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MONITORING AND ANALYSIS OF A REACTIVATED LANDSLIDE WITH UNCERTAIN BOUNDARIES IN AN URBAN AREA

Sevki Ozturk

Civil Engineering Department, Cankaya University, Ankara, Türkiye. E-mail: sevkiozturk@cankaya.edu.tr

Volkan Kalpakci

Civil Engineering Department, Middle East Technical University, Ankara, Türkiye. E-mail: kalpakci@metu.edu.tr

Nejan Huvaj

Civil Engineering Department, Middle East Technical University, Ankara, Türkiye. E-mail: nejan@metu.edu.tr

Ufuk Ergun

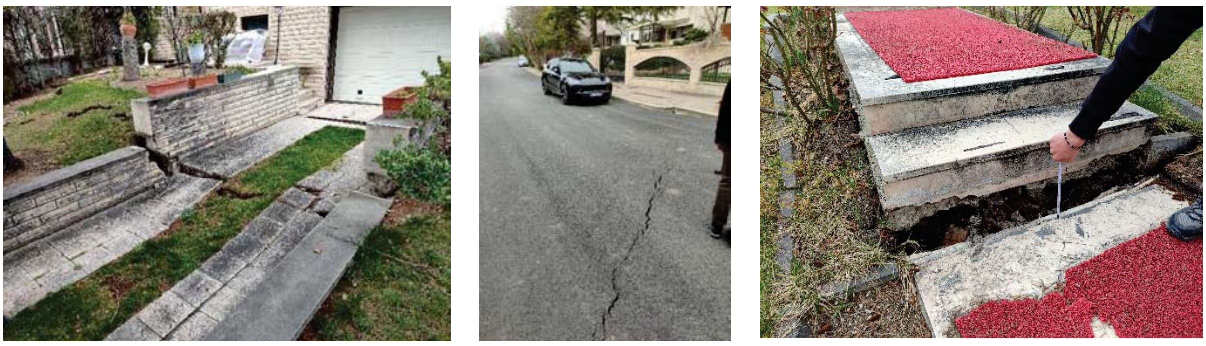
Civil Engineering Department (Emeritus), Middle East Technical University, Ankara, Türkiye. E-mail: eruf@metu.edu.tr

A landslide was triggered in January 2024 in an urban area in Ankara, Türkiye. During reconnaissance survey, it was found out that an old landslide which was stabilized at the same site almost 30 years ago was reactivated by an excavation in the sliding mass.A site investigation and monitoring program was executed which revealed that the depth and mechanism of the old landslide was altered such that the new landslide had roughly a rotational mechanism while the old landslide was more translational. After the excavation started in January 2024, the landslide movement continued with an increasing displacement rate. So, it was decided to analyze the landslide and design a stabilization system using reinforced concrete bored pile rows. The analyses were carried out by both limit equilibrium and finite element methods.Although the boundaries of the landslide were determined at the upper parts (crown, main scarp and sides) of the sliding mass, the boundaries at the lower part were uncertain due to the on-going construction activities in the excavated area. So, a series of analyses were carried out to estimate the probable toe of the sliding mass. The depth of the landslide observed from inclinometer readings and idealized soil profile obtained from site investigations were utilized as input in the analyses while the uncertainty both in the soil parameters and landslide geometry were considered for probable failure scenarios.The analyses have revealed that the most probable landslide toe was almost in the middle of the excavation area,which was then proved by site observations. The potential parametric and geometric uncertainties wereevaluated in the design of the geostructure for landslide stabilization. The effects of the spacing of the pile rows and pile cap dimensions were also investigated in this stage.

Keywords: uncertainty; landslide,geostructure, pile, monitoring, rotational, translational, stabilization

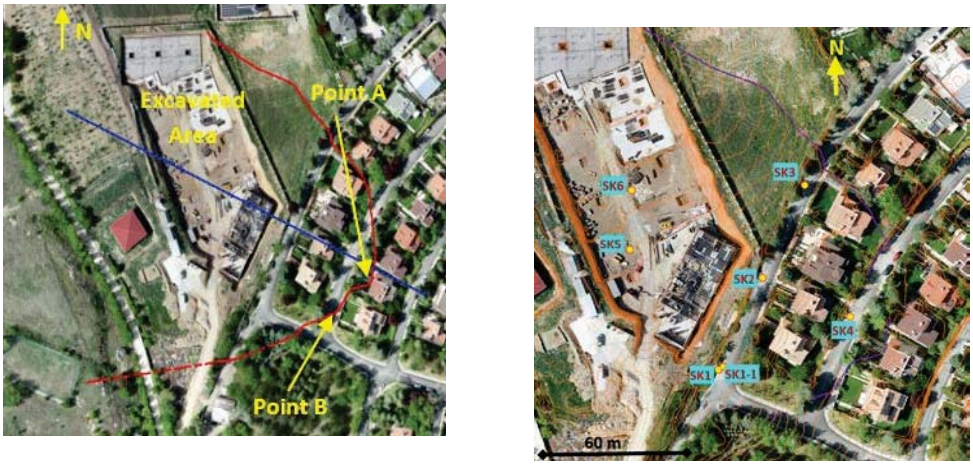
1. Introduction

In 1993, a landslide had occurred in an urban area in Ankara, Türkiye affecting a residential building site. This landslide was stabilized with a combined solution including a toe embankment and dewatering. Between 1994 and 2024 there were no movements. In January 2024, a new landslide was triggered in the same area due to an excavation in the lower body of the sliding mass (maximum excavation depth of about 8 m). Serious damages in the buildings, roads, gardens and infrastructure have been observed (Fig. 1).Although the boundaries of the landslide at the upper parts (crown, main scarp and northern side boundary) could be clearly observed during site visits aerial imaging was carried out at site by LIDAR, to determine the landslide boundaries. However, there was still anuncertainty at the toe of the landslidedue to ongoing construction activities.



(a) (b) (c)

Fig. 1. Deformations and cracks observed due to landslide((a) and (c)near the crown, at location “A” in Fig 2(a), (b) at location “B” in Fig 2(a))



(a) (b)

Fig. 2. (a) Aerial image of the landslide site and boundaries of landslide, (b) approximate locations of the boreholes

2. Site Investigation Studies

6 boreholes with depths in the range of 16-25m and 6 inclinometers with depths up to 21 m were placed (Fig. 2(b)). Field tests (SPT and PMT) and laboratory experiments were conducted. The soil profile is composed of silty clay units but at certain depths sandy layers were encountered. White colored, weak limestone units (RQD values near zero and highly weathered, W5) were observed beneath these clay units (Fig. 3).Ground water table was measured at 7 m depth at boreholes SK1, SK2 and SK4. Soil parameters were determined based on field and laboratory tests. The selected soil parameters are summarized in Table 1. The residual shear strength parameters of the sliding layer weredetermined by aback-analysis process details of which are explained in the next section.

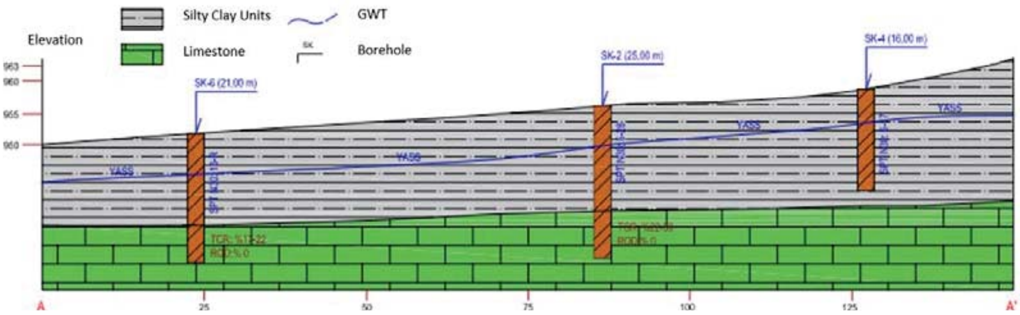
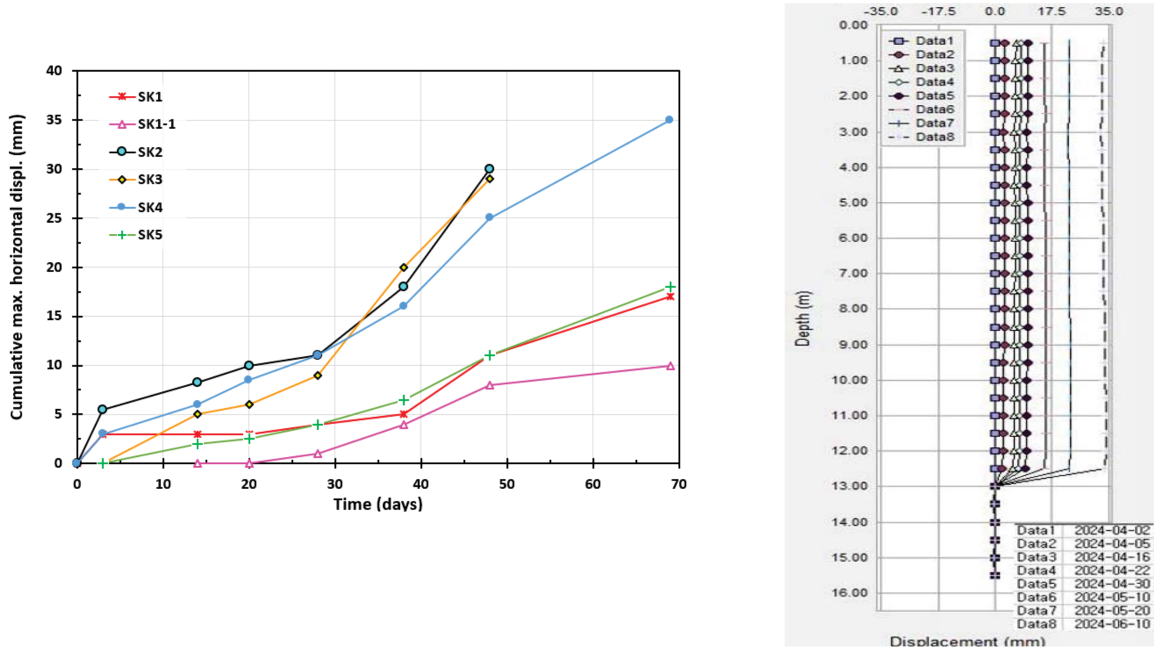


Fig. 3. Idealized soil profile (section A-A' (blue line shown in Fig. 2(a)) before the excavation)

Inclinometer readings between 2 April 2024 and 10 June 2024 (in approximately 70 days-time period) were used to investigate the landslide depth and the rate of movement (Fig. 4). According to theinclinometer data, the failure surface is observed at 17.5m depth in SK-2 inclinometer, 10.5m depth in SK-3 inclinometer, 13m depth in SK-4 inclinometer, 10-11m depth in SK-5 and SK-6 inclinometers.The monitoring program revealed that the depth and mechanism of the old landslide was altered such that the new sliding mass had a deeper sliding surface in a relatively short distance (maximum of about 17.5 m deep landslide having H100m length) while the old landslide had about 12m depth and 150m length. According to Skempton& Hutchinson (1969),the ratio of the depth to length (maximum depth of the surface of rupture below original ground surface, to the length of the surface of rupture) is less than 0.1 for translational landslides.In other words, the new landslide had roughly a rotational mechanism while the old landslide was more translational. The depth of failure surface from inclinometer measurements and the tension crack locations observed in the field were utilized in the back-analyses.



(a) (b)

Fig. 4. (a) Cumulative max. horizontal displacement with time, (b) Depth versus horizontal displacement measurements at different times at inclinometer located at SK4 location

Landslide movement rate was observed to be changing in time Fig. 4(a), however, the largest movement rate (12 mm / 10 days) is measured at the SK2 inclinometer location, which was at the mid-length of the landslide body, where the depth of the sliding surface was the largest. The highest rate of movement of 1.2 mm/day is less than 4.3 mm/day, which is the upper limit value for “very slow” landslide class,according to Cruden and Varnes (1996) landslide velocity classification. This case study illustrates that “very slow” landslides can cause significant damages on buildings and roads.

3. Geotechnical Analyses

As explained in the previous sections, there were uncertainties both at the toe of the sliding mass and residual shear strength parameters of soil at the sliding layer. A series of back analysesare performed based on inclinometer readings and soil investigation studies. After obtaining the residual parameters and deciding on the landslide geometry, a stabilization project is recommended based on the findings of limit equilibrium (Rocscience-Slide) and finite element analyses (Plaxis 2D) as described in the following sections.In the limit equilibrium method Spencer (1967) method of slices is used. In the finite element analyses, standard boundary conditions are used in the model; the bottom boundary is fixed in all-directions, and the right/left boundary conditions are roller-support where deformations only in the vertical direction are allowed.

3.1.*Back analysis*

Using inclinometer readings and idealized soil profile obtained from the site investigations, back analyses were performed considering uncertainty both in the soil parameters and landslide geometry. Based on limit equilibrium and finite element back-analyses (Fig. 5), the drained residual internal friction angle of the sliding layer was determined as 8^0 which was consistent with the $8\text{-}10^0$ reported in the 1993 landslide studies and the empirical correlations in the literature based on the plasticity index of clays (e.g. Mesri and Shahien, 2003).At borehole SK4 the depth of sliding surface was determined as 13 m based on inclinometer readings (Fig. 4b), and clay sample taken at 12.0-12.5 m depth (near the sliding surface) had liquid limit (LL) of 91% and plasticity index (PI) of 54% (determined at METU Civil Engineering Department, Soil Mechanics laboratory by the authors), which indicates the back-calculated residual friction angle is reasonable. Furthermore, at SK2 borehole at 10.0-11.0 m depth LL of 74% and PI of 47%, and at SK5, at 11.5-12.0 m depth LL of 50% and PI of 24% was measured. The variability of the plasticity of the clay at the site should be noted.

Based on the back analyses, it was determined that the landslide surface was located belowthe base of the excavation, passing underneath and coming to the surface within the excavation area atabout 50m before the toe of the old landslide. The toe of the new landslide estimated by a series of back analyses were later observed at the site at a very close location, which shows the accuracy of the analyses.

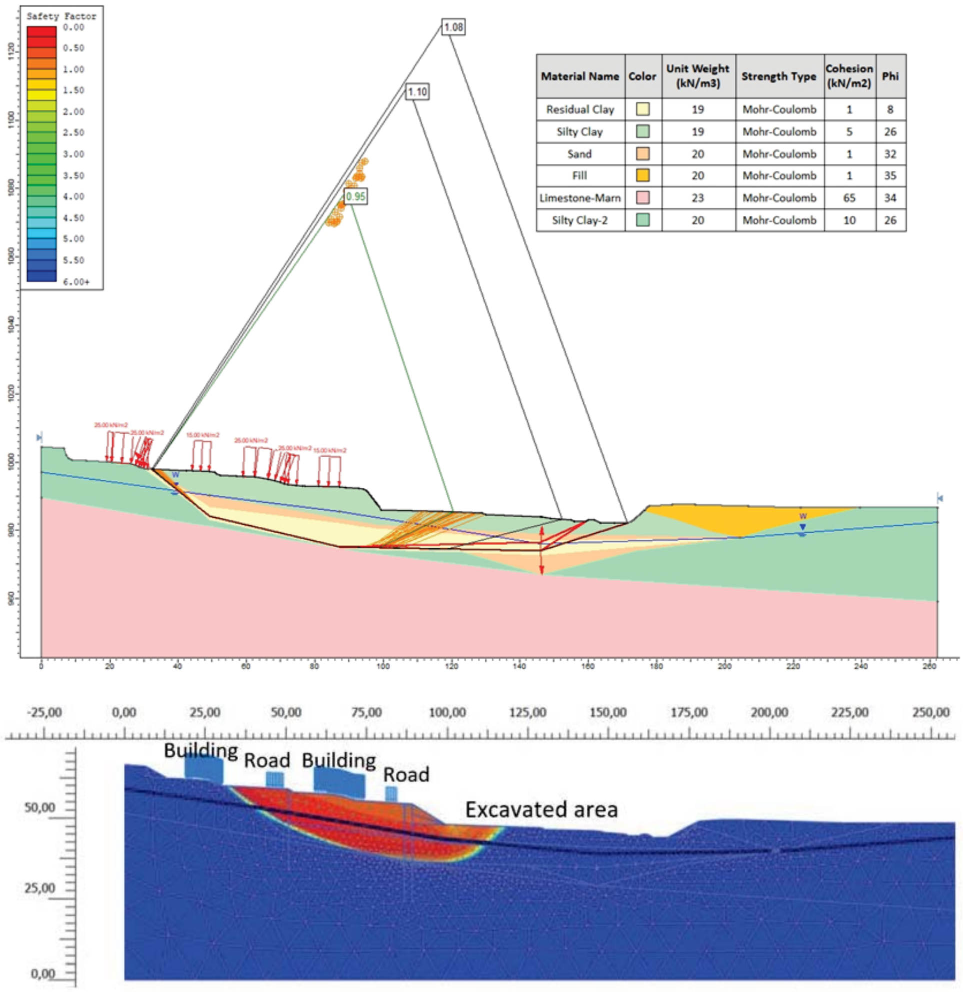


Fig. 5. Back analyses by limit equilibrium and finite element methods

Table 1. Soil parameters

Soil	γ kN/m^3	Average SPTN$_{60}$	PI %	c_u kPa	c' kPa	ϕ' ^{0}	E' MPa	E_u MPa
Silty Clay-1	19	10	29	50	5	26	21	30
Sand-Gravel	20	19	-	-	1	32	20	-
Silty Clay-2	20	18	28*	80	10*	26*	33.6	48
Limestone	23	-	-	-	65	34	150	-

* For the clay at the sliding surface,PI=47-74%, and the residual shear strength parameters of c'=1 kPa and ϕ'=8° is used.

3.2.*Limit Equilibrium and Finite Element Studies for Stabilization of the Landslide*

After the sliding mass geometry and residual shear strength parameters were determined, a landslide stabilization system using reinforced concrete bored piles were studied both with limit equilibrium and finite element methods (Fig. 6). Different alternatives were studied in order to see the effects of pile spacing and pile cap dimensions.During these analyses, it was seen that the bending moment capacity of the lower two rows of piles were exceeded, unless another row of pile at the upper part of the landslide was constructed. Also, the lower two rows of piles were connected with a single capping beam to limit the lateral displacements aiming to causeless damage in the surroundings. As a result, a stabilization system containing three rows of bored concrete piles (E_{conc}=28,000 MPa)was proposed.Two rows of piles(with pile diameter, D=120 cm and spacing of 320 cm with I=3.18x10^{-2} m^4/m) were placed at about the mid-length of the landslide, where the depth of the sliding surface was the largest; and they were connected with a single pile capping beam. Another row of piles (with D=100cm and spacing of 160 cm with I=3.07x10^{-2} m^4/m) were placed in the upper part, near the crown of the landslide, which had a separate pile capping beam.The stability of the landslide is checked both in the static and dynamic conditions (using pseudo-static dynamic loading represented by a horizontal seismiccoefficientsk_h=0.1g and the vertical k_v=0.05g, and reducing the undrained shear strength by 20%) and piles were structurally designed with the obtained loads and moments from finite element method.

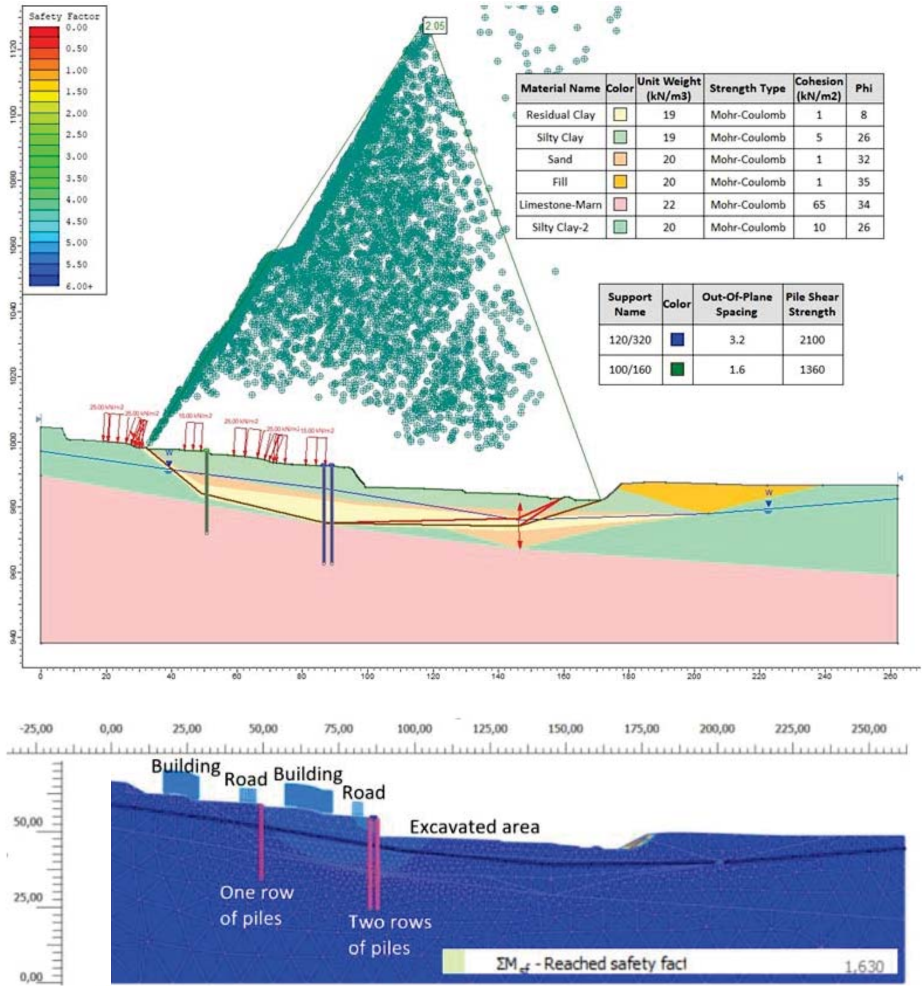


Fig. 6. Slope Stability after Improvement by rows of bored piles-Static Condition

In this study, a reactivated landslide with uncertainty both in the soil parameters and landslide geometryis studied in detail. Based on inclinometer data, reactivated sliding mass was observed to have a deep sliding surface (maximum depth is 17.5 m from ground surface)having about 100 m length,indicating a rotational slide mechanism. According to back analyses performed considering uncertainty both in the soil parameters and landslide geometry, drained residual internal friction angle of sliding layer was determined as 8^0 which was consistent with the 8-10^0 reported in the 1993 landslide studies. The back analyses have revealed that the landslide surface was located under the base of the excavation, passing underneath and coming to the surface within the excavation area, about 50m before the toe of the old landslide. The estimatedtoe of the new landslide was later observed at the site at a very close locationin agreement with the analyses.Different alternatives were studied in order to see the effects of pile spacing and pile cap dimensions. It is seen that placing a row of 100-cm-diameter bored piles located near to the crown of the landslide decreased both the lateral displacements and the shear and moment values acting on thelower rows of piles which had 120-cm-diameter near to the toe of the landslide. The pile spacing has a relatively minor effect on structural forcesas compared to the number of rows of piles for the studied range.

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