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Geotechnical test results at a prehistoric landslide site in Breckenridge, southwestern Quebec

Baolin Wang

ABSTRACT

Geotechnical field testing was conducted in and around a landslide occurred about 1020 cal BP in Breckenridge, southwestern Quebec. The objective of the study is to investigate the magnitude of an earthquake previously hypothesized to have triggered this and other landslides in the region. The purpose of this Open File is to document the data obtained from the Breckenridge landslide site. The landslide is about 980 m long and 370 m wide. Four cone penetration tests (CPT) were conducted to depths ranging from 37 m to 55 m. Two CPTs were inside the landslide scar and two outside. Field vane shear tests (VST) were carried out at three CPT locations. The CPT cone bearing factor N_{kt} was calibrated to be 11.5. The soil undrained shear strength (S_u) was found to have a correlation of $S_u = 15 + 2.66$ H (kPa), where H is depth in meters from the original ground surface elevation at the head scarp. All the CPT data consistently show the same trend for the undisturbed materials. The slip surface was found to be at about 15 m depth from the original ground surface.

1. INTRODUCTION

A previous study identified a cluster of 29 prehistoric glacialmarine landslides within a 4 km long corridor along the Breckenridge creek and its tributaries in Breckenridge, southwestern Quebec (Brooks et al., 2013). Thirteen of the landslides were dated to have ages of 180 to 7000 cal BP. Four of them occurred about 1020 cal BP. Five landslides at other locations within a 65 km range in the region were also identified to have an age of 1020 cal BP. It was hypothesized that those 1020 cal BP landslides were triggered by an earthquake and the magnitude was estimated to be at least 6.1 based on a magnitude to affected area correlation (Brooks, 2013). In order to further understand the landslide triggering mechanism and the earthquake magnitude, a geotechnical study was initiated at the Geological Survey of Canada (GSC) in 2014. Representative landslides were selected to investigate the 1020 cal BP event. One of the landslides in the Breckenridge valley was selected for this study in 2015. Cone penetration tests (CPT) and field vane shear tests (VST) were conducted in and around the landslide scar. This Open File documents detailed data from the test program.

2. SITE DESCRIPTION

The landslide site is located in Breckenridge of the Pontiac municipality, Quebec about 20 km northwest of Ottawa. Fig. 1 shows the general location of the site and the detailed LiDAR image of the landslide. The landslide area is within Champlain Sea sediments that were deposited between 13.9 and 11.5 ka cal BP (Dyke and Prest, 1987). Locally, the deposits are composed of 3 to 4 m of sand capping glaciomarine clay and silty clay of varying thickness that overlies glacial sediments or bedrock (Gadd, 1986). The Champlain Sea deposits became incised by the postglacial stream network in the early Holocene as the Champlain Sea receded because of regional postglacial uplift (Gadd, 1987).

Breckenridge creek discharges to the Ottawa River to the southwest. Landslide #1 (Fig. 1) is the study site that failed about 1020 cal BP (Brooks et al., 2013). It is near the confluence of a tertiary creek and the Breckenridge Creek. The landslide is about 980 m long and 370 m wide. The lateral extent of the landslide is constrained by four older landslides: one to the east (Landslide #2) aged 3000 cal BP (Brooks et al., 2013), and three older landslides of unknown age to the west. The three older landslides are along a scarp above an erosional fluvial terrace of the ancestral Ottawa River and have been eroded.

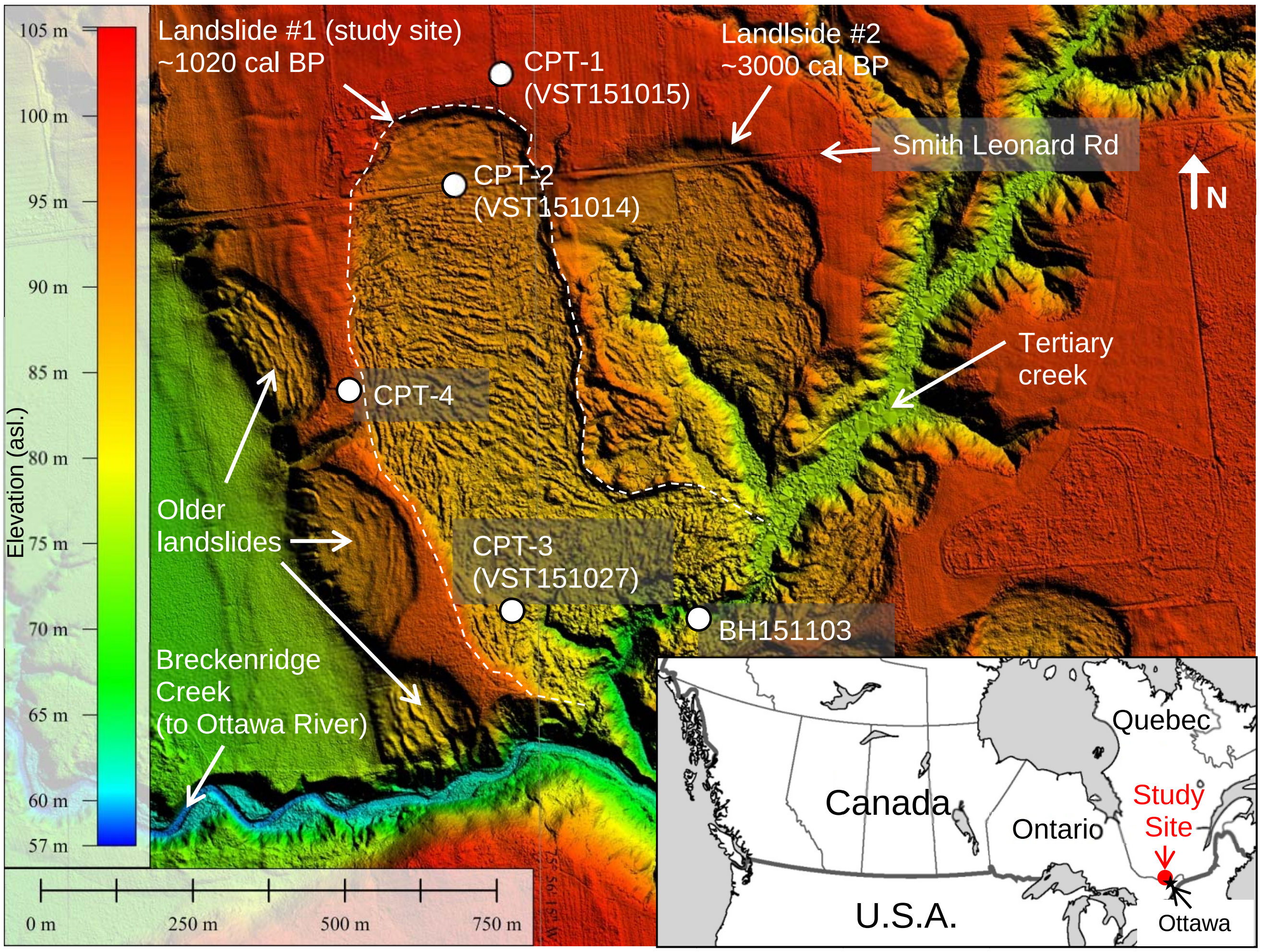

Fig. 1. Location map of study area based on LiDAR image (LiDAR image © Government of Quebec)

The ground surface within the scar of Landslide 1 dropped by about 8 to 15 m from its original elevation. The terrain inside the landslide scar is mostly hummocky and covered by dense trees and vegetation. The lands to the north of Smith Leonard Road and to the west of Landslide 1 are hayfield or pasture. Most of the scar zone of Landslide #2 is densely wooded. The woodlands in the general area have been designated by the Nature Conservancy of Canada as protected conservation area.

The creek downstream of Landslide #1 is about 15 m below the landslide depression or about 30 m below the undisturbed ground surface. The creek can be jumped across by foot at most places. Massive undisturbed clay is visible along the toe of the creek banks. The clay is often exposed along the bottom of the creek that provides convenient access for undisturbed soil sampling.

3. METHODS

3.1 Cone Penetration Tests

Cone penetration tests (CPT) were conducted at four locations on July 21, 2015 by ConeTec Investigations Ltd. and supervised by GSC personnel. The CPT locations are shown in Fig. 1. The location coordinates and tests conducted are listed in Table 1. CPT-1 and CPT-4 are in undisturbed areas outside the landslide scar and CPT-2 and CPT-3 are inside the scar. The CPT program was carried out to determine the in situ strength parameters of the soils.

Table 1. Drill hole locations and tests conducted

Site #	Location	Depth (m)	CPT	VST	Note
CPT-1	N45º29’27.3 W75º56’17.6	55.4	✓	✓	Outside landslide scar. VST151015 to 19 m depth (6m east of CPT-1).
CPT-2	N45º29’21.3 W75º56’20.8	53.1	✓	✓	Inside landslide scar. VST151014 to 19 m depth (5 m south of CPT-2).
CPT-3	N45º28’59.1 W75º56’16.7	36.7	✓	✓	Inside landslide scar. VST to 19 m depth (5m east of CPT-3).
CPT-4	N45º29’10.7 W75º56’29.7	53.3	✓		Outside landslide scar.

The CPT was conducted with a 20 ton track mounted rig. The rig was equipped with an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada. The penetrometer is compression type design in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocone uses strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The cone also has a platinum resistive temperature device for monitoring the temperature of the sensors, and an accelerometer type dual axis inclinometer. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

The cone had a tip cross-sectional area (A_c) of 15 cm^2, a friction sleeve area (A_s) of 225 cm^2, a maximum tip capacity of 150 MPa, a sleeve capacity of 1.5 MPa, and a pore pressure transducer capacity of 3.4 MPa (500 psi). The cone has an equal end area friction sleeve, a net end area ratio (a) of 0.8 and a tip of 60 degree apex angle.

A 6 mm thick pore pressure filter was used directly behind the cone tip. The filter, which is composed of porous plastic polyethylene, enables the cone penetrometer to measure dynamic pore pressures during penetration, and record pore pressure dissipations at selected depths. The filter has 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 cone and rods were pushed into the ground using a hydraulic ramset located inside the rig, at a steady rate of 2 cm/s. The CPT soundings were completed in accordance with ASTM D5778. The cone system used during the tests recorded the tip resistance and sleeve friction in kPa and the pore pressure in meters. The data acquisition frequency was every 10 cm.

The interpretation of the CPT data is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u_2). The interpretation of Soil Behaviour Type (SBT) is based on the correlations developed by Robertson (1990) and Robertson (2009).

The recorded tip resistance (q_c) and the recorded dynamic pore pressure behind the tip (u_2) were used for calculating q_t as follows:

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

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 (at shoulder of the cone)
a is the Net Area Ratio for the piezocone (0.8 for the probe used)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As the piezocone used in these tests has an equal end area friction sleeve, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u_2) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration was stopped to allow the dynamic pore pressures to dissipate. The variation of the pore pressure with time was measured and recorded. All pore pressure data was recorded immediately behind the cone tip (shoulder 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. The friction ratio gives an indication of the grain size characteristics of the material. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressure.

3.2 Vane Shear Tests

Vane shear tests (VST) were carried out at VST151014, VST151015 and VST151027 on the 14th, 15th and 27th of October 2015 respectively. The locations are shown in Fig. 1 and listed in Table 1. VST151014 is 6 m to the east of CPT-2, VST151015 is 5 m to the south of CPT-1 and VST151027 5 m to the east of CPT-3. The purpose of the VST is to measure the in-situ undrained shear strength of the soil and to calibrate the CPT data.

The vane shear tests were carried out with a portable heavy-duty M-1000 boring rig from RocTest Ltd. The vane borer consists of a torque recording head, boring rods of 20 mm diameter, various sized vanes and a slip coupling. The torque head is both a loading and a recording instrument. The torque record is scribed on a waxed paper disc mounted inside the torque head. The slip coupling installed between the vane and the rods allows a free slip of approximately 15^{o} before the vane is engaged. The torque recorded during the free slip reflects the rod friction that can be subtracted from the subsequent torque record for net vane shear resistance.

The tests were conducted from 6 m to 19 m depth at 1 m intervals for all three locations. The test holes were predrilled with a hand auger through the sand cap on the surface that is typically around a meter thick. The vane was then pushed down with the boring rig directly to the desired depth for shear test. The maximum depth of test was limited by the rods available. The vane shear tests were carried out in accordance with ASTM D2573.

The geometries of the vane used are shown in Fig. 2. The vane size was selected based on the capacity of the recording head and the anticipated shear resistance estimated from the CPT sounding at each site. A 65x130 mm vane was used for both VST151014 and VST151015. A smaller 50x110 mm vane was used for VST151027 due to anticipated higher shear strength. The soil undrained shear strength was calculated from the recorded torque with the factory calibrated conversion factor for the recording head and the coefficient for each vane.

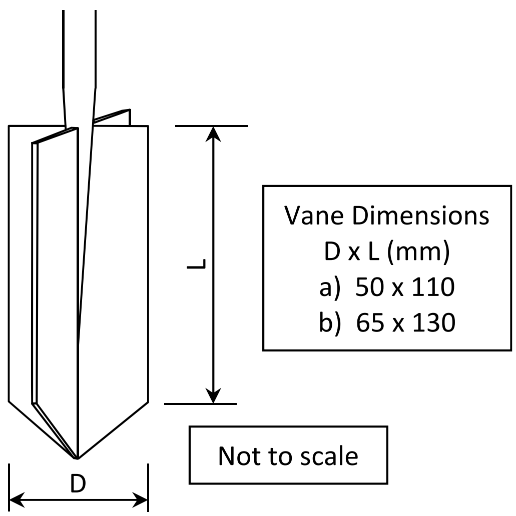

Fig. 2 Geometry of field vane used for soil undrained shear strength test

3.3 CPT Data Calibration

The VST results are used to calibrate the CPT data. The soil undrained shear strength (S_u) is conventionally calculated from the CPT tip resistance as follows:

$$S_u = (q_t - \sigma_{vo}) / N_{kt} \qquad (2)$$

where q_t is the total corrected cone tip resistance as calculated using Eq. 1;
σ_{vo} is the total vertical overburden stress;
N_{kt} is a bearing factor (Konrad and Law, 1987; Yu and Mitchell, 1998).

The S_u value can also be independently calculated entirely from the excess pore water pressure measurements (Δu) as follows:

$$S_u = \Delta u / N_{\Delta u} \qquad (3)$$

where Δu is commonly determined from $\Delta u = (u_2 - u_{eq})$ with u_2 being the recorded dynamic pore pressure behind the tip of the cone (shoulder element); u_{eq} is the equilibrium pore pressure determined from water table depth; and

$N_{\Delta u}$ is a pore pressure bearing factor (Tavenas and Leroueil, 1987).

Note that N_{kt} and $N_{\Delta u}$ are linked as:

$$N_{\Delta u} = B_q N_{kt} \qquad (4)$$

where B_q is a pore pressure parameter determined as follows:

$$B_q = (u_2 - u_{eq})/(q_t - \sigma_{vo}) \qquad (5)$$

With trial-and-error, a bearing factor N_{kt} is determined that brings the S_u from the CPT (Eq. 2) close to that from the VST. The pore pressure bearing factor $N_{\Delta u}$ can subsequently be calculated using Eq. 4 and S_u determined purely with pore water pressure data using Eq. 3.

3.4 Soil Sampling and Laboratory Testing

Undisturbed clay samples were collected with seamless thin-walled tubes at the toe of the landslide along the creek. The purpose was to determine the geotechnical index properties of the clay. The samples were taken on November 3, 2015. The sampling location is marked as BH151103 on Fig. 1. It is believed from the large area of exposed clay and its horizontal stratification that the creek has been incising through intact materials that have unlikely been affected by the landslide. The top 0.5 m materials were removed with hand shovel right beside the creek water. The creek was about 30 cm deep. An aluminum tube was pushed down by hand to collect core samples from the bottom of the pit. The diameters of the tubes are 7.6 cm (3 inch) and 5.1 cm (2 inch). The wall of the tube is 1.6 mm (1/16 inch) thick. The tubes are 115 cm long each. Two clay cores were taken: one 7.6 cm diameter and one 5.1 cm diameter. Both cores were taken from the same elevation at locations 1 m away from each other. The cores were sealed with plastic caps and electrical tape and shipped in vertical position to GSC's Sedimentology Laboratory. The cores were stored in a refrigerator until extruded for testing.

The 7.6 cm diameter core was extruded and cut into specimens of 5 cm long each on November 10, 2015. Each specimen was then put in a tin container of 8 cm in diameter and 5 cm high. The containers were sealed with electrical tape and refrigerated until testing. The other smaller sized core was preserved in fridge for future use.

Soil moisture contents, Atterberg limits, unit weight, specific gravity and gradations of the extruded specimens were determined in accordance with the respective ASTM standards by the GSC's Sedimentology Laboratory.

4. RESULTS

4.1 Soil Geotechnical Index Properties

The laboratory test results of the geotechnical index properties of the soil samples are presented in Table 2 and Figs. 3 and 4, which include moisture content, Atterberg limits, unit weight, specific gravity, and gradation. The data indicate that the materials are silty clay with high to very high plasticity.

Table 2. Geotechnical index properties of soil samples from BH151103

Depth (m)	Elevation asl. (m)	W_c (%)	PL (%)	LL (%)	I_p (%)	γ (kN/m^3)	G_s
0.5	71.3	67.6	24.7	55.5	30.8	16.2	-
0.6	71.2	68.1	-	-	-	15.5	2.77
0.7	71.1	-	-	-	-	15.7	-
0.8	71.0	75.3	33.0	82.8	49.8	16.1	-
0.9	70.9	65.8	-	-	-	15.7	2.78
1.0	70.8	-	-	-	-	15.7	-

Notes: W_c = water content; PL = plastic limit; LL = liquid limit; I_p = plasticity index; γ = unit weight; G_s = specific gravity

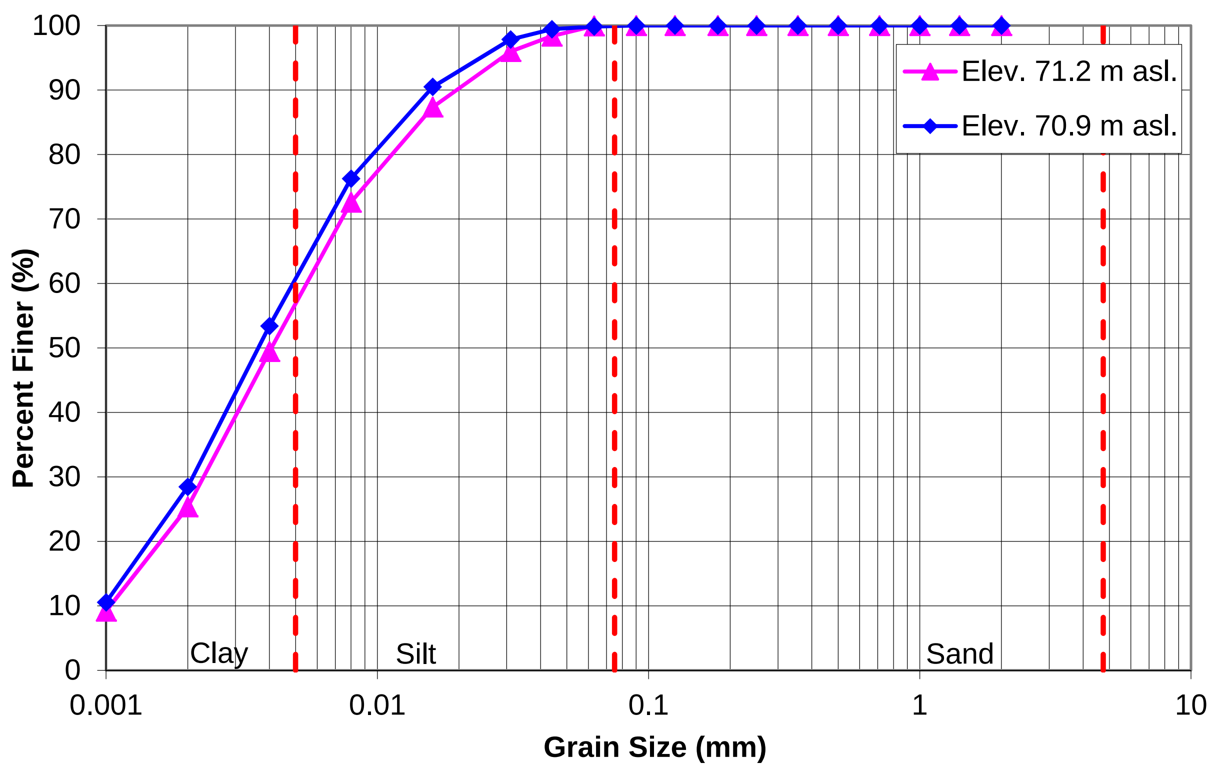

Fig. 3. Gradation chart of soil samples from BH151103

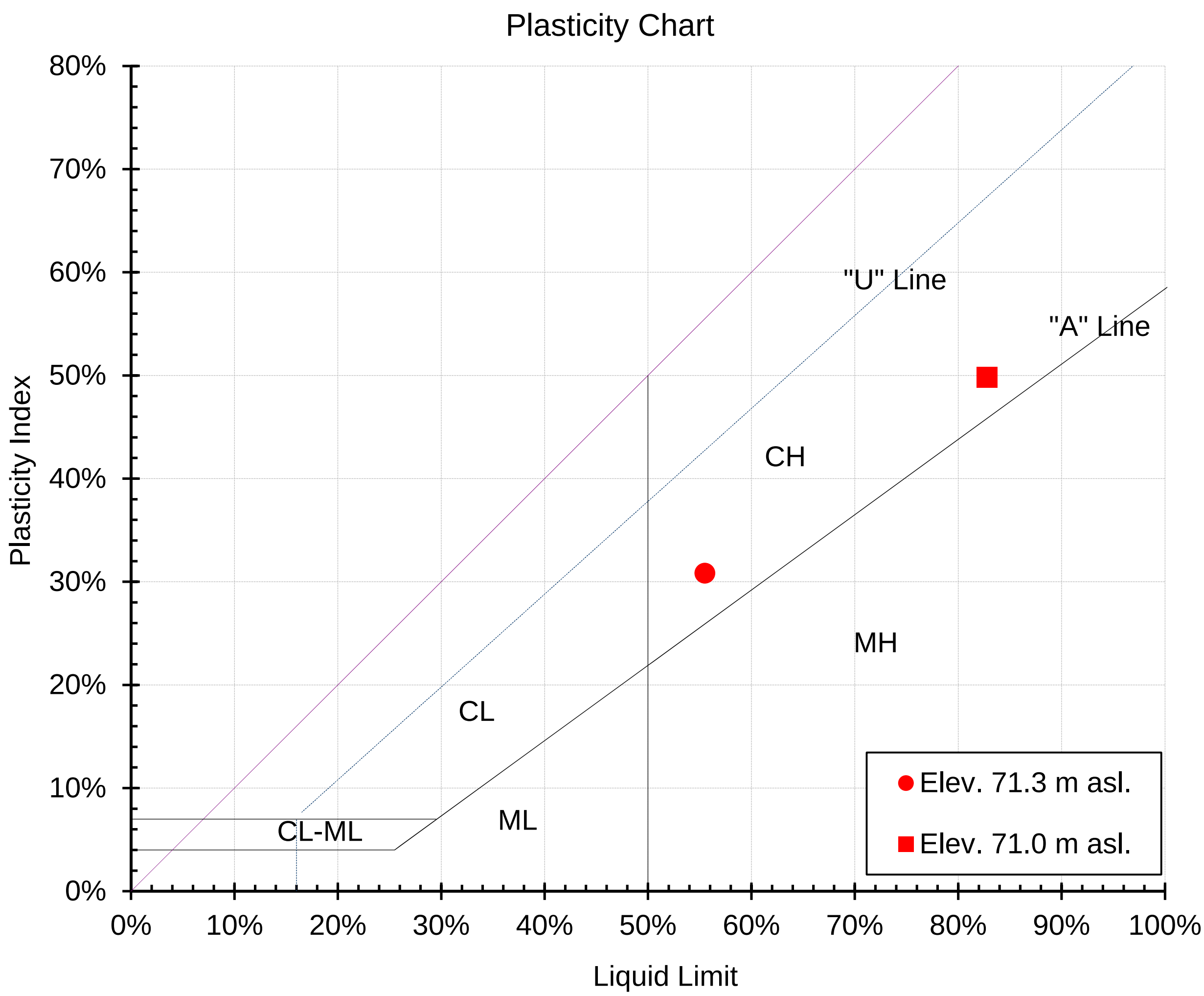

Fig. 4. Plasticity chart of soil samples from BH151103

4.2 CPT and VST Results

The cone penetration tests were conducted to a maximum depth of 55 m that was limited by the available rods and for time reasons. The CPT plots are provided in Appendix A that consist of q_t, f_s, R_f, u_2 and SBT (see Section 3.1 for descriptions). Limited pore pressure dissipation tests were carried out for time reasons. Appendix B provides data plots of the pore pressure dissipation tests.

The maximum depth of vane shear tests was 19 m that was limited by the available rods. The soil undrained shear strength from the CPT was calibrated with the VST results. The cone bearing factor $N_{kt} = 11.5$ was obtained which is consistent at all the three test locations (CPT-1, CPT-2 and CPT-3). The VST and calibrated CPT results are shown in Fig. 5.

The B_q values (Eq. 5) for the four CPT locations are shown in Fig. 6. An average B_q for each CPT (also shown in Fig. 6) is used to calculate the pore pressure bearing factor, $N_{\Delta u}$ at each location (Eq. 4). The $N_{\Delta u}$ values are 9.2, 9.1, 10.1 and 9.5 for CPT-01, CPT-02, CPT-03 and CPT-04 respectively.

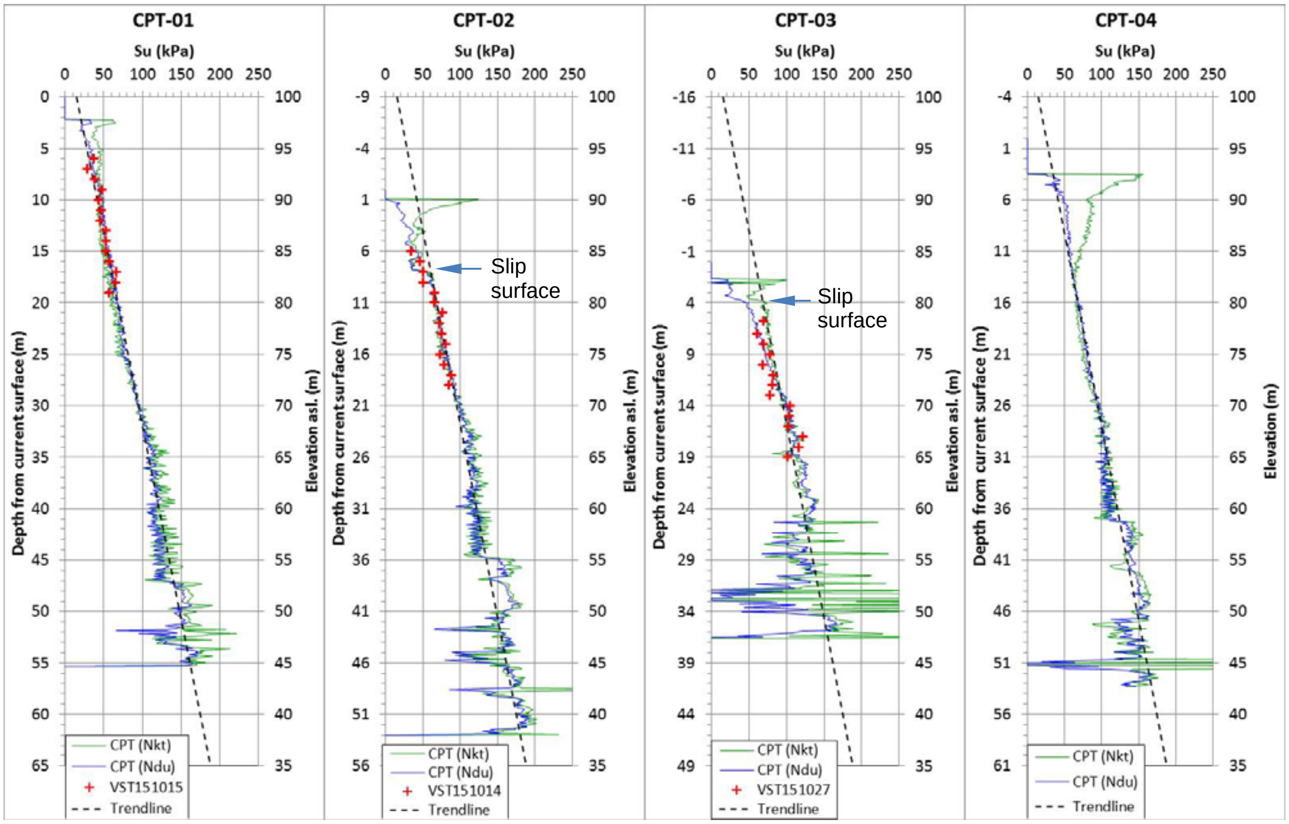

Fig. 5. Soil undrained shear strength (S_u)

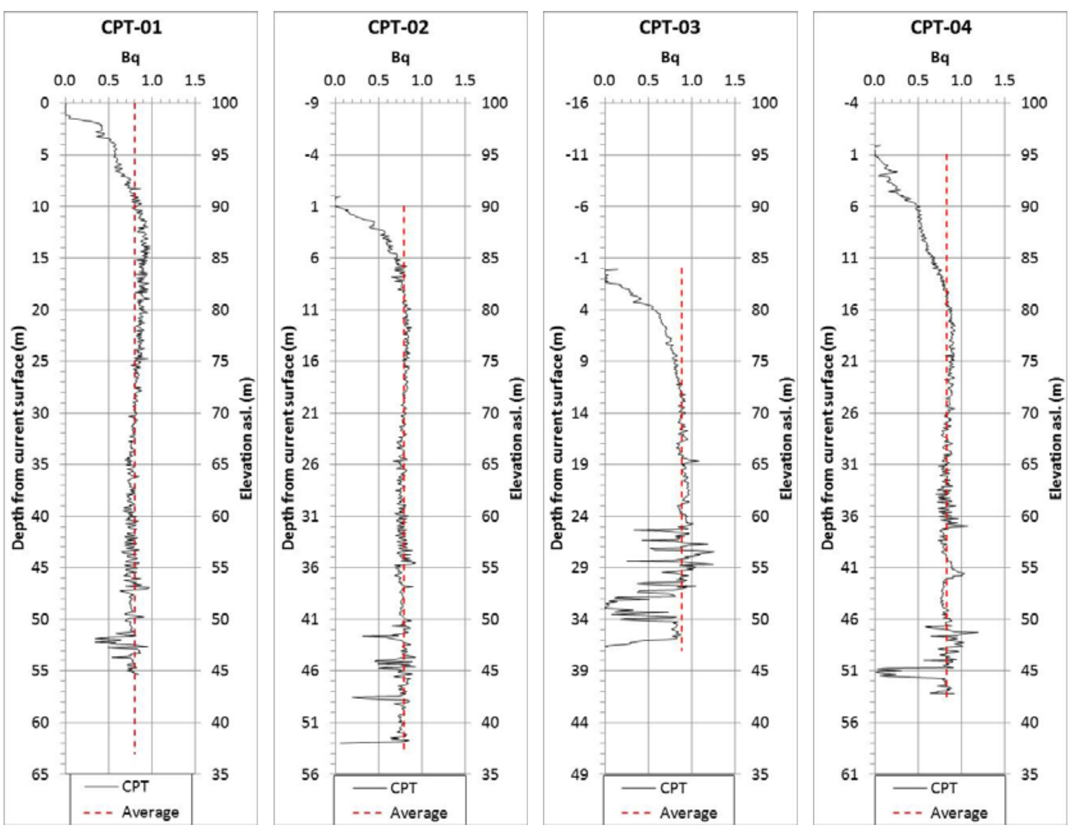

Fig. 6. Profiles of pore water pressure parameter B_q

The interpreted S_u values based on the pore pressure measurements (Eq. 3) are also included in Fig. 5. As seen in this figure, the S_u profiles interpreted from N_{kt} and $N_{\Delta u}$ are fairly close.

A linear correlation (trendline) applicable to all the four test sites was found as follows:

$$S_u = 15 + 2.66\ H \qquad (6)$$

where S_u = undrained shear strength (kPa);
H = depth (m) relative to ground surface at CPT-01 (or from 100 m elevation).

The above correlation is shown on the S_u plots in Fig. 5. It indicates that the main soil units are consistent at all the four test locations. The differences near surface are caused either by material variations or by hardening effect above water table. This is especially noticeable at CPT-04 where the water table is understandably lower as the test hole is on a ridge between two landslide depressions (Fig. 1). Refusal (bedrock) was encountered at 36.7 m depth at CPT-03. The swing of S_u profile right above the bedrock at CPT-03 is an indication of the glacial sediments typical in the Ottawa Champlain Sea region. Nevertheless, the main soil units below the surface crust have the same strength at the same elevation at all four locations despite the difference of the current surface elevations. This is an indication that the materials were horizontally and uniformly deposited. The materials of the same strength profile at the same elevation but different locations have unlikely been disturbed by the landslide. For this reason, by comparing the test results of CPT-02 and CPT-03 with that of CPT-01 (Fig. 5), the landslide slip surface is likely located at about 8 m below the current surface at CPT-02 and about 4 m below the current surface at CPT-03. The depth of the slip surface is about 15 m from the original ground surface at both CPT-02 and CPT-03 that can be interpreted from the undisturbed ground surface adjacent to the two drill holes.

5. CONCLUSIONS

This Open File presents factual data obtained from geotechnical test programs at a landslide site in Breckenridge. The in-situ shear strength results from CPT and VST agree well. The CPT bearing factor N_{kt} is determined to be a constant 11.5. The slope failed at 15 m depth from the original surface. A correlation between undrained shear strength and depth is found to be $S_u = 15 + 2.66$ H (kPa) for the undisturbed materials, where H is depth in meter relative to the crest of the head scarp (or 100 m elevation). Undisturbed soil samples collected were found to be silty clay with high to very high plasticity. The results provide input parameters for further study to evaluate the landslide failure mechanism that will lead to quantifying the magnitude of the earthquake that triggered the landslide.

ACKNOWLEDGEMENT

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APPENDIX A

Cone Penetration Test Plots

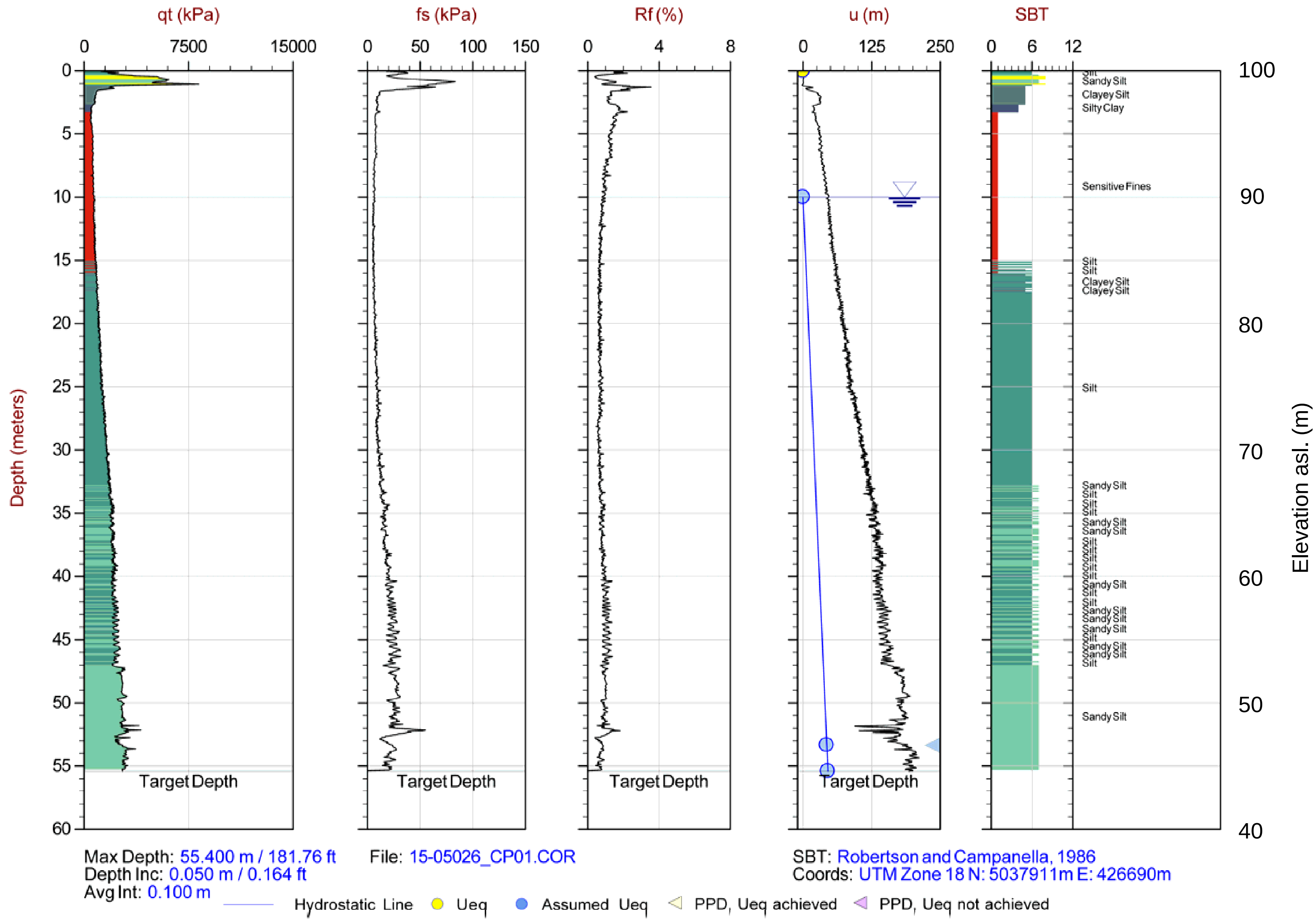

Fig. A-1. Cone penetration test plots at CPT-1

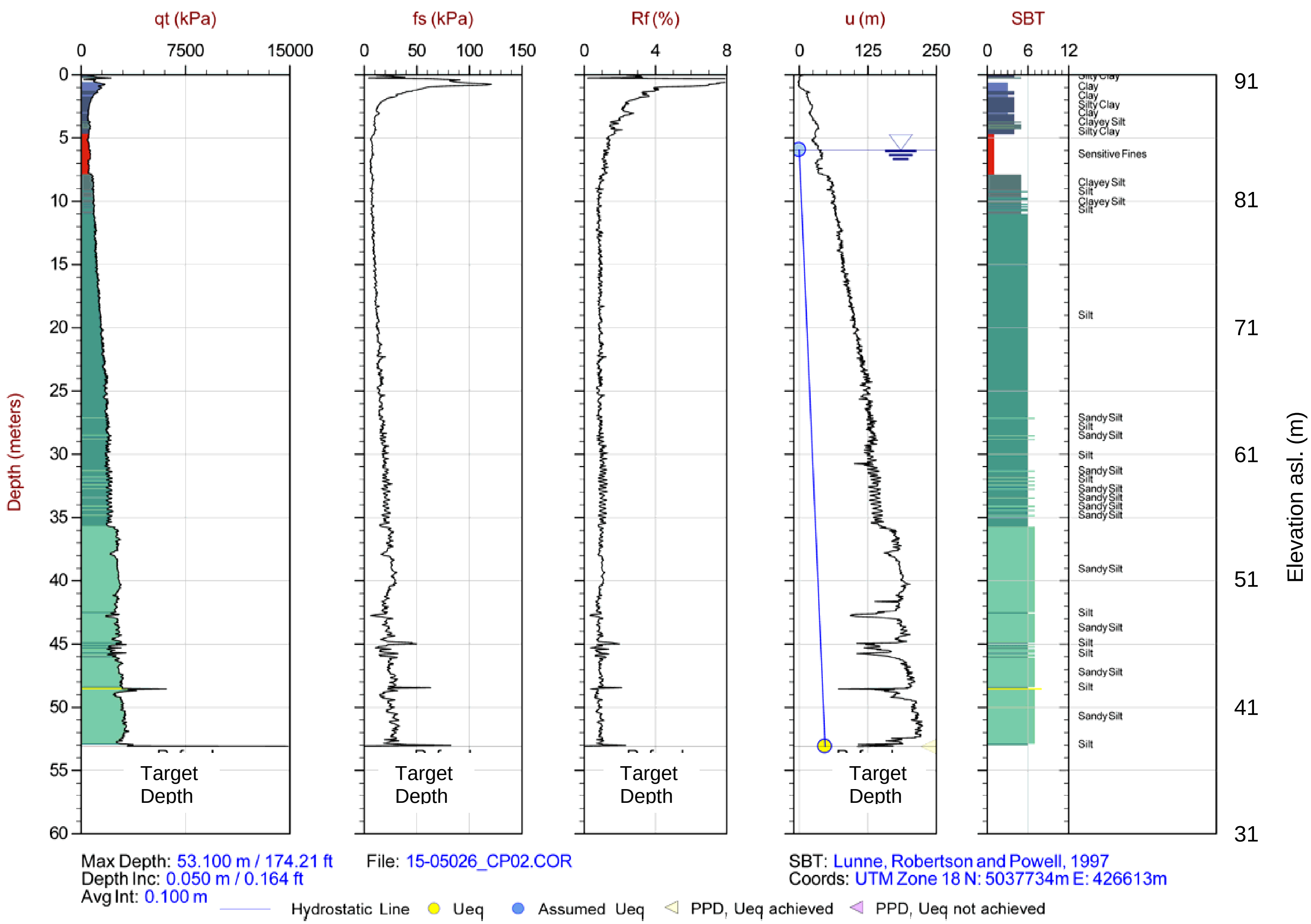

Fig. A-2. Cone penetration test plots at CPT-2

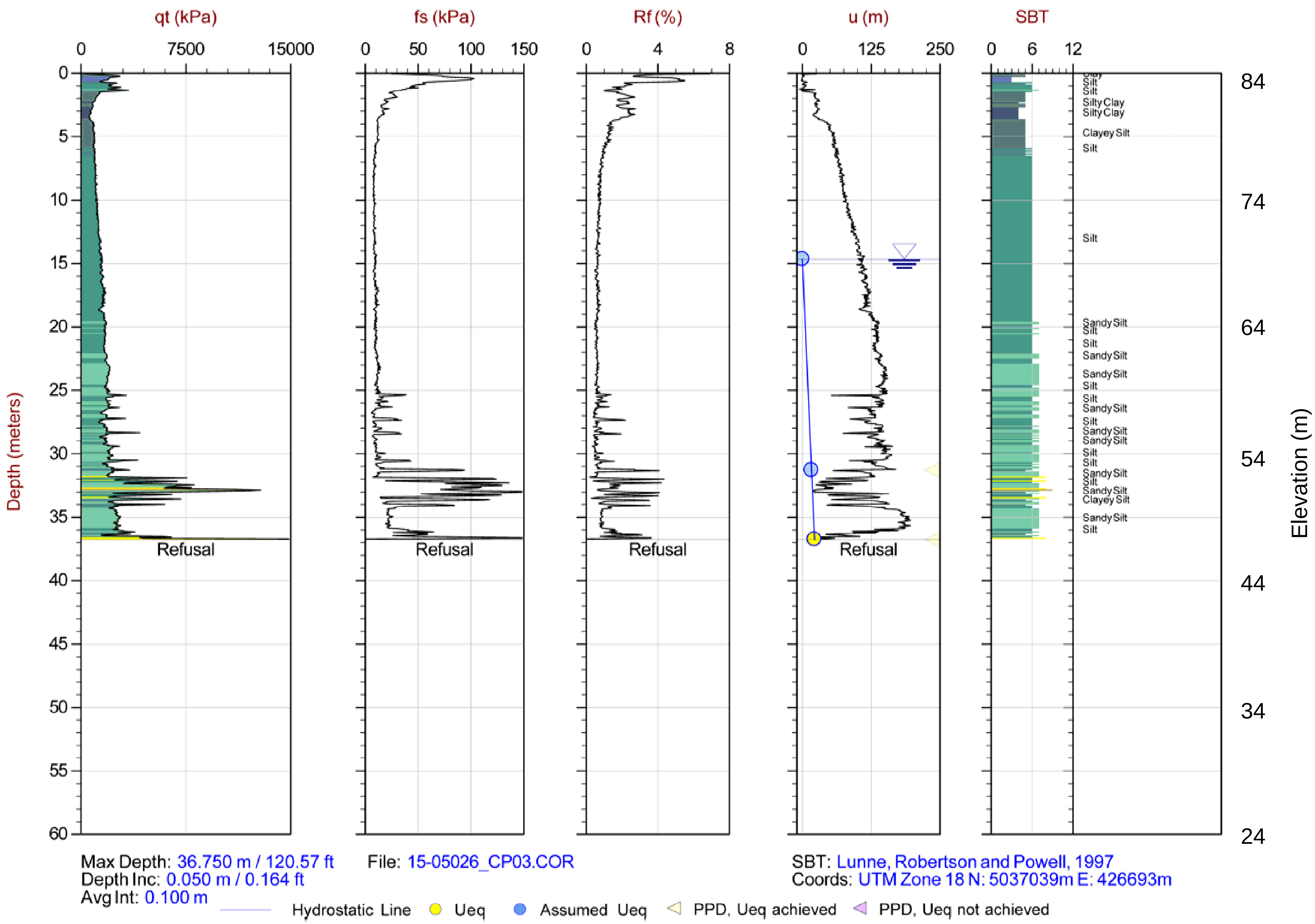

Fig. A-3. Cone penetration test plots at CPT-3

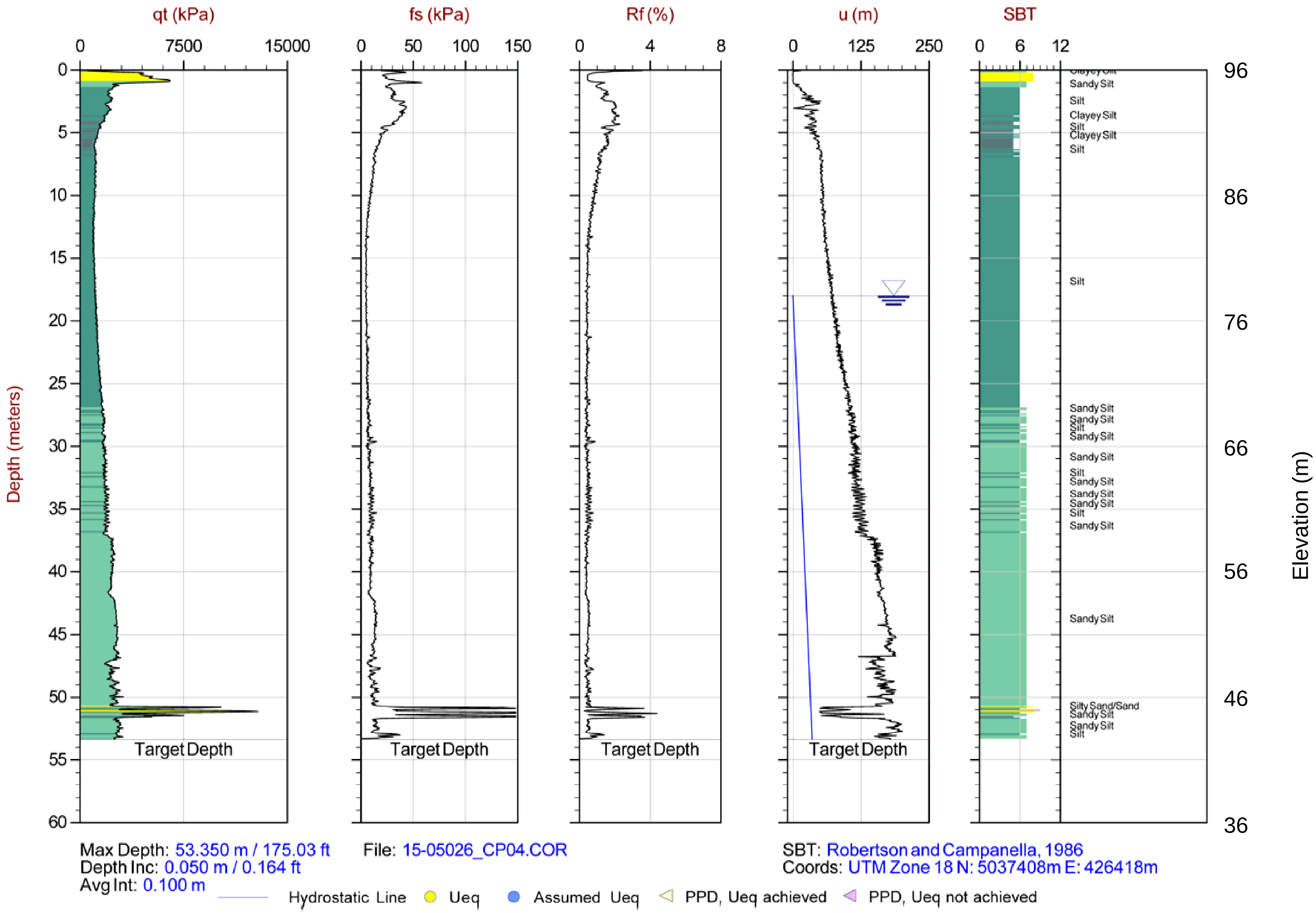

Fig. A-4. Cone penetration test plots at CPT-4

APPENDIX B

Pore Pressure Dissipation Test Plots

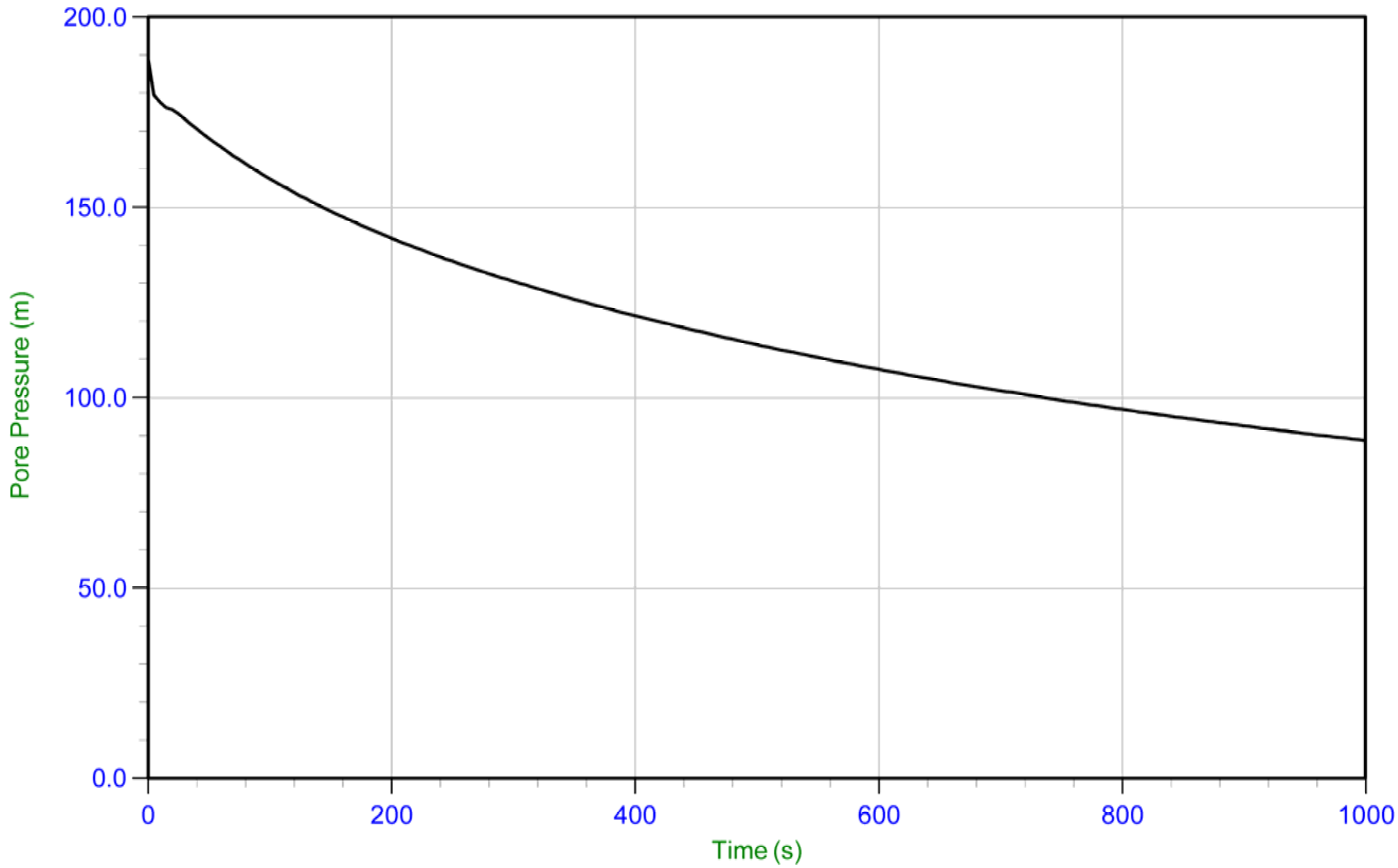

Fig. B-1. Pore Pressure Dissipation Test Plot – CPT-1 (Depth 53.35 m)

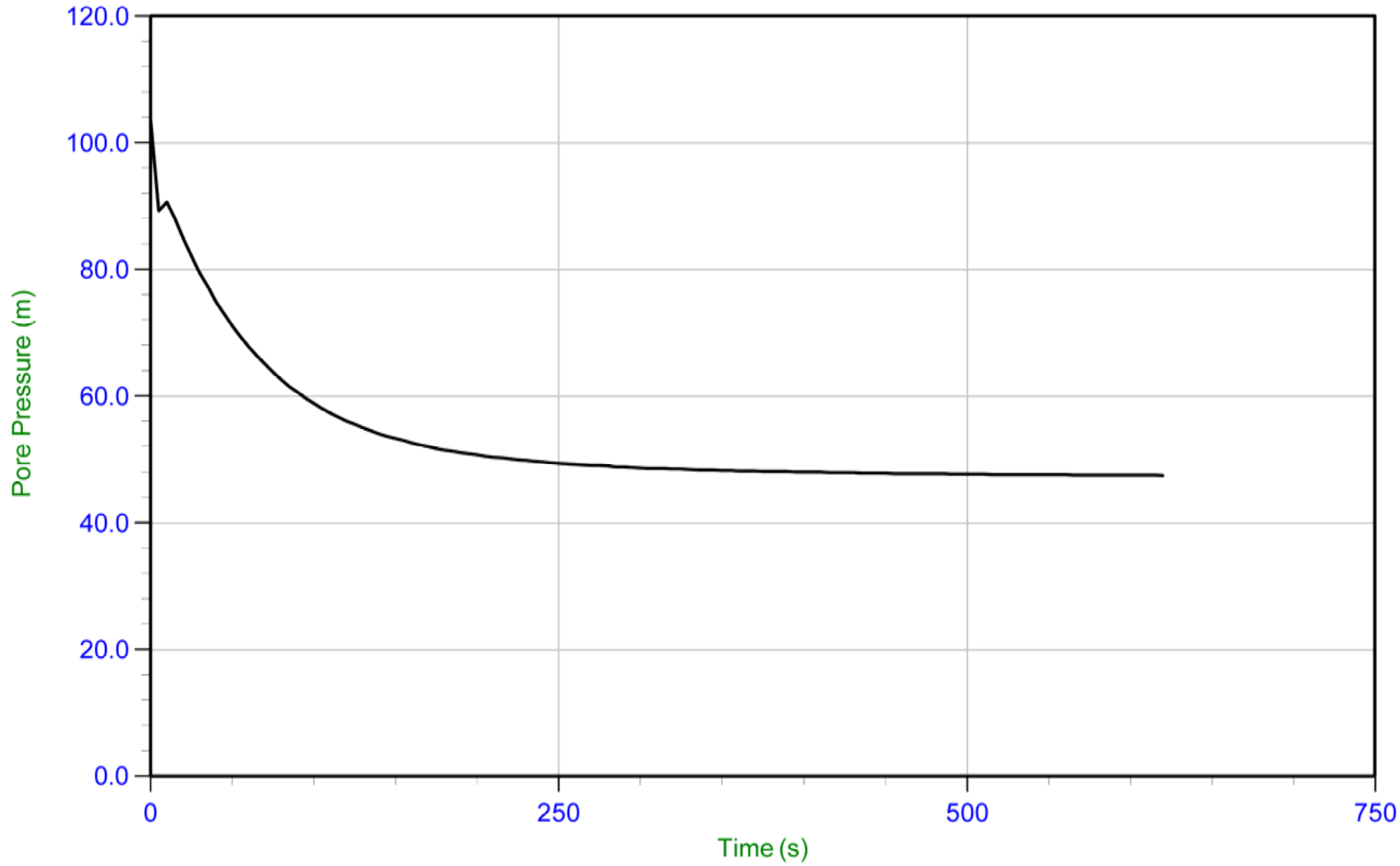

Fig. B-2. Pore Pressure Dissipation Test Plot – CPT-2 (Depth 53.10 m)

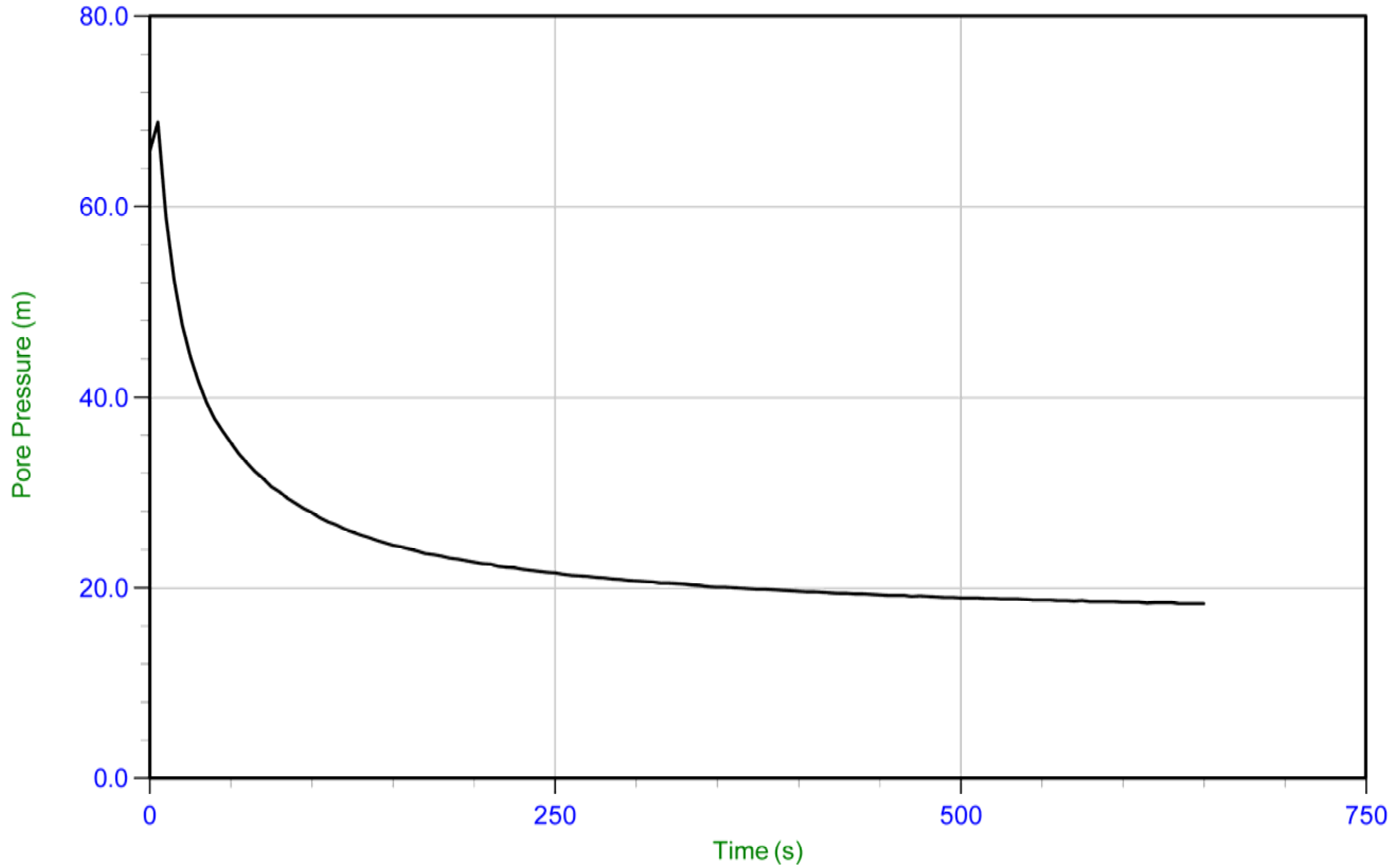

Fig. B-3. Pore Pressure Dissipation Test Plot – CPT-3 (Depth 31.30 m)

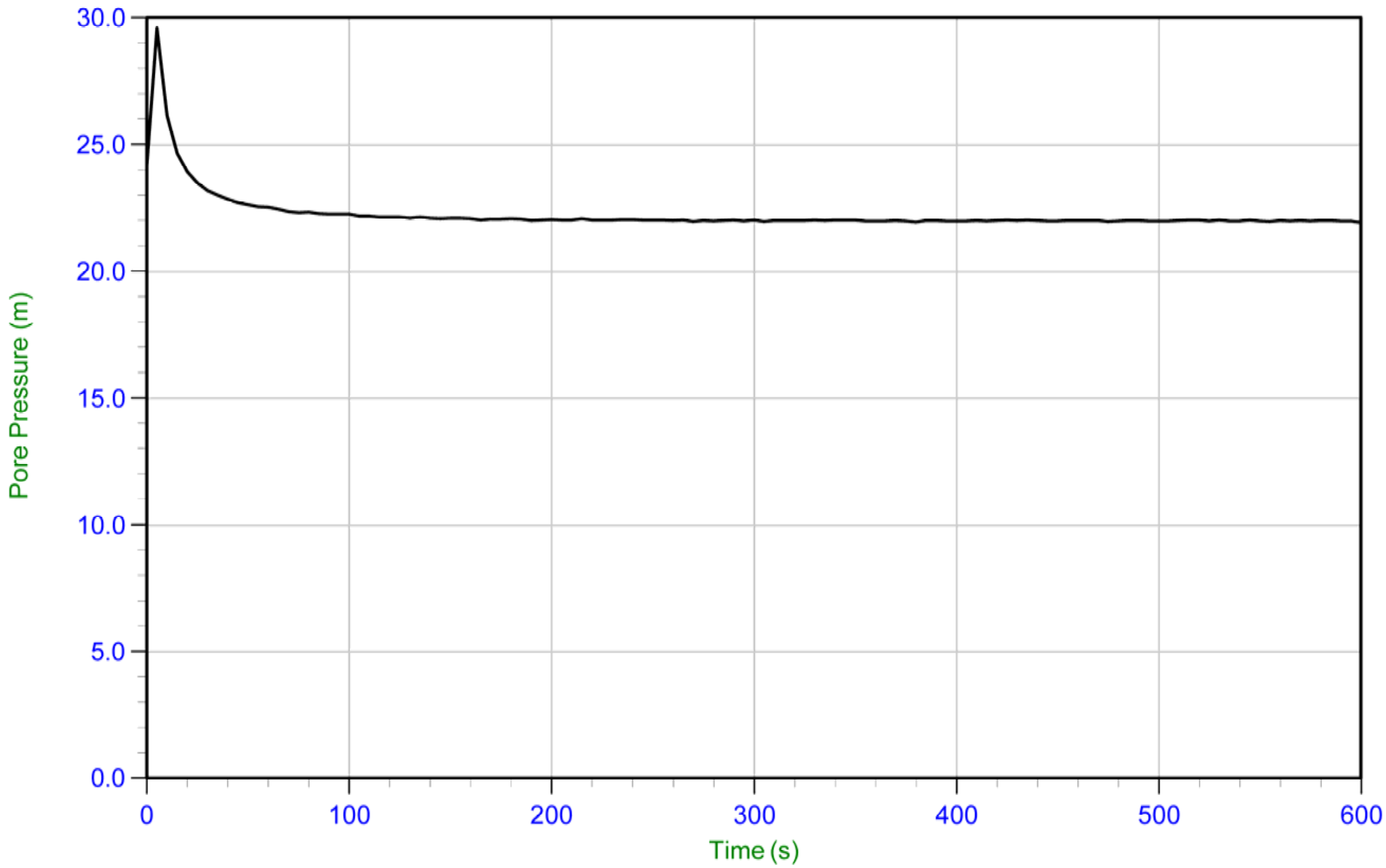

Fig. B-4. Pore Pressure Dissipation Test Plot – CPT-3 (Depth 36.75 m)