Geotechnical Engineering Design Report

Redmond Town Center (RTC) 5B Project

Redmond, Washington

Prepared for

RTC 74th Street Property, LLC

June 27, 2016
19162-02

HARTCROWSER

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Prepared by

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INTRODUCTION

This report provides our geotechnical engineering design recommendations for the Parcel 5B Development at Redmond Town Center (RTC) in Redmond, Washington (Figure 1). The project site is at Redmond Town Center on Parcel 5B, which is east of the existing Macy's store just south of NE 76th Street and just west of 168th Avenue NE.

The purpose of our study was to conduct a new subsurface investigation at the proposed site and provide design and construction recommendations for the proposed structure. Our scope of work included:

- Review relevant and readily available geotechnical reports and construction documents to improve our understanding of the local soil and groundwater conditions;
- Drill five exploratory soil borings, collect soil samples, and complete laboratory analysis;
- Complete geotechnical engineering analyses;
- Provide geotechnical engineering conclusions and recommendations; and
- Present our results in this design report.

Figures illustrating site information follow the report text. Appendices A and B present the results of field exploration and laboratory testing programs, respectively. Appendices C and D contain the historical explorations in the site area, and the results of slug tests, respectively. Attachments 1 and 2 provide the recommended procedures for tieback anchor testing and the shoring monitoring, respectively.

Our work was performed in general accordance with our Client-Consultant Agreement dated March 9, 2016. This report is for the exclusive use of RTC 74th Street Property, LLC, Lowe Enterprises Real Estate Group, and their design consultants for specific application to the subject project and site. We completed this work in accordance with generally accepted professional practices for work of similar nature done under similar conditions in the same or similar localities as those encountered at the time our work was performed. We make no other warranty, express or implied.

SITE AND PROJECT DESCRIPTIONS

We understand the project will consist of constructing an eight-story mid-rise residential/commercial structure on Parcel 5B, which is east of the existing Macy's store just south of NE 76th Street, and just west of 168th Avenue NE. Current plans from Encore Architects dated March 25, 2016 show an average ground surface elevation of 42 feet, with the basement floor

level at about elevation 29.5 feet on the average. Allowing for the thickness of the foundations, we anticipate a mass excavation depth of about 14 feet below existing grade (bottom of excavation at about elevation 28 feet) with local excavations that extend as deep as elevation 22 feet. The site is currently a paved parking lot.

We understand that the basement will be designed and constructed as a water-tight enclosure ("bath tub"), without allowing for a permanent dewatering system. This will require a close consideration of the effect of hydrostatic pressure on the basement structure and water-proofing system based on the proper assessment and selection of an appropriate design groundwater elevation. As the groundwater elevation at the site will vary seasonally, as well as from year to year depending on climatic changes over the life of the building, the design groundwater elevation for permanent condition will need to be selected such that the risk of the groundwater rising above the design elevation is kept low and acceptable.

Our understanding of the project is based on our correspondence and meetings with you, discussions with the project team, and our previous experience at Redmond Town Center.

SUBSURFACE CONDITIONS

Our understanding of the subsurface conditions at the site is based on five borings advanced for this study and relevant historical explorations at the site as shown on Figure 2. Our exploration program for this study included drilling five borings (designated HC-1 through HC-5) to depths ranging from 50.8 to 62 feet. HC-1 and HC-4 were completed as monitoring wells to allow for long-term groundwater-level monitoring at the site. Detailed logs of our borings are in Appendix A. The results of geotechnical laboratory tests performed on selected samples are in Appendix B.

Hart Crowser has conducted a number of geotechnical studies in this area associated with the original construction of Redmond Town Center. Other geotechnical studies were completed for the design of the structures in the vicinity of the site. Appendix C includes selected existing logs (borings B-15, B-9, B-5, and TP-7) from previous explorations.

The subsurface information used for this study represents conditions at discrete locations across the project site, and actual conditions in other areas could vary. Furthermore, the nature and extent of any variations may not become evident until construction begins. If significant variations are observed at that time, we may need to modify our conclusions and recommendations accordingly to reflect actual site conditions.

Soil

Redmond town center is located in an area of Redmond that is underlain by glacial recessional outwash deposits that consist of generally medium dense sand and gravel that becomes denser with depth. Compressible silt layers have been encountered in isolated areas in this deposit.

In general, the borings advanced for this study show medium dense to very dense gravelly sand to sandy gravel from the ground surface to a depth of about 30 to 37 feet (elevation 4 to 10 feet).

This soil unit contains varying amounts of cobbles and boulders. Below this unit, we encountered medium dense to very dense fine to coarse sand with occasional gravel and slightly silty layers. In general, the subsurface conditions are suitable for shallow foundations and slabs-on-grade with moderate loads.

Groundwater

Groundwater levels were estimated at the time of drilling and measured subsequently in monitoring wells (MW) installed in borings HC-1 and HC-4. Groundwater levels at the time of drilling (ATD) were encountered at depths from 10.5 to 15 feet below ground surface (bgs), corresponding to elevations ranging from 27.5 to 30.3 feet. Results of groundwater observations are summarized in Table 1. Previous studies have encountered groundwater at about 10 feet bgs, with locally higher water levels during periods of peak flow of Bear Creek.

Table 1 – Groundwater Observation in Borings and Monitoring Wells

Borings	Date Boring Completed	Groundwater ATD		Groundwater in Monitoring Well			Screened Elevation Range (ft)
		Depth (ft)	Elevation (ft)	Date Observed	Depth (ft)	Elevation (ft)	
HC-1 (MW)	09/25/2015	14.5	27.5	01/08/2016	11.0	31.0	7.0 – 27.0
HC-2	12/02/2015	10.5	30.3	–	–	–	–
HC-3	12/04/2015	11.0	29.5	–	–	–	–
HC-4 (MW)	12/03/2015	14.5	30.3	01/08/2016	12.2	32.6	8.4 – 28.4
HC-5	12/03/2015	15.0	29.2	–	–	–	–

Groundwater levels noted in existing and current borings are representative of the time of observations. Variation in groundwater level should be expected depending on location, season, and precipitation. In the Redmond Town Center area, groundwater level will also be controlled to a large extent by the flow (flood level) in the nearby Bear Creek.

Given the depth of planned excavation, groundwater will be encountered during excavation for construction of the building, and will require temporary dewatering. For temporary conditions (e.g. shoring design), we recommend using a design groundwater elevation of 31 feet.

Considering that the basement will be designed and constructed as a “bath tub” system, we recommend allowing a factor of safety to lessen the risk of groundwater rising above the design elevation during the life of the proposed development. This means considering a design groundwater elevation for permanent condition that is somewhat higher than that indicated by the transitory measurement at few discrete locations as described above. However, groundwater measurement records for periods comparable to the life of the structure are not available, and there is no definite way to ascertain the highest groundwater elevation that might occur during the life of structure. Data from a well in nearby area monitored by the City for the past few years indicate fluctuations in groundwater of up to about 5 feet can be expected. Based on our

understanding of the subsurface and topographical conditions, and our previous experience in the area, we recommend using a design groundwater elevation of 34 feet for the permanent condition.

Hydraulic Conductivity (Slug Testing)

Slug testing was conducted in wells HC-1 and HC-4 on January 8, 2016, to determine the hydraulic conductivity of the native soils. Slug tests are performed by rapidly inserting and removing a solid PVC rod into a well and measuring the recovery of the groundwater levels. When the PVC rod is inserted, it is a falling head test, and when the rod is removed, it is a rising head test. The water level data generated from the tests were analyzed using the Bouwer and Rice method.

Average hydraulic conductivities determined from slug tests range from 4.5×10^{-2} to 5.2×10^{-2} centimeters per second (cm/sec) (128.5 to 147.4 feet/day). This range of hydraulic conductivity is consistent with typical values for sand and gravel. The slug test results are summarized in Appendix D.

SEISMIC CONSIDERATIONS

Seismic Setting

The seismicity of Western Washington is dominated by the Cascadia Subduction Zone, in which the offshore Juan de Fuca Plate is subducting beneath the continental North American Plate. Three main types of earthquakes are typically associated with subduction zones: crustal, interface subduction, and intraslab subduction earthquakes.

Crustal Sources. Recent fault trenching and seismic records in the Puget Sound area clearly indicate a distinct shallow zone of crustal seismicity, the Seattle Fault, which may have surficial expressions and can extend 25 to 30 kilometers deep.

Subduction Zone Sources. The offshore Juan de Fuca Plate is subducting below the North American Plate. This causes two distinct types of events. Large-magnitude interface earthquakes occur at shallow depths near the Washington coast (e.g., the 1700 earthquake with magnitude of 8 to 9) at the interface between the two plates. A deeper zone of seismicity is associated with bending the Juan de Fuca Plate below the Puget Sound region that produces intraslab earthquakes at depths of 40 to 70 kilometers (e.g., the 1949, 1965, and 2001 earthquakes).

Seismic Parameters

The 2012 International Building Code seismic design parameters for the site latitude and longitude are provided in Table 2. These parameters should be used to develop the code-based design response spectrum. The parameters were obtained from the USGS US Seismic Design Maps web application (http://geohazards.usgs.gov/designmaps/us/application.php).

Site Class for Seismic Design. We determined the soil site class using information about the supporting foundation soils in accordance with ASCE 7-10. The soil site class is determined considering the soil characteristics and a weighted average of the blow counts observed to a depth of 100 feet bgs. For the borings in this study, which were not 100 feet deep, when determining the site class we assumed the material density below the deepest sample remains constant to 100 feet and determined that the site should be designated as seismic Site Class D.

Table 2 – 2012 IBC Seismic Design Parameters

Parameter	Value
Site Latitude	47.6705
Site Longitude	–122.117
Seismic Site Class	D
Mapped MCE_R spectral response acceleration at short periods, S_S	1.254 g
Mapped MCE_R spectral response acceleration at 1-second periods, S_1	0.480 g
Site coefficient at short periods, F_a	1.0
Site coefficient at 1-second periods, F_v	1.520
MCE_R spectral acceleration at short periods, S_{MS}	1.254 g
MCE_R spectral acceleration at 1-second periods, S_{M1}	0.730 g
Design spectral acceleration at short periods, S_{DS}	0.836 g
Design spectral acceleration at 1-second periods, S_{D1}	0.487 g

Liquefaction Assessment

Loose, saturated granular soil is generally susceptible to liquefaction as a result of cyclic shaking during an earthquake. Cyclic shaking causes a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils that rely on inter-particle friction for strength are susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upward, carrying soil particles with the draining water. In general, loose, saturated sandy soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are also susceptible to liquefaction under relatively higher levels of ground shaking. For any soil type, the soil must be saturated for liquefaction to occur. Liquefaction can cause ground surface settlement and lateral spreading.

We used empirical methods to estimate liquefaction potential based on the standard penetration test (SPT) data obtained at the site. We used the SPT-based liquefaction triggering procedure after Idriss and Boulanger (2008). For our analysis of the maximum considered earthquake (MCE) hazard level, which corresponds to an average 2,475-year return period, we used an earthquake magnitude of 7.0 and a peak ground acceleration of 0.506 g that we obtained from USGS (2008) for the site coordinates and Site Class D.

Our liquefaction assessment included five borings, HC-1 through HC-5. Our analysis indicated that there is some possibility of thin, localized soils layers in borings HC-2 and HC-5 having a low to moderate susceptibility to liquefaction. However, considering the overall results for all the borings, the susceptibility to liquefaction of the site is considered low. Accordingly, the potential for any liquefaction induced settlement at or below the foundation level of the building is also considered low.

GEOTECHNICAL ENGINEERING DESIGN RECOMMENDATIONS

This section of the report presents our conclusions and recommendations for the geotechnical aspects of design and construction on the project site. We developed these recommendations based on our current understanding of the project and the subsurface conditions encountered in the five borings completed for this study. If the nature or location of the development is different than we have assumed, we should be notified so we can change or confirm our recommendations.

Site Preparation

For site preparation, we anticipate that the asphalt pavements, concrete walkways, sidewalks, and landscape vegetation will be removed. Removed asphalt, brick, concrete, or topsoil should not be reused as structural fill.

It will likely be necessary to relocate or abandon some or all utilities from the building footprint area. Excavation of these utility lines will occur through fill materials. Abandoned underground utilities should be removed or completely grouted. Ends of remaining abandoned utility lines should be sealed to prevent soil or water from entering the pipe. Soft or loose backfill materials should be removed, and excavations should be backfilled with structural fill. Coordination with utility owners is generally required in addressing existing utilities.

Where used, the stability of the temporary cut slopes should be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. All excavation should be made in accordance with all local, state, and federal safety requirements

Temporary Shoring

Based on the soil and groundwater conditions, and the anticipated depth of excavation, it is our opinion that a dewatered excavation could be supported using cantilevered soldier piles, or conventional soldier piles with a single row of tieback anchors, or a combination of the two. In our opinion, a soil nail shoring is not suitable, considering the soil conditions encountered in HC-1 through HC-5. We understand that the temporary dewatering discharge during foundation excavation can be readily handled by the existing drainage system in the area.

Shoring should be designed by a professional structural engineer registered in the State of Washington. We also recommend that we review the geotechnical aspects of the shoring design

before construction. It is generally not the purpose of this report to provide specific criteria for the contractor's construction means and methods. It should be the responsibility of the shoring contractor to verify actual ground conditions and determine the construction methods and procedures needed to install an appropriate shoring system.

Lateral Earth Pressures

Given the soil conditions encountered at the site, lateral earth pressures for the design of conventional shoring depend on the type of shoring, its ability to deform, and the presence or absence of structures within the "deformation zone" behind the shoring wall. In general, this deformation zone can be approximated by a line extending into the retained soil from the base of the shoring wall at a slope of about 1H:1V.

The shoring may be designed using active earth pressure if the top of the shoring wall is allowed to deform about 0.001 to 0.002 times the wall height and if no settlement-sensitive structures or utilities are within the deformation zone behind the wall.

At-rest earth pressure should be used to design the shoring if settlement-sensitive structures or utilities exist within the potential deformation zone, or if the wall system is too stiff to allow sufficient lateral movement for the development of an active condition.

We recommend the following:

- For a cantilevered soldier pile wall or a soldier pile wall with single row of tieback anchors, and level ground behind the soldier pile wall, use lateral earth pressures presented on Figure 4. We will provide separate lateral earth pressure recommendations if a shoring system with two or more rows of tieback anchors is needed.

- For normal vehicular traffic behind the shoring wall, include a minimum traffic surcharge in the design, as shown on Figure 4.

- Lateral earth pressure from additional surcharge loads (e.g., adjacent footing loads, material stockpiles, or other large loads from heavy vehicles, machinery etc.) should be computed using the methods shown on Figure 6. These additional loads should be added to those calculated for the shoring wall based on Figure 4.

It should be noted that the lateral earth pressures recommendations described above are based on dewatered conditions, with the understanding that hydrostatic pressure does not act on the temporary shoring walls.

Soldier Pile Design

Soldier piles must be embedded deeply enough to resist lateral kick-out and vertical loads. Also, soldier piles must be designed to carry the bending and shear stresses from the lateral earth pressure acting against the soldier piles and the lagging between them. We also recommend the following:

- Use end-bearing and skin friction design values presented on Figure 4 to resist vertical loads.
- Embed piles at least 10 feet below the bottom of the excavation after allowing 2 feet for disturbance.
- Design soldier piles for bending using a uniform loading equivalent to 80 percent of the design values, and analyze for shear using the total load.
- For design against kick-out, compute the lateral resistance on the basis of the passive pressure presented on Figure 4, acting over two times the diameter of the concreted soldier pile section or the pile spacing, whichever is less.

The above recommendations are based on proper installation of the soldier piles as described below.

Soldier Pile Installation

Conditions such as caving soil and groundwater can loosen soils at the bottom of the soldier pile excavation and reduce bearing capacity of the zone of disturbed soils. Tieback destressing and shoring failure could occur if soldier pile bearing capacity is inadequate and soldier piles settle under the vertical component of the inclined tieback load. We recommend that a Hart Crowser representative monitor soldier pile installations so that construction methods can be adjusted in a timely manner, if needed.

We recommend the following for soldier pile installation:

- Require that the contractor to be prepared to case the soldier pile installations. The actual necessity of casing should be determined in the field at the time of installation.
- Prohibit the use of drilling mud unless reviewed and approved by the geotechnical and structural engineer.
- Place concrete in soldier pile holes with a tremie pipe to displace groundwater (or drilling mud where permitted) upward from the bottom of the hole. Hart Crowser must verify the integrity of the soil at the base of the hole before concrete is placed.

Lagging

Loss of ground between the soldier piles is prevented using lagging. Lagging typically consists of timber planks or concrete panels, but timber lagging is common in Seattle area. We recommend using the thickness (rough-cut) of temporary lagging provided in Federal Highway Administration (FHWA) Geotechnical Engineering Circular No. 4, “Ground Anchors and Anchor Systems,” based on “Competent Soils” and the selected clear span of soldier piles. Considering the expected excavation depth for this project, the recommended lagging thickness is 2 inches for a clear span of 5 feet or less. For clear spans between 5 and 8 feet, the recommended lagging thickness is 3 inches.

Prompt and careful installation of lagging, particularly in areas of seepage and loose soils, is important for maintaining the integrity of the excavation. The shoring contractor should be responsible for installing lagging to prevent soil failure, sloughing, and ground loss, and to provide safe working conditions.

Soldier pile wall construction may be difficult if cobbles, boulders, or loose sands and gravels are encountered in the excavation. If these conditions are encountered, substantial raveling of the soil could occur. The contractor should be prepared to place lagging in small vertical increments in areas of utility backfill or caving soil, and should be prepared to backfill voids behind the wall that may result from ground loss during construction.

We recommend the following for lagging:

- Backfill voids greater than 1 inch using sand, or a porous slurry. Backfill the void spaces progressively as the excavation deepens. The backfill must not allow potential hydrostatic pressure buildup behind the wall. Drainage behind the wall must be maintained. If not, hydrostatic water pressure should be added to the recommended lateral earth pressures.

- Install extra lagging above the shoring wall if there is a slope above the wall, to provide a partial barrier for material that could ravel down from the slope face and fall into the excavation.

Tieback Anchor Design and Construction

We expect that tieback anchors can be used for external lateral support of the soldier pile walls. Our tieback anchor design recommendations are based on the assumption that cased boreholes that are at least 6 inches in diameter will be used. We understand that the anchors will be installed by single stage grouting as the casings are withdrawn. Recommended allowable load transfer (adhesion) value on Figures 4 should be used for the preliminary design, and confirmed by load testing as described herein. We recommend the following for tieback anchor design:

- Locate anchor portions of the tiebacks behind the line defining the no-load zone, and below elevation 30 feet, as shown on Figure 4.

- For preliminary design and planning, use the allowable adhesion value presented on Figure 4, which is appropriate for cased, at least 6-inch-diameter boreholes. The shoring contractor must choose appropriate means and methods to achieve the design adhesion based on their experience on similar sites.

- Locate anchors at least three tieback borehole diameters apart.

- Pump structural grout into the anchor zone using a grout hose or tremie hose placed at the bottom of the anchor borehole.

- Install a suitable bond breaker such as plastic sheathing or a PVC pipe around the tieback within the no-load zone.
- Grout and backfill tiebacks immediately after placing the anchor. To prevent collapse of the holes, ground loss, and surface subsidence, do not leave anchor holes open overnight.
- Take care not to mine out large cavities in granular soil.
- Maintain continuous cutting return if using pneumatic drilling techniques so that air pressure is not channeled to nearby utility vaults, corridors, or subgrade slabs, because air pressure may damage such structures.
- Design anchor lengths so that they do not conflict with any underground utilities and/or support elements of the adjacent structures.
- Identify existing facilities adjacent to the project site including buried utilities and foundations, as these may affect the location and length of the anchors.
- Install anchors to minimize ground loss and do not disturb previously installed anchors. During tieback drilling, wet or saturated zones will be encountered, and caving or blow-in could occur. Drilling with a casing may reduce the potential for these conditions and ground loss.

We recommend that selection of the materials and the installation technique be left to the shoring contractor. The shoring contractor shall be made contractually responsible for the design of the tieback anchors, as tieback capacity is largely a function of the means and methods of installation. The selected tieback anchor installation method must be subject to field verification with performance testing and proof testing as discussed in Attachment 1.

Hart Crowser should review the design for anchor locations, capacities, and related criteria prior to implementation.

For anchor pullout resistance, we recommend a factor of safety of at least 2.0. This factor of safety provides for a reasonable additional load capacity in case of an unforeseen increase in loading that may develop because of variable field conditions during construction. The variable soil conditions and unit friction (adhesion) values mean that some field changes in anchor length may be necessary.

Tieback Anchor Testing

The tiebacks will be tested to confirm the appropriateness of the anchor design values and to verify that a suitable installation is achieved. The procedure for performance and proof-testing is presented in Attachment 1 and summarized below. For testing of tieback anchors, we recommend:

- Require the shoring contractor to complete two successful 200 percent performance tests on one tieback. Contract documents should be prepared such that additional verification tests could be performed on a unit price basis if different site conditions are encountered.

- For anchors installed for the 200 percent verification test, the specifications should include components to prevent friction contribution between the grout column and the soil in the no-load zone.

- Proof-load each production anchor to 133 percent of the design load to test for total movement and creep.

Shoring Monitoring

The shoring performance should be monitored using optical survey measurements of horizontal and vertical movement. A shoring monitoring program provides early warning if the shoring does not perform as expected. Our recommended shoring monitoring program is outlined in Attachment 2. In addition to the monitoring program, we recommend:

- The owner or its representative should make a complete inspection of all pavement and structures and other facilities adjacent to the project site. This inspection should focus on any signs of damage, particularly those caused by settlement. Notes should be made and photographs taken where necessary.

- The contractor and the shoring subcontractor should be familiar with the existing site conditions. They should be allowed to review the inspection data gathered by the owner and may also choose to complete a survey.

- The contract should clearly define the responsibilities of the owner, contractor, and shoring contractor in making inspections, reviewing data, and repairing possible damage.

Foundation Support

We recommend the following for design and construction of mats and footings:

Allowable Bearing Pressure

Allowable bearing pressures generally increase with increased footing width. We expect that footings for this development may involve isolated spread footings, a mat foundation, or some combination of the two. Continuous wall footings and isolated column footings should have a minimum width of 2 feet. Table 3 provides our recommendations for allowable bearing pressures by footing width, where the footing width is taken as the shortest lateral dimension of the footing. Bearing pressures of intermediate dimensions may be calculated by linear interpolation. Based on

the subsurface information, we expect that building foundations will bear on the undisturbed dense native soils.

Table 3 – Allowable Bearing Pressures for Building Foundations

Soil Type	Net Allowable Bearing Pressure		
	Footing width 2 feet or more but less than 5 feet	Footing width 5 feet or more but less than 10 feet	Footing width10 feet or more but less than 15 feet
Generally dense undisturbed native soils	4 ksf	5 ksf	6 ksf

We make the following recommendations for the design and construction of shallow foundations for the Redmond Town Center 5B Project:

- Use a maximum allowable soil bearing pressures provided in Table 3 for spread footings bearing on the generally dense, slightly silty, gravelly sand to sandy gravel layers. The allowable bearing pressure is for dead loads and frequently applied live loads.
- Allow an increase in the allowable soil bearing pressure of up to one-third for loads of short duration, such as those caused by wind or seismic forces.
- Footings should be founded outside of an imaginary 1H:1V plane projected upward from the bottom edge of adjacent footings or utility trenches.
- For resistance to lateral loads, use an equivalent fluid density to represent the passive resistance of the soil. For a typical footing poured against *in situ* soil around foundation subgrade elevation, (below about 28 feet elevation) we recommend an allowable passive equivalent fluid density of 140 pounds per cubic foot (pcf). A factor of safety of 1.5 has been applied to this passive resistance value.
- Use an allowable coefficient of friction of 0.25 for footings poured neat on the generally dense, slightly silty, gravelly sand to sandy gravel for resistance on the base of foundations. A factor of safety of 1.5 has been applied to this value.
- During construction of the footing, maintain groundwater levels at least 2 feet below the base of the excavation at all times to prevent the risk of heave, piping, boiling, and other loss or disturbance of foundation subgrade material.
- Have a qualified geotechnical engineer or geologist observe exposed subgrades before footing construction to verify suitable bearing surfaces.

Modulus of Subgrade Reaction

Modeling foundation behavior under vertical loads will require modulus of subgrade reaction (vertical spring constant) applicable to the soils on which the foundations bear, depending on the underlying soil density and consistency. Loading type, such as static or dynamic loading, has a dramatic effect on the stiffness of the springs. Determining the subgrade modulus value to be used depends on:

- The structural and geotechnical engineer's experience designing similar foundations in similar soil conditions;
- The quantity, magnitude, and area of the footing or mat foundation under various loads; and
- Back-checking settlement predicted from structural modeling with geotechnical settlement estimates for given foundation geometries.

As a preliminary or initial modeling under static vertical loads, we recommend using a subgrade modulus (Ks) of 80 pounds per cubic inch (pci) for the foundations bearing on the generally dense, slightly silty, gravelly sand to sandy gravel layers. We consider this value a reasonable starting point for an iterative design process. Hart Crowser should review the displacement estimates from the structural model and perform settlement evaluations of the specific geometry and loading for compatibility. Based on these settlement evaluations, modifications to the subgrade modulus used in the structural model may be required.

Foundation Settlement

Based on our understanding of the subsurface conditions at the site, we expect settlement to be primarily elastic with any time-dependent consolidation component occurring quickly. Settlement of foundations is expected to occur essentially as the loads are applied. Based on our explorations at the site, previous explorations near the site, laboratory test data, and our past experience on deep excavations for buildings and high-rise towers, we anticipate settlements on the order of 1/2-inch to up to about 1-inch depending on the foundation or footing sizes. Differential settlement is expected to be about half of the total settlement. When final loads become available, we can provide revised estimates of total and differential settlement.

Foundation Preparation and Construction

Careful preparation and protection of the exposed subgrade should occur before concrete placement. Any loose to medium dense sand or gravel or soft to medium stiff silt present at the subgrade should be overexcavated and replaced with lean or structural concrete. Any visible organic or other unsuitable material should be removed from the exposed subgrade.

The foundation settlement estimated herein assumes that careful preparation and protection of the exposed subgrade will occur before concrete placement. Any loosening of the subgrade during construction could result in greater settlement. It is important that all foundation excavations be cleaned of loose or disturbed soil prior to placing any concrete and that there be no standing

water in any foundation excavation. Also, groundwater should be controlled such that heave or boiling of the foundation subgrades does not occur. These conditions should be documented before construction.

Depending on the time of construction and conditions at the bottom of the excavation, it may be necessary to place a nominal 2- to 4-inch-thick “rat slab” consisting of lean concrete immediately after the excavation has been checked by the geotechnical engineer or representative.

Permanent Subgrade Building Walls

Lateral Earth Pressures for Walls Constructed Against Temporary Shoring

Permanent walls constructed flush with temporary shoring systems may be designed for the same active (or at-rest) earth pressures used in the design of the shoring system (Figures 4 and 5).

Seismic Earth Pressure

The lateral earth pressures for permanent basement walls must include a seismic earth pressure increment. This additional lateral earth pressure can be approximated as a rectangular uniform pressure of 7H, where H is the height of basement wall as shown on Figure 5. The seismic earth pressure is based on the design level hazard level for the site location per IBC 2012.

Additional Surcharge Pressures

We recommend applying a surcharge of 250 psf behind the top of the basement wall for computations to provide some allowance for possible surcharge loads such as light vehicles traffic or small material stockpiles. Surcharge pressures resulting from heavier loads such as buildings, footings, heavy equipment, or large material stockpiles should be calculated using Figure 6. These additional loads would be added to the soil pressure calculated for permanent foundation walls shown on Figure 5.

Hydrostatic Groundwater Pressure

We understand the building will be designed to be water tight enclosure (bath tub) and permanent basement walls will need to resist hydrostatic pressure as shown on Figure 5. Apply a triangular lateral hydrostatic pressure of $62.4h_w$ psf, where h_w is the depth of the basement wall below the groundwater table. As described above, we recommend using a design groundwater elevation of 36 feet for the permanent condition applicable to the design of basement wall.

This recommendation presumes that drainage will be installed above the groundwater table to prevent buildup of water pressure caused by perched water and precipitation. Walls without drainage must be designed for full horizontal hydrostatic pressure as well as hydrostatic uplift forces on the bottom of the basement floor slab. Additionally, the waterproofing and vapor barrier system would need to be designed and installed by others to facilitate a functioning, dry basement level.

Wall Drainage

We understand the building basement wall will be designed to be water tight and wall drainage will not be needed below the design groundwater elevation (36 feet) for permanent condition. For portion of the basement above the design groundwater elevation, we make the following recommendations for wall drainage:

To reduce the risk of potential hydrostatic pressure buildup, we recommend placing a dimpled geotextile drainage geocomposite (e.g., TenCate Mirafi G-Series) between the soil and the building wall. Alternatively, free-draining granular material (less than 3 percent passing the US No. 200 sieve) could be used as structural fill within an 18-inch-wide zone immediately behind the wall.

Floor Slab

The subgrade soils are expected to be adequate for a slab-on-grade design, but it should be noted that the floor slab design needs to consider uplift pressure from design groundwater elevation for permanent condition, as well as the waterproofing requirements for a watertight basement enclosure.

Construction Dewatering

The planned basement finished floor elevation is 29.5 feet and we anticipate a mass excavation elevation of 28 feet with local excavations extending to elevation 22 feet. This would place the bottom of excavation 3-feet below the design groundwater level of 31 feet.

Construction Dewatering Recommendations

Construction dewatering is required when excavating below the groundwater table to maintain dry and stable working conditions in the bottom of the excavation. Typically, contractors prefer to have the groundwater table lowered a foot or two below the bottom of excavations. At the project site, this equates to temporarily lowering the groundwater to about elevation 26 feet, which is 5 feet below the design temporary groundwater elevation. We recommend the following for dewatering during construction:

- Temporary construction dewatering will be required when excavations are below the groundwater table. The actual dewatering methods and schedule will be selected by the construction contractor, with our review. In our opinion, dewatering will require using a network of well points and/or dewatering wells around the perimeter of the excavation. Dewatering will need to be continuous during construction below the groundwater table.

- We recommend lowering the groundwater table by no more than 2 feet below the bottom of excavation to maintain stable working conditions in the bottom of the excavation while minimizing off-site settlement. This could be reduced to 1 foot during construction, if necessary.

- The contractor should use a qualified licensed hydrogeologist to design the dewatering system. The general contractor should retain an experienced dewatering contractor to install and operate the dewatering system. Hart Crowser should review the dewatering plan before it is implemented.

- Steady-state dewatering rates will vary depending on the excavation size and depth, season, dewatering method, schedule, and soil conditions. Generally, beginning dewatering one to two weeks before construction is recommended to allow sufficient time to reach design water levels. Initial dewatering rates can be much higher until a stabilized water level is achieved.

- We recommend disposing of dewatering discharges by a local storm drain if there are no contaminants. Disposal of dewatering discharges by infiltration or recharge will likely be impractical because of the relatively shallow groundwater table.

Our services were provided to help assess temporary dewatering for the planned excavation, shoring, and construction of new utilities that will be, in part, beneath the groundwater table. Dewatering rates will depend on the dewatering methods and schedule selected by the contractor. Site-specific information is limited to exploration borings and in situ hydraulic testing (Appendix D) completed at widely spaced locations; therefore, conditions may differ from those assumed.

Geotechnical Impacts of Dewatering

Geotechnical impacts of dewatering are primarily related to dewatering-induced settlement. Given the dense and incompressible nature of the soils underlying the site, lowering the groundwater table will likely not lead to excessive settlement. The amount of settlement that occurs depends on the soil conditions, as well as on the amount and duration of dewatering.

Waterproofing

We understand the foundation will be waterproofed. A specialty waterproofing subconsultant should be retained to design the waterproofing. We have seen waterproofing systems such as a heavy plastic membrane liners, bentonite clay panels (i.e., Volclay or equivalent), and other interior or exterior sealants used effectively on projects similar to this one.

Structural Fill

We anticipate that structural fill will be required for backfilling behind walls, for backfilling of utility trenches and other miscellaneous excavations, and possibly for replacement of overexcavated, soft, or wet soils. We recommend the following regarding placement of structural fill:

- Place structural fill in maximum 10-inch-thick loose lifts and compact it to a firm condition to support concrete placement as observed and verified by a Hart Crowser field representative.

- The moisture content of the fill should be controlled within 2 percent of the optimum moisture. Optimum moisture is the moisture content corresponding to the maximum Proctor dry density.

- If a select soil will be imported for use as structural fill, we recommend using a clean, well-graded sand or sand and gravel with less than 5 percent by weight passing the No. 200 mesh sieve (based on the minus 3/4-inch fraction). If imported soil is used during wet weather periods, we recommend a gravel content (material coarser than a US No. 4 sieve) of at least 30 percent.

- If small, hand-operated compaction equipment is used to compact structural backfill, fill lifts should not exceed 8 inches of loose thickness.

- Any import material to be used as structural fill should be sampled from the supplier's pit before delivery or use on site, to determine the maximum dry density, gradation, and optimum moisture content.

Use of On-Site Soils as Structural Fill

The suitability of excavated site soils for structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines (that portion passing the No. 200 sieve) increases, the soil becomes increasingly sensitive to small changes in moisture content, and adequate compaction becomes more difficult to achieve. Soil containing more than about 5 percent fines cannot be consistently compacted to a dense non-yielding condition when the water content is greater than about 2 percent above or below optimum.

Our borings indicate mostly sand and gravel soils with variable fines content at the project site. Some of this native soil may be used as structural fill, but should be verified by a Hart Crowser field representative before placement.

Temporary Open Cuts

The stability and safety of cut slopes depend on a number of factors, including:

- The type and density of the soil;
- The presence and amount of any seepage;
- The depth of cut;
- The proximity of the cut to any surcharge loads near the top of the cut (such as stockpiled material, traffic, or structures) and the magnitude of these surcharges;
- The duration of the open excavation; and
- The care and methods used by the contractor.

Temporary soil cuts for site excavations that are more than 4 feet deep should be adequately sloped back to prevent sloughing and collapse in accordance with Washington Department of Occupational Safety and Health (DOSH) guidelines (WAC Chapter 296-155 Part N). Based on these

guidelines, the fill and native subsurface materials at the site would be classified as Type C. We recommend the following for open cuts:

- Use a maximum allowable slope for excavations less than 20 feet deep of 1.5H:1V.
- Do not excavate below the bearing elevation of the existing footings or structural elements. Consult with the geotechnical engineer during construction to limit the size of these excavations and the amount of time they remain open.
- Protect the slope from erosion by using plastic sheeting, especially during wet weather excavation.
- Limit the maximum duration of the open excavation to the shortest time possible.
- Place no surcharge loads (equipment, materials) within 10 feet of the top of the slope, in general. However, more or less stringent requirements may apply depending on field conditions and actual surcharge loads.
- Use special care when excavating through the soft peat layer.

Because of the variables involved, actual slope angles required for stability in temporary cut areas can be only estimated (not determined precisely) before construction. We recommend that stability of the temporary slopes used for construction be the sole responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. All excavations should be made in accordance with all local, state, and federal safety requirements.

Utilities

Utility trench cut design should generally be the responsibility of the contractor. For shallow trench excavations (less than 4 feet in depth), open cutting may be used. Use of temporary shoring may be necessary if deeper excavations are required for placement of utilities. The contractor should verify the conditions of the side slopes during construction and layback trench cuts as necessary to conform to current standards of practice.

The minimum dry densities recommended below are a percentage of the modified Proctor maximum dry density as determined by the ASTM D1557 test procedure. Our recommendations for bedding and trench backfill materials are:

- At least 4 inches of bedding is recommended for all pipe utilities. Bedding materials should consist of well-graded sand and gravel with less than 3 percent material passing the No. 200 sieve (based on the minus 3/4-inch fraction). Bedding material should be compacted to a firm non-yielding condition.

- The recommended bedding backfill materials can be used as backfill around the pipe utilities (pipe zone backfill). Pipe zone backfill should extend to at least the top of the pipe utility.

- For bedding material beneath catch basins and manholes, we recommend 6 inches of imported structural fill (or acceptable on-site material) that consists of well-graded sand and

gravel with less than 3 percent material passing the No. 200 sieve (based on the minus 3/4-inch fraction). The bedding material should be compacted to 90 percent.

- Utilities that extend below the groundwater table should be evaluated for the potential to float out of the ground at the high groundwater level.

- Deeper utilities may require dewatering well points to obtain a suitable working base. The contractor may elect to place a geotextile fabric at the base of the excavation to help create a suitable working surface.

Utility Vaults. We recommend designing utility vaults to resist both the compressive load of the vault on the subgrade and the hydrostatic uplift force of the groundwater table acting on the base of the vault (i.e., design to preclude the vault from floating out of the ground when empty). Depending on space requirements, excavation for utility vaults may require temporary shoring.

CONTINUING GEOTECHNICAL SERVICES

Design and Consulting Services

Throughout this report, we recommend that we provide additional geotechnical input during the design and construction process. These recommendations are generally summarized in this section.

We recommend that, before construction begins, we:

- Continue to meet with the design team periodically as design concepts and design documents progress,
- Provide an update to this report as part of final design process, if necessary, and
- Review the final design plans to verify that the geotechnical engineering recommendations have been properly interpreted and implemented into the design.

Construction Services

During the construction phase of the project, we recommend retaining us to observe the following activities:

- Installation and testing of shoring system elements;
- Placement and density testing of structural fill at the site (if any);
- Installation of wall drainage;
- Backfilling of utility trenches or around subgrade walls; and
- Other observations as required by DPD.

We should also:

- Attend meetings as needed and

- Assist with other geotechnical engineering considerations that may arise during construction.

The purpose of our observations will be to verify compliance with design concepts and recommendations, and to allow design changes or evaluation of appropriate construction methods in case subsurface conditions differ from those anticipated before construction begins.

REFERENCES

ASCE 2010. Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-10.

FHWA 1999. Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems. FHWA-IF-99-015.

Idriss, I.M. and R.W. Boulanger 2008. Soil Liquefaction During Earthquakes. EERI Publication, MNO-12.

International Code Council 2012. 2012 International Building Code.

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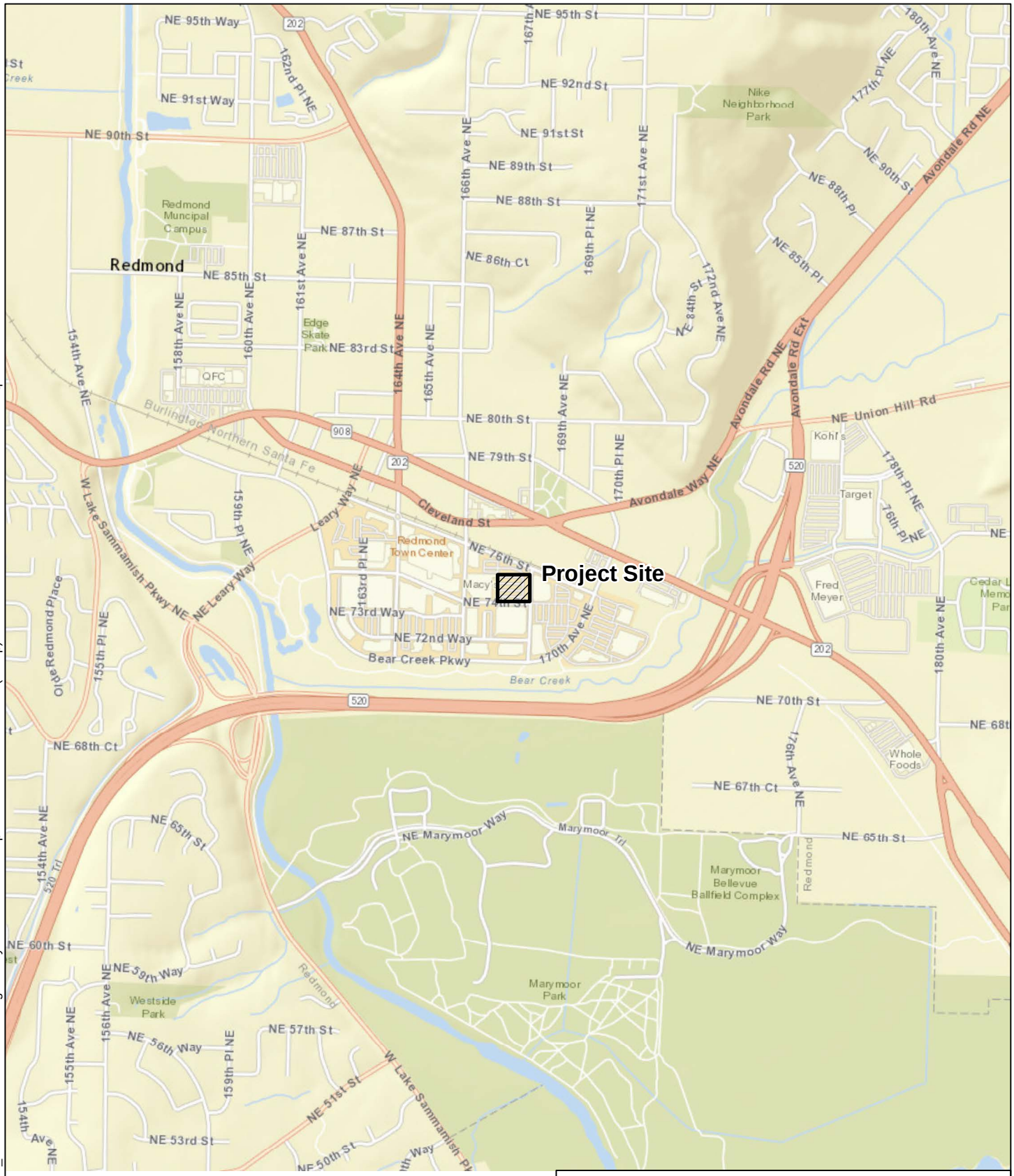

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1,000 500 0 1,000

Scale in Feet

N

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Redmond, Washington

Vicinity Map

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Figure

1

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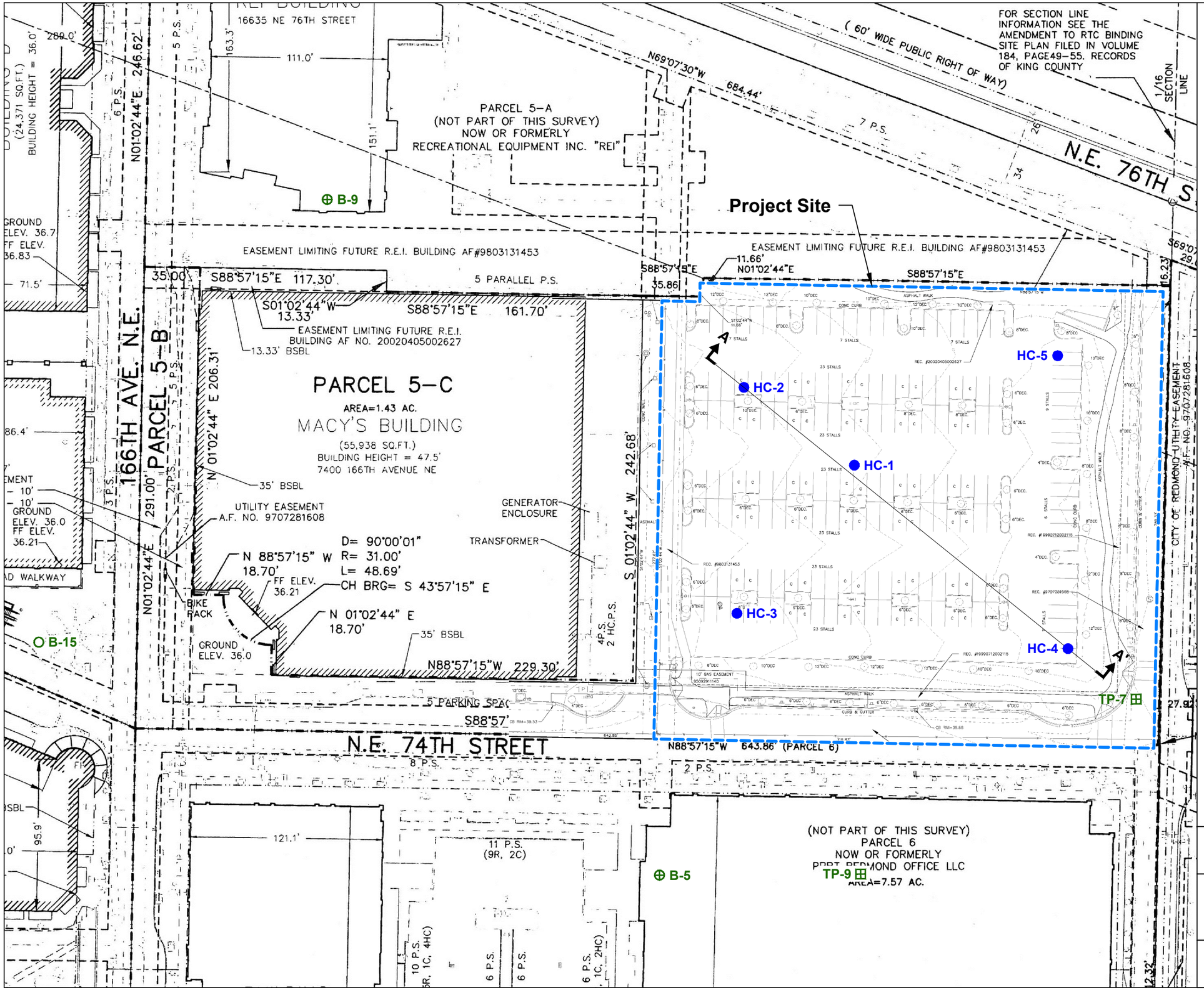

Legend

- HC-1 ● Boring (Hart Crowser 2015)
- B-15 ○ Boring (Hart Crowser 1982)
- TP-7 ⊞ Test pit (Hart Crowser 1982)
- B-5 ⊕ Boring (Hart Crowser 1979)
- A A' Cross section

0 60 120
Scale in Feet

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Site and Exploration Plan

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Figure
2

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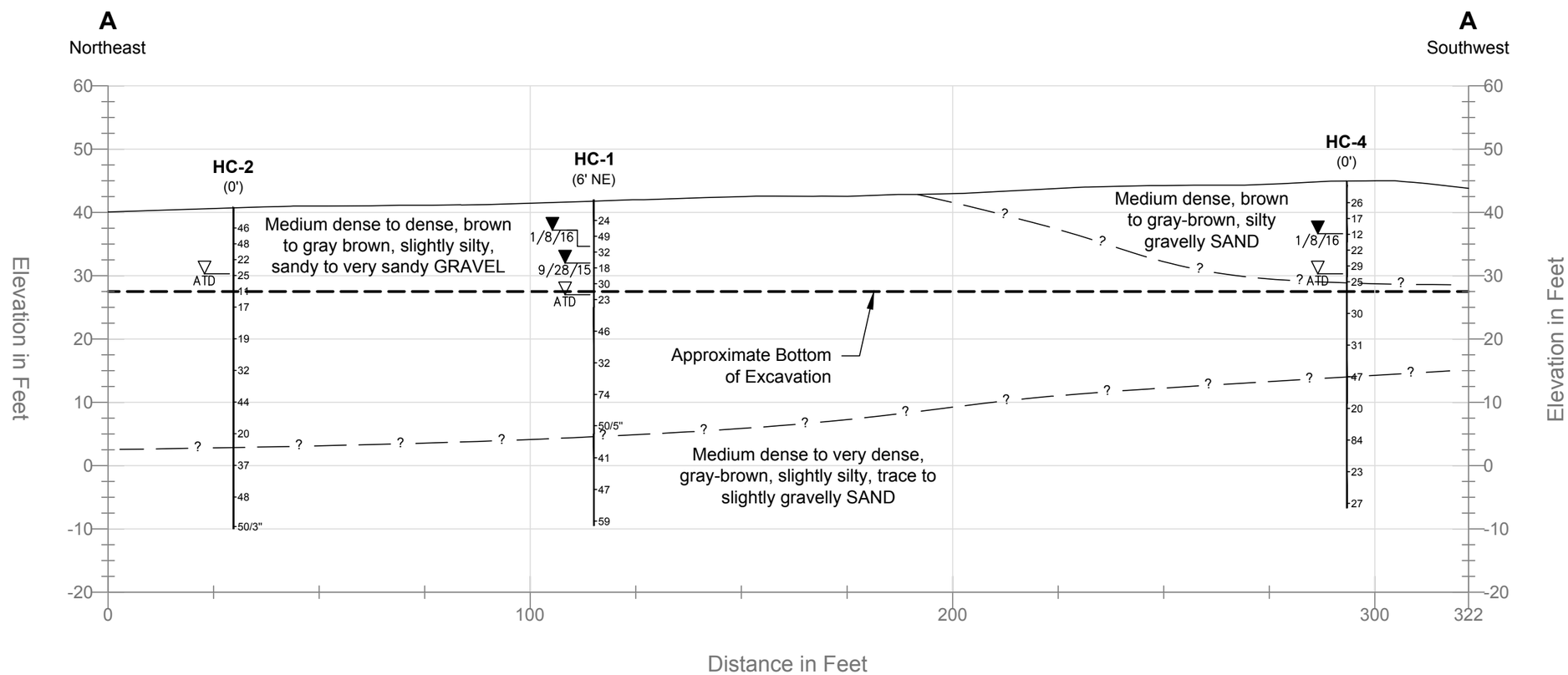

HC-1 (6' NE) — Exploration Number (Offset Distance and Direction)

Exploration Location

ATD — Water Level

9 — Standard Penetration Resistance in Blows per Foot

Horizontal Scale in Feet
0 30 60
0 20 40
Vertical Scale in Feet
Vertical Exaggeration x 1.5

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Generalized Subsurface Cross Section A-A'

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Figure
3

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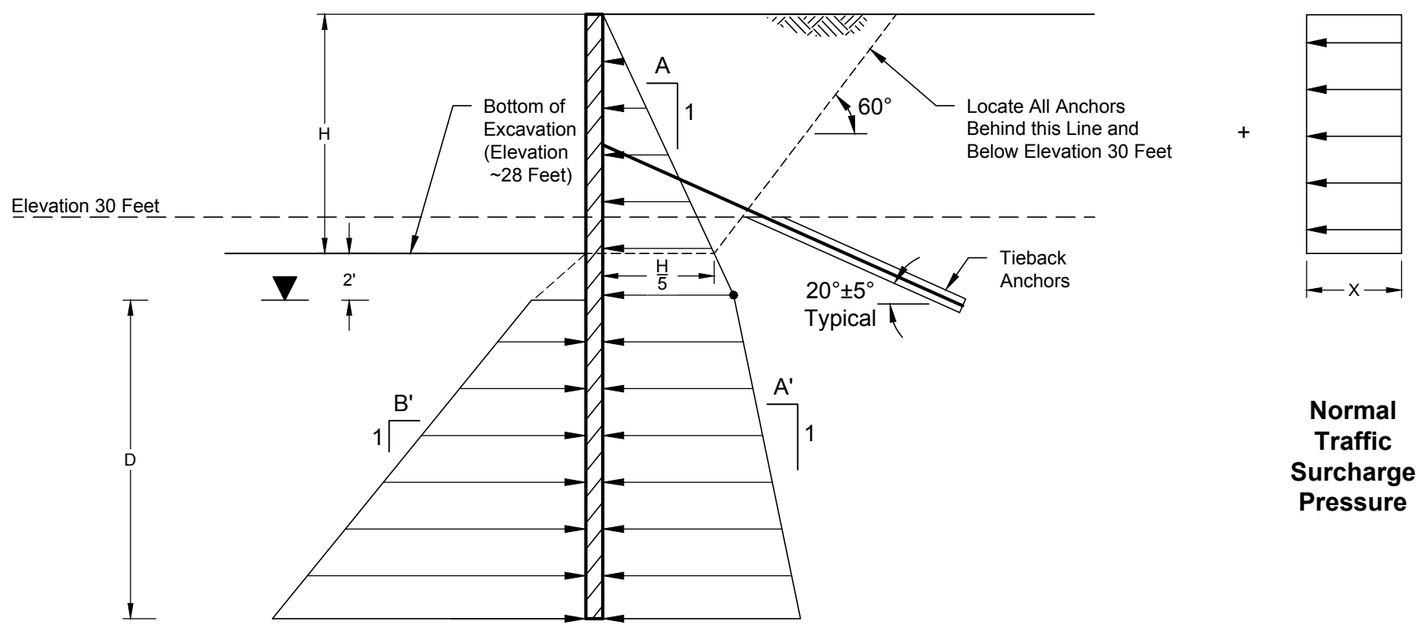
A. Lateral Soil Pressure



Passive Earth Pressure

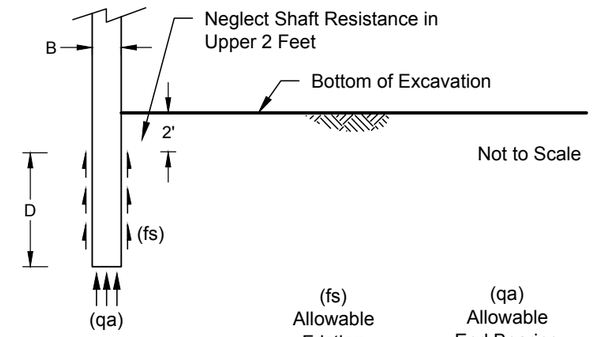
Active Earth Pressure

Normal Traffic Surcharge Pressure

Recommended Lateral Earth Pressures - Cantilever Wall 2					
	A (above GWT)	A' (below GWT)	B (above GWT)	B' (below GWT)	X
Active	37	18	--	--	75
At-Rest	57	29	--	--	125
Passive	--	--	280	140	--

GWT: Groundwater Table

B. Vertical Capacity of Soldier Pile



	(fs) Allowable Friction	(qa) Allowable End Bearing
Native Sand and Gravel	1.5 KSF	$\frac{4D}{B} \leq 40KSF$

C. Preliminary Anchor Pullout Resistance

For design proposes, use allowable load transfer (adhesion) as follows for cased, at least 6-inch-diameter boreholes

	Allowable Friction (Adhesion)
Native Sand and Gravel	2.0 KLF

1. A minimum anchor load length (bonded zone) of 15 feet is recommended.
2. Verify with Load Test to 200% of Design Stress Level. See Attachment 1.

Notes:

1. All earth pressures are in units of pounds per square foot.
2. Passive pressures are allowable values and include a factor of safety of 1.5.
3. Passive pressure acts over 2 times the concrete diameter of the soldier pile or pile spacing, whichever is less. Ignore upper 2 feet of passive resistance.
4. Active/At-Rest earth pressures and surcharge pressures act over the pile spacing.
5. Use at-rest lateral earth pressures if settlement sensitive utilities exist behind the wall.
6. All dimensions are in feet.
7. Diagrams are not to scale.
8. X is the lateral earth pressure for a minimum traffic surcharge of 250 psf. Additional surcharge should be considered if heavier traffic conditions apply. Refer to Figure 6 for surcharge scenarios.
9. Recommended minimum embedment depth (D) is 10 feet.
10. It is assumed that the groundwater will be lowered to at least 2 feet below the bottom of excavation (about elevation 28 feet) so that hydrostatic pressures do not act on the wall.

Legend

H = Excavation height

D = Soldier pile embedment depth

fs = Allowable skin friction resistance

qa = Allowable end-bearing resistance

KLF = kips per lineal foot

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Lateral Earth Pressures for Single Support or Cantilevered Soldier Pile Wall with Lagging

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Figure **4**

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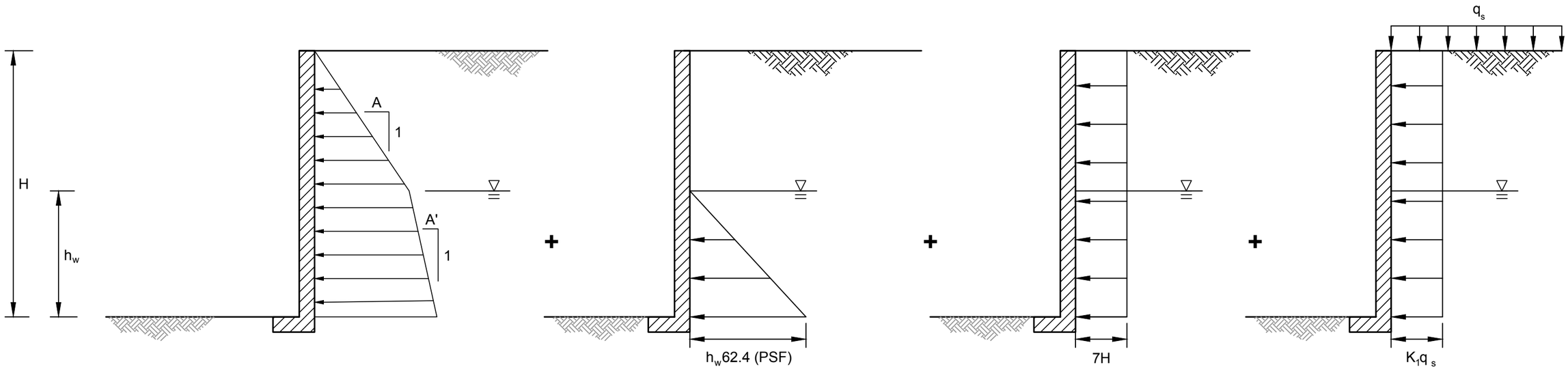

Earth Pressure*

Net Hydrostatic Pressure

Seismic Earth Pressure

Lateral Earth Pressure from Uniform Surcharge**

* The same earth pressure distributions (A and A' values) determined for temporary shoring should be used for permanent walls constructed against shoring (See Figure 4).

** See Figure 6 for K_1

Notes

1. All pressures are in units of pounds per square foot.
2. Diagrams do not include surcharge loading due to adjacent structures; see Figure 5.
3. Diagrams not to scale.
4. Assume a permanent groundwater elevation of 34 feet.

Legend

H	Height from bottom of excavation to ground surface in feet
q_s	Traffic surcharge
h_w	Depth of excavation below groundwater table
▽	Groundwater table

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Lateral Earth Pressures for Permanent Walls Constructed Against Shoring

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Figure **5**

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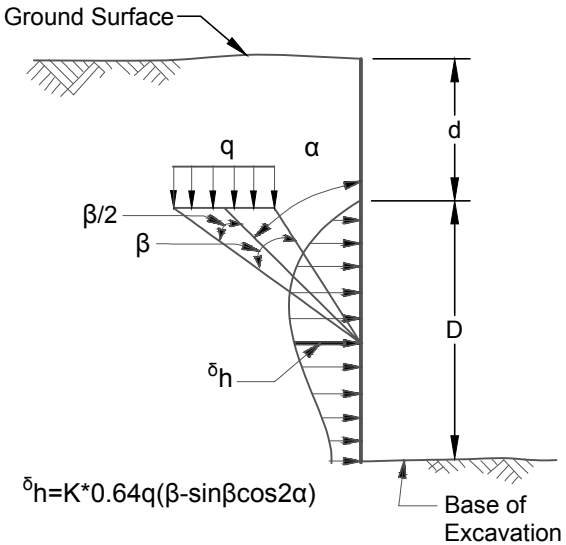
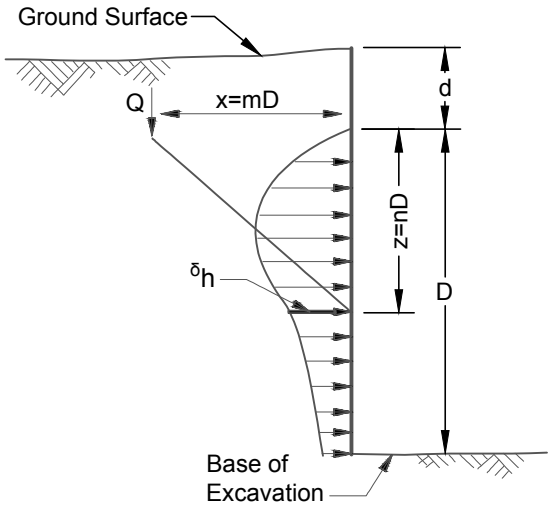
A. Strip Footing Cross Section View



$\delta h = K * 0.64q(\beta - \sin\beta\cos 2\alpha)$

B(1). Small Isolated Footing Cross Section View



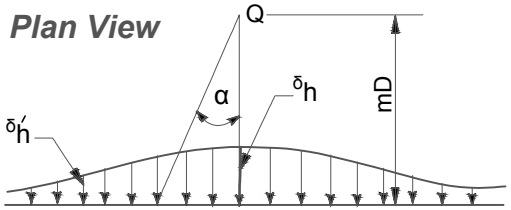
(For m>0.4)

$$\delta h = K_1 \frac{1.77Q}{D^2} \frac{m^2n^2}{(m^2n^2)^3}$$

(For m≤ 0.4)

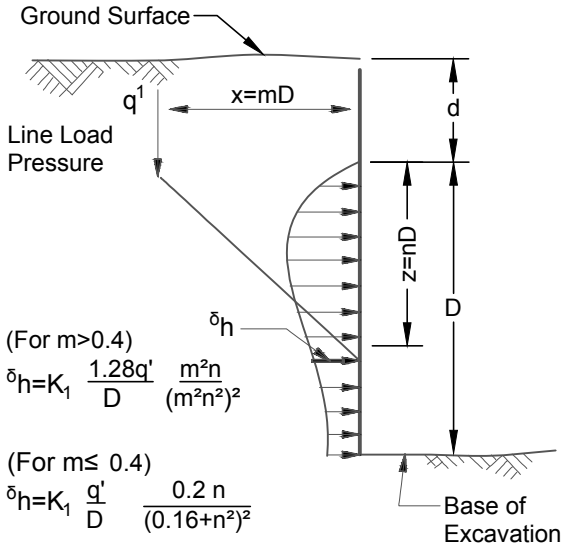
$$\delta h = K_1 \frac{0.28Q}{D^2} \frac{n^2}{(0.16+n^2)^3}$$

B(2). Plan View



$\delta h' = \delta h \cos^2(1.1\alpha)$

C. Continuous Wall Footing Parallel to Excavation Cross Section View



(For m>0.4)

$$\delta h = K_1 \frac{1.28q'}{D} \frac{m^2n}{(m^2n^2)^2}$$

(For m≤ 0.4)

$$\delta h = K_1 \frac{q'}{D} \frac{0.2\,n}{(0.16+n^2)^2}$$

Definition and Units

Q	Footing Load in Pounds
D	Excavation Depth below Footing in Feet
d	Depth to Base of Footing in Feet
δh	Lateral Soil Pressure in PSF
q	Unit Loading Pressure in PSF
q'	Footing Load in Pounds per Foot
α, β	Radians

K_1	Conditions
0.35	Active earth pressure on a flexible wall (e.g., shoring)
0.5	At-rest conditons, where surcharge loads exist prior to excavation
1.0	At-rest conditions, where surcharge loads are applied after construction on permanent wall

Notes:
1. Lateral pressures from adjacent structures should be added to lateral pressures on Figures 4 and 5.
2. Wall footings acting other than parallel to the excavation can be treated as series of discrete point loads, using Approach B.
3. Contact Hart Crowser for surcharge recommendations, if necessary.

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Lateral Earth Pressures on Adjacent Shoring Due to Surcharge Pressures

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Figure 6

APPENDIX A
Field Exploration Methods and Analysis

APPENDIX A

Field Exploration Methods and Analysis

This appendix documents the processes Hart Crowser used to determine the nature of the site soil and groundwater. This appendix includes information on the following subjects:

- Explorations and Their Location;
- Hollow-Stem Auger Borings;
- Standard Penetration Test Procedures;
- Modified California Sampler;
- Monitoring Well Installation;
- Monitoring Well Development; and
- Water Level Measurement

Explorations and Their Location

Subsurface explorations for this project included one hollow-stem auger (HSA) boring. The exploration logs in this appendix show our interpretation of the drilling, sampling, and testing data. The logs indicates the depth where soils change. Note that the actual change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on Figure A-1 - Key to Exploration Logs. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

Figure 2 illustrates the horizontal locations of explorations, which are based on field measurements from existing physical features. The elevations on the logs are taken from the elevation contours on the site plans provided to us. The vertical datum is NAVD88. All elevations presented in this report are based on NAVD88.

Hollow-Stem Auger Borings

We completed five HSA borings, designated HC-1 through HC-5. HC-1 was completed from September 25, 2015. HC-2 through HC-5 were completed from December 2 to December 4, 2015. The borings were completed to depths ranging from 50.8 to 62 feet using a 4-1/4-inch inside diameter hollow-stem auger. The borings were advanced with a truck-mounted drill rig subcontracted by Hart Crowser. The drilling was continuously observed by a geologist from Hart Crowser. Detailed field logs were prepared of each boring. Using the Standard Penetration Test (SPT), we obtained samples at 2-1/2- and 5-foot depth intervals.

Standard Penetration Test Procedures

The standard penetration test (SPT) method (as described in ASTM D 1586) was used to obtain disturbed samples. This test is an approximate measure of soil density and consistency. To be useful, the results must be used with engineering judgment in conjunction with other tests. The SPT test employs a standard 2-inch-outside-diameter split-spoon sampler. Using a 140-pound hammer, free-

falling 30 inches, the sampler is driven into the soil for 18 inches. The number of blows required to drive the sampler the last 12 inches only is the Standard Penetration Resistance. This resistance, or blow count, measures the relative density of granular soils and the consistency of cohesive soils. If a total of 50 blows are struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The blow counts are plotted on the boring logs at their respective sample depths.

Modified California Sampler

To improve soil recovery, a modified California split-spoon sampler was used during exploration. This 2.5-inch inside diameter (3.0-inch outside diameter) sampler is driven similar to a standard split-spoon sampler. Using a 140-pound hammer free-falling 30 inches, the sampler is driven into the soil for 18 inches. The density/consistency of the soil is interpreted to account for the larger diameter sampler and larger weight; however, actual sampler blow counts are presented on the logs as they were measured in the field.

Monitoring Well Installation

Monitoring wells were installed in boring HC-1 and HC-4 to allow for long-term groundwater level monitoring at the site.

We used 2-inch-diameter Schedule 40 PVC riser pipe and 2-inch-diameter 0.020-inch machine-slotted screen for the well casings and screens. The well screen and casing riser were lowered down through the hollow-stem auger. As the auger was withdrawn, No. 10/20 silica sand was placed in the annular (ring-shaped) space from the base of the boring to approximately 2 to 3 feet above the top of the well screen.

Well seals were constructed by placing bentonite chips in the annular space on top of the filter sand to within 3 feet of the ground surface. The remaining annular space was backfilled with concrete to complete the surface seal. For security, the monitoring wells were completed with a flush-mounted steel monument set in concrete. The monitoring well construction details are illustrated on the boring logs.

The monitoring wells were installed in accordance with Washington State Department of Ecology regulations.

Monitoring Well Development

Before well development work began, depth to water and depth to sediment were sounded with a Waterline well probe. Casing volume was calculated along with the target development volume of water to be extracted (usually 10 casing volumes).

The monitoring well was purged using a Whale electric submersible pump placed at the bottom of the casing. The volume of water extracted during purging was measured with a 5-gallon bucket and recorded. Observations of water clarity or turbidity were also documented.

After several casing volumes of water were removed from the well, the submersible pump was removed and the depths to water, sediment, and bottom of the well were measured. To effectively stress the filter pack to dislodge and remove fine particles, the well was surged with a surge block.

The procedure was continued until the pumped water contained a low visible sediment load and the sediment on the bottom of the well was removed, typically after removal of approximately 10 casing volumes.

Water Level Measurement

Water level in the monitoring well was measured using a Waterline water level probe, graduated in 0.01-foot increments. Depth to water was measured from the top of well casing, then adjusted to the ground surface elevation (casing is 0.25 feet below the ground surface).

Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 to 4	Very soft	0 to 2	<0.125
Loose	4 to 10	Soft	2 to 4	0.125 to 0.25
Medium dense	10 to 30	Medium stiff	4 to 8	0.25 to 0.5
Dense	30 to 50	Stiff	8 to 15	0.5 to 1.0
Very dense	>50	Very stiff	15 to 30	1.0 to 2.0
		Hard	>30	>2.0

Sampling Test Symbols

- 1.5" I.D. Split Spoon
- Grab (Jar)
- 3.0" I.D. Split Spoon
- Shelby Tube (Pushed)
- Bag
- Cuttings
- Core Run

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, likely below optimum
Moist	Likely near optimum moisture content
Wet	Much perceptible moisture, likely above optimum

Minor Constituents	Estimated Percentage
Trace	<5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Laboratory Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits

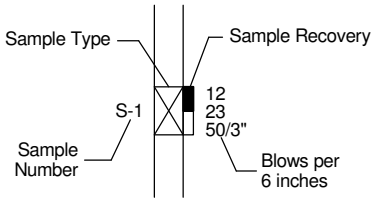
Water Content in Percent
Liquid Limit
Natural
Plastic Limit

PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	In Situ Density in PCF
OT	Tests by Others

Groundwater Indicators

- Groundwater Level on Date or (ATD) At Time of Drilling
- Groundwater Seepage (Test Pits)

Sample Key



HARTCROWSER

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Figure A-1

KEY SHEET 1916201-BL.GPJ HC_CORP.GDT 4/8/16

Boring Log HC-1

Location: Lat: 47.670450 Long: -122.117100
Approximate Ground Surface Elevation: 42 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Hollow Stem Auger
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 8 inches
Logged By: B. McDonald Reviewed By: J. Bruce

USCS Class	Soil Descriptions	Depth in Feet
GW-GM	6 inches of Asphalt over medium dense to dense, moist, brownish gray to gray, slightly silty, very sandy GRAVEL with occasional cobbles and boulders.	0
GW-GM	Dense to very dense, wet, gray, slightly silty, very sandy GRAVEL.	15
SW-SM	Dense to very dense, wet, brownish gray to gray, slightly silty SAND to SAND with trace gravel.	37

Well Construction: Flush mount monument; Concrete; Bentonite chips; 10-20 Silica sand; Screened 2" PVC; 9/28/15

Sample	Blows per Foot	LAB TESTS
S-1		
S-2	2 12 12	
S-3	13 27 22	
S-4	9 14 18	
S-5	5 8 10	
S-6	10 15 15	GS
S-7	3 10 13	
S-8	13 19 27	GS
S-9	9 15 17	GS
S-10	12 30 44	GS
S-11	19 38 50/5"	
S-12	6 11 30	GS

STANDARD PENETRATION RESISTANCE: ▲ Blows per Foot (0, 10, 20, 30, 40, 50+)

● Water Content in Percent (0, 20, 40, 60, 80, 100+)

NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

Boring Log HC-1

Location: Lat: 47.670450 Long: -122.117100
Approximate Ground Surface Elevation: 42 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Hollow Stem Auger
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 8 inches
Logged By: B. McDonald Reviewed By: J. Bruce

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Well Construction	Sample	Blows per Foot	LAB TESTS
SW-SM		Dense to very dense, wet, brownish gray to gray, slightly silty SAND to SAND with trace gravel. (cont'd)	45		S-13 (16, 25, 22)		
			50		S-14 A/B (12, 22, 37)		
		Sandy GRAVEL.					
		Bottom of Boring at 51.5 Feet.					
		Started 09/25/15.					
		Completed 09/25/15.					
		Ecology Well Tag #BJB-600	55				
			60				
			65				
			70				
			75				
			80				
			85				
			90				

STANDARD PENETRATION RESISTANCE

Blows per Foot: 0 10 20 30 40 50+

Water Content in Percent: 0 20 40 60 80 100+

NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

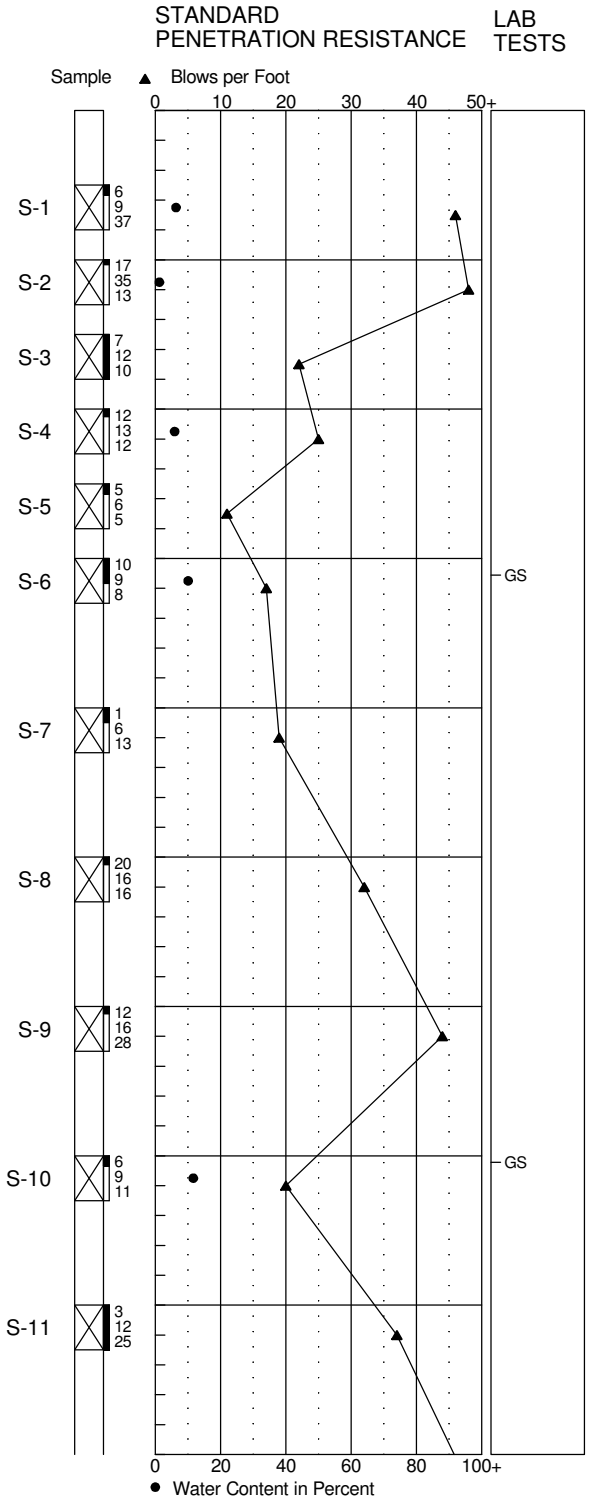
HARTCROWSER

Boring Log HC-2

Location: Lat: 47.670614 Long: -122.117303
Approximate Ground Surface Elevation: 40.8 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
		6 inches of Asphalt.	0
GW-GM		Medium dense to dense, moist to wet, brown to gray-brown, slightly silty, very sandy GRAVEL.	
		Cobbles moving up flights.	10 ATD
SP		Becomes very gravelly SAND with trace silt.	
		Heaving conditions; blow count might not be representative.	35
SP		Dense to very dense, wet, gray, fine to coarse SAND with trace gravel.	



NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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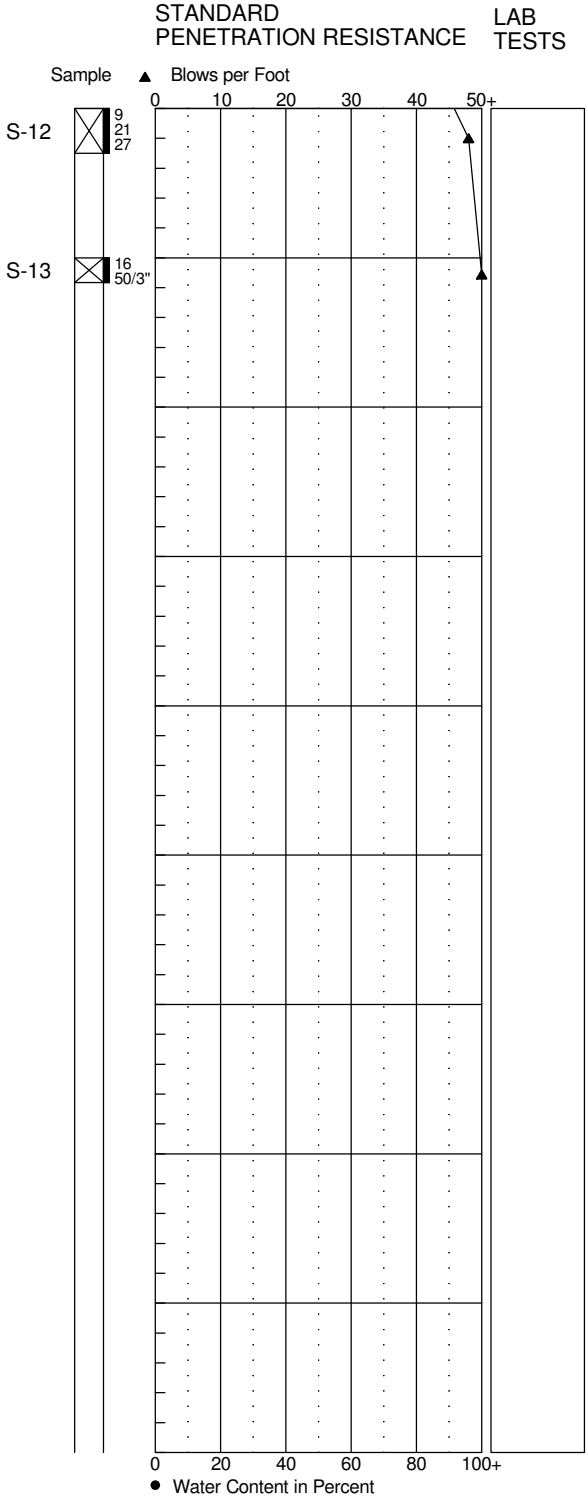
Figure A-3 1/2

Boring Log HC-2

Location: Lat: 47.670614 Long: -122.117303
Approximate Ground Surface Elevation: 40.8 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
SP		Dense to very dense, wet, gray, fine to coarse SAND with trace gravel. (cont'd)	45
		Heaving conditions; blow count might not be representative.	50
		Bottom of Boring at 50.8 Feet. Started 12/02/15. Completed 12/02/15.	55
			60
			65
			70
			75
			80
			85
			90



NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

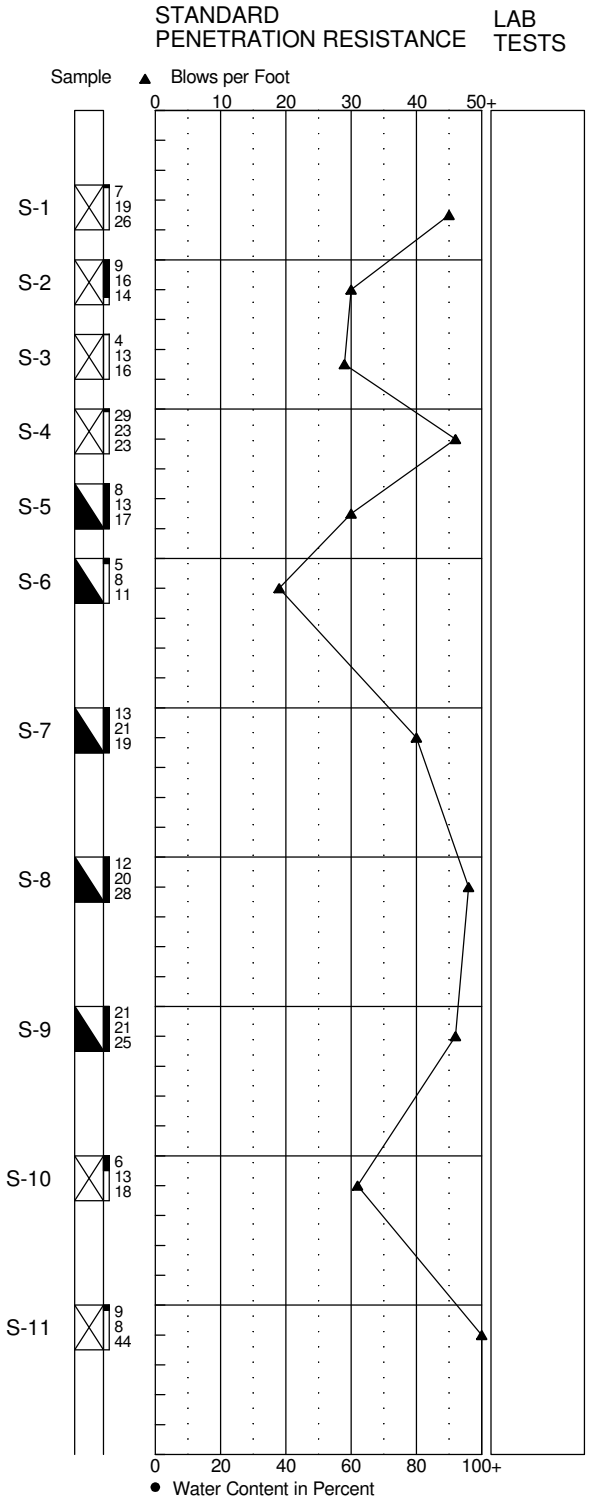
19162-01, -02 *12/15*
Figure A-3 *2/2*

Boring Log HC-3

Location: Lat: 47.670228 Long: -122.117336
Approximate Ground Surface Elevation: 40.5 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
		6 inches of Asphalt.	0
SW		Dense, moist, gray-brown, slightly silty, very gravelly, fine to coarse SAND. (FILL?)	5
GP		Medium dense to dense, wet, gray-brown, fine to coarse sandy GRAVEL.	10 ATD
		Trace silt and cobbles.	15, 20, 25
SW SP		Dense to very dense, wet, gray-brown to gray, gravelly, fine to coarse SAND.	30, 35, 40, 45



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

19162-01, -02 *12/15*

Figure A-4 *1/2*

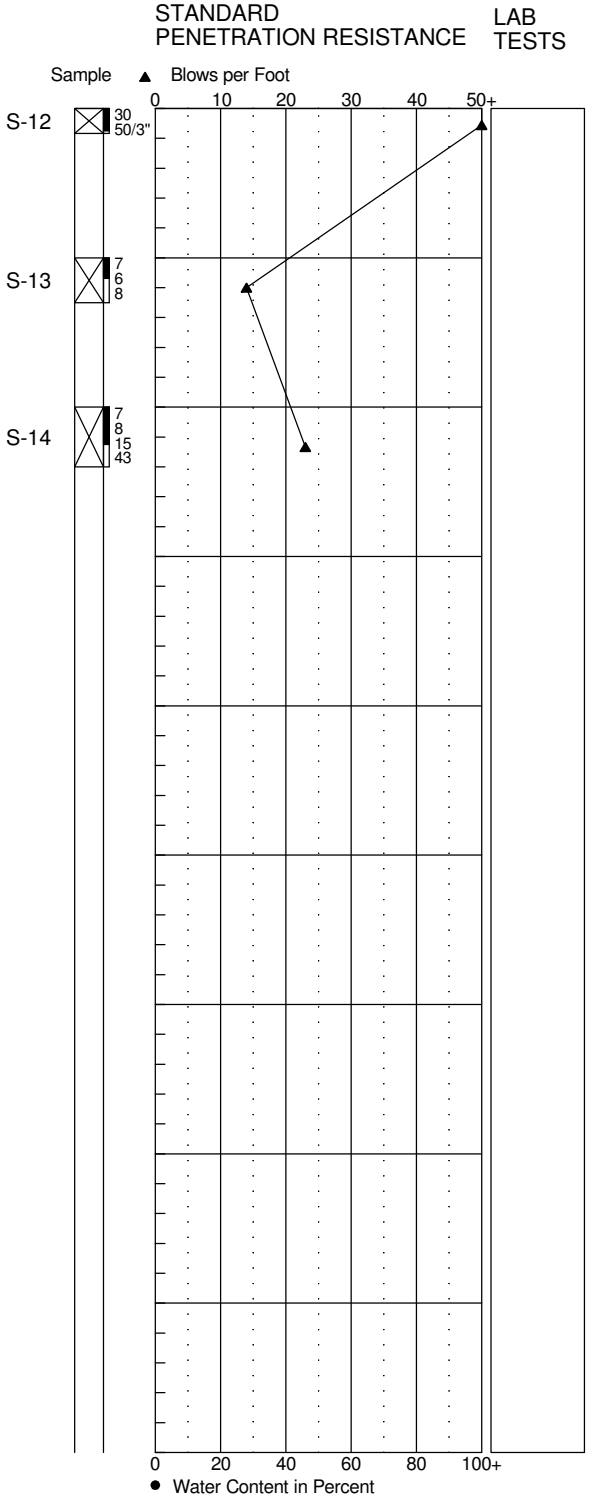
NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

Boring Log HC-3

Location: Lat: 47.670228 Long: -122.117336
Approximate Ground Surface Elevation: 40.5 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
SW SP		Dense to very dense, wet, gray-brown to gray, gravelly, fine to coarse SAND. (cont'd)	45
		Slightly silty last 7 feet.	50
			55
		Bottom of Boring at 57.0 Feet. Started 12/04/15. Completed 12/04/15.	60
			65
			70
			75
			80
			85
			90



NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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Figure A-4 *2/2*

Boring Log HC-4

Location: Lat: 47.670175 Long: -122.116483
Approximate Ground Surface Elevation: 44.8 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
	7 inches of Asphalt.	0
SM	Medium dense, moist, brown, silty, gravelly SAND. (FILL)	5
SM	Medium dense, moist, gray-brown, silty, gravelly SAND.	10
GP	Dense, wet, gray-brown, slightly sandy GRAVEL.	15 ATD
		20
		25
SP-SM	Becomes slightly silty, very gravelly SAND.	30
SW	Medium dense, wet, gray-brown, slightly gravelly to gravelly, fine to coarse SAND.	35
	Heaving conditions; blow count might not be representative.	40
		45

Well Construction: Flush mount monument; Concrete; Bentonite chips; 10-20 Silica sand; Screened 2" PVC

Sample	Blows per Foot
S-1	3, 4, 22
S-2	9, 9, 8
S-3	3, 3, 9
S-4	6, 12, 10
S-5	7, 14, 15
S-6	15, 15, 10
S-7	21, 21, 9
S-8	6, 14, 17
S-9	13, 18, 29
S-10	6, 9, 11
S-11	26, 45, 39

STANDARD PENETRATION RESISTANCE — Blows per Foot (0, 10, 20, 30, 40, 50+)

LAB TESTS: GS

Water Content in Percent (0, 20, 40, 60, 80, 100+)

NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

Boring Log HC-4

Location: Lat: 47.670175 Long: -122.116483
Approximate Ground Surface Elevation: 44.8 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Well Construction	Sample	Blows per Foot	LAB TESTS
SW		Medium dense, wet, gray-brown, slightly gravelly to gravelly, fine to coarse SAND. (cont'd) Slightly silty last 6.5 feet.	45		S-12 (11, 13, 10)		
			50		S-13 (14, 15, 12)		
		Bottom of Boring at 51.5 Feet. Started 12/03/15. Completed 12/03/15. Ecology Well Tag #BJZ-013	55				
			60				
			65				
			70				
			75				
			80				
			85				
			90				

STANDARD PENETRATION RESISTANCE

Blows per Foot: 0, 10, 20, 30, 40, 50+

Water Content in Percent: 0, 20, 40, 60, 80, 100+

NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

19162-01, -02 *12/15*

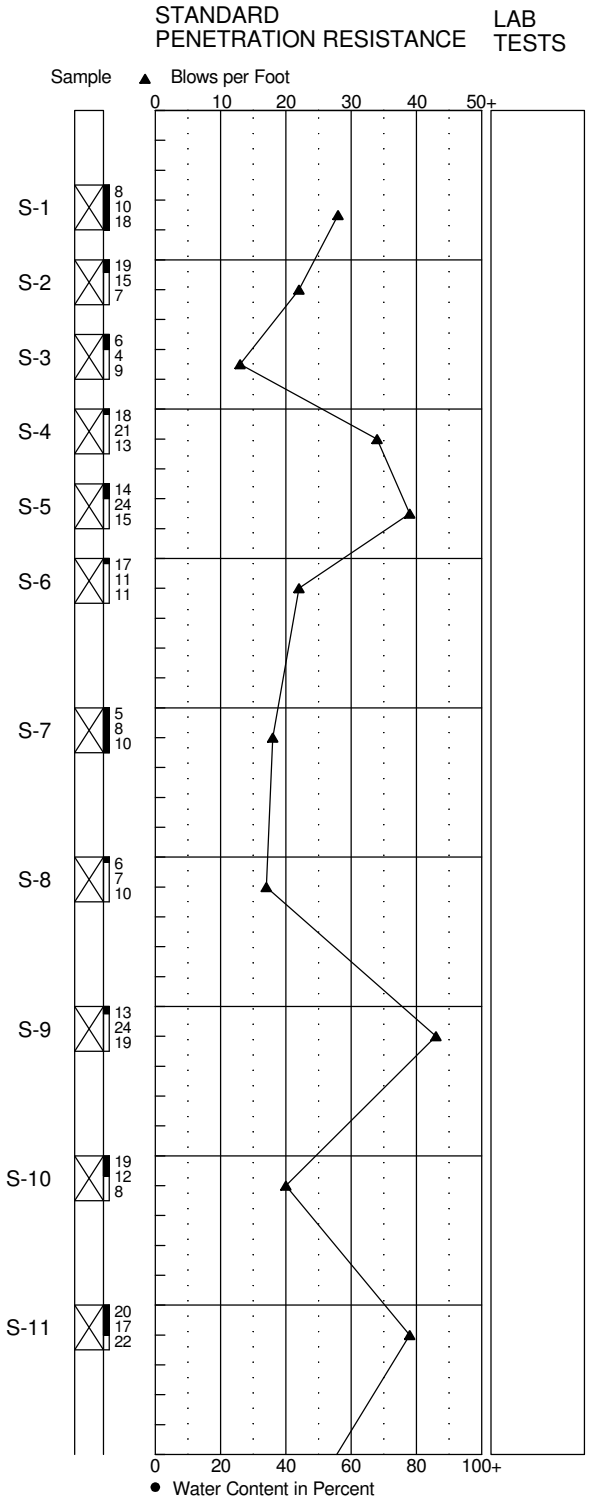
Figure A-5 *2/2*

Boring Log HC-5

Location: Lat: 47.670672 Long: -122.116511
Approximate Ground Surface Elevation: 44.2 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
	7 inches of Asphalt.	0
SM	Medium dense to dense, moist, gray to brown, silty, gravelly, fine to coarse SAND. (FILL)	5, 10
GP	Medium dense to dense, wet, gray-brown, fine to coarse sandy GRAVEL.	15 ATD
SP	Medium dense, wet, gray, fine to coarse SAND with trace gravel.	20, 25, 30, 35
	Slightly gravelly zones.	
SP	Medium dense, wet, gray, gravelly, fine to medium SAND.	40, 45



NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

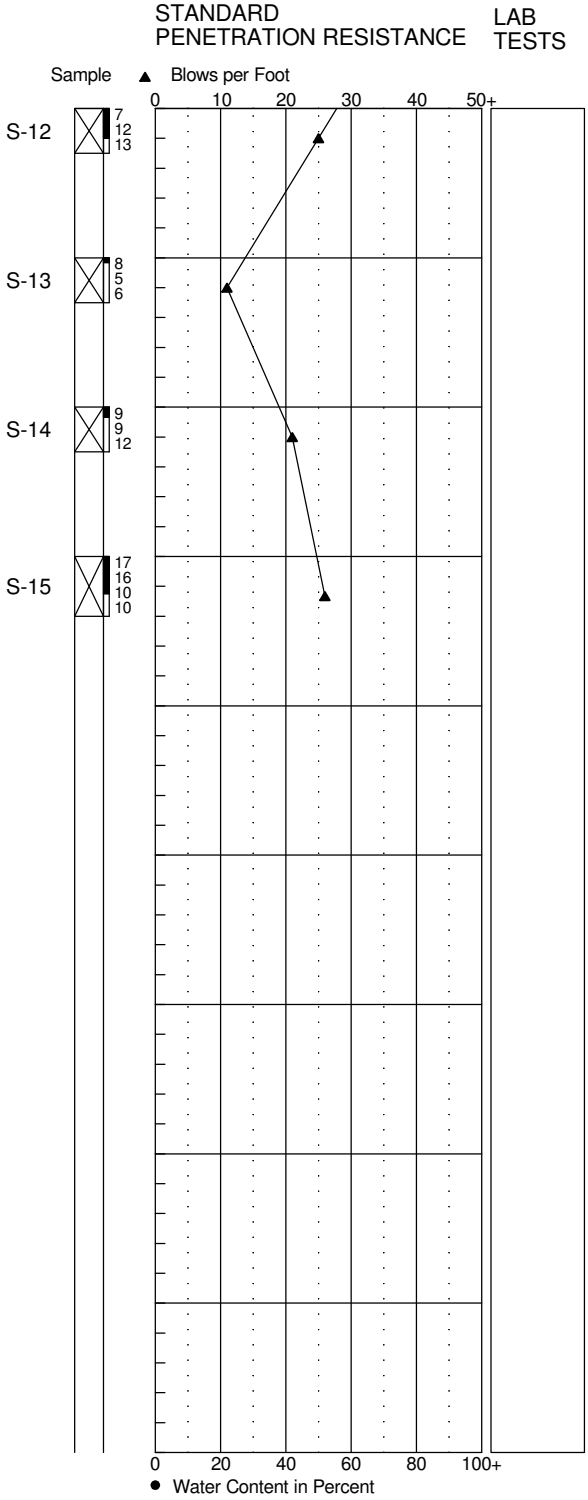
HARTCROWSER

Boring Log HC-5

Location: Lat: 47.670672 Long: -122.116511
Approximate Ground Surface Elevation: 44.2 Feet
Horizontal Datum: WGS84
Vertical Datum: NAVD88

Drill Equipment: Mobile B-60 Truck/HSA
Hammer Type: SPT w/ 140 lb. Autohammer
Hole Diameter: 4-1/4 ID inches
Logged By: W. McDonald Reviewed By: J. Thomas

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
SP		Medium dense, wet, gray, gravelly, fine to medium SAND. (cont'd)	45
			50
			55
			60
		Bottom of Boring at 62.0 Feet. Started 12/03/15. Completed 12/03/15.	
			65
			70
			75
			80
			85
			90



NEW BORING LOG 1916201-BL.GPJ HC_CORP.GDT 4/8/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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Figure A-6 *2/2*

APPENDIX B
Laboratory Testing Program

APPENDIX B

Laboratory Testing Program

A laboratory testing program was performed for this study to evaluate the basic index and geotechnical engineering properties of the site soils. The tests performed and the procedures followed are outlined below.

Soil Classification

Soil samples from the explorations were visually classified in the field and then taken to our laboratory where the classifications were verified in a relatively controlled laboratory environment. Field and laboratory observations include relative density/consistency, moisture condition, and grain size estimates.

The classifications of selected samples were checked by laboratory tests such as grain size analyses. Classifications were made in general accordance with the Unified Soil Classification (USC) System, ASTM D 2487, as presented on Figure B-1.

Water Content Determinations

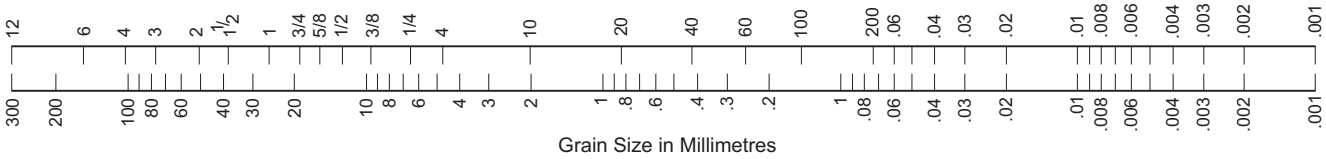
Water content was determined for most samples recovered in the explorations in general accordance with ASTM D 2216, as soon as possible following their arrival in our laboratory. Water contents were not determined for very small samples or samples where large gravel contents would result in unrepresentative values. The results of these tests are plotted or presented at the respective sample depth on the exploration logs. In addition, water contents are routinely determined for samples subjected to other testing. These are also presented on the exploration logs.

Grain Size Analysis

Grain size distribution was analyzed on representative samples in general accordance with ASTM D 422. Wet sieve analysis was used to determine the size distribution greater than the U.S. No. 200 mesh sieve. The results of the tests are presented as curves on Figures B-2 through B-4 plotting percent finer by weight versus grain size.

Unified Soil Classification (USC) System
Soil Grain Size

Size of Opening In Inches	Number of Mesh per Inch (US Standard)	Grain Size in Millimetres



COBBLES	GRAVEL	SAND	SILT and CLAY
Coarse-Grained Soils			Fine-Grained Soils

Coarse-Grained Soils

G W	G P	G M	G C	S W	S P	S M	S C
Clean GRAVEL <5% fines	*	GRAVEL with >12% fines		Clean SAND <5% fines	*	SAND with >12% fines	
GRAVEL >50% coarse fraction larger than No. 4				SAND >50% coarse fraction smaller than No. 4			
Coarse-Grained Soils >50% larger than No. 200 sieve							

G W and S W $\left(\frac{D_{60}}{D_{10}}\right)$ >4 for G W, >6 for S W & $1 \leq \left(\frac{(D_{30})^2}{D_{10} \times D_{60}}\right) \leq 3$

G P and S P Clean GRAVEL or SAND not meeting requirements for G W and S W

G M and S M Atterberg limits below A line with PI <4

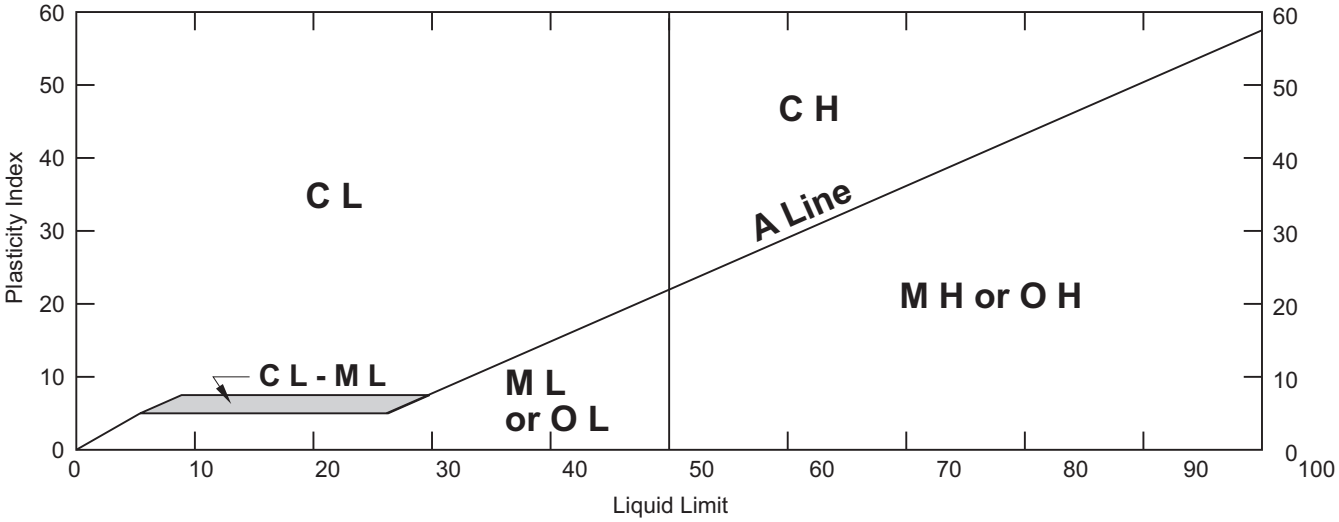
G C and S C Atterberg limits above A Line with PI >7

* Coarse-grained soils with percentage of fines between 5 and 12 are considered borderline cases requiring use of dual symbols.

D_{10}, D_{30}, and D_{60} are the particles diameter of which 10, 30, and 60 percent, respectively, of the soil weight are finer.

Fine-Grained Soils

M L	C L	O L	M H	C H	O H	Pt
SILT	CLAY	Organic	SILT	CLAY	Organic	Highly Organic Soils
Soils with Liquid Limit <50%			Soils with Liquid Limit >50%			
Fine-Grained Soils >50% smaller than No. 200 sieve						



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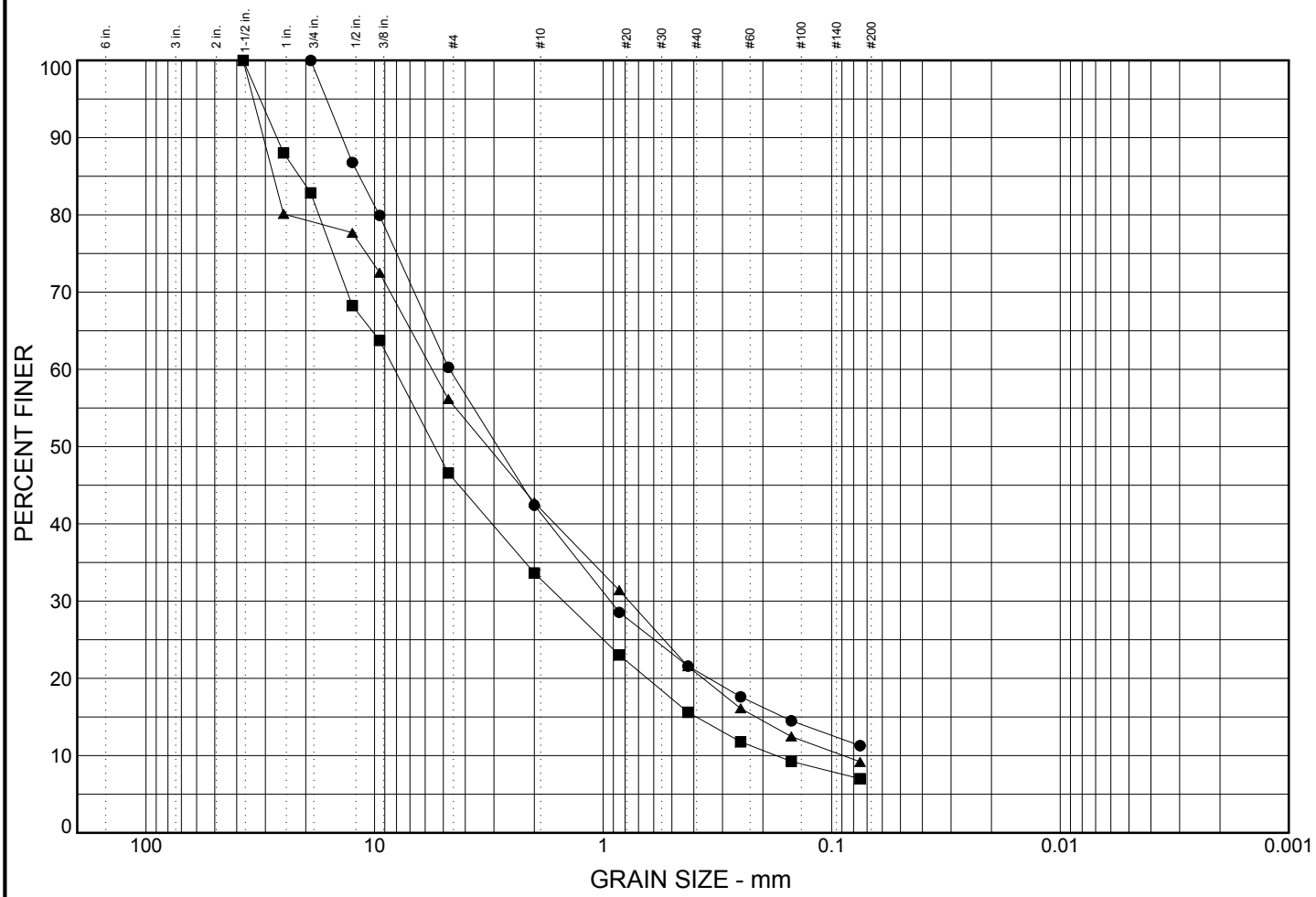
SRF Grain Size (B-1).cdr 3/06

19162-01 9/15

Figure B-1

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	39.7	49.0	11.3	
■	0.0	53.4	39.6	7.0	
▲	0.0	43.8	47.0	9.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			11.63	4.687	2.886	0.929	0.163		3.25	82.76
■			21.293	8.157	5.447	1.491	0.391	0.175	1.56	46.71
▲			27.614	5.59	3.191	0.769	0.214	0.089	1.19	62.89

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty, very gravelly SAND	SP-SM	5.8%
■ slightly silty, very sandy GRAVEL	GW-GM	8.9%
▲ slightly silty very gravelly SAND	SW-SM	9.5%

Remarks:

●

■

▲

Project: Macy's Parking Lot

Client:

● Source: HC-1	Sample No.: S-6	Depth: 12.5 to 14.0
■ Source: HC-1	Sample No.: S-8	Depth: 20.0 to 21.5
▲ Source: HC-1	Sample No.: S-9	Depth: 25.0 to 26.5

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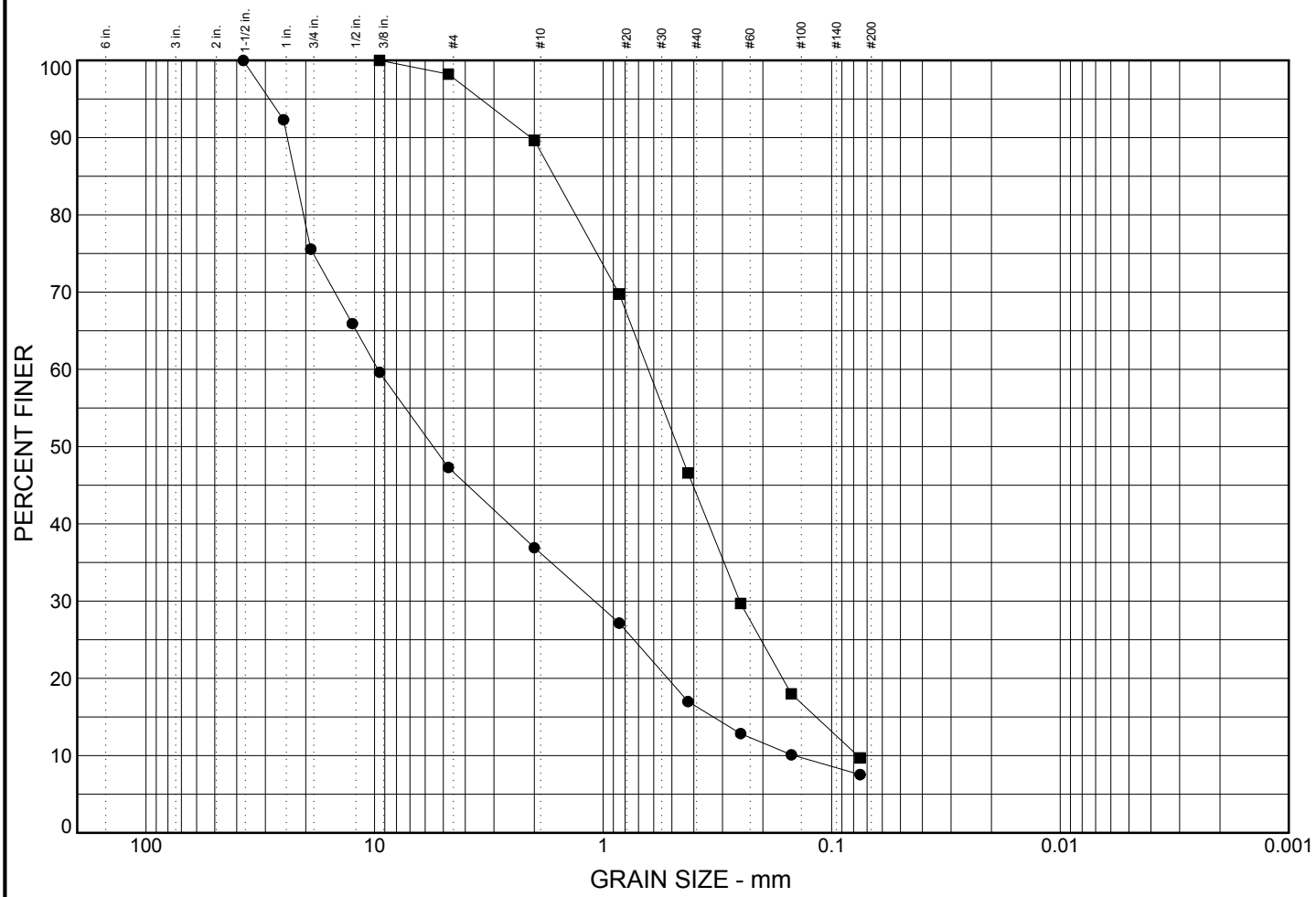
19162-01 *9/15*

Figure B-2

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	52.7	39.8	7.5	
■	0.0	1.8	88.5	9.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			22.172	9.65	5.525	1.09	0.329	0.146	0.85	66.30
■			1.636	0.635	0.471	0.252	0.117	0.077	1.30	8.24

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty, very sandy GRAVEL	GP-GM	8.5%
■ slightly silty SAND, trace gravel	SW-SM	17.5%

Remarks:

●

■

Project: Macy's Parking Lot

Client:

● Source: HC-1 Sample No.: S-10 Depth: 30.0 to 31.5

■ Source: HC-1 Sample No.: S-12 Depth: 40.0 to 41.5

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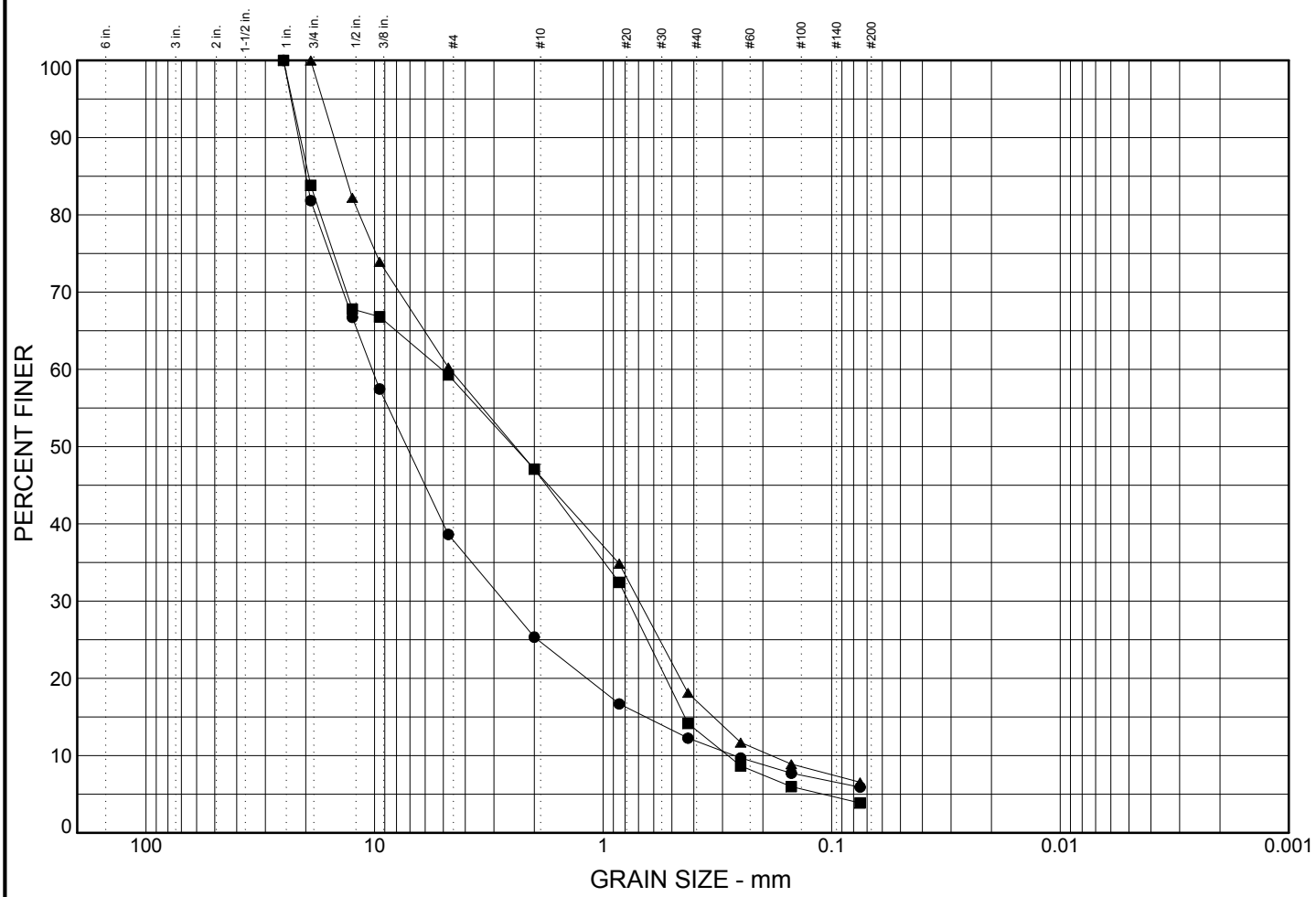
19162-01 *9/15*

Figure B-3

GRAIN SIZE 1916201-BL.GPJ HC_CORP.GDT 10/1/15

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	61.4	32.7	5.9	
■	0.0	40.7	55.4	3.9	
▲	0.0	39.7	53.7	6.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			19.926	10.239	7.216	2.708	0.652	0.267	2.69	38.41
■			19.379	5.069	2.456	0.775	0.438	0.283	0.42	17.88
▲			13.345	4.671	2.427	0.695	0.328	0.183	0.56	25.46

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty, very sandy GRAVEL	GW-GM	10.1%
■ very gravelly SAND, trace silt	SP	11.7%
▲ slightly silty, very gravelly SAND	SP-SM	10.9%

Remarks:

●

■

▲

Project: Redmond Town Center Parcel 5-B

Client:

● Source: HC-2 Sample No.: S-6 Depth: 15.0 to 16.5
■ Source: HC-2 Sample No.: S-10 Depth: 35.0 to 36.5
▲ Source: HC-4 Sample No.: S-9 Depth: 30.0 to 31.5

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19162-02 *9/15*

Figure B-4

GRAIN SIZE 1916201-BL.GPJ HC_CORP.GDT 1/11/16

APPENDIX C
Historical Explorations

APPENDIX C

Historical Explorations

We reviewed existing historical explorations to gain an understanding of the subsurface conditions near the site. Historical borings located in the vicinity are presented as they appear in original reports. Approximate locations of these historical explorations are shown on Figure 2; actual locations may differ from those shown.

Boring Log B-15

SOIL INTERPRETATION

STANDARD PENETRATION RESISTANCE (140 pound weight, 30 inch drop)

LABORATORY TESTS

Approximate Ground Surface Elevation in Feet

MEDIUM DENSE, WET, DARK BROWN, GRAVELLY, SILTY SAND TO SANDY SILT.

SLIGHTLY ORGANIC.

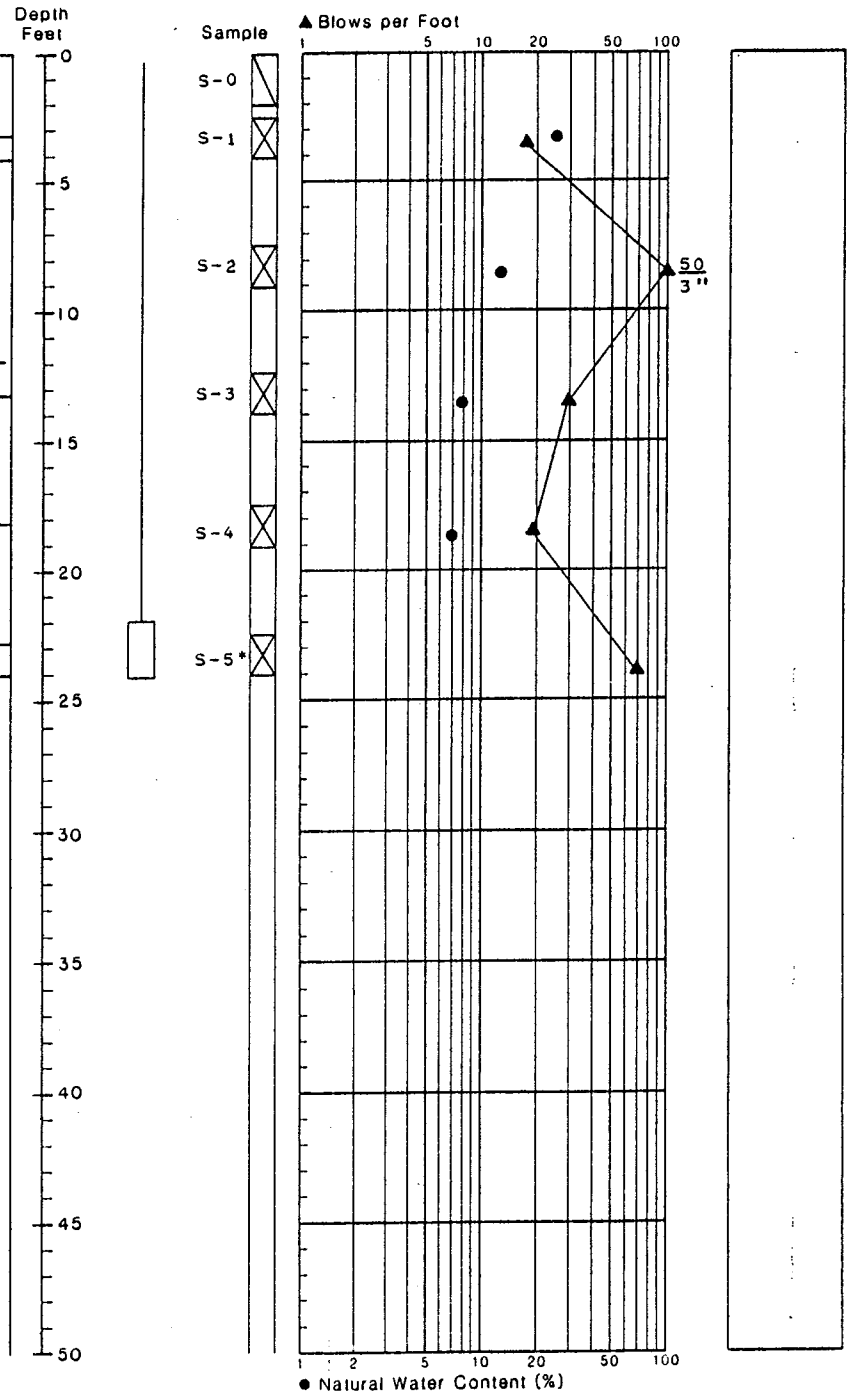
VERY DENSE, SATURATED, BROWN, GRAVELLY, SILTY SAND.

DENSE, GRAY AND TRACE SILT BELOW 12 FEET

MEDIUM DENSE.

VERY DENSE.

BOTTOM OF BORING 24.0 FEET
COMPLETED 12/15/81



Groundwater Level

Bentonite Seal

2/4/81 Water Level (Date)

ATD At Time of Drilling

Observation Well Tip or Slotted Section

Sampling

2" O.D. Split Spoon Sample

3" O.D. Shelby Tube Sample

Cutting Sample

* No Sample Recovery

Laboratory Tests

GS Grain Size Analysis

CN Consolidation Test

K Permeability Test

DS Direct Shear

QU Unconfined Compression, tsf

TV Torvane, tsf

PP Pocket Penetrometer, tsf

TUU Triaxial Unconsolidated Undrained

TCU Triaxial Consolidated Undrained

TCD Triaxial Consolidated Drained

Water Content (%)

Plastic Limit — Natural Water Content — Liquid Limit

Notes

1. Soil descriptions are interpretive and actual changes may be gradual.
2. Water Level, if indicated, is for the date specified and may vary with the time of year

J-791-01 December 1981

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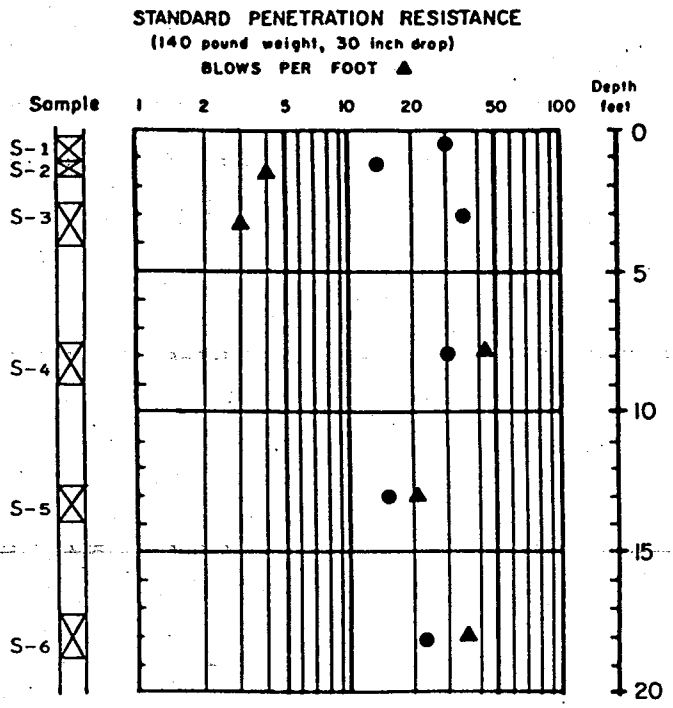
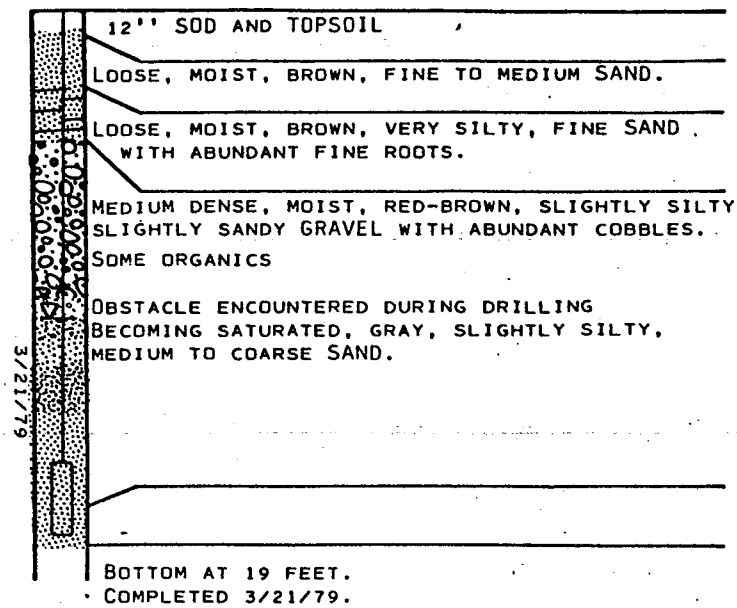
Figure A-5

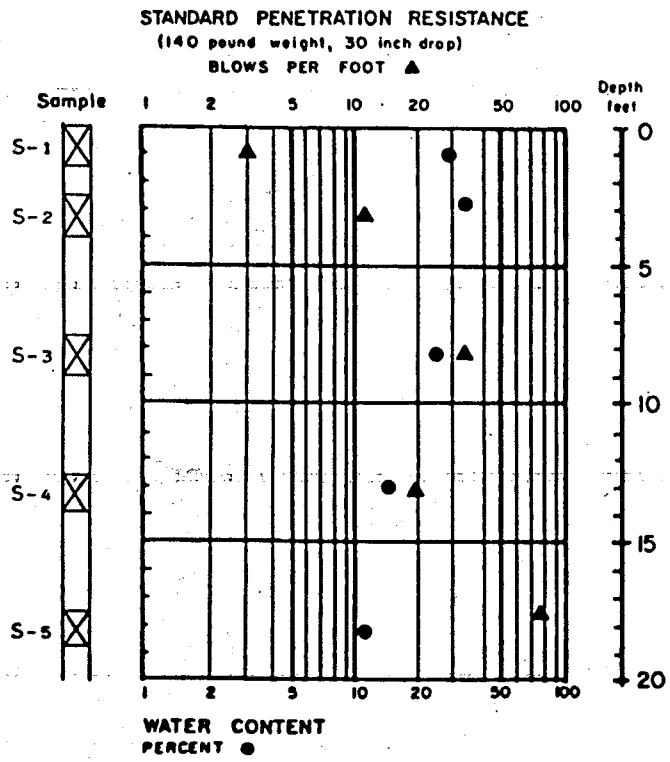
BORING LOG B-9



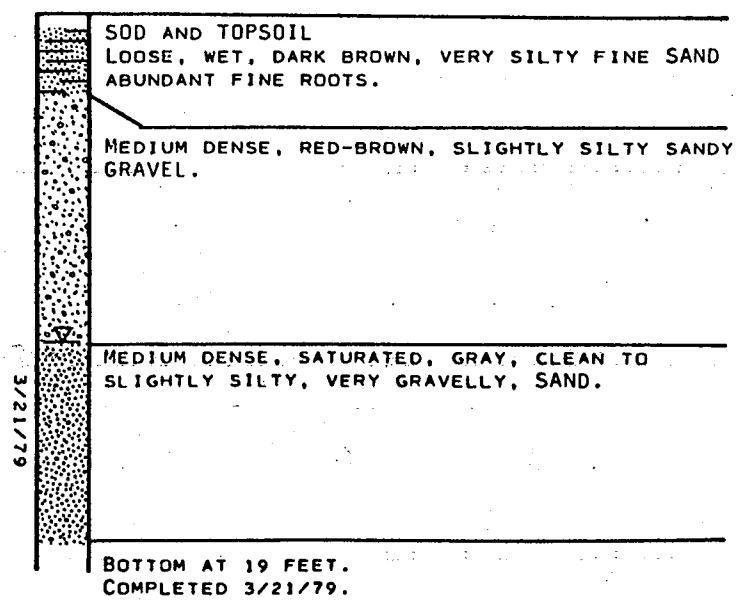
SOIL INTERPRETATION



BORING LOG B-10



SOIL INTERPRETATION



LEGEND

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- ★ No Sample Recovery
- ▽ Water Level
- ▯ Observation Well

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

J-791 March 1979

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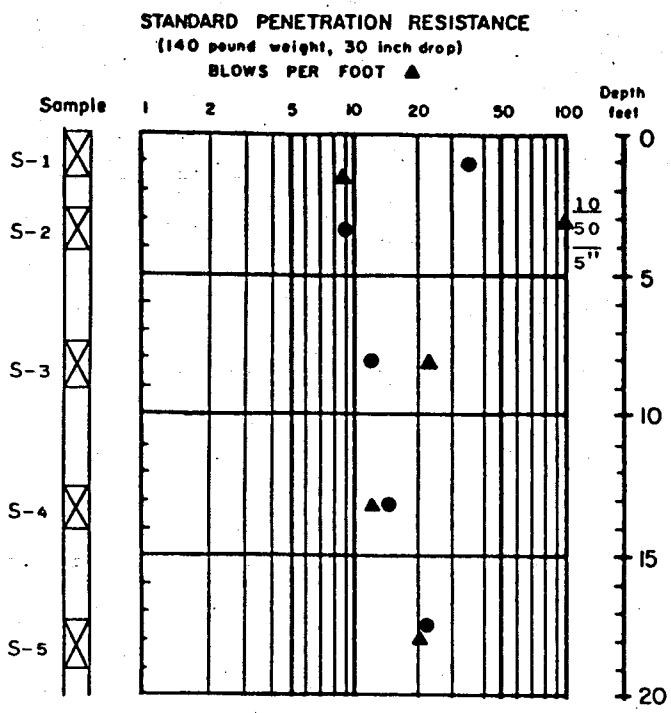
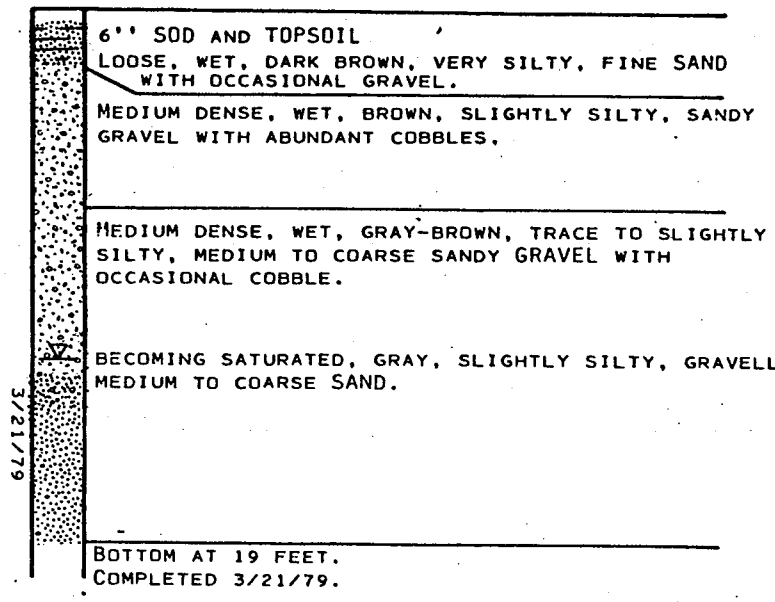
Figure A-6

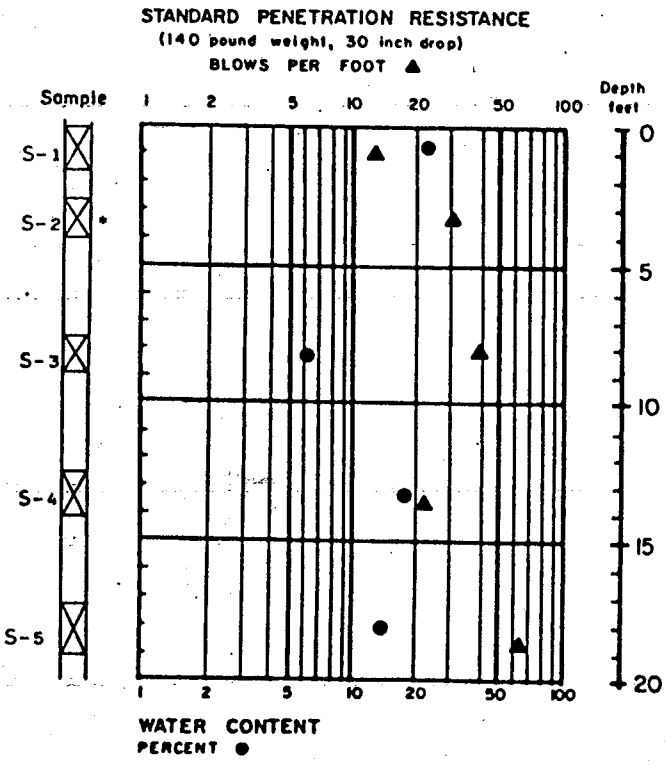
BORING LOG B-4



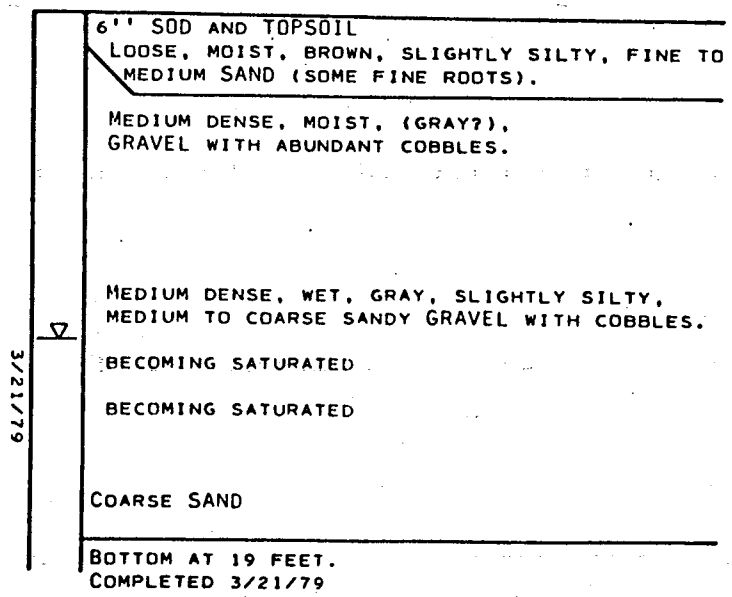
SOIL INTERPRETATION



BORING LOG B-5



SOIL INTERPRETATION



LEGEND

- ⊠ 2" O.D. Split Spoon Sample
- ⊠ 3" O.D. Shelby Sample
- ★ No Sample Recovery
- ▽ Water Level
- ▯ Observation Well

NOTE: Soil descriptions are interpretive and actual changes may be gradual.

J-791 March 1979

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Figure A-3

Test Pit Log TP-7

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
BS-1 S-1	32		0	SOD.
S-2	15		1	(LOOSE), WET, BROWN, SILTY FINE SAND WITH FEW FINE ROOTS.
S-3	46	PP=2.25	2	(LOOSE), MOIST, GRAY, FINE SAND.
			3	(STIFF), WET, GRAY-BROWN, FINE SANDY SILT.
S-4	25			(MEDIUM DENSE), WET, GRAY, SILTY, FINE SAND.
			4	BECOMING GRAVELLY.
				(MEDIUM DENSE), MOIST-WET, GRAY-BROWN, GRAVELLY SAND WITH COBBLES.
S-5	8		5	
			6	
			7	
S-6	14		8	SATURATED BELOW 7.5 FEET WITH INCREASING GRAVEL.
			9	BOTTOM OF TEST PIT 8.0 FEET COMPLETED 12/10/81
			10	NOTE: MODERATE SEEPAGE AT 7.5 FEET.
			11	
			12	
			13	
			14	
			15	

Test Pit Log TP-8

Sample	Water Content %	Other Tests	Depth feet	SOIL INTERPRETATION
BS-1 S-1	24		0	SOD.
S-2	7		1	(LOOSE), WET, BROWN, SLIGHTLY GRAVELLY, SILTY FINE TO MEDIUM SAND, SLIGHTLY ORGANIC AND FEW FINE ROOTS.
BS-3 S-3	33			
		INFILTROMETER	2	(MEDIUM DENSE), MOIST, TAN-BROWN SAND.
S-4	7	TEST		
		GRAIN	3	(MEDIUM DENSE), WET, BROWN, SLIGHTLY GRAVELLY, SILTY SAND, WEATHERED.
BS-2	6	SIZE		
			4	(MEDIUM DENSE), MOIST, GRAY-BROWN, SLIGHTLY COBBLY, GRAVELLY SAND.
			5	
S-5	7		6	GRADING TO: SANDY GRAVEL.
			7	
			8	BOULDER?
			9	BOTTOM OF TEST PIT 8.5 FEET COMPLETED 12/10/81
			10	
			11	
			12	
			13	
			14	
			15	

J-791-01 December 1981
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Figure A-9

APPENDIX D
Hydraulic Conductivity Memo (Slug Test)

MEMORANDUM

DATE: January 12, 2016

TO: Project File

FROM: Bridgette Brown
Roy Jensen, LHG

RE: **Summary of Slug Test Results**
Proposed Parcel 5B Development
Redmond, Washington
19162-01

This technical memorandum presents the results of slug testing that was conducted for the Proposed Parcel 5B Development in Redmond, Washington. Slug tests were performed to determine hydraulic conductivity of formation for use in estimating flow rates during dewatering.

Slug tests are performed by suddenly inserting or removing a solid PVC rod in a well and measuring the recovery of the water levels during the test. A test conducted by the insertion of the PVC rod into the well is referred to as a falling head test and the following removal of the rod is called a rising head test. The water level data generated from the tests were analyzed using the commercial software Aquifer^{Win32} Version 3 (Environmental Simulations, Inc., 2003). The slug test analysis is based on the Bouwer and Rice method (Bouwer and Rice 1976; Bouwer 1989) to obtain an estimated value of hydraulic conductivity of the aquifer.

Slug Testing Results

Slug testing was conducted in wells HC-1 and HC-4 January 8, 2016. A summary of monitoring well construction details is provided in Table 1. Shallow soils at the project site consist of sandy Gravel. The wells were screened in same stratigraphic units and are summarized below:

- HC-1 was screened in the sandy Gravel; and
- HC-4 was screened in the slightly silty, sandy Gravel.

A summary of slug testing results is provided in Table 2. Hydrographs of HC-1 and HC-4 are provided as Figure 1. The slug test plots are provided as Figure 2 and Figure 3. Multiple sets of falling and rising head tests were performed on each well. The results of the falling and rising head tests compare

favorably. Average hydraulic conductivities determined from slug tests range from 4.5×10^{-2} to 5.2×10^{-2} cm/sec (129 to 147 feet/day). This hydraulic conductivity range is typical for clean sand and gravel (Freeze and Cherry 1979).

References

Bouwer H. 1989. The Bouwer and Rice Slug Test – An Update. Ground Water 27(3): 304-309.

Bouwer H. and R.C. Rice 1976. A Slug Test for Determining Hydraulic Conductivity of Unconfined Aquifers with Completely or Partially Penetrating Wells. Water Resources Research 12(3): 423-428.

Environmental Simulations, Inc. 2003. Guide to Using Aquifer^{Win32} Version 3.

Freeze, R.A. and J.A. Cherry 1979. Groundwater. Prentice-Hall, Englewood Cliffs, New Jersey.

Attachments:
Table 1 – Monitoring Well Construction Summary
Table 2 – Summary of Slug Test Results
Figure 1 – HC-1 and HC-4 Hydrographs
Figure 2 – HC-1 Representative Slug Tests Results
Figure 3 – HC-4 Representative Slug Tests Results

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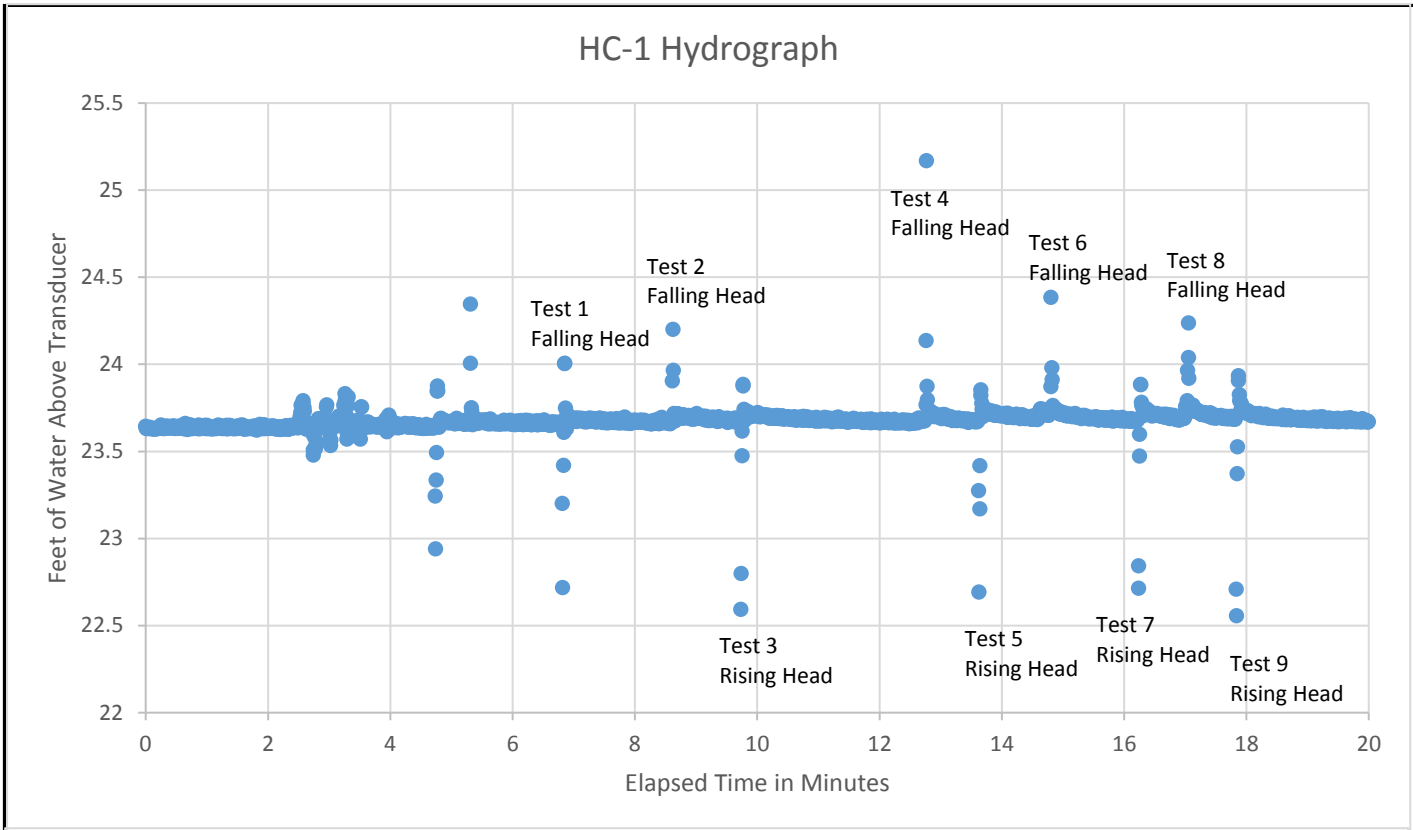
Table 1 - Monitoring Well Construction Summary

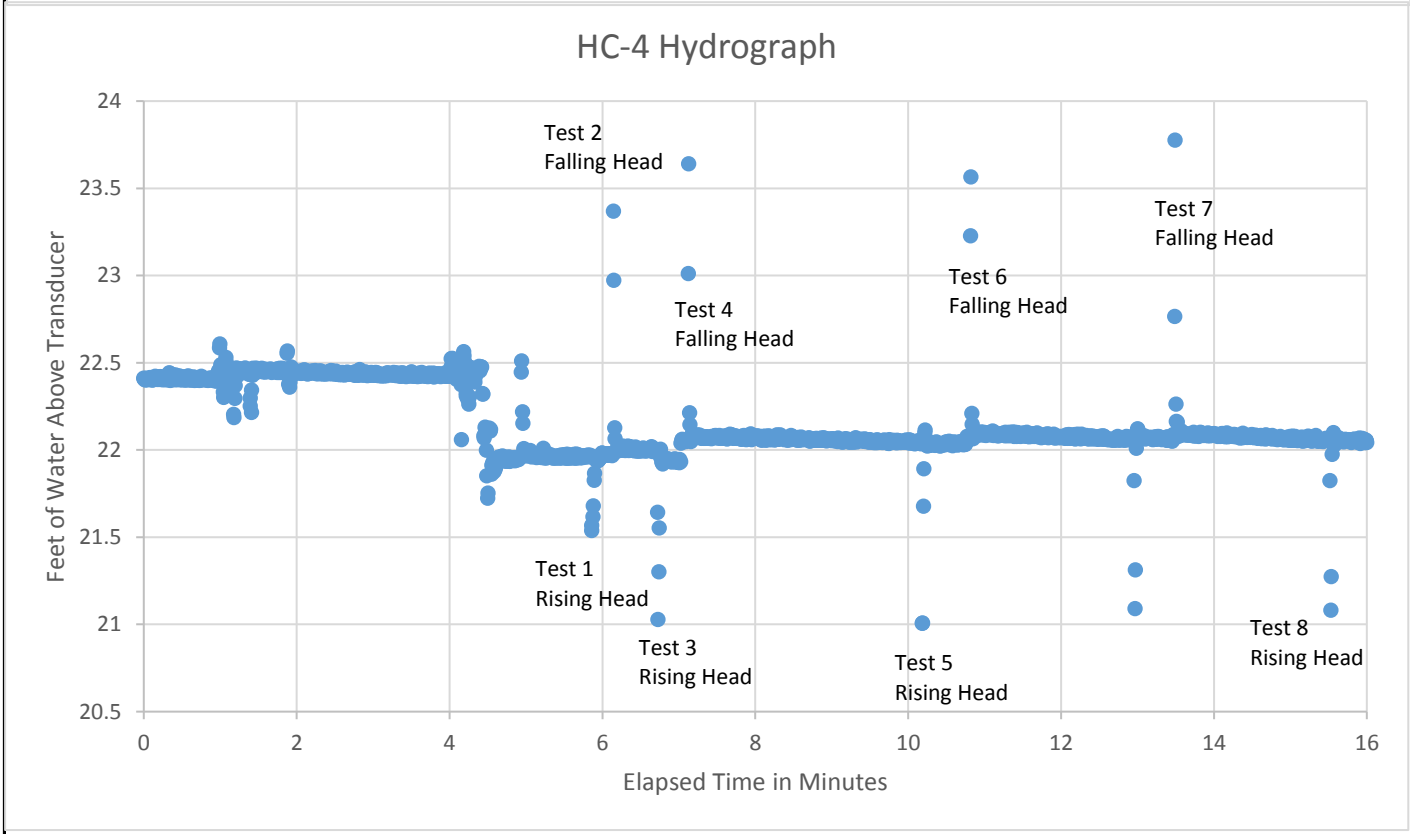
Well ID	HC-1	HC-4
Boring Depth in Feet	51.5	51.5
Well Depth in Feet	37	36.5
Screen Interval Depth in Feet	15 to 35	16.4 to 36.4
Depth to Sediment in Feet (1)	34.00	36.70
Depth to Water in Feet (1)	11.00	12.20
Saturated Thickness in Feet	23.0	24.5
Screened Interval Soil Description	GP	GP

Notes:
(1) Depth to sediment and depth to water was measured on January 8, 2016.
GP = Sandy GRAVEL
NA = Data not available.

Table 2 - Summary of Slug Test Results

Well ID	Test Type	Test Number	Bouwer and Rice		
				K in ft/day	K in cm/sec
HC-1	Falling Head	Test 1		70.9	2.5E-02
	Falling Head	Test 2		150.2	5.3E-02
	Rising Head	Test 3		144.6	5.1E-02
	Falling Head	Test 4		167.2	5.9E-02
	Rising Head	Test 5		87.9	3.1E-02
	Falling Head	Test 6		99.2	3.5E-02
	Rising Head	Test 7		119.1	4.2E-02
	Falling Head	Test 8		172.9	6.1E-02
	Rising Head	Test 9		144.6	5.1E-02
			Average	*128.5*	*4.5E-02*
HC-4	Rising Head	Test 1		99.2	3.5E-02
	Falling Head	Test 2		124.7	4.4E-02
	Rising Head	Test 3		150.2	5.3E-02
	Falling Head	Test 4		164.4	5.8E-02
	Rising Head	Test 5		119.1	4.2E-02
	Falling Head	Test 6		178.6	6.3E-02
	Falling Head	Test 7		164.4	5.8E-02
	Rising Head	Test 8		178.6	6.3E-02
			Average	*147.4*	*5.2E-02*





Proposed Parcel 5B Development
Redmond, Washington

HC-1 and HC-4 Hydrographs

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Figure
1

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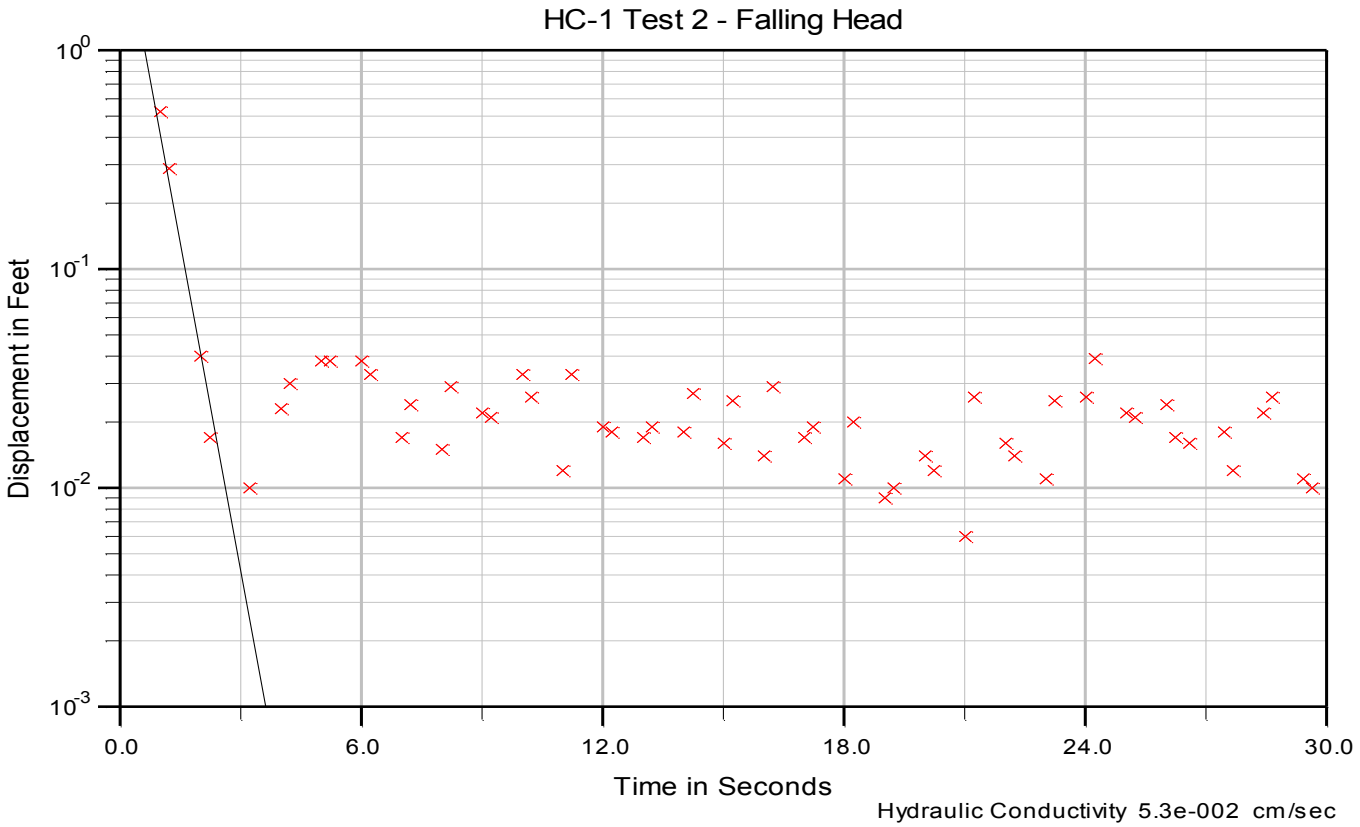

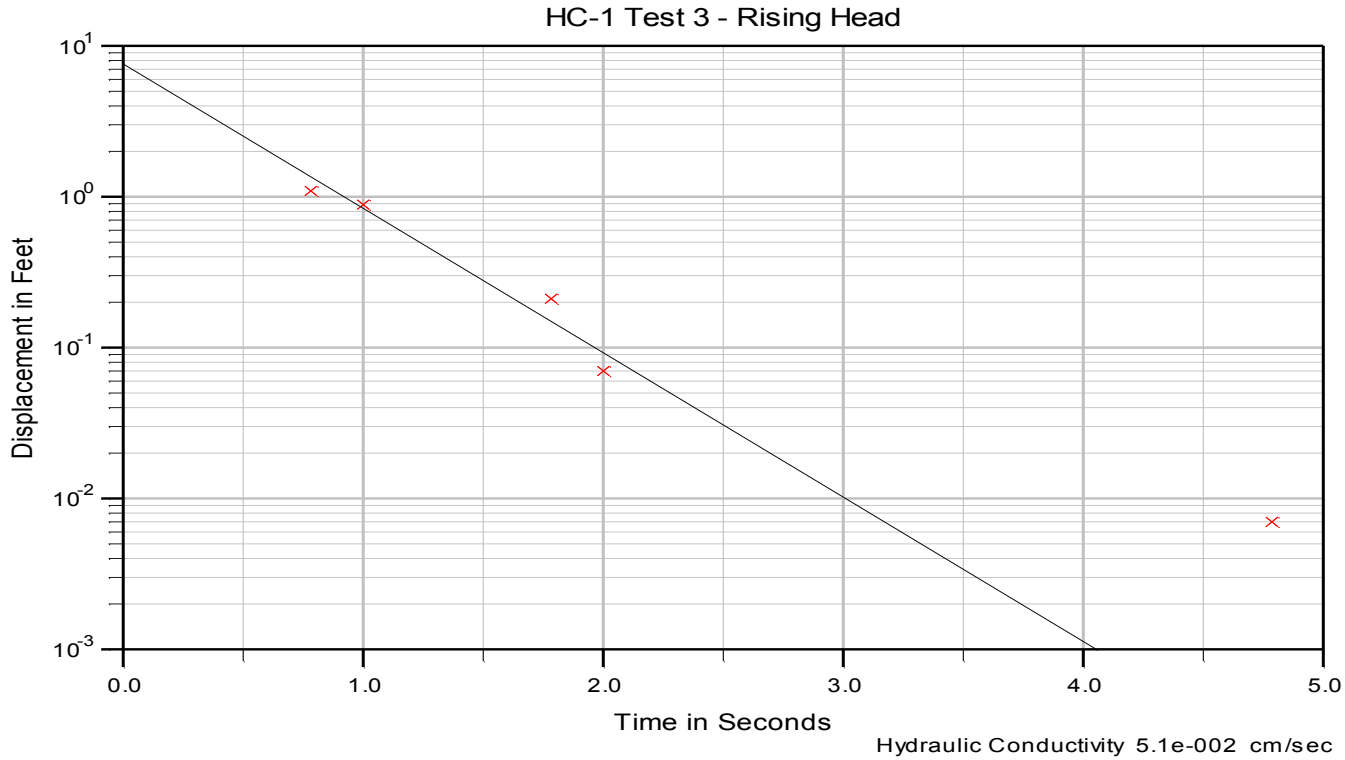

Proposed Parcel 5B Development
Redmond, Washington

HC-1 Representative Slug Tests Results

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Figure
2

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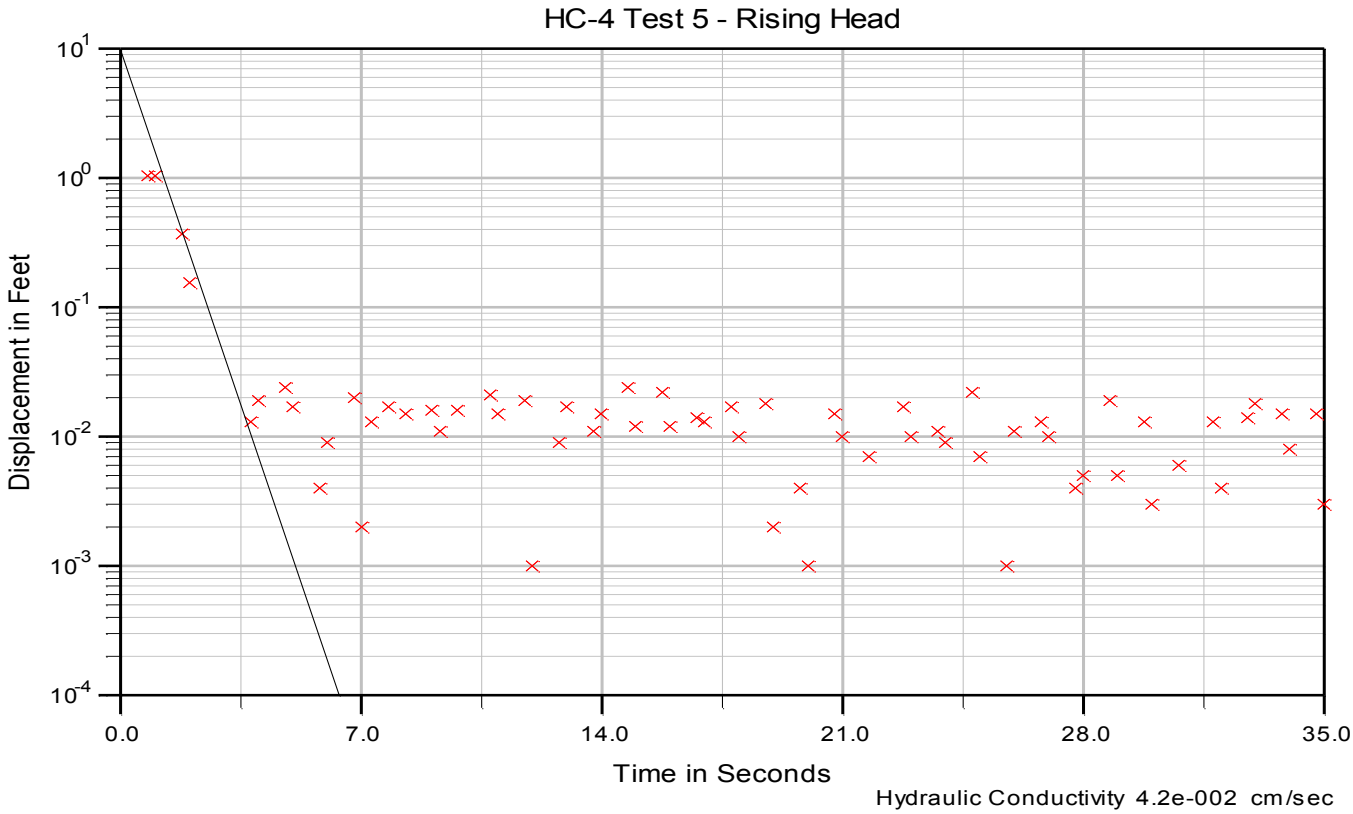

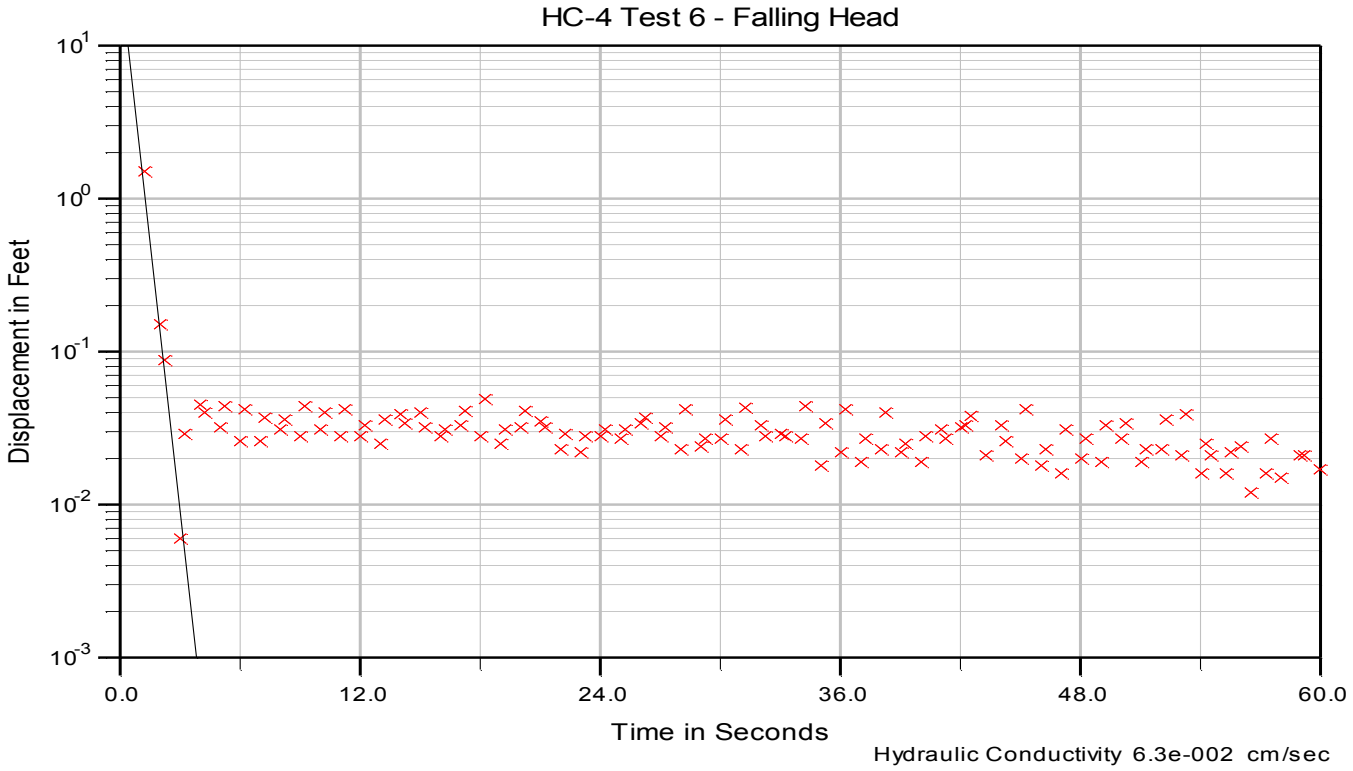

Note:
Bouwer and Rice method was used for the slug test analysis.

Proposed Parcel 5B Development
Redmond, Washington

HC-4 Representative Slug Tests Results

19162-01 1/16

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Figure
3

BLB 1/11/16 L:\Notebooks\1916201_Redmond 5B Geotech Feasibility Study\Field Data\Measurement Records\Slug Test\Slug Test Data.xls

ATTACHMENT 1
Tieback Anchor Testing Program

ATTACHMENT 1

Tieback Anchor Testing Program

Conduct the performance and proof tests as follows:

Performance Test

At least two performance tests should be completed before installation of production anchors. Each performance test should be conducted according to the following procedure:

1. The geotechnical engineer will select the testing locations with input from the shoring subcontractor.

2. The maximum stress in the prestressing steel should not exceed 80 percent of the ultimate tensile strength during performance testing [based on the Post Tensioning Institute manual]. The soldier pile and tieback may require extra reinforcement to permit stressing to 200 percent of design load as required for the performance test.

3. The performance test will measure anchor stress and displacement incrementally to values of unit skin friction equal to 200 percent of the design load. Load the anchor and measure deflections following the load sequence in Table 2-1.

Table 2-1 – Performance Test for Temporary Shoring

Load Level	Hold Time	Load Level	Hold Time
AL	Until Stable	1.75DL	Until Stable
0.25DL	10 min	1.50DL	Until Stable
0.50DL	10 min	1.25DL	Until Stable
0.75DL	10 min	1.00DL	Until Stable
1.00DL	10 min	0.75DL	Until Stable
1.25DL	10 min	0.50DL	Until Stable
1.50DL	60 min (Creep)	0.25DL	Until Stable
1.75DL	10 min	AL	Until Stable
2.00DL	10 min		

4. For 10-minute hold times, obtain and record deflection measurements during loading at intervals of 30 seconds, 1 minute, 2 minutes, 3 minutes, 5 minutes, 6 minutes, and 10 minutes. Measurements shall be made to an accuracy of 0.01 inch.

5. Perform a creep test at the 150 percent of design stress reading by holding the load constant to within 50 psi and recording readings at 30 seconds, 1 minute, 2 minutes, 3 minutes, 5 minutes, 6 minutes, and 10 minutes, 20 minutes, 30 minutes, 50 minutes, and 60 minutes.

6. A successful test does not experience pullout failure, holds the maximum test unit stress without considerable creep, and satisfies the apparent free length criteria.

- Pullout failure occurs when test measurements no longer exhibit a linear or near-linear relationship between unit stress and movement over the entire 200 percent stress range.

- Noticeable creep is defined as a rate of movement of not more than 0.04 inch between the 1- and 10-minute readings, or not more than 0.08 inch between the 6- and 60-minute readings. If the reading does not stabilize to 0.08 inch or less per log cycle of time, the test shall be considered to fail the creep criteria.

- Minimum apparent free length, based on the measured elastic and residual movement, should be greater than 80 percent of the designed free length plus the jack length.

7. Perform tests without backfill ahead of the anchor, if the hole will remain open, to avoid any contributory resistance by the backfill. If the hole will not remain open during testing, provide a bond breaker on the tie rods and backfill the no load zone with a non-cohesive non-structural mixture.

Proof Test

For each production tieback anchor, follow the proof testing procedures outlined below:

1. Load each anchor to 133 percent of the design load in increments of approximately 25 percent of the design load (i.e., 0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL, 1.25 DL, and 1.33 DL). The maximum stress in the prestressing steel should not exceed 80 percent of the ultimate tensile strength during proof testing.

2. Hold each incremental load for a period long enough to obtain a stable deflection measurement while recording deflections at each load increment. Hold the 133 percent load for a minimum of 10 minutes, recording the movement at times of 30 seconds, 1 minute, 2 minutes, and 5 minutes, 6 minutes, and 10 minutes.

3. A successful test is one that meets the same acceptance criteria as performance anchors, except that the creep portion of the test need not exceed 10 minutes if the 10-minute creep criteria is meet.

4. Following proof loading, lock off each tieback anchor to 80 to 100 percent of the design load, except as specified.

ATTACHMENT 2
Shoring Monitoring Program

ATTACHMENT 2

Shoring Monitoring Program

A shoring monitoring program provides early warning if the shoring does not perform as anticipated. We recommend that the following components be included in the shoring monitoring program during construction:

- Adjacent building surveys;
- Optical surveying; and
- Geotechnical instrumentation (Inclinometer).

All monitoring data should be submitted to Hart Crowser for weekly review. The data will be included in our field transmittals to the project team and DPD during construction. Details of our expectations for shoring monitoring are included below.

Adjacent Building Surveys

We recommend that adjacent buildings be surveyed before, during, and after construction. The pre-construction survey will establish the baseline of existing conditions (e.g., identifying the size and locations of any cracks). The surveys should consist of a videotape and/or photographs of the interior and exterior of adjacent buildings and detailed mapping of all cracks. Any existing cracks could be monitored with a crack gauge.

Optical Surveying

We recommend optical surveys of horizontal and vertical movements of: (1) the surface of the adjacent streets, (2) buildings on and adjacent to the site, and (3) the shoring system itself. The contractor, in coordination with the geotechnical engineer, should establish two reference lines adjacent to the excavation at horizontal distances back from the excavation face of about 1/3 H and H, where H is the final excavation height. Typically, these lines will be established near the curb line and across the street from the excavation face. The points on the adjacent buildings can be set either at the base or on the roof of the buildings.

Shoring system monitoring should include measuring vertical and horizontal movement at the top of every other soldier pile, and any geotechnical instrumentation (e.g., inclinometers) used for the project.

The measuring system for the shoring monitoring should have an accuracy of at least 0.005 foot. All reference points on the ground surface should be installed and read before excavation begins. The frequency of readings will depend on the results of previous readings and the rate of construction. At a minimum, readings on the external points should be taken once a week through construction until below-grade structural elements (floors, decks, columns, etc.) are completed, or as specified by the structural and geotechnical engineers. Readings on the top of soldier piles and the face of existing buildings on or adjacent to the property should be taken at least twice a week during this time. We

recommend that an independent surveyor hired by the owner to record the data at least once per week with the other reading taken by the surveyor or contractor