Chapter

Techniques to Evaluate and Remediate the Slope Stability in Overconsolidated Clay

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Abstract

In this chapter, the origin and remediation of an important sliding in the overconsolidated Boom Clay in Kruibeke (Belgium) is discussed. Local and environmental factors caused an unstable slope about 30 m deep, A larger sensitivity to erosion resulted finally in the instability of the slope. Because of the formation of fine cracks in the soil there was a possibility for the water to penetrate in the clay close to the surface, resulting in the presence of higher water pressures. Also, the presence of the excavator on top of the slope during exploitation had an important impact on the stress state of the soil. Both an analytical and numerical approach were used to estimate the factor of safety. Because of the change of the soil characteristics, the factor of safety decreases, which can be estimated through a numerical analysis (using the Strength Reduction Method). This chapter also discussed the applied techniques for the remediation using numerical analysis. Also, the importance of the field test is discussed. An integrated approach, using numerical analysis and field tests in combination, is capable of predicting the instability. This approach can also be used to evaluate the stability of the slope after remediation.

Keywords: slope stability, monitoring, in situ testing, DMT (dilatometer), rehabilitation

1. Introduction

A natural or an artificial slope in a clay deposit results in a change of the morphological properties and the stress state in the soil This evolution of the stress state can be measured by in situ tests considering the variations over time of the lateral stress at rest K_0 [1].

K_0 is an important parameter in the design and monitoring in many geotechnical applications (retaining walls, building foundations, and definitely the stability of slopes). For the evaluation of the stability of slopes, it can be estimated out of the results of field tests (Pressuremeter Tests [1], Vane Tests and Piezometer Cone Tests [2], Dilatometer Test [3–5] or numerical modeling, usually considering elasto-viscoplastic models.

Although the analysis of the lateral stress, in case of slope stability, also can be made based on the results of laboratory tests [6–8], one of the main problems is that K_0, due to the natural variations of horizontal and vertical stresses, at each location depends on the actual stress conditions.

Silvestri [9] performed a post slide analysis using the critical stress curve method for two failures that occurred in overconsolidated sensitive clay. The results of the analysis yielded the shear strength parameters. They found that the strength at the time of failure was due to the fissured nature of the clay, the spreading of the discontinuities caused by stress relief after excavation, and subsequent desiccation of the exposed clay mass.

Grefsheim [10] made a back analysis calculation from full scale landslides in overconsolidated clays using stress-strain plots based on the results of laboratory tests. The author compared the results of his research with those for similar overconsolidated clay soils available in the literature (London Clay and Panama Canal Clay). The correlation was made based on the relationship between shear strength and the plasticity index.

Besides the stress state of the soil, also the groundwater conditions and groundwater pressure plays an important role in the stability of slopes in clay.

The groundwater pressure also determines the stress state in the soil and plays an important role in the equilibrium of the slope.

Chandler [11] discussed twelve case records of failures of slopes in the Upper Lias Clay cutting slopes. The author evaluated that the groundwater behavior based on the results of piezometric measurements. The results of effective stress stability analyses revealed a similar pattern to that found earlier in London Clay (which has the same geological origin as the Boom Clay).

Dehandschutter [12] studied the effect of fracture-development in the Boom Clay on the fluid-flow changes (fracture permeability) and mechanical strength of the soil. This fracture-development is partially due to the soil relaxation after excavating the pit.

D'Elia et al. [13] (1996) and Totani et al. [2] used the Dilatometer Test for the evaluation of the soil conditions after a sliding occurred.

Slope stability is influenced by many factors, among which are subsoil structure, hydrologic conditions (e.g., soil moisture, groundwater position), climatic factors (e.g., rainfall), and the hydraulic boundaries. This chapter presents a multi-disciplinary approach for the remediation of the stability of slopes, which includes a numerical analysis based on the results of the in situ geotechnical monitoring, laboratory geotechnical tests (for the determination of the shear resistance and stiffness of the soil) and piezometric measurements. Thus, a reliable 2D model of the subsoil can be obtained, with well-defined boundaries on which it is possible to apply appropriate hydraulic conditions. This geotechnical model has been used for studying the remediation of the stability of the slope.

2. Methodology

2.1 General

In this chapter, a comparative analysis is made considering the results of in-situ tests in order to evaluate the sensitivity of these testing methods for the evaluation of slope stability.

The research was conducted in Kruibeke (Belgium) (**Figure 1**.). This research site has overconsolidated Boom clay. In the period of 1963–2010, 30 meters deep pit was excavated in the Boom Clay. The clay was used for the fabrication of expanded clay granulates.

In 2010, the borders of the concession were reached, and the exploitation of this pit was stopped. One year later, in 2011, a first (limited) instability of the slope of this pit occurred (sliding). Apparently, it became clear that more stability problems

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Figure 1.
Layout of the slide (2014) (as adapted from Peiffer (2016) [5]).

could be expected. In 2012 and 2013, additional (limited) slidings occurred. In the beginning of 2014, it was decided to set up an extensive monitoring program in order to evaluate the risk of further instability of the slopes.

In addition to the monitoring of the settlements and the pore water pressures in the environment, in-situ-tests were carried out on a regular basis to evaluate the stability of the slope.

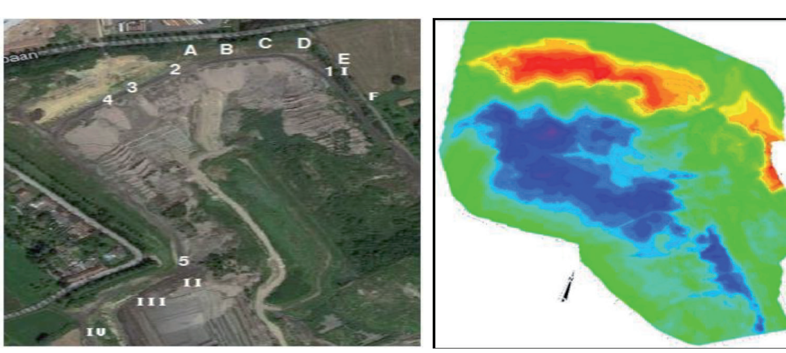
For the analysis, cone penetration tests (CPT) and dilatometer tests (DMT) were executed. A layout of the test locations is presented in **Figure 2a**. Four campaigns have been organized:

- January 2014—DMT 1 to 5 (before the major sliding) (**Figure 2a**)
- July 2014—DMT A to F (about 1 month after the sliding) (**Figure 2a**)
- January 2015—DMT I to IV (after stabilisation) : only DMT 1 was conducted in the neighborhood of the sliding (**Figure 2a**)
- March 2016—DMT1' (in the immediate neighborhood of DMTI)

2.2 Site investigation: CPT-tests and DMT-tests

2.2.1 Cone penetration test (CPT)

The Cone Penetration Test (CPT) is internationally one of the most widely used and accepted test methods used to determine geotechnical properties of soils and the soil strategy.



vertical displacement slope			
Number	Min.difference	Max. difference	Color
1	−9.000	−8.000	
2	−8.000	−7.000	
3	−7.000	−6.000	
4	−6.000	−5.000	
5	−5.000	−4.000	
6	−4.000	−3.000	
7	−3.000	−2.000	
8	−2.000	−1.000	
9	−1.000	0.000	
10	0.000	1.000	
11	1.000	2.000	
12	2.000	3.000	
13	3.000	4.000	
14	4.000	5.000	
15	5.000	6.000	
16	6.000	7.000	
17	7.000	8.000	
18	8.000	9.000	

Figure 2.
Photo view of the site with indication of the DMT test locations (a) and presentation of the magnitude of the vertical movement (b) (as adapted from Peiffer (2016) [5]).

The test method consists of pushing an instrumented cone, with the tip facing down, into the ground at a controlled rate of 2 cm/s. The typical cone tip has a cross-sectional area of 10 cm^2, corresponding to a diameter of 36 mm (**Figure 3**).

During the CPT-test, the cone resistance q_c is measured over the whole depth of the soil profile, generally up to more than 20 m below the surface or until the cone reaches a hard layer. A typical setup for the execution of a CPT-test is presented in **Figure 4**. The cone is pushed in the soil using a counterweight (in general, a heavy truck) that is positioned horizontally. The cone resistance is recorded using force sensors in the tip. These values allow for a good understanding of soil behavior. In particular, the strength of the soil and the relative density (compaction degree) can be determined.

Figure 3.
The CPT-cone.

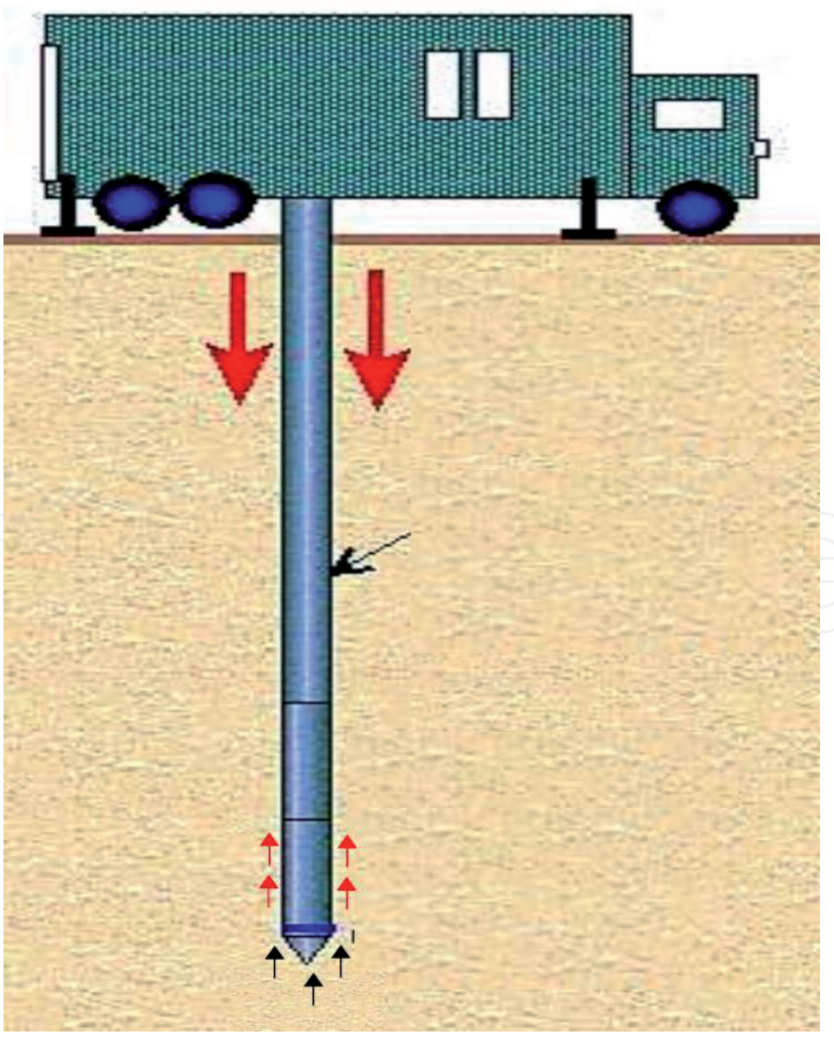


Figure 4.
Execution of the CPT-test.

A typical application of the CPT-test is the evaluation of the effect of the change in stress state on the strength of the soil by doing a test before and after, for example, ground improvement works.

2.2.2 Dilatometer test (DMT)

The flat dilatometer (DMT) is a stainless steel blade with a thickness of about 15 mm, having a flat, circular steel membrane (diameter 60 mm, thickness 0.1 to 0.2 mm) mounted on one side (**Figure 5**).

The blade is connected to a control unit on the ground surface by a pneumatic-electrical tube (transmitting gas pressure and electrical continuity) running through the insertion rods (same rods as for the CPT-test) (**Figure 6**). A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. At the beginning of the expansion, the horizontal pressure A can be measured. After an expansion of 1.1 mm at the center of the flexible membrane, the horizontal pressure B can be measured.

The pressure readings A and B must then be corrected by the values ΔA and ΔB, determined by calibration, in order to determine the soil stresses P0 (vertical position of the membrane) and P1 (expansion of 1.1 mm of the membrane). P0 is used for the determination of K_0, (P1-P0) is related to the stiffness of the soil.

The blade is pushed into the ground using common field equipment as for the CPT-test. The test starts by pushing the dilatometer into the ground. The blade is advanced into the ground of one depth increment (typically 20 cm), and the procedure for the A, B readings are repeated at each depth. A more detailed description of the test procedure is discussed in [4].

The field of application of the DMT is very wide. The immediate determination of the stress state and the stiffness of the soil are important advantages of this test technique.

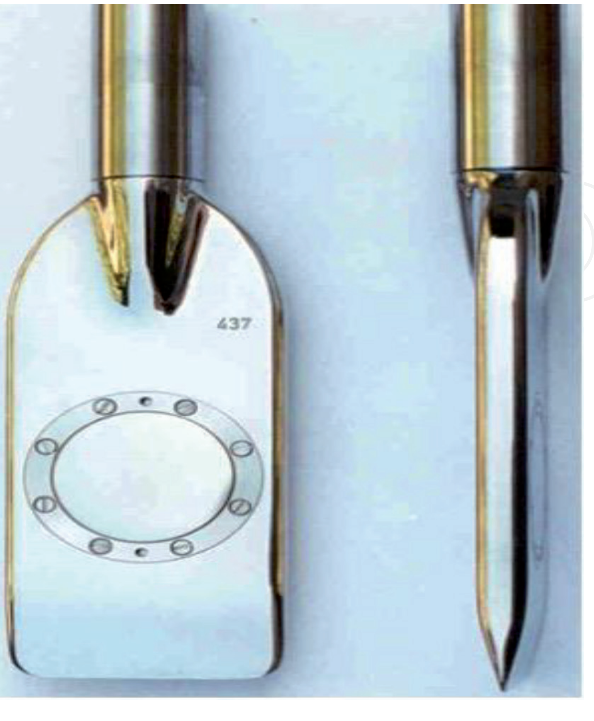


Figure 5.
Dilatometer blade.

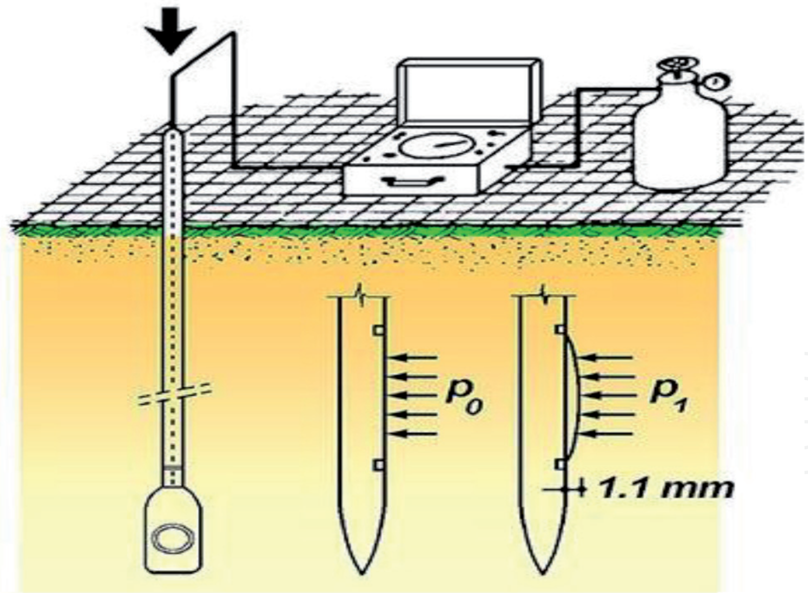


Figure 6.
DMT – Test procedure.

3. Instability of the studied slope

3.1 A general overview of the main sliding

In the next figures, a general view of the main sliding is presented. After excavation of the clay pit till a depth of about 30 m, in the Northern part of the pit, instability (small slidings) of the walls started to appears after some years. The main reason for this instability was the lack of sufficient resistance against horizontal stability. During the period of excavation, the pit became the central dewatering point for the region. Unfortunately, the surface between the upper quaternary layer and the underlying Boom clay was slightly declining towards the pit. The upper part (quaternary layer), with a thickness of about 4 to 5 m, moved towards the pit, resulting in instability. Also, the underlying Boom Clay was affected by this slide.

Figures 1 and **7** present a general view of the slides and the soil movement in 2014. As shown in these figures, a block of 100 m by 50 m in size started to move downward, resulting in a movement of about 9 m down at the corner of the pit.

In **Figure 2**, the extent of the sliding can be noticed. The legend in **Figure 2b** indicates the vertical displacement of the soil mass after the sliding.

After the first sliding in the northwestern part of the pit, a monitoring program was set up. The monitoring includes the execution of CPT-tests and DMT-test. Besides these tests, also the pore-water pressures were measured on a regular basis.

3.2 First sliding – period 2011 - 2012

The first slide occurred in 2011. The size of the slide was limited. In the second half of 2013, one more slide occurred in the neighborhood at point 5 shown in **Figure 2a**., A photo of the slide is presented in **Figure 8**.

3.3 More pronounced instability of the slope (2014)

After a more pronounced sliding in 2013, in January 2014, a monitoring program was established for different points at the top of the slope around the pit. Besides topographic surveys and the measurement of the phreatic water level,

Figure 7.
Detailed impression of the instable slope (as adapted from Peiffer (2016) [5]).

Figure 8.
Photo of the unstable zone after the instability of the slope (as adapted from Peiffer (2016) [5]).

Cone Penetration Tests (CPT) and Dilatometer-tests (DMT) were carried out. The in situ tests were executed in points 1 to 5 (**Figure 2a**).

A lot of in situ tests were conducted. In this section, the results of the DMT tests at point 1, and point 2 (**Figure 2a**) were discussed, because the sliding occurred in this zone. For the general discussion of the results of the in situ tests can be referred to [5].

In June 2014, a big sliding occurred (size of about 100 m x 50 m) between points 1 and 2 (**Figure 2a**).

The extent of the affected zone can be seen in the **Figure 9**. This is a CAD-translation of the zone presented in **Figure 2a**. Two profiles (cross-sections) are also presented in **Figures 10** and **11**. The general Belgian reference system for the heights indicated as 'm TAW' refers to the height above sea level. The arrows on the figures refer to measurement points.

In July 2014, 1 month after the sliding, additional site investigation was done at the top of the slope in the area where the sliding occurred. Six additional points were defined (points A to F) for the execution of CPT and DMT tests.

4. Analysis based on the site investigation

As discussed in 2.2.2., The DMT test can be used for risk assessment concerning the stability of slopes [3]. In recent publications [5, 14], the results of CPT and DMT tests were interpreted in order to evaluate the sensitivity of these testing methods for the evaluation of slope stability.

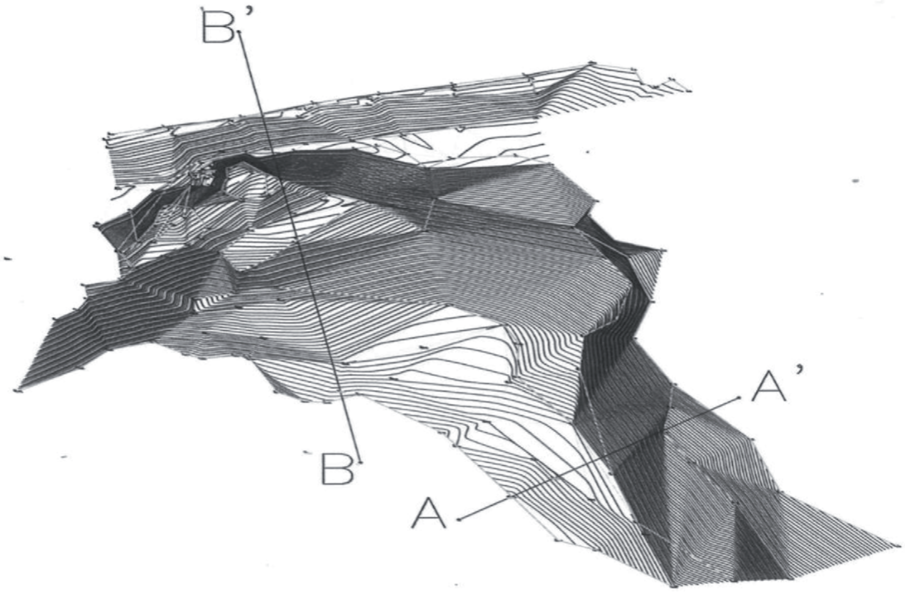


Figure 9.
Print of the NE-corner.

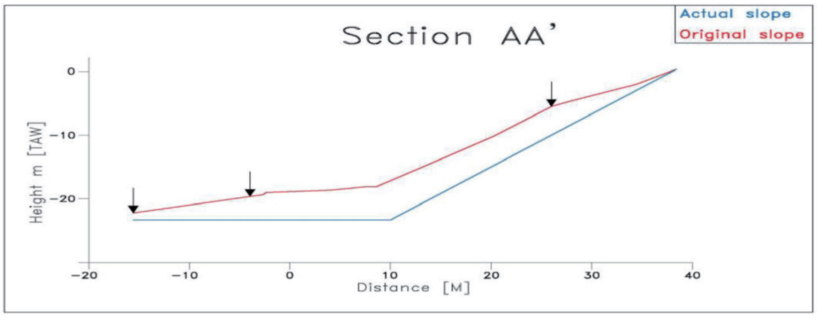


Figure 10.
Section A-A.

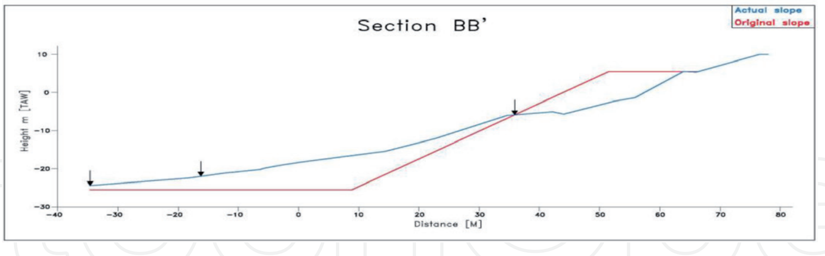


Figure 11.
Section B-B.

The consecutive DMT-measurements revealed an evolution towards instability. After a significant failure (sliding) in one zone in June 2014 (affecting an area of 100 m by 50 m), remedial works in the destabilized zone were undertaken (October – November 2014). After stabilization in 2015, further DMT-measurements were taken in order to investigate the evolution of the stress state of the soil in the potential sliding surfaces.

5. Boom clay

Based on geological investigations and data maps, the upper layer consists of sandy silt (quaternary deposit). Under this layer, the tertiary overconsolidated

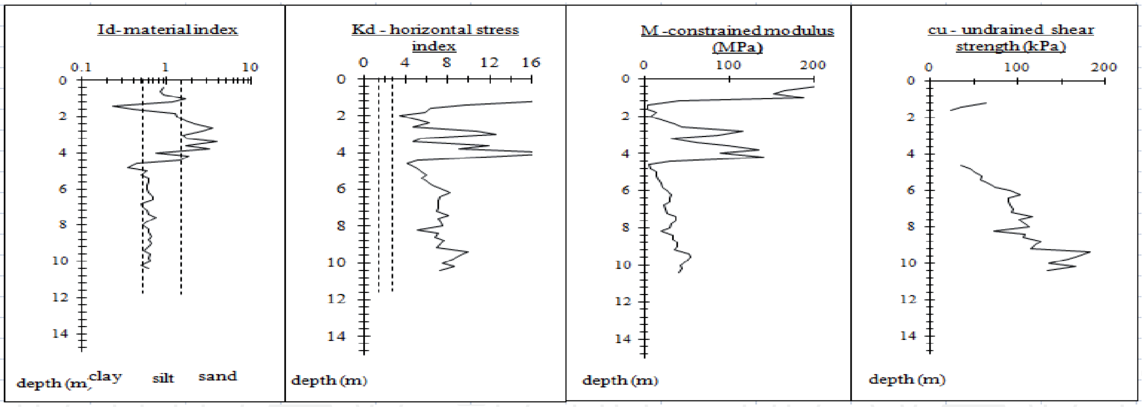


Figure 12.
DMT results in Kruibeke.

Boom Clay can be found. **Figure 9** shows the characteristics of this soil as measured by the DMT.

The tests allow for good identification of the soil. In the column on the left side, the results for the material index Id are presented. Based on Id, it is possible to make a reliable interpretation of the nature of the soil. One can identify in this graph a top layer of sand to silt and starting from a depth of about 4.5 m, a silty clay layer (the Boom Clay). In the second column, the horizontal stress index Kd is presented. A Kd-value between the two dashed lines (Kd = 1.8 and Kd = 2.4) refers to a normally consolidated soil. A Kd-value higher than 2.4 refers to an overconsolidated soil (OC), as can be seen for the Boom Clay.

Also, the stiffness (constrained modulus M) and the strength (undrained shear strength c_u) can be determined out of the basic measurements of the DMT. Both an undrained shear strength of 100 to 150 kPa and a constrained modulus up to 35 MPa refer to very stiff clay.

The profile confirms the results of the undisturbed sampling of the soil. This section focused on the horizontal stress index Kd. This parameter reflects the ratio of horizontal stress to vertical stress (**Figure 12**).

The phreatic level is at a depth of about 1.5 m. The thickness of the clay at the bottom of the pit is about 10 m. This thickness is sufficient to resist the upward artesian water pressures.

During the period of excavation, the pit became the central drainage point for the region. Unfortunately, the surface between the upper quaternary layer and the underlying Boom clay was declined slightly towards the pit.

6. DMT as a tool for the detection of slip surfaces – the DMT-Kd-method

The principle of the DMT-Kd-method for the detection of slip surfaces have been discussed by many authors [3, 13].

In many OC clay landslides, the sequence of sliding, remolding, and reconsolidation leave the clay in the slip zone in a normally consolidated (NC) or nearly NC state, with one or more loss of structure, aging or cementation.

Based on field data from different clay sites in various geographical areas, correlations between OCR and Kd could be established, as shown in **Figure 13** [2, 13, 15–18].

In genuinely NC clay (no structure, aging or cementation), the horizontal stress index Kd from the DMT is approximately equal to 2, while Kd values in OC clays are considerably higher (for the Boom Clay about 8).

For this reason, it is known that, if an OC clay slope contains clay layers with Kd ≈ 2, these layers are highly likely to be part of a slip surface (active or quiescent).

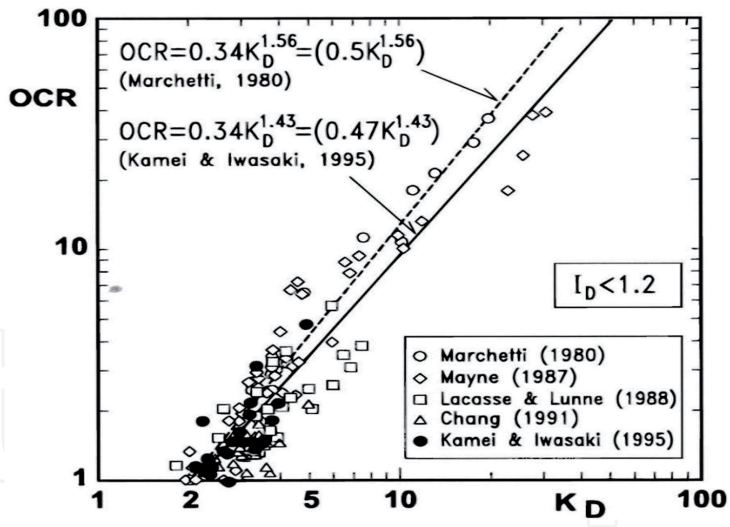


Figure 13.
Correlation Kd-OCR for cohesive soils [2, 13, 15–18] (as adapted from Peiffer (2016) [5]).

The DMT-Kd-method consists of identifying zones of NC-clay in a slope, using Kd ≈ 2 as the identifier of the NC zones.

7. Evaluation of the test results

The detailed evaluation of the test results of profiles with depth is discussed in [14, 19].

In **Figure 14**, the results are presented for the tests before (January 2014 – DMT 1), immediately after (July 2014 – DMT E) and about 2 years after the sliding (March 2016 – DMT 1′). In the test diagrams, the presence of the upper quaternary sandy silt layer can clearly be distinguished. Between January and July 2014, Kd decreased from about 3 to 5 t0, about 1.2 to 1.8 between 3.5 m and 4.5 m of depth. One can conclude that at this depth, the soil modified from an overconsolidated soil to an unstable soil (horizontal stress state lower than that for a normally consolidated soil). Above and below this strongly altered zone, the Kd-value decreased also, but less pronounced. Also, the stiffness, in particular the constrained modulus M, reduced sharply. The CPT diagram also shows a reduction of the cone resistance at a depth of about 3 to 4 m, but not that pronounced as in the results of the DMT, especially not at the top of the overconsolidated clay.

Out of the results of the DMT-tests, one can observe an unstable layer with a thickness of about 1.5 m where the soil is unstable where the slip was developed, but above and below this zone, the stress state is also affected by the sliding. Although cone resistance is decreased by a factor about 2, and a significant decrease of qc between 2 and 4 m, it can be seen in this layer that the qc-profile does not give a clear reflection of the sliding. It is also important to note that the Kd-values below the sliding zone decreased to a depth of about 5 m below the sliding surface.

It is worth noting how and to what extent the stresses were changed after the sliding. In **Figure 14c**, the results are presented for the measurements carried out in March 2016, after remedial works had been carried out in October–November 2014. The qc-value did not change significantly. The Kd-diagram shows clearly that the Kd-values exceeded again 1.8 (the lower limit for normally consolidated soils). For the clayey layer, the reading actually reflects a normally consolidated soil. For the intermediate sandy silt layer, one can see a slightly overconsolidated soil condition.

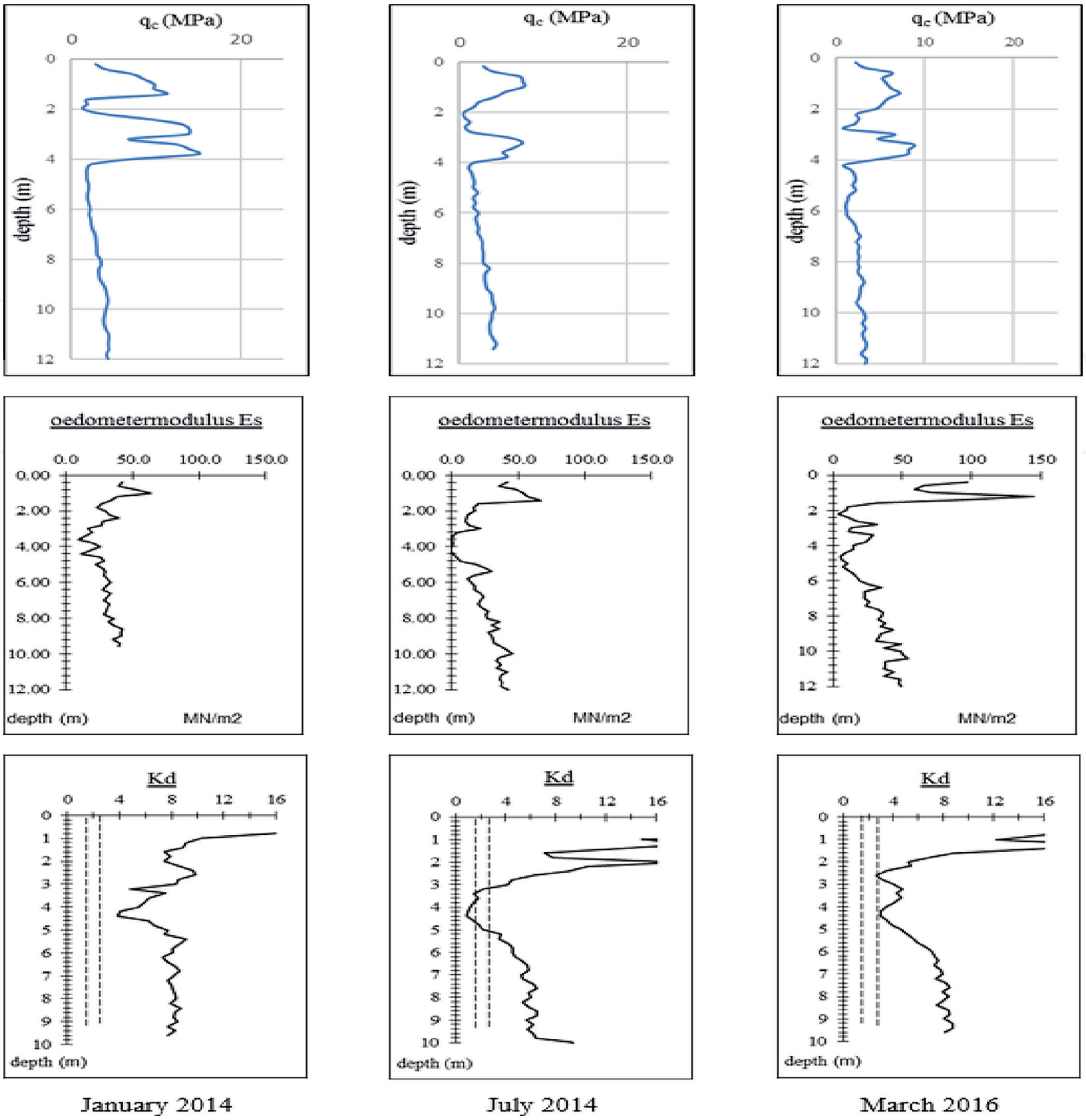


Figure 14.
Comparison CPT-DMT results – (a)January 2014 (DMT 1); (b) July 2014 (DMT E) and (c)march 2016 (DMT 1') (as adapted from Peiffer (2016) [5]).

8. Stability analysis

Based on the field measurements, the origin and the propagation of the instable zone where the sliding surface was initiated and developed could be clearly investigated.

A numerical analysis was performed, considering the geometry of section AA' (**Figure 10**) and the soil characteristics as they were measured in situ (before the sliding) and in laboratory tests. In this analysis, variable piezometric boundary conditions were considered to evaluate the impact on stability. Further investigations and additional calculations showed the importance of the groundwater pressures on the horizontal equilibrium.

Additional CPT-tests and laboratory tests were executed in order to make a detailed stability analysis. Based on an analytic and numerical analysis (PLAXIS-calculation), it was possible to prove the cause of the instability, and to design an appropriate improvement of horizontal stability.

Apparently, during the period of exploitation, the environment around the pit gradually became a central dewatering point. The flow of water towards the pit increased, and also the flow gradient increased.

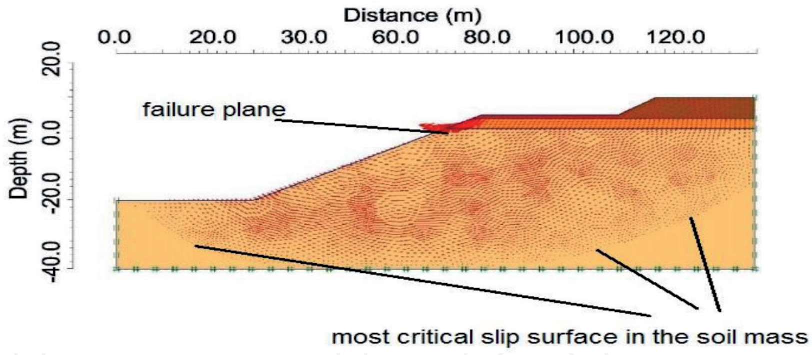


Figure 15.
PLAXIS-analysis and measurement of the slope after sliding (as adapted from Peiffer (2016) [5]).

Figure 15 [19] presents the calculated critical horizontal disequilibrium taking into account the presence of the groundwater and the piezometric site conditions. The red dots are points where the soil becomes plastic. The higher the density of the red dots, the more plastic the soil will behave, finally resulting in failure when there is no equilibrium of the soil stresses anymore. At the failure (the dark red line in **Figure 15**) zone, one can clearly detect the position of the sliding plane.

Out of these theoretical results, it became clear that the installation of drains would play an important role in rehabilitation.

9. Remediation

The slope was stabilized using three techniques:

- by installing deep drains in order to reduce the water stresses
- by smoothening the slope
- by the construction of a piled barrier

The PLAXIS-analysis resulted in a design where a deep drain has to be installed at a depth of 5 m., as shown in **Figures 16** and **17**. This drain was installed at the beginning of November 2015. The red line in **Figure 15** is the position of the drain in the plan view.

After the installation of this drain, there was an immediate effect on the depth of the groundwater. The original depth of 1.5 m increased to a depth of about 4.5 m. The safety factor of the slope stability increased from 0.9 to 1.31.

The original angle of the slope (before sliding) was 27° After the sliding, the clay was remolded.

Before the rehabilitation works of the slope, an angle of the slope of 18° was realized. These works were realized in order to reduce the risk of additional slidings. The result of this work is presented in **Figure 18.**

Although the slope angle was reduced, the soil mass remained moving to the center of the pit. It was not possible to remove the soil because of the bad soil conditions.

In order to stop the movement of the sliding mass, a piled barrier (6 rows of piles) was installed (**Figure 19**).

Between the piled barrier and the original slope, vertical drains were installed for the evacuation of water during consolidation.

Figure 16.
Installation of the deep drainage.

Figure 17.
Position of the drain in plan view (red line).

Figure 18.
Smoothening of the slope.

Since the beginning of 2019, the rehabilitation of the slope has been started. These works are executed in successive phases in order to guarantee the stability of the site during the works. Each phase has a height of 3 to 4 m, as can been seen on **Figure 20**.

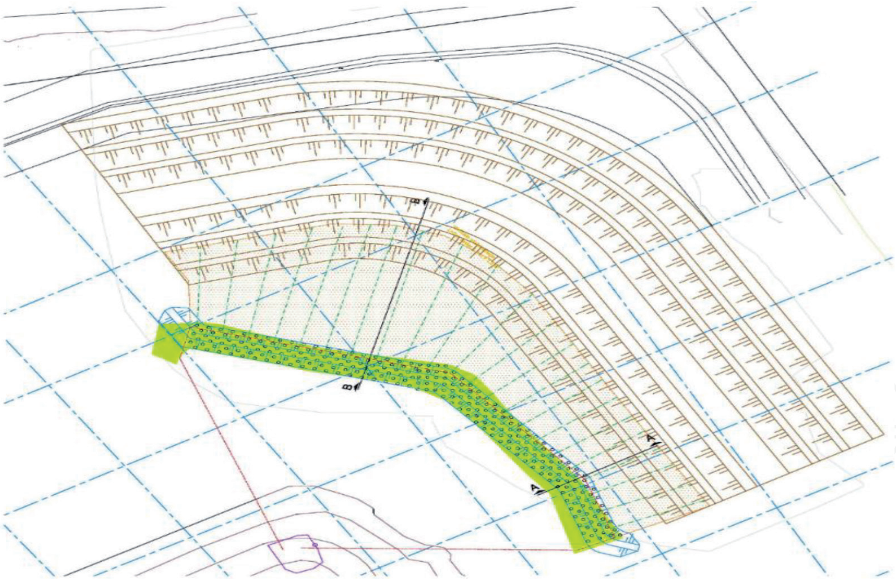


Figure 19.
Piled barrier.

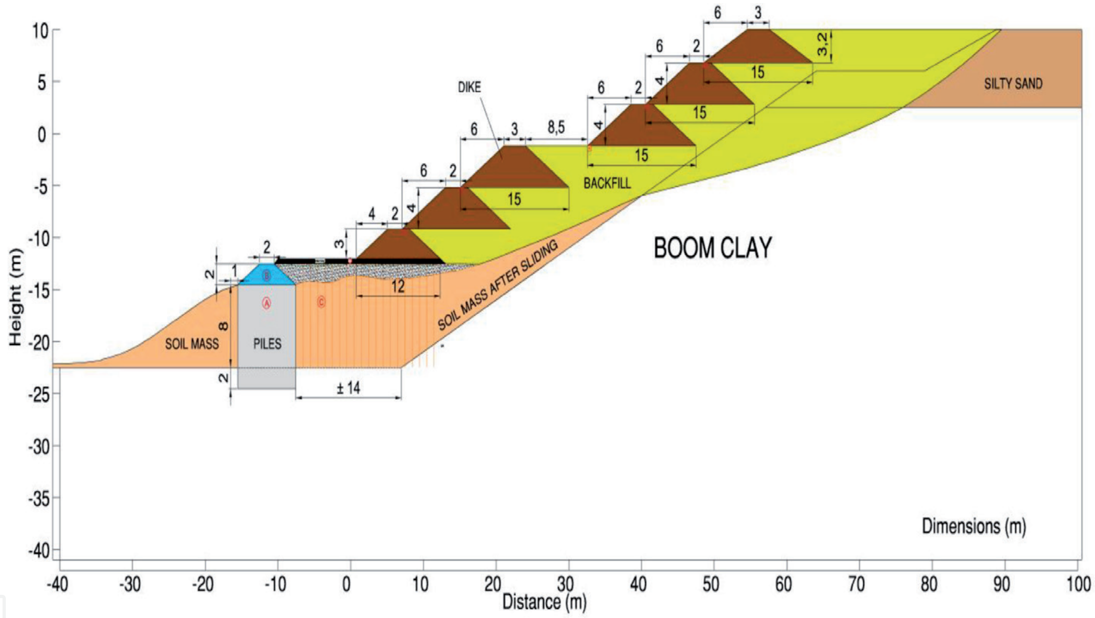


Figure 20.
New profile of the slope.

For each phase first a dike is constructed with well bearing sand. Between the dike and the actual slope, a backfill is made with sandy soil.

Today, about 50% of the total work has been completed.

10. Conclusions

In this chapter, DMT is applied as a tool for the monitoring of the stress state in a slope before and after the sliding.

From an analysis of the sliding observed in the overconsolidated Boom Clay, it can be concluded that it is possible to measure the effect of sliding on soil conditions and to evaluate the stability of the soil. In addition, it is possible to detect the position of a slip surface and the zone, which is influenced by the sliding.

For both applications, the results of a CPT-test in this type of soil are, as could be expected, less sensitive to changes in a stress state.

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