GEOTECHNICAL REPORT

Avalon Issaquah
1040 – 12th Avenue Northwest
Issaquah, Washington

Project No. T-8488

Terra Associates, Inc.

Prepared for:

Avalon Bay Communities
Bellevue, Washington

January 14, 2022

TERRA ASSOCIATES, Inc.
Consultants in Geotechnical Engineering, Geology
and
Environmental Earth Sciences

January 14, 2022
Project No. T-8488

Mr. Carl Shorett
Avalon Bay Communities
10885 Northeast 4th Street, Suite 500
Bellevue, Washington 98004

Subject: Geotechnical Report
Avalon Issaquah
1040 – 12th Avenue Northwest
Issaquah, Washington

Dear Mr. Shorett:

As requested, we have conducted a geotechnical engineering study for the subject project. The attached report presents our findings and recommendations for the geotechnical aspects of project design and construction.

Our field exploration indicates the site soils underlying approximately two to four inches of asphalt consisted of approximately two feet of medium dense to dense fill soils consisting of silty sand with gravel overlying very soft to medium stiff alluvial silts with varying sand, gravel, and clay contents with interbedded sands to silty sands to depths of approximately 25 to 60 feet below existing site grades. The upper silts are underlain by medium dense to very dense sands and gravels with varying amounts of silt and interbedded silt layers to the termination of the test boring and CPT termination depths. Groundwater was observed in all the test borings and CPTs at a depth of approximately seven feet below existing site grades.

Soil conditions observed will be suitable for support of the proposed structures provided recommendations contained herein are incorporated into building design and construction. The primary geotechnical concern would be the presence of the very soft to medium stiff, compressible silt layers and very loose to loose silty sand material typically observed in the upper 25 to 60 feet of the site. This material would not be suitable for support of the building foundation or floor slabs. Due to heavy building loads, a preload or surcharge would not be effective. Therefore, we recommend supporting the building on augercast piles.

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Mr. Carl Shorett
January 14, 2022

We trust the information presented in this report is sufficient for your current needs. If you have any questions or require additional information, please call.

Sincerely yours,
TERRA ASSOCIATES, INC.

Michael J. Xenos E.I.T.
Staff Engineer

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President

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1.0 PROJECT DESCRIPTION

The project consists of redeveloping the site with a seven-story multi-family building over two levels of below-grade structured parking, associated access, utility improvements, and surface parking. With two levels of below-grade parking, excavation depths of 20 to 25 feet are expected to achieve lower floor, foundation, and elevator pit levels. The below grade excavation will be stepped, so that adjacent the building property lines, the building will extend one level below grade, and interior to the site the building will extend two levels below grade.

As we understand the first two to three levels of the building would be concrete construction, with the upper five floors consisting of wood-framed construction. We expect foundation loading for the structure will be moderate, with isolated columns carrying loads of 500 to 800 kips, and continuous bearing walls carrying 5 to 8 kips per foot.

The recommendations in the following sections of this report are based on our understanding of the preceding design features. We should review design drawings as they become available to verify our recommendations have been properly interpreted and to supplement them, if required.

2.0 SCOPE OF WORK

Our work was completed in accordance with our authorized proposal dated February 12, 2021. Accordingly, on March 1, 2021, four Cone Penetration Tests (CPTs) were advanced to depths ranging from approximately 25 to 60 feet below existing grades. To supplement this data, on March 10, 2021, we observed soil conditions at three test borings drilled to depths of 50 feet below existing grade. Using the information obtained from the subsurface explorations, we performed analyses to develop geotechnical recommendations for project design and construction. Specifically, this report addresses the following:

- Soil and groundwater conditions.
- Geologic Hazards per the City of Issaquah Municipal Code.
- Seismic design parameters per the current International Building Code (IBC).
- Site preparation and grading.
- Excavation, shoring, and dewatering.
- Foundation support.
- Slab-on-grade floor support.
- Lateral earth pressures for below-grade wall design.
- Drainage.
- Utilities.
- Pavements.

It should be noted, recommendations outlined in this report regarding drainage are associated with soil strength, design earth pressures, erosion, and stability. Design and performance issues with respect to moisture as it relates to the structural environment are beyond Terra Associates, Inc.'s purview. A building envelope specialist or contractor should be consulted to address these issues, as needed.

3.0 SITE CONDITIONS

3.1 Surface

The project site consists of a single tax parcel totaling approximately 4.05 acres located at 1040 – 12th Avenue Northwest in Issaquah, Washington. The approximate site location is shown on Figure 1.

The site is currently developed with a single-story masonry building, along with paved access and parking. The site is bordered by a hotel and office space to the north and east, Newport Way Northwest to the south, and 12th Avenue Northwest to the west. Site topography consists of a gentle slope that descends from the southwest to the northeast, with an overall relief of approximately ten feet.

3.2 Subsurface

In general, the soil conditions observed underlying approximately two to four inches of asphalt consisted of approximately two feet of medium dense to dense fill soils consisting of silty sand with gravel overlying very soft to medium stiff alluvial silts with varying sand, gravel, and clay contents with interbedded sands to silty sands to depths of approximately 25 to 60 feet below existing site grades. The upper silts are underlain by medium dense to very dense sands and gravels with varying amounts of silt and interbedded silt layers to the termination of the test borings and to the CPT termination depths. The exceptions to this general condition were observed in Test Boring B-2 where the sands encountered in the lower 30 feet of the boring were in a loose consistency, and at Test Boring B-3 where very dense silty sand was encountered at a depth of approximately 23 feet underlain by very dense sand with silt at a depth of approximately 48 feet.

It should be noted, all the CPTs identified small layers of peat or other organic-heavy material within the upper 15 feet of the CPTs; Test Boring B-2 showed a roughly 7.5 foot-thick layer of peat at approximately 18.5 feet below existing grades, with another 3-inch layer at approximately 31 feet.

The *Geologic Map of the East Half of the Bellevue South 7.5'x15' Quadrangle, Issaquah Area, King County, Washington,* by D.B. Booth, T.J. Walsh, K.G. Troost, and S.A. Shimel (2012) maps the site as Peat Deposits (Qp). However, the native soils observed in the test borings are more consistent with Alluvium deposits (Qal), which is mapped roughly 400 feet to the west.

The preceding discussion is intended to be a general review of the soil conditions encountered. For more detailed descriptions, please refer to the Test Boring Logs in Appendix A. The approximate locations of the Test Borings and CPTs are shown on attached Figure 2.

3.3 Groundwater

We observed groundwater in all three test borings at a depth of approximately seven feet below existing grades. We performed pore water dissipation tests at CPT-1 and CPT-4. The test results indicate the static groundwater level to be located at approximately 7.3 feet and 6.89 feet below existing site grades for CPT-1 and CPT-4, respectively. Based on our experience in the area, the groundwater observed is part of the regional groundwater table and is influenced by flows in Tibbetts Creek located southeast of the site.

Fluctuations in the static groundwater level will occur seasonally. Typically, groundwater will reach maximum levels during the wet winter months. Based on the time of year the water levels were recorded, they likely represent near seasonal high groundwater levels.

3.4 Geologic Hazards

Section 18.02.050 of the City of Issaquah Municipal Code (IMC) classifies geologically related Critical Areas as erosion hazard areas, coal mine hazard areas, landslide hazard areas, steep slope hazard areas, and seismic hazard areas. The following is an assessment of these hazards concerning the project site.

3.4.1 Erosion Hazard Areas

Section 18.10.390 of the IMC defines erosion hazard areas as "…areas of King County and the City containing soils which, according to the USDA Soil Conservation Service, the 1973 King County Soils Survey and any subsequent revisions or additions thereto, may experience severe to very severe erosion hazard. This group of soils includes, but is not limited to, the following when they occur on slopes of fifteen (15) percent or greater: Alderwood gravelly sandy loam (AgD), Alderwood-Kitsap (Akf), Beausite gravelly sandy loam (BeD and BeF), Kitsap silt loam (Kpd), Oval gravelly sand loam (OvD and OvF), Ragnar fine sandy loam (RaD), Ragnar-Indianola Association (RdE), and any occurrence of River Wash (Rh)."

The majority of the soils observed onsite are classified as Sammamish silt loam (Sh) with soils in the southeast of the site classified as Kitsap silt loam, two to eight percent slopes (KpB) by the United States Department of Agriculture Natural Resources Conservation Service (NRCS). With the existing slope gradients, these soils will have a slight to moderate potential for erosion. Therefore, the site does not meet the above criteria for an erosion hazard area per the IMC. Regardless, the site soils would be susceptible to some erosion when exposed during construction. Proper implementation and maintenance of Best Management Practices (BMPs) for erosion prevention and sediment control would adequately mitigate the erosion potential in the planned development area in our opinion. Erosion protection measures as required by the City of Issaquah will need to be in place prior to and during grading activities at the site.

3.4.2 Coal Mine Hazard Areas

Section 18.10.390 of the IMC defines coal mine hazard areas as "…areas of the City directly underlain by or affected by abandoned coal mine working such as adits, tunnels, drifts or air shafts."

The King County Sensitive Areas Ordinance (SAO) Coal Mine Hazards map shows no workings exist below the site. Based on this review and the absence of any observed evidence of mine entrances, workings, or subsidence during our site reconnaissance, the site is not located within a coal mine hazard area in our opinion.

3.4.3 Landslide Hazard Areas

Section 18.10.390 of the IMC defines landslide hazard areas as "…areas of the City subject to a severe risk of landslide. A geotechnical report is required for all relevant projects to determine steepness of slope, permeability of soils, occurrence of springs, and groundwater level. The study shall be performed by a licensed geotechnical engineer. Landslide hazard areas include the following areas:

A. Slopes greater than forty (40) percent.

B. Any area with a combination of:

1. Slopes of greater than fifteen (15) percent;
2. Impermeable soils (typically silt and clay) frequently interbedded with granular soils (predominantly sand and gravel); and
3. Springs or groundwater seepage.

C. Any area which has shown movement during the Holocene epoch (from ten thousand (10,000) years ago to present) or which is underlain by mass wastage debris of that epoch.

D. Any area potentially unstable as a result of rapid stream incision, stream bank erosion, or undercutting by wave action.

E. Any area which shows evidence of, or is at risk from, snow avalanches.

F. Any area located on an alluvial fan, presently subject to or potentially subject to, inundation by debris flows or deposition of stream-transported sediments."

Site topography consists of a slight slope with little to no risk of mass movement due to geologic, topographic, or hydrologic factors. Therefore, the site is not a landslide hazard area as defined by the IMC in our opinion.

3.4.4 Steep Slope Hazard Areas

Section 18.10.390 of the IMC defines steep slope hazard areas as "Any ground that rises at an inclination of forty (40) percent or more within a vertical elevation change of at least ten (10) feet (a vertical rise of ten (10) feet or more for every twenty-five (25) feet of horizontal distance). A slope is delineated by establishing its toe and top and measured by averaging the inclination over at least ten (10) feet of vertical relief.

A. The "toe of a slope" is a distinct topographic break in a slope which separates slopes inclined at less than forty (40) percent from slopes equal to or in excess of forty (40) percent. Where no distinct break exists, the toe of a steep slope is the lowermost limit of the area where the ground surface drops ten (10) feet or more vertically within a horizontal distance of twenty-five (25) feet.

B. The "top of a slope" is a distinct, topographic break in a slope which separates slopes inclined at less than forty (40) percent from slopes equal to or in excess of forty (40) percent. Where no distinct break in slope exists, the top of a slope shall be the uppermost limit of the area where the ground surface rises ten (10) feet or more vertically within a horizontal distance of twenty-five (25) feet."

No slopes meeting the above criteria for a steep slope hazard exist at the site. Therefore, the site is not a steep slope hazard area as defined by the IMC in our opinion.

3.4.5 Seismic Hazard Areas

Section 18.10.390 of the IMC defines seismic hazard areas as "… areas of the City subject to severe risk of earthquake damage as a result of seismically induced settlement or soil liquefaction. These conditions may occur in areas underlain by cohesionless soils of low density usually in association with a shallow groundwater table."

Liquefaction is a phenomenon where there is a reduction or complete loss of soil strength due to an increase in water pressure induced by vibrations. Liquefaction mainly effects geologically recent deposits of fine-grained sand below the groundwater table. Soils of this nature derive their strength from intergranular friction. The generated water pressure or pore pressure essentially separates the soil grains and eliminates this intergranular friction; thus, eliminating the soil's strength.

We completed an analysis of soil liquefaction potential incorporating field soil strength values and soil types determined from the CPT soundings. A depth to groundwater of seven feet was used in the analysis based on our test pit observations. The analysis is based on a Magnitude 7 earthquake inducing ground motions having a peak ground acceleration (PGA) value of 0.66g. This acceleration represents an earthquake with a two percent probability of exceedance in 50 years. Results of the analysis are attached in Appendix B.

Based on our analysis, impacts to site structures should liquefaction occur during an earthquake will be in the form of settlement in the amount of about one to five inches. This amount of settlement would not structurally impact the building, but would result in damage of a cosmetic nature. As discussed, we recommend the building be supported on augercast piles to avoid settlement within the upper compressible alluvial silts. Supporting the building on augercast piles would also mitigate damage caused by liquefaction induced settlement. Liquefaction calculations are attached in Appendix B.

Based on soil conditions observed in the subsurface explorations and our knowledge of the area geology, per the current International Building Code (IBC), site class "E" should be used in structural design.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 General

Based on our study, the primary geotechnical concern would be the presence of the very soft to medium stiff, compressible silt layers and very loose to loose silty sand material typically observed in the upper 25 to 60 feet of the site. This material would not be suitable for support of the building foundation or floor slabs. Due to the heavy building loads, a preload or surcharge would not be effective; therefore, we recommend supporting the building on augercast piles.

The very loose to loose alluvial sands and silty sands typically observed in the upper 25 to 60 feet exhibit the potential for soil liquefaction during a design level earthquake. The primary impact, should soil liquefaction occur, would be potential building settlement estimated to be up to approximately five inches. This amount of settlement would not structurally impact the building but would result in damage of a cosmetic nature. Supporting the building foundations on augercast piles advanced to obtain support on the dense to very dense sands and gravels typically observed below approximately 25 to 60 feet, would be an acceptable means to mitigate building settlement due to soil liquefaction.

Due to site constraints, the excavation, needed to achieve the lowest building levels, will need to be shored. Due to the limited height, we recommend using conventional cantilevered soldier pile shoring with timber lagging for the shoring system. In addition, the contractor should be prepared to implement temporary dewatering to achieve excavation depths below approximately seven feet.

With one to two levels of below-grade construction, special consideration will need to be given to managing the elevated groundwater table both on a temporary and permanent basis. Excavation depths of 10 to 16 feet are anticipated to construct the lower building level, foundations, and elevator pits. With the winter static groundwater level at the site at a depth of approximately seven feet, temporary dewatering using well points will need to be completed to lower the water level three to nine feet to allow for construction of the first level during the winter. Additional dewatering will be required interior of the project site to construct the second lower level. Dewatering well points will be needed to lower the groundwater level the entire height of the second level excavation. The summer groundwater level will need to be determined for the project site.

The following sections provide detailed recommendations regarding the preceding issues and other geotechnical design and construction considerations. These recommendations should be incorporated into the final design drawings and construction specifications.

4.2 Site Preparation and Grading

To prepare the site for construction, all asphalt and other deleterious materials should be stripped and removed from the site. Demolition of existing structures should include removal of existing foundations and abandonment of underground septic systems and other buried utilities. Abandoned utility pipes that fall outside of new building areas can be left in place provided they are sealed to prevent intrusion of groundwater seepage and soil.

Once clearing and demolition operations are complete, cut and fill operations can be initiated to establish desired grades. Prior to placing fill, all exposed bearing surfaces should be observed by a representative of Terra Associates, Inc. to verify soil conditions are as expected and suitable for support of new fill. Our representative may request a proofroll using heavy rubber-tired equipment to determine if any isolated soft and yielding areas are present. If excessively yielding areas are observed and they cannot be stabilized in place by compaction, the affected soils should be excavated and removed to firm bearing and grade restored with new structural fill. If the depth of excavation to remove unstable soils is excessive beneath embankment fills or roadway subgrade, the use of geotextile fabrics such as Mirafi 500X or an equivalent fabric can be used in conjunction with clean, granular structural fill. Our experience has shown, in general, a minimum of 18 inches of a clean, granular structural fill placed and compacted over the geotextile fabric should establish a stable bearing surface.

The native soils encountered at the site contain a sufficient amount of soil fines that will make them difficult to compact as structural fill when too wet or too dry. The ability to use native soils from site excavations as structural fill will depend upon its moisture content and the prevailing weather conditions at the time of construction. When wet soils are encountered, the contractor will need to dry the soils by aeration during dry weather conditions. Alternatively, the use of an additive such as Portland cement or lime to stabilize the soil moisture can be considered. If the soil is amended, additional Best Management Practices (BMPs) addressing the potential for elevated pH levels will need to be included in the Stormwater Pollution Prevention Program (SWPPP) prepared with the Temporary Erosion and Sedimentation Control (TESC) plan.

If importing fill becomes necessary, we recommend importing a granular soil that meets the following grading requirements:

U.S. Sieve Size	Percent Passing
6 inches	100
No. 4	75 maximum
No. 200	5 maximum*

*Based on the 3/4-inch fraction.

Prior to use, Terra Associates, Inc. should examine and test all materials imported to the site for use as structural fill.

Structural fill should be placed in uniform loose layers not exceeding 12 inches, and compacted to a minimum of 95 percent of the soil's maximum dry density, as determined by American Society for Testing and Materials (ASTM) Test Designation D-698 (Standard Proctor). The moisture content of the soil at the time of compaction should be within two percent of its optimum, as determined by this ASTM standard. In nonstructural areas, the degree of compaction can be reduced to 90 percent. The silt material will need to be placed in six-inch lifts to achieve compaction.

4.3 Excavation, Shoring, and Dewatering

Conventional Excavation

Based on the regulations outlined in the Washington Industrial Safety and Health Act (WISHA), the soils at the site would be classified as Type C soil. Properly dewatered open excavation side slopes should be laid back at an inclination of 1.5:1 (Horizontal:Vertical).

Shoring

Given the expected excavation depth and building limits, site constraints will require the excavation sidewalls be supported by temporary shoring. Due to the limited shoring height, we recommend using conventional cantilevered soldier pile shoring with timber lagging for the shoring system.

The following sections outline our recommendations for design of the temporary shoring system.

Cantilevered Soldier Pile Shoring

Cantilevered soldier pile walls should be designed to resist lateral loads imposed by the adjacent soils and surcharge loadings that will be imposed.

We recommend soldier piles have a maximum center-to-center spacing of eight feet. Recommended design earth pressure diagrams with adjacent building surcharge are presented on Figure 3. For pile spacing of eight feet and less, the lateral soil pressure uniformly distributed over the width of the lagging can be reduced by 50 percent to account for soil arching between the soldier piles.

Unshored excavation heights should not exceed four feet during the excavation. Excavation lifts through the upper very loose to loose alluvial sands indicated in the test borings and CPTs should not exceed three feet without lagging. No excavation should remain unsupported for more than 24 hours.

Drilling obstructions, such as boulders, may be encountered. The soil and groundwater conditions will likely cave or collapse if open hole drilling is attempted. Therefore, the contractor must be prepared to case the drilled shafts as needed to prevent collapse and maintain a relatively clean, open hole. The shaft bottoms must be relatively clean of loose soil debris prior to insertion of the soldier pile beams and pouring concrete.

Over-break or gaps between the excavated soil face and the back of the lagging must be filled following each excavation lift. Filling with crushed rock or grouting with control density fill (CDF) is recommended. This will be an important consideration in limiting movement of the adjacent ground.

Temporary Dewatering

Based on groundwater conditions observed, dewatering efforts will need to be implemented as the excavation extends below depths of approximately seven feet below current site grades in the winter and approximately ten feet below grade in the summer. Given the groundwater conditions, dewatering using well points will likely be the most cost-efficient method in our opinion.

The temporary dewatering system should be designed and installed by an experienced dewatering well contractor. Additional testing should be performed to determine dewatering parameters.

Monitoring Program

A monitoring program must be implemented to verify the performance of the shoring system and possible excavation effects on adjacent properties. The first step of this program should consist of documenting the existing conditions of the adjacent properties and pavements. The documentation should include a visual survey and pictorial record.

We recommend optical survey monitoring be conducted by the owner and include the measurement of horizontal and vertical movements of:

1. The surface of the adjacent streets.
2. The adjacent buildings to the north and east.
3. The shoring system.

To monitor potential vertical and horizontal movements of the shoring, monitoring points should be established at the top of every other soldier pile. When the excavation reaches the halfway point, depending on readings obtained during the initial excavation phase, additional monitoring points may need to be established. Surface reference points should be established and monitored for elevation at distances of five and ten feet from the back of the shoring at spacing of 25 feet at the excavation perimeter.

Optical monitoring of the shoring system should be performed twice a week as the excavation proceeds, then every other week upon completion of the excavation. A registered land surveyor should be retained to perform the monitoring. Monitoring should continue until the basement walls are adequately braced at the ground surface level. The monitoring data should be submitted to the project shoring designer and Terra Associates, Inc. for review within one day.

4.4 Foundations

Augercast Piles

As noted above, the upper 25 to 60 feet of very soft to medium stiff, compressible silt and very loose to loose silty sand will not be suitable for support of conventional spread footing foundations for the expected building loads. Based on the soil conditions and proposed construction, we recommend supporting the structure on augercast piles advanced through the upper alluvial soils to obtain support on the lower dense to very dense sands and gravels. In our opinion, driven piles should not be used for foundation support due to the potential noise and vibration-related impacts to adjacent businesses.

We recommend the piles extend a minimum depth of five feet into the dense sands and gravel. Based on the test borings and CPT test data with this minimum embedment, pile tips would extend to depths of 30 to 65 feet below existing ground surface elevations. The dense sands and gravels were observed at approximately 25 feet below existing grades in the south-central portion of the site and at approximately 60 feet in the northern portion of the site. Recommended axial design capacities for 18- and 24-inch diameter augercast piles are as follows:

Axial Pile Capacities for 30-foot Pile

Allowable Axial Capacity		Allowable Uplift Capacity	
Pile Diameter		Pile Diameter	
18-inch	24-inch	18-inch	24-inch
97 kips	173 kips	35 kips	47 kips

Axial Pile Capacities for 65-foot Pile

Allowable Axial Capacity		Allowable Uplift Capacity	
Pile Diameter		Pile Diameter	
18-inch	24-inch	18-inch	24-inch
106 kips	188 kips	97 kips	129 kips

These allowable capacities include considerations for potential down drag due to subsidence caused by liquefaction during a seismic event. Estimated pile settlement excluding pile compression falls in the range of one-quarter to one inch. This settlement will primarily be immediate in nature, occurring as building loads are applied.

Lateral Pile Capacities

Lateral pile load capacity analyses were performed for a single pile. The analyses assume the pile will act as a beam under vertical loading. The vertical loading follows the allowable pile capacities above. For the analyses, we used the computer program LPILE Plus 10.0.

The design lateral load available will be dependent upon the allowable lateral deflection that can be tolerated. The following table provides single-pile lateral capacities for deflections of one inch at the top of the pile for both free and fixed head conditions for 18- and 24-inch diameter piles:

Lateral Pile Capacities

Pile Diameter	Lateral (Free Head)	Point of Fixity	Max Moment @ Point of Fixity	Lateral (Fixed Head)	Fixing Moment
18-inch	14 kips	9 feet	76 k-ft.	30 kips	-195 k-ft.
24-inch	22 kips	12 feet	152 k-ft.	47 kips	-380 k-ft.

In addition to the lateral pile capacities, additional lateral resistance will be provided by passive earth pressure acting adjacent to the buried portions of the pile caps and grade beams. Passive resistance equivalent to a fluid weighing 300 pcf can be used to calculate this lateral resistance.

Full single-pile capacities can be used in design for both vertical and lateral loading conditions, provided pile spacing equivalent to three pile diameters is maintained. Closer spacing will impact single-pile capacities as imposed stress fields overlap. If center to center pile spacing is less than three pile diameters, we should review the pile layout and determine the reduction impact to the individual pile capacities.

Construction Considerations

The auger should be extracted slowly and uniformly below a sufficient and consistent head of grout. If the auger is extracted too quickly, the pile may neck down and soil may collapse into the pile, reducing its structural integrity. At a point along the injection line, the piling contractor should use a pressure gauge to monitor the grout pressure during construction.

The pressure used to inject the grout and construct the pile column will compress the soils immediately adjacent to the pile. As a result, the amount of grout needed to form the pile will be greater than the computed grout volume. There will also be excess grout used to construct the piles because of the head of grout in the hollow stem auger required to construct the pile. Minimum grout takes should typically exceed the theoretical grout volume by 10 to 15 percent. Accounting for compression of the soils, maximum grout takes of 1.5 to 1.8 times the theoretical volumes should be expected. The contractor must take this into consideration in estimating grout volumes. The grout pump should be calibrated with a stroke counter to allow for monitoring and verifying the amount of grout used to construct the pile.

The pile installation sequence should be such that piles are constructed at a minimum spacing of five diameters. Once grout has achieved its initial set, usually in 24 hours, installation between these locations can be completed.

4.5 Slab-on-Grade Floors

Slab-on-grade floors should be supported on a minimum of 18 inches of new granular structural fill that replaces the existing very soft to medium stiff silts and very loose to loose silty sands. We recommend placing and compacting sub-slab fill as structural fill as described in Section 4.2 of this report. Immediately below the floor slab, we recommend placing a four-inch-thick capillary break layer composed of clean, coarse sand or fine gravel that has less than five percent passing the No. 200 sieve. This material will reduce the potential for upward capillary movement of water through the underlying soil and subsequent wetting of the floor slab.

The capillary break layer will not prevent moisture intrusion through the slab caused by water vapor transmission. Where moisture by vapor transmission is undesirable, such as covered floor areas, a common practice is to place a durable plastic membrane on the capillary break layer, then cover the membrane with a layer of clean sand or fine gravel to protect it from damage during construction and to aid in uniform curing of the concrete slab. It should be noted, if the sand or gravel layer overlying the membrane is saturated prior to pouring the slab, it will not be effective in assisting uniform curing of the slab and can actually serve as a water supply for moisture bleeding through the slab, potentially affecting floor coverings. Therefore, in our opinion, covering the membrane with a layer of sand or gravel should be avoided if floor slab construction occurs during the wet winter months and the layer cannot be effectively drained. We recommend floor designers and contractors refer to the American Concrete Institute (ACI) Manual of Concrete Practice for further information regarding vapor barrier installation below slab-on-grade floors.

4.6 Lateral Earth Pressures for Below-Grade Walls

Lower-level building walls should be designed for the earth pressure parameters presented on Figure 4. To prevent hydrostatic loading, the walls must be provided with adequate drainage. Typically, for walls constructed against temporary soldier pile/timber lagging, wall drainage is provided by attaching prefabricated drainage panels such as Miradrain G100N to the shoring. Drainpipes are attached to the Miradrain panels at the wall base and tightlined to discharge through the permanent wall. A typical drainage detail for permanent lower-level walls constructed against the temporary shoring system is shown on Figure 5.

For permanent lower-level walls constructed against cut excavations, the magnitude of earth pressures developing on below-grade walls will depend on the quality and compaction of the wall backfill. We recommend placing and compacting wall backfill as structural fill as described in Section 4.2 of this report. To prevent overstressing the walls during backfilling, heavy construction machinery should not be operated within five feet of the wall. Wall backfill in this zone should be compacted with hand-operated equipment. To prevent hydrostatic pressure development, wall drainage must also be installed. A typical wall drainage detail is shown on Figure 6. All drains should be routed to the storm sewer system or other approved point of controlled discharge.

With drainage properly installed, we recommend designing unrestrained walls for an active earth pressure equivalent to a fluid weighing 45 pounds per cubic foot (pcf). For restrained walls, an additional uniform load of 100 psf should be included in the wall design. To account for typical traffic surcharge loading, the walls can be designed for an additional imaginary height of two feet (two-foot soil surcharge). For evaluation of wall performance under seismic loading, a uniform pressure equivalent to 8H psf, where H is the height of the below-grade portion of the wall should be applied in addition to the static lateral earth pressure. These values assume a horizontal backfill condition and no other surcharge loading, sloping embankments, or adjacent buildings will act on the wall. If such conditions exist, then the imposed loading must be included in the wall design. Friction at the base of foundations and passive earth pressure will provide resistance to these lateral loads.

For any building walls that are constructed without drainage the active earth pressure equivalent fluid weight should be 95 pcf for unrestrained walls. For restrained walls, an additional uniform load of 100 psf should be included in the wall design. All other loading recommendations in the above sections should be included in the lower level building wall design.

4.7 Drainage

Surface

Final exterior grades should promote free and positive drainage away from the site at all times. Water must not be allowed to pond or collect adjacent to foundations or within the immediate building areas. We recommend providing a positive drainage gradient away from the building perimeters. If this gradient cannot be provided, surface water should be collected adjacent to the structures and disposed to appropriate storm facilities.

Subsurface

In addition to the drainage for the walls, we recommend installing perimeter foundation drains adjacent to shallow foundations. The drains can be laid to grade at an invert elevation equivalent to the bottom of footing grade. The drains can consist of four-inch diameter perforated PVC pipe enveloped in washed pea gravel-sized drainage aggregate. The aggregate should extend six inches above and to the sides of the pipe. Roof and foundation drains should be tightlined separately to the storm drains. All drains should be provided with cleanouts at easily accessible locations.

4.8 Utilities

Utility pipes should be bedded and backfilled in accordance with American Public Works Association (APWA) or the City of Issaquah specifications. At a minimum, trench backfill should be placed and compacted as structural fill, as described in Section 4.2 of this report. As noted, depending on the soil moisture when excavated most inorganic native soils on the site should be suitable for use as backfill material during dry weather conditions. However, if utility construction takes place during the wet winter months, it will likely be necessary to import suitable wet-weather fill for utility trench backfilling. The native silt soils will need to be placed in six-inch loose lifts to achieve compaction.

4.9 Pavements

Pavement subgrades should be prepared as described in the Section 4.2 of this report. Regardless of the degree of relative compaction achieved, the subgrade must be firm and relatively unyielding before paving. The subgrade should be proofrolled with heavy construction equipment to verify this condition. If the soft silt soils are exposed at the pavement subgrade, we recommend over-excavating 18 inches, placing a geotextile fabric such as Mirafi 500X, and restoring the subgrade using crushed rock in order to prepare a stable surface.

The pavement design section is dependent upon the supporting capability of the subgrade soils and the traffic conditions to which it will be subjected. For access roadways, with traffic consisting mainly of light passenger vehicles with only occasional heavy traffic and with a stable subgrade prepared as recommended, we recommend the following pavement sections:

- Two inches of Hot Mix Asphalt (HMA) over eight inches of Crushed Rock Base (CRB)
- Four- and one-half inches full depth HMA

The paving materials used should conform to the Washington State Department of Transportation (WSDOT) specifications for half-inch class HMA and CRB.

Long-term pavement performance will depend on surface drainage. A poorly-drained pavement section will be subject to premature failure as a result of surface water infiltrating into the subgrade soils and reducing their supporting capability. For optimum pavement performance, we recommend surface drainage gradients of at least two percent. Some degree of longitudinal and transverse cracking of the pavement surface should be expected over time. Regular maintenance should be planned to seal cracks when they occur.

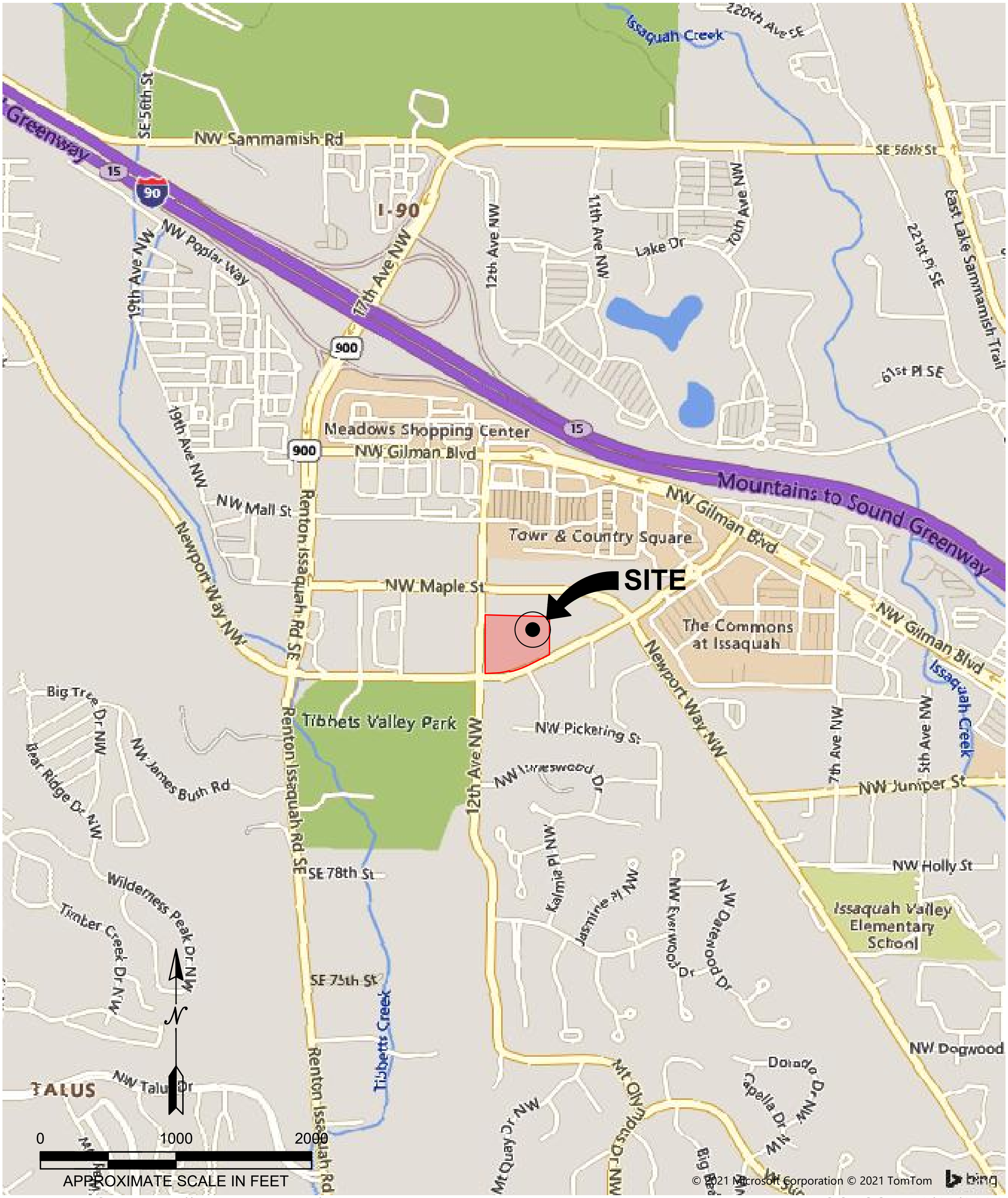
5.0 ADDITIONAL SERVICES

Terra Associates, Inc. should review the final design drawings and specifications in order to verify earthwork and foundation recommendations have been properly interpreted and implemented in project design. We should also provide geotechnical service during construction to observe compliance with our design concepts, specifications, and recommendations. This will allow for design changes if subsurface conditions differ from those anticipated prior to the start of construction.

6.0 LIMITATIONS

We prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made. This report is the copyrighted property of Terra Associates, Inc. and is intended for specific application to the Avalon Issaquah project in Issaquah, Washington. This report is for the exclusive use of Avalon Bay Communities and its authorized representatives.

The analyses and recommendations presented in this report are based on data obtained from the subsurface explorations performed onsite. Variations in soil conditions can occur, the nature and extent of which may not become evident until construction. If variations appear evident, Terra Associates, Inc. should be requested to reevaluate the recommendations in this report prior to proceeding with construction.



REFERENCE: https://www.bing.com/maps ACCESSED 3/16/2021

Terra Associates, Inc. Consultants in Geotechnical Engineering Geology and Environmental Earth Sciences	VICINITY MAP AVALON ISSAQUAH ISSAQUAH, WASHINGTON	
Proj.No. T-8488	Date: JAN 2022	Figure 1

NOTE:

THIS SITE PLAN IS SCHEMATIC. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE. IT IS INTENDED FOR REFERENCE ONLY AND SHOULD NOT BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

REFERENCE: SITE PLAN PROVIDED BY BING MAPS.

LEGEND:

- APPROXIMATE TEST BORING LOCATION
- APPROXIMATE CONE PENETRATION TEST LOCATION

0 60 120

APPROXIMATE SCALE IN FEET

N

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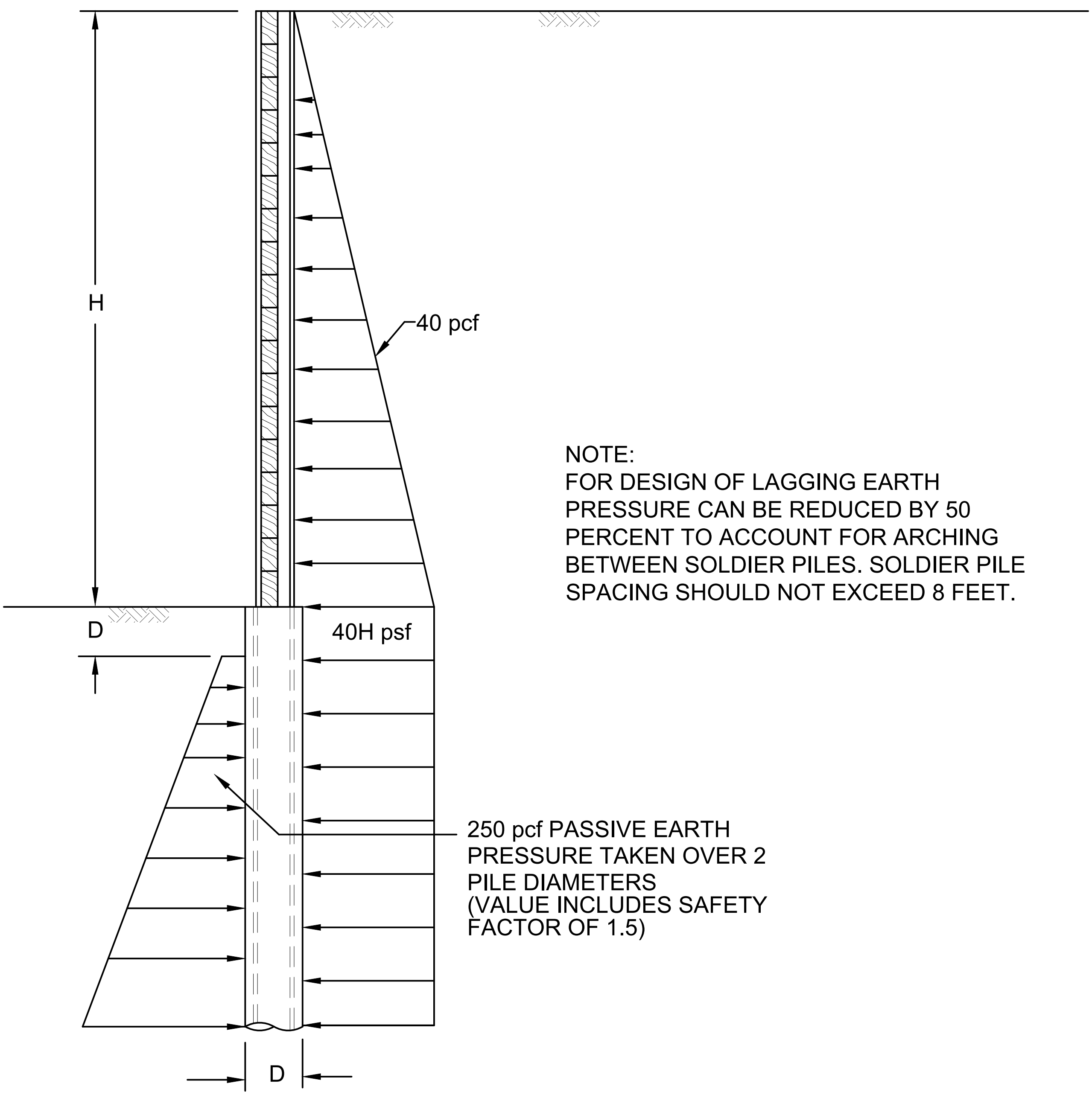
Geology and
Environmental Earth Sciences

EXPLORATION LOCATION PLAN
AVALON ISSAQUAH
ISSAQUAH, WASHINGTON

Proj.No. T-8488	Date: JAN 2022	Figure 2

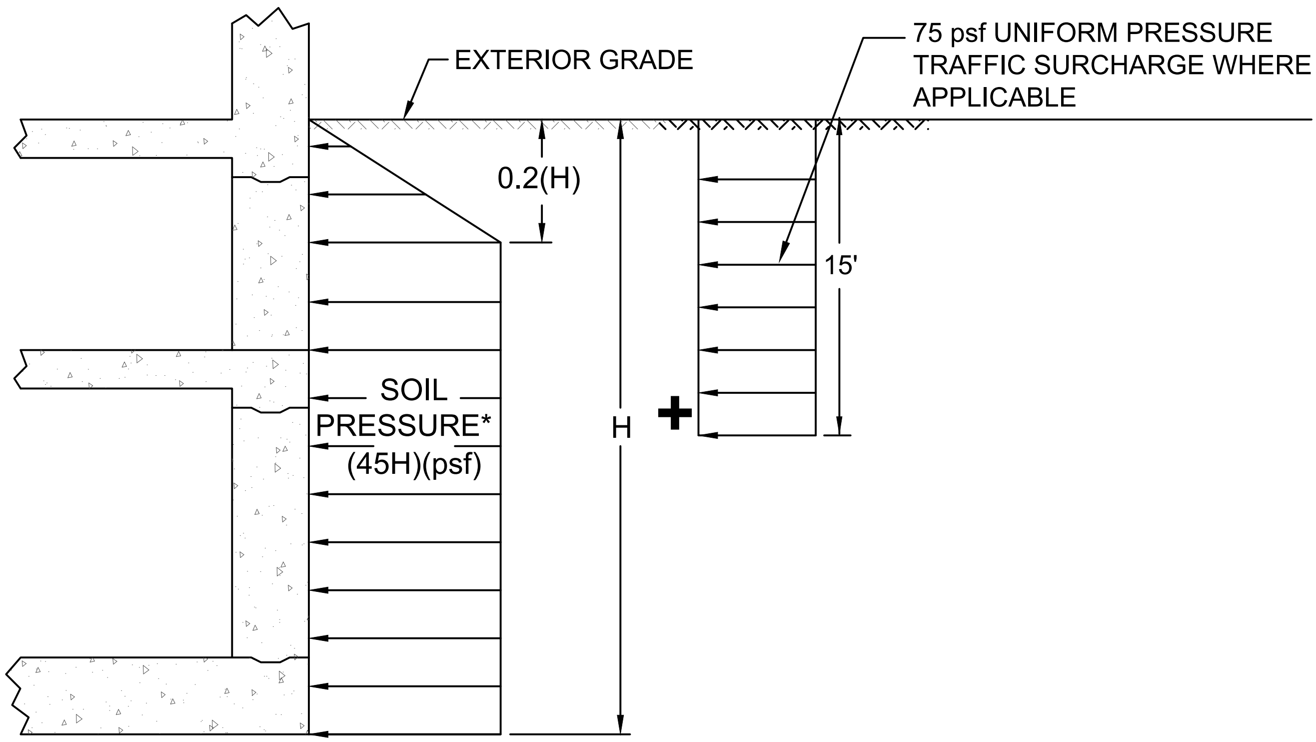
EARTH PRESSURE LOADING DIAGRAM
CANTILEVERED SOLDIER PILE SHORING



NOTE:
FOR DESIGN OF LAGGING EARTH PRESSURE CAN BE REDUCED BY 50 PERCENT TO ACCOUNT FOR ARCHING BETWEEN SOLDIER PILES. SOLDIER PILE SPACING SHOULD NOT EXCEED 8 FEET.

NOT TO SCALE

Terra Associates, Inc. Consultants in Geotechnical Engineering Geology and Environmental Earth Sciences	EARTH PRESSURE LOADING DIAGRAM AVALON ISSAQUAH ISSAQUAH, WASHINGTON		
	Proj.No. T-8488	Date: JAN 2022	Figure 3



NOT TO SCALE

* FOR SEISMIC LOADING EVALUATION INCREASE SOIL PRESSURE TO 53H

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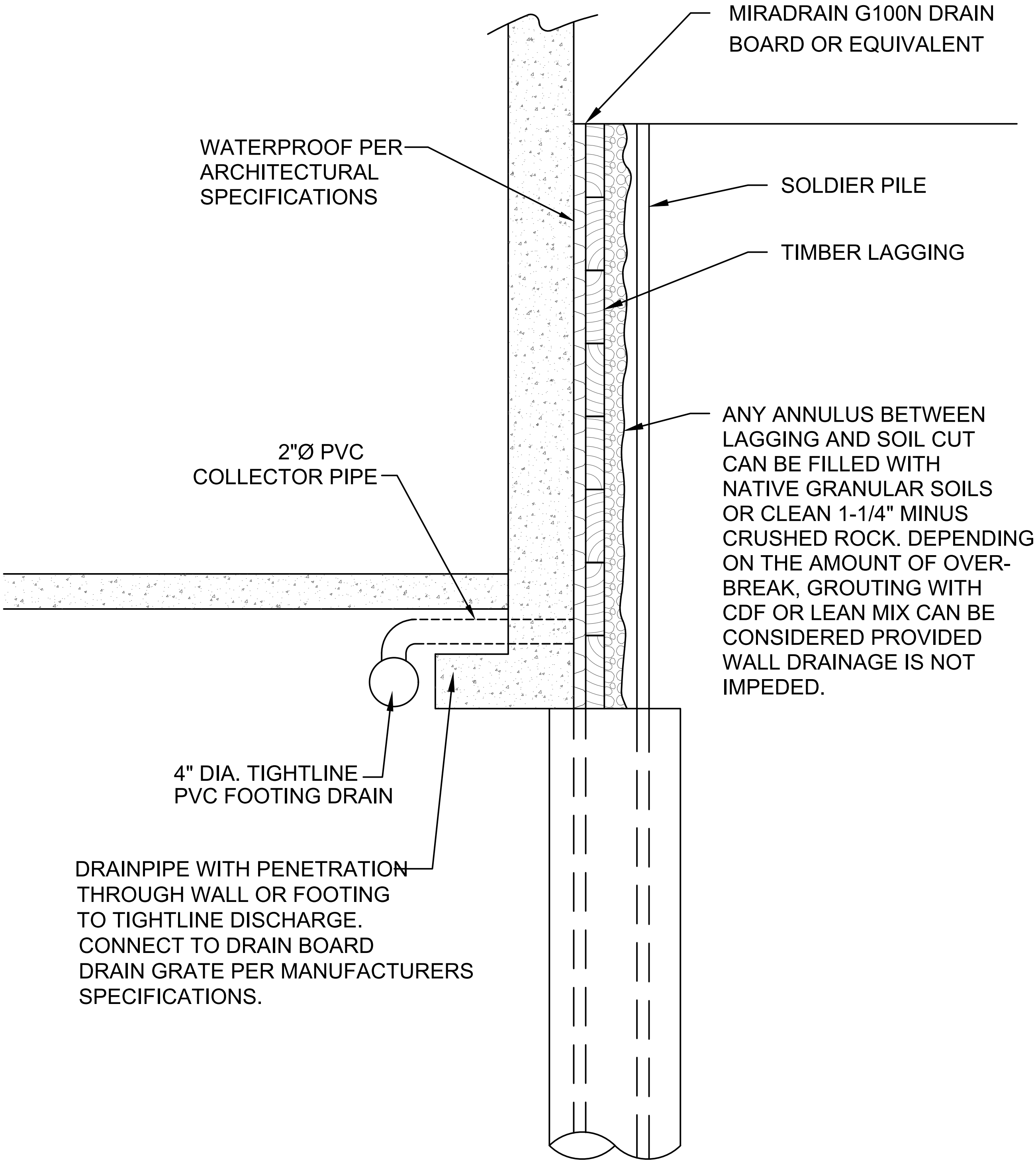
LATERAL EARTH PRESSURE ON BASEMENT WALLS AVALON ISSAQUAH ISSAQUAH, WASHINGTON		
Proj.No. T-8488	Date: JAN 2022	Figure 4



NOT TO SCALE

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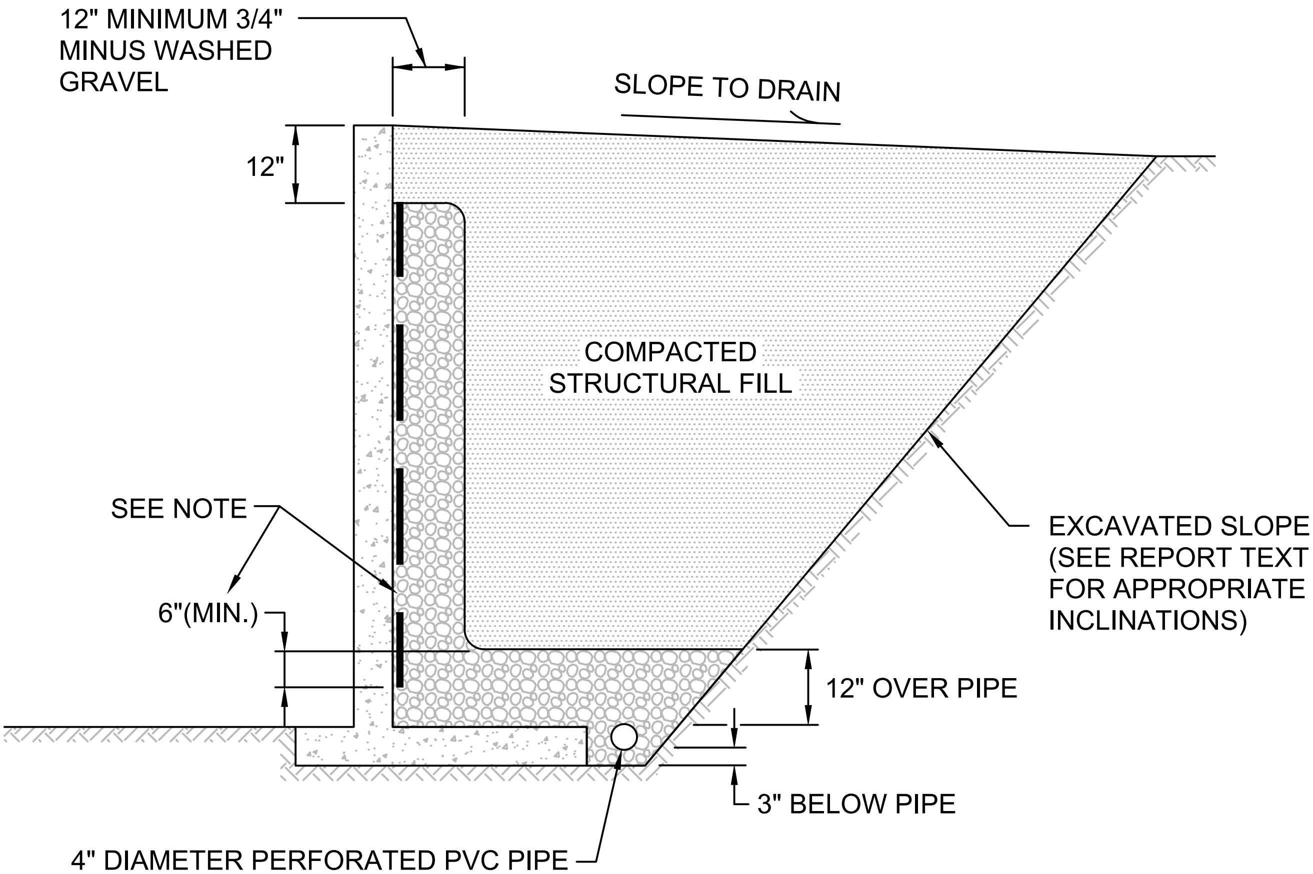
Geology and
Environmental Earth Sciences

SOLDIER PILE WALL DRAINAGE DETAIL
AVALON ISSAQUAH
ISSAQUAH, WASHINGTON

Proj.No. T-8488	Date: JAN 2022	Figure 5



NOT TO SCALE

NOTE:

MIRADRAIN G100N PREFABRICATED DRAINAGE PANELS OR SIMILAR PRODUCT CAN BE SUBSTITUTED FOR THE 12-INCH WIDE GRAVEL DRAIN BEHIND WALL. DRAINAGE PANELS SHOULD EXTEND A MINIMUM OF SIX INCHES INTO 12-INCH THICK DRAINAGE GRAVEL LAYER OVER PERFORATED DRAIN PIPE.

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Geology and
Environmental Earth Sciences

TYPICAL WALL DRAINAGE DETAIL
AVALON ISSAQUAH
ISSAQUAH, WASHINGTON

Proj.No. T-8488	Date: JAN 2022	Figure 6

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

Avalon Issaquah
Issaquah, Washington

On March 1, 2021, we explored subsurface conditions at the site by performing four cone penetration tests (CPTs) advanced to depth of approximately 25 to 60 feet below existing site grades. On March 10, 2021, we observed soil conditions at three test borings drilled to depths of 50 feet below existing grades. The CPT and test boring locations were approximately determined in the field by sighting and measuring from existing site features. The approximate locations of the test borings and CPTs are shown on the attached Exploration Location Plan, Figure 2. Test Boring Logs are attached as Figures A-2 through A-4.

A geotechnical engineer from our office conducted the field exploration. Our representative classified the soil conditions encountered, maintained a log of each test boring, obtained representative soil samples, and recorded water levels observed during drilling. During drilling, soil samples were obtained in general accordance with ASTM Test Designation D-1586. Using this procedure, a 2-inch (outside diameter) split barrel sampler is driven into the ground 18 inches using a 140-pound hammer free falling from a height of 30 inches. The number of blows required to drive the sampler 12 inches after an initial 6-inch set is referred to as the Standard Penetration Resistance value or N value. This is an index related to the consistency of cohesive soils and relative density of cohesionless materials. N values obtained for each sampling interval are recorded on the Test Boring Logs, Figures A-2 through A-4. All soil samples were visually classified in accordance with the Unified Soil Classification System (USCS) described on Figure A-1.

Representative soil samples obtained from the test borings were placed in closed containers and taken to our laboratory for further examination and testing. The moisture content of each sample was measured and is reported on the individual Test Boring Logs. Grain size analyses were performed on selected samples, the results of which are shown on Figure A-5.

InSitu Engineering, Inc., under subcontract to Terra Associates, Inc. performed the CPTs at locations selected by Terra Associates, Inc. The CPT consists of pushing an instrumented, approximately 1.5-inch diameter cone into the ground at a constant rate. During advancement, continuous measurements are made of the resistance to penetration of the cone and the friction of the outer surface of a sleeve. The cone is also equipped with a porous filter and a pressure transducer for measuring groundwater or pore water pressure generated. Measurements of tip and sleeve frictional resistance, pore pressure, and interpreted soil conditions are summarized in graphical form on the attached CPT Logs.

MAJOR DIVISIONS			LETTER SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS — More than 50% material larger than No. 200 sieve size	**GRAVELS** More than 50% of coarse fraction is larger than No. 4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS More than 50% of coarse fraction is smaller than No. 4 sieve	Clean Sands (less than 5% fines)	SW	Well-graded sands, sands with gravel, little or no fines.
			SP	Poorly-graded sands, sands with gravel, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — More than 50% material smaller than No. 200 sieve size	**SILTS AND CLAYS** Liquid Limit is less than 50%		ML	Inorganic silts, rock flour, clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity. (Lean clay)
			OL	Organic silts and organic clays of low plasticity.
	SILTS AND CLAYS Liquid Limit is greater than 50%		MH	Inorganic silts, elastic.
			CH	Inorganic clays of high plasticity. (Fat clay)
			OH	Organic clays of high plasticity.
HIGHLY ORGANIC SOILS			PT	Peat.

DEFINITION OF TERMS AND SYMBOLS

COHESIONLESS

Density	Standard Penetration Resistance in Blows/Foot
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	>50

COHESIVE

Consistancy	Standard Penetration Resistance in Blows/Foot
Very Soft	0-2
Soft	2-4
Medium Stiff	4-8
Stiff	8-16
Very Stiff	16-32
Hard	>32

Symbol	Meaning
I	2" OUTSIDE DIAMETER SPILT SPOON SAMPLER
II	2.4" INSIDE DIAMETER RING SAMPLER OR SHELBY TUBE SAMPLER
▼	WATER LEVEL (Date)
Tr	TORVANE READINGS, tsf
Pp	PENETROMETER READING, tsf
DD	DRY DENSITY, pounds per cubic foot
LL	LIQUID LIMIT, percent
PI	PLASTIC INDEX
N	STANDARD PENETRATION, blows per foot

Terra Associates, Inc.
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Geology and
Environmental Earth Sciences

UNIFIED SOIL CLASSIFICATION SYSTEM
AVALON ISSAQUAH
ISSAQUAH, WASHINGTON

Proj.No. T-8488	Date: JAN 2022	Figure A-1

LOG OF BORING NO. B-1

Figure No. A-2

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** -7 ft **Approx. Elev:** NA

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot	Moisture Content (%)
0	(2-inches ASPHALT) FILL: Gray silty SAND with gravel, fine to coarse sand, fine to coarse gravel, moist. (SM)	medium dense	18	7.0
	Gray sandy SILT with gravel, fine to coarse sand, fine to coarse gravel, moist to wet. (ML)	medium stiff	5	12.9
5	Brownish-gray to gray SILT to clayey SILT, moist to wet. (ML) *3-inch layer of sandy SILT observed at approximately 6 feet.	very soft	2	29.4
	Gray SILT with sand to sandy SILT, fine sand, wet, trace gravel, trace oxidized inclusions, occasional inclusion of brown silt. (ML)	very soft	2	36.7
10			2	36.9
	Gray clayey SILT to clayey SILT with sand, fine sand, wet. (ML) *2-inch layer of silty SAND with gravel observed at approximately 13.5 feet.	medium stiff	6	29.4
15	Gray sandy SILT, fine sand, wet. (ML) *3-inch layer clayey SILT observed at approximately 16 feet.	very soft	2	36.6

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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LOG OF BORING NO. B-1

Figure No. A-2

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** -7 ft **Approx. Elev:** NA

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot (10 30 50)	Moisture Content (%)
		Gray SILT to SILT with sand, fine sand, wet. (ML)			
20	sample			6	29.8
		Gray clayey SILT, moist to wet, infrequent sand with silt seams, occasional oxidized lamination. (ML)			
25	sample		medium stiff	4	37.2
30	sample			4	40.0
		Gray SAND to SAND with gravel, fine to coarse sand, fine to coarse gravel, wet, stratified silt seams. (SP)	medium dense		
35	sample			10	27.1

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

LOG OF BORING NO. B-1

Figure No. A-2

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** -7 ft **Approx. Elev:** NA

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot (10 30 50)	Moisture Content (%)
		Gray SAND to SAND with gravel, fine to coarse sand, fine to coarse gravel, wet, stratified silt seams. (SP)			
40		Gray silty SAND, fine to coarse sand, wet, trace gravel, interbedded silt layers. (SM)		10	20.1
45			medium dense	11	24.9
		Gray SAND, fine to coarse sand, wet, interbedded silty sand and silt seams. (SP)			
50				10	21.7
		Test Boring terminated at approximately 50 feet. Groundwater seepage observed at approximately 7 feet.			
55					

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

LOG OF BORING NO. B-2

Figure No. A-3

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** 7 ft **Approx. Elev:** NA

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot	Moisture Content (%)
0	(2.5- inches ASPHALT) FILL: Brownish-gray SAND with silt and gravel, fine to coarse sand, fine to coarse gravel, moist. (SP-SM)	dense	36	5.9
	Brownish-gray SILT with sand and gravel, fine to coarse sand, fine to coarse gravel, moist, trace rootlets. (ML)	medium stiff	6	15.1
5	Brown grading to gray silty SAND with gravel, fine to coarse sand, fine to coarse gravel, wet. (SM)	loose	5	13.3
	Gray SILT to clayey SILT, moist to wet, alternating layers of silty sand with gravel. (ML)	soft	2	39.3
10		medium stiff	5	32.4
	Gray SILT to clayey SILT, moist to wet, interbedded silty sand layers, infrequent sand seams. (ML)	medium stiff	7	27.3 27.1
15	*scattered organics observed at approximately 15 feet.	soft	2	43.9 26.3

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

LOG OF BORING NO. B-2

Figure No. A-3

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** 7 ft **Approx. Elev:** NA

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot	Moisture Content (%)
18–26		Brown to black organic PEAT, fibrous, wet, scattered inclusions of gray silt, infrequent sand with silt layers. (PT)	soft		
20	20–21.5			2	105.6 22.9
25	25–26.5			3	119.4 75.1
26–33		Gray SILT to clayey SILT, moist to wet, trace silty sand seams, trace organics. (ML) *3-inch layer of PEAT observed at approximately 31 feet.	soft (to ~28); medium stiff		
30	30–31.5			5	56.3
33–36.5		Gray silty SAND, fine to medium sand, wet, interbedded silt layers. (SM)	loose		
35	35–36.5			7	26.2

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
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LOG OF BORING NO. B-2

Figure No. A-3

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** 7 ft **Approx. Elev:** NA

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot (10 30 50)	Moisture Content (%)
		Gray silty SAND, fine to medium sand, wet, interbedded silt layers. (SM)	loose		
40	Sample (40)	Gray SILT, moist, trace organics, interbedded sand with silt and silty sand seams. (ML)	medium stiff	6	35.8
45	Sample (45)	Gray silty SAND, fine to medium sand, wet, infrequent silt layers. (SM)	loose	8	27.6
50	Sample (50)			10	22.1
		Test Boring termintated at approximately 50 feet. Groundwater seepage observed at approximately 7 feet.			
55					

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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Geology and
Environmental Earth Sciences

LOG OF BORING NO. B-3

Figure No. A-4

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** 7 ft **Approx. Elev:** NA

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot	Moisture Content (%)
0	(4-inches ASPHALT) FILL: Gray silty SAND with gravel, fine to coarse sand, fine to coarse gravel, moist. (SM)	dense	40	5.0
	Gray SILT with sand to sandy SILT, fine sand, moist to wet, occasional organic, stratified silty sand seams in upper 18 inches. (ML) *3-inch layer of sand with silt and gravel observed at approximately 6 feet.	medium stiff	5 7	20.7 22.0
5				
	Brownish-gray to gray SILT to clayey SILT, moist to wet, trace sand, trace inclusions of brown silt. (ML)	soft	2 4	37.1 32.6
10				
	Gray SILT with sand to sandy SILT, fine sand, wet. (ML)	stiff	12	30.0
15	Gray silty SAND, fine to medium sand, wet, occasional gravel, infrequent silt layers. (SM)	very loose	4	26.8

SPT scale: 10 30 50

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

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Geology and
Environmental Earth Sciences

LOG OF BORING NO. B-3

Figure No. A-4

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** 7 ft **Approx. Elev:** NA

Depth (ft)	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot (10 30 50)	Moisture Content (%)
	Gray SILT with sand to sandy SILT, fine sand, wet, occasional gravel, frequent silty sand seams. (ML)			
20		medium stiff	7	20.1
	Gray silty SAND with gravel, fine to coarse sand, fine to coarse gravel, moist to wet, occasional layer of sandy silt. (SM)	very dense		
25			58	11.7
30			70/9"	12.9
	Gray sandy SILT with gravel, fine to medium sand, fine to coarse gravel, moist to wet. (ML)	hard		
35			50/3"	17.8

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

Terra Associates, Inc.
Consultants in Geotechnical Engineering
Geology and
Environmental Earth Sciences

LOG OF BORING NO. B-3

Figure No. A-4

Project: Avalon Issaquah **Project No:** T-8488 **Date Drilled:** March 10, 2021

Client: Avalon Bay Communities **Driller:** BoreTec **Logged By:** MJX

Location: Issaquah, Washington **Depth to Groundwater:** 7 ft **Approx. Elev:** NA

Depth (ft)	Sample Interval	Soil Description	Consistency/ Relative Density	SPT (N) Blows / foot (10 30 50)	Moisture Content (%)
		Gray silty SAND with gravel, fine to coarse sand, fine to coarse gravel, moist, infrequent silt seams. (SM)	very dense		
40	X			50/3"	21.8
45	X			70	14.8
		Gray SAND with silt, fine to coarse sand, wet. (SP-SM)			
50	X			50/5"	20.3
		Test Boring terminated at approximately 50 feet. Groundwater seepage observed at approximately 7 feet.			
55					

NOTE: This borehole log has been prepared for geotechnical purposes. This information pertains only to this boring location and should not be interpeted as being indicative of other areas of the site

Terra Associates, Inc.
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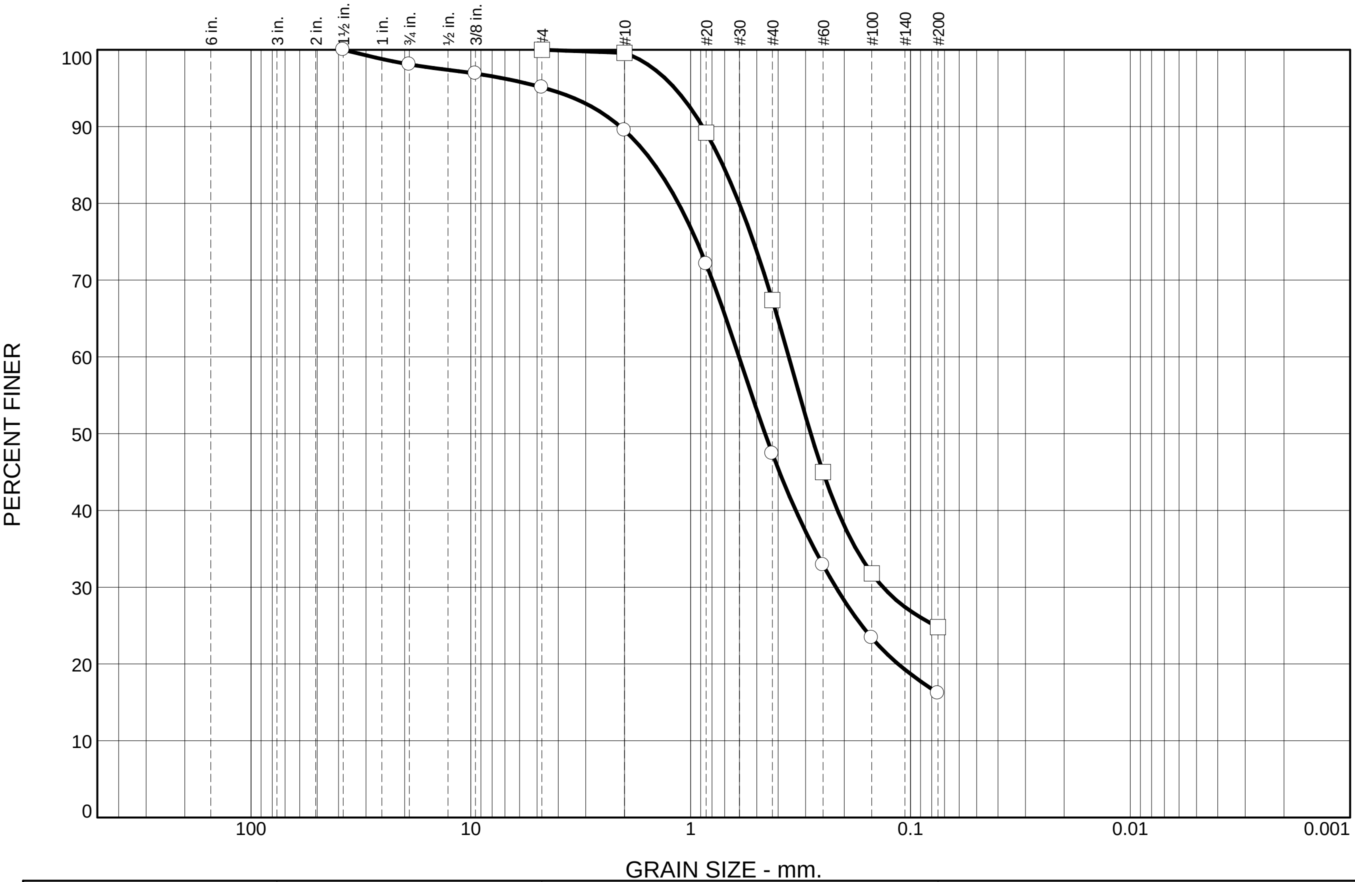
Geology and
Environmental Earth Sciences

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	1.9	3.0	5.6	42.1	31.2	16.2	
□	0.0	0.0	0.0	0.4	32.2	42.6	24.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			1.4568	0.6026	0.4587	0.2185				
□			0.7146	0.3579	0.2842	0.1333				

Material Description	USCS	AASHTO
○ silty SAND	SM	
□ silty SAND	SM	

Project No. T-8488 **Client:** Avalon Bay Communities

Project: Avalon Issaquah

○ **Location:** Test Boring B-1 **Depth:** 40 ft **Sample Number:** 12

□ **Location:** Test Boring B-2 **Depth:** 50 ft **Sample Number:** 18

Remarks:

○Tested on March 22, 2021

□Tested on March 22, 2021

Terra Associates, Inc.

Kirkland, WA

Figure A-5

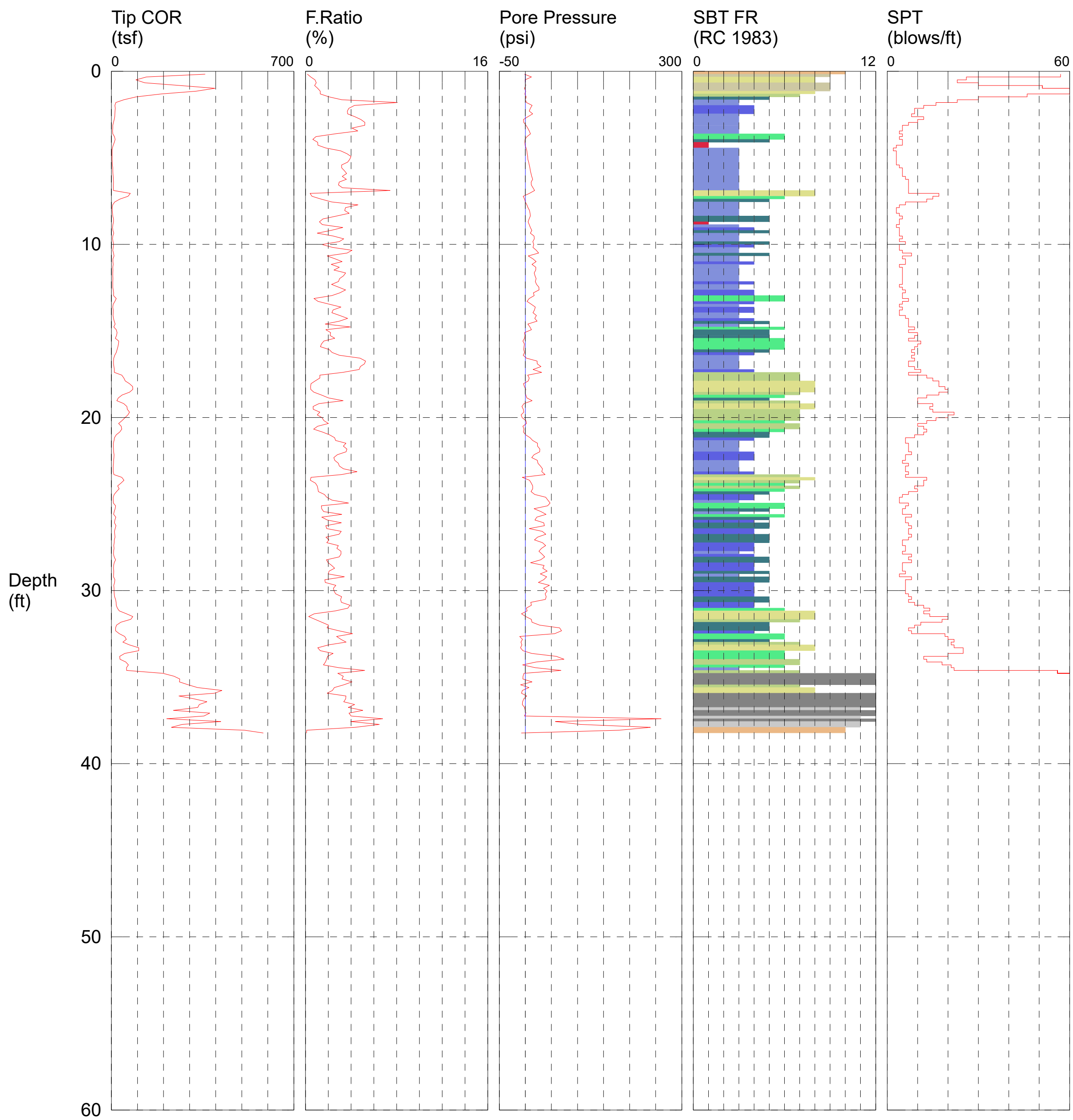
Tested By: FQ

CPT LOGS

CPT-01

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 12:08:00 PM
Predrill:
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

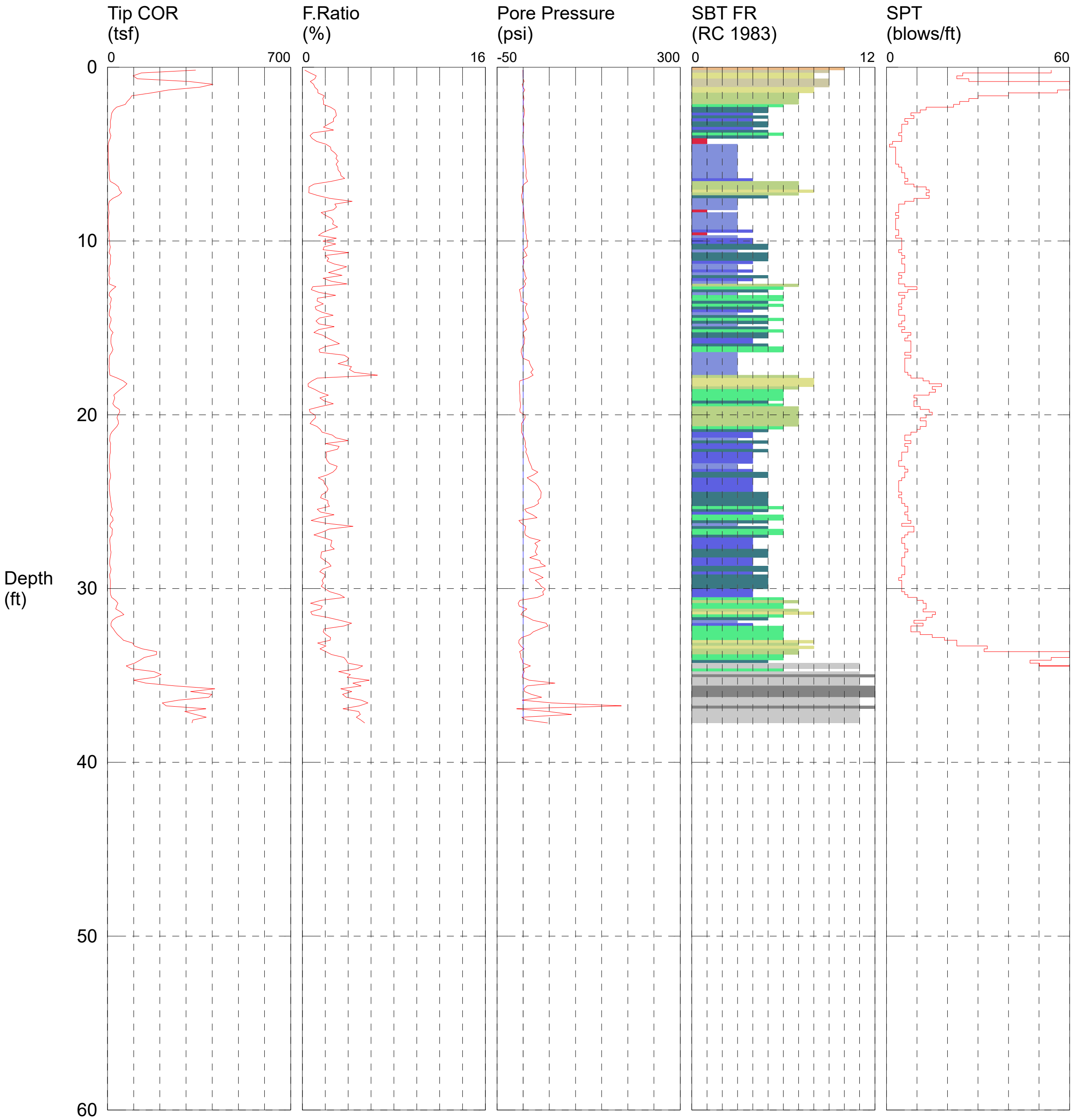
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

CPT-01A

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 1:01:28 PM
Predrill:
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

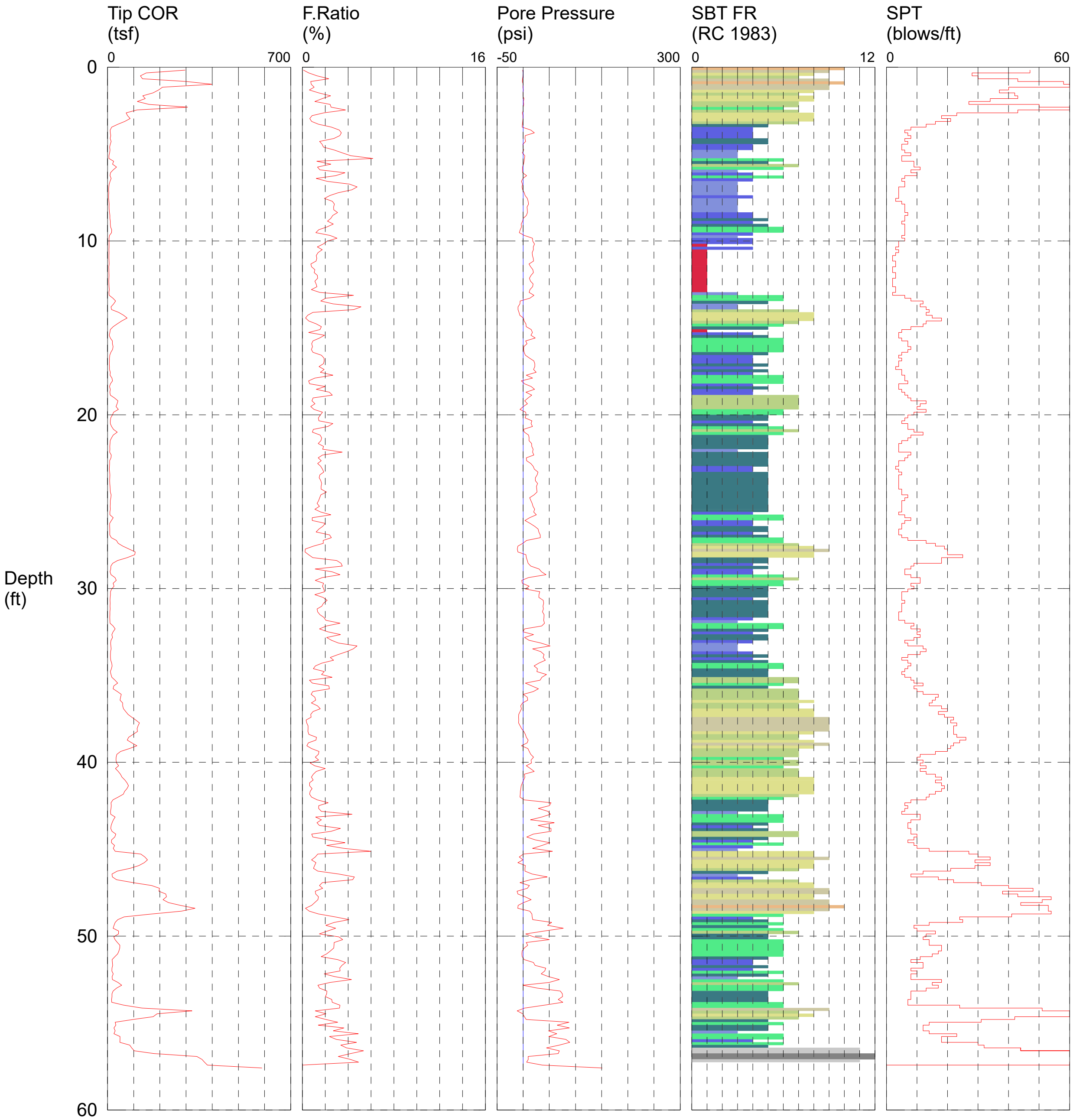
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

CPT-01.1

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 9:21:45 AM
Predrill:
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

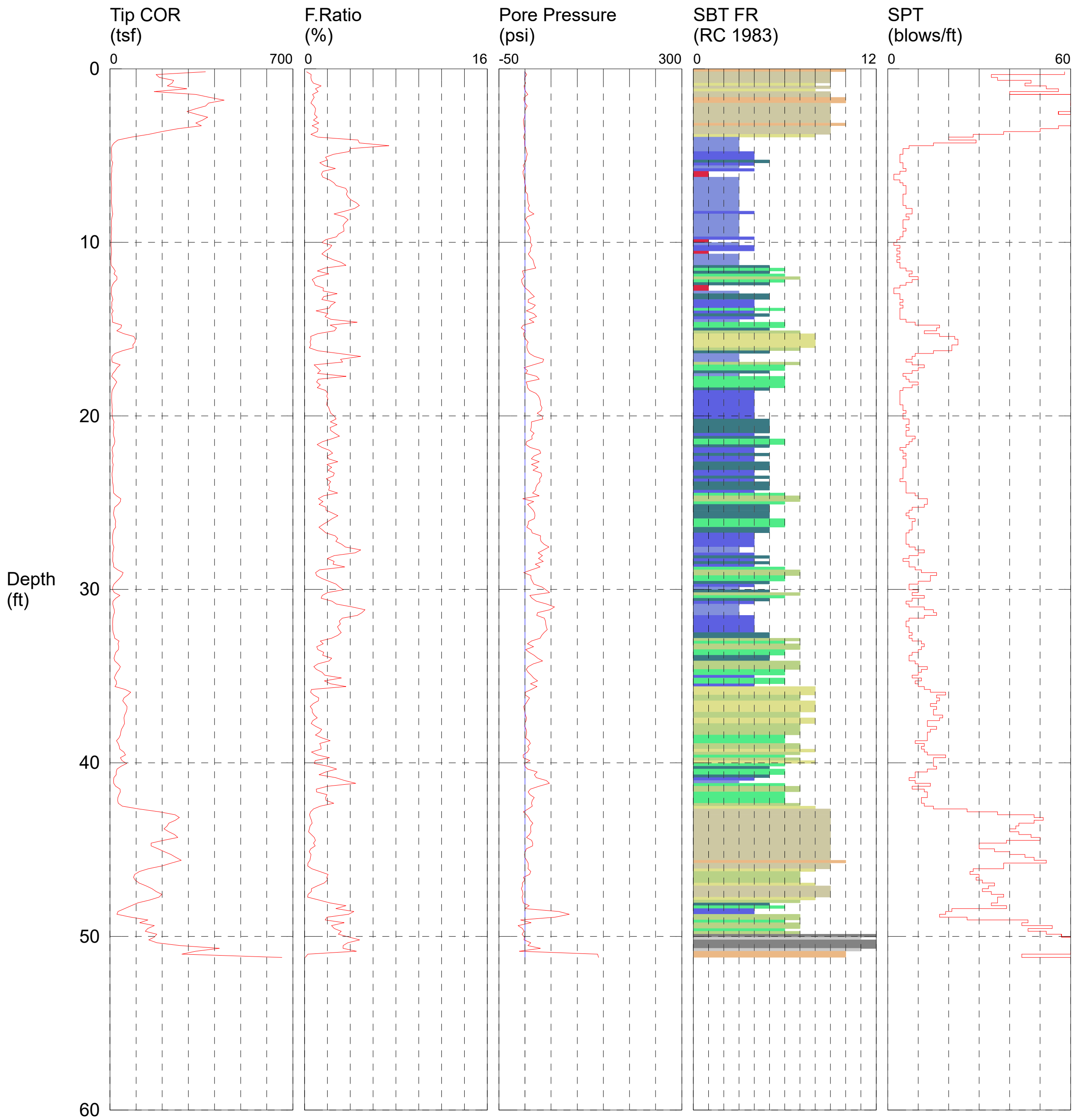
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

CPT-02

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 2:10:28 PM
Predrill:
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

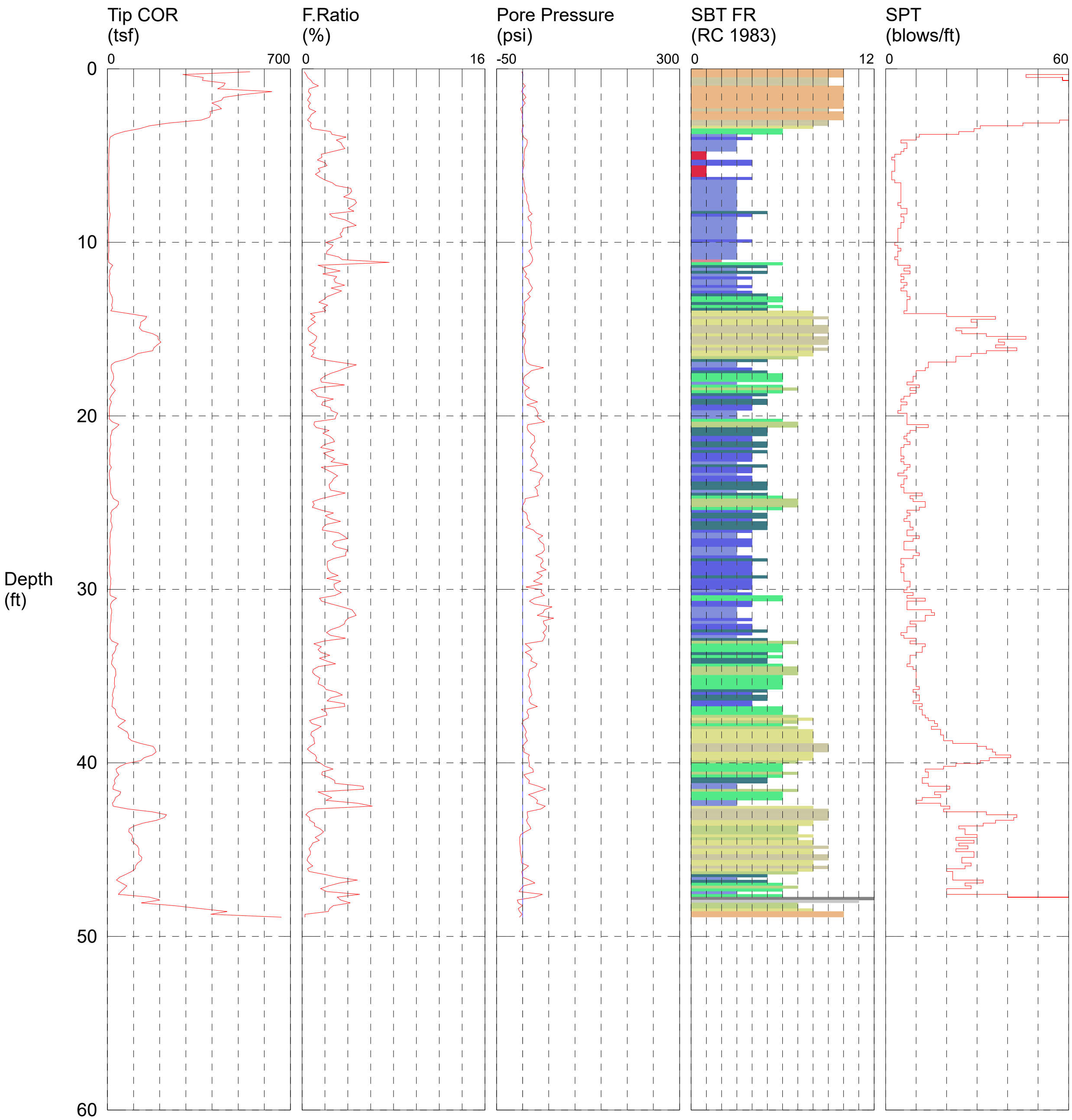
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

CPT-02A

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 7:13:02 PM
Predrill:
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

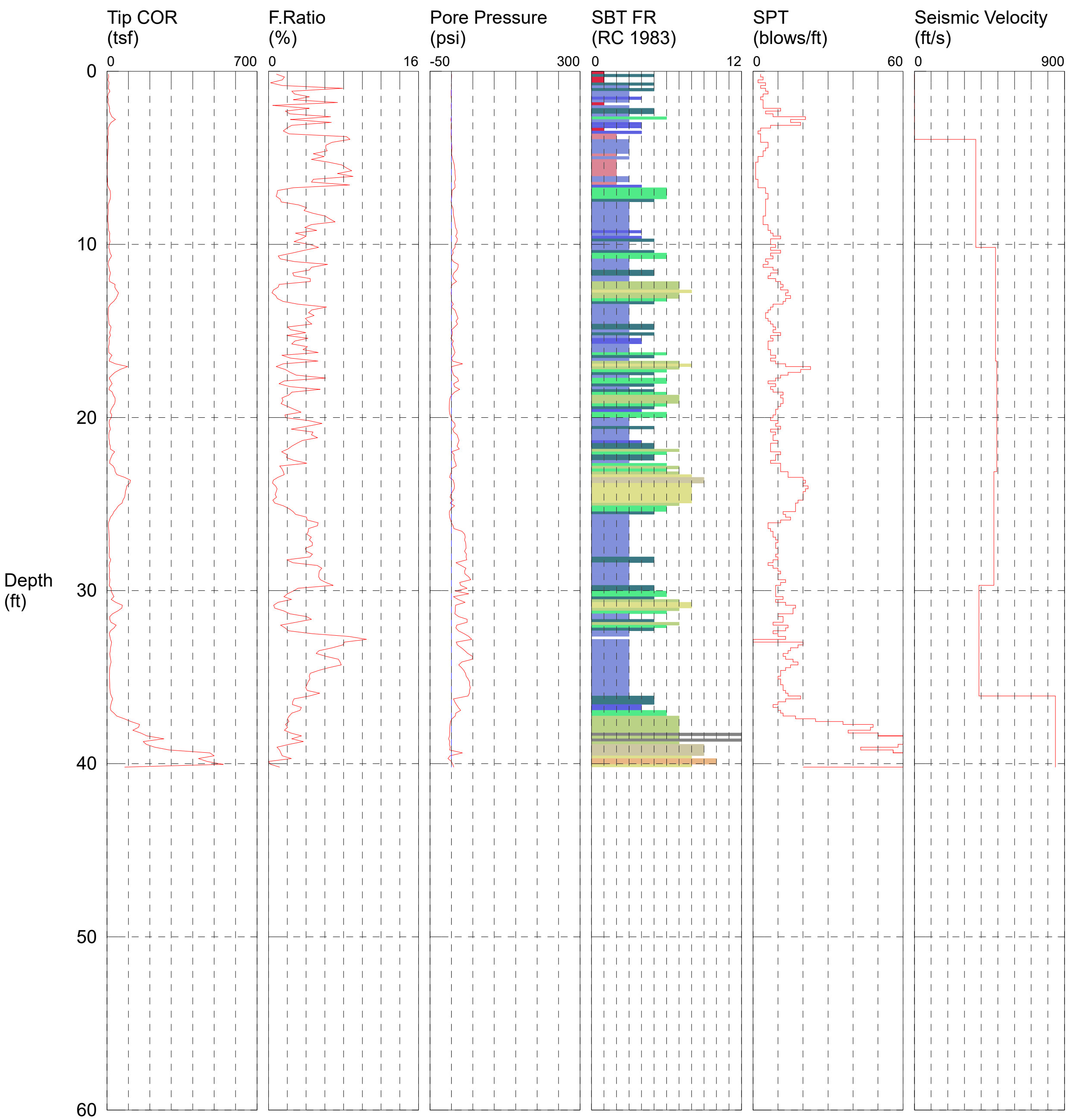
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

sCPT-03A

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 3:55:59 PM
Predrill: 4'
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

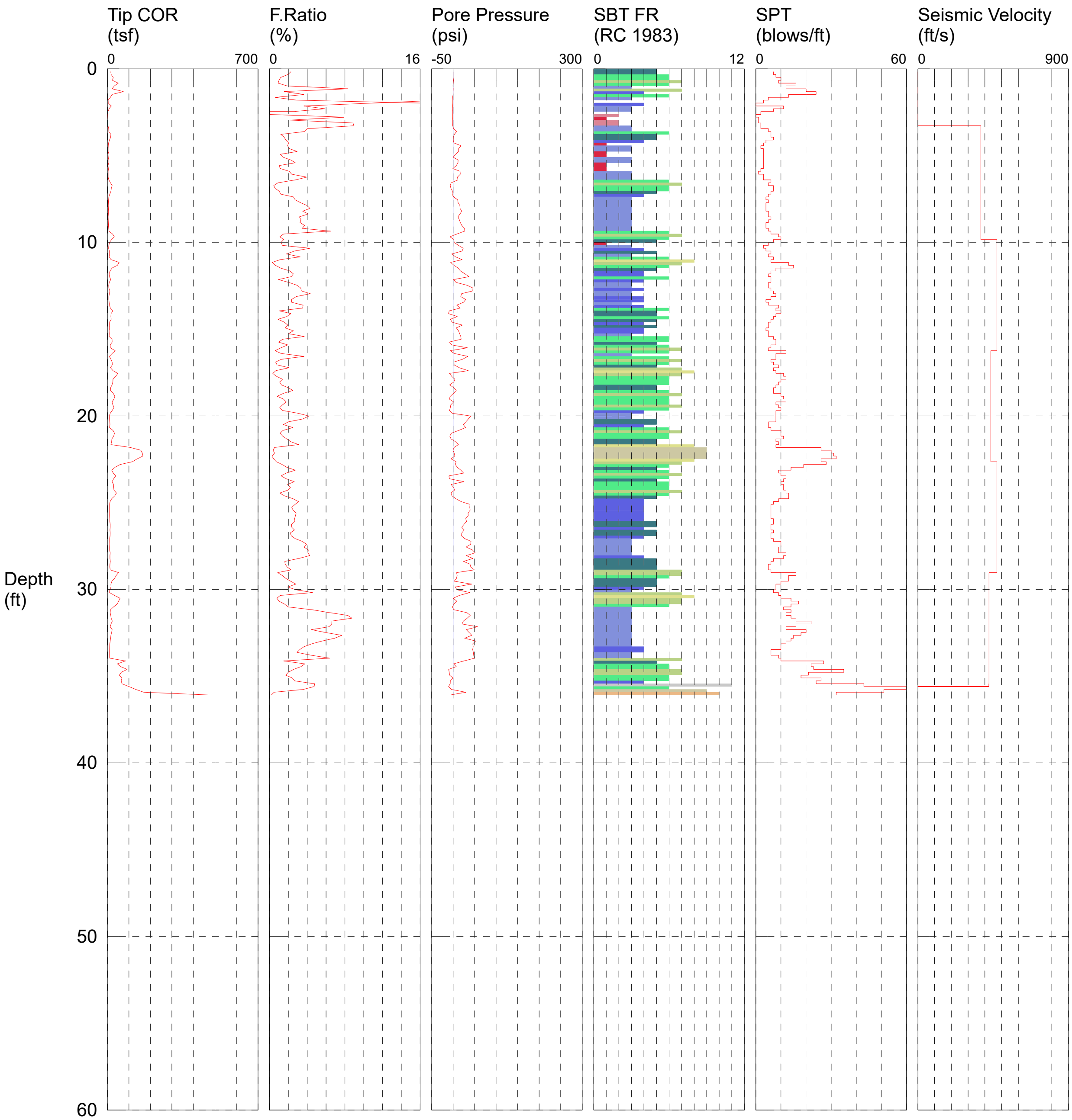
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

sCPT-03B

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 5:25:15 PM
Predrill: 4'
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

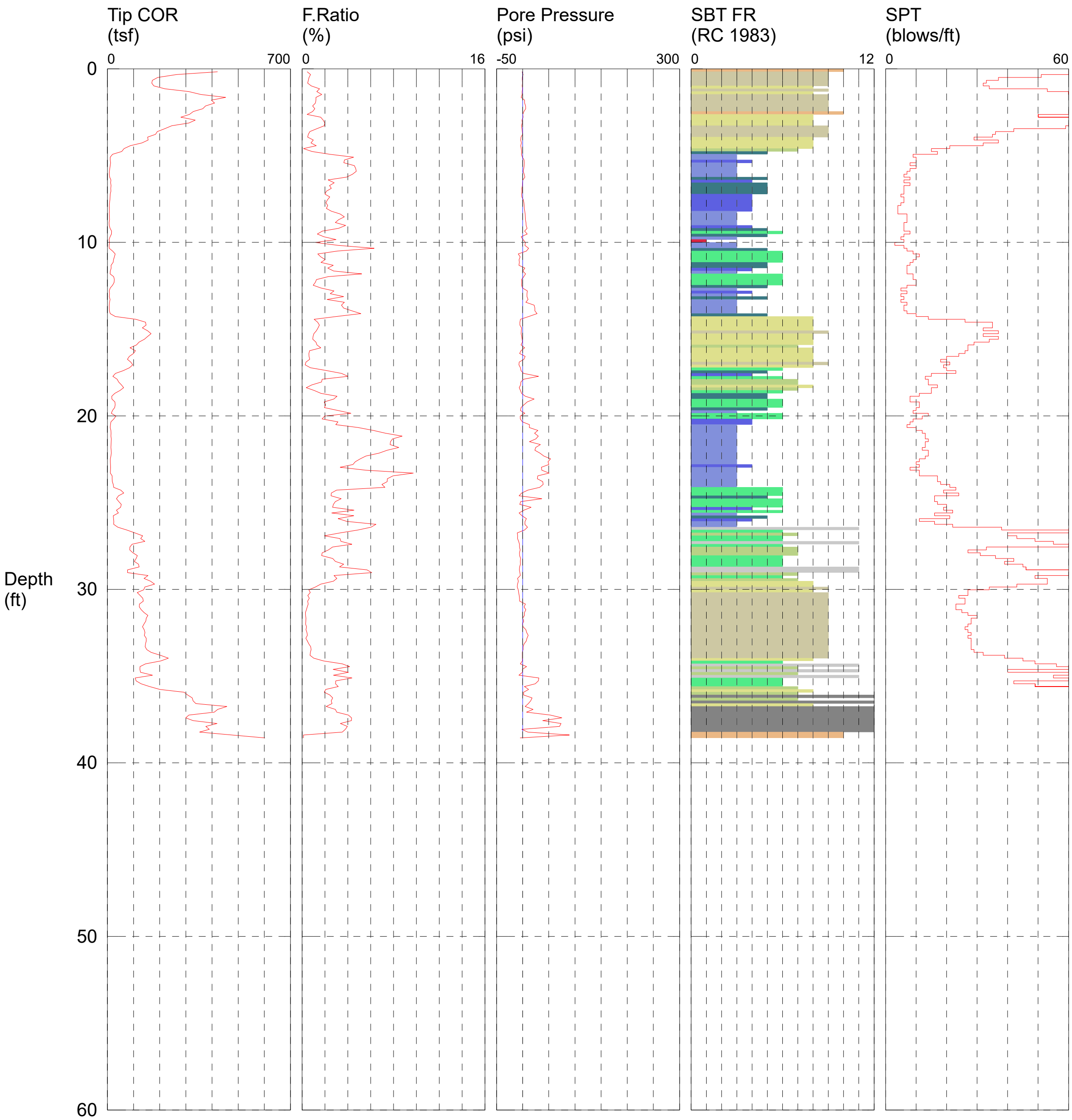
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

CPT-04

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 11:01:40 AM
Predrill:
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



COMMENT:

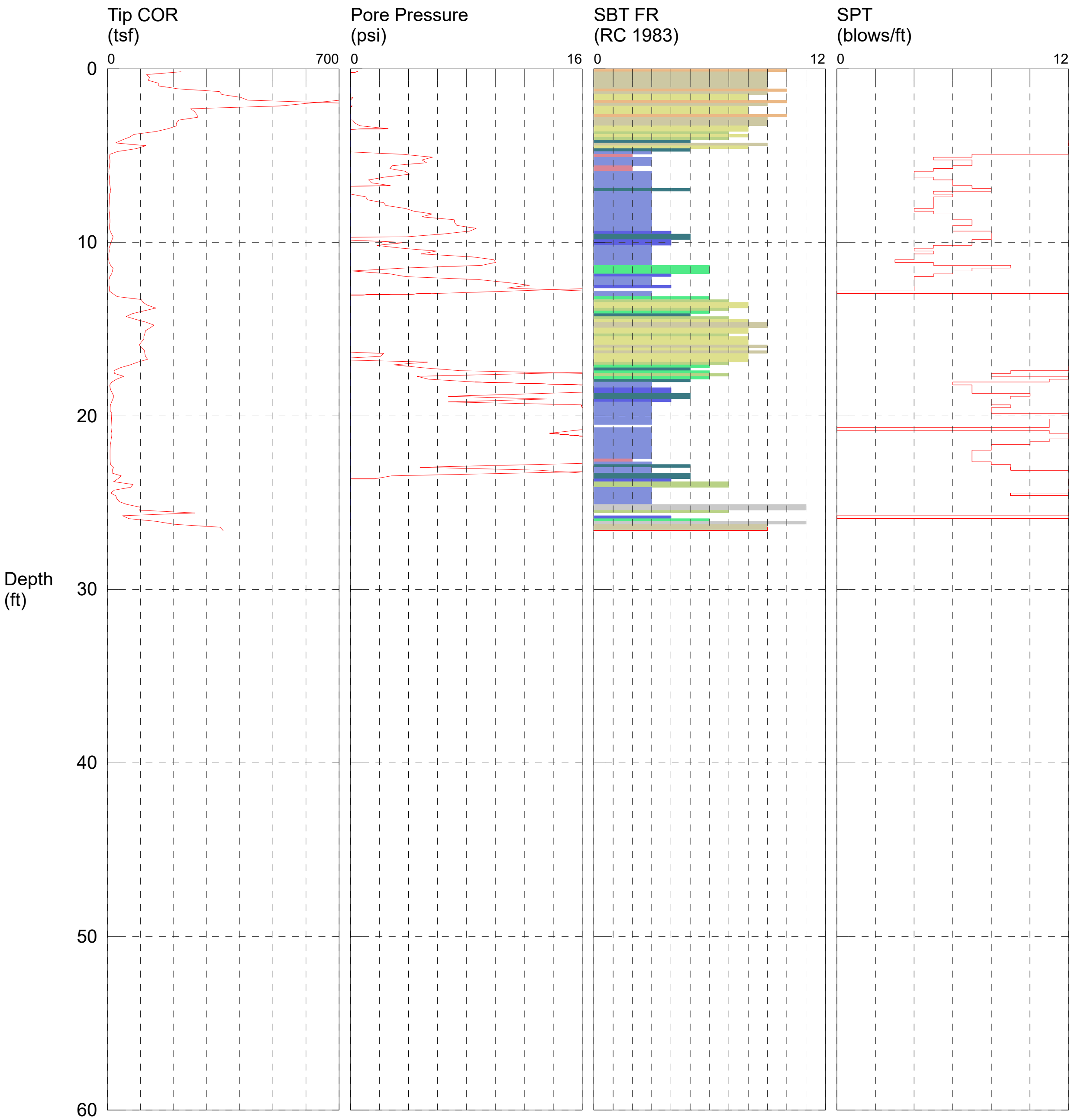
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

CPT-04A

CPT Contractor: In Situ Engineering
CUSTOMER: Terra
LOCATION: Issaquah
JOB NUMBER: T-8488

OPERATOR: Mayfield
CONE ID: DDG1369
TEST DATE: 3/1/2021 6:34:08 PM
Predrill:
Backfill: 20% Bentonite Slurry + Bentonite Chip
Surface Patch: Cold Patch



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

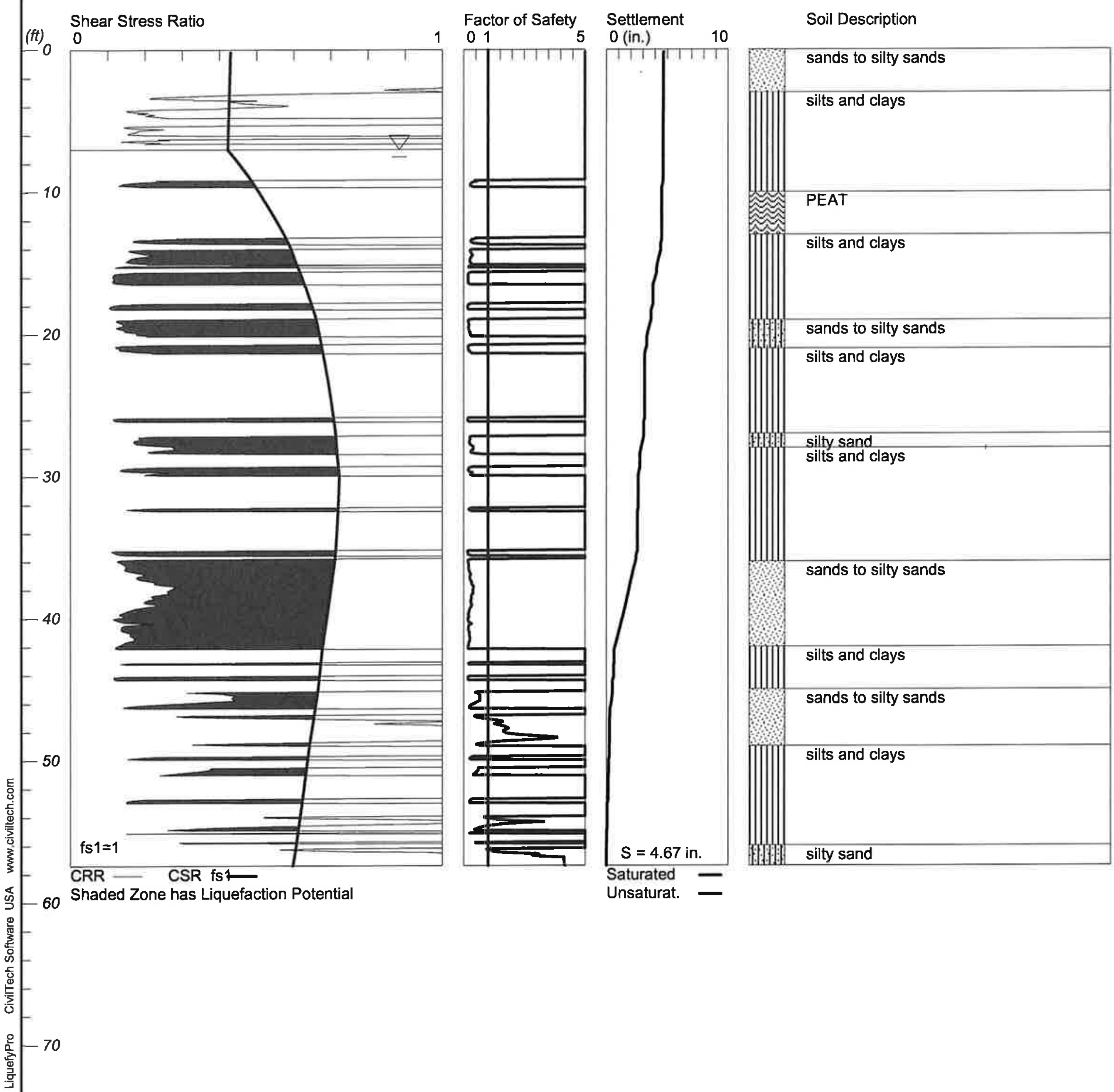
APPENDIX B
LIQUEFYPRO OUTPUT

LIQUEFACTION ANALYSIS

Avalon Issaquah

Hole No.=CPT 1.1 ***Water Depth=7 ft*** ***Magnitude=7***

Acceleration=.6619g

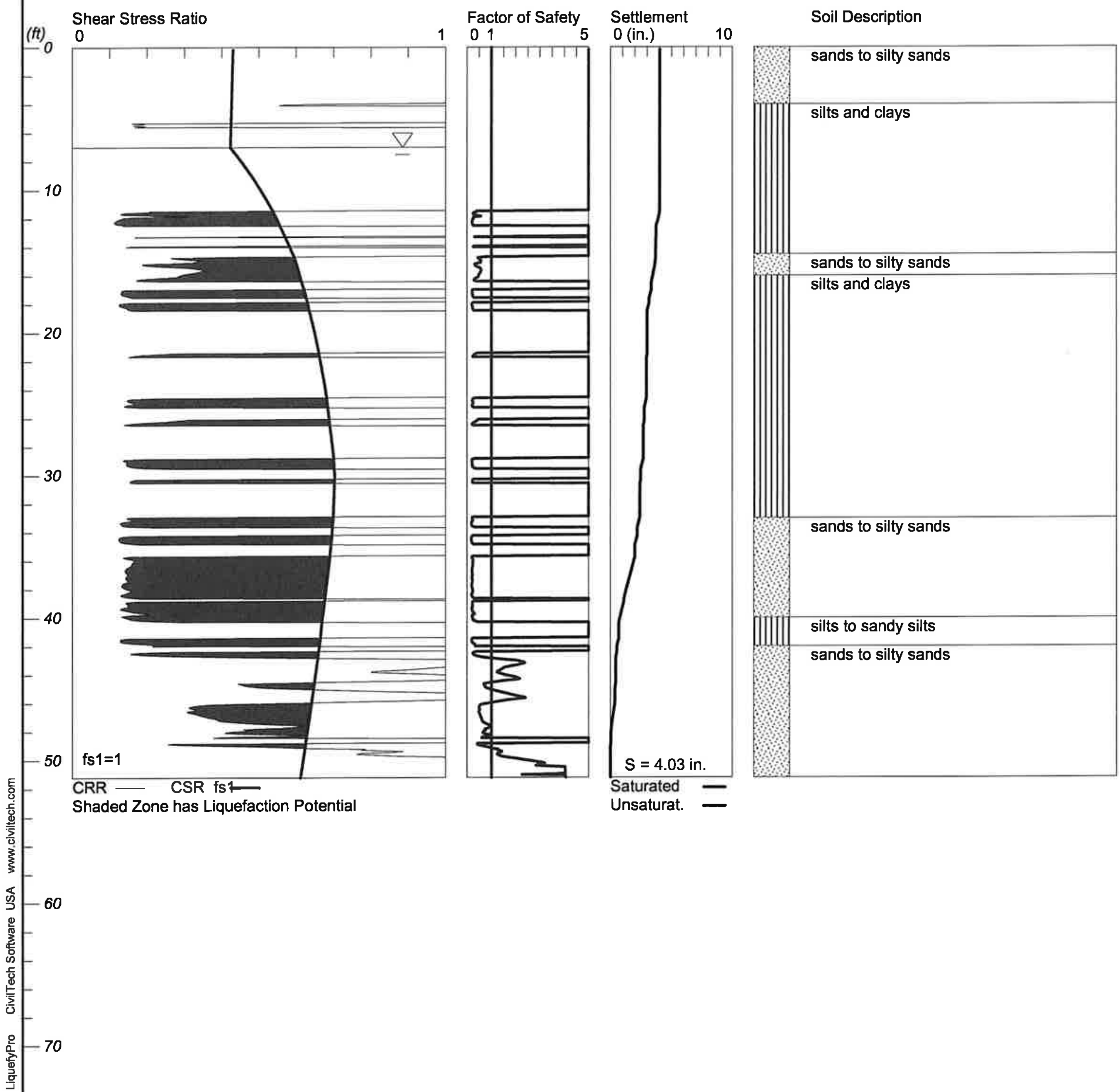


LIQUEFACTION ANALYSIS

Avalon Issaquah

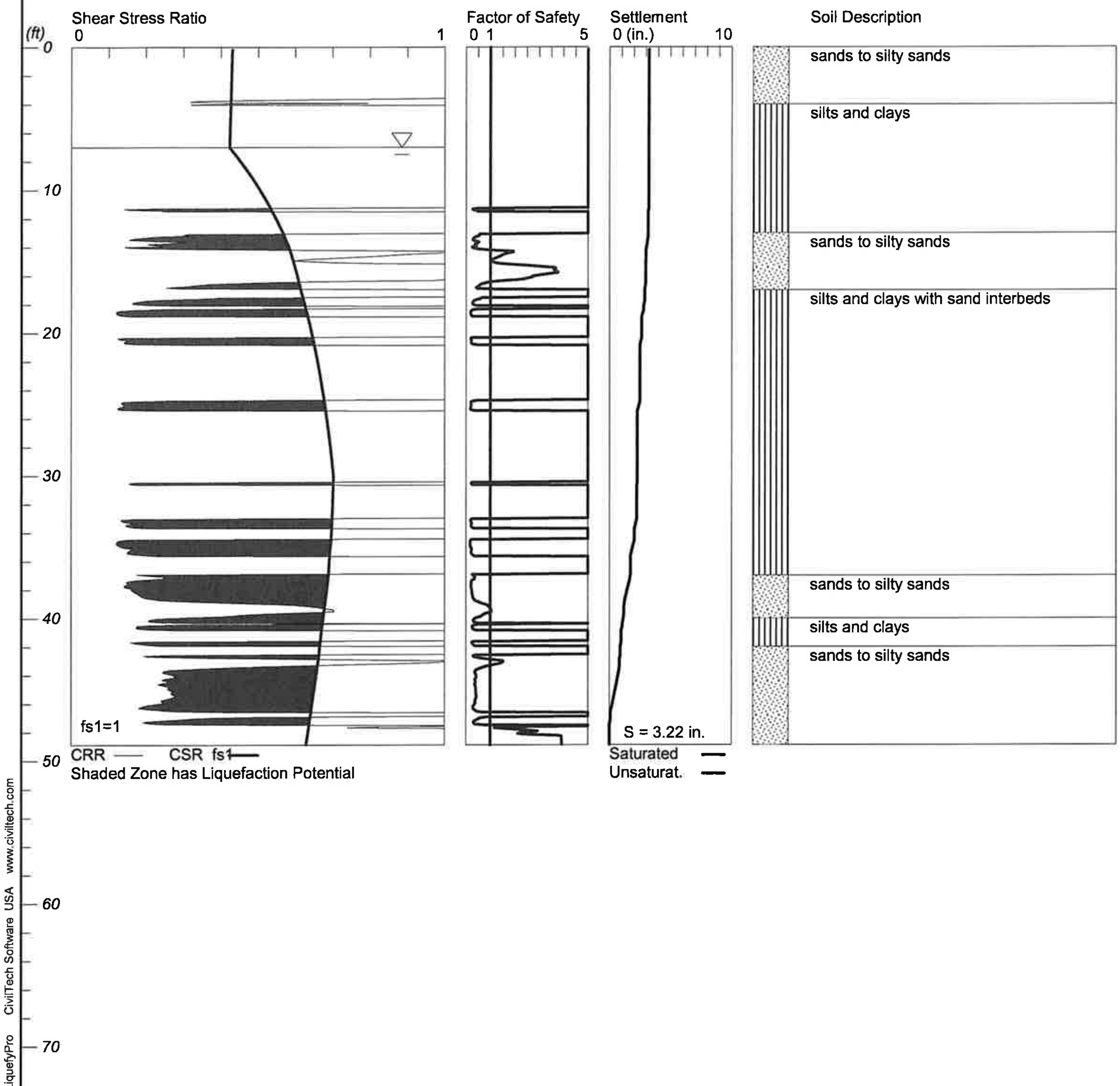
Hole No.=CPT 2 Water Depth=7 ft

Magnitude=7
Acceleration=.6619g



LIQUEFACTION ANALYSIS
Avalon Issaquah

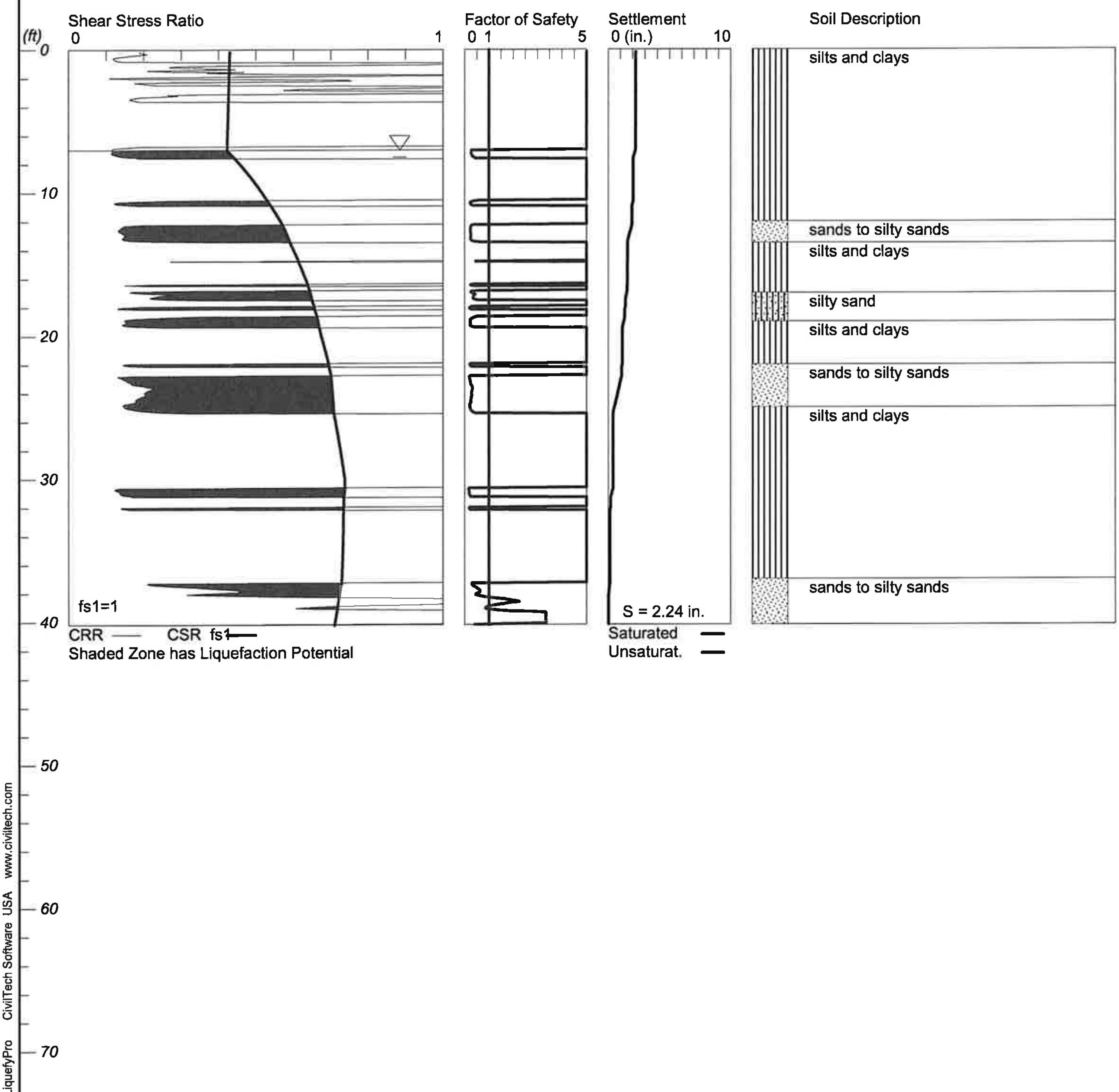
Hole No.=CPT 2A ***Water Depth=7 ft*** ***Magnitude=7***
Acceleration=.6619g



LIQUEFACTION ANALYSIS
Avalon Issaquah

Hole No.=CPT 3A   Water Depth=7 ft

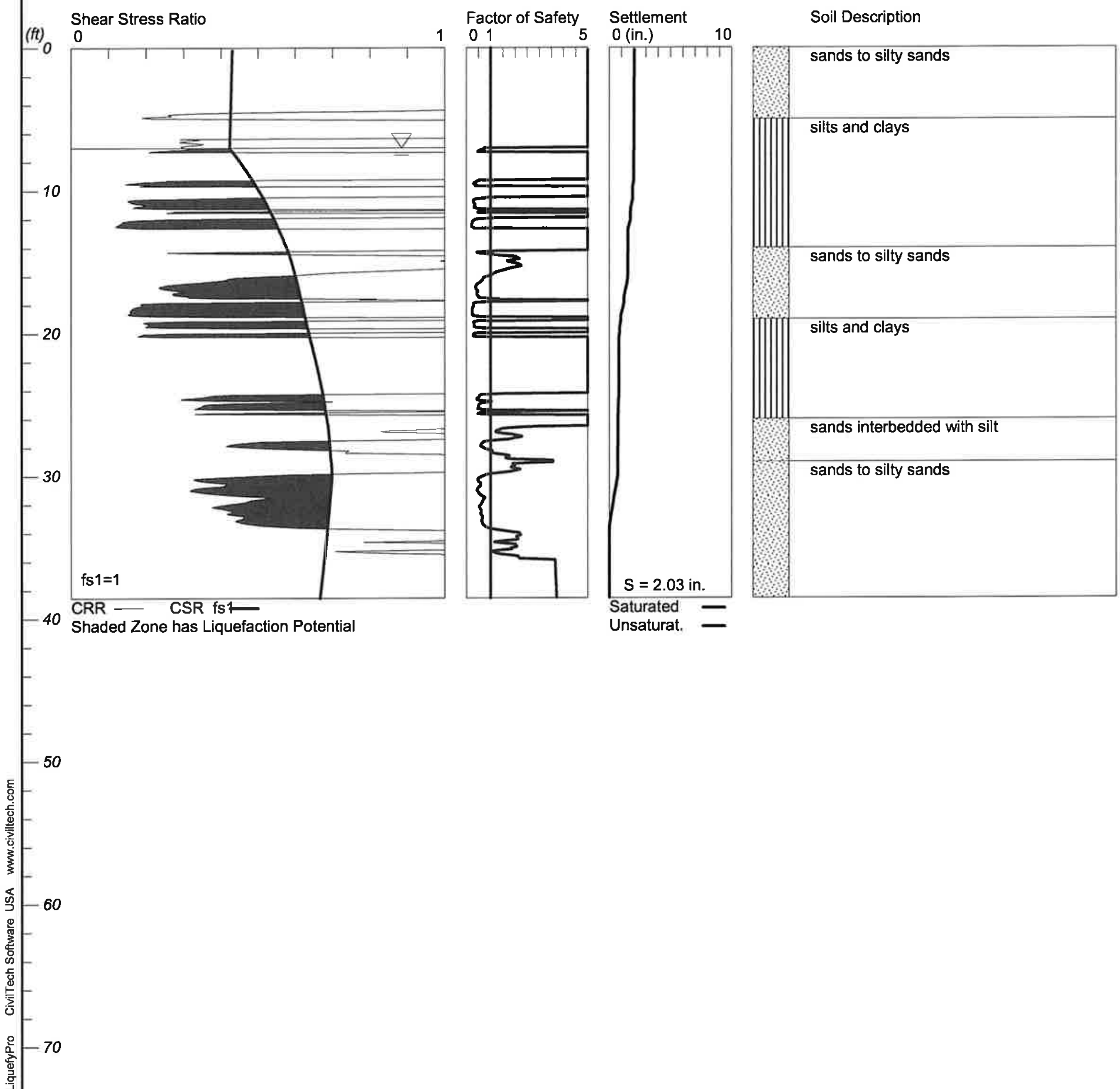
Magnitude=7
Acceleration=.6619g



LIQUEFACTION ANALYSIS
Avalon Issaquah

Hole No.=CPT 4   Water Depth=7 ft

Magnitude=7
Acceleration=.6619g



LIQUEFACTION ANALYSIS
Avalon Issaquah

Hole No.=CPT 4A   Water Depth=7 ft

Magnitude=7
Acceleration=.6619g