GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED ARCO AMPM FUELING FACILITY
1402 S MERIDIAN AVENUE
PUYALLUP, WASHINGTON

PROJECT NO. 062-22010
MAY 6, 2022

Prepared for:

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May 6, 2022 KA Project No. 062-22010

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Email: randall.arnold@sevansolutions.com
Tel: (206) 310.1851

Reference: Geotechnical Engineering Investigation
Proposed ARCO ampm Fueling Facility
1402 S Meridian Avenue
Puyallup, WA

Dear Mr. Arnold,

In accordance with your request, we have completed a Geotechnical Engineering Investigation for the referenced site. The results of our investigation are presented in the attached report.

If you have any questions, or if we can be of further assistance, please do not hesitate to contact our office.

Respectfully submitted,
KRAZAN & ASSOCIATES, INC.

Theresa R. Nunan

Theresa R. Nunan
Project Manager

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INTRODUCTION

This report presents the results of our Geotechnical Engineering Investigation for the Proposed ARCO ampm Fueling Facility located at 1402 S Meridian Avenue in Puyallup, Washington, as shown on the Vicinity Map in Figure 1. Discussions regarding site conditions are presented in this report, together with conclusions and recommendations pertaining to site preparation, excavations, foundations, structural fill, utility trench backfill, concrete slabs and exterior flatwork, drainage, erosion control, and pavements.

A site plan showing the approximate locations of the test pits is presented following the text of this report in Figure 2. A description of the field investigation and laboratory testing, as well as the test pit and Cone Penetration Test (CPT) logs, are presented in Appendix A. Appendix B contains a guide to aid in the development of earthwork specifications. Pavement design guidelines are presented in Appendix C. The recommendations in the main text of the report have precedence over the more general specifications in the appendices.

PURPOSE AND SCOPE

This investigation was conducted to evaluate the subsurface soil and groundwater conditions at the site, to develop geotechnical engineering recommendations for use in design of specific construction elements, and to provide criteria for site preparation and earthwork construction.

Our scope of services for this project was performed in general accordance with our proposal number G22018WAT dated March 24, 2022, and included the following:

- Exploration of the subsurface soil and groundwater conditions by conducting six (6) CPT borings to depths of about 27.0 to 46.3 feet below existing ground surface (bgs)using subcontracted rig and operator under the direction of a Krazan geotechnical engineer;

- Conduct two (2) small-scale Pilot Infiltration Tests (PITs), utilizing a subcontracted excavator and operator to dig the test pits and a rented water wagon for the water source;

- A Site Plan showing the CPT and PIT locations;

- Comprehensive CPT and test pit logs, including soil stratification and classification, and groundwater levels where applicable;

- Conduct laboratory testing on samples obtained from the explorations;
- Liquefaction analysis based on the data acquired from the CPTs;
- Recommendations for seismic design considerations including site coefficient and ground acceleration based on the 2018 IBC assuming that the structure will have a fundamental period of vibration equal to or less than 0.5 sec or if non-liquefiable soils are encountered in our explorations;
- Provide opinions and recommendations regarding stormwater infiltration feasibility and a design infiltration rate as per the 2014 Department of Ecology (DOE) Stormwater Management Manual for Western Washington (SWMMWW);
- Evaluation of the two (2) City of Puyallup mapped "landslide hazard" areas indicated on the Preliminary Site Plan, prepared by Barghausen Consulting Engineers, Inc. (Barghausen) dated July 8, 2021;
- Shallow foundation recommendations for the proposed structure, including allowable soil bearing pressure, anticipated settlements (both total and differential), coefficient of horizontal friction for footing design, and frost penetration depth;
- Deep foundation recommendations, if applicable based on the subsurface conditions encountered in the CPTs;
- Recommendations for design of slabs-on-grade, as well as subgrade preparation, slab drainage, capillary break, and/or moisture barriers;
- Recommendations for static and seismic active and passive lateral earth pressures for below grade and retaining structures, including surcharge loadings;
- Recommendations for structural fill materials, placement, and compaction;
- Recommendations for suitability of onsite soils as structural fill;
- Recommendations for temporary excavations including shoring;
- Recommendations for site drainage and erosion control; and
- Recommendations for asphalt and concrete pavement sections, including subgrade preparation recommendations for truck loading and pavement areas.

Environmental services, such as chemical analysis of soil and groundwater for possible environmental contaminants, were not included in our geotechnical engineering scope of services for this project.

PROPOSED CONSTRUCTION

Based on the Preliminary Site Plan, Sheet SP-5, dated July 8, 2021, and the Request for Proposal (RFP) for Geotechnical Services document dated March 10, 2022, which were prepared by Barghausen, we understand that the proposed development will include construction of a 3,349 square foot, single-story ampm building at the northern end of the site, a canopy fuel island structure with eight multi-product dispensers (MPDs) in the middle of the site, with underground storage tanks planned south of the fuel island, and a 24-foot by 28-foot car wash structure located at the southern end of the site. Other site improvements include paved access drives and parking areas, paved entry driveways from S Meridian Avenue, landscaped areas, and installation of associated utilities.

We understand a typical dead load reaction of 4 kips and live load reaction of 16 kips is anticipated for each canopy column, and independent pier foundations at each column are preferred for support of the canopy structure. Although no loading information was provided for the ampm building or the carwash structure, we have assumed typical column and wall loads for these structures will not exceed 30 kips and 3 kips per lineal foot, respectively, for our soil bearing capacity and settlement analyses. We have also assumed that the existing site grades are at or within a foot of the planned finish grades.

SITE LOCATION AND DESCRIPTION

The subject property consists of four parcels (APNs 770000021, -31, -281, and -288) that encompass 1.18 acres of land located at 1402 S Meridian Avenue in Puyallup. The site is bordered by Highway 512 to the north, and entry drive and commercial development to the south, S Meridian Avenue to the east, and commercial development and Highway 512 to the west. Historical aerial photos indicate the site was agricultural farmland from at least 1940 to around the mid-70's. The existing one-story restaurant building was constructed in 1976 based on parcel information presented on the Pierce County Parcel and Property Information web portal. The remainder of the site is asphalt paved parking areas and access drives, with the exception of the northernmost portion of the site which served as gravel surfaced overflow parking. Numerous underground utilities are located within the site, and especially within the utility corridor transecting the southern half of the gravel-surfaced lot in an east-west direction.

We have reviewed the Land Title Survey, prepared by Barghausen, dated April 19, 2022. The site is relatively level with the ground surface generally sloping east to west, and ranging from Elev. 47 to 49 feet. The land surrounding the general vicinity of the site is generally higher in elevation and slopes towards the project site. There is an isolated slope in the southeast corner of the site, at the access drive to the site from S Meridian Ave., which is roughly 8 feet in height and has an inclination of about 30 degrees (58 percent). This slope is partially supported by stacked rock boulders that showed signs of erosion and instability. There is another isolated slope near the northwestern property line (outside the site boundary, Highway 512 off-ramp embankment), which is roughly 7 feet in height and has an inclination of about 14 degrees (25 percent). Signs of significant erosion or slope instability were not observed along the northwestern slope during our site visit. A drainage ditch is situated between Hwy 512 and the northern side of the site. Water was observed over a portion of this drainage ditch to a depth of 1-foot or less during our field work on March 28, 2022.

Two existing monitoring wells were observed on the property. One monitoring well is located within the northeastern portion of the gravel lot, and a second monitoring well (DOE # BJI 189) is located in the paved parking area south of the existing building.

GEOLOGIC SETTING

The site lies within the Puget Lowland, a north-south trending depression bounded by the Cascade Mountain Range in the east, and the Olympic Mountains in the west. The surficial geology of the Puget Lowland has been shaped by glacial activity that deposited sediments during numerous cycles of advance and retreat over the past 2 million years.

The Washington Department of Natural Resources (DNR) Geologic Information Portal website indicates that the property is located in an area that is predominantly underlain by Quaternary alluvium (Qa) consisting of "unconsolidated or semiconsolidated alluvial clay, silt, sand, gravel, and (or) cobble deposits; locally includes peat, muck, and diatomite". The southern portion of the site, extending south from about the southern side of the existing restaurant building, is mapped as Continental Glacial Drift (Qgd) consisting of "till and outwash clay, silt, sand, gravel, cobbles, and boulders deposited or originating from continental glaciers; locally includes peat, nonglacial sediments, modified land, and artificial fill".

FIELD INVESTIGATION

Six (6) Cone Penetration Tests (CPTs) were completed to evaluate the subsurface soil and groundwater conditions at the project location. The CPTs were conducted on March 30, 2022, using a subcontracted test rig and operator under the direction of a Krazan geotechnical engineer. The CPTs, designated CPT-1 through CPT-5 and CPT-2B, were advanced to depths of 27.0 to 46.3 feet bgs. The CPT method consists of pushing an instrumented cone into the ground at a controlled rate and recording measured soil parameters, such as tip resistance, friction ration, and pore pressure. In addition, shear wave testing was also conducted every 3 feet in CPT-2B, CPT-4, and CPT-5. These measured parameters are used to determine geotechnical engineering properties of the soils encountered and to delineate soil stratigraphy, particularly for use with seismic and liquefaction analyses, and to develop seismic design parameters. Soil samples are not obtained with cone penetration testing.

Infiltration Testing: Two infiltration test pits, designated IP-1 and IP-2, were excavated at the site on March 28, 2022, at the locations indicated on the Site Plan, Figure 2, to conduct small scale PITs. Test pits IP-1 and IP-2 were excavated to depths of 7.1 and 4.7 feet bgs and to a bottom area of 18.5 and 13.0 sf, respectively. The subsurface soil and groundwater conditions encountered in the test pits are described in the following section of this report. Based on the subsurface conditions encountered, infiltration testing was not conducted in the test pits or at any other location on the site.

A detailed description of the field investigation is presented in Appendix A. The logs for the CPTs depict soil stratigraphy based on published correlations of the measured cone tip resistance and side friction with soil types. The test pit and CPT logs are also included in Appendix A. The approximate locations of the test pits and CPTs are shown on the Site Plan in Figure 2.

SOIL PROFILE AND SUBSURFACE CONDITIONS

Our field investigation exposed undocumented fill underlain by native alluvial and glacial soil deposits to the termination depths of the test pits and CPT explorations. The relative density and/or consistency of the soils described below are based on either observation of the excavation effort of the equipment used to conduct the test pits, or on the measured tip resistances of the cone for the CPTs.

Asphalt Pavement and Undocumented Fill: CPT-4, CPT-5, and IP-2 were conducted within the paved areas of the site and encountered 3 to 3.5 inches of asphalt pavement underlain by 6 to 7.5 inches of moist, brown, silty sand (SM) with gravel base course material. Up to roughly 3 feet of undocumented fill was encountered beneath the base course material and at the ground surface in the remaining explorations.

Native Alluvial and Glacial Soils: The undocumented fill was underlain by highly compressible, very soft to medium stiff organic silt, peat, sandy silt, and clay followed by very loose to medium dense sand with varying silt content to a depth of about 20 to 23 feet bgs. The compressible alluvial soils ranged from about 2 feet thick in CPT-2 and CPT-2B to up to 9.5 feet thick in CPT-1 conducted within the northeastern portion of the site, to occasional layers up to 1-foot thick in the explorations conducted within the southern part of the site (CPT-3, CPT-4, and CPT-5).

An approximately 12-foot thick layer of dense to very dense sand with gravel to gravel with sand was encountered beneath the loose alluvial sands in CPT-2 and CPT-2B, and extended to depths of 27 to 33 feet bgs in the remaining CPTs due to refusal of the cone to further penetration in this dense soil layer.

The dense sand in CPT-2 and CPT-2B was underlain by another stratum of very loose to medium dense alluvial sand ranging from about 5.5 to 12 feet thick, followed by dense to very dense glacial sand and gravel soils to their termination depths at about 39.1 and 46.3 feet bgs, respectively.

Groundwater: Porewater pressure dissipation tests conducted on March 30, 2022 in the CPTs indicated a groundwater level ranging between 1.2 to 3.7 feet bgs. Shallow groundwater was also encountered in the test pits; however, after waiting 3 hours the water level was still rising so the test pits were backfilled for safety reasons. Two monitoring wells installed by others, one near CPT-1 and the other near CPT-4, indicated water levels at 4.6 and 1.5 feet bgs. A manhole cover for the communications line at the northeast side of the site was removed during our March 28, 2022 site visit and the water level was measured at a depth of about 5.5 feet bgs.

It should be recognized that groundwater elevations generally fluctuate with time. The groundwater level will be dependent upon seasonal precipitation, irrigation, land use, climatic conditions, as well as other factors. Therefore, groundwater levels at the time of our field investigation may be different from those encountered during the construction phase of the project. The evaluation of such factors was beyond the scope of this report. Design and operation of temporary dewatering systems to remove or lower groundwater to facilitate construction should be the responsibility of the contractor.

The subsurface soils encountered in the test pits and CPTs were in general agreement with the mapped geology for the project area. Groundwater conditions were consistent with the available DOE well data in the site vicinity.

Shear Wave Velocity: Shear wave velocity were obtained from the CPT-2B, CPT-4, and CPT-5, which were advanced to depths of about 27.0 to 46.3 feet bgs. The shear wave velocities were measured to the maximum explored depth, and we have assumed similar site conditions continue below the explored depth. The measured shear wave velocities to the maximum explored depth ranged from about 333 feet per second to 1680 feet per second. The average measured shear wave velocities in the upper 100 feet were estimated to be in the range of 778 to 1217 feet per second.

GEOLOGIC HAZARDS

Erosion Concern/Hazard

The USDA Natural Resources Conservation Services (NRCS) map for Pierce County Area, Washington (WA653), classifies the soils in the site area as Shalcar muck (38A), 0 to 1 percent slopes. These soils are formed from organic material over alluvium deposited in flood plains, and are considered very poorly drained. The typical shallow soil profile consists of muck and peat over silty clay and fine sandy loam. The NRCS Soil Survey indicates that the Shalcar muck soils belong to Hydrologic Soil Group D, whereby surface runoff is slow, and the erosion hazard is very low due to flowing water or wind. The majority of the site is presently gravel-surfaced or asphalt paved, with the sloping ground along the northern, southern, and eastern sides of the property covered with grass, landscaping, and trees. Measures to address potential erosion during construction are presented in the Erosion and Sediment Control (ESC) section of this report.

Steep Slope Hazard

Review of the City of Puyallup Hazards Map website indicate that there is an isolated slope in the northwestern corner of the site, which has been mapped as moderate susceptibility to deep seated landslide. There are slopes near the southeastern portion of the site that have been mapped as moderate susceptibility to deep seated landslide as well. During our site visit we did not observe signs of recent slide scarps, tension cracks, or slumps within the site that would indicate current deep-seated instability on the slopes within or near the property. Signs of shallow soil movement and soil creep, such as curved tree trunks, were not observed on either of the slope areas. Based on our exploration and surficial site reconnaissance, it is our opinion the mapped landslide hazard areas should not have an adverse effect on the proposed site development or vice-versa.

Although the southeastern slope does not show signs of shallow or deep-seated hazard, this man-made embankment does show signs of construction-related issues with regard to erosion and instability. Rock boulders in a sand matrix appear to support the southern slope embankment from the corner near the intersection of S Meridian Ave. extending westward. Loose sand was noted between some of the rock boulders and a steel T-probe was able to penetrate to a depth of at least 3.5 feet bgs, while voids were noted at other locations between the rock boulders. Signs of erosion were evident in the bare section of the embankment, and it appears rebar rods have been inserted into the ground near the top of slope at this location possibly as a measure to hinder lateral movement. We recommend the erosion and instability concerns for this constructed embankment slope be addressed by either 1) re-constructing the access road embankment from its intersection with S Meridian Ave. down to the site level or 2) injecting high strength grout into this portion of the embankment through a series of horizontal and vertical holes. All bare areas should then be properly vegetated following remediation of this portion of the southeaster slope.

Seismic Hazard

The 2018 International Building Code (IBC), Section 1613.3.2, refers to Chapter 20 of ASCE 7-16 for Site Class Definitions. The site soil conditions encountered in CPT-2B, CPT-4 and CPT-5 correspond to "Site Class F" based on their liquefaction potential and, therefore, require a site-specific response analysis as per Section 20.3.1 of ASCE 7-16, unless the structure's fundamental period of vibration is equal to or less than 0.5 seconds. <u>We have assumed that the structure will have a fundamental period of vibration of equal to or less than 0.5 seconds.</u> Therefore, a site response analysis was not performed. Based on this exception, the site class was determined as per Section 20.3 of ASCE 7-16. The spectral accelerations were determined as per Sections 11.4.4 and 11.4.5 of ASCE 7-16.

The mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response parameters for short periods and at 1 second (S_S *and* S_1) were obtained from the Applied Technology Council (ATC) Hazards website, which utilizes the most updated published data on seismic conditions from the United States Geological Survey. The site coefficients (F_a *and* F_v) for "Site Class D" were selected based on the estimated average shear wave velocity of 1217, 778, and 899 feet per second in the upper 100 feet of cone penetration tests CPT-2B, CPT-4, and CPT-5, respectively. The spectral response acceleration parameters (S_{MS}, S_{DS}, S_{M1}, S_{D1}) and short period (*Ts*) were determined as per Sections 11.4.4. 11.4.5, and 11.4.6 of ASCE 7-16. The seismic design parameters for this site are based on a Risk Category II for the proposed structure and are presented in Table 1:

Table 1: Seismic Design Parameters*

(Reference: 2018 IBC Section 1613.2.2, ASCE 7-16, and ATC)

Seismic Item	Value
Site Coefficient F_a	1.000
S_s	1.268
S_{MS}	0.1.268
S_{DS}	0.846
Site Coefficient F_v	1.863
S_1	0.437
S_{M1}	0.814
S_{D1}	0.543
Ts	0.642

*Based on Equivalent Lateral Force (ELF) Design Procedure being used.

Note: If the structure's fundamental period of vibration exceeds 0.5 seconds, a site response analysis will be required, which is beyond the scope of this report.

Additional seismic considerations include liquefaction potential and amplification of ground motions by loose/soft soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. Soil liquefaction is a state where soil particles lose contact with each other and become suspended in a viscous fluid. This suspension of the soil grains results in a complete loss of strength as the effective stress drops to zero. Liquefaction normally occurs under saturated conditions in soils such as sand in which the strength is purely frictional. However, liquefaction has occurred in soils other than clean sand. Liquefaction usually occurs under vibratory conditions such as those induced by seismic events.

We have reviewed the Washington DNR Geologic Information web-portal interactive map, the liquefaction Susceptibility Map of Pierce County, Washington (Palmer et al., 2004), and the USDA Soil Survey Map (WA653) with regards to soils and liquefaction susceptibility. The maps indicate that the site is underlain by alluvial soils with the surface soils generally consisting of Shalcar muck (an organic, peat type soil). The Shalcar muck is not susceptible to liquefaction but may experience large displacements during an earthquake event. The alluvial soils are highly susceptible to liquefaction. The Hazard Zones are based on the combined effects of ground shaking amplification, liquefaction, and earthquake-induce landslides. At the request of our client, we have conducted a site-specific liquefaction analysis for this project.

To evaluate the liquefaction potential of the site, we analyzed the following factors:

1) Soil type
2) Groundwater depth
3) Relative soil density
4) Initial confining pressure
5) Maximum anticipated intensity and duration of ground shaking

Liquefaction Analysis: The commercially available liquefaction analysis software, NovoCPT from NovoTech, was used to evaluate the liquefaction potential and the possible liquefaction induced settlement for the site soil and groundwater conditions based on our explorations. The analysis was performed using the information from seismic cone penetration tests CPT-2B and CPT-5. The Maximum Considered Earthquake (MCE) was selected in accordance with the 2018 International Building Code (IBC) Chapter 16 and the U.S. Geological Survey (USGS) Earthquake Hazards Program website. For this analysis, a maximum earthquake magnitude of 7.1 and peak horizontal ground surface acceleration of 0.70g were used.

We ran our analyses for groundwater at a depth of 1-foot bgs during the earthquake. Our analyses indicated that the soils from the depth that groundwater was encountered to about 14 feet bgs were liquefiable under the maximum earthquake magnitude of 7.1. The maximum liquefaction induced settlement for this type of seismic event is estimated to be on the order of approximately 1.3 to 2.4 inches (total settlement). The dynamic differential settlement is estimated to be on the order of about ¼ to 1-inch over 50 feet.

The CPT data revealed two zones of liquefiable soils at the site. The upper zone encountered interbedded liquefiable layers ranging from 1 to 4 feet thick between a depth of about 4 to 21.5 feet bgs. A second deeper zone contained frequent liquefiable soil layers up to 1-foot thick from a depth of about 33 to 43 feet bgs. The deeper liquefaction zone accounted for roughly sixty percent of the total dynamic settlement.

Liquefaction-induced lateral spreading is lateral displacement of gently sloping ground as a result of pore pressure build-up or liquefaction in shallow deposits during an earthquake. The conditions conducive to lateral spreading include gentle surface slope, shallow water table, and liquefiable cohesionless soils. Based on the relatively shallow groundwater level and sand soils encountered in the explorations, about 4 to 10 inches of lateral spreading could occur as a result of a 7.1 magnitude earthquake event.

The liquefaction analysis plots showing the factor of safety, vertical settlement, and lateral displacement are presented in Appendix A.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion from a geotechnical standpoint that the site is compatible with the planned development, **provided that the geotechnical recommendations presented in this report are included in the project design and implemented during construction**.

Our field explorations at this site encountered very loose to medium dense sands with varying silt content, as well as highly compressible, very soft to soft organic silt/peat (Shalcar muck), clay, and sandy silt soils to a depth of about 23 feet bgs. These soils are considered unsuitable bearing soils for support of the proposed ampm building on a shallow foundation system. In addition, our liquefaction analyses indicated that the soils within the upper 21.5 feet of the site, as well as the soils encountered in a deeper zone between a depth of roughly 33 to 43 feet bgs, are liquefiable under a maximum earthquake magnitude of 7.1. The maximum liquefaction induced settlement for this type of seismic event is estimated to be on the order of approximately 1.3 to 2.4 inches (total settlement), with dynamic differential settlement estimated to be on the order of about ¼ to 1-inch over 50 feet. Therefore, a deep foundation system is recommended for support of the proposed ampm building. A shallow foundation system may be considered for the fuel canopy and car wash structures, provided a portion of the unsuitable soils are over-excavated and replaced with structural fill and the risks associated with seismic-induced settlement are deemed acceptable. Recommendations for shallow and deep foundations are presented in the Foundations section of this report.

Due to the shallow groundwater level and very loose to medium dense soils encountered at the proposed location of the USTs, temporary dewatering and shoring of the excavation sidewalls is anticipated to allow for installation of the tanks.

The subsurface soils encountered on this site during our field exploration are considered extremely moisture-sensitive and may disturb easily in wet conditions. We recommend that construction take place during the drier summer months, if possible. In our opinion, the onsite undocumented fill and native soils are considered unsuitable for re-use as structural fill, and the cost to import structural fill should be included in the project budget.

Stormwater Infiltration

The City of Puyallup Municipal Code has adopted the 2014 (DOE) SWMMWW. The SWMMWW references the small-scale PIT for field infiltration testing. We excavated two test pits, IP-1 and IP-2, at the site to conduct infiltration testing. However, due to the presence of undocumented fill material, organic silt/peat (Shalcar muck) and clay, and shallow groundwater, field infiltration tests were not conducted. Based on the subsurface soil and groundwater conditions encountered at the site, it is our opinion that onsite management of stormwater by infiltration is not considered feasible.

Site Preparation

General site clearing should include removal of topsoil material, asphaltic concrete, abandoned utilities, and structures including foundations, slabs, rubble, and trash, down to native suitable soils. In addition, any buried structures, such as grease traps, septic tanks, underground storage tanks, debris pits, cesspools, or similar structures, should be completely removed and backfilled with structural fill.

The undocumented fill and the native very loose sands and very soft to medium stiff organic silt/peat, clay, and sandy or clayey silt encountered in our field explorations are considered unsuitable for support of the ampm building, fuel canopy structure, car wash structure, floor slabs and exterior slabs-on-grade, and pavement loads. Based on the shallow groundwater levels encountered in our explorations conducted in March 2022, temporary dewatering measures will likely be required to conduct the over-excavation of unsuitable soils, especially if construction takes place during the “wet weather” season.

We recommend the undocumented fill and unsuitable native soils be over-excavated to a depth of at least 2 feet below the footing bearing level for shallow foundations or the planned subgrade elevation for slabs-on-grade or pavements. Deeper excavations may be required if soft and yielding soil conditions are exposed at the bottom of the over-excavation. A layer of rock spalls should be placed on the excavation bottom and tamped in-place to provide a stable working surface for placement of structural fill. We recommend a high-strength geotextile separation fabric, such as Mirafi 600X or equivalent, then be placed over the rock spalls. After the fabric is placed, the area should be filled to the planned pavement subgrade elevation with structural fill. The structural fill should be compacted to at least 95 percent of the maximum dry density (ASTM D1557) and to within 2 percent of the optimum moisture content. In-place density tests should be performed to verify proper moisture content and adequate compaction levels are achieved in the structural fill.

An existing restaurant building is located within the eastern central portion of the property where the Canopy and fuel pumps are planned, and extends into part of the proposed area of the future USTs. The debris from demolition of the existing building should be hauled off-site. As-built records for the existing building were not available at the time of this report. Assuming the restaurant is supported on a shallow foundation system, then existing concrete footings should be completely removed within the footprint of the canopy structure, and to a depth of at least 1-foot below the planned subgrade elevation in new pavement or exterior slab-on-grade areas. If the existing building is pile supported, the type and location of the piles will need to be evaluated prior to or during construction as information becomes available to determine if the piles should be left in-place, or partially or completely removed.

Krazan & Associates should be onsite full-time during the demolition activities to document that all below-grade structures have been properly removed and backfilled with properly placed and compacted structural fill, and that the resulting debris from the demolition activities has been hauled off-site and not re-used as fill at any location on the property.

All existing utilities should be completely removed from within planned structure areas. For any utility line to be considered acceptable to remain, i.e. be abandoned in-place, within the structure footprint, the utility line must be completely filled with grout or sand-cement slurry, the ends outside the building area capped with concrete, and the existing trench backfill removed and replaced with properly placed and compacted structural fill. Assessment of the level of risk posed by a particular utility line to the structure will determine whether the utility may be abandoned in-place or needs to be completely removed. The risks associated with abandoning utilities in-place include the potential for future differential settlement of existing trench fills and/or potential ground loss into utility lines that are not completely filled with grout if the abandonment requirements stated above are not followed.

Based on our field explorations, the near surface soils expected to be encountered at the site during construction are considered extremely moisture sensitive and will likely disturb easily in wet conditions. During wet weather conditions, subgrade stability problems and grading difficulties may develop due to the excess moisture, disturbance of sensitive soils, shallow groundwater levels, and/or the presence of perched groundwater. Construction during extended periods of wet weather could result in the need to remove wet disturbed soils if they cannot be suitably compacted due to elevated moisture contents. The prepared subgrade should be protected from construction traffic and surface water should be diverted around the prepared subgrade. Soils that have become unstable may require over-excavation, or drying and recompaction. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry, warm weather (typically during the summer months). If the soils cannot be dried back to a workable moisture condition, removal of the unstable soils or the use of remedial measures may be required. These remedial measures could include placement of a blanket of rock spalls to protect the exposed subgrade and construction traffic areas. The lateral extent and depth of rock spalls, if required, should be determined based on evaluation of the near surface soil conditions at the time of construction.

General project site winterization should consist of the placement of aggregate base and the protection of exposed soils during the construction phase. It should be understood that even if Best Management Practices (BMP's) for wintertime soil protection are implemented and followed there is a significant chance that moisture disturbed soil mitigation work will still be required.

A representative of our firm should be present during all site clearing and grading operations to test and observe earthwork construction. This testing and observation are an integral part of our services, as acceptance of earthwork construction is dependent upon compaction and stability of the material. The geotechnical engineer may reject any material that does not meet compaction and stability requirements. Further recommendations, contained in this report, are predicated upon the assumption that earthwork construction will conform to the recommendations set forth in this section and in the Structural Fill Section.

Dewatering

Excavations will be required for installation of the USTs and site utilities, as well as for over-excavations required for construction of the slabs-on-grade, pavements, and structures supported on shallow foundations. Based on the anticipated excavation depths and the shallow groundwater level encountered at the site, the excavations will extend below the groundwater table and thus require some method of dewatering.

Sump pit and pumping methods may be able to handle groundwater encountered in shallow excavations depending on the time of year construction takes place, the planned excavation depth, and the soils encountered within the excavation. The test pits conducted for this exploration encountered groundwater as shallow as 1.5 feet bgs, and cave-in of the pit sidewalls occurred in the very loose to loose soils at about the level groundwater was encountered.

Deeper excavations, such as for installation of the USTs, will require more a more aggressive dewatering method, such as well points. To maintain the stability of the excavation bottom, groundwater levels should be drawn down a minimum of 2 feet below the lowest portion of the excavation. The groundwater level should be maintained below the recommended level until the backfill has been placed and compacted.

Analysis of contractor dewatering needs or the design of contractor dewatering systems was not within the scope of our services. A competent dewatering contractor should provide these services. However, we have included some discussion of potential dewatering methods in the following paragraphs. Krazan and Associates should review the contractor's dewatering design for consistency with the geotechnical recommendations contained in this report.

The method of dewatering ultimately selected is dependent on a number of factors, e.g. quantity of groundwater to be removed, cone of depression (zone of influence) of dewatering measures within the excavation, stability of the undocumented fill and native soils, the presence of seepage zones, and cost to name a few.

Lowering the water table could induce settlements of the dewatered and underlying soils. The dewatering engineer should evaluate the potential for dewatering-related settlement, and mitigation measures should be taken, as necessary. If structures or utilities are located within the anticipated cone of depression, groundwater levels, settlement, and deflections at and near the structure or utility should be monitored during dewatering to observe if the groundwater level is changing and movement is occurring. Dewatering should stop and appropriate corrective action should be taken if settlement or changes in groundwater levels are noted at these locations.

Temporary Excavations

The onsite soils have variable friction and cohesion strengths, therefore the safe angles to which these materials may be cut for temporary excavations is variable, as the soils may be prone to caving and slope failures in temporary excavations deeper than about 2 feet or at the level where groundwater is encountered. Temporary excavations in the fill material and underlying native soils should be sloped no steeper than 2H:1V (horizontal to vertical) where room permits. Depending on site soil and groundwater conditions, it may be necessary to flatten the side slopes of the excavation and lower the groundwater level as necessary to achieve stable conditions. Slope cuts into excavations greater than 20 feet in depth should be designed by a professional engineer for the contractor.

All temporary cuts should be in accordance with Washington Administrative Code (WAC) Part N, Excavation, Trenching, and Shoring. The temporary slope cuts should be visually inspected daily by a qualified person during construction work activities and the results of the inspections should be included in daily reports. The contractor is responsible for maintaining the stability of the temporary cut slopes and minimizing slope erosion during construction. The temporary cut slopes should be covered with plastic sheeting to help minimize erosion during wet weather and the slopes should be closely monitored as the area is backfilled.

A Krazan & Associates geotechnical engineer should observe, at least periodically, the temporary cut slopes during the excavation work. The reason for this is that all soil conditions may not be fully delineated by the limited testing at the site. In the case of temporary slope cuts, the existing soil conditions may not be fully revealed until the excavation work exposes the soil. Typically, as excavation work progresses, the maximum inclination of the temporary slope will need to be evaluated by the geotechnical engineer so that supplemental recommendations can be made. Soil and groundwater conditions can be highly variable. If any variations or undesirable conditions are encountered during construction, Krazan & Associates should be notified so that supplemental recommendations can be made.

Underground Storage Tanks (USTs)

The specific plans for installation of the two new tanks were not available at the time of this report. However, we have assumed installation of the new tanks will generally follow the Underground Storage Tank Standards Element TP01 V-14.0 2019 Series Core drawings prepared by Barghausen Consulting Engineers, Inc. and dated January 25, 2019. Based on these drawings and side by side tank installations, we anticipate the excavation will extend to a minimum depth of about 16 to 20 feet bgs. We anticipate excavations for fuel lines, vent lines, and other utilities will generally be less than 4 feet deep. Therefore, some type of temporary shoring system will be necessary to support the excavation sidewalls. Due to the high groundwater level encountered at the site and the very loose to medium dense soils to be retained, we do not recommend the use of a soldier pile retaining wall system for support of the UST Excavation. Recommendations for a temporary sheet pile shoring system are provided below.

Lateral Earth Pressures: The parameters presented in Table 2 may be used for design of a temporary shoring and/or bracing system.

Table 2 - SOIL PARAMETERS FOR TEMPORARY SHORING DESIGN

Material Description	Depth (ft.)	Angle of Internal Friction (degrees)	Cohesion (psf)	Moist Unit Weight (pcf)	Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)
Soil Layer 1: very loose to medium dense Sands	0 - 22	22	0	105	0.45	2.20
Soil Layer 2 (Native Soils): Dense to very dense Silty Sand, Gravelly Sand, or Sandy Gravel	22 to 33	40	50	135	0.22	4.60

The temporary shoring should be designed to resist the full hydrostatic pressure over the entire depth of the excavation. The excavation support system may also be subjected to surcharge loads due to construction equipment, storage of materials, temporary storage of the tanks near the excavation, or loading of the tanks onto trucks for transport offsite. We recommend the temporary shoring system be designed for a uniform lateral surcharge pressure of 300 pounds per square foot (psf) to account for these surcharge loads. In addition, outriggers for cranes may impose point loads adjacent to the excavation and these loads should be included in design of the shoring system. The shoring design should also consider loads from any structures, foundations, or existing utilities located within the zone of influence, which is taken as a 1 Horizontal to1 Vertical (1H:1V) line projected upwards from the bottom of the excavation. Excavations for installation of the USTs will require dewatering as discussed in the previous section of this report.

The temporary sheet pile retaining wall should be designed by an experienced structural engineer licensed in the state of Washington. In many cases, the contractor may have qualified structural engineers on board, or have a working relationship with qualified wall designers. In any case, the wall designer should be provided a copy of our report, and we should be retained to review the geotechnical aspects of the shoring wall design prior to construction.

If the shoring wall is allowed to yield at the top at least one thousandth of the height of the above ground portion of the wall, the wall should be designed for an active loading condition. If the wall is restrained from yielding by external bracing, tiebacks, or wall stiffness, the wall should be designed for an at-rest loading condition. Active or at-rest pressure acting on the cantilevered sheet piles should be calculated based on a triangular pressure distribution using the soil parameters provided in Table 2. Single- or multiple-braced walls should be designed using a trapezoidal earth pressure distribution. A factor of safety of 1.5 should be applied to the calculated passive resistance.

Our explorations did not encounter boulders. However, boulders may be present in glacial soils and may cause obstruction. Additionally, there may be obstructions in unexplored areas of the site. The contractor should be prepared to penetrate or remove obstructions if they are encountered.

***Dewatering*: -** Porewater pressure dissipation tests conducted in the CPTs indicated groundwater levels at the time of testing in March 2022 at a depth of 1.5 to 3.7 feet bgs. Installation of monitoring wells, piezometers, or conducting slug tests to evaluate site specific groundwater levels and pumping rates for dewatering analysis was not included in our scope of services for this project. Analysis of contractor dewatering needs or the design of contractor dewatering systems was also not within the scope of our services.

Excavation Subgrade: - Based on the referenced standard tank drawings, we understand that the new tanks will bear on a minimum of 12 inches of pea gravel placed over the native soils. Based on the CPT results, the soils at the anticipated excavation bottom will likely consist of dense to very dense sand and gravel soils. The contractor should be prepared to remove any accumulations of soft soils due to standing water in the excavation prior to placement of the pea gravel base layer. Any over-excavation to remove soft soils should be backfilled with pea gravel meeting the requirements of the Structural Fill section of this report.

Construction Considerations: - The excavation and backfilling activities associated with installation of the new tanks may cause ground movement. Prior to conducting the excavation activities, a pre-construction survey should be conducted on existing structures within a horizontal distance of at least 17 feet from the edges of the excavation. The pre-construction survey should include elevation measurements as well as photos of the existing structures. Additional elevation measurements should be obtained at a reasonable frequency, but not less than once per week, to monitor movements during the excavation and backfilling process.

The new tanks should be designed to resist hydrostatic uplift forces. Concrete deadmen with straps could be utilized to provide additional uplift resistance for the fuel tank system.

Utility Trenches and Backfill

Excavations of up to 4 feet in depth are anticipated to install utilities associated with the new fuel tanks. Deeper excavations may be required to install site utilities. The temporary excavations for installation of utilities should follow the recommendations of the Temporary Excavations section of this report.

All utility trench backfill should consist of structural engineered fill as per the Structural Fill section of this report. The onsite undocumented fill and native soils are considered unsuitable for re-use as trench backfill. Trench backfill lifts should be placed in equal measures on each side of the utility pipe to the top of the pipe. Trench backfill lifts should not exceed 8 inches in loose thickness prior to compaction, with the exception that the first lift placed over the pipe may be up to 14 inches in loose thickness. Each lift of trench backfill should be moisture conditioned to within 2 percent of its optimum moisture content and compacted to the required relative density prior to placement of additional fill lifts.

A firm and unyielding subgrade (i.e. bearing soils at bottom of trench) should allow for the proper placement of subsurface utilities. If unstable soils are encountered at the utility trench bottom, we recommend placement of geotextile and quarry rock (rock spalls) on the bottom of utility trenches prior to placement of pipe bedding to provide a stable subgrade for placement of the pipe bedding, utility, and trench backfill. The thickness of the rock spall layer will depend on the instability of the subgrade soils at the time of excavation. Pipe bedding should be in accordance with the pipe manufacturer's recommendations.

Utility trench backfill placed within or adjacent to buildings and exterior slabs should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. It is recommended that utility trenches located within the building pad be compacted, as specified above, to minimize the transmission of moisture through the utility trench backfill. The upper 5 feet of utility trench backfill placed in pavement areas should be compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557. Below 5 feet, utility trench backfill in pavement areas should be compacted to at least 90 percent of the maximum dry density based on ASTM Test Method D1557. Pipe bedding should be in accordance with the pipe manufacturer's recommendations.

The contractor is responsible for removing all moisture-sensitive soils from the trenches regardless of the backfill location and compaction requirements. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction.

Structural Fill

Fill placed beneath foundations, pavement, or other settlement-sensitive structures should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician under the direction of the geotechnical engineer. Field monitoring procedures would include the performance of a representative number of in-place density tests on the soils to document the attainment of the desired degree of relative compaction and moisture content. The area to receive the fill should be suitably prepared as described in the Site Preparation subsection of this report prior to beginning fill placement.

Best Management Practices (BMP's) should be followed when considering the suitability of the existing materials for use as structural fill. Based on our field exploration, the undocumented fill and native soils that will be encountered within roughly the upper 10 feet during site development are considered unsuitable for re-use as structural fill material due to their high fines content (percent silt and/or clay material passing the No. 200 Sieve), as well as organic content for the Shalcar muck encountered in our explorations. These soils are considered extremely moisture-sensitive and will likely disturb easily in wet conditions. Also, debris was observed in the undocumented fill within the test pits.

An allowance for importing structural fill should be incorporated into the construction cost of the project. If deeper excavations, such as for installation of site utilities, are extended into the sands encountered beneath the organic silt/peat, clayey silt or clay soils, the sands may be re-used as structural fill provided that they can be dried back to near their optimum moisture content to attain the required level of compaction and they are separated from the organic silt, clayey silt, layers encountered within the sand stratum. During excavations, the sand and sandy silt soils should be stockpiled separately if plans are to try to re-use the sand as structural fill material. If soil types other than those revealed during our field exploration are encountered during construction, then we should be consulted regarding the suitability of these soils for use as structural fill.

Imported fill material should be all-weather structural fill consisting of well-graded gravel or a sand and gravel mixture with a maximum grain size of 3 inches and less than 5 percent fines (material passing the U.S. Standard No. 200 Sieve). Structural fill may also consist of crushed rock, rock spalls, or Controlled Density Fill (CDF). All structural fill material should be submitted for approval to the geotechnical engineer at least 48 hours prior to delivery to the site.

Fill soils should be placed in horizontal lifts not exceeding 8 inches loose thickness, moisture-conditioned as necessary (moisture content of soil shall not vary by more than ±2 percent of its optimum moisture content), and compacted to at least 95 percent of the maximum dry density based on ASTM Test Method D1557 (Modified Proctor). In-place density tests should be performed on all structural fill to document proper moisture content and adequate compaction levels have been attained. Additional fill lifts should not be placed if the previous lift did not meet the compaction requirements or if soil conditions are not considered stable. Placing several lifts of fill and then potholing down to each lift to conduct compaction testing is not acceptable, and will require complete removal of the fill down to the first lift. Ponding or jetting the soil is not an approved method of soil compaction.

Foundation Recommendations

Liquefiable soils were encountered throughout the site and consideration of the risks associated with constructing on such soils should be considered when selecting a particular foundation system for support of a structure. Our liquefaction analyses indicated that the soils within the upper 21.5 feet of the site, as well as the soils encountered in a deeper zone between a depth of roughly 33 to 43 feet bgs, are liquefiable under a maximum earthquake magnitude of 7.1. The maximum liquefaction induced settlement for this type of seismic event is estimated to be on the order of approximately 1.3 to 2.4 inches (total settlement), with dynamic differential settlement estimated to be on the order of about ¼ to 1-inch over 50 feet. The following sections discuss the subsurface conditions anticipated at the ampm building, fuel canopy and pump stations, and car wash structure, and discusses the recommended foundation system for each of these structures.

ampm Building: The proposed ampm building will be located within the northeastern portion of the site. CPT-1, conducted within the footprint of the building, encountered undocumented fill overlying highly compressible organic silts, peat, and clay and loose sands to a depth of about 20 feet bgs. The subsurface conditions are not considered suitable for foundation support on typical spread footings for both static and dynamic case scenario. Therefore, a deep foundation system is recommended to completely penetrate through liquefiable zones and transfer the building loads through the undocumented fill and compressible native soils to be supported on the underlying dense to very dense native sand and gravel soils.

Pin Piles: A deep foundation system consisting of pin piles bearing at a minimum depth of 20 feet bgs is recommended for support of the ampm building, **provided that the potential for liquefaction induced settlements of the deeper soils is considered acceptable**. Installation recommendations and allowable pile loads for 2-, 3-, and 4-inch diameter pipe piles are provided below. The pile capacities stated are based on pile center to center spacing of at least 3 pile diameters to avoid group effects.

For 2-inch diameter pipe piles driven to refusal using a hand-held, 90-pound jackhammer, we recommend a design axial compression capacity of three tons for each pile. The refusal criterion for this pile and hammer size is defined as less than one inch of pile penetration during 60 seconds of continuous driving. We recommend using extra strong (Schedule 80) galvanized steel pipe for the 2-inch diameter pipe piles.

We recommend that the 3-inch diameter pipe piles be driven using a hydraulic hammer with a weight class of at least 850 lbs. For this pile diameter and hammer size, we recommend a design axial compression capacity of six tons for each pile driven to refusal. The refusal criterion for this pile and hammer size is defined as less than one inch of pile penetration during 20 seconds of continuous driving.

We recommend that the 4-inch diameter pipe piles be driven using a hydraulic hammer, with a weight class of at least 1,100 lbs. For this pile and hammer size, we recommend a design capacity of ten tons for each pile driven to refusal. The refusal criterion for this pile and hammer size is defined as less than one inch of pile penetration during 20 seconds of continuous driving.

The above design capacities are based on theoretical numerical pile driving analysis. We should be retained to review final plans, monitor installation of the piles, and evaluate pile refusal. The pin piles should penetrate a minimum of 4 feet into the dense to very dense sand and gravel encountered at a depth of 20 feet bgs in order to develop the design capacity. Piles that do not meet this minimum embedment criterion or piles that are obstructed on debris in the fill should be rejected, and replacement piles should be driven after consulting with the structural engineer regarding the new pile locations. Due to the relatively small slenderness ratio of pin piles, maintaining pin pile confinement and lateral support is essential to preventing pile buckling. Pin piles should not stick above the finished ground surface.

Although pin piles bearing at a depth of at least 20 feet bgs will mitigate the dynamic settlements anticipated from the liquefiable soils within the upper zone, it **will not** reduce the seismic induced settlements anticipated due to the deeper liquefiable soils. It is estimated that about sixty percent of the dynamic settlement is attributed to the deeper liquefiable soils encountered at a depth of about 33 to 43 feet bgs in our explorations. It is anticipated that the pin piles will encounter refusal within the dense to very dense sand and gravel layer encountered at a depth of about 20 to 33 feet bgs.

Steel Pipe or Auger Cast Piles: In order to mitigate the magnitude of seismic-induced settlement associated with the deeper liquefiable soils, open-ended steel pile piles or auger cast piles, extending below the deeper liquefiable soils to bear at a minimum of 4 feet into the dense to very dense sand and gravel encountered at a depth of about 43 feet bgs, are recommended for support of the ampm building.

Driven open-ended pile piles may be used to support the anticipated 30-kip foundation loads for the ampm building. The allowable axial pile capacity for 8 and 10-inch diameter pipe pile are provided in Table 3. A factor of safety of 3.0 was used in the axial pile capacity calculations.

Table 3: Pipe Pile Capacities

PILE DIAMETER (Inch)	PILE CAPACITY (Kips)
8	25
10	38

Auger cast piles may also be used to support the ampm building. Auger cast piles are constructed with a hollow stem auger drilled to the desired depth. After reaching the minimum recommended penetration into bearing soils, a pressure head is created when grout is pumped through the hollow stem of the auger and into the borehole before starting withdrawal of the auger. After the head is developed, withdrawal of the auger is timed to maintain the grout pressure head and limit intrusion of loose soil into the sides of the pile excavation or discontinuity or “necking’ of the pile. The actual volume of the grout pumped into each pile is recorded and compared to the theoretical volume of the pile. Piles with a ratio of actual to theoretical volume less than 1.1 should be re-drilled. Due to the loose/soft conditions of the near surface soils on this site, we recommend that the auger cast piles be allowed to cure for at least 12 hours prior to the installation of the adjacent piles or maintain at least 12 feet of horizontal distance.

Table 4 lists the allowable capacity for 10 and 12-inch diameter auger cast piles. For design purposes, we recommend that these piles penetrate a minimum of 4 feet into the dense to very dense sand and gravel deposits encountered at a depth of 43 feet below the existing ground surface to provide adequate bearing.

Table 4: Auger Cast Pile Capacities

PILE DIAMETER (Inches)	ALLOWABLE PILE CAPACITY (Kips)
10	41
12	60

General - Final pile depths should be expected to vary somewhat and will depend on the actual depth of the existing fill and loose/soft native soils, and the nature of the underlying competent bearing soils. Debris consisting of chunks asphalt pavement and broken clay pipe was present in the undocumented fill encountered in test pits IP-1 and IP-2, and may be encountered within the proposed building footprint. There is a possibility some piles may be obstructed. There should be contingencies in the budget and design for removal of obstructions and/or additional/relocated piles to replace piles that may be obstructed by debris in the fill. A structural engineer should prepare the structural design of the pile foundation system.

The pile capacities listed in Tables 3 and 4 do not account for the effects of down drag forces. Since finish grades are anticipated to be at or near existing grades, we do not anticipate that down drag will have an appreciable effect on the capacity of the deep foundation system provided our site preparation and foundation recommendations are followed.

We recommend dynamic testing be conducted on at least one (1) indicator test pile installed within the building area in order to observe the installation characteristics of the piles, evaluate the suitability of the pile installation methods and equipment, and evaluate potential differences in the elevation that bearing soils are encountered, as well as the condition of the competent bearing soils. The indicator test pile should be installed and tested prior to driving the production piles to obtain the installation driving criteria and provide a better indication of the optimum pile length of production piles. Indicator test pile length and location should be selected by the geotechnical engineer, in conjunction with the structural engineer and contractor. We recommend that the dynamic testing consist of taking measurements using a Pile Driving Analyzer (PDA) during driving, as well as during a re-strike of the indicator test pile following a minimum of 24 hours of driving, if necessary. The purpose of the re-strike testing with the PDA is to determine the amount of additional pile capacity achieved once the pore pressures from pile driving have dissipated.

The indicator pile length should allow extra length for attachment of the PDA transducers and additional driving, if necessary due to soil conditions. We should be retained to review final plans, monitor installation of the indicator and production piles (including recording of blows counts, depth to bearing soils, and embedment within competent bearing soils), and evaluate the PDA tests results. The contractor should use the same equipment to install both the indicator and production piles, unless the results of the PDA testing indicates otherwise.

We recommend a baseline survey of the nearby structures, consisting of photo documentation of the existing condition of the buildings, be conducted prior to the start of construction activities. We also recommend the nearby existing structures be monitored for movement during pile driving activities. A system of survey points should be established and baseline readings should be established prior to commencing with the pile driving activities. Readings should be taken periodically until the piles are installed and these readings should be compared to the original baseline measurements.

Deep Foundation Alternative - As an alternative to supporting the ampm building on a deep foundation system, consideration could be given to locating the proposed building within the southern portion of the site where more suitable subsurface conditions were encountered in terms of anticipated total static settlement. However, dynamic settlement due to liquefiable soils would still be present at this alternative location, and the risks associated with seismic-induced settlements would have to be acceptable in order to support the building on a shallow foundation system. In addition, some over-excavation of the undocumented fill and loose/soft native soils and replacement with structural fill would still be required to provide a stable bearing surface for the anticipated foundation loads. Shallow foundation recommendations for this alternative would be similar to those presented in the following subsection for Canopy and Car Wash Structures.

Canopy and Car Wash Structures**:** We have assumed that design of the foundation system for the proposed canopy and car wash structures does not require consideration of seismic-induced dynamic settlements. Therefore, these structures may be supported on a shallow foundation system provided that the recommendations stated in this section are followed during design and construction of the foundations.

Based on CPT-3, CPT-4, and CPT-5, conducted within and near the locations of the proposed fuel canopy and car wash structures, the near surface soils within a depth of 10 feet bgs are anticipated to be undocumented fill underlain by loose native sands, with occasional soft silt or clay layers up to 1-foot thick. The near surface soils are not considered suitable for support of the foundation loads. We recommend that the undocumented fill and loose/soft native soils be removed to a depth of two (2) feet beneath the footings, with the over-excavation extending laterally from the outside edges of the footing a horizontal distance of one-half the width of the footing. A layer of rock spalls or a high strength geotextile fabric should be placed over the soils at the bottom of the over-excavation. The resulting excavation should then be backfilled with properly placed and compacted structural fill up to the planned footing subgrade elevations. Shallow foundations for the fuel canopy and car wash structures may then be supported on the structural fill.

Based on the size of the structures and the minimum over-excavation requirements, it may be economical to remove the unsuitable bearing soils to a depth of two (2) feet below the bottom of the footings (bearing level) throughout the entire footprint of each structure, and extending a horizontal distance of 12 inches beyond the perimeter of the canopy or car wash foundations. A representative of Krazan and Associates should evaluate the over-excavation grade and observe structural fill placement.

New utilities should not be located within the load influence zone of the footing defined as an imaginary line extending out at 1 horizontal to 1 vertical (1H:1V) from the bottom outside edge of the footing. Depending on the location of the utility, it may be necessary to deepen the planned footing elevation such that the utility pipe is located above the footing zone of influence so the footing does not impose a surcharge load on the utility.

We recommend that exterior footings bear a minimum depth of 18 inches below pad subgrade (soil grade) or adjacent exterior grade, whichever is lower, for frost protection and bearing capacity considerations. Interior footings should have a minimum depth of 12 inches below pad subgrade (soil grade) or adjacent exterior grade, whichever is lower. Footing widths should be based on the anticipated loads and allowable soil bearing pressure, but should not be less than 12 inches wide regardless of load. Additionally, footings should conform to current International Building Code (IBC) guidelines. Water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend that an allowable bearing capacity of **1,500 pounds per square foot (psf)** be used for foundation design for this project. A representative of Krazan and Associates should evaluate the foundation bearing soil prior to footing form construction and evaluate all structural fill subgrade and monitor all structural fill placement.

Resistance to lateral footing displacement can be computed using an allowable friction factor of 0.35 acting between the bases of foundations and the supporting subgrade. Lateral resistance for footings can alternatively be developed using an allowable equivalent fluid passive pressure of 300 pounds per cubic foot (pcf) <u>for granular structural fill</u> acting against the appropriate vertical footing faces (neglecting the upper 12 inches). The allowable friction factor and allowable equivalent fluid passive pressure values include a factor of safety of 1.5. The frictional and passive resistance of the soil may be combined without reduction in determining the total lateral resistance. A 1/3 increase in the above values may be used for short duration wind and seismic loads.

For foundations constructed as recommended, the total static settlement is not expected to exceed 1-inch. Differential settlement should be less than ½ inch. Most settlement is expected to occur during construction, as the loads are applied.

Up to 2.4 inches of total seismic settlement and about ¼ to 1-inch of differential settlement could occur during and/or following a seismic event. The foundation elements, i.e. spread and wall footings, could be structurally tied together to create a stiffer structure. It should be noted that although this may reduce the damage associated with the anticipated seismic settlement, particularly that caused by differential settlement, it would not mitigate the anticipated total seismic settlement. If the anticipated magnitude of the seismic settlement is deemed unacceptable, a deep foundation system could also be considered for support of either of these structures. The deep foundation recommendations presented for the ampm building would be applicable for the fuel canopy or car wash if seismic-induced dynamic settlements are to be considered in design of the foundation system.

Seasonal rainfall, water run-off, and the normal practice of watering trees and landscaping areas around the proposed structures should not be permitted to flood and/or saturate foundation subgrade soils. To prevent the build-up of water within the footing areas, continuous footing drains (with cleanouts) should be provided at the base of footings. The footing drains should consist of a minimum 4-inch diameter rigid perforated PVC pipe, sloped to drain, with perforations placed near the bottom and enveloped in all directions by washed rock and wrapped with filter fabric to limit the migration of silt and clay into the drain.

Drilled Pier Alternative for Fuel Canopy Foundation - As an alternative to spread footings, the fuel canopy columns may be supported on drilled piers. Drilled pier foundations are constructed by augering through the soils down to the design depth, installing steel reinforcement in the shaft, and then backfilling the shaft with concrete. Typical drilled pier diameters for support of the lightly loaded fuel canopy structure generally range from 18 to 48 inches in diameter. The drilled pier foundation supported on competent native alluvial soils may be designed with the following soil design parameters:

- Estimated angle of internal friction: 30 degrees.
- Estimated moist unit weight: 125 pounds per cubic foot (pcf).
- Allowable fluid passive resistance: 350 pcf (neglecting the upper 24 inches and includes 1.5 factor of safety).

Krazan & Associates should observe construction of the drilled piers to verify that the suitable bearing soils have been encountered at the bottom of the shaft prior to placement of steel reinforcement and concrete.

Due to the shallow groundwater conditions encountered at the site, the use of temporary casing will likely be required to prevent caving of the surrounding soil during construction of the drilled piers. Alternatively, construction of the drilled piers may use a slurry to maintain the integrity of the shaft during drilling and backfilling with concrete. The reinforcement and concrete should be placed immediately following excavation of the drilled shaft. The concrete should be placed by tremie method and a head of at least 2 feet of concrete should be maintained above the bottom of the casing during withdrawal from the shaft.

Floor Slabs and Exterior Flatwork

Based on the results of this investigation, undocumented fill and loose/soft native soils are anticipated to be encountered in the floor slabs and exterior flatwork subgrade. The floor slab and exterior flatwork subgrade should be prepared in accordance with the recommendations presented in the **Site Preparation** section of this report, and may be designed using a modulus of subgrade reaction value of k = 150 pounds per cubic inch (pci) for slabs supported on structural fill extending to the native soils.

In areas where it is desired to reduce floor dampness, such as areas covered with moisture sensitive floor coverings, we recommend that concrete slab-on-grade floors be underlain by a water vapor retarder system. According to ASTM guidelines, the water vapor retarder should consist of a vapor retarder sheeting underlain by a minimum of 6-inches of compacted clean (less than 5 percent passing the U.S. Standard No. 200 Sieve) open-graded coarse rock of ¾-inch maximum size. The vapor retarder sheeting should be protected from puncture damage. In addition, ventilation of the structure may be prudent to reduce the accumulation of interior moisture.

The exterior floors should be placed separately in order to act independently of the walls and foundation system.

Lateral Earth Pressures and Retaining Walls

It is not anticipated that permanent retaining walls will be required for this project. However, in case retaining walls will be incorporated into the project design, we have developed criteria for the design of retaining or below grade walls. Our design parameters are based on retention of the in-place soils and/or imported granular structural fill. The parameters are also based on level, well-drained wall backfill conditions. If other wall slope configurations are planned, we should be contacted to evaluate and provide additional recommendations for these cases.

Walls may be designed as "restrained" retaining walls based on "at-rest" earth pressures, plus any surcharge on top of the walls as described below, if the walls are braced to restrain movement and/or movement is not acceptable. Unrestrained walls may be designed based on "active" earth pressures, if the walls are not part of the building and some movement of the retaining walls is acceptable. Acceptable lateral movement equal to at least 0.2 percent of the wall height would warrant the use of "active" earth pressure values for design. We recommend that walls supporting horizontal backfill and not subjected to hydrostatic forces be designed using a triangular earth pressure distribution equivalent to that exerted by a fluid with a density of 35 pcf for yielding (active condition) walls, and 55 pcf for non-yielding (at-rest condition) walls.

If vehicular loads are expected to act on the surface of the wall backfill within a horizontal distance of less than or equal to one-half of the wall height behind the back face of the wall, a live load surcharge should be applied for the design. In this case, we recommend the addition of vehicle surcharges of 70 psf and 100 psf to the active and at-rest earth pressures, respectively.

The stated lateral earth pressures do not include the effects of hydrostatic pressure generated by water accumulation behind the retaining walls or loads imposed by construction equipment, slopes, foundations, or roadways adjacent to the wall (surcharge loads). To minimize the lateral earth pressure and prevent the build-up of water pressure against the walls, continuous footing drains should be provided at the base of walls. The footing drains should consist of a minimum 4-inch diameter perforated pipe, sloped to drain, and with perforations placed near the bottom. The drainpipe should be enveloped by 6 inches of washed gravel in all directions wrapped in filter fabric to prevent the migration of silt and clay into the drain. Below grade structures should be designed to withstand hydrostatic pressures due to the shallow groundwater encountered at the site.

The backfill placed adjacent to the wall and extending a lateral distance of at least 2 feet behind the wall should consist of free-draining granular material. All free-draining backfill should contain less than 5 percent fines (material passing the U.S. Standard No. 200 Sieve) with at least 30 percent of the material retained on the U.S. Standard No. 4 Sieve. Alternatively, a drainage composite may be used. It should be realized that the primary purpose of the free-draining material is the reduction of hydrostatic pressure. Some potential for the moisture to contact the back face of the wall may exist, even with treatment, which may require that more extensive waterproofing be specified for walls that require interior moisture sensitive finishes.

We recommend that backfill placed within a lateral distance of 3 feet behind the wall be compacted to between 92 and 95 percent of the maximum dry density based on ASTM D1557 Test Method to limit stressed on the retaining wall from compaction of the backfill. In-place density tests should be performed to verify adequate compaction and moisture content. Soil compaction equipment places transient surcharge loads on the backfill. Consequently, only light, hand-operated equipment is recommended for fill compaction within a 3-foot horizontal distance of the wall so that excessive stress is not imposed on the wall. Backfill placed greater than 3 feet from the wall should be compacted to at least 95 percent relative density in accordance with ASTM D1557, which may be conducted using conventional compaction equipment.

Erosion and Sediment Control

Erosion and sediment control (ESC) is used to minimize the transportation of sediment to wetlands, streams, lakes, drainage systems, and adjacent properties. Erosion and sediment control measures should be taken and these measures should be in general accordance with local regulations. At a minimum, the following basic recommendations should be incorporated into the design of the erosion and sediment control features of the site:

1) Phase the soil, foundation, utility, and other work, requiring excavation or the disturbance of the site soils, to take place during the dry season (generally May through September). However, provided precautions are taken using Best Management Practices (BMPs), grading activities can be undertaken during the wet season (generally October through April). It should be noted that this typically increases the overall project cost.

2) All site work should be completed and stabilized as quickly as possible.

3) Additional perimeter erosion and sediment control features may be required to reduce the possibility of sediment entering the surface water. This may include additional silt fences, silt fences with a higher Apparent Opening Size (AOS), construction of a berm, or other filtration systems.

4) Any runoff generated by dewatering discharge should be treated through construction of a sediment trap if there is sufficient space. If space is limited other filtration methods will need to be incorporated.

It has been our experience that soil erosion potential can be minimized by limiting the amount of bare areas exposed during construction activities, frequently wetting the surface soils during construction, and with proper landscaping of the site following completion of construction. Construction activities can alter the erosion potential of soils due to water. Typically, erosion of exposed soils will be most noticeable during periods of rainfall and may be mitigated by the use of temporary erosion control measures, such as silt fences, hay bales, straw wattles, mulching, control ditches or diversion trenching, and contour furrowing. The walls of excavations should be covered with plastic sheeting during periods of rainfall. Erosion control measures should be in place before the onset of wet weather.

Groundwater Influence on Structures and Earthwork Construction

Groundwater was encountered at depths of ranging between 1.5 to 3.7 feet bgs based on observations during excavation of the test pits and pore water dissipations tests conducted in the CPTs. It should be recognized that groundwater elevations may fluctuate with time. The groundwater level will be dependent upon seasonal precipitation, irrigation, land use, and climatic conditions, as well as other factors. Therefore, groundwater levels at the time of the field investigation may be different from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

If earthwork is performed during or soon after periods of precipitation, the subgrade soils may become saturated. These soils may not respond to densification techniques due to the excessive moisture. Typical remedial measures include: disking and aerating the soil during dry weather; mixing the soil with drier materials; removing and replacing the soil with an approved fill material. Krazan & Associates should be consulted prior to implementing remedial measures to observe the unstable subgrade conditions and provide appropriate recommendations.

Due to the shallow groundwater encountered at the site, below grade structures such as the USTs, should be designed to result uplift pressures.

Drainage and Landscaping

The ground surface should slope away from building pads and pavement areas, toward appropriate drop inlets or other surface drainage devices. It is recommended that adjacent exterior grades be sloped a minimum of 2 percent for a minimum distance of 5 feet away from structures. Roof drains should be tight lined away from foundations. Roof drains should not be connected to the footing drains, but may use the same outfall piping if connected well away from the structure and with enough fall such that roof water will not back-up into the footing drains.

Subgrade soils in pavement areas should be inclined at a minimum of 1 percent and drainage gradients should be maintained to carry all surface water to collection facilities, and suitable outlets. These grades should be maintained for the life of the project.

Water should not be allowed to collect adjacent to the structures. Excessive irrigation within landscaped areas adjacent to the structure should not be allowed to occur.

Pavement Design

The undocumented fill and native soils encountered at the site are unsuitable for support of pavement loads. The pavement subgrade should be prepared in accordance with the recommendations presented in the Site Preparation section of this report. Traffic loads were not provided, however, based on our knowledge of the proposed project, we expect the traffic to range from light duty (passenger automobiles) to heavy duty (fire trucks and delivery trucks). The following tables show the minimum recommended pavement sections for both light and heavy-duty traffic loads.

ASPHALTIC CONCRETE (FLEXIBLE) PAVEMENT

Asphaltic Concrete	Aggregate Base*
3.0 in.	6.0 in.

PORTLAND CEMENT CONCRETE (RIGID) PAVEMENT
4000 psi with FIBER MESH

Min. PCC Depth	Aggregate Base*
6.0 in.	6.0 in.

** 95% compaction based on ASTM Test Method D1557*

The asphaltic concrete depth in the flexible pavement tables should be a surface course type asphalt, such as Washington Department of Transportation (WSDOT) ½-inch Hot Mix Asphalt (HMA). The pavement specification in Appendix C provides additional recommendations including aggregate base material. The rigid pavement design is based on a Portland Cement Concrete (PCC) mix that has a 28-day compressive strength of 4,000 pounds per square inch (psi) with a fiber mesh. The design is also based on a concrete flexural strength or modulus of rupture of 575 psi.

Testing and Inspection

A representative of Krazan & Associates, Inc. should be present at the site during the earthwork activities to confirm that actual subsurface conditions, including foundation bearing soils, are consistent with those exposed during our exploratory field work. This activity is an integral part of our services as acceptance of earthwork construction is dependent upon compaction testing and stability of the material. This representative can also verify that the intent of our recommendations has been incorporated into the project design and construction. Krazan & Associates, Inc. will not be responsible for grades or staking, since this is the responsibility of the Prime Contractor. Furthermore, Krazan & Associates is not responsible for the contractor's procedures, methods, scheduling, or management of the work site.

LIMITATIONS

Geotechnical engineering is one of the newest divisions of Civil Engineering. This branch of Civil Engineering is constantly improving as new technologies and understanding of earth sciences improves. Although your site was analyzed using the most appropriate current techniques and methods, undoubtedly there will be substantial future improvements in this branch of engineering. In addition to improvements in the field of geotechnical engineering, physical changes in the site either due to excavation or fill placement, new agency regulations, or possible changes in the proposed structure after the time of completion of the soils report may require the soils report to be professionally reviewed. In light of this, the owner should be aware that there is a practical limit to the usefulness of this report without critical review. Although the time limit for this review is strictly arbitrary, it is suggested that two years be considered a reasonable time for the usefulness of this report.

This report has been prepared for the exclusive use of BP Products North America Inc. and their assigns, for the specific application to the subject site. Foundation and earthwork construction are characterized by the presence of a calculated risk that soil and groundwater conditions have been fully revealed by the original geotechnical investigation. This risk is derived from the practical necessity of basing interpretations and design conclusions on limited sampling of the earth. Our report, design conclusions, and interpretations should not be construed as a warranty of the subsurface conditions. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report.

The recommendations made in this report are based on the assumption that soil conditions do not vary significantly from those encountered during our field investigation. The findings and conclusions of this report can be affected by the passage of time, seasonal weather conditions, manmade influences such as construction on or adjacent to the site, and natural events such as earthquakes, slope instability, flooding, or groundwater fluctuations. If any variations or undesirable conditions are encountered during construction, the geotechnical engineer should be notified so that supplemental recommendations can be made.

The conclusions of this report are based on the information provided regarding the proposed construction. If the proposed construction is relocated or redesigned, the conclusions in this report may not be valid. The geotechnical engineer should be notified of any changes so that the recommendations can be reviewed and re-evaluated.

Misinterpretations of this report by other design team members can result in project delays and cost over-runs. These risks can be reduced by having Krazan & Associates, Inc. involved in the design team's meetings and discussions prior to and following submission of the geotechnical report. Krazan & Associates, Inc. should also be retained to review pertinent elements of the design team's plans and specifications. To reduce the risk of contractors misinterpreting the recommendations of this report, Krazan & Associates should participate in pre-bid and preconstruction meetings, and provide construction observations and testing during the site work.

This report is a geotechnical engineering investigation with the purpose of evaluating the soil conditions in terms of foundation design. The scope of our geotechnical engineering services did not include any environmental site assessment for the presence or absence of hazardous and/or toxic materials in the soil, groundwater or atmosphere, or the presence of wetlands. Any statements, or absence of statements, in this report or on any test pit or CPT logs regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous and/or toxic assessments.

The geotechnical information presented herein is based upon professional interpretation utilizing standard engineering practices and a degree of conservatism deemed proper for this project. It is not warranted that such information and interpretation cannot be superseded by future geotechnical developments. We emphasize that this report is valid for this project as outlined above, and should not be used for any other site. Our report is prepared for the exclusive use of our client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing.

If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office at (253) 939-2500.

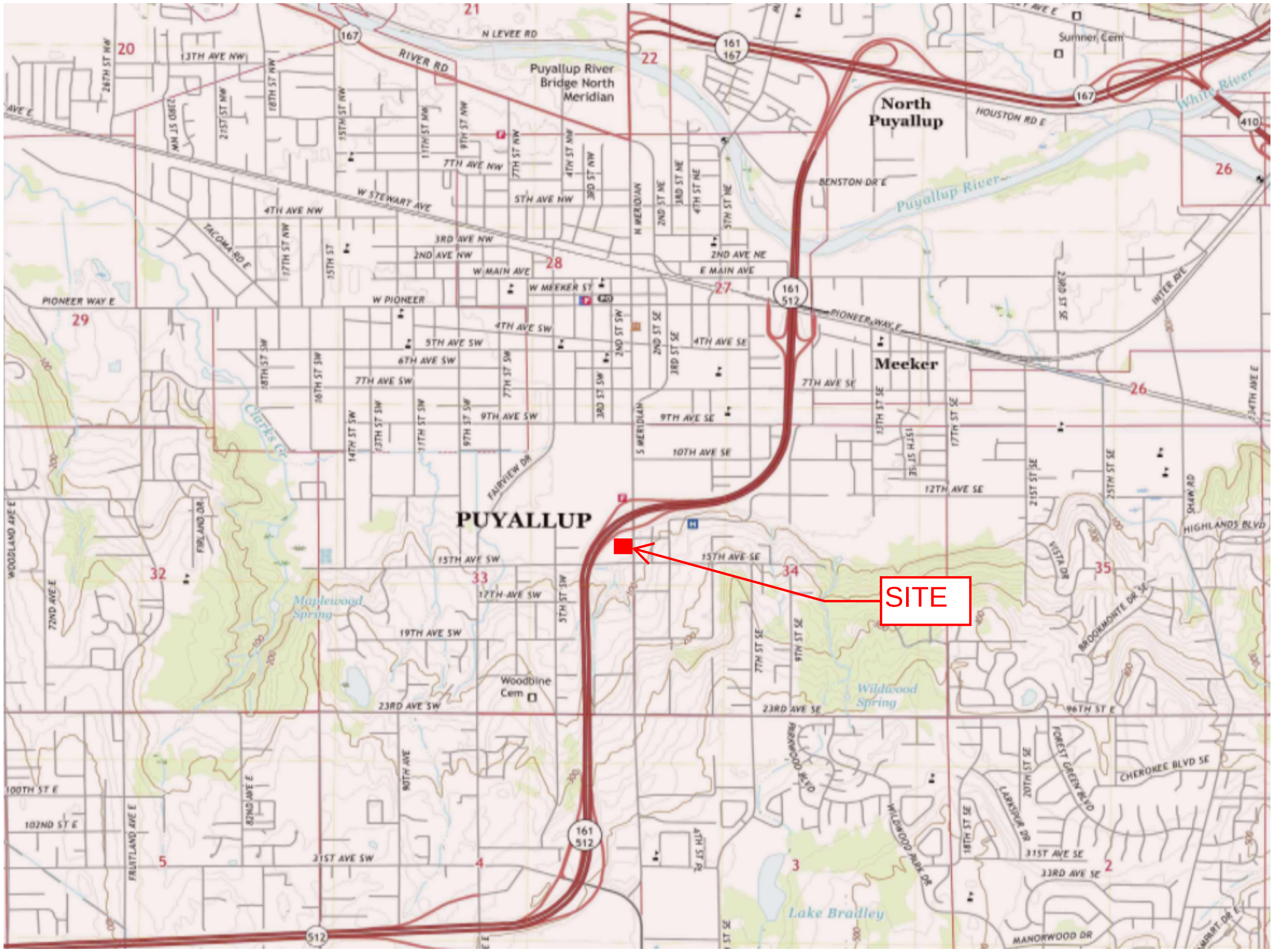
Respectfully submitted,

KRAZAN & ASSOCIATES, INC.

5/6/2022

Theresa R. Nunan

Theresa R. Nunan	Vijay Chaudhary, P.E.
Project Manager	Assistant Regional Engineering Manager



N

Figure 1: Vicinity Map

Reference: USGS Topographic Map titled "Puyallup Quadrangle, Washington-Pierce County, 7.5-minute series, Puyallup, WA" dated 2020.

Krazan

Project: ARCO ampm Fueling Facility, 1402 S Meridian Avenue, Puyallup, WA		
Date: April 2022	Project Number: 062-22010	
Drawn By: KC	Not to Scale	Figure 1

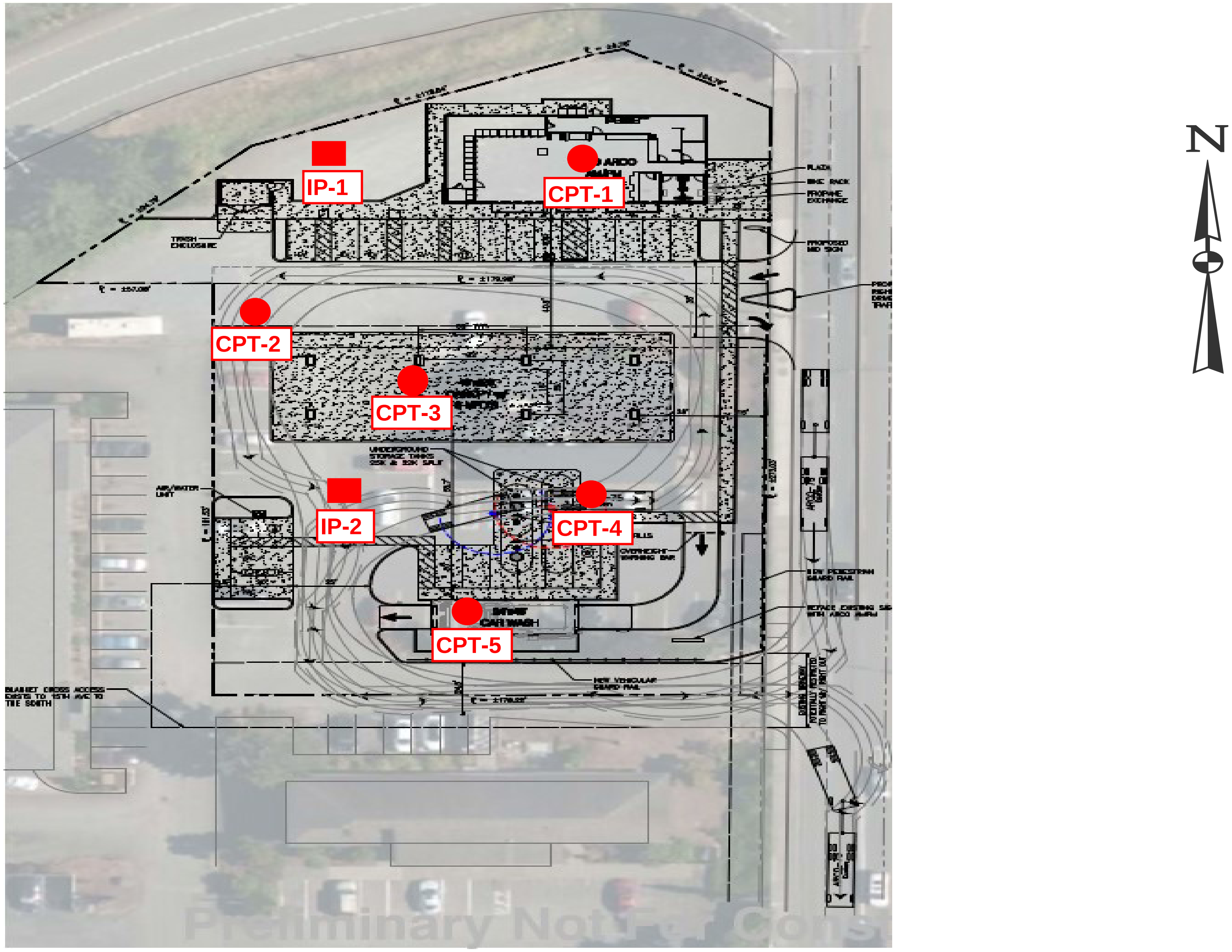


Figure 2: Site Plan

Legend:

TP-1 Test Pit Location

CPT-01 Cone Penetration Test Location

Reference: Preliminary Site Plan, Sheet SP-5, prepared by Barghausen Consulting Engineers, Inc. dated July 9, 2021.

Krazan

Proposed ARCO ampm Fueling Facility: 1402 S Meridian Ave., Puyallup, WA		
Date: April 9 2020	Project Number: 062-22010	
Drawn By: TRN	Not to Scale	Figure 2

APPENDIX A

FIELD INVESTIGATION AND LIQUEFACTION ANALYSIS

Field Investigation

The field investigation consisted of a surface reconnaissance and a subsurface exploration program. Six (6) Cone Penetrometer Tests (CPTs) were conducted for the subsurface investigation at this site. The CPTs were advanced to depths of about 27.0 to 46.3 feet bgs using a subcontracted testing rig. Porewater pressure dissipation tests were conducted in all of the CPTs for evaluation of the static groundwater level at the time of the explorations. Seismic shear wave testing was conducted in CPT-2B, CPT-4, and CPT-5 for use in determining seismic design parameters.

Two (2) test pits were excavated on March 28, 2022 to depths of 4.7 and 7.1 feet bgs using a subcontracted excavator and equipment operator. A geotechnical engineer from Krazan and Associates was present during the explorations, visually classified the soils obtained in the test pits in general accordance with the Unified Soil Classification System (USCS), and maintained logs of the test pits.

The test pit and CPT explorations were located in the field based on existing site features, and their approximate locations are shown on the Site Plan (Figure 2). The test pit and CPT logs are presented in this Appendix. The depths shown on the attached logs are from the existing ground surface at the time of our exploration. The ground surface elevations included on the CPT logs are based on information presented on the Alta/NSPS Land Title Survey prepared by Barghausen Consulting Engineering, Inc. and dated April 19, 2022.

Liquefaction Analysis

The commercially available liquefaction analysis software, NovoCPT from NovoTech, was used to evaluate the liquefaction potential and the possible liquefaction induced settlement for the site soil and groundwater conditions based on our explorations. The analysis was performed using the information from the CPTs. The results of the liquefaction analyses are included in this appendix.

Krazan & ASSOCIATES, INC.

Project: Prop. ARCO ampm Fueling Facility	Project Number: 062-22010	Client: BP Products North America	Test Pit No.:	IP-1
Location: 1402 S. Meridian Ave., Puyallup, WA			Contractor: Strickland & Sons Excavation	
Project Manager: Theresa Nunan	Date — Started: 3.28.22		Equipment: CAT 306 Excavator with WW	
Field Engineer: Theresa Nunan	Date — Completed: 3.28.22			
Groundwater Depth: 5.8 +/- feet (and rising)		Ground Elevation: 46.0 +/- feet	Total Depth of Test Pit: 7.1 feet	

Elev. (feet)	Depth (feet)	Sample Type	Sample ID	Blow Counts	N-Value (blows/ft)	Graphic Log	Soil Classification	Notes / Lab Test Results
	0						2" **PEA GRAVEL** Mixed with Silty Sand **(FILL)**	
	0–2.2						Brown **Silty SAND (SM)** with gravel, trace cobbles, chunks of asphalt, broken clay pipe pieces, medium dense, moist - - - At 1.5 ft., encountered chunks of 6-in. thick asphalt pavement slab - - - becomes loose **(FILL)**	
	2.2–4.0	BULK	S-1				Dark Brown **MUCK** (Organic Silt and Clay with some fibrous organics), very soft, moist **(SHALCAR MUCK)**	
	4.0–7.1	BULK	S-2				Dark Grey **SAND (SP-SM) with silt**, fine grained sand, very loose, moist to wet **(ALLUVIUM)**	
	7.1						**Test Pit Terminated at 7.1 Feet**	

Krazan & ASSOCIATES, INC.

Project: Prop. ARCO ampm Fueling Facility	**Project Number:** 062-22010	**Client:** BP Products North America	**Test Pit No.:**	**IP-2**
Location: 1402 S. Meridian Ave., Puyallup, WA			**Contractor:** Strickland & Sons Excavation	
Project Manager: Theresa Nunan	**Date** — **Started:** 3.28.22		**Equipment:** CAT 306 Excavator with WW	
Field Engineer: Theresa Nunan	**Completed:** 3.28.22			
Groundwater Depth: 2.3 +/- feet (and rising)		**Ground Elevation:** 47.0	**Total Depth of Test Pit:** 4.7 feet	

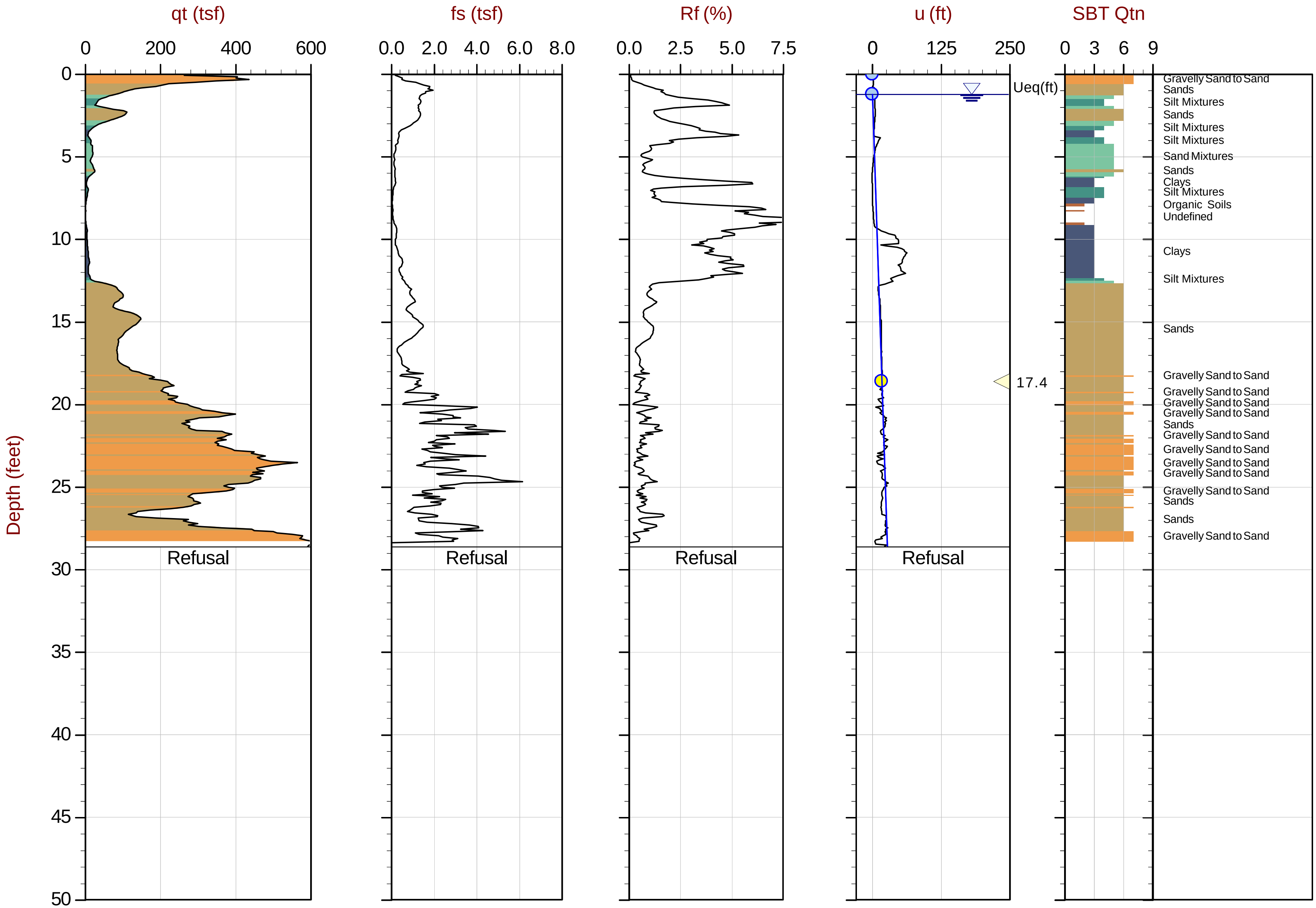
Elev. (feet)	Depth (feet)	Sample Type	Sample ID	Blow Counts	N-Value (blows/ft)	Graphic Log	Soil Classification	Notes / Lab Test Results
	0						3.5 Inches of **Asphalt Concrete Pavement** underlain by 6 inches of Brown Silty SAND (SM) with gravel **BASE COURSE**	
	1						Tan **Poorly Graded SAND (SP-SM)** with silt, fine to medium grained, loose, moist **(FILL)**	
	2	BULK	S-1				Bluish Grey **SAND (SP-SM)** with silt, very loose, wet **(ALLUVIUM)**	
	3	BULK	S-2				Dark Brown **Organic SILT (OH)**, very soft, wet **(SHALCAR MUCK)**	
	4	BULK	S-3				Dark Grey/Black **Poorly Graded SAND (SP-SM)** with silt, very loose, wet **(ALLUVIUM)**	
	5						**Test Pit Terminated at 4.7 Feet**	
	6							
	7							
	8							

ConeTec

Krazan

Job No: 22-59-23911
Date: 2022-03-30 13:42
Site: Puyallup CPT

Sounding: CPT-01
Cone: 781:T1500F15U35



Max Depth: 8.725 m / 28.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17887 Long: -122.29399

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

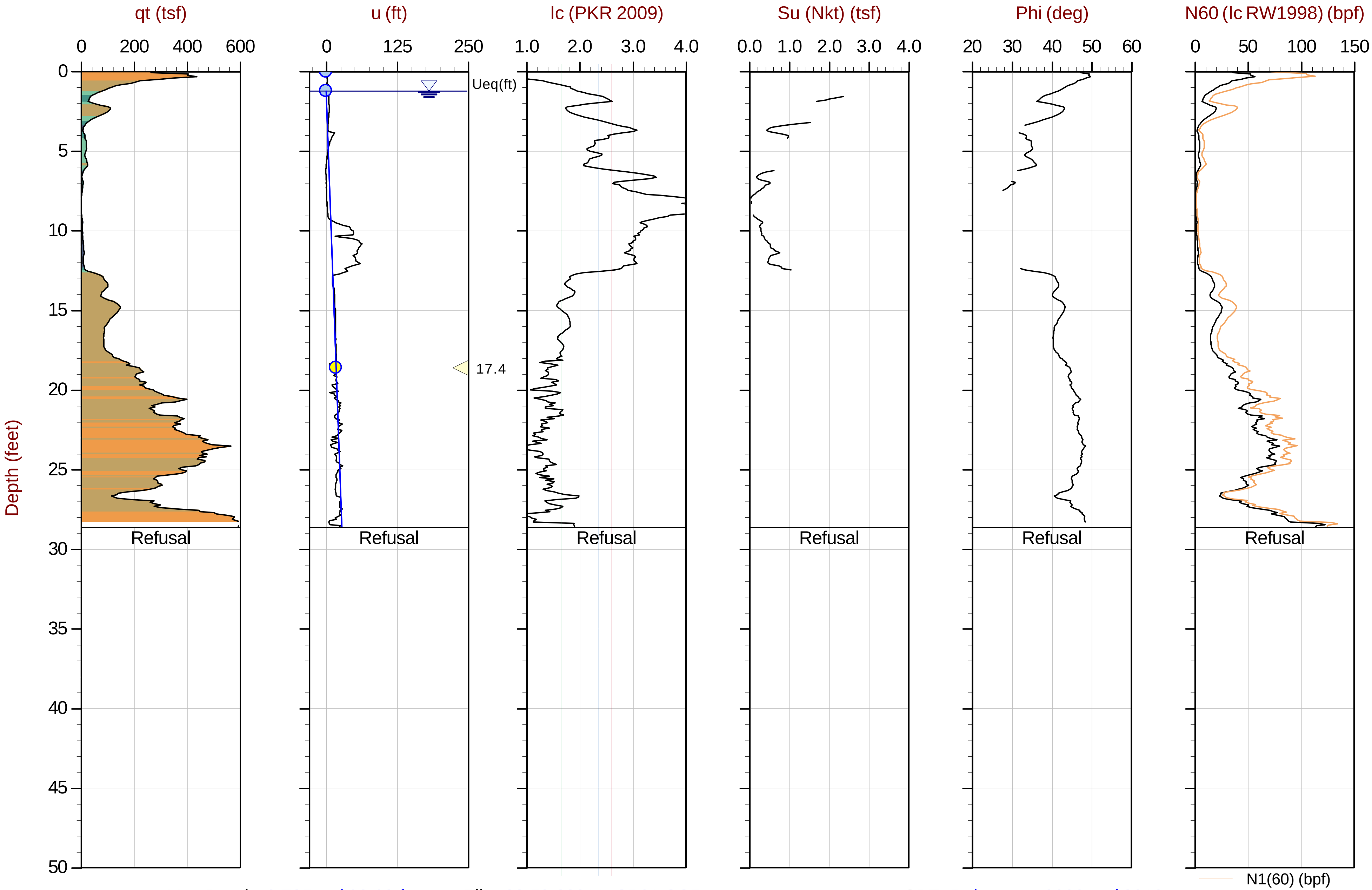
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 13:42
Site: Puyallup CPT

Sounding: CPT-01
Cone: 781:T1500F15U35



Max Depth: 8.725 m / 28.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

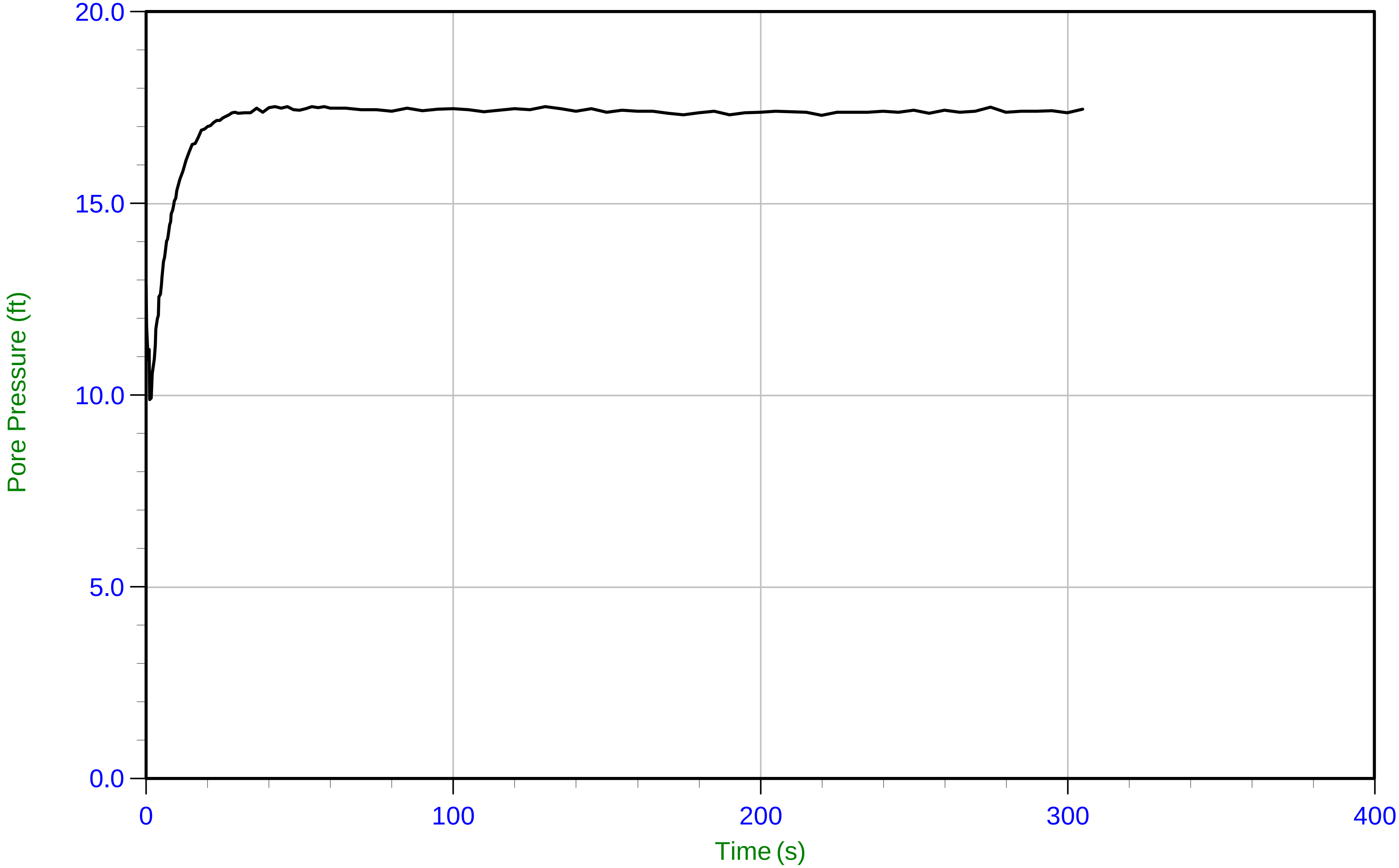
File: 22-59-23911_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17887 Long: -122.29399

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Krazan	Job No: 22-59-23911 Date: 03/30/2022 13:42 Site: Puyallup	Sounding: CPT-01 Cone: 781:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-59-23911_CP01.ppd2
Depth: 5.675 m / 18.619 ft
Duration: 305.0 s

u Min: 9.9 ft
u Max: 17.5 ft
u Final: 17.5 ft

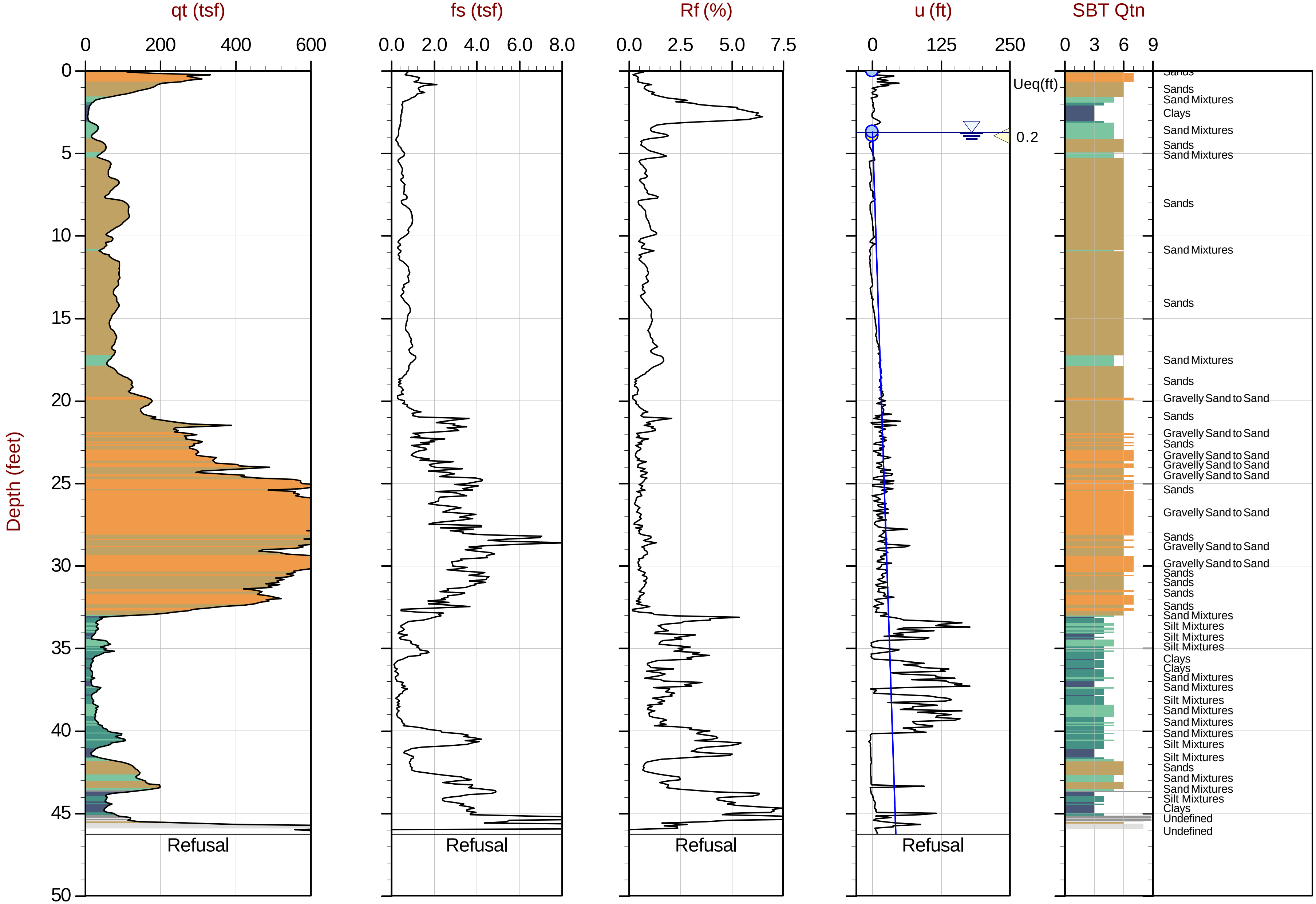
WT: 0.374 m / 1.227 ft
Ueq: 17.4 ft

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 17:07
Site: Puyallup CPT

Sounding: CPT-02B
Cone: 781:T1500F15U35



Max Depth: 14.100 m / 46.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.47872 Long: -122.29435

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

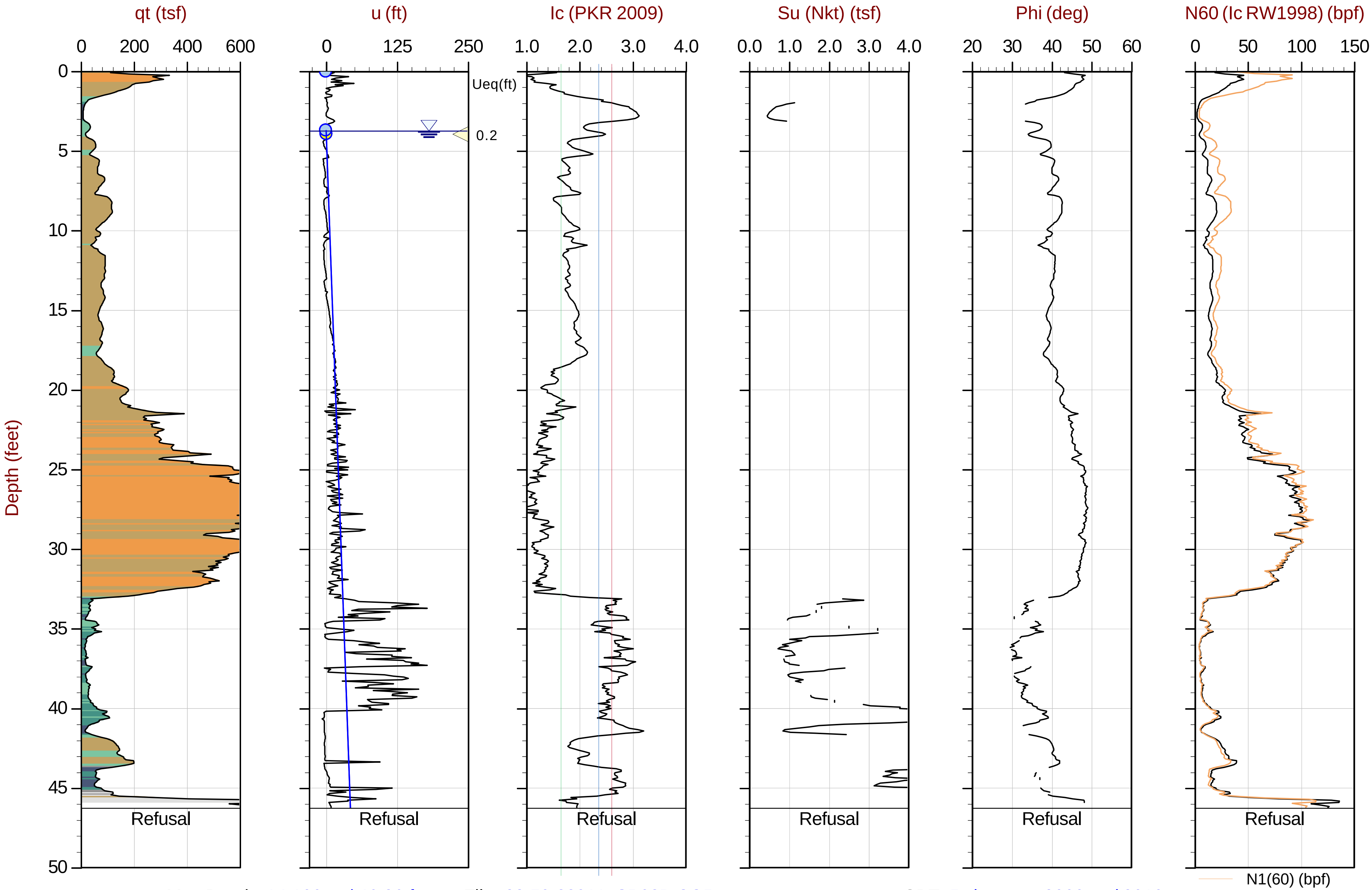
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 17:07
Site: Puyallup CPT

Sounding: CPT-02B
Cone: 781:T1500F15U35



Max Depth: 14.100 m / 46.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP02B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.47872 Long: -122.29435

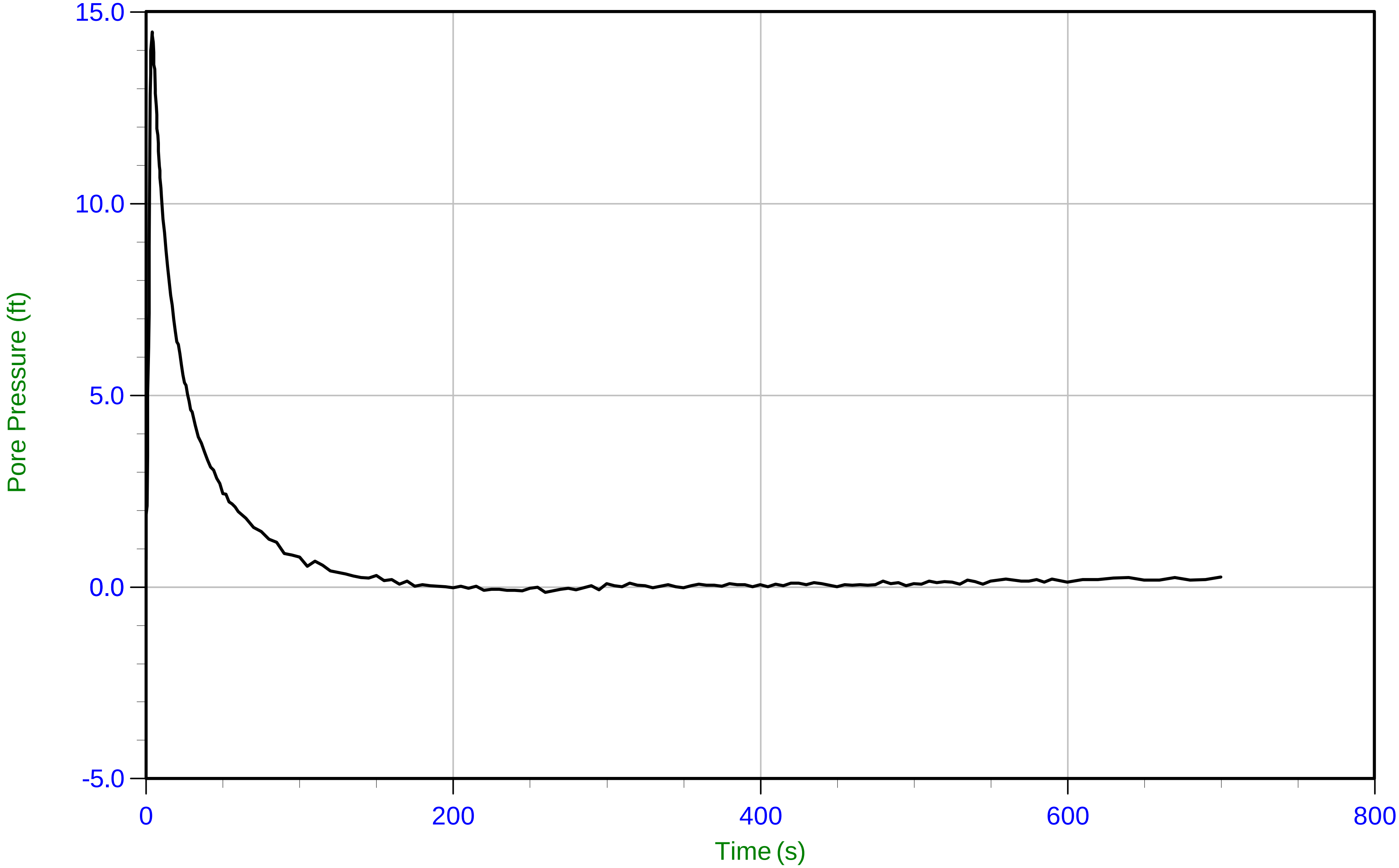
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Krazan

Job No: 22-59-23911
Date: 03/30/2022 17:07
Site: Puyallup

Sounding: CPT-02B
Cone: 781:T1500F15U35 Area=15 cm²



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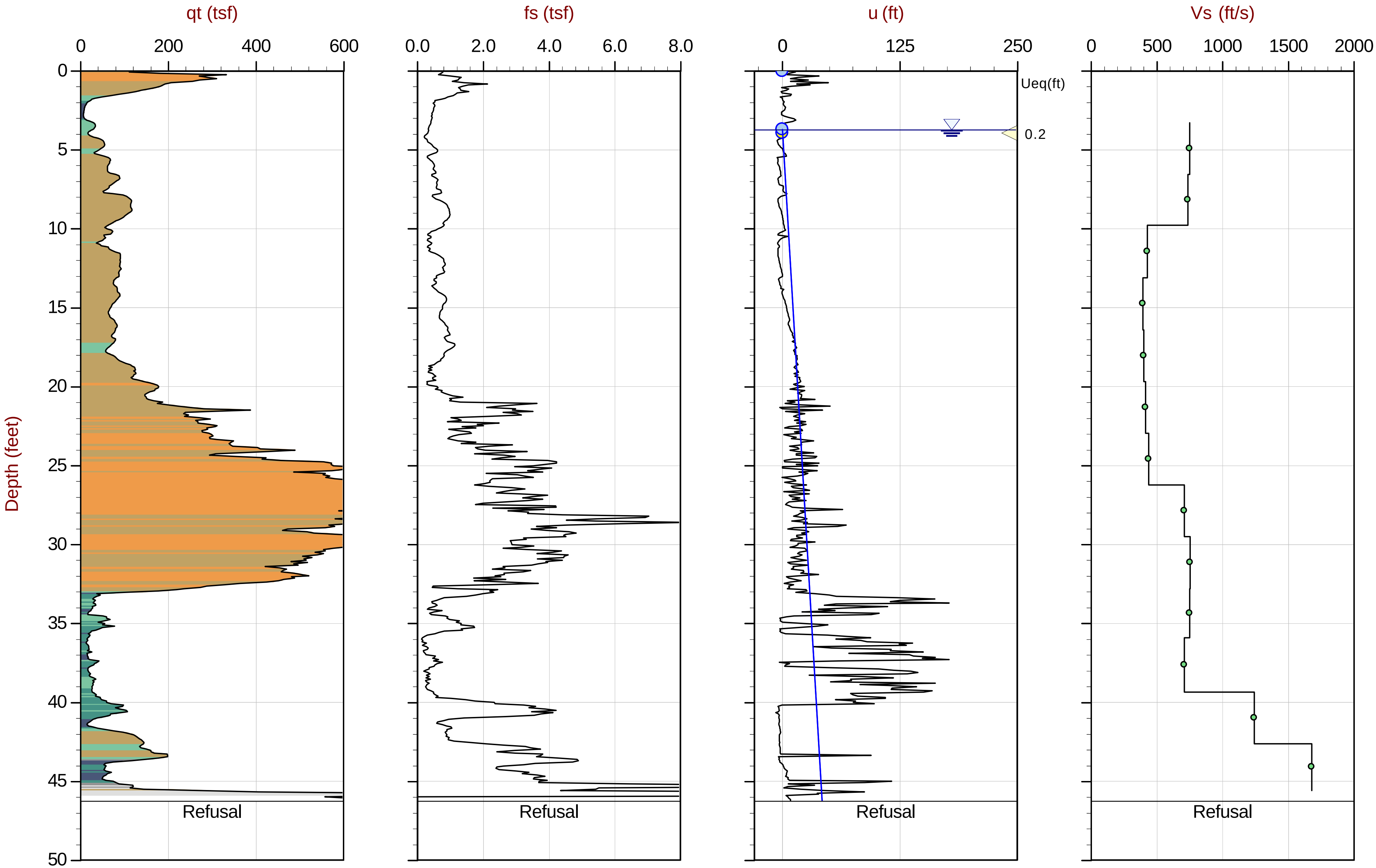
	Depth: 1.200 m / 3.937 ft	u Max: 14.5 ft	Ueq: 0.2 ft
	Duration: 700.0 s	u Final: 0.3 ft	

ConeTec | Krazan

Job No: 22-59-23911
Date: 2022-03-30 17:07
Site: Puyallup CPT
Sounding: CPT-02B
Cone: 781:T1500F15U35



Max Depth: 14.100 m / 46.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.47872 Long: -122.29435

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

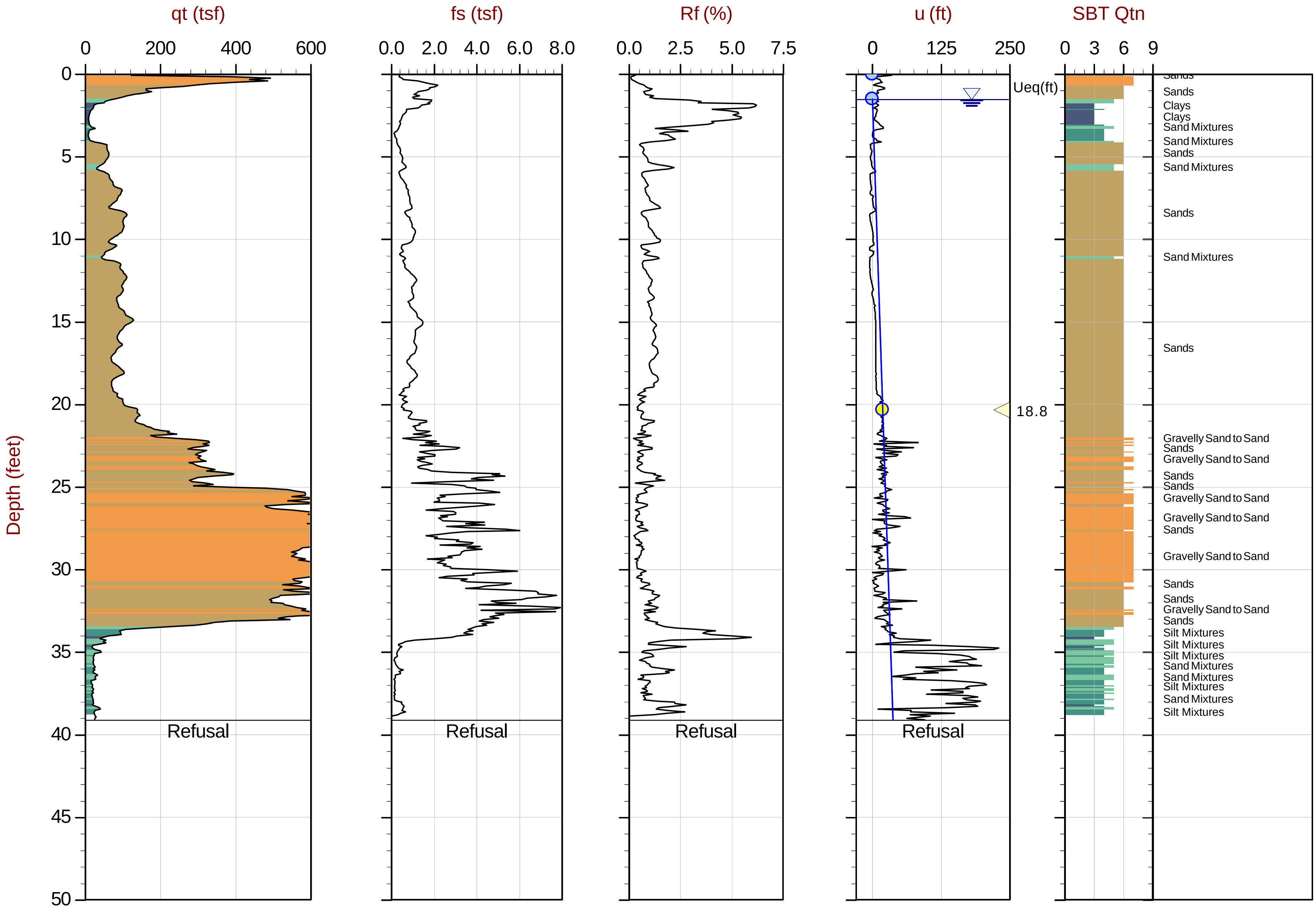
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 14:45
Site: Puyallup CPT

Sounding: CPT-02
Cone: 781:T1500F15U35



Max Depth: 11.925 m / 39.12 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17871 Long: -122.29435

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

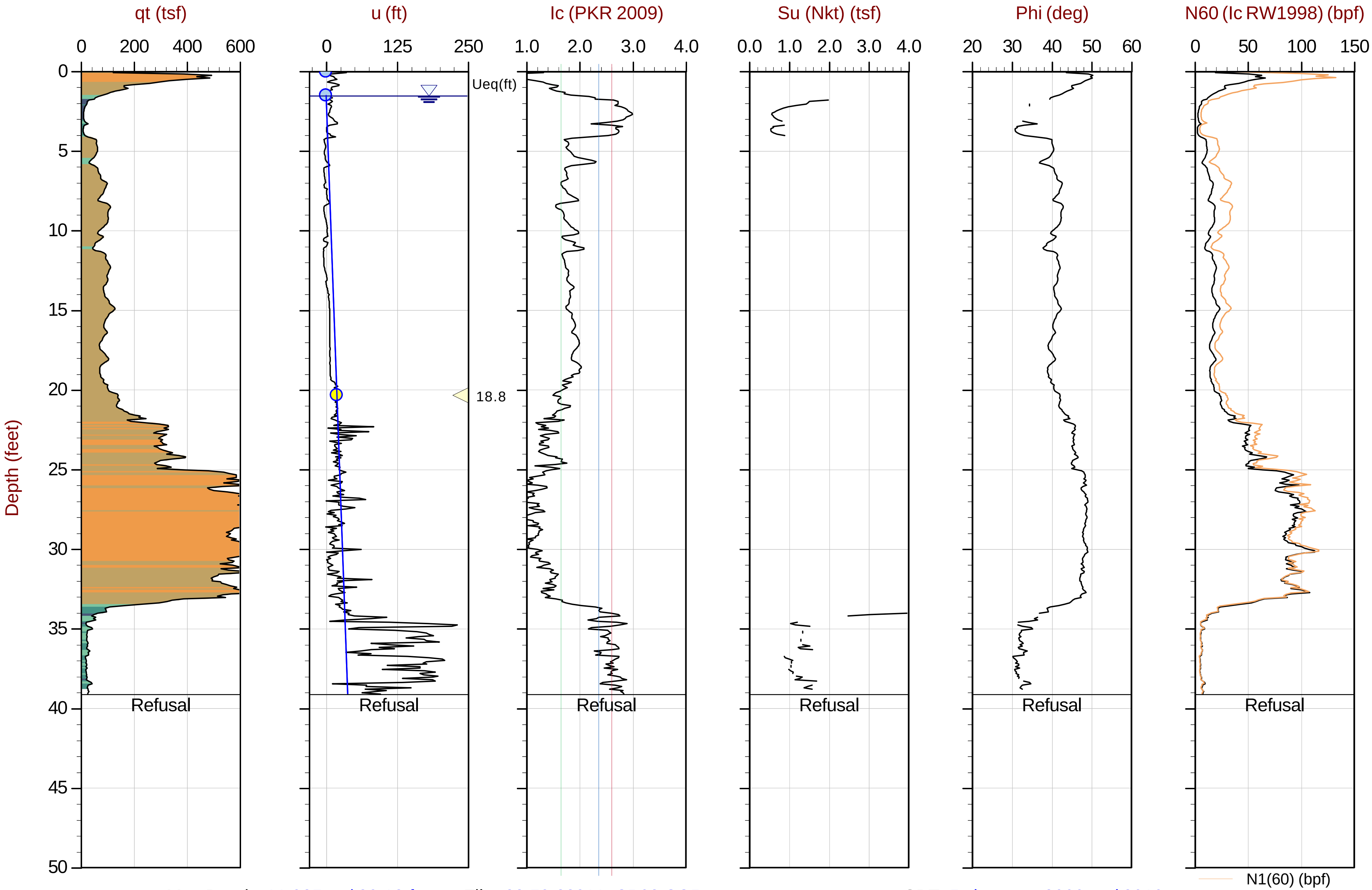
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 14:45
Site: Puyallup CPT

Sounding: CPT-02
Cone: 781:T1500F15U35



Max Depth: 11.925 m / 39.12 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17871 Long: -122.29435

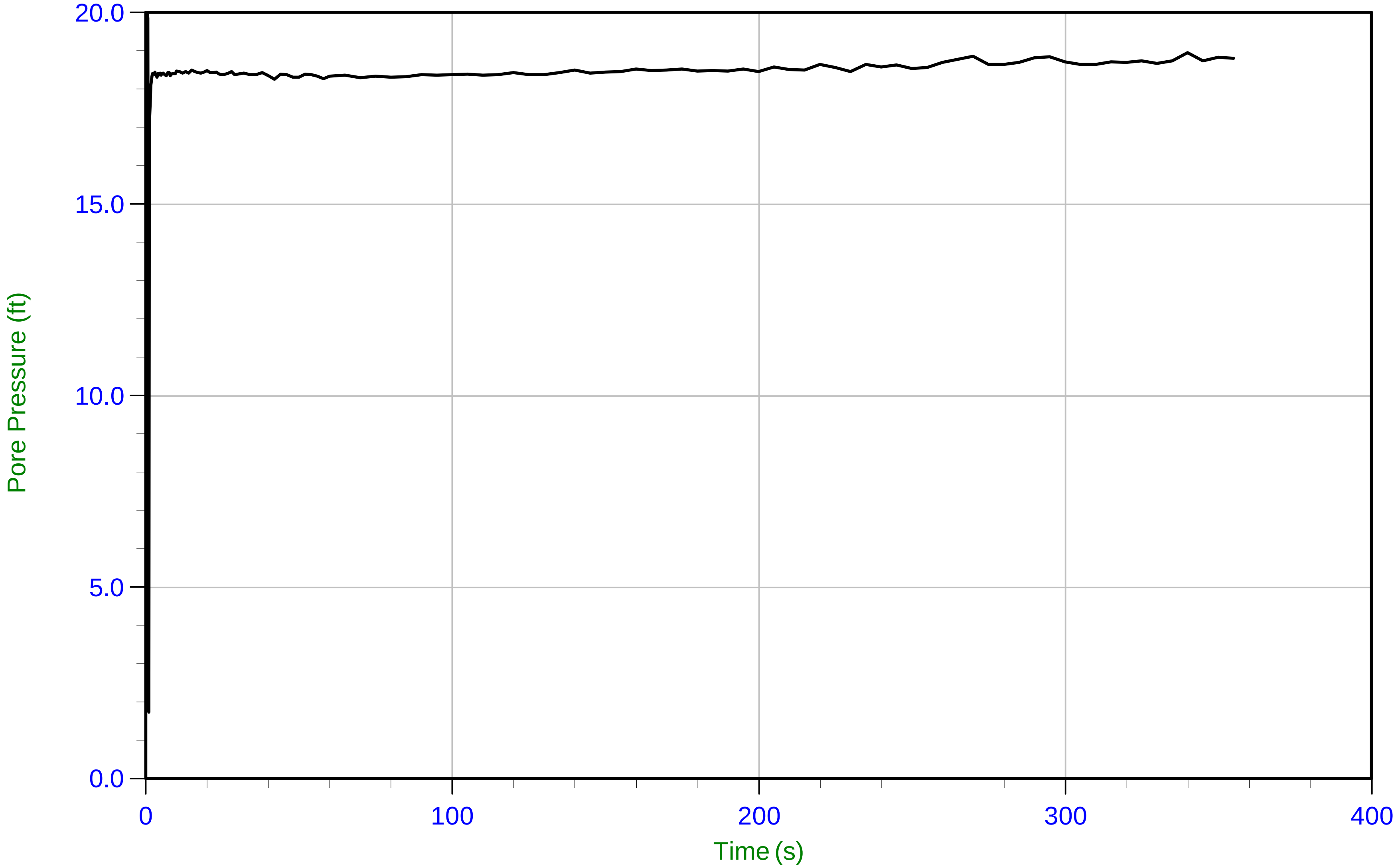
Equilibrium Pore Pressure (Ueq) · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Krazan

Job No: 22-59-23911
Date: 03/30/2022 14:45
Site: Puyallup

Sounding: CPT-02
Cone: 781:T1500F15U35 Area=15 cm²



Trace Summary:

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Depth: 6.200 m / 20.341 ft
Duration: 355.0 s

u Min: 1.7 ft
u Max: 20.1 ft
u Final: 18.8 ft

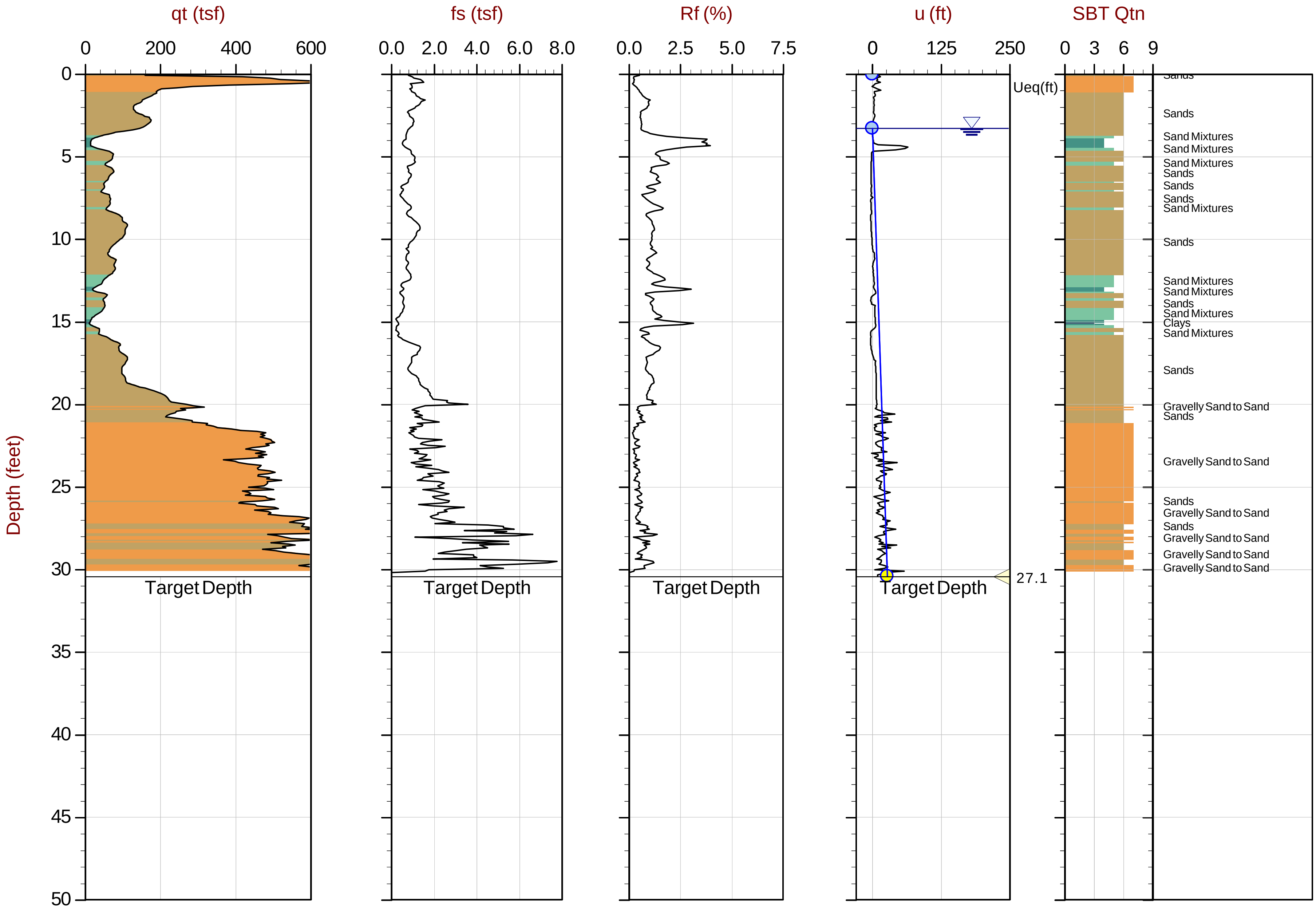
WT: 0.461 m / 1.513 ft
Ueq: 18.8 ft

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 15:29
Site: Puyallup CPT

Sounding: CPT-03
Cone: 781:T1500F15U35



Max Depth: 9.275 m / 30.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17856 Long: -122.29418

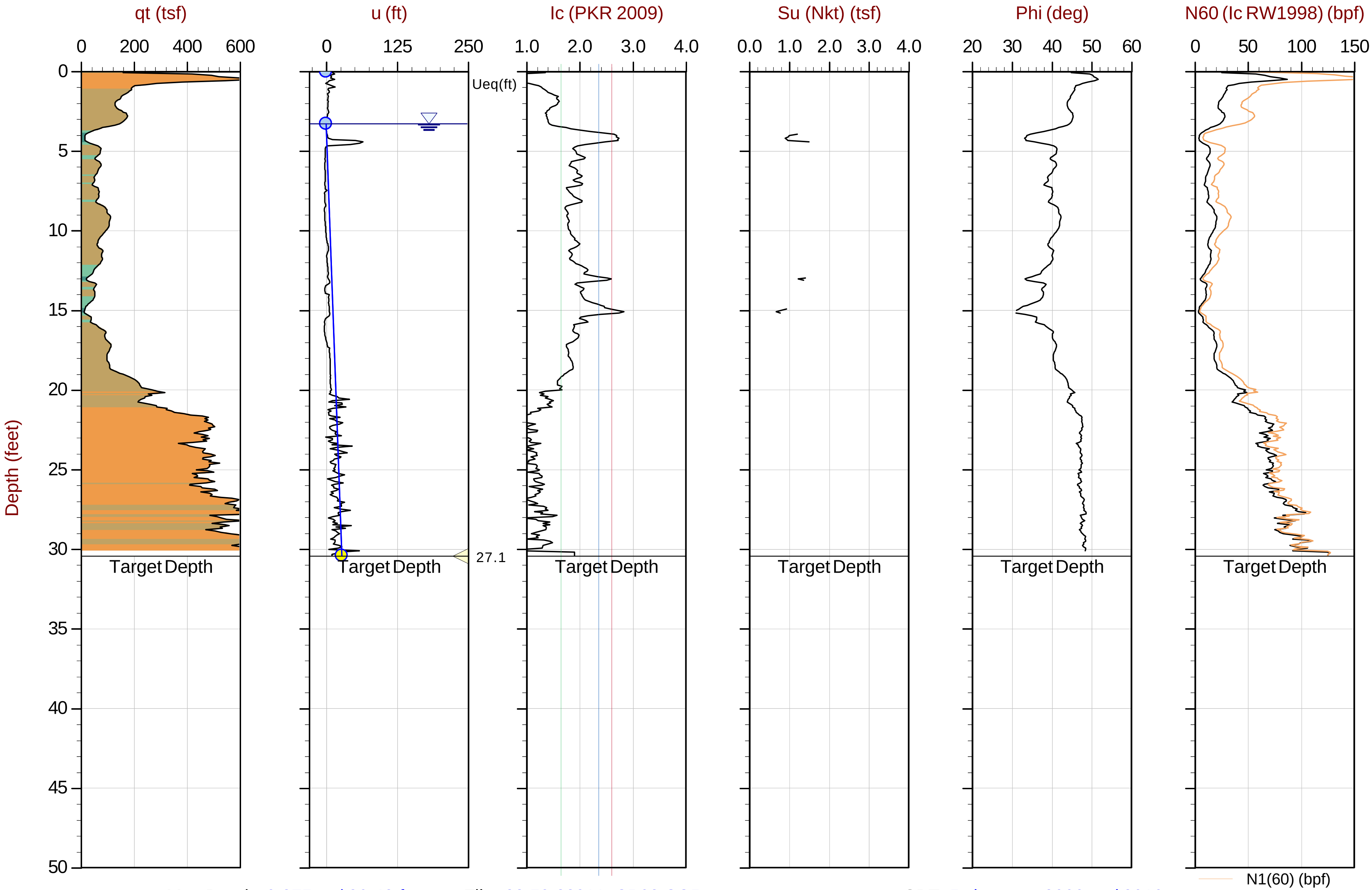
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Krazan

Job No: 22-59-23911
Date: 2022-03-30 15:29
Site: Puyallup CPT

Sounding: CPT-03
Cone: 781:T1500F15U35



Max Depth: 9.275 m / 30.43 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

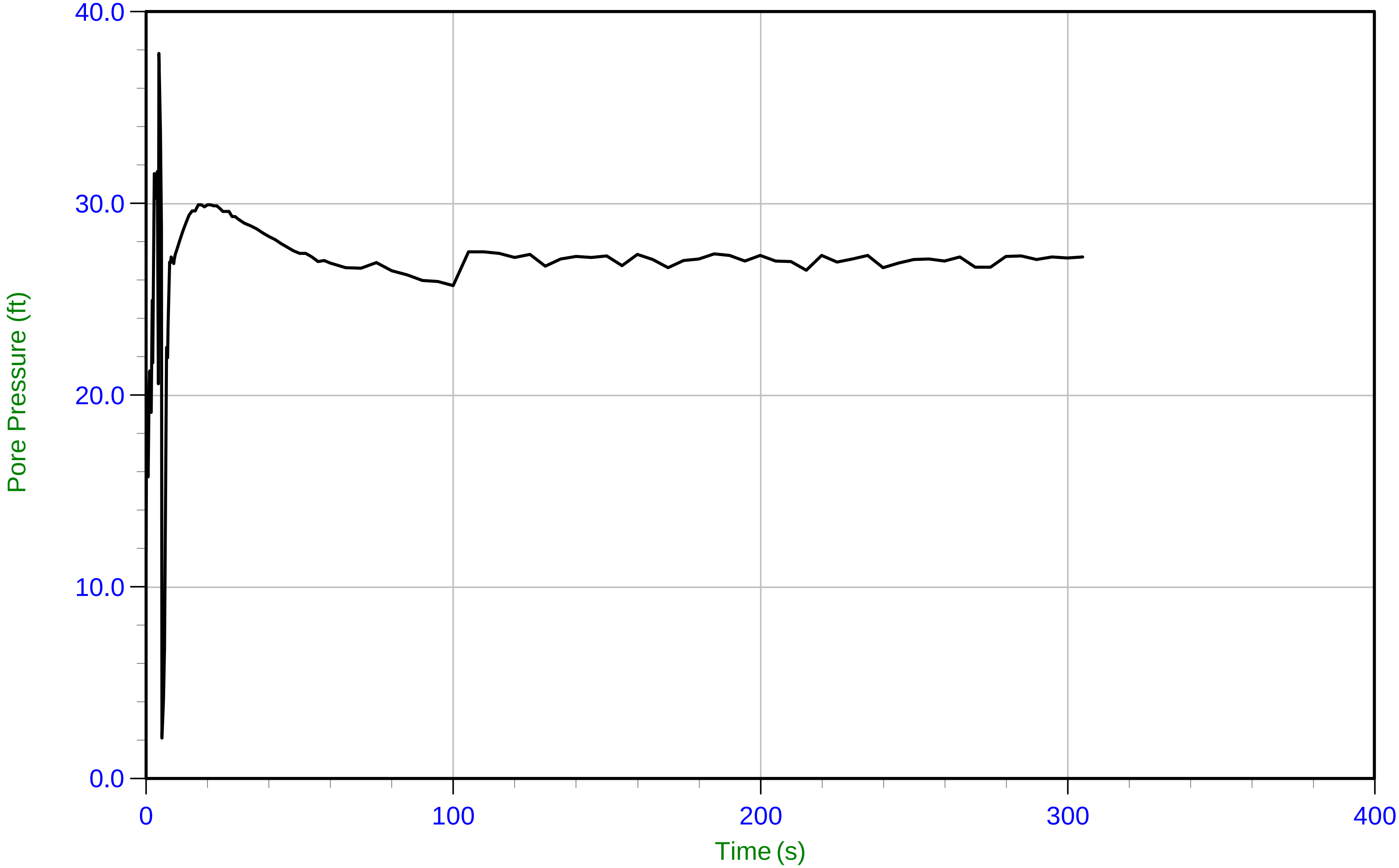
File: 22-59-23911_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17856 Long: -122.29418

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Krazan	Job No: 22-59-23911 Date: 03/30/2022 15:29 Site: Puyallup	Sounding: CPT-03 Cone: 781:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-59-23911_CP03.ppd2
Depth: 9.275 m / 30.429 ft
Duration: 305.0 s

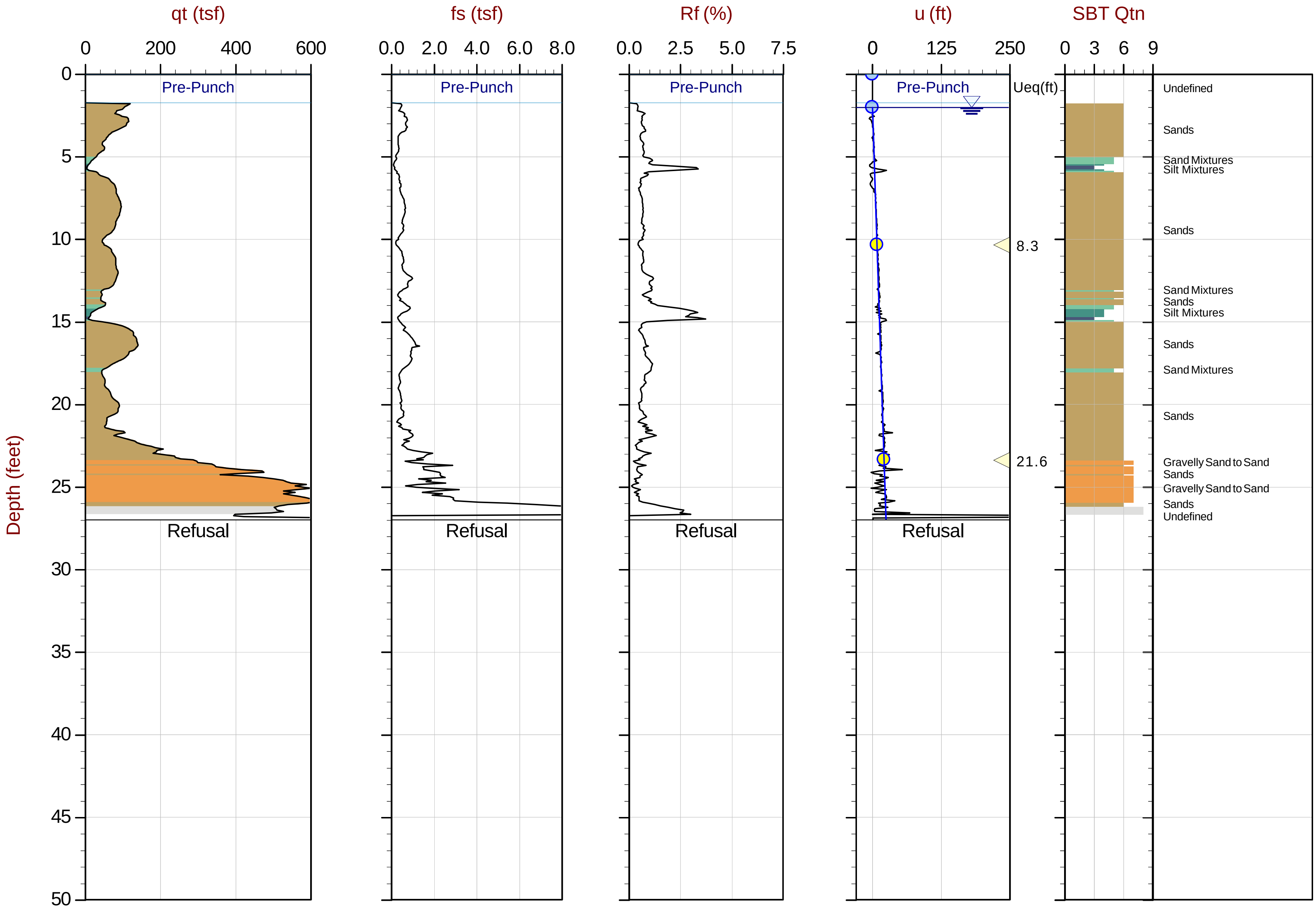
u Min: 2.1 ft
u Max: 37.8 ft
u Final: 27.2 ft

WT: 1.001 m / 3.284 ft
Ueq: 27.1 ft

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 12:23
Site: Puyallup CPT
Sounding: CPT-04
Cone: 781:T1500F15U35



Max Depth: 8.225 m / 26.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17839 Long: -122.29395

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

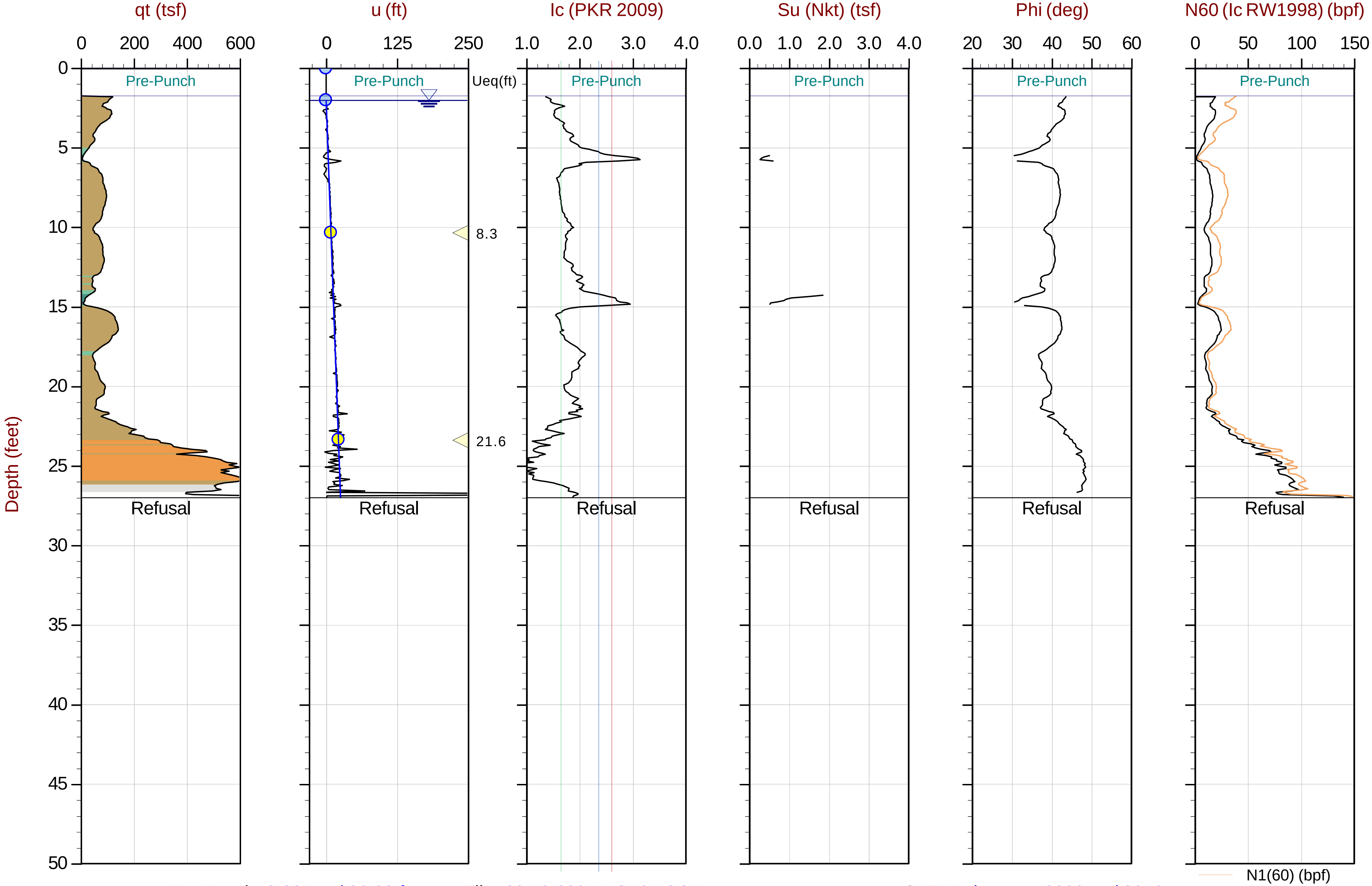
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 12:23
Site: Puyallup CPT

Sounding: CPT-04
Cone: 781:T1500F15U35



Max Depth: 8.225 m / 26.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17839 Long: -122.29395

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

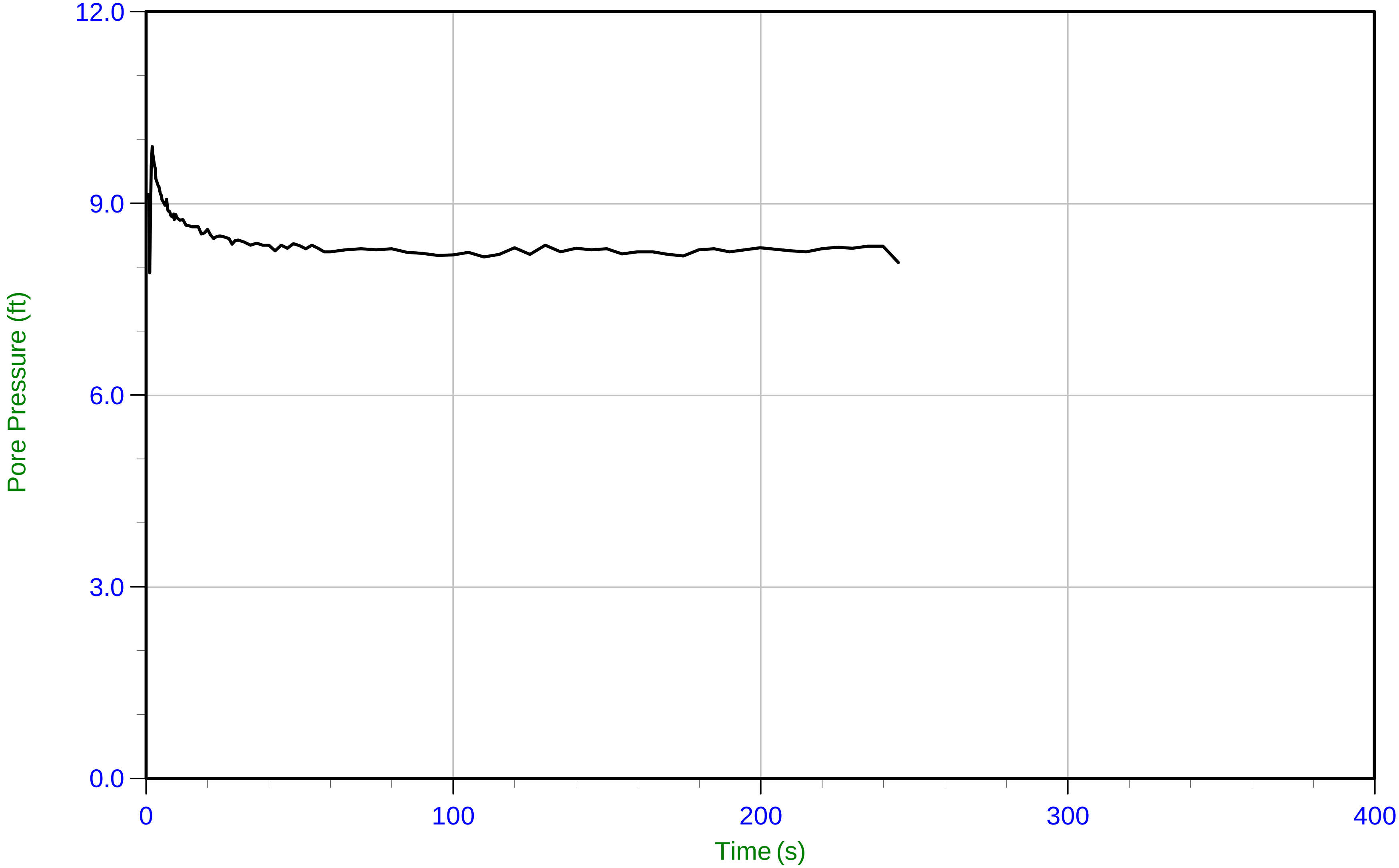
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Krazan

Job No: 22-59-23911
Date: 03/30/2022 12:23
Site: Puyallup

Sounding: CPT-04
Cone: 781:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-59-23911_SP04.ppd2
Depth: 3.150 m / 10.335 ft
Duration: 245.0 s

u Min: 7.9 ft
u Max: 9.9 ft
u Final: 8.1 ft

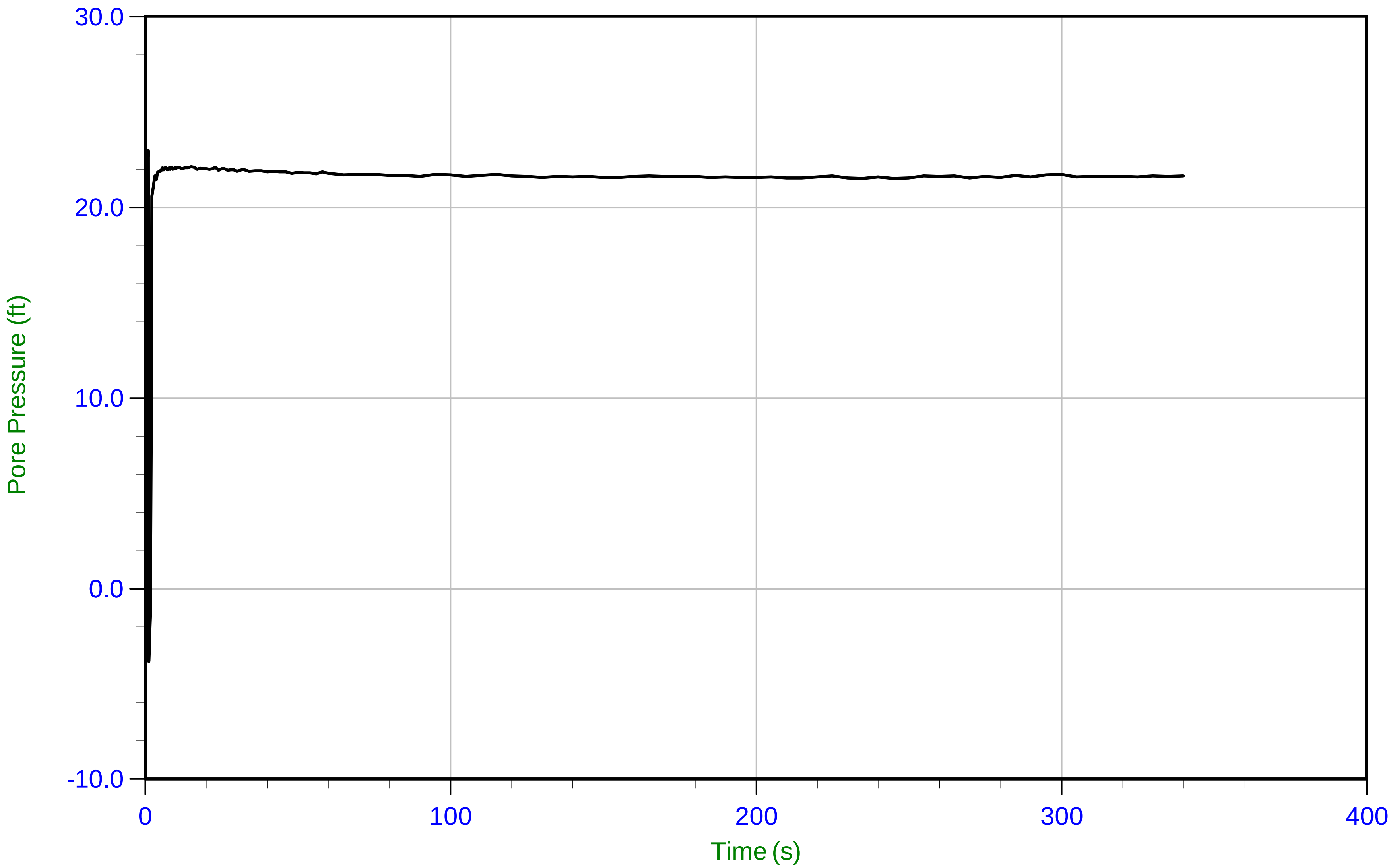
WT: 0.612 m / 2.009 ft
Ueq: 8.3 ft

ConeTec

Krazan

Job No: 22-59-23911
Date: 03/30/2022 12:23
Site: Puyallup

Sounding: CPT-04
Cone: 781:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-59-23911_SP04.ppd2
Depth: 7.125 m / 23.376 ft
Duration: 340.0 s

u Min: -3.8 ft
u Max: 23.0 ft
u Final: 21.7 ft

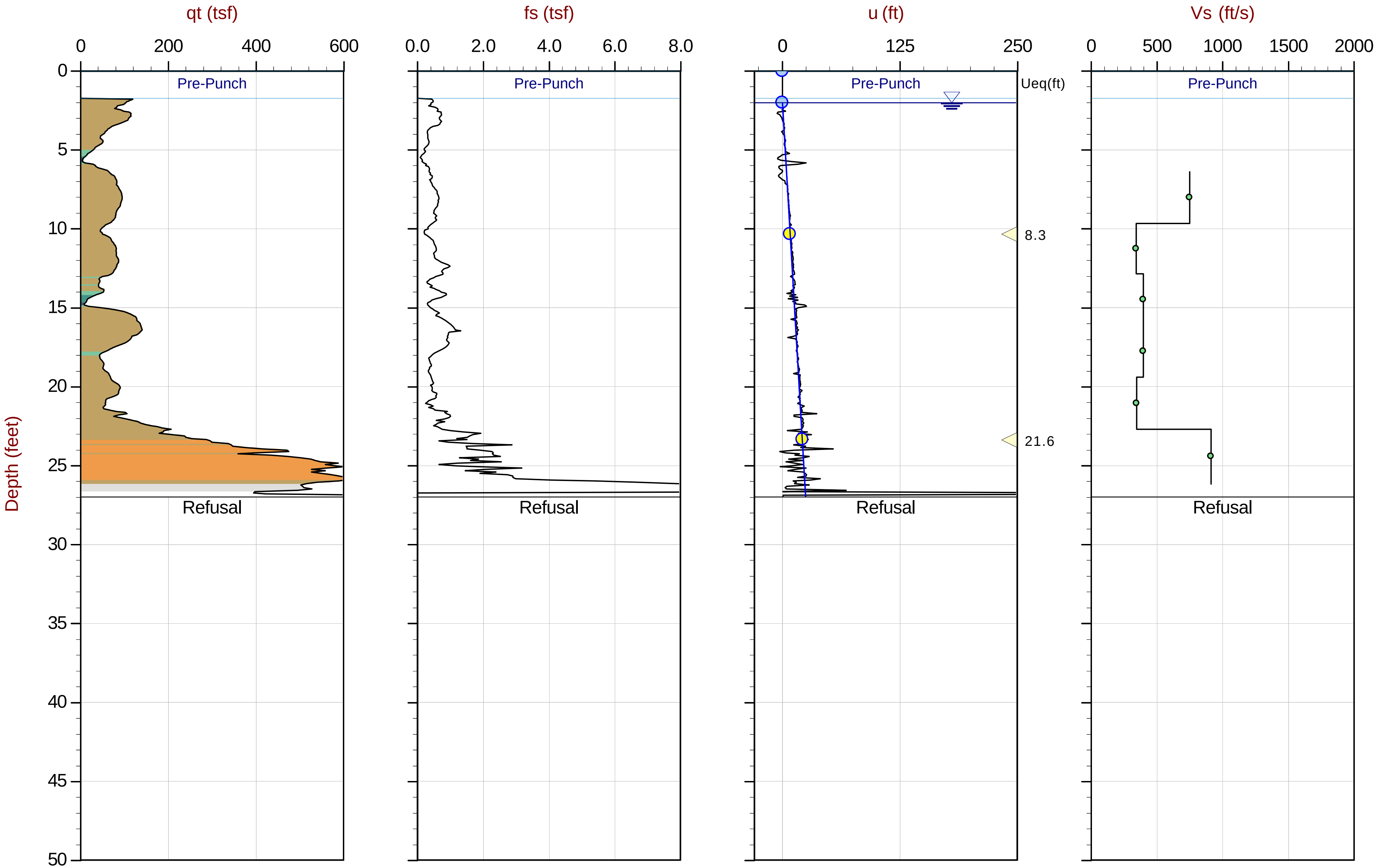
WT: 0.542 m / 1.779 ft
Ueq: 21.6 ft

ConeTec

Krazan

Job No: 22-59-23911
Date: 2022-03-30 12:23
Site: Puyallup CPT

Sounding: CPT-04
Cone: 781:T1500F15U35



Max Depth: 8.225 m / 26.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17839 Long: -122.29395

Equilibrium Pore Pressure (Ueq) · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Hydrostatic Line

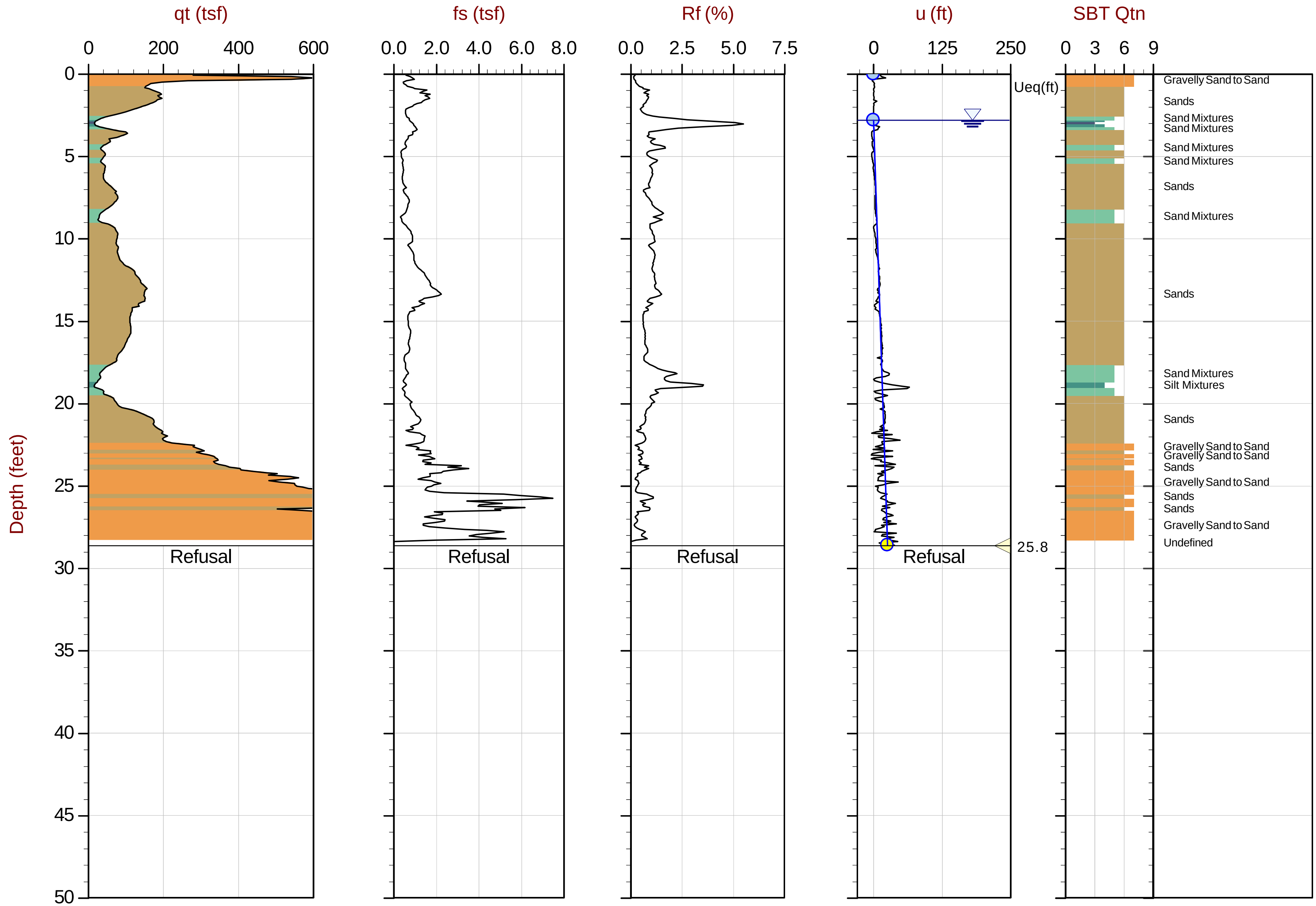
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Krazan

Job No: 22-59-23911
Date: 2022-03-30 16:06
Site: Puyallup CPT

Sounding: CPT-05
Cone: 781:T1500F15U35



Max Depth: 8.725 m / 28.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP05.COR
Unit Wt: SBTQtn (PKR2009)

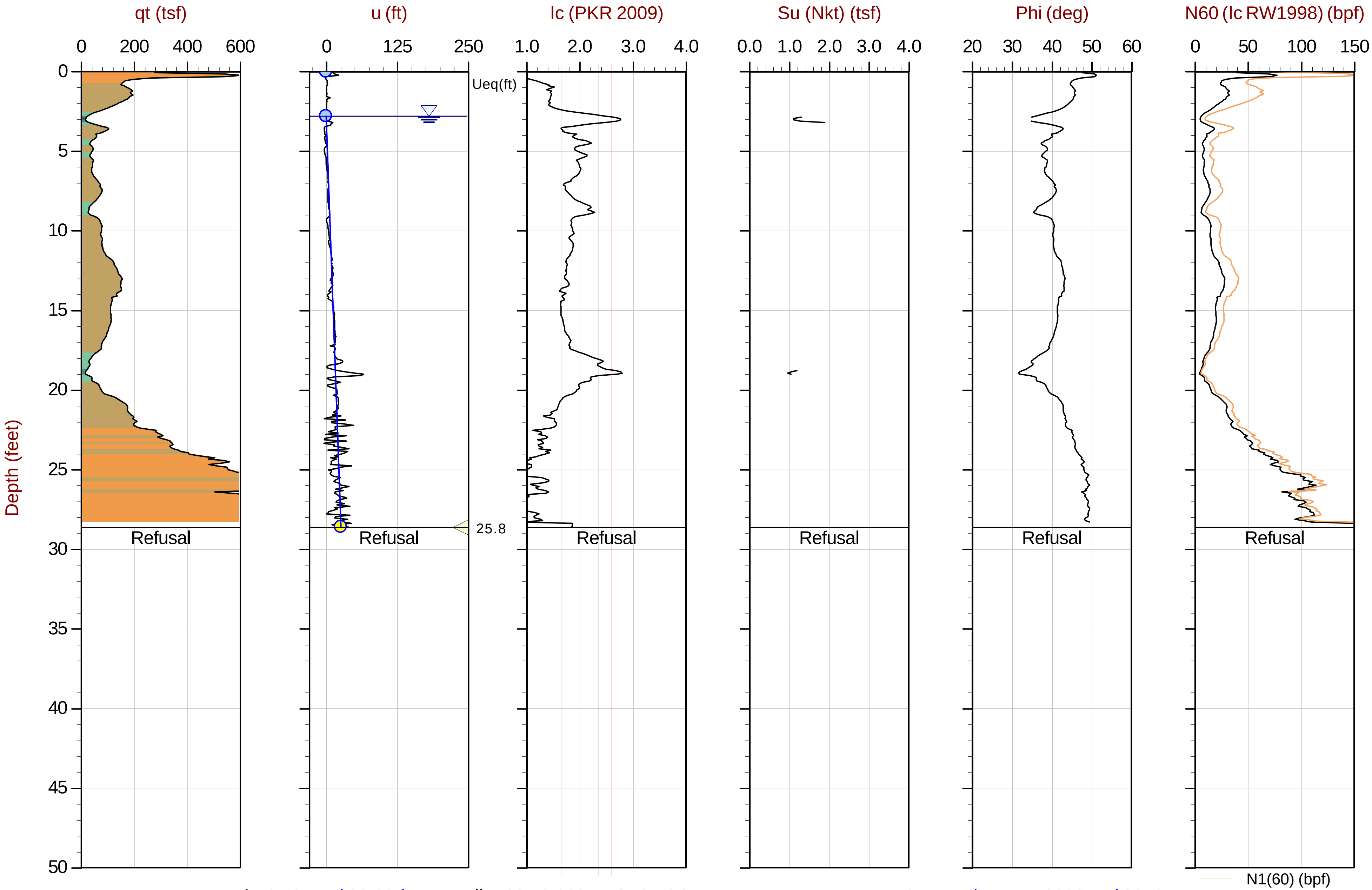
SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17827 Long: -122.29415

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Job No: 22-59-23911
Date: 2022-03-30 16:06
Site: Puyallup CPT
Sounding: CPT-05
Cone: 781:T1500F15U35

Krazan



Max Depth: 8.725 m / 28.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17827 Long: -122.29415

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

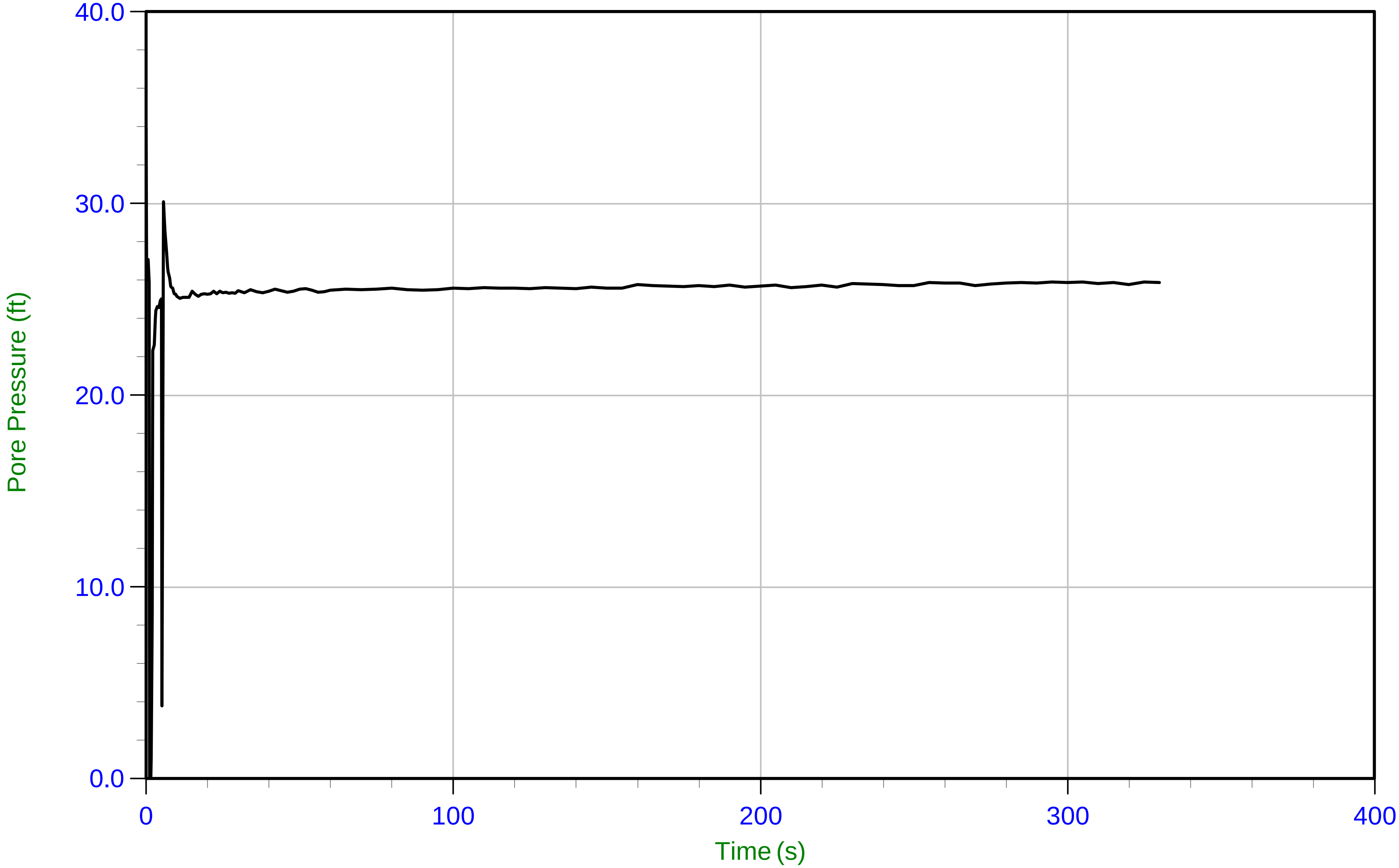
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Krazan

Job No: 22-59-23911
Date: 03/30/2022 16:06
Site: Puyallup

Sounding: CPT-05
Cone: 781:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 22-59-23911_SP05.ppd2
Depth: 8.725 m / 28.625 ft
Duration: 330.0 s

u Min: -3.3 ft
u Max: 33.9 ft
u Final: 25.9 ft

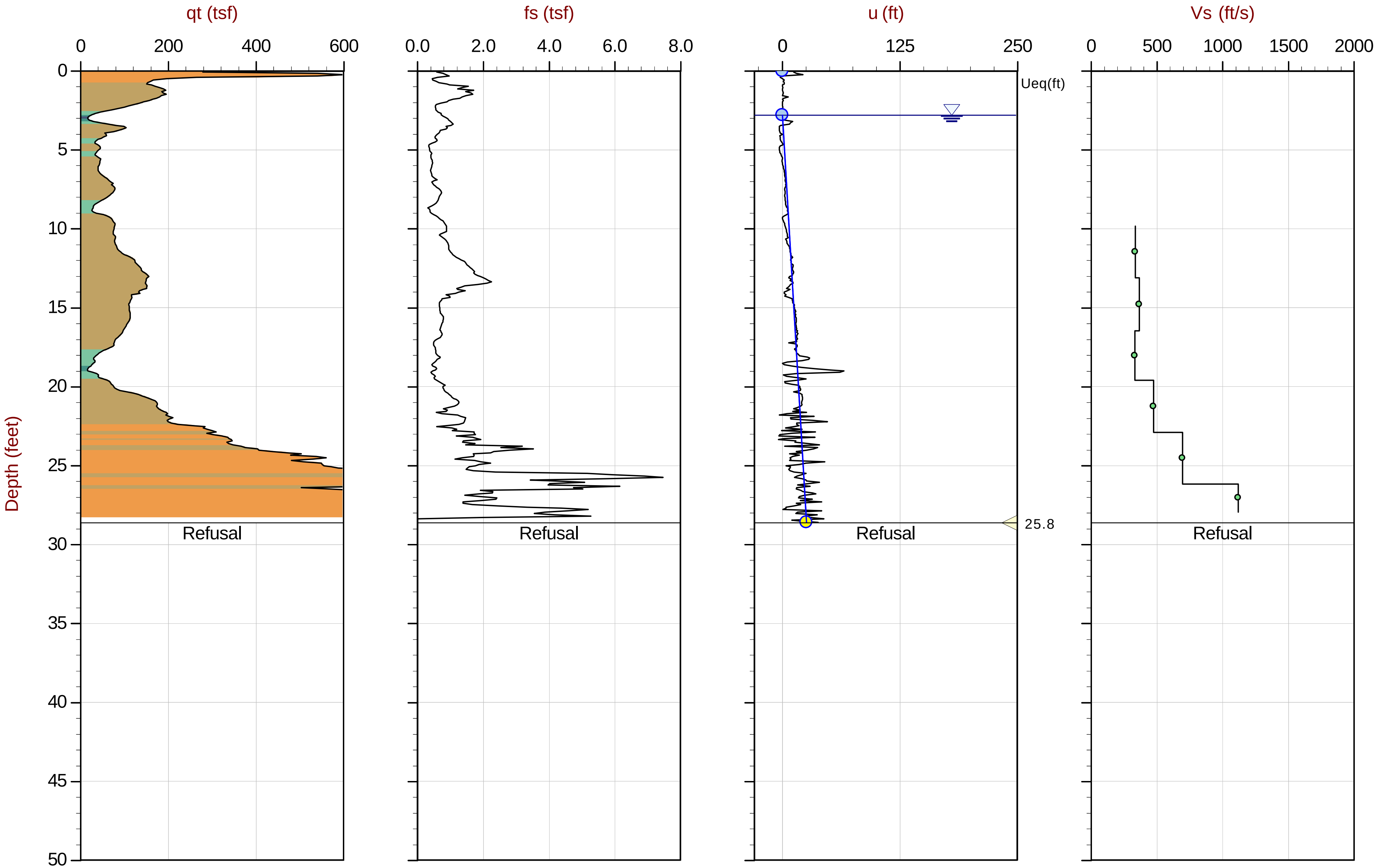
WT: 0.854 m / 2.802 ft
Ueq: 25.8 ft

ConeTec

Krazan

Job No: 22-59-23911
Date: 2022-03-30 16:06
Site: Puyallup CPT

Sounding: CPT-05
Cone: 781:T1500F15U35



Max Depth: 8.725 m / 28.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 22-59-23911_SP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 47.17827 Long: -122.29415

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

APPENDIX B

EARTHWORK SPECIFICATIONS

GENERAL

When the text of the report conflicts with the general specifications in this appendix, the recommendations in the report have precedence.

SCOPE OF WORK: These specifications and applicable plans pertain to and include all earthwork associated with the site rough grading, including but not limited to the furnishing of all labor, tools, and equipment necessary for site clearing and grubbing, stripping, preparation of foundation materials for receiving fill, excavation, processing, placement and compaction of fill and backfill materials to the lines and grades shown on the project grading plans, and disposal of excess materials.

PERFORMANCE: The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications. This work shall be inspected and tested by a representative of Krazan and Associates, Inc., hereinafter known as the Geotechnical Engineer and/or Testing Agency. Attainment of design grades when achieved shall be certified by the project Civil Engineer. Both the Geotechnical Engineer and Civil Engineer are the Owner's representatives. If the contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary readjustments until all work is deemed satisfactory as determined by both the Geotechnical Engineer and Civil Engineer. No deviation from these specifications shall be made except upon written approval of the Geotechnical Engineer, Civil Engineer or project Architect.

No earthwork shall be performed without the physical presence or approval of the Geotechnical Engineer. The Contractor shall notify the Geotechnical Engineer at least 2 working days prior to the commencement of any aspect of the site earthwork. The Contractor agrees that he shall assume sole and complete responsibility for job site conditions during the course of construction of this project, including safety of all persons and property; that this requirement shall apply continuously and not be limited to normal working hours; and that the Contractor shall defend, indemnify and hold the Owner and the Engineers harmless from any and all liability, real or alleged, in connection with the performance of work on this project, except for liability arising from the sole negligence of the Owner of the Engineers.

TECHNICAL REQUIREMENTS: All compacted materials shall be moisture conditioned to within 2 percent of the materials optimum moisture content and compacted to a density not less than 95 percent of maximum dry density as determined by ASTM Test Method D1557, unless specified otherwise in the technical portion of the Geotechnical Engineering Report. The results of these tests and compliance with these specifications shall be the basis upon which satisfactory completion of work will be judged by the Geotechnical Engineer.

SOIL AND FOUNDATION CONDITIONS: The Contractor is presumed to have visited the site and to have familiarized himself with existing site conditions and the contents of the data presented in the soil report. The Contractor shall make his own interpretation of the data contained in said report, and the Contractor shall not be relieved of liability under the contractor for any loss sustained as a result of any variance between conditions indicated by or deduced from said report and the actual conditions encountered during the progress of the work.

DUST CONTROL: The work includes dust control as required for the alleviation or prevention of any dust nuisance on or about the site or the borrow area, or off-site if caused by the Contractor's operation either during the performance of the earthwork or resulting from the conditions in which the Contractor leaves the site. The Contractor shall assume all liability, including Court costs of codefendants, for all claims related to dust or windblown materials attributable to his work.

SITE PREPARATION

Site preparation shall consist of site clearing and grubbing and preparations of foundation materials for receiving fill.

CLEARING AND GRUBBING: The Contractor shall accept the site in this present condition and shall demolish and/or remove from the area of designated project earthwork all structures, both surface and subsurface, trees, brush, roots, debris, organic matter, and all other matter determined by the Geotechnical Engineer to be deleterious. Such materials shall become the property of the Contractor and shall be removed from the site.

Tree root systems in proposed building area should be removed to a minimum depth of 3 feet and to such an extent which would permit removal of all roots larger than 1 inch. Tree roots removed in parking areas may be limited to the upper 1½ feet of the ground surface. Backfill or tree root excavation should not be permitted until all exposed surfaces have been inspected and the Geotechnical Engineer is present for the proper control of backfill placement and compaction. Burning in areas which are to receive fill materials shall not be permitted.

SUBGRADE PREPARATION: Subgrade should be prepared as described in our site preparation section of this report.

EXCAVATION: All excavations shall be accomplished to the tolerance normally defined by the Civil Engineer as shown on the project grading plans. All excavations extending beyond the excavation or over-excavation limits specified shall be backfilled at the Contractor's expense and shall be compacted in accordance with the applicable technical requirements.

FILL AND BACKFILL MATERIAL: No material shall be moved or compacted without the presence of the Geotechnical Engineer. Material from the required site excavation may be utilized for construction site fills provided prior approval is given by the Geotechnical Engineer and the compaction requirements can be met. All materials utilized for constructing site fills shall be free from vegetable or other deleterious matter as determined by the Geotechnical Engineer.

PLACEMENT, SPREADING AND COMPACTION: The placement and spreading of approved fill materials and the processing and compaction of approved fill and native materials shall be the responsibility of the Contractor. However, **compaction of fill materials by flooding, ponding, or jetting shall not be permitted**. Both cut and fill shall be compacted to the satisfaction of the Geotechnical Engineer prior to final acceptance.

SEASONAL LIMITS: No fill material shall be placed, spread, or rolled while it is frozen or thawing or during unfavorable wet weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until the Geotechnical Engineer indicates that the moisture content and density of previously placed fill are as specified.

APPENDIX C

PAVEMENT SPECIFICATIONS

1. DEFINITIONS – The term “pavement” shall include asphalt concrete surfacing, untreated aggregate base, and aggregate subbase. The term “subgrade” is that portion of the area on which surfacing, base, or subbase is to be placed.

2. SCOPE OF WORK – This portion of the work shall include all labor, materials, tools, and equipment necessary for and reasonably incidental to the completion of the pavement shown on the plans and as herein specified, except work specifically notes as “Work Not Included.”

3. PREPARATION OF THE SUBGRADE – Subgrade should be prepared as described in our site preparation and pavement design sections of this report.

4. AGGREGATE BASE – The aggregate base shall be spread and compacted on the prepared subgrade in conformity with the lines, grades, and dimensions shown on the plans. The aggregate base should conform to WSDOT Standard Specification for Crushed Surfacing Base Course or Top Course (Item 9-03.9(3)). The base material shall be compacted to a minimum compaction of 95% as determined by ASTM D1557 Modified Proctor. Each layer of subbase shall be tested and approved by the Geotechnical Engineer prior to the placement of successive layers.

5. ASPHALTIC CONCRETE SURFACING – Asphaltic concrete surfacing shall consist of a mixture of mineral aggregate and paving grade asphalt, mixed at a central mixing plant and spread and compacted on a prepared base in conformity with the lines, grades, and dimensions shown on the plans. The drying, proportioning, and mixing of the materials shall conform to WSDOT Specifications. The surface course shall be placed with an approved self-propelled mechanical spreading and finishing machine.

The prime coat, spreading and compaction equipment, as well as the process of spreading and compacting the mixture, shall conform to WSDOT Specifications, with the exception that no surface course shall be placed when the atmospheric temperature is below 50 degrees F. The surfacing shall be rolled with combination steel-wheel and pneumatic rollers, as described in WSDOT Specifications. The surface course shall be placed with an approved self-propelled mechanical spreading and finishing machine.

6. TACK COAT – The tack (mixing type asphaltic emulsion) shall conform to and be applied in accordance with the requirements of WSDOT Specifications.