DRAFT - FOR LAND USE SUBMITTAL

STORMWATER DRAINAGE CONTROL CERTIFICATE

Multnomah County

Land Use Planning Division

www.multco.us/landuse ▪ Email: land.use.planning@multco.us ▪ Phone: (503) 988-3043

> 500 SQUARE FEET OF NEW / REPLACED IMPERVIOUS SURFACES

NOTE TO PROPERTY OWNER/APPLICANT: Please have an Oregon Licensed Professional Engineer fill out this Certificate and attach a signed site plan, stamped and signed storm water system details, and stamped and signed storm water calculations used to support the conclusion. Please note that replacement of existing structures does not provide a credit to the square footage threshold.

Property Address or Legal Description: 17645 NW St Helens Rd, Portland, OR 97231, Map & Tax Lot: 2N1W18D, 500

Description of Project: Treatment center with 70 beds

The following stormwater drainage control system will be required:

- ☐ Use of Gutter, downspout, and splash block drainage control system;
- ☐ Natural Infiltration Process; or
- ☑ Construction of an on-site storm water drainage control system.

The rate of stormwater runoff attributed to the new/replaced development for a 10-year/24-hour storm event will be no greater than that which existed prior to any development as measured from the property line or from the point of discharge into a water body with the use of the designated system [MCC 39.6235].

CONFLICT WITH CODE

I certify the attached ~~signed~~ site plan showing the areas needed for the chosen system type, ~~stamped and signed~~ storm water system design details, and ~~stamped and signed~~ calculations dated 02/20/26 **will meet the requirements listed above.**

Signature: Robert J. Léger

Print Name: Robert Léger

Business Name: DOWL

Address: 309 SW 6th Ave., Suite 700, Portland, OR 97204

Phone #: 971-280-8656

Email: rleger@dowl.com

Date: 02-20-2026

Engineer's Stamp Below:

REGISTERED PROFESSIONAL ENGINEER
61270PE
OREGON
JULY 15, 2003
ROBERT JAMES LÉGER

NOTE TO ENGINEER: Please check one box above. Multnomah County does not use the City of Portland's storm water ordinance. As part of your review, MCC 39.6235 requires that you must consider all new, replaced, and existing structures and impervious areas and determine that the newly generated stormwater from the new or replaced impervious surfaces is in compliance with Multnomah County Code for a 10-year/24-hour storm event. This Storm Water Drainage Control Certificate does not apply to shingle or roof replacement on lawfully established structures.

Exhibit A.4

§ 39.6235 STORMWATER DRAINAGE CONTROL.

(A) Persons creating new or replacing existing impervious surfaces exceeding 500 square feet shall install a stormwater drainage system as provided in this section. This subsection (A) does not apply to shingle or roof replacement on lawful structures.

(B) The provisions of this section are in addition to and not in lieu of any other provision of the code regulating stormwater or its drainage and other impacts and effects, including but not limited to regulation thereof in the SEC overlay.

(C) The provisions of this section are in addition to and not in lieu of stormwater and drainage requirements in the Multnomah County Road Rules and Design and Construction Manual, including those requirements relating to impervious surfaces and proposals to discharge stormwater onto a county right-of-way.

(D) The stormwater drainage system required in subsection (A) shall be designed to ensure that the rate of runoff for the 10-year 24-hour storm event is no greater than that which existed prior to development at the property line or point of discharge into a water body.

(E) At a minimum, to establish satisfaction of the standards in this section and all other applicable stormwater-related regulations in this code, the following information must be provided to the planning director:

> (1) A site plan drawn to scale, showing the property line locations, ground topography (contours), boundaries of all ground disturbing activities, roads and driveways, existing and proposed structures and buildings, existing and proposed sanitary tank and drainfields (primary and reserve), location of stormwater disposal, trees and vegetation proposed for both removal and planting and an outline of wooded areas, water bodies and existing drywells;
>
> (2) Documentation establishing approval of any new stormwater surcharges to a sanitary drainfield by the City of Portland Sanitarian and/or any other agency authorized to review waste disposal systems;
>
> (3) Certified statement, and supporting information and documentation, by an Oregon licensed Professional Engineer that the proposed or existing stormwater drainage system satisfies all standards set forth in this section and all other stormwater drainage system standards in this code; and
>
> (4) Any other report, information, plan, certification or documentation necessary to establish satisfaction of all standards set forth in this section and all other applicable stormwater-related regulations in this code, such as, but not limited to, analyses and explanations of soil characteristics, engineering solutions, and proposed stream and upland environmental protection measures.

DOWL

MEMORANDUM

TO: Multnomah County Engineering
FROM: DOWL
DATE: 02/20/2026
SUBJECT: PA-2025-0007 – NARA NW – Stormwater Design

To Whom it May Concern,

Please find below a preliminary summary of the stormwater design for the proposed NARA NW redevelopment project. This project will remove the existing residential treatment facility and surrounding surfacing and construct a new building, parking, and utilities to serve the property.

Detention analyses were performed using HydroCAD 10.20-5c utilizing the Santa Barbara Urban Hydrograph (SBUH) method. The system has been evaluated for its compliance with the standards described in *Multnomah County Zoning Code (MCZC) Chapter 39.*

Existing Conditions

The 11.13-acre project site is situated at the south corner of the intersection of Route 30 and NW Cornelius Pass Road. The existing NARA building roof and surrounding parking lot drain undetained to an existing 16” outfall to McCarthy Creek, in the northwest corner of the site. McCarthy Creek flows into the Multnomah Channel, and ultimately to the Columbia River.

The existing property has more than 90 feet of drop across it, from the southern forested hills to the creek bed where it flows under Route 30. However, the majority of the project area is predominantly flat, with slopes of 0-2% generally draining to the north, as the property was heavily graded when originally developed. Per the NRCS Soil Survey (see appendix B), project area soils are predominantly Burlington fine sandy loam and Haploxerolls, with limited Quatamata loam deposits along the southern property line. A March 2023 geotechnical report by Columbia West (see appendix F) indicates that groundwater can be expected as shallow as 17 feet below ground level, and that infiltration rates, while not directly measured, were anticipated to be low despite the Hydrologic Soil Group rating of A.

Basin Areas:

Per MCZC, developed stormwater runoff rates shall be compared to those of the site prior to development. However, the Multnomah County Stormwater Certificate notes “prior to **any** development” (**emphasis** added), or the “undeveloped” condition, which conflicts with the MCZC. Meting stormwater rates prior to any development is a significant challenge for this property, and we **request clarification** on this matter. For this submittal, calculations were performed for the undeveloped site. The detention system required for an undeveloped site as the pre-development condition is large, and depends on adequate soil infiltration. This may or may not be available, due to varying soils found in the geotechnical study.

As noted above, the site today has been heavily graded during development. It is believed that prior to development, the site was likely hilly forest, similar to the forests that surround the site today. To model this condition, the existing development area perimeter is modelled to match the surrounding grades and graded to gently slope across the site – rather than the flat, stepped condition that exists today as a result of historical development.

For the purposes of this report, the proposed stormwater design considers only the limits of the project area and the portions of the property that directly drain into it, a total area of 140,189 sf (3.22 acres).

The existing site is modelled in two basins. The first basin is the steep, forested, type-C soil hill on the south property line, which drains into the project area. The second basin is the remainder of the “undeveloped” project area, with type A soil and assumed grades as described above. These basins are modelled with curve numbers of 70 and 30, respectively, and a time of concentration of 15.6 minutes. See provided appendices A and D for basin maps and time of concentration calculations.

The proposed development is similarly modelled with two basin areas: the forested southern hillside, and the development area. The development area is assumed to be 80% impervious, with the remainder of the area being landscaped areas over type A soils. The development area is modelled with a conservative minimum time of concentration of 5 minutes.

Design Storm:

Per section 39.6235 of the MCZC, the project must match the post-development peak rate runoff rate of the 10-yr 24-hr storm to that of the pre-development condition. Per ODOT GIS data (see appendix C) the 10-yr 24-hr storm at the property is 3.06 inches.

Analysis:

The existing undeveloped basin areas produce a total peak runoff rate of 0.04cfs for the 10-yr 24-hr storm; the proposed stormwater detention system is designed to match this flow rate.

To meet the undeveloped pre-development condition flow rate, the project would need 88 StormTech MC-3500 chambers with 12” of stone above and below the chambers and a flow control manhole on the downstream side of the facility. The chamber system is designed with an open bottom to allow for infiltration into native soils. With limited data, the design infiltration rate of the existing onsite type A soil is conservatively assumed to be 0.5 in/hr. This assumption will be confirmed later in the design process, and the detention system will be resized accordingly.

A flow control manhole downstream of the detention facility meters flow leaving the system. It consists of a 1-3/8” orifice set 9” below the top of the chambers, and an overflow at the top of the stone layer above the chambers to allow for larger storm events to leave the system. The flow control orifice is intentionally set high in the chamber to allow for extended storage of the design storm, allowing for onsite retention despite assumed low infiltration rates.

The proposed basin areas, detained by the above-described system, produce a total peak runoff rate of 0.04cfs for the 10-yr 24-hr storm, which matches the undeveloped pre-development condition flow rate.

For all calculations, see provided HydroCAD report in appendix E.

Conclusion:

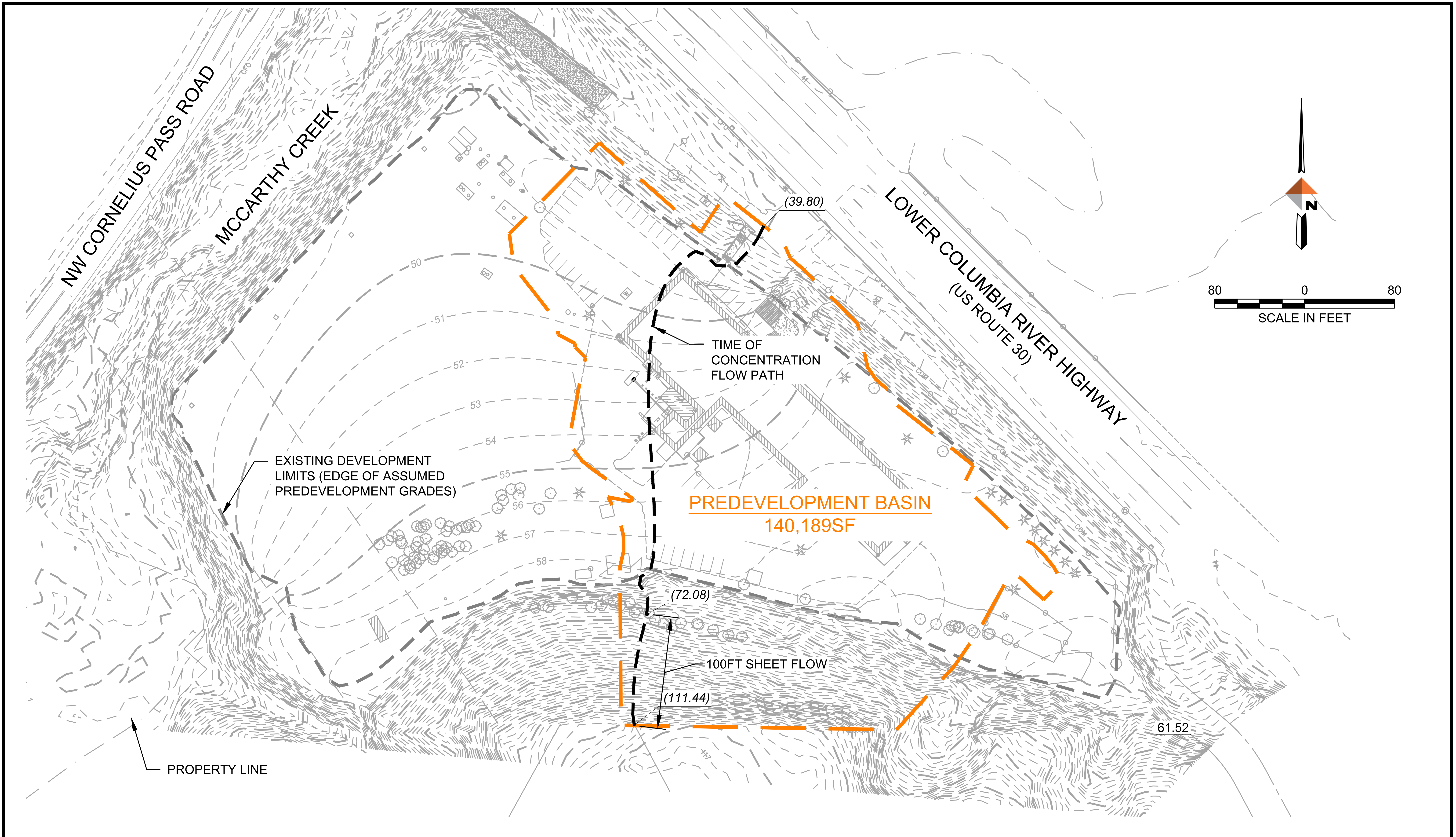
We believe this memo adequately summarizes our proposed approach to meet the detention requirements noted in the Stormwater Certificate, which conflicts with the MCZC. We **request clarification on this code requirement prior to construction plan submittal.** For this submittal, calculations were performed for the undeveloped site.

Please feel free to reach out directly with any questions.
Thank you,

Robert Leger, PE

APPENDIX A:

BASIN MAPS



J:\80\16023-01\65CAD\EXHIBITS\2026-02-13 - prelim storm study\existing basin.dwg PLOT DATE 2026-2-19 15:38 SAVED DATE 2026-02-19 15:38 USER: cguillaume

DOWL
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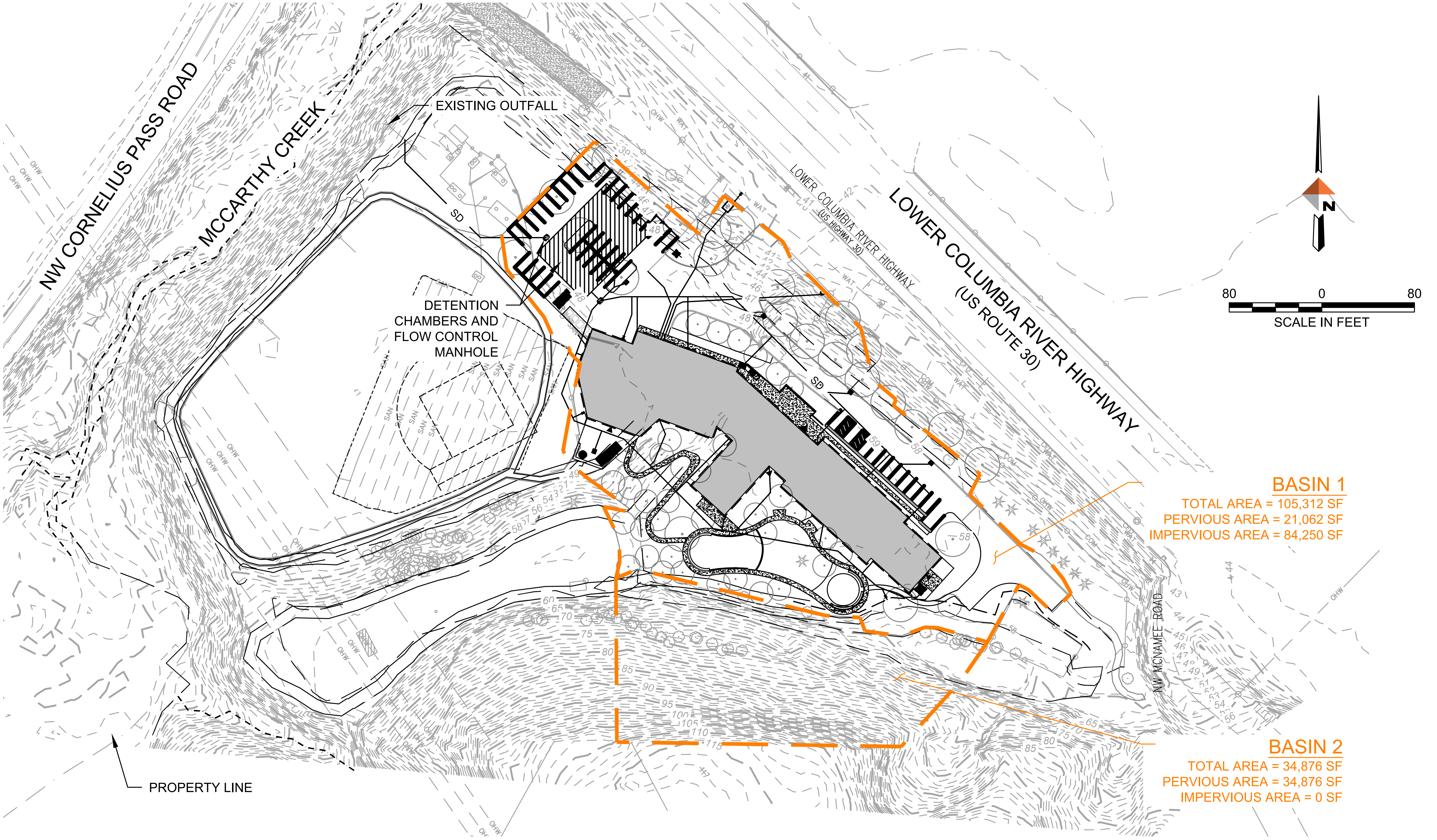
309 SW 6th Avenue, Suite 700
Portland, Oregon 97204
971-280-8641

NARA NW

EXISTING BASIN MAP

PROJECT 16023
DATE 02-20-2026

FIGURE 1



BASIN 1
TOTAL AREA = 105,312 SF
PERVIOUS AREA = 21,062 SF
IMPERVIOUS AREA = 84,250 SF

BASIN 2
TOTAL AREA = 34,876 SF
PERVIOUS AREA = 34,876 SF
IMPERVIOUS AREA = 0 SF

DOWL
WWW.DOWL.COM
309 SW 6th Avenue, Suite 700
Portland, Oregon 97204
971-280-8641

NARA NW

PROPOSED BASIN MAP

PROJECT	16023
DATE	02-20-2026

FIGURE 2

J:\80\16023-01\65CAD\EXHIBITS\2026-02-13 - prelim storm study\proposed basin.dwg PLOT DATE 2026-2-19 15:39 SAVED DATE 2026-02-19 08:19 USER: cguillaume

APPENDIX B:
NRCS SOIL REPORT

Hydrologic Soil Group—Multnomah County Area, Oregon

Map Scale: 1:1,290 if printed on A landscape (11" x 8.5") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)

▢ Area of Interest (AOI)

Soils

Soil Rating Polygons

■ A

■ A/D

■ B

■ B/D

■ C

■ C/D

■ D

▢ Not rated or not available

Soil Rating Lines

〰 A

〰 A/D

〰 B

〰 B/D

〰 C

〰 C/D

〰 D

〰 Not rated or not available

Soil Rating Points

■ A

■ A/D

■ B

■ B/D

■ C

■ C/D

■ D

▣ Not rated or not available

Water Features

〰 Streams and Canals

Transportation

┼┼┼ Rails

〰 Interstate Highways

〰 US Routes

〰 Major Roads

〰 Local Roads

Background

■ Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Multnomah County Area, Oregon
Survey Area Data: Version 24, Sep 9, 2025

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 1, 2024—Jul 1, 2024

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
6B	Burlington fine sandy loam, 0 to 8 percent slopes	A	2.1	66.3%
19E	Haploxerolls, steep	A	0.6	20.1%
37B	Quatama loam, 3 to 8 percent slopes	C	0.3	10.5%
37C	Quatama loam, 8 to 15 percent slopes	C	0.1	3.2%
Totals for Area of Interest			**3.2**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

APPENDIX C:

ODOT STORM MAP

10-Year 24-Hour Precipitation (in)

Pixel Value	3.06
Class value	4

2/19/2026

10-Year 24-Hour Precipitation (in)

- ▨ Less than 1.50
- ▨ 1.50 - 2.00
- ▨ 2.00 - 2.50
- ▨ 2.50 - 3.00
- ▨ 3.00 - 3.50
- ▨ 3.50 - 4.00
- ▨ 4.00 - 4.50
- ▨ 4.50 - 5.00
- ▨ 5.00 - 6.00
- ▨ 6.00 - 7.00

1:7,425

0 0.04 0.09 0.17 mi

0 0.05 0.1 0.2 km

Source: Esri, Vantor, Earthstar Geographics, and the GIS User Community,
Oregon Department of Transportation, Geographic Information Services Unit

APPENDIX D:

TIME OF CONCENTRATION

Time of Concentration

Based on *the TR-55 by the USDA SCS, Second Edition, June 1986*

SHEET FLOW

Less than 100' Length

$T_c = T_{t1} + T_{t2} + \ldots + T_n$ Time of Concentration

$$T_t = (0.42 \cdot (\eta \cdot L)^{0.8}) / (P_2^{0.5} \cdot S^{0.4})$$

Time of Travel T_{t1} = 13.11 minutes

Length of Flow L = 100 ft

Manning's value η = 0.80

Intensity of storm I = 2.40 in

Slope of Surface S = 0.3936 ft/ft

High Point Elevation = 111.44 ft

Low Point Elevation = 72.08 ft

SHALLOW CONCENTRATED FLOW

$$T_t = L / 60 \cdot v$$

Time of Travel T_{t2} = 2.51 minutes

Velocity v = 3.8 fps TR-55 Figure 3-1

Length of Flow L = 571.9 ft

Slope of Surface S = 0.0564 ft/ft

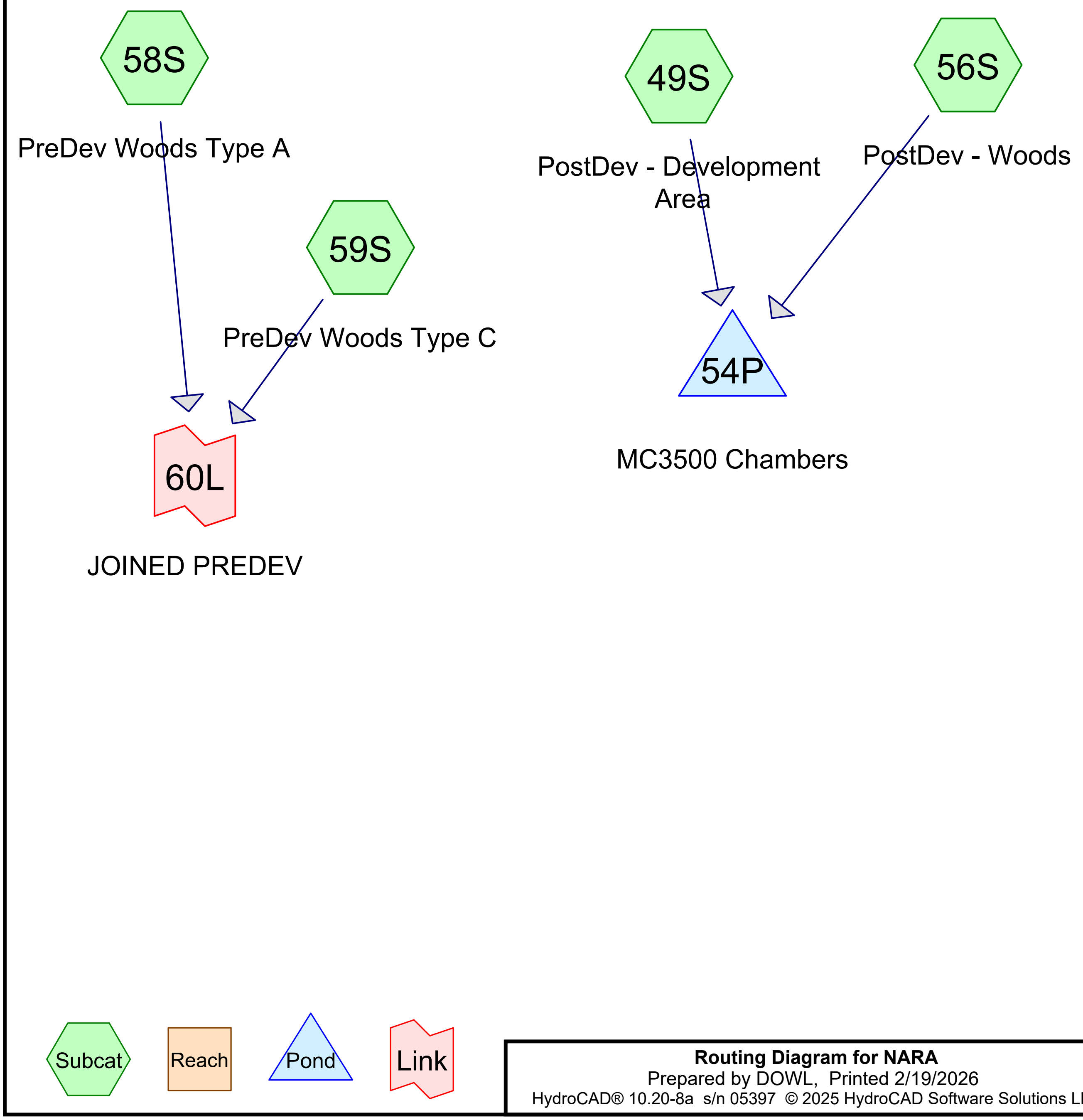
High Point Elevation = 72.08 ft

Low Point Elevation = 39.80 ft

$T_c = T_{t1} + T_{t2}$ T_c = 15.62 minutes

APPENDIX E:

HYDROCAD REPORT



Routing Diagram for NARA
Prepared by DOWL, Printed 2/19/2026
HydroCAD® 10.20-8a s/n 05397 © 2025 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
0.484	39	>75% Grass cover, Good, HSG A (49S)
1.934	98	Paved parking, HSG A (49S)
3.144	30	Woods, Good, HSG A (56S, 58S)
0.875	70	Woods, Good, HSG C (56S, 59S)
6.437	**57**	**TOTAL AREA**

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
5.561	HSG A	49S, 56S, 58S
0.000	HSG B	
0.875	HSG C	56S, 59S
0.000	HSG D	
0.000	Other	
6.437		**TOTAL AREA**

Ground Covers (all nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.484	0.000	0.000	0.000	0.000	0.484	>75% Grass cover, Good	49S
1.934	0.000	0.000	0.000	0.000	1.934	Paved parking	49S
3.144	0.000	0.875	0.000	0.000	4.019	Woods, Good	56S, 58S, 59S
5.561	**0.000**	**0.875**	**0.000**	**0.000**	**6.437**	**TOTAL AREA**	

Time span=0.00-160.00 hrs, dt=0.01 hrs, 16001 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

```
Subcatchment49S: PostDev -            Runoff Area=105,312 sf   80.00% Impervious   Runoff Depth=2.26"
                                                Tc=5.0 min   CN=39/98   Runoff=1.38 cfs  0.456 af

Subcatchment56S: PostDev - Woods        Runoff Area=34,876 sf   0.00% Impervious   Runoff Depth=0.14"
                                               Tc=15.9 min   CN=52/0   Runoff=0.01 cfs  0.009 af

Subcatchment58S: PreDev Woods Type A   Runoff Area=121,123 sf   0.00% Impervious   Runoff Depth=0.00"
                                               Tc=15.6 min   CN=30/0   Runoff=0.00 cfs  0.000 af

Subcatchment59S: PreDev Woods Type C    Runoff Area=19,066 sf   0.00% Impervious   Runoff Depth=0.75"
                                               Tc=15.6 min   CN=70/0   Runoff=0.04 cfs  0.027 af

Pond 54P: MC3500 Chambers               Peak Elev=45.96'  Storage=0.344 af   Inflow=1.38 cfs  0.465 af
                      Discarded=0.06 cfs  0.437 af   Primary=0.04 cfs  0.028 af   Outflow=0.10 cfs  0.465 af

Link 60L: JOINED PREDEV                                                      Inflow=0.04 cfs  0.027 af
                                                                            Primary=0.04 cfs  0.027 af

           Total Runoff Area = 6.437 ac   Runoff Volume = 0.493 af   Average Runoff Depth = 0.92"
                                      69.95% Pervious = 4.502 ac     30.05% Impervious = 1.934 ac
```

Summary for Subcatchment 49S: PostDev - Development Area

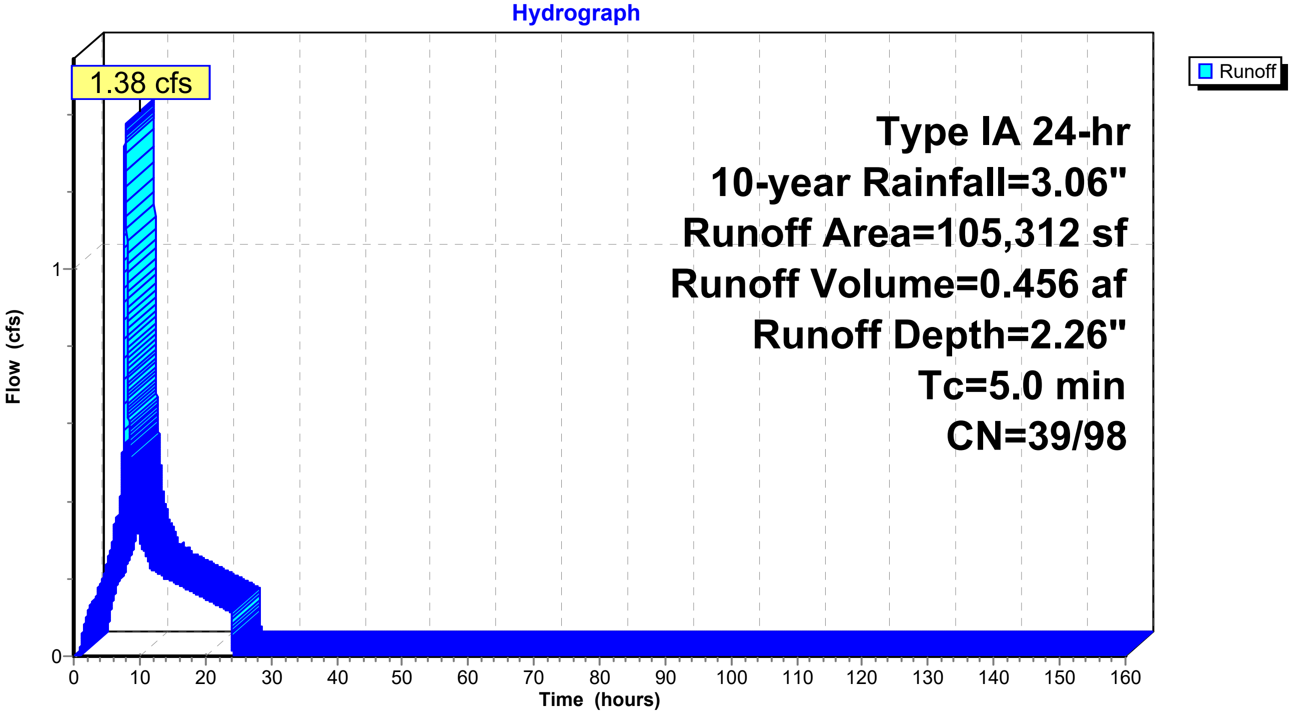
Runoff = 1.38 cfs @ 7.88 hrs, Volume= 0.456 af, Depth= 2.26"
Routed to Pond 54P : MC3500 Chambers

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-160.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.06"

Area (sf)	CN	Description
84,250	98	Paved parking, HSG A
21,062	39	>75% Grass cover, Good, HSG A
105,312	86	Weighted Average
21,062	39	20.00% Pervious Area
84,250	98	80.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry,**

Subcatchment 49S: PostDev - Development Area



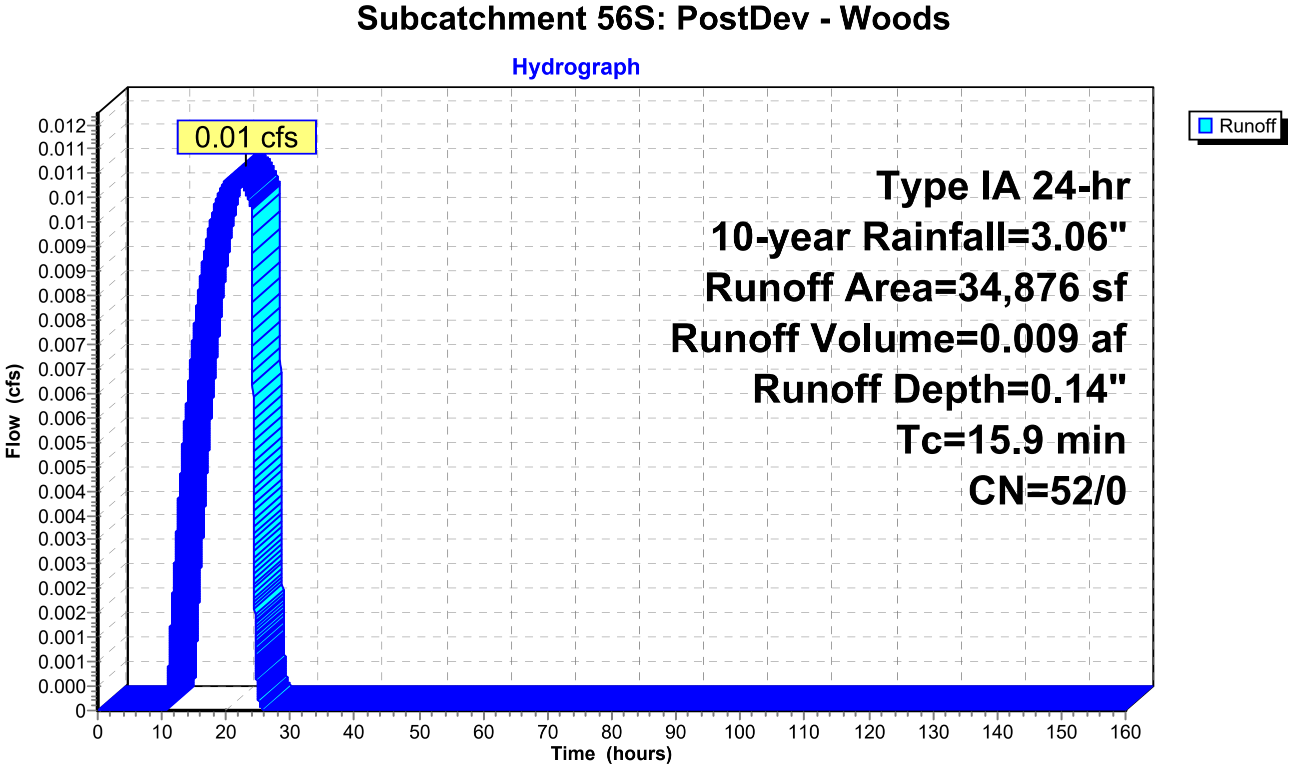
Summary for Subcatchment 56S: PostDev - Woods

Runoff = 0.01 cfs @ 20.87 hrs, Volume= 0.009 af, Depth= 0.14"
Routed to Pond 54P : MC3500 Chambers

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-160.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.06"

Area (sf)	CN	Description
19,066	70	Woods, Good, HSG C
15,810	30	Woods, Good, HSG A
34,876	52	Weighted Average
34,876	52	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
15.9					**Direct Entry,**



Summary for Subcatchment 58S: PreDev Woods Type A

[45] Hint: Runoff=Zero

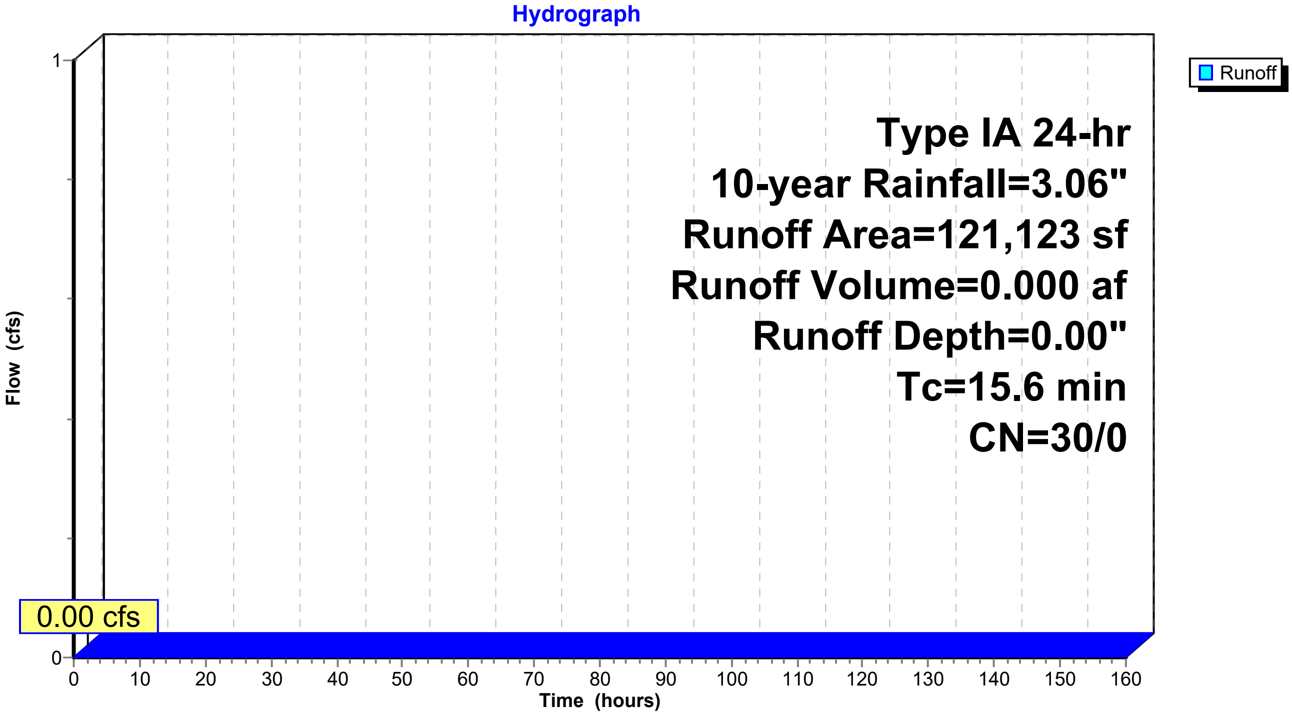
Runoff = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af, Depth= 0.00"
Routed to Link 60L : JOINED PREDEV

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-160.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.06"

Area (sf)	CN	Description
121,123	30	Woods, Good, HSG A
121,123	30	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
15.6					**Direct Entry,**

Subcatchment 58S: PreDev Woods Type A



Summary for Subcatchment 59S: PreDev Woods Type C

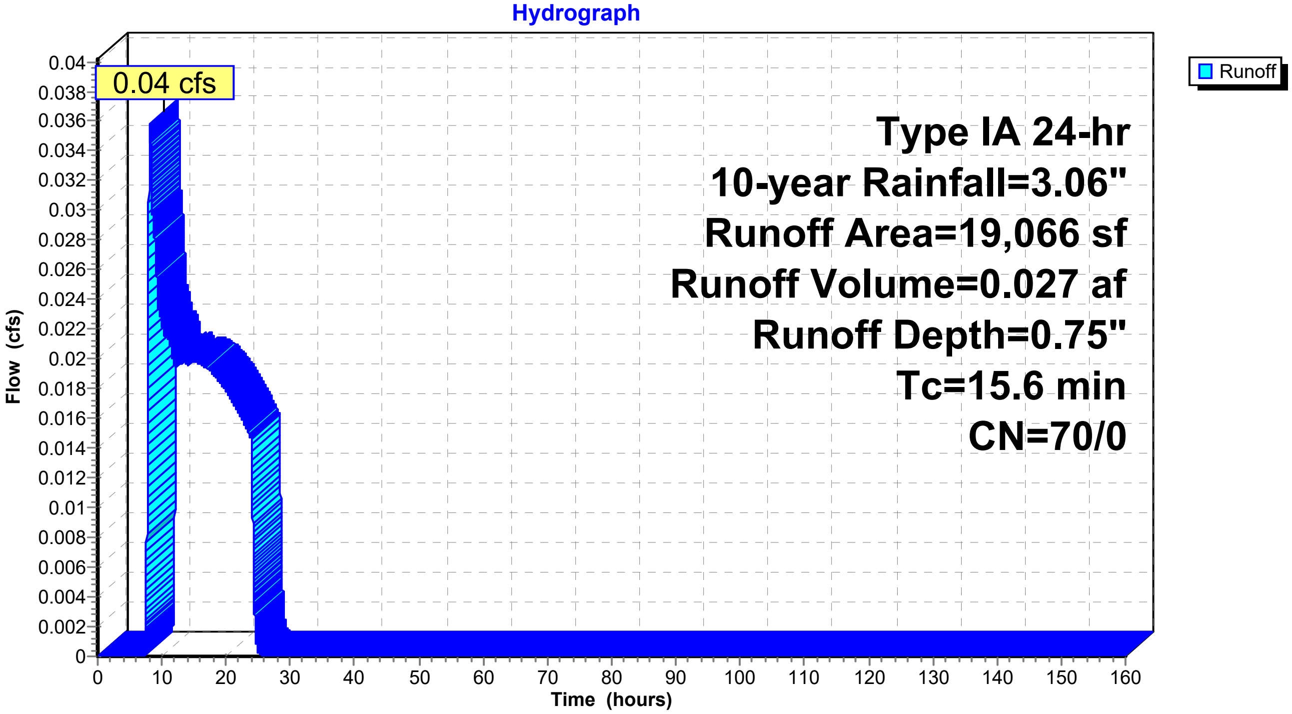
Runoff = 0.04 cfs @ 8.06 hrs, Volume= 0.027 af, Depth= 0.75"
Routed to Link 60L : JOINED PREDEV

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-160.00 hrs, dt= 0.01 hrs
Type IA 24-hr 10-year Rainfall=3.06"

Area (sf)	CN	Description
19,066	70	Woods, Good, HSG C
19,066	70	100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
15.6					**Direct Entry,**

Subcatchment 59S: PreDev Woods Type C



Summary for Pond 54P: MC3500 Chambers

Inflow Area = 3.218 ac, 60.10% Impervious, Inflow Depth = 1.73" for 10-year event
Inflow = 1.38 cfs @ 7.88 hrs, Volume= 0.465 af
Outflow = 0.10 cfs @ 24.03 hrs, Volume= 0.465 af, Atten= 93%, Lag= 968.9 min
Discarded = 0.06 cfs @ 1.85 hrs, Volume= 0.437 af
Primary = 0.04 cfs @ 24.03 hrs, Volume= 0.028 af
Routed to nonexistent node 57L

Routing by Stor-Ind method, Time Span= 0.00-160.00 hrs, dt= 0.01 hrs
Peak Elev= 45.96' @ 24.03 hrs Surf.Area= 0.110 ac Storage= 0.344 af

Plug-Flow detention time= 2,172.2 min calculated for 0.465 af (100% of inflow)
Center-of-Mass det. time= 2,172.2 min (2,847.9 - 675.7)

Volume	Invert	Avail.Storage	Storage Description
#1A	41.25'	0.163 af	**56.83'W x 84.57'L x 5.75'H Field A** 0.634 af Overall - 0.228 af Embedded = 0.407 af x 40.0% Voids
#2A	42.25'	0.228 af	**ADS_StormTech MC-3500 d +Cap** x 88 Inside #1 Effective Size= 70.4"W x 45.0"H => 15.33 sf x 7.17'L = 110.0 cf Overall Size= 77.0"W x 45.0"H x 7.50'L with 0.33' Overlap 88 Chambers in 8 Rows Cap Storage= 14.9 cf x 2 x 8 rows = 238.4 cf
		0.390 af	Total Available Storage

Storage Group A created with Chamber Wizard

Device	Routing	Invert	Outlet Devices
#1	Primary	45.25'	**1.375" Vert. Orifice/Grate** C= 0.620 Limited to weir flow at low heads
#2	Primary	46.00'	**4.0' long Sharp-Crested Rectangular Weir** 2 End Contraction(s)
#3	Discarded	41.25'	**0.500 in/hr Exfiltration over Horizontal area**

Discarded OutFlow Max=0.06 cfs @ 1.85 hrs HW=41.31' (Free Discharge)
3=Exfiltration (Exfiltration Controls 0.06 cfs)

Primary OutFlow Max=0.04 cfs @ 24.03 hrs HW=45.96' (Free Discharge)
1=Orifice/Grate (Orifice Controls 0.04 cfs @ 4.01 fps)
2=Sharp-Crested Rectangular Weir (Controls 0.00 cfs)

Pond 54P: MC3500 Chambers - Chamber Wizard Field A

Chamber Model = ADS_StormTechMC-3500 d +Cap (ADS StormTech®MC-3500 d rev 03/14 with Cap volume)
Effective Size= 70.4"W x 45.0"H => 15.33 sf x 7.17'L = 110.0 cf
Overall Size= 77.0"W x 45.0"H x 7.50'L with 0.33' Overlap
Cap Storage= 14.9 cf x 2 x 8 rows = 238.4 cf

77.0" Wide + 6.0" Spacing = 83.0" C-C Row Spacing

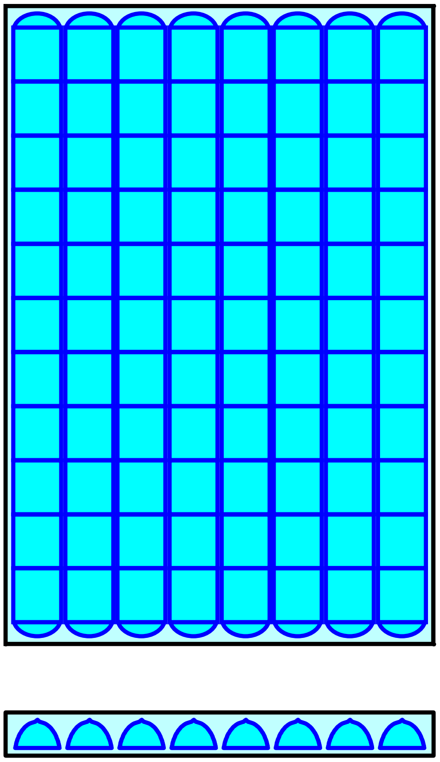
11 Chambers/Row x 7.17' Long +1.85' Cap Length x 2 = 82.57' Row Length +12.0" End Stone x 2 = 84.57' Base Length
8 Rows x 77.0" Wide + 6.0" Spacing x 7 + 12.0" Side Stone x 2 = 56.83' Base Width
12.0" Stone Base + 45.0" Chamber Height + 12.0" Stone Cover = 5.75' Field Height

88 Chambers x 110.0 cf + 14.9 cf Cap Volume x 2 x 8 Rows = 9,914.2 cf Chamber Storage

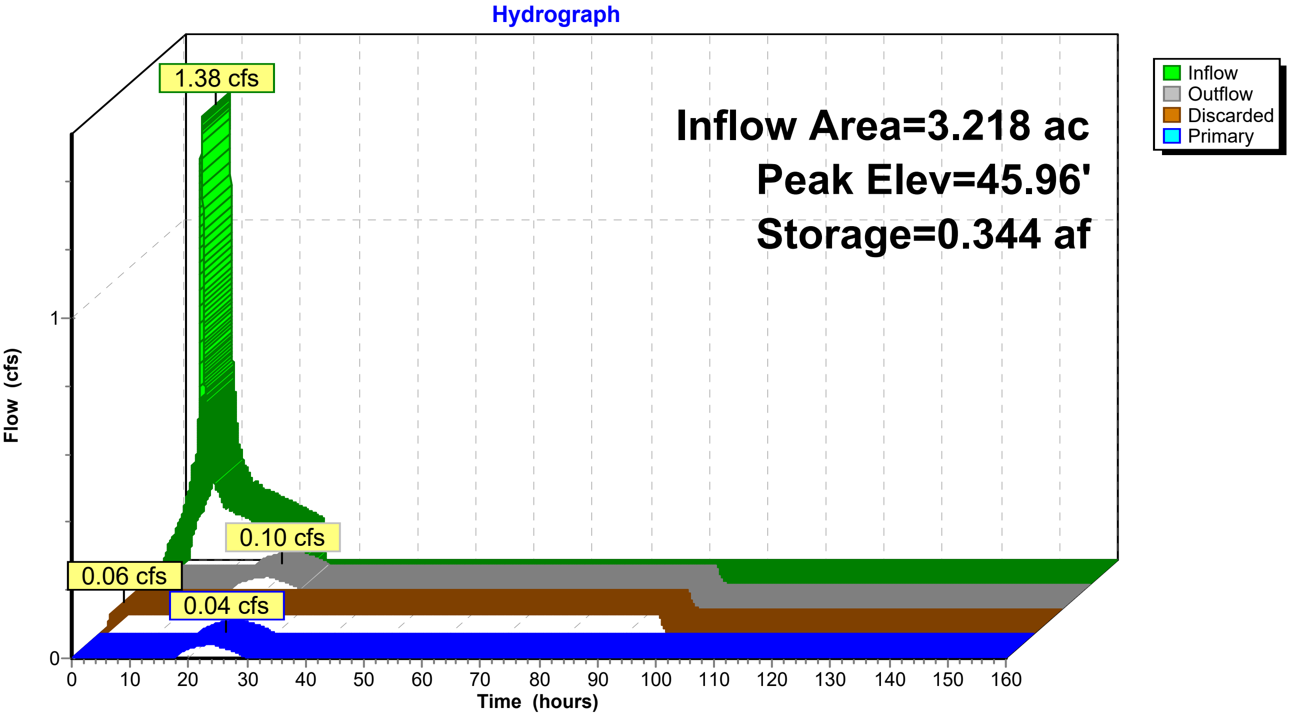
27,636.8 cf Field - 9,914.2 cf Chambers = 17,722.6 cf Stone x 40.0% Voids = 7,089.0 cf Stone Storage

Chamber Storage + Stone Storage = 17,003.2 cf = 0.390 af
Overall Storage Efficiency = 61.5%
Overall System Size = 84.57' x 56.83' x 5.75'

88 Chambers
1,023.6 cy Field
656.4 cy Stone



Pond 54P: MC3500 Chambers

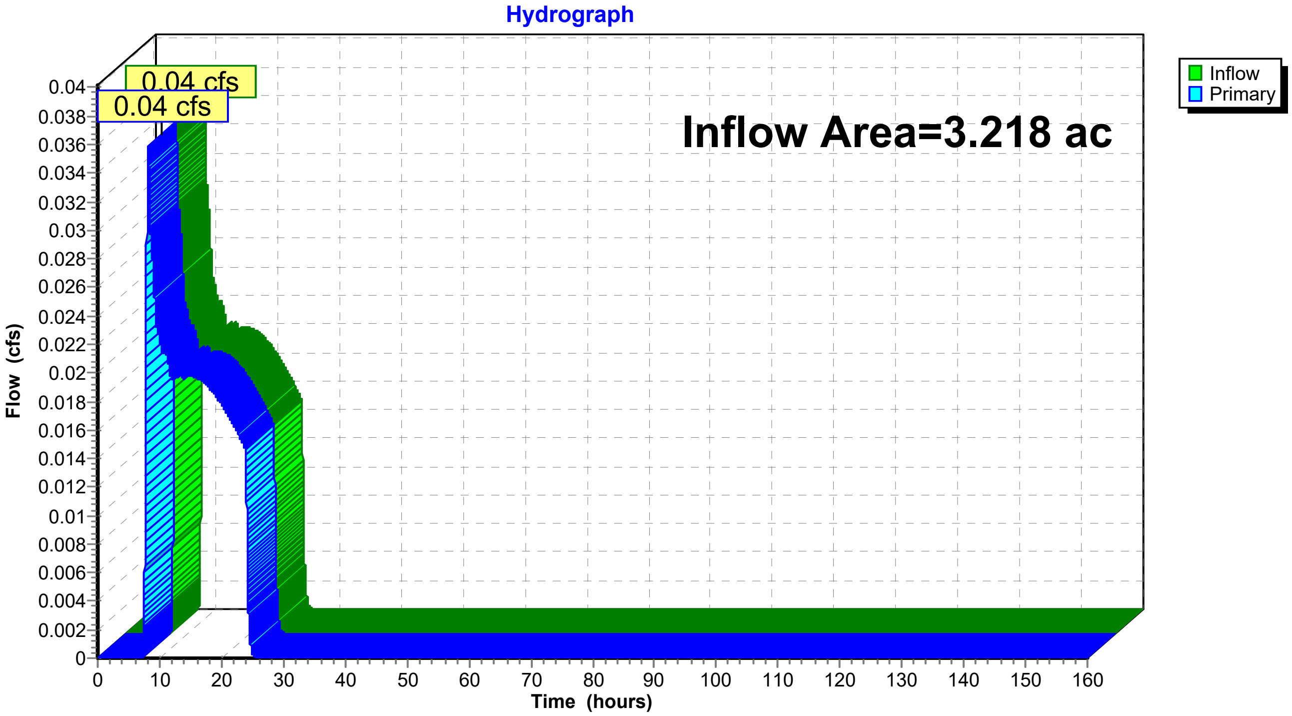


Summary for Link 60L: JOINED PREDEV

Inflow Area = 3.218 ac, 0.00% Impervious, Inflow Depth = 0.10" for 10-year event
Inflow = 0.04 cfs @ 8.06 hrs, Volume= 0.027 af
Primary = 0.04 cfs @ 8.06 hrs, Volume= 0.027 af, Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-160.00 hrs, dt= 0.01 hrs

Link 60L: JOINED PREDEV



APPENDIX F:

GEOTECHNICAL REPORT

Geotechnical Site Investigation

NARA Residential Treatment Center

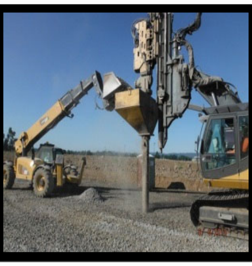
Portland, Oregon

March 1, 2023

Geotechnical ■ Environmental ■ Special Inspections

Columbia West Engineering, Inc

11917 NE 95th Street
Vancouver, Washington
98682
Phone: 360-823-2900



www.columbiawestengineering.com

GEOTECHNICAL SITE INVESTIGATION
NARA RESIDENTIAL TREATMENT CENTER
FEASIBILITY STUDY
PORTLAND, OREGON

Prepared For:

Native American Rehab Association of the Northwest, Inc.
Attn: Joey Phelan
PO Box 1569
Portland, Oregon 97207

Site Location:

17645 NW St Helens Road
Portland, Oregon

Prepared By:

Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
Phone: 360-823-2900
Work Order Number 23020

Date Prepared: **March 1, 2023**

EXECUTIVE SUMMARY

This executive summary presents the primary geotechnical considerations associated with the proposed NARA Residential Treatment Center project located in Portland, Oregon. Our conclusions and recommendations are based upon the subsurface information presented in this report and proposed development information provided by the design team. Detailed discussion of the geotechnical considerations summarized here is presented in respective sections of the report.

- Based on subsurface exploration and testing, site soils are not susceptible to do liquefaction. The buildings can therefore be supported on conventional spread footings bearing on firm native soil or engineered structural fill underlain by firm native soil.
- To reduce the risk of slope instability, structures, stormwater facilities, and structural fill placement should be avoided upslope and cuts avoided downslope within the geotechnical buffers identified on Figure 2 unless a slope stability analysis determines that the slope has an adequate factor of safety against slope failure. Additional discussion is presented in Section 5.3, *Landslide Hazard.*
- Although not observed in the explorations, geologic mapping indicates undocumented fill may be present at the site. Foundations should not be supported on undocumented fill and should be completely removed if encountered under footings. Additional discussions and our recommendations are provided in the report.
- Prior to being a treatment center the site was occupied by an elementary school. Any remnant structures should be removed and any disturbed soil should be removed from structural areas if present.
- Construction of settlement sensitive structures should be postponed in areas that receive 3 or more feet of fill until survey information shows that fill induced settlement is complete.
- The topsoil zone thickness at the site is expected to extend approximately 12 to 18 inches below ground surface (BGS). The topsoil zone is not suitable to support foundations, floor slabs, and pavements and should be removed from structural areas or mitigated in place. If left in place, topsoil zones should be improved, stabilized, or cement-amended to provide adequate subgrade support. Recommendations are presented in Section 7.1.1, *Topsoil Zone*.
- Groundwater was observed at approximately 20 feet BGS in the borings and CPTs explored in the vicinity of the existing drain field. Groundwater may impact site cuts for foundations, utilities, and stormwater detention ponds and dewatering may be needed depending on location and elevation of construction operations.

- The moisture content of native soil at the time of exploration was considerably higher than the optimum moisture content required for compaction. Moisture conditioning (drying) will be required to use the material as structural fill. Addition of moisture may also be necessary during periods of warm, dry weather. If moisture conditioning is not feasible, soils may require cement-amendment to be used as structural fill.

- Fine-grained soils will be sensitive to disturbance when at a moisture content that is above optimum. Haul roads and staging areas will be necessary to minimize damage to exposed subgrade soils during construction. Subgrade protection is discussed in Section 7.2, *Construction Traffic and Staging*.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL SITE INVESTIGATION
NARA RESIDENTIAL TREATMENT CENTER
FEASIBILITY STUDY
PORTLAND, OREGON

1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Native American Rehab Association of the Northwest, Inc. (NARA) to conduct a feasibility-level geotechnical site investigation for use in general planning purposes for various site improvements at the existing NARA Residential Treatment Center in Portland, Oregon. This report is subject to the limitations expressed in Section 8.0, *Conclusion and Limitations*, and Appendix G.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located at 17645 NW St Helens Road in Portland, Oregon. The site is comprised of a portion of tax lot R32501 totaling approximately 11 acres. The approximate latitude and longitude are N 45° 39' 01" and W 122° 51' 06". The regulatory jurisdictional agency is Multnomah County.

1.2 Project Understanding

Based on client correspondence and review of the various proposed site layouts presented in Appendix A, the proposed development includes construction of an approximately 30,000 square-foot, two-story residential treatment facility with associated parking areas, drive aisles and other amenities. We understand the exact location of the new facility has not been determined at the time of this report. We have assumed maximum column, wall, and slab loads for the building will be less than 200 kips, 4 kips per foot, and 100 psf, respectively. We have also assumed that cuts and fills will be no greater than 3 feet each. Columbia West should be contacted once the final building location and structural details are available.

2.0 SCOPE OF SERVICES

Columbia West's scope of services was outlined in a proposal dated January 20, 2023. In accordance with our proposal, we performed the following geotechnical services:

- Reviewed information available in our files from previous geological and geotechnical studies conducted in the vicinity of the site.
- Reviewed preliminary plans provided by the design team.
- Conducted subsurface exploration program at the site that included:
    - Five borings to depths between 17 and 21.5 feet BGS
    - Five cone penetration tests (CPTs) to depths ranging from 13 to 40 feet BGS
- Collected disturbed and relatively undisturbed soil samples from the soil borings for laboratory analysis.
- Classified and logged observed soil and groundwater conditions.
- Reviewed electronic logs of the CPT soundings provided by the CPT subcontractor.
- Prepared this geotechnical site investigation report for the proposed development, which includes:

- Summary of soil index properties, regional geology, soil conditions, and observed groundwater conditions
- Summary of geologic and seismic literature research used to evaluate relevant seismic risks, including locations of faults, earthquake magnitudes, and seismic coefficients in accordance with the 2022 State of Oregon Structural Specialty Code
- Liquefaction analysis and predicted seismic settlement
- Evaluation of existing slopes in the site vicinity
- Fill- and load-induced settlement potential
- Geotechnical design and construction recommendations for:
    - Shallow foundations
    - Slab subgrade preparation
    - Retaining walls, including drainage, backfill, and lateral earth pressures
    - Site preparation and grading, organic stripping, fill placement and compaction, over-excavation, and construction monitoring and testing
    - Structural fill materials, onsite soil suitability, and import aggregate specifications
    - Utility trench excavation and backfill
    - Drainage and management of groundwater conditions
    - Asphaltic concrete pavement construction for access roads and parking lots, including section thicknesses for base aggregate and asphalt layers
    - Portland cement concrete pavement construction for heavy access drives, including section thicknesses for base aggregate and PCC layers
    - Seismic design parameters in accordance with *ASCE 7-16*

3.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

The subject site lies at the toe of the Portland Hills approximately 2,000 feet southwest of the Multnomah Channel of the Willamette River and approximately 4 miles west of the confluence of the Willamette and Columbia Rivers.

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located immediately west and southwest of the site. According to *Seismic Design Mapping, State of Oregon* (Geomatrix Consultants, 1995), there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested.

According to the *USGS Earthquake Hazards Program*, the fault was originally mapped as a down-to-the-northeast normal fault but has also been mapped as part of a regional-scale zone of right-lateral, oblique slip faults, and as a steep escarpment caused by asymmetrical folding above a south-west dipping, blind thrust fault. The Portland Hills fault offsets Miocene Columbia River Basalts, and Miocene to Pliocene sedimentary rocks of the Troutdale Formation. No fault scarps on surficial Quaternary deposits have been described along the fault trace, and the fault is mapped as buried by the Pleistocene-aged Missoula flood deposits.

However, evidence suggests that fault movement has impacted shallow Holocene deposits and deeper Pleistocene sediments. Seismologists recorded a M3.2 earthquake thought to be associated with the fault zone near Kelly Point Park in November 2012, a M3.9 earthquake thought to be associated with the fault zone near Kelly Point Park in April 2003, and a M3.5 earthquake possibly associated with the fault zone approximately 1.3 miles east of the fault in 1991. Therefore, the Portland Hills Fault Zone is generally thought to be potentially active and capable of producing possible damaging earthquakes.

According to the *Geology Map of the Sauvie Island Quadrangle, Multnomah and Columbia Counties, Oregon and Clark County, Washington* (Evarts, R.C. O'Connor, J.E., and Cannon, C.M., USGS, 2016), near-surface soils are expected to consist of Holocene and Pleistocene, unconsolidated, poorly to well-sorted sand, silt, and minor gravel alluvial deposits (Qa) in the western portion of the site. Near-surface soils in the eastern portion of the site are expected to consist of Holocene, unconsolidated, massive fine sand and silt forming dunes and benches eoloian deposits (Qe). A portion of the northwest corner of the site is mapped as artificial fill (af).

The *Web Soil Survey* (USDA, NRCS, 2023 Website) identifies surface soils as Burlington fine sandy loam, Quatama loam, and Haploxerolls. Burlington, Quatama, and Haploxeroll series soils are generally fine-textured sands and silts with medium permeability, moderate to high water capacity, and low shear strength. Wapato series soils are generally fine-textured clays and silts with low permeability, moderate water capacity, and low shear strength. Wapato series soils are generally fine-textured clays and silts with low permeability, moderate water capacity, and low shear strength. Quatama and Wapato soils are generally moisture-sensitive, somewhat compressible, and described as having moderate shrink-swell potential. The erosion hazard is slight primarily based upon slope grade.

4.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation that consists of visual surface reconnaissance, drilling five borings (SB-1 through SB-5) and performing five CPTs (CPT-1 through CPT-5) was conducted at the site on January 31 and February 10, 2023. The CPTs were conducted with a truck-mounted CPT rig. The borings were drilled with a trailer-mounted drill rig.

Disturbed samples were collected from the borings at representative depth intervals using 1½-inch diameter split-barrel (SPT) samples in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-poind hammer free falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration log, unless otherwise noted. The hammer was lifted using a rope and cathead with two wraps. Sampling methods and intervals are shown on the exploration logs. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications.

Analytical laboratory test results are presented in Appendix B. Exploration locations are shown on Figure 2. Soil boring logs are presented in Appendix C. Soil descriptions and classification information are provided in Appendix D. The CPT data report is presented in Appendix E. A photo log is presented in Appendix F.

4.1 Surface Investigation and Site Description

The approximately 10-acre subject site is situated immediately south of the intersection of NW Cornelius Pass Road and NW St Helens Road. Current development consists of the existing NARA Residential Treatment Center in the eastern portion of the site and associated paved and landscaped areas extending into the central portion. The southern and southwestern property boundaries are undeveloped, heavily wooded, and comprise the toe of the adjacent Portland Hills. McCarthy Creek flows along the west to northwest property boundary separating the parcel from NW Cornelius Pass Road.

Further discussion on site topography is presented in Section 5.3, *Landslide Hazard.*

4.2 Subsurface Conditions

Borings and CPTs were advanced to a maximum depth of 21.5 and 41.2 feet BGS, respectively. Exploration locations were selected to observe subsurface soil characteristics in proximity to proposed development areas and are shown on Figure 2.Boring logs are presented in Appendix C. The CPT data report is presented in Appendix E.

4.2.1 Soil Type Description

In existing paved areas, 3 to 4 inches of asphalt underlain by 10 to 12 inches of crushed aggregate was observed. Surficial soils in other explored areas of the site generally consist of a 2 to 4-inch root zone underlain by an additional 6 to 10 inches of topsoil. Although not encountered in the explorations, geological mapping indicates undocumented fill may be present at the site. The site was previously occupied by an elementary school and disturbed soil may be present as a result of demolition activities.

Underlying the pavement and topsoil, alluvial clay, silt, and sand mixtures were observed to the maximum explored depth of approximately 41.2 feet. Subsurface lithology may generally be described by the soil units identified in the following text.

Existing Paved Areas

Pavement sections in existing paved areas were observed to consist of 3 to 4 inches of asphalt underlain by 10 to 12 inches of crushed aggregate.

Root Zone and Topsoil

Grassy and landscaped areas of the site generally consisted of 2 to 4 inches of heavy organic root sections underlain by 6 to 10 inches of silt and clay intermixed with minor proportions of organic material.

Undocumented Fill

As noted above, geological mapping indicates undocumented fill may be present at the site. Additional recommendations pertaining to undocumented fill are presented in Section 7.1.2, *Undocumented Fill.*

Silt, Clay, and Sand Mixtures

Underlying the pavement and topsoil, soft to medium stiff clays and silts and loose to medium dense sands mixtures were encountered to approximately 20 feet BGS. Within soil borings SB-1, SB-4, and SB-5 and CPT-5, increased relative density of native soils was observed between 20 and 25 feet BGS. Within CPT-4, a decrease in relative density of soils and a transition to soft to medium stiff clay and silt mixtures was observed to the terminal depth of 41.2 feet BGS. Laboratory tested native silts and clays had moisture contents generally ranging from 20 to 35 percent and exhibited low-plasticity behavior. Tested native sandy soils had moisture contents generally under 10 percent.

4.2.2 Groundwater

Groundwater seepage was observed at depths of approximately 17 to 22 feet BGS in explorations conducted in the vicinity of the drainage field west of the existing facility (SB-1, SB-2, and CPT-3 through CPT-5). Groundwater was not observed in other explorations conducted at higher elevations throughout the site.

Groundwater levels are likely governed by fluctuations of nearby water features such as the Multnomah Channel and McCarthy Creek, are subject to seasonal variance, and may rise during extended periods of increased precipitation. Perched groundwater may also be present in localized areas. Seeps and springs may become evident during site grading, primarily along slopes or in areas cut below existing grade. Structures, pavements, and drainage design should be planned accordingly.

4.2.3 Infiltration Testing

Evaluation of infiltration potential of site soils through field testing was not included in the current scope. Based on the presence of fine-textured, low permeability site soils, infiltration of stormwater may not be a feasible option for stormwater management. Columbia West should be contacted for additional study if subsurface disposal of stormwater is considered.

5.0 GEOLOGIC HAZARDS

5.1 Liquefaction

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand with low silt and clay content is the most susceptible to liquefaction. Silty soil wit low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking. Our subsurface exploration program did not encounter soils that are susceptible to liquefaction under design levels of ground shaking.

5.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard that occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement.

Since the site soils are not susceptible to liquefaction lateral spreading is not considered a hazard

5.3 Landslide Hazard

As indicated on Figures 2A and 2B, DOGAMI HazVu maps the site as having moderate to high potential for a landslide hazard with isolated areas of head scarps and landslide deposits. A site reconnaissance of the property was completed on February 10, 2023. General observations of site slopes are summarized in the following text. Slope inclinations and heights are approximate and are based on available topographic mapping. The recommendations presented below should be confirmed with Columbia West once an updated topographic survey and site layout are available.

The northeast site boundary adjacent to NW St Helens road consists of a northeast-facing slope with approximate maximum inclination and height of 25 percent and 10 to 15 feet, respectively. The northwest site boundary slopes downward northwest to McCarthy Creek with an approximate maximum inclination of 35 to 45 percent and height of 25 feet. The area southwest of the existing drain field consists of a southwest-facing slope with an approximate inclination of 45 percent and height of 15 feet before transitioning to an approximate 15 percent slope for an additional 10 feet in height (total 25-ft slope height between two tiers).

Existing slopes to the southeast of the existing drain field consist of a two-tiered, northwest-facing slope. The lower tier adjacent to the drain field slopes up southeastward from the field at approximately 25 percent for a height of 10 feet to a 50-ft wide, relatively flat landscaped area associated with the existing facility. The ground surface then continues upward southeast at an inclination of approximately 35 percent and a height of 50 feet before continuing upwards into the adjacent Portland Hills.

Vegetation on the surface of the slopes primarily consists of moderate to heavily dense mature trees and underbrush Indications of soil creep such as curved tree trunks were noted in several portions of the slopes. It is our opinion that the proposed development is not at significant risk of slope failure or instability provided the recommendations in this report are incorporated in design and implemented during construction.

We recommend trees and vegetation remain on the existing slopes where they would not otherwise interfere with proposed improvements. If vegetation must be removed, bare soil should be protected against erosion until permanent vegetation can be re-established. Columbia West should be retained to review all plans indicating removal of vegetation on the existing slopes.

We recommend that water flow from storm drainage controls and surface runoff (roof drains, retaining wall back drains, and other subsurface drains) be connected to non-perforated pipes leading to the storm drain facilities. Drainage discharged onto sloped areas of the site could result in slope instability and should not be permitted.

Cuts and fills should be limited to 3 feet in height. If cuts and fills greater than 3 feet are required, Columbia West should be contacted to conduct a slope stability analysis.

5.3.1 Geotechnical Buffer

To reduce the risk of slope instability affecting the development, structures and stormwater facilities should be avoided within the geotechnical buffers identified on Figure 2 unless slope stability analysis determines that the slope and proposed development are stable. We also recommend that fill not be placed at the top of slopes in the buffer zone and cuts be avoided in the buffer zone at the toe of slopes. The buffer for the northeast, northwest, and southwest-facing slopes is defined by a 10-ft setback measured from the existing top-of-slope. The buffer for the northwest-facing slopes southeast of the drain field is defined by a 10-ft setback measured from the existing toe-of-slope. Construction activities should be avoided in the 50-ft wide planar area between the two tiers of the northwest-facing slopes situated southeast of the drain field.

Placement of engineered structural fill or stockpiles of disturbed soil should be avoided inside the geotechnical buffer without conducting a slope stability analysis. Soil excavation may be acceptable within the top-of-slope buffers as driving forces may be reduced by removing soil mass but are not permitted within the toe-of-slope buffer indicated for the area southeast of the drain field. Columbia West should review mass grading plans as they relate to the geotechnical buffer prior to final design approval.

Areas within the geotechnical buffer are not intended to be do-not-disturb conservation areas. Small disturbances such as minor landscaping, fence building, or walk-path construction are acceptable. Deep-rooted vegetation generally results in reduced slope erosion and increased near-surface soil shear strength. The risk of slope instability increases with disturbance or alteration of existing slope vegetation. Removal of established slope vegetation within the buffer should be minimized.

6.0 DESIGN RECOMMENDATIONS

The geotechnical site investigation suggests the proposed development is generally compatible with surface and subsurface soils, provided the recommendations presented in this report are incorporated in design and implemented during construction. The primary geotechnical considerations for the project were summarized previously in the *Executive Summary*. Specific design and construction recommendations are presented in the following sections.

6.1 Areal Settlement Considerations

Grading plans were not available at the time of this report. We have assumed cuts and fills at the site will be less than 3 feet each. Alluvial site soils, particularly the silt and clay, will be compressible under proposed foundation, slab, and/or fill-induced loads. Our experience indicates that fills not exceeding 3 feet above existing grade combined with anticipated footing and floor slab loads are unlikely to exceed the static settlement tolerances of the buildings.

In areas where new fills exceed 3 feet, construction of hardscapes and settlement sensitive structures should be postponed until survey information confirms that settlement is complete.

6.2 Shallow Foundation Support

Foundation loads were not available at the time of this report. We have assumed maximum column and wall loads for the buildings will be less than 200 kips and 4 kips per foot, respectively. Provided maximum floor slab loading is less than 100 psf, the proposed building can be supported by conventional spread footings bearing on firm native soil or engineered structural fill.

Foundations should not be supported by topsoil, agricultural till zones, or undocumented fill material. If encountered, these materials should be improved or removed and replaced with structural fill. If footings are constructed during wet-weather conditions or when footing subgrade soils are above their optimum moisture content, we recommend that a minimum of 6 inches of compacted aggregate be placed over exposed subgrade soils. The aggregate pad should extend 6 inches beyond the edge of the foundations and consist of imported granular material as described in Section 7.6.1, *Structural Fill*. Columbia West should observe exposed subgrade conditions prior to placement of crushed aggregate to verify adequate subgrade support.

6.2.1 Footing Dimensions and Bearing Capacity

Continuous perimeter wall and isolated spread footings should have minimum width dimensions of 18 and 24 inches, respectively. The base of exterior footings should bear at least 18 inches below the lowest adjacent exterior grade. The base of interior footings should bear at least 12 inches below the base of the floor slab.

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. As the allowable bearing pressure is a net bearing pressure, the weight of the footing and associated backfill may be ignored when calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by 50 percent for transient lateral forces such as seismic or wind.

6.2.2 Shallow Foundation Settlement

Foundations designed in accordance with this report are expected to experience a post construction settlement of less than one inch. Differential post construction settlement between comparably-loaded footing elements is not expected to exceed 0.5 inch over a span of 50 feet.

6.2.3 Resistance to Sliding

Lateral foundation loads can be resisted by passive earth pressure on the sides of the footing and by friction at the base of the footings. Recommended passive earth pressure for footings confined by native soil or engineered structural fill is 350 pcf. The upper 12 inches of soil should be neglected when calculating passive pressure resistance. Adjacent floor slabs and pavement, if present, should also be neglected from the analysis. The recommended passive pressure resistance assumes that a minimum horizontal clearance of 10 feet is maintained between the footing face and adjacent downgradient slopes.

The estimated coefficient of friction between in situ native soil or engineered structural fill and in-place poured concrete is 0.35. The estimated coefficient of friction between compacted crushed aggregate and in-place poured concrete is 0.4.

6.2.4 Subgrade Observation

Footing and floor subgrade soils should be evaluated by Columbia West prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report. Subgrade observation should confirm that all disturbed material, organic debris, unsuitable fill, remnant topsoil zones, and softened subgrades (if present) have been removed. Over-excavation of footing subgrade soils may be required to remove deleterious material, particularly if footings are constructed during wet-weather conditions.

6.2.5 Floor Slabs

Floor slabs can be supported on firm, competent, native soil or engineered structural fill prepared as described in this report. Disturbed soils and unsuitable fills in proposed slab locations, if encountered, should be removed and replaced with structural fill. Floor slab settlement risks were discussed previously in Section 6.1, *Areal Settlement Considerations*.

To provide a capillary break, slabs should be underlain by at least 6 inches of compacted crushed aggregate that has less than 5 percent by dry weight passing the No. 200 Sieve. Geotextile may be used below the crushed aggregate layer to increase subgrade support. Recommendations for floor slab base aggregate and subgrade geotextile are discussed in Section 7.6, *Materials*.

Elevated soil moisture was observed within near-surface silt and clay at the time of exploration. Some flooring manufactures will only warranty their product if a vapor barrier is installed. Selection of an appropriate vapor barrier should be selected by consulting with the design team.

Slab thickness and reinforcement should be designed by the project structural engineer assuming a modulus of subgrade reaction, k, of 125 pci.

6.3 Seismic Design Considerations

Seismic design for proposed structures is prescribed by the *2022 Oregon Structural Specialty Code (OSSC)* which refers to the 2022 IBC. Based on results of subsurface exploration, site soils meet the criteria for Site Class D. Seismic design parameters for Site Class D are presented in Table 1.

For Site Class D sites with mapped maximum considered earthquake spectral response acceleration parameter S_1 greater than 0.2, a ground motion hazard analysis may be required according to *ASCE 7-16, Section 11.4.8* unless the seismic response coefficient, C_s, is calculated in accordance with *ASCE 7-16 Section 11.4.8, Exception 2.* However, if an alternative method is utilized to determine the seismic response coefficient, the structure is seismically isolated, or structural damping systems are proposed, *ASCE 7-16* requires a ground motion hazard analysis be conducted. Columbia West recommends that the project structural engineer evaluate these requirements and exceptions to determine if a site-specific ground motion hazard evaluation will be required for proposed structures.

Table 1. ASCE 7-16 Seismic Design Parameters[1]

	Short Period (T_s = 0.2 s)	1 Second Period (T_1 = 1.0 s)
MCE Spectral Acceleration	0.909	0.424
Site Class	D^2	
Site Coefficient	Fa = 1.14	Fv = 1.88
Adjusted Spectral Response Acceleration	S_{MS} = 1.03	S_{M1} = 0.80
Design Spectral Response Acceleration	S_{DS} = 0.69	S_{D1} = 0.53

1. The structural engineer should evaluate *ASCE 7-16* code requirements and exceptions to determine if these parameters are valid for design.

Since we do not believe that there will be a loss of bearing capacity, in our opinion foundation ties will not be required.

6.4 Retaining Structures

Lateral earth pressures should be considered during design of retaining walls and below-grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Wall foundation construction and bearing capacity should adhere to specifications provided previously in Section 6.2, *Shallow Foundation Support*.

Permanent retaining walls that are not restrained from rotation should be designed for active earth pressures using an equivalent fluid pressure of 35 pcf. Walls that are restrained from rotation should be designed for an at-rest, equivalent fluid pressure of 55 pcf. The recommended earth pressures assume a maximum wall height of 10 feet with well-drained, level backfill. These values also assume that adequate drainage is provided behind retaining walls to prevent hydrostatic pressures from developing. Lateral earth pressures induced by surcharge loads may be estimated using the criteria presented on Figure 3.

Seismic forces may be calculated by superimposing a uniform lateral force of $7H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The force should be applied as a distributed load with the resultant located at 0.6H from the base of the wall.

6.4.1 Wall Drainage and Backfill

A minimum 4-inch-diameter, perforated collector pipe should be placed at the base of retaining walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of finished grade. The drain rock and geotextile drainage fabric should meet the specifications provided in Section 7.6, *Materials*. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drainage systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Backfill material placed behind the walls and extending a horizontal distance of ½ H, where H is the height of the retaining wall, should consist of select granular material placed and compacted as described in Section 7.6.1, *Structural Fill.*

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be delayed at least four weeks after placement of wall backfill, unless survey data indicates that settlement is complete prior to that time.

6.5 Pavement Design

6.5.1 Design Parameters and Traffic

Pavement should be installed on firm, competent native subgrade soil or engineered structural fill prepared as described in this report. Our pavement recommendations are based on the following design parameters and assumptions:

- 12 inches of subgrade soil directly below the pavement sections are compacted to at least 95 percent of maximum dry density, as determined by *ASTM D1557*.
- Resilient moduli for subgrade soil and aggregate base materials were assumed to be 4,500 psi and 20,000 psi, respectively.
- Pavement design life of 20 years with no expected traffic growth.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 85 percent and standard deviation of 0.4.
- Pavement may be exposed to a fire apparatus load of 75,000 pounds on an infrequent basis.

The specific type and frequency of traffic was not available at the time we prepared this report. Based on experience, we assume that heavy truck traffic will consist of approximately 40 percent FHWA Class Group 6 type trucks (4-axle, single unit) and 60 percent FHWA Class Group 8 type trucks (tractor/trailer 2- to 3-axle). Lightly-loaded drive aisles and parking stalls are expected to service typical passenger vehicle traffic.

6.5.2 Asphaltic Concrete (AC) Pavement Design Sections

Pavement design recommendations for a range of traffic conditions and loading scenarios are presented in Table 2. Material properties and compaction recommendations for asphalt surfacing and crushed aggregate base layers are presented in Section 7.6, *Materials*.

Table 2. Recommended AC Pavement Sections Constructed over Native Soil or Engineered Fill

Traffic	Trucks Per Day	Equivalent Single-Axle Loads (ESALs)	AC Thickness (in)	Base Aggregate Thickness (in)
Passenger Vehicle Parking	0	10,000	2.5	8
Passenger Vehicle Drive Aisles	0	20,000	3	9
Heavy Truck Areas	10	92,000	4	10.5
	25	229,000	4.5	12.5
	50	458,000	5	14
	100	916,000	5.5	16.5

Pavement sections may be reduced in areas where subgrade soils are cement-amended to a minimum depth of 12 inches with a minimum of 6 percent cement by weight. Provided the cement-amended subgrade soil achieves a seven-day unconfined compressive strength of 100 psi, AC pavement sections may be constructed as presented in Table 3.

Table 3. Recommended AC Pavement Sections Constructed over Cement-Amended Subgrade Soil

Traffic	Trucks Per Day	Equivalent Single-Axle Loads (ESALs)	AC Thickness (in)	Base Aggregate Thickness (in)	Cement-Amendment Thickness (in)
Passenger Vehicle Parking	0	10,000	2.5	4	12
Passenger Vehicle Drive Aisles	0	20,000	3	4	
Heavy Truck Areas	10	92,000	4	4	
	25	229,000	4.5	4	
	50	458,000	5	4	
	100	916,000	5.5	6	

6.5.3 General Pavement Recommendations

Recommended pavement section thicknesses are intended to be minimum acceptable values and do not include construction traffic loading. The recommendations assume that pavement construction will be completed during an extended period of warm, dry weather. Wet weather construction may require an increased thickness of base aggregate as discussed later in Section 7.2, *Construction Traffic and Staging*.

Cement-amended soil should be allowed to cure for at least four days prior to aggregate base placement or exposure to construction traffic. Prior to construction traffic access, the cement-amended subgrade should be protected by a minimum 4-inch-thick layer of compacted crushed aggregate. Construction traffic should be limited to dedicated haul roads or non-structural, unpaved portions of the site. Construction traffic should not be permitted on new pavement, unless accounted for in the pavement design section. Base aggregate and cement-amended soils supporting pavement are also not intended for construction traffic. Haul roads and staging areas supporting construction traffic are discussed later in Section 7.2, *Construction Traffic and Staging.*

Asphalt paving is generally not recommended during cold weather conditions where ambient air temperatures are less than 40 degrees Fahrenheit. Compacting asphalt in low-temperature conditions can result in low relative density of the asphalt layer and premature pavement distress.

Asphalt mix designs have a recommended compaction temperature range that is specific to the AC binder used. In low-temperature conditions, maintaining the temperature of the AC mix is difficult as heat can be lost during transport, placement, and compaction. The ambient air temperature during paving should be at least 40 degrees Fahrenheit for a lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for a lift thickness between 2 and 2.5 inches. If AC paving must take place during cold-weather construction as defined in this section, the contractor and design team should discuss options for minimizing risk to pavement serviceability.

6.6 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to Multnomah County regulations. Finished site grading should be conducted with positive drainage away from structures at a minimum 2 percent slope for a distance of at least 10 feet. Depressions or shallow areas that may retain ponding water should be avoided.

6.6.1 Subsurface Drainage

Site improvements construction may occur in areas where springs or seepage is present. If encountered during construction, footing drains or subdrains beneath slabs-on-grades can be installed. Figure 3 shows a typical foundation drain detail. Figure 5 shows a typical trench drain detail. A typical drainage mat is shown on Figure 6. Columbia West should determine drainage mat location, extent, and thickness when subsurface conditions are exposed.

7.0 CONSTRUCTION RECOMMENDATIONS

7.1 Site Preparation and Grading

As discussed previously, root zones of 2 to 4 inches were observed in grassy areas of the site. Root zones approaching 12 inches may be present in other areas of thick vegetation, trees, and shrubs. Approximately 3 to 4 inches of asphalt underlain by 10 to 12 inches of crushed aggregate were observed in paved areas of the site. Vegetation, organic material, unsuitable fill, and deleterious material that may be encountered should be cleared from areas identified for structures and site grading. Vegetation, root zones, organic material, and debris should be removed from the site. Stripped topsoil should also be removed or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot.

The required stripping depth may increase in areas of existing fill or where structures existed previously. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed.

Previously disturbed soil, debris, or undocumented fill encountered during grading or construction activities should be removed completely and thoroughly from structural areas. This includes old remnant foundations, basement walls, utilities, associated soft soils, and debris. Excavation areas should be backfilled with engineered structural fill.

Site grading activities should be performed in accordance with requirements specified in the *2018 International Building Code* (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation, soil stripping, and grading activities should be observed and documented by Columbia West.

7.1.1 Topsoil Zone

A topsoil layer was encountered underling the root zone within site explorations and extended to an observed depth of approximately 1 to 1.5 feet BGS. As soil strength and compressibility properties are difficult to estimate for topsoil, there is a high risk for poor performance of floor slabs and pavement constructed directly over the unimproved topsoil layer. To reduce settlement risk, we recommend the topsoil be improved or mitigated in areas where proposed cuts will not extend to the base of the topsoil layer (up to 12 inches below existing grade). The topsoil zone should be improved mitigated through removal and replacement with structural fill or improved through scarifying and re-compacting as engineered structural fill.

Alternatively, the topsoil layer may be cement-amended as discussed in Section 7.6.3, *Soil Amendment with Cement*. Cement-amendment is generally preferred in areas where the proposed cut is less than 12 inches and treated topsoil zone soils will serve as subgrade for building slabs and pavement.

7.1.2 Undocumented Fill

Although not observed within the explorations, geologic mapping indicates undocumented fill may be present at the site. Undocumented fill and other previously disturbed soils or debris are not suitable for bearing structures and should be removed completely removed from the influence zone of foundations until firm native soils are encountered prior to the placement of additional fill.

To minimize long-term risk of adverse impacts to pavement structures, existing fill should also be thoroughly removed from proposed pavement areas. If existing fill is left in place, pavement structures may experience a reduction in long-term serviceability due to premature pavement distress which could include asphalt cracking, localized grade depressions, and inadequate drainage. The decision to construct pavements over existing fill and acceptance of the associated risk should be made by the owner and project stakeholders.

Partial mitigation of premature pavement distress risk may be accomplished by over-excavation and backfill with granular structural fill or application of cement amended materials. Identification of specific engineered mitigation plans is beyond the scope of this report. If this option is selected, Columbia West should be contacted for additional analysis and study, but would likely consist of improving the upper 18-inches of undocumented fill. This can be accomplished by scarifying and compacting it in place, cement emending it, or removing it and replacing it with structural fill.

7.1.3 Subgrade Evaluation

Upon completion of stripping and prior to the placement of structural fill or pavement improvements, exposed subgrade soil should be evaluated by proof rolling with a fully-loaded dump truck or similar heavy, rubber tire construction equipment. When the subgrade is too wet for proof rolling, a foundation probe may be used to identify areas of soft, loose, or unsuitable soil. Subgrade evaluation should be performed by Columbia West. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over-excavated and backfilled with compacted imported granular fill.

7.2 Construction Traffic and Staging

Near-surface clay and silt will be easily disturbed during construction. If not carefully executed, site preparation, excavation, and grading can create extensive soft areas resulting in significant repair costs. Earthwork planning should include considerations for minimizing subgrade disturbance, particularly during wet-weather conditions.

If construction occurs during wet-weather conditions, or if the moisture content of the surficial soil is more than a few percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Under these conditions, granular haul roads and staging areas will also be necessary provide a firm support base and sustain construction equipment.

The recommended base aggregate thickness for pavement sections is intended to support post-construction design traffic loads and will not provide adequate support for construction traffic. Staging areas and haul roads will require an increased base thickness during wet weather conditions. The configuration of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor’s means and methods. Therefore, design and construction of staging areas and haul roads should be the responsibility of the contractor. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul road areas. In areas of heavy construction traffic, geotextile separation fabric may be placed between the subgrade soil and imported granular material to increase subgrade support and minimize silt migration into the base aggregate layer.

As an alternative to thickened aggregate sections, haul roads and staging areas may be constructed using a combination of cement-amended subgrade and crushed aggregate surfacing. If cement-amendment is used, the base aggregate thickness for staging areas and haul roads can typically be reduced to between 6 and 9 inches, respectively. This recommendation is based on a minimum seven-day unconfined compressive strength of 100 psi for the cement-amended soil with a treatment depth of 12 to 16 inches. Based on experience, 6 to 7 percent cement by weight is typically required to achieve the indicated compressive strength.

Project stakeholders should understand that wet weather construction is risky and costly. Proper construction methods and techniques are critical to overall project integrity and should be observed and documented by Columbia West.

7.3 Cut and Fill Slopes

Fill slopes should consist of structural fill material as discussed in Section 8.1.1, *Structural Fill*. Fill placed on existing grades steeper than 5H:1V should be horizontally benched at least 5 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 7. Drainage implementations, including subdrains or perforated drainpipe trenches, may also be necessary in proximity to cut and fill slopes if seeps or springs are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by Columbia West during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion.

Final cut or fill slopes at the site should not exceed 2H:1V or 10 feet in height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 8.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed and documented by Columbia West.

7.4 Excavation

The site was explored to a maximum depth of 21.5 feet BGS with a drill rig and 41.3 feet BGS with a CPT rig. In general, conventional earthmoving equipment in proper working condition should be capable of making necessary site excavations. CPT-5 met early refusal in very dense deposits at 13.2 feet BGS. The CPT was then offset approximately 10 feet southwest (CPT-5B) where it met refusal at 22.8 feet BGS. Increased penetration resistance was noted in the SPTs performed within soil borings SB-1, SB-4, and SB-5 at approximately 20 feet BGS. The contractor should be prepared with appropriately-sized equipment if site excavations extend to these depths.

Groundwater was observed within the vicinity of the drain field west of the existing facility at approximately 20 feet BGS. Recommendations as described in Section 7.5, *Dewatering,* should be considered where subsurface construction activities intersect the groundwater table.

Temporary excavation sidewalls should maintain a vertical cut to a depth of approximately 4 feet in the near-surface silt and clay, provided groundwater seepage is not present in the sidewalls. In sandy soil, excavations will likely slough and cave, even at shallow depths. Open-cut excavation techniques may be used to excavate trenches between 4 and 8 feet deep, provided the walls of the excavation are cut at a maximum slope of 1H:1V and groundwater seepage is not present. Excavation slopes should be reduced to 1.5H:1V or 2H:1V if excessive sloughing or raveling occurs.

Shoring may be required if open-cut excavations are infeasible or if excavations are proposed adjacent to existing infrastructure. Typical methods for stabilizing excavations consist of solider piles and timber lagging, sheet pile walls, tiebacks and shotcrete, or pre-fabricated hydraulic shoring. As a wide variety of shoring and dewatering systems are available, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

The contractor should be held responsible for site safety, sloping, and shoring. All excavation activity should be conducted in accordance with applicable OSHA requirements. Columbia West is not responsible for contractor activities and in no case should excavation be conducted in excess of applicable local, state, and federal laws.

7.5 Dewatering

Groundwater was observed at approximately 20 feet BGS within subsurface explorations performed in the vicinity of the existing drain field. Depending on final site elevations, groundwater may be encountered in utility trench excavations and in areas of significant cut. Generalized recommendations for temporary construction dewatering are presented in the following section.

7.5.1 Construction Dewatering

The contractor should be responsible for temporary drainage of surface water, perched water, and groundwater. Dewatering should be performed to the extent necessary to prevent standing water and/or erosion of exposed site soils. During rough and finished grading of building pad areas, the contractor should keep all footing excavations and slab subgrade soils free of standing water.

The contractor’s proposed dewatering plan should be capable of maintaining groundwater levels at least two feet below the base of proposed trench excavations. Without adequate trench dewatering, running soil, caving, and sloughing will increase backfill volumes and may result in damage to adjacent structures or utilities. Significant pumping and dewatering may be required to temporarily reduce the groundwater elevation to the recommended depth. Dewatering via a sump within excavation zones may be insufficient to control groundwater and provide excavation side slope stability. Dewatering may be more feasibly conducted by installing a system of temporary well points and pumps around proposed excavation areas or utility trenches. Depending on proposed utility depths, a site-specific dewatering plan may be necessary.

If groundwater is present at the base of utility excavations, we recommend placing 18 to 24 inches of stabilization material at the base of the excavation. Subgrade geotextile placed directly over trench subgrade soils may reduce the required thickness of the stabilization material. The actual thickness of stabilization material should be determined at the time of construction based on observed field conditions. Trench stabilization material should be placed in one lift and compacted until well keyed. Stabilization material and geotextile fabric should meet the requirements presented in Section 7.6, *Materials*.

7.6 Materials

7.6.1 Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in Section 7.1, *Site Preparation and Grading*. Engineered fill placement should be observed by Columbia West. Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with *ASTM D6938*. Field compaction testing should be performed for each vertical foot of engineered fill placed.

Various materials may be acceptable for use as structural fill. Structural fill should be free of organic material or other unsuitable material and meet specifications provided in the following sections. Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by Columbia West prior to placement.

7.6.1.1 Onsite Soil

Most onsite native soil (silt, clay, and sand) will be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native clay soil with a plasticity index greater than 25, if encountered, should be evaluated and approved by Columbia West prior to use as structural fill. Laboratory analysis indicated that the moisture content of site soil was above optimum at the time of exploration. Moisture conditioning will likely be necessary to dry the soil prior to applying compaction effort. In addition, the near-surface silt and clay will be moisture sensitive and difficult, if not impossible, to compact during wet weather conditions. Therefore, structural fill placement using onsite soil should be performed during dry summer months if possible. Onsite soil may also require addition of moisture during extended periods of dry weather.

Onsite soil used as structural fill should be placed in loose lifts not exceeding 8 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within a few percentage points of optimum conditions. The soil should be compacted to at least 95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557).* Compacted onsite fill soils should be covered shortly after placement.

Onsite soil will likely expand during excavation and transport and consolidate during compaction. Development of site-specific expansion and consolidation factors is beyond the scope of this investigation. We can provide site-specific factors upon request.

Recommendations for the topsoil zone were provided previously in Section 7.1.1, *Topsoil Zone*.

7.6.1.2 Imported Granular Material

Imported granular material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be durable, angular, and fairly well graded between coarse and fine material; should have less than 5 percent fines (material passing the U.S. Standard No. 200 sieve) by dry weight; and should have at least two mechanically fractured faces. Imported granular material should be placed in loose lifts not exceeding 12 inches in depth and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557).* During wet-weather conditions or where wet subgrade conditions are present, the initial loose lift of granular fill should be approximately 18 inches thick and should be compacted with a smooth-drum roller operating in static mode.

7.6.1.3 Stabilization Material

Stabilization material should consist of durable, 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is free of organics and other deleterious material. The material should have a maximum particle size of 6 inches with less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve. The material should have at least two mechanically-fractured faces.

Stabilization material should be placed in loose lifts between 12 and 24 inches thick and be compacted to a firm, unyielding condition. Equipment with vibratory action should not be used when compacting stabilization material over wet, fine-textured soils. If stabilization material is used to stabilize soft subgrade below pavement or construction haul roads, a subgrade geotextile should be placed as a separation barrier between the soil subgrade and the stabilization material.

7.6.1.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of durable, well-graded granular material with a maximum particle size of 1½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of durable, well-graded granular material with a maximum particle size of 2½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

7.6.1.5 Floor Slab Base Aggregate

Imported granular material used as base rock for building floor slabs should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight and at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.6.1.6 Pavement Base Aggregate

Imported granular material used as base rock for pavement should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight and at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.6.1.7 Retaining Wall Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material as described above and should have less than 7 percent fines by dry weight. We recommend the wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

The wall backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D1557. However, backfill located within a horizontal distance of 3 feet from a retaining wall should only be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (sidewalks or pavement) will be placed atop the wall backfill, we recommend that the upper 2 feet of material be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

7.6.1.8 Retaining Wall Leveling Pad

Imported granular material placed at the base of retaining wall footings should consist of select granular material. The granular material should be ¾- to 1-inch-minus aggregate size and should have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch-thick lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.6.1.9 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and less than 2 percent by weight passing the No. 200 sieve. Drain rock should be free of roots, organic debris, and other unsuitable material and should have at least two mechanically-fractured faces. Drain rock should be compacted to a firm, unyielding condition. Drain rock should be completely wrapped in a geotextile drainage fabric meeting the requirements presented below.

7.6.1.10 Existing Concrete and Crushed Rock

Concrete and crushed rock from the existing pavement areas and improvements can be used in general structural fill, provided particles greater than 3 inches are not present, it is thoroughly mixed and well graded so that there are no voids between the fragments, and the resulting mix is moisture conditioned for compaction. This material can be used as trench backfill if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.6.2 Geotextile Fabric

7.6.2.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-4 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile.

7.6.2.2 Drainage Geotextile

Drainage geotextile should conform to Type 2 material of OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

7.6.3 Soil Amendment with Cement

The on-site soil can be amended with Portland cement to obtain suitable properties for use as wet-weather structural fill or subbase for pavement. The effectiveness of soil amendment is highly dependent on proper mixing techniques, soil moisture conditioning, and the quantity of cement. The quantity of cement applied during amendment should be based on an assumed dry unit weight of 100 pcf for site soil.

7.6.3.1 Subbase Stabilization

Specific recommendations for soil amendment should be based on exposed site conditions at the time of construction. For preliminary design purposes, we recommend cement-amended subgrade for building pads and pavement subbase (below the base aggregate layer) achieve a target strength of 100 psi. The quantity of cement required to achieve the target strength will vary with moisture content and soil type. Laboratory testing of cement-amended soil should be used to confirm design expectations.

Based on our experience, near-surface silt and clay will require approximately 6 to 7 percent cement by weight to achieve the target strength of 100 psi. This cement percentage assumes that the soil moisture content does not exceed 20 percent at the time of amendment. If the soil moisture content is in the range of 25 to 35 percent, 7 to 8 percent cement by weight may be required to achieve the target strength. The amount of cement added to the soil at the time of construction should be based on observed field conditions and subgrade performance. During extended periods of dry weather, water may need to be applied during the amendment and tilling process to achieve the optimum moisture content required for compaction.

Cement-amendment of the topsoil zone will likely require higher quantities of cement due to the organic content and high-plasticity characteristics of the material. A minimum cement percentage of 7 to 8 percent by weight should be assumed for topsoil zone soil. In addition, increased mixing effort and tilling passes will likely be required to adequately blend the cement into the high plasticity material.

Cement-amendment equipment should have balloon tires to minimize softening, rutting, and disturbance of fine-grained site soil. A sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction. Rollers with vibratory action should not be used to compact fine-grained, cement-amended soil. Final compaction should be conducted with a smooth-drum roller with a minimum applied linear force of 700 pounds per inch. The amended soil should be compacted to at least 95 percent of the maximum dry density as determined by *ASTM D558*.

Following cement amendment, a minimum curing time of four days is required prior to exposure to construction traffic. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect cement-amended areas from damage, the finished surface should be covered with 4 to 6 inches of imported granular material. The protective layer of crushed rock often becomes contaminated with soil during construction, particularly in staging and haul road areas. Contaminated aggregate, where present, should be removed and replaced with clean crushed aggregate prior to construction of pavement or other permanent site improvements supported by base aggregate.

Cement amendment should not be attempted during moderate to heavy precipitation or when the ambient air temperature is below 40 degrees Fahrenheit. Cement should not be placed in areas of standing water or where saturated subgrade conditions exist.

7.6.3.2 Cement-Amended Structural Fill

If adequate compaction is not achievable with onsite silt and clay due to moisture or weather conditions, the soil may be cement-amended and placed as general structural fill. Prior to placement of cement-amended fill, subgrade soils should be prepared as described in Section 7.1, *Site Preparation and Grading*. Where multiple lifts of cement-amended fill are necessary to meet finished grade, consecutive lifts may be placed immediately following amendment and compaction of the underlying lift. However, where the final lift of cement-amended fill will serve as building pad or pavement subbase material, the four-day cure period as discussed above is recommended.

7.6.3.3 Verification Testing

Cement-amendment of site soils should be observed and tested by Columbia West to document conformance with design recommendations. Cement spread rate should be verified with a pan sample test conducted at one random location per lift per 20,000 square-feet of cement-amended fill. Treatment depth should be verified through excavation of a small test pit and measurement at one random location per lift of cement-amended fill. Adequate compaction and moisture content should be verified by conducting nuclear gauge density testing at a frequency of approximately one test per 5,000 square feet of cement-amended fill in accordance with ASTM D6938. At least one representative sample should be collected per day of cement-amendment, cured for 7 days, and tested for unconfined compressive strength in accordance with ASTM D1633. The tested samples should have a minimum 7-day, unconfined compressive strength of 100 psi.

7.6.3.4 Drainage Considerations

Cement-amended soil will be poorly-drained and will not be suitable for planting areas. The material may also be difficult to excavate with light-duty landscaping equipment. Proposed landscape areas should not be cement-amended unless accommodations are made for drainage and planting.

Cement-amendment within building pad areas should consider the potential for trapped water below the floor slab. Columbia West should be consulted to provide appropriate recommendations if cement-amendment is proposed within building pad areas.

7.6.4 Pavement

7.6.4.1 Asphaltic Concrete

Asphaltic concrete should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2 and 3 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingles in the contractor’s mix design submittal.

7.7 Erosion Control Measures

Soil at this site is susceptible to erosion by wind and water; therefore, erosion control measures should be carefully planned and installed before construction begins. Surface water runoff should be collected and directed away from sloped areas to prevent water from running down the slope face. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads. All erosion control methods should be in accordance with local jurisdiction standards.

8.0 CONCLUSION AND LIMITATIONS

This geotechnical site investigation report was prepared in accordance with accepted standard conventional principles and practices of geotechnical engineering. This investigation pertains only to material tested and observed as of the date of this report and is based upon proposed site development as described in the text herein. This report is a professional opinion containing recommendations established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. Soil conditions may differ between tested locations or over time. Slight variations may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent construction observation and testing to verify soil conditions are as encountered during our study and documented in this report-. in this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Columbia West cannot accept responsibility for deviations from recommendations described in this report. Future performance of structural facilities is often related to the degree of construction observation by qualified personnel. These services should be performed to the full extent recommended.

This report is not an environmental assessment and should not be construed as a representative warranty of site subsurface conditions. The discovery of adverse environmental conditions, or subsurface soils that deviate from those described in this report, should immediately prompt further investigation. The above statements are in lieu of all other statements expressed or implied.

This report was prepared solely for the client and is not to be reproduced without prior authorization from Columbia West. Final engineering plans and specifications for the project should be reviewed and approved by Columbia West as they relate to geotechnical and grading issues prior to final design approval. Columbia West is not responsible for independent conclusions or recommendations made by other parties based upon information presented in this report. Unless a particular service was expressly included in the scope, it was not performed and there should be no assumptions based upon services not provided. Additional report limitations and important information about this document are presented in Appendix F. This information should be carefully read and understood by the client and other parties reviewing this document.

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Jason F. Merritt, P.E.
Senior Project Engineer

Brett A. Shipton, PE, GE
Principal

REFERENCES

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ASCE 7-16, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 2016.

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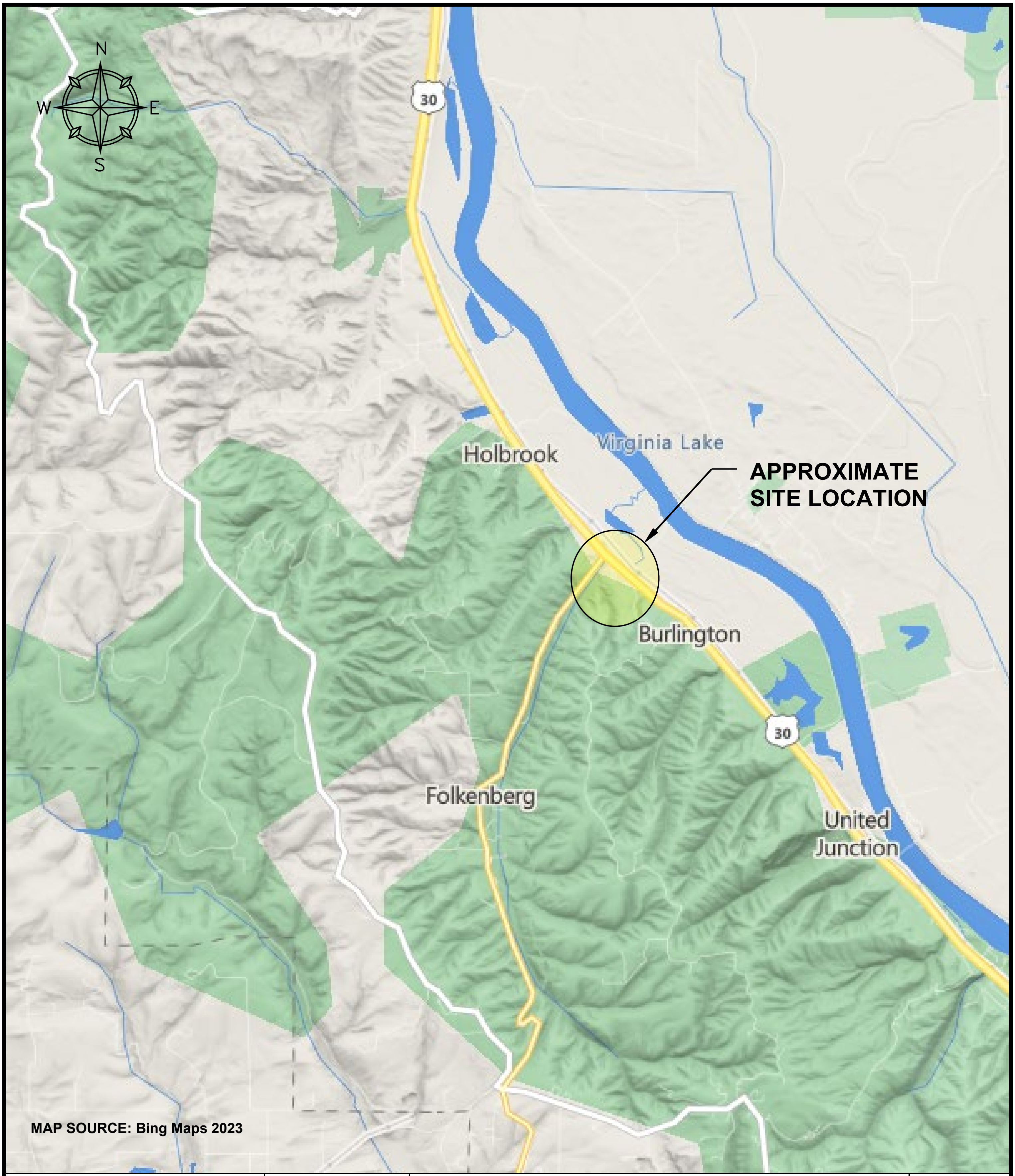
FIGURES



MAP SOURCE: Bing Maps 2023

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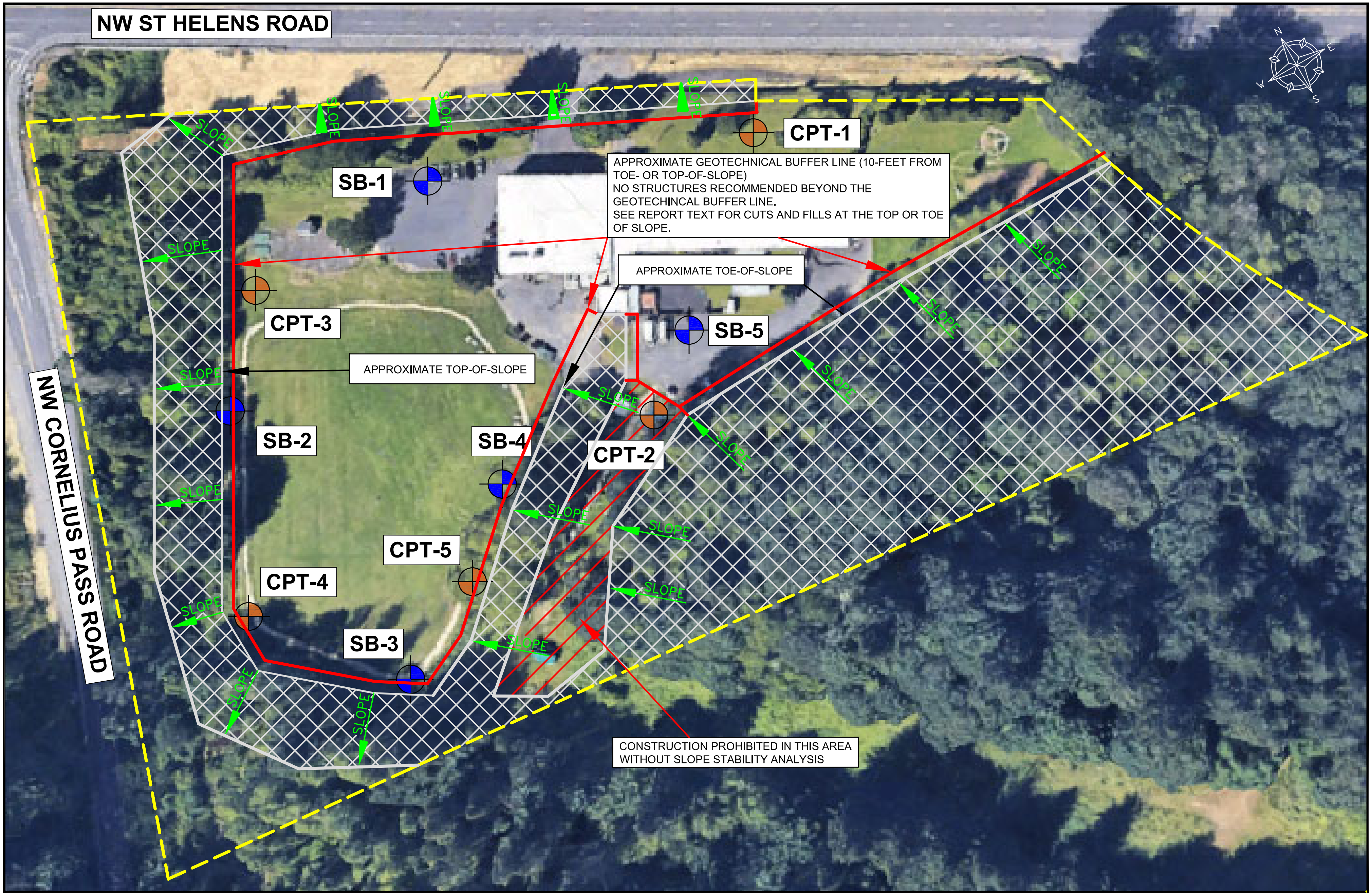
Columbia West
Engineering, Inc

Job No: 23020
Date: 2/20/23
Drawn: SSC
Checked: JFM

SITE LOCATION MAP
NARA RESIDENTIAL TREATMENT CENTER
PORTLAND, OREGON

FIGURE
1



APPROXIMATE GEOTECHNICAL BUFFER LINE (10-FEET FROM TOE- OR TOP-OF-SLOPE)
NO STRUCTURES RECOMMENDED BEYOND THE GEOTECHINCAL BUFFER LINE.
SEE REPORT TEXT FOR CUTS AND FILLS AT THE TOP OR TOE OF SLOPE.

APPROXIMATE TOE-OF-SLOPE

APPROXIMATE TOP-OF-SLOPE

CONSTRUCTION PROHIBITED IN THIS AREA WITHOUT SLOPE STABILITY ANALYSIS

⊕ APPROXIMATE CONE PENETRATION TEST LOCATION	⊕ APPROXIMATE SOIL BORING LOCATION	- - - - APPROXIMATE SITE BOUNDARY

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Project: 23020
Date: 02/24/23
Drawn: SSC
Checked: JFM

EXPLORATION LOCATION MAP
NARA RESIDENTIAL TREATMENT CENTER

NOTES:
1. SITE LOCATION: 17645 NW ST HELENS ROAD, PORTLAND, OREGON 97231.
2. SITE CONSISTS OF PROPERTY ID R325018 TOTALING APPROXIMATELY 10 ACRES.
3. DRAWING IS NOT TO SCALE.
4. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
5. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
6. CONE PENETRATION TESTS BACKFILLED WITH BENTONITE ON JANUARY 31, 2023.
6. SOIL BORINGS BACKFILLED WITH BENTONITE ON FEBRUARY 10, 2023.

FIGURE
2

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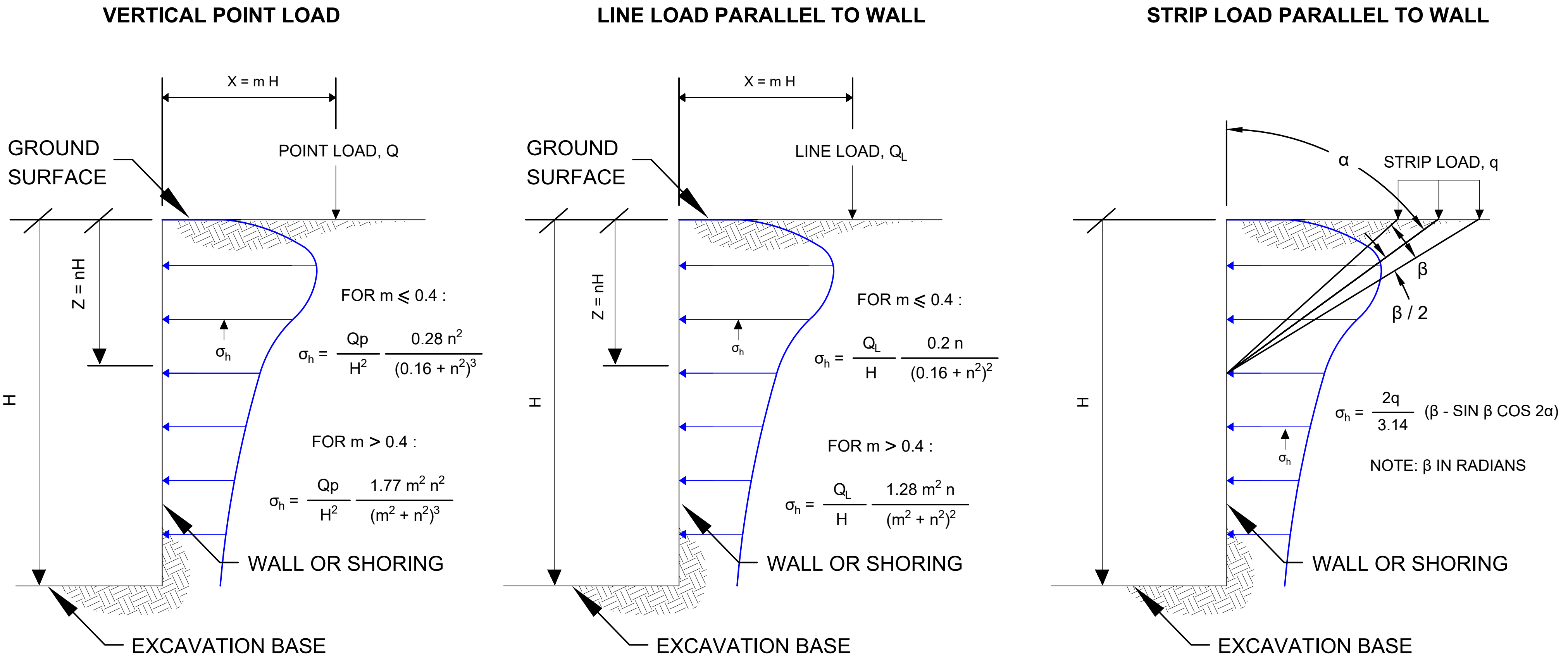
Job No: 23020
Date: 02/28/23
Drawn: EMU
Checked: JFM

LANDSLIDE INVENTORY MAP
NARA RESIDENTIAL
TREATMENT CENTER

NOTES:
1. SITE LOCATION: 17645 NW ST HELENS ROAD, PORTLAND, OREGON 97231.
2. SITE CONSISTS OF PROPERTY ID R325018 TOTALING APPROXIMATELY 10 ACRES.
3. MAP SOURCED FROM DOGAMI HAZVU.

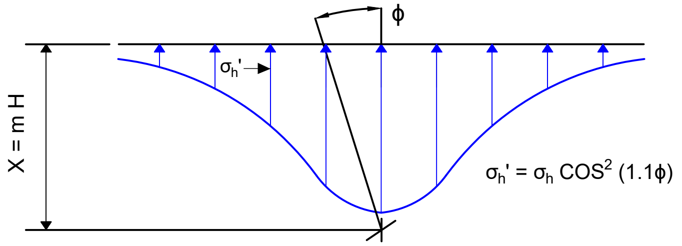
FIGURE
2B


VERTICAL POINT LOAD

LINE LOAD PARALLEL TO WALL

STRIP LOAD PARALLEL TO WALL

VERTICAL POINT LOAD
HORIZONTAL PRESSURE DISTRIBUTION



NOTES:
1. FIGURE SHOULD BE USED JOINTLY WITH RECOMMENDATIONS PRESENTED IN THE REPORT TEXT.
2. LATERAL EARTH PRESSURES ASSUME RIGID WALLS WITH BACKFILL MATERIALS HAVING A POISSON'S RATIO OF 0.5.
3. TOTAL LATERAL EARTH PRESSURES RESULTING FROM COMBINED LOADS MAY BE CALCULATED USING SUPERPOSITION.
4. DRAWING IS NOT TO SCALE.

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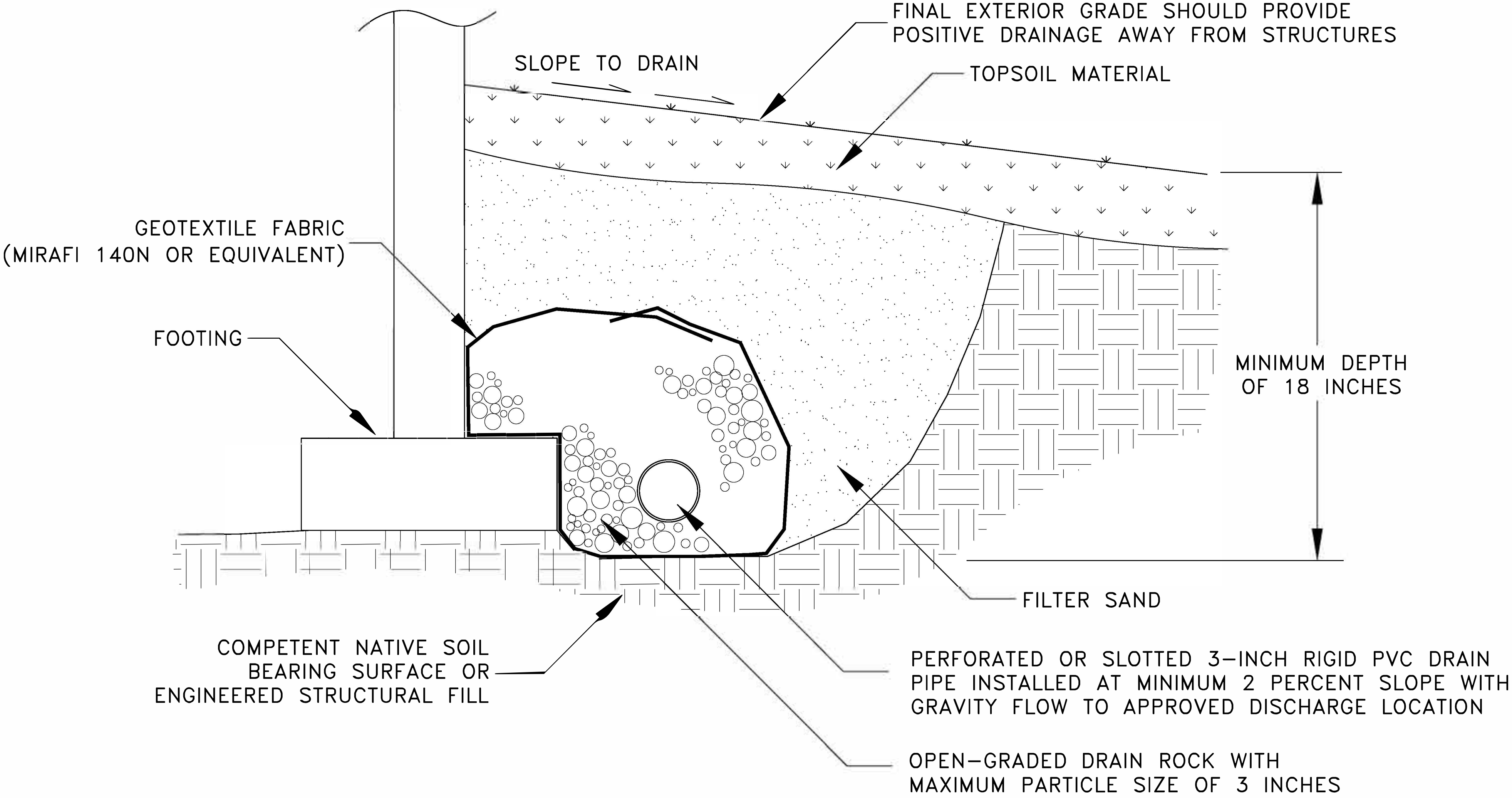
SURCHARGE–INDUCED LATERAL EARTH PRESSURES

FIGURE
3



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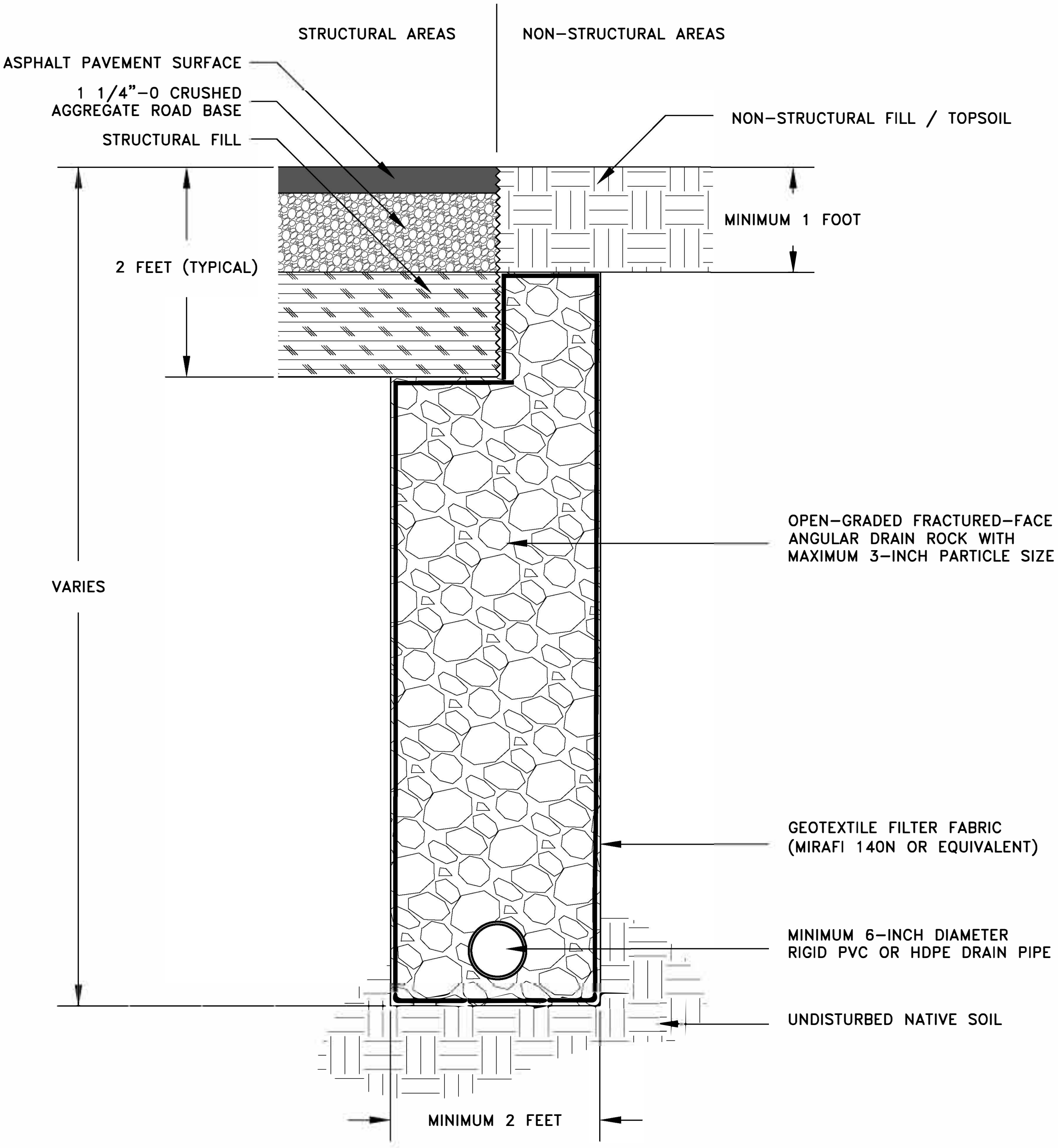
TYPICAL PERIMETER FOOTING
DRAIN DETAIL

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL PERIMETER FOOTING DRAIN DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE
4



NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION BASED UPON FIELD OBSERVATION AND SITE-SPECIFIC SOIL CONDITIONS.

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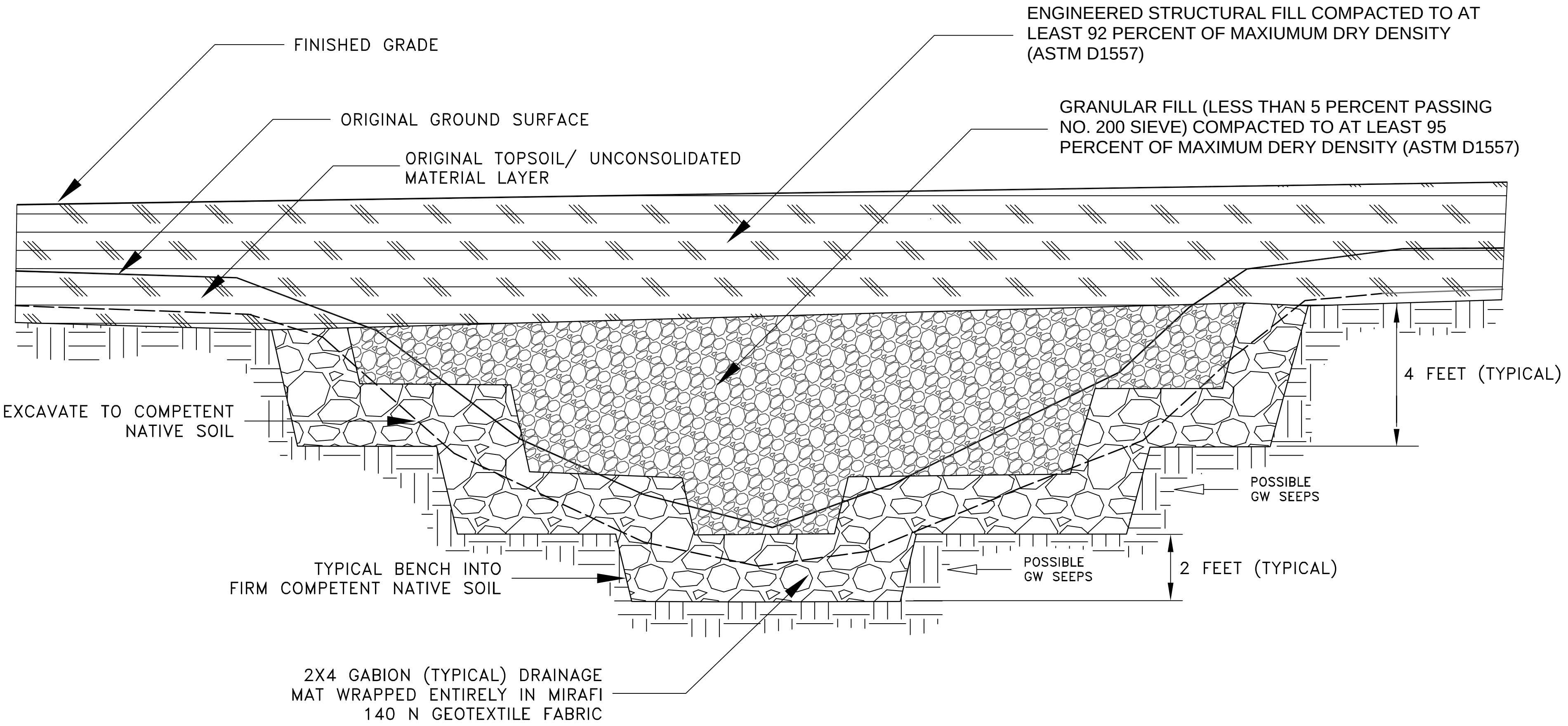
TYPICAL PERFORATED DRAINPIPE TRENCH DETAIL

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE
5

TYPICAL DRAINAGE MAT CROSS-SECTION



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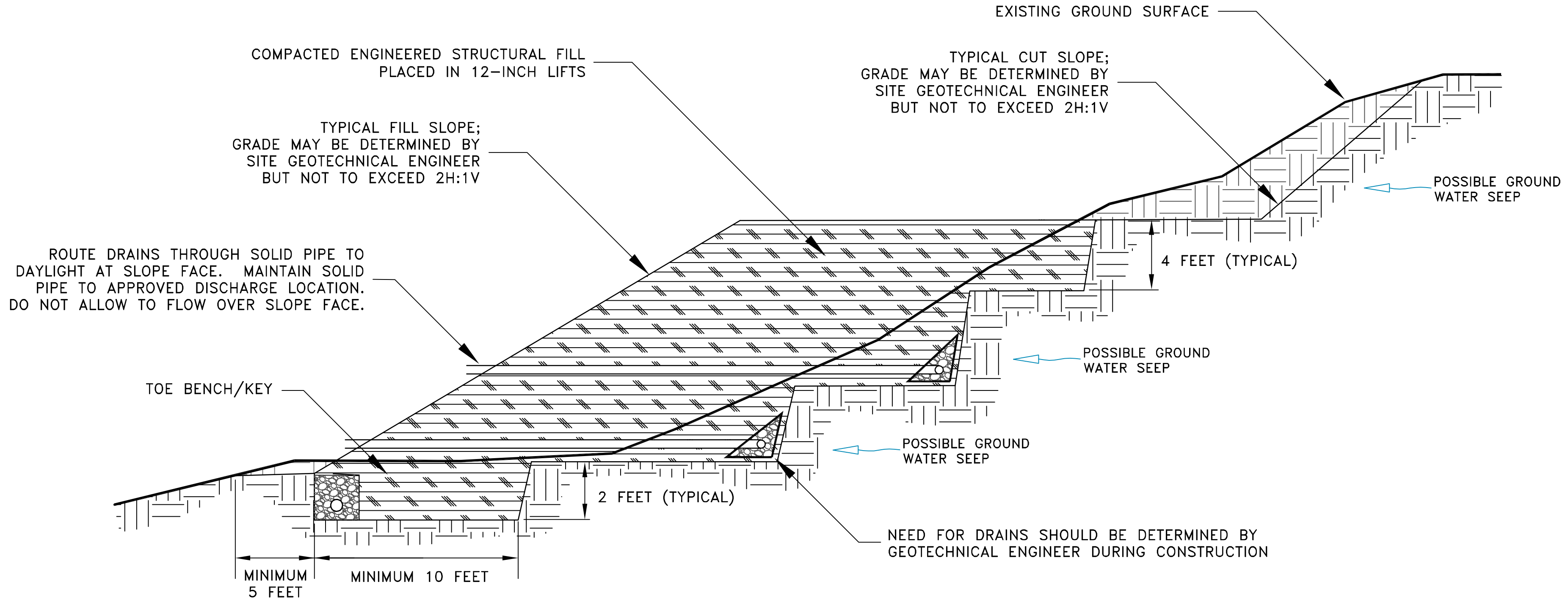
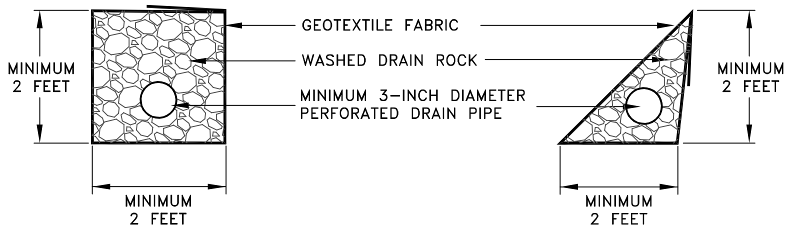
TYPICAL DRAINAGE MAT
CROSS SECTION

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL DRAINAGE MAT SECTION AND MAY NOT BE SITE-SPECIFIC.

FIGURE
6



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT WITH AOS BETWEEN No. 70 AND No. 100 SIEVE.

WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE No. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 3 INCHES.

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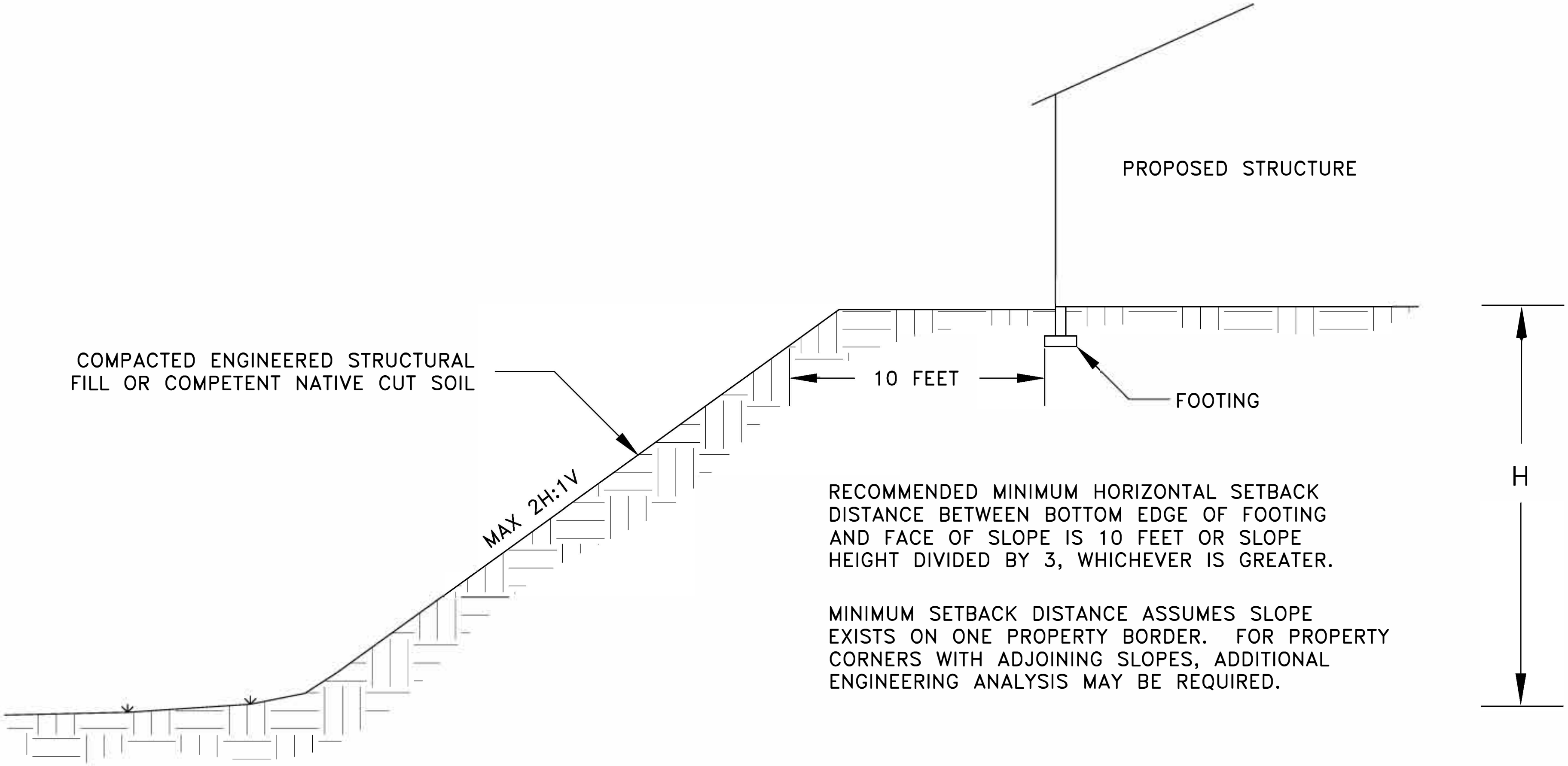
TYPICAL CUT AND FILL SLOPE CROSS SECTION

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL CUT AND FILL SLOPE CROSS SECTION AND MAY NOT BE SITE-SPECIFIC.

FIGURE

7



RECOMMENDED MINIMUM HORIZONTAL SETBACK DISTANCE BETWEEN BOTTOM EDGE OF FOOTING AND FACE OF SLOPE IS 10 FEET OR SLOPE HEIGHT DIVIDED BY 3, WHICHEVER IS GREATER.

MINIMUM SETBACK DISTANCE ASSUMES SLOPE EXISTS ON ONE PROPERTY BORDER. FOR PROPERTY CORNERS WITH ADJOINING SLOPES, ADDITIONAL ENGINEERING ANALYSIS MAY BE REQUIRED.

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TYPICAL MINIMUM FOUNDATION SLOPE SETBACK DETAIL

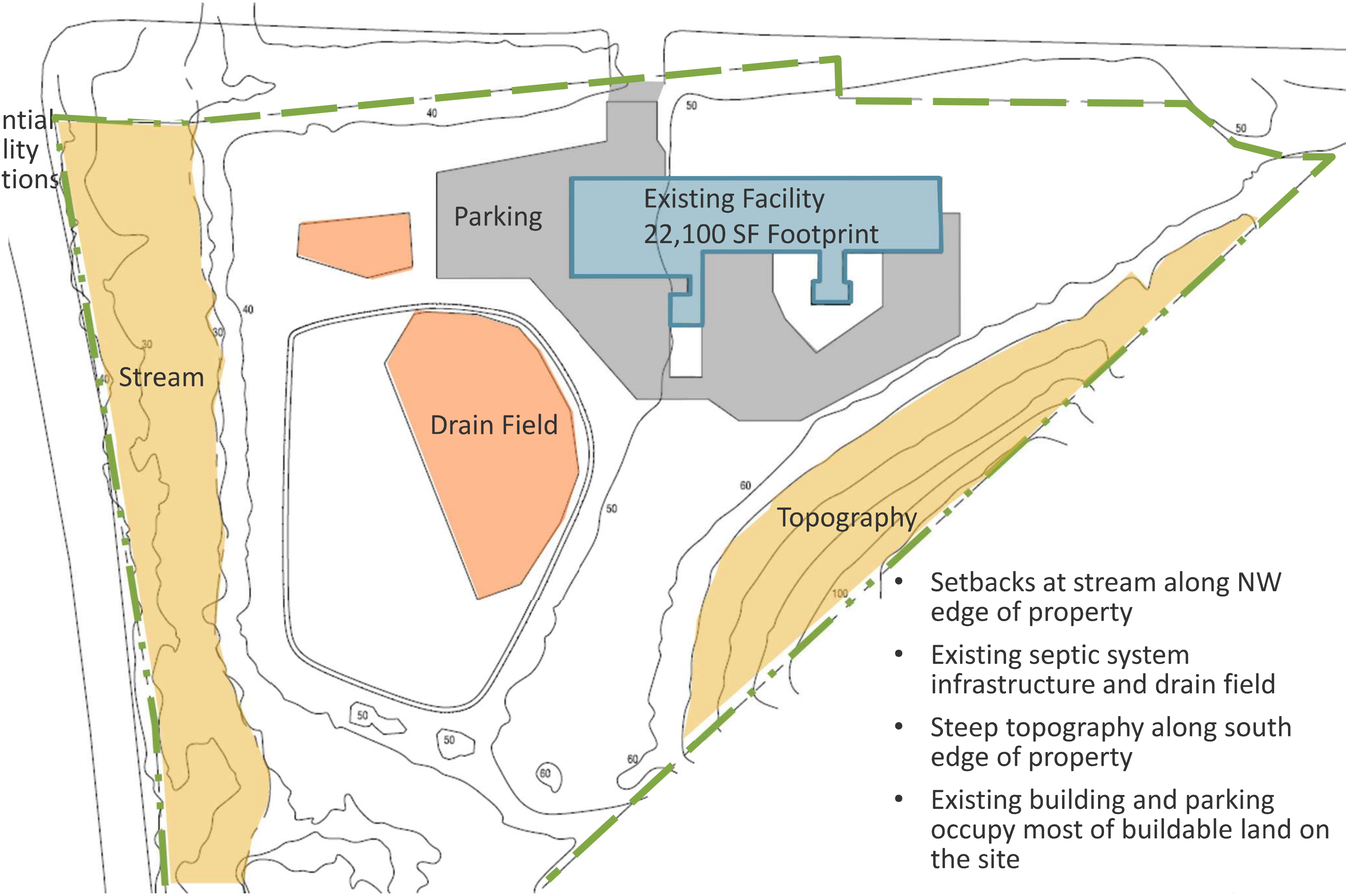
NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FOUNDATION SETBACK DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE
8

APPENDIX A
PRELIMINARY SITE PLANS

NARA

Existing Residential Treatment Facility and Site Limitations



- Setbacks at stream along NW edge of property
- Existing septic system infrastructure and drain field
- Steep topography along south edge of property
- Existing building and parking occupy most of buildable land on the site

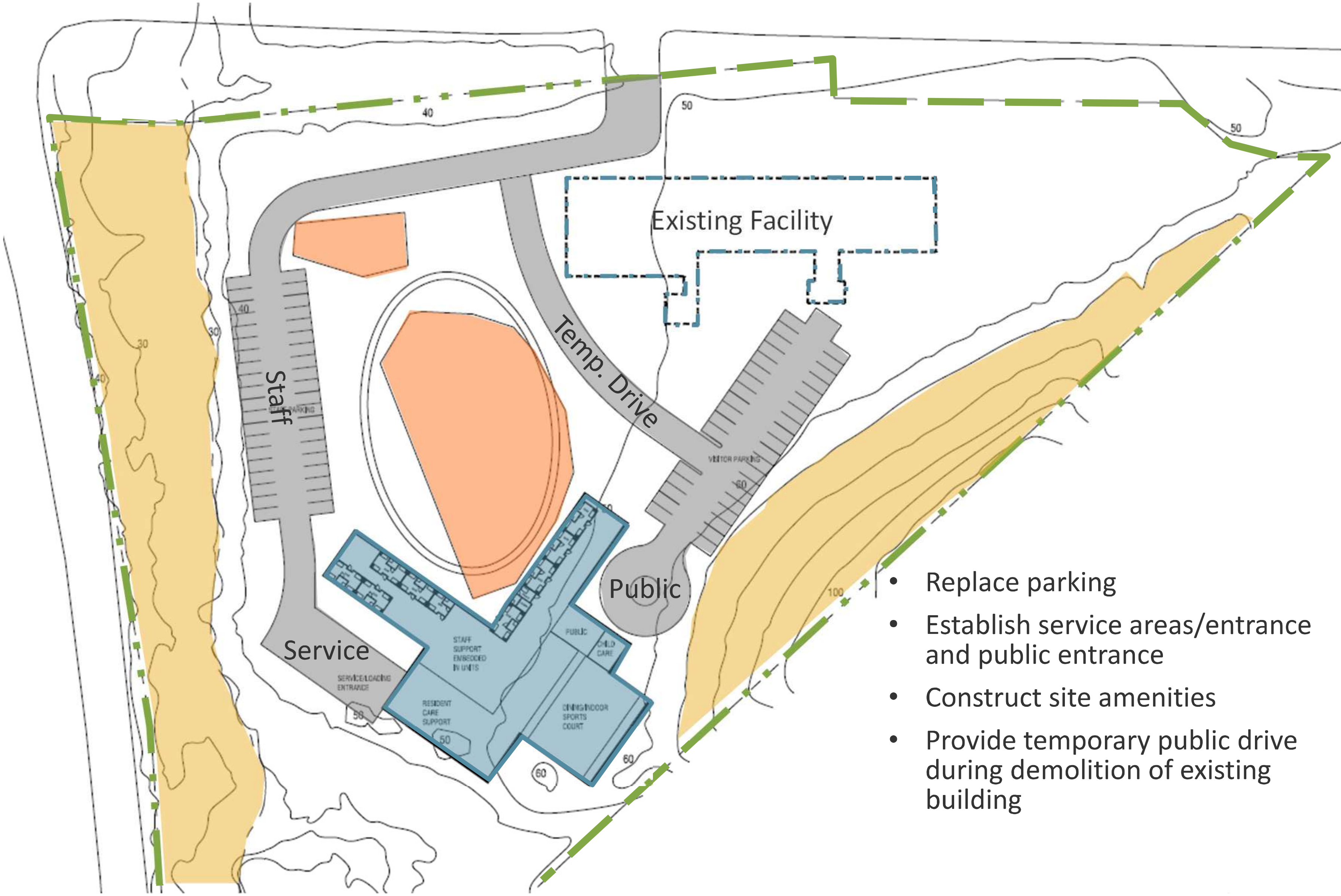
NARA

Replacement
Facility

- Two story facility
- ~50,000 BGSF, (28K-30K First floor footprint)
- Existing facility remains operational during construction
- Phasing and construction lay down areas to be considered

NARA

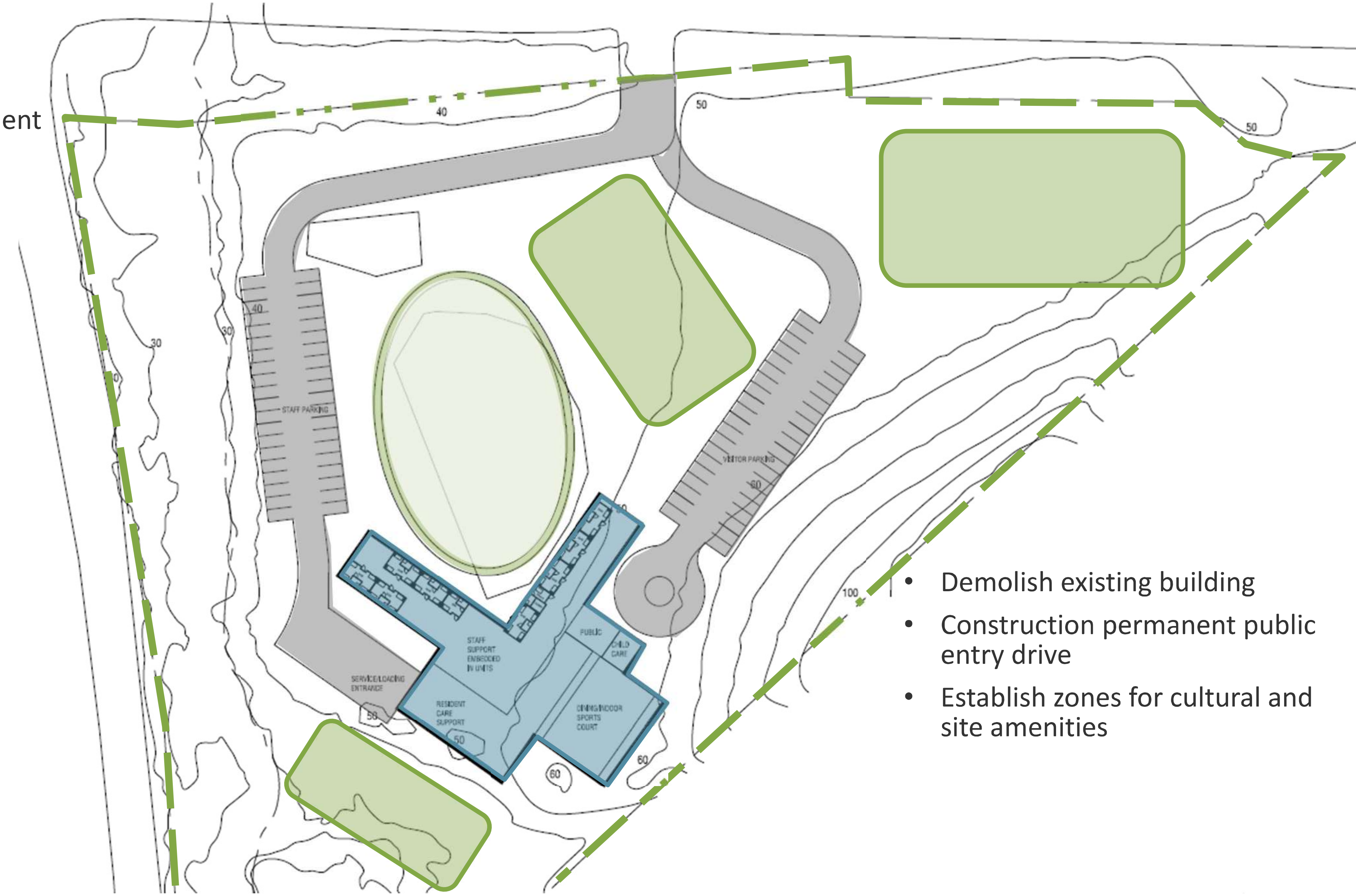
Replacement Facility



- Replace parking
- Establish service areas/entrance and public entrance
- Construct site amenities
- Provide temporary public drive during demolition of existing building

NARA

Full Replacement Build Out



- Demolish existing building
- Construction permanent public entry drive
- Establish zones for cultural and site amenities

APPENDIX B
LABORATORY TEST REPORTS

APPENDIX B

LABORATORY TESTING

CLASSIFICATION

The soil samples collected in the filed were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT

We determined the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

We completed particle-size analyses on select soil samples in general accordance with ASTM D6913. This test is a quantitative determination of the soil particle size distribution expressed as a percentage of dry soil weight. The test results are presented in this appendix.

ATTERBERG LIMITS

We determined the Atterberg Limits on selected samples in general accordance with ASTM D4318. Atterberg limits include the liquid limit, plastic limit, and the plasticity index of soils. These index properties are used to classify soils and for correlation with other engineering properties of soils. The test results are presented in this appendix.

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MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT	CLIENT	PROJECT NO.	REPORT DATE
NARA Residential Treatment Center Portland, Oregon	Native American Rehab Association of the Northwest, Inc. PO Box 1569 Portland, Oregon 97207	23020	02/21/23
		DATE SAMPLED: 02/10/23	
		SAMPLED BY: SSC/EMU	

LABORATORY TEST DATA

TEST PROCEDURE
ASTM D2216 - Method B, ASTM D1140

LAB ID	CONTAINER MASS	MOIST MASS + PAN	DRY MASS + PAN	AFTER WASH DRY MASS + PAN	MATERIAL DESCRIPTION	FIELD ID	SAMPLE DEPTH	MOISTURE CONTENT	PASSING NO. 200 SIEVE
S23-0181	87.79	295.42	252.60	-	brown silt with sand	SB1.2	2.5 feet	26.0%	-
S23-0182	556.04	780.35	724.71	sieved sample	Brown Sandy SILT	SB1.6	15 feet	33.0%	61.7%
S23-0183	556.52	760.48	724.82	653.40	gray/blue/brown silty sand	SB2.4	10 feet	21.2%	42.4%
S23-0184	86.85	313.50	262.87	-	gray/blue/brown silt with sand	SB2.6	20 feet	28.8%	-
S23-0185	541.93	774.04	735.06	sieved sample	Brown Sandy SILT	SB3.2	5 feet	20.2%	55.4%
S23-0186	87.25	233.77	200.81	-	brown silt	SB3.4	10 feet	29.0%	-
S23-0187	542.59	769.67	711.34	607.10	gray/blue sandy silt	SB3.7	20 feet	34.6%	61.8%
S23-0188	765.56	970.47	925.08	sieved sample	Tan/Orange Lean CLAY	SB4.1	2.5 feet	28.5%	99.2%
S23-0189	86.65	290.97	247.26	-	gray/blue clay with sand	SB4.4	10 feet	27.2%	-
S23-0190	86.99	305.54	260.17	-	gray/blue clay with sand	SB4.5	15 feet	26.2%	-
S23-0191	87.71	217.24	205.53	-	brown/gray sand with silt	SB5.1	2.5 feet	9.9%	-
S23-0192	87.22	202.36	193.07	-	brown/gray sand with silt	SB5.2	5 feet	8.8%	-
S23-0193	87.05	213.27	188.80	-	brown/gray sandy silt	SB5.5	15 feet	24.0%	-

NOTES:	DATE TESTED	TESTED BY
	02/16/23	AGM/KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
NARA Residential Treatment Center Portland, Oregon	Native American Rehab Association of the Northwest, Inc. PO Box 1569 Portland, Oregon 97207	23020	S23-0182
		REPORT DATE	FIELD ID
		02/21/23	SB1.6
		DATE SAMPLED	SAMPLED BY
		02/10/23	SSC/EMU

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Brown Sandy SILT	Soil Boring, SB-01 depth = 15 feet	ML, Sandy Silt

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-4(0)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter, moist prep, hand washed, 12" single sieve-set	ASTM D6913, Method A

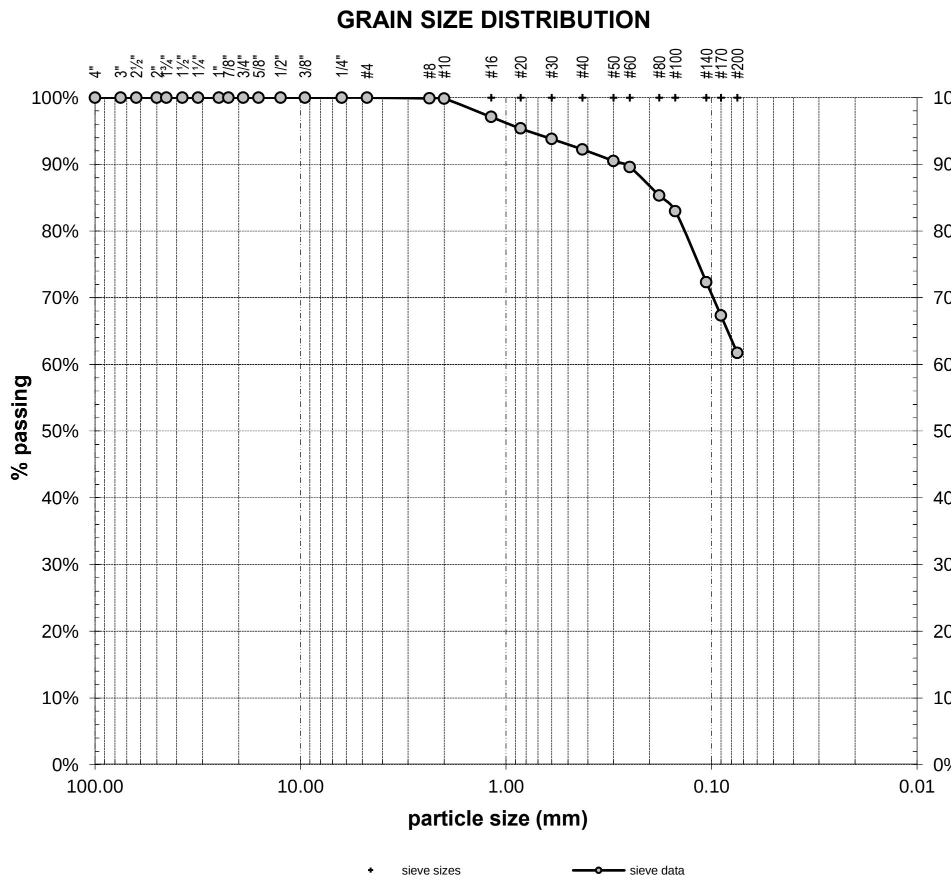
ADDITIONAL DATA

initial dry mass (g) =	168.67		
as-received moisture content =	33.0%	coefficient of curvature, C_C =	n/a
liquid limit =	28	coefficient of uniformity, C_U =	n/a
plastic limit =	25	effective size, $D_{(10)}$ =	n/a
plasticity index =	3	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 38.3%
% silt and clay = 61.7%

	SIEVE SIZE		PERCENT PASSING			
			SIEVE		SPECS	
	US	mm	act.	interp.	max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		97%		
	#20	0.850	95%			
	#30	0.600		94%		
	#40	0.425	92%			
	#50	0.300		90%		
	#60	0.250	90%			
	#80	0.180		85%		
	#100	0.150	83%			
	#140	0.106		72%		
	#170	0.090		67%		
	#200	0.075	62%			



DATE TESTED	TESTED BY
02/20/23	AGM/KMS

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ATTERBERG LIMITS REPORT

PROJECT NARA Residential Treatment Center Portland, Oregon	CLIENT Native American Rehab Association of the Northwest, Inc. PO Box 1569 Portland, Oregon 97207	PROJECT NO. 23020	LAB ID S23-0182
		REPORT DATE 02/21/23	FIELD ID SB1.6
		DATE SAMPLED 02/10/23	SAMPLED BY SSC/EMU

MATERIAL DATA

MATERIAL SAMPLED Brown Sandy SILT	MATERIAL SOURCE Soil Boring, SB-01 depth = 15 feet	USCS SOIL TYPE ML, Sandy Silt

LABORATORY TEST DATA

LABORATORY EQUIPMENT Liquid Limit Machine, Hand Rolled	TEST PROCEDURE ASTM D4318

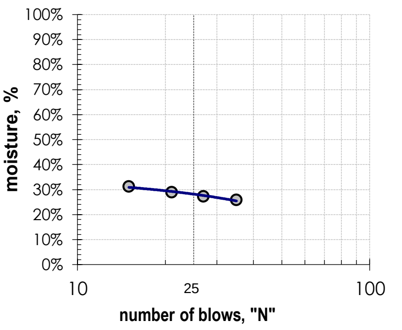
ATTERBERG LIMITS

liquid limit = 28
plastic limit = 25
plasticity index = 3

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.02	34.41	35.11	34.33
dry soil + pan weight, g =	31.32	31.51	31.91	31.18
pan weight, g =	20.85	20.87	20.84	21.11
N (blows) =	35	27	21	15
moisture, % =	25.8 %	27.3 %	28.9 %	31.3 %

LIQUID LIMIT



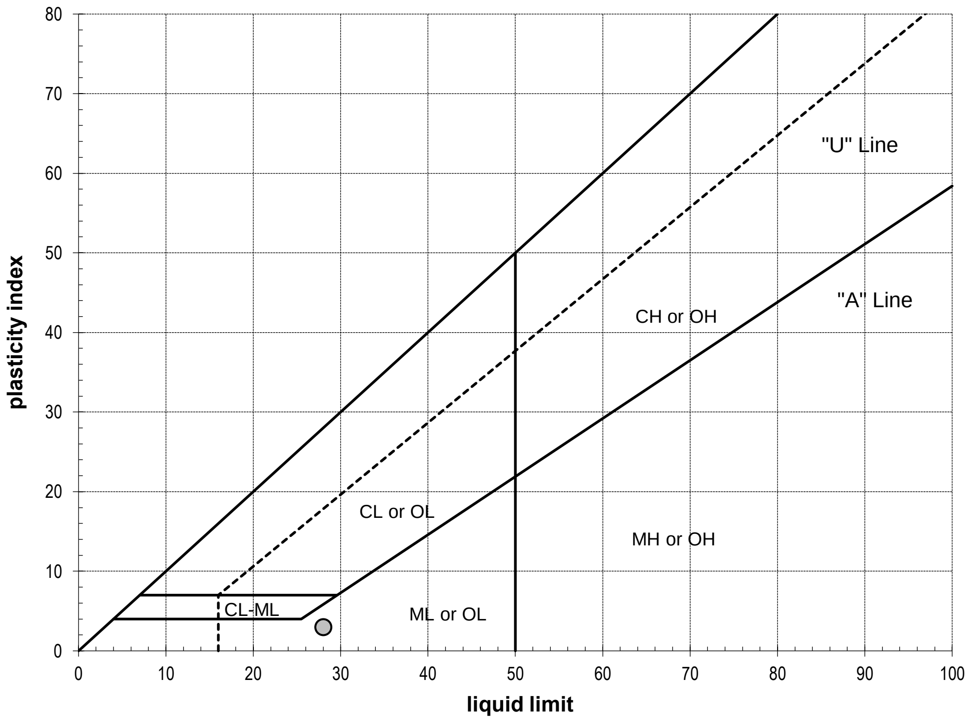
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.69	28.24		
dry soil + pan weight, g =	26.33	26.73		
pan weight, g =	20.95	20.80		
moisture, % =	25.3 %	25.5 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 38.3%
% silt and clay = 61.7%
% silt = n/a
% clay = n/a
moisture content = 33.0%

DATE TESTED 02/17/23	TESTED BY AGM/KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
NARA Residential Treatment Center Portland, Oregon	Native American Rehab Association of the Northwest, Inc. PO Box 1569 Portland, Oregon 97207	23020	S23-0185
		REPORT DATE	FIELD ID
		02/21/23	SB3.2
		DATE SAMPLED	SAMPLED BY
		02/10/23	SSC/EMU

MATERIAL DATA

MATERIAL SAMPLED
Brown Sandy SILT

MATERIAL SOURCE
Soil Boring, SB-03
depth = 5 feet

USCS SOIL TYPE
ML, Sandy Silt

SPECIFICATIONS
none

AASHTO CLASSIFICATION
A-4(0)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter, moist prep, hand washed, 12" single sieve-set

TEST PROCEDURE
ASTM D6913, Method A

ADDITIONAL DATA

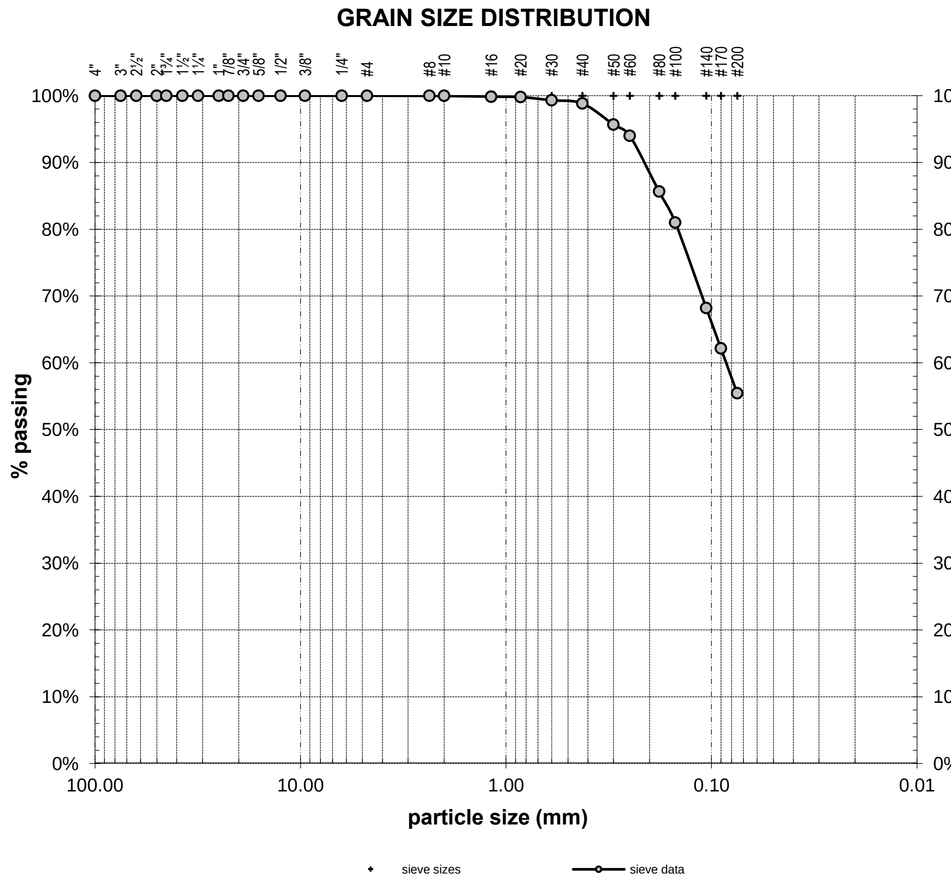
initial dry mass (g) = 193.13
as-received moisture content = 20.2%
liquid limit = -
plastic limit = -
plasticity index = NP
fineness modulus = n/a

coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = n/a
$D_{(60)}$ = 0.085 mm

SIEVE DATA

% gravel = 0.0%
% sand = 44.6%
% silt and clay = 55.4%

	SIEVE SIZE		PERCENT PASSING			
			SIEVE		SPECS	
	US	mm	act.	interp.	max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	100%			
	#30	0.600		99%		
	#40	0.425	99%			
	#50	0.300		96%		
	#60	0.250	94%			
	#80	0.180		86%		
	#100	0.150	81%			
	#140	0.106		68%		
	#170	0.090		62%		
	#200	0.075	55%			



DATE TESTED
02/20/23

TESTED BY
AGM/KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
NARA Residential Treatment Center Portland, Oregon	Native American Rehab Association of the Northwest, Inc. PO Box 1569 Portland, Oregon 97207	23020	S23-0188
		REPORT DATE	FIELD ID
		02/21/23	SB4.1
		DATE SAMPLED	SAMPLED BY
		02/10/23	SSC/EMU

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Tan/Orange Lean CLAY	Soil Boring, SB-04 depth = 2.5 feet	CL, Lean Clay

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-4(11)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter, moist prep, hand washed, 12" single sieve-set	ASTM D6913, Method A

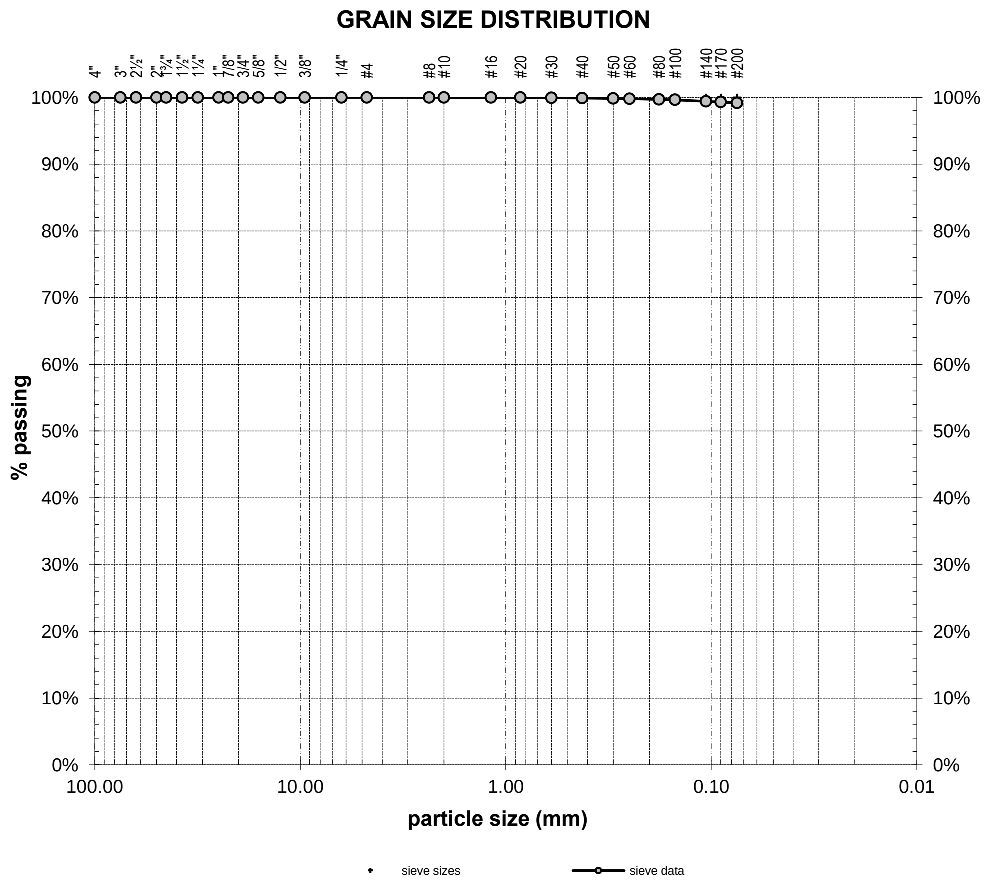
ADDITIONAL DATA

initial dry mass (g) =	159.52		
as-received moisture content =	28.5%	coefficient of curvature, C_C =	n/a
liquid limit =	33	coefficient of uniformity, C_U =	n/a
plastic limit =	23	effective size, $D_{(10)}$ =	n/a
plasticity index =	10	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 0.8%
% silt and clay = 99.2%

	SIEVE SIZE		PERCENT PASSING			
			SIEVE		SPECS	
	US	mm	act.	interp.	max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		100%		
	#20	0.850	100%			
	#30	0.600		100%		
	#40	0.425	100%			
	#50	0.300		100%		
	#60	0.250	100%			
	#80	0.180		100%		
	#100	0.150	100%			
	#140	0.106		99%		
	#170	0.090		99%		
	#200	0.075	99%			



DATE TESTED	TESTED BY
02/20/23	AGM/KMS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
NARA Residential Treatment Center Portland, Oregon	Native American Rehab Association of the Northwest, Inc. PO Box 1569 Portland, Oregon 97207	23020	S23-0188
		REPORT DATE 02/21/23	FIELD ID SB4.1
		DATE SAMPLED 02/10/23	SAMPLED BY SSC/EMU

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Tan/Orange Lean CLAY	Soil Boring, SB-04 depth = 2.5 feet	CL, Lean Clay

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

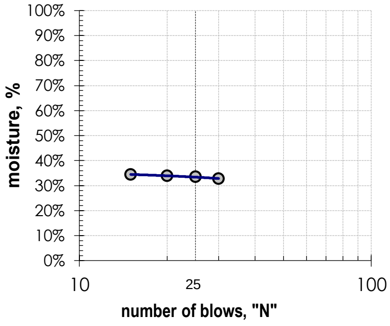
ATTERBERG LIMITS

liquid limit = 33
plastic limit = 23
plasticity index = 10

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.59	31.13	32.46	32.31
dry soil + pan weight, g =	28.92	28.51	29.53	29.35
pan weight, g =	20.77	20.69	20.89	20.75
N (blows) =	30	25	20	15
moisture, % =	32.8 %	33.5 %	33.9 %	34.4 %

LIQUID LIMIT



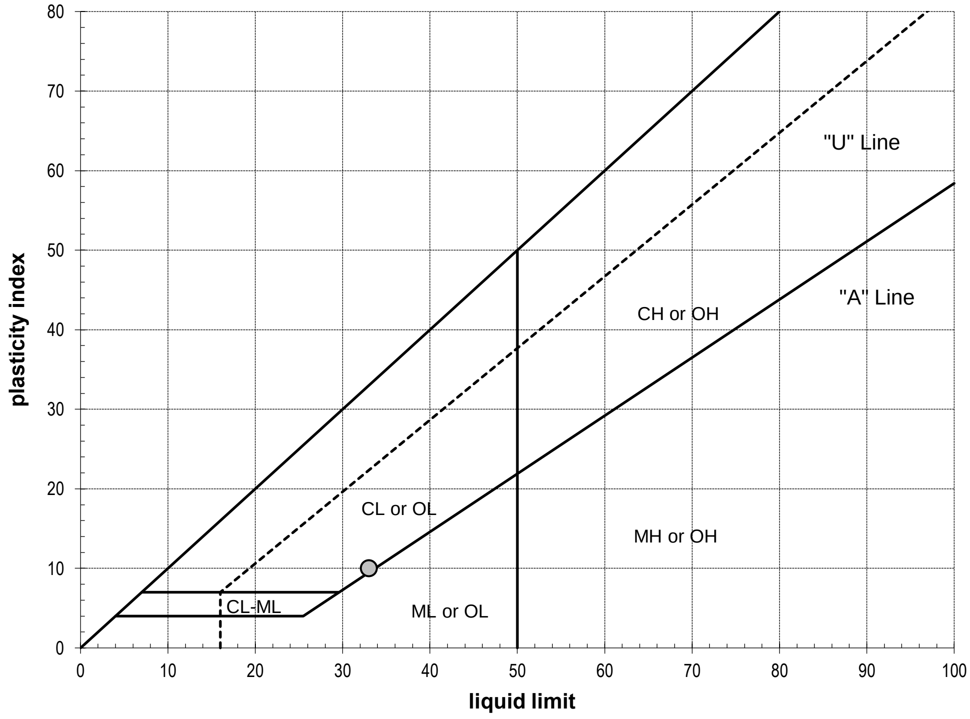
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.65	30.32		
dry soil + pan weight, g =	26.36	28.52		
pan weight, g =	20.80	20.91		
moisture, % =	23.2 %	23.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel =	0.0%
% sand =	0.8%
% silt and clay =	99.2%
% silt =	n/a
% clay =	n/a
moisture content =	28.5%

DATE TESTED	TESTED BY
02/17/23	AGM/KMS

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APPENDIX C
SOIL BORING LOGS

APPENDIX C

SUBSURFACE EXPLORATION PROGRAM

GENERAL

We explored subsurface conditions at the site by drilling five borings using a trailer-mounted drill rig. The borings were drilled by Dan Fischer Excavating, Inc. on February 10, 2023, to a maximum depth of 21.5 feet BGS. The test pit logs are presented in this appendix.

SOIL SAMPLING

Representative disturbed soil samples were collected from the auger bucket for classification and laboratory testing.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the Unified Soil Classification System presented in Appendix E. The exploration log indicates the depths at which the soil or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Soil classifications are shown on the exploration logs.

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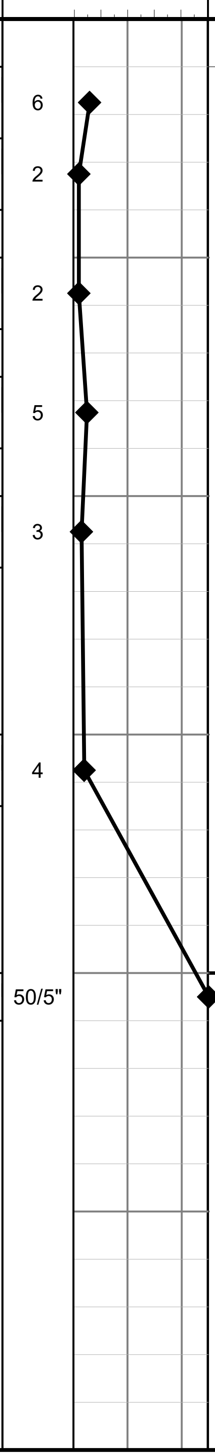
SOIL BORING LOG

PROJECT NAME NARA Residential Treatment Center	CLIENT NARA	PROJECT NO. 23020	BORING NO. SB-1
PROJECT LOCATION Portland, Oregon	DRILLING CONTRACTOR Dan Fisher Excavating, Inc DRILL RIG Trailer Rig	TECHNICIAN SSC/EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Solid Stem Auger SAMPLING METHOD SPT/Shelby	START DATE 2/10/23	START TIME 0759
REMARKS None	GROUNDWATER DEPTH Perched groundwater seepage at 20 feet BGS	FINISH DATE 2/10/23	FINISH TIME 0856



Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
							FILL. Approximately 3 to 4 inches of apshalt underlain by 10 to 12 inches of baserock.					
		SPT SB1.1	6	ML			SILT with sand, brown, damp, medium stiff, nonplastic to low plasticity.					
		SPT SB1.2	2				Becomes very soft to soft with trace organics, trace fine gravels, and mottling at 2.5 feet.		26.0			
		SPT SB1.3	2									
							Becomes Sandy SILT, moist, medium stiff at 7.5 feet.					
		SPT SB1.4	5				Trace organics at 8 feet.					
		SPT SB1.5	3				Becomes soft at 10 feet.					
		SPT SB1.6	4				Becomes soft to medium stiff at 15 feet.		33.0	61.7	28	3
							Difficult drilling from 18.5 to 19 feet. Gravels and possible cobbles observed in auger cuttings.					
		SPT SB1.7	50/5"	▼			Becomes very hard at 20 feet.					
							Bottom of soil boring at 20.5 feet bgs. Perched groundwater seepage observed at 20 feet on 2/10/23.					

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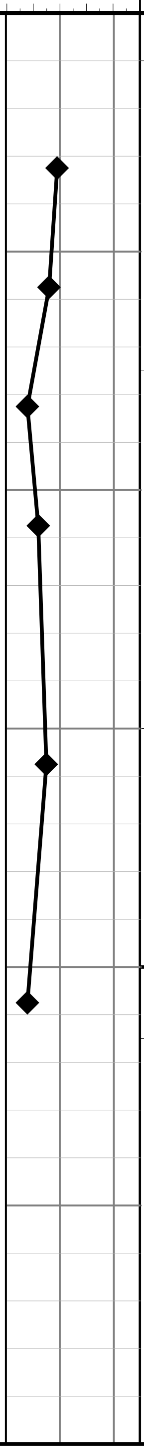
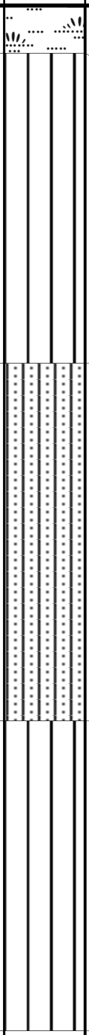
SOIL BORING LOG

PROJECT NAME NARA Residential Treatment Center	CLIENT NARA		PROJECT NO. 23020	BORING NO. SB-2
PROJECT LOCATION Portland, Oregon	DRILLING CONTRACTOR Dan Fisher Excavating, Inc	DRILL RIG Trailer Rig	TECHNICIAN SSC/EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Solid Stem Auger	SAMPLING METHOD SPT/Shelby	START DATE 2/10/23	START TIME 0920
REMARKS None	GROUNDWATER DEPTH Perched groundwater seepage at 20 feet BGS		FINISH DATE 2/10/23	FINISH TIME 1000





Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0							Approximate 3 to 4 inches of root zone underlain by 8 to 10 inches of topsoil.					
				ML			SILT with sand, brown, moist, very stiff, nonplastic to low plasticity.					
2												
		SPT SB2.1	19									
4												
							Increased fine-textured sand content at 5 feet.					
		SPT SB2.2	16									
6												
							Becomes brown and gray at 7.5 feet.					
8		SPT SB2.3	8	SM			Silty SAND, gray and blue, moist, loose, nonplastic, fine-textured sand.					
10		SPT SB2.4	12				Becomes medium dense at 10 feet.		21.2	42.4		
12												
14							Trace decomposed wood debris observed at 15.5 feet.					
		SPT SB2.5	15	ML			SILT with sand, gray trace brown and blue, stiff to very stiff, nonplastic to low plasticity, fine-textured sand.					
16												
18												
20		SPT SB2.6	8	▼			Becomse loose at 20 feet.		28.8			
22							Bottom of soil boring at 21.5 feet bgs. Perched groundwater seepage observed at 20 feet on 2/10/23.					
24												
26												
28												
30												

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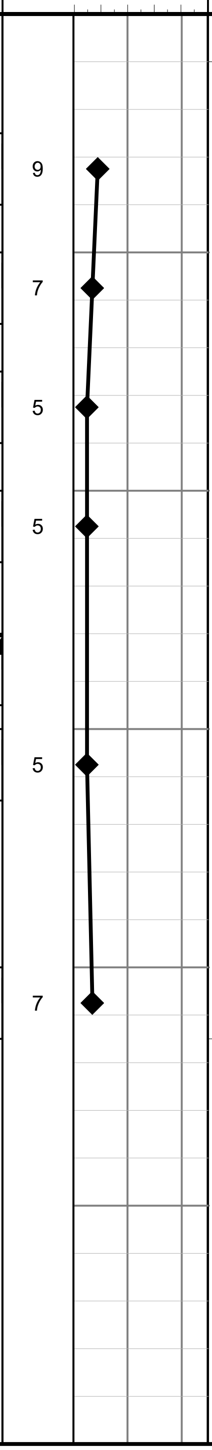
SOIL BORING LOG

PROJECT NAME NARA Residential Treatment Center	CLIENT NARA		PROJECT NO. 23020	BORING NO. SB-3
PROJECT LOCATION Portland, Oregon	DRILLING CONTRACTOR Dan Fisher Excavating, Inc	DRILL RIG Trailer Rig	TECHNICIAN SSC/EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Solid Stem Auger	SAMPLING METHOD SPT/Shelby	START DATE 2/10/23	START TIME 1009
REMARKS None	GROUNDWATER DEPTH Not encountered		FINISH DATE 2/10/23	FINISH TIME 1103



Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0							Approximate 2 to 3 inches of root zone underlain by 6 to 8 inches of topsoil.					
				ML			Sandy SILT, brown, moist, stiff, nonplastic to low plasticity.					
2		SPT SB3.1	9									
4							Becomes medium stiff at 5 feet.					
6		SPT SB3.2	7						20.2	55.4	NP	NP
8		SPT SB3.3	5				Increased fine-textured sand content at 7.5 feet.					
10		SPT SB3.4	5				Becomes SILT at 10 feet.		29.0			
12												
14		SHELBY SB3.5										
16		SPT SB3.6	5				Becomes sandy SILT, blue and gray with trace decomposed wood debris observed at 15 feet.					
18												
20		SPT SB3.7	7				Becomes very moist at 20 feet..		34.6	61.8		
22							Bottom of soil boring at 21.5 feet bgs. Groundwater not encountered on 2/10/23.					
24												
26												
28												
30												

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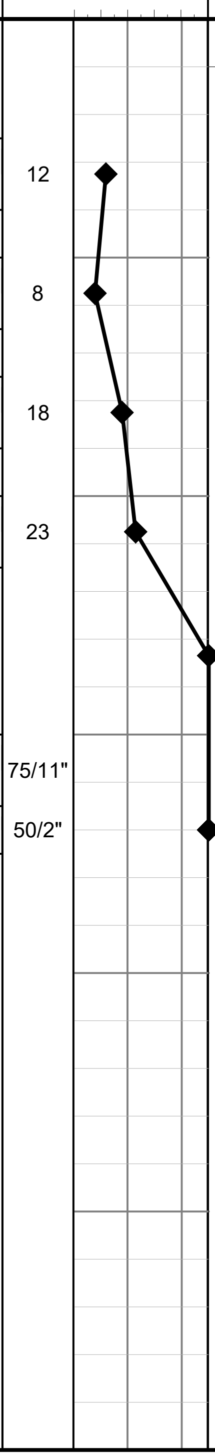
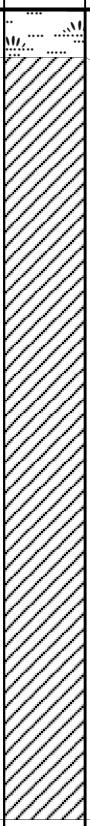
SOIL BORING LOG

PROJECT NAME NARA Residential Treatment Center	CLIENT NARA	PROJECT NO. 23020	BORING NO. SB-4
PROJECT LOCATION Portland, Oregon	DRILLING CONTRACTOR Dan Fisher Excavating, Inc DRILL RIG Trailer Rig	TECHNICIAN SSC/EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Solid Stem Auger SAMPLING METHOD SPT/Shelby	START DATE 2/10/23	START TIME 1109
REMARKS None	GROUNDWATER DEPTH Not encountered	FINISH DATE 2/10/23	FINISH TIME 1201





Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0							Approximate 3 to 4 inches of root zone underlain by 8 to 10 inches of topsoil.					
				CL			Lean CLAY, tan and orange, moist, stiff, nonplastic to low plasticity.					
2		SPT SB4.1	12						28.5	99.2	33	10
4							Becomes tan and gray and medium stiff at 5 feet.					
6		SPT SB4.2	8									
8		SPT SB4.3	18				Becomes lean CLAY with sand, gray and blue and very stiff at 7.5 feet.					
10		SPT SB4.4	23						27.2			
12							Trace decomposed wood debris observed at 11 feet.					
14							Becomes very hard at 15 feet.					
16		SPT SB4.5	75/11"						26.2			
		SPT SB4.6	50/2"									
18							Bottom of soil boring at 17 feet bgs. Groundwater not encountered on 2/10/23.					
20												
22												
24												
26												
28												
30												

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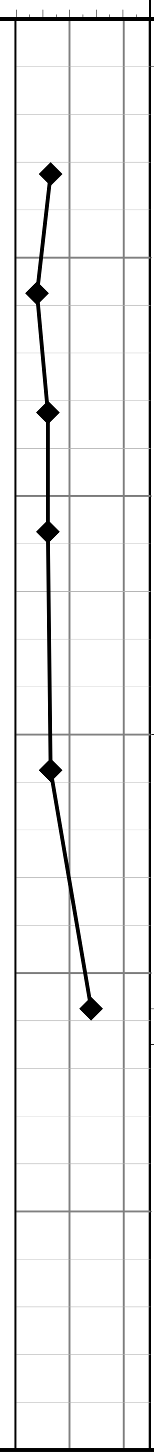
SOIL BORING LOG

PROJECT NAME NARA Residential Treatment Center	CLIENT NARA		PROJECT NO. 23020	BORING NO. SB-5
PROJECT LOCATION Portland, Oregon	DRILLING CONTRACTOR Dan Fisher Excavating, Inc	DRILL RIG Trailer Rig	TECHNICIAN SSC/EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD Solid Stem Auger	SAMPLING METHOD SPT/Shelby	START DATE 2/10/23	START TIME 1209
REMARKS None	GROUNDWATER DEPTH Not encountered		FINISH DATE 2/10/23	FINISH TIME 1304



Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0							FILL. Approximately 3 to 4 inches of apshalt underlain by 10 to 12 inches of baserock.					
				SP-SM			SAND with silt, brown and gray, moist, medium dense, nonplastic to low plasticity, fine-texuted sand.					
2												
		SPT SB5.1	13						9.9			
4							Increased fine-textured sand content at 4 feet.					
		SPT SB5.2	8				Becomes loose at 5 feet.		8.8			
6												
							Becomes medium dense at 7.5 feet.					
8		SPT SB5.3	12									
10		SPT SB5.4	12									
12												
14												
		SPT SB5.5	13	ML			Sandy SILT, brown and ray, moist, stiff, nonplastic to low plasticity.		24.0			
16												
18												
20		SPT SB5.6 (A/B)	28				Becomes SILT, brown with black and orange mottles, and very stiff at 20 feet.					
				SP-SM			SAND with silt, brown and orange, moist, medium dense, nonplasic to low plasticity, fine-texuted sand.					
22							Bottom of soil boring at 21.5 feet bgs. Groundwater not encountered on 2/10/23.					
24												
26												
28												
30												

APPENDIX D
SOIL CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

COMPONENT	ASTM/USCS		AASHTO	
	size range	sieve size range	size range	sieve size range
Cobbles	> 75 mm	greater than 3 inches	> 75 mm	greater than 3 inches
Gravel	75 mm – 4.75 mm	3 inches to No. 4 sieve	75 mm – 2.00 mm	3 inches to No. 10 sieve
Coarse	75 mm – 19.0 mm	3 inches to 3/4-inch sieve	-	-
Fine	19.0 mm – 4.75 mm	3/4-inch to No. 4 sieve	-	-
Sand	4.75 mm – 0.075 mm	No. 4 to No. 200 sieve	2.00 mm – 0.075 mm	No. 10 to No. 200 sieve
Coarse	4.75 mm – 2.00 mm	No. 4 to No. 10 sieve	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve
Medium	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve	-	-
Fine	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve
Fines (Silt and Clay)	< 0.075 mm	Passing No. 200 sieve	< 0.075 mm	Passing No. 200 sieve

Consistency for Cohesive Soil

CONSISTENCY	SPT N-VALUE (BLOWS PER FOOT)	D&M N-VALUE (BLOWS PER FOOT)	POCKET PENETROMETER (UNCONFINED COMPRESSIVE STRENGTH, tsf)
Very Soft	Less than 2	Less than 3	less than 0.25
Soft	2 to 4	3 to 6	0.25 to 0.50
Medium Stiff	4 to 8	6 to 12	0.50 to 1.0
Stiff	8 to 15	12 to 25	1.0 to 2.0
Very Stiff	15 to 30	25 to 65	2.0 to 4.0
Hard	30 to 60	65 to 145	greater than 4.0
Very Hard	greater than 60	greater than 145	-

RELATIVE DENSITY	SPT N-VALUE (BLOWS PER FOOT)	D&M N-VALUE (BLOWS PER FOOT)
Very Loose	0 to 4	0 to 11
Loose	4 to 10	11 to 26
Medium Dense	10 to 30	26 to 74
Dense	30 to 50	74 to 120
Very Dense	more than 50	More than 120

Relative Density for Granular Soil

Moisture Designations

TERM	FIELD IDENTIFICATION
Dry	No moisture. Dusty or dry.
Damp	Some moisture. Cohesive soils are usually below plastic limit and are moldable.
Moist	Grains appear darkened, but no visible water is present. Cohesive soils will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grains. Sand and silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is much wetter than optimum moisture content and is above plastic limit.

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing .075 mm)			Silt-Clay Materials (More than 35 Percent Passing 0.075)			
Group Classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, percent passing:							
2.00 mm (No. 10)	-	-	-				
0.425 mm (No. 40)	50 max	51 min	-	-	-	-	-
0.075 mm (No. 200)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.		10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to good			Fair to poor			

Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

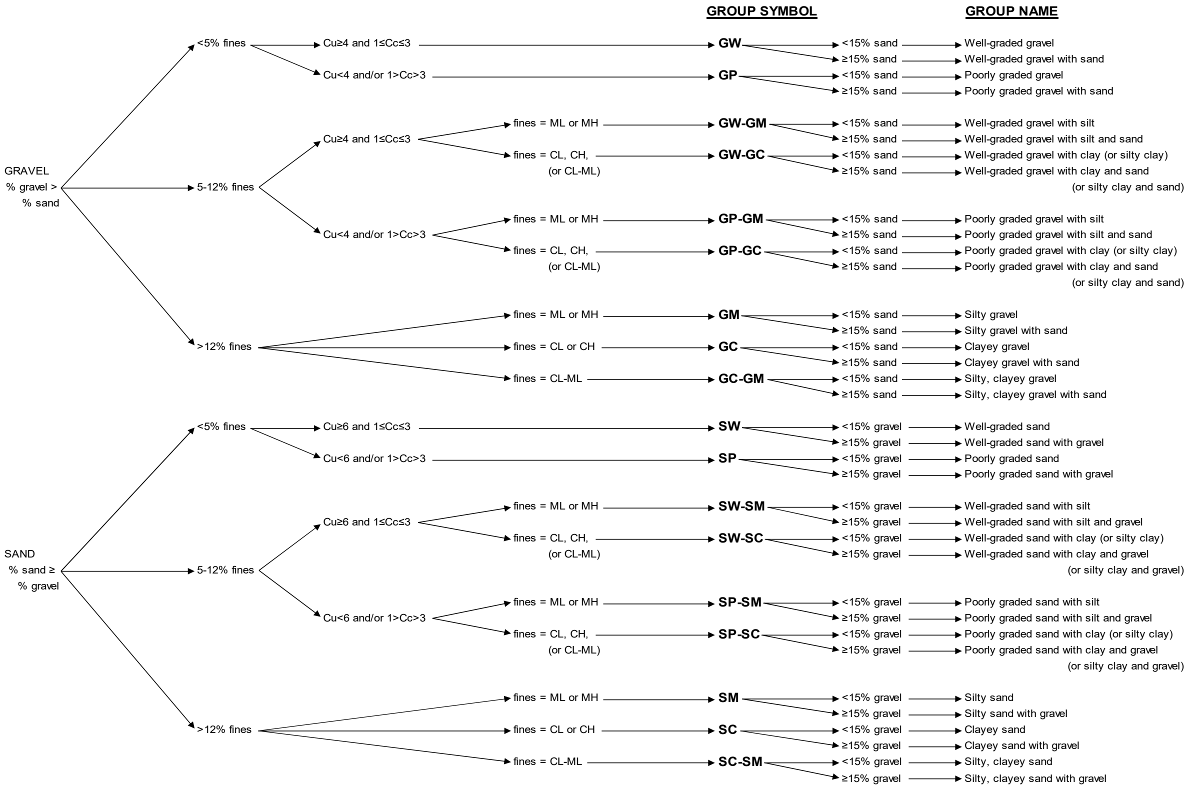
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max	-	-	-	-	-	-	-	-	-	-
0.425 mm (No. 40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11min
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General ratings as subgrade	Excellent to Good							Fair to poor			

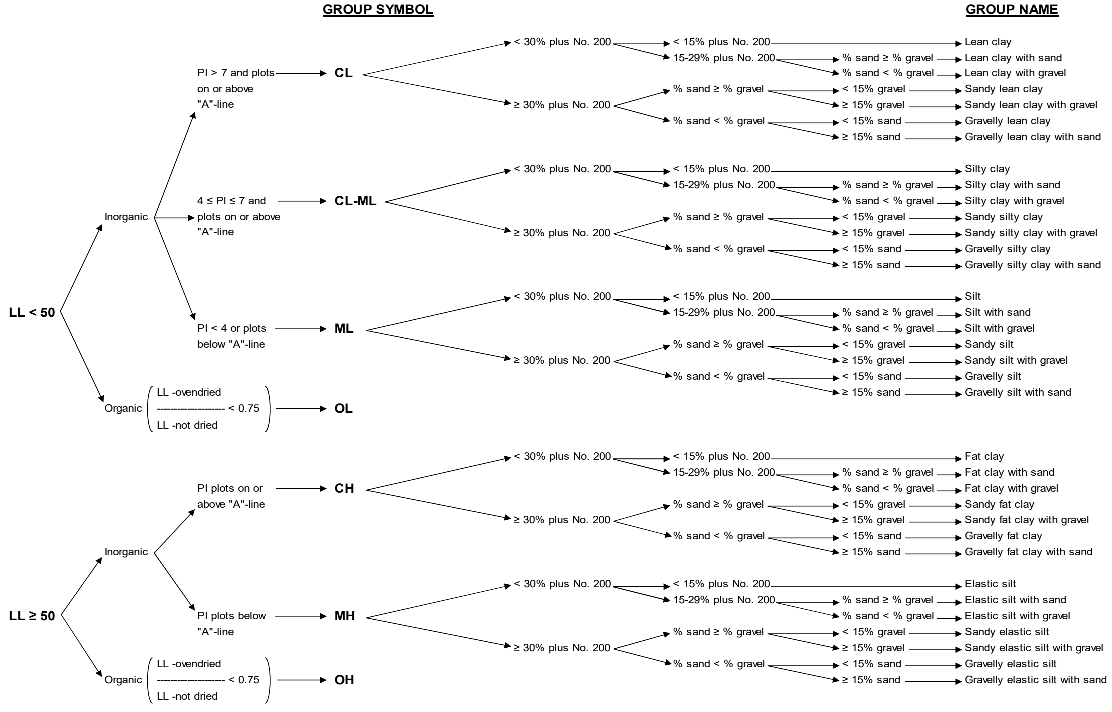
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

UNIFIED SOIL CLASSIFICATION SYSTEM



Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)



Flow Chart for Classifying Fine-Grained Soil (50% or More Passes No. 200 Sieve)

APPENDIX E
CPT RESULTS REPORT

PRESENTATION OF SITE INVESTIGATION RESULTS

St Helens Road CPT

Prepared for:

Columbia West Engineering, Inc.

ConeTec Job No: 23-59-25320

Project Start Date: 31-JAN-2023
Project End Date: 31-JAN-2023
Report Date: 02-FEB-2023

Prepared by:

ConeTec Inc.
3530 NW St Helens Rd
Portland, OR 97210

Tel: (253) 397-4861

ConeTecWA@conetec.com
www.conetec.com
www.conetecdataservices.com

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Inc. for Columbia West Engineering, Inc. at 17645 NW St Helens Road, Portland, OR 97231. The program consisted of five (5) cone penetration tests, with one (1) location being repushed due to shallow refusal.

Project Information

Project	
Client	Columbia West Engineering, Inc.
Project	St Helens Road CPT
ConeTec project number	23-59-25320

An aerial overview from Google Earth including the CPTu test locations is presented below.

Rig Description	Deployment System	Test Type
C02-023_25-Ton Truck Rig	Integrated Push Cylinders	CPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu	Consumer grade GPS	4326

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC870:T1500F15U35	870	15	225	1500	15	35
Cone 870 was used for all CPTu soundings						

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	• Advanced plots with Ic, Su, phi and N(60)/N1(60) • Soil Behaviour Type (SBT) scatter plots

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile.

Limitations

This report has been prepared for the exclusive use of Columbia West Engineering, Inc. (Client) for the project titled “St Helens Road CPT”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Inc. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

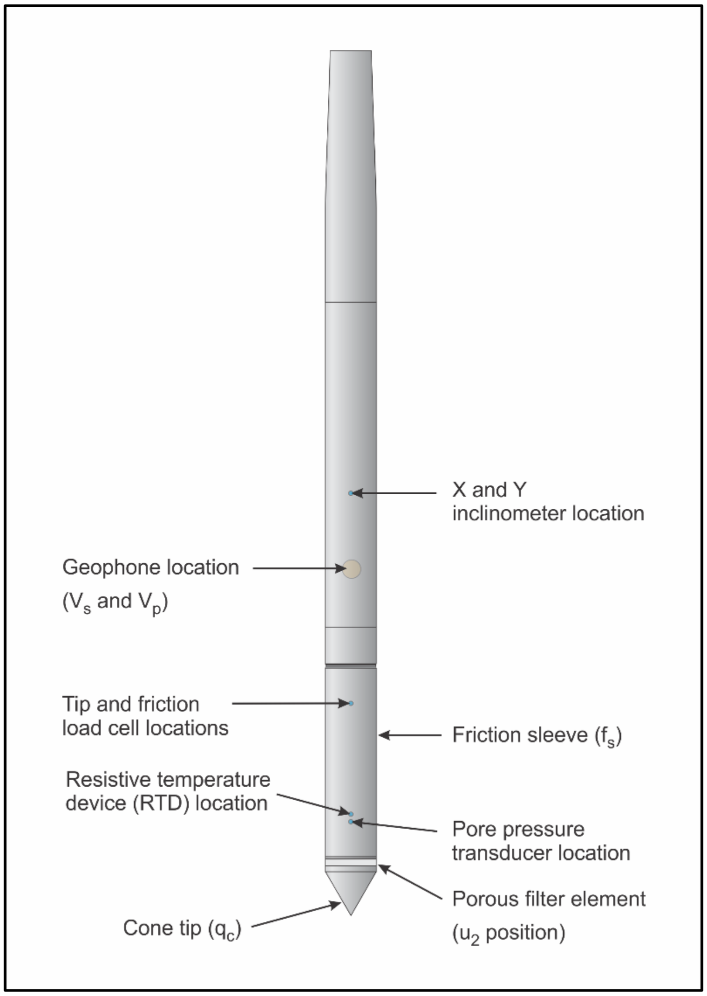


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches (38.1 millimeters) are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

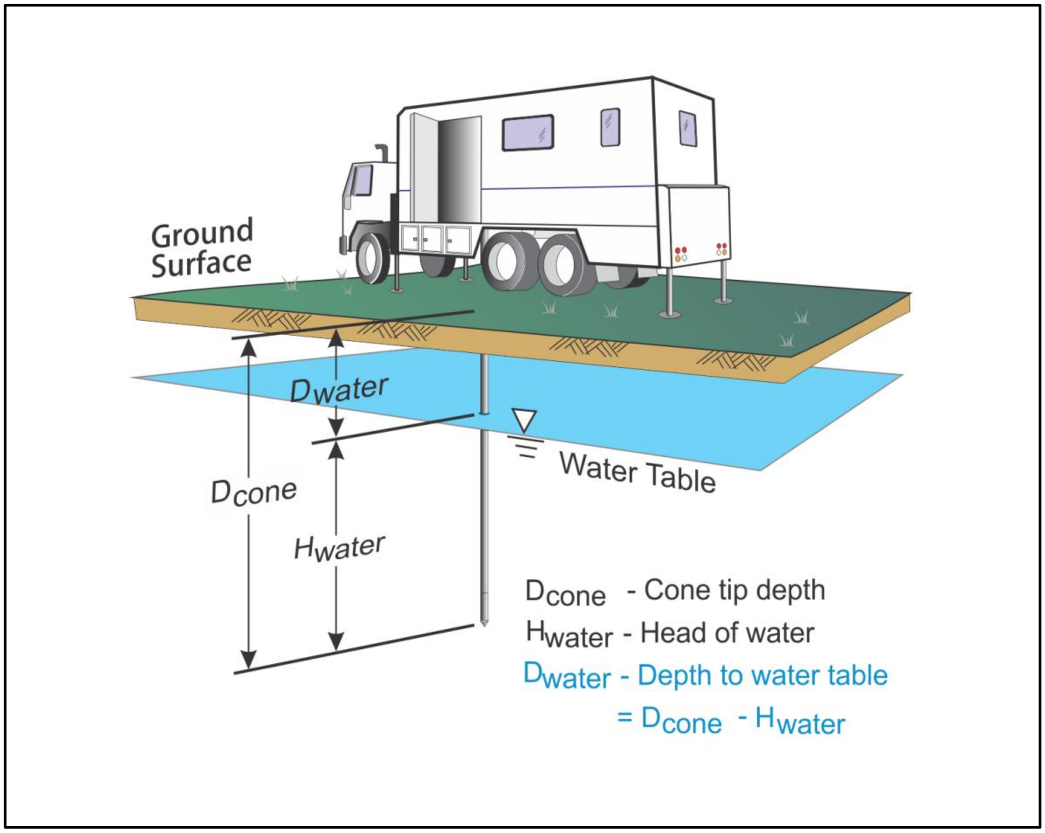


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

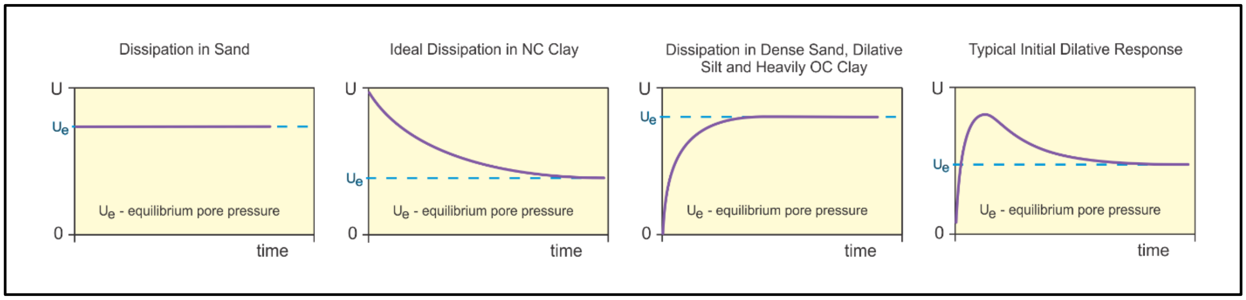


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:
T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-12.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073. DOI: 1063-1073/T98-062.

Burns, S.E. and Mayne, P.W., 2002, “Analytical cavity expansion-critical state model cone dissipation in fine-grained soils”, Soils & Foundations, Vol. 42(2): 131-137.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: 10.1061/9780784412770.027.

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158. DOI: 10.1139/T90-014.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 539-550. DOI: 10.1139/T92-061.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381. DOI: 10.1139/T98-105.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N(60)Ic/N1(60)Ic
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

CONETEC

Job No: 23-59-25320
Client: Columbia West Engineering, Inc.
Project: St Helens Road CPT
Start Date: 31-Jan-2023
End Date: 31-Jan-2023

CONE PENETRATION TEST SUMMARY

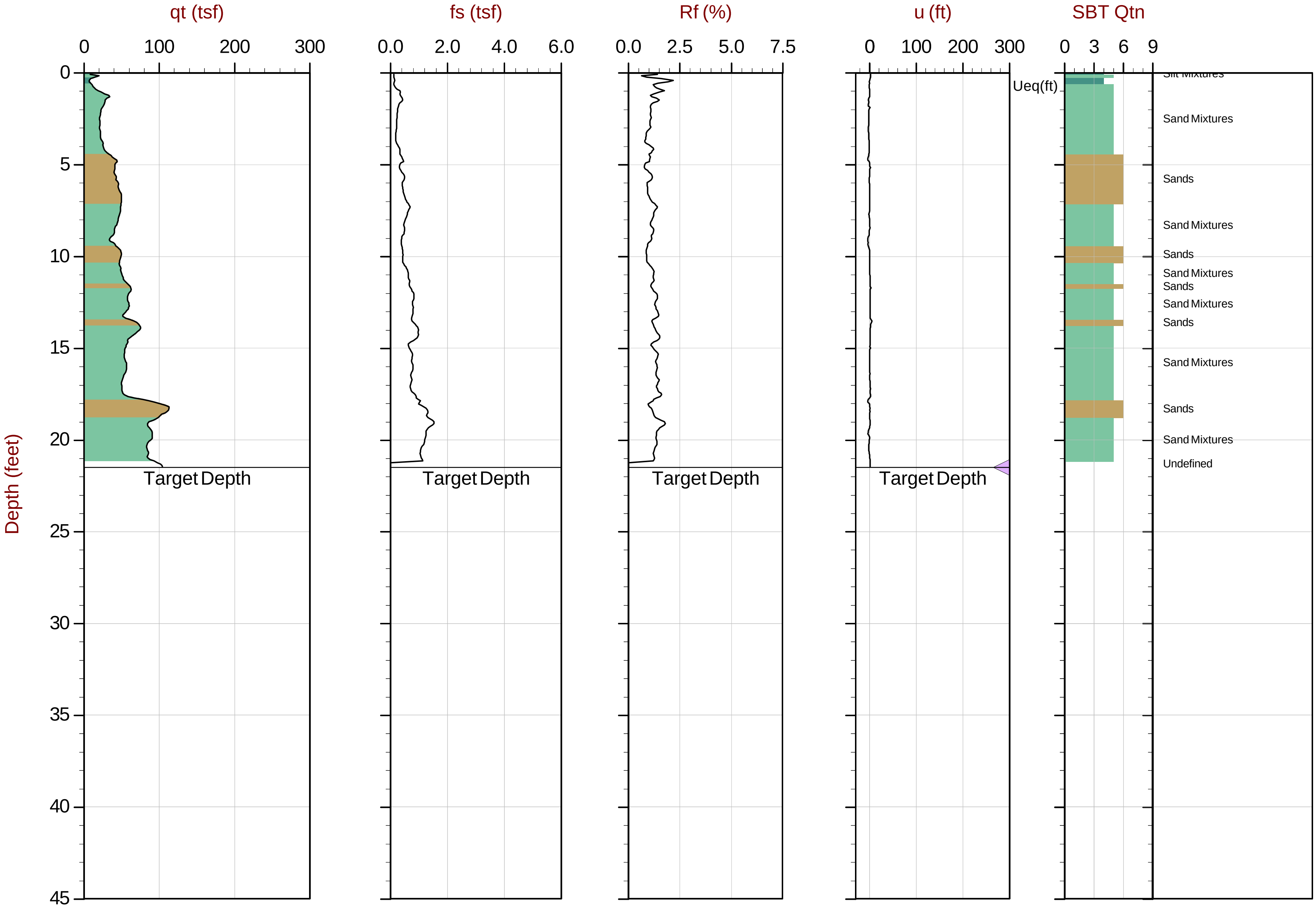
Sounding ID	File Name	Date	Cone	Assumed [1] Phreatic Surface (ft)	Final Depth (ft)	Latitude[2] (deg)	Longitude[2] (deg)	Refer to Notation Number
CPT-01	23-59-25320_CP01	31-Jan-2023	EC870:T1500F15U35		21.5	45.65014	-122.84998	3
CPT-02	23-59-25320_CP02	31-Jan-2023	EC870:T1500F15U35		22.0	45.64973	-122.85106	3
CPT-03	23-59-25320_CP03	31-Jan-2023	EC870:T1500F15U35	17.4	23.6	45.65069	-122.85166	
CPT-04	23-59-25320_CP04	31-Jan-2023	EC870:T1500F15U35	22.3	41.3	45.65010	-122.85229	
CPT-05	23-59-25320_CP05	31-Jan-2023	EC870:T1500F15U35		13.2	45.64981	-122.85177	3
CPT-05B	23-59-25320_CP05B	31-Jan-2023	EC870:T1500F15U35	19.7	22.8	45.64980	-122.85188	
Totals	6 soundings				144.4			

1. Phreatic surface based on pore pressure dissipation test unless otherwise noted. Hydrostatic profile applied to interpretation tables
2. Coordinates were collected using consumer grade GPS - WGS 84 Lat/Long
3. Phreatic surface is assumed to deeper than final sounding depth at this location

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 08:30
Site: St Helens Road CPT
Sounding: CPT-01
Cone: 870:T1500F15U35



Max Depth: 6.550 m / 21.49 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.65014 Long: -122.84998

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

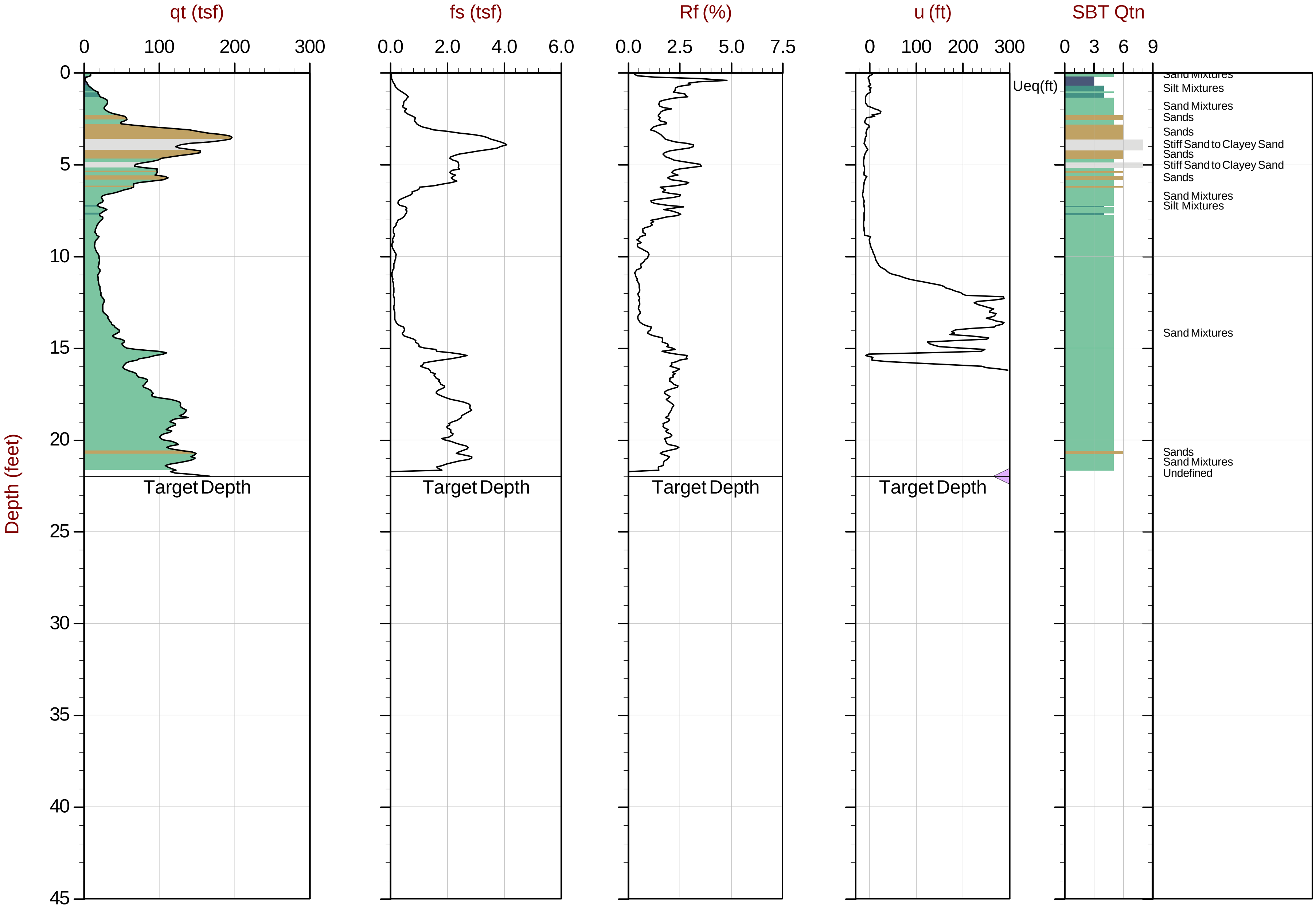
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 09:18
Site: St Helens Road CPT

Sounding: CPT-02
Cone: 870:T1500F15U35



Max Depth: 6.700 m / 21.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP02.COR
Unit Wt: SBTQtn (PKR2009)

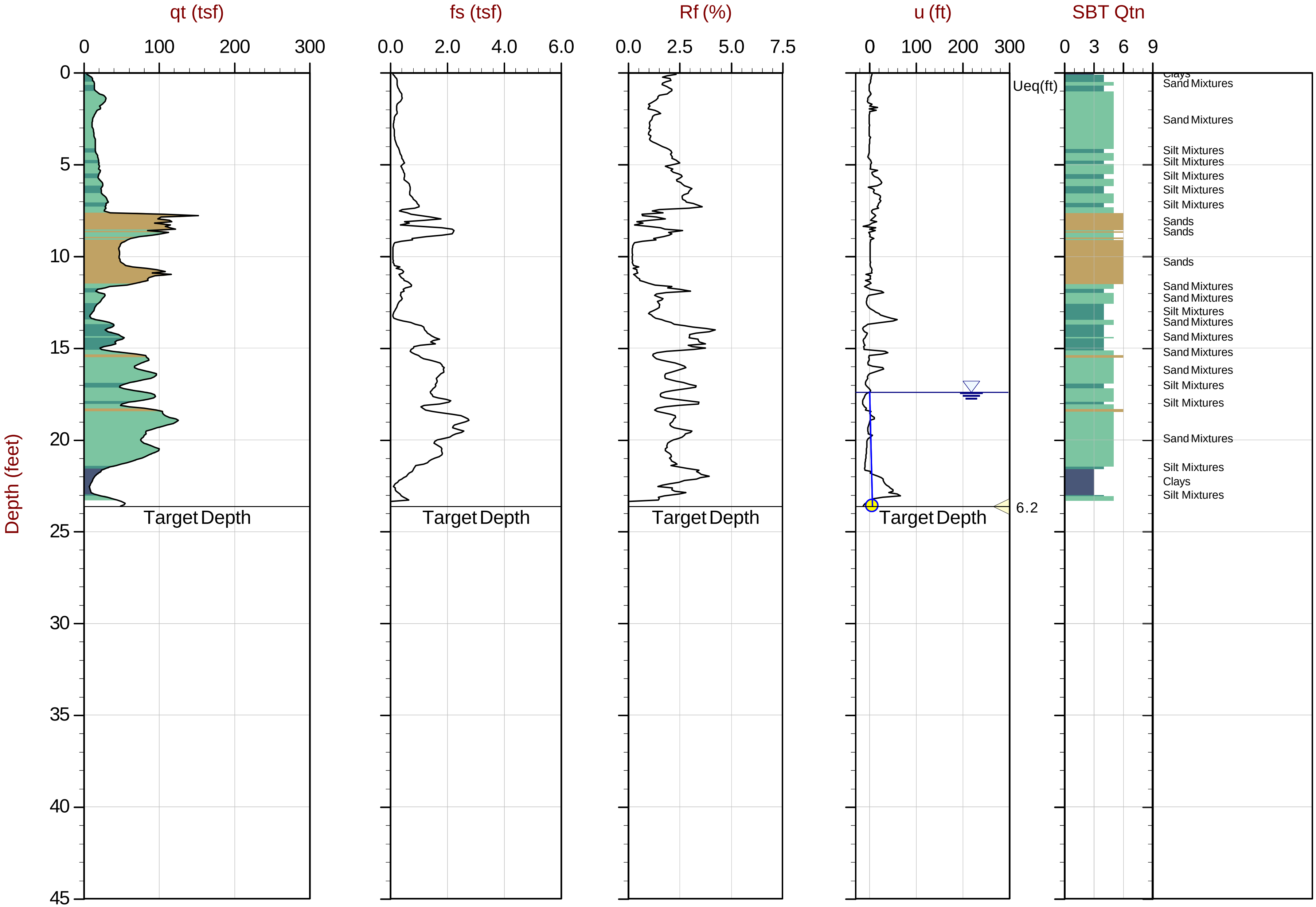
SBT: Robertson, 2009 and 2010
Coords: Lat: 45.64973 Long: -122.85106

● 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.

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 10:05
Site: St Helens Road CPT
Sounding: CPT-03
Cone: 870:T1500F15U35



Max Depth: 7.200 m / 23.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.65069 Long: -122.85166

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

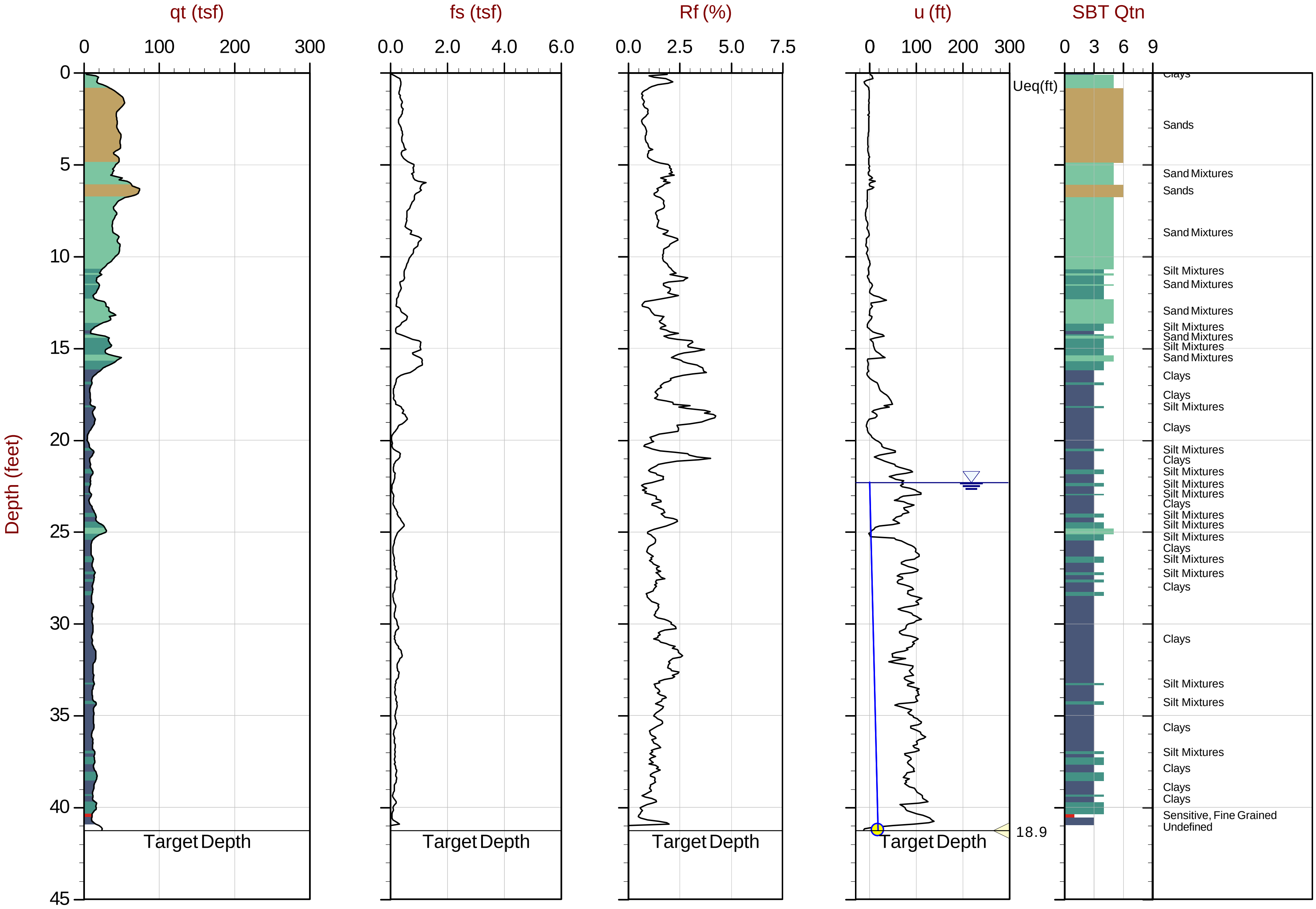
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 10:51
Site: St Helens Road CPT

Sounding: CPT-04
Cone: 870:T1500F15U35



Max Depth: 12.575 m / 41.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.65010 Long: -122.85229

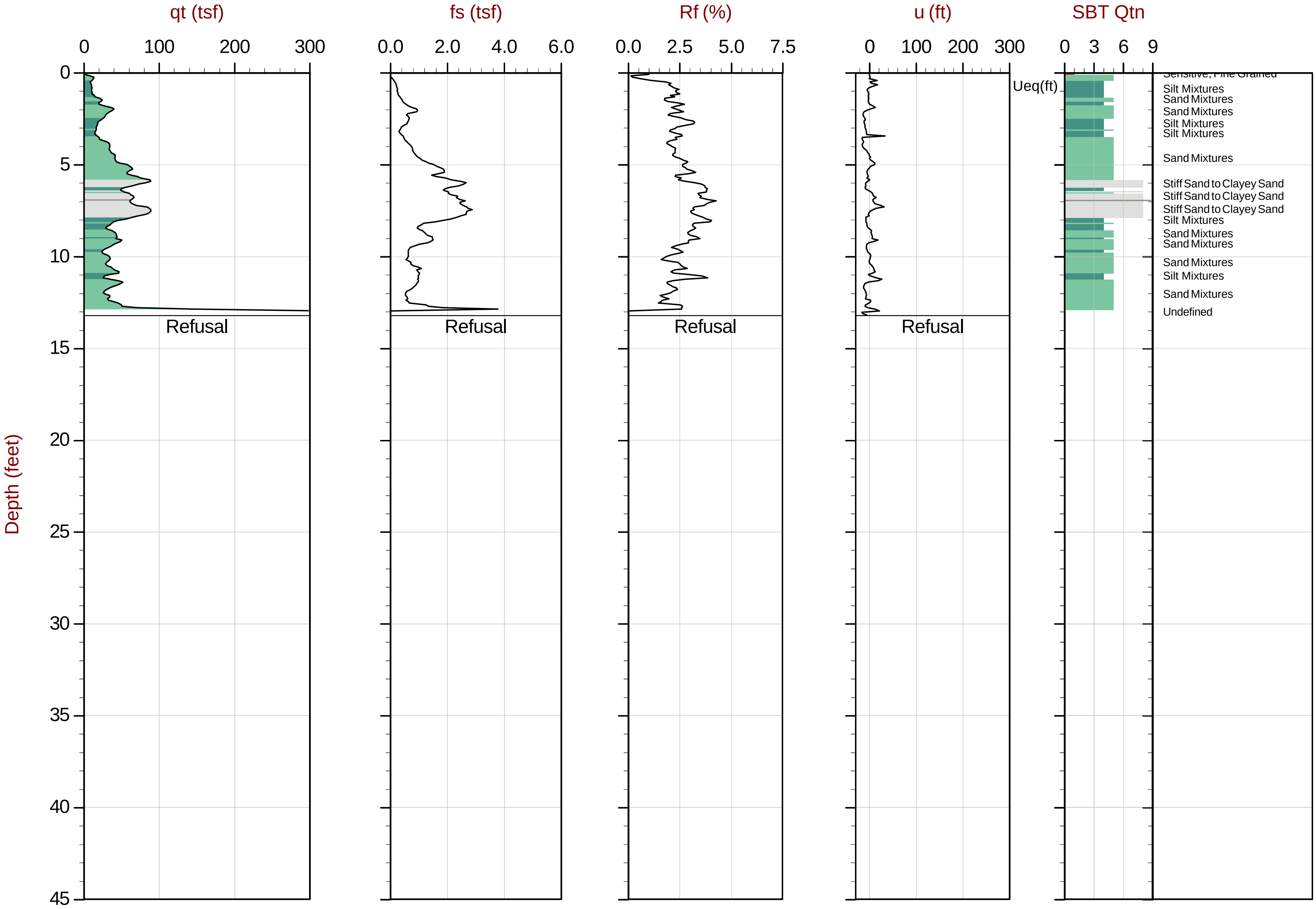
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

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

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 11:49
Site: St Helens Road CPT

Sounding: CPT-05
Cone: 870:T1500F15U35



Max Depth: 4.025 m / 13.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.64981 Long: -122.85177

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

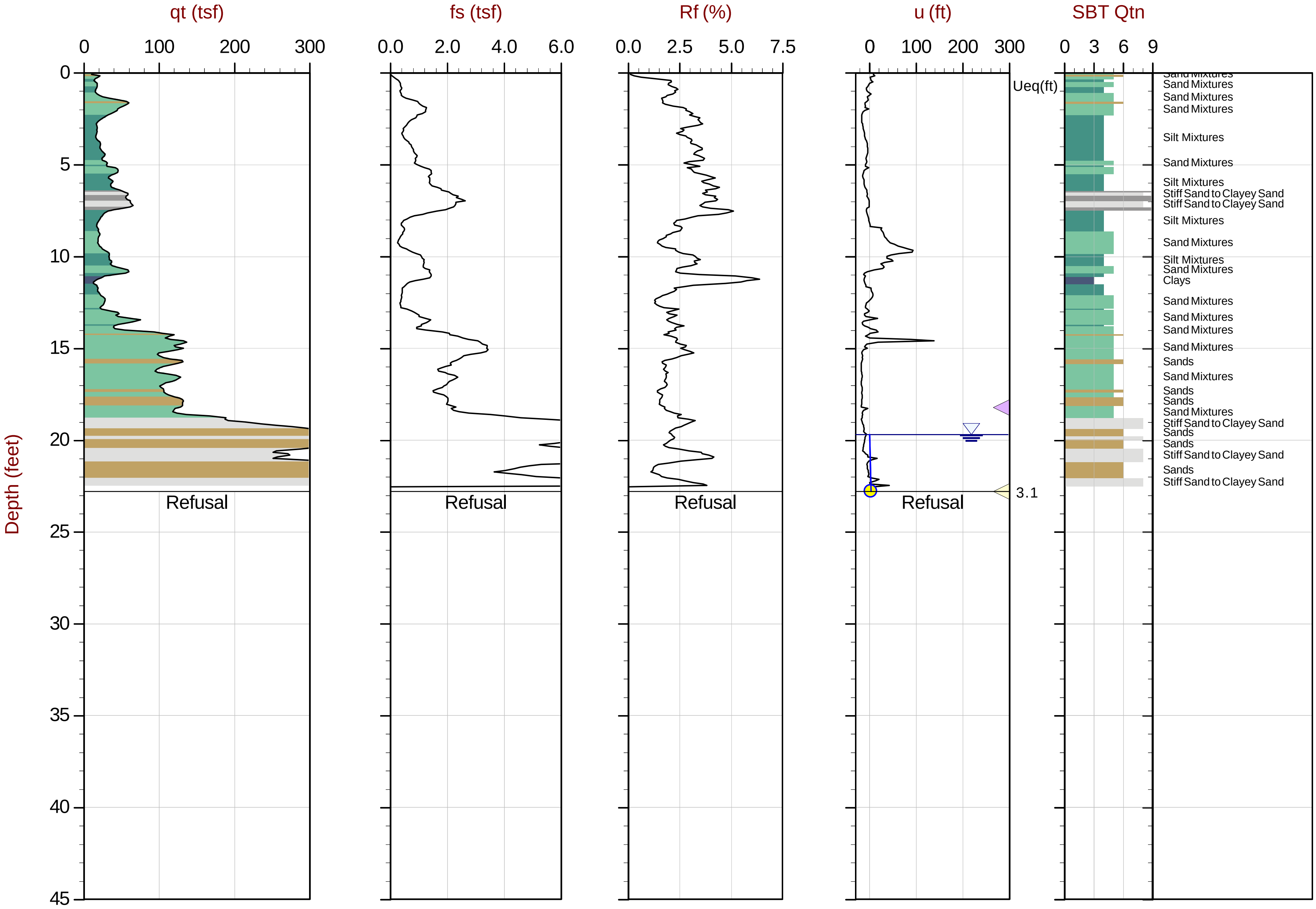
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 12:25
Site: St Helens Road CPT

Sounding: CPT-05B
Cone: 870:T1500F15U35



Max Depth: 6.950 m / 22.80 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP05B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.64980 Long: -122.85188

● 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.

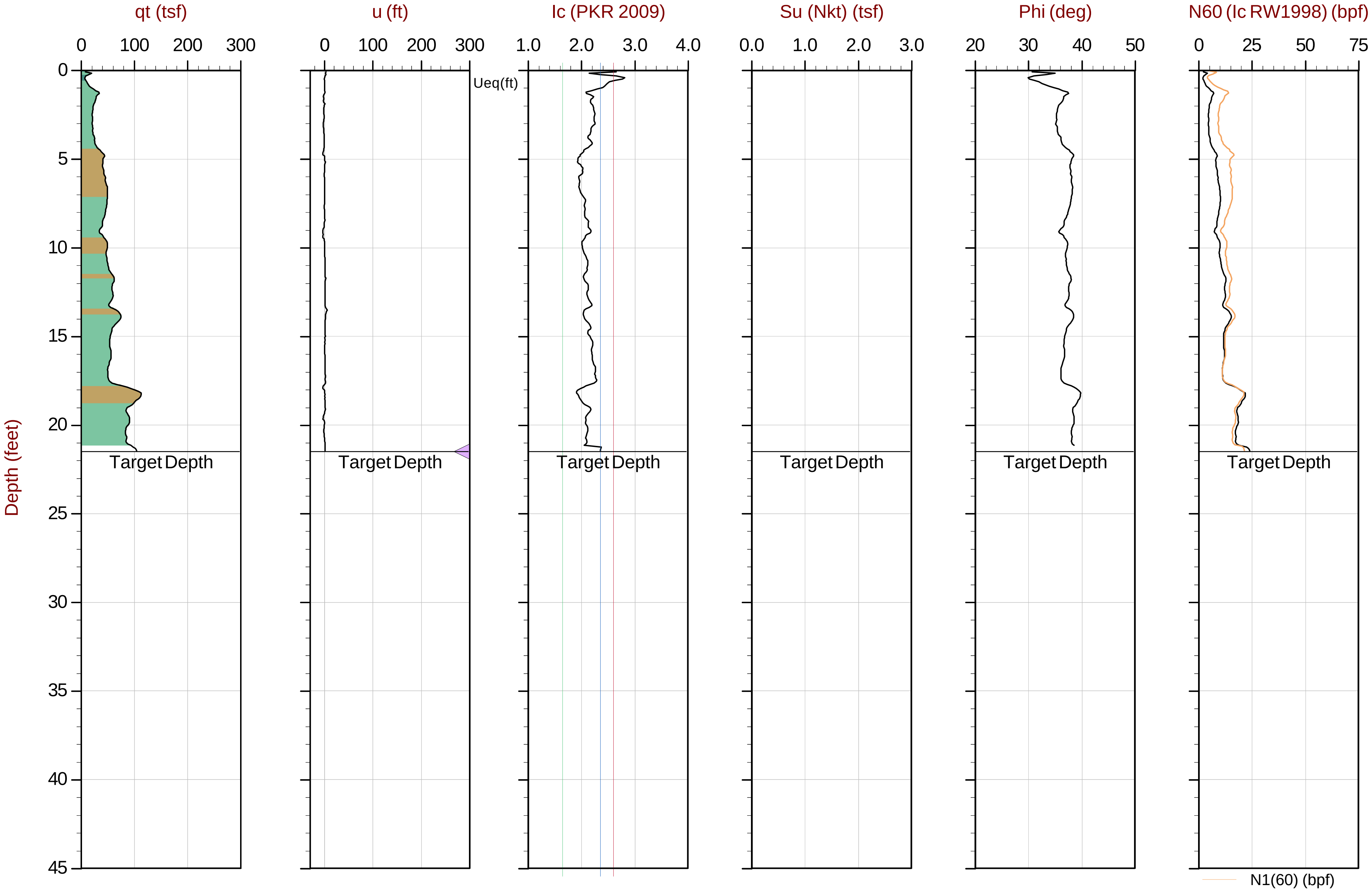
Advanced Cone Penetration Test Plots with Ic, Su, Phi and N(60)/N1(60)

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 08:30
Site: St Helens Road CPT

Sounding: CPT-01
Cone: 870:T1500F15U35



Max Depth: 6.550 m / 21.49 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.65014 Long: -122.84998

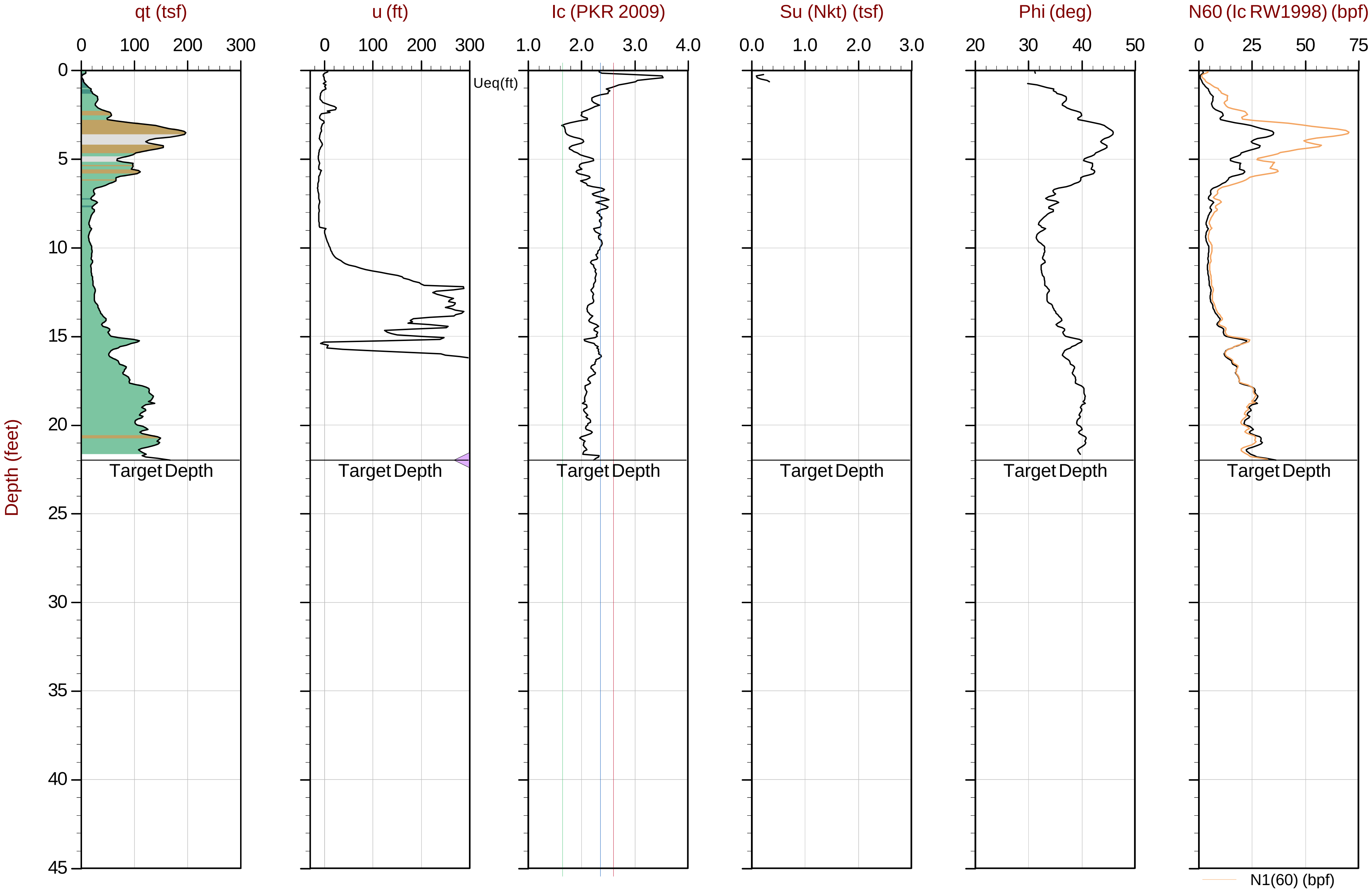
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

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

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 09:18
Site: St Helens Road CPT

Sounding: CPT-02
Cone: 870:T1500F15U35



Max Depth: 6.700 m / 21.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.64973 Long: -122.85106

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

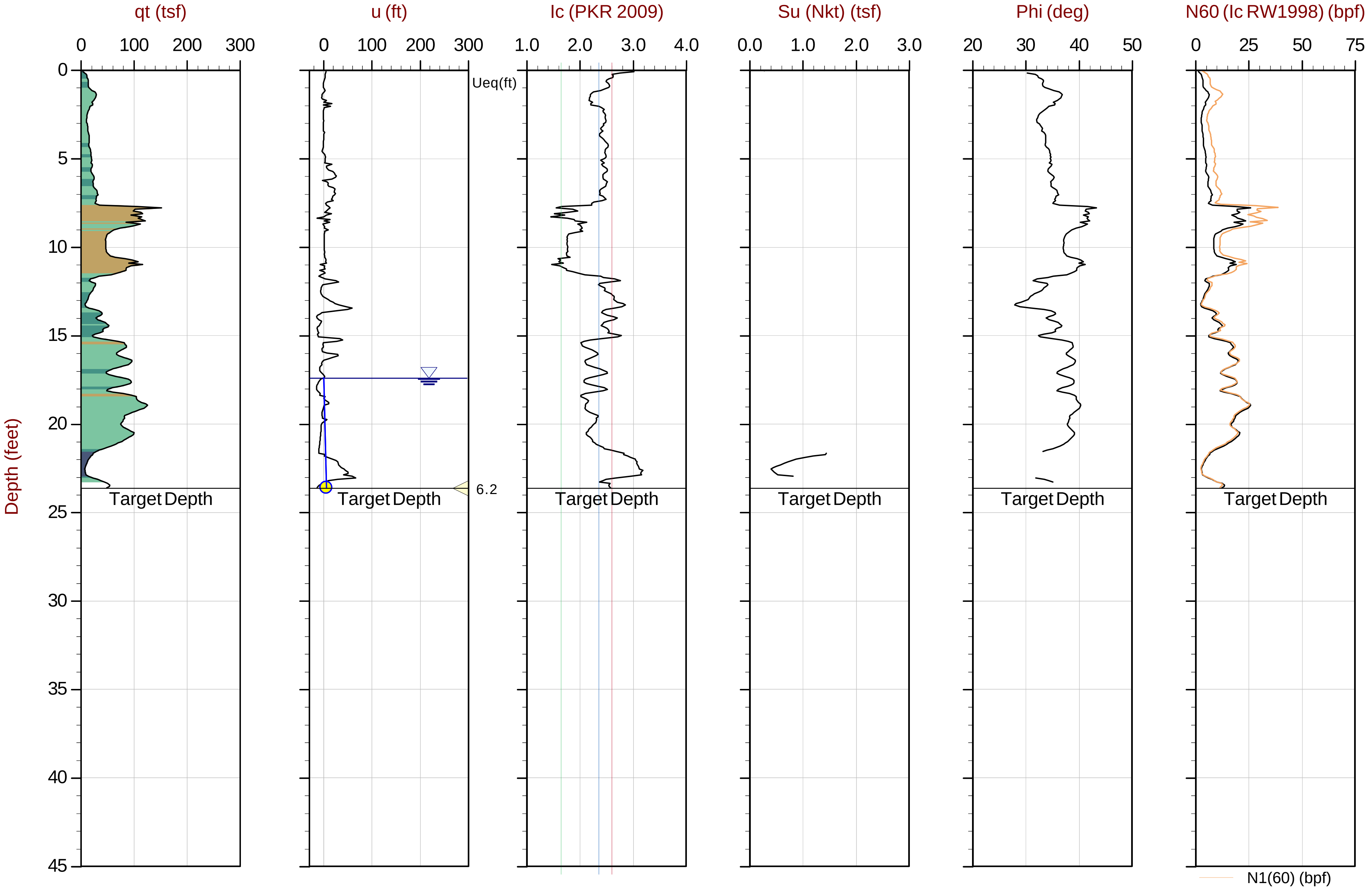
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 10:05
Site: St Helens Road CPT

Sounding: CPT-03
Cone: 870:T1500F15U35



Max Depth: 7.200 m / 23.62 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.65069 Long: -122.85166

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

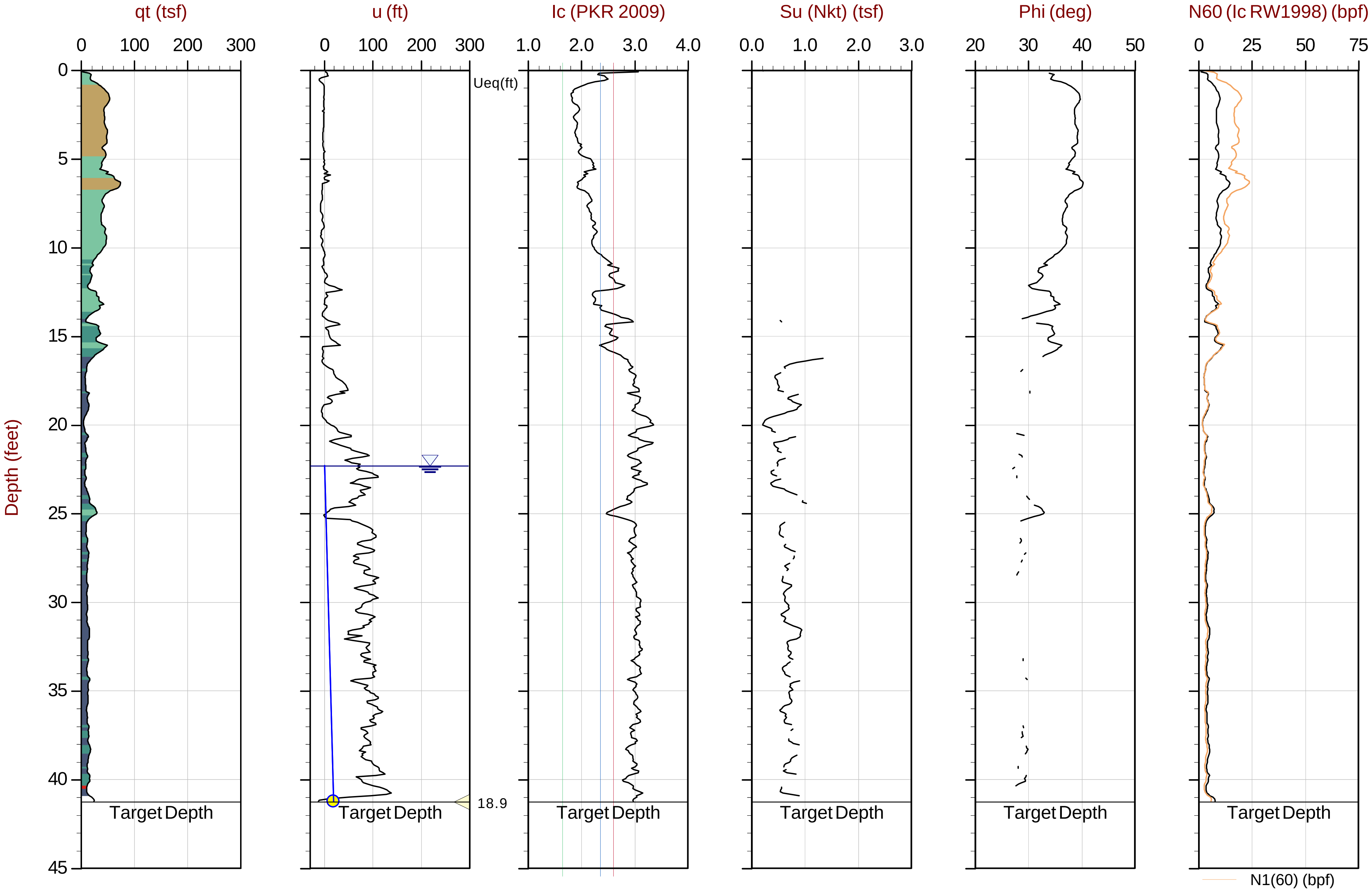
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 10:51
Site: St Helens Road CPT

Sounding: CPT-04
Cone: 870:T1500F15U35



Max Depth: 12.575 m / 41.26 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

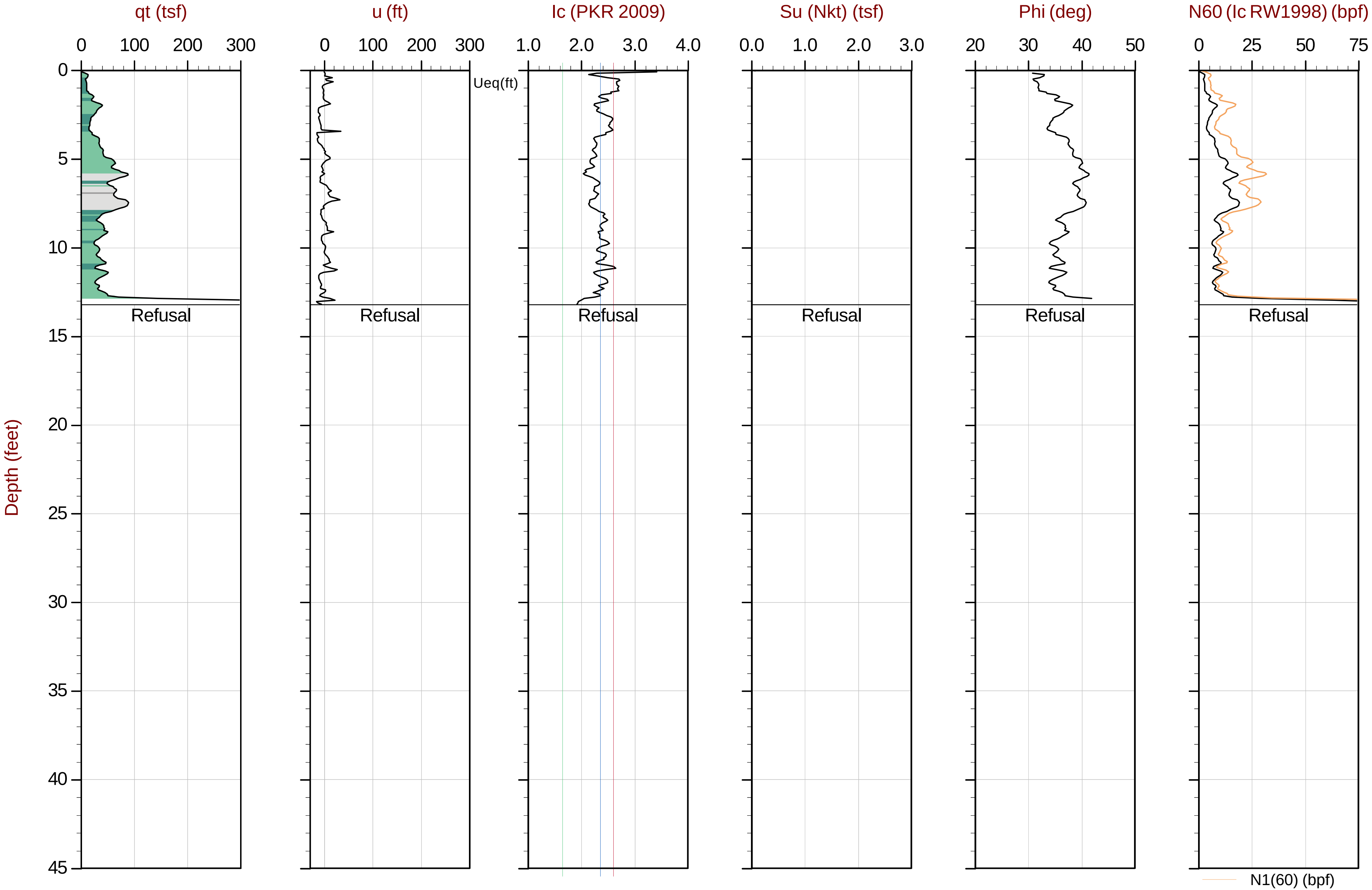
SBT: Robertson, 2009 and 2010
Coords: Lat: 45.65010 Long: -122.85229

● 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.

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 11:49
Site: St Helens Road CPT
Sounding: CPT-05
Cone: 870:T1500F15U35



Max Depth: 4.025 m / 13.21 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.64981 Long: -122.85177

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

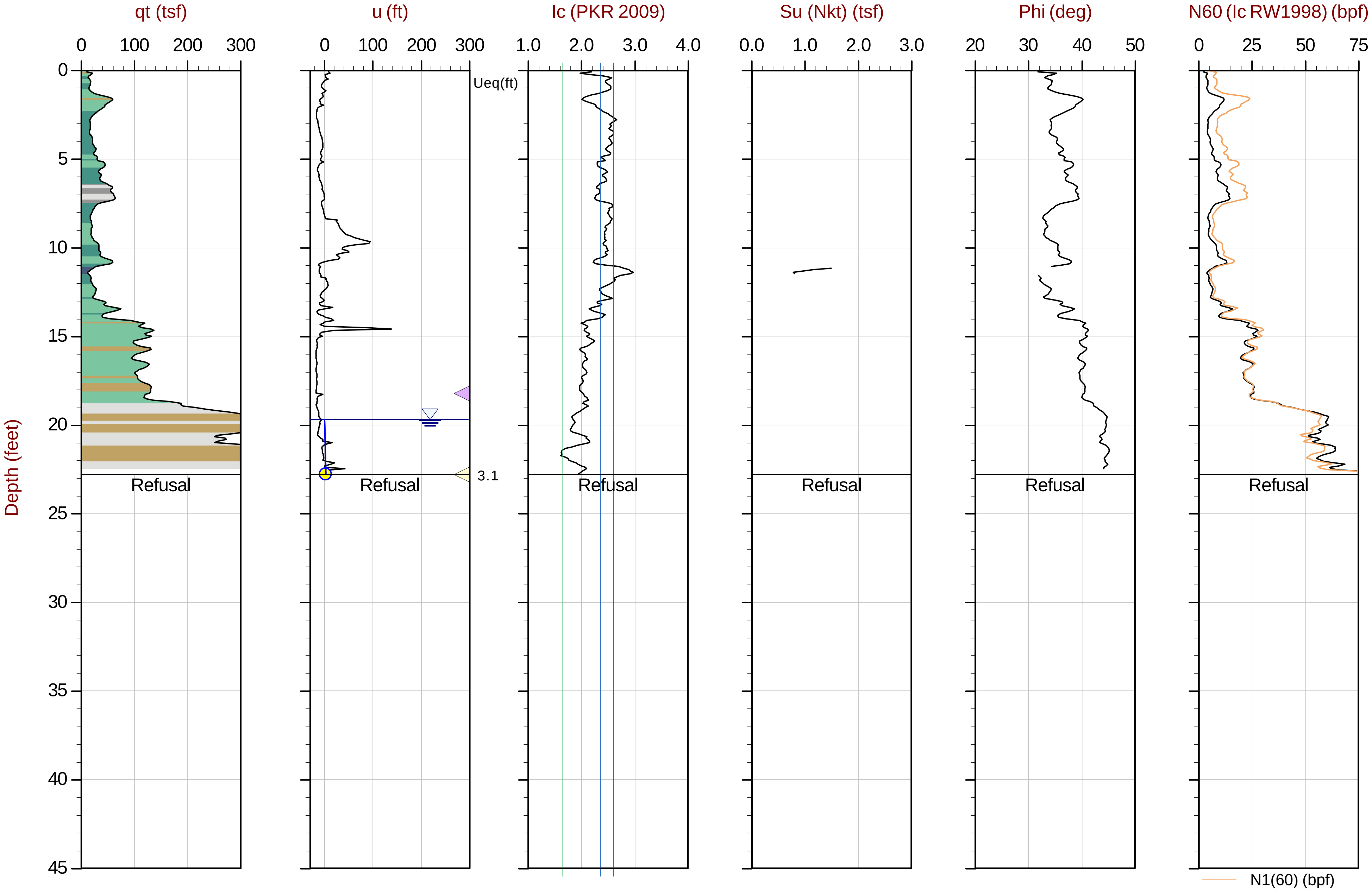
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Columbia West

Job No: 23-59-25320
Date: 2023-01-31 12:25
Site: St Helens Road CPT

Sounding: CPT-05B
Cone: 870:T1500F15U35



Max Depth: 6.950 m / 22.80 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-25320_CP05B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.64980 Long: -122.85188

● 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.

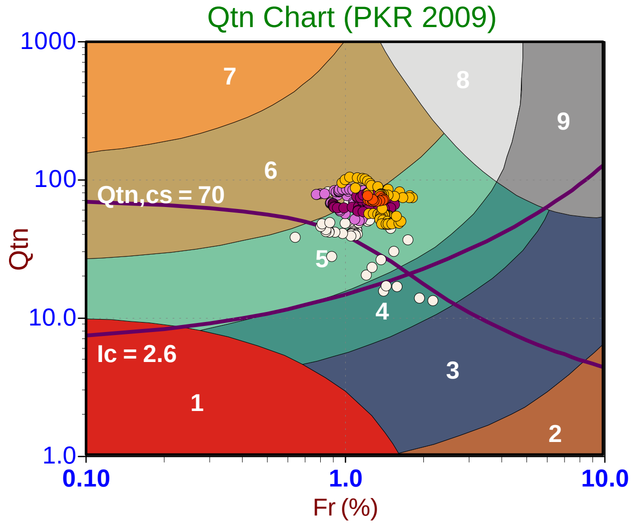
Soil Behavior Type (SBT) Scatter Plots

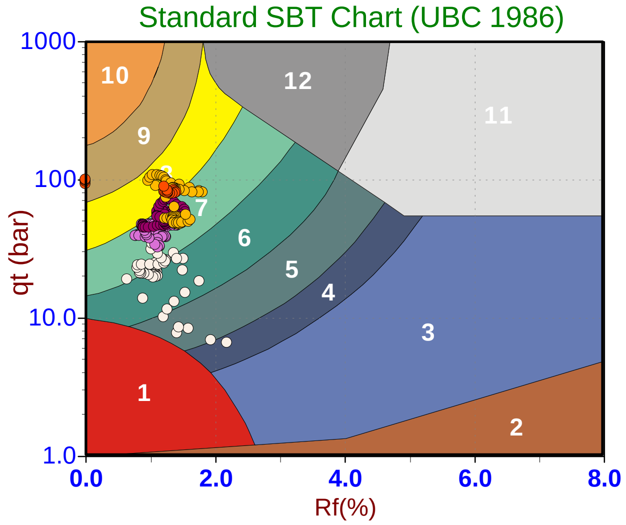
ConeTec

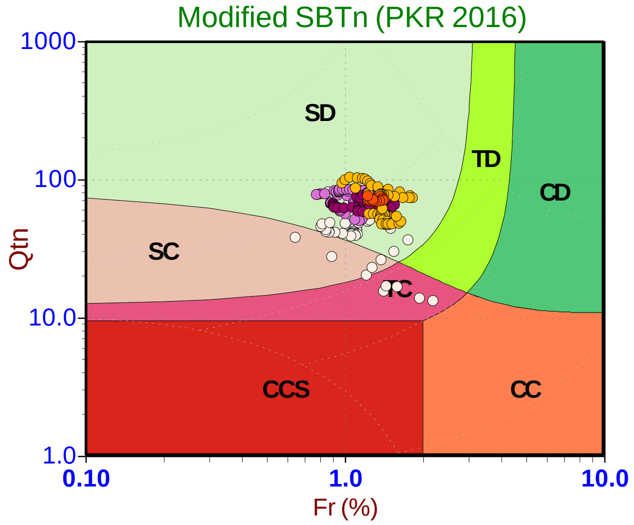
Columbia West

Job No: 23-59-25320
Date: 2023-01-31 08:30
Site: St Helens Road CPT

Sounding: CPT-01
Cone: 870:T1500F15U35


Qtn Chart (PKR 2009)


Standard SBT Chart (UBC 1986)


Modified SBTn (PKR 2016)

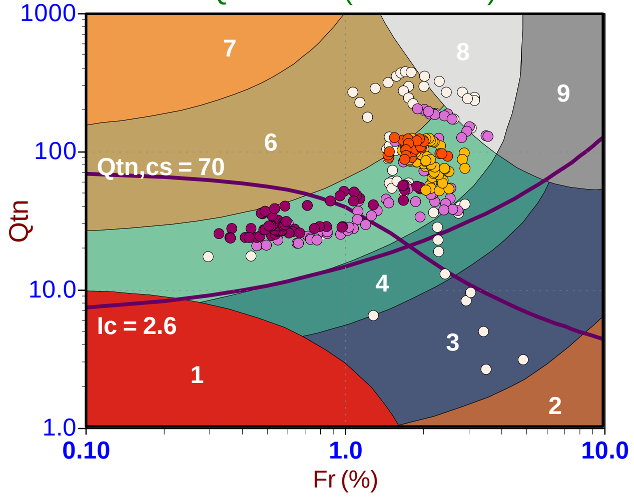
Depth Ranges
- ● >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend
- ■ Sensitive, Fine Grained
- ■ Organic Soils
- ■ Clays
- ■ Silt Mixtures
- ■ Sand Mixtures
- ■ Sands
- ■ Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
- ■ Very Stiff Fine Grained

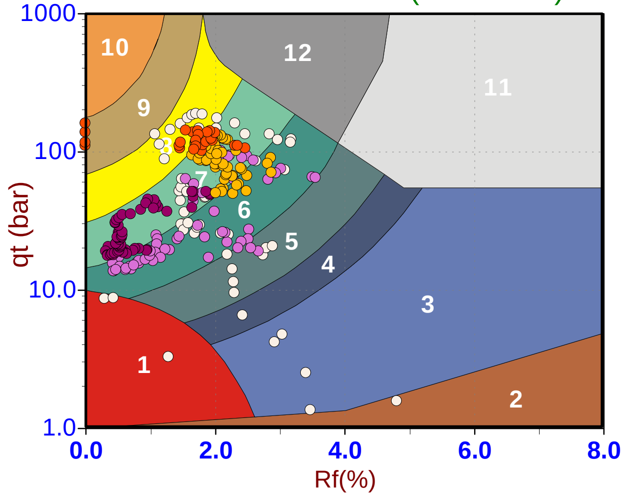
Legend
- ■ Sensitive Fines
- ■ Organic Soil
- ■ Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- ■ Silty Sand/Sand
- ■ Sand
- ■ Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

Legend
- ■ CCS (Cont. sensitive clay like)
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

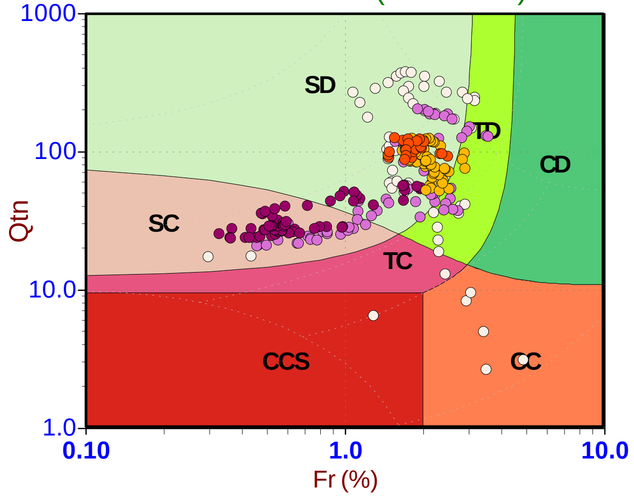
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- ● >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend

- ■ Sensitive, Fine Grained
- ■ Organic Soils
- ■ Clays
- ■ Silt Mixtures
- ■ Sand Mixtures
- ■ Sands
- ■ Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
- ■ Very Stiff Fine Grained

Legend

- ■ Sensitive Fines
- ■ Organic Soil
- ■ Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- ■ Silty Sand/Sand
- ■ Sand
- ■ Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

Legend

- ■ CCS (Cont. sensitive clay like)
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

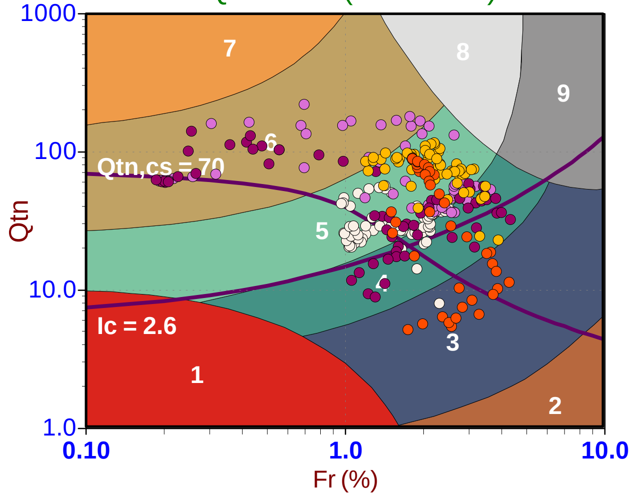
CONETEC

Columbia West

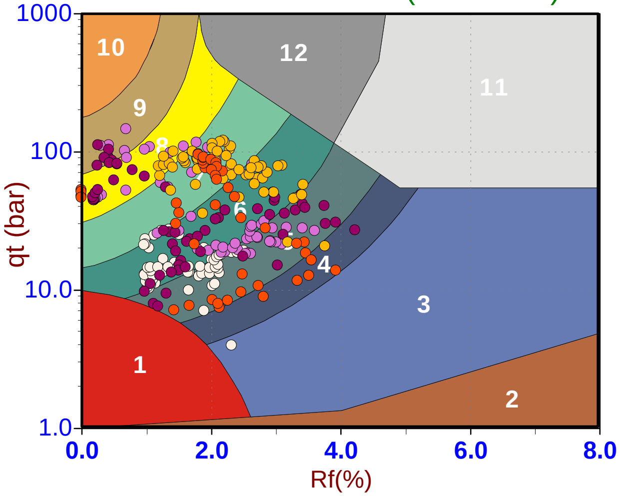
Job No: 23-59-25320
Date: 2023-01-31 10:05
Site: St Helens Road CPT

Sounding: CPT-03
Cone: 870:T1500F15U35

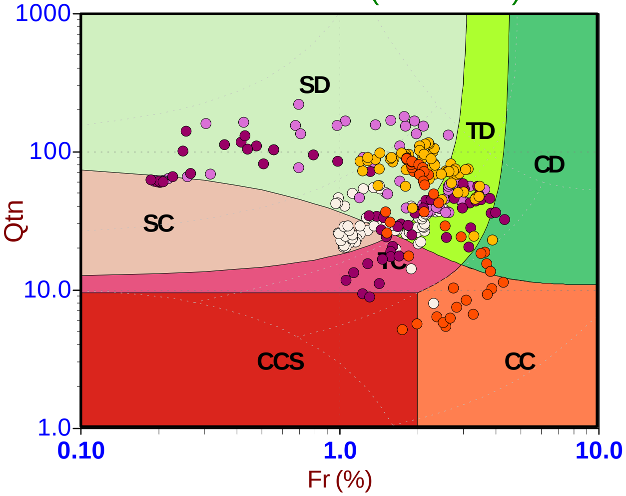
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges
- ● >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend
- 🟥 Sensitive, Fine Grained
- 🟫 Organic Soils
- ■ Clays
- ■ Silt Mixtures
- 🟩 Sand Mixtures
- ■ Sands
- 🟧 Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
- ■ Very Stiff Fine Grained

Legend
- 🟥 Sensitive Fines
- 🟫 Organic Soil
- 🟦 Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- 🟩 Sandy Silt
- 🟨 Silty Sand/Sand
- ■ Sand
- 🟧 Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

Legend
- 🟥 CCS (Cont. sensitive clay like)
- 🟧 CC (Cont. clay like)
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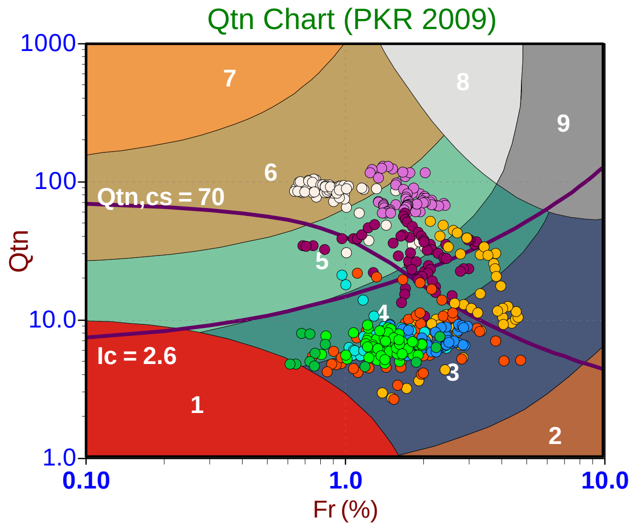
- ■ SC (Cont. sand like)
- 🟩 CD (Dil. clay like)
- 🟩 TD (Dil. transitional)
- ■ SD (Dil. sand like)

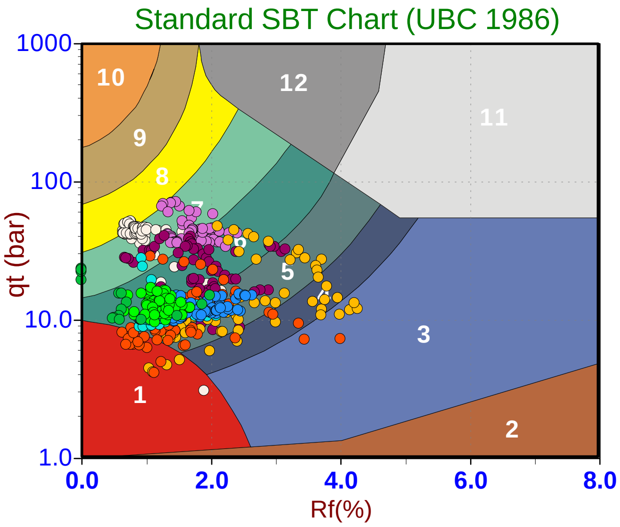
ConeTec

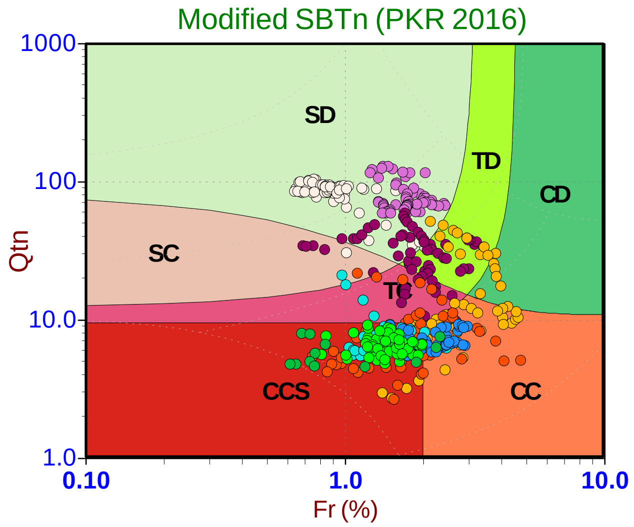
Columbia West

Job No: 23-59-25320
Date: 2023-01-31 10:51
Site: St Helens Road CPT

Sounding: CPT-04
Cone: 870:T1500F15U35


Qtn Chart (PKR 2009)


Standard SBT Chart (UBC 1986)


Modified SBTn (PKR 2016)

Depth Ranges
- ● >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend
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Legend
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Legend
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- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

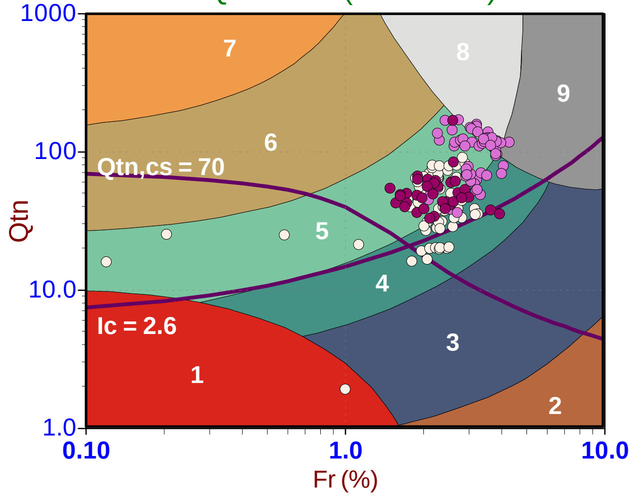
CONETEC

Columbia West

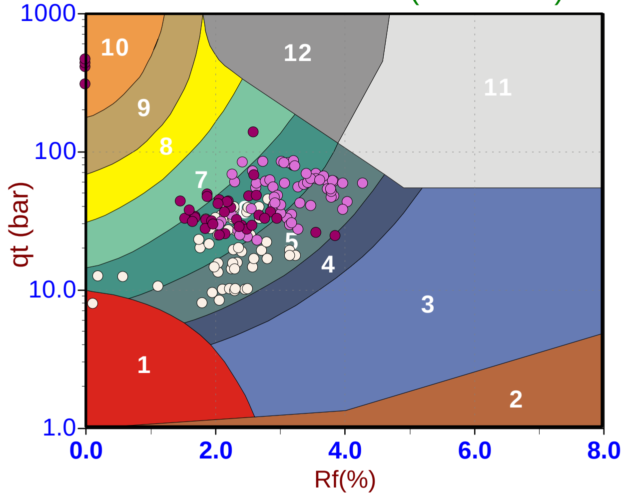
Job No: 23-59-25320
Date: 2023-01-31 11:49
Site: St Helens Road CPT

Sounding: CPT-05
Cone: 870:T1500F15U35

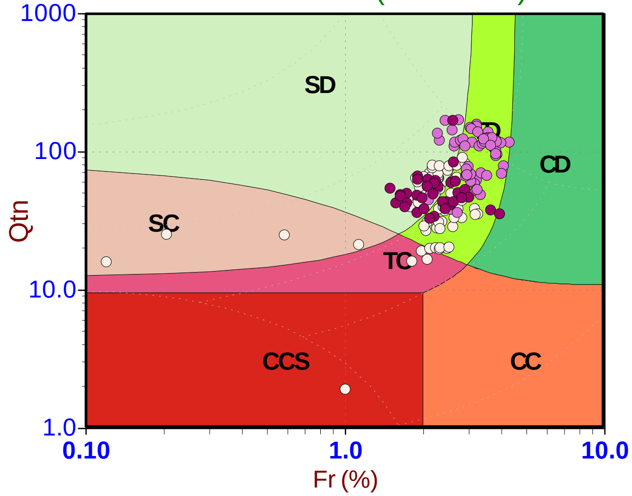
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges
- ● >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend
- ■ Sensitive, Fine Grained
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- ■ Sand Mixtures
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- ■ Stiff Sand to Clayey Sand
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Legend
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- ■ Sand
- ■ Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

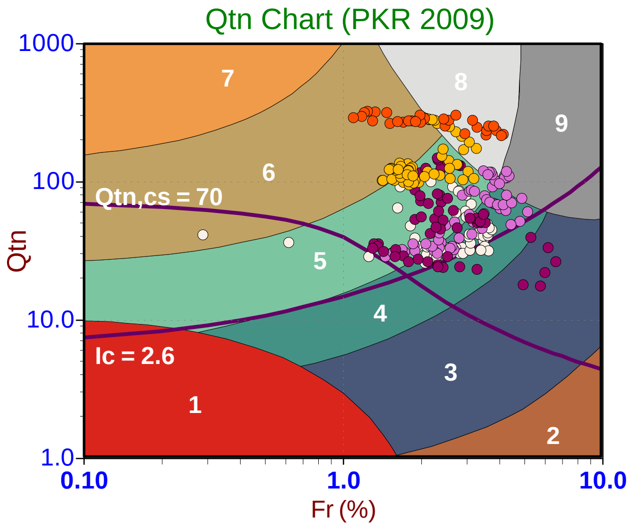
Legend
- ■ CCS (Cont. sensitive clay like)
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

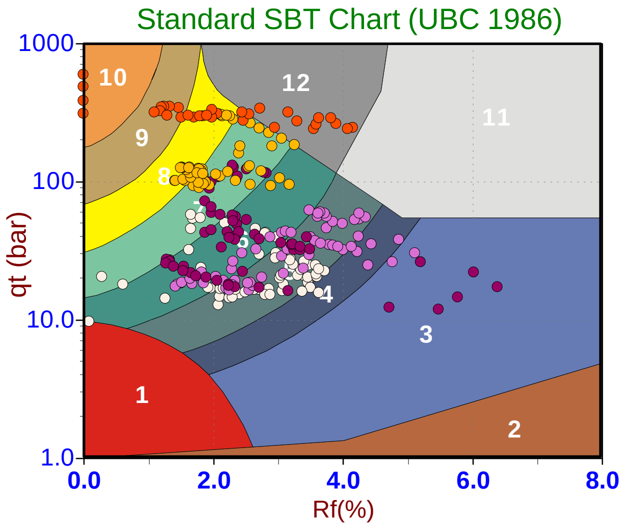
CONETEC

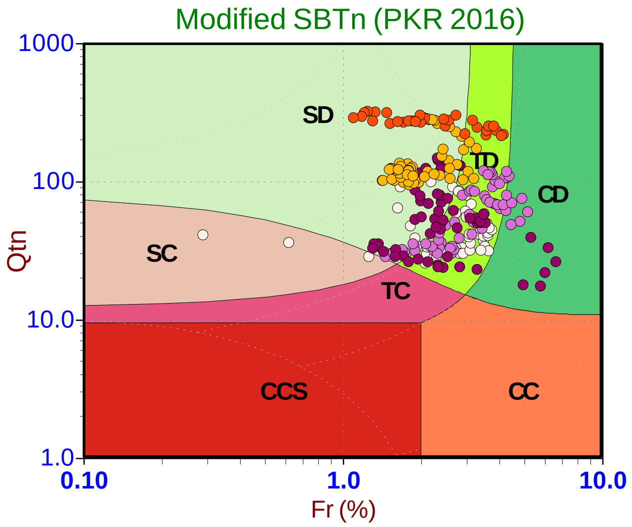
Columbia West

Job No: 23-59-25320
Date: 2023-01-31 12:25
Site: St Helens Road CPT

Sounding: CPT-05B
Cone: 870:T1500F15U35







Depth Ranges
- ○ >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend
- ■ Sensitive, Fine Grained
- ■ Organic Soils
- ■ Clays
- ■ Silt Mixtures
- ■ Sand Mixtures
- ■ Sands
- ■ Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
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Legend
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- ■ Sand
- ■ Gravelly Sand
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- ■ Cemented Sand

Legend
- ■ CCS (Cont. sensitive clay like)
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

CONETEC

Job No: 23-59-25320
Client: Columbia West Engineering, Inc.
Project: St Helens Road CPT
Start Date: 31-Jan-2023
End Date: 31-Jan-2023

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft)	Calculated Phreatic Surface (ft)	Notes
CPT-01	23-59-25320_CP01	15	300	21.5			1
CPT-02	23-59-25320_CP02	15	440	22.0			1,2
CPT-03	23-59-25320_CP03	15	305	23.6	6.2	17.4	
CPT-04	23-59-25320_CP04	15	910	41.3	18.9	22.3	
CPT-05B	23-59-25320_CP05B	15	355	18.2			1
CPT-05B	23-59-25320_CP05B	15	1140	22.8	3.1	19.7	
Total Duration			57 min				

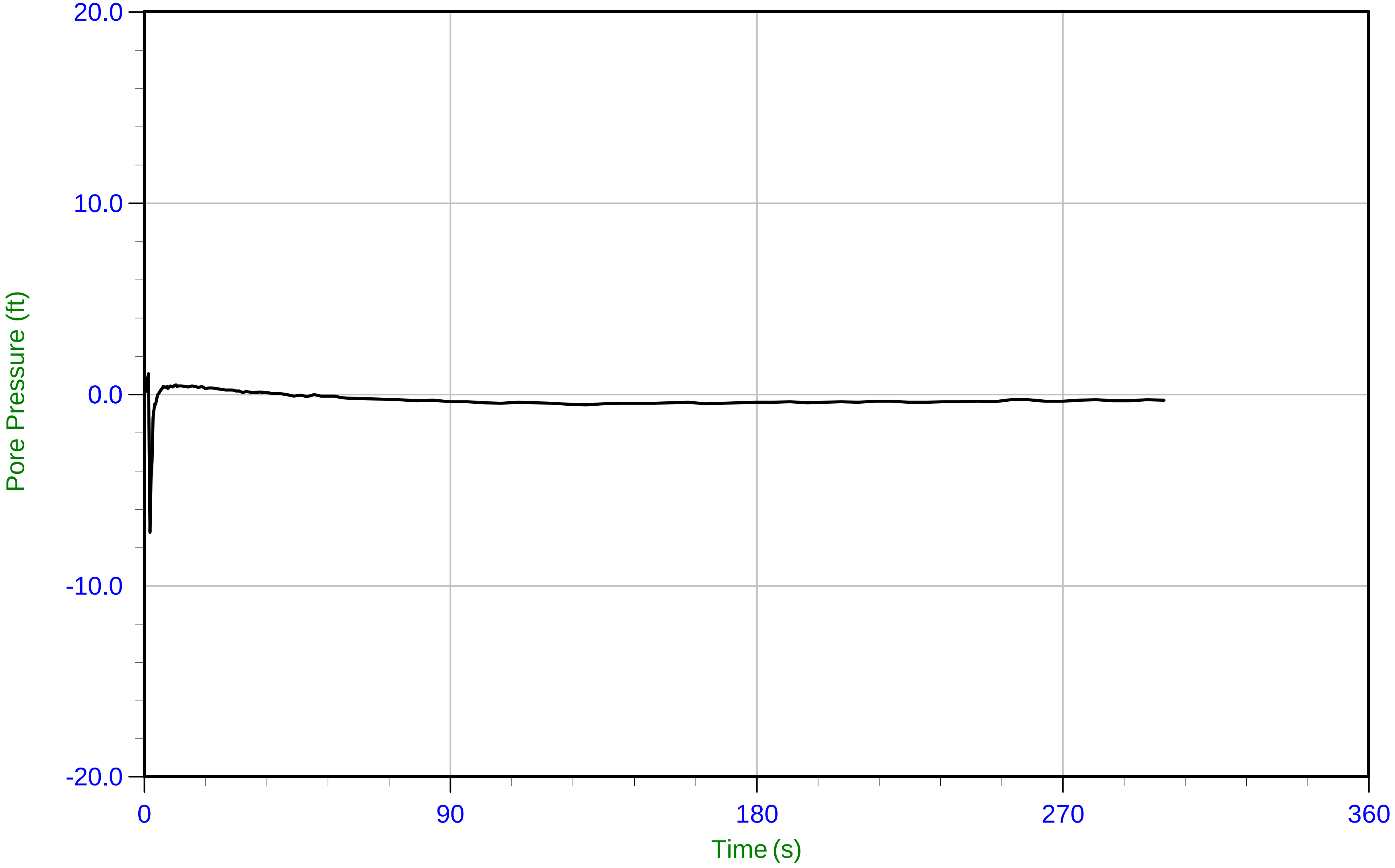
1. Dissipation assumed to have been performed shallower than the phreatic surface
2. Dissipation ended before Ueq was achieved

CONETEC

Columbia West

Job No: 23-59-25320
Date: 01/31/2023 08:30
Site: St Helens Road CPT

Sounding: CPT-01
Cone: 870:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-25320_CP01.PPR2
Depth: 6.550 m / 21.489 ft
Duration: 300.0 s

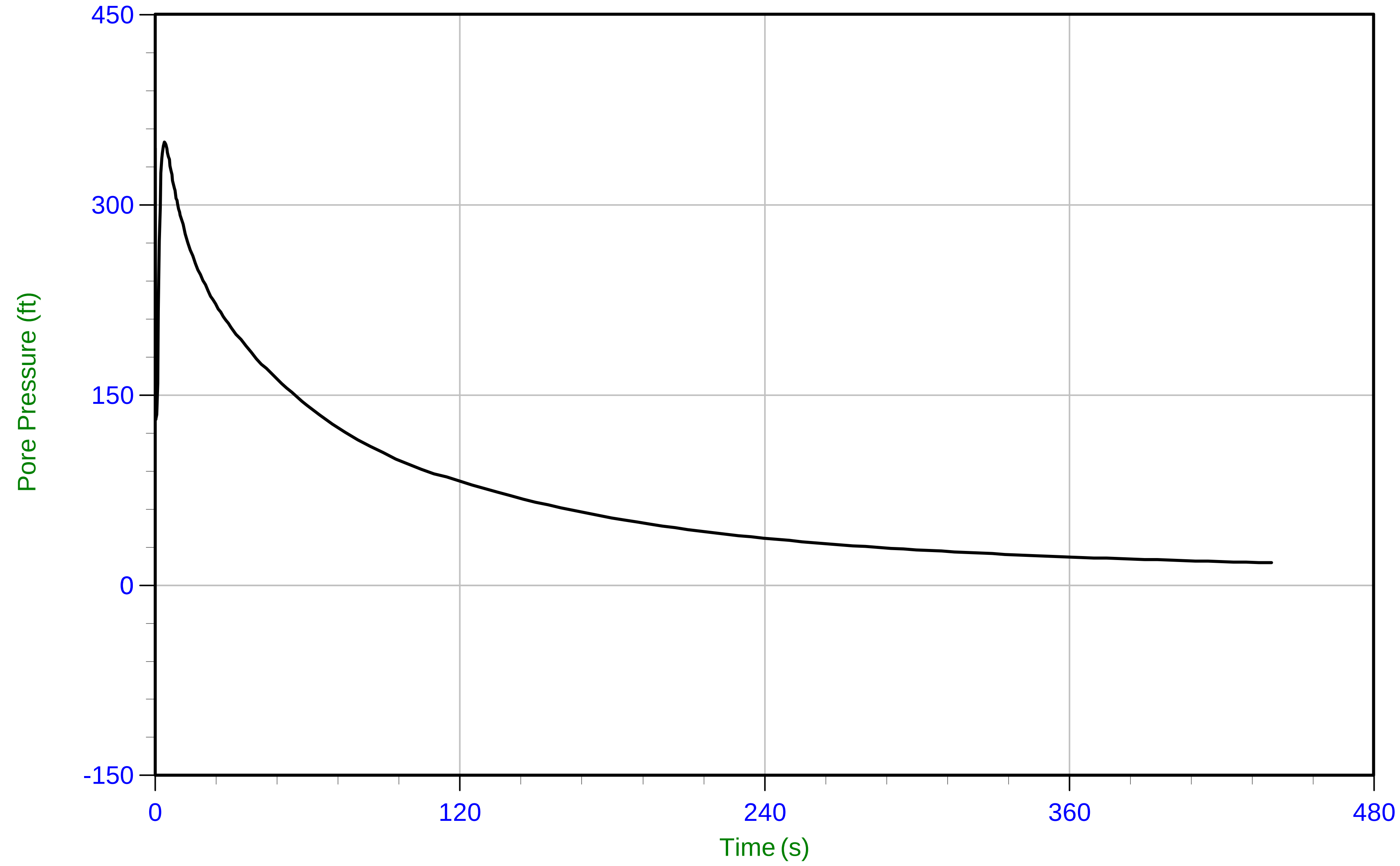
u Min: -7.2 ft
u Max: 1.4 ft
u Final: -0.3 ft

CONETEC

Columbia West

Job No: 23-59-25320
Date: 01/31/2023 09:18
Site: St Helens Road CPT

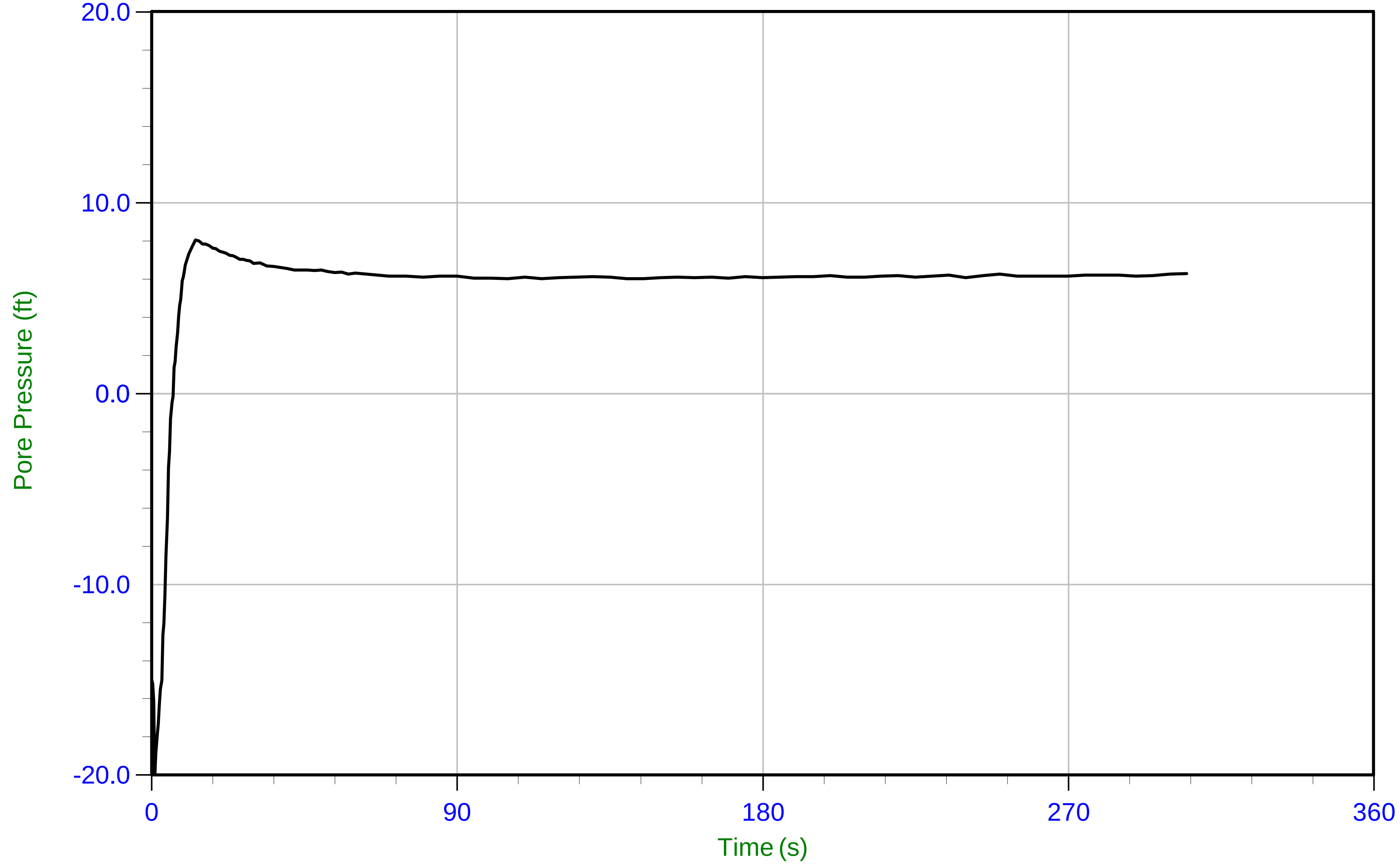
Sounding: CPT-02
Cone: 870:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-25320_CP02.PPR2
Depth: 6.700 m / 21.981 ft
Duration: 440.0 s

u Min: 17.6 ft
u Max: 349.2 ft
u Final: 17.6 ft

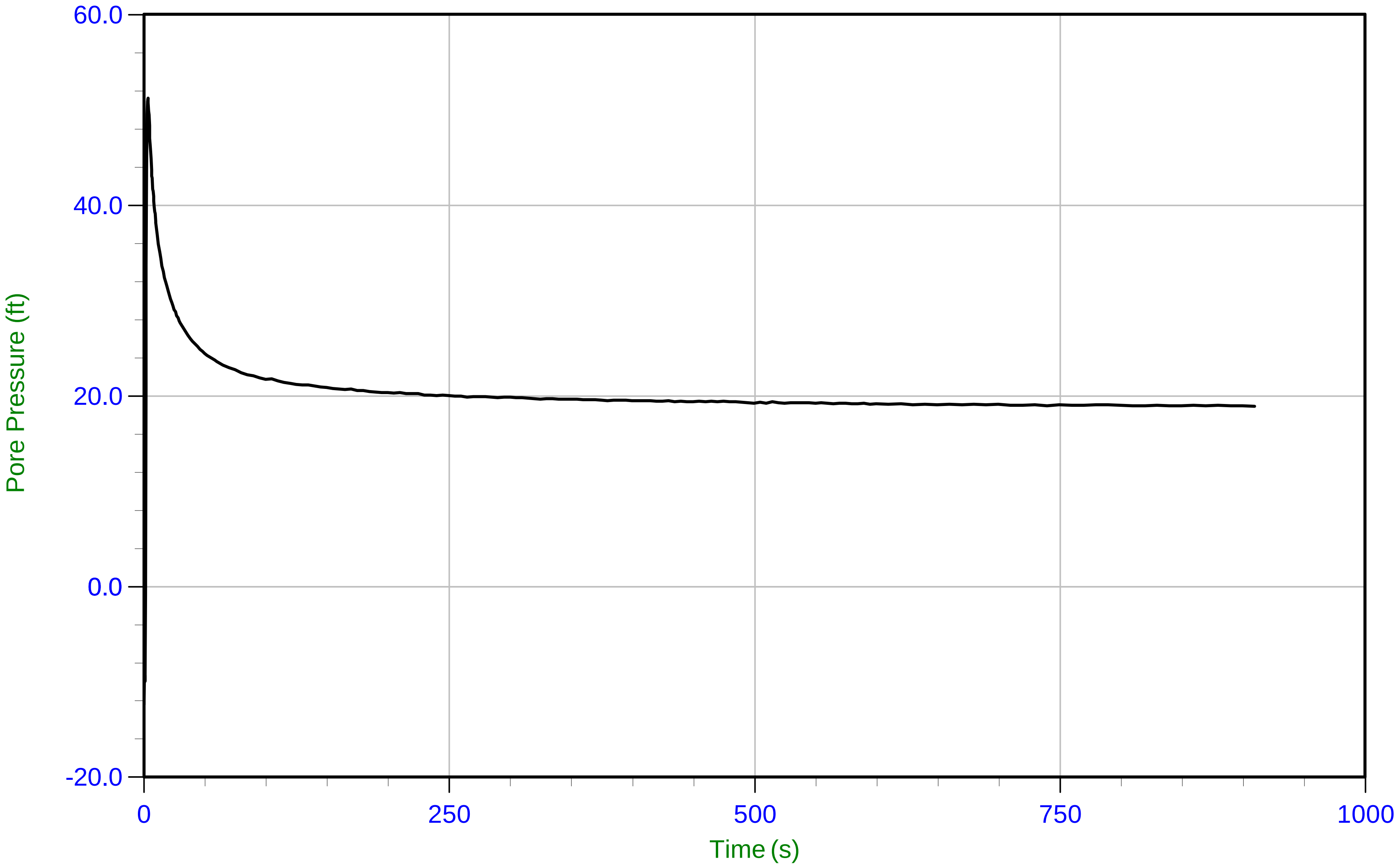


Trace Summary:

Filename: 23-59-25320_CP03.PPR2
Depth: 7.200 m / 23.622 ft
Duration: 305.0 s

u Min: -20.2 ft
u Max: 8.0 ft
u Final: 6.3 ft

WT: 5.306 m / 17.408 ft
Ueq: 6.2 ft



Trace Summary:

Filename: 23-59-25320_CP04.PPR2
Depth: 12.575 m / 41.256 ft
Duration: 910.0 s

u Min: -12.4 ft
u Max: 51.2 ft
u Final: 18.9 ft

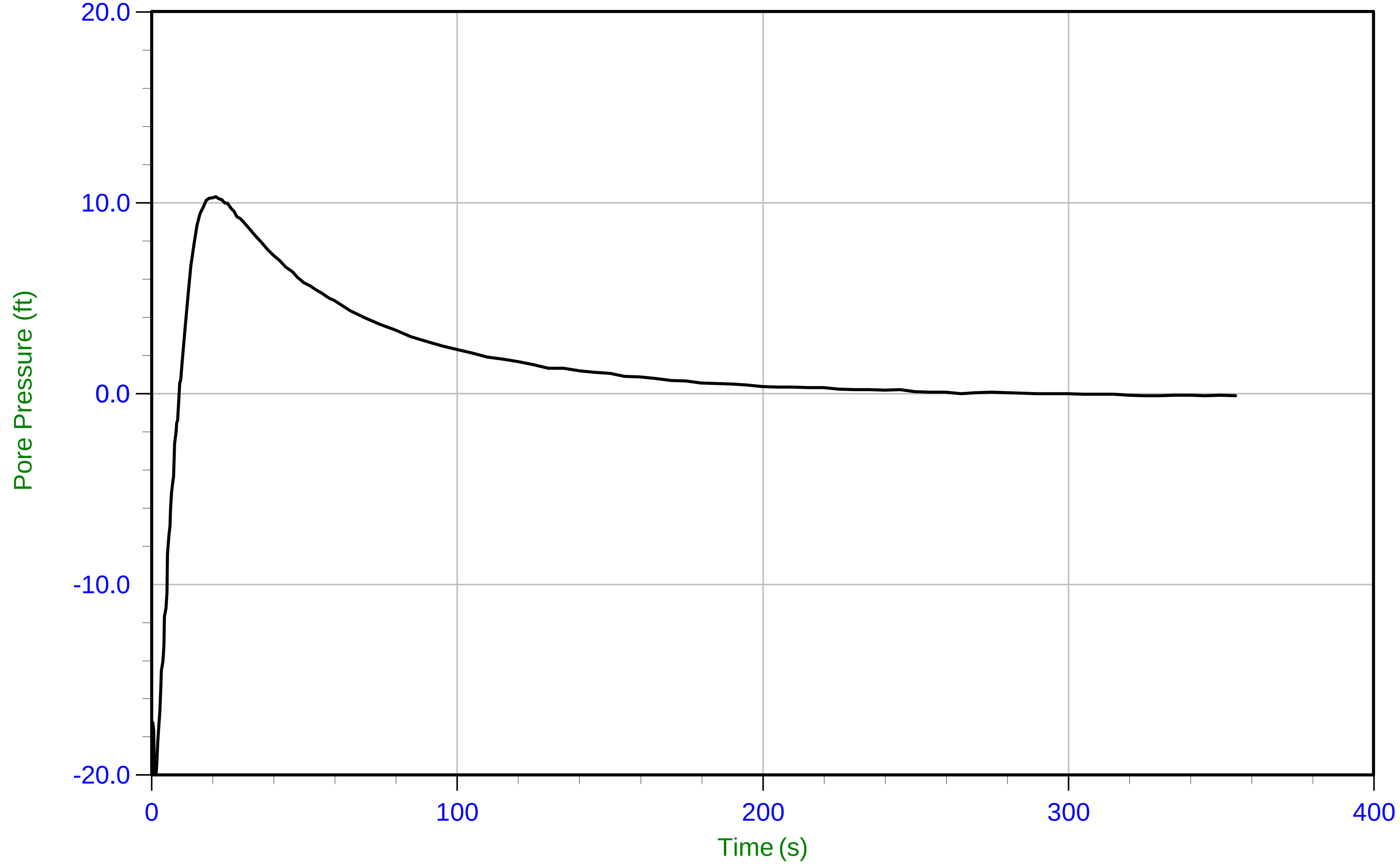
WT: 6.805 m / 22.326 ft
Ueq: 18.9 ft

CONETEC

Columbia West

Job No: 23-59-25320
Date: 01/31/2023 12:25
Site: St Helens Road CPT

Sounding: CPT-05B
Cone: 870:T1500F15U35 Area=15 cm²



Trace Summary:

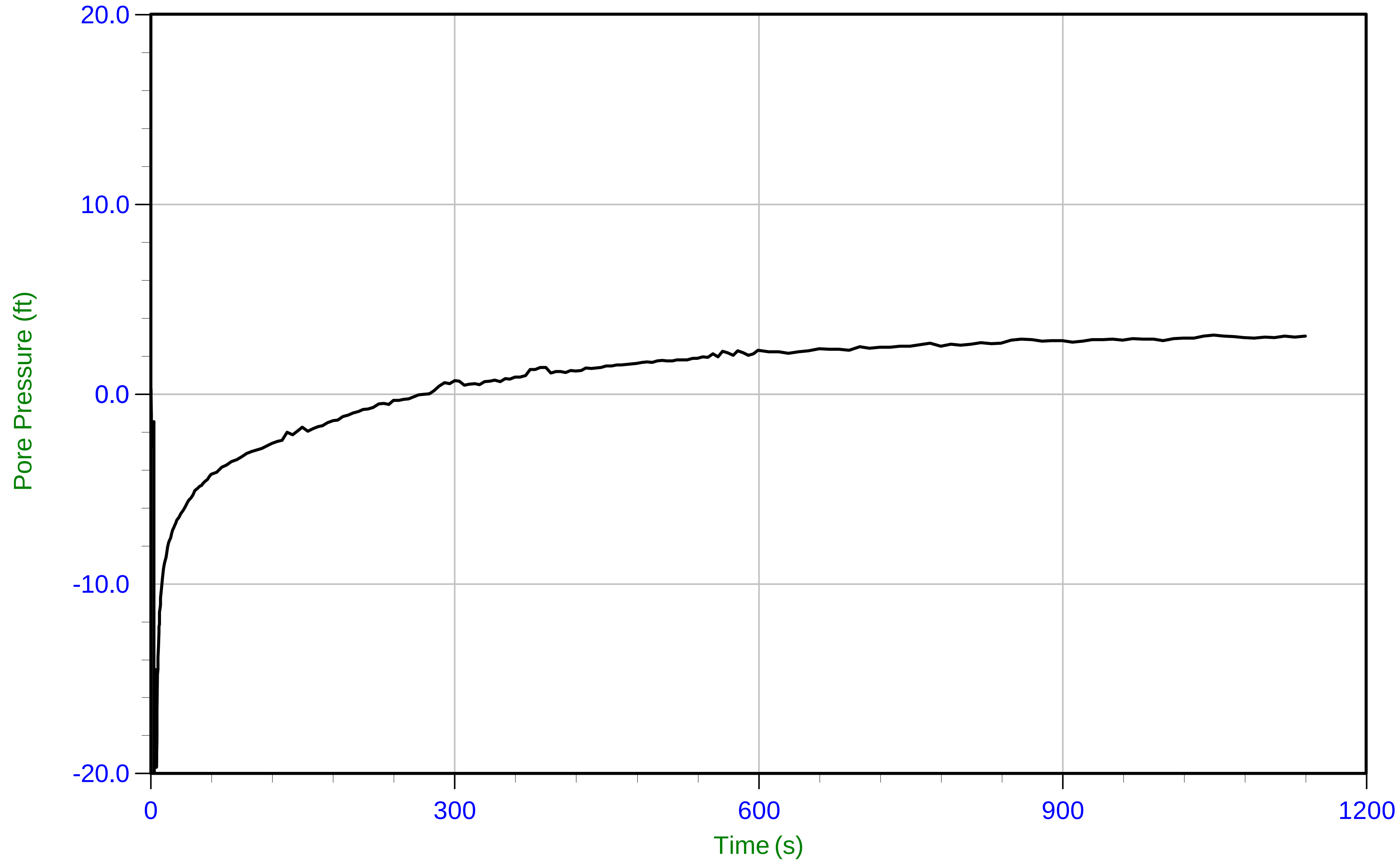
Filename: 23-59-25320_CP05B.PPR2
Depth: 5.550 m / 18.208 ft
Duration: 355.0 s

u Min: -21.4 ft
u Max: 10.3 ft
u Final: -0.1 ft

Columbia West

Job No: 23-59-25320
Date: 01/31/2023 12:25
Site: St Helens Road CPT

Sounding: CPT-05B
Cone: 870:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-25320_CP05B.PPR2
Depth: 6.950 m / 22.802 ft
Duration: 1140.0 s

u Min: -21.1 ft
u Max: 3.1 ft
u Final: 3.1 ft

WT: 6.002 m / 19.691 ft
Ueq: 3.1 ft

APPENDIX F
PHOTO LOG

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Drilling SB-1, Facing Southwest

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Drilling SB-3, Facing East

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Performing CPT-2, Facing South

Split Spoon Sample, SB 5.5

Slope on Western Edge of site, Facing North

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Slope on Western Edge of site adjacent to NW Cornelius Pass Rd,

Facing Northeast

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Slope on Southwest Edge, Facing West

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Central Site Area, Facing West

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Northwestern edge, Facing South

NARA RESIDENTIAL TREATMENT CENTER

PORTLAND, OREGON

Slope on Northeast Edge of site adjacent to NW St Helens Rd, Facing Northwest

APPENDIX G
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Date: March 1, 2023
Project: NARA Residential Treatment Center
Portland, Oregon

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

This report has been prepared in accordance with standard fundamental principles and practices of geotechnical engineering and/or environmental consulting, and in a manner consistent with the level of care and skill typical of currently practicing local engineers and consultants. This report has been prepared to meet the specific needs of specific individuals for the indicated site. It may not be adequate for use by other consultants, contractors, or engineers, or if change in project ownership has occurred. It should not be used for any other reason than its stated purpose without prior consultation with Columbia West Engineering, Inc. (Columbia West). It is a unique report and not applicable for any other site or project. If site conditions are altered, or if modifications to the project description or proposed plans are made after the date of this report, it may not be valid. Columbia West cannot accept responsibility for use of this report by other individuals for unauthorized purposes, or if problems occur resulting from changes in site conditions for which Columbia West was not aware or informed.

Report Conclusions and Preliminary Nature

This geotechnical or environmental report should be considered preliminary and summary in nature. The recommendations contained herein have been established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. The exploration and associated laboratory analysis of collected representative samples identifies soil conditions at specific discreet locations. It is assumed that these conditions are indicative of actual conditions throughout the subject property. However, soil conditions may differ between tested locations at different seasonal times of the year, either by natural causes or human activity. Distinction between soil types may be more abrupt or gradual than indicated on the soil logs. This report is not intended to stand alone without understanding of concomitant instructions, correspondence, communication, or potential supplemental reports that may have been provided to the client.

Because this report is based upon observations obtained at the time of exploration, its adequacy may be compromised with time. This is particularly relevant in the case of natural disasters, earthquakes, floods, or other significant events. Report conclusions or interpretations may also be subject to revision if significant development or other manmade impacts occur within or in proximity to the subject property. Groundwater conditions, if presented in this report, reflect observed conditions at the time of investigation. These conditions may change annually, seasonally or as a result of adjacent development.

Additional Investigation and Construction QA/QC

Columbia West should be consulted prior to construction to assess whether additional investigation above and beyond that presented in this report is necessary. Even slight variations in soil or site conditions may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions do not differ materially or significantly from the interpreted conditions utilized for preparation of this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Actual subsurface conditions are more readily observed and discerned during the earthwork phase of construction when soils are exposed. Columbia West cannot accept responsibility for deviations from recommendations described in this report or future performance of structural facilities if another consultant is retained during the construction phase or Columbia West is not engaged to provide construction observation to the full extent recommended.

Collected Samples

Uncontaminated samples of soil or rock collected in connection with this report will be retained for thirty days. Retention of such samples beyond thirty days will occur only at client's request and in return for payment of storage charges incurred. All contaminated or environmentally impacted materials or samples are the sole property of the client. Client maintains responsibility for proper disposal.

Report Contents

This geotechnical or environmental report should not be copied or duplicated unless in full, and even then only under prior written consent by Columbia West, as indicated in further detail in the following text section entitled *Report Ownership*. The recommendations, interpretations, and suggestions presented in this report are only understandable in context of reference to the whole report. Under no circumstances should the soil boring or test pit excavation logs, monitor well logs, or laboratory analytical reports be separated from the remainder of the report. The logs or reports should not be redrawn or summarized by other entities for inclusion in architectural or civil drawings, or other relevant applications.

Report Limitations for Contractors

Geotechnical or environmental reports, unless otherwise specifically noted, are not prepared for the purpose of developing cost estimates or bids by contractors. The extent of exploration or investigation conducted as part of this report is usually less than that necessary for contractor's needs. Contractors should be advised of these report limitations, particularly as they relate to development of cost estimates. Contractors may gain valuable information from this report, but should rely upon their own interpretations as to how subsurface conditions may affect cost, feasibility, accessibility and other components of the project work. If believed necessary or relevant, contractors should conduct additional exploratory investigation to obtain satisfactory data for the purposes of developing adequate cost estimates. Clients or developers cannot insulate themselves from attendant liability by disclaiming accuracy for subsurface ground conditions without advising contractors appropriately and providing the best information possible to limit potential for cost overruns, construction problems, or misunderstandings.

Report Ownership

Columbia West retains the ownership and copyright property rights to this entire report and its contents, which may include, but may not be limited to, figures, text, logs, electronic media, drawings, laboratory reports, and appendices. This report was prepared solely for the client, and other relevant approved users or parties, and its distribution must be contingent upon prior express written consent by Columbia West. Furthermore, client or approved users may not use, lend, sell, copy, or distribute this document without express written consent by Columbia West. Client does not own nor have rights to electronic media files that constitute this report, and under no circumstances should said electronic files be distributed or copied. Electronic media is susceptible to unauthorized manipulation or modification, and may not be reliable.

Consultant Responsibility

Geotechnical and environmental engineering and consulting is much less exact than other scientific or engineering disciplines, and relies heavily upon experience, judgment, interpretation, and opinion often based upon media (soils) that are variable, anisotropic, and non-homogenous. This often results in unrealistic expectations, unwarranted claims, and uninformed disputes against a geotechnical or environmental consultant. To reduce potential for these problems and assist relevant parties in better understanding of risk, liability, and responsibility, geotechnical and environmental reports often provide definitive statements or clauses defining and outlining consultant responsibility. The client is encouraged to read these statements carefully and request additional information from Columbia West if necessary.