# Landslide stabilization by piles: A case history

## Stabilisation des glissements de terrain par des pieux : un cas d'étude

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ABSTRACT: During the foundation excavation of an industrial plant in Eskişehir, Turkey, a landslide occurred in the neighbouring slope, and consequently, the excavation had to be stopped. The instability occurred in a natural slope which had an average slope angle of 9-13 degrees. The aerial extent of the slide mass was 400 m by 115 m. The material in the slope was 7 to 20 meter-thick colluvium (slide debris), which constitutes a historic landslide mass, underlain by a gravelly sandy silty clay matrix containing limestone blocks, weathered tuff and tuff. Soil profile and location of the slip surface were determined through a site investigation program including nine boreholes and inclinometers. A back analysis was carried out and shear strength parameters of the slope forming materials were determined. Slope stability analyses were carried out, stabilization alternatives were evaluated and slope stabilization by piled retaining walls was considered as the most feasible alternative. The analyses and design procedure for the piles will be presented together with a summary of existing analyses and design methods for landslide stabilizing piles.

RÉSUMÉ : Pendant les excavation des fondations d'un site industriel à Eskisehir, en Turquie, un glissement de terrain a eu lieu dans un talus voisin, et par conséquent, les travaux ont été arrêtés. L'instabilité s'est produite dans un terrain naturel présentant une pente moyenne de 9 – 13 degrés. Les dimensions en plan du glissement étaient de 400 m par 115 m. Le matériau du glissement était composé de colluvions, sur 7 à 20 m d'épaisseur, qui constituaient une masse de glissement postérieur. Les colluvions reposaient sur une argile graveleuse et sablo-limoneuse contenant des blocs de calcaire, du tuf sain et du tuf altéré. La coupe géotechnique du site et la localisation de la surface de glissement ont été mises en évidence par une campagne d'investigations incluant neuf forages et inclinomètres. Les paramètres de résistance au cisaillement des terrains en place ont été déterminés par une rétro-analyse. Suite aux études de stabilité des pentes et à l'examen des systèmes de renforcement alternatifs, la solution de stabilisation par une paroi en pieux s'est avérée comme la plus judicieuse. Les analyses et la procédure de dimensionnement pour ces pieux vont être présentées en même temps qu'un résumé des analyses et des méthodes de calcul existantes sur la stabilisation des glissements par pieux.

KEYWORDS: landslide, piles, inclinometer.

## 1 INTRODUCTION AND SOILS AT THE SITE

Instability occurred in a natural slope which had an average slope angle of 9-13 degrees, during excavation works near the toe. The aerial extent of the slide mass was 400 m by 115 m. The construction site was located at the toe of this sliding mass (Figure 1). The slope movement had a NW-SE direction. The material in the slope was 7 to 20 meter-thick colluvium (slide debris), which constitutes a historic landslide mass, underlain by gravelly sandy silty clay layers with limestone blocks, weathered tuff and unweathered tuff at different borehole locations. The colluvium was also composed of pieces of limestone cobbles and blocks in a clayey silty matrix.

It was observed that the slide debris lies on tuff. The pliosen age tuffs are approximately 100 m thick and lies on limestone. In the borehole descriptions the tuff layer at the upper levels are weathered to highly weathered condition and can be considered as a transition between rock and soil and identified as a layer of stiff clay. Samples from this weathered tuff had natural water content 19-43% (high natural water content such as 43% was obtained near the shear surface at about 20 m depth at borehole SK9). Since this material can be considered clay-like, Atterberg limits are determined LL=45-49%, PI=11-28% (for the material near the shear surface LL=49-66% and PI=27-36%), fines content=34-74% and clay-size fraction was 23-57%, and the material can be classified as CL, ML, GC and SC. The natural unit weight of samples were 20-21 kN/m$^3$. Unconfined compressive strength of this material was 30-240 kPa and UU triaxial tests gave $c_u$=13-44 kPa and $\phi_u$=11-22°. Since the residual shear strength will be of interest in this reactivated landslide case, direct shear tests were carried out on limited number of undisturbed samples taken from near the shear surface in order to find the residual friction angle. 46 mm diameter specimens were sheared undrained without submerging in water, then sheared continuously and slowly to reach residual condition. From the lab tests it was concluded that the 100 and 250 kPa undrained shear strength for tuffaceous clay and clayey tuff are reasonable values. Drained friction angle of slope debris/colluvium was 15-20 degrees, and tuffaceous clay taken from near the shear surface had 10.2 degrees residual friction angle.

### 1.1 *Soil Profile and The Mechanism of the Slope Movement*

The sliding mass has grown progressively backward into the slope as multistage rupture surfaces from the toe of the landslide developed. First, the tension crack (scarp) C1 has been observed while excavation works were going on. Then, within about 15 days, there were total of four major tension cracks (visible scarps) (C1 to C4 in Figures 1 and 2) and one meter horizontal and vertical movements have been observed at the ground surface. Fifty days after the start of the first scarp, a small road located at the slope was observed to move 15 m downslope and 4 m in vertical direction.

Nine boreholes are opened at a line of NW-SE direction which is also the direction of the movement and inclinometers are also placed in these boreholes. In all boreholes except SK7, SK8 (at the toe) and SK6; the top layer is slope debris and below that tuffs sometimes in clayey state and sometimes in harder state are encountered.

## 2 ANALYSES AND RESULTS

In this section the types of the analyses , the results of the back-analyses of the landslide movement and then the stabilization works will be presented.

### 2.1 *Back-Analyses and the Shear Strength Parameters*

In all boreholes inclinometer readings were taken, and the depth of the slip has been determined. As it is a fast movement, all inclinometer pipes were sheared by the landslide (except SK-3 and SK-8) shortly after they are placed. These depths were considered as the depth of the slip surface. The cross section of the landslide were prepared according to the borehole data and inclinometer readings and the scarps of the progressive slips. The positions of the slope debris and tuff layers and also the scarps of the progressive failure slides are shown in Figure 2. The shear surface passes mostly through the colluvium material in the upper half of the sliding mass, whereas it passes near the contact between the colluvium and tuff (more within the tuff) in the lower half of the sliding mass. There was no groundwater level, however in stability analysis to be on the safe side some water level is considered.

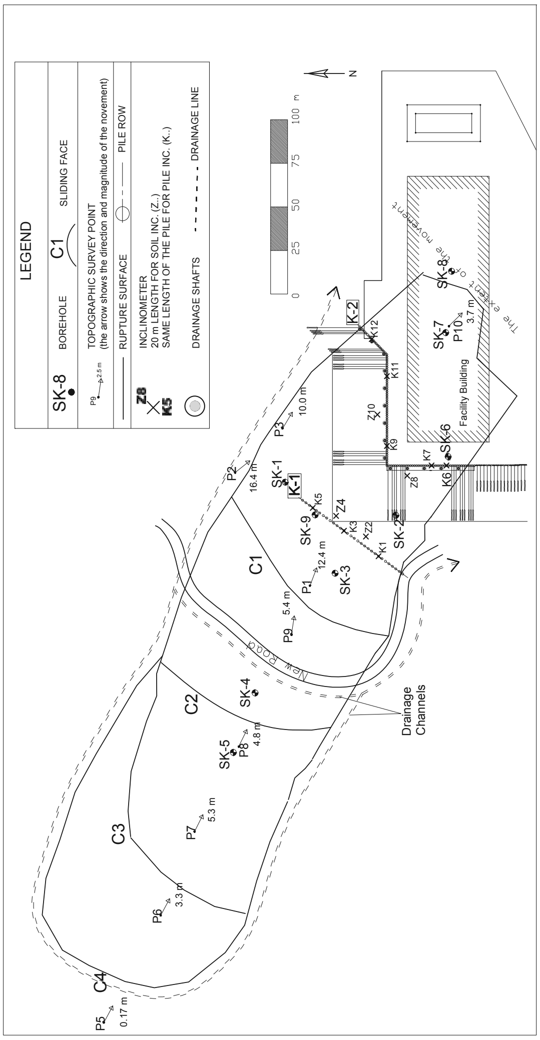

Figure 1. General layout plan

It was not easy to take samples from the sliding mass near the shear surface, therefore only a limited number of soil samples were tested in the laboratory. Determination of the soil parameters at the sliding surface of a landslide at the limit equilibrium state by the back analyses of the movement is a widely used method. In this method the c and $\phi$ parameter couples are determined which give a factor of safety value of one at the sliding surface. In the back analyses the cohesion and angle of friction parameter couples for the tuff and the slope debris layers are identified. In this case the number of the unknowns is four. They are decreased progressively using a methodology given below.

At the first stage, the back analyses of sliding surfaces of C3 and C4 were performed. By this way the parameters for the slope debris were determined as $c'$=5.6 kPa and $\phi'$=15.7°, $E'$=35 MPa are used in the analyses.

At the second stage, calculations for sliding surfaces of C1 and C2 were performed. The above parameters were used for slope debris, and the shear strength parameters for the tuff layers were determined. The parameters for the weathered tuff layer at the sliding level are assessed as $c'$=0 kPa and $\phi'$= 9.2°. But these parameters were not suitable to use for the whole tuff layers. Table 1 summarizes the parameters used in the analyses.

Table 1. Material properties used in analyses

| Material | Drained $c'$ (kPa) | Drained $\phi'$ (°) | Drained $E'$ (MPa) | Undrained $c_u$ (kPa) | Undrained $E_u$ (MPa) |
|---|---|---|---|---|---|
| Slope debris | 5.6 | 15.7 | 35 | 60-75 | 20 |
| Weathered tuff (first 3 m) | 0 | 9.2 | | 100 | 60 |
| Unweathered tuff | 20 | 25 | 60 | 250 | 130 |

Undrained triaxial compression and consolidated drained direct shear tests were performed on the debris and tuff soil samples from the boreholes which are located between the slope debris and tuff layers. The results of these tests show that the internal friction angle for the slope debris and for the clay soil originated form tuff are 15 to 20° and 10.2° respectively. This findings support the results of the back analyses.

### 2.2. *Slope Stabilization by Piles*

Piles used in slope stabilization are subject to lateral force caused by the movement of surrounding soil, and they are called “passive piles” (Viggiani 1981; Poulos 1995). One of the major issues in the design of these piles is the magnitude of the force on the pile. Since this is related to the soil movement and soil movement is influenced by the presence of piles, the interaction between the passive piles and the soil is quite complicated. There exists a number of empirical, analytical, and numerical methods available in the literature about the design of piles used in slope stabilization (Brinch Hansen 1961, De Beer 1977, Fukuoka 1977, Ito and Matsui 1977, Sommer 1977, Winter et al. 1983, Popescu 1991, Reese et al. 1992). Some of these methods are pressure or displacement-based and some of them are numerical methods, such as finite element and finite difference (Kourkoulis et al., 2012).

In the pressure or displacement based methods, the pile is modeled as a beam supported by springs at its sides. A single laterally loaded pile is considered, and the ultimate soil-pile resistance is correlated with the undrained shear strength for clays, and with the overburden stress and friction angle for sands. In these methods group effects are taken into account by using reduction factors. Although spring constants are dependent on pile deflection, or the movement of the soil, they are typically assumed constant. In the current study this approach is used.

The general design procedure consists of two main steps: Step 1: Provision of the required total lateral force needed to increase the factor of safety of the slope to the desired value (based on analysis of the unreinforced slope). Step 2: Calculation of loading on piles and pile lateral capacity.

In the last few years, 3-dimensional finite-element and finite-difference methods are becoming increasingly popular. Using these methods complex geometries and complicated phenomenon such as soil arching, pile group effects, nonlinearity of soil and pile could be modeled. Use of 3D numerical methods is still not very attractive to practitioner engineers because of the long computational time and learning effort. Kourkoulis et al. (2012) developed a hybrid solution to the problem, combining the benefits of accurate 3D finite element simulation with the simplicity of widely accepted analytical techniques.

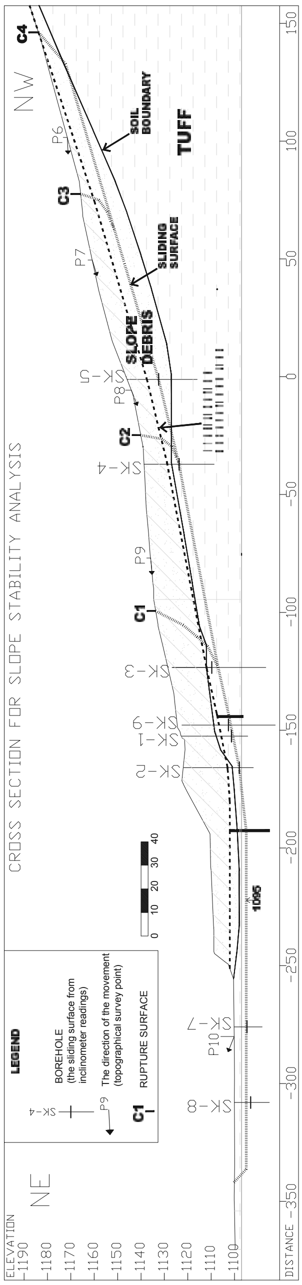


Figure 2. Cross section for the analyses

## 2.2 *Stabilization Works for Foundation Excavation and Prevention of the Slope Movements*

As a result of the preliminary works it is understood that the diameter of the stabilizing passive piles should be 1.2 m and more than single pile row would be needed. The depth of the sliding mass was 20 m at the centerline. It was not possible to stabilize this amount of soil mass by two rows of piles only. So stabilizing excavations were needed to decrease the amount of sliding mass (about half of the sliding depth was removed-see Figure 2), and then the following piled analyses were performed.

1.Slope Stability (GEOSLOPE 6.02, SLIDE 5.014) / Structural Analyses (SAP 2000, V8.4)

2. Finite Element Method (PLAXIS 2D, V8.6)

In this stabilization program, one row of stabilizing piles are placed at an upper level (Pile Row K1 in Figure 1). The lower row of piles (Pile Row K2 in Figure 1) will also have a function of retaining wall for the facility. To decrease the moment and the displacements of the piles in this second row, permanent ground anchorages were planned at the top beam of the piles.

The most critical issue in the analyses were the excavation works for the foundation of the facility which would be performed after the stabilizing works were completed. Because of the excavations just behind the second row of the piles, the pile displacements and the moments were increasing for both row of piles. This critical concept was best configured in 2D finite element analyses (figure 3).

## 2.3 *Finite Element and Limit State Analyses*

The place of the two rows of the stabilizing piles is shown in the plan view in Figure 1. In the analyses the construction stages were excavations for stabilization, the construction of the piles and the foundation excavations to the 1095 m elevation just in front of the K2 Pile Row. For the drained analyses of the K1 pile row it was found that the pile head displacement as 18 cm, the shear force as 468 kN/m and the bending moment as 1240 kN.m/m. The same values for the K2 pile row were 10.8 cm, 387 kN/m and 970 kN.m/m, respectively.

The horizontal peak ground acceleration at the construction site was accepted as 0.25g and accordingly the seismic stability of the stabilizing system was also checked. The slope stability analyses of the final stage of the works were repeated for a horizontal seismic ground acceleration of 0.125g.

The cohesive soil exerts a pressure on the piles when it tries to pass between the piles forming a failure mechanism. This pressure being related to the undrained cohesion of the soil and is a limit value to be checked (see Eq. 1). This equation is valid for s≥3d where s is the pile center-to-center spacing and d is the pile diameter. If s<3d there will be a reduction up to a factor of 0.5.

$$p = c_u . k . D \tag{1}$$

Active and passive lateral earth pressures are represented by spring constants k above and below sliding surface. The k values for this case were 1.30 for the passive side of the lower level (K2) piles and 1.34 and 0.86 for the passive and active side of the upper level (K1) piles. These are lower than the limit values (Ergun, 2000).

## 3 CONCLUSION

A long landslide which progresses backwards had occurred because of the foundation excavations of an industrial plant which coincided with the toe of a historic landslide. The construction works were stopped, and the landslide was investigated through boreholes. The clayey debris (colluvium) containing limestone cobbles and boulders had slided over plastic tuffs. Back analyses and the laboratory tests were performed for the determination of the shear strength parameters at the sliding surface. Two rows of stabilizing piles were designed through the sliding mass and the structural analyses and also FE analyses of these piles were performed independently.

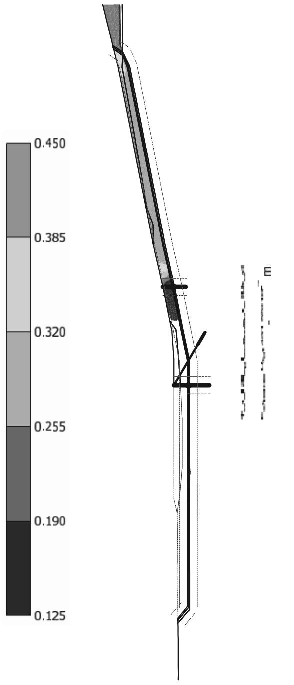


Figure 3. Total displacements plot of FE analysis

10 to 15 m deep excavations were performed for the first stage of the stabilization works. The upper level piles (K1) were 12 m in length and 1.2 m in diameter. Center to center spacing of piles were 2.5 m. The total number of stabilizing piles is 30 covering a longitudinal length of 72 m reaching the South edge of the landslide. The 2nd row of stabilizing piles (K2) was also used as the retaining walls of the facility. The 1.2 m diameter bored reinforced concrete piles are placed with 1.25 m spacing in this row. The pile lengths are 17 m. The total length of the pile row is 137 m in plan view passing over the South edge of the landslide.

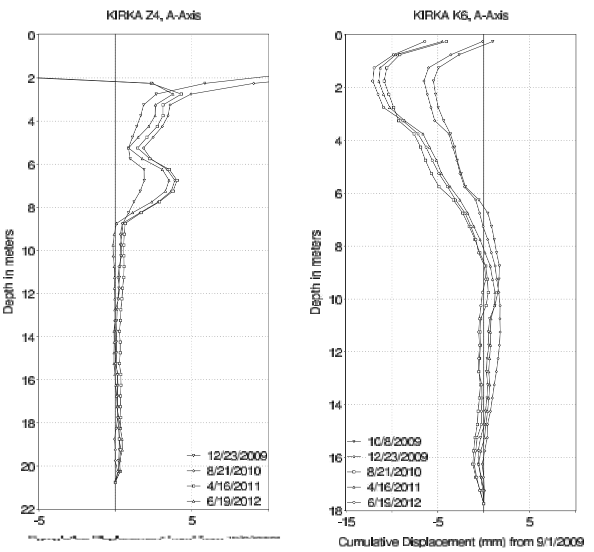


Figure 4. Cumulative displacements (mm) at Z4 (soil) and K6 (pile) inclinometers

The permanent ground anchors were constructed at the head beam of the piles with 1.25 m spacing, 30 to 35 m in length and inclined at 30 and 45 degree to the horizontal.

No drainage system was planned in the sliding mass. Instead surface drainage was implemented as reinforced concrete drainage channels at the edges of the landslide and at the upper levels of the new road. Behind the retaining walls of the construction site 0.60 m diameter vertical drainage shaft between the piles were planned and the collected groundwater was transferred to the water drainage system of the facility.

The deformations of the system are controlled by the 12 inclinometers (8 in the soil, 4 in the piles) at each stage of the construction works. There are no displacements in three years (Figure 4).

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