GEOTECHNICAL DATA FROM A PREHISTORIC LANDSLIDE SITE AT LOW, QUEBEC

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ABSTRACT
Geotechnical site investigations were conducted at a prehistoric landslide site at Low, Quebec. The landslide occurred in Champlain Sea clay of up to 43 m thick. Cone penetrometer tests (CPT) and vane shear tests resulted in a CPT bearing factor N_{kt} of 17.0 for the clay undisturbed by the landslide and 11.5 for that of the disturbed materials. A correlation of the peak undrained shear strength (C_u) of the undisturbed clay was found to be C_u = 28 + 1.42 H (kPa), where H (m) is depth from the pre-failure ground surface.

RÉSUMÉ
Des études géotechniques ont été réalisées sur le site d'un glissement de terrain préhistorique à Low, au Québec. Le glissement s'est produit dans le sol argileux de la mer de Champlain dont l'épaisseur a été mesurée à 43 mètres. Des essais de pénétration au cône (EPC) et des essais scissométriques ont donné un facteur de portance pénétrométrique N_{kt} de 17,0 pour l'argile non perturbée par le glissement de terrain, et de 11,5 pour l'argile de la zone perturbée. Nous avons déterminé que la résistance au cisaillement sans consolidation (C_u) du sol avait une corrélation de C_u = 28 + 1,42H (kPa), où H (m) est la profondeur en mètres, dans le sol avant le glissement.

1 INTRODUCTION

A prehistoric sensitive clay landslide at Low, Quebec is postulated by Brooks (2013, 2014, and 2015) to have been triggered by an earthquake about 1020 cal yr BP. A further geotechnical study was initiated to investigate the magnitude of the earthquake. Passive micro-seismic surveys were carried out to measure the thickness of the clay sediments in and around the landslide area. Cone penetrometer tests and field vane share tests were conducted to determine in-situ soil strength. Soil samples were collected and laboratory tests were carried out to determine geotechnical index properties. This paper presents the results of the field and laboratory tests.

2 SITE DESCRIPTION

The study site is located at Low, Quebec about 50 km NNW from Ottawa, Ontario (Fig. 1). The landslide is situated on the east bank of Stag Creek, a tributary to the Gatineau River. Regionally the topography is hilly with elevations ranging from about 100 m to 250 m above sea level. More locally around the landslide ground elevations vary from about 113 m at Stag Creek to about 170 m at nearby rock outcrops. Champlain Sea sediments are present in the relatively flat areas where the elevation is around 153 m. Access to the head scarp is via McCrank and O'Connor Roads (Fig. 1).

The landslide is about 790 m long and 430 m wide and about 15 to 20 m deep. Open land mostly hayfields and pastures surround the landslide. The area inside the landslide scar is wooded in the southeast quadrant and pasture or wetland in other areas and does not appear to have been cultivated. The post failure terrain appears well preserved. The terrain outside the landslide zone also appears little altered by human activities (see LiDAR image in Fig. 1).

East of O'Conner Road, surface water discharges to a gully and flows eastward away from the landslide zone. The drainage pattern inside the landslide zone is westward to Stag Creek. The creek is about 5 m wide and about a meter deep in summer. Spring run-off may temporarily elevate the creek level by about 1 to 2 m as observed from the bending pattern of the dead grass along the creek banks. The creek banks are mostly tree covered. The bank slopes are generally about 2.3H:1V, with the height ranging from about 10 m to 35 m. Surface sloughing or small scale slope failures are visible from vegetation changes or freshly exposed soils. The creek is incised nearly to bedrock. Probing along the creek encountered bedrock at about 2 to 3 m depth at some locations. Large areas of horizontally bedded clay are visible along the creek. At some locations, the clays are exposed by some tens of meters along the creek.

3 FIELD INVESTIGATIONS

Field investigations were conducted at the landslide site in 2016 and 2017. An ultra-portable tri-axial seismograph (Tromino®) was used to determine sediment thickness. The instrument measures horizontal-to-vertical spectral ratio (HVSR) of the ambient noise in the ground. Correlations between the sediment thickness and HVSR by Hunter et al. (2010) and Crow et al. (2017) were used to calculate the sediment thickness. A total of 27 locations (Fig. 1) were surveyed with the instrument. The interpreted sediment thicknesses were used for planning of the subsequent geotechnical investigations.

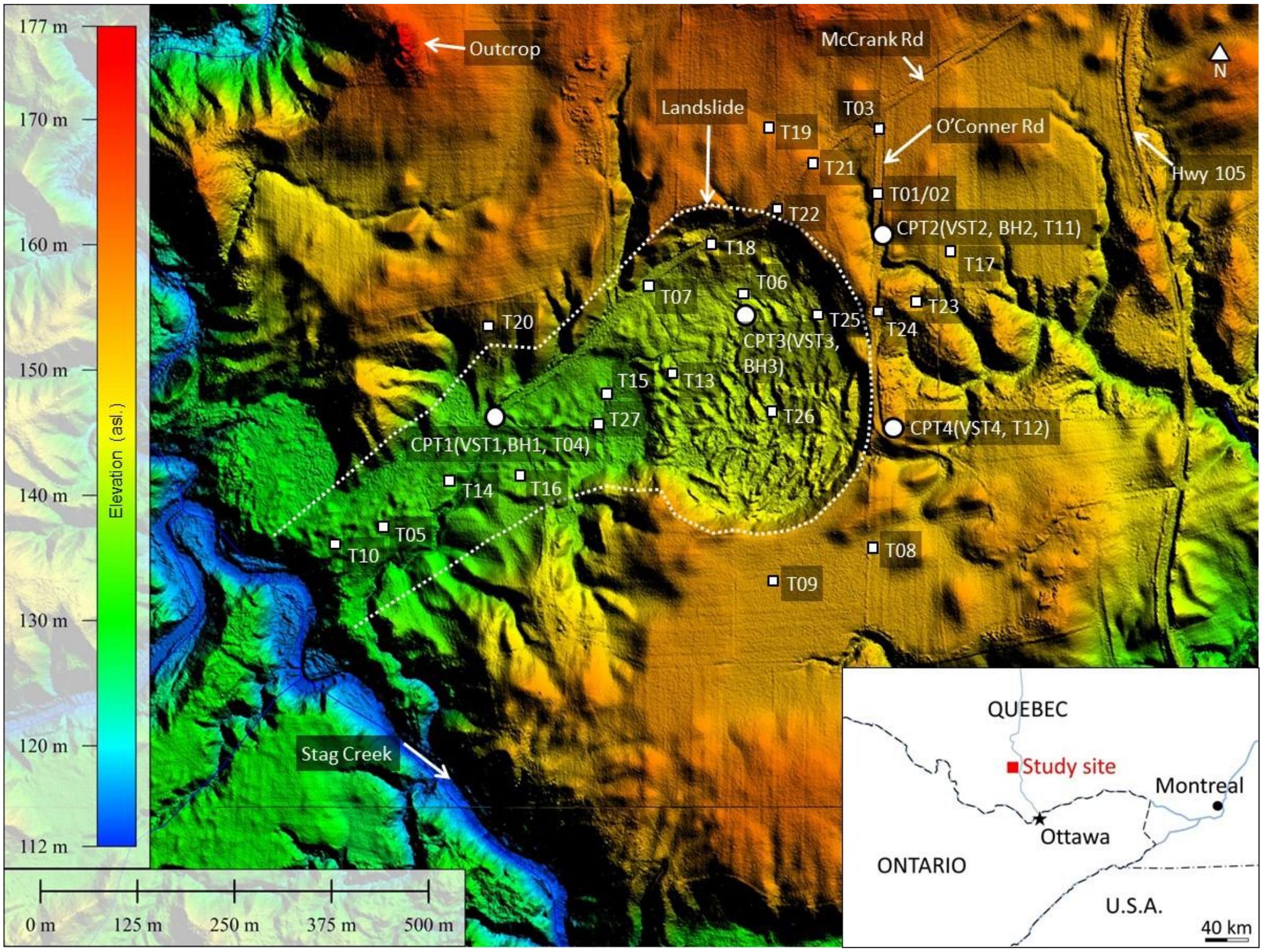

Figure 1. Location map of study area (LiDAR image ©Government of Quebec). Dotted lines mark the landslide perimeter. Rounded dots are drill hole locations. Square dots are locations of seismic survey for sediment thickness.)

Cone penetrometer tests (CPT) and field vane shear tests (VST) were carried out at four locations shown in Fig. 1 and Table 1. CPT1 and CPT3 are inside the landslide scar and CPT2 and CPT4 outside the scar.

Table 1. CPT and VST locations and total depths

Test #	Coordinates	Depth of CPT (m)	Depth of VST (m)
CPT1, VST1	N45º49.633 W75º59.393	19.4	19.0
CPT2, VST2	N45º49.760 W75º59.011	36.0	36.0
CPT3, VST3	N45º49.701 W75º59.142	28.4	23.0
CPT4, VST4	N45º49.625 W75º58.996	42.7	24.0

The CPT's were carried out to determine the soil in-situ strength parameters. A commercial 30-ton truck mounted CPT rig was used for the tests. The rig was equipped with an integrated electronic piezocone penetrometer and data acquisition system. The cone had a maximum tip capacity of 37.5 MPa, a sleeve capacity of 1.0 MPa, and a pore pressure transducer capacity of 1.4 MPa. The cone was pushed to refusal (bedrock) at all four test locations.

Field vane shear tests were carried out about 5 m away from the CPT holes. A portable Nilcon Vane Borer (RocTest M-1000) was used to determine the in-situ undrained shear strength of the clay. The equipment consists of a torque recording head, boring rods, various sized vanes and a slip coupler. During testing, the torque is scribed on a waxed paper disc mounted inside the torque head. The slip coupler installed between the vane and rod allows a free slip of approximately 15º before the vane is engaged. The torque recorded during the free slip reflects the rod friction that is subtracted from the subsequent reading for net shear resistance of the soil.

Soil samples were collected at BH1, BH2 and BH3 (near CPT1, CPT2 and CPT3 respectively) with thin wall aluminum tubes of 38 and 48 mm diameter. A portable auger was used to pre-drill the holes before coring. The samples were taken from depth ranging from 2.6 m to a maximum of 23 m below surface. The sample tubes were sealed with plastic caps and electrical tape and stored in a fridge until extruded for laboratory testing. The samples were tested for geotechnical index properties at the Sedimentology Laboratory of the Geological Survey of Canada.

4 RESULTS

The geophysics survey results of the Champlain Sea sediment thickness are provided in Table 2. The CPT depths to bedrock are also shown in this table for comparison, and are fairly consistent with the seismic survey results. The difference ranged from 0 to 5 m, which are within the expected error margin. The largest discrepancy (5 m) occurred between T04 and CPT1. Bedrock slope might account for the discrepancy. Nevertheless, the sediment thickness surveyed with the geophysics instrument was useful for the purpose of this study.

Table 2. Sediment thickness

Test #	Seismic survey (m)	CPT to bedrock (m)	Test #	Seismic survey (m)	CPT to bedrock (m)
T01/02	25		T15	26	
T03	21		T16	14	
T04	24	19 (CPT1)	T17	26	
T05	17		T18	27	
T06	28	28 (CPT3)	T19	11	
T07	19		T20	10	
T08	30		T21	29	
T09	18		T22	28(?)	
T10	18		T23	39	
T11	33	36 (CPT2)	T24	39	
T12	44	43 (CPT4)	T25	39(?)	
T13	27		T26	36	
T14	22		T27	27	

The field vane shear test results of the peak undrained shear strength (C_u), remoulded shear strength (C_r) and sensitivity (S_t) are provided in Fig. 2. The ranges of the strength values are in Table 3. The CPT results of the peak undrained shear strength were calculated from the CPT corrected tip resistance (q_t) and the overburden pressure (σ_{vo}) with a bearing factor N_{kt} as: $C_u = (q_t - \sigma_{vo}) / N_{kt}$ (Konrad and Law, 1987; Yu and Mitchell, 1998). The N_{kt} factor was calibrated with the VST data. The C_u results are shown in Fig. 3. A factor of N_{kt} = 17.0 was obtained for the undisturbed clay at CPT2 and CPT4. However, CPT1 yielded N_{kt} = 11.5, which is inside the landslide disturbed zone. At CPT3, N_{kt} = 11.5 was obtained above elevation 134 m and N_{kt} = 17.0 below elevation 134 m, which coincides approximately with the phreatic elevation of 133 m measured at the bottom of the CPT hole (Fig. 3). Note that CPT3 was located inside the landslide zone. From the N_{kt} perspective, the materials above 134 m elevation at CPT3 behaved similarly to that of CPT1 and that below 134 m elevation behaved similarly to that of CPT2 and CPT4. In other words, the material above 134 m elevation at CPT3 was likely disturbed by the landslide and the lower part was likely not. The sudden change of the CPT profile at elevation 133 m also indicates that a shear band is likely located at the vicinity of 133 m to 134 m.

Table 3. Range of VST results

Location	C_u (kPa)	C_r (kPa)	S_t
VST1	32 ~ 73	1 ~ 10	6 ~ 58
VST2	45 ~ 76	7 ~ 16	3 ~ 9
VST3	36 ~ 95	1 ~ 11	9 ~ 68
VST4	55 ~ 81	2 ~ 12	5 ~ 34

The C_u results were also independently calculated from the excess pore pressure behind the cone tip (Δu) by $C_u = \Delta u / N_{\Delta u}$ where $N_{\Delta u}$ is a pore water bearing factor (Tavenas and Leroueil, 1987). However, the two bearing factors, N_{kt} and $N_{\Delta u}$, are correlated as $N_{\Delta u} = B_q N_{kt}$, where B_q is pore pressure parameter calculated as $B_q = \Delta u/(q_t - \sigma_{vo})$. The B_q values vary with location and depth. The calculated B_q profiles at all the CPT locations are shown in Fig. 4. The approximate average B_q values below the surface crust are 0.83, 0.97, 0.84, and 0.97 for CPT1, CPT2, CPT3, and CPT4 respectively. With the N_{kt} values discussed above, the corresponding $N_{\Delta u}$ values were calculated to be 9.5, 16.5, 14.3, and 16.5 for the materials below the phreatic line at CPT1, CPT2, CPT3, and CPT4 respectively. The C_u results (below phreatic surface) calculated from $N_{\Delta u}$ agree well with that from N_{kt} at all CPT locations.

The soil gradation and index properties are provided in Figs. 5 and 6 as well as in Table 4. As seen in Fig. 5, the materials tested are silty-clay or clayey-silt. Higher plasticity was observed at shallower depths as shown in Fig. 6 and Table 4. The liquidity indexes of samples from above 134 m elevation are mostly less than 1.2 and those below 134 m elevation are mostly greater than 1.2.

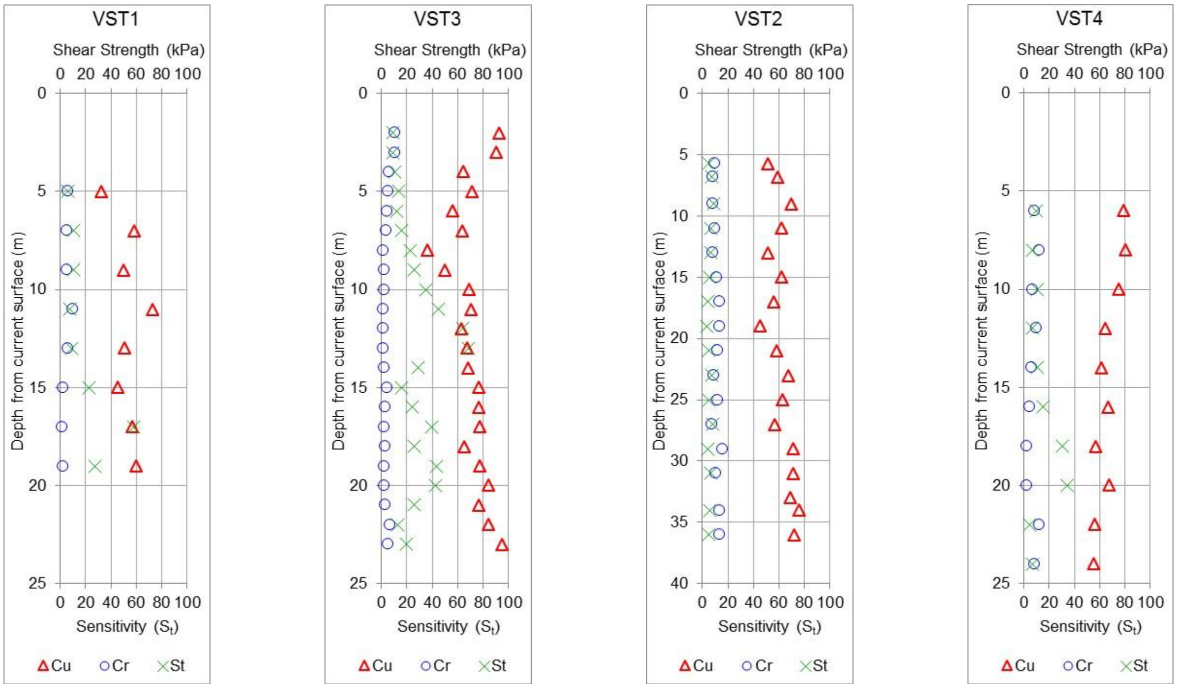

Figure 2. Vane shear test results (C_u = peak undrained shear strength; C_r = remoulded shear strength; S_t = sensitivity)

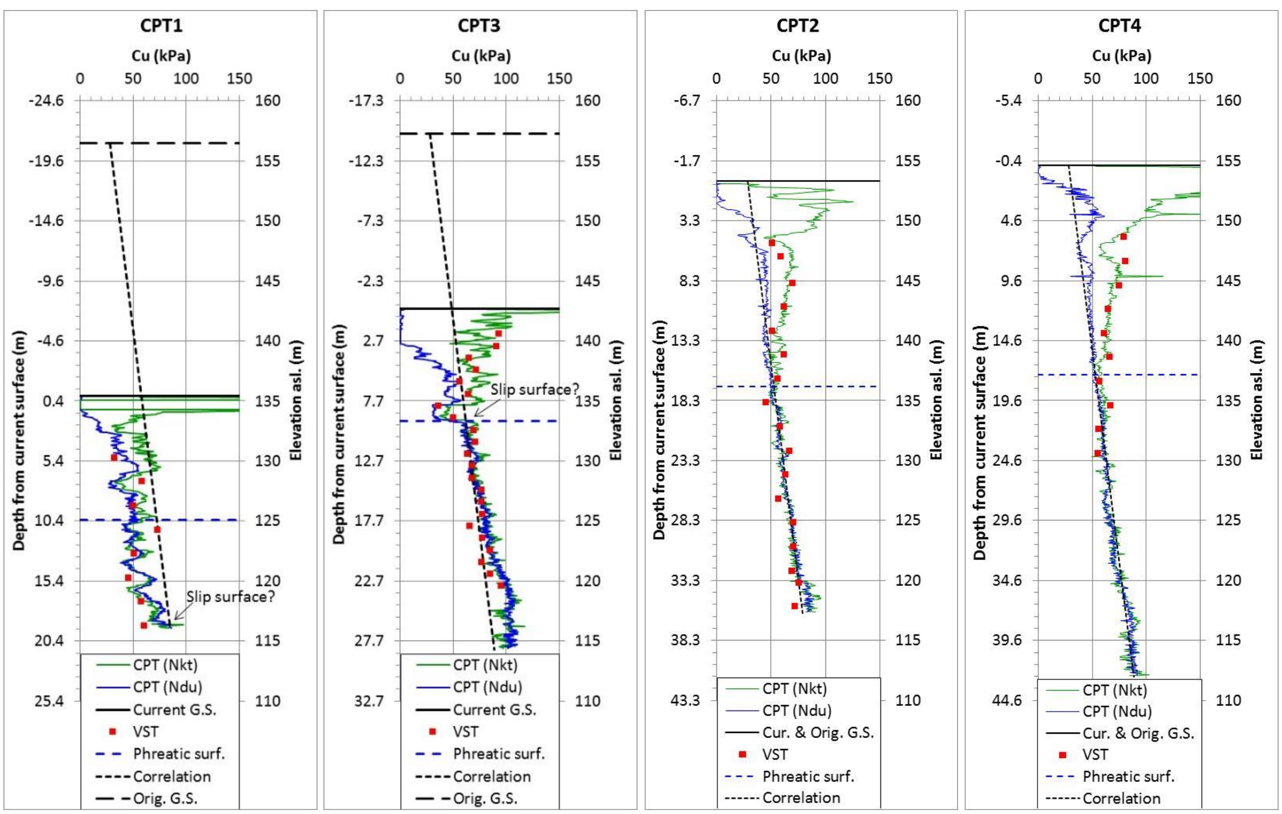

Figure 3. CPT peak undrained shear strength (C_u) calibrated with VST results (All CPT's stopped at depth of refusal)

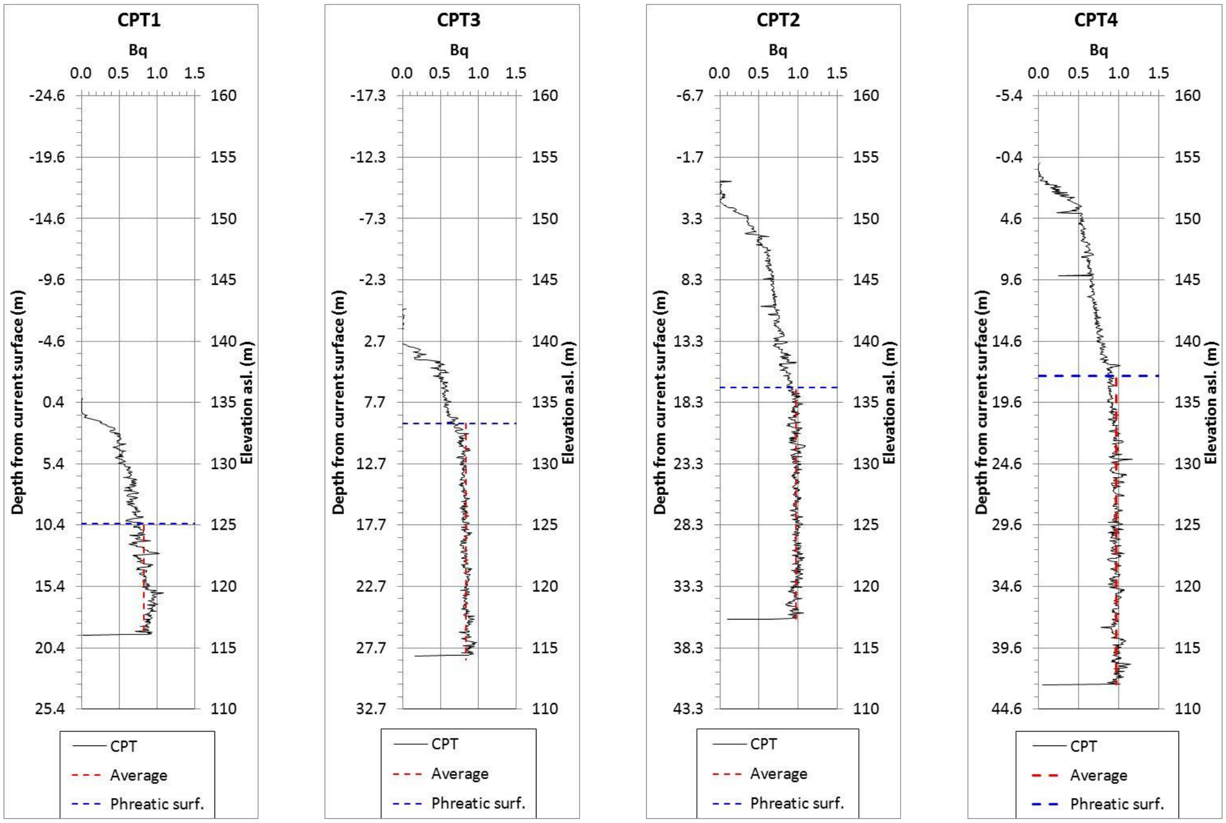

Figure 4. Profiles of pore water pressure parameter B_q

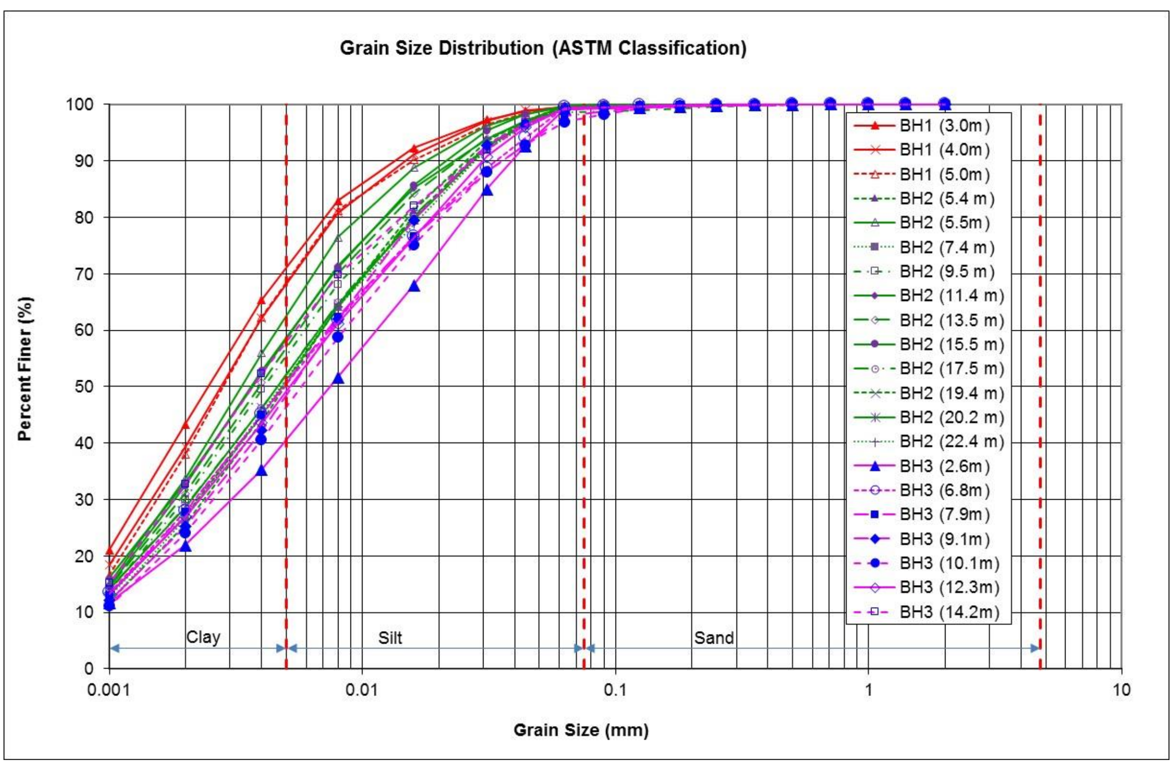

Figure. 5. Gradation chart of soil samples

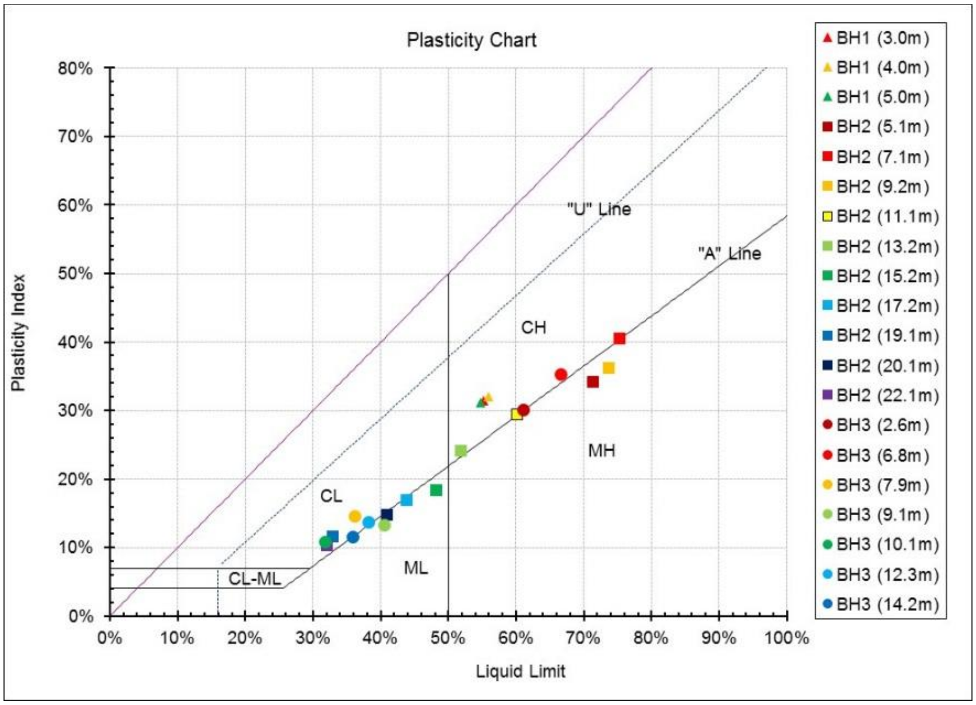

Figure 6. Plasticity chart of soil samples

Table 4. Geotechnical index properties of soil samples

Bore hole #	Depth (m)	Elevation above sea level (m)	Water content W_c (%)	Plastic limit PL (%)	Liquid limit LL (%)	Plasticity index I_p (%)	Liquidity index I_L	Unit weight γ (kN/m^3)	Specific gravity G_s
BH1	3.0	132.4	57.7	23.9	55.2	31.3	1.08	16.2	2.79
	4.0	131.4	55.9	23.8	55.9	32.1	1.00	16.9	2.79
	5.0	130.4	54.7	23.6	65.7	31.1	1.00	16.8	2.79
BH2	5.1	147.3	56.1	37.2	71.4	34.2	0.55	16.1	2.81
	7.1	145.3	57.7	34.8	75.3	40.5	0.57	16.3	2.81
	9.2	143.2	54.3	37.5	73.7	36.2	0.46	16.2	2.81
	11.1	141.3	49.9	30.9	60.3	29.4	0.65	16.5	2.80
	13.2	139.2	40.3	27.8	51.9	24.1	0.52	17.4	2.79
	15.2	137.2	52.5	29.8	48.2	18.4	1.23	16.6	2.81
	17.2	135.2	40.6	26.8	43.8	17.0	0.81	17.2	2.80
	19.1	133.3	49.7	21.3	33.0	11.7	2.44	17.2	2.80
	20.1	132.3	50.4	26.1	40.9	14.8	1.64	16.8	2.81
	22.1	130.3	46.1	21.6	32.0	10.4	2.36	17.3	2.81
BH3	2.6	140.0	52.0	31.3	61.2	30.0	0.69	16.7	2.78
	6.8	135.8	52.9	31.6	66.7	35.2	0.61	16.4	2.80
	7.9	134.7	42.1	21.8	36.3	14.5	1.40	17.2	2.79
	9.1	133.5	45.6	27.4	40.6	13.3	1.38	17.1	2.80
	10.1	132.6	52.2	21.3	32.0	10.7	2.89	17.3	2.79
	12.3	130.3	36.5	24.7	38.3	13.7	0.86	17.4	2.78
	14.2	128.4	38.8	24.5	36.0	11.5	1.25	17.6	2.79

5 DISCUSSION

The pre-failure ground surface at CPT1 and CPT3 are inferred from the adjacent terrain undisturbed by the landslide based on the LiDAR image (Fig. 1). The surfaces at CPT2 and CPT4 are not affected by the landslide. The interpreted pre-failure ground surface and the current surface at all CPT locations are shown in Fig. 3. By comparing the CPT results in Fig. 3, it was found that the C_u profiles at CPT2 and CPT4 follow the same trendline for the sediment below phreatic surface, which is:

$$C_u = 28 + 1.42\ H \qquad [1]$$

where C_u = peak undrained shear strength (kPa); and H = depth (m) from the pre-failure ground surface.

It is noted that the C_u profile of CPT3 follows approximately the same correlation expressed in eq. [1] for the portion below the phreatic line. This indicates that the materials below the phreatic line at CPT3 might not have been disturbed by the landslide and the landslide slip surface is likely in the vicinity of the phreatic surface at CPT3. Given that the regional groundwater regime may not have been affected significantly by the landslide, the pre-failure phreatic surface at CPT3 might be in proximity to the currently measured level. In other words, the lowest C_u at CPT3 when failure occurred could have been in the vicinity of the phreatic line (similar to that observed at CPT2 and CPT4). It is therefore reasonable to believe that the landslide slip surface likely coincides with the phreatic surface (around 133 m elevation or about 24 m below the inferred pre-failure surface) at CPT3.

As shown in Fig. 3, the C_u profile at CPT1 deviates entirely from the trendline (eq. 1). The materials at this location exhibit lower shear strength than the trendline. There are two probable explanations for the deviation: (1) The materials at CPT1 were relocated from an upper elevation. In other words, the landslide slip surface could be near the bedrock. (2) The pre-failure ground surface at CPT1 was much lower than that shown in Fig. 3. However, the C_u profile of CPT1 "swings" considerably. The irregularity of the C_u profile is not consistent with that of the undisturbed clay at other locations. It is therefore believed that explanation (1) above is more likely the case.

6 CONCLUSION

The peak undrained shear strength (C_u) of the Champlain Sea clay at the landslide site ranged from 32 to 95 kPa from four vane shear test holes at 2 m to 36 m depth range. The remoulded shear strength ranged from 1 to 16 kPa, and sensitivity from 3 to 68. The CPT cone tip bearing factor, N_{kt}, was calculated as 17.0 for the clay undisturbed by the landslide and 11.5 for the disturbed materials. The CPT pore water bearing factor $N_{\Delta u}$ varied with location. A correlation of the C_u profile was found to be $C_u = 28 + 1.42$ H, where H is depth in meters from the pre-failure ground surface. The landslide slip surface is likely in the vicinity of the phreatic surface of the general groundwater regime.

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