STUDY OF A LATERAL SPREAD FAILURE IN AN EASTERN CANADA CLAY DEPOSIT IN RELATION WITH PROGRESSIVE FAILURE: THE SAINT-BARNABÉ-NORD SLIDE

A. Locat[1]
[1]*Département de Génie civil, Université Laval, Québec, Québec, Canada, ariane.locat.1@ulaval.ca*
S. Leroueil[1], S. Bernander[2], D. Demers[3], J. Locat[4], L. Ouehb[1]
[2] *Former adjunct professor, Luleå Technical University, Sweden*
[3] *Ministère des Transports du Québec, Section Mouvements de terrain, Service de la géotechnique et de la géologie, Québec, Québec, Canada*
[4]*Département de géologie et de génie géologique, Université Laval, Québec, Québec, Canada*

RÉSUMÉ
Une revue du concept de rupture progressive et l'application de ce concept à une rupture par étalement survenue dans les argiles de l'Est du Canada ont été réalisées dans le but d'obtenir une meilleure compréhension du rôle de la rupture progressive dans ce type de mouvement de masse. Cette étude a abouti à la distinction de deux modes de rupture probables pour les glissements par étalement. Dans le premier cas, l'agent perturbateur est localisé dans le haut de la pente (e.g. construction d'un remblai en haut de pente) et augmente la pression des terres pouvant provoquer une rupture passive dans la pente. Le deuxième cas est celui où l'agent perturbateur est localisé au bas de la pente (l'érosion d'une rivière par exemple) et qu'il diminue la pression des terres dans le talus provoquant possiblement une rupture active. L'application de la rupture progressive à une rupture par étalement a permis le développement de nouvelles idées sur le mécanisme de rupture des grands glissements pour lesquels les méthodes d'analyse de stabilité à l'équilibre limite ne sont pas applicables.

ABSTRACT
A review of the concept of progressive failure in the context of slope stability and an application of this concept to a spread failure in an eastern Canada clay deposit have been made in the search of a better understanding of the role of progressive failure in this type of mass movement. The study led to the identification of two failure modes for lateral spread failures. In the first one, the disturbing agent is located up slope (e.g. presence of a fill on top of the slope) resulting in an increase of earth pressure and possibly entailing a passive failure. In the second one, the disturbing agent is located down slope (e.g. river erosion at the toe of the slope) causing a decrease of earth pressure and possibly an active failure. The application of progressive failure to a lateral spread failure in clay has therefore enabled the development of new ideas regarding the failure mechanism in large landslides where limit equilibrium stability analysis is not applicable.

1. INTRODUCTION

In this study, the progressive failure concept is used to explain the displacement of large volumes of soil on an nearly horizontal failure surface as is occurring in spread failures (Turner and Schuster, 1996, chapter 3). The progressive failure analysis makes use of the strain-softening behaviour of the soil to define deformations in the failure zone and to explain failure propagation along a failure surface. The effect of such behaviour is that when the peak shear strength of an element in a slope is reached reduction in shear stress will be transferred to the neighbouring element because of the loss of shear strength in the adjoining element. This propagation of failure will continue until the triggering load becomes smaller then the resistance of the soil. This process may result in a failure surface where part of the soil material is mobilized beyond its peak shear strength and other is below (Skempton, 1964; Bishop, 1967; Bjerrum, 1967).

The objective of this paper is to define a concept, based on progressive failure formation that will be used in the elaboration of a numerical method for stability analysis in connection with lateral spread failures in clay slopes. The paper begins with a review of the concept of progressive failure, including the description of a progressive failure model for downward progressive slides (i.e. when the disturbing agent is located up slope, in accordance with Bernander, 1978 and 2000; Bernander and Gustås, 1984; Bernander and Olofsson, 1981; Bernander and Svensk, 1982; Bernander *et al.* 1988 and 1989) as well as the modifications made to apply it to upward progressive slides (i.e. when the disturbing agent is located down slope, Bernander, 2005, personal communications; Locat, 2007). An example of upward failure, the Saint-Barnabé-Nord slide, in the eastern Canada clay deposit, will be presented, followed by the application of the model of progressive failure to this slide. The paper emphasis the importance of taking progressive failure into account in stability evaluation of lateral spread failures.

2. PROGRESSIVE FAILURE

2.1 Concept of progressive failure

Skempton (1964) is one of the first to use the residual shear strength in order to explain the large difference between the laboratory shear strength and the shear strength actually mobilised in many failures of natural slopes. Using the

In : J. Locat, D. Perret, D. Turmel, D. Demers et S. Leroueil, (2008).
Comptes rendus de la 4e Conférence canadienne sur les géorisques: des causes à la gestion.
Proceedings of the 4th Canadian Conference on Geohazards : From Causes to Management. Presse de l'Université Laval, Québec, 594 p.

strain-softening behaviour of clays, he clearly describes the failure mode occurring in a progressive failure with the following statement:

"[…] if for any reason a clay is forced to pass the peak at some particular point within its mass, the strength at that point will decrease. This action will throw additional stress on to the clay at some other point, causing the peak to be passed at that point also. In this way a progressive failure can be initiated and, in the limit, the strength along the entire length of a slip surface will fall to the residual value (Skempton, 1964). "

This definition of progressive failure has been the basic idea of the subsequent studies including Bjerrum (1967) who summarizes three important conclusions from the work of Skempton (1964): 1) the residual strength probably depends on the size, shape and mineralogical composition of the particles of soil. 2) The average shear stress along a surface of rupture computed from a number of slides in overconsolidated plastic clays is closer to the residual strength of the clay than to the peak strength. Analyses of some slides in natural slopes proved that the shear stresses at failure could be almost equal to the residual shear strength. 3) Slides in overconsolidated clays begin with the progression of a failure surface and, when sufficient time is available for the development of a rupture surface by progressive failure, the stability depends on the residual shear strength only.

On the basis of this concept, it was proposed by Bjerrum (1967) and Bishop (1967) that stresses and deformations acting in a slope do not mobilise the same shear strength along a potential failure surface but that it varies from peak shear strength to residual strength. It was suggested that the mobilised shear strength acting on a failure surface prior to failure was determined by the strain-softening behaviour of the soil. The soil in the potential sliding mass is therefore subjected to local failure, when elements of soil reach their peak resistance, prior to global failure. Riedel (1929) proposed a mechanism of shearing that illustrates well the development of a failure surface. The study describes how shearing can create single separate shear surfaces that are slightly inclined in the direction of principal shear, but do not permit any movements. It is only at the last stage of shearing that those separate shear planes are linked together by displacement discontinuities in the direction of imposed shear. This mechanism of failure can be used to illustrate how local failure along a potential slip surface can evolve in a global failure.

Urciuoli *et al.* (2007) illustrate progressive failure along a weak layer in a strain-softening material. Figure 1a shows the shear stresses mobilised along a failure plane parallel to the ground surface due to an excavation in an infinite slope. The excavation mobilises the residual shear stress (τ_r) of the soil at point A (Figure 1a), at the toe of the excavation. Further up slope, the peak shear strength (τ_f) is reached and the mobilised shear stress gradually decreases to its value prior excavation (τ_i, at point B on Figure 1a, where the excavation has no effect). It is worth noting that this change in shear stress decreases the earth pressure naturally present in the slope (E_i). Therefore, an excavation at the toe of a slope creates a shear zone along a potential failure plane propagating up slope which can cause local failure. This effect has been observed in the field by Burland *et al.* (1977) who observed the deformations induced by a shear zone at the toe of an excavation in clay.

Figure 1b presents the same infinite slope previously described but, instead of being subjected to an excavation, it is subject to an overburden pressure at the top of the slope. The changes in the shear stress distribution are similar; the only difference being that the shear zone is propagating down slope, increasing *in situ* earth pressure further down slope, possibly inducing a passive failure.

In Sweden, this last mode of failure, triggered by a disturbance localised at the top of a slope, has been well studied by Bernander and his colleagues (Bernander, 1978 and 2000, Bernander and Gustås, 1984, Bernander and

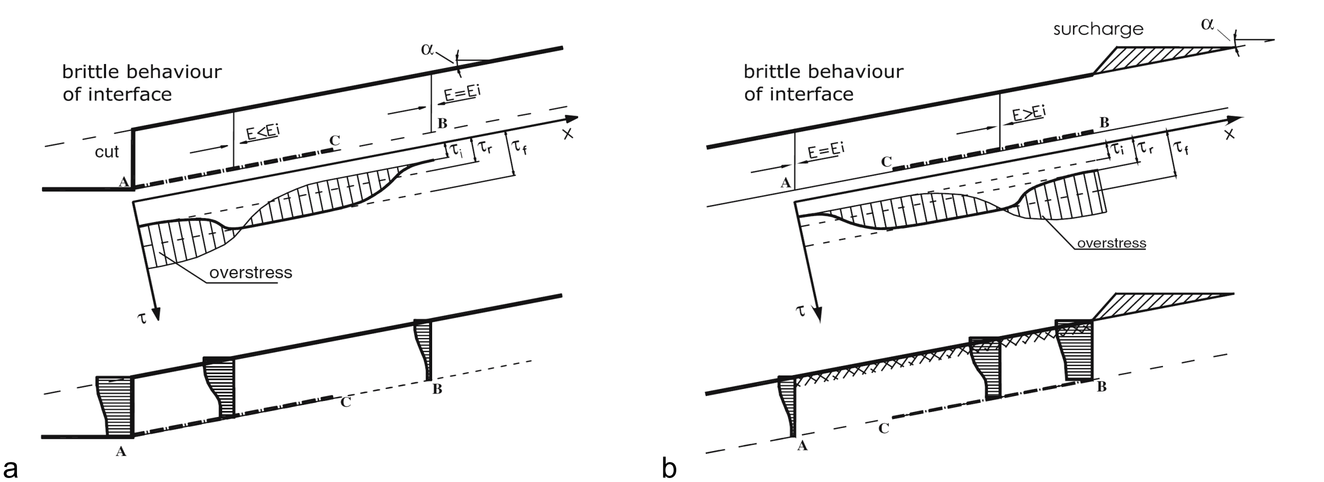

Figure 1: Shear stresses (upper graphs) and displacements (lower graphs) mobilised by a) an excavation and b) an overburden load along a discontinuity parallel to the ground surface in an infinite slope (Urciuoli *et al.* 2007).

Olofsson, 1981; Bernander and Svensk, 1982; Bernander *et al.* 1988 and 1989). These studies led to the development of a progressive failure model that enables the modeling of shear stresses and displacements along a potential failure zone generated by the presence of an overburden pressure at the top of a slope.

2.2 Bernander and colleagues' progressive failure concept

The studies cited in the above section considered progressive failure in stability problem by applying a mean shear resistance along a discrete potential failure surface. As explain above, that was done in the objective to take into account the deformations and the strain-softening behaviour of the soil, necessary in the modeling of progressive failure. In those cases, the stability of the slope was still defined by a conventional global safety factor, equal to the ratio of the mean shear resistance to the mean shear stress. That conventional safety factor can give too high values when peak shear resistance is used and too low values when using the residual shear resistance for slides occurring on long horizontal failure surface. Instead, Bernander *et al.* (1988 and 1989) and Bernander (2000) proposed an analysis that models the stresses and the shear stains in the entire zone subjected to additional stress and deformations, while keeping shear deformations and the down-slope displacements mutually compatible in this zone. This model, originally intended for the analysis of downward progressive failures, is summarized in this section and has been applied in the present study to a spread failure that occurred in eastern Canada clays.

After Bernander (2000) and Bernander *et al.* (1988 and 1989) as a force N, which is parallel to the failure surface and acting at the top of the slope, increases, shear stresses are gradually mobilised along a presupposed failure zone and deformations occur. At the point of load application the change in shear stresses in terms of (τ_x - τ_o), where τ_0 is the *in situ* shear stress and τ_x is the resulting total shear stress after disturbance, increases proportionally with the force N. At some point, the peak shear strength will be mobilised by the increasing force N, inducing deformations (γ_f) in the slope. If the force N increases and the stress in the soil pass beyond its peak shear strength (τ_f), displacement occurs at the point of application of the load and to the deformation is added a new component, the slip (δ_S) along the failure plane.

Figure 2 shows the situation when the shear resistance at x = x1 is reduce beyond the peak shear strength to the *in situ* shear stress, i.e. τ_{x1} = τ_o, in an infinite slope (Figure 2a and b). At a distance x = x1, all the available shear resistance of the soil is mobilised along the failure surface and the force N_{x1} = N_{cr} has reached its maximum possible value that can be applied without causing any instabilities. The shear stresses in the potential failure zone, induced by the force N, decrease down slope to the point where the force N has no effect (point X3 on Figure 2c). The distance (x1 - x3 = L_{cr}) represents the distance necessary for the mobilised strength along the failure plane to be in equilibrium with the

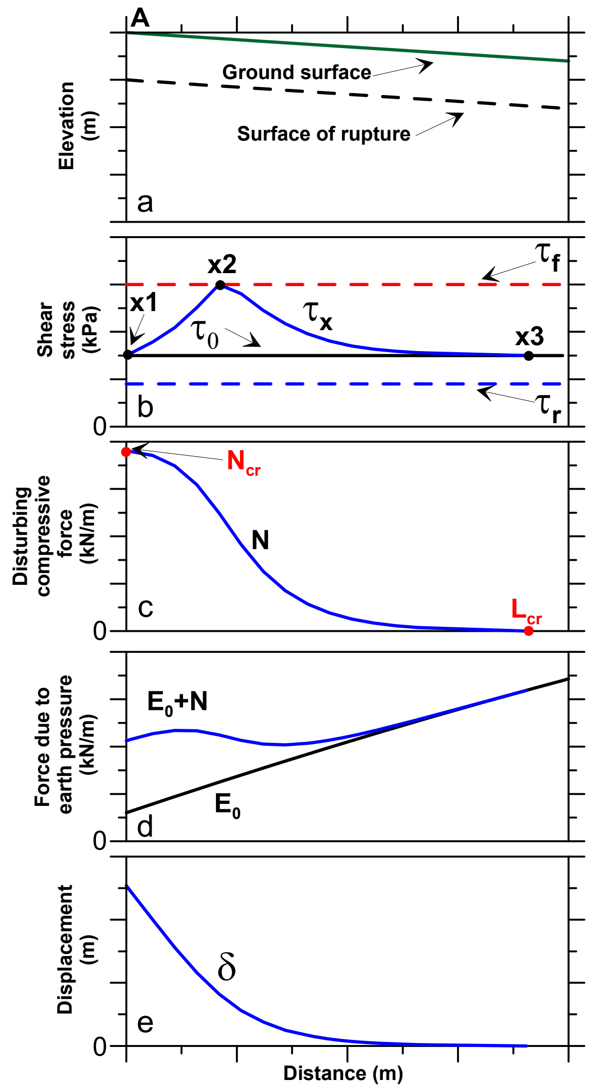

Figure 2: Schematic illustration of the initiation of local instability induced by an overburden load on top of the slope (point A) after Bernander (2000) and Bernander *et al.* (1988 and 1989).

force N_{cr} applied at point A, up slope. It may be noted that the curve representing (τ_x - τ_o) is in principle an asymptote to the curve for τ_o implying that the point defined by x3 is virtually infinitely far away. As L_{cr} (as well as x3) is theoretically of infinite length, a practical limit for the L_{cr} may be defined by a point x3, where ratio of $\Delta\tau$ = (τ_x - τ_o) / τ_f is given a predetermined value, not significantly affecting the outcome of the analysis.

Figures 2d and 2e represent respectively the force due to earth pressures before (E_0) and after the disturbance (E_0+N) in the slope (Figure 2d) and the displacement (δ) induced by the disturbance (Figure 2e).

If the force N_{cr} is exceeded, inducing instability in the slope, failure may propagate progressively down slope in which case global failure can occur. Figure 3 presents the case where global progressive failure occurs in a slope reaching an almost horizontal ground. The diagrams illustrate the shear distribution and the associated pressure wave subsequent to a global progressive failure. If the earth pressures, at the foot of the slope, exceed passive Rankine resistance ($E_{P\ Rankine}$, i.e. $E_0 + N > E_{P\ Rankine}$, see Figure 3c), this will merely form a transient stage, which immediately develops into the actual global slide movement. The soil mass between x0 and x1 (Figure 3a), below which the residual shear resistance has been mobilised, slides along the failure surface and increases the earth pressure further down the slope. The soil down slope is therefore compressed by this sliding mass. As Bernander (2000) and Bernander *et al.* (1988 and 1989) explain, "this movement should, however, not be understood as a regular slide but rather as a progressive pressure wave, by which unbalanced forces in the zones subject to deformation softening are transmitted to less sloping ground further down the gradient."

On the other hand, if the $E_{P\ Rankine}$ is not exceeded (i.e. $E_0 + N < E_{P\ Rankine}$), the progressive failure will only result in the redistribution of earth pressures and cracking in the active zone of the slide area. Hence in that latter case, equilibrium can be reached if the destabilizing force (N) added to the initial earth pressure (E_0 + N) remains lower than the passive Rankine pressure ($E_{P\ Rankine}$).

The transition between the initiation of an instability (Figure 2) and a global failure (Figure 3), identified with this method, is of enormous practical importance and can explain why a slope, that may have been stable for thousands of years and would have continued to remain stable, failed due to a minor temporary disturbance.

2.3 Assumptions for eastern Canada spread failures triggered by erosion

As described in the previous sections, Bernander and colleagues' model for progressive failure has primarily been developed for failures, where the disturbing agent is located up slope (downward progressive failure). In eastern Canada clay deposits, the majority of large slides, as flow slides and spread failures, are triggered by erosion at the toe of the slope, generating an upward progressive failure. It is therefore interesting to adapt the Bernander method of analysing progressive failure to spread failures slide, thus modeling in a similar way the shear zones, the forces and the deformations induced by a disturbing agent at the toe of the slope.

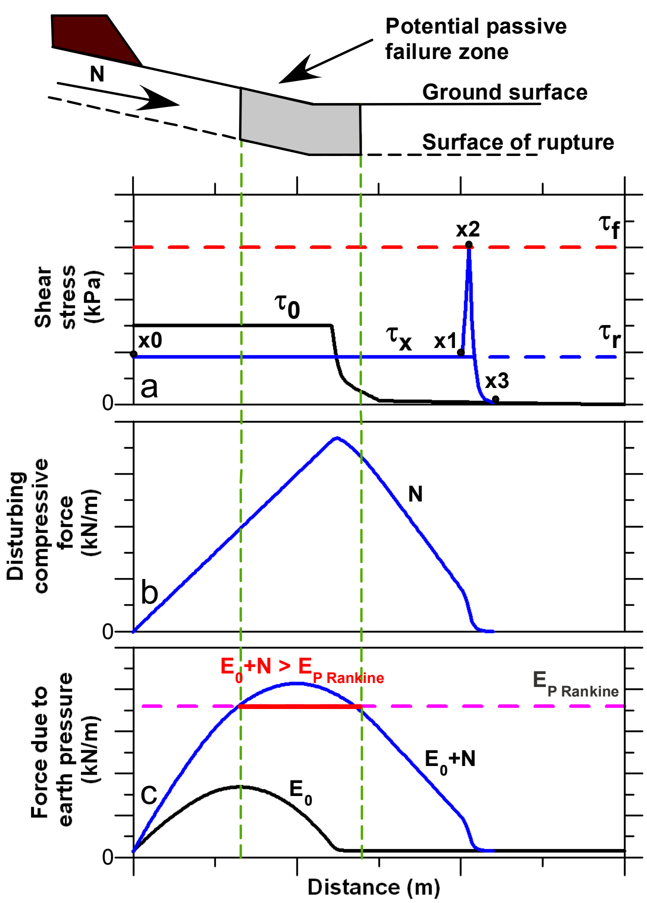

Figure 3: Schematic diagram of global failure induced by an overburden load on top of the slope (point A) after Bernander (2000) and Bernander *et al.* (1988 and 1989).

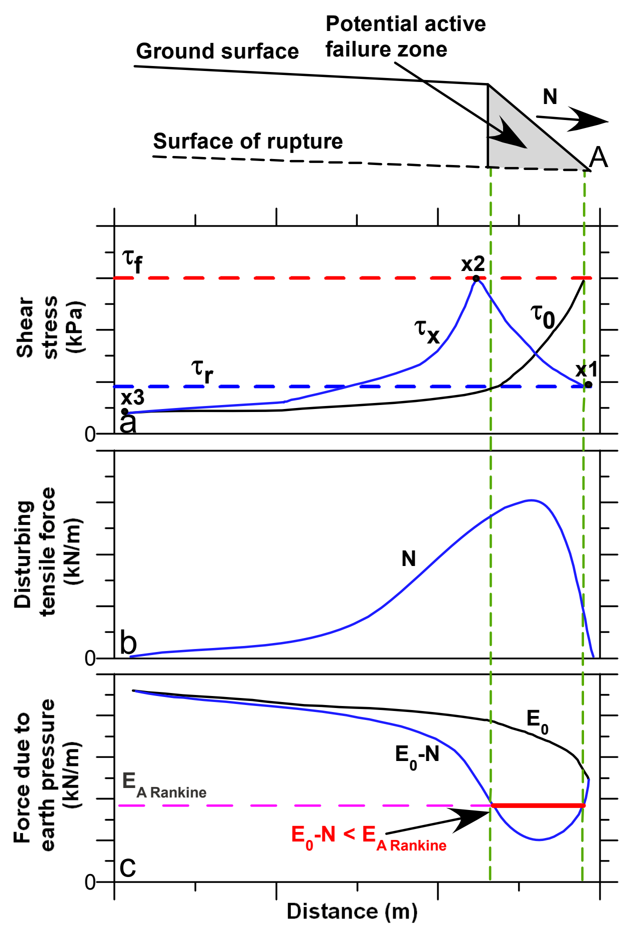

Figure 4: Shematic representation of the upward progressive failure model (Bernander, 2005, personal communication).

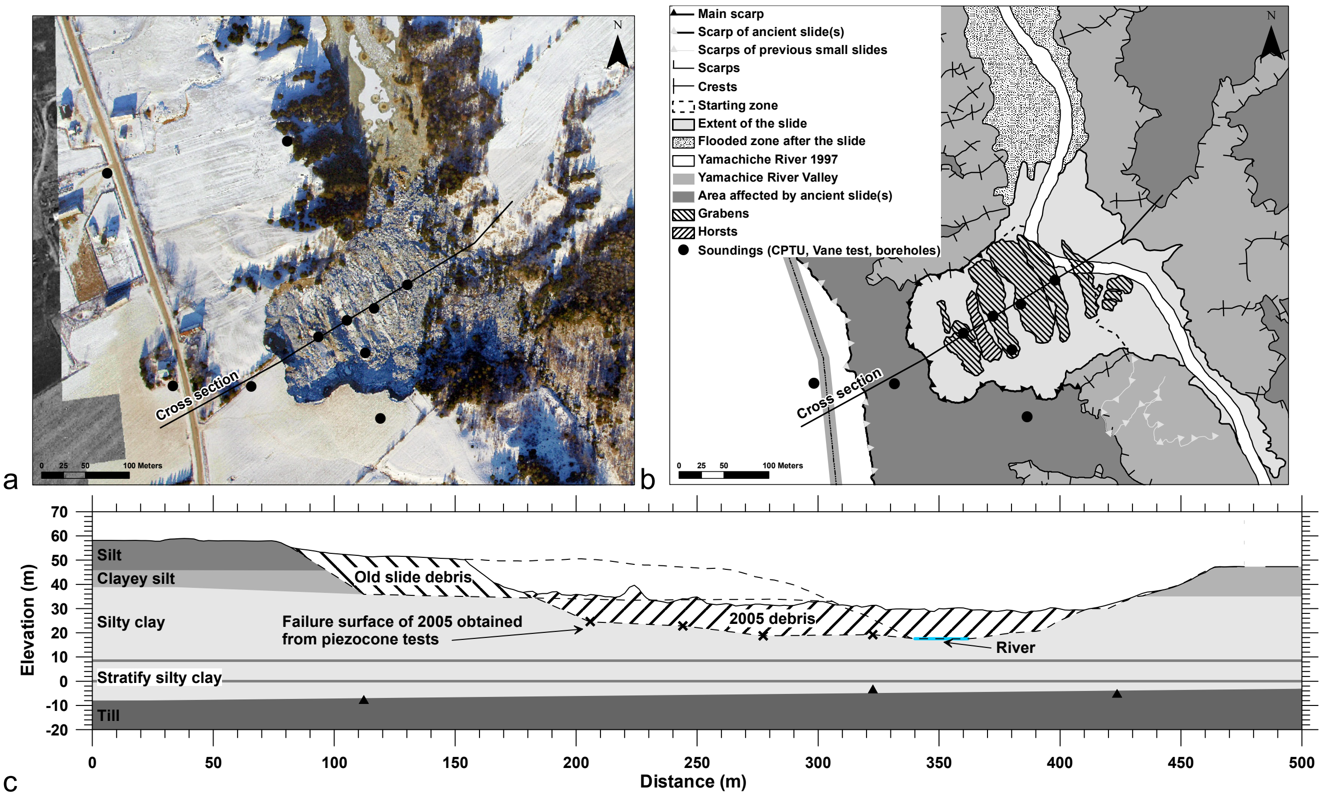

Figure 5: a) Aerial photo (Source: MTQ), b) morphological features and c) cross section of the Saint-Barnabé-Nord slide.

In order to apply the Bernander concept to spread failures triggered by erosion, some modifications have to be made regarding the failure mechanisms and the propagation of the shear zone. In this case, as Figure 4 shows, the disturbing force (N) does not act as a compressive force but as a tensile force. The force N therefore has negative values in the direction of the slope. In the case when the disturbing agent is located near the toe of the slope (Figure 4c) horizontal earth pressures are therefore reduced. Slope failures starting from the toe of the slope consequently form a process of active failure (Bernander, 2005, personal communications).

Figure 4 illustrates the effect of erosion at the toe of the slope (point A), briefly presenting the same information as Figure 1a, i.e. the decreasing earth pressure causing deformations along a failure surface. The force N is taken positive in the upslope direction, but represents a tensile force actually decreasing the earth pressure initially present in the slope.

Another major difference in upward progressive failure compared to downward progressive failure is the conditions for global failure. In upward progressive failure, no final equilibrium conditions exist once a local instability has generated a progressive failure, as no support is present at the foot of the slope to stop the downward movement (Bernander, 2005, personal communications). In fact, when a failure occurs at the toe of the slope, the instable mass slides down the failure surface and nothing is there to form a passive thrust that can stop the movement, unless the displaced mass is limited by some nearby obstacle such as the opposite side of a valley for example. As failure in this case is taken to be an active mechanism, one hypothesis is that progressive failure will occur when active Rankine pressure is reached ($E_{A\ Rankine}$,see Figure 4c). The propagation will stop at the point where the resulting earth pressures after disturbance are higher then the active Rankine pressure.

Now that the progressive failure mechanism has been introduced, the next section describes a spread failure that occurred in eastern Canada clay deposit: the Saint-Barnabé-Nord slide, which was likely triggered by erosion at the toe of the slope.

3. UPWARD SPREAD FAILURE – THE 2005 SAINT-BARNABÉ-NORD SLIDE

The Saint-Barnabé-Nord slide (Figure 5a) occurred, along the Yamachiche River, on the 10th December 2005, about 20km north-west of Trois-Rivières (Québec, Canada) in the municipality of Saint-Barnabé-Nord. The slope located in the Champlain sea sediments was 30m high with a maximum dip toward the river of about 25°. Prior to the slide, the toe of the slope was partly in contact with the erosive side of a river meander (Figure 5b). The debris of the finished slide completely blocked the river creating a temporary lake that disappeared during the next spring.

A few days after the slide, the Quebec Ministry of Transportation performed airborne LIDAR scans over the area enabling a detailed study of the morphology of the slide. Figure 5b shows the morphological features of the Saint-Barnabé-Nord slide resulting from that study. One of the interesting features of this figure is the presence of the scarp of an ancient slide with its backscarp about 100m west of the 2005 slide and dated to about 2300 BP (Locat 2007). Upper part of the 2005 slide therefore implied debris of this ancient slide.

The slide geometry corresponds to a retrogression length of 180m, a width of 160m and an average thickness of 22m. A total volume of 219 000m^3 of debris spread onto the valley floor over an area of 70 000m^2. The displaced mass broke into horsts and grabens of intact clay (Figure 5b). The main horst, located in the western part of the debris, is 6m high and 78m wide (Locat, 2007). The grabens are clearly identifiable on Figure 5a by the soil covered with snow that was kept intact on top of the grabens during the slide.

A few months after the event, a thorough investigation of the site was carried out by the Quebec Ministry of Transportation: 12 piezocone tests, one vane test and two boreholes were performed near the slide. It enabled the detailed reconstruction of the stratigraphy of the slope (Figure 5c). Five different layers of soil resting on the rock bottom were identified by the different soundings (Locat, 2007). Starting from the top, a 10m layer of silt covers an 8m layer of clayey silt, deposited during the retreat of the Champlain Sea. Underneath, 28m of intact silty clay rest on 16m of stratified silty clay covering 20m of what could be a glacial deposit (till) (Figure 5c).

Five of the twelve piezocone tests were located inside the 2005 crater. They were very useful in locating the failure surface. It is possible to see on Figure 5c that, at the toe of the slope, the failure surface is located at about the same elevation as the Yamachiche River. This fact implies that erosion at the toe of the slope is a possible cause of failure

The two boreholes provided core samples for characterisation tests (Locat, 2007). The water content was found to be about 40% in the more silty zones, i.e. for the first 12m of the profile and between elevations 11 and 7m, and ranges between 40 to 60% in the more clayey zones between elevations 33 and 11m. The clayey soil has a sensitivity ranging from 38 to 71 near the 2005 failure surface. This sensitivity is not related to low remoulded shear strength (3 to 12kPa between 26 and 19m of elevation), as is normally the case in classical sensitive clays, but by very high intact shear strength (100 to 120kPa between 26 and 19m of elevation). The results of oedometer tests compared with the vertical effective stress indicate that the soil is overconsolidated with an overconsolidation ratio (OCR) varying between 3.2 at the top of the clay layer to 1.6 at bottom.

No obvious cause was found explaining the timing of the present failure; no major precipitation during the days before the slide or earthquakes were reported. It is proposed that failure was initiated either by erosion of the Yamachiche River, acting only on a part of the slope toe, or by fatigue. Because the triggering factor is located at the toe of the slope and as the failure surface is almost horizontal, the slope likely failed by an upward progressive failure. This may be the case for most of spread failures in Québec.

4. MODELING UPWARD SPREAD FAILURES

4.1 The model

To analyse the upward spread failure, Bernander's downward progressive failure model was used and modified so it can be applied to spread failure triggered at the toe of the slope producing an upward progressive failure. The same equations are used but the major difference is the meaning of the force N generated by the disturbing agent which is, in an upward progressive failure, a tensile force reducing earth pressures in the slope (see section 2.3).

The first step in modeling the upward progressive failure is to calculate the shear stresses acting naturally in the slope (*in situ* shear stresses τ_0). As shown on Figure 4 (black line), for a slope subject to erosion, the shear stress is increasing towards the crest of the slope and reaches a maximum at the toe of the slope, where the erosion is present.

When the *in situ* shear stresses are known, it is possible to evaluate the stress change due to the disturbing agent. The additional shear stresses ($\Delta\tau$) are then determined over a length Δx by an iteration process considering the strain-softening behaviour of the soil. The actual shear stresses after disturbance (τ_x) can therefore be evaluated by adding or subtracting $\Delta\tau$ to τ_0.

Once those shear stresses are known, the resulting changes of earth pressure can be calculated with the following equation (see blue line in Figure 4b):

$$N = \int_{xa}^{xb} \left(\tau_0 - \tau_x\right) dx \qquad [1]$$

N is the tensile force in relation with the change in shear stresses acting over a length Δx (xa - xb) along the potential failure surface and it has the effect of decreasing the prevailing earth pressure (E_0). In the diagram (Figure 4a and b), this force mathematically represents the area between the *in situ* shear stress curve and the curve representing the shear stress after disturbance over the length of the shear zone. It reaches its maximum value when all the available shear strength of the soil is fully mobilised (τ_0-τ_x = 0) (as explained for downward progressive failure in section 2.2 Figure 2b and c).

It is therefore possible to model the changes in shear stresses, in earth pressure and the deformation resulting from loss of support at the toe of the slope along a portential failure surface. More details on the model are provided by

Bernander (2000), Bernander *et al.* (1988 and 1989) and Locat (2007).

4.2 Testing the model on the Saint-Barnabé-Nord slide

In order to study the applicability of this upward spread failure model, it was preliminarily applied on the Saint-Barnabé-Nord slide by Locat (2007). At this stage, the analysis was not performed to explain the actual failure that occurred at Saint-Barnabé-Nord. The objective of this study was to test the model in order to get more information on the relevant parameters acting during a spread failure triggered by erosion and to get an example of the kind of results obtained from this analysis. The results obtained are presented in Figure 6. Figure 6a shows the geometry of the slope and of the failure surface considered in the modeling.

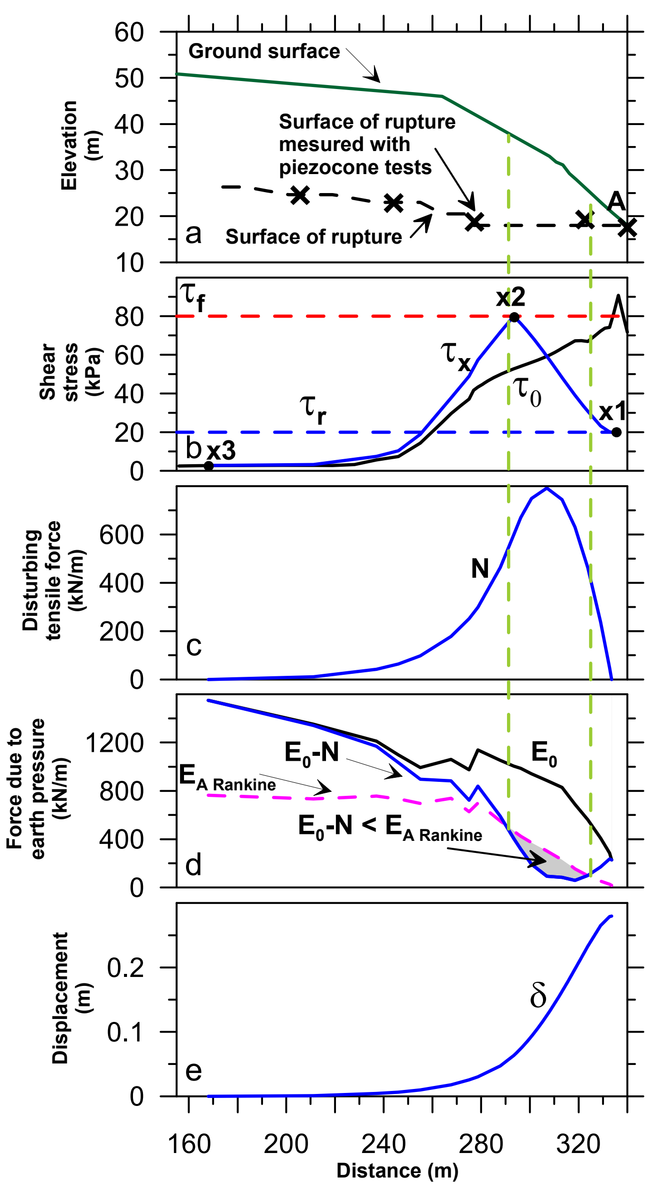

Figure 6: Tests results of the upward spread failure model on the case the Saint-Barnabé-nord slide (Locat, 2007).

The actual failure surface as measured with piezocone tests is also presented with "x".

First, the state of stress acting in the slope was evaluated by modeling the excavation created by river erosion and the dissipation of induced water pressures by consolidation (Locat, 2007). The modeling was performed using the SIGMA/W program of GeoStudio. With this modeling it has been possible to evaluate vertical and horizontal effective stresses (σ_v', σ_h') with realistic coefficients of earth pressure at rest (K_0) and to obtain the horizontal shear stresses (τ_0, Figure 6b) as well as the earth pressures (E_0, Figure 6d) acting naturally in the slope. The results of this analysis were used as initial conditions in the modeling of the progressive failure and are plotted on Figure 6b and d.

The strain-softening behaviour of the soil was taken as obtained from direct simple shear tests performed on samples taken close to the failure surface elevation (Locat, 2007). The results gave a peak shear strength (τ_f) of 80kPa at a shear deformation (γ_f) of 3.5%. The large deformation shear strength (τ_r) was approximated to 20kPa because the shear tests did not reach steady sate conditions. For modeling purposes, the large deformation resistance was taken to be mobilised at a shear displacement of 0.3m. These resistance values are illustrated by horizontal dash lines in Figure 6b.

In order to model the impact of a local instability triggered by river erosion, the *in situ* shear stress (τ_0) was decreased to the residual shear resistance (τ_r) at the toe of the slope (point x1, Figure 6b). Using the upward spread failure model described above and taking the strain-softening behaviour of the soil into account, it was possible, not only to model the shear zone induced by this disturbance, but also the following parameters, namely: the resulting shear stress (τ_x) (Figure 6b), the change in earth pressure (N_x) as calculated with equation [1] (Figure 6c) and the displacements (δ_x, Figure 6e). The decrease of *in situ* stresses to residual strength caused the propagation of a tensile load that has a maximal value of 792kN/m at 307m and a displacement of 0.28m at the toe of the slope (point x1, see Figure 6c and e). The destabilising force was then subtracted from the *in situ* earth pressure, i.e. (E_0 - N), to obtain the earth pressure after disturbance (Figure 6d).

With the intention to evaluate the possibility of failure, the earth pressures after disturbance were compared to the active Rankine pressure. It can be seen on Figure 6d that there is a zone, at the bottom of the slope, were active failure is likely to occur between x=288m and x=324m, i.e. over a distance of 36m, due to the fact that the earth pressures resulting from erosion are lower than the active Rankine pressure. This failure could be triggered by a displacement of only 0.28m (see Figure 6e) without any application of a sustained force (Bernander, 2000). As explained before, this modeling does not explain the full extent of the failure that actually occurred at Saint-Barnabé-Nord, but explains the formation of the active failure along a 36m long horizontal failure surface. More modeling including the study of the different parameters involved in the failure process has to be done in order to model the formation of an entire failure surface as seen for spread failures.

5. CONCLUDING REMARKS

The concept of progressive failure has been studied and two types of progressive failures have been defined: upward and downward spread failures. In order to illustrate this concept, the Saint-Barnabé-Nord slide has been examined. The slide occurred in a deposit consisting of overconsolidated ($3.2 > OCR > 1.6$) Champlain Sea sediments with high intact shear strength (~100kPa) at the failure surface. The soil failed by the mechanism of spreading, characterised by the formation of horst and grabens, still present after the movement. The blocks of essentially intact clay slid on an almost horizontal failure surface. This slide is thought to be an upward spread failure as it seems to have failed because of toe erosion.

Downward progressive failures have been well studied by Bernander and his colleagues (Bernander, 2000 and Bernander *et al.* 1988 and 1989). In the current case, his progressive failure analysis has been modified to model upward progressive failures (Bernander, 2005, personal communication, and Locat, 2007) and has been tested on the Saint-Barnabé-Nord slide (Locat, 2007). With this modeling, the propagation of the shear zone caused by erosion at the toe of the slope was evaluated as an upward spread failure. The analysis showed that only 0.28m of displacement, caused by erosion at the toe of the slope, could be enough to trigger a first active failure over a distance of 36m along the actual horizontal failure plane. This analysis suggests the following comments:

- The knowledge of the state of stress (σ_v', σ_h' and K_0) acting in a slope subjected to erosion is necessary for the stability analysis of spread failures. This knowledge comes from a good field study of the ground water regime in a slope, the assessments of the *in situ* states of stress and of the coefficient of earth pressure at rest and the effect of time and fatigue phenomena.
- More investigation of case studies and more modeling of finished slides have to be performed to be able to model a complete failure surface of a potential spread slide and to make sure that the right failure criteria are used. A finite element model may have to be developed and apply to several cases of spread failures.
- Shear tests must be carried out to properly determine the strain-softening that can be used as an input in the modeling.
- Further modeling has to take the formation of horsts and grabens into account, maybe as a post-failure mechanism of spread failures.

6. ACKNOWLEDGEMENTS

The authors would like to thank the reviewers of this paper Réjean Couture, from the Geological Survey of Canada and Hans Petter Jostad, from the Norway Geotechnical Institute. The investigation of the Saint-Barnabé-Nord has been made possible by the Quebec Ministry of Transportation, especially with the help of Catherine Thibault.

7. REFERENCES

BERNANDER, S. 1978. Brittle failures in normaly consolidated soil. *Väg-och Vattenbyggaren*, No. 8-9, pp.49-52.

BERNANDER, S. 2000. Progressive landslides in long natural slopes, formation, potential extension and configuration of finished slides in strain-softening soils. *Licentiate thesis*, *Luleå University of Technology*, 105p.

BERNANDER, S. 2005. Analytical FDM-model for uphill progressive (retrogressive) slides - theory. *Personal communications.*

BERNANDER, S. and GUSTÅS, H. 1984. Consideration of *in situ* stresses in clay slopes with special reference to progressive failure analysis. *IV^th^ International Symposium on Landslides*, Toronto, pp. 235-240.

BERNANDER, S., GUSTÅS, H. and OLOFSSON, J. 1988. Improved Model for Progressive failure Analysis of Slope stability. *Nordic Geotechnical Conference*, *NGM 88*, Oslo. 5p.

BERNANDER, S., GUSTÅS, H. and OLOFSSON, J. 1989. Improved model for progressive failure analysis of slope stability. *Proceedings 12^th^ ICSMFE*, Rio de Janeiro, pp. 1539-1542.

BERNANDER, S. and OLOFSSON, I. 1981. On formation of progressive failures in slope. *Proceeding of the 10^th^ ICSMFE Conference*, Stockholm, pp.357-362.

BERNANDER, S. and SVENSK, I. 1982. On the brittleness of soft clays and its effect on slope stability. *Väg-och Vattenbyggaren*, No. 7-9, pp.3-7.

BISHOP, A. W. 1967. Progressive failure - with special reference to the mechanism causing it. *Proceedings of the Geotechnical Conference*, Oslo, Vol. 2, pp. 142-150.

BJERRUM, L. 1967. Progressive failure in slopes in overconsolidated plastic clay and shales. *Terzaghi Lectures*, *Journal of Soil Mechanics and Foundations Division*, *ASCE*, pp. 139-190.

BURLAND, J. B., LONGWORTH and MOORE, J. F. A. 1977. A study of ground movement and progressive failure caused by a deep excavation in Oxford clay. *Géotechnique*, Vol. 27, No. 4, pp. 557-591.

LOCAT, A. 2007. Étude d'un étalement latéral dans les argiles de l'Est du Canada et de la rupture progressive, Le cas du glissement de Saint-Barnabé-Nord. *M.Sc. Thesis, Université Laval*, Québec, 280p.

RIEDEL, W. 1929. Zur mechanik geologischer brucherscheinungen, Ein Beitrag zum Problem der `Fiederspalten'. *Centralblatt fuÈr Mineralogie, Geologie, und Paleontologie*, Part B, pp. 354±368.

SKEMPTON, A. W. 1964. 4th rankine lecture : long-term stability of clay slopes. *Géotechnique*, Vol. 14, No. 2, pp. 77-102.

TURNER, A. K. and SCHUSTER, R. L. 1996. Landslides investigation and mitigation. Special Report 247, *Transportation, Research Board, National Research Council*, 674p.

URCIUOLI, G., PICARELLI, L. and LEROUEIL, S. 2007. Local soil failure before general slope failure. *Geotechnical and Geological Engineering*, Vol. 25, No. 1, pp. 103-122.