Geotechnical Report
18th Avenue and Cherry Street
Medical Office Building
Seattle, Washington

February 8, 2017

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Excellence. Innovation. Service. Value.

Since 1954.

Submitted To:
Ms. Kara M. Anderson
Sabey Corporation
12201 Tukwila International Boulevard, Fourth Floor
Seattle, Washington 98168-5121

By:
Shannon & Wilson, Inc.
400 N 34th Street, Suite 100
Seattle, Washington 98103

21-1-22308-001

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

TABLES

FIGURES

APPENDICES

GEOTECHNICAL REPORT
18TH AVENUE AND CHERRY STREET MEDICAL OFFICE BUILDING
SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of our geotechnical studies for a new 18th Avenue and Cherry Street medical office building in Seattle, Washington. The purpose of this study is to evaluate the subsurface conditions at the proposed building site and to provide geotechnical engineering recommendations to aid in the design and construction of the proposed structure. This report has been prepared in coordination with the Sabey design team, to develop specific recommendations for the appropriate foundation and shoring systems. Our work included review of existing geotechnical information from the previous project (the Bob Hope Hart Institute, 1983), subsurface explorations, laboratory testing, groundwater observation, and engineering studies to develop the recommendations presented in this report. Our work was performed in general accordance with our proposal dated July 21, 2016.

2.0 SITE AND PROJECT DESCRIPTION

The project location is on 18th Avenue between East Cherry Street and East Jefferson Street, as shown on the Vicinity Map, Figure 1. The site currently contains parking lots at the north and south ends and three older residential structures within the central portion of the site. The ground surface slopes up from about elevation 342 feet at the south end to approximately elevation 372 feet at the north end, as shown in the Site and Exploration Plan, Figure 2. The proposed building will generally occupy the entire property with a three-story medical office building, including three levels of underground parking extending down to elevation 320 feet on the south end (approximately 22 feet deep) and to elevation 334 feet on the north end (approximately 38 feet deep), as shown in Generalized Subsurface Profile A-A’ and Generalized Subsurface Profile B-B’, Figures 3 and 4, respectively. Excavations for building foundations will be approximately 3 feet deeper than the lowest garage floor slabs. The project will include installation of temporary excavation shoring along all four sides of the new medical office building structure.

3.0 SUBSURFACE EXPLORATIONS

To characterize subsurface conditions across the project site, we reviewed five existing soil borings drilled for a previously proposed expansion of the Bob Hope International Hart Research Institute in June 1983 (Rittenhouse-Zeman & Associates [RZA], 1983) and performed six additional borings in November 2016 at the approximate locations shown in the Site and Exploration Plan, Figure 2. The recent explorations, designated SW-1 through SW-6, were advanced approximately 35.5 to 56.5 feet below ground surface (bgs). Two groundwater monitoring wells were installed in borings SW-1 and SW-6 at the approximate locations shown in Figure 2. A description of the methodology and procedures used for locating, drilling, and sampling the borings is discussed in Appendix A, Subsurface Exploration. The recent exploration logs are presented in Appendix A as Figures A-2 through A-7. The approximate location of previous soil borings drilled for the Bob Hope International Hart Research Institute proposed expansion, designated B-1 through B-5, are shown in Figure 2. These logs are included in Appendix A as Figures A-8 and A-12.

4.0 LABORATORY TESTING

Laboratory tests were performed on selected soil samples retrieved from the borings. The laboratory testing program included a variety of tests to classify the soils and to provide data for engineering studies. Classification and index laboratory tests included visual classification and tests to determine natural water content and grain size distribution. The results from the laboratory tests are included in Appendix B, Laboratory Testing.

4.1 Water Content Determinations

Water content was determined on selected samples in general accordance with ASTM International (ASTM) D2216, Test Method for Determination of Water (Moisture) Content of Soil and Rock (ASTM, 2010). The water content is shown graphically in the boring logs.

4.2 Grain Size Analyses

The grain size distribution of selected samples was determined in general accordance with ASTM D422, Standard Test Method for Particle-Size Analysis of Soils (ASTM, 2007). Results of these analyses are presented as gradation curves in Appendix B. Each gradation sheet provides the Unified Soil Classification System (USCS) group symbol, the sample description, and water content. The USCS for samples with fewer than 50 percent fines were classified in general accordance with ASTM D2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure) (ASTM, 2009).

5.0 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Local Geology

The project site is located within the Puget Lowland, a structural trough between the Cascade Range and Olympic Mountains. This trough was subjected to several major glaciations during the Pleistocene Epoch. As a result of these glaciations, the Puget Lowlands were filled to significant depths with glacial and nonglacial sediments. Many of these glacial sediments have been glacially overridden and consolidated to dense to very dense or stiff to hard conditions. The last glaciation experienced by the Puget Lowlands, the Vashon Stade, occurred approximately 13,000 years ago. As the glacier advanced southward sediments were deposited at the base of the ice and overridden (glacial lodgement till). Sometimes the glacial sediments were reworked by sub-glacial streams, forming lenses and layers of sorted sediments. As the glacier receded northward, meltwater streams emanating from the glacial ice deposited stratified layers of sand and gravel as well as lacustrine deposits of silty sand and silts. The native soils at the project site consist of Vashon-age glacial lodgement till. These materials have been glacially overridden and have high shear strength and low compressibility.

5.2 Soils

Based on the soils encountered in the borings, the site is underlain by a thin layer of fill over a thick deposit of glacial till. Detailed descriptions of the different soil layers encountered in the subsurface explorations are presented in the boring logs, Figures A-2 through A-12, Appendix A. The site’s subsurface conditions are presented in the Generalized Subsurface Profile A-A’ and Generalized Subsurface Profile B-B’, Figures 3 and 4, respectively.

The site is covered by asphalt on the south and the central portion of the site and crushed rock surfacing on the north portion of the site. The asphalt and crushed rock are underlain by fill to depths ranging from about 3 to 8 feet bgs. In general, the fill consists of very loose to dense, poorly graded sand with silt and gravel to silty sand with gravel. The fill thickness appears to be thinnest at the south portion of the site and thicker towards the north. It is likely that the thickness of fill will vary across the site. The thickness of fill may also be locally greater than 8 feet behind the foundation walls of the existing buildings.

Below the fill, a relatively thick deposit of very dense, silty, gravelly sand (glacial till), was encountered to the bottom of the borings. This glacial till typically has Standard Penetration Test “N” values that exceed 100 blows per foot. The till contains discontinuous lenses of relatively clean, fine to medium sand that contain perched groundwater.

5.3 Local Groundwater

Perched groundwater was evidenced by relatively wet soil encountered during drilling of borings SW-3, SW-4, SW-5, and SW-6 at depths of approximately 37, 3, 15, and 55 feet, respectively. Two wells were installed in borings SW-1 and SW-6 to measure groundwater levels on the south and on the north sections of the project site. Subsequent readings of the well SW-1 indicated no presence of groundwater while readings of the well SW-6 indicated groundwater at approximately 53 feet bgs, as illustrated in the boring logs.

Based on the groundwater observations during drilling, the observed inflow rates and information from the wells installed in borings SW-1 and SW-6, it appears that groundwater inflow will not be a construction issue for this project. Perched groundwater observed in localized zones within glacial till is expected to cause relatively minor and temporary seepage during excavation. This could be addressed with the installation of temporary sumps during site excavation.

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

6.1 General

In general, the soils that are present at the proposed excavation depth consist of very dense, gravelly, silty sand that will provide suitable support for conventional spread footing foundations with relatively high bearing pressures.

The proposed excavation will require the use of temporary shoring to support vertical excavations and maintain structural support of the city property on the north, south, and west sides of the site and the residential properties to the east. We recommend that top-down soil nail wall construction be used as a suitable method of shoring for all sides of the site.

Because it is unlikely that temporary easement for soil nails will be granted by the property owners on the east side of the site, we recommend that the excavation along the east property line be supported using a combination of sloped cuts on the top and vertical soil nail wall at the bottom of excavation. To provide additional temporary shoring support in loose to medium dense fill along the east property line, we recommend the shoring designer use a series of closely-spaced vertical elements installed prior to commencing excavation. This would only be required where small wood-framed residential structures are within 5 feet of the property line.

The following sections present our design recommendations for foundations, excavation, and shoring, as well as other pertinent geotechnical design issues, including lateral resistance and lateral earth pressures, stormwater infiltration, seismic design, wall drainage, floor slab design, fill placement, and construction considerations.

6.2 Foundation Design

Based on the subsurface explorations, very dense, overconsolidated, granular soil underlies the proposed building site at the proposed excavation depth. Spread footing foundations for the proposed building should bear in the very dense soil. If loose or medium dense, wet, disturbed soils develop due to construction activity, these soils should be removed to expose competent native soils consisting of very dense glacial till. The base of all excavations should be reasonably dry and cleared of loose soil at the time of concrete placement. Foundation subgrades should be evaluated by a geotechnical engineer (or representative) from our firm to confirm the presence of competent bearing soil prior to placement of steel and formwork. It is acceptable to place a 3- to 6-inch layer of lean-mix concrete on the subgrade immediately after excavation and approval by the geotechnical engineer to protect it from moisture and disturbance during rebar steel and form installation.

Individual spread footings and continuous strip footings bearing in undisturbed, very dense, native soils may be designed for a maximum allowable bearing pressure of 16,000 pounds per square foot (psf). This allowable bearing pressure requires that the footing bear at least 3 feet below the adjacent ground surface and have a minimum width of 5 feet. Alternatively, an allowable bearing pressure of 10,000 psf may be used for less heavily loaded columns and walls; however, these column footings and continuous wall footings should have minimum widths of 24 and 18 inches, respectively. The recommended minimum footing widths may govern design. The elevation difference of adjacent footings should not be greater than one-half the clear distance between them. Where adjoining, continuous wall footings that are at different elevations, the upper footing should be stepped down to the lower footing. Recommended bearing pressures may be increased by one-third for short-term wind or seismic loading.

6.3 Estimated Settlements

Individual footings designed for the bearing pressures noted above are anticipated to settle less than ½ inch. Differential settlements between footings or along a continuous 20-foot-long portion of a footing may approach ¼ inch if there is some variation between the relative densities of bearing soils. The majority of the settlement is expected to take place as the load is applied during construction.

6.4 Site Preparation and General Excavation

Site preparation should commence by collecting and diverting all sources of surface water flow into temporary storm drainage/treatment system. The site should be cleared of pavements, structures, trees, and vegetation. Excavations will then remove the surface layer of mixed fills that are about 3 to 8 feet thick. No soil contaminants were detected during subsurface explorations; however, we recommend monitoring to evaluate the excavated fills for potential contaminants. After the fills are removed, the remaining excavation will be in dense to very dense glacial till.

Based on the subsurface conditions of the site, we anticipate that the excavations can be accomplished with conventional excavating equipment, such as an excavator equipped with hardened teeth on the bucket. Note that the glacial till on the site will be very dense and relatively slow to excavate. The fill on the site is expected to be medium dense and relatively easy to excavate. While our subsurface explorations did not encounter hazardous materials in the soil, potentially hydrocarbon-impacted soils may exist in the fill.

Vertical elements such as cantilevered soldier piles with 2-inch timber lagging or plywood equivalent may be used to support the upper 5 to 8 feet of fill materials present at the north end of the site and along a portion of the east side near the location of boring SW-3. Cantilevered vertical elements (soldier piles) can be designed using recommended lateral earth pressures provided in Section 6.6. Additional design recommendations for soil nail walls are provided in Section 6.10.

If there is sufficient space available and vertical elements are not used, we recommend that temporary unsupported, open-cut slopes in the loose fill at the top of the proposed excavation be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). The slopes in dense to very dense glacial till may be cut at 1H:1V for slopes less than 30 feet high. Vertical cuts up to 5 feet high may be made in the very dense soils during the shoring wall construction. No vertical cuts should be left unsupported for more than a 24-hour period. If seepage zones are encountered, the maximum height of temporary vertical cuts should be reduced to 4 feet and dewatering requirements should be evaluated by a representative of our firm.

We recommend that all exposed cut slopes be protected with a waterproof covering during periods of wet weather to reduce sloughing and erosion. The cut slopes should be covered with plastic sheeting or lined with a thin layer of shotcrete if the slope is used in conjunction with the soil nail wall installed below the sloped cut.

Excavation of temporary slopes should be made the responsibility of the Contractor. The Contractor is continually at the site and is able to observe the natural conditions of the subsurface materials encountered, including groundwater, and also has responsibility for methods, sequence, and schedule of construction. If instability is detected, slopes should be flattened or shored. Regardless of the construction method used, all shoring and excavation work (and all project work) should be accomplished in compliance with applicable local, state, and federal safety codes.

Except as otherwise designed and/or specifically covered in the contract, the Contractor should be made responsible for control of all surface and groundwater encountered during construction. In this regard, sloping, slope protection, ditching sumps, trench drains, dewatering, and other measures should be employed as necessary to permit proper completion of the work. Perched groundwater was encountered in relatively thin layers of soil within central and east side of the site while drilling the borings. Therefore, we anticipate that groundwater would not present significant construction related problems while excavating the site. The presence of water bearing layers should be evaluated during the excavation process so that temporary sumps can be added as necessary.

6.5 Seismic Design Considerations

6.5.1 Design Ground Parameters

The proposed medical office building will be designed in accordance with the International Building Code (IBC) 2015 (International Code Council, 2015). For the IBC 2015, the seismological inputs are short-period spectral acceleration, S_S, and spectral acceleration at the 1-second period, S_1. The coefficients, S_S and S_1, are for a maximum considered earthquake, which corresponds to a ground motion with a 2 percent probability of exceedance in 50 years, or a 2,475-year return period (with a deterministic maximum cap in some regions). The coefficients are based on regional probabilistic ground motion studies completed in 2008 by the U.S. Geological Survey.

The spectral response acceleration values are scaled by site soil response factors to account for site amplification/damping effects. The site classification determines the site soil response factors. Our analysis of geologic conditions indicates that the building site is underlain by competent glacially overridden sediments and, therefore, the site can be classified as Site Class C. The IBC does not require a site-specific ground motion evaluation for Site Class C sites.

The seismological inputs are short-period spectral acceleration, S_S, and spectral acceleration at the 1-second period, S_1, taken from approved National Earthquake Hazards Reduction Program spectral response acceleration contour map for Class B sites (shown in Figure 1613 in the code). Sites classified as Class B are defined as firm rock having a shear-wave velocity between 2,500 and 5,000 feet per second in the top 100 feet. The seismological inputs are modified for Site Class C. The mapped S_S and S_1 values, site coefficients and design values corresponding to Site Class C are presented in Table 1.

TABLE 1
INTERNATIONAL BUILDING CODE 2015
GROUND MOTION PARAMETERS

S_S (g's)	S_1 (g's)	Site Class	S_{MS} (g's)	S_{M1} (g's)	S_{DS} (g's)	S_{D1} (g's)
1.357	0.525	C	1.357	0.682	0.904	0.455

6.5.2 Earthquake-induced Geologic Hazards

Earthquake-induced geologic hazards that may affect a given site include fault-related ground rupture and liquefaction and associated effects (loss of shear strength, bearing capacity failure, loss of lateral support, ground oscillation, lateral spreading, etc.). The following provides a brief discussion of these hazards.

6.5.2.1 Liquefaction

Soil liquefaction is a phenomenon which occurs in loose, saturated granular soil when the water pressure in the pore spaces increases to a level that is sufficient to separate the soil grains from each other. When a saturated soil experiences partial or full liquefaction, porewater pressures between the soil grains increases, which causes a reduction in the soil's strength and stiffness. As a result, ground settlement, lateral spreading, and landslides may occur. Based on the relatively dense nature of the glacially overridden soils at the site, the gentle topography, and the estimated depth to groundwater, it is our opinion that the risk of liquefaction, settlement, and landsliding at the site is low and, therefore, not considered a design issue for this project.

6.5.2.2 Fault Rupture

The development is located approximately ¼ mile north of the Seattle Fault Zone (SFZ), which extends approximately east-west adjacent to Interstate 90 and has been estimated to range from about 1 to 2 miles wide (Johnson and others, 2004). Radiocarbon dating evidence suggests that the last ground-rupturing earthquake associated with the SFZ occurred about 1,100 years ago and caused nearly 22 feet of permanent vertical displacement across the fault on Bainbridge Island. The rupture history of the fault zone is the subject of current and ongoing research in the scientific community. Preliminary geologic evidence developed to date suggests that the current recurrence rate of large, ground-surface-deforming earthquakes on the Seattle Fault may be on the order of thousands of years. Therefore, it is our opinion that the potential for ground surface fault rupture at the project site is low.

6.6 Lateral Earth Pressures

Lateral earth pressures act on the buried portions of the building walls. For walls allowed to move at least 0.001 times the wall height, we recommend using the static, active lateral earth pressure of 30H as presented in Figure 5. For buried walls that are not allowed to move 0.001 times the wall height (rigid condition), a static, at-rest lateral earth pressure equivalent to 50 pounds per cubic foot (pcf) should be used. If a slope will exist above the top of a foundation wall, 1 pcf should be added the design lateral earth pressure for each degree of slope inclination. These lateral earth pressures are based on the assumption that the wall backfill includes proper drainage (i.e., geocomposite drainage mat installed with shoring wall) so hydrostatic pressures will not build up. Earth pressure recommendations for nearby footings, equipment, or other surcharges are presented in Figure 6.

The total earth pressure should be analyzed for seismic loading conditions using a uniformly distributed pressure of 14H, as shown in Figure 5. The load increase for seismic conditions is consistent with a pseudo-static analysis using the Mononobe-Okabe equation for lateral earth pressures and a horizontal seismic coefficient of 0.28g. This horizontal seismic coefficient is consistent with ground motion criteria in the 2015 IBC and is approximately one-half the peak ground acceleration.

6.7 Lateral Resistance

Lateral forces would be resisted by passive earth pressure against the buried portions of the structure and friction against the bottom of the footings. In our opinion, passive earth pressures developed from compacted granular fill could be estimated using an equivalent fluid unit weight (using Hf Fill criteria) of 300 pcf. This value is based on the assumption that the structures extend at least 1.5 feet below the lowest adjacent exterior grade, are properly drained, and that the backfill around the structure is compacted in accordance with the recommendations for structural fill outlined herein. If footings are cast directly against glacial till, passive earth pressure could be increased to an equivalent fluid unit weight of 450 pcf. Similarly, passive resistance against below-grade retaining walls can be assumed to be equivalent to an equivalent fluid unit weight of 450 pcf. The above equivalent fluid unit weight includes a factor of safety of 1.5 to limit lateral deflection.

We recommend that a coefficient of friction of 0.5 be used between cast-in-place concrete and undisturbed, dense, glacially overridden soil. A coefficient of friction of 0.35 should be used for footings bearing on compacted structural fill. These values include a factor of safety of 1.5.

6.8 Floor Slab

We recommend that floor slabs be supported by glacially overridden soil or compacted imported structural fill placed directly onto these materials. If loose, soft, or otherwise unsuitable soil is encountered, it should be removed and replaced with compacted structural fill. Structural fill should be compacted to a dense, unyielding condition. A modulus of subgrade reaction of 300 pounds per cubic inch may be used to design the slab, assuming dense structural fill or native subgrades are present.

We recommend placing a capillary break consisting of a minimum 6-inch-thick layer of ⅝-inch minus crushed rock or washed pea gravel (⅜-inch to No. 8 sieve size), below the floor slab. This capillary break layer should be hydraulically connected to a sump within the lowest portion of the parking garage. We recommend that a perforated 4-inch-diameter under-slab drain pipe be installed in the capillary break layer to convey any accumulated water to the sump pump. Crushed rock should have less than 3 percent fines passing the No. 200 sieve. The crushed rock should be compacted to a dense and unyielding condition with at least two passes with a vibrating plate compactor or smooth-drum roller. Crushed rock would provide a firmer working surface than washed pea gravel on which to place the slab reinforcement and concrete. Prior to placing pea gravel and/or crushed rock, the exposed subgrade surface should be compacted as needed to achieve a dense, unyielding condition and should be evaluated by the geotechnical engineer. A vapor barrier consisting of 10-mil plastic sheeting should be included above the capillary break in all heated spaces such as offices, maintenance rooms, restrooms, etc., or where floor surfaces will be covered with finishing materials.

6.9 Drainage and Infiltration

We recommend installing a prefabricated geocomposite drainage mat system along the perimeter of the building in conjunction with the shoring wall and an under-slab drainage system to prevent the buildup of hydrostatic pressures. The geocomposite drainage mat behind temporary shoring walls should connect into a tightline, in accordance with manufacturer's instructions. The subdrain system below the flor slab should consist of a perforated, 4-inch-diameter plastic pipe bedded in ⅜-inch to No. 8 size washed pea gravel or ⅝-inch minus crushed gravel. Subdrains under the floor slabs should extend north to south under the center of the building.

Cleanouts should be provided at convenient locations along all drain lines, such as at the building corners. Figure 7 shows typical wall drainage recommendations.

Precipitation water from roof downspouts should be collected into tightlines and routed away from the building using tightline pipes. Downspout water should not be introduced into foundation backfill. Provisions should be made to divert surface water away from structures and prevent it from seeping into the ground adjacent to structures or excavations. Surface water should be collected in catch basins and, along with downspout water, should be conveyed in a nonperforated pipe (tightline) to an approved discharge point.

The site is underlain by a thin layer of fill consisting of very loose to dense, poorly graded sand with silt and gravel to silty sand with gravel (reworked till fill) over a thick deposit of relatively impervious glacial till. Infiltration rates within the glacial till deposit underlying the site would likely be less than 0.2 inch per hour and would not be suitable for use with an infiltration facility. If required by the civil engineer, further exploration and testing could be accomplished to measure the specific infiltration rates within the project area.

6.10 Temporary Shoring

We recommend using a temporary soil nail shoring wall to facilitate excavation of the site. Soil nailing consists of drilling and grouting a series of steel bars or “nails” behind the excavation face and covering the face with temporary or permanent reinforced shotcrete. The placement of relatively closely spaced steel nails in the retained soil mass increases the shear resistance of the soil against rotational sliding, increases the tensile strength of the soil behind potential slip surfaces, and moderately increases shear resistance at a potential slip surface due to the bending stiffness of the nails.

Soil nailing is most effective in dense, granular soils and stiff, low plasticity, fine-grained soil. It is generally not used in loose granular soils, soft cohesive soils, highly plastic clays, or where uncontrolled groundwater exists above the bottom of the excavation. In general, excavation faces with heights of 7 to 9 feet must be able to stand unsupported for 24 hours in order for soil nailing to be cost-effective unless vertical steel elements are installed to provide additional support of the soil face.

Shored excavations will be required along all sides of the site. Proposed excavation depths are about 22 to 38 feet. Soils to be retained mostly consist of dense to very dense, silty, gravelly sand. These soils are suitable for temporary soil nail shoring walls.

As summarized in Section 6.1, we recommend that the excavation along the east property line be supported using a combination of sloped cuts and a series of closely spaced vertical elements (soldier piles) located on the top of vertical soil nail wall. The upper sloped cut and shored sections using vertical elements should be designed as a surcharge load to the vertical soil nail wall system located at the bottom of excavation. We recommend using the static, active lateral earth pressure of 35 pcf for design of temporary shoring consisting of closely spaced vertical elements. Active earth pressures should be adjusted for surcharge loads as recommended in Figure 6.

Construction of soil nail walls involves a top-down procedure that generally includes three steps for each horizontal row of nails: (a) staged excavation, (b) nail installation and select nail testing, and (c) drainage and facing construction. This sequence of staged excavation, nail installation, and drainage/facing construction in horizontal rows is repeated until the excavation and shoring is complete. Soil nails consist of steel bars (typically ¾- to 1⅜-inch-diameter), which are installed by tremie-grouting the nail into a predrilled hole. Soil nails are located in square or rectangular grid patterns and are typically installed at an inclination angle of 15 degrees below horizontal. Drainage is provided behind these walls by placing vertical rows of geosynthetic drainage composites between the grids of soil nails and connecting these to tightlines in the bottom of the wall. Facing typically consists of shotcrete sprayed over a steel mesh (temporary walls) or reinforcing steel bars (permanent walls) on the face of the cut excavation that connects to the soil nails and bearing plates.

In general, the first row of nails is installed not more than 2 to 4 feet bgs, and the bottom row of nails is installed less than 4 feet above the bottom of the cut or excavation. Soil nail lengths typically range from 0.7 to 1.0 times the wall height. For very dense glacially overridden soil that is present at this site, the soil nail lengths likely would be about 0.7 times the wall height.

The shoring designer should be retained to provide the design of the soil nail wall. For soil nail wall design, we recommend the following strength parameters for glacial till, presented in Table 2:

TABLE 2
RECOMMENDED SOIL NAIL WALL DESIGN PARAMETERS

Soil Unit	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Design Pullout Friction (kips/foot)
Glacial Till	40	200	135	5.0

Notes:
Design pullout friction assumes 6-inch-diameter tremie-grouted soil nails.
pcf = pounds per cubic foot
psf = pounds per square foot

Means and methods of constructing the shoring system should be specified by the designer. We recommend that a monitoring program be established to evaluate performance of the shoring wall. This would include deflection monitoring points on the adjacent sidewalks, streets, and the structures on the property line east of the excavation.

6.11 Utility Trenches

Soil used for utility trench backfill should meet the recommended requirements outlined in Section 7.3. The backfill should be placed in lifts not exceeding 4 inches if compacted with hand-operated equipment, or 10 inches if compacted with heavy equipment. Each lift should be compacted to a dense, unyielding condition and to at least 92 percent of Modified Proctor maximum dry density (ASTM D 1557-70) 12 inches or more below the pavement subgrade, and a minimum of 95 percent within 12 inches of the pavement subgrade. We recommend a minimum 2 feet of soil cover over utility pipes and/or conduits, as measured from the pavement subgrade elevation. Catch basins, utility vaults, and other structures installed flush with the pavement surface should be designed to transfer wheel loads to the base of the structure.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Fill Placement, Compaction, and Use of On-site Soils

In areas to be filled, such as beneath foundations, floor slabs, and pavements, the exposed soil surface, after clearing and stripping and prior to any fill placement or foundation or pavement construction, should be compacted using a heavy vibratory roller or backhoe-mounted hydraulic plate compactor, and should be evaluated by an experienced geotechnical engineer probing with a steel T-bar. Where unsuitable soil that is loose, soft, wet, or contains organic material is encountered during the compaction process, it should be removed and replaced with densely compacted structural fill.

Native granular soil in dry conditions and granular on-site fill material without debris, wood, and free of topsoil (abundant organic material) would be suitable for use as structural fill provided the soil is within ±2 percent of its optimum moisture content for compaction. On-site fill soil could be re-used as structural fill provided it is evaluated and approved by a geotechnical engineer.

Imported structural fill soil should consist of a well-graded mixture of sand and gravel; free of organics, debris, and rubbish; and should contain no more than 20 percent fines (material passing the No. 200 mesh sieve, based on the minus ¾-inch fraction). The fines should be nonplastic, and the moisture content of the soil should also be within ±2 percent of its optimum. All structural fill should have a maximum particle size of 3 inches.

Structural fill should be placed in uniform lifts and compacted to a dense and unyielding condition, and to at least 95 percent of the Modified Proctor maximum dry density (ASTM D 1557-70). All subgrades to receive structural fill should be dense and unyielding and should be evaluated by the geotechnical engineer. In general, the thickness of soil layers before compaction should not exceed 10 inches for heavy equipment compactors and 6 inches for hand-operated mechanical compactors. The most appropriate lift thickness should be determined in the field using the Contractor's selected equipment and fill, and verified with in situ soil density testing (nuclear gauge methods). All compacted surfaces should be sloped to drain to prevent ponding. Structural fill operations should be observed and evaluated by an experienced geotechnical engineer or technician.

During wet weather or in wet conditions where control of soil moisture is difficult, structural fill material should consist of clean, granular soil, of which not more than 5 percent by dry weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch sieve. The fines should be nonplastic.

7.2 Excavation Monitoring

We recommend that an optical survey program be implemented to monitor movements during excavation and shoring installation. A preconstruction crack survey of adjacent streets, buildings, and facilities should be completed prior to any excavation or shoring, and monitoring baseline readings should be established before excavation begins. We recommend that optical survey points be established on existing structures located within a distance equal to the height of the wall.

We recommend that optical survey points be established at no more than 20-foot spacing along the top of the shoring wall. Monitoring points should be evaluated on a weekly basis during construction or as excavation progress dictates. If total horizontal movements are observed to be in excess of ½-inch, construction of the soil nail wall should be stopped to determine the cause of the movement and to establish the type and extent of remedial construction. If movements exceed ½-inches, geotechnical engineer and shoring designer should determine the cause of displacement and implement remedial measures required to limit total movement at 1-inch. All earthwork and construction activities must be directed toward immediate implementation of remedial measures to limit deformations to 1-inch. The survey points should be monitored until lateral loads are fully supported by the permanent structure. The top row of soil nails should be recorded for vertical and horizontal movement, and survey points behind the soil nail wall should also be installed in the streets and monitored similarly, as recommended above.

7.3 Wet Weather Earthwork

Wet weather generally begins about mid-October and continues through about May, although rainy periods may occur at any time of year. The soil at the site typically contains sufficient silts and fines to produce an unstable mixture when wet. Such soils are susceptible to changes in water content, and they tend to become unstable and difficult, or impossible, to compact if their moisture content significantly exceeds the optimum. If earthwork at the site continues into the wet season, or if wet conditions are encountered, we recommend the following:

- The ground surface in and surrounding the construction area should be sloped as much as possible to promote runoff of precipitation away from work areas and to prevent ponding of water.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill can be accomplished on the same day. The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, or equivalent, located so that equipment does not traffic over the excavated area. Thus, subgrade disturbance caused by equipment traffic will be minimized.
- Fill material should consist of clean, well-graded, pit-run sand and gravel soils of which not more than 5 percent fines by dry weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve. The gravel content should range between 20 and 60 percent retained on a No. 4 mesh sieve. The fines should be nonplastic.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.

- In-place soils or fill soils that become wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil (see third bullet).
- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.
- We suggest that these recommendations for wet weather earthwork be included in the contract specifications.

7.4 Erosion Control

The Contractor should employ proper erosion control measures during construction, especially if construction takes place during wet weather. Covering work areas, soil stockpiles, or slopes with plastic and using sandbags, sumps, and other measures should be employed as necessary to permit proper completion of the work. Bales of straw, geotextile silt fences, rock-stabilized entrance, wheel wash and, as appropriate, street sweeper, and drain inlet sediment screens should be appropriately located to control soil movement and erosion.

7.5 Obstructions

Buildings previously and currently on site, such as floor slabs and basement walls, could be partially or completely buried. The existing foundations, walls, and slabs should be anticipated and could require special demolition measures during site excavation.

Although boulders were not encountered in the explorations, it has been our experience that cobbles and boulders are commonly encountered in glacial soils. We recommend that contract documents contain an advisory statement that cobbles, boulders, and other obstructions that could be encountered in the mass excavation and soil nail drill holes. The presence of these materials could require installing additional nails or altering construction procedures. The Contractor should be prepared to remove any cobbles, boulders, or other obstructions that protrude into the soil face of the excavation. The void produced by removing face obstructions should be backfilled with shotcrete.

8.0 ADDITIONAL SERVICES

We recommend that Shannon & Wilson, Inc. be retained to review the geotechnical aspects of plans and specifications to determine that they are consistent with our recommendations. In addition, we should be retained to observe the geotechnical aspects of construction, particularly foundation installation, shoring construction, drainage and backfill. Observation will allow us to evaluate the subsurface conditions as they are exposed during construction and to determine that the work is accomplished in accordance with our recommendations and the project specifications.

9.0 LIMITATIONS

This report was prepared for the exclusive use of Sabey Corporation and their design team for specific application to this project. This report should be provided to prospective contractors for information of factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the exploratory borings made for this project are representative of the subsurface conditions through the site; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If conditions different from those described in this report are observed or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between submission of our report and the start of work at the site, or if conditions have changed because of natural forces or construction operations at or near the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations.

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or completing test borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

The scope of our services included no environmental assessment or evaluation regarding the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air at the subject site. Shannon & Wilson, Inc. has qualified personnel to assist you with these services should they be necessary. Shannon & Wilson, Inc. has prepared Appendix C, "Important Information About Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports.

We appreciate the opportunity to work with the Sabey Corporation and their design team on this challenging project. Please feel free to contact me at mwp@shanwil.com or (206) 695-6875.

SHANNON & WILSON, INC.

MARTIN W. PAGE
STATE OF WASHINGTON
30308
REGISTERED PROFESSIONAL ENGINEER

Martin Page
2/8/17

Martin W. Page, PE, LEG
Geotechnical Engineer
Vice President

MXR:MWP/mxr

10.0 REFERENCES

ASTM International (ASTM), 2007, Standard test method for particle-size analysis of soils, D422-63(2007)e2: West Conshohocken, PA., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 – D5876, 8 p., available: www.astm.org.

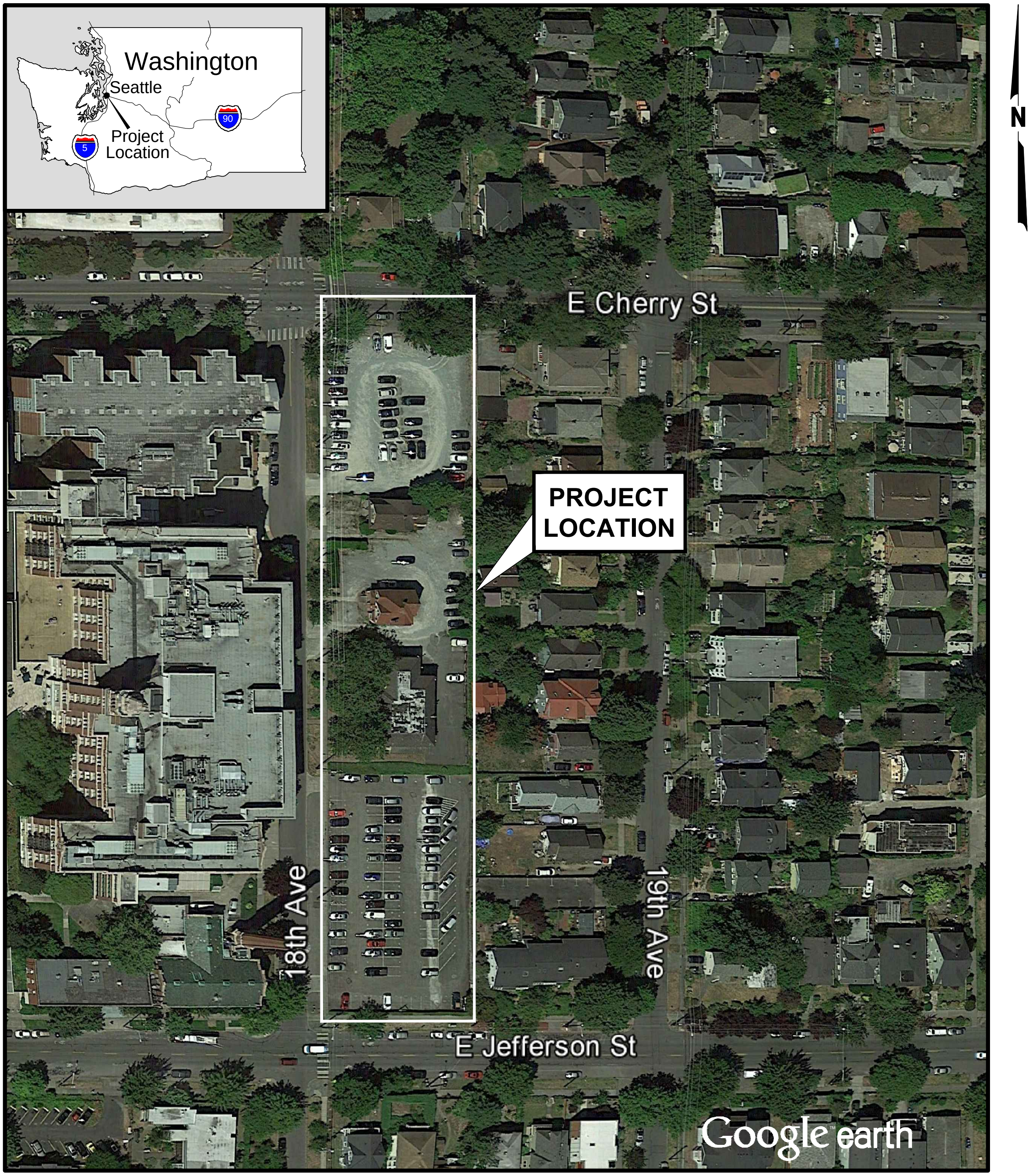
ASTM International (ASTM), 2009, Standard practice for description and identification of soils (visual/manual procedure), D2488-09a: West Conshohocken, PA., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 – D5876, 12 p., available: www.astm.org.

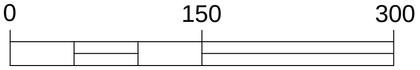
ASTM International (ASTM), 2010, Standard test methods for laboratory determination of water (moisture) content of soil and rock by mass, D2216-10: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 7 p., available: www.astm.org.

International Code Council, Inc., 2015, International building code: Country Club Hills, Ill., International Code Council, Inc., 690 p.

Johnson, S.Y., Blakely, R.J., Brocher, T.M., and others, 2004, Fault number 570, Seattle fault zone, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults.

Rittenhouse-Zeman & Associates (RZA), 1983, Geotechnical exploration and engineering report, The Bob Hope International Hart Research Institute, 528 18th Avenue, Seattle, Washington: Report prepared by Rittenhouse-Zeman & Associates, Bellevue, Washington, for HDI Architects, P.O. Box 4087, Bellevue, Washington.





Approximate Scale in Feet

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

18th Avenue & Cherry Street
Medical Office Building
Seattle, Washington

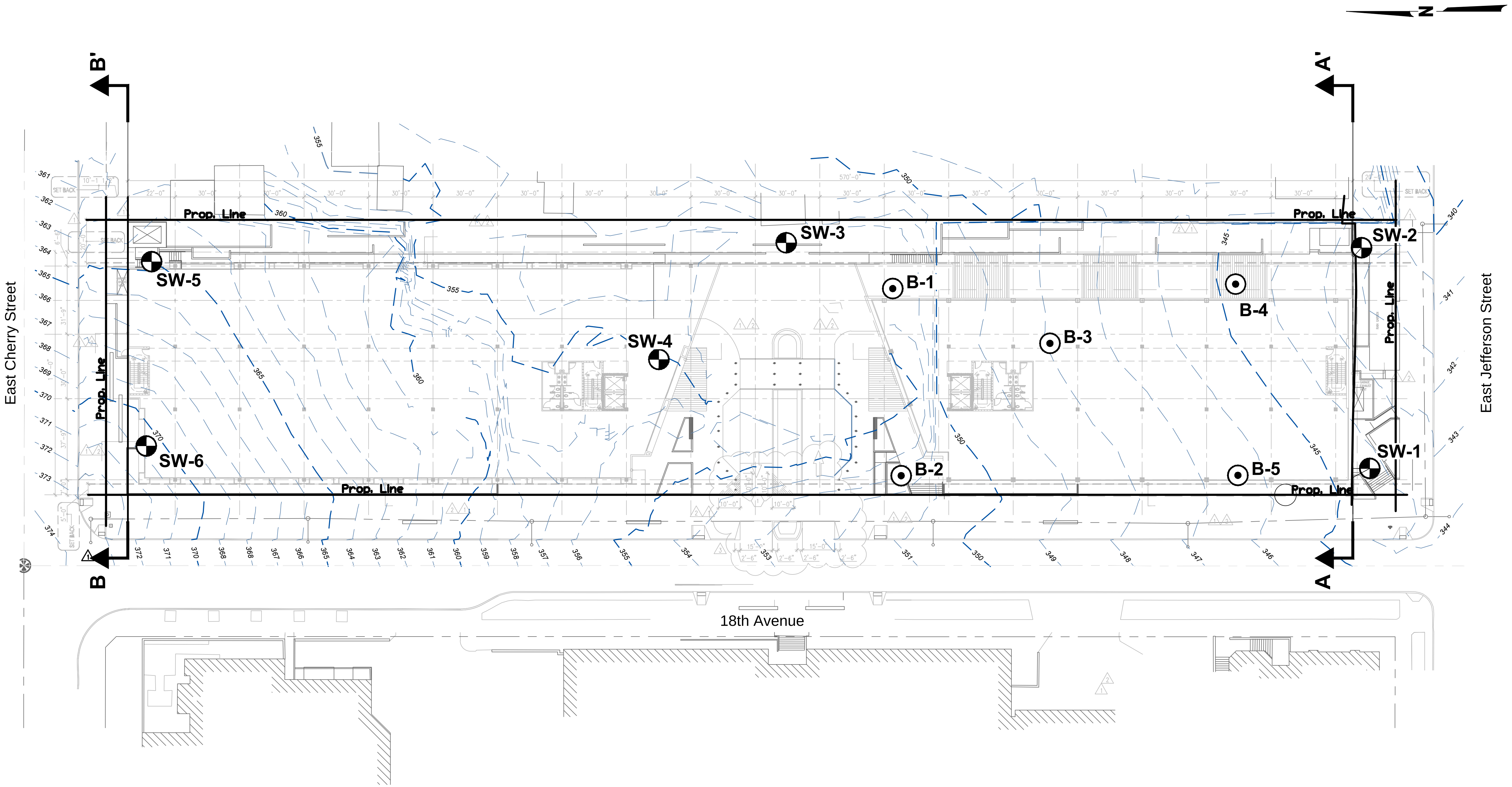
VICINITY MAP

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

Filename: J:\211\22308\001\21-1-22308-001 Fig 1 - Vicinity Map.dwg Date: 02-07-2017 Login: SAC



0 50 100

Scale in Feet

Contour Interval = 1 Foot

LEGEND

SW-1 ⊕	Boring Designation and Approximate Location (Shannon & Wilson, November 2006)
B-1 ⊙	Boring Designation and Approximate Location (Rittenhouse-Zeman & Associates, June 1983)
A	Generalized Subsurface Profile (See Figures 3 and 4)

NOTE

Figure adapted from file, *Overall Site Plan.dwg*, Page A1.1 by Collins Woerman Architects. Original survey performed using old City of Seattle Datum. 9.7 feet has been added to all elevations to adjust to NAVD88 Vertical Datum.

18th Avenue & Cherry Street
Medical Office Building
Seattle, Washington

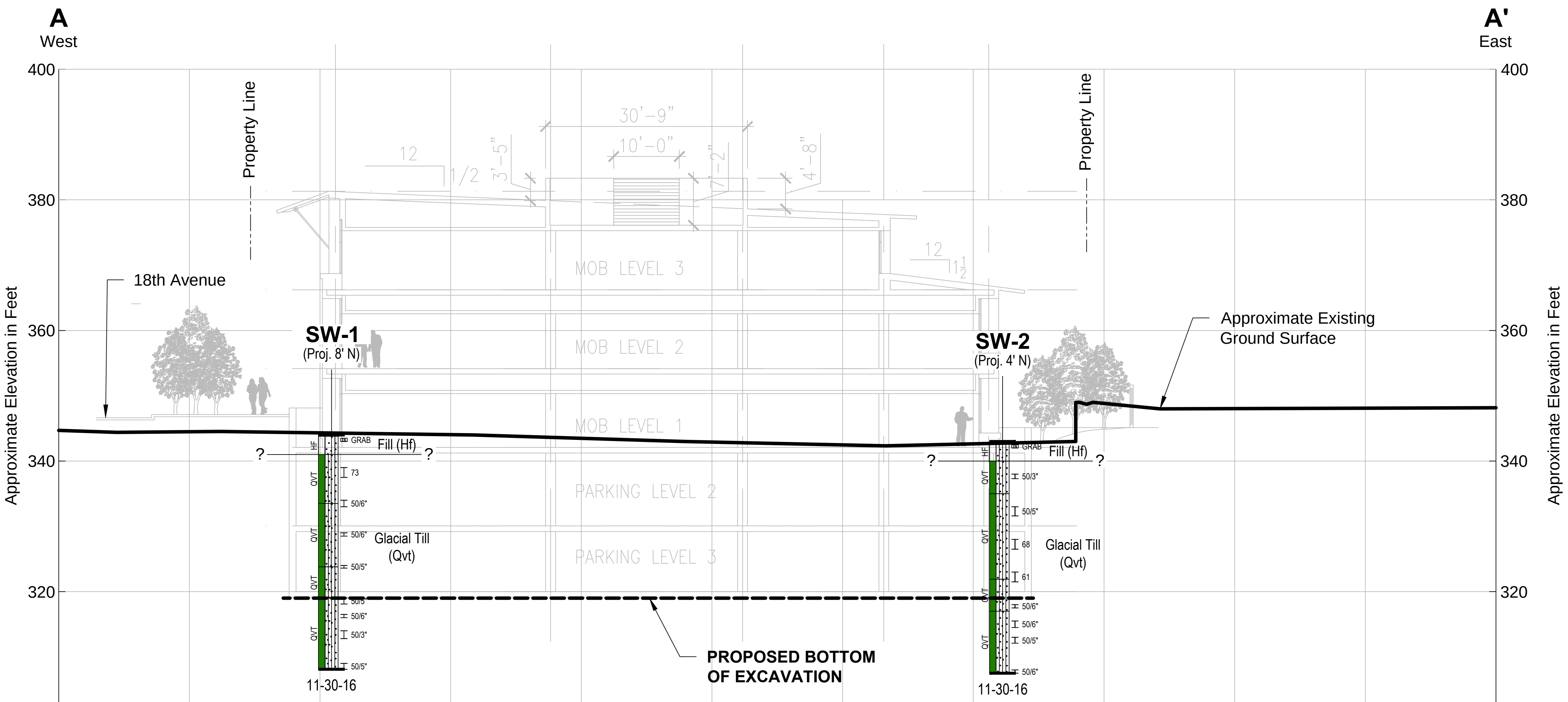
SITE AND EXPLORATION PLAN

February 2017
21-1-22308-001

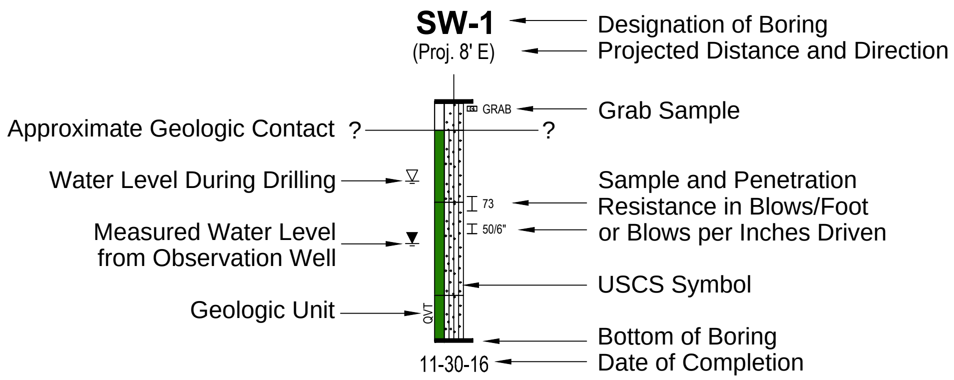
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

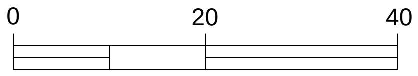
FIG. 2

Filename: J:\211\22308\001\21-1-22308-001 Plan and Profiles.dwg Layout: Plan Date: 02-07-2017 Login: SAC



LEGEND





Scale in Feet
Vertical = Horizontal

NOTES

1. Figure adapted from file, *Overall Site Plan.dwg*, Page A1.1 by Collins Woerman Architects.

2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

18th Avenue & Cherry Street
Medical Office Building
Seattle, Washington

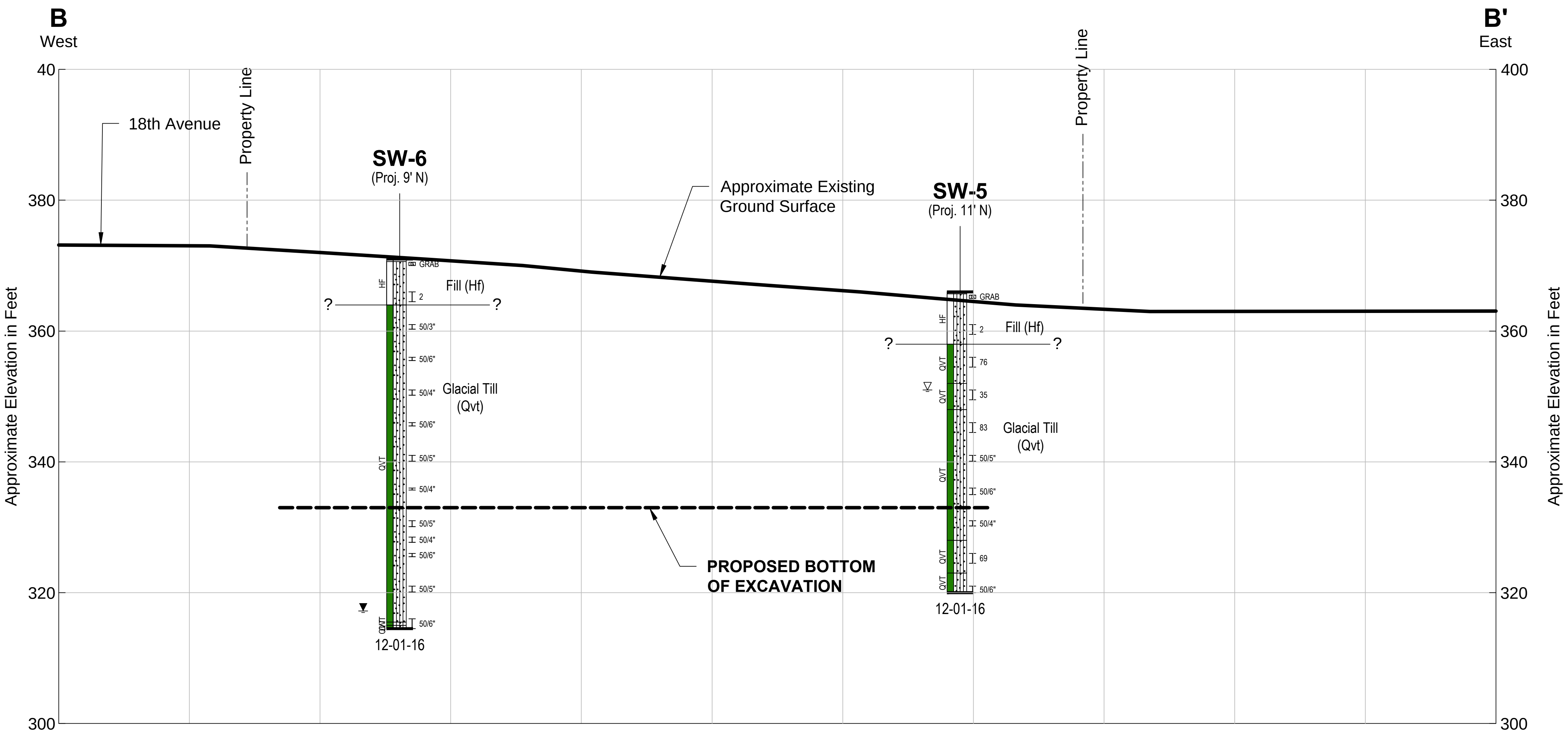
GENERALIZED SUBSURFACE PROFILE A-A'

February 2017 21-1-22308-001

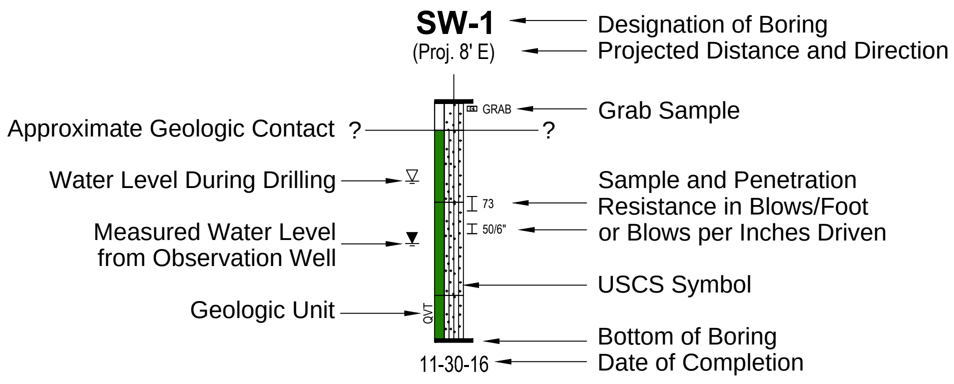
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

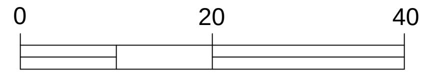
FIG. 3

Filename: J:\211\22308\001\21-1-22308-001 Plan and Profiles.dwg Layout: A-A' Date: 02-07-2017 Login: SAC



LEGEND





Scale in Feet

Vertical = Horizontal

NOTES

1. Figure adapted from file, *Overall Site Plan.dwg*, Page A1.1 by Collins Woerman Architects.
2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

18th Avenue & Cherry Street
Medical Office Building
Seattle, Washington

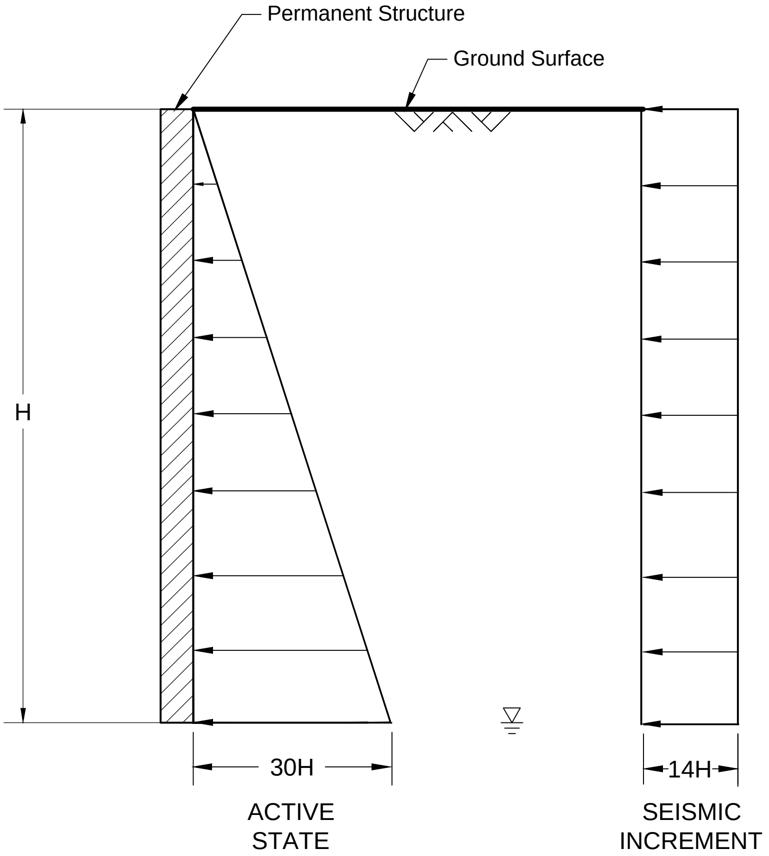
GENERALIZED SUBSURFACE PROFILE B-B'

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 4

Filename: J:\211\22308\001\21-1-22308-001 Plan and Profiles.dwg Layout: B-B' Date: 02-07-2017 Login: SAC



Not to Scale

NOTES

1. All pressures shown are in units of pounds per square foot per foot of wall.
2. The lateral pressures shown above assume drained conditions behind the wall.
3. Earth pressures assume a horizontal backslope. If a sloping ground surface results, the earth pressures should be increased depending on the sloping angle.
4. Appropriate surcharge should be included. See Figure 66.
5. Seismic increment was developed based on a horizontal seismic acceleration of 0.28g.

18th Avenue & Cherry Street
Medical Office Building
Seattle, Washington

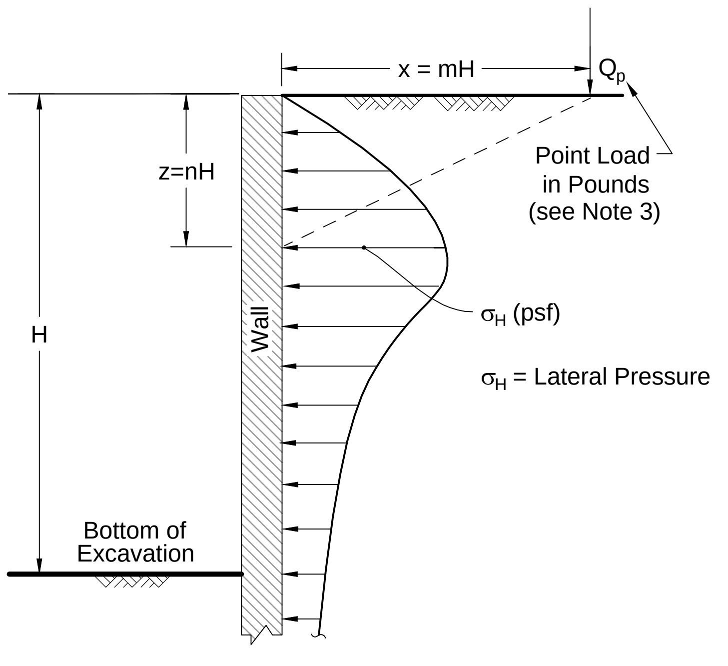
RECOMMENDED LATERAL EARTH PRESSURES

February 2017
21-1-22308-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 5

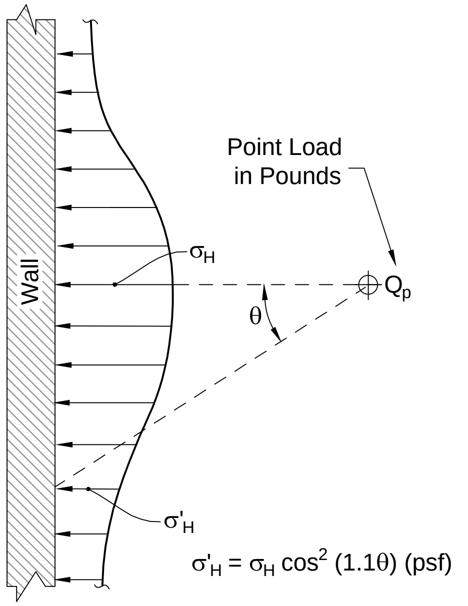
File: J:\211\22308\001\21-1-22308-001 Fig 5 - Recommended Earth Pressures.dwg Date: 02-07-2017 Author: SAC



ELEVATION VIEW

For m ≤ 0.4: $\sigma_H = 0.28 \frac{Q_p}{H^2} \frac{n^2}{(0.16 + n^2)^3}$ (psf) (see Note 3)

For m > 0.4: $\sigma_H = 1.77 \frac{Q_p}{H^2} \frac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)

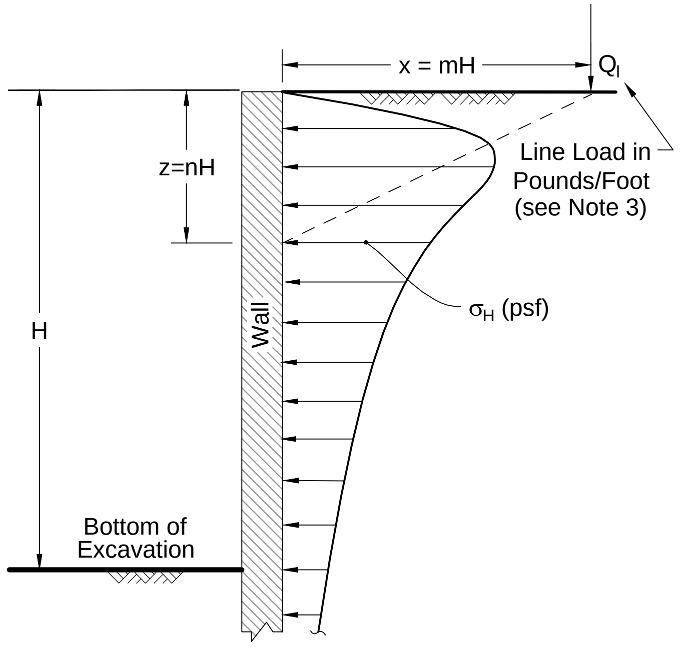


$\sigma'_H = \sigma_H \cos^2 (1.1\theta)$ (psf)

PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.2, 1986)



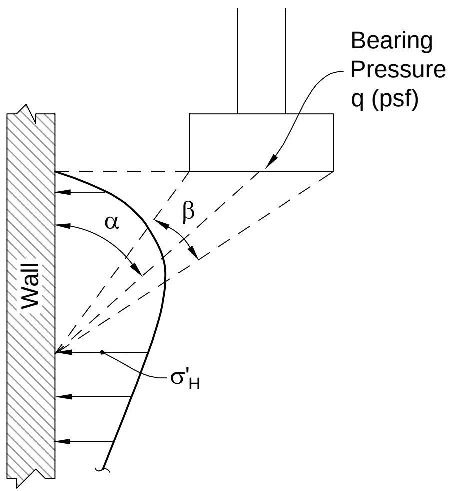
ELEVATION VIEW

For m ≤ 0.4: $\sigma_H = 0.20 \frac{Q_l}{H} \frac{n}{(0.16 + n^2)^2}$ (psf) (see Note 3)

For m > 0.4: $\sigma_H = 1.28 \frac{Q_l}{H} \frac{m^2 n}{(m^2 + n^2)^2}$ (psf)

B) LATERAL PRESSURE DUE TO LINE LOAD i.e. NARROW CONTINUOUS FOOTING PARALLEL TO WALL

(NAVFAC DM 7.02, 1986)

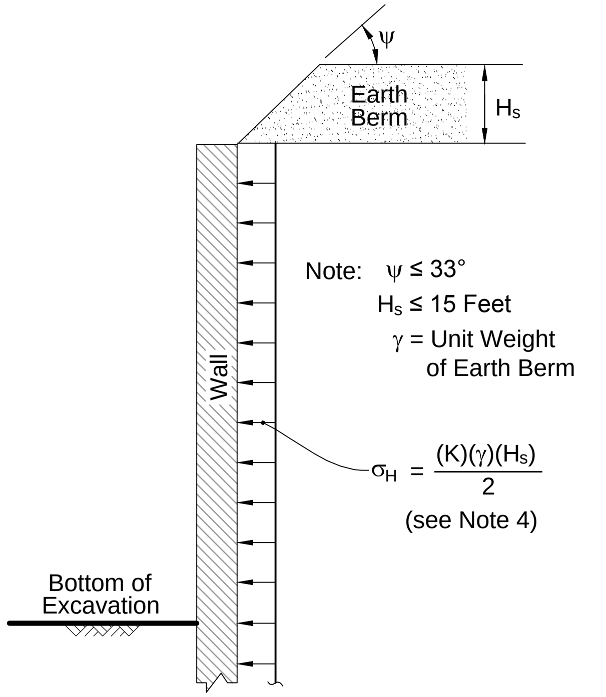


in radians

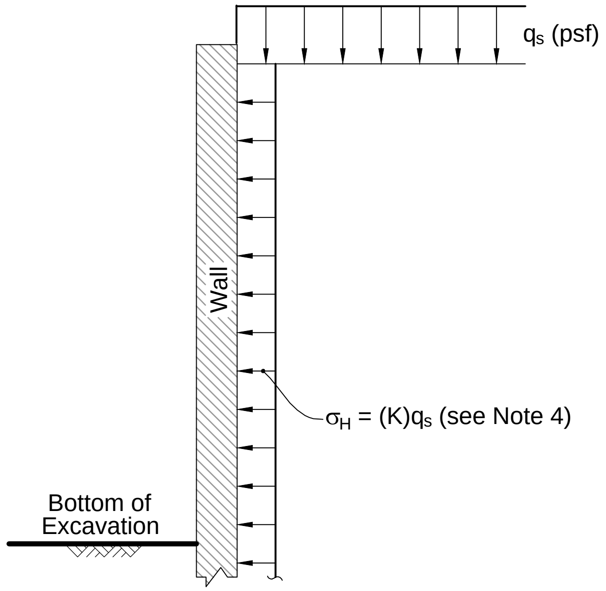
$\sigma_H = \frac{2q}{\pi} (\beta - \sin \beta \cos 2\alpha)$ (psf)

C) LATERAL PRESSURE DUE TO STRIP LOAD

(derived from Fang, *Foundation Engineering Handbook*, 1991)



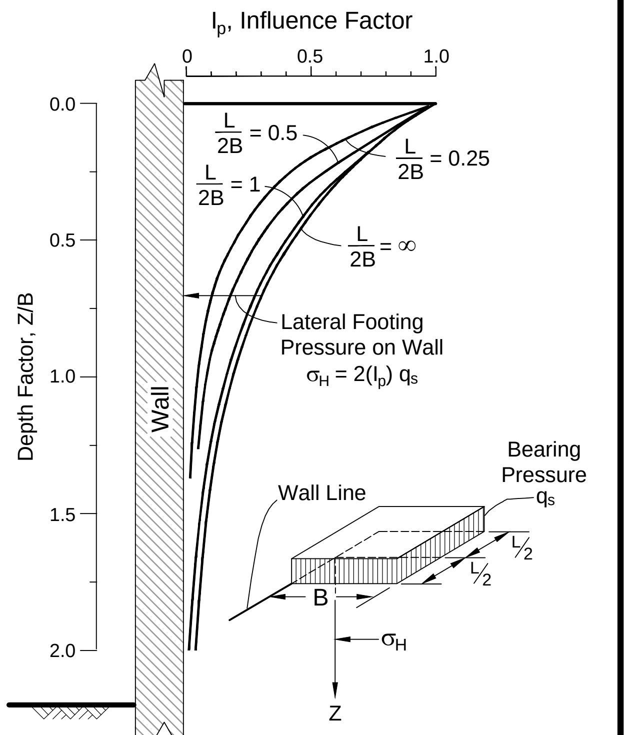
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE

(derived from Poulos and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE TO ADJACENT FOOTING
(see Notes 5 and 6)

(derived from NAVFAC DM 7.02, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that m ≤ 0.4, it may be more appropriate to model the actual load distribution (i.e., Detail E) or use more rigorous analysis methods.
4. Use a K_a value of K_a = 0.27.
5. The stress is estimated on the back of the wall at the center of the length, L, of loading.
6. The estimated stress is based on a Poisson's ratio of 0.5.
7. For areas where fill will be placed immediately behind and above the rop elevation of the wall, Diagram D can be used to determine loads on the wall. For narrow fills adjacent to the wall, Diagram C can be used.

18th Avenue & Cherry Street
Medical Office Building
Seattle, Washington

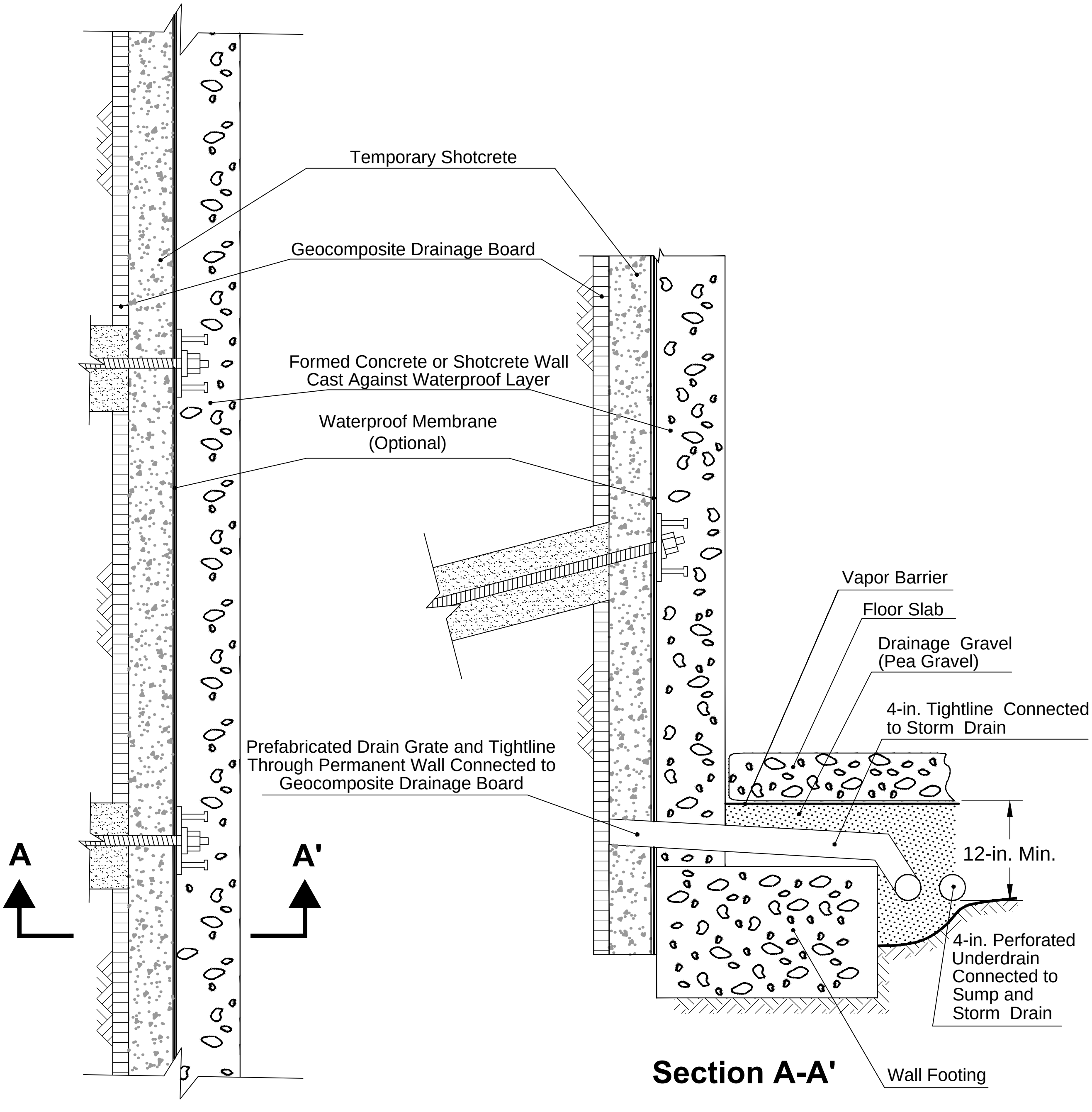
RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 6

File: J:\211\22308\001\21-1-22308-001 Fig 6 - Surcharge Loading.dwg Date: 02-07-2017 Author: SAC



Section A-A'

PLAN VIEW

Not to Scale

NOTES

1. Wall drainage material: Geodrain, Miradrain 6000 or equivalent drainage composite.
2. Clean-outs should be provided in the underdrain system.
3. Wall drain tightline to be separate from sub-slab drain system.

18th Avenue & Cherry Street
Medical Office Building
Seattle, Washington

TYPICAL WALL DRAINAGE

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 7

Filename: J:\211\22308\001\21-1-22308-001 Fig 7 - Wall Drainage.dwg Layout: Layout1 Date: 02-07-2017 Login: SAC

APPENDIX A

SUBSURFACE EXPLORATION

APPENDIX A

SUBSURFACE EXPLORATION

TABLE OF CONTENTS

FIGURES

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1] All percentages are by weight of total specimen passing a 3-inch sieve.
[2] The order of terms is: *Modifying Major with Minor*.
[3] Determined based on behavior.
[4] Determined based on which constituent comprises a larger percentage.
[5] Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer: 140 pounds with a 30-inch free fall.
Rope on 6- to 10-inch-diam. cathead
2-1/4 rope turns, > 100 rpm

NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.

Sampler: 10 to 30 inches long
Shoe I.D. = 1.375 inches
Barrel I.D. = 1.5 inches
Barrel O.D. = 2 inches

N-Value: Sum blow counts for second and third 6-inch increments.
Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND	
Fine	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.)
Medium	#40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.)
Coarse	#10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL	
Fine	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.)
Coarse	3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

▧	Bentonite Cement Grout	⁘	Surface Cement Seal
▨	Bentonite Grout	■	Asphalt or Cap
▦	Bentonite Chips	▨	Slough
⁘	Silica Sand	◫	Inclinometer or Non-perforated Casing
▤	Perforated or Screened Casing	▮	Vibrating Wire Piezometer

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1] Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2] Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

SOIL DESCRIPTION AND LOG KEY

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 1 of 3

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL		TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	●	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP	○	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	●	Silty Gravel; Silty Gravel with Sand
			GC	▨	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	⁘	Well-Graded Sand; Well-Graded Sand with Gravel
			SP	⁘	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	▥	Silty Sand; Silty Sand with Gravel
			SC	▨	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	▥	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	▨	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	☰	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	▥	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	▨	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	▨	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	〰	Peat or other highly organic soils (see ASTM D4427)

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

SOIL DESCRIPTION
AND LOG KEY

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 2 of 3

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASTICITY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20
High	It takes considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

---

[1] Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

[2] Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

SOIL DESCRIPTION AND LOG KEY

February 2017
21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 3 of 3

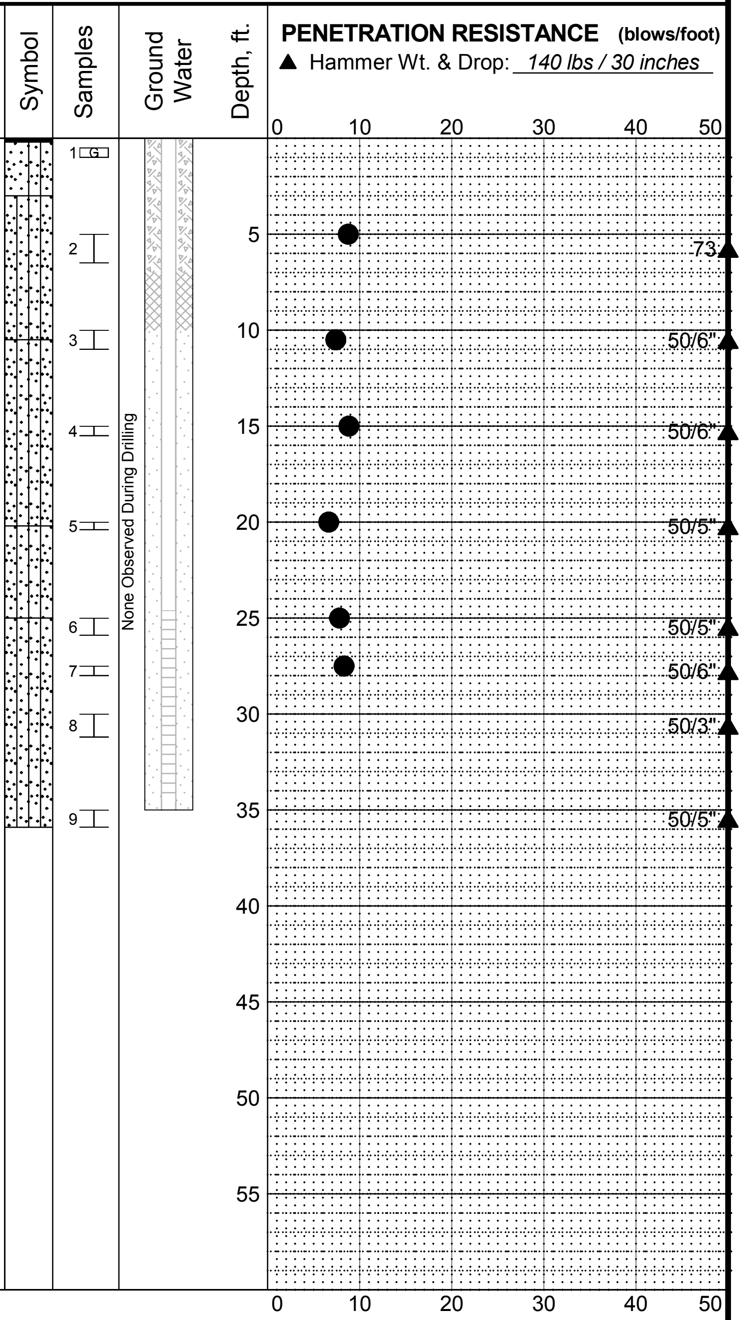
Total Depth:	*35.9 ft.*	Northing:		Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*9 in.*
Top Elevation:	*~ 344 ft.*	Easting:		Drilling Company:	*Holocene Drilling*	Rod Diam.:	*MWJ*
Vert. Datum:		Station:		Drill Rig Equipment:	*Mobile B-61*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.
Asphalt pavement.	0.2
Medium dense to dense, brown-gray, *Poorly Graded Sand with Silt and Gravel (SP-SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines; trace of roots. Fill (Hf)	3.0
Very dense, gray, *Silty Sand with Gravel (SM) and Poorly Graded Sand with Silt and Gravel (SP-SM)*; moist to wet; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	10.5
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	20.2
Very dense, gray, *Silty Sand with Gravel (SM) and Poorly Graded Sand with Silt and Gravel (SP-SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	25.0
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM);* moist; subrounded gravel; fine to coarse sand; nonplastic fines; poorly graded sand with silt and gravel from 30.8 to 31 feet. Quaternary Till (Qvt)	35.9

BOTTOM OF BORING
COMPLETED 11/30/2016
NO GROUNDWATER MEASURED ON
12/30/2016



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

*	Sample Not Recovered	⬚	Well Screen and Sand Filter
G	Grab Sample	▧	Bentonite-Cement Grout
⌶	2.0" O.D. Split Spoon Sample	⊠	Bentonite Chips/Pellets
		▨	Bentonite Grout
		▼	Ground Water Level in Well

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

LOG OF BORING SW-1

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2

REV 3 - Approved for Submittal

Log: MXR Rev: MXR Typ: LKN

MASTER_LOG_E 21-22308.GPJ SHAN_WIL.GDT 2/8/17

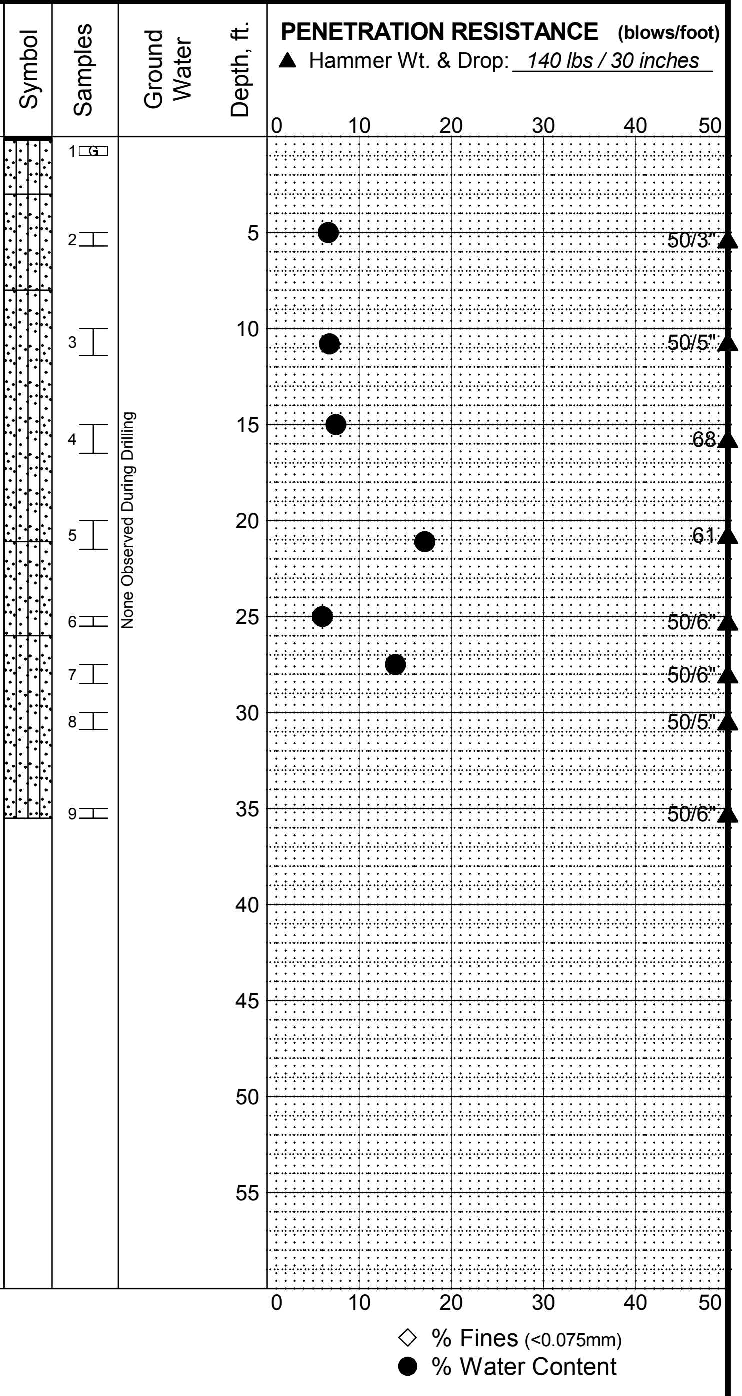
Total Depth: 35.5 ft.
Top Elevation: ~ 343 ft.
Vert. Datum:
Horiz. Datum:
Northing:
Easting:
Station:
Offset:
Drilling Method: Hollow Stem Auger
Drilling Company: Holocene Drilling
Drill Rig Equipment: Mobile B-61
Other Comments:
Hole Diam.: 9 in.
Rod Diam.: MWJ
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.
Asphalt pavement.	0.2
Medium dense, gray, *Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Fill (Hf)	3.0 8.0
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	
Very dense, gray, *Silty Sand with Gravel (SM) and Poorly Graded Sand with Silt and Gravel (SP-SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	21.1
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	26.0
Very dense, gray, *Silty Sand with Gravel (SM) and Poorly Graded Sand with Silt and Gravel (SP-SM)*; moist to wet; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	35.5

BOTTOM OF BORING
COMPLETED 11/30/2016



LEGEND

* Sample Not Recovered
G Grab Sample
⌶ 2.0" O.D. Split Spoon Sample

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

LOG OF BORING SW-2

February 2017
21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3

REV 3 - Approved for Submittal

Log: MXR Rev: MXR Typ: LKN

MASTER_LOG_E 21-22308.GPJ SHAN_WIL.GDT 2/8/17

Total Depth: 46.5 ft.
Top Elevation: ~ 354 ft.
Vert. Datum:
Horiz. Datum:
Northing:
Easting:
Station:
Offset:
Drilling Method: Hollow Stem Auger
Drilling Company: Holocene Drilling
Drill Rig Equipment: Mobile B-61
Other Comments:
Hole Diam.: 9 in.
Rod Diam.: MWJ
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

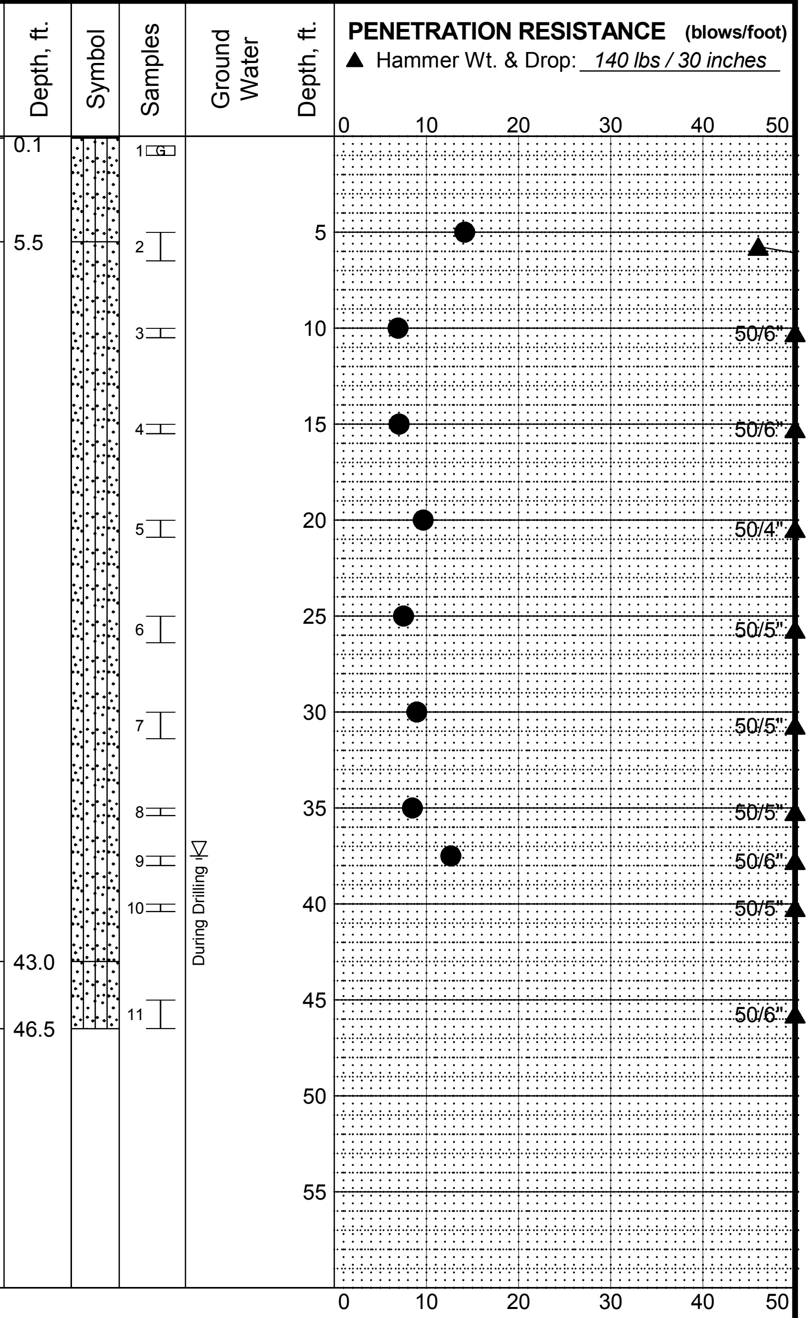
Asphalt pavement.

Medium dense, dark brown-gray, *Silty Sand with Gravel (SM)*; moist; trace of construction debris; subrounded to subangular gravel; fine to coarse sand; nonplastic fines.
Fill (Hf)

Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist to wet; subrounded gravel; fine to coarse sand; nonplastic fines; observed wet, *Poorly Graded Sand with Silt and Gravel (SP-SM)* from 37.0' to 38.0' bgs;
Quaternary Till (Qvt)

Very dense, gray, *Silty Sand with Gravel (SM) and Poorly Graded Sand with Silt and Gravel (SP-SM)*; wet; subrounded gravel; fine to coarse sand; nonplastic fines; observed moist, *Silty Sand with Gravel (SM)* from 46.0' to 46.5' bgs;
Quaternary Till (Qvt)

BOTTOM OF BORING
COMPLETED 11/30/2016



LEGEND

* Sample Not Recovered
G Grab Sample
⌶ 2.0" O.D. Split Spoon Sample
∇ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

LOG OF BORING SW-3

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

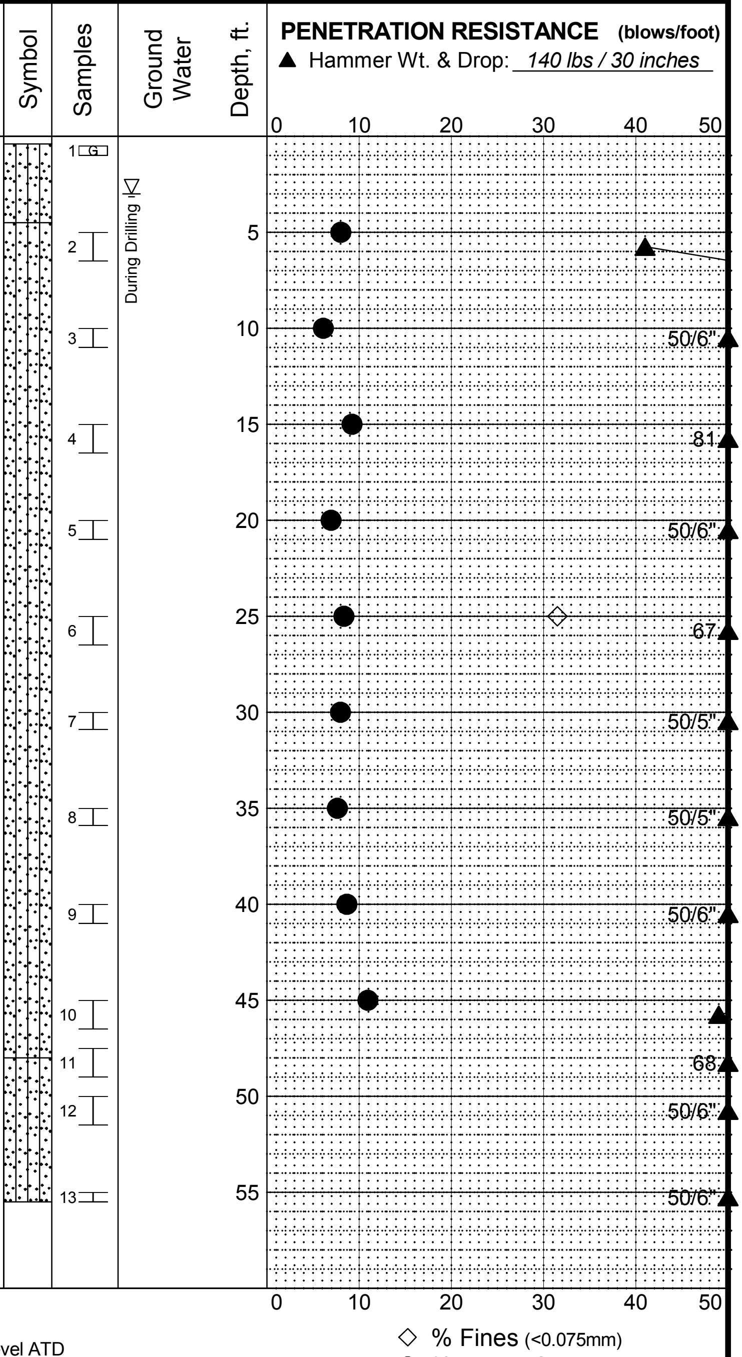
FIG. A-4

Total Depth: 55.5 ft.
Top Elevation: ~ 355 ft.
Vert. Datum:
Horiz. Datum:
Northing:
Easting:
Station:
Offset:
Drilling Method: Hollow Stem Auger
Drilling Company: Holocene Drilling
Drill Rig Equipment: Mobile B-61
Other Comments:
Hole Diam.: 9 in.
Rod Diam.: MWJ
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.
Gravel surfacing.	0.4
Medium dense, light brown, *Silty Sand with Gravel (SM)*; wet; subrounded to subangular gravel; fine to coarse sand; nonplastic fines. Fill (Hf)	4.5
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines; observed wet, *Poorly Graded Sand with Silt and Gravel (SP-SM)* from 47.5' to 48.0' bgs. Quaternary Till (Qvt)	48.0
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	55.5
BOTTOM OF BORING COMPLETED11/30/2016	



LEGEND

* Sample Not Recovered
G Grab Sample
⌶ 2.0" O.D. Split Spoon Sample
⊻ Ground Water Level ATD

◇ % Fines (<0.075mm)
● % Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

LOG OF BORING SW-4

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-5

REV 3 - Approved for Submittal

MASTER_LOG_E 21-22308.GPJ SHAN_WIL.GDT 2/8/17 Log: MXR Rev: MXR Typ: LKN

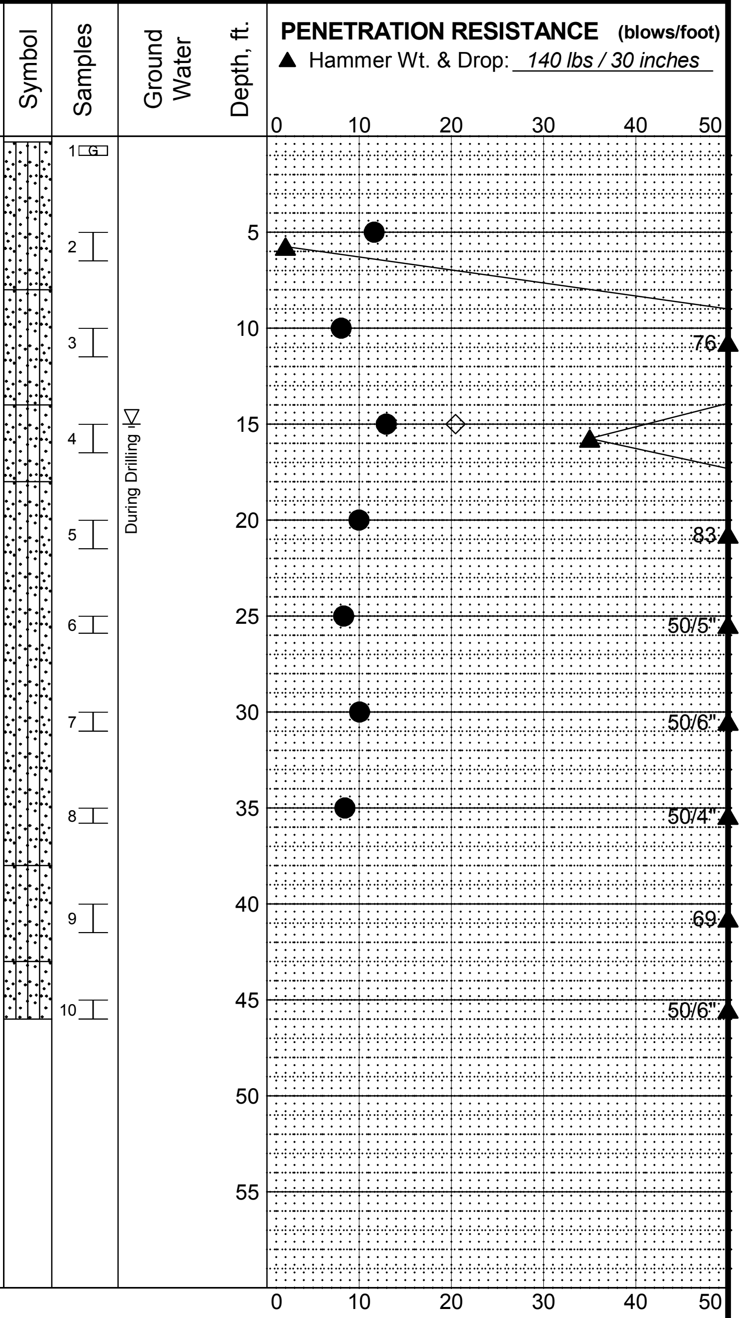
Total Depth: 46 ft.
Top Elevation: ~ 366 ft.
Vert. Datum:
Horiz. Datum:
Northing:
Easting:
Station:
Offset:
Drilling Method: Hollow Stem Auger
Drilling Company: Holocene Drilling
Drill Rig Equipment: Mobile B-61
Other Comments:
Hole Diam.: 9 in.
Rod Diam.: MWJ
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.
Gravel surfacing.	0.3
Very loose, light brown and gray, *Silty Sand with Gravel (SM)*; moist; subrounded to subangular gravel; fine to coarse sand; nonplastic fines. Fill (Hf)	8.0
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	14.0
Dense, gray, *Silt Sand with Gravel (SM)*; wet; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	18.0
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	38.0
Very dense, gray, *Silty Sand with Gravel (SM) and Poorly Graded Sand with Silt and Gravel (SP-SM)*; wet; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	43.0
Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist to wet; subrounded gravel; fine to coarse sand; nonplastic fines. Quaternary Till (Qvt)	46.0

BOTTOM OF BORING
COMPLETED 12/1/2016



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

- * Sample Not Recovered
- G Grab Sample
- ⌶ 2.0" O.D. Split Spoon Sample
- ⊻ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

LOG OF BORING SW-5

February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-6

REV 3 - Approved for Submittal

MASTER_LOG_E 21-22308.GPJ SHAN_WIL.GDT 2/8/17
Log: MXR Rev: MXR Typ: LKN

Total Depth: 56.5 ft.
Top Elevation: ~ 371 ft.
Vert. Datum:
Horiz. Datum:

Northing:
Easting:
Station:
Offset:

Drilling Method: Hollow Stem Auger
Drilling Company: Holocene Drilling
Drill Rig Equipment: Mobile B-61
Other Comments:

Hole Diam.: 9 in.
Rod Diam.: MWJ
Hammer Type: Automatic

SOIL DESCRIPTION

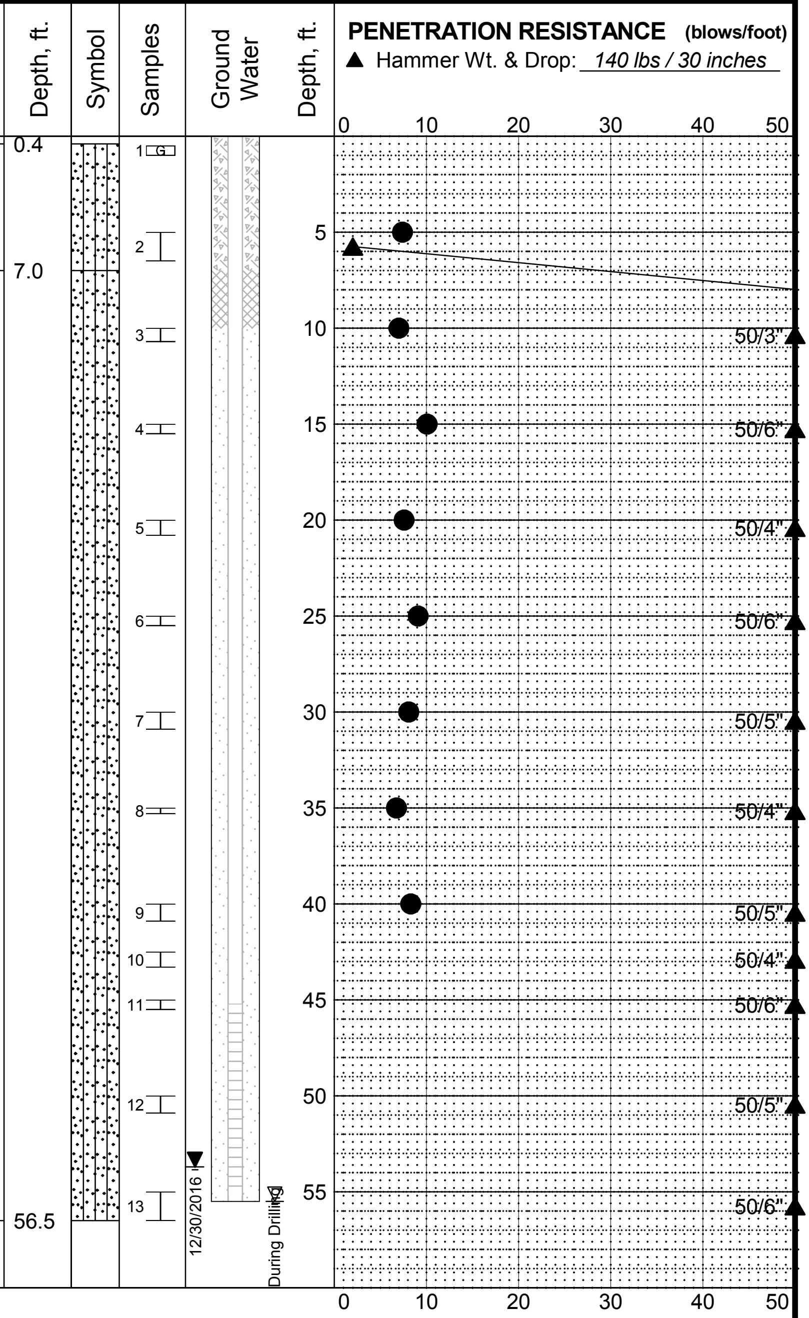
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Gravel surfacing.

Very loose to medium dense, dark gray to gray, *Silty Sand with Gravel (SM)*; moist; subrounded to subangular gravel; fine sand; nonplastic fines.
Fill (Hf)

Very dense, gray, *Silty Sand (SM) and Silty Sand with Gravel (SM)*; moist; subrounded gravel; fine to coarse sand; nonplastic fines; observed wet, *Poorly Graded Sand with Silt and Gravel (SP-SM)* from 55.5' to 56.0' bgs.
Quaternary Till (Qvt)

BOTTOM OF BORING
COMPLETED 12/1/2016



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

* Sample Not Recovered	⊡ Well Screen and Sand Filter
G Grab Sample	▧ Bentonite-Cement Grout
I 2.0" O.D. Split Spoon Sample	⊠ Bentonite Chips/Pellets
	▨ Bentonite Grout
	▽ Ground Water Level ATD
	▼ Ground Water Level in Well

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

LOG OF BORING SW-6

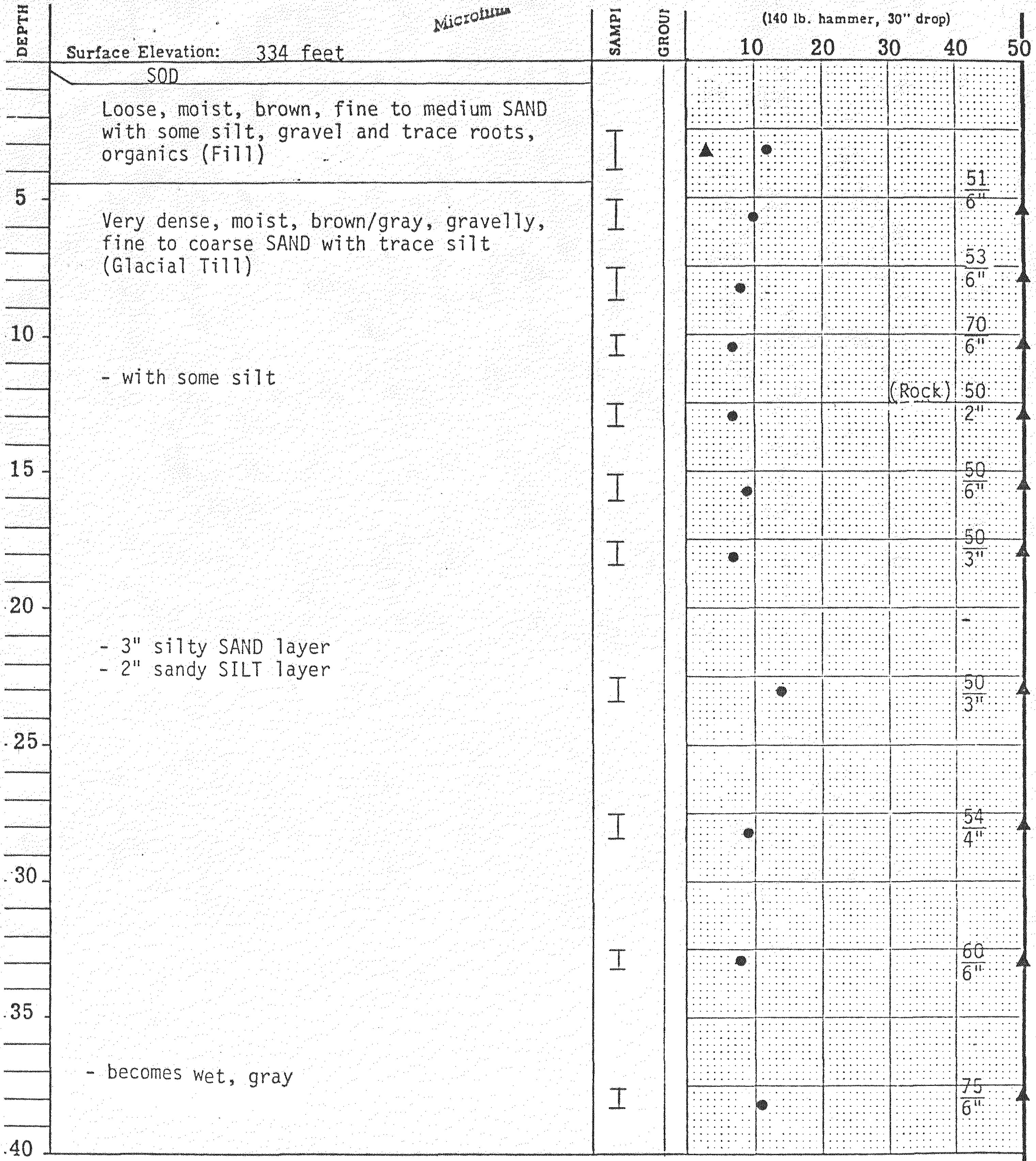
February 2017 21-1-22308-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-7

REV 3 - Approved for Submittal

Log: MXR Rev: MXR Typ: LKN



Surface Elevation: 334 feet

SOD

Loose, moist, brown, fine to medium SAND with some silt, gravel and trace roots, organics (Fill)

Very dense, moist, brown/gray, gravelly, fine to coarse SAND with trace silt (Glacial Till)

- with some silt

- 3" silty SAND layer
- 2" sandy SILT layer

- becomes wet, gray

LEGEND

I	2.0" O.D. split spoon sampler	•	Sample not recovered
II	3.0" O.D. undisturbed sampler	▮	Piezometer tip
P	Sampler pushed	▽	Water level

Atterberg limits: ├─●─┤ ← Liquid limit ← Natural water content ← Plastic Limit

● % Water Content

Bob Hope International Heart Research Inst.

W-4011

LOG OF BORING NO. B-1

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

FIG. A-8
Sheet 1 of 2

DEPTH,

Surface Elevation:

Microfilm

SAMPLI

GROUN

Very dense, wet, brown/gray, silty fine to coarse SAND with some gravel

45

-becomes gray with some silt

50

55

Boring terminated at 53.5 feet

Completed 6 June 1983

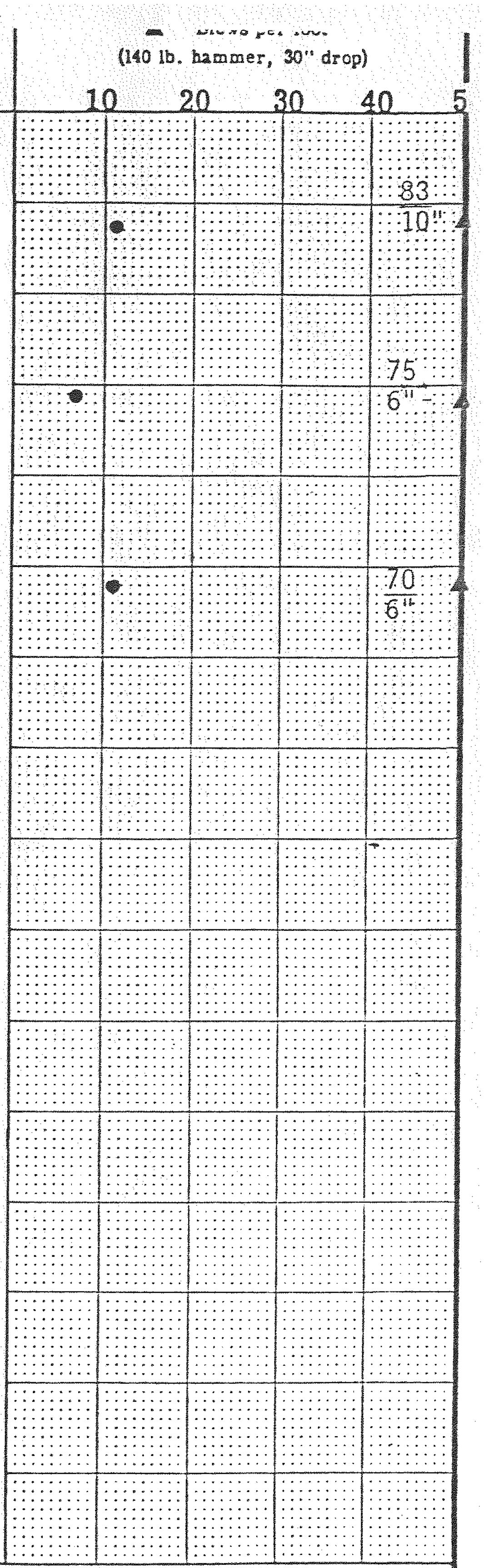
60

65

70

75

80



LEGEND

I 2.0" O.D. split spoon sampler

II 3.0" O.D. undisturbed sampler

P Sampler pushed

* Sample not recovered

Piezometer tip

▽ Water level

Atterberg limits: ├─●─┤ ← Liquid limit

← Natural water content

← Plastic Limit

● % Water Content

Bob Hope International Heart Research Inst.

LOG OF BORING NO. B-1(cont.)

W-4011

RITTENHOUSE-ZEMAN & ASSOC.

SOILS ENGINEERING AND GEOLOGY

FIG. A-8

Sheet 2 of 2

DEPTH

Surface Elevation: 334 Feet ..crofilm

SOD

Medium dense, moist, brown, fine to medium SAND with some silt, gravel (Fill)

5

Very dense, damp, brown/gray, fine to medium SAND with some silt, gravel (Glacial Till)

10

- becomes moist, fine to coarse SAND with some silt, gravel and very silty zones

15

- becomes gravelly

20

- 1" fine sandy SILT layer

- becomes moist to wet, silty

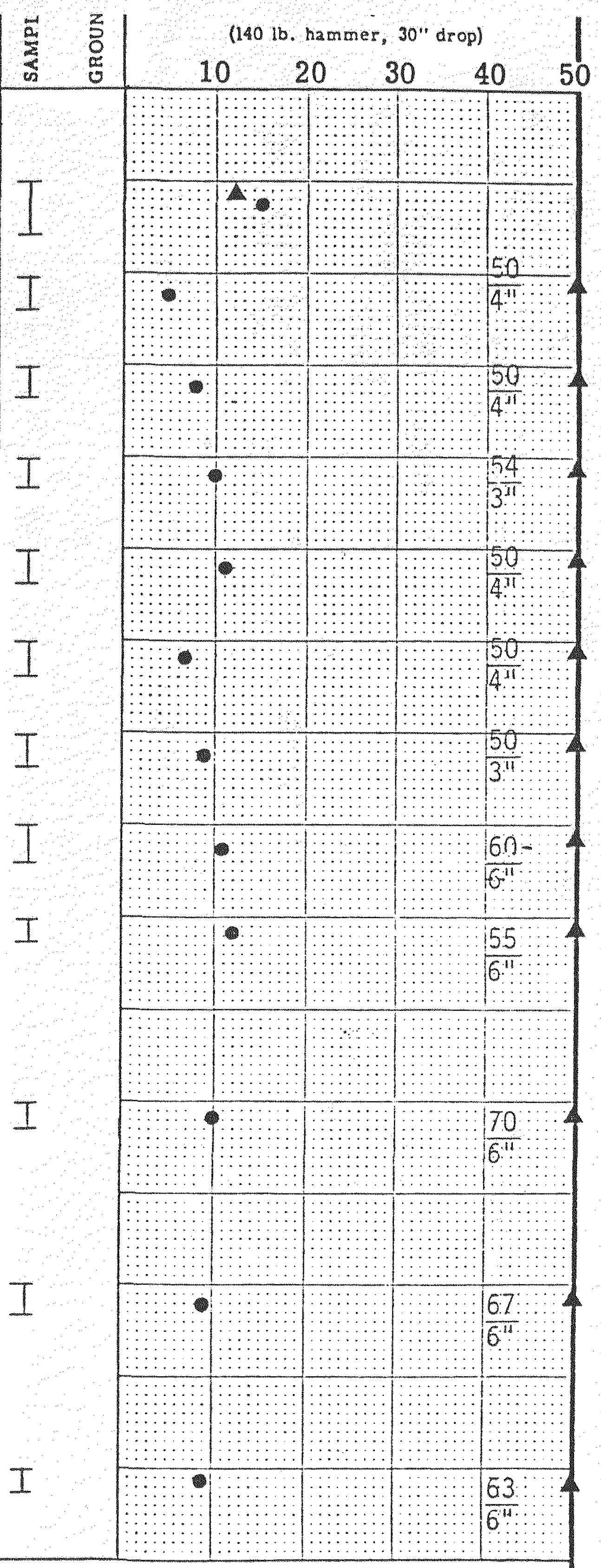
25

- decreasing gravel content

30

35

40



LEGEND

[2.0" O.D. split spoon sampler
I 3.0" O.D. undisturbed sampler
P Sampler pushed
• Sample not recovered
▮ Piezometer tip
▽ Water level
Atterberg limits: ├─●─┤ ← Liquid limit; ← Natural water content; ← Plastic Limit

● % Water Content

Bob Hope International Heart
Research Institute

LOG OF BORING NO. B-2
W-4011

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

FIG. A-9
Sheet 1 of 2

DEPTH,

Microfilm

Surface Elevation:

SAMPLE

GROUND

Very dense, moist to wet, gray, silty, gravelly, fine to medium SAND (Glacial Till)

I

45

I

50

I *

55

Boring terminated at 53.0 feet

Completed 6 June 1983

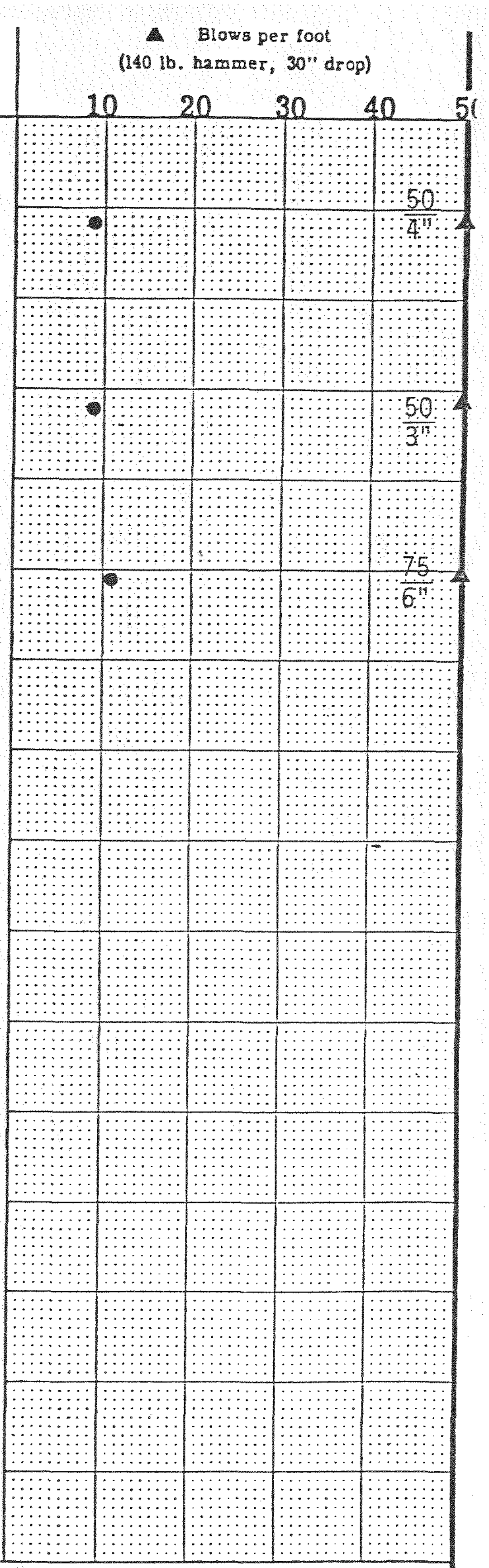
60

65

70

75

80



LEGEND

I 2.0" O.D. split spoon sampler

II 3.0" O.D. undisturbed sampler

P Sampler pushed

* Sample not recovered

Piezometer tip

▽ Water level

Atterberg limits: ← Liquid limit

Natural water content

Plastic Limit

● % Water Content

Bob Hope International Heart Research Institute

LOG OF BORING NO. B-2 (cont.)

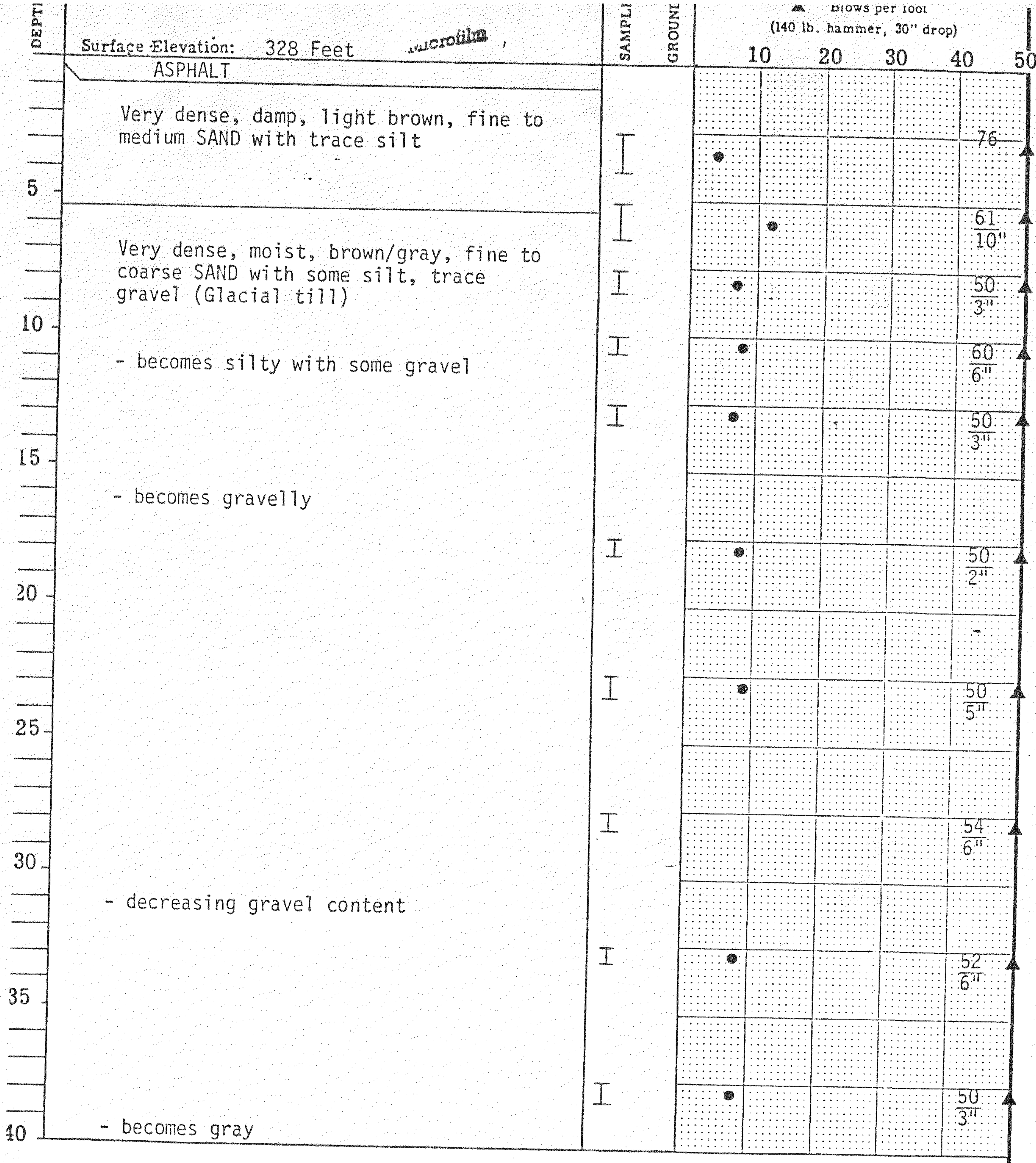
W-4011

RITTENHOUSE-ZEMAN & ASSOC.

SOILS ENGINEERING AND GEOLOGY

FIG. A-9

Sheet 2 of 2



Surface Elevation: 328 Feet

ASPHALT

Very dense, damp, light brown, fine to medium SAND with trace silt

Very dense, moist, brown/gray, fine to coarse SAND with some silt, trace gravel (Glacial till)

- becomes silty with some gravel

- becomes gravelly

- decreasing gravel content

- becomes gray

LEGEND

[2.0" O.D. split spoon sampler
I 3.0" O.D. undisturbed sampler
P Sampler pushed
* Sample not recovered
▮ Piezometer tip
▽ Water level
Atterberg limits: ├●┤ ← Liquid limit
← Natural water content
← Plastic Limit

● % Water Content

Bob Hope International Heart
Research Institute

LOG OF BORING NO.B-3

W-4011

RITTENHOUSE-ZEMAN & ASSOC.

SOILS ENGINEERING AND GEOLOGY

FIG. A-10
Sheet 1 of 2

DEPTH

Surface Elevation:

SAMPL

GROUNI

- 45

Very dense, moist, gray, silty fine to medium SAND with trace gravel (Glacial till)

I

I

- 50

Boring terminated at 48.5 feet

Completed 8 June 1983

- 55

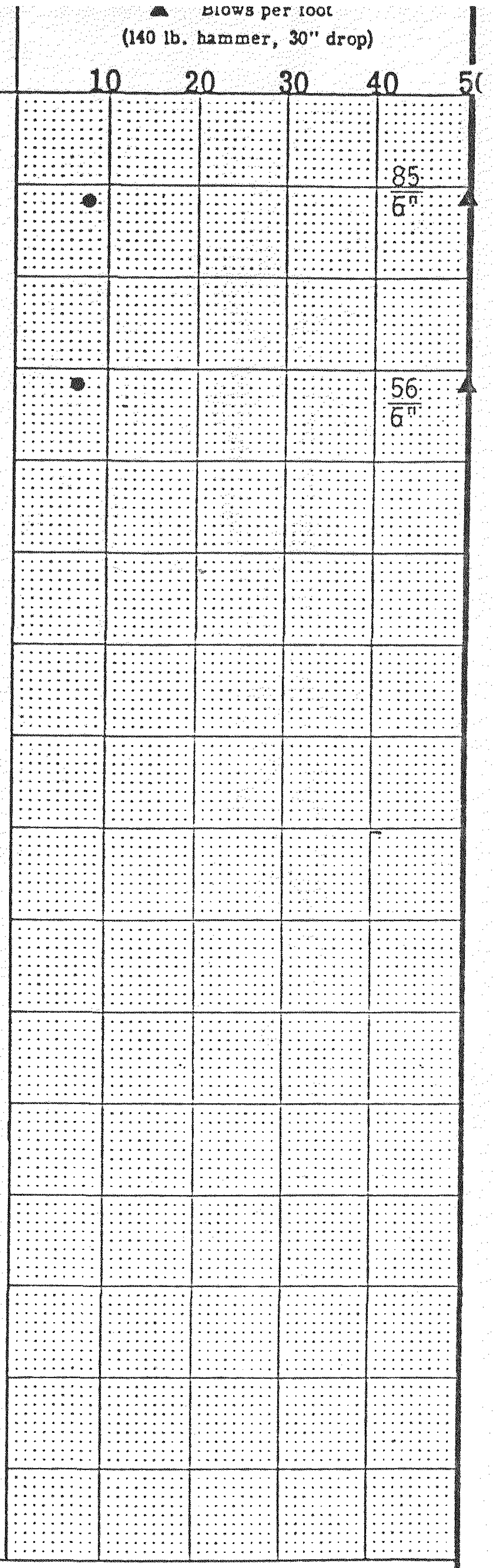
- 60

- 65

- 70

- 75

- 80



● % Water Content

LEGEND

I 2.0" O.D. split spoon sampler

II 3.0" O.D. undisturbed sampler

P Sampler pushed

* Sample not recovered

Piezometer tip

▽ Water level

Atterberg limits: ├─●─┤ ← Liquid limit

← Natural water content

← Plastic Limit

Bob Hope International Heart
Research Institute

LOG OF BORING NO. B-3 (cont.)
W-4011

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

FIG. A-10
Sheet 2 of 2

DEPTI

Surface Elevation: 325 Feet

ASPHALT

Very dense, damp, light brown, fine to medium SAND with trace silt, gravel

5

Very dense, damp, brown, fine to coarse SAND with some gravel, trace silt (Glacial till)

10

- becomes wet with some gravel, silt and occasional clay and sandy silt lenses

15

- becomes moist

20

- becomes moist, brown/gray, gravelly fine to coarse sand with some silt

25

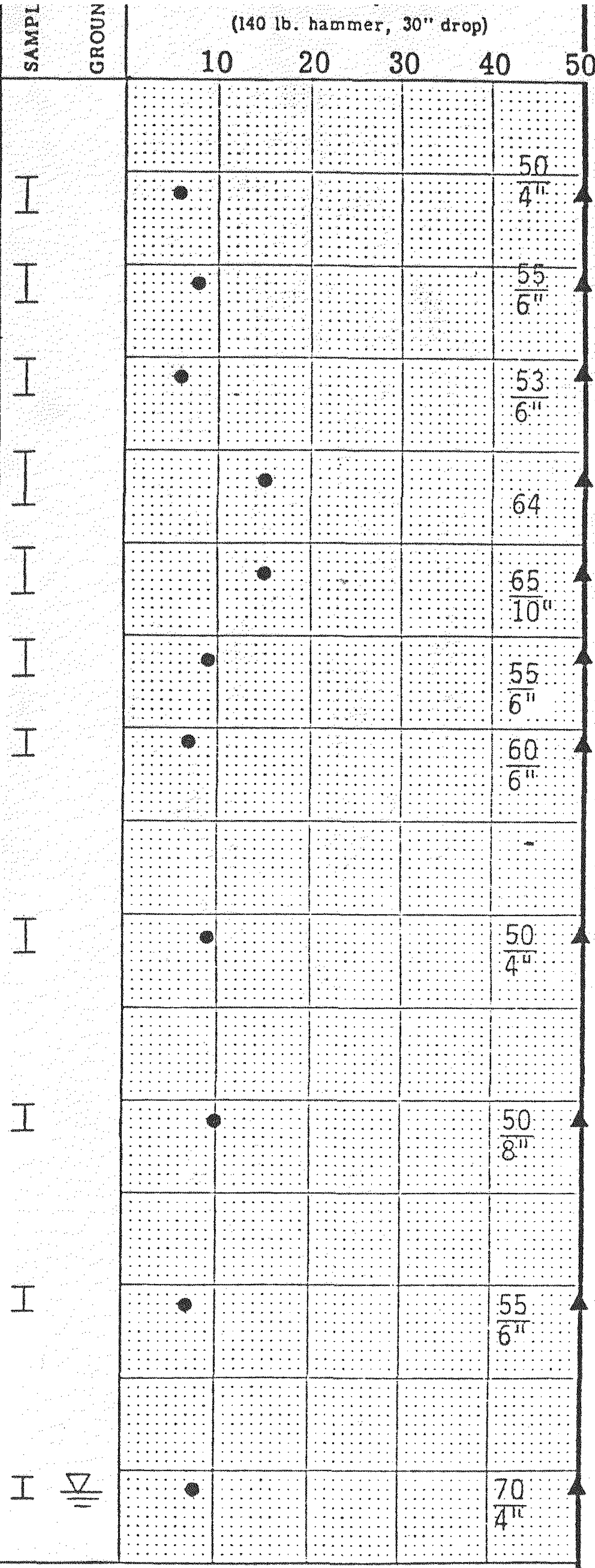
30

- becomes silty with some gravel

35

- becomes gray, decreasing silt content

ł0



LEGEND

2.0" O.D. split spoon sampler
3.0" O.D. undisturbed sampler
Sampler pushed
tterberg limits: ├●┤ ← Liquid limit
Natural water content
Plastic Limit

• Sample not recovered
Piezometer tip
▽ Water level

● % Water Content

Bob Hope International Heart
Research Institute

LOG OF BORING NO. B-4
W-4011

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

FIG. A-11
Sheet 1 of 2

DEPTH,

Surface Elevation:

Microfilm

SAMPLE

GROUND

Very dense, wet to saturated, gray, silty, gravelly, fine to coarse SAND (Glacial till)

I

45

Boring terminated at 43.0 feet

Completed 7 June 1983

50

55

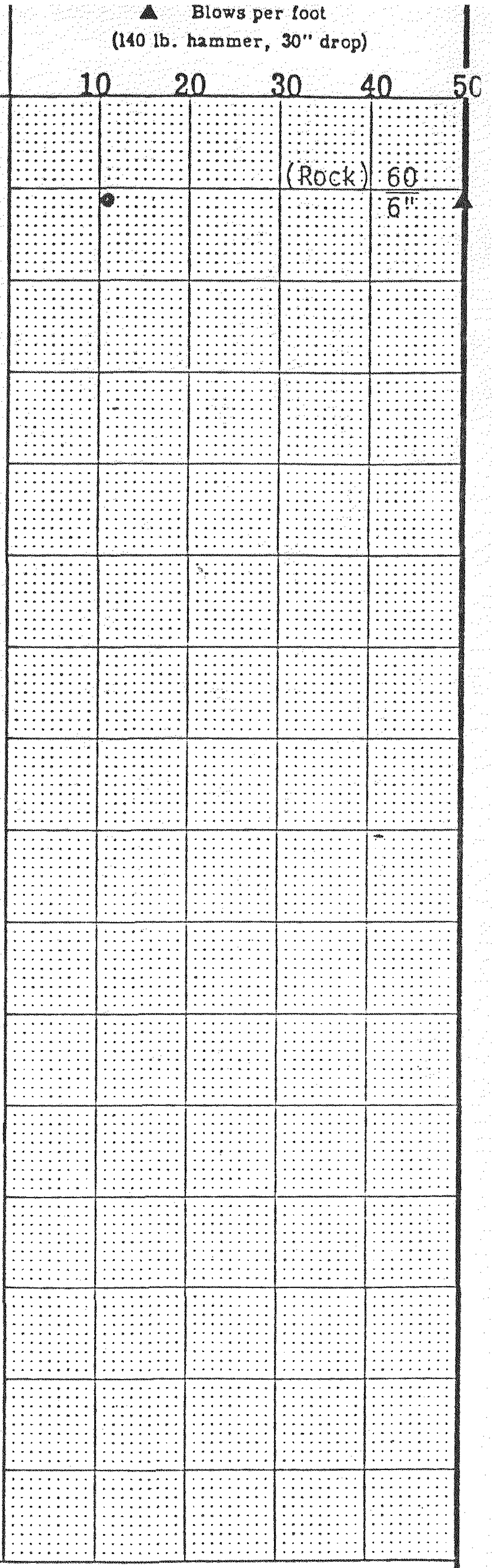
60

65

70

75

80



● % Water Content

LEGEND

I 2.0" O.D. split spoon sampler

II 3.0" O.D. undisturbed sampler

P Sampler pushed

* Sample not recovered

Piezometer tip

∇ Water level

Atterberg limits: ├─●─┤ ← Liquid limit

Natural water content

Plastic Limit

Bob Hope International Heart
Research Institute

LOG OF BORING NO. B-4 (cont.)

W-4011

RITTENHOUSE-ZEMAN & ASSOC.

SOILS ENGINEERING AND GEOLOGY

FIG. A-11

Sheet 2 of 2

DEPTH

Surface Elevation: 320 Feet

ASPHALT

Microfilm

Very dense, damp, light brown, fine to medium SAND with trace silt

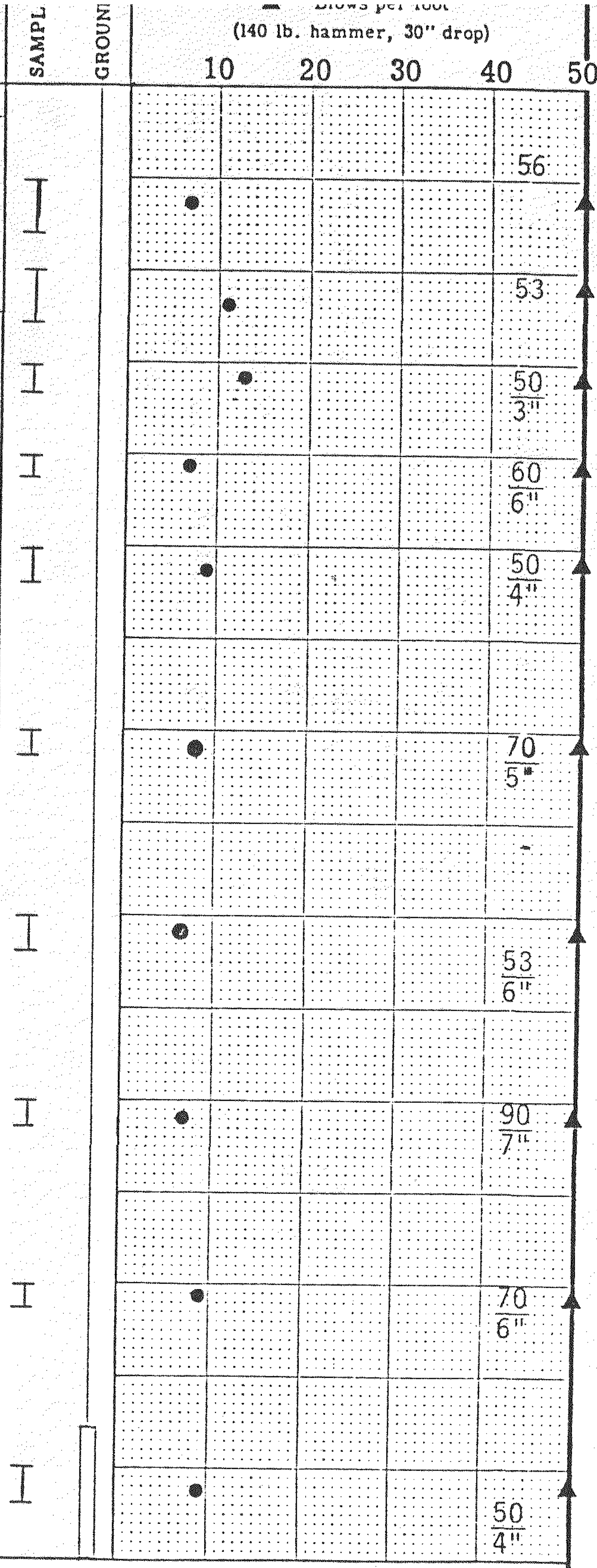
Very dense, moist, brown, silty, fine to medium SAND with trace fine gravel and clay (Glacial till)

- becomes brown/gray, fine to coarse SAND with some gravel, silt

- becomes silty

- decreasing silt content

- becomes gravelly



5
10
15
20
25
30
35
40

LEGEND

2.0" O.D. split spoon sampler

3.0" O.D. undisturbed sampler

Sampler pushed

Atterberg limits: ├─●─┤ ← Liquid limit; ← Natural water content; ← Plastic Limit

● Sample not recovered

Piezometer tip

▽ Water level

● % Water Content

Bob Hope International Heart Research Inst.

LOG OF BORING NO. B-5

W-4011

RITTENHOUSE-ZEMAN & ASSOC.

SOILS ENGINEERING AND GEOLOGY

FIG. A-12

Sheet 1 of 2

DEPTH,

Surface Elevation:

Microfilm

SAMPL

GROUNI

Very dense, wet, brown/gray, fine to coarse SAND with some gravel (Glacial till)

I

45

Boring terminated at 43.5 feet

Completed 8 June 1983

50

55

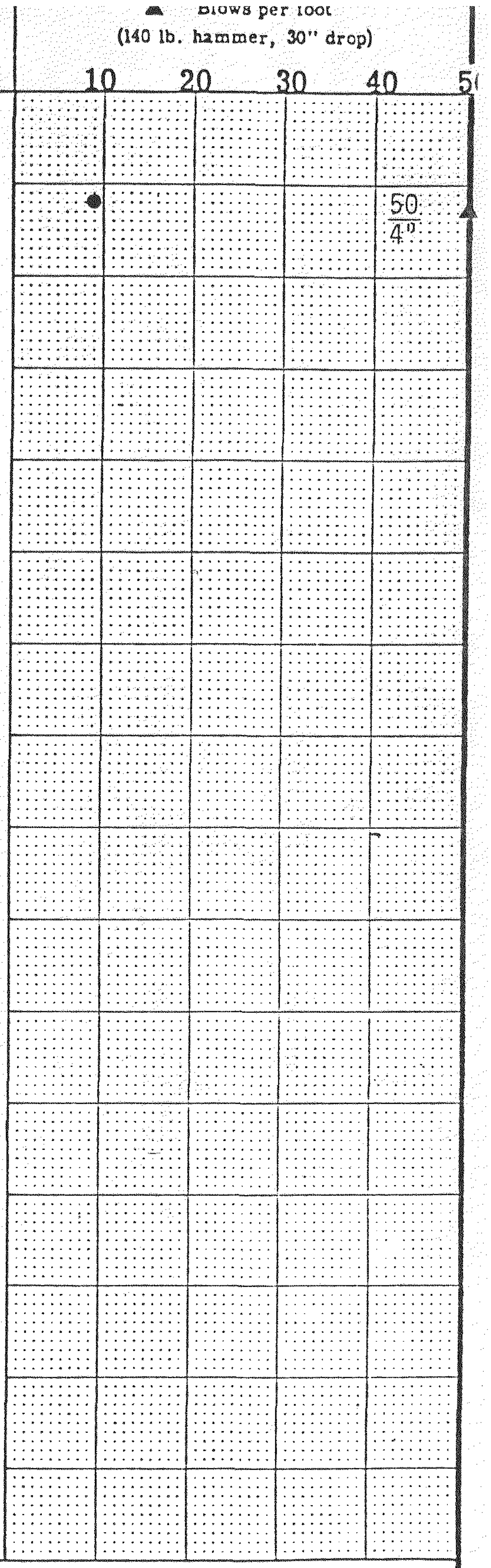
60

65

70

75

80



● % Water Content

LEGEND

I 2.0" O.D. split spoon sampler
II 3.0" O.D. undisturbed sampler
P Sampler pushed
Atterberg limits: ├─●─┤ ← Liquid limit
← Natural water content
← Plastic Limit

* Sample not recovered
▮ Piezometer tip
▽ Water level

Bob Hope International Heart Research Institute

LOG OF BORING NO. B-5 (cont.)
W-4011

RITTENHOUSE-ZEMAN & ASSOC.
SOILS ENGINEERING AND GEOLOGY

FIG. A-12
Sheet 2 of 2

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

TABLE OF CONTENTS

TABLES

TESTS

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

We performed geotechnical laboratory testing on selected soil samples retrieved from the six borings completed for the 18th Avenue and Cherry Street Medical Office Building Project. The laboratory testing program included tests to classify the soil and provide data for engineering studies. We performed visual classification on all retrieved samples. Our laboratory testing program included water content determinations, and grain size distribution analyses.

The following sections describe the laboratory test procedures.

B.1 VISUAL CLASSIFICATION

We visually classified soil samples retrieved from the borings using a system based on ASTM International (ASTM) D2487-11, Standard Test Method for Classification of Soil for Engineering Purposes (ASTM, 2011), and ASTM D2488-09a, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure) (ASTM, 2009). We summarize our classification system in Appendix A. We assigned a Unified Soil Classification System (USCS) group name and symbol, based on our visual classification of particles finer than 76.2 millimeters (3 inches). We revised visual classifications using results of the index tests discussed below.

B.2 WATER CONTENT DETERMINATION

We tested the water content of selected samples in accordance with ASTM D2216-10, Standard Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures (ASTM, 2010). Comparison of the water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. We present water content test results in the Laboratory Test Summary table in this appendix, and graphically on boring logs in Appendix A.

B.3 GRAIN SIZE DISTRIBUTION ANALYSIS

Grain size distribution analyses separate soil particles through mechanical or sedimentation processes. Grain size distributions are used to classify the granular component of soils and can correlate with soil properties, including frost susceptibility, permeability, shear strength, liquefaction potential, capillary action, and sensitivity to moisture. We plot grain size distribution analysis results in this appendix. Grain size distribution plots provide tabular information about each specimen, including: USCS group symbol and group name; water content; constituent (i.e., cobble, gravel, sand, and fines) percentages; coefficients of uniformity and curvature, if applicable; personnel initials; ASTM standard designation; and testing remarks. Constituent percentages are presented in the Lab Summary Table in this appendix and fines contents are plotted as data points on borings logs in Appendix A.

We performed mechanical sieve analyses on selected soil specimens to determine the grain size distribution of coarse-grained soil particles, in accordance with ASTM C136/C136M-14, Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates (ASTM, 2014).

B.4 CONSIDERATIONS

Drilling and sampling methodologies may affect the outcome of prescribed geotechnical laboratory tests. Refer to the field exploration discussion in this report for a discussion of these potential effects. Instances of limited recovery may have resulted in test samples not meeting specified minimum mass requirements, per ASTM standards. Test plots show which samples do not meet ASTM specified minimum mass requirements.

B.5 REFERENCES

ASTM International (ASTM), 2009, Standard practice for description and identification of soils (visual/manual procedure), D2488-09a: West Conshohocken, PA., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 – D5876, 12 p., available: www.astm.org.

ASTM International (ASTM), 2010, Standard test methods for laboratory determination of water (moisture) content of soil and rock by mass, D2216-10: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 7 p., available: www.astm.org.

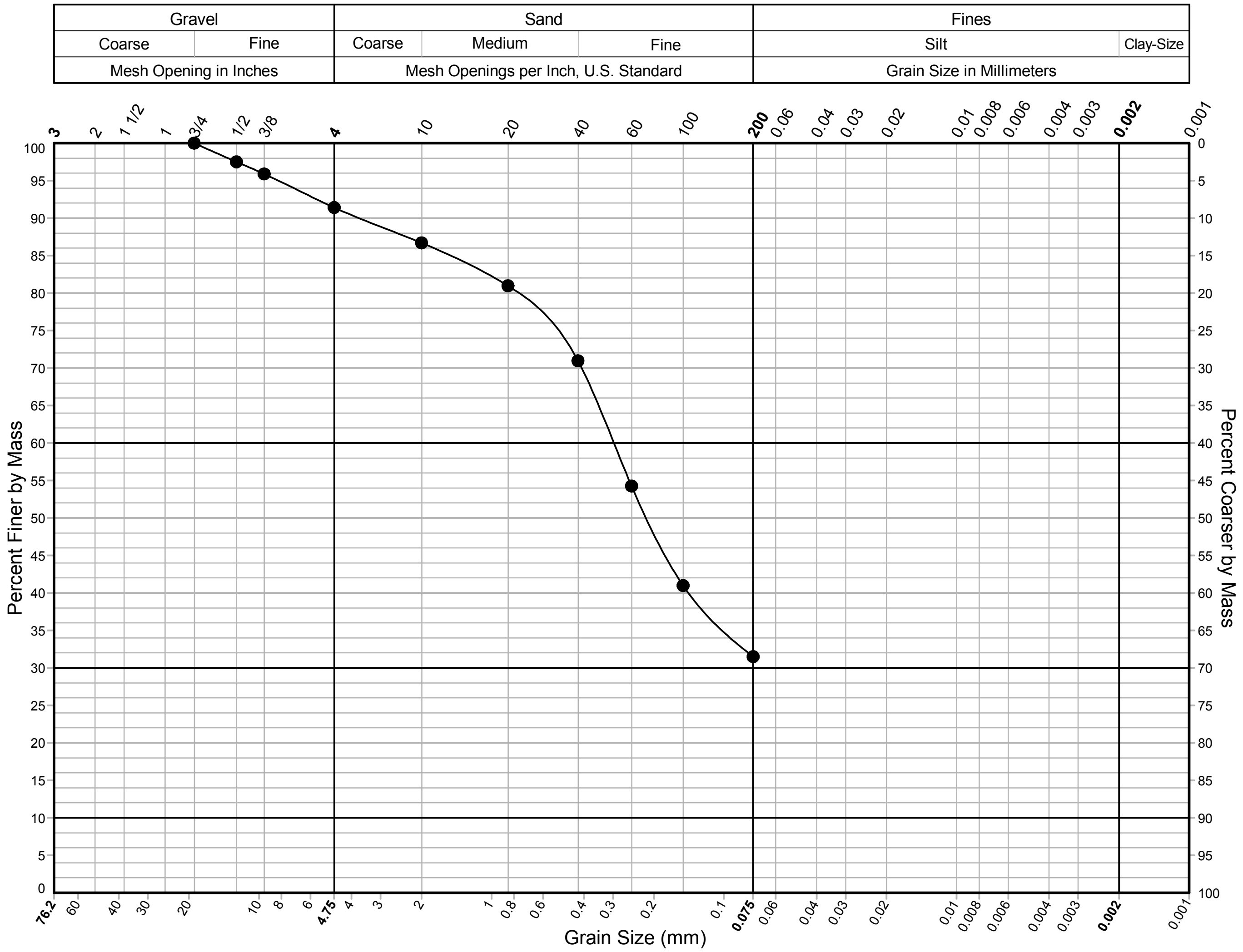
ASTM International (ASTM), 2011, Standard practice for classification of soils for engineering purposes (unified soil classification system), D2487-11: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 12 p., available: www.astm.org.

ASTM International (ASTM), 2014, Standard test method for sieve analysis of fine and coarse aggregates, C136-14: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.02, concrete and aggregates, 5 p., available: www.astm.org.

GRAIN SIZE DISTRIBUTION PLOT

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

BORING SW-4



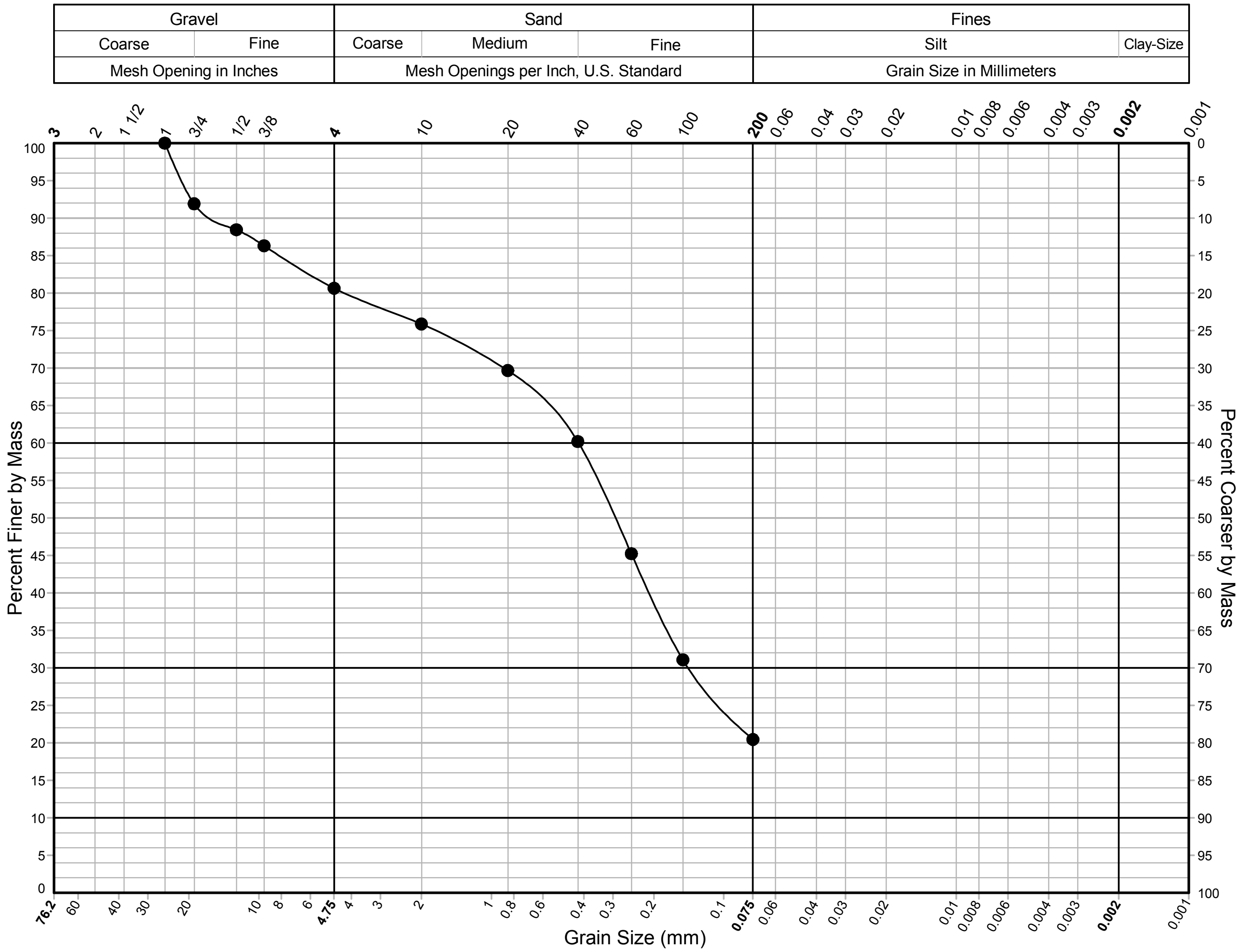
Sample Identification	Depth (ft)	USCS Group Symbol	USCS Group Name	Gravel %	Sand %	Fines %	< 20μm %	< 2μm %	WC %	Tested By	Review By	ASTM Std.
● SW-4, S-6*	25.0	SM	Silty Sand	9	60	31			8.3	AKV	JFL	C136

* *Test specimen did not meet minimum mass recommendations.*

GRAIN SIZE DISTRIBUTION PLOT

18th Ave & Cherry St
Medical Office Building
Seattle, Washington

BORING SW-5



Sample Identification	Depth (ft)	USCS Group Symbol	USCS Group Name	Gravel %	Sand %	Fines %	< 20µm %	< 2µm %	WC %	Tested By	Review By	ASTM Std.
● SW-5, S-4*	15.0	SM	Silty Sand with Gravel	19	60	20			12.9	AKV	JFL	C136

* *Test specimen did not meet minimum mass recommendations.*

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-22308-001

Date: February 8, 2017
To: Ms. Kara M. Anderson
Sabey Corporation

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland