Preliminary Geotechnical Engineering Services

Block 37 – South Lake Union Development
Seattle, Washington

for
City Investors XI, LLC

August 1, 2014

GeoEngineers
Earth Science + Technology

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File No. 7087-027-00

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INTRODUCTION

This report presents the results of GeoEngineers' preliminary geotechnical engineering services for the proposed Block 37 – South Lake Union development. The project site is located in Seattle's South Lake Union neighborhood and is bounded by Valley Street to the north, Terry Avenue North to the east, Mercer Street to the south, and Westlake Avenue North to the west. The site is shown relative to surrounding physical features on the Vicinity Map (Figure 1) and the Site Plan (Figure 2).

The purpose of this report is to provide preliminary geotechnical engineering conclusions and recommendations for the design of the new development. The site consists of six King County parcels (parcel numbers 408880-3355, 1987200015, 408880-3345, 408880-3240, 408880-3235, and 408880-3236) and covers approximately 1.59 acres. GeoEngineers' preliminary geotechnical engineering services have been completed in general accordance with our Master Services Agreement executed February 21, 2014. Our scope of work includes:

- Review available reports and studies for the site and surrounding area available from our files;
- Complete explorations at the site to characterize soil and groundwater conditions;
- Providing International Building Code (IBC) 2012 seismic design criteria;
- Providing preliminary foundation, temporary shoring, slab-on-grade and permanent below-grade wall recommendations;
- Providing preliminary recommendations for temporary and permanent dewatering and groundwater seepage estimates; and
- Preparing this report.

PROJECT DESCRIPTION

GeoEngineers understands that the planned development may consist of two towers, one residential tower and one office tower with up to three below-grade levels of parking. Temporary shoring will be required on each of the four sides of the planned excavation. Temporary dewatering will be necessary for the planned building configuration. Given that the planned building configuration will extend below the static groundwater table and that the site is capable of generating a significant dewatering flow rate, it is anticipated that the lower portion of the building will need to resist hydrostatic pressures. Variable soil conditions are present across the site at the anticipated foundation elevation and structural mat foundations bearing on native soils or on improved ground, where necessary, is anticipated for foundation support.

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

The subsurface conditions at the site were evaluated by drilling two borings, B-37-1 and B-37-2 to depths of 80 feet. Monitoring wells equipped with automatic dataloggers were installed in both borings to observe and monitor groundwater conditions. The approximate locations of the explorations for this and previous studies are shown in Figure 2. Descriptions of the field exploration program and the boring logs are presented in Appendix A.

Laboratory Testing

Soil samples were obtained during drilling and were taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of the fines contents and moisture contents. A description of the laboratory testing is presented in Appendix A and the test results are presented in the boring logs in Appendix A.

PREVIOUS SITE EVALUATIONS

In addition to the explorations completed as part of this evaluation, the logs of selected explorations from previous site evaluations in the project vicinity were reviewed. The logs of explorations from previous projects referenced for this study are presented in Appendix B. The existing subsurface information includes:

- The log for one boring (B-43-1) completed by GeoEngineers in 2013.
- The log for one boring (CHB-111) completed by CH2MHILL in 2008.
- The logs of two borings (B-3-98 and B-4-98) completed by GeoEngineers in 1998.
- The logs of two borings (BB-6 and BB-14) completed by HWA GeoSciences in 1998.
- The logs of eight borings (B-401, B-402, B-422, B-423, B-424, B-425, B-426, and B-430) completed by Shannon and Wilson in 1970.

SITE CONDITIONS

Surface Conditions

Block 37 is bounded by Valley Street to the north, Terry Avenue North to the east, Mercer Street to the south, and Westlake Avenue North to the west. The north portion of the site is an asphalt paved parking lot and the south portion of the site is occupied by a water treatment system and job trailers to support the construction of Block 43. Site grades are relatively flat throughout the site, with approximately a 2-foot grade change across the site.

Subsurface Soil Conditions

The site is located in a low lying area of mapped fill, recent clay and granular deposits, and glacially consolidated soils. Previous borings in the vicinity encountered a significant thickness of variable fill (that in many cases in this area contains wood debris from an historic lumber mill located in this area), recent clay, and recent granular deposits overlying very dense/hard glacially consolidated deposits. The fill, recent clay, and recent granular deposits are compressible and typically represent poor bearing soils for new structures with significant foundation loading.

GeoEngineers' understanding of subsurface conditions is based on review of existing geotechnical information and the results of two new borings (B-37-1 and B-37-2) drilled as part of this study. The approximate locations of these explorations are presented in the Site Plan, Figure 2. Interpreted subsurface conditions are presented in Sections A-A' and B-B' (Figures 3 and 4).

The soils encountered at the site consist of fill, recent silt, clay and granular deposits, and glacially consolidated soils. Wood debris is present near the transition between the fill soils and recent deposits. The fill, wood debris and organic soil are unsuitable for foundation support. The recent silt, clay, and granular deposits are compressible and may not be suitable bearing soils for new structures with significant foundation loading and/or stringent static/seismic settlement tolerances. The glacially consolidated soils represent competent bearing soils for shallow foundations and/or deep foundations.

The fill generally consists of very loose to medium dense silty sand with variable gravel content and gravel with varying amounts of sand and fines. The thickness of fill encountered in the explorations completed for this study ranged from 17 to 30 feet, with the thickest area of fill found in borings along the north side of the site. Construction debris consisting of brick, concrete, and wood were observed to be present in the borings and is typical of the area.

Below the fill, recent clay, silt and granular deposits were encountered. These recent deposits were encountered in each the borings and consist of layered very soft to stiff sandy silt, silt, and clay and very loose to dense sand with varying amounts of silt and gravel. The thickness of the recent deposits ranges up to 15 feet to 30 feet across the site.

The glacially consolidated soils consist of cohesionless sand and gravel deposits. The cohesionless sand and gravel deposits were encountered below the recent deposits and consist of dense to very dense sand with varying amounts of silt and gravel and very stiff to hard silt with variable sand content. The glacially consolidated deposits extend to depths explored.

Groundwater Conditions

Measurements of depth to groundwater in the monitoring wells installed in both borings on Block 37 (B-37-1 and B-37-2) indicate that the site groundwater is present in a shallow aquifer and a deeper aquifer. This shallow and deep aquifer condition was also noted on recent nearby projects (Block 43, Block 44, Block 45, UW Medicine Phase 2 and 3, among others). Automatic dataloggers were installed in both monitoring wells to observe the variability in groundwater levels seasonally and following significant rainfall events. Figure 5 presents the groundwater elevations observed on Block 37 using the data from the automatic dataloggers as well as discrete groundwater measurements.

The table below provides a summary of the monitoring wells and recent groundwater measurements at the site.

Well ID	Ground Surface Elevation (feet)	Top of Casing Elevation (feet)	Bottom of Casing Elevation (feet)	Well Screen Elevation Range (feet)	Measured Groundwater Elevation Range Between 5/5/14 to 7/1/14 (feet)
B-37-1	28.4	28.0	-8.5	1.5 to -8.5	9.9 to 8.7
B-37-2	29.6	29.4	-30.9	-20.9 to -30.9	-1.4 to -2.3

The measured groundwater levels in the shallow aquifer fluctuate between Elevation 9.9 and 8.7 feet and groundwater levels in the deeper aquifer fluctuate between Elevation -1.4 and -2.4 feet based on the groundwater measurements discussed above. Additional groundwater measurements will be taken during the design phase of the project to further assess variations in groundwater elevations. Groundwater level readings taken to date are anticipated to be at a significantly lower level than typical due to recent dewatering on nearby projects in the site vicinity.

CONCLUSIONS AND RECOMMENDATIONS

Summary

A summary of the preliminary geotechnical considerations is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The site meets the characteristics of Site Class F in the 2012 IBC due to the presence of potentially liquefiable soils. For preliminary design purposes, we recommend that the design response spectrum developed through a site-specific seismic response analysis for the Block 43 (located across Westlake Avenue to the west of the site) be used for the Block 37 site (see Figure 11). Once the building configuration, foundation elevation, and foundation type has been more clearly defined, a site specific seismic response analysis will be completed for the Block 37 site to develop the design response spectrum.
- Based on review of the site specific groundwater levels collected at the site and in the site vicinity since 2011, a design groundwater table of Elevation 20 feet is recommended for Block 37 for the design of the permanent below grade walls and structural mat foundations. The planned excavation will extend below approximate Elevation 20 feet; therefore, temporary dewatering will be required to complete the planned excavation.
- Temporary dewatering can be completed using vacuum wellpoints or deep dewatering wells. The type of dewatering system and the system's configuration will depend on the type of temporary shoring system implemented and on the contractor's preferences for completing excavation and construction of the below grade portion of the building. Significant dewatering flows are anticipated where excavations extend below the groundwater level. The depth of the excavation, the type of temporary shoring system, and the type dewatering system design will influence the dewatering flow rates.
- Excavation support can be provided by either conventional soldier pile and tieback shoring system or through the use of an anchored diaphragm shoring wall. Several options for anchored diaphragm walls are feasible for Block 37 including secant pile walls, cutter soil mix (CSM) walls, sheet pile walls, cast in-situ reinforced concrete walls using slurry trench techniques, or ground freezing. Conventional soldier pile and tieback walls are feasible with temporary dewatering; however, the extent of drawdown of groundwater off-site is significant and settlement of nearby improvements due to lowering the groundwater table is possible.
- Further evaluation by the project team is necessary to select the excavation support system and should consider the following variables: cost, settlement of adjacent off-site improvements, impact of obstructions such as wood debris or concrete from previous development at the site, impact of dense soils at depth, impact on dewatering costs and effluent treatment and discharge costs, whether the system can be used as the permanent below grade basement wall, installation vibrations, and below grade waterproofing.
- For a building with two to three levels below grade, permanent dewatering flows are anticipated to be significant and designing the below grade portion of the building to resist hydrostatic pressures is

recommended due to life cycle pumping costs, effluent discharge constraints, and soil and groundwater conditions at the site.

- Recent deposits are anticipated at the foundation elevation. Given the depth of the planned building below the groundwater table, the need for a structural mat designed to resist hydrostatic pressure, and the variable soil conditions at the foundation elevation, a structural mat foundation bearing on improved ground, where necessary, is the preferred foundation system. For preliminary design purposes, an allowable bearing pressure of 4 to 6 kips per square foot (ksf) can be assumed. The allowable bearing is highly dependent on the foundation elevation, the type and extent of ground improvement and the settlement tolerances of the building and will be further evaluated during design.
- Ground improvement can be implemented to provide uniform foundation bearing across the variable soil conditions at the foundation elevation and to limit settlement to acceptable levels. Several options for ground improvement are available including rigid inclusions, compaction grouting, soil mixed columns, and driven timber piles. Stone columns or similar permeable ground improvement options are not recommended as this type of improvement will increase dewatering requirements.
- Buoyant pressures acting on the portion of the building extending below the groundwater table should be evaluated to determine if tiedown anchors are required and to determine when the temporary dewatering system can be turned off.

Our specific geotechnical recommendations are presented in the following sections of this report.

Earthquake Engineering

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts that are below the water table.

The results of our preliminary analyses indicate that the very loose to medium dense sandy fill and recent deposit soils have a moderate potential for liquefaction during a design earthquake event. At this time, it is not known whether these soils will be present below the foundation elevation.

Depending on the foundation elevation, ground improvement may be required to mitigate the potential for differential settlement and to transfer the building loads through the potentially liquefiable soils to the underlying bearing soils.

Lateral Spreading

Lateral spreading involves lateral displacement of large, surficial blocks of soil as the underlying soil layer liquefies. Due to the distance to Lake Union and given that the building will bear on non-liquefiable and improved soils, the potential for lateral spreading is considered to be low for the Block 37 site.

Other Seismic Hazards

Due to the location of the site and the site’s topography, the risk of adverse impacts resulting from seismically induced slope instability, differential settlement, or surface displacement due to faulting is considered to be low.

Seismic Design Information

The site meets the characteristics of Site Class F in the 2012 IBC due to the presence of potentially liquefiable soils. GeoEngineers completed a site specific seismic response analyses for the Block 43 project site located one block west of the Block 37 site to develop design spectra for use in the design of the building. For preliminary design purposes, we recommend that the Block 43 design response spectrum presented on Figure 11 be used for Block 37. Once the building configuration, foundation elevation, and foundation type has been more clearly defined, a site specific seismic response analysis will be completed for the Block 37 site to develop the design response spectrum.

Temporary Dewatering

Temporary dewatering is anticipated to be required to complete the planned excavation. Temporary dewatering may be accomplished using a variety of means; however, the use of either deep dewatering wells, vacuum wellpoints, or a combination of these two methods, is anticipated for this site. The type of temporary dewatering system will depend on the depth of excavation, type of temporary shoring system, extent of offsite drawdown, constructability considerations, and other factors.

Deep dewatering wells can also be used for temporary dewatering. Deep wells can be located either inside or outside the temporary shoring system where conventional shoring is implemented. If a diaphragm type shoring system is used, the deep wells should be located within the excavation in order to reduce dewatering flows. Deep well locations should be coordinated with the foundation design to allow for foundation construction prior to decommissioning of the wells. Where the deep wells are located within the excavation, careful detailing of the structural mat foundation/dewatering well penetration is required to provide a reliable and watertight seal following well decommissioning.

Vacuum wellpoints will be effective where the groundwater table is to be lowered by up to 15 to 20 feet below current levels. Where conventional shoring is used, the vacuum wellpoints should be installed from within the perimeter of the excavation and extend through the shoring wall at a steeply inclined angle. Where a diaphragm type shoring system is used, the wellpoints should be installed within the excavation to reduce dewatering flows. The header pipe should be located near the static groundwater table elevation prior to completing the excavation below this elevation. Vacuum wellpoints should be designed with an appropriately graded filter pack of sufficient thickness to promote groundwater inflow while limiting the migration fines, and should be constructed by an experienced dewatering contractor who is also a licensed well driller registered with the State of Washington (per WASC 173-162). Depending on the depth of the planned excavation, deep dewatering wells in addition to the vacuum wellpoints may be necessary at the center of the excavation.

The temporary dewatering system should be designed to maintain the groundwater level at least 3 feet below the foundation subgrade elevation until the below-grade portion of the structure is capable of withstanding the hydrostatic pressures resulting in uplift on the bottom of the foundation and structural slab and lateral pressures against below-grade walls.

Most of the groundwater flow into the planned excavation is anticipated to be produced from the recent granular deposits and the glacially consolidated cohesionless sand and gravel deposits. Based on previous temporary dewatering experience at Block 43 and similar soil conditions, we anticipate similar dewatering flow rates, which may be up to 600 gallons per minute (gpm). Once the depth of excavation and type of shoring system has been determined, a more refined estimate of dewatering flow rates can be developed.

GeoEngineers recommends that groundwater monitoring wells or piezometers be installed throughout the excavation in order to monitor groundwater levels inside and outside of the planned excavation during construction. The purpose of the groundwater monitoring wells is to confirm that the dewatering system is performing as intended and to confirm that dewatering is functioning to reduce the potential for excessive buoyant pressures acting on the building until sufficient structural loads are present to resist buoyancy.

Settlement Impacts to Adjacent Improvements

Settlement of the adjacent streets, buildings and utilities caused by increases in effective stress as groundwater levels are lowered by temporary dewatering is possible given that potential groundwater drawdown will occur in the fill, recent granular deposits, and recent silt and clay deposits. Based on review of the subsurface information for the Block 37 site, the soils that are considered to be prone to dewatering induced settlement consist primarily of the fill, wood waste and portions of the recent deposits located above approximate Elevation 0 feet. Previous temporary dewatering in the site vicinity has lowered water levels to below Elevation 0 feet in the Block 37 vicinity. As a result, the majority of potential settlement associated with temporary dewatering has likely already occurred.

On the adjacent Block 43 project, settlement in the rights-of-way around the site was determined to result from three factors: (1) dewatering induced settlement, (2) settlement resulting from shoring wall deformation, and 3) settlement resulting from installation of tieback anchors (high pressure compressed air was used to drill the tiebacks on Block 43). Given that the majority of settlement associated with dewatering has likely occurred, the settlement related shoring wall movement and tieback drilling can be managed by the selection of the earth pressures for the temporary shoring wall design and methodology used to install tieback anchors.

It is recommended that a settlement monitoring program be implemented to confirm that dewatering induced settlements do not adversely impact existing facilities. Settlement monitoring can be combined with the optical survey monitoring typically implemented as part of the construction of temporary shoring.

Excavation Support

Based on current design concepts, excavation depths will extend up to 35 feet below existing grades. Excavation support can be provided by either a conventional soldier pile and tieback shoring system or through the use of an anchored diaphragm shoring wall. Several options for anchored diaphragm walls are feasible for Block 37 including secant pile walls, CSM walls, sheet pile walls, cast in-situ reinforced concrete walls using slurry trench techniques, or soil freezing.

Conventional soldier pile and tieback walls are feasible with temporary dewatering; however, the extent of drawdown of groundwater off-site is significant and settlement of nearby improvements due to lowering the groundwater table is possible.

Diaphragm type shoring systems that are nearly impermeable and extend below the base of the excavation may reduce the dewatering pumping rates and extent of off-site drawdown, depending on the thickness of lower permeability soils present at the base of excavation. However, where less than approximately 20 feet of low permeability soils (such as recent silt or clay deposits) remain below the base of excavation, sand boils and/or base heave may result and may require higher pumping rates and extent of drawdown, thus reducing the benefit of a diaphragm type shoring system.

Further evaluation by the project team is necessary to select the excavation support system and should consider the following variables: cost, off-site settlement impacts, impact of obstructions such as wood debris or concrete from previous development at the site, impact of dense soils at depth, impact on dewatering costs and effluent treatment and discharge costs, whether the system can be used as the permanent below grade basement wall, installation vibrations, and below grade waterproofing.

In the sections below the shoring options will be described, including advantages and disadvantages.

Ground anchors should be designed to maintain an acceptable clearance from buried utilities. The shoring system will be required to be temporary because the ground anchors will extend into the City of Seattle right-of-way and a street use permit will be required.

Excavation Considerations

The site soils may be excavated with conventional excavation equipment, such as trackhoes or dozers. The contractor should be prepared for occasional cobbles and boulders in the site soils. Likewise, the surficial fill may contain foundation elements and/or utilities from previous site development, debris, rubble and/or cobbles and boulders. Significant wood debris and timber piling were encountered in the northeast portion of the excavation for the building at Block 44, along the eastern half of Block 43 excavation, and wood debris was noted in the explorations for the Block 37 project.

Wood debris was observed in many of the soil samples obtained from the borings and it is known that a saw mill operated in the site vicinity. Further exploration to determine the extent and nature of the wood debris is recommended if sheet piles are selected as the preferred shoring system. The further exploration would likely consist of test pits with a large excavator.

The fill and recent deposits have a significant amount of fine grained soils with high moisture contents. These soils are anticipated to provide poor support for construction equipment and to be highly susceptible to disturbance due to construction traffic and wet weather. The earthwork and shoring contractors should be prepared to operate equipment on poor subgrade conditions and to excavate soils disturbed by equipment loading or wet weather.

Conventional Shoring

Conventional shoring systems consisting of soldier pile walls with timber lagging and tieback anchors are considered an option for this project. Conventional shoring would require temporary dewatering to allow for the shoring system to be designed for fully dewatered conditions (no hydrostatic pressures acting on the shoring wall.

Soldier Pile and Tieback Walls

Soldier pile walls consist of steel beams that are concreted into drilled vertical holes located along the wall alignment, typically about 8 feet on center. After excavation to specified elevations, tiebacks are installed, if necessary. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles.

The advantages of soldier pile and tieback walls are that it is a standard shoring system that has been used successfully in Seattle and the South Lake Union area. There are several local contractors and the system is cost effective. The disadvantages with soldier pile and tieback shoring on this site include: (1) dewatering will result in a larger extent of drawdown and this will come with the risk of potential settlement to improvements in the drawdown zone, (2) dewatering pumping rates will be higher than the pumping rates for a diaphragm type shoring wall, (3) the system will require a separate permanent building wall to be constructed adjacent to the temporary wall, (4) waterproofing will be required between the temporary and permanent walls, (5) blockouts will be required around tieback heads to allow for de-stressing following completion of the below grade portion of the building, and (6) waterproofing around the tieback heads following de-stressing can be problematic.

For preliminary design purposes, soldier pile walls can be designed using the earth pressure diagram presented in Figure 7. The earth pressures presented in Figure 7 are for soldier pile walls with single or multiple levels of tiebacks, and the pressures represent the estimated loads that will be applied to the wall system for various wall heights.

The earth pressures presented in Figure 7 include the loading from traffic surcharge. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be considered by GeoEngineers on a case-by-case basis. No seismic pressures have been included in Figure 7 because it is assumed that the shoring will be temporary.

We recommend that the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 15 feet below the base of the excavation to resist “kick-out.” The axial capacity of the soldier piles must resist the downward component of the anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 40 ksf for piles supported on the glacially consolidated soils and 10 ksf in the fill or recent granular and recent silt and clay deposits. The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 2.5. The allowable end bearing value assumes that the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction of 1.5 ksf may be used on the embedded portion of the soldier piles within the glacially consolidated soils to resist the vertical loads.

Tiebacks

Tieback anchors will be required to resist the lateral pressures acting on the shoring wall. Tieback anchors should extend far enough behind the wall to develop anchorage beyond the “no-load” zone and within a stable soil mass. The anchors should be inclined downward at 15 to 45 degrees below the horizontal. Steeper anchor declinations may be required to achieve higher tieback capacities. Corrosion protection will not be required for the temporary tiebacks.

Centralizers should be used to keep the tieback in the center of the hole during grouting. Structural grout or concrete should be used to fill the bond zone of the tiebacks. A bond breaker, such as plastic sheathing, should be placed around the portion of the tieback located within the no-load zone if the shoring contractor plans to grout both the bond zone and unbonded zone of the tiebacks in a single stage. If the shoring contractor does not plan to use a bond breaker to isolate the no-load zone, GeoEngineers should be contacted to provide recommendations.

It is anticipated that the tiebacks will be drilled with casing. Holes drilled for tiebacks should be grouted/filled promptly to reduce the potential for loss of ground. Additionally, based on our experience of shoring installation at Block 43, it was discussed that some of the settlement along the perimeter of the excavation was attributed to high pressure compressed air during installation of the tiebacks.

We recommend the contractor develop tieback installation procedures or methods to reduce excessive air pressure during tieback installation.

Tieback anchors should develop anchorage in the recent deposits or glacially consolidated soils. We recommend that spacing between tiebacks be at least three times the diameter of the anchor hole to minimize group interaction. We recommend a preliminary design load transfer value between the anchor and soil of 4 kips per foot for glacially consolidated soils and 1.5 kips per foot for recent deposits. Higher adhesion values may be developed, depending on the anchor installation technique. The contractor should be given the opportunity to use higher adhesion values by conducting performance tests prior to the start of installing the production tieback anchors.

The tieback anchors should be verification- and proof-tested to confirm that the tiebacks have adequate pullout capacity. The pullout resistance of tiebacks should be designed using a factor of safety of 2. The pullout resistance should be verified by completing at least two successful verification tests in each soil type and a minimum of four total tests for the project. Each tieback should be proof-tested to 133 percent of the design load. Verification and proof tests should be completed as described in Appendix C, Ground Anchor Load Tests and Shoring Monitoring Program.

The tieback layout and inclination should be checked to confirm that the tiebacks do not interfere with adjacent buried utilities. The City of Seattle minimum clearances between ground anchors and existing utilities should be maintained.

Lagging

We recommend that the temporary timber lagging be sized using the procedures outlined in the Federal Highway Administration's Geotechnical Engineering Circular No. 4. The site soils are best described as competent soils. The following table presents recommend lagging thicknesses (roughcut) as a function of soldier pile clear span and depth.

Depth (feet)	**Recommended Lagging Thickness (roughcut) for clear spans of:**					
	5 feet	**6 feet**	**7 feet**	**8 feet**	**9 feet**	**10 feet**
0 to 25	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches
25 to 50	3 inches	3 inches	3 inches	4 inches	4 inches	5 inches

Lagging should be installed promptly after excavation, especially in areas where perched groundwater is present or where clean sand and gravel soils are present and caving soils conditions are likely. The workmanship associated with lagging installation is important for maintaining the integrity of the excavation.

The space behind the lagging should be filled with soil as soon as practicable. The City of Seattle requires that voids be backfilled immediately or within a single shift, depending on the selected method of backfill. Placement of this material will help reduce the risk of voids developing behind the wall and damage to existing improvements located behind the wall.

Material used as backfill in voids located behind the lagging should not cause buildup of hydrostatic pressure behind the wall. Lean concrete is a suitable option for the use of backfill behind the walls. Lean concrete will reduce the volume of voids present behind the wall. Alternatively, lean concrete may be used for backfill behind the upper 10 to 20 feet of the excavation to limit caving and sloughing of the upper soils, with on-site soils used to backfill the voids for the remainder of the excavation. Based on our experience, the voids between each lean concrete lift are sufficient for preventing the buildup of hydrostatic pressure behind the wall.

Drainage

A suitable drainage system should be installed to prevent the buildup of hydrostatic groundwater pressures behind the soldier pile and lagging wall. Seepage flows at the bottom of the excavation should be contained and controlled. Drainage should be provided for permanent below-grade walls as described below in the "Below-Grade Walls" section of this report.

Diaphragm Type Shoring Walls

Diaphragm type shoring systems that are relatively impermeable are considered an option for this project. Diaphragm shoring systems considered feasible for the Block 37 project include sheet piles, secant pile walls, CSM walls, cast in-situ reinforced concrete walls using slurry trench techniques (slurry walls), and ground freezing. The diaphragm shoring system should be designed to resist the full hydrostatic pressure resulting from the static pre-dewatering groundwater condition (approximately Elevation 21 feet).

The advantages of diaphragm type shoring walls include: (1) dewatering may result in a reduced extent of drawdown and a reduced risk of potential settlement to improvements in the drawdown zone (compared to conventional shoring), (2) dewatering pumping rates may be lower than the pumping rates for a conventional soldier pile and tieback shoring wall, (3) the sheet pile system and slurry trench system can result in a wall that can be used for both temporary and permanent below grade walls, and (4) the sheet pile system and slurry trench system can eliminate the need for a separate waterproofing system. The primary disadvantage with the diaphragm type shoring system is the cost of the system, particularly for systems where a permanent below grade wall is constructed in front of a temporary wall.

For preliminary design purposes, we recommend that diaphragm type shoring walls be designed using the earth pressure diagram presented in Figure 8. The earth pressures presented in Figure 8 are for multiple levels of tiebacks, and the pressures represent the estimated loads that will be applied to the wall system for various wall heights.

The earth pressures presented in Figure 8 include the loading from traffic surcharge. Other surcharge loads, such as cranes, construction equipment or construction staging areas, should be considered by GeoEngineers on a case-by-case basis. No seismic pressures have been included in Figure 8 because these pressures are intended for the temporary construction condition.

Tiebacks for diaphragm type shoring walls should be designed using the same recommendations presented above for conventional soldier pile and tieback walls. Other design aspects such as embedment depth, axial capacity, and skin friction are dependent on the shoring system chosen. At this time a preferred shoring option has not been chosen. As the project becomes more defined we can provide the necessary design information for the preferred shoring system. Each of the diaphragm walls are discussed in more detail below for preliminary design purposes.

Sheet Piles

Sheet pile walls consist of thin, prefabricated interlocking steel sheets that are driven, vibrated, or pushed to a specified embedment depth and are located along the wall alignment. The interlocking design on the sides allows the sheet piles to connect to each other to form a continuous wall. After excavation to specified elevations, tiebacks are installed, if necessary. Tiebacks will be required to be connected to walers in order to distribute the anchor loads over multiple sheet piles.

The sheet pile option has many potential advantages for the Block 37 project. These advantages include the ability to act as both the temporary and permanent below grade wall, measures can be taken to reduce seepage through the sheet pile interlock, the extent of groundwater drawdown off-site may be reduced, and the dewatering pumping rates may be reduced. Potential disadvantages include constructability concerns related to large wood debris, concrete rubble, and boulders; potential vibration and noise impacts if an impact hammer is used for installation; and achieving minimum embedment requirements in the very dense glacially consolidated soils.

The steel sheet piles are impermeable, but seepage can enter the excavation/building through the interlocking joints between adjacent sheets. This seepage can be reduced by using either bituminous or water swelling joint filler compounds. Alternatively, the seepage can be eliminated by welding the seams between adjacent sheets. Special detailing will be required to waterproof the interface between the vertical sheet piles and the horizontal mat foundation/structural slab.

If sheet piles are to be used for the Block 37 project, it is recommended that test pit explorations be completed to assess the extent and nature of wood debris and other potential obstructions. Additionally, a test pile installation program is recommended to assess the most efficient means of installation and to measure vibrations during installation to assess the impact to adjacent improvements.

Secant Pile Walls

Secant pile walls are formed by constructing intersecting reinforced concrete piles. This is accomplished by drilling and installing the primary piles first with the secondary piles constructed between the primary piles. The secondary piles are typically reinforced with wide flange steel beams. The advantages of secant pile walls are wall alignment flexibility, increased stiffness compared to sheet piles, the system can be installed in difficult ground, and reduced noise and vibrations during construction compared to driven or vibrated sheet piles.

The disadvantages of secant pile walls are vertical tolerances may be hard to achieve for deep piles and as a result, achieving an impermeable wall is difficult. The mix design of the lean concrete and structural concrete is an important consideration so that excavation to the face of the wide flange beam can be completed without difficulty while maintaining sufficient strength in the wall.

Cutter Soil Mix Walls

CSM walls are a type of soil mixing ground modification system that blends cementitious grout with the in-situ soil to form soil-cement elements, such as panels. Cutter soil mixing technology uses a vertically mounted, counter rotating cutter wheels. The wheels cut through the surrounding soil while blending and mixing the grout mixture with the in-situ soil forming cement panels. The panels can vary in width up to 4 feet and can be constructed to depths up to 130 feet. The CSM method uses sophisticated instrumentation to control different aspects of the system and insure that the panels are meeting specifications and tolerances.

The advantages of CSM are the instrumentation can accurately control the cutting tool, constructs panels instead of columns, which allows easier constructability for building walls, and can be used in a variety of soil types.

The disadvantages of CSM are it can be cost prohibitive when compared to conventional shoring and sheet piles, requires the use of an on-site batch plant for grout, there can be increased spoil disposal costs due to the cement content/pH of the spoils, and there is a significant amount of equipment associated with CSM production that a smaller site may not allow.

In-Situ Cast Reinforced Concrete Walls

Another type of diaphragm wall considered feasible for the Block 37 site is a cast in-situ reinforced concrete wall using a slurry supported trench, also known as a slurry wall. This type of wall is used frequently in the eastern United States. Slurry walls are advantageous where noise and vibration must be limited, geology and groundwater preclude the use of conventional shoring, and dewatering is costly or not practical. The construction process involves the following steps: pre-trenching to remove obstructions, construction of a guidewall at the ground surface, excavation of a vertical panel typically 20 to 24 feet long along the planned wall alignment to a specified depth, placement of an endstop for seepage control between adjacent panels, installation of a steel reinforcing cage, and tremie placement of structural concrete. This process is then repeated for the adjacent panels until the wall fully surrounds the planned excavation. Once the panel construction is complete, excavation commences and tiebacks are installed through pockets inserted into the reinforcing cages in the panels.

The primary advantage with the slurry wall technique is that the slurry wall combines into a single structural element the function of temporary shoring, permanent basement wall, reduced dewatering pumping rates, and vertical support of the future building. Disadvantages with this system is that it is a technique not commonly used in the Pacific Northwest and the initial cost of the system is high compared to conventional excavation support options.

Soil Freezing

Soil freezing is another ground improvement option that can be used to create an impermeable wall. Soil freezing can be implemented in two scenarios for a building project such as Block 37: (1) use soil freezing in combination with a conventional soldier pile and tieback wall with the soil freezing consisting of a line of vertical freeze pipes located some distance behind the wall (on the order of 5 to 10 feet) and (2) using soil freezing as the primary shoring system. The advantages and disadvantages of these two soil freezing options are discussed further below.

Ground freezing can create an impermeable barrier by freezing the available moisture in the ground. The frozen moisture acts similar to cement in concrete with the soil as the aggregate. Frozen soil is strong (can be up to ⅓ the strength of concrete, depending on soil type and temperature) and is essentially impermeable. Freeze pipes are typically spaced 3 to 5 feet center to center. There is a smaller plastic interior pipe that extends to the end of the steel pipe and a special head that connects to the top of the freeze pipe. A manifold system is then attached to each freeze pipe and then to a pump and chillers. The entire freeze system is checked for leaks prior to filling with brine. The system is then charged with calcium chloride brine, which is non-toxic salt water (the fluid that is sprayed on roads for de-icing and on dry roads for dust control). Brine is circulated evenly through each freeze pipe and chilled to -20 degrees Fahrenheit or colder as it passes through the chillers. The soil around each freeze pipe immediately begins to freeze when the system is turned on. A frozen soil zone slowly grows over time until a continuous frozen soil wall is formed. This usually takes 3 to 6 weeks, depending on soil type, pipe spacing, brine temperature and other factors.

Success of the soil freeze system is dependent upon the accuracy of the installation and the maintenance of the system during construction. If the vertical freeze pipes are installed out of alignment, gaps and weak points in the freeze wall may occur. Hoses that supply the brine to the freeze pipes must be protected during the excavation process and a staging area is required to house the chiller units. A constant source of electricity to power the chillers and the pumps is required to ensure the performance of the frozen wall.

Foundation Support

Recent deposits overlying the glacially consolidated soils are present at the anticipated foundation elevation across the site. Given the depth of the planned building below the groundwater table, the need for a structural mat designed to resist hydrostatic pressure, and the variable soil conditions at the foundation elevation, a structural mat foundation bearing on improved ground, where necessary, is the preferred foundation system.

Based on the data obtained from the borings completed at the site, GeoEngineers has developed a contour map estimating the top of the glacially consolidated soils. This contour map is presented in Figure 6. The glacially consolidated soils represent competent bearing and foundation elements bearing in these soils will have high capacities. The recent silt and clay soils represent a bearing layer with a reduced capacity, but likely still adequate for a structural mat foundation. The consistency of the recent deposits is variable across the site and ground improvement may be necessary to provide a consistent bearing across the site. Given that the glacially consolidated soils are present below the recent granular soils, ground improvement can be implemented to transfer the structural mat loading to the glacially consolidated soils.

For preliminary design purposes, an allowable bearing pressure of 4 to 6 ksf can be assumed. The allowable bearing is highly dependent on the foundation elevation, the type and extent of ground improvement and the settlement tolerances of the building and will be further evaluated during design. For preliminary design purposes, the use of ground improvement can be assumed for foundations bearing above approximate Elevation 0 feet.

Once the lowest finish floor elevations have been established for the project, the type/location of foundation elements should be reviewed by the project team. Additional explorations can be completed to reduce uncertainty with the extent of ground improvement required. More detail regarding recommended subgrade preparation and allowable bearing pressures for shallow foundations are presented below.

Allowable Bearing Pressure

Where foundations bearing directly on improved ground, stiff to hard recent silt and clay deposits, or on glacially consolidated soils, a preliminary allowable bearing pressure of 4 to 6 ksf can be assumed. The allowable soil bearing pressure applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. The allowable soil bearing pressures are net values.

Settlement

Provided that all loose soil is removed and that the subgrade is prepared as recommended under “Construction Considerations” below, we estimate that the total settlement of the structural mat foundations will be about 1 inch or less. The static settlements will occur rapidly, essentially as loads are applied. Differential settlements between footings could be half of the total settlement. Note that smaller settlements will result from lower applied loads.

Lateral Resistance

Given the planned deep excavation and structural mat foundation, lateral resistance of the planned building is anticipated to be high. GeoEngineers can provide design recommendations for lateral resistance upon request during the design phase of the project.

Ground Improvement

Ground improvement is recommended to provide uniform foundation support across the site, where necessary. Feasible ground improvement options include rigid inclusions, compaction grouting, soil mixed columns, and driven timber piles and would be completed within the recent granular and silt/clay soils. Each of these ground improvement systems would be completed on a grid pattern, where necessary, to transfer the foundation loading to the bearing soils. Stone columns or similar permeable ground improvement options are not recommended as this type of improvement will increase dewatering requirements. The type of ground improvement technique should be reviewed with the project team to identify constructability issues, provide a range of cost, and to establish the allowable bearing that can be achieved using the method selected. GeoEngineers can design the ground improvement system in collaboration with the general contractor and structural engineer.

Structural Slab

The lowest level of the planned building will extend below the groundwater table and permanent dewatering is not planned due to significant dewatering pumping rates, life cycle pumping costs, effluent discharge constraints, and groundwater treatment costs. As a result, the building should be designed to resist hydrostatic/uplift pressures.

Based on review of the site specific groundwater levels collected at the site and in the site vicinity since 2011, a design groundwater table of Elevation 20 feet is recommended for Block 37 for the design of the structural mat foundation.

A relief drain is recommended to be installed at the design groundwater elevation (Elevation 20 feet) and typically consists of a series of weepholes located along the permanent exterior below grade wall at a constant elevation. These weepholes are connected to a collector pipe and directed to a suitable discharge point. The benefit of the relief drain system is that it will limit the hydrostatic pressure that the building will need to be designed for and will reduce the risk to the building associated with unanticipated fluctuations in the groundwater table elevation.

The design groundwater elevation may be modified based on the structural aspects of the building and the location of floor levels. This may be desirable to keep the relief drain collection pipe from becoming damaged by vehicles in the below grade parking garage. The ideal location for the collector pipe is typically just below and elevated building diaphragm.

The structural slab should be designed to resist the hydrostatic uplift force. The uplift force acting on the proposed structure can be estimated by multiplying the volume of the structure located below the design groundwater elevation, in cubic feet, by the unit weight of water, 62.4 pounds per cubic foot (pcf). We assume that resistance to the uplift force will be provided by the weight of the structure. If necessary, tiedown anchors can be used to resist the hydrostatic uplift pressure acting on the structural mat foundation. Tiedown anchors for this application typically consist of small diameter vertical anchors constructed similar to a soil nail. GeoEngineers can assist the project team with design recommendations and capacities of tiedown anchors, should these elements be necessary.

Permanent below-grade walls that extend below the design groundwater table should be designed to resist hydrostatic pressures, as discussed in “Permanent Subsurface Walls” below.

Below-Grade Walls

Permanent Subsurface Walls

Permanent subsurface walls should be designed using the earth pressure diagram presented in Figure 9. The static and seismic earth pressures presented in Figure 9 represent the best estimate of actual loads and do not include a factor of safety. Other surcharge loads, such as from foundations, construction equipment or construction staging areas, should be considered on a case-by-case basis.

As discussed in Structural Slab above, a relief drain system consisting of weep pipes located around the perimeter of the permanent below grade building wall at the design groundwater table elevation. The purpose of the weep pipes/drainage system is to allow for wall drainage in the event that groundwater levels rise above the design groundwater elevation over the life of the structure.

Other Cast-in-Place Walls

Conventional cast-in-place walls may be necessary for small retaining structures located on-site. The lateral soil pressures acting on conventional cast-in-place subsurface walls will depend on the nature, density and configuration of the soil behind the wall and the amount of lateral wall movement that can occur as backfill is placed.

For walls that are free to yield at the top at least 0.1 percent of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming that the walls are backfilled and drainage is provided as outlined in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 35 pcf (triangular distribution), and that non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). For seismic loading conditions, a rectangular earth pressure equal to 8H pounds per square foot (psf), where H is the height of the wall, should be added to the active/at-rest pressures. Other surcharge loading should be applied as appropriate. Lateral resistance for conventional cast-in-place walls can be provided by frictional resistance along the base of the wall and passive resistance in front of the wall in accordance with the “Lateral Resistance” discussion earlier in this report.

The above soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed in the paragraphs below.

Drainage

Drainage behind the permanent below-grade walls is typically provided using prefabricated drainage board attached to the temporary shoring walls. For the Block 37 project, the prefabricated drainage board should extend at least 5 feet below the design groundwater elevation. If a diaphragm type shoring system that will act as the permanent below grade wall and temporary shoring (for instance sheet piles or a slurry wall) is used, prefabricated drainage material is not necessary.

Weep pipes that extend through the permanent below-grade wall should be installed around the perimeter of the building at the design groundwater elevation. The weep pipes should have a minimum diameter of 2 inches. The weep pipes should be considered as a safety valve that is activated only when groundwater builds up to the weep pipe elevation. The weep pipes should be connected to a collector pipe and directed to a suitable discharge location. The weep pipes should be spaced approximately 20 feet on center or less.

Positive drainage should be provided behind cast-in-place retaining walls by placing a minimum 2-foot-wide zone of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.10, with the exception that the percent passing the U.S. No. 200 sieve is to be less than 3 percent. A perforated or slotted drainpipe should be placed near the base of the retaining wall to provide drainage. The drainpipe should be surrounded by a minimum of 6 inches of Mineral Aggregate Type 22 (¾-inch crushed gravel) or Type 5 (1-inch washed gravel), City of Seattle Standard Specifications 9-03.11 and 9-03.12(6), respectively, or an alternative approved by GeoEngineers. The Type 22 or Type 5 material should be wrapped with a geotextile filter fabric meeting the requirements of construction geotextile for underground drainage, WSDOT Standard Specification 9-33. The wall drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger-diameter pipe will allow for easier maintenance of drainage systems.

Earthwork

Structural Fill

Fill placed to support structures, placed behind retaining structures, and placed below pavements and sidewalks will need to be specified as structural fill as described below:

- Structural fill placed behind cast in place retaining walls should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.10.
- Structural fill placed around cast-in-place wall drains should meet the requirements of Mineral Aggregate Type 5 (1-inch washed gravel), or Type 22 (¾-inch crushed gravel), City of Seattle Standard Specification 9-03.11.
- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of Mineral Aggregate Type 17 (bank run gravel), City of Seattle Standard Specification 9-03.10.
- Structural fill placed as crushed surfacing base course below pavements and sidewalks should meet the requirements of Mineral Aggregate Type 2 (1¼-inch minus crushed rock), City of Seattle Standard Specification 9-03.9(3).

On-site Soils

The on-site soils are moisture-sensitive and generally have natural moisture contents higher than the anticipated optimum moisture content for compaction. As a result, the on-site soils will likely require moisture conditioning in order to meet the required compaction criteria during dry weather conditions and will not be suitable for reuse during wet weather. Furthermore, most of the fill soils required for the project have specific gradation requirements, and the on-site soils do not meet these gradation requirements.

Therefore, imported structural fill meeting the requirements described above should be used where structural fill is necessary.

Fill Placement and Compaction Criteria

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

- Structural fill placed in building areas (around foundations or below slab-on-grade floors) and in pavement and sidewalk areas (including utility trench backfill) should be compacted to at least 95 percent of the maximum dry density (MDD) estimated in general accordance with ASTM International (ASTM) D 1557.
- Structural fill placed against subgrade walls should be compacted to between 90 and 92 percent. Care should be taken when compacting fill against subsurface walls to avoid overcompaction and hence overstressing the walls.

We recommend that GeoEngineers be present during probing of the exposed subgrade soils in building and pavement areas, and during placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

Weather Considerations

During wet weather, some of the exposed soils could become muddy and unstable. If so affected, we recommend that:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- 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 to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

Temporary Slopes

Temporary slopes may be used around the site to facilitate early installation of shoring or in the transition between levels at the base of the excavation. We recommend that temporary slopes constructed in the fill and recent deposits be inclined at 1½H:1V (horizontal to vertical). Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs. For open cuts at the site, we recommend that:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut;

- Exposed soil along the slope be protected from surface erosion by 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 practicable;
- Erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practicable;
- Surface water be diverted away from the slope; and
- The general condition of the slopes be observed periodically by the geotechnical engineer to confirm adequate stability.

Because 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. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

Recommended Additional Geotechnical Services

GeoEngineers will complete a design-level geotechnical engineering evaluation for the project, which is anticipated to confirm or modify as appropriate the preliminary design recommendations presented in this report. GeoEngineers should also be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.

During construction, GeoEngineers should observe the installation of the shoring system, review/collect shoring monitoring data, evaluate the suitability of the foundation subgrades, observe installation of subsurface drainage measures, evaluate structural backfill, observe the condition of temporary cut slopes, 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 F, Report Limitations and Guidelines for Use.

LIMITATIONS

We have prepared this report for the exclusive use of City Investors XI, LLC and their authorized agents for the Block 37 – South Lake Union Development project in Seattle, 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

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FIGURES

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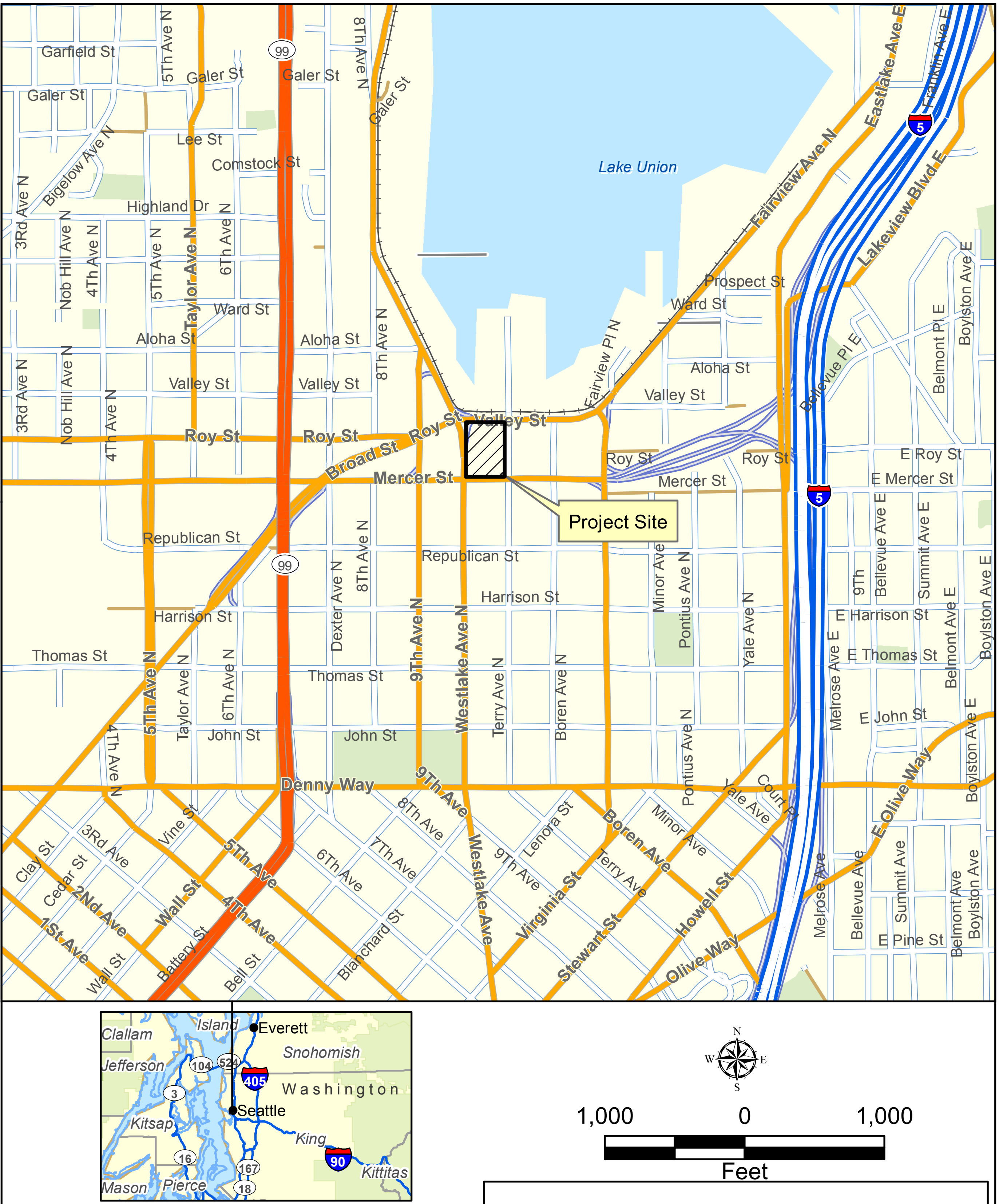

Notes:
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Data Sources: ESRI Data & Maps

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Block 37
Seattle, Washington

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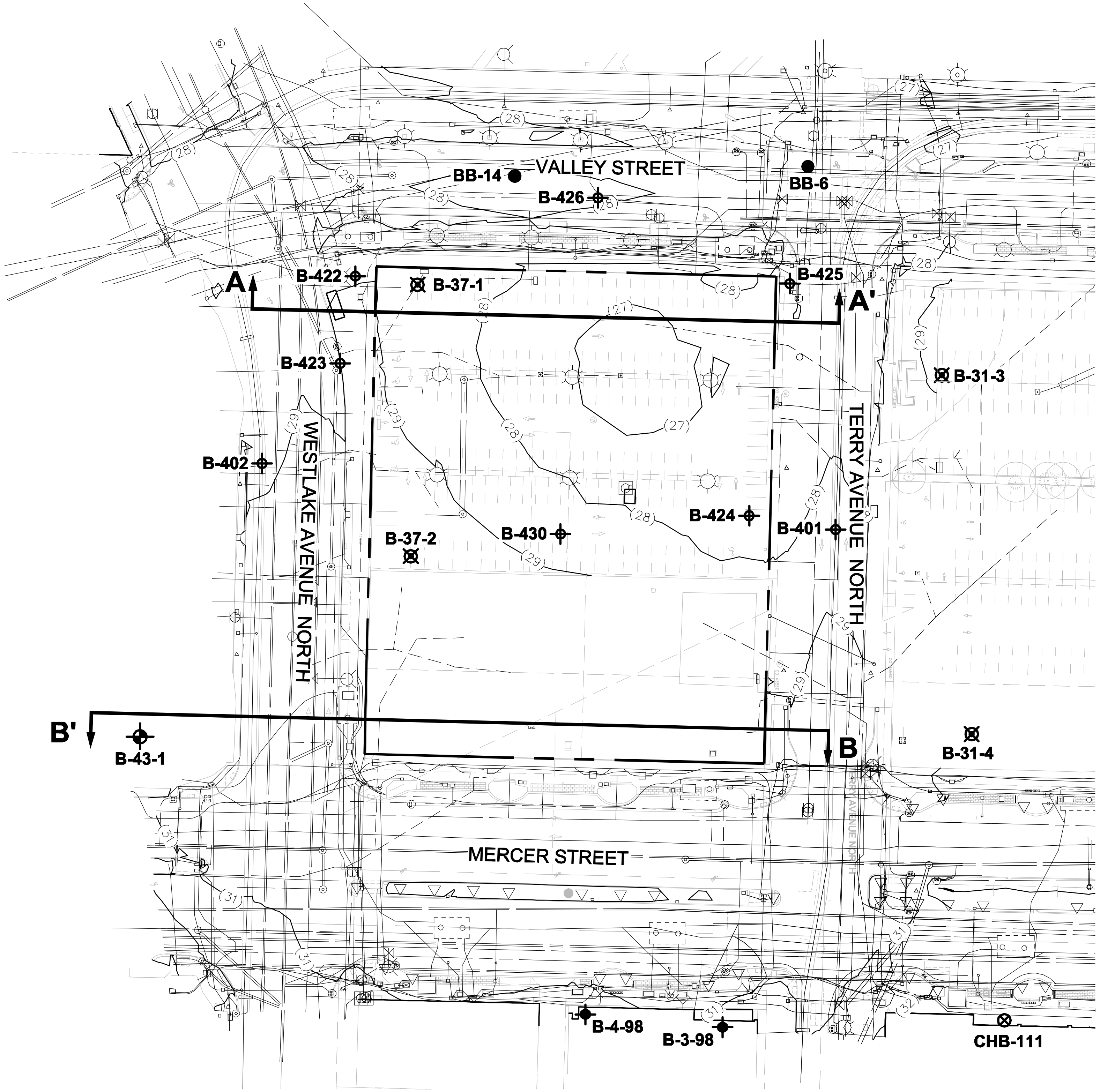
Figure 1

Path: \\red\projects\7\7087027\GIS\7087027_VicinityMapBlock37.mxd Map Revised: 26 June 2014 glohrmeyer



Legend

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CHB-111	⊗	Boring Completed by CH2MHILL (2008)
B-3-98	✦	Boring Completed by GeoEngineers (1998)
BB-6	●	Boring Completed by HWA GeoSciences (1998)
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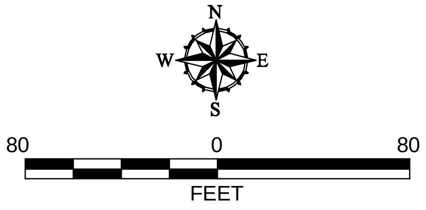

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Reference: Base topographic survey by Bush, Roed & Hitchings, Inc. dated 5/19/14.

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Site Plan

Block 37
Seattle, Washington

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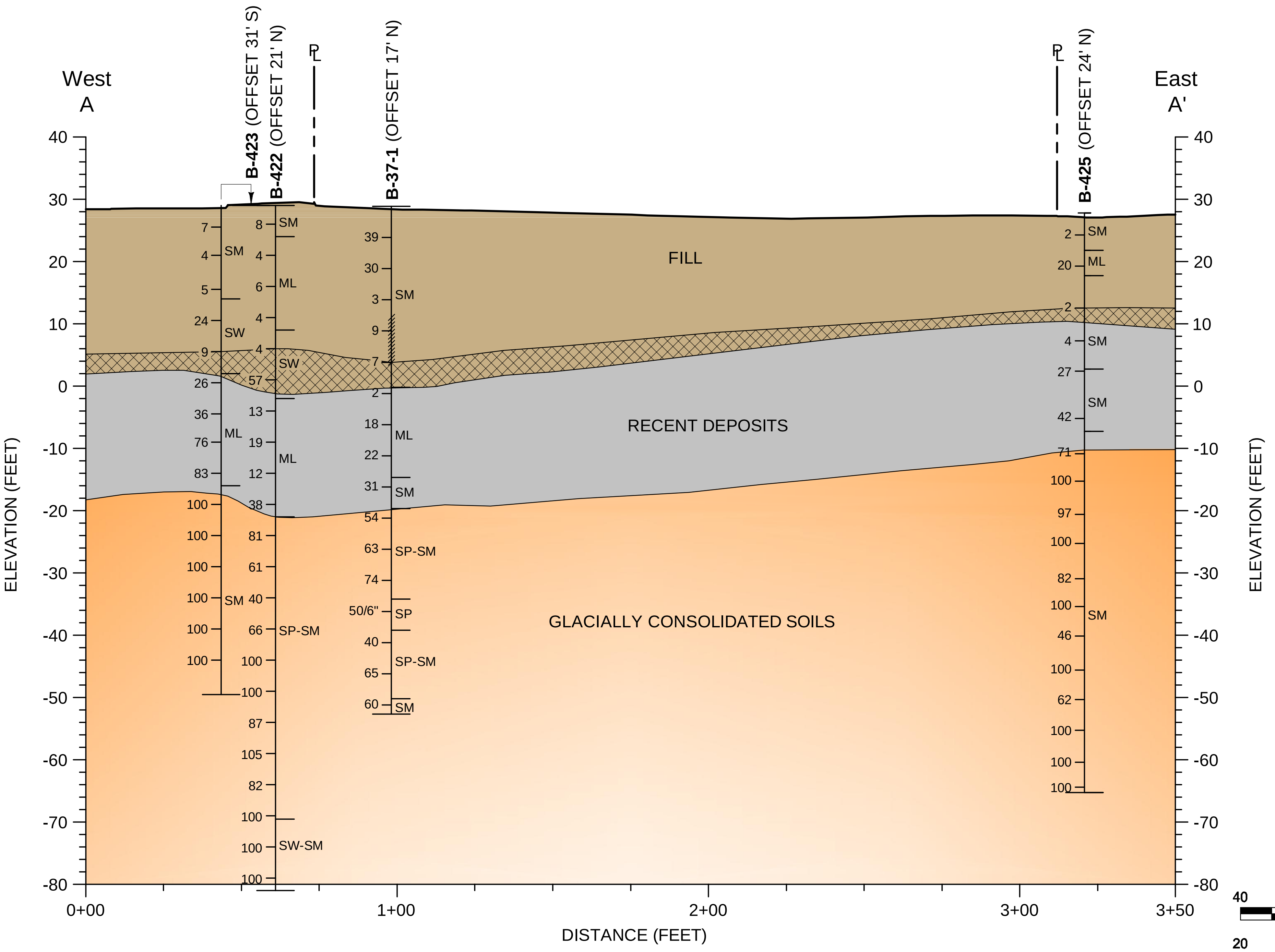
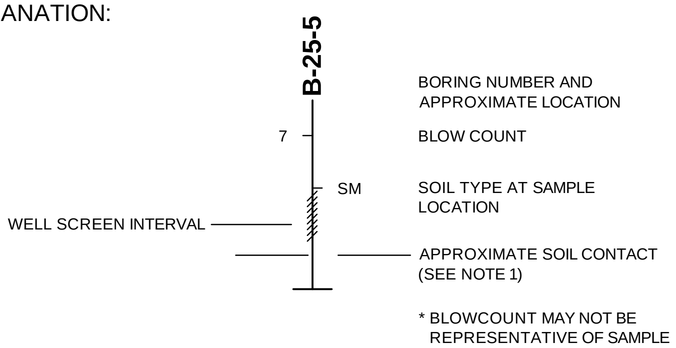
Figure 2



40 0 40
Horizontal Scale in Feet
20 0 20
Vertical Scale in Feet
Vertical Exaggeration: 2X

EXPLANATION:

▨	FILL
▨	RECENT DEPOSITS
▨	GLACIALLY CONSOLIDATED SOILS
▨	ZONE OF WOOD DEBRIS/ORGANIC SOILS



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Section A-A'

Block 37
Seattle, Washington

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Figure 3

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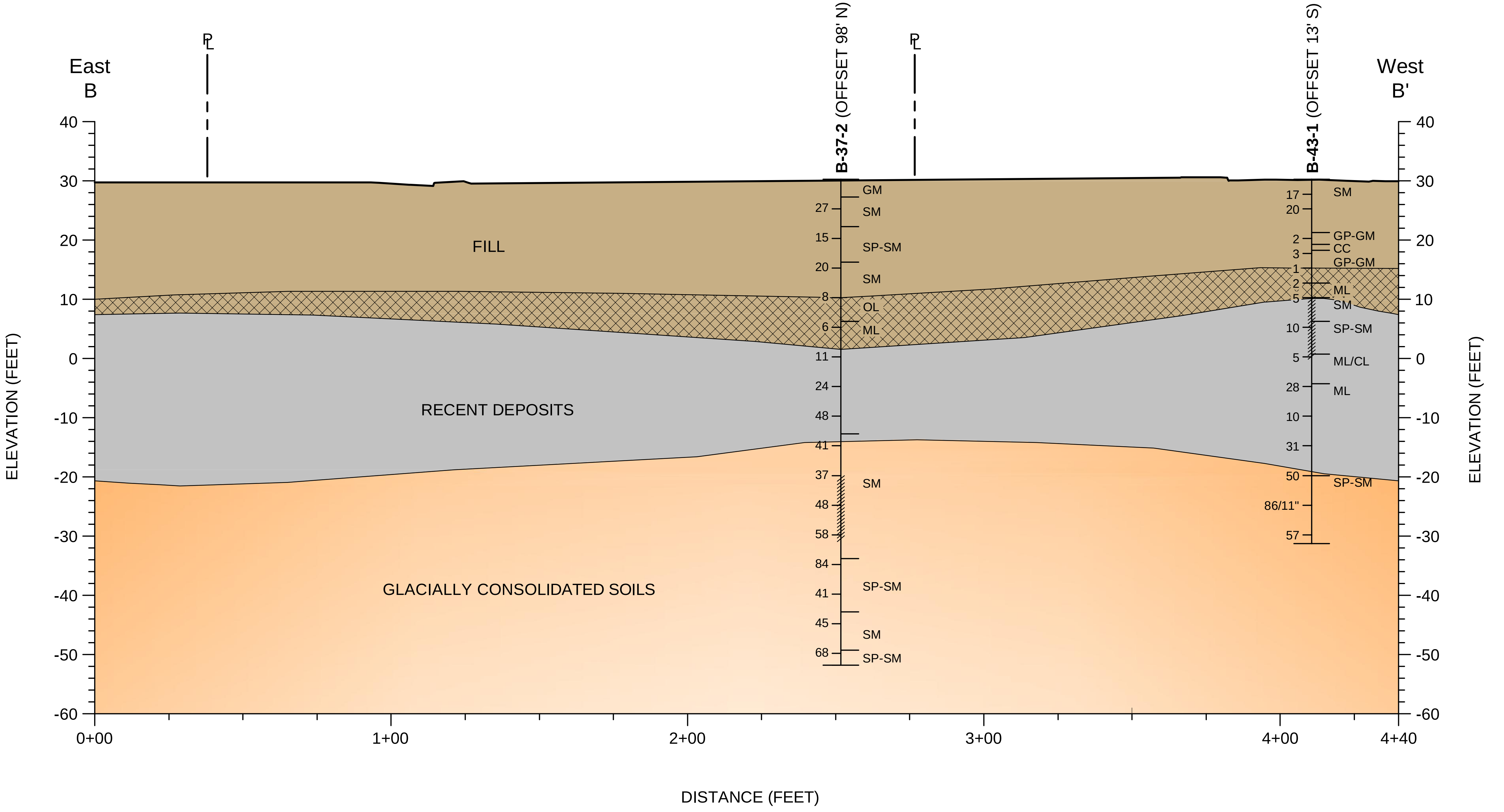
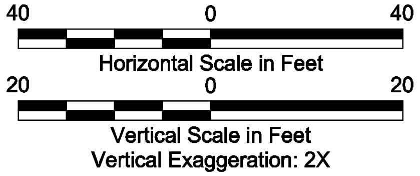




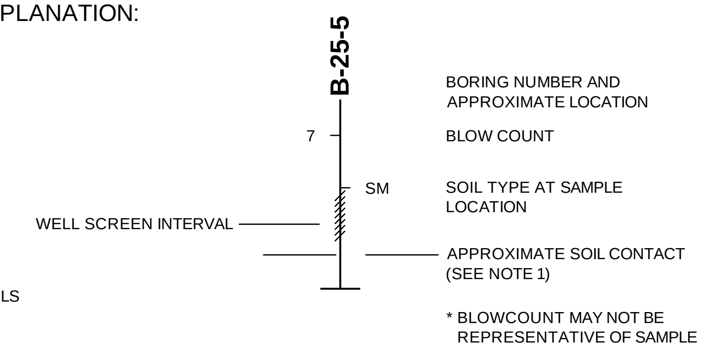
EXPLANATION:

🟫 FILL

⬜ RECENT DEPOSITS

🟧 GLACIALLY CONSOLIDATED SOILS

▨ ZONE OF WOOD DEBRIS/ORGANIC SOILS



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Section B-B'

Block 37
Seattle, Washington

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Figure 4

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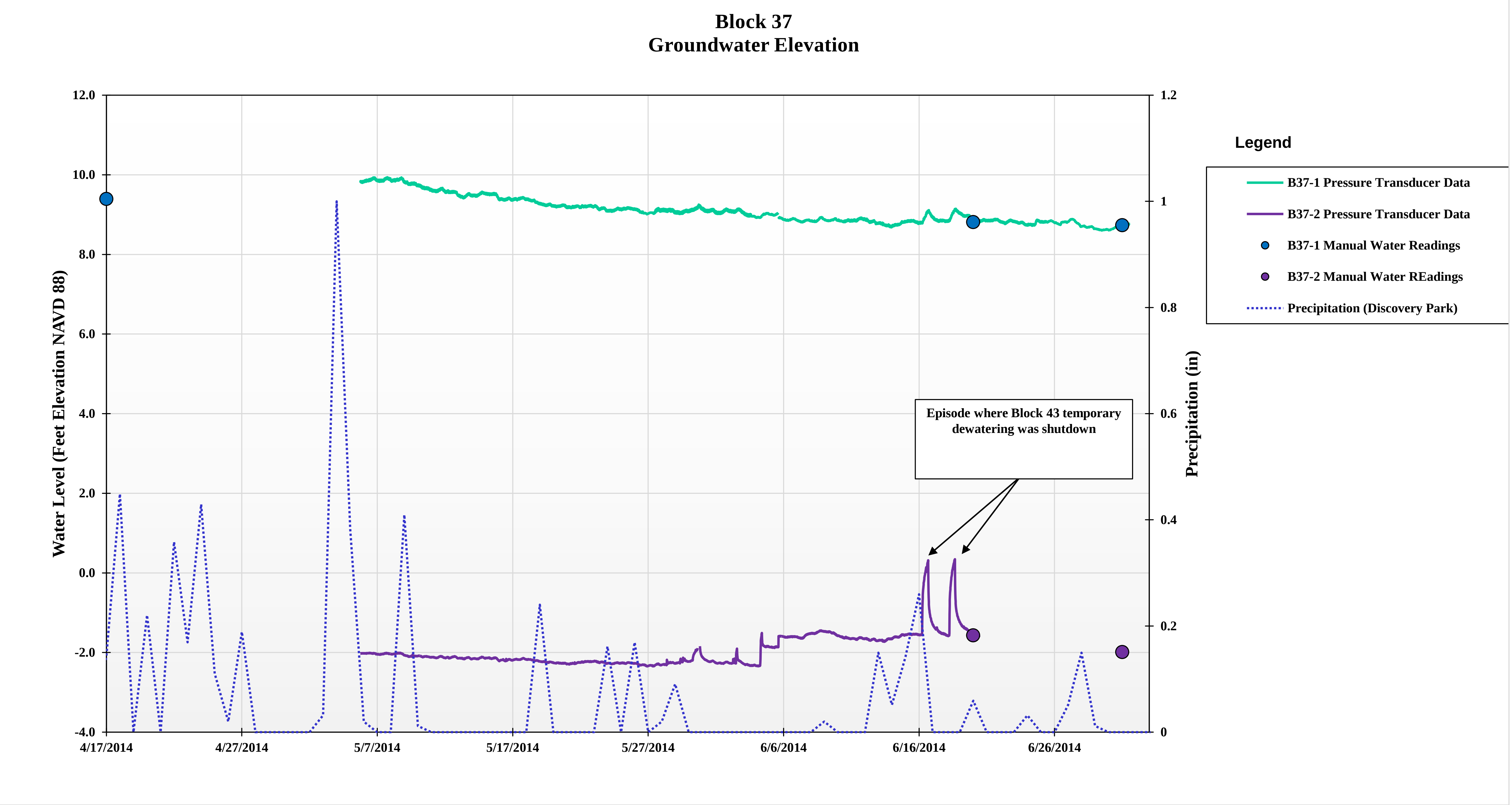

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Groundwater Elevation Data

Block 37
Seattle, Washington

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Figure 5

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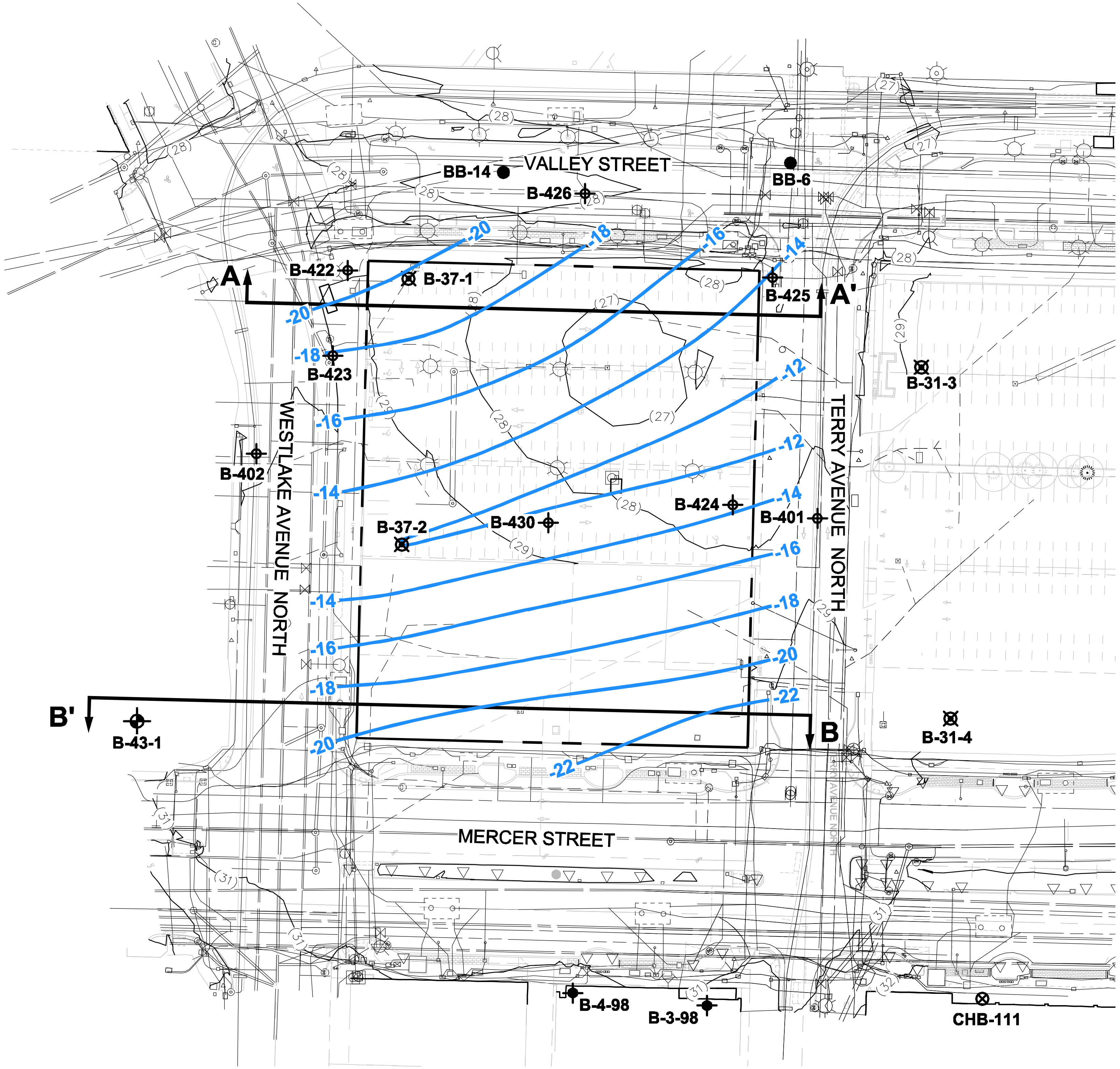

Legend

18 —— 18		Estimated Elevation Contour to Top of Glacially Consolidated Soils
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B-41-3	⊕	Boring Completed by GeoEngineers (2013)
CHB-111	⊗	Boring Completed by CH2MHILL (2008)
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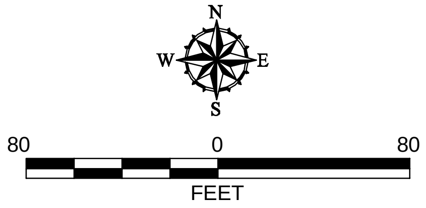

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Reference: Base topographic survey by Bush, Roed & Hitchings, Inc. dated 5/19/14.

Estimated Top Elevation of Glacially Consolidated Soils

Block 37
Seattle, Washington

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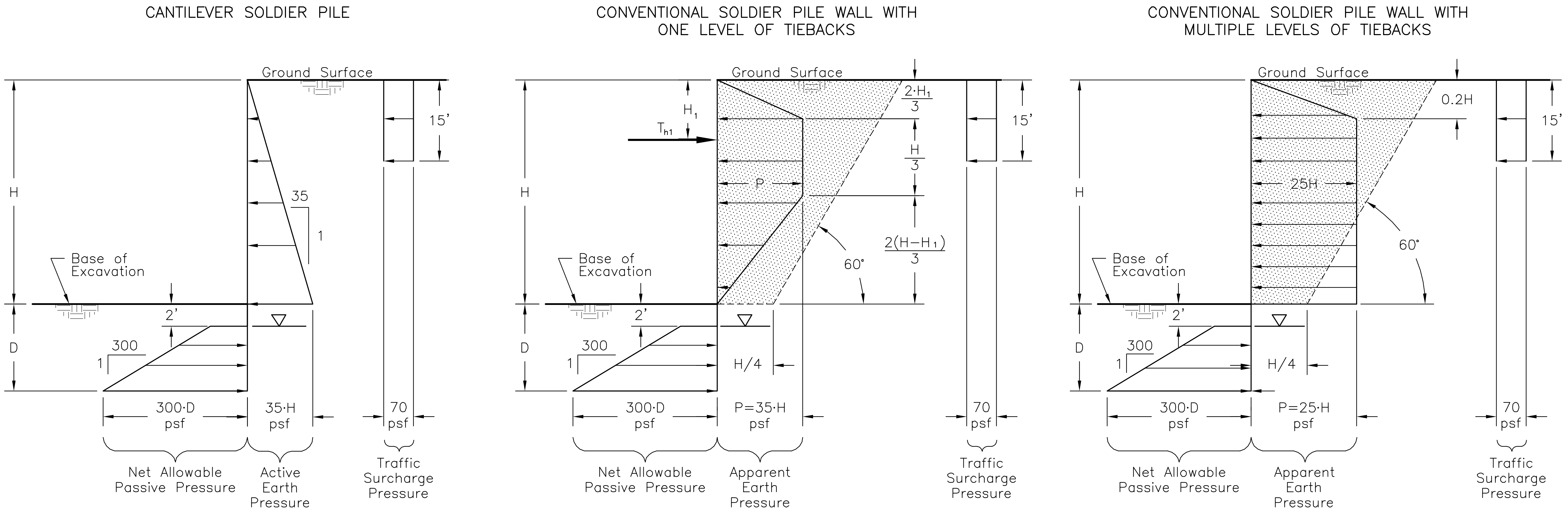
Figure 6

W:\Redmond\Projects\7\0787027\CAD\0787-027-00_BLK_37 Glacially Consolidated Elevs.DWG\TAB:Fig 3 modified on Aug 01, 2014 - 9:39am



NOT TO SCALE

Notes:

1. Apparent earth pressure and surcharge act over the pile spacing above the base of the excavation.
2. Passive earth pressure acts over 2.5 times the concreted diameter of the soldier pile, or the pile spacing, whichever is less.
3. Passive pressure includes a factor of safety of 1.5
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 10.
5. This pressure diagram is appropriate for temporary soldier pile and tieback walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Legend

▨ No Load Zone

H = Height of Excavation, Feet

D = Soldier Pile Embedment Depth, Feet

H_1 = Distance From Ground Surface to Uppermost Tieback, Feet

T_{h1} = Horizontal Load in Uppermost Ground Anchor

P = Maximum Apparent Earth Pressure Pounds per Square Foot

▽ Design Groundwater Elevation for Drained Walls/ Passive Resistance Design

Earth Pressure Diagrams - Permeable Walls
Temporary Soldier Pile & Tieback Wall

Block 37
Seattle, Washington

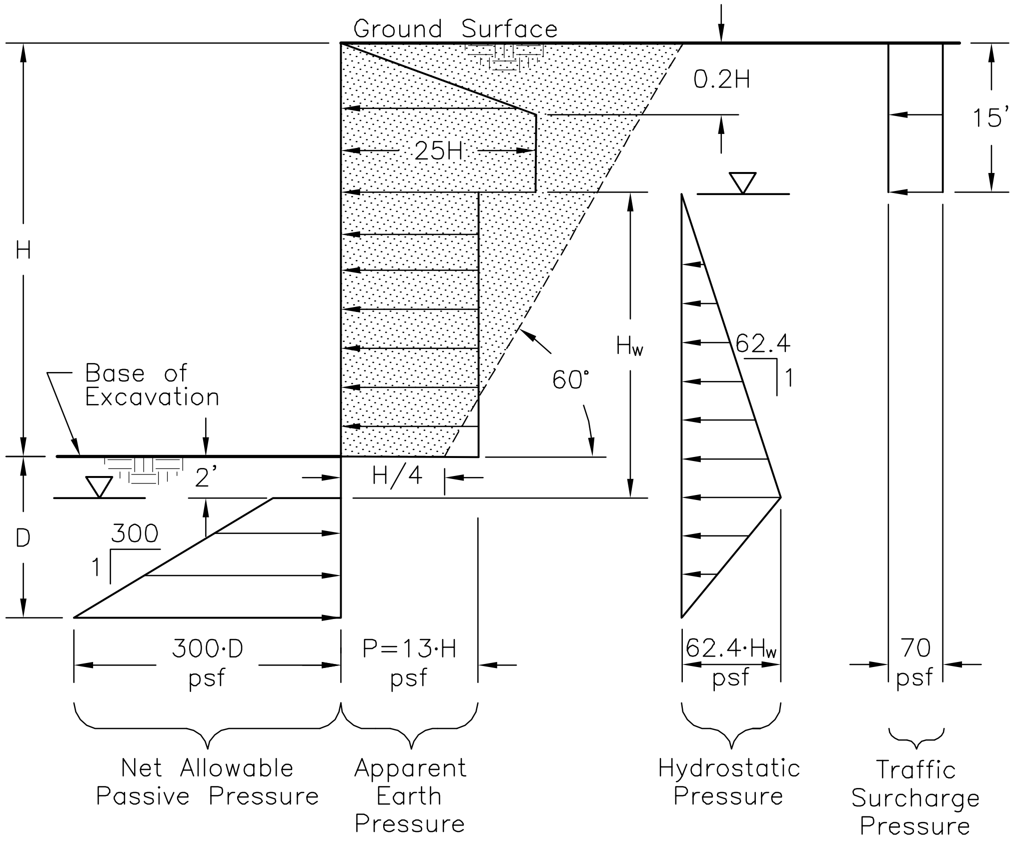
GEOENGINEERS

Figure 7

W:\REDMOND\PROJECTS\7\7087027\CAD\0787-027-00_BLK_37 EPDs.DWG\TAB:FIGURE 7 MODIFIED BY TMICHAUD ON JUL 16, 2014 - 13:52

W:\REDMOND\PROJECTS\7\7087027\CAD\0787-027-00_BLK_37 EPDS.DWG\TAB:FIGURE 8 MODIFIED BY TMICHAUD ON JUL 16, 2014 - 13:54

DIAPHRAGM TYPE SHORING WALL WITH MULTIPLE LEVELS OF TIEBACKS



NOT TO SCALE

Legend

▨ No Load Zone

H = Height of Excavation, Feet

D = Diaphragm Wall Embedment Depth, Feet

H_1 = Distance From Ground Surface to Uppermost Tieback, Feet

T_{h1} = Horizontal Load in Uppermost Ground Anchor

P = Maximum Apparent Earth Pressure Pounds per Square Foot

▽ Design Groundwater Elevation for Undrained Walls/ Passive Resistance Design

H_W = Design Height of Excavation Located Below Design Ground Water Table, feet

Notes:

1. Apparent earth pressure and surcharge act over the exposed face of wall.
2. Passive earth pressure acts over the embedded portion of the wall excluding the upper 2'.
3. Passive pressure includes a factor of safety of 1.5
4. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 10.
5. This pressure diagram is appropriate for temporary diaphragm walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.

Earth Pressure Diagrams -
Impermeable Walls
Temporary Diaphragm Walls with Tiebacks

Block 37
Seattle, Washington

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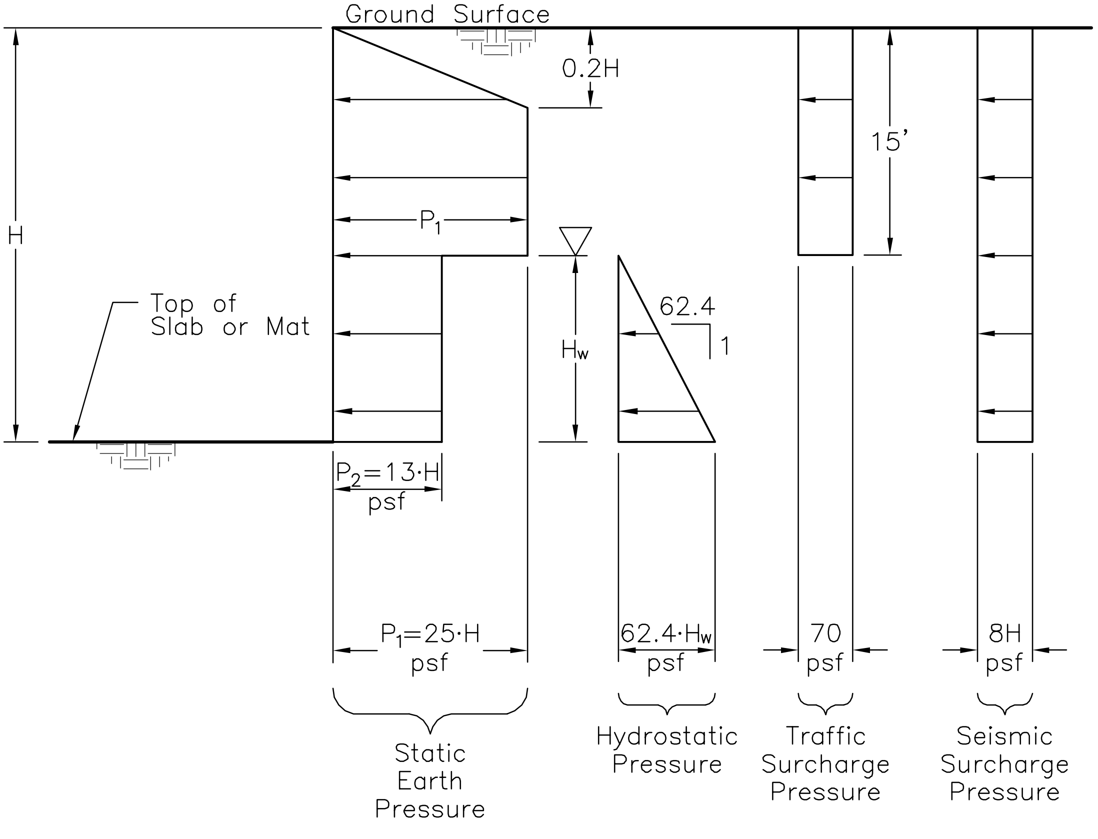
Figure 8

PERMANENT BASEMENT WALL DESIGN PRESSURES



NOT TO SCALE

Notes

1. This pressure diagram is appropriate for permanent basement walls. If additional surcharge loading (such as from soil stockpiles, excavators, dumptrucks, cranes, or concrete trucks) is anticipated, GeoEngineers should be consulted to provide revised surcharge pressures.
2. The static earth pressure does not include a factor of safety and represents the actual anticipated static earth pressure.

Legend

H = Height of Basement Wall, Feet
D = Foundation Embedment Depth, Feet
P_1, P_2 = Maximum Static Earth Pressure Pounds per Square Foot
H_w = Design Height of Excavation Located Below Design Ground Water Table, Feet
▽ = Design Ground Water Table at Elevation 21 Feet

Earth Pressure Diagram
Permanent Below Grade Walls

Block 37
Seattle, Washington

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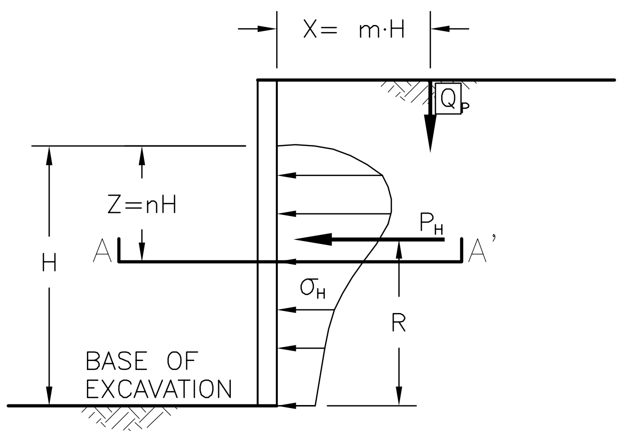
Figure 9

W:\REDMOND\PROJECTS\7\7087027\CAD\0787-027-00_BLK_37 EPDS.DWG\TAB:FIGURE 9 MODIFIED BY TMICHAUD ON JUL 16, 2014 - 13:56

LATERAL EARTH PRESSURE FROM POINT LOAD, Q_P (SPREAD FOOTING)



FOR m ≤ 0.4

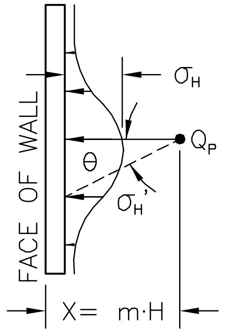
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2 (0.16+n^2)^3}$$

FOR m > 0.4

$$\sigma_H = \frac{1.77 Q_P m^2 n^2}{H^2 (m^2+n^2)^3}$$

$$\sigma_H' = \sigma_H \, COS^2 (1.1\theta)$$

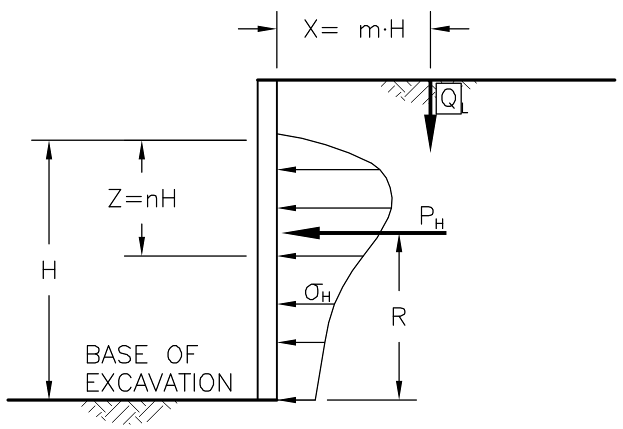
m	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



SECTION A–A'

Pressures from Point Load Q_P

LATERAL EARTH PRESSURE FROM LINE LOAD, Q_L (CONTINUOUS WALL FOOTING)



FOR m ≤ 0.4

$$\sigma_H = \frac{0.2n \cdot Q_L}{H(0.16+n^2)^2}$$

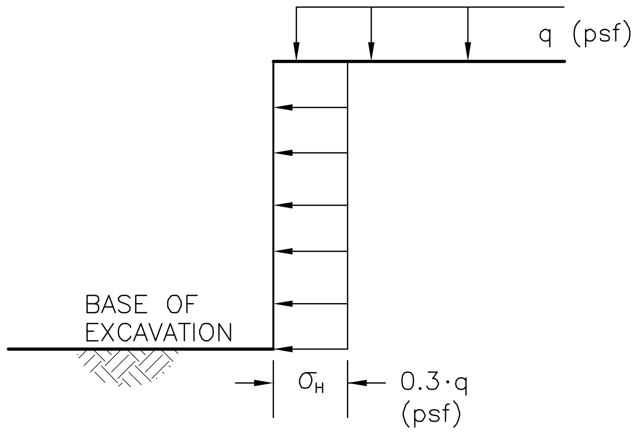
FOR m > 0.4

$$\sigma_H = \frac{1.28 m^2 \, n \cdot Q_L}{H(m^2+n^2)^2}$$

$$\text{RESULTANT } P_H = \frac{0.64 Q_L}{(m^2+1)}$$

m	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

UNIFORM SURCHARGES, q (FLOOR LOADS, LARGE FOUNDATION ELEMENTS OR TRAFFIC LOADS)



σ_H = LATERAL SURCHARGE PRESSURE FROM UNIFORM SURCHARGE

Definitions:

Q_P = Point load in pounds
Q_L = Line load in pounds/foot
H = Excavation height below footing, feet
σ_H = Lateral earth pressure from surcharge, psf
q = Surcharge pressure in psf
θ = Radians
σ_H' = Distribution of σ_H in plan view
P_H = Resultant lateral force acting on wall, pounds
R = Distance from base of excavation to resultant lateral force, feet

Notes:

1. Procedures for estimating surcharge pressures shown above are based on Manual 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures 7, 8, and 9.
3. See report text for where surcharge pressures are appropriate.

Recommended Surcharge Pressure

Block 37
Seattle, Washington

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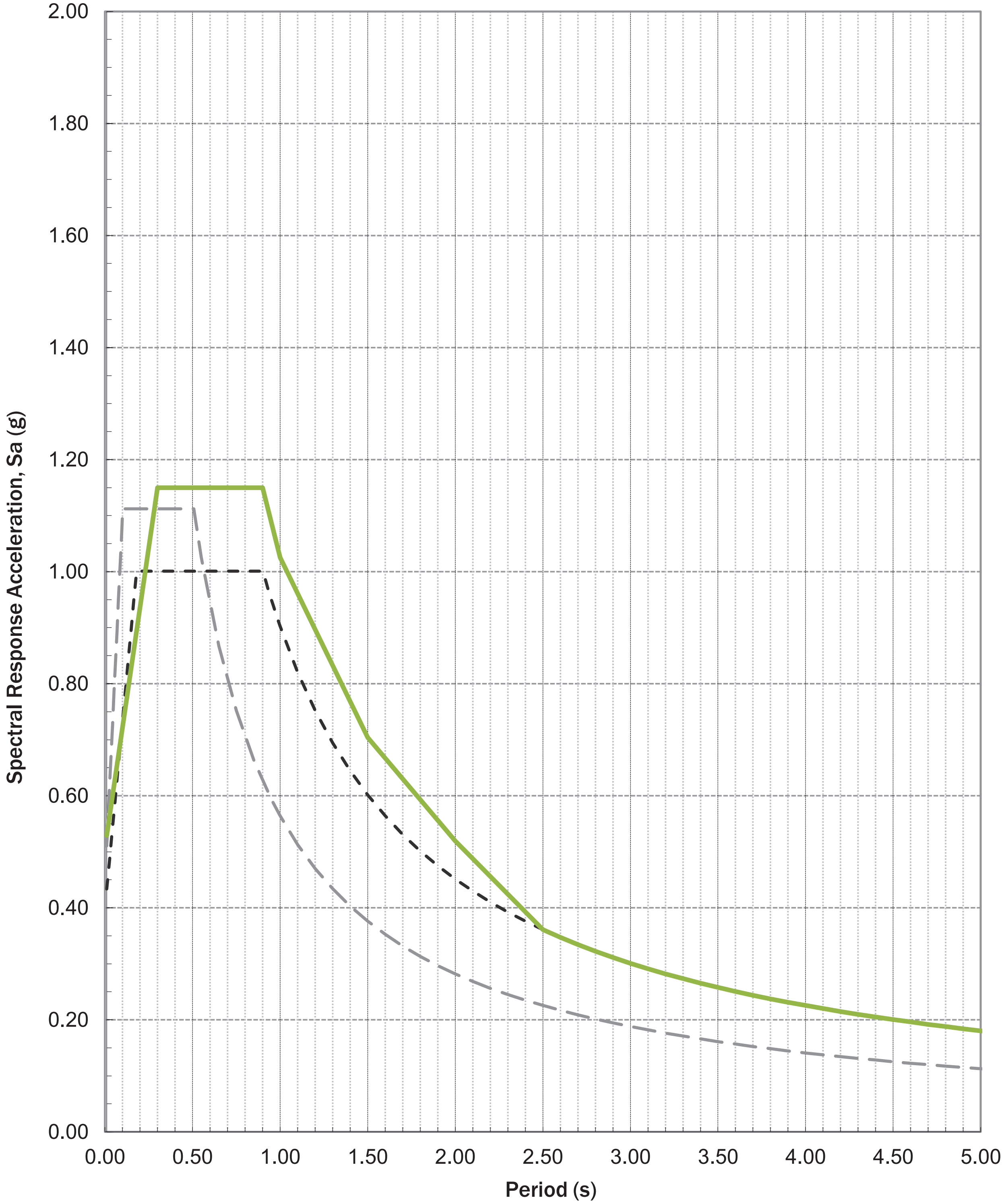
Figure 10

W:\REDMOND\PROJECTS\7\7087027\CAD\0787-027-00_BLK_37 EPDS.DWG\TAB:FIGURE 10 MODIFIED BY TMICHAUD ON JUL 16, 2014 - 13:57



- – – 0.8 x Site Class D Generalized Response Spectrum (ASCE 7-05)
- - - - 0.8 x Site Class E Generalized Response Spectrum (ASCE 7-05)
- ━ Recommended Site Specific MCE Response Spectrum

Recommended Site Specific MCE Response Spectrum (5% Damped)

Block 43 - South Lake Union Development
Seattle, Washington

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Figure 11

APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Subsurface conditions were explored at the site by drilling four borings (B-37-1, B-37-2, B-31-3 and B-31-4). The borings were completed to depths 81½ feet below the existing ground surface. The borings were completed by Geologic Drill Exploration Inc. between April 7 and 17, 2014.

The locations and elevations of the explorations were surveyed the project surveyor, Bush, Roed, and Hitchings (BRH). The approximate exploration locations are shown on the Site Plan, Figure 2.

Borings

Borings were completed using track- and trailer-mounted, continuous-flight, hollow-stem auger drilling equipment. The borings were continuously monitored by a geotechnical engineer or geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration.

The soils encountered in the borings were generally sampled at 2½- and 5-foot vertical intervals with a 2-inch outside diameter split-barrel standard penetration test (SPT) sampler. The disturbed samples were obtained by driving the sampler 18 inches into the soil with a 140-pound auto-hammer free-falling 30 inches. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions precluded driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings are presented in Figures A-2 to A-5. The boring logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change may actually be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented on the boring logs. Groundwater conditions observed during drilling represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

Monitoring Wells

A representative of GeoEngineers observed the installation of monitoring wells in both borings. The monitoring wells were constructed using 2-inch-diameter polyvinyl chloride (PVC) casing. The depth to which the casing was installed was selected based on our understanding of subsurface soil and groundwater conditions in the project area. The lower portion of the casing was slotted to allow entry of water into the casing. Medium sand was placed in the borehole annulus surrounding the slotted portion of the casing. A bentonite seal was placed above and below the slotted portion of the casing. The monitoring wells were protected by installing flush-mount steel monuments set in concrete. Completion details for the monitoring wells are shown on the logs presented in Figures A-2 through A-5.

Groundwater Measurements

Groundwater levels were measured on April 17, June 20, and July 1, 2014, in the monitoring wells installed at the site. Additionally, groundwater readings were taken continuously between April 28 to July 1, 2014 in the borings by means of automated dataloggers (refer to Figure 5 for a summary plot of measured groundwater levels).

Laboratory testing

Soil samples obtained from the explorations were transported to GeoEngineers' laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soil samples. Representative samples were selected for laboratory testing to determine the moisture content and percent fines (material passing the U.S. No. 200 sieve). The tests were performed in general accordance with test methods of ASTM International (ASTM) or other applicable procedures.

The results of the moisture content and percent fines determinations are presented at the respective sample depths on the exploration logs in Appendix A.

Moisture Content

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

Percent Passing U.S. No. 200 Sieve (%F)

Selected samples were "washed" through the U.S. No. 200 mesh sieve to estimate 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 estimate the fines content for analysis purposes. The tests were conducted in accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

SOIL CLASSIFICATION CHART

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

Groundwater Contact

▼	**Measured groundwater level in exploration, well, or piezometer**
▼	**Measured free product in well or piezometer**

Graphic Log Contact

—	**Distinct contact between soil strata or geologic units**
/	**Approximate location of soil strata change within a geologic soil unit**

Material Description Contact

—	**Distinct contact 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**
PI	**Plasticity index**
PP	**Pocket penetrometer**
PPM	**Parts per million**
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

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FIGURE A-1

Drilled	Start 4/16/2014	End 4/16/2014	Total Depth (ft) 81.5	Logged By TKC Checked By DPC	Driller Geologic Drill
Drilling Method	Hollow-Stem Auger				

Hammer Data	Pneumatic 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Turbo
Surface Elevation (ft) Vertical Datum	28.35 NAVD88	Top of Casing Elevation (ft)	27.95
Easting (X) Northing (Y)	1269362.7704 231828.1831	Horizontal Datum	NAD83

DOE Well I.D.: BIJ 461
A 2 (in) well was installed on 4/16/2014 to a depth of 36.75 (ft).

Groundwater

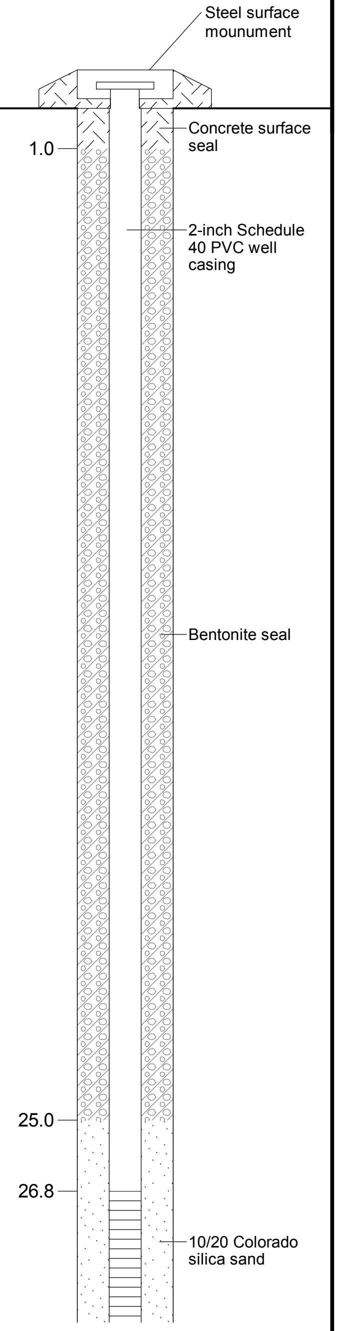
Date Measured	Depth to Water (ft)	Elevation (ft)
7/1/2014	19.2	8.7

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0							AC	3 inches asphalt concrete pavement		
								CR	8 inches base course		
								SM	Brown silty fine to medium sand with gravel and occasional cobbles (dense, moist) (fill)		
25											
	5								(Brick debris)		
20											
	10	■		30		3					
									(Easier drilling at 12 feet below ground surface)		
15											
	15	■	18	3		2			Grades to gravel		
10							▼				
	20	■	12	9		3					
									(Groundwater encountered at approximately 21 feet at the time of drilling)		
5											
	25	■	18	7		4			(Peat layer in sampler shoe)		
0								ML	Gray sandy silt (stiff to very stiff, wet) (recent deposits)		
	30										

WELL LOG



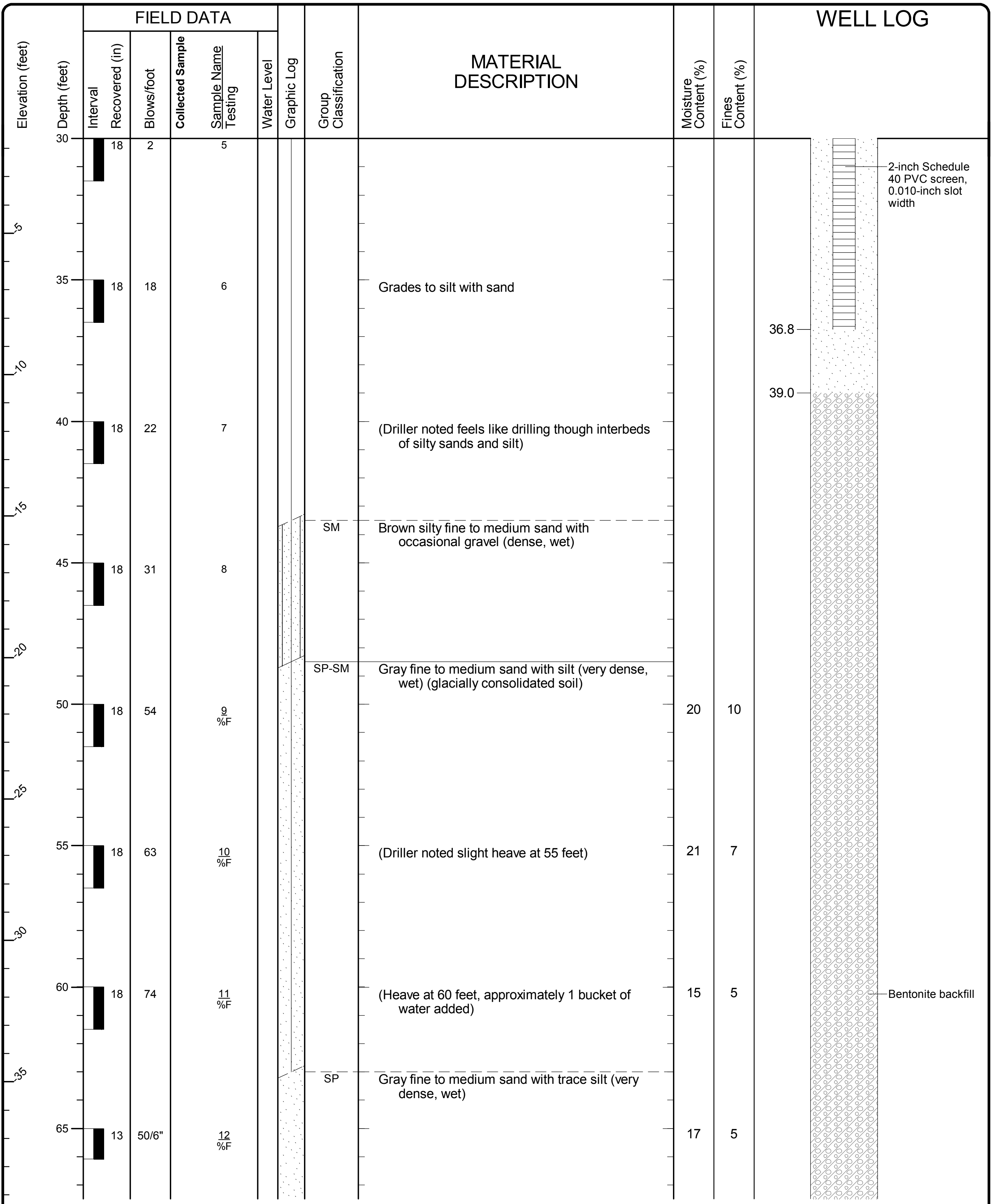
Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-37-1

GEOENGINEERS

Project: Block 37
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-2
Sheet 1 of 3



Depth (feet)	Recovered (in)	Blows/foot	Sample Name Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
30	18	2	5				
35	18	18	6		Grades to silt with sand		
40	18	22	7		(Driller noted feels like drilling though interbeds of silty sands and silt)		
				SM	Brown silty fine to medium sand with occasional gravel (dense, wet)		
45	18	31	8				
				SP-SM	Gray fine to medium sand with silt (very dense, wet) (glacially consolidated soil)		
50	18	54	9 %F			20	10
55	18	63	10 %F		(Driller noted slight heave at 55 feet)	21	7
60	18	74	11 %F		(Heave at 60 feet, approximately 1 bucket of water added)	15	5
				SP	Gray fine to medium sand with trace silt (very dense, wet)		
65	13	50/6"	12 %F			17	5

Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-37-1 (continued)

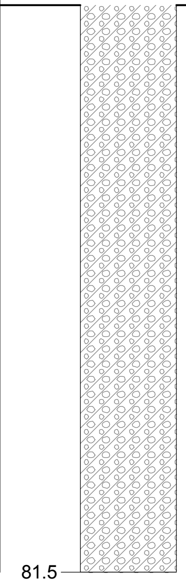
GEOENGINEERS

Project: Block 37
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-2
Sheet 2 of 3

		FIELD DATA									
Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
40								SP-SM	Gray fine to medium sand with silt (dense to very dense, wet)		
	70	18	40		13 %F					19	6
45											
	75	18	65		14						
50											
								SM	Gray silty fine to medium sand (very dense, wet)		
	80	18	60		15 %F					21	16

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-37-1 (continued)

GEOENGINEERS

Project: Block 37
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-2
Sheet 3 of 3

Drilled	Start 4/16/2014	End 4/17/2014	Total Depth (ft)	81.5
Logged By	TKC	Checked By	DPC	
Driller	Geologic Drill	Drilling Method	Hollow-Stem Auger	
Hammer Data	Pneumatic 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Turbo	
Surface Elevation (ft) Vertical Datum	29.6 NAVD88	Top of Casing Elevation (ft)	29.38	
Easting (X) Northing (Y)	1269358.7011 231666.0835	Horizontal Datum	NAD83	

DOE Well I.D.: BIJ 462
A 2 (in) well was installed on 4/16/2014 to a depth of 60.61 (ft).

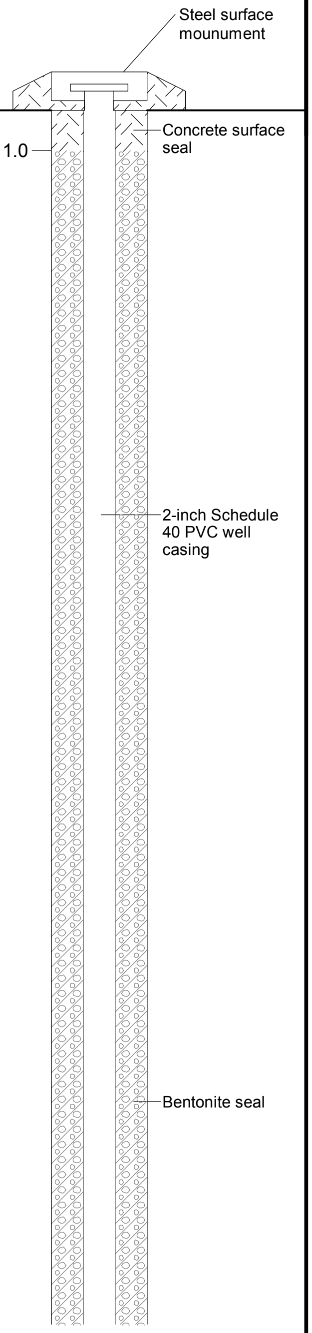
Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
7/1/2014	31.4	-2.0

Notes:

		FIELD DATA										
Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0								AC	3 inches asphalt concrete pavement		
									CR	8 inches base course		
		☒				1			GM	Brown silty fine to medium gravel with sand (moist) (fill)		
									SM	Brown silty fine to medium sand with gravel and occasional cobbles (medium dense, moist)		
25												
	5	■	12	27		2						
									SP-SM	Brown fine to medium sand with silt and occasional gravel (medium dense, moist)		
20												
	10	■	15	15		3						
									SM	Gray-brown silty fine to medium sand with occasional gravel (medium dense, moist) (with geogrid debris)		
15												
	15	■	12	20		4						
10												
	20	■	10	8		5			SM	Gray silty sand (medium dense, moist to wet) (with up to 2 feet wood debris)		
5												
									ML	Gray sandy silt or silt with sand and occasional gravel (medium stiff, wet) (with wood debris)		
	25	■	18	6		6						
									ML	Gray sandy silt (stiff, wet) (recent deposits) (with wood debris)		
0	30											

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-37-2

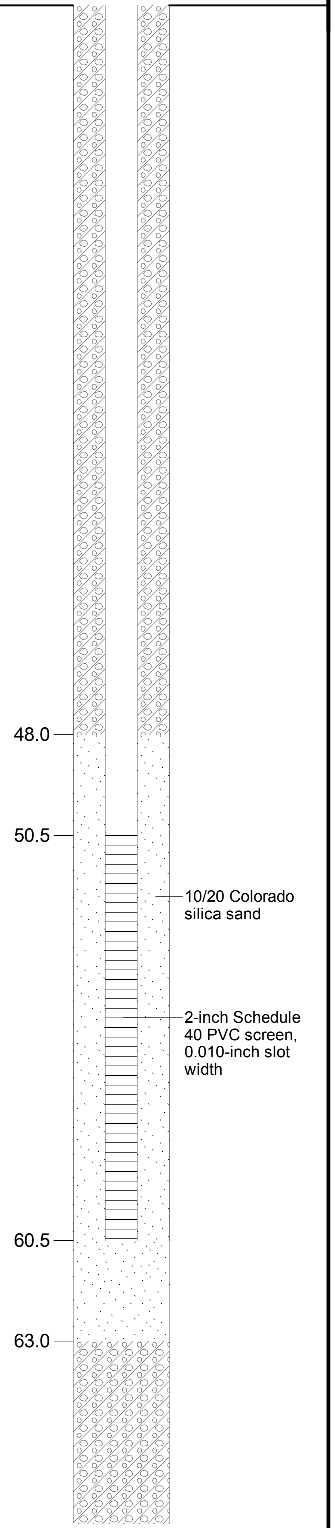
GEOENGINEERS

Project: Block 37
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-3
Sheet 1 of 3

Elevation (feet)	Depth (feet)	FIELD DATA Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	30		18	11		7 %F					26	62
							▼		ML	Gray sandy silt (medium stiff, wet)		
-5												
	35		12	24		8						
-10												
	40		18	48		9						
									SM	Gray silty fine to medium sand (dense, wet) (glacially consolidated soils)		
-15												
	45		15	41		10						
-20												
	50		15	37		11						
-25										(Driller added bucket of water at approximately 54 feet) (4 inch silt layer in the middle of sample, interbedded)		
	55		18	48		12						
-30												
	60		18	58		13 %F					18	13
									SM	Gray silty fine to medium sand (very dense, wet)		
									SP-SM	Gray fine to medium sand with silt (very dense, wet)		
-35												
	65		18	84		14						

WELL LOG



Notes: See Figure A-1 for explanation of symbols

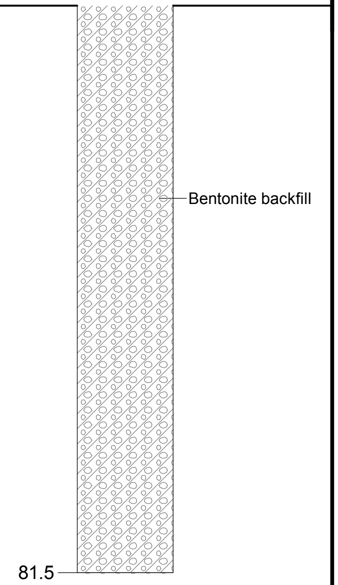
Log of Monitoring Well B-37-2 (continued)

GEOENGINEERS

Project: Block 37
Project Location: Seattle, Washington
Project Number: 7087-027-00

		FIELD DATA										
Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	**Collected Sample**	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
										(Driller added 2 buckets of water at approximately 68 feet)		
-40												
	70	■	18	41		15 %F					19	8
									SM	Gray silty fine to medium sand (dense, wet)		
-45												
	75	■	18	45		16 %F					19	35
-50												
									SP-SM	Gray fine to medium sand with silt and occasional gravel (very dense, wet)		
	80	■	18	68		17 %F					19	9

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Redmond: Date:7/31/14 Path:\\RED\PROJECTS\7\7087027\GINT\708702700.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_WELL

Log of Monitoring Well B-37-2 (continued)

GEOENGINEERS

Project:	Block 37
Project Location:	Seattle, Washington
Project Number:	7087-027-00

Figure A-3
Sheet 3 of 3

Drilled	Start 4/10/2014	End 4/10/2014	Total Depth (ft) 81	Logged By DTM Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger
Hammer Data	Pneumatic 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Turbo		
Surface Elevation (ft) Vertical Datum	28.7 NAVD88	Top of Casing Elevation (ft)	28.41		
Easting (X) Northing (Y)	1269675.6231 231774.012	Horizontal Datum	NAD83		

DOE Well I.D.: BIJ 460
A 2 (in) well was installed on 4/10/2014 to a depth of 30 (ft).

Groundwater

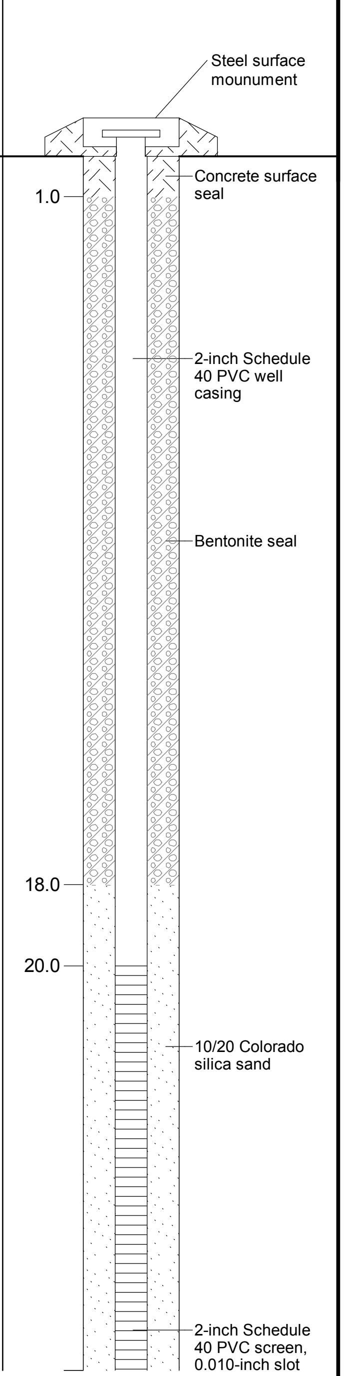
Date Measured	Depth to Water (ft)	Elevation (ft)
7/1/2014	13.0	15.4

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0							AC	4 inches asphalt concrete pavement		
								CR	5 inches base course		
								ML/CL	Gray silt/clay with fine sand and occasional gravel (soft, moist) (fill)		
25											
	5		12	56/10"		1			(Wood debris, blow counts may not be representative of soil)		
20								SM	Grayish brown silty fine to coarse sand with gravel (loose, moist to wet) (Construction debris; brick fragments)		
	10		6	2		2					
							▼				
15											
	15		10	2		3 MC			(Perched groundwater; constructions debris)		
10											
	20		12	6		4			(Wood waste)		
5								SM	Gray silty fine to medium sand with occasional gravel (medium dense, wet) (recent deposits)		
	25		18	25		5					
0											
	30										

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Redmond: Date:7/31/14 Path:\\RED\PROJECTS\7\7087027\GINT\708702700.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_WELL

Log of Monitoring Well B-31-3

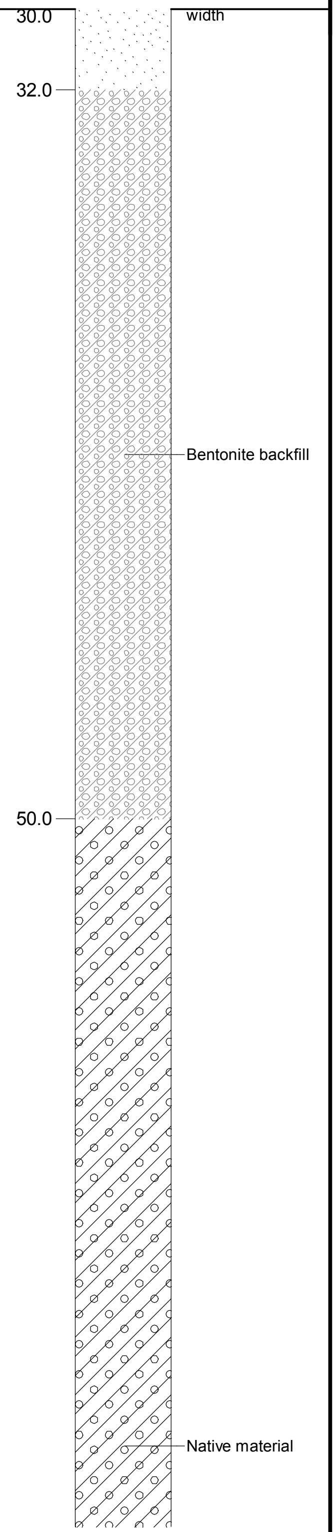
GEOENGINEERS

Project: Block 31
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-4
Sheet 1 of 3

		FIELD DATA									
Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	30		18	16		6 %F				(Adding mud at 30 feet)	26	79
									ML	Gray silt with sand (medium stiff to stiff, wet)		
									SM	Gray silty fine to medium sand (dense, moist?)		
-5												
	35		16	39		7				(Silt interbedding)		
-10												
									SP-SM	Gray fine to medium sand with silt (dense, wet) (glacially consolidated soil)		
	40		18	44		8 %F					18	8
-15												
									SM	Gray silty fine to medium sand (dense, wet)		
	45		18	41		9 %F				(Interbeds of silt with fine sand)	21	17
									ML	Gray sandy silt (hard, wet) (Driller notes firmed up at 48.5 feet)		
-20												
	50		14	52		10						
-25												
									SM	Grayish brown silty fine sand (dense, wet)		
	55		16	47		11						
									ML	Gray silt with sand (hard, wet)		
-30												
	60		18	67		12 %F					21	63
									SP-SM	Brownish-gray sand with silt (very dense, wet)		
-35												
	65		18	58		13 %F					19	10

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-31-3 (continued)

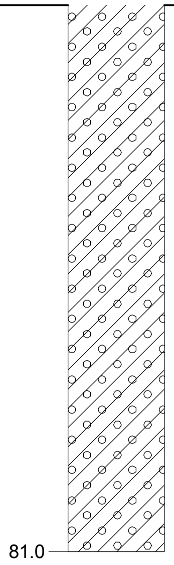
GEOENGINEERS

Project: Block 31
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-4
Sheet 2 of 3

Elevation (feet)	Depth (feet)	FIELD DATA Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
-40												
	70	■	18	50		14						
-45												
	75	■	6	50/6"		15						
-50												
	80	■	12	50/6"		16 %F					21	8

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-31-3 (continued)

GEOENGINEERS

Project: Block 31
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-4
Sheet 3 of 3

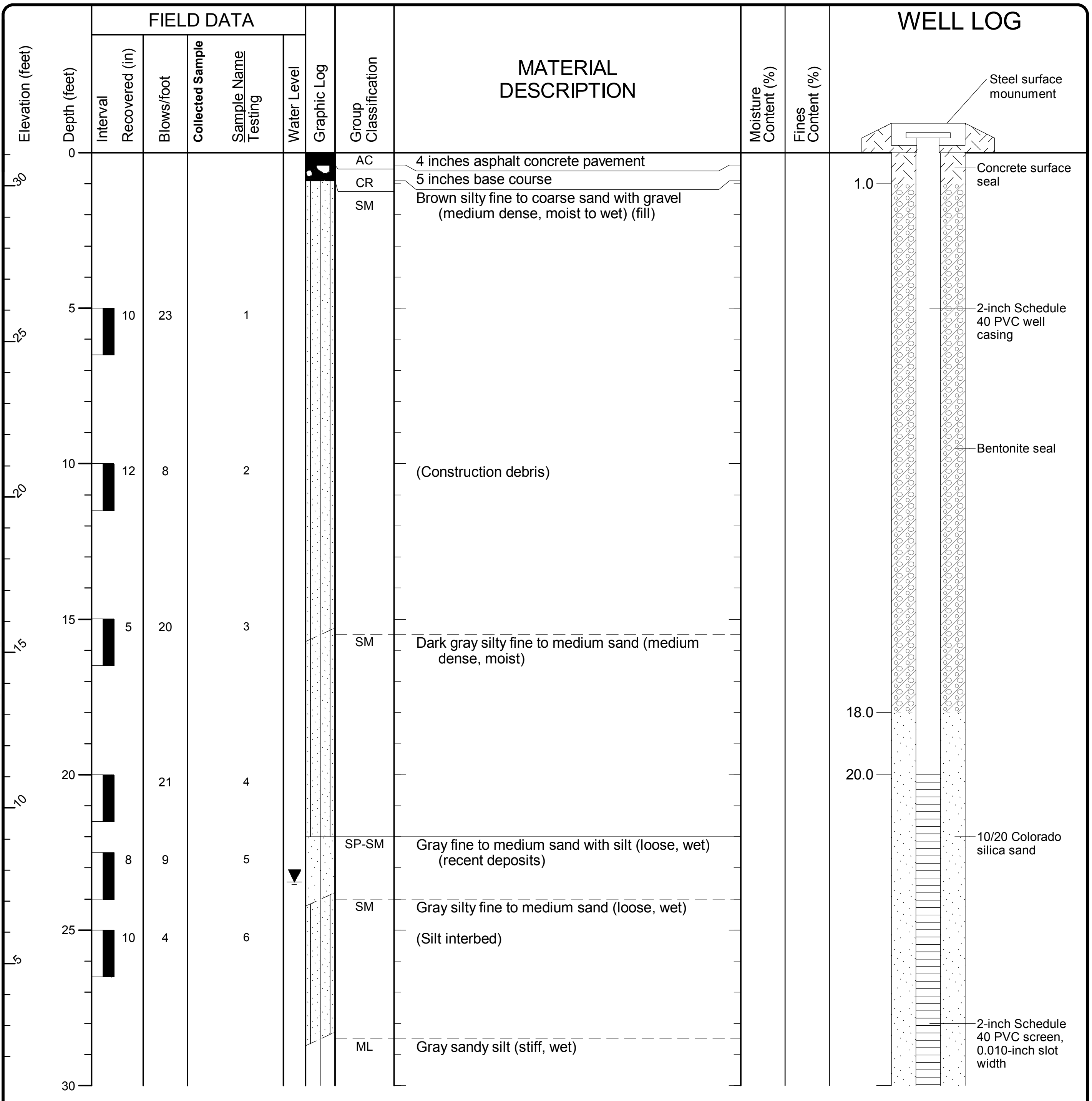
Drilled	Start 4/7/2014	End 4/9/2014	Total Depth (ft) 86.5	Logged By DTM Checked By DPC	Driller Geologic Drill	Drilling Method Hollow-Stem Auger

Hammer Data	Pneumatic 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D-50 Turbo	DOE Well I.D.: BIJ 459 A 2 (in) well was installed on 4/7/2014 to a depth of 30 (ft).
Surface Elevation (ft) Vertical Datum	31.07 NAVD88	Top of Casing Elevation (ft)	30.65	
Easting (X) Northing (Y)	1269693.3304 231559.8674	Horizontal Datum	NAD83	

Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
7/1/2014	23.5	7.2

Notes:



Depth (feet)	Recovered (in)	Blows/foot	Sample Name
5	10	23	1
10	12	8	2
15	5	20	3
20		21	4
	8	9	5
25	10	4	6

Group Classification	MATERIAL DESCRIPTION
AC	4 inches asphalt concrete pavement
CR	5 inches base course
SM	Brown silty fine to coarse sand with gravel (medium dense, moist to wet) (fill)
	(Construction debris)
SM	Dark gray silty fine to medium sand (medium dense, moist)
SP-SM	Gray fine to medium sand with silt (loose, wet) (recent deposits)
SM	Gray silty fine to medium sand (loose, wet)
	(Silt interbed)
ML	Gray sandy silt (stiff, wet)

Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-31-4

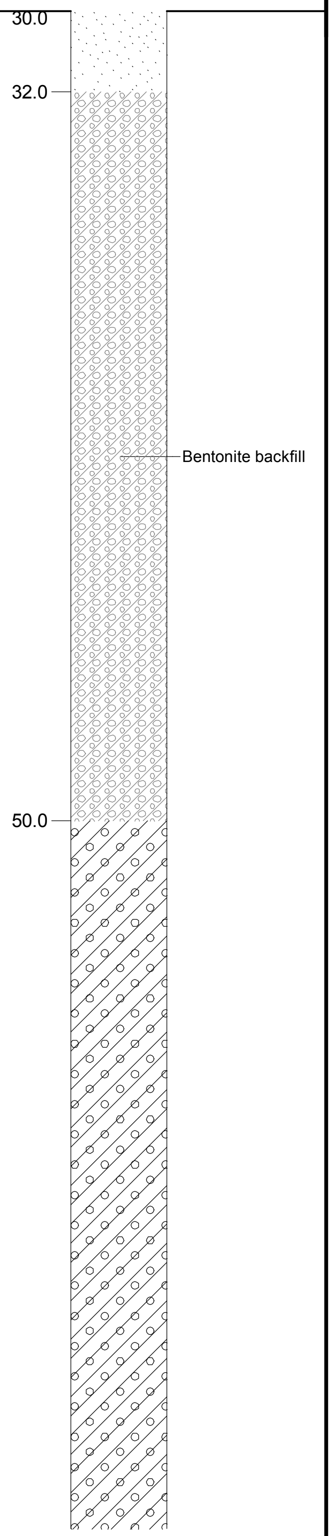
GEOENGINEERS

Project: Block 31
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-5
Sheet 1 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	30	■	18	20		7 %F					26	72
0												
										Becomes stiff to hard		
	35	■	18	21		8 %F					24	64
-5												
	40	■	18	39		9 %F					21	51
-10												
	45	■	18	34		10				(Adding mud at 45 feet)		
-15												
	50	■	18	24		11 %F				Grades to sand with silt	21	79
-20												
									SM	Gray silty fine sand (dense, wet) (glacially consolidated soils)		
	55	■	15	40		12 %F					19	13
-25												
									SP-SM	Light brown fine to medium sand with silt (dense,wet)		
	60	■		36		13 %F					19	8
-30												
	65	■		31		14						
-35												

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Redmond: Date:7/31/14 Path:\\RED\PROJECTS\7\7087027\GINT\708702700.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_WELL

Log of Monitoring Well B-31-4 (continued)

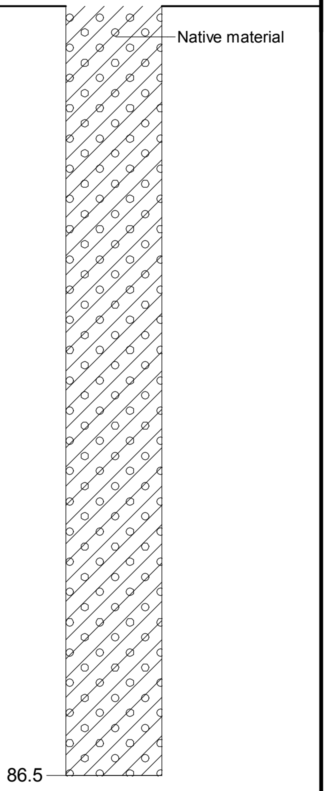
GEOENGINEERS

Project: Block 31
Project Location: Seattle, Washington
Project Number: 7087-027-00

Figure A-5
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	70	▮		32		15						
-40												
	75	▮		42		16						
-45												
										(Gravel at 77 feet)		
	80	▮	18	93/11.5"		17						
-50												
										(Cobbles at 81.5 feet, hard drilling)		
	85	▮		89		18						
-55												

WELL LOG



Notes: See Figure A-1 for explanation of symbols

Log of Monitoring Well B-31-4 (continued)

Project:	Block 31
Project Location:	Seattle, Washington
Project Number:	7087-027-00

Figure A-5
Sheet 3 of 3

APPENDIX B

Boring Logs from Previous Studies

APPENDIX B
BORING LOGS FROM PREVIOUS STUDIES

Included in this section are logs from the following previous studies completed in the immediate vicinity of the project site.

- The log for one boring (B-43-1) completed by GeoEngineers in 2013.
- The log for one boring (CH-111) completed by CH2MHILL in 2008.
- The logs of two borings (B-3-98 and B-4-98) completed by GeoEngineers in 1998.
- The logs of two borings (BB-6 and BB-14) completed by HWA GeoSciences in 1998.
- The logs of eight borings (B-401, B-402, B-422, B-423, B-424, B-425, B-426, and B-430) completed by Shannon and Wilson in 1970.

Drilled	Start 3/14/2012	End 3/14/2012	Total Depth (ft) 61.5	Logged By TML Checked By DPC	Driller Geologic Drill	Drilling Method HSA

Hammer Data	Manual 140 (lbs) / 30 (in) Drop	Drilling Equipment	XL Trailer Mounted
Surface Elevation (ft) Vertical Datum	31.8	Top of Casing Elevation (ft)	30.40
Easting (X) Northing (Y)	1222019.21 473730.19	Horizontal Datum	NAD83

A 2 (in) well was installed on 3/14/2012 to a depth of 30 (ft).

Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
3/19/2012	10.6	21.2

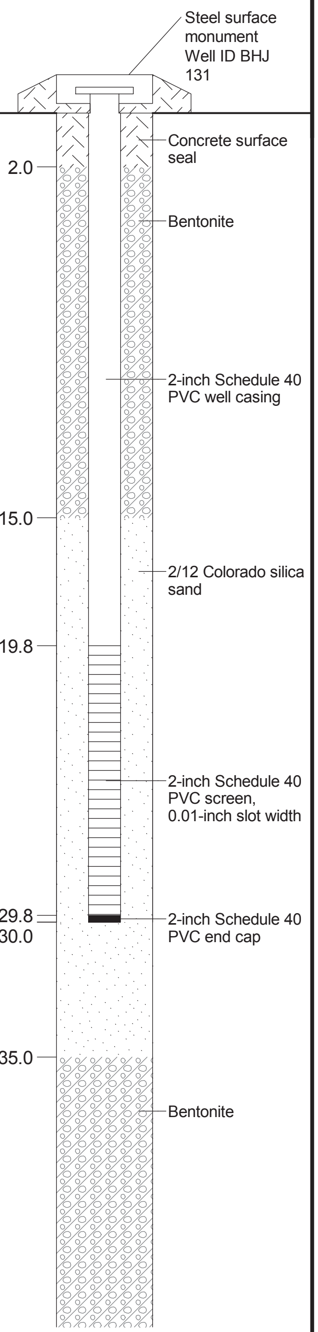
Notes: 4.25" I.D./4.75" O.D.

FIELD DATA

Depth (feet)	Recovered (in)	Blows/foot	Sample Name	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)
0					SM	Light brown silty fine to coarse sand with gravel and wood debris (medium dense, moist to wet)		
	3		1					
	6	17	2			Brick fragments		
					GP-GM	Gray fine to coarse gravel with silt and sand (very loose, wet)		
5	6	20	3			Sample is wet		
10	5	2	4	▼				
					CC	Concrete from 11 to 12 feet; brick fragments		
	4	3	5		GP-GM	Gray fine to coarse gravel with silt and sand (very loose, wet) (fill)		
15	4	1	6					
	5	2	7		ML	Gray silt with sand (soft, wet)		
20	18	5	8		SM	Gray silty fine to medium sand (loose, wet) (recent deposits)		
					SP-SM	Gray fine to coarse sand with silt (loose, wet)		
25	9	10	9					
30	14	5	10		ML/CL	Gray silt/clay with sand (medium stiff, wet)		
35	14	28	11 %F		ML	Gray silt with sand (stiff to very stiff, wet) %F = 74	23	
40	6	10	12 %F			Grades to with occasional sand %F = 89	28	
45								

Elevation (feet): 30, 25, 20, 15, 10, 5, 0, -5, -10

WELL LOG



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

Log of Monitoring Well B-43-1

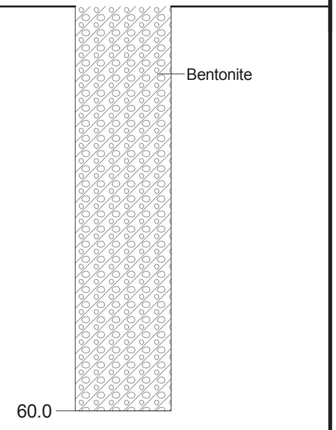
GEOENGINEERS

Project: Block 43 - South Lake Union Development
Project Location: Seattle, Washington
Project Number: 7087-017-00

Figure A-2
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)
	45		15	31		13 %F			ML	Grades to with sand %F = 71	19	
-15												
	50		7	50		14 %F			SP-SM	Gray fine to medium sand with silt (very dense, wet) (glacially consolidated soils) %F = 6	20	
-20												
	55		10	86/11"		15						
-25												
	60		18	57		16				Heave at 60 feet		

WELL LOG



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

Log of Monitoring Well B-43-1 (continued)

GEOENGINEERS

Project:	Block 43 - South Lake Union Development
Project Location:	Seattle, Washington
Project Number:	7087-017-00

Figure A-2
Sheet 2 of 2

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:	
314749.AA.P3.18.01	CHB-111	SHEET 1 OF 2

SOIL BORING LOG

PROJECT : Mercer Corridor Improvements

LOCATION : Mercer St, 125' East of Terry Ave N, (N 231434.4, E 1269717.9, WA North Zone, NAD 83/91)

ELEVATION : 32.7 feet (NAVD88)

DRILLING CONTRACTOR : Gregory Drilling, Inc., Redmond, WA

DRILLING METHOD AND EQUIPMENT : Hollow Stem Auger (HSA), 140-lb auto-hammer with 30-in drop, CME 85 truck-mounted rig

WATER LEVELS : See graph in Appendix C

START : 9/14/2008

END : 9/15/2008

LOGGER : M. Bouchedid

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"-6" (N)	SOIL DESCRIPTION **SOIL NAME (USCS GROUP SYMBOL),** COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					Surface is 4-in asphalt pavement overlying 6-in old concrete pavement.	Started drilling at 10:07.
5	5.0 7.0	1.3	SS-1	3-2-3-4 (5)	**SILTY SAND (SM),** brown, dry to slightly moist, loose, fine to coarse sand, predominantly fine sand, estimated 15 to 25% nonplastic fines, estimated 10% subrounded gravel.	SS-1 at 10:11
10	10.0 12.0	1.4	SS-2	1-2-1-2 (3)	**SILTY SAND (SM),** similar to above, except very loose, estimated 15% nonplastic fines, estimated less than 5% gravel, homogeneous.	SS-2 at 10:18
	12.5 14.5	1.5	SS-3	1-1-2-3 (3)	**0 to 10 in: SILTY SAND, SM,** similar to above, except gray, wet. (SS-3A) **10 to 18 in: SILTY SAND (SM),** brown and black, moist, very loose, fine to medium sand, predominantly fine sand, estimated 25 to 35% fines, decomposed wood chips throughout. (SS-3B)	SS-3 at 10:23 SS-3A Index Test Results Gravel = 0.3% P200 = 33.7% Sand = 66.0% Driller reports: water at 13 feet.
15	15.0 16.5	1.2	SS-4	2-3-50/2" (53/8")	**SILTY SAND (SM),** brown and black, moist to slightly wet, very dense, predominantly fine sand, estimated 20 to 30% nonplastic fines, trace fine subrounded gravel, decomposed wood chips.	SS-4 at 10:35 Driller reports: wood log at 16 feet, gravel at 16.5 feet. Performed percolation test at 16.5 feet from 11:12 to 11:36.
	17.5 19.0	0.8	SS-5	5-10-5 (15)	**POORLY-GRADED GRAVEL WITH SILT AND SAND, GP-GM,** brown, wet, medium dense, fine to coarse sand, nonplastic fines, fine subrounded gravel, fresh to decomposed wood chips in bottom 3 inches, bottom 2 inches is sandy silt.	SS-5 at 11:45 SS-5 Index Test Results Gravel = 51.4% P200 = 6.1% Sand = 42.5%
20	20.0 21.5	0.9	SS-6	1-1-2 (3)	**SILT AND PEAT,** dark brown, wet, soft, plastic silt, trace fine sand, peat and decomposed wood, 0.1-in gray sand lens at 20.8 feet.	SS-6 at 11:51 SS-6 Index Test Results M.C. = 191%
						Driller reports: harder at 23 feet.
25	25.0 26.5	1.3	SS-7	2-3-8 (11)	**LEAN CLAY (CL),** gray, moist, stiff, plastic fines, trace sand, bottom 1 inch is sand.	SS-7 at 11:56 PPh= 1.0; 1.25; 1.0 tsf PPv= 1.0; 1.0; 1.1 tsf
30						

CH2MHILL

PROJECT NUMBER: 314749.AA.P3.18.01

BORING NUMBER: CHB-111

SHEET 2 OF 2

SOIL BORING LOG

PROJECT : Mercer Corridor Improvements

LOCATION : Mercer St, 125' East of Terry Ave N, (N 231434.4, E 1269717.9, WA North Zone, NAD 83/91)

ELEVATION : 32.7 feet (NAVD88)

DRILLING CONTRACTOR : Gregory Drilling, Inc., Redmond, WA

DRILLING METHOD AND EQUIPMENT : Hollow Stem Auger (HSA), 140-lb auto-hammer with 30-in drop, CME 85 truck-mounted rig

WATER LEVELS : See graph in Appendix C

START : 9/14/2008

END : 9/15/2008

LOGGER : M. Bouchedid

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME (USCS GROUP SYMBOL), COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	30.0	1.2	SS-8	11-16-17 (33)	**SILTY SAND/POORLY-GRADED SAND WITH SILT (SM/SP-SM),** gray, wet, dense, fine to coarse sand, predominantly fine to medium sand, estimated 10 to 20% nonplastic fines, trace fine subrounded gravel, 0.1-inch silt lenses at 30.5 and 31.1 feet. Bottom of hole at 31.5 ft below ground surface.	SS-8 at 12:05 Well Installation 0-1.5 ft: Concrete monument 1.5-5 ft: Bentonite chips 5-17 ft: Colorado sand 17-30 ft: Bentonite chips 6-16 ft: 2-in factory slotted PVC screen Tag No. BAA 852
35						
40						
45						
50						
55						
60						

BORING B-3

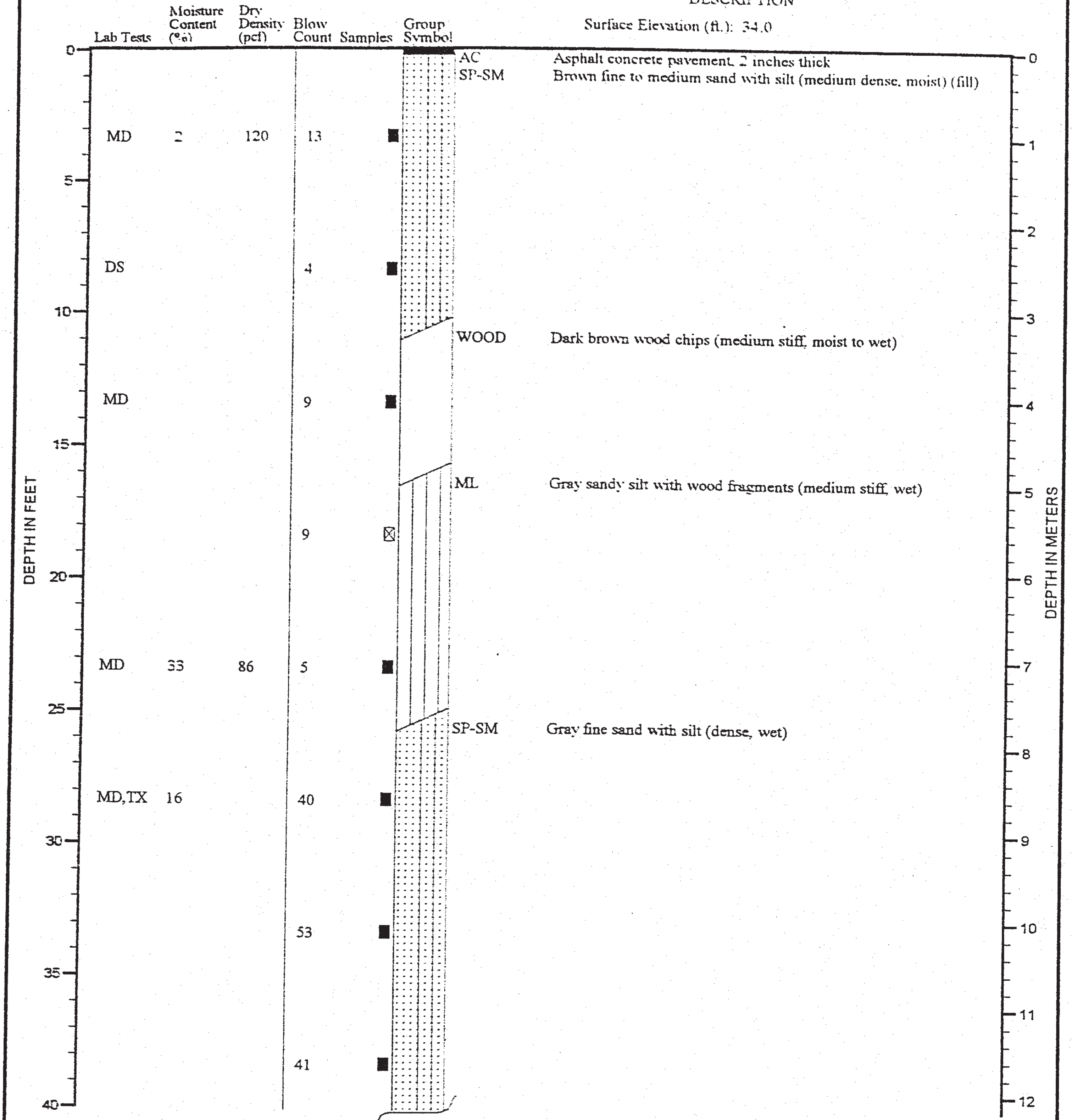
TEST DATA

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples
MD	2	120	13	■
DS			4	■
MD			9	■
			9	☒
MD	33	86	5	■
MD,TX	16		40	■
			53	■
			41	■

DESCRIPTION

Surface Elevation (ft.): 34.0

Group Symbol	
AC	Asphalt concrete pavement, 2 inches thick
SP-SM	Brown fine to medium sand with silt (medium dense, moist) (fill)
WOOD	Dark brown wood chips (medium stiff, moist to wet)
ML	Gray sandy silt with wood fragments (medium stiff, wet)
SP-SM	Gray fine sand with silt (dense, wet)



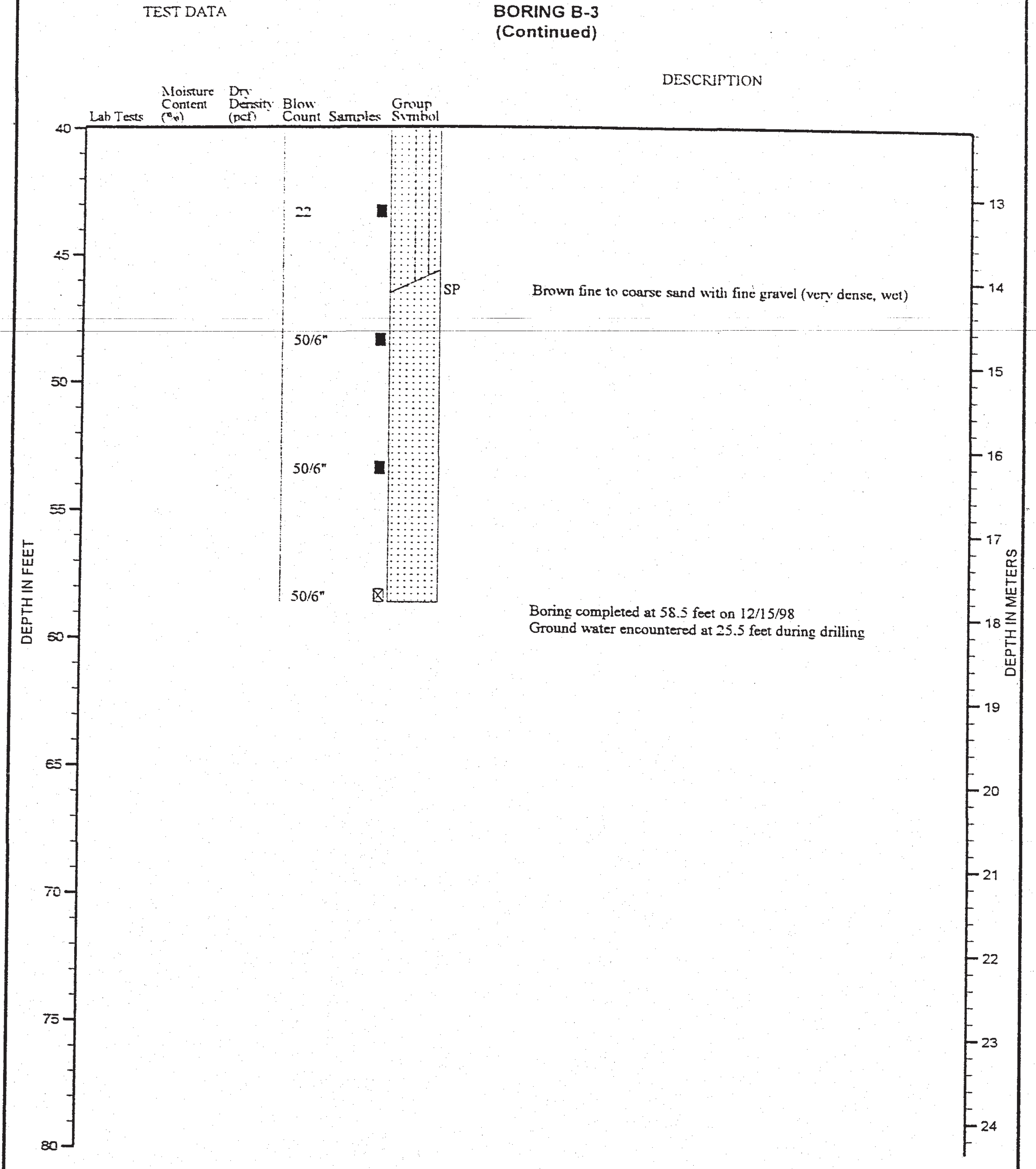
Note: See Figure A-2 for explanation of symbols

GeoEngineers

LOG OF BORING

FIGURE A-5

BORING B-3
(Continued)



Brown fine to coarse sand with fine gravel (very dense, wet)

Boring completed at 58.5 feet on 12/15/98
Ground water encountered at 25.5 feet during drilling

Note: See Figure A-2 for explanation of symbols

DCO:ja 2/9/99

7131-001-00a

GeoEngineers

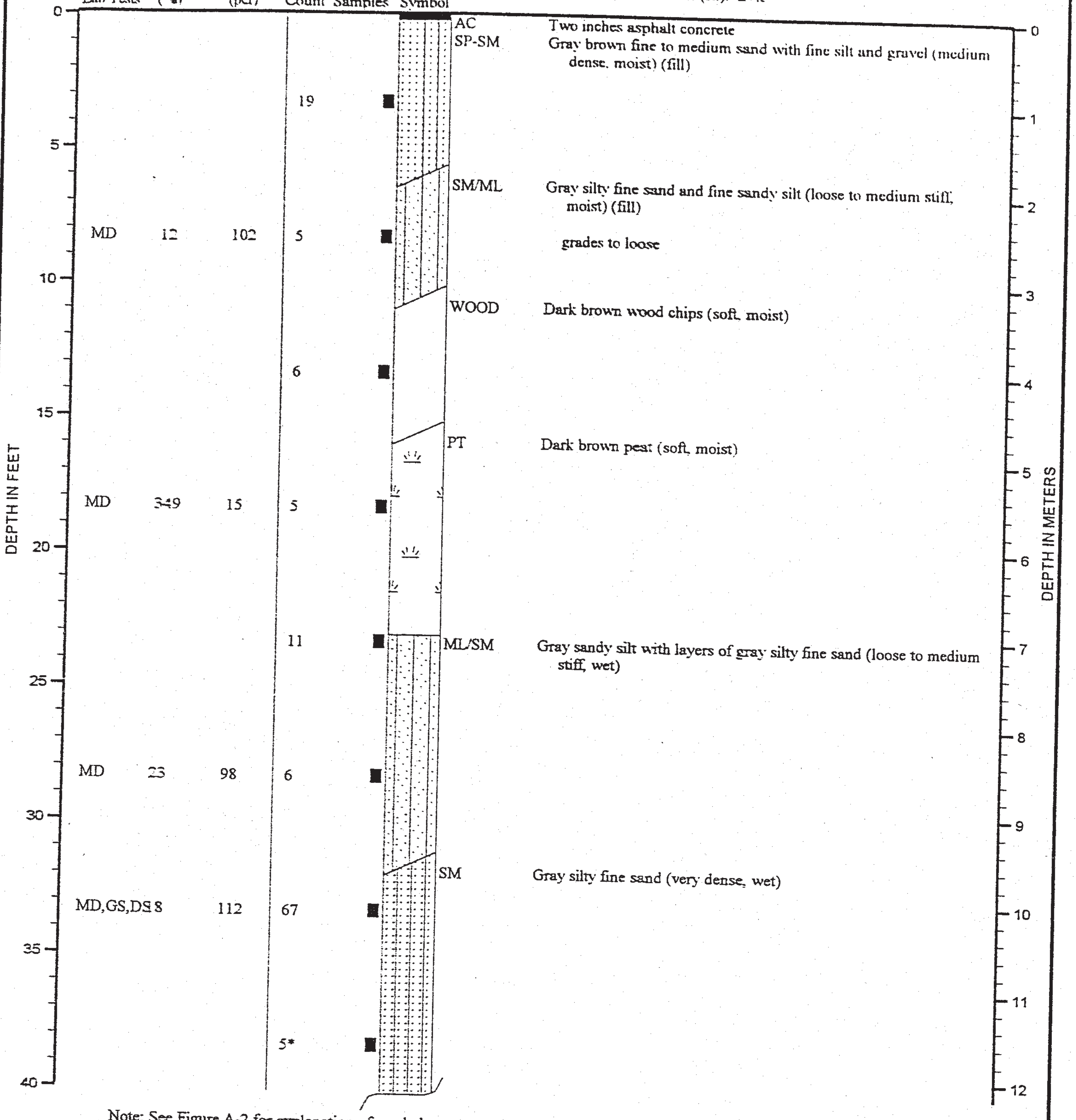
LOG OF BORING

FIGURE A-5

BORING B-4

TEST DATA

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count
			19
MD	12	102	5
			6
MD	349	15	5
			11
MD	23	98	6
MD,GS,DS	8	112	67
			5*



DESCRIPTION

Surface Elevation (ft.): 29.5

Group Symbol	Description
AC	Two inches asphalt concrete
SP-SM	Gray brown fine to medium sand with fine silt and gravel (medium dense, moist) (fill)
SM/ML	Gray silty fine sand and fine sandy silt (loose to medium stiff, moist) (fill) grades to loose
WOOD	Dark brown wood chips (soft, moist)
PT	Dark brown peat (soft, moist)
ML/SM	Gray sandy silt with layers of gray silty fine sand (loose to medium stiff, wet)
SM	Gray silty fine sand (very dense, wet)

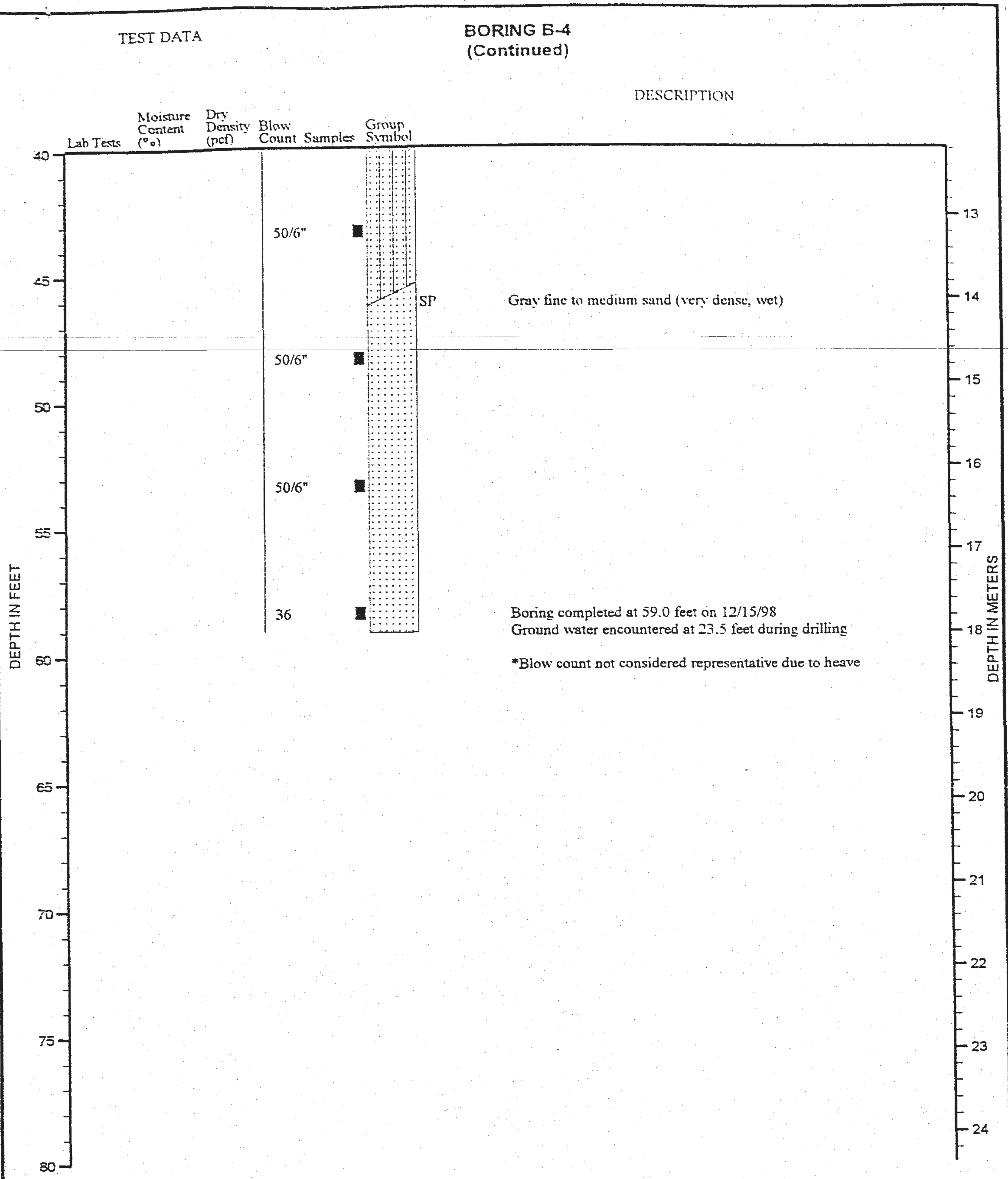
Note: See Figure A-2 for explanation of symbols

GeoEngineers

LOG OF BORING

FIGURE A-6

BORING B-4
(Continued)



Gray fine to medium sand (very dense, wet)

Boring completed at 59.0 feet on 12/15/98
Ground water encountered at 23.5 feet during drilling

*Blow count not considered representative due to heave

Note: See Figure A-2 for explanation of symbols

7131-001-00a DCO:ja 2/9/99

GeoEngineers

LOG OF BORING

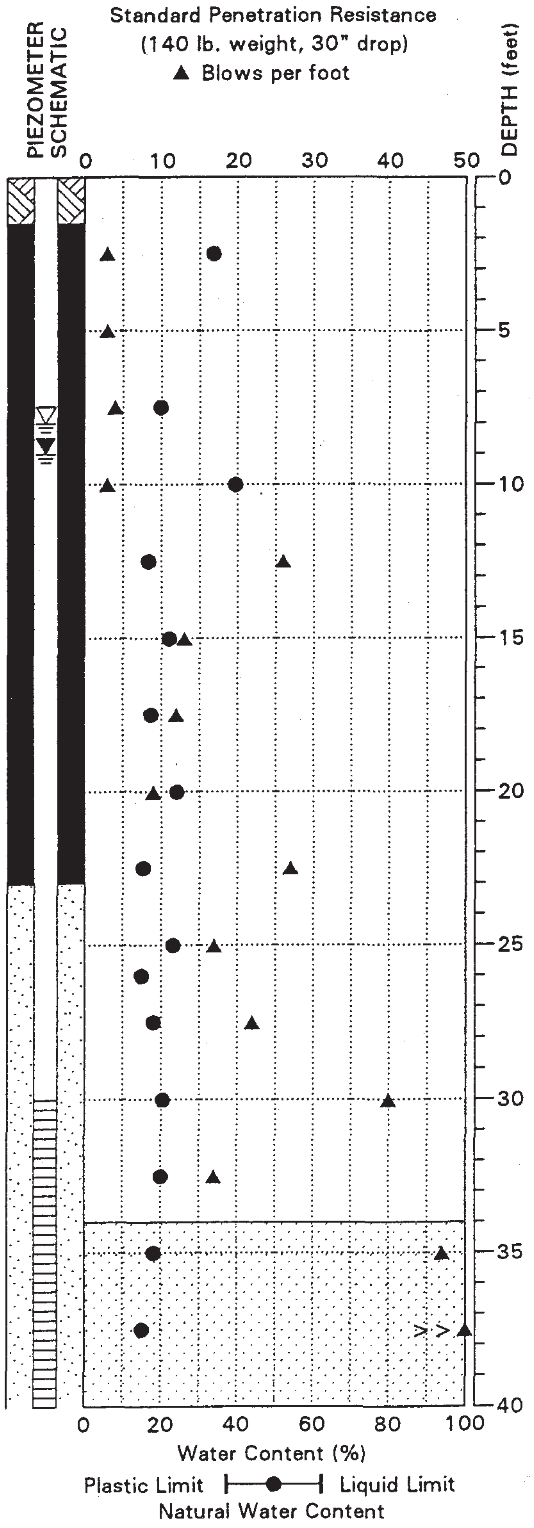
FIGURE A-6

DRILLING COMPANY: Tacoma Pump & Drill
DRILLING METHOD: Acker Soil Sampler, HSA
SURFACE ELEVATION: 125 ± Feet

Doc ID 3364

LOCATION:
DATE COMPLETED: 10/15/97
LOGGED BY: ADM/MLR

ASTM SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
SM	Railroad tie over very loose, dark brown, silty SAND, very moist. Petroleum odor. (FILL)	S-1	1-1-2	
ML	Soft, very dark brown, sandy SILT, very moist.	S-2	1-1-2	
SM	Loose to medium dense, gray, olive and dark brown mottled, silty fine to medium SAND, very moist to wet. Some gravel. Few silt layers. Trace wood chips / organics. Chunks of slickensided clay.	S-3	2-2-2	
		S-4	1-1-1	
		S-5	4-12-14	
SM ML	Medium dense, gray silty SAND to sandy SILT, wet.	S-6	5-6-7	GS
	At 19', brick fragments.	S-7	6-5-7	
SM	Loose to medium dense, mottled gray and dark brown, silty SAND, wet. Occasional small fragment of brick. Some wood chips/sawdust, gravel, and pockets of gray silt.	S-8	3-3-6	GS
		S-9	10-12-15	
SP	Medium dense to dense, gray, fine to coarse SAND, wet.	S-10 S-10B	6-8-9	
		S-11	6-10-12	
		S-12	10-15-25	
	At 33', pieces of glass, string and brick.	S-13	6-7-10	
SP	Dense to very dense, gray, fine to medium SAND, wet. Some coarse gravel. (RECESSIONAL OUTWASH)	S-14	20-21-26	
		S-15	30-32-36	



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.

BORING: BB- 6

HWA GEOSCIENCES INC.

Denny Way / Lake Union CSO, Contract B
Seattle, Washington

PAGE: 1 of 2

PROJECT NO.: 97061

FIGURE: A-7

PZO DWB 5/1/98

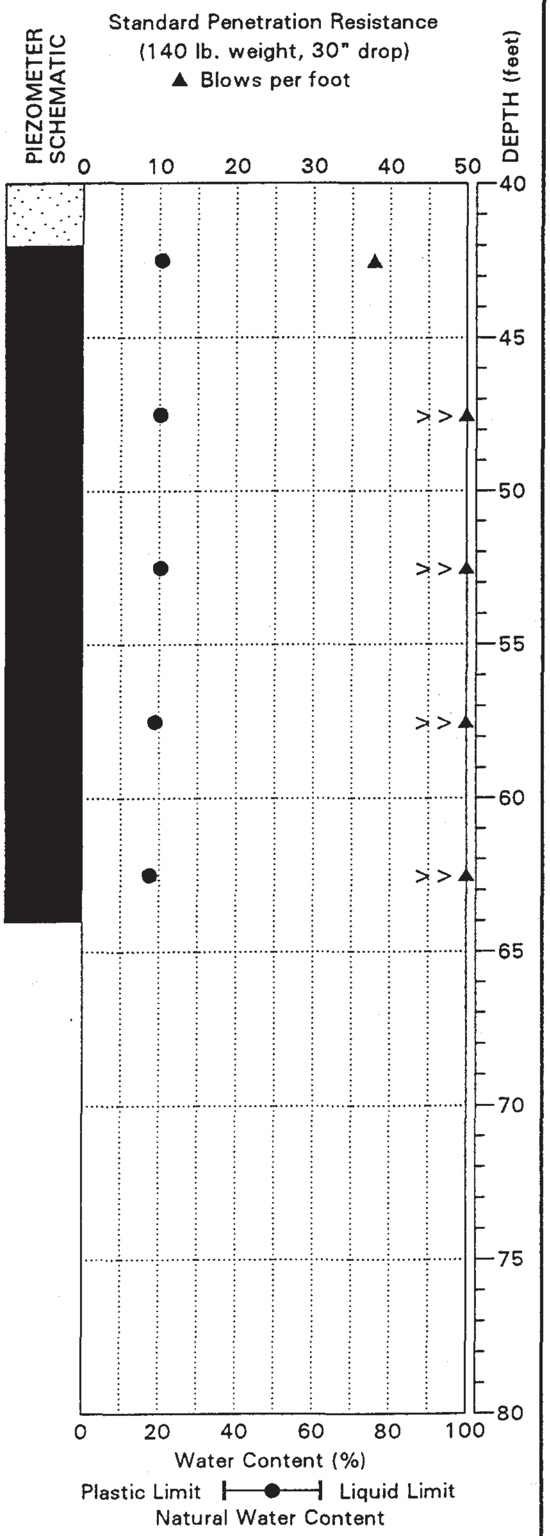
DRILLING COMPANY: Tacoma Pump & Drill
DRILLING METHOD: Acker Soil Sampler, HSA
SURFACE ELEVATION: 125 ± Feet

DOC ID 3364

LOCATION:
DATE COMPLETED: 10/15/97
LOGGED BY: ADM/MLR

DEPTH (feet)	ASTM SOIL CLASS	DESCRIPTION	SAMPLE TYPE SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)
40		Dense to very dense, fine to medium SAND, wet. Some coarse gravel. (RECESSIONAL OUTWASH) Occasional nodule of silt.	S-16	16-18-20
			S-17	20-50/5"
			S-18	42-50/5"
		Lenses of gray silt.	S-19	25-30-36
60	SP SM	Very dense, gray, fine to medium SAND with silt, wet.	S-20	36-50/4"

Total Depth = 64 feet.
2" PVC piezometer installed to 40 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.

BORING: BB- 6

HWA GEOSCIENCES INC.

Denny Way / Lake Union CSO, Contract B
Seattle, Washington

PAGE: 2 of 2

PROJECT NO.: 97061
FIGURE: A-7

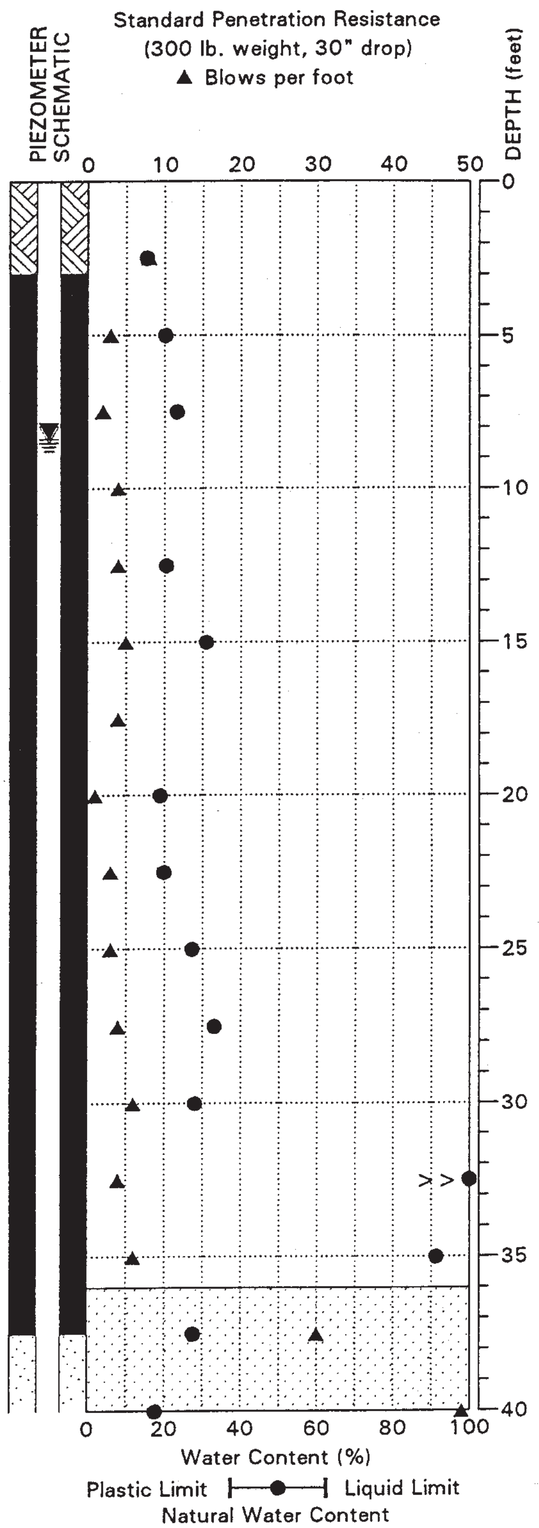
PZO DWB 5/1/98

DRILLING COMPANY: Cherokee
DRILLING METHOD: B-59 Mobile, 4.5" ID HSA
SURFACE ELEVATION: 125 ± Feet

Doc ID 3364

LOCATION:
DATE COMPLETED: 3/3/98
LOGGED BY: MB

DEPTH (feet)	ASTM SOIL CLASS	DESCRIPTION	SAMPLE TYPE SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	SM	Loose, dark gray, slightly gravelly, silty fine SAND, moist. Trace wood (organics). (FILL)	S-1	3-3-5	
5		At 5 feet, becomes very loose; few brick fragments.	S-2	2-1-2	
	ML	Very soft, dark gray, slightly gravelly, sandy SILT, moist to wet. Some fine gravel and coarse sand.	S-3	1-1-1	
10		No sample recovery	S-4	1-2-2	
	SM	Loose, dark gray, silty fine SAND, wet. Brick fragment.	S-5	1-2-2	
15	ML	Soft, dark gray, slightly sandy SILT, wet. Two 2"-thick layers of stiff dark grayish green clay.	S-6	1-2-3	
	SC	Very loose, olive gray, clayey fine SAND, wet. Some gravel to 1" diameter. Few silt lenses.	S-7	3-2-2	
20		No sample recovery at 17.5 feet.	S-8	2-1/12"	
			S-9	1-2-1	
25		At 25 feet, 12" layer of saw dust and wood chips.	S-10	2-2-1	GS
		At 27.5 feet, becomes fine to medium grained. Wood chips.	S-11	3-1-1	
30		At 30 feet, wood fragments.	S-12	1-2-4	
	PT	Very soft, dark brown PEAT, wet, interbedded with layers of dark gray fine sand to silt. Trace wood chips.	S-13	1-2-2	
35	PT SM	Loose, dark gray, silty fine SAND, wet, interbedded with soft, dark brown PEAT.	S-14	1-2-4	
	SP SM	Medium dense to dense, dark gray, slightly silty fine to medium SAND, wet. Trace wood. (RECESSIONAL OUTWASH)	S-15	8-16-14	
40					



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.

BORING: BB-14

HWA GEOSCIENCES INC.

Denny Way / Lake Union CSO, Contract B
Seattle, Washington

PAGE: 1 of 2

PROJECT NO.: 97061

FIGURE: A-15

PZO300 DWB 5/1/98

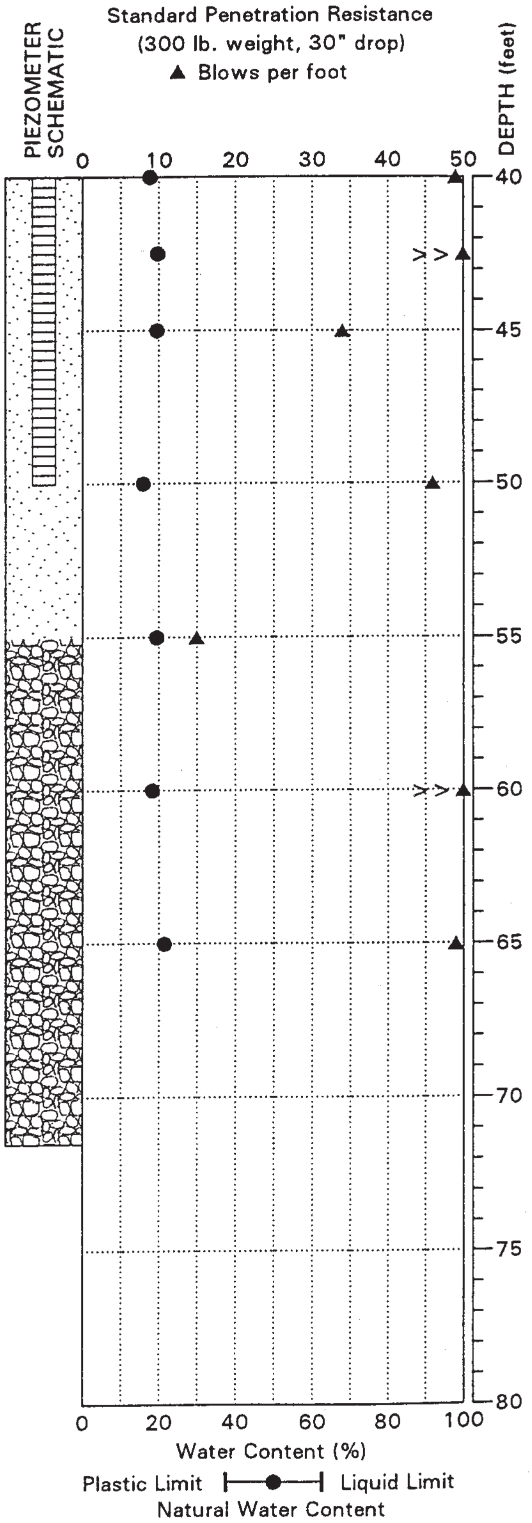
DRILLING COMPANY: Cherokee
DRILLING METHOD: B-59 Mobile, 4.5" ID HSA
SURFACE ELEVATION: 125 ± Feet

Doc ID 3364

LOCATION:
DATE COMPLETED: 3/3/98
LOGGED BY: MB

DEPTH (feet)	SYMBOL	ASTM SOIL CLASS	DESCRIPTION	SAMPLE TYPE SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
40			At 40 feet, becomes dense, slightly silty, slightly gravelly. 3/8" minus subrounded gravel.	S-16	5-23-26	GS
			(RECESSIONAL OUTWASH)	S-17	4-18-35	
45		SM	Dense, dark gray, silty fine to medium SAND, wet.	S-18	5-12-20	GS
50		SP SM	Dense, dark gray, slightly silty fine SAND, wet.	S-19	9-18-28	
55			At 55 feet, becomes medium dense. 2" thick layer of gray silt.	S-20	2-6-9	
		SP	Very dense, dark gray, fine to medium SAND, wet.			
60			(ADVANCE OUTWASH)	S-21	18-25-35	
65			At 65 feet, becomes medium grained.	S-22	9-19-30	
70			At 70 feet, some heave. No sample recovery.	S-23		
75						
80						

Total Depth = 71.5 feet.
2" PVC piezometer installed to 50 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.

BORING: BB-14

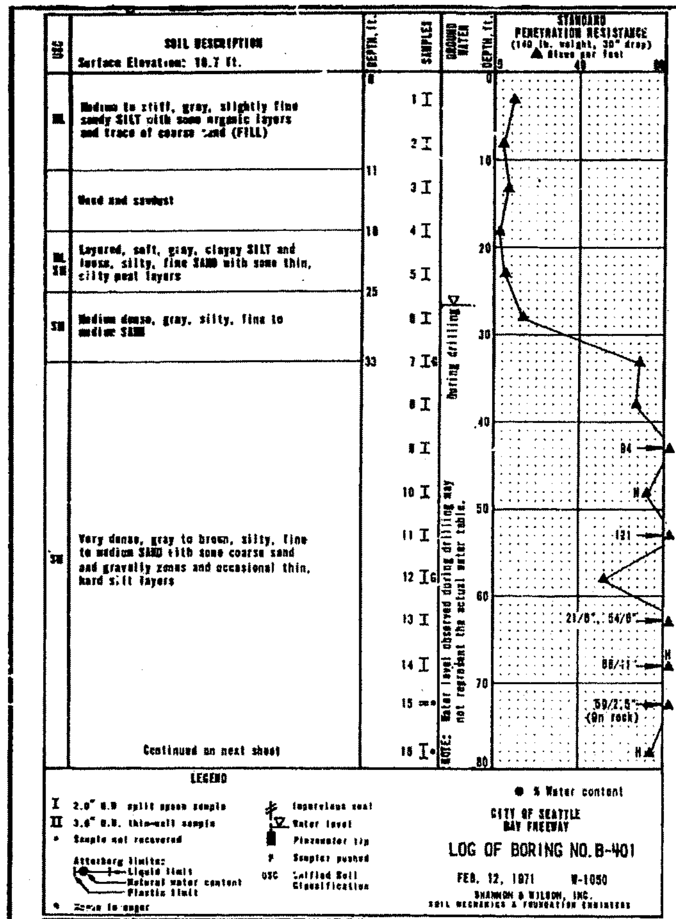
HWA GEOSCIENCES INC.

Denny Way / Lake Union CSO, Contract B
Seattle, Washington

PAGE: 2 of 2

PROJECT NO.: 97061

FIGURE: A-15



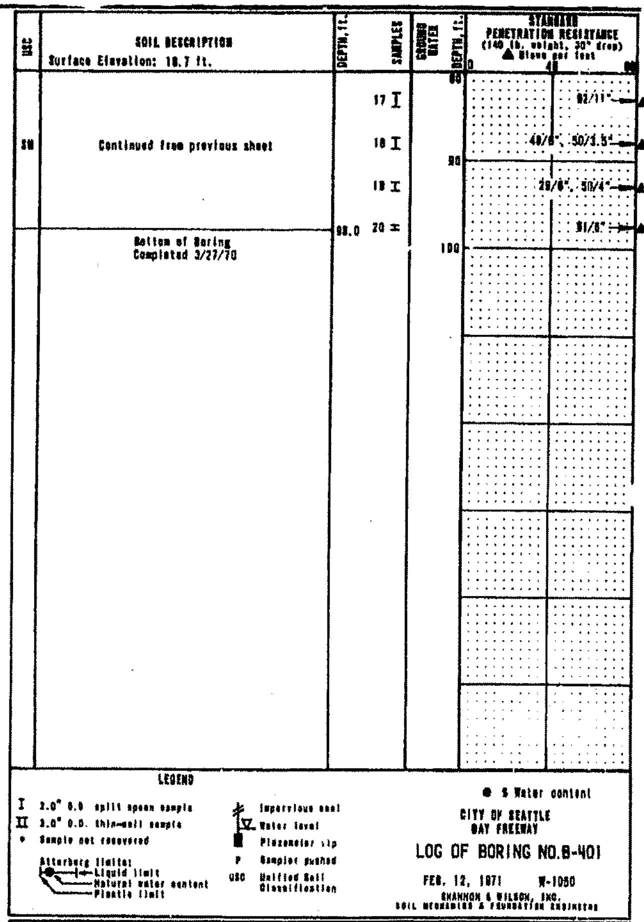
Surface Elevation: 18.7 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.
ML	Medium to stiff, gray, slightly fine sandy SILT with some organic layers and trace of coarse sand (FILL)	0
	Wood and sawdust	11
ML SM	Layered, soft, gray, clayey SILT and loose, silty, fine SAND with some thin, silty peat layers	18
SM	Medium dense, gray, silty, fine to medium SAND	25
SM	Very dense, gray to brown, silty, fine to medium SAND with some coarse sand and gravelly zones and occasional thin, hard silt layers	33

Samples: 1 I, 2 I, 3 I, 4 I, 5 I, 6 I, 7 I G, 8 I, 9 I, 10 I, 11 I, 12 I G, 13 I, 14 I, 15 ==*, 16 I*

Continued on next sheet

CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-401
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

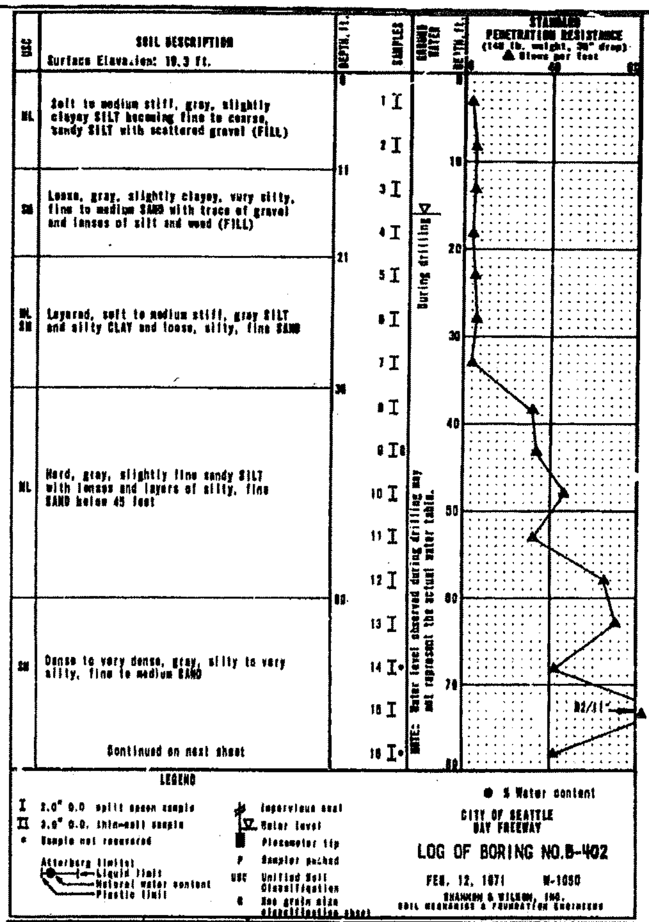


Surface Elevation: 18.7 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.
SM	Continued from previous sheet	
	Bottom of Boring Completed 3/27/70	98.0

Samples: 17 I, 18 I, 19 I, 20 ==

CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-401
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS



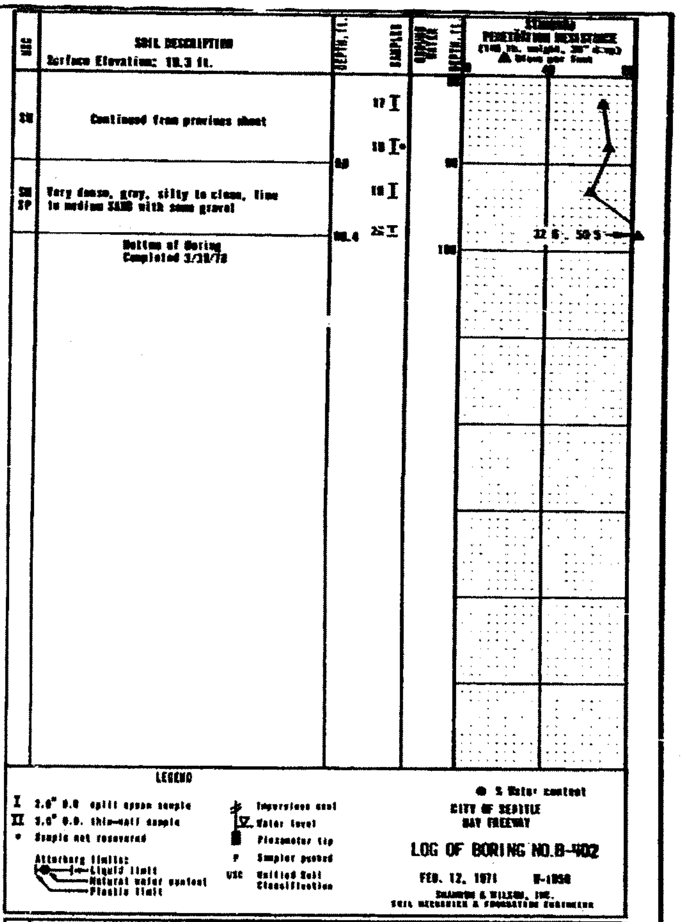
Surface Elevation: 19.3 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.
ML	Soft to medium stiff, gray, slightly clayey SILT becoming fine to coarse, sandy SILT with scattered gravel (FILL)	0
SM	Loose, gray, slightly clayey, very silty, fine to medium SAND with trace of gravel and lenses of silt and wood (FILL)	11
ML SM	Layered, soft to medium stiff, gray SILT and silty CLAY and loose, silty, fine SAND	21
ML	Hard, gray, slightly fine sandy SILT with lenses and layers of silty, fine SAND below 45 feet	36
SM	Dense to very dense, gray, silty to very silty, fine to medium SAND	60

Samples: 1 I, 2 I, 3 I, 4 I, 5 I, 6 I, 7 I, 8 I, 9 I G, 10 I, 11 I, 12 I, 13 I, 14 I*, 15 I, 16 I*

Continued on next sheet

CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-402
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

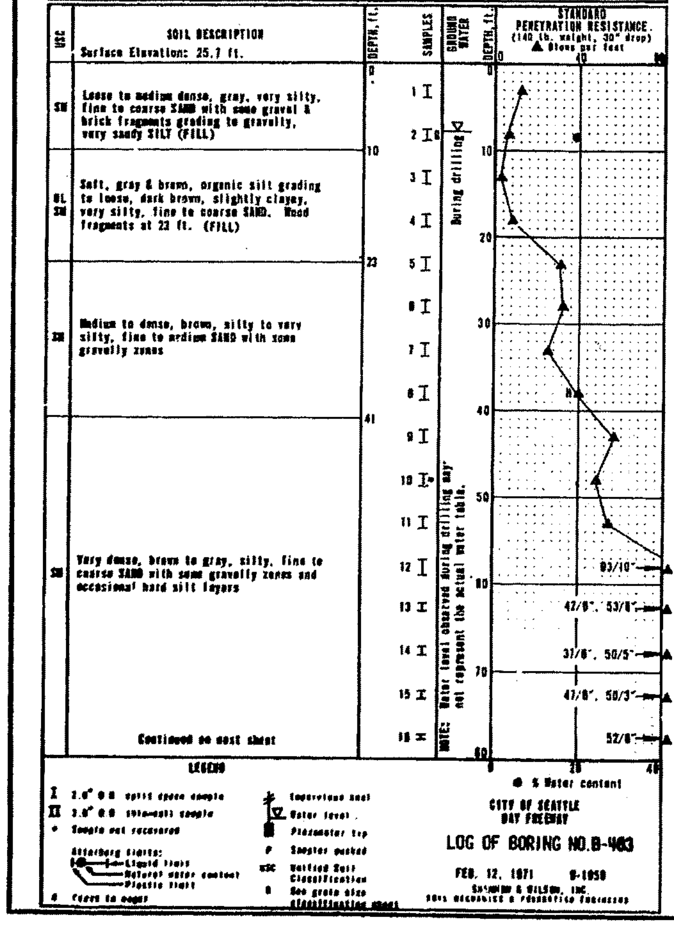


Surface Elevation: 19.3 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.
SM	Continued from previous sheet	
SM SP	Very dense, gray, silty to clean, fine to medium SAND with some gravel	88
	Bottom of Boring Completed 3/30/70	98.4

Samples: 17 I, 18 I*, 19 I, 20 I

CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-402
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS



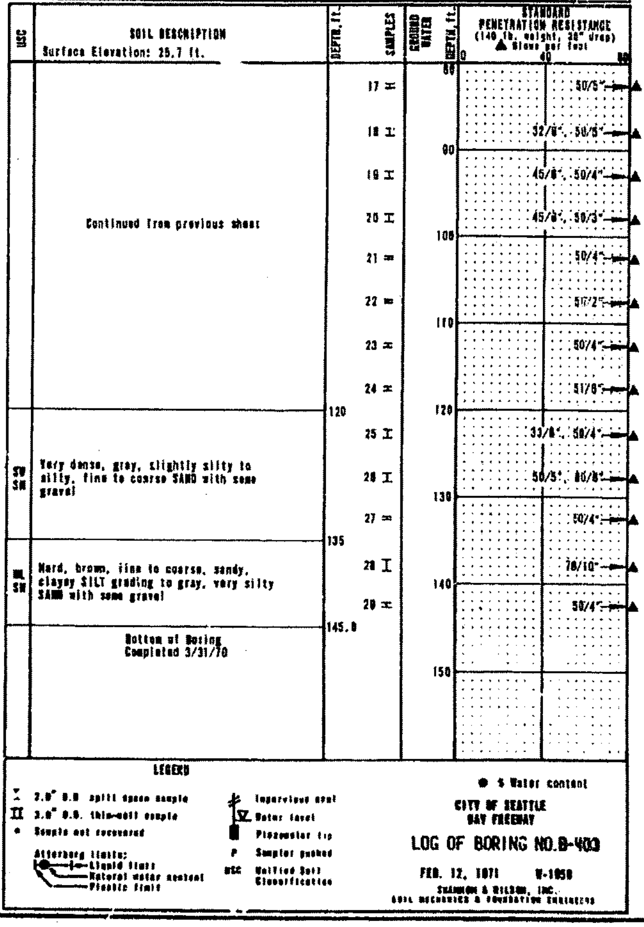
Surface Elevation: 25.7 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.
SM	Loose to medium dense, gray, very silty, fine to coarse SAND with some gravel & brick fragments grading to gravelly, very sandy SILT (FILL)	0
OL SM	Soft, gray & brown, organic silt grading to loose, dark brown, slightly clayey, very silty, fine to coarse SAND. Wood fragments at 22 ft. (FILL)	10
SM	Medium to dense, brown, silty to very silty, fine to medium SAND with some gravelly zones	23
SM	Very dense, brown to gray, silty, fine to coarse SAND with some gravelly zones and occasional hard silt layers	41

Samples: 1 I, 2 I G, 3 I, 4 I, 5 I, 6 I, 7 I, 8 I, 9 I, 10 I*, 11 I, 12 I, 13 I, 14 I, 15 I, 16 ==

Continued on next sheet

CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-403
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

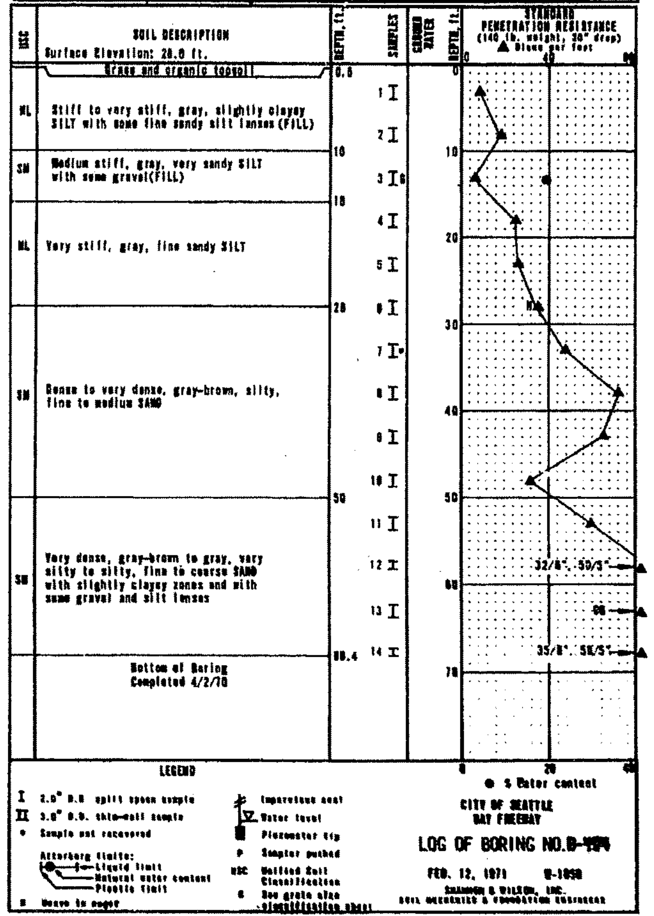


Surface Elevation: 25.7 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.
	Continued from previous sheet	
SW SM	Very dense, gray, slightly silty to silty, fine to coarse SAND with some gravel	120
ML SM	Hard, brown, fine to coarse, sandy, clayey SILT grading to gray, very silty SAND with some gravel	135
	Bottom of Boring Completed 3/31/70	145.0

Samples: 17 ==, 18 I, 19 I, 20 I, 21 ==, 22 ==, 23 ==, 24 ==, 25 I, 26 I, 27 ==, 28 I, 29 ==

CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-403
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

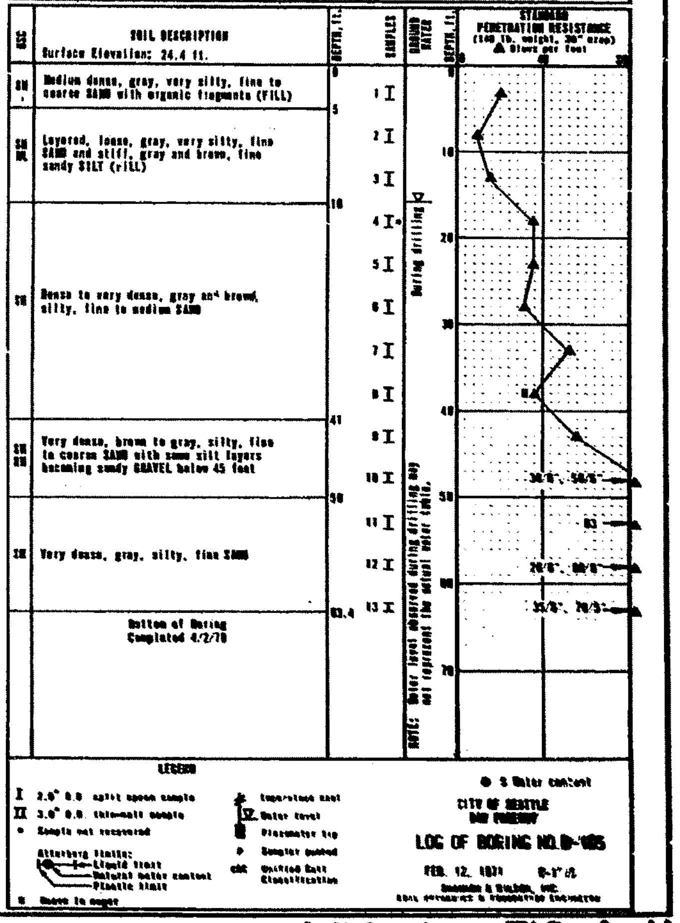


Surface Elevation: 28.8 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.
	Grass and organic topsoil	
ML	Stiff to very stiff, gray, slightly clayey SILT with some fine sandy silt lenses (FILL)	0.8
SM	Medium stiff, gray, very sandy SILT with some gravel (FILL)	10
ML	Very stiff, gray, fine sandy SILT	16
SM	Dense to very dense, gray-brown, silty, fine to medium SAND	28
SM	Very dense, gray-brown to gray, very silty to silty, fine to coarse SAND with slightly clayey zones and with some gravel and silt lenses	50
	Bottom of Boring Completed 4/2/70	68.4

Samples: 1 I, 2 I, 3 I G, 4 I, 5 I, 6 I, 7 I*, 8 I, 9 I, 10 I, 11 I, 12 I, 13 I, 14 I

CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-404
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS



Surface Elevation: 24.4 ft.

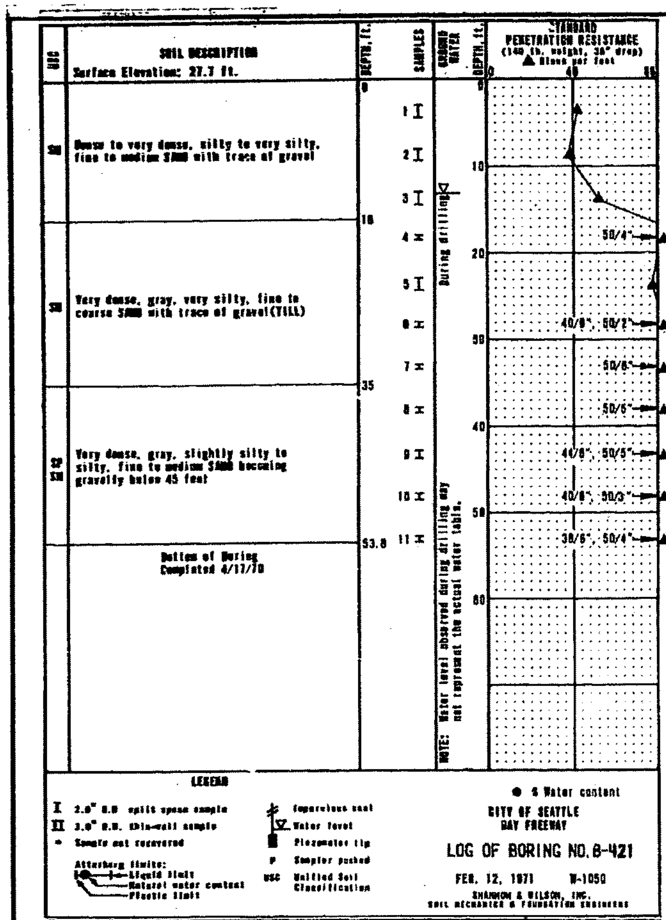
USC	SOIL DESCRIPTION	DEPTH, ft.
SM	Medium dense, gray, very silty, fine to coarse SAND with organic fragments (FILL)	0
SM ML	Layered, loose, gray, very silty, fine SAND and stiff, gray and brown, fine sandy SILT (FILL)	5
SM	Dense to very dense, gray and brown, silty, fine to medium SAND	16
SM GM	Very dense, brown to gray, silty, fine to coarse SAND with some silt layers becoming sandy GRAVEL below 45 feet	41
SM	Very dense, gray, silty, fine SAND	50
	Bottom of Boring Completed 4/2/70	63.4

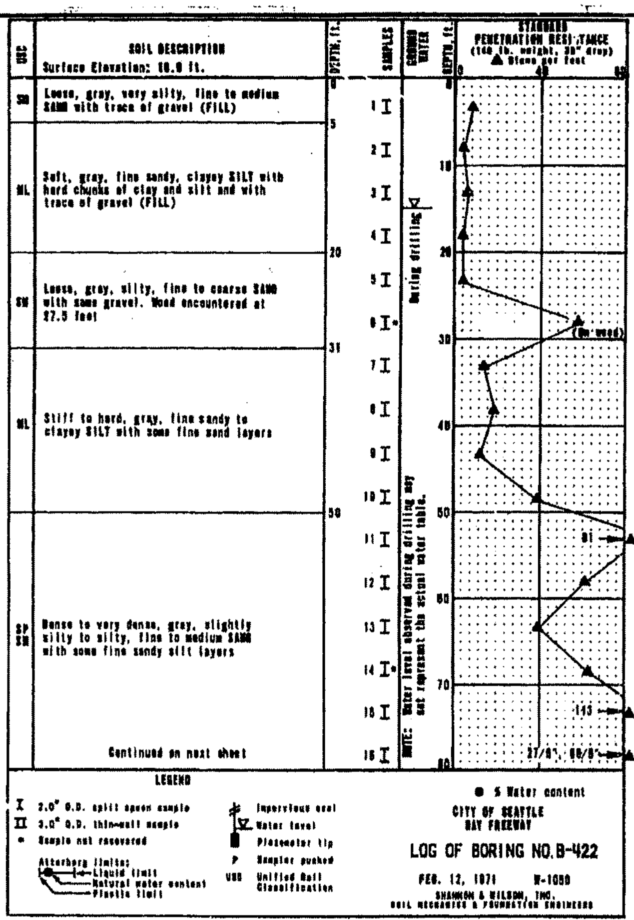
Samples: 1 I, 2 I, 3 I, 4 I*, 5 I, 6 I, 7 I, 8 I, 9 I*, 10 I, 11 I, 12 I, 13 I

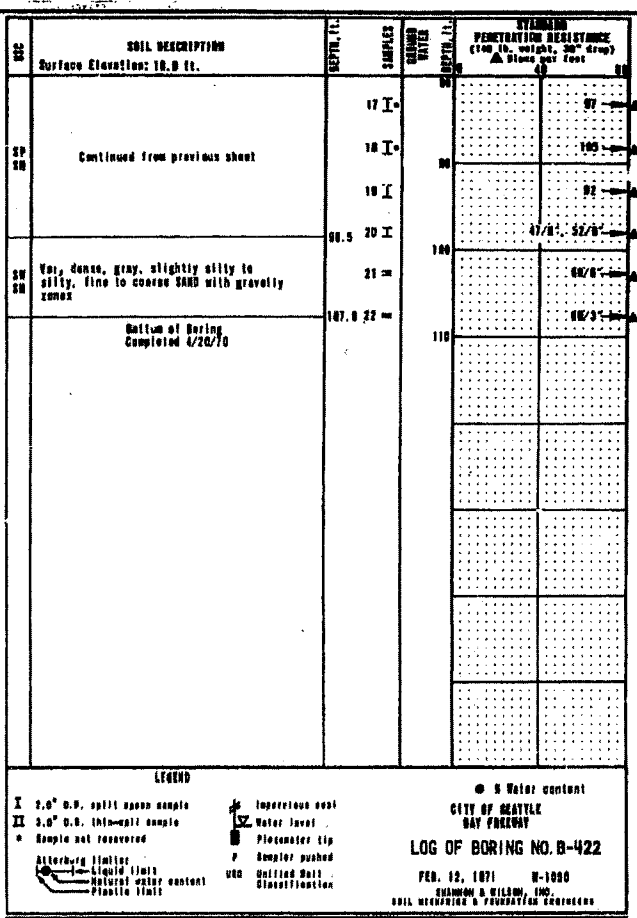
CITY OF SEATTLE
BAY FREEWAY
LOG OF BORING NO.B-405
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

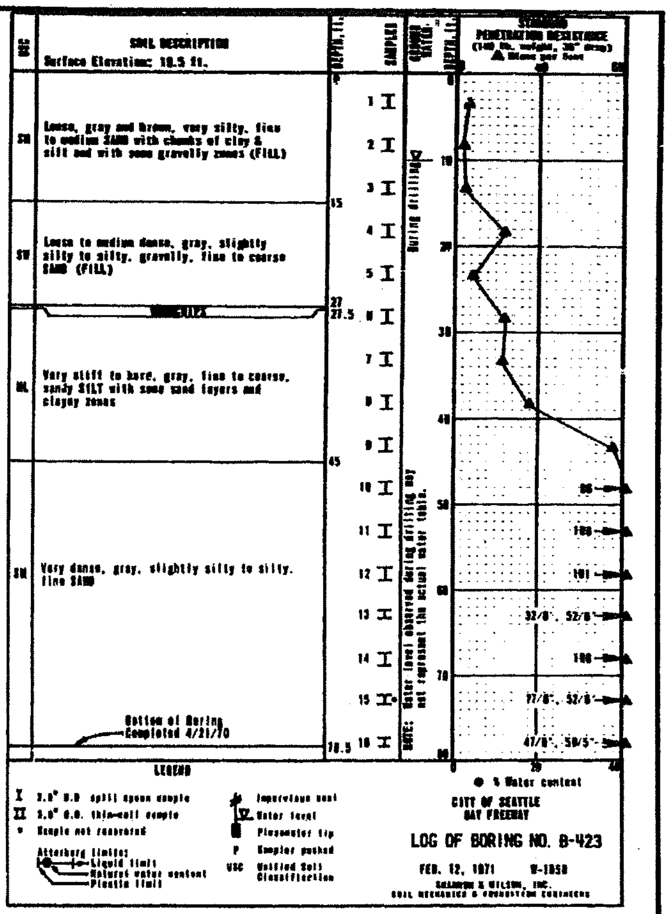
07621

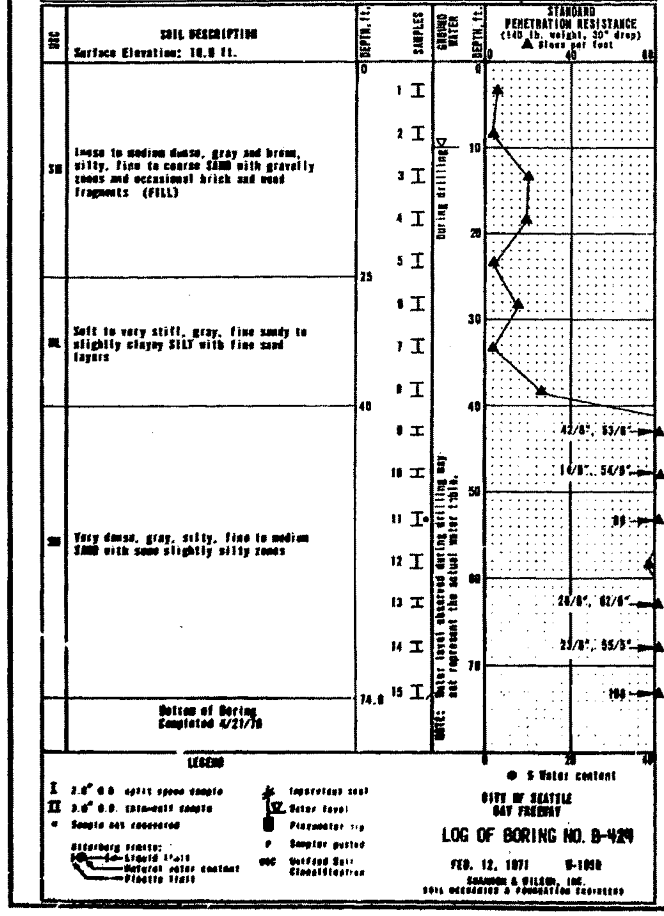
FIG. A-11

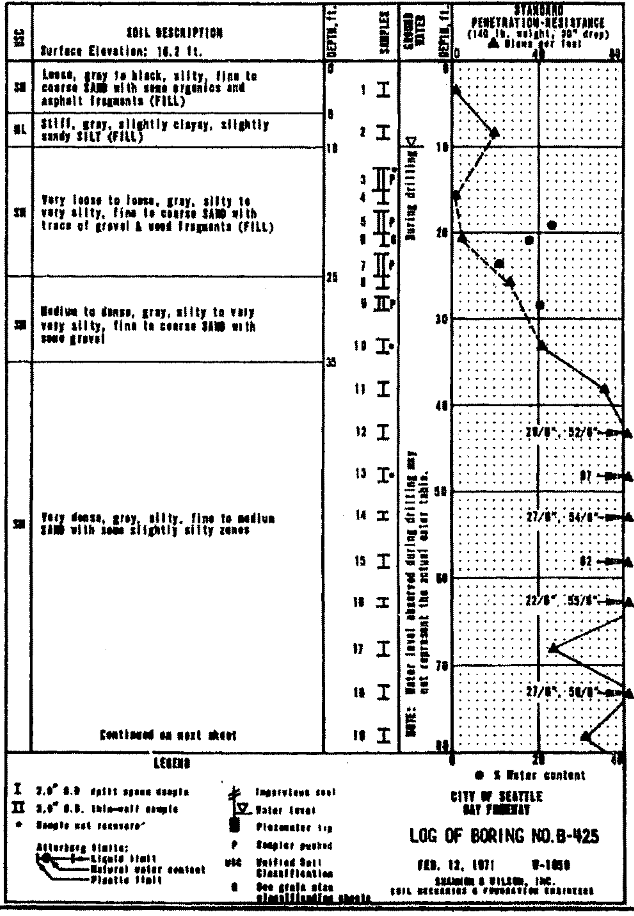


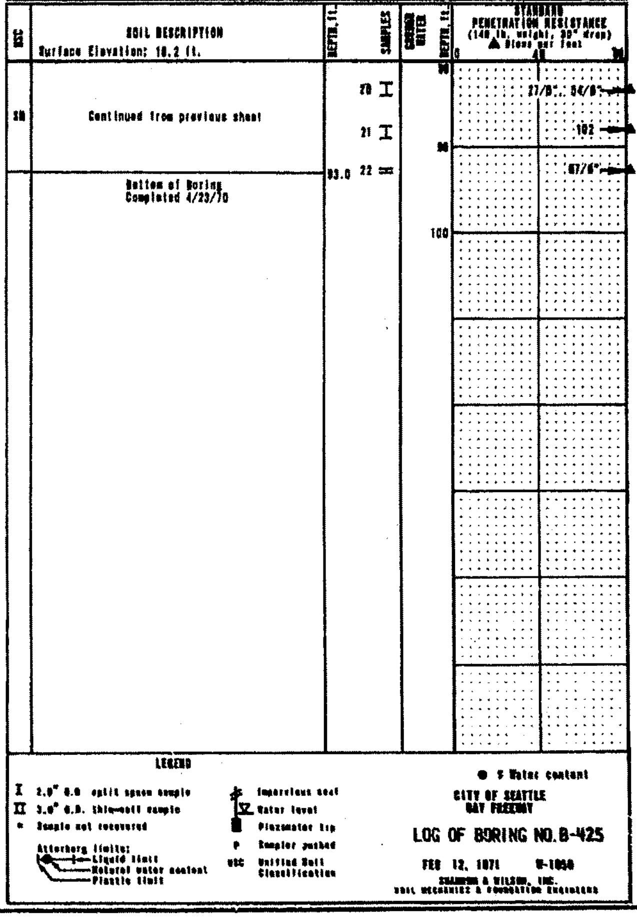












07024 FIG A-14

LOG OF BORING NO. B-426

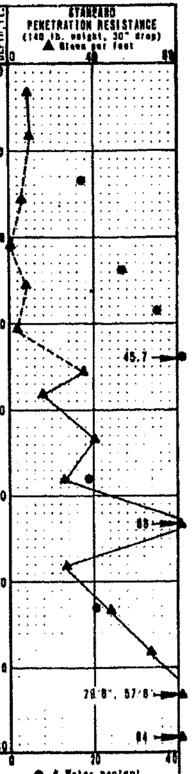
SOIL DESCRIPTION
Surface Elevation: 18.7 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
SM	Very loose to loose, gray, very silty, slightly clayey, fine to medium SAND with sawdust fragments gravel and wood or logs 8 inches or greater in diameter (FILL)	30	1, 2, 3, 4, 5, 6, 7, 8, 9, 10
OL	Soft, brown to gray, peaty SILT	32	
SM ML	Dense, gray, very silty, fine SAND with layers of very stiff to hard, sandy SILT	45	11, 12, 13, 14
SM	Medium to very dense, gray, silty to slightly silty, fine to medium SAND	61	15, 16, 17
SM	Dense to very dense, gray, silty to slightly silty, fine to medium SAND	70	18, 19
SM	Very dense, gray, silty, fine to coarse SAND with some slightly silty zones		20, 21

Continued on next sheet

GROUND WATER: During drilling

NOTE: Water level observed during drilling may not represent the actual water table.



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification
- G See grain size classification sheet

CITY OF SEATTLE
BAY FREEWAY

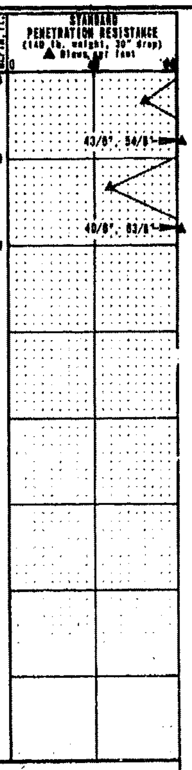
LOG OF BORING NO. B-426

FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

LOG OF BORING NO. B-426

SOIL DESCRIPTION
Surface Elevation: 18.7 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
SM	Continued from previous sheet	98.5	22, 23, 24, 25
	Bottom of Boring Completed 4/23/70		



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification

CITY OF SEATTLE
BAY FREEWAY

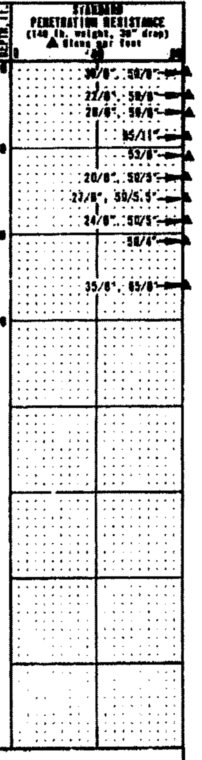
LOG OF BORING NO. B-426

FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

LOG OF BORING NO. B-427

SOIL DESCRIPTION
Surface Elevation: 20.3 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
SM	Very dense, gray-brown, very silty, fine to coarse SAND with trace of gravel and some silty pockets (TILL)	17	1, 2, 3, 4, 5, 6, 7
SM	Very dense, gray-brown, very silty, fine SAND	20	8
SM	Very dense, gray-brown, very silty, fine to coarse SAND with trace of gravel (TILL)	28.5	9, 10
	Bottom of Boring Completed 4/24/70		



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification

CITY OF SEATTLE
BAY FREEWAY

LOG OF BORING NO. B-427

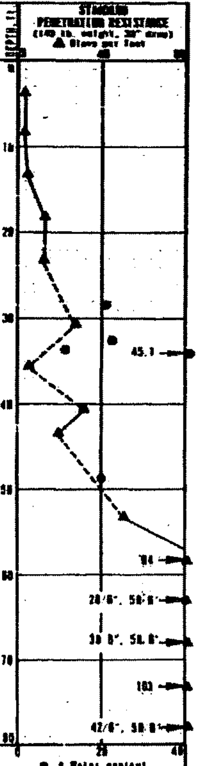
FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

LOG OF BORING NO. B-428

SOIL DESCRIPTION
Surface Elevation: 10.2 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
SM	Loose, gray, very silty, fine SAND with some wood and silt layers (FILL)	3	1
ML	Soft to medium, gray, fine sandy, slightly clayey SILT with trace of gravel (FILL)	10	2, 3
ML	Stiff, gray, slightly clayey to clayey SILT with some fine sand pockets (FILL)	30	4, 5, 6
SM ML CL	Medium to dense, gray, very silty, fine to medium SAND with layers of medium stiff, gray, sandy SILT & silty CLAY	50	7, 8, 9, 10, 11, 12, 13
SM ML	Very dense, gray, very silty, fine to medium SAND with hard, sandy silt layers	60	14, 15
SW	Very dense, gray, clean, fine to coarse SAND with trace of gravel	78.5	16, 17, 18, 19
	Bottom of Boring Completed 4/24/70		

GROUND WATER: During drilling



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification
- G See grain size classification sheet

CITY OF SEATTLE
BAY FREEWAY

LOG OF BORING NO. B-428

FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

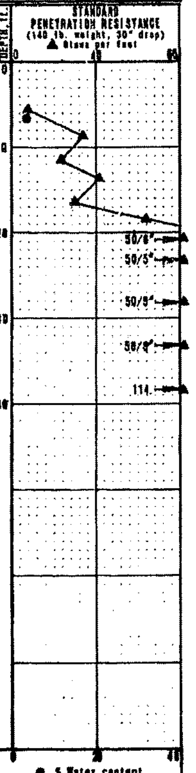
LOG OF BORING NO. B-429

SOIL DESCRIPTION
Surface Elevation: 20.0 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
SW	Loose, brown, fine to coarse SAND & GRAVEL (FILL)	7	1, 2
SM	Medium to dense, silty, fine to coarse SAND with trace of gravel	18	3, 4, 5, 6
SP	Very dense, clean, fine to medium SAND	20	7
SM	Very dense, gray, very silty, fine to medium SAND with trace of gravel	39.0	8, 9, 10, 11, 12
	Bottom of Boring Completed 4/27/70		

GROUND WATER: During drilling

NOTE: Water level observed during drilling may not represent the actual water table.



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification
- G See grain size classification sheet

CITY OF SEATTLE
BAY FREEWAY

LOG OF BORING NO. B-429

FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

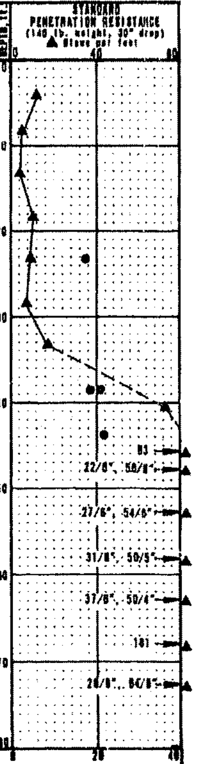
LOG OF BORING NO. B-430

SOIL DESCRIPTION
Surface Elevation: 10.8 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
SM	Loose to medium, brown, very silty, fine to coarse SAND with trace of gravel, layers of slightly clayey SILT and some brick fragments (FILL)	16	1, 2, 3
	Wood and sawdust	20	4
SP-SM	Loose, gray, gravelly, slightly silty, fine to medium SAND	25	5
ML	Medium to very stiff, gray, fine sandy SILT with clayey zones and with some sand layers	35	6, 7
SP-SM	Very dense, gray, slightly silty to silty, fine to medium SAND with occasional thin, hard silt layers and with gravel below 70 feet	73.5	8, 9, 10, 11, 12, 13, 14, 15, 16, 17
	Bottom of Boring Completed 4/27/70		

GROUND WATER: During drilling

NOTE: Water level observed during drilling may not represent the actual water table.



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification
- G See grain size classification sheet

CITY OF SEATTLE
BAY FREEWAY

LOG OF BORING NO. B-430

FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

LOG OF BORING NO. B-431

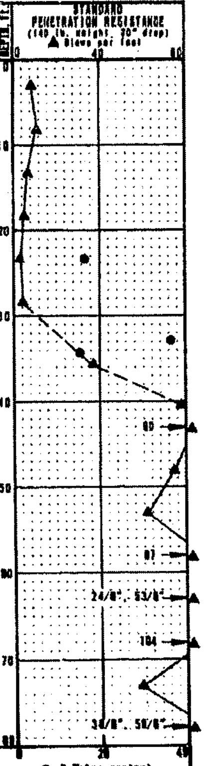
SOIL DESCRIPTION
Surface Elevation: 26.2 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
SM	Loose, gray, very silty, fine to coarse SAND with some silt layers and trace of gravel (FILL)	11	1, 2
ML	Medium stiff, tan to gray, slightly clayey to clayey SILT with some very silty sand layers and some wood fragments (FILL)	27	3, 4, 5
SM OL CL	Layered, very loose, gray, silty, fine to medium SAND and medium-stiff, gray, organic clayey SILT and silty CLAY	35	6, 7
SM	Dense, gray, silty, fine to medium SAND	38	8
SM	Very dense, gray, silty to very silty, fine to medium SAND with some slightly silty zones and some gravelly zones		9, 10, 11, 12, 13, 14, 15, 16, 17, 18

Continued on next sheet

GROUND WATER: During drilling

NOTE: Water level observed during drilling may not represent the actual water table.



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification
- G See grain size classification sheet

CITY OF SEATTLE
BAY FREEWAY

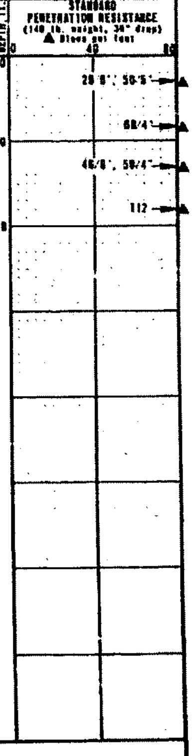
LOG OF BORING NO. B-431

FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

LOG OF BORING NO. B-431

SOIL DESCRIPTION
Surface Elevation: 26.2 ft.

USC	SOIL DESCRIPTION	DEPTH, ft.	SAMPLES
	Continued from previous sheet	98.0	19, 20, 21, 22
	Bottom of Boring Completed 4/28/70		



LEGEND
- I 2.0" O.D. split spoon sample
- II 3.0" O.D. thin-wall sample
- * Sample not recovered
- Atterberg limits: Liquid limit, Natural water content, Plastic limit
- Impervious seal
- Water level
- Piezometer tip
- P Sampler pushed
- USC Unified Soil Classification

CITY OF SEATTLE
BAY FREEWAY

LOG OF BORING NO. B-431

FEB. 12, 1971 W-1050
SHANNON & WILSON, INC.
SOIL MECHANICS & FOUNDATION ENGINEERS

07625 FIG. A-15

APPENDIX C

Ground Anchor Load Tests and Shoring Monitoring Program

APPENDIX C
GROUND ANCHOR LOAD TESTS AND SHORING MONITORING PROGRAM

Ground Anchor Load Testing

General

The locations of the load tests shall be approved by the Engineer and shall be representative of the field conditions. Load tests shall not be performed until the nail/tieback grout and shotcrete wall facing, where present, have attained at least 50 percent of the specified 28-day compressive strengths.

Where temporary casing of the unbonded length of test nails/tiebacks is provided, the casing shall be installed to prevent interaction between the bonded length of the nail/tieback and the casing/testing apparatus.

The testing equipment shall include two dial gauges accurate to 0.001 inch, a dial gauge support, a calibrated jack and pressure gauge, a pump and the load test reaction frame. The dial gauge should be aligned within 5 degrees of the longitudinal nail/tieback axis and shall be supported independently from the load frame/jack and the shoring wall. The hydraulic jack, pressure gauge and pump shall be used to apply and measure the test loads.

The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit. The pressure gauge shall be graduated in 100 pounds per square inch (psi) increments or less and shall have a range not exceeding twice the anticipated maximum pressure during testing unless approved by the Engineer. The ram travel of the jack shall be sufficient to enable the test to be performed without repositioning the jack.

The jack shall be supported independently and centered over the nail/tieback so that the nail/tieback does not carry the weight of the jack. The jack, bearing plates and stressing anchorage shall be aligned with the nail/tieback. The initial position of the jack shall be such that repositioning of the jack is not necessary during the load test.

The reaction frame should be designed/sized such that excessive deflection of the test apparatus does not occur and that the testing apparatus does not need to be repositioned during the load test. If the reaction frame bears directly on the shoring wall facing, the reaction frame should be designed so as not to damage the facing.

Verification Tests

Prior to production soil nail/tieback installation, at least two soil nails/tiebacks for each soil type shall be tested to validate the design pullout value. All test nails/tiebacks shall be installed by the same methods, personnel, material and equipment as the production anchors. Changes in methods, personnel, material or equipment may require additional verification testing as determined by the Engineer. At least two successful verification tests shall be performed for each installation method and each soil type. The nails/tiebacks used for the verification tests may be used as production nails/tiebacks if approved by the Engineer.

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars. The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Tieback design test loads should be the design load specified on the shoring drawings. Verification test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL	1 minute
1.5DL	60 minutes
1.75DL	1 minute
2.0DL	10 minutes

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.5DL test load shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes.

Proof Tests

Proof tests shall be completed on approximately 5 percent of the production nails at locations selected by the owner's representative. Additional testing may be required where nail installation methods are substandard. Proof tests shall be completed on each production tieback.

For soil nails, the unbonded length of the test nails shall be at least 3 feet unless approved otherwise by the Engineer. The bond length of the test nails shall not be less than 10 feet and shall not be longer than the bond length that would prevent testing to 200 percent of the design load while not exceeding the allowable bar load. The allowable bar load during testing shall not exceed 80 percent of the steel ultimate strength for Grade 150 bars or 90 percent of the steel ultimate strength for Grade 60 and 75 bars. The allowable tieback load should not exceed 80 percent of the steel ultimate strength.

For soil nails, the design test load shall be determined by multiplying the bond length of the nail times the design load pullout resistance (load transfer). Tieback design test loads should be the design load specified on the shoring drawings. Proof test nails/tiebacks shall be incrementally loaded and unloaded in accordance with the following schedule:

Load	Hold Time
Alignment Load	1 minute
0.25 Design Load (DL)	1 minute
0.5DL	1 minute
0.75DL	1 minute
1.0DL	1 minute
1.25DL (soil nails)	1 minute
1.33DL (tiebacks)	10 minutes
1.5DL (soil nails)	

The alignment load shall be the minimum load required to align the testing apparatus and should not exceed 5 percent of the design load. The dial gauge should be zeroed after the alignment load is applied. Nail/tieback deflections during the 1.33DL and 1.5DL test loads shall be recorded at 1, 2, 3, 5, 6 and 10 minutes.

Depending upon the nail/tieback deflection performance, the load hold period at 1.33DL (tiebacks) or 1.5DL (soil nails) may be increased to 60 minutes. Nail/tieback movement shall be recorded at 1, 2, 3, 5, 6 and 10 minutes. If the nail/tieback deflection between 1 minute and 10 minutes is greater than 0.04 inches, the 1.33DL/1.5DL load shall be continued to be held for a total of 60 minutes and deflections recorded at 20, 30, 50 and 60 minutes.

Test Nail/Tieback Acceptance

A test nail/tieback shall be considered acceptable when:

1. For verification tests, a nail/tieback is considered acceptable if the creep rate is less than 0.08 inches per log cycle of time between 6 and 60 minutes and the creep rate is linear or decreasing throughout the creep test load hold period.
2. For proof tests, a nail/tieback is considered acceptable if the creep rate is less than 0.04 inches per log cycle of time between 1 and 10 minutes or the creep rate is less than 0.08 inches per log cycle of time between 6 and 60 minutes, and the creep rate is linear or decreasing throughout the creep test load hold period.
3. The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.
4. Pullout failure does not occur. Pullout failure is defined as the load at which continued attempts to increase the test load result in continued pullout of the test nail/tieback.

Acceptable proof-test nails/tiebacks may be incorporated as production nails/tiebacks provided that the unbonded test length of the nail/tieback hole has not collapsed and the test nail/tieback length and bar size/number of strands are equal to or greater than the scheduled production nail/tieback at the test location. Test nails/tiebacks meeting these criteria shall be completed by grouting the unbonded length. Maintenance of the temporary unbonded length for subsequent grouting is the contractor's responsibility.

The Engineer shall evaluate the verification test results. Nail/tieback installation techniques that do not satisfy the nail/tieback testing requirements shall be considered inadequate. In this case, the contractor shall propose alternative methods and install replacement verification test nails/tiebacks.

The Engineer may require that the contractor replace or install additional production nails/tiebacks in areas represented by inadequate proof tests.

Shoring Monitoring

Preconstruction Survey

A shoring monitoring program should be established to monitor the performance of the temporary shoring walls and to provide early detection of deflections that could potentially damage nearby improvements. We recommend that a preconstruction survey of adjacent improvements, such as streets, utilities and buildings, be performed prior to commencing construction. The preconstruction survey should include a video or photographic survey of the condition of existing improvements to establish the preconstruction condition, with special attention to existing cracks in streets or buildings.

Optical Survey

The shoring monitoring program should include an optical survey monitoring program. The recommended frequency of monitoring should vary as a function of the stage of construction as presented in the following table.

Construction Stage	Monitoring Frequency
During excavation and until wall movements have stabilized	Twice weekly
During excavation if lateral wall movements exceed 1 inch and until wall movements have stabilized	TBD
After excavation is complete and wall movements have stabilized, and before the floors of the building reach the top of the excavation	Twice monthly

Monitoring should include vertical and horizontal survey measurements accurate to at least 0.01 feet. A baseline reading of the monitoring points should be completed prior to beginning excavation. The survey data should be provided to GeoEngineers for review within 24 hours.

For shoring walls, we recommend that optical survey points be established: (1) along the top of the shoring walls and (2) at the curb lines around the perimeter of the site. The survey points should be located on every other soldier pile along the wall face for soldier pile and tieback shoring and at every 25 feet for diaphragm type shoring systems. The points along the curb line should be located at an approximate spacing of 50 feet. If lateral wall movements are observed to be in excess of ½ inch between successive readings or if total wall movements exceed 1 inch, construction of the shoring walls should be stopped to determine the cause of the movement and to establish the type and extent of remedial measures required.

Inclinometers

Inclinometers may be beneficial to monitor shoring wall deformations at selected locations, depending on the depth of the excavation and the type of shoring system used. Where necessary, the inclinometers should be installed on the back of the soldier piles if conventional shoring is used. For diaphragm type walls, the inclinometers should be installed in boreholes installed immediately behind the shoring wall at a location where tieback anchors will not damage the inclinometer casing. For soldier pile walls, the inclinometer should extend to the tip of the soldier pile. For diaphragm walls, the inclinometer should extend at least 30 feet below the base of excavation.

APPENDIX D

Report Limitations and Guidelines for Use

APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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 XI, LLC and other project team members for the Block 37 – South Lake Union Development project. 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 the Client 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 Block 37 – South Lake Union Development project in Seattle, 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.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org .

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.

Have we delivered World Class Client Service?

Please let us know by visiting **www.geoengineers.com/feedback**.

GeoEngineers