Failure mechanism of a prehistoric landslide in Champlain Sea clay at Breckenridge, Quebec

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ABSTRACT
This paper discusses the failure mechanism of a prehistoric landslide at Breckenridge, Quebec. The landslide is a Champlain Sea clay failure triggered by an earthquake about 1020 cal yr BP. Field and laboratory test data and other evidence indicate that the landslide likely occurred as a translational failure of the overall slope rather than a retrogressive failure as is commonly observed in sensitive clays. Slope stability analysis indicates that a threshold ground acceleration of 0.28 g is required to trigger the landslide. The findings provide knowledge about sensitive clay failure process as well as prehistoric seismicity in the region.

RÉSUMÉ
Nous discutons du mécanisme d'effondrement d'un glissement de terrain préhistorique à Breckenridge au Québec. Ce glissement de terrain est un effondrement d'argile de la Mer de Champlain déclenché par un séisme survenu il y a 1200 années étalonnées avant aujourd'hui. Les données d'essais obtenues sur le terrain et en laboratoire et d'autres éléments probants indiquent que ce glissement a probablement été causé par un effondrement plan de la pente globale et non par un effondrement rétrogressif, le type glissement d'argile sensible habituellement observé. L'analyse de stabilité de la pente indique qu'une accélération-seuil du sol de 0,28 g fut nécessaire pour enclencher le glissement. Ces conclusions nous informent sur les processus d'effondrement de l'argile sensible et sur la sismicité préhistorique de la région.

1 INTRODUCTION

Landslides caused by earthquakes can preserve information about the triggering seismic events. Studies of paleolandslides can decode such information that is useful for understanding prehistoric seismicity. Seismic data extending beyond the historic earthquake record can help improve the understanding of seismic hazards. A relatively large landslide in Champlain Sea sediments at Breckenridge, Quebec was investigated for this purpose. The landslide is interpreted to have been triggered by an earthquake about 1020 cal yr BP (Brooks et al., 2013). A geotechnical study was carried out to evaluate the ground acceleration required to trigger the slope failure. While the terrain and geotechnical conditions in and around the landslide are relatively well preserved, understanding its failure mechanism and process presents a challenge. Wang (2017) presented preliminary field and laboratory test results from the landslide site. The current paper continues the evaluation of the landslide failure mechanism with the presentation of new data and other evidence. Slope stability analysis was carried out to determine the critical ground acceleration required to trigger the landslide.

2 STUDY SITE

Breckenridge is located about 20 km northwest of Ottawa in the municipality of Pontiac, Quebec (Figure 1). The area is covered by Champlain Sea sediments at the foot of the Gatineau Hills. The Champlain Sea Plain is fairly level with an average elevation of about 100 m above sea level (asl). Underlying the soft Champlain Sea sediments is a deep oval-shaped bedrock basin about 5 km long and 3 km wide with the long axis parallel to Eardley Escarpment (Crow et al., 2017). The thickness of the sediments range from zero at the adjacent to the Eardley Escarpment to about 98 m at the centre of the basin (Figure 1). Most of the flat terrain in the area is farmland, hayfields or pasture.

A network of drainage channels developed across the Champlain Sea plain; Breckenridge Creek is the main channel flowing into the Ottawa River. The creek and its tributaries are incised into the sediments to a depth of about 30 m. A total of 38 landslide scars are discernable from LiDAR images (Figure 1). Brooks et al. (2013) reported the ages of 13 landslides in this area. The dated landslide ages range from modern to 7800 cal yr BP. The age of the study landslide is about 1020 cal yr BP, as is marked in Figure 1.

The study landslide (hereinafter called the large Breckenridge landslide or the large landslide) is the largest among the 38 landslides in the Breckenridge basin. An enlarged view of the landslide and other details are shown in Figure 2. The landslide scar is about 980 m long and about 370 m wide. Its lateral extent is constrained by four older landslides, one on the east side and three on the west side. The one to the east is dated about 3600 cal yr BP (Brooks et al., 2013). The three to the west have not been dated but are apparently much older. Their deposition zones were eroded by the long abandoned ancestral Ottawa River (Figure 2).

The elevation of the bedrock beneath the large landslide ranges from 40 m to 50 m asl. The Champlain Sea sediments are about 50 m to 60 m thick and become thicker north towards (Crow et al., 2017). The landslide bowl is about 8 to 15 m below the adjacent Champlain Sea Plain. The landslide scar is hummocky and wooded. Debris ridges in the scar are well preserved in the wooded areas and are clearly visible in the LiDAR image (Figure 2). The portion of the scar north of Smith Leonard Road is pasture and debris ridges are not as well preserved.

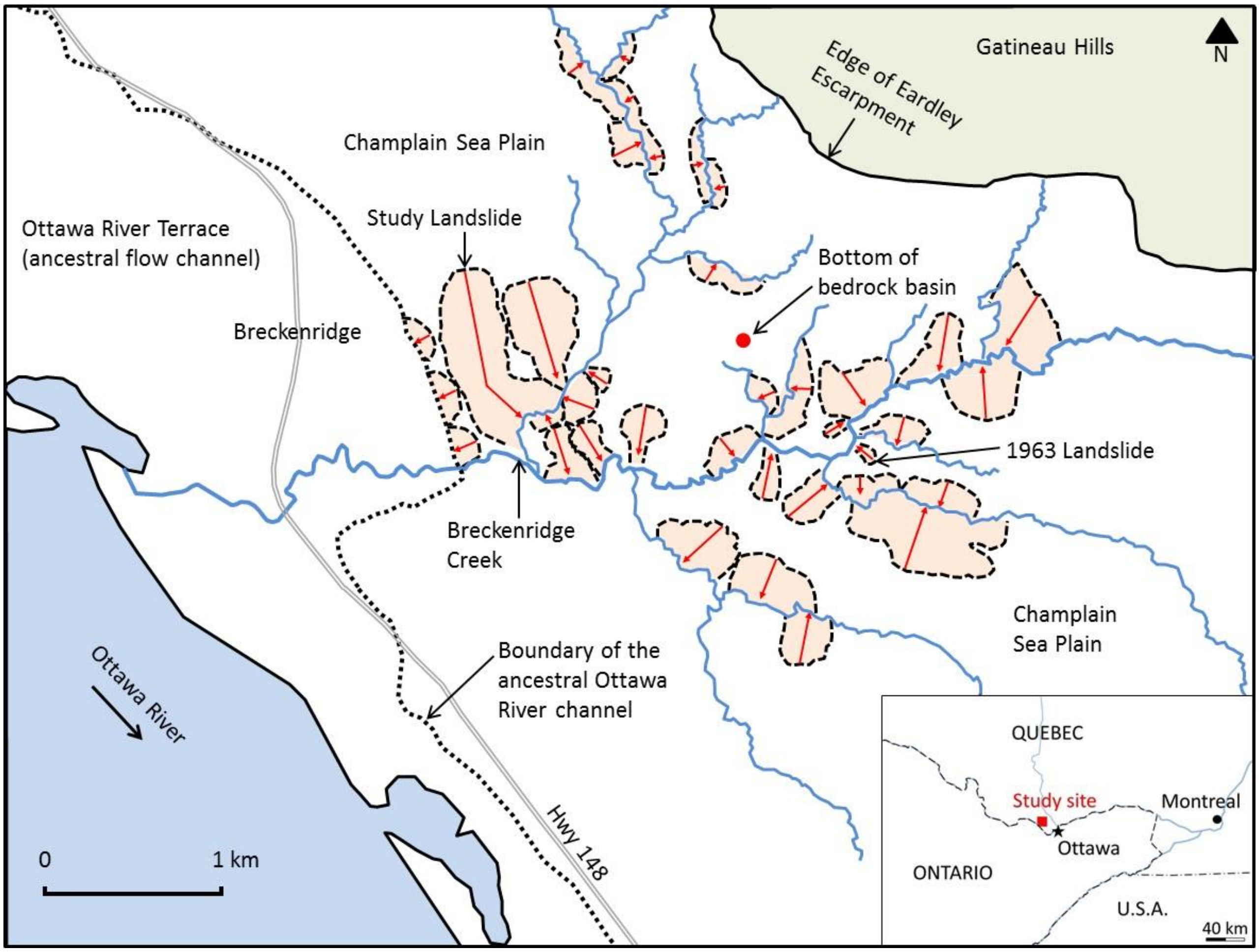

Figure 1. Location map showing landslide scars in the Breckenridge basin. (Dashed black lines mark the perimeters of the landslide scars with red arrows indicating direction and extent of the failure. Drainage indicated in blue)

The stream at the toe of the large landslide is tributary of Breckenridge Creek. Its channel is about 15 m below the landslide depression or about 30 m below the original Champlain Sea Plain. The flow is about one to two meters wide in the summer season. Clay deposits are visible along the flow channel that exhibit horizontal stratification and appear to be undisturbed by the landslide.

3 FIELD AND LABORATORY TESTS

Field tests were carried out to determine the in-situ shear strength of the Champlain Sea sediments (Wang, 2017). Field vane shear tests (VST) were conducted at three locations, VST1 to VST3 and cone penetration tests (CPT) at four locations, CPT1 to CPT4 (Figure 2). Additional soil sampling was conducted later at BH1 (adjacent to CPT1 and VST1, Figure 2). Laboratory tests were conducted for geotechnical index properties of the samples collected.

The vane shear test results are shown in Figure 3. The VST data are used to calibrate the CPT data as shown in Figure 4 where the peak undrained shear strength C_u from VST and CPT are plotted. Two CPT profiles are shown on the chart for each test location in Figure 4, one calculated from a cone tip bearing factor N_{kt} (Konrad and Law, 1987; Yu and Mitchell, 1998) and the other from a pore water bearing factor $N_{\Delta u}$ (Tavenas and Leroueil, 1987). A constant factor N_{kt} of 11.5 is obtained from CPT1 to CPT3. The same factor is applied to CPT4 in Figure 4. The $N_{\Delta u}$ factor is 9.2, 9.1, 10.1 and 9.5 for CPT1 to CPT4 respectively.

A trendline of the peak undrained shear strength is obtained from the CPT data as follows:

$$C_u = 15 + 2.66\ H \qquad [1]$$

Where C_u = peak undrained shear strength (kPa); and H = depth (m) relative to the undisturbed ground surface at CPT1 (100 m elevation).

This trendline is plotted on the CPT charts in Figure 4. As seen from those charts, the shear strength of the sediments has approximately the same trend at all four test locations. The exceptions are at the upper crust and at the lower elevations where tills are commonly observed near bedrock (Gadd, 1986). The deviation from the trendline in

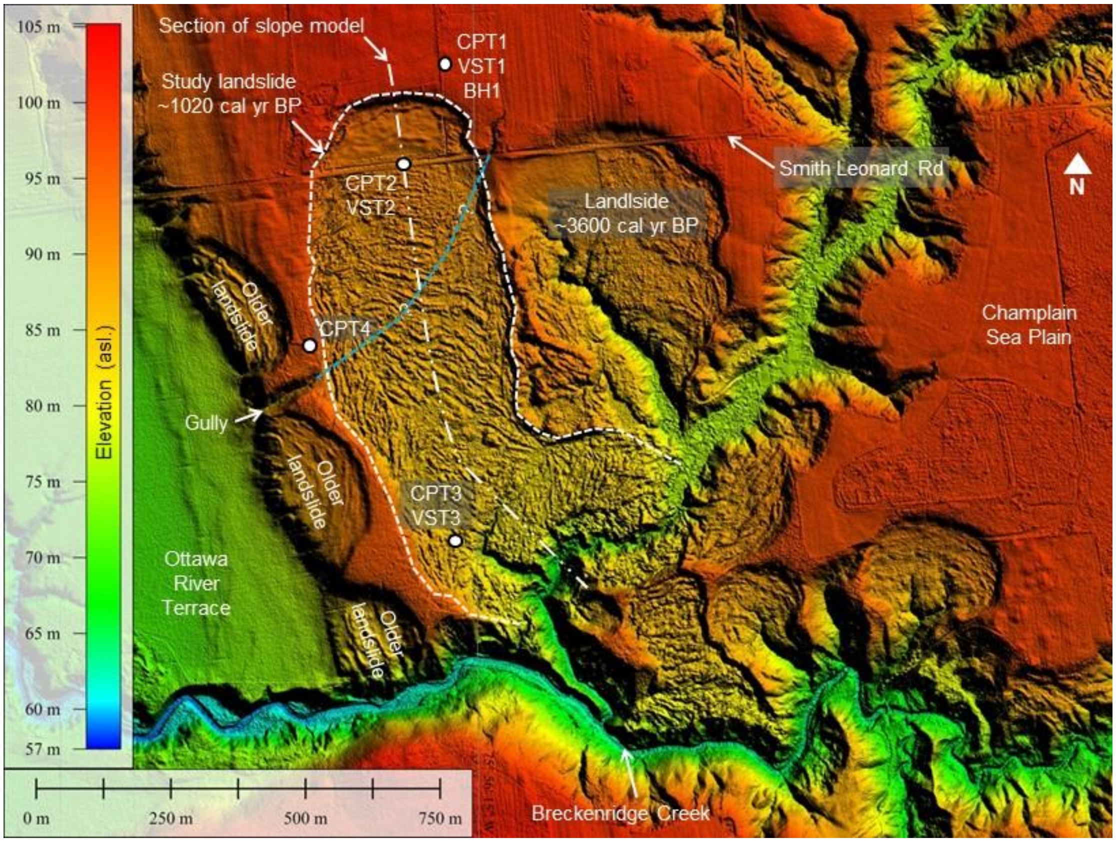

Figure 2. The study landslide, borehole locations and other details (LiDAR image © Government of Quebec)

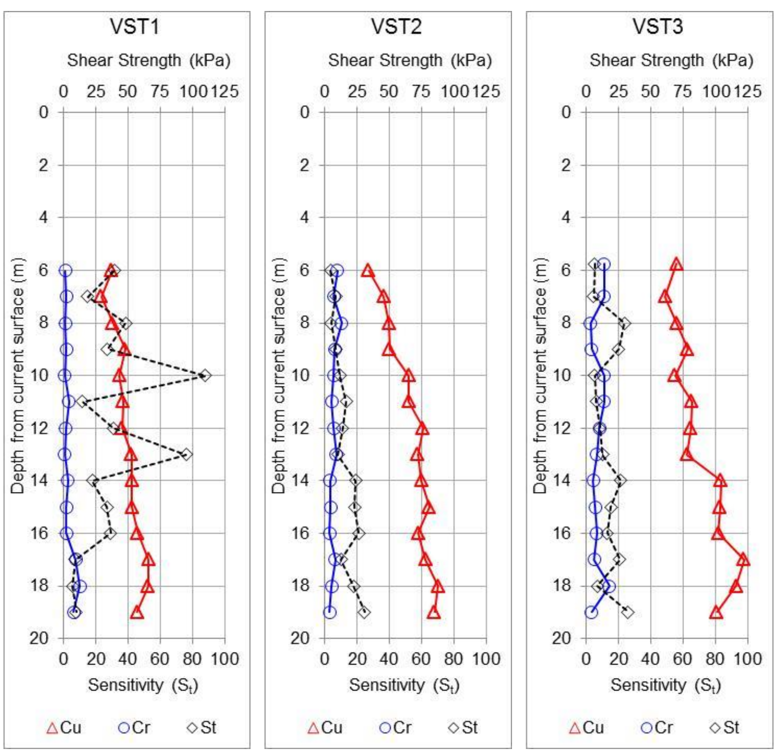

Figure 3. Vane shear test results (C_u = peak undrained shear strength, C_r = remoulded shear strength, S_t = sensitivity)

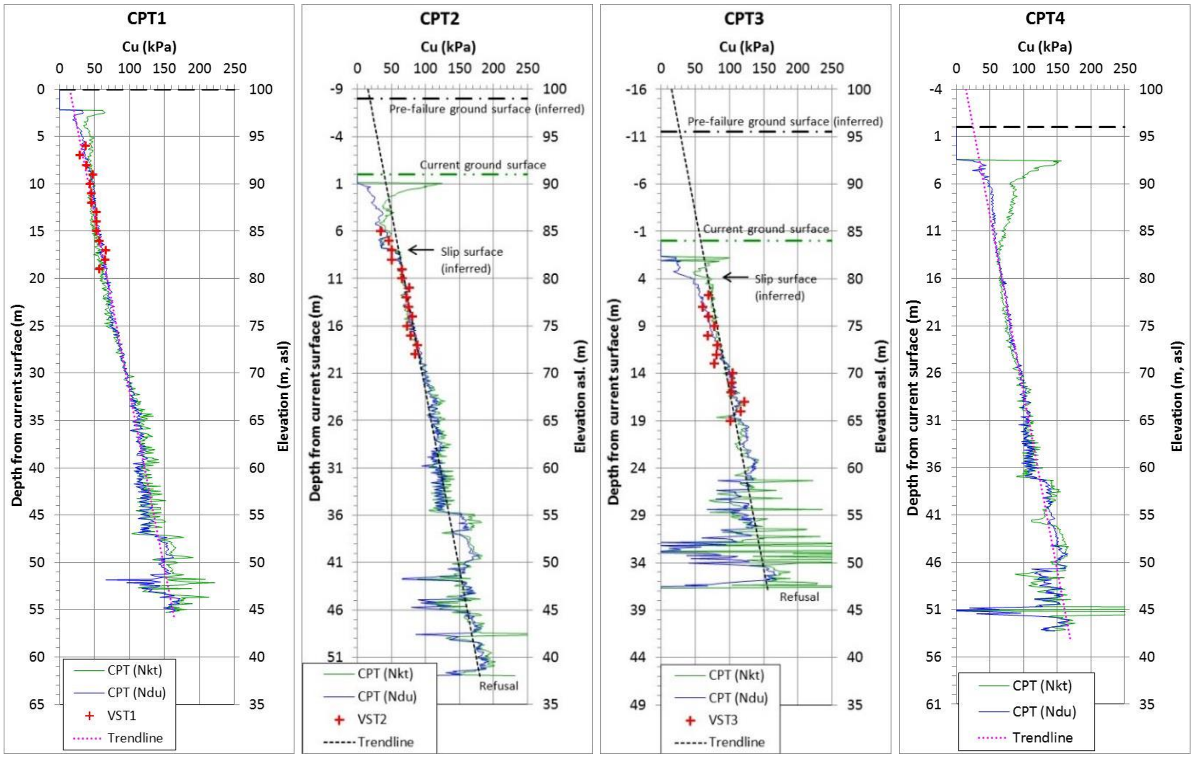

Figure 4. CPT peak undrained shear strength (C_u) calibrated from VST results (Black dashed lines are undisturbed or inferred pre-failure ground surface. Green dashed lines are post-failure ground surface.)

the upper crust is likely due to the hardening effect near the surface. This is especially obvious at CPT4 where the test hole is located on a ridge between two landslide depressions (Figure 2). The deviation from the trendline at the upper elevations at CPT2 and CPT3 are perhaps also attributed to landslide disturbance. A drop of shear strength is observed above 83 m elevation at CPT2 (Figure 4). A similar feature is noted at 80 m elevation at CPT3. It is an indication that the softer materials were likely relocated from an upper elevation. The breaking point of the profile is likely an indication of the location where shear band developed. In other words, the slip surface is likely located at about 83 m elevation at CPT2 and about 80 m elevation at CPT3 (Figure 4). The depth of the slip surface is about 15 m from the original surface at both locations (8 m and 4 m depth from the post failure surface at CPT2 and CPT3 respectively). This is consistent with the observations from the typical Champlain Sea clay failures. Demers et al. (2014) indicate that the bases of the Champlains Sea clay landslides are usually much higher than the level of the watercourse in most cases. They also indicate that the thickness of the debris materials above the floor of the scars of the historical landslides is generally low, with a mean value of 0.18 times the height of the original slope. The creek at Breckenridge is about 30 m deep. Based on the observations by Demers et al. (2014), the debris materials of the Breckenridge landslide can be calculated to have an average thickness of 30 x 0.18 = 5.4 m. The above CPT interpretation of 4 m to 8 m thick debris is therefore consistent with the typical observations.

Soil core samples were collected from BH1 at 1 m interval from 3 m to 18 m depth. The geotechnical index properties of the samples tested are provided in Table 1 and Figures 5 and 6. The data indicate that the materials are clayey silt to silty clay of high plasticity for most cases.

4 SLOPE FAILURE PROCESS

The objective of this study is to calculate the threshold ground acceleration required to trigger the landslide. Undoubtedly, the calculation depends on the slope failure mechanism. Most landslides in Champlain Sea clays are retrogressive failures (Demers et al., 2014). Such landslides start from a small scale failure of a steep slope

Table 1. Geotechnical index properties of soil samples

H (m)	Elev. asl.(m)	W_c (%)	PL (%)	LL (%)	I_P (%)	I_L	γ (kN/m^3)	G_s
3.1	96.9	75.6	31.0	70.7	39.7	1.1	14.6	2.76
5.1	94.9	85.0	32.2	65.5	33.3	1.6	14.9	2.76
7.2	92.8	78.2	29.7	63.1	33.4	1.5	14.9	2.76
9.2	90.8	85.3	32.1	68.8	36.6	1.5	14.9	2.76
11.2	88.8	76.5	31.9	53.1	21.2	2.1	14.9	2.77
13.1	86.9	82.9	32.6	62.8	30.2	1.7	14.9	2.76
15.2	84.8	52.0	28.1	44.6	16.5	1.5	14.9	2.76
17.2	82.8	52.9	29.8	54.3	24.5	0.9	-	2.76

Note: H = depth; W_c = water contents; PL = plastic limit; LL = liquid limit; I_P = plasticity index; I_L = liquidity index; γ = unit weight; G_s = specific gravity.

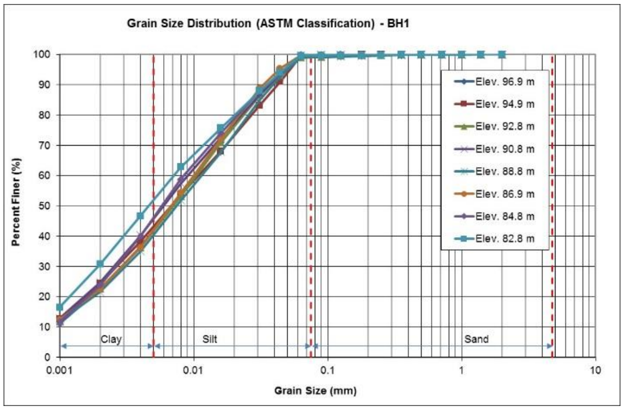

Figure 5. Grain sizes of soil samples from BH1

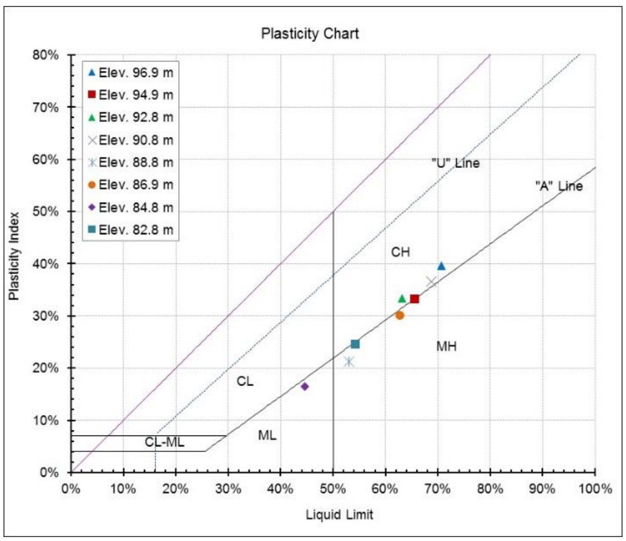

Figure 6. Plasticity chart of soil samples from BH1

usually along a water course. Upon failure, the remoulded clay carries little shear stress due to a substantial loss of strength. The higher stress initially carried by the intact clay is transferred up slope causing failure to propagate inland progressively (Carson, 1977 and 1979, Locat et al., 2011, Odenstad, 1951, and Quinn et al., 2011). If the large Breckenridge landslide failed in this manner, it may have not needed much shaking to trigger the failure had the creek valley side slope been near its critical state under static conditions. However, such a failure process may not necessarily be the case here. An example is the enormous Quyon valley landslide about 30 km west of Breckenridge. The Quyon landslide was triggered by the same earthquake about 1020 cal yr BP (Brooks, 2013). Compelling evidences indicate that the landslide is a translational failure of the overall slope (Wang, 2016). The large Breckenridge landslide resembles the Quyon landslide in many ways although it is substantially smaller. The following discusses the likelihood of its failure process.

There are various publications that discuss the governing factors and failure mechanisms of retrogressive slope failures (e.g., Mitchell and Markell 1974; Carson 1977, 1979; Tavenas et al. 1983; Locat and Demers 1988; Leroueil et al. 1996; Trak and Lacasse 1996; Locat et al. 2011; Quinn et al. 2011; Thakur and Degago 2013; and Thakur et al. 2014). One of the widely quoted criteria is by comparing a Stability Number (N_s) that is defined as $N_s = \gamma H/C_u$, where γ is soil unit weight; and H is height of the slope; and C_u is peak undrained shear strength. Leroueil et al. (1996) indicated that retrogressive failure could occur if $N_s > 4$ with a plasticity index (I_p) of around 10; or $N_s > 7$ or 8 for I_p of around 40.

As discussed earlier, the slip surface of the study landslide is at about 15 m depth (from the original Champlain Sea Plain). From Eq. 1, the peak undrained shear strength at this depth is $C_u = 15 + 2.66 \times 15 = 55$ kPa. Based on the test data in Table 1, the average unit weight of the sediment is $\gamma = 14.9$ kN/m^3. The stability number can therefore be calculated as $N_s = 14.9 \times 15 / 55 = 4$. The plasticity index in Table 1 ranges from 16.5 to 39.7 with an average of 29.4, or closer to 40 than to 10. According to the criteria by Leroueil et al. (1996), a stability factor of between 4 and 8 (say $N_s > 6$) should be required for clays of $I_p = 29.4\%$ to fail retrogressively. Apparently, the calculated $N_s = 4$ does not meet this requirement. The large Breckenridge landslide is therefore more likely to be a non-retrogressive failure.

There are 37 other landslide scars in the Breckenridge area. There is no doubt that many of these landslides are static retrogressive failures. However, it is noted that the lengths of those landslides ranged from 70 m to 370 m, except for the 3600 cal yr BP landslide to the east of the large landslide that is 600 m long (Figure 2). While the failure mechanism of the 3600 cal yr BP event is unknown, those other landslides are all significantly shorter than the large landslide that is 980 m long. Among those is the 1963 landslide (Figure 1) that is reported to be a retrogressive failure following a rainstorm (Eden et al., 1965). The failure took three days to retrogress a total of 140 m. The creek is about 30 m below the Champlain Sea Plain and the sediment is nearly 80 m thick at this location. There is no other apparent reason why the landslide stopped short but to believe that the limited capacity of the creek may have well allowed the debris to build up and buttress the failure fairly quickly. Nevertheless, the much smaller size of those other landslides in this area is compelling evidence that if a landslide occurs as a retrogressive failure it would be short. In other words, the large landslide is likely a translational failure of the overall slope rather than a retrogressive failure. This is consistent with the stability factor theory discussed earlier.

Retrogressive landslides in sensitive clays often terminate when approaching a reverse break in slope such as a stream gully (Lawrence et al. 1997, Quinn et al., 2011). This is because of clay hardening to a greater depth around the gully that increases slope stability. The shape of the large Breckenridge landslide itself is evidence of such effect. It is confined between four older landslide scars instead of crossing them. Note in Figure 2 that a small gully had likely existed prior to the landslide. The gully is about 10 m deep at the downstream end. It was truncated by the landslide instead of stopping the failure. This is another indication that the failure might have been a translational failure that undercut the gully.

Theoretically, a retrogressive failure may continue to propagate a long distance until the unbalanced stress released from the remoulded clay is balanced (Locat et al. 2011 and Quinn et al. 2011). A stress equilibrium can be reached when (1) the shear band is long enough so that the total shear resistance of the remoulded clay becomes sufficiently high to support the slope; (2) the clay at the head scarp is strong enough, such as near a gully, to withstand the additional load transferred from the remoulded clay; and/or (3) the toe of the landslide is buttressed by the debris deposit.

Observations from some recent large landslides in the region may also help understand the failure process of the large Breckenridge landslide. Eden et al. (1971) documented a large retrogressive landslide at the South Nation River about 48 km east of Ottawa in 1971. The failure occurred in Champlain Sea clay following a heavy rainstorm. The failed valley side was about 24 m high. The head scarp extended landward to about 490 m from the river. The debris flowed both upstream and downstream of the river filling about 2450 m of riverbed, and raising the river level by more than 11 m before it overtopped.

Evans and Brooks (1994) described another landslide along the South Nation River at Lemieux in 1993. It was about 4.5 km downstream of the 1971 landslide. The landslide started at the riverbank that was about 23 m high. The average depth of failure is about 18 m. A debris plug of about 12 m high and 3.3 km long dammed the river. The trigger is believed to be related to late wet spring condition. The head scarp retrogressed by about 680 m inland.

The above landslides are relatively large among the well documented Champlain Sea clay failures in the region. Based on the descriptions, debris plugs must have played a key role in stopping the failure from retrogressing further. In other words, debris build up in the river buttressed the toe and stabilized the slope. There is no doubt that the height of the debris plug depends on the volume of debris generated and the volume capacity of the river channel. The average width of the South Nation River was about 50 m as reported by Evans and Brooks in 1994. The average width of the river flow at the landslide site is also measured to be about 50 m from the Google Earth satellite image dated September 3, 2016. The volume capacity of the creek valley at Breckenridge is nowhere near that of the South Nation River. The water flow in the creek is only a couple of meters wide and the average width of the valley bottom is about 5 m as measured from the LiDAR image (Figure 2), which is a clear contrast to the 50 m wide flow channel of the South Nation River. It is therefore much easier for the creek to be plugged by landslide debris than for the river. However, the large Breckenridge landslide is much longer than those at the South Nation River mentioned above. This is another indication that the large Breckenridge landslide may well be a translational failure of the overall slope rather than a retrogressive one.

In summary, the large Breckenridge landslide most likely started from an overall slope failure under seismic loading. Similar to that described in Wang (2016) for the Quyon landslide, the translational failure should have been followed by disintegration of the sliding blocks and gradual discharge of the remoulded clay. Due to the relatively small discharge capacity of the creek valley and depending on the fluidity of the remoulded clay, the debris evacuation process might have taken a much longer period of time than the typical retrogressive landslides.

5 SLOPE STABILITY ANALYSIS

The threshold ground acceleration required to trigger the landslide can be calculated through slope stability analysis. There are various types of models available for slope stability analysis. They range from a simple closed form solution, limit equilibrium methods, finite element, discrete element and coupled methods. Some models can simulate material flow process, e.g., Roy and Hawlayder (2017). However, the most sophisticated models are not without serious limitations. For example, the material parameters cannot be easily acquired and there is lack of standard of acquiring them. The assumptions made at microscopic level, e.g. particles and their interactions for some latest models still have considerable room for improvement. The analysis in the current study uses a limit equilibrium model that is considered appropriate. Limit equilibrium models are the most widely used and tested methods that calculate the factor of safety by comparing soil resistance with driving force. The calculations are straight forward and the required parameters can be obtained with well established and standardized methods. Although its simplicity comes with limitations that encourage development of alternative tools, its rich case history and proven success rate justify its use for the purpose of this study.

A two-dimensional limit equilibrium model, Slope/W developed by Geo-Slope International (2010) is used in this study. Slope/W is an industry standard software that has been widely used internationally for slope stability design. The model is well suited for translational slope failures as the case here in this study. A Slope/W model is constructed (Figure 7) for a longitudinal section at location shown in Figure 2. The pre-failure ground surface is interpolated from the adjacent Champlain Sea terraces. The post-failure ground surface is shown in Figure 7 for reference. The bedrock is far below the landslide based on the CPT results and with reference to Crow et al. (2017). One soil unit is assumed overlying bedrock in the model. The hardened surface crust has negligible effect due to the great length of the slope and is therefore ignored for simplicity. The geometry of the pre-failure creek bank slope is extrapolated from the current bank slope. The sensitivity of this assumption diminishes with the relatively long slope. The slip surface is inferred from the CPT test results as discussed earlier. There could be some uncertainty of the slip surface locally near the creek bank slope, but again its effect diminishes with the relatively long slope. The clay peak undrained shear strength given by Eq. 1 is used in the analysis. A unit weight of 14.9 kN/m^3 is assumed based on the average test result in Table 1.

Pseudo-static total stress analysis is performed with the Morgenstern-Price method (Verification with other methods resulted in very marginal differences). A horizontal seismic coefficient is applied to calculate the factor of safety (FOS). Vertical seismic load is ignored as it has negligible effect on the relatively flat ground. The analysis is performed with a trial-and-error approach. A

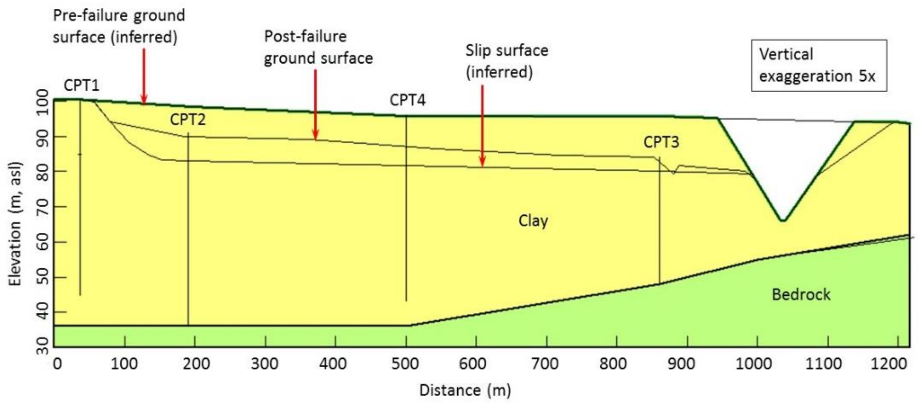

Figure 7. Slope stability model configuration

random seismic coefficient is applied to the model and its corresponding factor of safety is calculated. The coefficient is then adjusted for another calculation and the cycle is repeated until a FOS = 1.0 is achieved. A seismic coefficient of 0.28 g is obtained for a FOS = 1.0. This is considered as a critical or threshold seismic load required to trigger the landslide.

6 DISCUSSION

The threshold ground acceleration of 0.28 g for the large Breckenridge landslide is fairly consistent with other information from the region. The ~1020 cal yr BP earthquake that triggered the large Breckenridge landslide is interpreted to have triggered at least 9 other Champlain Sea clay landslides (Brooks et al., 2013). This group includes the massive Quyon Valley landslide (600 Mm^3), which is one of the largest sensitive clay failures in eastern Canada (Brooks, 2013). Wang (2016) estimated a critical ground acceleration of 0.27 g to trigger the Quyon landslide. The threshold ground acceleration of 0.28 g for the large Breckenridge landslide is similar to that of the Quyon landslide.

An earthquake occurred at Val-des-Bois, Quebec on June 23, 2010. The epicenter is located at about 58 km northeast of Breckenridge. Atkinson and Assatourians (2010) and Lin and Adams (2010) reported the earthquake as moment magnitude M_w 5.0. It is indicated by Ma and Motazedian (2012) as M_w 5.2. Lin and Adams (2010) reported a horizontal peak ground acceleration of 0.15 g recorded at a soil site about 49 km away from the epicenter. While the 0.28 g for the large Breckenridge landslide cannot be directly compared with the 2010 measurement, it does indicate that the 1020 cal yr BP earthquake is either closer than 49 km to Breckenridge or greater than M_w 5.0-5.2. Notably, that the 2010 earthquake is known to have triggered two Champlain Sea clay landslides about 12 km and 18 km away from the epicenter (Perret et al. 2013). The sizes of the landslides are 420 m x 150 m and 80 m x 180 m, which are much smaller than that of the Breckenridge or Quyon landslides. The higher ground acceleration (0.28 g) obtained from this study is consistent with the greater number of landslides triggered, their substantially larger sizes, and a greater area affected.

7 CONCLUSION

Cone penetration tests and vane shear tests were conducted at the large Breckenridge landslide site. The calibrated cone bearing factor N_{kt} is a constant 11.5. The clay peak undrained shear strength (C_u) is found to follow a correlation with depth (H) as $C_u = 15 + 2.66$ H. The depth of the slope failure is about 15 m below the Champlain Sea Plain. The landslide is likely a translational failure of the overall slope. Slope stability analysis indicates a threshold horizontal seismic coefficient of 0.28 g required to trigger the landslide.

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REFERENCE

Atkinson, G.M. and Assatourians, K., 2010. Attenuation and source characteristics of the June 23, 2010 M5.0 Val-des-Bois, Quebec earthquake. *Seismological Research Letters*, 81(5), pp. 849-860.

Brooks, G.R., 2013. A massive sensitive clay landslide, Quyon Valley, southwestern Quebec, Canada, and evidence for a paleoearthquake triggering mechanism. *Quaternary Research*, 80: 425-434.

Brooks, G.R., B.E. Medioli, J.M. Aylsworth, and D.E. Lawrence, 2013. A compilation of radiocarbon dates relating to the age of sensitive clay landslides in the Ottawa Valley, Ontario-Quebec, *Geological Survey of Canada, Open File 7432*, 58 p.

Carson, M.A., 1977. On the retrogression of landslides in sensitive muddy sediments. *Can. Geot. J.,* 14: 582-602.

Carson, M.A., 1979. On the retrogression of landslides in sensitive muddy sediments: Reply. *Can. Geot. J.,* 16: 431-444.

Crow, H., Alpay, S., Hinton, M., Knight, R., Oldenborger, G., Percival, J.P., Pugin, A.J.-M., Pelchat, P., 2017. Geophysical, geotechnical, geochemical, and mineralogical data sets collected in Champlain Sea sediments in the Municipality of Pontiac, Quebec. *Geological Survey of Canada Open File 7881*, doi:10.4095/301664. 50 p.

Demers, D., Robitaille, D., Locat, P., Potvin, J., 2014. Inventory of large landslides in sensitive clay in the province of Québec, Canada: Preliminary analysis. J.-S. L'Heureux et al (eds), *Landslides in Sensitive Clays: From Geosciences to Risk Management, Advances in Natural and Technological Hazards Research 36*, 77-89.

Eden, W.J., Fletcher, E.B. and Mitchell, R.J., 1971. South Nation River landslide, 16 May 1971. *Can. Geot. J.,* 8, 446-451.

Eden, W.J., Matyas, E.L. and Irwin, W., 1965. The Breckenridge landslide. *National Research Council Canada Division of Building Research, Internal Report* No. 316. 13 p.

Evans, S.G. and Brooks, G.R., 1994. An earthflow in sensitive Champlain Sea sediments at Lemieux, Ontario, June 20, 2993, and its impact on the South Nation River. *Can. Geot. J.,* 31, 384-394.

Gadd, N.R., 1986. Lithofacies of Leda clay in the Ottawa basin of the Champlain Sea. *Geological Survey of Canada, Paper 85-21*.

Geo-Slope International Ltd., 2010. *Stability Modeling with SLOPE/W 2007 Version – An Engineering Methodology*, Fourth Edition. 367 p.

Konrad, J.M. and Law, K.T. 1987. Undrained shear strength from piezocone tests. *Can. Geot. J.,* 24 (3): 392-405.

Lawrence, D.E., Morey, C.R. and Aylsworth, J.A., 1997, Landslides in sensitive marine clays, South Nation River valley; the role of ravines in limiting retrogression. *Proc. 50th Can. Geot. Conf.*, Ottawa, 20-22 OCT 1997, 815-822.

Leroueil, S., Locat, J., Vaunat, J., Picarelli, L., Lee, H., and Faure, R., 1996. Geotechnical characterization of slope movements. *Proc. of the 7th Int'l Symp. on Landslides*, Trondheim, pp.53-74.

Lin, L. and Adams, J., 2010. Strong motions from Nearby seismometer records of the Val-des-Bois, Québec, earthquake of June 23, 2010. *Canadian Hazard Information Service Internal Report 2010-2.1*, 23 pages plus digital Appendix.

Locat, A., Leroueil, S., Bernander, S., Demers, D., Jostad, H.P., Ouehb, L., 2011. Progressive failures in eastern Canadian and Scandinavian sensitive clays. *Can. Geot. J.,* 48: 1696-1712.

Locat, J. and Demers, D., 1988. Viscosity, yield stress, remolded strength, and liquidity index relationships for sensitive clays. *Can. Geot. J.,* 25 (4): 799-806.

Ma, S. and Motazedian, D., 2012. Studies on the June 23, 2010 north Ottawa Mw 5.2 earthquake and vicinity seismicity. *J. Seismol.* 16:513-534.

Mitchell, R.J. and Markell, A.R., 1974. Flowsliding in sensitive soils. *Can. Geot. J.,* 11 (1): 11-31.

Odenstad, 1951. The landslide at Skottorp on the Lidan River February 2, 1946. *Royal Swedish Geotechnical Institute Proceedings,* No. 4, pp. 7-39.

Perret, D., Mompin, R., Demers, D., Lefebvre, G. and Pugin, A.J.-M., 2013. Two large sensitive clay landslides triggered by the 2010 Val-Des-Bois earthquake, Quebec (Canada) – Implications for risk management. Poster, *First Int'l Workshop on Landlsides in Sensitive Clays (IWLSC),* Quebec, Canada, 28-30 Oct. 2013.

Quinn, P.E., Diederichs, M.S., Rowe, R.K., Hutchinson, D.J., 2011. A new model for large landslides in sensitive clay using a fracture mechanics approach. *Can. Geot. J.*, 48: 1151-1162.

Roy, S. and Hawlayder, B., 2017. Finite element modelling of pregressive and retrogressive failure of submarine slopes for varying angle of inclination. *Proc. Of the 70th Can. Geot. Conf.*, Ottawa, Oct. 1-4, 2017.

Tavenas, F., Flon, P., Leroueil, S., and Lebuis, J., 1983. Remolding energy and risk of slide retrogression in sensitive clays. *Symposium on Slopes on Soft Clays, Linkoping*, March 8-10, 1982; Swedish Geotechnical Institute Report No. 17, pp. 423-454.

Tavenas, F. and Leroueil, S., 1987. State-of-the-art on laboratory and in-situ stress-strain-time behavior of soft clays. *Proc. Int'l Symp. on Geot. Engineering of Soft Soils*, Mexico City, Vol. 2: 1-46.

Thakur, V. and Degago, S.A., 2013. Disintegration energy of sensitive clays. *Geotechnique Lett*. 3, 20-25.

Thakur, V., Degago, S.A., Oset, F., Aaboe, R., Dolva, B.K., Aunaas, K., Nyheim, T., Lyche, E., Jensen, O.A., Sæter, M.B., Robsrud, A., Viklund, M., Nigussie, D., and L'Heureux, J.-S., 2014. Characterization of post-failure movements of landslides in soft sensitive clays. J.-S. L'Heureux et al. (eds.), *Landslides in Sensitive Clays: from Geosciences to Risk Management, Advances in Natural and Technological Hazards Research 36*. DOI 10.1007/978-94-007-7079-9_8, pp.91-103.

Trak, B. and Lacasse, S., 1996. Soils susceptible to flow slides and associated failure mechanisms. *Proc. of the 7th Int'l Symp. on Landslides*, Trondheim, pp.497-506.

Wang, B., 2016. Local shaking intensity of an earthquake that triggered Quyon Valley landslide 1020 cal yr BP. *Proc. of the 69th Can. Geot. Conf.*, Vancouver, Oct. 2-5, 2016.

Wang, B., 2017. Geotechnical data from a Champlain Sea clay landslide site at Breckenridge, southwestern Québec. *Proc. of the 70th Can. Geot. Conf.*, Ottawa, Oct. 1-4, 2017.

Yu, H.S. and Mitchell, J.K., 1998. Analysis of cone resistance: review of methods. *J. Geot. & Geoenvir. Engineering*, 124 (2): 140-149.