Stability of Natural Slopes in Quick Clay

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Abstract

Norway and Sweden have been subjected to landslides for several centuries, with some of the largest landslides involving a large block sliding out as a continuous flake on a deposit of soft quick clay. Attempts to calculate the stability conditions for this type of landslides by using a conventional effective stress analysis have shown to considerably overestimate the safety factor. The introduction of the concept of limiting or yielding stresses in the course of the last 3 to 5 decades resulted in the general acceptance that stability calculations, even for natural slopes, should be based on undrained shear strength. Early in the seventies the ADP-type of analysis was proposed (Bjerrum, 1973; Ladd and Foott, 1974), based on triaxial and direct simple shear tests on tests specimens reconsolidated to *in situ* stresses, and thus simulating the stress conditions along different parts of the failure surface. The present study describes recently developed relations between undrained shear strength parameters and effective shear strength parameters, friction and attraction. The new relationships make it possible to do an effective stress analysis, and thus take existing pore pressures into consideration in a better way than in an ADP-type analyses. These effective stress-strength parameters have been determined for several Norwegian and Swedish clays, and used for a recalculation of some of the older Scandinavian flake type landslides.

Introduction

It is commonly accepted today that in soft, sensitive clays yielding and failure take place at a critical stress, occurring before the clay has been able to fully mobilize its effective stress strength parameters. This has led to the conclusion that stability calculations even for natural slopes should be carried out with an undrained (ADP-type[1]) stability analysis, based on normalized values of active and passive triaxial and direct simple shear strengths. Figure 1 shows the results from ADP-analyses of five old Norwegian quick clay flake landslides presented earlier by Aas (1981). For simplicity, the stability conditions were expressed by comparing the calculated average shear stress along the shear plane $(\tau_\beta/\sigma'_0)_{aver.}$ with the direct simple shear test results from the laboratory (s_{uD}/σ'_0), and in one case (the Furre landslide), with the shear stress ratio measured from large shear box tests *in situ*, $(\tau_{\beta\,Cr}/\sigma'_0)$.

The present paper presents the relationships between the normalized undrained shear strengths and the effective shear strength parameters, friction and attraction, thus confirming that the ADP analysis can be considered as theoretically well founded, and in accordance with the fundamental and well-established effective stress strength concept. Since the occurrence of quick clay is often tied to flowing water and artesian pore pressures, it is important to determine exact values of pore pressures and corresponding effective stresses in the clay. This also applies for an ADP-analysis.

[1] ADP-approach: A denotes Active, D Direct simple shear and P Passive, as determined, e.g. from laboratory tests, and where three different undrained shear strengths are used along the sliding surface.

Reliable values of pore pressure and effective stresses are of great importance for an alternative effective stress analysis based on the effective shear strength parameters, and for a new failure criterion for soft, sensitive clays. Aas and Lacasse (2021a) presented a reliable relationship between effective stress strength parameters and plasticity, illustrated in Figure 2.

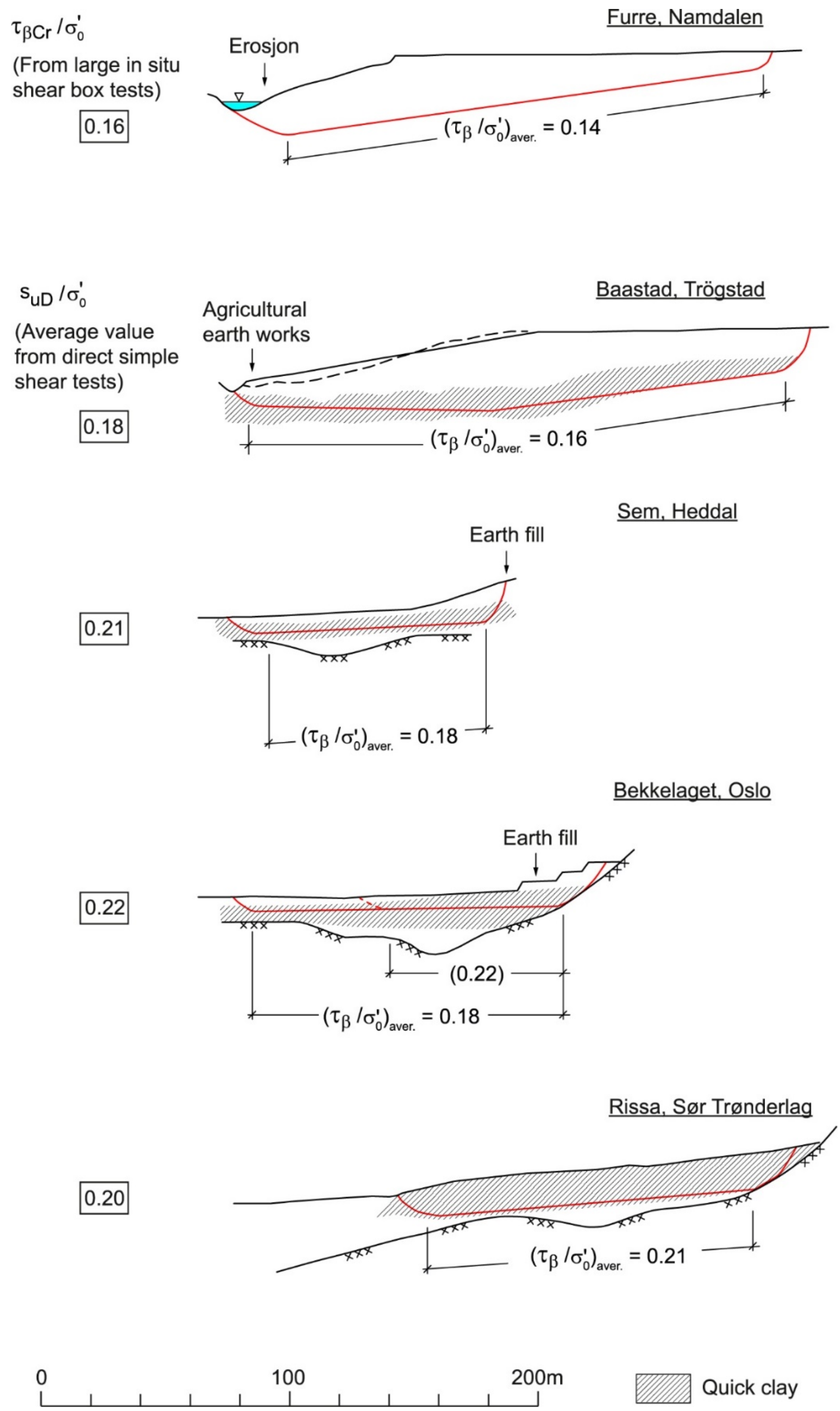

Figure 1. Comparison of calculated average shear stress acting along plane sliding surface $(\tau_\beta/\sigma'_0)_{aver.}$ and measured direct simple shear strength for five landslides in Norwegian quick clay.

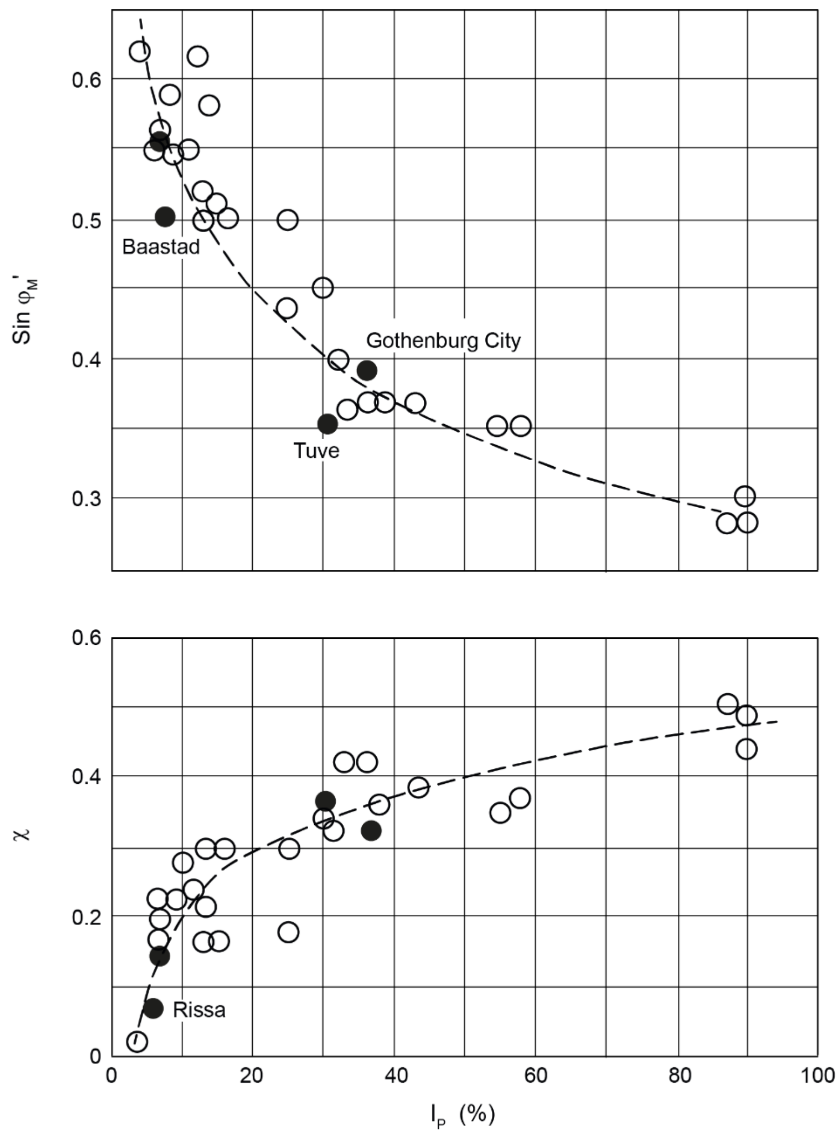

Figure 2. Relationship between effective stress strength parameters and plasticity index (Aas and Lacasse 2021a).

In this paper, an estimate was done to back-calculate the stability conditions for five older Scandinavian quick clay slides, using an effective stress analysis as mentioned above. However, a limitation of the approach is that pore pressure data are very limited or sometimes are not available. To compensate for this, one has had to investigate the relationship between calculated safety factor and assumed pore pressures, and evaluate whether the pore pressure values that lead to SF = 1.0 seem realistic. The calculations were based on effective shear strength parameters determined from the actual landslide area, or if those were missing, on data from other locations with similar clays and stresses. In the present study, information about geometric and geotechnical conditions was collected from reports and publications, and from personal communication with people with knowledge of the landslides.

Shear strength of soft, sensitive clays

In the following description, the clay considered is assumed to have been consolidated under a condition of no lateral yield. If one assumes in addition that the mobilization of friction is closely related to the condition that plastic deformation takes place, this means that the amount of mobilized friction in the quick clay is the result of vertical stresses and strains alone. Consequently, in active shear, the mobilized friction is equal to $^{1}/_{2}\sigma'_v \cdot sin\varphi'_M$. In passive shear, decreasing vertical stress and plastic deformation make possible to mobilize friction equal to $^{1}/_{2}K_0 \cdot \sigma'_v \cdot sin\varphi'_M$. This means full mobilization of friction due to σ'_v in active shear and in passive shear due to σ'_h (= $K_0 \cdot \sigma'_v$), using the following notation:

σ'_v	=	Effective vertical stress
φ'_M	=	Material friction angle
K_0	=	Coefficient of earth pressure at rest
σ'_h	=	Effective horizontal stress

An attempt to mobilize friction also due to the horizontal stress in active shear, or due to the vertical stress in passive shear, results in structural collapse, mobilization of high pore pressures and complete failure. For a normally consolidated clay sample reconsolidated under in situ stresses, such attempts would imply a transition from a state of uniaxial vertical strain, to a condition of large shear strains (Fig. 3), which is believed to be responsible for the structural reorganization and disturbance of the clay structure (Aas and Lacasse, 2021d).

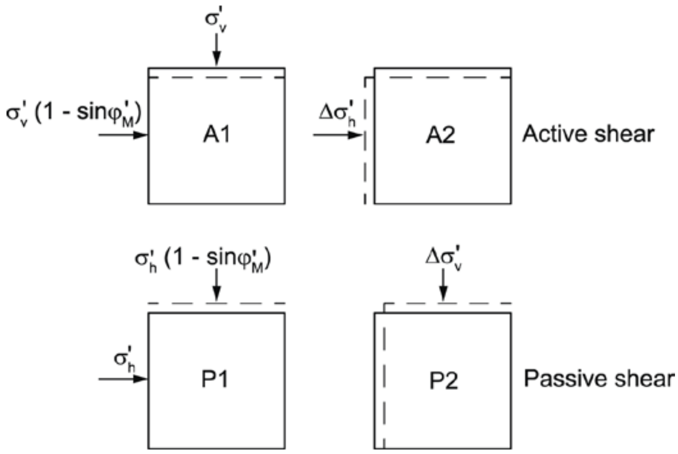

Figure 3: Transition from elastic, uniaxial strain to critical shear stress condition

Figure 4 shows examples of such dramatic failures in active undrained triaxial tests for five different clays (Aas, 1981). The clays have a plasticity index between 3 and 85% and a sensitivity between 3 and 94.

During the reloading up to failure, the increase in pore pressure equals the increase in maximal shear stress, thus confirming initially perfect elastic conditions. The major principal stresses at failure in active and passive shear equal σ'_{vE} and $\sigma'_{vE}\ (1 - sin\varphi'_M)$ respectively, where σ'_{vE} represents an equivalent effective vertical stress larger than the *in situ* stress, σ'_{vo}, and is determined by consolidation strain conditions, as for instance ageing or weathering. The minor principal stresses for a purely frictional soil is $\sigma'_{vo}(1 - sin\varphi'_M)$ and $K_0 \cdot \sigma'_{v0}(1 - sin\varphi'_M)$ in active and passive shear respectively. These values are, however, reduced with $\chi \cdot \sigma'_{v0}$ and $\chi \cdot K_0 \cdot \sigma'_{v0}$, in a clay exhibiting attraction.

The parameters φ'_M and χ denote *the material friction angle*, and the *normalized material attraction*, the term "material" indicating pure material constants, and hence are independent of stress level, stress direction and stress history. The attraction acts like a tensile reinforcement in the clay, allowing the clay to reduce the minor principal stresses and thereby increase the shear strength prior to failure.

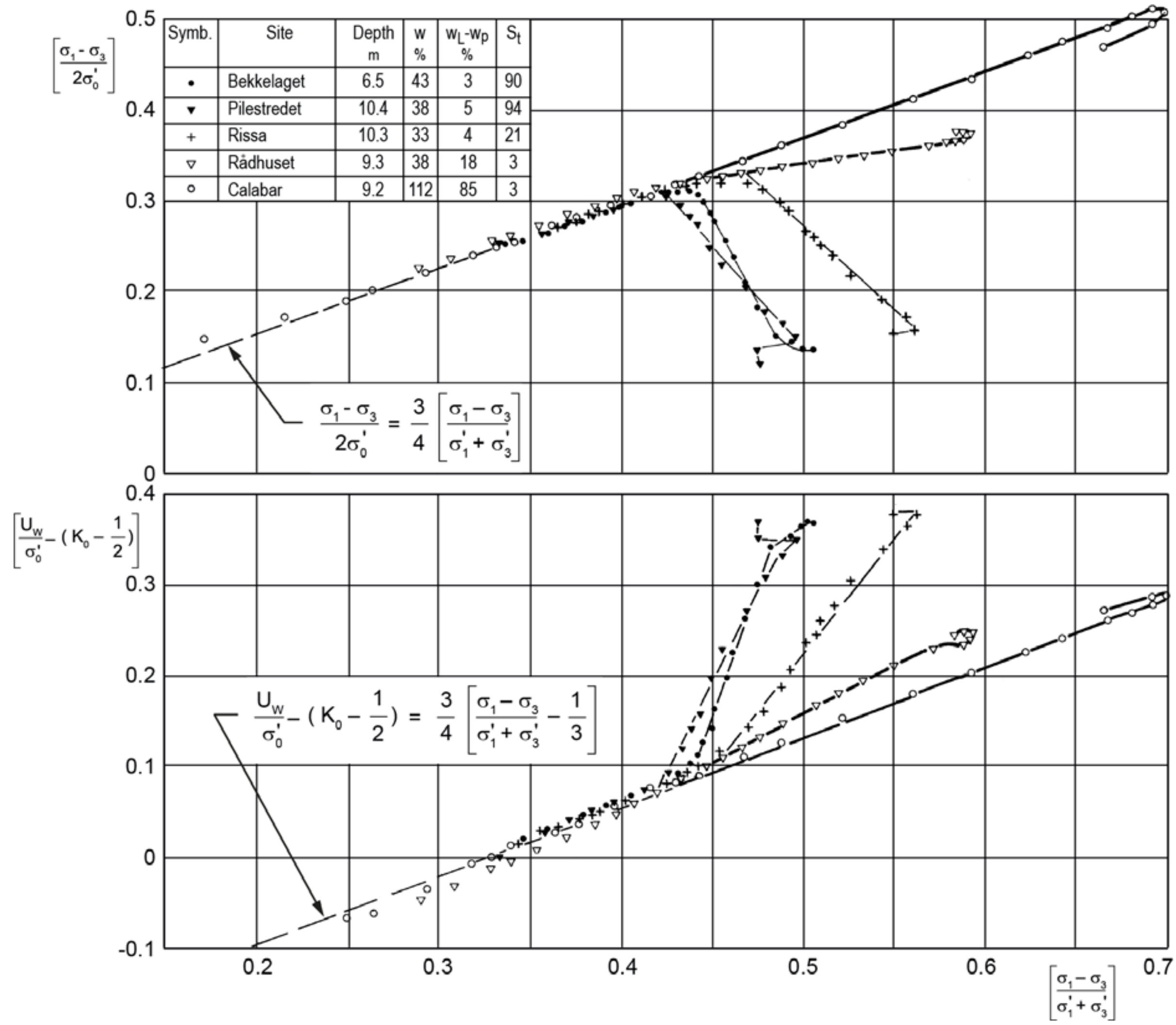

Figure 4. Results from undrained, active triaxial tests on specimens consolidated to in-situ stresses.

Combining the principal stresses outlined above leads to the following general expressions for the active, passive and direct simple shear strengths, s_{uA}, s_{uP} and s_{uD}:

$$s_{uA} = {}^{1}/_{2}\sigma'_{vo}\,[(\chi + \sin\varphi'_M) + \sigma'_{vE}/\sigma'_{vo} - 1] \quad (1)$$

$$s_{uP} = {}^{1}/_{2}\sigma'_{vo}\,[K_0\,(\chi + \sin\varphi'_M) + \sigma'_{vE}/\sigma'_{vo}\,(1 - \sin\varphi'_M) - K_0] \quad (2)$$

$$s_{uD} = {}^{1}/_{4}\sigma'_{vo}\,[(1+K_0')\,(\chi + \sin\varphi'_M) + \sigma'_{vE}/\sigma'_{vo}\,(2 - \sin\varphi'_M) - (1 + K_0] \quad (3)$$

Laboratory shear tests on quick clay close to landslide sites

Figure 5 shows the results from undrained triaxial tests on quick clay from a location close to the large Tuve landslide 1977, 10 km north of Gothenburg (NGI, 1988). Effective stress paths from active and passive triaxial tests on specimens from 15.5 and 20 m depths made it possible to determine effective stress strength parameters of *sin*φ'_M = 0.35 and χ = 0.36. Parallel tests (NGI, 1979) on a specimen in Gothenburg City gave *sin*φ'_M = 0.39 and χ = 0.32 (Fig. 6). The parameters seem to vary little from one location to the other in the "Gothenburg area". For the tree Swedish clay landslides close to Gothenburg analysed below, values of *sin*φ'_M of 0.36 and χ of 0.35 were used.

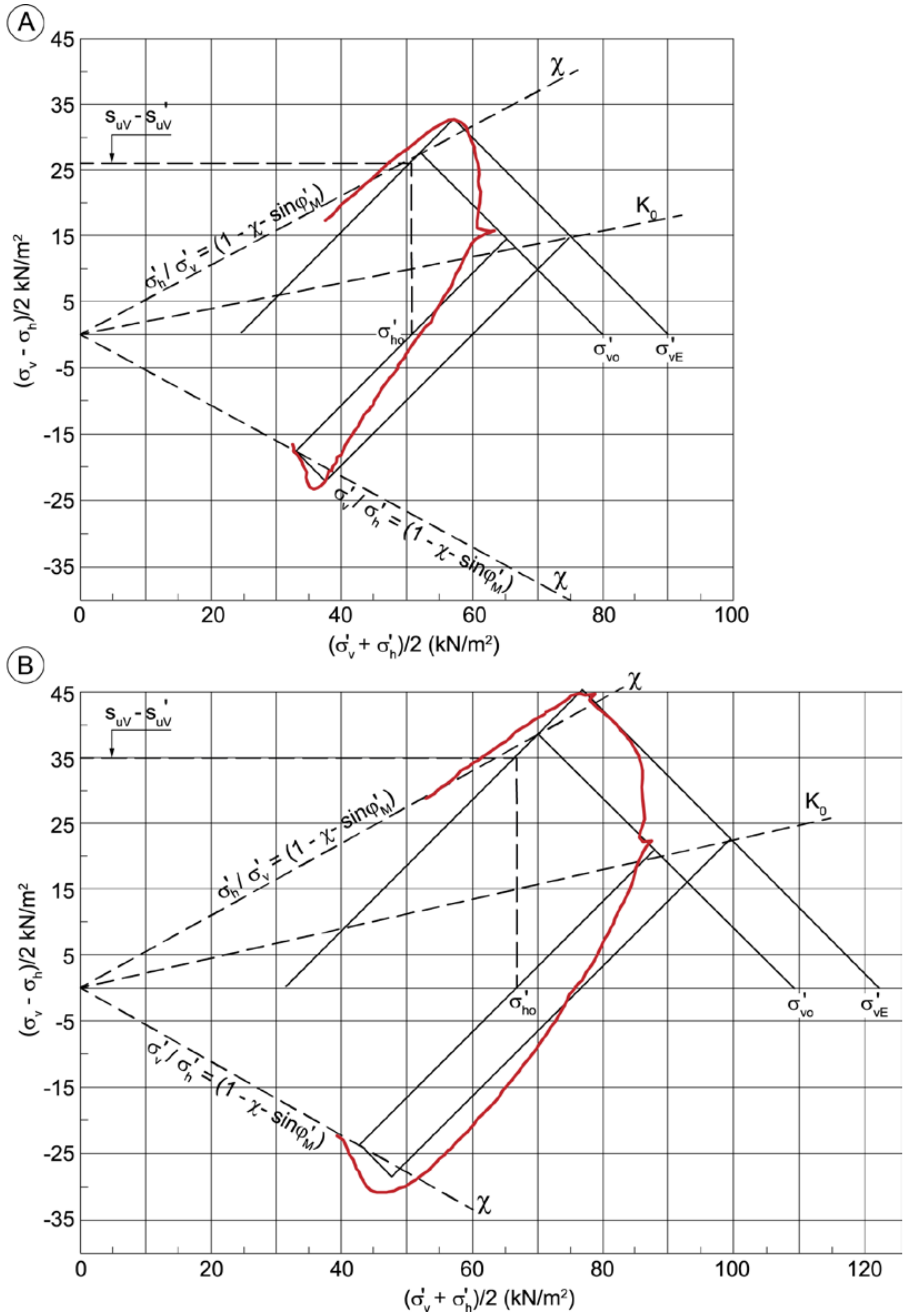

Figure 5. Results from active and passive triaxial tests on Tuve specimens from 15.5m (A) and 20m (B).

In the stress-strain diagrams, the crossing point of the two lines representing σ'_{vE} and $\sigma'_{vE}(1 - \sin\varphi'_M)$ is on the "K_0-line" ($\sigma'_h/\sigma'_v = 1 - \sin\varphi'_M$). This point defines the value of $\sin\varphi'_M = 1 - K_0$. Similarly, the crossing point of the two lines representing σ'_{vo} and the minor principal stress in active shear is on the χ-line ($\sigma'_h/\sigma'_v = 1 - \chi - \sin\varphi'_M$), thus making it possible to determine the value of χ. In the case of the Tuve clay, however, due to a certain degree of weathering, K_0 is no longer equal to $\sigma'_{vE}(1 - \sin\varphi'_M)$, and has to be determined from vane shear data.

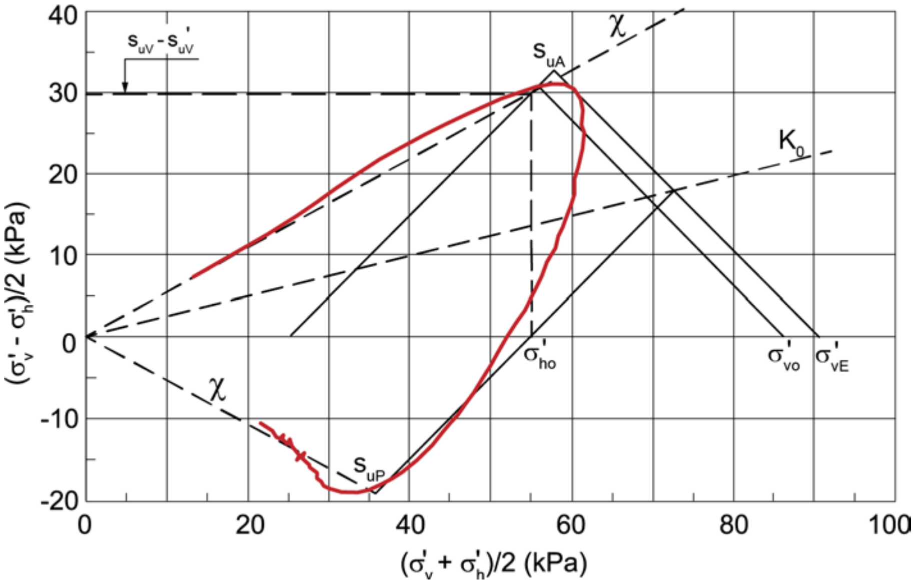

Figure 6. Results from active and passive triaxial tests on samples from 14 m depth in Gothenberg.

The same type of tests on representative clays from two older Norwegian quick clay landslides, Baastad and Rissa, suggest the values of $\sin\varphi'_M = 0.50$, $\chi = 0.15$ and $\sin\varphi'_M = 0.55$, $\chi = 0.06$, respectively (Figs 7 and 8).

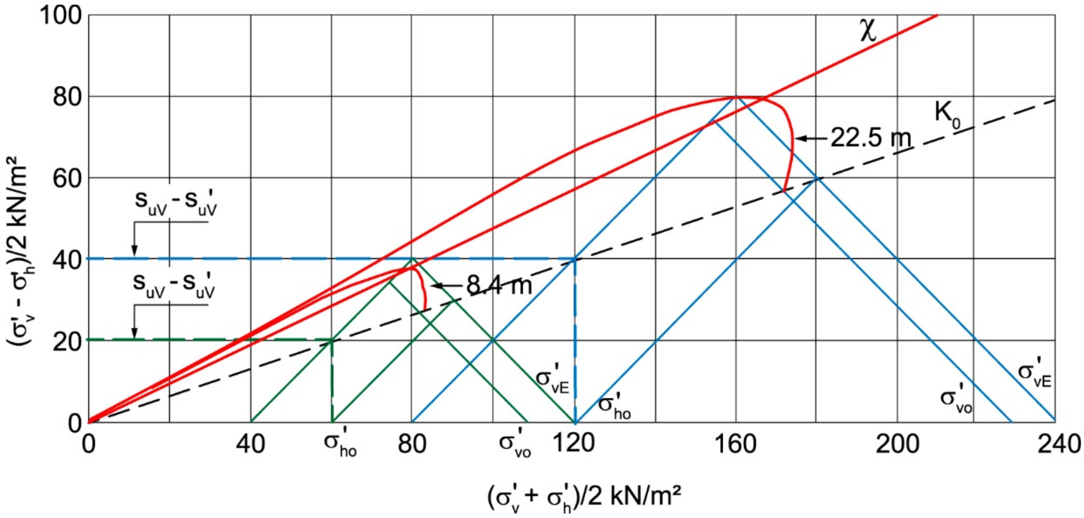

Figure 7. Results from active triaxial tests on specimens 8.4 and 22.5m depth at Baastad.

Typically for soft, quick Norwegian clays, and contrarily to Swedish clays, the effective stress path from passive tests does not give a sufficiently accurate peak value to determine K_0. Aas *et al.*, 1986) demonstrated though that through a combination of in situ vane boring and active triaxial test results, one can determine the K_0-parameter with a construction as done on Figures

7 to 9. Figure 6 give examples showing that the use of both methods leads to the same result.

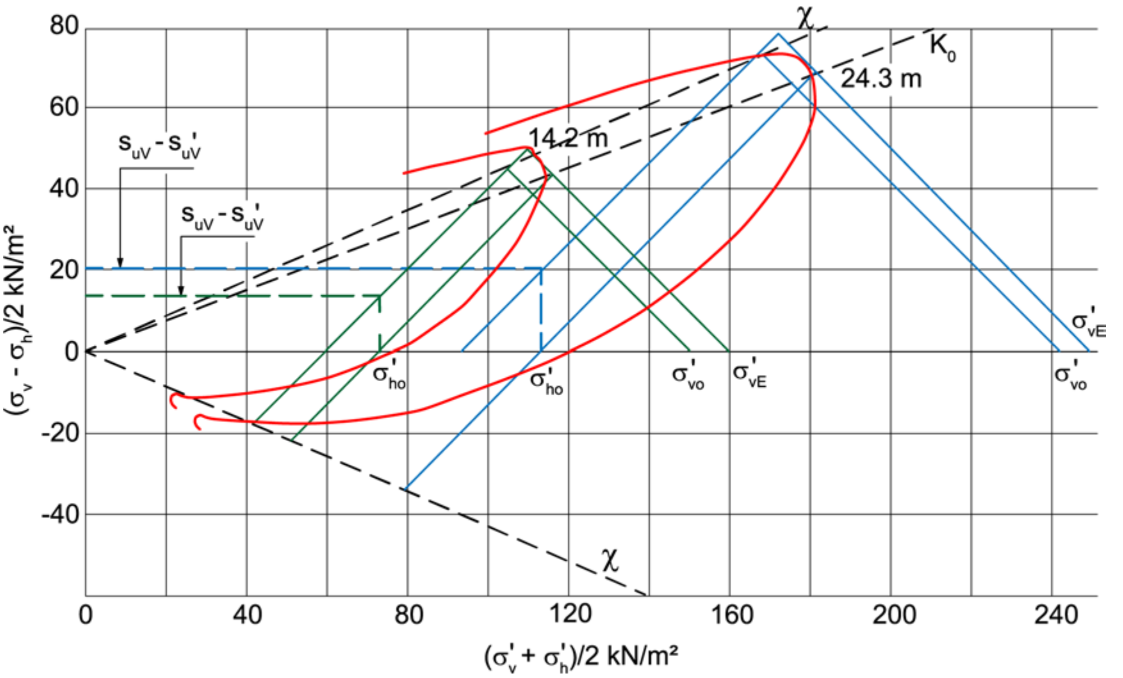

Figure 8. Results from active and passive triaxial tests on specimens from 14.2 and 24.3 m at Rissa

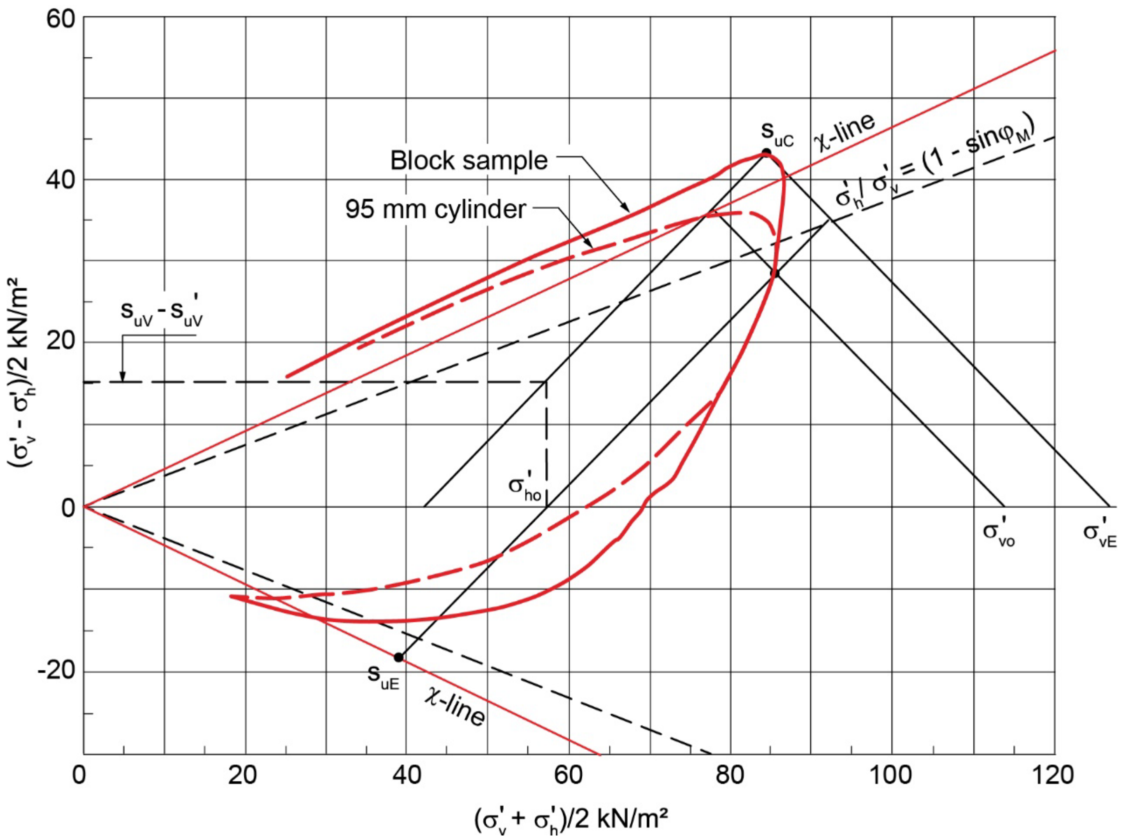

Figure 9. Results from active and passive triaxial tests on samples from 10.2m in Drammen.

The values of the parameters $sin\varphi'_M$ and χ used in the stability analysis are shown as a function of the plasticity index of each clays in Figure 1.

Figure 9 shows the test results for a quick clay sampled at 10.2m in Drammen City. The test program included both block sampling and sampling with a 95-mm piston cylinder. In Figures 5 to 9, s_{uv} and s_{uv}' denote undisturbed and remoulded field vane strength. This is due to the disturbance related to the use of piston sampler in this extremely quick clay almost nullifies the effect of ageing, appearing in the test on the block sample. This disturbance results in a reduction in $sin\varphi'_M$ (from 0.55 to 0.51) and an increase in χ (from 0.08 to 0.12). However, from a practical point of view, it is important to recognize that the sum of the two parameters (Aas and Lacasse 2021a) is almost not affected by sampling disturbance.

Stability analysis

Stability analyses were performed for two Norwegian and three Swedish flake type landslides. A simplified model was used, including an active earth pressure at the landslide rear scarp, a passive earth pressure at the front of the landslide, and a sliding body in between.

$$\begin{aligned} &\text{Active pressure:} && P_a = \gamma D^2/2 - 2s_{uA}/\sigma'_{vo}\,(\gamma' D - 10x)D/2 \\ &\text{Passive pressure:} && P_p = \gamma D^2/2 + 2s_{uP}/\sigma'_{vo}\,(\gamma' D - 10x)D/2 \\ &\text{Sliding body:} && P_\beta = \gamma DL \cdot sin\beta \\ &\text{Mobilized shear strength:} && P_s = (\gamma' D - 10x)\, L\, s_{u\beta}/\sigma'_{vo} \end{aligned} \tag{4}$$

$$\text{Safety factor} \qquad F = P_s/(P_a + P_\beta - P_P) \tag{5}$$

where γ and γ' denote total and submerged unit weight, D depth down to the sliding surface, L length of the sliding body, and β the inclination of the failure surface. For the Swedish clays, a total unit weight of 16 kPa/m^3 was assumed, whereas for the Norwegian clays, a total unit weight of 19 kPa/m^3 was used at Rissa and 20 kPa/m^3 at Baastad. The term "x" in the equations expresses the piezometric elevation in meters above the ground surface in the failure plane.

Isostatic uplift and erosion have caused increasing shear stresses and increased pore pressures and water flow in the clay deposits. This has led to a washing out of salt in the pore water, making the clay "quick", and causing a strain-softening. This process is still in progress today. Therefore, it is reasonable to believe that the clay does not have significant ageing, and can be considered as "young" and almost normally consolidated. For the stability analyses, the ratio $\sigma'_{vE}/\sigma'_{vo}$ was assumed equal to 1.1 and 1.05 for the Swedish and Norwegian clays, respectively, in Eqs (1) to (3), based on the results of the triaxial tests. The shear strength on a gently dipping (β degrees) failure plane is given by the formula:

$$s_{u\beta} = s_{uA} \cdot cos^2(\beta - 45^o) + s_{uP} \cdot sin^2(\beta - 45^o) \tag{6}$$

Since s_{uA} and s_{uP} are expressed in terms of effective stresses, one in reality performs an effective stress analysis, which makes it simple to consider the effect of pore pressures in the calculations.

The formation of quick clay is often tied to percolating water through a layer of marine clay. Therefore, it is reasonable to assume that artesian pore pressures may be a primary contributing factor leading to the occurrence of quick clay slides. Pore pressures usually increase with depth, with a value of zero at the groundwater surface and increase with depth to a maximum value in permeable soils at the bottom of the deposit or in permeable rock below. The magnitude of the pore pressures in quick clay generally increase gradually over the course of a very long period of time. They are generally only slightly affected by short periods of heavy rain. This means that the natural slopes studied in this paper may have been on the brink of failure for many years. In general, most quick clay slides are triggered by a moderate reduction in the stability of the slope due to small earth works, pile driving, erosion or similar events.

Based on this reasoning, stability calculations can be performed as an effective stress analysis using the known effective strength parameters χ and $\sin\varphi'_M$ together with a failure criterion for soft contracting clays. This will give a relationship between theoretical safety factor and average artesian water pressure along the failure surface. In the quick clay landslide examples reported below, where pore pressures records are often missing, the calculations had the purpose to estimate the existing pore pressures conditions that were necessary to obtain a safety factor of 1.0, and then evaluate how realistic the pore pressures were.

Stability analyses for five landslides

The 1974 Baastad landslide, Norway

The landslide took place on 5 December 1974 and involved about 1.5 million m^3 of soil (Gregersen and Løken, 1978). The landslide area covered about 80.000 m^2 of farmland. The debris extended over another 45.000 m^2 of the downstream valley bottom. The slide terminated near the dwellings on two farms. About 10 individuals were on the farms at the time of the landslide, but fortunately no lives were lost. Nobody actually witnessed the landslide. According to the persons on the farms, the landslide occurred very fast, probably within one minute or less.

The sliding masses at Baastad consisted of a non-sensitive clay. The clay was fairly homogeneous down to 16 m, and contained layers of sand and silt at greater depths. A quick clay layer of considerable thickness (20 m or more) was found at 21 m. The water content of the quick clay was about 28%, the plastic limit, w_P, 13% and liquid limit, w_L, 20%.

Stability analyses were carried out for a profile in the centre of the landslide. The initial depths to the sliding surface, 22 m in the active zone and 12 m in the passive zone (Fig. 10) were determined with vane borings and sampling trough the slide masses. The inclination of the failure surface under the block in between, was 4.9°. The calculated safety factor was 1.22 if one assumed hydrostatic pore pressures with groundwater level at ground surface, and 1.0 if one assumed the water rising 1.2 m above the ground surface.

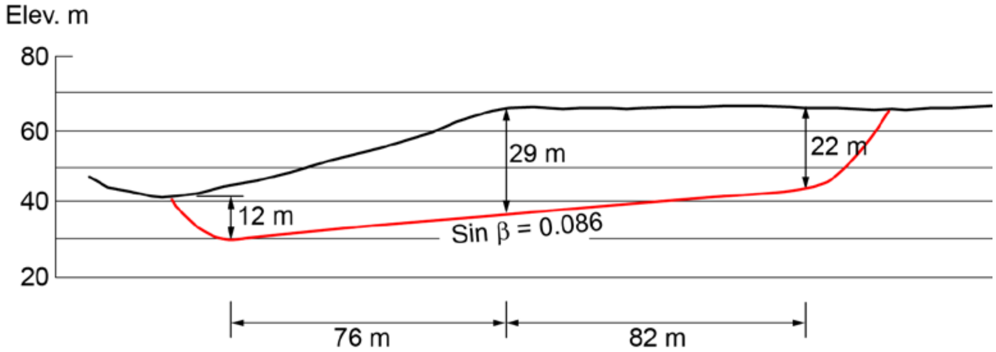

Figure 10. The Baastad landslide.

The 1978 Rissa landslide, Norway

The Rissa landslide happened on 29 April 1978 and involved an area of about 333.000 m^2 and 5 to 6 million m^3 quick clay (Gregersen, 1981). The main part of the landslide slid out in the course of 5 minutes. The sliding area damaged seven farms and five dwellings, either taken by the slide or condemned from safety reasons. About 40 persons were present in the area, and one live was lost. After initial, smaller, progressive slides, the main landslide started, resulting in an extensive disaster! An area 250 m in width (normal to the sliding direction) and 150 m long slid out as a monolithic block, downwards, into Lake Botnen. The major part of the landslide, analysed below, was assumed to have been only slightly influenced by the initial slides. The slope was considered to have been standing with a safety factor close to one prior to the landslide.

A boring just behind the slide showed quick clay down to 15 to 20 m. The clay had a natural water content of 30 to 35%, a liquid limit of about 20 to 25% and a plastic limit of 17%. New site investigations at the site in 2009 (NGI, 2009; Liu *et al.*, 2021) confirmed a water content of 35%, a plasticity index less than 10% and an overconsolidation ratio of about 1.4. An artesian pore pressure of 10 kPa (1 m) was measured close to Lake Botnen, just south of the Rissa landslide. At the top of the slope, it was estimated that the porewater pressures were either hydrostatic or slightly below hydrostatic.

In front of the slide (Fig. 11), the sliding mass met stiffer clay, and ended up in a 8 m deep passive zone. Between this passive zone and the 12 m deep active zone in the back of the slide, a 127-m long block of quick clay was sliding along a shear plane dipping 5.0°. The calculated safety factor was 1.33 for groundwater at the ground surface. The factor of safety reduced 1.0 for pore pressures corresponded to the groundwater 1.9 m above ground surface.

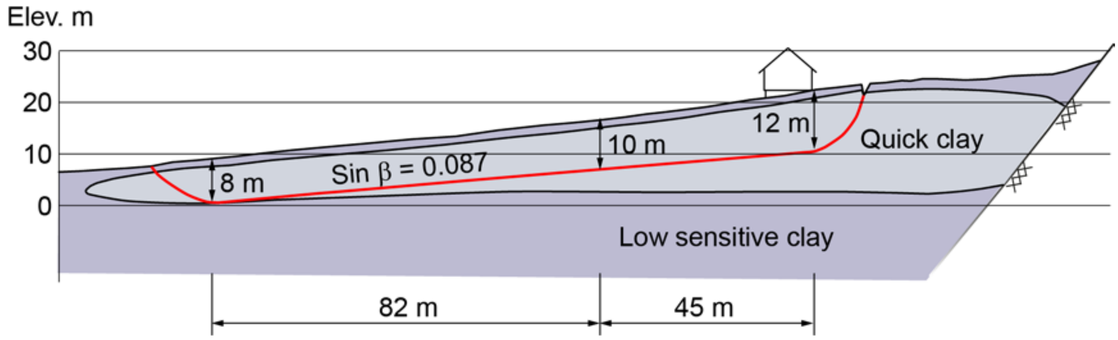

Figure 11. The Rissa landslide

The 1950 Surte landslide, Sweden

The Surte landslide that took place on 29 September 1950 was located in a village about 15 km upstream of Gothenburg (Jacobsson and Mohren, 1952). The area that slid out had a width of about 400 m, parallel to the river, a total length of about 600 m, and a volume estimated at 4 million m^3. Thirty one domestic houses and about 10 outhouses were swept 50 − 150 m down towards the Göta River. Three hundred persons became homeless, and one elderly woman died.

The river bank had practically no height, and the terrain was practically horizontal within 200 m from the river. The landslide lasted for 2-3 minutes, and during that time the clay masses raised the bottom of the river several meters, blocking the river and impeding shipping.

The sliding masses consisted of clay containing several thinner layers of more permeable material. Samples from the back part of the slide showed a thick layer of quick clay with the following properties: sensitivity 50 – 100, a plastic limit of 25 – 30%, a liquid limit of 50 – 55%, and a natural water content of about 70%. Artesian pore pressures corresponding to a water rise above terrain of 6 – 7 m in the sliding masses and 3 – 4 m just outside the edge of the landslide were recorded.

Stability analysis were done for three cases, including a 100-m long part of the total slide, which was extending further about 300 m out to the river (Fig. 12). The most critical case gave a safety factor 2.7, if assuming pore pressures corresponding to a water rise to terrain, and 1.0, for a water rise 4.0 m above terrain. This alternative included depths in active and passive zone 12 and 14 m, leading to an sliding surface inclination of 5.7°. A five meter deeper shear surface gave corresponding values: safety factor of 2.8 and water level rise 5.2 m above terrain, respectively.

It should be emphasized that the stability analysis included a length of about only 100 m of the total slide. If the analysis included the 300 m of land extending to the river, the calculated safety factors was considerably higher. The mechanism of failure in this outer zone is discussed below.

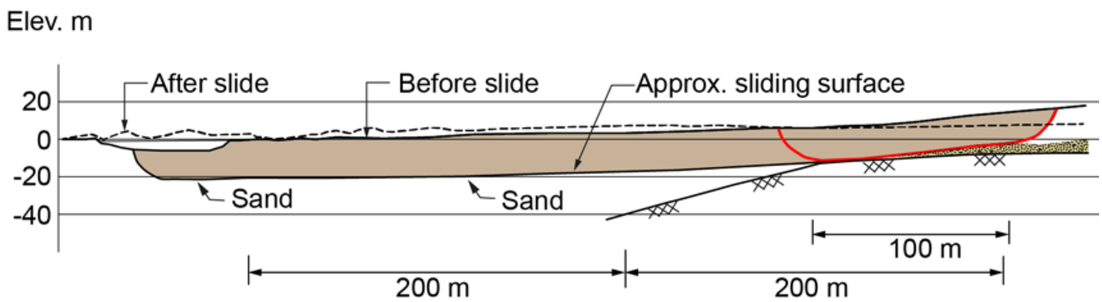

Figure 12. The Surte landslide

The 1957 Göta landslide, Sweden

The first sign of instability on 7 June 1957 was a 130-m long crack in the ground just inside the river bank, which was observed about three hours before the landslide rapidly expanded to 200 to 300 m inland from the Göta River shore (Odenstad, 1958). The river bank was displaced up to 60 − 70 m towards the river, and the ground surface sank down 7 to 8 m within the slide area. A pulp-mill, located where the slide started, was totally destroyed. Of the 200 persons working in the factory area, three lost their lives. Shipping on the Göta River was suspended for a month. Prior to the landslide, the depth of the river seemed to have been 10−12 m.

Based on borings in the upper part and upstream of the sliding area, the soil profile was sensitive and quick clay, with a plastic limit 25 to 30%, a liquid limit 50 to 60%, and a natural water content of about 75%.

Stability analyses were carried out for two profiles (Fig. 13) oriented normally to the Göta River. One profile passed through the factory area, where the slide was initiated by a local failure in the river bank. The other profile extended through an area located upstream of the factory, where the clay seemed to have been sliding toward the river as a whole flake. The terrain at Göta River consisted of a steep river bank, followed by an almost horizontal terrace extending several hundred meters from the Göta River.

The slide that destroyed the factory, had a total length of about 280 m, a depth to the sliding surface of 5 and 10 m in the active and passive zones, a maximum depth of 22 m, and an inclination of the 266 m long sliding surface equal to 6.1°. The calculated safety factor was 2.2 for pore pressures at the sliding surface rising to terrain, and 1.0 if the pore pressures corresponded to 4.3 m above ground level.

The upstream slide had a total length of about 230 m, a depth to the sliding surface 3 and 6 m in the active and passive zone, and a maximum depth of 20 m. The inclination of the sliding surface was 6.9° in the upper 134 m and 1.2° in the lower 86 m of the sliding surface. The calculated safety factor was 2.4 for the case where pore pressures corresponded to a water rise up to ground level, and 1.0 if 4.3 m above ground level.

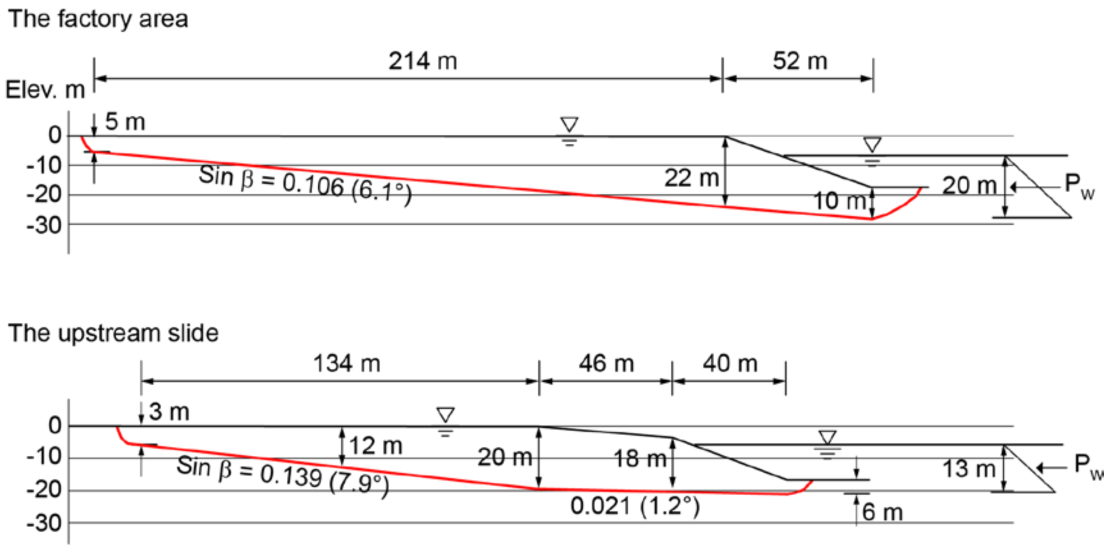

Figure 13. The Göta landslide.

The 1977 Tuve Slide, Sweden

An initial slide occurred suddenly on 30 November 1977 (Larsson and Jansson, 1982). Within about five minutes, the appearance of an area of 270.000 m^2 changed completely. As a consequence of the slide, 65 domestic houses were completely destroyed, and another 84 houses condemned for safety reasons. Nine peoples died from injuries caused by the landslide. Fortunately, most of the people who lived in the area, were absent on the afternoon when the disaster took place.

Most of the borings recorded from Swedish sources (SGI, Swedish Geotechnical Institute) were performed in the slide area, and give results with considerable variations and uncertainty. This made it difficult to choose reliable shear strength parameters from the data for a stability analysis of the landslide. However, in the spring of 1979, NGI received a series of undisturbed cylinder samples for research on the Tuve clay (NGI, 1979). The samples, from a depth interval between 15 and 20 m, were taken at a location well outside the slide area. They gave the following approximate results with small variations: plastic limit = 35%, liquid limit = 65% , natural water content = 70%, and unit weight = 16 kN/m^3. Furthermore, active and passive triaxial tests on specimens between 15.5 and 20 m provided a set of effective stress strength parameters χ and $sin\varphi'_M$, as described above.

A few alternative stability analyses were carried out for the Tuve landslide which had a total length of approximately 220 m long. This study consisted of the 180 m segment between the active and passive zones with an inclination of 6°. The depths to the sliding surface were 11 m at the active zone and 19 m at the passive zone. The calculated factors of safety were equal to 3.2 if the pore pressures were assumed to correspond to a piezometric elevation up to ground level, and 1.0 if the piezometric elevation was 5.3 m above ground level. The calculated artesian pressures were assumed to be average values for the entire sliding surface.

As for the Surte landslide, the total slide length at Tuve also extended over a long distance, namely 120 m in front of the 180 m long part included in the stability analysis. If one combines these areas in a conventional stability calculation, this would require unreasonably high pore pressures to yield a safety factor equal to 1.0. Nevertheless, both zones took part in the slide, and there must be an explanation for the failure.

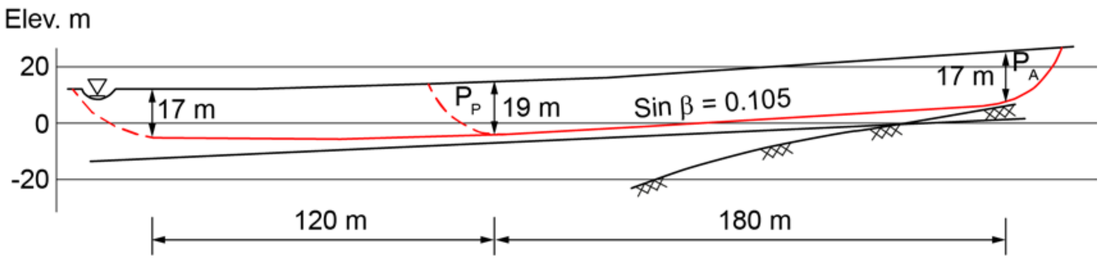

Figure 14. The Tuve landslide.

Typical for the Surte and Tuve landslides, the secondary parts of the sliding consisted of a block with almost constant height, and a constant capacity with respect to resisting a passive earth pressure. As a result of isostatic land uplift and erosion over thousands of years, this clay block had undergone a decreasing vertical effective stress under conditions of no lateral yield. This is exactly the way a clay specimen is brought to failure in a passive triaxial test, indicating that the clay in the sliding block was exposed to a pure passive shear involving no shear stress acting on the vertical and horizontal planes. Hence, the explanation for the failure might be that before the slide started, this block ensured the stability of the area behind, such that mobilization of shear strength under the block was not necessary. Under such circumstances, this lower part of the slide would act as a compression member and then fail as a house of cards at the same time as the area above.

Discussion of stability calculations

The two Norwegian slides at Baastad and Risa show safety factors of 1.0 for pore pressures corresponding to a piezometric level of 1.2 and 1.9 m above terrain, respectively. However, the reality of the calculated pore pressures is encumbered with an obvious uncertainty. As pointed out by Bjerrum et al. (1969), the downward percolation of surface water is usually negligible with respect to formation of quick clay. The leaching of salt water that transforms the clay to quick clay is, in almost all known cases, caused by an upward flow of fresh groundwater, originating from deeper permeable layers or bedrock below. Although reliable pore pressure measurements are limited at these two locations, the magnitude of the calculated pore pressures needed to provide a safety factor of 1.0 are reasonable, if one takes into consideration the precision of the performed calculations.

The landslides at Göta show safety factors of approximately 1.0 assuming the pore pressure at failure corresponds to a water level 4.3 m above the ground surface. The most critical example at Tuve shows a safety factor of about 1.0 assuming that the pore pressure at failure corresponds to a water level 5.3 m above the ground surface. For the landslide at Surte, the calculated safety factor is 1.0 if the water level rises to 4.0 m above ground surface.

To verify the whether or not an artesian pore pressures of 40 to 53 kPa was possible in the Gothenburg valley, samples were taken in the sampling location in Gothenburg City, and artesian pressures of 50 to 70 kPa were measured at depth of about 25 m. At this location, the terrain level was 18 to 22 m above sea level. The depth to the groundwater table was about 2 m, and the observed artesian pore pressure increased steadily with depth. Persson *et al.* (2011) also reported measurements of up to 7 m artesian water pressures at locations in the southern part of the Göta River valley. At Tuve, very high artesian pressures were measured in the slide area (Berntsson and Lindt, 1981). However, the values of the pore pressures prior to the slide are uncertain. Alte *et al.* (1989) presented recorded artesian pressures of 90 to 100 kPa down to 60 m depth at a test site in Gothenburg.

Even with only these limited actual measurements, it is the authors' opinion that the measured pore pressure are reliable and that the magnitude of the pore pressures derived to obtain factors of safety of unity in the landslide case studies are realistic. One limitation in the analyses is that the expected most realistic slip surface was analysed. It may not be the absolute most critical sliding surface. Therefore, the calculated artesian pore pressures may be overestimated.

According to Swedish experts, high artesian pore pressures are believed to have played an important role in triggering the Swedish landslides. In addition, kinetic energy, due to the moving soil mass during the first part of the slide, has been assumed, for instance by Lundstrøm, (1981), to be the only possible explanation for the failure in the almost horizontal, lower part of the Surte and Tuve landslides. This explanation assumes that there must have been a certain time interval between the two main parts of the slide. However, there were no observations that could confirm this assumption.

The relation between pore pressures and calculated safety factors in the five case studies show the importance, even the necessity, of pore pressure measurements for the assessment of the stability of quick clay slopes. It is reasonable to believe that landslides like those reported in this paper occur at locations where non-hydrostatic pore pressures conditions exist. The authors consider that ignoring pore pressure measurements and doing stability calculations using hydrostatic porewater pressures from the groundwater can lead to catastrophic slope failures.

Conclusions

The paper presents the relationship between the normalized undrained shear strength parameters used in the ADP analysis and the effective stress strength parameters, friction and attraction. This makes it possible to include pore pressure in an effective stress stability analysis, as a more realistic alternative to the ADP analysis. In a quick clay, however, the failure criteria will differ from the conventional Mohr Coulomb failure criterion, because only the vertical stress contributes to friction in active shear, and only the horizontal stress contributes in passive shear.

A good reason for applying an effective stress analysis especially for flake slides is that they often occur in clay slopes which can have existed nearly unchanged for very long periods of time, and then suddenly fail almost for no apparent reason. Consequently, the effective shear strength parameters are fully mobilized, and the magnitude of effective stresses at failure are determined by the *in situ* pore pressures. Effectively, one then performs a drained analysis, with increasing reliability as the calculated value of SF approaches 1.0.

Based on undrained triaxial tests on clay samples from two locations in the Göta River area in Sweden, the approximate representative values of $sin\varphi'_M$ and χ are both equal to 0.35. For Norwegian quick clays from two different locations $sin\varphi'_M$ and χ were found to be respectively 0.49 and 0.18 at one location 0.55 and 0.08 at the other site. These effective stress parameters were used in an effective stress stability analysis for two Norwegian and three Swedish quick clay landslides. These five slides were 40 to 67 years old. Therefore, geotechnical data are sparse, and pore pressure conditions prior to the slides are not well known. Consequently, somewhat approximate factor of safety calculations were carried out. The main objective of the study was to investigate the magnitude of the pore pressures needed to give a safety factor of unity for each landslide, and to discuss the realis of these pore pressures at each site.

The two Norwegian landslides at Baastad and Rissa required an artesian pressure, expressed as the rise in the groundwater level above ground level, of 1.5 and 1.8 m.

Corresponding values for the Swedish landslides, Gøta, Tuve and Surte, was found to be 3.6, 5.3, and 3.9, respectively. Since the occurrence of quick clay, in almost in all known cases, is a result of streaming ground water and increasing artesian pore pressure with depth, the values for the Norwegian landslides seem realistic. Due to lack of actual data, the values calculated for the Swedish slides are difficult to evaluate. However, measured artesian pore pressures of 50 – 70 kPa at a location in Gothenburg City, and reported artesian pressures up to 70 kPa in the southern part of the Göta River valley support the hypothesis of rather large artesian pore pressures at the location of the three landslides.

The slides at Surte and Tuve extended a long distance in front of the part included in the stability analysis. If these areas, if the stability analysis were run in a conventional way (i.e. based on solely the undrained shear strength), this would require unrealistic high pore pressures to explain a safety factor equal to 1.0. In this respect, a theory put forward is that the front parts of the landslide, because of a historic vertical unloading and horizontal loading under conditions of no lateral yield, have been brought into a condition of pure passive shear involving no shear stress acting on the horizontal and vertical planes. Under such circumstances, the clay blocks would act as compression members and fail like a card house simultaneously with the upper part of the slide.

Pore pressures have a dominating influence on calculated safety factors, especially in an effective stress analysis. Likewise, in order to have full control on effective stresses in an ADP analysis, one cannot neglect the pore pressures. Consequently, piezometer installations or pore pressure measurements (e.g. with the piezocone) should be included in all site investigations where quick clay is found.

In closing, it is appropriate to quote the last sentence in Bent Jacobsson’s (1952) comprehensive paper on the landslide at Surte:

"The main conclusions to be drawn from the [Surte] slide are that a small slide can under certain conditions grow to a great extent, and that we have to take the pore water pressure into consideration in a greater degree than hitherto".

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