Modeling landslide triggering in layered soils

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ABSTRACT: The Flemish Government (represented by AMINAL; Administration of Environment, Nature, Land and Water management) commissioned a research project to study the triggering of landslides in the Flemish Ardennes. A few representative sites subject to landslides were studied from a geotechnical point of view. Several sites were selected for geotechnical calculations in order to predict the conditions necessary to trigger a landslide and to verify the predictions with the observations on site. Two of those sites are discussed in this paper. A hypothetical collapse mechanism, possibly responsible for many landslides in the Flemish Ardennes, was numerically verified. The presence of a sand layer (high water permeability) between two clay layers (low water permeability) causes the building up of pore water overpressures, decreasing the effective stresses, eventually resulting in the uplift and/or collapse of the slope. The understanding of the mechanisms responsible for the studied landslides resulted in specific recommendations to prevent future landslides on these sites, but also on similar sites in the region.

1 INTRODUCTION

1.1 *Landslides in layered soils: causes and parameters*

Insufficient safety of the global stability of a slope can cause a landslide that occurs along a slip surface (circular or not), or a slow displacement (creep) of the entire slope. Landslides can also be triggered by the seepage of ground water out of the slope, causing local erosion and caving.

An insufficient safety of the global stability of the slope may be induced by several parameters. The geometry of the slope is obviously a very important factor. A steeper slope will collapse more rapidly.

The shear strength characteristics of the involved soil layers play an even more important role. The friction angle f [°] and cohesion c [MPa] (cfr. The Mohr-Coulomb soil model) must be determined for every layer when evaluating the global stability of the slope. When checking the safety of a slope which already experienced a landslide, it is best to evaluate the stability also with the residual shear resistance parameters (De Beer, (1979)).

Another important parameter is the piezometric height of the water in the different soil layers, or the ground water level. The shear resistance is determined (in drained circumstances) by the effective stresses in the soil, which are directly correlated with the piezometric heights in the soil layers. When dealing with permeable layers with significant thickness, the groundwater level or the piezometric height is easily measured using water level tubes. The determination of correct piezometric heights becomes much more difficult when thin permeable layers occur within little permeable or impermeable layers. The variation of piezometrc height within an enclosed layer can be very high (i.e. during periods with heavy rainfall). In Figure 1 a hypothetical collapse mechanism caused by high piezometric levels is illustrated.

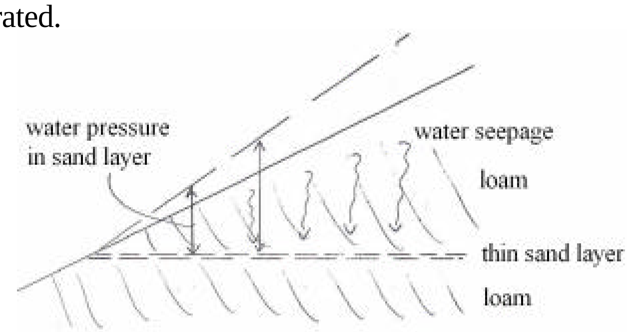

Figure 1. Typical failure mechanism. A permeable (sand) layer is enclosed by little permeable layers (e.g. loam or clay) resulting in high water overpressures and thus low effective stresses in the sand layer, resulting in collapse during heavy rainfall.

Seepage of rainwater through the top layer into the sand layer will cause high water overpressures in this permeable layer when the entrance water amount (e.g. true cracks) is bigger than the exit water amount (which is very limited because of the inclusion between two impermeable layers). This high water pressures result in low effective stresses (= total stress minus water pressure) and thus lowers the shear resistance, causing the landslide to occur.

The global stability can also be affected when the load on the slope is altered. This can have a stabilizing or a destabilizing effect. When adding load (buildings, soil deposits, …) at the top of the slope, this will have a destabilizing effect. Adding load at the toe of the slope will have a stabilizing effect.

1.2 *Research aim*

The Flemish Government (represented by AMINAL; Administration of Environment, Nature, Land and Water management) commissioned a research project to study the triggering of landslides in the Flemish Ardennes. This particular rolling region in Belgium has a history of landslides and general stability problems of slopes. The project marked out a part of the Flemish Ardennes, then produced an inventory, a classification, a statistical and spatial analysis and a methodology for the production of hazard maps (Van Den Eeckhaut et al. (2005)). Complementary, a few representative sites subject to landslides were studied from a geotechnical point of view. This geotechnical study is the topic of this paper.

Three sites were selected for geotechnical calculations in order to predict the conditions necessary to trigger a landslide and to verify the predictions with the observations on site. Two of those sites are discussed in this paper. The study of each site started with the execution of soil investigation tests (Cone Penetration Tests, borings and triaxial shear tests on representative non disturbed soil samples) to draw a geotechnical profile of the slope, determining the stratification of layers and their geotechnical parameters. Combined with topographical data of the collapsed slope and the assumed profile before the landslide occurred, numerical models of these sites were implemented using two software programmes based on numerical (finite elements method using PLAXIS, www.plaxis.nl) and analytical (SLOPE, www.geo-slope.com) mathematical algorithms.

With these models it is possible to calculate an overall factor of safety to evaluate the stability of the slopes. An hypothetical collapse mechanism as defined in paragraph 1.1 (See Figure 1), possibly responsible for many landslides in the Flemish Ardennes, will be numerically verified. The presence of a sand layer (high water permeability) between two clay layers (low water permeability) causes the building up of pore water overpressures, decreasing the effective stresses, eventually resulting in the collapse of the slope.

Numerical simulations also proved to be very useful to determine the negative influence on the overall slope stability of adding additional loads to the upper slope surface (buildings), the slightly positive influence of vegetation and the negative influence of excavations (swimming pools, ponds) at the bottom of the slope.

The understanding of the mechanisms responsible for the studied landslides results in specific recommendations to prevent future landslides on these sites, but also on similar sites in the region. For a more detailed publication in Dutch of this work see (Keersmaekers et al. (2005)).

2 MODELLING

2.1 *Analytical method: SLOPE (Bishop)*

The development of calculation methods to check the global stability of slopes with arbitrary shapes and materials with cohesion and friction goes back for decades. Firstly only circular slide surfaces were considered (Figure 2). Later on calculation methods were developed for irregular slide surfaces. Recently, the use of finite element methods is becoming more and more common.

The most used analytical method to evaluate circular slip surfaces is the Bishop method. The soil above the slip surface is divided in vertical strips, bounded by vertical surfaces. A shear stress t is mobilized along the slip surface, which is supposed to be a factor SF (safety factor) smaller than the maximum possible shear resistance. A 2D-geometry is assumed.

Verifying the stability of the slope consist of the expression of the momentum equilibrium to a center point of the considered circular slip surface. The calculation of SF is done iteratively (starting with SF = 1) and must be done for many center points (i.e. many possible circular slip surfaces) to find the lowest value of the SF.

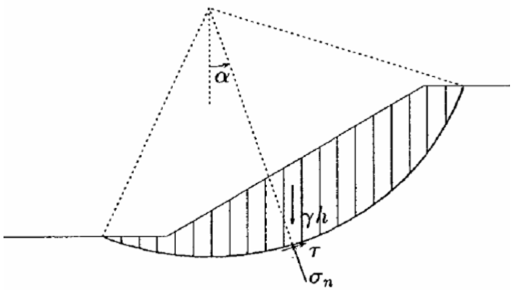

Figure 2. Circular slide surface for analytical calculations conform Bishop. This analytical method divides the slide volume in vertical strips.

The software package used in this research project to calculate the Bishop method is Slope.

2.2 *Finite element method: PLAXIS*

Most calculations in this research project were made with Plaxis, a finite element based software package, especially developed for geotechnical applications. The volume is divided into small elements which are numerically coupled to each other. The stress equilibrium and the soil deformation are described by a system of regular and partial differential equations which are solved numerically. In this way the soil stresses and deformations can be calculated.

The advantage of using finite elements is that the real behavior of the soil is better simulated and that the real stresses occurring in the soil are taken into account. There is for example a clear division between vertical and horizontal stresses which is not the case in analytical methods.

To model the behavior of the soil a so called Mohr-Coulomb model has been used. This model assumes a complete elastic behavior until the shear stress in the element equals the shear resistance. After this point the soil behaves completely plastic.

For the determination of the safety factor (SF as defined above), the so called phi,c (f,c)-reduction method is used. The shear resistance parameters f' and c' are reduced in the same way until collapse of the slope. This collapse is verified by the displacement of one or more well chosen physical points.

2.3 *Safety factor*

When slopes are designed with the analytical Bishop method, a safety factor SF of minimum 1.3 is normally required. The calculation is then based on the momentum equilibrium of a circular slip surface.

The calculation of the safety factor using finite element methods is based on the phi,c (f,c)-reduction method. Many calculations in the past learned that the overall safety factor, obtained from analytical methods like Bishop do not differ a lot from the safety factor obtained using the phi,c (f,c)- reduction method (finite elements).

When evaluating the Slope and Plaxis calculations in this project, an overall SF of 1.3 is required to have a save slope. SF-values lower than one indicate slopes that will collapse under the given parameters. SF-values between 1 and 1.3 correspond to slopes with insufficient safety, but will not necessarily collapse.

3 SITE1: SOCCER FIELD "KORTE KEER" AT MAARKEDAL

3.1 *Situation*

The first site is the football field Korte Keer at Maarkedal (Nukerke, Belgium). The site is located above a very large, deep and old landslide. The football field itself was built on a slope by rectifying the terrain by filling the site with a sandy embankment. From a geotechnical point of view, this added material has similar properties as the top layer of the original slope (see hereafter).

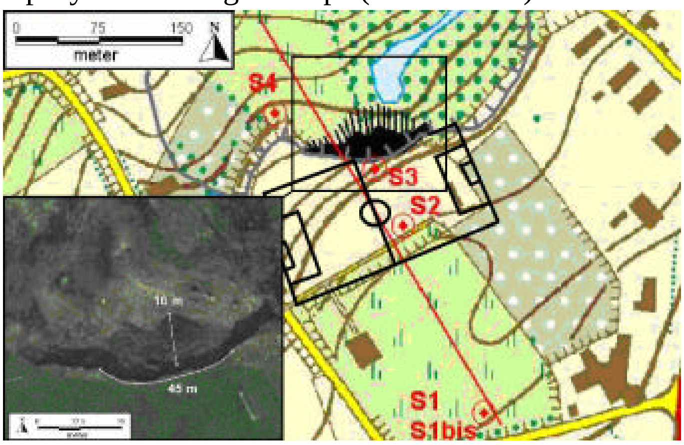

Figure 3. Situation of the site Korte Keer. The line defines the topographic profile used to model the geometry of the collapsed and original slope.

Figure 3 shows the situation of the site Korte Keer. The line defines the topographic profile used to model the geometry of the collapsed and original slope (see also Figure 4). The original profile of the football field had a steep inclination of 1/1.

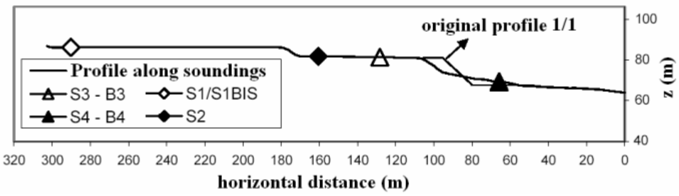

Figure 4. Profile before and after the event of the landslide. Notice the location of the soundings.

3.2 *Characteristics of the soil layers*

There were five CPT-tests made on site with a 200 kN apparatus, which probed to depths until 25 meters. Also two borings and triaxial tests on undisturbed soil samples were performed. The information obtained from these tests are than used to determine the stratification and characteristics of the different soil layers. The locations of these CPT-tests are given in Figures 3 and 4. Also a piezometric pipe was installed to monitor the variation of the water table over one year.

From the on-site investigations it was concluded that the site consists of two major layers. The top layer is a well permeable sand layer which is partly constructed from sand deposits to rectify the slope and the underlying original sandy material (quaternary origin). Both layers have very similar characteristics and are therefore modeled as one top layer. Under this top layer, a clay layer (from tertiar origin, i.e. "Ieperiaan") is found with an almost horizontal orientation. Table 1 summarizes the main characteristics of both layers.

A Mohr-Coulomb soil model is used to define the two layers in Plaxis. Two models are constructed. The first one with the original slope geometry and one with the present profile. This second profile, representative for the situation after the landslide, aims to determine the present safety of the site (see Figure 5).

Table 1. Ground characteristics of the different soil layers.

Parameter	Top sand layer		Bottom clay layer	
	value	unit	value	unit
γ_{dry}	18	kN/m³	19	kN/m³
γ_{wet}	19	kN/m³	19	kN/m³
E-mod	2E+4	kN/m²	1E+4	kN/m²
Poisson ν	0.3	[-]	0.35	[-]
Cohesion c	0.0	kN/m²	25	kN/m²
φ	27.5	°	23	°

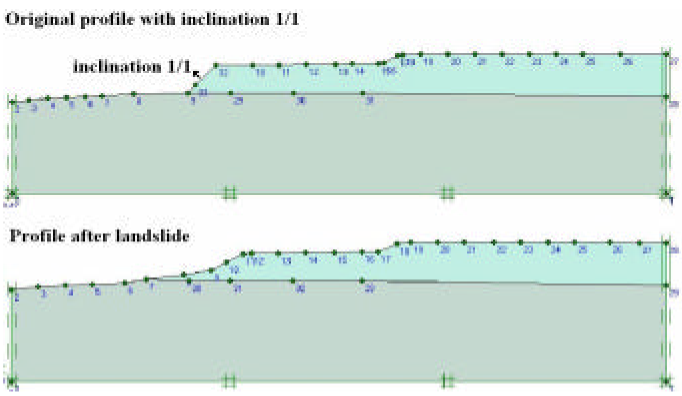

Figure 5. Plaxis geometric models for the original profile and after the landslide occurred.

3.3 *Results of the calculations*

3.3.1 *Original profile with inclination 1/1*

Figure 6 gives the slip surface of the original slope. The safety factor SF is calculated to be 0.561, meaning that the collapse of the site was inevitable. The inclination 1/1 is too steep. The calculation is made with a deep water table, which proves the landslide was not due to water overpressures.

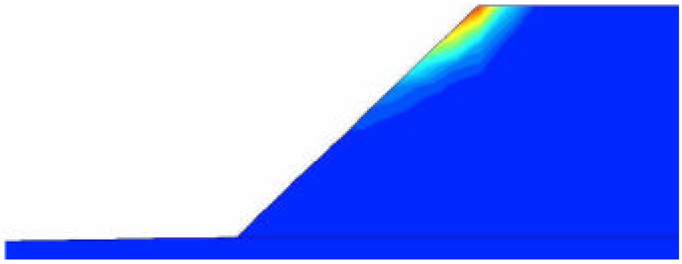

Figure 6. Slip surface of the original slope. The safety factor SF is 0.561, meaning that the collapse of the site was inevitable.

3.3.2 *Profile after landslide*

The measurements of the water level pipe showed a maximum piezometric height of +73.37 TAW (TAW is the reference level for Belgium) between September and October 2004.

The SF-value obtained was 1.218, meaning the slope will not collapse, but has an insufficient SF according to literature (minimum SF = 1.3). When the piezometric height of the top layer increases to +76 TAW (i.e. 2.63 m higher than in the above situation), the SF-value drops to 0.997, meaning the slope has just reached equilibrium, the landslide can occur at any moment.

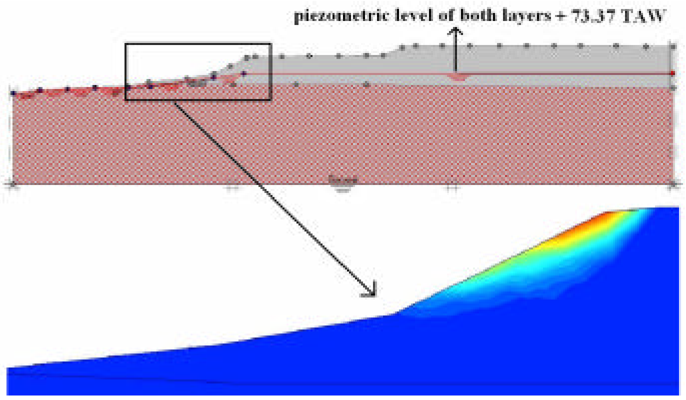

Figure 7. Top: Piezometric heights at +73.37 TAW of both layers. Notice the drop in water level towards the toe of the slope. Bottom: Preferential slip surface, the safety factor SF is 1.218.

The influence of the growth of plants on the site is incorporated by giving the first meter of the top soil layer a cohesion of 5 kN/m². The SF increases from 0.997 to 1.074, which proves the positive effect of vegetation.

Comparison with a Slope calculation (analytical model) resulted in a SF-value of 1.08 (compared to 0.997), which proves that the overall safety factor, obtained from analytical methods like the Bishop method does not differ a lot from de safety factor obtained using the phi,c (f,c)-reduction method (based on finite elements modeling). Also the slip surface showed a very similar sliding surface, showing a rather shallow collapse of the slope.

Finally a calculation was made to rebuild the football field without the danger of triggering a landslide. The piezometric height of the top layer was chosen one meter below the surface (this can be done by placing a drainage system under the new football terrain and a drainage at the toe of the slope). A minimum inclination of the slope of 12/4 resulted in a SF of 1.333, reaching the minimum required SF-value of 1.3.

4 SITE2: SCHERPENBERG RONSE

4.1 *Situation*

The Scherpenberg is a complex landslide on a slope without buildings. Therefore it is easily accessible for cone penetration test and chosen for this project.

Figure 8 shows the situation of the site Scherpenberg. The dotted line defines the topographic profile used to model the geometry of the original slope. The continuous line defines the topographic profile used to model the geometry of the collapsed slope. Notice the location of the CPT-tests (see Figure 9).

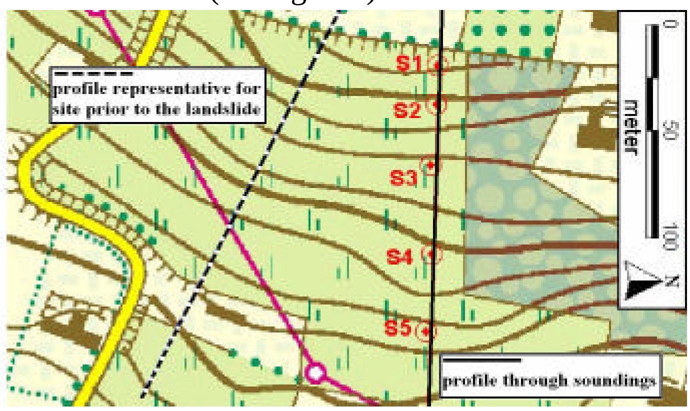

Figure 8. Situation of the site Scherpenberg. The dotted line is representative for the original slope (no landslide has occurred there). The continuous line defines the topographic profile used to model the geometry of the collapsed slope. Notice the location of the soundings.

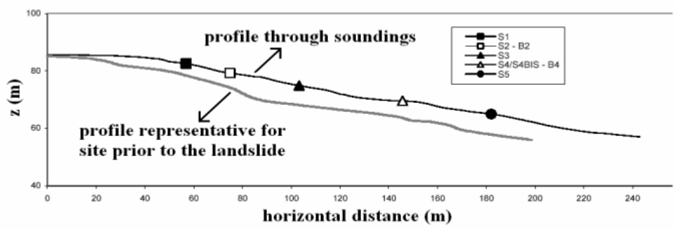

Figure 9. Profile before and after the advent of the landslide. Notice the location of the soundings.

4.2 *Characteristics of the soil layers*

There were five CPT-tests made on site with a 200 kN apparatus, which probed to depths until 25 meters. Also two borings were made and triaxial shear tests on undisturbed soil samples were performed. The information obtained from these tests are then used to determine the stratification and characteristics of the different soil layers. The locations of these CPT-tests are given in Figures 8 and 9.

From the on-site investigations it was concluded that the site consists of three major layers. The middle layer is a well permeable sand layer which is enclosed between two low permeable clay layers. Table 2 summarizes the main characteristics of the three layers.

Table 2. Ground characteristics of the different soil layers.

Parameter	Top clay	Middle sand	Bottom clay	
	value	value	value	unit
γ_{dry}	18	17	19	kN/m³
γ_{wet}	18	20	19	kN/m³
E-mod	1E+4	1.3E+4	1E+4	kN/m²
Poisson ν	0.35	0.30	0.35	[-]
Cohesion c	10	1	20	kN/m²
φ	25	30	25	°

A Mohr-Coulomb soil model is used to define the three layers in Plaxis. Two models are constructed. The first one with the original slope geometry and the second one with the present profile. This second profile, representative for the situation after the landslide, aims to determine the present safety of the site (Figure 10).

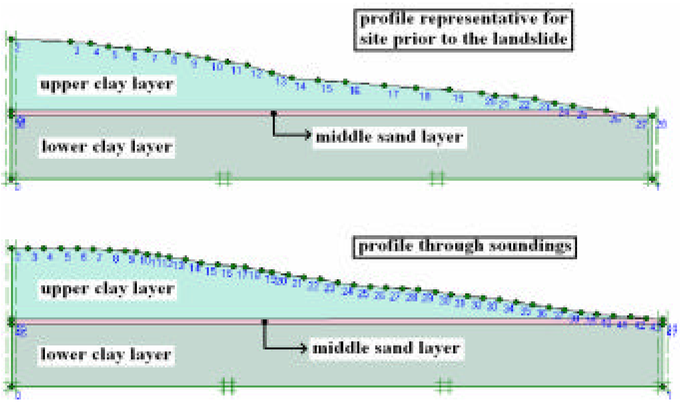

Figure 10. Plaxis geometric models for the presumed original profile and after the landslide occurred.

4.3 *Results of the calculations*

4.3.1 *Presumed original profile*

Figure 11 gives the slip surface of the presumed original slope. The piezometric height of +65 TAW (for the middle and bottom layers, for the top clay layer the piezometric height is assumed one meter under the ground surface) is the maximum valuemeasured on site between November 2004 and January 2005. The safety factor SF is calculated to be 1.526, meaning that the site is safe under these conditions. Notice the preferential slip surface at the top of the slope.

Raising the piezometric level of the middle and bottom layer to +68 TAW results in a SF of 1.026 (meaning the slope has just reached equilibrium, the landslide can occur at any moment), and a different preferential slip surface at the bottom of the slope is obtained (Figure 12). This means that when a landslide will occur, the slope will collapse according to this second slip surface.

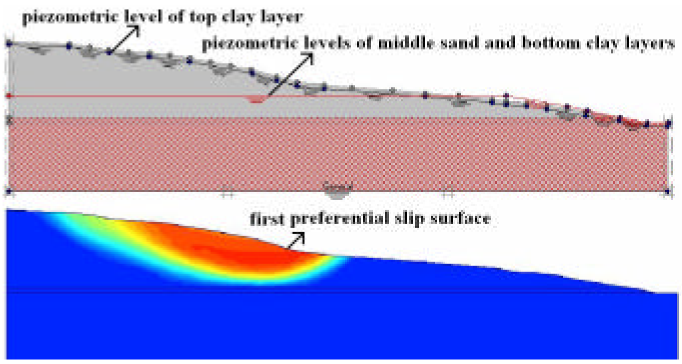

Figure 11. Top: Piezometric heights of the middle and bottom layers at +65 TAW. Bottom: Preferential slip surface at the top of the slope, SF = 1.526.

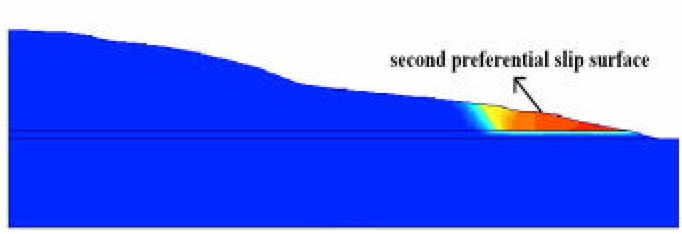

Figure 12. Piezometric heights of the middle and bottom layers at +68 TAW. Preferential slip surface, SF = 1.026.

When adding a load of 40 kN/m² at the top of the slope (representative for a building on a fill layer, Figure 13), for the same conditions of Figure 11 (+65 TAW, SF = 1.526), the SF drops to 1.264. In case the load is 60 kN/m², the SF drops to 1.155, proving the negative effect of load at the top of the slope.

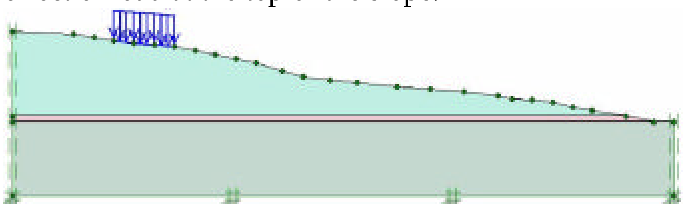

Figure 13. Plaxis geometric model including the load representative for a building at the top of the slope.

4.3.2 *Profile after landslide*

Figure 14 gives the slip surface of the slope after occurrence of the landslide. The piezometric height for the middle and bottom layers is set at +68 TAW, for the top clay layer the piezometric height is set one meter under the ground surface.

The safety factor SF is calculated to be 1.865, meaning that the site is safe under these conditions (which are unlikely to occur in reality). For lower water levels the slope gave even higher SF's and preferential slip surfaces at the top of the slope. Notice the shift from a high to a low preferential slip surface in Figure 14.

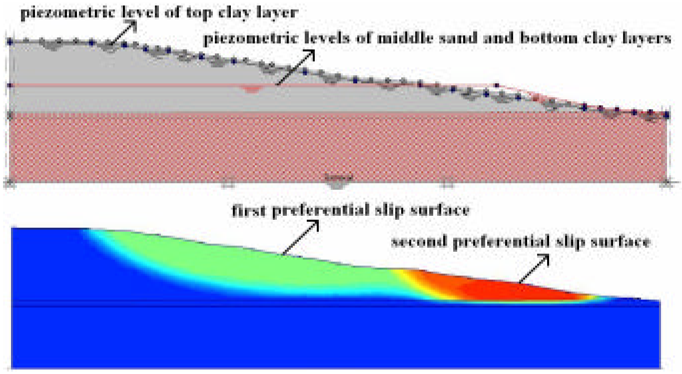

Figure 14. Top: Piezometric heights of the middle and bottom layers at +68 TAW. Bottom: The preferential slip surface shifts from the top to the bottom of the slope, SF = 1.865

When the piezometric height for the middle and bottom layers is set at +71.5 TAW, SF drops to 1,212, resulting in a slip surface at the bottom of the slope (cfr. Figure 15). For a TAW level +72.5, SF = 0.952, meaning the slope has just reached equilibrium, the landslide can occur at any moment.

Figure 15 shows the negative influence of a swimming pool at the bottom of the slope. The empty swimming pool is lifted, SF drops from 1.212 to 1.025, respectively without and with swimming pool.

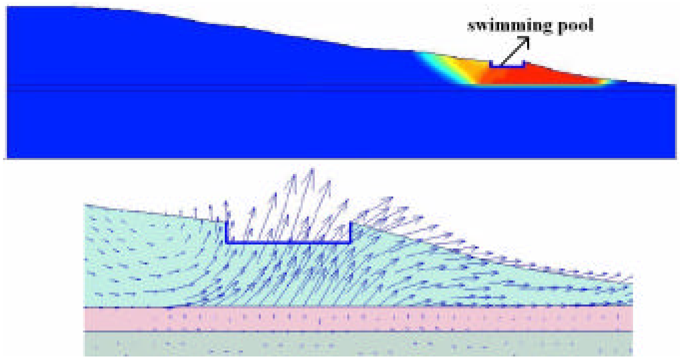

Figure 15. Top: Influence of a swimming pool at the bottom of the slope. Piezometric heights of the middle and bottom layers at +71,5 TAW. Bottom: Incremental displacement vectors: The empty swimming pool is lifted, SF drops from 1.212 to 1.025, respectively without and with swimming pool.

Finally the influence of the decrease of the strength characteristics due to an already occurred landslide is taken in to account. Figure 16 shows the Plaxis model including the presumed disturbed zone (based on the sounding profiles) due to the occurred landslide. In (De Beer, (1979)), values for the residual shear strength characteristics for quaternary clay are given: $f'_r = 12.5°$ and $c'_r = 5$ kN/m². The piezometric heights of the middle and bottom layers are set at +71,5 TAW. The SF is 1.037 (compared to 1.212 above) proving the negative influence on the present stability of the already disturbed zones in the top layer. This result was verified with Slope, using the Bishop method (Figure 17), giving a value $SF_{Slope} = 1.094$ (almost equal to 1.037 above).

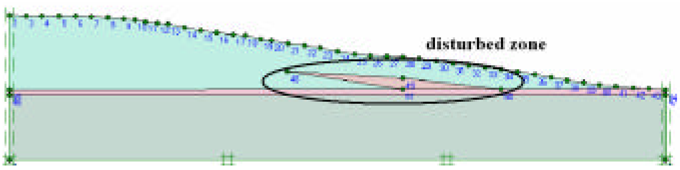

Figure 16. Plaxis geometric model including the presumed disturbed zone (based on the sounding profiles) of the former landslide.

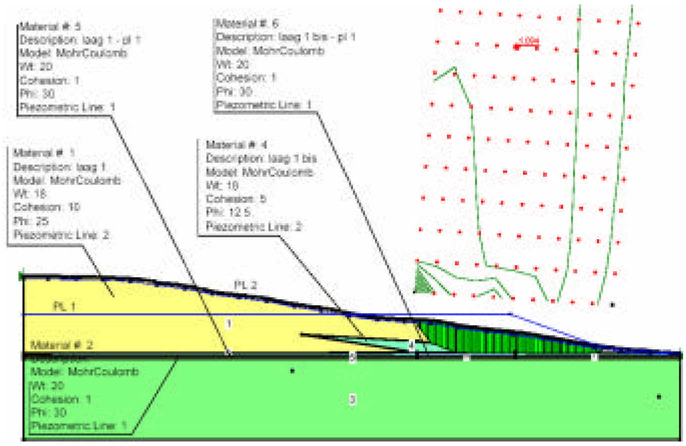

Figure 17. Slope calculation (Bishop method) of a geometric model including the presumed disturbed zone of the former landslide, $SF_{Slope} = 1.094$.

5 CONCLUSIONS

This study demonstrates that the presence of a sand layer between two clay layers may cause the building up of pore water overpressures, decreasing the effective stresses, eventually resulting in the collapse of a slope (cfr. Figure 1).

Numerical simulations proved very useful to determine the negative influence on the overall slope stability of adding additional loads to the upper slope surface (buildings), the positive influence of vegetation and the negative influence of excavations (swimming pools, ponds) at the bottom of the slope.

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