J3310-02-01
November 1, 2021

John Arthur, PE
President & CEO
Wireless EDGE Towers
6369 Mill Street, Suite 202
Rhinebeck, New York 12572

Re: Updated Soil Conditions and Engineering Properties
Park Street Lattice Tower
Northfield, New Hampshire

Dear Mr. Arthur:

O'Reilly, Talbot & Okun Associates, Inc. (OTO) is pleased to provide this updated letter report summarizing our Site investigation and geotechnical recommendations for the proposed Lattice Tower to be located to the south of the Tilton-Northfield Fire Department, which is located at 149 Park Street in Northfield, New Hampshire. A Site Locus is provided as Figure 1. A Site Plan is provided as Figure 2. This report supersedes our May 26, 2021 report.

Our services consisted of review of the boring logs and soil samples, engineering analyses, and preparation of this report. New England Boring Contractors of Derry, New Hampshire performed two soil borings, collected samples and prepared soil boring logs. In addition, ConeTec of West Berlin, New Jersey performed six Piezocone Penetration Test (CPTu) probes. Shear wave velocity measurements were made at three-foot intervals in one of the CPTu probes using a seismic CPTu. OTO did not observe the soil borings or CPTu probes. This report is subject to the attached limitations.

PROJECT DESCRIPTION

The project consists of the installation of a 178-foot-high self-supporting lattice tower and associated structures and utilities. The tower will be located to the south of the Northfield Fire Department located at 149 Park Street in Northfield, New Hampshire. A 15-foot-wide gravel access road will provide access to the area from Park Road.

The topography at the Site within the footprint of the proposed tower is relatively flat at an approximate elevation of 436 feet. Topography at the proposed access road slopes downward from the north, from approximate elevation 441, to the south, to approximate elevation 436 feet. The location of existing Site features and the proposed construction is shown on the attached Site Plan.

We understand that the lattice tower will be founded upon either a concrete mat (large spread footing) or a deep foundation system. We understand that preliminary plans call for a 7-foot round or square pedestal bearing on a 17-foot square, 2-foot thick mat foundation. The bottom of the footing/mat will likely bear about 5.5 feet below the surrounding ground surface.

We note that the recommendations provided in this report should be reviewed once the foundation type and tower design loads are finalized.

SUBSURFACE EXPLORATIONS

Subsurface explorations consisted of two soil borings (B-1 and B-2) and six CPTu probes (SCPT21-01, CPT21-02/02A through CPT21-06). The borings and probes locations are as shown on the attached Figure 2.

Soil Borings

The borings were performed on January 27, 2021 by New England Boring Contractors of Derry, New Hampshire using a truck mounted drill rig equipped with hollow stem augers. We note that OTO did not observe the borings.

In general, soil samples were collected continuously from the ground surface to a depth of eleven feet, at fourteen feet and every five feet thereafter. Soil samples were collected using a 2-inch diameter split spoon sampler driven 24-inches with a 140-pound safety hammer falling 30 inches (American Society for Testing and Materials Test Method D1586-99 “Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils”). The number of blows required to drive the samples each six inches was recorded. The standard penetration resistance, or N-Value, is the number of blows required to drive the sampler the middle 12 inches. Soil properties, such as strength and density, are related to the N-value.

New England Boring Contractors collected and classified soil samples and logged the borings. A Site sketch showing boring locations, boring logs and jarred soil samples were provided to OTO for review.

Piezocone Penetration Tests (CPTu) and Seismic CPTu Tests

Six cone penetration test probes (CPTu) were performed by ConeTec Inc. of West Berlin, New Jersey on September 22, 2021. A cone penetrometer is a cylindrical element with the cone shaped tip that is pushed into the ground at a fixed rate while the resistance at the tip and along the sides are measured on a continuous basis using electronic pressure load cells. Soil properties such as density, friction angle and strength can be approximated based upon the tip resistance and skin friction and ratios calculated using these measurements. An electronic cone penetration probe equipped with a pressure transducer to measure water pressure (CPTu tests), and a geophone to measure shear wave velocities (SCPTu tests), was used for this project. The seismic response of the Site soils under earthquake loadings can be estimated based upon shear wave velocity measurements. A detailed discussion of the CPTu and SCPTu tests is presented in the ConeTec data report, attached.

The CPT test probes were performed within or near the footprint of the proposed tower. One SCPTu test probe (SCPT21-01) was performed to a depth of 50 feet to provide a profile of shear wave velocity with depth at the Site. Shear wave velocity measurements are presented in Table 1. The remaining CPT probes were completed to a depth of between 44 and 50 feet using the CPTu probe. OTO did not observe the probes. A detailed discussion of the piezocone and shear wave velocity tests are presented in the ConeTec data report, attached.

Since CPT measurements of tip resistance and skin friction are collected on what is effectively a continuous basis, detailed profiles of soil properties (such as density, friction angle and strength) were developed. In general, the data from the CPTu probes were in agreement with published correlations for the Standard Penetration Tests (SPT) performed in the soil borings.

SUBSURFACE CONDITIONS

Subsurface conditions were interpreted based upon the soil boring logs prepared by New England Boring Contractors and the CPTU data. We also obtained and reviewed the jarred soil samples collected from the borings. Granular material observed at the Site consists of sand and silt soils to a depth of approximately 45 to 50 feet. The loose sand and silt layers present between a depth of 10 and 45 to 50 feet are the significant geotechnical issue identified in the investigations. Probe CPT21-02 encountered shallow refusal at a depth of 2 feet, this probe was off-set and probing resumed (CPT21-02A).

Soil Conditions

A medium dense, sand and gravel was encountered in the upper four to six feet at each location. This surficial layer was underlain by generally medium dense fine sand. At a depth of between 10 and 15 feet, the soil transitioned to very loose to loose, silty sand or sandy silt. Boring B-1 fully penetrated the loose, sandy silt/ sandy silt layer while boring B-2 was terminated within this layer at a depth of 16 feet. Probes CPT21-03 through CPT21-06 were terminated within the silty sand layer at a dept of 50 feet. We note that the loose, layer is of concern due to potential settlement that may occur during the design earthquake.

Boring B-1 and probes CPT21-01 and -02A, which fully penetrated the loose, sandy silt layer, encountered a very dense, fine to coarse sand at a depth of between 44 and 47 feet below ground surface.

The CPTu data indicates a soil profile which is generally consistent with the soil borings. The CPTu probes indicate most of the profile consists of thin layers of silt and fine to medium sand to the maximum depth explored. CPTu probes CPT21-02A and -03 encountered refusal within the lower granular layer, indicating the layer is very dense and/or large cobbles or boulders are present.

Groundwater Conditions

Groundwater was encountered at an approximate depth of between 5 and 7 feet below ground surface, as noted on the boring logs. Groundwater was encountered at similar depths in the CPTu probes. Therefore, groundwater may be encountered during construction activities. If groundwater is encountered during excavations for footings and utilities, it should be possible to dewater these excavations by trenching or using sump pumps provided the excavations only penetrate a short distance into the water table.

Shear Wave Velocity Measurements

Fourteen (14) seismic shear wave velocity measurements were obtained in SCPTu probe SCTP21-01. The measurements were obtained to a maximum depth of 50 feet. Shear wave velocity measurements are presented in the attached report. The velocities ranged from 518 to 795 feet per second (fps), with an harmonic averages greater than 600 fps.

Shear wave velocity measurements are used to determine the Seismic Site Classification. Based upon these measurements the Seismic Site Classification is Class D.

GEOTECHNICAL ISSUES

The significant geotechnical issues for this report include the following:

- Potential liquefaction during design earthquake event of the loose, sand and silt layer present between a depth of 10 and 45 feet below ground surface and the resulting settlement;
- Foundation type;
- Geotechnical design parameters including bearing capacity for a potential mat foundation upon ground improved alternative (if selected), end bearing and skin friction for the design of drilled shafts or mini-piles (if selected); and lateral resistance;
- Seismic design parameters; and
- and the re-use of Site soils as engineered fill.

Seismic Design Concerns and Liquefaction Potential

The significant concern for this project Site includes the impacts of the design earthquake on Site soil, and the resulting potential settlement and/or failure of a shallow foundation under seismic loading condition. The sandy silt encountered between below a depth of 10 feet may be susceptible to liquefaction during the design earthquake.

The phenomenon of liquefaction is simply stated as the complete loss of strength experienced by loose granular soils during earthquake shaking. Structures supported on sand or silty sand deposits may liquefy, may settle or tilt excessively and in extreme cases can collapse. Many complete structure failures have occurred during earthquakes during ground failure resulting from liquefaction.

Based upon soil densities estimated from N-values and the subsequent CPTu data, the saturated granular soils were observed to be loose between 10 and 45 to 50 feet. These soils may be susceptible to liquefaction.

Using the data collected for this study, we performed calculations to estimate the amount of liquefaction induced settlement the structure might experience (if liquefaction were to occur). Conservatively, we estimate that without ground improvement over 12 inches total liquefaction induced settlement could occur. The amount of settlement is significant and exceeds typical settlement tolerances. Therefore, we recommend that this concern is mitigated.

Two options are available to address this concern:

- **Option 1** - Ground Improvement: the loose, soil layer is improved via the installation of rammed aggregate piers; or
- **Option 2** - Deep Foundations System: a deep foundation is utilized.

Additional information and preliminary recommendations for both alternatives are provided below. Further analysis, recommendations and design may be needed depending on the chosen alternative. Based upon the type of structure (tower upon a stiff mat) and its tolerance to foundation settlement, along with the acceptance of some risk by the owner, we have proposed alternatives to support the structure and mitigate the concern. We recommend that the owner review the proposed alternatives, their effectiveness to mitigate the concern and reduce risks of settlement and failure.

PRELIMINARY RECOMMENDATIONS

The following preliminary recommendations are based upon the construction described above. The recommendations assume that the liquefaction concern will be fully or partially mitigated as described above.

Seismic Design Considerations

Since the project is associated with the Tilton-Northfield Fire Station, we have assumed that the 2019 New Hampshire Building Code (NHBC) applies to this project. The NHBC includes amendments to the 2015 International Building Code (IBC), which refers to ASCE-7 (2010). Procedures for the Site-specific determination of Site Classification are provided in Chapter 20 of ASCE-7(2010), *Minimum Design Loads for Buildings and Other Structures*. Section 15.6.6 of ASCE 7 requires that self-supporting and guyed telecommunication towers be designed to resist seismic lateral forces determined from a substantiated analysis using reference documents. The design team should evaluate this assumption.

Site Class Determination

Structures should be designed and constructed to resist the effects of lateral forces imposed by earthquake motion in accordance with ASCE-7. Lateral forces are dependent on the type and properties of soils present beneath the Site, along with the geographic location. At this Site, we evaluated Site Classification using one of the parameters allowed (Shear Wave Velocity). The Site Class was determined to be Class D based upon SCPTu soil data collected. We note that this assumes that a Site-specific analysis is not required for this type of structure. This interpretation of code requirements should be confirmed by the structural engineer.

The maximum considered earthquake spectral response acceleration at short periods (S_s) and at 1-sec (S_1) was determined to be 0.297 and 0.088, respectively, for Northfield, New Hampshire. Furthermore, the Site coefficients F_a and F_v were determined to be 1.56 and 2.4, respectively.

Liquefaction

As described above, the liquefaction potential was evaluated for saturated Site soils and is a significant concern for this project. Although New England is not generally considered one of the more seismically active areas in the country; however, relatively large earthquakes have occurred in the region within the last 300 years.

We performed calculations based upon both the SPT blow counts obtained in the soil borings and upon the CPT data. Both methods indicated the soils between a depth of approximately 15 and 45 feet are potentially liquefiable under the design earthquake. The SPT based calculation assumed no drilling-related disturbances to estimate the amount of liquefaction induced settlement the structure might experience if liquefaction were to occur based upon the data collected during this study. We conservatively estimate that during a seismic event resulting in liquefaction of the loose sand and silt layer the mat could experience up to 10 to 12 inches of total settlement. Therefore, we recommend ground improvement to densify the loose soil layer, or a deep foundation alternative is recommended, as described above. As we discussed above, if the owner is willing to accept the risk of some settlement under a seismic event it may be feasible to reduce the amount of settlement by densifying only a portion of the loose layer.

Foundations

We understand that the communications tower will be founded on either a mat foundation bearing on improved soil or upon a deep foundation system. we recommend that a mat foundation be founded densified (improved) ground to reduce settlement under both static and seismic conditions. For a mat foundation, ground improvement would be necessary to improve (densify) existing Site soils to reduce liquefaction concerns. For the deep foundation alternative, it is likely that either drilled shafts or mini-piles would be the most cost effective alternatives due to their lower mobilization cost. The deep foundation alternative would transfer loads to the denser soil layers at depths below 45 to 50 feet.

Option 1: Ground Improvement

The ground improvement option provides an alternative to construct the tower upon traditional stiff mat foundation, while increasing the soil resistance to earthquake loading and reducing the amount of settlement due to liquefaction.

Many ground improvement techniques are available. Our recommended ground improvement method for this project is Rammed Aggregate Piers (RAPs, also known as GeoPiers) to densify the loose layer. These piers are basically stone columns, which are mechanically tamped into the ground to displace and densify the existing fill layer. The densified soils are less susceptible to liquefaction and settlement. Structures founded on rammed aggregate columns can, in turn, be supported on shallow foundations, such as a mat foundation.

Rammed aggregate piers (or RAPs) are a proprietary foundation technique that uses drilled and vertical columns of aggregate that are placed and mechanically tamped in thin lifts within an augured drill hole, to stiffen the soil mass and/or transmit loads vertically through the soft and compressible soils to the underlying denser soils. In New England, these piers are installed by Helical Drilling of Braintree, Massachusetts (781-848-2110) or Keller (formerly Hayward Baker) at 401-334-2565.

The installation process introduces lateral stresses into the soil mass to stiffen, densify, and reinforce the surrounding soil matrix. RAPs are typically designed by the contractor's engineer, who determines the number, size, depth and location of piers for a given project. RAPs are typically installed in a relatively tight spacing beneath structural elements.

Based upon our experience, spread footings or mats on RAPs may be designed based on an allowable bearing pressure of upwards of 5,000 pounds per square foot. The depth of the bottom of the loose layer (approximately 45 feet) is greater than the typical maximum depth of aggregate piers. However, it may be practical to densify only the upper portion of the loose sand layer, which would reduce the potential for bearing capacity failure, and reduce the magnitude of total and differential settlement under both static loading and seismic induced liquefaction. The magnitude of settlement should be directly proportional to the thickness of the loose (liquefiable) layer (which is present between a depth of approximately 15 to 45 feet below ground surface and is about 30 feet thick). Therefore, densifying the loose layer to a depth of 30 feet would reduce the thickness of the liquefiable layer by a half and reduce the amount of settlement by about 50%.

A structure founded on soils improved using 30-foot long aggregate piers should also experience less differential settlement since the foundation would bear on a 30-foot layer of dense soil, which would reduce the amount of differential movement.

The resulting estimated total and differential settlement under static and transient loads for both zones would need to be less than the acceptable settlement tolerances for this type of construction under the various loading conditions. The settlement tolerances under an earthquake event should be determined by the owner and/or tower manufacturer.

For a mat foundation placed upon improved soil (large spread footing), the size of the mat will depend on the weight of concrete (and soil) needed to resist uplift and the final footprint; however, preliminary plans call for the base of the foundation to be on the order of 17 feet square and will extend 5.5 feet below ground surface. The mat foundation can bear upon the upper medium dense native, granular soils, provided they are densified prior to the placement of the concrete. If wet or disturbed soils are encountered at the base of the excavation, we recommend that the subgrade be over-excavated by 6 to 12 inches and that Crushed Stone be placed to provide a firm bearing surface. The Crushed Stone should be compacted by tamping, vibratory compactor or other suitable methods to form a dense base. Provided these recommendations are followed, a maximum allowable bearing pressure of 5,000 pounds per square foot can be used for preliminary design of a mat upon improved soil. As noted above, the allowable bearing capacity will be determined by the specialty designer (based upon final design of system) if this option is chosen.

If the designer elects to use passive resistance to resist sliding of the mat, we recommend that only the portion of the passive resistance below five feet (the anticipated maximum frost depth) be used to resist sliding. Additional design parameters are provided in Table 1.

Option 2: Deep Foundation System

We understand that a deep foundation system may be the most cost efficient option based upon preliminary studies. Types of deep foundations to consider include (but not limited to) ductile iron pipe piles, drilled caissons and/or drilled mini-piles. Due to the presence of the thick layer of potentially liquefiable layers, it may be appropriate to found the piles/piers in or on the dense soil layer present at a depth of approximately 45 feet. Additional borings may be required to further define the nature and depth of the suitable bearing strata, and confirm the depth to bedrock. Pile/pier design and construction should be consistent with requirements set forth in appropriate codes. Furthermore, once a pile type is selected, the structural and geotechnical engineers should evaluate the lateral and tensile capacity of the selected system and provide supplemental recommendations and soil properties, as needed.

We recommend further review by a structural engineer and pile specialty contractor. We recommend that the pile system fully penetrate the loose soil layer. Generally, the pile system will be designed by the specialty contractor in general accordance with the project’s specifications (loading and settlement tolerances). Some pile types have a limited lateral capacity due to their slender cross-section. To accommodate higher lateral loads, we recommend the use of battered piles and/or a mat (pile cap) be considered by the structural engineer. Thicker casing and larger diameter mini piles may be required for lateral resistance. Lateral loading of foundation elements should be determined during final design. In addition, considerations to down drag forces under the seismic event will need to be considered during design, as well as lateral reinforcement (such as a grade beam) to provide additional structural stiffness during a seismic event.

General Foundation Recommendations

This report provides general recommendations for both types of foundations (shallow and deep); however, supplemental information and evaluation will be necessary for final design. Soil properties for use during preliminary design are presented in Table 1.

Depending on the type of foundation, the lateral and vertical loads will be resisted through a combination of skin friction, end bearing against the base, and if the designer elects to do so, passive resistance against the side of a mat foundation or pile cap. Vertical loads (predominantly dead loads) consist of both downward pressures due to the dead weight of the mat foundation and the tower structure and both downward and upward live loads (such as wind and seismic). Since the lattice tower is relatively light, uplift and overturning pressures under live loads will likely govern design.

We recommend that the mat foundation (or pile/pier cap for a deep foundation system) bear a minimum of five feet below ground surface, or to the design frost depth. If a deep (pile) foundation is used, we have assumed that the piles/piers will extend through the loose soils and into the dense (non-liquefiable) sand layer below to provide capacity during a potential seismic event (when liquefaction may occur).

If winter construction occurs, the mat foundation should not be placed on frozen soils. The excavation should be free of loose or disturbed materials. Any boulders or cobbles larger than 4 inches in diameter should be removed from within one foot of the bottom of the foundation and replaced with sand and gravel fill. If loose materials are present in the excavation, they shall be recompacted to form a firm, dense, bearing surface.

Table 1
Design Parameters

Soil Property	Recommended Value — Soil Type		
	Upper Sand (Unimproved) (0-10')	Lower Silt (Unimproved) (10-45')	Lower Granular Soil* (Dense) (>50')
Total Unit Weight	125 pcf	110 pcf	135 pcf
Buoyant Weight	62 pcf	48 pcf	72 pcf
Angle of Internal Friction Friction Factor	30 degrees 0.36	24 degrees 0.28	45 degrees 0.55
Cohesion	0	0	0
Coefficient of Active Earth Pressure (Ka)	0.33	0.40	0.17
Coefficient of At-Rest Earth Pressure (Ko)	0.50	0.59	0.30
Coefficient of Passive Earth Pressure (Kp)*	3.0	2.5	5.8
Allowable Static Bearing Pressure (Qall)	5000	--	Dependent on pile type

* We recommend an equivalent fluid pressure of 275 pounds per cubic foot be used to compute passive resistance based upon a factor of safety of 1.5.
* Note depth to bedrock unknown

Earthwork Considerations

We anticipate that earthwork for this project will include excavations for the foundation and placement of fill to backfill the mat foundation or pile cap.

Any deleterious materials, vegetation, organic soils (topsoil) or wet and disturbed soils, should be removed from beneath the mat foundation (Option 1) and replaced with Sand and Gravel Fill. Fill, debris, topsoil or organic soils stripped from the excavation should not be re-used as fill beneath structures. For the ground improvement option, any fill placed between the improved ground and the communication tower foundation should consist of a well-graded Sand and Gravel (approved by the geotechnical engineer), that is compacted to at least 95% of the Modified Proctor dry density as defined in ASTM D1557, Method C. We note that Crushed Stone can be used to form the foundation subgrade and provide a firm bearing layer, if needed and as described above. To avoid point loads, any cobbles or boulders larger than 4-inch diameter, encountered at the subgrade for the mat foundation, should be removed and replaced with compacted Sand and Gravel fill.

Based upon the planned construction and the description of materials observed in the borings, it does not appear that there are sufficient quantities of suitable materials present on-Site to meet requirements for Sand and Gravel Fill. Therefore, these materials will likely need to be imported. Fill should be placed in lifts of no more than 12-inches (loose lifts). To facilitate compaction, the moisture content should be maintained within 2 percent of the optimum moisture content, as determined by ASTM D1557, Method C. Grain size distribution requirements are presented in Table 2.

Table 2
Grain Size Distribution Requirements

Size	Sand and Gravel	Crushed Stone
	Percent Finer by Weight	
3 inch	100	---
1 inch	---	100
¾ inch	---	90-100
½ inch	50-85	10-50
⅜ inch	---	0-20
No. 4	40-75	0-5
No. 40	10-35	---
No. 200	0-10	---

SUPPLEMENTAL INVESTIGATIONS AND/OR FINAL DESIGN

We recommend that the project team consider if supplement investigations are warranted in order to complete final design. OTO has initiated discussions with a specialty contractor to provide input in the feasibility and cost of the recommended options described in the report. Once final load demands are available, this information should be provided to the design team to further evaluate options and provide supplement investigations and/or recommendations as needed.

If you have any questions, please do not hesitate to contact the undersigned.

Sincerely yours,
O'Reilly, Talbot & Okun Associates, Inc.

Ashley L. Sullivan, P.E.
Principal

Michael J. Talbot, PE
Reviewer

Attachments: Limitations, Site Locus, Site Plan, Boring Logs, ConeTec Report

O:\J3300\3310 Wireless EDGE Towers\02-03 Northfield NH Telecommunication - Supplemental Geotech\Updated Northfield Tower Geotech 11-1-21.docx

LIMITATIONS

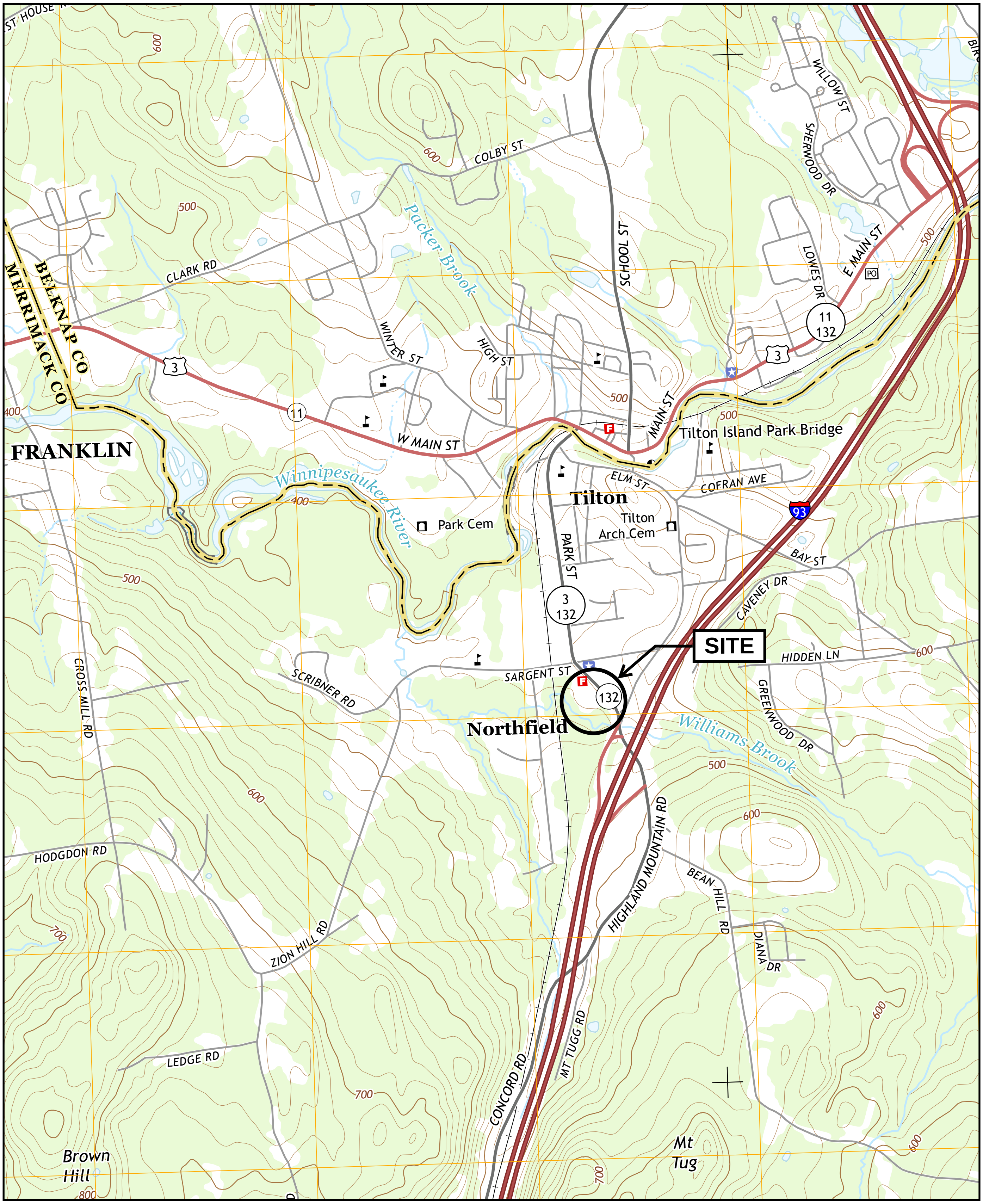
1. The observations presented in this report were made under the conditions described herein. The conclusions presented in this report were based solely upon the services described in the report and not on scientific tasks or procedures beyond the scope of the project or the time and budgetary constraints imposed by the client. The work described in this report was carried out in accordance with the Statement of Terms and Conditions attached to our proposal.

2. The analysis and recommendations submitted in this report are based in part upon the data obtained from widely spaced subsurface explorations. The nature and extent of variations between these explorations may not become evident until construction. If variations then appear evident, it may be necessary to reevaluate the recommendations of this report.

3. The generalized soil profile described in the text is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretations of widely spaced explorations and samples; actual soil transitions are probably more erratic. For specific information, refer to the boring logs.

4. In the event that any changes in the nature, design or location of the proposed structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by O'Reilly, Talbot & Okun Associates Inc. It is recommended that we be retained to provide a general review of final plans and specifications.

5. Our report was prepared for the exclusive benefit of our client. Reliance upon the report and its conclusions is not made to third parties or future property owners.



0 1000 FEET

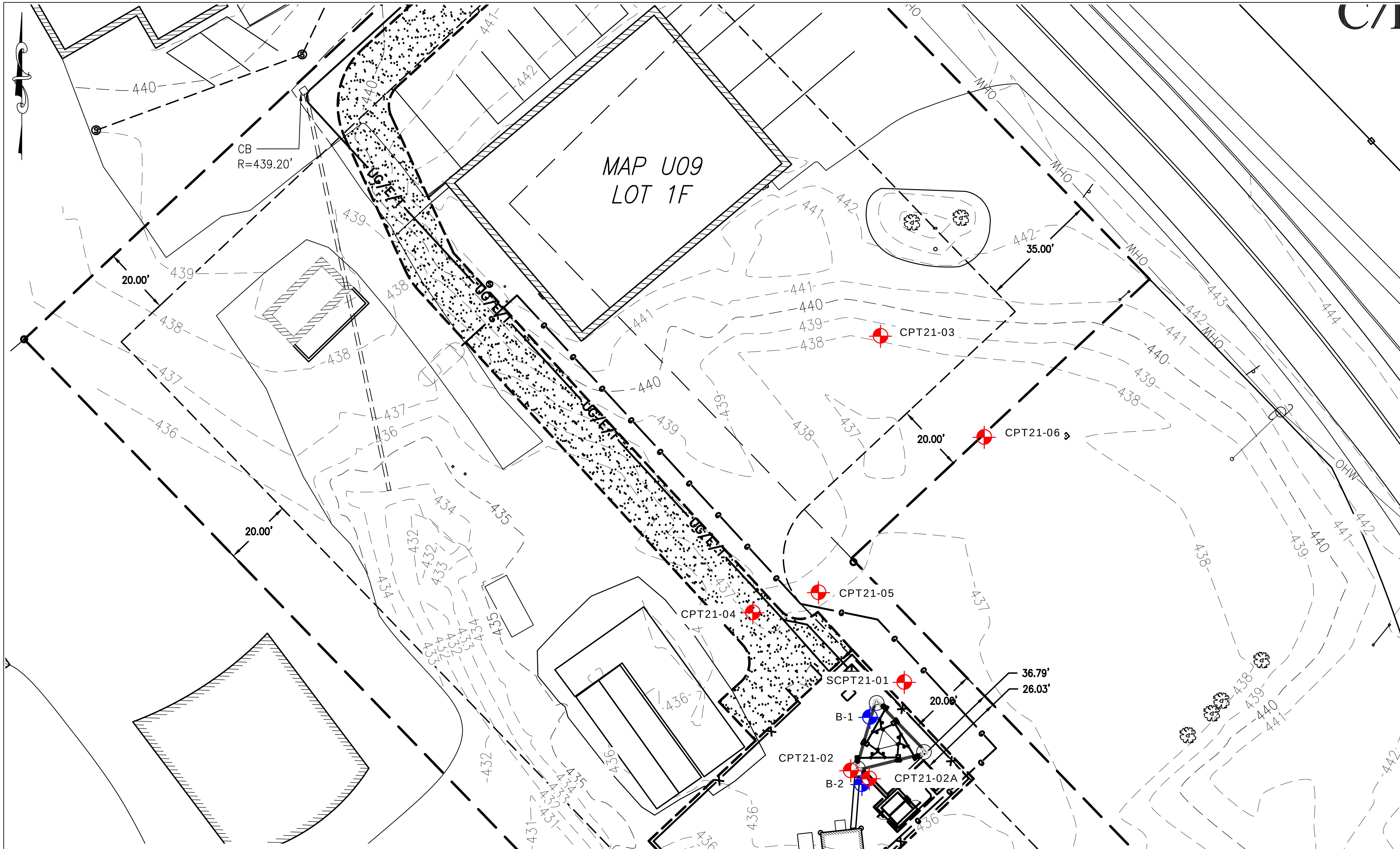
0 0.5 1.0 MILES

0 0.5 1 KILOMETERS

1:24,000 SCALE NORTH AMERICAN VERTICAL DATUM 1988 20 FOOT CONTOUR INTERVAL

O:\J3300\3310 Wireless EDGE Towers\02-01 Northfield, NH Telecommunications Tower - Geotech\Figures\Figure 1 - Site Locus.pdf

O'Reilly, Talbot & Okun ENGINEERING ASSOCIATES 293 Bridge Street, Suite 500 Springfield, MA 01103 413.788.6222 www.OTO-ENV.com	178' SELF-SUPPORTING LATTICE TOWER 149 PARK STREET NORTHFIELD, NEW HAMPSHIRE SITE LOCUS	Topographic Map Quadrants: NORTHFIELD, NH Map Version: 2015 Current As Of: 2015 Date: FEBRUARY 2021	PROJECT No. J3310-02-01 FIGURE No. 1



NOTES:

1. PLAN CREATED IN PART FROM PLAN TITLED "SITE PLAN" BY CHAPPELL ENGINEERING ASSOCIATES, LLC. DATED 9/21/2020
2. SAMPLE LOCATIONS ARE SHOWN ACCORDING TO TAPED MEASUREMENTS TAKEN FROM EXISTING SITE FEATURES
3. ALL DATA IS TO BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD(S) USED IN THE DEVELOPMENT OF THIS PLAN

LEGEND:

- APPROXIMATE SOIL BORING LOCATIONS PERFORMED BY NEW ENGLAND BORING CONTRACTORS ON JANUARY 27, 2021
- APPROXIMATE CPT SOUNDING LOCATIONS PERFORMED BY CONETEC ON SEPTEMBER 22, 2021

SCALE IN FEET
1" = 30'
0' 15' 30' 60'

O'Reilly, Talbot & Okun
ENGINEERING ASSOCIATES
293 Bridge Street, Suite 500 Springfield, MA 01103 413.788.6222
www.OTO-ENV.com

Designed By: JE
Drawn By: JE
Checked By:
Date: 10/27/2021
Revised Date:

178' SELF-SUPPORTING LATTICE TOWER
149 PARK STREET
NORTHFIELD, NEW HAMPSHIRE

SITE SKETCH

PROJECT NO.
J3310-02-03

FIGURE NO.
2

O:\J3300\3310 Wireless EDGE Towers\02-03 Northfield NH Telecommunication - Supplemental Geotech\Figure 2 - Site Sketch.pdf

(603) 437-1610 | New England Boring Contractors | Fax: (603) 437-0034
P.O. Box 165
Derry, NH 03038
E-Mail: nebc@neboring.com

Boring # B-1 **Project**: Seaboard Drilling – Northfield, NH **Project #** 175052

Project Address: 149 Park St **City:** Northfield **State:** NH **Zip:** 03276

Date Start: 01/27/2021 **Date End:** 01/27/2021 **Location:** See Plan

Casing Type: HW	**Sampler:**	**Casing:** 4"ID	**Sampler:** SS
Hammer: 300lb.	S/S	Size:	1-3/8 in.
	140lbs	Fall:	30in.

G R O U N D W A T E R   O B S E R V A T I O N

Date:	Depth:	Casing:	Stabilization Period
1/27/21	5'		

DP	S#	DEPTH	PEN	REC	BLOWS/6"	S/C	SAMPLE DESCRIPTION
-	S-1	0'-2'	24"	24"	17-52-72-25		Brown, fine to coarse SAND, some Gravel, trace inorganic Silt. Dry.
-							
-	S-2	2'-4'	24"	11"	11-14-20-14		Brown, fine to coarse SAND, some Gravel, trace inorganic silt. Dry.
-							
5'0"	S-3	4'-6'	24"	22"	9-8-7-8		Brown, FINE SAND, Trace medium to coarse sand, trace Inorganic silt. Wet.
-							
-	S-4	6'-8'	24"	18"	9-6-5-6		Brown, wet, FINE SAND, trace medium to coarse sand, Trace inorganic Silt.
-							
-	S-5	9'-11'	24"	10"	2-2-1-2		Brown, wet, FINE SAND, Trace medium to coarse sand, Trace inorganic Silt
10'0"							
-							
-							
-							
-	S-6	14'-16'	24"	13"	2-3-7-7	14'	Gray, wet, FINE SAND, Some Inorganic Silt.
15'0"							
-							
-							
-							
-	S-7	19'-21'	24"	15"	1-1-1-1		Gray, Wet, INORGANIC SILT, Some Fine Sand.
20'0"							
-							
-							
-							
-	S-8	24'-26'	24"	17"	1-1-1-1		Gray, wet, INORGANIC SILT, Some Fine Sand.
25'0"							
-							
-							
-							
-	S-9	29'-31'	24"	18"	WHO-2-2-2		Gray, wet, INORGANIC SILT, Some Fine Sand.
30'0"							
-							

Driller: Walter Hoeckele **Helpers:** Mike Misiaszek **Inspector:**

Remarks: Page 1 of 2 - continued on next page

S/#: Sample	**PEN:** Penetration	**REC:** Recovery	**S/C:** Strata Change

(603) 437-1610 | New England Boring Contractors | Fax: (603) 437-0034
P.O. Box 165
Derry, NH 03038
E-Mail: nebc@neboring.com

Boring # B-1 continued **Project**: Seaboard Drilling – Northfield, NH **Project #** 175052

Project Address: 149 Park St **City:** Northfield **State:** NH **Zip:** 03276

Date Start: 01/27/2021 **Date End:** 01/27/2021 **Location:** See Plan

Casing Type: HW	**Sampler:**	**Casing:** 4"ID	**Sampler:** SS
Hammer: 300lb.	S/S	Size:	1-3/8 in.
	140lbs	Fall:	30in.

G R O U N D W A T E R O B S E R V A T I O N

Date:	**Depth:**	**Casing:**	**Stabilization Period**
1/27/21	5'		

DP	S#	DEPTH	PEN	REC	BLOWS/6"	S/C	SAMPLE DESCRIPTION
-							
-							
-	S-10	24'-36'	24"	19"	2-3-3-4		Gray, Wet, INORGANIC SILT, Some Fine Sand, Trace Clay.
35'0"							
-							
-							
-							
-	S-11	39'-41'	24"	15"	2-2-2-4		Gray, Wet, INORGANIC SILT, Some Fine Sand, Trace Clay.
40'0"							
-							
-							
-							
-	S-12	44'-46'	24"	12"	2-1-2-11		Gray, Wet, INORGANIC SILT, Some Fine Sand, Trace Clay
45'0"						45.5'	Gray, Wet, Fine to Coarse SAND, Trace Gravel, Trace Inorganic Silt.
-							
-							
-							
-	S-13	49'-51'	24"	17"	24-39-56-68		Gray, Wet Fine to Coarse SAND, Trace Gravel, Trace Inorganic Silt.
50'0"							- BOE 51'
-							
-							
-							
-							
55'0"							
-							
-							
-							
-							
60'0"							
-							
-							

Driller: Walter Hoeckele **Helpers:** Mike Misiaszek **Inspector:**

Remarks: Page 2 of 2 **BOE 51'**

S/#: Sample	**PEN:** Penetration	**REC:** Recovery	**S/C:** Strata Change

(603) 437-1610 | New England Boring Contractors | Fax: (603) 437-0034
P.O. Box 165
Derry, NH 03038
E-Mail: nebc@neboring.com

Boring # B-2 **Project**: Seaboard Drilling – Northfield, NH **Project #** 175052

Project Address: 149 Park St **City:** Northfield **State:** NH **Zip:** 03276

Date Start: 01/27/2021 **Date End:** **Location:** See Plan

Casing Type: HW	Sampler:	Casing: 4"ID	Sampler: SS
Hammer: 300lb.	S/S	Size:	1-3/8 in.
	140lbs	Fall:	30in.

G R O U N D W A T E R O B S E R V A T I O N

Date:	Depth:	Casing:	Stabilization Period
1/27/21	7'		

DP	S#	DEPTH	PEN	REC	BLOWS/6"	S/C	SAMPLE DESCRIPTION
-	S-1	0'-2'	24"	21"	16-32-26-19		Brown, Dry, Fine to Coarse SAND, Some Gravel, Trace Inorganic Silt.
-							
-	S-2	2'-4'	24"	7"	14-11-11-8		Brown, Dry, Fine to Coarse SAND, Some Gravel, Trace Inorganic Silt.
-							
5'0"	S-3	4'-6'	24"	5"	6-5-8-8		Brown, Dry, Fine to Coarse SAND, Some Gravel, Trace Inorganic Silt.
-						6.5'	
-	S-4	6'-8'	24"	23"	8-8-7-9		Brown, Wet, FINE SAND, Trace medium to coarse Sand, Trace Inorganic Silt.
-							
-	S-5	9'-11'	24"	9"	5-4-5-9		Brown, Wet, FINE SAND, Some Inorganic Silt.
10'0"							
-							
-							
-							
-	S-6	14'-16'	24"	12"	3-4-5-4		Brown, Wet, FINE SAND, Some Inorganic Silt.
15'0"							
-							- BOE 16'
-							
-							
-							
20'0"							
-							
-							
-							
-							
25'0"							
-							
-							
-							
-							
30'0"							
-							

Driller: Walter Hoeckele	Helpers: Mike Misiaszek	Inspector:

Remarks: BOE 16'

S/#: Sample	PEN: Penetration	REC: Recovery	S/C: Strata Change

PRESENTATION OF SITE INVESTIGATION RESULTS

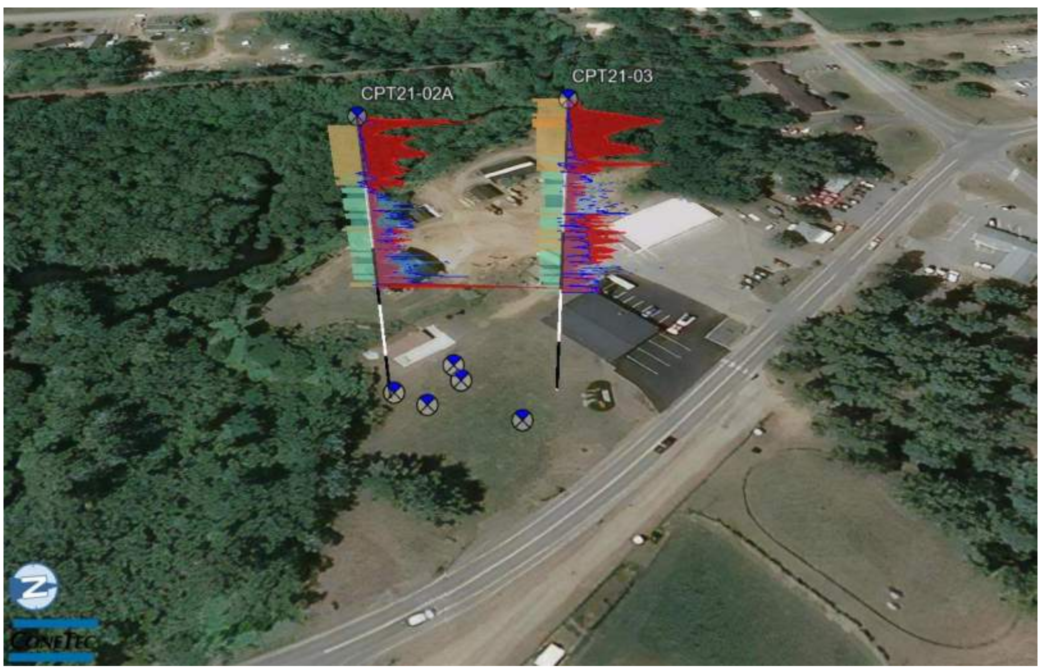
149 Park Street
Northfield, New Hampshire

Prepared for:

O'Reilly Talbot & Okun Engineering Associates

ConeTec Job No: 21-53-23025

Project Start Date: 22-Sep-2021
Project End Date: 22-Sep-2021
Report Date: 30-Sep-2021



Prepared by:

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www.conetecdataservices.com

Introduction

The enclosed report presents the results of a piezocone penetration testing (CPTu or CPT) and seismic piezocone penetration testing (SCPTu or SCPT) program carried out at 149 Park Street in Northfield, New Hampshire. The site investigation program was conducted by ConeTec Inc. (ConeTec), under contract to O'Reilly Talbot & Okun Engineering Associates (OTO Engineering Associates) of Springfield, Massachusetts.

A total of 6 cone penetration tests and 1 seismic cone penetration test were completed at 7 locations (CPT21-02 was offset and attempted again due to shallow refusal). The CPT and SCPT program was performed to evaluate the subsurface soil conditions. CPT and SCPT sounding locations were selected and numbered under supervision of OTO Engineering Associates personnel (Ashley Sullivan).

Project Information

Project	
Client	OTO Engineering Associates
Project	149 Park Street, Northfield, NH
ConeTec project number	21-53-23025

A map from CESIUM including the CPT and SCPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT Truck Rig	25 ton truck mounted (twin cylinders)	CPT and SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT and SCPT	GPS (GlobalSat MR-350)	32619 (WGS 84 / UTM North)

Cone Penetration Test (CPT)	
Depth reference	Ground surface at the time of the investigation.
Tip and sleeve data offset	0.1 meter. This has been accounted for in the CPT data files.
Pore pressure dissipation (PPD) tests	Eight pore pressure dissipation tests were completed primarily to determine the phreatic surface and consolidation characteristics.
Additional plots	Advanced, Seismic and Soil Behavior Type (SBT) scatter plots are included in the data release package.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
558:T1500F15U35	558	15	225	1500	15	35
Cone 558 was used for each sounding.						

Calculated Geotechnical Parameters Tables	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Qtn) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT 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 behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behavior Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of OTO Engineering Associates (Client) for the project titled “149 Park Street, Northfield, NH”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec. ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

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

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

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

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm 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 6 mm 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 meet or exceed those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

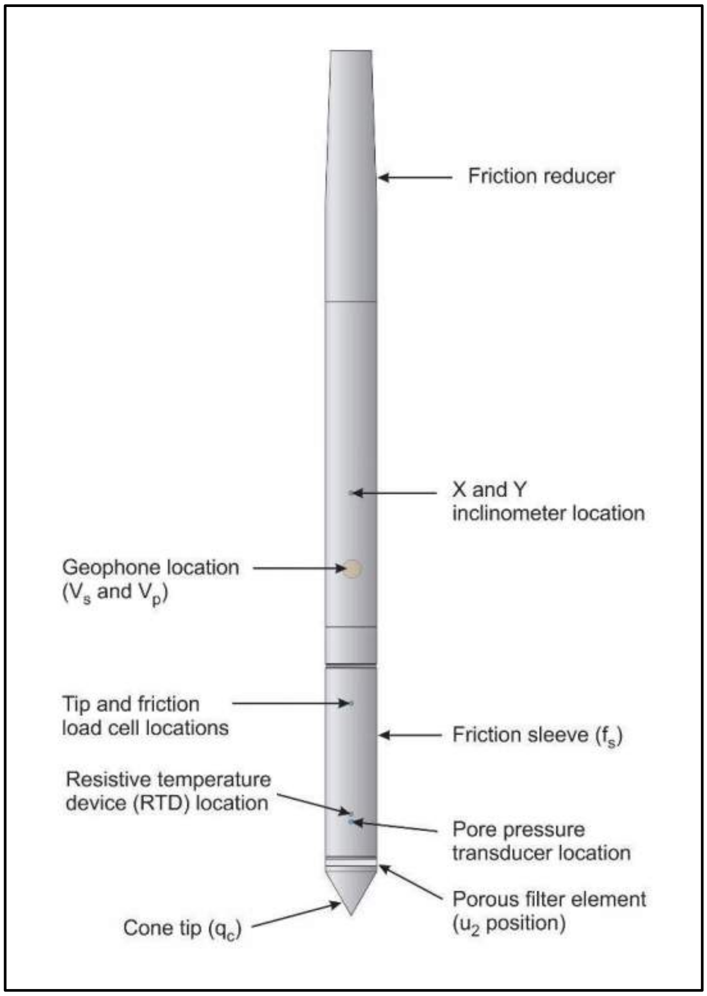


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 cm; 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 CPT 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 either glycerin or 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 2 cm/s, 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
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

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

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

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

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

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

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

The friction ratio (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.

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

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

References

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

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.

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

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

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

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

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

Shear wave velocity (Vs) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity (Vp) testing is also performed.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

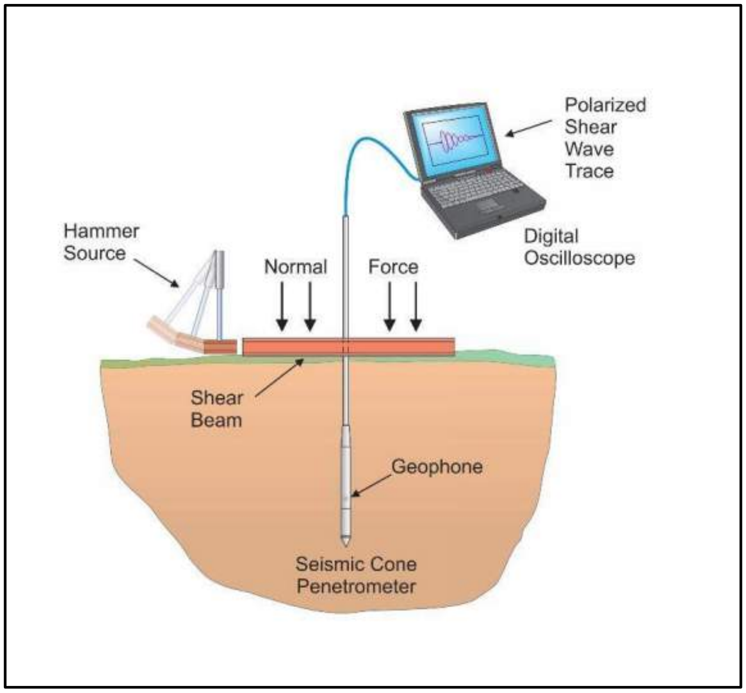


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM 5778 and ASTM D7400 standards.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control purposes and uncertainty analysis. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et. al. (1986).

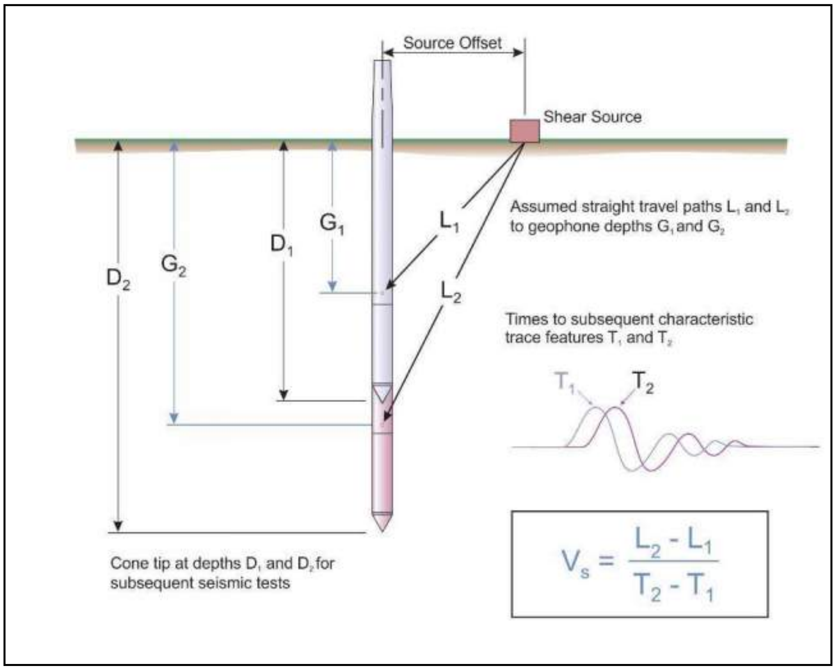


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

For all SCPTu soundings that have achieved a depth of at least 100 feet (30 meters), the average shear wave velocity to a depth of 100 feet ($\bar{v}_s$) has been calculated using the following equation presented in ASCE (2010).

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_s$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = 100 ft (30 m)

Average shear wave velocity, $\bar{v}_s$ is also referenced to V_{s100} or V_{s30}.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

References

American Society of Civil Engineers (ASCE), 2010, “Minimum Design Loads for Buildings and Other Structures”, Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

ASTM D7400-14, 2014, "Standard Test Methods for Downhole Seismic Testing", ASTM, West Conshohocken, US.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803.

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

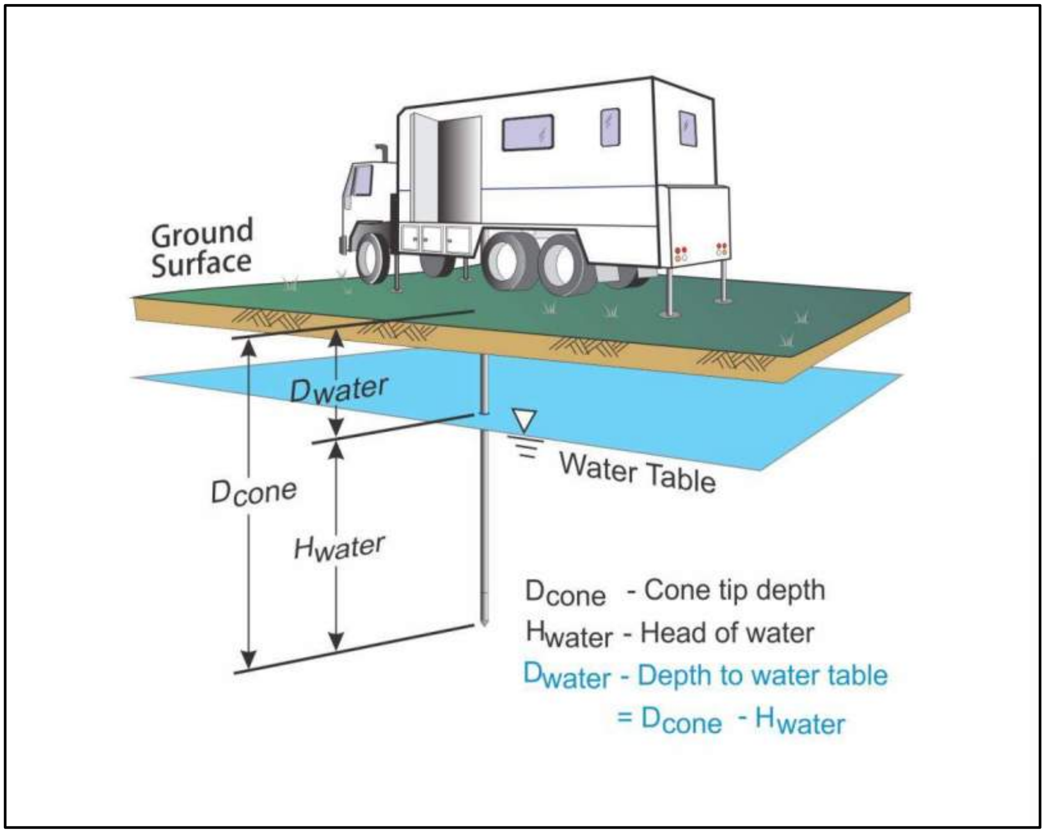


Figure PPD-1. Pore pressure dissipation test setup

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

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

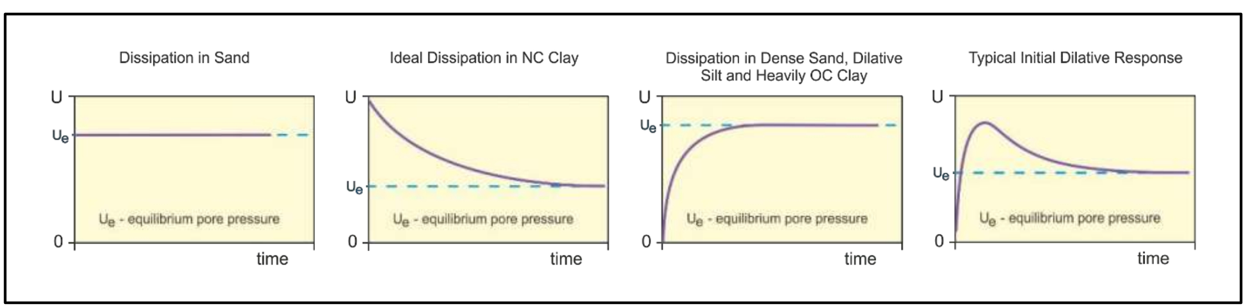


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

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

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

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

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

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

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

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

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

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

References

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

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

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

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

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

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

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N1(60)Ic
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave (Vs) Traces
- Seismic Cone Penetration Test Tabular Results
- Soil Behavior Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No:	21-53-23025
Client:	OTO Engineering Associates
Project:	149 Park Street, Northfield, NH
Start Date:	22-Sep-2021
End Date:	22-Sep-2021

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Shear Wave Velocity Tests	Northing[2] (m)	Easting[2] (m)	Refer to Notation Number
SCPT21-01	21-53-23025_SP01	22-Sep-2021	558:T1500F15U35	4.9	50.03	14	4812043	290345	
CPT21-02	21-53-23025_CP02	22-Sep-2021	558:T1500F15U35		1.97		4812035	290340	4
CPT21-02A	21-53-23025_CP02A	22-Sep-2021	558:T1500F15U35	5.0	43.88		4812034	290342	3
CPT21-03	21-53-23025_CP03	22-Sep-2021	558:T1500F15U35	4.0	50.03		4812074	290344	
CPT21-04	21-53-23025_CP04	22-Sep-2021	558:T1500F15U35	4.0	50.03		4812050	290332	3
CPT21-05	21-53-23025_CP05	22-Sep-2021	558:T1500F15U35	4.0	50.03		4812051	290338	3
CPT21-06	21-53-23025_CP06	22-Sep-2021	558:T1500F15U35	3.0	50.03		4812065	290353	3
Totals	7 soundings				296.01	14			

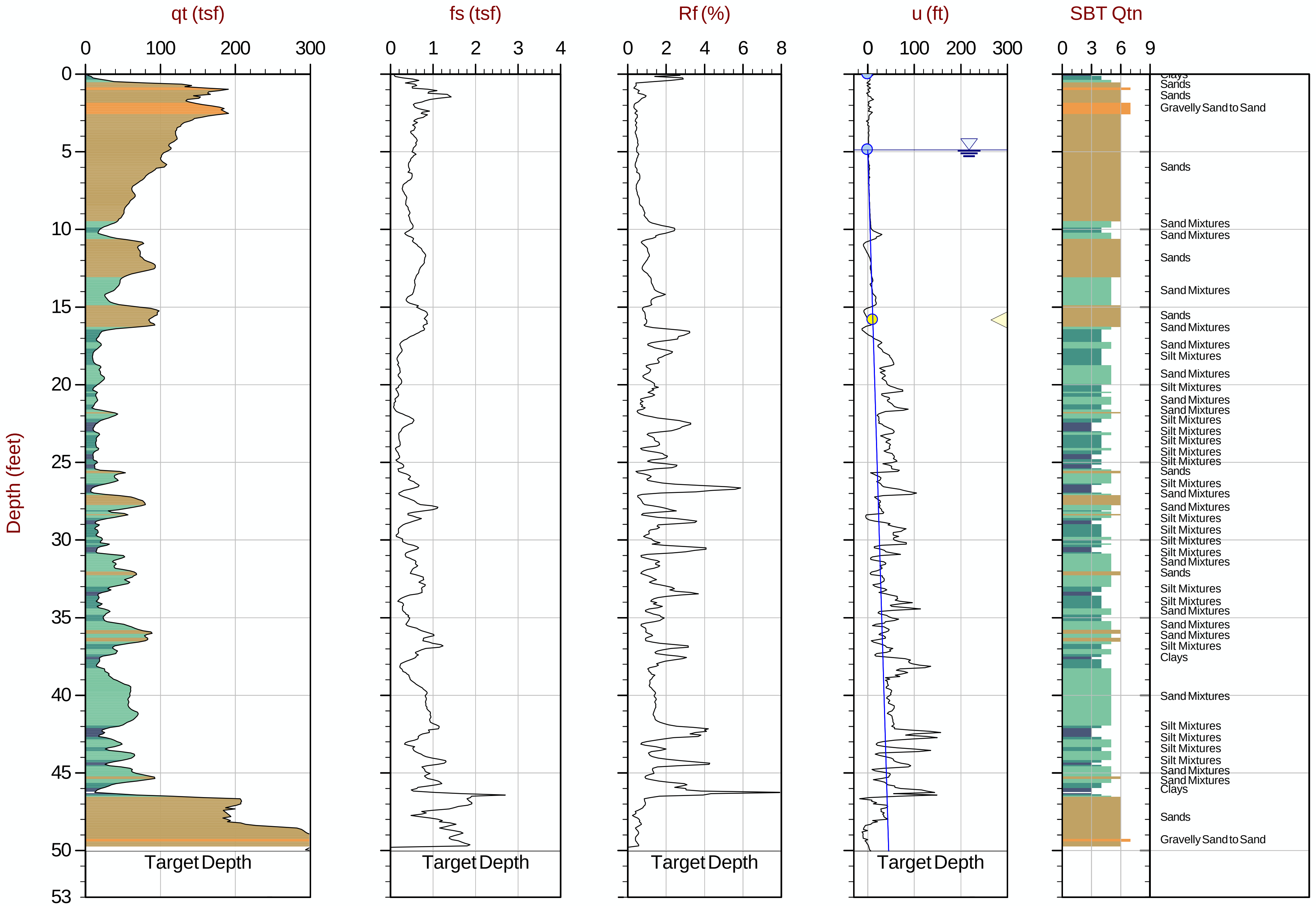
1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic data were used for the calculated parameters.
2. Coordinates were acquired using a MR-350 GlobalSat GPS Receiver in datum: WGS84 / UTM Zone 19 North.
3. The assumed phreatic surface was estimated from the dynamic pore pressure data.
4. No phreatic surface detected.

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 08:18
Site: 149 Park Street, Northfield, NH

Sounding: SCPT21-01
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812043m E: 290345m

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

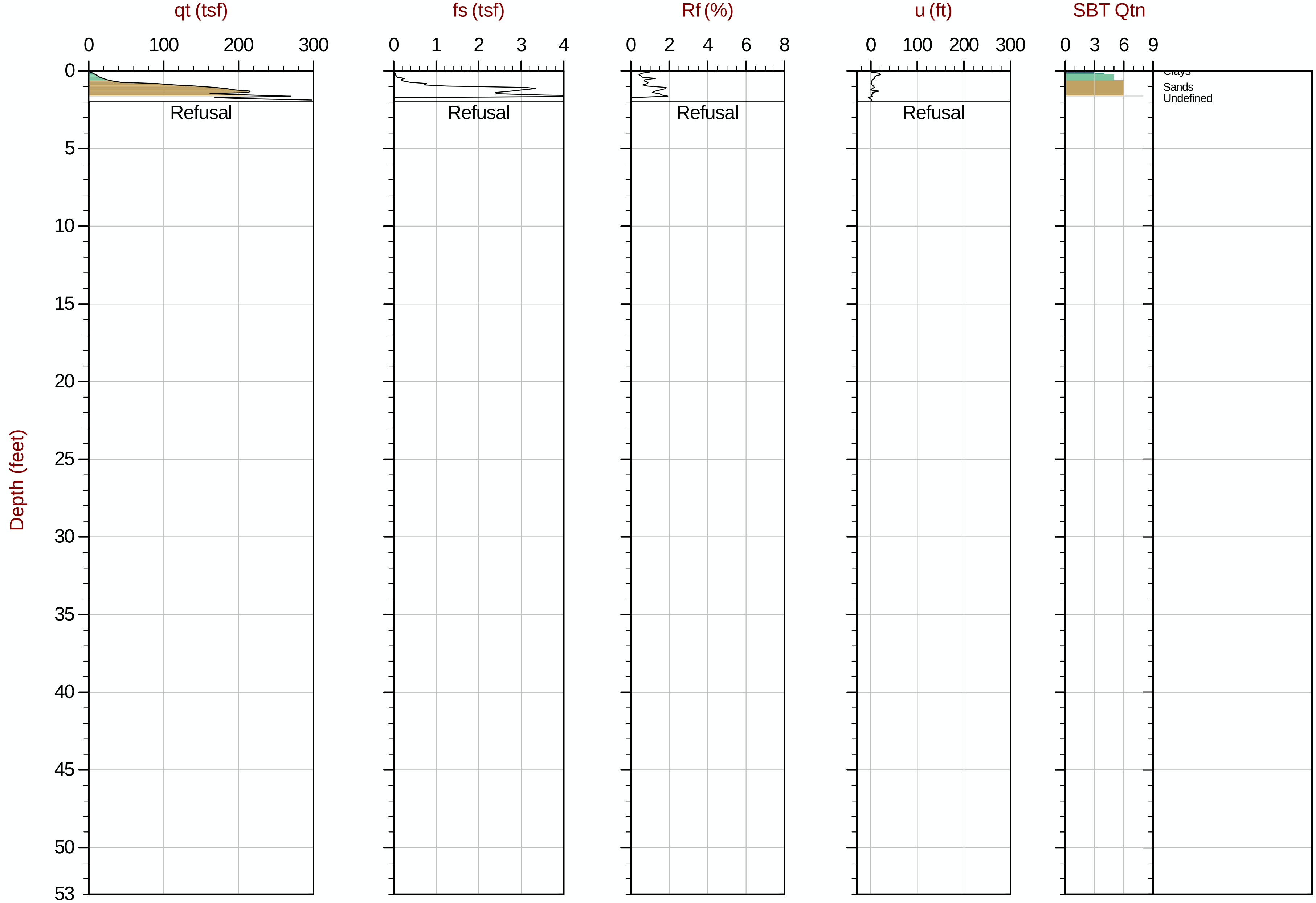
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 09:07
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-02
Cone: 558:T1500F15U35



Max Depth: 0.600 m / 1.97 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

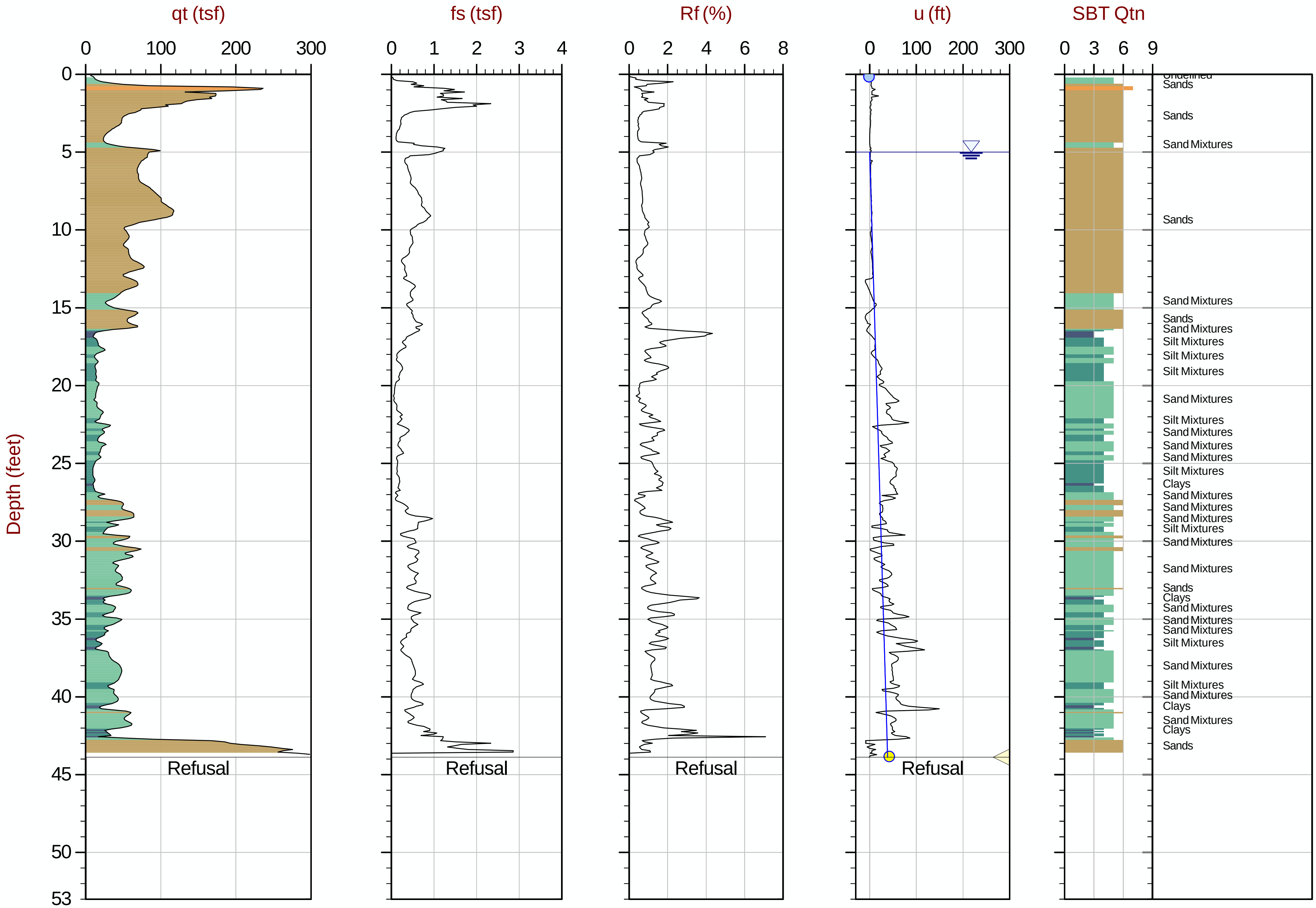
File: 21-53-23025_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812035m E: 290340m

Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

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

OTO Engineering Associates	Job No: 21-53-23025	Sounding: CPT21-02A
	Date: 2021-09-22 09:24	Cone: 558:T1500F15U35
	Site: 149 Park Street, Northfield, NH	



Max Depth: 13.375 m / 43.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP02A.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812034m E: 290342m

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

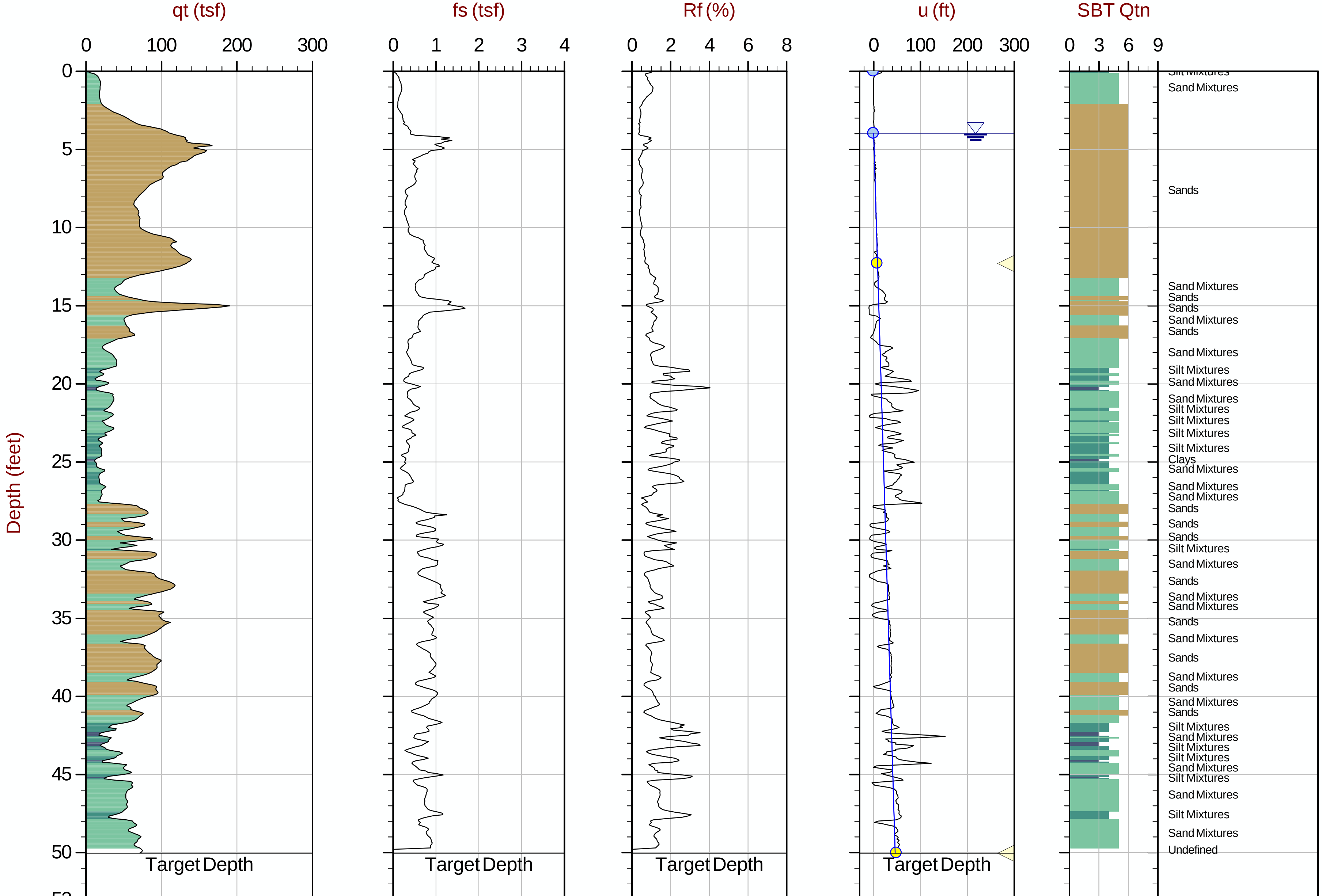
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 10:37
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-03
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812074m E: 290344m

Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

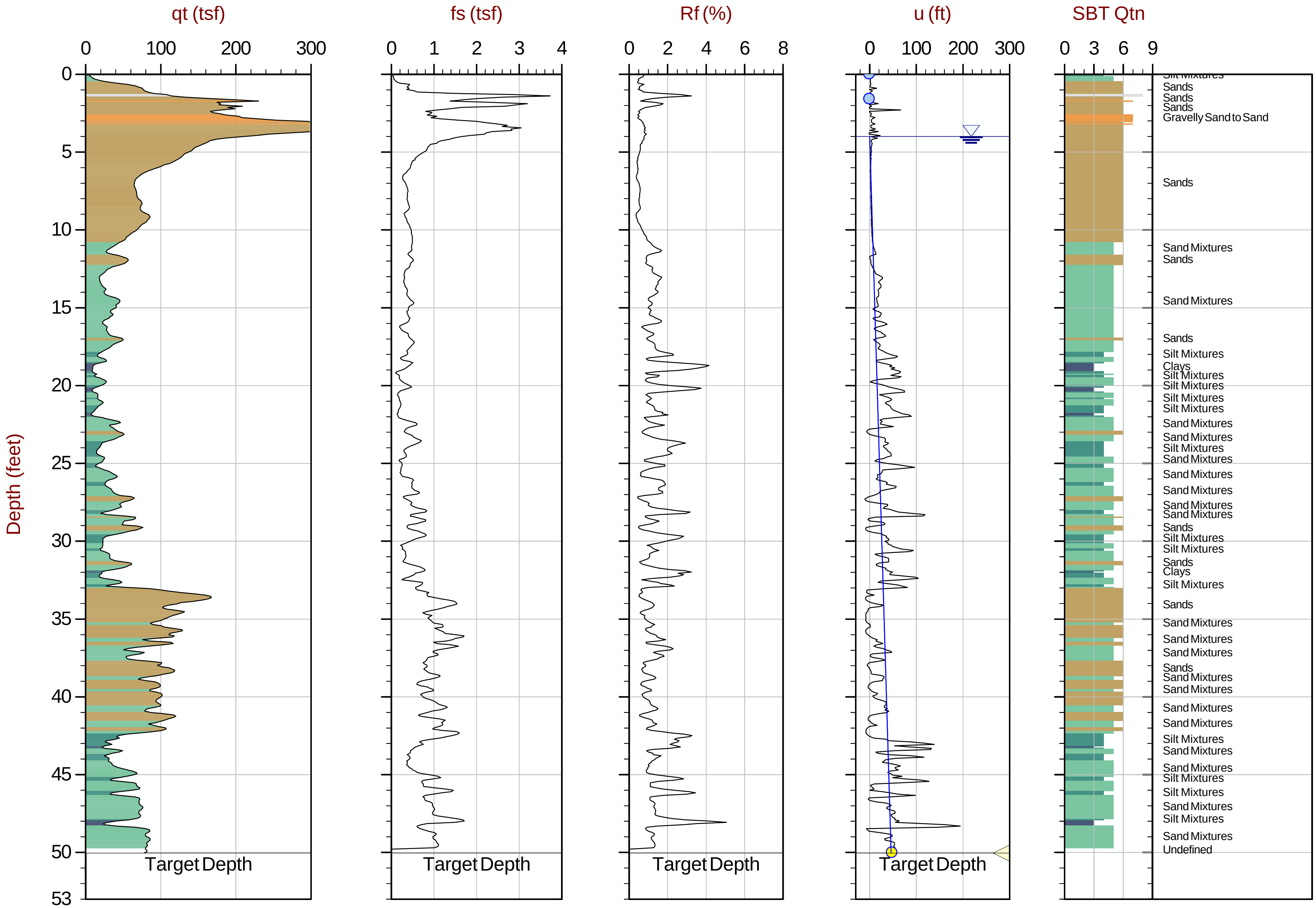
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 12:37
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-04
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812050m E: 290332m

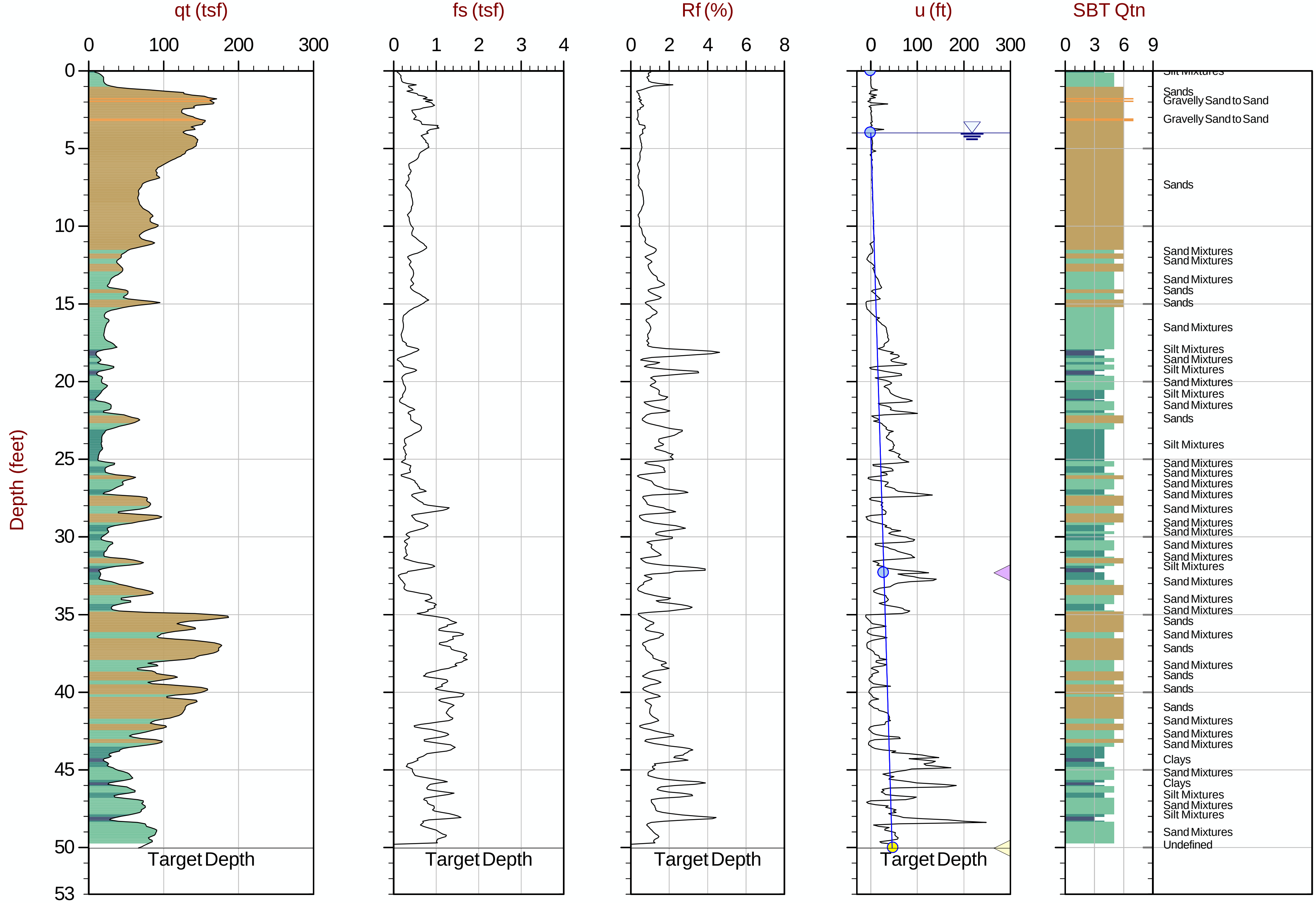
Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

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

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 11:31
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-05
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP05.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812051m E: 290338m

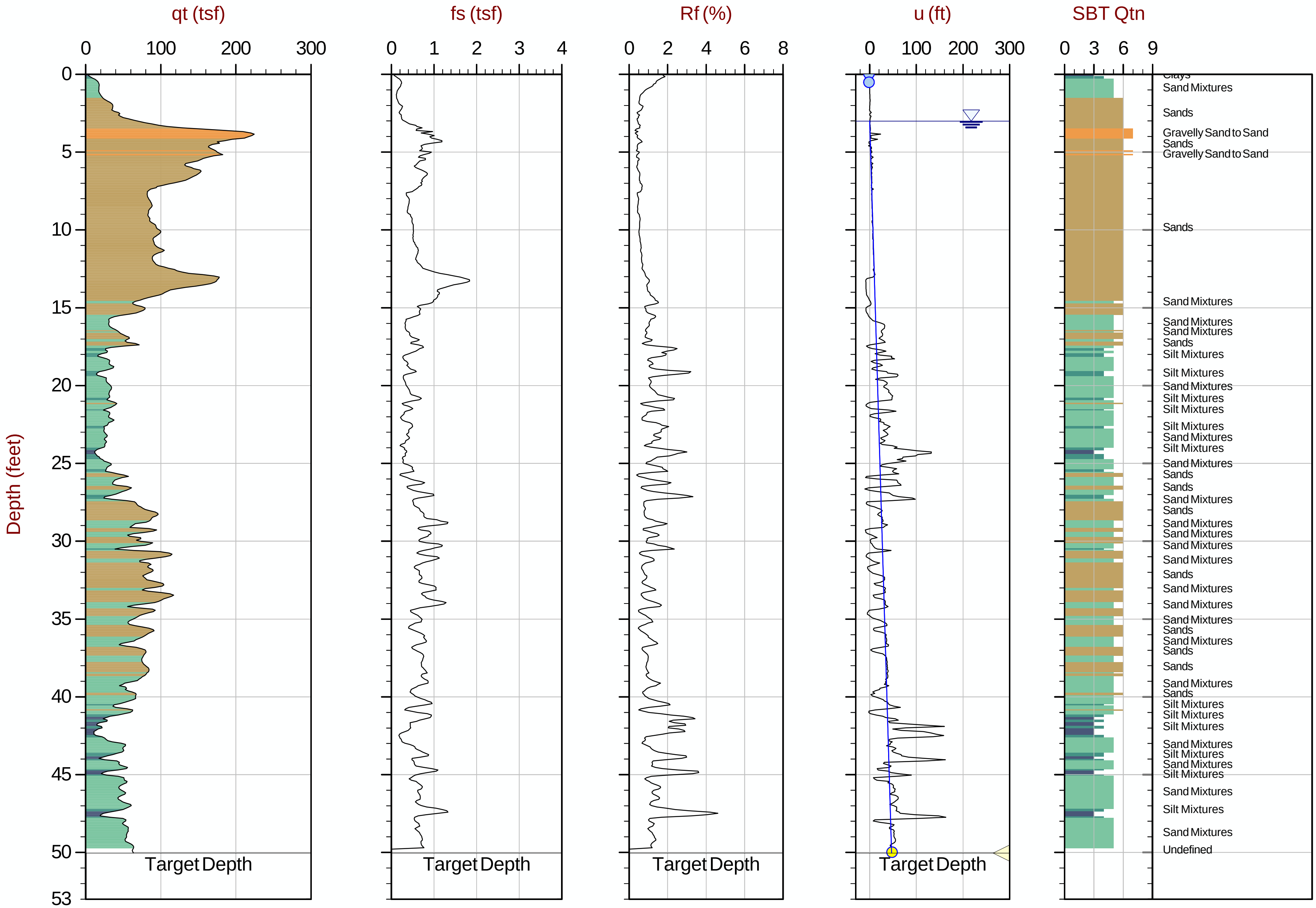
Hydrostatic Line | Ueq | Assumed Ueq | PPD, Ueq achieved | PPD, Ueq not achieved

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

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 09:58
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-06
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP06.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812065m E: 290353m

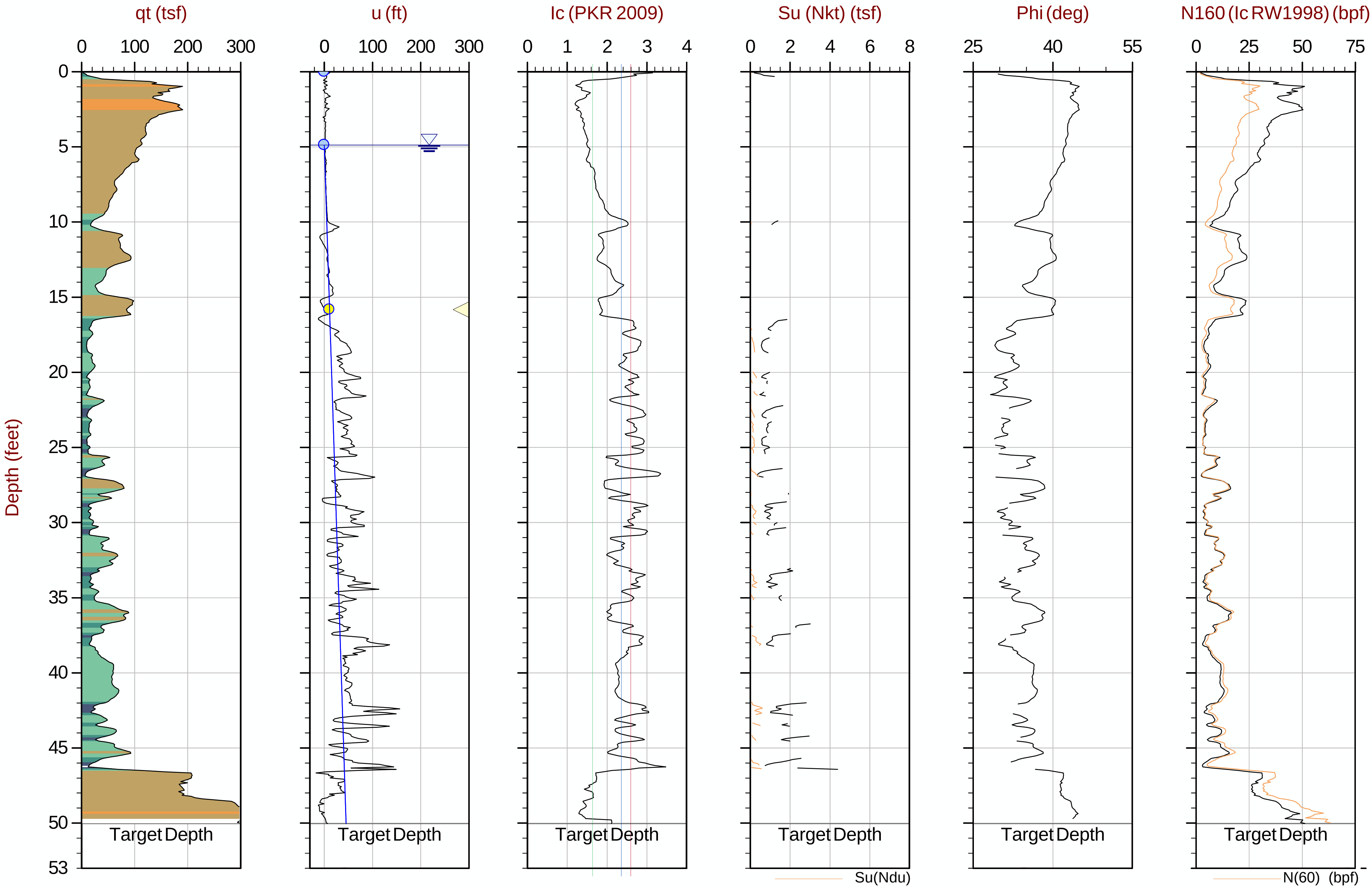
Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

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 Plots with Ic, Su(Nkt), Phi and N1(60)Ic

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 08:18
Site: 149 Park Street, Northfield, NH
Sounding: SCPT21-01
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812043m E: 290345m

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

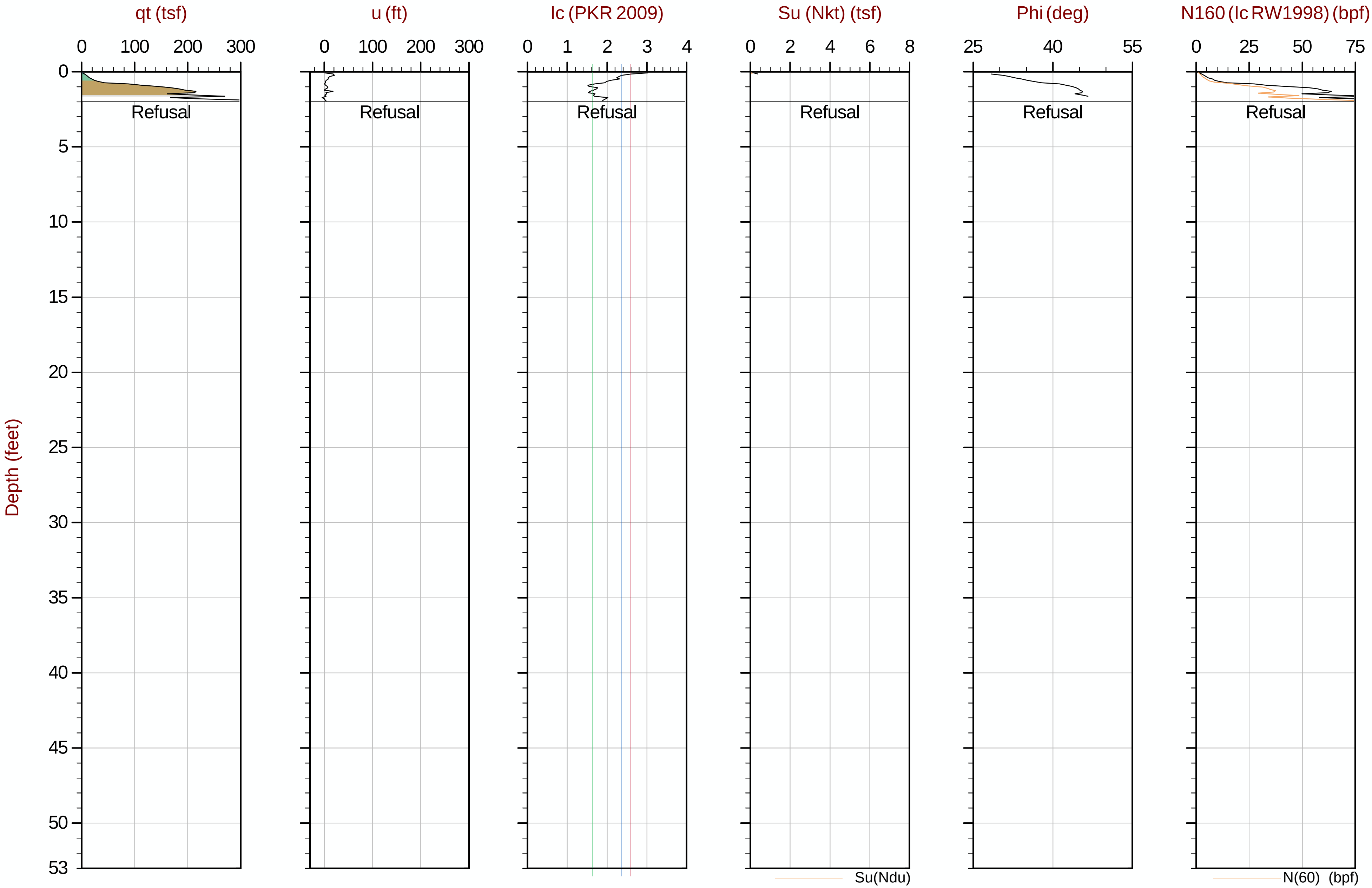
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 09:07
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-02
Cone: 558:T1500F15U35



Max Depth: 0.600 m / 1.97 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812035m E: 290340m

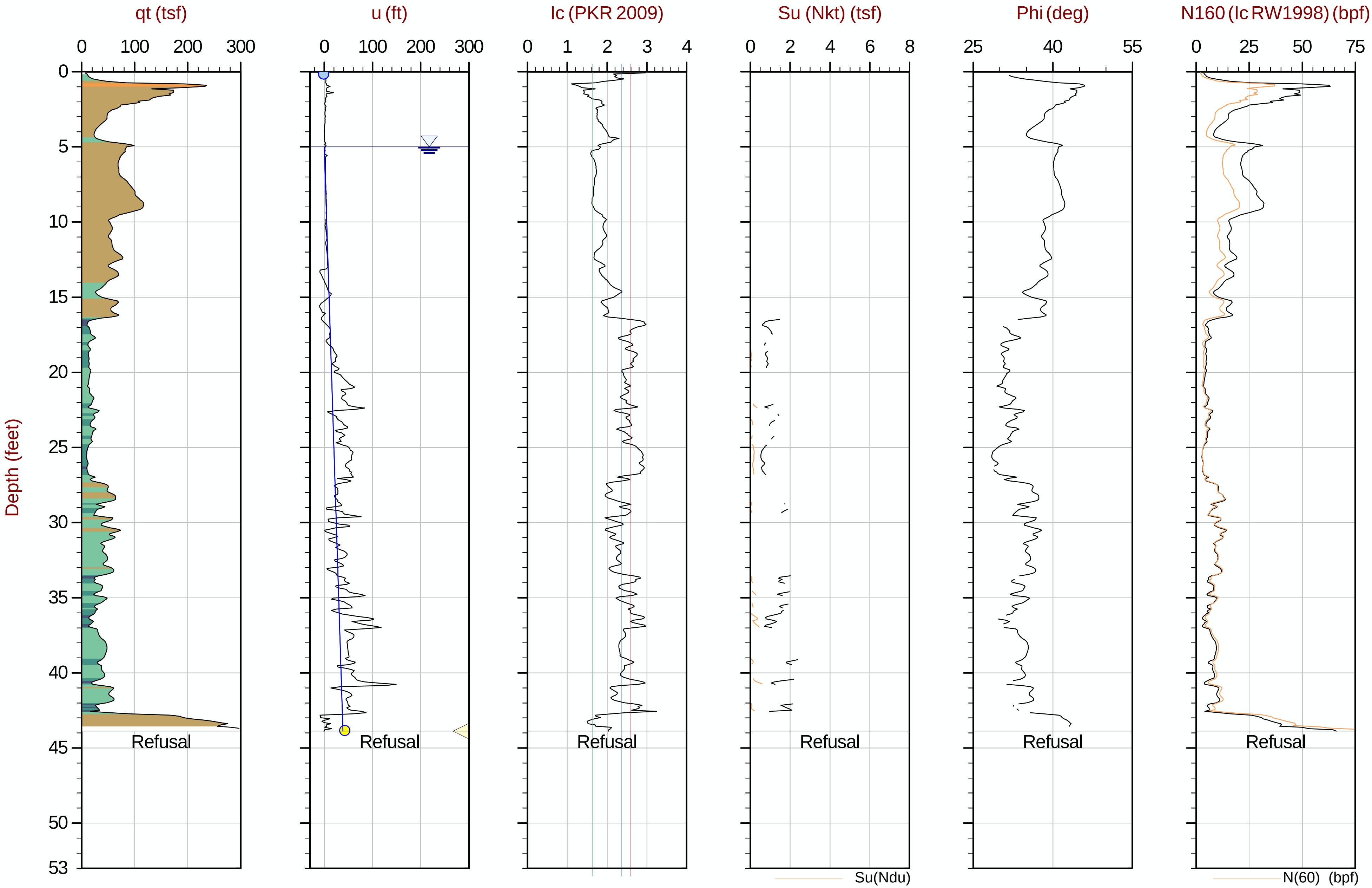
Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

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

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 09:24
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-02A
Cone: 558:T1500F15U35



Max Depth: 13.375 m / 43.88 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

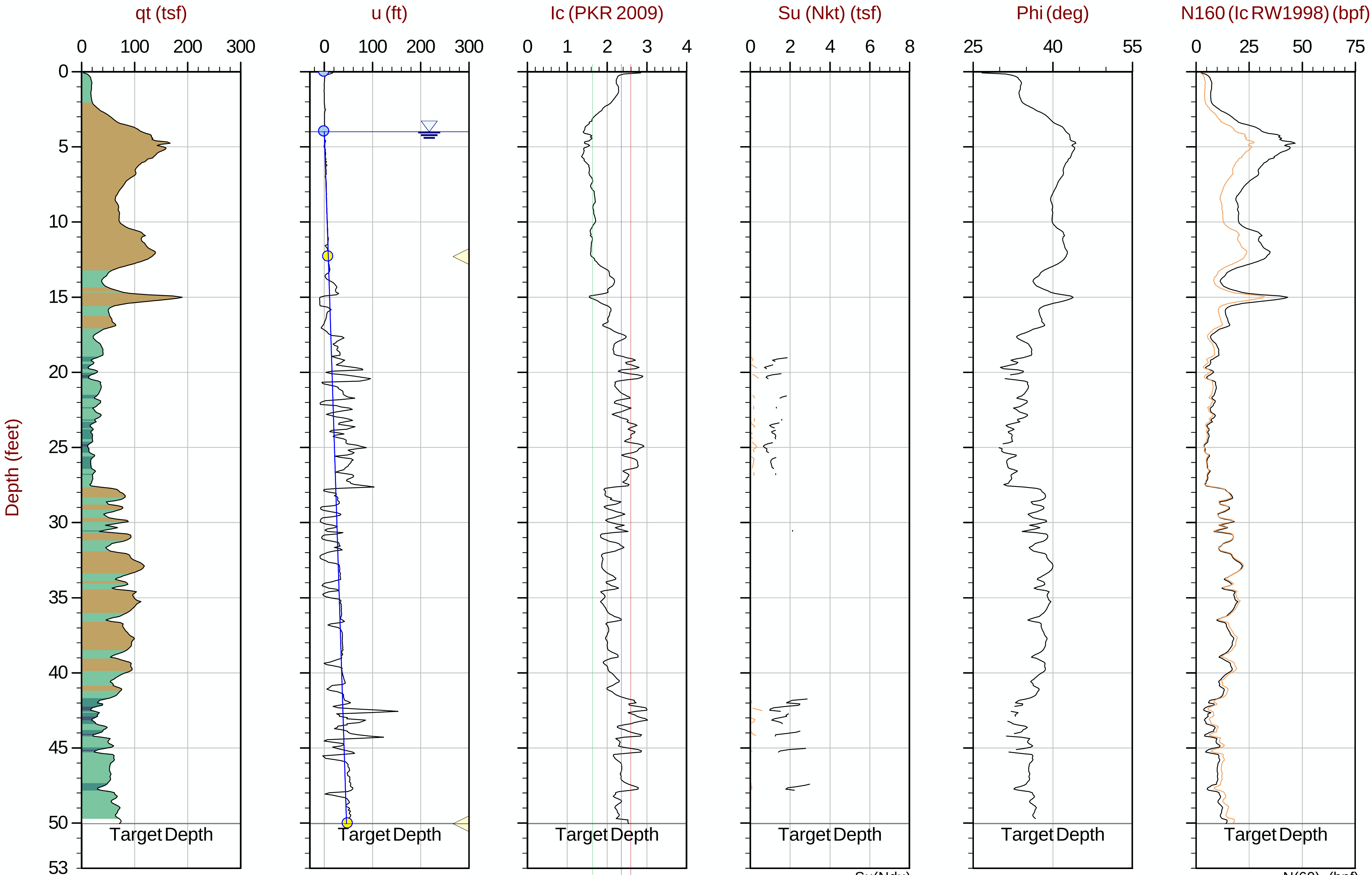
File: 21-53-23025_CP02A.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812034m E: 290342m

Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

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

OTO Engineering Associates	Job No: 21-53-23025	Sounding: CPT21-03
	Date: 2021-09-22 10:37	Cone: 558:T1500F15U35
	Site: 149 Park Street, Northfield, NH	



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812074m E: 290344m

Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

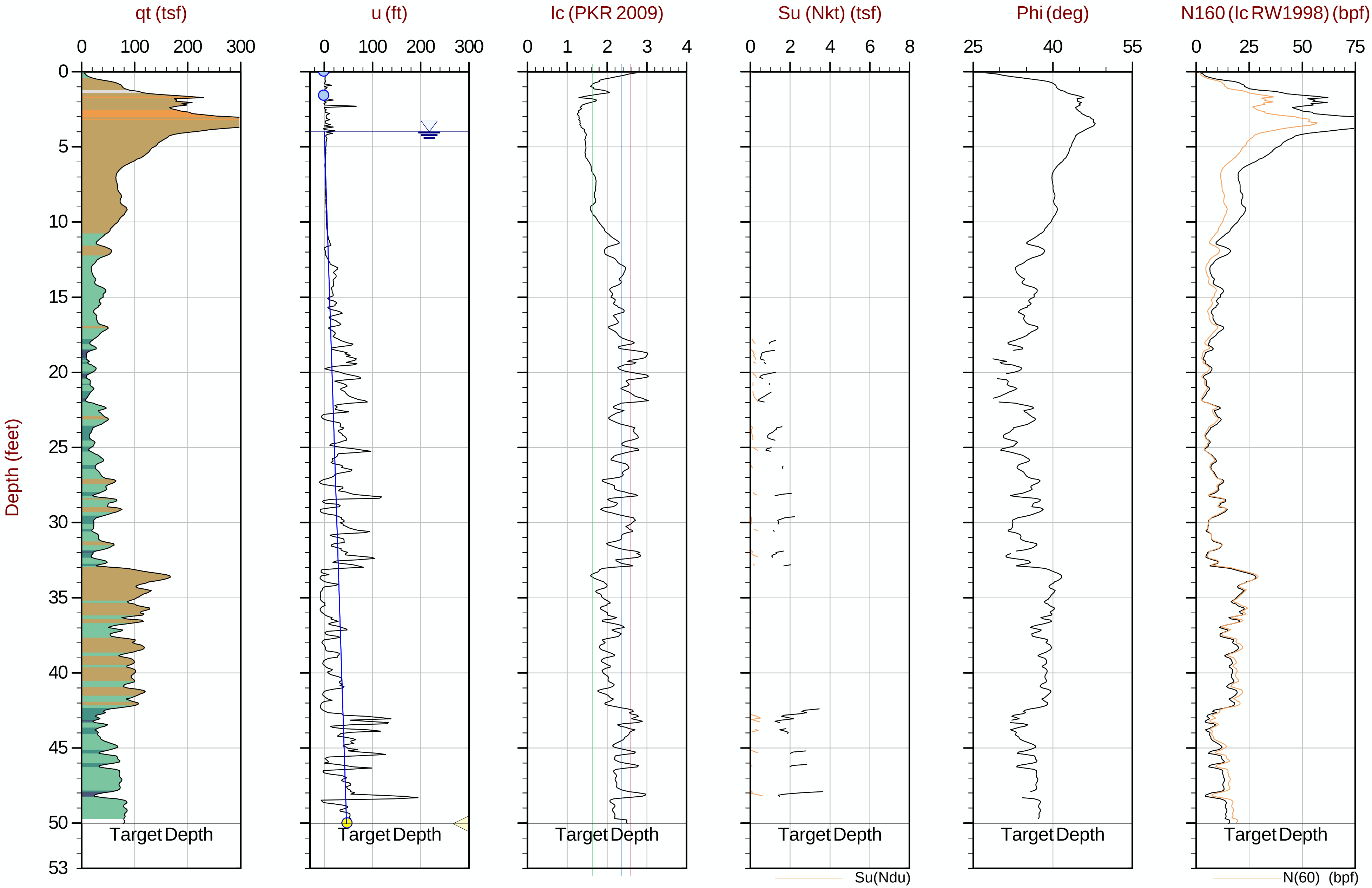
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 12:37
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-04
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812050m E: 290332m

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

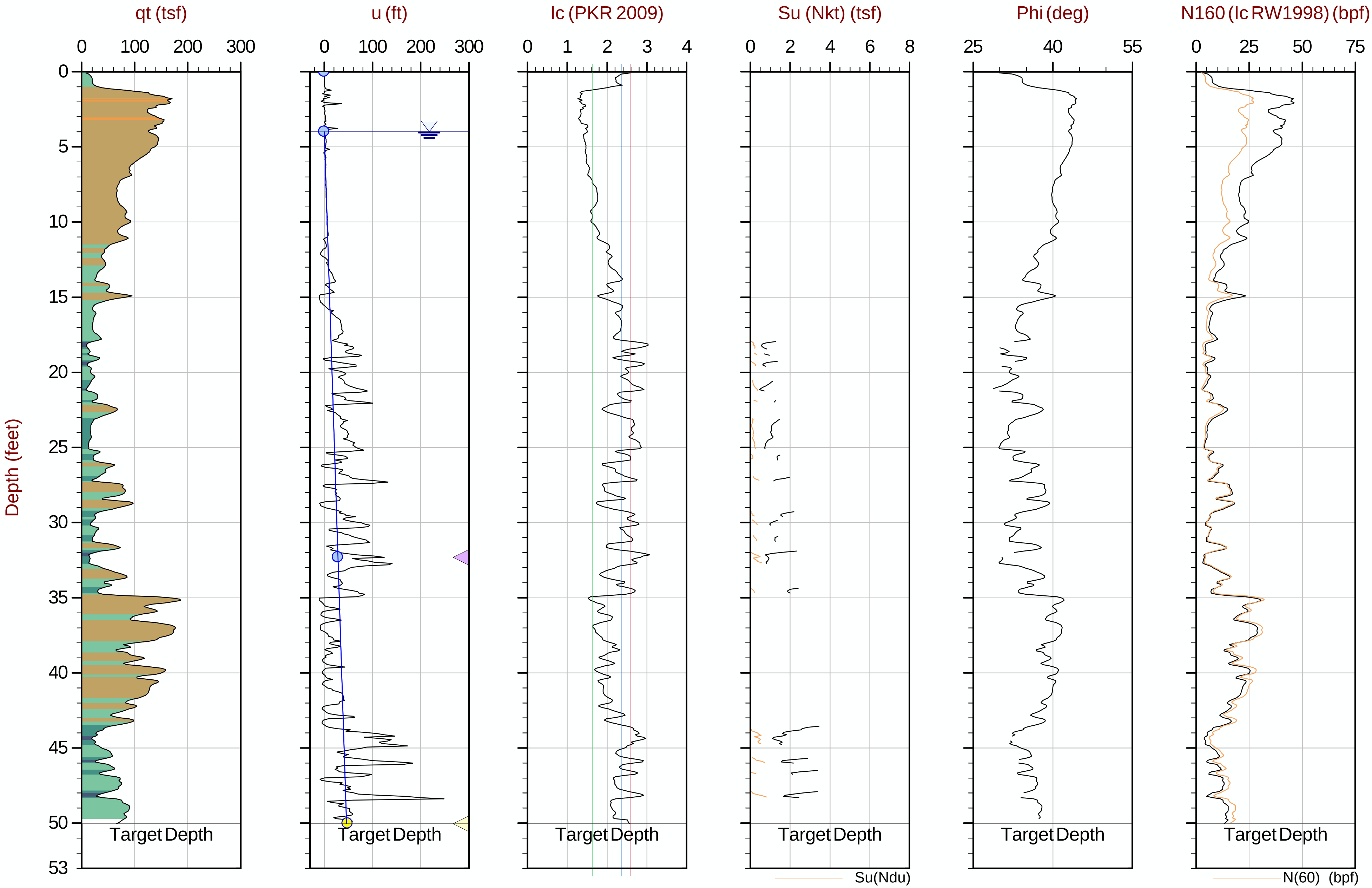
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 11:31
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-05
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP05.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

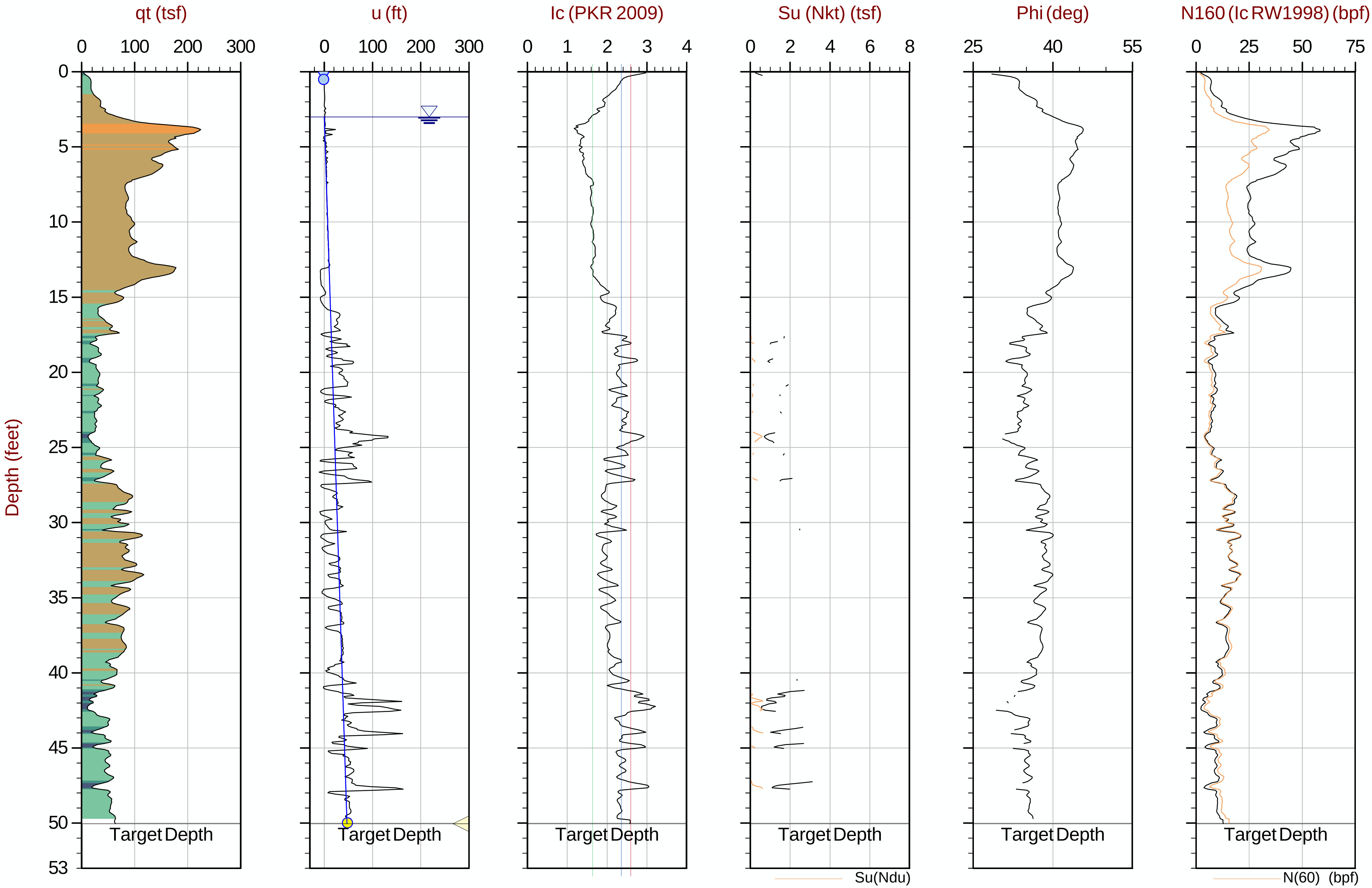
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812051m E: 290338m

Hydrostatic Line Ueq Assumed Ueq PPD, Ueq achieved PPD, Ueq not achieved

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

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 09:58
Site: 149 Park Street, Northfield, NH
Sounding: CPT21-06
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_CP06.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812065m E: 290353m

Hydrostatic Line   Ueq   Assumed Ueq   PPD, Ueq achieved   PPD, Ueq not achieved

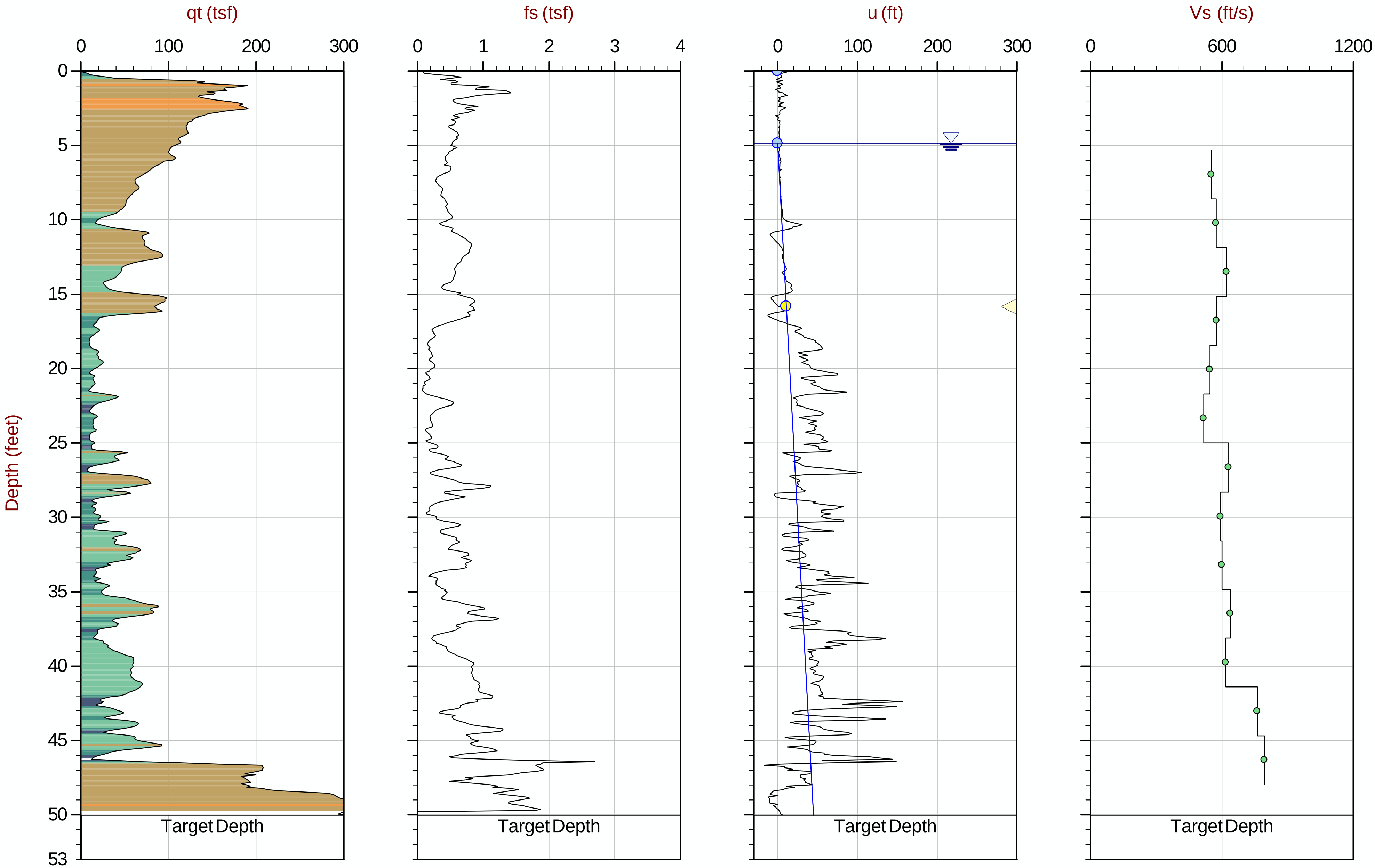
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Plots

OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 08:18
Site: 149 Park Street, Northfield, NH

Sounding: SCPT21-01
Cone: 558:T1500F15U35



Max Depth: 15.250 m / 50.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 21-53-23025_SP01.COR
Unit Wt: SBTQtn (PKR2009)

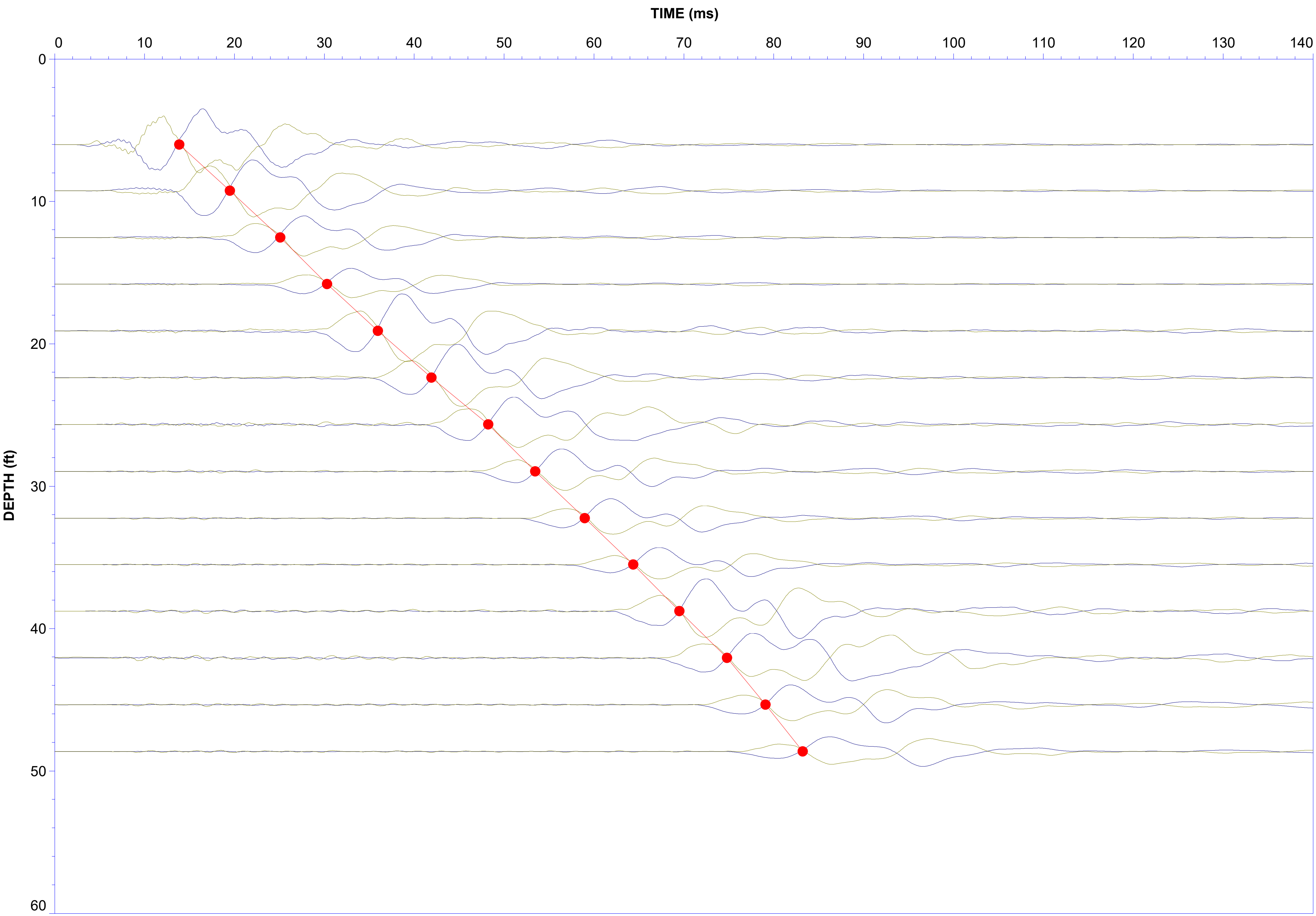
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 19 N: 4812043m E: 290345m

Hydrostatic Line · Ueq · Assumed Ueq · PPD, Ueq achieved · PPD, Ueq not achieved

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) Traces

Job No: 21-53-23025
Date: 22-Sep-2021
Client: OTO Engineering Associates
Project: 149 Park Street, Northfield, NH
Sounding: SCPT21-01
Filter: None



Seismic Cone Penetration Test Tabular Results

Job No: 21-53-23025
Client: OTO Engineering Associates
Project: 149 Park Street, Northfield, NH
Sounding ID: SCPT21-01
Date: 22-Sep-2021

Seismic Source: Beam
Source Offset (ft): 1.97
Source Depth (ft): 0
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)
6.00	5.35	5.70			
9.25	8.60	8.82	3.12	5.64	553
12.53	11.88	12.04	3.22	5.60	575
15.81	15.16	15.28	3.25	5.22	623
19.09	18.44	18.54	3.26	5.65	577
22.38	21.72	21.81	3.27	5.97	547
25.66	25.00	25.08	3.27	6.31	518
28.97	28.31	28.38	3.30	5.23	631
32.25	31.59	31.66	3.27	5.50	595
35.50	34.84	34.90	3.24	5.39	601
38.78	38.12	38.17	3.28	5.13	639
42.06	41.40	41.45	3.28	5.30	618
45.34	44.68	44.73	3.28	4.30	762
48.62	47.97	48.01	3.28	4.12	795

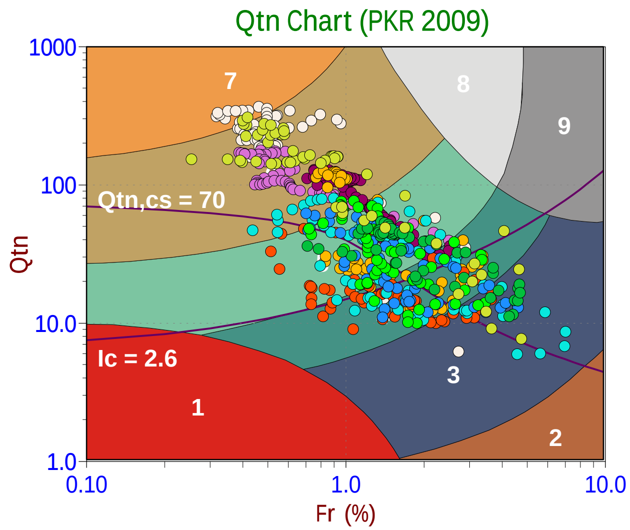
Soil Behavior Type (SBT) Scatter Plots

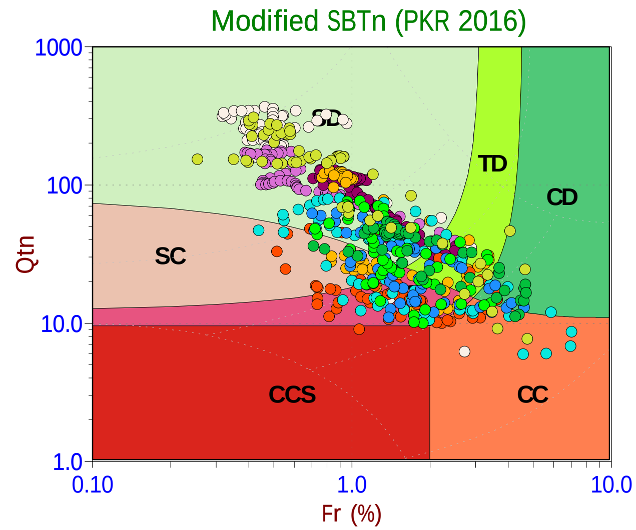
CONETEC

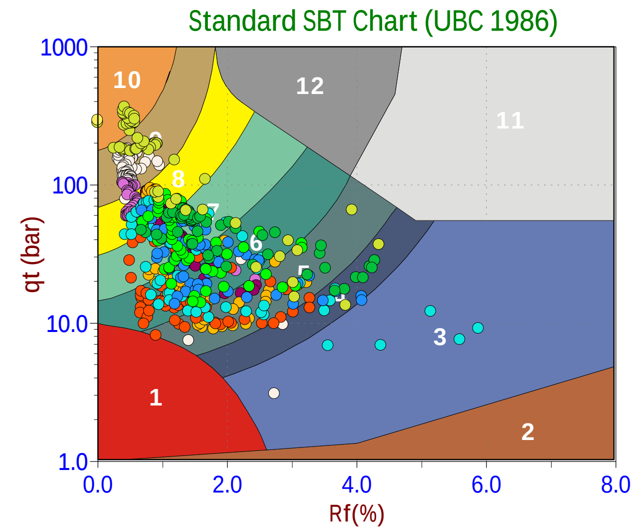
OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 08:18
Site: 149 Park Street, Northfield, NH

Sounding: SCPT21-01
Cone: 558: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
- 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)

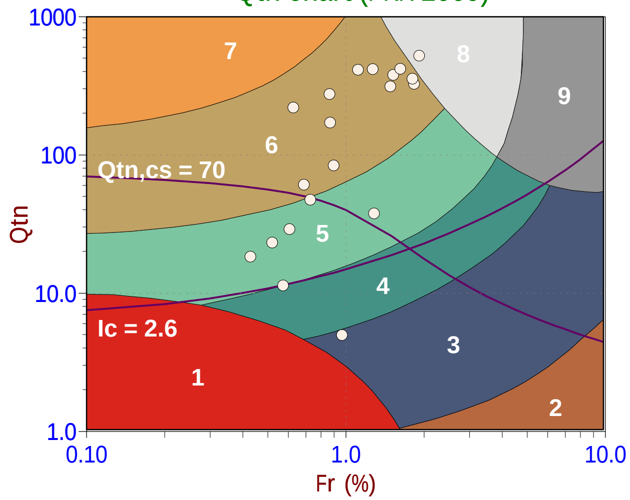
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

OTO Engineering Associates

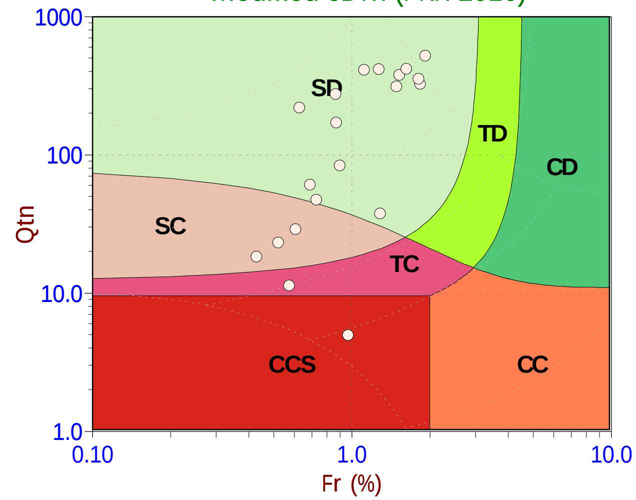
Job No: 21-53-23025
Date: 2021-09-22 09:07
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-02
Cone: 558:T1500F15U35

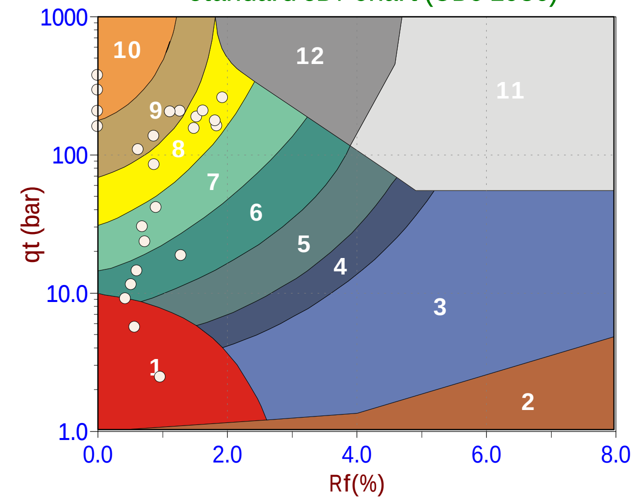
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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

- 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)

Legend

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

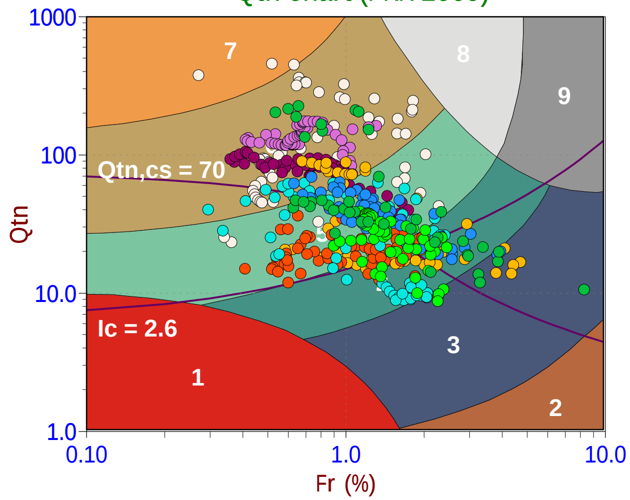
CONETEC

OTO Engineering Associates

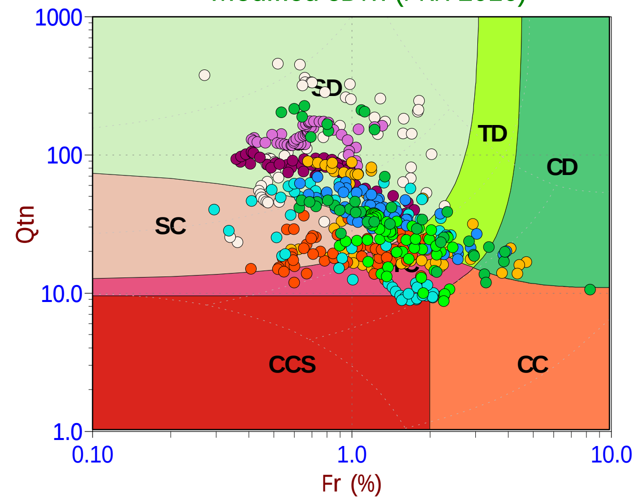
Job No: 21-53-23025
Date: 2021-09-22 09:24
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-02A
Cone: 558:T1500F15U35

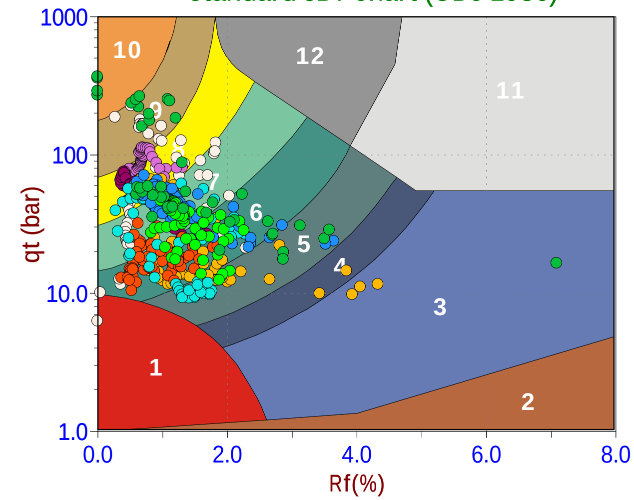
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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
- 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)

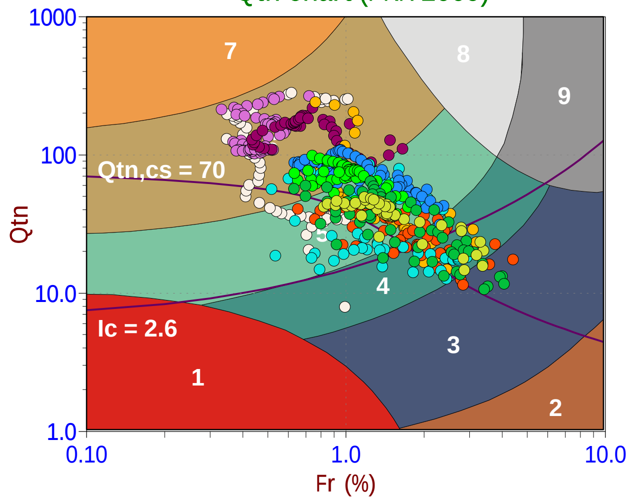
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

OTO Engineering Associates

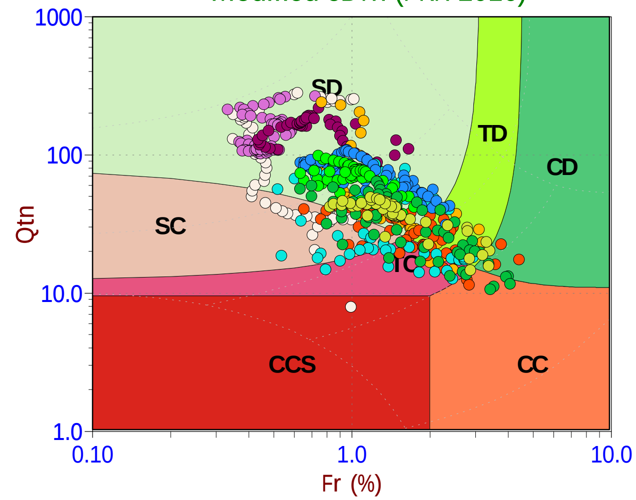
Job No: 21-53-23025
Date: 2021-09-22 10:37
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-03
Cone: 558:T1500F15U35

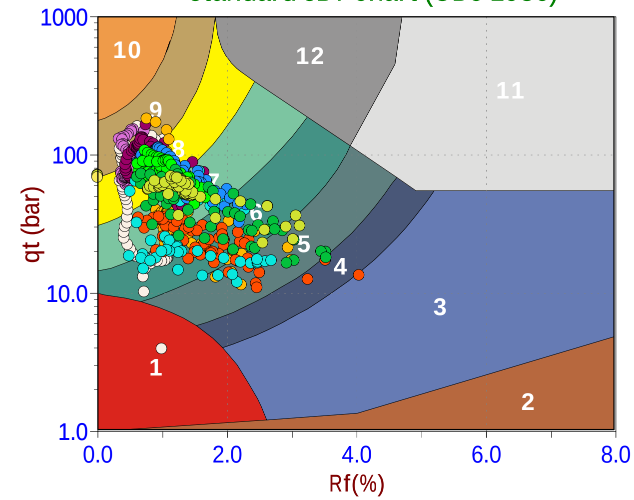
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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

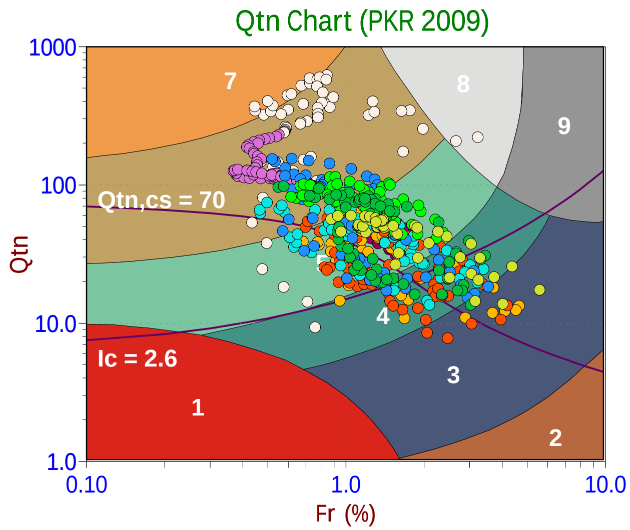
Legend

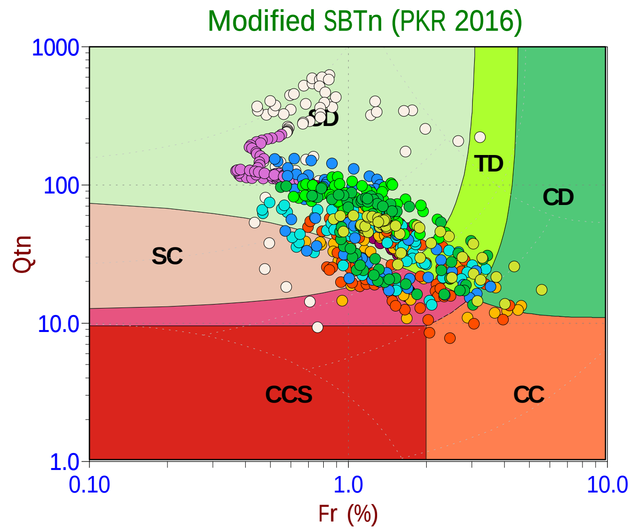
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

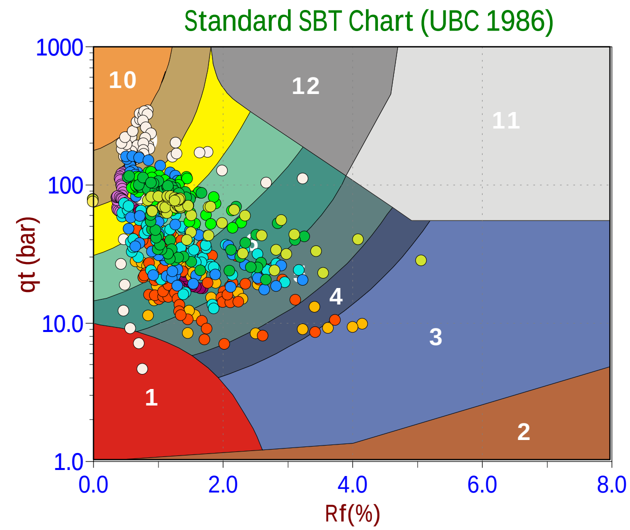
Legend

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

OTO Engineering Associates	Job No: 21-53-23025 Date: 2021-09-22 12:37 Site: 149 Park Street, Northfield, NH
Sounding: CPT21-04 Cone: 558:T1500F15U35	







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- >20.0 to 25.0 ft
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- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

Legend

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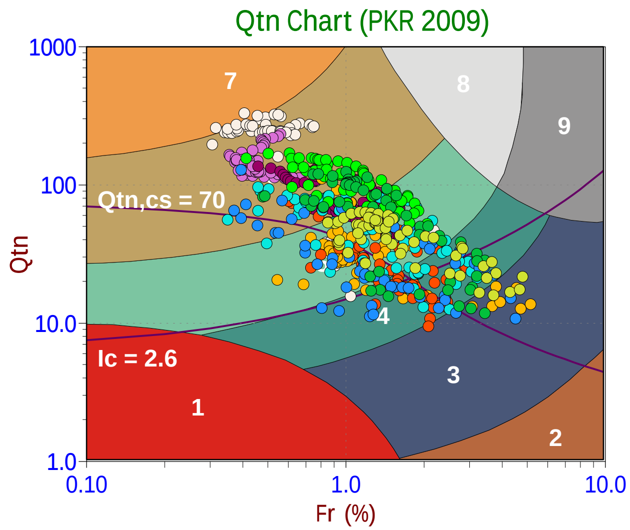
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

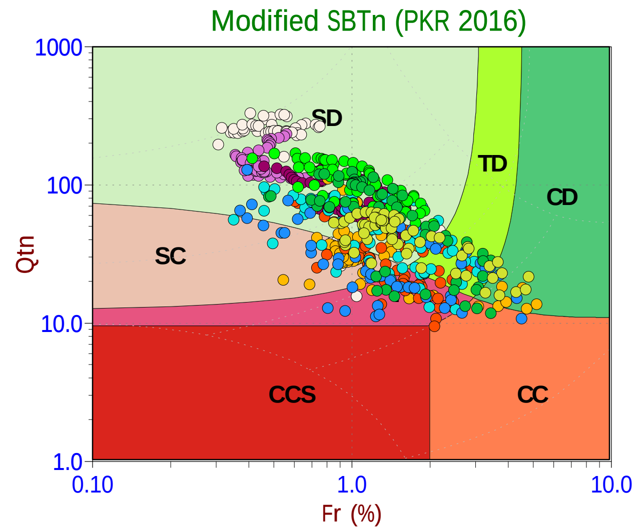
CONETEC

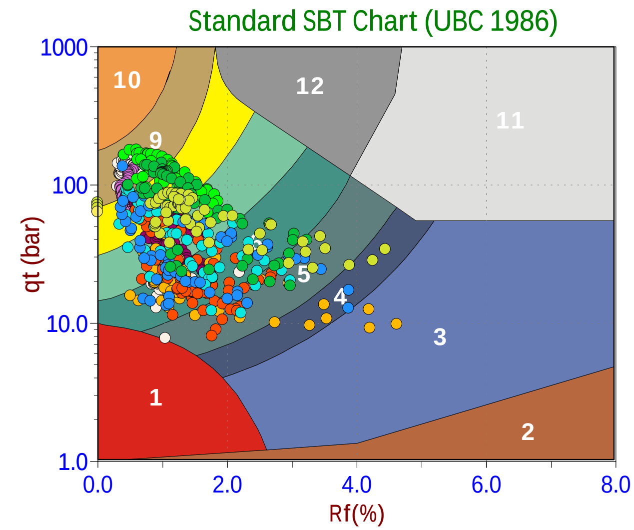
OTO Engineering Associates

Job No: 21-53-23025
Date: 2021-09-22 11:31
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-05
Cone: 558:T1500F15U35







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- >35.0 to 40.0 ft
- >40.0 to 45.0 ft
- >45.0 to 50.0 ft
- >50.0 ft

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- Silty Sand/Sand
- Sand
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- Stiff Fine Grained
- Cemented Sand

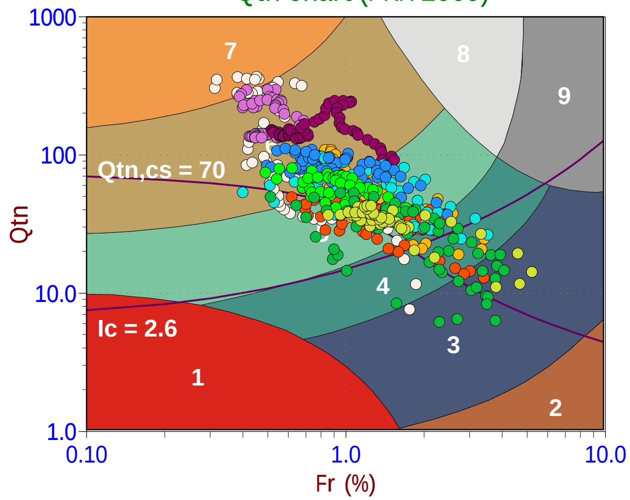
CONETEC

OTO Engineering Associates

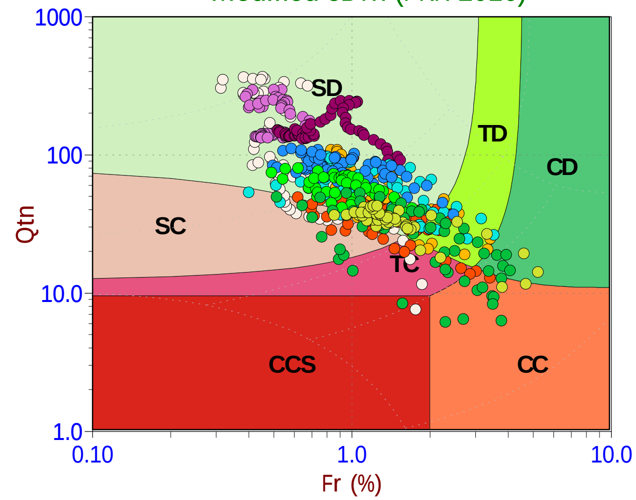
Job No: 21-53-23025
Date: 2021-09-22 09:58
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-06
Cone: 558:T1500F15U35

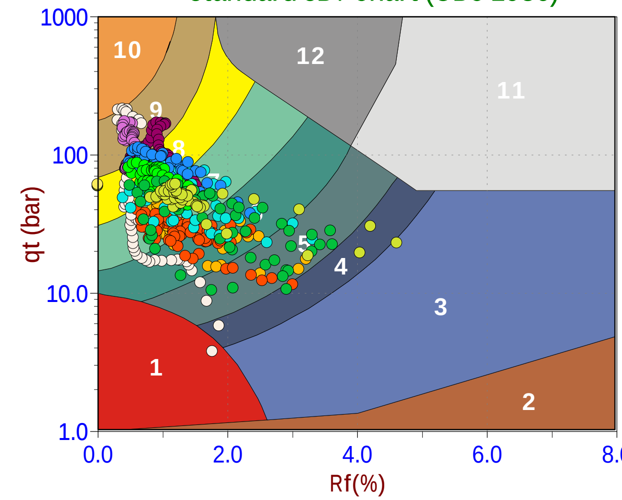
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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- >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

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Legend
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- Organic Soil
- Clay
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- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

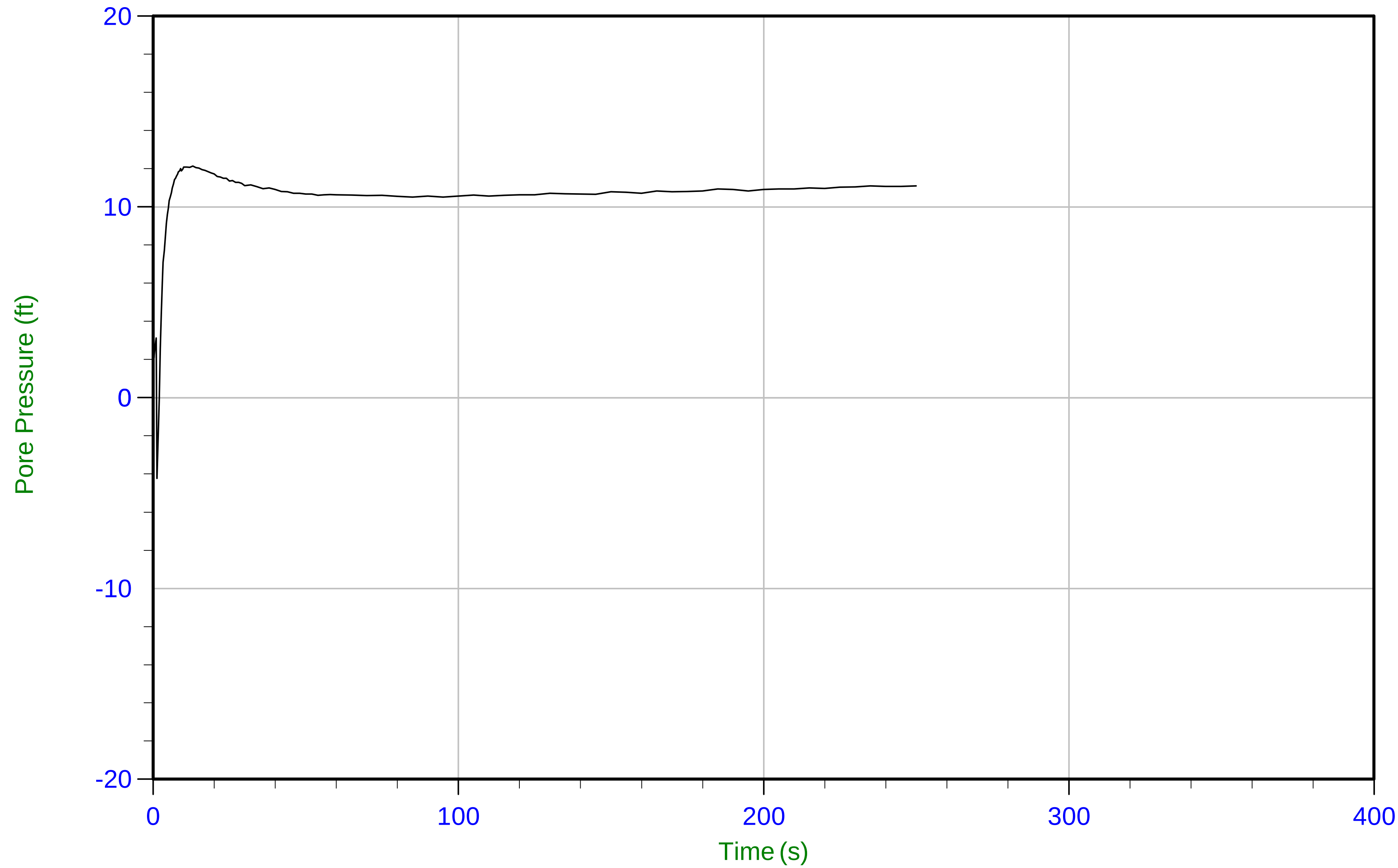
Job No: 21-53-23025
Client: OTO Engineering Associates
Project: 149 Park Street, Northfield, NH
Start Date: 22-Sep-2021
End Date: 22-Sep-2021

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)	Estimated Phreatic Surface (ft)	t_{50} [a] (s)	Assumed Rigidity Index (I_r)	c_h [b] (cm^2/min)
SCPT21-01	21-53-23025_SP01	15	250	15.83	10.9	4.9				
CPT21-02A	21-53-23025_CP02A	15	250	43.88	43.7	0.2				
CPT21-03	21-53-23025_CP03	15	555	12.30	8.3	4.0				
CPT21-03	21-53-23025_CP03	15	400	50.03	49.1	1.0				
CPT21-04	21-53-23025_CP04	15	510	50.03	48.4	1.6				
CPT21-05	21-53-23025_CP05	15	1000	32.32	28.3		4.0	127	100	5.5
CPT21-05	21-53-23025_CP05	15	300	50.03	48.4	1.6				
CPT21-06	21-53-23025_CP06	15	350	50.03	49.5	0.6				
Totals	8 dissipations		60.3 min							

a. Time is relative to where umax occurred.
b. Houlsby and Teh, 1991.

OTO Engineering Associates	Job No: 21-53-23025 Date: 09/22/2021 08:18 Site: 149 Park Street, Northfield, NH	Sounding: SCPT21-01 Cone: 558:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-53-23025_SP01.ppd2
Depth: 4.825 m / 15.830 ft
Duration: 250.0 s

u Min: -4.2 ft
u Max: 12.1 ft
u Final: 11.1 ft

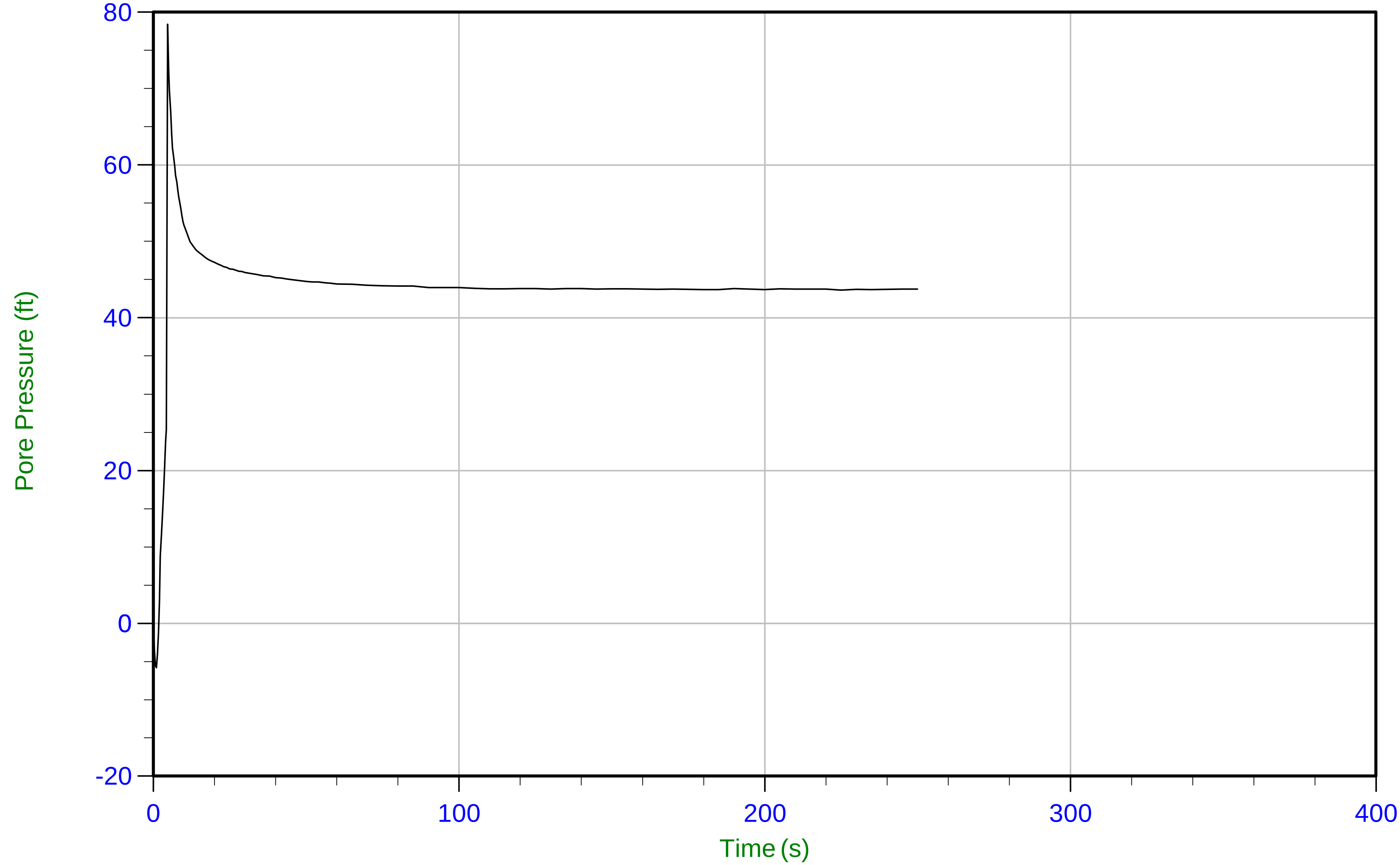
WT: 1.488 m / 4.883 ft
Ueq: 10.9 ft

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 09/22/2021 09:24
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-02A
Cone: 558:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-53-23025_CP02A.ppd2
Depth: 13.375 m / 43.881 ft
Duration: 250.0 s

u Min: -5.8 ft
u Max: 78.4 ft
u Final: 43.7 ft

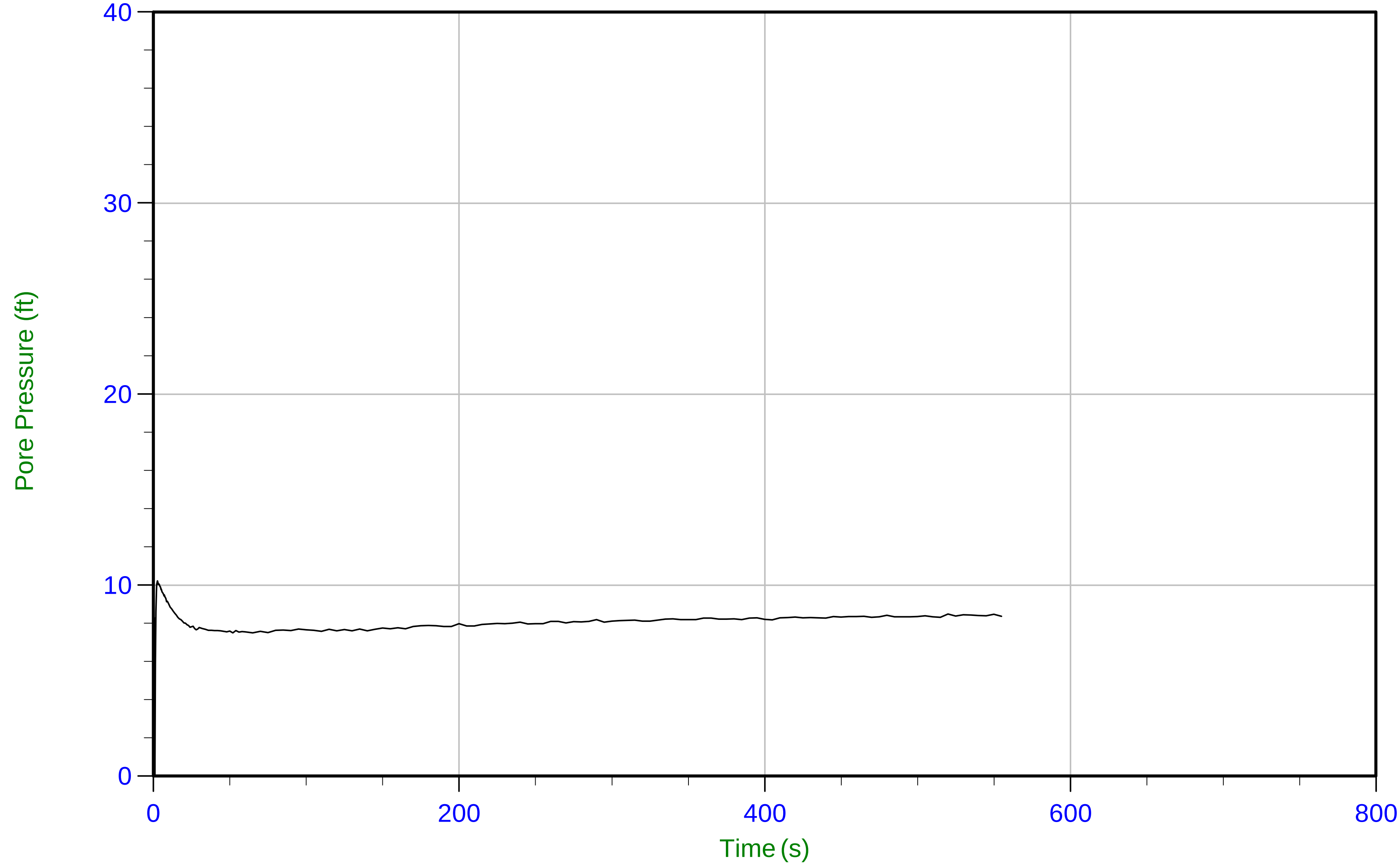
WT: 0.060 m / 0.197 ft
Ueq: 43.7 ft

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 09/22/2021 10:37
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-03
Cone: 558:T1500F15U35 Area=15 cm²



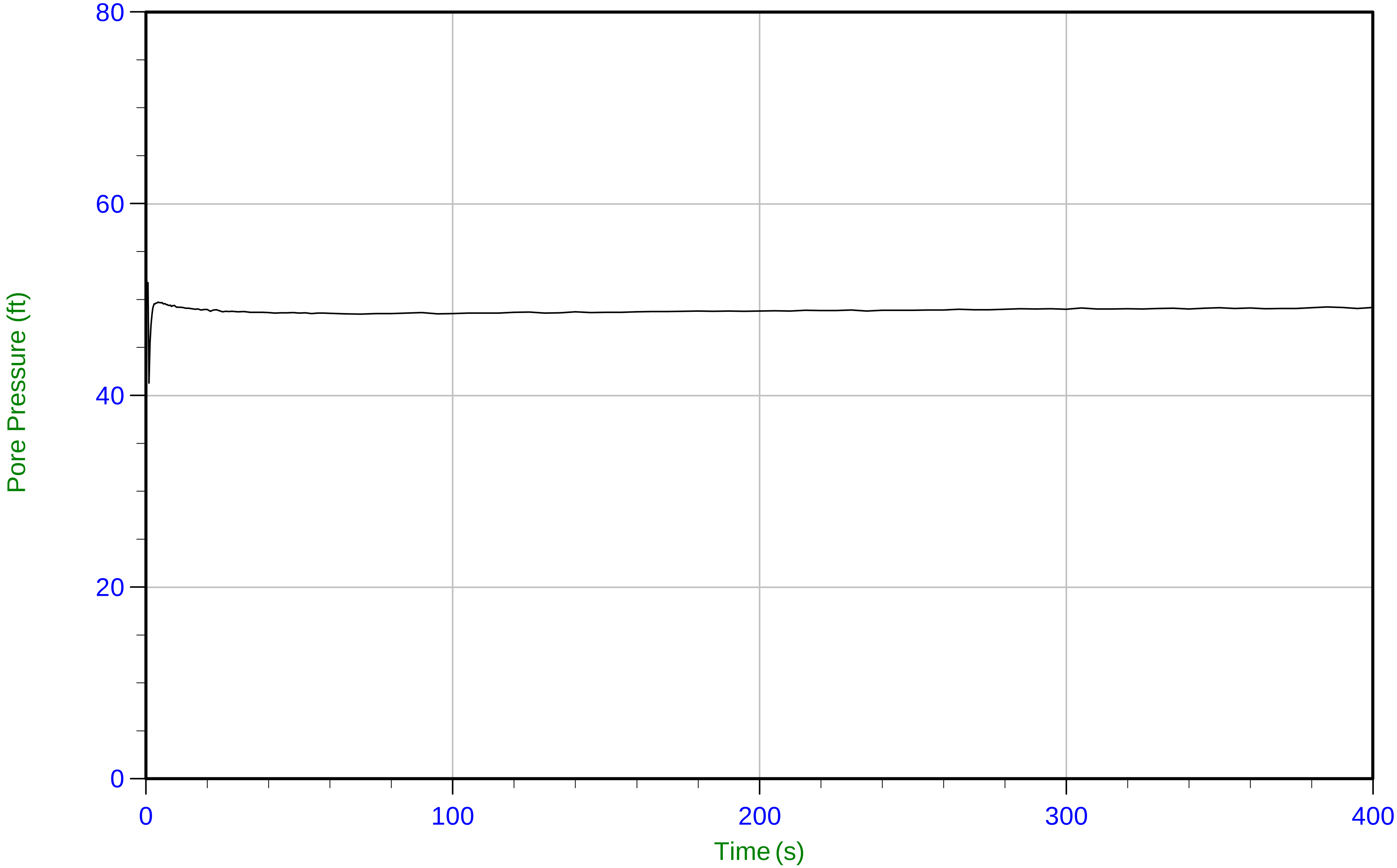
Trace Summary:

Filename: 21-53-23025_CP03.ppd2
Depth: 3.750 m / 12.303 ft
Duration: 555.0 s

u Min: -0.1 ft
u Max: 10.2 ft
u Final: 8.4 ft

WT: 1.215 m / 3.987 ft
Ueq: 8.3 ft

OTO Engineering Associates	Job No: 21-53-23025 Date: 09/22/2021 10:37 Site: 149 Park Street, Northfield, NH	Sounding: CPT21-03 Cone: 558:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-53-23025_CP03.ppd2
Depth: 15.250 m / 50.032 ft
Duration: 400.0 s

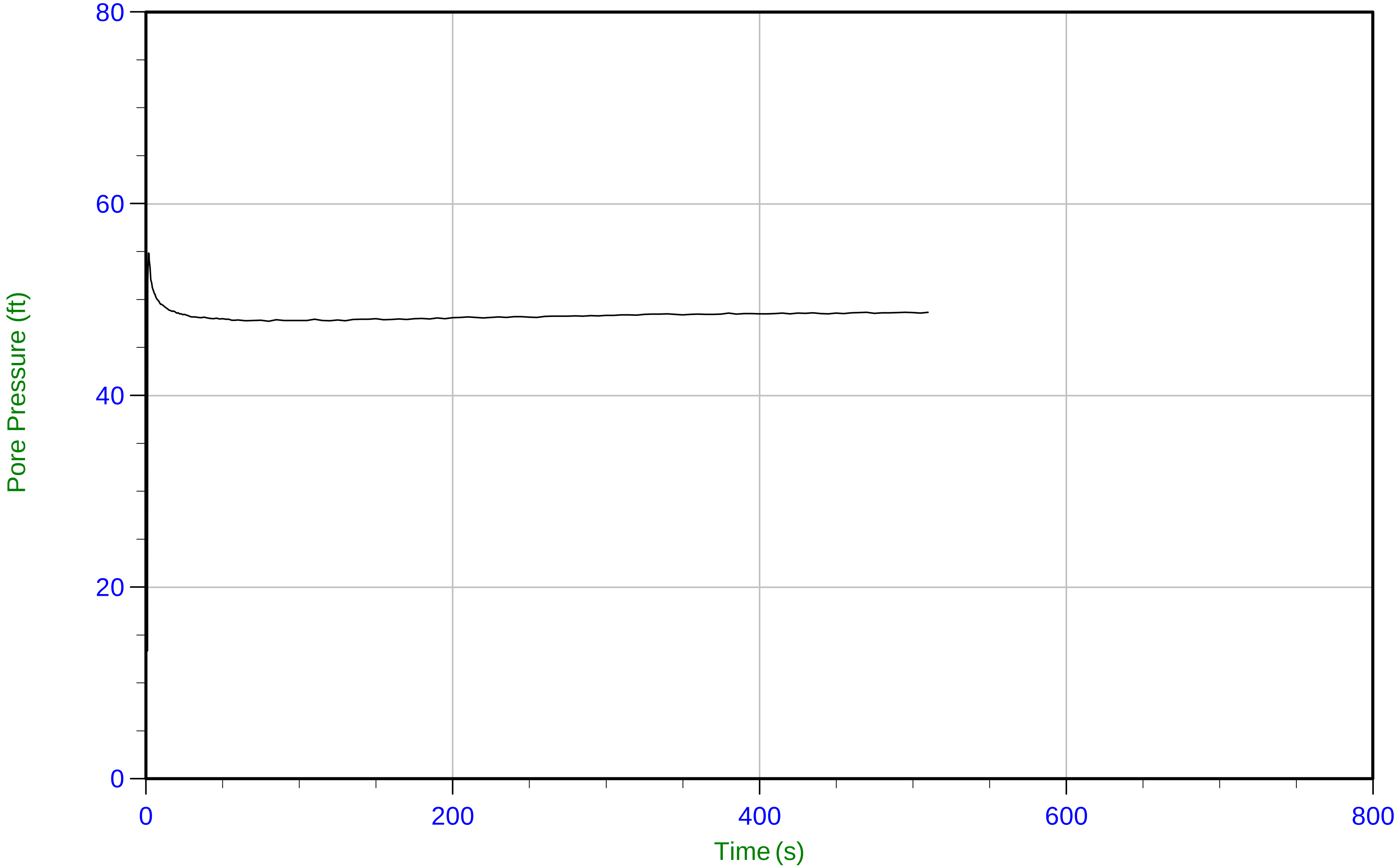
u Min: 41.3 ft
u Max: 53.3 ft
u Final: 49.2 ft

WT: 0.299 m / 0.980 ft
Ueq: 49.1 ft

OTO Engineering Associates

Job No: 21-53-23025
Date: 09/22/2021 12:37
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-04
Cone: 558:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-53-23025_CP04.ppd2
Depth: 15.250 m / 50.032 ft
Duration: 510.0 s

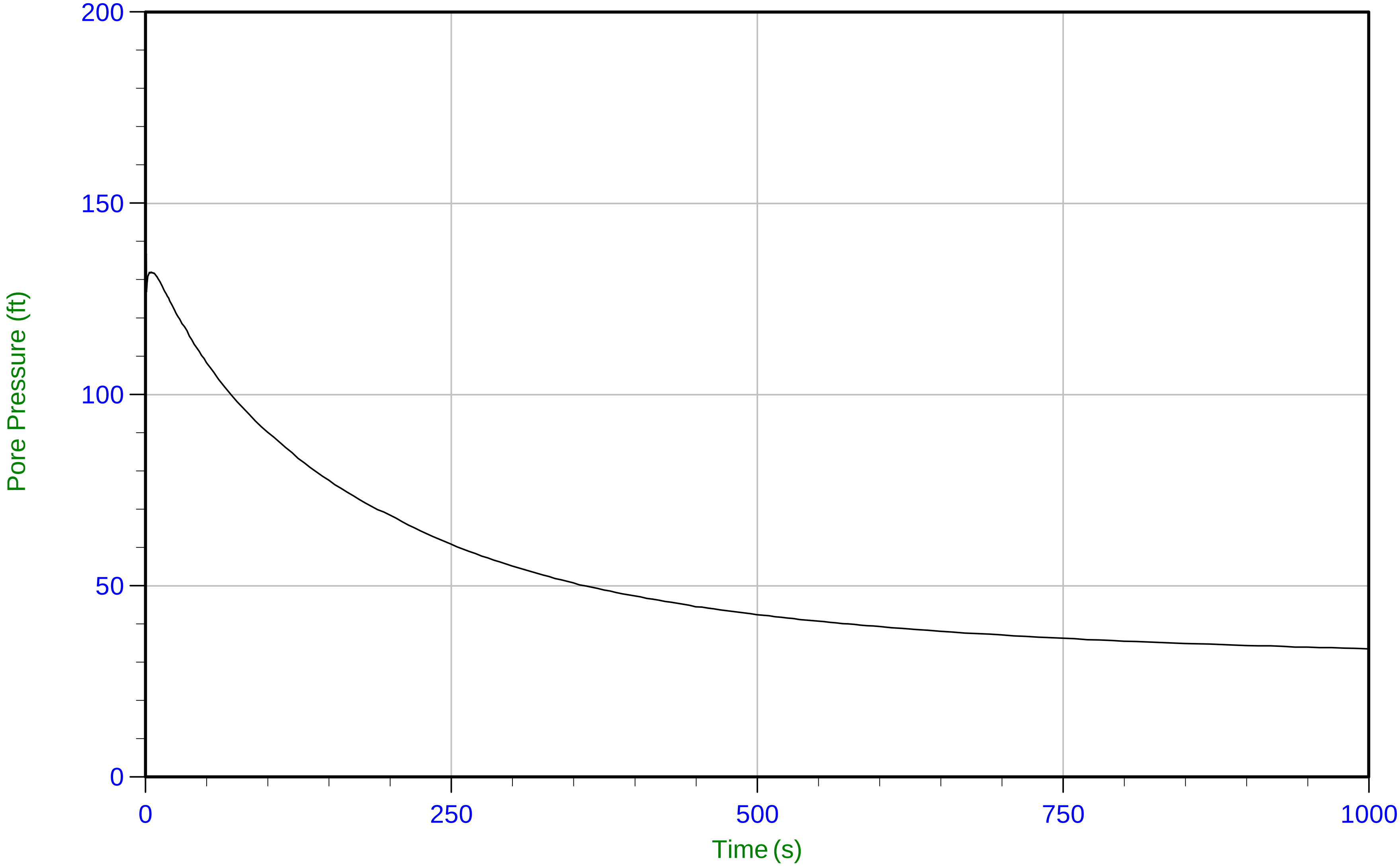
u Min: 13.3 ft
u Max: 54.9 ft
u Final: 48.7 ft

WT: 0.491 m / 1.612 ft
Ueq: 48.4 ft

OTO Engineering Associates

Job No: 21-53-23025
Date: 09/22/2021 11:31
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-05
Cone: 558:T1500F15U35 Area=15 cm²



Trace Summary:

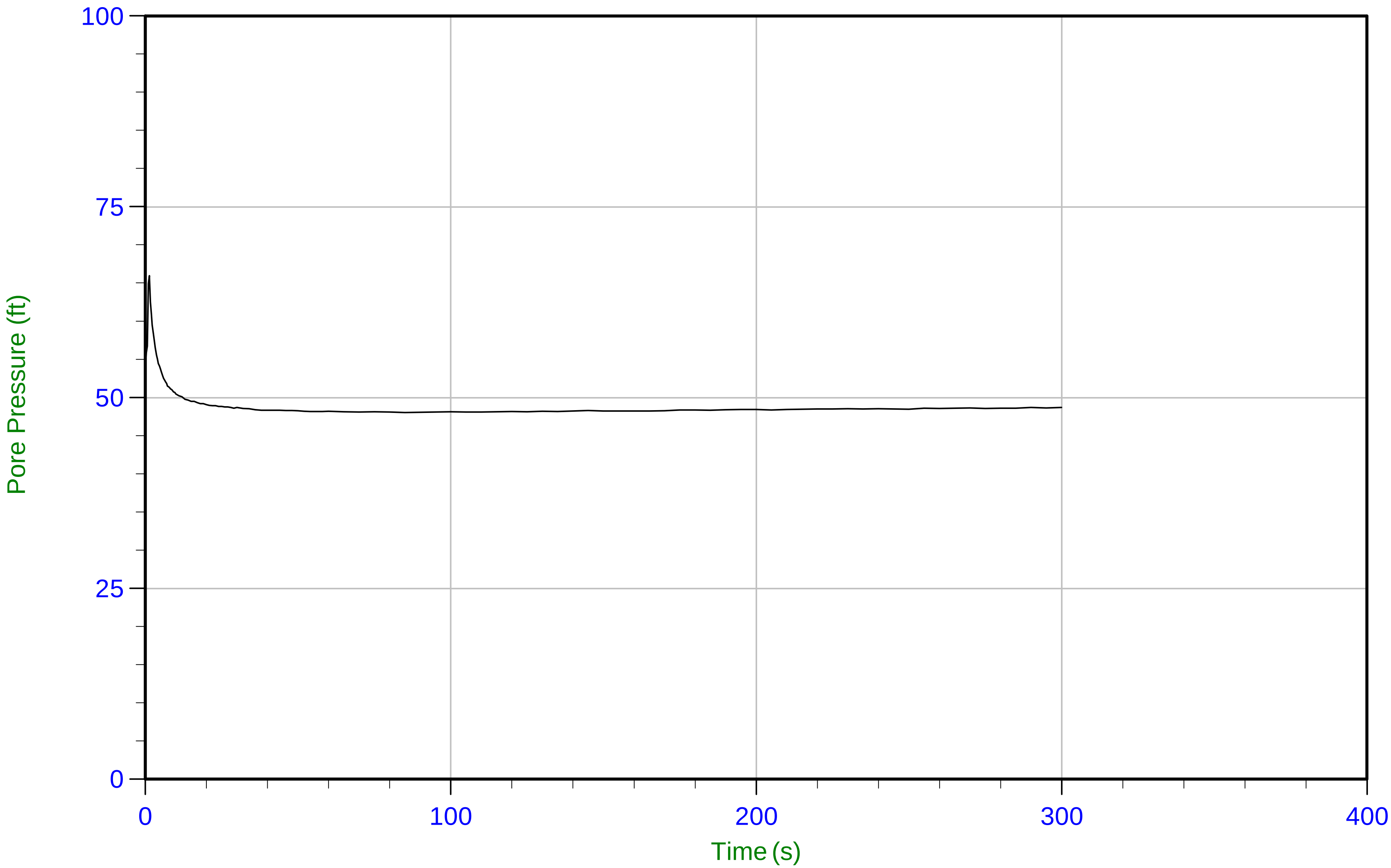
Filename: 21-53-23025_CP05.ppd2	u Min: 33.5 ft	WT: 1.219 m / 4.000 ft	T(50): 127.1 s
Depth: 9.850 m / 32.316 ft	u Max: 137.3 ft	Ueq: 28.3 ft	Ir: 100
Duration: 1000.0 s	u Final: 33.5 ft	U(50): 82.83 ft	Ch: 5.5 cm²/min

CONETEC

OTO Engineering Associates

Job No: 21-53-23025
Date: 09/22/2021 11:31
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-05
Cone: 558:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-53-23025_CP05.ppd2
Depth: 15.250 m / 50.032 ft
Duration: 300.0 s

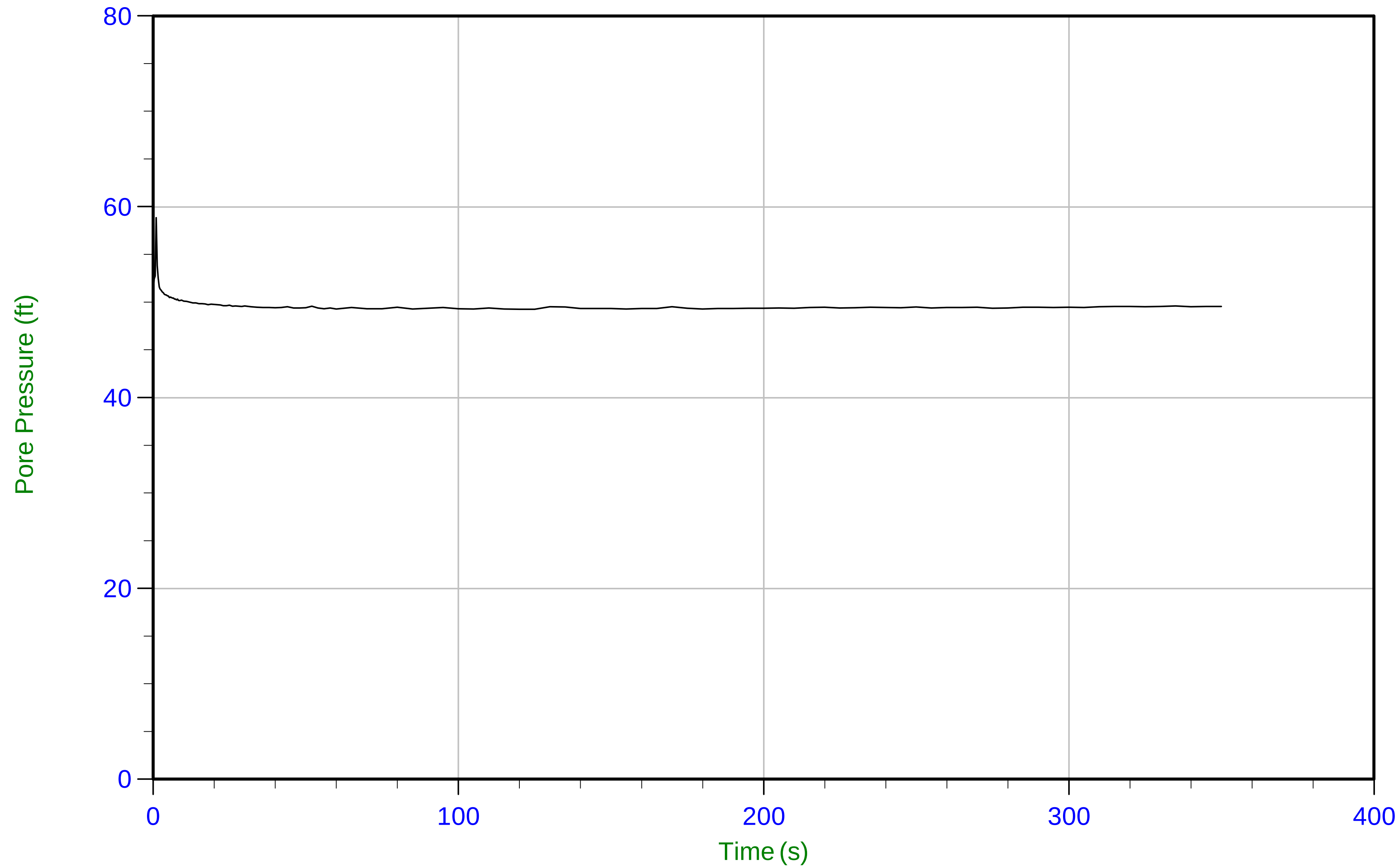
u Min: 48.1 ft
u Max: 66.0 ft
u Final: 48.7 ft

WT: 0.491 m / 1.612 ft
Ueq: 48.4 ft

OTO Engineering Associates

Job No: 21-53-23025
Date: 09/22/2021 09:58
Site: 149 Park Street, Northfield, NH

Sounding: CPT21-06
Cone: 558:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 21-53-23025_CP06.ppd2
Depth: 15.250 m / 50.032 ft
Duration: 350.0 s

u Min: 49.3 ft
u Max: 58.9 ft
u Final: 49.6 ft

WT: 0.170 m / 0.559 ft
Ueq: 49.5 ft