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ORIGINAL ARTICLE

Investigation of the failure mechanism and stabilization of a landslide in weathered tuffite, Giresun, northeastern Turkey

Özgür Avşar · Haluk Akgün · Mustafa Kerem Koçkar

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Abstract This study investigates the causes and failure mechanism of the Aksu landslide that occurred during the construction of the Giresun–Espiye road between KM: 1 + 030–1 + 170 in northern Turkey and recommends proper stabilization techniques. For the purpose of investigating the causes and mechanism of this slope failure, engineering geological mapping, geotechnical investigation and rock mass characterization were performed. From top to bottom, weathered tuffite, tuffite, flysch, and dacitic tuffite were the major units in the study area. The disturbance of the slope by the excavations performed at the toe of the slope (i.e., due to the foundation excavation for the Tünel restaurant building and for the road cut) led to a "translational slide". The "translational slide" occurred in completely weathered tuffite due to the disturbance of the stability of the slope by the excavations performed at the toe of the slope, particularly for the foundation excavation of the Tünel restaurant building and for the road cut along the Giresun–Espiye road. The rise in the groundwater level was also another important factor that has contributed to the occurrence of the landslide. After establishing the geometry of the landslide in detail, the shear strength parameters of the failure surface were determined by the back analysis method. Sensitivity analyses were performed and landslide failure mechanisms were modeled to quantify the contributing factors that have caused the formation of the Aksu landslide. The influence of an earthquake was investigated through pseudostatic slope stability analysis. Toe buttressing, ground water drainage, and surface water drainage alternatives were considered for stabilizing the slope.

Keywords Landslide · Failure mechanism · Inclinometer · Weathered tuffite · Back analysis · Sensitivity analysis · Slope stabilization · Pseudostatic slope stability analysis

Introduction

In Turkey, a large proportion of the landslides take place in the Black Sea region, especially in the eastern Black Sea region. Rainy climate and steep topography of the region make it susceptible to mass movements. Between 1929 and 1990 huge mass movements took place in the Black Sea region which resulted in tremendous economic losses and losses of lives. For example, on 21 June 1990, a landslide occurred due to heavy rain in Maçka/Çatak (Trabzon) and 65 people lost their lives. In addition, an economic loss of several million dollars was recorded (Öztürk 2002).

Landslides should be considered as serious as any other natural hazard in Turkey. Between the years 1950 and 2005, 12,791 events have been reported within 4,161 residential settlement units due to landslides. 6,347 buildings were destroyed partly or completely and then abandoned, which adversely affected 63,969 people (Gökçe et al. 2005). Figure 1 presents a landslide density map of Turkey which is based on observed landslide phenomena in residential areas that occurred between 1950 and 2005 (Gökçe

Ö. Avşar
Department of Geological Engineering, Muğla Sıtkı Koçman University, Muğla, Turkey

H. Akgün (✉)
Geotechnology Unit, Department of Geological Engineering, Faculty of Engineering, Middle East Technical University, Ankara, Turkey
e-mail: hakgun@metu.edu.tr

M. K. Koçkar
Earthquake Engineering Implementation and Research Center, Gazi University, Ankara, Turkey

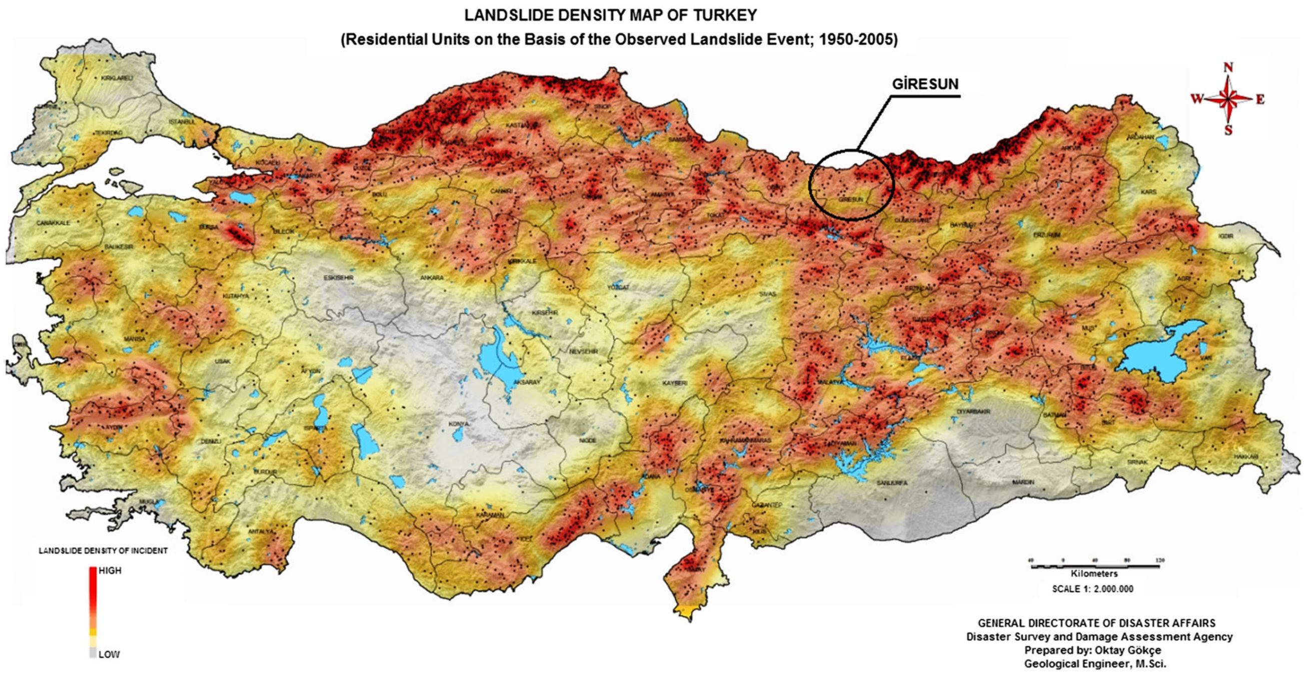

Fig. 1 Landslide density map of Turkey which is based on observed landslide phenomena in residential areas that occurred between 1950 and 2005 (Gökçe et al. 2005)

et al. 2005). The map indicates that most of the landslide-occurring provinces are in the Black Sea region. It should be noted that Giresun, which is highly susceptible to landsliding, is located in the Black Sea region.

The Giresun–Espiye road, which is a part of the Black Sea Coastal Highway Project of the General Directorate of Highways, is designed to connect Giresun and Espiye (Fig. 2). Figure 2 gives the location of the Aksu landslide that is located 5 km away from the Giresun city center (Yüksel Project International, Inc. 2003) and is accessible through the 010 State road along the shoreline. The length of the road is 25 km and during its construction, some amount of rock and soil was excavated from the south side of the road between KM: 1 + 030–1 + 170 with the purpose of increasing the width of the existing road from 13.5 to 27 m. The removal of the earth material disturbed the stability of the natural slope in the southwestern side of the road and led to the triggering of the Aksu landslide which was a 120-m long and 90-m wide mass movement (Figs. 3, 4). The purpose of this study is to investigate the causes and the mechanism of this slope failure and to suggest proper stabilization techniques (Avşar 2004; Yüksel Project International Inc. 2003).

Geomorphology

The geomorphology of the study area is characterized by a steep topography. Generally, E–W trending mountains are cut by N–S trending running water systems. The topography is very rough in general with several peaks around 3,000 m. In the eastern Pontides, rock exposures are very limited and are observed mainly along the road cuts and deep valleys. The other areas are usually covered with soil and vegetation. The soil cover varies in thickness from a few tens of centimeters to a few meters, where the thickest profiles are observed at the toes of the slopes as well as in rather flat-lying terrains that have formed as a result of paleo-landslides. Relief and altitude are the main factors affecting the development of the soil profile in the area. Due to high rainfall, the region is densely vegetated with a variety of species (Akçay et al. 1998).

General geology

Introduction

Eastern Turkey, which is situated in the Alpine belt, is subdivided into three tectonic units, namely Pontides (Pontian magmatic arc), Anatolides (mainly ophiolitic and metamorphic complexes), and Taurides (carbonate platform) from north to south (Ketin 1966; Bektaş and Gedik 1988; Şengör and Yılmaz 1981). The study area is located towards the east of the Pontides (Eastern Pontides, Fig. 5), which has the overall features of a volcanic arc and contains volcanics ranging in age from Liassic to recent (Akçay et al. 1998; Aslaner et al. 1995).

Fig. 2 Location map of the study area

Stratigraphy

Six rock groups, namely Late Carboniferous-Early Permian, Jurassic-Early Cretaceous, Late Cretaceous-Early Eocene, Middle-Late Eocene, Oligocene–Miocene, and Pliocene and Plio-Quaternary are differentiated on the metamorphic basement of the Eastern Pontides and these units are bounded with unconformities (Korkmaz et al. 1995). The complementary paper of Korkmaz et al. (1995) on the stratigraphy of Eastern Pontides constitutes the basis of the stratigraphy of the project area.

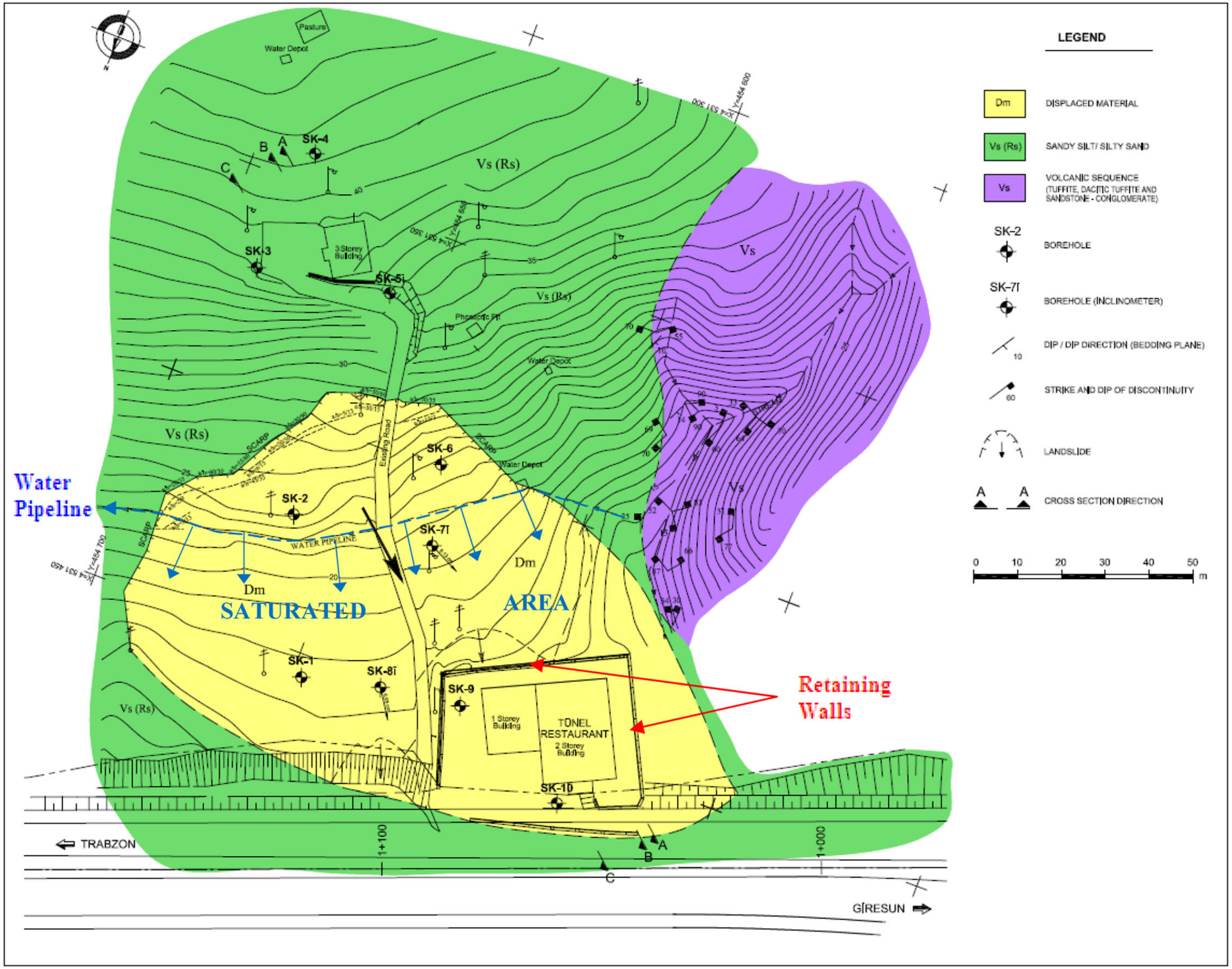

Fig. 3 Engineering geological map of the study area (scale: 1/1,000) which includes the outline of the Aksu landslide that occurred between KM: 1 + 030–1 + 170 of the Giresun–Espiye road. The map shows the outline of the excavated area for placing the foundation of the Tünel restaurant and the trend of the water pipeline (*thick blue dashed line*). Note that the soils between the pipeline and the Giresun–Espiye road were most probably saturated due to the leakage of water from the pipeline

Fig. 4 Photographs of the Aksu landslide area. *Top photos* the landslide area looking SSE. *Bottom photos* the scarp of the landslide looking NE. Due to dense vegetation, the entire outline of the landslide cannot be observed in the photos

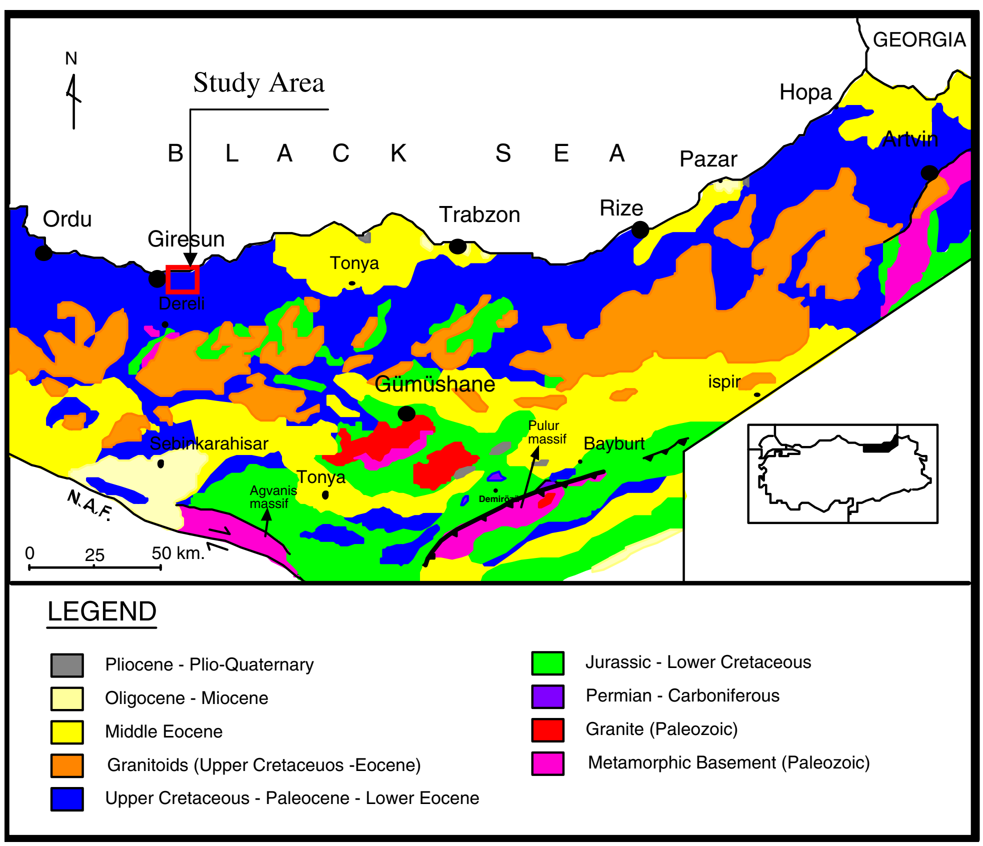

Fig. 5 Geological map of the eastern Pontides (generalized from Korkmaz et al. 1995)

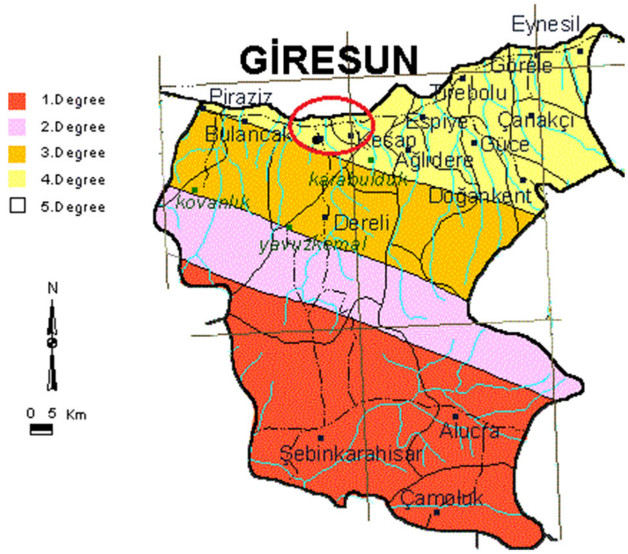

Fig. 6 Earthquake zonation map of the Giresun area, Turkey (modified from AFAD 2012)

Among these rock groups, only the Late Cretaceous-Early Eocene aged rock unit, which is the most widespread rock unit in the Eastern Pontides outcrops in the study area (Fig. 5). This unit is called the Cankurtaran group since it is outcropping continuously at the Cankurtaran Mountain (Hopa). It is unconformably underlain by Lower Cretaceous rocks of the Gümüşhane group. In this unit, Campanian and Maastrichtian aged rocks are composed of basic and partly acidic volcanic rocks and sedimentary rocks. Starting with Upper Maastrichtian, the formation is characterized by an alternation of red-colored marl and limestone, up to the end of Paleocene. Lower Eocene rocks are represented by gray marl and sandstone (Korkmaz et al. 1995).

Seismicity

According to the "Turkish earthquake zonation map" prepared by the Turkish Ministry of Public Works and Settlement, General Directorate of Disaster Affairs (AFAD 2012; Fig. 6), the study area is situated in a 4th degree earthquake zone. An investigation of the earthquakes which took place around Giresun during the occurrence of the Aksu landslide revealed that no significant earthquake has occurred around Giresun during that period of time. Hence, it may be inferred that the Aksu landslide was not triggered by an earthquake.

Engineering geological mapping and geotechnical investigations

A 1/1,000 scale engineering geological map of the landslide area was prepared based on previous studies (Göksu 1974) and actual field observations (Avşar 2004; Fig. 3). The engineering geological map contains the boundary of the main geological units, the boundary of the landslide, the location of the boreholes, and the man-made structures in the study area. Dip/dip direction of discontinuities were measured in the field and plotted on the engineering geological map. The main lithological units of the area are dacitic tuffite, tuffite, flysch, and weathered tuffite. An engineering geological characterization of these rock units is given in the next section.

Drilling, standard penetration test (SPT), and inclinometer measurements have been performed as part of the geotechnical investigation program since sufficient geological/geotechnical investigation and careful interpretation of ground conditions are mandatory prior to cut slope design and for locating the slip surface in a correct fashion (e.g., Koçkar and Akgün 2003a; Sun et al. 2013). Borings with a total depth of 137 m have been performed at ten locations in order to disclose the type, thickness, contact relations, and geological–geotechnical properties of the units that are present in the study area. Inclinometer casings were installed in boreholes SK-5i, SK-7i, and SK-8i, which were selected to be on a line cutting the landslide to make inclinometer measurements for determining the geometry of the slip surface and for measuring the amount, direction, and velocity of the movement. Figure 3 gives the locations of the boreholes with inclinometers. SK-5i was located just in front of the three-storey building to investigate the existence of any movement which might have affected the building deleteriously. In order to determine the location of the failure surface as well as to find the velocity and direction of the mass movement, inclinometer measurements have been taken periodically. These measurements were recorded and analyzed to plot depth versus cumulative displacement graphs. Figure 7 presents an example for inclinometer measurements in borehole SK-7i.

A summary of the inclinometer measurements is given in Table 1. As can be observed from Table 1, no significant movement was recorded in SK-5i whereas SK-7i and SK-8i indicated movement at depths of 5.31 and 4.85 m, respectively. A total of 4 proper inclinometer measurements were taken for a period of 24 days after which the measurements were terminated because the displacement of the ground caused a shift of the casing in such a way that the probe could not be inserted back into the boreholes.

Engineering geological characterization of the rock masses

All of the data gathered from the geotechnical investigations were utilized to obtain three cross-sections

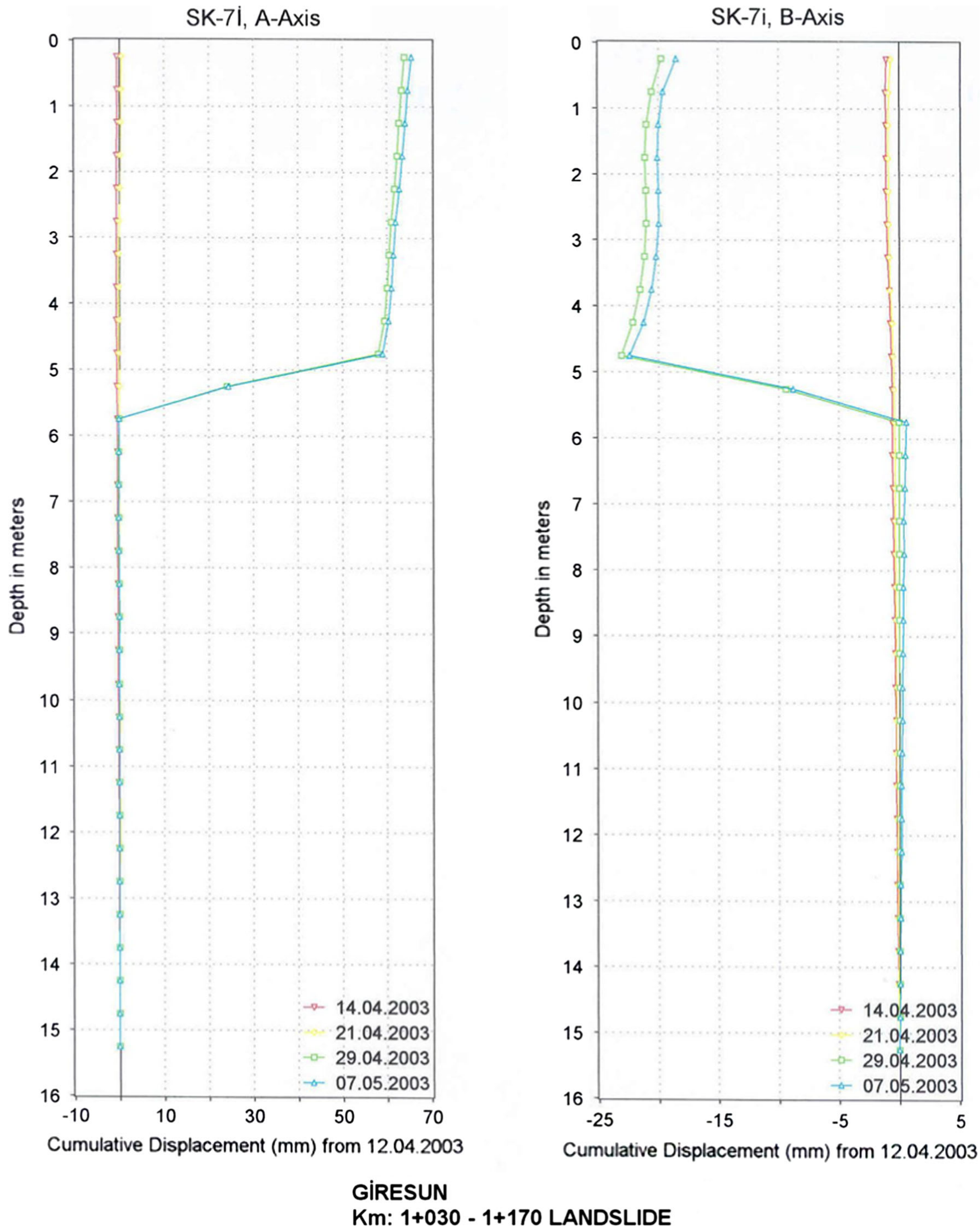

Fig. 7 Depth versus cumulative displacement graph of borehole SK-7i which contained an inclinometer casing

(Fig. 8) which lay in four major units that were differentiated in the field on the basis of their lithological and engineering geological properties, namely from top to bottom: weathered tuffite, flysch, tuffite, and dacitic tuffite which belong to the Late Cretaceous-Early Eocene volcano sedimentary and sedimentary rocks of the Eastern Pontides.

Engineering geological characterization of each unit that is present in the project area is presented below.

Dacitic tuffite

Dacitic tuffite is the oldest unit among the rock units present in the study area and lies at the bottom of all

Table 1 A summary of the inclinometer measurements performed to locate the depth of the landslide slip surface

Borehole no.	Depth (m)	Movement direction	Velocity (mm/day)
SK-5i	N/A	N/A	N/A
SK-7i	5.31	295°N	2.52
SK-8i	4.85	326°N	2.55

units (Fig. 8). This lithology outcrops towards the west of the study area (Fig. 3). The lithology is green-bluish green, moderately weathered, and moderately strong. The joint walls are rough and undulated (ISRM 1981). The discontinuities are generally clean, but calcite mineralization and FeO–MnO coating are observed occasionally. Apertures are 0.5–2.0 mm wide. According to ISRM (1981), the aperture is classified as "open". Average spacing of joints is about 300 mm. Therefore, the spacing can be classified as "moderate" according to ISRM (1981).

Flysch

This unit is underlain by dacitic tuffite and overlain by tuffite (Fig. 8). Flysch is generally composed of an alternation of conglomerate and sandstone and it varies in thickness from 2 to 4.5 m. Marl intercalations are occasionally observed. The sandstone component is yellowish/greenish brown to dark brown, completely weathered, and possesses a very weak strength. Conglomerate, the other component of the unit, is characterized by brown to gray color. Conglomerate contains rounded, strong rock fragments with a maximum diameter of 50–70 mm within a completely weathered and loosely cemented matrix. Due to the intense weathering, discontinuities can hardly be observed. Dip amounts of the discontinuities observed from the sandstone cores range from 0° to 45°. The joint walls are MnO–FeO coated and can generally be classified as "rough" according to ISRM (1981).

Standard penetration tests (SPT) were performed along the highly weathered portions of the flysch unit. The statistics of the SPT and laboratory index tests is presented in Table 2.

Referring to the groundwater level measurements obtained in boreholes SK-2, SK-3, SK-4, SK-5, SK-6, SK-9, and SK-10, it was observed that the groundwater table in the study area lay within the flysch unit. However, groundwater levels measured in boreholes SK-1, SK-7i and SK-8i revealed a rise in the groundwater table. The reason for this anomaly was most probably due to the ENE–WSW trending broken water pipeline in the study area whose footprint area is displayed in Fig. 3. Infiltration of water from this broken pipeline had most probably saturated the underlying units and caused an increase in the groundwater level. Figure 8 illustrates the anomaly in the groundwater level.

Tuffite

Tuffite is underlain by flysch and overlain by weathered tuffite in the study area. The thickness of this unit, as traced from boreholes SK-3, SK-4, and SK-5, ranges between 2.5 and 3.0 m. It is brownish green–brown, moderately to highly weathered, has weak to very weak strength, and contains agglomeratic levels. Because of the weathered nature of the unit, discontinuities could hardly be observed in the outcrops and in the core samples.

In addition to the core samples, disturbed samples were also taken from the highly weathered portions of the tuffite unit by means of SPT. The statistics of the SPT and laboratory index tests are given in Table 3. The groundwater table lies below the tuffite unit in the study area (Fig. 8).

Weathered tuffite

This unit is the overlying unit of the study area (Fig. 8) and has a thickness ranging from approximately 7.0 to 9.0 m. The weathered tuffite unit is composed of an alternation of sandy silt and silty/gravely silty sand layers, which both are formed by the weathering of tuffite. The slope failure took place along the boundary between the weathered tuffite and flysch.

The groundwater table level was below the weathered tuffite in the study area prior to the unstable condition of the Aksu landslide. However, the effect of the dissipation of water from the pipeline, which is discussed above, has caused an increase in the groundwater table level at the north of the pipeline (i.e., as revealed by the groundwater level measurements in boreholes SK-1, SK-7i, and SK-8i) and might have consequently caused the saturation of the weathered tuffite unit at this location (Fig. 8).

The sandy silt level of weathered tuffite

The sandy silt level constitutes the major part of the weathered tuffite unit and its thickness is about 5.5–6.0 m. The landslide (displaced) material is generally composed of this level of the weathered tuff unit. The level is generally brown–greenish brown in color, generally stiff, rarely soft to hard, and of medium plasticity. The statistics of the SPT and laboratory index test results are given in Table 4.

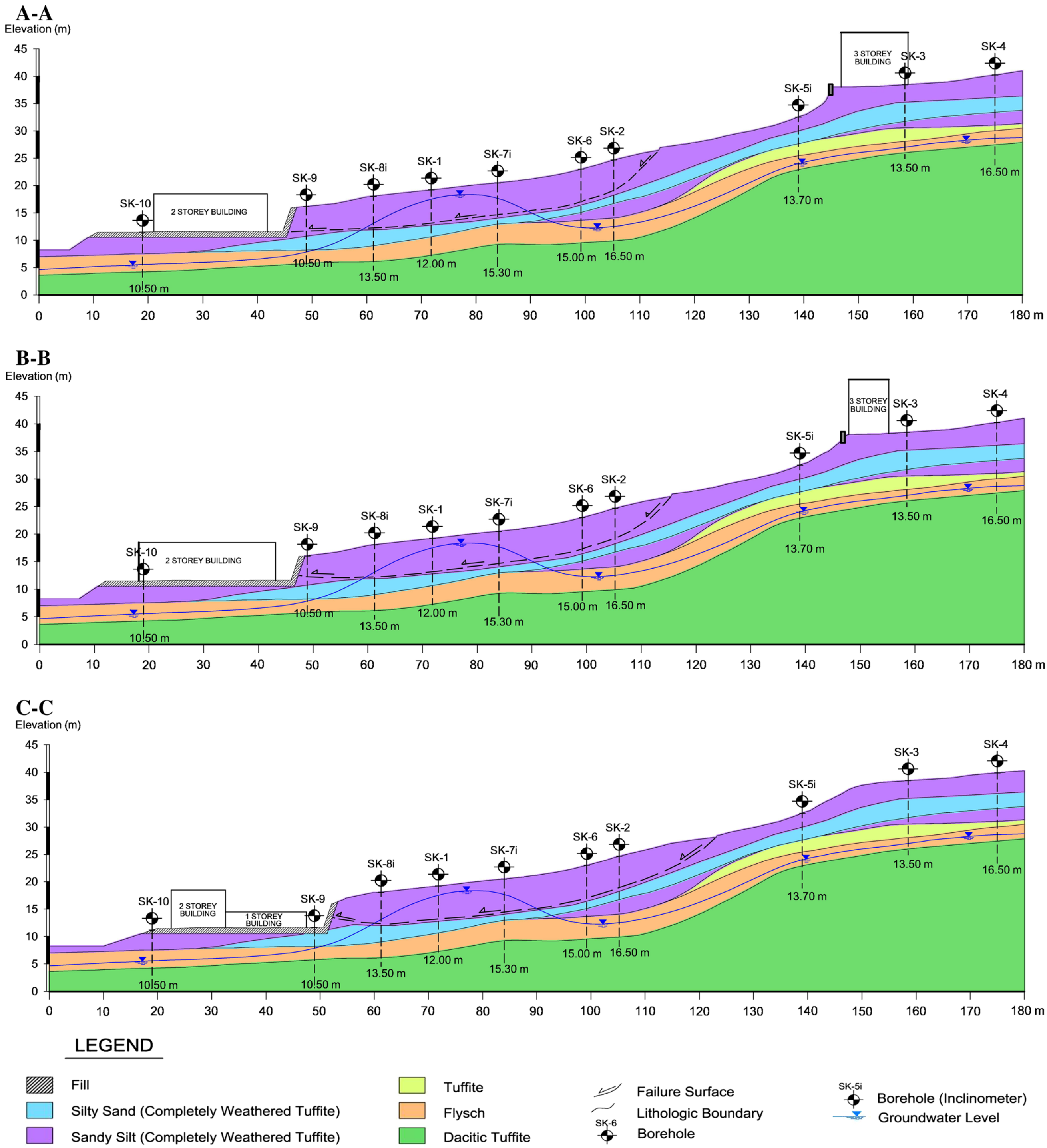

Fig. 8 Cross-sections *A–A*, *B–B* and *C–C*. Note that Fig. 3 gives the trends of the cross-sections. The groundwater levels measured in boreholes SK-1, SK-7i, and SK-8i revealed a rise in the groundwater table. The reason for this anomaly was the ENE–WSW trending broken water pipeline which caused an increase in the groundwater level

The silty/gravelly silty sand level of the weathered tuffite unit

This unit is the second member of the weathered tuffite unit and is generally observed just below the slip surface. Silty/gravely silty sand is brown to yellowish/greenish brown colored, moist, loose to moderately dense. The statistics of the SPT and laboratory index test results in the silty/gravelly silty sand level of the weathered tuffite unit is given in Table 5.

Table 2 The statistics of the standard penetration test (SPT) and laboratory index test results in the highly weathered portions of the flysch unit

	Lower bound	Upper bound	Mean	Standard deviation
SPT (N) (18)	24	100	69	32.3
Water content (W_n) (%) (18)	1	26	15.3	5.94
Retained on # 4 sieve (%) (18)	0	99	27.4	25.9
Passing # 200 (%) (18)	0	21	11.9	5.09
Unified soil class	SM, SP-SM, SW-SM, GP, GM			

The number of tests is given in parentheses

Table 3 The statistics of the SPT and laboratory index test results in the highly weathered portions of tuffite

	Lower bound	Upper bound	Mean	Standard deviation
SPT (N) (4)	47	92	61.3	21.3
Water content (W_n) (%) (4)	12	28	18.5	6.95
Retained on # 4 sieve (%) (4)	1	37	24.5	16.3
Passing # 200 (%) (4)	14	30	20.5	6.81
Unified soil class	SM, SP-SM			

The number of tests is given in parentheses

Table 4 The statistics of the SPT, laboratory index and uniaxial compressive strength test results in the sandy silt level of the weathered tuffite unit

	Lower bound	Upper bound	Mean	Standard deviation
SPT (N) (27)	4	37	13	8.15
Water content (W_n) (%) (30)	33	55	41.9	5.69
Natural unit weight (γ_n) (kN/m^3) (3)	17.16	17.85	17.65	3.922
Saturated unit weight (γ_{sat}) (kN/m^3) (3)	18.54	19.29	19.07	3.240
Liquid limit (LL) (%) (28)	40	86	57.8	8.10
Plasticity index (PI) (28)	12	42	17.9	5.81
Retained on # 4 sieve (%) (28)	0	8	0.63	1.56
Passing # 200 (%) (28)	50	93	64.1	10.6
Uniaxial compressive strength (q_u) (kPa) (1)	–	–	89.0	–
Unified soil class	MH, ML			

The number of tests is given in parentheses

Assessment of slope instability

Mechanism and factors that have contributed to the formation of the Aksu landslide

The Aksu landslide occurred within the upper Cretaceous-Lower Eocene volcano-sedimentary and sedimentary rocks at the eastern Pontides. The formation is subdivided into four major lithologies, namely weathered tuffite, flysch, tuffite, and dacitic tuffite from top to bottom. The upper 7.0–9.0 m of the study area consists of weathered tuffite, which is characterized by a soil-like lithology. The weathered tuffite is underlain by the relatively resistant flysch unit. The geotechnical investigation performed in the area along with the inclinometer measurements indicated that shear failure initiated between these two units. As indicated by Table 1, the inclinometer data coming from SK-5i, which was located in front of the three storey building, did not indicate any significant ground movement which might have deleteriously affected the building. However, inspection of the depth versus cumulative displacement graphs of SK-7i and SK-8i led to a justification of a movement at a depth of about 5.5 m. Taking the average values of the inclinometer measurements of SK-7i

Table 5 The statistics of the SPT and laboratory index test results in the silty/gravelly silty sand level of the weathered tuffite unit

	Lower bound	Upper bound	Mean	Standard deviation
SPT (N) (14)	4	24	13.4	6.48
Water content (W_n) (%) (15)	18	51	36.7	9.43
Natural unit weight (γ_n) (kN/m^3) (1)	–	–	18.93	–
Saturated unit weight (γ_{sat}) (kN/m^3) (3)	–	–	19.79	–
Liquid limit (LL) (%) (15)	NP	66	54.5	6.04
Plasticity index (PI) (15)	NP	23	13.4	3.92
Retained on # 4 sieve (%) (15)	0	41	12.6	12.1
Passing # 200 (%) (15)	12	49	29.6	11.3
Unified soil class	SM			

The number of tests is given in parentheses

Fig. 9 A view showing Tünel restaurant (to the *right*) and the tilted retaining walls after the occurrence of the landslide (to the *left*)

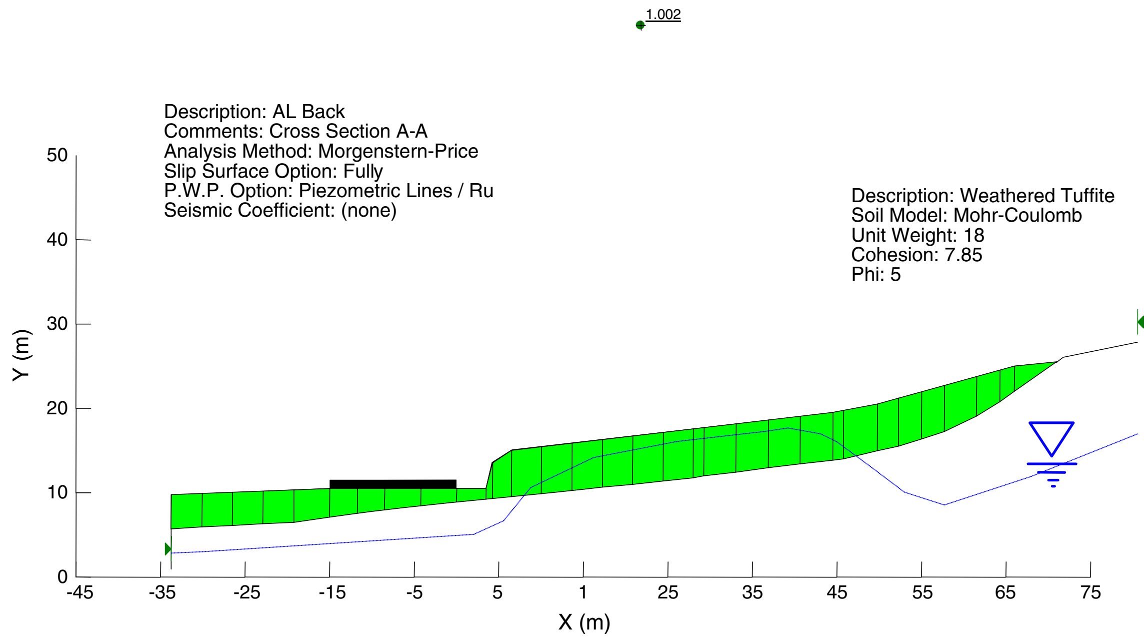

Fig. 10 The results of the back analysis for landslide cross-section *A–A*. Figure 3 gives the trend of the cross-section *A–A*. *AL Back* Aksu landslide back analysis

and SK-8i, it was concluded that the landslide mass moved towards 311°N with a velocity of 2.54 mm/day.

There are two main factors, which most probably caused the formation of the Aksu landslide: (1) excavations at the toe of the slope and (2) the influence of groundwater. The influence of these factors on the stability of the Aksu landslide is investigated below.

Excavations at the toe of the slope

There have been two different excavation activities performed at the toe of the slope at different times. The first excavation was performed several years prior to the landslide movement, for the construction of the Tünel restaurant (Figs. 3, 9). About 4,600 m^3 of earth material was excavated for the purpose of flattening the ground. Vertical slopes of the excavated area were supported by concrete retaining walls which were tilted after the landslide occurred.

The other excavation was made to enlarge the width of the Giresun–Espiye road between KM: 1 + 030–1 + 170. During construction operations, some amount of earth was excavated from the toe of the slope to increase the width of the existing road from 13.5 to 27.0 m (Fig. 3). The Aksu landslide occurred after this excavation activity.

The influence of groundwater

The second factor that has contributed to failure in the project area is suspected to be the effect of groundwater. The natural state of the groundwater level lies within the flysch unit in the study area. However, the water pipeline with a trend of ENE-WSW that passed through the study area was broken and the water, which leaked from this pipeline, has caused an increase in the groundwater level in the down-slope direction (Fig. 8). In addition to the leakage from the pipeline, the Tünel restaurant building might have also adversely affected the groundwater condition in the study area. As mentioned earlier, vertical cut slopes of the excavated area of the Tünel restaurant were supported by concrete retaining walls (Fig. 9). The absence of drain pipes within these concrete retaining walls most probably caused them to act as impervious barriers for the groundwater flow and led to an increase in the groundwater level behind the relatively impervious retaining walls.

Determination of the shear strength parameters

A back analysis was performed to determine the shear strength parameters of the Aksu landslide. Koçkar and Akgün (2003b), Akgün and Koçkar (2004) and Sharifzadeh et al. (2010) present examples of the back analysis procedure for slope stability analysis. Cross-sections A–A, B–B and C–C were utilized to determine the shear strength parameters of the failure surface of the Aksu landslide. The trends of the cross-sections are shown in Fig. 3. A back analysis on cross-section A–A is given by Fig. 10. In this figure, the green-colored area represents the displaced material. The Morgenstern–Price (Morgenstren and Price 1965) method was utilized for the back analysis through

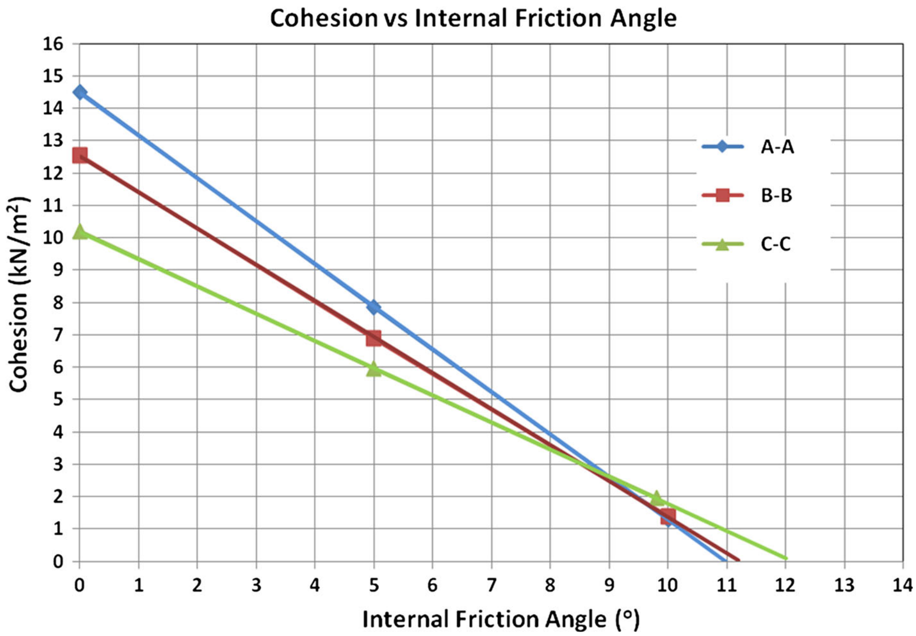

Fig. 11 Graphical representation of the back analysis results

using the slope stability software Slope/W (Geo-Slope International Ltd. 2007).

Figure 11 presents a graphical representation of the back analysis results where the values of c' and ϕ', as obtained from the points of intersections were determined to be 2.5 kPa and 9°, respectively.

The influence of the main factors on the formation of the landslide

In this section, the contribution of the main factors, namely the influence of excavation and groundwater conditions and/or both on the formation of the landslide are analyzed by utilizing slope stability analyses. The stages or sequence of events that took place in the slope was incorporated into the slope stability analysis and presented in Table 6. For the initial Stage 1, which represents natural slope conditions prior to any excavation activity where the groundwater level lies below the failure surface (i.e., within the flysch unit), the factor of safety of the slope was calculated to be 1.258 (Table 6). Next, for Stage 2, the excavation of the foundation and construction of the Tünel restaurant on the natural slope of Stage 1 decreased the factor of safety to 1.192, which led to a decrease in the factor of safety of 5.25 %. Incorporation of leakage to Stage 2 led to Stage 3 for which the groundwater level distribution is as that presented by Fig. 8 for slope cross-section A–A. For Stage 3, due to the occurrence of leakage from the pipeline, the factor of safety decreased to 1.057 which represented a decrease of 15.98 % as compared to the natural slope condition represented by Stage 1. Further disturbance due to the excavation related to the enlargement of the width of the existing road led to Stage 4 (landslide triggering condition) which gave a factor of safety of 0.985 that represented a decrease of 21.70 % as compared to the natural slope condition represented by Stage 1. Subsurface drainage measures applied to Stage 4 led to Stage 5 where the groundwater level was assumed to be lowered below the failure surface of slope cross-section A–A (Fig. 8). For Stage 5, the factor of safety increased to 1.147 which represented a decrease of 8.82 % as compared to the natural slope condition represented by Stage 1. Stage 6 represents full saturation of the slope of Stage 4 where the groundwater level is assumed to rise to the ground surface. For Stage 6, the factor of safety decreased to 0.884 which represented a decrease of 29.73 % as compared to the natural slope condition represented by Stage 1. Note that Stages 5 and 6 were modeled as complementary analyses to identify the contribution of the groundwater level (i.e., leakage) to the stability of the slope.

It follows from Table 6 that the landslide area has not posed any instability problems prior to the construction of Tünel restaurant and that the natural, undisturbed slope was stable (FS = 1.258) in its original topographic setting (Stage 1). Even though the construction of the Tünel restaurant has decreased the factor of safety by 5.25 %, since the factor of safety was 1.192, the slope may be assumed to have been stable after the construction of the Tünel restaurant (Stage 2). Incorporation of leakage from the pipeline to Stage 2 led to Stage 3, where the factor of safety decreased to 1.057 which represented a decrease of 15.98 % as compared to the natural slope condition represented by Stage 1. This indicates that the stability problem was elevated and approached a critical condition with

Table 6 Factor of safety (FS) of the slope (i.e., cross-section A–A; Fig. 8) as a function of the slope disturbance stages (i.e., sequence of events)

Stage (i.e., sequence of event)	FS	Change in FS (%)[a]
Stage 1[b]: Natural slope prior to any excavation for which the groundwater level lies below the failure surface	1.258	0.00
Stage 2[b]: Natural slope of Stage 1 + excavation of the foundation and construction of the Tünel restaurant	1.192	−5.25
Stage 3: Stage 2 + leakage from the pipeline leading to the groundwater level distribution for slope cross-section A–A as presented by Fig. 8	1.057	−15.98
Stage 4: Stage 3 + excavation related to the enlargement of the width of the existing road (landslide triggering condition)	0.985	−21.70
Stage 5[c]: Stage 4 without any leakage from the pipeline (i.e., groundwater level below the failure surface)	1.147	−8.82
Stage 6[c]: Stage 4 + groundwater level at the ground surface	0.884	−29.73

[a] Change in FS is presented with respect to Stage 1 (i.e., the natural slope prior to any disturbance)

[b] Note that the natural state of the groundwater level lies within the flysch unit (i.e., below the failure surface)

[c] Note that Stages 4, 5, and 6 are modeled to identify the contribution of the groundwater level (i.e., leakage) to the stability of the slope

the incorporation of leakage from the pipeline. Further disturbance due to the excavation related to the enlargement of the width of the existing road led to Stage 4 that gave a factor of safety of 0.985 which represented a decrease by 21.70 % as compared to the natural slope condition represented by Stage 1 and triggered the landslide. It may be inferred from Table 6 that about 10.97 % of the instability of the slope may be attributed to the excavations related to the Tünel restaurant and to the existing road, whereas 10.73 % of the instability may be related to the leakage from the pipeline that led to the groundwater level distribution presented by Fig. 8 for the slope cross-section A–A. Hence, the contribution of the excavations and the groundwater level rise to the instability of the slope may be considered to be almost equal. Subsurface drainage measures represented by Stage 5 where the groundwater level was assumed to be lowered below the failure surface of the landslide led to an increased factor of safety of 1.147 which represented a decrease by 8.82 % as compared to the natural slope condition represented by Stage 1. This result indicated that the main reason for the formation of the landslide was the contribution of leakage from the broken pipeline since the factor of safety increased from 0.985 (Stage 4) to 1.147 (Stage 5) with the lowering of the groundwater level below the failure surface through subsurface drainage measures. Full saturation of the slope of Stage 4 where the groundwater level is assumed to rise to the ground surface led to Stage 6 for which the factor of safety decreased to 0.884 which represented a decrease by 29.73 % as compared to the natural slope condition represented by Stage 1.

A sensitivity analysis performed in regards to the influence of the groundwater level on slope stability, where Stage 4 and Stage 6 are compared with Stage 5 indicates that the factor of safety decreases by 14.12 % when the groundwater level in the slope rises from the bottom of the landslide surface (Stage 5) to the level indicated by Stage 4 (i.e., the groundwater level distribution as presented by Fig. 8 for slope cross-section A–A) and decreases by 22.93 % when the groundwater level rises to the ground surface (Stage 6; Table 6).

Slope stabilization

General

Surface and subsurface drainage accompanying a rock fill buttress were the slope stabilization techniques considered for the Aksu landslide.

Drainage

The effective surface water potential of the Black Sea region also controls the groundwater dynamics. After rainy periods, the groundwater level increases dramatically and affects the stability adversely. In order to minimize the adverse effects of groundwater, both surface and subsurface drainage needed to be performed in the landslide area.

Surface drainage

Considering the rainy climate of the Black Sea region, the surface water in the study area was one of the major problems to be dealt with. For draining the surface water around the landslide area, a surface water drainage system consisting of a ditch surrounding the displaced material was designed (Fig. 12a). The details of the geometry of the ditch are given in Fig. 12b. In addition to the surface water drainage system, as a precaution, the tension cracks were recommended to be sealed with clay material to prevent the build-up of water pressure in them (Simons et al. 2001). In addition to the water pipeline, there was also considerable seepage from the water depots and sewer tanks located in the area (Fig. 3). All of these structures including the water pipeline, water depots, and sewer tanks were recommended to be either removed or strictly isolated.

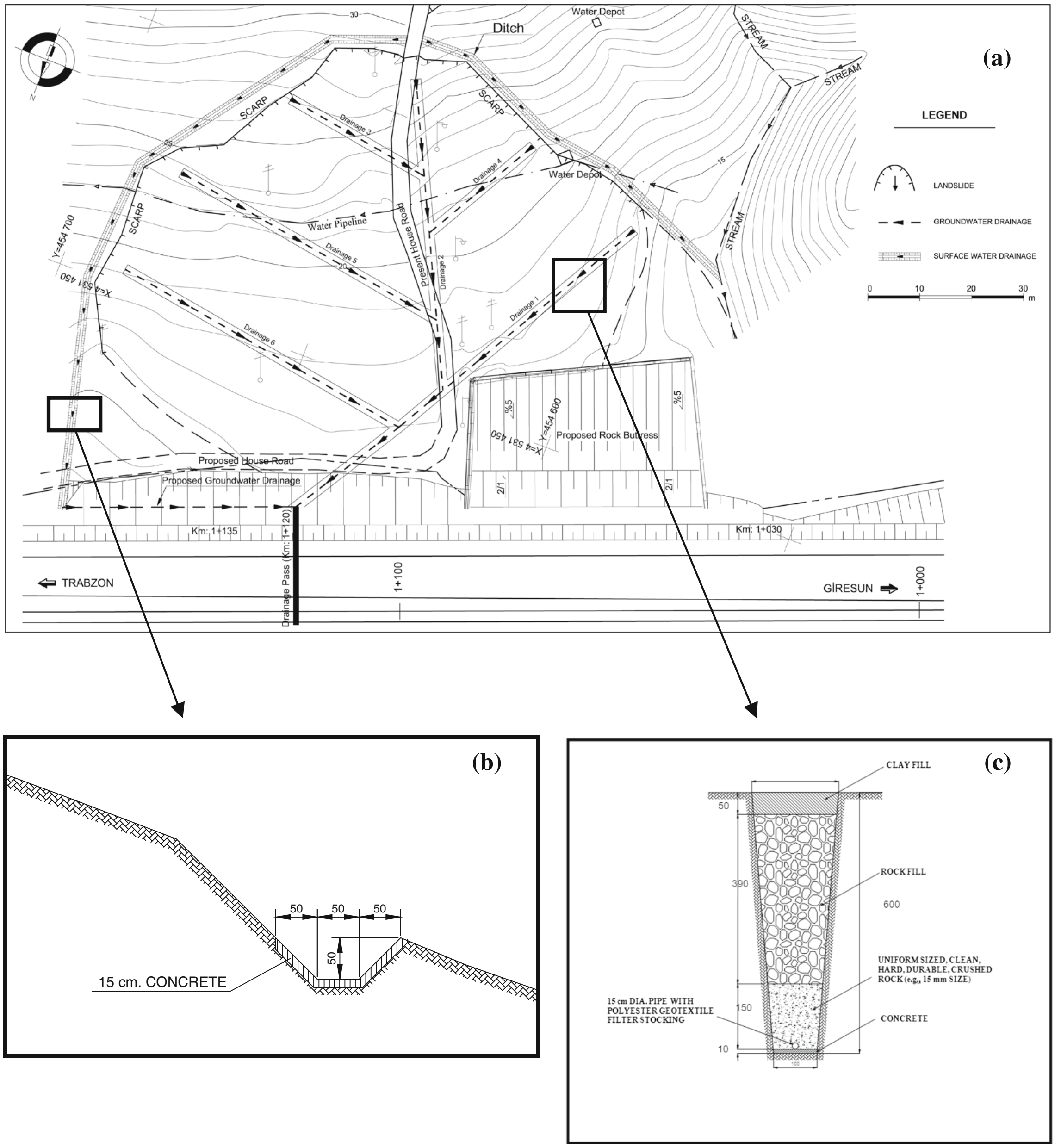

Fig. 12 a Slope stabilization applications of the surface water drainage system consisting of a ditch surrounding the displaced material, **b** the details of the geometry of the ditch (values are in cm) (Avşar 2004), and **c** the subsurface drains (values are in cm) (Avşar 2004; Jeffery 1987) that are proposed to be constructed for surface drainage in the study area

Subsurface drainage

Upon preventing the leakage from the broken pipeline, the groundwater level was expected to drop below the failure surface, to its natural position. The boreholes, which supplied information on the groundwater level were drilled and groundwater level observations were made. However, since the observations were made during the rainiest season in the Giresun province, the groundwater level was expected to increase during that time. Considering the possibility of an increase in the groundwater level due to heavy rain, construction of a subsurface drainage system was proposed. The location of the proposed subsurface drainage system is given in Fig. 12a. Figure 12c gives the details of the subsurface drain. The drains were recommended to be excavated progressively down to the bottom of the weathered tuffite unit and backfilled, noting that the design of subsurface drains could be modified during construction by considering the difficulties in the excavation operation.

After surface and subsurface water drainage applications, the groundwater level was lowered below the failure

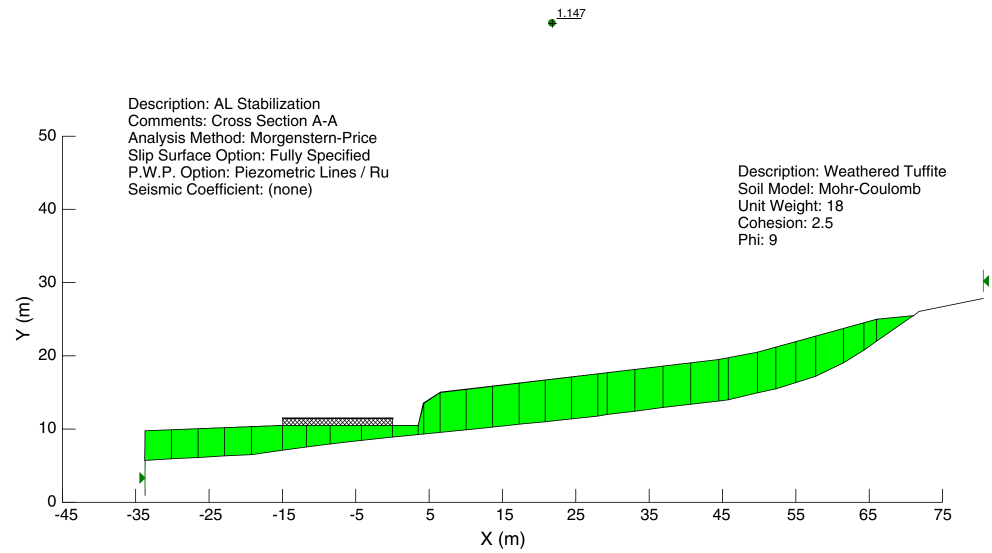

Fig. 13 Demonstration of the reduction in the groundwater level below the failure surface (Stage 5, Table 6) leading to an increase in the factor of safety (FS) to 1.147 for cross-section *A–A* (Fig. 8). *AL* Aksu landslide

surface. With the lowering of the groundwater level below the failure surface, the factor of safety (FS) of the Aksu landslide increased to 1.147 (Fig. 13). It should be noted that the back analysis was performed to estimate the Mohr–Coulomb shear strength parameters, namely cohesion (c) and angle of internal friction (ϕ) by using the slope model having a groundwater distribution as shown in Fig. 8. The results were that $c = 2.5$ kPa and $\phi = 9°$. The shear strength parameters were then used to calculate the factor of safety of the slope without groundwater (i.e., groundwater level below the failure surface; Stage 5 in Table 6; Fig. 13) or with a full column of groundwater (i.e., groundwater level at the ground surface; Stage 6 in Table 6). It should be noted that when the groundwater level is lower than the failure surface or when the groundwater level rises to the ground surface, the shear strength parameters and the unit weight of rock masses will change. For example, in a very general sense, for a moist c–ϕ soil, both cohesion and angle of internal friction tend to increase due to the negative pore water pressure, i.e., suction, which increases cohesion and angle of internal friction by drawing the grains together, whereas for a saturated c–ϕ soil, positive pore water pressures could develop which push the grains apart, act against the normal stress, and reduce it by leading to a decrease in cohesion and angle of internal friction. This general trend may also be assumed to hold for silty soils (i.e., sandy silt and/or silty sand) even though the shear strength behavior of silts has not been studied as extensively and is not well understood as much as the behavior of granular materials or clays. Although the strengths of silts are governed by the same principles as the strengths of other soils, the range of their behavior is wide, and sufficient data are not available to anticipate or estimate their properties with the same degree of reliability as it is possible in case of granular soils or clays (Duncan and Wright 2005). Hence, the slope stability analysis should be performed with the appropriate shear strength parameters that are determined after performing back analyses on slope sections possessing the appropriate groundwater levels and with the appropriate unit weights (i.e., moist or saturated unit weight).

Rock fill buttress

Since the excavation at the toe of the slope with the purpose of constructing the Tünel restaurant was one of the factors that contributed to the slope instability in the study area, it was proposed that the Tünel restaurant building be removed and a rock fill buttress be placed in its foundation excavation area. The rock fill buttress was modeled for cross-section A–A (Fig. 3) by utilizing the Slope/W software (Geo-Slope International Ltd. 2007; Fig. 14). The unit weight, cohesion, and angle of internal friction of the rock fill buttress were assigned values of 20.86 kN/m^3, 0 kPa, and 35°, respectively, which are the lower bound values given by Leps (1988). The reason for assigning the lower bound values was to perform a fairly conservative approach.

The effect of the rock fill buttress was analyzed by using the Morgenstern–Price method (Morgenstren and Price 1965). For limit equilibrium analyses of critical slopes adjacent to highways, a factor of safety of 1.50 is preferred (Turkish Department of State Highways 2002). The results

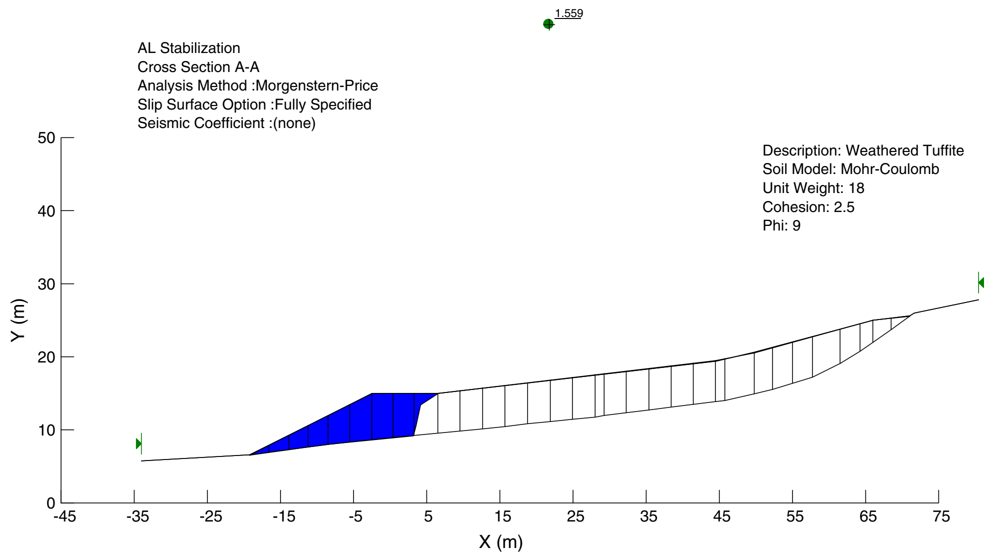

Fig. 14 Slope stability limit equilibrium model with rock fill buttress along cross-section *A–A*. *AL* Aksu landslide

of the slope stability analysis showed that the rock fill buttress was sufficient to stabilize the landslide which led to a factor of safety (FS) of 1.559 (Fig. 14).

Pseudostatic slope stability analysis

Since the study area is situated in a 4th degree earthquake zone, utilization of pseudostatic slope stability analysis is suggested (e.g., Akgün and Koçkar 2004; Özcep et al. 2012). Inspection of the earthquakes that have occurred in a 100-km radius around the study area between 1900 and 2014 revealed that only two earthquakes had a magnitude ranging between 6.0 and 6.9 (AFAD 2012). Regarding these results, Makdisi and Seed (1978) proposed to select a horizontal seismic coefficient of 0.15 g in the case of a 6.5-magnitude earthquake with a factor of safety of at least 1.15 for stability. When a horizontal seismic coefficient of 0.15 g was included in the calculations, the factor of safety (FS) of the Aksu landslide was reduced to 0.853 (Fig. 15). In an attempt to increase the FS above 1.15, the geometry of the rock fill buttress was modified. Four alternative rock fill buttress geometries (Fig. 16), which led to different factor of safety values (Table 7), were analyzed. Alternatives 1, 2, and 3 involved a gradual increase of the height of the rock buttress (note that the FS slightly decreased with an increased height of the rock buttress which may most

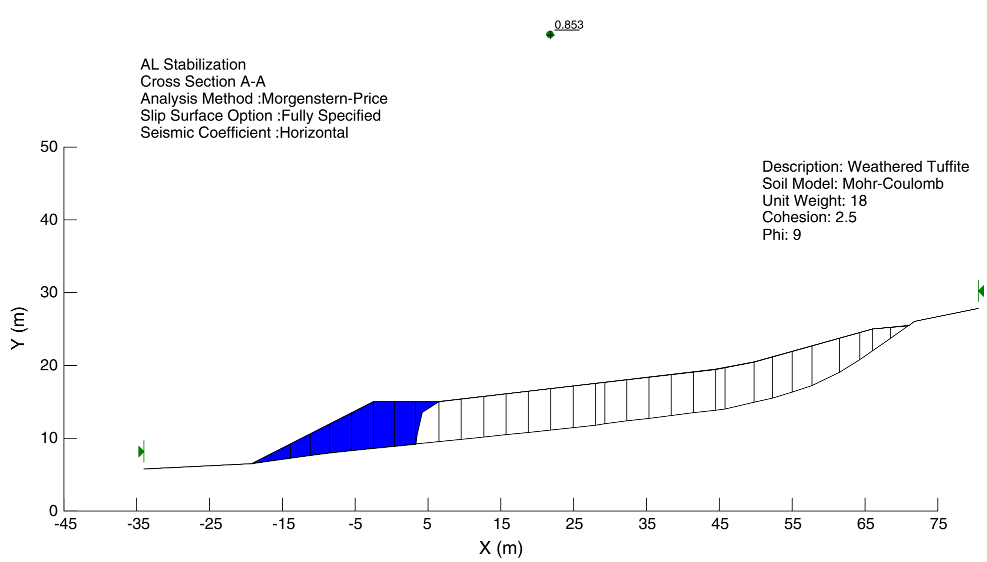

Fig. 15 Demonstration of the decrease of the factor of safety (FS) along cross-section *A–A* of the Aksu landslide from 1.559 to 0.853 when a horizontal seismic coefficient of 0.15 g is included in the limit equilibrium analysis. *AL* Aksu landslide

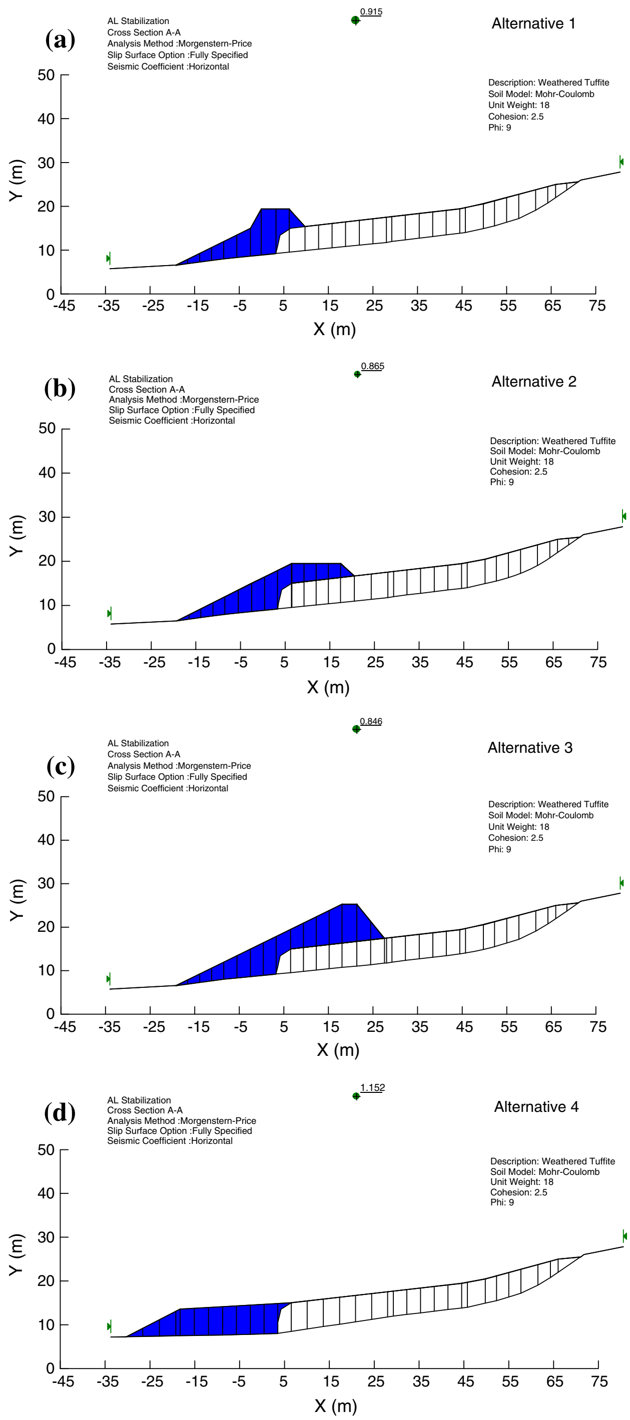

Fig. 16 Rock fill buttress alternatives **a** 1, **b** 2, **c** 3, and **d** 4, respectively

probably be attributed to an increase in the rate of the driving force exceeding the increase in the rate of the resisting force with an increased buttress height going from Alternative 1–3). As presented by Alternative 4 (Fig. 16), what really mattered was the extent of the rock buttress towards the toe which led to an increase in the FS to about 1.152. Hence, Alternatives 1, 2, and 3 failed to satisfy the

Table 7 Factor of safety results corresponding to alternative buttress geometries

Buttress geometry	Factor of safety (FS)
Alternative 1	0.915
Alternative 2	0.865
Alternative 3	0.846
Alternative 4	1.152

FS $\geq$ 1.15 condition while with Alternative 4, the Aksu landslide was stabilized with a FS of 1.152.

Conclusions and recommendations

During the construction of the Giresun–Espiye road between KM: 1 + 030–1 + 170, some amount of soil was excavated from the toe of a major slope to increase the width of the existing road. The removal of the earth material disturbed the stability of the natural slope on the south side of the road and gave way to a mass slope movement with a length of 110 m and width of 90 m. In order to investigate the causes and the mechanism of this slope failure, a detailed site investigation study, including engineering geological mapping, drilling work, in situ, and laboratory testing was performed.

As a result of the engineering geological investigation studies, the following conclusions were drawn:

- The formation was subdivided into four major lithologies from top to bottom: weathered tuffite, flysch, tuffite, and dacitic tuffite. The upper 7.0–9.0 m of the study area consisted of weathered tuffite, which was characterized by a soil-like lithology. The weathered tuffite was underlain by a relatively resistant flysch unit. The inclinometer measurements combined with the engineering geological field investigations revealed that the shear failure initiated between these two units.
- The failed mass continued to move towards 311°N with a velocity of 2.54 mm/day.

Two different excavations, which were performed for the foundation of the Tünel restaurant building and for the widening of the Giresun–Espiye road, combined with an increase in the groundwater level due to a broken water pipeline, were the reasons for the occurrence of the Aksu landslide.

A back analysis performed on the landslide led to the determination of the shear strength parameters of the failure surface as $\phi' = 9^{\circ}$ and $c' = 2.5$ kPa. The results of the slope stability analyses indicated that the contribution of the excavations (i.e., excavations related with the Tünel restaurant and with the existing road) and the groundwater level rise to the instability of the slope were almost equal.

The slope stability analysis indicated that the main reason for the formation of the landslide was the contribution of leakage from the broken pipeline since the factor of safety increased from 0.985 to 1.147 with the lowering of the groundwater level below the failure surface through subsurface drainage measures.

A sensitivity analysis performed in regards to the influence of the groundwater level on slope stability indicated that the factor of safety decreased by 14.12 % when the groundwater level in the slope rised from the bottom of the landslide surface to the level indicated by present groundwater level conditions and decreased by 22.93 % when the groundwater level rised to the ground surface. The influence of an earthquake was investigated through pseudostatic slope stability analysis. Utilization of surface drainage, subsurface drainage, and rock fill buttressing at the toe of the Aksu landslide revealed that the landslide could be stabilized statically and dynamically with a factor of safety (FS) of 1.152.

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