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2D and 3D numerical simulations of a reinforced landslide: A case study in NE Turkey

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The purpose of this study is to investigate the slope stability problem that occurred in the Ulubey (NE Turkey) during the construction of a hospital building and to propose a reliable support design. The borehole applications, geophysical surveys, groundwater measurements, soil sampling and SPT were performed to establish the geotechnical model. Based on the site characterization investigations, three units were defined as sliding material, residual regolith and volcanic rocks. Strength parameters of the sliding and residual soil materials were obtained from the back analysis. The long-term performance of the double row-bore piles was proposed as support measures and was controlled using the limit equilibrium (LE) and finite element (FEM) analyses methods under a dynamic condition. The 2D-LE and 2D-FEM analysis results showed that the suggested support design is reliable for long-term stability. The locations of the critical shear surface determined by 2D methods were almost the same as those obtained from 3D-FEM method and the total displacement values obtained from the 3D-FEM model were smaller than those obtained from the 2D-FEM model. These results indicated that 2D and 3D stability analyses were sufficient to evaluate the stability of uncomplex slope geometry when a reliable design with simple solutions was required.

Keywords. Landslide; limit equilibrium; FEM; support design; Ulubey.

1. Introduction

Landslides and floods are the major natural geological hazards that cause substantial fatalities and property loss in Turkey, and they affect residential areas, settlement infrastructures, agriculture, transportation lines, and engineering projects such as hydroelectric power plants and dams. In the literature related to landslides, it was reported that the 70% of the landslides in Turkey have occurred in the eastern Black Sea region because of the geological, morphological structures, and climatic conditions (Bulut *et al.* 2000; Akgün *et al.* 2008; Akgün 2011; Alemdağ *et al.* 2014, 2015; Kaya *et al.* 2016a, 2017; Kul Yahşi and Ersoy 2018; Ersoy *et al.* 2019). Because the region is rainy during all seasons and the amount of precipitation is over 1500 mm, more than 50% of the landslide incidents in the eastern Black Sea region were recorded as rainfall-triggered landslides (Ersoy *et al.* 2019). The Çayeli (Rize) disaster in 2002, Gündoğdu (Rize) disaster in 2010, and Hopa (Artvin) disaster in 2015 are the most well-known examples. Approximately 20% of the average annual

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precipitation in Turkey (about 600 mm) was measured in 1 hr in the Hopa district and nearly 100 rainfall-induced landslide events were recorded (Durmuş 2016). Over 50 deaths were reported only because of these three disasters. In another catastrophic example, 170 mm of rainfall in 6 hrs was measured in 2016 and landslides caused many deaths in Ordu, which is another city in the north-eastern part of Turkey. In addition to sudden and very heavy rainfall, uncontrolled excavation is another important triggering factor that causes landslides in the eastern Black Sea region. For example, the Çatak landslide disaster (Trabzon city) in 1988, which was caused by prolonged heavy rainfall and incorrectly designed road cut application, caused 66 fatalities (Jones *et al.* 1989; Genç 1993).

Ulubey (Ordu, NE Turkey) is a small and developing district in the north-eastern section of Turkey (figure 1) with over 1500 mm of annual precipitation, and it is one of the highest precipitation areas in the region. It has steep peaks, sporadic regolith soils covering the topography, and instant flurries in the winter season. Thus, rainfall-triggered landslides are frequently encountered around Ulubey district. In addition, the natural topographic structure of the study area complicates planned urbanization and the buildings are often necessarily constructed on slopes. Highly weathered volcanic rocks are widely exposed in this area; therefore, often slope stability problems occur especially in urban areas where slope design errors also play a significant role. Thus, landslides are likely to occur in almost every uncontrolled excavation during construction activities in rainy seasons in the study area.

In this study, an Ulubey landslide (figure 2) triggered by an uncontrolled slope excavation was investigated to determine the failure mechanism and to design the appropriate retaining structures for stabilization. For this purpose, seven investigation boreholes were drilled and a seismic survey on a profile was performed to reveal vertical and horizontal continuity of the undisturbed and landslide materials. In addition, standard penetration tests (SPTs) were

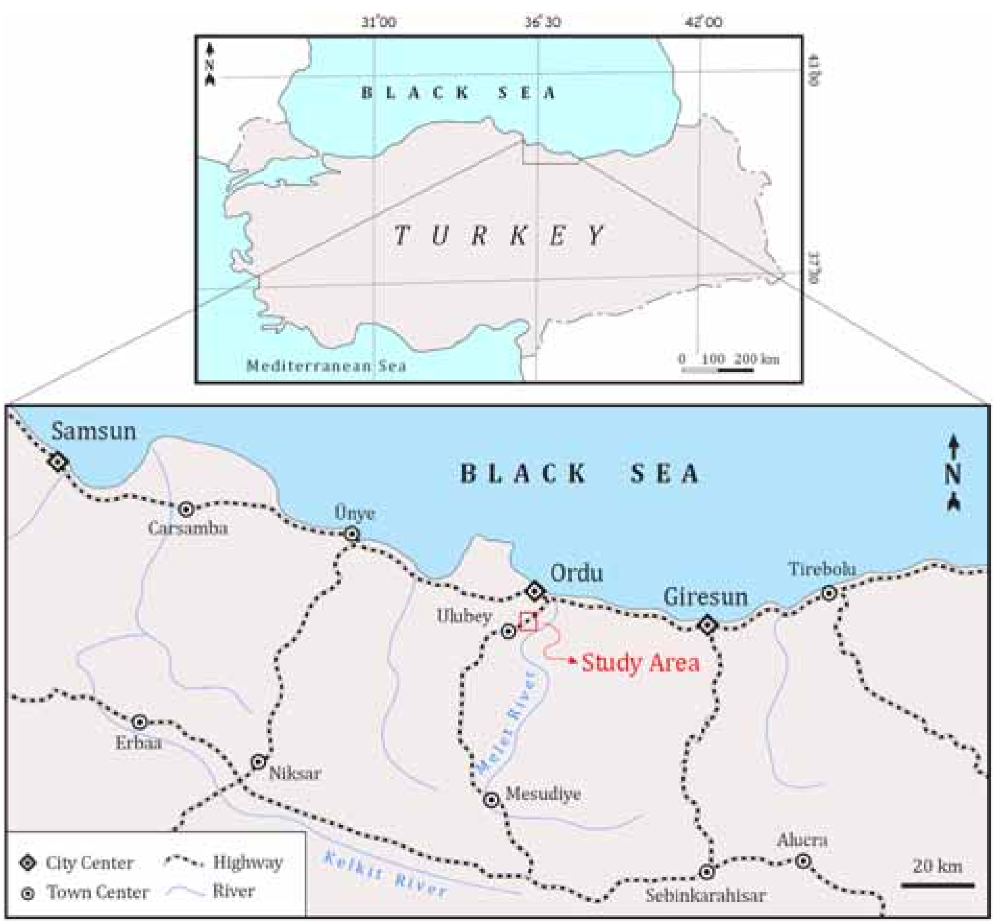

Figure 1. Location map of the study area.

Figure 2. Field view of the Ulubey landslide.

performed in three boreholes to investigate the variation of stiffness properties of the landslide material in depth. Undisturbed and disturbed specimens were obtained, and laboratory experiments were performed to define the geotechnical properties. Finally, to control the performance of the recommended stabilization technique, the limit equilibrium (LE) and finite element method (FEM)-based two- and three-dimensional (2D and 3D) numerical analyses were performed. The obtained results were then compared.

2. Geological setting

Considering the structural differences and lithological properties, the eastern Pontide is described as a geological and tectonic unit in Turkey (Özsayar *et al.* 1981; Bektaş *et al.* 1995; Okay and Şahintürk 1997). The study area is situated on the west side of the eastern Pontide.

In the study area, Late Cretaceous-aged formations are characterized by three different lithologies. The oldest unit, consisting of dacite, rhyodacite, and pyroclastites, is conformably overlain by claystone, siltstone, marl, and sandstone. The youngest unit in the Late Cretaceous contains andesite, basalt, and pyroclastites. These units are cross-cut by the Middle Miocene-aged acidic rocks. The Plio-Quaternary-aged regolith soil located in the study area includes saprolite in the lower zone and soil material in the upper zone. The widely exposed 20–25-m thick reddish-brown regolith soil indicates deep weathering of the bedrock surface. In the study area, flat areas are covered with residual regolith and the studied landslide occurred in the upper zone of the residual regolith that originated from Eocene–Miocene-aged basic volcanic rocks (figure 3).

3. Geotechnical investigations

The main objectives of the investigations and surveys that were performed on the site of potential slope instability are collecting all the information and data necessary for the stability analyses, assessing the risk of instability, and designing the remedial measures to relieve instability. For this purpose, seven boreholes were drilled (total length, 151.5 m) to describe the geological model, to establish groundwater conditions (table 1), and to obtain undisturbed samples for the laboratory tests. Four boreholes were opened on the landslide materials and three boreholes were drilled in the upper elevations from the main scarp of the landslide.

According to borehole and site investigations, the thickness of soil material varies between 4.0 and 20.7 m, and the groundwater level is located at a depth of 1.0–4.0 m from the surface. The length of the landslide, which occurred in residual soil with a maximum thickness of 20 m, is approximately 130 m, while the width ranges between 80 and 110 m. Investigations on the site characterization indicated that three different materials were defined depending on the depth from borehole logs. These materials are as follows: (1) sliding material (disturbed soil); (2) residual regolith (undisturbed soil); and (3) bedrocks consisting of andesite and trachyandesite.

SPTs were performed on the landslide material at 50-cm intervals in the BH-4, BH-5, and BH-6 boreholes to define the location of the failure surface. The SPT values with graphs are presented in figure 4. It was concluded that the landslide did not occur between the bedrock and soil contact, and the failure surface was located 1–2 m above the bedrock (figure 5). Furthermore, the seismic refraction survey on a profile was performed to obtain the dynamic elastic characteristics of the

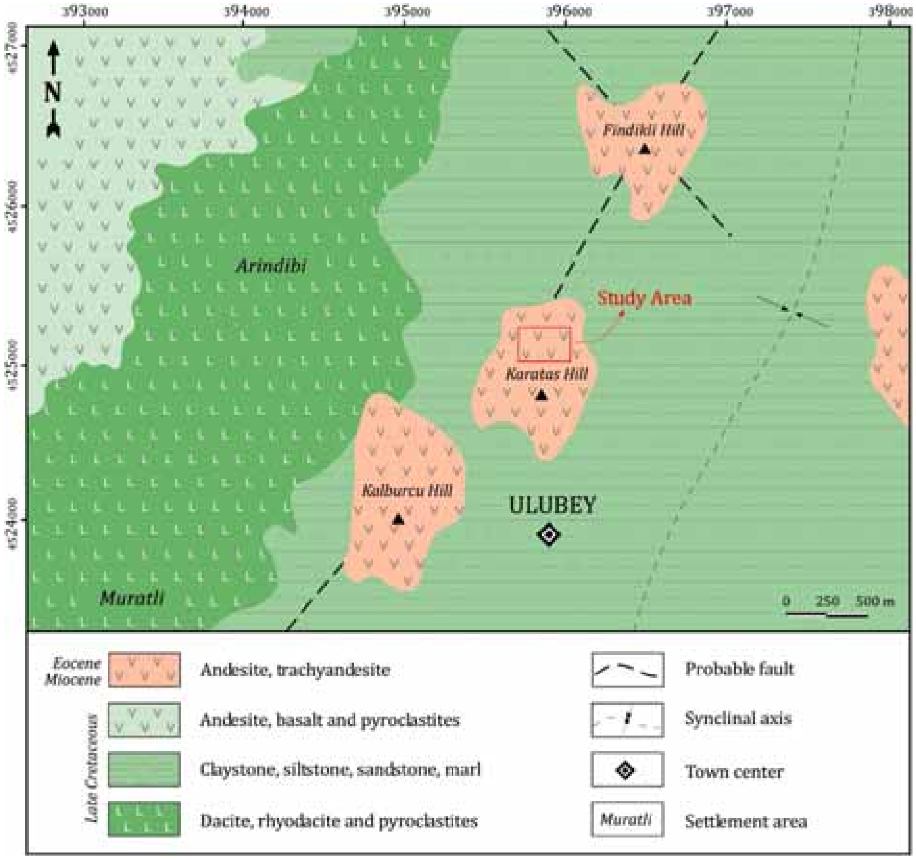

Figure 3. Geological map of the study area (modified from Temizel *et al.* 2012).

Table 1. *Summary of the borehole data.*

Borehole no.	Altitude (m)	Borehole depth (m)	Main rock depth (m)	Groundwater depth (m)
BH-1	563.0	24.0	10.0	4.00
BH-2	562.0	29.0	8.3	3.50
BH-3	561.0	22.0	8.8	3.00
BH-4	546.0	19.0	15.0	1.00
BH-5	546.5	21.0	15.3	2.00
BH-6	546.5	23.0	15.2	3.00
BH-7	530.0	13.5	2.2	4.00

geological materials. Using the outcomes of the data obtained from all investigations and the field observations, the engineering geological map with the scale of 1/2500 and the cross-sections of the landslide area were prepared according to IAEG (1976) procedures (figure 5).

The laboratory experiments were performed on the undisturbed soil samples acquired from BH-1, BH-2, and BH-3 boreholes to determine the physico-mechanical properties (ASTM D422-69 2007; ASTM D4718 2015; ASTM D4318-17 2017a). Based on the Unified Soil Classification System (USCS) (ASTM 2487-06 2017b), all samples were classified as the high plasticity clay (CH) (table 2). To determine the peak shear strength parameters (c_p and ϕ_p) of the residual regolith (undisturbed soil), consolidated-undrained (CU) triaxial compression tests were performed. In addition, the residual shear strength parameters (c_r and ϕ_r) were obtained from a back analysis method using the three different geological cross-sections (figure 6). The locations of the shear surface in different cross-sections were determined by the evaluation of drilling logs and SPT results as well as the main scarp location. As seen from table 3, the peak parameters were 50 kPa and 9°,

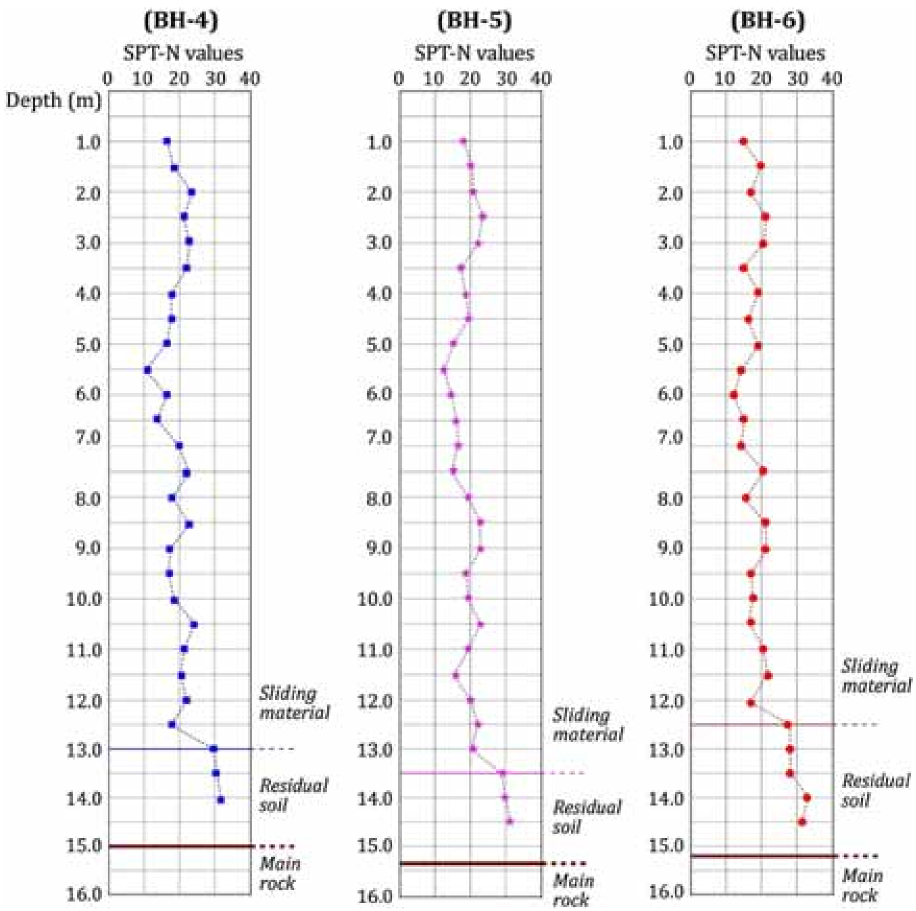

Figure 4. Results of SPT conducted in some boreholes.

while residual strength parameters were 19 kPa and 9°.

To define the elastic properties of the undisturbed soil and landslide material, the seismic refraction method was applied using the Geometrics model 12-channel seismograph to obtain a profile. The dynamic Poisson's ratio (v) and dynamic deformation modulus (E) were determined based on the following equations that were recommended by Tezcan *et al.* (2006) and the elastic theory using the P and S-wave velocity values (table 4).

$$v = \left[(V_P/V_S)^2 - 2\right]/2\left[(V_P/V_S)^2 - 1\right], \quad (1)$$

$$E = \mu\left(3V_P^2 - 4V_s^2\right)/\left(V_P^2 - V_s^2\right), \quad (2)$$

$$\mu = \rho V_s^2/100, \quad (3)$$

$$\rho = 0.44 V_s^{0.25}, \quad (4)$$

where v is Poisson's ratio, E is the deformation modulus (MPa), μ is the shear modulus (MPa), ρ is the density (kN/m^3) and V_p, V_s are P-S wave velocities (m/s).

The geophysical measurements show that P and S wave velocities are 411 and 184 m/s for landslide material and 814 and 384 m/s for residual soil, respectively. The dynamic Poisson's ratio and the dynamic deformation modulus were calculated as 0.37 and 150 MPa for landslide materials and 0.35 and 790 MPa for residual soil, respectively.

4. Stability analysis

Because quantitative determination of the slope stability is necessary for many engineering activities, the researchers have studied both rock and soil slope stability problems in the eastern Black Sea region (e.g., Akgün and Bulut 2007; Kesimal *et al.* 2008; Akgün 2011; Karaman *et al.* 2013; Alemdağ *et al.* 2014; Gelişli *et al.* 2015; Kaya *et al.* 2016a, 2017; Kul Yahşi and Ersoy 2018, Ersoy *et al.* 2019). Although several techniques are available for slope analysis, based on the type of matter and

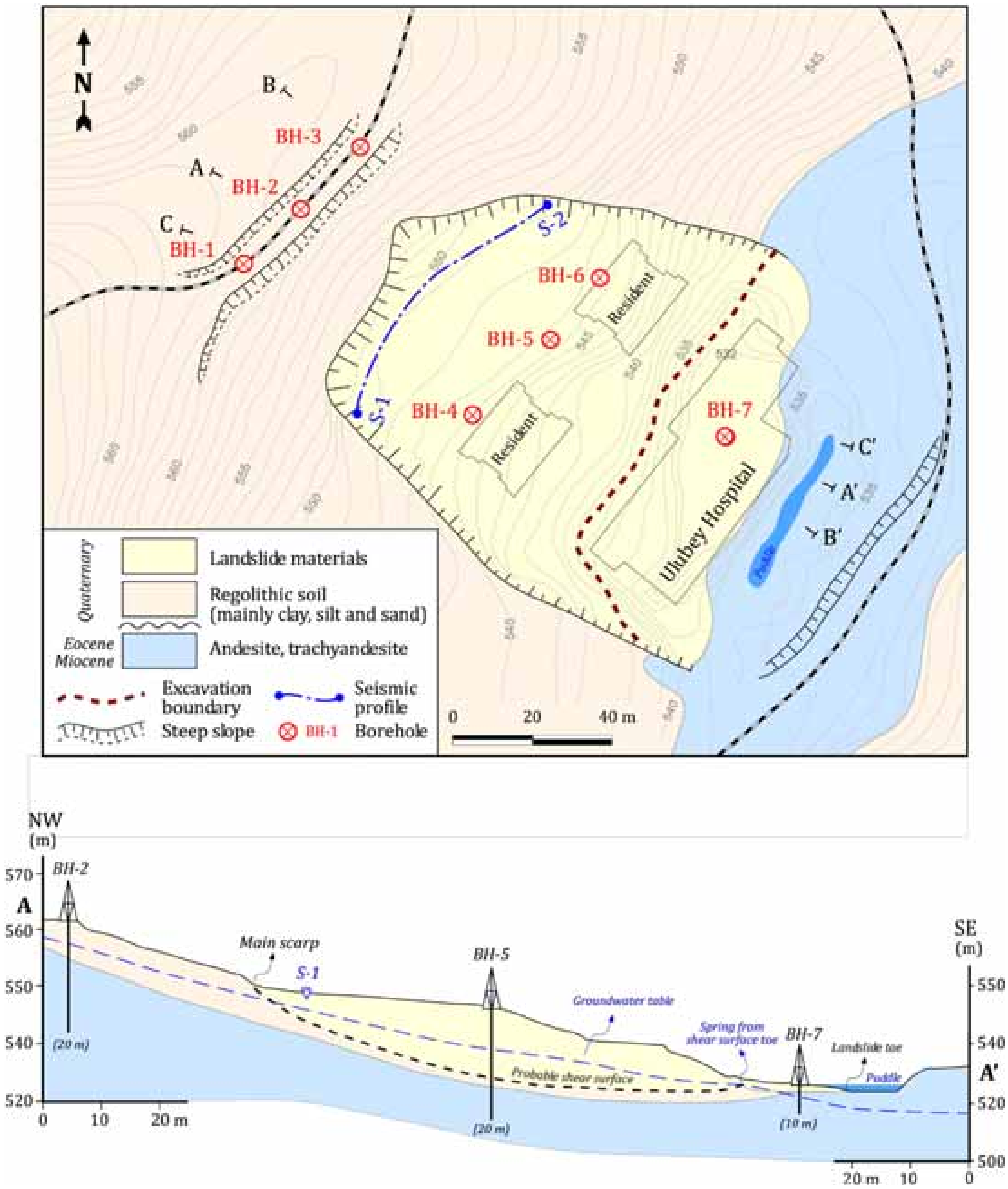

Figure 5. 1/2500 scaled engineering geological map and geological cross-section of the landslide area.

Table 2. *The index properties of the different geological materials in the landslide area.*

		Grain size distribution* (%)			Atterberg limits* (%)		Unit weight* (kN/m^3)		
Geological units	Thickness (m)	Gravel	Sand	Silt + clay	LL	PI	γ_n	γ_s	USCS
Residual soil (1)	8.5–10.0	4	7	89	61	30	19.9	22.1	CH
Landslide material (2)	> 14.0	6	7	87	56	32	19.4	19.8	CH

*Average values, *LL* liquid limit, *PL* plastic limit, γ_n natural unit weight, γ_s saturated unit weight.

the characteristics of the landslide materials, LE analysis, and numerical analysis were chosen as the most preferred stability methods in these studies. Landslide control and precaution techniques must be implemented after adequate investigation of site characterization and understanding of the causes of the landslide occurrence. If a slope has already moved, a control measure to stop the movement is

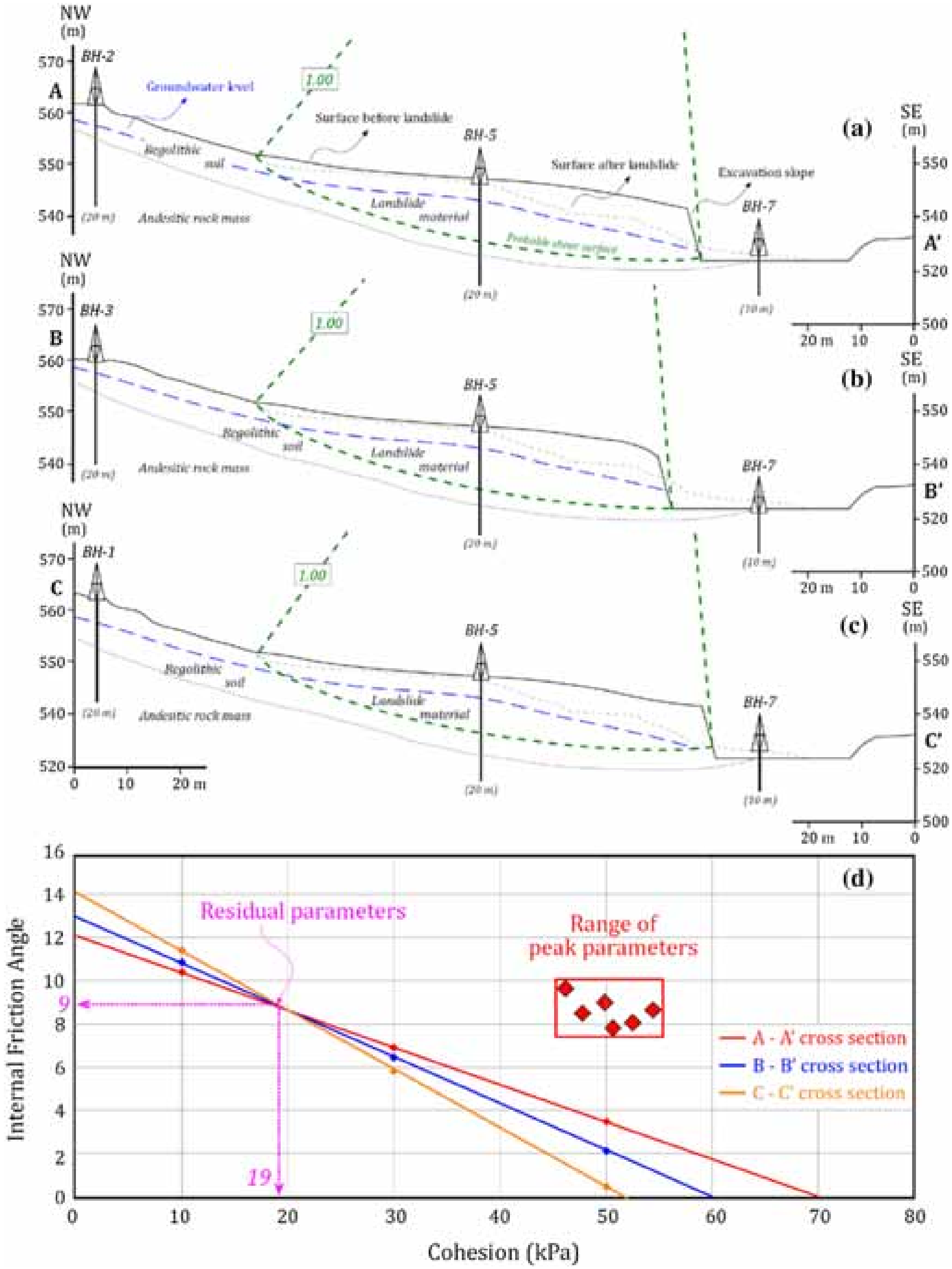

Figure 6. Back analysis carried out on three different geological cross-sections and the obtained residual parameters.

Table 3. *Strength parameters of different geological materials of the landslide area.*

Geotechnical units	Thickness (m)	Residual shear strength* c (kPa)	Residual shear strength* ϕ (°)	Peak shear strength** c (kPa)	Peak shear strength** ϕ (°)
Residual soil (1)	8.5–10.0	–	–	50	9
Landslide material (2)	> 15.5	19	9	–	–
Main rock (3)	–	Infinite strength			

c, cohesion; ϕ, internal friction angle.

*Obtained from back analysis on three different geological cross-sections.

**Obtained from consolidated-undrained triaxial compression tests.

Table 4. *Average values of elastic parameters of the geological materials obtained from seismic wave velocities.*

Geotechnical units	h (m)	V_p (m/s)	V_s (m/s)	v	E (MPa)
Residual soil (1)	8.5–10.0	814	384	0.35	790
Landslide material (2)	>15.5	411	184	0.37	150

h, thickness; V_p, P-wave velocity; V_s, S-wave velocity; v, Poisson's ratio; E, deformation modulus.

required to counteract the processes that started the slide.

In this study, the stability of Ulubey landslide was evaluated using different analysis techniques. The 2D-LE analysis was performed using Slide v9.0 (Rocscience Inc. 2017) software to calculate the factor of safety (FOS) for the failed slope after the application of support. In addition to 2D-LE analysis, the FEM-based 2D and 3D numerical analyses were used with the RS^2 v9.0 (Rocscience Inc. 2016a) and RS^3 v1.0 (Rocscience Inc. 2016b) softwares. In the analysis models, the soil and landslide materials were identified using table 2 and the bedrock was defined as the infinite strength material.

Slide is a PC program that is constantly used for the computation of FOS for soil slopes considering the LE approach. The different geometrical surfaces with different lithology can be demonstrated in both humble and multipart form using Slide. This program also permits the users to delineate and examine groundwater matter using a similar model as for the slope stability matter. In the LE method, which accounts for a significant portion of the slope stability analysis approaches, the Mohr–Coulomb failure criterion is grounded, a surface having a slide probability risk is selected, and the stress situation that would be the basis for the failure along this surface is examined. Then, the shear stress that saves and steadies the mass in the shear zone is determined. The considered stress values are compared, and FOS is then calculated (Alemdağ *et al.* 2015; Kaya 2017).

RS^2 is an influential and supple program in which the Shear Strength Reduction (SSR) technique is joined into the FEM and clarification modules. Because of the fast developments in computer science, the FEM–SSR (finite element method–shear strength reduction) method-based programs are being used more frequently than in the past. In the FEM–SSR technique, the elastic parameters and forces acting on the slope-forming material are considered to be dissimilar to the LE method. In the FEM–SSR method, the FOS is represented by the Strength Reduction Factor (SRF). The SSR technique includes systematic use of FEM to define a SRF value that transports a slope to the limit of failure. The shear strength parameters are scaled until the stability limit is reached. The SRF value is the ratio between the actual and the model strength at the stability limit. One of the benefits of the SSR method is that there is no requirement for the principal estimate at the description of the critical failure surface. However, the FOS value could not be calculated in the 3D analyzed model because v1.0 of the RS^3 program does not have a SSR option. Therefore, the results of the 2D-FEM and 3D-FEM models are compared based on the location of the stress concentration and total displacement outcome (Kaya *et al.* 2016b, 2018).

To control the long-term performance of the suggested support design under dynamic conditions, the horizontal seismic coefficient (k) of 0.22 g was applied to the model. The k value was calculated using the following formula recommended by Towhata (2008) for pseudo-static analysis. In this equation, the peak horizontal ground acceleration (PHA) was chosen as 0.25 g for the study area based on the iso-acceleration map prepared by Erdik *et al.* (2006), which took into account the active faults in Turkey.

$$k = PGA^{0.333}/3, \tag{5}$$

where k is the horizontal seismic coefficient (for PGA > 0.2 g) and PGA is the peak ground acceleration value.

In common field applications, such as retaining walls, drainage, geogrids in embankments, flattening, buttressing, anchors, and vegetation, are the most preferred landslide remediation methods. However, in this study, double row-bore piles (ϕ, 100 cm) that was supported with grouted tieback were chosen as the most realistic support design for the landslide because there is an inadequate area for slope flattening. The characteristics of the applied support elements are given in figure 7. The dynamic long-term efficiency of the supported slope was controlled in three steps.

In the first step, the 2D-LE analysis was performed taking into account the earthquake and building loads. The groundwater condition was defined using the data obtained from borehole investigations and the auto water table (Hu) option was used (table 1). The Mohr–Coulomb failure criterion (Mohr 1900) was applied, and the Janbu Method (Janbu 1973) was chosen for the combined slide-type analyses. The landslide material was characterized using the residual strengths that were obtained from the back analysis. However, the undisturbed zone was identified using the peak strengths that were obtained from laboratory studies. According to the 2D-LE analysis, the FOS values were determined as 1.017 for the dynamic long-term stability condition (figure 7a).

In the second step, the efficiency of the suggested remediation planning was controlled using the FEM-based 2D numerical analysis. In the analysis model, the ground surface was assumed to be a free boundary. However, the bottom and vertical boundaries of the model were closed to prevent displacements. In the mesh, 500 six-nodded triangular finite elements were used. Gravitational loading was applied in the numerical solutions. In this method, the Mohr–Coulomb failure criterion was taken into account, as in the applied 2D-LE analysis method. According to the 2D-FEM analysis, the FOS value was determined to be 1.13 for the dynamic long-term stability (figure 7b).

For the slope stability under an earthquake effect, a FOS value of 1.0–1.1 is necessary. In this study, a safety factor of 1.0 (LE condition) was taken into account, and it was useful for dynamic long-term stability (TGDH 2013). Therefore, it was concluded that the results acquired from the 2D-LE and 2D-FEM analyses are acceptable, and the reinforcement suggestions provided convincing outcomes. Furthermore, figures 7(a and b) show that the location of the critical failure surface

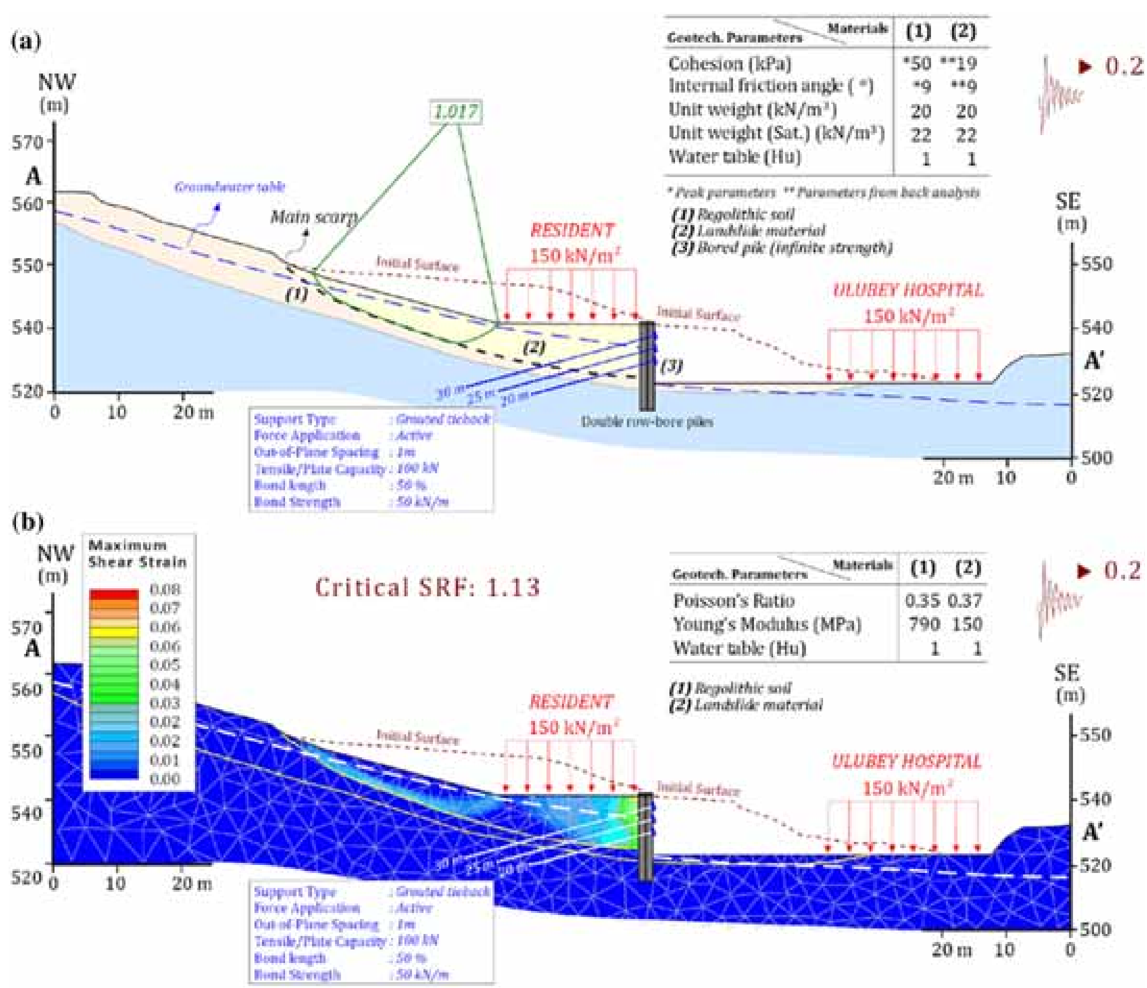

Figure 7. LE (**a**) and 2D-FEM–SSR (**b**) analyses evaluated for pseudo-static condition along the A–A′ cross-section.

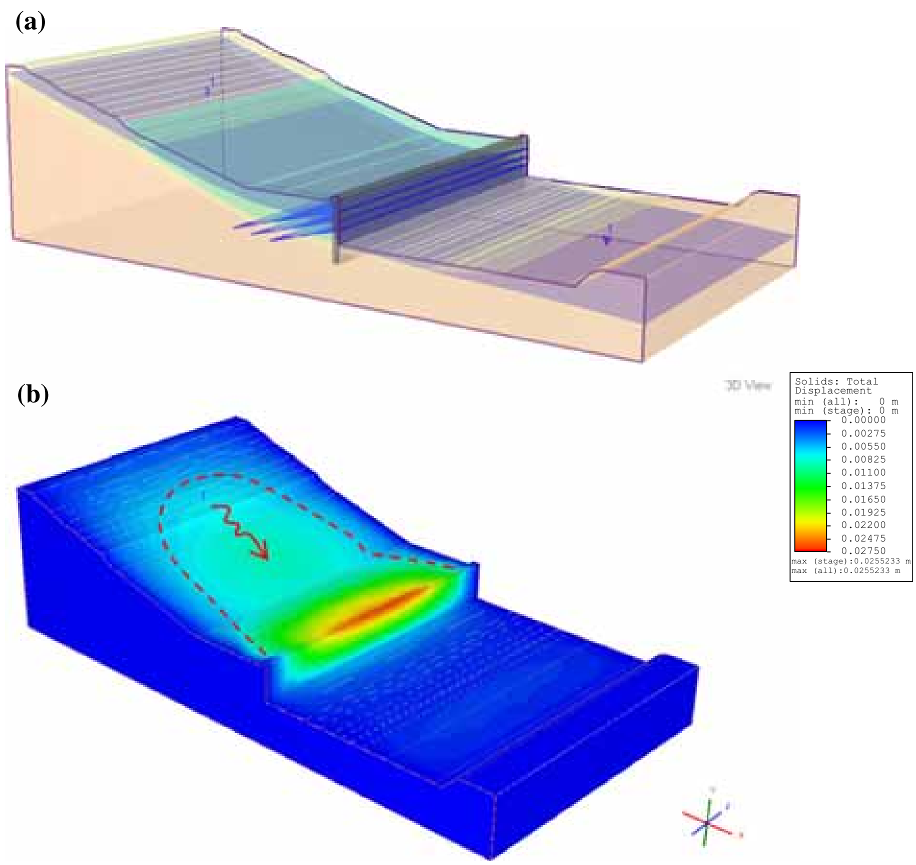

Figure 8. 3D-FEM–SSR analysis result evaluated for pseudo-static condition.

completely coincides with the results of the 2D-LE and 2D-FEM analyses.

In the third step, to check the 2D-LE and 2D-FEM stability analyses outcomes, the 3D-FEM model was used. The thickness of the model was chosen 80 m, which was the same as the actual excavation width that was applied in the field. Because there were no unevenness differences that would affect the 3D analysis results, the same surface topography was used as in the 2D model. The location of the stress concentrations obtained from the 3D model showed a close agreement with the 2D analysis results (figure 8). However, the total displacement values showed that the 3D model leads to a more conservative result than the 2D model. While the total displacement value obtained from 2D-FEM was 8.24 cm, the total displacement value for 3D-FEM model was 2.55 cm.

5. Conclusions

The main reasons of almost all landslides in NE Turkey are uncontrolled excavations and heavy rainfalls. In this study, the remediation design for landslides that occurred on a slope that was exposed to excessive rainfall after uncontrolled excavation during construction of a hospital in Ulubey (Ordu city, NE Turkey) was investigated.

To define the geotechnical characteristics of the study area, seven boreholes were drilled, seismic reflection measurement on a profile was conducted, and standard penetration tests were performed. In addition, undisturbed specimens were collected from the boreholes to define the index and strength characteristics. The peak shear strength parameters of the residual soils were determined from consolidated–undrained triaxial compression experiments. However, the residual shear strength parameters of the landslide material were determined using the back analysis technique on different geological cross-sections.

The following three different geotechnical units were identified as a result of the site investigations and laboratory tests: (1) sliding materials (failed or disturbed soil); (2) residual regolith (undisturbed soil); and (3) bedrocks (basic volcanic rocks). According to the SPT results, the failure surface was 1–2 m above the bedrock. A double row-bore piles (ϕ, 100 cm) supported with grouted tieback was chosen as the safest and most economical support design for the landslide for long-term stability. To control the efficiency of the suggested support design under dynamic conditions, 2D and 3D slope stability analyses were performed. This study showed that LE and FEM-based softwares are easy and advantageous simulation instruments to determine the landside characteristics.

Acknowledgements

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