PRELIMINARY REPORT
GEOTECHNICAL ENGINEERING SERVICES
EVERETT RIVERFRONT REDEVELOPMENT PROJECT - SIMPSON PAD
EVERETT, WASHINGTON

JUNE 7, 2007

FOR
OLIVERMCMILLAN EVERETT, LLC

GEOENGINEERS

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Preliminary Report
Geotechnical Engineering Services
Everett Riverfront Redevelopment Project
Simpson Pad
Everett, Washington
File No. 6191-002-04

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EXECUTIVE SUMMARY

This report contains the results of our preliminary geotechnical engineering services for use in preliminary design of the proposed development at the Simpson Pad site, which is a part of the Everett Riverfront Redevelopment project, in Everett, Washington. The Simpson Pad site is located on an approximately 45-acre parcel adjacent to and west of the Snohomish River. We understand that development of the Simpson Pad site may include residential housing units. The housing units may consist of a mixture of single-family houses, cottages, apartments and 4/6/8-plex buildings. The development will be constructed near existing grades; although, minor site grading will be needed.

In our opinion, the site may be developed as planned, provided the recommendations contained in this report are included in the design of the project. The site is underlain by 10 to 20 feet of fill overlying highly compressible fine-grained flood deposits and liquefiable soils. Mitigation of settlement due to consolidation of the compressible fine-grained deposits will be required at the site. In addition, settlement due to potential seismic liquefaction may need to be mitigated depending on the project design requirements. Following mitigation of potential settlement, lightly loaded buildings may be founded on conventional shallow spread footings bearing on a zone of compacted structural fill. Preloading of the site may be used to mitigate potential consolidation settlement at the site. Ground improvement, such as stone columns, may be used to mitigate consolidation settlement and liquefaction settlement concerns at the site. Heavily loaded structures may need to be supported on pile foundations.

SUBSURFACE CONDITIONS

The subsurface conditions at the site were evaluated by drilling five borings and advancing nine cone penetration tests (CPTs) in and along the perimeter of the Simpson Pad. Based on our explorations, the near-surface soils generally consist of 10 to 20 feet of fill overlying recent alluvial deposits consisting of compressible fine-grained soils composed of peat, organic silt, and silt with interbedded sands. The compressible fine-grained alluvial soils are generally underlain at depth by coarser-grained alluvial deposits consisting of sand with variable silt content and occasional silt layers. Where the explorations were extended to sufficient depth, a deeper compressible fine-grained silt layer was encountered below the coarser-grained alluvial deposits. A deep deposit of soft fine-grained sediment was observed in an east-west trending channel located in the southern portion of the site. Groundwater was generally observed ranging from 3 to 9 feet below the ground surface.

SETTLEMENT

The site soils are susceptible to consolidation settlement as a result of raising site grades and due to anticipated building loads. Settlement resulting from raising site grades by 2 to 4 feet may be on the order of 4 to 8 inches over the northern portion of the site and 6 to 12 inches where deeper fine-grained deposits are located under the southern portion of the site. Estimated primary consolidation settlement due to anticipated building loads is 3 to 8 inches. Estimated secondary consolidation settlement ranges from 1 to 2 inches over a period of 50 years.

The site soils are susceptible to seismically induced liquefaction, typically to depths of 27 to 46 feet below the ground surface. Liquefaction is characterized by the loss of soil strength during seismic shaking that results in ground settlement. We estimate that liquefaction induced ground settlement in the range of 3 to 6 inches could occur at the site during a significant earthquake.

Although there is a moderate to high potential that soil liquefaction would occur at the site during a large seismic event, our analyses indicate that there is a low potential for lateral spreading to occur that would

affect the planned buildable portion of the site. Lateral spreading and/or earthquake induced landsliding is anticipated to occur within about 100 to 200 feet of the Snohomish River.

SEISMIC DESIGN

In accordance with the 2006 International Building Code (IBC), the site is classified as Site Class F. This designation is required due to the high liquefaction potential of the native soils during a design earthquake event. However, for structures with a natural period of less than 0.5 seconds, the IBC allows the designation of Site Class E. If ground improvement, such as stone columns, is used to mitigate the liquefaction hazard at the site, the IBC 2006 Site Class should change to D.

FOUNDATION DESIGN

Buildings supported on shallow foundations on the existing soils will have a high risk of settlement during a design level earthquake and may also be impacted by consolidation settlement of compressible soils. The site soils are susceptible to seismically induced liquefaction to depths ranging from 27 to 46 feet below the ground surface. In addition, consolidation of the soft fine-grained soils will result in settlement across the site. Consolidation settlement may be mitigated by preloading the site; however, preloading will not mitigate potential liquefaction induced settlement.

With proper design and structural detailing, the planned wood-frame structures could be constructed to perform in a life-safe manner consistent with the intent of the 2006 IBC. However, the damage resulting from liquefaction induced settlements could be severe. If the risk of settlement due to seismic liquefaction is not acceptable, then we recommend that potentially liquefiable soils be mitigated by installation of stone columns. Stone columns may be designed to mitigate not only the liquefaction hazard at the site, but they may also be used to reduce the risk of damage resulting from consolidation of the fine-grained soils. The required depth of ground improvement will likely range from 30 to 50 feet below existing grades.

The proposed residential structures can be designed using conventional shallow foundations bearing on improved ground. We recommend a maximum allowable bearing pressure of 2.5 kips per square foot for conventional shallow foundations. A modulus of subgrade reaction of 50 pounds per cubic inch may be used for mat foundations. Additional reinforcement steel should be considered in the building foundations and floor slabs to reduce the risk of damage due to differential settlement at the site.

PRELIMINARY REPORT
GEOTECHNICAL ENGINEERING SERVICES
EVERETT RIVERFRONT REDEVELOPMENT PROJECT
SIMPSON PAD
EVERETT, WASHINGTON

INTRODUCTION

This report presents the results of our geotechnical engineering services for Simpson Pad site, which is a part of the Everett Riverfront Redevelopment project in Everett, Washington. The Simpson Pad is an approximately 45-acre parcel of developable land located within the Simpson site. The Simpson Pad is located south of the landfill site, west of the Snohomish River, east of Second Avenue, and north of Lowell-Snohomish River Road. The site is shown relative to surrounding physical features on the Vicinity Map, Figure 1, and in the Project Location Map, Figure 2.

PROJECT DESCRIPTION

We understand that OliverMcMillan Everett, LLC is proposing redevelopment of the Simpson Pad site for residential use. We further understand the proposed redevelopment will include:

- Fill materials imported to the site to achieve grade across the site.
- Construction of one- to three- story single and multi-family wood-framed residential structures. Residential housing may consist of a mixture of single family houses, cottages, apartments, and 4/6/8-plex buildings.
- Construction of roadways.
- Construction of hardscapes.

PURPOSE AND SCOPE OF SERVICES

The purpose of our services is to evaluate subsurface soil and groundwater conditions as a basis for developing preliminary design criteria for geotechnical aspects of the proposed development on the Simpson Pad, which is a part of the Everett Riverfront Redevelopment project. Field explorations and laboratory testing were performed to identify and evaluate subsurface conditions across the site in order to develop engineering recommendations for use in preliminary design of the project.

Our services were performed in general accordance with our proposal (Change Order No. 4) dated January 5, 2007 and the Owner and Consultant Agreement, which was accepted by OliverMcMillan Everett, LLC on October 10, 2006. Written authorization to proceed with our initial services was provided by OliverMcMillan Everett, LLC on January 12, 2007.

PREVIOUS STUDIES

In addition to the explorations conducted as part of this study, the logs of explorations completed as part of previous studies at the site were reviewed. The existing geotechnical information includes the logs of borings completed by HWA GeoSciences (2003) and explorations contained in the Floyd & Snider Team report (1999). Logs of these borings are presented in Appendix B, and their locations are shown on the Site Plan, Figure 3.

FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions were evaluated by a field exploration program that consisted of advancing five geotechnical soil borings and performing nine cone penetration tests (CPTs). The soil borings (designated B-1 through B-5) were advanced to depths ranging from 99½ to 131½ feet below the existing ground surface. The CPTs (designated CPT-1 through CPT-9) were advanced to depths ranging from 73 to 139 feet below the existing ground surface. The approximate locations of the explorations are shown on Figure 3. Details of the subsurface exploration program and logs of the borings (Figures A-2 through A-6) and CPTs (Figures A-7 through A-15) are summarized in Appendix A. Boring logs from previous projects in the vicinity of the Simpson Pad are included in Appendix B, and their locations are shown on Figure 3.

LABORATORY TESTING

Soil samples obtained from the borings were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index and engineering properties of the soil. Representative samples were selected for laboratory testing consisting of moisture content, dry density, Atterberg limits, particle size analyses, percent passing the U.S. No. 200 sieve, and consolidation tests. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures. Appendix C includes a brief discussion of the laboratory tests and the test results.

SITE CONDITIONS

SURFACE CONDITIONS

The Simpson Pad site is located along the western shoreline of the Snohomish River approximately 3 miles upstream from Possession Sound, as shown in Figure 2. The Simpson Pad was previously an industrial site with uses including a former sawmill and paper mill. Subsequent to the industrial uses, site grades have been raised with hydraulic fill as well as other imported materials. The Simpson Pad has been filled such that it is elevated above the surrounding wetland areas. The ground surface across the pad ranges from approximately Elevation 18 to 21 feet. A low area on the pad is located in the west-central portion of the site at about Elevation 15 feet. Fill embankments along the perimeter of the pad are up to 8 feet high on the east side and are generally inclined at about 3H:1V.

Primitive access roads surround the pad and access onto the pad is from the south and west-central areas. The northern edge of the site is adjacent to the northern end of the Snohomish River Trail. The pad is generally vegetated with tall grasses, while brush and blackberry bushes exist along the perimeter between the pad and the wetland areas.

REGIONAL GEOLOGY

The Puget Sound basin is a region of Quaternary (last 3 million years) sediments that range in thickness between 800 and 2,400 feet. Bedrock exposures are present on the basin margins to the east and west in the Cascade and Olympic Mountains, respectively. The basin area has been repeatedly overridden by Pleistocene (between 11,000 and 3 million years ago) continental glacial ice depositing till, glacial sand and gravel, and glacially formed lake clay and silt. The repeated glacial action has resulted in numerous north-south trending ridges, with intervening valleys filled with post-glacial alluvium and/or marine deposits (Galster, 1989). The most recent glacial cycle of sediment deposits is referred to as the Vashon Drift, occurring between 13,500 and 15,000 years ago.

The site is situated on the west side of the Snohomish River in the Snohomish River Valley. The river follows the eastern edge of the northern upland area of Everett and bends to the west north of the site prior to flowing into Possession Sound.

Geologic information for the site was obtained from the "Geologic Map of the Everett 7.5-Minute Quadrangle, Snohomish County, Washington", by Minard (1985). The map indicates that the site is generally underlain by younger alluvium (Holocene), as shown on Figure 4. Younger alluvium deposits underlying the site consist of unconsolidated, stratified, clay, silt, and very fine to fine sand with abundant organic material. Medium to coarse sand and gravel underlie much of the fine-grained flood plain sediment. Glacial till, advance outwash and Transitional Beds are mapped west of the site in the Everett upland area.

SUBSURFACE CONDITIONS

General

The Simpson Pad was constructed with fill placed during previous industrial activities, hydraulic fill originating from the Snohomish River, and recent fill placed by the City of Everett. The fill was placed over the alluvial deposits, including areas containing peat. The upper portion of the recent alluvial deposits consists of compressible fine-grained soils composed of peat, organic silt and silt with interbedded sands and occasional organic matter. The compressible fine-grained alluvial soils are generally underlain by coarser-grained alluvial deposits consisting of sand with variable silt content and occasional silt layers. In some of the explorations, the coarse-grained alluvial deposits are underlain by fine-grained deposits. Generalized subsurface profiles generated across the site are shown on Figures 4 through 10. Locations of the profiles are shown on Figure 3. Each of the units encountered is described below.

Topsoil

Topsoil consisting of brown sandy silt with roots from surface vegetation was observed in the upper 3 inches of the Simpson Pad. Deeper topsoil and organic laden soils should be expected beyond the perimeter of the Simpson pad.

Fill Soils

Fill exists across the entire site that forms the Simpson Pad. The fill varies from about 10 to 20 feet thick and consists primarily of silty sand, silt, and fine to medium sand with varying amounts of silt and gravel. Occasional organic matter (wood chips and decayed vegetation) and scattered bricks fragments were observed in the fill soils during our field exploration. Fill consisting of silty sand and silt appears to have been placed on the western portion of the pad in the areas of borings B-1 and B-2. The fill is typically loose to medium dense with decreasing density near the base of the fill. Although not encountered in our explorations, we understand that many of the concrete foundations from the previous industrial facilities are still in place within the fill below the ground surface (HWA, 2003). Gravel was encountered towards the bottom of the fill layer in borings B-1, B-2, and B-5.

Fine-grained Alluvial Deposits

Fine-grained deposits consisting of recent alluvium directly underlie the fill. These deposits are composed of very soft to soft organic silt, silt, and interbedded silts and sands. Peat layers were observed throughout this deposit. The fine-grained alluvium was encountered in all the explorations underlying the fill and was generally observed extending to depths ranging from 32 to 47 feet below the existing ground surface. However, what appears to be a historic channel infilled with fine-grained deposits is situated

along the southern part of the site in an east-west direction. A deep thickness of fine-grained alluvium was observed in Boring B-1, CPT-5, and CPT-8. Fine-grained deposits observed in this possible channel were encountered to depths ranging from 68 to 93 feet below the ground surface, as shown on Figure 11. The upper portions of the fine-grained deposits commonly contain variable amounts of fibrous organic peat material. Logs were observed throughout this deposit and created refusal conditions on the first two attempts (CPT-5A and CPT-5B) in advancing CPT-5.

Coarse-grained Alluvial Deposits

The fine-grained alluvial deposits are underlain by coarse-grained alluvial deposits generally consisting of loose to medium dense, fine to medium sand with variable silt content and interbedded layers of silt. The explorations indicate that the upper surface of the coarse-grained alluvial deposits is typically encountered at depths between about 32 and 46 feet below existing ground surface, except where a historic river channel may traverse the south portion of the site and the sand was observed at depths ranging from 68 to 93 feet below the ground surface. The approximate depth to the top of the upper coarse-grained deposit is shown in Figure 11.

Deep Fine-grained Alluvial Deposits

Fine-grained alluvium was also observed underlying the coarse-grained deposits in several explorations, including B-2, B-3, CPT-1 through CPT-6, and CPT-9. The deeper fine-grained alluvial deposits were typically observed between 61 and 92 feet below the ground surface when they were encountered. The deep fine-grained alluvial deposits generally consist of soft to stiff, organic silt and silt with variable sand content, including interbedded sand layers and organic soils.

Deep Course-grained Alluvial Deposits

Deeper coarse-grained deposits including sand with gravel and gravel layers were observed at depth below the deep fine-grained deposits. The deep course-grained alluvium was generally observed at depths of 80 to 95 feet below the existing ground surface. These deposits generally consist of dense to very dense silty sand, fine to medium sand with variable silt content, and gravel with occasional interbeds of fine-grained soils.

GROUNDWATER CONDITIONS

Previous studies at the site (HWA GeoSciences, 2003) indicate that a shallow aquifer exists below the site at depths ranging from about 3½ to 12 feet below the ground surface. The aquifer reportedly flows in a northerly direction and is not hydraulically connected to the Snohomish River.

Groundwater was observed in all five of our borings at depths typically ranging from 3 to 7 feet below the ground surface. Groundwater was encountered at a depth of 14 feet below the existing ground surface in boring B-2. Previous explorations at the site, BF-1 and BF-2, encountered groundwater at depths of about 3½ to 9 feet below the ground surface. Explorations performed by the Floyd & Snider team, including B-43, B-44, and B-54 on the Simpson Pad encountered groundwater at depths of 2.5 to 9 feet below the ground surface. Groundwater conditions in the borings can vary due to the limited time the borings are left open such that groundwater does not have sufficient time to equilibrate. We anticipate that groundwater levels will fluctuate as a function of the season, precipitation, and other factors. Observed groundwater conditions are summarized in Table 1.

Table 1. Observed Groundwater Conditions

Exploration	Approximate Surface Elevation (feet)	Observed Groundwater Depth (feet)	Groundwater Elevation (feet)
B-1	17.5	7	10.5
B-2	21.5	14	7.5
B-3	18	3	15
B-4	22	7	15
B-5	19	5	14
BF-1	16.3	3.6	12.7
BF-2	21.7	9	12.7
B-43	16.8	5	13.8
B-44	17.2	8.8	8.4
B-54	15.2	2.5	12.7

SEISMICITY

General

Major portions of the site are characterized by the City of Everett's Sensitive Areas maps as having moderate to high potential for seismic hazards, including liquefaction and landslides. Seismic hazard areas are typically defined as those areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, liquefaction, or lateral spreading.

Seismicity in the Puget Sound area is primarily driven by the Cascadia Subduction Zone, which is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. Three potential seismic source zones are generally acknowledged for the Puget Sound area: (1) shallow crustal earthquakes within the North American Plate associated with known and/or unknown faults; (2) Cascadia Subduction Zone interface earthquakes, which occur along the boundary located between the Juan de Fuca and North American plates; and (3) Cascadia Subduction Zone intraplate earthquakes, which occur within the subducting Juan de Fuca Plate.

Shallow Crustal Earthquakes

Shallow crustal earthquakes occur within the North American Plate to depths up to 15 miles. Shallow earthquakes in the Puget Sound region are expected to have durations ranging up to 60 seconds. Four magnitude 7 or greater known earthquakes have occurred in the last 1,100 years in the Cascadia region, two of these occurred on Vancouver Island and two in Western Washington.

The largest historic earthquake in Western Washington occurred in 1872 in the North Cascades and is estimated to have had a magnitude of 7.4. This earthquake is believed to have occurred at a depth of less than 10 miles. The other magnitude 7+ Western Washington earthquake occurred on the Seattle Fault approximately 1,100 years ago.

Shallow crustal faults with known or suspected Quaternary displacements within the general project area include the Southern Whidbey Island Fault. The Southern Whidbey Island Fault is a northwest-southeast trending structure located near Whidbey Island and extending as far southeast as Everett. Recent evidence suggests that the Southern Whidbey Island Fault is an active fault system with Quaternary

displacements. This fault system is considered capable of magnitude 7 earthquakes and is located approximately 6 miles southwest of the site.

Interface Earthquakes

Interface earthquakes occur on the boundary between the Juan de Fuca and North American tectonic plates. The Cascadia Subduction Zone extends from Vancouver Island to Northern California. Interface earthquakes on the Cascadia Subduction Zone are anticipated to have durations ranging up to 4 minutes.

Paleogeologic evidence of the occurrence of large (magnitude 8 to 9+) earthquakes occurring on the Cascadia Subduction Zone has recently been discovered. The last large interface earthquake is believed to have occurred in the year 1700. It is estimated that the recurrence interval for interface earthquakes on the Cascadia Subduction Zone is about 400 to 600 years; however, the interval between earthquakes appears irregular.

Intraplate Earthquakes

Cascadia Subduction Zone intraplate earthquakes occur within the subducting Juan de Fuca Plate at depths of 30 to 40 miles within the Puget Sound area. Intraplate earthquakes are expected to have durations ranging up to 30 seconds and magnitudes ranging up to 7.5. The Olympia 1949 (magnitude 7.1), the Seattle 1965 (magnitude 6.5), and the Nisqually 2001 (magnitude 6.8) were intraplate earthquakes. Other earthquakes that are considered to be intraplate events occurred in 1882, 1909 and 1939.

CONCLUSIONS AND RECOMMENDATIONS

EARTHQUAKE ENGINEERING

GeoEngineers evaluated the site for seismic hazards including liquefaction and lateral spreading. Our analyses indicate that the Simpson Pad site has a high risk of liquefaction induced settlement, but a low risk of lateral spreading during a design level earthquake. The liquefaction and lateral spreading hazards and building code site coefficients are discussed in detail below.

2006 IBC Seismic Design Information

As discussed, the site is located in a seismically active area. Newer structures, designed in accordance with the latest seismic codes and that have proper foundations and structural detailing, have performed well during recent earthquakes. However, modern seismic codes are formulated to provide only life safety protection during a large earthquake. Cosmetic and structural damage are considered acceptable. If better performance during a large earthquake is desirable, it may be necessary to upgrade the design of the structure beyond the current seismic code levels. We can provide additional information for site-specific earthquake analyses, if requested.

We recommend the use of the parameters presented in Tables 2 and 3 for 2006 International Building Code (IBC) for Site Class, short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1), and seismic coefficients for the project site. Table 2 presents the IBC 2006 seismic parameters for the site if a stone column ground improvement program is implemented.

Table 2. IBC Seismic Parameters – Shallow Foundations on Stone Columns

2006 IBC Parameter	Recommended Value
Average Field Standard Penetration Resistance	$15 \leq N \geq 50$
Site Class	D
Short Period Spectral Response Acceleration, S_S (percent g)	117.6
1-Second Period Spectral Response Acceleration, S_1 (percent g)	41.2
Seismic Coefficient, F_A	1.03
Seismic Coefficient, F_V	1.59

If ground improvement is not employed to mitigate the liquefaction hazard, the Site Class per the 2006 IBC is F, and a site specific seismic response analysis will be required. However, for structures with a natural period of less than 0.5 seconds, the IBC allows the designation of Site Class using the procedure outlined in Section 1615.1.5.1 in lieu of completing a site response analyses. Table 3 presents the 2006 IBC seismic parameters for the site if piles are selected for foundation support and/or the natural period of the buildings are less than 0.5 seconds.

Table 3. IBC Seismic Parameters – Pile Foundations

2006 IBC Parameter	Recommended Value
Average Field Standard Penetration Resistance	$15 > N$
Site Class	E
Short Period Spectral Response Acceleration, S_S (percent g)	117.6
1-Second Period Spectral Response Acceleration, S_1 (percent g)	41.2
Seismic Coefficient, F_A	0.9
Seismic Coefficient, F_V	2.4

Liquefaction

The City of Everett Liquefaction Hazard Map depicts the site as having moderate to high potential for liquefaction. Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as pore water pressures increase in response to strong ground shaking. The increased pore water pressure may temporarily meet or exceed soil overburden pressures to produce conditions that allow soil and water to flow, deform, or erupt from the ground surface. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures, such as buildings, supported on or within liquefied soils may suffer foundation settlement or lateral movement that can be damaging to the buildings. Based on our analyses, the potential exists for liquefaction to occur within zones of the loose to medium dense sand deposits encountered in the explorations completed at the site.

The evaluation of liquefaction potential is a complex procedure and is dependent on numerous site parameters, including soil grain size, soil density, site geometry, static stresses, and the design ground acceleration. Typically, the liquefaction potential of a site is evaluated by comparing the cyclic shear stress ratio (the ratio of the cyclic shear stress to the initial effective overburden stress) induced by an earthquake to the cyclic shear stress ratio required to cause liquefaction. Estimation of the cyclic shear stress required to initiate liquefaction and the cyclic shear stress initiated by a design earthquake were completed using the empirical method developed by Seed et al. (1985) as revised at the National Control

for Earthquake Engineering Research (NCEER) workshop in 1997 (Youd, et al, 2001). The cyclic shear stress ratio required to cause liquefaction at the site was estimated using empirical procedures based on correlations from the standard penetration tests (SPTs) and cone penetration tests (CPTs). The CPT method relates the cyclic shear stress ratio required to cause liquefaction to the cone tip and sleeve resistance values. Estimated ground settlement resulting from earthquake-induced liquefaction was analyzed using an empirical procedure that relates settlement to average SPT N-values, which we correlated from the CPT tip values. A design earthquake with a magnitude of 7.2 and a peak horizontal acceleration of 0.28g (28 percent of the acceleration due to gravity) was used for our analysis. This analysis also assumes a level ground surface.

The results of our analyses indicate that zones in the loose to medium dense sandy soils encountered in the explorations near the bottom of the fill and immediately below the organic silt have a moderate to high potential for liquefaction during a design earthquake event, and a low to moderate potential during an event with a lower level of ground shaking. The potentially liquefiable layers vary across the site but are generally within 10 to 23 feet of the ground surface where located immediately beneath the fill, and between 27 and 46 feet of the ground surface where located below the upper fine-grained alluvial deposits. The organic silt and silt deposits located above the sand have a low potential for liquefaction during a design.

Our analyses indicate that settlements caused by liquefaction of the saturated loose to medium dense sand layers at this site during a design earthquake could be on the order of 3 to 6 inches. Foundations for the structures will be constructed over the liquefiable soils and will therefore be prone to liquefaction induced settlement. Because of the random nature of liquefaction, differential settlements may be on the same order as the total settlements. However, surface expression of the differential settlement may be reduced due to the thickness of the existing fill soils overlying the potentially liquefiable soil layers.

Structures constructed at the site should be evaluated and designed based on the risk and potential magnitude of soil liquefaction. It is possible to design wood-frame structure to accommodate the magnitude of liquefaction induced displacement expected and still perform in a life-safe manner, which is consistent with the IBC philosophy. However, even though the buildings can be designed to perform in a life-safe manner, they maybe severely damage. Mitigation of the liquefaction induced settlement would be required to prevent the structures from being damaged.

Lateral Spreading

Lateral spreading occurs when the shear strength of the liquefied soil is incrementally exceeded by the lateral inertial forces induced during an earthquake. The result of lateral spreading is typically horizontal movement of non-liquefied soils located above liquefied soils. Lateral spreading generally develops in areas where sloping ground is present or near a free face, such as a river.

If liquefaction were to occur within the sands underlying the site, we anticipate that there would be a low potential for lateral spreading to occur within the proposed site building area. Lateral spreading would be characterized by movement of the soils towards the Snohomish River. We anticipate that most of the movement would be along the banks of the river. Movements near the river can be excessive and very destructive. During the 1995 earthquake in Kobe, Japan, lateral displacement on the order of 5 to 15 feet occurred resulting in collapse of over 40 miles of waterfront walls and piers.

Lateral spreading associated with liquefaction was evaluated by performing limit-equilibrium slope stability analyses using liquefied soil strength parameters. In general, the magnitude of lateral spread will decrease with increasing distance between the point of interest and the Snohomish River. The results of our analyses suggest that lateral spreading will generally occur within 100 to 200 feet of the Snohomish

River during the design earthquake and that lateral spread displacements are expected to develop east of the Simpson Pad.

Ground Rupture

Geologic maps indicate that the south Whidbey Island fault is located roughly 6 miles southwest of the site. Because of the thickness of the Quaternary sediments below the site, which are commonly more than 1,000 feet thick, the potential for surface fault rupture is considered remote.

Landsliding

Based on literature review, site location, and site topography, there does not appear to be active landsliding on the site. Thus, earthquake shaking will have a low likelihood of initiating large-scale landsliding, except along the banks immediately adjacent to the Snohomish River. As discussed above, lateral spreading induced movement of the river banks and ground within 100 to 200 feet of the river bank is probable during a significant seismic event. The final design phase needs to further address the potential of landsliding (lateral spreading) during excessive earthquake shaking at it relates to proposed site improvements.

The City of Everett Liquefaction Hazard Map also shows the slopes to the west of the project site near Interstate 5 as moderate to high potential for landslides. As with all slopes in western Washington, shallow surficial sliding is possible, particularly when the ground is saturated. Surficial slides typically occur in the upper 2 to 5 feet of soil and movement occurs episodically, generally in response to heavy rainfall. Earthquake shaking would tend to increase the size of the surficial slide area as well as the frequency of movement.

CONSOLIDATION SETTLEMENT

Compressible silt, organic silt, and peat deposits were observed in the explorations completed at the site. We have completed analyses to evaluate the consolidation settlement of these compressible soils under anticipated building loads and under 2 to 4 feet of fill that is anticipated to be placed across the site. Based on the results of our analyses, we estimate that settlement resulting from raising site grades by 2 to 4 feet may be on the order of 4 to 8 inches over the northern portion of the site and 6 to 12 inches where the deeper fine-grained deposits are located under the southern portion of the site. Estimated primary consolidation settlement due to anticipated building loads is 3 to 8 inches. The estimated secondary consolidation settlement ranges from 1 to 2 inches over a period of 50 years. Consolidation settlement may take several months to possibly years. It is possible to accelerate the consolidation process by placing additional fill (preload and/or surcharge) above the prepared subgrade to induce a major portion of the settlement, which would otherwise occur when building loads are applied.

Potential building settlement may be mitigated if shallow foundations are used in conjunction with ground improvement such as preloading or stone columns. Consolidation of the lower compressible fine-grained deposits encountered in the explorations also affect the design of the deep foundations and the ground improvement as described below.

GROUND IMPROVEMENT

General

Ground improvement may be needed to mitigate potential liquefaction hazards underlying the site and will be needed to mitigate settlement due to static loading. Ground improvement techniques used to mitigate the risk of liquefaction induced settlement include dynamic compaction, stone columns

(vibro-replacement), compaction grouting, and gravel drains, among others. Potential consolidation settlement at the site due to static loading can be mitigated by use of preloading or by supporting the structures on deep foundations.

Dynamic compaction is basically the dropping of a heavy weight on to the ground surface to densify soils at depth. Dynamic compaction is not effective in reducing the settlement potential of the saturated fine-grained alluvial deposits underlying the site. However, dynamic compaction can reduce the risk of liquefaction of the looser sand deposits, which are 10 to 20 feet below the ground surface and beneath the existing fill. This technique is less effective in densifying the loose to medium dense potentially liquefiable soils located at deeper depths, especially where overlain by the fine-grained alluvial deposits.

Grouting, such as compaction grouting, involves injection of grout into the subsurface soils to densify the soil as well as to create a mass having a relatively high shear strength. Grouting is effective in loose granular soils, but less effective in fine-grained alluvial soils. Grouting techniques are not economical on projects such as this where the depth of application is deep and widespread.

Based on our evaluations, we recommend that preloading be implemented to mitigate settlement at the site caused by consolidation of the fine-grained soils. In addition, if the risk of soil liquefaction is not acceptable, then we recommend that stone columns be considered to mitigate this risk. Stone columns may also be used to support the buildings, such that preloading may not be needed. Preloading alone does not mitigate the liquefaction hazard at the site. Stone columns and preloading are discussed in detail in the following sections.

Stone Columns

Construction of stone columns involves the partial replacement of loose, liquefiable soils with a vertical column of compacted stone. Typically, a hollow tube or probe is vibrated, jetted or driven into the ground to the desired depth. As the tube or probe is withdrawn, crushed stone is fed to the bottom of the hole and compacted. The end result is a column of dense stone which penetrates through the loose, liquefiable soil and is capable of transferring loads into the underlying non-liquefiable soils. The presence of the column also creates a composite material of higher density and higher shear strength than the native soil alone. Lastly, the process of constructing the stone columns can densify potentially liquefiable soils reducing their susceptibility to liquefaction. The increased density and shear strength of the resulting composite material provides a significant reduction in the liquefaction potential as compared to areas where ground improvement is not employed.

If chosen as a ground improvement technique, we recommend that the stone columns be installed within the building areas to depths of approximately 30 to 50 feet below existing grades. The actual depth and final design of the stone columns should be based on the planned building footprints and actual subsurface conditions below the buildings. We recommend that the area in which stone columns are installed extend at least 5 feet laterally beyond the edges of the proposed building footprints.

In addition to the stone columns, we recommend that a minimum 2-foot thick layer of structural fill consisting of pit run sand and gravel or crushed rock, both with less than 3 percent fines, be placed between the stone columns and the foundations. This layer of structural fill will help transfer loads from the foundations and slabs-on-grade to the stone columns and will help reduce differential settlement. A 1-foot thick layer of this material should be placed between the floor slab and the stone columns.

The stone column center-to-center spacing and diameter should be such that the corrected Standard Penetration Test (SPT) blow counts ($N_{1(60)CS}$) are increased in the lower sand soils such that they are no

longer susceptible to liquefaction during a design event. Before production ground improvement work is initiated, the contractor should complete a test section to demonstrate that the submitted installation equipment and methods will meet the performance requirements for the project. Performance testing of the test section should be completed using SPT testing completed in accordance with ASTM D1586 using automatic trip, safety hammers. Cathead or wireline-operated hammers should not be allowed. Performance testing should be completed both prior to and after ground improvement in the immediate vicinity of the test section to confirm that the submitted design meets the performance requirements. The performance test should be completed at the location equidistant from adjacent stone columns.

Performance testing during production should be completed at a frequency of one SPT test per 5,000 square feet of ground improvement. The stone used for the stone columns should consist of clean, hard, unweathered crushed stone free of organics, debris and other deleterious materials.

GeoEngineers can develop the Stone Column ground improvement performance specification, upon request; if this technique is selected for the site.

Preload Fill

If the building areas are not improved through the use of the stone columns or similar technique and if the buildings are not be supported on pile foundations, a preload program will be necessary to mitigate settlement from consolidation of the underlying soils from static loads (e.g., new fill or building loads). The purpose of the preload fill is to pre-induce a major portion of the settlement that would otherwise occur when site fill and structure loads are applied.

Preload Height

For planning purposes, we suggest a preload height of at least 5 feet above the finished grade elevation. The preload height is the thickness of the preload fill that is placed above the finished grade elevation. Higher preloads may be needed depending on planned structures/loads of the site. The crest of the preload should extend laterally at least 5 feet beyond the footprint of the structures in each direction. It may be beneficial to expand the preload areas in case the locations of structures are expected to move.

The preload fill should consist of structural fill quality material compacted only to the extent necessary to support construction equipment. Side slopes should be planned no steeper than 1H:1V. The preload surface should be crowned slightly to promote drainage of surface water.

Settlement Magnitude and Rate

The site soils are susceptible to consolidation settlement as a result of raising site grades and due to anticipated building loads. Settlement resulting from raising site grades by 2 to 4 feet may be on the order of 4 to 8 inches over the northern portion of the site and up to 12 inches where the deeper fine-grained deposits are located under the southern portion of the site. Estimated primary consolidation settlement due to potential future building loads is 3 to 8 inches.

We estimate that the 5-foot high preload will induce approximately 2 to 8 inches of settlement in addition to settlement caused by site fills. In the southern portion of the site where the deep deposits of fine-grained soils exists, the preload may induce up to 12 inches of settlement. This settlement will reduce the post-construction settlements observed in the areas where the preload was placed. Fill volume estimates to achieve proposed site grades should include an allowance for the anticipated settlement.

The estimated settlement assumes that the soils have not been previously preloaded. The estimated settlement magnitudes may vary substantially based on preloading caused by previous uses at the site and due to the thickness of the underlying compressible soils.

The estimated time for the settlement to occur is 12 to 16 weeks, but the actual duration of preloading should be based on settlement readings. The thicker compressible silt layers in the southern portion of the site will require more time to consolidate and will likely be on the 16 week end of this range. The consolidation process could be accelerated by placing a surcharge fill on top of the preload fill. We can provide specific surcharge recommendations if the preload time must be shortened.

We have also estimated the long-term settlement for structures that may be constructed as part of the project. The long-term settlement estimates assume that the site grading fill and preload settlements are achieved prior to the construction and loading of planned structures. The estimated secondary consolidation settlement ranges from 1 to 2 inches over a period of 50 years. Differential settlement between similarly loaded footings should be on the order of ½-inch. These long-term estimated settlements are based on the preloads being left on the site long enough to substantially complete the primary consolidation process for each structure.

Settlement Monitoring

In order to evaluate the magnitude and time rate of settlement of the site fill or the preload fill, we recommend that settlement monitoring plates be installed prior to placing the preload fill. Settlement plate locations will depend on the preload fill phasing and should be established to adequately measure the preload fill induced settlement. A detail of a typical settlement plate is shown in Figure 12.

If a settlement plate is damaged, it should be repaired and resurveyed immediately. Plate elevations should be referenced to a stable benchmark, away from the influence of the preload or any fill placed at the site. The settlement data should be provided to the geotechnical engineer immediately after the readings are taken so that we may review and comment as appropriate.

Initial elevation readings of the settlement plates must be obtained when they are installed and before any fill is placed – site fill or preload fill. If this is not done, the initial settlement behavior of the preload fill will not be recorded and the total magnitudes of settlement and rate of settlement will be unknown.

The elevations of the plates and the adjacent ground surface should be measured twice weekly during fill placement and once a week after completion of filling so that settlement progress in relation to the amount of fill in place can be observed.

Measurement rods that extend from the settlement plates through the fill will inhibit the mobility of earthmoving equipment to some extent. The contractor will have to exercise care to avoid damaging the rods. The construction documents should emphasize the importance of protecting the settlement plates and measuring rods from disturbance.

EARTHWORK

Excavation Considerations

Surficial soils consisting of fill and soft native organic silt were observed in the explorations. We anticipate that these materials can be excavated with conventional excavation equipment, such as excavators or dozers. However, because of the soft and wet nature of the native fine-grained soils, we recommend the contractor use low ground pressure equipment including wide-tracked dozers, when operating over the native soils. In addition, we recommend that all excavations by excavators be accomplished using a bucket with a smooth edge to reduce disturbance to the native soils, especially when performing excavations in areas under planned structures.

Clearing and Grubbing

Most of the site is vegetated with grasses and shrubs. The surficial soils at the site include thin topsoil deposits and fill soils. We recommend that the topsoil and organic materials be completely stripped and removed from planned building, pavement and hardscape areas.

Subgrade Preparation

Prior to placing new fills, pavement base course materials or gravel below on-grade floor slabs, subgrade areas should be proofrolled to locate any soft or pumping soils. Prior to proofrolling, all unsuitable soils should be removed from below building areas. Proofrolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather, the exposed subgrade areas should be probed to determine the extent of soft soils. If soft or pumping soils are observed they should be removed and replaced with structural fill.

After completing the proofrolling, the subgrade areas should be recompacted to a firm and unyielding condition, if possible. The degree of compaction that can be achieved will depend on when the construction is performed. If the work is performed during dry weather conditions, we recommend that all subgrade areas be recompacted to at least 95 percent of the maximum dry density (MDD) in accordance with the American Society for Testing and Materials (ASTM) D 1557 test procedure (modified Proctor). If the work is performed during wet weather conditions, it may not be possible to recompact the subgrade to 95 percent of the MDD. In this case, we recommend that the subgrade be compacted to the extent possible without causing undue weaving or pumping of the subgrade soils.

We recommended that the upper 2 feet of subgrade soil below all asphalt pavement and hardscape areas be compacted to at least 95 percent of the MDD per ASTM D 1557. This will require excavation and replacing existing soils or excavation and removal of existing soils and replacement with imported Gravel Borrow. Prior to placing the 2-foot thick structural fill layer the exposed subgrade should be conditioned, aerated (if needed) and be compacted to at least 95 percent of the MDD.

We also recommend that spread footings and floor slabs be supported on at least 2 feet of compacted structural fill for uniform support and to provide adequate bearing. This will require overexcavating and replacing existing loose or soft soils with compacted structural fill within 2 feet of foundation subgrades. Imported Gravel Borrow should be planned below the buildings. Prior to placing the structural fill material, we recommend that the exposed subgrade surface at the 2-foot depth be conditioned, aerated (if needed) and be recompacted to at least 95 percent of the MDD.

Prior to placing new fills, crushed rock, or foundations subgrade areas should be evaluated and/or proofrolled to locate any soft or pumping soils. All unsuitable soils should be removed. Proofrolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather or in foundation excavations, the exposed subgrade areas should be probed to determine the extent of soft soils. If soft or pumping soils are observed they should be removed and replaced with structural fill.

Subgrade disturbance or deterioration could occur if the subgrade is wet and can not be dried. If the subgrade deteriorates during proofrolling or compaction, it may become necessary to modify the proofrolling or compaction criteria or methods.

Structural Fill Materials

Materials placed as fill to raise site grades in the vicinity of proposed structures, pavement areas, or other improvements are classified as structural fill. Structural fill material quality varies depending upon its use.

Structural fill placed to raise site grades should meet the criteria for common borrow, WSDOT Standard Specification 9 03.14(3). Common borrow will be suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, the structural fill should consist of gravel borrow as described in Section 9-03.14(1) with the exception that the percent passing the U.S. No. 200 Sieve should be less than 5 percent. Common borrow should not be planned as structural fill during the wet season, as compaction can be very difficult if not impossible to achieve. Common borrow would still be very susceptible to disturbance from construction traffic during the wet season.

Structural fill placed as crushed surfacing base course below pavements should meet WSDOT Standard Specification 9-03.9(3) for Base Course.

On-site Soils

The on-site fill soils generally consist of silty sand and fine to medium sand with silt. Due to the fine-grained nature of the soils, they are susceptible to changes in moisture content. We anticipate that the on-site fill may be reused as structural fill during dry weather conditions; however, they will be difficult to compact in wet weather conditions.

The on-site native soils consist of silt, organic silt and silty sands for the most part, with the silt and organic soils near the ground surface in most areas. These soils contain a high percentage of fines and organics, are typically wet, and are highly sensitive to changes in moisture content. They are also highly susceptible to disturbance from construction traffic. The on-site native soils should not be planned for use as structural fill.

Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. The structural fill should be placed in lifts. The lift thickness should be sufficiently thin so that following compaction, the entire lift meets the compaction criteria recommended below. In general, each loose lift thickness should not exceed 1 foot. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Thinner loose lifts may be needed to achieve compaction depending on the actual compaction equipment used. Structural fill should be compacted to the following criteria:

- Structural fill placed to raise site grades in areas where no proposed structures are planned should be compacted to at least 90 percent of the MDD per ASTM D 1557.
- Structural fill used to construct or reconstruct embankments should be compacted to at least 90 percent of the MDD.
- Structural fill placed below proposed structures should be compacted to at least 95 percent of the MDD per ASTM D 1557.
- Structural fill placed within 2 feet of the pavement or gravel driveway subgrade elevations should be compacted to at least 95 percent of the MDD per ASTM D 1557.

- Structural fill placed for crushed surfacing base course below pavements should be compacted to at least 95 percent of the MDD per ASTM D 1557.
- Fill placed as preload fill should be compacted to the extent necessary to allow access for construction equipment.
- Fill placed in landscape areas and other areas where no settlement-sensitive improvements will be located should be compacted to at least 90 percent of the MDD per ASTM D 1557.

The recommendations presented in this section are for the placement of structural fill during the dry season. We recommend that a representative from our firm be present during placement of the structural fill. Our representative will evaluate subgrade conditions prior to placing fill, perform in-place moisture density tests in the fill to evaluate if the work is being done in accordance with the compaction specifications, and advise on any modifications to procedure that may be appropriate for the prevailing conditions.

Weather Considerations

The on-site fill soils and common borrow contain a sufficient percentage of fines (silt and fine organics) to be highly moisture sensitive. When the moisture content of these soils is more than a few percent above the optimum moisture content, these soils become unstable. Operation of equipment on these soils during wet weather conditions may be difficult and may generate mud due to severe rutting and pumping, and it will be difficult or impossible to meet the required compaction criteria. Additionally, disturbance of near surface soils should be expected any time of the year where the native soils are exposed at the ground surface, especially if earthwork is completed during periods of wet weather. The wet weather season generally begins in October and continues through May in the Puget Sound region; however, periods of wet weather may occur during any month of the year. It will be preferable to schedule site preparation and earthwork activities during periods of dry weather when the soils will (1) be less susceptible to disturbance, (2) provide better support for construction equipment, and (3) more likely to meet the required compaction criteria.

The optimum earthwork period for these types of soils is typically June through September. If wet weather earthwork is unavoidable, as we understand for this project, we recommend that:

- Structural fill placed during the wet season or during periods of wet weather consist of gravel borrow with less than 5 percent fines (material passing the U.S. No. 200 Sieve).
- Structural fill be placed as soon as possible after subgrade is achieved to prevent softening of the subgrade areas and to provide support for equipment.
- The ground surface in and around the work area be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities not take place during periods of heavy precipitation.
- Slopes with exposed soils be covered with plastic sheeting or similar means.
- Measures be taken to prevent on-site soils and soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting, sumps with pumps, and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing the

surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent that these soils become wet or unstable.

- Construction and foot traffic be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance. The contractor should keep equipment off as much of the site as possible and restrict access to as small of areas as possible.
- Construction activities be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.

Temporary Slopes

We recommend temporary slopes constructed in fill placed to raise site grades or cuts made in the unsaturated native organic silt soils be inclined no steeper than 1½H:1V. Flatter slopes may be necessary if groundwater is encountered or if localized sloughing occurs. In addition, vibrations from construction equipment can cause sloughing of temporary slopes and thus slopes may have to be flattened if soughing is observed. For temporary slopes at the site, we recommend that:

- No traffic, construction equipment, stockpiles or construction supplies be allowed at the top of the cut slopes within a distance of at least 10 feet from the top of the slope.
- Exposed soil along the slope be protected from surface erosion using waterproof tarps or plastic sheeting.
- Construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practical.
- Erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practical.
- Surface water is diverted away from the excavation.
- The general condition of the slopes be observed periodically by a geotechnical engineer to confirm adequate stability.

Since the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. All shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

Permanent Slopes

We recommend that permanent cut and fill slopes be constructed no steeper than 2H:1V. To achieve uniform compaction, we recommend that fill slopes be overbuilt by about 2 feet and subsequently cut back to expose properly compacted fill.

We recommend that all existing fill slopes that will remain be removed and replaced using engineered fill compacted to at least 90 percent of the MDD. For planning purposes, the width of engineered fill should be equal to the height of the slope.

To minimize erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and raveling of the slopes should be expected. This may require localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, jute fabric, loose straw or rolled erosion control mats should be used to protect the slopes during periods of rainfall.

Erosion and Sedimentation Control

Potential sources or causes of erosion and sedimentation depend upon construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. The project impact on erosion-prone areas can be reduced by implementing an erosion and sedimentation control plan. The plan should be designed in accordance with the city of Everett and/or applicable county standards. The plan should incorporate basic planning principles including:

- Scheduling grading and construction to minimize soil exposure,
- Retaining existing vegetation whenever feasible,
- Revegetating or mulching denuded areas,
- Directing runoff away from denuded areas,
- Minimizing the length and steepness of slopes with exposed soils,
- Decreasing runoff velocities,
- Preparing drainage ways and outlets to handle concentrated or increased runoff,
- Confining sediment to the project site, and
- Inspecting and maintaining control measures frequently.

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help minimize erosion and reduce transport of sediment to adjacent areas and receiving waters. Permanent erosion protection should be provided by re-establishing vegetation by hydroseeding or landscape planting.

Until the permanent erosion protection is established and the site is stabilized, site monitoring should be performed by qualified personnel to evaluate the effectiveness of the erosion control measures and repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the erosion and sedimentation control plan.

SHALLOW FOUNDATIONS

General

We anticipate that it will be most economical to support the residential buildings on conventional spread footings following ground improvement - preloading or stone columns. As discussed in the "Stone Columns" section above, a minimum 2-foot thick layer of structural fill should be placed between the stone columns and the foundations and 1-foot below slabs-on-grade if Stone Column ground improvement is used at the site. This layer of structural fill would help transfer loads from the foundations and slab-on-grade to the stone columns and would help reduce differential settlement. Following preloading, shallow foundation should also be supported on at least 2 feet of structural fill.

Mat Foundations

Allowable Bearing Pressure. Mat foundations may be designed using a modulus of subgrade reaction of 50 pounds per cubic inch bearing on a minimum of 2 feet of structural fill placed over improved native soils.

The allowable soil bearing values apply to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

Settlement. Provided all loose soil is removed and the subgrade is prepared as recommended under "Construction Considerations" below, we estimate the total settlement of mat foundations will be on the order of 1 to 2 inches. The settlements will occur rapidly, essentially as loads are applied. Differential settlements are expected to be less than ½ inch over a 50-foot length.

Conventional Shallow Foundations

Allowable Bearing Pressure. Footings may be designed using a maximum net allowable soil bearing value of 2.5 kips per square foot (ksf) on properly compacted structural fill above improved ground (stone columns or preloaded). The net allowable soil bearing values apply to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. If stone columns are installed and underlie a granular pad below the foundations, an allowable soil bearing value of 4 kips per square foot (ksf) may be used.

Size and Embedment. Exterior footings should be founded a minimum of 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below top of slab. Continuous wall footings and individual column footings should have minimum widths of 18 and 24 inches, respectively.

Settlement. Provided all loose soil is removed and the subgrade is prepared as recommended under "Construction Considerations" below, total settlement of shallow foundations are anticipated to be on the order of ½ to 1 inch. The settlements will occur rapidly, essentially as loads are applied. Differential settlements measured along 25 feet of wall foundations or between similarly loaded column footings are expected to be less than ½ inch. One to two inches of secondary settlement will occur over a 50-year design life in areas where stone columns are not installed.

Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the footings. For footings supported on native soils or on structural fill placed and compacted in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 350 pounds per cubic foot (pcf) (triangular distribution). This value is appropriate for foundation elements that are poured directly against undisturbed native soils or surrounded by structural fill. The structural fill should extend out from the face of the foundation element for a distance at least equal to three times the height of the element and be compacted to at least 95 percent of the MDD.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

Construction Considerations

Immediately prior to placing concrete, all debris and soil slough that accumulated in the footings during forming and steel placement must be removed. Debris or loose soils not removed from the footing excavations will result in increased settlement. We recommend that the footing excavations be cut using a smooth-edged bucket to reduce the amount of disturbed soil exposed at the subgrade.

The condition of all footing excavations should be observed by the Geotechnical Engineer to evaluate if the work is completed in accordance with our recommendations and that the subsurface conditions are as anticipated.

Footing Drains

We recommend that perimeter footing drains be installed around the buildings. The perimeter drains should be installed at the base of the exterior footings. The perimeter drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non woven geotextile fabric such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material. We recommend that the drainpipe consist of either heavy-wall solid pipe (SDR-35 PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). We recommend against using flexible tubing for footing drainpipes. The drainage material should consist of pea gravel or "Gravel Backfill for Drains" per Washington State Department of Transportation (WSDOT) standard specifications Section 9-03.12(4). The perimeter drains should be sloped to drain by gravity, if practicable, to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered, and be placed in flush mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

PILE FOUNDATIONS

General

Pile foundations may be required for support of heavily loaded buildings or sensitive facilities. Preliminary recommendations for driven timber piles, steel piles, and augercast piles are provided below. We can provide alternate deep foundation design and more detailed recommendations for support of structures, if necessary, when the final layout and foundation loads are determined during final design.

Axial Capacity

Based on the conditions encountered in our explorations, pile foundations will be resisted primarily by friction in the medium dense to dense sand generally encountered below the fine-grained deposits. The upper surface of the sand layer is shown on Figure 11. We recommend that pile tips extend at least 10 to 15 feet into the dense sand deposits. Piles may need to be at least 40 to 60 feet long in the north half of the site and at the south end of the site. However, in the south-central area where a former river channel may exist, piles may need to extend 70 to 110 feet below the ground surface. Recommended pile capacities for 8-inch-diameter timber piles, 12-inch-diameter closed-end steel pipe piles, and 14-inch-diameter augercast piles are provided in Table 4. Because of the corrosive environment in organic deposits, the steel and timber piles will need to be protected from corrosion.

Table 4. Axial Capacity

Pile Type	Recommended Minimum Tip Depth Below Grade	Allowable Axial Compression Load (tons)	Allowable Uplift (tons)
8-inch-Diameter Timber	15 feet into sand	25	5
12-inch Closed-End Pipe	15 feet into sand	45	15
14-inch Augercast	15 feet into sand	60	25

These preliminary values are for the total of dead and long-term loads and may be increased by one-third

when considering design loads of short duration such as wind or seismic forces. The allowable axial capacities presented above are based on the strength of the supporting soils for the penetrations indicated and include a factor of safety of at least 2. These preliminary capacities apply to single piles and final design recommendations should be provided based on the layout of the buildings and the building loads. If piles within groups are spaced at least three pile diameters on-center, no reduction for pile group action need be made. The structural characteristics of pile materials and structural connections might impose limitations on pile capacities and should be evaluated by the structural engineer. The timber and steel piles should be driven to obtain the design capacity based on refusal criteria developed during the initial phases of driving. Installation recommendations for all pile types are provided in a following section, "Pile Installation."

Pile downdrag forces develop when surrounding compressible soils settle relative to a pile, thus interacting with and adding load to the pile. The recommendations presented above for allowable axial capacity in compression include the effects of downdrag on the piles, provided that significant new fill thicknesses are not placed above existing grade (less than about 2 feet). We recommend that all fills/preloads be placed and allowed to settle prior to constructing piles.

Lateral Pile Capacity

Preliminary ultimate lateral capacities for the various pile types are presented in Table 5. These lateral capacities are based on a center-to-center pile spacing of at least three pile diameters, adequate steel reinforcement for the first line of piles in a pile groups and are in the augercast piles, and pile-head fixity against rotation. The capacities are based on a maximum pile deflection of approximately ½ inch. Piles behind the first row of piles in pile groups have reduced lateral capacities because of shadowing effects. For preliminary estimating purpose, the lateral capacities of piles in trailing rows can be estimated as one-half the values presented in Table 5.

Table 5. Ultimate Lateral Capacity

Pile Type	Ultimate Lateral Capacity (kips)
8-inch-diameter Timber	6
12-inch-diameter Steel, 3/8-inch Wall Thickness	20
14-inch-diameter Augercast	22

Resistance to lateral loads can also be developed by passive earth pressure on the faces of pile caps and other buried foundation elements. The passive pressure recommended in the "Lateral Resistance" subsection of "Conventional Shallow Foundations" can be used to estimate the lateral resistance against the sides of pile caps.

Settlement

We estimate that settlement of the pile foundations, designed and installed as recommended, will be on the order of 1/2 inch or less. Most of this settlement will occur rapidly as loads are applied. Post-construction differential settlements should be negligible.

Pile Installation

Augercast Piles: Augercast piles should be installed to the recommended penetrations using a continuous-flight, hollow-stem auger. The pile grout is pumped under pressure through the hollow-stem

as the auger is slowly withdrawn. Reinforcing steel for bending and uplift is placed in the fresh grout column immediately after withdrawal of the auger.

We recommend that the augercast piles be installed by a contractor experienced in their placement and using suitable equipment. Grout pumps should be fitted with a volume-measuring device and pressure gauge so that the volume of grout placed in each pile and the pressure head can be readily determined. While grouting, the rate of auger withdrawal should be controlled such that the rate is uniform and the volume of grout pumped is equivalent to at least 115 percent of the theoretical hole volume. A minimum grout line pressure of 100 psi should be maintained while grouting. We recommend that there be a waiting period of at least eight hours between installation of piles spaced closer than 8 feet center-to-center, in order to avoid disturbance of concrete undergoing curing in a previously cast pile.

It should be noted that the recommended pile embedments and allowable capacities presented above are based on assumed uniformity of soil conditions between the explorations. Obstructions could be encountered within the fill soils during installation such that new pile locations may need to be selected and/or pile capacities may need to be re-evaluated. In addition, obstructions (timber logs) are often encountered within the upper alluvial soils at the site. There may be unexpected variations in the depth to and characteristics of the supporting soils across the site. In addition, no direct information regarding the capacity of augercast piles (e.g., driving resistance data) is obtained while this type of pile is being installed. Therefore, it is particularly important that the installation of augercast piles be carefully monitored by staff from our office.

Driven Piles: Steel and timber piles should be driven to obtain the design capacity based on refusal criteria developed during the initial phases of driving (discussed below). The piles should be installed using an appropriately sized pile hammer. The pile hammer should be of sufficient size to drive the pile to refusal without damaging the pile. Proper selection of pile hammer can reduce pile damage during driving. Since the contractor has control of materials handling and driving equipment, we recommend that the contractor be made responsible for installing an acceptable pile to design depths without damaging the piles. Based on our experience with similar projects, we anticipate that a pile hammer with a rated energy of 15,000 to 20,000 foot-pounds may be appropriate. We should be consulted to provide recommendations for the specific size of hammer to be used and to evaluate appropriate refusal criteria, once the contractor has been selected. We recommend that the installation of all piles be monitored by a member of our staff who would observe installation procedures and evaluate the adequacy of individual pile installations.

It should be noted that the preliminary pile penetrations and allowable capacities presented above are based on the subsurface conditions encountered in our explorations. Piles with insufficient resistance at the end of driving may require splicing and additional driving to meet the required refusal criteria or additional piles may be necessary to support the design loads.

SLAB-ON-GRADE FLOORS

Slabs may be supported on-grade providing the subgrade soils are prepared as recommended under the "Earthwork" section of this report and that the preload is applied in accordance with the recommendations of the preloading/settlement section of this report. We recommend that the slabs be founded on structural fill placed over the native soils. For slabs designed as a beam on an elastic foundation, a subgrade modulus of 50 pounds per cubic inch (pci) may be used for subgrade soils prepared as recommended.

We recommend that the slab be underlain by a 4-inch minimum thickness of 1-½ inch minus crushed rock with negligible sand and silt. The base course will provide uniform support and serve as a capillary break to reduce moisture migration through the slab.

If moisture-sensitive coverings are used on interior floor slabs, a vapor barrier consisting of plastic sheeting should be installed between the slab and the base course. The contractor should be made responsible for maintaining the integrity of the vapor barrier during construction.

PAVEMENT SUBGRADE PREPARATION

New pavement sections must be installed over a dense and unyielding subgrade. Structural fill placed to establish subgrade elevation in pavement areas should be compacted to at least 90 percent, except the upper 24 inches of the subgrade, which should be compacted to a minimum of 95 percent of its maximum dry density, as determined using test method ASTM D 1557. We recommend that the upper 12 inches of the existing soils, when encountered at subgrade elevation, be compacted to at least 95 percent of the maximum dry density per ASTM D 1557 if the work is completed during extended periods of dry weather. For wet weather construction, we recommend placing a 12-inch-thick subbase layer of imported fill meeting the requirements for gravel borrow as described in Section 9-03.14(1) of the 2006 WSDOT Standard Specifications with the exception that the fines content (percent passing the U.S. No. 200 sieve) should be less than 5 percent. The subbase should be placed after the utilities have been installed to reduce the risk of disturbance from construction equipment.

Prior to the placement of base course materials, the exposed subgrade should be proof rolled. Proof rolling should be accomplished with a loaded dump truck or equivalent piece of equipment. The purpose of this effort is to identify possible loose or soft soil and recompact disturbed areas of subgrade.

Proof rolling should be carefully observed by qualified geotechnical personnel. Areas exhibiting significant deflection, pumping, or saturated soils that cannot be readily compacted should be overexcavated to firm soil. Overexcavated areas should be backfilled with compacted granular fill. During periods of wet weather, proof rolling could damage the exposed subgrade. Under these conditions, qualified geotechnical personnel should observe subgrade conditions to determine if proof rolling is feasible.

DRAINAGE CONSIDERATIONS

We recommend that all paved and landscaped areas be graded so that surface drainage is directed away from the buildings to appropriate catch basins. Pavement surfaces and open space areas should be sloped such that the surface water is collected and routed to suitable discharge points.

Water collected in roof downspout lines must not be routed or discharged into the perforated pipes intended for providing footing or wall drainage.

RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

Throughout this report, recommendations are provided where we consider additional geotechnical services to be appropriate. These additional services are summarized below:

- The recommendations provided in this report should be considered preliminary. Final geotechnical recommendations should be provided based on the actual building types and layouts developed during the design phase.

- Additional subsurface explorations may be needed to better understand the soil conditions under planned structures and across the site.
- GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.
- GeoEngineers should be retained to develop/provide input regarding the ground improvement performance specification.
- During construction, GeoEngineers should evaluate the suitability of the foundation, pavement and slab subgrades, observe installation of ground improvement and subsurface drainage measures, and provide a summary letter of our construction observation services. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix D, Report Limitations and Guidelines for Use.

LIMITATIONS

We have prepared this report for use by OliverMcMillan Everett LLC, and their authorized agents for planning purposes in development of the Simpson Pad as part of the Everett Riverfront Redevelopment project located in Everett, Washington.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix D, titled Report Limitations and Guidelines for Use, for additional information pertaining to use of this report.

REFERENCES

American Society of Testing and Materials, "ASTM Book of Standards Volume 4.08: Construction: Soil and Rock (I): D 420 - D 5611", 2007.

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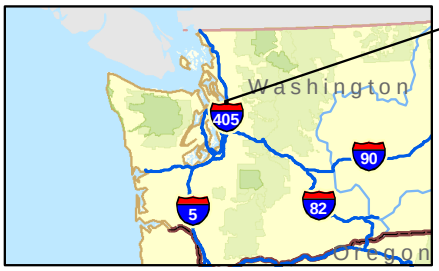
The Floyd & Snider Team “Existing Conditions and Precedent Site Memos”, dated May 28, 1999.

U.S. Geological Survey – National Seismic Hazard Mapping Project – Interactive Deaggregations.

WSDOT/APWA. 2006. Standard Specifications for Road, Bridge, and Municipal Construction. Washington State Department of Transportation and the American Public Works Association.



N
W E
S

2,000 0 2,000
Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information 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 of this communication.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources: ESRI Data & Maps, Street Maps 2005

Transverse Mercator, Zone 10 N North, North American Datum 1983
North arrow oriented to grid north

Vicinity Map

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 1

Office: EVER Path: P:\6\6191002\04\GIS\619100201Figure1.mxd Map Revised: February 15, 2007

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information 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 of this communication.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources:
Aerial image obtained from TerraServer dated Thursday, June 13, 2002.
OHWM and Snohomish River delineated from sketch provided by GeoEngineers field staff, aerial image and GPS data collected with Trimble GeoXT unit by field crew on December 12 and 22, 2006.
Universal Transverse Mercator, Zone 10, North American Datum 1983

Legend

Site Boundary

700 0 700
Feet

Project Location Map

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

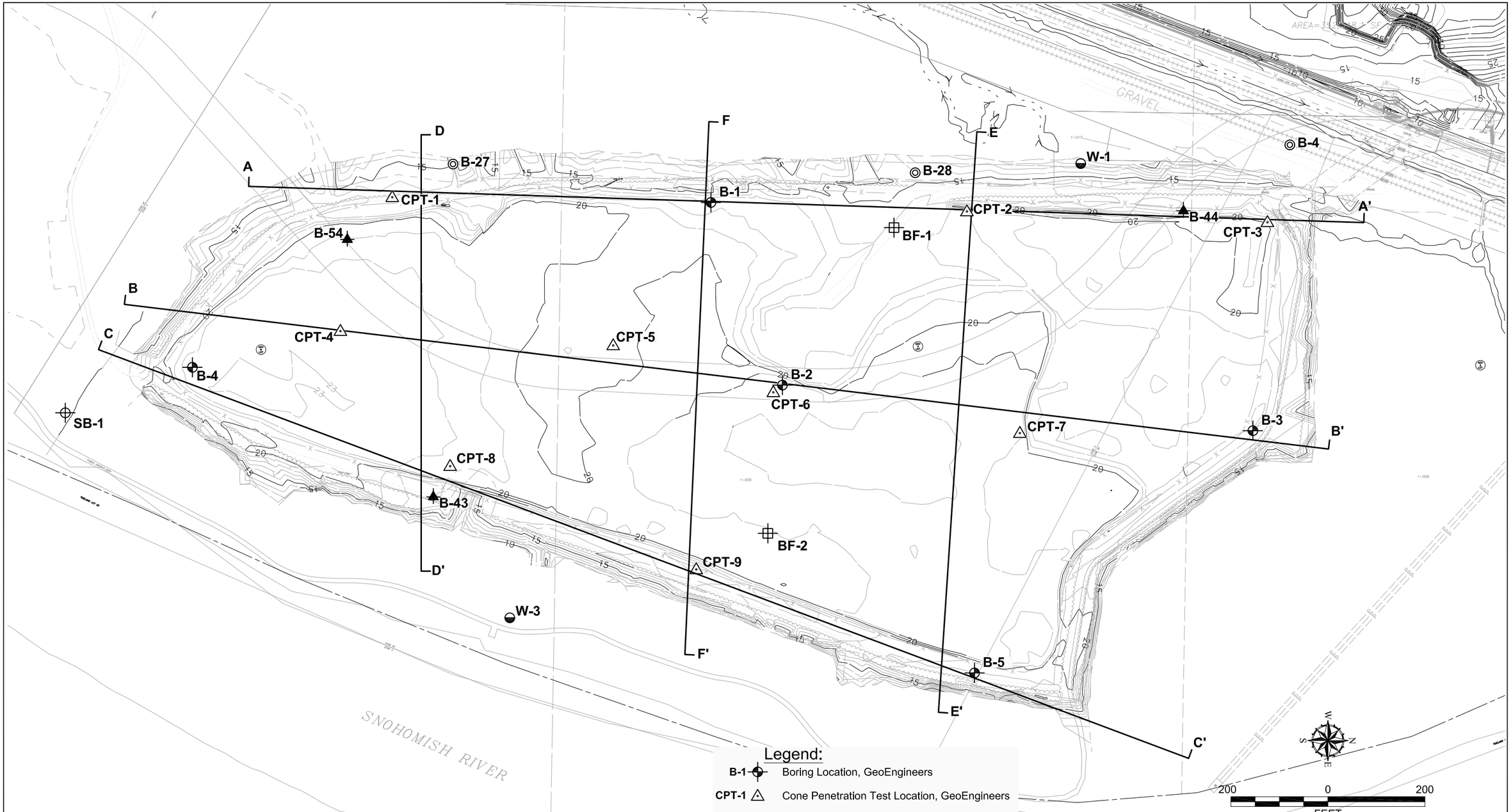
GeoEngineers

Figure 2

Map Revised: February 15, 2007

Path: P:\06\06191002\04\GIS\Siteboundary_LayoutFigure2.mxd

Office: EVER



Legend:

B-1	Boring Location, GeoEngineers
CPT-1	Cone Penetration Test Location, GeoEngineers
B-4	Boring Location, Shannon & Wilson, 1990
B-44	Boring Location, Floyd & Snider Team, 1999
SB-1	Boring Location, EMR-Northwest (Pacific Groundwater Group)
W-1	Monitoring Well, EMR-Northwest, 1990
BF-1	Boring Location, HWA GeoSciences, Inc., 2003
A — A'	Subsurface Profile

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information 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 of this communication.

Reference: Drawing from AutoCAD file name CAD Site Survey.dwg

Site Plan

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 3

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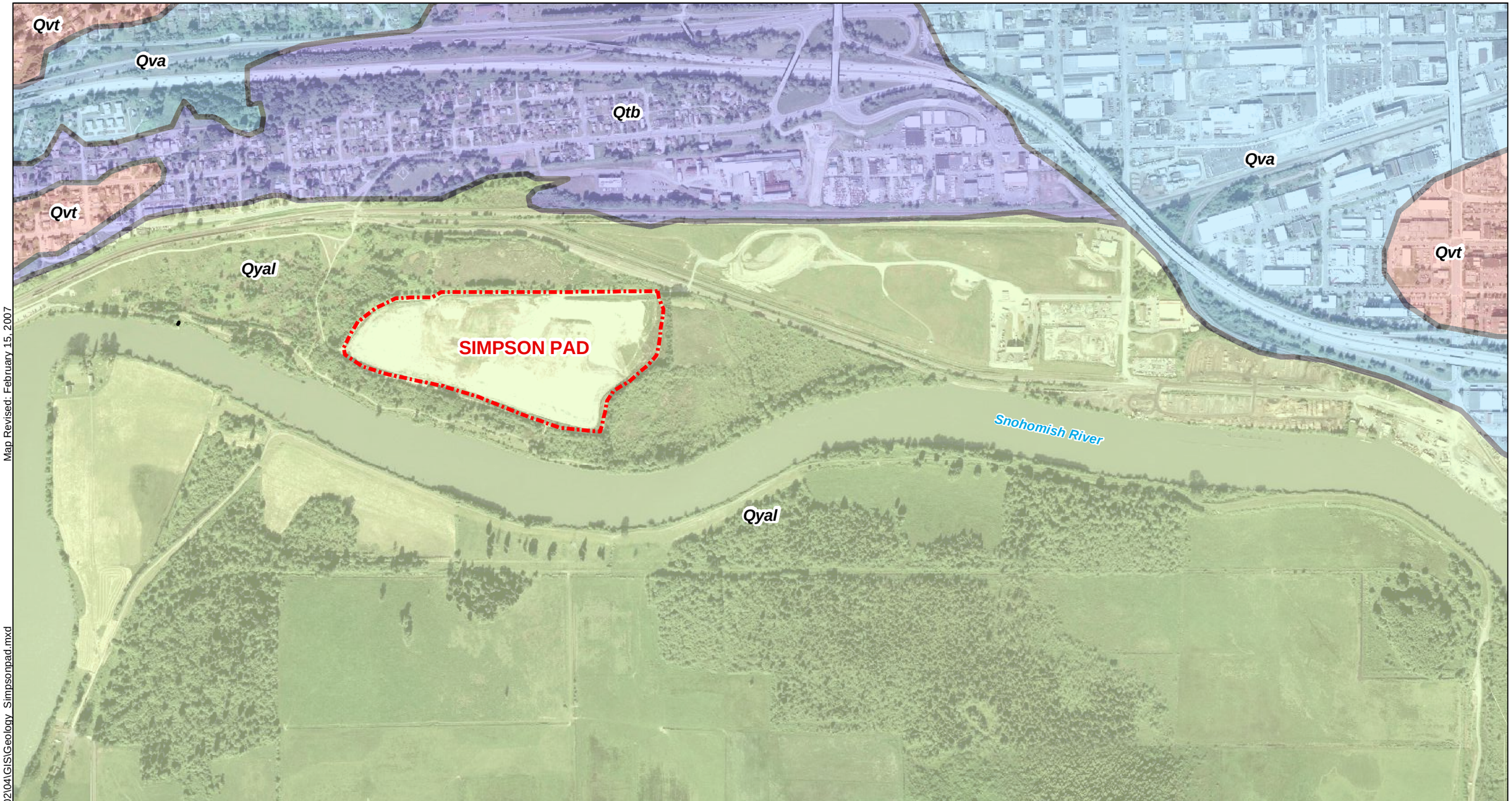


Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources:
Aerial image obtained from TerraServer dated Thursday, June 13, 2002.
Geology digitized USGS Geologic Map of Everett 7.5 Quadrangle
Snohomish County, WA. James P Minard 1985

Washingto State Plane, North, North American Datum 1983

Legend

Site Boundary

Geologic Unit

- Qtb - Transitional Beds (Fraser Glaciation to Pre-Frase Glaciation)
- Qva - Advance Outwash
- Qvt - Till
- Qyal - Younger Alluvium (Holocene)

W S N E

700 0 700
Feet

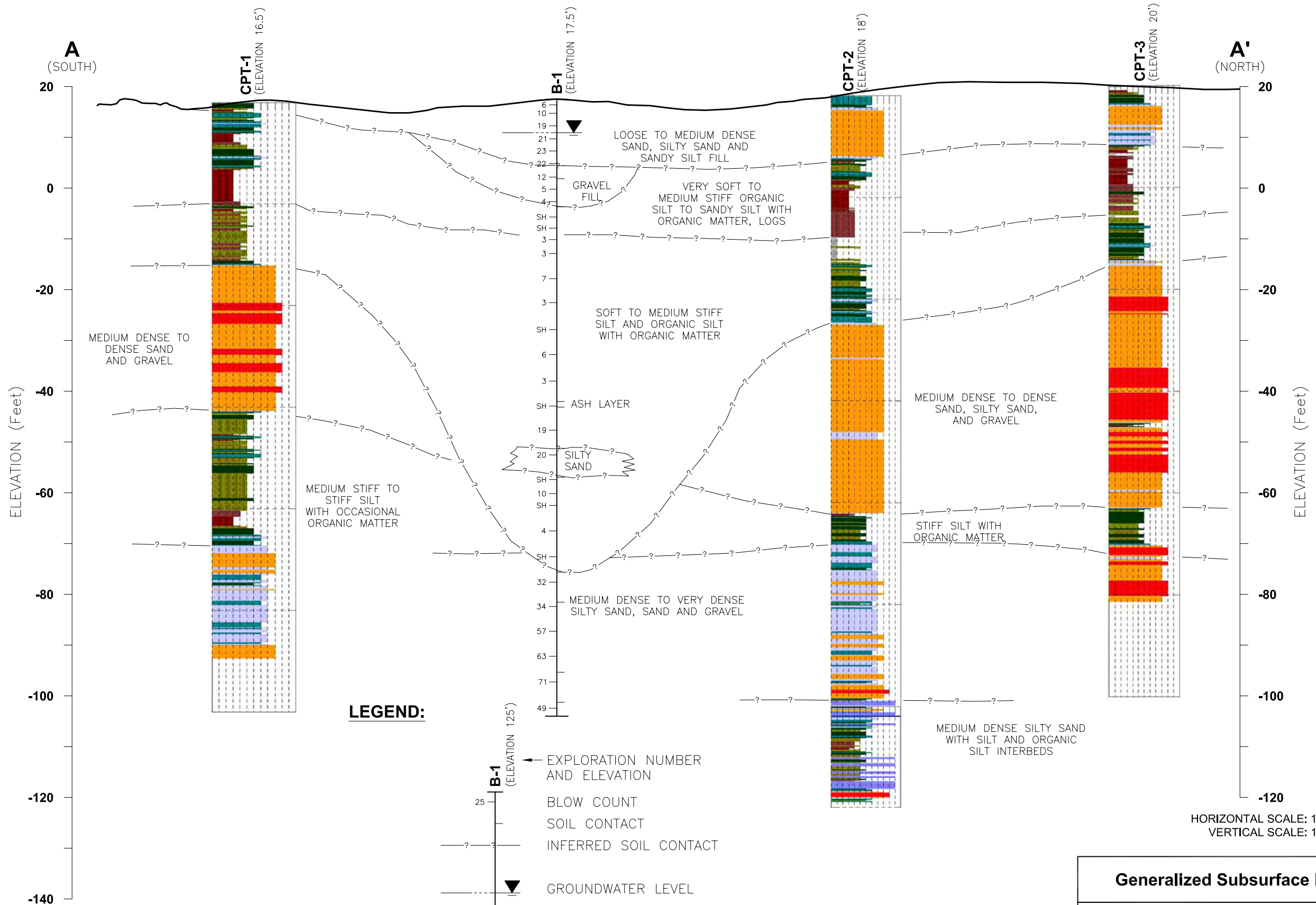
Geology Map

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GeoEngineers

Figure 4

Office: EVER Path: P:\06\06191002\04\GIS\Geology_Simpsonpad.mxd Map Revised: February 15, 2007



LEGEND:

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 (*)

NOTES:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 3 for location of Profile A-A'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

Generalized Subsurface Profile A-A'

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 5

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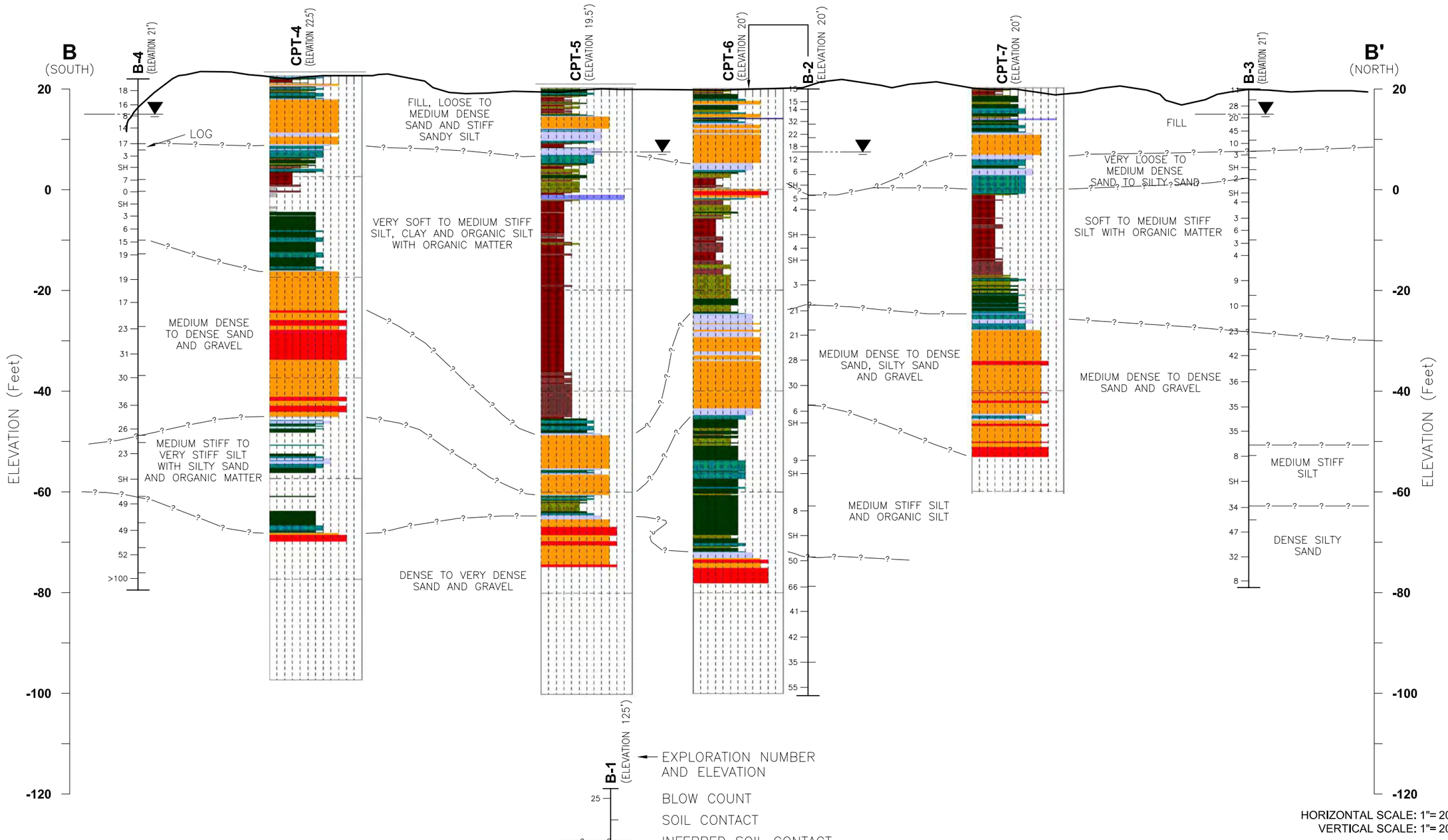

HORIZONTAL SCALE: 1"= 200'
VERTICAL SCALE: 1"= 20'

NOTES:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 3 for location of Profile B-B'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

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 (*)

Generalized Subsurface Profile B-B'

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 6

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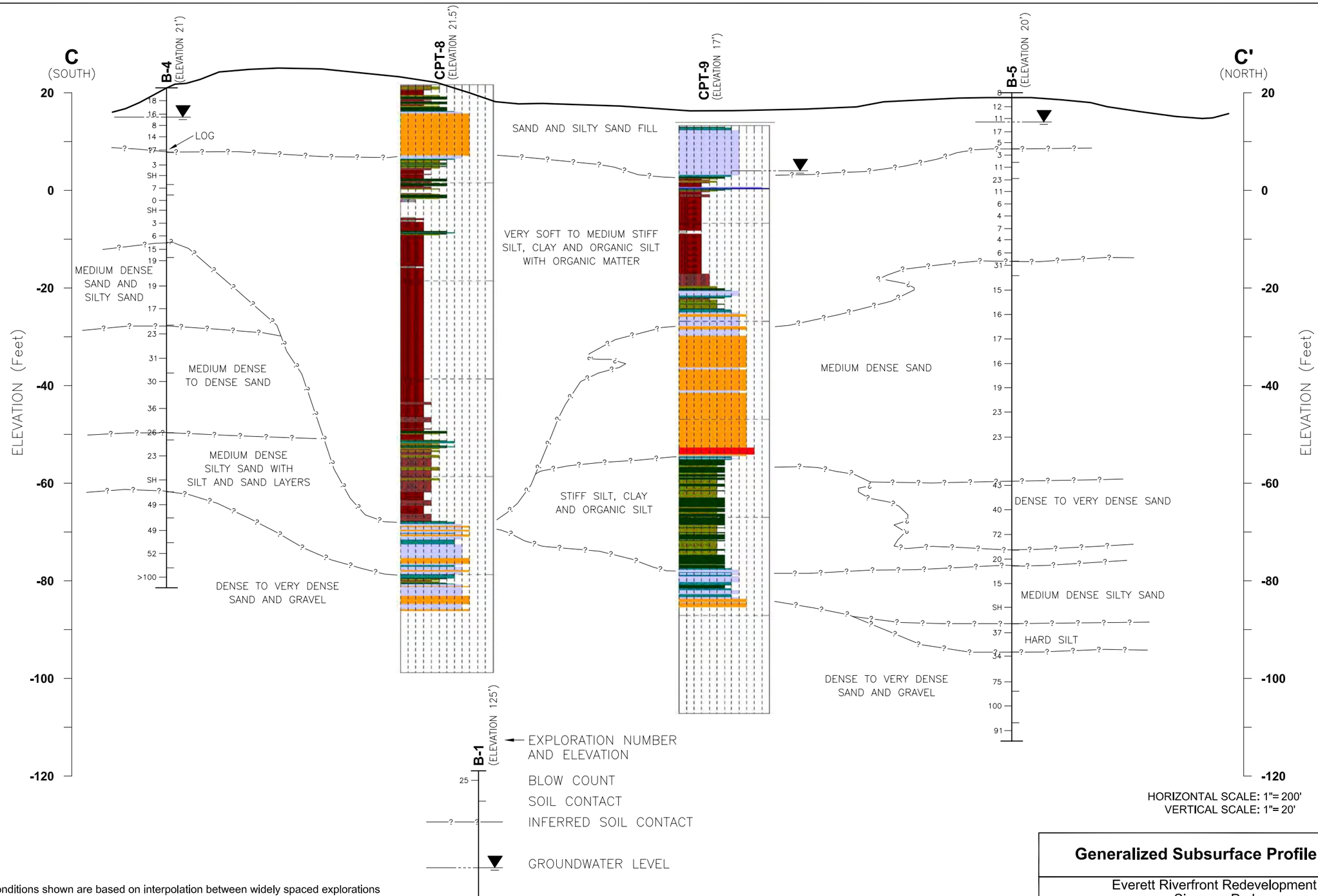

NOTES:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 3 for location of Profile C-C'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

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 (*)

Generalized Subsurface Profile C-C'

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 7

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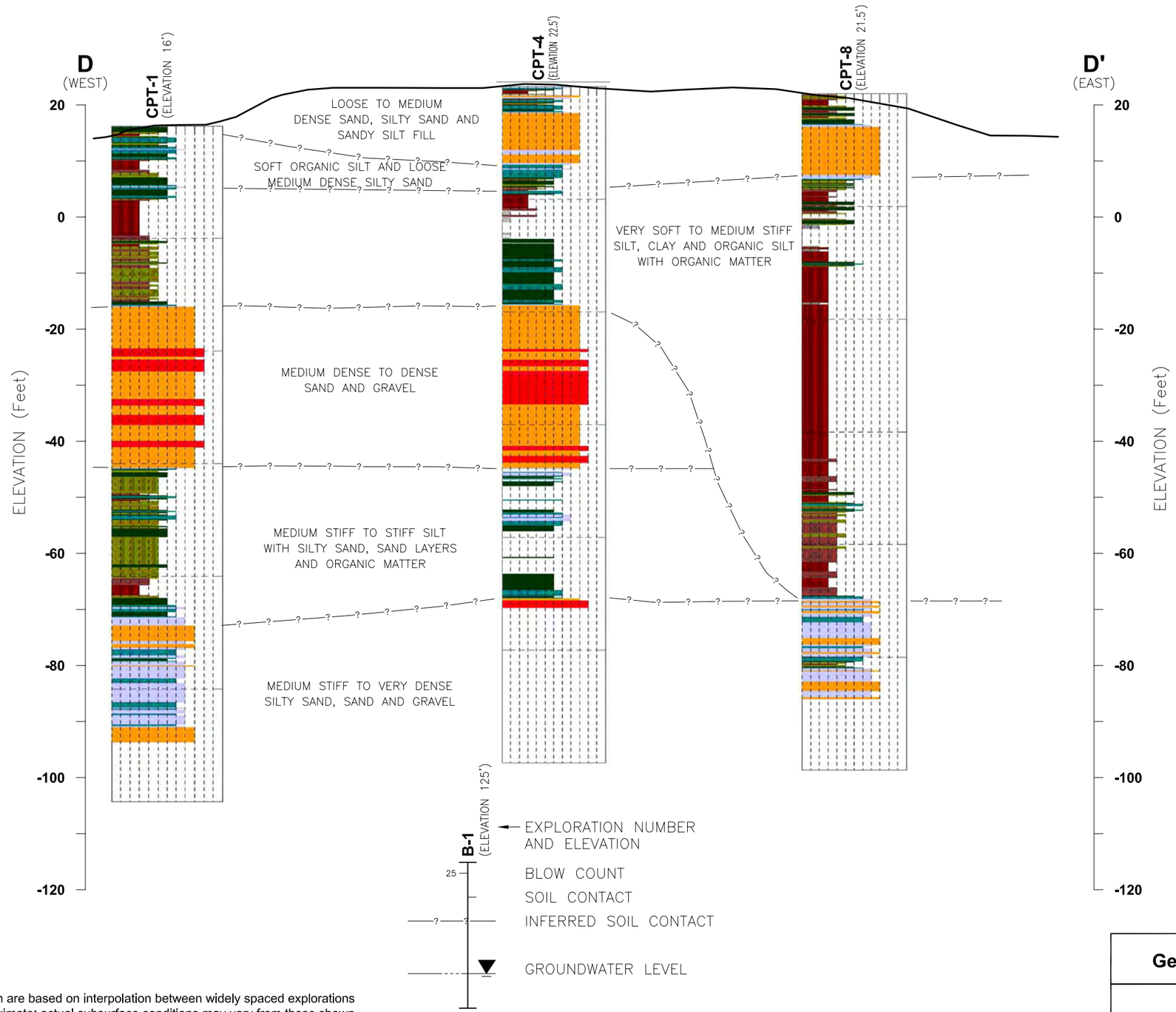

HORIZONTAL SCALE: 1"= 100'
VERTICAL SCALE: 1"= 20'

NOTES:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 3 for location of Profile D-D'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

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 (*)

Generalized Subsurface Profile D-D'

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 8

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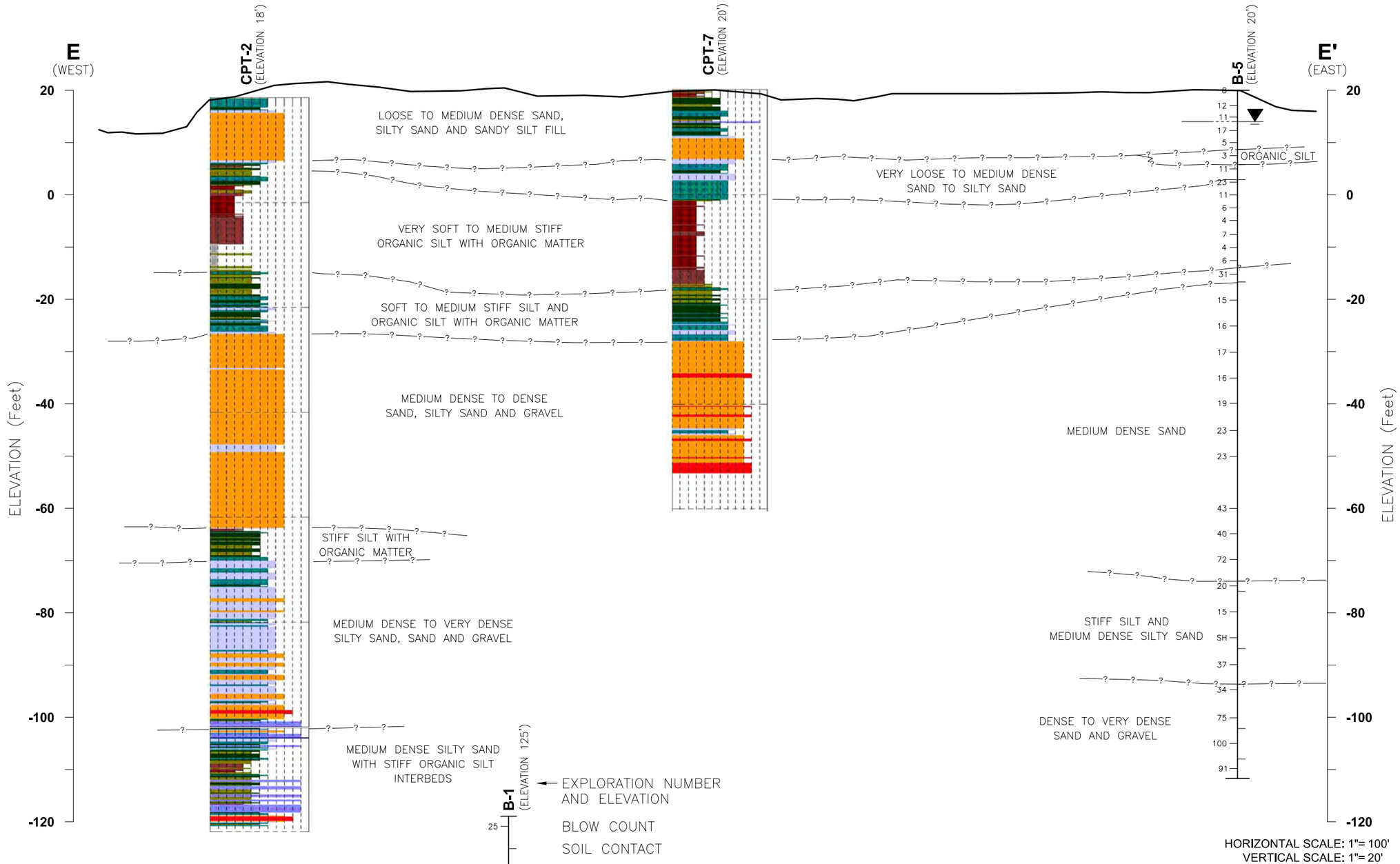


NOTES:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 3 for location of Profile E-E'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

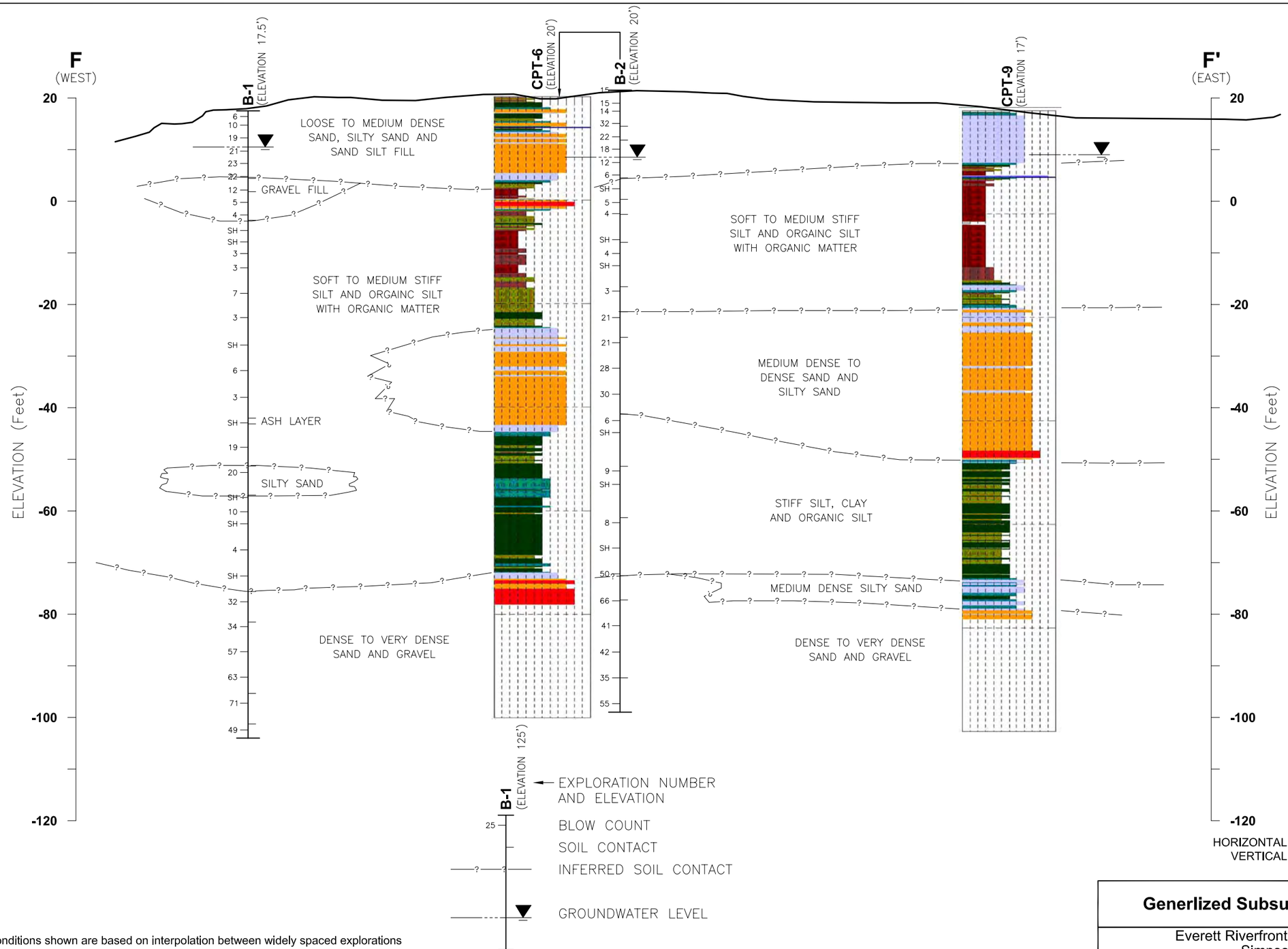
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

Generalized Subsurface Profile E-E'

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 9



HORIZONTAL SCALE: 1"= 100'
VERTICAL SCALE: 1"= 20'

NOTES:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. Refer to Figure 3 for location of Profile F-F'.
3. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record.

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 (*)

Generlized Subsurface Profile F-F'

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 10

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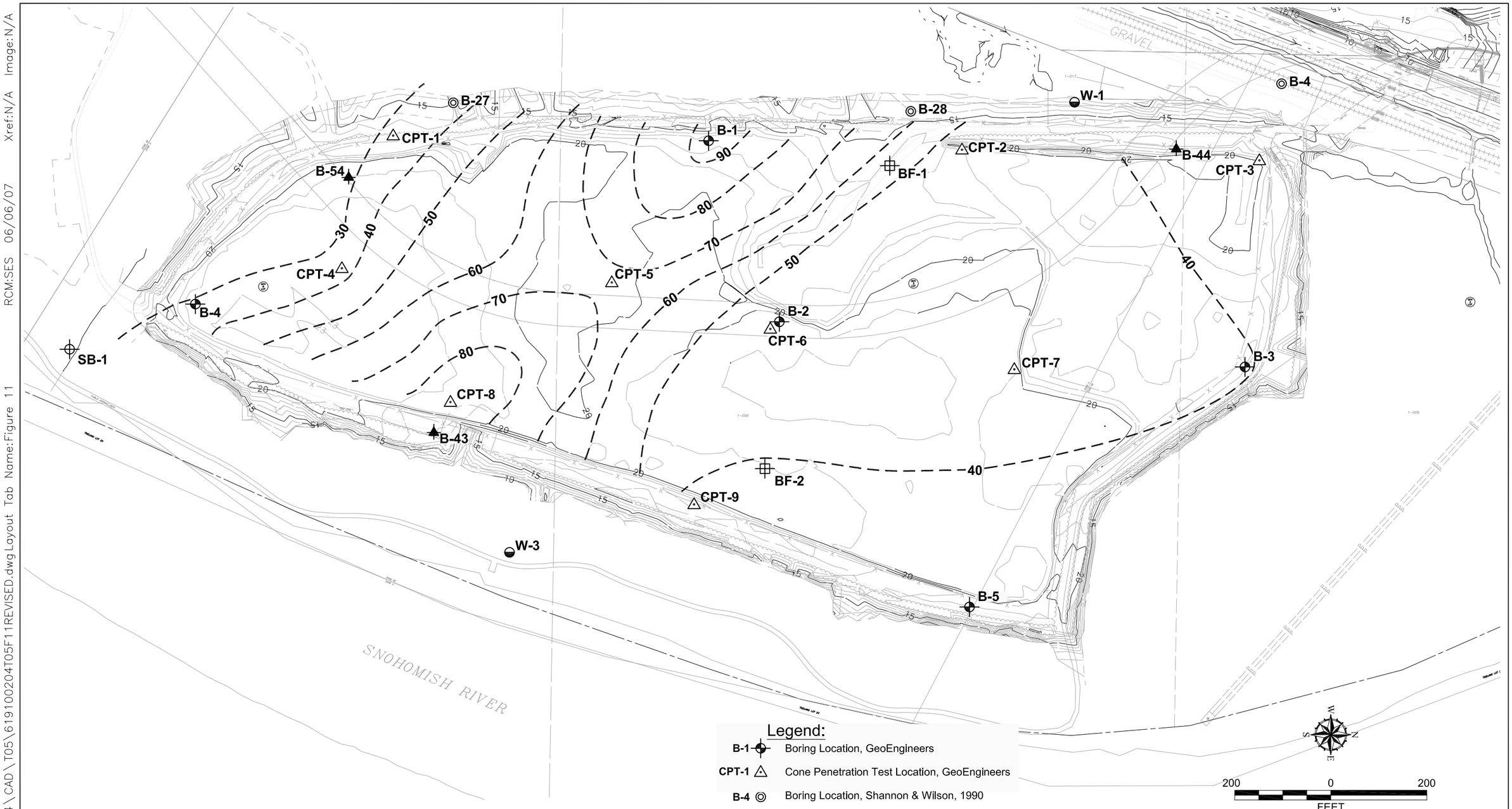
Ever \P:\ 6191002\04\CAD\T05\61910020 4T05F11REVISED.dwg Layout Tab Name:Figure 11 RCM:SES 06/06/07 Xref:N/A Image:N/A



Legend:

B-1	Boring Location, GeoEngineers
CPT-1	Cone Penetration Test Location, GeoEngineers
B-4	Boring Location, Shannon & Wilson, 1990
B-44	Boring Location, Floyd & Snider Team, 1999
SB-1	Boring Location, EMR-Northwest (Pacific Groundwater Group)
W-1	Monitoring Well, EMR-Northwest, 1990
BF-1	Boring Location, HWA GeoSciences, Inc., 2003
80	Top of Upper Sand Deposit Contour (Estimated)

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information 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 of this communication.

Reference: Drawing from AutoCAD file name CAD Site Survey.dwg

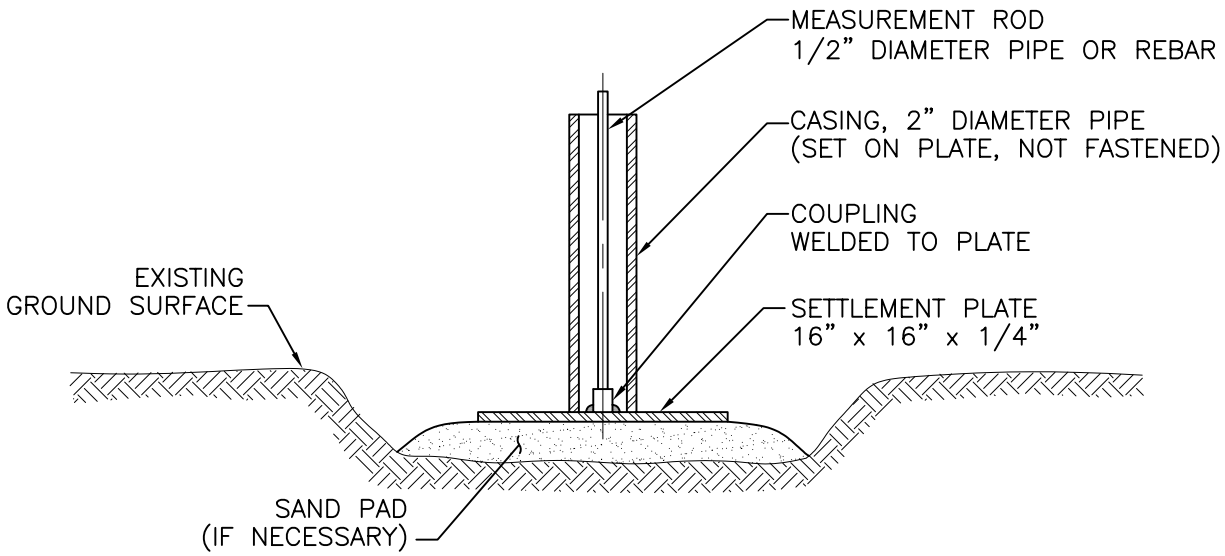
Depth to Upper Sand Deposit

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 11

Ever \P:\ 6191002\04\CAD\T05\6191002O4T05F12.dwg Layout Tab Name:Figure 12 CMK:SES 05/03/07 Xref:N/A Image:N/A



NOT TO SCALE

Notes:

1. Install markers on firm ground or on sand pads if needed for stability. Take initial reading on top of rod and at adjacent ground level prior to placement of any fill.

2. For ease in handling, rod and casing are usually installed in 5-foot sections. As fill progresses, couplings are used to install additional lengths. Continuity is maintained by reading the top of the measurement rod, then immediately adding the new section and reading the top of the added rod. Both readings are recorded.

3. Record the elevation of the top of the measurement rod in each marker at the recommended time intervals. Each time, note the elevation of the adjacent fill surface.

4. Read the marker to the nearest 0.01 foot, or 0.005 foot if possible. Note the fill elevation to the nearest 0.1 foot.

5. The elevations should be referenced to a temporary benchmark located on stable ground at least 100 feet from the embankment.

Settlement Plate Detail

Everett Riverfront Redevelopment
Simpson Pad
Everett, Washington

GEOENGINEERS

Figure 12

APPENDIX A
FIELD EXPLORATIONS

APPENDIX A
FIELD EXPLORATION

GENERAL

Subsurface conditions were explored at the site by drilling five borings and advancing nine cone penetration tests (CPTs). The locations and elevations of the explorations were estimated in the field by taping from existing site features and by using a hand-held global positioning device. The approximate exploration locations are shown on the Site Plan, Figure 3.

SOIL BORINGS

Five borings, designated B-1 through B-5, were drilled using either a track-mounted (CME-850) or a truck-mounted (Mobile B-61) drill rig owned and operated by Boart Longyear on January 23 through January 30, 2007. The borings were advanced to depths ranging from 99½ to 131½ feet below the existing ground surface. The borings were continuously monitored by a geotechnical engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and prepared detailed logs of the borings. Soils were visually classified in general accordance with ASTM D 2488, which is described in Figure A-1. A key to the boring log symbols is shown in Figure A-1.

The samples were obtained using either a 2.4-inch inside-diameter, split-barrel sampler driven into the soil using a 140-pound hammer free-falling 30 inches or a Standard Penetration Test (SPT) sampler using a 140-pound hammer. The number of blows required to drive the samplers the last 12 inches or other indicated distances are recorded on the boring logs. Select samples of very soft soils were also obtained using direct-push Shelby tube samplers. Logs of the borings are shown on Figures A-2 through A-6. Observations of groundwater conditions were made during drilling and are presented on the boring logs. Groundwater conditions observed during drilling should be considered approximate as they represent a short term condition and may or may not be representative of the long term groundwater conditions at the site. The logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. They also indicate the depths at which these soils or their characteristics change; although, the change might actually be gradual.

CONE PENETRATION TESTS (CPT)

Subsurface soil conditions were also explored by completing nine CPT soundings (designated CPT-1 through CPT-9) at the site on January 22 through January 30, 2007. Track and truck-mounted equipment were used to advance the nine CPTs to depths ranging from 73 to 139 feet below existing ground surface. CPT-5 was relocated twice due to refusal (possible logs) encountered at depths of 25½ and 29 feet.

Northwest Cone Explorations, Inc. provided cone penetration testing services under subcontract to GeoEngineers. The CPTs were completed by advancing a small-diameter steel tip with an adjacent sleeve with hydraulically operated equipment. Measurements of tip and sleeve resistance allow interpretation of the soil profile and the density/consistency and shear strength. A piezometer-equipped cone tip was used allowing pore water pressure measurements. Logs of the CPTs are shown on Figures A-7 through A-15. The logs were developed concurrently with the tests.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	CC	Cement Concrete
	AC	Asphalt Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Stratigraphic Contact

Distinct contact between soil strata or geologic units

Gradual change between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE A-1

Date(s) Drilled	01/24/07, 01/25/07	Logged By	SH	Checked By	RCM
Drilling Contractor	Boart Longyear/Holt Drilling	Drilling Method	Mud Rotary	Sampling Methods	SPT/DM/Shelby Tube
Auger Data	4-7/8" Tricone	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	CME-850 Track Rig
Total Depth (ft)	121.5	Surface Elevation (ft)	17.5	Groundwater Elevation (ft)	10.5
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0		10	6		1			SM	Gray silty sand with gravel, brick debris (loose to medium dense, moist) (fill)			
15			6	10		2					12		SA, %F=34
										Becomes medium dense			
	5		9	19		3					14		
10			0	21		4							
	10		0	23		5							
5			8	22		6			GP-GM	Gray gravel with silt and sand (medium dense, wet) (fill)	7		
	15		6	12		7			GP	Brown/white/gray fine gravel with medium to coarse sand (loose to medium dense, moist) (fill)			
0			14	5		8							
	20		6	4		9a							
						9b			OH	Dark brown organic silt with organic matter including twig fragments (soft, moist)	145		
-5			0	P		10							
	25		24	P		11			MH	Dark gray elastic silt with fine sand and trace organic matter (soft, wet)	79		AL LL=52, PI=19
-10			18	3		12			ML	Brown and gray silt with organic matter including wood fragments (soft, moist)	79		
	30		18	3		13					80		
-15													
	35												

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

LOG OF BORING B-1

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-2
Sheet 1 of 3

V6_GTBORING P:\6\6191002\04\FINAL\619100204.GPJ GEIV6_1.GDT 5/7/07

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	35		6	7		14							
-20													
	40		18	3		15					82		
-25													
	45		0	P		16							
-30													
	50		0	6		17							
-35													
	55		12	3		18			OH	Brown organic silt (soft, moist)	70		AL LL=95, PI=31
-40													
	60		0	P		19							
-45													
	65		10	19		20a			ML	White/gray volcanic ash (medium stiff, moist)			
						20b			ML	Gray silt with organic matter (stiff, moist)	58		
-50													
									SM	Gray silty sand (medium dense, moist)			
	70		12	20		21				1/4 inch thick gray/white volcanic ash layer	27		SA, %F=33
-55													
	75		0	P		22			MH	Gray elastic silt with organic matter (medium stiff to stiff, wet)			
-60			12	10		23					42		

LOG OF BORING B-1 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-2
Sheet 2 of 3

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	80	12	P	24				Increasing fine sand content	39	81	Consol, AL LL=84, PI=23
-65											
	85	18	4	25					66		
-70							ML	Black/gray silt (soft, moist)			
	90	16	P	26							
-75							SM	Gray silty fine sand (dense, wet)			
	95	12	32	27							
-80											
	100	8	34	28			SP	Gray and black fine to coarse sand with gravel (dense, moist)	12		Hard drilling
-85											
	105	10	57	29				Becomes very dense	11		
-90							SP	Gray fine to medium sand with silt interbeds (very dense, wet)			
	110	12	63	30							Hard drilling
-95							GP	Gray gravel with fine to coarse sand (very dense, wet)			
	115	3	71	31							Rock in shoe
-100							SP	Black/gray/white fine to medium sand (very dense, moist)			
	120	12	49	32							

LOG OF BORING B-1 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-2
Sheet 3 of 3

Date(s) Drilled	01/25/07, 01/26/07	Logged By	SH	Checked By	RCM
Drilling Contractor	Boart Longyear/Holt Drilling	Drilling Method	Mud Rotary	Sampling Methods	SPT/Shelby Tube
Auger Data	4-7/8" Tricone	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	CME-850 Track Rig
Total Depth (ft)	121.5	Surface Elevation (ft)	21.5	Groundwater Elevation (ft)	7.5
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0		8	5		1			TS	6 inches brown fine sandy silt (topsoil) (soft, moist)			
20			8	15		2			ML	Gray fine sandy silt with coarse sand and gravel, slight plasticity (stiff, moist) (fill)	16		
	5		4	14		3					12		
15			12	32		4			SM	Brown silty fine to medium sand (dense, moist) (fill)	11		
	10		14	22		5			SP-SM	Gray fine to medium sand with silt (medium dense, moist) (fill)	20		
10			6	18		6					21		%F=6
	15		12	12		7			SP-SM	Dark gray fine to medium sand with silt, micaceous (medium dense, wet) (fill)	33		%F=10
5			2	6		8			GP	Black/gray/white gravel with fine to coarse sand (loose, wet) (fill)	39		
	20		8	P		9			SM	Gray/black silty fine sand with organic matter, micaceous (loose, moist)	51		%F=41 Shelby tube refusal on wood
0			4	5		10							
	25		14	4		11			ML	Brown/gray silt with fine sand and organic matter (soft, wet)	61		
-5			0	P		12							
	30		16	P		13			OH	Brown organic silt with trace organic matter and peat (soft, wet)	70	56	Consol, AL LL=86, PI=26
-10			18	4		14					135		
	35												

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

LOG OF BORING B-2

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-3
Sheet 1 of 3

V6_GTBORING P:\6\6191002\04\FINAL\619100204.GPJ GEIV6_1.GDT 5/7/07

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	35		18	P		15							
-15									ML	Gray silt with organic matter (soft, wet)			
	40		18	3		16					49		
-20									SM	Gray silty fine sand with occasional organic matter (medium dense, moist)			
	45		15	20		17					31		%F=19
-25									SP-SM	Gray fine to medium sand with silt (medium dense, wet)			
	50		18	21		18					25		%F=10
-30													
	55		8	26		19							
-35									SP-SM	Gray fine to medium sand with silt (medium dense, moist)			
	60		12	30		20					23		%F=9
-40									MH	Gray elastic silt with organic matter (medium stiff, moist)			
	65		18	6		21					50		
-45			16	P		22					55	70	Consol, AL LL=73, PI=27
	70												
-50									ML	Gray silt with trace fine sand (medium stiff, moist)			
	75		18	9		23					33		
-55			20	P		24							

V6_GTBORING P:\6\6191002\04\FINAL\61910020 4.GPJ GEIV6_1.GDT 5/7/07

LOG OF BORING B-2 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-3
Sheet 2 of 3

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	80										
	-60						OL	Gray organic silt with organic matter (medium stiff, moist)			
	85	18	8	25					68		
	-65										
	90	18	P	26							
	-70										
							SP-SM	Gray fine to medium sand with silt (dense to very dense, moist)			
	95	10	50	27					22		
	-75										
							SP	Black/gray/white fine to medium sand with trace gravel (very dense, moist)			
	100	12	66	28							
	-80										
							SP	Gray fine to coarse sand with occasional gravel (dense, moist)			
	105	12	41	29							
	-85										
							SP	Gray fine to coarse sand with gravel, interbedded silty sand layers and occasional organic matter (dense, moist)			
	110	8	42	30							
	-90										Hard drilling
							SP-SM	Gray fine to medium sand with silt and gravel, logs (dense, moist)			
	115	10	35	31							6 inches wood core in sample
	-95							Wood debris at 116 feet			
							SP-SM	Black/gray/white fine to medium sand with silt (very dense, wet)			
	120	10	55	32							
	-100										

V6_GTBORING P:\6\6191002\04\FINAL\619100204.GPJ GEIV6_1.GDT 5/7/07

LOG OF BORING B-2 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-3
Sheet 3 of 3

Date(s) Drilled	01/23/07	Logged By	SH	Checked By	RCM
Drilling Contractor	Boart Longyear/Holt Drilling	Drilling Method	Mud Rotary	Sampling Methods	SPT/DM/Shelby Tube
Auger Data	4-7/8" Tricone	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	CME-850 Track Rig
Total Depth (ft)	99.5	Surface Elevation (ft)	18	Groundwater Elevation (ft)	15
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	Interval	Recovered (in)	Blows/foot	Sub-Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0		6	11	1			SM	Brown silty fine to medium sand with occasional gravel and roots (medium dense, moist) (fill)			
								SP	Gray fine to medium sand (medium dense, wet) (fill)			
15			11	28	2					13		
	5		10	20	3				Occasional gravel			
10			18	45	4				Becomes dense	51	69	
	10		14	10	5			SP-SM	Brown fine to medium sand with silt and occasional gravel (medium dense, wet) (fill)	29		Easy drilling %F=5
5			15	3	6a			SM	Black fine to medium sand with organic matter (very loose, wet)			
					6b			SP	Gray fine to medium sand with trace silt (very loose, wet)	26		%F=4
	15		0	P	7			SP-SM	Gray fine to medium sand with occasional gravel and trace silt (very loose, wet)			
0			16	2	8a					29		%F=5
					8b			ML	Brown silt with fine sand and occasional gravel, organic matter (very soft, wet)			
	20		24	P	9			OL	Brown organic silt with interbedded silt layers (soft, wet)	39	50	Consol, AL LL=36, PI=11
-5			18	4	10				Occasional sand and gravel, organic matter	58		
	25		18	3	11			ML	Brown silt with organic matter (soft, moist)	62		
-10			18	6	12				Occasional gravel	35	84	
	30		16	3	13			ML	Gray fine sandy silt with trace organic matter (soft, moist)	37		
-15			18	4	14					35		%F=60
	35											

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

LOG OF BORING B-3

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-4
Sheet 1 of 3

V6_GTBORING P:\6\6191002\04\FINAL\61910020 4.GPJ GEIV6_1.GDT 5/7/07

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	35										
-20		12	9	15					31		
	40						SM	Gray silty fine sand (medium dense, moist)			
-25		15	10	16					32		%F=32
	45						SP-SM	Gray fine sand with silt (medium dense, wet)			
-30			23	17					24		%F=12
	50						SP	Gray fine sand with trace silt (dense, wet)			
-35		14	40	18					22		
	55						SP-SM	Gray fine sand with silt (dense, wet)			
-40		12	36	19					21		
	60										
-45		12	35	20					19		Artesian conditions
	65										
-50		10	35	21					20		%F=8
	70						ML	Brown silt with trace fine sand (medium stiff, moist)			
-55		18	8	22				Log at 73 feet	35		
	75										
-60		18	P	23							

V6_GTBORING P:\6\6191002\04\FINAL\61910020 4.GPJ GEIV6_1.GDT 5/7/07

LOG OF BORING B-3 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-4
Sheet 2 of 3

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	80										
							ML	Gray silt with silty fine sand interbeds, slight plasticity (very stiff, wet)			
-65		18	24	24					28		
	85										
							SM	Gray silty fine sand (dense, moist)			
-70		12	47	25					20		
	90										
-75		12	32	26				Occasional organic matter, wet	26		
	95										
							SM	Gray silty fine sand (loose, wet)			
-80			8	27					28		
	100										
-85											
	105										
-90											
	110										
-95											
	115										
-100											
	120										

V6_GTBORING P:\6\6191002\04\FINAL\619100204.GPJ GEIV6_1.GDT 5/7/07

LOG OF BORING B-3 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-4
Sheet 3 of 3

Date(s) Drilled	01/29/07, 01/30/07	Logged By	SH	Checked By	RCM
Drilling Contractor	Boart Longyear/Holt Drilling	Drilling Method	Mud Rotary	Sampling Methods	SPT/Shelby Tube
Auger Data	3-7/8" Tricone	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	Mobile B-59 Truck Rig
Total Depth (ft)	101.5	Surface Elevation (ft)	22	Groundwater Elevation (ft)	15
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0							SP	Brown fine to medium sand (medium dense, moist) (fill)			
20			18	18	1							
								SP	Brown fine to medium sand with occasional gravel and trace silt (medium dense, moist) (fill)			
5			12	16	2							
15			13	8	3	▼		SP	Brown fine to medium sand with trace silt (loose, wet) (fill)	20		%F=4
	10		9	14	4			SP	Black fine to medium sand with occasional gravel, organic matter, micaceous (medium dense, wet) (fill)	40		
10			10	17	5							
								ML	Log, wood fragments Brown silt with organic matter, grass, wood pieces (soft, wet)			
	15		7	3	6					102		
5												
			12	P	7							
	20		4	7	8			SM	Gray silty fine sand with occasional gravel (loose, wet)	18		%F=27
0			13	P	9			ML	Gray fine sandy silt (soft, wet)	35		
	25		24	P	10							
-5									Increasing sand content			
			13	3	11					32		%F=52
	30		10	6	12				Becomes medium stiff with decreasing sand content	36		
-10			10	15	13			SM	Gray silty sand with occasional gravel (medium dense, wet)	27		%F=31
	35											

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

LOG OF BORING B-4

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-5
Sheet 1 of 3

V6_GTBORING P:\6\6191002\04\FINAL\61910020 4.GPJ GEIV6_1.GDT 5/7/07

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	35		8	19	14			SP	Black/gray/white fine to coarse sand with gravel, trace silt (medium dense, wet)			
-15												
								SP	Black/gray/white medium to coarse sand with gravel (medium dense, wet)			
	40		8	19	15							
-20												
	45		13	17	16				Sand becomes coarser			
-25												
								SP-SM	Gray fine to coarse sand with silt and occasional gravel (medium dense, wet)			
	50		12	23	17					12		%F=7
-30												
	55		12	31	18				2-inch thick gravel layer			
-35												
								SP	Gray fine to medium sand with occasional gravel and trace silt (dense, wet)			
	60		11	30	19					17		
-40												
	65		12	36	20					22		
-45												
	70		14	26	21							
								ML	Gray fine sandy silt with organic matter (very stiff, wet)			
-50								SM	Gray silty fine sand (medium dense, wet)			
	75		15	23	22				Light brown volcanic ash layer	49		
-55												

LOG OF BORING B-4 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-5
Sheet 2 of 3

Elevation feet	Depth feet	Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	80		P	23							
	-60						SP	Gray fine to medium sand with trace silt (dense, moist)			
	85	14	49	24							
	-65						SP-SM	Gray fine sand with silt (dense, moist)			
	90	14	49	25							
	-70						SP	Gray fine to medium sand with occasional gravel and trace silt (very dense, moist)			
	95	13	52	26							
	-75						SP/GP	Gray sand and gravel (very dense, moist)			
	100	2	50/6"	27							
	-80										
	105										
	-85										
	110										
	-90										
	115										
	-95										
	120										
	-100										

LOG OF BORING B-4 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-5
Sheet 3 of 3

V6_GTBORING P:\6\6191002\04\FINAL\619100204.GPJ GEIV6_1.GDT 5/7/07

Date(s) Drilled	01/25/07, 01/26/07	Logged By	SH	Checked By	RCM
Drilling Contractor	Boart Longyear/Holt Drilling	Drilling Method	Mud Rotary	Sampling Methods	SPT/DM/Shelby Tube
Auger Data	3-7/8" Tricone	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	Mobile B-59 Truck Rig
Total Depth (ft)	131.5	Surface Elevation (ft)	19	Groundwater Elevation (ft)	14
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	0		13	8		1			SP	Brown/gray fine to medium sand with trace silt (loose to medium dense, moist) (fill)			
			13	12		2							
15													
	5		12	11		3	▼			Becomes wet	22		SA, %F=3
			11	17		4				Increasing gravel			
10													
	10		11	5		5					37		
			0	3		6			OL	Dark brown/black organic silt with wood fragments and organic matter, occasional gravel (soft, moist)			
5													
	15		18	11		7			SM	Gray silty fine sand, micaceous (medium dense, wet)	29		%F=17
			4	23		8			ML	Gray fine sandy silt with organic matter and wood debris (medium stiff, wet) Log, wood debris			Blow count affected by log
0													
	20		6	11		9				Log, wood debris	98		
			0	6		10							
-5													
	25		18	4		11					46		
			10	7		12							Consol, %F
-10													
	30		18	4		13					39		
			18	6		14				Increasing sand content	36		
-15													
	35												

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

LOG OF BORING B-5

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-6
Sheet 1 of 4

V6_GTBORING P:\6\6191002\04\FINAL\61910020 4.GPJ GEIV6_1.GDT 5/7/07

Elevation feet	Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	35		10	31	15			SM	Gray silty sand grading to fine to medium sand (medium dense, wet)			
								SP-SM	Gray fine to medium sand with silt and occasional gravel (medium dense, wet)			
-20												
	40		11	15	16					22		%F=7
-25												
	45		12	16	17				No gravel, trace silt	25		
-30												
	50		14	17	18					25		%F=8
-35												
	55		11	16	19					22		
-40												
	60		12	19	20				Occasional gravel	21		%F=7
-45												
	65		13	23	21				Trace silt	22		
-50												
	70		11	23	22				Occasional gravel			
-55												
	75		2	-	23				Becomes dense			Blow count not representative

V6_GTBORING P:\6\6191002\04\FINAL\6191002O4.GPJ GEIV6_1.GDT 5/7/07

LOG OF BORING B-5 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-6
Sheet 2 of 4

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
-60											
	80	12	43	24							
-65											
	85	12	40	25				Trace silt			
-70											
	90	15	72	26							
-75							ML	Gray silt to silty clay, plastic (stiff, wet)			
	95	18	20	27					32		
							SM	Gray silty fine sand (medium dense, wet)			
-80											
	100	6	15	28					27		
-85											
	105			29							
							ML	Gray fine sandy silt (hard, wet)			
-90											
	110	17	37	30					29		
							SP	Gray fine to medium sand, trace silt (dense, moist)			
-95											
	115	13	34	31							
-100							SP	Gray fine to medium sand with occasional gravel (very dense, moist)			
	120	13	75/9"	32							
							SP/GP	Gray medium to coarse sand and gravel (very dense, moist)			

LOG OF BORING B-5 (continued)

GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-6
Sheet 3 of 4

Elevation feet	Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
-105	125	7	98/9"	33			SP	Gray medium to coarse sand with gravel (very dense, moist)			
-110	130	7	91	34							
-115	135										
-120	140										
-125	145										
-130	150										
-135	155										
-140	160										
-145	165										

LOG OF BORING B-5 (continued)

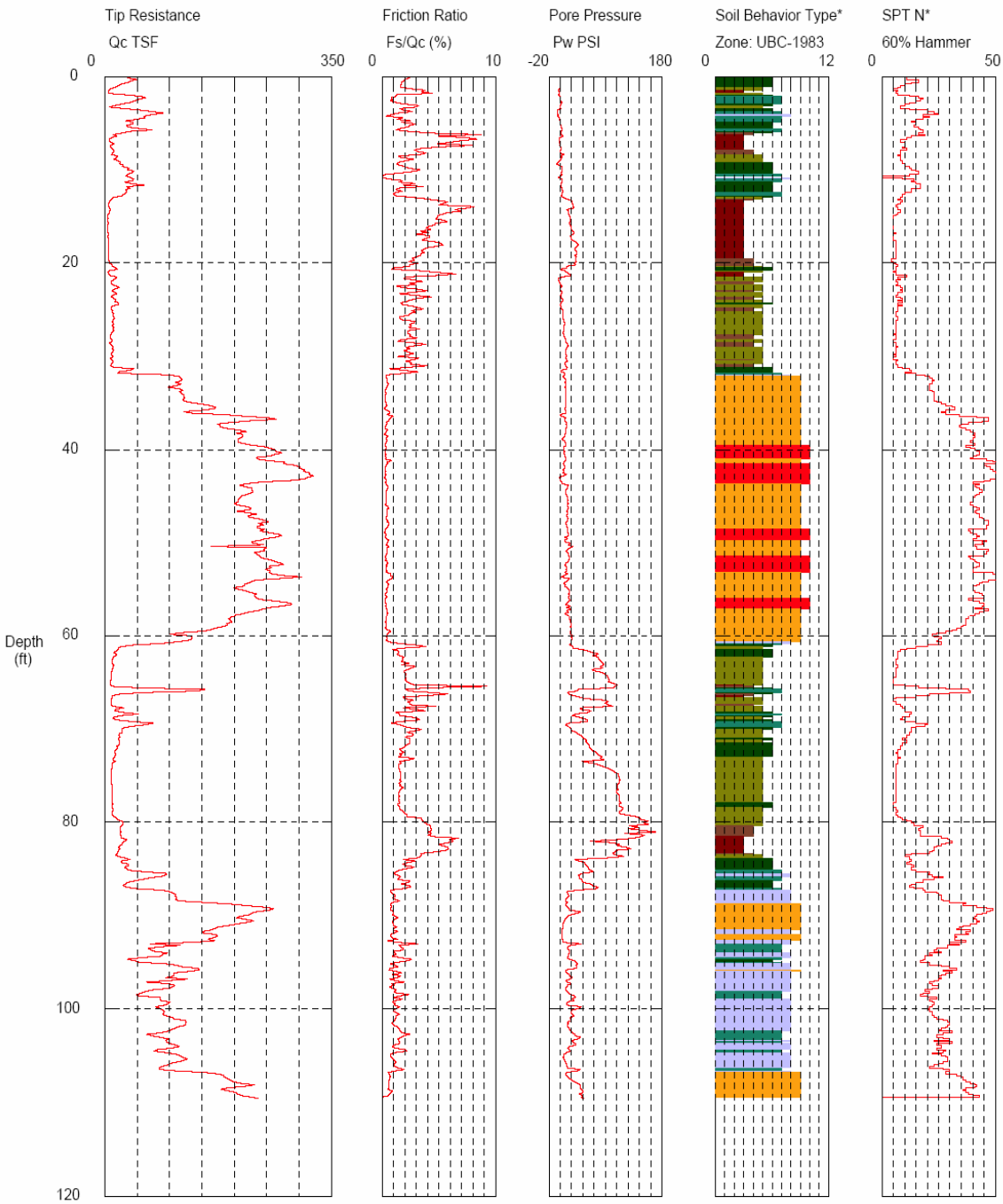
GEOENGINEERS

Project: Everett Riverfront Redevelopment Project
Project Location: Everett, Washington
Project Number: 6191-002-04

Figure A-6
Sheet 4 of 4

Operator: Brown
Sounding: CPT-01
Cone Used: DSG1015

CPT Date/Time: 1/23/2007 2:23:47 PM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 109.58 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 (*)

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

Northwest Cone Exploration

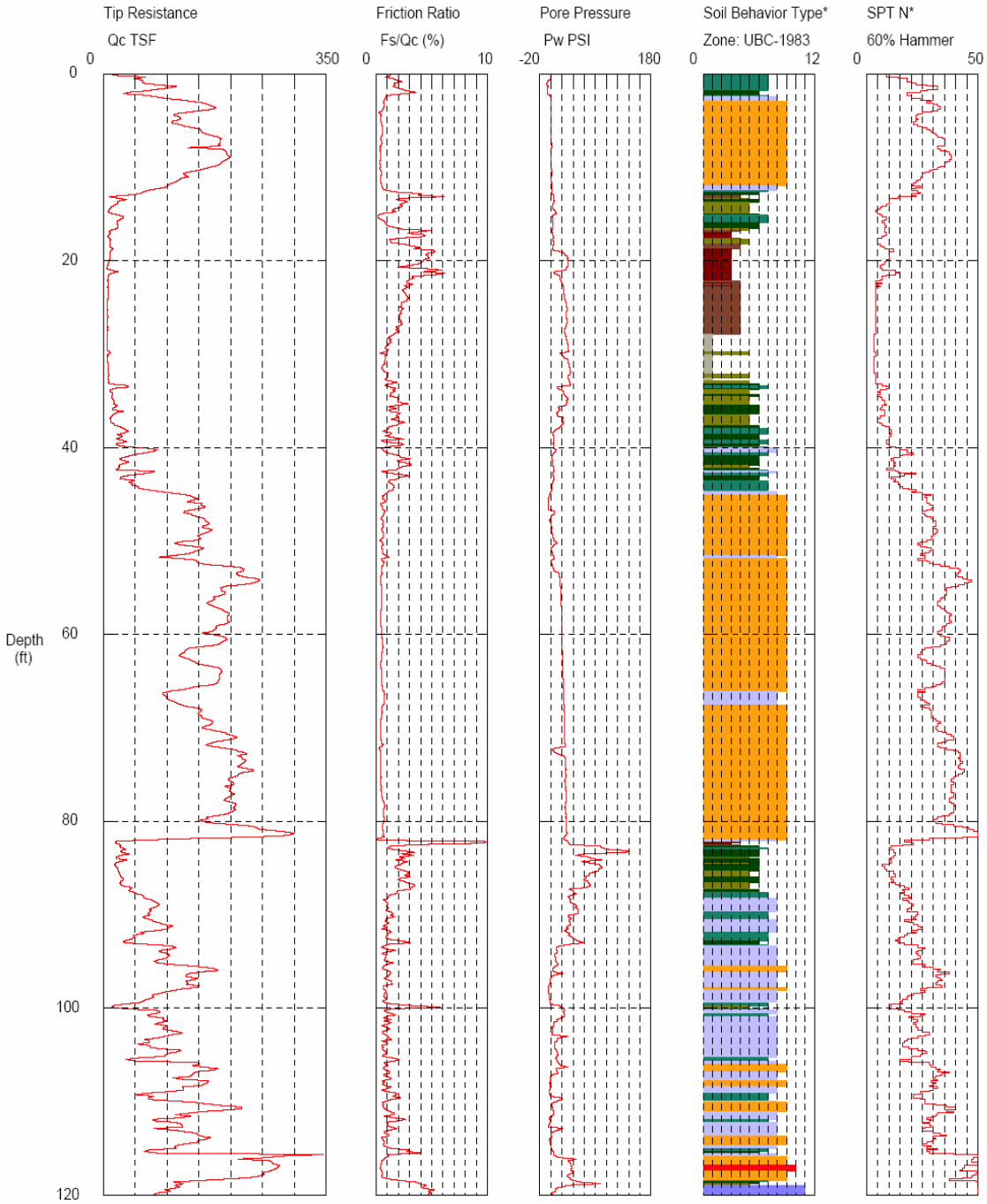
GeoEngineers
Earth Science + Technology

Cone Penetration Test, CPT-1

Figure A-7

Operator: Brown
Sounding: CPT-02
Cone Used: DSA0902

CPT Date/Time: 1/29/2007 1:30:19 PM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 139.27 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 (*)

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

Northwest Cone Exploration

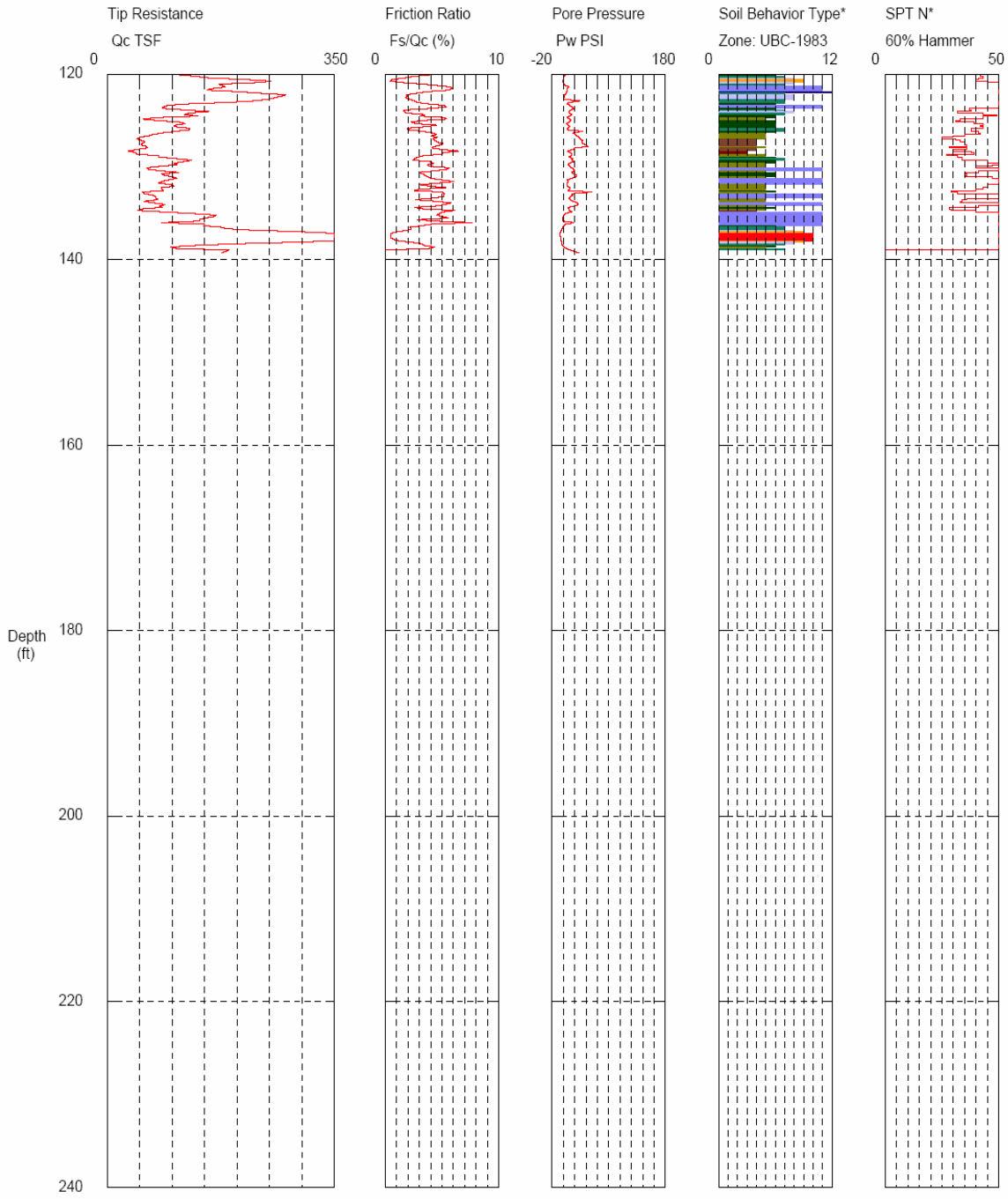
GeoEngineers
Earth Science + Technology

Cone Penetration Test, CPT-2

Figure A-8 (1 of 2)

Operator: Brown
Sounding: CPT-02
Cone Used: DSA0902

CPT Date/Time: 1/29/2007 1:30:19 PM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 139.27 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 (*)

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

Northwest Cone Exploration

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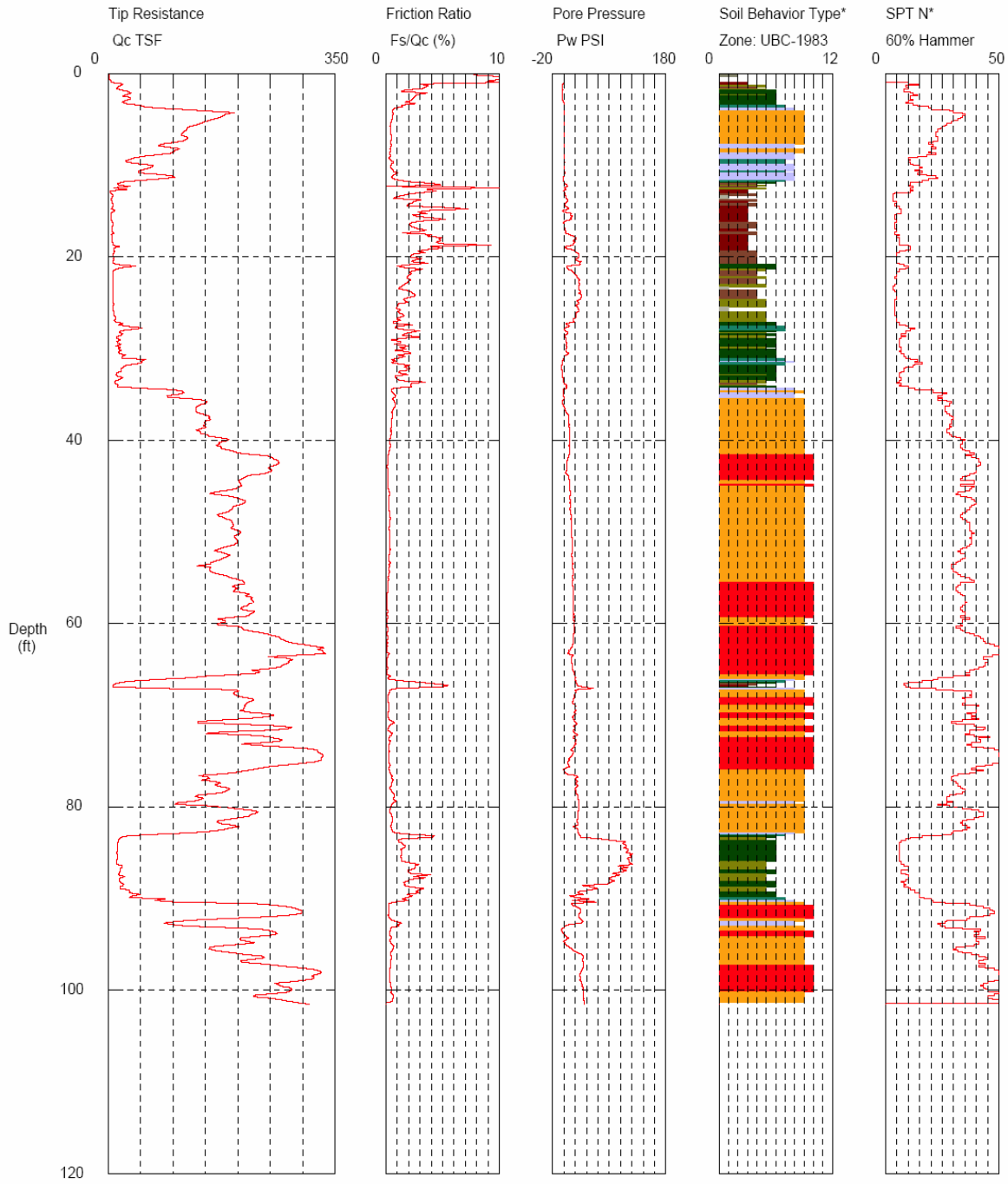
Earth Science + Technology

Cone Penetration Test, CPT-2

Figure A-8 (2 of 2)

Operator: Brown
Sounding: CPT-03
Cone Used: DSA0902

CPT Date/Time: 1/29/2007 11:43:19 AM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 101.54 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 (*)

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

Northwest Cone Exploration

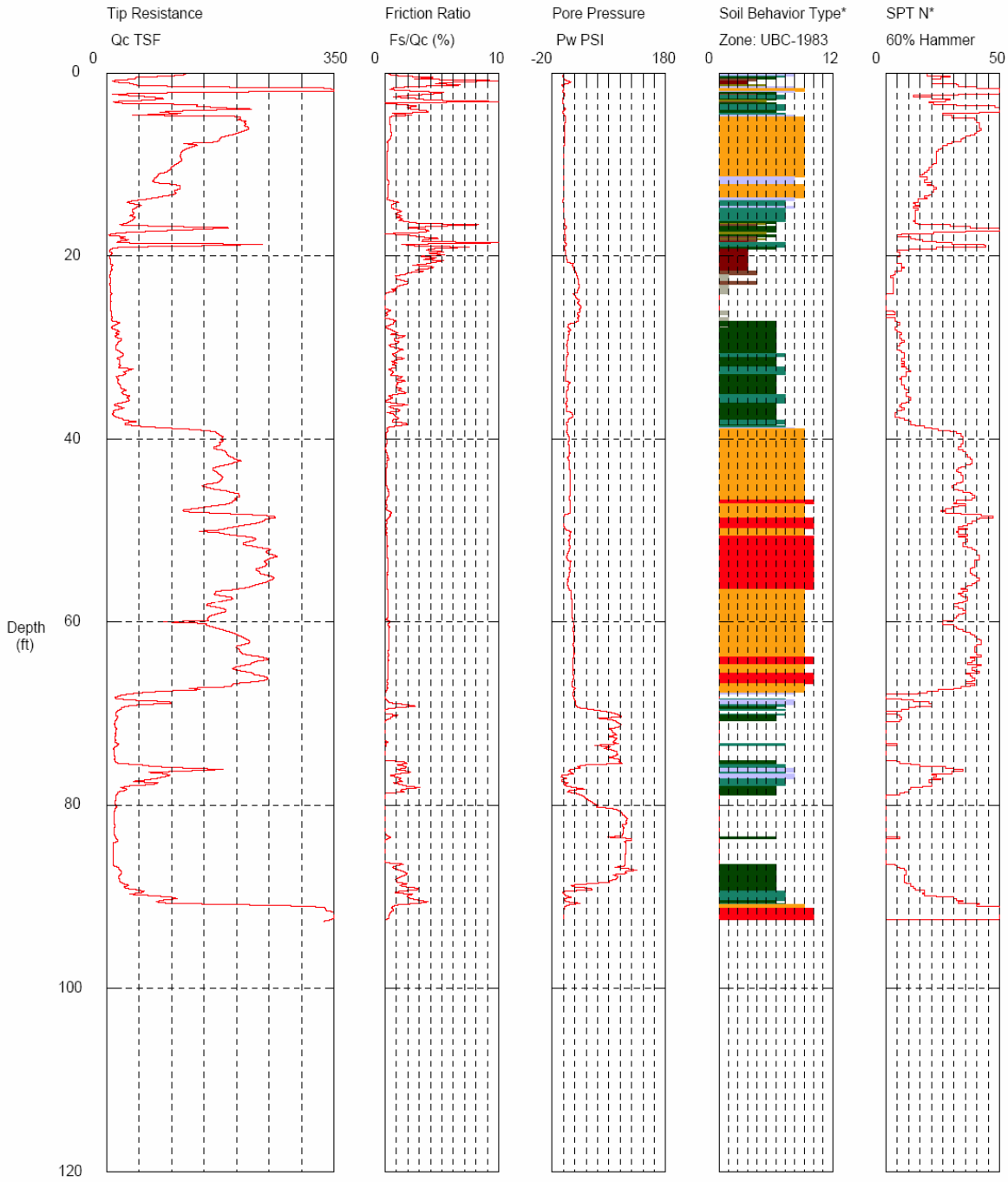
GeoEngineers
Earth Science + Technology

Cone Penetration Test, CPT-3

Figure A-9

Operator: Brown
Sounding: CPT-04
Cone Used: DSG1015

CPT Date/Time: 1/23/2007 11:54:53 AM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 92.68 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 (*)

Friction sleeve jammed in wood debris at about 18 feet.

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

Northwest Cone Exploration

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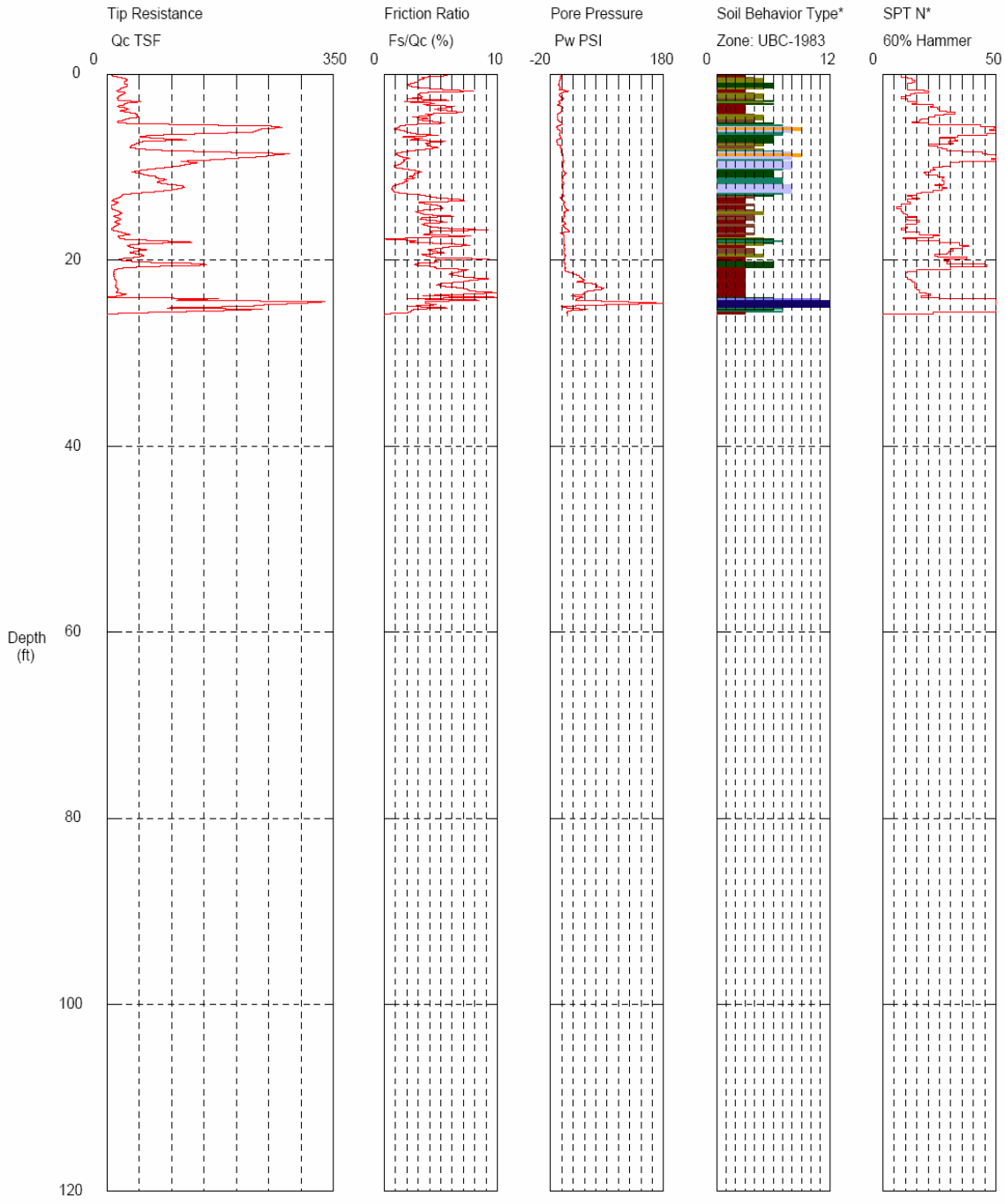
Earth Science + Technology

Cone Penetration Test, CPT-4

Figure A-10

Operator: Brown
Sounding: CPT-05A
Cone Used: DSG1015

CPT Date/Time: 1/23/2007 10:37:56 AM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 25.92 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
Refused in log?
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 (*)

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

Northwest Cone Exploration

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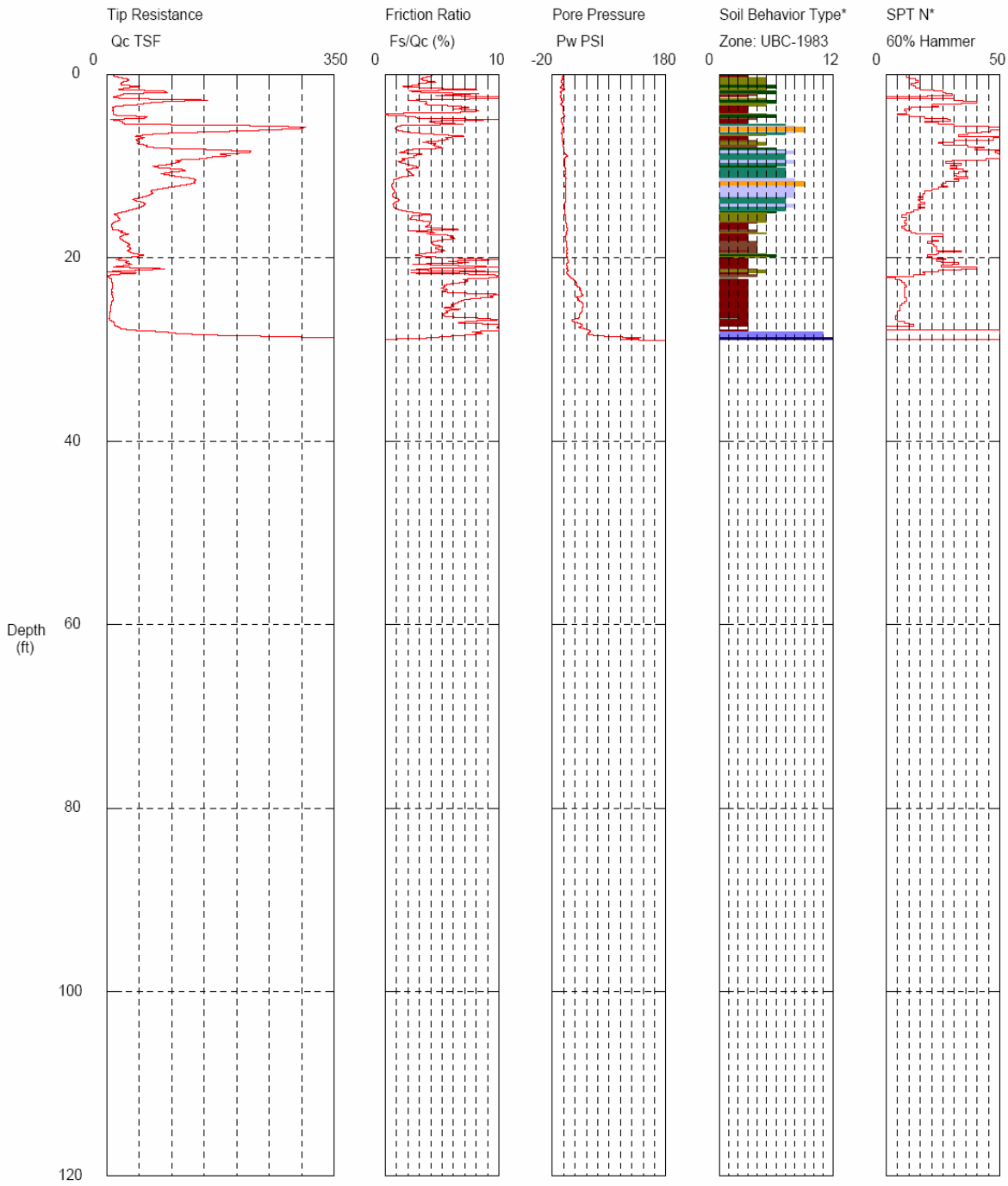
Earth Science + Technology

Cone Penetration Test, CPT-5A

Figure A-11

Operator: Brown
Sounding: CPT-05B
Cone Used: DSG1015

CPT Date/Time: 1/23/2007 11:10:00 AM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 29.04 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
Refused in log?
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 (*)

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

Northwest Cone Exploration

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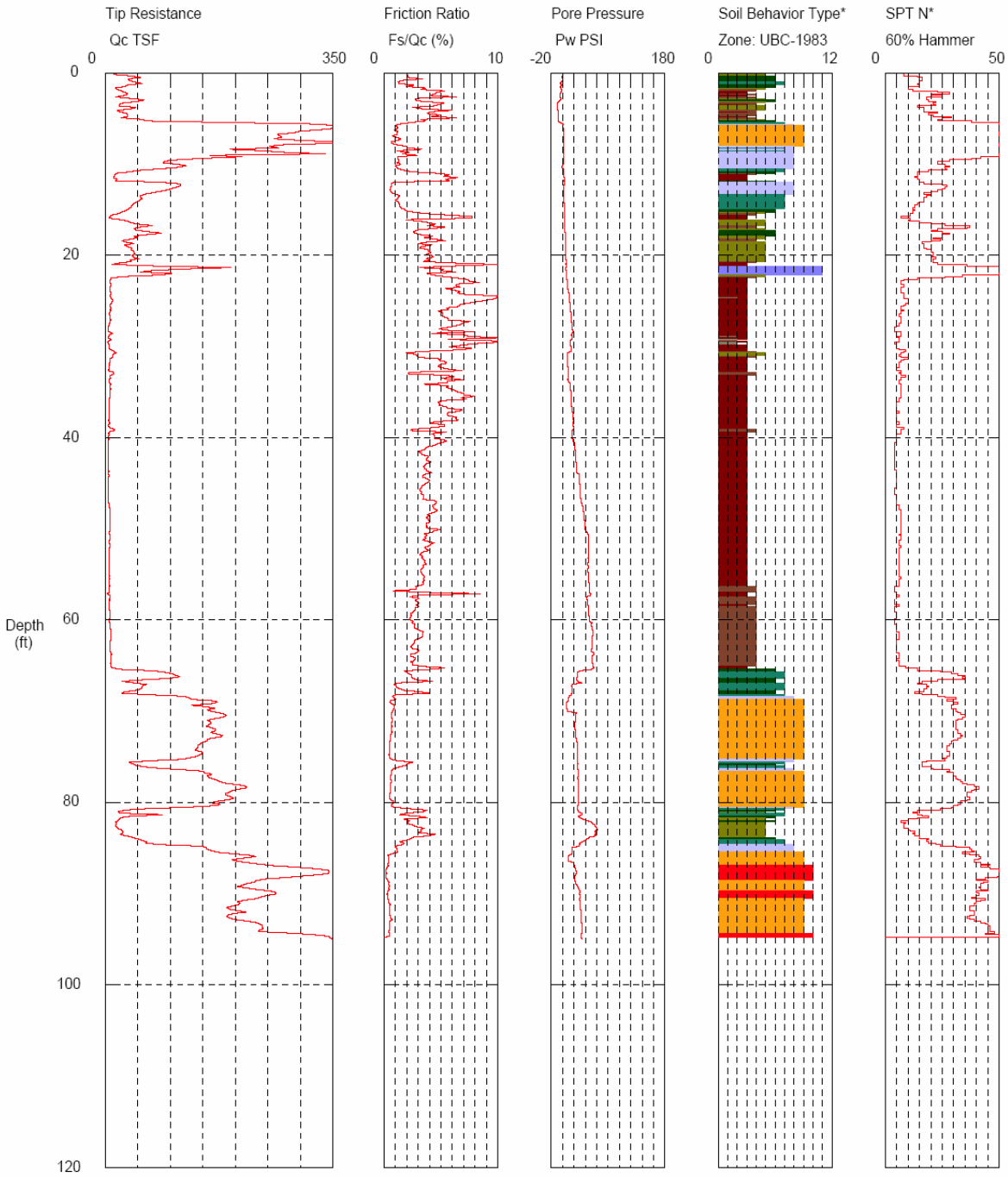
Cone Penetration Test, CPT-5B

Figure A-12

Operator: Brown
Sounding: CPT-05C
Cone Used: DSA0902

CPT Date/Time: 1/30/2007 2:40:09 PM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 94.98 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 (*)

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

Northwest Cone Exploration

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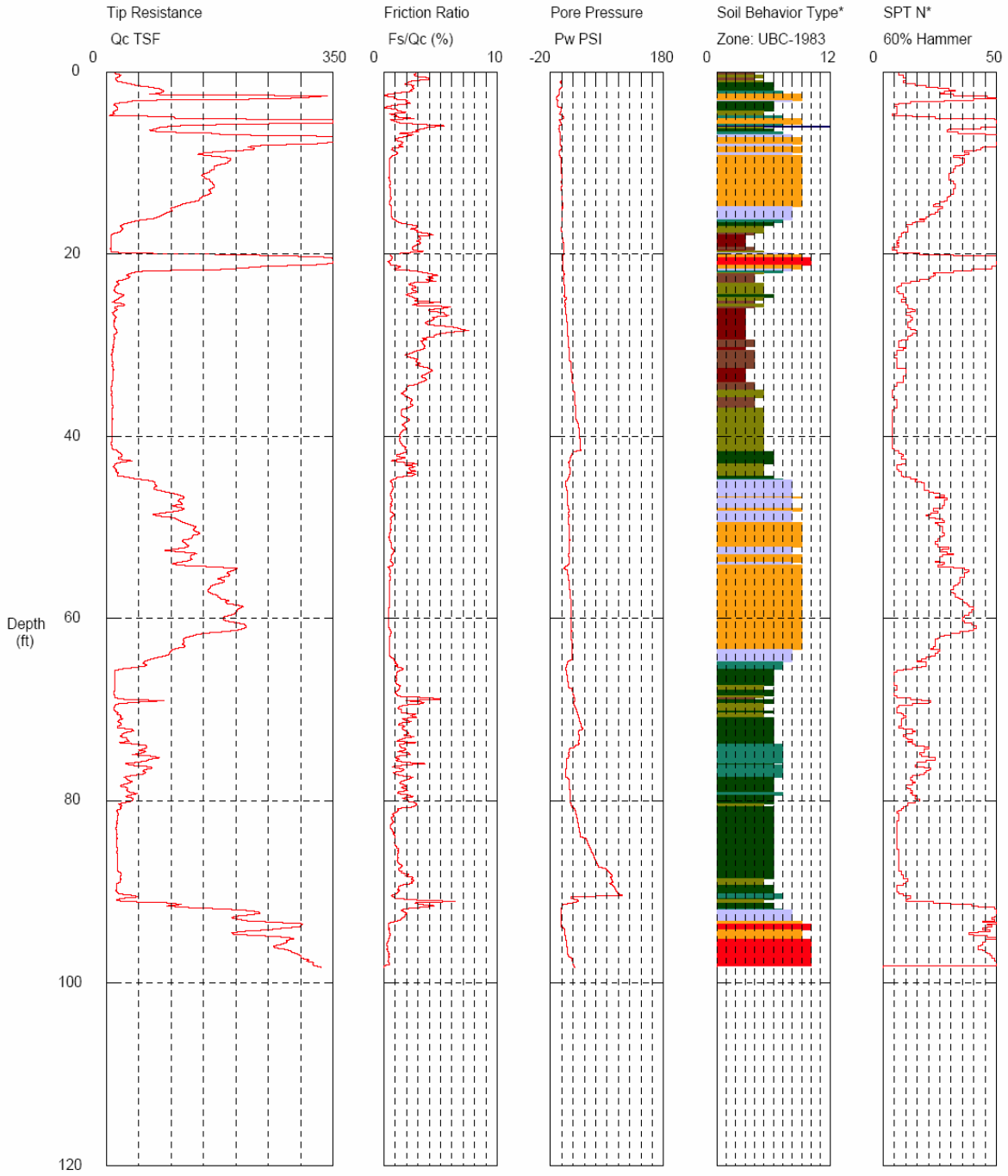
Cone Penetration Test, CPT-5C

Figure A-13

Operator: Brown
Sounding: CPT-06
Cone Used: DSA0902

CPT Date/Time: 1/30/2007 10:24:05 AM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 98.26 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 (*)

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

Northwest Cone Exploration

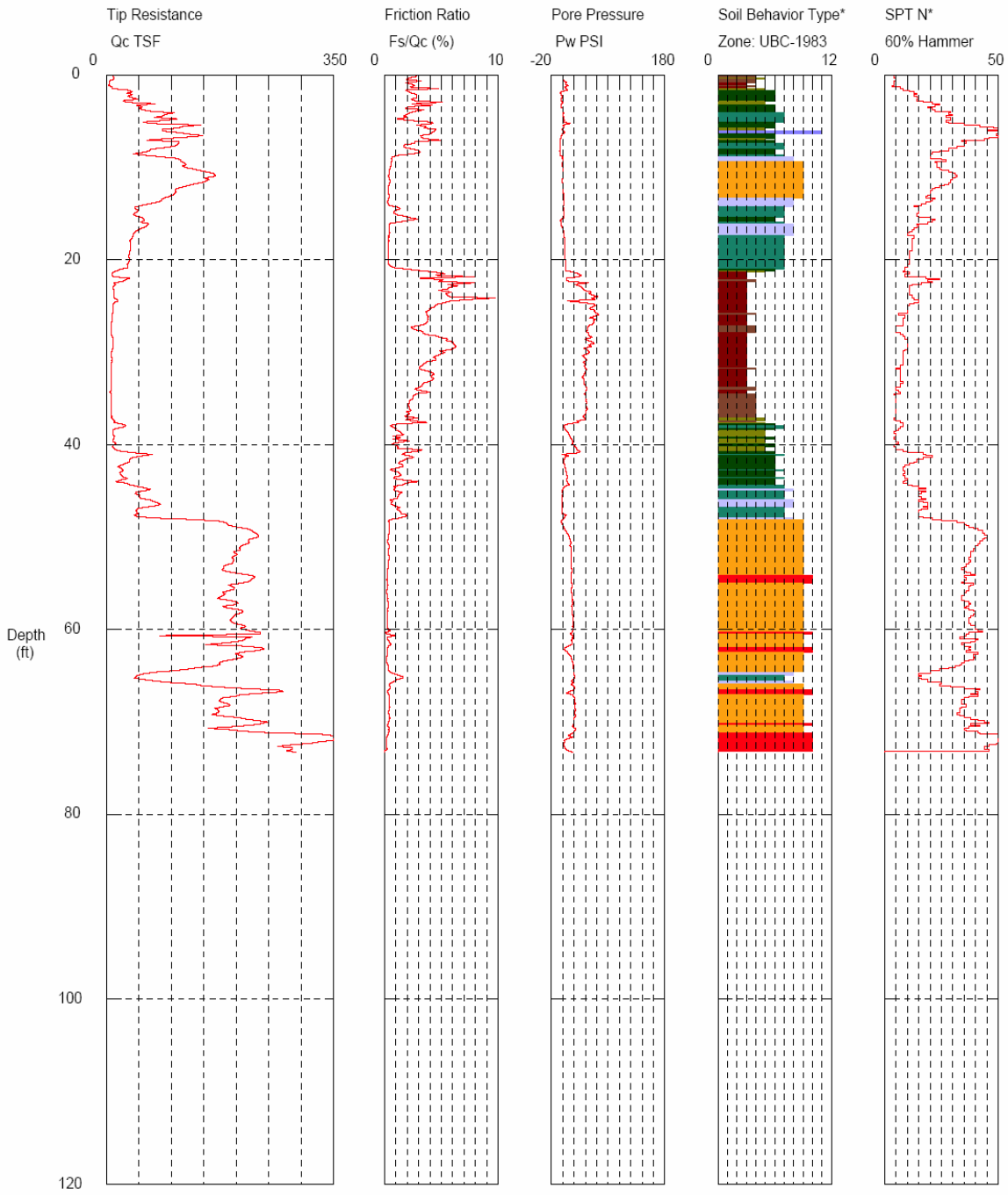
GeoEngineers
Earth Science + Technology

Cone Penetration Test, CPT-6

Figure A-14

Operator: Brown
Sounding: CPT-07
Cone Used: DSG1015

CPT Date/Time: 1/22/2007 11:05:37 AM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 73.33 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 (*)

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

Northwest Cone Exploration

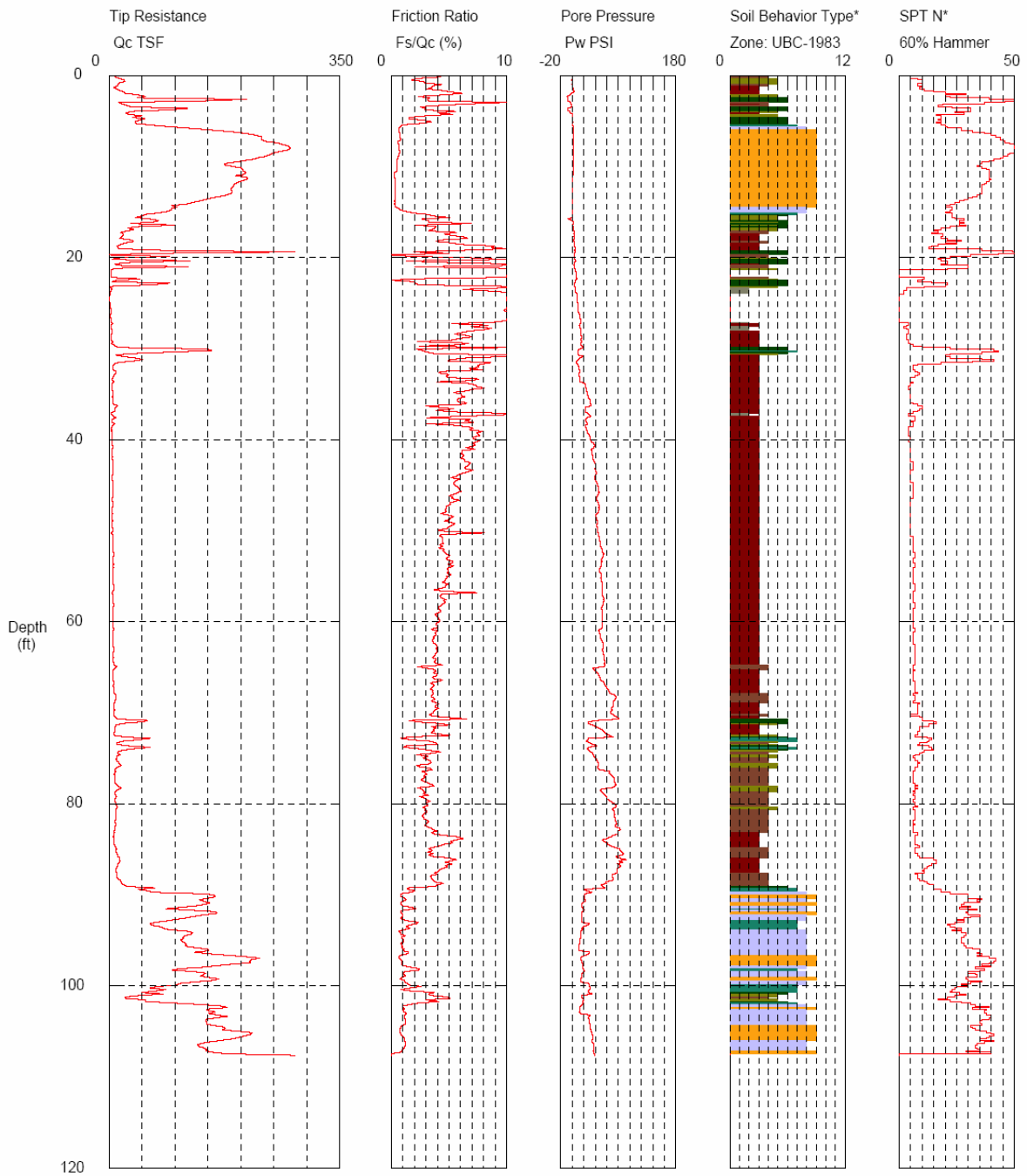
GeoEngineers
Earth Science + Technology

Cone Penetration Test, CPT-7

Figure A-15

Operator: Brown
Sounding: CPT-08
Cone Used: DSG1015

CPT Date/Time: 1/22/2007 12:53:32 PM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 107.61 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 (*)

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

Northwest Cone Exploration

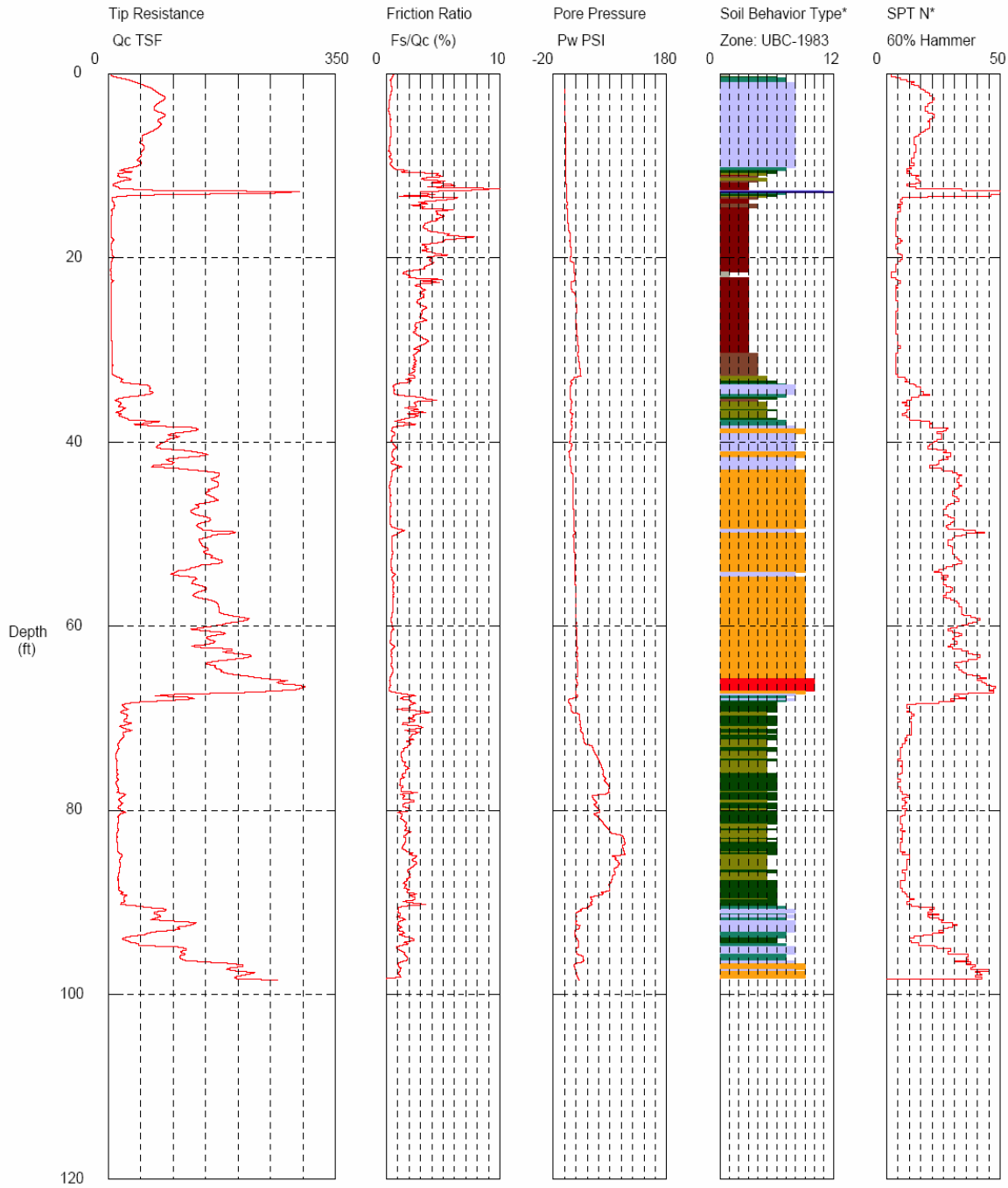
GeoEngineers
Earth Science + Technology

Cone Penetration Test, CPT-8

Figure A-16

Operator: Brown
Sounding: CPT-09
Cone Used: DSA0902

CPT Date/Time: 1/30/2007 1:02:45 PM
Location: Everett Riverfront Redevelopment Project - Simpson Site
Job Number: 6191-002-04



Maximum Depth = 98.43 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 (*)

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

Northwest Cone Exploration

GeoEngineers
Earth Science + Technology

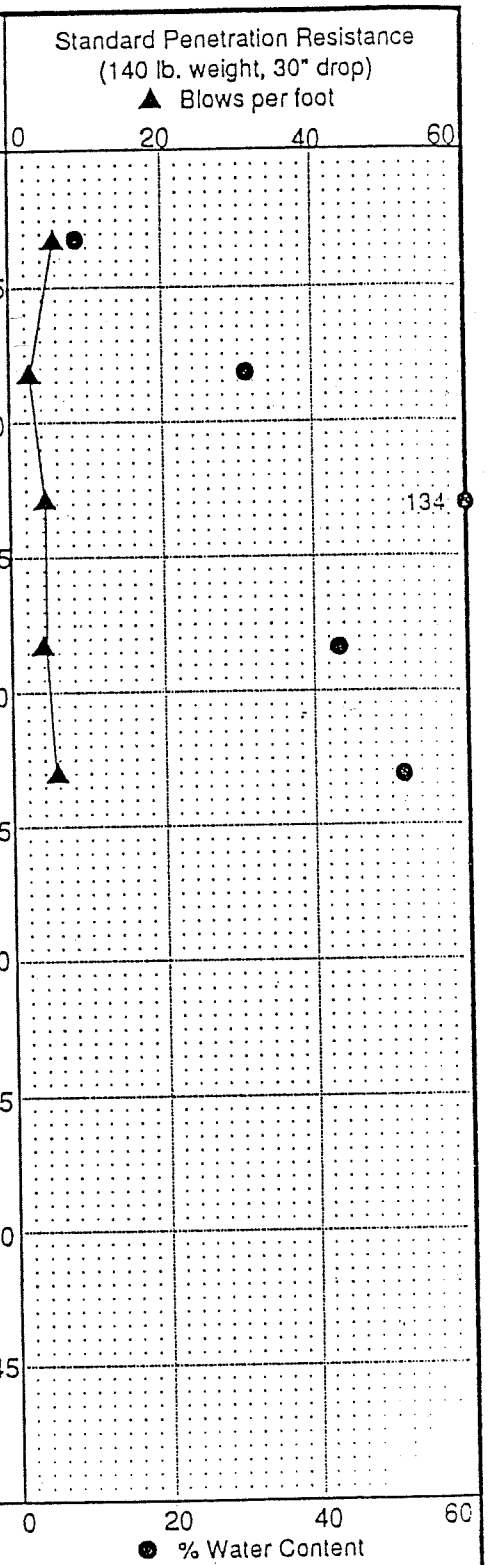
Cone Penetration Test, CPT-9

Figure A-17

APPENDIX B
PREVIOUS EXPLORATIONS

- Shannon & Wilson, 1990, borings B-4, B-27 and B-28
- Pacific Groundwater Group, date unknown, boring SB-1
- The Floyd & Snider Team, 1999, borings B-43, B-44 and B-54
- HWA Geosciences, Inc., 2003, borings BF-1 and BF-2

SOIL DESCRIPTION Surface Elevation: Approx. 110 feet	Depth, Ft.	Samples	Ground Water	Depth, Ft.
Very loose to loose, brown, slightly silty, fine to medium SAND; moist (FILL)		1, 2	During drilling	
Soft, brown woody, fibrous PEAT and organic SILT; moist	13	3		
Loose, gray, SILT and fine sandy SILT; moist, numerous wood fragments	17.5	4, 5		
BOTTOM OF BORING COMPLETED 08-21-89	24			



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

South End Interceptor Extension
Everett, Washington

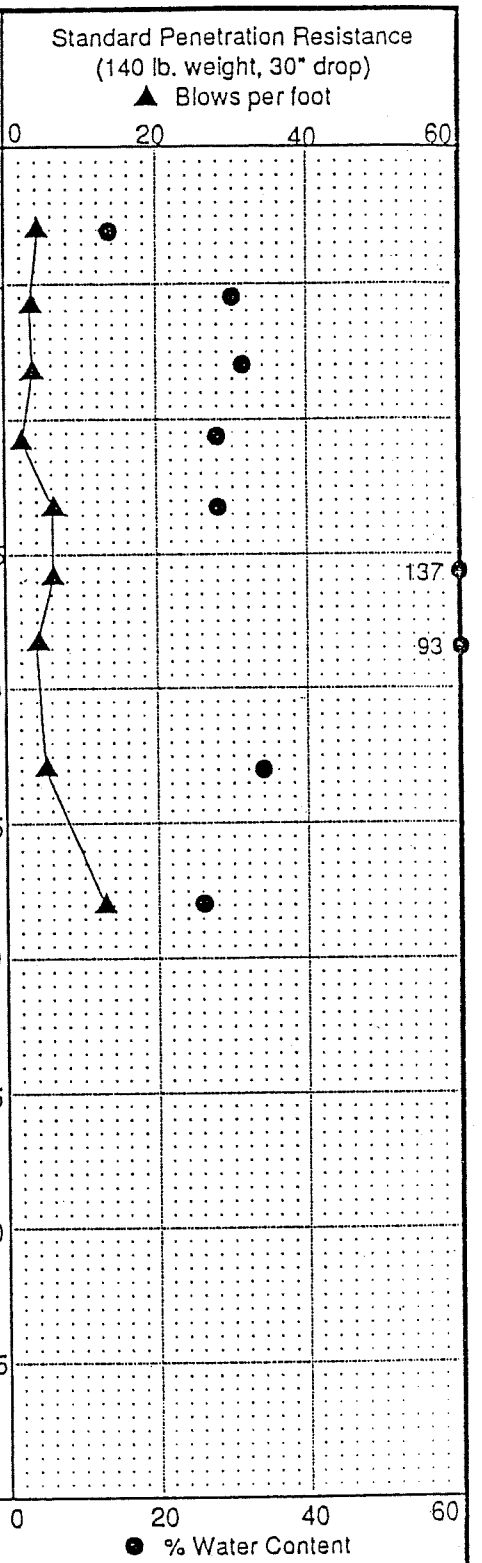
LOG OF BORING B-4

March 1990 W-5002-02

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-4

SOIL DESCRIPTION Surface Elevation: Approx. 115 feet	Depth, Ft.	Samples	Ground Water	Depth, Ft.
Medium to very dense, brown to gray, slightly silty fine to medium SAND; moist (FILL)		1, 2, 3, 4, 5	During drilling	5, 10, 15
Medium stiff, brown to gray, PEAT and peaty SILT; moist, numerous wood fragments, occasional logs	13.5	6, 7		20
Loose to medium dense, gray, fine sandy SILT and silty fine SAND; moist	18.5	8, 9		25
BOTTOM OF BORING COMPLETED 10-24-89	29			30, 35, 40, 45



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- • Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

South End Interceptor Extension
Everett, Washington

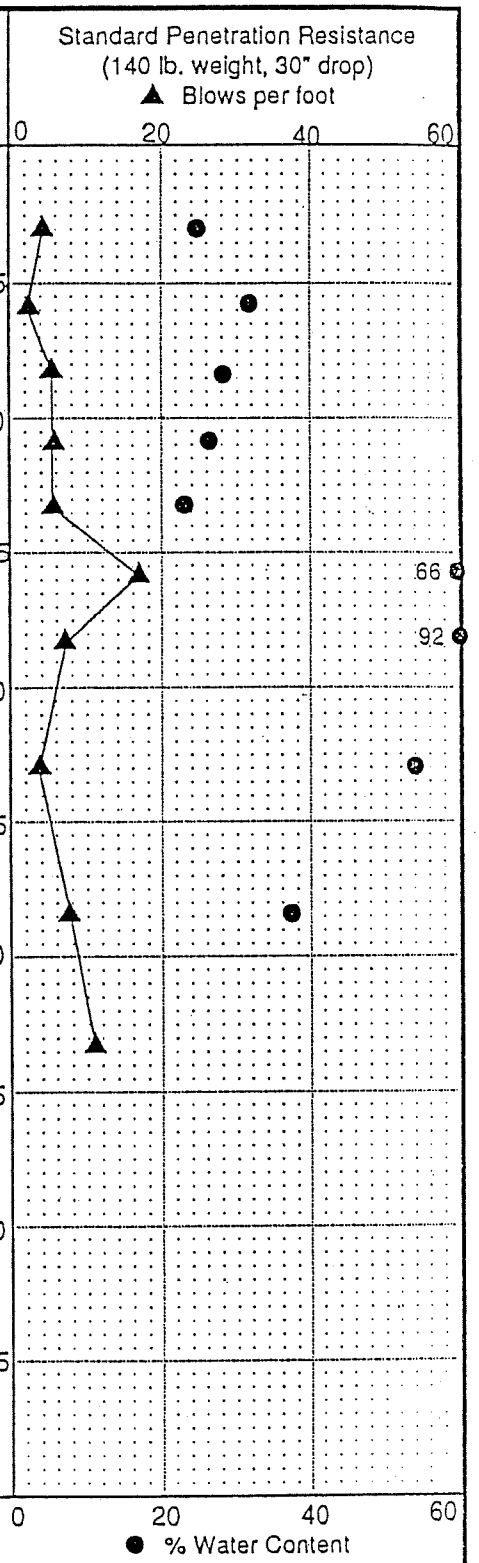
LOG OF BORING B-27

March 1990 W-5002-02

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-20

SOIL DESCRIPTION Surface Elevation: Approx. 115 feet	Depth, Ft.	Samples	Ground Water	Depth, Ft.
Very loose to loose, brown to gray, slightly silty fine to medium SAND; moist (FILL)		1, 2, 3, 4, 5	Estimated During Drilling	5, 10
Soft to loose, brown to gray, peaty SILTwith lenses of PEAT; moist	13.5	6, 7, 8		15, 20
Loose to medium dense, gray, fine sandy SILT and silty fine SAND; moist	25	9, 10*		25, 30
BOTTOM OF BORING COMPLETED 10-24-89	34			35, 40, 45



LEGEND

- 2" O.D. split spoon sample
- 3" O.D. thin-wall sample
- * Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:

- Liquid limit
- Natural water content
- Plastic limit

The stratification lines represent the approx. boundaries between soil types, and the transition may be gradual.

South End Interceptor Extension
Everett, Washington

LOG OF BORING B-28

March 1990 W-5002-02

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-21

EVERETT LANDFILL

S8--1

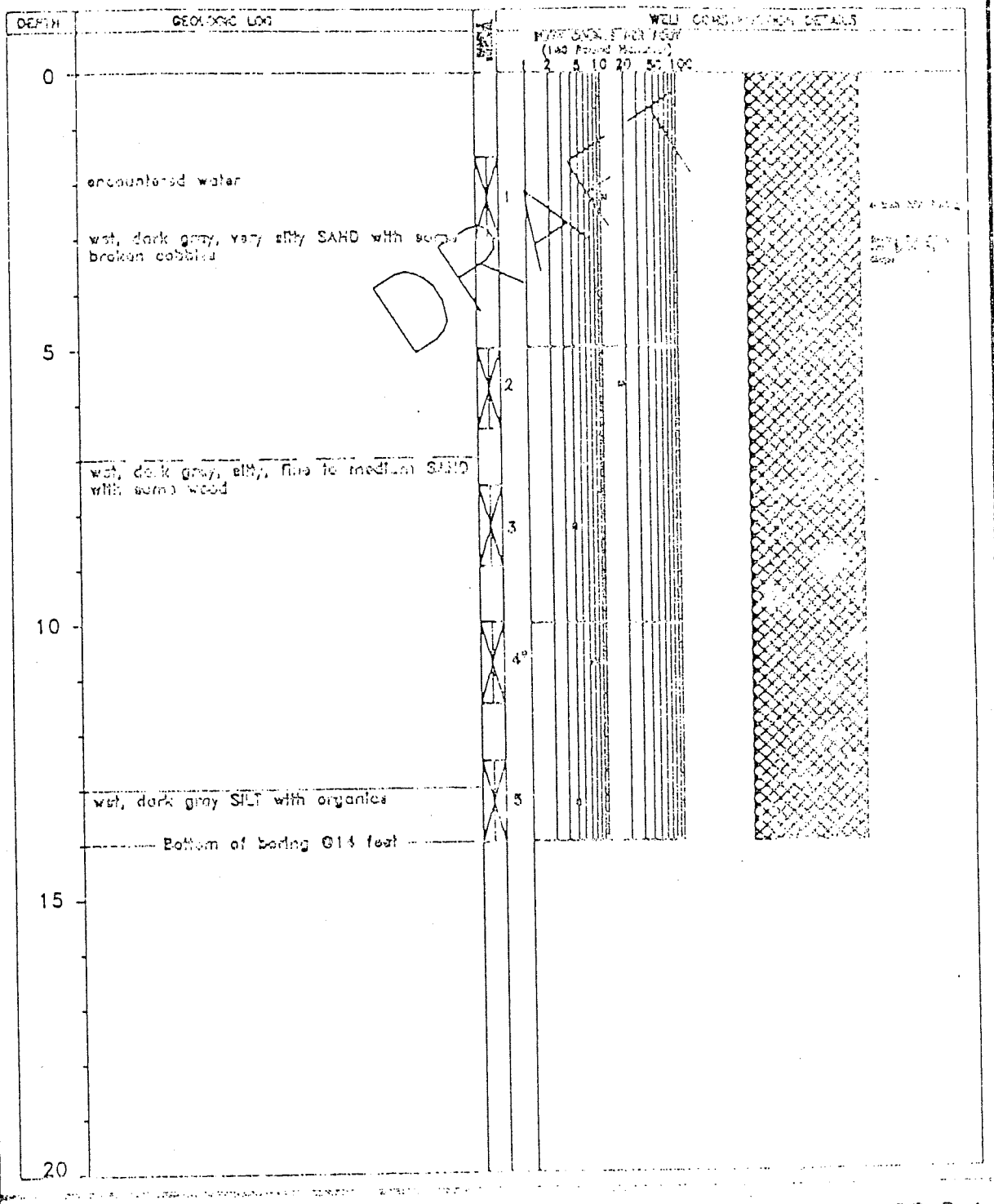


FIGURE B-1

California Sampler

SPT Sample

Thin-Walled Tube Push Sample

• No Recovery

PROJECT NAME Everett Landfill
WELL INSTALLATION METHOD
DRILLING METHOD
DRILLER
FIRM
COMPANY
REPRESENTATIVE
DRILLED 4/9/85

KEY:

- ■ Indicates 3-inch OD Dames & Moore Sample.
- ◪ Indicates 2-inch OD Split Spoon Sample (SPT).
- ☒ Indicates Disturbed Sample.
- I Indicates No Recovery.
- □ Indicates Bag Sample.
- ⧅ Indicates Shelby Tube Sample.

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5mm) to No. 200 (0.074mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074 mm)

COMPONENT PROPORTIONS

DESCRIPTIVE TERMS	RANGE OF PROPORTION
Trace or little	0 - 5%
Some	5 - 12%
Clayey, silty, sandy, gravelly	15 - 30%
And	30 - 50%

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
DAMP	Some perceptible moisture; below optimum
MOIST	No visible water; near optimum moisture content
WET	Visible free water, usually soil is below water table.

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N -VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft) Shear	Approximate Relative Density (%)	Consistency	N (blows/ft)	Approximate Undrained Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	< 250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	> 4000

FIGURE A-17

LOG OF BORING B-43

RIVERFRONT REDEVELOPMENT PROJECT
EVERETT, WASHINGTON

Drilling Method: HS Auger
Sampling Method: SPT
Logged By: D. Kramer
Total Depth: 81.5 feet
Groundwater: 5' BGS ATD

Date Started: 3-22-99
Date Completed: 3-23-99
Elevation: 16.8 feet
Coordinates: N 1635.99
E 8026.58

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Dense, moist, brown-gray SAND (Fill)		0				
				33	20	
Loose to dense, wet, black to dark gray SAND with occasional wood and organics		10		57/11"	21	wood in sampler
				50/3"	61	sampler pushing log
		20		11		poor recovery
Soft to medium stiff, wet, gray clayey SILT with occasional peat				3	60	fines content = 81%
						poor recovery, wood at bottom of tube
grades to medium stiff		30		4	73	
						no recovery
grades to soft, gray, clayey Silt/silty Clay				0	102	
		40		3	56	
				0	80	
				4	80	
		50				
grades to very soft				0	59	weight of rod push sampler 18", fine content = 89%
		60		0	50	
grades to gray				0	52	2" ash layer with leaves
		70				

Floyd & Snider Inc.

LOG OF BORING B-43

RIVERFRONT REDEVELOPMENT PROJECT
EVERETT, WASHINGTON

Drilling Method: HS Auger
Sampling Method: SPT
Logged By: D. Kramer
Total Depth: 81.5 feet
Groundwater: 5' BGS ATD

Date Started: 3-22-99
Date Completed: 3-23-99
Elevation: 16.8 feet
Coordinates: N 1635.99
E 8026.58

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Medium stiff, wet, gray fine sandy SILT		70		6	32	fines content = 71%
grades to very stiff				17	34	fines content = 87%
grades to wet, very soft gray-black peaty clayey SILT		80		1	82	
Boring completed at a depth of about 81.5 feet below the surface. Groundwater was encountered at about 5 feet below the ground surface at the time of drilling.		90				
		100				
		110				
		120				
		130				
		140				

Floyd & Snider Inc.

LOG OF BORING B-44

RIVERFRONT REDEVELOPMENT PROJECT
EVERETT, WASHINGTON

Drilling Method: HS Auger
Sampling Method: SPT
Logged By: D. Kramer
Total Depth: 76.5 feet
Groundwater: 8.8' BGS ATD

Date Started: 3-30-99
Date Completed: 3-30-99
Elevation: 17.2 feet
Coordinates: N 3181.24
E 7449.69

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Very loose to medium dense, moist, gray silty SAND, trace of gravel		0				
				28	19	
Very soft, wet, dark gray to black organic SILT		10		2	190	
grades to soft, dark brown to gray clayey SILT with significant organics				4	209	
		20		1.5	58	fines content = 89%
Medium stiff to stiff, wet, gray sandy SILT				5	36	
		30		16	34	fines content = 51%
Medium dense, wet, dark gray, silty, fine to coarse SAND				15	30	fines content = 26%, Heave noted and drilling mud added
grades to dense to very dense, gray fine clean SAND		40		53	21	Heave noted
				75	23	
grades to fine to medium SAND				30	24	
		50		50/6"	25	
				44	24	
				52	26	heave noted
				36	29	
		60		10	34	fines content = 32%
Stiff to very stiff, wet, gray sandy SILT				20	43	fines content = 51%
Dense to very dense, wet, gray coarse SAND				48	26	
grades to fine to medium sand		70		68	22	

Floyd & Snider Inc.

LOG OF BORING B-44

RIVERFRONT REDEVELOPMENT PROJECT
EVERETT, WASHINGTON

Drilling Method: HS Auger
Sampling Method: SPT
Logged By: D. Kramer
Total Depth: 76.5 feet
Groundwater: 8.8' BGS ATD

Date Started: 3-30-99
Date Completed: 3-30-99
Elevation: 17.2 feet
Coordinates: N 3181.24
E 7449.69

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Dense to very dense, wet, gray coarse SAND		70		52	28	
grades to fine sand				62	23	
				49	29	
Boring completed at a depth of about 76.5 feet below the ground surface. Groundwater was encountered at about 9 feet below the ground surface at the time of drilling.		80				
		90				
		100				
		110				
		120				
		130				
		140				

Floyd & Snider Inc.

LOG OF BORING B-54

RIVERFRONT REDEVELOPMENT PROJECT
EVERETT, WASHINGTON

Drilling Method: HS Auger
Sampling Method: SPT
Logged By: D. Kramer
Total Depth: 91.5 feet
Groundwater: 2.5' BGS ATD

Date Started: 3-29-99
Date Completed: 3-29-99
Elevation: 15.2 feet
Coordinates: N 1463.19
E 7508.13

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Loose, moist, brown fine to medium SAND (Fill)		0				
				5	24	
				10		
grades to very loose		10		2		
Soft, wet, gray to dark brown clayey SILT with organics				3	153	
grades to sandy SILT		20		3	38	fines content = 48%
grades to medium stiff				7	36	
Dense, wet, gray fine to coarse SAND		30		36	14	
				34	17	
		40		48	16	
				41	25	
				48	18	
		50		48	17	
				39	16	
				29	22	
Medium stiff to stiff, gray clayey SILT				6	48	
grades with organics		60		9	49	fines content = 79%
Stiff to very stiff, wet, gray sandy SILT				19	48	fines content = 63%
				12	31	
		70				

Floyd & Snider Inc.

LOG OF BORING B-54

RIVERFRONT REDEVELOPMENT PROJECT
EVERETT, WASHINGTON

Drilling Method: HS Auger
Sampling Method: SPT
Logged By: D. Kramer
Total Depth: 91.5 feet
Groundwater: 2.5' BGS ATD

Date Started: 3-29-99
Date Completed: 3-29-99
Elevation: 15.2 feet
Coordinates: N 1463.19
E 7508.13

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Medium stiff to stiff, wet, gray clayey SILT with organics		70		9	41	
				7	60	
grades to very stiff		80		22	32	
Dense, wet, gray fine to medium SAND with occasional thin silt layers				22	23	heave noted
				58	20	heave noted
		90		80/11"	21	
Boring completed at a depth of about 91.5 feet below the ground surface. Groundwater was encountered at about 2.5 feet below the ground surface at the time of drilling.		100				
		110				
		120				
		130				
		140				

Floyd & Snider Inc.

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils	Gravel and Gravelly Soils	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
	More than 50% of Coarse Fraction Retained on No. 4 Sieve	Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
More than 50% Retained on No. 200 Sieve Size	Sand and Sandy Soils	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
	50% or More of Coarse Fraction Passing No. 4 Sieve	Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILT/Organic CLAY
Highly Organic Soils			PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit, LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
PID	Photoionization Device Reading
PP	Pocket Penetrometer Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
TV	Torvane Approx. Shear Strength (tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- 3-1/4" OD Split Spoon with Brass Rings
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- Non-standard Penetration Test (3.0" OD split spoon)

GROUNDWATER SYMBOLS

- Groundwater Level (measured at time of drilling)
- Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)
Components are arranged in order of increasing quantities.	

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation. Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

HWA GEOSCIENCES INC.

EVERETT LANDFILL
EVERETT, WASHINGTON

LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 98165 FIGURE: B-1

DRILLING COMPANY: Cascade Drilling, Inc.
DRILLING METHOD: Direct Push Soil Probe
SAMPLING METHOD: core
LOCATION:

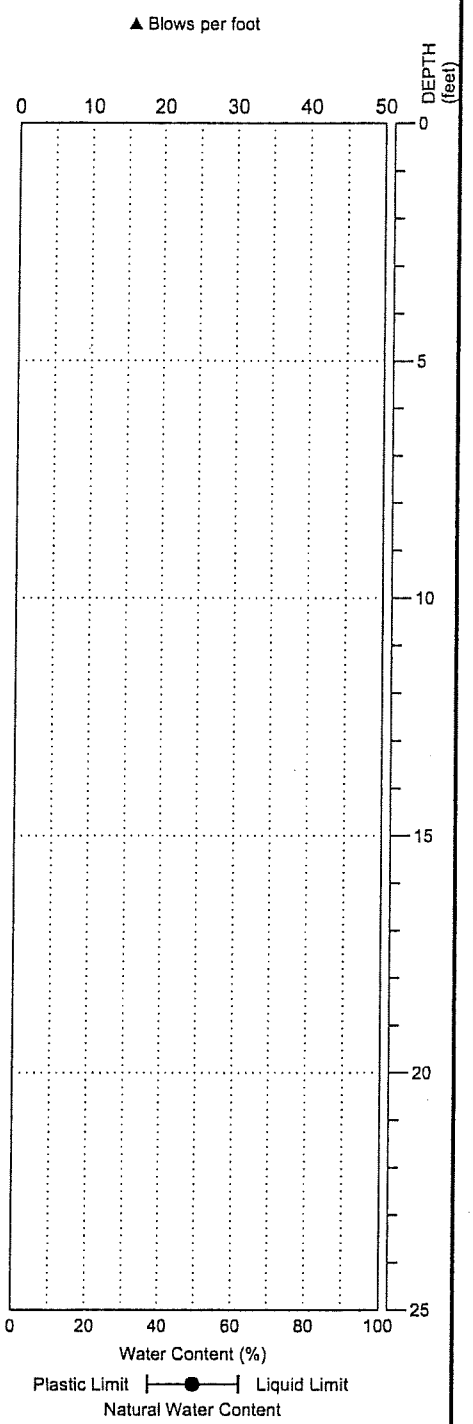
SURFACE ELEVATION: 16.33 ± feet

DATE STARTED: 4/28/2003
DATE COMPLETED: 4/28/2003
LOGGED BY: A. Sugar

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	PID (ppm)
0	GM	Brown and gray silty GRAVEL/crushed rock, dry.	0
5	SP SM	Gray silty fine to medium SAND, and SILTY SAND moist.	0
	SP	Dark gray fine SAND, wet. Trace wood chips.	
	OL	Dark brown organic plastic SILT, wet.	0
	ML	Gray silt with GRAVEL, wet, plastic.	
10	ML	Gray SILT with sand and gravel, wet, plastic.	
	ML SM	Gray silty fine SAND, wet, plastic. [FILL] very easy probing 12-16'	0
15	PT	Brown fibrous PEAT with patches of soft gray plastic silt, wet. [NATIVE ?]	0
20		Brown PEAT, few 3-4" gray silt layers, wet.	
25			

SAMPLE TYPE
SAMPLE NUMBER
PEN. RESISTANCE (blows/6 inches)
GROUNDWATER

Water sample collected, 3/4" PVC screen with 0.010" slots set at 0-10 feet



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

EVERETT LANDFILL
EVERETT, WASHINGTON

BORING:
BF-1

PAGE: 1 of 1

PROJECT NO.: 98165 FIGURE: B-2

BORINGE 98165.GPJ 6/4/03

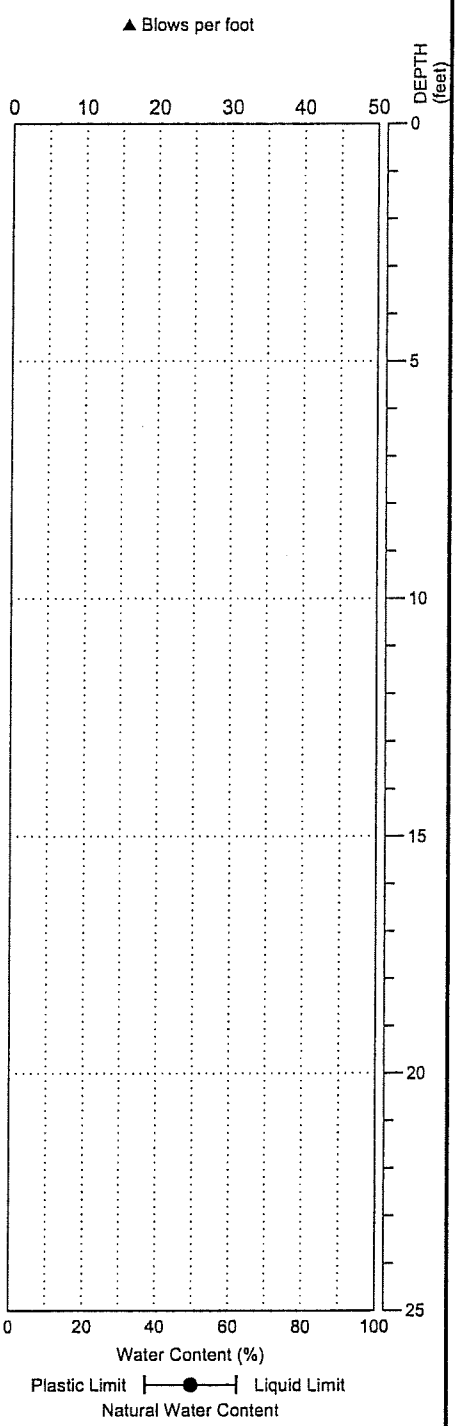
DRILLING COMPANY: Cascade Drilling, Inc.
DRILLING METHOD: Direct Push Soil Probe
SAMPLING METHOD: core
LOCATION:

SURFACE ELEVATION: 21.74 ± feet

DATE STARTED: 4/28/2003
DATE COMPLETED: 4/28/2003
LOGGED BY: A. Sugar

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	PID (ppm)	GROUNDWATER
0		SM	Brown, tan and black silty sand, gravel and silt with bricks, crushed rock, charcoal, dry. [FILL]			0	
5						0	
		ML	Light gray SILT, dry, non-plastic.				
		SM	Light gray silty fine sand, moist, with woody debris and bricks. [FILL]			0	▽
		SM	Gray, silty, fine to medium SAND, wet.				
10		SP	Gray fine to medium SAND, wet, few organics, woody/peat laminae. [FILL]			0	
15			Dark gray fine to medium SAND, wet.				
		PT	Dark gray and brown PEAT (woody debris and chips) and fine to medium SAND. 16-20' 6" recovery (wood at sampler tip): Black/dark gray silty SAND and woody debris, wet.			0	
20		PT	Soft, brown, plastic SILT and wood, wet. [PEAT]			0	
25							

Water sample: 3/4" PVC screen, 0.010" slot @8-18 feet



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

EVERETT LANDFILL
EVERETT, WASHINGTON

BORING:
BF-2

PAGE: 1 of 1

PROJECT NO.: 98165 FIGURE: B-3

BORINGE 98165.GPJ 6/4/03

APPENDIX C
LABORATORY ANALYSIS

APPENDIX C
LABORATORY ANALYSIS

GENERAL

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate engineering properties of the soil samples. Representative samples were selected for laboratory testing consisting of moisture content determination, dry density determination, percent fines content, particle size analyses, Atterberg limit tests, and consolidation tests. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

The results of the laboratory tests are presented in Figures C-1 through C-7. The results of the moisture content and dry density determinations are presented on the boring logs at the respective sample depths in Appendix A.

SOIL CLASSIFICATIONS

All soil samples obtained from the borings were visually classified in the field and/or in our laboratory using a system based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM test method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory tests results. These classification procedures are incorporated in the boring logs shown in Figures A-2 through A-6 in Appendix A.

MOISTURE CONTENT TESTING

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the borings. The results of these tests are presented on the boring logs in Appendix A at the depths at which the samples were obtained.

PERCENT PASSING U.S. NO. 200 SIEVE

Selected samples were "washed" through the No. 200 mesh sieve to determine the relative percentages of coarse and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to determine the fines content for analysis purposes. The tests were conducted in general accordance with ASTM D 1140, and the results are shown on the boring logs at the respective sample depth.

SIEVE ANALYSIS

Sieve analyses were performed on three samples obtained from the borings. The analyses were conducted in general accordance with ASTM D 422. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the USCS, and presented on Figure C-1.

ATTERBERG LIMITS TESTING

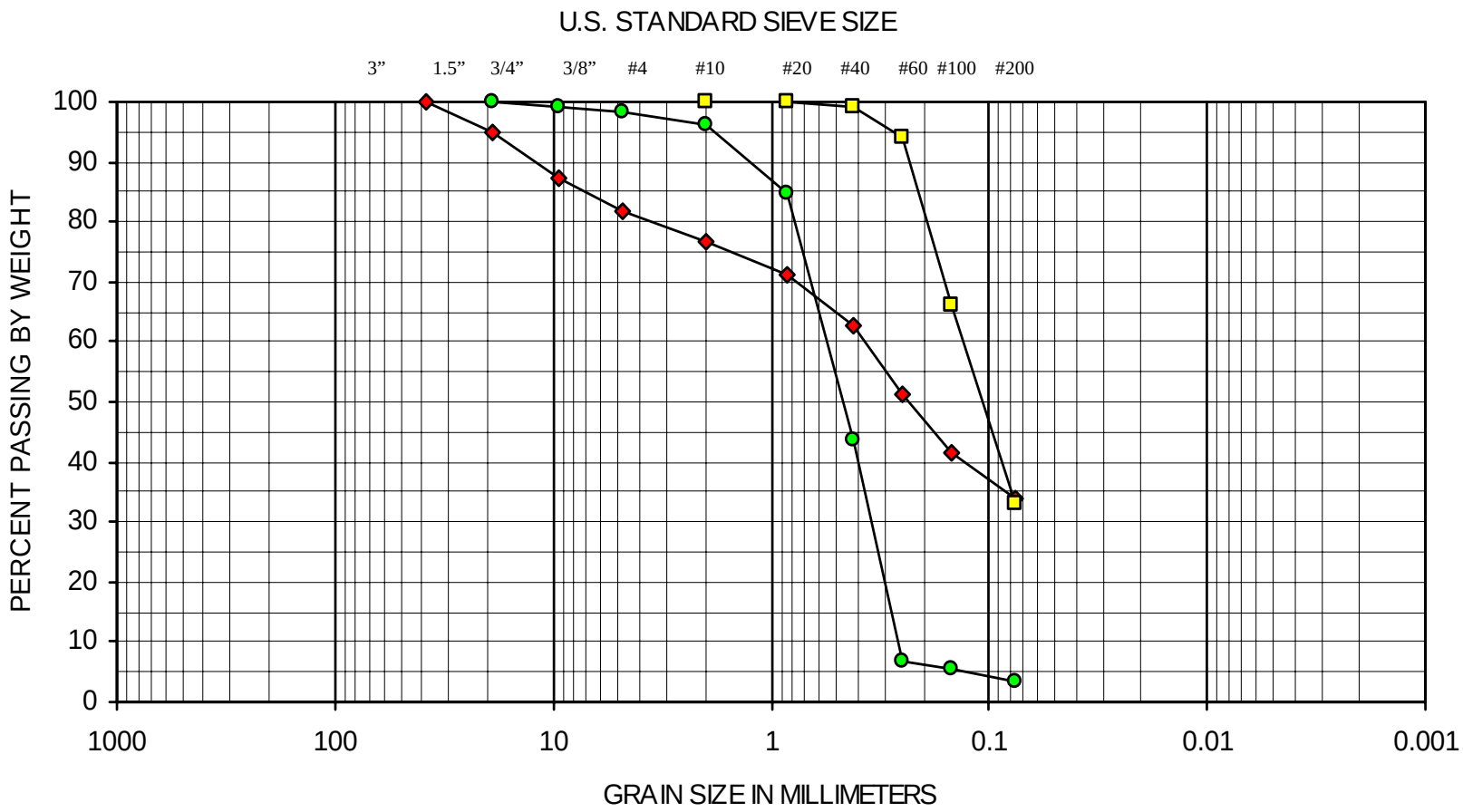
Atterberg limits testing (plasticity characteristics) was performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and the plastic limit were estimated through a procedure performed in general accordance with ASTM D 4318. The results of the Atterberg limits testing are summarized in Figures C-2 and C-3.

CONSOLIDATION TESTS

One-dimensional consolidation tests were conducted on four relatively undisturbed soil samples. The consolidation testing was conducted in general accordance with ASTM D 2435, using a fixed-ring consolidometer. The results of the consolidation tests are presented in Figures C-4 through C-7. The coefficient of consolidation, Cv, computed for selected load increments from the consolidation tests ranged from 0.19 to 8.37 ft^2/day. The modified compression index, $C_{C\varepsilon}$, ranged from 0.13 to 0.28.

6191-002-04 RCM : CTS : cmk 4-5-07 (Sieve.ppt)

GEOENGINEERS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	B-1	2.5	Silty sand with gravel (SM)
□	B-1	70	Silty sand (SM)
●	B-5	5	Poorly graded sand (SP)

SIEVE ANALYSIS RESULTS

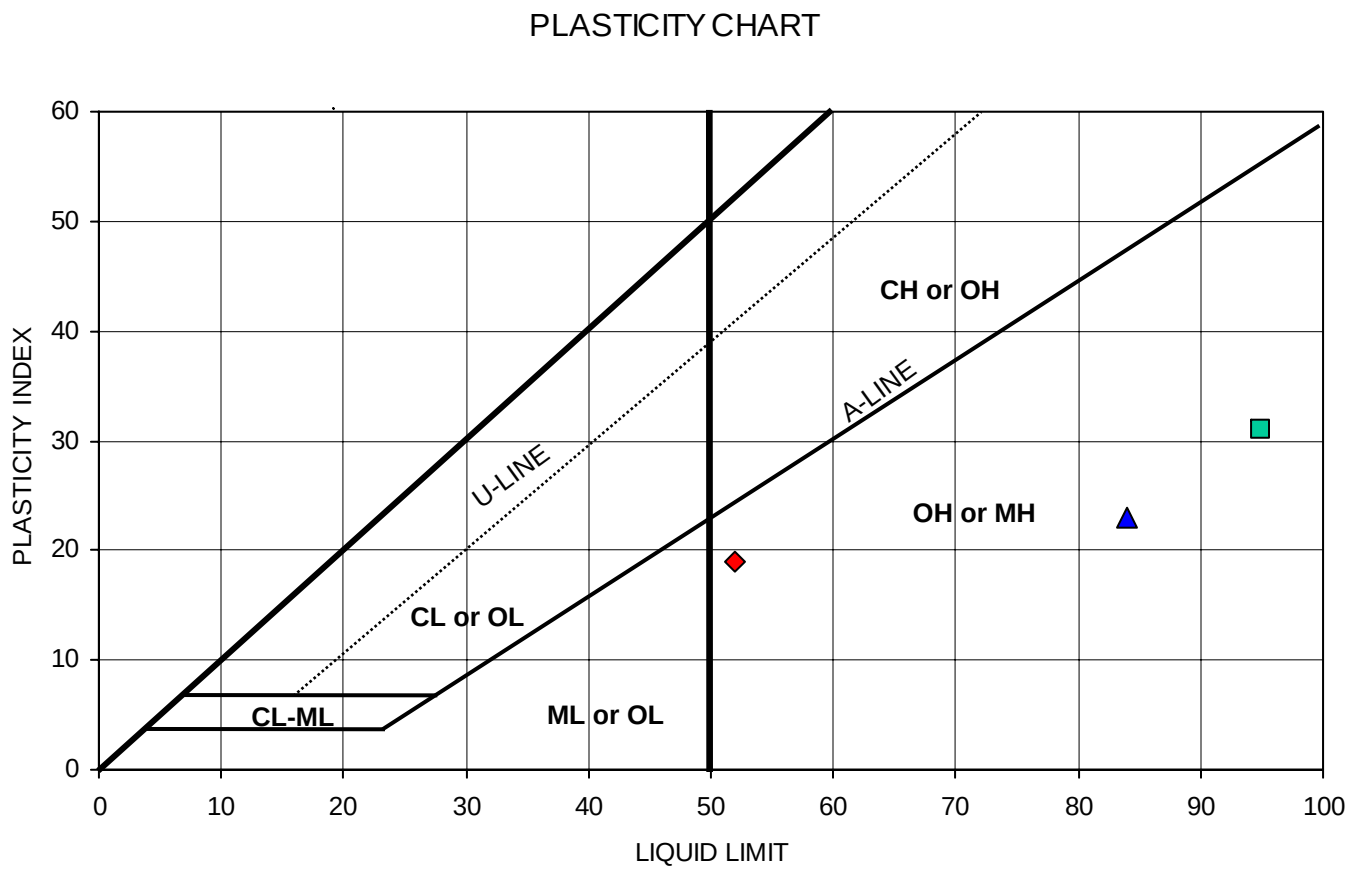
FIGURE C-1

6191-002-04 RCM : CTS : cts 3-19-07 (Atterbergs.ppt)

GEOENGINEERS

FIGURE C-2

ATTERBERG LIMITS TEST RESULTS



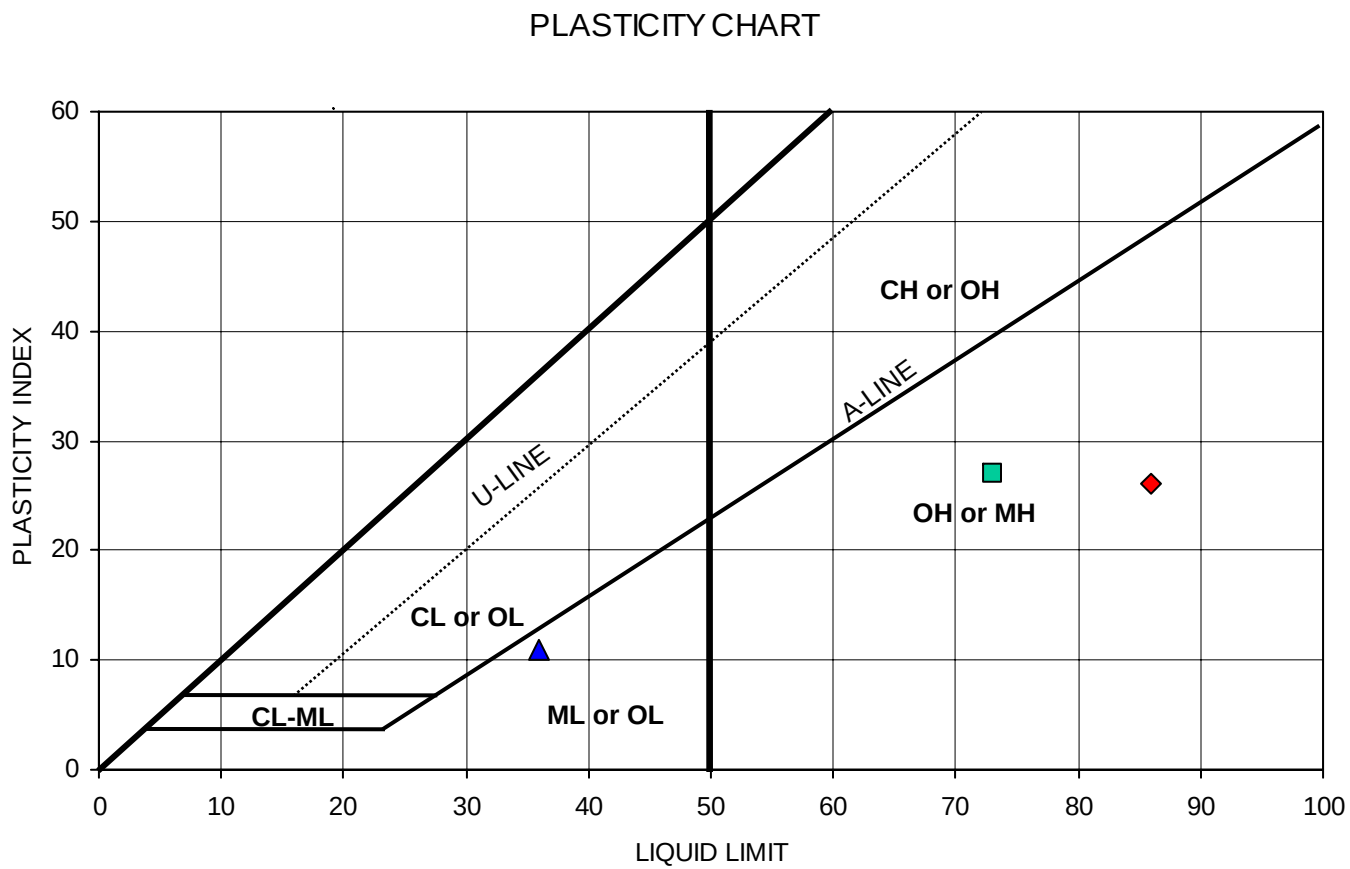
SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆	B-1	25.5	79	52	19	Elastic silt with trace organics (MH)
■	B-1	55	70	95	31	Organic silt (OH)
▲	B-1	80	56	84	23	Elastic silt with organics (MH)

6191-002-04 RCM : CTS : cmk 4-5-07 (Atterbergs.ppt)

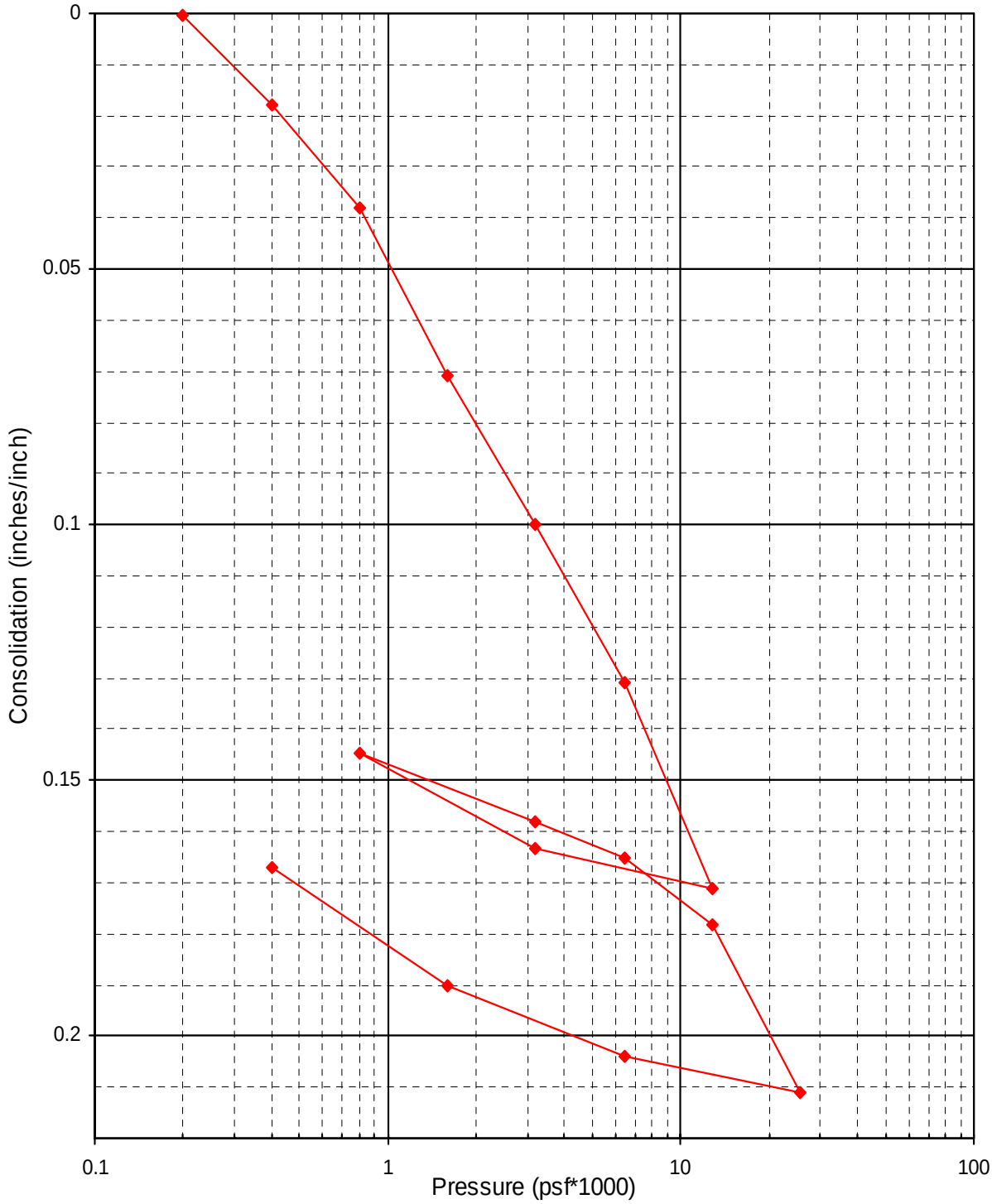
GEOENGINEERS

FIGURE C-3

ATTERBERG LIMITS TEST RESULTS



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆	B-2	30.5	75	86	26	Organic silt (OH)
■	B-2	67	55	73	27	Elastic silt with organics (MH)
▲	B-3	20.5	39	36	11	Silt (ML)



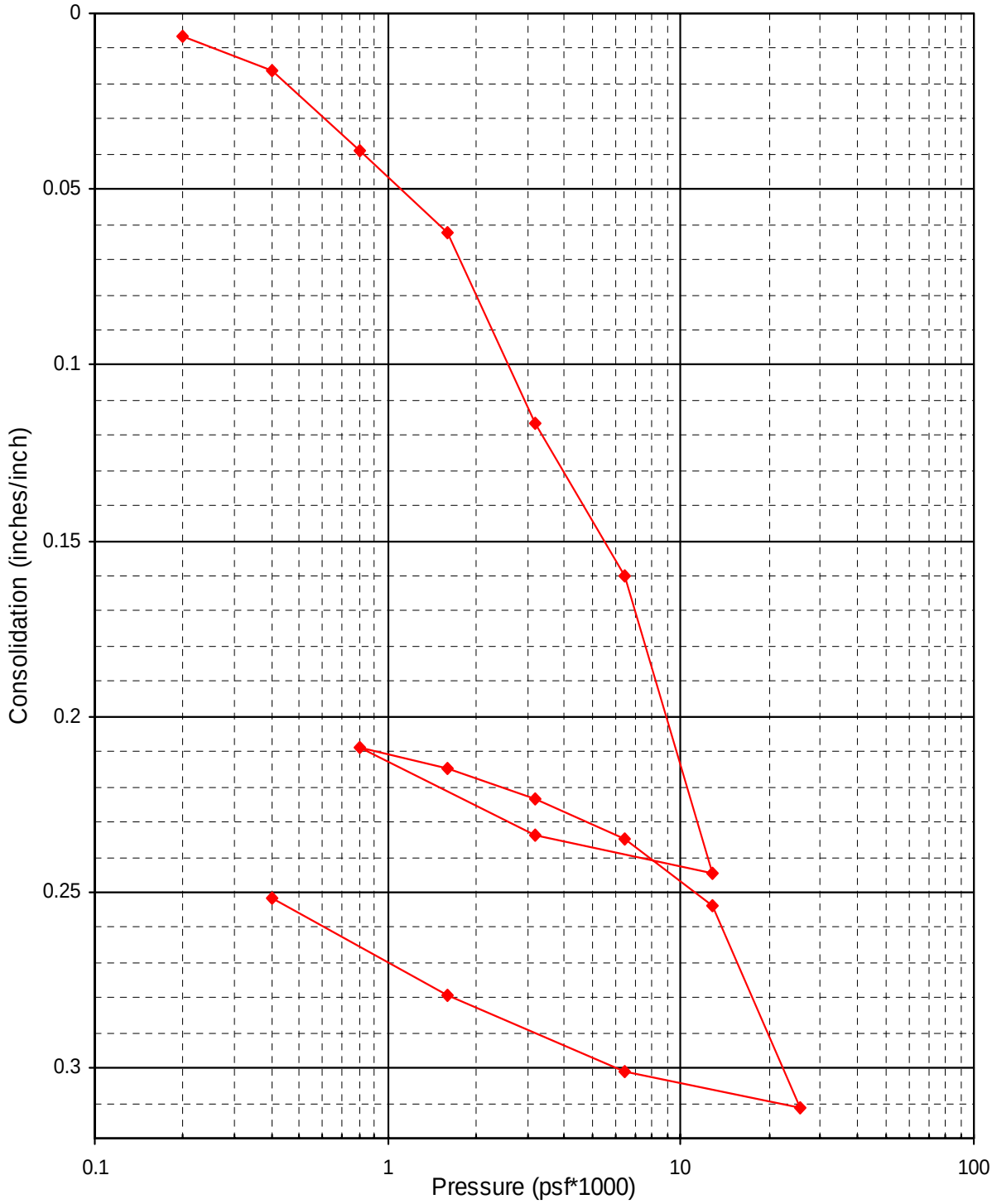
BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT³)
B-1	81	Elastic silt with fine sand and organic matter (MH)	39	81

6191-002-04 RCM/JRS/cmk 04/5/07 (consol.ppt)

GEOENGINEERS

CONSOLIDATION TEST RESULTS

FIGURE C-4



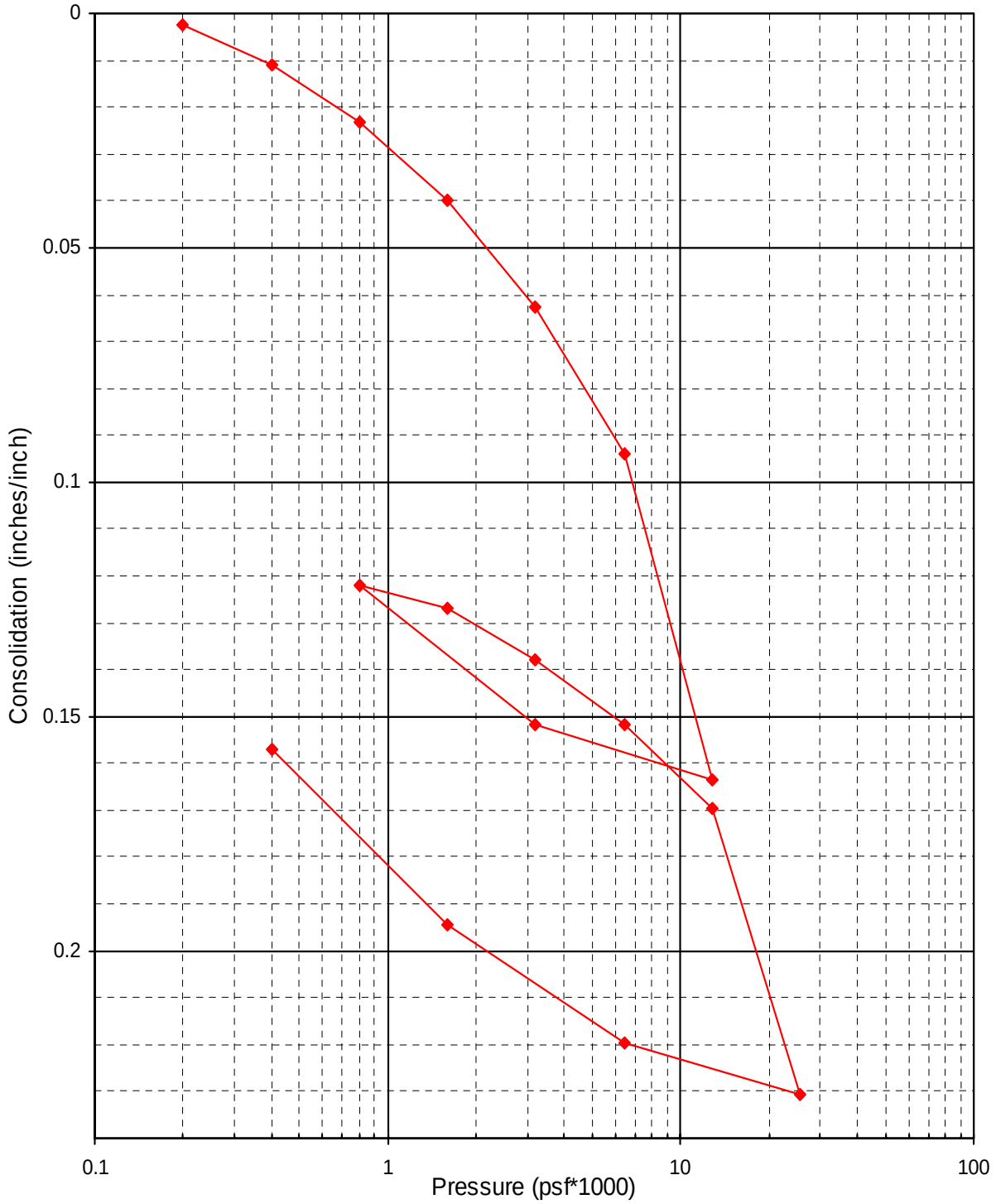
BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT³)
B-2	30	Organic silt with trace organics (OH)	70	56

6191-002-04 RCM/JRS/cmk 04/5/07 (consol.ppt)

GEOENGINEERS

CONSOLIDATION TEST RESULTS

FIGURE C-5



BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT³)
B-2	67.5	Elastic silt with organic matter (MH)	55	70

6191-002-04 RCM/JRS/cmk 04/5/07 (consol.ppt)

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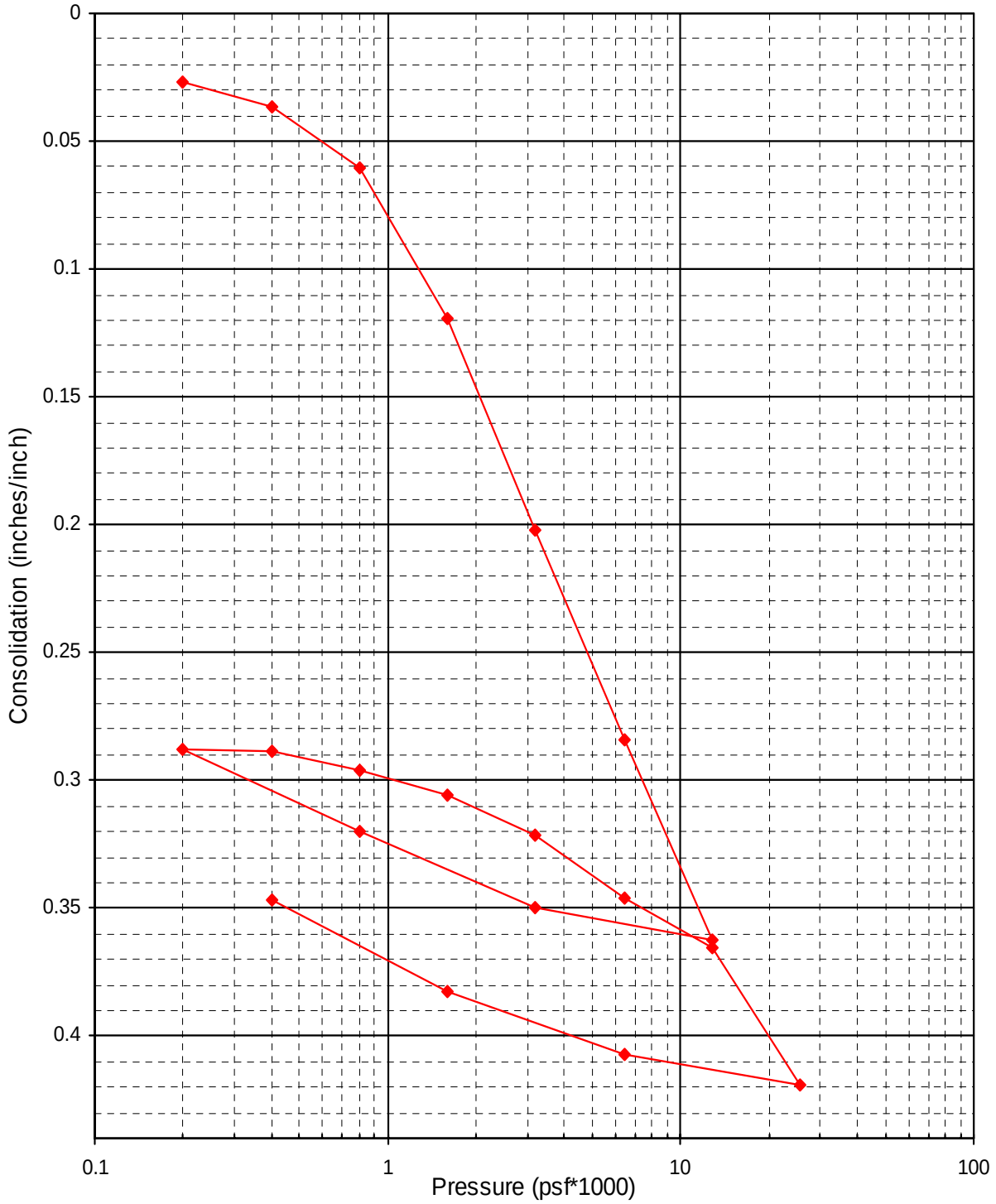
CONSOLIDATION TEST RESULTS

FIGURE C-6



BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT³)
B-3	20.5	Organic silt (OH)	83	50

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CONSOLIDATION TEST RESULTS

FIGURE C-7

6191-002-04 RCM/JRS/cmk 04/5/07 (consol.ppt)

APPENDIX D

REPORT LIMITATIONS AND GUIDELINES FOR USE

APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE

This appendix provides information to help you manage your risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report has been prepared for the exclusive use of Oliver McMillan Everett, LLC., and their authorized agents. This report may be made available to other members of the design team. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with Oliver McMillan Everett, LLC, and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the proposed development of the Simpson Pad as part of the Everett Riverfront Redevelopment Project located in Everett, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

MOST GEOTECHNICAL AND GEOLOGIC FINDINGS ARE PROFESSIONAL OPINIONS

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

DO NOT REDRAW THE EXPLORATION LOGS

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

GIVE CONTRACTORS A COMPLETE REPORT AND GUIDANCE

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

CONTRACTORS ARE RESPONSIBLE FOR SITE SAFETY ON THEIR OWN CONSTRUCTION PROJECTS

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

READ THESE PROVISIONS CLOSELY

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

GEOTECHNICAL, GEOLOGIC AND ENVIRONMENTAL REPORTS SHOULD NOT BE INTERCHANGED

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

BIOLOGICAL POLLUTANTS

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of biological pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of biological pollutants and no conclusions or inferences should be drawn regarding biological pollutants, as they may relate to this project. The term "biological pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.