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Geotechnical Factual Data Report

Campbell Heights Property Assessment - Site 14



Prepared for:
BC Hydro

October 15, 2024

Prepared by:
Stantec Consulting Ltd.

Project/File:
123317336

Geotechnical Factual Data Report: Campbell Heights Property Assessment - Site 14

Revision	Date	Description
A	October 15, 2024	Draft, issued for review

DRAFT



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Table of Contents

1	Introduction.....	1
1.1	Project Understanding	1
1.2	Scope of Work	1
2	Site Description and Geology	3
2.1	Physical Setting	3
2.2	Geological Setting	3
3	Geotechnical Site Investigation	4
3.1	Site Reconnaissance and Preparation	4
3.2	Property Access and Permits	4
3.3	Field Program	5
3.3.1	Utility Locating	5
3.3.2	Drilling, Sampling, and In-Situ Testing	5
3.4	Standpipe Piezometers and Borehole Closure	7
3.5	Laboratory Testing.....	8
3.5.1	Natural Moisture Content.....	9
3.5.2	Particle Size Distribution.....	9
3.5.3	Chemical Tests of Soils	9
4	Closure	10
5	References	11
List of Tables		
Table 3-1 Borehole Summary.....		5
Table 3-2 Standpipe Piezometer Summary and Groundwater Depths		8
Table 3-3 Laboratory Testing Summary.....		9
List of Appendices		
Appendix A Statement of General Conditions		A-1
Appendix B Borehole Location Plan		B-1
Appendix C Borehole Records		C-1
Appendix D ConeTec RSCPT Report		D-1
Appendix E Laboratory Test Results		E-1



1 Introduction

BC Hydro (BCH) has engaged Stantec Consulting Ltd. (Stantec) to provide geotechnical consulting services in support of the Campbell Heights Property Assessment – Site 14 project, herein referred to as ‘the Project’. Stantec has prepared this report for BCH to summarize the factual geotechnical data for the Project.

This report should be read in conjunction with the Statement of General Conditions presented in **Appendix A**.

1.1 Project Understanding

Based on BCH Request for Proposal (RFP) 17635, we understand that BCH is evaluating candidate properties for the future development of a new 200 MVA substation to service the South Surrey and Langley areas. The candidate properties are within the Campbell Heights neighbourhood of the Township of Langley, British Columbia.

This report pertains to Site 14, which is one of BCH’s candidate properties and is located between 196 Street and 197A Street, and between 30 Avenue and 31A Avenue. We understand that Site 14 was selected as a candidate property for further assessment based on a high-level review undertaken by BCH. BCH requires an assessment of subsurface conditions, seismic conditions, foundation design considerations, and a qualitative geotechnical risk assessment to progress the evaluation of Site 14 prior to property acquisition. Details about the substation layout, final grading, proposed infrastructure, and loading have not been provided to Stantec and are currently unknown.

Per BCH RFP 17635, the key objectives of the property assessment are as follows:

- Identification of the potential for adverse impacts to the property due to a seismic event, including unacceptable post-seismic displacements and/or failure resulting from any of the seismically induced mechanisms such as liquefaction, strain softening and or unacceptable amplification of seismic loading, etc.
- Identification of potential geohazards at the property including flooding and landslides.
- Identification of site preparation (i.e., cuts and fills) and ground improvement requirements in order for the site to be used as a substation.

The results of the geotechnical site assessment are presented separately.

1.2 Scope of Work

The work described in this report was carried out in accordance with our proposal letter (“Campbell Heights Property Assessment – Site 14”, dated July 18, 2024), email correspondence from BCH with suggested geotechnical site investigation changes on July 22, 2024, and the terms of the BCH RFP



Geotechnical Factual Data Report: Campbell Heights Property Assessment - Site 14

Introduction

17635 Geotechnical Engineering and Geohazard Services Master Services Agreement. Stantec's scope of work for this Geotechnical Factual Data Report included the following:

- Review of existing Project information and available geotechnical and geological information for the Project site.
- Site reconnaissance with BCH, Stantec, and Stantec's drilling subcontractor.
- Obtaining permits to unload and load geotechnical site investigation equipment within the road right-of-way.
- A geotechnical site investigation consisting of:
 - Two (2) auger boreholes to a target depth of 15 m with Standard Penetration Testing (SPT) carried out at 1.5 m intervals.
 - Seismic Cone Penetration Tests with concurrent soil Resistivity measurements (RSCPT) near both auger boreholes and advanced to a maximum depth of 30 m or until practical equipment refusal.
 - Installing standpipe piezometers within the auger boreholes to monitor the groundwater levels at the Project site.
- Geotechnical and chemical laboratory testing of select samples collected during the geotechnical site investigation.
- Preparation of this report.



2 Site Description and Geology

2.1 Physical Setting

The Project site, identified as 'Site 14', is situated between 196 Street and 197A Street, and between 30 Avenue and 31A Avenue in the Township of Langley, BC, within the Fraser Lowland physiographic sub-region of the greater Georgia Depression physiographic region (Holland, 1976).

The Township of Langley's interactive mapping tool, GeoSource (Township of Langley, 2024), shows that the Project site is approximately 13 acres in area, within Lots 9 and 11 of Plan NWP 1168, and is rectangular in shape. The site is currently undeveloped and vegetated with second growth forest. Surrounding land use generally consists of residential and agricultural properties.

The existing ground surface elevation at Site 14 is between approximately El. 47 m and 49 m geodetic at the west and east sides of the site, respectively.

2.2 Geological Setting

Based on the published Geological Survey of Canada Map 1484A, Surficial Geology of New Westminster (Armstrong and Hickock, 1980), the surficial soils at the site consist of geological unit Se (Pleistocene – Sumas Drift sediments): outwash, ice-contact, and deltaic deposits consisting of raised proglacial deltaic gravel and sand up to 40 m thick.



3 Geotechnical Site Investigation

Stantec carried out a geotechnical site investigation field program consisting of utility locating, drilling of two (2) auger boreholes (BH24-01 and BH24-02) with in-situ testing and standpipe piezometer installation, and two RSCPT soundings located near the auger borehole locations (RSCPT24-01 and RSCPT24-02). A borehole location plan that shows the approximate locations of the auger boreholes and RSCPT soundings is provided in **Appendix B**.

3.1 Site Reconnaissance and Preparation

BCH, Stantec, and our drilling subcontractor carried out a site reconnaissance on August 21, 2024, to select and confirm access to the borehole locations, and identify any potential geohazards. The borehole locations were selected to minimize tree clearing requirements and to avoid the need for fence removals, waterway crossings, or the construction of new access trails. During the site reconnaissance, the parties also identified trees that could obstruct equipment access to borehole locations or that could be potentially hazardous to the geotechnical site investigation personnel.

Stantec retained Central Valley Tree and Arborist Services Ltd. (Central Valley) of Abbotsford, BC to remove potentially hazardous trees and branches that were obstructing equipment access routes. BCH obtained approval from the Township of Langley prior to the tree and branch removal work. Two standing dead trees were removed near BH24-01, one live tree was removed near BH24-02, and various tree limbs were trimmed along the access trail to BH24-02. Central Valley carried out the tree and branch clearing work on September 6, 2024.

3.2 Property Access and Permits

BCH negotiated with the Township of Langley to obtain access to the site and receive permission to carry out the geotechnical site investigation and tree clearing.

Boreholes BH24-01 and BH24-02 were accessed via existing trails that originated near 196 Street at 31A Avenue and near 197A Street at 30 Avenue, respectively. For borehole BH24-01, the geotechnical site investigation equipment was unloaded and loaded in a clearing beyond the south end of 196 Street.

At the time of the reconnaissance, the trail entrance near 197A Street at 30 Avenue was blocked by concrete barriers, which would impede access to BH24-02. For this reason, Stantec obtained a Highway Use Permit (HUP HG002218) from the Township of Langley on September 6, 2024, in order to unload and load geotechnical site investigation equipment in the 30 Avenue right-of-way.

A rubber-tracked John Deere 323D Skid Steer was used to temporarily re-locate the concrete barriers positioned at the trail access point near the intersection of 30 Avenue and 197A Street. The concrete barriers were returned to their original position after completion of the geotechnical site investigation.



3.3 Field Program

3.3.1 Utility Locating

In advance of the geotechnical site investigation, Stantec submitted requests to BC One Call, Dig Shaw, and Metro Vancouver to obtain information on existing utilities in the vicinity of the proposed borehole locations. Prior to drilling, Stantec retained the services of a utility locating subconsultant, Quadra Utility Locating Ltd. (Quadra) of Port Coquitlam, BC, to review available information on existing utilities and to carry out a survey of existing subsurface utilities at the site prior to finalizing the borehole locations. Quadra carried out the on-site survey of existing utilities on September 6, 2024, using electromagnetic (EM) and ground penetrating radar (GPR) utility locating methodologies.

3.3.2 Drilling, Sampling, and In-Situ Testing

The auger boreholes and RSCPT soundings were completed between September 9 and 11, 2024. A summary of the borehole and RSCPT locations, existing ground surface elevations, and termination depths is provided in **Table 3-1**.

Table 3-1 Borehole Summary

Borehole ID	Dates	Northing ¹ (m)	Easting ¹ (m)	Approximate Ground Surface Elevation ² (m)	Termination Depths (m)
BH24-01	September 9, 2024	5433903	523338	48	16.6
BH24-02	September 11, 2024	5433791	523505	49	15.1
RSCPT24-01	September 10, 2024	5433903	523336	48	1.4
RSCPT24-01A	September 10, 2024	5433903	523337	48	6.2, 14.0
RSCPT24-02	September 11, 2024	5433794	523504	49	1.9

Notes:

1. Borehole and RSCPT coordinates collected using handheld GPS device and are accurate to approximately +/- 5 m.
2. Ground surface elevations estimated using Township of Langley ArcGIS 1 m contour map (2018) open data and are approximate.

The geotechnical site investigation was carried out under the full-time supervision of a Stantec geotechnical engineer who located the boreholes, observed the drilling and testing operations, classified the soils encountered, prepared field records, and collected soil samples for laboratory testing.



3.3.2.1 Auger Boreholes

Geotechnical auger borehole drilling was carried out by Mud Bay Drilling Ltd. (Mud Bay), a subsidiary of ConeTec Investigations Ltd. (ConeTec). The drilling was completed using a truck-mounted GT8 6x6 drill rig.

Borehole drilling commenced using the solid-stem auger drilling methodology in general accordance with American Society for Testing and Materials (ASTM) standard D1452. However, this methodology was abandoned at approximately 2.3 m and 0.8 m depth below the ground surface in boreholes BH24-01 and BH24-02, respectively, due to borehole collapse. Mud Bay subsequently switched to the hollow-stem auger drilling methodology to advance the boreholes to the target depth.

Hollow-stem auger drilling was carried out in general accordance with ASTM standard D6151, using approximately 110 mm inner diameter (ID) hollow-stem auger flights. Due to the presence of sandy and gravelly soils, Mud Bay chose to advance the hollow-stem augers without utilizing a plug at the bottom of the augers. Instead, 44 mm outer diameter (OD) drill rods (AWJ size) were used during drilling to prevent soil from rising through the inside of the hollow-stem augers.

Mud Bay initially attempted hollow-stem auger drilling without using any water or drilling fluid within the auger-casing. Approximately 300 mm to 450 mm of heave occurred inside the hollow-stem augers at 4.3 m and 4.7 m depth below the ground surface in BH24-01 and BH24-02, respectively. Subsequently, Mud Bay introduced water to the inside of the hollow-stem auger-casing to maintain positive head at the bottom of the hole to mitigate further heave.

Standard Penetration Tests (SPTs) were performed in both boreholes, either after removing the solid-stem augers from the borehole or through the inside of the hollow-stem auger-casing. The SPTs were carried out at approximately 1.5 m depth intervals in general accordance with ASTM D1586, using a 51 mm OD, un-lined split spoon sampler. Upon reaching the desired test interval, the split spoon sampler was connected to 44 mm OD (AWJ size) drilling rods, lowered to the base of the borehole, and driven into the ground using a 63.5 kg automatic safety hammer, falling from a height of 760 mm. Blow counts were recorded for penetration intervals of 150 mm during the testing, up to a maximum of four consecutive intervals (i.e., maximum 600 mm test). The SPT N-values are the cumulative blows for the second and third 150 mm penetration (for a total 300 mm of penetration) and are reported on the Borehole Records.

Soil and groundwater conditions, results of the laboratory classification and index testing, borehole backfilling, and standpipe installation details are shown on the Borehole Records in **Appendix C**. Soil descriptions in the Borehole Records are in general accordance with ASTM D2487 and D2488 for the Unified Soil Classification System (USCS).

3.3.2.2 Resistivity Seismic Cone Penetration Testing (RSCPT)

RSCPT soundings were carried out as per BCH RFP 17635 and email correspondence from BCH. The RSCPT soundings were located within approximately 3 m (horizontal distance) of the boreholes.



The RSCPTs were carried out in accordance with ASTM D5778 and ASTM D7400/D7400-19 using a 15 cm² stainless steel piezocone, with additional shear wave velocity and soil resistivity measuring equipment, attached to 36 mm OD rods. The piezocone was advanced into the ground at a rate of approximately 2 cm/s using a 14-ton drill rig head cylinder attached to the truck mounted GT8 6x6 drill rig. All RSCPT soundings were terminated by ConeTec short of the target depth due to excessive rod flex caused by the presence of coarse-grained granular soils.

At the location of RSCPT24-01, the soils within approximately 0.9 m of the ground surface were drilled out prior to initiating the RSCPT sounding, which was subsequently terminated due to excessive rod flex at approximately 1.4 m depth below ground surface.

The drill rig was repositioned approximately 1 m from the initial test location, and the sounding was re-attempted as RSCPT24-01A. At this new location, ConeTec drilled out the soils within approximately 5.9 m of the existing ground surface and installed casing to provide support to the RSCPT rods. The RSCPT sounding was advanced from approximately 5.9 m to 6.2 m depth, at which point ConeTec terminated the sounding due to very high cone tip resistance. Subsequently, ConeTec drilled out the coarse-grained granular soils and installed additional support casing from approximately 6.2 m to 13.5 m depth below the ground surface. The RSCPT24-01 sounding resumed below 13.5 m depth but was terminated due to excessive rod flex at approximately 14.0 m depth. Further advancement of the RSCPT24-01 sounding was not attempted due to the soil conditions encountered.

Sounding RSCPT24-02 was carried out near BH24-02. The sounding was advanced to approximately 1.9 m depth, at which point excessive rod flex occurred. Further advancement of this RSCPT sounding was not attempted due to the similarity in soil conditions and resistance to penetration encountered at BH24-01.

Pore pressure dissipation tests were conducted in all soundings. Shear wave velocity measurements were taken at select depths in soundings RSCPT24-01 and -02.

The RSCPT data is provided along with additional details in ConeTec's report in **Appendix D**. It should be noted that the RSCPT coordinates collected by ConeTec (using a consumer grade GPS device in datum WGS84/UTM 10 North), provided in ConeTec's report, do not agree with the coordinates collected by Stantec (as presented in **Table 3-1**) and have not been verified by Stantec.

3.4 Standpipe Piezometers and Borehole Closure

Standpipe piezometers were installed after the completion of drilling in boreholes BH24-01 and BH24-02. The standpipes consisted of 50 mm diameter PVC Schedule 40 pipe with a slotted section (screen) at the bottom. The screened length was 4.5 m and 3.0 m in BH24-01 and BH24-02, respectively.

Soil sloughing was observed in both boreholes after removing the hollow-stem auger flights. BH24-01 sloughed below approximately 9.4 m depth, and BH24-02 sloughed below approximately 7.9 m depth. Coated bentonite pellets were used to backfill the boreholes over a length of approximately 0.3 m between the sloughed soils and the bottom of the standpipe piezometers. Filter sand was used to backfill



the borehole annulus and extended approximately 0.3 m above the top of the screen. Bentonite pellets were used to backfill the boreholes above the filter sand.

Flush-mount protective well covers were installed at the ground surface and sealed with concrete for both standpipe piezometers.

Standpipe piezometer installation details are summarize in **Table 3-2** and presented on the borehole records in **Appendix C**. A summary of the groundwater depth readings is also provided in **Table 3-2**.

Table 3-2 Standpipe Piezometer Summary and Groundwater Depths

Borehole ID	Standpipe Installation Date	Ground Surface Elevation ¹ (m)	Screen Depth Interval (m)	Groundwater Depth (m)	
				September 9 / 11, 2024 ²	October 4, 2024
BH24-01	September 9, 2024	48	4.6 – 9.1	4.1	4.2
BH24-02	September 11, 2024	49	4.6 – 7.6	4.6	5.0

Notes:

1. Ground surface elevations estimated using 1 m contour map (2018) open data from GeoSource (Township of Langley, 2024) and are approximate.
2. Initial water level measurements taken following standpipe piezometer installation (i.e., depth measurements taken the day of standpipe installation).

The boreholes were backfilled in accordance with the *BC Groundwater Protection Regulation*.

3.5 Laboratory Testing

The objective of the Stantec laboratory testing program was to aid in the classification of the representative, disturbed soil samples and to measure pertinent soil parameters. The tests included natural moisture content and particle size distribution tests. In general, geotechnical laboratory testing was performed in accordance with applicable ASTM test procedures and/or laboratory equipment manufacturer's recommended test procedures.

Chemical tests were carried out on select soil samples at the Bureau Veritas laboratory in Burnaby, BC.

A summary of the laboratory testing is included in **Table 3-3**.



Table 3-3 Laboratory Testing Summary

Test	Test Procedure	Number of Tests Performed
Moisture Content	ASTM D2216	2
Particle Size Distribution	ASTM D6913	7
Soluble pH	BCMOE BCLM Mar2005 m	6
Soluble Conductivity	SM 23 2510 B m	6
Soluble Chloride	SM 23 4500-Cl- E m	6
Soluble Sulphate	SM 23 4500-SO42- E m	6

3.5.1 Natural Moisture Content

Natural Moisture Content tests were performed on select samples, with emphasis on silt or silty samples, in accordance with ASTM D2216. The results of the moisture content tests are presented on the Borehole Records in **Appendix C**.

3.5.2 Particle Size Distribution

Particle size distribution tests were completed in accordance with ASTM D6913 using select soil samples. The results of the particle size distribution tests are presented on the Borehole Records in **Appendix C** and on the test reports in **Appendix E**.

3.5.3 Chemical Tests of Soils

Chemical tests including pH, electrical conductivity, chloride, and sulphate content were carried out on six soil samples from the boreholes. The chemical test results are presented in **Appendix E**.



4 Closure

This report was prepared for the exclusive use of BC Hydro for specific application to the Campbell Heights Property Assessment – Site 14 project. Use of this report is subject to the Statement of General Conditions included in **Appendix A**. It is the responsibility of BC Hydro, who is identified as “the Client” within the Statement of General Conditions, to review the conditions and notify Stantec should any of them not be satisfied.

We trust that this report meets your present requirements. If you have any questions or require additional information, please contact the undersigned.

Regards,

STANTEC CONSULTING LTD.

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5 References

1. Holland, S.S. (1976). *Landforms of British Columbia, A Physiographic Outline*. Bulletin 48. The Government of the Province of British Columbia
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Appendix A Statement of General Conditions

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STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This professional work product ("hereinafter referred to as the Report") has been prepared for the sole benefit of the Client in accordance with Stantec's contract with the Client. While the Report may be provided by the Client to applicable authorities having jurisdiction and to other third parties in connection with the project, Stantec disclaims any legal duty based upon warranty, reliance, or any other theory to any third party, and will not be liable to such third party for any damages or losses of any kind that may result.

BASIS OF THIS REPORT: This Report relates solely to the site-specific project for which Stantec was retained and the stated purpose for which the Report was prepared. The information, opinions, conclusions and/or recommendations made in this Report are in accordance with Stantec's present understanding of the site-specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time the scope of work was conducted and do not take into account any subsequent changes. If the proposed site-specific project differs or is modified from what is described in this Report or if the site conditions are altered, this Report is no longer valid unless Stantec is requested by the Client to review and revise the Report to reflect the differing or modified project specifics and/or the altered site conditions. This Report is not to be used or relied on for any variation or extension of the project, or for any other project or purpose or site, and any unauthorized use or reliance is at the recipient's own risk.

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PROVIDED INFORMATION: Stantec has assumed all information received from the Client and third parties in the preparation of this Report to be correct. While Stantec has exercised a customary level of judgment or due diligence in the use of such information, Stantec assumes no responsibility for the consequences of any error or omission contained therein.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this Report are based on site conditions encountered by Stantec at the time of the scope of work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behaviour. Extrapolation of in-situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this Report or encountered at the test and/or sample locations, Stantec must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the Report conclusions or recommendations are required. Stantec will not be responsible to any party for damages incurred as a result of failing to notify Stantec that differing site or subsurface conditions are present upon becoming aware of such conditions.

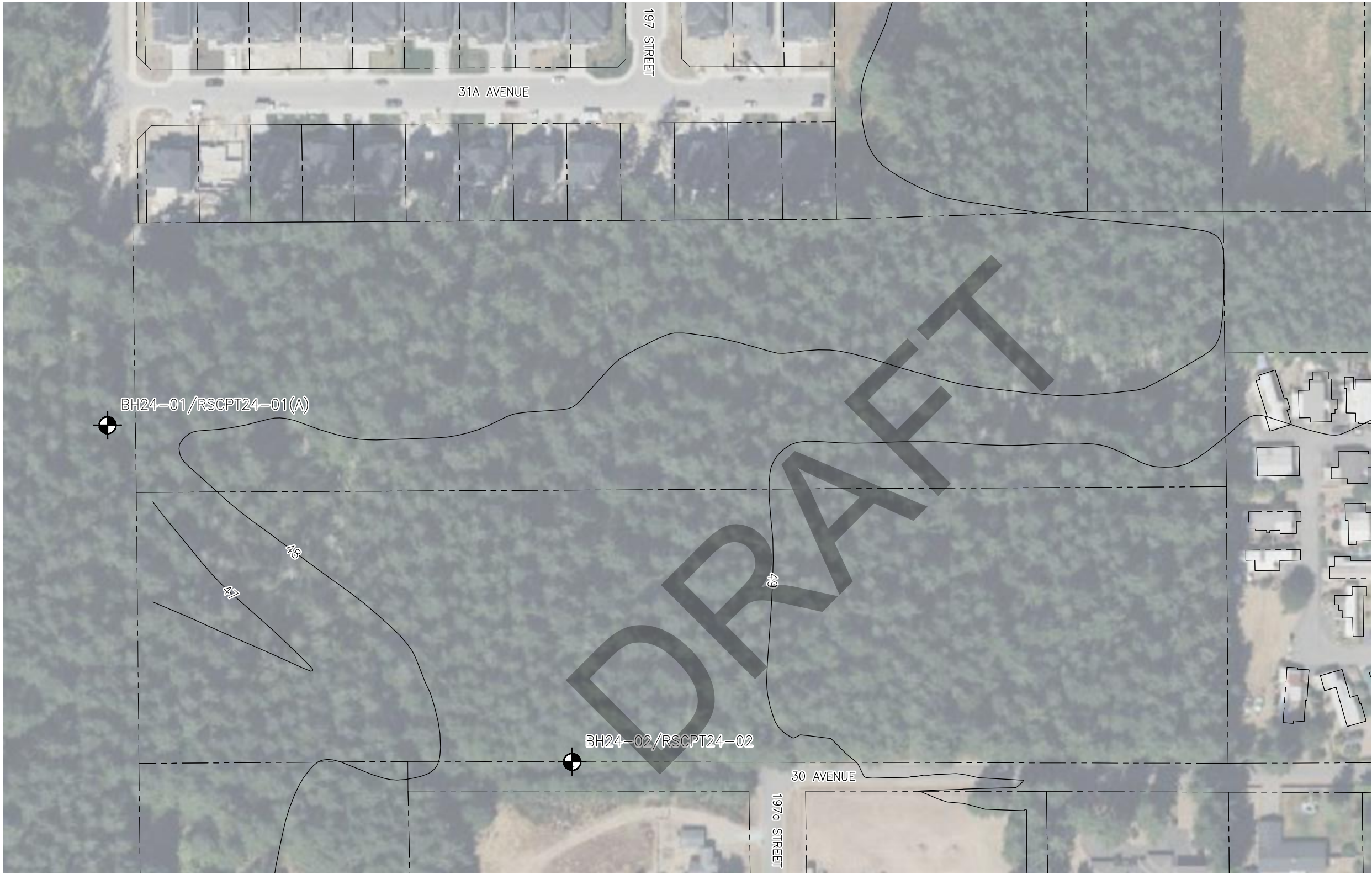
PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec geotechnical engineers, sufficiently ahead of initiating the next project stage (e.g., property acquisition, tender, construction, etc.), to confirm that this Report completely addresses the elaborated project specifics and that the contents of this Report have been properly interpreted. Specialty quality assurance services (e.g., field observations and testing) during construction are a necessary part of the evaluation of subsurface conditions and site work. Site work relating to the recommendations included in this Report should only be carried out in the presence of a qualified geotechnical engineer; Stantec cannot be responsible for site work carried out without being present.

Appendix B Borehole Location Plan

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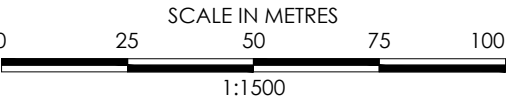


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2024/10/08 4:58 PM By: Loewen, Caroline



LEGEND

-  BOREHOLE LOCATION
-  PROPERTY / LEGAL LINE
-  GROUND CONTOUR (2018)



DISCLAIMER: The Contractor shall verify and be responsible for all dimensions. DO NOT scale the drawing - any error or omissions shall be reported to Stantec without delay. The Copyrights to all designs and drawings are the property of Stantec. Reproduction or use for any purpose other than that authorized by Stantec is forbidden.

ORIGINAL SHEET - ANSI B

Sources

Project Information

Project No.: 123317336
Scale: 1:400
Date: 2024-OCT-04
Drawn by: C. LOEWEN
Checked by: K. ALDER

Project Location

TOWNSHIP OF LANGLEY, BRITISH COLUMBIA

Client/Project

BC HYDRO

CAMPBELL HEIGHTS PROPERTY ASSESSMENT - SITE 14

TITLE

BOREHOLE SITE PLAN

Figure No.

1

Appendix C Borehole Records

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Stantec

BOREHOLE RECORD

BH24-01

CLIENT: **BC Hydro**

BH COORDINATES

PROJECT NO.: **123317336**

PROJECT: **Campbell Heights Site Assessment - Site 14**

[UTM10]

BH ELEVATION: **48m**

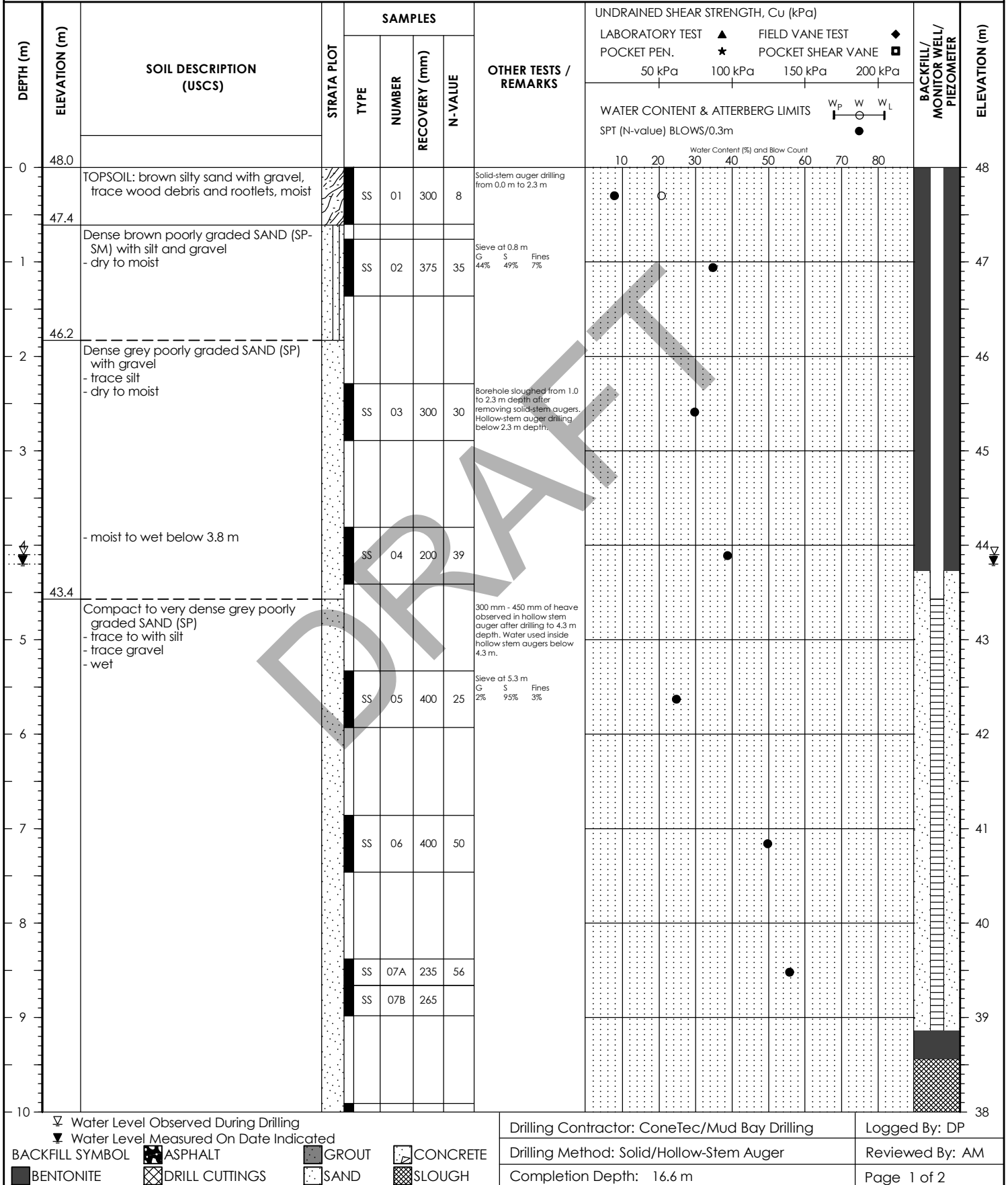
LOCATION: **Lots 9 & 11, Sec. 22, Twsp. 7, NWD, Township of Langley, BC**

5433903N 523338E

DATUM: **Geodetic**

DATE BORED: **September 9, 2024**

WATER LEVEL: **4.2 m on October 4, 2024**



PROJECT NO.: 123317336

BH ELEVATION: 48m

DATUM: **Geodetic**

WATER LEVEL: 4.2 m on October 4, 2024

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION (USCS)	STRATA PLOT	SAMPLES				OTHER TESTS / REMARKS	UNDRAINED SHEAR STRENGTH, Cu (kPa)		FIELD VANE TEST	POCKET PEN.	POCKET SHEAR VANE	BACKFILL / MONITOR WELL / PIEZOMETER	ELEVATION (m)		
				TYPE	NUMBER	RECOVERY (mm)	N-VALUE		LABORATORY TEST	WATER CONTENT & ATTERBERG LIMITS							
									50 kPa	100 kPa	150 kPa	200 kPa					
									SPT (N-value) BLOWS/0.3m								
									Water Content (%) and Blow Count								
									10	20	30	40	50	60	70	80	
10		Compact to very dense grey poorly graded SAND (SP) - trace to with silt - trace gravel - wet		SS	08	500	60	Sieve at 9.9 m G 7% S 85% Fines 8%								38	
11																	37
12				SS	09	600	70										36
13				SS	10	450		Sieve at 13.0 m G 1% S 94% Fines 5%									35
14																	34
15				SS	11	325	18										33
16	31.4	Borehole BH24-01 terminated at 16.6 m		SS	12	0	14	No recovery for SS-12 due to inferred gravel obstructing sampler.								32	
17																31	
18																30	
19																29	
20																28	

▽ Water Level Observed During Drilling

▼ Water Level Measured On Date Indicated

BACKFILL SYMBOL

ASPHALT

BENTONITE

DRILL CUTTINGS

SAND

GROUT

CONCRETE

SLOUGH

Drilling Contractor: ConeTec/Mud Bay Drilling

Drilling Method: Solid/Hollow-Stem Auger

Completion Depth: 16.6 m

Logged By: DP

Reviewed By: AM

Page 2 of 2



Stantec

BOREHOLE RECORD

BH24-02

CLIENT: **BC Hydro**

BH COORDINATES

PROJECT NO.: **123317336**

PROJECT: **Campbell Heights Site Assessment - Site 14**

[UTM10]

BH ELEVATION: **49m**

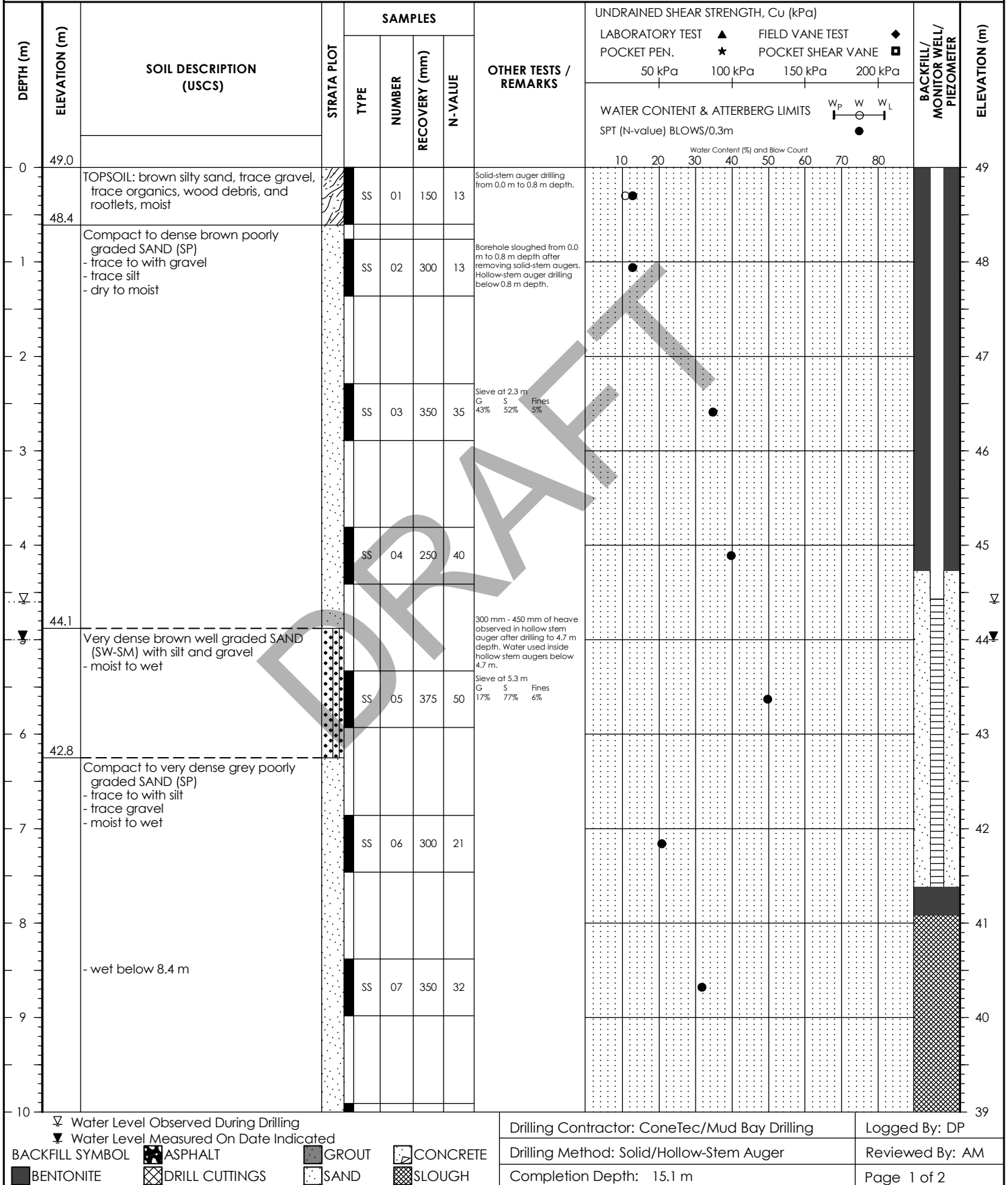
LOCATION: **Lots 9 & 11, Sec. 22, Twsp. 7, NWD, Township of Langley, BC**

5433791N 523505E

DATUM: **Geodetic**

DATE BORED: **September 11, 2024**

WATER LEVEL: **5.0 m on October 4, 2024**





Stantec

BOREHOLE RECORD

BH24-02

CLIENT: **BC Hydro**

BH COORDINATES

PROJECT NO.: **123317336**

PROJECT: **Campbell Heights Site Assessment - Site 14**

[UTM10]

BH ELEVATION: **49m**

LOCATION: **Lots 9 & 11, Sec. 22, Twsp. 7, NWD, Township of Langley, BC**

5433791N 523505E

DATUM: **Geodetic**

DATE BORED: **September 11, 2024**

WATER LEVEL: **5.0 m on October 4, 2024**

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION (USCS)	STRATA PLOT	SAMPLES				OTHER TESTS / REMARKS	UNDRAINED SHEAR STRENGTH, Cu (kPa)				BACKFILL / MONITOR WELL / PIEZOMETER	ELEVATION (m)			
				TYPE	NUMBER	RECOVERY (mm)	N-VALUE		LABORATORY TEST POCKET PEN.	FIELD VANE TEST POCKET SHEAR VANE	50 kPa	100 kPa			150 kPa	200 kPa	
10		Compact to very dense grey poorly graded SAND (SP) - trace to with silt - trace gravel - moist to wet		SS	08	475	32	Sieve at 9.9 m G S Fines 0% 94% 6%								39	
11																	38
12				SS	09	300	13										37
13																	36
14				SS	10	350	26										35
15	33.9	Borehole BH24-02 terminated at 15.1 m.		SS	11A	460	50									34	
				SS	11B	65										34	
16																33	
17																32	
18																31	
19															30		
20															29		

Water Level Observed During Drilling

Water Level Measured On Date Indicated

BACKFILL SYMBOL

ASPHALT

GROUT

CONCRETE

BENTONITE

DRILL CUTTINGS

SAND

SLOUGH

Drilling Contractor: ConeTec/Mud Bay Drilling

Drilling Method: Solid/Hollow-Stem Auger

Completion Depth: 15.1 m

Logged By: DP

Reviewed By: AM

Page 2 of 2

Appendix D ConeTec RSCPT Report

DRAFT



PRESENTATION OF SITE INVESTIGATION RESULTS

197A Street – 30 Ave, Langley

Prepared for:

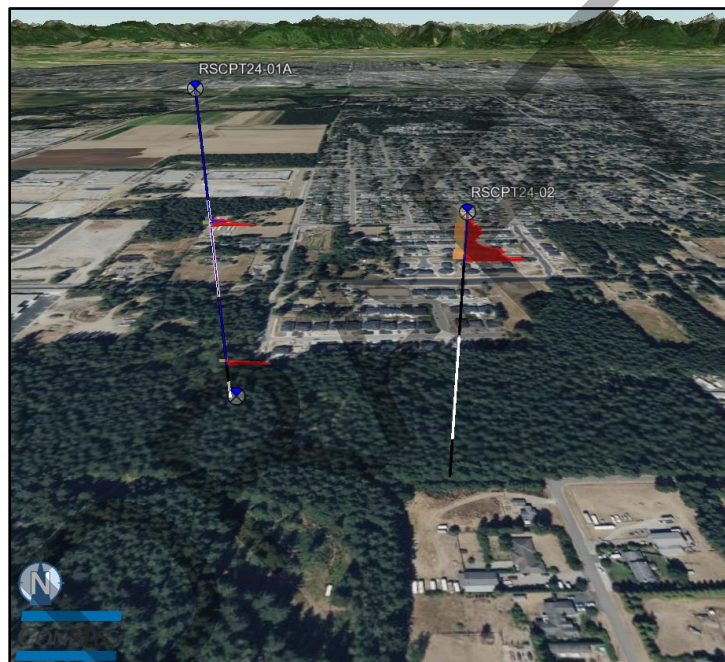
Stantec Consulting Ltd.

ConeTec Job No: 24-02-28291

Project Start Date: 2024-09-10

Project End Date: 2024-09-11

Report Date: 2024-10-02



Prepared by:

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www.conetecdataservices.com



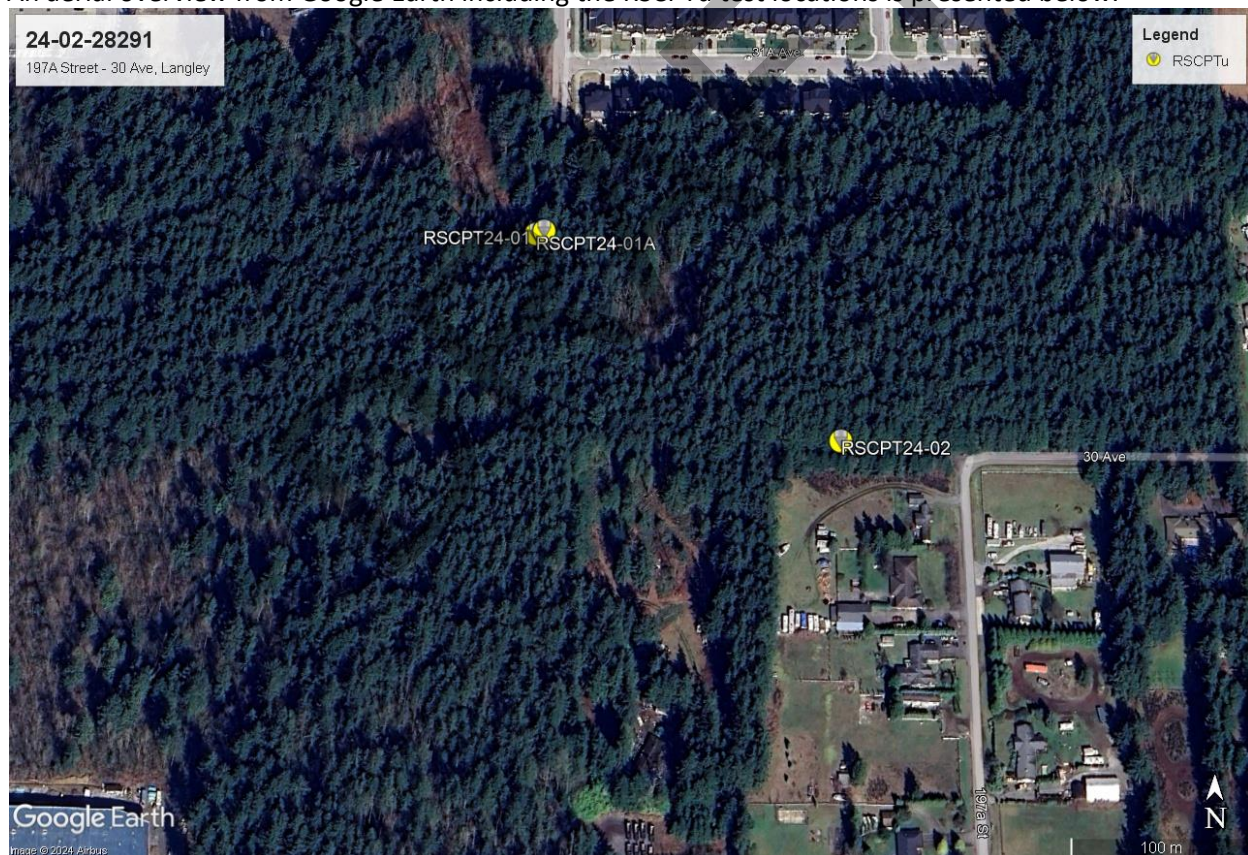
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Stantec Consulting Ltd. near 197A St and 30th Ave, Langley, BC. The program consisted of 3 resistivity seismic cone penetration tests (RSCPTu) with shear wave velocity measurements. 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 Information

Project	
Client	Stantec Consulting Ltd.
Project	197A Street - 30 Ave, Langley
ConeTec project number	24-02-28291

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



Rig Description	Deployment System	Test Type
Truck mounted drill rig (GT8)	14 ton rig cylinder	RSCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
RSCPTu	Consumer grade GPS	32610

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)
898:T1500F15U35	898	15	225	1500	15	35
Cone 898 was used for all CPTu soundings.						

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	- Resistivity plots - Advanced plots with I_c, S_u, ϕ and $N1(60)I_c$ - Seismic shear wave velocity plots

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and hydrostatic conditions were assumed. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Closure

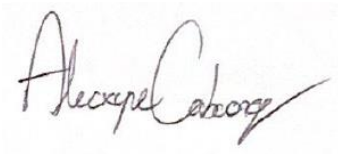
Thank you for the opportunity to work on this project. The equipment used and the field procedures followed complied with current accepted practice standards.

ConeTec Investigations Ltd.

Permit Number: 1002866

Report prepared by:

Report reviewed by:



Alecxyne Cabaong, GIT (AB)

James Greig, P.Eng.

Limitations

3rd Party Disclaimer

This report titled “197A Street - 30 Ave, Langley”, referred to as the (“Report”), was prepared by ConeTec for Stantec Consulting Ltd. 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 Stantec Consulting Ltd. to collect and provide the raw data (“Data”) which is included in this report titled “197A Street - 30 Ave, Langley”, which is referred to as 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. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively “Interpretations”) included in the Report, including those based on the Data, are outside the scope of ConeTec’s retainer and are included in the Report as a courtesy only. 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.

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 cm² and 15 cm² 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 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² 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 "u₂" 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](#).

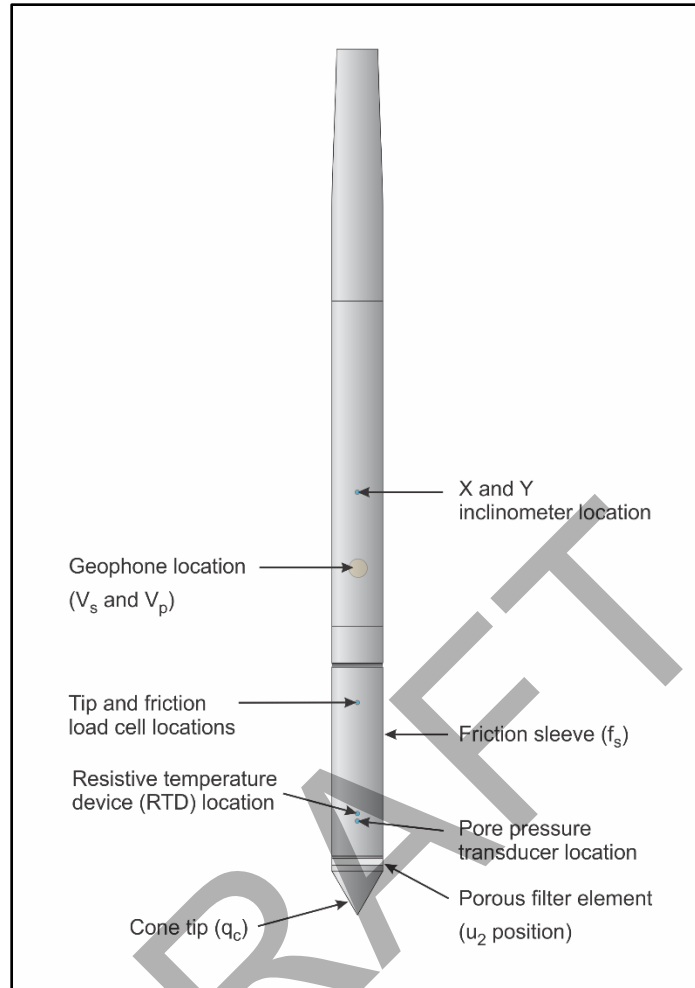


Figure CPTu. Piezocone Penetrometer (15 cm²)

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

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

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

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

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

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

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

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

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

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

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

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

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to [Robertson et al. \(1986\)](#), [Lunne et al. \(1997\)](#), [Robertson \(2009\)](#), Mayne (2013, 2014), [Mayne and Peuchen \(2012\)](#) and [Mayne et al. \(2023\)](#).

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.

Mayne, P.W., Cargill, E. and Greig, J. (2023). The Cone Penetration Test: Better Information, Better Decisions. produced by ConeTec Group, Burnaby, B.C. [www.conetec.com](#)

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

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158. DOI: [10.1139/T90-014](#).

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: [10.1139/T09-065](#).

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).

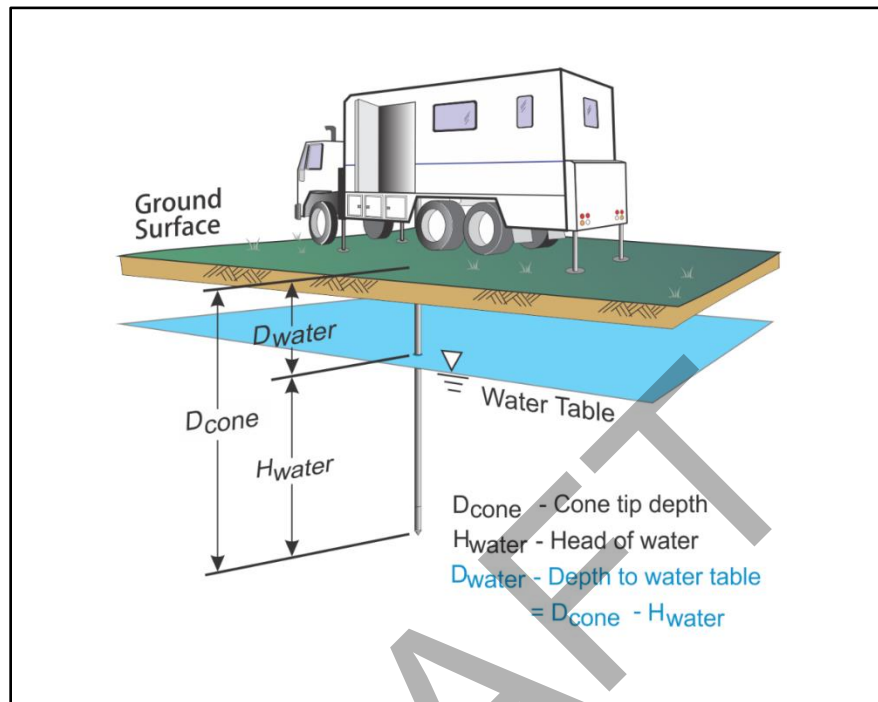


Figure PPD-1. Pore pressure dissipation test setup

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

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.

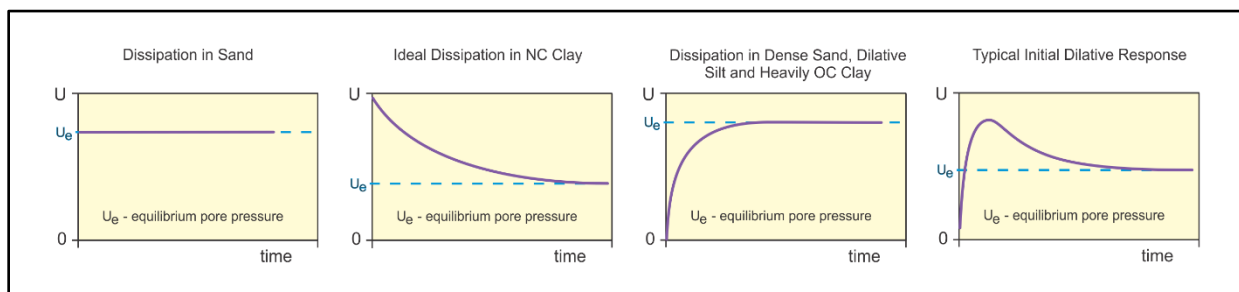


Figure PPD-2. Pore pressure dissipation curve examples

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 dilatatory 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.

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

References

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", *Geotechnique*, 41(1): 17-34. DOI: [10.1680/geot.1991.41.1.17](https://doi.org/10.1680/geot.1991.41.1.17).

To determine the electrical resistivity of the soil (the inverse of electrical conductivity), resistivity measurements are performed in conjunction with piezocone penetration testing (RCPTu) using a resistivity cone. An illustration of the resistivity piezocone penetrometer is presented in [Figure RCPTu](#).

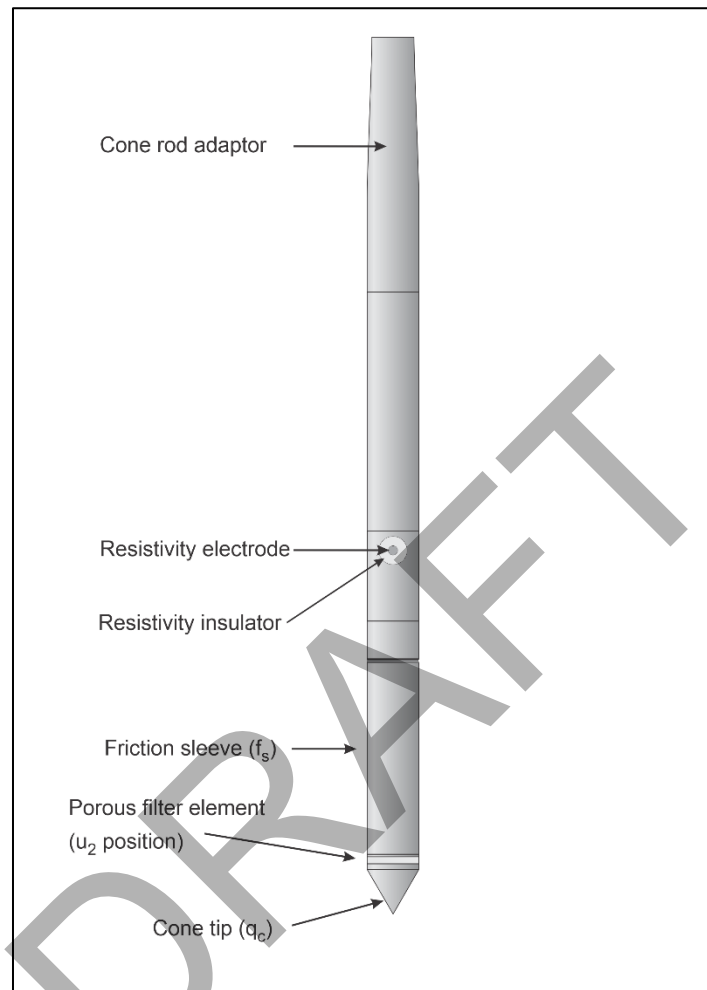


Figure RCPTu. Resistivity piezocone penetrometer (15 cm²)

The resistivity cone has a single, wear resistant, 8 mm diameter stainless steel electrode set within a 25 mm diameter Delrin plastic insulator. The measurement of resistivity is confined to the region of soil in contact with the electrode and the cone body just beyond the boundary of the insulator. The configuration of the electrode provides vertical resolution of resistivity changes of approximately 25 mm.

The electrical resistance is determined by measuring the voltage drop and current applied across the soil in contact with the electrode and cone body. A 960 hertz alternating current source is used to avoid polarization of the electrode. Polarization is the process where ions accumulate at the electrode thus increasing the measured resistance.

Electrical resistance is not a material property, it is a function of the resistivity of the material being measured, and the electrode configuration. To convert from resistance (ohm) to resistivity (ohm-m) a lab calibration is necessary. Resistivity cones are calibrated in a water tank with solutions of known resistivity. The resistance across the electrode and ground is measured in the various solutions and a calibration

curve is generated. It is necessary to assume that the calibration factors determined in the homogeneous isotropic medium do not vary significantly as the cone is advanced into the ground through soil.

Prior to the start of a test, the procedures described in the cone penetration test section are followed and the resistivity output is verified using various resistors. The resistivity measurements are recorded on a continuous basis at the same time as the tip, friction, and pore pressure measurements. Due to the vertical offset between the cone tip and the electrode, resistivity data is not available for the last 0.275 meters of each profile.

The resistivity of soil is for the most part influenced by the resistivity of the pore fluid, which in turn is a measure of the groundwater chemical composition. Electrical conduction in saturated sandy soils is largely by electrolytic conduction in the pore fluid whereas for clayey soils, ion exchange contributes significantly within the soil skeleton. Resistivity measurements will increase as the saturation of the soil decreases. For additional information on resistivity cone penetration testing, refer to [Campanella and Weemees \(1990\)](#).

Resistivity CPTu plots are presented in the relevant appendix.

References

Campanella, R.G. and Weemees, I., 1990, "Development and Use of an Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5: 557-567. DOI: [10.1139/T90-071](#).

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

ConeTec's 15 cm² piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves; however, it is often affected by the compression wave travelling through the cone rods.

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 may be 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 in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in [Figure SCPTu-1](#).

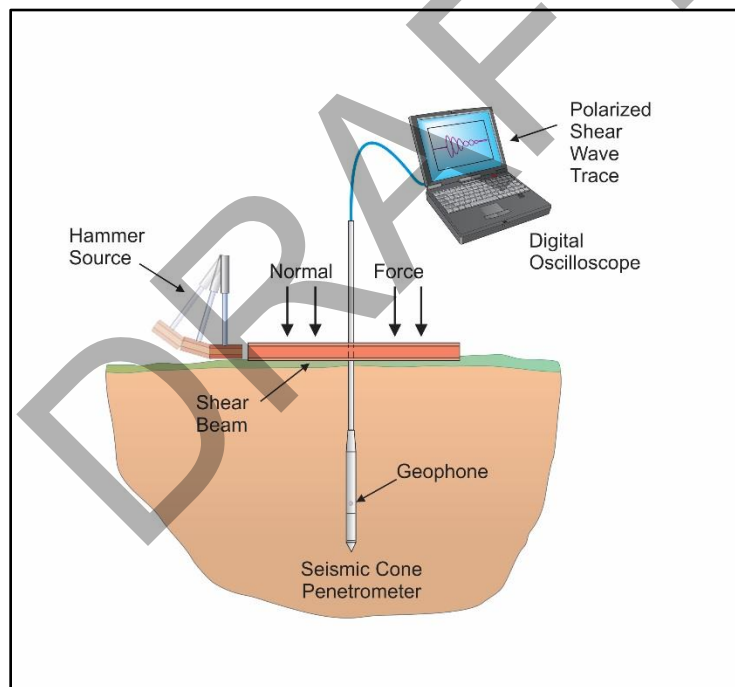


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current [ASTM D5778](#) 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 and uncertainty analysis purposes. 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. \(1986\)](#).

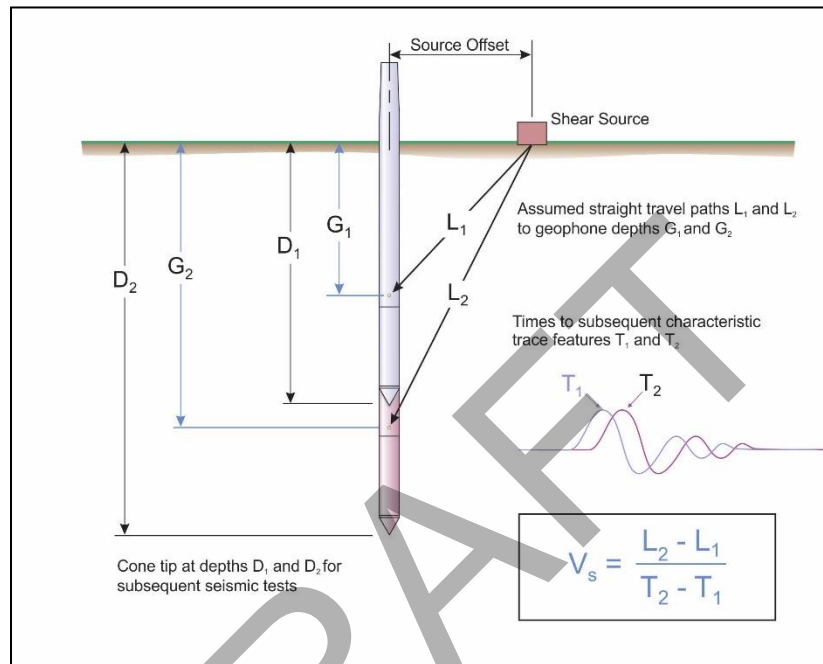


Figure SCPTu-2. Illustration of a seismic cone penetration test

For the determination of interval travel times the wave traces from all depths are displayed in analysis software. The results of the interval picks are supplied in the relevant appendix of this report. Standard practice for ConeTec is to record five wave traces for each source direction at each test depth. Outlier impacts are identified in the field and the impacts are repeated. For the final wave trace profile, the traces are stacked in the time domain to display a single average trace.

Determination of the shear wave interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the trace depths and taking the difference in ray path divided by the time difference between features at subsequent depths. The same process is used for compression waves, however the first break is most commonly used for selecting an arrival time. For velocity calculation, the 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.

In some cases, usually for shear wave velocity testing, more than one characteristic marker may be used. If there is an overlap between different sets of characteristic markers, then the average time value for those sets of interval times is applied to the determination of velocity.

Ideally, all depths are used for the determination of the velocity profile. However, an interval may be skipped if there is some ambiguity or quality concern with a particular depth, resulting in a larger interval.

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

The average shear wave velocity to a depth of thirty meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in [Crow et al. \(2012\)](#).

$$V_{s30} = \frac{\text{total thickness of all layers (30m)}}{\sum(\text{layer travel times})}$$

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

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](#).

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: [10.1520/D7400_D7400M-19](#).

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

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. DOI: [10.1061/\(ASCE\)0733-9410\(1986\)112:8\(791\)](#).

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Resistivity Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave (V_s) Tabular Results
- Seismic Cone Penetration Test Shear Wave (V_s) Traces
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots
- Description of Methods for Calculated CPT Geotechnical Parameters

DRAFT

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

DRAFT



Job No: 24-02-28291
Client: Stantec Consulting Ltd.
Project: 197A Street - 30 Ave, Langley
Start Date: 2024-09-10
End Date: 2024-09-11

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm ²)	Assumed Phreatic Surface ¹ (m)	Final Depth (m)	Northing ² (m)	Easting ² (m)	Refer to Notation Number
RSCPT24-01	24-02-28291_RS01	2024-09-10	898:T1500F15U35	15		1.425	5433913	523333	3, 4
RSCPT24-01A	24-02-28291_RS01A	2024-09-10	898:T1500F15U35	15	4.1	13.950	5433911	523328	5
RSCPT24-02	24-02-28291_RS02	2024-09-11	898:T1500F15U35	15		1.875	5433790	523508	3

1. The assumed phreatic surface was based on pore pressure dissipation tests, unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device in datum WGS84/UTM Zone 10 North.
3. No clear phreatic surface was detected.
4. No seismic was reported due to the shallow refusal.
5. The material from 13.525m to 13.700m is potentially disturbed due to auger drilling.



Stantec

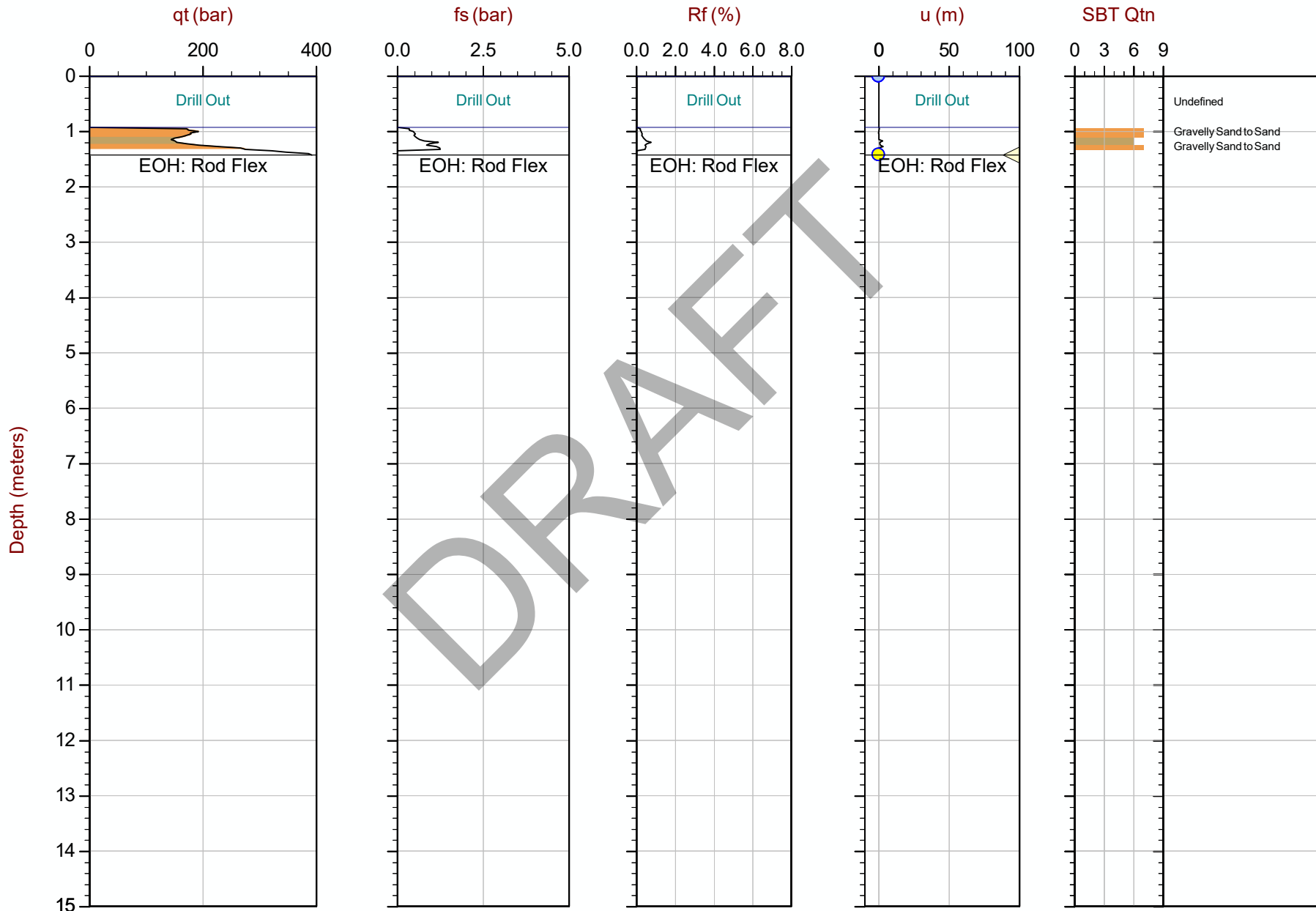
Job No: 24-02-28291

Date: 2024-09-10 09:28

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 1.425 m / 4.68 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved

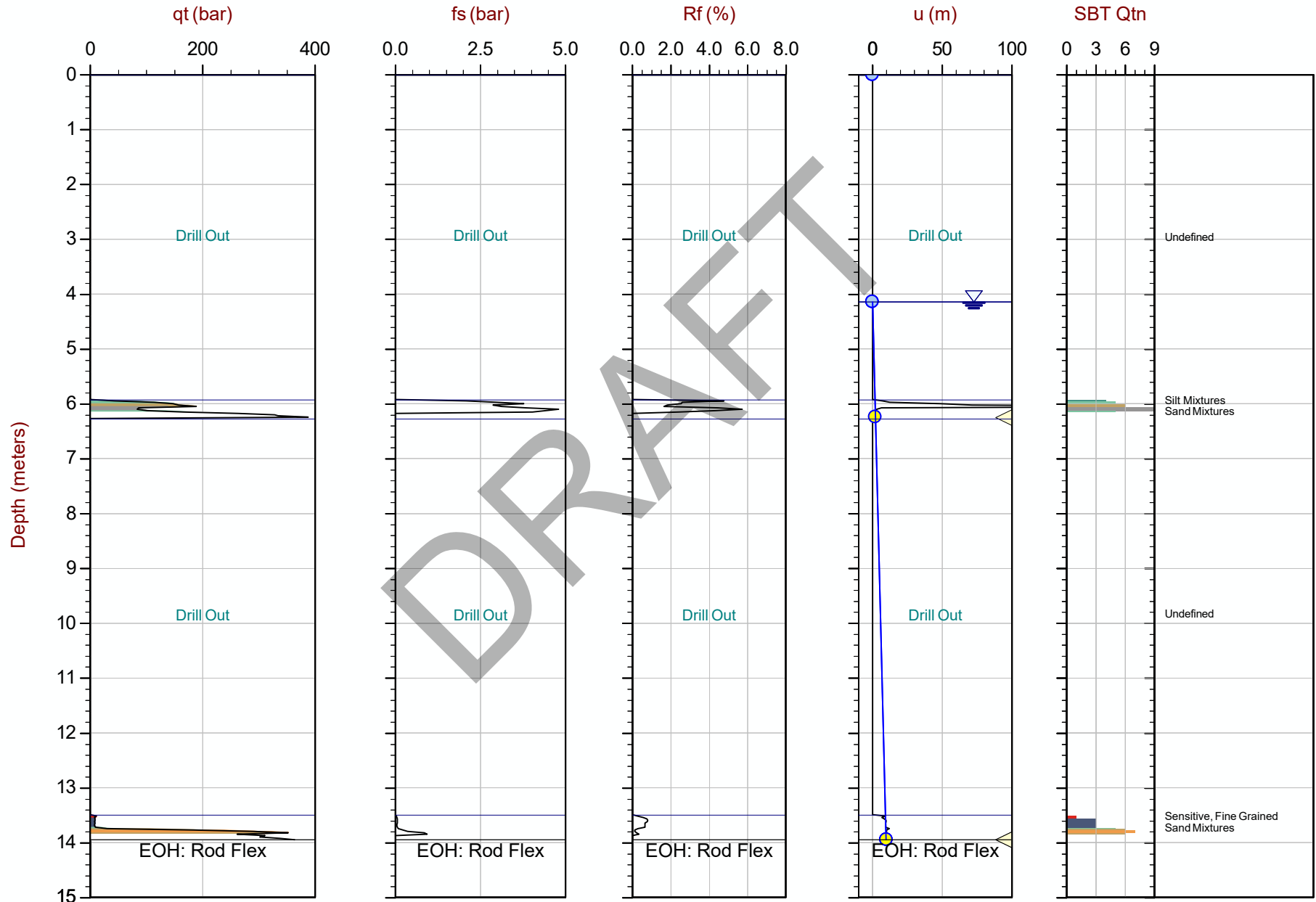
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433913m E: 523333m

Sheet No: 1 of 1

◀ Dissipation, Ueq assumed — Hydrostatic Line



Max Depth: 13.950 m / 45.77 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS01A.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433911m E: 523328m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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.



Stantec

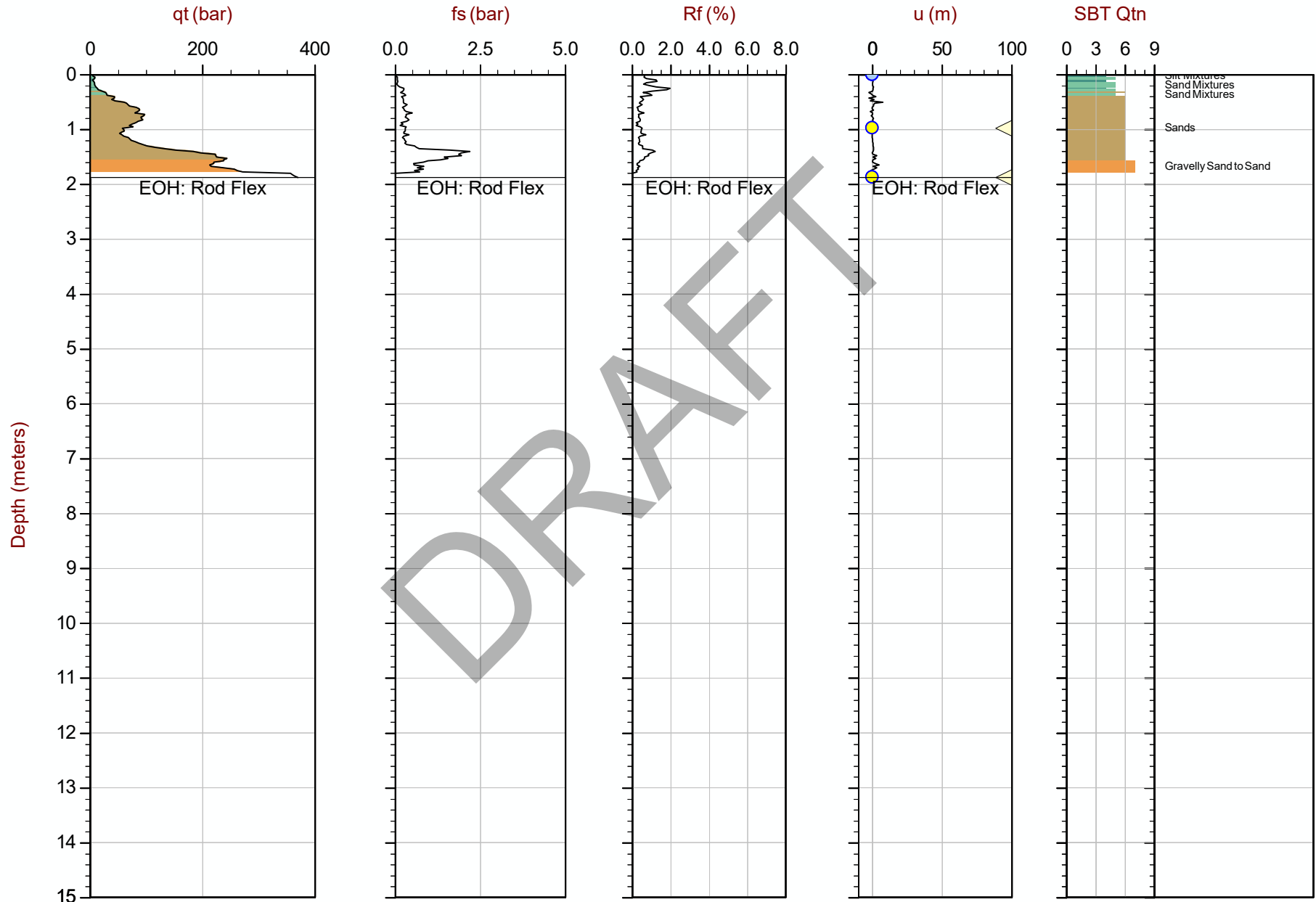
Job No: 24-02-28291

Date: 2024-09-11 14:15

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-02

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 1.875 m / 6.15 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433790m E: 523508m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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.

Resistivity Cone Penetration Test Plots

DRAFT



Stantec

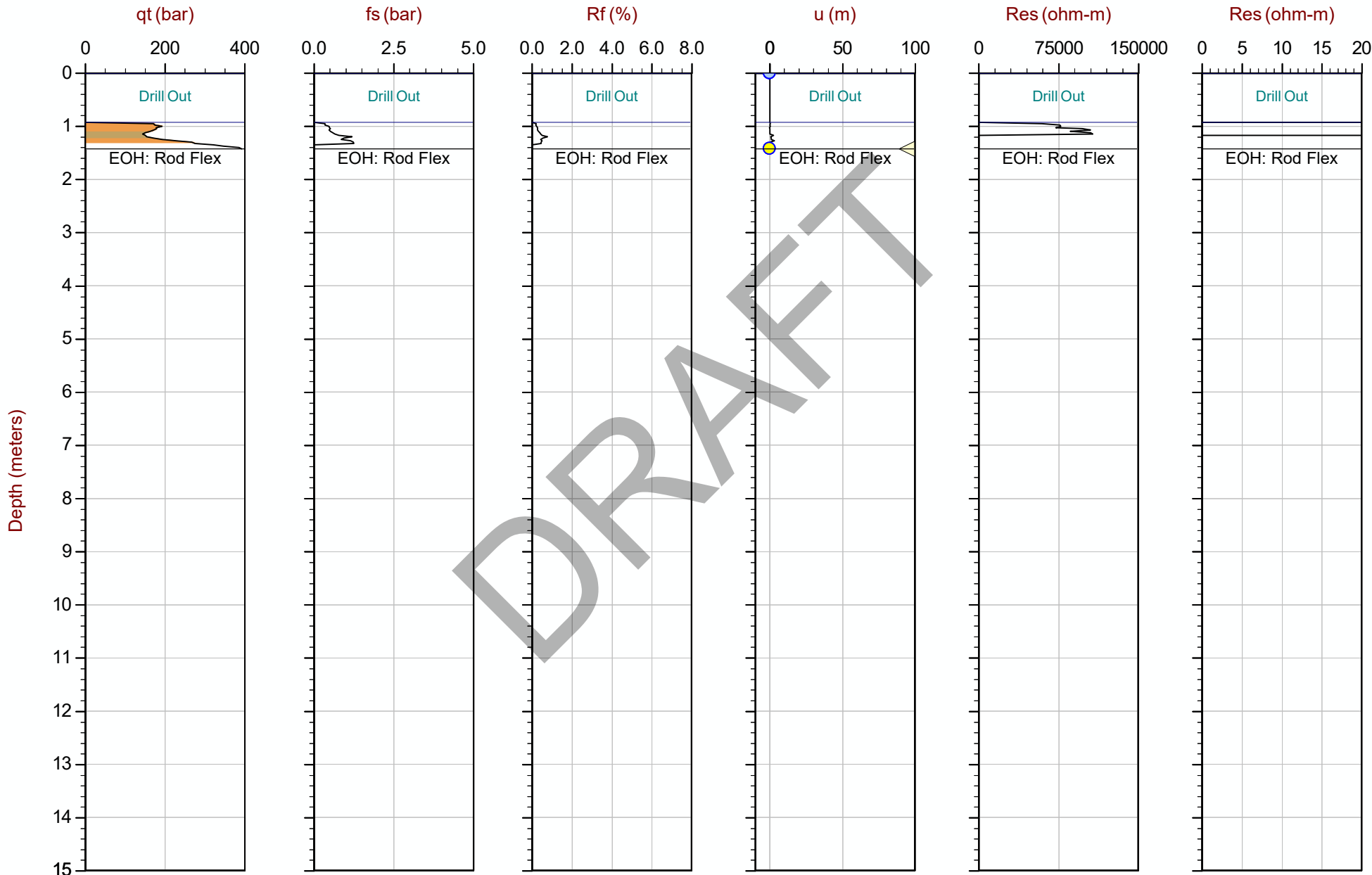
Job No: 24-02-28291

Date: 2024-09-10 09:28

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 1.425 m / 4.68 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS01.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433913m E: 5233333m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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.



Stantec

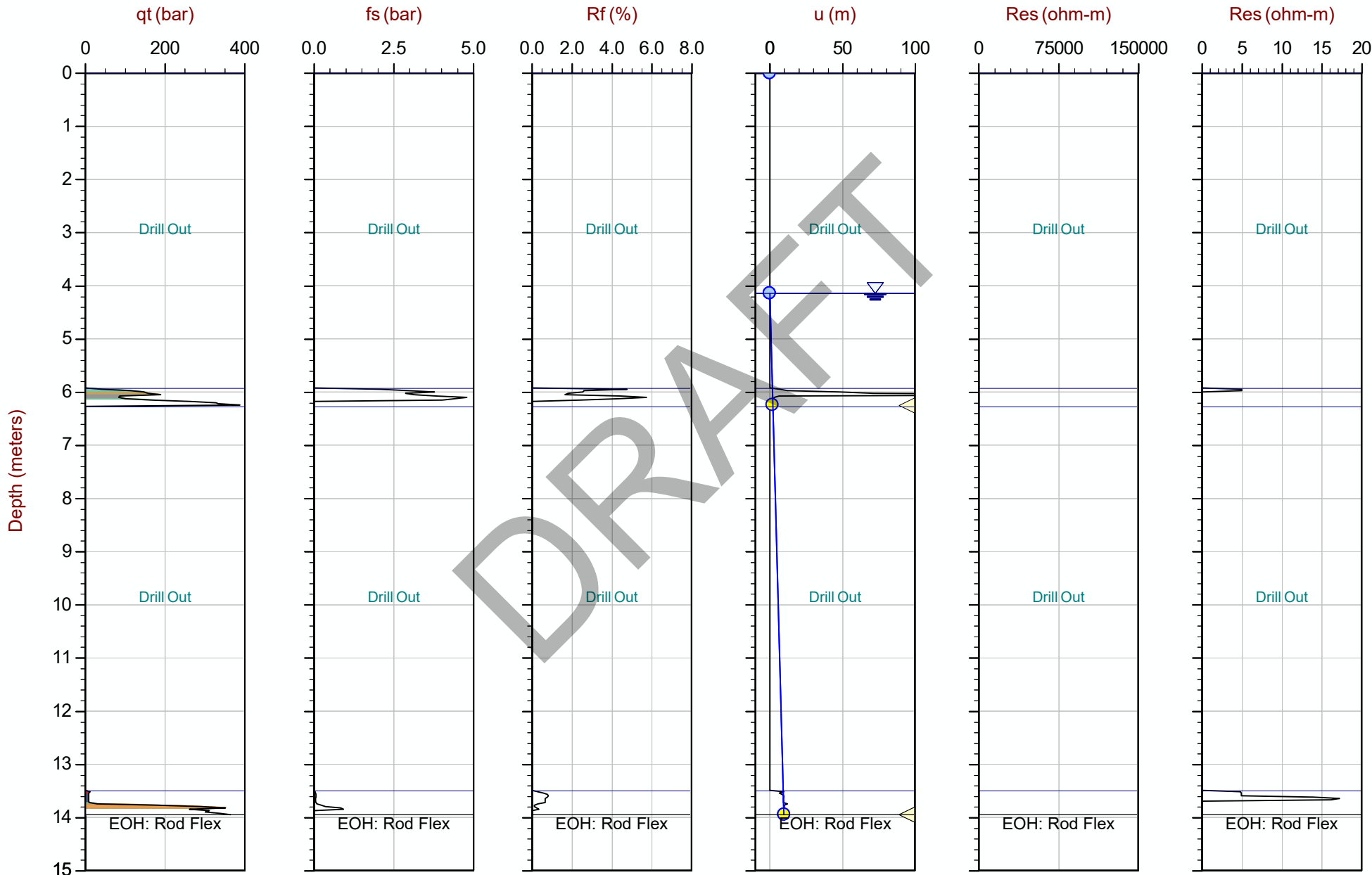
Job No: 24-02-28291

Date: 2024-09-10 10:09

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01A

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 13.950 m / 45.77 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS01A.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433911m E: 523328m

Sheet No: 1 of 1

Overplot Item: ● Ueq ○ Assumed Ueq ◀ Dissipation, Ueq achieved ▶ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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.



Stantec

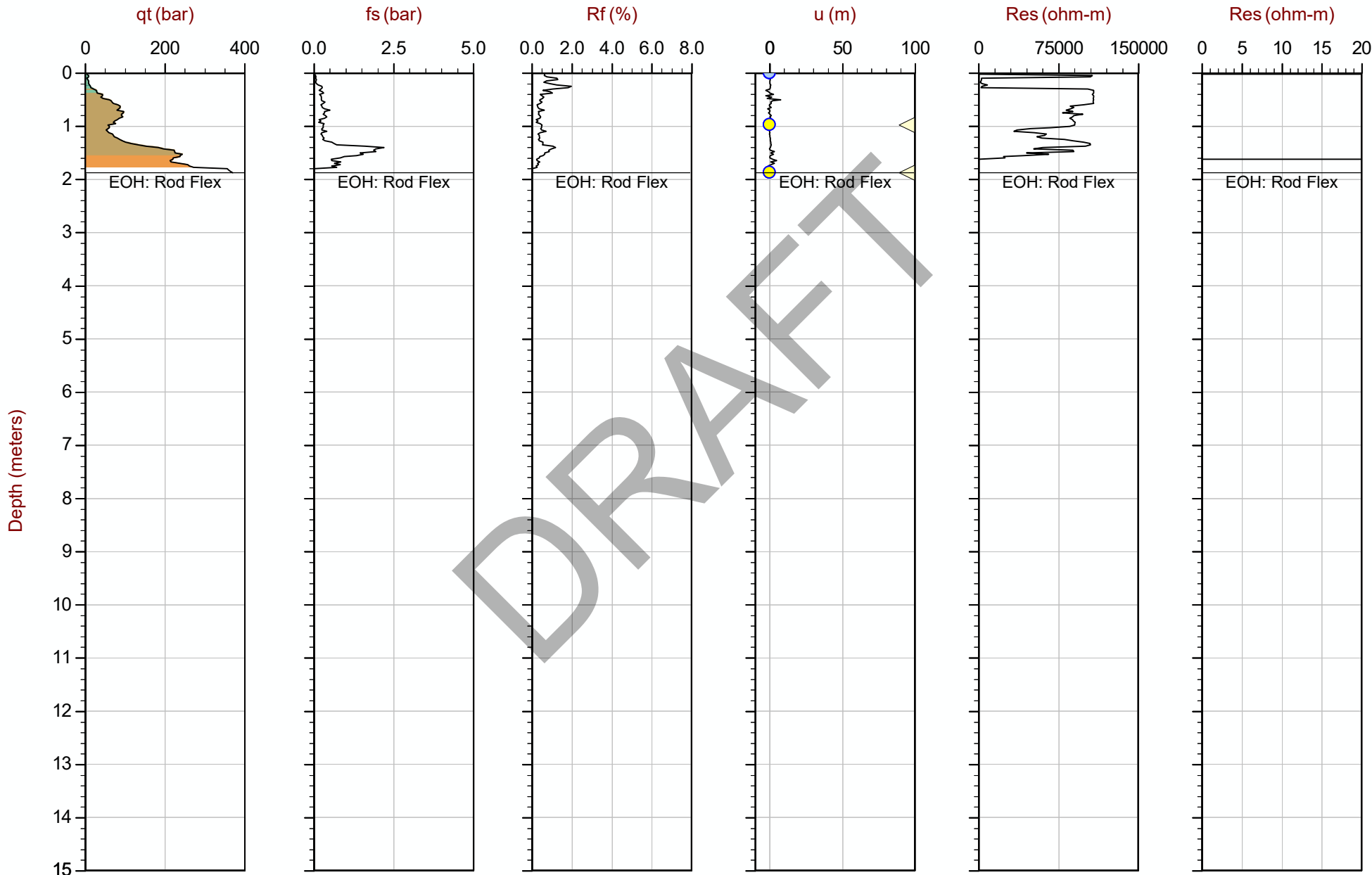
Job No: 24-02-28291

Date: 2024-09-11 14:15

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-02

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 1.875 m / 6.15 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433790m E: 523508m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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 Plots

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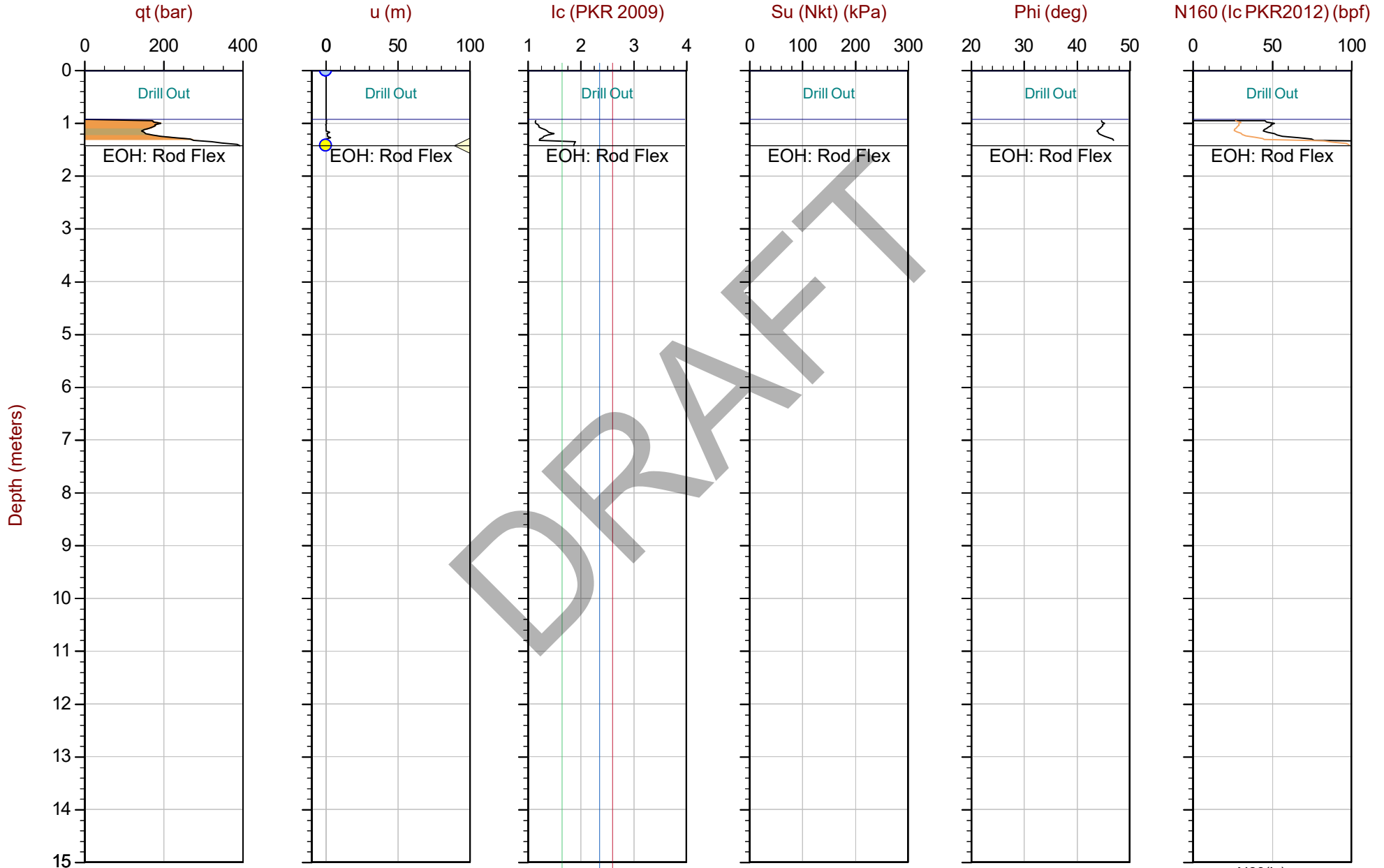
Job No: 24-02-28291

Date: 2024-09-10 09:28

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 1.425 m / 4.68 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS01.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433913m E: 523333m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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.



Stantec

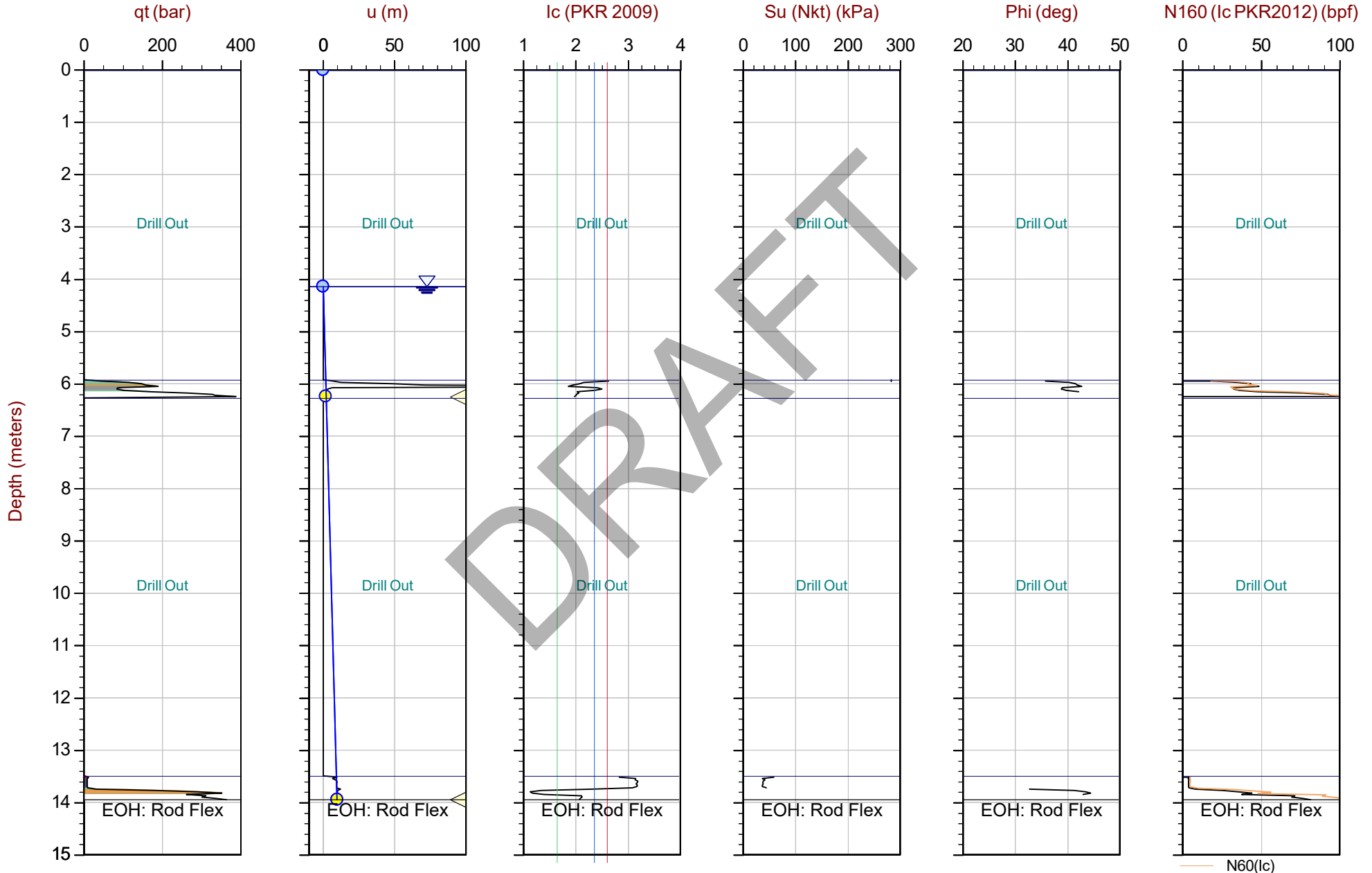
Job No: 24-02-28291

Date: 2024-09-10 10:09

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01A

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 13.950 m / 45.77 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS01A.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433911m E: 523328m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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.



Stantec

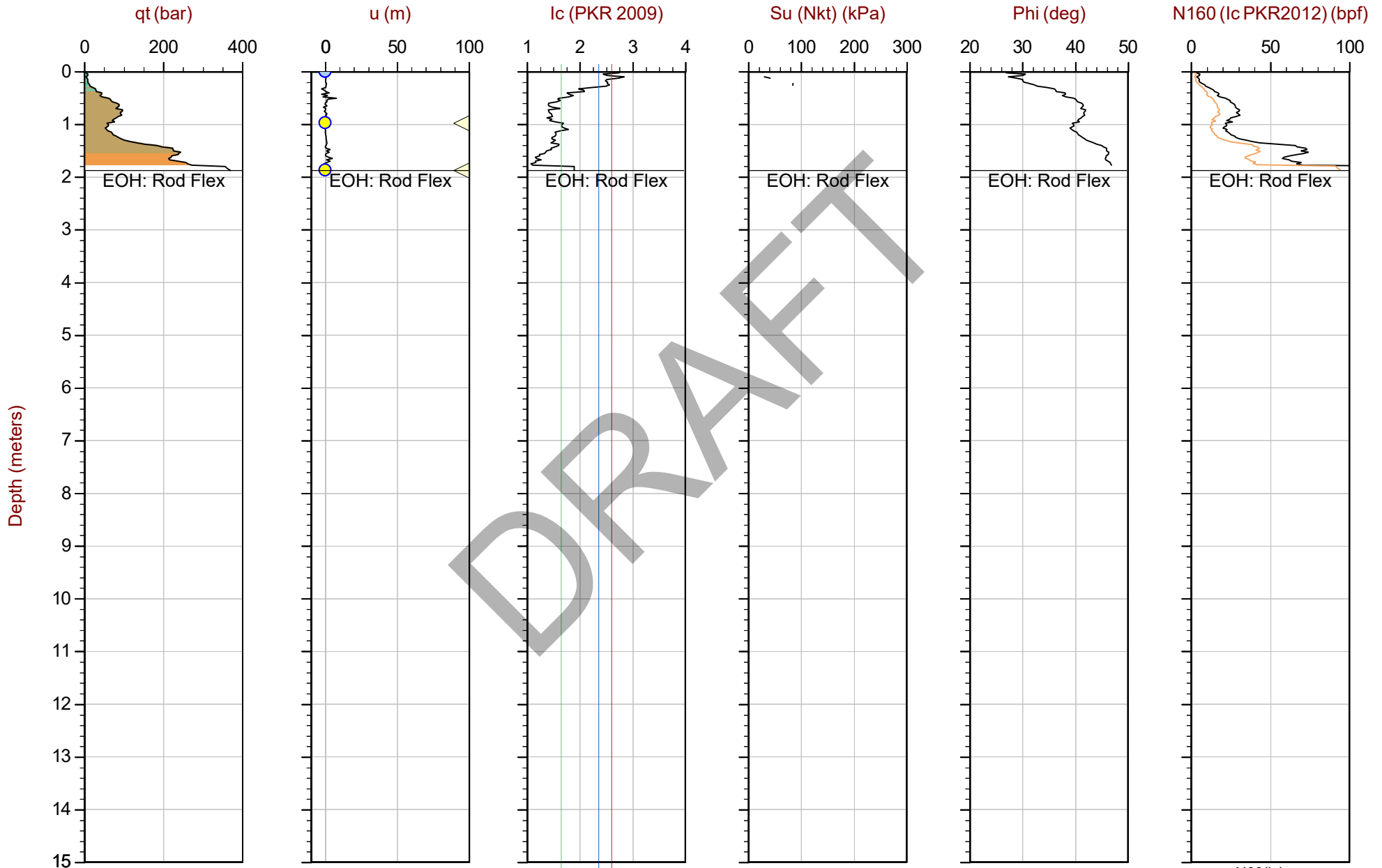
Job No: 24-02-28291

Date: 2024-09-11 14:15

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-02

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 1.875 m / 6.15 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS02.COR

Unit Wt: SBTQtn(PKR2009)

Su Nkt: 15.0

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433790m E: 523508m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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

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Stantec

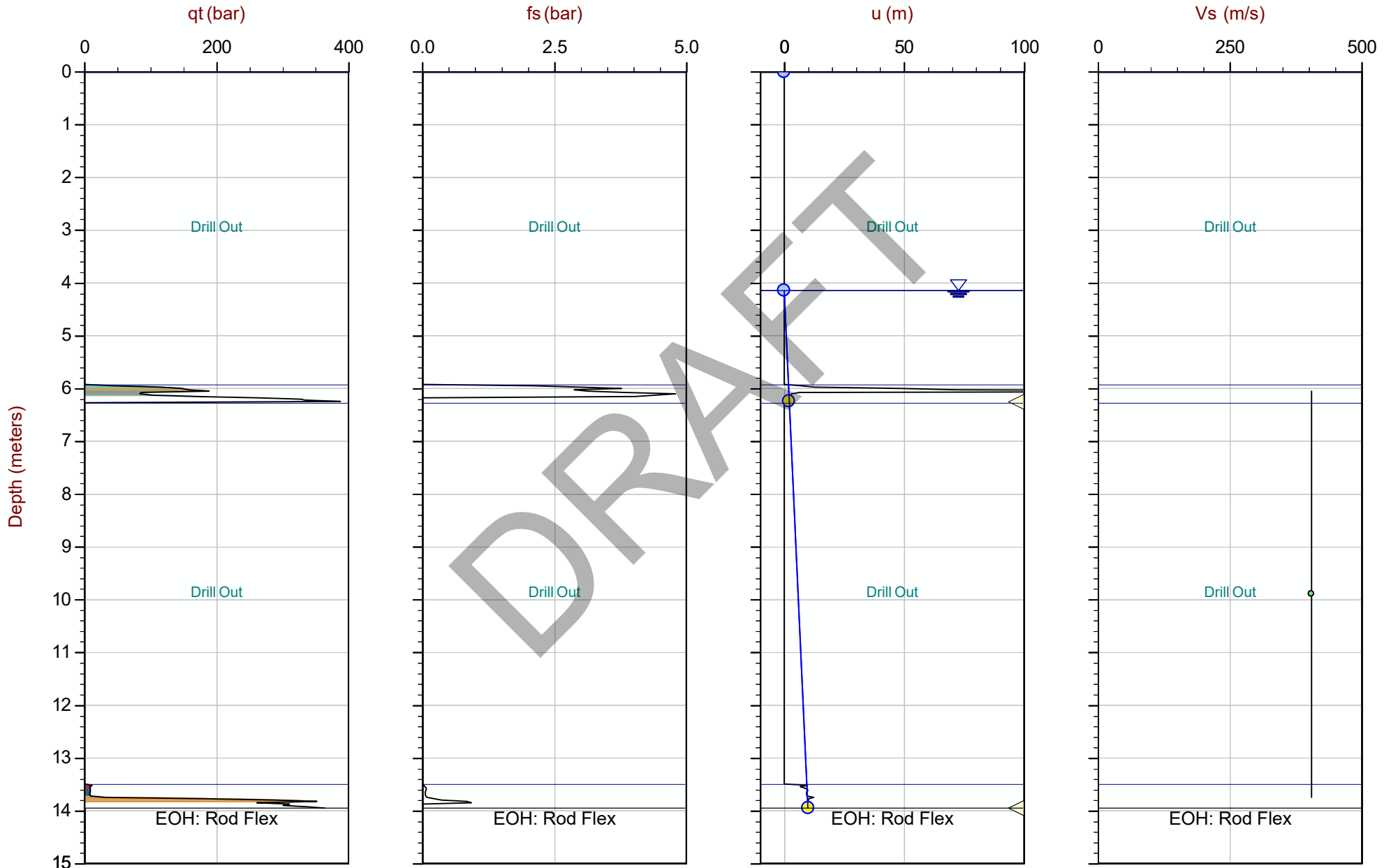
Job No: 24-02-28291

Date: 2024-09-10 10:09

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01A

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 13.950 m / 45.77 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 24-02-28291_RS01A.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433911m E: 523328m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ◀ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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.



Stantec

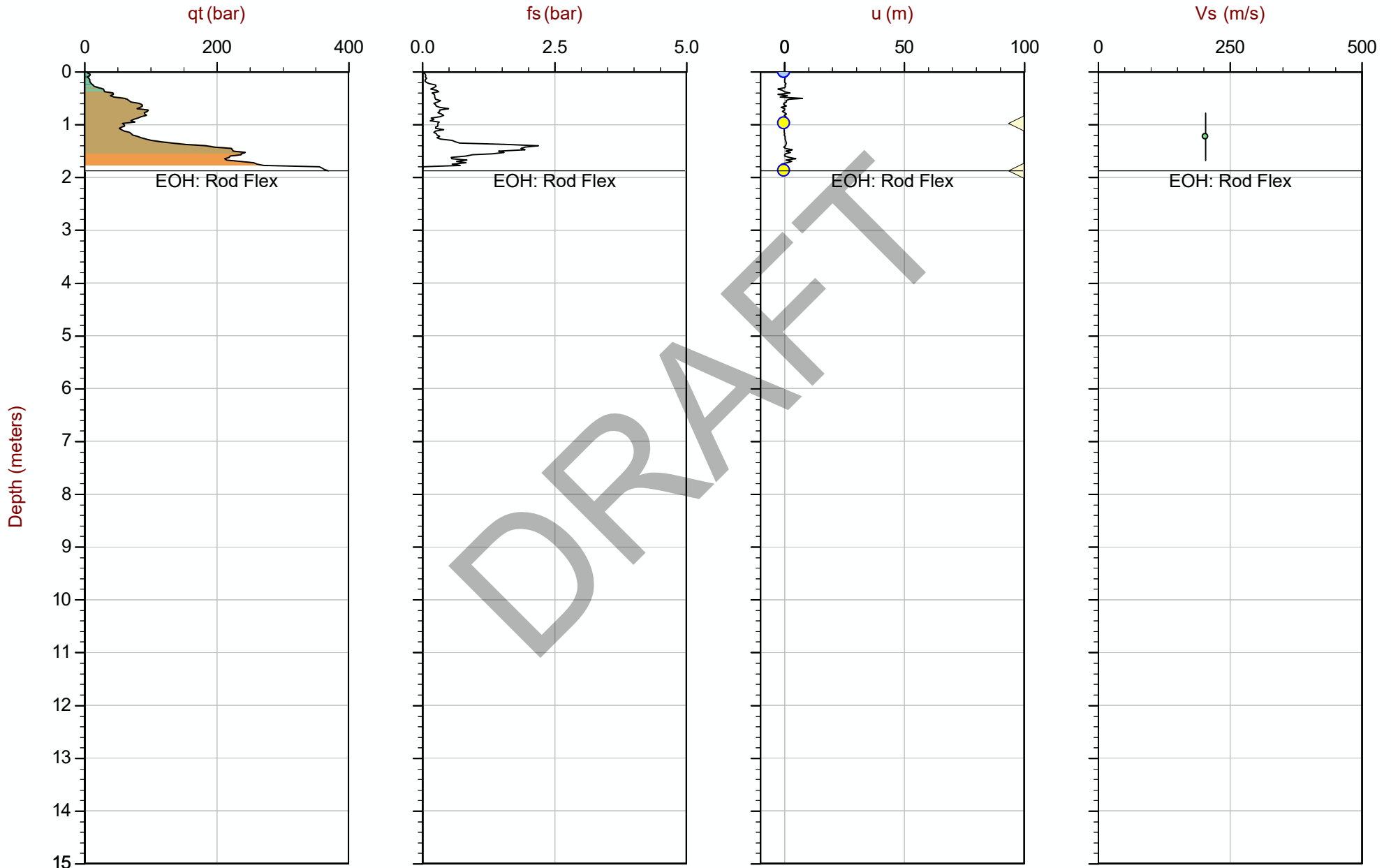
Job No: 24-02-28291

Date: 2024-09-11 14:15

Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-02

Cone: 898:T1500F15U35 Area=15 cm²



Max Depth: 1.875 m / 6.15 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 24-02-28291_RS02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10N N: 5433790m E: 523508m

Sheet No: 1 of 1

Overplot Item: ● Ueq ● Assumed Ueq ◀ Dissipation, Ueq achieved ▶ Dissipation, Ueq not achieved ◀ Dissipation, Ueq assumed — 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 (V_s) Tabular Results

DRAFT



Job No: 24-02-28291
Client: Stantec Consulting Ltd.
Project: 197A Street - 30 Ave, Langley
Sounding ID: RSCPT24-01A
Date: 2024-09-10

Seismic Source: Beam
Seismic Offset (m): 1.00
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
6.25	6.05	6.13			
13.95	13.75	13.79	7.65	18.90	405

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Job No: 24-02-28291
Client: Stantec Consulting Ltd.
Project: 197A Street - 30 Ave, Langley
Sounding ID: RSCPT24-02
Date: 2024-09-11

Seismic Source: Beam
Seismic Offset (m): 1.00
Source Depth (m): 0.00
Geophone Offset (m): 0.20

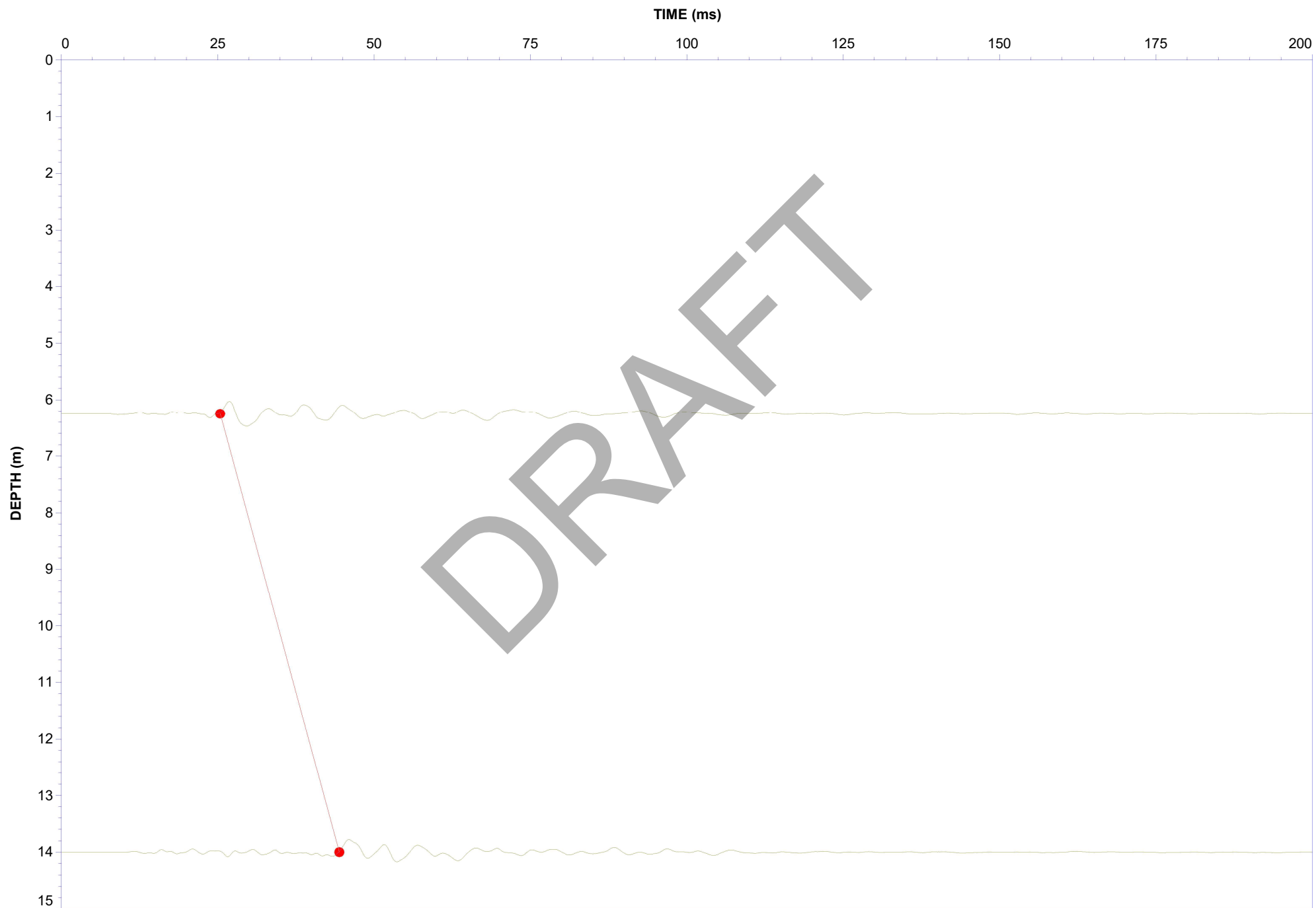
SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.98	0.78	1.27			
1.88	1.68	1.96	0.69	3.37	204

DRAFT

Seismic Cone Penetration Test Shear Wave (V_s) Traces

DRAFT





Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

DRAFT



Job No: 24-02-28291
Client: Stantec Consulting Ltd.
Project: 197A Street - 30 Ave, Langley
Start Date: 2024-09-10
End Date: 2024-09-11

CPT_u PORE PRESSURE DISSIPATION SUMMARY

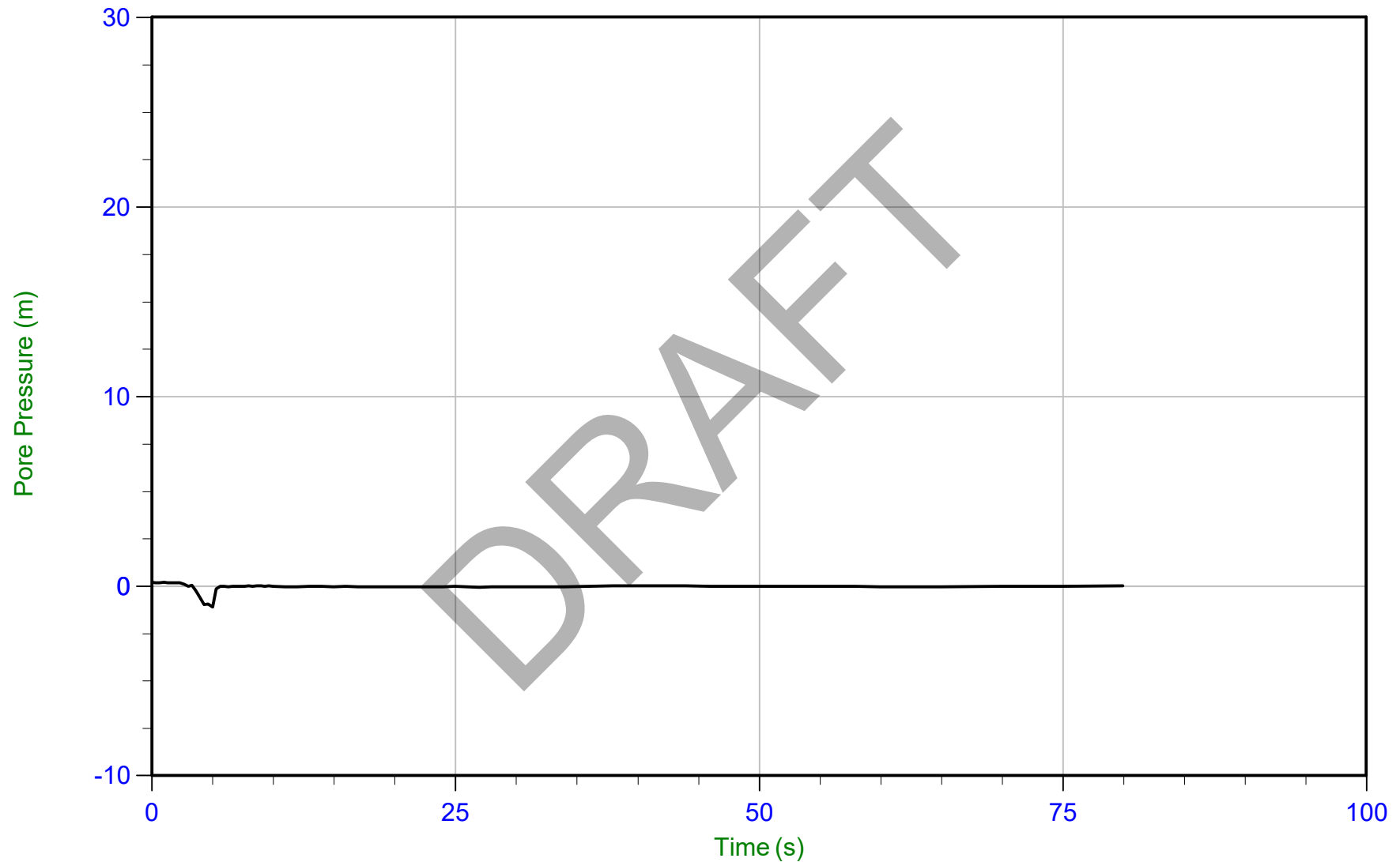
Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (m)	U _{initial} (m)	U _{max} (m)	U _{min} (m)	U _{final} (m)	Observed Equilibrium Pore Pressure U _{eq} (m)	Assumed Phreatic Surface (m)	Refer to Notation Number
RSCPT24-01	24-02-28291_RS01	15	80	1.425	0.2	0.2	-1.1	0.0	0.0		
RSCPT24-01A	24-02-28291_RS01A	15	270	6.250	2.6	2.7	1.7	2.1	2.1	4.1	
RSCPT24-01A	24-02-28291_RS01A	15	170	13.950	10.0	10.0	9.9	10.0	10.0	4.0	
RSCPT24-01A	24-02-28291_RS01A	15	125	13.950	10.5	10.6	4.4	10.0	10.0	4.0	
RSCPT24-02	24-02-28291_RS02	15	145	0.975	0.5	0.5	-5.8	0.0	0.0		
RSCPT24-02	24-02-28291_RS02	15	200	1.875	3.6	3.8	-1.8	0.0	0.0		



Stantec

Job No: 24-02-28291
Date: 2024-09-10 09:28
Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01
Cone: 898:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-02-28291_RS01.PPF2
Depth: 1.425 m / 4.675 ft
Duration: 80.0 s

u Min: -1.1 m
u Max: 0.2 m
u Final: 0.0 m

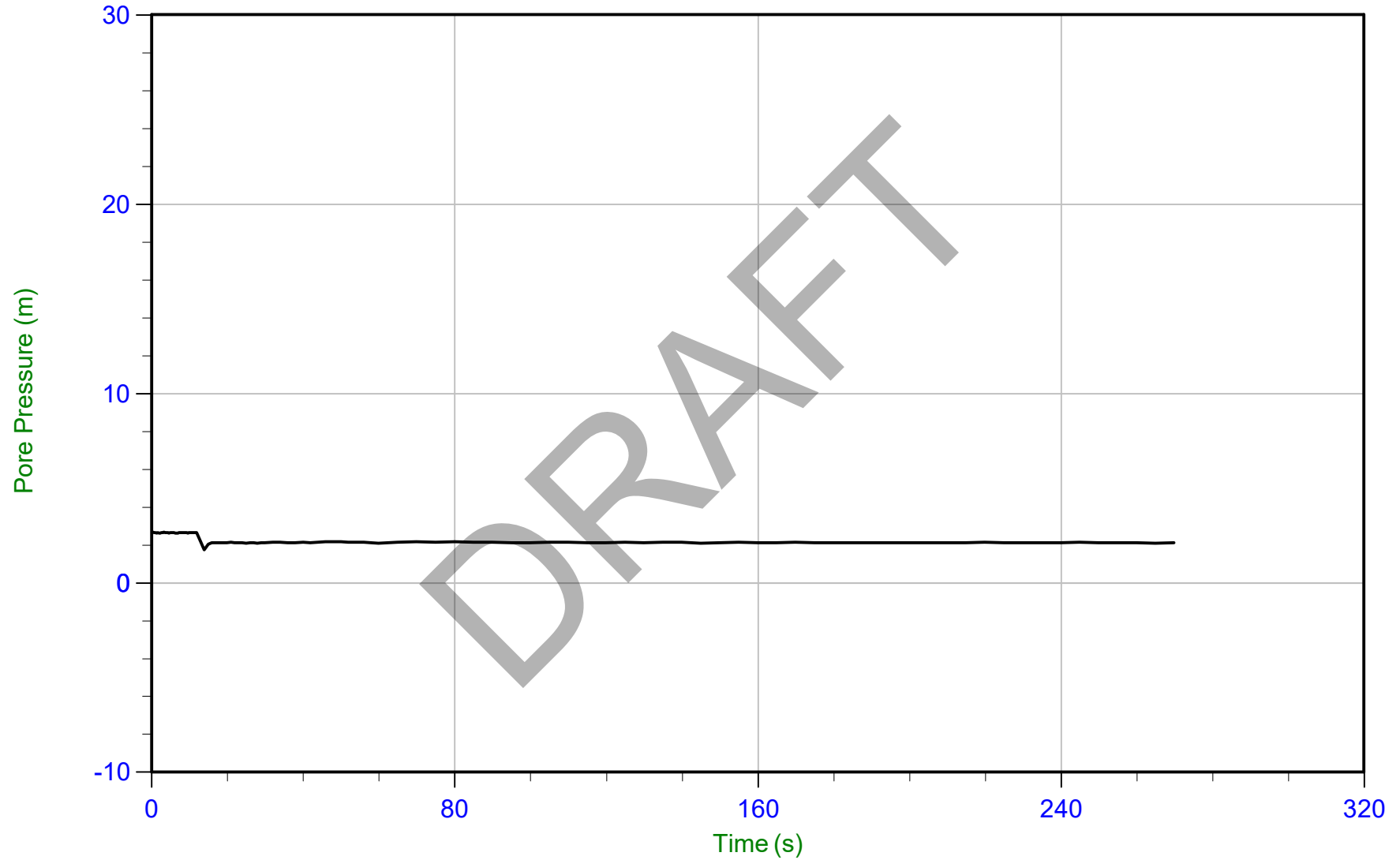
WT: 1.4 m / 4.7 ft
Ueq: 0.0 m



Stantec

Job No: 24-02-28291
Date: 2024-09-10 10:09
Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01A
Cone: 898:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-02-28291_RS01A.PPF2
Depth: 6.250 m / 20.505 ft
Duration: 270.0 s

u Min: 1.7 m
u Max: 2.7 m
u Final: 2.1 m

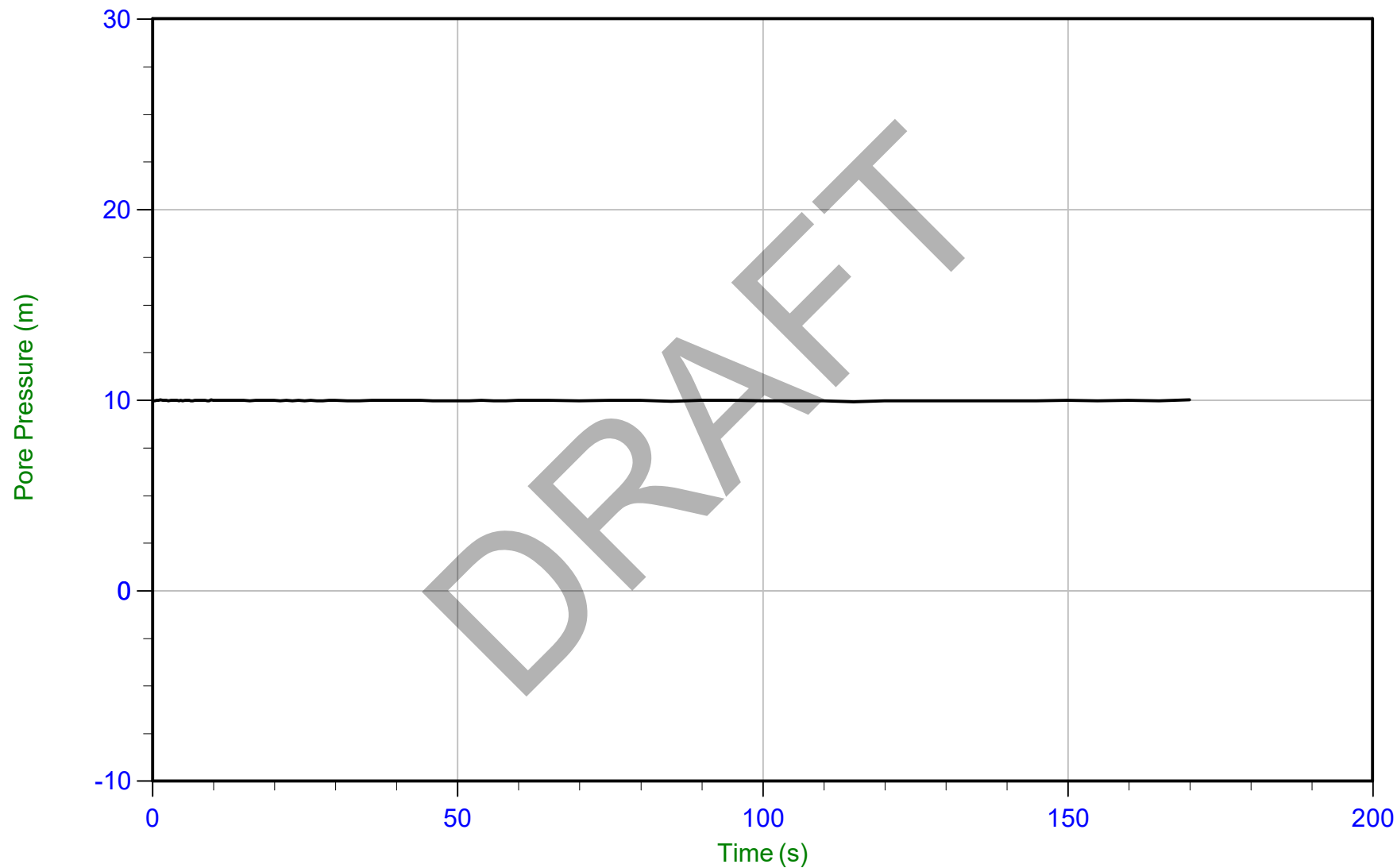
WT: 4.1 m / 13.6 ft
Ueq: 2.1 m



Stantec

Job No: 24-02-28291
Date: 2024-09-10 10:09
Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01A
Cone: 898:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-02-28291_RS01A.PPF2
Depth: 13.950 m / 45.767 ft
Duration: 170.0 s

u Min: 9.9 m
u Max: 10.0 m
u Final: 10.0 m

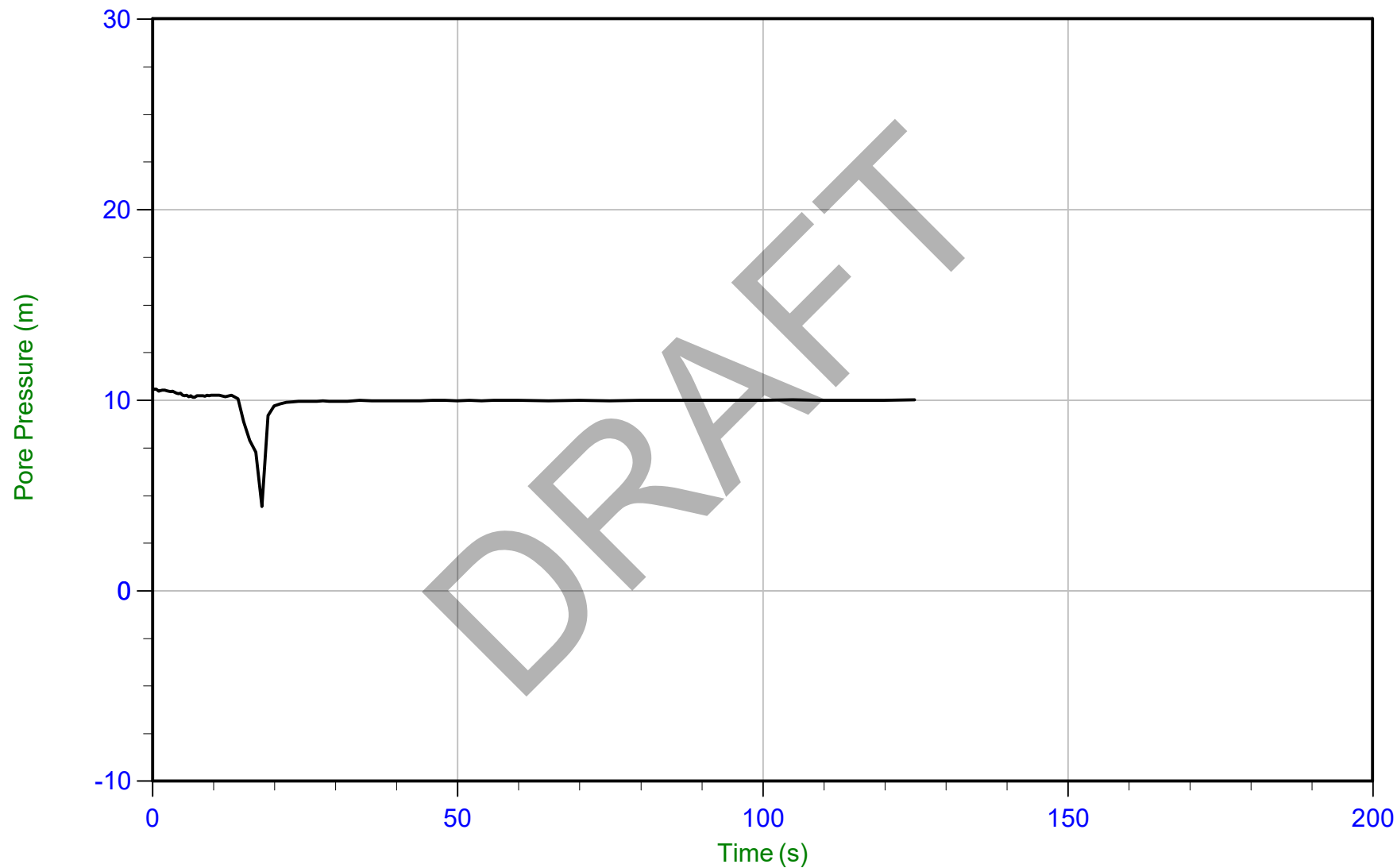
WT: 4.0 m / 13.0 ft
Ueq: 10.0 m



Stantec

Job No: 24-02-28291
Date: 2024-09-10 10:09
Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-01A
Cone: 898:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-02-28291_RS01A.PPF2
Depth: 13.950 m / 45.767 ft
Duration: 125.0 s

u Min: 4.4 m
u Max: 10.6 m
u Final: 10.0 m

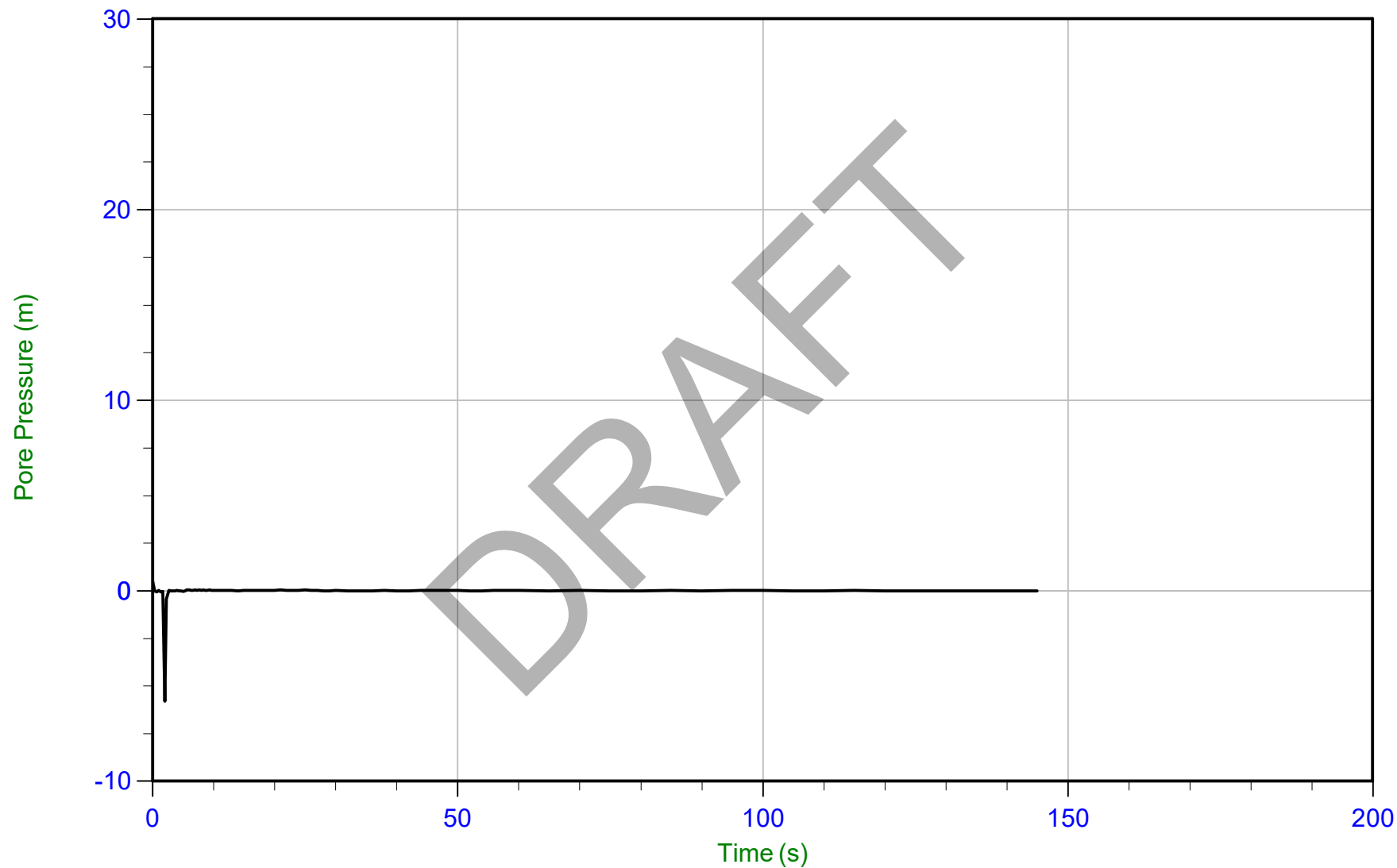
WT: 4.0 m / 13.0 ft
Ueq: 10.0 m



Stantec

Job No: 24-02-28291
Date: 2024-09-11 14:15
Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-02
Cone: 898:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-02-28291_RS02.PPF2
Depth: 0.975 m / 3.199 ft
Duration: 145.0 s

u Min: -5.8 m
u Max: 0.5 m
u Final: -0.0 m

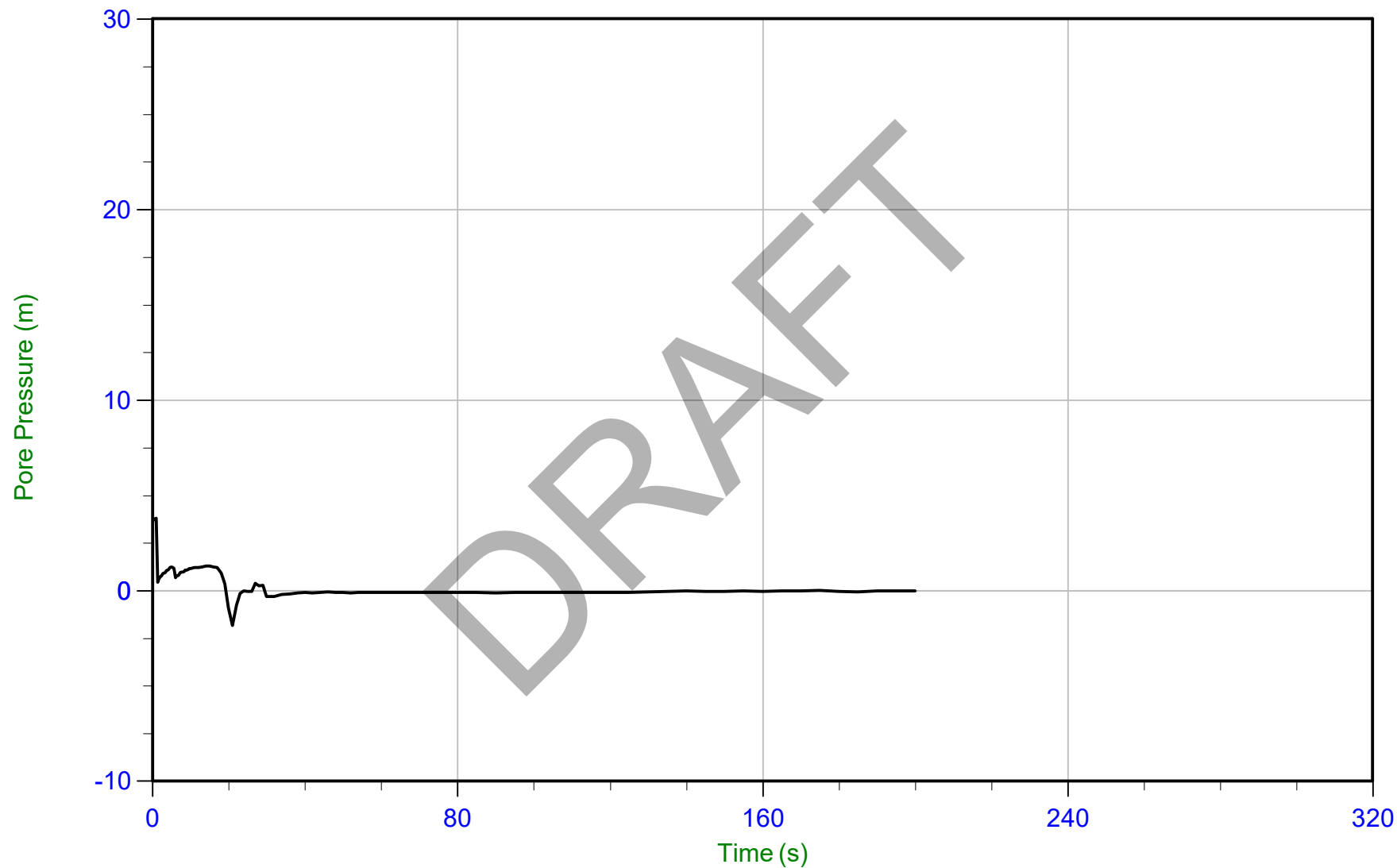
WT: 1.0 m / 3.2 ft
Ueq: 0.0 m



Stantec

Job No: 24-02-28291
Date: 2024-09-11 14:15
Site: 197A St 30 Ave, Langley

Sounding: RSCPT24-02
Cone: 898:T1500F15U35 Area=15 cm²



Trace Summary:

Filename: 24-02-28291_RS02.PPF2
Depth: 1.875 m / 6.152 ft
Duration: 200.0 s

u Min: -1.8 m
u Max: 3.8 m
u Final: -0.0 m

WT: 1.9 m / 6.2 ft
Ueq: 0.0 m

Description of Methods for Calculated CPT Geotechnical Parameters

DRAFT

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)

CONETEC

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: $q_t = q_c + (1-a) \cdot u_2$ (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 B_q parameter. The normalized Q_{tn} 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.

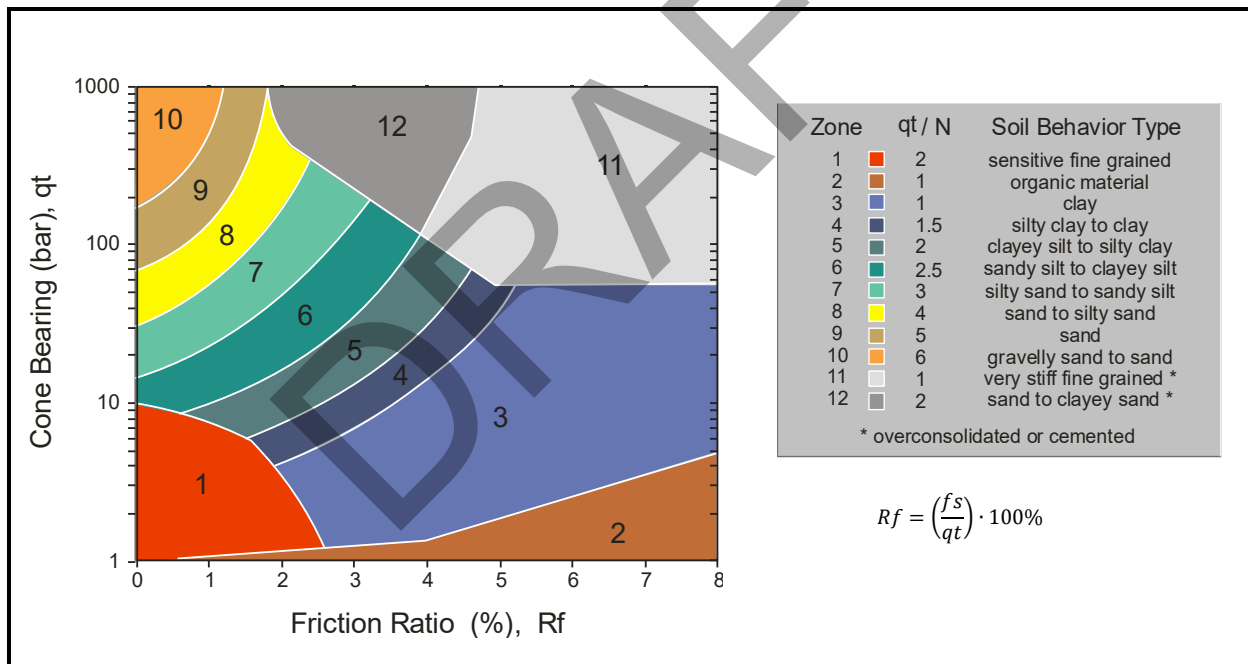


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

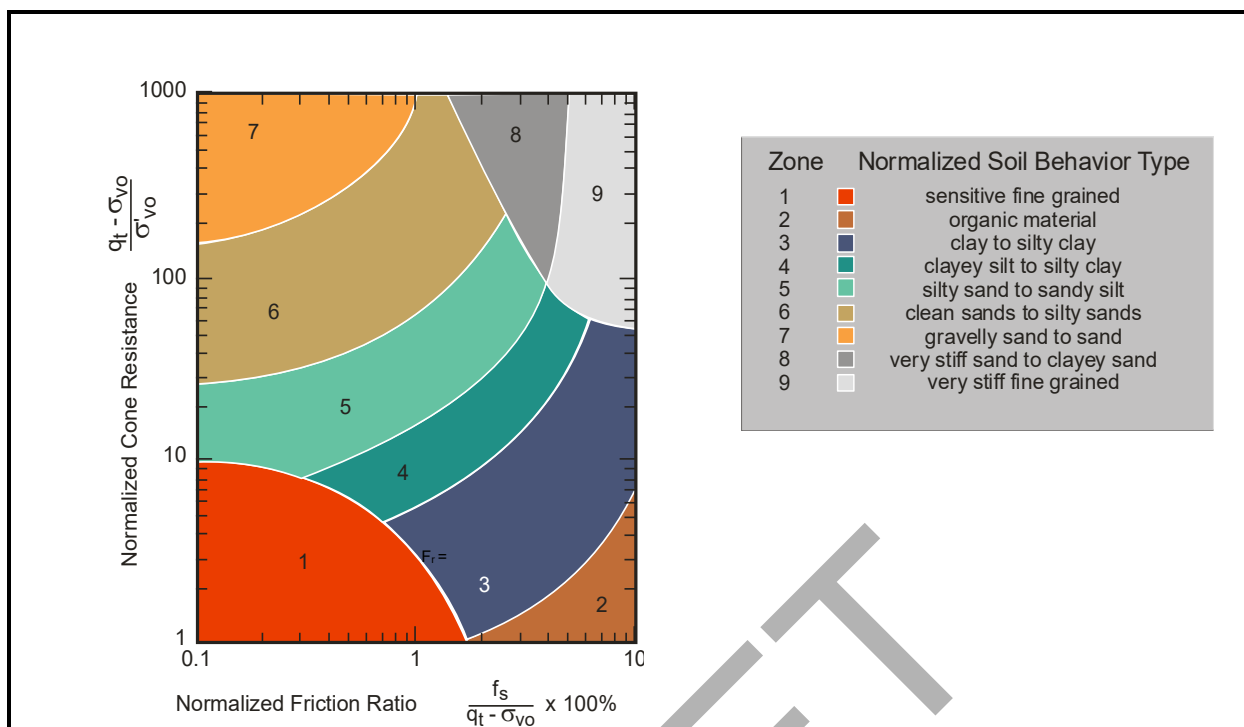
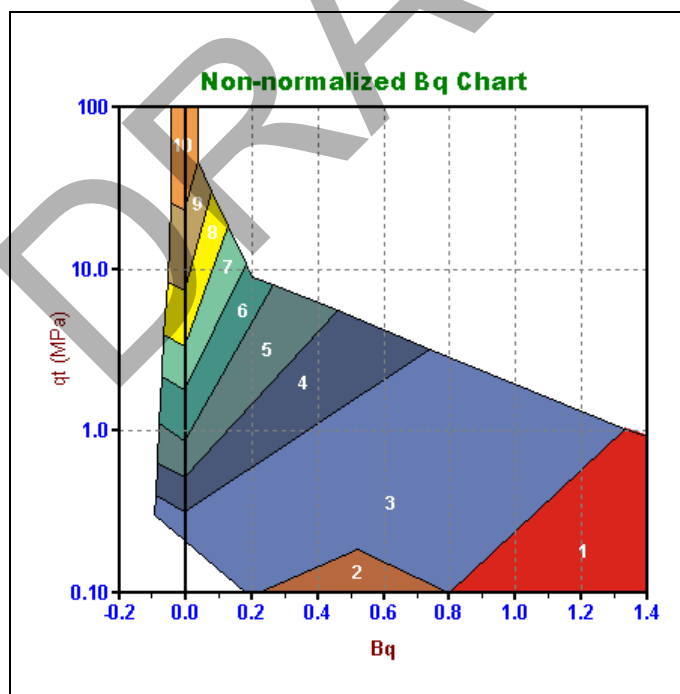
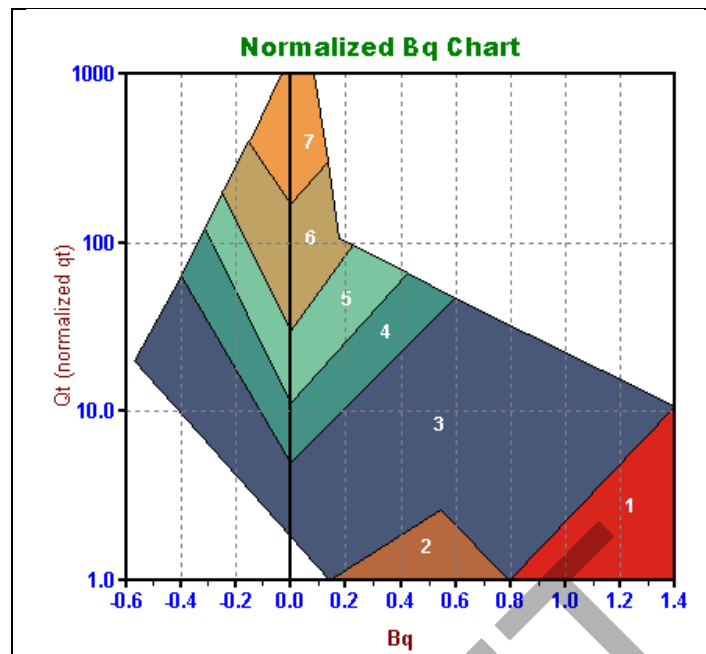
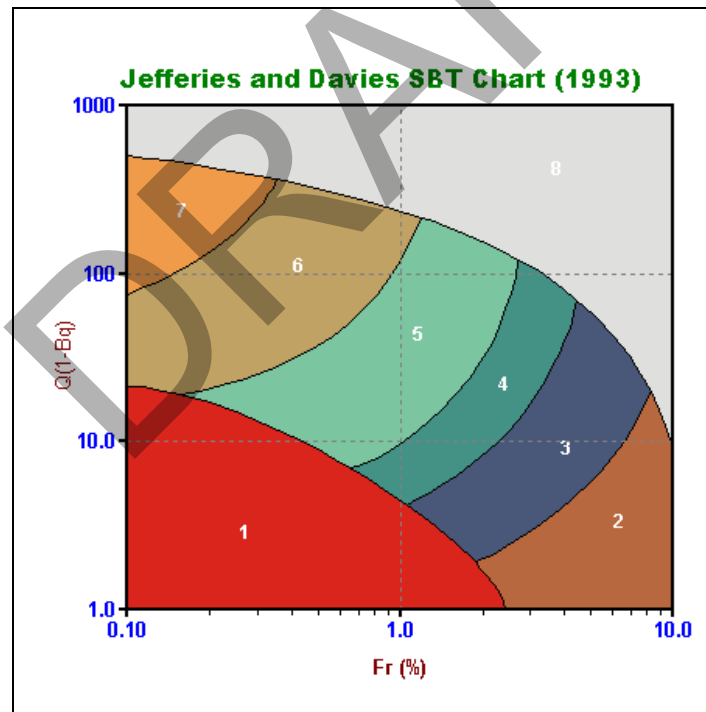


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

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

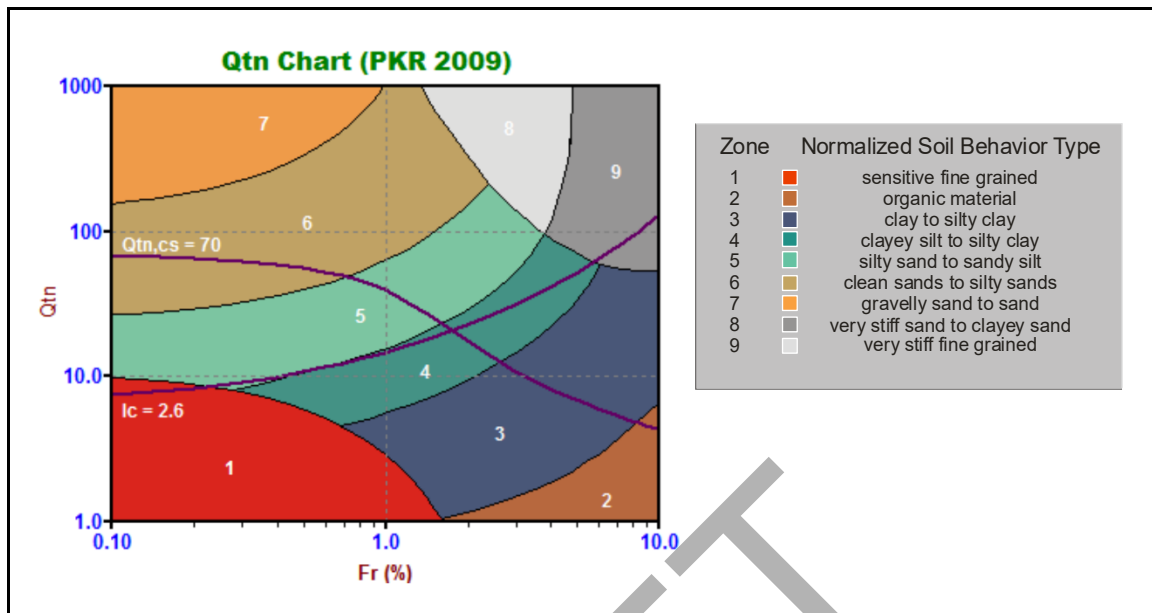
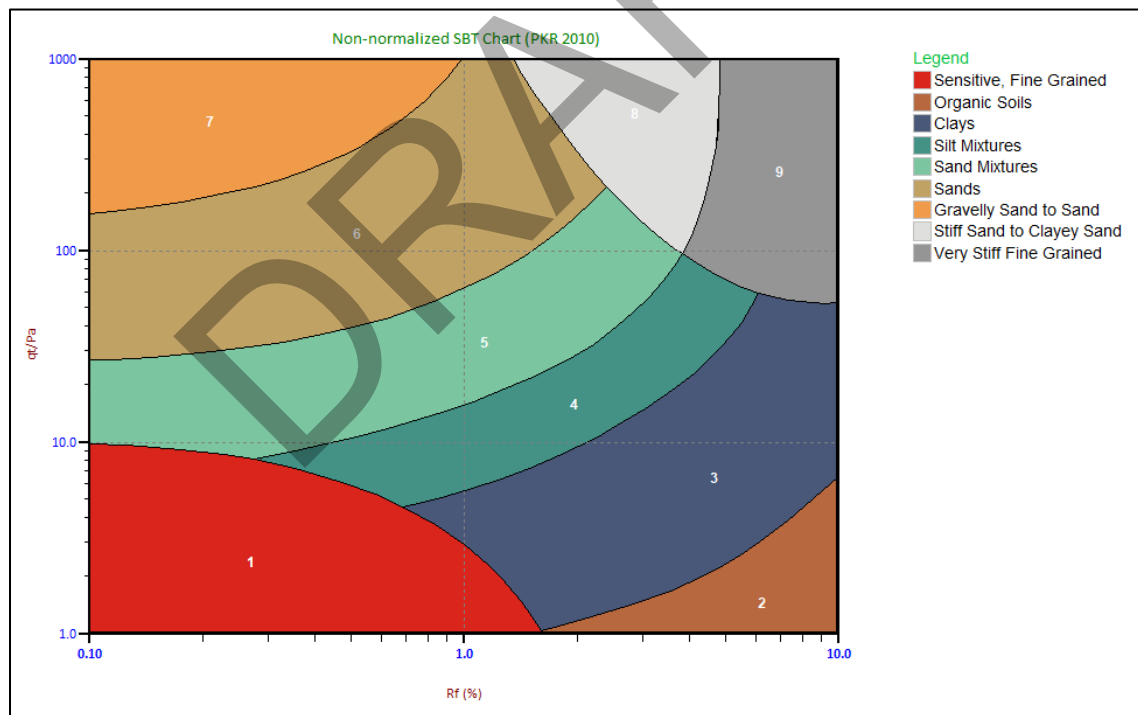
Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Q_{tn})

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

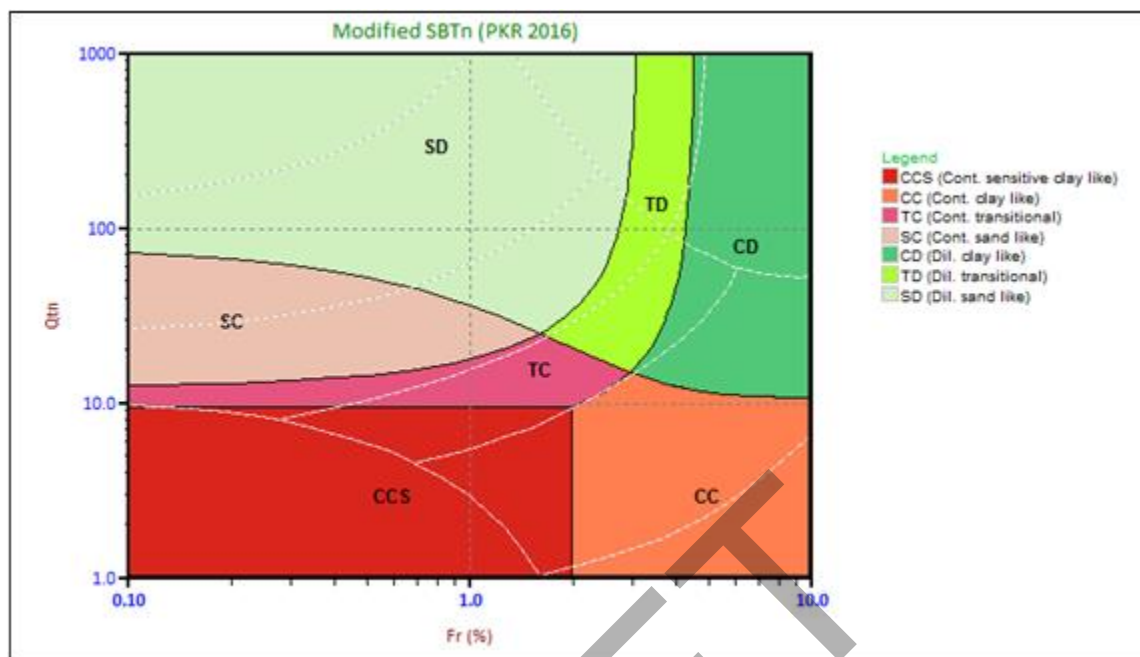


Figure 6. Modified SBTn Behavior Based Chart

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 1a and 1b 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 <i>(where calculations are done at each point then Mid Layer Depth = Recorded Depth)</i>	$[Depth (Layer Top) + Depth (Layer Bottom)] / 2.0$	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)	$Avgqc = \frac{1}{n} \sum_{i=1}^n q_c$ <i>n=1 when calculations are done at each point</i>	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1 - \alpha) \cdot u_2$ 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.	$Avgqt = \frac{1}{n} \sum_{i=1}^n q_t$ <i>n=1 when calculations are done at each point</i>	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s .	$Avgfs = \frac{1}{n} \sum_{i=1}^n f_s$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{f_s}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ <i>not an average of individual R_f values</i>	CK*
Avg u	Averaged dynamic pore pressure (u)	$Avgu = \frac{1}{n} \sum_{i=1}^n u_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n} \sum_{i=1}^n Resistivity_i$ <i>n=1 when calculations are done at each point</i>	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)	$AvgUVIF = \frac{1}{n} \sum_{i=1}^n UVIF_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n} \sum_{i=1}^n Temperature_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n} \sum_{i=1}^n Gamma_i$ <i>n=1 when calculations are done at each point</i>	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_{t1} , now referred to as Q_{t1})	See Figure 2	2, 5

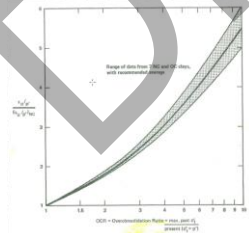
Calculated Parameter	Description	Equation	Ref
SBT-B _q	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B _q parameter	See Figure 3a	1, 2, 5
SBT-B _{qn}	Normalized Soil Behavior type based on normalized tip resistance (Q _t , now called Q _{t1}) and the B _q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Q _{tn}	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_{cs1n} 5) values assigned to SBT Q_{tn} 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 <i>A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.</i> 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 • • • Repeats for each layer Layer i Layer $i+1$ Final Layer	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 point 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) <i>Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.</i>	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.	$(N_1)_{60} = C_n \cdot N_{60}$	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).	$(q_t/P_a)/N_{60} = 8.5 (1 - I_c/4.6)$ $(q_t/P_a)/N_{60} = 10^{(1.1268 - 0.2817I_c)}$ 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) $(N_1)_{60}I_c = C_n \cdot (N_{60} I_c)$ 2) $q_{c1n}/(N_1)_{60}I_c = 8.5 (1 - I_c/4.6)$ 3) $(Q_{tn})/(N_1)_{60}I_c = 10^{(1.1268 - 0.2817I_c)}$	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.	$S_u = \frac{q_t - \sigma_v}{N_{kt}}$	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.	$S_u = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hoksund 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)	$= \frac{\Delta u}{q_t}$ 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	$B_q = \frac{\Delta u}{q_t - \sigma_v}$ where: $\Delta u = u - u_{eq}$ and u = dynamic pore pressure u_{eq} = equilibrium pore pressure	1, 2, 5
Net q_t or qt_{Net}	Net tip resistance (used in many subsequent correlations)	$q_t - \sigma_v$	36
q_e or qE or qE	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qe_{Norm}	Normalized effective tip resistance	$\frac{q_t - u_2}{\sigma_v}$	36
Q_t or Norm: Q_t 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.	$Q_t = \frac{q_t - \sigma_v}{\sigma_v}$	2, 5, 15
F_r or Norm: F_r	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$F_r = 100\% \cdot \frac{f_s}{q_t - \sigma_v}$	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.	$Q \cdot (1 - B_q)$ $Q \cdot (1 - B_q) + 1$ <i>where B_q is defined as above and Q is the same as the normalized tip resistance, Q_{t1}, defined above</i>	6, 7, 34
q_{c1}	Normalized tip resistance, q_{c1} , using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (P_a / \sigma_v')^{0.5}$ 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)	$q_{c1} (0.5) = (q_t / P_a) \cdot (P_a / \sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
$q_{c1} (C_n)$	Normalized tip resistance, q_{c1} , based on C_n (this method has stress units)	$q_{c1}(C_n) = C_n * q_t$	5, 12
$q_{c1} (C_q)$	Normalized tip resistance, q_{c1} , based on C_q (this method has stress units)	$q_{c1} (C_q) = C_q * q_t$ (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 I_c (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. I_c (RW1998) is different from that of Jefferies and Davies (7) and is different from I_c (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).	$I_c \text{ (PKR 2009)} = [(3.47 - \log_{10} Q_{tn})^2 + (1.22 + \log_{10} Fr)^2]^{0.5}$	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).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	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 (%)	$FC = 1.75(I_c^{3.25}) - 3.7$ $FC = 100 \text{ for } I_c > 3.5$ $FC = 0 \text{ for } I_c < 1.26$ $FC = 5\% \text{ if } 1.64 < I_c < 2.6 \text{ AND } Fr < 0.5$	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 Q_{tn} 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) 1 st order approximation using q_t Net (clays) 3) 1 st order approximation using Δu_2 (clays) 4) 1 st order approximation using q_e (clays) 5) Based on V_s	All stresses in kPa 1) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v) m' (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1 + (I_c / 2.65)^{2.5}}$ 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) $\sigma_p' = (V_s/4.59)^{1.47}$	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 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 Q_t	1) requires a user defined value for NC S_u/P_c' ratio 2 through 5) based on yield stresses 6) $YSR (V_s) = \sigma_p' (V_s) / \sigma_v'$ 7) $OCR = 0.25 \cdot (Q_t)^{1.25}$	9 19 20 20 20 18 32
E_s/q_t	Intermediate parameter for calculating Young's Modulus, E , in sands. It is the Y axis of the reference chart. Note that Figure 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 Es 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	$= S_u (N_{kt}) / \sigma_v'$	9, 23
Vs or Vs	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 Vp	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.	$V_{s30} = \frac{\text{total thickness of all layers to 30 m}}{\Sigma \left(\frac{\text{layer thickness}}{\text{layer shear wave velocity}} \right)}$ $V_{s30} = \frac{\text{total thickness of all layers to 30 m}}{\Sigma (\text{layer travel times})}$	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)	$= (q_t - \sigma_v) / G_{max}$ 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	$G_o = 0.0188 [10^{(0.55I_c + 1.68)}] (q_t - \sigma_v)$	15
Estimated E ₂₅	Estimated value for Young’s Modulus, E, at a 25% working load	$E_{25} = \alpha_E (qtNet)$ where $\alpha_E = 0.015 [10^{(0.55I_c + 1.68)}]$	15
k _{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I _c values.	For 1.0 < I _c ≤ 3.27: $k = 10^{(0.952 - 3.04I_c)} \quad \text{in m/s}$ For 3.27 < I _c < 4.0: $k = 10^{(-4.52 - 1.37I_c)} \quad \text{in m/s}$	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) $\alpha_M = Q_t \quad \text{when } Q_t < 14$ $\alpha_M = 14 \quad \text{when } Q_t > 14$ I _c < 2.2 (coarse grained) $\alpha_M = 0.0188 [10^{(0.55I_c + 1.68)}]$ D’ = α _D (q _t - σ _v) where α _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 $(N_1)_{60}$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	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) $(N_1)_{60cs} I_c = \alpha + \beta((N_1)_{60} I_c)$ 2) $(N_1)_{60cs} I_c = K_{SPT} \cdot ((N_1)_{60} I_c)$ 3) $(q_{c1ncs}) / ((N_1)_{60cs} I_c) = 8.5 (1 - I_c/4.6)$ FC $\leq$ 5%: $\alpha = 0, \beta = 1.0$ FC $\geq$ 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}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	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
$S_u(Liq)/ES_v$ 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
$S_u(Liq)/ES_v$ 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
K_g 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 $\Psi = -0.05$ curve	25
URS NP Fr	Normalized friction ratio point on $\Psi = -0.05$ curve used in SP distance calculation		25
URS NP Q_{tn}	Normalized tip resistance (Q_{tn}) point on $\Psi = -0.05$ curve used in SP Distance calculation		25

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Table 2. References

No.	Reference
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No.	Reference
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36	Mayne, P.W., Cargill, E. and Greig, J., 2023, "The Cone Penetration Test: A CPT Design Parameter Manual", ConeTec Group
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Appendix E Laboratory Test Results

DRAFT





Grain Size Analysis
ASTM D6913

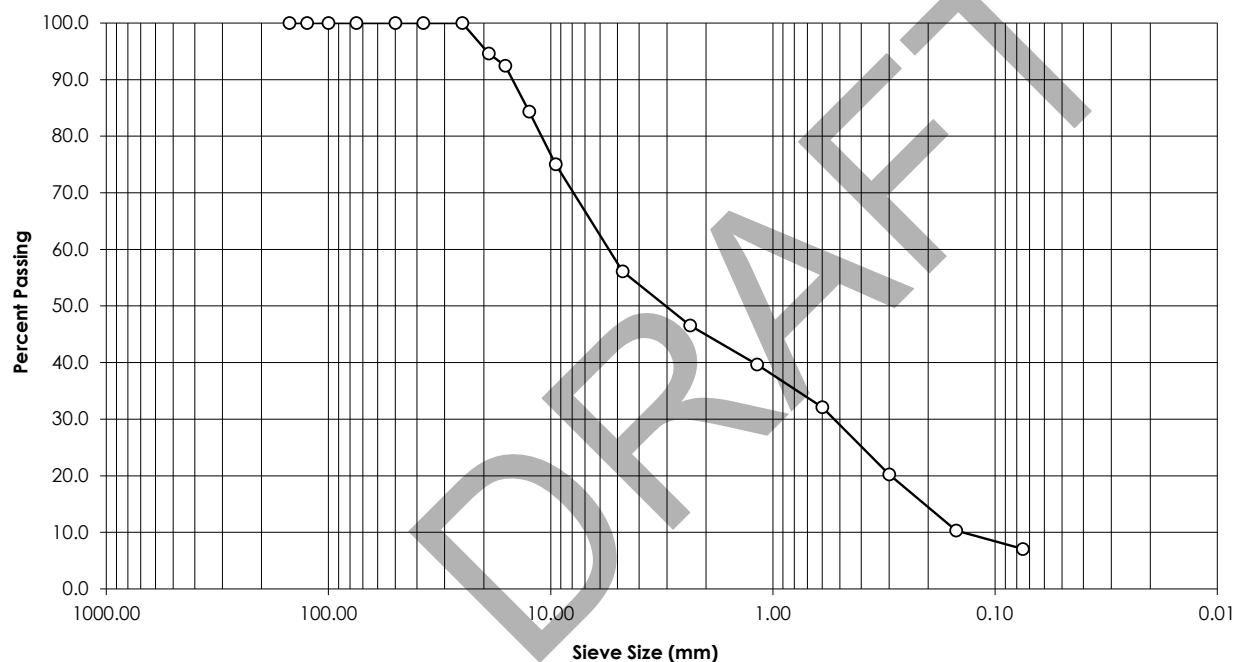
Client: BC Hydro
Project Name: Campbell Heights Site Assessment
-
Project No: 123317336.200.800

OFFICE
4515 Central Blvd.
Suite 500
Burnaby, BC
Canada V5H 0C6
Tel: (604) 436-3014

LABORATORY
3711 North Fraser Way
Suite 400
Burnaby, BC
Canada V5J 5J2
Tel: (604) 436-3014

SAMPLE No.: BH24-01, SS-2, 0.76m-1.37m
SOURCE: -
TESTED BY: PS, FG

DATE RECEIVED: September 13, 2024
DATE TESTED: September 27, 2024
SAMPLE DESCRIPTION: poorly graded SAND with silt and gravel, (SP-SM)



Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
75.0	100.0	-	-
50.0	100.0	-	-
37.5	100.0	-	-
25.0	100.0	-	-
19.0	94.6	-	-
16.0	92.5	-	-
12.5	84.3	-	-
9.5	75.1	-	-
4.75	56.1	-	-
2.36	46.5	-	-
1.18	39.6	-	-
0.600	32.1	-	-
0.300	20.2	-	-
0.150	10.3	-	-
0.075	7.1	-	-
Cobble: -		D ₁₀ :	0.14
Gravel: 43.9%		D ₃₀ :	0.56
Sand: 49.1%		D ₆₀ :	5.84
Fines: 7.0%		C _u :	40.61
		C _c :	0.37

Comments: The available minimum test sample size did not meet the requirement of ASTM D6913/D6913M - 17, section 10.2. Nominal Maximum Size Opening of 19.0mm and 25.4mm, require 1.3kg and 3kg minimum test sample size, respectively.

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. Stantec is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of Stantec.

Reviewed by:

Wil de Castro, ASCE



Stantec

**Grain Size
Analysis**

ASTM D6913

Client: BC Hydro

Project Name: Campbell Heights Site Assessment

Project No: 123317336.200.800

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Tel: (604) 436-3014

SAMPLE No.: BH24-01, SS-5, 5.33m-5.94m

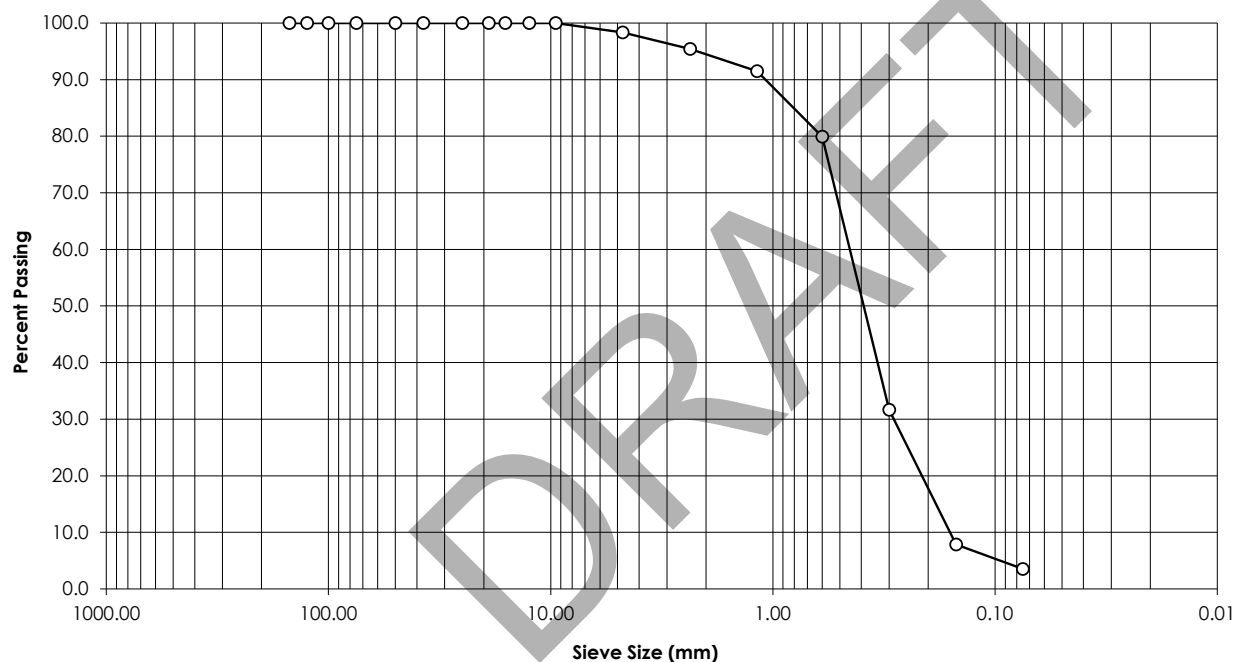
SOURCE: -

TESTED BY: PS, FG

DATE RECEIVED: September 13, 2024

DATE TESTED: September 27, 2024

SAMPLE DESCRIPTION: poorly graded SAND, (SP)



Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
75.0	100.0	-	-
50.0	100.0	-	-
37.5	100.0	-	-
25.0	100.0	-	-
19.0	100.0	-	-
16.0	100.0	-	-
12.5	100.0	-	-
9.5	100.0	-	-
4.75	98.3	-	-
2.36	95.4	-	-
1.18	91.5	-	-
0.600	79.9	-	-
0.300	31.7	-	-
0.150	7.9	-	-
0.075	3.5	-	-
Cobble: -		D ₁₀ :	0.18
Gravel: 1.7%		D ₃₀ :	0.29
Sand: 94.8%		D ₆₀ :	0.51
Fines: 3.5%		C _u :	2.88
		C _c :	0.97

Comments:

Reviewed by:

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Grain Size Analysis
ASTM D6913

Client: BC Hydro
Project Name: Campbell Heights Site Assessment
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Project No: 123317336.200.800

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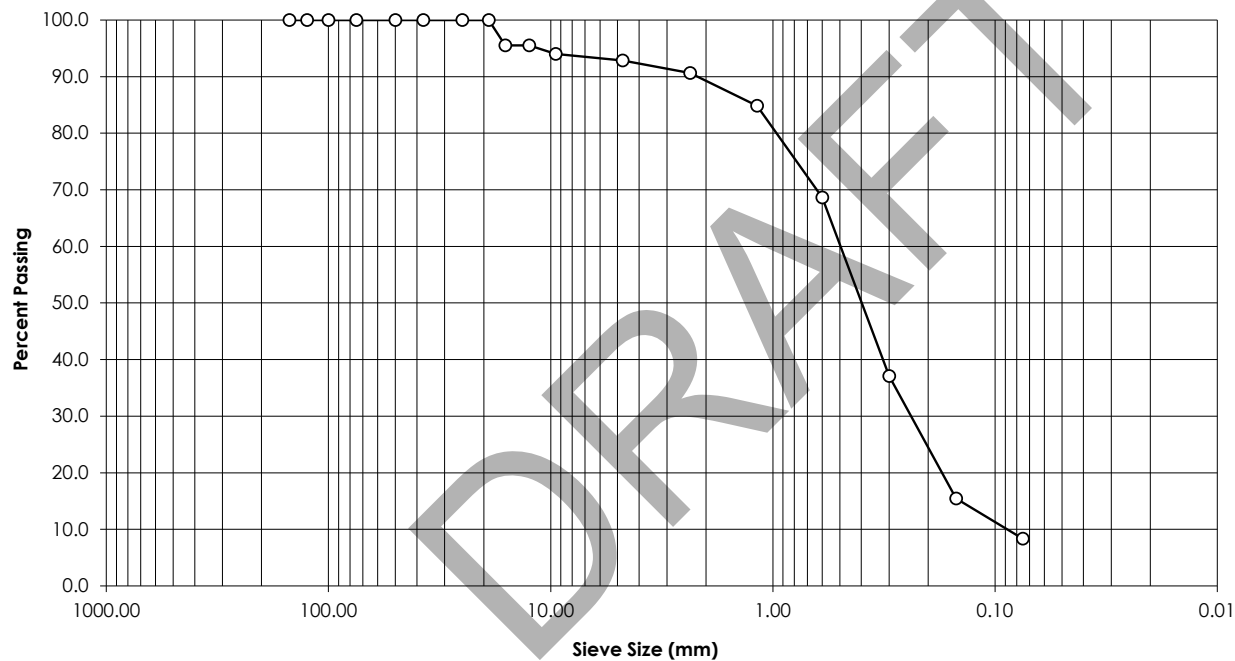
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SAMPLE No.: BH24-01, SS-8, 9.91m-10.52m
SOURCE: -
TESTED BY: PS, FG

DATE RECEIVED: September 13, 2024
DATE TESTED: September 27, 2024
SAMPLE DESCRIPTION: poorly graded SAND with silt, (SP-SM)



Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
75.0	100.0	-	-
50.0	100.0	-	-
37.5	100.0	-	-
25.0	100.0	-	-
19.0	100.0	-	-
16.0	95.5	-	-
12.5	95.5	-	-
9.5	94.0	-	-
4.75	92.8	-	-
2.36	90.6	-	-
1.18	84.8	-	-
0.600	68.7	-	-
0.300	37.1	-	-
0.150	15.4	-	-
0.075	8.3	-	-
Cobble: -		D ₁₀ :	0.10
Gravel: 7.2%		D ₃₀ :	0.26
Sand: 84.5%		D ₆₀ :	0.53
Fines: 8.3%		C _u :	5.50
		C _c :	1.34

Comments:

Reviewed by:

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**Stantec****Grain Size
Analysis**

ASTM D6913

Client: BC Hydro

Project Name: Campbell Heights Site Assessment

Project No: 123317336.200.800

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LABORATORY

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Canada V5J 5J2

Tel: (604) 436-3014

SAMPLE No.: BH24-01, SS-10, 12.95m-13.56m

SOURCE: -

TESTED BY: PS, FG

DATE RECEIVED: September 13, 2024

DATE TESTED: September 27, 2024

SAMPLE DESCRIPTION: poorly graded SAND with silt, (SP-SM)



Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
75.0	100.0	-	-
50.0	100.0	-	-
37.5	100.0	-	-
25.0	100.0	-	-
19.0	100.0	-	-
16.0	100.0	-	-
12.5	100.0	-	-
9.5	100.0	-	-
4.75	99.5	-	-
2.36	98.4	-	-
1.18	93.0	-	-
0.600	79.2	-	-
0.300	37.9	-	-
0.150	11.7	-	-
0.075	5.1	-	-
Cobble: -		D ₁₀ :	0.14
Gravel: 0.5%		D ₃₀ :	0.27
Sand: 94.4%		D ₆₀ :	0.49
Fines: 5.1%		C _u :	3.60
		C _c :	1.11

Comments:

Reviewed by:

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Wil de Castro
 Wil de Castro, ASCE



Grain Size Analysis
ASTM D6913

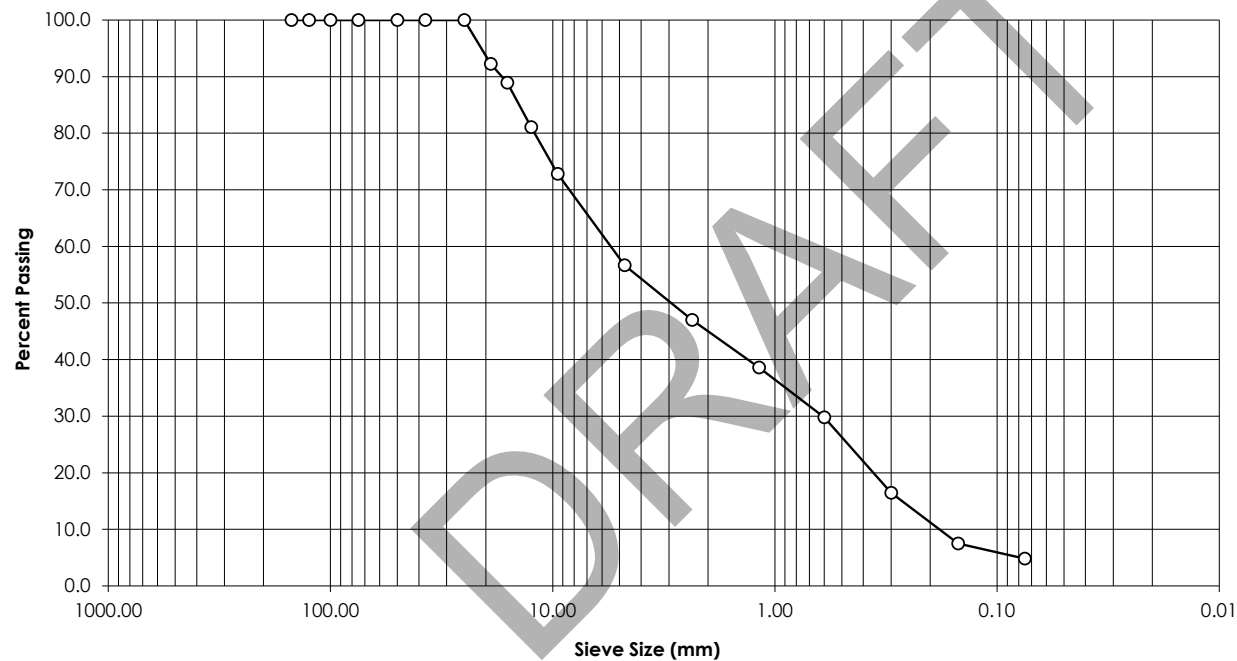
Client: BC Hydro
Project Name: Campbell Heights Site Assessment
-
Project No: 123317336.200.800

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SAMPLE No.: BH24-02, SS-3, 2.29m-2.90m
SOURCE: -
TESTED BY: PS, FG

DATE RECEIVED: September 13, 2024
DATE TESTED: September 27, 2024
SAMPLE DESCRIPTION: poorly graded SAND with gravel, (SP)



Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
75.0	100.0	-	-
50.0	100.0	-	-
37.5	100.0	-	-
25.0	100.0	-	-
19.0	92.2	-	-
16.0	88.9	-	-
12.5	81.1	-	-
9.5	72.8	-	-
4.75	56.6	-	-
2.36	47.0	-	-
1.18	38.6	-	-
0.600	29.8	-	-
0.300	16.5	-	-
0.150	7.5	-	-
0.075	4.8	-	-
Cobble: -		D ₁₀ :	0.21
Gravel: 43.4%		D ₃₀ :	0.62
Sand: 51.8%		D ₆₀ :	5.84
Fines: 4.8%		C _u :	28.46
		C _c :	0.32

Comments: The available minimum test sample size did not meet the requirement of ASTM D6913/D6913M - 17, section 10.2. Nominal Maximum Size Opening of 19.0mm and 25.4mm, require 1.3kg and 3kg minimum test sample size, respectively.

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. Stantec is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of Stantec.

Reviewed by:


Wil de Castro, ASCT



Grain Size Analysis
ASTM D6913

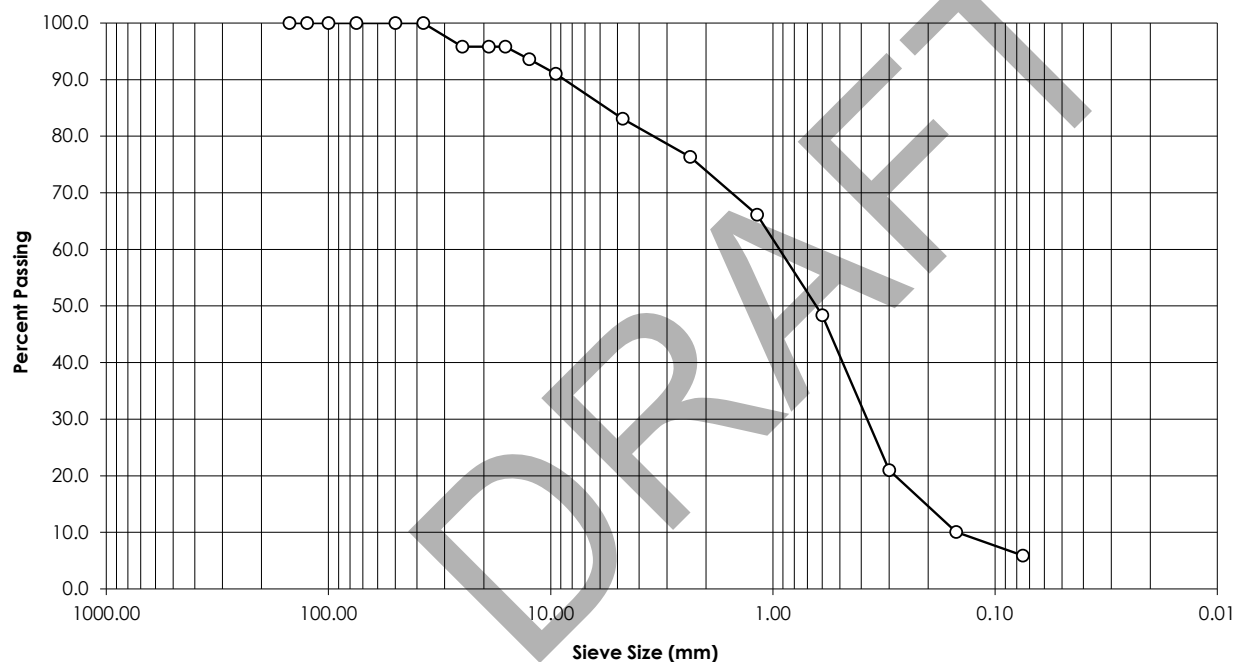
Client: BC Hydro
Project Name: Campbell Heights Site Assessment
-
Project No: 123317336.200.800

OFFICE
4515 Central Blvd.
Suite 500
Burnaby, BC
Canada V5H 0C6
Tel: (604) 436-3014

LABORATORY
3711 North Fraser Way
Suite 400
Burnaby, BC
Canada V5J 5J2
Tel: (604) 436-3014

SAMPLE No.: BH24-02, SS-5, 5.33m-5.94m
SOURCE: -
TESTED BY: PS

DATE RECEIVED: September 13, 2024
DATE TESTED: September 27, 2024
SAMPLE DESCRIPTION: well graded SAND with silt and gravel, (SW-SM)



Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
75.0	100.0	-	-
50.0	100.0	-	-
37.5	100.0	-	-
25.0	95.8	-	-
19.0	95.8	-	-
16.0	95.8	-	-
12.5	93.6	-	-
9.5	91.1	-	-
4.75	83.1	-	-
2.36	76.3	-	-
1.18	66.1	-	-
0.600	48.3	-	-
0.300	21.0	-	-
0.150	10.1	-	-
0.075	5.9	-	-
Cobble: -		D ₁₀ :	0.15
Gravel: 16.9%		D ₃₀ :	0.43
Sand: 77.2%		D ₆₀ :	1.00
Fines: 5.9%		C _u :	6.72
		C _c :	1.23

Comments: The available minimum test sample size did not meet the requirement of ASTM D6913/D6913M - 17, section 10.2. Nominal Maximum Size Opening of 19.0mm and 25.4mm, require 1.3kg and 3kg minimum test sample size, respectively.

Reviewed by:

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Wil de Castro, ASCE



Stantec Grain Size
Analysis
ASTM D6913

Client: BC Hydro
Project Name: Campbell Heights Site Assessment
-
Project No: 123317336.200.800

OFFICE

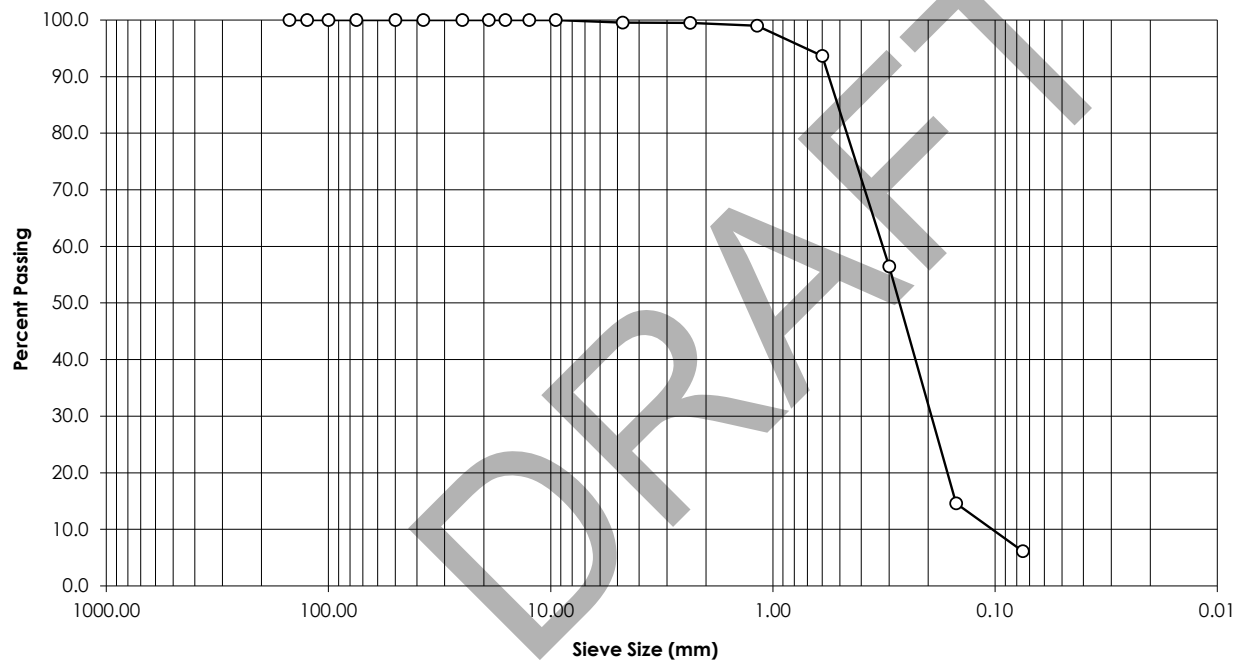
4515 Central Blvd.
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LABORATORY

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Suite 400
Burnaby, BC
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Tel: (604) 436-3014

SAMPLE No.: BH24-02, SS-8, 9.91m-10.52m
SOURCE: -
TESTED BY: PS

DATE RECEIVED: September 13, 2024
DATE TESTED: September 27, 2024
SAMPLE DESCRIPTION: poorly graded SAND with silt, (SP-SM)



Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
75.0	100.0	-	-
50.0	100.0	-	-
37.5	100.0	-	-
25.0	100.0	-	-
19.0	100.0	-	-
16.0	100.0	-	-
12.5	100.0	-	-
9.5	100.0	-	-
4.75	99.6	-	-
2.36	99.5	-	-
1.18	99.0	-	-
0.600	93.7	-	-
0.300	56.4	-	-
0.150	14.6	-	-
0.075	6.1	-	-
Cobble: -		D ₁₀ :	0.12
Gravel: 0.4%		D ₃₀ :	0.23
Sand: 93.5%		D ₆₀ :	0.34
Fines: 6.1%		C _u :	2.86
		C _c :	1.34

Comments:

Reviewed by:

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. Stantec is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of Stantec.

Wil de Castro, ASCE



BUREAU
VERITAS

Bureau Veritas Job #: C476862

Report Date: 2024/10/03

STANTEC CONSULTING LTD

Client Project #: 123317336

Site Location: -

Your P.O. #: 123317336

Sampler Initials: WDC

RESULTS OF CHEMICAL ANALYSES OF SOIL

Bureau Veritas ID		CWM337		CWM338	CWM339		
Sampling Date		2024/09/10		2024/09/10	2024/09/10		
COC Number		92321		92321	92321		
	UNITS	BH24-01, SS-3, 2.29m - 2.90m	RDL	BH24-01, SS-6, 6.86m - 7.47m	BH24-01, SS-10, 12.95m - 13.56m	RDL	QC Batch

ANIONS

Soluble Sulphate (SO4)	mg/L	<20	20	<20	<20	20	B547337
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Calculated Parameters

Soluble Sulphate (SO4)	mg/kg	<6.2	6.2	<6.0	<6.0	6.0	B540124
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Soluble Parameters

Soluble Conductivity	uS/cm	106	5.0	70.2	115	5.0	B547806
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Soluble pH	pH	5.91	N/A	6.51	7.03	N/A	B546760
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Saturation %	%	30.9	N/A	29.9	29.9	N/A	B546723
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RDL = Reportable Detection Limit

N/A = Not Applicable

Bureau Veritas ID		CWM340		CWM341		CWM342		
Sampling Date		2024/09/10		2024/09/10		2024/09/10		
COC Number		92321		92321		92321		
	UNITS	BH24-02, SS-2, 0.76m - 1.37m	RDL	BH24-02, SS-6, 6.86m - 7.47m	RDL	BH24-02, SS-9, 11.43m - 12.04m	RDL	QC Batch

ANIONS

Soluble Sulphate (SO4)	mg/L	<20	20	<20	20	<20	20	B547337
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Calculated Parameters

Soluble Sulphate (SO4)	mg/kg	<6.9	6.9	<6.0	6.0	<6.1	6.1	B540124
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Soluble Parameters

Soluble Conductivity	uS/cm	56.7	5.0	84.9	5.0	73.3	5.0	B547806
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Soluble pH	pH	6.26	N/A	7.14	N/A	7.71	N/A	B546760
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Saturation %	%	34.6	N/A	29.9	N/A	30.6	N/A	B546723
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RDL = Reportable Detection Limit

N/A = Not Applicable



BUREAU
VERITAS

Bureau Veritas Job #: C476862

Report Date: 2024/10/03

STANTEC CONSULTING LTD

Client Project #: 123317336

Site Location: -

Your P.O. #: 123317336

Sampler Initials: WDC

RESULTS OF CHEMICAL ANALYSES OF SOIL

Bureau Veritas ID		CWM342		
Sampling Date		2024/09/10		
COC Number		92321		
	UNITS	BH24-02, SS-9, 11.43m - 12.04m Lab-Dup	RDL	QC Batch

ANIONS				
Soluble Sulphate (SO4)	mg/L	<20	20	B547337
Soluble Parameters				
Soluble Conductivity	uS/cm	73.9	5.0	B547806
Soluble pH	pH	7.72	N/A	B546760
Saturation %	%	30.6	N/A	B546723

RDL = Reportable Detection Limit

Lab-Dup = Laboratory Initiated Duplicate

N/A = Not Applicable