

Overview of Retrogressive Landslide Risk Analysis in Sensitive Clay Slope

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Abstract: Sensitive clays are known for producing retrogressive landslides, also called spread or flowslides. The key characteristics associated with the occurrence of these landslides on a sensitive clay slope must be assessed, and the potential retrogressive distance must be evaluated. Common risk analysis methods include empirical methods for estimating the distance of potential retrogression, analytical limit equilibrium methods, numerical modelling methods using the strength reduction technique, and the integration of a progressive failure mechanism into numerical methods. Methods developed for zoning purposes in Norway and Quebec provide conservative results in most cases, even if they don't cover the worst cases scenario. A flowslide can be partially analysed using analytical limit equilibrium methods and numerical methods having strength reduction factor tools. Numerical modelling of progressive failure mechanisms using numerical methods can define the critical parameters of spread-type landslides, such as critical unloading and the retrogression distance of the failure. Continuous improvements to the large-deformation numerical modeling approach allow its application to all types of sensitive clay landslides.

Keywords: sensitive clay; quick clay; landslide; stability analysis; failure type; retrogressive distance

1. Introduction

Clays are constituted of fine-grained soil material having a grain size of less than approximately 0.05 mm. A clay soil is cohesive with a plastic behaviour. The word clay can also be used to indicate specific minerals, such as kaolinite or illite. Here, we refer to clay as a fine-grained soil that is plastic, cohesive, and that may or may not contain clay minerals [1].

A soil is called sensitive if its sensitivity (S_t) is greater than 1. The sensitivity is the ratio of the undisturbed soil strength to the remoulded shear strength (Equation (1)) [2]:

$$S_t = \frac{\text{Undisturbed undrained shear strength}}{\text{Remoulded undrained shear strength}} \quad (1)$$

The term “quick clay” also has multiple definitions. In Canada, clays having a sensitivity value above 16 are called quick clays [2]. In Sweden, this term is applied to a clays having a sensitivity greater or equal to 50 and a fully remoulded shear strength of less than 0.4 kPa [2,3], whereas in Norway, quick clays are defined as clays having a remoulded shear strength of less than 0.5 kPa and a sensitivity of 30 or more [2,4] (Table 1). In this paper, the expression ‘sensitive clay’ is used to represent both sensitive and quick clays. Sensitive clay is characterized by the marked difference of behaviour between its undisturbed and remoulded stages, as it behaves more or less as a liquid when disturbed.

Table 1. Definition of quick clay in three countries.

| Parameters | Canada | Sweden | Norway |
|------------------------------------|--------|-----------|-----------|
| Sensitivity (S_t) | >16 | ≥ 50 | ≥ 30 |
| Remoulded shear strength (S_r) | - | <0.4 kPa | <0.5 kPa |

Generally, sensitive clays are found in areas that were ice covered during the last glaciation and drowned by a marine invasion that followed immediately the glacial retreat [5]. Eastern Canada and the coasts of Scandinavia are the two main regions characterized by sensitive clays. Understanding slope stability in this soil type is vital for ensuring the security of people and infrastructures in these areas.

Sensitive clays are formed in a post-glaciation context. In North America, the retreat of the ice sheet, which started around 21,400 years ago [6], was followed by a marine invasion along much of the isostatically depressed coastline. The clay deposited on the coastal seafloor contained an important quantity of polyvalent cations that flocculated to form a relatively strong structure. Following the isostatically forced regression, the clay was exposed to leaching by fresh meteoric water. This leaching led to an increase of the inter-particle repulsive forces. According to the classification of Varnes [7] and its modification by Hungr et al. [8], sensitive clays can produce a progressive spread or a flow landslide, also called a flowslide. Both categories of landslide are highly retrogressive. A third type of sensitive clay landslide, a progressive translational (flake) slide, falls into the flowslide category and occurs when the source material is almost entirely remoulded and flows out of the slide area. In this paper, we only address the first two types, although we remain conscious of this third type.

Retrogression occurs when the failure surface progresses upslope (Figure 1). In Stage 1, a failure surface forms at the bottom of the intact slope (Figure 1a). In Stage 2, after an initial landslide has occurred, a new failure surface forms further upslope (Figure 1b). In Stage 3, the landslide continues its progression via sequential failure surfaces that progress upslope (Figure 1c). This landslide mechanism is characterized by a series of retrogressive rotational slips that collectively form a final landslide crater [9,10]. This sequence of events can be considered as a flowslide type of landslide [11].

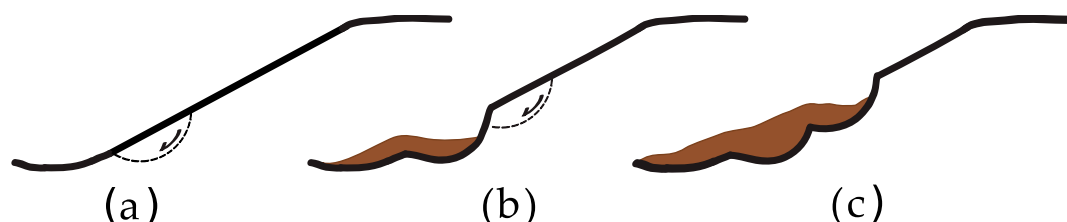


Figure 1. Schematic cross-sections of a landslide with retrogression. (a) A failure surface forms toward the base of the intact slope; (b) a new failure surface forms further upslope; (c) the landslide continues its progression following sequential failure surfaces that progress upslope.

Recently, the progressive failure mechanism has been identified as one of the key processes for explaining retrogressive landslides and spread in sensitive clays [12–14]. This mechanism considers that the complete failure surface develops before any significant movement occurs (Figure 2) [12]. The dismemberment of the soil layer above the failure surface appears retrogressive. In reality, however, this is not the case (Figure 3) [12,15,16]. This mechanism is consistent with the horst and graben structures that are often observed within sensitive clay spreading.

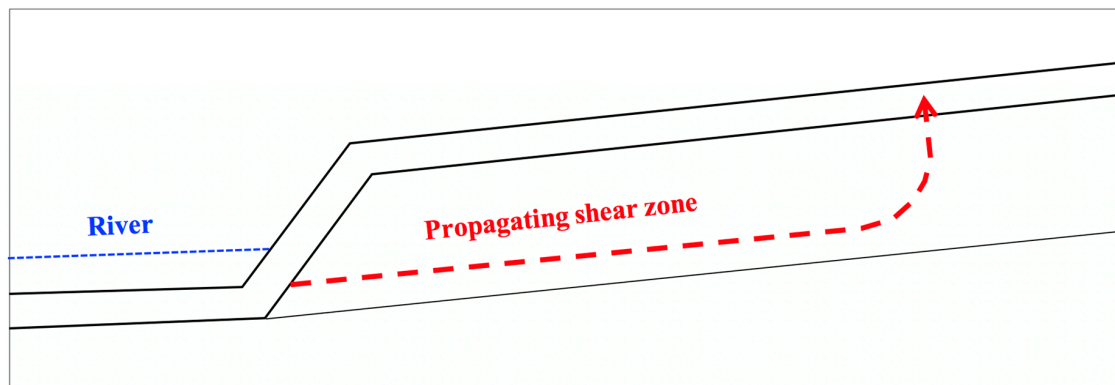


Figure 2. Development of a failure surface via a weak zone caused by progressive erosion at the foot of the slope [12].

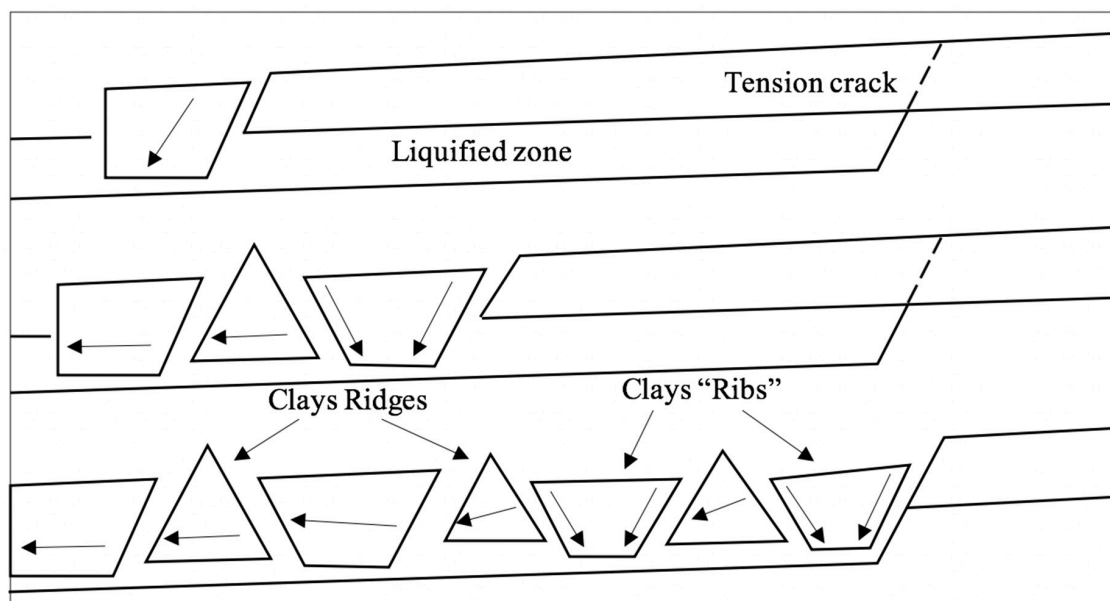


Figure 3. Progression of a “retrogressive” landslide in sensitive clays via the dismembering of an undisturbed soil monolith evolving on a layer of liquefied soil [12].

Unloading the toe of a slope favours shearing along the potential failure surface and can ultimately initiate slope failure. The shear stress may then increase up to peak shear strength. When failure occurs, the shear strength decreases to a residual level and thereby allows the failure to propagate. The qualifier critical is used to define the unloading at the toe of the slope that is required to initiate slope instability and allow the failure to propagate.

Retrogressive landslides in sensitive clays often occur following a single major perturbation, such as an earthquake, or they may occur after a large number of annual load cycles at a geological time scale, with the final trigger related to a minor and seemingly innocuous perturbation [13]. These failure events can therefore occur without any obvious warning signs. The potential retrogressive distance of these landslides can also reach values much greater than 100 m [11]. This uncertainty in their behaviour emphasizes the importance of determining where these landslides can occur.

Multiple methods have been developed to prevent or to predict the occurrence of landslides. The identification of slope at risk by empirical zonation methods are commonly used in Norway and Quebec. Limit equilibrium techniques, for example, have been used for decades as an analytical approach [17–19] and with the advent of computers, numerical modelling using finite element and finite difference techniques began to be applied to slope stability problems and elements of other

engineering fields. This paper reviews existing methods used in landslide risk analysis and their application to sensitive clays slopes.

2. Materials and Methods

Landslide risk analysis relies mainly on assessments of slope stability analysis, which can be performed via either analytical methods, i.e., limit equilibrium methods, or numerical modelling. In stability analysis, an equilibrium must be satisfied in terms of total stress (σ), it being the sum of effective stress (σ') and pore pressure (u). Shear strength (τ) is the maximum shear stress that a soil can withstand, and this value is related to total stress. In drained soils, pore pressure is not affected by a change of load because water can flow freely. In undrained soils, the pore pressure is affected by a change of load because there is no water flow. Total stress analyses are not applicable to drained soils. In these cases, we use the effective stress, that is the pore pressure is subtracted from total stress to evaluate the effective stress on the shear surface [20]. Before the stability analysis is carried out, the hazard area must be identified and the slopes that are prone to large retrogression should be pinpoint.

2.1. Retrogression Potential

Given the complexity of estimating the distance of retrogression, the first step requires identifying all parameters that affect the propagation of a landslide. Geertsema and L'Heureux [21] highlighted the various controls of sensitive clay slopes; they subdivided these controls into three categories: geometric, material, and external controls. Material controls include the undrained (S_u) and remoulded shear strength (S_r), as well as sensitivity (S_t). These controls are the underlying requisites for producing large retrogressive landslides, as material needs to pass from a relatively strong state to a weaker state. Landslide type affects the dimensions of a slide as spread materials are less mobile than those of flowslides. The S_u of spread tends to have a higher value than that of flowslides [22], illustrating that S_u is a crude predictor of the relative dimensions of a landslide.

The ability of clay to be remoulded, entirely or partially, during a landslide controls the potential for retrogression in sensitive clays. As part of this idea, the stability number (N_s) (Equation (2)) is also a part of the material controls. The stability number was developed by considering the potential energy available to remould the clay of a slope, and it has been used to predict retrogression distance.

$$N_s = \gamma H / S_u, \quad (2)$$

where N_s = stability number, γ = unit weight of the soil (kN/m^3), H = height of the slope (m), and S_u = undrained shear strength (kPa).

Geertsema and L'Heureux [21] illustrate that the remoulding index (I_r), the speed, and the mobility of the depleted mass are other important parameters in the material controls over the dimensions of a sensitive clay landslide. Overall, these parameters represent the ability of clay to move quickly out the slide area to allow (or not) a greater progression of a landslide.

The geometric controls of slides include the slope and orientation of the ground surface, as well as the bedding planes, valley width and geometry, depth of failure, and the presence and incision depth of bounding streams [21]. Other structures, such as the proximal presence of the bedrock, can also act as barriers to a possible slide. External controls, including the presence of dense vegetation on the slope, can vary greatly and affect the mobility of the soil mass.

Proposed approaches for estimating the distance of retrogression include the methods of Mitchell and Markell [23] and Mitchell [24]. Both of these studies use the stability number, as well as the methods of Carson [25] and Quinn et al. [13].

Mitchell and Markell [23] assessed the correlation between the stability number (Equation (2)) and the retrogressive distance of 75 documented retrogressive landslides, flowslides, and spreads. They observed that the retrogressive distance increases as a parabolic function of the stability number

(Figure 4). This function was further modified by Mitchell [24] to improve its applicability (Figure 4), and by Carson [25], who considered the ratio of the retrogressive distance over the height of the slope and the sensitivity of the clay. Quinn et al. [13], on the other hand, based their assumptions on the progressive failure mechanism, suggesting that the final failure surface is developed before the first landslide movement has occurred. As explained by Demers et al. [11], however, none of these abovementioned methods produce accurate results. The application of these methods to estimate retrogression distance yields rather conservative estimates having minimal precision. Figure 4 illustrates the absence of a correlation between the stability number and retrogression distance as observed in historical cases and ancient scars. The predicted results may differ from reality by a factor 10, even when considering a progressive failure mechanism, such as that proposed by Quinn et al. [13].

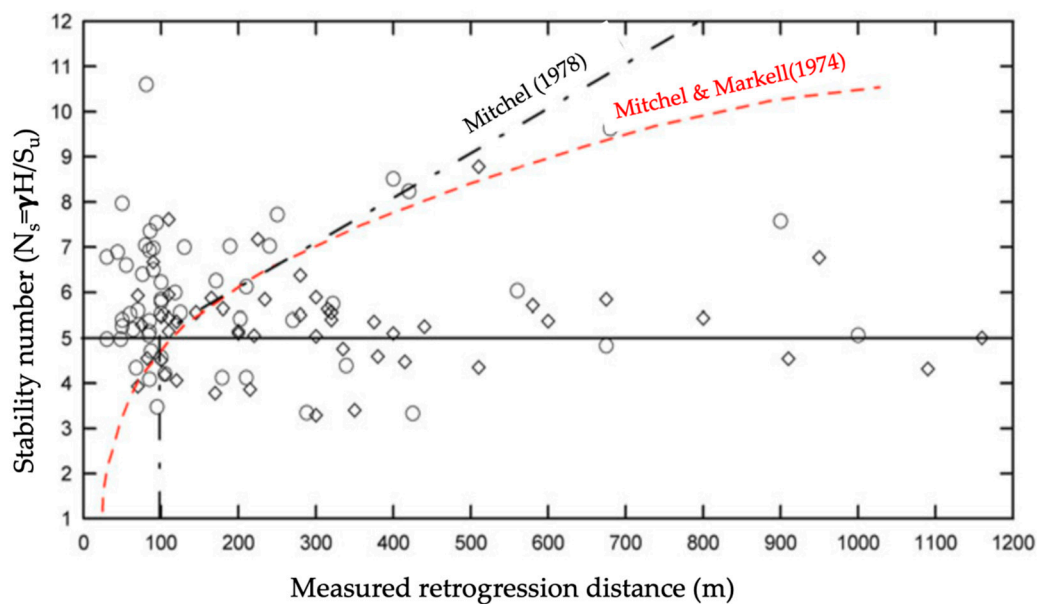


Figure 4. Retrogression distance as a function of the stability number for historical cases (circles) and ancient scars (diamonds) [11].

On the other hand, specific methodologies for estimating retrogression and the runout distance of sensitive clay flowslides are currently being applied with success for sites in Norway and Quebec. As presented by Turmel [26], the first step requires determining whether a site is prone to flowslide. The height of the slope must be sufficient enough for the potential energy to remould the material, and the consistency of the clay must be fluid enough to flow as a liquid. In Eastern Canada, sensitive clays require a liquidity index (I_L) of at least 1.2 and a S_r smaller than 1 kPa [27].

Many parameters have been proposed for identifying material that is most prone to producing large retrogressions (Table 2).

Table 2. Parameters that render a slope susceptible to large retrogression.

| Parameters | Proposed by |
|------------------------------|-------------------------|
| $N_s > 4$, if $I_p \sim 10$ | Leroueil [28] |
| $N_s > 8$, if $I_p \sim 40$ | |
| $S_r < 1$ kPa | Mitchell & Markell [23] |
| $I_L > 1.2$ (flowslide) | Leroueil et al. [29] |
| Rapidity number > 8 | Söderblom [30] |
| $Q > 15\%$ | Thakur and Degago [4] |

2.2. Zoning of the Slope at Risk

In Norway, there are three primary methods that have been developed for zoning retrogressive distances. The 1:15 method, used by the Norwegian Water Resources and Energy Directorate (NVE), consists of establishing the limit of the retrogressive distance at 15 times the height of the slope from the toe of the slope [31], based on Karlsrud et al. [32]. The NIFS (Natural hazards – infrastructure for floods and slides) method rates all sites having 40% sensitive clays found above a critical failure surface as having a S_r less than 1 kPa and classifies these sites into three possible outcomes: 1:5, 1:10, and 1:15 classes [33]. The propagated region of retrogressive failure is then drawn from the base of the critical failure surface, unlike the 1:15 method where it is drawn from the base of the toe of the slope. The runout distance (E), as determined via the NIFS, is method recommended [34] to be 1.5 times the retrogressive distance (R) in open terrain and three times this distance in channelized terrain. The Norwegian Geotechnical Institute (NGI) method draws the same 1:15 line from the base of the critical failure surface as that proposed by the NIFS line. However, when the slip surface is not within sensitive clays, they apply a 1:3 to 1:2 line, depending on the soil properties and pore-water pressure conditions [26,31]. L'Heureux [35] demonstrated that some ancient landslides in Norway have a ratio larger than 1:15. From a probabilistic point of view, these slides are considered as being quite rare.

In Quebec, the MTMDDET (Ministère des Transports, de la Mobilité durable et de l'Électrification des Transports) applies a statistical approach where ancient flowslides within a sector of interest are used [27]. Slides that have been diverted by natural barriers, such as ancient scars, are excluded from the analyses. This approach relies on two major assumptions. First, ancient flowslides that have occurred in an area are a product of the area's specific conditions. Second, areas sharing similar characteristics produce flowslides that have similar retrogressive distances. From these two assumptions, a moving average of retrogressive distances is produced along the study slopes. The runout distance is calculated using Equation (3) for open terrain [26]:

$$E = \frac{8 \times W \times R}{\pi}, \quad (3)$$

where E = radius of the circular shape, W = width of the flowslide determined empirically, based on the average W/R ratio for the scars of the sector, and R = retrogressive distance.

For the runout distance in channelized terrain, it is hypothesized that a quarter of the debris will flow upstream, and the remaining three quarters will flow downstream. Knowing the dimensions of the crater and channel, it is possible to calculate the length of the debris trail [26].

As presented by Turmel [26], both the Quebec and Norwegian approaches produce conservative results and are safe in a majority of cases. Nonetheless, the potential worst-cases scenarios are not covered by these approaches.

2.3. Stability Analysis: Analytical Methods

For a given slope in a hazard area, its stability can be evaluated. Analytical methods are a way to achieve it. Those methods are based on the principle of limit equilibrium and they consider successive slices of soil of finite thickness and compute the forces and moments acting on each slice. The equations relating these variables can be solved, and a range of assumptions are applied to reduce the number of unknowns. From these equations, we can determine the shape of the failure surface based on the forces, the moments, and their application [19]. Each method can estimate a safety factor (SF) using the ratio of a material's shear strength to the mobilized shear stress along the likely slip surface (Equation (4)). A safety factor of 1 or less indicates a failure.

$$SF = \frac{\sum \text{Shear strength}}{\sum \text{Shear stress}}. \quad (4)$$

Shear strength can be determined using the Mohr–Coulomb failure criterion [1]:

$$\tau_{ff} = \sigma'_{ff} \tan \varphi' + c' \quad (5)$$

where τ_{ff} = shear strength (kPa), σ'_{ff} = normal effective stress on the failure surface (kPa), φ' = internal friction angle ($^{\circ}$), and c' = Cohesion (kPa).

Shear stress can then be calculated using various stability analysis methods. The earliest limit equilibrium methods date back centuries and relate to Coulomb's corner method [36]. The considered failure surface is a plane, and the slope is subdivided into one or more blocks that lack internal movement [19]. Fellenius [17] later introduced the slice method for slope stability analysis (Figure 5). The long sides of the blocks are vertical, and the slope is subdivided into several slices. The slice method was then modified by others, including Bishop [18].

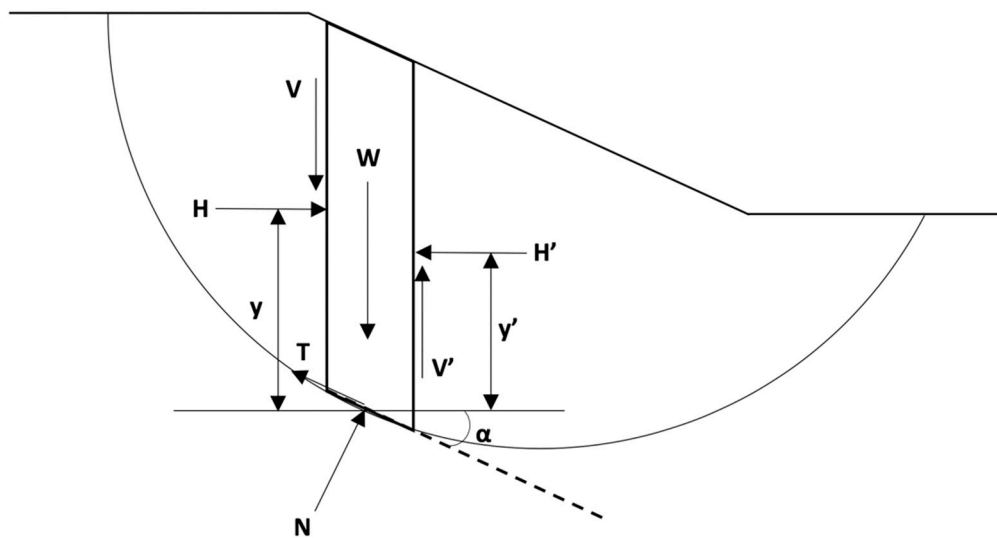


Figure 5. The unknown parameters in the slice method [17].

There are more unknown variables than equations in the slice method. A number of simplifying assumptions must then be included in each slice method to obtain results. Simplification usually requires selecting one or more unknown variables and setting their value or orientation, thereby making them known variables. Thus, the equilibrium can be satisfied with the sums of forces in the x ($\sum F_x$) and y ($\sum F_y$) axes and the sum of moments ($\sum M$), where they should all equal zero. It is important to note that several simplifications lead to the inapplicability of one or two equilibrium conditions. Other assumptions are based on the geometry of the failure surface.

In a soil slope stability problem, the failure surface is complex and can be approximated by a circular or another curved shape, such as used by Fellenius [17] and Bishop [18]. Collin [37] introduced the idea of a circular failure surface, and this concept was then used in an equation by Fellenius [17] to reduce the number of variables needed to be considered when analysing a slope stability problem. The circular failure surface and other commonly used slice methods are summarized in Table 3.

Table 3. Commonly used slope stability analysis methods and their assumptions (modified from Sivakugan & Das [38]).

| Procedure | Assumption and Characteristics | Equilibrium Conditions Satisfied | | |
|---|---|----------------------------------|----------------|--------------|
| | | $\sum F_x = 0$ | $\sum F_y = 0$ | $\sum M = 0$ |
| Ordinary method of slices/Fellenius method [17] | Circular slip surfaces only; interslice forces are zero | No | No | Yes |
| Simplified Bishop [18] method | Circular slip surfaces only; interslice shear forces are zero | No | Yes | Yes |
| Corps of engineers [39] modified Swedish method | Slip surfaces of any shape, interslice force is parallel to the ground or inclined at an angle equal to the slope of a line connecting the crest and the toe (called average embankment slope) | Yes | Yes | No |
| Lowe and Karafiath's [40] method | Slip surface of any shape; interslice force is inclined at an angle of $(1/2 \alpha + \beta)$ | Yes | Yes | No |
| Janbu's [41] simplified method | Slip surface of any shape, interslice shear force is zero | Yes | Yes | No |
| Spencer's [42] method | Slip surface of any shape; interslice forces are parallel with an unknown inclination | Yes | Yes | Yes |
| Morgenstern-Price [43] method | Slip surface of any shape; interslice shear forces are related to the interslice normal forces by $X = \lambda f(x)E$, where λ is an unknown scaling factor, and $f(x)$ is an assumed function with prescribed values at slice boundaries | Yes | Yes | Yes |
| Sarma's [44] method | Slip surface of any shape; interslice shear force is related to the interslice shear strength by $X = \lambda f(x)S_y$, where λ is an unknown scaling factor, $f(x)$ is an assumed function with prescribed values at slice boundaries, and S_y is the available shear force depending on c' and φ' along the slice boundaries | Yes | Yes | Yes |

The assumptions commonly used in limit equilibrium methods include a regular shape for the failure surface, e.g., plane, circular, and spiral logarithm, simplified equilibrium equations, as well as the presence or absence (or considered as negligible) of certain forces in the analysis, e.g., interslice forces, and their orientation or relation to other variables. The great advantages of these methods include the speed of calculation, allowing the analysis to be repeated thousands of times in a few seconds, and the possibility of considering uncertainties for several factors and parameters, such as the shear strength of the materials and groundwater level. On the other hand, the required simplifying assumptions decrease the accuracy of the results, sometimes to an inappropriate level characterized by a high difference in results between methods.

These methods can estimate the safety factor for a possible retrogressive flowslide composed of successive circular slides. The limit equilibrium approach, however, neglects some important aspects of sensitive clay slopes susceptible to progressive failure. Analytical methods consider the soil as a rigid-perfect plastic material. This assumption is inadequate for correctly evaluating the stability conditions of a slope susceptible to progressive failure that is accompanied by deformations which occurred before the overall failure. In particular, considering the peak strength parameters for stability analysis could severely overestimate the stability conditions. As presented by Skempton [45], the shear strength value obtained using these methods does not necessarily represent the actual values produced during the slide. It is more likely to be a value situated between the peak strength and the residual strength. Skempton [45] and Burland et al. [46], using back analysis of actual landslides on natural slopes, showed that the strength parameters mobilized during an overall failure could vary markedly for each point along the failure surface, ranging from residual to peak values. Skempton [45], in particular, defined a “residual factor” to quantify the magnitude of the progressive failure. Therefore, the utilization of peak strength, rather than a more accurate value, can greatly overestimate the safety factor.

Also, Burland et al. [46] demonstrated that after an excavation of a sensitive clay slope, the shear surface is more likely to develop horizontally into a non-circular surface, which is not always considered by limit equilibrium methods. These behaviours, typical of progressive failure mechanisms, highlight the limits of analytical approaches when evaluating spread landslides.

2.4. Stability Analysis: Numerical Methods

The arrival of powerful computers gave rise to numerical modelling, making it possible to rapidly resolve complex equations. During the second part of the 20th century, multiple numerical computation methods emerged, such as the finite element method (FEM) [47], with the aim of obtaining more realistic results than with the then-existing methods.

Numerical methods have been applied to a wide range of engineering fields and problems, including soil mechanics. FEM also allows the use of the Mohr–Coulomb failure criterion. A common goal of FEM in slope stability analysis is to evaluate a variety of possible slip surfaces and determine the most likely failure surface. This approach differs from limit equilibrium methods where the failure surface is assumed.

FEM relies on a displacement-based approach, which is commonly used in geotechnical engineering. In addition to FEM, boundary element methods (BEM) and the finite difference methods (FDM) can simulate homogeneous domains. In heterogeneous environments, discrete element methods (DEM) are more commonly used. For example, their use in cases of rock slope stability analysis [48]. All these methods use a mesh to create a simplified model. The difference between these methods is the discretizing format of the differential equations and how the equations are resolved.

In the following sections, we present a number of modifications to the displacement-based approach when using numerical methods to analyzed slope stability.

2.5. Shear Strength Reduction Technique

Multiple approaches based on numerical methods have been developed to determine the critical failure surface and safety factor. These approaches include those of Zienkiewicz and Corneau [49], Donald & Giam [50], and Matsui and San [51], the latter having proposed the “strength reduction technique”. Meanwhile, Griffiths and Lane [52], Chang and Huang [53], and many others have demonstrated the efficacy of this technique.

The strength reduction technique is based on the Mohr–Coulomb failure criterion; the most important parameters in this numerical method are therefore the same as those of the analytical limit equilibrium approach. These parameters are total unit weight γ , the shear strength parameters c' and φ' , cohesion and the angle of internal friction, the geometry of the problem, and the groundwater condition [52]. The strength reduction technique consists of dividing the shear strength parameters of soil c' and φ' by the strength reduction factor (SRF), as presented in Equations (6) and (7). The SRF gradually increases until convergence cannot be achieved within the iteration limit; this situation therefore means that a failure has occurred. The SRF at failure is then equivalent to the safety factor.

$$c'_f = \frac{c'}{\text{SRF}} \quad (6)$$

$$\varphi'_f = \arctan\left(\frac{\tan\varphi'}{\text{SRF}}\right). \quad (7)$$

The main advantage of the strength reduction technique is that, unlike the limit equilibrium technique, assumptions are not required either in regard to the forces acting on the sides of the slices or in relation to the location and shape of the slip surface [52]. The critical surface and the associated safety factor, which is equivalent to the SRF in this case, are computed by numerical modelling.

The particular conditions that must be considered to explain the retrogressive aspects of this type of slip are rarely discussed in the reports that describe these slides [10,54]. The literature recognizes

the particular behaviour of retrogressive slides [8,10,54,55]. Until recently, however, no modelling approach had been proposed for spread landslides, as described in the next section.

2.6. Modelling of the Progressive Failure Mechanism

Recently, the modelling of sensitive clay spreads has advanced due to a new interpretation of this type of slide. The importance of plasticity and brittleness of clay in the progressive failure mechanism was first presented by Bjerrum [56], who was studying overconsolidated clay. He suggested that each landslide in overconsolidated clay is preceded by the formation of a continuous sliding surface through progressive failure due to a large amount of recoverable strain energy. Clays having weak diagenetic bonds recover the strain energy immediately after unloading. On the other hand, clays having strong diagenetic bonds recover this energy gradually through near-surface weathering, which destroys these bonds. Quinn et al. [12] (see Figures 2 and 3) initially proposed the landslide formation model in sensitive clays, and this model was reworked subsequently by Quinn et al. [13,15], Locat et al. [14], and Dey et al. [57]. This approach includes an analytical model to determine the parameters associated with the propagation of the shear zone [13], and these parameters are then integrated into finite element modelling [15]. This FEM recognizes, among other things, that the brittleness of non-liquefied clay and erosion at the foot of the slope have a significant effect on the occurrence of landslides in sensitive clays.

The progressive failure mechanism is thus based mainly on the brittleness of the clay, its post-peak behaviour, and the effect of the unloading of the slope toe. The brittleness is either the rate at which the strength decreases or the amount of strength loss after the peak, as shown in Figure 6. The more brittle the clay, the more likely progressive failure will propagate. The brittleness can also be related to the nominal displacement of the clay $\bar{\delta}$, which represents the fraction of the shear displacement along the shear surface required to fully remould the clay. The post-peak behaviour reflects the residual strength left after the post-peak decrease, whereas the unloading of the toe of the slope can stem from various causes, including erosion, previous smaller slides, or excavation.

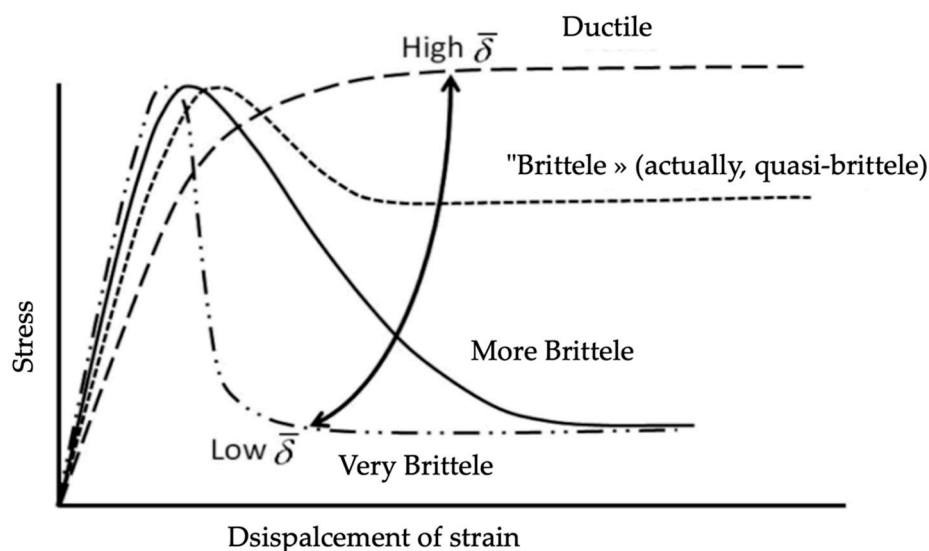


Figure 6. Representation of the brittleness of clay [15].

Numerical modelling of a progressive failure mechanism consists of a two-step procedure. Step 1 involves calculating the in situ initial stresses on the slope. This is followed by the second step of initiating the propagation of progressive failure. In Wang and Hawlader [58], this step is subdivided into two substages, the triggering factor and the continuity of the experiment. The triggering factor can be a downslope unloading or an upslope loading.

The first step can be achieved by conducting an elastoplastic finite element analysis, such as the coupled Eulerian–Lagrangian (CEL) approach used by Dey et al. [57] and Wang and Hawlader [58]. This large-deformation FE modelling technique overcomes the non-convergence of the solution that occurs due to the significant mesh distortion around the failure plane in most FEM. Undrained conditions are used given the very short time required for sensitive clay landslides to occur. A post-peak softening behaviour in the undrained loading is defined for the clay. The hardening soil model can also be applied, such as the model presented by the software PLAXIS [59] and used by Locat et al. [14].

The initiation of the propagation of progressive failure depends on the method used in Step 1. In an approach similar to that of CEL, it is possible to model an eroded block at the toe of the slope or an upslope embankment load. In Step 2, this block is displaced laterally at a constant speed to induce movement within the rest of the slope [57], or this block is simply removed to initiate the propagation of shear bands [49]. In a hardening soil model, the initial shear stress is input within a finite element program, such as the finite element software BIFURC developed by the Norwegian Geotechnical Institute [60,61].

These methods continue to improve and have been recently applied to simulate large-scale landslides in sensitive clays due to earthquake loading [62] or to model complex landslides [63] for better understanding the behaviour of shear stress during these slides. Although these models were initially developed for sensitive clay spread, Wang and Hawlader [58] successfully applied them to downhill progressive slides and flowslides.

3. Discussion

Risk analysis in sensitive clays slope is undertaken to prevent or minimize the risk of two principal landslide types: sensitive clay spread and sensitive clay flowslides. These two types of landslide develop from different mechanisms. Flowslides are a retrogressive sequence of circular slides and are more likely to occur in sensitive clays. Spreads result from the progressive formation of a single failure, and these failures can occur in less sensitive clays than required for flowslides [11]. Generally, it is difficult to predict the type of landslide that may be produced on a given slope. The type will depend highly on the composition of the slope, e.g., the presence (or not) of a stiff crust above the sensitive clay layer, and the triggering factor, e.g., the unloading at the toe and loading at the top [58]. This uncertainty underlines the importance of analysing each slope for both types of slide and triggering factors.

Methods have been developed for zoning purposes in Norway and Quebec, and these approaches provide conservative results in most cases [26]. Those are based on empirical factor developed to identify material susceptible to produce large retrogression. When a slope meets certain criteria, as a liquidity index (I_L) of at least 1.2 and a S_r smaller than 1 kPa for example, a limit of retrogressive distance is established for this given slope. Therefore, a hazard zone is delimited. For further analysis, stability can be assessed.

The limit equilibrium methods neglect some important aspects of sensitive clay slopes susceptible to progressive failure. The shear strength value obtained with these methods does not necessarily represent the actual values produced during a slide; it is more likely to be a value between peak strength and residual strength. The utilization of the peak strength rather than a more accurate value can greatly overestimate the safety factor. These results illustrate the limits of analytical approaches for evaluating progressive failure.

In the case of flowslides, using an analytical method or applying SRF to numerical methods can determine the initial rotational slip surface and its safety factor. However, in the case of a spread landslide, the limit equilibrium methods do not considerate the loss of strength after the initiating of the deformation and the horizontal propagation of the shear bands. Wang and Hawlader [58] demonstrated how to model the effect of unloading at the toe of the slope on the propagation of shear bands. For this, they applied a large-deformation FE modelling technique.

The use of numerical modelling has been adapted to the progressive failure mechanism approach for the stability analysis of spreads. Numerical modelling has recently been used for the analysis of

sensitive clay spreads by focusing on the behaviour of the propagating shear bands [14,57,58,62,63]. It then becomes possible to simulate a weak zone and its potential distance of propagation [15] or to simulate scenarios of triggering factors and their effects on failure initiation. Dey et al. [57] were among the first to model the conditions required to obtain horst and graben structures in sensitive clay slides. However, the safety factor is not mentioned in this study as the behaviour of the weak zone was the primary focus of interest.

4. Conclusions

The main objective of this paper is to provide a better understanding of the risk analysis of sensitive clay slopes. Our overview of the commonly applied methods makes it possible to develop some generalizations.

Empirical determinations of retrogression potential are used in the mapping of hazards. Such maps have been developed for specific areas in Norway and Quebec. All these methods have limitations, such as the necessity to have ancient scars near the study area for the MTMDET approach, the presence of flowslides in Norway that exceed the 1:15 ratio, and the lack of development of these methods for spreads. Nonetheless, empirical approaches remain in use in these countries. They also represented the first step to conduct in a landslide risk analysis study in sensitive clay slope.

Analytical methods for stability analysis can be used for calculating the safety factor of the possible initial slip in the case of flowslides. The simple use of SRF in numerical modelling can achieve the same result and identify the critical slip surface. Combined with the risk area zone delimited with empirical methods, it is possible to answer those two questions: Will it slide? How far can it retrogress?

Numerical modelling via the progressive mechanism approach can help determine the critical unloading at the toe of a slope, the retrogressive distance of a failure, and even the pattern of horst and graben structure after a slide. The applicability of a large deformation FE modelling technique can be extended to other types of sensitive clay landslides because this approach focuses on the propagation of shear bands. The most recent studies based on this modelling technique continue to further our understanding of the mechanisms behind all types of retrogressive landslides in sensitive clays.

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Abbreviation

Symbols

| | |
|-----------|---|
| S_t | Sensitivity |
| S_r | Remoulded shear strength |
| σ | Total stress |
| σ' | Effective stress |
| u | Pore pressure |
| τ | Shear strength |
| S_u | Undrained shear strength |
| N_s | Stability number |
| I_r | Remoulding index |
| I_L | Liquidity index |
| Q | Quickness |
| N | Normal force at slice base |
| T | Shear force at slice base |
| H | Interslice normal force |
| V | Interslice shear force |
| y | Location of interslice normal force |
| NVE | Norwegian Water Resources and Energy Directorate |
| NIFS | Natural hazards – infrastructure for floods and slides |
| NGI | Norwegian Geotechnical Institute |
| MTMDET | Ministère des Transports, de la Mobilité durable et de l'Électrification des Transports |
| E | Runout distance |
| R | Retrogressive distance |
| SF | Safety factor |
| FEM | Finite element methods |
| BEM | Boundary element methods |
| FDM | Finite difference methods |
| DEM | Discrete element methods |
| SRF | Strength reduction factor |
| CEL | Coupled Eulerian–Lagrangian |

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