



**BARGHAUSEN**

# **PRELIMINARY STORMWATER SITE PLAN**

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## **Convenience Store with Gas**

1402 South Meridian  
Puyallup, Washington 98371

City File No. PLPSP20220079



03/09/2023

Prepared for:  
BP Fuels, NA  
c/o Accenture  
P.O. Box 696049  
San Antonio, Texas 78269-9931

March 9, 2023  
Our Job No. 21730

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# **Preliminary Stormwater Site Plan**

Barghausen Consulting Engineers, Inc.

**Convenience Store with Gas**

Puyallup, Washington

Our Job No. 21730

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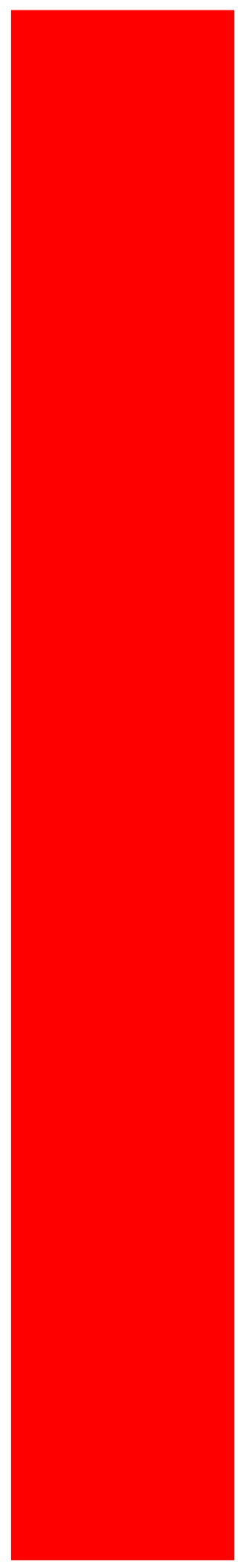
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# Tab 1.0



## 1.0 PROJECT OVERVIEW

The proposed redevelopment consists of an ARCO ampm convenience store, a fuel station canopy, two underground storage tanks (USTs), required fuel system components, and an automatic car wash located at 1402 S Meridian Street, Puyallup, Washington 98371. The development is located within Parcel Nos. 773000-028-1, 773000-028-8, 773000-003-1, and 773000-002-1 that consist of a total area of 1.20 acres. Construction activities include site clearing, grading, paving, structures, utilities, drainage, road improvements, and landscaping.

The existing site is primarily developed with a restaurant, asphalt pavement, parking stalls, an empty gravel truck parking area to the north, shrubs, trees, and other landscaping which shall all be demolished for the proposed development. The overall site topography varies, to the north of the site the slopes are between 0% to 3% and to the south of the site the slopes are between 2% and 14%. Per the draft geotechnical report by Krazan Associates, Inc., Appendix A in this report, stormwater infiltration is not feasible for this site. Additionally, groundwater was encountered at shallow depths.

The project proposes to add a new ARCO ampm convenience store, a fuel canopy with six multi-dispensers, two USTs, required fuel systems components, an automatic car wash, ADA parking, pavement, drainage, and associated utilities. Based on the Flow Chart for Determining Minimum Requirements for redevelopment found in the 2019 Washington State Department of Ecology Surface Water Management Manual for Western Washington (SWMMWW), Figure 1.6 of this report, the project will include more than 5,000 square feet of new plus replaced impervious areas; therefore, the project will meet Minimum Requirements Nos. 1 through 9. Refer to the Minimum Flow Chart included in this report for additional information.

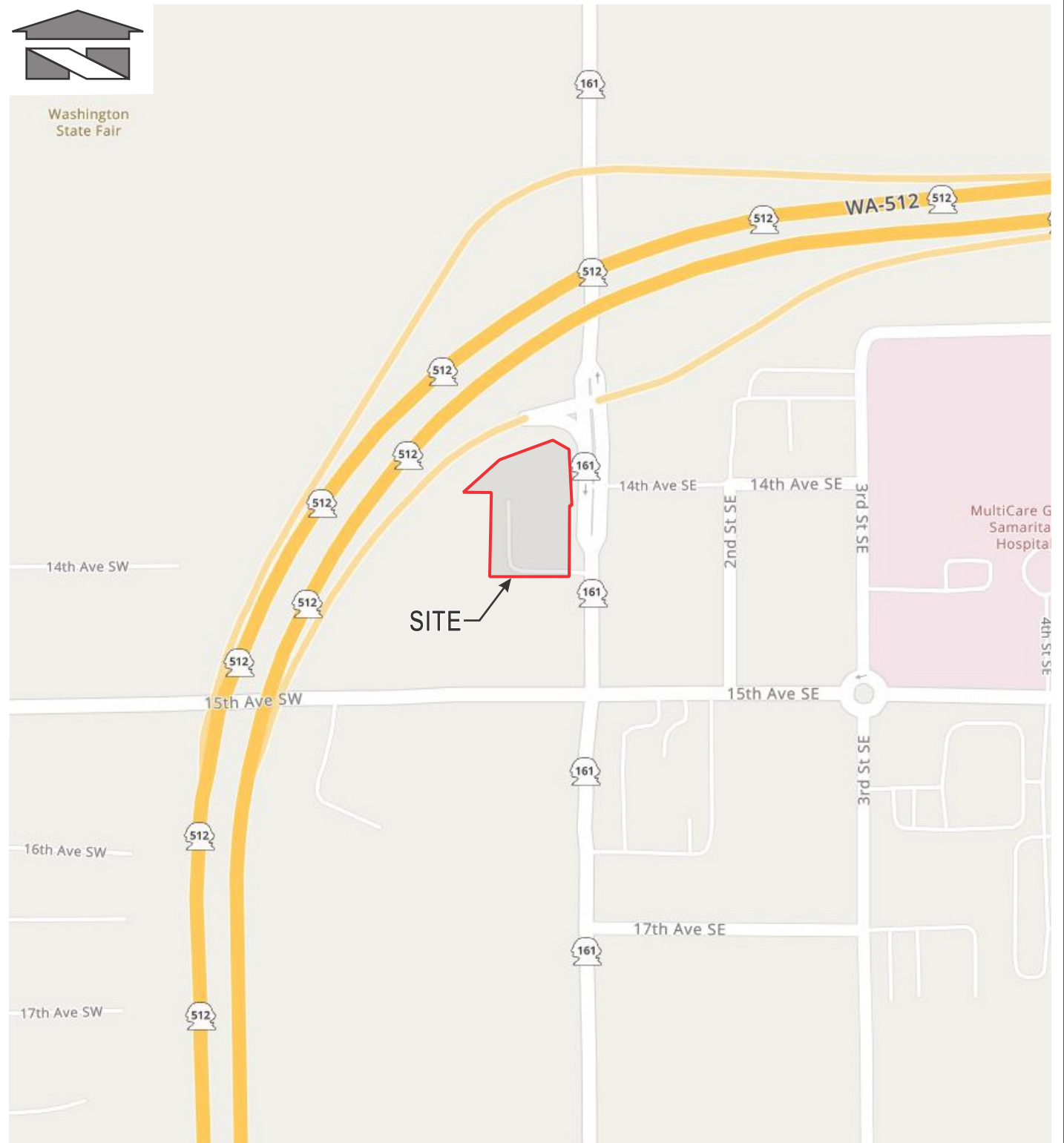
On-site drainage improvements include new catch basins, conveyance structures, proprietary runoff treatment facilities and a detention facility. The proposed detention facility shall be designed to meet Flow Control. Enhanced Treatment will be provided for pollution generating hard surfaces (PGHS) prior to discharge from the site.

# Figure 1.1 Vicinity Map





Washington State Fair



REFERENCE: MapQuest (2022)

Scale:

Horizontal: N.T.S. Vertical: N/A

For:

ARCO ampm  
Puyallup, Washington

Job Number

21730



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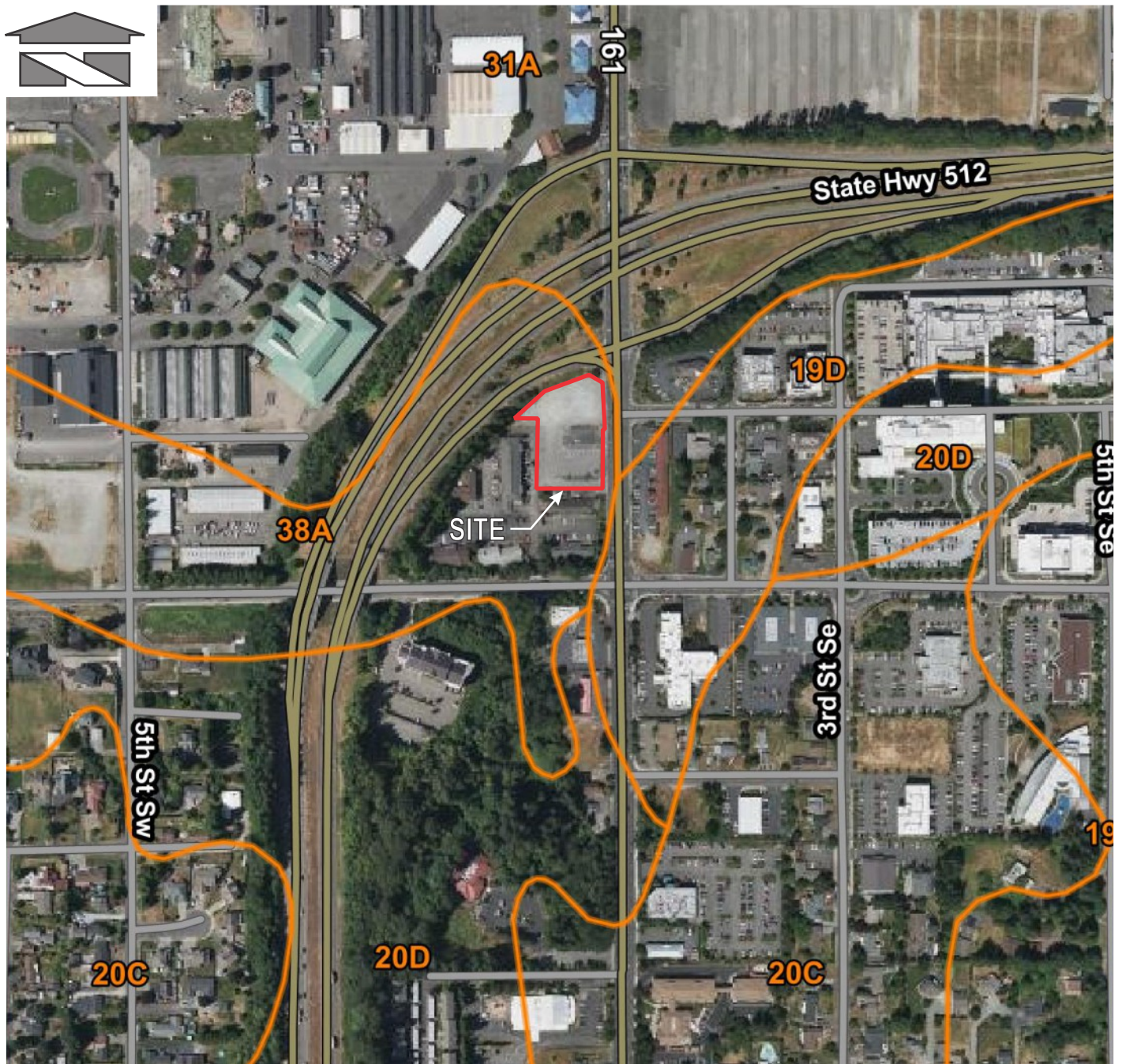
Title:

VICINITY MAP

DATE: 05/10/22

# Figure 1.2 Soil Survey Map





REFERENCE: USDA, Natural Resources Conservation Service

**LEGEND:**

38A = Shalcar muck  
 31A = Puyallup fine sandy loam

**HSG**

D  
 A

Scale:

Horizontal: N.T.S.      Vertical: N/A

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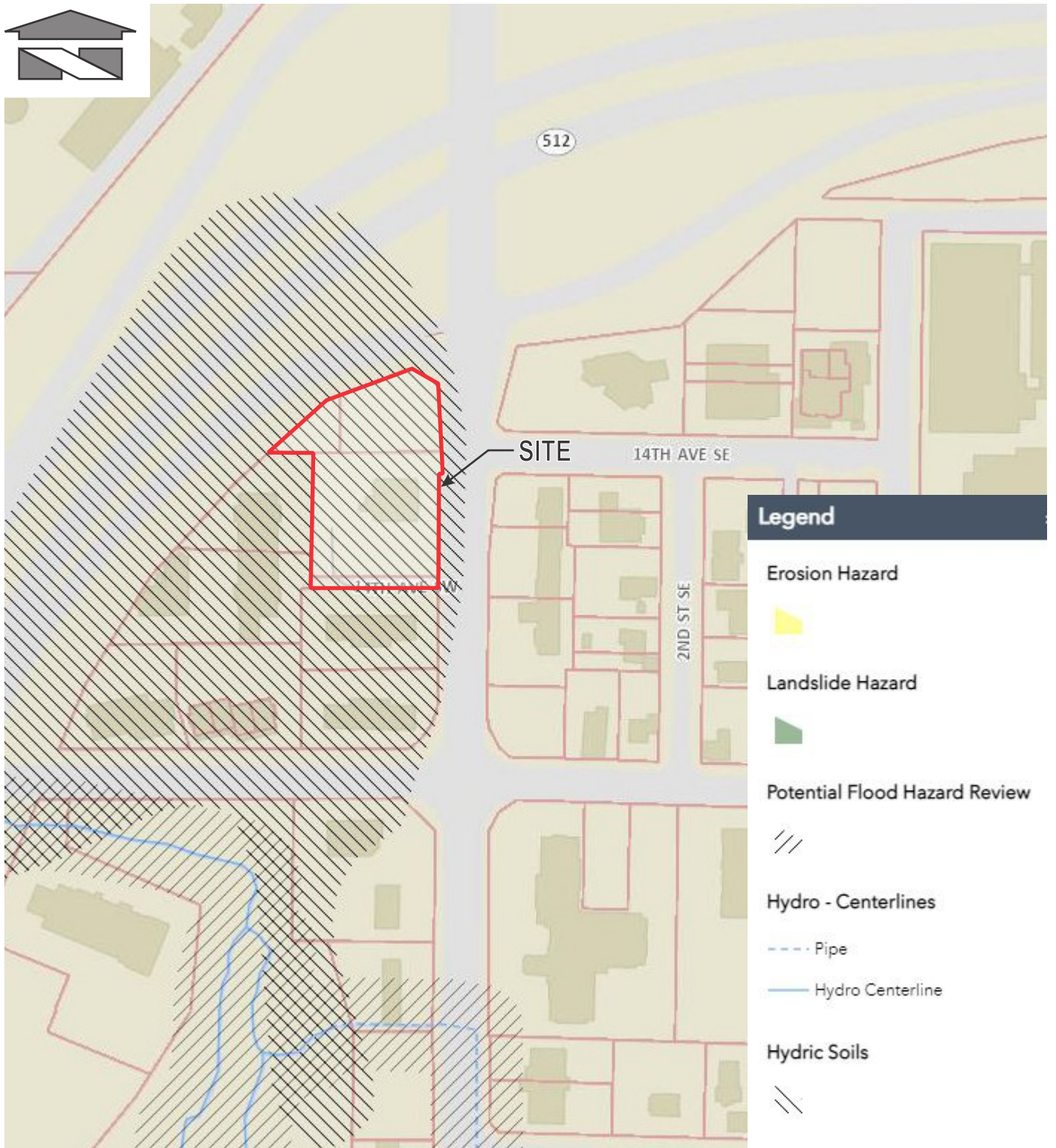
SOIL SURVEY MAP

DATE: 05/10/22

# Figure 1.3 Sensitive Areas Map







REFERENCE: Pierce County PublicGIS

Scale:

Horizontal: N.T.S.

Vertical: N/A

For:

ARCO ampm  
Puyallup, Washington

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21730

Title:

SENSITIVE AREAS  
MAP

DATE: 05/10/22



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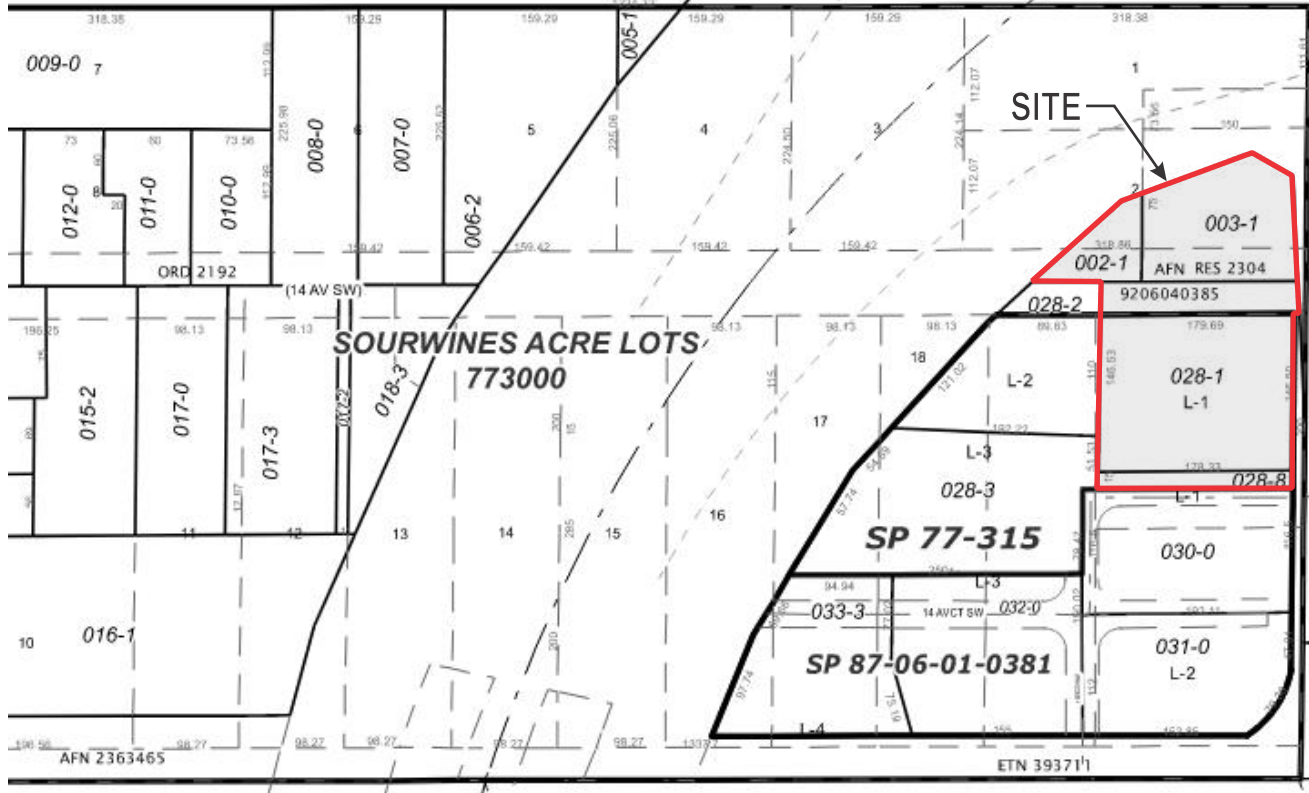


# Figure 1.4 Assessor's Map





1-121



REFERENCE: Pierce County Department of Assessments (Feb. 2021)

Scale:

Horizontal: N.T.S. Vertical: N/A

For:

ARCO ampm  
Puyallup, Washington

Job Number

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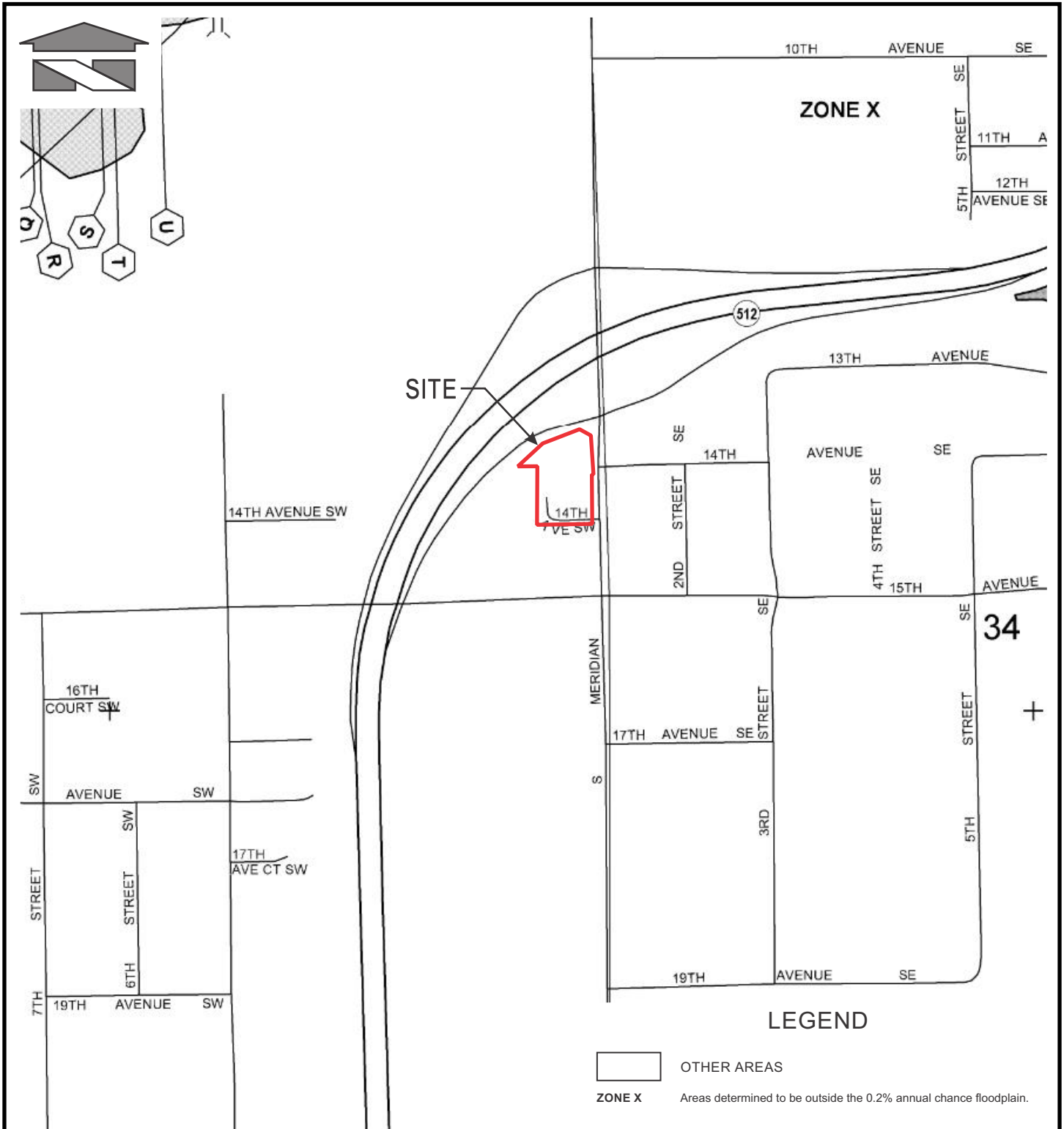
Title:

ASSESSOR MAP

DATE: 05/10/22

# Figure 1.5 FEMA Map





REFERENCE: Federal Emergency Management Agency (Portion of Map 53053C0341E, March 2017)


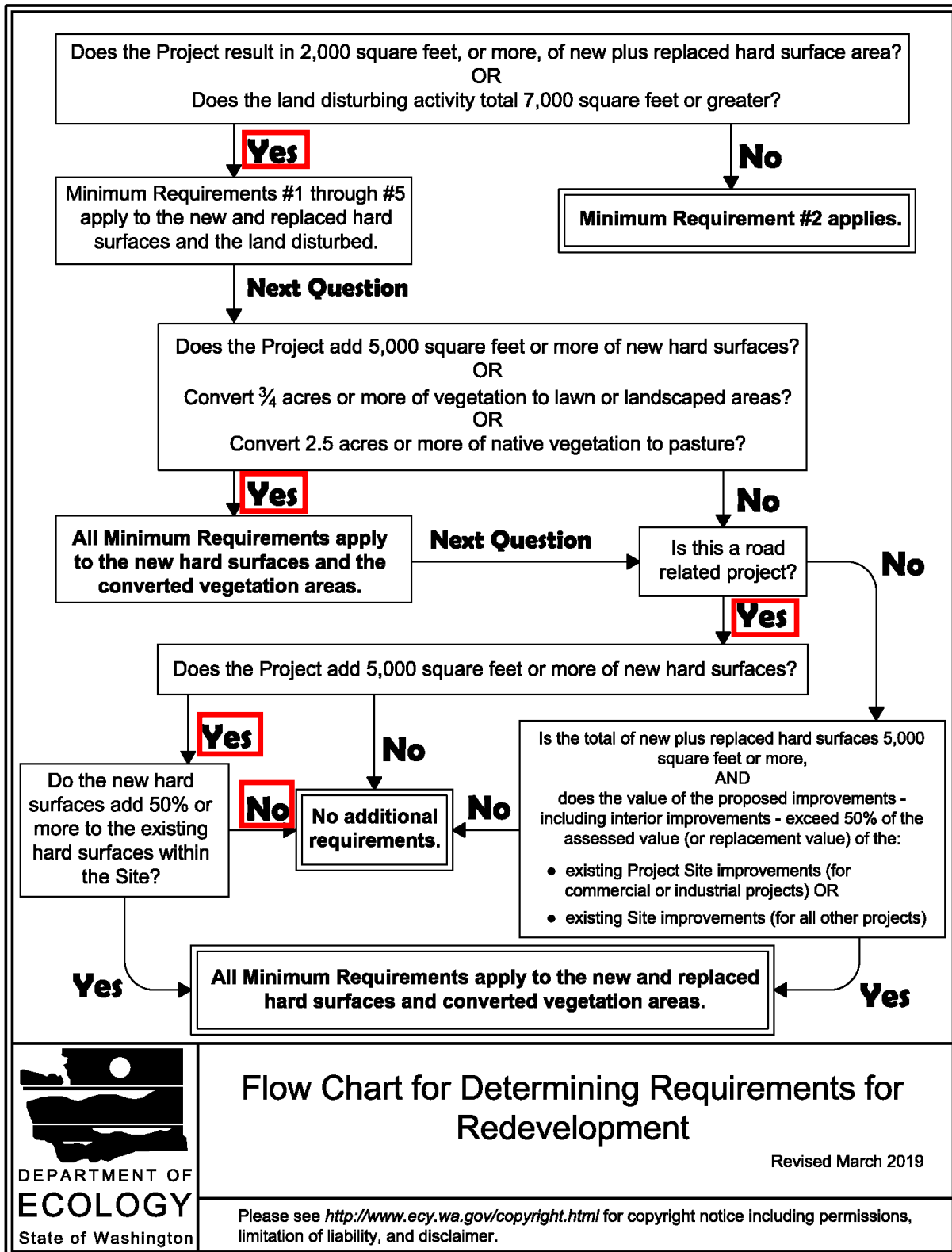
<p>Scale: Horizontal: N.T.S.      Vertical: N/A</p>	<p>For: <b>ARCO ampm Puyallup, Washington</b></p>	<p>Job Number <b>21730</b></p>
 <p><b>Barghausen Consulting Engineers, Inc.</b> 18215 72nd Avenue South Kent, WA 98032 425.251.6222      <b>barghausen.com</b></p>	<p>Title: <b>FEMA MAP</b></p>	<p>DATE: 05/10/22</p>

Figure 1.6  
Minimum  
Requirements  
Flow Chart



**Figure I-3.2: Flow Chart for Determining Requirements for Redevelopment**



**Flow Chart for Determining Requirements for Redevelopment**

Revised March 2019

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# Tab 2.0



## **2.0 CONDITIONS AND REQUIREMENTS SUMMARY**

This section contains the following information:

2.1 Analysis of the Minimum Requirements



## 2.1 Analysis of the Minimum Requirements

MINIMUM REQUIREMENTS	HOW PROJECT HAS ADDRESSED REQUIREMENT
<b>No. 1: Preparation of Stormwater Site Plans</b>	This Minimum Requirement has been fulfilled by the preparation and completion of this Stormwater Site Plan.
<b>No. 2: Construction Stormwater Pollution Prevention (SWPP)</b>	A completed Construction Stormwater Pollution Prevention Plan (SWPPP) shall be prepared under a separate cover and will be submitted together with the Construction Permit submittal. A Department of Ecology Construction Stormwater Permit will be obtained prior to construction.
<b>No. 3: Source Control of Pollution</b>	The proposed redevelopment will provide appropriate source control measures applicable to the project. The proposed design ensures that the fuel station canopy is hydraulically isolated and is routed to an oil water separator with a stop valve prior to discharge into the sanitary sewer system. Additionally, water used for the proposed car wash will be collected and routed to a reclaim system prior to discharge into the sanitary sewer system. Solid waste will be stored in dumpsters that have leak proof lids, and will be stored in an enclosed space within the site.
<b>No. 4: Preservation of Natural Drainage Systems and Outfalls</b>	The proposed redevelopment will preserve the natural drainage system and discharges from the project site shall occur at the natural location, to the maximum extent practicable. The proposed detention facility shall be designed so that stormwater discharge will not cause a significant adverse impact to downstream receiving waters and downgradient properties.
<b>No. 5: On-site Stormwater Management</b>	As required by the 2019 SWMMWW, On-Site Stormwater Management is required where is feasible based on onsite conditions. This project triggers Minimum Requirements Nos. 1 through 9 and is defined as a redevelopment on a parcel inside the UGA; therefore, this project must either apply the Low Impacted Development Performance Standard and BMP T5.13: Post Construction Soil Quality and Depth; or evaluate the feasibility of the BMPs in List No. 2. This project will choose to evaluate the feasibility of BMPs from List No. 2 and apply them to the maximum extent feasible; however, it appears that all on-site stormwater management BMPs for proposed impervious surfaces are infeasible for this site.
<b>No. 6: Runoff Treatment</b>	The proposed redevelopment will trigger enhanced Treatment requirements for all pollution generating surfaces. See the Runoff Treatment Flow Chart, Figure 5.3, in this report for additional detail. Enhanced Treatment will be provided for the pollution generating hard surfaces via a modular wetland unit prior to discharging off-site. See Section 5.6 for further information.

<b>No. 7: Flow Control</b>	This project proposes more than 10,000 square feet of new and replaced hard surface, and must provide flow control. A detention facility has been sized with WHHM2012 to match developed discharge durations to predeveloped durations for the range of predeveloped discharge rates from 50 percent of the 2-year recurrence interval peak flow up to the full 50-year peak flow. The detention facility will route to a catch basin prior to discharging via gravity into the existing on-site stormwater main which ultimately drains to Clarks Creek. See Section 5.5 for further information.
<b>No. 8: Wetlands Protection</b>	There are no documented wetlands recorded on-site.
<b>No. 9: Operation and Maintenance</b>	The drainage facility for this project will be a private facility, owned and maintained by the Owner. An Operation and Maintenance Manual will be provided in Section 9.0 of this Stormwater Site Plan during Final Engineering Review.

# Tab 3.0



### 3.0 EXISTING CONDITIONS SUMMARY

The site consists of four parcels, which together consist of a total area of 1.20 acres. Those four parcels are Parcel Nos. 773000-028-1, 773000-028-8, 773000-003-1, and 773000-002-1. Currently the existing site is primarily developed with a restaurant, asphalt pavement, parking stalls, an empty gravel truck parking area to the north, shrubs, trees, and other landscaping.

See Figure 5.1 for the existing condition map that is included in this report for additional detail.

The overall site topography varies, to the north of the site the slopes are between 0% to 3% and to the south of the site the slopes are between 2% and 14%. The site possibly receives an incidental amount of drainage from adjacent properties. There are several existing storm drainage systems found on-site which collect and convey runoff from the roof and paved areas into the existing public stormwater infrastructure. Existing stormwater runoff appears to be collected by on-site catch basins and routed to the storm conveyance pipe that passes through the northern half of the site.

The project is not located within the 100-year floodplain and no wetlands are known onsite or nearby. The project site is within the Clarks Creek basin.

Per the draft geotechnical report by Krazan & Associates, Inc., the soils encountered on-site generally consisted of poorly graded sand (SP-SM) with silt at a depth of 0.0 to 2.0 feet below ground surface (bgs), bluish grey sand (SP-SM) with silt that was very loose at a depth of 2.0 to 2.5 feet bgs, organic silt (OH, Shalcar Muck) that was very soft at a depth of 2.5 to 4.0 feet bgs, and very loose to medium dense sand with varying silt content at a depth of 4.0 to 46.3 feet bgs. Also, shallow groundwater was encountered in the test pits.

# Tab 4.0



#### **4.0 OFF-SITE ANALYSIS REPORT**

The immediate upstream basin of the site consists of properties to the south and west, and S Meridian Street to the east. Runoff from these upstream properties appear to collect into existing catch basins and conveyance systems found within each individual property. Runoff from S Meridian Street appears to collect into existing conveyance systems within S Meridian Street. It is not anticipated that runoff from the proposed development will contribute a negative impact on upstream properties.

The immediate downstream basin of the site appears to be confined to S Meridian Street and an existing ditch found just north of the site. Runoff from the existing ditch and S Meridian Street shall be collected into catch basins and conveyed northwest. It appears that stormwater within this conveyance system ultimately discharges to Clarks Creek. This project intends to detain stormwater runoff to the maximum extent feasible to meet flow control standards specified in MR No. 7. Additionally, this project intends to provide enhanced stormwater quality treatment, and thus it is not anticipated to create a negative impact on the downstream basin or receiving freshwater bodies.

# Tab 5.0



## **5.0 PERMANENT STORMWATER CONTROL PLAN**

This section contains the following information:

- 5.1 Existing Site Hydrology
- 5.2 Developed Site Hydrology
- 5.3 Performance Standards and Goals
- 5.4 Low Impact Development Features
- 5.5 Flow Control System
- 5.6 Water Quality System
- 5.7 Conveyance System Analysis and Design



## **5.1 Existing Site Hydrology**

### **Predeveloped Basins**

See Figure 5.1 for the existing condition map and tabular table with the predeveloped basin tributary areas and calculations.

The existing 1.20 AC site consists of soils that are classified as Shalcar muck according to the National Cooperative Soil Survey by the USDA Natural Resources Conservation Service. The overall site topography varies, to the north of the site the slopes are between 0% to 3% and to the south of the site the slopes are between 2% and 14%.

There are several existing storm drainage systems found on-site which collect and convey runoff from the roof and paved areas into the existing public stormwater infrastructure. Runoff from the north gravel area sheet flows north into an off-site ditch found within the right-of-way.

## 5.2 Developed Site Hydrology

### Developed Basins

See Figure 5.2 for the developed basin map which contains a tabular table with the developed basin tributary areas and calculations.

The project proposes to add a new ARCO ampm convenience store, a fuel canopy with six multi-dispensers, two USTs, required fuel systems components, an automatic car wash, ADA parking, pavement, drainage and associated utilities

In the developed condition, the site will contain a new ARCO ampm convenience store, a fuel canopy with six multi-dispensers, two USTs, required fuel systems components, an automatic car wash, ADA parking, pavement, detention facility, drainage and associated utilities. Catch basins and an underground pipe conveyance system will be installed to convey stormwater runoff from the developed site to the proposed detention facility located on-site.

This project proposes the use of a concrete detention facility to provide flow control for the site. Per the 2019 DOE SWMMWW, project proponent is allowed to provide Stormwater Management BMPs for an equivalent (flow and pollution characteristics) area. The proposed project proposes to collect existing off-site non-replaced impervious hard surface with an equivalent area to the replaced/new impervious hard surface bypass that will not be collected. The collected existing off-site non-replaced impervious hard surface will have similar flow and pollution characteristics to the replaced/new impervious hard surface bypass area. A detention facility will collect runoff and shall detain and discharge runoff into the public stormwater infrastructure at rates that meet Flow Control standards. The site shall be designed and graded so that all runoff is collected and conveyed by catch basins, pipes, and other associated storm utilities. Oil control and enhanced water quality treatment will be provided for stormwater runoff from all collected at-grade hard surfaces.

A modular wetland unit will be sized using the water quality flow rates from WWHM and proposed upstream of the detention facility to provide treatment to pollution generating surfaces. The detention facility shall be sized in WWHM to ensure that all trigger Flow Control standards are met. Please see Figure 5.5 and 5.6 for the detention facility sizing and the modular wetland unit sizing, respectively.

The proposed drainage design shall meet the required flow control and water quality per the 2019 SWMMWW.

### 5.3 Performance Standards and Goals

The proposed project is required to comply with the runoff treatment and flow control standards set forth by the 2019 SWMMWW.

Since the project proposes to add over 5,000 square feet of new plus replaced impervious surface, the project requires water quality treatment. The project is required to provide enhanced treatment and oil control. To provide water quality, the design flow rate shall be the flow rate at or below which 91% of the total runoff volume, as estimated by an approved continuous runoff model (WWHM) will be treated. In addition, the site will achieve the goals of no ongoing or recurring visible sheen, and to have a 24-hour average Total Petroleum Hydrocarbon (TPH) concentration no greater than 10 mg/l, and a maximum of 15 mg/l for a discrete sample. See the Runoff Treatment Flowchart included in this report for additional information.

The detention facility system shall be designed so that stormwater discharges shall match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow as required by the 2019 SWMMWW. Please see Figure 5.5, the WWHM Modeling report, the pre-developed conditions shall match the forested land cover in the continuous simulation model. The pre-developed conditions show existing off-site non-replaced impervious hard surfaces modeled as forested/flat because they will be used for the purpose of providing Stormwater Management BMPs for an equivalent replaced/new impervious hard surface area that will not be collected. Therefore, with the existing off-site non-replaced impervious hard surface modeled as an equivalent area to the replaced/new impervious hard surface area, there shall be no impervious hard surface area bypassing the detention facility.

## 5.4 Low Impact Development Features

Please see Figure 5.4, this project triggers Minimum Requirements Nos. 1 through 9 and must either use on-site stormwater management BMPs from List No. 2 or demonstrate compliance with the LID Performance Standard and BMP T5.13. This project will choose to evaluate the feasibility of on-site stormwater management BMPs from List No. 2.

### Lawn and Landscaped Areas

1. Soil preservation and Amendment BMP in Volume III, Section 3.1.

**Feasible:** Post Construction Soil Quality and Depth in accordance with BMP T5.13 in Chapter 5 Volume V of the SWMMWW will be applied to all proposed landscaping areas.

### Roofs:

1. Full Dispersion in accordance with BMP T5.30 in Chapter 5 of Volume V of the SWMMWW, or Downspout Full Infiltration Systems in accordance with BMP T5.10A in Section 3.1.1 of Volume III of the SWMMWW.

**Infeasible:** This project will not preserve 65 percent of the site area as forest or native vegetation. Additionally, infiltration is infeasible for this project because of the presence of undocumented fill material, organic silt/peat (Shalcar muck) and clay, and shallow groundwater.

2. Bioretention (See Chapter 7 of Volume V of the SWMMWW) facilities that have a minimum horizontally projected surface area below the overflow, which is at least 5 percent of the total surface area draining to it.

**Infeasible:** Bioretention is infeasible due to the infeasibility of on-site infiltration because of the presence of undocumented fill material, organic silt/peat (Shalcar muck) and clay, and shallow groundwater.

3. Downspout Dispersion Systems in accordance with BMP T5.10B in Section 3.1.2, Volume III, of the SWMMWW.

**Infeasible:** Downspout dispersion systems are infeasible due to the lack of available vegetated area and flow path space.

4. Perforated Stub-out Connections in accordance with BMP T5.10C in Section 3.1.3, Volume III, of the SWMMWW.

**Infeasible:** Perforated Stub-out Connections are infeasible. All rooftop runoff is proposed to be collected and discharge to a stormwater detention facility designed to meet Minimum Requirement No. 7 of Flow Control Requirements.

### Other Hard Surfaces:

1. Full Dispersion in accordance with BMP T5.30 in Chapter, Volume V, of the SWMMWW.

**Infeasible:** This project will not preserve 65 percent of the site area as forest or native vegetation.

2. Permeable Pavement No. 2 is in accordance with BMP T5.15 in Chapter 5, Volume V, of the SWMMWW.

**Infeasible:** This site is defined as high use, and therefore, does not require the evaluation of permeable pavement. Additionally, this site is not able to use infiltration BMPs because of the presence of undocumented fill material, organic silt/peat (Shalcar muck) and clay, and shallow groundwater.

3. Bioretention (See Chapter 7, Volume V of the SWMMWW) facilities that have a minimum horizontally projected surface area below the overflow which is at least 5 percent of the total surface area draining to it.

**Infeasible:** Bioretention is infeasible due to the infeasibility of on-site infiltration because of the presence of undocumented fill material, organic silt/peat (Shalcar muck) and clay, and shallow groundwater.

4. Sheet Flow Dispersion in accordance with BMP T5.12, or Concentrated Flow Dispersion in accordance with BMP T5.11 in Chapter 5, Volume V, of the SWMMWW.

**Infeasible:** The site lacks the available vegetated flow path space for sheet flow dispersion per BMP T5.12, or concentrated flow dispersion per BMP T5.11.

## **5.5 Flow Control System**

This project proposes greater than 10,000 square feet of new and replaced impervious surfaces; therefore, must provide flow control such that post development stormwater discharge rates will match durations for the range of pre-developed discharge rates from 50 percent of the 2-year recurrence interval peak flow up to the full 50-year peak flow. The pre-developed condition to be matched has been modeled as forested land cover.

This project proposes the use of a concrete detention facility to provide flow control for the site. Per the 2019 DOE SWMMWW, project proponent is allowed to provide Stormwater Management BMPs for an equivalent (flow and pollution characteristics) area. The proposed project proposes to collect existing off-site non-replaced impervious hard surface with an equivalent area to the replaced/new impervious hard surface bypass that will not be collected. The collected existing off-site non-replaced impervious hard surface will have similar flow and pollution characteristics to the replaced/new impervious hard surface bypass area. Please see Figure 5.2 and 5.5 for the WWHM Basin Map and the WWHM Model report, respectively. The proposed concrete detention facility shall be routed to a catch basin prior to discharging via gravity into Clarks Creek.

## 5.6 Water Quality System

The project proposes greater than 5,000 square feet of new and replaced PGHS, and is a commercial development, therefore, Enhanced water quality treatment will be required. Additionally, the site is defined as "high-use" and therefore, must provide oil control treatment. The project will propose an off-line coalescing plate oil/water separator to provide oil control treatment, followed by a Bioclean Modular Wetland Unit to provide Enhanced Treatment as allowed under the DOE TAPE program. A flow splitter, in accordance with Section V-1.4.1 of Volume 1 of the 2019 DOE SWMMWW, is proposed upstream of the proposed modular wetland unit and will be designed to ensure that low flows are delivered to the modular wetland unit up to the Water Quality design flow rate and any additional flows above this rate will be diverted to the bypass system with minimal increase in head at the flow splitter structure to avoid surcharging the Runoff Treatment BMP under high flow conditions. Please see Figure 5.6 for the WWHM calculations and sizing of the proposed modular wetland unit which meets Minimum Requirement No. 6.

BMP S409 will also be implemented for the fueling pad. The fuel pad shall be graded to be hydraulically isolated from the surrounding areas and will discharge incidental stormwater from under the canopy to an oil/water separator prior to discharge into the sanitary sewer system.

## **5.7 Conveyance System Analysis and Design**

Conveyance calculations shall be included within this report on the final submittal.



Figure 5.1  
Existing  
Condition Map





# Figure 5.2 Developed Basin Map









Figure 5.3  
Drainage Facility –  
Runoff Treatment  
Facility Selection Flow  
Chart

**Figure III-1.1: Runoff Treatment BMP Selection Flow Chart**

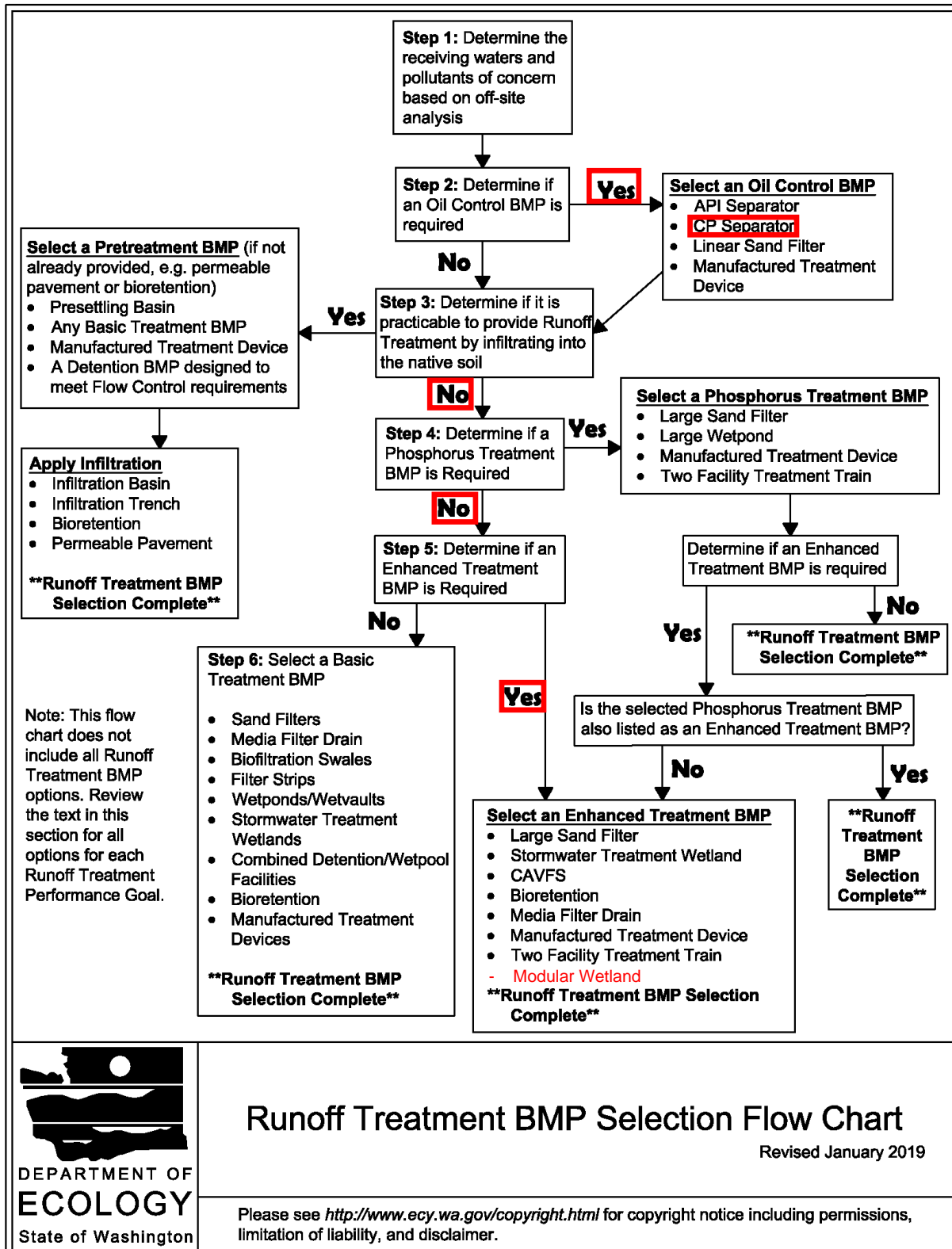
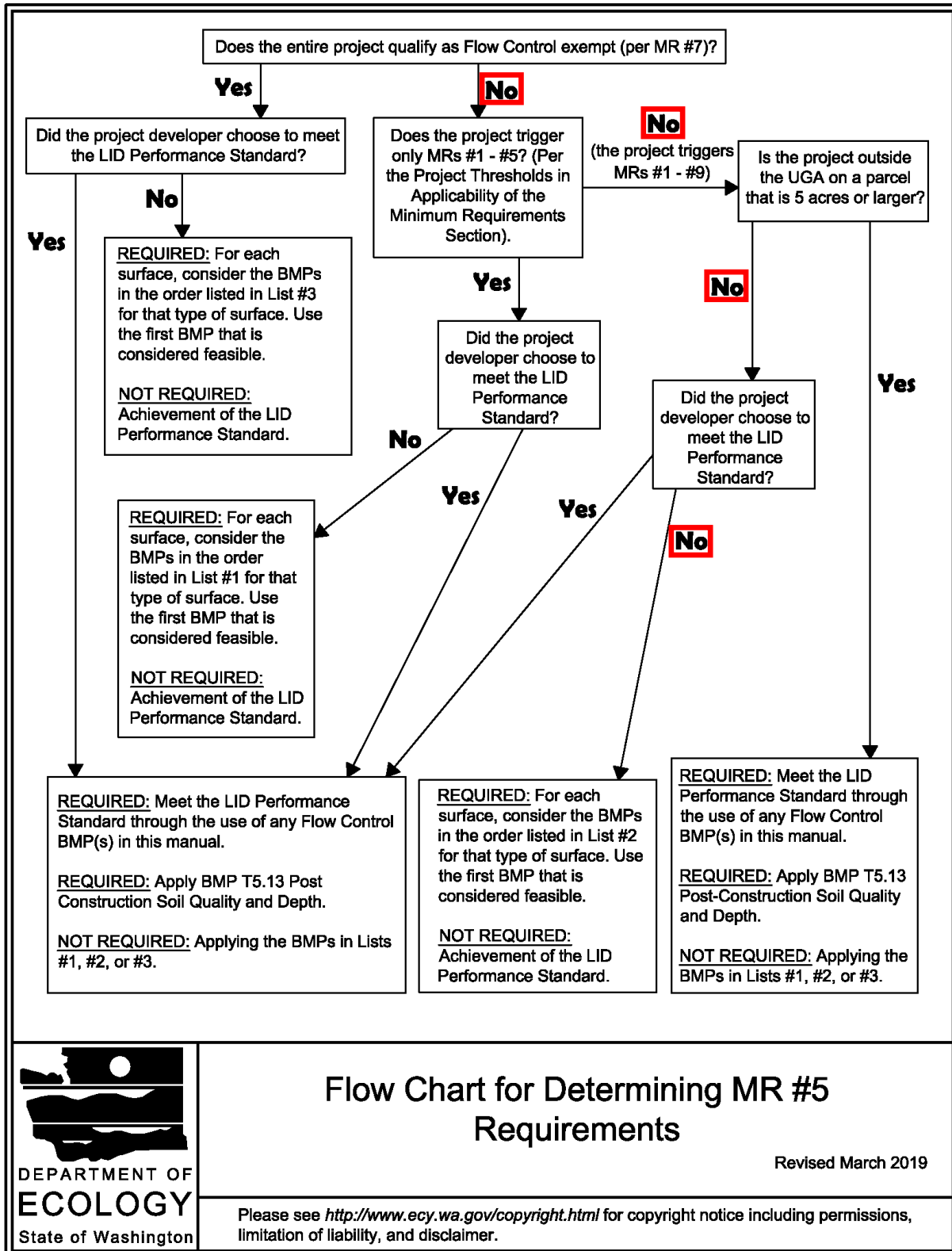


Figure 5.4  
Flow Chart for  
Determining MR No.  
Requirements





**Figure I-3.3: Flow Chart for Determining MR #5 Requirements**



## Flow Chart for Determining MR #5 Requirements

Revised March 2019

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Figure 5.5  
WWHM Calculations  
and Detention Facility  
Sizing

**WWHM2012**  
**PROJECT REPORT**

**WWHM MODELING  
REPORT FOR  
DETENTION FACILITY**

## *General Model Information*

Project Name: 2023-3-7-WWHM-6' depth  
Site Name: ARCO  
Site Address:  
City: PUYALLUP  
Report Date: 3/9/2023  
Gage:  
Data Start: 10/01/1901  
Data End: 09/30/2059  
Timestep: 15 Minute  
Precip Scale: 1.000  
Version Date: 2018/03/02  
Version: 4.2.14

## *POC Thresholds*

---

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

---

## Landuse Basin Data

### Predeveloped Land Use

#### Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use C, Forest, Flat	acre 0.887
Pervious Total	0.887
Impervious Land Use	acre
Impervious Total	0
Basin Total	0.887

Element Flows To:		
Surface	Interflow	Groundwater

## Basin 2

Bypass:	No
GroundWater:	No
Pervious Land Use C, Forest, Flat	acre 0.052
Pervious Total	0.052
Impervious Land Use	acre
Impervious Total	0
Basin Total	0.052

Element Flows To:		
Surface	Interflow	Groundwater

### Basin 3

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROADS FLAT	0.01
Impervious Total	0.01
Basin Total	0.01

Element Flows To:		
Surface	Interflow	Groundwater

## Mitigated Land Use

### Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROADS FLAT	0.887
Impervious Total	0.887
Basin Total	0.887

Element Flows To:		
Surface	Interflow	Groundwater
Vault 1	Vault 1	



## Basin 2

Bypass:	Yes
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROADS FLAT	0.053
Impervious Total	0.053
Basin Total	0.053

Element Flows To:		
Surface	Interflow	Groundwater
Vault 1	Vault 1	

*Routing Elements*  
*Predeveloped Routing*

## Mitigated Routing

### Vault 1

Width: 69 ft.  
 Length: 69 ft.  
 Depth: 6 ft.  
 Discharge Structure  
 Riser Height: 5 ft.  
 Riser Diameter: 12 in.  
 Notch Type: Rectangular  
 Notch Width: 0.011 ft.  
 Notch Height: 1.391 ft.  
 Orifice 1 Diameter: 0.485 in. Elevation:0 ft.  
 Element Flows To:  
 Outlet 1                      Outlet 2

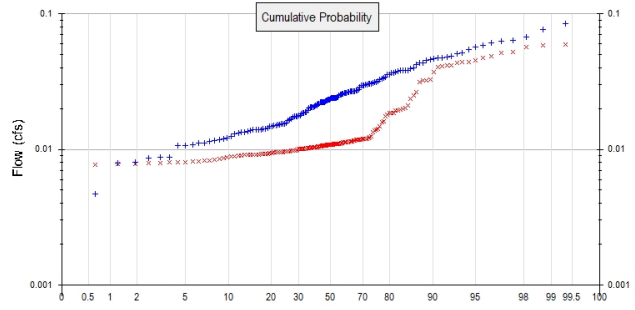
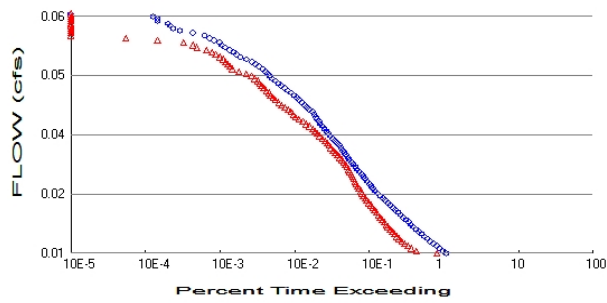
Vault Hydraulic Table

Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
0.0000	0.109	0.000	0.000	0.000
0.0667	0.109	0.007	0.001	0.000
0.1333	0.109	0.014	0.002	0.000
0.2000	0.109	0.021	0.002	0.000
0.2667	0.109	0.029	0.003	0.000
0.3333	0.109	0.036	0.003	0.000
0.4000	0.109	0.043	0.004	0.000
0.4667	0.109	0.051	0.004	0.000
0.5333	0.109	0.058	0.004	0.000
0.6000	0.109	0.065	0.004	0.000
0.6667	0.109	0.072	0.005	0.000
0.7333	0.109	0.080	0.005	0.000
0.8000	0.109	0.087	0.005	0.000
0.8667	0.109	0.094	0.005	0.000
0.9333	0.109	0.102	0.006	0.000
1.0000	0.109	0.109	0.006	0.000
1.0667	0.109	0.116	0.006	0.000
1.1333	0.109	0.123	0.006	0.000
1.2000	0.109	0.131	0.007	0.000
1.2667	0.109	0.138	0.007	0.000
1.3333	0.109	0.145	0.007	0.000
1.4000	0.109	0.153	0.007	0.000
1.4667	0.109	0.160	0.007	0.000
1.5333	0.109	0.167	0.007	0.000
1.6000	0.109	0.174	0.008	0.000
1.6667	0.109	0.182	0.008	0.000
1.7333	0.109	0.189	0.008	0.000
1.8000	0.109	0.196	0.008	0.000
1.8667	0.109	0.204	0.008	0.000
1.9333	0.109	0.211	0.008	0.000
2.0000	0.109	0.218	0.009	0.000
2.0667	0.109	0.225	0.009	0.000
2.1333	0.109	0.233	0.009	0.000
2.2000	0.109	0.240	0.009	0.000
2.2667	0.109	0.247	0.009	0.000
2.3333	0.109	0.255	0.009	0.000
2.4000	0.109	0.262	0.009	0.000

2.4667	0.109	0.269	0.010	0.000
2.5333	0.109	0.276	0.010	0.000
2.6000	0.109	0.284	0.010	0.000
2.6667	0.109	0.291	0.010	0.000
2.7333	0.109	0.298	0.010	0.000
2.8000	0.109	0.306	0.010	0.000
2.8667	0.109	0.313	0.010	0.000
2.9333	0.109	0.320	0.010	0.000
3.0000	0.109	0.327	0.011	0.000
3.0667	0.109	0.335	0.011	0.000
3.1333	0.109	0.342	0.011	0.000
3.2000	0.109	0.349	0.011	0.000
3.2667	0.109	0.357	0.011	0.000
3.3333	0.109	0.364	0.011	0.000
3.4000	0.109	0.371	0.011	0.000
3.4667	0.109	0.378	0.011	0.000
3.5333	0.109	0.386	0.012	0.000
3.6000	0.109	0.393	0.012	0.000
3.6667	0.109	0.400	0.012	0.000
3.7333	0.109	0.408	0.013	0.000
3.8000	0.109	0.415	0.015	0.000
3.8667	0.109	0.422	0.016	0.000
3.9333	0.109	0.429	0.018	0.000
4.0000	0.109	0.437	0.020	0.000
4.0667	0.109	0.444	0.022	0.000
4.1333	0.109	0.451	0.025	0.000
4.2000	0.109	0.459	0.027	0.000
4.2667	0.109	0.466	0.029	0.000
4.3333	0.109	0.473	0.031	0.000
4.4000	0.109	0.480	0.034	0.000
4.4667	0.109	0.488	0.036	0.000
4.5333	0.109	0.495	0.039	0.000
4.6000	0.109	0.502	0.041	0.000
4.6667	0.109	0.510	0.044	0.000
4.7333	0.109	0.517	0.047	0.000
4.8000	0.109	0.524	0.050	0.000
4.8667	0.109	0.531	0.054	0.000
4.9333	0.109	0.539	0.057	0.000
5.0000	0.109	0.546	0.060	0.000
5.0667	0.109	0.553	0.243	0.000
5.1333	0.109	0.561	0.570	0.000
5.2000	0.109	0.568	0.968	0.000
5.2667	0.109	0.575	1.379	0.000
5.3333	0.109	0.582	1.744	0.000
5.4000	0.109	0.590	2.021	0.000
5.4667	0.109	0.597	2.199	0.000
5.5333	0.109	0.604	2.361	0.000
5.6000	0.109	0.612	2.501	0.000
5.6667	0.109	0.619	2.633	0.000
5.7333	0.109	0.626	2.758	0.000
5.8000	0.109	0.633	2.878	0.000
5.8667	0.109	0.641	2.994	0.000
5.9333	0.109	0.648	3.104	0.000
6.0000	0.109	0.655	3.211	0.000
6.0667	0.109	0.663	3.315	0.000
6.1333	0.000	0.000	3.415	0.000

# Analysis Results

## POC 1



+ Predeveloped x Mitigated

### Predeveloped Landuse Totals for POC #1

Total Pervious Area: 0.939  
Total Impervious Area: 0.01

### Mitigated Landuse Totals for POC #1

Total Pervious Area: 0  
Total Impervious Area: 0.94

Flow Frequency Method: Log Pearson Type III 17B

### Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.023355
5 year	0.035927
10 year	0.044562
25 year	0.055664
50 year	0.064014
100 year	0.0724

### Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0.012028
5 year	0.019392
10 year	0.02623
25 year	0.037746
50 year	0.048895
100 year	0.062721

## Annual Peaks

### Annual Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1902	0.023	0.011
1903	0.015	0.009
1904	0.031	0.010
1905	0.014	0.012
1906	0.008	0.008
1907	0.037	0.010
1908	0.026	0.009
1909	0.025	0.011
1910	0.036	0.011
1911	0.023	0.010

1912	0.084	0.012
1913	0.036	0.020
1914	0.012	0.008
1915	0.017	0.014
1916	0.023	0.010
1917	0.009	0.010
1918	0.024	0.032
1919	0.021	0.010
1920	0.024	0.010
1921	0.027	0.012
1922	0.026	0.011
1923	0.021	0.012
1924	0.011	0.010
1925	0.014	0.009
1926	0.024	0.010
1927	0.019	0.010
1928	0.018	0.011
1929	0.037	0.014
1930	0.023	0.010
1931	0.022	0.011
1932	0.018	0.012
1933	0.021	0.011
1934	0.048	0.052
1935	0.022	0.025
1936	0.021	0.012
1937	0.034	0.010
1938	0.020	0.011
1939	0.003	0.008
1940	0.022	0.012
1941	0.015	0.008
1942	0.033	0.044
1943	0.016	0.011
1944	0.039	0.019
1945	0.027	0.011
1946	0.017	0.009
1947	0.012	0.010
1948	0.049	0.011
1949	0.043	0.023
1950	0.014	0.010
1951	0.019	0.010
1952	0.064	0.021
1953	0.058	0.045
1954	0.022	0.011
1955	0.019	0.009
1956	0.011	0.009
1957	0.031	0.015
1958	0.061	0.057
1959	0.038	0.049
1960	0.011	0.008
1961	0.038	0.043
1962	0.022	0.011
1963	0.011	0.008
1964	0.013	0.009
1965	0.044	0.044
1966	0.014	0.010
1967	0.022	0.009
1968	0.022	0.012
1969	0.020	0.011

1970	0.031	0.011
1971	0.045	0.031
1972	0.030	0.011
1973	0.040	0.020
1974	0.024	0.010
1975	0.048	0.059
1976	0.026	0.011
1977	0.013	0.008
1978	0.042	0.041
1979	0.013	0.010
1980	0.025	0.011
1981	0.024	0.011
1982	0.014	0.008
1983	0.038	0.013
1984	0.019	0.010
1985	0.029	0.010
1986	0.024	0.012
1987	0.047	0.032
1988	0.028	0.019
1989	0.026	0.010
1990	0.030	0.010
1991	0.024	0.011
1992	0.030	0.033
1993	0.031	0.011
1994	0.046	0.011
1995	0.012	0.010
1996	0.051	0.042
1997	0.023	0.009
1998	0.026	0.010
1999	0.005	0.009
2000	0.019	0.012
2001	0.012	0.008
2002	0.038	0.011
2003	0.029	0.011
2004	0.027	0.011
2005	0.052	0.011
2006	0.015	0.010
2007	0.017	0.011
2008	0.025	0.010
2009	0.018	0.010
2010	0.015	0.013
2011	0.014	0.010
2012	0.024	0.010
2013	0.015	0.008
2014	0.011	0.008
2015	0.021	0.010
2016	0.009	0.010
2017	0.033	0.018
2018	0.063	0.052
2019	0.068	0.048
2020	0.020	0.009
2021	0.032	0.026
2022	0.014	0.009
2023	0.027	0.011
2024	0.077	0.010
2025	0.024	0.011
2026	0.038	0.018
2027	0.015	0.009

2028	0.013	0.008
2029	0.026	0.020
2030	0.047	0.012
2031	0.015	0.009
2032	0.011	0.008
2033	0.014	0.009
2034	0.015	0.010
2035	0.055	0.059
2036	0.030	0.012
2037	0.009	0.009
2038	0.027	0.018
2039	0.008	0.007
2040	0.015	0.010
2041	0.017	0.009
2042	0.057	0.037
2043	0.027	0.019
2044	0.034	0.016
2045	0.023	0.012
2046	0.027	0.040
2047	0.020	0.012
2048	0.027	0.011
2049	0.024	0.011
2050	0.018	0.010
2051	0.030	0.011
2052	0.016	0.011
2053	0.026	0.041
2054	0.031	0.018
2055	0.013	0.009
2056	0.012	0.009
2057	0.018	0.011
2058	0.022	0.014
2059	0.037	0.016

### Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.0843	0.0593
2	0.0769	0.0587
3	0.0677	0.0569
4	0.0641	0.0523
5	0.0626	0.0517
6	0.0607	0.0485
7	0.0583	0.0475
8	0.0570	0.0451
9	0.0546	0.0444
10	0.0517	0.0439
11	0.0511	0.0434
12	0.0488	0.0418
13	0.0477	0.0414
14	0.0475	0.0411
15	0.0470	0.0405
16	0.0467	0.0373
17	0.0459	0.0326
18	0.0452	0.0323
19	0.0436	0.0321
20	0.0435	0.0315
21	0.0421	0.0264
22	0.0401	0.0251



23	0.0394	0.0234
24	0.0384	0.0212
25	0.0384	0.0201
26	0.0384	0.0200
27	0.0381	0.0197
28	0.0377	0.0193
29	0.0372	0.0193
30	0.0368	0.0185
31	0.0366	0.0185
32	0.0357	0.0185
33	0.0357	0.0181
34	0.0343	0.0176
35	0.0335	0.0162
36	0.0333	0.0158
37	0.0329	0.0147
38	0.0318	0.0142
39	0.0313	0.0140
40	0.0313	0.0138
41	0.0309	0.0134
42	0.0305	0.0125
43	0.0305	0.0123
44	0.0305	0.0121
45	0.0299	0.0120
46	0.0299	0.0120
47	0.0296	0.0119
48	0.0295	0.0119
49	0.0294	0.0119
50	0.0288	0.0118
51	0.0277	0.0117
52	0.0272	0.0117
53	0.0272	0.0117
54	0.0270	0.0116
55	0.0268	0.0116
56	0.0268	0.0115
57	0.0268	0.0115
58	0.0267	0.0114
59	0.0265	0.0114
60	0.0265	0.0114
61	0.0262	0.0113
62	0.0262	0.0113
63	0.0260	0.0113
64	0.0259	0.0112
65	0.0258	0.0112
66	0.0257	0.0112
67	0.0255	0.0112
68	0.0253	0.0110
69	0.0251	0.0110
70	0.0242	0.0110
71	0.0242	0.0110
72	0.0241	0.0110
73	0.0240	0.0109
74	0.0240	0.0109
75	0.0240	0.0109
76	0.0239	0.0109
77	0.0239	0.0109
78	0.0238	0.0109
79	0.0235	0.0108
80	0.0233	0.0108

81	0.0232	0.0108
82	0.0232	0.0108
83	0.0230	0.0108
84	0.0229	0.0107
85	0.0226	0.0107
86	0.0224	0.0107
87	0.0224	0.0107
88	0.0222	0.0106
89	0.0222	0.0106
90	0.0222	0.0105
91	0.0222	0.0105
92	0.0217	0.0105
93	0.0216	0.0105
94	0.0214	0.0104
95	0.0212	0.0104
96	0.0212	0.0104
97	0.0207	0.0104
98	0.0206	0.0104
99	0.0205	0.0104
100	0.0205	0.0103
101	0.0205	0.0103
102	0.0201	0.0103
103	0.0194	0.0102
104	0.0189	0.0102
105	0.0188	0.0101
106	0.0186	0.0101
107	0.0186	0.0101
108	0.0182	0.0101
109	0.0178	0.0101
110	0.0178	0.0101
111	0.0176	0.0099
112	0.0176	0.0099
113	0.0175	0.0098
114	0.0174	0.0098
115	0.0169	0.0098
116	0.0167	0.0097
117	0.0162	0.0097
118	0.0157	0.0097
119	0.0154	0.0096
120	0.0154	0.0096
121	0.0153	0.0096
122	0.0153	0.0095
123	0.0152	0.0095
124	0.0150	0.0095
125	0.0150	0.0095
126	0.0147	0.0094
127	0.0147	0.0093
128	0.0144	0.0093
129	0.0142	0.0093
130	0.0141	0.0092
131	0.0141	0.0092
132	0.0141	0.0092
133	0.0140	0.0092
134	0.0139	0.0092
135	0.0138	0.0091
136	0.0135	0.0091
137	0.0135	0.0091
138	0.0134	0.0090

139	0.0132	0.0090
140	0.0130	0.0089
141	0.0124	0.0088
142	0.0122	0.0087
143	0.0119	0.0086
144	0.0118	0.0085
145	0.0117	0.0084
146	0.0115	0.0083
147	0.0112	0.0082
148	0.0111	0.0082
149	0.0108	0.0081
150	0.0107	0.0081
151	0.0107	0.0081
152	0.0088	0.0080
153	0.0087	0.0080
154	0.0086	0.0079
155	0.0080	0.0078
156	0.0080	0.0078
157	0.0047	0.0077
158	0.0033	0.0072

## Duration Flows

The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0117	59389	44448	74	Pass
0.0122	53567	23739	44	Pass
0.0127	48525	20038	41	Pass
0.0133	43988	18149	41	Pass
0.0138	39966	16443	41	Pass
0.0143	36387	15080	41	Pass
0.0148	33224	13983	42	Pass
0.0154	30326	12919	42	Pass
0.0159	27717	12044	43	Pass
0.0164	25457	11257	44	Pass
0.0170	23468	10565	45	Pass
0.0175	21662	9900	45	Pass
0.0180	20016	9280	46	Pass
0.0185	18520	8742	47	Pass
0.0191	17097	8238	48	Pass
0.0196	15756	7767	49	Pass
0.0201	14509	7313	50	Pass
0.0207	13446	6897	51	Pass
0.0212	12421	6521	52	Pass
0.0217	11485	6149	53	Pass
0.0223	10615	5789	54	Pass
0.0228	9817	5449	55	Pass
0.0233	9064	5136	56	Pass
0.0238	8371	4860	58	Pass
0.0244	7712	4625	59	Pass
0.0249	7152	4437	62	Pass
0.0254	6670	4235	63	Pass
0.0260	6238	4036	64	Pass
0.0265	5878	3830	65	Pass
0.0270	5528	3675	66	Pass
0.0275	5215	3544	67	Pass
0.0281	4900	3419	69	Pass
0.0286	4600	3308	71	Pass
0.0291	4348	3179	73	Pass
0.0297	4089	3047	74	Pass
0.0302	3846	2897	75	Pass
0.0307	3615	2784	77	Pass
0.0312	3390	2662	78	Pass
0.0318	3220	2506	77	Pass
0.0323	3014	2364	78	Pass
0.0328	2866	2231	77	Pass
0.0334	2694	2075	77	Pass
0.0339	2568	1951	75	Pass
0.0344	2439	1817	74	Pass
0.0349	2319	1699	73	Pass
0.0355	2196	1600	72	Pass
0.0360	2068	1508	72	Pass
0.0365	1926	1415	73	Pass
0.0371	1782	1305	73	Pass
0.0376	1666	1196	71	Pass
0.0381	1561	1104	70	Pass
0.0386	1470	1034	70	Pass
0.0392	1381	962	69	Pass

0.0397	1300	877	67	Pass
0.0402	1226	783	63	Pass
0.0408	1169	712	60	Pass
0.0413	1108	627	56	Pass
0.0418	1054	569	53	Pass
0.0423	1011	530	52	Pass
0.0429	947	493	52	Pass
0.0434	867	450	51	Pass
0.0439	806	406	50	Pass
0.0445	748	368	49	Pass
0.0450	689	339	49	Pass
0.0455	636	304	47	Pass
0.0460	595	284	47	Pass
0.0466	541	262	48	Pass
0.0471	497	249	50	Pass
0.0476	448	230	51	Pass
0.0482	404	214	52	Pass
0.0487	367	199	54	Pass
0.0492	334	188	56	Pass
0.0497	304	177	58	Pass
0.0503	278	162	58	Pass
0.0508	254	146	57	Pass
0.0513	240	124	51	Pass
0.0519	224	99	44	Pass
0.0524	206	81	39	Pass
0.0529	184	75	40	Pass
0.0534	165	70	42	Pass
0.0540	147	65	44	Pass
0.0545	125	61	48	Pass
0.0550	103	55	53	Pass
0.0556	91	48	52	Pass
0.0561	84	43	51	Pass
0.0566	75	36	48	Pass
0.0571	65	27	41	Pass
0.0577	58	24	41	Pass
0.0582	54	18	33	Pass
0.0587	46	8	17	Pass
0.0593	40	3	7	Pass
0.0598	34	0	0	Pass
0.0603	24	0	0	Pass
0.0608	16	0	0	Pass
0.0614	13	0	0	Pass
0.0619	12	0	0	Pass
0.0624	11	0	0	Pass
0.0630	8	0	0	Pass
0.0635	8	0	0	Pass
0.0640	7	0	0	Pass

## Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume: 0 acre-feet

On-line facility target flow: 0 cfs.

Adjusted for 15 min: 0 cfs.

Off-line facility target flow: 0 cfs.

Adjusted for 15 min: 0 cfs.

# LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Vault 1 POC	<input type="checkbox"/>	387.40			<input type="checkbox"/>	0.00			
Total Volume Infiltrated		387.40	0.00	0.00		0.00	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Failed

## *Model Default Modifications*

Total of 0 changes have been made.

### *PERLND Changes*

No PERLND changes have been made.

### *IMPLND Changes*

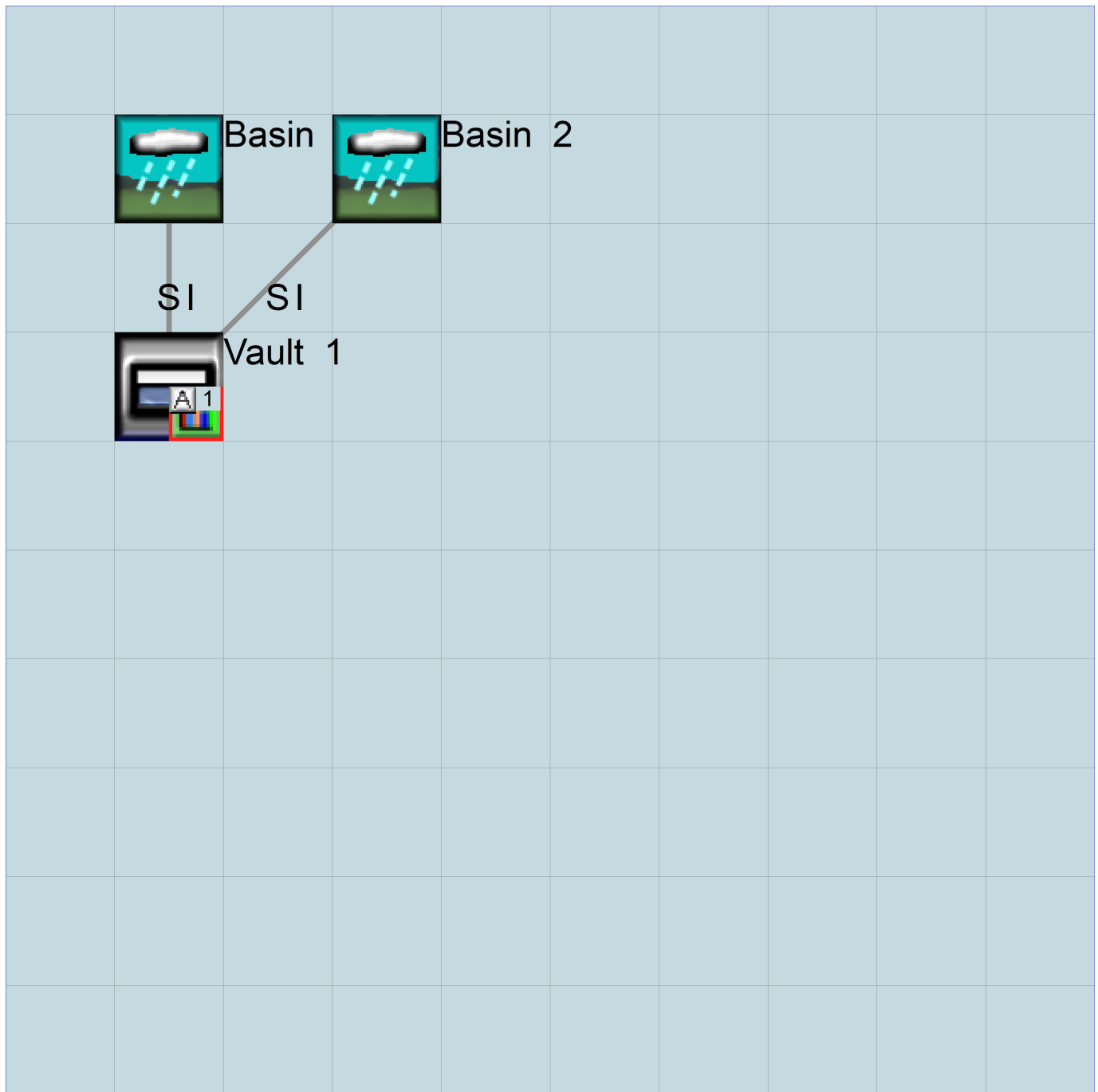
No IMPLND changes have been made.



Appendix  
Predeveloped Schematic



Mitigated Schematic



# Predeveloped UCI File

RUN

GLOBAL

```
WVHM4 model simulation
START      1901 10 01      END      2059 09 30
RUN INTERP OUTPUT LEVEL   3      0
RESUME     0 RUN         1
UNIT SYSTEM 1
```

FILES

```
<File> <Un#> <-----File Name----->***
<-ID->                                     ***
WDM      26      2023-3-7-WVHM-6' depth.wdm
MESSU    25      Pre2023-3-7-WVHM-6' depth.MES
          27      Pre2023-3-7-WVHM-6' depth.L61
          28      Pre2023-3-7-WVHM-6' depth.L62
          30      POC2023-3-7-WVHM-6' depth1.dat
```

END FILES

OPN SEQUENCE

```
INGRP          INDELT 00:15
  PERLND        10
  IMPLND         1
  COPY          501
  DISPLY         1
```

END INGRP

END OPN SEQUENCE

DISPLY

DISPLY-INFO1

```
# - #<-----Title----->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
1   Basin 1          MAX          1   2   30   9
```

END DISPLY-INFO1

END DISPLY

COPY

TIMESERIES

```
# - # NPT NMN ***
1   1   1
501 1   1
```

END TIMESERIES

END COPY

GENER

OPCODE

```
# # OPCD ***
```

END OPCODE

PARM

```
# # K ***
```

END PARM

END GENER

PERLND

GEN-INFO

```
<PLS ><-----Name----->NBLKS Unit-systems Printer ***
# - # User t-series Engl Metr ***
          in out ***
```

```
10 C, Forest, Flat 1 1 1 1 27 0
```

END GEN-INFO

\*\*\* Section PWATER\*\*\*

ACTIVITY

```
<PLS > ***** Active Sections *****
```

```
# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC ***
10 0 0 1 0 0 0 0 0 0 0 0 0 0
```

END ACTIVITY

PRINT-INFO

```
<PLS > ***** Print-flags ***** PIVL PYR
```

```
# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC *****
10 0 0 4 0 0 0 0 0 0 0 0 0 0 1 9
```

END PRINT-INFO

```

PWAT-PARM1
  <PLS > PWATER variable monthly parameter value flags ***
  # - # CSNO RTOP UZFG VCS VUZ VNN VIFW VIRC VLE INFC HWT ***
  10 0 0 0 0 0 0 0 0 0 0 0
END PWAT-PARM1

```

```

PWAT-PARM2
  <PLS > PWATER input info: Part 2 ***
  # - # ***FOREST LZSN INFILT LSUR SLSUR KVARY AGWRC
  10 0 4.5 0.08 400 0.05 0.5 0.996
END PWAT-PARM2

```

```

PWAT-PARM3
  <PLS > PWATER input info: Part 3 ***
  # - # ***PETMAX PETMIN INFEXP INFILD DEEPFR BASETP AGWETP
  10 0 0 2 2 0 0 0
END PWAT-PARM3

```

```

PWAT-PARM4
  <PLS > PWATER input info: Part 4 ***
  # - # CEPSC UZSN NSUR INTFW IRC LZETP ***
  10 0.2 0.5 0.35 6 0.5 0.7
END PWAT-PARM4

```

```

PWAT-STATE1
  <PLS > *** Initial conditions at start of simulation
  ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
  # - # *** CEPS SURS UZS IFWS LZS AGWS GWVS
  10 0 0 0 0 2.5 1 0
END PWAT-STATE1

```

END PERLND

IMPLND

```

GEN-INFO
  <PLS ><-----Name-----> Unit-systems Printer ***
  # - # User t-series Engl Metr ***
  in out ***
  1 ROADS/FLAT 1 1 1 27 0
END GEN-INFO
*** Section IWATER***

```

```

ACTIVITY
  <PLS > ***** Active Sections *****
  # - # ATMP SNOW IWAT SLD IWG IQAL ***
  1 0 0 1 0 0 0
END ACTIVITY

```

```

PRINT-INFO
  <ILS > ***** Print-flags ***** PIVL PYR
  # - # ATMP SNOW IWAT SLD IWG IQAL *****
  1 0 0 4 0 0 0 1 9
END PRINT-INFO

```

```

IWAT-PARM1
  <PLS > IWATER variable monthly parameter value flags ***
  # - # CSNO RTOP VRS VNN RTLI ***
  1 0 0 0 0 0
END IWAT-PARM1

```

```

IWAT-PARM2
  <PLS > IWATER input info: Part 2 ***
  # - # *** LSUR SLSUR NSUR RETSC
  1 400 0.01 0.1 0.1
END IWAT-PARM2

```

```

IWAT-PARM3
  <PLS > IWATER input info: Part 3 ***
  # - # ***PETMAX PETMIN
  1 0 0

```

END IWAT-PARM3

IWAT-STATE1

<PLS > \*\*\* Initial conditions at start of simulation

# - # \*\*\* RETS SURS

1 0 0

END IWAT-STATE1

END IMPLND

SCHEMATIC

<-Source->	<-Area-->	<-Target->	MBLK	***
<Name> #	<-factor->	<Name> #	Tbl#	***
Basin 1***				
PERLND 10	0.887	COPY 501	12	
PERLND 10	0.887	COPY 501	13	
Basin 2***				
PERLND 10	0.052	COPY 501	12	
PERLND 10	0.052	COPY 501	13	
Basin 3***				
IMPLND 1	0.01	COPY 501	15	

\*\*\*\*\*Routing\*\*\*\*\*

END SCHEMATIC

NETWORK

<-Volume->	<-Grp>	<-Member->	<-Mult-->	Tran	<-Target vols>	<-Grp>	<-Member->	***
<Name> #		<Name> #	#	<-factor->strg	<Name> #	#	<Name> #	***
COPY 501	OUTPUT	MEAN	1	1	48.4		DISPLY 1	INPUT TIMSER 1

<-Volume->	<-Grp>	<-Member->	<-Mult-->	Tran	<-Target vols>	<-Grp>	<-Member->	***
<Name> #		<Name> #	#	<-factor->strg	<Name> #	#	<Name> #	***

END NETWORK

RCHRES

GEN-INFO

RCHRES	Name	Nexits	Unit Systems	Printer	***
# - #	<----->	<---->	User T-series	Engl Metr LKFG	***
			in out		***

END GEN-INFO

\*\*\* Section RCHRES\*\*\*

ACTIVITY

<PLS > \*\*\*\*\* Active Sections \*\*\*\*\*

# - # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFQ PKFG PHFG \*\*\*

END ACTIVITY

PRINT-INFO

<PLS > \*\*\*\*\* Print-flags \*\*\*\*\* PIVL PYR

# - # HYDR ADCA CONS HEAT SED GQL OXRX NUTR PLNK PHCB PIVL PYR \*\*\*\*\*

END PRINT-INFO

HYDR-PARM1

RCHRES Flags for each HYDR Section \*\*\*

# - #	VC A1 A2 A3	ODFVFG for each	***	ODGTFG for each	FUNCT for each
	FG FG FG FG	possible exit	***	possible exit	possible exit
	* * * *	* * * *		* * * *	***

END HYDR-PARM1

HYDR-PARM2

# - #	FTABNO	LEN	DELTH	STCOR	KS	DB50	***
	<----->	<----->	<----->	<----->	<----->	<----->	***

END HYDR-PARM2

HYDR-INIT

RCHRES Initial conditions for each HYDR section \*\*\*

# - #	*** VOL	Initial value of COLIND	Initial value of OUTDGT
	*** ac-ft	for each possible exit	for each possible exit
	<----->	<----->	<----->

END HYDR-INIT  
END RCHRES

SPEC-ACTIONS  
END SPEC-ACTIONS  
FTABLES  
END FTABLES

EXT SOURCES  
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member-> \*\*\*  
<Name> # <Name> # tem strg<-factor->strg <Name> # # <Name> # # \*\*\*  
WDM 2 PREC ENGL 1 PERLND 1 999 EXTNL PREC  
WDM 2 PREC ENGL 1 IMPLND 1 999 EXTNL PREC  
WDM 1 EVAP ENGL 1 PERLND 1 999 EXTNL PETINP  
WDM 1 EVAP ENGL 1 IMPLND 1 999 EXTNL PETINP

END EXT SOURCES

EXT TARGETS  
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd \*\*\*  
<Name> # <Name> # #<-factor->strg <Name> # <Name> tem strg strg\*\*\*  
COPY 501 OUTPUT MEAN 1 1 48.4 WDM 501 FLOW ENGL REPL  
END EXT TARGETS

MASS-LINK  
<Volume> <-Grp> <-Member-><--Mult--> <Target> <-Grp> <-Member->\*\*\*  
<Name> <Name> # #<-factor-> <Name> <Name> # #\*\*\*  
MASS-LINK 12  
PERLND PWATER SURO 0.083333 COPY INPUT MEAN  
END MASS-LINK 12  
  
MASS-LINK 13  
PERLND PWATER IFWO 0.083333 COPY INPUT MEAN  
END MASS-LINK 13  
  
MASS-LINK 15  
IMPLND IWATER SURO 0.083333 COPY INPUT MEAN  
END MASS-LINK 15

END MASS-LINK

END RUN

# Mitigated UCI File

RUN

GLOBAL

```
WVHM4 model simulation
START      1901 10 01      END      2059 09 30
RUN INTERP OUTPUT LEVEL   3      0
RESUME     0 RUN          1
UNIT SYSTEM 1
```

END GLOBAL

FILES

```
<File> <Un#> <-----File Name----->***
<-ID->                                     ***
WDM      26      2023-3-7-WVHM-6' depth.wdm
MESSU    25      Mit2023-3-7-WVHM-6' depth.MES
          27      Mit2023-3-7-WVHM-6' depth.L61
          28      Mit2023-3-7-WVHM-6' depth.L62
          30      POC2023-3-7-WVHM-6' depth1.dat
```

END FILES

OPN SEQUENCE

```
INGRP          INDELT 00:15
  IMPLND        1
  RCHRES        1
  COPY          1
  COPY         501
  COPY         601
  DISPLY        1
```

END INGRP

END OPN SEQUENCE

DISPLY

DISPLY-INF01

```
# - #<-----Title----->***TRAN PIVL DIG1 FIL1 PYR DIG2 FIL2 YRND
1      Vault 1      MAX          1      2      30      9
```

END DISPLY-INF01

END DISPLY

COPY

TIMESERIES

```
# - # NPT NMN ***
1      1      1
501    1      1
601    1      1
```

END TIMESERIES

END COPY

GENER

OPCODE

```
#      # OPCD ***
```

END OPCODE

PARM

```
#      #      K ***
```

END PARM

END GENER

PERLND

GEN-INFO

```
<PLS ><-----Name----->NBLKS Unit-systems Printer ***
# - # User t-series Engl Metr ***
                               in out      ***
```

END GEN-INFO

\*\*\* Section PWATER\*\*\*

ACTIVITY

```
<PLS > ***** Active Sections *****
# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC ***
```

END ACTIVITY

PRINT-INFO

```
<PLS > ***** Print-flags ***** PIVL PYR
# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC *****
```

END PRINT-INFO

```

PWAT-PARM1
  <PLS > PWATER variable monthly parameter value flags ***
  # - # CSNO RTOP UZFG VCS VUZ VNN VIFW VIRC VLE INFC HWT ***
END PWAT-PARM1

PWAT-PARM2
  <PLS > PWATER input info: Part 2 ***
  # - # ***FOREST LZSN INFILT LSUR SLSUR KVARY AGWRC
END PWAT-PARM2

PWAT-PARM3
  <PLS > PWATER input info: Part 3 ***
  # - # ***PETMAX PETMIN INFEXP INFILD DEEPFR BASETP AGWETP
END PWAT-PARM3

PWAT-PARM4
  <PLS > PWATER input info: Part 4 ***
  # - # CEPSC UZSN NSUR INTFW IRC LZETP ***
END PWAT-PARM4

PWAT-STATE1
  <PLS > *** Initial conditions at start of simulation
  ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
  # - # *** CEPS SURS UZS IFWS LZS AGWS GWVS
END PWAT-STATE1

```

END PERLND

IMPLND

```

GEN-INFO
  <PLS ><-----Name-----> Unit-systems Printer ***
  # - # User t-series Engr Metr ***
  in out ***
  1 ROADS/FLAT 1 1 1 27 0
END GEN-INFO
*** Section IWATER***

```

```

ACTIVITY
  <PLS > ***** Active Sections *****
  # - # ATMP SNOW IWAT SLD IWG IQAL ***
  1 0 0 1 0 0 0
END ACTIVITY

```

```

PRINT-INFO
  <ILS > ***** Print-flags ***** PIVL PYR
  # - # ATMP SNOW IWAT SLD IWG IQAL *****
  1 0 0 4 0 0 0 1 9
END PRINT-INFO

```

```

IWAT-PARM1
  <PLS > IWATER variable monthly parameter value flags ***
  # - # CSNO RTOP VRS VNN RTLI ***
  1 0 0 0 0 0
END IWAT-PARM1

```

```

IWAT-PARM2
  <PLS > IWATER input info: Part 2 ***
  # - # *** LSUR SLSUR NSUR RETSC
  1 400 0.01 0.1 0.1
END IWAT-PARM2

```

```

IWAT-PARM3
  <PLS > IWATER input info: Part 3 ***
  # - # ***PETMAX PETMIN
  1 0 0
END IWAT-PARM3

```

```

IWAT-STATE1
  <PLS > *** Initial conditions at start of simulation
  # - # *** RETS SURS

```



1 0 0  
END IWAT-STATE1

END IMPLND

SCHEMATIC

<-Source-> <Name> #	<--Area--> <-factor-->	<-Target-> <Name> #	MBLK Tbl#	*** ***
Basin 1***				
IMPLND 1	0.887	RCHRES 1	5	
Basin 2***				
IMPLND 1	0.053	RCHRES 1	5	

\*\*\*\*\*Routing\*\*\*\*\*

IMPLND 1	0.887	COPY 1	15	
IMPLND 1	0.053	COPY 1	15	
RCHRES 1	1	COPY 501	16	

END SCHEMATIC

NETWORK

<-Volume-> <Name> #	<-Grp>	<-Member-> <Name> #	<--Mult--> #	Tran strg	<-Target <Name> #	vols #	<-Grp>	<-Member-> <Name> #	*** ***
COPY 501	OUTPUT	MEAN 1	1	48.4	DISPLY 1	1	INPUT	TIMSER 1	

<-Volume-> <Name> #	<-Grp>	<-Member-> <Name> #	<--Mult--> #	Tran strg	<-Target <Name> #	vols #	<-Grp>	<-Member-> <Name> #	*** ***

END NETWORK

RCHRES

GEN-INFO

RCHRES #	Name #	Nexits <----->	Unit User	Systems T-series	Printer Engl	Metr	LKFG	*** *** ***
1	Vault 1	1	1	1	1	28	0	1

END GEN-INFO

\*\*\* Section RCHRES\*\*\*

ACTIVITY

<PLS >	*****	Active Sections	*****
#	-	#	HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFQ PKFG PHFG ***
1		1	0 0 0 0 0 0 0 0 0 0

END ACTIVITY

PRINT-INFO

<PLS >	*****	Print-flags	*****	PIVL	PYR	*****
#	-	#	HYDR ADCA CONS HEAT SED GQL OXRX NUTR PLNK PHCB PIVL PYR			
1		4	0 0 0 0 0 0 0 0 0 0 1 9			

END PRINT-INFO

HYDR-PARM1

RCHRES #	Flags for each VC A1 A2 A3	HYDR Section ODFVFG for each possible exit	***	ODGTFG for each possible exit	***	FUNCT for each possible exit	***
*	*	*	*	*	*	*	*
1	0 1 0 0	4 0 0 0 0		0 0 0 0 0		2 2 2 2 2	

END HYDR-PARM1

HYDR-PARM2

#	-	#	FTABNO	LEN	DELTH	STCOR	KS	DB50	*** ***
1		1		0.01	0.0	0.0	0.5	0.0	

END HYDR-PARM2

HYDR-INIT

RCHRES #	Initial conditions for each HYDR section *** ac-ft	Initial value of COLIND for each possible exit	Initial value of OUTDGT for each possible exit	***
<----->	<----->	<----->	<----->	<----->
1	0	4.0 0.0 0.0 0.0 0.0	0.0 0.0 0.0 0.0 0.0	

END HYDR-INIT  
 END RCHRES

SPEC-ACTIONS  
 END SPEC-ACTIONS

F'TABLES

F'TABLE 1  
 92 4

Depth (ft)	Area (acres)	Volume (acre-ft)	Outflow1 (cfs)	Velocity (ft/sec)	Travel Time*** (Minutes)***
0.000000	0.109298	0.000000	0.000000		
0.066667	0.109298	0.007287	0.001648		
0.133333	0.109298	0.014573	0.002331		
0.200000	0.109298	0.021860	0.002855		
0.266667	0.109298	0.029146	0.003296		
0.333333	0.109298	0.036433	0.003685		
0.400000	0.109298	0.043719	0.004037		
0.466667	0.109298	0.051006	0.004361		
0.533333	0.109298	0.058292	0.004662		
0.600000	0.109298	0.065579	0.004944		
0.666667	0.109298	0.072865	0.005212		
0.733333	0.109298	0.080152	0.005466		
0.800000	0.109298	0.087438	0.005709		
0.866667	0.109298	0.094725	0.005942		
0.933333	0.109298	0.102011	0.006167		
1.000000	0.109298	0.109298	0.006383		
1.066667	0.109298	0.116584	0.006593		
1.133333	0.109298	0.123871	0.006795		
1.200000	0.109298	0.131157	0.006993		
1.266667	0.109298	0.138444	0.007184		
1.333333	0.109298	0.145730	0.007371		
1.400000	0.109298	0.153017	0.007553		
1.466667	0.109298	0.160303	0.007731		
1.533333	0.109298	0.167590	0.007904		
1.600000	0.109298	0.174876	0.008074		
1.666667	0.109298	0.182163	0.008241		
1.733333	0.109298	0.189449	0.008404		
1.800000	0.109298	0.196736	0.008564		
1.866667	0.109298	0.204022	0.008721		
1.933333	0.109298	0.211309	0.008876		
2.000000	0.109298	0.218595	0.009027		
2.066667	0.109298	0.225882	0.009177		
2.133333	0.109298	0.233168	0.009323		
2.200000	0.109298	0.240455	0.009468		
2.266667	0.109298	0.247741	0.009610		
2.333333	0.109298	0.255028	0.009751		
2.400000	0.109298	0.262314	0.009889		
2.466667	0.109298	0.269601	0.010025		
2.533333	0.109298	0.276887	0.010160		
2.600000	0.109298	0.284174	0.010293		
2.666667	0.109298	0.291460	0.010424		
2.733333	0.109298	0.298747	0.010553		
2.800000	0.109298	0.306033	0.010681		
2.866667	0.109298	0.313320	0.010808		
2.933333	0.109298	0.320606	0.010933		
3.000000	0.109298	0.327893	0.011056		
3.066667	0.109298	0.335179	0.011178		
3.133333	0.109298	0.342466	0.011299		
3.200000	0.109298	0.349752	0.011419		
3.266667	0.109298	0.357039	0.011537		
3.333333	0.109298	0.364325	0.011654		
3.400000	0.109298	0.371612	0.011770		
3.466667	0.109298	0.378898	0.011885		
3.533333	0.109298	0.386185	0.011999		
3.600000	0.109298	0.393471	0.012111		
3.666667	0.109298	0.400758	0.012205		
3.733333	0.109298	0.408044	0.012342		
3.800000	0.109298	0.415331	0.012478		
3.866667	0.109298	0.422617	0.012613		
3.933333	0.109298	0.429904	0.012746		

4.000000	0.109298	0.437190	0.020728
4.066667	0.109298	0.444477	0.022809
4.133333	0.109298	0.451763	0.024984
4.200000	0.109298	0.459050	0.027236
4.266667	0.109298	0.466336	0.029550
4.333333	0.109298	0.473623	0.031913
4.400000	0.109298	0.480909	0.034314
4.466667	0.109298	0.488196	0.036741
4.533333	0.109298	0.495482	0.039186
4.600000	0.109298	0.502769	0.041639
4.666667	0.109298	0.510055	0.044536
4.733333	0.109298	0.517342	0.047587
4.800000	0.109298	0.524628	0.050726
4.866667	0.109298	0.531915	0.053950
4.933333	0.109298	0.539201	0.057258
5.000000	0.109298	0.546488	0.060648
5.066667	0.109298	0.553774	0.242976
5.133333	0.109298	0.561061	0.570499
5.200000	0.109298	0.568347	0.968606
5.266667	0.109298	0.575634	1.379103
5.333333	0.109298	0.582920	1.744584
5.400000	0.109298	0.590207	2.021242
5.466667	0.109298	0.597493	2.199625
5.533333	0.109298	0.604780	2.361554
5.600000	0.109298	0.612066	2.501173
5.666667	0.109298	0.619353	2.633232
5.733333	0.109298	0.626639	2.758841
5.800000	0.109298	0.633926	2.878862
5.866667	0.109298	0.641212	2.993981
5.933333	0.109298	0.648499	3.104754
6.000000	0.109298	0.655785	3.211640
6.066667	0.109298	0.663072	3.315020

END FTABLE 1

END FTABLES

EXT SOURCES

<-Volume->	<Member>	SsysSgap	<--Mult-->	Tran	<-Target vols>	<-Grp>	<-Member->	***	
<Name>	#	<Name>	#	tem strg	<-factor-->	strg	<Name>	# #	***
WDM	2	PREC		ENGL	1		PERLND	1 999	EXTNL PREC
WDM	2	PREC		ENGL	1		IMPLND	1 999	EXTNL PREC
WDM	1	EVAP		ENGL	1		PERLND	1 999	EXTNL PETINP
WDM	1	EVAP		ENGL	1		IMPLND	1 999	EXTNL PETINP

END EXT SOURCES

EXT TARGETS

<-Volume->	<-Grp>	<-Member->	<--Mult-->	Tran	<-Volume->	<Member>	Tsys	Tgap	Amd	***
<Name>	#	<Name>	#	#<-factor-->	strg	<Name>	#	<Name>	tem strg	strg***
RCHRES	1	HYDR	RO	1 1	1	WDM	1000	FLOW	ENGL	REPL
RCHRES	1	HYDR	STAGE	1 1	1	WDM	1001	STAG	ENGL	REPL
COPY	1	OUTPUT	MEAN	1 1	48.4	WDM	701	FLOW	ENGL	REPL
COPY	501	OUTPUT	MEAN	1 1	48.4	WDM	801	FLOW	ENGL	REPL
COPY	601	OUTPUT	MEAN	1 1	48.4	WDM	901	FLOW	ENGL	REPL

END EXT TARGETS

MASS-LINK

<Volume>	<-Grp>	<-Member->	<--Mult-->	<Target>	<-Grp>	<-Member->	***
<Name>		<Name>	# #<-factor-->	<Name>		<Name>	# #***
MASS-LINK			5				
IMPLND	IWATER	SURO	0.083333	RCHRES		INFLOW	IVOL
END MASS-LINK			5				
MASS-LINK			15				
IMPLND	IWATER	SURO	0.083333	COPY		INPUT	MEAN
END MASS-LINK			15				
MASS-LINK			16				
RCHRES	ROFLOW			COPY		INPUT	MEAN
END MASS-LINK			16				

END MASS-LINK

END RUN

*Predeveloped HSPF Message File*

## Mitigated HSPF Message File

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1929/ 8/31 24: 0

RCHRES : 1

RELERR	STORS	STOR	MATIN	MATDIF
-0.01272	0.00000	0.0000E+00	0.00000	-1.652E-08

Where:

RELERR is the relative error (ERROR/REFVAL).

ERROR is (STOR-STORS) - MATDIF.

REFVAL is the reference value (STORS+MATIN).

STOR is the storage of material in the processing unit (land-segment or reach/reservoir) at the end of the present interval.

STORS is the storage of material in the pu at the start of the present printout reporting period.

MATIN is the total inflow of material to the pu during the present printout reporting period.

MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1955/ 9/30 24: 0

RCHRES : 1

RELERR	STORS	STOR	MATIN	MATDIF
-1.861E-02	0.00000	0.0000E+00	0.00000	-1.118E-08

Where:

RELERR is the relative error (ERROR/REFVAL).

ERROR is (STOR-STORS) - MATDIF.

REFVAL is the reference value (STORS+MATIN).

STOR is the storage of material in the processing unit (land-segment or reach/reservoir) at the end of the present interval.

STORS is the storage of material in the pu at the start of the present printout reporting period.

MATIN is the total inflow of material to the pu during the present printout reporting period.

MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1960/ 8/31 24: 0

RCHRES : 1

RELERR	STORS	STOR	MATIN	MATDIF
-6.345E-02	0.00000	0.0000E+00	0.00000	-3.145E-09

Where:

RELERR is the relative error (ERROR/REFVAL).  
ERROR is (STOR-STORS) - MATDIF.  
REFVAL is the reference value (STORS+MATIN).  
STOR is the storage of material in the processing unit (land-segment or reach/reservior) at the end of the present interval.  
STORS is the storage of material in the pu at the start of the present printout reporting period.  
MATIN is the total inflow of material to the pu during the present printout reporting period.  
MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1980/ 8/31 24: 0

RCHRES : 1

RELERR	STORS	STOR	MATIN	MATDIF
-1.342E-03	0.00000	0.0000E+00	0.00000	-1.590E-07

Where:

RELERR is the relative error (ERROR/REFVAL).  
ERROR is (STOR-STORS) - MATDIF.  
REFVAL is the reference value (STORS+MATIN).  
STOR is the storage of material in the processing unit (land-segment or reach/reservior) at the end of the present interval.  
STORS is the storage of material in the pu at the start of the present printout reporting period.  
MATIN is the total inflow of material to the pu during the present printout reporting period.  
MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1989/ 8/31 24: 0

RCHRES : 1

RELERR	STORS	STOR	MATIN	MATDIF
-9.547E-02	0.00000	0.0000E+00	0.00000	-2.008E-09

Where:

RELERR is the relative error (ERROR/REFVAL).  
ERROR is (STOR-STORS) - MATDIF.

REFVAL is the reference value (STORS+MATIN).

STOR is the storage of material in the processing unit (land-segment or reach/reservoir) at the end of the present interval.

STORS is the storage of material in the pu at the start of the present printout reporting period.

MATIN is the total inflow of material to the pu during the present printout reporting period.

MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

The count for the WARNING printed above has reached its maximum.

If the condition is encountered again the message will not be repeated.

---



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### *Legal Notice*

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Figure 5.6  
WWHM Calculations  
for Proposed  
Modular Wetland  
Unit



**WWHM2012**

**PROJECT REPORT**

**Modular Wetland  
Calculations**

## General Model Information

Project Name: 2023-3-7-WWHM-6' depth-WQ  
Site Name: ARCO  
Site Address:  
City: PUYALLUP  
Report Date: 3/9/2023  
Gage: 42 IN EAST  
Data Start: 10/01/1901  
Data End: 09/30/2059  
Timestep: 15 Minute  
Precip Scale: 1.000  
Version Date: 2018/03/02  
Version: 4.2.14

## POC Thresholds

---

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

---

# Landuse Basin Data

## Predeveloped Land Use

### Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use C, Forest, Flat	acre 0.887
Pervious Total	0.887
Impervious Land Use	acre
Impervious Total	0
Basin Total	0.887

Element Flows To:		
Surface	Interflow	Groundwater

## Basin 2

Bypass:	No
GroundWater:	No
Pervious Land Use C, Forest, Flat	acre 0.052
Pervious Total	0.052
Impervious Land Use	acre
Impervious Total	0
Basin Total	0.052

Element Flows To:		
Surface	Interflow	Groundwater

### Basin 3

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROADS FLAT	0.01
Impervious Total	0.01
Basin Total	0.01

Element Flows To:		
Surface	Interflow	Groundwater

## Mitigated Land Use

### Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROADS FLAT	0.887
Impervious Total	0.887
Basin Total	0.887

Element Flows To:		
Surface	Interflow	Groundwater
Flow Splitter 1	Flow Splitter 1	



## Basin 2

Bypass:	Yes
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROADS FLAT	0.053
Impervious Total	0.053
Basin Total	0.053

Element Flows To:		
Surface	Interflow	Groundwater
Flow Splitter 1	Flow Splitter 1	

*Routing Elements*  
*Predeveloped Routing*

## Mitigated Routing

### Vault 1

Width: 69 ft.  
Length: 69 ft.  
Depth: 6 ft.  
Discharge Structure  
Riser Height: 5 ft.  
Riser Diameter: 12 in.  
Notch Type: Rectangular  
Notch Width: 0.011 ft.  
Notch Height: 1.391 ft.  
Orifice 1 Diameter: 0.485 in. Elevation:0 ft.  
Element Flows To:  
Outlet 1                      Outlet 2

Vault Hydraulic Table

Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
0.0000	0.109	0.000	0.000	0.000
0.0667	0.109	0.007	0.001	0.000
0.1333	0.109	0.014	0.002	0.000
0.2000	0.109	0.021	0.002	0.000
0.2667	0.109	0.029	0.003	0.000
0.3333	0.109	0.036	0.003	0.000
0.4000	0.109	0.043	0.004	0.000
0.4667	0.109	0.051	0.004	0.000
0.5333	0.109	0.058	0.004	0.000
0.6000	0.109	0.065	0.004	0.000
0.6667	0.109	0.072	0.005	0.000
0.7333	0.109	0.080	0.005	0.000
0.8000	0.109	0.087	0.005	0.000
0.8667	0.109	0.094	0.005	0.000
0.9333	0.109	0.102	0.006	0.000
1.0000	0.109	0.109	0.006	0.000
1.0667	0.109	0.116	0.006	0.000
1.1333	0.109	0.123	0.006	0.000
1.2000	0.109	0.131	0.007	0.000
1.2667	0.109	0.138	0.007	0.000
1.3333	0.109	0.145	0.007	0.000
1.4000	0.109	0.153	0.007	0.000
1.4667	0.109	0.160	0.007	0.000
1.5333	0.109	0.167	0.007	0.000
1.6000	0.109	0.174	0.008	0.000
1.6667	0.109	0.182	0.008	0.000
1.7333	0.109	0.189	0.008	0.000
1.8000	0.109	0.196	0.008	0.000
1.8667	0.109	0.204	0.008	0.000
1.9333	0.109	0.211	0.008	0.000
2.0000	0.109	0.218	0.009	0.000
2.0667	0.109	0.225	0.009	0.000
2.1333	0.109	0.233	0.009	0.000
2.2000	0.109	0.240	0.009	0.000
2.2667	0.109	0.247	0.009	0.000
2.3333	0.109	0.255	0.009	0.000
2.4000	0.109	0.262	0.009	0.000

2.4667	0.109	0.269	0.010	0.000
2.5333	0.109	0.276	0.010	0.000
2.6000	0.109	0.284	0.010	0.000
2.6667	0.109	0.291	0.010	0.000
2.7333	0.109	0.298	0.010	0.000
2.8000	0.109	0.306	0.010	0.000
2.8667	0.109	0.313	0.010	0.000
2.9333	0.109	0.320	0.010	0.000
3.0000	0.109	0.327	0.011	0.000
3.0667	0.109	0.335	0.011	0.000
3.1333	0.109	0.342	0.011	0.000
3.2000	0.109	0.349	0.011	0.000
3.2667	0.109	0.357	0.011	0.000
3.3333	0.109	0.364	0.011	0.000
3.4000	0.109	0.371	0.011	0.000
3.4667	0.109	0.378	0.011	0.000
3.5333	0.109	0.386	0.012	0.000
3.6000	0.109	0.393	0.012	0.000
3.6667	0.109	0.400	0.012	0.000
3.7333	0.109	0.408	0.013	0.000
3.8000	0.109	0.415	0.015	0.000
3.8667	0.109	0.422	0.016	0.000
3.9333	0.109	0.429	0.018	0.000
4.0000	0.109	0.437	0.020	0.000
4.0667	0.109	0.444	0.022	0.000
4.1333	0.109	0.451	0.025	0.000
4.2000	0.109	0.459	0.027	0.000
4.2667	0.109	0.466	0.029	0.000
4.3333	0.109	0.473	0.031	0.000
4.4000	0.109	0.480	0.034	0.000
4.4667	0.109	0.488	0.036	0.000
4.5333	0.109	0.495	0.039	0.000
4.6000	0.109	0.502	0.041	0.000
4.6667	0.109	0.510	0.044	0.000
4.7333	0.109	0.517	0.047	0.000
4.8000	0.109	0.524	0.050	0.000
4.8667	0.109	0.531	0.054	0.000
4.9333	0.109	0.539	0.057	0.000
5.0000	0.109	0.546	0.060	0.000
5.0667	0.109	0.553	0.243	0.000
5.1333	0.109	0.561	0.570	0.000
5.2000	0.109	0.568	0.968	0.000
5.2667	0.109	0.575	1.379	0.000
5.3333	0.109	0.582	1.744	0.000
5.4000	0.109	0.590	2.021	0.000
5.4667	0.109	0.597	2.199	0.000
5.5333	0.109	0.604	2.361	0.000
5.6000	0.109	0.612	2.501	0.000
5.6667	0.109	0.619	2.633	0.000
5.7333	0.109	0.626	2.758	0.000
5.8000	0.109	0.633	2.878	0.000
5.8667	0.109	0.641	2.994	0.000
5.9333	0.109	0.648	3.104	0.000
6.0000	0.109	0.655	3.211	0.000
6.0667	0.109	0.663	3.315	0.000
6.1333	0.000	0.000	3.415	0.000

## Flow Splitter 1

Bottom Length: 10.00 ft.  
 Bottom Length: 10.00 ft.  
 Depth: 10 ft.  
 Side slope 1: 0 To 1  
 Side slope 2: 0 To 1  
 Side slope 3: 0 To 1  
 Side slope 4: 0 To 1

### Threshold Splitter Hydraulic Table

Stage(feet)	Area(ac.)	Volume(ac-ft.)	Primary(cfs)	Secondary(cfs)
0.000	0.002	0.000	0.082	0.000
0.111	0.002	0.000	0.082	0.000
0.222	0.002	0.000	0.082	0.000
0.333	0.002	0.000	0.082	0.000
0.444	0.002	0.001	0.082	0.000
0.555	0.002	0.001	0.082	0.000
0.666	0.002	0.001	0.082	0.000
0.777	0.002	0.001	0.082	0.000
0.888	0.002	0.002	0.082	0.000
1.000	0.002	0.002	0.082	0.000
1.111	0.002	0.002	0.082	0.000
1.222	0.002	0.002	0.082	0.000
1.333	0.002	0.003	0.082	0.000
1.444	0.002	0.003	0.082	0.000
1.555	0.002	0.003	0.082	0.000
1.666	0.002	0.003	0.082	0.000
1.777	0.002	0.004	0.082	0.000
1.888	0.002	0.004	0.082	0.000
2.000	0.002	0.004	0.082	0.000
2.111	0.002	0.004	0.082	0.000
2.222	0.002	0.005	0.082	0.000
2.333	0.002	0.005	0.082	0.000
2.444	0.002	0.005	0.082	0.000
2.555	0.002	0.005	0.082	0.000
2.666	0.002	0.006	0.082	0.000
2.777	0.002	0.006	0.082	0.000
2.888	0.002	0.006	0.082	1000
3.000	0.002	0.006	0.082	1000
3.111	0.002	0.007	0.082	1000
3.222	0.002	0.007	0.082	1000
3.333	0.002	0.007	0.082	1000
3.444	0.002	0.007	0.082	1000
3.555	0.002	0.008	0.082	1000
3.666	0.002	0.008	0.082	1000
3.777	0.002	0.008	0.082	1000
3.888	0.002	0.008	0.082	1000
4.000	0.002	0.009	0.082	1000
4.111	0.002	0.009	0.082	1000
4.222	0.002	0.009	0.082	1000
4.333	0.002	0.009	0.082	1000
4.444	0.002	0.010	0.082	1000
4.555	0.002	0.010	0.082	1000
4.666	0.002	0.010	0.082	1000
4.777	0.002	0.011	0.082	1000
4.888	0.002	0.011	0.082	1000
5.000	0.002	0.011	0.082	1000
5.111	0.002	0.011	0.082	1000

5.222	0.002	0.012	0.082	1000
5.333	0.002	0.012	0.082	1000
5.444	0.002	0.012	0.082	1000
5.555	0.002	0.012	0.082	1000
5.666	0.002	0.013	0.082	1000
5.777	0.002	0.013	0.082	1000
5.888	0.002	0.013	0.082	1000
6.000	0.002	0.013	0.082	1000
6.111	0.002	0.014	0.082	1000
6.222	0.002	0.014	0.082	1000
6.333	0.002	0.014	0.082	1000
6.444	0.002	0.014	0.082	1000
6.555	0.002	0.015	0.082	1000
6.666	0.002	0.015	0.082	1000
6.777	0.002	0.015	0.082	1000
6.888	0.002	0.015	0.082	1000
7.000	0.002	0.016	0.082	1000
7.111	0.002	0.016	0.082	1000
7.222	0.002	0.016	0.082	1000
7.333	0.002	0.016	0.082	1000
7.444	0.002	0.017	0.082	1000
7.555	0.002	0.017	0.082	1000
7.666	0.002	0.017	0.082	1000
7.777	0.002	0.017	0.082	1000
7.888	0.002	0.018	0.082	1000
8.000	0.002	0.018	0.082	1000
8.111	0.002	0.018	0.082	1000
8.222	0.002	0.018	0.082	1000
8.333	0.002	0.019	0.082	1000
8.444	0.002	0.019	0.082	1000
8.555	0.002	0.019	0.082	1000
8.666	0.002	0.019	0.082	1000
8.777	0.002	0.020	0.082	1000
8.888	0.002	0.020	0.082	1000
9.000	0.002	0.020	0.082	1000
9.111	0.002	0.020	0.082	1000
9.222	0.002	0.021	0.082	1000
9.333	0.002	0.021	0.082	1000
9.444	0.002	0.021	0.082	1000
9.555	0.002	0.021	0.082	1000
9.666	0.002	0.022	0.082	1000
9.777	0.002	0.022	0.082	1000
9.888	0.002	0.022	0.082	1000
10.00	0.002	0.023	0.082	1000
10.11	0.002	0.023	0.082	1000

Discharge Structure

Riser Height: 0 ft.

Riser Diameter: 0 in.

Element Flows To:

Outlet 1	Outlet 2
MWS 1	Vault 1

**MWS 1**

MWS Model Number: MWS-L-8-8  
Media Filter Rate (in/hr): 25

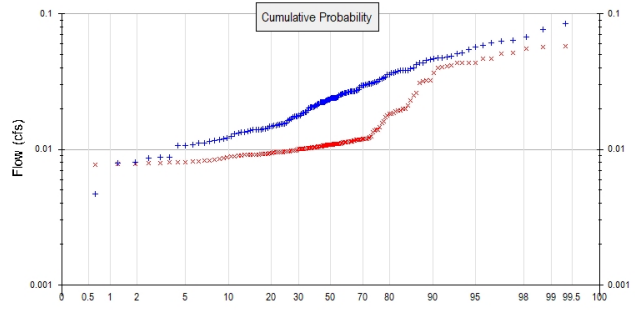
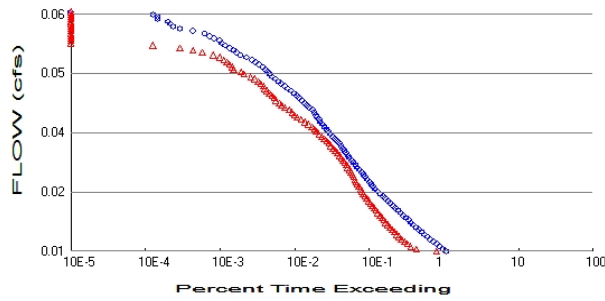
Element Flows To:  
Outlet 1                      Outlet 2  
Vault 1                        Vault 1

MWS Hydraulic Table

Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
10.000	0.000	0.000	0.000	0.000
10.000	0.000	0.000	0.000	0.006
10.000	0.000	0.000	0.000	0.013
10.000	0.000	0.000	0.000	0.019
10.000	0.000	0.000	0.000	0.026
10.000	0.000	0.001	0.000	0.032
10.000	0.000	0.001	0.000	0.039
10.000	0.000	0.002	0.000	0.045
10.000	0.000	0.003	0.000	0.052
10.000	0.000	0.003	0.000	0.058
10.000	0.000	0.004	0.000	0.065
10.000	0.000	0.005	0.000	0.072
10.000	0.000	0.006	0.000	0.078
10.000	0.000	0.007	0.000	0.085
10.000	0.000	0.009	0.000	0.091
10.000	0.000	0.010	0.000	0.098
10.000	0.000	0.011	0.000	0.104
10.000	0.000	0.013	0.000	0.111
10.000	0.000	0.014	0.000	0.117
10.000	0.000	0.016	0.000	0.124
10.000	0.000	0.018	0.000	0.130
10.000	0.000	0.019	0.000	0.137
10.000	0.000	0.021	0.000	0.143
10.000	0.000	0.023	0.000	0.150
10.000	0.000	0.025	0.000	0.157
10.000	0.000	0.027	0.000	0.163
10.000	0.000	0.030	0.000	0.170
10.000	0.000	0.032	0.000	0.176
10.000	0.000	0.034	0.000	0.183
10.000	0.000	0.037	0.000	0.189
10.000	0.000	0.039	0.000	0.196
10.000	0.000	0.042	0.000	0.202
10.000	0.000	0.045	0.000	0.209
10.000	0.000	0.048	0.000	0.215
10.000	0.000	0.051	0.000	0.222
10.000	0.000	0.054	6.000	0.222

# Analysis Results

## POC 1



+ Predeveloped x Mitigated

### Predeveloped Landuse Totals for POC #1

Total Pervious Area: 0.939  
 Total Impervious Area: 0.01

### Mitigated Landuse Totals for POC #1

Total Pervious Area: 0  
 Total Impervious Area: 0.94

Flow Frequency Method: Log Pearson Type III 17B

### Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.023355
5 year	0.035927
10 year	0.044562
25 year	0.055664
50 year	0.064014
100 year	0.0724

### Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0.011997
5 year	0.019235
10 year	0.025928
25 year	0.037156
50 year	0.047988
100 year	0.061382

## Annual Peaks

### Annual Peaks for Predeveloped and Mitigated. POC #1

Year	Predeveloped	Mitigated
1902	0.023	0.011
1903	0.015	0.009
1904	0.031	0.010
1905	0.014	0.012
1906	0.008	0.008
1907	0.037	0.010
1908	0.026	0.009
1909	0.025	0.011
1910	0.036	0.011
1911	0.023	0.010



1912	0.084	0.012
1913	0.036	0.019
1914	0.012	0.008
1915	0.017	0.014
1916	0.023	0.010
1917	0.009	0.010
1918	0.024	0.032
1919	0.021	0.010
1920	0.024	0.010
1921	0.027	0.012
1922	0.026	0.011
1923	0.021	0.012
1924	0.011	0.010
1925	0.014	0.009
1926	0.024	0.010
1927	0.019	0.010
1928	0.018	0.011
1929	0.037	0.014
1930	0.023	0.010
1931	0.022	0.011
1932	0.018	0.012
1933	0.021	0.011
1934	0.048	0.051
1935	0.022	0.025
1936	0.021	0.012
1937	0.034	0.010
1938	0.020	0.011
1939	0.003	0.008
1940	0.022	0.012
1941	0.015	0.008
1942	0.033	0.044
1943	0.016	0.011
1944	0.039	0.019
1945	0.027	0.011
1946	0.017	0.009
1947	0.012	0.010
1948	0.049	0.011
1949	0.043	0.023
1950	0.014	0.010
1951	0.019	0.010
1952	0.064	0.021
1953	0.058	0.044
1954	0.022	0.011
1955	0.019	0.009
1956	0.011	0.009
1957	0.031	0.015
1958	0.061	0.055
1959	0.038	0.047
1960	0.011	0.008
1961	0.038	0.043
1962	0.022	0.011
1963	0.011	0.008
1964	0.013	0.009
1965	0.044	0.043
1966	0.014	0.010
1967	0.022	0.009
1968	0.022	0.012
1969	0.020	0.011

1970	0.031	0.011
1971	0.045	0.031
1972	0.030	0.011
1973	0.040	0.020
1974	0.024	0.010
1975	0.048	0.057
1976	0.026	0.011
1977	0.013	0.008
1978	0.042	0.041
1979	0.013	0.010
1980	0.025	0.011
1981	0.024	0.011
1982	0.014	0.008
1983	0.038	0.012
1984	0.019	0.010
1985	0.029	0.010
1986	0.024	0.012
1987	0.047	0.032
1988	0.028	0.019
1989	0.026	0.010
1990	0.030	0.010
1991	0.024	0.011
1992	0.030	0.032
1993	0.031	0.011
1994	0.046	0.011
1995	0.012	0.010
1996	0.051	0.041
1997	0.023	0.009
1998	0.026	0.010
1999	0.005	0.009
2000	0.019	0.012
2001	0.012	0.008
2002	0.038	0.011
2003	0.029	0.011
2004	0.027	0.011
2005	0.052	0.011
2006	0.015	0.010
2007	0.017	0.011
2008	0.025	0.010
2009	0.018	0.010
2010	0.015	0.013
2011	0.014	0.010
2012	0.024	0.010
2013	0.015	0.008
2014	0.011	0.008
2015	0.021	0.010
2016	0.009	0.010
2017	0.033	0.018
2018	0.063	0.052
2019	0.068	0.046
2020	0.020	0.009
2021	0.032	0.026
2022	0.014	0.009
2023	0.027	0.011
2024	0.077	0.010
2025	0.024	0.011
2026	0.038	0.018
2027	0.015	0.009

2028	0.013	0.008
2029	0.026	0.020
2030	0.047	0.012
2031	0.015	0.009
2032	0.011	0.008
2033	0.014	0.009
2034	0.015	0.010
2035	0.055	0.058
2036	0.030	0.012
2037	0.009	0.009
2038	0.027	0.018
2039	0.008	0.007
2040	0.015	0.010
2041	0.017	0.009
2042	0.057	0.037
2043	0.027	0.018
2044	0.034	0.016
2045	0.023	0.012
2046	0.027	0.040
2047	0.020	0.012
2048	0.027	0.011
2049	0.024	0.011
2050	0.018	0.010
2051	0.030	0.011
2052	0.016	0.011
2053	0.026	0.041
2054	0.031	0.017
2055	0.013	0.009
2056	0.012	0.009
2057	0.018	0.011
2058	0.022	0.014
2059	0.037	0.016

### Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.0843	0.0575
2	0.0769	0.0568
3	0.0677	0.0555
4	0.0641	0.0518
5	0.0626	0.0507
6	0.0607	0.0469
7	0.0583	0.0464
8	0.0570	0.0436
9	0.0546	0.0436
10	0.0517	0.0433
11	0.0511	0.0432
12	0.0488	0.0414
13	0.0477	0.0411
14	0.0475	0.0407
15	0.0470	0.0402
16	0.0467	0.0366
17	0.0459	0.0322
18	0.0452	0.0321
19	0.0436	0.0318
20	0.0435	0.0311
21	0.0421	0.0262
22	0.0401	0.0249

23	0.0394	0.0230
24	0.0384	0.0209
25	0.0384	0.0200
26	0.0384	0.0199
27	0.0381	0.0195
28	0.0377	0.0192
29	0.0372	0.0190
30	0.0368	0.0184
31	0.0366	0.0184
32	0.0357	0.0183
33	0.0357	0.0178
34	0.0343	0.0174
35	0.0335	0.0160
36	0.0333	0.0156
37	0.0329	0.0145
38	0.0318	0.0140
39	0.0313	0.0140
40	0.0313	0.0138
41	0.0309	0.0134
42	0.0305	0.0125
43	0.0305	0.0122
44	0.0305	0.0120
45	0.0299	0.0120
46	0.0299	0.0120
47	0.0296	0.0119
48	0.0295	0.0119
49	0.0294	0.0118
50	0.0288	0.0118
51	0.0277	0.0117
52	0.0272	0.0117
53	0.0272	0.0117
54	0.0270	0.0116
55	0.0268	0.0116
56	0.0268	0.0115
57	0.0268	0.0115
58	0.0267	0.0114
59	0.0265	0.0114
60	0.0265	0.0114
61	0.0262	0.0113
62	0.0262	0.0113
63	0.0260	0.0113
64	0.0259	0.0112
65	0.0258	0.0112
66	0.0257	0.0112
67	0.0255	0.0112
68	0.0253	0.0110
69	0.0251	0.0110
70	0.0242	0.0110
71	0.0242	0.0110
72	0.0241	0.0110
73	0.0240	0.0109
74	0.0240	0.0109
75	0.0240	0.0109
76	0.0239	0.0109
77	0.0239	0.0108
78	0.0238	0.0108
79	0.0235	0.0108
80	0.0233	0.0108

81	0.0232	0.0108
82	0.0232	0.0108
83	0.0230	0.0108
84	0.0229	0.0107
85	0.0226	0.0107
86	0.0224	0.0107
87	0.0224	0.0107
88	0.0222	0.0106
89	0.0222	0.0106
90	0.0222	0.0105
91	0.0222	0.0105
92	0.0217	0.0105
93	0.0216	0.0105
94	0.0214	0.0104
95	0.0212	0.0104
96	0.0212	0.0104
97	0.0207	0.0104
98	0.0206	0.0104
99	0.0205	0.0104
100	0.0205	0.0103
101	0.0205	0.0103
102	0.0201	0.0103
103	0.0194	0.0102
104	0.0189	0.0102
105	0.0188	0.0101
106	0.0186	0.0101
107	0.0186	0.0101
108	0.0182	0.0101
109	0.0178	0.0101
110	0.0178	0.0101
111	0.0176	0.0099
112	0.0176	0.0099
113	0.0175	0.0098
114	0.0174	0.0098
115	0.0169	0.0098
116	0.0167	0.0097
117	0.0162	0.0097
118	0.0157	0.0097
119	0.0154	0.0096
120	0.0154	0.0096
121	0.0153	0.0096
122	0.0153	0.0095
123	0.0152	0.0095
124	0.0150	0.0095
125	0.0150	0.0095
126	0.0147	0.0094
127	0.0147	0.0093
128	0.0144	0.0093
129	0.0142	0.0093
130	0.0141	0.0092
131	0.0141	0.0092
132	0.0141	0.0092
133	0.0140	0.0092
134	0.0139	0.0092
135	0.0138	0.0091
136	0.0135	0.0091
137	0.0135	0.0091
138	0.0134	0.0090

139	0.0132	0.0090
140	0.0130	0.0089
141	0.0124	0.0088
142	0.0122	0.0087
143	0.0119	0.0086
144	0.0118	0.0085
145	0.0117	0.0084
146	0.0115	0.0083
147	0.0112	0.0082
148	0.0111	0.0082
149	0.0108	0.0081
150	0.0107	0.0081
151	0.0107	0.0081
152	0.0088	0.0081
153	0.0087	0.0080
154	0.0086	0.0079
155	0.0080	0.0078
156	0.0080	0.0078
157	0.0047	0.0077
158	0.0033	0.0072

## Duration Flows

The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0117	59389	44088	74	Pass
0.0122	53567	23545	43	Pass
0.0127	48525	19867	40	Pass
0.0133	43988	18005	40	Pass
0.0138	39966	16315	40	Pass
0.0143	36387	14936	41	Pass
0.0148	33224	13850	41	Pass
0.0154	30326	12825	42	Pass
0.0159	27717	11944	43	Pass
0.0164	25457	11152	43	Pass
0.0170	23468	10471	44	Pass
0.0175	21662	9817	45	Pass
0.0180	20016	9197	45	Pass
0.0185	18520	8654	46	Pass
0.0191	17097	8183	47	Pass
0.0196	15756	7712	48	Pass
0.0201	14509	7252	49	Pass
0.0207	13446	6836	50	Pass
0.0212	12421	6449	51	Pass
0.0217	11485	6089	53	Pass
0.0223	10615	5740	54	Pass
0.0228	9817	5424	55	Pass
0.0233	9064	5105	56	Pass
0.0238	8371	4825	57	Pass
0.0244	7712	4589	59	Pass
0.0249	7152	4397	61	Pass
0.0254	6670	4207	63	Pass
0.0260	6238	4023	64	Pass
0.0265	5878	3810	64	Pass
0.0270	5528	3662	66	Pass
0.0275	5215	3524	67	Pass
0.0281	4900	3397	69	Pass
0.0286	4600	3282	71	Pass
0.0291	4348	3158	72	Pass
0.0297	4089	3003	73	Pass
0.0302	3846	2852	74	Pass
0.0307	3615	2740	75	Pass
0.0312	3390	2598	76	Pass
0.0318	3220	2486	77	Pass
0.0323	3014	2333	77	Pass
0.0328	2866	2213	77	Pass
0.0334	2694	2068	76	Pass
0.0339	2568	1935	75	Pass
0.0344	2439	1817	74	Pass
0.0349	2319	1687	72	Pass
0.0355	2196	1590	72	Pass
0.0360	2068	1491	72	Pass
0.0365	1926	1379	71	Pass
0.0371	1782	1269	71	Pass
0.0376	1666	1176	70	Pass
0.0381	1561	1086	69	Pass
0.0386	1470	1021	69	Pass
0.0392	1381	936	67	Pass

0.0397	1300	851	65	Pass
0.0402	1226	760	61	Pass
0.0408	1169	671	57	Pass
0.0413	1108	595	53	Pass
0.0418	1054	548	51	Pass
0.0423	1011	509	50	Pass
0.0429	947	466	49	Pass
0.0434	867	407	46	Pass
0.0439	806	369	45	Pass
0.0445	748	345	46	Pass
0.0450	689	309	44	Pass
0.0455	636	293	46	Pass
0.0460	595	273	45	Pass
0.0466	541	252	46	Pass
0.0471	497	233	46	Pass
0.0476	448	219	48	Pass
0.0482	404	211	52	Pass
0.0487	367	195	53	Pass
0.0492	334	179	53	Pass
0.0497	304	161	52	Pass
0.0503	278	143	51	Pass
0.0508	254	116	45	Pass
0.0513	240	105	43	Pass
0.0519	224	84	37	Pass
0.0524	206	80	38	Pass
0.0529	184	74	40	Pass
0.0534	165	68	41	Pass
0.0540	147	62	42	Pass
0.0545	125	55	44	Pass
0.0550	103	45	43	Pass
0.0556	91	32	35	Pass
0.0561	84	24	28	Pass
0.0566	75	16	21	Pass
0.0571	65	7	10	Pass
0.0577	58	0	0	Pass
0.0582	54	0	0	Pass
0.0587	46	0	0	Pass
0.0593	40	0	0	Pass
0.0598	34	0	0	Pass
0.0603	24	0	0	Pass
0.0608	16	0	0	Pass
0.0614	13	0	0	Pass
0.0619	12	0	0	Pass
0.0624	11	0	0	Pass
0.0630	8	0	0	Pass
0.0635	8	0	0	Pass
0.0640	7	0	0	Pass



## Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume: 0.0203 acre-feet

On-line facility target flow: 0.0102 cfs.

Adjusted for 15 min: 0.0102 cfs.

Off-line facility target flow: 0.007 cfs.

Adjusted for 15 min: 0.007 cfs.

# LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Vault 1 POC	<input type="checkbox"/>	386.91			<input type="checkbox"/>	0.00			
Flow Splitter 1	<input type="checkbox"/>	387.05			<input type="checkbox"/>	8.51			
MWS 1	<input checked="" type="checkbox"/>	354.02	389.03	389.03	<input checked="" type="checkbox"/>	100.00	389.03	100.00	Treat. Credit
Total Volume Infiltrated		1127.97	389.03	389.03		34.30	389.03	389 / 389 = 100%	Treat. Credit = 100%
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Failed

Treated 100% of runoff up to the WQ flow rate of 0.0821 CFS.

## *Model Default Modifications*

Total of 0 changes have been made.

### *PERLND Changes*

No PERLND changes have been made.

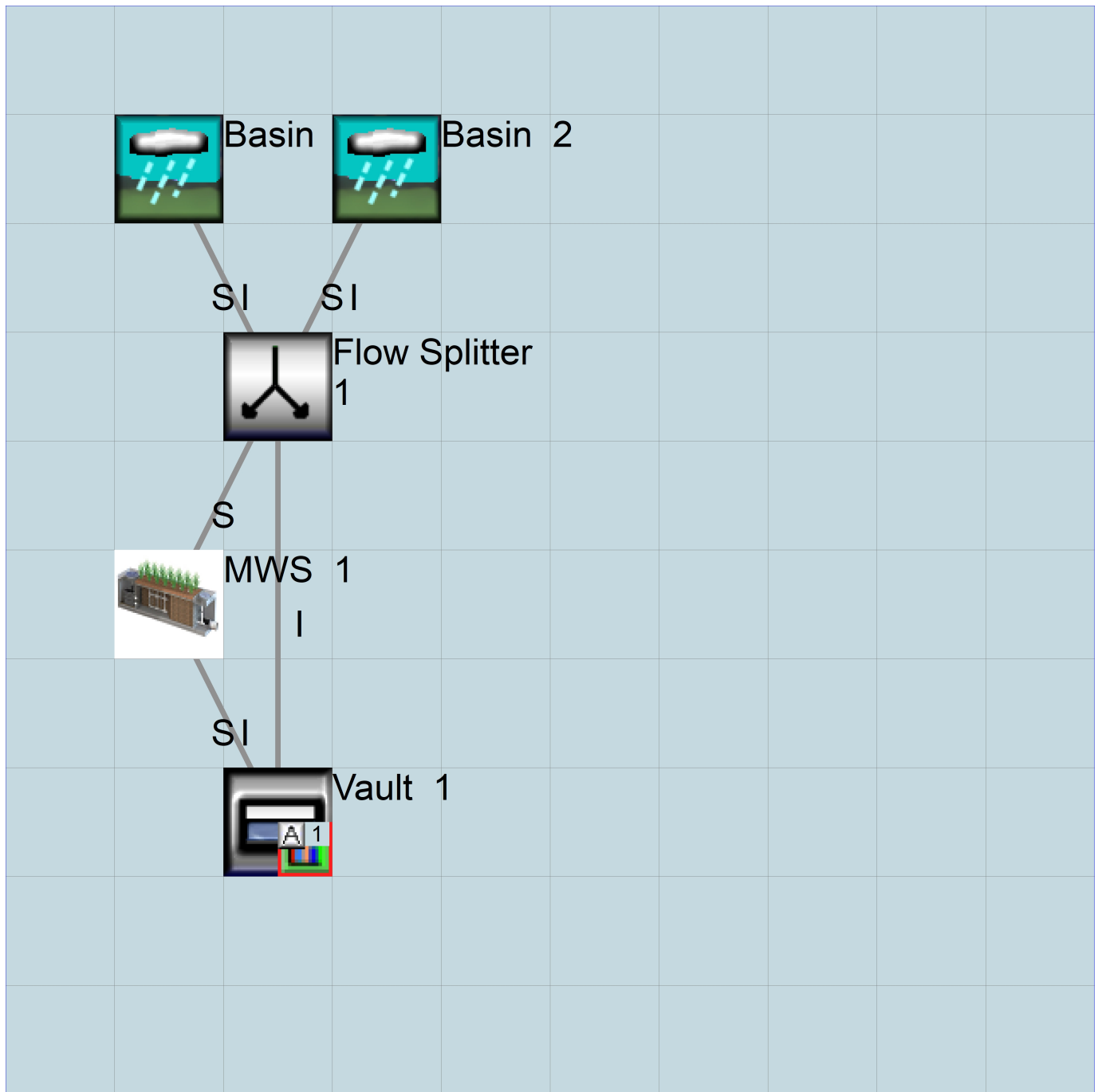
### *IMPLND Changes*

No IMPLND changes have been made.

Appendix  
Predeveloped Schematic



Mitigated Schematic



# Predeveloped UCI File

RUN

GLOBAL

```
WVHM4 model simulation
START      1901 10 01      END      2059 09 30
RUN INTERP OUTPUT LEVEL   3      0
RESUME     0 RUN         1
UNIT SYSTEM 1
```

END GLOBAL

FILES

```
<File> <Un#> <-----File Name----->***
<-ID->                                     ***
WDM      26      2023-3-7-WVHM-6' depth-WQ.wdm
MESSU    25      Pre2023-3-7-WVHM-6' depth-WQ.MES
          27      Pre2023-3-7-WVHM-6' depth-WQ.L61
          28      Pre2023-3-7-WVHM-6' depth-WQ.L62
          30      POC2023-3-7-WVHM-6' depth-WQ1.dat
```

END FILES

OPN SEQUENCE

```
INGRP          INDELT 00:15
  PERLND        10
  IMPLND         1
  COPY          501
  DISPLY         1
```

END INGRP

END OPN SEQUENCE

DISPLY

DISPLY-INFO1

```
# - #<-----Title----->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
1      Basin 1          MAX          1      2      30      9
```

END DISPLY-INFO1

END DISPLY

COPY

TIMESERIES

```
# - # NPT NMN ***
1      1      1
501    1      1
```

END TIMESERIES

END COPY

GENER

OPCODE

```
# # OPCD ***
```

END OPCODE

PARM

```
# # K ***
```

END PARM

END GENER

PERLND

GEN-INFO

```
<PLS ><-----Name----->NBLKS Unit-systems Printer ***
# - # User t-series Engl Metr ***
          in out ***
```

```
10      C, Forest, Flat      1      1      1      1      27      0
```

END GEN-INFO

\*\*\* Section PWATER\*\*\*

ACTIVITY

```
<PLS > ***** Active Sections *****
# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC ***
10      0      0      1      0      0      0      0      0      0      0      0      0
```

END ACTIVITY

PRINT-INFO

```
<PLS > ***** Print-flags ***** PIVL PYR
# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC *****
10      0      0      4      0      0      0      0      0      0      0      0      0      1      9
```

END PRINT-INFO

```

PWAT-PARM1
<PLS > PWATER variable monthly parameter value flags ***
# - # CSNO RTOP UZFG VCS VUZ VNN VIFW VIRC VLE INFC HWT ***
10 0 0 0 0 0 0 0 0 0 0 0
END PWAT-PARM1

```

```

PWAT-PARM2
<PLS > PWATER input info: Part 2 ***
# - # ***FOREST LZSN INFILT LSUR SLSUR KVARY AGWRC
10 0 4.5 0.08 400 0.05 0.5 0.996
END PWAT-PARM2

```

```

PWAT-PARM3
<PLS > PWATER input info: Part 3 ***
# - # ***PETMAX PETMIN INFEXP INFILD DEEPFR BASETP AGWETP
10 0 0 2 2 0 0 0
END PWAT-PARM3

```

```

PWAT-PARM4
<PLS > PWATER input info: Part 4 ***
# - # CEPSC UZSN NSUR INTFW IRC LZETP ***
10 0.2 0.5 0.35 6 0.5 0.7
END PWAT-PARM4

```

```

PWAT-STATE1
<PLS > *** Initial conditions at start of simulation
ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
# - # *** CEPS SURS UZS IFWS LZS AGWS GWVS
10 0 0 0 0 2.5 1 0
END PWAT-STATE1

```

END PERLND

IMPLND

```

GEN-INFO
<PLS ><-----Name-----> Unit-systems Printer ***
# - # User t-series Engl Metr ***
in out ***
1 ROADS/FLAT 1 1 1 27 0
END GEN-INFO
*** Section IWATER***

```

```

ACTIVITY
<PLS > ***** Active Sections *****
# - # ATMP SNOW IWAT SLD IWG IQAL ***
1 0 0 1 0 0 0
END ACTIVITY

```

```

PRINT-INFO
<ILS > ***** Print-flags ***** PIVL PYR
# - # ATMP SNOW IWAT SLD IWG IQAL *****
1 0 0 4 0 0 0 1 9
END PRINT-INFO

```

```

IWAT-PARM1
<PLS > IWATER variable monthly parameter value flags ***
# - # CSNO RTOP VRS VNN RTLI ***
1 0 0 0 0 0
END IWAT-PARM1

```

```

IWAT-PARM2
<PLS > IWATER input info: Part 2 ***
# - # *** LSUR SLSUR NSUR RETSC
1 400 0.01 0.1 0.1
END IWAT-PARM2

```

```

IWAT-PARM3
<PLS > IWATER input info: Part 3 ***
# - # ***PETMAX PETMIN
1 0 0

```

END IWAT-PARM3

IWAT-STATE1

<PLS > \*\*\* Initial conditions at start of simulation

# - # \*\*\* RETS SURS

1 0 0

END IWAT-STATE1

END IMPLND

SCHEMATIC

<-Source->	<-Area-->	<-Target->	MBLK	***
<Name> #	<-factor->	<Name> #	Tbl#	***
Basin 1***				
PERLND 10	0.887	COPY 501	12	
PERLND 10	0.887	COPY 501	13	
Basin 2***				
PERLND 10	0.052	COPY 501	12	
PERLND 10	0.052	COPY 501	13	
Basin 3***				
IMPLND 1	0.01	COPY 501	15	

\*\*\*\*\*Routing\*\*\*\*\*

END SCHEMATIC

NETWORK

<-Volume->	<-Grp>	<-Member->	<--Mult-->	Tran	<-Target vols>	<-Grp>	<-Member->	***
<Name> #		<Name> #	#	<-factor->	strg	<Name> #	#	<Name> #
COPY 501	OUTPUT	MEAN	1	1	48.4	DISPLY	1	INPUT TIMSER 1

<-Volume->	<-Grp>	<-Member->	<--Mult-->	Tran	<-Target vols>	<-Grp>	<-Member->	***
<Name> #		<Name> #	#	<-factor->	strg	<Name> #	#	<Name> #

RCHRES

GEN-INFO

RCHRES	Name	Nexits	Unit Systems	Printer	***
# - #	<----->	<---->	User T-series	Engl Metr LKFG	***
			in out		***

END GEN-INFO

\*\*\* Section RCHRES\*\*\*

ACTIVITY

<PLS > \*\*\*\*\* Active Sections \*\*\*\*\*

# - # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUGF PKFG PHFG \*\*\*

END ACTIVITY

PRINT-INFO

<PLS > \*\*\*\*\* Print-flags \*\*\*\*\* PIVL PYR

# - # HYDR ADCA CONS HEAT SED GQL OXRX NUTR PLNK PHCB PIVL PYR \*\*\*\*\*

END PRINT-INFO

HYDR-PARM1

RCHRES Flags for each HYDR Section \*\*\*

# - #	VC A1 A2 A3	ODFVFG for each	***	ODGTFG for each	FUNCT for each
	FG FG FG FG	possible exit	***	possible exit	possible exit
	* * * *	* * * *		* * * *	***

END HYDR-PARM1

HYDR-PARM2

# - #	FTABNO	LEN	DELTH	STCOR	KS	DB50	***
	<----->	<----->	<----->	<----->	<----->	<----->	***

END HYDR-PARM2

HYDR-INIT

RCHRES Initial conditions for each HYDR section \*\*\*

# - #	*** VOL	Initial value of COLIND	Initial value of OUTDGT
	*** ac-ft	for each possible exit	for each possible exit
	<----->	<----->	<----->



END HYDR-INIT  
END RCHRES

SPEC-ACTIONS  
END SPEC-ACTIONS  
FTABLES  
END FTABLES

EXT SOURCES  
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member-> \*\*\*  
<Name> # <Name> # tem strg<-factor->strg <Name> # # <Name> # # \*\*\*  
WDM 2 PREC ENGL 1 PERLND 1 999 EXTNL PREC  
WDM 2 PREC ENGL 1 IMPLND 1 999 EXTNL PREC  
WDM 1 EVAP ENGL 1 PERLND 1 999 EXTNL PETINP  
WDM 1 EVAP ENGL 1 IMPLND 1 999 EXTNL PETINP

END EXT SOURCES

EXT TARGETS  
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd \*\*\*  
<Name> # <Name> # #<-factor->strg <Name> # <Name> tem strg strg\*\*\*  
COPY 501 OUTPUT MEAN 1 1 48.4 WDM 501 FLOW ENGL REPL  
END EXT TARGETS

MASS-LINK  
<Volume> <-Grp> <-Member-><--Mult--> <Target> <-Grp> <-Member->\*\*\*  
<Name> <Name> # #<-factor-> <Name> <Name> # #\*\*\*  
MASS-LINK 12  
PERLND PWATER SURO 0.083333 COPY INPUT MEAN  
END MASS-LINK 12  
  
MASS-LINK 13  
PERLND PWATER IFWO 0.083333 COPY INPUT MEAN  
END MASS-LINK 13  
  
MASS-LINK 15  
IMPLND IWATER SURO 0.083333 COPY INPUT MEAN  
END MASS-LINK 15

END MASS-LINK

END RUN

# Mitigated UCI File

RUN

GLOBAL

```
WVHM4 model simulation
START      1901 10 01      END      2059 09 30
RUN INTERP OUTPUT LEVEL   3      0
RESUME     0 RUN          1
UNIT SYSTEM 1
```

END GLOBAL

FILES

```
<File> <Un#> <-----File Name----->***
<-ID->                                     ***
WDM      26      2023-3-7-WVHM-6' depth-WQ.wdm
MESSU    25      Mit2023-3-7-WVHM-6' depth-WQ.MES
          27      Mit2023-3-7-WVHM-6' depth-WQ.L61
          28      Mit2023-3-7-WVHM-6' depth-WQ.L62
          30      POC2023-3-7-WVHM-6' depth-WQ1.dat
```

END FILES

OPN SEQUENCE

```
INGRP          INDELT 00:15
  IMPLND        1
  RCHRES        1
  RCHRES        2
  RCHRES        3
  COPY          1
  COPY          501
  COPY          601
  DISPLY        1
```

END INGRP

END OPN SEQUENCE

DISPLY

DISPLY-INFO1

```
# - #<-----Title----->***TRAN PIVL DIG1 FIL1 PYR DIG2 FIL2 YRND
1      Vault 1      MAX      1      2      30      9
```

END DISPLY-INFO1

END DISPLY

COPY

TIMESERIES

```
# - # NPT NMN ***
1      1      1
501    1      1
601    1      1
```

END TIMESERIES

END COPY

GENER

OPCODE

```
#      # OPCODE ***
```

END OPCODE

PARM

```
#      #      K ***
```

END PARM

END GENER

PERLND

GEN-INFO

```
<PLS ><-----Name----->NBLKS Unit-systems Printer ***
# - # User t-series Engl Metr ***
                               in out      ***
```

END GEN-INFO

\*\*\* Section PWATER\*\*\*

ACTIVITY

```
<PLS > ***** Active Sections *****
# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC ***
```

END ACTIVITY

PRINT-INFO

```
<PLS > ***** Print-flags ***** PIVL PYR
```

# - # ATMP SNOW PWAT SED PST PWG PQAL MSTL PEST NITR PHOS TRAC \*\*\*\*\*  
END PRINT-INFO

PWAT-PARM1  
<PLS > PWATER variable monthly parameter value flags \*\*\*  
# - # CSNO RTOP UZFG VCS VUZ VNN VIFW VIRC VLE INFC HWT \*\*\*  
END PWAT-PARM1

PWAT-PARM2  
<PLS > PWATER input info: Part 2 \*\*\*  
# - # \*\*\*FOREST LZSN INFILT LSUR SLSUR KVARY AGWRC  
END PWAT-PARM2

PWAT-PARM3  
<PLS > PWATER input info: Part 3 \*\*\*  
# - # \*\*\*PETMAX PETMIN INFEXP INFILD DEEPFR BASETP AGWETP  
END PWAT-PARM3

PWAT-PARM4  
<PLS > PWATER input info: Part 4 \*\*\*  
# - # CEPSC UZSN NSUR INTFW IRC LZETP \*\*\*  
END PWAT-PARM4

PWAT-STATE1  
<PLS > \*\*\* Initial conditions at start of simulation  
ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 \*\*\*  
# - # \*\*\* CEPS SURS UZS IFWS LZS AGWS GWVS  
END PWAT-STATE1

END PERLND

IMPLND

GEN-INFO  
<PLS ><-----Name-----> Unit-systems Printer \*\*\*  
# - # User t-series Engl Metr \*\*\*  
in out \*\*\*  
1 ROADS/FLAT 1 1 1 27 0  
END GEN-INFO  
\*\*\* Section IWATER\*\*\*

ACTIVITY  
<PLS > \*\*\*\*\* Active Sections \*\*\*\*\*  
# - # ATMP SNOW IWAT SLD IWG IQAL \*\*\*  
1 0 0 1 0 0 0  
END ACTIVITY

PRINT-INFO  
<ILS > \*\*\*\*\* Print-flags \*\*\*\*\* PIVL PYR  
# - # ATMP SNOW IWAT SLD IWG IQAL \*\*\*\*\*  
1 0 0 4 0 0 0 1 9  
END PRINT-INFO

IWAT-PARM1  
<PLS > IWATER variable monthly parameter value flags \*\*\*  
# - # CSNO RTOP VRS VNN RTLI \*\*\*  
1 0 0 0 0 0  
END IWAT-PARM1

IWAT-PARM2  
<PLS > IWATER input info: Part 2 \*\*\*  
# - # \*\*\* LSUR SLSUR NSUR RETSC  
1 400 0.01 0.1 0.1  
END IWAT-PARM2

IWAT-PARM3  
<PLS > IWATER input info: Part 3 \*\*\*  
# - # \*\*\*PETMAX PETMIN  
1 0 0  
END IWAT-PARM3

IWAT-STATE1

```

<PLS > *** Initial conditions at start of simulation
# - # *** RETS      SURS
1       0          0
END IWAT-STATE1

```

```
END IMPLND
```

```

SCHEMATIC
<-Source->          <--Area-->          <-Target->          MBLK          ***
<Name> #           <-factor->          <Name> #          Tbl#          ***
Basin 1***
IMPLND 1           0.887          RCHRES 1          5
Basin 2***
IMPLND 1           0.053          RCHRES 1          5

```

```

*****Routing*****
RCHRES 1           1          RCHRES 2          7
RCHRES 1           1          RCHRES 3          8
RCHRES 1           1          COPY 1          18
RCHRES 2           1          RCHRES 3          7
RCHRES 2           1          COPY 1          17
RCHRES 2           1          RCHRES 3          8
RCHRES 2           1          COPY 1          18
RCHRES 3           1          COPY 501         16
END SCHEMATIC

```

```

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member-> ***
<Name> #           <Name> # #<-factor->strg <Name> # #           <Name> # # ***
COPY 501 OUTPUT MEAN 1 1 48.4          DISPLY 1          INPUT TIMSER 1

```

```

<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member-> ***
<Name> #           <Name> # #<-factor->strg <Name> # #           <Name> # # ***
END NETWORK

```

```

RCHRES
GEN-INFO
RCHRES          Name          Nexits          Unit Systems          Printer          ***
# - #<-----><----> User T-series Engl Metr LKFG          ***
                               in out
1      Flow Splitter 1-013      2      1      1      1      28      0      1          ***
2      MWS 1          2      1      1      1      28      0      1
3      Vault 1        1      1      1      1      28      0      1
END GEN-INFO
*** Section RCHRES***

```

```

ACTIVITY
<PLS > ***** Active Sections *****
# - # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFQ PKFG PHFG ***
1      1      0      0      0      0      0      0      0      0      0
2      1      0      0      0      0      0      0      0      0      0
3      1      0      0      0      0      0      0      0      0      0
END ACTIVITY

```

```

PRINT-INFO
<PLS > ***** Print-flags ***** PIVL PYR
# - # HYDR ADCA CONS HEAT SED GQL OXRX NUTR PLNK PHCB PIVL PYR *****
1      4      0      0      0      0      0      0      0      0      0      1      9
2      4      0      0      0      0      0      0      0      0      0      1      9
3      4      0      0      0      0      0      0      0      0      0      1      9
END PRINT-INFO

```

```

HYDR-PARM1
RCHRES          Flags for each HYDR Section          ***
# - # VC A1 A2 A3 ODFVFG for each *** ODGTFG for each          FUNCT for each
          FG FG FG FG possible exit *** possible exit          possible exit
          * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
1      0 1 0 0      4 5 0 0 0          0 0 0 0 0          2 2 2 2 2

```

```

2      0 1 0 0      4 5 0 0 0      0 0 0 0 0      2 2 2 2 2
3      0 1 0 0      4 0 0 0 0      0 0 0 0 0      2 2 2 2 2
END HYDR-PARM1

```

```

HYDR-PARM2
# - #      FTABNO      LEN      DELTH      STCOR      KS      DB50      ***
<-----><-----><-----><-----><-----><-----><----->      ***
1      1      0.01      0.0      0.0      0.5      0.0
2      2      0.01      0.0      10.0     0.5      0.0
3      3      0.01      0.0      0.0      0.5      0.0

```

END HYDR-PARM2

```

HYDR-INIT
RCHRES Initial conditions for each HYDR section      ***
# - # *** VOL      Initial value of COLIND      Initial value of OUTDGT
      *** ac-ft      for each possible exit      for each possible exit
<-----><----->      <---><---><---><---><--->      *** <---><---><---><---><--->
1      0      4.0 0.0 0.0 0.0 0.0      0.0 0.0 0.0 0.0 0.0
2      0      4.0 5.0 0.0 0.0 0.0      0.0 0.0 0.0 0.0 0.0
3      0      4.0 0.0 0.0 0.0 0.0      0.0 0.0 0.0 0.0 0.0

```

END HYDR-INIT

END RCHRES

SPEC-ACTIONS  
END SPEC-ACTIONS

FTABLES

```

FTABLE      3
92      4
Depth      Area      Volume      Outflow1 Velocity      Travel Time***
(ft)      (acres) (acre-ft) (cfs)      (ft/sec)      (Minutes)***
0.000000  0.109298  0.000000  0.000000
0.066667  0.109298  0.007287  0.001648
0.133333  0.109298  0.014573  0.002331
0.200000  0.109298  0.021860  0.002855
0.266667  0.109298  0.029146  0.003296
0.333333  0.109298  0.036433  0.003685
0.400000  0.109298  0.043719  0.004037
0.466667  0.109298  0.051006  0.004361
0.533333  0.109298  0.058292  0.004662
0.600000  0.109298  0.065579  0.004944
0.666667  0.109298  0.072865  0.005212
0.733333  0.109298  0.080152  0.005466
0.800000  0.109298  0.087438  0.005709
0.866667  0.109298  0.094725  0.005942
0.933333  0.109298  0.102011  0.006167
1.000000  0.109298  0.109298  0.006383
1.066667  0.109298  0.116584  0.006593
1.133333  0.109298  0.123871  0.006795
1.200000  0.109298  0.131157  0.006993
1.266667  0.109298  0.138444  0.007184
1.333333  0.109298  0.145730  0.007371
1.400000  0.109298  0.153017  0.007553
1.466667  0.109298  0.160303  0.007731
1.533333  0.109298  0.167590  0.007904
1.600000  0.109298  0.174876  0.008074
1.666667  0.109298  0.182163  0.008241
1.733333  0.109298  0.189449  0.008404
1.800000  0.109298  0.196736  0.008564
1.866667  0.109298  0.204022  0.008721
1.933333  0.109298  0.211309  0.008876
2.000000  0.109298  0.218595  0.009027
2.066667  0.109298  0.225882  0.009177
2.133333  0.109298  0.233168  0.009323
2.200000  0.109298  0.240455  0.009468
2.266667  0.109298  0.247741  0.009610
2.333333  0.109298  0.255028  0.009751
2.400000  0.109298  0.262314  0.009889
2.466667  0.109298  0.269601  0.010025
2.533333  0.109298  0.276887  0.010160
2.600000  0.109298  0.284174  0.010293
2.666667  0.109298  0.291460  0.010424

```

2.733333	0.109298	0.298747	0.010553
2.800000	0.109298	0.306033	0.010681
2.866667	0.109298	0.313320	0.010808
2.933333	0.109298	0.320606	0.010933
3.000000	0.109298	0.327893	0.011056
3.066667	0.109298	0.335179	0.011178
3.133333	0.109298	0.342466	0.011299
3.200000	0.109298	0.349752	0.011419
3.266667	0.109298	0.357039	0.011537
3.333333	0.109298	0.364325	0.011654
3.400000	0.109298	0.371612	0.011770
3.466667	0.109298	0.378898	0.011885
3.533333	0.109298	0.386185	0.011999
3.600000	0.109298	0.393471	0.012111
3.666667	0.109298	0.400758	0.012705
3.733333	0.109298	0.408044	0.013842
3.800000	0.109298	0.415331	0.015278
3.866667	0.109298	0.422617	0.016933
3.933333	0.109298	0.429904	0.018761
4.000000	0.109298	0.437190	0.020728
4.066667	0.109298	0.444477	0.022809
4.133333	0.109298	0.451763	0.024984
4.200000	0.109298	0.459050	0.027236
4.266667	0.109298	0.466336	0.029550
4.333333	0.109298	0.473623	0.031913
4.400000	0.109298	0.480909	0.034314
4.466667	0.109298	0.488196	0.036741
4.533333	0.109298	0.495482	0.039186
4.600000	0.109298	0.502769	0.041639
4.666667	0.109298	0.510055	0.044536
4.733333	0.109298	0.517342	0.047587
4.800000	0.109298	0.524628	0.050726
4.866667	0.109298	0.531915	0.053950
4.933333	0.109298	0.539201	0.057258
5.000000	0.109298	0.546488	0.060648
5.066667	0.109298	0.553774	0.242976
5.133333	0.109298	0.561061	0.570499
5.200000	0.109298	0.568347	0.968606
5.266667	0.109298	0.575634	1.379103
5.333333	0.109298	0.582920	1.744584
5.400000	0.109298	0.590207	2.021242
5.466667	0.109298	0.597493	2.199625
5.533333	0.109298	0.604780	2.361554
5.600000	0.109298	0.612066	2.501173
5.666667	0.109298	0.619353	2.633232
5.733333	0.109298	0.626639	2.758841
5.800000	0.109298	0.633926	2.878862
5.866667	0.109298	0.641212	2.993981
5.933333	0.109298	0.648499	3.104754
6.000000	0.109298	0.655785	3.211640
6.066667	0.109298	0.663072	3.315020

END FTABLE 3

FTABLE 1

90 5

Depth (ft)	Area (acres)	Volume (acre-ft)	Outflow1 (cfs)	Outflow2 (cfs)	Velocity (ft/sec)	Travel Time*** (Minutes)***
0.000000	0.002296	0.000000	0.082100	0.000000		
0.111111	0.002296	0.000255	0.082100	0.000000		
0.222222	0.002296	0.000510	0.082100	0.000000		
0.333333	0.002296	0.000765	0.082100	0.000000		
0.444444	0.002296	0.001020	0.082100	0.000000		
0.555556	0.002296	0.001275	0.082100	0.000000		
0.666667	0.002296	0.001530	0.082100	11.00000		
0.777778	0.002296	0.001786	0.082100	21.00000		
0.888889	0.002296	0.002041	0.082100	31.00000		
1.000000	0.002296	0.002296	0.082100	41.00000		
1.111111	0.002296	0.002551	0.082100	51.00000		
1.222222	0.002296	0.002806	0.082100	61.00000		
1.333333	0.002296	0.003061	0.082100	71.00000		
1.444444	0.002296	0.003316	0.082100	81.00000		

1.555556	0.002296	0.003571	0.082100	91.00000
1.666667	0.002296	0.003826	0.082100	101.0000
1.777778	0.002296	0.004081	0.082100	111.0000
1.888889	0.002296	0.004336	0.082100	121.0000
2.000000	0.002296	0.004591	0.082100	131.0000
2.111111	0.002296	0.004846	0.082100	141.0000
2.222222	0.002296	0.005102	0.082100	151.0000
2.333333	0.002296	0.005357	0.082100	161.0000
2.444444	0.002296	0.005612	0.082100	171.0000
2.555556	0.002296	0.005867	0.082100	181.0000
2.666667	0.002296	0.006122	0.082100	191.0000
2.777778	0.002296	0.006377	0.082100	201.0000
2.888889	0.002296	0.006632	0.082100	211.0000
3.000000	0.002296	0.006887	0.082100	221.0000
3.111111	0.002296	0.007142	0.082100	231.0000
3.222222	0.002296	0.007397	0.082100	241.0000
3.333333	0.002296	0.007652	0.082100	251.0000
3.444444	0.002296	0.007907	0.082100	261.0000
3.555556	0.002296	0.008162	0.082100	271.0000
3.666667	0.002296	0.008418	0.082100	281.0000
3.777778	0.002296	0.008673	0.082100	291.0000
3.888889	0.002296	0.008928	0.082100	301.0000
4.000000	0.002296	0.009183	0.082100	311.0000
4.111111	0.002296	0.009438	0.082100	321.0000
4.222222	0.002296	0.009693	0.082100	331.0000
4.333333	0.002296	0.009948	0.082100	341.0000
4.444444	0.002296	0.010203	0.082100	351.0000
4.555556	0.002296	0.010458	0.082100	361.0000
4.666667	0.002296	0.010713	0.082100	371.0000
4.777778	0.002296	0.010968	0.082100	381.0000
4.888889	0.002296	0.011223	0.082100	391.0000
5.000000	0.002296	0.011478	0.082100	401.0000
5.111111	0.002296	0.011733	0.082100	411.0000
5.222222	0.002296	0.011989	0.082100	421.0000
5.333333	0.002296	0.012244	0.082100	431.0000
5.444444	0.002296	0.012499	0.082100	441.0000
5.555556	0.002296	0.012754	0.082100	451.0000
5.666667	0.002296	0.013009	0.082100	461.0000
5.777778	0.002296	0.013264	0.082100	471.0000
5.888889	0.002296	0.013519	0.082100	481.0000
6.000000	0.002296	0.013774	0.082100	491.0000
6.111111	0.002296	0.014029	0.082100	501.0000
6.222222	0.002296	0.014284	0.082100	511.0000
6.333333	0.002296	0.014539	0.082100	521.0000
6.444444	0.002296	0.014794	0.082100	531.0000
6.555556	0.002296	0.015049	0.082100	541.0000
6.666667	0.002296	0.015305	0.082100	551.0000
6.777778	0.002296	0.015560	0.082100	561.0000
6.888889	0.002296	0.015815	0.082100	571.0000
7.000000	0.002296	0.016070	0.082100	581.0000
7.111111	0.002296	0.016325	0.082100	591.0000
7.222222	0.002296	0.016580	0.082100	601.0000
7.333333	0.002296	0.016835	0.082100	611.0000
7.444444	0.002296	0.017090	0.082100	621.0000
7.555556	0.002296	0.017345	0.082100	631.0000
7.666667	0.002296	0.017600	0.082100	641.0000
7.777778	0.002296	0.017855	0.082100	651.0000
7.888889	0.002296	0.018110	0.082100	661.0000
8.000000	0.002296	0.018365	0.082100	671.0000
8.111111	0.002296	0.018621	0.082100	681.0000
8.222222	0.002296	0.018876	0.082100	691.0000
8.333333	0.002296	0.019131	0.082100	701.0000
8.444444	0.002296	0.019386	0.082100	711.0000
8.555556	0.002296	0.019641	0.082100	721.0000
8.666667	0.002296	0.019896	0.082100	731.0000
8.777778	0.002296	0.020151	0.082100	741.0000
8.888889	0.002296	0.020406	0.082100	751.0000
9.000000	0.002296	0.020661	0.082100	761.0000
9.111111	0.002296	0.020916	0.082100	771.0000
9.222222	0.002296	0.021171	0.082100	781.0000

```

9.333333 0.002296 0.021426 0.082100 791.0000
9.444444 0.002296 0.021681 0.082100 801.0000
9.555556 0.002296 0.021937 0.082100 811.0000
9.666667 0.002296 0.022192 0.082100 821.0000
9.777778 0.002296 0.022447 0.082100 831.0000
9.888889 0.002296 0.022702 0.082100 841.0000

```

```

END FTABLE 1
FTABLE 2
36 5

```

Depth (ft)	Area (acres)	Volume (acre-ft)	Outflow1 (cfs)	Outflow2 (cfs)	Velocity (ft/sec)	Travel Time*** (Minutes)***
0.000000	0.000735	0.000000	0.000000	0.000000		
0.100000	0.000735	0.000086	0.000000	0.006541		
0.200000	0.000735	0.000258	0.000000	0.013083		
0.300000	0.000735	0.000515	0.000000	0.019624		
0.400000	0.000735	0.000859	0.000000	0.026166		
0.500000	0.000735	0.001289	0.000000	0.032707		
0.600000	0.000735	0.001804	0.000000	0.039248		
0.700000	0.000735	0.002405	0.000000	0.045790		
0.800000	0.000735	0.003093	0.000000	0.052331		
0.900000	0.000735	0.003866	0.000000	0.058873		
1.000000	0.000735	0.004725	0.000000	0.065414		
1.100000	0.000735	0.005670	0.000000	0.071955		
1.200000	0.000735	0.006701	0.000000	0.078497		
1.300000	0.000735	0.007817	0.000000	0.085038		
1.400000	0.000735	0.009020	0.000000	0.091580		
1.500000	0.000735	0.010309	0.000000	0.098121		
1.600000	0.000735	0.011683	0.000000	0.104663		
1.700000	0.000735	0.013143	0.000000	0.111204		
1.800000	0.000735	0.014690	0.000000	0.117745		
1.900000	0.000735	0.016322	0.000000	0.124287		
2.000000	0.000735	0.018040	0.000000	0.130828		
2.100000	0.000735	0.019844	0.000000	0.137370		
2.200000	0.000735	0.021734	0.000000	0.143911		
2.300000	0.000735	0.023710	0.000000	0.150452		
2.400000	0.000735	0.025771	0.000000	0.156994		
2.500000	0.000735	0.027919	0.000000	0.163535		
2.600000	0.000735	0.030152	0.000000	0.170077		
2.700000	0.000735	0.032472	0.000000	0.176618		
2.800000	0.000735	0.034877	0.000000	0.183159		
2.900000	0.000735	0.037368	0.000000	0.189701		
3.000000	0.000735	0.039946	0.000000	0.196242		
3.100000	0.000735	0.042609	0.000000	0.202784		
3.200000	0.000735	0.045358	0.000000	0.209325		
3.300000	0.000735	0.048192	0.000000	0.215866		
3.400000	0.000735	0.051113	0.000000	0.222408		
4.400000	0.000735	0.054120	6.000000	0.222408		

```

END FTABLE 2
END FTABLES

```

EXT SOURCES

```

<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member-> ***
<Name> # <Name> # tem strg<-factor->strg <Name> # # <Name> # # ***
WDM 2 PREC ENGL 1 PERLND 1 999 EXTNL PREC
WDM 2 PREC ENGL 1 IMPLND 1 999 EXTNL PREC
WDM 1 EVAP ENGL 1 PERLND 1 999 EXTNL PETINP
WDM 1 EVAP ENGL 1 IMPLND 1 999 EXTNL PETINP

```

END EXT SOURCES

EXT TARGETS

```

<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name> # <Name> # #<-factor->strg <Name> # <Name> tem strg strg***
RCHRES 3 HYDR RO 1 1 1 WDM 1000 FLOW ENGL REPL
RCHRES 3 HYDR STAGE 1 1 1 WDM 1001 STAG ENGL REPL
COPY 1 OUTPUT MEAN 1 1 48.4 WDM 701 FLOW ENGL REPL
COPY 501 OUTPUT MEAN 1 1 48.4 WDM 801 FLOW ENGL REPL
COPY 601 OUTPUT MEAN 1 1 48.4 WDM 901 FLOW ENGL REPL

```

END EXT TARGETS



```

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->   <Target>   <-Grp> <-Member->***
<Name>     <Name> # #<-factor->   <Name>     <Name> # #***
  MASS-LINK          5
IMPLND      IWATER  SURO          0.083333   RCHRES      INFLOW  IVOL
  END MASS-LINK     5

  MASS-LINK          7
RCHRES      OFLOW   OVOL    1           RCHRES      INFLOW  IVOL
  END MASS-LINK     7

  MASS-LINK          8
RCHRES      OFLOW   OVOL    2           RCHRES      INFLOW  IVOL
  END MASS-LINK     8

  MASS-LINK          16
RCHRES      ROFLOW                    COPY        INPUT  MEAN
  END MASS-LINK     16

  MASS-LINK          17
RCHRES      OFLOW   OVOL    1           COPY        INPUT  MEAN
  END MASS-LINK     17

  MASS-LINK          18
RCHRES      OFLOW   OVOL    2           COPY        INPUT  MEAN
  END MASS-LINK     18

END MASS-LINK

END RUN

```

*Predeveloped HSPF Message File*

## Mitigated HSPF Message File

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1929/ 8/31 24: 0

RCHRES : 3

RELERR	STORS	STOR	MATIN	MATDIF
-0.01257	0.00000	0.0000E+00	0.00000	-1.665E-08

Where:

RELERR is the relative error (ERROR/REFVAL).

ERROR is (STOR-STORS) - MATDIF.

REFVAL is the reference value (STORS+MATIN).

STOR is the storage of material in the processing unit (land-segment or reach/reservoir) at the end of the present interval.

STORS is the storage of material in the pu at the start of the present printout reporting period.

MATIN is the total inflow of material to the pu during the present printout reporting period.

MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1955/ 9/30 24: 0

RCHRES : 3

RELERR	STORS	STOR	MATIN	MATDIF
-1.866E-02	0.00000	0.0000E+00	0.00000	-1.121E-08

Where:

RELERR is the relative error (ERROR/REFVAL).

ERROR is (STOR-STORS) - MATDIF.

REFVAL is the reference value (STORS+MATIN).

STOR is the storage of material in the processing unit (land-segment or reach/reservoir) at the end of the present interval.

STORS is the storage of material in the pu at the start of the present printout reporting period.

MATIN is the total inflow of material to the pu during the present printout reporting period.

MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1960/ 8/31 24: 0

RCHRES : 3

RELERR	STORS	STOR	MATIN	MATDIF
-6.434E-02	0.00000	0.0000E+00	0.00000	-3.095E-09

Where:

RELERR is the relative error (ERROR/REFVAL).  
ERROR is (STOR-STORS) - MATDIF.  
REFVAL is the reference value (STORS+MATIN).  
STOR is the storage of material in the processing unit (land-segment or reach/reservior) at the end of the present interval.  
STORS is the storage of material in the pu at the start of the present printout reporting period.  
MATIN is the total inflow of material to the pu during the present printout reporting period.  
MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1980/ 8/31 24: 0

RCHRES : 3

RELERR	STORS	STOR	MATIN	MATDIF
-1.326E-03	0.00000	0.0000E+00	0.00000	-1.601E-07

Where:

RELERR is the relative error (ERROR/REFVAL).  
ERROR is (STOR-STORS) - MATDIF.  
REFVAL is the reference value (STORS+MATIN).  
STOR is the storage of material in the processing unit (land-segment or reach/reservior) at the end of the present interval.  
STORS is the storage of material in the pu at the start of the present printout reporting period.  
MATIN is the total inflow of material to the pu during the present printout reporting period.  
MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

ERROR/WARNING ID: 238 1

The continuity error reported below is greater than 1 part in 1000 and is therefore considered high.

Did you specify any "special actions"? If so, they could account for it.

Relevant data are:

DATE/TIME: 1989/ 8/31 24: 0

RCHRES : 3

RELERR	STORS	STOR	MATIN	MATDIF
-9.459E-02	0.00000	0.0000E+00	0.00000	-2.030E-09

Where:

RELERR is the relative error (ERROR/REFVAL).  
ERROR is (STOR-STORS) - MATDIF.

REFVAL is the reference value (STORS+MATIN).

STOR is the storage of material in the processing unit (land-segment or reach/reservoir) at the end of the present interval.

STORS is the storage of material in the pu at the start of the present printout reporting period.

MATIN is the total inflow of material to the pu during the present printout reporting period.

MATDIF is the net inflow (inflow-outflow) of material to the pu during the present printout reporting period.

---

The count for the WARNING printed above has reached its maximum.

If the condition is encountered again the message will not be repeated.

---

## *Disclaimer*

### *Legal Notice*

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# Tab 6.0



## 6.0 CONSTRUCTION STORMWATER POLLUTION PREVENTION PLAN

A SWPPP has been prepared and submitted with this project under a separate cover. The following is a list of the thirteen SWPPP elements and how they have been addressed for this project:

**Element No. 1 - Preserve Vegetation / Mark Clearing Limits:** Clearing Limits will be delineated on the engineering plans and will be flagged in the field.

**Element No. 2 - Establish Construction Access:** A stabilized gravel construction entrance will be shown on the engineering plans.

**Element No. 3 - Control Flow Rates:** A temporary sediment pond will be shown on the engineering plans. Once the permanent infiltration facilities are constructed the temporary sediment ponds can be removed. The permanent facilities can be used throughout the remainder of construction.

**Element No. 4 - Install Sediment Controls:** Silt fence will be shown on the engineering plans for perimeter protection. In addition, temporary ditches to divert runoff to the sediment pond will be shown on the engineering plans.

**Element No. 5 - Stabilize Soils:** Cover measures will be addressed in the TESC notes on the engineering plans.

**Element No. 6 - Protect Slopes:** Runoff will be diverted away from the site steep slopes during and after construction. Any erosion on the site steep slopes will be rectified immediately.

**Element No. 7 - Protect Permanent Drain Inlets:** A detail for catch basin inserts will be shown on the final engineering plans along with a note specifying that they be installed once the permanent storm system is completed. A note will also be included that the contractor shall keep public roadways clear of dirt and debris.

**Element No. 8 - Stabilize Channels and Outlets:** Notes regarding outfall protection will be shown on the engineering plans. Temporary ditches shall be armored with rip rap for slopes greater than 5 percent.

**Element No. 9 - Control Pollutants:** A note will be added to the engineering plans that the contractor shall dispose of all pollutants and waste materials in a safe and timely manner.

**Element No. 10 - Control Dewatering:** Groundwater removed from dewatering activity shall be discharged from the site in accordance with the requirements of the City, and Washington State DOE.

**Element No. 11 - Maintain Best Management Practices** Once the engineering plans are completed the contractor shall maintain all erosion control measures in accordance with Snohomish County Standards and manufactures recommendations. In addition, the contractor shall maintain a stockpile of erosion control materials onsite.

**Element No. 12 - Manage the Project:** Once the engineering plans are completed, the clearing, grading, and seasonal work shall be performed in accordance with Snohomish County Code. The contractor shall inspect, maintain, and repair all BMPs as needed to assure continued performance of their intended function. In addition to the engineering plans the contractor will be required to follow and maintain the Construction SWPPP which has been prepared according to Department of Ecology NPDES requirements. The completed SWPPP and TESC Plans will be provided during Final Engineering Review.



**Element No. 13 – Protect Low Impact Development BMPs:** LID BMPs are not feasible for this project, this element does not appear to apply to the project site.

# Tab 7.0



## **7.0 SPECIAL REPORTS AND STUDIES**

Draft Geotechnical Engineering Investigation by Krazan & Associates, Inc, dated May 6, 2022, shall be provided within this report as Appendix A.

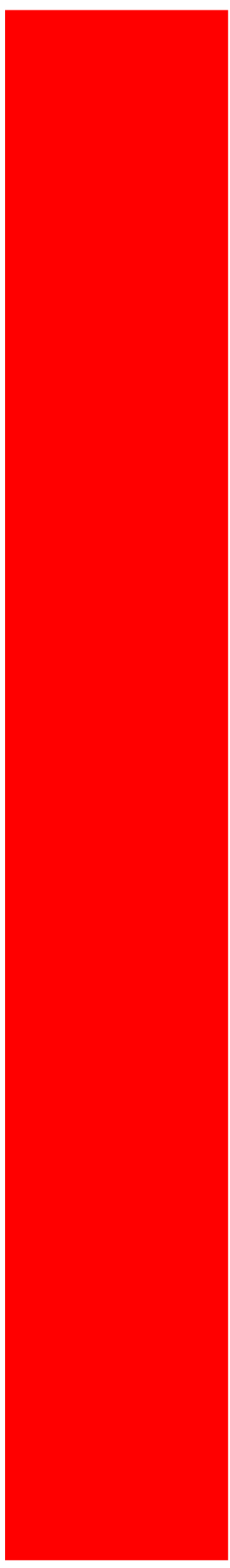
# Tab 8.0



## **8.0 OTHER PERMITS**

Other permits associated with this project shall be included within the final report.

# Tab 9.0



## **9.0 OPERATIONS AND MAINTENANCE MANUAL**

An Operations and Maintenance Manual shall be included within the final report.

# Tab 10.0

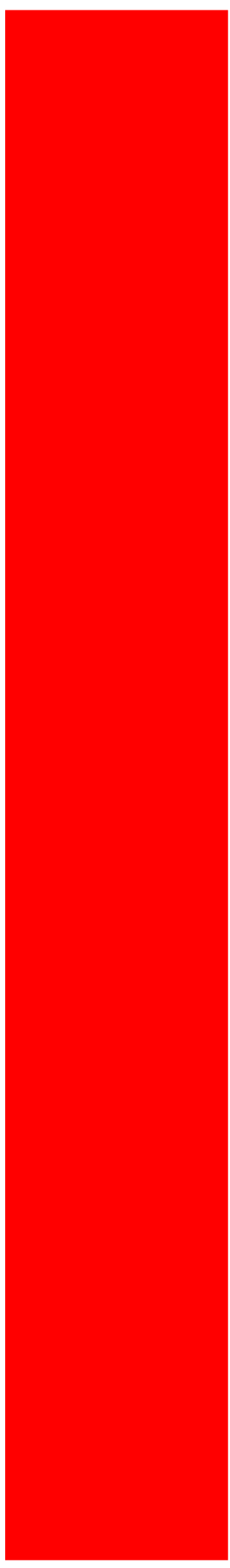




## **10.0 DECLARATION OF COVENANT FOR PRIVATELY MAINTAINED FLOW CONTROL AND TREATMENT FACILITIES**

A declaration of covenant for privately maintained flow control and treatment facilities is not listed within the requirements of Puyallup's drainage report requirements.

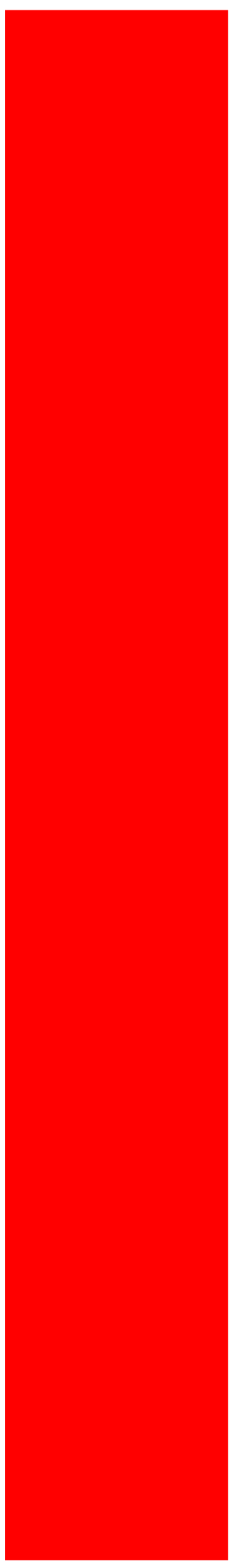
# Tab 11.0



**11.0 DECLARATION OF COVENANT FOR PRIVATELY MAINTAINED ON-SITE STORMWATER MANAGEMENT BMPS**

A declaration of covenant for privately maintained on-site stormwater management BMPs is not listed within the requirements of Puyallup's drainage report requirements.

# Tab 12.0

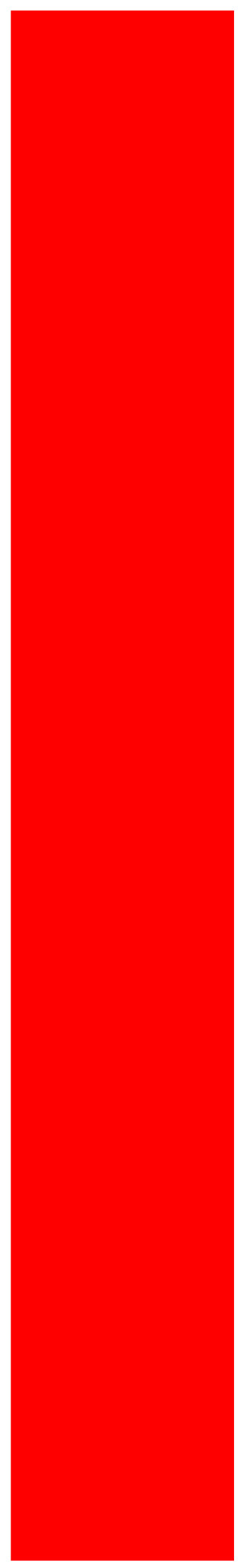


## **12.0 BOND QUANTITIES WORKSHEET**

A completed Bond Quantities Worksheet shall be included within the final report.

# Appendix A

## Draft Geotechnical Report



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

**PROJECT NO. 062-22010  
MAY 6, 2022**

**Prepared for:**

**BP PRODUCTS NORTH AMERICA, INC.  
30 SOUTH WACKER DRIVE, SUITE 900  
CHICAGO, IL 60606**

**Prepared by:**

**KRAZAN & ASSOCIATES, INC.  
GEOTECHNICAL ENGINEERING DIVISION  
825 CENTER STREET, STE A  
TACOMA, WASHINGTON 98409  
(253) 939-2500**

# **Krazan** & ASSOCIATES, INC.

---

GEOTECHNICAL ENGINEERING • ENVIRONMENTAL ENGINEERING  
CONSTRUCTION TESTING & INSPECTION

May 6, 2022

KA Project No. 062-22010

**BP Products North America Inc.**  
30 South Wacker Drive, Suite 900  
Chicago, IL 60606

**Attn: Mr. Randall Arnold**  
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  
Project Manager



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May 6, 2022

KA Project No. 062-22010

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

**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 Kraزان 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 ratio, 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. We have assumed that the structure will have a fundamental period of vibration of equal to or less than 0.5 seconds. 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  $S_I$ ) 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 ( $S_{MS}$ ,  $S_{DS}$ ,  $S_{MI}$ ,  $S_{DI}$ ) and short period ( $T_s$ ) 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
$T_s$	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-induced 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.

<b>Table 2 - SOIL PARAMETERS FOR TEMPORARY SHORING DESIGN</b>						
<b>Material Description</b>	<b>Depth (ft.)</b>	<b>Angle of Internal Friction (degrees)</b>	<b>Cohesion (psf)</b>	<b>Moist Unit Weight (pcf)</b>	<b>Active Earth Pressure Coefficient (K<sub>a</sub>)</b>	<b>Passive Earth Pressure Coefficient (K<sub>p</sub>)</b>
<b>Soil Layer 1:</b> very loose to medium dense Sands	0 - 22	22	0	105	0.45	2.20
<b>Soil Layer 2 (Native Soils):</b> 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 to 1 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

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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  $\pm 2$  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  $\frac{1}{4}$  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**

<b>PILE DIAMETER</b>  <b>(Inches)</b>	<b>ALLOWABLE PILE CAPACITY</b>  <b>(Kips)</b>
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) for granular structural fill 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.

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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  $\frac{3}{4}$ -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 stress 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 dissipation 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**

<b>Asphaltic Concrete</b>	<b>Aggregate Base*</b>
3.0 in.	6.0 in.

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

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<b>Min. PCC Depth</b>	<b>Aggregate Base*</b>
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 overruns. 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  
Project Manager

Vijay Chaudhary, P.E.  
Assistant Regional Engineering Manager

DRAFT



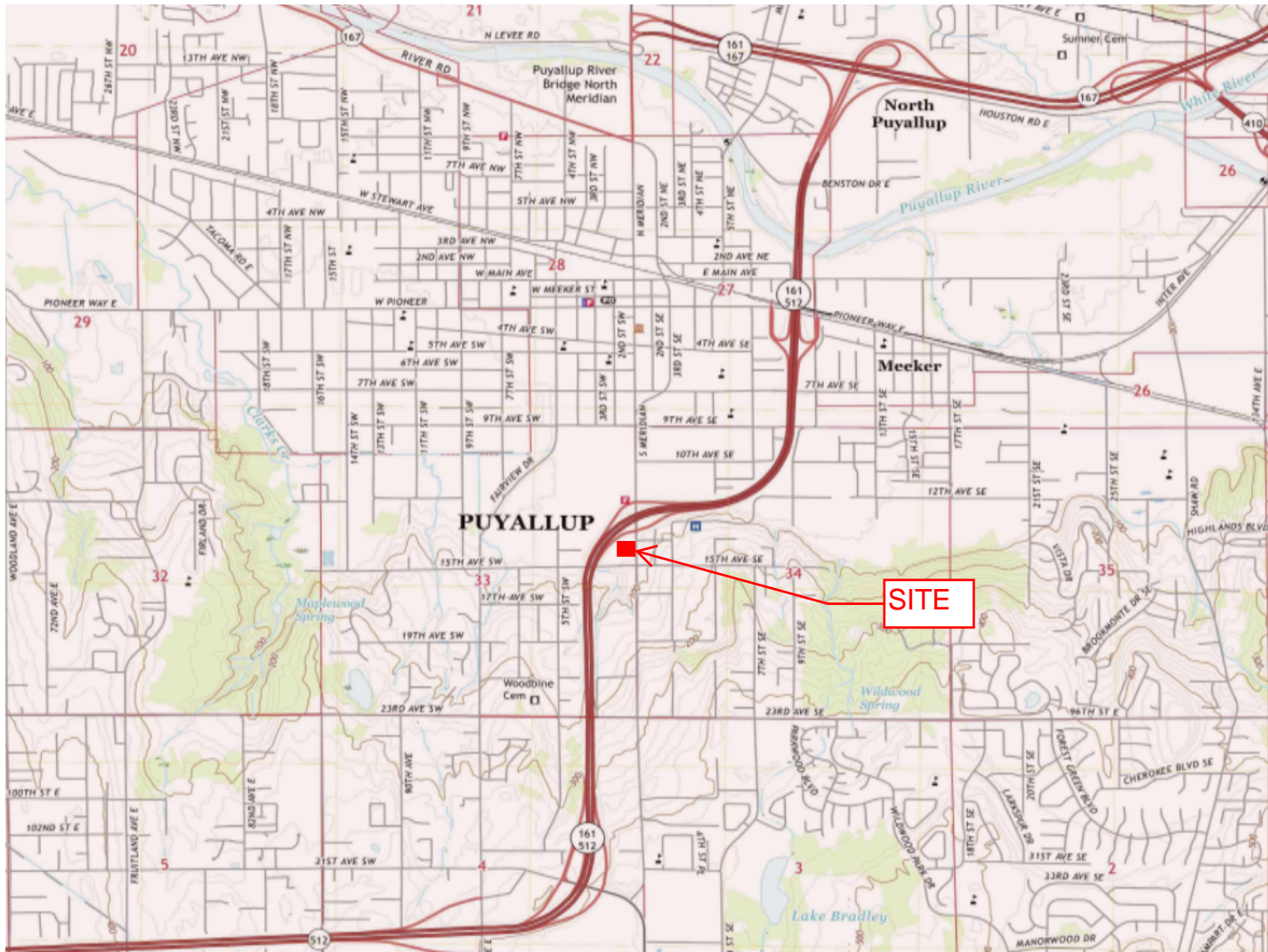



Figure 1: Vicinity Map

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

		
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

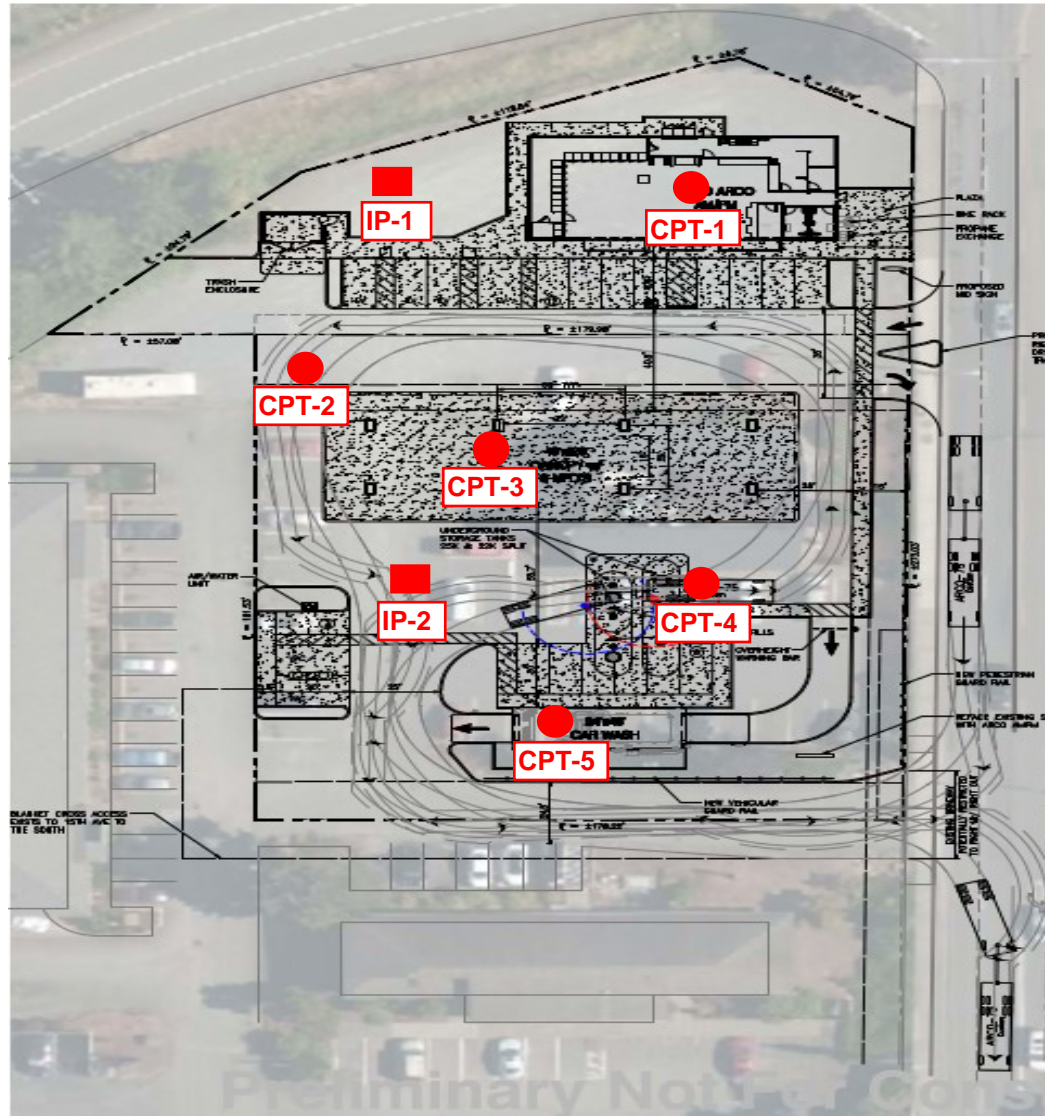


Figure 2: Site Plan

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.

		
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.

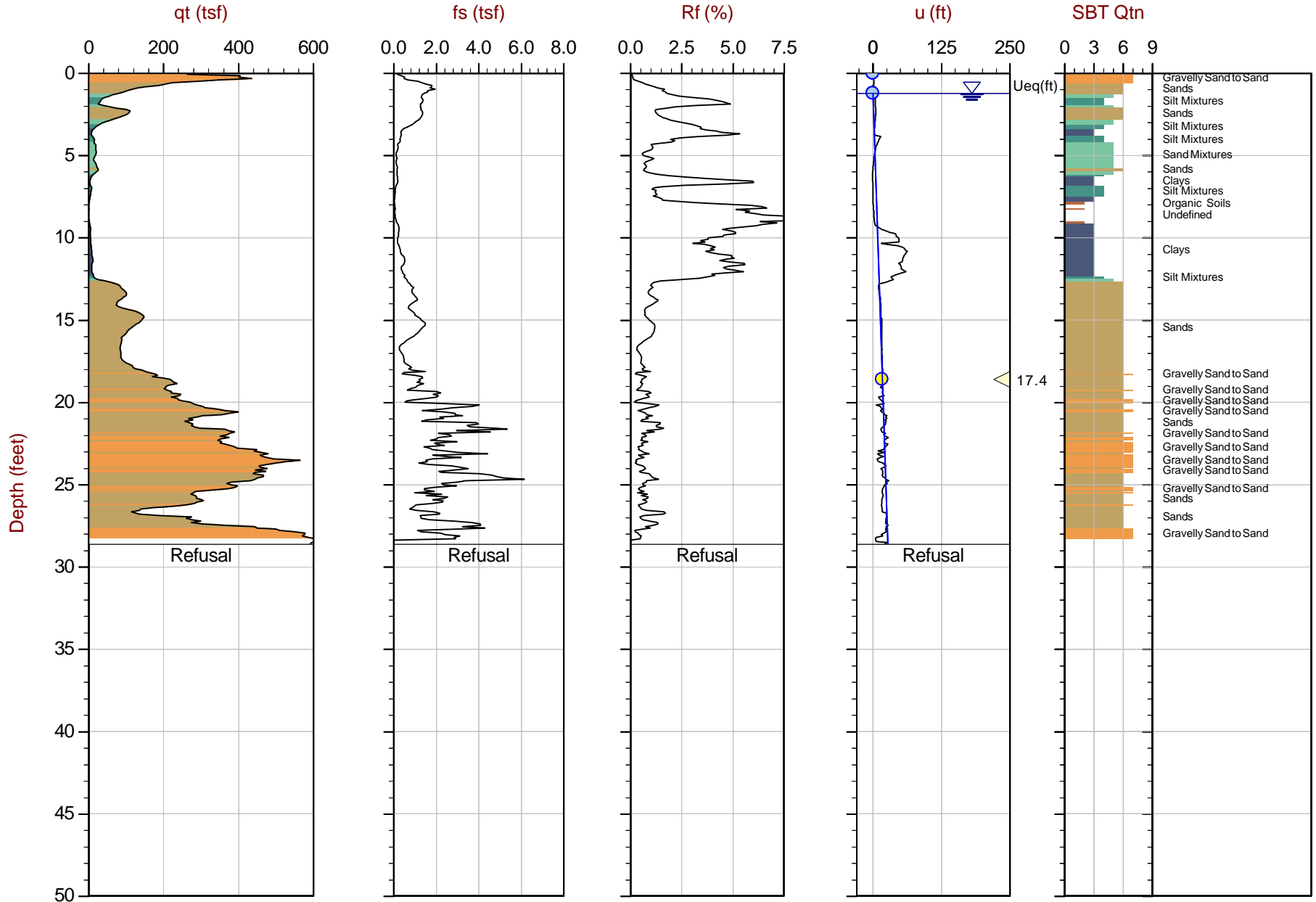
The test pit and CPT explorations were located in the field based on existing site features, and their approximate locations are shown on the Site Plan (Figure 2). The test pit and CPT logs are presented in this Appendix. The depths shown on the attached logs are from the existing ground surface at the 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.

<b>Project:</b> Prop. ARCO ampm Fueling Facility		<b>Project Number:</b> 062-22010		<b>Client:</b> BP Products North America		<b>Test Pit No.:</b>		<b>IP-1</b>	
<b>Location:</b> 1402 S. Meridian Ave., Puyallup, WA						<b>Contractor:</b> Strickland & Sons Excavation			
<b>Project Manager:</b> Theresa Nunan			<b>Date</b>	<b>Started:</b> 3.28.22		<b>Equipment:</b> CAT 306 Excavator with WW			
<b>Field Engineer:</b> Theresa Nunan				<b>Completed:</b> 3.28.22					
<b>Groundwater Depth:</b> 5.8 +/- feet (and rising)				<b>Ground Elevation:</b> 46.0 +/- feet		<b>Total Depth of Test Pit:</b> 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)		
	1						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		
	2						(FILL)		
	3	BULK	S-1				Dark Brown MUCK (Organic Silt and Clay with some fibrous organics), very soft, moist		
	4						(SHALCAR MUCK)		
	5	BULK	S-2				Dark Grey SAND (SP-SM) with silt, fine grained sand, very loose, moist to wet		
	6								
	7						(ALLUVIUM)		
	8						Test Pit Terminated at 7.1 Feet		

<b>Project:</b> Prop. ARCO ampm Fueling Facility		<b>Project Number:</b> 062-22010		<b>Client:</b> BP Products North America		<b>Test Pit No.:</b>		<b>IP-2</b>	
<b>Location:</b> 1402 S. Meridian Ave., Puyallup, WA						<b>Contractor:</b> Strickland & Sons Excavation			
<b>Project Manager:</b> Theresa Nunan			<b>Date</b>	<b>Started:</b> 3.28.22		<b>Equipment:</b> CAT 306 Excavator with WW			
<b>Field Engineer:</b> Theresa Nunan				<b>Completed:</b> 3.28.22					
<b>Groundwater Depth:</b> 2.3 +/- feet (and rising)				<b>Ground Elevation:</b> 47.0		<b>Total Depth of Test Pit:</b> 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 <b>Asphalt Concrete Pavement</b> underlain by 6 inches of Brown Silty SAND (SM) with gravel <b>BASE COURSE</b>			
	1					Tan <b>Poorly Graded SAND (SP-SM)</b> with silt, fine to medium grained, loose, moist  <b>(FILL)</b>			
	2	BULK	S-1			Bluish Grey <b>SAND (SP-SM)</b> with silt, very loose, wet  <b>(ALLUVIUM)</b>			
	3	BULK	S-2			Dark Brown <b>Organic SILT (OH)</b> , very soft, wet  <b>(SHALCAR MUCK)</b>			
	4	BULK	S-3			Dark Grey/Black <b>Poorly Graded SAND (SP-SM)</b> with silt, very loose, wet  <b>(ALLUVIUM)</b>			
	5					Test Pit Terminated at 4.7 Feet			
	6								
	7								
	8								



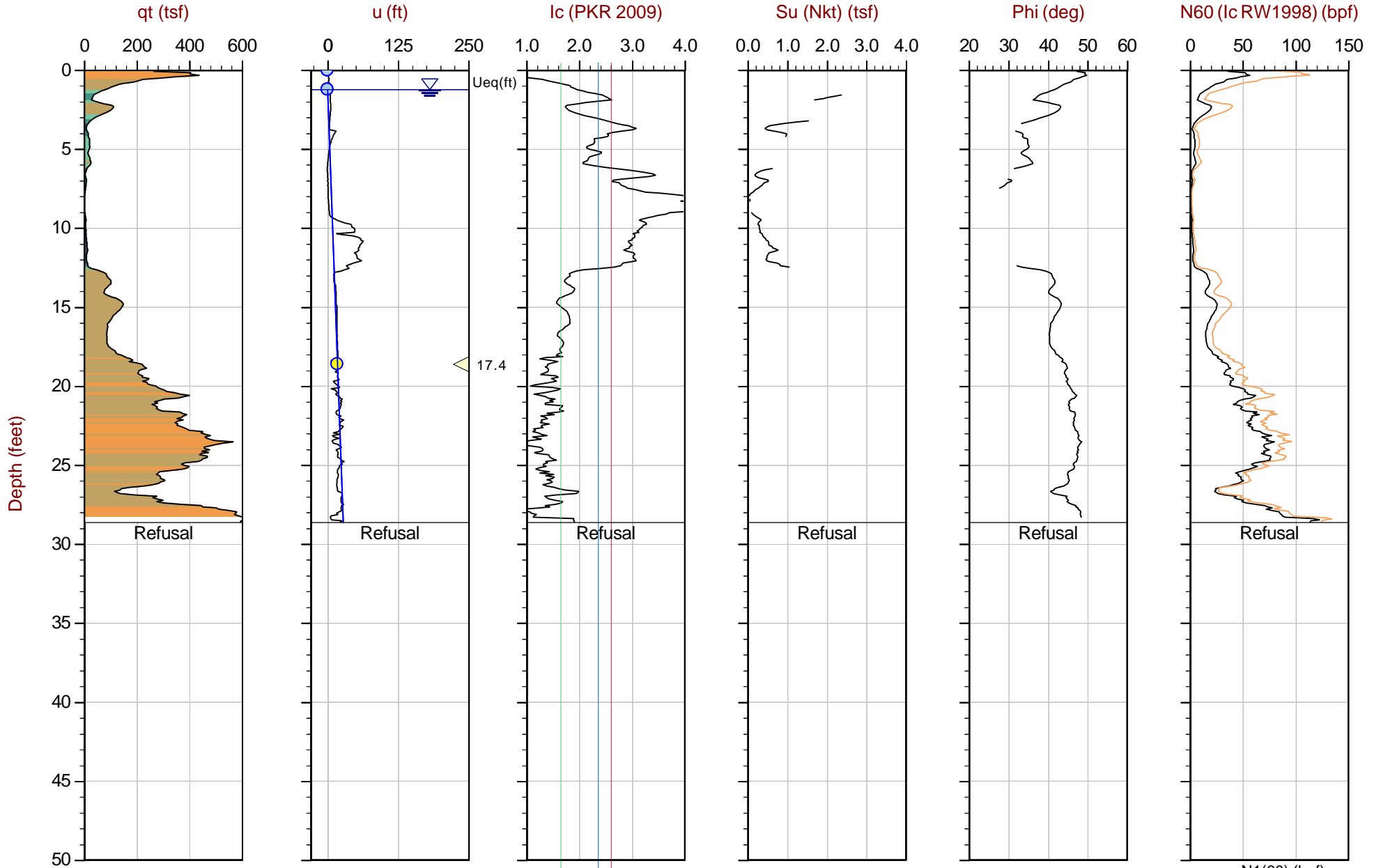
Max Depth: 8.725 m / 28.62 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.





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

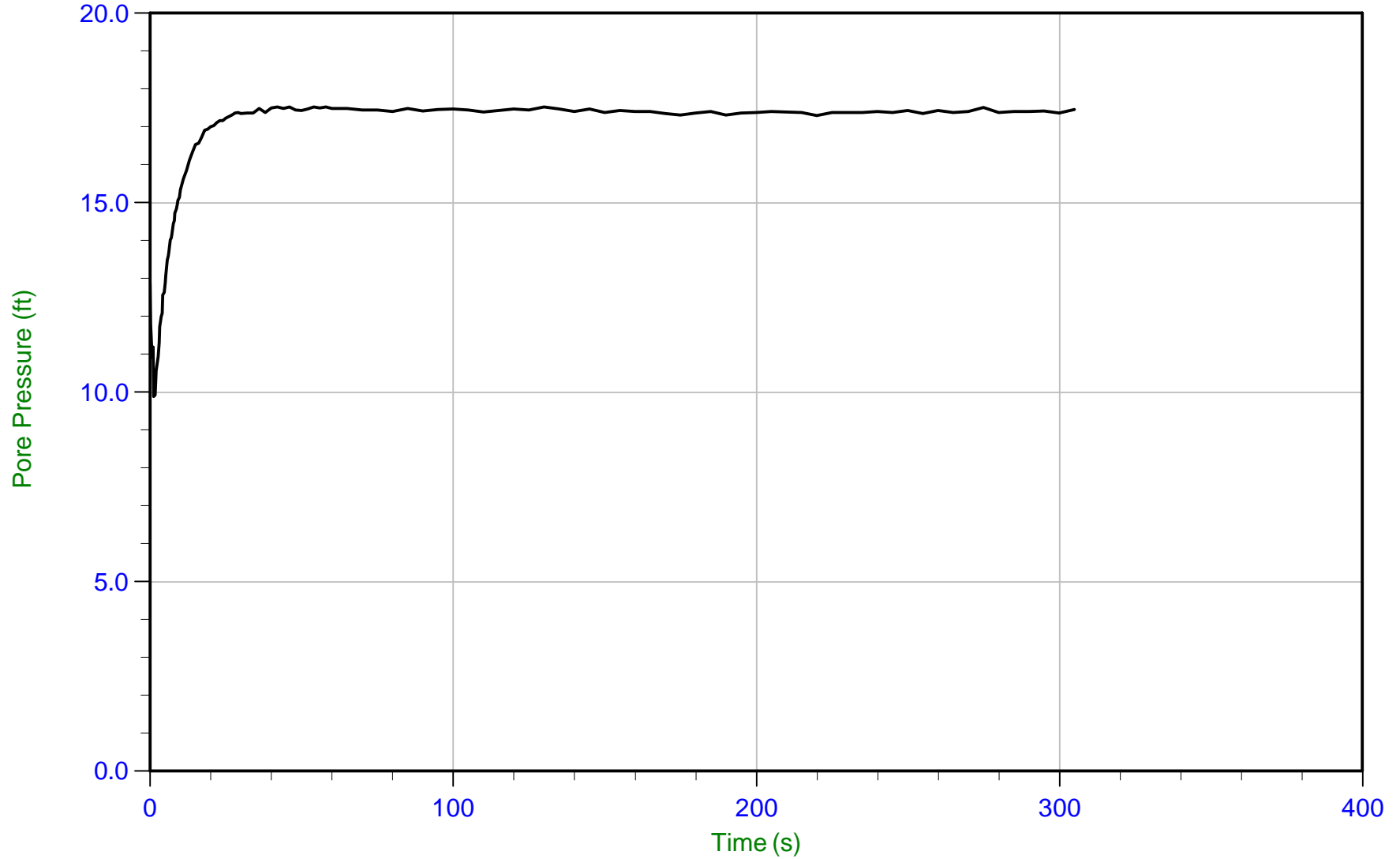
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-01  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>



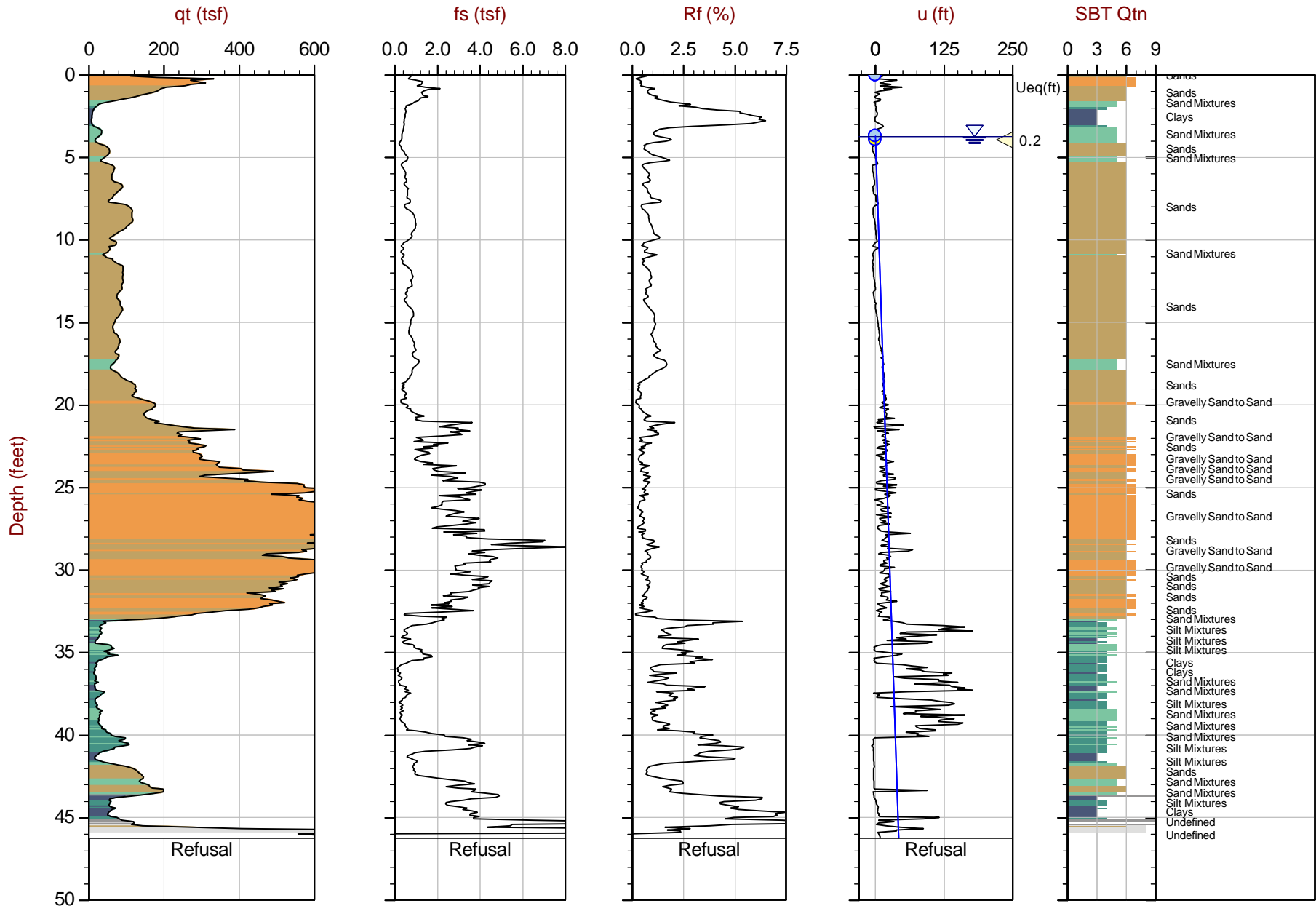
Trace Summary:

Filename: 22-59-23911\_CP01.ppd2  
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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





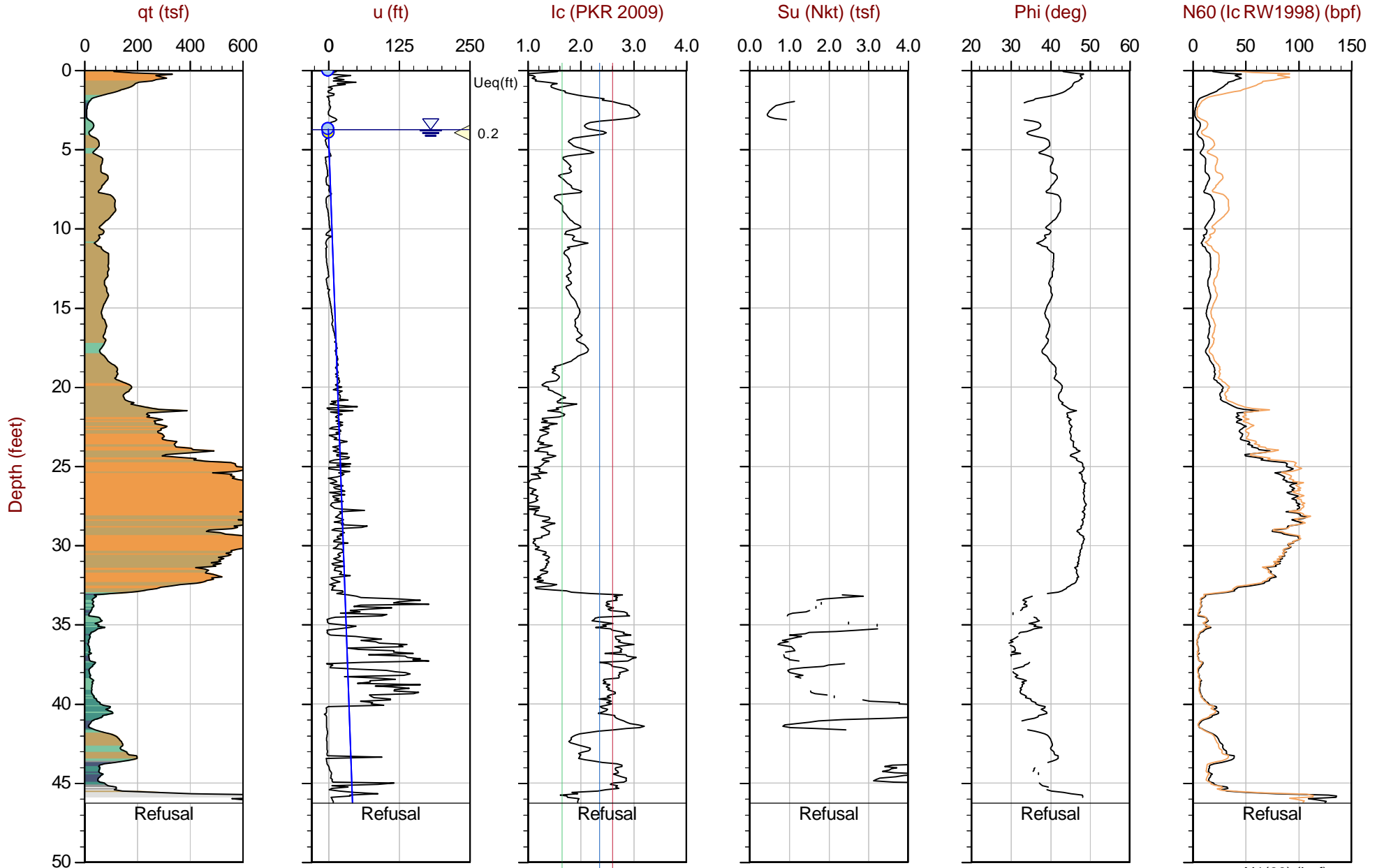
Max Depth: 14.100 m / 46.26 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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: SBTQn(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

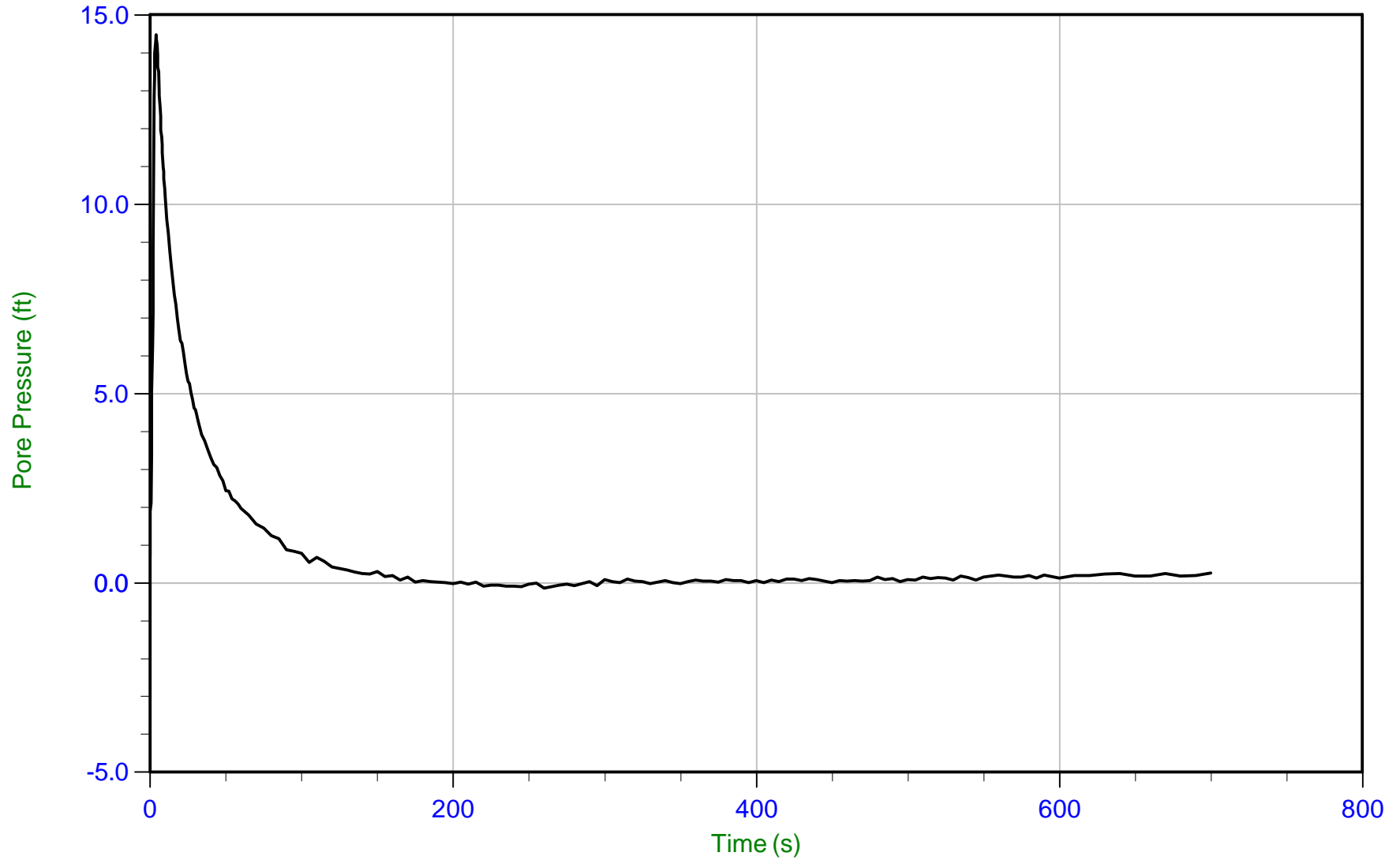
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-02B  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 22-59-23911\_SP02B.ppd2  
Depth: 1.200 m / 3.937 ft  
Duration: 700.0 s

u Min: -0.1 ft  
u Max: 14.5 ft  
u Final: 0.3 ft

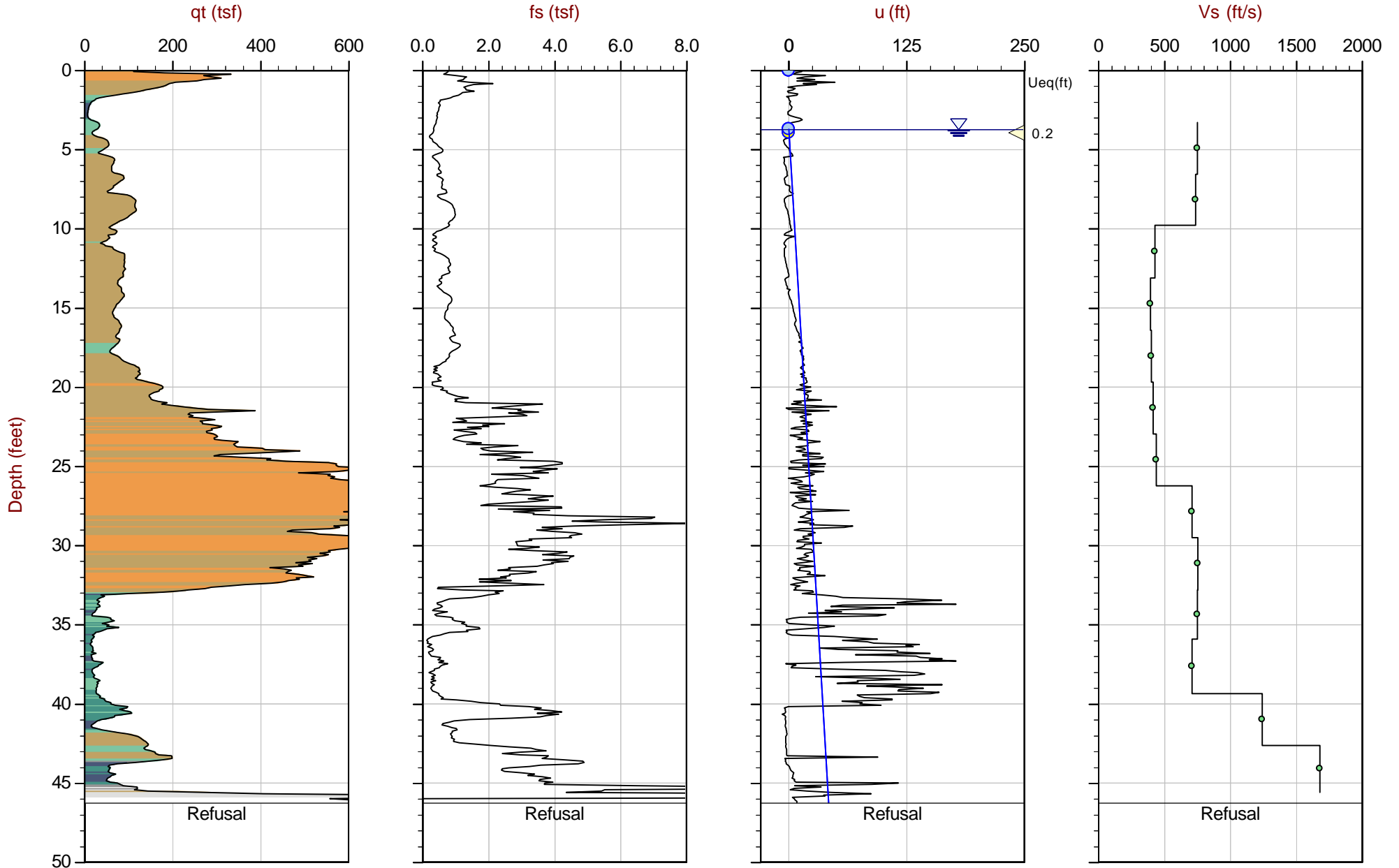
WT: 1.134 m / 3.720 ft  
Ueq: 0.2 ft



Krazan

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

Sounding: CPT-02B  
Cone: 781:T1500F15U35



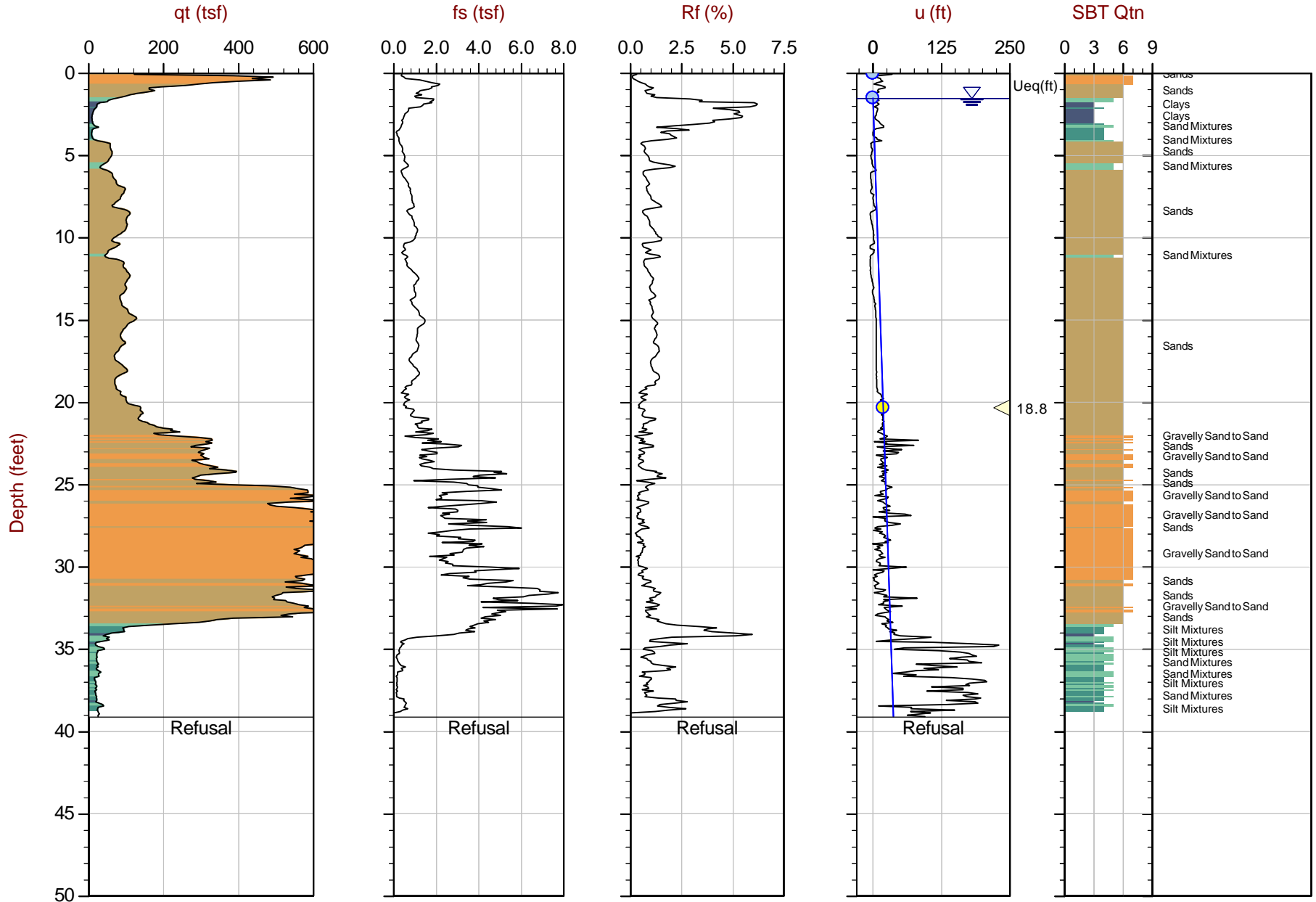
Max Depth: 14.100 m / 46.26 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

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.



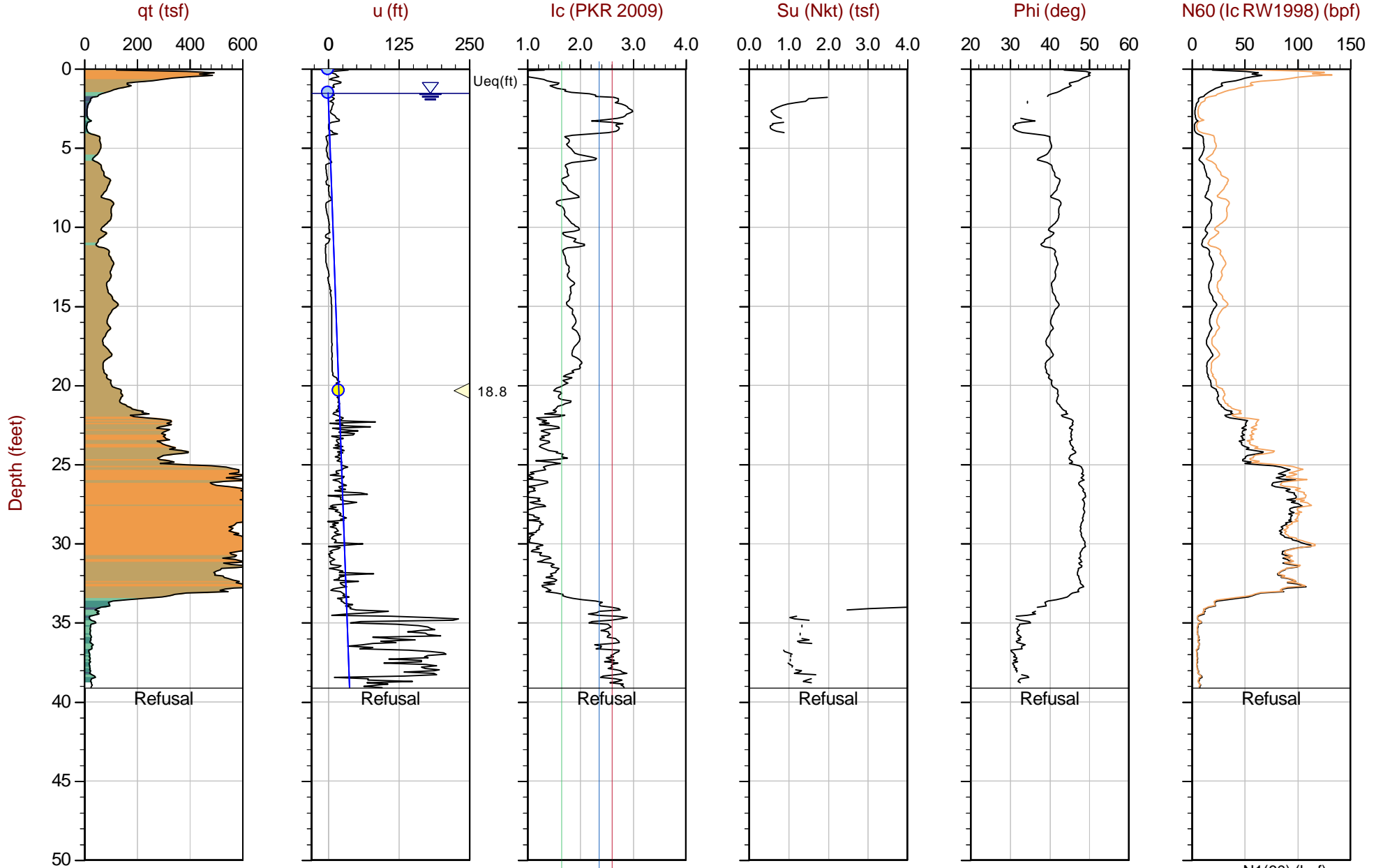
Max Depth: 11.925 m / 39.12 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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

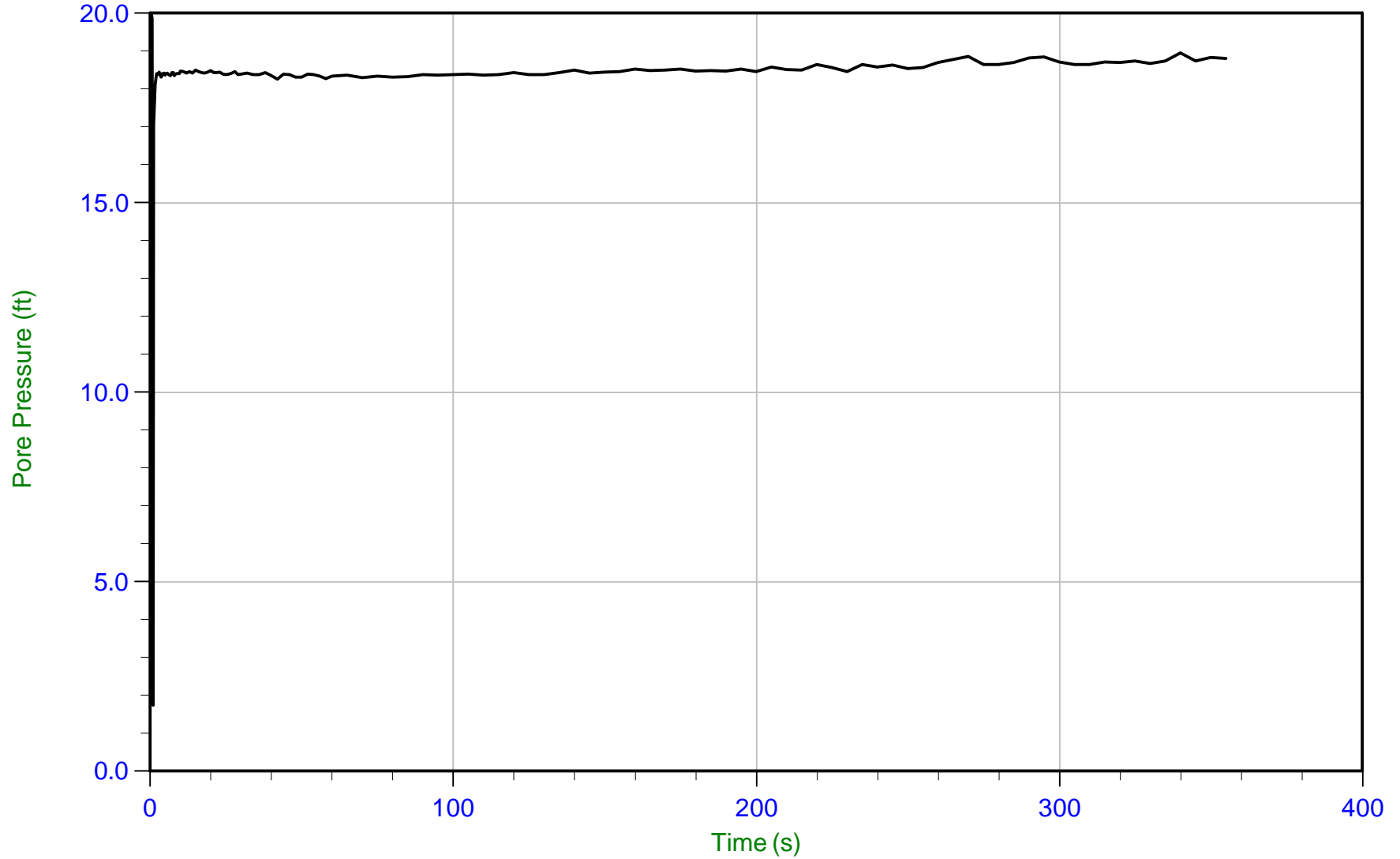
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-02  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

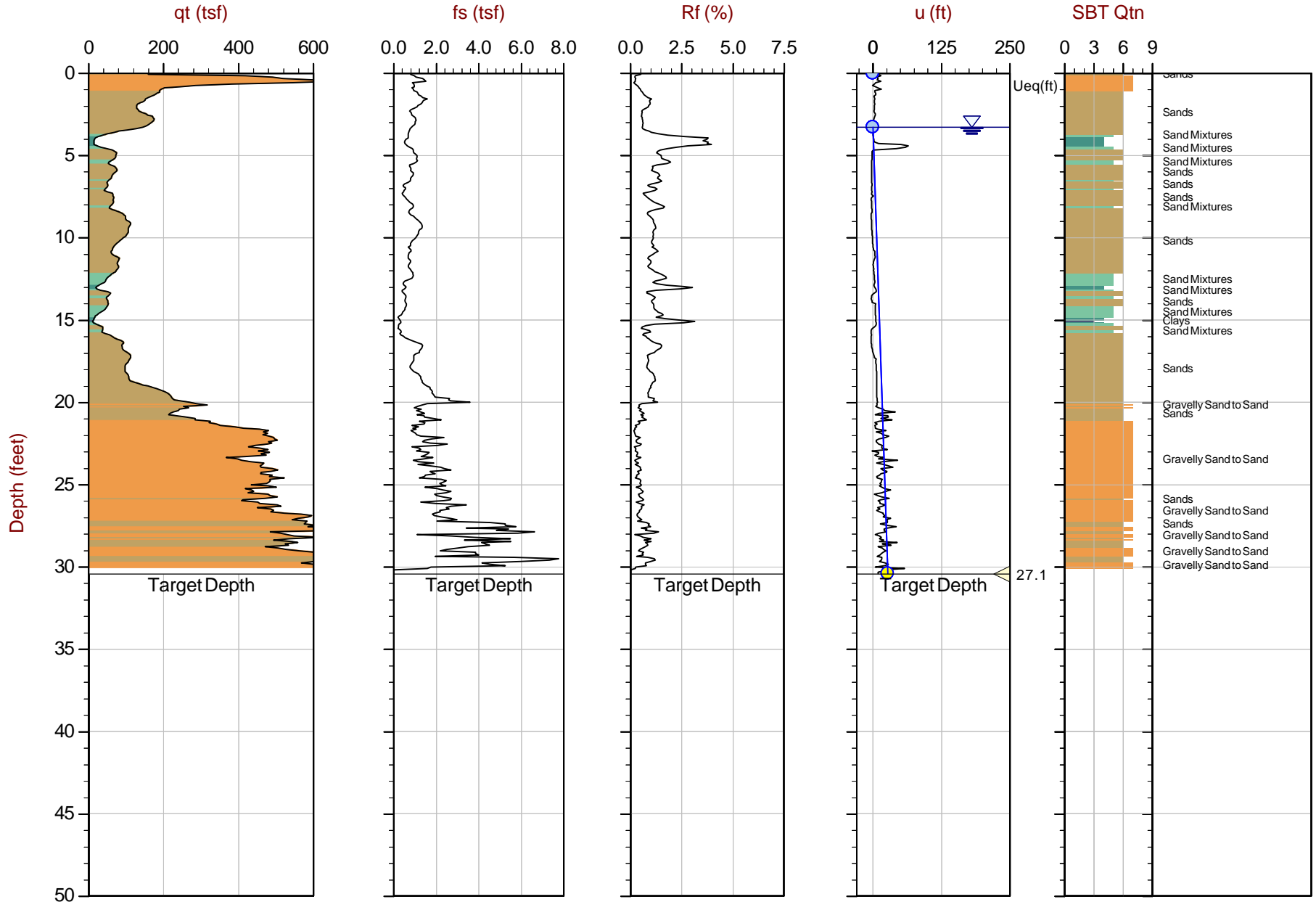


Trace Summary:

Filename: 22-59-23911\_CP02.ppd2  
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



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

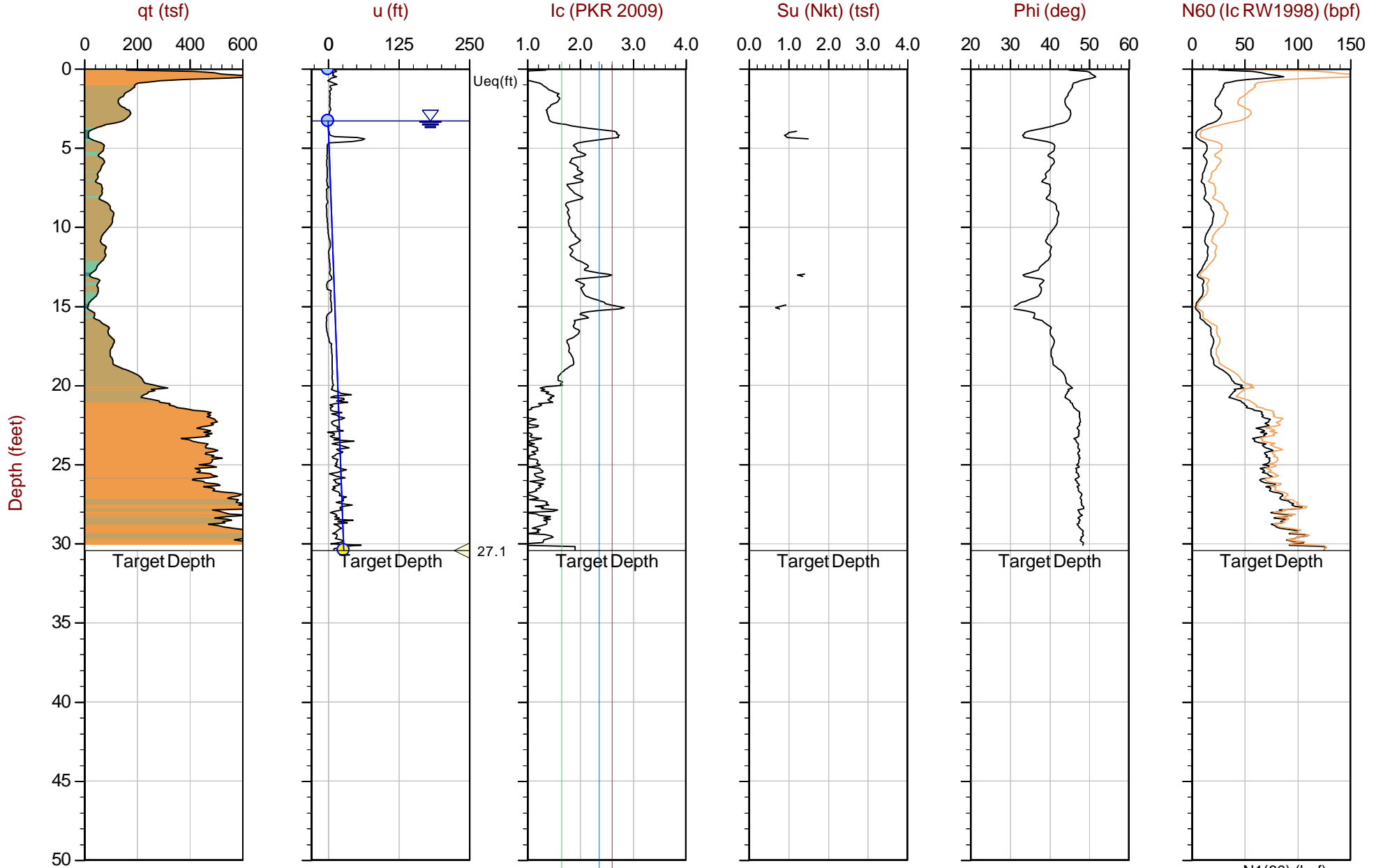
File: 22-59-23911\_CP03.COR  
 Unit Wt: SBTQtn(PKR2009)

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.





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

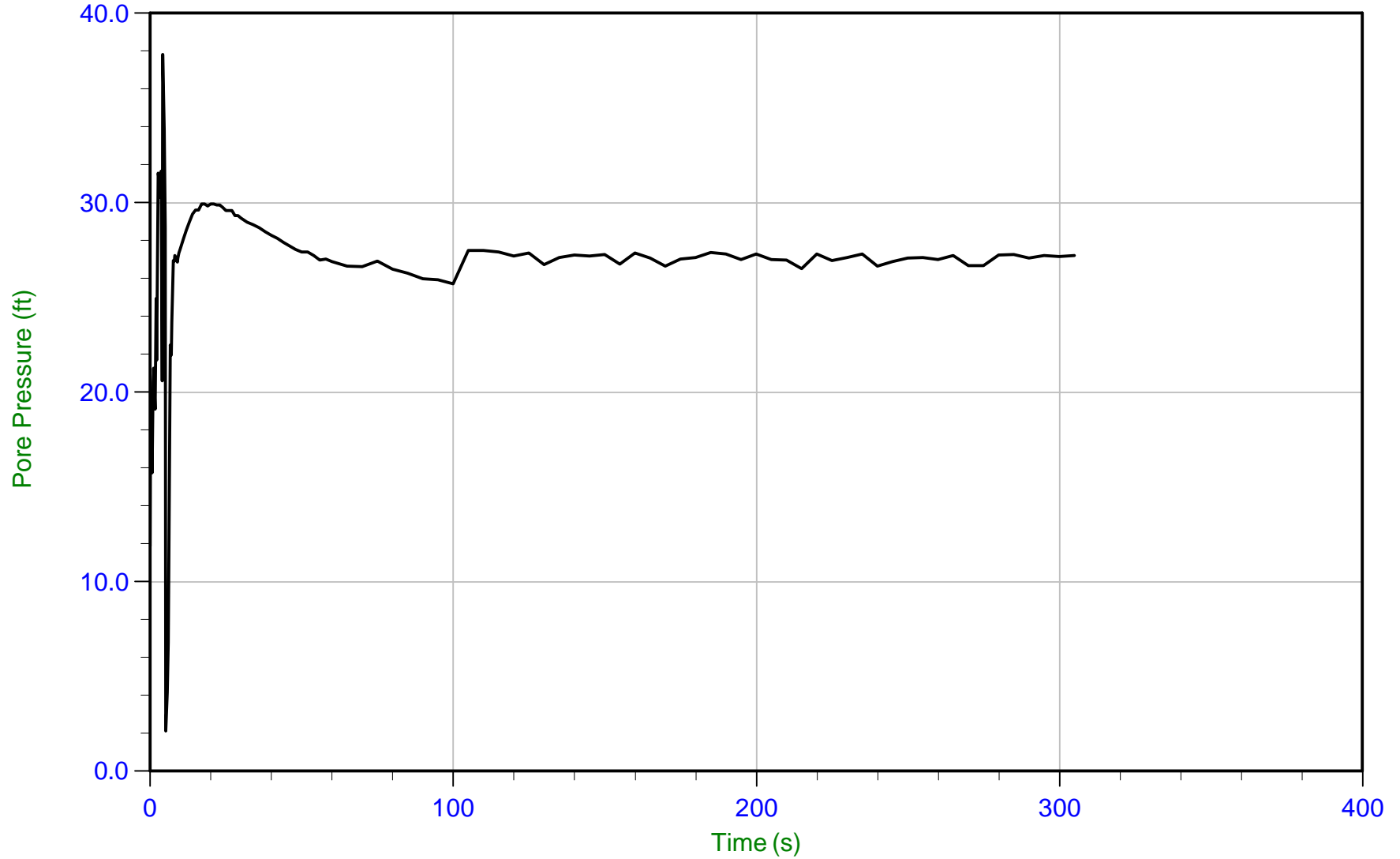
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-03  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

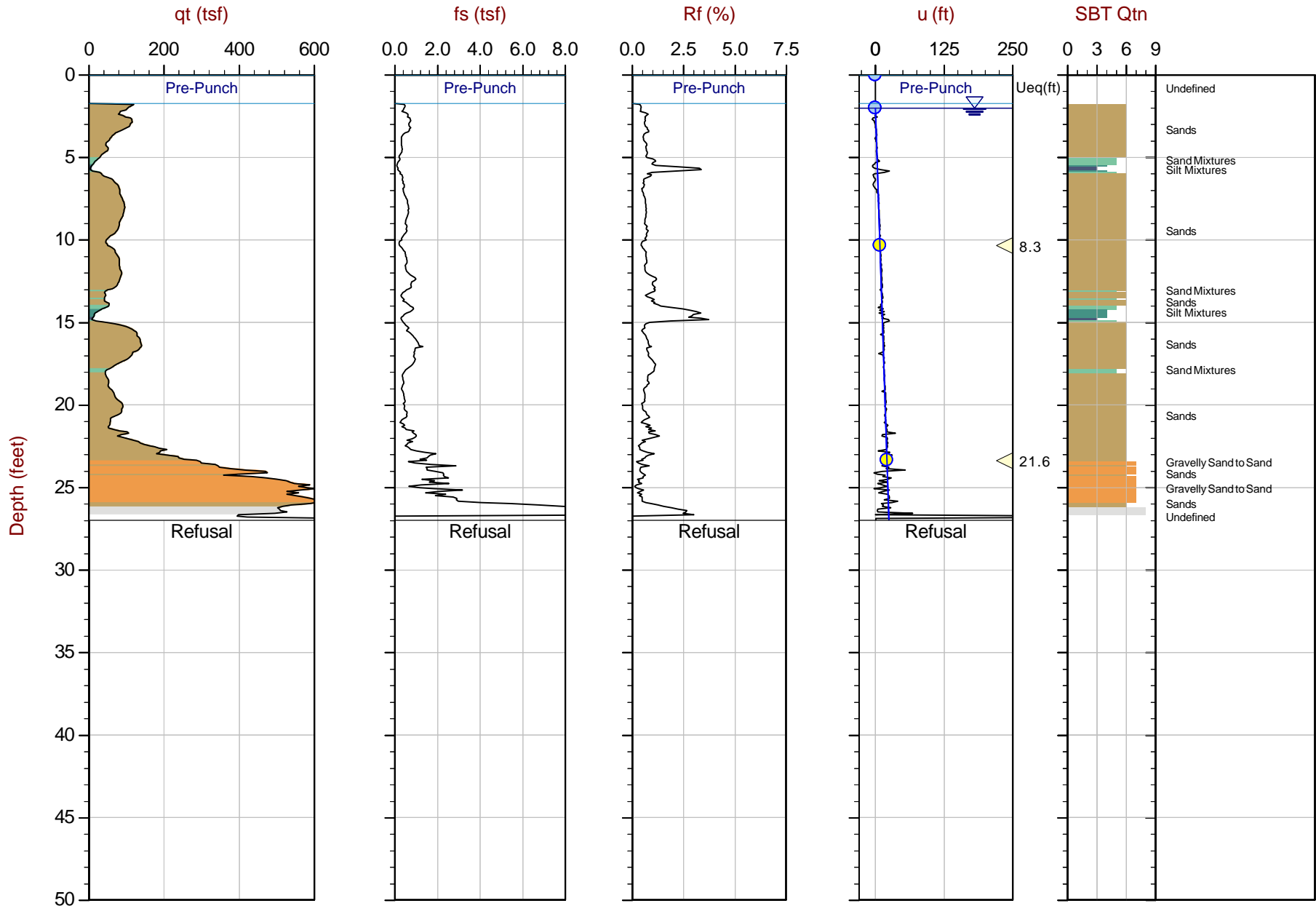


Trace Summary:

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

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u Max: 37.8 ft  
u Final: 27.2 ft

WT: 1.001 m / 3.284 ft  
Ueq: 27.1 ft



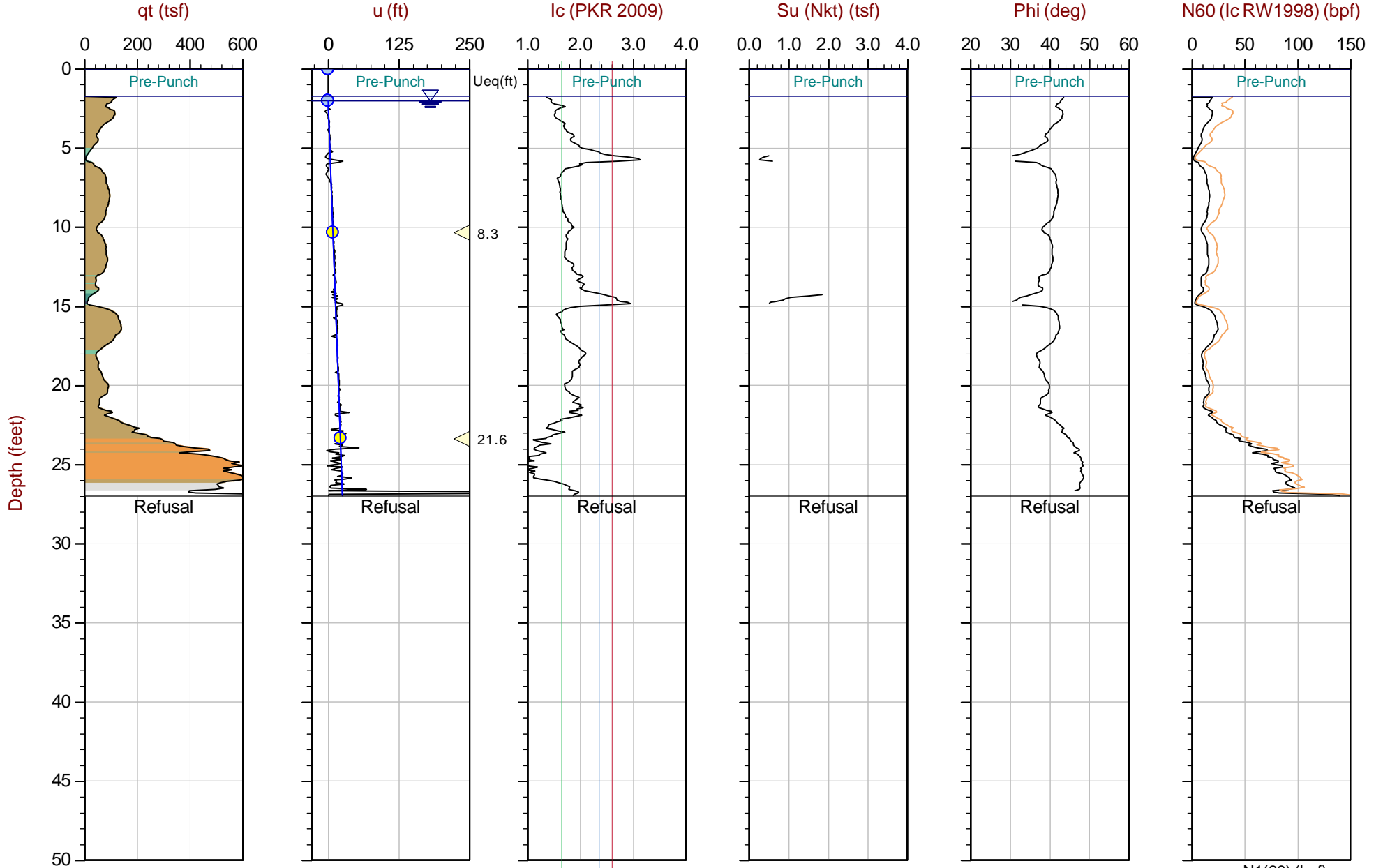
Max Depth: 8.225 m / 26.98 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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

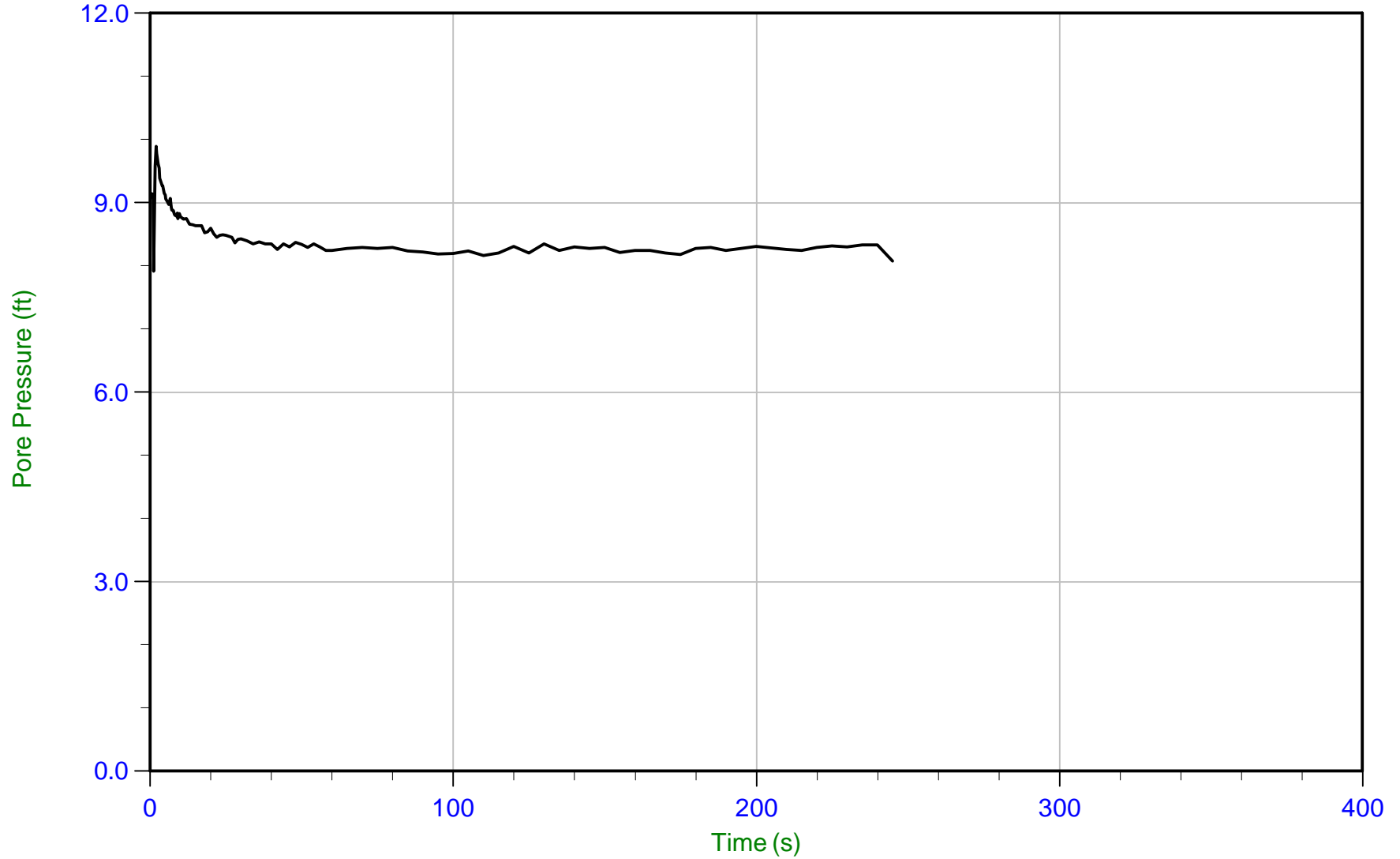
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-04  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

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

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u Max: 9.9 ft  
u Final: 8.1 ft

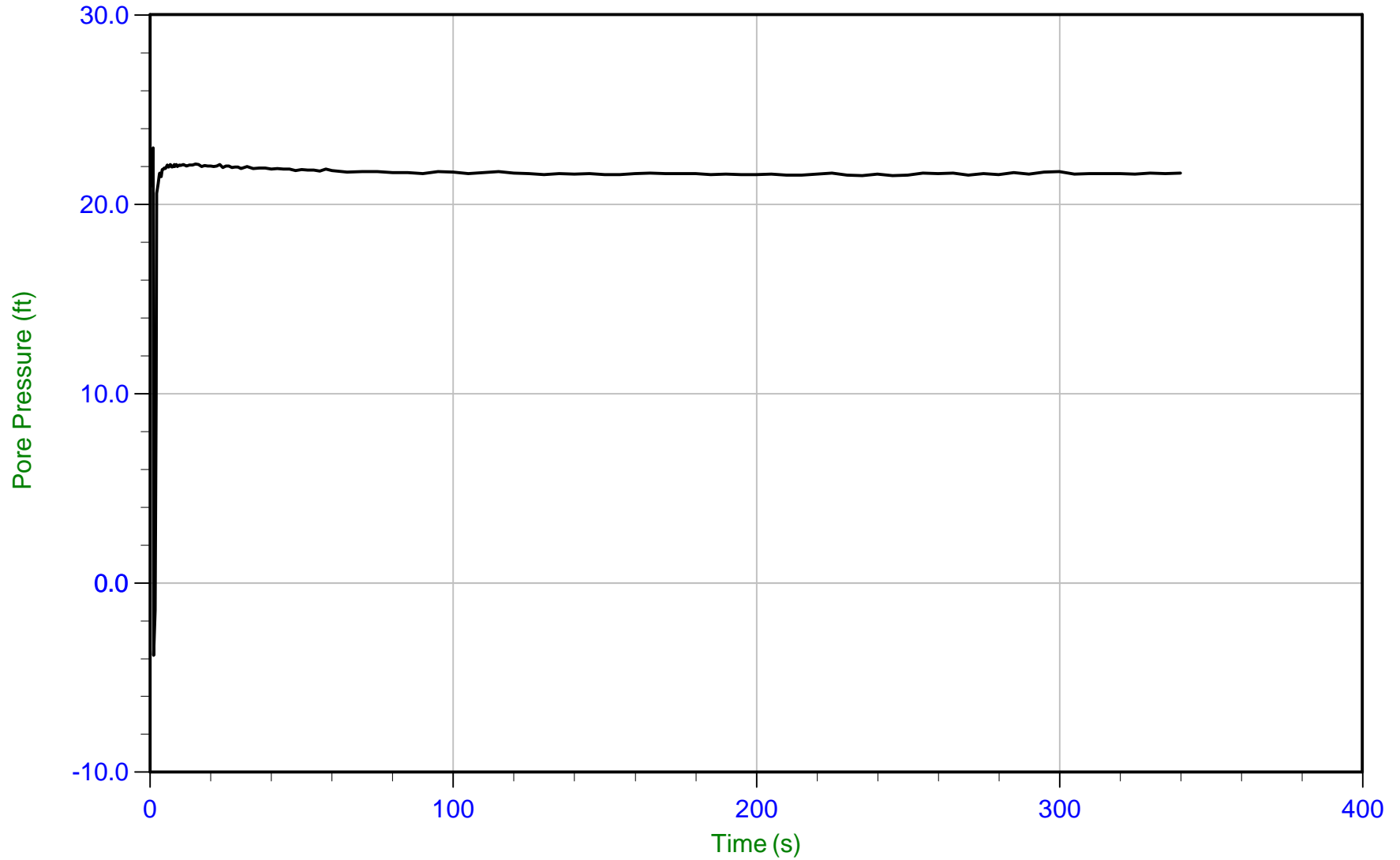
WT: 0.612 m / 2.009 ft  
Ueq: 8.3 ft



*Krazan*

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

Sounding: CPT-04  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

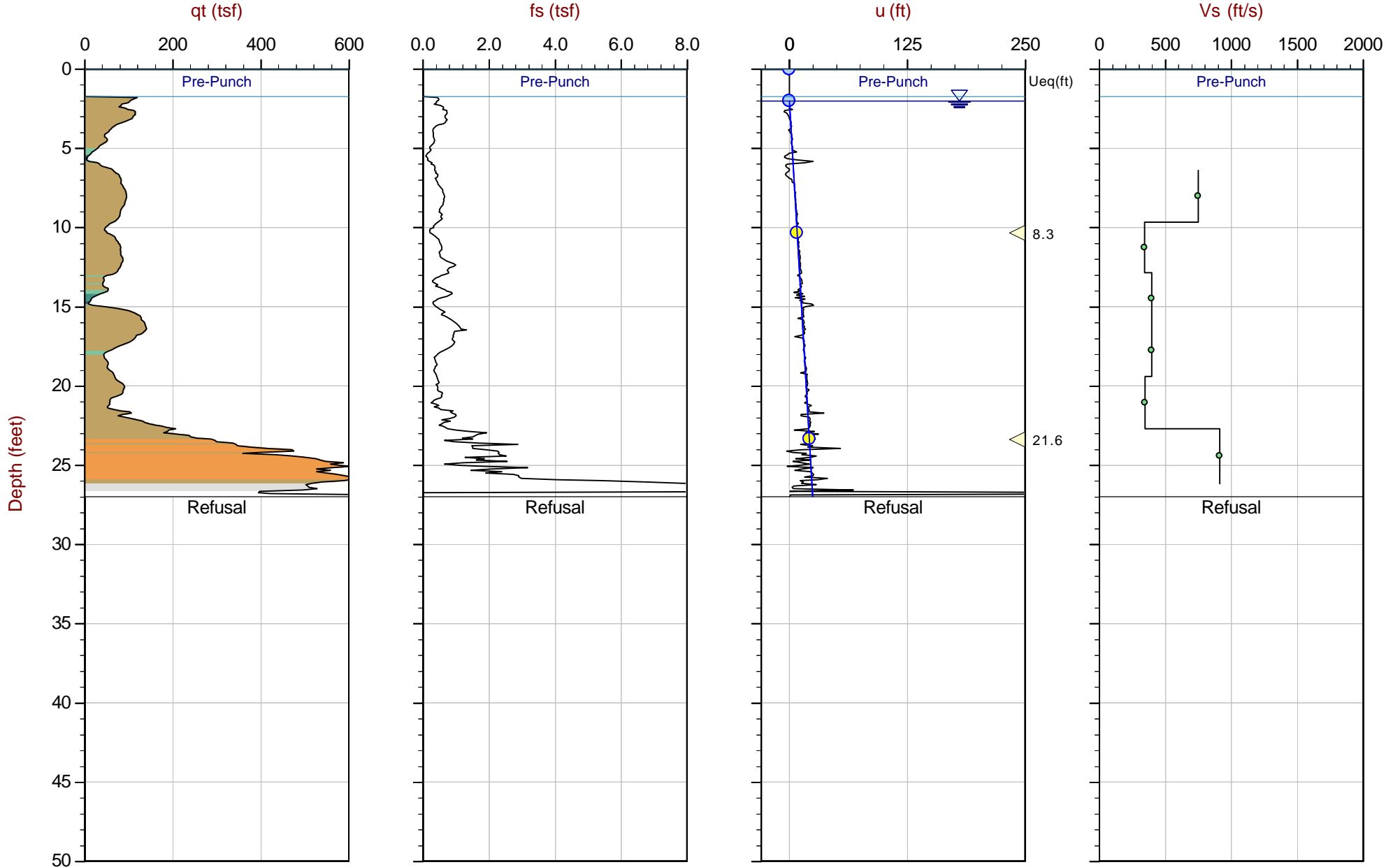


Trace Summary:

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

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u Max: 23.0 ft  
u Final: 21.7 ft

WT: 0.542 m / 1.779 ft  
Ueq: 21.6 ft



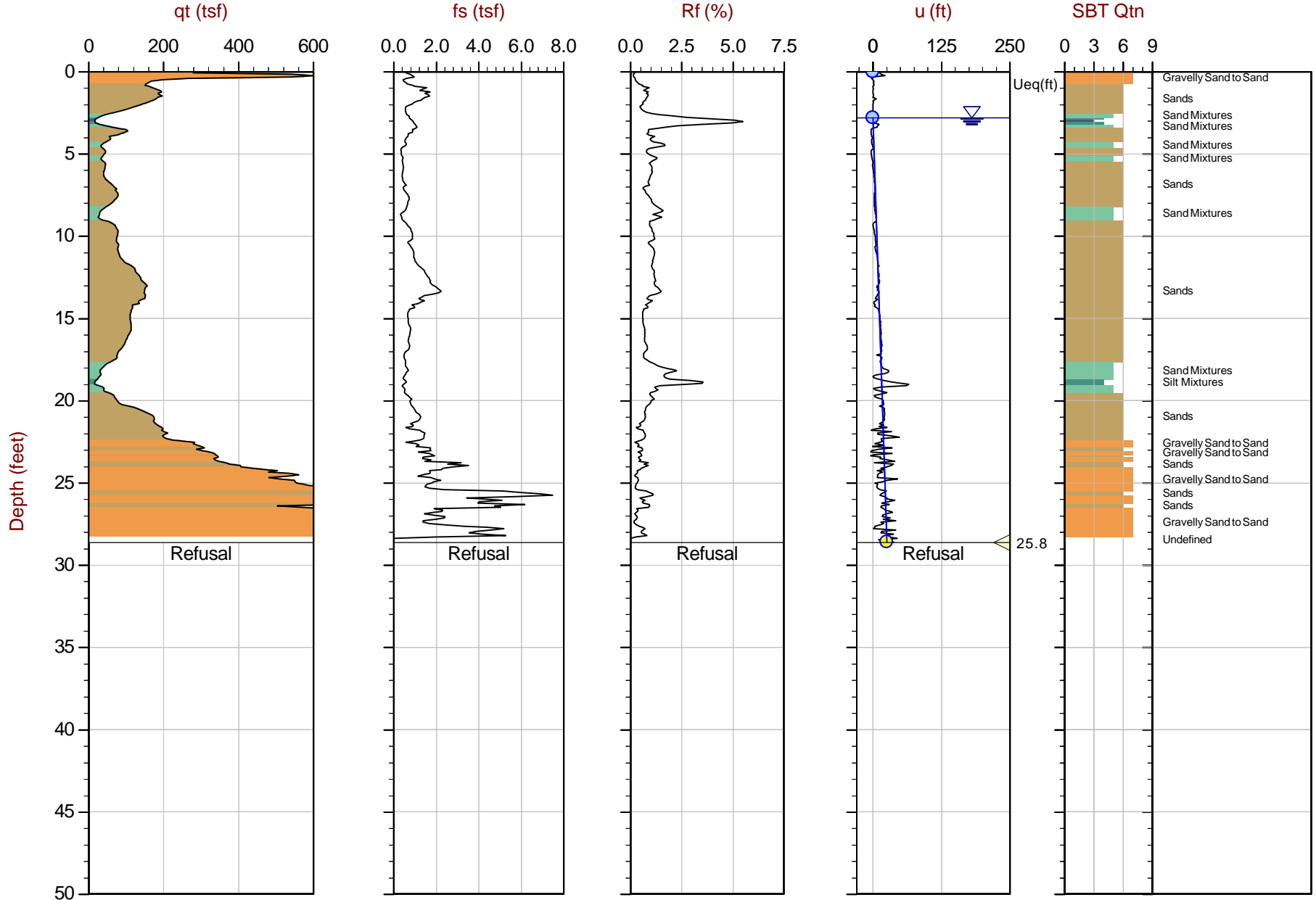
Max Depth: 8.225 m / 26.98 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



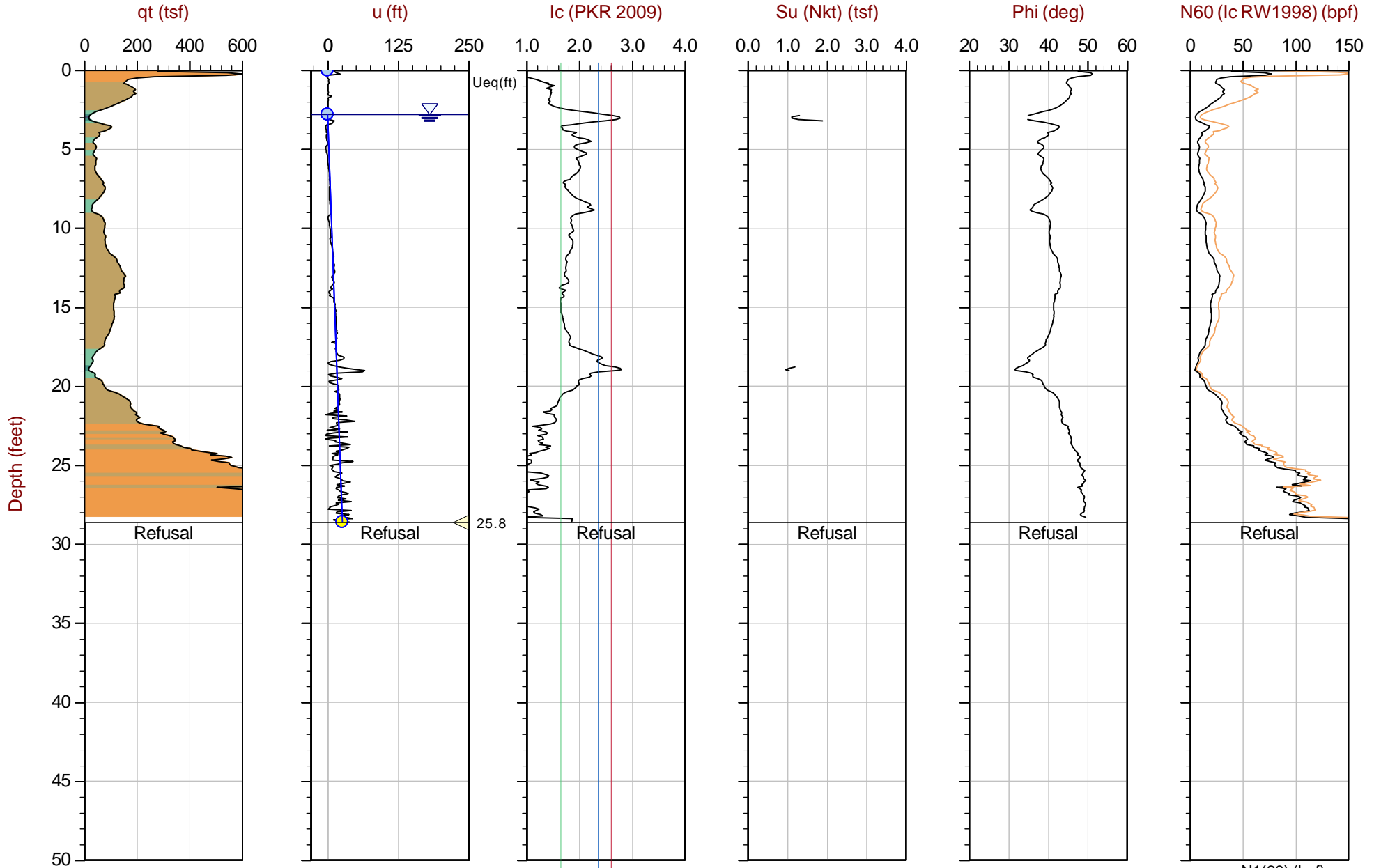
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 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.





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

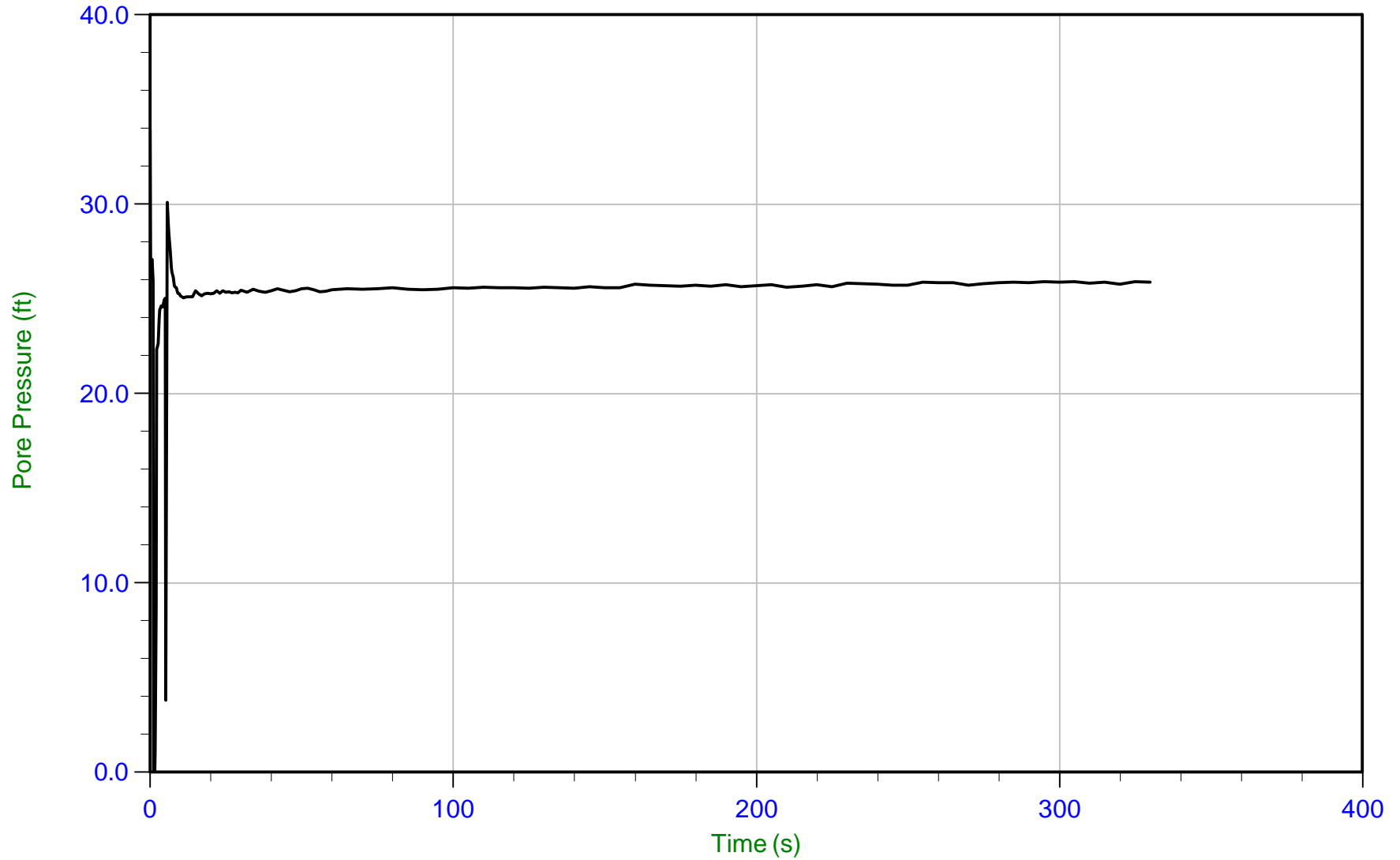
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-05  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

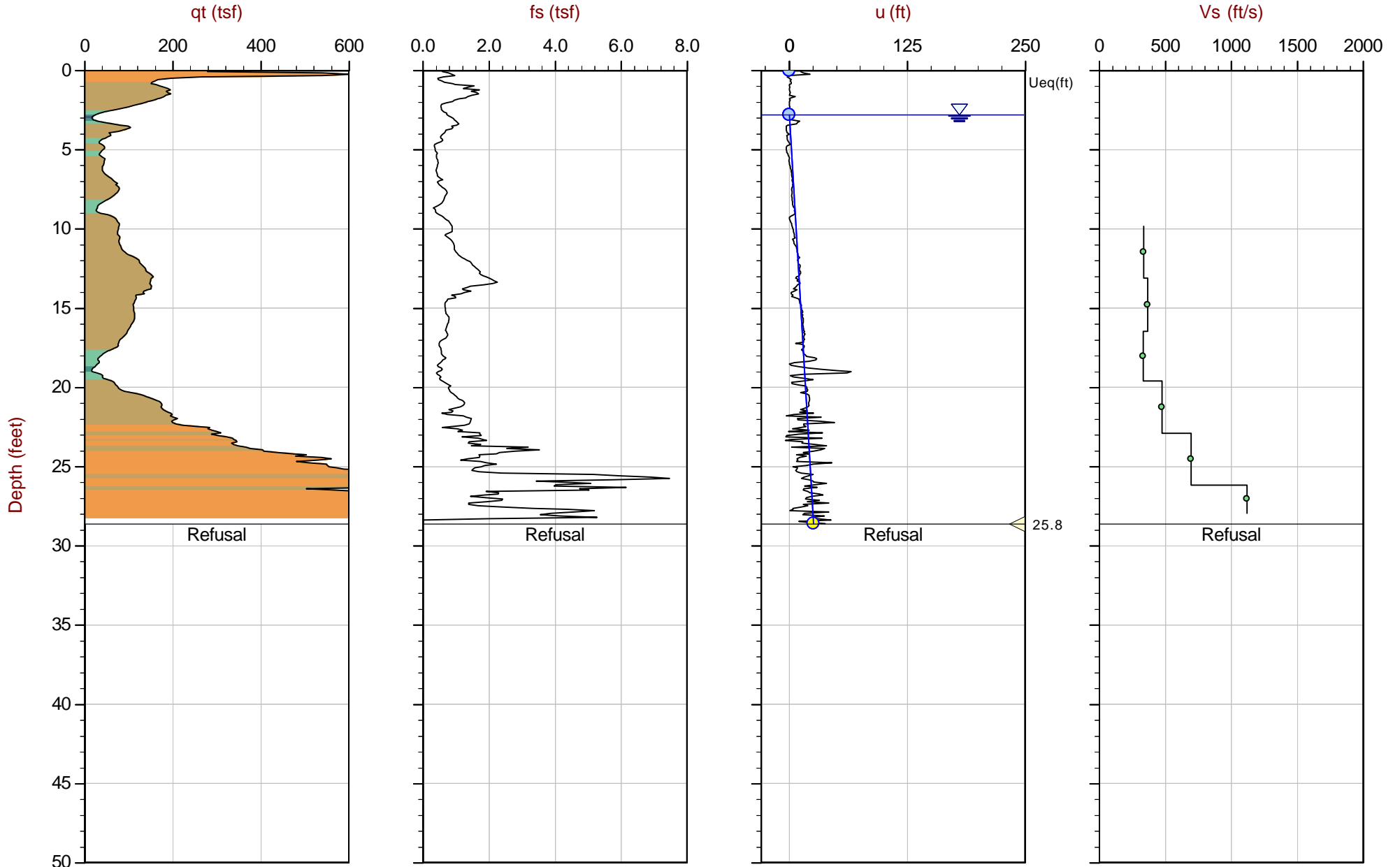


Trace Summary:

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



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 ( $U_{eq}$ )    
 ● Assumed  $U_{eq}$     
 ◁ Dissipation,  $U_{eq}$  achieved    
 ◁ Dissipation,  $U_{eq}$  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 contract 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.

Appendix A  
Draft Geotechnical  
Engineering  
Investigation by Krazan  
& Associates, Inc,  
dated May 6, 2022

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

**PROJECT NO. 062-22010**

MAY 6, 2022

**Prepared for:**

**BP PRODUCTS NORTH AMERICA, INC.**  
30 SOUTH WACKER DRIVE, SUITE 900  
CHICAGO, IL 60606

**Prepared by:**

**KRAZAN & ASSOCIATES, INC.**  
**GEOTECHNICAL ENGINEERING DIVISION**  
825 CENTER STREET, STE A  
TACOMA, WASHINGTON 98409  
(253) 939-2500



# **Krazan & ASSOCIATES, INC.**

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GEOTECHNICAL ENGINEERING • ENVIRONMENTAL ENGINEERING  
CONSTRUCTION TESTING & INSPECTION

May 6, 2022

KA Project No. 062-22010

**BP Products North America Inc.**  
30 South Wacker Drive, Suite 900  
Chicago, IL 60606

**Attn: Mr. Randall Arnold**  
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  
Project Manager

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May 6, 2022

KA Project No. 062-22010

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

**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 Kraزان 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 ratio, 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. We have assumed that the structure will have a fundamental period of vibration of equal to or less than 0.5 seconds. 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  $S_I$ ) 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 ( $T_s$ ) 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
$T_s$	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-induced 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.

<b>Table 2 - SOIL PARAMETERS FOR TEMPORARY SHORING DESIGN</b>						
<b>Material Description</b>	<b>Depth (ft.)</b>	<b>Angle of Internal Friction (degrees)</b>	<b>Cohesion (psf)</b>	<b>Moist Unit Weight (pcf)</b>	<b>Active Earth Pressure Coefficient (K<sub>a</sub>)</b>	<b>Passive Earth Pressure Coefficient (K<sub>p</sub>)</b>
<b>Soil Layer 1:</b> very loose to medium dense Sands	0 - 22	22	0	105	0.45	2.20
<b>Soil Layer 2 (Native Soils):</b> 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 to 1 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  $\pm 2$  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  $\frac{1}{4}$  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**

<b>PILE DIAMETER</b>  <b>(Inches)</b>	<b>ALLOWABLE PILE CAPACITY</b>  <b>(Kips)</b>
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) for granular structural fill 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.

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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  $\frac{3}{4}$ -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 stress 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 dissipation 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**

<b>Asphaltic Concrete</b>	<b>Aggregate Base*</b>
3.0 in.	6.0 in.

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

<b>Min. PCC Depth</b>	<b>Aggregate Base*</b>
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 overruns. 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  
Project Manager

Vijay Chaudhary, P.E.  
Assistant Regional Engineering Manager

DRAFT

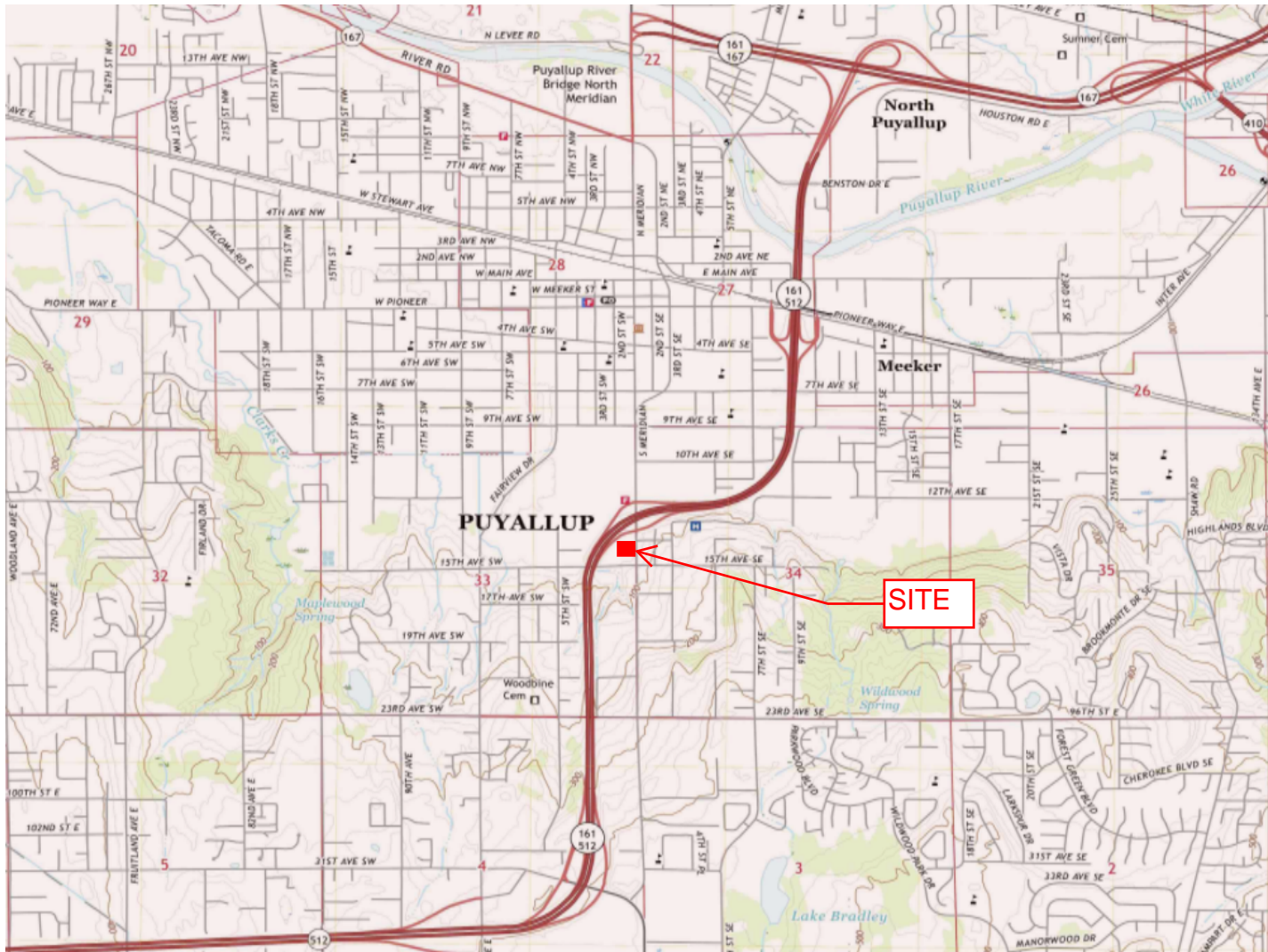



Figure 1: Vicinity Map

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

		
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

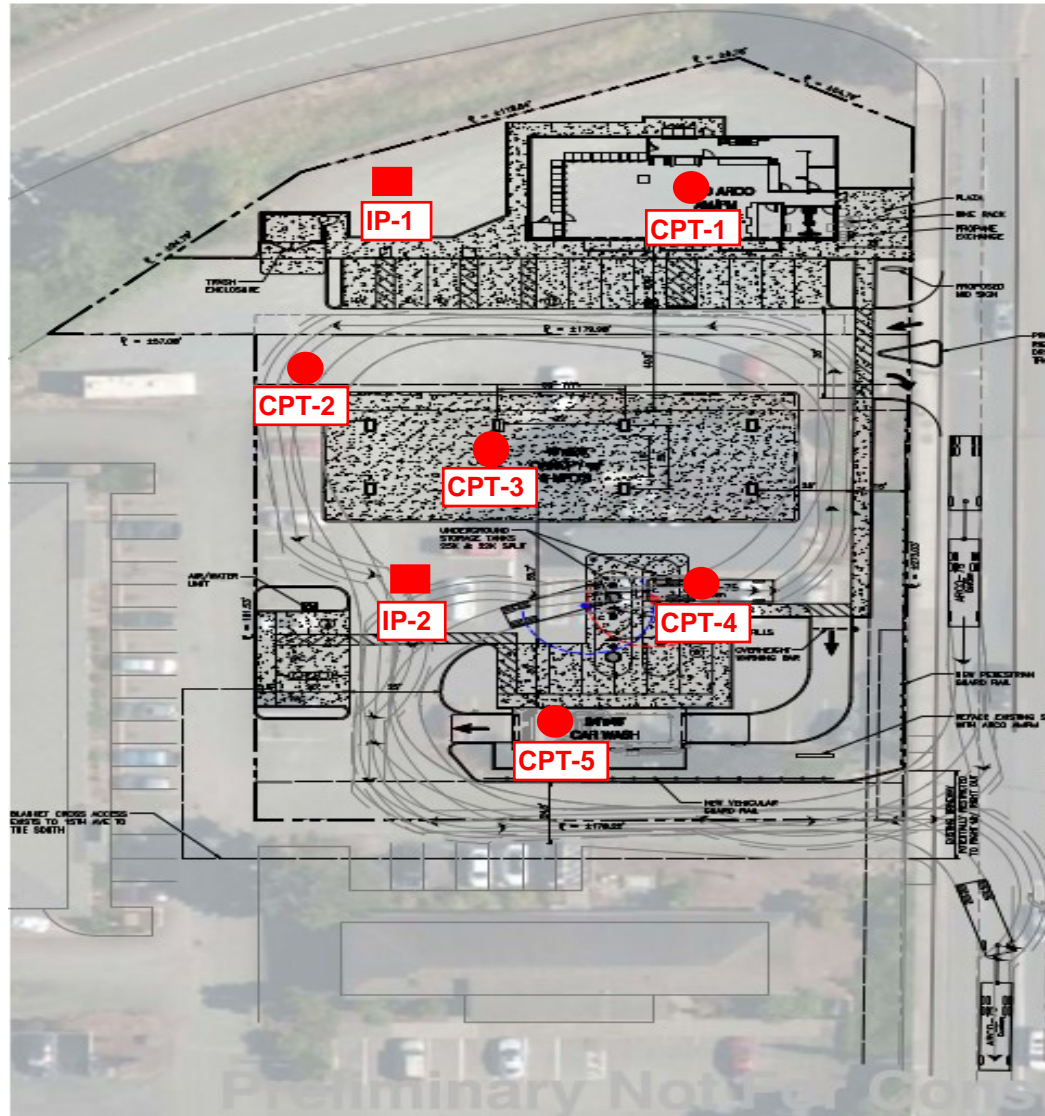


Figure 2: Site Plan

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.

		
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.

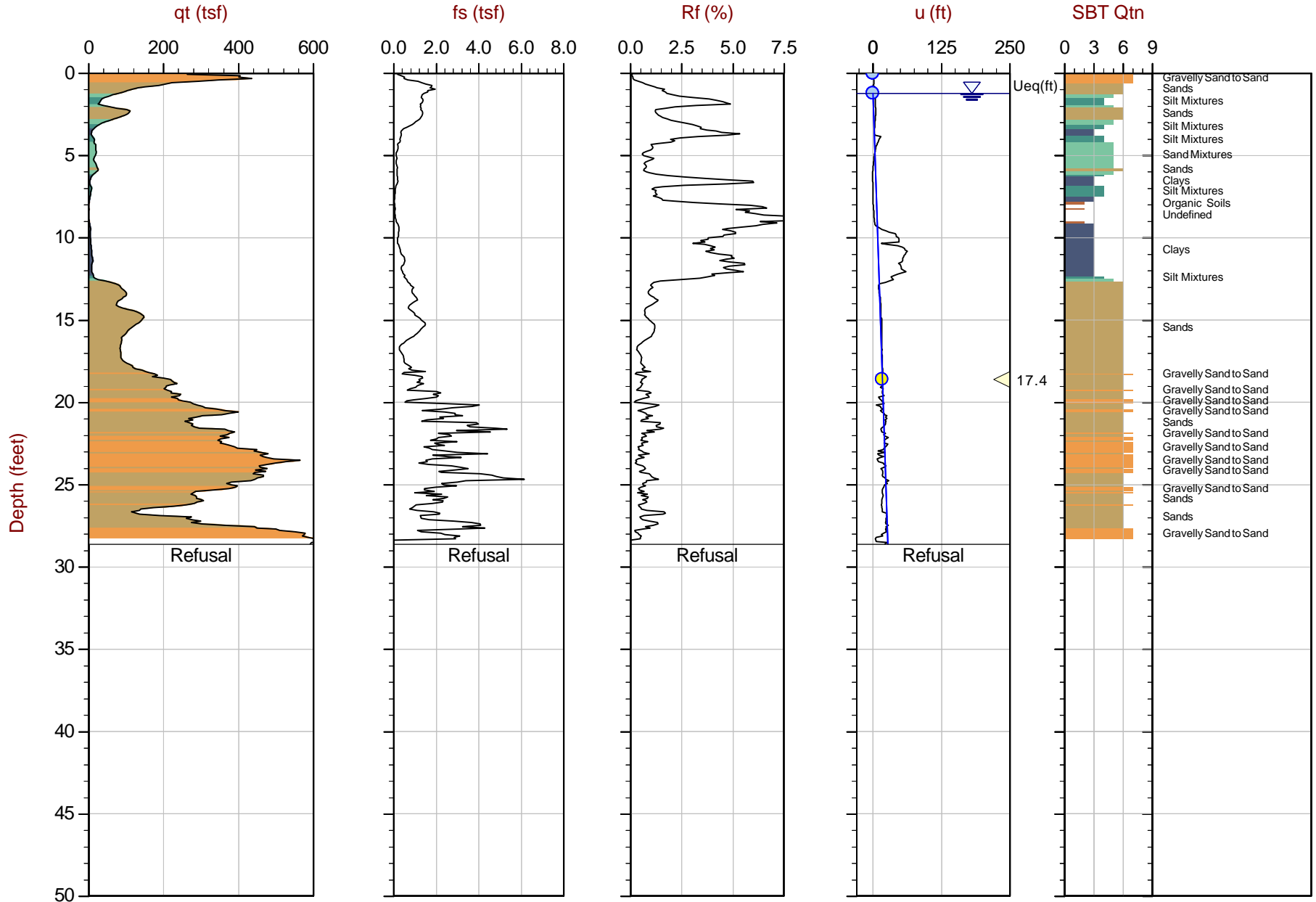
#### **Liquefaction Analysis**

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.

<b>Project:</b> Prop. ARCO ampm Fueling Facility		<b>Project Number:</b> 062-22010		<b>Client:</b> BP Products North America		<b>Test Pit No.:</b>		<b>IP-1</b>	
<b>Location:</b> 1402 S. Meridian Ave., Puyallup, WA						<b>Contractor:</b> Strickland & Sons Excavation			
<b>Project Manager:</b> Theresa Nunan			<b>Date</b>	<b>Started:</b> 3.28.22		<b>Equipment:</b> CAT 306 Excavator with WW			
<b>Field Engineer:</b> Theresa Nunan				<b>Completed:</b> 3.28.22					
<b>Groundwater Depth:</b> 5.8 +/- feet (and rising)				<b>Ground Elevation:</b> 46.0 +/- feet		<b>Total Depth of Test Pit:</b> 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)			
	1					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			
	2						(FILL)		
	3	BULK	S-1			Dark Brown MUCK (Organic Silt and Clay with some fibrous organics), very soft, moist			
	4						(SHALCAR MUCK)		
	5	BULK	S-2			Dark Grey SAND (SP-SM) with silt, fine grained sand, very loose, moist to wet			
	6								
	7						(ALLUVIUM)		
	8					Test Pit Terminated at 7.1 Feet			

<b>Project:</b> Prop. ARCO ampm Fueling Facility		<b>Project Number:</b> 062-22010		<b>Client:</b> BP Products North America		<b>Test Pit No.:</b>		<b>IP-2</b>	
<b>Location:</b> 1402 S. Meridian Ave., Puyallup, WA						<b>Contractor:</b> Strickland & Sons Excavation			
<b>Project Manager:</b> Theresa Nunan			<b>Date</b>	<b>Started:</b> 3.28.22		<b>Equipment:</b> CAT 306 Excavator with WW			
<b>Field Engineer:</b> Theresa Nunan				<b>Completed:</b> 3.28.22					
<b>Groundwater Depth:</b> 2.3 +/- feet (and rising)				<b>Ground Elevation:</b> 47.0		<b>Total Depth of Test Pit:</b> 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 <b>Asphalt Concrete Pavement</b> underlain by 6 inches of Brown Silty SAND (SM) with gravel <b>BASE COURSE</b>			
	1					Tan <b>Poorly Graded SAND (SP-SM)</b> with silt, fine to medium grained, loose, moist  <b>(FILL)</b>			
	2	BULK	S-1			Bluish Grey <b>SAND (SP-SM)</b> with silt, very loose, wet  <b>(ALLUVIUM)</b>			
	3	BULK	S-2			Dark Brown <b>Organic SILT (OH)</b> , very soft, wet  <b>(SHALCAR MUCK)</b>			
	4	BULK	S-3			Dark Grey/Black <b>Poorly Graded SAND (SP-SM)</b> with silt, very loose, wet  <b>(ALLUVIUM)</b>			
	5					<b>Test Pit Terminated at 4.7 Feet</b>			
	6								
	7								
	8								



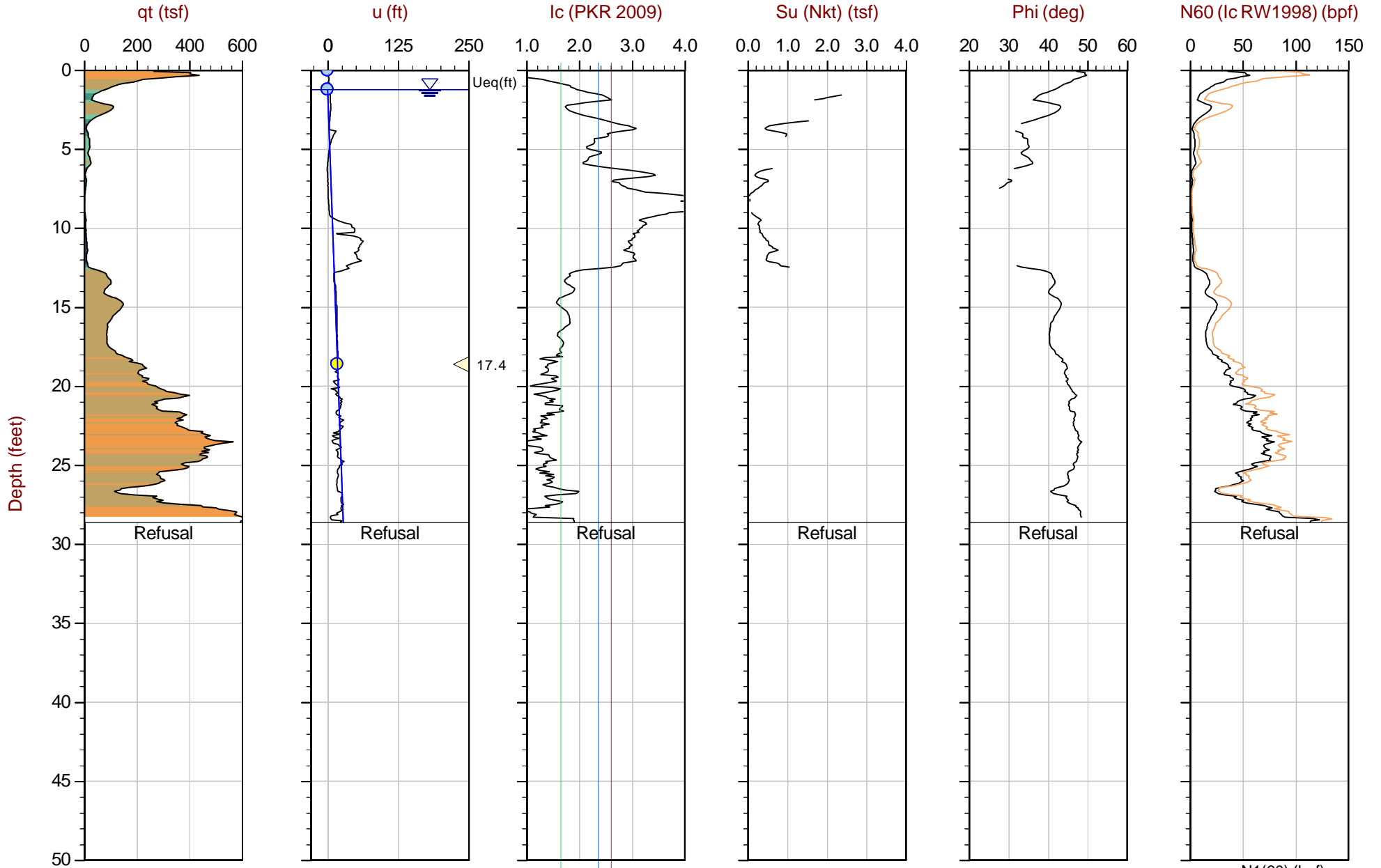


Max Depth: 8.725 m / 28.62 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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.

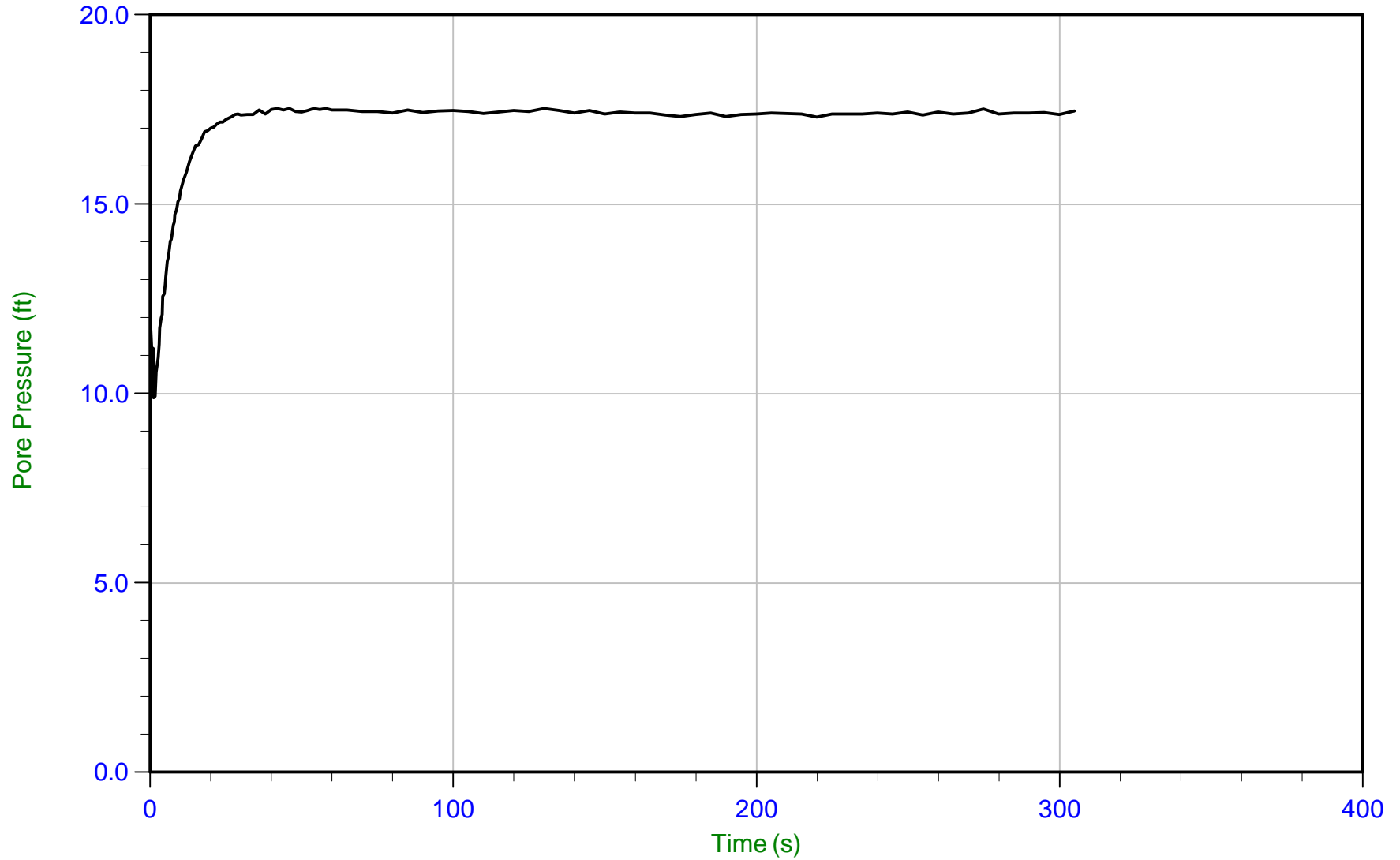




*Krazan*

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

Sounding: CPT-01  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

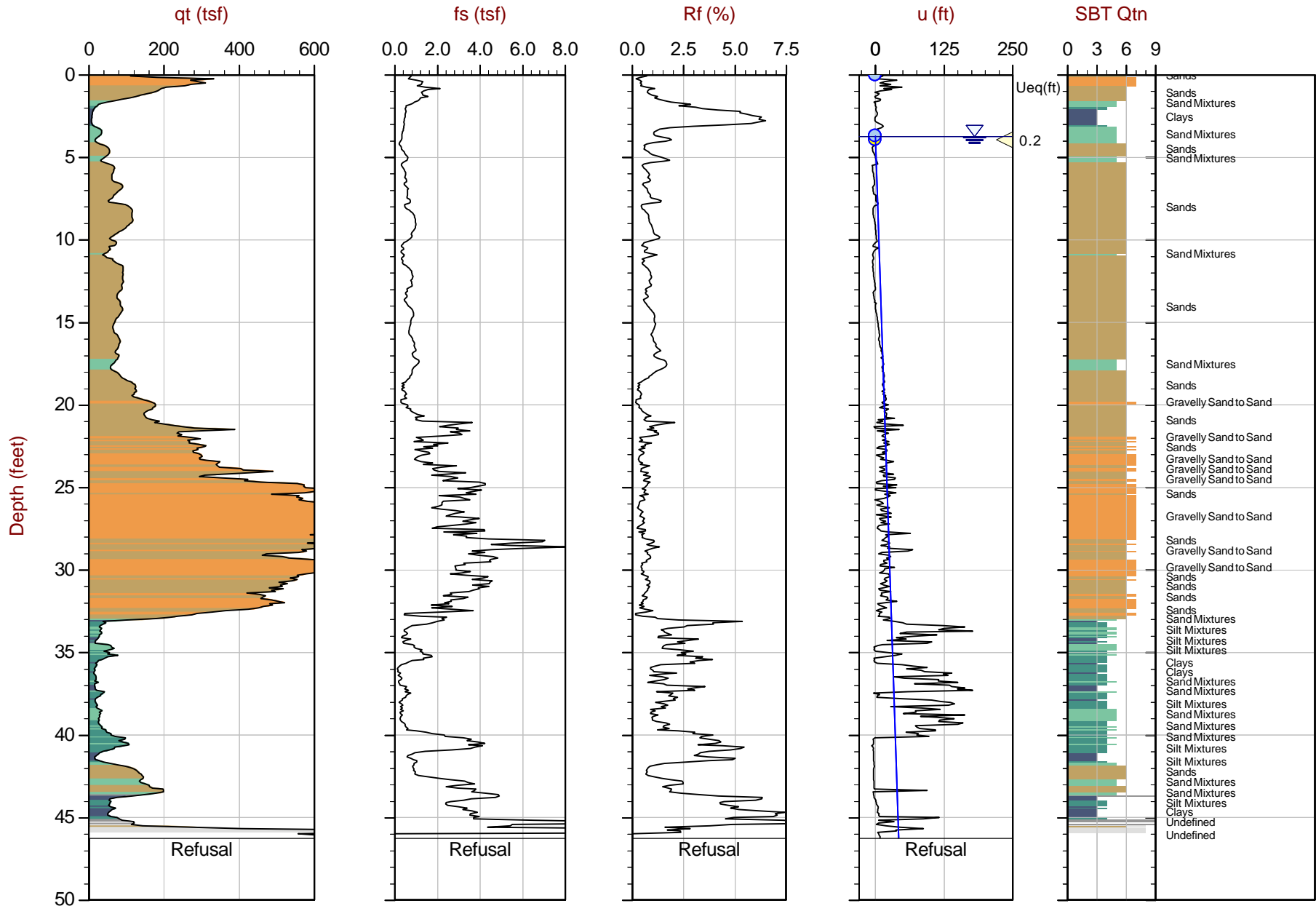


Trace Summary:

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



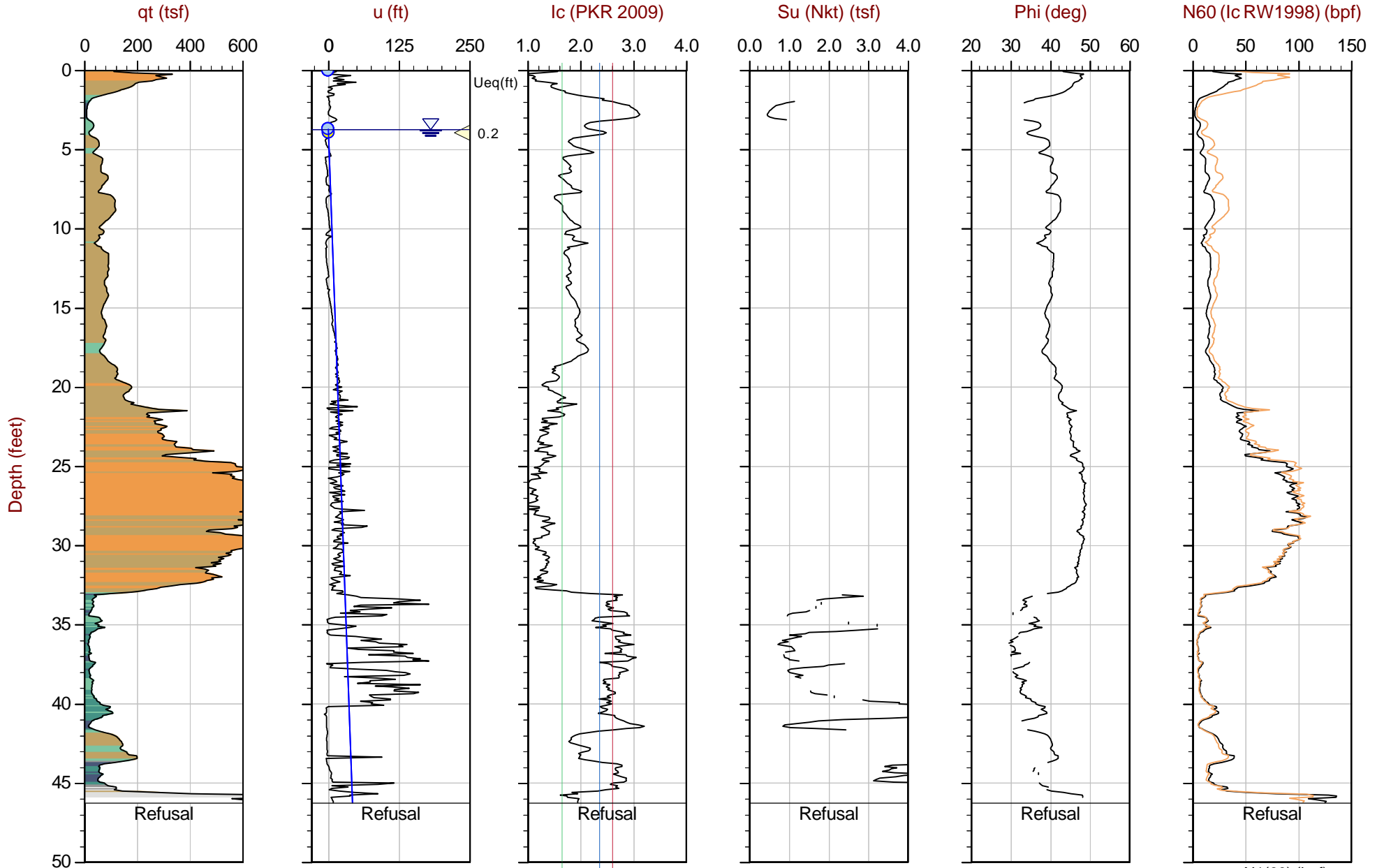
Max Depth: 14.100 m / 46.26 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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

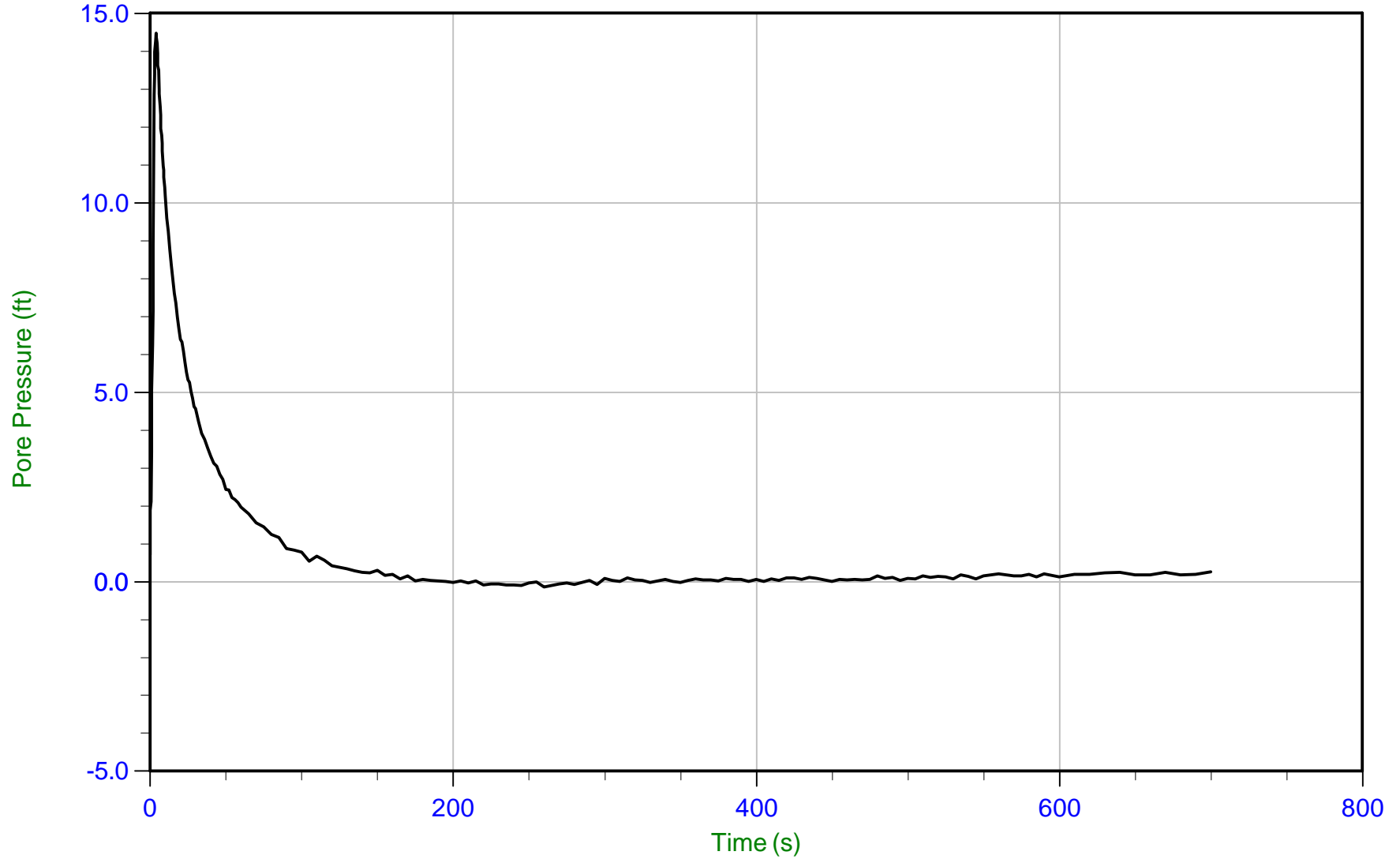
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-02B  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>



Trace Summary:

Filename: 22-59-23911\_SP02B.ppd2  
Depth: 1.200 m / 3.937 ft  
Duration: 700.0 s

u Min: -0.1 ft  
u Max: 14.5 ft  
u Final: 0.3 ft

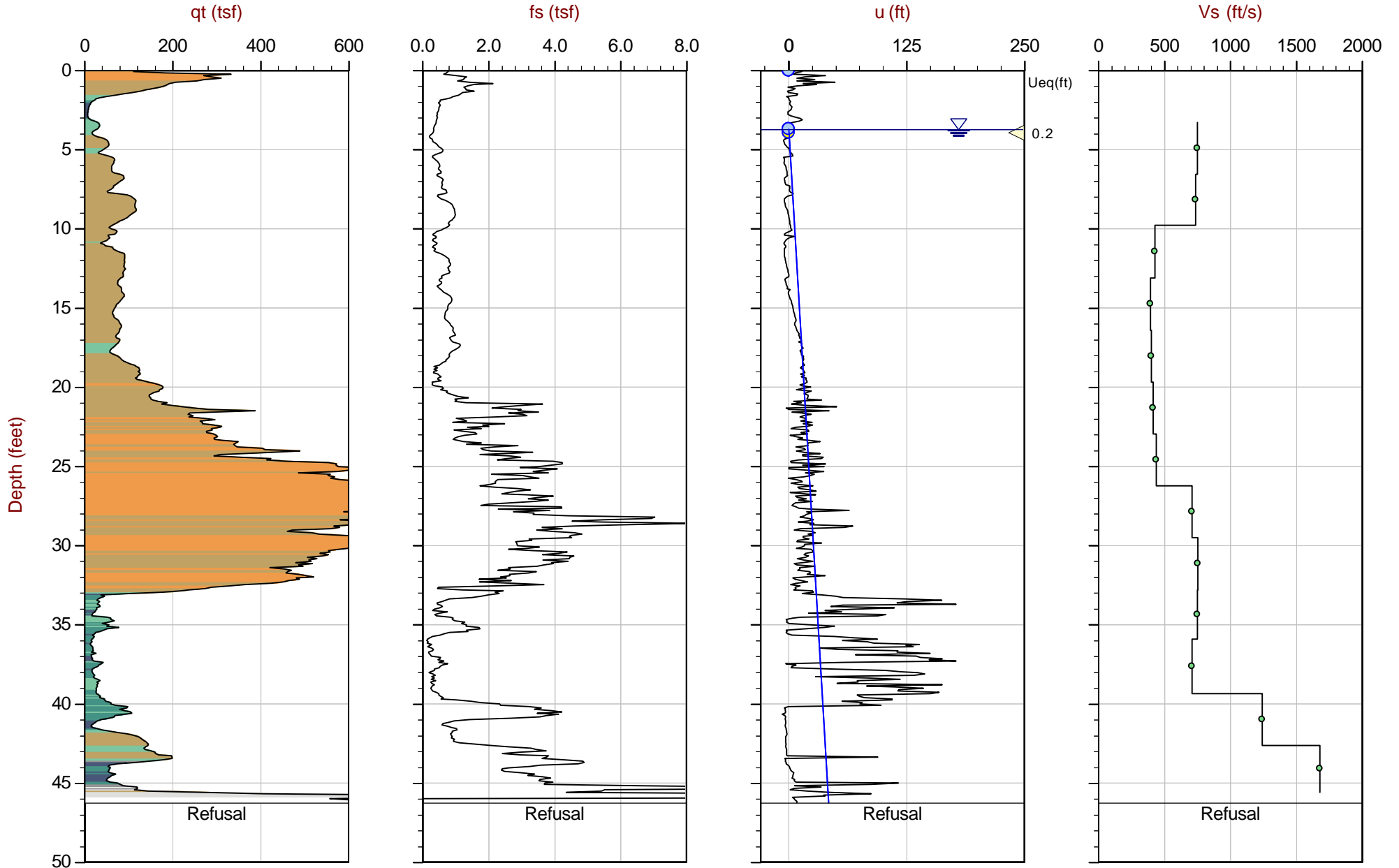
WT: 1.134 m / 3.720 ft  
Ueq: 0.2 ft



Krazan

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

Sounding: CPT-02B  
Cone: 781:T1500F15U35



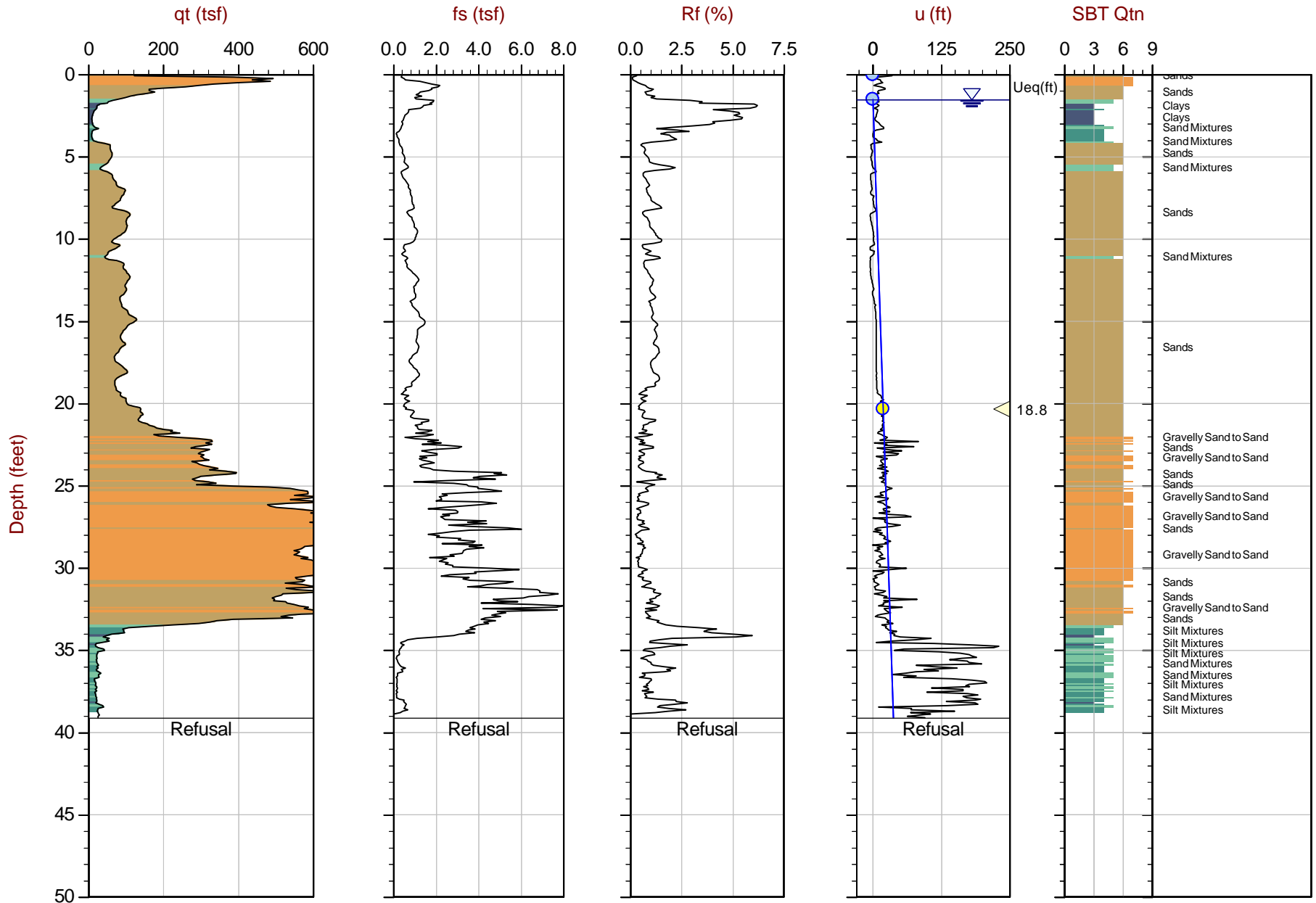
Max Depth: 14.100 m / 46.26 ft  
Depth Inc: 0.025 m / 0.082 ft  
Avg Int: Every Point

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.



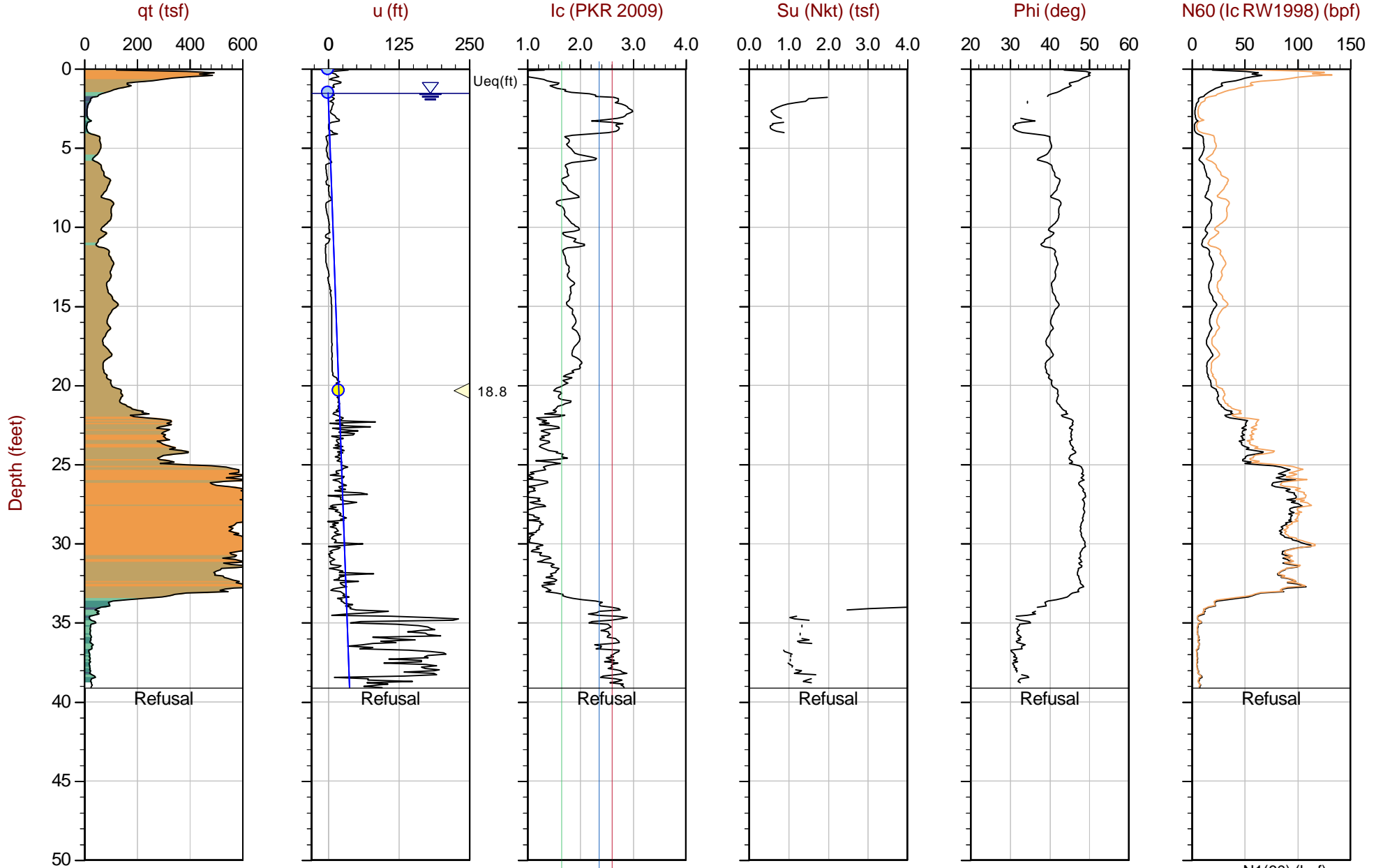
Max Depth: 11.925 m / 39.12 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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

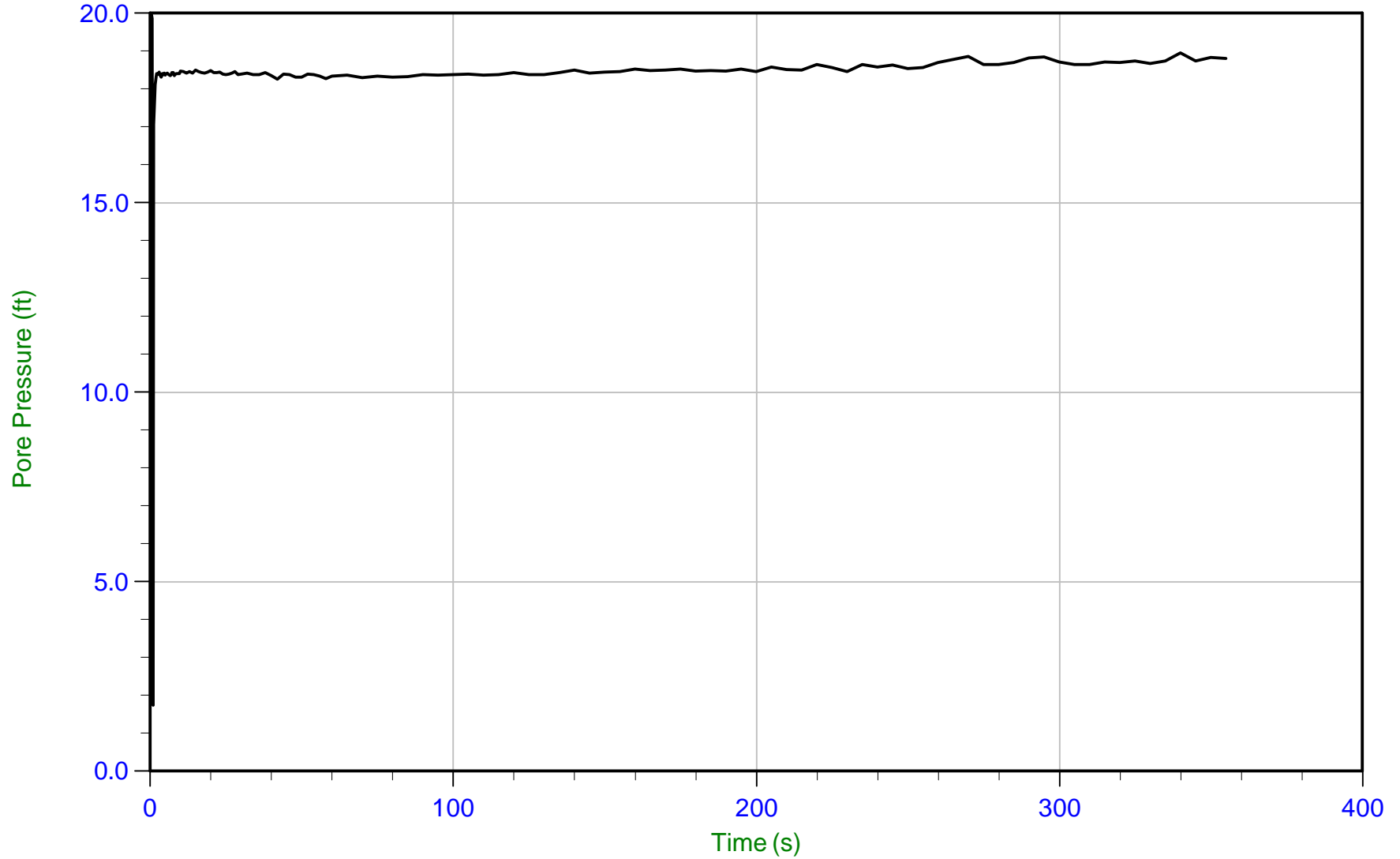
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-02  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>



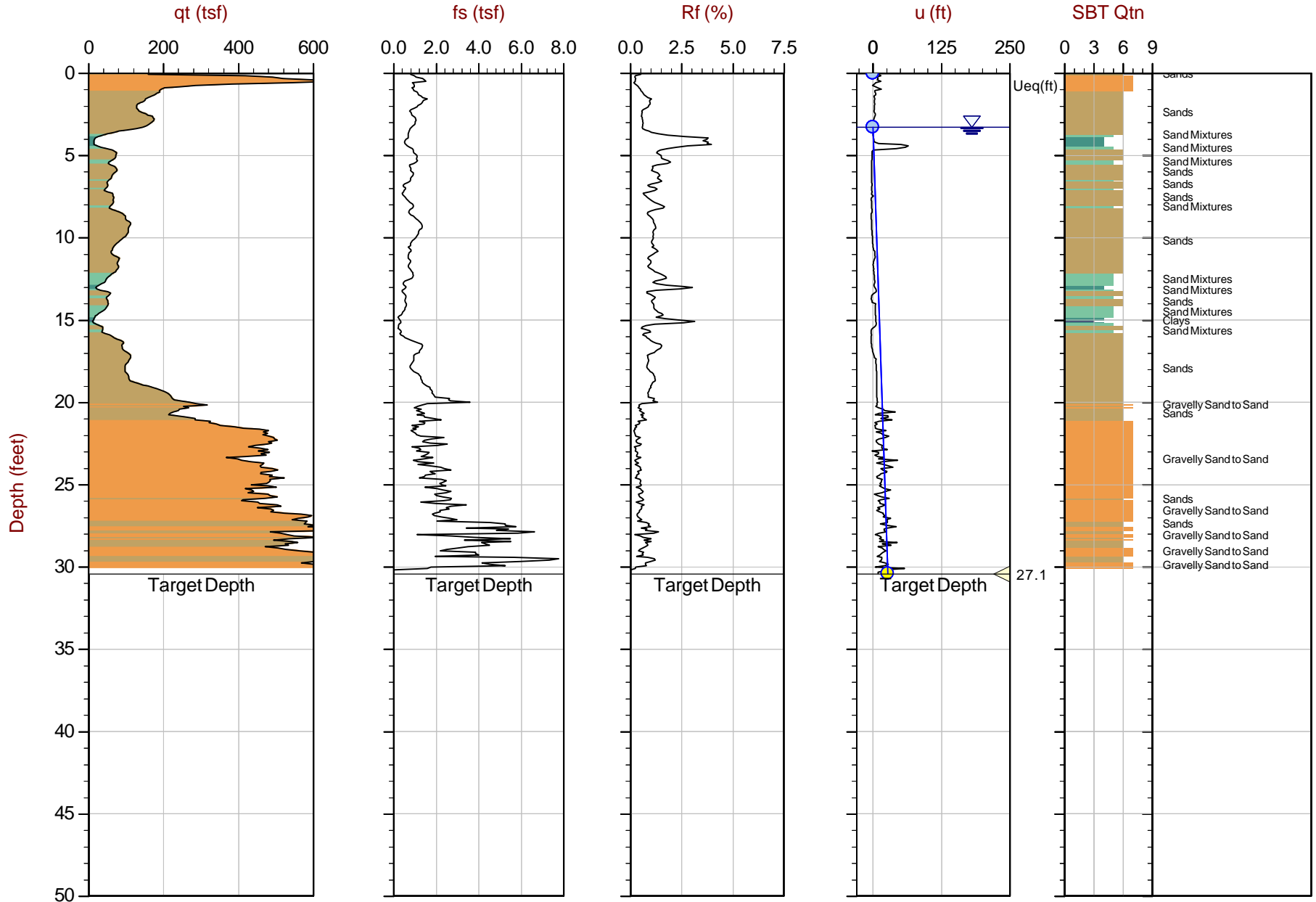
Trace Summary:

Filename: 22-59-23911\_CP02.ppd2  
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





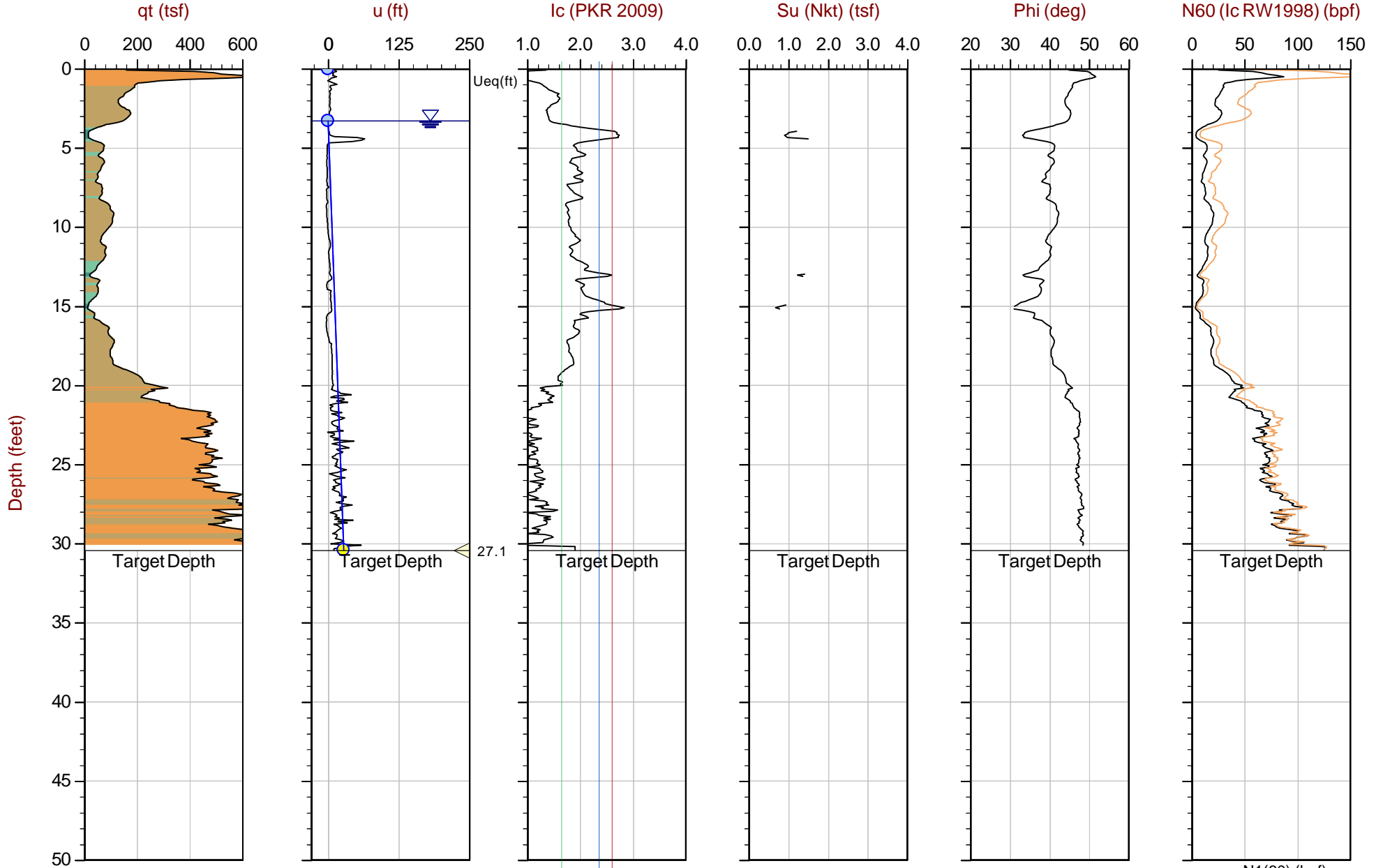
Max Depth: 9.275 m / 30.43 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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

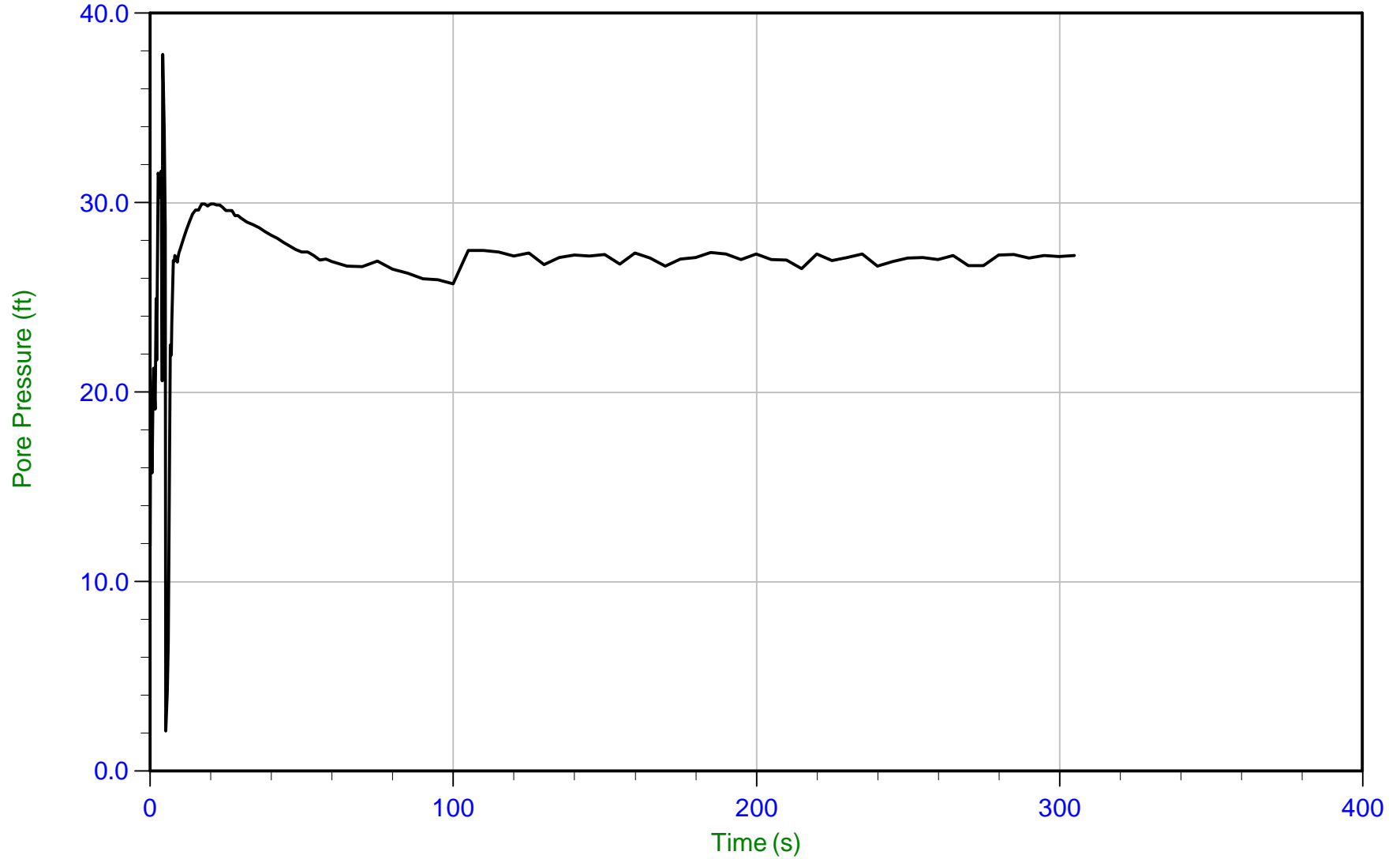
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-03  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

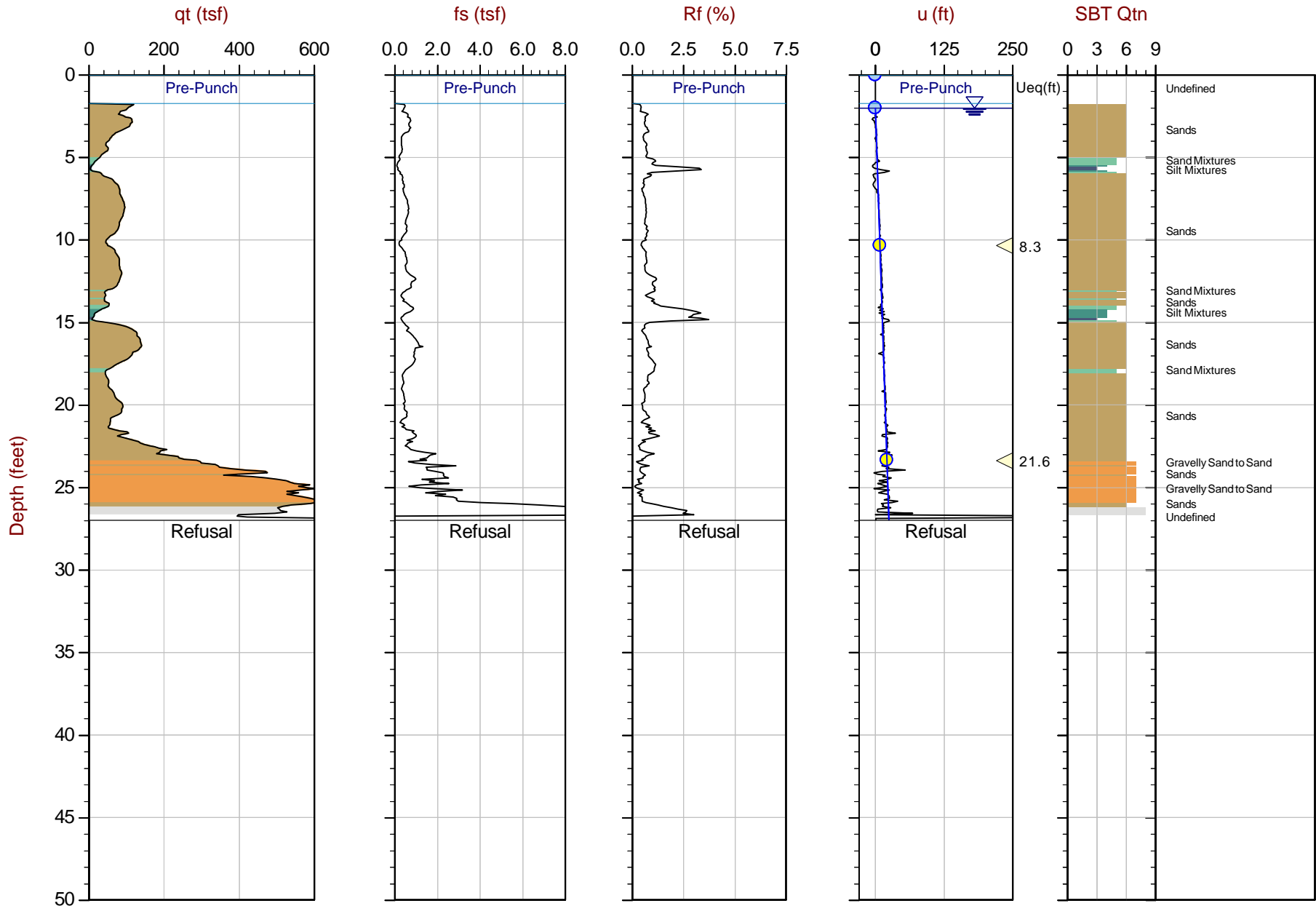


Trace Summary:

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



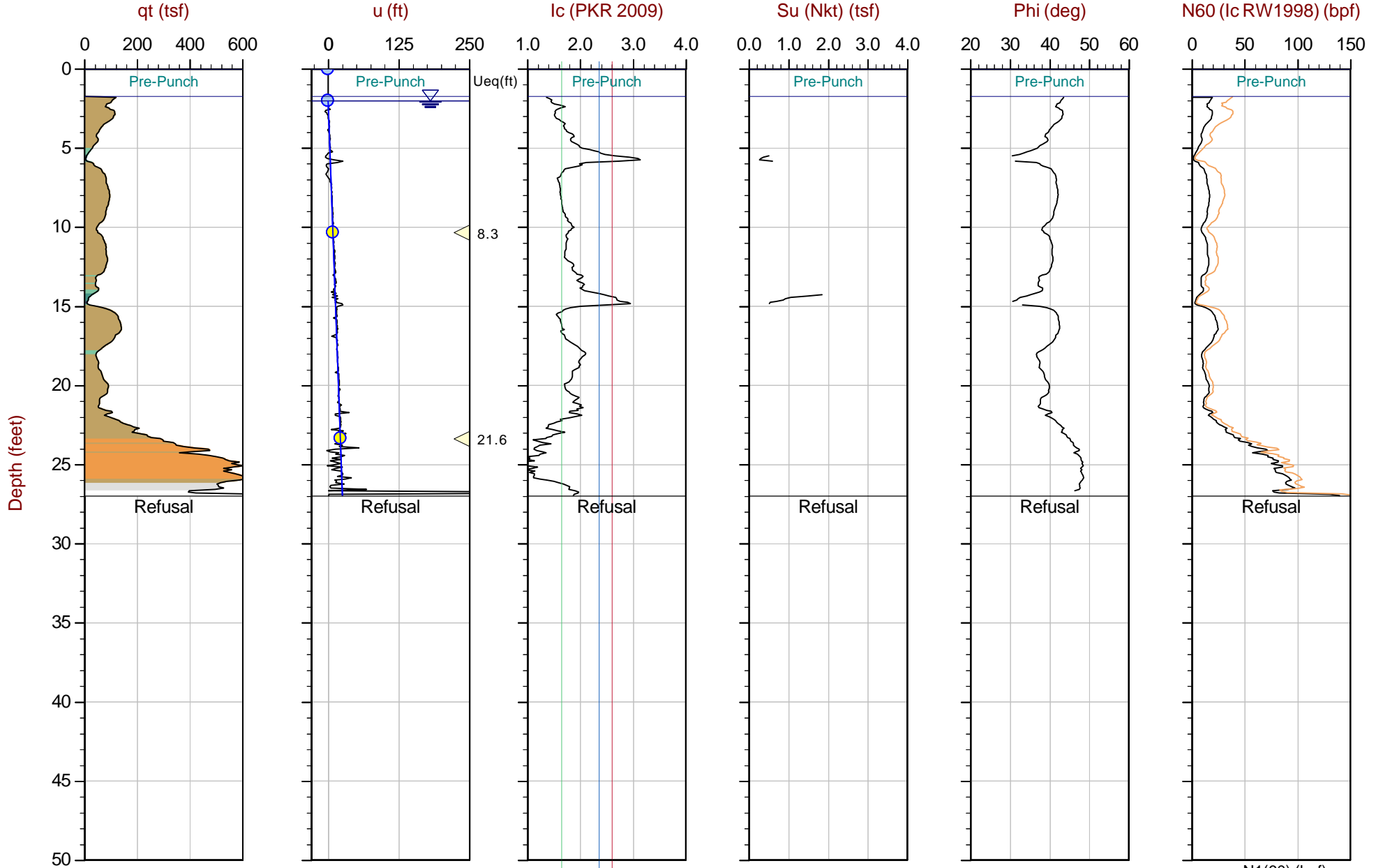
Max Depth: 8.225 m / 26.98 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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

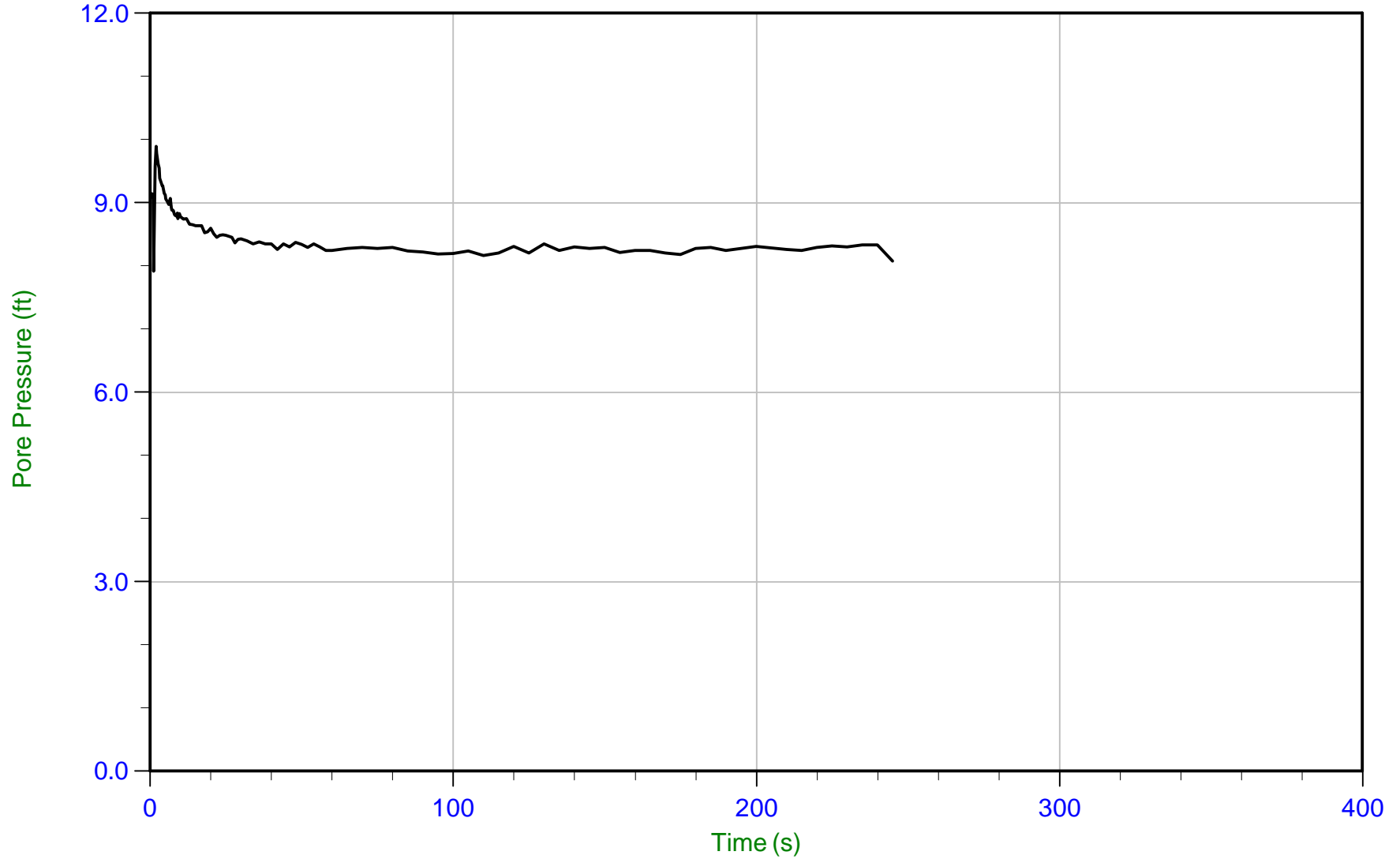
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-04  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>



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

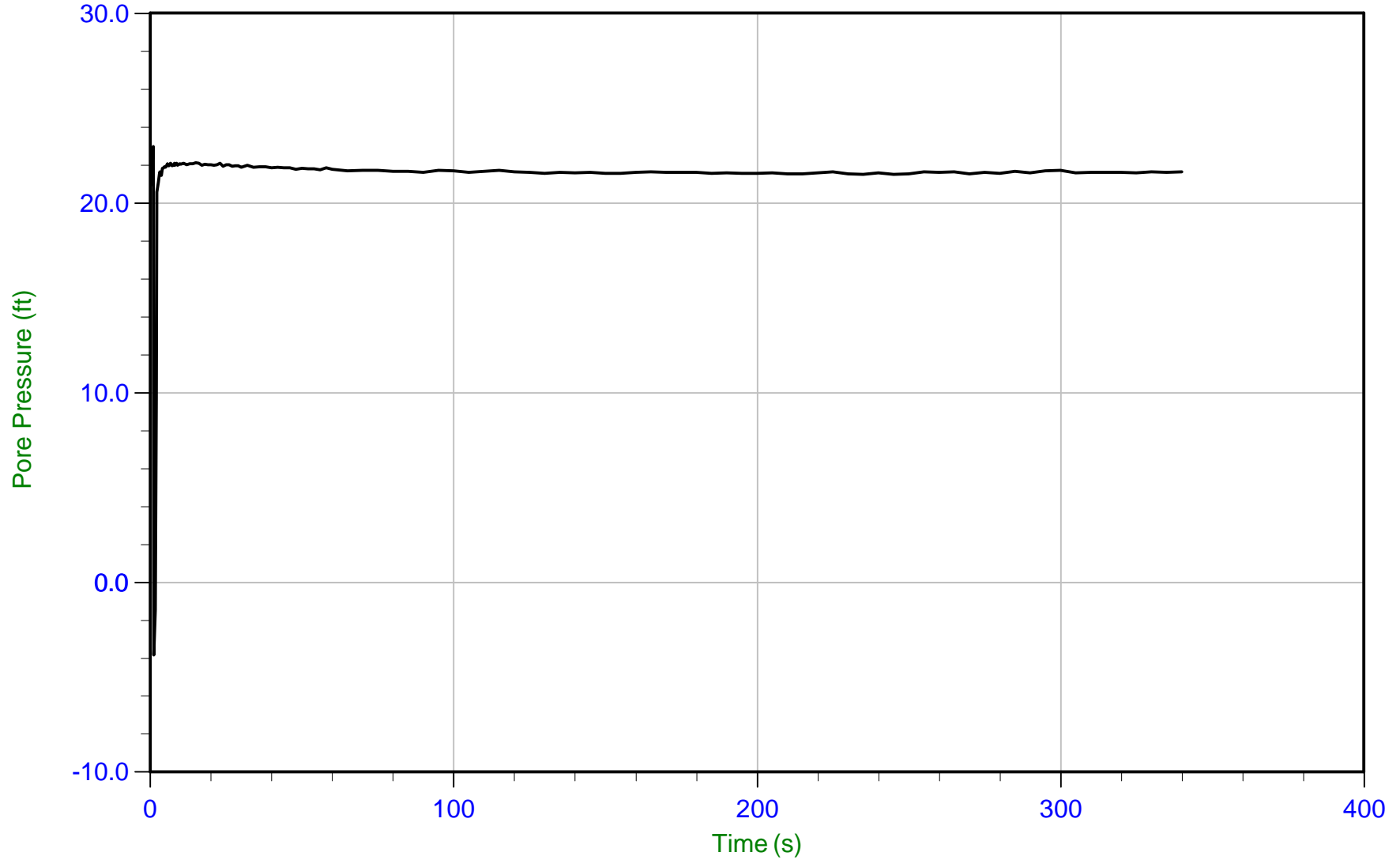
WT: 0.612 m / 2.009 ft  
Ueq: 8.3 ft



*Krazan*

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

Sounding: CPT-04  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

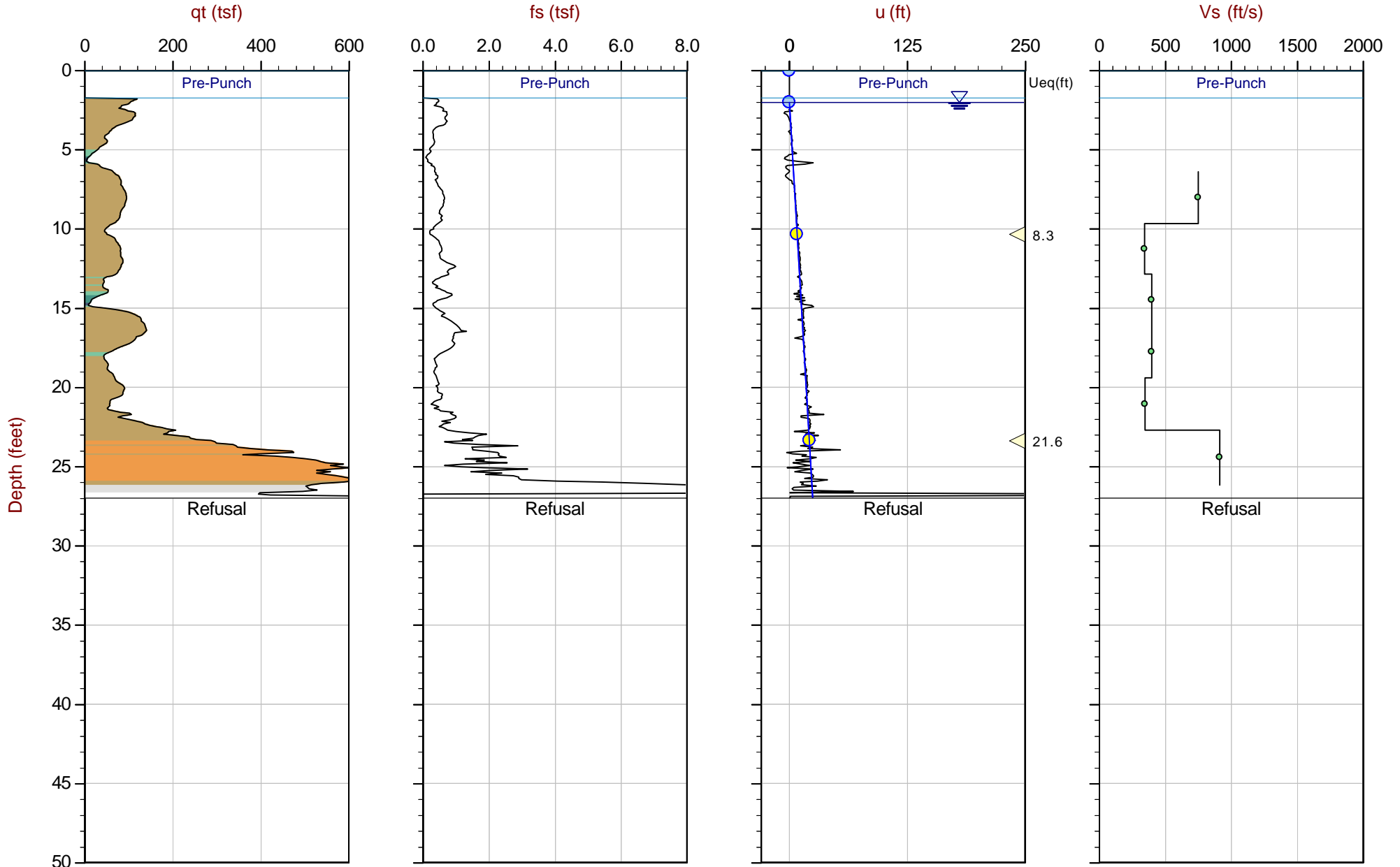


Trace Summary:

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



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

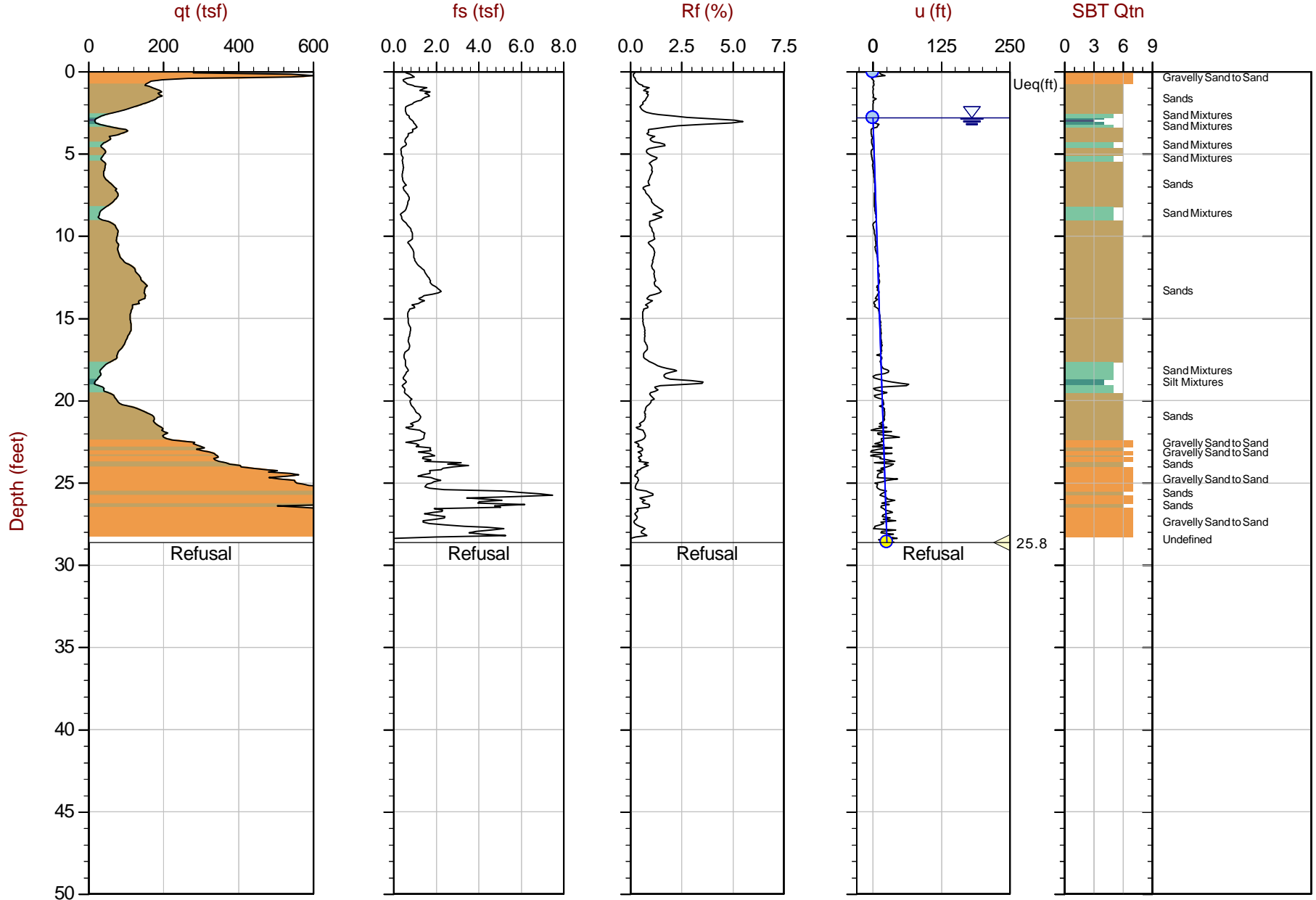
File: 22-59-23911\_SP04.COR  
 Unit Wt: SBTQtn(PKR2009)

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

● Equilibrium Pore Pressure ( $U_{eq}$ )    
 ● Assumed  $U_{eq}$     
 ◁ Dissipation,  $U_{eq}$  achieved    
 ◁ Dissipation,  $U_{eq}$  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.



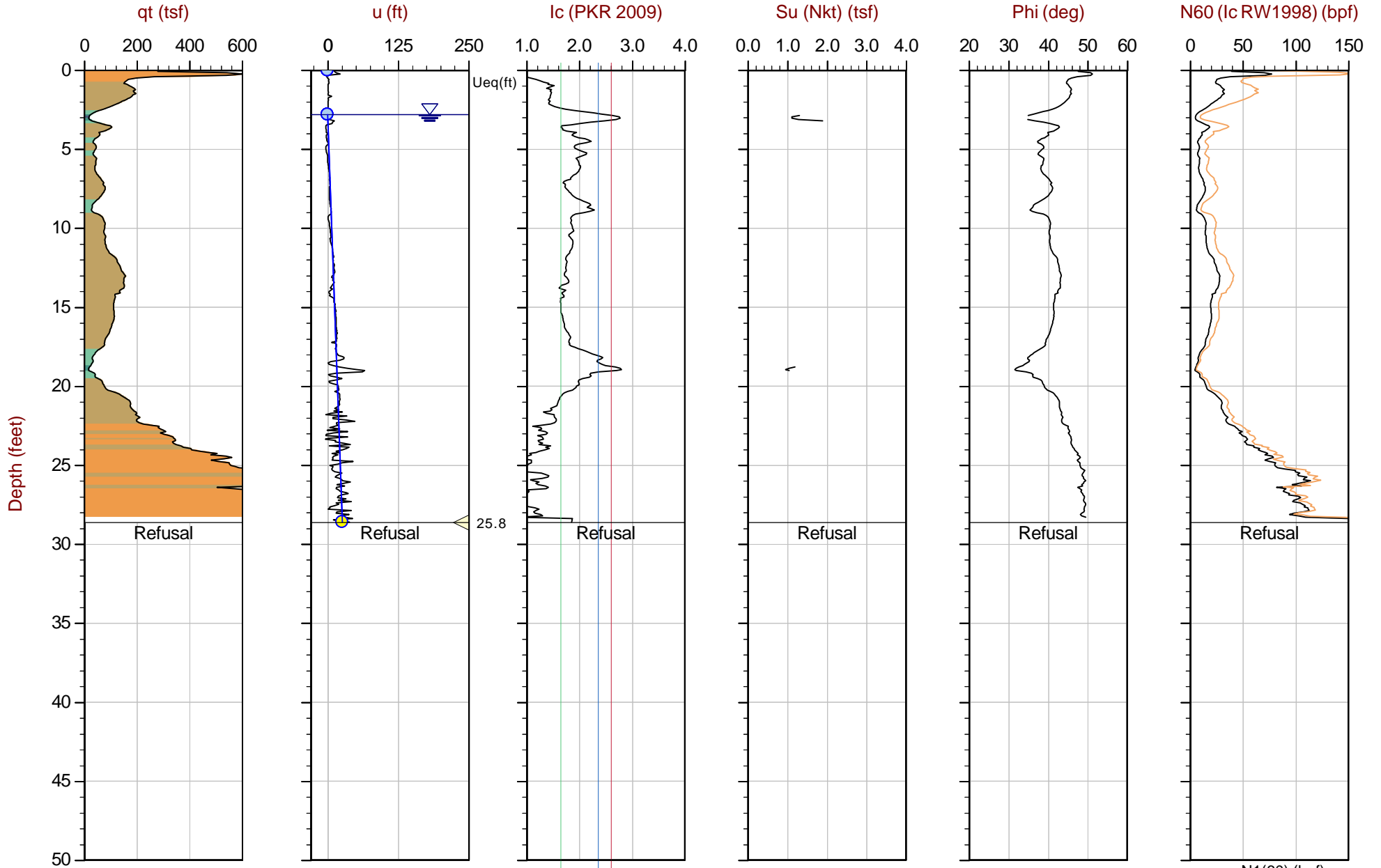


Max Depth: 8.725 m / 28.62 ft  
 Depth Inc: 0.025 m / 0.082 ft  
 Avg Int: Every Point

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.



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

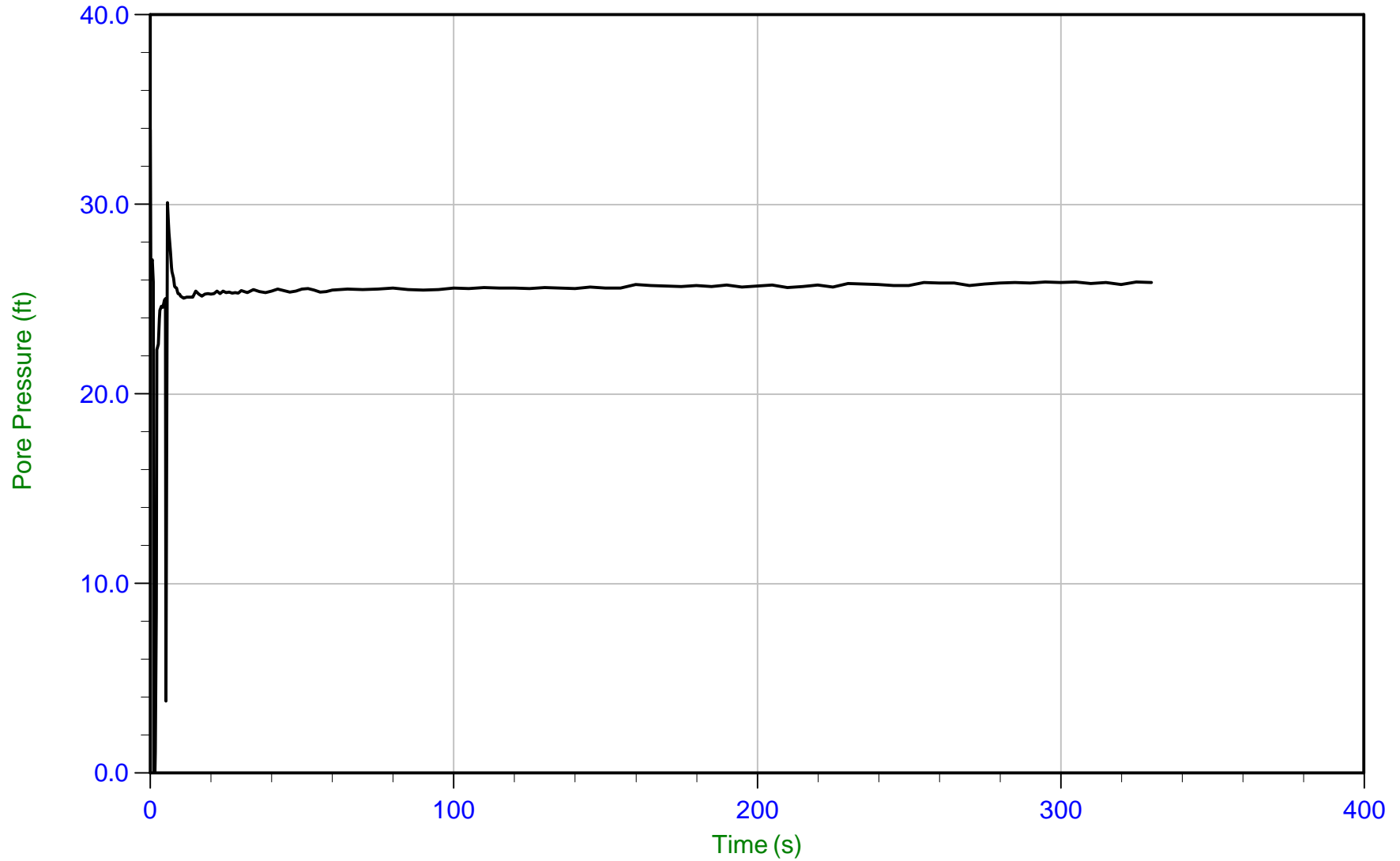
● Equilibrium Pore Pressure (Ueq)    
 ● Assumed Ueq    
 ◀ Dissipation, Ueq achieved    
 ◀ Dissipation, Ueq not achieved    
 — Hydrostatic Line  
 The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



*Krazan*

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

Sounding: CPT-05  
Cone: 781:T1500F15U35 Area=15 cm<sup>2</sup>

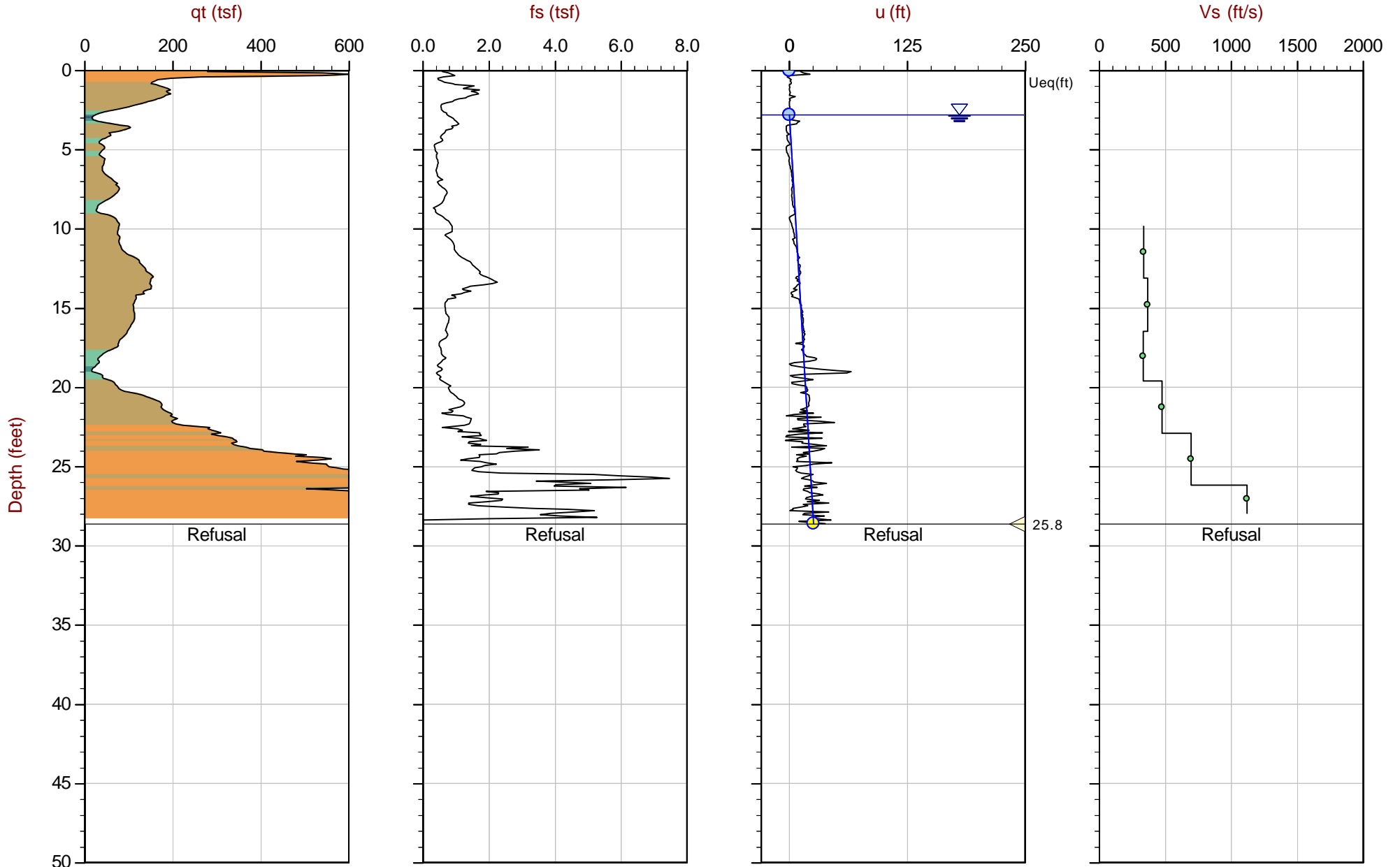


Trace Summary:

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



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 contract 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.