Geotechnical Engineering Report
Juanita Beach Park Bathhouse
Kirkland, Washington

April 19, 2017

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GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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KIRKLAND, WASHINGTON

1.0 INTRODUCTION

Juanita Beach Park is located on Juanita Bay on the northeast side of Lake Washington in Kirkland, Washington, as illustrated on the Vicinity Map (Figure 1). The proposed facility improvements include a bathhouse structure, a sewer connection to an existing manhole, a pavilion, two play areas, and several pathways. The purpose of this study was to evaluate subsurface soil and groundwater conditions to aid in design and planning for proposed facilities improvements. Our geotechnical scope of services included drilling three soil borings, performing hydrogeologic testing, performing engineering analyses, and preparing this report. We researched available geotechnical engineering reports and geologic maps of the area. We reviewed the boring logs from the Juanita Bay Pumping Station project, located about 400 feet northwest of the proposed Bathhouse (Metropolitan Engineers, 1966).

2.0 SITE AND PROJECT DESCRIPTION

Juanita Beach Park slopes gently towards the south from about elevation 30 feet on the north side to elevation 18 feet on the south side at Lake Washington. The site includes grassy lawn areas, sidewalks, a beach on the south side, a parking lot on the north side, and an existing bathhouse and playground. A creek flows along the west side of the site into Lake Washington.

The proposed bathhouse will be approximately 2,000 to 3,000 square feet and will partially occupy the footprint of the existing playground. The bathhouse will connect a sewer line to an existing King County Metro manhole approximately 100 feet southeast of the bathhouse. We understand the existing bathhouse will be demolished. The proposed pavilion will cover approximately 1,000 square feet and will partially occupy the footprint of the existing bathhouse.

3.0 SITE GEOLOGY

3.1 Regional Geology

Kirkland is located in the central portion of the Puget Lowland, an elongated topographic and structural depression bordered by the Cascade Mountains on the east and the Olympic Mountains on the west. This lowland is characterized by low, rolling relief with some deeply cut ravines and broad valleys. In general, the ground surface elevation is within 500 feet of sea level.

The Puget Sound area underwent six or more major glaciations during the Pleistocene Epoch (2 million years ago to about 10,000 years ago), which filled the Puget Lowland to significant depths with a complex sequence of glacial and nonglacial (deposited during interglacial times) sediments. These glaciers originated in the coastal mountains of British Columbia. The maximum southward advance of the ice was about halfway between Olympia and Centralia (about 50 miles south of Seattle). During the most recent glaciation of the central Puget Lowland (Vashon Stade of Fraser Glaciation), the thickness of ice was about 3,000 feet in the project area, resulting in overconsolidation of the underlying soils. Since the last glaciation, complete or partial erosion of some deposits, as well as local deposition of alluvial deposits, further complicates the geology of the region.

3.2 Regional Tectonics and Seismicity

Tectonically, the Puget Lowland is located in the fore arc of the Cascadia Subduction Zone. The tectonics and seismicity of the region are the result of the relative northeastward subduction of the Juan de Fuca Plate beneath the North American Plate. The convergence of these two plates results not only in the east-west compressive strain, but also in dextral shear, clockwise rotation and north-south compression of the crustal blocks that form the leading edge of the North American Plate. It is estimated that the compression rate for these blocks is about 0.03 to 0.04 inch per year, and much of the compression may be occurring within the more fractured, northern Washington block that underlies the Puget Lowland.

While the bedrock and structure of the portion of the northern block that underlies the Puget Lowland is largely concealed by thick Quaternary deposits, it has been the subject of recent and ongoing research (Yount and Gower, 1991; Yount and others, 1985). This research suggests that the north-south compression of the block is being accommodated primarily beneath the Lowland by a series of west and northwest-trending thrust faults that extend to depths of about 12 miles. The thrust faults are presumably bounded by strike slip or shear zones on the east at the Cascade Mountains, and on the west along Hood Canal at the base of the Olympic Mountains.

The nearest potentially active fault to the project is the Seattle Fault, a collective term for a series of four or more east-west-trending, south-dipping fault splays. The mapped location of the fault is about 8 miles south of Juanita Beach Park (Booth and Minard, 1992). This thrust fault zone is approximately 2.5 to 4 miles wide (north-south) and extends from the west end of the Kitsap Peninsula near Hood Canal, eastward to the Sammamish Plateau east of Lake Sammamish. The locations of the fault splays are largely determined from overwater seismic reflection profiles with some recent fault trenching studies by the U.S. Geological Survey (USGS) on the west side of Puget Sound on Bainbridge Island and the Kitsap Peninsula. East of Puget Sound, the fault splay locations have been extrapolated and are not precisely known. Recent geologic evidence indicates that ground surface rupture from movement on this fault zone occurred as recently as 1,100 years before present.

4.0 SUBSURFACE EXPLORATIONS

Holocene Drilling, Inc. (Holocene) drilled three soil borings, designated B-1 to B-3. Boring B-1 was drilled on October 27, 2015, and borings B-2 and B-3 were drilled on March 23, 2017. Holocene installed a well in boring B-2. The boring locations are shown in Figure 2. Logs of the borings and description of drilling methods are presented in Appendix A. We performed geotechnical laboratory testing on select samples from the borings. Appendix B presents laboratory test results and procedures.

5.0 SUBSURFACE CONDITIONS

The subsurface conditions of the site have been summarized based on the soil and groundwater conditions observed in the boring, and review of previous geotechnical reports and boring logs.

5.1 Soil

The soil at the project site consists of:

- Alluvial sand with silt and recessional outwash was encountered below a thin layer of topsoil. The alluvium and outwash is generally very loose to medium dense and extends 12 to 15 feet below ground surface (bgs).
- Lacustrine deposits, consisting of silt, were encountered below the alluvium and extended to 17 feet bgs. The lacustrine deposit is generally medium dense or stiff and contains variable amounts of sand.
- Recessional outwash was encountered below the lacustrine silt and extended to the bottom of the borings at 31.5 feet bgs. The outwash generally consists of medium dense to dense, fine to medium sand. The low blow counts encountered in boring B-2 are likely influenced by heaving sands and are not representative of soil density.

More detailed information is presented on the boring logs in Appendix A.

5.2 Groundwater

Groundwater was encountered at about elevation 18 feet, or about 3 to 5 feet bgs. The elevation of Lake Washington is at about elevation 18 feet. Therefore, we anticipate the groundwater level is closely tied to the elevation of Lake Washington and probably varies seasonally with Lake Washington.

Our groundwater measurements from the monitoring well installed in boring B-2 indicate that a confined aquifer is present below the lacustrine layer. The confined aquifer extends from approximately 17 feet bgs to at least 31.5 feet bgs (where boring B-2 was terminated). We observed artesian groundwater pressures in the confined aquifer with pressures corresponding to about elevation 22 feet. Review of boring logs from the nearby Juanita Bay Pump Station Replacement Project provide a similar hydrogeologic profile to boring B-2, with alluvial sand unconfined aquifer above a silt/clay confining unit, in turn underlain by a confined aquifer consisting of alluvial and recessional outwash sand. The alluvial and recessional outwash sand at the Juanita Bay Pump Station Replacement Project was encountered to depths of 45 feet bgs.

6.0 ENGINEERING RECOMMENDATIONS AND CONCLUSIONS

6.1 Earthquake-induced Geologic Hazards

Earthquake-induced geologic hazards that may affect a given site include landsliding, fault rupture, settlement, and liquefaction, and associated effects (loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, lateral spreading, etc.). Because of the relatively flat topography at the site, the risk of landsliding is considered low.

The project site is about 8 miles from the potentially active Seattle Fault zone. Therefore, the risk of fault rupture is considered negligible. The hazards associated with liquefaction are discussed below.

6.2 Liquefaction Potential

Soil liquefaction is a phenomenon that occurs during seismic shaking in loose, saturated, cohesionless soils. During liquefaction, the pore pressure of the water in the soil increases while the effective stress between soil grains decreases. When the two approach equal states, the result is a reduction in shear strength of the soil. This reduction in strength can cause ground settlement and lateral spreading.

We have evaluated the liquefaction potential of the site soils using the data from borings B-1 to B-3. We used the procedure by Youd and others (2001), and updated by Idriss and Boulanger (2004) to calculate factors of safety (FSs). This method involves comparing the liquefaction resistance of the soil (expressed as cyclic resistance ratio) to the earthquake-induced loading (expressed as cyclic stress ratio). Our liquefaction analyses indicate that the soil in the upper 15 feet is susceptible to liquefaction during the 2,500-year earthquake. This could induce settlement of the project site, as described below in Section 6.5.

6.3 Seismic Design

We understand that the bathhouse project will be designed in accordance with the International Code Council's 2014 International Building Code (IBC) (International Code Council, 2015). The IBC requires that the seismicity of the region be considered in building design by requiring that structures be designed for earthquake ground motions with a 2 percent chance of being exceeded in 50 years (2,500-year recurrence).

The subsurface conditions at the site correspond to IBC 2014 Site Class F because of the presence of potentially liquefiable soil. If liquefaction were not considered, the site would correspond to IBC 2014 Site Class D, based on standard penetration resistance values in the boring. The IBC 2014 requires a site-specific ground response evaluation for Site Class F sites, with the exception of structures with periods of less than 0.5 second, which we assume is the case for the proposed bathhouse. Therefore, we recommend that the site be classified as Site Class D for purposes of structural design.

Table 1 summarizes the mean earthquake magnitude value from the USGS probabilistic seismic hazard analysis, M_W, and a ground motion that corresponds to Site Class D for the 2,500-year seismic event.

TABLE 1
EARTHQUAKE MAGNITUDE AND SITE CLASS D
PEAK GROUND ACCELERATIONS

2,500-year Earthquake	Design Value
Magnitude	7.0
S_1 g (1 sec)	0.48
Ss g (0.2 sec)	1.25
PGA (ground motion*)	0.67
Design PGA **	0.45

Notes:
* Peak ground accelerations based upon the maximum considered earthquake spectral response acceleration.
** Two-thirds of peak ground accelerations.
PGA = ground motion
sec = second

Because of the potential for ground settlements and lateral movements during a design-level earthquake, we recommend the foundations for the bathhouse be structurally tied together to resist differential settlements. Some structural damage due to settlement could be expected in a design-level earthquake, however, we do not expect it would result in a life-safety hazard.

6.4 Lateral Spreading

Liquefaction of soils at the site may result in permanent lateral displacement, or lateral spreading, toward Lake Washington. Lateral spreading occurs when the ground surface displaces towards a nearby sloping ground surface at a lower elevation than the site during liquefaction. We estimate that there is moderate risk of liquefaction-induced lateral spreading using the results of our liquefaction analyses and the empirical procedure by Youd and others (2002). As such the shoreline may experience minor lateral displacement during an earthquake event.

6.5 Seismically Induced Settlement

Loose, cohesionless soils that are susceptible to liquefaction are also susceptible to earthquake-induced settlement. The resulting ground surface settlements are not likely to occur uniformly over an area. Differential settlement can be damaging to structures founded on loose soils.

We estimated seismically-induced settlements for the subsurface conditions encountered in boring B-1 using the empirical correlations for volumetric strain by Tokimatsu and Seed (1987). The Tokimatsu and Seed procedure for estimating seismically-induced settlements is an approximate method; however, this method is the current state-of-practice. We estimate seismic-induced settlements would be about 3 to 8 inches over the width of the building. It is common to assume that differential settlement may be a large percentage of or equal to the total settlement because of potential variations in subsurface conditions across a given site. If this potential differential settlement is unacceptable for spread footings, we recommend using a mat foundation to reduce the potential for differential settlement.

6.6 Foundations

The proposed bathhouse may be supported on spread footings with a slab-on-grade floor slab or on a mat foundation. We recommend that an allowable bearing pressure of 2,000 pounds per square foot be used in the design of the spread footings and mat foundations. The native subgrade should be compacted to a dense and unyielding condition prior to constructing foundations. This pressure could be increased by up to one-third for seismic and wind loads.

The base of all foundations should be located at least 18 inches below the adjacent grade. We recommend that a representative from our firm be retained to evaluate foundation excavations during construction and to verify the presence of competent bearing soil or compacted structural fill. This should be done immediately prior to placement of reinforcing steel and concrete forms.

6.6.1 Spread Footings

Continuous footings should have a minimum width of 18 inches, and column footings should have a minimum width of 24 inches. Spread footing foundations designed and constructed as recommended in this report are estimated to undergo total settlement on the order of 1 inch under static loading conditions. We estimate differential settlement would be on the order of ½ inch between adjacent footings. Due to the granular nature of the foundation soils, we estimate that the majority of this settlement would occur during construction as the load is applied.

6.6.2 Mat Foundations

Mat foundations designed and constructed as recommended in this report are estimated to undergo total settlement on the order of 1 inch under static loading conditions. We estimate differential settlement would be on the order of ½ inch across the width of the foundation. Due to the granular nature of the foundation soils, we estimate that the majority of this settlement would occur during construction as the load is applied.

We recommend designing the mat foundation using a modulus of vertical subgrade reaction of 14 pounds per cubic inch (pci). This value was calculated based on the allowable bearing pressure and estimated static settlement.

6.7 Floor Slabs

We recommend that floor slabs be supported by densely compacted native soil, or compacted structural fill placed directly onto compacted native soil. If unanticipated loose, soft, or unsuitable soil is encountered, it should be removed and replaced with compacted structural fill. Structural fill should be compacted to a dense, unyielding condition, according to our recommendations presented in the construction considerations section of this report below. A modulus of subgrade reaction of 250 pci may be used to design the slab, assuming that densely compacted structural fill will be present.

We recommend placing a capillary break consisting of a minimum 4-inch layer of washed pea gravel (⅜ inch to No. 8 sieve size) and a vapor barrier consisting of plastic sheeting, as shown in Figure 3.

6.8 Lateral Resistance

Lateral forces would be resisted by passive earth pressure against the buried portions of the structure and friction against the bottom. In our opinion, passive earth pressures developed from compacted granular fill could be estimated using an equivalent fluid unit weight of 300 pounds per cubic foot (pcf). This value is based on the assumption that the structure extends at least 18 inches below the lowest adjacent exterior grade, is properly drained, and the backfill around the structure is compacted in accordance with the recommendations for structural fill outlined herein. The above equivalent fluid unit weight includes a FS of 1.5 to limit lateral deflection.

6.9 Base Footing Friction

We recommend that a coefficient of friction of 0.35 be used between cast-in-place concrete and native granular soil. This value includes an FS of 1.5.

6.10 Sewer Line Excavation

We understand an approximately 16-foot-deep excavation is proposed to connect the bathhouse sewer line to an existing King County Metro manhole. The following sections present our recommendations for temporary slopes, temporary shoring, and dewatering related to the proposed excavation.

6.10.1 Temporary Excavation Slopes

Temporary excavation slopes should be made the responsibility of the Contractor who is continually at the site, is able to observe the nature and conditions of the subsurface materials encountered, including groundwater, and has responsibility for the methods, sequence, and schedule of construction.

For planning purposes, we recommend that temporary, unsupported, open-cut slopes be no steeper than 1.5 Horizontal to 1 Vertical. This recommendation is applicable if groundwater seepage is not present. Flatter slopes may be required based on the actual conditions encountered, particularly where groundwater seepage is encountered. We recommend that all exposed slopes be protected with waterproof covering during periods of wet weather to reduce sloughing and erosion.

All traffic and/or construction equipment loads should be set back from the edge of the temporary cut slopes a minimum of 5 feet. Excavated material, stockpiles of construction materials, and equipment should not be placed closer to the edge of any excavation than the depth of the excavation, unless the excavation is shored and such materials are accounted for as a surcharge load.

6.10.2 Temporary Shoring

Temporary shoring will be required for the proposed excavation. We anticipate temporary shoring would consist of temporary sheet piles and/or trench boxes. We recommend designing temporary shoring for an equivalent fluid weight of 40 pcf above the water table or 85 pcf below the water table. Surcharge loads such as traffic and construction equipment will also induce lateral loads on retaining walls and buried structures. Figure 4 presents recommendations for lateral pressure due to surcharge loads that could be applied to walls. We recommend using a K_a value of 0.33.

6.10.3 Dewatering

The hydrogeologic conditions impacting construction dewatering include an unconfined sand aquifer overlying a silt/clay confining unit, which is underlain by a confined sand aquifer.

We recommend that the dewatering system design be made the Contractor's responsibility as part of the project plans and specifications. The design should be provided by a Washington State-Licensed Hydrogeologist experienced in the design and construction of dewatering systems.

This section provides groundwater parameters that can be used for preliminary dewatering design, conceptual dewatering recommendations, and dewatering considerations.

6.10.3.1 Hydraulic Conductivity

Hydraulic conductivity estimates for the unconfined aquifer are based on visual comparison of boring B-2 samples within the unconfined aquifer with boring B-2, sample S-9 from the confined aquifer. The samples within the unconfined aquifer have a visually similar grain size distribution to sample S-9 and are expected to have a similar hydraulic conductivity. Hydraulic conductivity estimates for the confined aquifer are based on the results of slug testing (single-well field hydraulic conductivity testing, described in Appendix C) performed in observation well B-2 and grain size analysis values.

Estimated hydraulic conductivities are as follows:

- Unconfined aquifer: 70 feet per day (ft/day) to 250 ft/day
- Confined Aquifer: 70 ft/day to 250 ft/day

These hydraulic conductivity ranges are generally consistent with the fine to medium sand encountered in the observation well screen interval of this exploration.

Groundwater depth measured in observation well B-2 in April 2017 varied from 0.56 to 0.8 foot above ground surface (approximate elevation 21.6 to 21.8 feet).

These groundwater depths/elevations are expected to be at or near the annual high which typically occurs in late winter or spring.

6.10.3.2 Dewatering Analysis

Installation of the proposed below-ground sewer connection will involve excavating below groundwater, with anticipated excavation depths up to 16 feet. This excavation will require construction dewatering and depressurization to control groundwater inflow, reduce instability and erosion of the side slopes, and reduce hydrostatic pressures and subgrade instability at the base of the excavation.

The maximum required groundwater drawdown was assumed to be 16 feet below existing ground surface (elevation 5 feet). Additionally, we assume the excavation will be supported using sheet pile shoring and/or trench boxes and will require dewatering of the unconfined aquifer and depressurization of the confined aquifer. Dewatering of the unconfined aquifer is accomplished by physically draining groundwater from the pore space within the sediment. This process requires lowering of the water table, by pumping, which induces groundwater flow by gravity towards the area of lowered water table. Depressurization of the confined aquifer is accomplished by lowering the piezometric surface that extends above the top of the aquifer while the pore space within the aquifer remains saturated.

6.10.3.3 Dewatering-induced Settlement

Dewatering of the unconfined aquifer and depressurizing of the confined aquifer will result in settlement due to the decrease in water pressure and subsequent increase in effective stress. We anticipate settlement due to dewatering could be on the order of ½ to 1 inch. Settlement would be greatest near the excavation, but could potentially impact areas several hundred feet away. The dewatering designer should evaluate potential settlement impacts prior to construction.

We recommend completing the excavation and associated dewatering prior to construction of the bathhouse and pavilion to avoid causing settlement of the new structures.

6.10.3.4 Construction Dewatering Approach and Available Technologies

Numerous factors influence the type of dewatering approach employed by the Contractor, including soils, aquifer thickness, the relationship of the excavation base to the base of the aquifer, drawdown requirements, shoring and excavation approaches, the amount of dewatering flow anticipated, and the experience of the Contractor working in dewatered and wet soils.

Available dewatering technologies include:

- **Sumps and/or Trenches** generally provide the least costly method and are the most common dewatering methods. Sumps consist of excavations immediately adjacent to or in an excavation. Sump pumping should be limited to areas where no more than 2 or 3 feet of drawdown is required. Sumps work well in either fine- or coarse-grained soils, which typically provide low or high dewatering flow rates, respectively. Sumps and trenches generally pump finer-formation material which can undermine excavations. Sump pumping usually requires considerable treatment such as settlement of fines in the dewatering discharge prior to disposal.
- **Pumped Wells (Dewatering Wells)** typically consist of large-diameter holes (24 to 36 inches) and large-diameter casings/screens (i.e., 8- to 16-inch-diameter). Pumped wells, often called deep wells, are relatively deep compared to sumps and vacuum wellpoints. Pumped wells include individual pumps which typically discharge to a common manifold. Pumped wells work best (most efficiently) in relatively coarse-grained (high permeability) formations (silty sand, sand and gravel) that allow wide spacing of wells (typically 25 to 250 feet) due to a large radius of influence.
- **Vacuum Wellpoints** connect to a common vacuum header and typically operate using a single pump for the whole system, and are suitable for both fine- and coarse-grained soils. They are generally 15 to 25 feet deep and constrained by the limits of the vacuum to pull water out of the ground (typically 15 to 20 feet at sea level). The wellpoints typically have a 3-foot length of slotted well screen at the bottom and are spaced 2 to 10 feet apart with the closer spacing for finer-grained soils (i.e., silt, clay, and/or peat). For coarser soils and wider spacing, pumped wells typically prove more efficient and less costly than vacuum wellpoints.
- **Eductors/Ejectors** typically are closely spaced, and are rarely used except in fine-grained soils due to their higher cost. However, because eductors require little maintenance, they are particularly suited for excavations in both coarse- and fine-grained soils needing large drawdown over a long period of time (months or years). Because eductors employ pressurized flow and are not limited by vacuum constraints, they can achieve greater drawdowns than vacuum wellpoints.

6.10.3.5 Construction Dewatering Recommendations

Based on our understanding of the hydrogeologic conditions at the project site and their relation to the proposed structures and sewer line construction, we recommend assuming that a Contractor would propose to use large-diameter pumped wells lower groundwater levels in the unconfined aquifer and depressurize the confined aquifer during construction. As an alternative, the Contractor may select a vacuum well point system to dewater the unconfined aquifer, and would still use pumped wells to depressurize the confined aquifer.

Additionally, the use of localized sump pumping within the trench excavation should be anticipated to capture perched or pocketed groundwater not captured by the wells and or vacuum well points within the unconfined aquifer. Sumps should be designed to produce discharge that is free of sediment or high levels of turbidity. Using a “trash pump” directly in the excavation (open sumping) to remove groundwater typically mobilizes sediment, produces very turbid discharge, and should be prohibited.

We also note that the existing sewer line which connects to the manhole may have relatively high-permeability bedding material, which could contribute significant volumes of water to the excavation. Water flow from the pipe bedding may also mobilize soil, which could result in soil loss and associated ground settlement. The Contractor should anticipate this likelihood, and should be required to submit to the owner their plan for capturing or controlling water flow from the existing pipe bedding, and preventing soil loss and related impacts.

We recommend that construction dewatering, and the design of dewatering systems, be the responsibility of the Contractor. The Contractor should be required to use the services of a Washington State-Licensed Hydrogeologist experienced in the design and construction of dewatering systems.

Discharge from the temporary dewatering systems should be collected and disposed of in accordance with discharge permit requirements.

6.11 Subdrainage and Surface Water Drainage Control

We recommend installing a footing subdrain system along the outside of the perimeter footings to prevent the buildup of hydrostatic pressures. The subdrain system should consist of a perforated or slotted, 4-inch (minimum)-diameter plastic pipe bedded in ⅜ inch to No. 8 size washed pea gravel or crushed gravel. Please refer to Figure 3 for subdrainage recommendations.

To promote surface water drainage, provisions should be made to direct water away from structures and prevent water from seeping into the ground adjacent to the structures. The ground surface should be sloped away, and surface and downspout water should not be introduced into backfill. Surface water should be collected in catch basins and, along with downspout water, should be conveyed in a non-perforated pipe (tightline) into an approved discharge point.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Foundations

The recommended allowable bearing capacities presented previously in this report are contingent upon the following construction considerations:

- Foundation subgrade excavations should be cleaned of all fill, debris, and loose, soft, wet, or disturbed soil prior to placing the reinforced concrete.
- All excavations for spread footing foundations should be observed by a geotechnical engineer to evaluate the adequacy of the bearing stratum and to confirm that subsurface conditions at and below the bearing elevation are suitable for the design bearing values provided.

7.2 Fill Material, Placement, Compaction, and Use of On-site Soils

Care should be taken to select the granular soil suitable for use as structural fill. All fill material placed beneath structures, pavements, or other areas where settlements are to be reduced and where backfill will provide passive resistance, should be structural fill. Onsite native soils are suitable for reuse, but may be difficult to compact during wet weather conditions because it contains significant quantities of silts and clays. Structural fill should consist of reasonably well-graded sand and gravel, free of organics and debris, and with a maximum particle size of 3 inches for wall and footing backfills.

Structural fill should be placed in uniform lifts and compacted to a dense and unyielding condition, to at least 95 percent of the Modified Proctor maximum dry density (ASTM Designation: D1557-70, Method C or D). The thickness of soil layers before compaction should not exceed 12 inches for heavy equipment compactors or 6 inches for hand-operated mechanical compactors.

7.3 Wet Weather Earthwork

In the Puget Sound region, wet weather generally begins about mid-October and continues through about May, although rainy periods may occur at any time of year. Therefore, it would be advisable to schedule earthwork during the normally dry weather months of June through September. Earthwork conducted during wet weather generally is more costly and time-consuming than work conducted in dry weather.

The following recommendations are applicable if earthwork construction takes place during wet weather or in wet conditions:

- The ground surface in and surrounding the construction area should be sloped and sealed with a smooth-drum roller to promote runoff of precipitation, to prevent surface water from flowing into excavations, and to prevent ponding of water.
- Work areas and soil stockpiles should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work. Bales of straw and/or geotextile silt fences should be suitably located to control soil movement and erosion.
- Earthwork should be accomplished in small sections to reduce exposure to wet weather. If there is to be traffic over the exposed subgrade, the subgrade should be protected with a compacted layer of clean sand and gravel or crushed rock. The size of construction equipment may have to be limited to prevent soil disturbance.
- Fill material should consist of clean, granular soil, of which not more than 5 percent by weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve. The fines should be nonplastic. Such soils may need to be imported to the site.
- No fill should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should be used to seal the ground surface. Soil that becomes too wet for compaction should be removed and replaced with clean granular soil.
- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer or the engineer's representative experienced in wet weather earthwork to determine that all unsuitable aggregates are removed and suitable compaction and site drainage is achieved.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

We suggest that these recommendations for wet weather earthwork be included in the contract specifications.

8.0 ADDITIONAL SERVICES

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

9.0 CLOSURE

This report was prepared for the exclusive use of Patano Studio Architecture for design and construction of the proposed development at Juanita Beach Park in Kirkland, Washington. The report should be provided to the design team and prospective subcontractors for information of factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

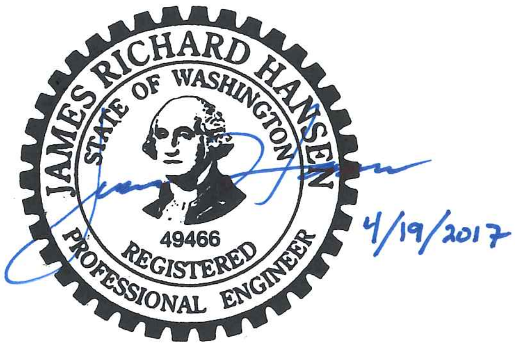
The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the exploratory boring made for this project is representative of the subsurface conditions throughout the site; i.e., the subsurface conditions everywhere are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

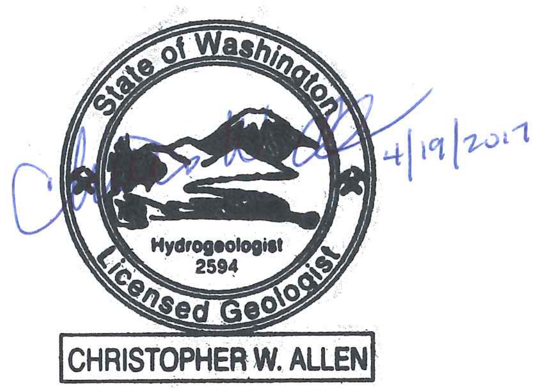
Within the limitations of the scope, schedule and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering and hydrogeologic principles and practice in this area at the time this report was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

The scope of our services did not include any environmental assessment or evaluation of hazardous or toxic materials in the soil, surface water, groundwater, or air at the subject site. Shannon & Wilson has qualified personnel to assist you with these services should they be necessary.

Shannon & Wilson has prepared Appendix D, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

SHANNON & WILSON, INC.





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Geotechnical engineering recommendations were prepared by or prepared under the direct supervision of James R. Hansen, PE.

Hydrogeologic recommendations were prepared by or prepared under the direct supervision of Chris W. Allen, LG, LHG.

10.0 REFERENCES

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N

0 1,000 2,000

Approximate Scale in Feet

NOTE

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

Juanita Beach Park Bathhouse
Kirkland, Washington

VICINITY MAP

April 2017 21-1-22161-008

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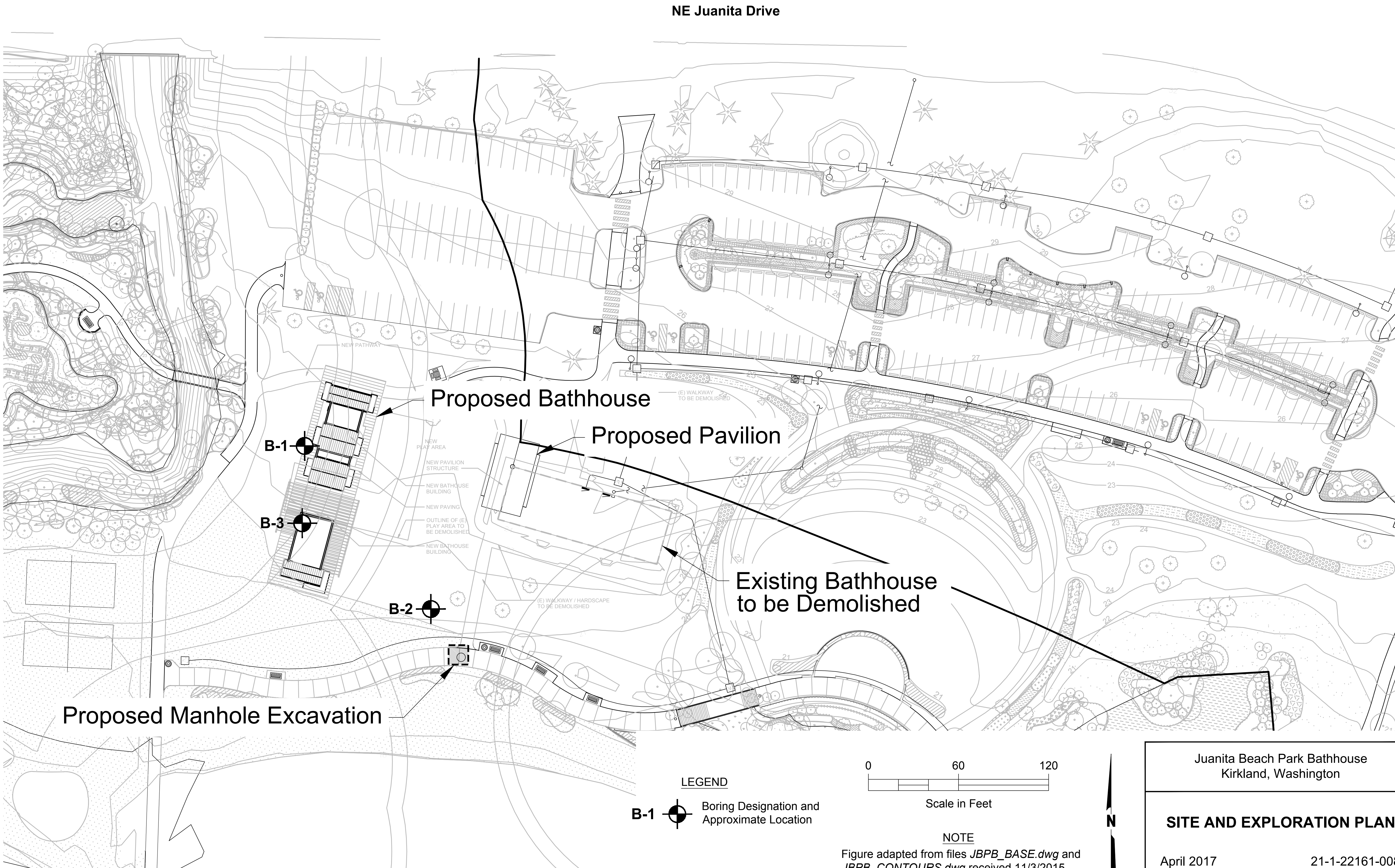
FIG. 1

Filename: J:\211\22161-008\21-1-22161-008 Vicmap.dwg Date: 04-06-2017 Login: jrs



NOTE
Figure adapted from files *JBPB_BASE.dwg* and *JBPB_CONTOURS.dwg* received 11/3/2015.

Juanita Beach Park Bathhouse
Kirkland, Washington

SITE AND EXPLORATION PLAN

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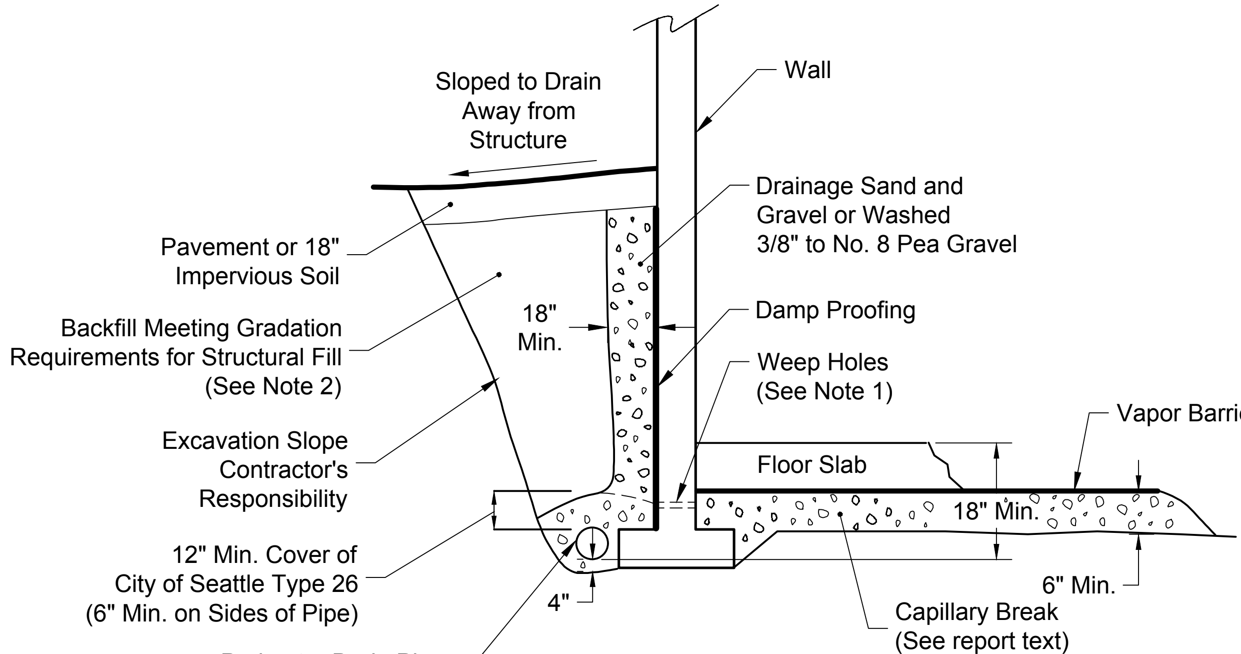
FIG. 1

Filename: J:\211\22161-008\21-1-22161-008 Fig 2 - Site Plan.dwg Layout: Layout1 Date: 04-17-2017 Login: jrs



Not to Scale

MATERIALS

Drainage Sand & Gravel with
the Following Specifications:

Sieve Size	% Passing by Weight
1-1/2"	100
3/4"	90 to 100
1/4"	75 to 100
No. 8	65 to 92
No. 30	20 to 65
No. 50	5 to 20
No. 100	0 to 2
(by wet sieving)	(non-plastic)

PERIMETER DRAIN PIPE

4" minimum diameter perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs.

Perforated pipe holes (3/16" to 3/8" dia.) to be in lower half of the pipe with lower quarter segment unperforated for water flow.

Slotted pipe to have 1/8" maximum width slots.

NOTES

1. Capillary break beneath floor slab could be hydraulically connected to perimeter drain pipe. Use of 2-inch-diameter weep holes as shown is one applicable method.

2. If wet conditions render on-site soil unsuitable for compaction, backfill the zone shown above with imported structural fill. Imported structural fill should meet WSDOT Gravel Borrow Specification 9-03.14(1) but should have a maximum size of 3 inches, and should not have more than 5% fines (by weight based on minus 3/4" portion) passing No. 200 sieve (by wet sieving) with no plastic fines during wet conditions or wet weather.

3. Backfill within 3 feet of wall should be compacted with hand-operated equipment. Heavy equipment should not be used to compact backfill, as such equipment operated near the wall could increase lateral earth pressures and possibly damage the wall.

4. All backfill should be placed in layers not exceeding 4" loose thickness for light equipment and 8" for heavy equipment and densely compacted. Beneath paved or sidewalk areas, compact to at least 95% Modified Proctor maximum dry density (ASTM: D1557, Method C or D). Landscape areas could be compacted to 90% minimum.

Juanita Beach Park Bathhouse
Kirkland, Washington

TYPICAL WALL DRAINAGE AND BACKFILL

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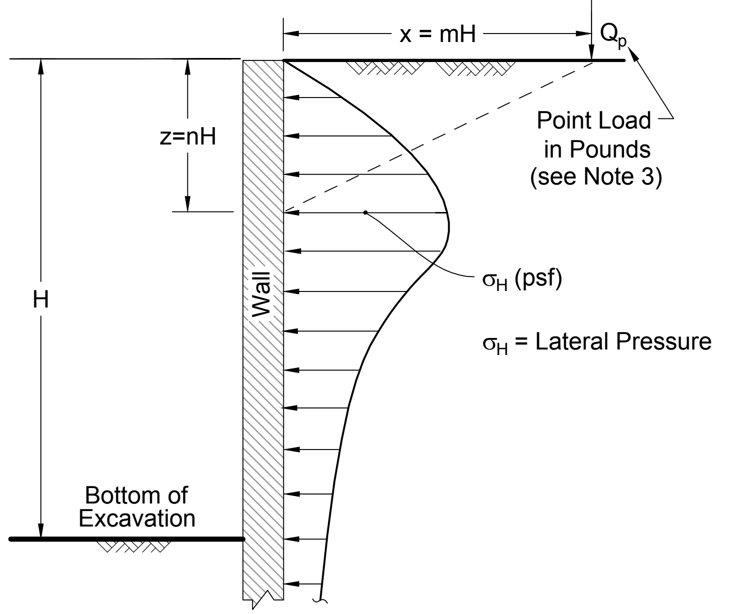
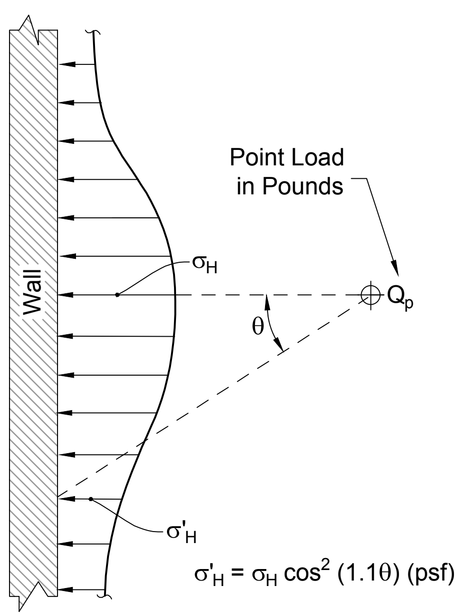
FIG. 3



ELEVATION VIEW

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

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

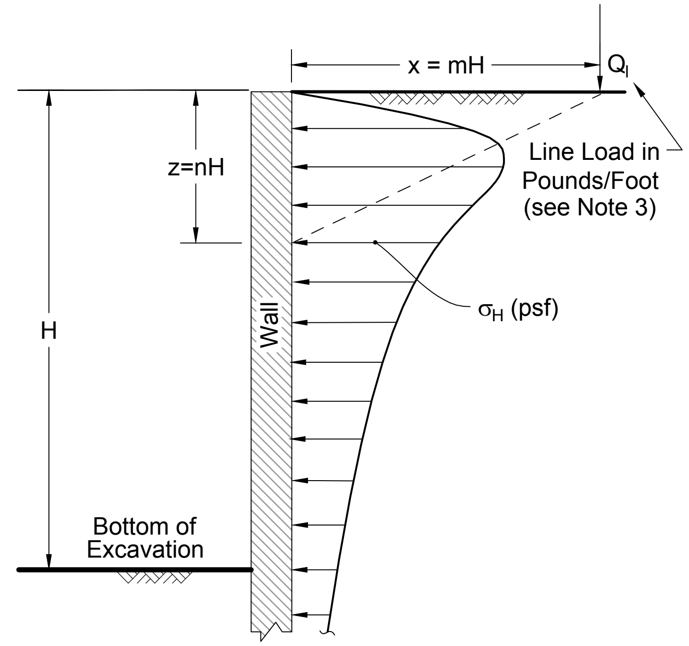


$\sigma'_H = \sigma_H \cos^2 (1.1\theta)$ (psf)

PLAN VIEW

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

(NAVFAC DM 7.2, 1986)



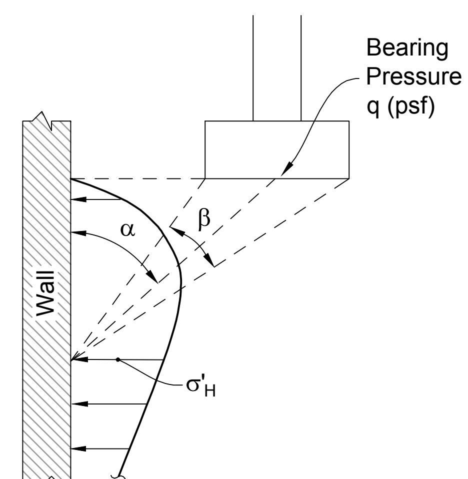
ELEVATION VIEW

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

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

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

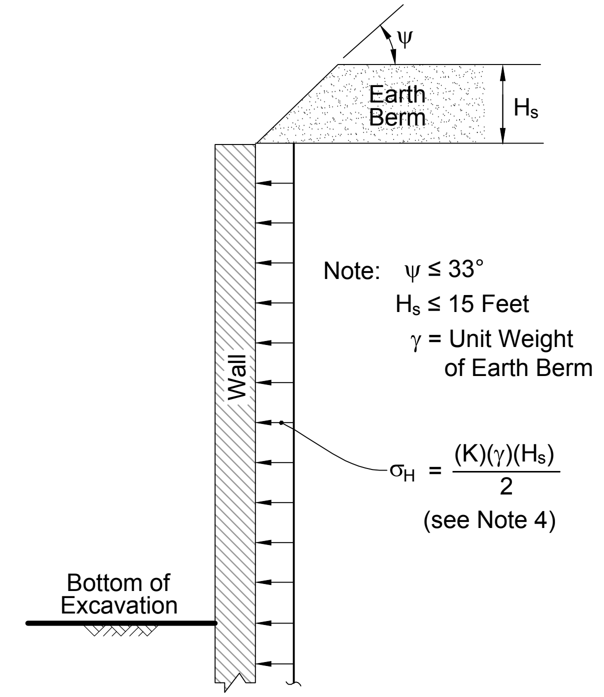
(NAVFAC DM 7.02, 1986)



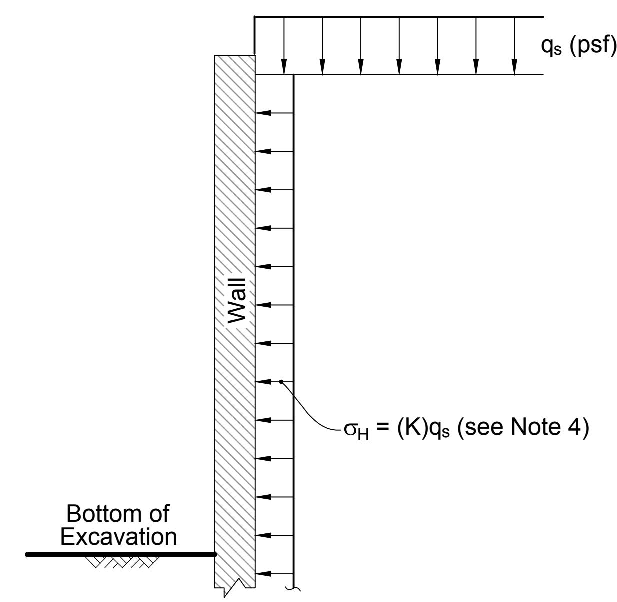
$\sigma_H = \dfrac{2q}{\pi} (\beta - \sin \beta \cos 2\alpha)$ (psf) (β in radians)

C) LATERAL PRESSURE DUE TO STRIP LOAD

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



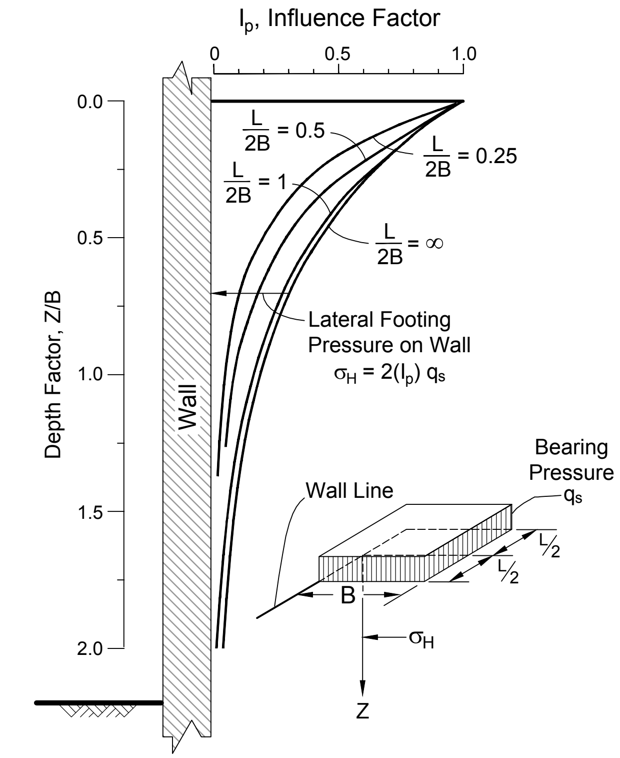
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE

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



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

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

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that m ≤ 0.4, it may be more appropriate to model the actual load distribution (i.e., Detail E) or use more rigorous analysis methods.
4. Use a K_a value of K_a = 0.33.
5. The stress is estimated on the back of the wall at the center of the length, L, of loading.
6. The estimated stress is based on a Poisson's ratio of 0.5.
7. For areas where fill will be placed immediately behind and above the rop elevation of the wall, Diagram D can be used to determine loads on the wall. For narrow fills adjacent to the wall, Diagram C can be used.

Juanita Beach Park Bathhouse
Kirkland, Washington

RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

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FIG. 4

File: J:\211\22161-008\21-1-22161-008 Fig 4 - Surcharge.dwg Date: 04-17-2017 Author: jrs

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

SUBSURFACE EXPLORATIONS

TABLE OF CONTENTS

FIGURES

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 GENERAL

The subsurface exploration program for the project was conducted by Shannon & Wilson, Inc. (Shannon & Wilson). The exploration program consisted of three soil borings, designated B-1 to B-3. The approximate locations of the explorations are shown in Figure 2.

The logs of the soil borings are presented as Figures A-2 to A-4. Figure A-1 presents a key to our classification of the soils encountered in the explorations.

A.2 SOIL BORINGS

The soil borings were drilled by Holocene Drilling, Inc. (Holocene). Boring B-1 was completed on October 27, 2015, and borings B-2 and B-3 were completed on March 23, 2017. Borings B-1 and B-2 extended 31.5 feet below existing grade. Boring B-3 extended 11.5 feet below existing grade. Disturbed samples were obtained in conjunction with the Standard Penetration Test (SPT). The SPT test is an in situ soil test, which can be used to interpret the several engineering properties of soils (see Section A.4). The Unified Soil Classification System (USCS), as described in Figure A-1, was used to classify the soils.

Holocene completed the soil borings using a track-mounted drill rig using hollow-stem auger (HSA) drilling techniques. HSA drilling consists of advancing continuous-flight augers to remove soil from the borehole. Soil samples are taken from the bottom of the boring by removing the center rod and lowering a split-spoon sampler through the hollow stem. Soil samples were taken in 2.5-foot intervals in the upper 20 feet and 5-foot intervals beyond 20 feet deep. After completing drilling, borings B-1 and B-3 were backfilled with bentonite chips. Holocene installed a 2-inch-diameter polyvinyl chloride well in boring B-2 and backfilled with sand and bentonite chips. Drill cuttings and spoils were put into drums, and the drums were taken off site by Holocene.

A.3 GROUNDWATER OBSERVATIONS

Groundwater was observed during drilling at about 3 to 5 feet below ground surface. The depth of groundwater is noted on the boring logs. We measured groundwater in the monitoring well installed in boring B-2 within the confined aquifer in April 2017. Observed water levels in the confined aquifer varied from 0.56 to 0.8 foot above the ground surface.

A.4 SOIL SAMPLING AND CLASSIFICATION

A Shannon & Wilson geologist observed and logged the drilling operations. Representative soil samples collected were transferred to our laboratory in Seattle, Washington, for analysis. The field logs and soil samples were reviewed by Shannon & Wilson personnel in the Seattle laboratory using the USCS field classification method. The boring logs in this report represent our interpretation of the field logs.

Disturbed soil samples were obtained in conjunction with the SPT. SPTs were performed in general accordance with the ASTM International (ASTM) Designation: D1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM, 2010).[1] SPTs were collected in the borings at 2.5-foot intervals. The SPT consists of driving a 2-inch outside diameter split-spoon sampler a total distance of 18 inches below the bottom of the drill hole with a 140-pound hammer falling 30 inches. The number of blows required to advance the split spoon from 6 to 18 inches of penetration is termed the Standard Penetration Resistance (N-value). The N-values are plotted in the boring logs presented in this appendix. These values provide a means for evaluating the relative density of granular soils and the relative consistency (stiffness) of cohesive soils.

A.5 EXISTING EXPLORATIONS

We reviewed subsurface explorations previously completed for the Juanita Bay Pumping Station project (Metropolitan Engineers, 1966).[2] Boring logs, boring locations, descriptions of the drilling methods, and sampling procedures can be found in the referenced report, available on the Washington State Department of Natural Resources website.

[1] ASTM International (ASTM), 2010, 2010 Annual book of standards, Construction, v. 04.08, Soil and rock (I): D420 - D5876: West Conshohocken, Pa.
[2] Metropolitan Engineers, 1966, Final report, soils investigation, Juanita Bay pumping station, Kirkland, Washington: Report prepared by Metropolitan Engineers, Seattle, Washington.

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

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

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

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

MOISTURE CONTENT TERMS

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

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

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

PARTICLE SIZE DEFINITIONS

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

RELATIVE DENSITY / CONSISTENCY

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

WELL AND BACKFILL SYMBOLS

Bentonite Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Inclinometer or Non-perforated Casing
Perforated or Screened Casing	Vibrating Wire Piezometer

PERCENTAGES TERMS [1, 2]

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

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

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

Juanita Beach Park Bathhouse
Kirkland, Washington

SOIL DESCRIPTION AND LOG KEY

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SHANNON & WILSON, INC.
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FIG. A-1
Sheet 1 of 3

SOIL_CLASS_KEY_PG1 21-22161.GPJ SHAN_WIL.GDT 4/19/17

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

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

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

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.

Juanita Beach Park Bathhouse
Kirkland, Washington

SOIL DESCRIPTION AND LOG KEY

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FIG. A-1
Sheet 2 of 3

GRADATION TERMS

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

CEMENTATION TERMS[1]

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

PLASTICITY[2]

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

ADDITIONAL TERMS

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

PARTICLE ANGULARITY AND SHAPE TERMS[1]

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

ACRONYMS AND ABBREVIATIONS

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

STRUCTURE TERMS[1]

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

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

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SOIL DESCRIPTION AND LOG KEY

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FIG. A-1
Sheet 3 of 3

Total Depth:	*31.5 ft.*	Northing:		Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*8 in.*
Top Elevation:	*~ 23 ft.*	Easting:		Drilling Company:	*Holocene Drilling*	Rod Diam.:	*2-inch*
Vert. Datum:		Station:		Drill Rig Equipment:	*Diedrich D50*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:		Other Comments:			

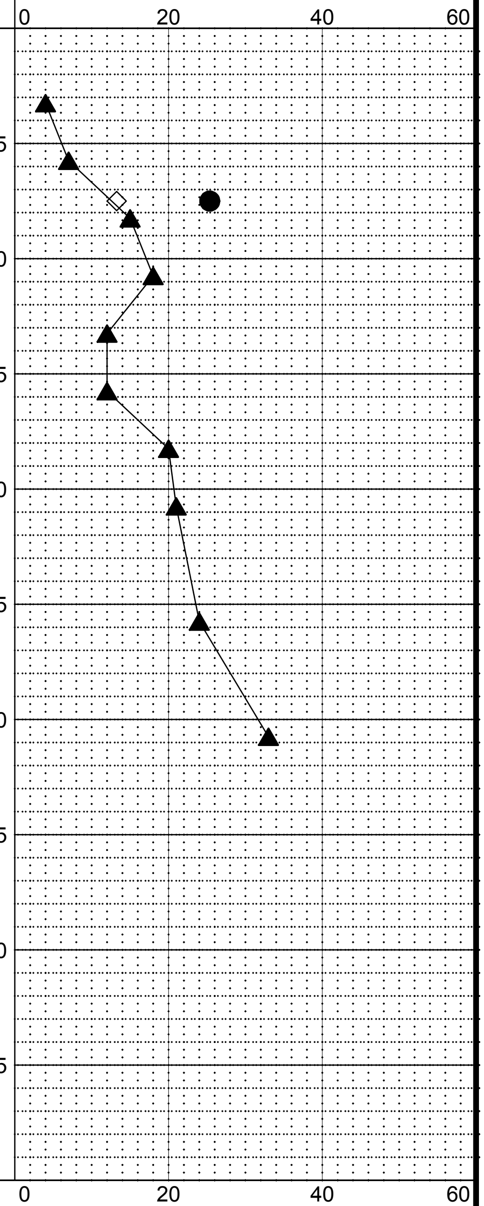
SOIL DESCRIPTION

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

Soil Description	Depth, ft.	Samples	Ground Water
Topsoil.	0.5		
Loose, brown, *Poorly Graded Sand with Silt (SP-SM)*; moist to wet; abundant wood fragments. (Ha)	7.0	1, 2	During Drilling ∇
Medium dense, dark brown, *Poorly Graded Sand with Silt to Silty Sand (SP-SM/SM)*; wet. (Ha)	15.0	3, 4, 5	
Stiff, gray-brown, *Sandy Silt (ML)*; wet. (Ha)	17.0	6	
Medium dense to dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet. (Ha)	31.5	7, 8, 9, 10	
BOTTOM OF BORING COMPLETED 10/27/2015			

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



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

LEGEND

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

NOTES

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

Juanita Beach Park Bathhouse
Kirkland, Washington

LOG OF BORING B-1

April 2017 21-1-22161-008

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2

REV 3 - Approved for Submittal

MASTER_LOG_E 21-22161.GPJ SHAN_WIL.GDT 4/18/17 Log: KJW Rev: JRH Typ: LKN

Total Depth:	*31.5 ft.*	Northing:		Drilling Method:	*Hollow Stem Auger*
Top Elevation:	*~ 21 ft.*	Easting:		Drilling Company:	*Holocene Drilling*
Vert. Datum:		Station:		Drill Rig Equipment:	*Diedrich D50*
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.:	*8 in.*
Rod Diam.:	*2-inch*
Hammer Type:	*Automatic*

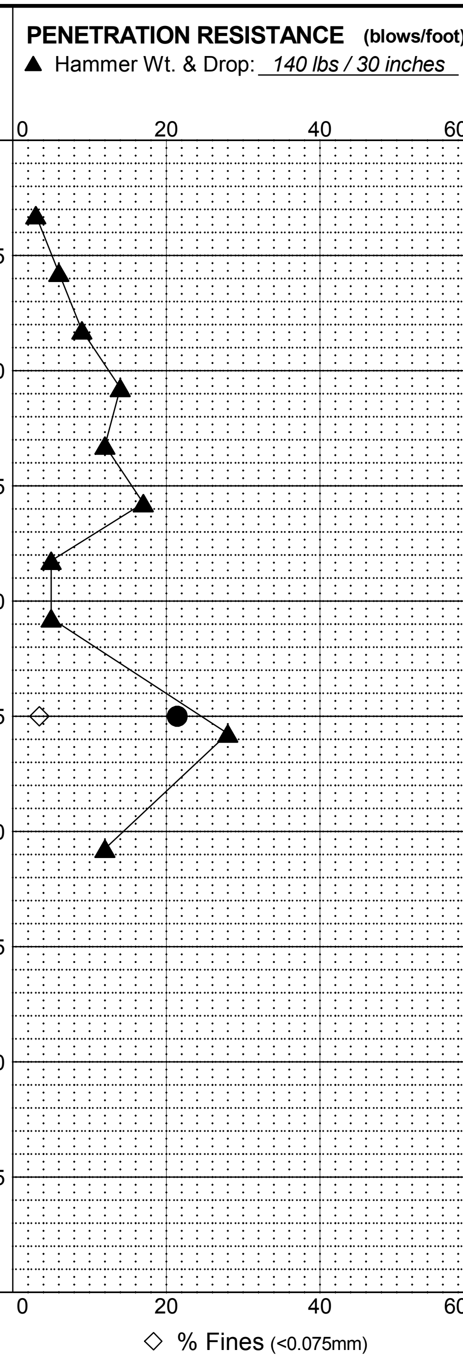
SOIL DESCRIPTION

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

Soil Description	Depth, ft.	Samples
Very loose to loose, gray-brown, *Poorly Graded Sand (SP)* to *Poorly Graded Sand with Silt (SP-SM)*; wet; trace fine subrounded gravel; fine to coarse sand; nonplastic fines; trace organics. (Ha)	7.0	1, 2
Medium dense, gray-brown, *Poorly Graded Sand (SP)*; wet; fine to medium sand; trace nonplastic fines; trace organics; organic odor. (Qvro/Ha)	12.0	3, 4
Medium dense, blue-gray to gray and brown, *Silt with Sand (ML)* and *Silt (ML)*; moist; fine sand; nonplastic to low plasticity fines; trace iron-oxide staining; trace silty sand seams and partings. (Qvrl)	17.0	5, 6
Loose to medium dense, gray-brown, *Poorly Graded Sand (SP)*; wet; fine to medium sand; nonplastic fined; trace organics. (Qvro) - 2 feet of heave was encounterd prior to the well installation.	31.5	7, 8, 9, 10

BOTTOM OF BORING
COMPLETED 3/23/2017

Ground Water: During Drilling



LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Well Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level ATD

NOTES

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

Juanita Beach Park Bathhouse
Kirkland, Washington

LOG OF BORING B-2

April 2017 21-1-22161-008

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FIG. A-3

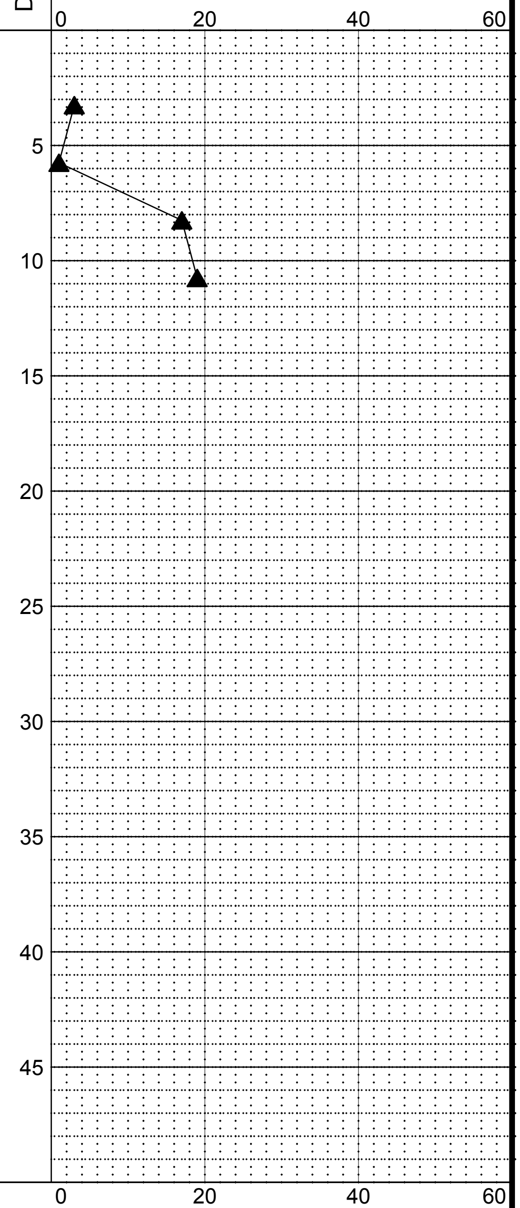
REV 3 - Approved for Submittal

MASTER_LOG_E 21-22161.GPJ SHAN WIL.GDT 4/18/17 Log: JJM Rev: EAS Typ: LKN

Total Depth:	11.5 ft.	Northing:		Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~ 22 ft.	Easting:		Drilling Company:	Holocene Drilling	Rod Diam.:	2-inch
Vert. Datum:		Station:		Drill Rig Equipment:	Diedrich D50	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	Samples	Ground Water
Very loose, gray-brown, *Poorly Graded Sand (SP)* to *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; few organics. (Ha)			1, 2	During Drilling
Medium dense, gray, *Poorly Graded Sand (SP)*; wet; fine to medium sand; trace nonplastic fines; trace organics. (Qvro/Ha)	7.0		3, 4	
BOTTOM OF BORING COMPLETED 3/23/2017	11.5			

PENETRATION RESISTANCE (blows/foot)
▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered
2.0" O.D. Split Spoon Sample
Ground Water Level ATD

NOTES

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

Juanita Beach Park Bathhouse
Kirkland, Washington

LOG OF BORING B-3

April 2017 21-1-22161-008

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FIG. A-4

REV 3 - Approved for Submittal

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

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FIGURE

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

Samples collected from the boring B-1 were sealed in jars and returned to the Shannon & Wilson, Inc. (Shannon & Wilson) laboratory for testing. The Shannon & Wilson laboratory conducted the tests.

B.1 WATER CONTENT DETERMINATION

The water content was determined for select boring samples. Water content determination tests are generally performed in accordance with ASTM International (ASTM) D2216, Standard Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock. Comparison of water content of a soil with its index properties can be useful in characterizing soil unit weight, compactness, consistency, compressibility, and strength. Water content is plotted in the boring logs presented in Appendix A.

B.2 GRAIN SIZE ANALYSIS

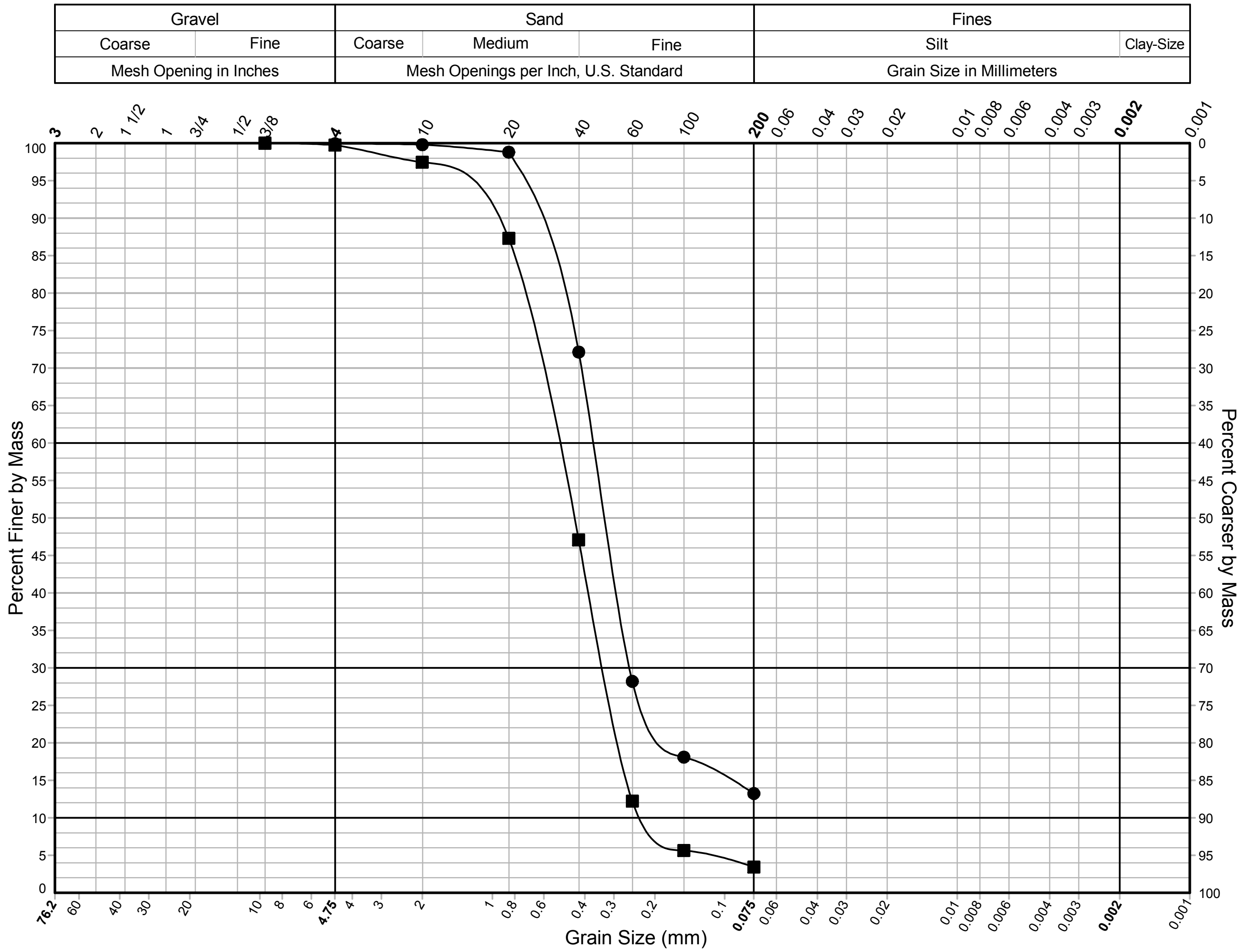
Two grain size analyses were performed from one sample each in borings B-1 and B-2. Grain size analyses are generally performed in accordance with ASTM D422, Standard Method for Particle Size Analysis of Soils.[1] Results of the grain size analyses are presented in Figure B-1. This figure also shows percent fines in tabular form.

[1] ASTM International (ASTM), 2007, Annual book of standards, construction, v. 4.08, soil and rock (I): D420 – D5611: West Conshohocken, Pa.

GRAIN SIZE DISTRIBUTION PLOT

Juanita Beach Park Bathhouse
Kirkland, Washington

B-1|7.5, B-2|25



Sample Identification	Depth (ft)	USCS Group Symbol	USCS Group Name	Gravel %	Sand %	Fines %	< 20µm %	< 2µm %	WC %	Tested By	Review By	ASTM Std.
● B-1, S-3*	7.5	SM	Silty Sand	0	87	13			25.4	AKV	JFL	D422
■ B-2, S-9*	25.0	SP	Poorly Graded Sand	0	96	3.4			21.4	AKV		C136

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

21-1-22161-008 A_GSA_MAIN 21-22161.GPJ SHAN_WIL.GDT 4/18/17

APPENDIX C

HYDROGEOLOGIC TESTING AND GROUNDWATER LEVEL MONITORING

APPENDIX C

HYDROGEOLOGIC TESTING AND GROUNDWATER LEVEL MONITORING

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APPENDIX C

HYDROGEOLOGIC TESTING AND GROUNDWATER LEVEL MONITORING

C.1 SLUG TESTING

Single-well field hydraulic conductivity tests (slug tests) were performed in observation well B-2 to estimate the horizontal hydraulic conductivity of the soils. The slug tests were performed on March 27, 2017. A slug test provides an in situ means of estimating the horizontal hydraulic conductivity of the saturated sediments surrounding the screened zone of a well. Slug tests do not provide data regarding large-scale aquifer properties, aquifer geometry, or boundary conditions affecting groundwater flow.

Slug testing consists of rapidly raising or lowering the water level within an observation well and measuring the recovery of the water level over time to the static level. Raising the water level is achieved by lowering a slug (a sealed, sand-filled, polyvinyl chloride [PVC] pipe) below the static water level to displace water within the well casing. This procedure is termed a “falling head test” because the water level falls with time back to the static level. Lowering the water level is achieved by quickly removing the slug from the well. This is termed a “rising head test” because the water level rises back to the static level after the slug is removed. Both rising and falling head tests were performed as part of the slug testing at each location.

Field staff measured and recorded the variation in water level during the testing period at the well using a downhole combination pressure transducer/data logger, with additional water level measurements being made with an electronic water level indicator. The transducer was secured in the well below the depth to which the slug would be lowered, and rapid water level measurements were made by the transducer and recorded by the data logger.

The slug test data were analyzed using the Bouwer and Rice solution (Bouwer and Rice, 1976; and Bouwer, 1989). Figures C-1 through C-8 present the slug test data in semi-log plots of water level change versus time.

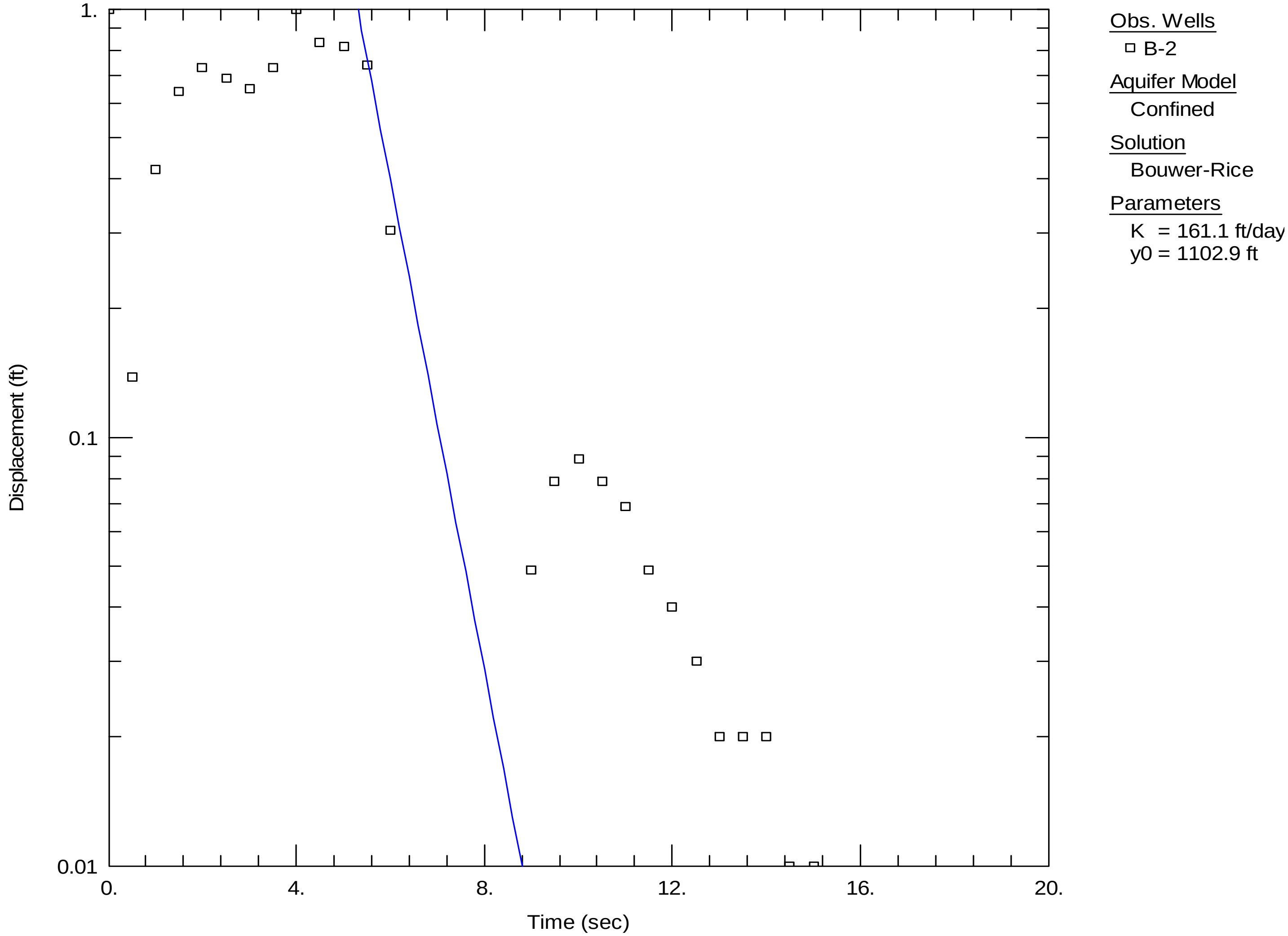
C.2 GROUNDWATER LEVEL MONITORING

Groundwater levels in the observation well originally installed for the project were measured in March 2017. Groundwater levels were measured in Observation well water level measurements were made using an electric water level indicator measured relative to the top of the PVC well casing. The water level at observation well B-2 varied from 0.56 to 0.8 foot above ground surface.

C.3 REFERENCES

Bouwer, Herman, 1989, The Bouwer and Rice slug test – an update: Ground Water, v. 27, no. 3, May-June, p. 304-309.

Bouwer, Herman, and Rice, R.C., 1976, A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells: Water Resources Research, v. 12, no. 3, June, p. 423-428.



NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

FIG. C-1

Juanita Beach Park Bathhouse
Kirkland, Washington

FALLING HEAD TEST 1
OBSERVATION WELL B-2

April 2017 21-1-22161-008

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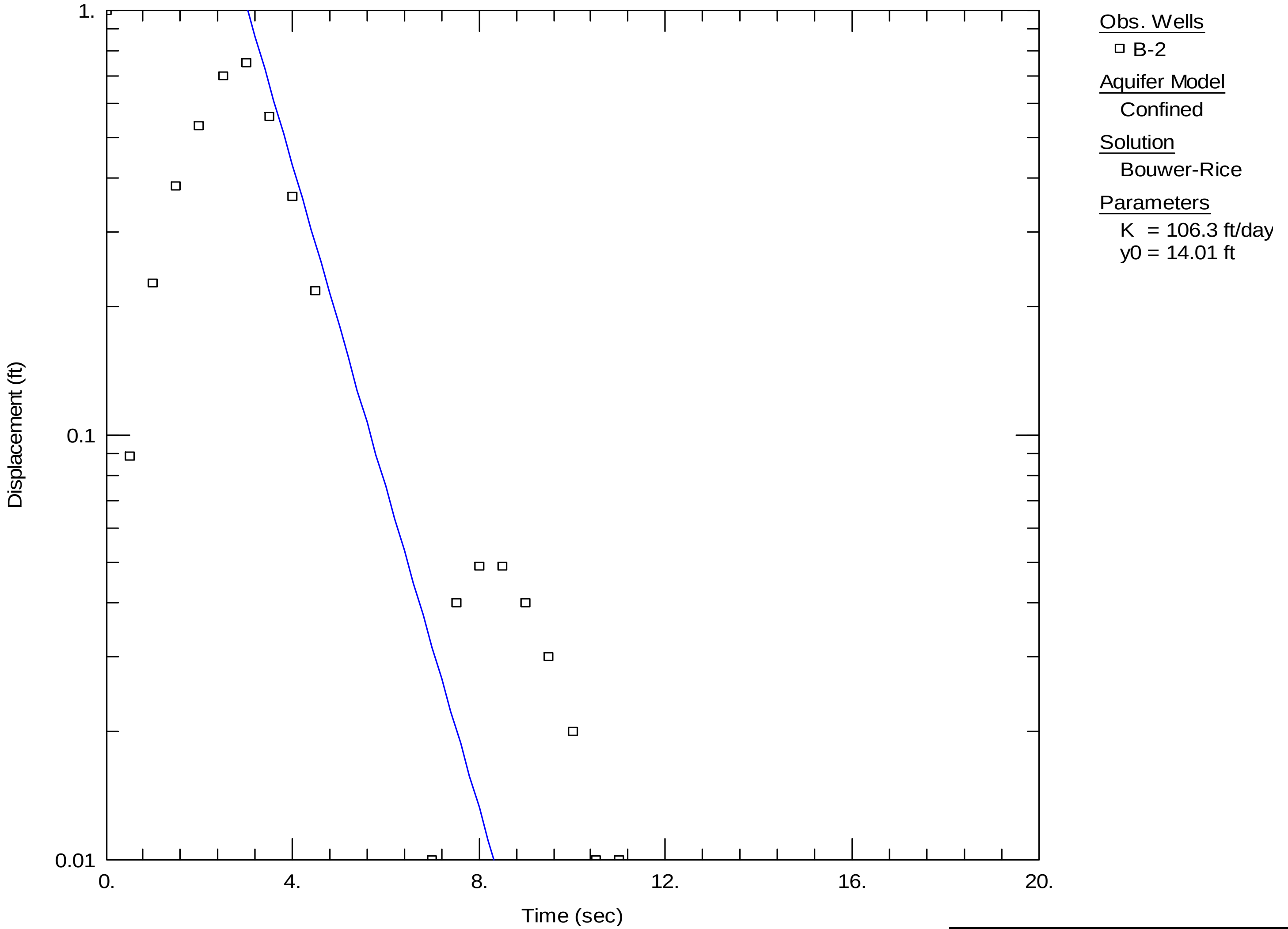
FIG. C-1



NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

FIG. C-2

Juanita Beach Park Bathhouse
Kirkland, Washington

FALLING HEAD TEST 2
OBSERVATION WELL B-2

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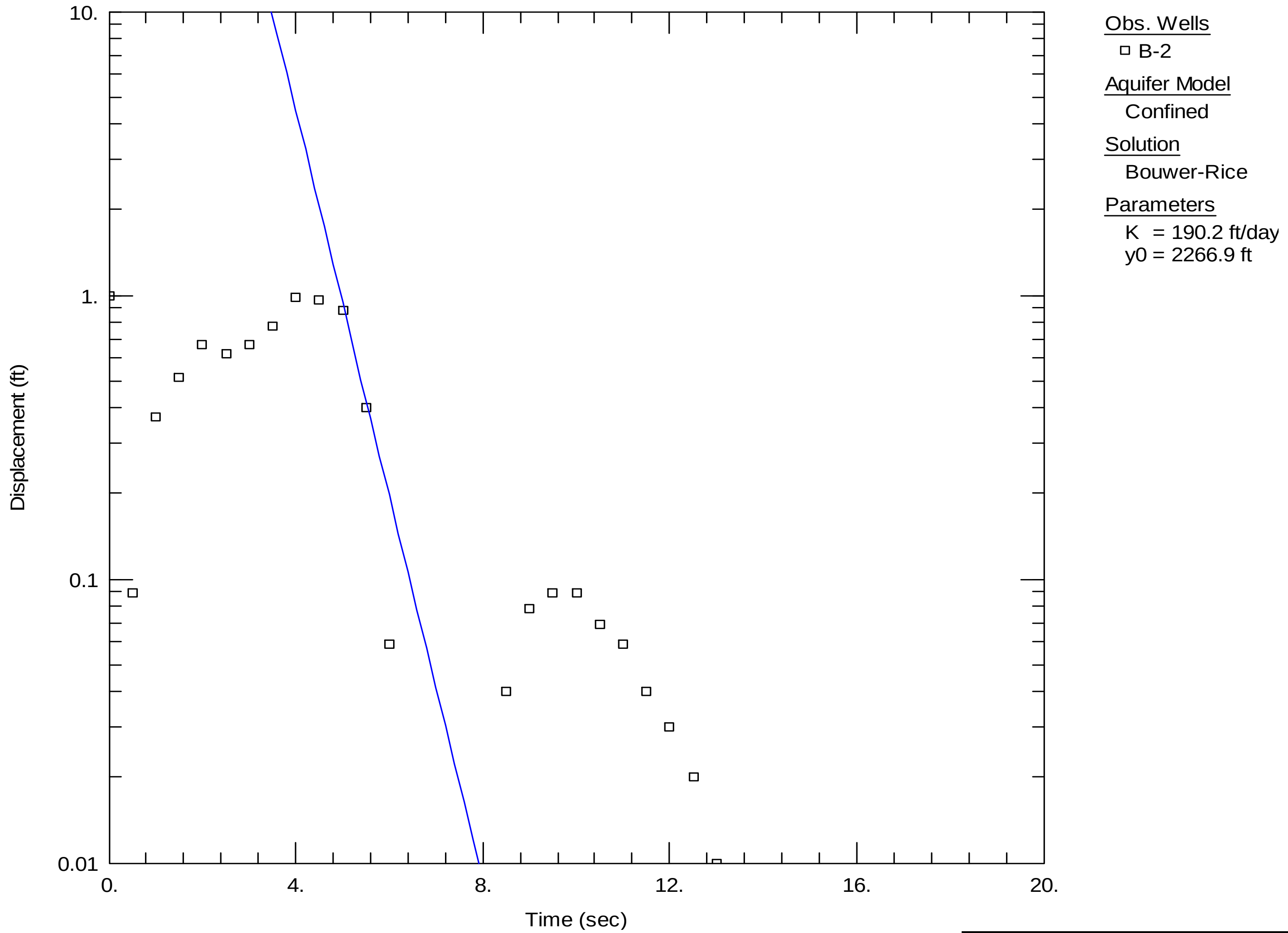
FIG. C-2



NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

FIG. C-3

Juanita Beach Park Bathhouse
Kirkland, Washington

FALLING HEAD TEST 3
OBSERVATION WELL B-2

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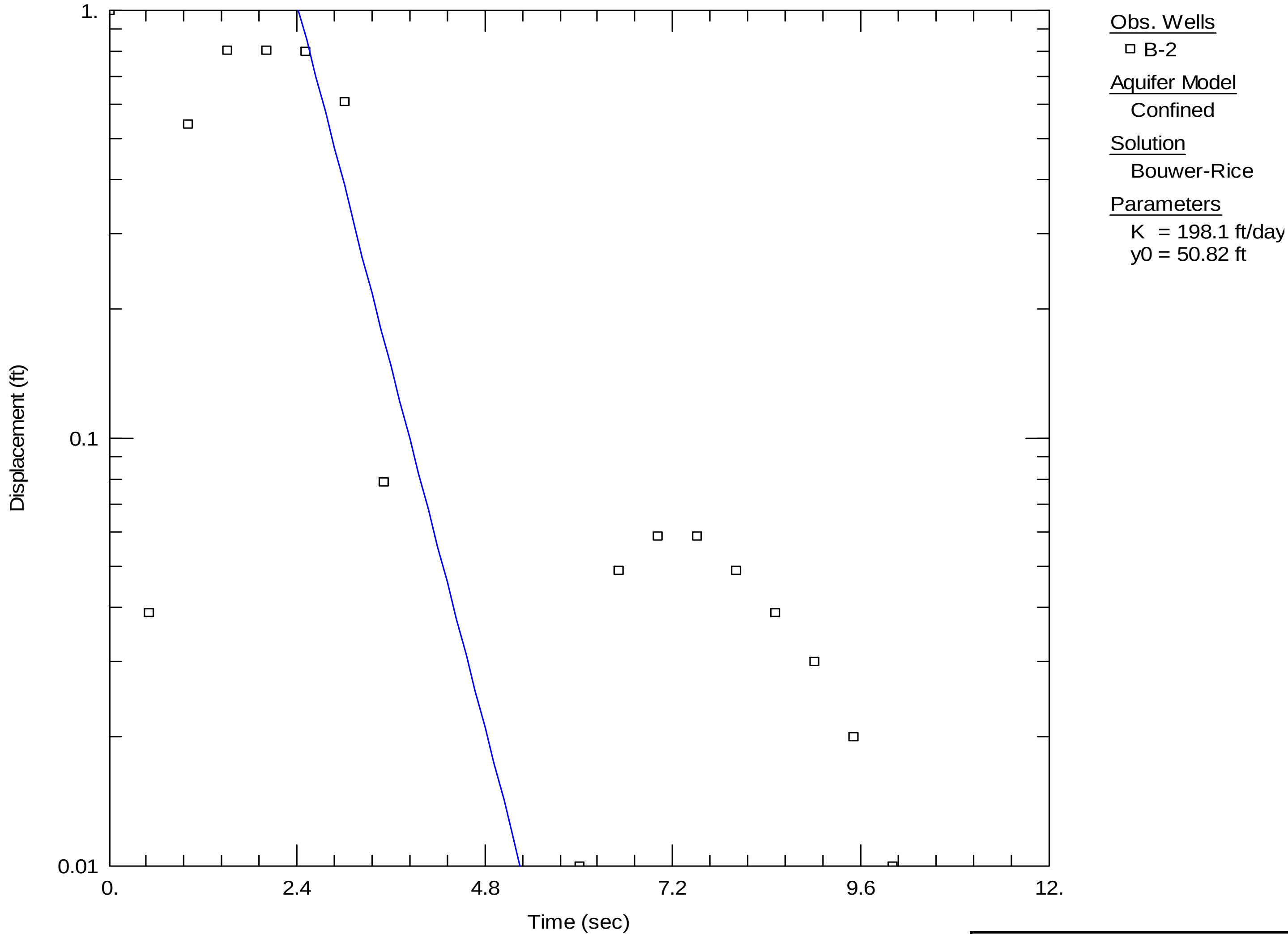
FIG. C-3



NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

FIG. C-4

Juanita Beach Park Bathhouse
Kirkland, Washington

FALLING HEAD TEST 4
OBSERVATION WELL B-2

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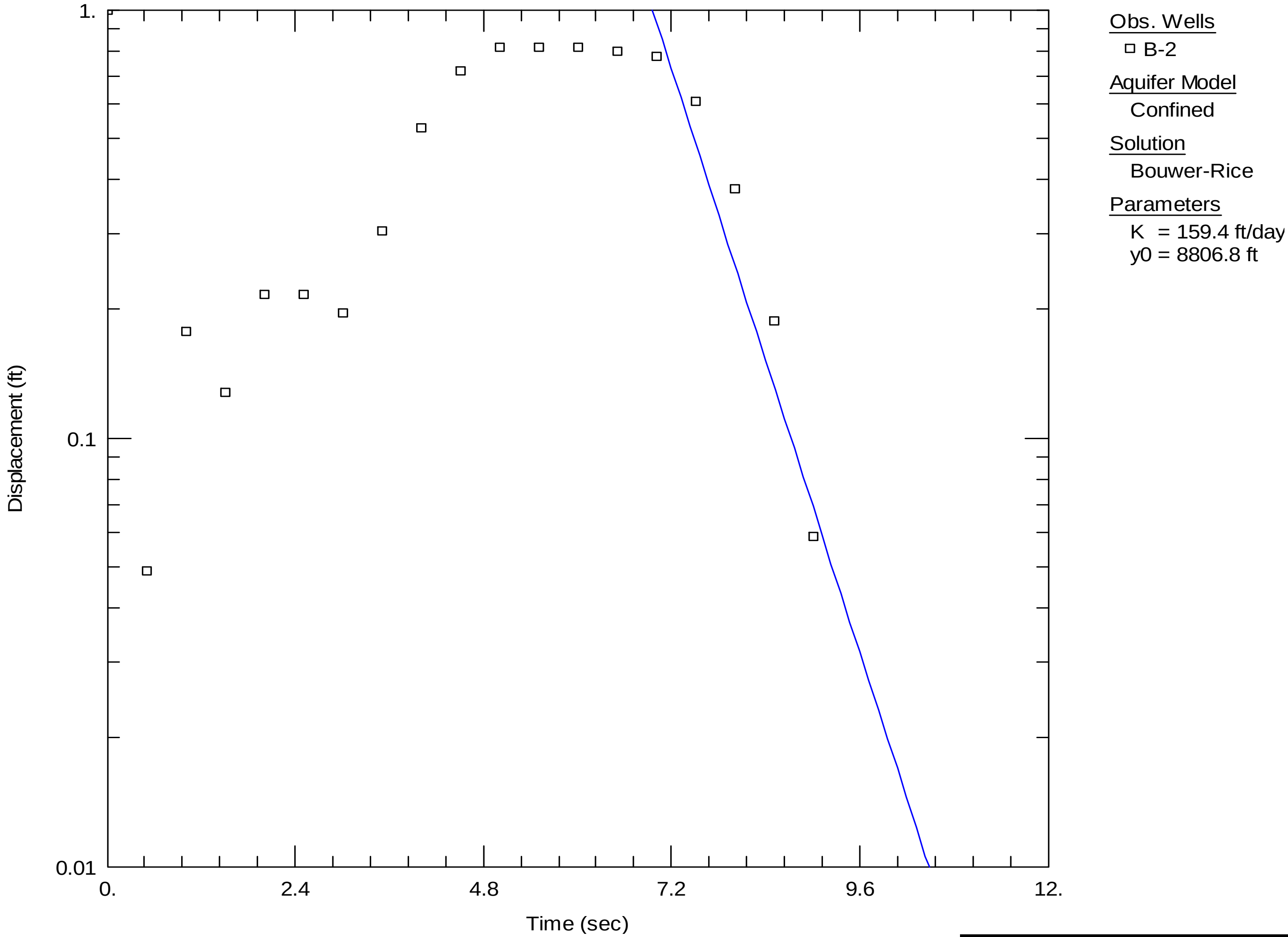
FIG. C-4



NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

FIG. C-5

Juanita Beach Park Bathhouse
Kirkland, Washington

RISING HEAD TEST 1
OBSERVATION WELL B-2

April 2017 21-1-22161-008

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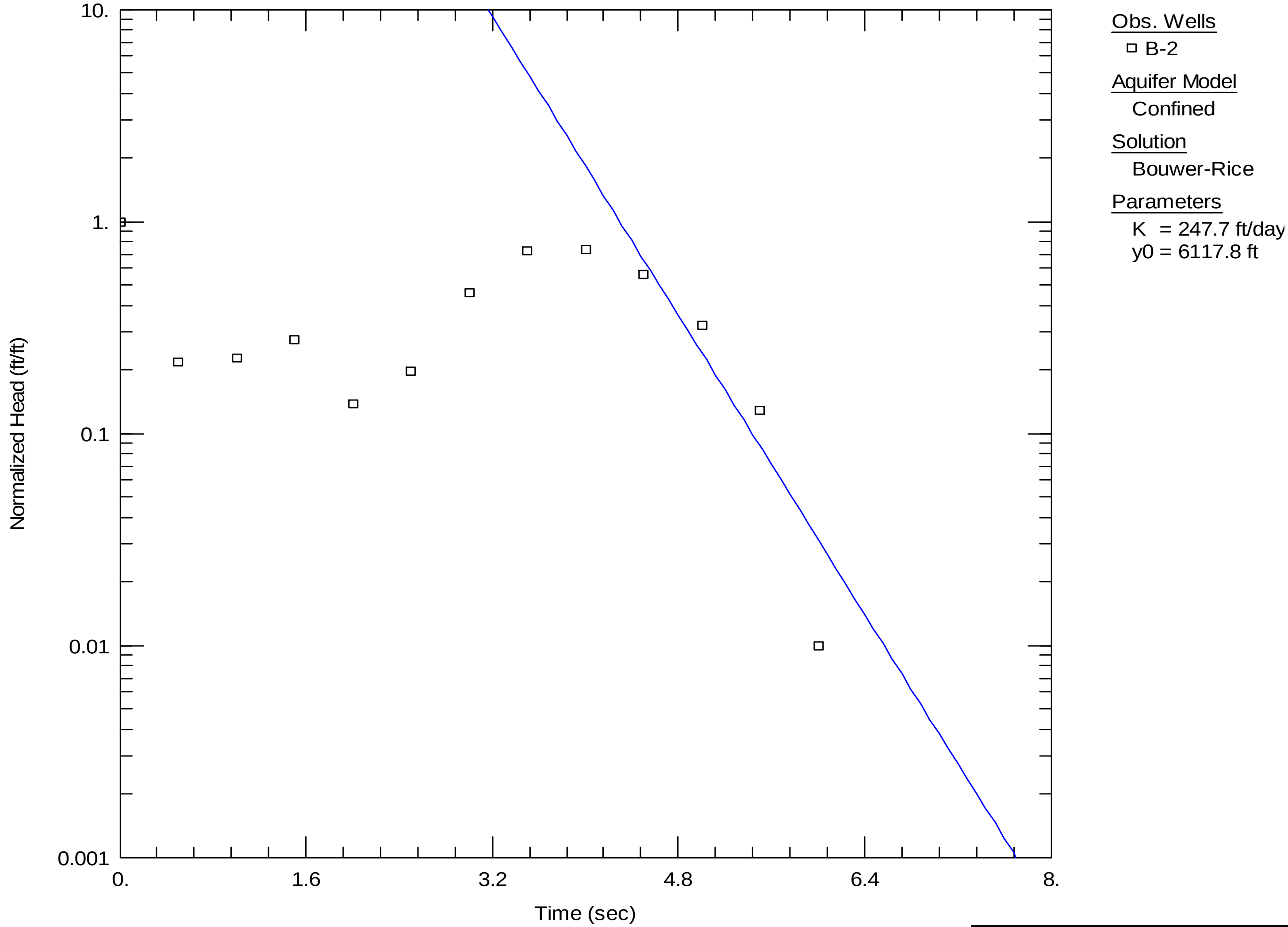
FIG. C-5



NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

FIG. C-6

Juanita Beach Park Bathhouse
Kirkland, Washington

RISING HEAD TEST 2
OBSERVATION WELL B-2

April 2017 21-1-22161-008

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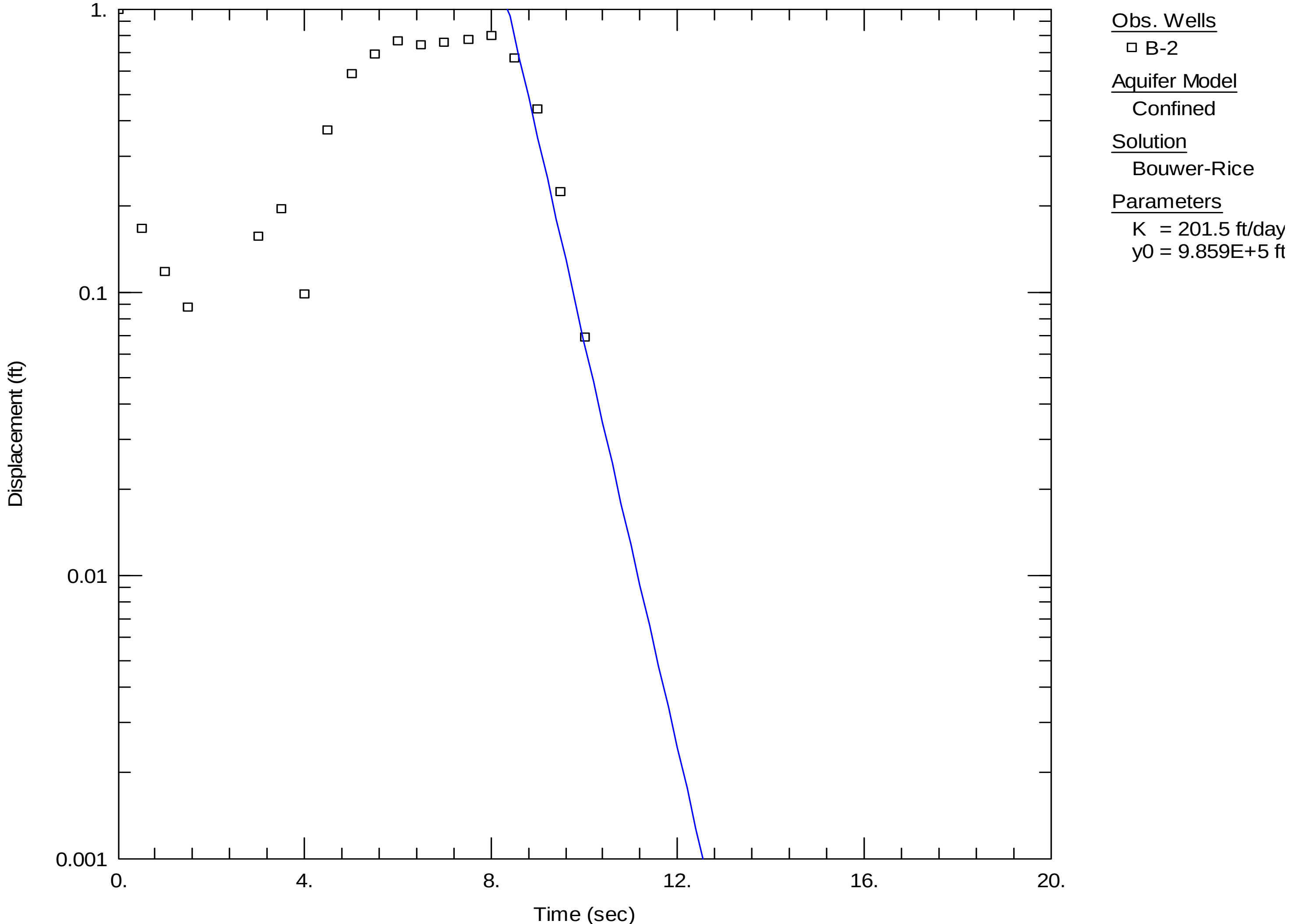
FIG. C-6



NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

FIG. C-7

Juanita Beach Park Bathhouse
Kirkland, Washington

RISING HEAD TEST 3
OBSERVATION WELL B-2

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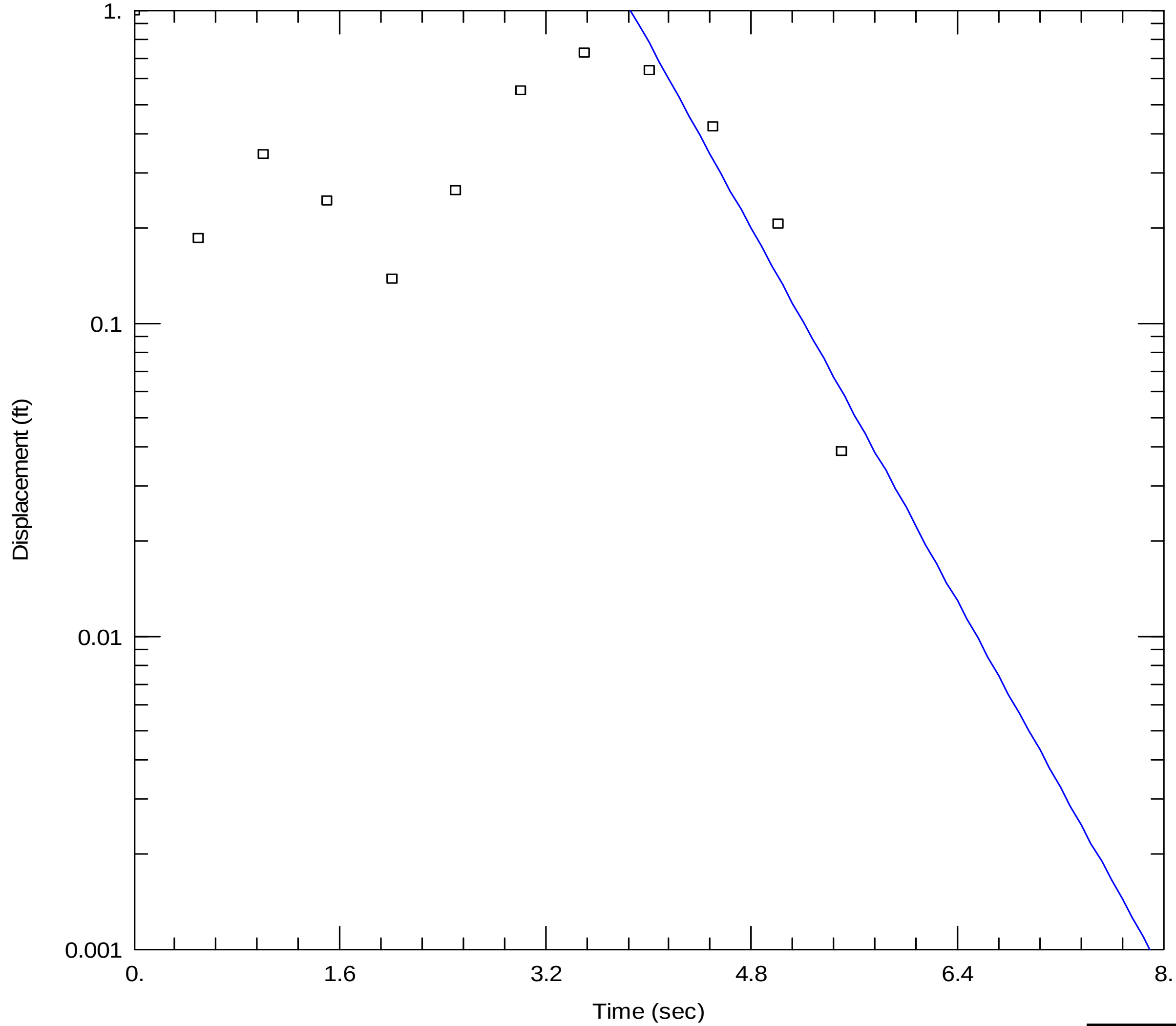
FIG. C-7



Obs. Wells
B-2

Aquifer Model
Confined

Solution
Bouwer-Rice

Parameters
K = 209.2 ft/day
y0 = 747.2 ft

NOTES

1. Observed test data represented by squares in plot
2. Modeled line in plot is selected based on Bower and Rice (1976)

ft= feet
min= minutes

Juanita Beach Park Bathhouse
Kirkland, Washington

RISING HEAD TEST 4
OBSERVATION WELL B-2

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FIG. C-8

FIG. C-8

APPENDIX D

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-22161-008

Date: April 19, 2017
To: Mr. Erik Barr
Patano Studio Architecture

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

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