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Slope stability problems in Denmark - a joke or dire reality?

Problèmes de stabilité de pente au Danemark - une plaisanterie ou une grande réalité ?

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ABSTRACT: Contrary to common belief, even Denmark suffers slope failures in natural and man-made slopes. In a majority of the failures water is the catalyst combined with oversight or neglect of fundamental soil mechanics principles. The paper describes three cases of slope failures, two natural and one man-made, and the subsequent forensic geotechnical engineering investigations and remediation.

1 INTRODUCTION

Even though Denmark is a level country (the highest point is about 177 m above sea level) both natural and excavated slopes fail, as evidenced in Figure 1. Failure in the slopes normally occurs after a wet period with rainfalls, which increase the pore pressure in the slopes and decreases the effective shear strength of the soils.

The paper describes three case histories with failure of slopes some 12 to 20 m high: One with a warning prior to the slope failure and two without a warning. The result of the failures was rather dramatic, as app. 100 m of a highway lane disappeared, a two-family house was severely damaged and finally, extensive earthwork was required in remediation. Comprehensive investigations and analyses were performed as all the parties involved in the cases requested a highly competent technical basis for the determination of the reasons for the failures and the subsequent remediation.

The case histories amply demonstrate that our capabilities in terms of testing do not always meet our expectations. But back-calculation based on well-winnowed experience proved to be invaluable, hence re-iterating the need for case histories in geotechnical engineering. Even smaller slope failures carry loud messages as seen from Figure 1.

Figure 1. Dire reality of a minor slope failure leading to total demolition of a one-family house in Denmark

2 PORE WATER PRESSURES AND STRENGTH PARAMETERS

Slope failures are quintessential manifestations of the ultimate limit state. To assess the risk of failure and to make sense of a forensic geotechnical engineering study three items must be understood and quantified: the geology of the site, the groundwater and pore water pressure regimes, and finally the strength parameters for the individual soil layers.

As most failures occur with little or no warning (and are surprisingly unexpected!) the forensic investigations are hampered by the fact that the pore water pressures and ground water conditions at failure are unknown. Even with thorough, previous instrumentation these would be difficult to assess and subsequent measurements are subject to biased back calculations. However, based on systematic recordings of rainfall and the water table position at the site a likely adverse distribution of pore pressures prior to failure may be established.

The same difficulties are associated with the determination of strength parameters. This is particularly true for clays of Tertiary Age, where the assessment of effective strength parameters is notably difficult due to the very low coefficient of permeability.

To obtain reliable strength parameters routine tests do not suffice. To counteract the inevitable sample disturbance, even associated with the best possible sample retrieval, it is necessary to re-establish the stress history of the sample prior to the failure phases in shear box or triaxial testing. The same problems are associated with oedometer testing to establish the preconsolidation stress for the sample.

In the present cases the strengths were determined by a combination of laboratory and field testing and back calculation of previous slope failures in the same deposits. The failure area and the adjoining non-failed slopes were carefully mapped. Based on these data, the geological profile established from geotechnical boreholes and assessment of the strength parameters for the "stable" soil types back calculations were carried out for the slope failures. The parameters for the failure susceptible soils were "fitted" to match the recorded failure mechanisms. The resulting parameters are shown in Table 1.

Table 1.Summary of characteristic strength parameters from tests and back calculations

Parameter		Sundsøre: Tertiary, marine clay		Høgsholtparken: clay fill/solifluction deposit		Type of test
		Test	Back calculation	Test	Back calculation	
w	%	19 - 32		17 - 36		Classification
I_P	%	17 - 42				Classification
s_u	kPa	50	35	50	35	Field vane
c'	kPa	0	0	14	7	Triaxial
φ'_{tr}	°	35 - 23	19.2	24	24	Triaxial

It is apparent from Table 1 that the strength parameters determined from the tests exceeded the values from back calculation. The uncertainty associated with the pore pressures at the time of failure and difficulties associated with "correct" testing procedures are most likely the main sources of the discrepancy. To err on the safe side, the strength parameters from the back calculation were used in the subsequent remediation.

3 SUNDSØRE

3.1 Site description

The Sundsøre site is situated in north-eastern Jutland where Highway 472 runs parallel to the top of a 21 m high slope towards Hvalpsund cove. Over a period of eight years, prior to the failure in 1995, a series of smaller failures had taken place. As a result the distance from the highway to the crown of the slope was down to six metres. The Road Authorities had therefore instigated a review of the stability of the slope.

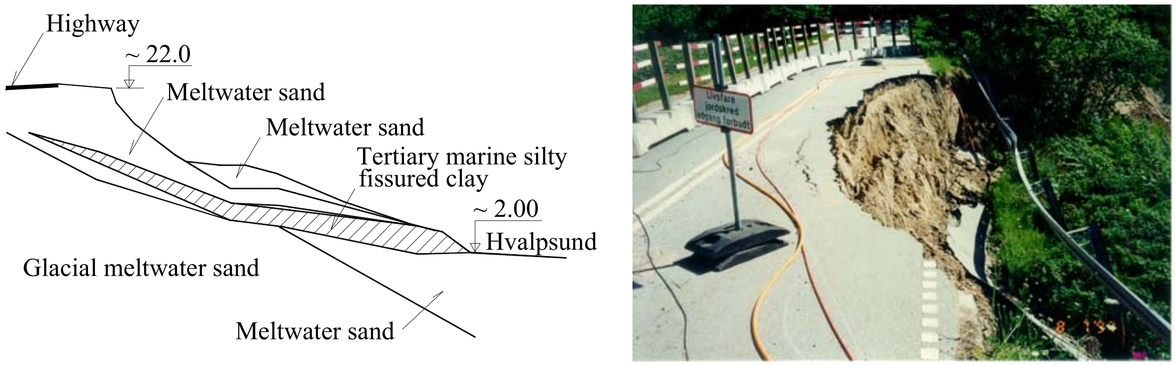

Figure 2. (a) Cros section at slope at Sundsøre prior to failure; (b) Second failure involving half the highway

Geotechnical borings in the slope revealed rather complicated slope geology. One boring at the crown showed 1.0 m of leached, glacial clay till above 6.5 m of glacial meltwater sand with grain sizes and gravel contents increasing with depth. This was underlain by some 2 m thick wedge of Tertiary, marine silty fissured clay. Below the Tertiary clay, glacial fine to coarse meltwater sand was found to a depth of 14.5 m below the crown. The borings carried out down the slope confirmed the presence of a floe of Tertiary clay, wedged at an angle along and across the slope profile. A cross section of the slope appears from Figure 2.

Based on the site investigation and a preliminary assessment of strength parameters and ground water conditions it was found that the factor of safety for the slope was below the requirements of the Danish Code DS 415 (1984). For safety reasons half the highway was therefore blocked pending remediation work on the slope.

3.2 The dilemmas presented by laboratory testing

As part of the forensic investigation laboratory testing was carried out on the Tertiary clay at Aalborg University. The samples were retrieved as 70 mm diameter undisturbed tube samples from the borings and from block samples excavated in the slope debris.

Strength and deformation tests on the Tertiary clay are notably difficult to carry out due to effects from sample disturbance, inherent fissuring and long consolidation times. The difficulties became apparent when the oedometer test (w_i = 26.9%; I_P = 17.0 %) was carried out. Pre-consolidation pressures in the range σ'_{pc} = 500 to 2000 kPa, were estimated depending on the method applied!

To check this ambiguity two tests were subsequently carried out on one of the block samples (I_P = 30.1 %). In one test the clay was remoulded at its initial water content (w_i = 33.2 %) and in the other the clay was reconstituted at a water content of w_i = 54.0 %. The remoulded sample suggested σ'_{pc} = 400-1000 kPa depending on method, whereas the reconstituted sample gave σ'_{pc} = 5-10 kPa. Notably, an evaluation of the latter based on the rate of secondary consolidation would suggest σ'_{pc} =1000 kPa!

In all three cases the value of the modified compression index C_{ce} (= $d\varepsilon/d\log\sigma'$) was about the same, ie. 11.3%, 12.8% and 12.5%, taking the difference in I_P into consideration. The same applied to the modified secondary compression index $C_{\alpha e}$ (= $d\varepsilon/d\log t$), where values of 0.5, 0.5-0.6 and 0.5% were found for consolidation stresses exceeding some 1000 to 2000 kPa.

$CU_{u=0}$ triaxial tests were carried out on 70 mm diameter specimens (I_P = 17.0 %) with smooth pressure heads and a height/diameter ratio of H/D = 1, in accordance with Danish practice. However, as very high values of secant friction angle, $\varphi'_{tr,\,secant}$ = 34.1 and 37.9° were achieved, the third specimen was tested with H/D = 2 (I_P = 30.1 %). This resulted in $\varphi'_{tr,\,secant}$ = 35.9° and surprisingly no reduction from "live" fissures or the change in H/D ratio could be detected.

Based on an extensive database of back calculated slope failures in Tertiary clay, an empirical equation, $\varphi'_{tr,\,secant} = 45° - 15° \log I_P$, has been derived (DGI, 1991). This is customarily used in the absence of laboratory testing. Here, this lead to values of $\varphi'_{tr,\,secant}$ = 26.5° and 22.8°, respectively! Subsequent research (Thøgersen, 2001) on Tertiary clays indicate that the difference between the empirical relation and laboratory triaxial tests may be due to osmotic pressure generated during testing. If the pore pressure at the base of the sample is measured and the site pore fluid (or compounded water) is used instead of deionised water, the test values are very close to the empirical formula.

In the present case the values from back-calculation were luckily considered most reliable and used in the risk assessment for the slope.

3.3 Solution

A solution involving a decrease of slope angle was not considered feasible, and hence the road authorities decided to permanently change the alignment of the road away from the slope. The wisdom of the decision was borne out by a new slope failure taking place shortly after the decision and without warning. Half the highway disappeared over a length of about 100 m (cf. Figure 2b).

4. HØGSHOLTPARKEN, VEJLE

4.1 Site description

The slope at the residential area "Høgsholtparken" is situated in the northern part of Vejle in eastern Jutland. The slope was about 12-15 m high and to accommodate a series of new domestic houses, up to 16 m of the slope was removed at the bottom of the slope (cf. Figure 3a).

The slope could not be cut back due to a series of existing apartment buildings close to the crown of the slope. Hence, an up to five m high reinforced sand retaining wall was established at the base of the slope with the new houses as close as 2 m away. However, the reinforced slope was not stable, and over a period of time a series of smaller slope failures took place up to October 1998, where the gable on one of the houses collapsed due to a major, sudden slide (fortunately without any bodily harm to the inhabitants), as seen in Figure 3b.

The forensic investigation with geotechnical borings carried out in the slope showed recent sand and clay fill deposits (or solifluction deposits) to a depth of some 1-3 m below the surface. This was underlain by rather plastic glacial clay till to a depth of 12 m below the surface. Glacially, fine to medium meltwater sand was found 12 to 15 m below ground surface (end of borings). The clay till deposit was interlaced with thin horizons of medium to coarse sand till. The primary groundwater table was found to be well under the level of the foot of the slope. A cross section with interpreted soil boundaries is shown in Figure 3a.

The residents in the damaged building were evacuated immediately after the slope failure and a monitoring programme for the slope was initiated. The primary goal was to record the water pressures in the slope with a view to assessing the risk to the other buildings at the site.

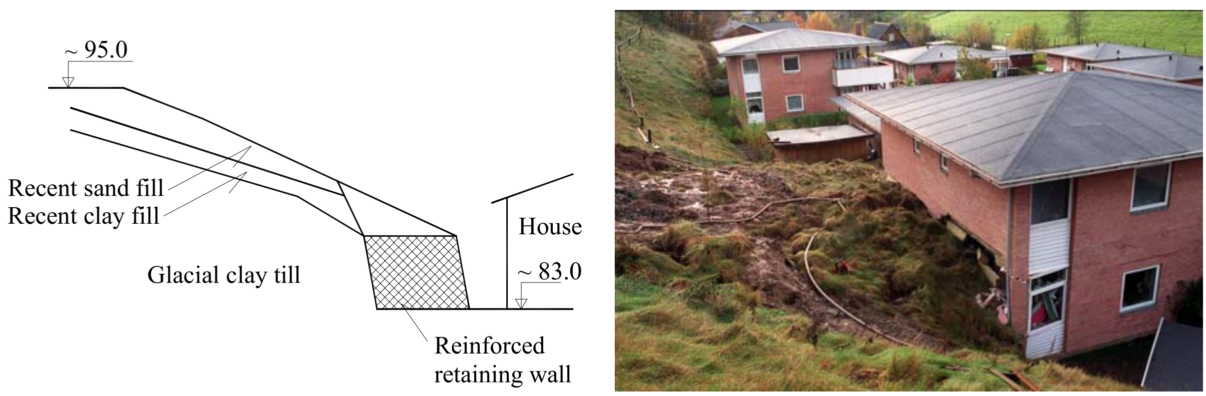

Figure 3. (a) Interpreted soil boundaries for slope at "Høgsholtparken"; (b) Damage from the slope failure

4.2 Laboratory testing

Oedometer and triaxial tests were carried out on the two types of soils believed to be involved in the failure, the clay fill/solifluction deposits and the relatively plastic clay till. The oedometer tests on clay till indicated σ'_{pc} = 400 - 450 kPa, $C_{c\varepsilon}$ = 10.3 - 10.5% and $C_{\alpha\varepsilon}$ = 0.30 - 0.35%. The triaxial $CU_{u=0}$ tests indicated $\varphi'_{tr,\ tangent}$ = 23.9°, $c'_{tangent}$ = 13.6 kPa for the clay fill and $\varphi'_{tr,\ tangent}$ = 36.3°, $c'_{tangent}$ = 5.2 kPa for the clay till, respectively.

4.3 Solution

The reason for the failure was clearly a build-up of pore pressure during heavy rainfalls in the interface between the top deposits (of sandy, gravelly, fill and clay) and the underlying clay till in combination with an inadequate soil reinforced retaining wall at the foot of the slope.

Due to the limitations imposed by the existing building at both the top and the foot of the slope, it was not possible to reduce the slope angle per se. Instead two different solutions were tendered:

- A traditional Danish solution with a tied back sheet pile wall at the top of the slope (retaining the crown of the slope) allowing a reduction of slope angle
- Soil nailing of the existing slope maintaining the slope angle. This solution invites itself, as the very competent clay till is at shallow depth

Soil nailing has seen no proliferation in Denmark and hence, the calculation principles had to be re-evaluated in connection with the tender project. Both British and French standards for reinforced soil use total factors of safety as opposed to the partial factor of safety system imbedded in the Danish Code of Practice. A number of test cases were calculated for comparison of methods and it was concluded that by and large the number of nails and the required lengths were similar. The comparison showed that a total factor of safety >1.4 would be equivalent to the Danish system with partial factors on loads and material parameters and a factor of safety 1 in the stability calculation.

Although the soil nailing solution was technically feasible the (Danish) traditional solution, with a sheet pile wall at the top, was preferred as this allowed a transparent pricing of the remediation works.

5 HOPTRUP

5.1 Site description

A minor slope at the top of a 15 m high slope along Highway 519 in the southern part of Jutland took place in February 2002. As seen on Figure 4 the bicycle lane disappeared over a length of some five metres causing major concern for the safety of the highway. In the failure escarpment water was visibly seeping from the soil layers below the road. At the foot of the slope some failures had taken place too, and the immediate reaction was that failures in clay deposits due to dead elm trees (caused by elm disease) might be the main reason for the slope failure.

Figure 4. (a) Slide failure blocking private road; (b) The missing bicycle lane; (c) The hunt for Tertiary clay floes

The geology on site is rather chaotic, due to the ice movements during the latest glaciation (responsible for the attractive, highly undulating landscape). Thus, fat Tertiary clay deposits of varying thickness may be interbedded in glacial sand and gravel deposits. Due to very difficult access conditions because of the failure, only two borings were carried out, one at the top of the slope and one halfway down. The top boring indicated plastic Tertiary clay at the top of the slope underlain by glacially deposited sand. The midway boring indicated sand only and could not contribute to a plausible explanation of the failure. However, the presence of Tertiary clay and sand combined with excess water from rainfall and possibly defunct drain systems along the highway seemed the most likely explanation (notice exposed sewage discharge pipe on Figure 4b).

5.2 Remediation solution

A rather untraditional and proactive approach was used in the remediation. In a very co-operative spirit between Road Authorities, geotechnical consultant and contractor, remediation was carried out by trial and error. The premise was that removal of the fat Tertiary clay floes and an effective drainage of water from the top of the road to the foot of the slope was the cure.

To reduce the uncertainty in the extent of remediation a number of test pits were excavated by back hoe at the foot of the slope. These revealed the presence of up to two meter thick Tertiary clay floes over sand, substantiating the intuitive explanation for the failure. A very experienced geotechnical engineer (the second author) instructed the contractor's crew in the hunt for the culprit, the Tertiary clay, which was found in abundance with thicknesses ranging from 0.5 to 1.5 m along the slope. Some of the floes had an area extent of some 200 m^2 and were sandwiched in between sand layers. Due to the significant differences in height, build-up of pore water pressures could reduce the effective stresses and strength in the sand layers to near zero.

Where the clay could not be removed it was perforated and the holes filled with sand to prevent future pore pressure build-up to take place. The excavated clay was placed at an inclination of 1:15 at the foot of the slope using a total of 5000 m^3 of sand as replacement in the slope at an inclination of 1:2.2 and with re-establishment of the 3 m wide private road approximately at mid-height. To further reduce the risk of future failures, the existing sewage line from the road was connected to two sets of four metre deep, one meter diameter drainage wells down the slope.

The solution was a fast track intuitive and robust solution, where "proper" testing of strength and deformation parameters was deselected. It was estimated that the potential gains in reduction of clay replacement would not justify the expenses and delays associated with the soil testing!

6 LESSONS LEARNED

In an editorial in Ground Engineering it was postulated that "You pay for the geotechnical investigations you *don't* carry out". It may seem trite but unfortunately the truth of it is repeatedly proved by the occurrence of failures, remediation and unnecessary spending on civil engineering structures. But geotechnical investigations and laboratory testing, per se, are not sufficient.

In line with well-winnowed Danish tradition interplay with engineering geology is a prerequisite for understanding the potential risks of sites. As amplified by the present case histories investigations should include the neighbouring structures and geological features.

Strength and deformation parameters obtaining by laboratory testing should be critically reviewed with emphasis on the quintessential back-calculations, provided the latter are not accepted on face value.

7 REFERENCES

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