Landslide hazards in sensitive clays: Recent advances in assessment and mitigation strategies

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ABSTRACT
Sensitive clays, when provoked by artificial or natural causes, have led to several landslide disasters throughout history. Such landslides sometimes involve massive soil movements in the order of millions of cubic meters and represent a major threat to human life, constructed facilities, the infrastructure, and the natural environment. Several efforts are being made in Norway at different levels to ameliorate society's ability to cope with landslides in sensitive clays. In doing so, numerous assessment and mitigation strategies are being developed and implemented. Here, geoscientists (geotechnical engineers and geologists) have a purposeful and systematic role to play by providing reliable tools that can be used for the assessment and mitigation of landslide in sensitive clays. To exemplify this, this paper will present some research results dedicated to sensitive clays from an ongoing intra-governmental research program, Natural Hazard—Infrastructure, Flood and Slides, in Norway. The paper first presents some of the challenges related to the landslides in sensitive clays. Next, it addresses principles regarding requirements with a partial safety factor for local stability as well as a partial safety factor or percentual improvement for large landslides. This paper also points out the manner in which regulations impose increased attention to safety when sensitive clay is encountered. An attempt is made to describe the indicator concerning the prediction of the post-failure movement of landslides. Finally, this paper addresses some ongoing fundamental research being carried-out in Norway to develop new mitigation measures.

Keywords: landslide, sensitive clays, local and overall stability, run-out, natural slopes

1 INTRODUCTION

The work presented in this paper mainly originates from the national research program titled *"Natural Hazards: Infrastructure, Floods, and Slides (NIFS),"* which involves a close collaboration between the Norwegian Water Resources Energy Directorate, the Norwegian National Railways Administration (NNRA), and the Norwegian Public Roads Administration (NPRA). The NIFS project conducted several research activities with the Norwegian Geotechnical Institute (NGI), SINTEF Building and Infrastructure, Multiconsult, Norwegian Geological Survey (NGU), and Norwegian University of Science and Technology (NTNU). In this paper, an effort is made to provide insight into some of the research results related to the assessment and the mitigation of landslide hazards in sensitive clays. The primary aim of this paper is to initiate critical discussions among the relevant scientists, practitioners, and authorities so that well-verified research results can be formulated and implemented into practice.

2 LANDSLIDE CHALLENGES IN SENSITIVE CLAYS

A significant part of the transport infrastructures in Eastern and Central Norway is placed in/on sensitive clays, and a large number of new railways or roads in these regions are being planned on sensitive clay deposits. Sensitive clays constitute a major threat to nearby infrastructures as they have a tendency to rapidly lose their strength when subjected to excessive shear loading. One must consider the most well-known landslides in Norway: Verdal in 1893 (55 million m³, 116 casualties) and Rissa in 1979 (5 million m³, 1 casualty). It is evident from these that sensitive clays in Norway can cause severe disasters when provoked by artificial or natural causes. Over the last 40 years, there has been approximately 1 or 2 such slides per decade, with a volume exceeding 500,000 m^3 (Oset et al. 2014). Since 1996, no lives have been lost in quick clay landslides, but residential areas and transport infrastructures have been affected to varying degrees of destruction. This is illustrated in Figure 1 (left) using an example of the Mofjellbekken landslides in 2015 that caused the partial collapse of the E 18 Bridge located on the landslide scarp. More than 30 such landslides in Norwegian sensitive clays are reported in the literature listed in Table 1.

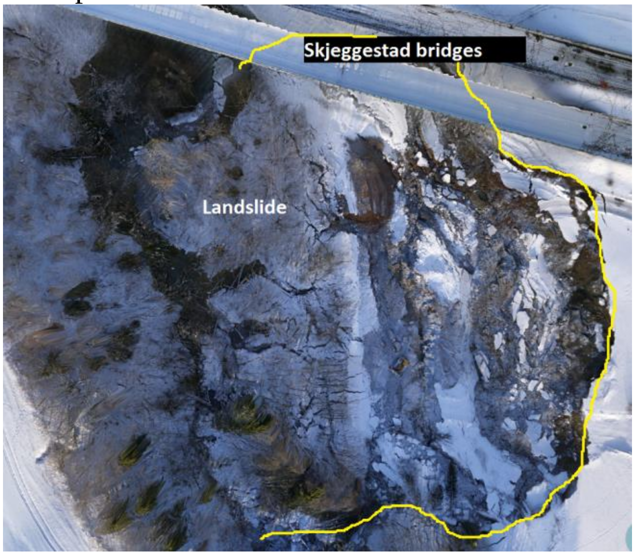

Figure 1 Mofjellbekken landslides in 2015 (sources: Orbiton, and Bjorn K Dolva, NPRA).

From these landslides, it is known that a seemingly stable area can be subjected to a major landslide after a small initial slide. Post-failure movement in terms of retrogression distance (L) and the run-out distance (Lu) in these clays is occasionally fast moving, which may involve massive soil volume in the order of millions of cubic meters (See Figure 2).

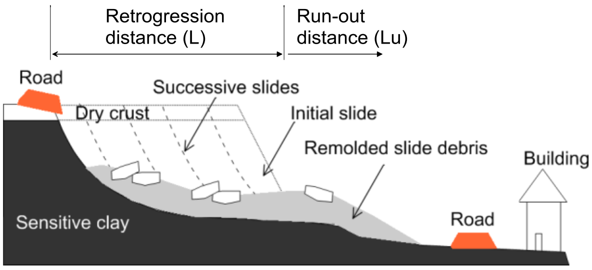

Figure 2 A sketch of retrogressive flow slide in sensitive clays. The retrogression distance (L) and the run-out distance (L_u) are measured from the toe of the slope (Thakur and Degago, 2014).

3 RESEARCH FOCUS

The overall purpose of the work package *sensitive clays* regarding the NIFS project was to obtain a sound basis for the harmonization of the guidelines by the NPRA, NVE, and NNRA. Accordingly, the following research topics were identified:

1. Equity between the local and overall stability of slopes
2. Site investigation database and sensitive clay mapping
3. Sensitive clay landslides along shorelines
4. Detection and characterization of sensitive clays
5. Quantification of the effect of strain softening
6. Post-failure movements
7. Stabilization of critically stable slopes

Research institutions and consultants were engaged to carry out the academic and industrial research as a part of the NIFS project. Some of the key results from the research activities are discussed in this paper.

Table 1 Landslide in sensitive clay (updated after Natterøy (2011), L'Heureux (2012), Thakur et al. (2014a))

Year	Landslide	Type	*L* [m]	L_u [m]	*Volume* [10^5 x m^3]	c_{ur} [kPa]	S_t [-]	I_L [-]	I_P [%]	*References**
1940	Asrumvannet	F				0.1	200	3.1	13	Mayerhof (1957)
1626	Bakklandet	FK	70	50		0.1	30	2	6	Egeland and Flateland (1988)
1988	Balsfjord	F	400		8	1	30	3	6	Rygg and Oset(1996)
1974	Båstad	F	230	700	15	0.53	35	1.8	8	Gregersen and Løken (1979)
1953	Bekkelaget	FK	145	20	1	0.11	150	2.4	11	Eide and Bjerrum (1955)
1953	Borgen	F	165		1.6	0.7	100	1.2	20	Trak and Lacasse (1996)
1928	Brå	FK	197	300	5	0.24	75	2		Holmsen (1929), Reite et al. (1999)
2012	Byneset	F/FK	400	870	3.5	0.12	120	3.9	4.8	Thakur (2012)
1955	Drammen	RS	45		0.04	2.5	4	1.1	11	Eide and Bjerrum (1955)
1625	Duedalen	FK	410		5	0.07	209			Trondheim Municipality reports , Furseth (2006)
1996	Finneidfjord	F	150	850	10	0.4	60			Longva et al. (2003)
1980	Fredrikstad	RS	45	22	1	<0.5	20	1	20	Holmsen and Holmsen (1946), Karlsreud (1983)
1959	Furre	FK/F	300	90	30	0.1	115	2.1	11	Huchinson(1961)
1974	Gullaug	F/FK	150		1.25	2	7.5			Karlsrrud (1979)
1967	Hekseberg	F/FK	700	300	2	0.25	100	2.4	4	Drury (1968)
2014	Hobbel	RS/F	140	200	0.015	<0.2				NGI report nr
2009	Kattmarka	F	300	350	3-5	0.24	63	2.9	8	Nordal et al. (2009)
1994	Kåbbel	F	100	10	1	<0.5	>50	>1.2	20	NVE reports
1944	Lade	F	40	62	0.05	2.12	6.6	1		Reite et al. (1999) Holmsen and Holmsen (1946),
2002	Leistad	FK	250	25		0.15	110	1.5	6	NPRA reports
1989	Lersbakken	RS	65	75	0.75		38-62			NPRA reports
1954	Lodalen	FK	40	10	0.1	17	3	0.8	17	Sevaldsen (1956)
2010	Lyngen	F	153	411	2-3	0.14	51.4	2.1		NVE reports
2015	Motfjellsbakke n	FK	80	30	1	0.2	220	2.1	5	NIFS report nr 53/2015
2000	Nedre Kåbbel	RS	120	10	1.8	<0.5	>50	>1.2	20	NVE reports
1978	Rissa	F	120 0		50-60	0.25	100	2	5	Gregersen (1981)
1995	Røesgrenda	F	100	50	0.02	0.1	186	>1.2	<10	Larsen (2002)
1974	Sem	FK	100	20	0.68	1.4	8-14			NGI(1974)
1965	Selnes	F	230	>400	1.4	0.35	100	2.3	7	Kenney (1967)
1962	Skjelstadmarka	F	600	2800	20	0.83	80	1.1	10	Janbu (2005)
2014	Statlandet	F	150	1300	3.5	0.2	100	4	3.6	NIFS report nr 93/2014
1816	Tiller	FK			55	0.1	90	2.7	4	Reite et al. (1999)
2012	Torsnes	RS	25		0.063	<0.5			22	NVE reports
1893	Verdal	F	200 0	5000	650	0.2	300	2.2	5	Reite et al. (1999)
1959	Vibstad	FK	250	250	10	5	8	0.2	17	Hutchinson JN (1965)

* can be found in the reference list provided by Thakur et al. (2014a)

Landslide types: F = flow slide, FL = flake slide, RS = rotational slide;

Soil poperties: c_{ur} = remolded shear strength, S_t = sensitivity, I_L = liquidity index, Ip = plasticity index

4 RECENT ADVANCES

The NIFS projects and their associated activities have significantly advanced our knowledge related to landslides in sensitive clays. Several research reports are published at www.naturfare.no. In addition, the following articles were released at this conference (Nordic Geotechnical Meeting, 2016):

- *An attempt towards harmonizing the Norwegian guidelines related to construction on sensitive clay*, by Aunaas et al. (2016)
- *In-situ detection of sensitive clays : Part I Selected test methods* by Sandven et al. (2016a)
- *In-situ detection of sensitive clays : Part II Results* by Sandven et al. (2016b)
- *Extended interpretation basis for the vane shear test*, by Gylland et al. (2016)
- *Sample disturbances in the block samples of low plastic soft clays*, by Amundsen et al. (2016)
- *A procedure for the assessment of the undrained shear strength profile of soft clays*, by Thakur et al. (2016)

These papers address sample quality and sample disturbances, regulatory frameworks, the in-situ detection of sensitive clays using e.g., CPTU-R, ERT and AEM, the interpretation of vane shear test results, the effect of strain softening in the stability calculation, strength anisotropy, and the selection of a strength profile. To avoid any repetition and to respect the limited amount of space in this paper, this particular paper is confined to the latest advancements that are not covered by any of the aforementioned papers. At the same time, the authors are referring to these papers to get a complete overview of the contributions made by the NIFS project.

4.1 Local stability, overall stability, and perceptual improvement of material factors

It is a well-known fact that sensitive clays are associated with various risks related to the loss of stability and bearing capacity, as well as substantial ground deformation, which can lead to structural damage and risk regarding the overall stability of an area. The current code of practice (NVE, 2014) suggests that the overall stability of areas that consist of these sensitive clays must be investigated. This overall stability assessment considers areas where there is potential for retrogression and/or the propagation of landslides. It also considers areas that are outside the slide zone but may still be subjected to dangers, such as large movements and structural damages. Therefore, in comparison with local, the overall stability addresses a much wider perspective with respect to the stability of a vast area. Landslide assessments of sensitive clay slopes demand both a local and overall stability calculation (Thakur et al. 2012; Thakur and Degago 2013 &2014; Oset et al. 2014, Aunaas et al. 2016).

However, there has been a challenge in differentiating the extent of local stability from overall stability. In 2014, Frode Oset from the NPRA proposed a pragmatic solution. He suggested that the extent of local stability is the area where the external loading (for example, fill, excavation, etc.) deteriorates the calculated material factor (γ_{mi}) more than 5% in a stability calculation (see Figure 3). This requirement applies for both circular and non-circular slip surfaces.

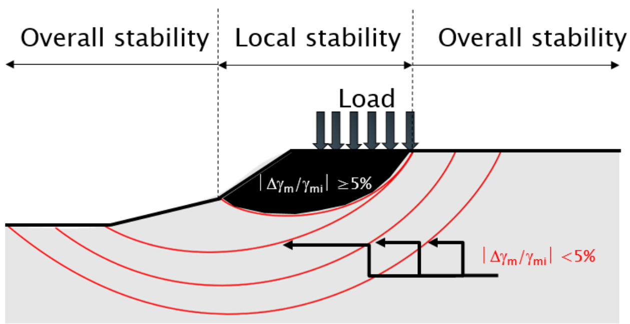

Figure 3 Local and overall stability (Based on NIFS report no. 8/2016).

In 2015, a working group was established by the NIFS project to assess the extent of local stability according to this suggestion. The working group consisted of

- Kristian Aunaas, Frode Oset, and Hanne Bratlie Ottesen, NPRA
- Stein-Are Strand, Einar Lyche, Trude Nyheim and Ingrid Havnen, NVE
- Margareta Viklund and Mostafa Abokhakil, NNRA
- Odd Arne Fauskerud, Multiconsult AS
- Stein Christensen, SINTEF Building and Infrastructure
- Vidar Gjelsvik, NGI
- Vikas Thakur, NTNU

The NIFS report no. 8/2016, prepared by the working group, validated the proposed solution to determine the extent of local stability using several real cases. The recommended step-wise procedure to calculate the extent of local stability is as follows:

- Select a slope geometry
- Calculate the material factor (γ_{mi}) for sliding surfaces before the external load is placed
- Include the external load in the calculation and find a new material factor (γ_{mo}) for the sliding surfaces from the previous step
- Compare the material factors of the sliding surfaces of the situation before and after the external loading
    - Find $\Delta\gamma_m = \gamma_{mi} - \gamma_{mo}$
    - Calculate $|\Delta\gamma_m / \gamma_{mi}|$
- Identify the part of the slope geometry that is covered by the slip surfaces having $|\Delta\gamma_m / \gamma_{mi}| \geq 5\ \%$ and characterize it as the extent of local stability. This is illustrated in Figure 3.
- If the external loading causes a change in the location of critical sliding surfaces, then the corresponding material factor for this particular slip surface should be calculated for both before and after the loading.
- The safety requirement for the overall stability is based on percentual improvement, as shown in Figure 4.

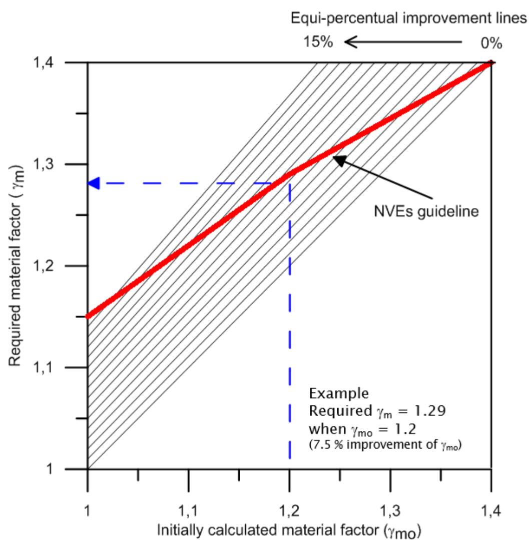

Figure 4 Required material factor (γ_m) for the overall stability of sensitive clay slopes.

- The safety requirement for local stability within the 5% limit deterioration shall be based on the absolute material factor in accordance with the Norwegian Standard or regulations established by the NPRA and NNRA. Safety requirements for impact extend outside the limit based on regulations in TEK 10 and NVEs guideline.

4.2 *Total versus effective stress parameters based on stability analysis of natural slopes*

The stability of slopes in clays is a geotechnical problem of great social, economic, and technical importance. A good appraisal of slopes' stability is, thus, very important, and it is probably the topic that has received the largest attention in geotechnical engineering. However, there is no unanimity on a specific method to determine strength parameters and analyse the stability of slopes.

In Norway, total stress analysis (ϕ = 0) is mostly used. The validity of the ϕ = 0 analysis for calculating the "end of construction" stability of slopes, cuttings, the bearing capacity of footings, and fillings on clay has been very well adopted in Norway and elsewhere. Leroueil et al. (1983) suggest a pragmatic approach to estimate the factor of safety from total stress based on the over consolidation ratio (OCR) because it seems to be a major factor influencing the factor of safety in total stress analyses. Canadian scientists propose a rough estimation as follows:

Factor of Safety ~ 0.9 OCR

It appears from this equation that, for normally or nearly normally consolidated clays, the factor of safety is not far from 1.0, and it is thought that this is the main reason why total stress analysis has enjoyed certain success in Scandinavia and not in Eastern Canada, where the clays are more heavily overconsolidated. Leroueil et al. (1983) have tested this equation for several landslide cases and found that this simplified approach is rather pragmatic.

However, there has been a longstanding discussion in Norway whether $\phi = 0$ analysis is a rational approach to calculate the stability of natural slopes and if one can use the safety level obtained from using the c-ϕ method (effective stress parameter-based drained analysis) for natural slopes. To achieve a consensus in the geotechnical community in Norway, a working group was established by the NIFS project regarding this definition. The working group consisted of:

- Kristian Aunaas, Frode Oset and Hanne Bratlie Ottesen, NPRA
- Stein-Are Strand, Einar Lyche, Trude Nyheim and Ingrid Havnen, NVE
- Margareta Viklund and Mostafa Abokhalil, NNRA
- Odd Arne Fauskerud and Anders Gylland, Multiconsult AS
- Stein Christensen, SINTEF Building and Infrastructure
- Kjell Karlsrud, Vidar Gjelsvik, NGI
- Steinar Nordal and Arnfinn Emdal, NTNU

Due to the lack of space, only selected recommendations are presented in this paper. Readers are referred to the NIFS report to be published in 2016 to get complete information.

The working group recommends that the drained analysis of slope stability analysis can be allowed for natural slopes in sensitive clays if:

- The slope has a stable and steady stress state situation, that is, the slope is not experiencing stress changes due to, for example, natural erosion
- Natural seasonal variations in ground water conditions are included in the calculation
- The slope is not influenced by extreme precipitation
- The slope has an over consolidation ratio (OCR) greater than 6

Assessing the pore pressure situation is the biggest challenge using the effective stress parameters based analysis. The effect of natural variations in the pore pressure situation due to seasonal variations, extreme precipitation must be accounted in the calculation. Climate-induced changes must be taken into consideration in relation to changes in future ground water levels and the ground water flow regime. The basic premise of adding effective stress parameters based drained analysis as a basis for the stability assessment of a natural slope is that it is not exposed to the geological or human-activities resulting in a deterioration of safety level.

Drained analysis should be considered only if the initial slide (local stability) is not involving the sensitive clay layer in the slope. Stress changes leading to increased shear mobilization in a short duration should be treated as an undrained condition. Both human activities and natural causes such as erosion over a short time can cause an undrained situation in natural slopes. A pore pressure profile should be included in the design basis based on effective stress analysis. Robustness against other natural load variations must be taken into account in the parameter selections. The working group has further advised new criteria for perceptual improvements of the safety level of natural slopes.

4.3 National database for site investigation results

White paper no. 15 (March 30, 2012) by the Norwegian Parliament emphasizes, among other things, the importance of information being made available from site investigations. The Norwegian Geological Survey (NGU), together with the NIFS project, among others, has developed a national database for site investigation (NADAG), which is a tool for the more effective collection and use of site investigation data. The cost-benefit analysis performed by VISTA assesses the annual economic benefit from such a database as 6 to 7 times higher than the costs for the construction and operation of the database itself (the NIFS project). The NADAG database is now available at http://tempgeo.ngu.no/kart/nadag/.

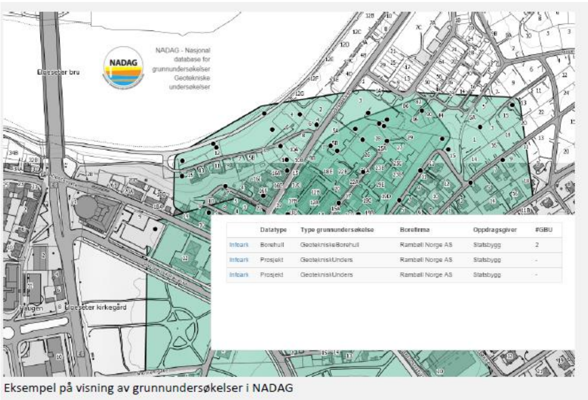

Figure 5 A snapshot from the NADAG database. The dots on the figure refer to the location of boreholes.

4.4 Post-failure movements

Thakur et al. (2014a) present a comprehensive overview of several parameters that may influence the extent of landslides, for example, topography, stability number (*Nc*), remolded shear strength (c_{ur}), liquidity index (I_L), and quickness (Q). The Norwegian landslide data support the fact that large landslides with a retrogression greater than 100 meters are only possible when c_{ur} < 1.0 kPa or I_L >1.2 or Q> 15% and N_c > 4. These criteria are useful and can be utilized as indicators to assess the potential for the occurrence of large landslides (Thakur et al. 2013). However, determining the extent of a landslide with only an individual geotechnical parameter may not be sufficient. A working group was established by the NIFS project to propose an empirical approach to estimate the run-out distance of landslides in sensitive clays. The group consisted of:

- Kristian Aunaas, Frode Oset og Hanne Bratlie Ottesen, NPRA
- Stein-Are Strand, Einar Lyche, NVE
- Odd Arne Fauskerud, Multiconsult AS
- Kjell Karlsrud, Jean-Sebastien L'hueruex, Vidar Gjelsvik, NGI
- Vikas Thakur, NTNU

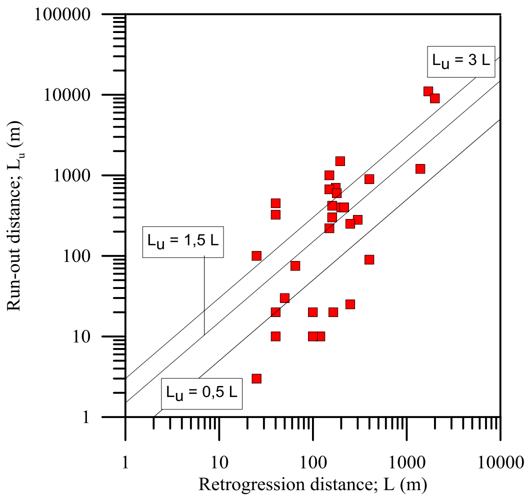

Figure 6 Retrogression distance versus run-out distance (based on Table 1 and Thakur et al. 2014a).

To determine the probable post-failure movements of sensitive clay landslides, it is necessary to identify the landslide type, the the retrogression length (L), and the topography of the downstream region. Accordingly, the following empirical relationships have been proposed by the working group to calculate the run-out distance in sensitive clay landslides (See also Figure 6):

Retrogressive landslide in channelized terrain

$$Lu = 3 * L \quad (1)$$

Retrogressive landslide in open terrain

$$Lu = 1.5 * L \quad (2)$$

Flakes or rotational landslide

$$Lu = 0.5 * L \quad (3)$$

A retrogressive landslide occurs when at least 40% of the landslide volume is sensitive clay. Flow slide-type retrogressive slides occur when the remolded shear strength of involved sensitive clays is lower than 1.0 kPa. A flak landslide may occur when the normal thickness of sensitive clay is less than 40% relative to the critical sliding surface, typically in which the sensitive clay is located in layers approximately parallel to the terrain. A flake-type landslide is often registered when the thickness of sensitive clays is relatively low, typically less than 10% to 20%. Rotational landslides, without further retrogressive landslide development, occur when the thickness of sensitive clays is under 40% compared to the critical sliding surface and the location of the slip surface is below terrain level at the toe.

The numerical modeling of the run-out distance of sensitive clay debris has so far received very little focus. Different approaches and methods have been developed in the past for a quantitative risk analysis using dynamic run-out models for debris flows and avalanches. Some of the commonly used models to estimate run-out distances are quasi-two-dimensional numerical models, for example, BING (Imran et al. 2001) and NIS (Norem et al. 1987), and quasi-three-dimensional models, for example, DAN3D (Hungr 1995; McDougall & Hungr 2004), MassMov2D (Beguería et al. 2009), LS-RAIPD (Sassa 1988), and RAMMS (Christen et al. 2002). However, none of these tools is specifically developed for the estimation of the run-out distance of sensitive clay debris. This can be perhaps attributed to, among other factors, an insufficient knowledge about the complex rheological behavior of sensitive clay debris. Accordingly, as a first step, it would be logical to focus on the rheology of sensitive clay debris using existing and available numerical tools before heading for a new numerical tool for modeling the run-out of such types of material. There exist several rheological models, in the numerical tools mentioned above, for run-out modelling (Thakur et al. 2014b). This is illustrated using a very preliminary study presented by Thakur et al. (2014b) to model the run-out of sensitive clay debris using a plastic rheological set-up in DAN3D using a real case, the Byneset flow slide, from Norway. The Byneset flow slide took place on January 1, 2012, in a highly sensitive clay deposit. The slide is located in the central part of Norway. The actual reason for the initiation of the flow slide is unknown, but it is believed that the slide was initiated due to natural erosion at the toe of the slope. The slide area was approximately 150 m in width. The flow slide retrogressed backward to a distance approximately 450 m from the toe of the slope. The slip surface was located between 10-12 m below the terrain. The volume of the slide debris was estimated to be on the order of $3 – 3.5 \times 10^5$ m^3.

Table 2. Soil properties of the sensitive clay involved in the Byneset flow slide.

Unit weight γ kN/m^3	Undrained shear strength c_{ui} kPa	Plasticity index I_p %	Liquidity index I_L	Fall cone yield strength c_{ur} kPa
20	20 – 30	5 –7	5 –7	0.1

The plastic rheology is related to a pseudo-static motion of liquefied debris. The base shear resistance (τ) is assumed to be equivalent to a constant yield strength (c_{ur}) value.

$$\tau = c_{ur} \quad (4)$$

The yield strength (c_{ur}) of sensitive clays in this case is fully remolded shear strength obtained using the fall cone test. The plastic rheology is found to be the simplest among all the rheological models implemented in DAN3D. The rheology required only γ and c_{ur} values, which are easily obtainable (Thakur et al. 2014b). The flow deposit contours obtained from the plastic rheology, with c_{ur} = 0.1 kPa, are shown in Figure 7. The total run-out of the slide debris obtained at the end of the simulation is quite similar to that observed in the field.

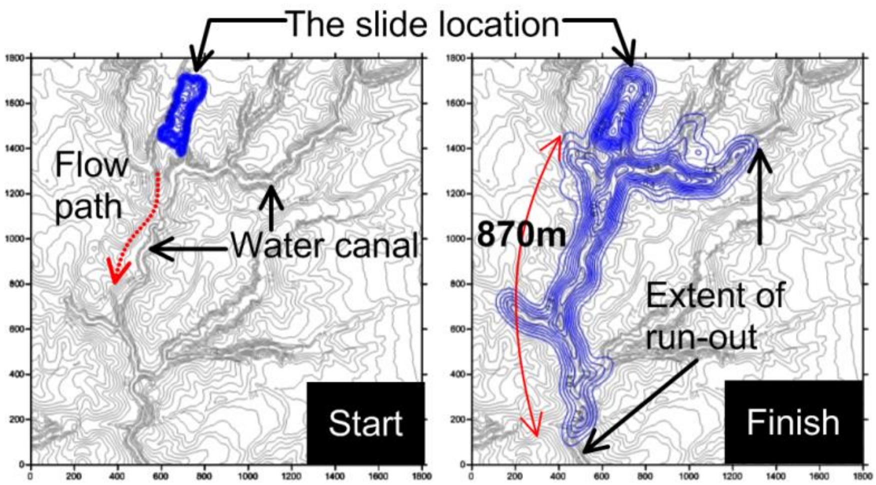

Figure 7 Flow contours obtained from the plastic rheology with c_{ur} = 0.1 kPa (Thakur et al. 2014b).

Thakur et al. (2014) further studied the plastic rheology to see the effect of c_{ur} on the run-out distance for a given geometrical set-up. In Figure 8, the run-out distance was plotted at varying values of c_{ur} to depict the effect of the shear strength on the flow. The result shows that run-out distance logarithmically reduces with increasing c_{ur}. Similarly, the development of run-out with simulation run-out time, plotted in Figure 6, shows that sensitive clays having higher c_{ur} will need less time to reach their final extent of run-out. As an exception, when the basal shear resistance is close to zero, for example, c_{ur}= 0.01 kPa, the extent of run-out and the velocity of debris will primarily be controlled by the topographical aspect.

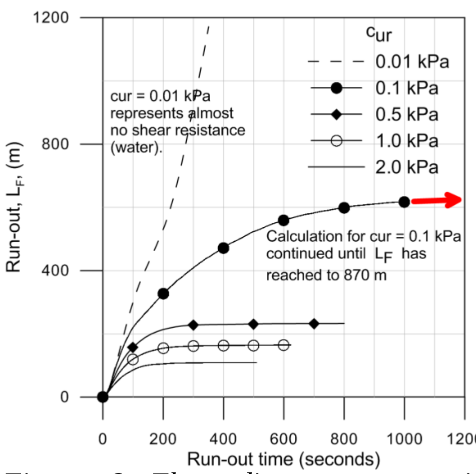

Figure 8 Flow distance at varying shear strengths (Thakur et al. 2014b). L_F in the figure refers to Lu in the context of this paper.

Thakur et al. (2014) use a simplified approach to investigate a very complex problem that has yet to be fully understood. Therefore, several simple approximations were necessary in order to focus on the role of certain parameters governing the basal rheology on the run-out of sensitive clay debris. Thakur et al. (2014) also found that the plastic rheology and the friction rheology seem to predict the run-out distance of the flow slide in Byneset reasonably well. However, the Bingham rheology and the Voellmy rheology, which are sophisticated models, require more parameters that are not readily available for sensitive clays. To advance the knowledge in this area, Issler et al. (2012), NIFS reports no. 38/2013 and 39/2013, Nigussie (2013), Yifru (2014) compared the input parameters of the various models implemented in DAN3D and RAMMS. Similarly, Grue (2015) attempted to measure viscosity on Norwegian sensitive clays. These studies are valuable in developing an advance phehological model that can be used for quick clay slides. Run-out modelling of sensitive clay landslides is an ongoing research activity in Norway as a part of the GeoFuture II project.

5 RESEARCH ON MATERIAL BEHAVIOR

The NIFS project supports two PhD studies at NTNU that deal with the material behavior of sensitive clays. The PhD study by Tonje E. Helle is related to quantifying the improved geotechnical properties as a result of treating the clay with salt (KCl). The research work includes a large-scale field test related to salt infiltration in sensitive clay layers. Helle et al. (2015a & b) show a significant improvement in the undrained shear strength and the pre-consolidation pressure of a sensitive clay sample after salt treatment. Refer Figure 9 for a typical laboratory test result. The preliminary observations made by this PhD work advocates that due to its beneficial effect on the strength and the deformation properties e.g., intact/remolded shear strength, liquid limit, and plasticity, the salt infiltration technique seems to suit for the stabilization method in sensitive clay slopes.

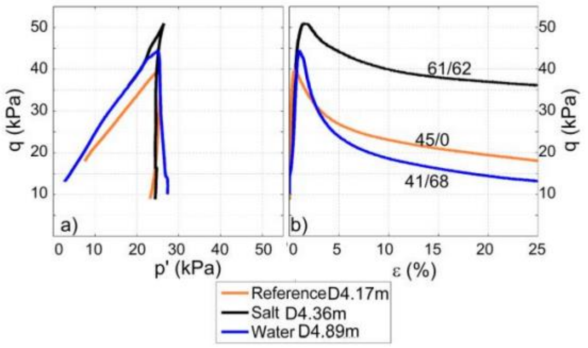

Figure 9 Results from the anisotropically consolidated triaxial test on a block sample and samples stored in de-aired water and KCL. Here, q is the deviatoric stress, p' the mean effective stress, and e the axial strain. Storage time in days is marked in the figure to the right (source: Helle et al. 2015b).

The reliable characterization of fine-grained soil samples requires undisturbed sampling, followed by careful material handling. However, this is not a straightforward task.
Moreover, experiences show that, because of the restricted capacity of geotechnical laboratories, fine-grained soil samples are rarely tested immediately after sampling. The storage time of samples may vary from days to weeks and can be as long as several months. A long storage time can significantly alter the behavior of fine-grained soil samples. A PhD study by Helene Alexandra Amundsen is exploring this aspect. Amundsen et al. (2015) show that low plastic sensitive clay samples may behave very differently, even with the matter of a few hours of storage, if the impact of stress relief during the sampling is great (See Figure 10). This PhD work is aiming to find suitable measures to handle this type of challenges in sensitive clay soil samples.

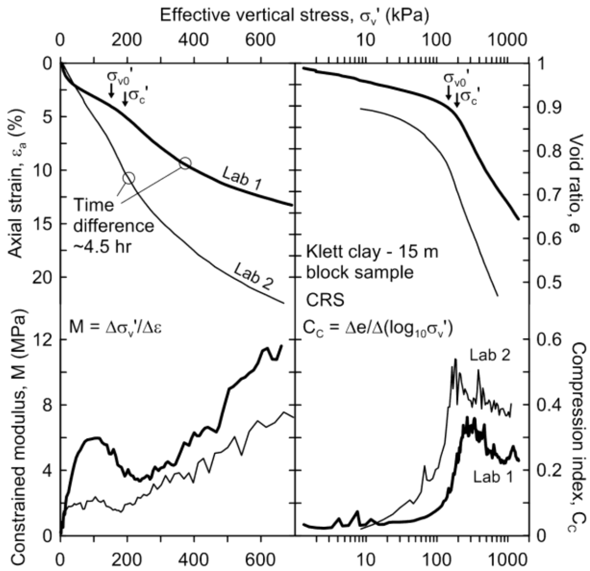

Figure 10 Effect of stress relief and the storage time is illustrated using the non-unique response from a single block sample from Klett tested by two different laboratories (source: Amundsen et al. 2015).

6 CONCLUDING REMARKS

This paper presents some recent advancements made by the NIFS project in relation to the assessment and the mitigation of landslides in sensitive clays. In doing so, a criterion for the estimation of the extent of local stability, the condition for the use of drained stability analysis in natural slopes, and the empirical correlations to estimate run-out distance is addressed in brief. Fundamental research related to salt infiltration in sensitive clays and the effect of storage time in sample quality is discussed. These advancements provides a necessary basis to harmonizing the guidelines by the Norwegian agencies such as NNRA, NVE and NPRA.

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