PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

February 20, 2018
File No. 17-296

King County Housing Authority
c/o Spectrum Development Solutions
1809 Seventh Avenue, Suite 501
Seattle, WA 98112
Attention: Matt Wiley

Subject: **GEOTECHNICAL REPORT - DRAFT**
Issaquah TOD
1505 Newport Way NW, Issaquah, Washington

Dear Mr. Wiley:

As requested, PanGEO, Inc. completed a geotechnical study to assist the project team with the design and construction of the proposed mixed-use workforce housing project in Issaquah, Washington. We understand that the two proposed buildings will be at-grade eight-story structures. The results of our study are summarized in the attached draft report. We will finalize our report after we receive review comments from the project team.

In summary, the site is underlain by a shallow groundwater table and thick layer of compressible soil that is susceptible to liquefaction. It is our opinion that the site may be developed generally as planned, provided the effects of compressible soils and the risk of liquefaction are properly considered into the design of the building foundation. Ground improvement methods such as aggregate piers or supporting the buildings on deep foundation elements (i.e. piles) are considered appropriate measures to support the buildings.

We appreciate the opportunity to work with you on this project. Please call if there are any questions regarding this report.

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Seattle, WA 98102
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Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer

Encl.: Draft Geotechnical Report

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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering study that was undertaken to support the design and construction of the proposed development at the subject site. This study was conducted in general accordance with our scope of work outlined in our proposal dated September 27, 2017. Our service scope included reviewing readily available geologic and geotechnical data in the vicinity of the project site, conducting a site reconnaissance, advancing four cone penetration tests, and performing engineering analyses to develop the geotechnical recommendations outlined in this report.

2.0 SITE AND PROJECT DESCRIPTION

The subject site consists of an approximately 174,189 square-foot (4 acres) rectangular-shaped parcel located at 1505 Newport Way Northwest in Issaquah, Washington. The proposed project location is shown in the attached Figure 1, Vicinity Map. The subject site is bound to the north by Northwest Maple Street, to the south by Newport Way Northwest, to the east by a business park, and to the west by the Sound Transit Issaquah Transit Center (see aerial photo on following page).

The site is currently occupied by a Centurylink operations center that includes a one-story approximately 33,680 square-foot building of concrete construction with a slab-on-grade floor that we understand was constructed in 1981. Asphalt paved parking and storage areas surround the existing building and a stormwater detention pond is located in the northwestern portion of the site. According to the site topographic survey prepared by Triad, site grades gently slope down to the north with about 7 feet of vertical relief between the north and south property lines.

Plate 1 on the following page depicts the approximate limits of the site in relation to current site developments and adjacent streets. The location of previous and current subsurface explorations are also indicated in Plate 1.

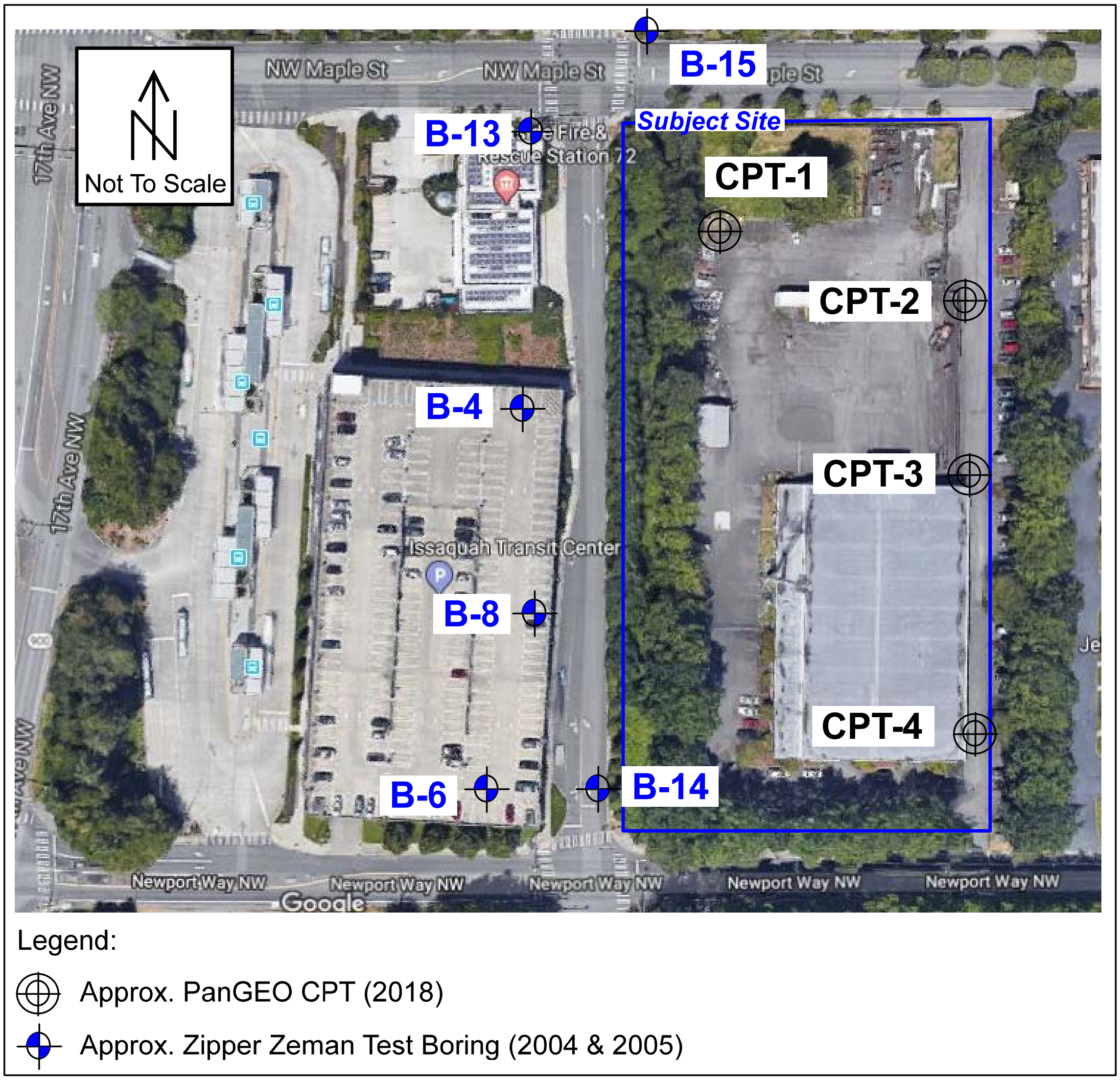

Plate 1. Aerial View of Existing Project Site Including Subsurface Explorations (Modified from Google Maps).

As currently planned, the proposed development will consist of a new mixed-use workforce housing project that will feature two at-grade eight story buildings. We understand each building will be 5 levels of wood frame construction over 3 levels of concrete construction. We anticipate foundation excavations will be less than 4 feet deep.

3.0 SUBSURFACE EXPLORATIONS

3.1 Cone Penetration Tests

Four cone penetration tests (CPT) were performed by In-Situ Engineering of Snohomish, Washington on January 2, 2018. The approximate CPT locations are shown in the attached Figure 2 and on Plate 1 on page 2 of this report. The CPTs were advanced approximately 61 to 72 feet below grade before encountering practical refusal in a dense gravelly sand deposit. A piezometer-equipped cone tip was utilized for the tests. Summary CPT logs are included in Appendix A as Figures A-1 through A-4.

A CPT consists of pushing an instrumented cone, approximately one-inch in diameter, into a soil deposit from a truck mounted reaction frame, and measuring the resistance and pore water pressure on the tip and side of the cone. Higher tip resistance measurements indicate the soil deposit has a higher strength or density than lower tip resistance measurements. The resistances to continuous penetration encountered by the cone tip and adjacent friction sleeve also exhibit high sensitivity to changes in soil type, which may be correlated to differing soil types and strength parameters. The principal advantages of using a CPT are minimum site disturbance and continuous profiling of the underlying soil.

3.2 Previous Subsurface Investigations

We reviewed the results of previous test borings performed for the Issaquah Transit Center located immediately west of the site. Specifically, we reviewed the logs of test borings B-4, B-6, B-8, B-13, B-14, and B-15, which were advanced on the east side of the Issaquah Transit Center site. The test borings for the Issaquah Transit Center were completed by Zipper Zeman Associates, Inc. (ZZA) between June 2004 and May 2005. The above-mentioned borings were advance between 21½ and 74 feet below grade. The previous test boring locations for the Issaquah Transit Center are shown in Plate 1 on page 2 of this report. The logs of the previous explorations are included in Appendix B of this report.

4.0 SUBSURFACE CONDITIONS

4.1 SOIL

According to the geologic map of the area compiled by Booth, et al. (2012), the surficial geologic unit mapped at the site is alluvium. Alluvium typically consists of loose to medium dense gravel, sand and sandy silt that has been deposited along stream channels. Based on the results of previous and recent subsurface explorations at the site, the site is underlain by a sequence of recent fill and soft silt and clay deposits over the mapped alluvium. The following is a summary of the subsurface conditions encountered in the explorations. Details can be found on the CPT logs included in Appendix A and the nearby previous test boring logs in Appendix B of this report.

Unit 1: Fill – Based on our interpretation of the CPTs, we anticipate the site to be underlain by about 3 to 6 feet of granular fill material. The fill generally consists of dense relatively clean to silty sand with gravel.

Unit 2: Lacustrine/Alluvial Deposits – Underlying the existing fill, very soft to medium stiff silty clay to clayey silt with a varying sand content was encountered to between 35 and 48 feet below grade at the CPT locations. Due to the generally fine-grained nature of this soil unit, we interpret is as a lacustrine (i.e. lake) or an alluvial deposit from a low-energy stream. This soil unit contained scattered medium dense to dense relatively clean to silty sand lenses, and the upper portion of this soil unit contained occasional 1- to 2-foot thick organic silt lenses. This unit is generally consistent with the mapped geology of the area compiled by Booth, et al., 2012. ZZA test borings B-13, B-14, and B-15 were terminated in this soil unit.

Unit 3: Older Alluvium – Underlying the Lacustrine/Alluvial soil unit, medium dense to dense relatively clean sand to silty sand with a varying gravel content was encountered to the maximum depth explored at all of the CPT locations as well as ZZA test borings B-4, B-6, and B-8. This soil unit contained occasional stiff to very stiff silt lenses.

4.2 GROUNDWATER

Review of the ZZA boring logs for the Issaquah Transit Center indicates that the static groundwater table was measured between about 5 and 7 feet below grade (i.e. between approx. 66 feet and 70 feet El.) in their groundwater monitoring wells on July 7, 2004. It should be noted that groundwater elevations may vary depending on the season, local subsurface conditions, and other factors. Groundwater levels are normally highest during the winter and early spring.

5.0 SEISMIC DESIGN CONSIDERATIONS

5.1 IBC SEISMIC PARAMETERS

The 2015 International Building Code (IBC) seismic design section provides a basis for seismic design of structures. Because the submerged Unit 2 and Unit 3 deposits are prone to soil liquefaction (see additional discussions in Section 5.2 of this report), Site Class F should be assumed for the seismic design of the project. With Site Class F, a site-specific ground response analysis will be required unless the natural period of the building is less than 0.5 second. However, based the currently-proposed building height of eight stories, we anticipate the natural building period to exceed 0.5 second, but this should be confirmed by the project structural engineer.

Building Period Less than 0.5 Second. If the building period is less than 0.5 second, it is our opinion that Site Class D is appropriate. As such, seismic parameters outlined in Table 1, on the following page, may be used for the seismic design of the building. These parameters are in conformance with the 2015 IBC, which specifies a design earthquake having a 2% probability of occurrence in 50 years (return interval of 2,475 years), and the 2008 USGS seismic hazard maps.

Table 1. IBC Seismic Design Parameters
(For buildings with period less than 0.5 second)

Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters		Control Periods (sec.)		Design PGA (S_{DS}/2.5)
			F_a	F_v	S_{DS}	S_{D1}	T_O	T_S	
D	1.33	0.50	1.0	1.5	0.89	0.50	0.11	0.57	0.35

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2008 data) for the project latitude and longitude.

If a site-specific ground response analysis will be needed, PanGEO will provide a separate proposal to complete the analysis

5.2 Soil Liquefaction

Liquefaction occurs when saturated sands are subjected to cyclic loading, and causes the pore water pressure to increase in the sand thereby reducing the inter-granular stresses. As the inter-granular stresses are reduced, the shearing resistance of the sand decreases. If pore pressures develop to the point where the effective stresses acting between the grains become zero, the soil particles will be in suspension and behave like a viscous fluid. Typically, loose, saturated, clean granular soils, that have a low enough permeability to prevent drainage during cyclic loading, have the greatest potential for liquefaction, while more dense soil deposits with higher silt or clay contents have a lesser potential. Soil liquefaction may cause the temporary loss/reduction of foundation capacity and settlement.

We evaluated the liquefaction-induced settlement at the site considering the subsurface conditions encountered at CPT-1 through CPT-4, and a Magnitude 7.5 earthquake with a design ground acceleration (PGA) of 0.35g. We utilized CLiq software by GeoLogimiki for liquefaction analysis. During a 2,475-year code level earthquake, our analysis, using the procedure proposed by the 1996 and 1998 NCEER/NSF workshops (Youd et al., 2001), indicated that the site has a high potential for soil liquefaction.

For levels of ground shaking consistent with 2015 IBC, the effect of liquefaction of the underlying soils is estimated to result in surface settlements on the order of 3 to 4 inches. Our analyses indicate that the depth of liquefaction extends about 56 to 68 feet below grade at our CPT locations. Although theoretical computations may suggest that liquefaction could extend below a depth of 50 feet, as a practical matter, effects of soil liquefaction occurring deeper than about 50 to 60 feet are not likely to be manifest at the ground surface.

In addition to the liquefaction-induced settlement, the occurrence of soil liquefaction will likely lead to temporary loss of footing bearing capacity and potential foundation failure. As such, for conventional footings founded on or near liquefiable soils, the occurrence of soil liquefaction could lead to significant settlement. Given that the groundwater at the site is only about 5 feet deep, and the bottom of the footings may be less than 3 feet above potentially liquefiable soils. It is our opinion that conventional footings are not appropriate for the proposed buildings without ground improvement such as aggregate piers.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 FOUNDATION SUPPORT OPTIONS

The alluvial soils beneath the site are subject to compression and settlement upon an increase in overburden stress under static conditions. It is our opinion that conventional footings founded on surficial soils are likely to undergo large, unacceptable levels of settlements under the anticipated foundation loads, unless ground improvements such as aggregate piers are installed.

A pile foundation would provide good performance during both static and seismic conditions, however, it would likely be the most costly. Based on our understanding of subsurface conditions in the area and considering that liquefaction may extend up to about 70 feet below grade, piles are anticipated to be quite long, possibly about 100 feet long. PanGEO is available to provide pile foundation recommendations if it is desired to pursue this foundation support option.

Preloading has been widely used to improve soft soils supporting relatively light structures. Although preloading will reduce the compressibility of site soils, it does not mitigate the liquefaction hazard at the site, and the performance of preloaded soils will not be as good as ground improvement such as aggregate piers. Furthermore, for an at-grade 8 story building, the fill for preload would be quite thick and the cost to import/export preload soil as well as the duration of a preload program would likely prevent this option from being cost-effective.

Based on the various factors discussed above, it is our opinion that conventional footings or a mat founded on aggregate piers likely will be the most cost-effective foundation. Additional discussions on aggregate piers are outlined below. PanGEO will also be available to provide additional foundation design input for other foundation alternatives will be considered.

6.1.1 Aggregate Piers

A shallow foundation may be utilized for the proposed structures provided that the near surface soils are adequately improved using aggregate piers. Aggregate piers consist of compacting columns of well-graded crushed rock to increase the bearing capacity of poor soils, to mitigate liquefaction potential within the improved zones, and to reduce settlements. Cementitious mixtures may be added to improve the stiffness of the pier. Because the aggregate piers increase the stiffness of the subsurface soils, and provide additional drainage pathways for excess pore water pressure during a seismic event, the potential for earthquake induced liquefaction in the improved soils is reduced.

After the aggregate piers are installed, conventional spread footings or a mat foundation is constructed directly on the improved soil. Aggregate piers should extend at least 40 feet below the existing surface such that a thick crust of relatively incompressible soils will be present below the foundation level. The actual depth of improvements should be determined by the aggregate pier designer, based on settlement criteria provided by the structural engineer.

Because specialty contractors install aggregate piers using a proprietary system, the contractor determines the lengths/depths and spacing of piers, the allowable soil bearing pressure of the improved soil, improved soil characteristics and anticipated settlements.

The aggregate pier contractors will base their design on the settlement criteria provided by the project owner and the project structural engineer.

6.1.2 Foundation Design Parameters

It is our opinion that either a mat foundation or a conventional footing system on aggregate piers will provide adequate support for the proposed buildings. A mat foundation, however, is anticipated to have better performance in terms of mitigating the risk of differential settlement, especially during a strong seismic event that is consistent with IBC. The performance of conventional footings may be improved by tying the individual footings together with concrete grade beams.

The footings and mat foundation should be sized using the following parameters:

- Allowable Bearing Pressure – 4,000 psf
- Allowable Friction Coefficient 0.40
- Allowable Passive Pressure – 350 pcf

These parameters may be increased by one-third for transient loads.

6.1.3 Buoyancy

Portions of the buildings such as elevator pits may be positioned below the groundwater table. Building elements extending below the groundwater table should be designed to resist the hydrostatic uplift pressure and the bending stress from the uplift pressure. The weight of the structure and friction along the sides of the structure will resist uplift forces. If needed, the base slabs of the below-grade structures may be extended outside its wall to increase its uplift resistance. For design purposes, we recommend that the groundwater level should be assumed to rise within 3 feet of the existing grade when calculating the hydrostatic uplift.

6.2 Retaining and Foundation Wall Design Parameters

Retaining walls should be properly designed to resist the lateral earth pressures exerted by the soils behind the wall. Adequate drainage provisions should also be provided behind the walls to intercept and remove groundwater that may be present behind the wall. Our geotechnical recommendations for the design and construction of new retaining walls are presented below.

Wall Foundation- For foundation walls supported on aggregate piers, the recommendations parameters outlined in Section 6.1.2 of this report remain applicable for retaining wall design and construction.

For site retaining walls, wall footings should be supported on at least two feet of granular structural fill such as crushed rock. An allowable bearing pressure of 2,000 psf may be used to size site retaining wall footings.

Lateral Earth Pressures – Foundation walls with level backslopes should be designed for a static at-rest lateral earth pressure based upon an equivalent fluid weight of 45 pcf. Cantilevered site retaining walls with level backslopes should be designed for a static active earth pressure based upon an equivalent fluid weight of 35 pcf. Walls retaining sloping backfills or surcharge loads should be designed for higher forces. PanGEO is available to provide additional recommendations if needed.

Buried structures such as elevator pits may extend below groundwater table and it is not feasible to incorporate footing drains for these structures. In this event, a lateral earth pressure of 90 pcf should be used to design walls of these structures. The recommended 90 pcf includes the effects of hydrostatic pressure. For design purposes, we recommend that the groundwater level should be assumed to rise within 3 feet of the existing grade.

In addition, permanent walls should be designed for an incremental uniform lateral pressure of 9H psf for seismic loading, where H corresponds to the retained height of the wall. The recommended lateral pressures assume that the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

Surcharge – Surcharge loads, where present, should be included in the design of retaining walls. We recommend that a lateral load coefficient of 0.3 be used to compute the lateral pressure on the wall face resulting from surcharge loads located within a horizontal distance of one-half wall height.

Wall Drainage – Provisions for wall drainage should consist of a rigid 4-inch diameter perforated drainpipe at the base of the wall footings. The drainpipe should be embedded in 12 to 18 inches of pea gravel. A minimum 12-inch wide layer of open-graded, free draining granular material (i.e. pea gravel or washed rock) is recommended adjacent to the wall for the full height of the wall. Alternatively, a composite drainage material, such as Miradrain 6000 may be used in lieu of open-graded, free draining granular material. The composite drainage material should be installed per the manufacturer's recommendations. The drainpipe at the base of the wall should be graded to direct water to a suitable outlet.

Wall Backfill – Given the relatively high fines content of the alluvial soils anticipated in site excavations, we do not recommend using the on-site soils for wall backfill. Imported granular soils such as Gravel Borrow (Section 9-03.14(1) WSDOT) are recommended for use as retaining wall backfill. In areas where the space is limited between the wall and the face of excavation, pea gravel may be used as backfill without compaction.

In structural areas, wall backfill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and relatively unyielding condition and to at least 95 percent of the maximum dry density, as determined using test method ASTM D1557. In landscaping areas and within 5 feet of the wall, the wall backfill should be compacted to at least 90 percent of its laboratory determined maximum dry density.

Damp Proofing – The exterior of all foundation walls should be protected with a damp proofing compound. Recommendations for damp proofing is beyond our area of expertise. A building envelope specialist or product vendors may be consulted for specific recommendations regarding this matter.

6.3 Concrete Slab-On-Grade Floors

It is our opinion that concrete slab-on-grade floors are appropriate for the site. However, a conventional on-grade floor slab may settle if subjected to earthquake shaking.

Existing undocumented fill is anticipated to be present at the slab subgrade elevation. The existing fill should be compacted in-place to a firm and unyielding condition or overexcavated to competent soil and replaced with Gravel Borrow. The exposed bottom of the overexcavation should be compacted to a dense and unyielding condition before placing the new structural fill. The adequacy of the floor subgrade should be evaluated by PanGEO during construction.

Concrete floors should be underlain by a capillary break consisting of at least of 4 inches of ¾-inch, clean crushed rock (less than 3 percent fines) compacted to a firm and unyielding condition. The capillary break should be placed on a suitable subgrade as confirmed by PanGEO. A minimum 10-mil polyethylene vapor barrier should also be placed directly below the slab. We also recommend that control joints be incorporated into the floor slab to control cracking.

6.4 Pavement

New asphalt pavement will be constructed as part of the proposed development. Because the site soils are prone to settlement, we recommend that the fill soil in the pavement areas be placed as early in the project as possible, and allowed to settle prior to final grading and pavement construction.

Assuming the pavement will generally be used by light passenger cars and trucks, with only occasional heavy truck, bus, or garbage truck use, as a minimum, we recommend that the new pavement section consist of 4-inches HMA, overlying a 6-inch thick layer of crushed surfacing base course (CSBC), overlying a minimum of 12 inches of properly compacted granular structural fill. Both the structural fill and crushed rock base should be compacted to a minimum of 95% of the materials maximum dry density (Modified Proctor ASTM D-1557). It should be noted that actual pavement performance will depend on a number of factors, including the actual traffic loading conditions. The recommended pavement section will need to be revised if the traffic level will be more or less than our assumed value.

6.5 UNDERGROUND UTILITIES

6.5.1 Pipe Support and Bedding

Based on our field explorations, we anticipate the exposure of variable, but generally adequate subsoil conditions at pipe invert elevations less than about 4 feet below the existing ground surface. Below about 4 feet, soft silt, clay, and some organics were encountered. In our opinion, the relatively undisturbed silty sand and sands should provide suitable support for the proposed pipelines; however, for utilities deeper than about 4 feet, if soft silt, clay, or organic-rich soil is exposed along the bottom of any trench, we recommend about 6 to 12 inches of the soft soils be removed and replaced with additional bedding material.

In general, pipe bedding materials should be placed in loose lifts not exceeding 6 inches in thickness, and compacted to a minimum relative compaction of 95 percent maximum dry density, per ASTM D1557. Bedding materials and thicknesses provided should be suitable for the utility system and materials installed, and in accordance with any applicable manufacturers' recommendations. Pipe bedding materials should be placed on relatively undisturbed native soil, or compacted structural fill soils. If the native subgrade soils are disturbed, the disturbed material should be removed and replaced with compacted structural fill or bedding material.

6.5.2 Trench Backfill

Beneath structural or paved areas, we recommend that trench backfill be select granular material, meeting the requirements for structural fill. During placement of the initial lifts, the trench backfill material should not be bulldozed into the trench or dropped directly on the pipe. Furthermore, heavy vibratory equipment should not be permitted to operate directly over the pipe until a minimum of 3 feet of backfill has been placed.

In order to minimize subsequent settlement of the trench backfill, it is recommended that the trench backfill be placed in 8- to 12-inch, loose lifts and compacted using mechanical equipment to about 90 percent maximum dry density, as determined by Standard Proctor (per ASTM D698). In structural or paved areas, the upper 2 feet of the backfill should be compacted to at least 95 percent maximum dry density, per ASTM D1557.

It is anticipated that selected excavation spoils may be used as trench backfill if they are placed at or near optimum content and proper compaction control is utilized. In our opinion, the top approximately 3 to 6 feet of soil at the site (sand and silty sand) may be potentially re-used as trench backfill. However, some of the soils may be too wet to achieve the recommended compaction requirements. If the material is not compacted as recommended, the potential for backfill settlement will be increased. Below a depth of about 3 to 4 feet, the silt and clay will not be suitable for re-use as trench backfill.

Underground utilities should be designed to accommodate differential and total settlements on the order of several inches over the design life of the project.

6.6 PERMANENT CUT AND FILL SLOPES

We recommend that the permanent cut and fill slopes be constructed no steeper than 2H:1V (horizontal:vertical). For fill slopes constructed at the angles recommended above, and comprised of fill soils placed and compacted as recommended in this report, we anticipate that adequate factors of safety against global failure will be maintained.

Measures should be taken to prevent surficial instability and/or erosion on slopes. For a permanent fill slope, this can be accomplished by conscientious compaction of the fills all the way out to the slope face, by maintaining adequate drainage, and planting the slope face as soon as possible after construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to design finish grade. In our experience, compaction of slope faces by “track-walking” is generally not as effective.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 SITE PREPARATION

Site preparation includes striping and clearing of topsoil and sod, surface vegetation, root balls, existing foundations and pavements, and any other deleterious materials within the proposed development areas, and excavating to the design subgrade.

All stripped materials should be properly disposed off-site or be “wasted” on site in non-structural landscaping areas. Soil disturbed during stripping and clearing activities should be compacted to a firm and unyielding condition.

In areas where existing foundations, slab-on-grade floors, and pavements are removed, it may be possible to crush the existing materials for use as structural fill. Materials reclaimed by crushing and used as fill should have a maximum particle size of four inches and should be mixed with soil to provide a well-graded material.

Following the removal of deleterious and unsuitable materials, the exposed subgrade within the development area, such as building foundation, slab, and pavement areas, should be proof-rolled with a fully loaded dump truck or a smooth roller compactor. The proof-rolling operation should be observed by a representative of PanGEO. If loose or unstable subgrade soils are observed during the proof roll, the soil should be overexcavated and replaced with structural fill.

7.2 Material Reuse

Based on our CPTs and review of the previous nearby test borings, the top 3 to 6 feet of on-site soils consist of silty to relatively clean sand with gravel (fill) that may be suitable for use as structural fill at the site. Below the 3- to 6-foot thick layer of granular fill, the site soils consist of soft, wet, silt and clay with some sand, that will likely not be suitable as structural fill.

The re-use of on-site materials as structural fill may be possible only if the materials are properly handled and can be compacted to the required density. The re-use of the on-site soils during wet times of the year will be more difficult or impossible. If use of the on-site soils is planned, any excavated soil should be stockpiled and protected with plastic sheeting to prevent softening from rainfall.

7.3 Structural Fill and Compaction

We recommend using a granular fill material such as Gravel Borrow (WSDOT 9-03.14(1)), Seattle Type 17 mineral aggregate, or another approved equivalent. The structural fill should be moisture conditioned to within about 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and systematically compacted to a dense and unyielding condition, and to at least 95 percent of the maximum dry density, as determined using test method ASTM D 1557.

7.4 Temporary Excavation and Dewatering

Temporary excavations greater than 4 feet deep should be properly sloped or shored. All temporary excavations should be performed in accordance with Part N of WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. For planning purposes, the temporary excavations in the upper fill and soft to medium stiff silt and clay may be sloped to as steep as 1.5H:1V (Horizontal:Vertical). To stabilize the toe of excavation slopes below the groundwater level, such as elevator pit excavations, the soils at the toe of the slope may need to be replaced with angular rock such as 2- to 4-inch quarry spalls. A sheet of geotextile separator should be placed below the quarry spalls to prevent the native fine sand and silt from migrating into the spalls. The temporary cut slopes should be re-evaluated by a representative of PanGEO during construction based on actual observed soil conditions.

We anticipate excavations deeper than about 5 feet below grade to encounter groundwater seepage. The contractor should be prepared to provide a temporary dewatering system for the excavations. Due to the generally fine-grained nature of the Unit 2 deposits (Lacustrine/Alluvium), we anticipate that groundwater seepage inflow will be relatively slow and sumps and pumps will likely be adequate for controlling the groundwater seepage. The spacing of the sumps should be determined by the contractor during construction based on field observations at the time of construction.

7.5 Erosion and Drainage Considerations

Surface runoff can be controlled during construction by careful grading practices. This may include the construction of shallow, upgrade perimeter ditches or low earthen berms to collect runoff and prevent water from entering the excavation. All collected water should be directed to a positive and permanent discharge system such as a storm sewer. It should be noted that some of the site soils are prone to surficial erosion. Special care should be taken to prevent surface water from flowing over open cut excavations, and exposed slopes should be protected with plastic sheeting.

Permanent control of surface water and roof runoff should be incorporated in the final grading design. In addition to these sources, irrigation and rain water infiltrating into any landscape and/or planter areas adjacent to paved areas or building foundations should also be controlled. Water should not be allowed to pond immediately adjacent to buildings or paved areas. All collected runoff should be directed into conduits that carry the water away from pavements or the structure and into storm drain systems or other appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from structures.

7.6 Wet Weather Earthwork and Erosion Considerations

The site soils contain a moderate to high amount of fines, and are therefore considered moisture sensitive. As a result, it may be more economical to perform earthwork in the drier summer months to reduce the potential of site soils becoming soft due to excessive moisture. Any softened soils should be removed and replaced with structural fill.

General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below:

- Because site soils are considered moisture sensitive, all subgrade surfaces should be protected against inclement weather.
- Earthwork may need to be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of structural fill. The size and type of construction equipment used may have to be limited to reduce soil disturbance.
- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing ¾-inch sieve. The fines should be non-plastic.
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water, and to prevent surface water from entering the excavations.
- Bales of straw and/or geotextile silt fences should be strategically located to control erosion and the movement of sediment. Erosion control measures should be installed along all the property boundaries.
- Excavation slopes and soils stockpiled on site should be covered with plastic sheeting.

- Under no circumstances should soil be left uncompacted and exposed to moisture.

8.0 LIMITATIONS

We have prepared this report for use by the King County Housing Authority and the project team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as being preventative of mold development. A mold specialist should be consulted for all mold-related issues.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the Report or its contents were prepared. No warranty, express or implied, is made.

We appreciate the opportunity to be of service to you on this project. Please feel free to contact our office with any questions you have regarding our study, this report, or any geotechnical engineering related project issues.

Sincerely,

PanGEO, Inc.

(Draft)

Steven T. Swenson, L.G.
Project Geologist

(Draft)

Siew L. Tan, P.E.
Principal Geotechnical Engineer

9.0 REFERENCES

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International Building Code (IBC), 2015, International Code Council.

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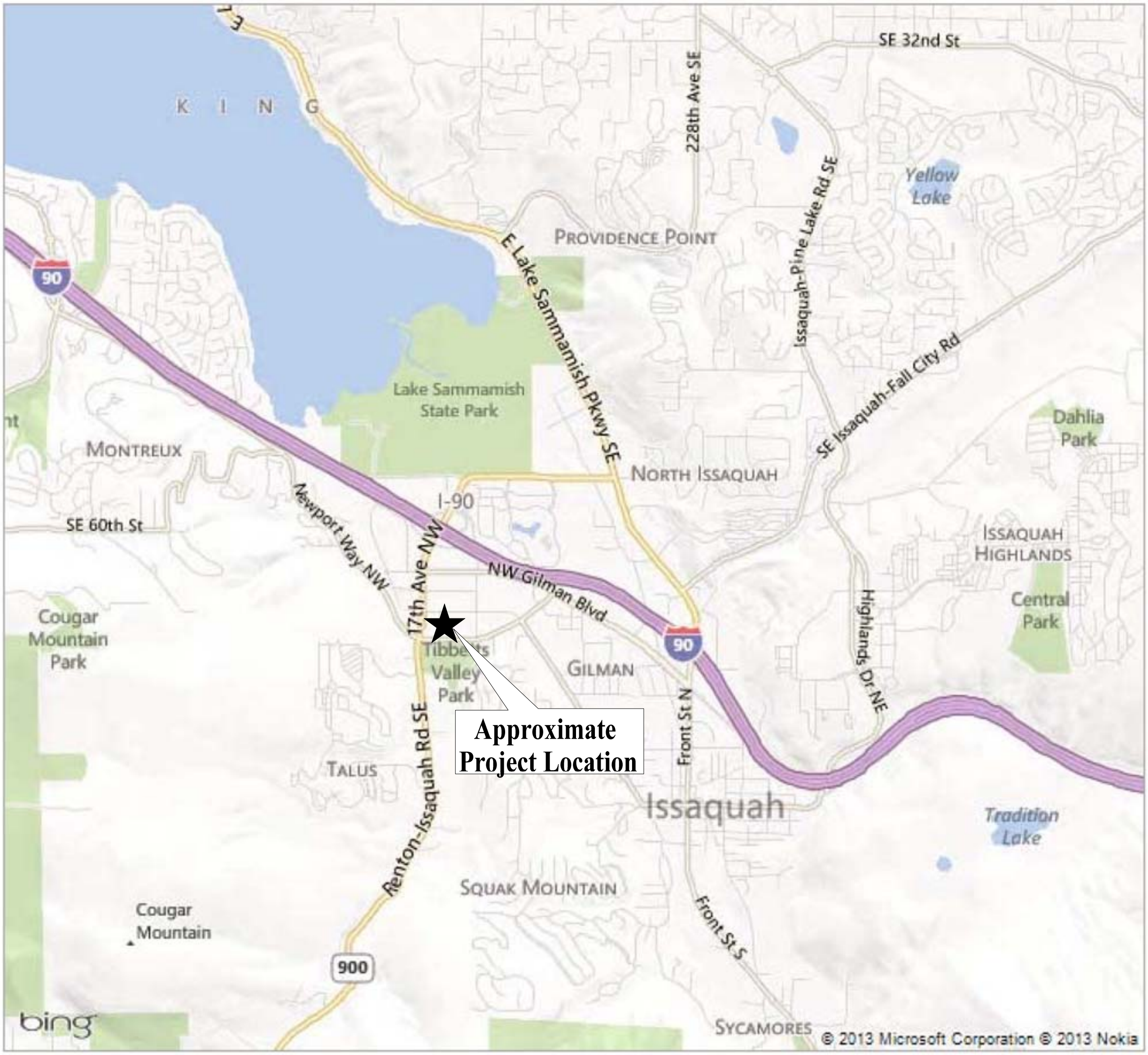
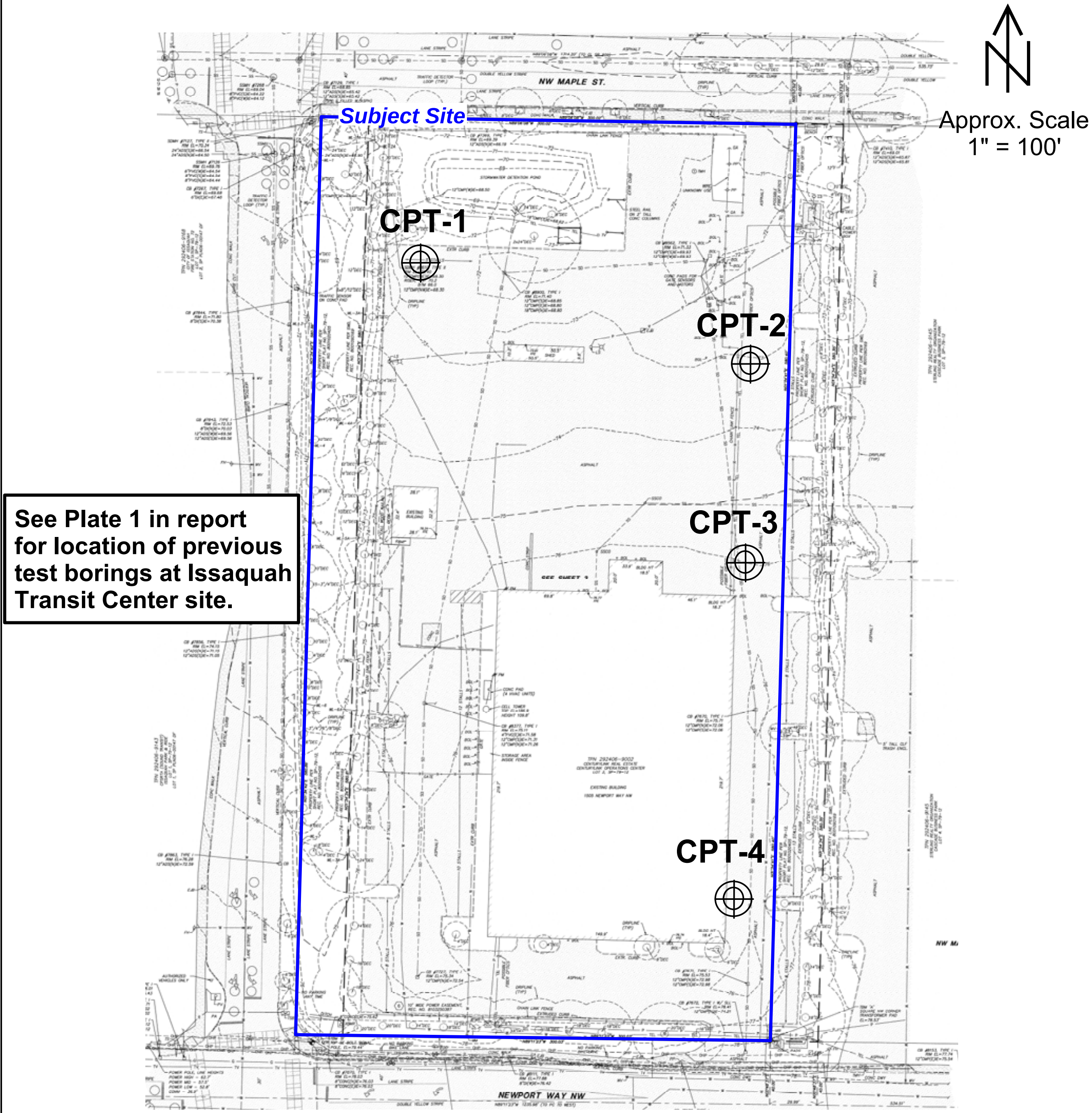
Zipper Zeman Associates, Inc. (2005), *Boring Logs B-4, B-6, B-8, B-13, B-14, and B-15, Report of Geotechnical Investigation, Sound Transit Regional Express, Issaquah Transit Center, Issaquah, Washington, project number J-1875, June 24, 2005.*



Scale
1 inch = 4,000 feet
(Approximate)

17-296 Fig 1 Vicinity Map.grf 2/20/18 (11:18)

PanGEO INCORPORATED	Issaquah TOD 1505 Newport Way NW Issaquah, Washington	VICINITY MAP	
		Project No. 17-296	Figure No. 1



Legend:

Approx. CPT Location (PanGEO 2018)

Note: Base modified from ALTA survey prepared by Triad (dated 7/27/2017).

PanGEO INCORPORATED

Issaquah TOD
1505 Newport Way NW
Issaquah, Washington

SITE AND EXPLORATION PLAN

Project No.	Figure No.
17-296	2

17-296 Fig 2 Site Plan.grf 1/19/18 (08:37) STS

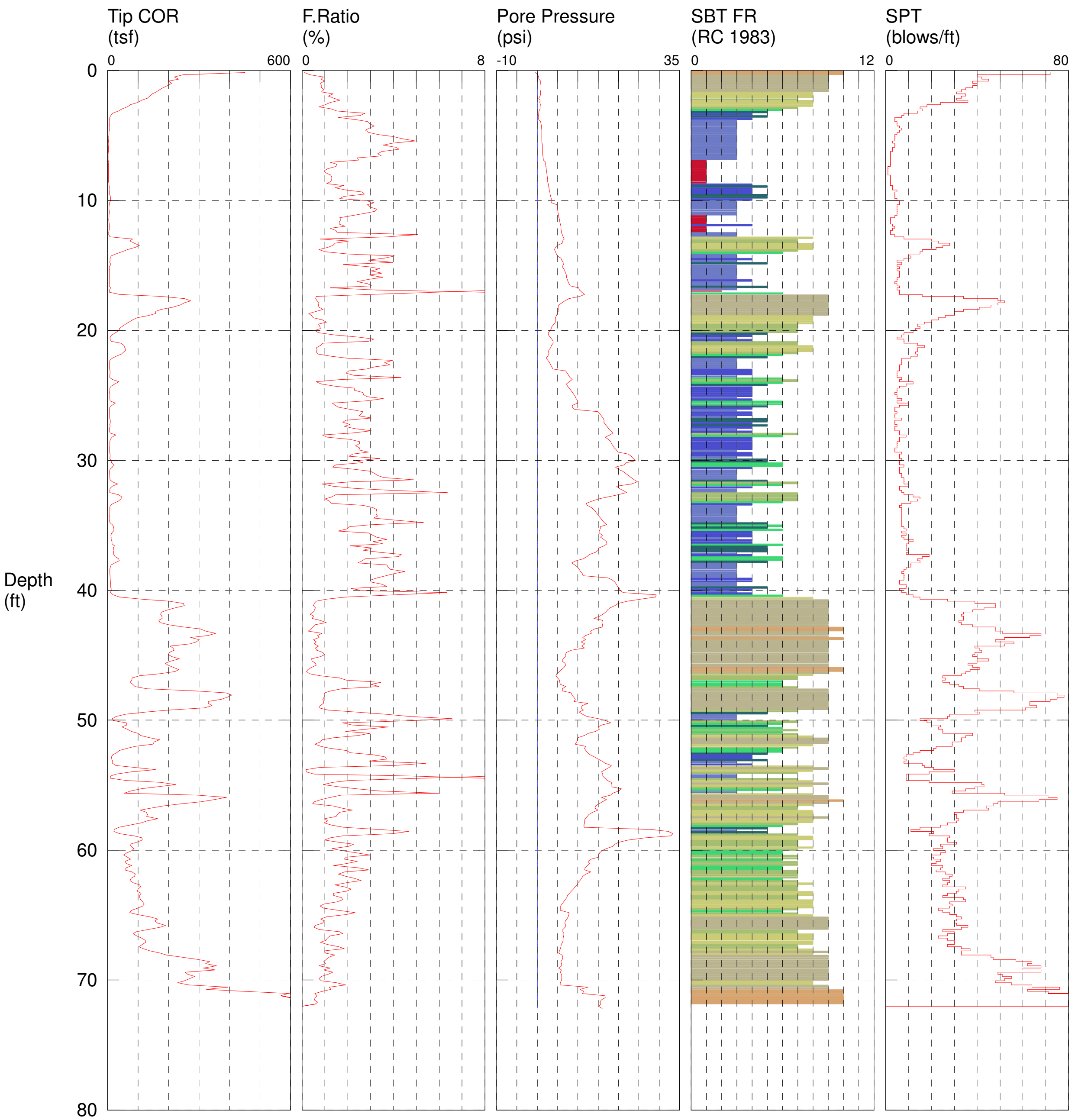
APPENDIX A

CONE PENETRATION TEST LOGS

CPT-01

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 10:42:29 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

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

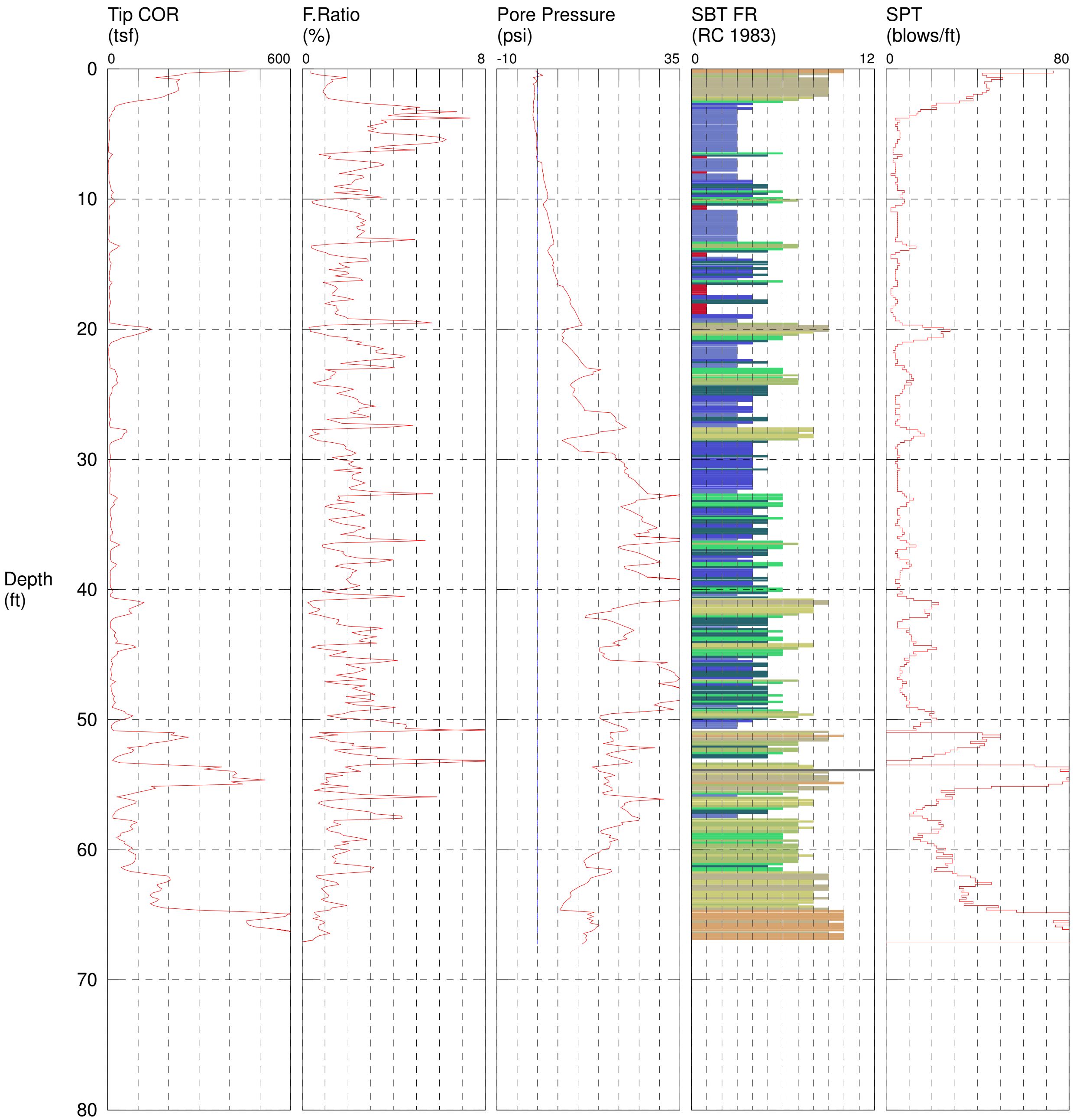
*SBT/SPT CORRELATION: UBC-1983

Figure A-1

CPT-02

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 9:45:30 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

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

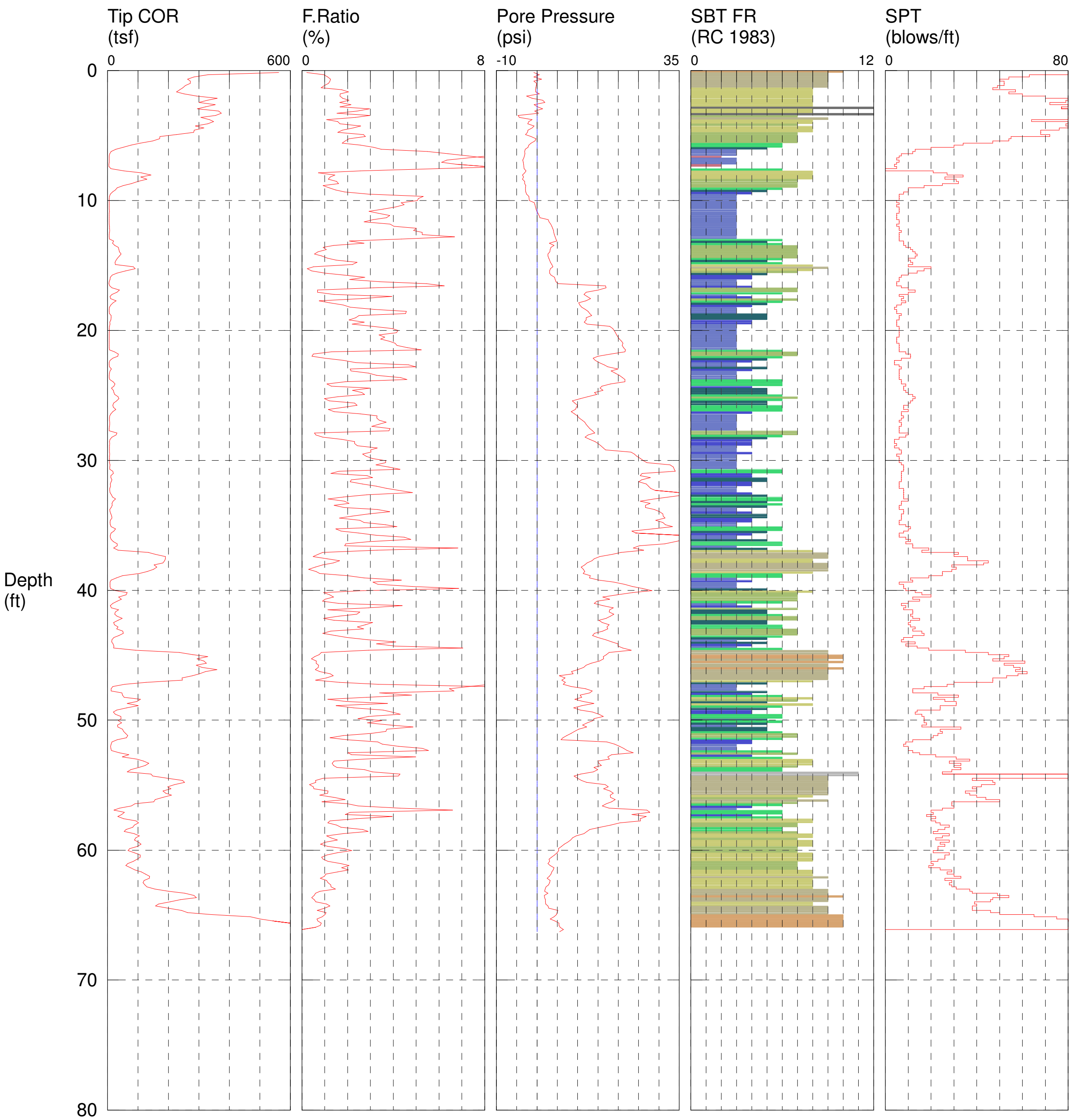
*SBT/SPT CORRELATION: UBC-1983

Figure A-2

CPT-03

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 8:34:46 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

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

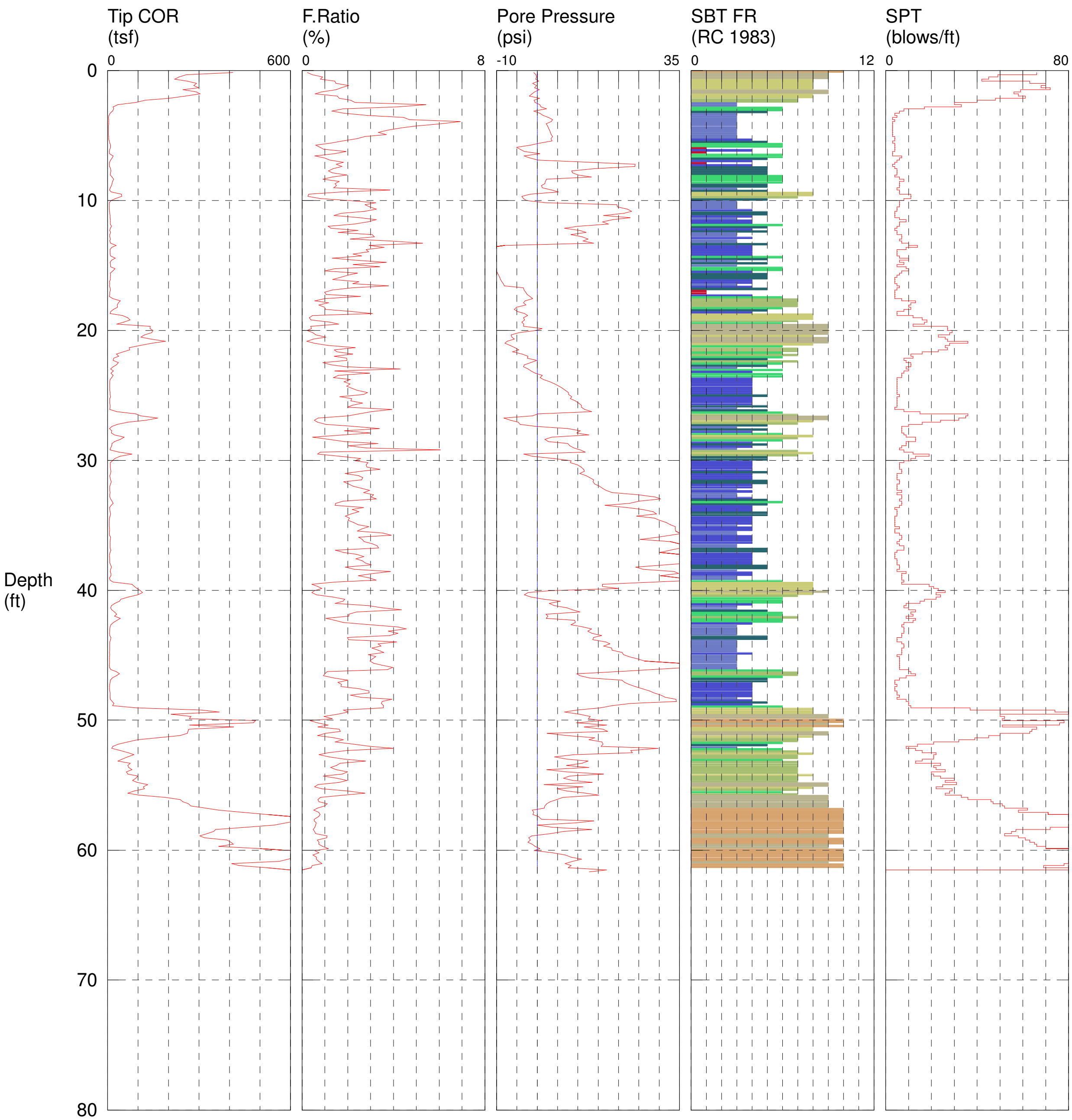
*SBT/SPT CORRELATION: UBC-1983

Figure A-3

CPT-04

CPT CONTRACTOR: InSitu Engineering
CUSTOMER: PanGEO
LOCATION: Issaquah
JOB NUMBER: 17-296

OPERATOR: Romanelli
CONE ID: DDG1424
TEST DATE: 1/2/2018 7:14:16 AM
PREDRILL: N/A
BACKFILL: 20% Bentonite Grout
SURFACE PATCH: Granular Bentonite Chip



COMMENT:

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

*SBT/SPT CORRELATION: UBC-1983

Figure A-4

APPENDIX B

PREVIOUS SUBSURFACE INVESTIGATIONS

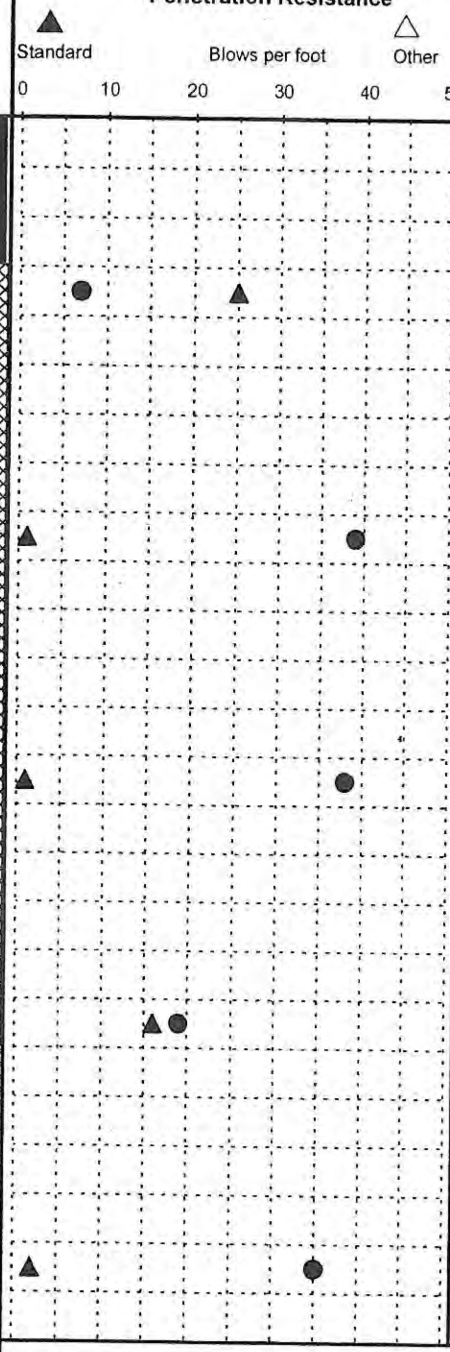
Borings B-4, B-6, B-8, B-13, B-14, and B-15, Issaquah Transit Center (ZZA, 2005)

PROJECT: Issaquah Transit Center | JOB NO.: J-1875 | BORING: B-4 | PAGE 1 OF 3

Location: Issaquah, Washington | Approximate Elevation: 74 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	4 inches asphalt					
	Medium dense, moist, brown-gray, silty, gravelly, SAND (fill)	I	S-1		25	
5	Very soft, saturated, black, sandy, organic SILT			7/7/04		
	Very soft, staurated, gray with orange, sandy, clayey SILT with trace fine organic material	I	S-2		1	
10						
	Grades to gray, sandy SILT with some clay and trace fine organic material	I	S-3		1	
15	Medium dense, saturated, gray, silty, sandy GRAVEL with trace organics			ATD		
		I	S-4		16	
20						
	Very soft, saturated, gray, clayey SILT with trace fine organic material	I	S-5		2	
25						

Penetration Resistance



Explanation

- 2-inch O.D. split spoon sample 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG | Figure A-4

Date Drilled: 6/23/2004 | Logged By: KTH

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-4	PAGE 2 OF 3
Location: Issaquah, Washington	Approximate Elevation: 74 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Standard ▲ / Other △, Blows per foot 0–50)	N-values	Testing
25							
	Grades to soft, sandy SILT with some gravel and trace clay and fine organic material	I	S-6		▲	5	
30							
	Grades to very soft, gray-black, sandy SILT with some clay and trace fine organic material	I	S-7		▲	2	
35							
	Grades to medium stiff, sandy SILT with 1/8-inch thick organic layers	I	S-8		▲	6	
40	Loose, saturated, gray, gravelly SAND with trace silt						
	Very soft, saturated, dark gray, sandy SILT with organics and trace gravel	I	S-9		▲	2	
45	Medium dense, saturated, gray- black, organic, silty, fine SAND with trace gravel						
		I	S-10		▲	12	
50							

Explanation

- I — 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- I — 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-4	PAGE 3 OF 3
Location: Issaquah, Washington	Approximate Elevation: 74 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Standard ▲ / Other △, Blows per foot 0–50)	N-values	Testing
50				Bentonite			
~53	Medium dense,saturated, dark gray, sandy, silty GRAVEL with trace organics	2-inch O.D. split spoon	S-11	Bentonite	▲ 33	33	
55				Bentonite			
~58		3-inch O.D. Dames & Moore	S-12	Bentonite	▲ 23	23	
60				Bentonite			
~63		2-inch O.D. split spoon	S-13	Bentonite	▲ 29	29	
65				Bentonite			
~68	Grades to silty, sandy GRAVEL	2-inch O.D. split spoon	S-14	Bentonite	▲ 38	38	
70				Bentonite			
~71	(2' heave)	2-inch O.D. split spoon	S-15	Bentonite	▲ 21	21	
75	Boring completed at 74 feet on 6/23/04 Groundwater observed at 15 feet at time of drilling Groundwater measured at 5.4 feet on 7/7/04						

Explanation

- I — 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- I (solid) — 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

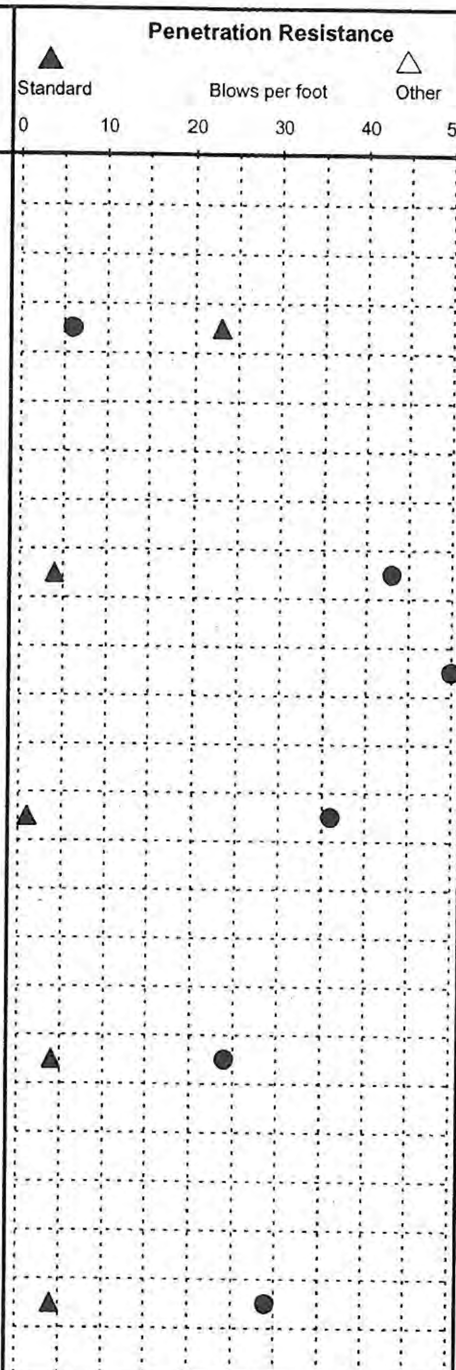
Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

ZZA Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-4
	Date Drilled: 6/23/2004	Logged By: KTH

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-6	PAGE 1 OF 3
Location: Issaquah, Washington	Approximate Elevation: 79 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	4 inches asphalt					
	Medium dense, moist, brown, silty, gravelly SAND (Fill)					
		2-inch O.D. split spoon	S-1		23	
5						
	Soft, wet, black, sandy, organic SILT			ATD		
	Soft, saturated, brown-gray, clayey SILT with some sand and trace organics and gravel					
		2-inch O.D. split spoon	S-2		4	
10		3-inch I.D Shelby tube	S-3			Con. Att.
	Grades to very soft, wet to saturated, orange-gray, sandy SILT with some clay and with trace fine organic material					
	Very soft, saturated, gray, sandy SILT to silty SAND with trace organics	2-inch O.D. split spoon	S-4		1	200W
15						
		2-inch O.D. split spoon	S-5		4	
20						
	Grades to soft, clayey SILT with some sand and fine organic material	2-inch O.D. split spoon	S-6		4	
25						



Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- No Recovery
- ATD: Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-6
	Date Drilled: 6/24/2004	Logged By: KTH

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-6	PAGE 2 OF 3
Location: Issaquah, Washington	Approximate Elevation: 79 Feet		

Penetration Resistance: ▲ Standard, △ Other; Blows per foot: 0, 10, 20, 30, 40, 50

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
25						
	Very loose, saturated, dark gray, clayey, fine SAND with gravel lenses	I	S-7		0	200W
30						
	Grades to fine to medium SAND with trace organics	I	S-8		4	
35						
	Medium stiff, wet to saturated, dark gray, sandy SILT with 1/8-inch thick organic lenses and medium sand lens with trace gravel	I	S-9		5	
40						
	Medium dense, saturated, gray, sandy GRAVEL	I	S-10		22	
45						
	Grades to silty, gravelly, SAND with 2-inch thick silt lenses	I	S-11		26	
50						

Explanation

- 2-inch O.D. split spoon sample 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test

ZZA Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-6
	Date Drilled: 6/24/2004	Logged By: KTH

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-6	PAGE 3 OF 3
Location: Issaquah, Washington	Approximate Elevation: 79 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (▲ Standard / △ Other; Blows per foot 0–50)	N-values	Testing
50							
53	Grades to dense silty, sandy GRAVEL (7 ' of heave flushed from augers)	2-inch O.D. split spoon	S-12		▲	40	
55							
58	Grades to medium dense, silty, gravelly SAND	2-inch O.D. split spoon	S-13		▲	26	
60							
63	Grades to sandy GRAVEL with some silt	3-inch O.D. Dames & Moore	S-14		▲	27	
65							
68	Grades to very dense, gravelly SAND with trace silt	2-inch O.D. split spoon	S-15		▲	74	
69	Boring completed at 69 feet on 6/24/04 Groundwater observed at 7 feet at time of drilling						
70							
75							

Explanation

- 2-inch O.D. split spoon sample
  140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample
  300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-6
	Date Drilled: 6/24/2004	Logged By: KTH

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-8	PAGE 1 OF 3
Location: Issaquah, Washington	Approximate Elevation: 77 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Blows per foot)	N-values	Testing
0	3 inches asphalt						
	Medium dense, moist, brown, sandy GRAVEL with some silt (Fill)	2-inch O.D. split spoon	S-1			25	GSA
5	Soft, wet, black, sandy, organic SILT	3-inch O.D. Dames & Moore	S-2			3	
	Very soft, wet, brown-gray, clayey SILT with trace to some sand and organics	2-inch O.D. split spoon	S-3			0	
10							
	Grades to saturated, gray, sandy, clayey SILT with trace organics	2-inch O.D. split spoon	S-4	▼ ATD		0	
15							
	Medium dense, saturated, gray, silty, sandy GRAVEL	2-inch O.D. split spoon	S-5			18	
20							
	Grades to sandy GRAVEL with trace silt	2-inch O.D. split spoon	S-6			17	
25							

Penetration Resistance: ▲ Standard, △ Other; Blows per foot 0, 10, 20, 30, 40, 50

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

PROJECT: Issaquah Transit Center **JOB NO.:** J-1875 **BORING:** B-8 **PAGE 2 OF 3**

Location: Issaquah, Washington **Approximate Elevation:** 77 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Standard ▲ / Other △, Blows per foot 0–50)	N-values	Testing
25							
	Soft, saturated, gray, sandy SILT with some clay and organics	I	S-7		▲	4	
30							
	Loose, saturated, dark gray, clayey, fine SAND with gravel and organics	I	S-8		▲	7	
35							
	Soft, saturated, gray, sandy SILT with organics	I	S-9		▲	5	
40							
	Very loose, saturated, gray, silty, gravelly, fine to medium SAND with organics	I	S-10		▲	2	
45							
	Dense, saturated, gray, silty, sandy GRAVEL to gravelly SAND	I	S-11		▲	31	
50							

Explanation

- I 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-8
	Date Drilled: 6/25/2004	Logged By: KTH

Location: Issaquah, Washington | **Approximate Elevation:** 77 Feet

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Blows per foot)	N-values	Testing
50							
~53	Grades to sandy GRAVEL with trace silt	2-inch O.D. split spoon	S-12		40	40	
55							
~58	Grades to medium dense, gravelly SAND with trace silt and organics	3-inch O.D. Dames & Moore	S-13		26	26	
59	Boring completed at 59 feet on 06/25/04 Groundwater observed at 14 feet at time of drilling						
60							
65							
70							
75							

Penetration Resistance: ▲ Standard, △ Other; Blows per foot 0, 10, 20, 30, 40, 50

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD: Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test

PROJECT: Issaquah Transit Center | **JOB NO.:** J-1875 | **BORING:** B-13 | **PAGE 1 OF 1**

Location: Issaquah, Washington | **Approximate Elevation:** 69.5 Feet

Penetration Resistance: ▲ Standard, △ Other — Blows per foot (0, 10, 20, 30, 40, 50)

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	N-values	Testing
0	Loose, moist, brown, silty SAND. (Fill)					
	Grades to medium dense, damp, brown, gravelly SAND.	I	S-1		16	
5	Very loose, wet, mottled gray, silty SAND with some clay.	I	S-2		3	
10	Very soft, moist, gray, silty CLAY. (PP = 0.0-0.25 tsf)	I	S-3		2	
15	Grades to silty CLAY to sandy, silty CLAY (PP = 0.0 tsf)	I	S-4		2	
	Very loose, moist to wet, gray, silty SAND.					
20	Medium dense, saturated, gray, sandy GRAVEL.	I	S-5	▼ ATD	12	
	Boring completed at 21.5 feet on 5/15/05. Groundwater observed at 20.0 feet at time of drilling					
25						

Explanation

- I — 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- I — 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- II — 3-inch I.D Shelby tube sample
- ⊗ — No Recovery
- ▼ ATD — Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test
- PP = Pocket Penetrometer

ZZA Zipper Zeman Associates, Inc.
Geotechnical and Environmental Consulting

BORING LOG	Figure A-13
Date Drilled: 5/15/2005	Logged By: BAG

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-14	PAGE 1 OF 1
Location: Issaquah, Washington	Approximate Elevation: 79 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Blows per foot; ▲ Standard, △ Other; 0 10 20 30 40 50)	N-values	Testing
0	4 inches asphalt over loose to medium dense, damp, brown, sandy GRAVEL with trace silt. (Fill)						
	Grades to very dense.	I	S-1		▲	59	
5	(gravelly drilling action)	⊗	S-2		▲	6	
	Very loose, moist to wet, mottled gray-brown, fine to medium silty SAND with 2 inch thick lense of sandy, clayey SILT.						
10		I	S-3		▲	3	
15	Grades to loose, saturated, gray, clayey SAND with 2 inch thick sandy, clayey silt lense. (PP = 0.0 tsf)	I	S-4	▼ ATD	▲	5	
20	Grades to very loose, saturated, gray, silty SAND. Very soft, saturated, gray, sandy, clayey SILT. Very loose, saturated, gray, silty SAND.	I	S-5		▲	3	
	Boring completed at 21.5 feet on 5/15/05. Groundwater observed at 15.0 feet at time of drilling.						
25							

Explanation

- 2-inch O.D. split spoon sample, 140 lb hammer with 30-inch free fall
- 3-inch O.D. Dames & Moore sample, 300 lb hammer with 30-inch free fall
- 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

- GSA = Grain Size Analysis
- 200W = 200 Wash Analysis
- Att. = Atterberg Limits
- Con. = Consolidation Test
- PP = Pocket Penetrometer

ZZA Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-14
	Date Drilled: 5/15/2005	Logged By: BAG

PROJECT: Issaquah Transit Center	JOB NO.: J-1875	BORING: B-15	PAGE 1 OF 1
Location: Issaquah, Washington	Approximate Elevation: 70 Feet		

Depth (ft)	Soil Description	Sample Type	Sample Number	Ground Water	Penetration Resistance (Standard ▲ / Other △, Blows per foot, 0–50)	N-values	Testing
0	4-3/4 inches concrete over loose to medium dense, damp, brown, sandy GRAVEL with trace silt. (Fill)						
	Grades to medium dense, moist, brown, gravelly SAND.	I	S-1		▲	16	
5	Very loose, wet, mottled gray, silty SAND with sandy, clayey SILT interbeds. (PP - 0.0 tsf)	I	S-2		▲	3	
10	Grades to medium dense, saturated, gray, gravelly SAND with some silt and a 6" thick clayey SILT interbed. (PP = 0.0 tsf)	I	S-3	▼ ATD	▲	14	
15	Grades to very loose silty SAND.	I	S-4		▲	3	
20	Grades to silty SAND interbedded with very soft, saturated, gray, sandy SILT. (PP = 0.0 tsf)	I	S-5		▲	3	
	Boring completed at 21.5 feet on 5/15/05. Groundwater observed at 10.0 feet at time of driling.						
25							

Explanation

- I 2-inch O.D. split spoon sample 140 lb hammer with 30-inch free fall
- I 3-inch O.D. Dames & Moore sample 300 lb hammer with 30-inch free fall
- II 3-inch I.D Shelby tube sample
- ⊗ No Recovery
- ▼ ATD Groundwater level at time of drilling or date of measurement

Monitoring Well Key

- Clean Sand
- Bentonite
- Grout/Concrete
- Screened Casing
- Blank Casing

Moisture Content

Plastic Limit — Natural — Liquid Limit

Testing Key

GSA = Grain Size Analysis
200W = 200 Wash Analysis
Att. = Atterberg Limits
Con. = Consolidation Test
PP = Pocket Penetrometer

ZZA Zipper Zeman Associates, Inc. Geotechnical and Environmental Consulting	BORING LOG	Figure A-15
	Date Drilled: 5/15/2005	Logged By: BAG