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STABILIZATION OF A FAILED SLOPE WITH PILED STRUCTURES

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ABSTRACT

Neogene aged units of a densely populated region of Western Turkey along the Aegean Sea coastline is susceptible to landslides causing frequent economic loss especially following raining seasons. Several landslides took place in the area covering a narrow band of the coastline between Izmir and Söke (Aydın). Countermeasures against these relatively small-scale slope failures in the region often involve construction of either reinforced concrete retaining walls or stabilizing piles, which can be easily constructed by local contractors. In this study borings, in-situ and laboratory soil mechanics tests, geophysical and geological investigations have been performed in order to investigate the landslide occurred in the yard of an elementary school in Söke township. The analysis of two rows of piled retaining system constructed to reuse the school building against a potential slides are presented. Three inclinometer measurements have been performed after completion of the bored pile system. It has been concluded that the measured and the calculated displacement values are both small. There is no problem of the built project by means of moments and displacements.

Key Words : Landslide, Investigations of sliding surface, Stabilizing piles, Two row of retaining system.

KAYAN BİR ŞEVİN KAZIKLI SİSTEMLERLE STABİLİZASYONU

ÖZET

Türkiye’nin batısında, Ege Denizi kıyısı boyunca uzanan yoğun nüfusa sahip bölgedeki neogen yaşlı birimlerde, özellikle yağışların da etkisiyle büyük ekonomik kayıplara yol açan şev kaymaları oluşmaktadır. İzmir-Söke arasındaki dar bandı da içine alan bölgede meydana gelen, bölgesel ve küçük ölçekli şev kaymalarına karşı önlem olarak genellikle yerel müteahhitler tarafından imalatı kolaylıkla gerçekleştirilebilen betonarme istinat duvarları veya kazıklı iksa sistemleri uygulanmaktadır. Söke’de bir ilköğretim okulu bahçesinde meydana gelen heyelanın oluşma mekanizmasını araştırmak amacıyla yapılan zemin sondajları, zemin mekaniği laboratuvar ve arazi deneyleri, jeolojik ve jeofizik incelemeler yapılmıştır. Bu çalışmada, yeni bir heyelan riski altında bulunan mevcut okul binasının tekrar kullanıma geçirilebilmesi için inşa edilen çift sıra kazıklı iksa sistemine ilişkin analizler verilmektedir. Kazıkların imalatından sonra 3 inklinometre ölçümü yapılmıştır. Gözlenen ve analizler sonucu hesaplanan deplasmanların oldukça küçük değerlerde olduğu sonucuna varılmıştır. İnşaatı tamamlanan sistemde, taşıdığı moment ve gerçekleşen deplasmanlar açısından bir problem görülmemektedir.

Anahtar Kelimeler : Heyelan, Kayma yüzeyinin belirlenmesi, Stabilizasyon amaçlı iksa kazıkları, Çift sıra iksa sistemi.

1. INTRODUCTION

When a slope fails, it is often called a landslide or a slope failure. A landslide is a mass downward movement of either rock or unconsolidated material (Abramson et al., 1996). The movement is caused by gravity acting upon materials that are in an unstable state of equilibrium. A case study was presented of the effect of lateral movement and the analysis for preventing landslide using piled retaining system.

In order to determine the landslide mechanism, site and laboratory investigations were performed. Possible causes and the mechanism of the landslide have been thoroughly investigated. A low cost remediation project consisting of two rows of drilled shafts connected with a single continuous pile-cap has been prepared through effective use of the findings of the investigations. The most important two reasons for the usage of piles are wideness of the area where landslide occurs or has the potential and the magnitude of force that affects the constituted construction. Piled retaining systems lead failure surfaces go deeper. Because of the opportunity of arching affect in the soil, it is not necessary to constitute as continuous curtain. A retaining wall may only be constructed on a plane in a slope and it needs excavation during construction. Excavation work during construction even it is temporary leads to decrease the safety of the slope that is also in the limit state.

A landslide has occurred during a sloping ground excavation in the backyard of an elementary school located in Söke, which is a town of Aydın Province. The studies that have been done in order to rescue the school building from unserviceable position and to relieve the present unacceptable situation are declared.

2. ENGINEERING GEOLOGY

Several landslides took place in the area covering a narrow band of the coastline between İzmir and Söke where the site of interest is also located. General geology map of Aegean Region and the location of the study area are shown in Figure 1 below.

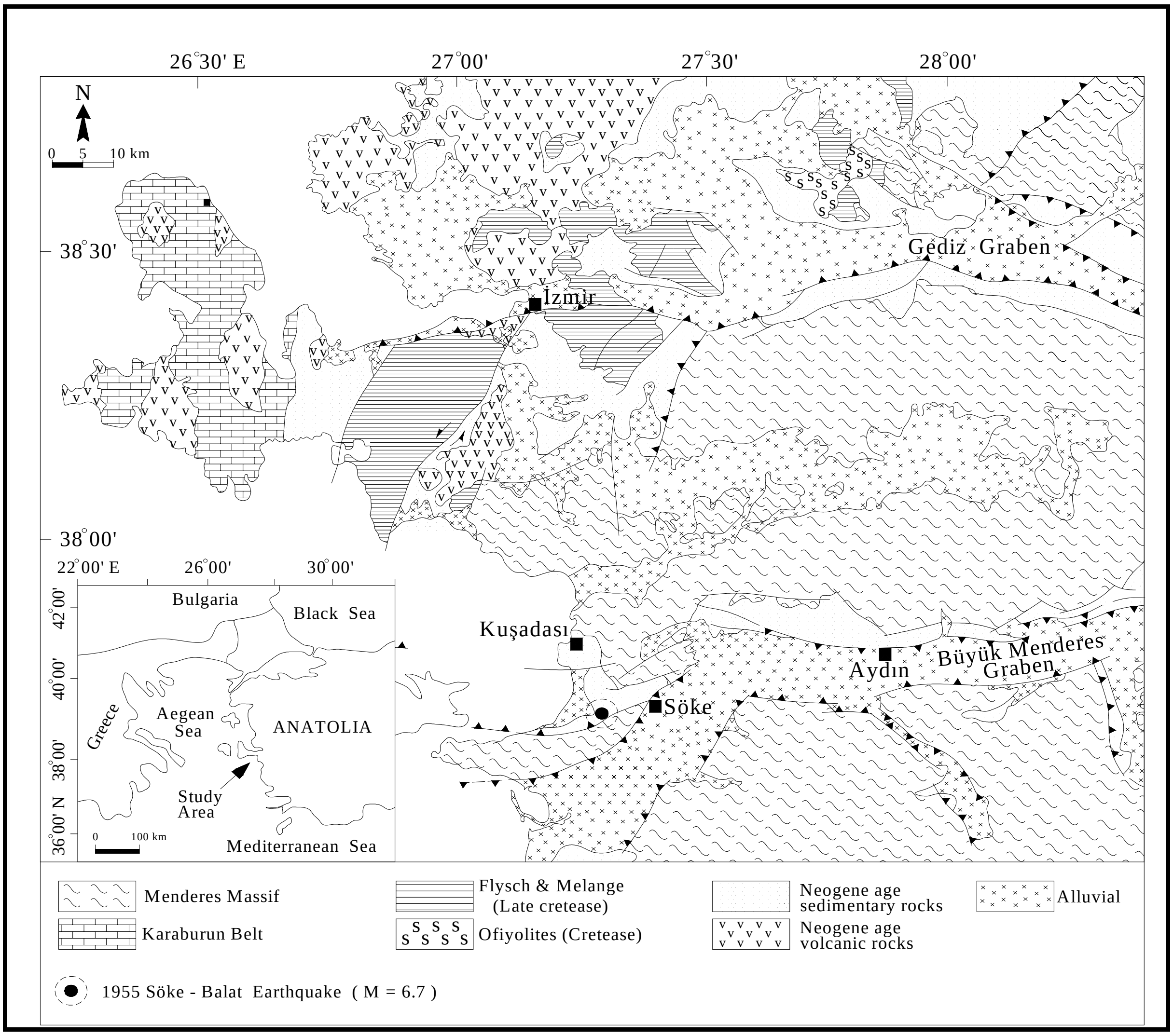

Figure 1. Geological map of aegean region and location of the study area (Genc et al., 2001).

Paleozoic age metamorphic mica schist and marble rocks belonging to the Menderes Massif constitute the base rock in the study area. Neogene age alternating formations of siltstone, claystone and sandstone were formed by means of sedimentation covering the metamorphic base rock in the west and north of Söke with angular unconformity displaying a widespread dispersion in the region. Quaternary alluvial soils, on the other hand, cover all these units. The mica schist rocks observed in the north of the investigation area are mainly composed of chlorite, muscovite, and quartz schist (Genç et al., 2001).

The Menderes Massif and the Lycian belt govern seismic activity of the study area. The İzmir-Ankara zone to the northwest and Lycian nappes to the south tectonically border the Menderes Massif, which consists of mainly Paleozoic age metamorphic rocks and stayed for a long period as the continent shield. It has been stated that no graben formation occurred until Miocene period. The Menderes Massif, however, has been broken along deep lines in the east-west direction during the neotectonic regime after mid-Miocene, forming the famous grabens of the west Anatolia (Güngör, 1998). Normal faulting is the common mechanism of major faults in the neighborhood generating earthquakes as large as M = 6.7 on Richter scale as shown in Figure 1.

3. THE LANDSLIDE

The landslide took place on the grounds of the recently constructed five-story elementary school building. The area has already been defined to carry low landslide risk in official geological maps of local authorities. The excavation activities pursued during construction, however, disturbed the delicate balance of the former residual shear planes and triggered a major landslide.

Neogene aged geological units take place within and around the location area of the school. These units, in general, are composed of layers of weak cemented siltstone and sandstone. As a result of field surveys, drills and geophysical experiments, it was found that geological layers and their slopes and directions were developed in different directions, and that layers in the location of the school were immerged with angles of 20^o – 25^o towards the East. The area of investigation presents topography with a slope from the North East to the East of Söke. This area contains fossil landslides and this situation can be seen upon investigation of the former topographic maps.

The school building itself was built on an area with a 30% slope, after some slope arrangement, on the South East-East part of a morphology showing a protrusion towards the East. This slope is on a morphology that forms a stair-like structure. One gets the impression that “This morphology also reflects the presence of a fossil landslide formed during geological ages” (Aydar, 1998).

Slope failure, a sliding caused by tectonic forces aftermath the formation of these units during Quaternary period, occurred at the edge of a sliding plane. Slope failure occurred as a result of the fact that the balances attained in time by disturbed geological units and rainwater that increase the pore water pressure and decrease soil strength. The fact that slopes and directions of the geological units in that area were directed towards the East is effective in the development of the slope failure to happen towards the school building.

As a result of deterioration of the balance of the disturbed rocks along the sliding surface and of the natural ground water flow due to the excavations, the soil mass slid to the west of the school building. The slide was mapped on Figure 2.

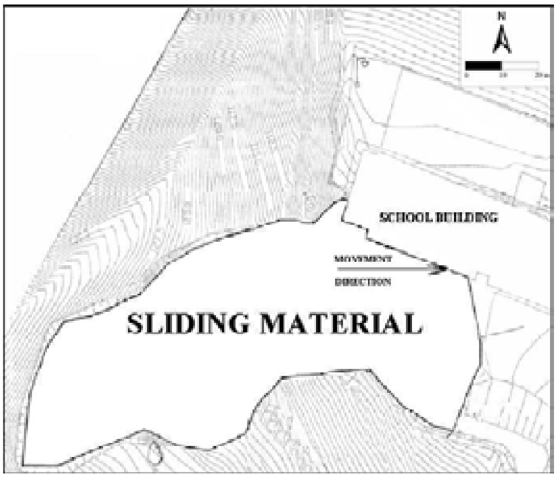

Figure 2. Map of sliding material (zone) in the school area.

4. SITE INVESTIGATIONS

Bore holes were drilled for idealizing the landslide profile, standard penetration tests (SPT) were performed and representative SPT samples took for the laboratory tests. In addition, to understand the underlying structures and discontinuities of the terrain, geophysical studies were conducted alongside the borings locations.

Bore holes were conducted with rotary drilling technique. The total number of the bore holes was eight. Six of these were located on and the remaining two were off the sliding zone. The locations of the bore holes were shown in Figure 3.

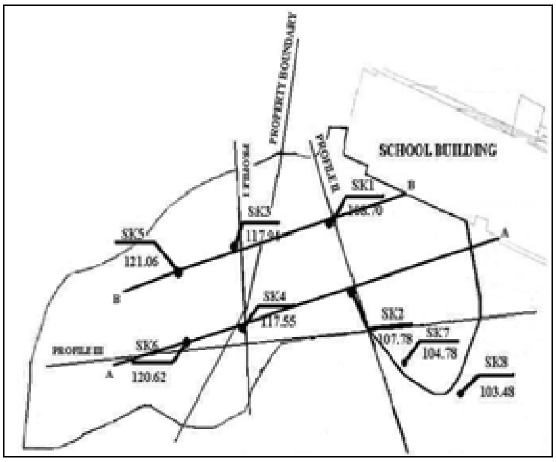

Figure 3. Map of borings and geophysical profiles' locations.

The thickness of the landslide zone was determined to be varying in between 4.5 m–10 m. The minimum was in the SK-2 location and the maximum was in the SK-5 location. This variation reflects the three-dimensional geometry of the sliding wedge. Geophysical investigations have also indicated similar results.

SPT samples have been taken at various depths along the slid mass. Also disturbed samples have been extracted from the middle and top part of exposed sliding surface. Standard penetration resistance values (SPT – N_{30}) of the slid material have been found to vary in between 10 and 27. The penetration resistances of the rock below the sliding surface were essentially greater than 50.

In order to understand underlying structure (bed rock slope, discontinuities etc.) of the terrain better, geophysical studies have been conducted on three different axes (Profile I, Profile II and Profile III in Figure 3) alongside selected borings.

Resistivity tomographs have been performed along these three axes. The first two refraction profiles (Profile I and II) were in NW-SE and the third (Profile III) was in E-W direction.

P and S waves elevation of Profile II and resistivity tomograph of Profile III were selected to be good representations of sliding mass and findings related with these profile were briefly summarized.

P and S wave velocities, given in Profile II, were calculated 397 m/s and 195 m/s at the top, and 2140 m/s and 719 m/s at the bottom, respectively. Rather big differences of wave velocities in the upper and lower parts of related sections reflected the quality differences between the upper soils that participated sliding and lower stable formations. Poisson's ratios were calculated as 0.34 m/s at the top and 0.44 m/s at the bottom. The thickness of upper weak layer represented in this profile varies in between 4-8 meters. This thickness range was in accordance with the borehole findings (Figure 4).

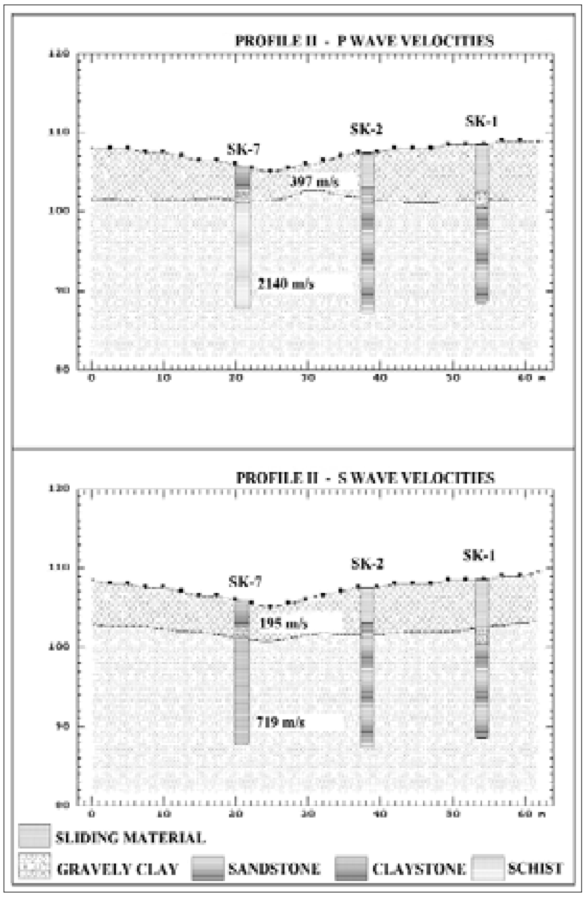

Figure 4. Profile II – Elevation of P and S wave velocities.

Profile III (tomograph of resistivity) passes just from the south of SK-6 and SK-4 borings and extends towards the west. Landslide geometry is seen in the mid part of the profile as a half circle (Figure 5).

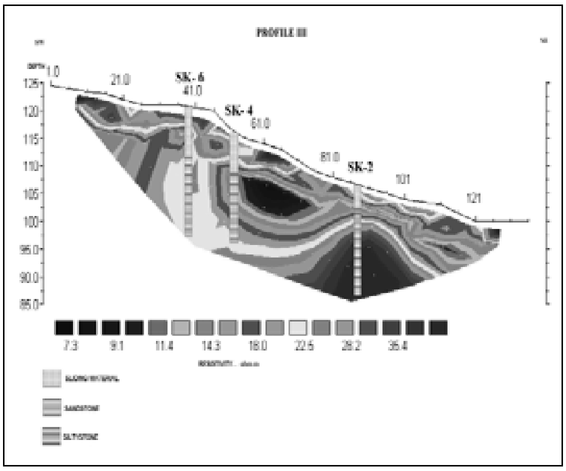

Figure 5. Profile III - Two-dimensional resistivity tomography.

5. LABORATORY TEST

Water content, consistency limits, sieve analyses and direct shear tests were performed on the forty three soil samples (Kayalar et al., 2004).

According to classification tests, gravel was the dominant element of the four disturbed samples. The eighteen of the soil samples were sand and; the remaining twenty one of them were clay or silt. As a result of these classification tests, it was seen that material on the sliding surface was silty sand and clayey sand (SC / SM). The soil in the sliding mass was mainly silty sand (SM).

The water contents of soil samples that are taken from 8 borings have been determined. Water contents range of all samples is 10%-28%. The samples that show plastic behavior were clay with low plasticity (CL).

Direct shear tests were performed in order to obtain shear strength parameters for sliding surface. The samples taken from various borings and close to failure surface depths were kneaded in their natural water contents and the disturbed samples have been prepared. They were compacted to gain a unit of $\gamma_n = 18$ kN/m^3. This value was the average of the unit weights of undisturbed hand samples.

These samples have been sheared under vertical stresses of approximately 0.25-0.50-1.0 and 2.0 kg/cm^2. Samples have been firstly sheared in their natural water contents, and then sheared under full saturation. Residual strength parameters have also been obtained.

Residual cohesion intercepts have been obtained for most of the samples tested. Peak and residual effective cohesion intercept values have been found to be very low. So, samples have been considered as cohesionless. Residual internal friction angles have been determined in between 9.3° and 17.4°. The average is $\overline{\phi} \cong 12°$.

Physical properties tests, unconfined compression tests, and ultrasonic velocity tests have been performed on the weak cemented rock samples. Physical properties tests (dry and saturated unit weights, porosity and void ratio) were conducted over 15 samples. Average values of these physical properties were given as 1.83 gr/cm^3 for dry unit weight, 2.12 gr/cm^3 for saturated unit weight, 29 % for porosity and, 0.42 for void ratio. Average of three unconfined compression strengths has been reported as 204 kg/cm^2.

Lab-Ultrasonic Velocity Determination Tests have also been done on 15 samples and; minimum, maximum and average values of primary wave velocity has been determined to be 1775 m/s, 3222 m/s and 2583 ± 420 m/s respectively.

It has been stated that there was a clear correlation between the results of geophysical data and boring data. Values of primary wave velocities of rock just below the sliding surface by seismic refraction method have been reported to be 3020 m/s and 2140 m/s along Profile I and Profile II, respectively. Laboratory determined mean value of ultrasonic velocity (2583 m/s) for rock samples taken from close depths was in accordance with the findings of refraction method.

6. THE PILED RETAINING SYSTEM

This prevention of landslide problem had several features. The first of them has been the difficulty of determining the failure surface. The other one was that the building exposed to the landslide belongs to an educational institution. Interests of the local press and families of the students have enforced the institutional administration to make immediate precautions. Serious question marks have appeared about the stability of the building and the slope. The obligation of finding an economical solution had made up another dimension of the problem. The top part of the sliding mass taking place in a private property has also raised another problem.

The piled retaining system has been chosen as the most suitable method for preventing the slide and to supply long-term stability. Moreover, other slide prevention methods have been evaluated as uneconomic for this small-scale project.

Two factors have been distinctive during the selection of the best location of the pile system. The first one among them has been the obligation of staying within the parcel border shown in Figure 3.

The parcel border has brought restrictions also on coordinating the slope. The second distinctive factor for selecting the best location has been the thickness of the slid mass above the bedrock. After examining the alternatives, the direction of SK-1/SK-2 boreholes (Profile II) has been determined as the most economical and the most suitable location for the site topography.

The piled retaining system has been designed to consist of double-row 49 reinforced bored piles with 120 cm diameter and 15 m length having a 1,13 m^2 cross section and 32ϕ26 reinforcement. This length ensured a socket length of 8 m in the bedrock. Center to center spacing between the rows and between the piles in a row have been arranged as 3.15 m and 2.4 m, respectively.

In order to provide the drainage of groundwater and to improve the properties of soil on the sliding surface, a horizontal drain system has been designed.

Horizontal drain system consisted of double-row, overall 12 horizontal drains, each of which has 20 m length and 4.80 m spacing.

Soil has been modeled to be composed of three zones for the analysis of the piled retaining system. These zones were sandstone at the bottom, sliding mass at the top and a thin failure layer in between.

Parameters of soil shearing strength (effective internal friction angle ϕ' and effective cohesion c') have been idealized from the results of direct shear tests.

Shear strength parameters were chosen for failure layer are $c' = 5$ kN/m^2, $\phi' = 13^0$. Other parameters for failure layer around sliding surface were saturated unit weight, γ_{sat}, effective modulus, E', Poisson's ratio, v, and pile-soil friction factor, δ. The values of these parameters have been taken as 20 kN/m^3, 5000 kN/m^2, 0.3 and 0.5 respectively. Sliding mass above the failure surface has been found to be composed of silty sand and partially saturated. Assigned parameters for sliding mass were $c'=5$ kN/m^2, $\phi'=18^0$, $\gamma_{sat} = 20$ kN/m^3, E' = 5000 kN/m^2, $v = 0.3$, $\alpha = 0.5$ and, for sandstone are $c' = 200$ kN/m^2, $\phi'=36^0$, $\gamma_{sat} = 20.5$ kN/m^3, E' = 900 MPa, $v = 0.25$, $\alpha = 0.67$. The bedrock parameters have been idealized by using the approach given by Bell and Culshaw (1993). The soil model and assigned parameters were shown in Figure 6.

The most critical state for the piled retaining system has been earthquake loading. A thin layer of the sliding zone over the bottom sandstone has been modeled with residual strength parameters. Relatively high shear strength parameters' values have been artificially appointed to the sliding mass above the failure surface in order to provide leaning on the piles without shear failure above sliding surface. In the case of earthquake, it has been assumed that the horizontal drains would dissipate the pore water pressure on failure surface.

In this way, it has been also assumed that the excessive wet term would not occur at the same time with the design earthquake. Earthquake calculations has been performed as pseudo-static with a horizontal acceleration value of a_{max}=0.192 g according to Code of Disaster.

According to the results of two dimensional finite element analysis using Plaxis, the values of maximum moment, maximum normal force and maximum tensile force affected on piles have been found as 511.18 kNm, 579.63 kN, and 572.81 kN, respectively in the axis on the side of school building. For the piles placed in the axis on the side of landslide, these values have been found as 780.16 kNm, 296.80 kN, and 219.66 kN respectively. The maximum deflection of the piles has been calculated as 3.5 mm.

7. BACK ANALYSIS (STABLE ANALYSIS)

Back analysis of the failed slope has been performed in order to evaluate the shear strength parameters on the failure surface.

By using related topographic maps, two different cross sections (Section A-A and Section B-B) (plan views of these cross sections) of the ground surface before the excavation, after the excavation and after the slide have been drawn. The representative cross section, B-B was presented on Figure 7 with failure circle, ground water table and boreholes.

Locations of the failure circles have been determined on Excel sheets by considering the findings both in boreholes and geophysical sections; in such a way that failure circle passes through failure surface indicated on each borehole. The coordinates from Excel sheets were used as the coordinates of sliding circle in slope stability analysis by STABLE program.

The value of effective frictional angles that provide a value of F = 1 for factor of safety against sliding have been determined by back analysis using Stable $\phi'=12°$ for the section given in figure.

This value of $\phi'=12°$ for section B-B was the same as the value of $\overline{\phi}_r' \cong 12°$ which is the average of residual effective values determined by direct shear tests. It can be concluded that the sliding mass was dominantly composed of silty and clayey sand and, this soil class has been effective on the low value of friction angle. Stable analyses of the slope considering the ground surface before excavation and the value of angle of shearing resistance as $\phi'=12°$ have been performed and the minimum value of safety factor against sliding has been found to be as F=1.26. This value of F indicates that stability of this slope was not sufficient even before the slope arrangement.

8. INCLINOMETER MEASUREMENTS

Four PVC pipes with 100 mm diameter were placed in the central position of the pipe holes (9, 19, 27 and 38 numbered piles) before concreting. These pipes have penetrated 5 meters below the bottom of the pile in order to reach a stable position.

Four inclinometer measurements have been performed. On 08.12.2004 the inclinometer reading was done for determining the reference line for other inclinometer readings. After this, first two measurements were taken in December 2004 (09.12.2004 and 30.12.2004). The third measurement were taken in 22.04.2005 after a heavy rainy season.

The latest cumulative displacement graphics of four stations (Pile-9, Pile-19, Pile-27 and Pile-38) have been plotted in Figure 8 for the purpose of comparison. According to the latest measurements conducted on 22.04.2005, the greatest pile head displacement occurred in the direction of slide for Pile-38. The value of this maximum displacement was 11.03 mm. The greatest pile head displacements in the other inclinometer stations for Pile-9, Pile-19, and Pile-27 were 9.08 mm, 10.92 mm and 3.29 mm, respectively.

It can be observed that the displacements occurred seem not to increase linearly with time, but increase with a decreasing velocity even after the intense precipitation period of 2005. It has been thought that the causes of the measured displacements may be the weight increase of the sliding mass due to precipitation and, shear strength decrease due to possible pore water pressure increase on the sliding surface. Thus, the sliding mass has transmitted its expected load by leaning on the retaining system. The limited maximum value of pile head displacements and the fact that measured displacements increase with decreasing velocity suggest that this piled retaining system would not experience hazardous displacement in the future.

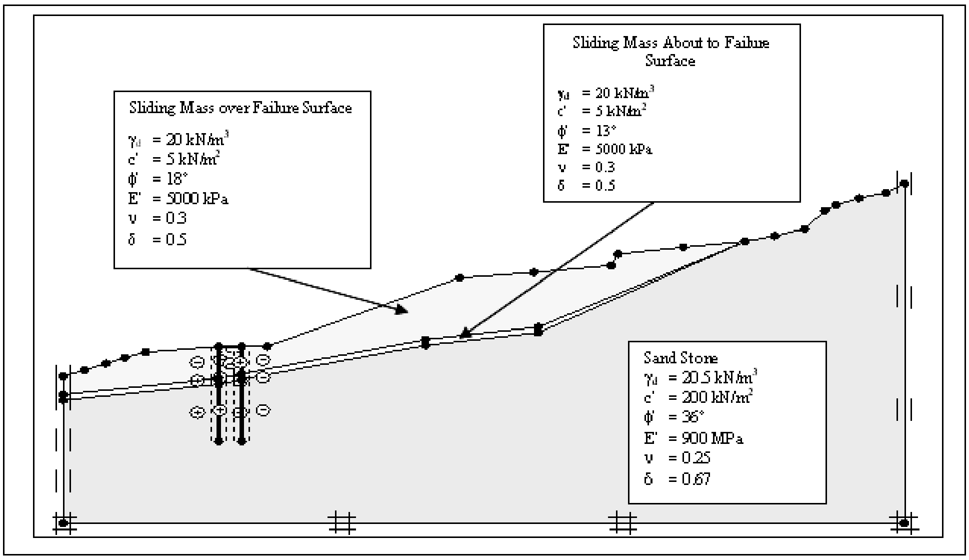

Figure 6. The profile for finite element analysis.

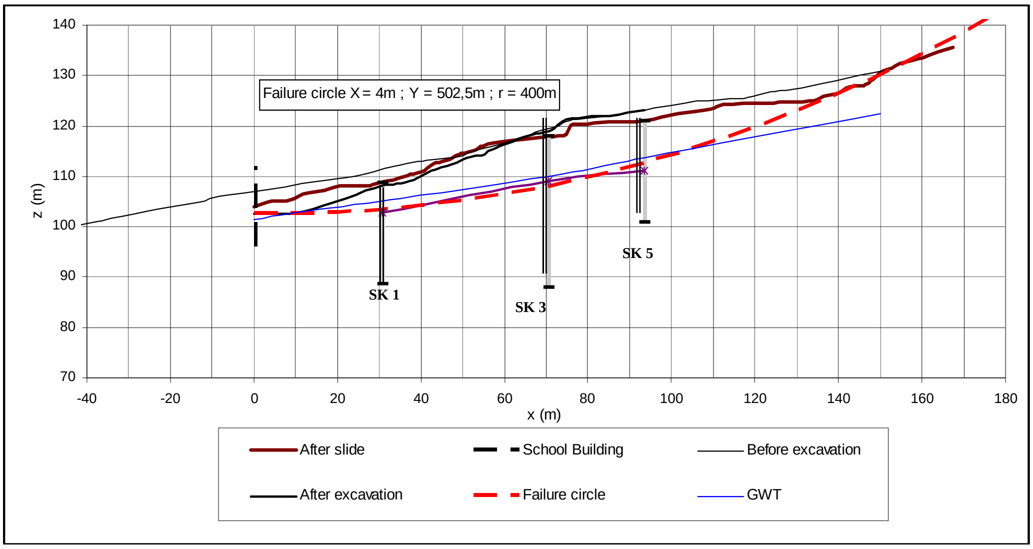

Figure 7. B-B cross section.

The slopes of the pile elastic curves were zero in the region of pile cap and pile bottom elevations. Pile elastic curves attained a reverse slope around 6-meter depth. The shapes of the elastic curves were consistent with the socket effect in the weak rock at the bottom and rigid pile cap at the top.

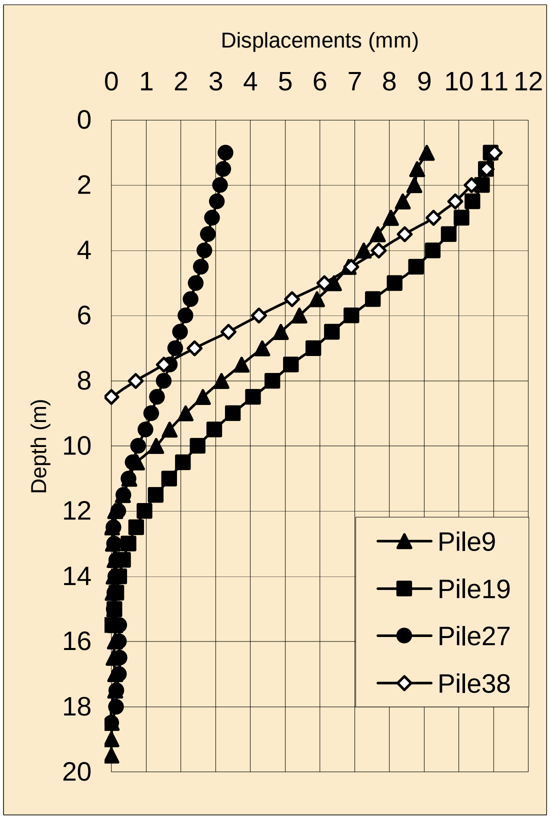

Figure 8. Cumulative displacement graphics using the data on 22.04.2005.

9. CONCLUSION

The landslide took place on the grounds of the recently constructed five-story elementary school building in Söke (Aydın). The area has been already defined to carry landslide risk to some degree in official geological maps of local authorities. Upon investigation of the former topographic maps, it has been concluded that this area contains fossil landslides. The area of investigation presents topography with a slope from the North East to the East of Söke. The excavation activities pursued during construction disturbed the delicate balance of the former residual shear planes and triggered a major landslide.

Slope failure occurred as a result of the fact that the balances attained in time by disturbed geological units and rainwater that increase the pore water pressure and decrease soil strength. The fact that slopes and directions of the geological units in that area were directed towards the East was effective in the development of the slope failure to happen towards the school building.

As a result of deterioration of the balance of the disturbed rocks along the sliding surface and of the natural ground water flow due to the excavations of school foundation, the soil mass to the west of the school building slid.

Based on several finite element analyses, the double-row piled retaining system has been found to be the most feasible alternative using locally available sources. After examining the alternatives, the direction of SK-1/SK-2 boreholes (Profile II, Figure 3) has been determined as the most economical and the most suitable location for the site topography. After investigations, soil has been modeled as composed of three zones for the analysis of the piled retaining system. These zones were sandstone at the bottom, sliding mass at the top and a thin failure layer in between.

A low cost remediation project consisting of two rows of drilled shafts connected with a single continuous pile-cap has been prepared by means of the findings of the investigations. The value of affected maximum moment on piles is 780.16 kNm determined by 2D finite element analysis (original project analysis), each pile in this system (having a 1.13 m^2 cross section and 32ϕ26 reinforcement) have 2500 kNm moment capacity. So the performance of the built project is found to be satisfactory.

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