Geotechnical Investigation Portland General Electric

PGE Harborton 230-kV Transmission Line

Portland, Oregon

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Prepared for

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1 INTRODUCTION

As requested, GRI is providing geotechnical design services for the proposed Portland General Electric (PGE) Harborton 230-kV Transmission Line project in Portland, Oregon. The general location of the project is shown on the Vicinity Map, Figure 1. The purpose of our services was to evaluate the subsurface conditions near existing or proposed transmission line towers and pole locations selected by you, provide design recommendations for new pole foundations, and evaluate sloping terrain within the project areas where new cuts and fills are planned. Our services to date have included a review of available subsurface and geologic information for the site and surrounding area, site reconnaissance, coordination and completion of subsurface explorations, collection of slope monitoring data, geotechnical analysis and design, and preparation of this report. Ongoing slope monitoring is also being completed at two discrete locations, with the updated results being provided to PGE through the duration of the project design phase.

1.1 Background Information

GRI has completed previous geotechnical investigations for PGE adjacent to the current project site. The following reports, memoranda, and letters were reviewed, and relevant information was used for this study:

Memorandum titled, "Geotechnical Engineering Services for Water Treatment Swale, PGE NW Marina Way, Portland, Oregon," dated March 2017.

Draft report titled, "Pavement Design Report, PGE NW Marina Way, Portland, Oregon," dated October 2017.

Report titled, "Geotechnical Investigation, PGE Harborton Transmission Line System, Portland, Oregon," dated March 2018.

Letter titled, "Geotechnical Engineering Services for Retaining Walls, PGE Harborton Frontage Road Improvements, NW Marina Way, Portland, Oregon," dated December 2019.

Report titled, "Geotechnical Investigation, PGE Harborton Substation Transmission Line Poles, Portland, Oregon," dated September 2020.

2 PROJECT DESCRIPTION

PGE is planning improvements to the existing Harborton transmission lines that extend southwest of Hwy 30 (NW St. Helens Road) and the Harborton substation and uphill into Forest Park. The project area is shown on the Site Plan, Figure 2. The improvements will include installation of new transmission line pole structures to augment or replace existing structures. Because the transmission lines cross relatively steep terrain along a portion of the alignment within Forest Park, improved access roads and new work pad areas will also be required as part of the transmission line improvements. We understand some work pad areas will be used to service existing towers, while other work pad areas will service new transmission line poles.

A site plan with observed existing conditions and proposed site development was prepared by David Evans and Associates (DEA) for PGE as part of the Portland Parks and Recreation Permit of Entry (Permit #2023-35). The figure by DEA includes site features, existing access roads, and existing transmission line tower locations. A modified version of the figure is included in Appendix D of this report and is used for referencing pertinent site features described in this report.

Concept-level design was completed for PGE by Mackay Sposito to develop preliminary grading plans for the work areas within Forest Park. To aid in discussion and description of the work areas, we included selected pages from the 90% Concept Design drawings in Appendix D of this report. A review of the concept-level design indicates proposed cuts and fills of up to about 10 feet for constructing the work areas, which are also referred to as "landings" in the design drawings. The plans also indicate that up to four new transmission line pole structures are planned within Forest Park. We understand that two pole structures, identified as SP3 and SP9, are planned at a downslope location in Forest Park (see "Sheet 3 of 12", Appendix D), one pole identified as SP5 is planned further upslope ("Sheet 4 of 12", Appendix D), and another possible pole structure identified as Alternative Option: 2998 may be constructed upslope as well ("Sheet 7 of 12", Appendix D).

In addition to the transmission line improvements noted above in Forest Park, we understand that up to three new pole structures will be added adjacent to the Harborton substation, between the substation and NW Marina Way. The pole structures are identified as SP2, SP7, and SP8. Each of the proposed pole structures, including those within Forest Park and those adjacent to Harborton substation, are expected to be supported by a single drilled shaft (i.e., drilled pier) foundation.

3 SITE DESCRIPTION

3.1 General

As noted above, the project alignment extends southwest from the Harborton substation, across Hwy 30, and uphill into and through Forest Park. The existing terrain along the project alignment varies with location. Between the Harborton substation and Hwy 30, the terrain slopes moderately to gently down to the northeast (i.e., toward the Willamette River). The flatter terrain is traversed by two parallel roads (Hwy 30 and NW Marina Drive) and railroad tracks. Southwest of Hwy 30, the project extends uphill into Forest Park, where the terrain is typically much steeper with some undulating and/or rolling hills. The area within Forest Park is mostly forested, with minor development consisting of meandering access roads and the existing Bonneville Power Administration (BPA) and PGE transmission line infrastructure. Selected swaths of the forested area have been thinned to accommodate the existing transmission lines. A gravel-surface road identified as BPA Road extends generally southwest and uphill from Hwy 30 to provide access to portions of the project site. A perennial stream and wetland areas were delineated by DEA (see figure in Appendix D). The stream generally flows northeast, crossing one of the upper access roads and the wetland areas on the lower access road above Hwy 30.

3.2 Geology

Published geologic mapping indicates the downslope portion of the project area near the substation is generally mantled with alluvium or fill soils associated with historic development of the properties adjacent to the Willamette River (Madin et al., 2008; Wells et al., 2020). The uphill portion of the project area within Forest Park is typically underlain at shallow depth by Columba River Basalt but also includes surficial alluvium and loess-basalt fragment colluvium (Madin et al., 2008; Wells et al., 2020). The mapped geology is generally consistent with the conditions encountered during previous and current investigations, as discussed in more detail in the following sections of this report.

Prehistoric landslide deposits are mapped in the uphill areas within Forest Park. The Oregon Department of Geology and Mineral Industries (DOGAMI) State Landslide Information Database for Oregon (SLIDO) interactive database identifies a landslide deposit and uphill scarp within the area where existing transmission line structures are presently located and where new structures and grading are planned. More recent mapping by Wells et al. (2020) identifies smaller areas of landslide deposits relative to the previous DOGAMI mapping. Geologic Map, Figure 3 includes the project area with overlays of the DOGAMI landslide mapping and Wells et al. (2020) mapped geology. We understand that the presence of mapped landslide terrain was a key item identified in the City of Portland Bureau of Development Services (BDS) review. Additional information regarding the landslide hazard is discussed in Section 5.2: Evaluation and Monitoring of Mapped Landslide Terrain.

4 SUBSURFACE CONDITIONS

4.1 General

Subsurface materials and conditions at the site were investigated between September 11 and September 26, 2023, with four machine-drilled borings, designated B-31, B-32, B-34, and B-35; two hand-auger borings, designated HA-1 and HA-2; two dynamic cone penetration (DCP) tests, designated DCP-1 and DCP-2; two cone penetrometer test (CPT) probes, designated CPT-1 and CPT-2; and a seismic refraction geophysical survey. The approximate locations of the explorations completed for this investigation are shown on Figure 2. The field and laboratory programs conducted to evaluate the physical engineering properties of the materials encountered in the borings are described in Appendix A. A summary of the fieldwork is provided below.

4.2 Machine-Drilled Borings

Borings B-31, B-32, B-34, and B-35 were drilled at locations selected by PGE to coincide with approximate new transmission pole and/or existing lattice tower locations and proposed work pad areas. The locations for B-32, B-34, and B-35 were chosen with consideration to site access and clearing restrictions within Forest Park. We understand the naming convention for these borings coincides with other previous PGE borings. Boring B-31 was located near the proposed poles SP2, SP7, and SP8 near the Harborton substation. Boring B-32 was located near the proposed poles SP3 and SP9 within Forest Park. Boring B-34 was located near the work pad area for the existing lattice tower 2998 and a possible new alternative pole 2998. Boring B-35 was located between the proposed work pad areas for existing lattice towers 2996 and 2997 and uphill of proposed pole SP5. A fifth boring, B-33, was originally planned near the proposed pole SP5 but not completed based on potential land use restrictions identified by PGE.

The borings were drilled to depths ranging from about 15.1 feet to 67.8 feet below existing surface grades. Disturbed and undisturbed soil samples were generally obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet to 20 feet and at 5-foot intervals below these depths. Standard penetration tests (SPTs) were conducted while collecting disturbed samples from the drilled borings. The SPT N-values provide a measure of the relative density of granular soils and the relative stiffness, or consistency, of cohesive soils. Continuous rock core was obtained in each of the borings after encountering relatively competent bedrock. Rock coring was generally completed within the bottom 8 feet to 12 feet of the borings. All of the soil and rock samples were returned to our laboratory for further examination and possible testing. Borings B-34 and B-35 also included installation of a vibrating wire piezometer (VWP) and datalogger for further measurement of groundwater and slope inclinometer casing for monitoring of lateral movement at the borehole locations. Additional details of the sampling, SPTs, rock coring, and monitoring installations are provided in Appendix A. Logs of the borings are provided on Figures 1A through 4A. The terms and symbols used to describe the materials encountered in the borings are defined in Tables 1A and 2A and on the attached legend.

4.3 Hand-Auger Borings and DCP Testing

Hand-auger borings HA-1 and HA-2 were completed to supplement the drilled borings at two locations that were generally not accessible by the drill rig used to complete the machine-drilled borings. HA-1 was excavated near existing lattice tower 2996 and extended to a depth of approximately 12 feet. HA-2 was excavated near existing lattice tower 2997 and extended to a depth of approximately 9 feet. Disturbed grab samples were collected at selected depths in each boring for further examination and laboratory testing. Logs of the borings are provided on Figure 5A.

Dynamic cone penetration (DCP) tests DCP-1 and DCP-2 were completed in conjunction with hand-auger borings HA-1 and HA-2, respectively, using a Wildcat DCP apparatus. The DCP test results are used to assess the density or stiffness characteristics of the soils. The DCPs were advanced to depths ranging from about 16 feet to 19 feet below the existing ground surface. The DCP test results are summarized on Figure 6A.

4.4 CPT Probes

Cone penetrometer test (CPT) probes CPT-1 and CPT-2 were advanced near the location of boring B-31 and proposed transmission line poles SP2, SP7, and SP8 adjacent to Harborton substation. The location of CPT-1 was selected in consultation with PGE and included seismic shear wave velocity (Vs) testing. CPT-2 was added to help further characterize the foundation soils and depth to bedrock near the proposed pole locations. The CPT probes were advanced to depths of approximately 47 feet at CPT-1 and 47.8 feet at CPT-2, where refusal of the CPTs was encountered. Logs of the CPT probes are provided on Figures 7A and 8A, and the full report prepared by ConeTec, Inc. is provided as Appendix B of this report. The terms used to describe the soils encountered in the CPT probes are defined in Table 3A.

4.5 Seismic Refraction

A seismic refraction study was completed to supplement the boring data near the proposed transmission line poles SP3 and SP9 and the associated work pad areas. The primary focus of the study was to help evaluate rock depths near boring B-32 and extend north where access with other exploration equipment was not feasible. A report prepared by Earth Dynamics, LLC, summarizing the seismic refraction study, is provided as Appendix C of this report.

4.6 Discussion of Soil and Rock Conditions

For discussion purposes, the soils and rock encountered during our investigation have been grouped into the following units based on their physical characteristics and engineering properties. The subsurface conditions typically varied between the exploration locations due to the varying terrain and distance between the explorations. Therefore, the soil units discussed herein were not necessarily encountered in each exploration. However, in general, the units encountered from the ground surface downward are as follows:

a. Alluvium
b. Portland Hills Silt/Possible Landslide Debris
c. Residual Soil/Possible Landslide Debris
d. Decomposed Columbia River Basalt

e. Columbia River Basalt

The following paragraphs provide a description of the materials encountered in the explorations and a discussion of the anticipated groundwater conditions along the project alignment.

a. Alluvium

Alluvial soils generally consisting of silt to silty sand were encountered in boring B-31, drilled nearest to the Harborton substation at the northeast end of the project site. Medium-stiff grading to very soft to soft silt with up to some clay and sand was encountered to a depth of about 25 feet, followed by loose grading to medium-dense silty sand extending to a depth of approximately 48 feet. The alluvial soils are underlain by basaltic rock.

Interpreted logs from CPT-1 and CPT-2 suggest similar conditions relative to those encountered in boring B-31. The CPT refusal depths of approximately 47 feet at CPT-1 and 47.8 feet at CPT-2 also suggest the likely interface between the alluvial soils and underlying basalt rock. Previous borings completed by GRI adjacent to boring B-31 and the Harborton substation encountered similar alluvial soils to depths ranging from about 45 feet to 52 feet.

b. Portland Hills Silt/Possible Landslide Debris

Portland Hills Silt, which is predominantly comprised of loess and/or eolian soils, mantles much of the upland area within Forest Park. These soils were encountered extending below the ground surface in boring B-32 to a depth of approximately 3 feet, in boring B-35 to a depth of approximately 20 feet, and in hand-auger borings HA-1 and HA-2 to depths of about 12 feet and 9 feet, respectively. In B-32, this unit consists of stiff silt with some clay and trace sand. In boring B-35, the unit consists of layers of silt and clay; B-35 encountered stiff clay with some silt to a depth of about 5 feet, followed by medium stiff to stiff silty clay with trace sand to about 15 feet, then stiff to very stiff silt with trace sand to about 20 feet. In borings HA-1 and HA-2, the Portland Hills Silt consists of medium-stiff to very stiff silt with trace sand.

We identified the unit encountered in boring B-35 as Portland Hills Silt/Possible Landslide Debris because the boring is located within an area of mapped landslide terrain. Therefore, although we did not encounter obvious conditions to indicate landslide materials, the possible presence of previous landslide activity should be acknowledged.

c. Residual Soil/Possible Landslide Debris

Residual soil is material that has generally weathered or decomposed from a parent rock to a soil-like condition. Residual soil was encountered in B-34, extending below the ground surface to a depth of approximately 30.5 feet, and in B-35, extending below the Portland Hills Silt to a depth of approximately 45.5 feet. The residual soil encountered in the borings consists of very stiff to hard clayey silt with trace to some sand and trace gravel.

Similar to the Portland Hills Silt unit, we identified the residual soil encountered in borings B-34 and B-35 as possible landslide debris. We did not encounter obvious conditions in the borings to indicate landslide materials within this stratum. However, the possible presence of previous landslide activity should be acknowledged.

d. Decomposed Columbia River Basalt

The Columbia River Basalt underlies the project site at varying depths. Within borings B-32, B-34, and B-35, the upper portion of the basalt is weathered to the consistency of very dense angular rock fragments with silt, clay, and sand infilling. The decomposed rock was encountered in boring B-32 from a depth of approximately 3 feet to 5 feet, in boring B-34 from a depth of approximately 30.5 feet to 40.3 feet, and in boring B-35 from a depth of approximately 45.5 feet to 60 feet.

e. Columbia River Basalt

Columbia River Basalt was encountered in each of the borings, extending to the maximum depth of the explorations. As noted above, the upper portion of the basalt encountered in borings B-32, B-34, and B-35 was decomposed to the consistency of very dense angular rock fragments. Less weathered basalt was encountered in the borings at depths of approximately 48 feet in B-31, 5 feet in B-32, 40.3 feet in B-34, and 60 feet in B-35.

Laboratory unconfined compressive strengths ranging from about 9,000 pounds per square inch (psi) to 23,300 psi were recorded for samples of rock core retained from the borings, indicating relative rock strengths ranging from strong to very strong (R4 to R5). However, not all of the rock core retained from the borings was testable due to higher fracturing, particularly for the deeper rock in borings B-34 and B-35. In general, we described the rock as being within the range of weak to very strong (R2 to R5), depending on boring location and test depth.

Rock depth near proposed transmission line poles SP3 and SP9 was also evaluated using the results of the seismic refraction study, which was compared to data collected from boring B-32. The results of that study suggest rock is relatively shallow across the test area, with estimated depths on the order of about 5 feet or less below the ground surface.

4.7 Groundwater

Based on data provided by the U.S. Geological Survey groundwater map of the Portland area (Snyder, 2008), the regional groundwater table varies from about 10 feet to 20 feet below the ground surface near the substation and up to 100 feet below the ground surface near the locations of B-34 and B-35 in Forest Park. However, it is our experience that perched groundwater conditions often develop on the surface of the bedrock or above fine-grained, low-permeability soil layers. As noted above, less weathered basalt was encountered, ranging from depths of about 40 feet to 60 feet below the ground surface. In addition, due to the steep terrain, temporary groundwater conditions may develop in drainages or surrounding perennial streams during heavy precipitation and surface runoff events.

Mud-rotary drilling methods precluded an accurate determination of groundwater in the machine-drilled borings at the time of the explorations. However, additional means were used to evaluate groundwater conditions across the site.

Pore pressure measurements in the CPT probes near the Harborton substation estimated groundwater depths in the range of approximately 11 feet to 13 feet below the ground surface at the time of the exploration. These measurements are generally consistent with groundwater depths estimated by Snyder (2008), as noted above. Based on the proximity of the substation to the Willamette River, we anticipate groundwater in this portion of the site remains relatively shallow throughout the year and may fluctuate with changes in the river elevation. Groundwater near the substation may approach the ground surface during periods of heavy or prolonged precipitation.

As noted above, VWPs with dataloggers were installed at the locations of borings B-34 and B-35 to measure groundwater conditions near the bedrock-soil interface over time. Measurements were automatically collected two times per day, and the information from the dataloggers was periodically downloaded by GRI staff. During the course of our readings, groundwater was not measured by the VWPs. This suggests possible dry conditions and/or no perched water within the zone of influence of the VWPs. GRI will continue obtaining VWP readings to evaluate any changes over the coming months.

5 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

5.1 General

The geotechnical explorations completed for this project indicate the subsurface conditions vary along the project alignment. At the northeast end of the project, near the Harborton substation (i.e., near proposed poles SP2, SP7, and SP8), the subsurface profile typically includes a relatively deep deposit of soft or loose grading to medium dense alluvial soils, with basaltic rock encountered at depths on the order of about 47 feet to 48 feet below existing grades. Across Hwy 30 near the base of the hillside within Forest Park (i.e., near proposed poles SP3 and SP9), basalt rock was encountered at shallow depths and is expected within about 5 feet of the ground surface. Further southwest and uphill within Forest Park (i.e., near proposed pole SP5 and work pad areas 2996, 2997, and

2998), the subsurface profile includes Portland Hills Silt over residual soils, followed by decomposed grading to less weathered basalt. The soils within the uphill portion of Forest Park were also identified as possible landslide debris based on mapping by DOGAMI and others.

We understand that the new transmission line pole structures will be supported by drilled shaft foundations, which consist of structural concrete with a reinforcing rebar cage. The required foundation depths will depend on the anticipated lateral loads and axial (i.e., compressive and uplift) loads and the depths required to resist the loads within acceptable performance criteria. Recommended soil parameters for evaluating the proposed drilled shaft foundations are provided in the following sections of this report.

Another consideration for the project is site grading and the construction of work pads, particularly within the uphill portion of Forest Park where landslide terrain has previously been mapped. Our current evaluation of the landslide topography within the work pad areas is discussed below along with geotechnical considerations for earthwork and construction of the work pads and access roads.

5.2 Evaluation and Monitoring of Mapped Landslide Terrain

5.2.1 General

The focus of our slope stability evaluation and monitoring is limited to the locations where new transmission line pole structures and work pad areas associated with this project are proposed, as described in the preceding sections of this report. As discussed above, portions of the project alignment cross through slopes with sections of mapped landslide terrain, as documented in the DOGAMI SLIDO interactive database and by Wells et al. (2020). Therefore, in addition to the subsurface explorations, GRI completed an assessment to evaluate surface features that may be associated with potential geologic and/or landslide hazards.

5.2.2 Statewide Landslide Database

DOGAMI is the state agency responsible for geologic hazard mapping for the State of Oregon and maintains SLIDO (version 4.4), a spatial database of landslide susceptibility and mapped and inferred landslides. As discussed in Section 3.2: Geology, a landslide (Unique ID Linnton_317) is mapped within the upper project area in Forest Park. SLIDO describes the landslide as a prehistoric (>150 years ago) deep-seated slump/earth flow. Numerous landslides are mapped in the surrounding area. The project area southwest of Hwy 30 is defined as having a moderate to high susceptibility to both shallow and deep landslides.

5.2.3 *Review of Lidar and Remote Sensing Data*

DOGAMI also maintains a bare earth Light Detection and Ranging (lidar) hillshade layer, accessible through SLIDO or other web mapping services. Due to the large volume of laser measurements utilized by lidar systems to map topography, enough measurements are able to penetrate the surface vegetation to develop a detailed terrain model with the vegetation removed. This is referred to as a "bare earth" model and uncovers many topographic and landslide features that would normally be obscured by vegetation. Within the mapped landslide area, a scarp feature and potential hummocky terrain were noted. Outside of the mapped landslide area, no other obvious large-scale landslide features or morphology were identified using the lidar data.

5.2.4 *Site Reconnaissance*

Several visits were made to the project area by experienced Oregon-licensed geotechnical engineers and certified engineering geologists from GRI. During our site visits, we did not observe obvious signs of significant landslides (i.e., recent scarps, ground cracks, hummocky terrain, leaning trees) within the proposed work areas. Localized areas of slumping and erosion were observed, including minor erosion along a natural drainage crossing the access road to the existing transmission line towers and associated proposed work pad areas 2996, 2997, and 2998. The location of observed stream erosion is identified on the modified figure originally developed by DEA in Appendix D. Such surface erosion is typical within drainages and steep terrain and does not necessarily indicate other deep-seated stability risks. However, along drainages within sloping terrain, there is always an inherent risk of future erosion and accompanying surface instability over time. Further discussion of mitigation of this risk in relation to PGE's proposed transmission line development is provided in later sections of this report.

Our observation of the existing transmission line structures located in or adjacent to the mapped landslide topography suggests that the existing structure foundations have performed satisfactorily over the life of these structures. In this regard, we did not observe indications of foundation and/or structure rotation or undermining in these areas. PGE also did not identify or report any ongoing maintenance issues with the existing structures.

5.2.5 *Slope Monitoring*

To further evaluate potential ongoing slope movement within the documented landslide terrain, we installed slope inclinometer casings in borings B-34 and B-35. Details of the installation are provided in Appendix A. We periodically visited the site to collect data using GRI's slope inclinometer instruments to measure lateral movement within the cased depths. The results are plotted on Figures 9A and 10A. The uncorrected readings provided on Figures 9A and 10A were taken between Fall 2023 and Spring 2024. In general, movements in unstable slopes are most often observed in the winter and spring months, when precipitation, run-off, and groundwater levels are highest. In B-34, the inclinometer readings indicate negligible lateral movement during the period of observation. In B-35, movement of less than 0.1 inch was observed in the downhill direction to a depth of about 17 feet. However, the pattern of movement over the period of readings in B-35 was not consistent. For example, the readings taken in November 2023 showed greater displacement compared to the later readings taken in December 2023, and the readings taken in February 2024 were nearly identical to the November 2023 readings. The most recent reading in June 2024 showed a very minor offset along almost the entire depth of the inclinometer casing. Therefore, it appears that the "movement" noted at boring B-35 is within the tolerance of the installed equipment and gear.

In consultation with PGE, further periodic monitoring is planned through the design phase of the project to monitor any changes and evaluate whether minor movement, such as observable slope creep, is occurring at the locations of borings B-34 and B-35. If appreciable movement is observed, further recommendations may be warranted to mitigate slope hazards. However, we do not anticipate such conditions based on the data collected to date.

5.2.6 Slope Stability Conclusions and Considerations

Based on our review of the available geologic data, lidar, site observations, and monitoring, in our opinion, there is a relatively low risk of landslide hazards within the noted project work areas. The absence of observable, active instabilities within the sloping terrain precluded numerical slope stability analysis at the work areas within the project site. Where portions of the project area are located within mapped landslide terrain, it is our opinion that previous deep-seated landslides are likely prehistoric (greater than 150 years ago) and the areas are presently stable in the current ground and slope configurations. Continued monitoring (e.g., collection of slope inclinometer data) is planned to further evaluate whether any hillside slope creep is occurring in the vicinity of the instrumented borings.

Proposed grading (i.e., cuts and fills) to create work pad areas adjacent to existing and proposed transmission line structures will alter surface grades at the noted locations. This work will include tree removal and/or thinning in work pad areas and beneath transmission lines. As with any earthwork within sloping terrain, the grading plan and finished site grades must account for the surface and subsurface conditions to limit the risk of future slope instability or the potential for debris flow (i.e., soil movement through narrow channels such as existing drainages). Further discussion and recommendations for earthwork are provided in a subsequent section of this report. In our opinion, the recommendations provided herein will reduce the risk of project improvements adversely affecting the existing slopes.

5.3 Seismic Considerations

5.3.1 Seismic Criteria

If necessary, we anticipate the 2022 Oregon Structural Specialty Code (OSSC) will be used to develop seismic parameters for the transmission line structures. The 2022 OSSC is based on the International Building Code (IBC) and incorporates recommendations for seismic design from the American Society of Civil Engineers (ASCE) document 7-16, *Minimum Design Loads for Building and Other Structures* (ASCE 7-16). The ASCE 7-16 seismic-hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCE_R). The ground motions associated with the probabilistic MCE_R represent a targeted risk level of 1% in 50 years probability of collapse in the direction of maximum horizontal response. In general, these risk-targeted ground motions are developed by applying adjustment factors of directivity and risk coefficients to the 2% probability of exceedance in 50 years, or a 2,475-year return-period hazard level. The risk-targeted probabilistic values are also subject to a deterministic limit.

The ASCE methodology uses two bedrock spectral response-mapped acceleration parameters, S_S and S_1, corresponding to periods around 0.2 second and 1.0 second to develop the MCE_R response spectrum. To establish the ground-surface MCE_R spectrum, these mapped bedrock spectral parameters are adjusted for site class using the short- and long-period site coefficients, F_a and F_v, in accordance with Section 11.4.3 of ASCE 7-16, which includes seismic site coefficients to adjust the mapped values for soil properties.

5.3.2 Recommended Seismic Design Parameters

We used the results from the subsurface investigation to determine the appropriate seismic design parameters and site class in accordance with Section 20.4 of ASCE 7-16. As noted above, the subsurface conditions vary across the project site, which is reflected with varying site class recommendations.

The project area adjacent to the Harborton substation where new poles SP2, SP7, and SP8 are planned is underlain by alluvial soils that may be subject to seismic-induced liquefaction and/or cyclic softening, as described in more detail below. Such conditions generally warrant a Site Class F designation. Sites classified as Site Class F require a site-specific, site-response analysis per Section 20.3.1 of ASCE 7-16 unless the structure has a fundamental period of vibration, T, less than or equal to 0.5 second. The design response spectrum for sites with structures having a fundamental period of less than or equal to 0.5 second can be derived using the non-liquefied subsurface profile and code-tabulated site coefficients. The corresponding non-liquefied condition at this location is Site Class D, based on the results of the shear wave velocity testing at CPT-1.

The project area where boring B-32 was drilled, near poles SP3 and SP9, is underlain at shallow depth by relatively hard basalt. Therefore, a Site Class B is recommended at this location.

The project area where B-34 was drilled, near an existing lattice tower and the proposed 2998 work pad, is underlain by residual soil, followed by decomposed basalt, grading to less weathered basalt. Site Class C is appropriate at this location based on recommended correlations with SPT N-values.

The project area where B-35 was drilled is near two existing lattice towers, the proposed 2996 and 2997 work pads, and upslope of proposed pole SP5. The boring encountered Portland Hills Silt, followed by residual soil, then decomposed basalt grading to less weathered basalt. Site Class D is appropriate at this location based on recommended correlations with SPT N-values.

The ASCE 7-16 S_S and S_1 mapped spectral response acceleration parameters were determined from the USGS National Seismic Hazard Map based on latitude and longitude coordinates corresponding to the approximate exploration locations. Due to the S_1 acceleration parameter being greater than or equal to 0.2 g, Section 11.4.8 of ASCE 7-16 requires a ground-motion hazard analysis for Site Class D locations unless the seismic response coefficient, C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16. Assuming the seismic response coefficient, C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16, the site coefficients F_a and F_v were determined from code-tabulated values. The design-level response spectrum is calculated as two-thirds of the ground-surface MCE_R spectra. The recommended MCE_R- and design-level spectral-response parameters are provided below in Table 5-1.

Table 5-1: RECOMMENDED SEISMIC DESIGN PARAMETERS (2022 OSSC/ASCE 7-16)

Exploration	Mapped Parameters				Recommended Values						
	Latitude	Longitude	S_S (g)	S_1 (g)	Site Class	F_a	F_v	S_{MS} (g)	S_{M1} (g)	S_{DS} (g)	S_{D1} (g)
B-31/CPT-1	45.6135	-122.7982	0.90	0.42	D	1.14	1.88	1.03	0.79	0.68	0.53
B-32	45.6115	-122.7979	0.90	0.42	B	1.0	1.0	0.90	0.42	0.60	0.28
B-34	45.6108	-122.8021	0.90	0.42	C	1.2	1.5	1.08	0.62	0.72	0.42
B-35	45.6102	-122.8026	0.90	0.42	D	1.14	1.88	1.03	0.79	0.69	0.53

Notes:
1. Site Class D was identified for the location of Boring 31/CPT-1 based on shear wave velocity testing and assumes the structure will have a fundamental period, T, of less than 0.5 second.
2. Exception 2 of Section 11.4.8 should be considered when evaluating base shear calculations in Section 12.8 where Site Class D conditions are noted.

S_{MS} = MCE_R 0.2-Sec Period Spectral Response Acceleration
S_{M1} = MCE_R 1.0-Sec Period Spectral Response Acceleration
S_{DS} = Design-Level 0.2-Sec Period Spectral Response Acceleration
S_{D1} = Design-Level 1.0-Sec Period Spectral Response Acceleration

5.3.3 Liquefaction/Cyclic Softening

Soils that are potentially susceptible to liquefaction and/or cyclic softening were encountered in boring B-31 located adjacent to the Harborton substation. Similar conditions were also encountered in previous explorations completed by GRI near the substation. Such conditions were not encountered in the other explorations completed for this project. A brief description of these hazards and a subsequent evaluation are provided below.

Liquefaction. "Liquefaction" is the process by which loose, saturated granular materials, such as clean sand and, to a somewhat lesser degree, nonplastic and low-plasticity silts, temporarily lose stiffness and strength during and immediately after a seismic event. This degradation in soil properties may be substantial and abrupt, particularly in loose sands. Liquefaction occurs as seismic shear stresses propagate through saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure causes the pore-water pressure to increase between the soil grains. If the pore-water pressure becomes sufficiently large, the intergranular stresses become small, and the granular layer temporarily behaves as a viscous liquid rather than a solid. After liquefaction is triggered, there is an increased risk of settlement, loss of bearing capacity, lateral spreading, and/or slope instability. Liquefaction-induced settlement occurs as the elevated pore-water pressures dissipate and the soil consolidates after the earthquake.

The potential for liquefaction is typically estimated using a simplified method that compares the cyclic shear stresses induced by the earthquake (demand) to the cyclic shear strength of the soil available to resist these stresses (resistance). Estimates of seismically induced stresses are based on earthquake magnitude (M_W) and peak ground acceleration (PGA). The cyclic resistance of soils is dependent on several factors, including the number of loading cycles, relative density, confining stress, plasticity, natural water content, stress history, age, depositional environment (fabric), and composition. The cyclic resistance of soils is evaluated using in-situ testing in conjunction with laboratory index testing, which may also include monotonic and cyclic laboratory strength tests. For sand-like soils, the cyclic resistance is typically evaluated using SPT N-values or CPT tip-resistance values normalized for overburden pressures and corrected for factors that influence cyclic resistance, such as fines content.

The potential for liquefaction of the alluvial soils encountered in boring B-31 was evaluated using the simplified method based on procedures recommended by Idriss and Boulanger (2008) with subsequent revisions (2014). Per ASCE 7-16 guidelines, our evaluation included the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site class effects (PGA_M). The USGS National Seismic Hazard Mapping Project was used to determine the contributing MCE_G hazard level earthquake magnitude associated with the mean PGA spectral period associated with the 2,475-year return-period hazard level. We evaluated the liquefaction risk assuming seismic loading parameters with PGA_M equal to 0.49g for the project location, a Site Class D profile, and an associated earthquake magnitude, M, equal to 7.91. The results of our analysis confirmed there is a high risk of liquefaction associated with the 2,475-year return-period hazard level if these soils are saturated. We do not anticipate significant liquefaction risk for the other soil units encountered in the other borings completed for the project.

We estimated associated free-field liquefaction-induced settlements near the substation based on the profile of boring B-31 using the methods of Tokimatsu and Seed (1987). The analysis indicates greater than 1 foot of seismic-induced settlements are possible. Settlement of the proposed transmission line poles at this location can be mitigated by extending the foundations to the underlying basalt. These structures will still incur downdrag loads at the foundation level from the settling soils. However, such downdrag loads are expected to be relatively small.

Cyclic Softening. "Cyclic softening" is a term that describes a relatively gradual and progressive increase in shear strain with load cycles and is more common within fine-grained soils. Excess pore pressures may increase due to cyclic loading but generally do not approach the total overburden stress. Shear strains accumulate with additional loading cycles, but an abrupt or sudden decrease in shear stiffness is not typically expected. Settlement due to post-seismic consolidation can occur, particularly in lower-plasticity silts. Large shear strains can develop, and strength loss related to soil sensitivity may be a concern. For clay-like soils, the cyclic resistance is typically evaluated using estimates of the undrained shear strength, overconsolidation ratio, and sensitivity or directly from cyclic laboratory tests.

For the soft, fine-grained alluvial soils (i.e., predominantly silts) encountered in boring B-31, there is a risk of cyclic softening occurring if the soils do not already liquefy during a design-level earthquake. It is our opinion that the risk of this occurrence is significantly less for the fine-grained soils encountered in the other explorations (i.e., borings B-32, B-34, and B-35) based on the stiffness of the soils encountered in the explorations and the deeper groundwater conditions.

5.3.4 Other Seismic Hazards

Although soft soils were identified adjacent to the Harborton substation site, our previous studies identified a low risk of lateral spreading or similar earthquake-induced slope instability because of the relatively flat site topography and distance from the edge of the Willamette River. Overall, we do not anticipate mitigation of such risks will be considered, as soft soil conditions are present across the area that includes the substation and are not limited to the new pole locations.

The explorations completed uphill within Forest Park for this project did not identify conditions indicating a significant risk of coseismic slope instability (i.e., slope stability under earthquake loading). With the existing landslide terrain mapped across the uphill portion of the project site (see Figure 3), it should be understood that there is some inherent risk of instability, particularly with the addition of seismic loads. However, based on our observations and understanding of the project, we do not anticipate the proposed project improvements will exacerbate this risk.

The nearest mapped fault is the Portland Hills Fault, which is presently mapped as crossing adjacent to the west side of the Harborton substation and, therefore, may cross the project alignment (see Figure 3). We did not identify traces of the fault in our explorations. If a proposed transmission line pole is located along the fault line, it may be prudent to move the structure in order to reduce the risk of damage with future fault movement. We anticipate PGE will review these criteria and determine the acceptable level of risk associated with locating the structures near the mapped fault line.

The risk of damage by a tsunami and/or seiche along the alignment is absent.

5.4 Transmission Pole Foundations

5.4.1 General

The new transmission line poles will be supported on drilled shafts and/or pier foundations. The required pier embedment may be controlled by the design lateral loading or axial loading conditions. The recommended soil and rock parameters for evaluating the proposed pier foundations are discussed in the following sections of this report and summarized below in Tables 5-2 through 5-5. The soil and rock profiles identified in the tables are based on the individual borings (B-31, B-32, B-34, and B-35) shown on Figure 2, supplemented with information from our other fieldwork and a review of other relevant information.

5.4.2 LPILE Analysis Parameters

Lateral structural loads can generally be resisted by the structural strength of the drilled pier or embedded pole in bending. We understand the drilled pier foundations will be evaluated using the computer software LPILE, developed by Ensoft, Inc., of Austin, Texas. Recommended input parameters for the soil units are provided in Tables 5-2 through 5-5 below. The lateral resistance within the top 2 feet of each pier should be neglected if soil disturbance near the ground surface during installation is a concern. Soil parameters satisfying both static and seismic loading conditions are provided for most soil units. Different static and seismic parameters are provided for the soil units where liquefaction risk has been identified.

5.4.3 Axial Resistance Parameters

The static axial resistances for the drilled piers were evaluated using methods discussed in the Federal Highway Administration (FHWA) publication FHWA-NHI-18-024, *Drilled Shafts: Construction Procedures and LRFD Design Methods*. The design method estimates axial (i.e., compression or uplift) resistances based on the estimated soil and rock parameters and the properties of the drilled pier.

Our analysis assumed the foundations would derive their axial resistance from skin friction that develops along the sides of the foundation and end-bearing resistance at the base of the foundation. The nominal (unfactored) static skin friction and end-bearing resistances for each soil and rock unit are tabulated in Tables 5-2 through 5-5 below to evaluate axial capacities for individual drilled piers. To provide a more conservative estimate of soil and rock strengths, the groundwater depths with submerged soil conditions are included at depths above where groundwater was observed in the explorations. Axial resistances within the upper 5 feet of the soil profiles should be neglected when calculating skin-friction resistances for the foundation elements.

As noted in Tables 5-2 through 5-5, the unit side and end-bearing values for axial resistances are provided as nominal or ultimate values. A typical factor of safety of 2.0 to 3.0 is applied to these values for establishing allowable resistances. Unit end-bearing resistances are not provided for soil units that are relatively weak or prone to seismically induced liquefaction. In this case, it is recommended that the pier tip extend below these soil units and into more competent soil or rock.

Table 5-2: SOIL AND ROCK PARAMETERS FOR DRILLED SHAFT ANALYSIS (B-31 PROFILE)

Unit	Depth, ft	Condition	LPILE Soil/Rock Type	Soil Properties: γ', pcf	c, psf	ε_{50}	ϕ', deg.	K, pci	Unit Side Resist., psf	Unit End Bearing, psf
Medium stiff SILT, up to some clay and sand (Alluvium)	0 to 5	Static and Seismic	Sand	110	N/A	N/A	32	50	-	-
Very soft to soft SILT, up to some clay and sand (Alluvium)	5 to 25	Static	Sand	50	N/A	N/A	28	20	300	-
		Seismic	Soft Clay	50	100	0.050	N/A	N/A	-	-
Loose silty SAND (Alluvium)	25 to 35	Static	Sand	50	N/A	N/A	32	20	800	-
		Seismic	Soft Clay	50	300	0.050	N/A	N/A	-	-
Medium dense silty SAND (Alluvium)	35 to 48	Static	Sand	55	N/A	N/A	34	60	1,500	-
		Seismic	Soft Clay	55	600	0.050	N/A	N/A	-	-
				Rock Properties: γ', pcf	**q_u, psi**	**E_{rm}, psi**	**RQD, %**	**k_{rm}**	**Unit Side Resist, psf**	**Unit End Bearing, psf**
Strong (R4), fresh BASALT	Below 48	Static and Seismic	Strong Rock	100	2,500	N/A	N/A	N/A	10,000	150,000

Notes:

1. The subsurface profile was interpreted based on the conditions disclosed in boring B-31 and CPT soundings CPT-1 and CPT-2.
2. Static groundwater with submerged conditions is assumed to be below a depth of 5 feet.
3. Unit side resistance and end-bearing for axial (compressive) resistance are provided as nominal or ultimate values. A typical factor of safety of 2.0 to 3.0 should be applied to these values for allowable axial resistances.
4. Side friction resistance within the upper 5 feet of the shaft should be neglected when evaluating axial (compressive) resistance.
5. No end-bearing resistance should be assumed for the soils above the basalt bedrock because of the potential for strength loss and settlement under seismic loading conditions. Side friction resistance in liquefiable soils is assumed to be negligible under seismic loading conditions.
6. Assumed strength properties for the basalt bedrock have been reduced based on typical limits applied to hard rock for software programs such as LPILE for evaluating lateral resistance and deflection of drilled shafts.

Table 5-3: SOIL AND ROCK PARAMETERS FOR DRILLED SHAFT ANALYSIS (B-32 PROFILE)

Unit	Depth, ft	Condition	LPILE Soil/Rock Type	Soil Properties γ', pcf	c, psf	ε_{50}	ϕ', deg.	K, pci	Unit Side Resist., psf	Unit End Bearing, psf
Stiff SILT, some clay (Portland Hills Silt)	0 to 3	Static and Seismic	Sand	110	N/A	N/A	32	50	-	-
Very dense ROCK FRAGMENTS (Decomposed Basalt)	3 to 6	Static and Seismic	Sand	75	N/A	N/A	42	125	1,500	-
				Rock Properties γ', pcf	**q_u, psi**	**E_{rm}, psi**	**RQD, %**	**k_{rm}**	**Unit Side Resist, psf**	**Unit End Bearing, psf**
Weak to very strong (R2 to R5), slightly weathered to fresh BASALT	Below 6	Static and Seismic	Strong Rock	100	2,500	N/A	N/A	N/A	10,000	150,000

Notes:

1. The subsurface profile is interpreted based on the conditions disclosed in boring B-32 and the seismic refraction profile.
2. Static groundwater with submerged conditions is assumed to be below a depth of 3 feet.
3. Unit side resistance and end-bearing for axial (compressive) resistance are provided as nominal or ultimate values. A typical factor of safety of 2.0 to 3.0 should be applied to these values for allowable axial resistances.
4. Side friction resistance within the upper 5 feet of the shaft should be neglected when evaluating axial (compressive) resistance.
5. No end-bearing resistance is assumed for the soils above the basalt bedrock based on the shallow depth to rock.
6. Assumed strength properties for the basalt bedrock have been reduced based on typical limits applied to hard rock for software programs such as LPILE for evaluating lateral resistance and deflection of drilled shafts.

Table 5-4: SOIL AND ROCK PARAMETERS FOR DRILLED SHAFT ANALYSIS (B-34 PROFILE)

Unit	Depth, ft	Condition	LPILE Soil/Rock Type	Soil Properties: γ', pcf	c, psf	ε_{50}	ϕ', deg.	K, pci	Unit Side Resist., psf	Unit End Bearing, psf
Very stiff to hard clayey SILT (Residual Soil/ Poss. Landslide Debris)	0 to 15	Static and Seismic	Stiff Clay w/o Free Water	120	4,000	0.005	N/A	N/A	1,000	-
Very stiff to hard clayey SILT (Residual Soil/ Poss. Landslide Debris)	15 to 30	Static and Seismic	Stiff Clay w/o Free Water	60	4,000	0.005	N/A	N/A	1,500	-
Very dense ROCK FRAGMENTS (Decomposed Basalt)	30 to 40	Static and Seismic	Sand	75	N/A	N/A	42	125	3,500	60,000
				Rock Properties: γ', pcf	**q_u, psi**	**E_{rm}, psi**	**RQD, %**	**k_{rm}**	**Unit Side Resist., psf**	**Unit End Bearing, psf**
Medium strong to strong (R3 to R4), moderately weathered to fresh BASALT	Below 40	Static and Seismic	Strong Rock	100	2,500	N/A	N/A	N/A	10,000	150,000

Notes:

1. The subsurface profile was interpreted based on the conditions disclosed in boring B-34.
2. Static groundwater with submerged conditions is assumed to be below a depth of 15 feet.
3. Unit side resistance and end-bearing for axial (compressive) resistance are provided as nominal or ultimate values. A typical factor of safety of 2.0 to 3.0 should be applied to these values for allowable axial resistances.
4. Side friction resistance within the upper 5 feet of the shaft should be neglected when evaluating axial (compressive) resistance.
5. Assumed strength properties for the basalt bedrock have been reduced based on typical limits applied to hard rock for software programs such as LPILE for evaluating lateral resistance and deflection of drilled shafts.

Table 5-5: SOIL AND ROCK PARAMETERS FOR DRILLED SHAFT ANALYSIS (B-35 PROFILE)

Unit	Depth, ft	Condition	LPILE Soil/Rock Type	Soil Properties: γ', pcf	c, psf	ε_{50}	ϕ', deg.	K, pci	Unit Side Resist., psf	Unit End Bearing, psf
Medium stiff to stiff CLAY to silty CLAY (Portland Hills Silt/ Poss. Landslide Debris)	0 to 15	Static and Seismic	Stiff Clay w/o Free Water	110	1,000	0.010	N/A	N/A	500	-
Stiff SILT, trace sand (Portland Hills Silt/ Poss. Landslide Debris)	15 to 20	Static and Seismic	Sand	50	N/A	N/A	34	60	1,000	-
Medium stiff SILT, some clay & sand (Residual Soil/ Poss. Landslide Debris)	20 to 30	Static and Seismic	Stiff Clay w/o Free Water	50	1,000	0.010	N/A	N/A	750	-
Very stiff to hard clayey SILT, some sand (Residual Soil/ Poss. Landslide Debris)	30 to 45	Static and Seismic	Stiff Clay w/o Free Water	60	3,000	0.005	N/A	N/A	1,500	30,000
Very dense ROCK FRAGMENTS (Decomposed Basalt)	45 to 60	Static and Seismic	Sand	75	N/A	N/A	42	125	3,500	60,000
				Rock Properties: γ', pcf	**q_u, psi**	**E_{rm}, psi**	**RQD, %**	**k_{rm}**	**Unit Side Resist, psf**	**Unit End Bearing, psf**
Medium strong (R3), decomposed to fresh BASALT	Below 60	Static and Seismic	Strong Rock	100	2,500	N/A	N/A	N/A	10,000	150,000

Notes:

1. The subsurface profile is interpreted based on the conditions disclosed in boring B-35, with supplemental information from hand auger explorations HA-1 and HA-2.
2. Static groundwater with submerged conditions is assumed to be below a depth of 15 feet.
3. Unit side resistance and end-bearing for axial (compressive) resistance are provided as nominal or ultimate values. A typical factor of safety of 2.0 to 3.0 should be applied to these values for allowable axial resistances.
4. Side friction resistance within the upper 5 feet of the shaft should be neglected when evaluating axial (compressive) resistance.
5. Assumed strength properties for the basalt bedrock have been reduced based on typical limits applied to hard rock for software programs such as LPILE for evaluating lateral resistance and deflection of drilled shafts.

5.4.4 Foundation Settlement

Vertical movement is required for mobilization of skin-friction resistance along the length of the foundation elements and end-bearing resistance at the base. Full mobilization of skin friction resistance typically develops with less than ½ inch of movement, while end-bearing resistance can require significantly greater settlement and is dependent on the size (e.g., diameter) of the foundation base and the subsurface materials at the foundation base.

Provided that a factor of safety of at least 2 is applied to the nominal (i.e., ultimate) end-bearing resistance, we anticipate limited movement will be required for mobilizing the required resistance. Furthermore, for pier foundations extending to rock, the movement required to mobilize the end-bearing resistances should be negligible. Therefore, we anticipate relatively small settlements (e.g., ½ inch or less) under static loading conditions or temporary loads such as the design wind loads. GRI should provide a more comprehensive settlement evaluation if the vertical loads are expected to approach the allowable end-bearing resistances calculated based on the values provided in Tables 5-2 through 5-5.

As noted above in Section 5.3.3, near the Harborton substation (i.e., poles SP2, SP7, and SP8), liquefaction-induced settlements associated with a design-level earthquake are expected to be on the order of 1 foot or greater. To mitigate liquefaction-induced settlements at these pole locations, the pier foundations need to be set below the liquefaction-prone soil layers. This corresponds to a minimum pier tip set at or below the top of rock. Rock in the current explorations was observed at depths of about 47 feet to 48 feet. However, the rock depth may be more variable, as noted from previous explorations that encountered rock at depths ranging from about 45 feet to 52 feet below surface grades.

5.4.5 Corrosion Considerations

We evaluated the corrosion potential of the near-surface soil at each of the boring locations by completing chloride, sulfate, oxidation-reduction potential, and soil resistivity testing. The results of the testing are summarized in Figure 18A. The conclusions and recommendations provided are based on the California Department of Transportation (Caltrans) Corrosion Guidelines, Version 3.2. A minimum resistivity value for soil and/or water less than 1,000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion. The samples tested had a minimum resistivity value of 2,485 ohm/cm or greater. For structural elements, a site is considered corrosive if the chloride concentration is 500 parts per million (ppm) or greater or sulfate concentration is 1500 ppm or greater. All samples tested had chloride concentrations less than 27 ppm and sulfate concentrations less than 30 ppm. Therefore, the test results suggest a low risk of corrosive soil conditions.

6 GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

6.1 General

As discussed above, the construction of the new transmission line poles and foundations will require granular work pads and improved haul roads within Forest Park. Work pads are also planned for servicing some of the existing lattice tower structures in the park. Review of the concept-level design indicated maximum cut and fill thicknesses of up to about 10 feet for constructing the work pad areas. The following sections provide a discussion of earthwork considerations associated with such efforts, as well as considerations for drilled shaft construction. GRI should review the final grading plan and site drainage details when they become available to confirm the plans are consistent with the recommendations provided in this report.

6.2 Site Preparation

The ground surface within all areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils or fill. All trees, brush, and surficial organic material should be removed from within the limits of the proposed fill areas. Excavations required to remove unsuitable soils, brush, and trees should be backfilled with structural fill. Organic strippings should be disposed of offsite or stockpiled on site for use in landscaped areas. Strippings should not be incorporated into structural fill materials.

Following stripping or excavation to design elevations, a qualified geotechnical engineer or an engineering geologist should evaluate the exposed subgrade. Proof rolling with a loaded dump truck may be part of this evaluation in larger work areas. Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm material and backfilled with structural fill. Due to the presence of moisture-sensitive, fine-grained soils near the ground surface, it should be anticipated that some overexcavation of the subgrade may be required.

6.3 Earthwork

Fine-grained soils mantle a significant portion of the proposed work pad areas. These soils are moisture sensitive. Therefore, in our opinion, earthwork can be completed most economically during the dry summer months, typically extending from June to mid-October. It has been our experience that the moisture content of the upper few feet of soils with a high-fines content will decrease during extended warm, dry weather. However, below this depth, the moisture content of the soil tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must use construction equipment and procedures that prevent disturbance and softening of the subgrade soils. To minimize disturbance of the moisture-sensitive soils, site work can be completed using track-mounted equipment. Excavations should be finished using a smooth-edged bucket to produce a firm, undisturbed surface. It may also be necessary to construct granular haul roads and work pads concurrently with excavation to minimize subgrade disturbance. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with structural fill.

We understand that, to the extent practical, cuts and fills will be balanced, and fill areas will utilize onsite materials to limit the need for significant importing or exporting. In general, it is our opinion that reusing onsite soils as structural fill in the work pad areas is feasible. However, as noted above, the practicality of using the onsite fine-grained soils will depend on the ability to properly moisture condition and compact the soils. Proper moisture conditioning will require construction to be completed during relatively dry weather (i.e., typically during summer months). The excavated soils will be variable depending on the location of the cuts. In no case should organic strippings, high-plasticity clays, overly wet and/or unsuitable soils, or other deleterious materials be placed as structural fill. Proposed materials to be used for structural fill should be observed by a GRI representative.

All fill placed as structural fill should be placed and compacted in relatively level lifts of no greater than 12 inches in thickness. Structural fill should be compacted to a minimum of 95% relative compaction based on the results of ASTM International (ASTM) D698. The moisture content of the fill should be adjusted to within about 2% of its optimum value prior to compaction. Field-density tests should be run frequently to confirm adequate compaction. Adequate compaction of fill materials that are too coarse or too variable for density testing should be evaluated by observation of the compaction method and proof rolling with a loaded dump truck or other approved heavy construction vehicle.

Where rock is present at shallow depths (e.g., work pad areas for poles SP3 and SP9), rock excavation techniques such as ripping, chipping, or controlled blasting may be required. Therefore, the contractor should have a thorough understanding of the conditions present and the anticipated depths to rock relative to the proposed finished grades. We anticipate that additional considerations for excavation techniques may be warranted in sensitive areas of Forest Park.

6.4 Temporary and Permanent Slopes and Embankments

We did not identify existing drainages or unstable slopes where proposed cuts and fills are planned that warrant specific considerations outside of what is discussed in this report. Final grading where work pads are planned should provide positive drainage of surface water away from adjacent properties and slopes to reduce the potential for erosion and ponding or future debris flows. The subgrade should be sloped to a minimum 0.5% slope to aid drainage.

Studies by the US Forest Service (e.g., Swanston, 1974) and others have shown that landslides within logged hillsides are more likely to occur where the slope gradient is steeper than 50% (i.e., greater than 2H:1V [Horizontal to Vertical]). Furthermore, erosion and/or shallow instabilities can occur as the existing root systems of felled trees rot or otherwise degrade over time. Therefore, in general, permanent cut and fill slopes should be no steeper than 2H:1V and protected with approved replanted vegetation to reduce the risk of surface erosion due to rainfall and loss of existing root systems. We understand some tree removal will occur in areas where existing natural slopes exceed 2H:1V. In these areas, we anticipate the risk of shallow instabilities can be mitigated with replanting of approved ground cover that will establish a root system as the existing tree roots degrade. Furthermore, we anticipate the site will be monitored after replanting to confirm the growth of the new vegetation. We note that the existing steep slopes along the existing transmission line corridor where tree cutting has occurred in the past within the project area do not appear to have ongoing or reoccurring instabilities. Therefore, it is our opinion that future shallow slope instability can be mitigated using the methods discussed above.

Where competent rock is encountered (e.g., work pad areas for poles SP3 and SP9), it will likely be feasible to excavate finished cut slopes within the rock at ½H:1V or steeper without risk of increased instability. However, final excavation and finished cuts in rock should be evaluated at the time of construction to confirm suitable cut slopes based on the exposed materials.

Temporary cuts no steeper than 1.5H:1V should be planned where fine-grained soils are exposed. If temporary cuts are open for a significant period of time or during wet weather, erosion control measures should be put in place, such as plastic sheeting, jute mats or netting, barriers, geosynthetics, or similar means. However, it should be understood that prolonged exposure of the temporary cuts increases the risk of erosion and slumps or instabilities to occur.

Finished embankment fill slopes should be properly keyed and benched into existing slopes. Benching should be completed where existing grades are steeper than 5H:1V, and new fill should be placed in level lifts to provide positive bond with the existing ground. A typical detail for fill placement and embankment construction modified from the Oregon Department of Transportation (ODOT) Detail No. DET2100 is provided on Standard Embankment Detail, Figure 4.

6.5 Foundation Construction

The design criteria presented above assume that drilled piers supporting the proposed transmission line poles will be installed in accordance with Section 00512 of the current Oregon Department of Transportation *Standard Specifications for Construction* (ODOT SSC). These specifications include selection criteria for the drilled shaft contractor.

In some locations, particularly downslope near the Harborton substation, excavations for the drilled piers could extend below the static groundwater level or below zones of perched groundwater, and there is a risk that caving and running overburden soils may be encountered during foundation excavation. Therefore, the use of temporary casing meeting the requirements of ODOT SSC Section 00512.43 should be specified, as needed, to reduce the risk of caving conditions that will affect the installation of the foundations. Drilling slurry meeting the requirements of ODOT SSC Section 00512.43 may also be considered for drilled piers in lieu of casing. If temporary casing is used, excavation in advance of the casing tip should not exceed 5 feet, and hydrostatic pressure inside and outside the casing should be consistent throughout the excavation. The pier excavation should be cleaned, the reinforcing cage set (if applicable), and the concrete placed in as short a time sequence as possible, and preferably on the same day. The concrete should be placed using tremie methods, beginning at the base of the shaft and maintaining the concrete at least 5 feet above the outlet of the tremie pipe. Temporary casing should be removed as the concrete is placed, and permanent casing should not be allowed.

For pier excavation where rock is shallow, particularly at proposed poles SP3 and SP9, the contractor should clearly understand the anticipated rock hardness and equipment necessary for drilling and/or excavating the rock. As previously noted, laboratory test results indicated an unconfined compressive strength of greater than 23,000 psi at relatively shallow depth near the location of proposed poles SP3 and SP9 (see B-32 boring log on Figure 2A and laboratory UCS results on Figures 15A through 17A). Excavation into massive rock with rock strengths this high typically requires rock coring bits, down hole hammers, or other specialty tooling in addition to augers or similar equipment.

7 DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. To observe compliance with the intent of our recommendations, the design concepts, and the plans and specifications, it is our opinion that all construction operations dealing with earthwork and foundation installations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions different from those described in this report.

8 LIMITATIONS

This report has been prepared to aid the project team in the design of the Harborton 230kV Transmission Line improvements. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to earthwork and design and construction of the transmission line structures and associated work pad areas. In the event any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations in this report are based on the data obtained from the subsurface explorations at the locations shown on Figure 2 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged variations in subsurface conditions may exist between exploration locations. This report does not reflect variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

We have included the Geoprofessional Business Association (GBA) guidance document "Important Information about This Geotechnical-Engineering Report/Geoenvironmental Report" to assist you and others in understanding the use and limitations of this report, included as Appendix E. We recommend you read this document.

Submitted for GRI,

RENEWS: 06/2025

RENEWS: 02/2025

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This document has been submitted electronically.

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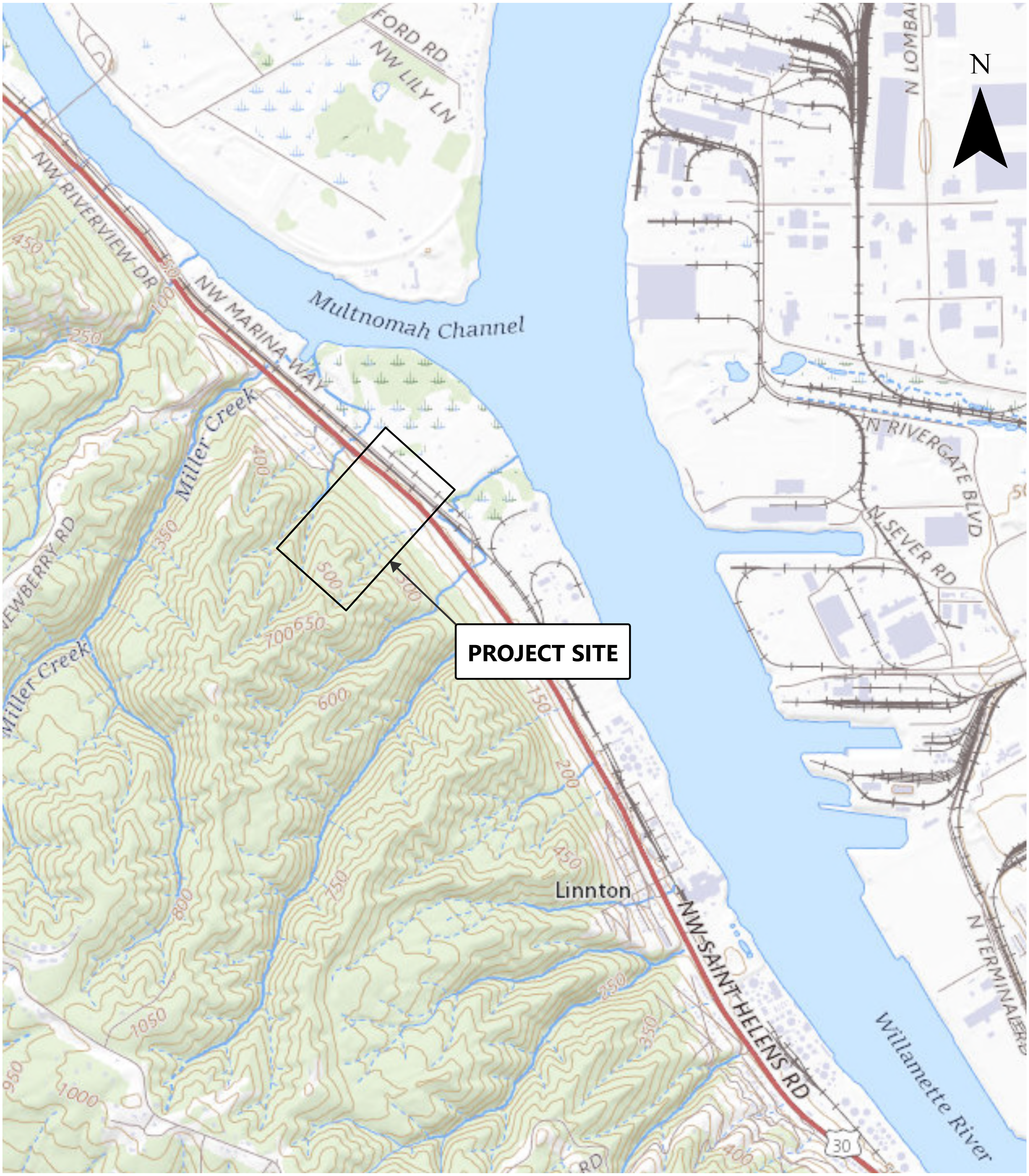
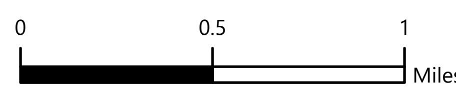


BASEMAP PROVIDED BY USGS NATIONAL MAP ONLINE LAYER

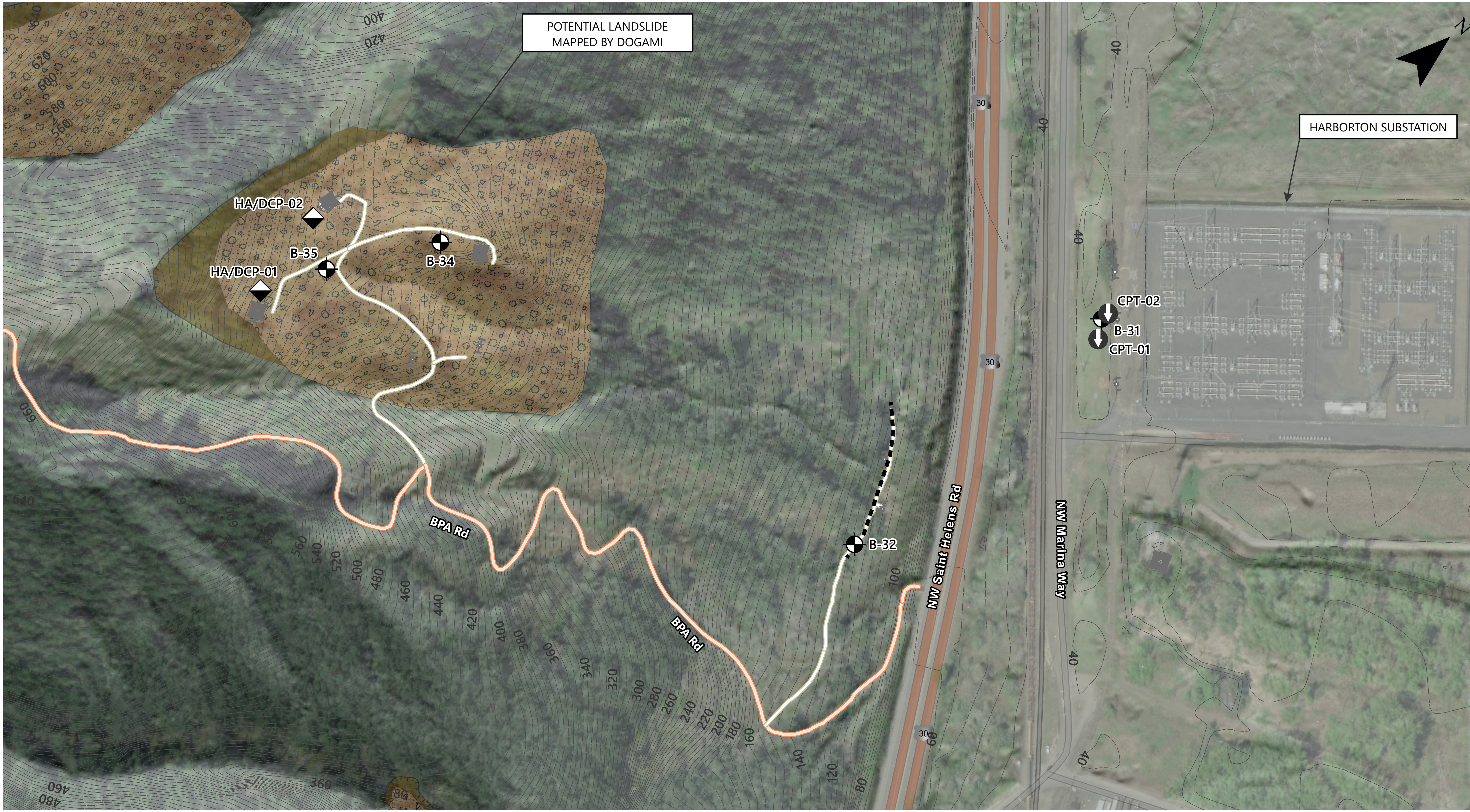


GRI

PORTLAND GENERAL ELECTRIC
HARBORTON 230-kV TRANSMISSION LINE

VICINITY MAP

JUN. 2024 | JOB NO. 6767-B | FIG. 1



APPROXIMATE LOCATION OF BORING

APPROXIMATE LOCATION OF DCP/HAND AUGER

APPROXIMATE LOCATION OF CPT

APPROXIMATE LOCATION OF GEOPHYSICAL LINE

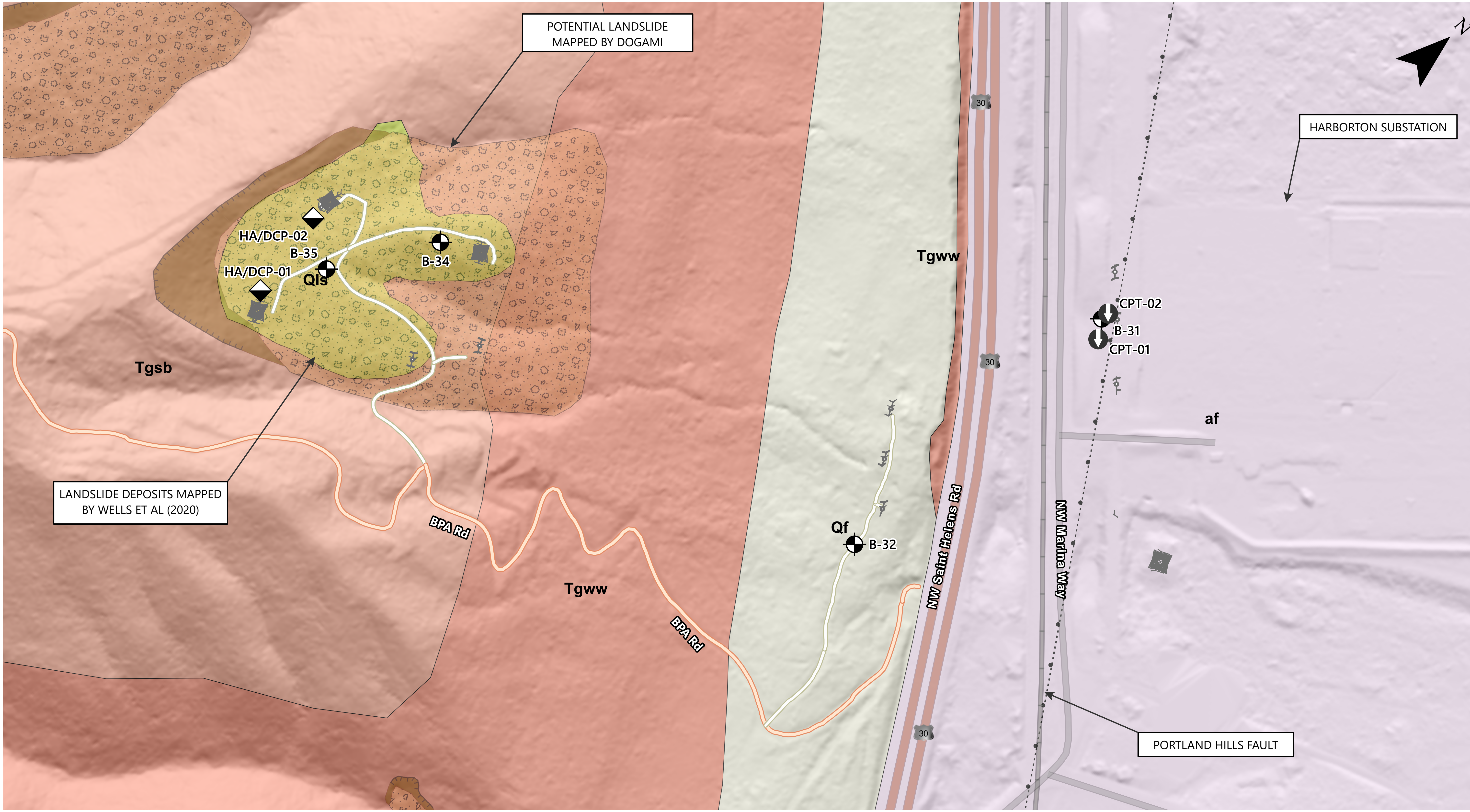
1. BASEMAP PROVIDED BY ESRI AND DOGAMI
2. CONTOURS PROVIDED BY METRO RLIS
3. LANDSLIDE MAPPING PROVIDED BY DOGAMI SLIDO 4 DATABASE
4. ACCESS ROADS AND TRANSMISSION STRUCTURE LOCATIONS PROVIDED BY PGE IN KMZ FILES ON DEC. 12, 2022.
5. EXPLORATION LOCATIONS MEASURED IN THE FIELD WITH HANDHELD OR MOBILE PHONE GPS AND ARE APPROXIMATE.

0 200 400 Feet

GRI PORTLAND GENERAL ELECTRIC
HARBORTON 230-kV TRANSMISSION LINE

SITE PLAN

JUN. 2024 JOB NO. 6767-B FIG. 2



APPROXIMATE LOCATION OF BORING
APPROXIMATE LOCATION OF DCP/HAND AUGER
APPROXIMATE LOCATION OF CPT
Scarp
Head Scarp
Talus-Colluvium

Fan
Landslide
af - Artificial Fill
Qf - Missoula Flood Deposits
Qls - Landslide Deposits
Tgsb - CRBG Sentinel Bluffs Member
Tgww - CRBG Winter Water Member

1. BASEMAP PROVIDED BY ESRI AND DOGAMI
2. GEOLOGY PROVIDED BY WELLS ET AL (2020)
3. LANDSLIDE MAPPING PROVIDED BY DOGAMI SLIDO 4 DATABASE
4. ACCESS ROADS AND TRANSMISSION STRUCTURE LOCATIONS PROVIDED BY PGE IN KMZ FILES ON DEC. 12, 2022.
5. EXPLORATION LOCATIONS MEASURED IN THE FIELD WITH HANDHELD OR MOBILE PHONE GPS AND ARE APPROXIMATE.

0 200 400 Feet

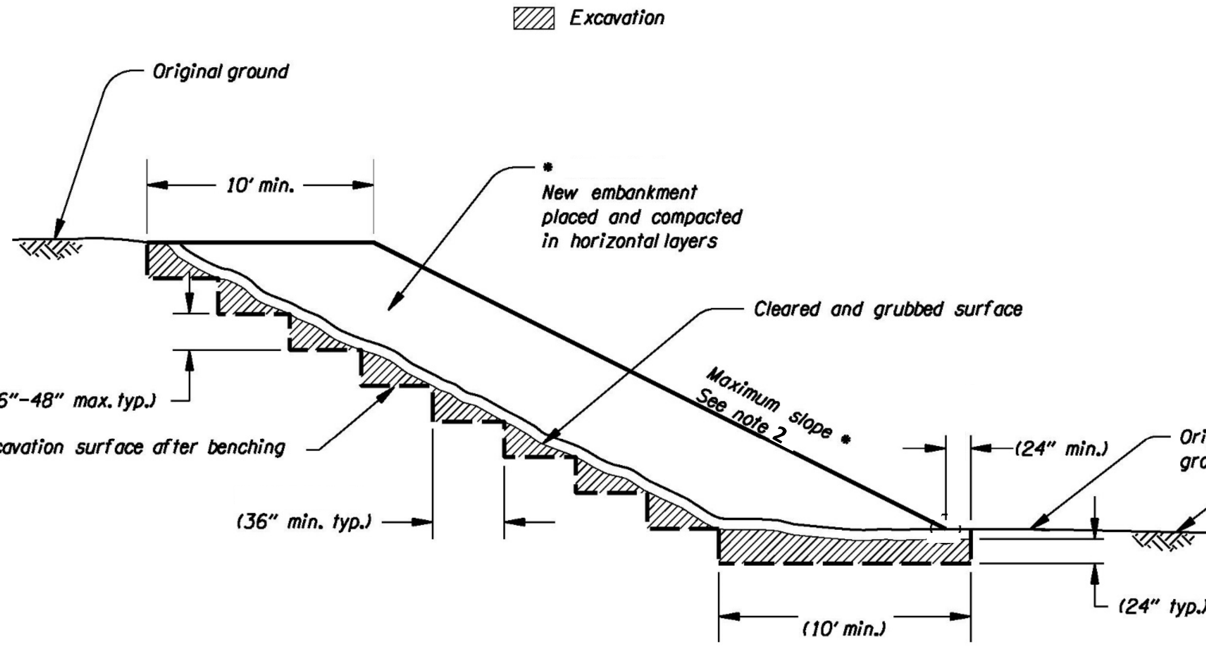
GRI PORTLAND GENERAL ELECTRIC
HARBORTON 230-kV TRANSMISSION LINE

GEOLOGIC MAP

JUN. 2024 JOB NO. 6767-B FIG. 3

STANDARD EMBANKMENT CONSTRUCTION

(Not to scale: Diagrammatic only)



Standard embankment general notes:

1. *Construct benches on slopes steeper than 1:5 (v:h) to provide positive bond with existing ground.*
2. *Benching work is incidental to embankment construction.*

NOTES:

1. FIGURE WAS ADAPTED FROM ODOT STANDARD EMBANKMENT DETAIL DET2100.
2. MAXIMUM FINISHED SLOPE OF 2H:1V SHOULD BE PLANNED WHEN USING FILL GENERATED FROM ONSITE EXCAVATIONS.
3. SEE REPORT FOR DISCUSSION OF SUBSURFACE CONDITIONS AND FURTHER RECOMMENDATIONS FOR FILL PLACEMENT AND EMBANKMENT CONSTRUCTION.

GRI

PORTLAND GENERAL ELECTRIC
HARBORTON 230-kV TRANSMISSION LINE

STANDARD EMBANKMENT DETAIL

JUN. 2024 | JOB NO. 6767-B | FIG. 4

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

A.1.1 General

Subsurface materials and conditions at the site were investigated between September 11 and September 26, 2023, with four machine-drilled borings, designated B-31, B-32, B-34, and B-35; two hand-auger borings, designated HA-1 and HA-2; two dynamic cone penetration (DCP) tests, designated DCP-1 and DCP-2; two cone penetrometer test (CPT) probes, designated CPT-1 and CPT-2; and a seismic refraction geophysical survey. The approximate locations of the explorations completed for this investigation are shown on the Site Plan, Figure 2. The field-exploration work was coordinated and documented by experienced members of GRI's geotechnical engineering staff.

A.1.2 Machine-Drilled Borings

Borings B-31, B-32, B-34, and B-35 were completed using a track-mounted CME 850 drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. Mud-rotary, open-hole drilling, and triple-barrel wire-line coring techniques were used to advance the borings and collect representative soil and rock samples. The drilled borings were advanced to depths of ranging from about 15.1 feet to 67.8 feet below existing site grades.

Disturbed and undisturbed soil samples were obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet to 20 feet and at 5-foot intervals below this depth. Disturbed soil samples were obtained using a standard split-spoon sampler. The standard penetration test (SPT) was completed while obtaining disturbed soil samples. This test is performed by driving a 2-inch-outside-diameter, split-spoon sampler into the soil a distance of 18 inches using the force of a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is known as the Standard Penetration Resistance, or SPT N-value. The SPT N-values provide a measure of the relative density of granular soils and the relative stiffness or consistency of cohesive soils. Samples obtained from the borings were placed in airtight plastic bags and returned to our laboratory for further classification and testing. In addition, relatively undisturbed samples were collected by pushing a 3-inch-outside-diameter Shelby tube into the undisturbed soil at a maximum distance of 24 inches using the hydraulic ram of the drill rig. The soil exposed at the end of the Shelby tube was examined and classified in the field. After classification, the tubes were sealed with rubber caps and returned to our laboratory for further examination and testing.

A continuous rock core was obtained in each of the borings after encountering relatively competent bedrock. Coring was completed using a triple-barrel HQ coring system. Rock coring was generally completed within the bottom 8 feet to 12 feet of the borings. The rock core was obtained with core runs extending up to 5 feet long. Shorter core runs were necessary for some instances at the discretion of the driller due to the rock core blocking off during drilling. The recovery and rock quality designation (RQD) values were recorded for each core run. RQD is determined by adding the length of all intact rock core lengths greater than 4 inches long and dividing that value by the total length of the core run. The relative rock hardness noted on the logs was estimated from visual inspection of the retained rock core and from laboratory testing.

The boring explorations were coordinated and documented by an experienced member of GRI's geotechnical engineering staff, who maintained a log of the materials and conditions disclosed during the course of work. Logs of the borings are provided on Figures 1A through 4A. Each log presents a summary of the various types of materials encountered in the boring and notes the depths at which the materials and/or characteristics of the materials change. To the right of the summary, the numbers and types of samples are indicated. Farther to the right, SPT N-values are shown graphically, along with the natural moisture contents, Atterberg limits, and percent passing the No. 200 sieve, where applicable. The terms and symbols used to describe the materials encountered in the borings are defined in Tables 1A and 2A and the attached legend.

A.1.3 Hand-Auger Borings

Two hand-auger borings, designated HA-1 and HA-2, were advanced to depths of about 12 feet and 9 feet, respectively, using a hand-operated auger. The borings were completed by an experienced member of GRI's geotechnical engineering staff, who maintained a log of the materials and conditions disclosed during the course of the work. Disturbed samples were obtained from the borings at selected depths. The samples were placed in plastic bags and returned to our laboratory for further classification and testing.

Logs of the borings are provided on Figure 5A. The logs present a summary of the various types of materials encountered in the borings and note the depth at which the materials and/or characteristics of the materials change. To the right of the summary, the numbers and types of samples taken during the drilling operation are indicated. Further to the right, moisture contents are shown graphically. The terms and symbols used to describe the materials encountered in the borings are defined in Table 1A and the attached legend.

A.1.4 Dynamic Cone Penetration Testing

Dynamic cone penetration (DCP) tests, DCP-1 and DCP-2 were completed in conjunction with the hand-auger borings and advanced to depths of about 19 feet and 16 feet, respectively. The DCP tests were completed using a Wildcat cone penetrometer manufactured by Triggs Technologies, Inc. The Wildcat DCP test consists of driving a 1.4-inch-diameter cone with a 35-pound weight falling 15 inches. The number of blows required to drive the cone 10 centimeters (approximately 4 inches) is recorded to assess the density or stiffness characteristics of the underlying soils. The DCP results are summarized on Figure 6A, which shows the blows required to drive the probe in 10-centimeter increments.

A.1.5 Cone Penetration Test Probes

Cone penetrometer test (CPT) probes CPT-1 and CPT-2 were advanced near the location of boring B-31 adjacent to Harborton substation. The CPT probes were advanced using a truck-mounted CPT rig provided and operated by ConeTec, Inc. of Seattle, Washington. CPT-1 and CPT-2 were advanced to depths of approximately 47 feet and 47.4 feet, respectively, before reaching practical refusal (i.e., the cone apparatus could not be pushed further without incurring possible damage to the equipment).

During a CPT, a steel cone is forced vertically into the soil at a constant rate of penetration. The force required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This force is measured and recorded every 2 inches. In addition to the cone measurements, measurements are obtained of the magnitude of force required to force a friction sleeve attached above the cone through the soil. The force required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point-bearing capacity provides an indicator of the type of soil penetrated. The cone penetration resistance and sleeve friction can be used to evaluate the relative consistency of cohesionless and cohesive soils, respectively. In addition, a piezometer fitted between the cone and the sleeve measures changes in water pressure as the probe is advanced and can also be used to estimate the groundwater depth. The probe is also operated using an accelerometer fitted to the probe, which allows measurement of the arrival time of shear waves from impulses generated at the ground surface and calculation of shear-wave velocities for the surrounding soil profile. Shear wave velocity testing was completed for CPT-1.

Logs of the CPT probes are provided on Figures 7A and 8A, which present a graphical summary of the tip resistance, local (sleeve) friction, friction ratio, pore pressure, and soil behavior type index. The terms used to describe the soils encountered in the CPT probes are defined in Table 3A. The full report prepared by ConeTec, Inc. is provided as Appendix B of this report and includes additional information such as shear wave velocity testing and pore pressure dissipation at selected depths.

A.1.6 Seismic Refraction

A seismic refraction geophysical study was completed to supplement the data from boring B-32 near the proposed transmission line poles SP3 and SP9 and the associated work pad areas. The study was completed by Earth Dynamics, LLC, of Portland, Oregon. Their report is included as Appendix C.

A seismic refraction exploration consists of measuring the time required for a seismic wave to travel from a seismic source to a receiver transducer. The seismic source is typically a large weight (e.g., a sledgehammer) that is dropped, and vertical geophones are used as receiving transducers. A seismograph records signals from the geophones, which are then analyzed by a geophysical expert to evaluate soil and/or rock units and the depth to geologic contacts based on the arrival time of the seismic waves as a function of the seismic source. The primary focus of this study was to help evaluate rock depths near boring B-32 and extend north where access with other exploration equipment was not feasible.

A.2 INSTRUMENTATION

Borings B-34 and B-35 included installation of a vibrating wire piezometer (VWP) with datalogger and slope inclinometer casing for future measurement of groundwater conditions and lateral movement at the borehole locations, as described below.

A.2.1 Vibrating-Wire Piezometers

Geokon Model 4500S VWPs were installed to a depth of approximately 43.9 feet in boring B-34 and 54 feet in boring B-35. At the time of installation, the piezometers were saturated with water, field calibrated, taped to the inclinometer casing (discussed below) in an inverted position to maintain saturation, and inserted into the open borehole to the desired depth. The borings were then filled with cement-bentonite grout to near the ground surface. Geokon Model 8940 dataloggers were connected to the VWPs to automatically collect and store measurements. The installations are equipped with 10-inch-diameter steel monuments cement-grouted into the borehole collar to protect the datalogger and readout cable. The VWPs will continue to be monitored through the remainder of the project design phase and reported to the project team.

A.2.2 Slope Inclinometer Casings

Inclinometer casings were installed in borings B-34 and B-35 to permit measurement of below-ground lateral movement. An inclinometer casing consists of 2.75-inch-outside-diameter ABS plastic pipe with orthogonal grooves. The vertical orientation of the casing is monitored by lowering a Durham Geo Slope Indicator Digitilt AT electronic measuring probe to the bottom of the grooved casing and obtaining readings at about 2-foot intervals as the instrument is withdrawn. The initial set of readings serves as a "benchmark" and is commonly portrayed as the vertical axis on a plot of depth versus casing deflection. All subsequent readings are referenced to the initial readings. By comparing relative movements at fixed depths over the length of the casing, zones of movement can be identified. The total, or cumulative, displacement with respect to the base of the casing is obtained by summing the relative displacements from the bottom to top. As indicated above, the installations were equipped with a 10-inch-diameter steel monument casing cement-grouted into the borehole collar for protection. Uncorrected, incremental measurements from the inclinometer readings are shown on Figures 9A and 10A. Additional readings will be obtained through the remainder of the project design phase and reported to the project team.

A.3 LABORATORY TESTING

A.3.1 General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included Torvane shear strength, dry unit weight measurements, Atterberg limits determinations, and grain-size analyses. A summary of the laboratory test results is provided in Table 4A. The following sections describe the testing program in more detail.

A.3.2 Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM International (ASTM) D2216. The results are summarized on Figures 1A through 6A and in Table 4A.

A.3.3 Grain-Size Analysis

A.3.3.1Washed-Sieve Method

To assist in classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve was oven-dried and weighed. The percentage of material passing the No. 200 sieve was then calculated. The results are summarized on Figures 1A through 4A, and in Table 4A.

A.3.4 Torvane Shear Strength

The approximate undrained shear strength of fine-grained soils retained in selected Shelby tube samples from boring B-31 was estimated using a Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear-strength tests are summarized on Figure 1A.

A.3.5 Undisturbed Unit Weight

The unit weight, or density, of undisturbed soil samples, was determined in the laboratory in conformance with ASTM D2937 on selected Shelby tube samples from boring B-31. The results are summarized on Figure 1A and in Table 4A.

A.3.6 Atterberg Limits

Atterberg limits testing was performed on samples of soil in conformance with ASTM D4318. The test results are summarized on the Plasticity Charts Figures 11A and 12A, Figures 1A through 4A, where applicable, and in Table 4A.

A.3.7 One-Dimensional Consolidation

One-dimensional consolidation testing was performed in accordance with ASTM D2435 on relatively undisturbed soil samples obtained from boring B-31 at depths of about 9.2 feet and 25.7 feet. The test provides data on the compressibility of underlying fine-grained soils. Test results are summarized on Figures 13A and 14A in the form of a curve showing effective stress versus percent strain. The initial dry unit weight and moisture content of the samples are also shown on the figures.

A.3.8 Uniaxial Compressive Strength of Intact Rock Cores

Samples of intact rock core specimens were delivered to the laboratory of FEI Testing & Inspection, Inc. of Corvallis, Oregon, for unconfined compressive strength testing in conformance with ASTM D7012 (Methods C and D). The test results are tabulated below and the stress-strain plots on Figures 15A through 17A. A summary is also provided on Figures 1A and 2A.

Boring (Run)	Depth, feet	Uniaxial Compressive Strength, psi	Stress-Strain Plot
B-31 (Run 3)	56.0 – 57.5	11,019	Figure 15A
B-32 (Run 1)	7.2 – 8.2	23,334	Figure 16A
B-32 (Run 2)	13.5 – 14.6	9,524	Figure 17A

A.3.9 Soil Corrosivity

Soil corrosivity testing was completed by Cooper Testing Labs, Inc. of Palo Alto, California. The testing was completed on three relatively shallow samples obtained from borings B-31, B-34, and B-35. Testing was not completed on samples from boring B-32 because of the shallow depth to rock and limited soil sampling. The corrosivity testing suite included Resistivity (100% saturated) testing in accordance with ASTM G57, chloride testing in accordance with ASTM D4327, sulfate testing in accordance with ASTM D4327, sulfide testing using lead acetate paper, and redox potential/ORP in accordance with ASTM G200. Test results are summarized on Figure 18A.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows/ft	California-Modified Penetration Resistance (SPT N*-values), blows/ft
Very Loose	0 - 4	0 – 11
Loose	4 - 10	11 – 26
Medium Dense	10 - 30	26 – 74
Dense	30 - 50	74 – 120
Very Dense	over 50	Over 120

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows/ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification

Boulders:
- >12 in.

Cobbles:
- 3-12 in.

Gravel:
- ¼ - ¾ in. (fine)
- ¾ - 3 in. (coarse)

Sand:
- No. 200 - No. 40 sieve (fine)
- No. 40 - No. 10 sieve (medium)
- No. 10 - No. 4 sieve (coarse)

Silt/Clay:
- Pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (By Weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Table 2A
GUIDELINES FOR CLASSIFICATION OF ROCK

Relative Rock Weathering Scale

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

Relative Rock Hardness Scale

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD and Rock Quality

Relation of RQD and Rock Quality	
RQD (Rock Quality Designation), %	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Terminology for Planar Surface		
Bedding	Joints and Fractures	Spacing
Laminated	Very Close	< 2 in.
Thin	Close	2 in. – 12 in.
Medium	Moderately Close	12 in. – 36 in.
Thick	Wide	36 in. – 10 ft
Massive	Very Wide	> 10 ft

Table 3A

CONE PENETRATION TEST (CPT) CORRELATIONS

Cohesive Soils

Cone Tip Resistance, tsf	Consistency
<5	Very Soft
5 to 15	Soft to Medium Stiff
15 to 30	Stiff
30 to 60	Very Stiff
>60	Hard

Cohesionless Soils

Cone Tip Resistance, tsf	Relative Density
<20	Very Loose
20 to 40	Loose
40 to 120	Medium
120 to 200	Dense
>200	Very Dense

Reference

Kulhawy, F. H., and Mayne, P. W., 1990, Manual on Estimating Soil Properties for Foundation Design, Electric Power Research Institute, EL-6800.

Table 4A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-31	S-1	2.5	--	24	--	--	--	--	SILT
	S-2	5.0	--	38	--	--	--	--	SILT
		8.0	--	39	--	--	--	86	SILT
	S-3	8.5	--	40	81	41	5	--	SILT
		9.0	--	39	--	--	--	56	SILT
	S-4	9.5	--	42	--	--	--	66	SILT
	S-6	14.5	--	47	--	--	--	--	SILT
	S-7	20.0	--	43	--	--	--	--	SILT
	S-8	25.0	--	41	--	--	--	61	Silty SAND
		26.0	--	42	79	--	--	--	Silty SAND
		26.5	--	34	--	--	--	53	Silty SAND
	S-9	27.0	--	40	--	--	--	30	Silty SAND
	S-10	30.0	--	39	--	--	--	--	Silty SAND
	S-11	35.0	--	37	--	--	--	--	Silty SAND
	S-12	40.0	--	42	--	--	--	25	Silty SAND
	S-13	45.0	--	33	--	--	--	--	Silty SAND
B-32	S-1	0.5	--	10	--	--	--	--	SILT
B-34	S-1	2.5	--	44	--	--	--	98	Clayey SILT
	S-2	5.0	--	48	--	--	--	--	Clayey SILT
	S-3	7.5	--	45	--	--	--	--	Clayey SILT
	S-4	10.0	--	49	--	52	11	--	Clayey SILT
	S-5	12.5	--	44	--	--	--	--	Clayey SILT
	S-6	15.0	--	44	--	--	--	--	Clayey SILT
	S-7	20.0	--	52	--	--	--	--	Clayey SILT
	S-8	25.0	--	56	--	--	--	--	Clayey SILT
B-35	S-1	2.5	--	28	--	--	--	--	CLAY
	S-2	5.0	--	31	--	41	18	--	Silty CLAY
	S-4	9.5	--	31	--	--	--	--	Silty CLAY
	S-6	14.5	--	26	--	--	--	--	Silty CLAY
	S-7	17.5	--	30	--	28	3	--	SILT
	S-8	20.0	--	32	--	--	--	75	SILT
	S-9	25.0	--	30	--	--	--	--	SILT
	S-10	30.0	--	63	--	73	23	--	Clayey SILT
	S-11	35.0	--	58	--	--	--	--	Clayey SILT
	S-12	40.0	--	60	--	63	13	--	Clayey SILT
	S-13A	45.0	--	45	--	--	--	--	Clayey SILT
	S-13-B	45.5	--	42	--	--	--	--	BASALT
	S-15	55.0	--	27	--	--	--	--	BASALT
HA-1	S-1	0.5	--	17	--	--	--	--	SILT
	S-2	2.0	--	24	--	--	--	--	SILT

Table 4A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
HA-1	S-3	5.0	--	28	--	--	--	--	SILT
	S-4	6.0	--	29	--	--	--	--	SILT
	S-5	11.0	--	31	--	--	--	--	SILT
	S-6	11.5	--	31	--	--	--	--	SILT
HA-2	S-1	0.5	--	21	--	--	--	--	SILT
	S-2	1.0	--	21	--	--	--	--	SILT
	S-3	3.0	--	26	--	--	--	--	SILT
	S-4	4.8	--	30	--	--	--	--	SILT
	S-5	8.5	--	31	--	--	--	--	SILT

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
	LANDSCAPE MATERIALS
	FILL
	GRAVEL; clean to some silt, clay, and sand
	Sandy GRAVEL; clean to some silt and clay
	Silty GRAVEL; up to some clay and sand
	Clayey GRAVEL; up to some silt and sand
	SAND; clean to some silt, clay, and gravel
	Gravelly SAND; clean to some silt and clay
	Silty SAND; up to some clay and gravel
	Clayey SAND; up to some silt and gravel
	SILT; up to some clay, sand, and gravel
	Gravelly SILT; up to some clay and sand
	Sandy SILT; up to some clay and gravel
	Clayey SILT; up to some sand and gravel
	CLAY; up to some silt, sand, and gravel
	Gravelly CLAY; up to some silt and sand
	Sandy CLAY; up to some silt and gravel
	Silty CLAY; up to some sand and gravel
	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	MUDSTONE
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
	Asphalt concrete PAVEMENT
	Portland cement concrete PAVEMENT
	Crushed rock BASE COURSE

SAMPLER SYMBOLS

Symbol	Sampler Description
	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Push probe sample interval

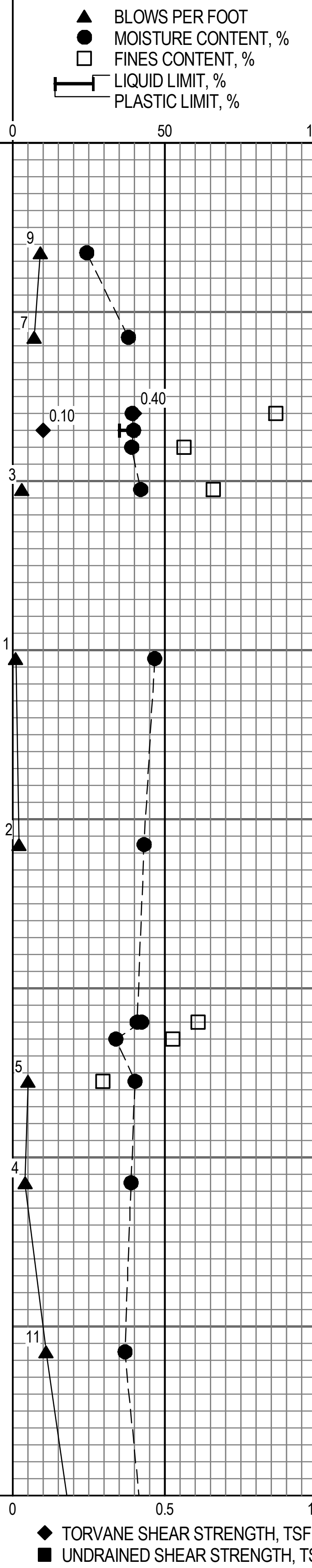
INSTALLATION SYMBOLS

Symbol	Symbol Description
	Flush-mount monument set in concrete
	Concrete, well casing shown where applicable
	Bentonite seal, well casing shown if applicable
	Filter pack, machine-slotted well casing shown where applicable
	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
	1-in.-diameter solid PVC
	1-in.-diameter hand-slotted PVC
	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
	Groundwater level during drilling and date measured
	Groundwater level after drilling and date measured
	Rock/sonic core or push probe recovery (%)
	Rock quality designation (RQD, %)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: Not Available						
		TOPSOIL						
		SILT, up to some clay and sand, ML, brown mottled black and iron stained, low plasticity, medium stiff to stiff, fine-grained sand (Alluvium)			S-1		6 5 4	
5					S-2		3 3 4	
					S-3			Dry Density = 81 pcf
10		---very soft to soft below 9.5 feet			S-4		1 2 1	
					S-5			
15		---some clay, trace fine-grained sand, brown below 14.5 feet			S-6		0 0 1	
20					S-7		1 1 1	
25		Silty SAND, SM, brown, very loose to loose, nonplastic fines, fine-grained sand (Alluvium)	25.0		S-8			Dry Density = 79 pcf
					S-9		1 2 3	
30					S-10		1 2 2	
35		---medium dense below 35 feet			S-11		2 5 6	
40								



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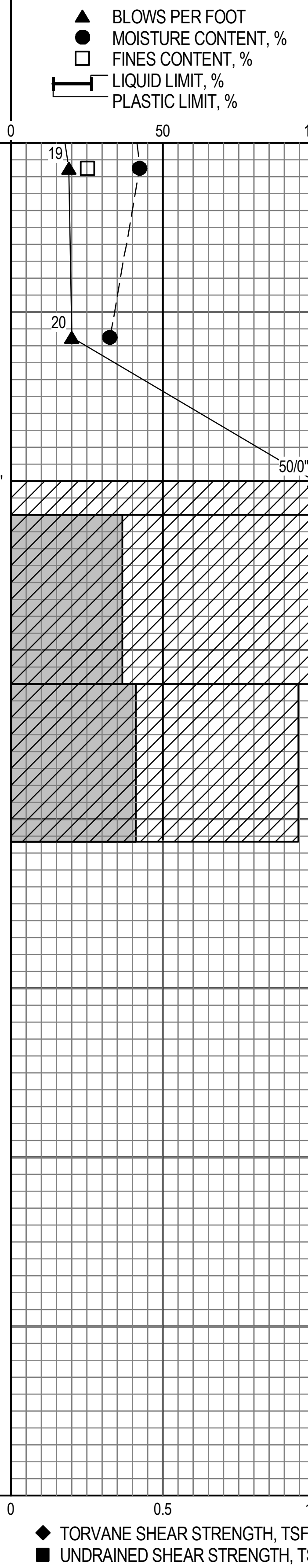
Logged By: M. Preciado	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/18/23	**GPS Coordinates:** 45.613435° N -122.79818° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.772
Note: See Legend for Explanation of Symbols	

GRI

BORING B-31

JUN. 2024 JOB NO. 6767-B FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: Not Available						
		Silty SAND, SM, brown to red, nonplastic fines, medium dense, fine-grained sand (Alluvium)			S-12		4 9 10	
45					S-13		7 8 12	
		BASALT, nonvesicular to some vesicles, gray, fresh, strong (R4), closely jointed at 5° to 60°, joints are open with blue mineralization on surfaces, scattered hairline and healed fractures (Columbia River Basalt)	48.0					Drill chatter below 48 feet
50					S-14		50/0"	
					R-1			
					R-2			
55		---joints at 0° to 90° below 56 feet						
					R-3			UCS = ±11,000psi
60		(9/18/2023)	60.6					
65								
70								
75								
80								



GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 11/29/23

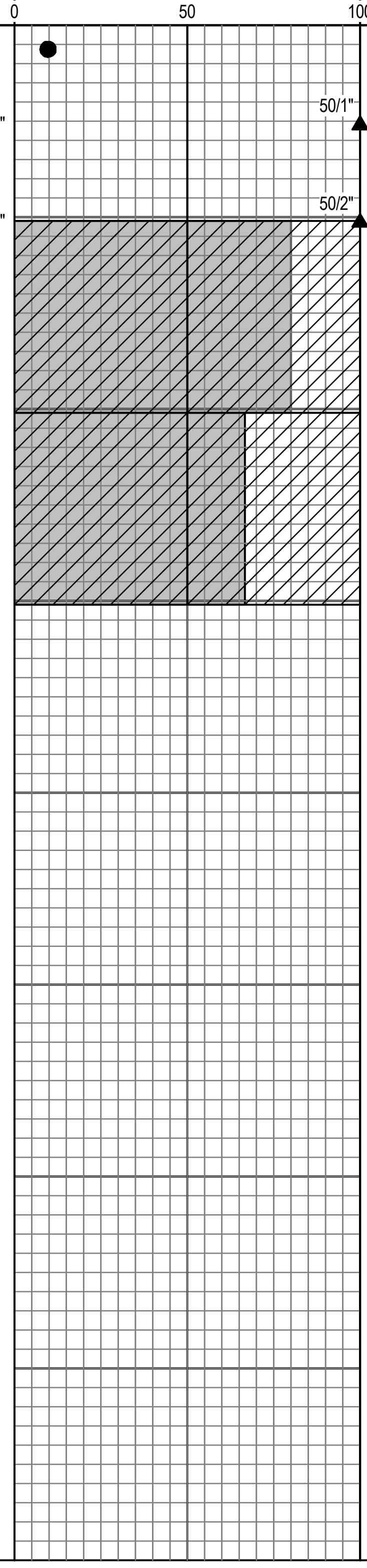
GRI

BORING B-31

JUN. 2024 JOB NO. 6767-B FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		TOPSOIL			S-1			
		SILT, some clay, trace sand, ML, brown mottled rust, low plasticity, stiff, fine-grained sand (Portland Hills Silt)	3.0		S-2		50/1"	
5		Decomposed BASALT *remolds to* ROCK FRAGMENTS and SAND, trace silt and clay, SP, gray mottled rust, very dense, medium- to coarse-grained sand, gravel-sized angular rock fragments (Decomposed Columbia River Basalt)	5.0		S-3		50/2"	
10		BASALT, trace vesicles to nonvesicular, gray, fresh to slightly weathered, weak to very strong (R2-R5), close to moderately close joints at 15° to 90°, joints are open with iron oxide staining, some joints contain secondary mineralization (Columbia River Basalt) ---vugs present below 8.2 feet			R-1			UCS = ±23,300psi
15			15.1		R-2			UCS = ±9,500psi
		(9/11/2023)						
20								
25								
30								
35								
40								

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
□ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %



◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

Logged By: M. Preciado	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/11/23	**GPS Coordinates:** 45.611482° N -122.798083° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.772
Note: See Legend for Explanation of Symbols	

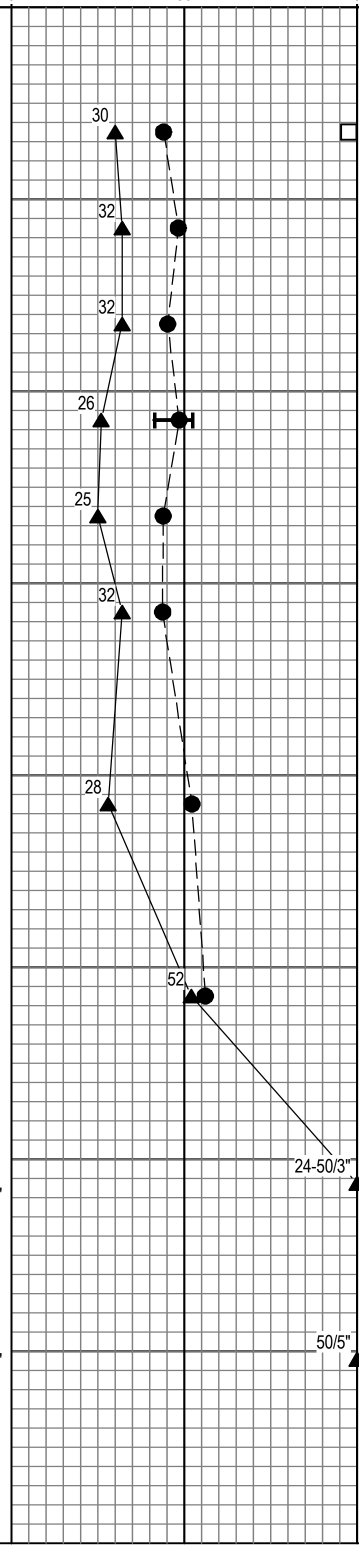
GRI

BORING B-32

JUN. 2024 JOB NO. 6767-B FIG. 2A

GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 11/29/23

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % 0 50 100	COMMENTS AND ADDITIONAL TESTS
		TOPSOIL							2.75-inch-OD inclinometer casing installed to 51 feet
		Clayey SILT, trace sand and gravel, MH, brown mottled red, black, and yellow, low to medium plasticity, very stiff to hard, fine- to coarse-grained sand (Residual Soil/Possible Landslide Debris)			S-1		7 14 16		
5					S-2		8 16 16		
					S-3		8 15 17		
10					S-4		8 12 14		
		---gray mottled black, yellow, and red below 12.5 feet			S-5		8 11 14		
15					S-6		8 16 16		
20					S-7		7 11 17		
25					S-8		7 19 33		
30									
		Decomposed BASALT *remolds to* silty to clayey ROCK FRAGMENTS, trace sand, GM, fine- to coarse-grained sand, gravel-sized angular rock fragments (Decomposed Columbia River Basalt)	30.5		S-9		24 50/3"		
35					S-10		50/5"		
40									



◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

(CONTINUED NEXT PAGE)

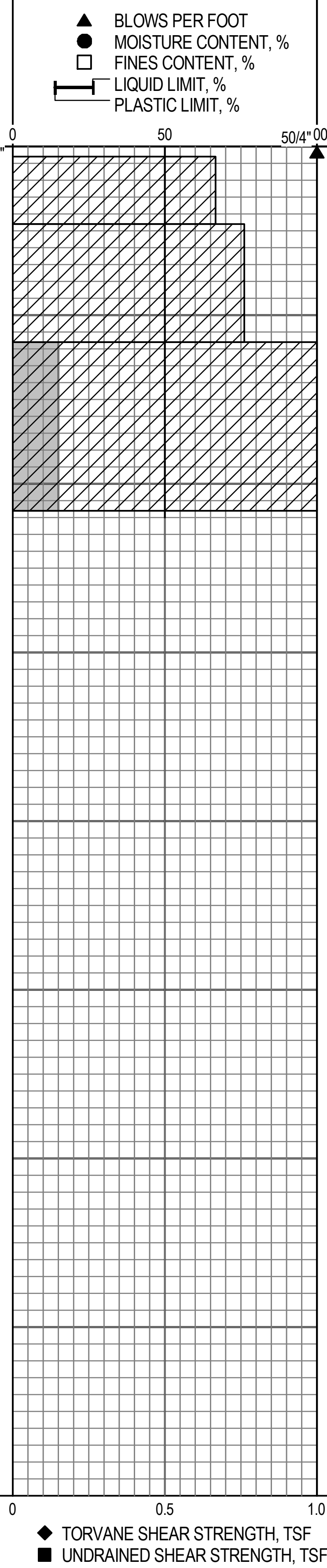
Logged By: M. Preciado	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/12/23	**GPS Coordinates:** 45.610757° N -122.802351° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.772
Note: See Legend for Explanation of Symbols	

GRI

BORING B-34

JUN. 2024 JOB NO. 6767-B FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: Not Available						
		BASALT, nonvesicular, gray, slightly weathered, strong (R4), very closely spaced open joints with iron oxide staining (Columbia River Basalt) ---core loss from 41.7 feet to 43 feet	40.3		S-11		50/4"	
					R-1			
		---slightly to moderately weathered, very close to close joints with clay infill below 43.5 feet			R-2			Geokon 4500S-700kPa VWP (SN2307611) installed at 43.9 feet
45		---fresh to slightly weathered, medium strong to strong (R3-R4)						
					R-3			
50								
		(9/13/2023)	51.0					
55								
60								
65								
70								
75								
80								



GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 11/29/23

GRI

BORING B-34

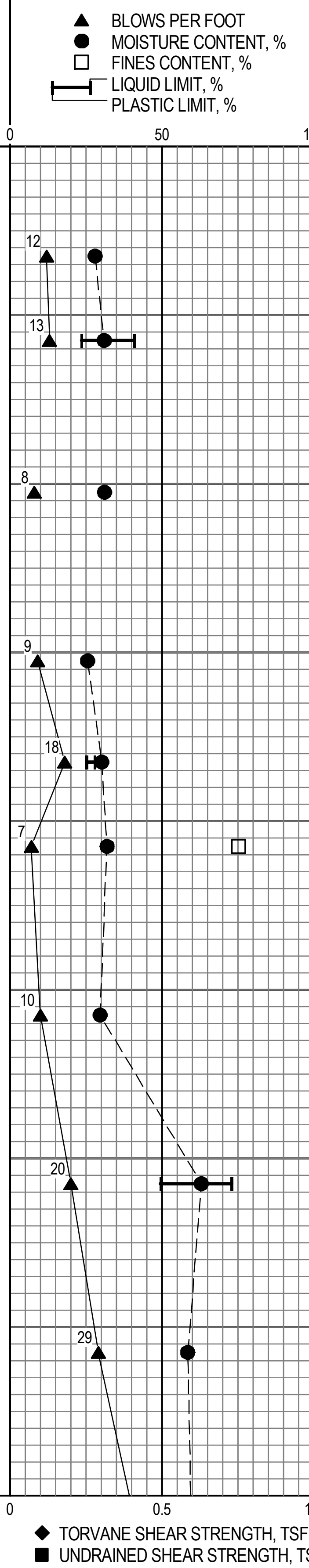
JUN. 2024 JOB NO. 6767-B FIG. 3A

BORING B-35

DEPTH, FT	CLASSIFICATION OF MATERIAL (Surface Elevation: Not Available)	ELEVATION, FT / DEPTH, FT
0	TOPSOIL	
	CLAY, some silt, trace sand, CH, brown mottled rust and gray, medium to high plasticity, stiff, fine-grained sand, contains fine organics (Portland Hills Silt/Possible Landslide Debris)	
5	Silty CLAY, trace sand, CL, light brown mottled dark brown and rust, medium plasticity, medium stiff to stiff, fine-grained sand (Portland Hills Silt/Possible Landslide Debris)	5.0
15	SILT, trace sand, ML, light brown mottled gray and rust, nonplastic, stiff to very stiff, fine-grained sand (Portland Hills Silt/Possible Landslide Debris)	15.0
20	SILT, some clay and sand, ML, brown mottled black, orange, and light brown, low to medium plasticity, medium stiff, fine- to coarse-grained sand, weathered texture (Residual Soil/Possible Landslide Debris)	20.0
30	Clayey SILT, some sand, trace gravel, MH, dark brown mottled black and yellow, very stiff, fine-grained sand, angular gravel (Residual Soil/Possible Landslide Debris)	30.0
40		

(CONTINUED NEXT PAGE)

SAMPLE NO.	BLOW COUNT
S-1	3, 6, 6
S-2	3, 6, 7
S-3	
S-4	3, 3, 5
S-5	
S-6	1, 4, 5
S-7	5, 8, 10
S-8	1, 4, 3
S-9	2, 4, 6
S-10	8, 9, 11
S-11	9, 14, 15

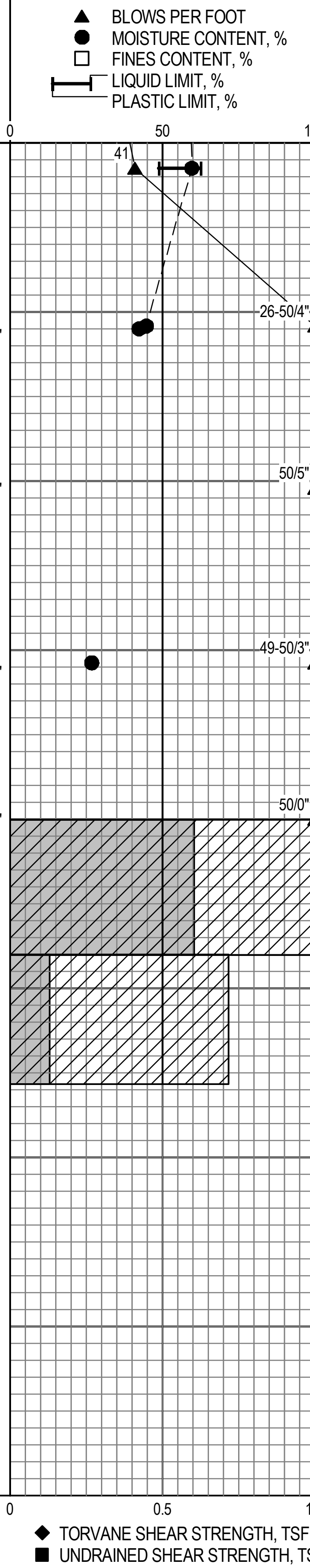


COMMENTS AND ADDITIONAL TESTS

2.75-inch-OD inclinometer casing installed to 62 feet

Logged By: M. Preciado	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/13/23	**GPS Coordinates:** 45.610238° N -122.802687° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 850 Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.772
Note: See Legend for Explanation of Symbols	

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Surface Elevation: Not Available				
	Clayey SILT, some sand, trace gravel, MH, dark brown mottled black and yellow, very stiff, fine-grained sand, angular gravel (Residual Soil/Possible Landslide Debris)		S-12	10 18 23	
45	Decomposed BASALT *remolds to* ROCK FRAGMENTS and CLAY, some sand, CH, hard, fine- to coarse-grained sand, angular gravel-sized rock fragments (Decomposed Columbia River Basalt)	45.5	S-13A S-13B	26 50/4"	
50	Decomposed BASALT *remolds to* Clayey ROCK FRAGMENTS, some sand, GC, very dense, fine- to coarse-grained sand, angular gravel-sized rock fragments (Decomposed Columbia River Basalt)	50.0	S-14	50/5"	
55			S-15	49 50/3"	Geokon 4500S-700kPa VWP (SN2307610) installed at 54 feet
60	BASALT, nonvesicular, gray, fresh to predominantly decomposed, medium strong (R3), closely spaced joints contain trace iron oxide staining (Columbia River Basalt)	60.0	S-16	50/0"	
			R-1		
65			R-2		
	(9/14/2023)	67.8			
70					
75					
80					



GRI BORING LOG (GPS) GRI DATA TEMPLATE.GDT 11/29/23

GRI

BORING B-35

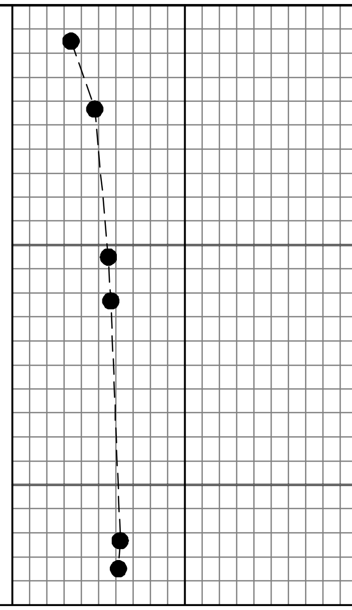
JUN. 2024 JOB NO. 6767-B FIG. 4A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO. / SAMPLE TYPE	● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 – 50 – 100)	COMMENTS AND ADDITIONAL TESTS

HA-1

Surface Elevation: Not Available

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	COMMENTS AND ADDITIONAL TESTS
	SILT, trace sand, brown, nonplastic to low plasticity, moist, fine-grained sand, contains organics consisting of roots (Portland Hills Silt/Possible Landslide Debris)		S-1	
			S-2	
5			S-3	
	---mottled gray to rust, up to some clay below 6.2 feet		S-4	
	---mottling absent below 7 feet			
10				Groundwater not observed during excavation
			S-5	
			S-6	
	(9/26/2023)	12.0		



◆ TORVANE SHEAR STRENGTH, TSF

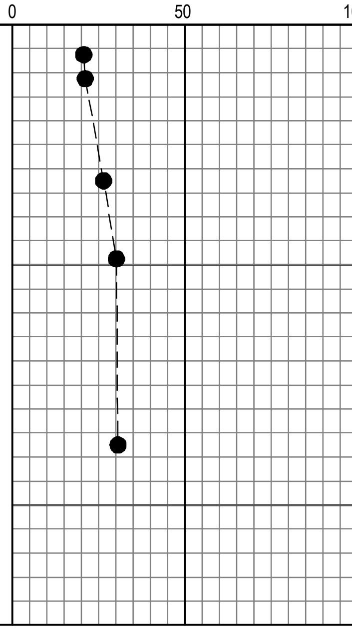
Logged By: M. Preciado | **Excavated by:** GRI | **Equipment:** Hand Auger

Date Started: 9/26/23 | **GPS Coordinates:** 45.609822° N -122.802886° W (WGS 84) | **Note:** See Legend for Explanation of Symbols

HA-2

Surface Elevation: Not Available

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	COMMENTS AND ADDITIONAL TESTS
	SILT, trace sand, ML, brown mottled gray and rust, low plasticity, damp, contains fine organics consisting of roots (Portland Hills Silt/Possible Landslide Debris)		S-1	
			S-2	
			S-3	
5			S-4	
			S-5	Groundwater not observed during excavation
	(9/26/2023)	9.0		
10				



◆ TORVANE SHEAR STRENGTH, TSF

Logged By: M. Preciado | **Excavated by:** GRI | **Equipment:** Hand Auger

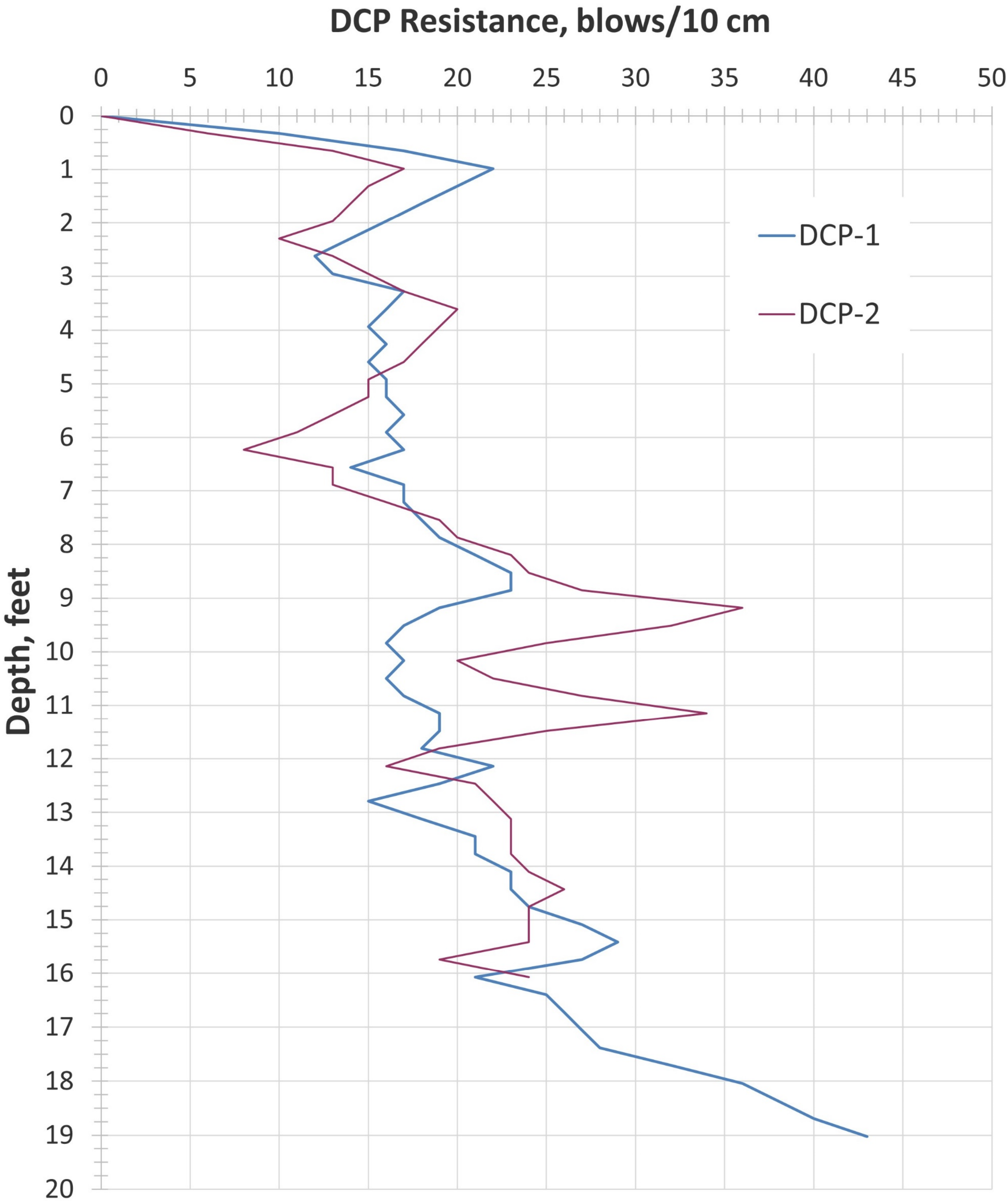
Date Started: 9/26/23 | **GPS Coordinates:** 45.610312° N -122.803066° W (WGS 84) | **Note:** See Legend for Explanation of Symbols

GRI HA LOG (GPS) - 2 PP GRI DATA TEMPLATE.GDT 11/29/23

GRI

BORINGS

JUN. 2024 JOB NO. 6767-B FIG. 5A



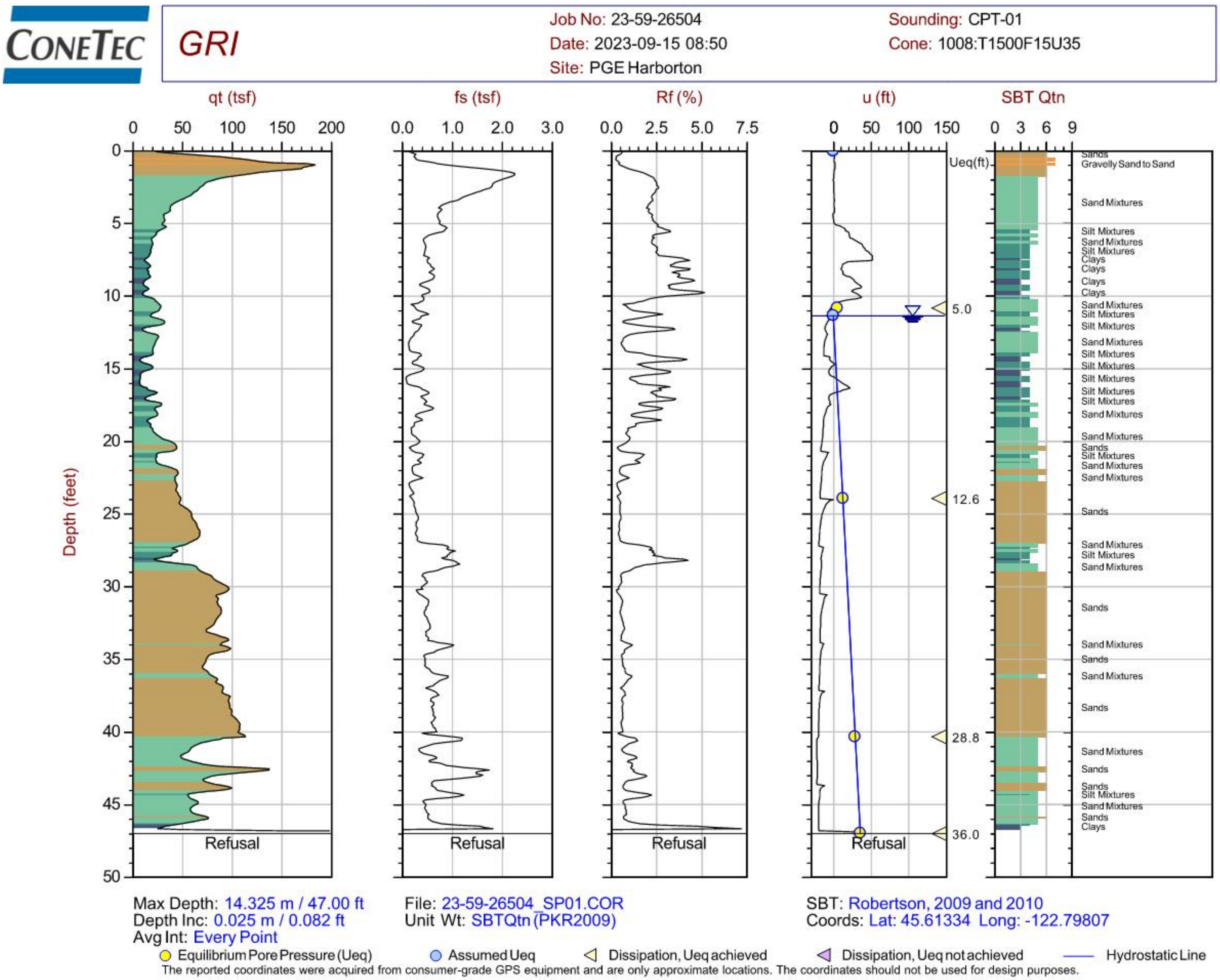
GRI

NOTES:

1. DCP TESTING WAS COMPLETED ON SEPTEMBER 26, 2023, IN CONJUCTION WITH SUBSURFACE EXPLORATIONS HA-1 AND HA-2.
2. SEE REPORT FOR DISCUSSION OF TEST DESCRIPTION AND INTERPRETATION OF THE TEST RESULTS.

"WILDCAT" DCP TEST SUMMARY

JUN. 2024 | JOB NO. 6767-B | FIG. 6A



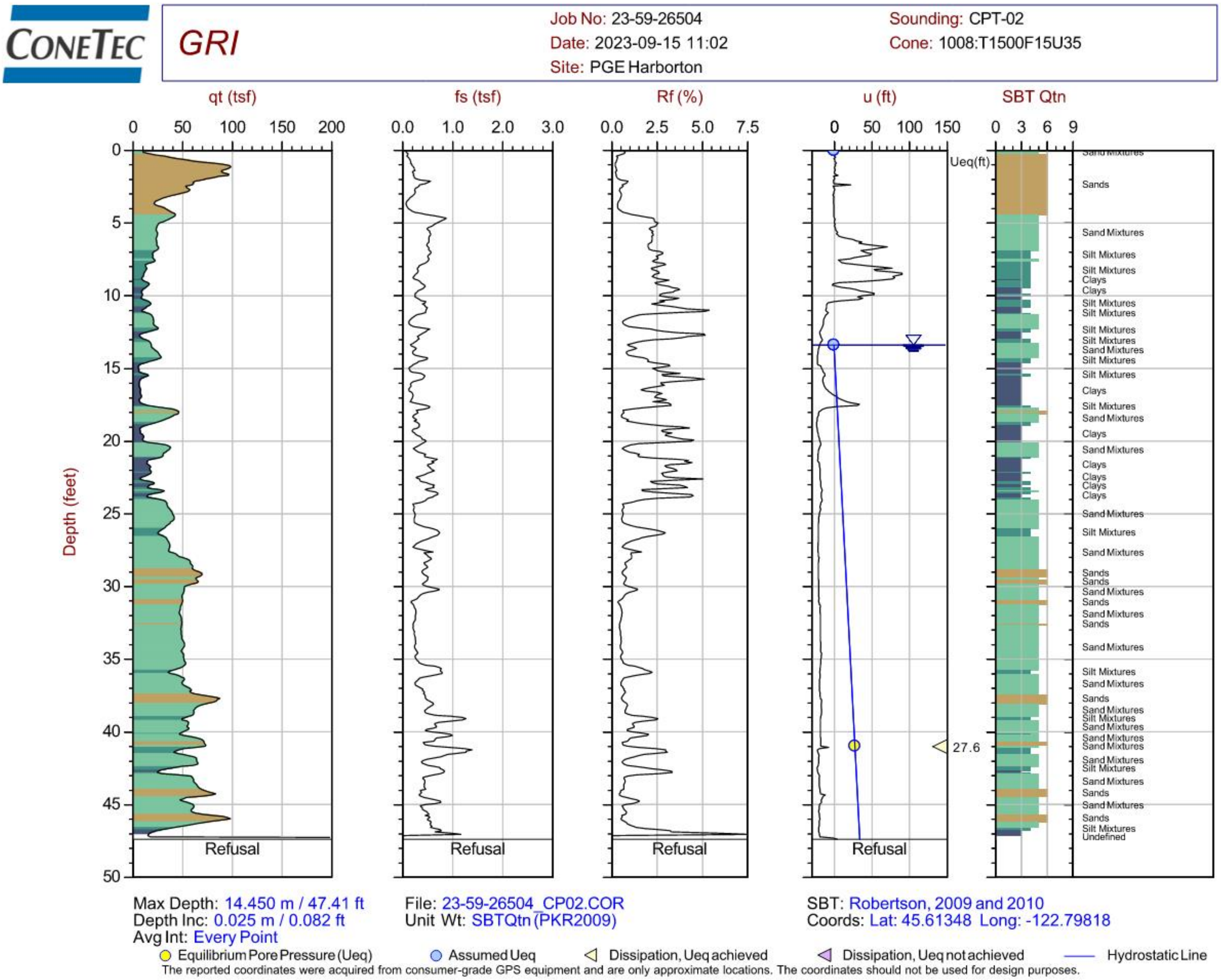
NOTES:

1. CPT PROBES WERE COMPLETED ON SEPTEMBER 15, 2023, BY CONETEC, INC.
2. SEE APPENDIX B FOR FULL REPORT PREPARED BY CONETEC, INC.

GRI

CONE PENETRATION TEST CPT-1

JUN. 2024 | JOB NO. 6767-B | FIG. 7A



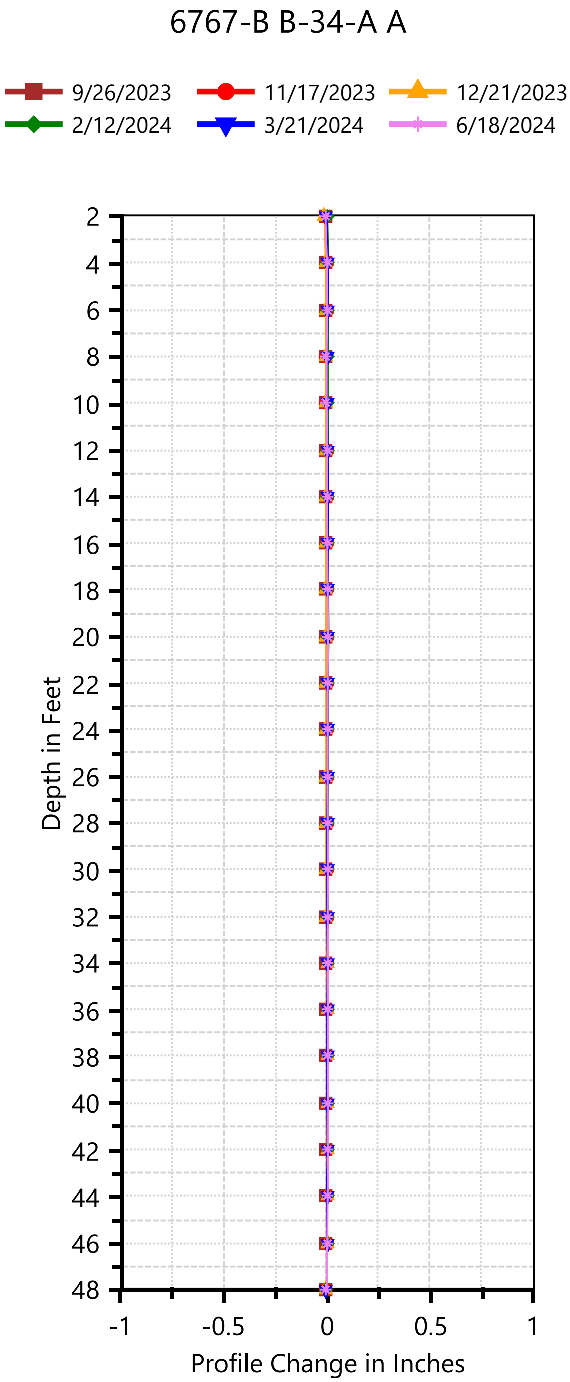
NOTES:

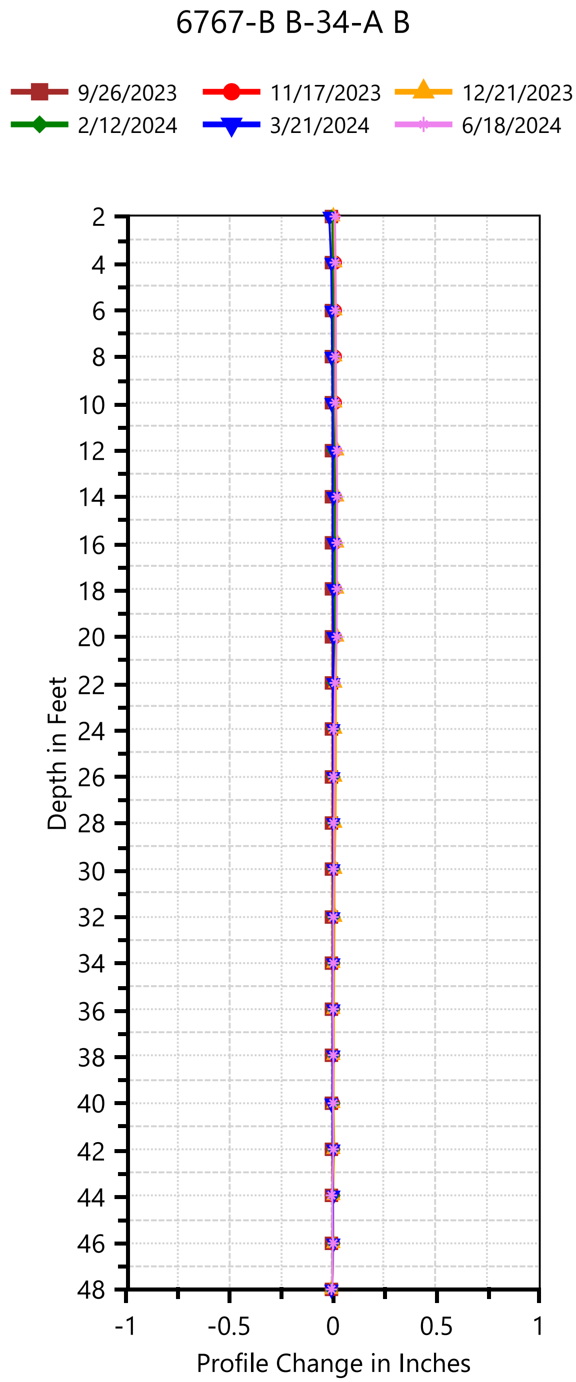
1. CPT PROBES WERE COMPLETED ON SEPTEMBER 15, 2023, BY CONETEC, INC.
2. SEE APPENDIX B FOR FULL REPORT PREPARED BY CONETEC, INC.

GRI

CONE PENETRATION TEST CPT-2

JUN. 2024 JOB NO. 6767-B FIG. 8A

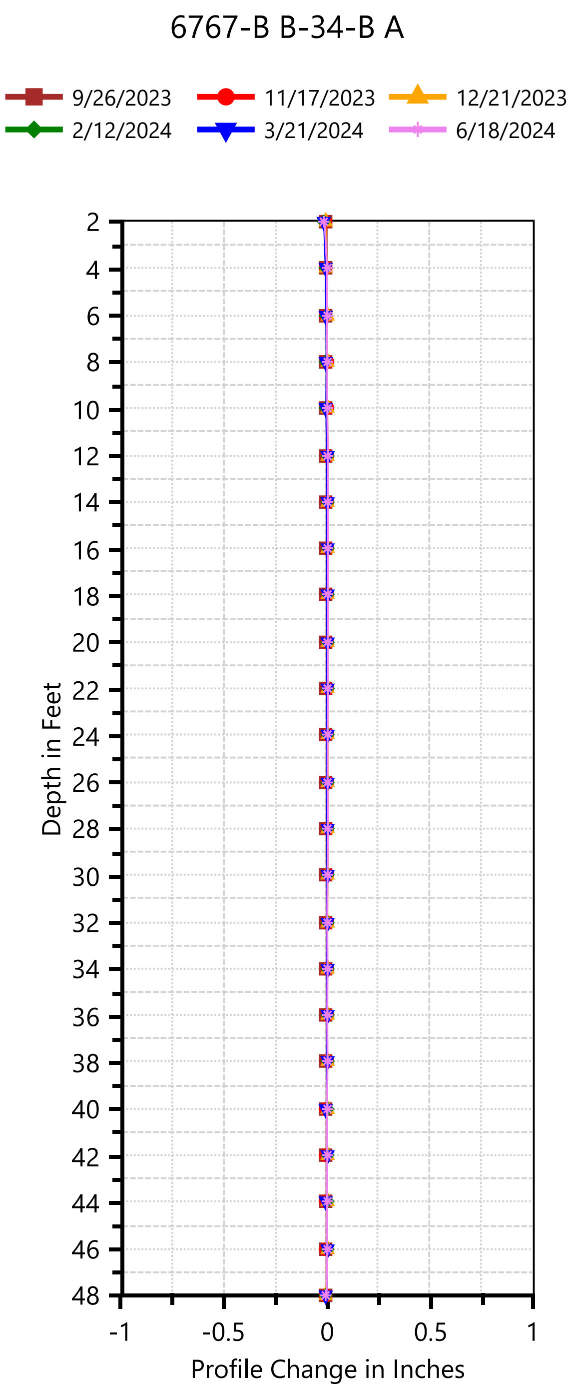


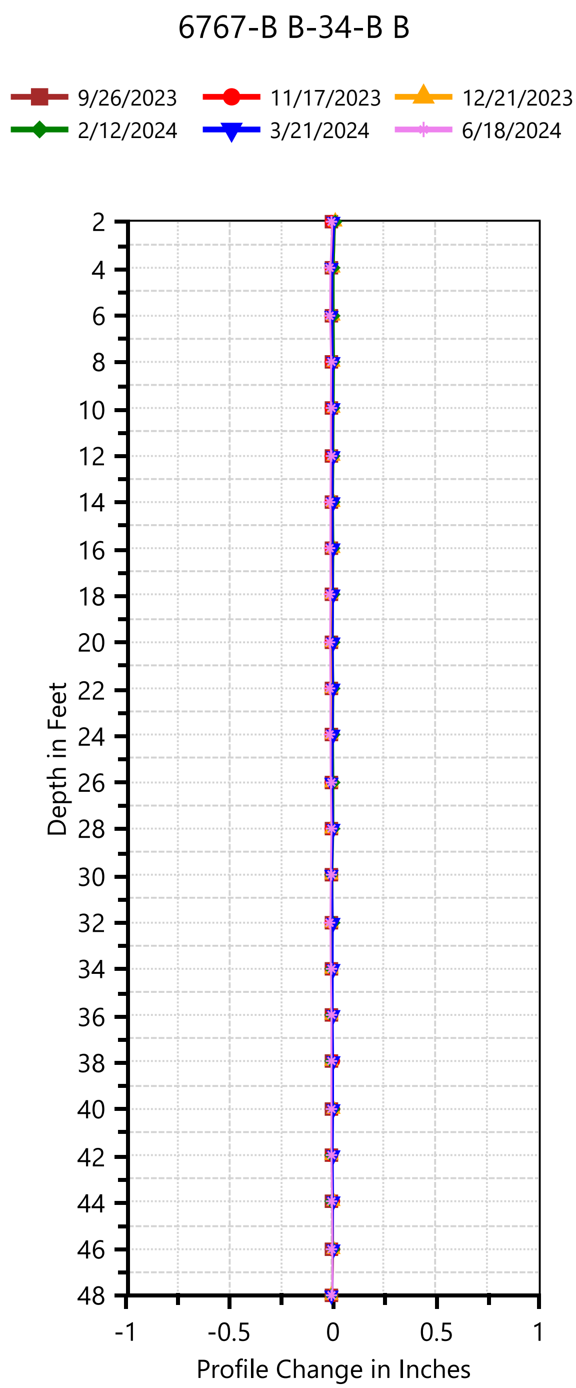


GRI

PROFILE CHANGE PLOT
B-34 A-AXIS INCLINOMETER

JUN. 2024 | JOB NO. 6767-B | FIG. 9A

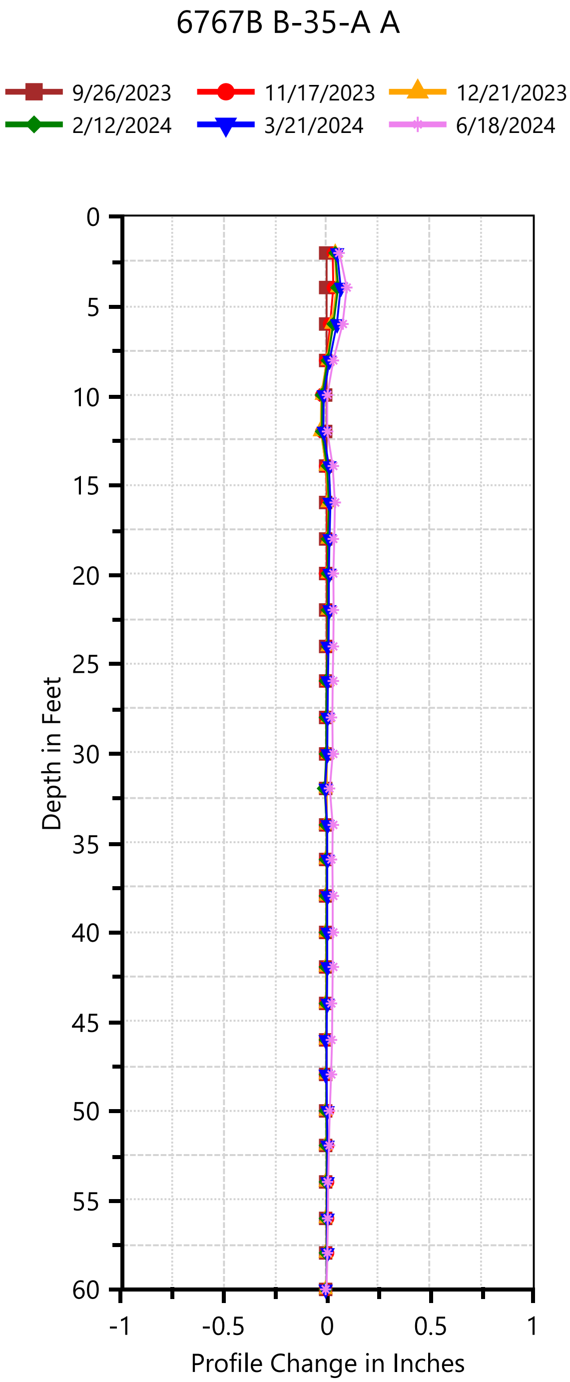


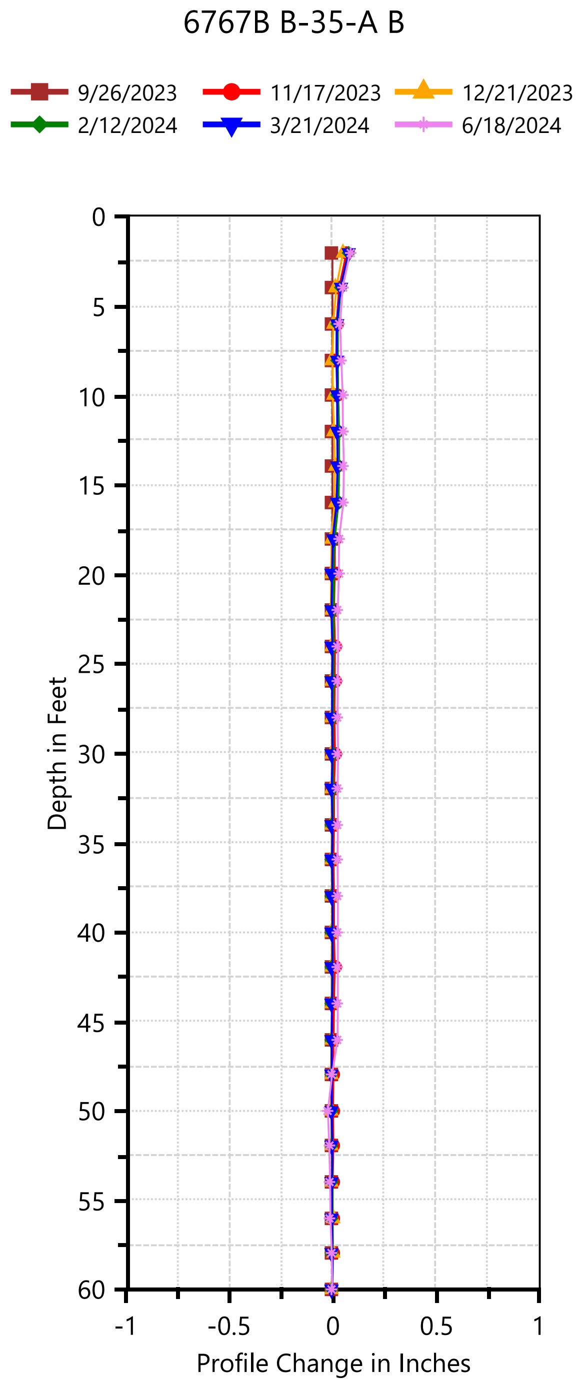


GRI

PROFILE CHANGE PLOT
B-34 B-AXIS INCLINOMETER

JUN. 2024 | JOB NO. 6767-B | FIG. 9A

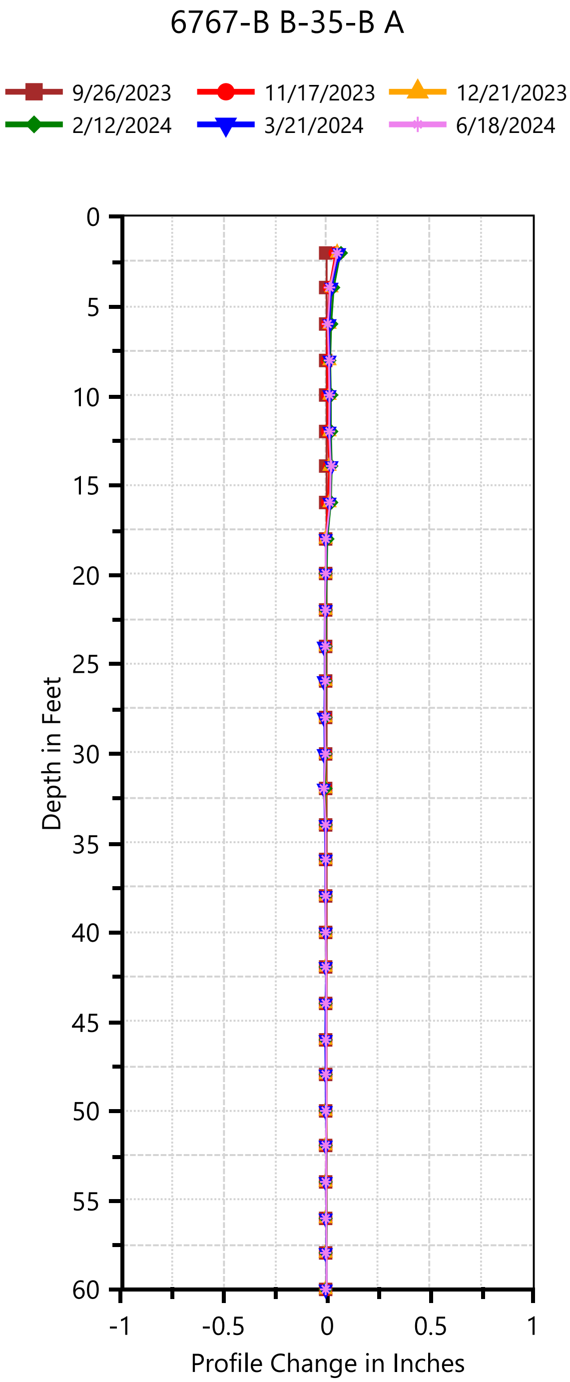


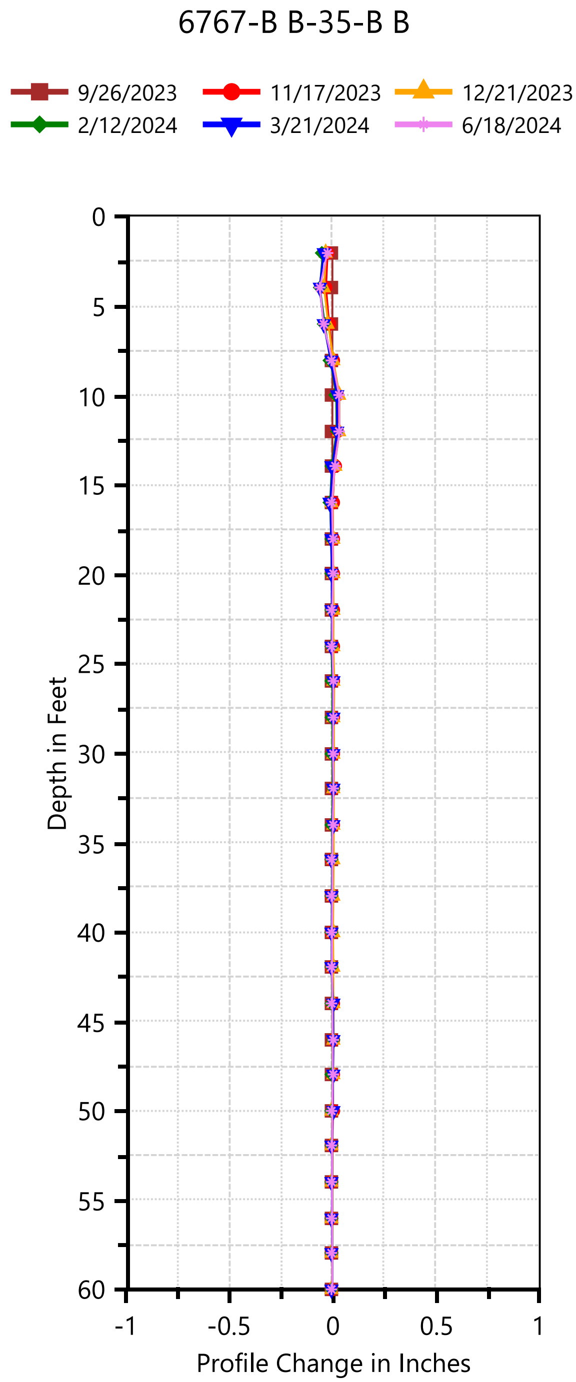


GRI

PROFILE CHANGE PLOT
B-35 A-AXIS INCLINOMETER

JUN. 2024 | JOB NO. 6767-B | FIG. 10A





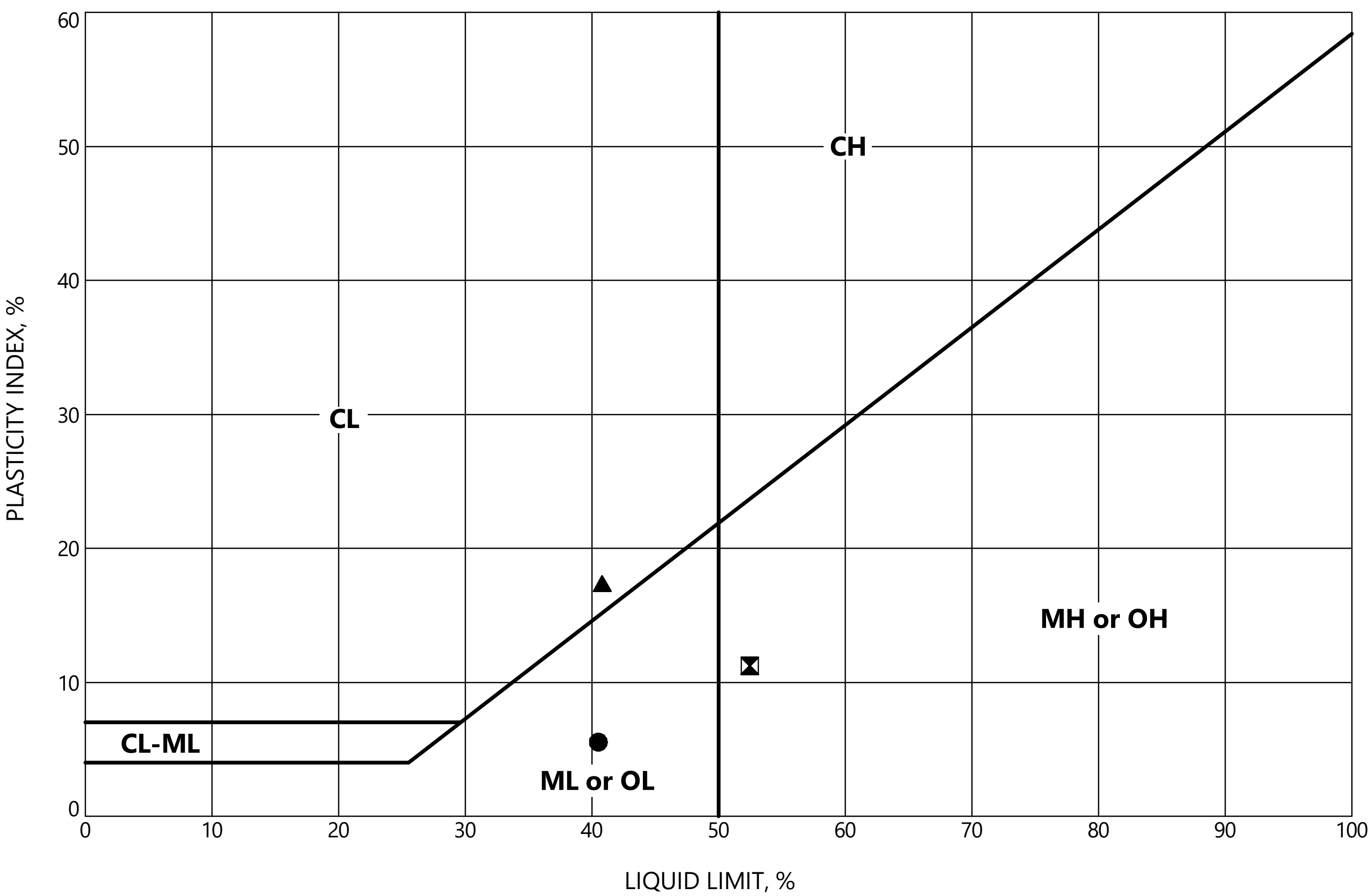
GRI

PROFILE CHANGE PLOT
B-35 B-AXIS INCLINOMETER

JUN. 2024 | JOB NO. 6767-B | FIG. 10A

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



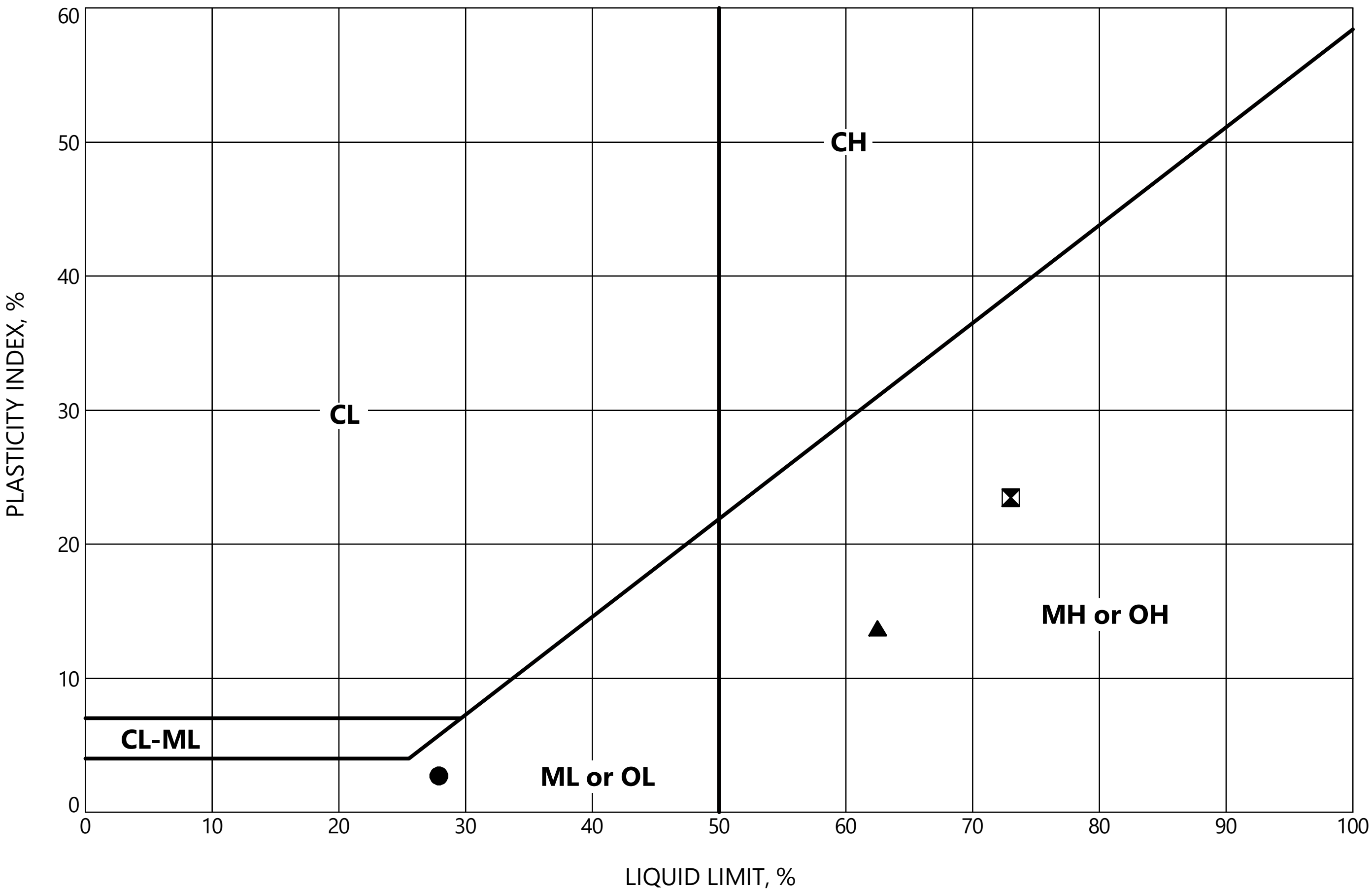
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-31	S-3	8.5	SILT, up to some clay and sand, ML	41	35	5	40
⧗	B-34	S-4	10.0	Clayey SILT, trace sand and gravel, MH	52	41	11	49
▲	B-35	S-2	5.0	Silty CLAY, trace sand, CL	41	23	18	31

GRI

PLASTICITY CHART

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



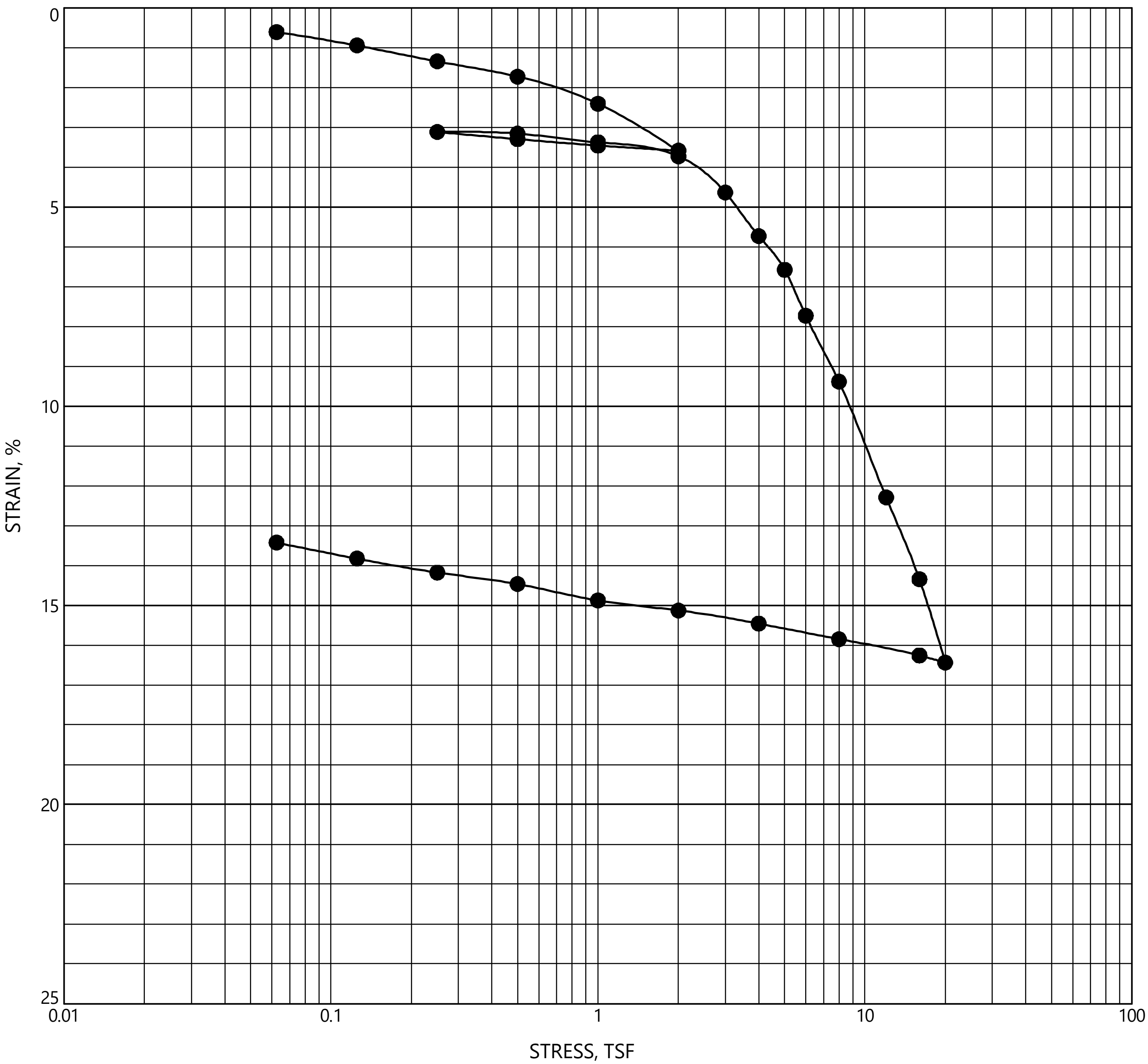
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-35	S-7	17.5	SILT, trace sand, ML	28	25	3	30
⧗	B-35	S-10	30.0	Clayey SILT, some sand, trace gravel, MH	73	50	23	63
▲	B-35	S-12	40.0	Clayey SILT, some sand, trace gravel, MH	63	49	13	60

ATTERBERG-PLASTICITY 3 PER PAGE GRI DATA TEMPLATE.GDT 2/9/24

GRI

PLASTICITY CHART

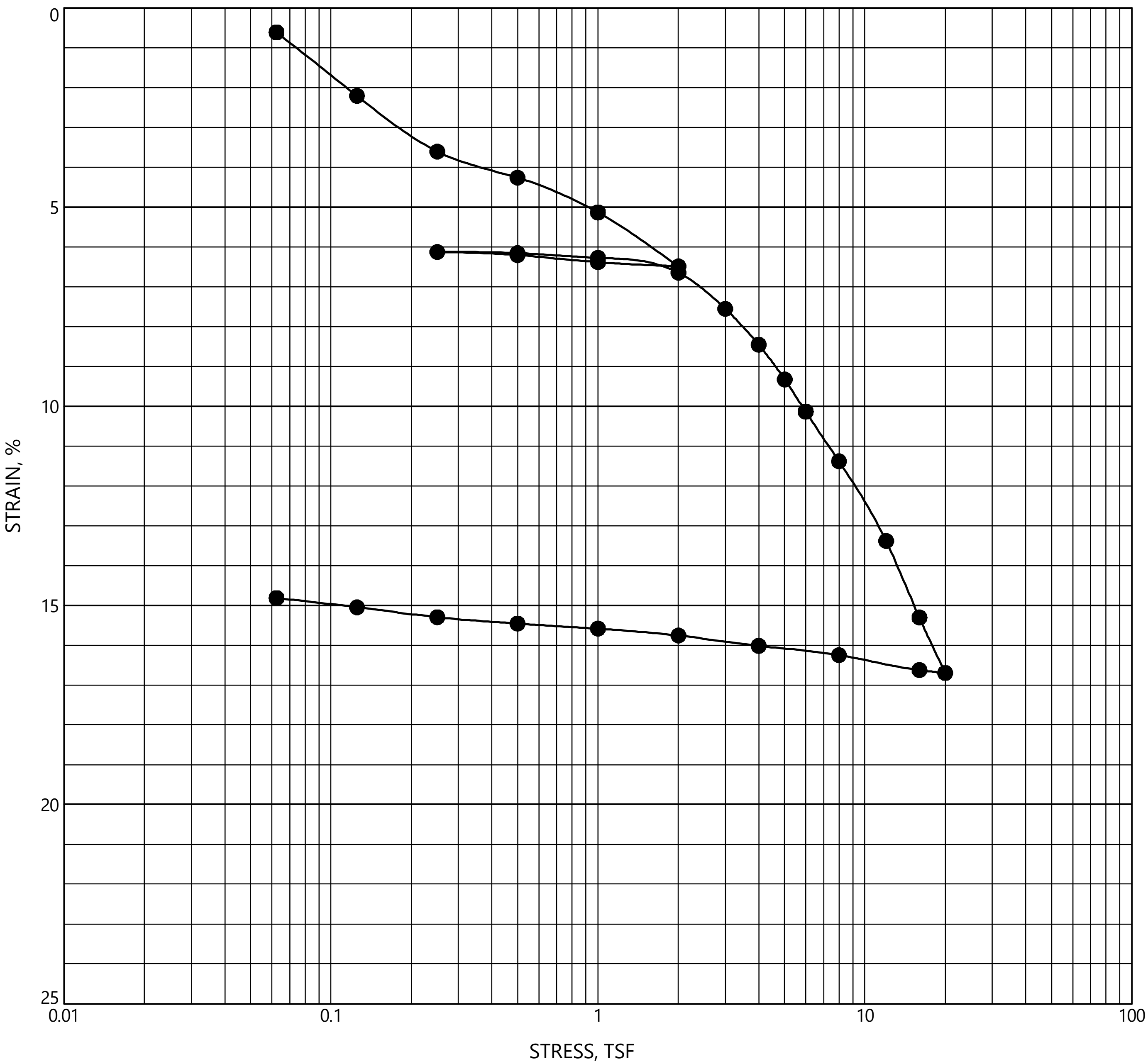
JUN. 2024 JOB NO. 6767-B FIG. 12A



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-31	S-3	9.2	SILT, up to some clay and sand, ML	81	33

GRI

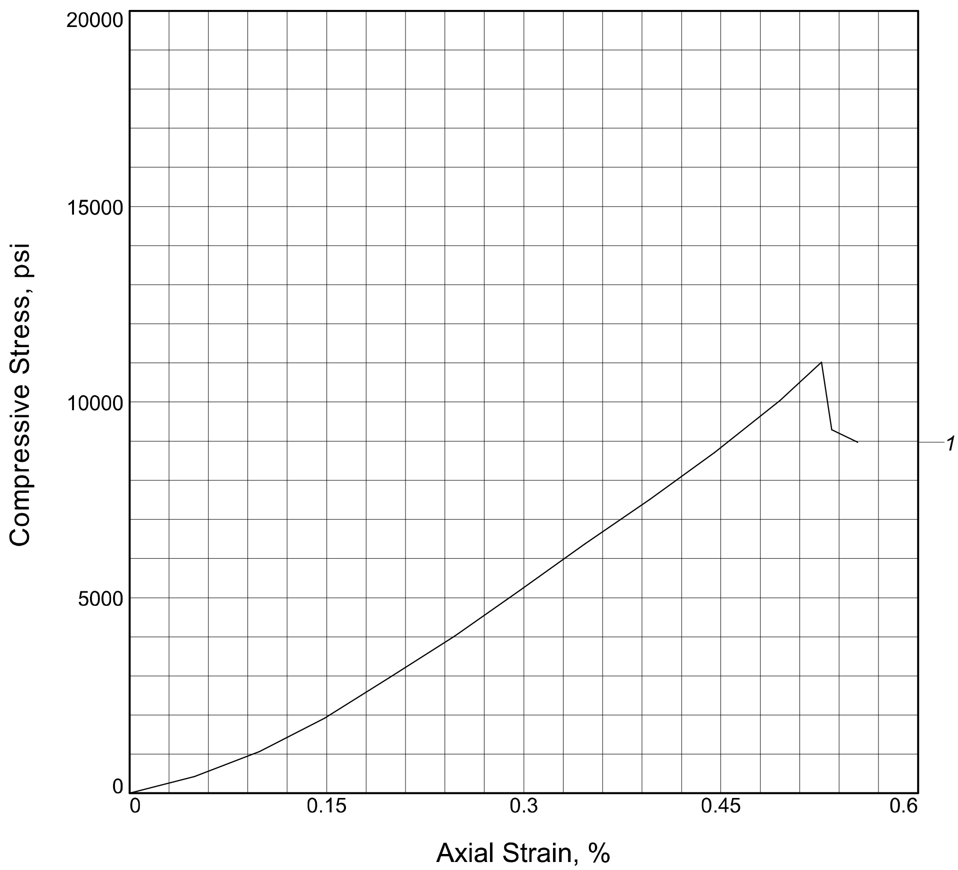
CONSOLIDATION TEST



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-31	S-8	25.7	Silty SAND, SM	82	43

CONSOLIDATION TEST

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	11019.45			
Failure strain, %	0.5			
Strain rate, in./min.	0.100			
Water content, %	0.2			
Wet density, pcf	170.1			
Dry density, pcf	169.7			
Saturation, %	58.0			
Void ratio	0.0118			
Specimen diameter, in.	2.40			
Specimen height, in.	5.05			
Height/diameter ratio	2.10			

Description:

LL =	**PL =**	**PI =**	**Assumed GS=** 2.75	**Type:**

Project No.: 2236060-606

Date Sampled:

Remarks:

Client: GRI (Project No. 6767-B)

Project: PGE Harborton

Source of Sample: 9244 **Depth:** 56.0-57.5'

Sample Number: B-31, R-3

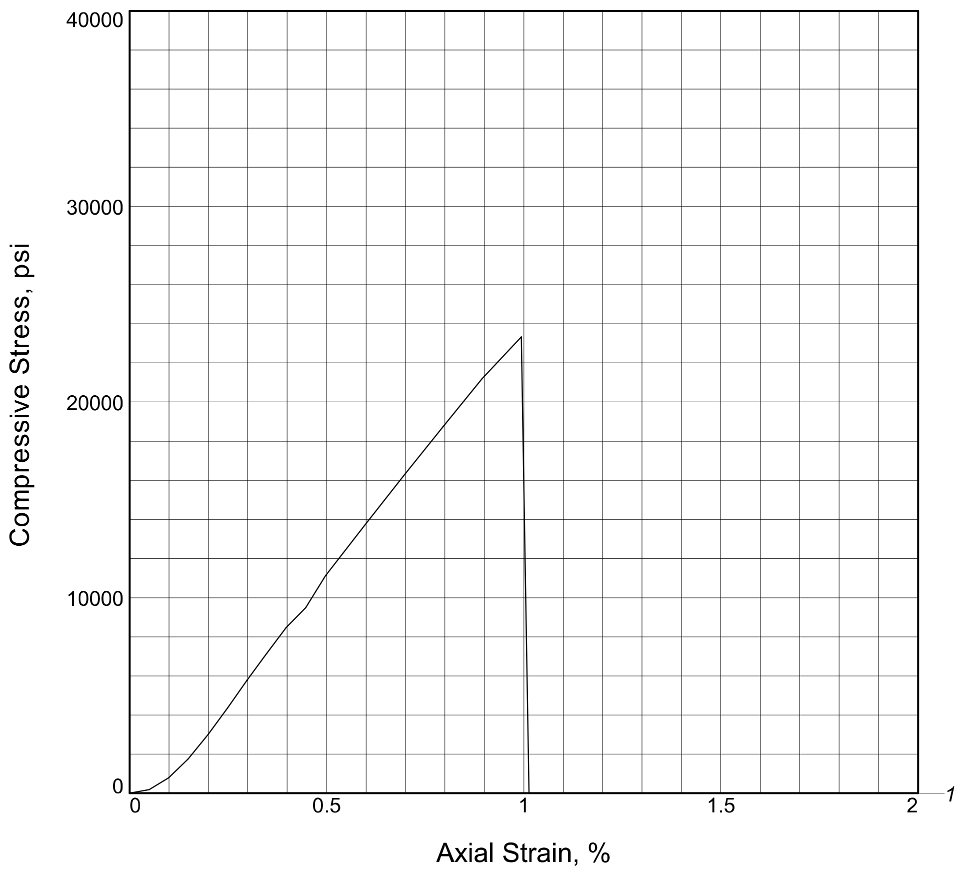
UNCONFINED COMPRESSION TEST

FEI Testing & Inspection, Inc.

Corvallis, OR

Figure 15A

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	23333.80			
Failure strain, %	1.0			
Strain rate, in./min.	0.100			
Water content, %	0.2			
Wet density, pcf	168.7			
Dry density, pcf	168.3			
Saturation, %	32.8			
Void ratio	0.0198			
Specimen diameter, in.	2.37			
Specimen height, in.	5.03			
Height/diameter ratio	2.12			

Description:

LL =	**PL =**	**PI =**	**Assumed GS=** 2.75	**Type:**

Project No.: 2236060-606
Date Sampled:
Remarks:

Client: GRI (Project No. 6767-B)

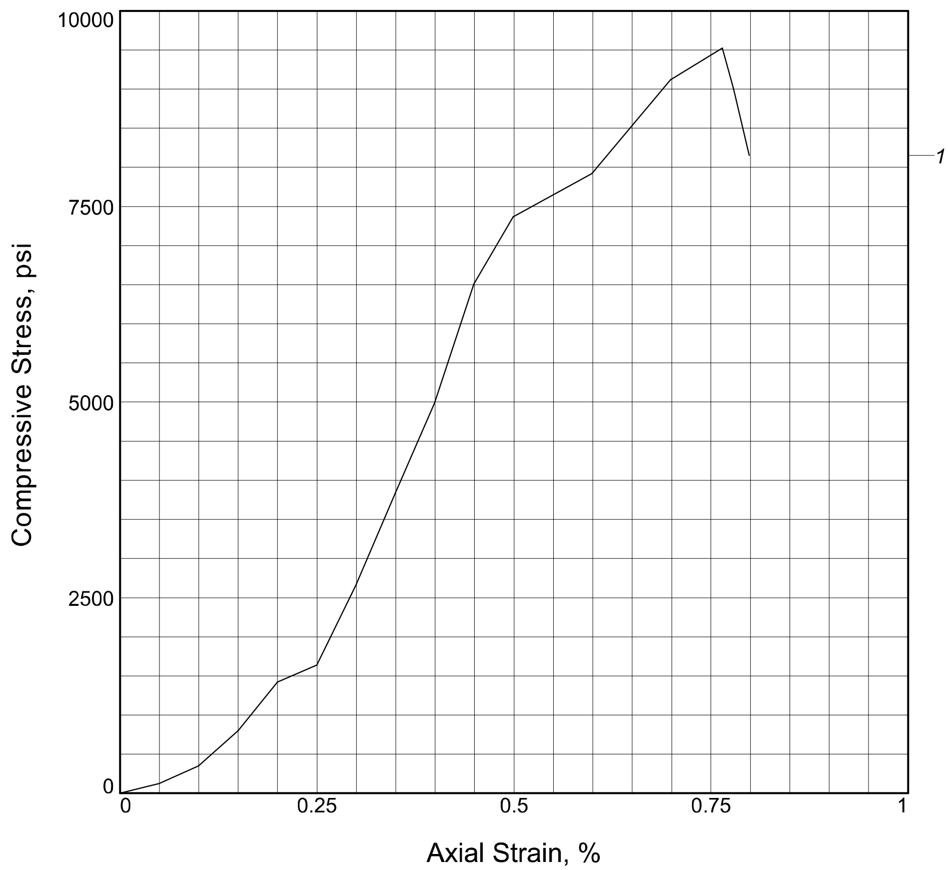
Project: PGE Harborton

Source of Sample: 9244 **Depth:** 7.2'-8.2'
Sample Number: B-32, R-1

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 16A

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psi	9523.93			
Failure strain, %	0.8			
Strain rate, in./min.	0.100			
Water content, %	0.8			
Wet density, pcf	168.7			
Dry density, pcf	167.4			
Saturation, %	87.2			
Void ratio	0.0256			
Specimen diameter, in.	2.37			
Specimen height, in.	5.01			
Height/diameter ratio	2.11			

Description:

LL =	PL =	PI =	Assumed GS= 2.75	Type:

Project No.: 2236060-606
Date Sampled:
Remarks:

Figure 17A

Client: GRI (Project No. 6767-B)

Project: PGE Harborton

Source of Sample: 9244 **Depth:** 13.5'-14.6'
Sample Number: B-32, R-2

UNCONFINED COMPRESSION TEST
FEI Testing & Inspection, Inc.
Corvallis, OR

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL # 823-067 **Date:** 11/7/2023 **Tested By:** PJ **Checked:** PJ
Client: GRI **Project:** PGE Harborton **Proj. No:** 6767-8
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B-31	S-2	5.0	-	-	4,917	10	30	0.0030	-	510	21	Negative	37.9	Brown CLAY w/ Sand
B-34	S-3	7.5	-	-	2,485	25	6	0.0006	-	513	21	Negative	45.7	Brown CLAY w/ Sand
B-35	S-1	2.5	-	-	7,497	27	9	0.0009	-	525	21	Negative	28.4	Brown CLAY w/ Sand

GRI

CORROSIVITY TESTS SUMMARY

JUN. 2024 JOB NO. 6767-B FIG. 18A

APPENDIX B

CPT Test Report by ConeTec, Inc.

PRESENTATION OF SITE INVESTIGATION RESULTS

PGE Harborton

Prepared for:

GRI

ConeTec Job No: 23-59-26504

Project Start Date: 2023-09-15
Project End Date: 2023-09-15
Report Date: 2023-10-03

Prepared by:

ConeTec Inc.

1237 S Director Street, Seattle, WA 98108

Tel: (253) 397-4861

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

ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. The program consisted of Resistivity Piezocone Penetration Testing and Pore Pressure Dissipation Testing. Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project	
Client	GRI
Project	PGE Harborton
ConeTec Project Number	23-59-26504
Rig Description	Geoprobe Track Rig (C05-032)
Test Types	CPTu/SCPTu
Additional Comments	None

Coordinates	
Collection Method	Consumer Grade GPS
EPSG Number	4326 (WGS84 / Lat Long)
Additional Comments	None

Please refer to the list of attached documents following the text of this report. A test summary, location map, and plots are included. Thank you for the opportunity to work on this project.

PROJECT INFORMATION

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 Meters. This has been accounted for in the CPT data files.
Additional Comments	Seismic data offset

Calculated Geotechnical Parameters	
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

3rd Party Disclaimer

- The “Report” refers to this report titled PGE Harborton
- The Report was prepared by ConeTec for GRI

The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

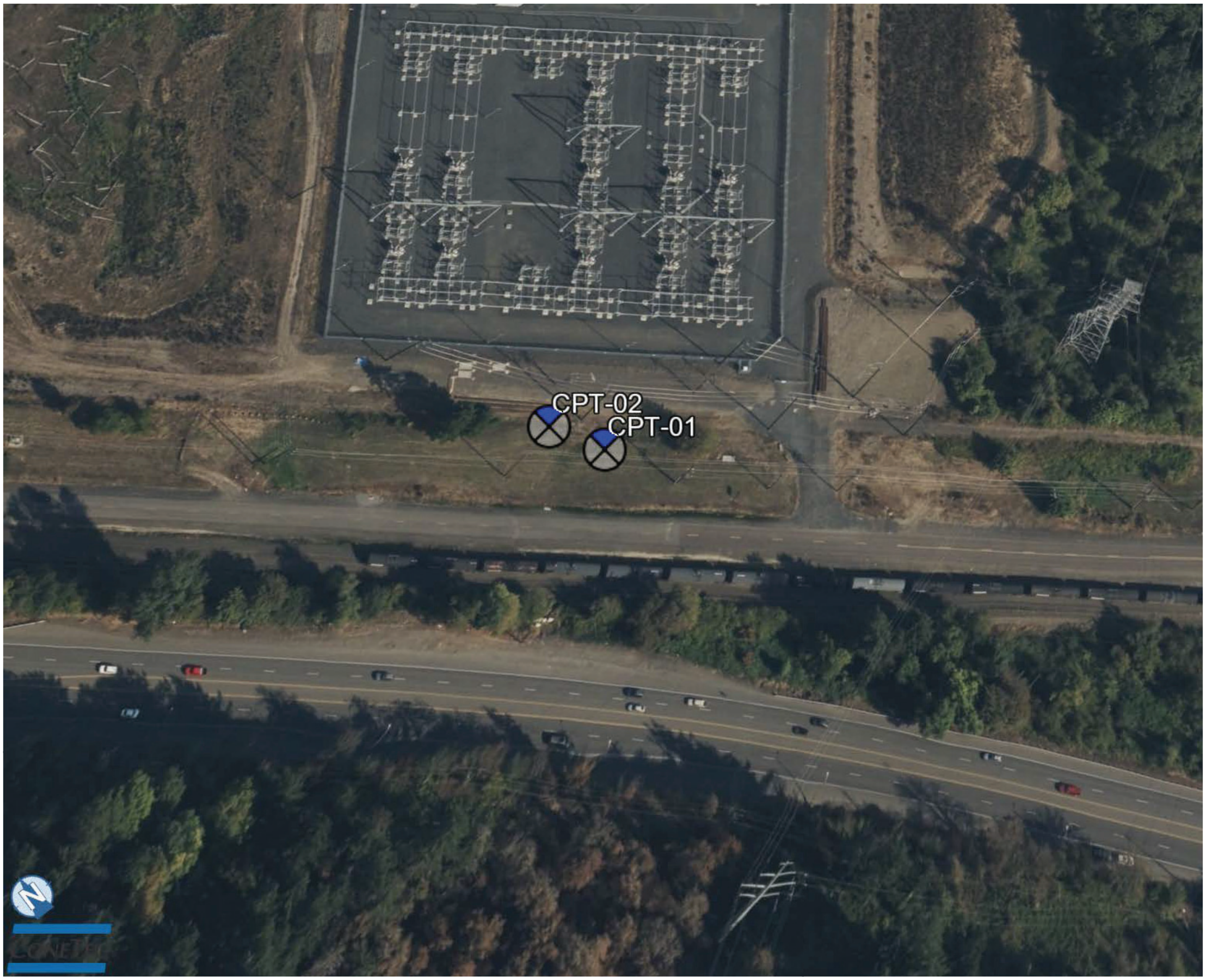
- ConeTec was retained by GRI
- The “Report” refers to this report titled PGE Harborton
- ConeTec was retained to collect and provide the raw data (“Data”) which is included in the Report.

ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

CONTENTS

The following listed below are included in the report:

SITE MAP



ConeTec Job Number: 23-59-26504

Client: GRI

Project: PGE Harborton

Report Date: 2023-10-03

Sounding Location

All sounding locations are approximate

ConeTec

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 23-59-26504
Client: GRI
Project: PGE Harborton
Start Date: 2023-09-15
End Date: 2023-09-15

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Rig	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Latitude[2]	Longitude[2]	Refer to Notation Number
CPT-01	23-59-26504_SP01	2023-09-15	C05-023	1008:T1500F15U35	15	11.3	47.0	45.61334	-122.79807	
CPT-02	23-59-26504_CP02	2023-09-15	C05-023	1008:T1500F15U35	15	13.4	47.4	45.61348	-122.79818	
Totals	2 Soundings						94.4 ft			

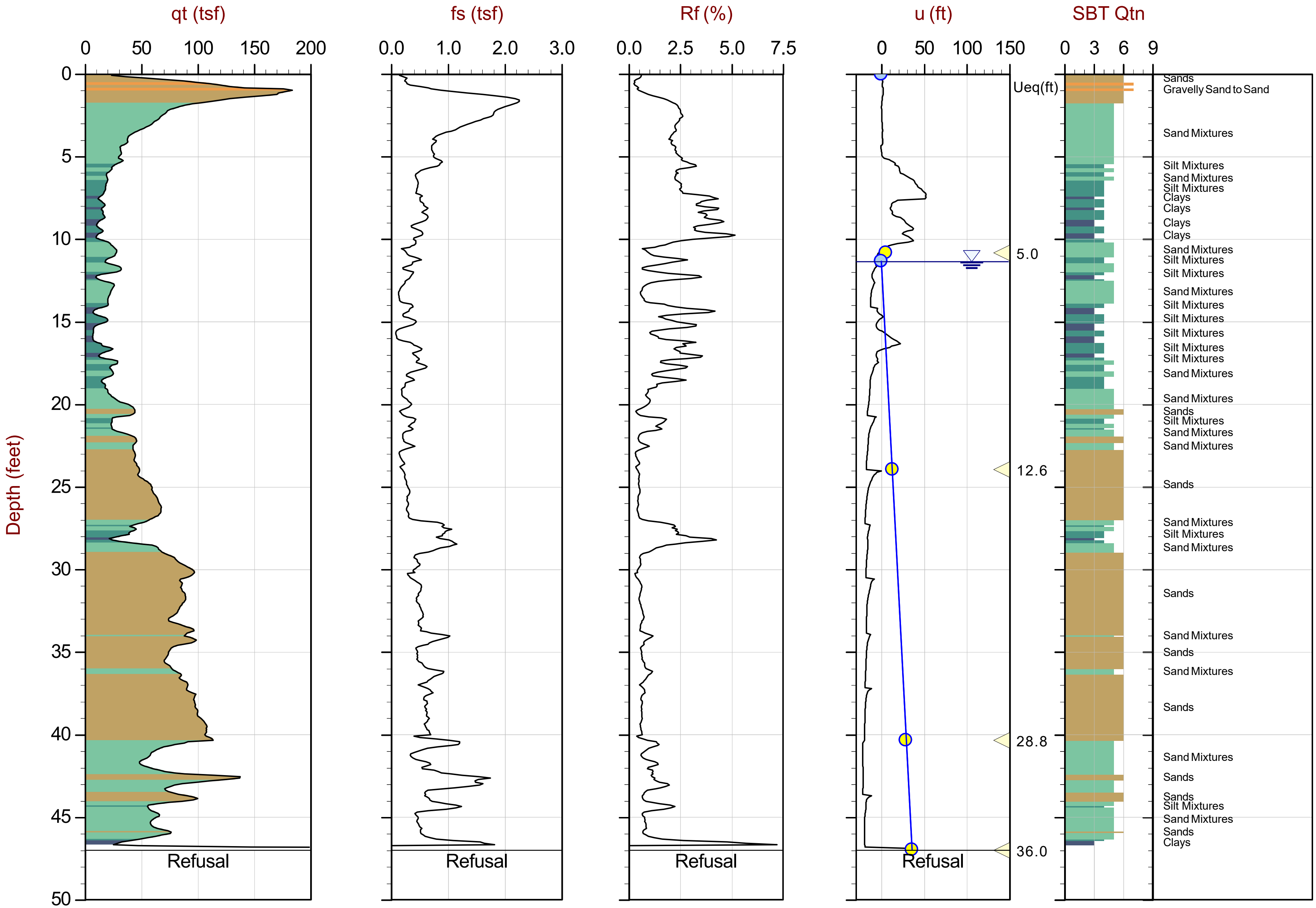
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 4326 (WGS84 / LatLong).

CONETEC

GRI

Job No: 23-59-26504
Date: 2023-09-15 08:50
Site: PGE Harborton

Sounding: CPT-01
Cone: 1008:T1500F15U35



Max Depth: 14.325 m / 47.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-26504_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.61334 Long: -122.79807

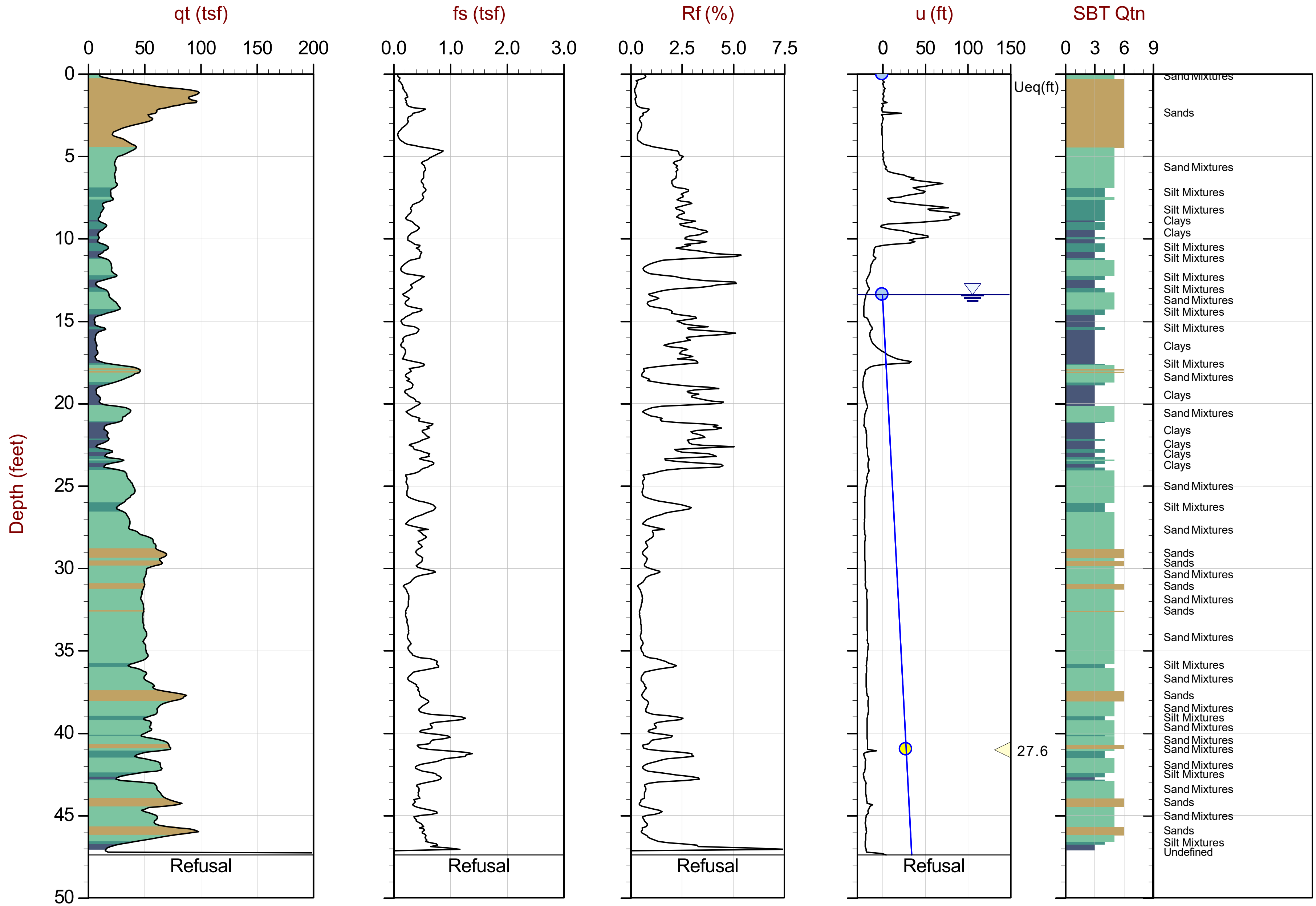
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

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

GRI

Job No: 23-59-26504
Date: 2023-09-15 11:02
Site: PGE Harborton

Sounding: CPT-02
Cone: 1008:T1500F15U35



Max Depth: 14.450 m / 47.41 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-26504_CP02.COR
Unit Wt: SBTQtn (PKR2009)

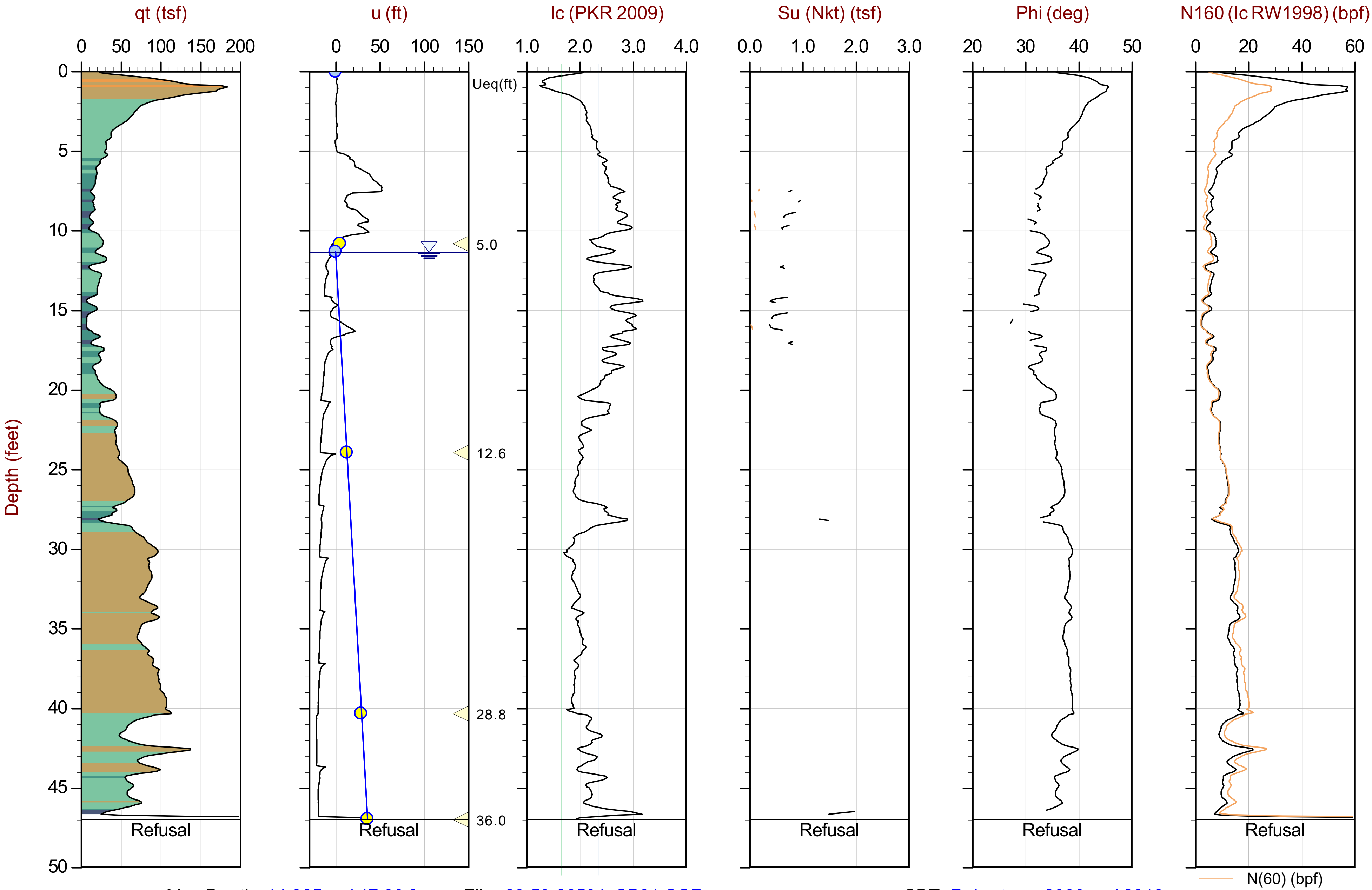
SBT: Robertson, 2009 and 2010
Coords: Lat: 45.61348 Long: -122.79818

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

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

Advanced Cone Penetration Test Plots

CONETEC GRI	Job No: 23-59-26504 Date: 2023-09-15 08:50 Site: PGE Harborton	Sounding: CPT-01 Cone: 1008:T1500F15U35



Max Depth: 14.325 m / 47.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-26504_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.61334 Long: -122.79807

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

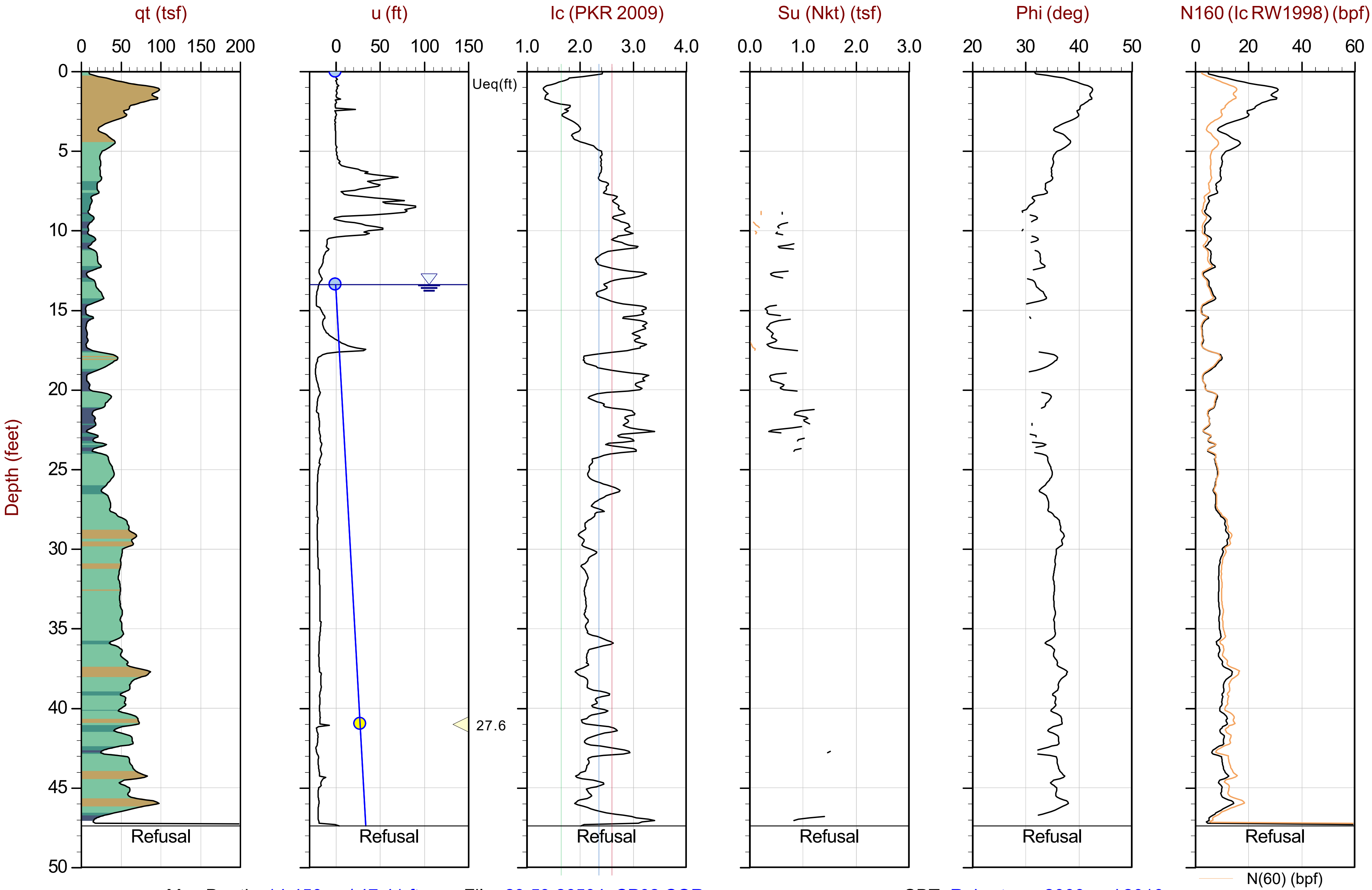
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

GRI

Job No: 23-59-26504
Date: 2023-09-15 11:02
Site: PGE Harborton

Sounding: CPT-02
Cone: 1008:T1500F15U35



Max Depth: 14.450 m / 47.41 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-26504_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 9.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.61348 Long: -122.79818

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

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

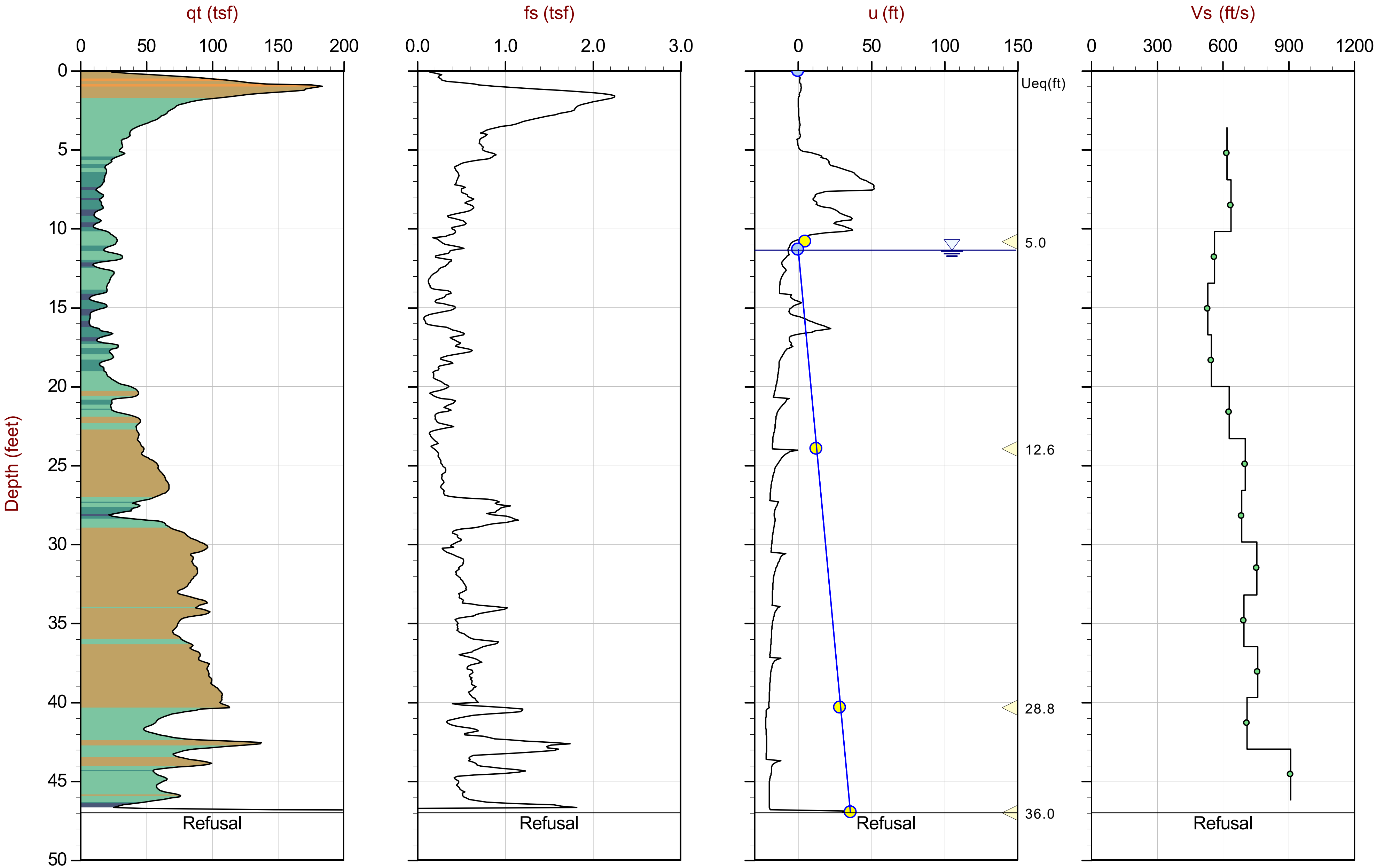
Seismic Cone Penetration Test Plots

CONETEC

GRI

Job No: 23-59-26504
Date: 2023-09-15 08:50
Site: PGE Harborton

Sounding: CPT-01
Cone: 1008:T1500F15U35



Max Depth: 14.325 m / 47.00 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-59-26504_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.61334 Long: -122.79807

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

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

Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results

ConeTec

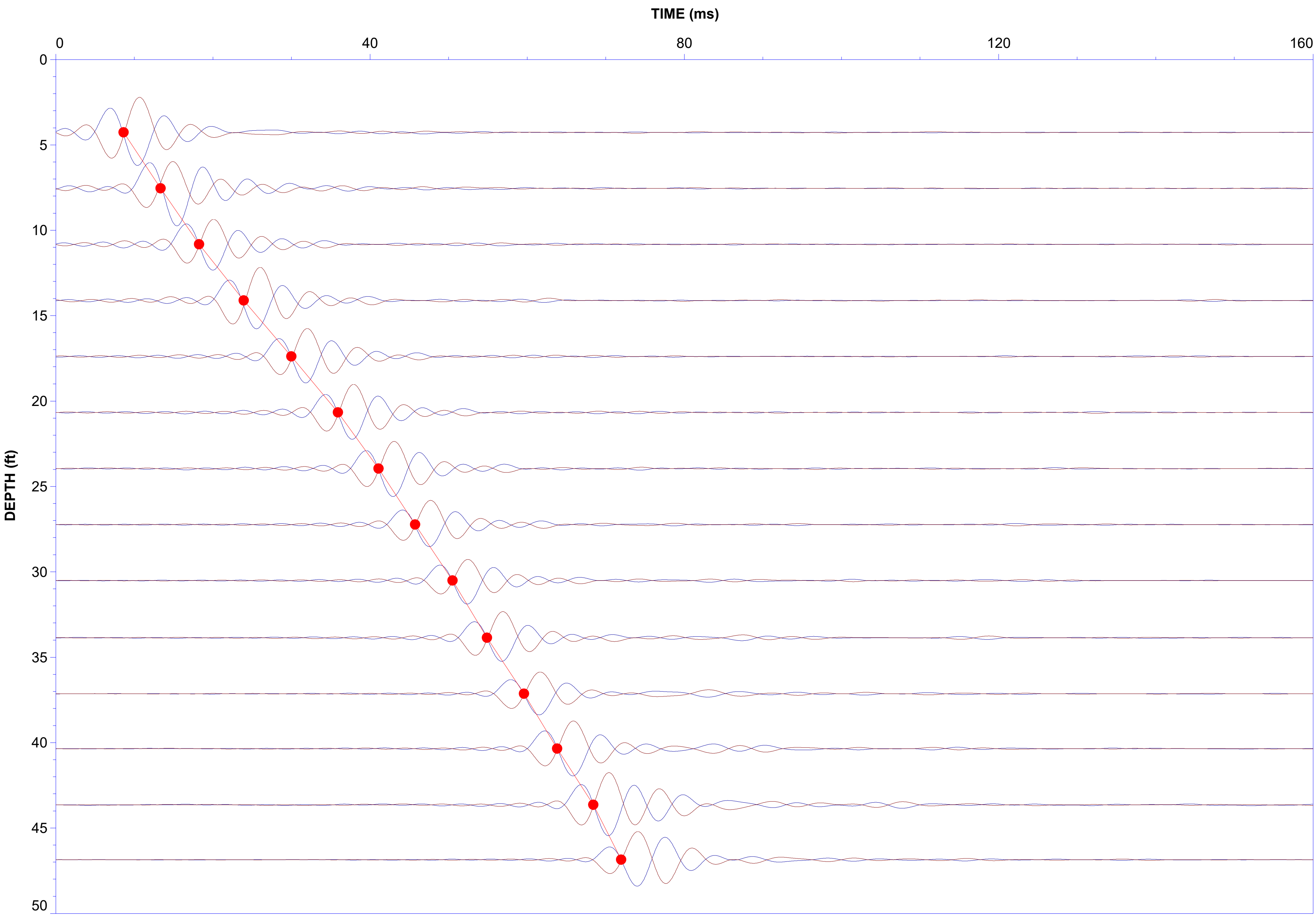
Job No:	23-59-26504
Client:	GRI
Project:	PGE Harborton
Sounding ID:	CPT-01
Date:	9/15/2023
Seismic Source:	Beam
Seismic Offset (ft):	2.62
Source Depth (ft):	0.00
Geophone Offset (ft):	0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
4.26	3.61	4.46			
7.55	6.89	7.37	2.91	4.70	619
10.83	10.17	10.50	3.13	4.91	638
14.11	13.45	13.71	3.20	5.69	563
17.39	16.73	16.94	3.23	6.07	532
20.67	20.01	20.18	3.25	5.92	549
23.95	23.29	23.44	3.26	5.17	629
27.23	26.57	26.70	3.26	4.64	703
30.51	29.86	29.97	3.27	4.76	687
33.86	33.20	33.31	3.33	4.41	757
37.14	36.48	36.58	3.27	4.70	696
40.35	39.70	39.78	3.21	4.22	760
43.64	42.98	43.06	3.27	4.61	711
46.85	46.19	46.27	3.21	3.53	910

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 23-59-26504 Client: GRI Project: PGE Harborton Analysis: Shear Wave Sounding: CPT-01 Filter: BP 10 - 200 Hz
Date: 15-Sep-2023 Cone: 1008:T1500F15U35



Soil Behavior Type (SBT) Scatter Plots

ConeTec

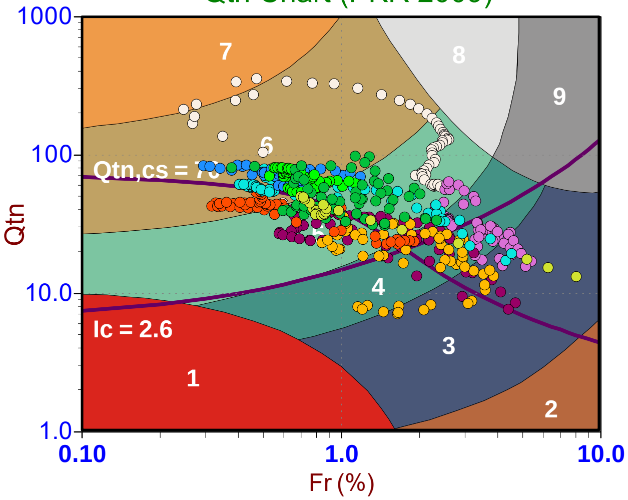
CONETEC

GRI

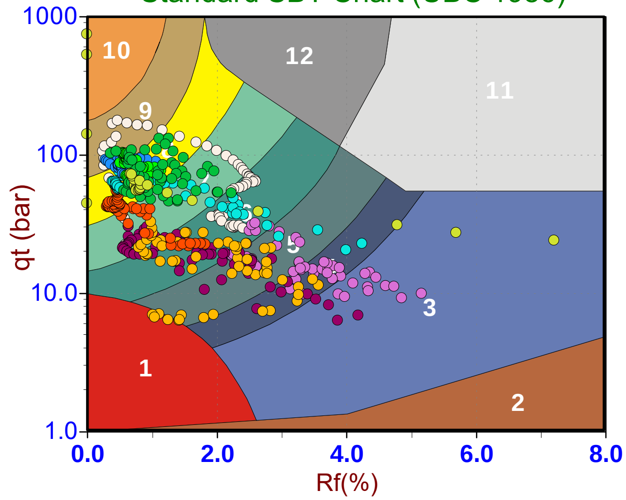
Job No: 23-59-26504
Date: 2023-09-15 08:50
Site: PGE Harborton

Sounding: CPT-01
Cone: 1008:T1500F15U35

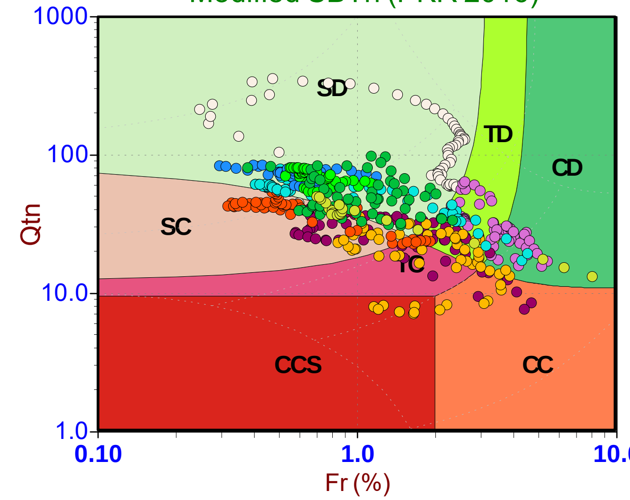
Qtn Chart (PKR 2009)



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

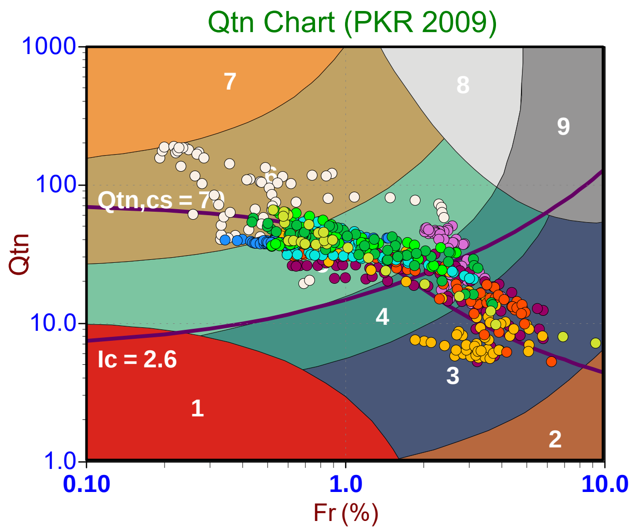
Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

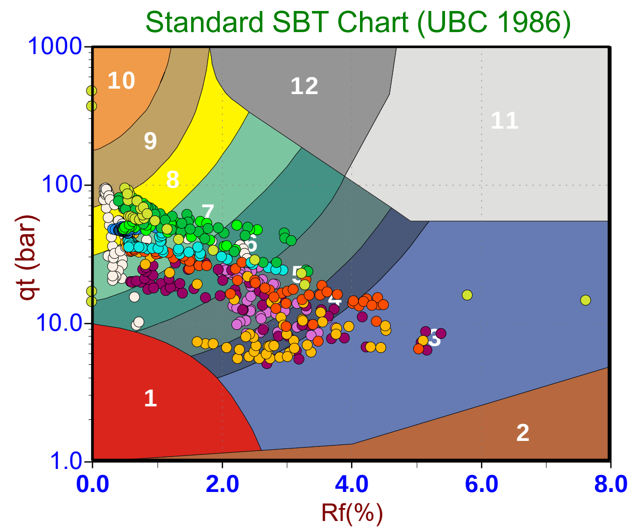
CONETEC

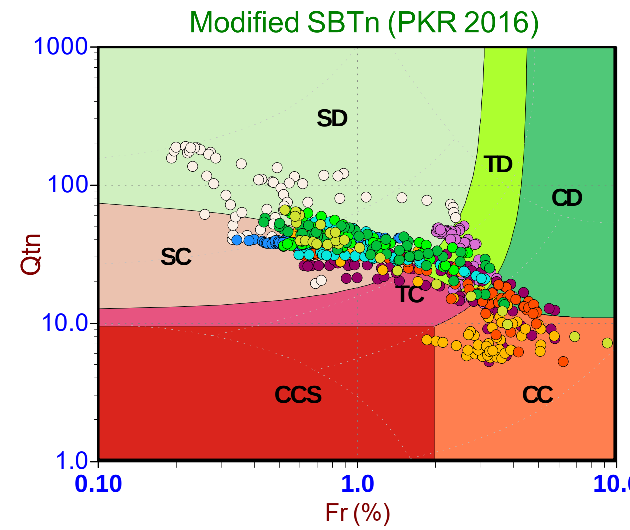
GRI

Job No: 23-59-26504
Date: 2023-09-15 11:02
Site: PGE Harborton

Sounding: CPT-02
Cone: 1008: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
- 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)

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

ConeTec

Job No: 23-59-26504
Client: GRI
Project: PGE Harborton
Start Date: 2023-09-15
End Date: 2023-09-15

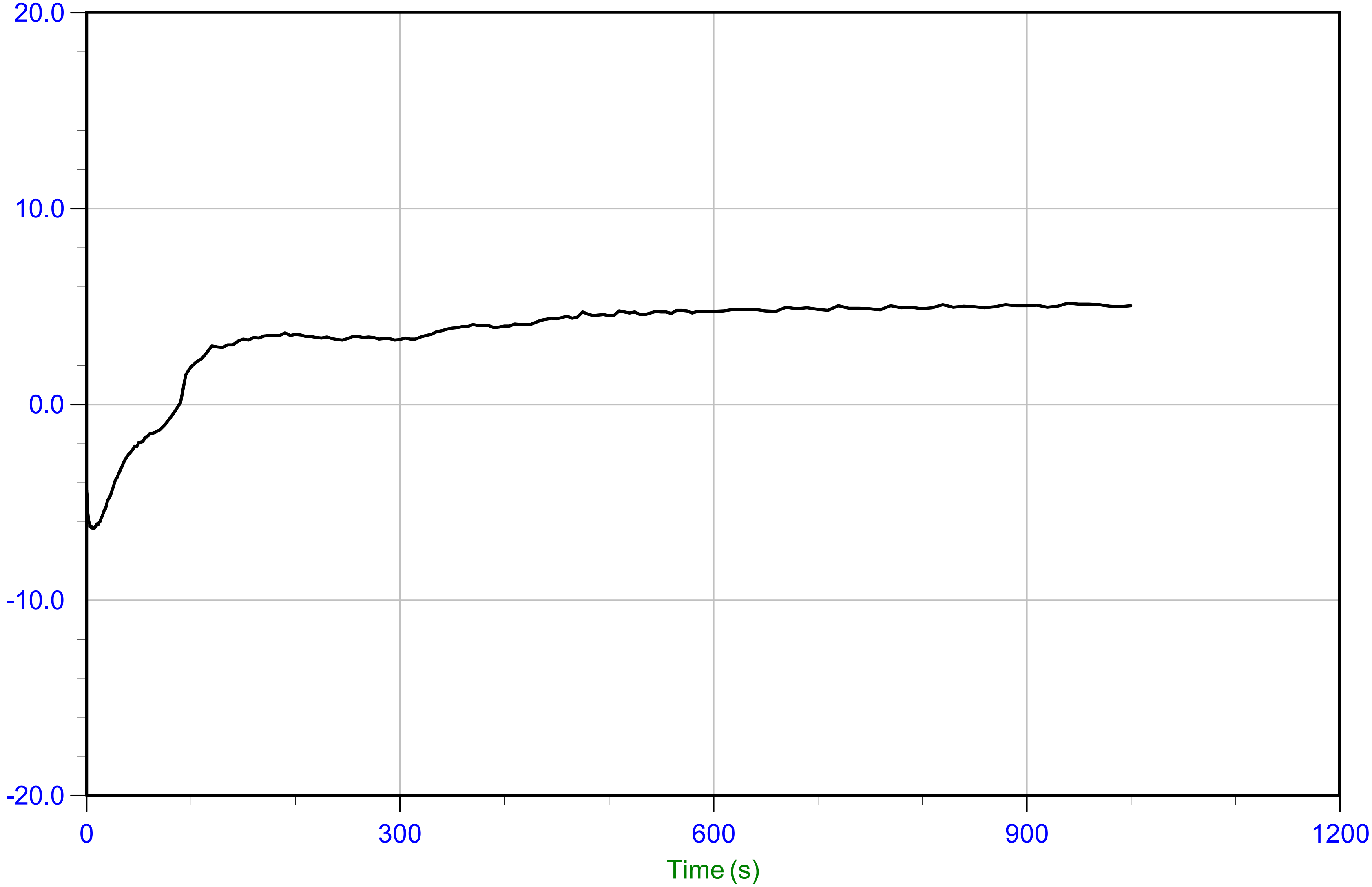
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U_{eq} (ft.)	Calculated Phreatic Surface (ft.)	Refer to Notation Number
CPT-01	23-59-26504_SP01	15	1000	10.8	5.0	5.9	
CPT-01	23-59-26504_SP01	15	830	24.0	12.6	11.4	
CPT-01	23-59-26504_SP01	15	760	40.4	28.8	11.5	
CPT-01	23-59-26504_SP01	15	620	47.0	36.0	11.0	
CPT-02	23-59-26504_CP02	15	400	41.0	27.6	13.4	
Total Duration			16.7 min				

GRI

Job No: 23-59-26504
Date: 09/15/2023 08:50
Site: PGE Harborton

Sounding: CPT-01
Cone: 1008:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-26504_SP01.ppd2
Depth: 3.300 m / 10.827 ft
Duration: 1000.0 s

u Min: -6.3 ft
u Max: 5.2 ft
u Final: 5.0 ft

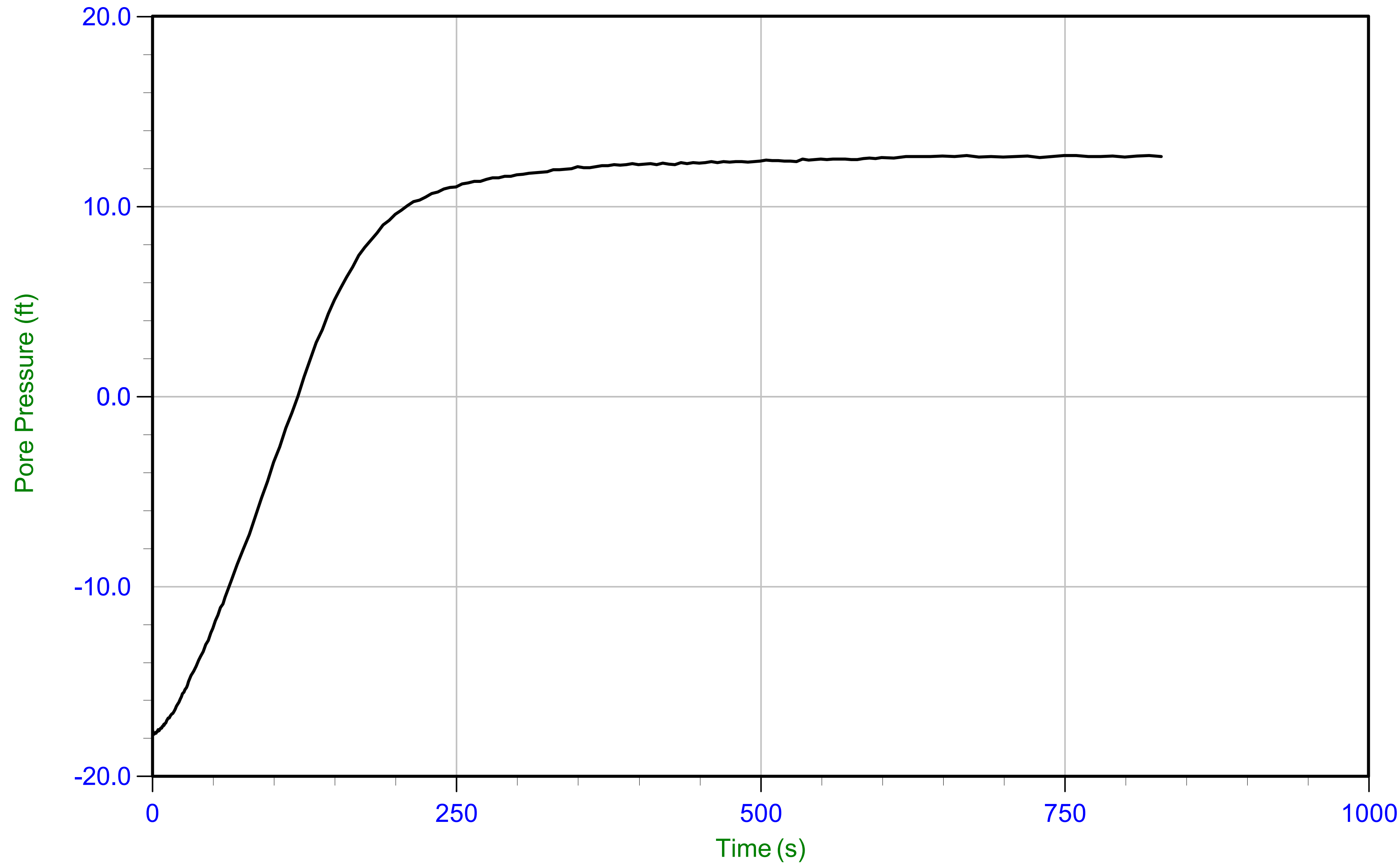
WT: 1.782 m / 5.846 ft
Ueq: 5.0 ft

CONETEC

GRI

Job No: 23-59-26504
Date: 09/15/2023 08:50
Site: PGE Harborton

Sounding: CPT-01
Cone: 1008:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-26504_SP01.ppd2
Depth: 7.300 m / 23.950 ft
Duration: 830.0 s

u Min: -17.8 ft
u Max: 12.7 ft
u Final: 12.6 ft

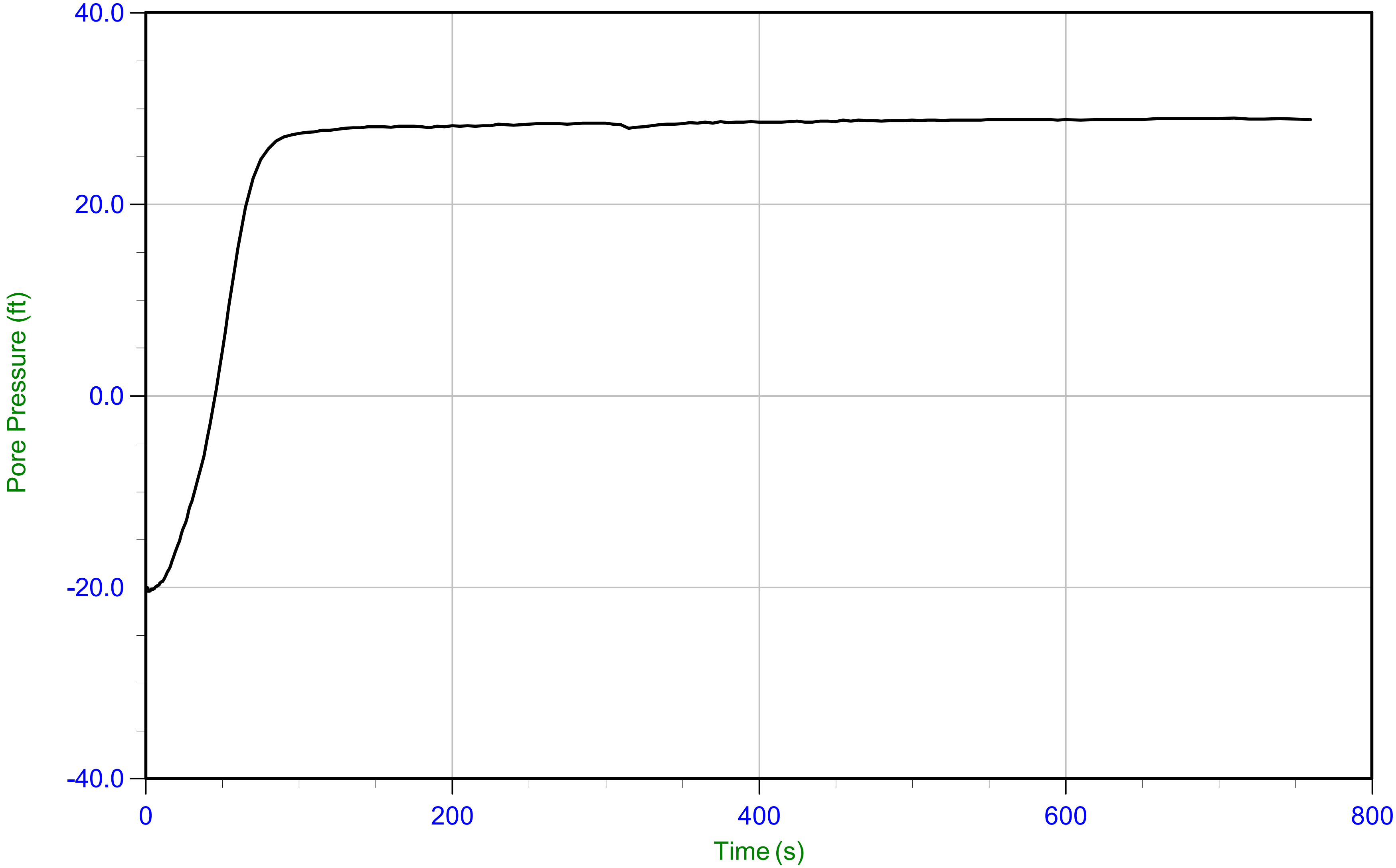
WT: 3.458 m / 11.346 ft
Ueq: 12.6 ft

ConeTec

GRI

Job No: 23-59-26504
Date: 09/15/2023 08:50
Site: PGE Harborton

Sounding: CPT-01
Cone: 1008:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-26504_SP01.ppd2
Depth: 12.300 m / 40.354 ft
Duration: 760.0 s

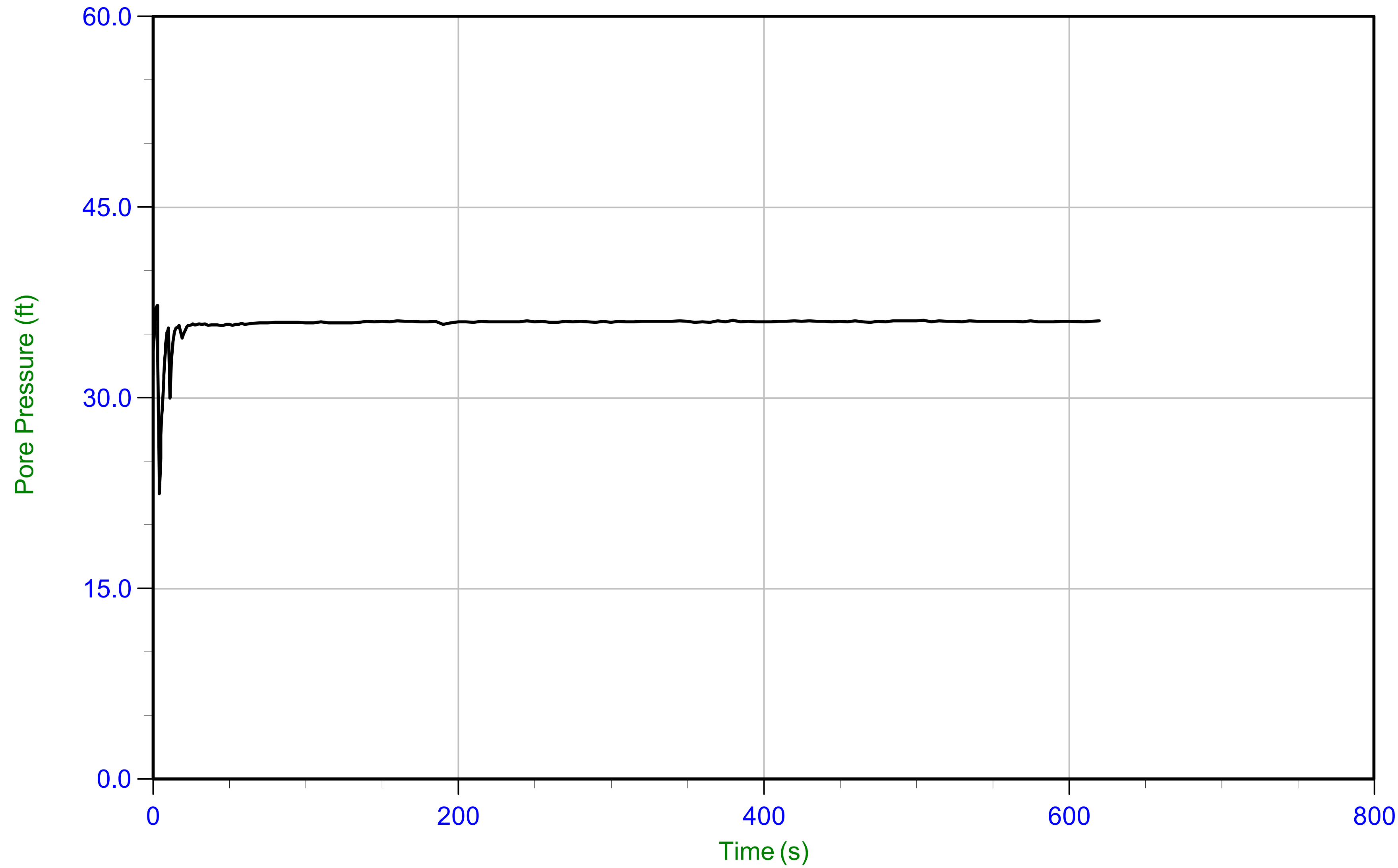
u Min: -20.4 ft
u Max: 29.0 ft
u Final: 28.9 ft

WT: 3.513 m / 11.524 ft
Ueq: 28.8 ft

GRI

Job No: 23-59-26504
Date: 09/15/2023 08:50
Site: PGE Harborton

Sounding: CPT-01
Cone: 1008:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-26504_SP01.ppd2
Depth: 14.325 m / 46.997 ft
Duration: 620.0 s

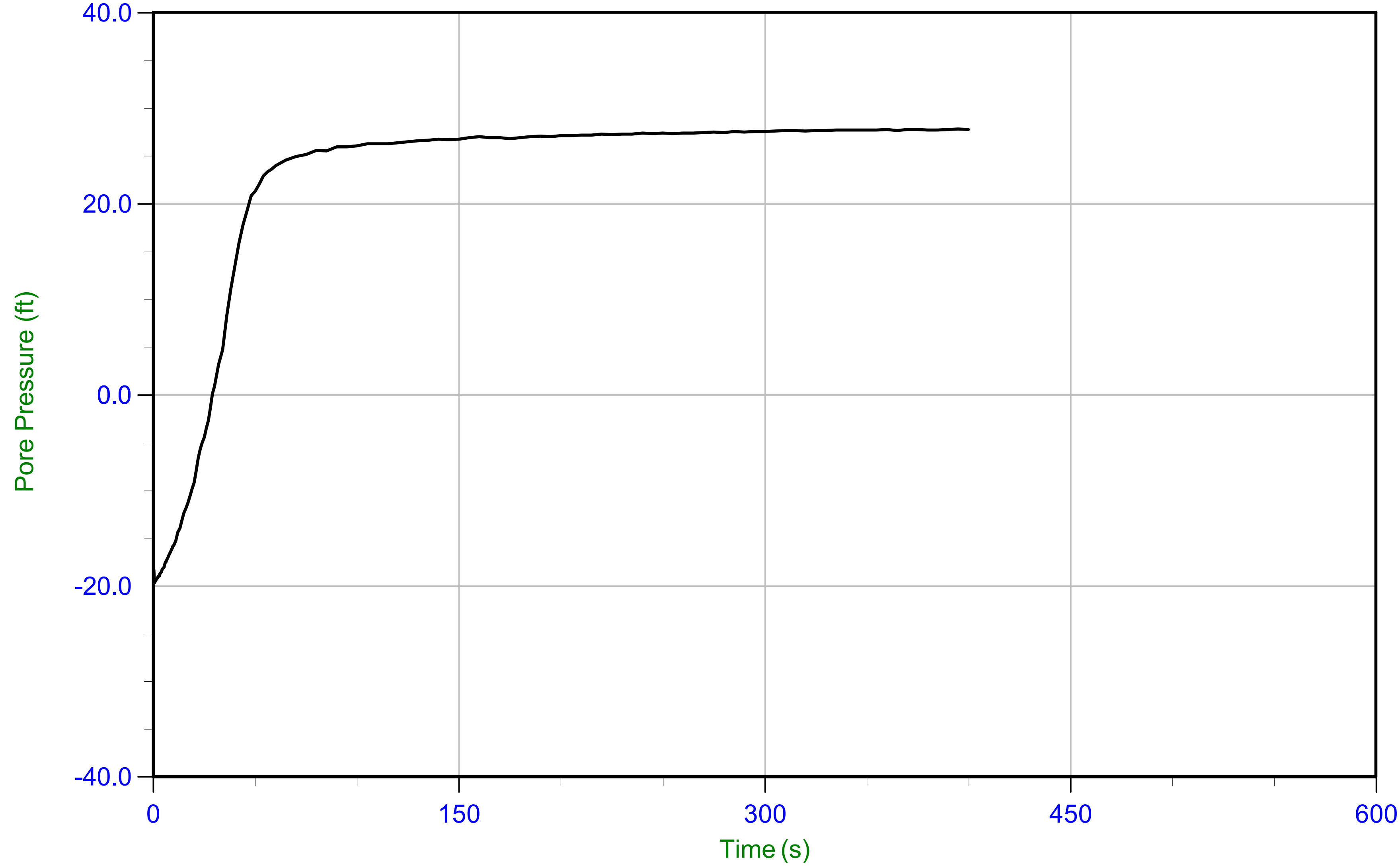
u Min: 22.5 ft
u Max: 37.3 ft
u Final: 36.1 ft

WT: 3.352 m / 10.998 ft
Ueq: 36.0 ft

GRI

Job No: 23-59-26504
Date: 09/15/2023 11:02
Site: PGE Harborton

Sounding: CPT-02
Cone: 1008:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 23-59-26504_CP02.ppd2
Depth: 12.500 m / 41.010 ft
Duration: 400.0 s

u Min: -19.7 ft
u Max: 27.8 ft
u Final: 27.8 ft

WT: 4.081 m / 13.388 ft
Ueq: 27.6 ft

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

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. 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.

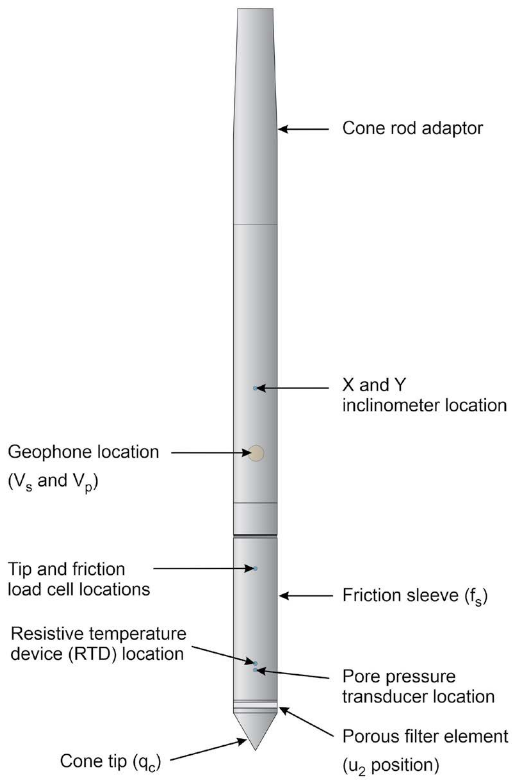


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition system consists of a Windows based computer, 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 encoder that is either portable or integrated into the rig. 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 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, P.K., 2010. The Soil Behavior Type (SBT) classification chart developed by Robertson, P.K., 2010 is presented in Figure SBT. 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.

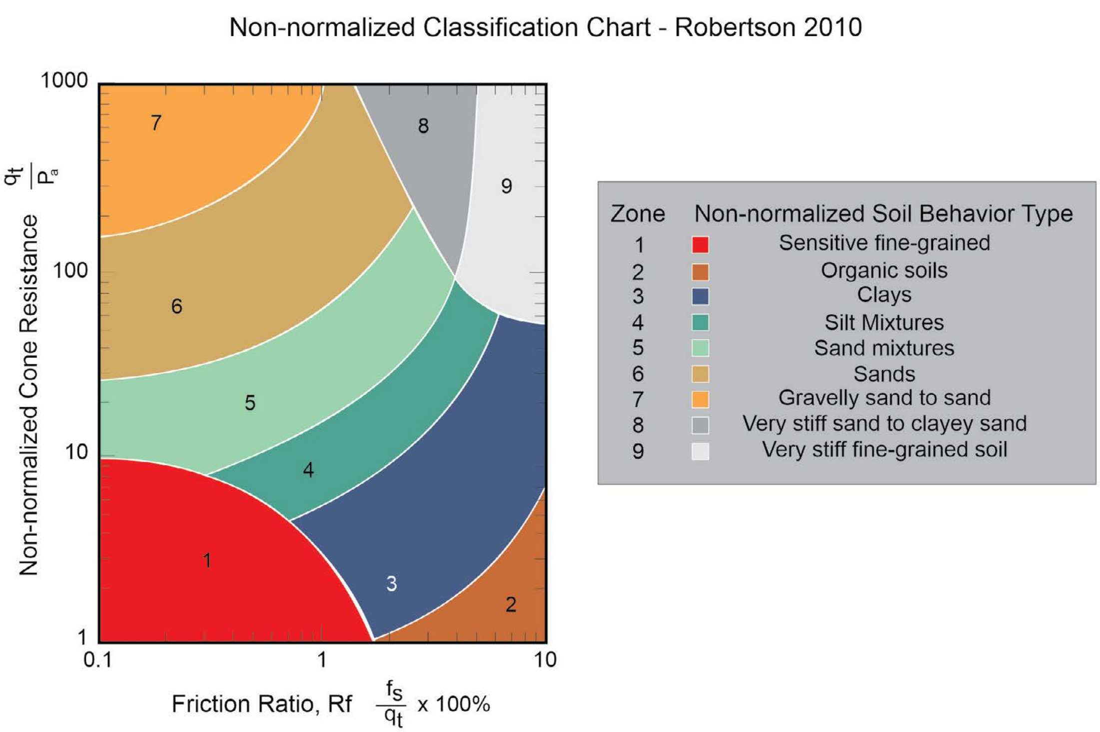


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

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):

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

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 (Rf) 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.

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).

REFERENCES

ASTM D5778-20, 2020, “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils”, ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-20.

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

Robertson, P.K., 2010. Soil behavior type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT’10, Huntington Beach, CA, USA

PORE PRESSURE DISSIPATION TEST

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).

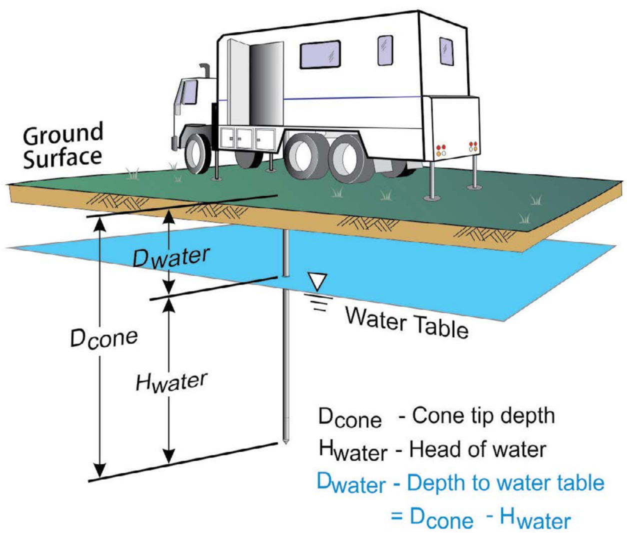


Figure PPD-1. Pore pressure dissipation test setup

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.

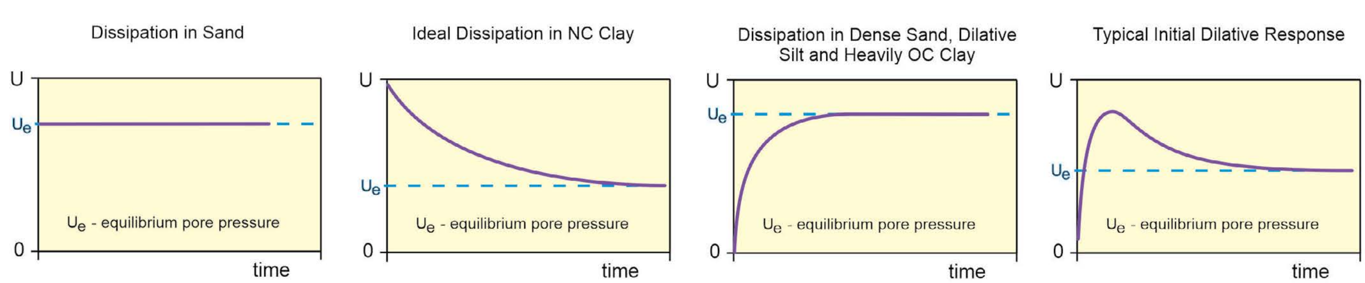


Figure PPD-2. Pore pressure dissipation curve examples

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.

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)
Columns 23-38 contain the sounding Operator
Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location
Columns 17-32 contain the Cone ID
Columns 33-47 contain the sounding number
Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)
Column 2: Tip (q_c), recorded in units selected by the operator
Column 3: Sleeve (f_s), recorded in units selected by the operator
Column 4: Dynamic pore pressure (u), recorded in units selected by the operator
Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "–PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm^2)	Tip Capacity (bar)	Sleeve Area (cm^2)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number
**Outer Cylindrical Area

CONETEC

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 18

Revised February 10, 2023

Prepared by Jim Greig, M.A.Sc, P.Eng (BC, AB, ON)

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of February 10, 2023.

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g., 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not performed.

Corrected tip resistance: $q_t = q_c + (1\text{-}a) \cdot u_2$ (consistent units are required)

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure from behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

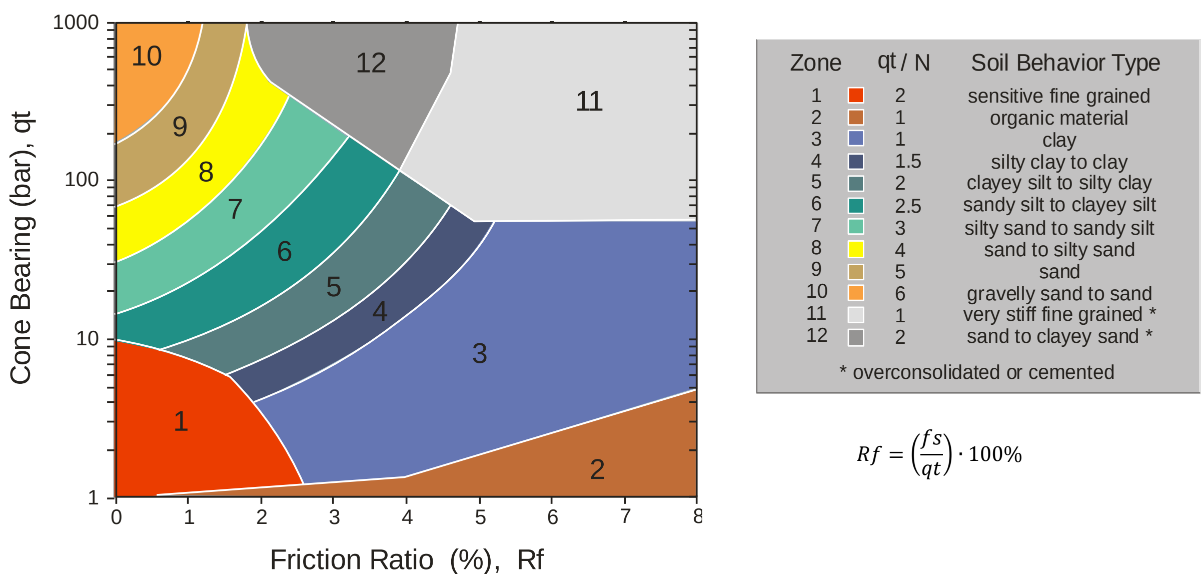
Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline are taken into account as is the appropriate unit weight of water. How this is done depends on where the instruments are zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived from or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 6. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBTn chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Take note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that defined by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised 1986 SBT Chart presented to CPT'10 by Robertson (2010b). It is known as the Updated non-normalized Soil Behavior Chart (also referred to as the Rev SBT Chart (PKR2010) in our output files). This chart was produced to be more in line with all post-1986 Robertson charts having the same 9 soil type zones, a $\log_{10}$ axis for friction ratio, R_f in this case, and a unitless tip resistance axis.

Figure 6 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson. A green palette was selected for the dilative (desirable) side of the chart and a red palette for the contractive side of the chart.



$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

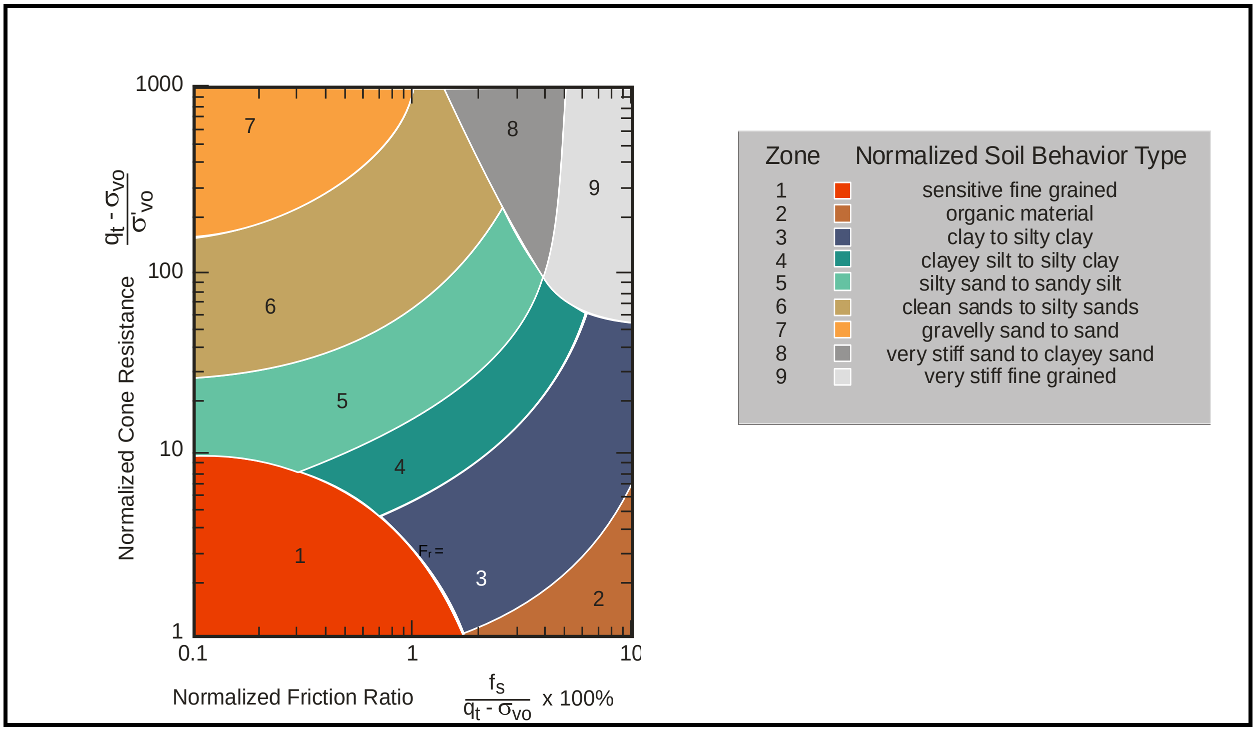


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

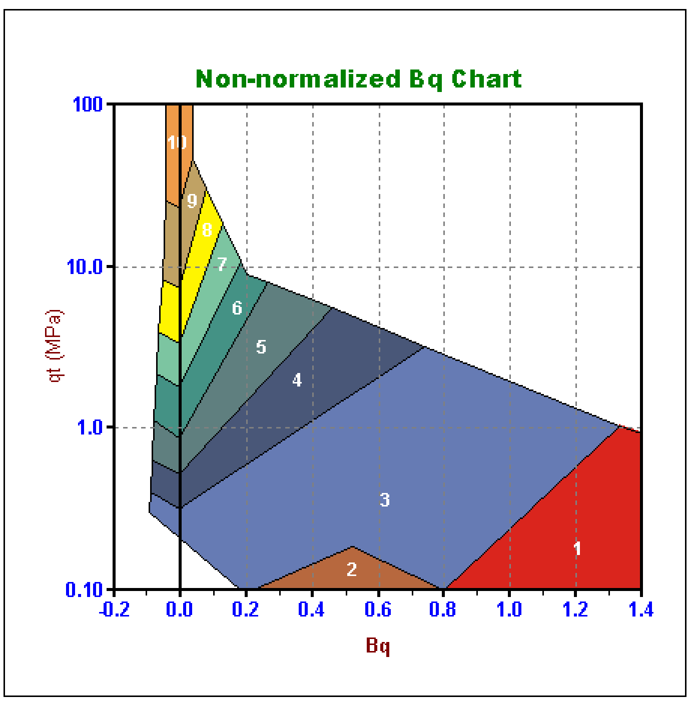


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

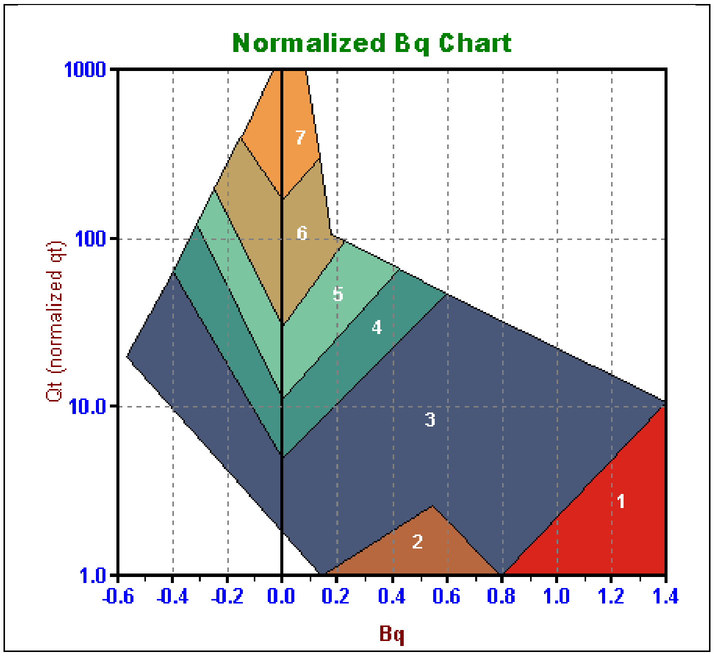


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

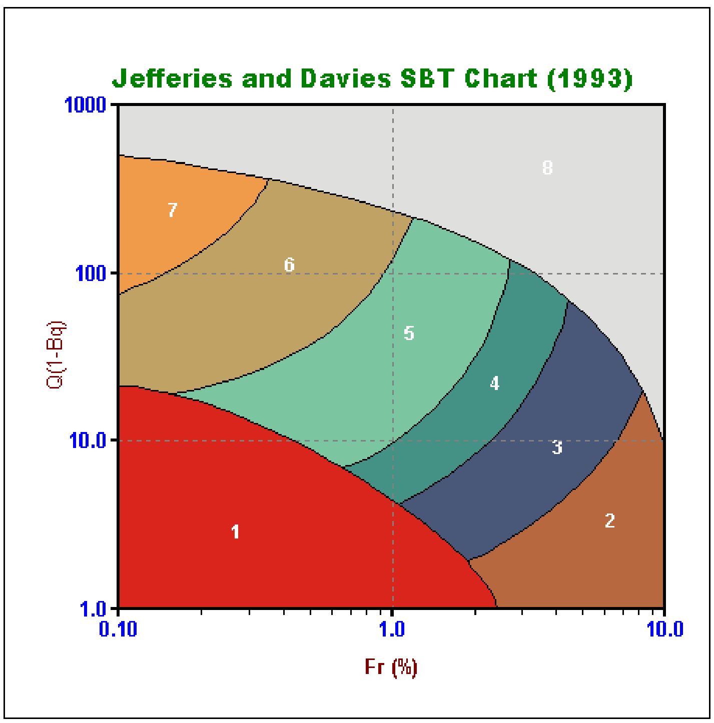


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1-B_q)$ - F_r

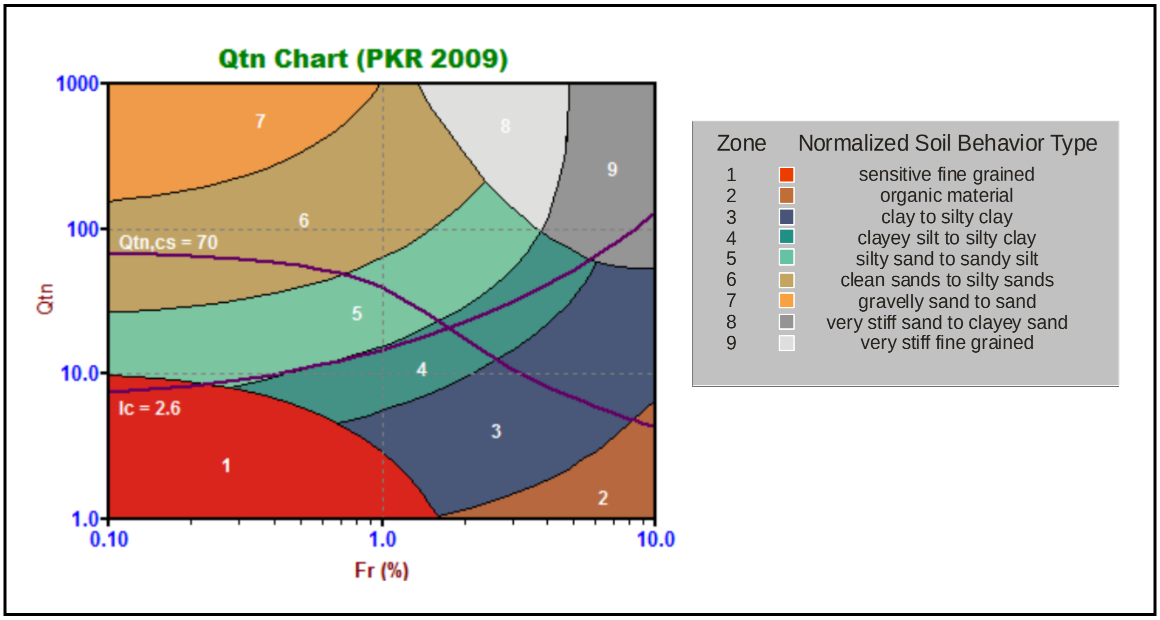


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

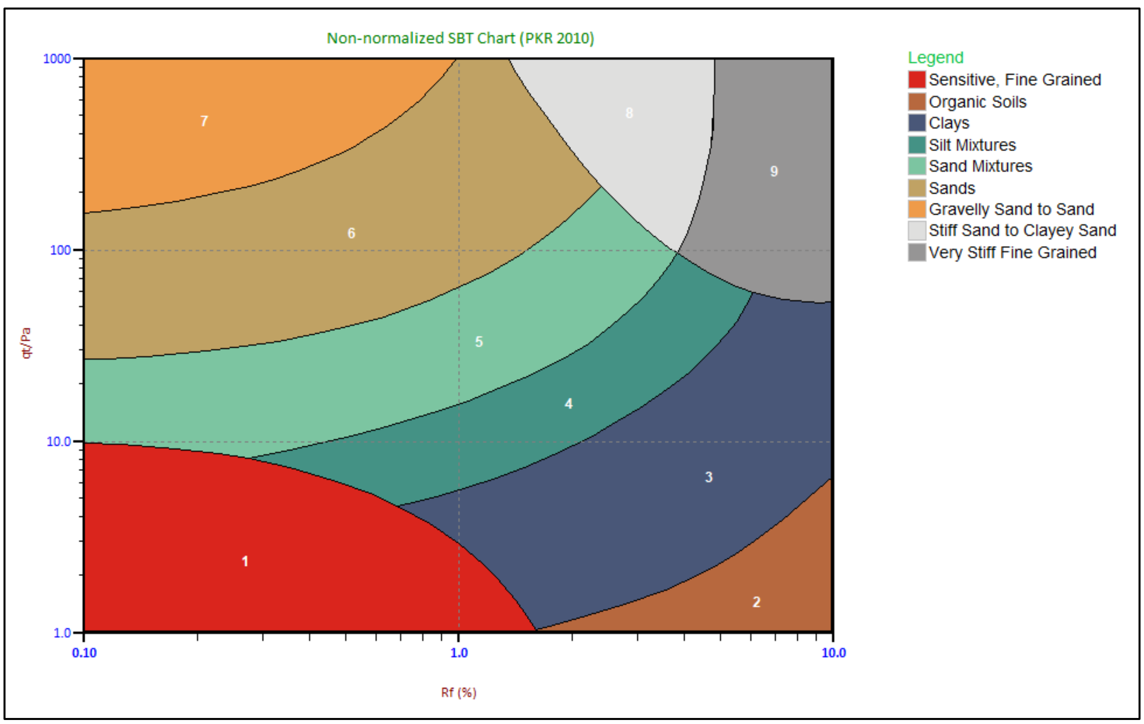


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

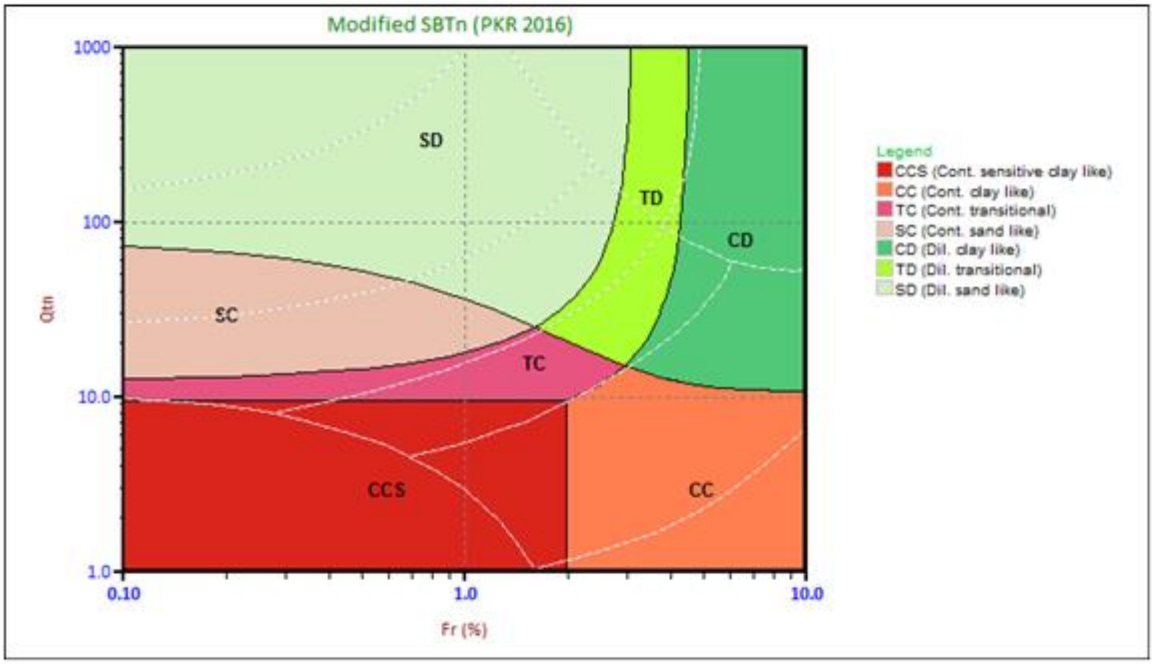


Figure 6. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary, we recommend that the user refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g., drilled out section or data gap).

2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving in an undrained manner (and vice versa).

3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.

4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Tables 1and 1a may be included in the output files delivered with this report.

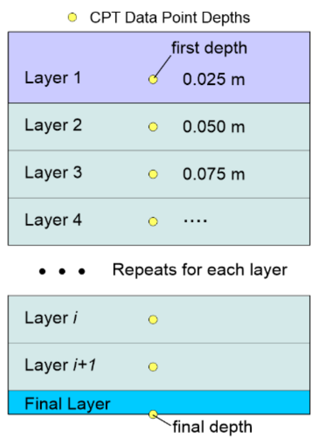
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file base name (from the .COR file) followed by a three or four character indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2, IFI3) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Reference Notes: CK* - Common Knowledge, U* - Unpublished

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer is based on the sounding collar elevation supplied by the client or through a site survey In Sweden a variation of elevation is used where the elevation increases with depth. We refer to this as inverse elevation.	Elevation = Collar Elevation – Depth InverseElevation = Collar Elevation + Depth	CK* N/A
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1 - a) \cdot u_2$ Averaged q_t is not calculated using the average q_c and averaged u values. Averaged q_t is based on the average of the q_t values calculated at each data point.	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s.	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *not an average of individual R_f values*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization using Q_t, now referred to as Q_{t1})	See Figure 2	2, 5

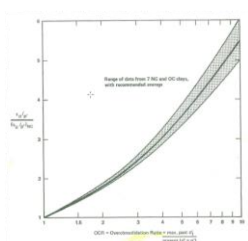
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_{c\ (PKR\ 2009)}$	See Figure 4	15
Modified Non-normalized SBT Chart SBT (PKR2010)	This is a revised version of the simple 1986 non-normalized SBT chart (presented at CPT '10). The revised version has been reduced from 12 zones to 9 zones to be similar to the normalized Robertson charts. Other updates include a dimensionless tip resistance normalized to atmospheric pressure, q_t/P_a, on the vertical axis and a log scale for non-normalized friction ratio, R_f, along the horizontal axis.	See Figure 5	33
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior. Note that ConeTec displays the chart with colors different from Robertson. ConeTec's colors were chosen to avoid confusion with soil type descriptions.	See Figure 6	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) values based on Robertson updated non-normalized Soil Behavior Type Chart (2010b) 6) Mayne f_s (sleeve friction) method 7) Robertson and Cabal 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options.	See references	3, 5, 15, 21, 24, 29, 33

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point. Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point. For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness 	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth.	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressures are determined from one of the following user selectable options: 1) hydrostatic below the water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For the hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is the unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0.	$K_0 = (1 - \sin\Phi')\, OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters.	$C_n = (P_a/\sigma_v')^{0.5}$ where *0.0 < C_n < 2.0 (user adjustable, typically ranging from 1.7 to 2.0)* *P_a is atmospheric pressure (100 kPa)*	4, 12

Calculated Parameter	Description	Equation	Ref
C_q	Overburden stress normalizing factor.	$C_q = 1.8 / [0.8 + (\sigma_v'/P_a)]$ where *$0.0 < C_q < 2.0$ (user adjustable)* *P_a is atmospheric pressure (100 kPa)* *Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.*	3, 12
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \bullet N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/ N_{60} = 8.5\ (1 – I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 – 0.2817Ic)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60}I_c$	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic= C_n \bullet (N_{60} I_c)$ 2) $q_{c1n}/ (N_1)_{60}I_c = 8.5\ (1 – I_c/4.6)$ 3) $(Q_{tn})/ (N_1)_{60}I_c = 10^{(1.1268 – 0.2817Ic)}$	4 5 15, 31
S_u or S_u (N_{kt})	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
S_u or S_u (N_{du}) or S_u ($N_{\Delta u}$)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable.	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hokksund Sand 3) Schmertmann (1978) 4) Jamiolkowski (1985) - All Sands 5) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods 1 through 4) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods 1 through 4 are for sands and method 5 is for silts and clays): 1) Campanella and Robertson 2) Durgunoglu and Mitchel 3) Janbu 4) Kulhawy and Mayne 5) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/q_t $\Delta u/q_t$ du/q_t	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	39

Calculated Parameter	Description	Equation	Ref
B_q	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where: $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* *u_{eq} = equilibrium pore pressure*	1, 2, 5
Net q_t or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	36
q_e or qE or q_E	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v'}$	36
Q_t or Norm: Qt or Q_{t1}	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}. This parameter was renamed to Q_{t1} in Robertson, 2009. Without normalization limits this parameter calculates to very high unrealistic values at low stresses.	$Qt = \frac{qt - \sigma_v}{\sigma_v'}$	2, 5, 15
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-B_q) Q(1-B_q) + 1	Q(1-B_q) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter. Later papers added the +1 term to the equation.	$Q \cdot (1 - Bq)$ $Q \cdot (1 - Bq) + 1$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_{t1}, defined above*	6, 7, 34
q_{c1}	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	21
q_{c1} (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\ (0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
q_{c1} (C_n)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
q_{c1} (C_q)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\ (Cq) = C_q * q_t$ (some papers use q_c)	5, 12
q_{c1n}	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, or 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3

Calculated Parameter	Description	Equation	Ref
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. Ic(RW1998) is different from that of Jefferies and Davies (7) and is different from Ic(PKR2009).	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining* I_c *And* *Fr is in percent* P_a *= atmospheric pressure* *n has the following distinct values:* *0.5, 0.75 and 1.0* *and is determined in an iterative manner based on the resulting* I_c *in each iteration* *Note that NCEER replaced 0.75 with 0.70*	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	I_c (PKR 2009) = $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	n (PKR 2009) = $0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	15
Q_{tn} (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Q_{tn} (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where* P_a *= atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
CD	The contractive / dilative boundary on Robertson's Modified SBTn (contractive/dilative) Chart shown in Figure 6 above. The boundary is marked as CD = 70 on the chart in the relevant paper. Similar to the $Q_{tn,cs}$ = 70 line in Figure 4.	$CD = 70 = (Q_{tn} - 11) (1 + 0.06F_r)^{17}$ lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5) (1 + 0.06F_r)^{17}$	30

Calculated Parameter	Description	Equation	Ref
I_B	Hyperbolic fit defining the boundary between SBT soil types proposed by Schneider as a better fit than the I_c circles. I_B = 32 represents the boundary for most sand like soils. I_B = 22 represents the upper boundary for most clay like soils. The region between I_B=22 and I_B=32 is the "transitional soil" zone.	$I_B = 100\ (Q_{tn} + 10) / (70 + Q_{tn}\ F_r)$	30
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) This method uses mean normal stresses based on a uniform value of K_0 or a calculated K_0 using methods described elsewhere in this document	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods 1) General method 2) 1st order approximation using q_tNet (clays) 3) 1st order approximation using Δu_2 (clays) 4) 1st order approximation using q_e (clays) 5) Based on Vs	All stresses in kPa 1) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)^{m'} (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1 + (I_c / 2.65)^{25}}$ 2) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)$ 3) $\sigma_p' = 0.54 \cdot (\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ 4) $\sigma_p' = 0.60 \cdot (q_t - u_2)$ 5) $\sigma_p' = (Vs/4.59)^{1.47}$	19 20 20 20 18
OCR OCR(JS1978) YSR(Mayne2014) YSR (qtNet) YSR (deltaU) YSR (qe) YSR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on 1) Schmertmann (1978) method involving a plot plot of $S_u/\sigma_v' / (S_u/\sigma_v')_{NC}$ and OCR  2) based on Yield stresses described above 3) approximate version based on qtNet 4) approximate version based on Δu 5) approximate version based on effective tip, q_e 6) approximate version based on shear wave velocity, V_s and σ_v' 7) based on Qt	1) requires a user defined value for NC Su/P_c' ratio 2 through 5) *based on yield stresses* 6) YSR (Vs) = $\sigma_p'(Vs) / \sigma_v'$ 7) OCR = $0.25 \cdot (Qt)^{1.25}$	9 19 20 20 20 18 32
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart. Note that Figured 5.59 from reference 5, Lunne, Robertson and Powell, (LRP) has an error. The X axis values are too high by a factor of 10. The plot is based on Baldi's (not Bellotti as cited in	Based on Figure 5.59 in the reference	5, 37

Calculated Parameter	Description	Equation	Ref
	LRP) original Figure 3 where the X axis is: $\frac{q_c}{\sqrt{\sigma_v'}}$ (both in kPa) with a range of 200 to 3000. Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values. Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^{0.5}$ The two expressions are not the same: they differ by a factor of $\frac{\sqrt{P_a}}{P_a}$. With P_a taken to be 100 kPa the factor is 1/10. Substituting typical values of 200 bar (20000 kPa) for q_c and 225 kPa for σ_v' one gets: 20000 / 15 = 1333.33 for Bellotti's axis and $(200/1)(100/225)^{0.5}$ = 200 * (10/15) = 133.3 for LRP's axis (noting that P_a = 1 bar) showing a factor of 10 difference.		
Es or E_s Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. E_s is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $$\sigma_m' = \frac{1}{3}(\sigma_v' + \sigma_h' + \sigma_h')$$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \cdot \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress $\Delta u / \sigma_v$	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	39
Delta U/EStress, P Value, Excess Pore Pressure Ratio $\Delta u/\sigma_v'$	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma_v'}$ *where:* $\Delta u = u - u_{eq}$	25, 25a
Su/EStress S_u/σ_v'	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su(N_{kt}) / \sigma_v'$	9, 23
Vs or V_s	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or V_p	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\left(\frac{layer\ thickness}{layer\ shear\ wave\ velocity}\right)}$ $V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\,(layer\ travel\ times)}$	38
G_{max}	G_{max} determined from SCPT shear wave velocities (not estimated values). Note that seismic data (V_s) is collected over set depth intervals (typically 1 meter). Each data point over the test segment is assigned the same V_s value. Since soil density changes with depth, slightly different G_{max} values may be calculated over the test depth interval.	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/G_{max}	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30
qUlt	A site specific and client specific parameter for estimating the limiting stress for “crane walk” accessibility	$q_{ult} = CraneWalkFactor \cdot S_u$ Where: *CraneWalkFactor* is client provided	U*
Estimated G_o	Estimated value for small strain shear modulus	$G_o = 0.0188[10^{(0.55Ic + 1,68)}](q_t - \sigma_v)$	15
Estimated E_{25}	Estimated value for Young’s Modulus, E, at a 25% working load	$E_{25} = \alpha_E\,(qtNet)$ where $\alpha_E = 0.015[10^{(0.55Ic + 1,68)}]$	15
k_{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I_c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04Ic)}$ in m/s For $3.27 < Ic < 4.0$: $k = 10^{(-4.52 - 1.37Ic)}$ in m/s	35
M or D’ Constrained Modulus	Constrained Modulus based on 1) Robertson, M 2) Mayne, D’	1) Robertson $M = \alpha_M\,(q_t - \sigma_v)$ $I_c > 2.2$ (fine grained) $\alpha_M = Qt$ when $Qt < 14$ $\alpha_M = 14$ when $Qt > 14$ $Ic < 2.2$ (coarse grained) $\alpha_M = 0.0188\,[10^{(0.55Ic + 1.68)}]$ $D' = \alpha_D\,(qt - \sigma_v)$ where $\alpha_D = 5$	32 23

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT} or K_s	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC – 5))$	10
K_{CPT} or K_c (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = – 0.403\ I_c^4 + 5.581\ I_c^3 – 21.63 I_c^2 + 33.75\ I_c – 17.88$	3, 10
K_c (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = – 0.403\ I_c^4 + 5.581\ I_c^3 – 21.63 I_c^2 + 33.75\ I_c – 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}I_c$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\ (1 – I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta=1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta=1.2$ 5% < FC < 35% $\alpha = exp[1.76 – (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{S_u(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{S_u(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
S_u (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress	$S_u(Liq) = \sigma_v' \cdot \left(\frac{S_u(Liq)}{\sigma_v'}\right)$	16
Cont/Dilat Tip	Contractive / Dilative q_{c1} Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ q_{c1} is calculated from specified q_t(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\ [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\ [q_{c1ncs}/1000]^3 + 0.08$	10
Kg or K_g	Small strain Stiffness Ratio Factor, K_g	$[G_{max}/q_t]/[q_{c1n}^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
K_g*	Revised K_g factor extended to fine grained soils (Robertson).	$K_g^* = (G_o / q_n)(Q_{tn})^{0.75}$ where q_n is the net tip resistance = $q_t - \sigma_v$	30
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Q_{tn} chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP distance calculation		25
URS NP Q_{tn}	Normalized tip resistance (Q_{tn}) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

No.	Reference
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APPENDIX C

Seismic Refraction Survey Report by Earth Dynamics, LLC

Report on

Geophysical Exploration for PGE Harborton Project Portland, OR

Data Acquisition Date: September 14, 2023
Report Date: September 25, 2023

Prepared for:

GRI
16950 SW Upper Boones Fry
Tigard, OR 97224

Prepared by:

EARTH DYNAMICS LLC
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(503) 227-7659
Project No. 23220

1.0 - Introduction

Portland General Electric (PGE) is planning improvements to the Harborton transmission lines in Portland, Oregon. The planned work is in Forest Park near Highway 30. GRI engaged Earth Dynamics LLC to conduct geophysical explorations to supplement a geotechnical investigation for the project. The purpose of the explorations is to determine the depth to and the compressional wave velocity of basalt bedrock at the site. These data are needed for site development and to determine the rippability of the basalt.

This work was requested and authorized by Mr. Jon Huffman of GRI. The geophysical field work was conducted on September 14, 2023 under the supervision of Mr. Daniel Lauer of Earth Dynamics LLC. Seismic refraction data were acquired along one profile. The desired location and length of the profile was specified by GRI. This report describes the methodology and results of the geophysical investigation.

2.0 - Method

2.1 - Seismic Refraction

The seismic velocity of soil and rock is a function of the density and elastic properties of the material. Therefore, variations in subsurface materials can be inferred from analysis of the seismic velocity. Application of the method is limited to areas where seismic velocity increases or is constant with depth. Low velocity zones, which are common in basalt, cannot be resolved with seismic refraction.

A seismic refraction exploration consists of measuring the time required for a seismic wave to travel from a seismic source to a receiving transducer. A sledgehammer, large weight dropped, or explosive device is typically used for the seismic source and vertical geophones are used as receiving transducers. A seismograph records signals from the geophones. By analyzing the arrival time of the seismic wave as a function of distance from the seismic source, the seismic velocities of the underlying soil/rock units and the depth to geologic contacts can be determined. The seismic refraction method requires that seismic sources be placed at each end of the geophone array. Intermediate and off end sources are also often used to increase resolution and penetration. The depth of penetration is typically one-quarter to one-third of the geophone array length, and lateral resolution is typically one-half of the geophone spacing.

The seismic refraction survey for this study was conducted using a Seismic Source 24-channel DAQ Link IV seismograph equipped with twenty-four vertical geophones. A 20-pound sledgehammer was used as the seismic source at six to seven shot points for each array. Data from several hammer impacts were acquired at each shot point. Stacking multiple impacts enhances the seismic signal by reducing random noise and makes picking of the first arrival time more accurate.

The seismic data are analyzed using SeisOpt@2D Ver. 6.0 by Optim Software. SeisOpt@2D uses a forward modeling global optimization technique. The technique consists of creating a finite element velocity model through which travel times are computed. The computed times are compared with the observed data. Thousands of iterations are completed to find the velocity model with the minimum travel time error. Comparison of the computed travel times to the measured values provides an indication of the validity of the model. Several velocity models are run using different grid resolution and depth values to obtain the best result for each data set. SeisOpt generates xyz data files that are input to Surfer® 17 for contouring, scaling, and data presentation. The SeisOpt modeling technique is generally superior to discrete layer modeling because lateral, as well as vertical variations can be resolved, and gradual increases in seismic velocity with depth can be quantified.

For this study, data were acquired for one extended profile using a 24-channel geophone array. A geophone spacing of five feet and an array length of 115 feet was used. Three consecutive geophone arrays were deployed to extend the length of the exploration to 345 feet. Data are acquired using six to seven shot points for each array. The data acquired for the three arrays are combined and processed to create one two-dimensional profile designated as S1.

2.2 - Location and Elevation Survey

Horizontal position data were obtained with a Trimble GEOXH 6000 GPS receiver equipped with a Tornado external antenna. The position data were post-processed to increase the accuracy of the GPS positions. GPS location data were recorded at the end points of each profile. Recorded GPS data are summarized in Table 2-2. The GPS data are displayed in degrees, decimal minutes Latitude and Longitude using the WGS 1984 datum. No absolute elevation data are available for this site. Therefore, elevations from the GPS data were used. The geophone elevations along the profile were surveyed with a rod and level to determine relative elevations. The elevations for profiles S1 are tied to S1-345' with a reported GPS elevation of 119.5 feet. The differential correction software suggests that the accuracies of the elevations used are better than ±1 foot.

Table 2-2. GPS Position and Elevation Data for Seismic Refraction Profile Endpoints and Selected Borings. (WGS 1984).

Profile Location	Latitude	Longitude	GPS Elevation (MSL- ft)	Estimated Accuracy Horiz/Vert (±ft)
S1 – 0'	45° 36.6835'N	122° 47.8824'W	N/A	0.5/N/A
S1 – 115'	45° 36.7018'N	122° 47.8936'W	N/A	0.8/N/A
S1 – 230'	45° 36.7178'N	122° 47.9076'W	N/A	0.8/N/A
S1 – 345'	45° 36.7304'N	122° 47.9277'W	119.5	0.3/0.5

3.0 - Results

The approximate locations of the geophysical profiles are shown in the Google Earth image in Figure 3-1. Computed seismic velocity models with interpreted geology for the seismic refraction profiles are contained in Appendix A.

Figure 3-1. Site plan showing approximate locations of seismic refraction profile.

4.0 - Discussion

The seismic refraction data acquired in this study are generally of good quality. The stacking of several hits at each shot point allows for good confidence in picking each first arrival.

Earth Dynamics LLC has completed numerous seismic refraction studies in Portland and surrounding areas. In many cases it is observed that the minimum velocity of un-weathered and fractured basalt is greater than approximately 5,000 feet per second (ft/sec). Weathered, fractured and/or residual/decomposed basalt typically has a seismic velocity range of 3,000 to 5,000 ft/s. Soils and silts and other unconsolidated sediments typically have a seismic velocity less than 3,000 ft/s.

The interpreted intact basalt bedrock contact is shown with a dashed black line on the geophysical model in Appendix A. The model indicates that intact basalt ranges from approximately two to six feet below the ground surface (bgs) along the length of the profile. Material with a seismic velocity less than 3,000 ft/s is likely soil or silt. Material with a seismic velocity in the range of 3,000 – 5,000 ft/s may be decomposed basalt and/or gravel. Preliminary information from an exploratory boring in the vicinity of S1-32' indicates that basalt bedrock was encountered at a depth of three feet bgs.

Compressional wave (p-wave) seismic velocity is related to ripper performance in the Caterpillar Performance Handbook (2019). Caterpillar performance data for basalt are summarized in Table 4-1. The data in Table 4-1 indicate that basalt with a seismic velocity less than 6,400 ft/s is generally rippable with moderately sized equipment and that basalt with seismic velocities greater than about 10,000 ft/s is generally not rippable. However, basalt rippability is very dependent on the characteristics of particular basalt formations. Basalt which contains interflows, joints or weathered zones may be rippable even when the modeled seismic velocity is greater than 10,000 ft/s.

Table 4-1. Ripper performance in basalt.
(Caterpillar Performance Handbook, Rev 49, 2019)

Ripper Model	P-wave Seismic Velocity (ft/sec)		
	Rippable	Marginal	Non-Rippable
D8R/D8T	<6,400	6,400 - 8,000	>8,000
D9R/D9T	<7,600	7,600 - 8,600	>8,600
D10T2	<8,000	8,000 - 9,000	>9,000
D11	<8,900	8,900 - 9,800	>9,800
D11CD	<9,100	9,100 – 10,100	>10,100

5.0 - Limitations

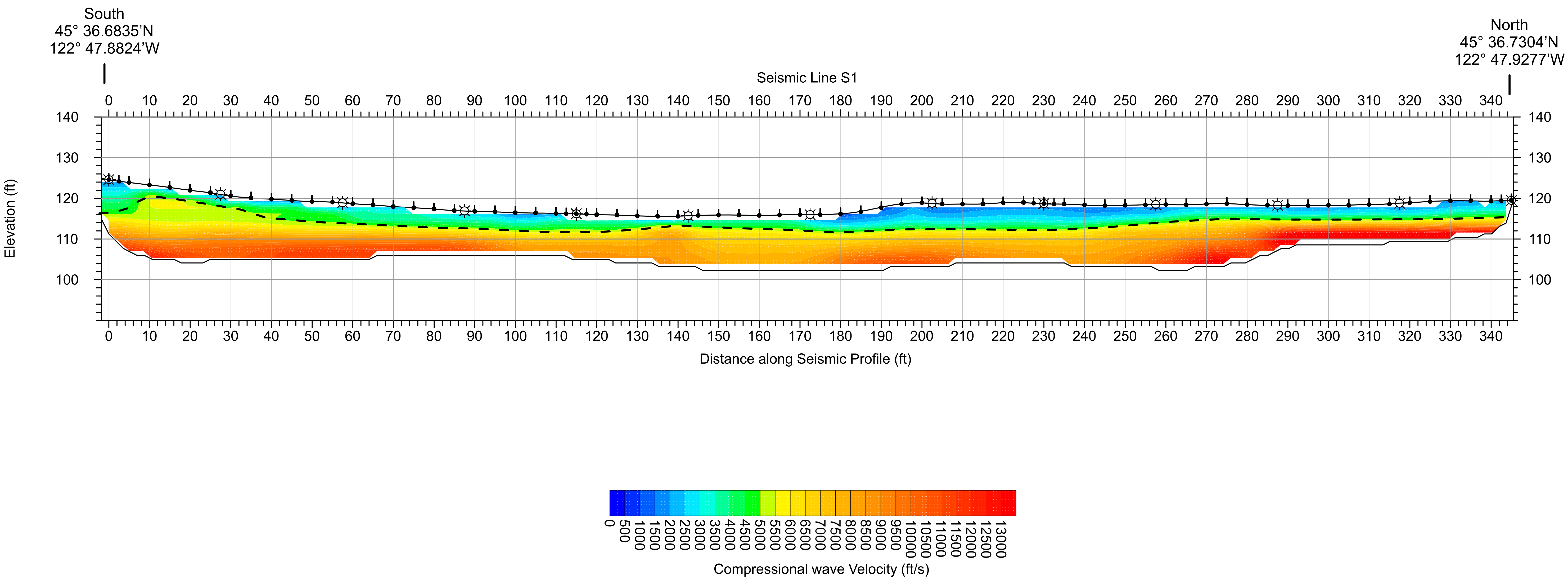
The inversion of seismic refraction data does not produce a unique model. Theoretically, there are an infinite number of models that will fit the data as well as the models presented in this report. Further, many geologic materials have similar seismic velocity. We have presented models and interpretations which we believe to be the best fit given the geology and known conditions at the site. However, no warranty is made or intended by this report or by oral or written presentation of this work. Earth Dynamics LLC accepts no responsibility for damages as a result of decisions made or actions taken based upon this report.

RESPECTFULLY SUBMITTED
EARTH DYNAMICS LLC

Daniel Lauer, M.S.
Principal - Senior Geophysicist

Appendix A

Seismic Refraction Profiles



☼ Shot point

• Geophone

Horizontal Scale: 1" = 25'
Vertical Scale: 1" =25'

Elevations surveyed with level and rod and tied to elevation from GPS data
Horizontal Positions surveyed with Trimble GeoXH 6000 GPS Receiver (Differentially Corrected)

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Portland, OR 97210
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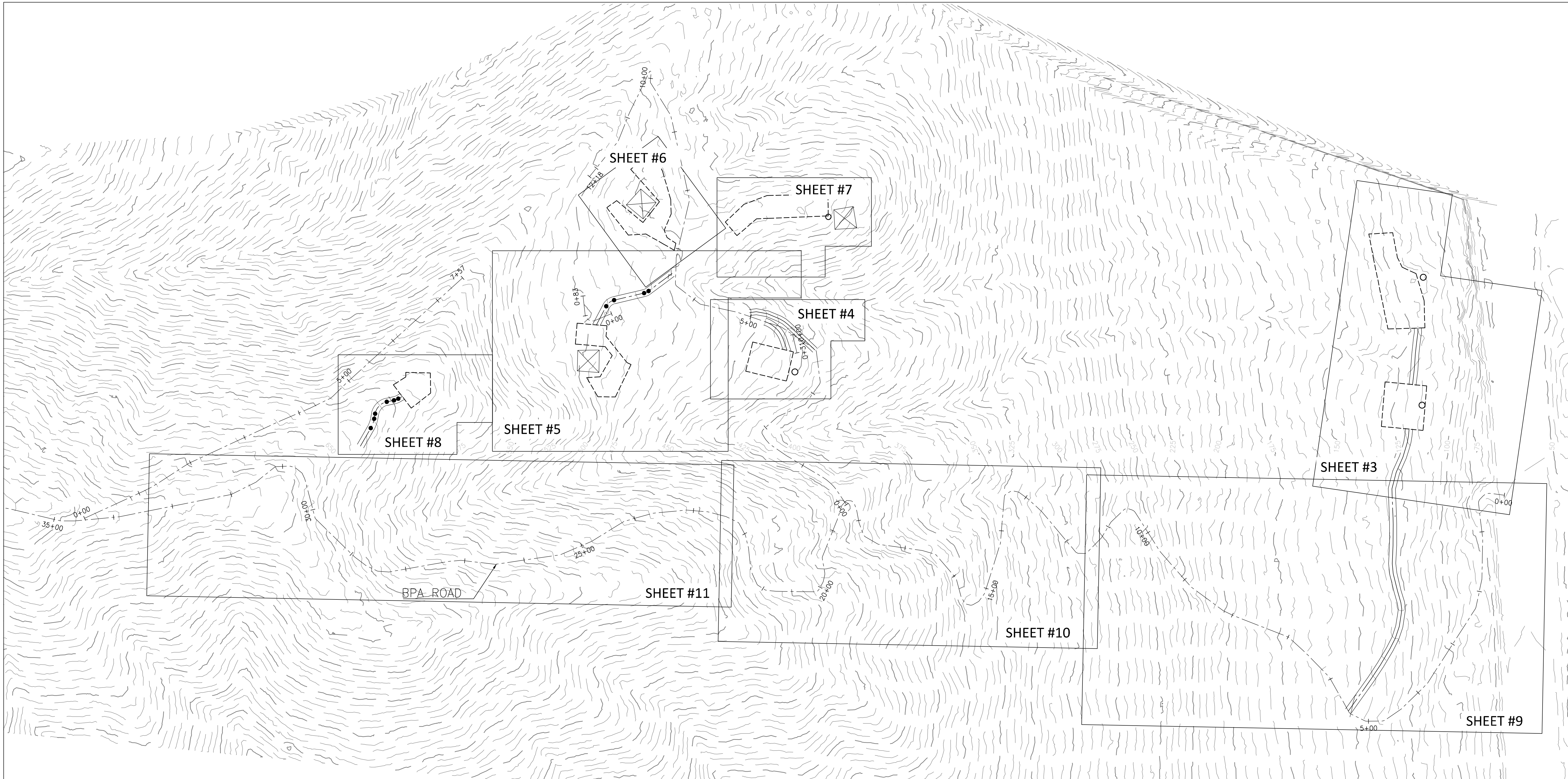
PGE Harborton
Portland, Oregon

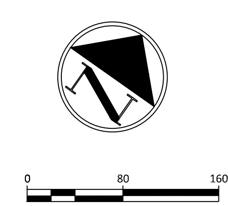
Seismic Refraction Profile S1 - SeisOpt2D Model

Job #:	23220	Date:	9/14/23	Figure:	A1

APPENDIX D

Selected Figures Prepared by DEA and Mackay Sposito for PGE





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OREGON
JULY 10, 2018
PRELIMINARY
EXPIRES: 12/31/2023

REVISIONS:

PGE CONSTRUCTION DRAWING

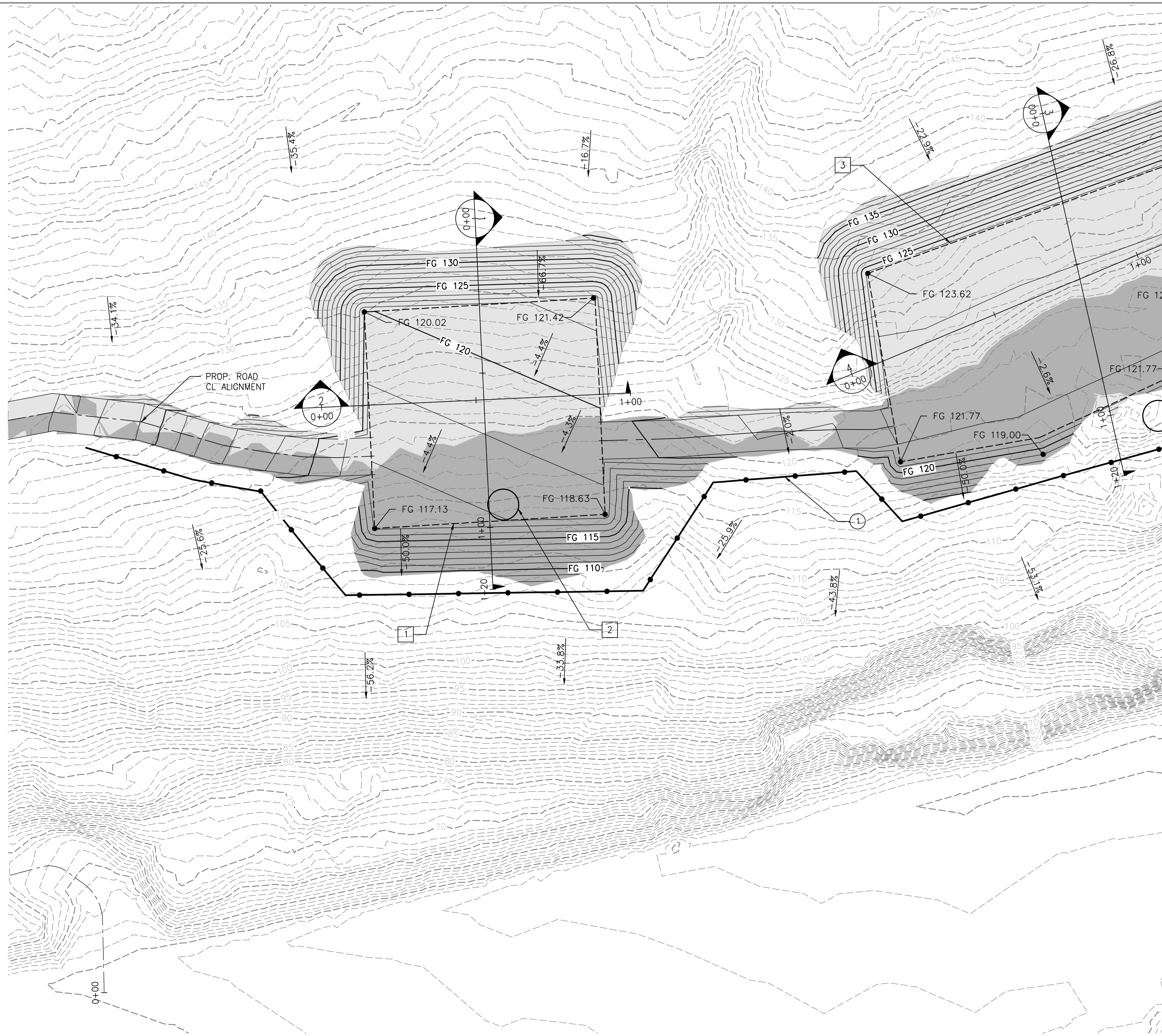
90% CONCEPT DESIGN

General Foreman: ______________________ Date: ______________
(Signature only required for field construction changes.)

CONST. PROJECT MGR:	PHONE:

DATE: 2023.11.30	SCALE: AS SHOWN
ACCOUNT: ----	AWO: ----
JOB NO: 17925	
TITLE:	HARBORTON ACCESS ROADS
CIRCUIT:	HARBORTON 230 KV TIE-IN
SIZE:	11X17
LOCATION:	PORTLAND
DESCRIPTION:	OVERALL LAYOUT
COUNTY:	MULTNOMAH
SECTION (S):	SEC 14&34, T2N, R1W
WORK WITH:	
SHEET:	2 OF 12
DESIGN BY:	JM/KF
PHONE: (503)	
DRAWN BY:	JM/KF

FILE: W:\17925\500 DESIGN\502 DRAWINGS\SHEETS\3 OVERALL LAYOUT.DWG



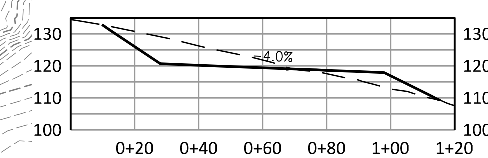
SCALE: 1" = 20'

EROSION CONTROL KEYNOTES

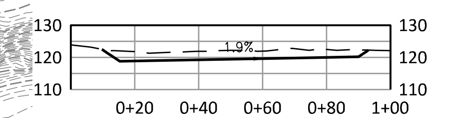
1. INSTALL TEMPORARY SEDIMENT FENCE PER DETAIL 4.3–A ON SHEET 12
2. INSTALL TREE PROTECTION FENCE. SEE DETAIL ON SHEET 12

CONSTRUCTION KEYNOTES

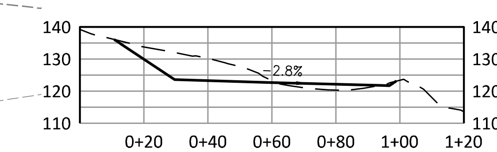
1. CONSTRUCT SP3 LANDING
2. SP3 NEW POLE LOCATION
3. CONSTRUCT SP9 LANDING
4. SP9 NEW POLE LOCATION



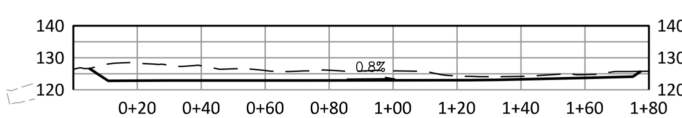
1 SP3 LANDING 1 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



2 SP3 LANDING 1 LENGTH VIEW
SCALE H:1"=30' V:1"=30'



3 SP9 LANDING 1 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



4 SP9 LANDING 1 LENGTH VIEW
SCALE H:1"=30' V:1"=30'

GENERAL NOTES

1. ELEVATIONS, LANDING DESIGN AND CUT/FILL QUANTITIES ARE BASED ON PGE LIDAR, NAD83 DATUM.
2. EXISTING UTILITIES AND SITE FEATURES SHOWN ON THESE PLANS ARE BASED ON GIS DATA.
3. THE CONTRACTOR SHALL REESTABLISH ANY SURVEY MONUMENTS DAMAGED OR DESTROYED DURING CONSTRUCTION. SURVEY MONUMENTS SHALL BE REESTABLISHED BY A LICENSED LAND SURVEYOR AT THE CONTRACTORS EXPENSE.
4. HYDROSEED ALL DISTURBED SOILS.
5. REFER TO ACCESS ROAD PHOTOMAPS FOR ACCESS ROAD IMPROVEMENT/CONSTRUCTION INFORMATION.
6. STRUCTURAL ENGINEERING FOR STRUCTURE IN FILL WORK AREA TO BE PERFORMED BY OTHERS.

DIRT BALANCE (IN CUBIC YARDS)		
ITEM	CUT	FILL
LOWER GRADING	2473	591
SHRINK/ SWELL(20%)	495	118
TOTAL	2968	709
NET	2258	

CUT/FILL LEGEND:

CUT

FILL

GRADING NOTES:

1. ALL STRIPPING, GRUBBING, GRADING, AND COMPACTION ARE TO BE DONE IN ACCORDANCE WITH THE SOIL ENGINEER'S REQUIREMENTS. THE CONTRACTOR IS ADVISED TO DIG TEST HOLES OR USE WHATEVER METHOD HE DEEMS NECESSARY TO DETERMINE EARTHWORK QUANTITIES.
2. THE CONTRACTOR SHALL VERIFY THE EXACT STRIPPING DEPTHS AND LOCATIONS WITH THE SOIL ENGINEER, AND IT IS THE CONTRACTOR'S RESPONSIBILITY TO NOTIFY AND COORDINATE THIS WITH THE SOIL ENGINEER. MACKAY SPOSITO, INC., DOES NOT HAVE THE RESPONSIBILITY FOR DETERMINING THE ADEQUACY OF THE STRIPPING OPERATION.
3. THE EARTHWORK QUANTITY SHOWN ON THE GRADING PLAN AND THE PROPOSAL IS AN ESTIMATED QUANTITY AND IS USED AS A BASIS OF BIDS. THE CUT AND FILL QUANTITY SHOWN IS CALCULATED BASED ON THE EXISTING CONDITIONS CONTOURS AND THE PROPOSED FINISH GRADE CONTOURS. THE DIRT MOVED IN THE FIELD MAY OR MAY NOT CORRESPOND TO THIS CALCULATED AMOUNT DEPENDING ON SUCH VARIABLES AS COMPACTION, SHRINKAGE, CONTRACTOR'S METHOD OF OPERATION, AND ACCURACY OF THE EARTHWORK TAKEOFF. THE CONTRACTOR IS ADVISED TO MAKE HIS OWN TAKEOFF OF EARTHWORK QUANTITIES AND DETERMINE HIS OWN QUANTITIES FOR BIDDING. WITH THE SIGNING OF THE CONTRACT FOR THE CONSTRUCTION OF THESE IMPROVEMENTS, THE CONTRACTOR AGREES THAT THE EARTHWORK QUANTITY IS SATISFACTORY AND THAT THERE WILL BE NO ADDITIONAL CHARGE FOR THIS ITEM.
4. IF ANY ARCHAEOLOGICAL DEPOSITS ARE FOUND DURING CONSTRUCTION, WORK SHALL STOP THE OREGON STATE DEPARTMENT OF ARCHAEOLOGY AND HISTORIC PRESERVATION SHALL BE NOTIFIED

MacKay Sposito

ENERGY PUBLIC WORKS LAND DEVELOPMENT

www.mackaysposito.com

REGISTERED PROFESSIONAL ENGINEER 83791PE OREGON JULY 10, 2018 JONATHAN WILLIAM FULTON

PRELIMINARY

EXPIRES: 12/31/2023

REVISIONS:

PGE CONSTRUCTION DRAWING

90% CONCEPT DESIGN

General Foreman: ______________________ Date: ____________

(Signature only required for field construction changes.)

CONST. PROJECT MGR: PHONE:

DATE: 2023.11.30	SCALE: AS SHOWN	ACCOUNT: ----	AWO: ----	JOB NO: 17925

TITLE: HARBORTON ACCESS ROADS

CIRCUIT: HARBORTON 230 KV TIE-IN — SIZE: 11X17

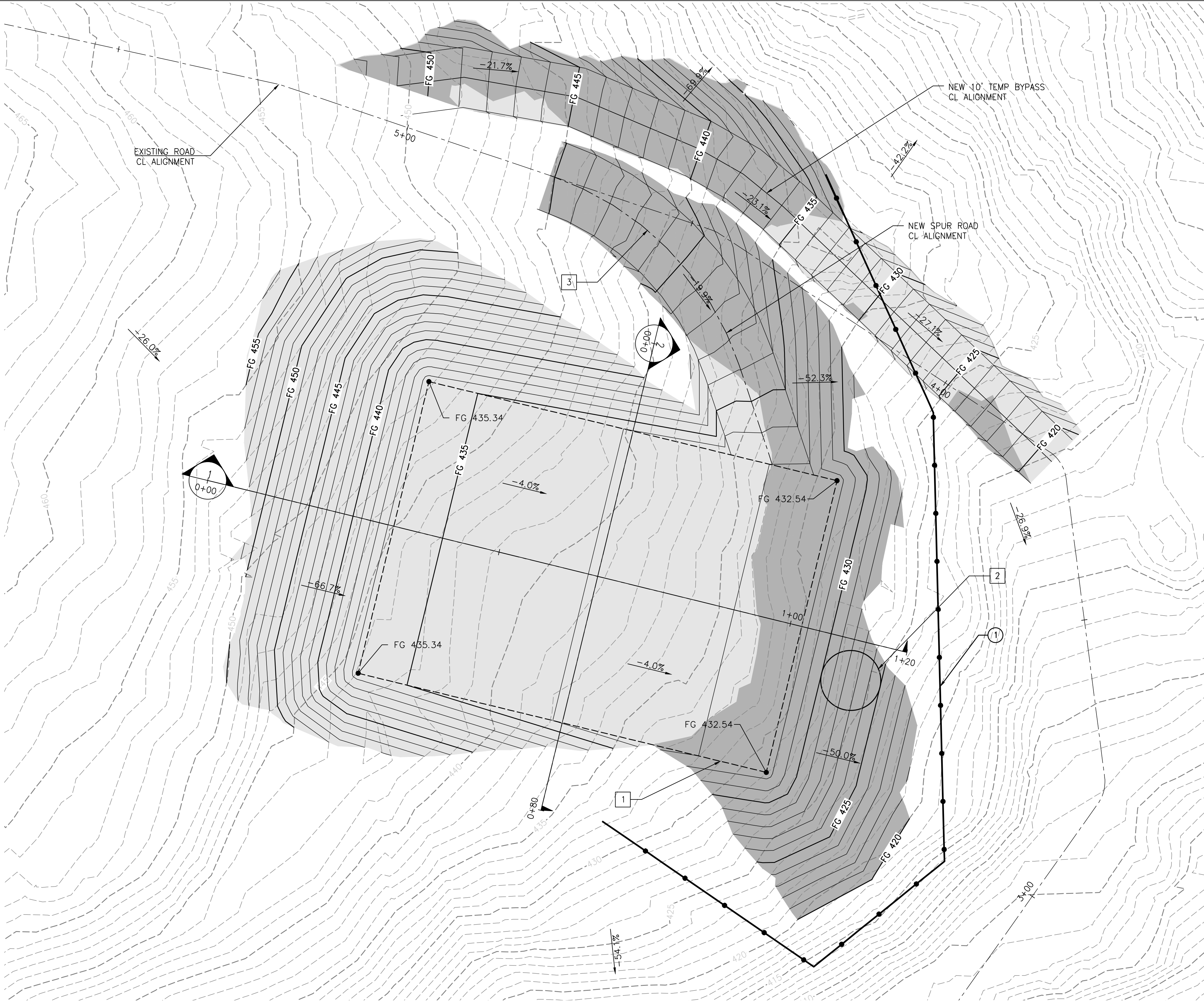
LOCATION: PORTLAND

DESCRIPTION: LOWER ROAD SITE PLAN

COUNTY: MULTNOMAH — SECTION (S): SEC 14&34, T2N, R1W — WORK WITH: — SHEET: 3 OF 12

DESIGN BY: JM/KF — PHONE: (503) — DRAWN BY: JM/KF

FILE: W:\17925\500 DESIGN\502 DRAWINGS\SHEETS\4 SP3-SITE PLAN.DWG

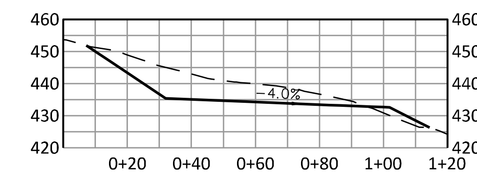


EROSION CONTROL KEYNOTES

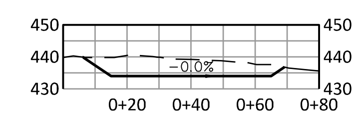
1. INSTALL TEMPORARY SEDIMENT FENCE PER DETAIL 4.3-A ON SHEET 12
2. INSTALL TREE PROTECTION FENCE. SEE DETAIL ON SHEET 12

CONSTRUCTION KEYNOTES

1. CONSTRUCT SP5 LANDING
2. SP5 NEW POLE LOCATION
3. TEMP. ABANDON MAIN ROAD DURING CONSTRUCTION TO BUILD LANDING ENTRANCE



1 SP5 LANDING 1 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



2 SP5 LANDING 1 LENGTH VIEW
SCALE H:1"=30' V:1"=30'

GENERAL NOTES

1. ELEVATIONS, LANDING DESIGN AND CUT/FILL QUANTITIES ARE BASED ON PGE LIDAR, NAD83 DATUM.
2. EXISTING UTILITIES AND SITE FEATURES SHOWN ON THESE PLANS ARE BASED ON GIS DATA.
3. THE CONTRACTOR SHALL REESTABLISH ANY SURVEY MONUMENTS DAMAGED OR DESTROYED DURING CONSTRUCTION. SURVEY MONUMENTS SHALL BE REESTABLISHED BY A LICENSED LAND SURVEYOR AT THE CONTRACTORS EXPENSE.
4. HYDROSEED ALL DISTURBED SOILS.
5. REFER TO ACCESS ROAD PHOTOMAPS FOR ACCESS ROAD IMPROVEMENT/CONSTRUCTION INFORMATION.

DIRT BALANCE (IN CUBIC YARDS)		
ITEM	CUT	FILL
LANDING 1	1145	141
BYPASS	19	31
SHRINK/ SWELL(20%)	229	28
TOTAL	1393	200
NET	1193	

CUT/FILL LEGEND:

CUT

FILL

GRADING NOTES:

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MacKay Sposito

ENERGY PUBLIC WORKS LAND DEVELOPMENT

www.mackaysposito.com

REGISTERED PROFESSIONAL ENGINEER
OREGON
PRELIMINARY
EXPIRES: 12/31/2023

REVISIONS:

PGE CONSTRUCTION DRAWING

90% CONCEPT DESIGN

General Foreman: ______________________ Date: ______________
(Signature only required for field construction changes.)

CONST. PROJECT MGR: PHONE:

DATE: 2023.11.30 | SCALE: AS SHOWN | ACCOUNT: ---- | AWO: --- | JOB NO: 17925

TITLE: HARBORTON ACCESS ROADS

CIRCUIT: HARBORTON 230 KV TIE-IN | SIZE: 11X17

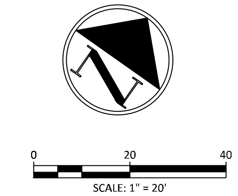
LOCATION: PORTLAND

DESCRIPTION: SP5 SITE PLAN

COUNTY: MULTNOMAH | SECTION (S): SEC 14&34, T2N, R1W | WORK WITH: | SHEET: 4 OF 12

DESIGN BY: JM/KF | PHONE: (503) | DRAWN BY: JM/KF

FILE: W:\17925\500 DESIGN\502 DRAWINGS\SHEETS\8 SP5-SITE PLAN.DWG

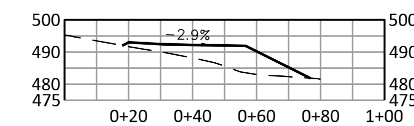


EROSION CONTROL KEYNOTES

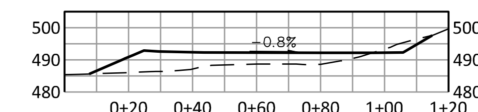
1. INSTALL TEMPORARY SEDIMENT FENCE PER DETAIL 4.3-A ON SHEET 12
2. INSTALL TREE PROTECTION FENCE. SEE DETAIL ON SHEET 12

CONSTRUCTION KEYNOTES

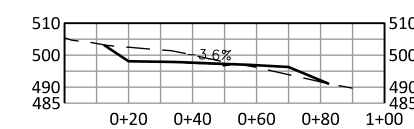
1. CONSTRUCT 2996 LANDING
2. 2996 EXIST. LATTICE LOCATION (TO REMAIN EXIST.)



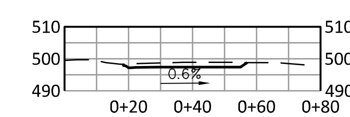
1 2996 LANDING 1 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



2 2996 LANDING 1 LENGTH VIEW
SCALE H:1"=30' V:1"=30'



3 2996 LANDING 2 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



4 2996 LANDING 2 LENGTH VIEW
SCALE H:1"=30' V:1"=30'

GENERAL NOTES

1. ELEVATIONS, LANDING DESIGN AND CUT/FILL QUANTITIES ARE BASED ON PGE LIDAR, NAD83 DATUM.
2. EXISTING UTILITIES AND SITE FEATURES SHOWN ON THESE PLANS ARE BASED ON GIS DATA.
3. THE CONTRACTOR SHALL REESTABLISH ANY SURVEY MONUMENTS DAMAGED OR DESTROYED DURING CONSTRUCTION. SURVEY MONUMENTS SHALL BE REESTABLISHED BY A LICENSED LAND SURVEYOR AT THE CONTRACTORS EXPENSE.
4. HYDROSEED ALL DISTURBED SOILS.
5. REFER TO ACCESS ROAD PHOTOMAPS FOR ACCESS ROAD IMPROVEMENT/CONSTRUCTION INFORMATION.

DIRT BALANCE (IN CUBIC YARDS)		
ITEM	CUT	FILL
LANDING 1	310	865
SHRINK/ SWELL(20%)	62	173
TOTAL	372	1038
NET		666

CUT/FILL LEGEND:

CUT

FILL

GRADING NOTES:

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MacKay Sposito

ENERGY PUBLIC WORKS LAND DEVELOPMENT
www.mackaysposito.com

REGISTERED PROFESSIONAL ENGINEER 83791PE
OREGON
JULY 10, 2018
... WILLIAM FULTON
PRELIMINARY
EXPIRES: 12/31/2023

REVISIONS:

PGE CONSTRUCTION DRAWING

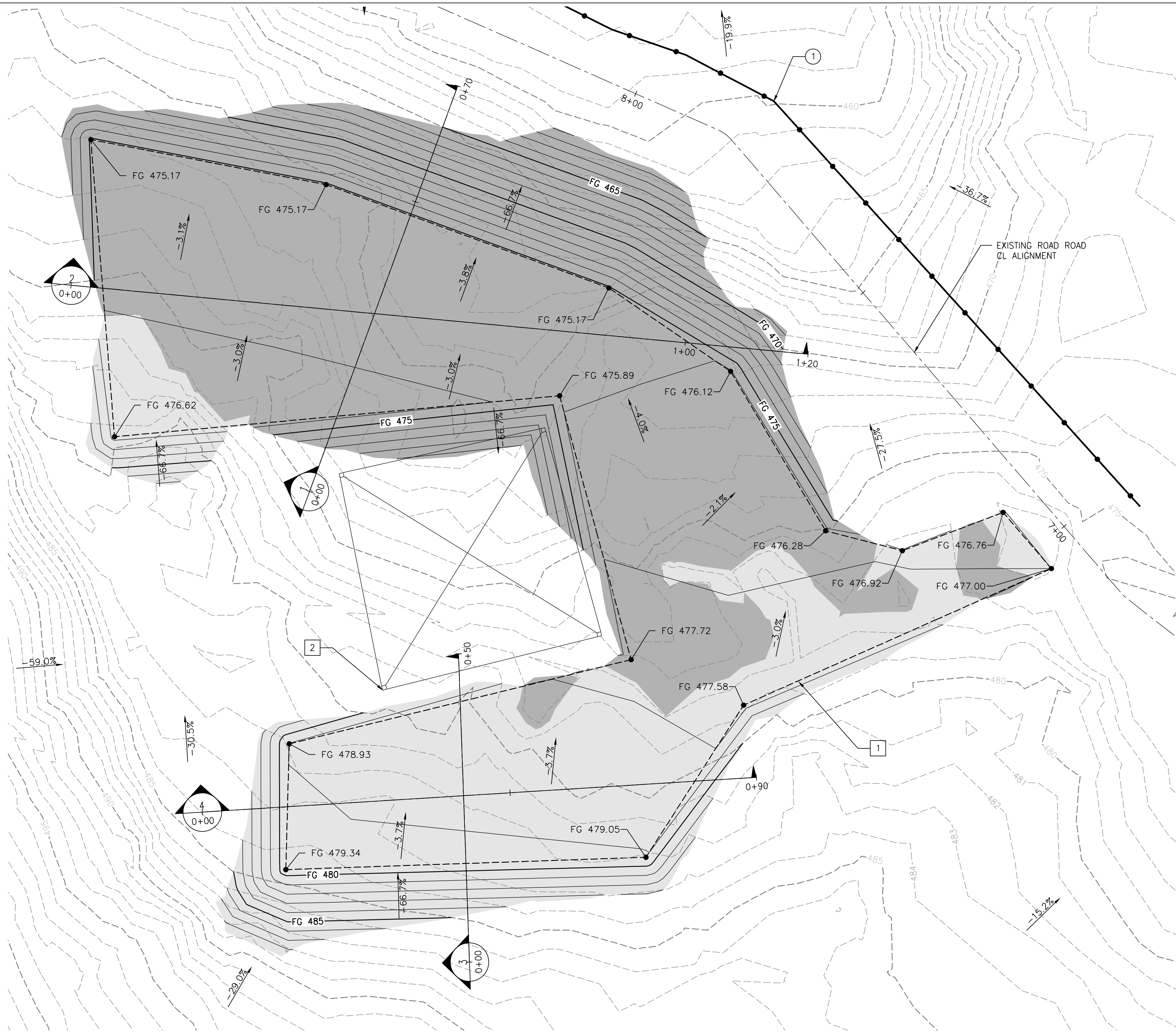
90% CONCEPT DESIGN

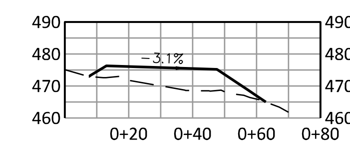
General Foreman: ______ Date: ______
(Signature only required for field construction changes.)

CONST. PROJECT MGR: PHONE:

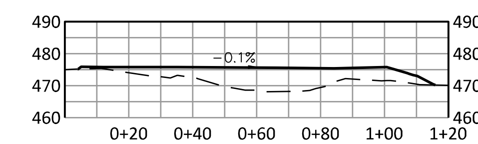
DATE: 2023.11.30	SCALE: AS SHOWN	ACCOUNT: ----	AWO: ----	JOB NO: 17925
TITLE: HARBORTON ACCESS ROADS				
CIRCUIT: HARBORTON 230 KV TIE-IN				SIZE: 11X17
LOCATION: PORTLAND				
DESCRIPTION: 2996 SITE PLAN				
COUNTY: MULTNOMAH	SECTION (S): SEC 14&34, T2N, R1W		WORK WITH:	SHEET: 5 of 12
DESIGN BY: JM/KF	PHONE: (503)	DRAWN BY: JM/KF		

FILE: W:\17925\500 DESIGN\502 DRAWINGS\SHEETS\8 SP5-SITE PLAN.DWG

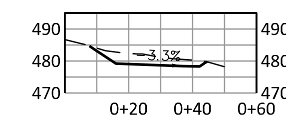




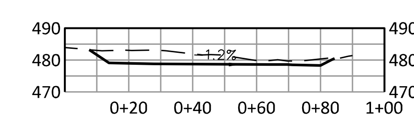
1 2997 LANDING 1 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



2 2997 LANDING 1 LENGTH VIEW
SCALE H:1"=30' V:1"=30'



3 2997 LANDING 2 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



4 2997 LANDING 2 LENGTH VIEW
SCALE H:1"=30' V:1"=30'

SCALE: 1" = 10'

EROSION CONTROL KEYNOTES

1. INSTALL TEMPORARY SEDIMENT FENCE PER DETAIL 4.3-A ON SHEET 12
2. INSTALL TREE PROTECTION FENCE. SEE DETAIL ON SHEET 12

CONSTRUCTION KEYNOTES

1. CONSTRUCT 2997 LANDING
2. 2997 EXIST. LATTICE STRUCTURE (TO REMAIN EXIST.)

GENERAL NOTES

1. ELEVATIONS, LANDING DESIGN AND CUT/FILL QUANTITIES ARE BASED ON PGE LIDAR, NAD83 DATUM.
2. EXISTING UTILITIES AND SITE FEATURES SHOWN ON THESE PLANS ARE BASED ON GIS DATA.
3. THE CONTRACTOR SHALL REESTABLISH ANY SURVEY MONUMENTS DAMAGED OR DESTROYED DURING CONSTRUCTION. SURVEY MONUMENTS SHALL BE REESTABLISHED BY A LICENSED LAND SURVEYOR AT THE CONTRACTORS EXPENSE.
4. HYDROSEED ALL DISTURBED SOILS.
5. REFER TO ACCESS ROAD PHOTOMAPS FOR ACCESS ROAD IMPROVEMENT/CONSTRUCTION INFORMATION.

DIRT BALANCE (IN CUBIC YARDS)		
ITEM	CUT	FILL
LANDING 1	236	737
SHRINK/ SWELL(20%)	47	147
TOTAL	283	884
NET		601

CUT/FILL LEGEND:

CUT

FILL

GRADING NOTES:

1. ALL STRIPPING, GRUBBING, GRADING, AND COMPACTION ARE TO BE DONE IN ACCORDANCE WITH THE SOIL ENGINEER'S REQUIREMENTS. THE CONTRACTOR IS ADVISED TO DIG TEST HOLES OR USE WHATEVER METHOD HE DEEMS NECESSARY TO DETERMINE EARTHWORK QUANTITIES.
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MacKay Sposito

ENERGY PUBLIC WORKS LAND DEVELOPMENT

www.mackaysposito.com

REGISTERED PROFESSIONAL ENGINEER 83781PE OREGON JULY 10, 2018 JOHN WILLIAM FULTON

PRELIMINARY

EXPIRES: 12/31/2023

REVISIONS:

PGE CONSTRUCTION DRAWING

90% CONCEPT DESIGN

General Foreman: ______ Date: ______
(Signature only required for field construction changes.)

CONST. PROJECT MGR: PHONE:

DATE: 2023.11.30 SCALE: AS SHOWN ACCOUNT: ---- AWO: ---- JOB NO: 17925

TITLE: HARBORTON ACCESS ROADS

CIRCUIT: HARBORTON 230 KV TIE-IN SIZE: 11X17

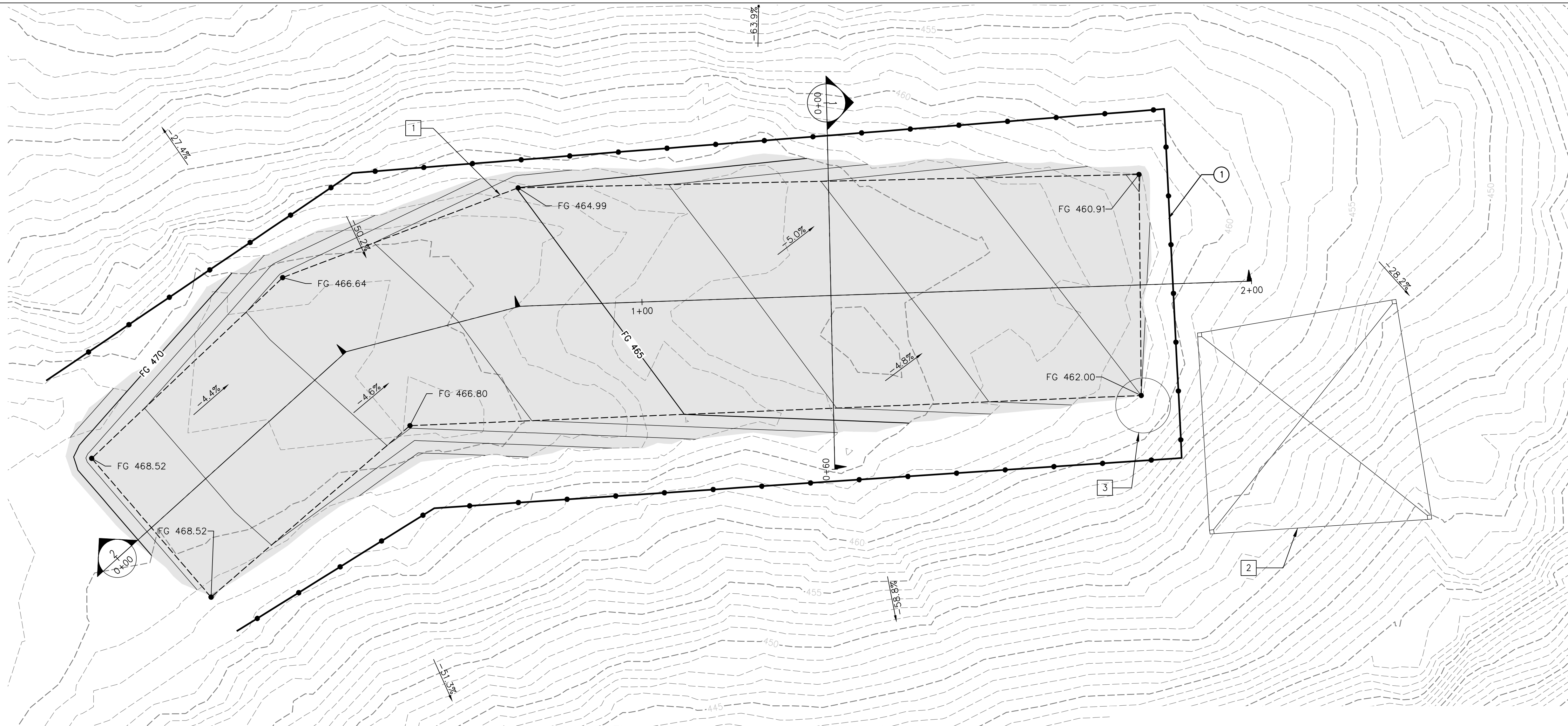
LOCATION: PORTLAND

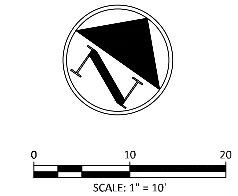
DESCRIPTION: 2997 SITE PLAN

COUNTY: MULTNOMAH SECTION (S): SEC 14&34, T2N, R1W WORK WITH: SHEET: 6 OF 12

DESIGN BY: JM/KF PHONE: (503) DRAWN BY: JM/KF

FILE: W:\17925\500 DESIGN\502 DRAWINGS\SHEETS\8 SP5-SITE PLAN.DWG



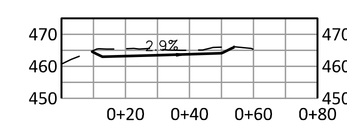


EROSION CONTROL KEYNOTES

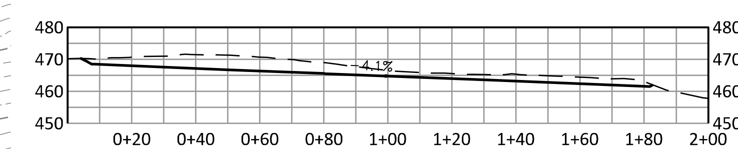
1. INSTALL TEMPORARY SEDIMENT FENCE PER DETAIL 4.3-A ON SHEET 12
2. INSTALL TREE PROTECTION FENCE. SEE DETAIL ON SHEET 12

CONSTRUCTION KEYNOTES

1. CONSTRUCT 2998 LANDING
2. PREFERRED OPTION: 2998 RE-USE EXIST. LATTICE STRUCTURE
3. ALTERNATIVE OPTION: 2998 NEW POLE LOCATION



1 2998 LANDING 1 WIDTH VIEW
SCALE H:1"=30' V:1"=30'



2 2998 LANDING 1 LENGTH VIEW
SCALE H:1"=30' V:1"=30'

GENERAL NOTES

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4. HYDROSEED ALL DISTURBED SOILS.
5. REFER TO ACCESS ROAD PHOTOMAPS FOR ACCESS ROAD IMPROVEMENT/CONSTRUCTION INFORMATION.

DIRT BALANCE (IN CUBIC YARDS)

ITEM	CUT	FILL
LANDING 1	694	0
SHRINK/ SWELL(20%)	139	0
TOTAL	833	0
NET	833	

CUT/FILL LEGEND:

CUT

FILL

GRADING NOTES:

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MacKay Sposito

ENERGY PUBLIC WORKS LAND DEVELOPMENT

www.mackaysposito.com

REGISTERED PROFESSIONAL ENGINEER 83791PE OREGON JULY 10, 2018 JONATHAN WILLIAM FULTON

PRELIMINARY

EXPIRES: 12/31/2023

REVISIONS:

-
-
-
-
-
-

PGE CONSTRUCTION DRAWING

90% CONCEPT DESIGN

General Foreman: ____________________ Date: __________
(Signature only required for field construction changes.)

CONST. PROJECT MGR: PHONE:

DATE: 2023.11.30	SCALE: AS SHOWN	ACCOUNT: ----	AWO: ---	JOB NO: 17925

PGE

TITLE: HARBORTON ACCESS ROADS

CIRCUIT: HARBORTON 230 KV TIE-IN

SIZE: 11X17

LOCATION: PORTLAND

DESCRIPTION: 2998 SITE PLAN

COUNTY: MULTNOMAH

SECTION (S): SEC 14&34, T2N, R1W

WORK WITH:

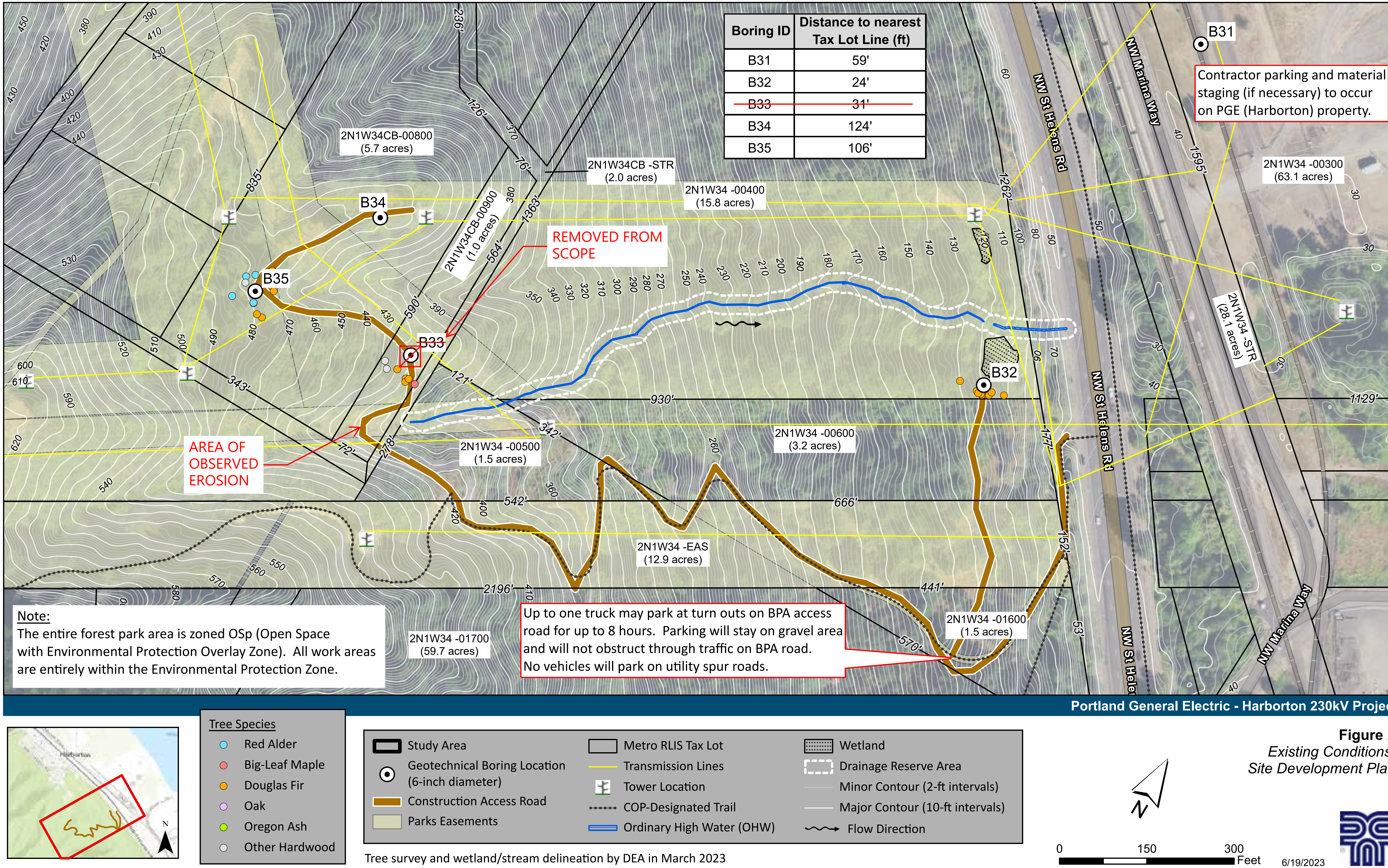
SHEET: 7 OF 12

DESIGN BY: JM/KF

PHONE: (503)

DRAWN BY: JM/KF

FILE: W:\17925\500 DESIGN\502 DRAWINGS\SHEETS\8 SP5-SITE PLAN.DWG



Boring ID	Distance to nearest Tax Lot Line (ft)
B31	59'
B32	24'
~~B33~~	~~31'~~
B34	124'
B35	106'

Path: P:\P\PGEX00008085\0600INFO\GS\PGEX8085.aprx

APPENDIX E

Geoprofessional Business Association Guidance Document

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

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