

Stability assessment of high-steep cut slope debris on a landslide (Gumushane, NE Turkey)

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Received: 26 February 2015 / Accepted: 24 April 2015 / Published online: 8 May 2015
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Abstract The objective of this study was to examine the effects on the stability of slope debris of planned slope excavations around the vicinity of Gumushane Imam-Hatip High School in NE Turkey. To assess the geotechnical properties of slope debris along the determined cross-sections, four exploration boreholes with a total length of 100 m were drilled and undisturbed soil samples were collected. Slope debris is composed of two soil zones and its depth varies between 15 m and 21 m. It was determined from the experimental results that the soil class of the upper zone is CL (low plasticity clay) and CH (high plasticity clay) whereas the soil class of the lower zone is SC (clayey sand) and GC (clayey gravel). Peak cohesions of the upper and lower zones were 26.3 kPa and 10.2 kPa, and their peak internal friction angles were 16° and 24°, respectively. Limit equilibrium (LE) analyses for the slopes were evaluated using Slide v5.0 software for the pre-excavation, post-excavation, and supported cases. The results of LE analyses showed the existence of a landslide along the SW direction; the landslide would extend to cover the whole region if the excavation were to be performed without applying engineering measures. Finally, the precautions to be taken were determined and stability analyses for the supported case were evaluated. These analyses showed that the active lateral forces applied by the proposed retaining

walls to the cut slope should be 8325 and 3103 kN, respectively.

Keywords Landslide · Slope excavation · Stability · Slope debris · Limit equilibrium analysis

Introduction

Slope debris is usually comprised of loose earth-fill, collapse or landslide accumulation, alluvial slope sediments and completely decomposed rocks. Thus, debris is composed of many complex components and is a heterogeneous multiphase material that exhibits complex mechanical characteristics due to the various structural characteristics of its complex ingredients. The physical and mechanical properties of debris are major factors leading to different development patterns and deformation characteristics for debris slides. Stability of slope debris is an important area of research for engineering geology. Assessment of the stability of slope debris and controlling the failures of debris is costly not only for economic reasons but also due to the considerable time required to define the boundary conditions of a slide. Selecting a suitable slope stability analysis method is also of great significance (Xu 2011; Shaorui et al. 2013).

Due to the geologic, geomorphologic, seismologic, and climatic conditions in Turkey, it is possible to encounter various types of large landslides triggered by heavy rainfall (Gokceoglu et al. 2005; Ocakoglu et al. 2009) or earthquakes (Duman et al. 2005). In addition to rainfall and earthquakes, the effects of human activities have been a significant landslide trigger for many years. Population growth leads to additional settlement areas and requires new infrastructure in the form of roads, railroads, tunnels,

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and towers, etc. Each human activity that impacts the natural environment changes the natural morphology, stress conditions, and natural balance (Alemdag et al. 2013, 2014), resulting in the triggering of various landslide types. Many landslides throughout the world have been investigated and reported by researchers (e.g., Genc 1993; Fell et al. 1996; Hungr et al. 1999; Tarcan and Koca 2001; Ferreira 2007; Mantovani et al. 2010; Jaiswal et al. 2011) and many researchers (e.g., Fellenius 1936; Janbu 1954a, b, 1968, 1973; Bishop 1955; Bishop and Morgenstern 1960; Lowe and Karafiath 1959; Morgenstern and Price 1967; Spencer 1967; US Army Corps of Engineers 1970; Sarma 1973; Chung and Fabbri 1999; Tunusluoglu et al. 2007; Gurocak et al. 2008; Kahatadeniya et al. 2009; Nefeslioglu et al. 2010; Di Maio et al. 2010; Dag et al. 2011; Ferrari et al. 2011; Zheng 2012; Liu and Zhao 2013; Jeldes 2013; Gu et al. 2014; Alemdag et al. 2015) have conducted slope stability studies as part of the landslide literature. Different methods have been used in these studies, which were conducted at either local or regional scale according to the type of problem and the properties of the soil. Assessing these studies, it has emerged that limit equilibrium (LE) analysis is the method most preferred by researchers.

The city of Gumushane is settled on the Harsit River valley in NE Turkey (Fig. 1), and selection of new settlement areas constitutes an important urbanization problem. Excavation of slopes in the area to obtain new settlement areas is necessary, which in turn can sometimes trigger local slope instabilities. Today, new settlement areas are generally constructed on either side of the Trabzon-Erzurum highway, which crosses the Harsit valley, as well as on the surrounding steep slopes.

In this study, stability analyses were evaluated to determine how the existing landslide and the Gumushane Imam-Hatip High School buildings were affected by slope excavations, to understand the mechanisms of failure that can occur, and to determine the precautions needed to stabilize the area. The studies were executed in three phases as field, laboratory and office studies. In the field studies, exploration boreholes were drilled and undisturbed soil samples were taken from the slope debris. Engineering properties of the debris, such as unit weight, internal friction angle, and cohesion, were determined during the laboratory studies, after which soil classification was carried out. In the office studies, stability analyses were conducted using the data obtained from the field and laboratory studies.

The main aims of the study were:

1. to understand the failure mechanism of slope debris composed of elements having different origin and size and therefore different engineering properties and,
2. to emphasize the importance of detailed geotechnical studies in slope stability analyses.

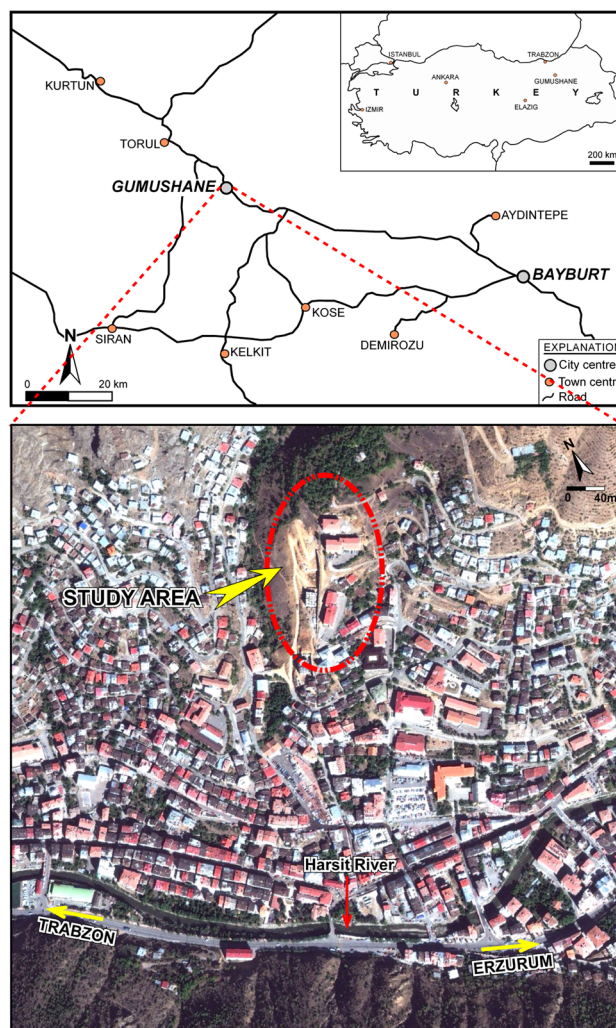


Fig. 1 Location map and satellite view of the study area

Geology of the study area

Based on structural and lithological evidence, the Eastern Pontides is subdivided into northern and southern zones and the study area is located in the southern zone. From bottom to top, the lithological units in the southern zone are represented by granitic, volcanic, and sedimentary rock associations, respectively (Fig. 2). The oldest lithological unit in and around the region is the Permo-Carboniferous aged Gumushane granitoid, which also constitutes the dominant lithology in the local study area (Topuz et al. 2010; Dokuz 2011). This unit is overlaid unconformably by the Liassic aged sedimentary rocks intercalated basalt, andesite, dacite, and their pyroclastites. The Early Jurassic-Late Cretaceous aged Berdiga limestone conformably overlies these sedimentary rocks. The Early Cretaceous aged sedimentary rocks give rise to the Berdiga limestone and all of these units are cut by the Paleocene-Early

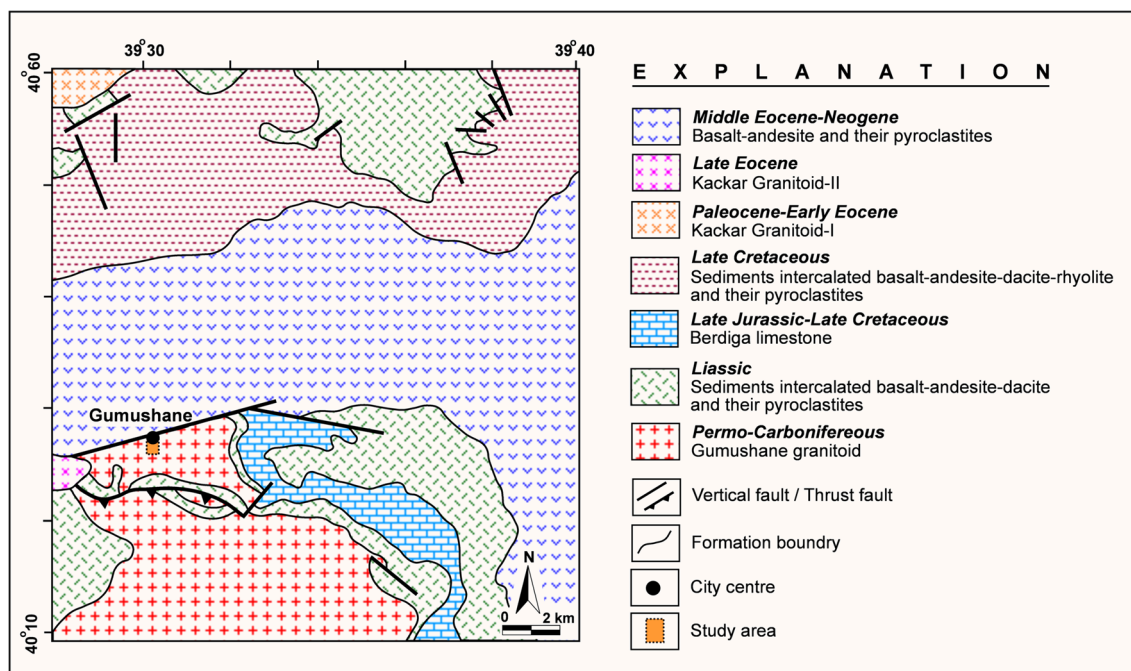


Fig. 2 Simplified geological map of the close vicinity of the study area (modified from Guven 1993)

Eocene and Late Eocene aged Kackar Granitoids I and II, respectively. During the Middle Eocene-Neogene period, volcanic activity was dominant in the region and is represented by basalt, andesite, and their pyroclastites. The Gumushane granitoid and Eocene volcanics are contacted by a vertical fault in the study area (Fig. 2).

The blocks and gravel outcrops in the slope where the study area resides are composed in general of granite and limestone and in part of basalt elements. Clay, silt and sand are more abundant than the other elements. This property plays an important role in sliding, and the shear resistance of the slope materials is decreased by increasing the water content.

Geotechnical investigations

Since the topography of the area where the Gumushane Imam-Hatip High School and the dormitory building reside is undulated, tilting of fences and tree bodies, stress cracks, openings, and compactions seen in the expansion joints among building blocks, and cracks occurring at the bases of buildings indicate the existence of a landslide (Fig. 3). Moreover, slope excavations with an approximate depth of 12 m are planned for new buildings to be built 8–10 m southeast of the existing structures (Fig. 4).

The A–A' and B–B' cross-sections (Figs. 4, 5) were defined as part of our geotechnical study to conduct stability analyses; these cross-sections pass through the school

and the dormitory building as well as the planned slope excavations. Four exploration boreholes having a total length of 100 m were drilled (Fig. 5) in order to determine the engineering properties of the slope debris existing in the study area and the change in its thickness, and to determine the ground water level. The data obtained from the exploration boreholes showed that the slope debris is composed of two different zones and its thickness ranges from 15 m to 21 m (Table 1; Fig. 5).

In order to determine the geotechnical properties of the slope debris in the study area, laboratory tests were carried out on undisturbed and disturbed soil samples in the Rock and Soil Mechanics Laboratory of Gumushane University. The index properties of the soil, shear strength parameters, and consistency limits were determined according to ASTM (2005, 2007, 2011, respectively) (Table 2). The samples were classified according to the USCS (unified soil classification system) (ASTM 2006) and it was determined that the upper zone of the slope debris is in CL (low plasticity clay) and CH (high plasticity clay) class soil whereas the lower zone is in SC (clayey sand) and GC (clayey gravel) class (Table 2). The shear box tests were carried out considering the worst conditions assuming the soil is saturated. Therefore, the tests were conducted as consolidated-undrained (CU). In the CU test, the sample was consolidated by keeping it in water and it was cut by not allowing drainage of water. By constructing normal stress (σ)–shear stress (τ) graphs, peak cohesion (c) and peak internal friction angle (ϕ) values of the soil were

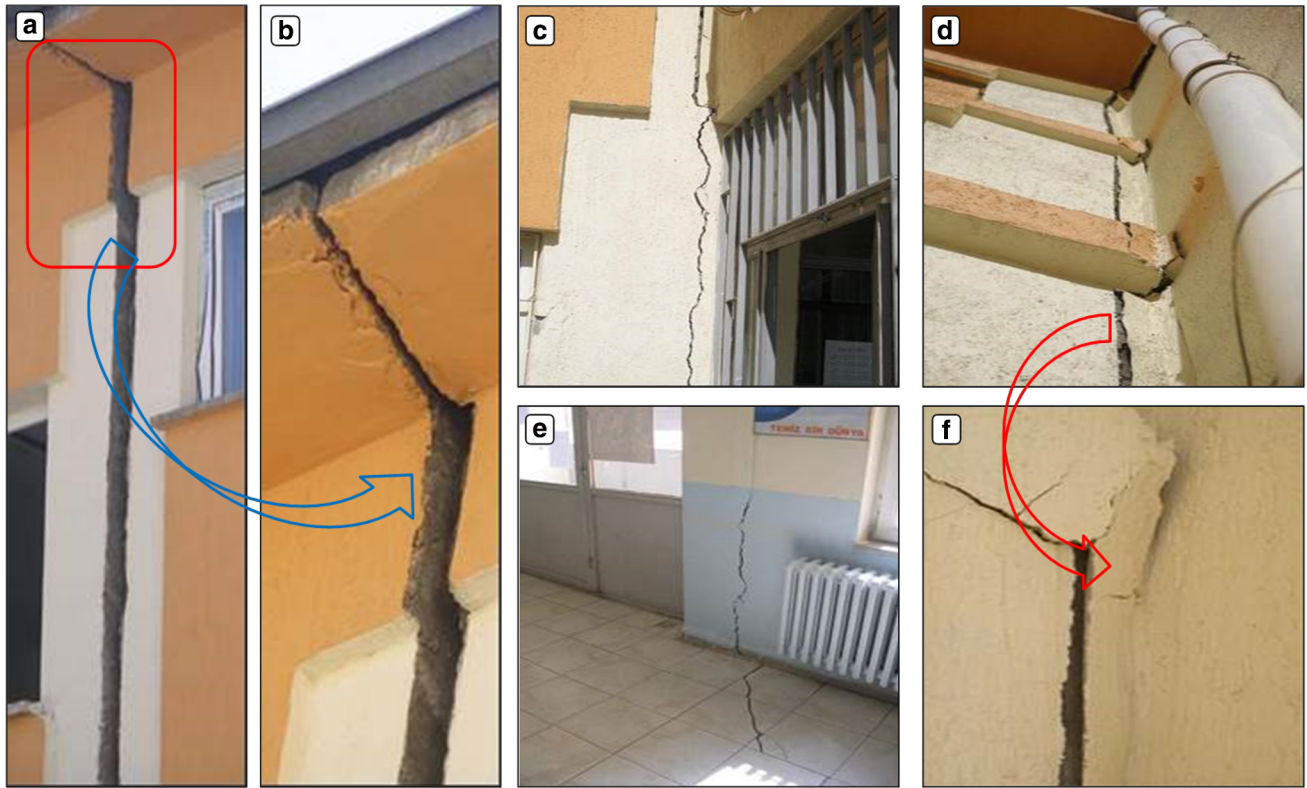
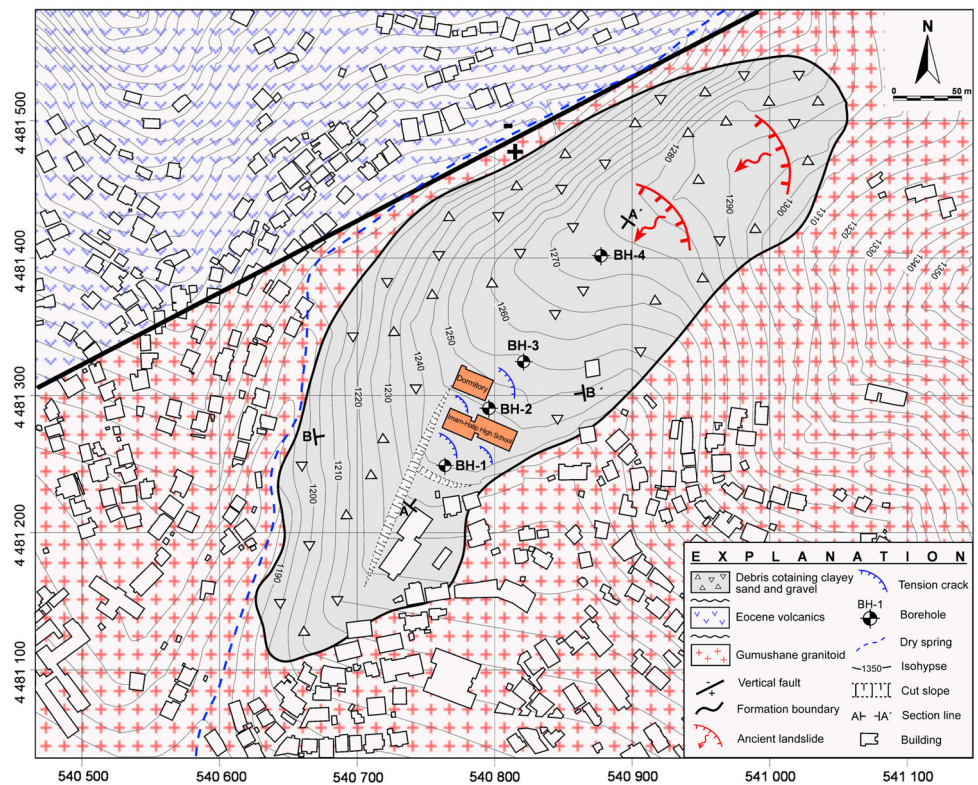


Fig. 3 Deformation of the dilatation joint (a, b), and cracks on the school wall (c, d, f) and floor (e) formed by mass movement

Fig. 4 Engineering geological map of the study area



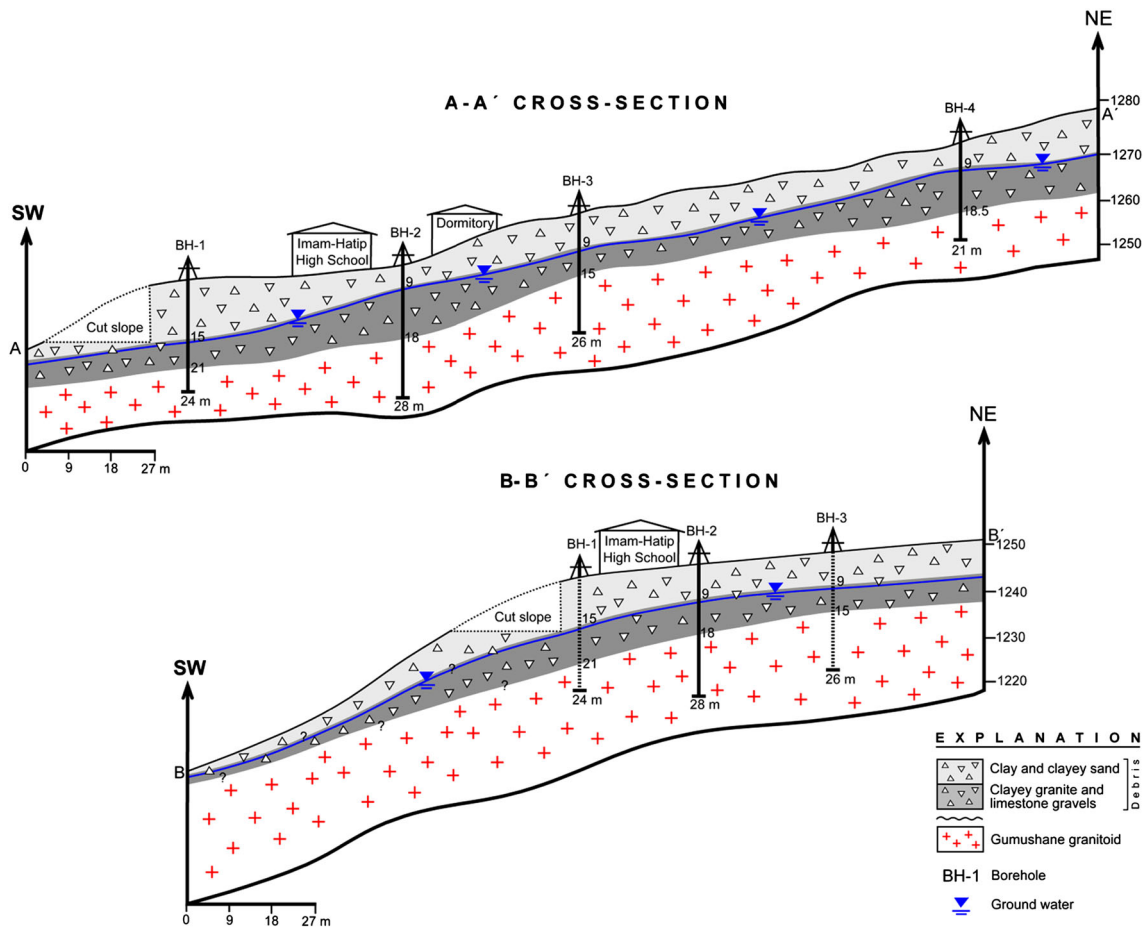


Fig. 5 Soil profiles of the study area along the A–A’ and B–B’ cross-sections

Table 1 Universal transverse mercator (UTM) 6° coordinate values of boreholes, thickness of debris and depth of the ground water in the study area

Borehole no.	Borehole depth (m)	Borehole elevation (m)	Debris thickness (m)	Ground water depth (m)	Y (right)	X (upward)
BH-1	24.0	1241	21.0	15.0	513,609	4,505,453
BH-2	28.0	1244	18.0	9.5	513,666	4,505,344
BH-3	26.0	1255	15.0	9.0	513,662	4,505,061
BH-4	21.0	1273	18.5	12.0	513,816	4,505,704

found. The test results determined the peak internal friction angles for CL-CH and SC-GC class soils as 16° and 24°, and peak cohesion values as 26.3 kPa and 10.2 kPa, respectively.

Modeling and stability analyses

In order to investigate the stability of the slope debris within the study area, LE analyses were evaluated on two cross-sections using Slide v5.0 (Rocscience 2003)

software. Slide v5.0 is a software that is used commonly for calculation of factor of safety (FOS) for slopes. Surfaces with different lithological properties and geometry can be modeled both in simple and complex form with this software. Furthermore, performing analyses taking into account both the dynamic loads due to seismicity and static loads owing to surrounding buildings makes the results more reliable and realistic.

The data obtained from the field and laboratory studies were used as input parameters in the stability analyses. In addition, the highest possible horizontal ground

Table 2 Distribution of soil classifications and geotechnical properties for the soil zones

Borehole no.	Soil zone	γ_n (kN m ⁻³)	γ_s (kN m ⁻³)	% Retained 4 no sieve	% Passed 200 no sieve	LL (%)	PL (%)	PI (%)	USCS	ϕ (°)	c (kPa)
BH-1	CL-CH	20.6–21.1 (20.9) ^a	20.9–21.4 (21.3) ^a	5.2	60.6	48	23	25	CL	19	24.1
	SC-GC	20.3–21.0 (20.6) ^a	21.1–21.8 (21.5) ^a	23.2	30.6	57	22	35	SC	23	9.6
BH-2	CL-CH	17.7–19.1 (18.9) ^a	19.3–19.8 (19.6) ^a	1.2	74.5	61	25	36	CH	14	28.4
	SC-GC	21.0–21.6 (21.3) ^a	22.2–22.6 (22.3) ^a	52.6	19.8	52	21	31	GC	–	–
BH-3	CL-CH	17.5–18.6 (18.3) ^a	18.6–19.4 (18.9) ^a	0.8	76.7	59	22	37	CH	15	26.9
	SC-GC	19.9–20.4 (20.1) ^a	20.3–20.9 (20.8) ^a	30.5	16.3	40	17	23	SC	25	10.7
BH-4	CL-CH	20.8–21.4 (21.1) ^a	21.4–21.8 (21.5) ^a	9.4	53.6	40	8	32	CL	17	25.7
	SC-GC	21.7–22.0 (21.8) ^a	22.3–22.8 (22.5) ^a	52.6	18.4	38	18	20	GC	–	–

CL Low plasticity clay, CH high plasticity clay, SC clayey sand, GC clayey gravel, γ_n natural unit weight, γ_s saturated unit weight, c peak cohesion, LL liquid limit, PL plastic limit, PI plasticity index, ϕ peak internal friction angle, USCS unified soil classification system

^a Average value

Table 3 Parameters used in the limit equilibrium (LE) analyses

Material	γ_s (kN m ⁻³)	ϕ (°)	c (kPa)	a (g)
CL-CH	20.3	16	26.3	0.2
SC-GC	21.8	24	10.2	0.2

acceleration (a) was taken as 0.2 g considering the static loads from the surrounding buildings and seismic activity of the region.

The Mohr-Coulomb failure criterion was used in the analyses, and circular and combined slide types were chosen. The Janbu method (Janbu 1973) was used in the LE analyses since the length of the slope debris is greater than its thickness and it has a combined slide hazard.

The LE analyses were conducted for each cross-section in three phases. In the first stage, the slope stability was investigated for the pre-excavation, and then the post-excavation condition was examined. In the final phase, the analyses were performed including the reinforcements proposed in this study and the FOS values were found. The parameters used in the analyses are given in Table 3.

LE analyses for pre-excavation case

The LE analyses were evaluated along the A–A' and B–B' cross-sections (Figs. 4, 5) for the pre-excavation in the study area. The FOS values for the slope were found to be 0.966 for the A–A' and 0.616 for the B–B' cross-sections (Fig. 6). The FOS values found are the lowest values calculated for the worst soil conditions and indicate the existence of a landslide along both cross-sections. Also, the cracks and openings, especially in the high-school building, clearly show the existence of movement (Fig. 3). The LE analyses performed along the A–A' and B–B' cross-

sections revealed that failure has occurred in both the CL-CH and SC-GC soils.

LE analyses for the post-excavation case

The LE analyses for the post-excavation case were evaluated along the A–A' and B–B' cross-sections in order to determine how the excavation planned for the region would affect the high school and dormitory buildings and to determine the FOS values for post-excavation (Fig. 7). It was determined by the analyses that the FOS values are 0.579 and 0.517 and, if slope excavation was carried out, landslides would occur along both cross-sections. The results of LE analyses revealed that reinforcement would definitely be needed to provide slope stability.

Reinforcement of cut slopes and LE analyses for the supported case

According to the LE analyses evaluated along the A–A' and B–B' cross-sections, it is considered that any slope excavation made in the area would make the landslide more active, thereby causing an extension in the whole region. Reinforcement is needed in order to stabilize the region. Because there is insufficient area for slope flattening, a retaining wall was selected as the most reliable and economic reinforcement method for the slopes planned to be excavated in this study. The lateral active forces needed to be applied to the slopes by the retaining walls to make the region stable were determined from the LE analyses (Fig. 8). In these analyses, lateral active forces that make the FOS value 1.50 were taken into account. As a result, if the proposed retaining wall is applied, lateral active forces of 8325 and 3103 kN to the slopes for the A–A' and B–B' cross-sections,

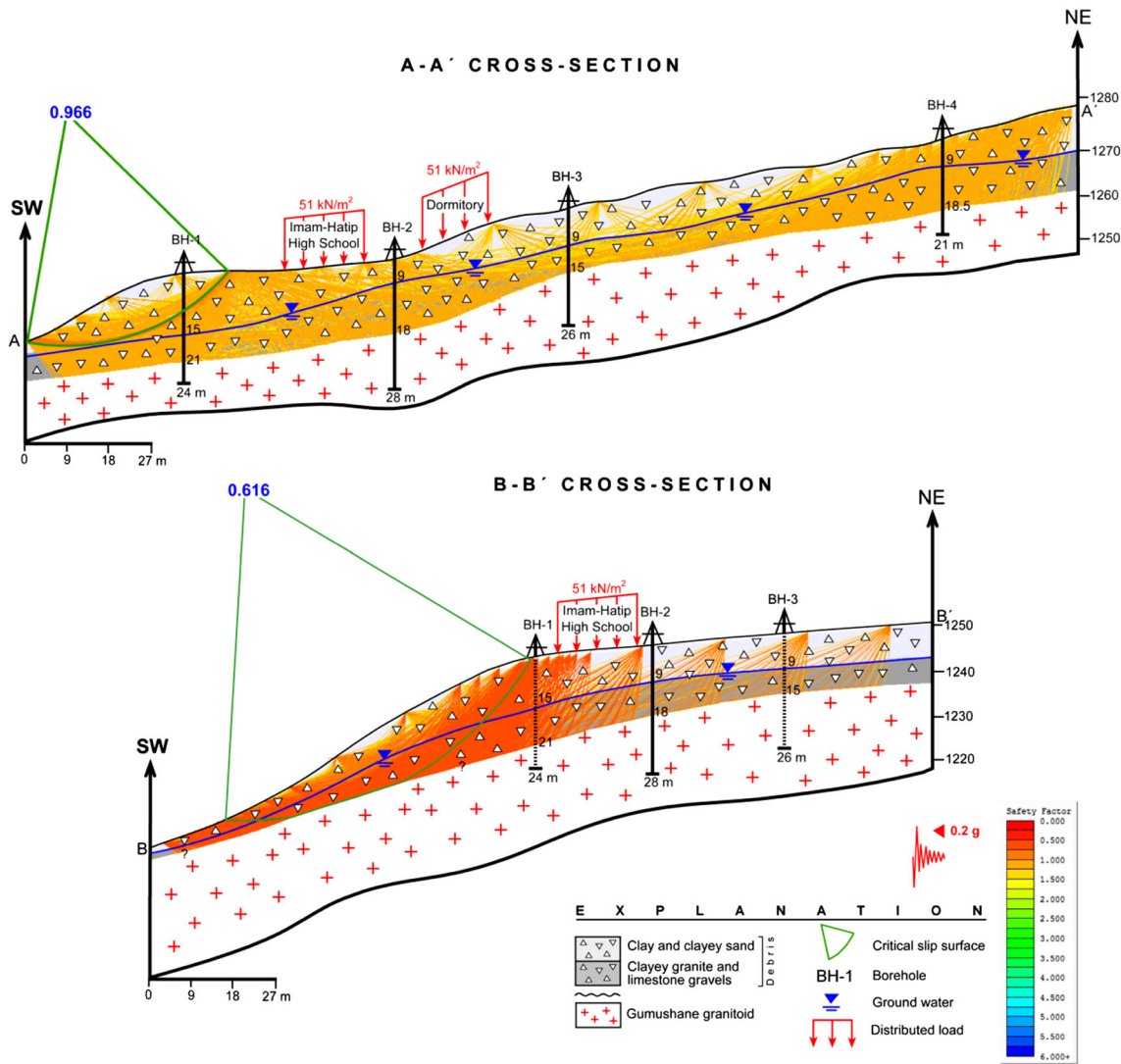


Fig. 6 Limit equilibrium (LE) analyses evaluated before slope excavations for the A–A’ and B–B’ cross-sections

respectively, under a seismic load of 0.2 g would result in FOS values of 1.50.

Conclusions

Landslides occurring in slope debris composed of elements of different size and origin usually have a complex development mechanism. Therefore, in engineering projects where slope debris is available, determination of the stability of the slope debris and the precautions to be taken are important to ensure the safety of the project and to limit costs. In such geotechnical units, engineers perform stability analyses using computer software that

applies different analysis methods over a short time frame. LE analysis is one of the most preferred methods with which to expose the development mechanism of landslides and to realistically determine the precautions to be taken.

In this study, LE analyses were performed via Slide v5.0 software to determine the effect on the stability of the region of the existing landslide and planned slope excavations close to the Gumushane Imam-Hatip High School and dormitory building where the main soil is slope debris, and to determine the precautions needing to be taken. The LE analyses were evaluated for the pre-excavation, post-excavation and supported cases by taking the worst conditions and seismicity into account.

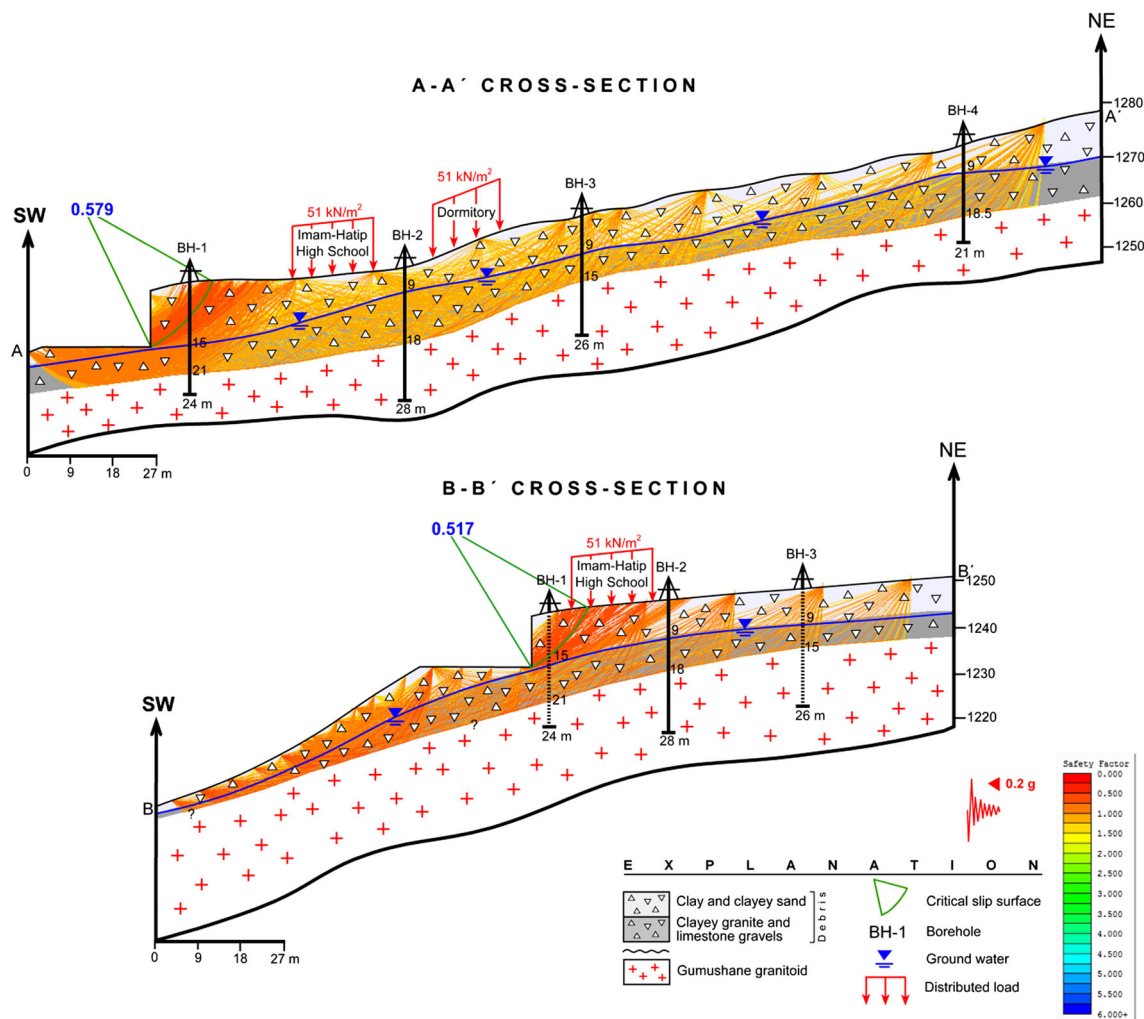


Fig. 7 LE analyses evaluated after slope excavations for the A–A' and B–B' cross-sections

The results of the LE analyses evaluated for the pre-excitation determined FOS values of 0.966 for A–A' and 0.616 for B–B' cross-sections and indicated the existence of a landslide along both cross-sections.

According to the results of LE analyses evaluated for the post-excitation, the FOS values reduced to 0.579 and 0.517. These FOS values indicated that the landslide would be spread over a wider area and cover the whole area. Some precautions are therefore needed to prevent the effects of landslide before excavation.

Based on the results of this study, we propose retaining walls as the most economic reinforcement method for the study area to prevent further landslide. The lateral active forces that need to be applied to the slopes to achieve FOS values of 1.50 were determined.

The analyses evaluated for the supported case showed that the lateral active forces along the A–A' and B–B' cross-sections should be 8325 and 3103 kN, respectively, to make the region stable and obtain a FOS value of 1.50. It was determined that the support type proposed in this study would stabilize the existing movement and prevent future landslides in the region and make the region stable.

This study shows that mass movements are the major geological hazard to high-steep excavation in mountainous areas, especially in landslide zones. Remedial measures have to be applied simultaneously with slope excavation to prevent potential failure hazard from becoming reality, thus reducing the need for, and cost of, intensive supporting.

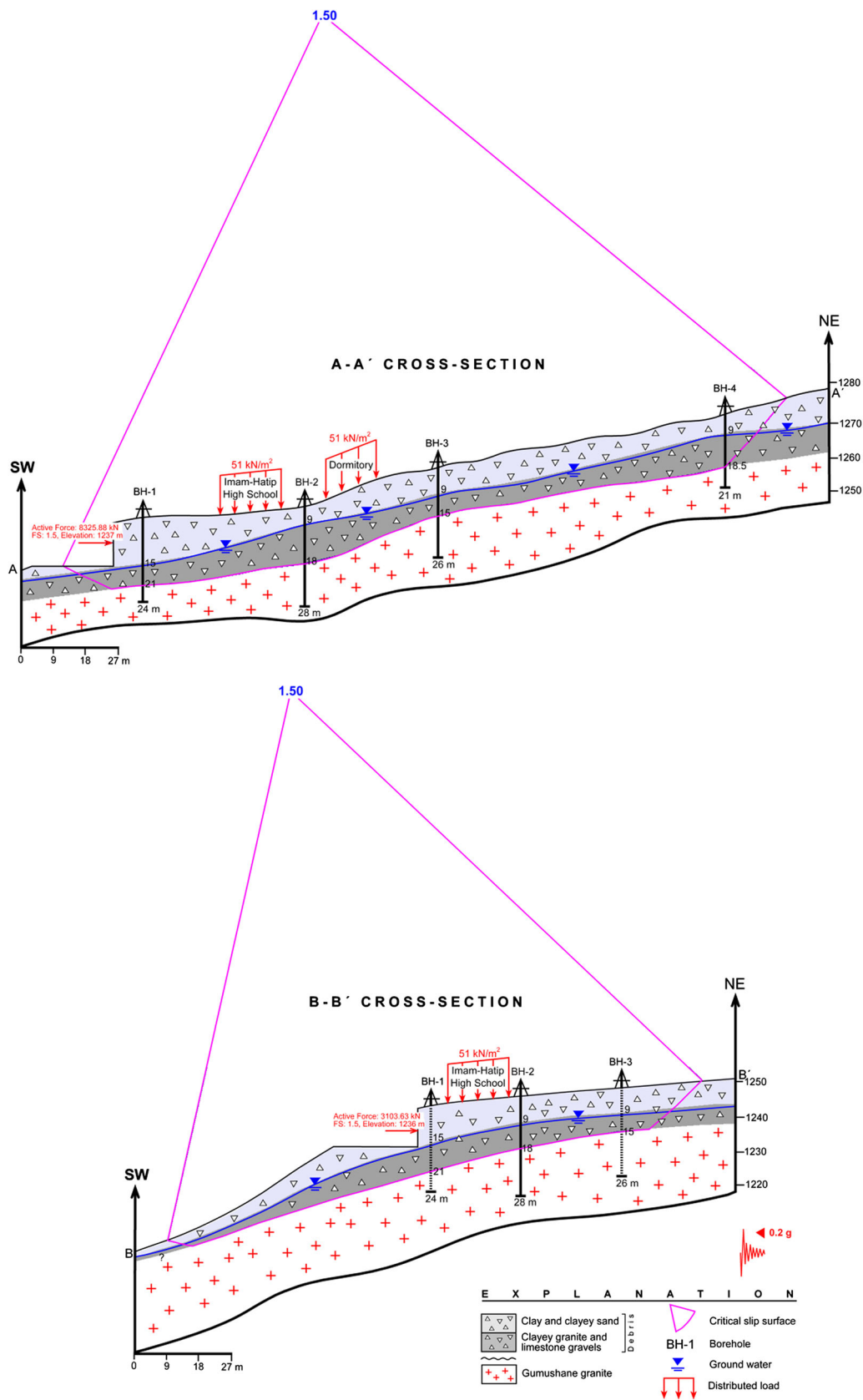


Fig. 8 LE analyses evaluated after supported cases for the A-A' and B-B' cross-sections

Acknowledgments The authors would like to express their sincerest gratitude to the editor and reviewers for their excellent comments. Thanks also to Ideo Interpretership Co. Ltd. for language editing of the text.

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