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Methodology for tunnel and portal support design in mixed limestone, schist and phyllite conditions: a case study in Turkey

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Abstract

The purpose of this study is to present a methodology for tunnel and support design in mixed limestone, schist and phyllite conditions through investigating two highway tunnel case studies that are located along the Antalya–Alanya Highway in southern Turkey. The main lithologies of the project area are regularly jointed, recrystallized limestone and the weak lithologies of the schist unit (i.e., pelitic schist, calc schist, graphitic phyllite and alternations of these lithologies). A detailed geological and geotechnical study was carried out in the project area, and the tunnel ground support types and categories were determined according to the Q-system, rock mass rating method and New Austrian Tunneling Method (NATM). The shear strength parameters and geomechanical properties of the rock masses were obtained by using the geological strength index (GSI). The deformation moduli and post-failure behavior of the rock masses have been determined. Slope stability analyses were performed at the portal, side or cut slope sections. Kinematic and limit equilibrium analyses incorporating the effects of water pressure were performed for the regularly jointed failed rock slopes. Circular failure analogy was used for the slope stability analyses of irregularly jointed, highly foliated lithologies. Slope support system recommendations were made. A back analysis on a failed slope was performed. The results of the back analysis compared well with the results obtained through the GSI method. The tunnel grounds were divided into sections according to their rock mass classes. The deformations and stress concentrations around each tunnel section were investigated and the interactions of the empirical support systems with the rock masses were analyzed by using the Phase^2 finite element software. The regularly jointed rock masses were modeled to be anisotropic and the irregularly jointed, highly foliated and very deformable soil-like lithologies were modeled to be isotropic in the tunnel finite element analyses.

1. Introduction

1.1. Methodology for assessing support requirements in mixed rock conditions

The purpose of this study is to present a methodology for tunnel and support design in mixed limestone, schist and phyllite conditions. The methodology was assessed through two highway tunnel case studies in southern Turkey which involved investigating the engineering geological and geotechnical characteristics of the rock material and rock mass of the tunnel grounds, and suggesting appropriate support and stabilization techniques. The tunnels which are named as Ilıksu 1 and Ilıksu 2 are located along the 4th division route of the Antalya–Alanya autoroad. Units that belong to the Yumrudağ Nappe that is the structurally highest unit of the Alanya Massif were observed in the project area. Yumrudağ Nappe consists of a thick carbonate sequence (regularly bedded, recystallized limestone) underlain by a relatively thin schist unit metamorphosed under low-grade greenschist facies. Pelitic schists, calc schists, graphitic phyllites and alternations of these lithologies are the major lithologies of the schist unit that are highly problematic. Fig. 1 gives a detailed flow chart of the methodology. Fig. 2 presents a location map of the project area.

The study started with literature review regarding the geology and geotechnical characteristics of the project site, a preliminary site reconnaissance visit to identify major lithologies, structural features and to decide on possible borehole locations (Fig. 1). Detailed geological and geotechnical field investigations in the project area encompassed geological mapping and geological

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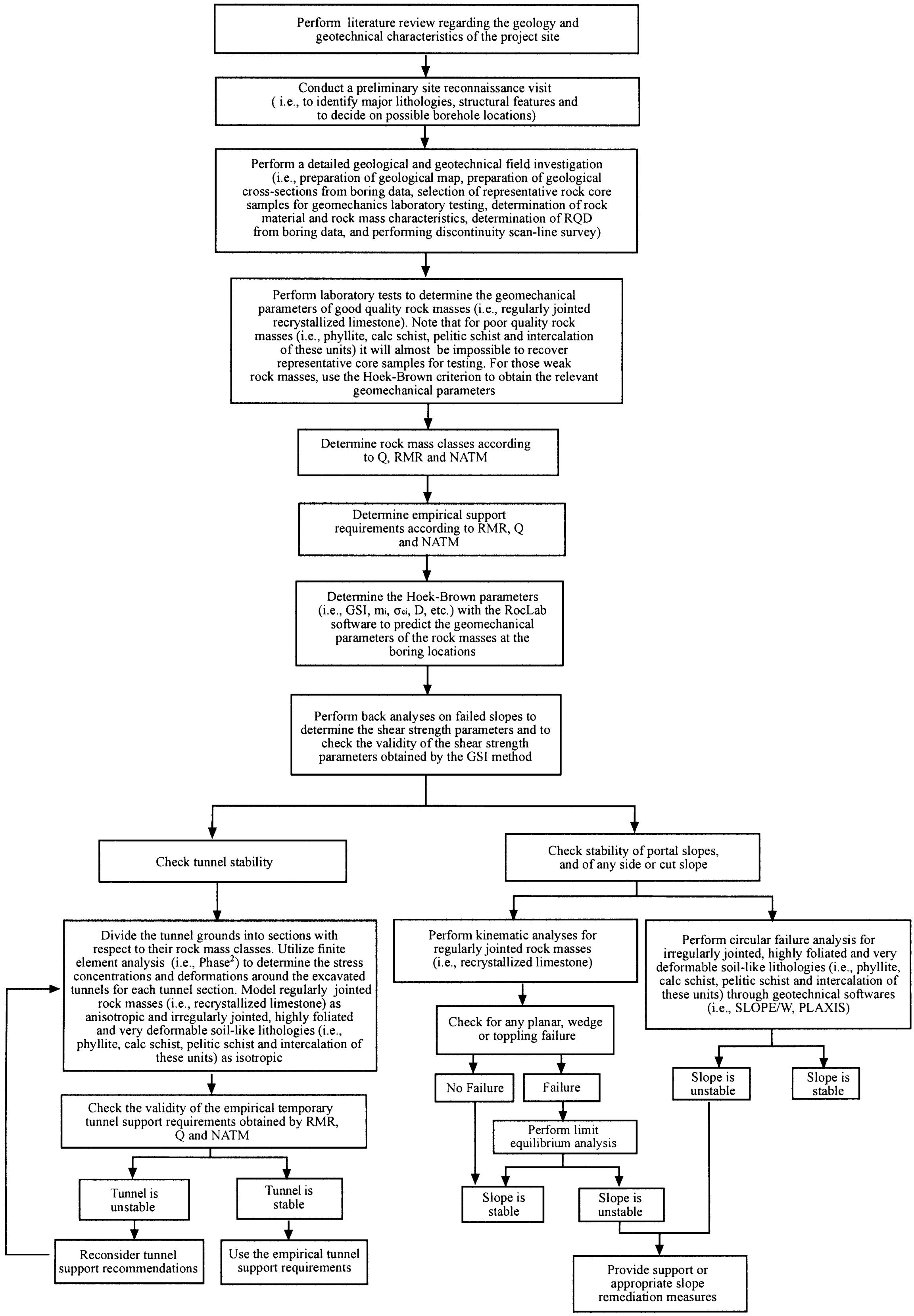


Fig. 1. Flow chart of methodology for assessing support requirements in mixed rock conditions.

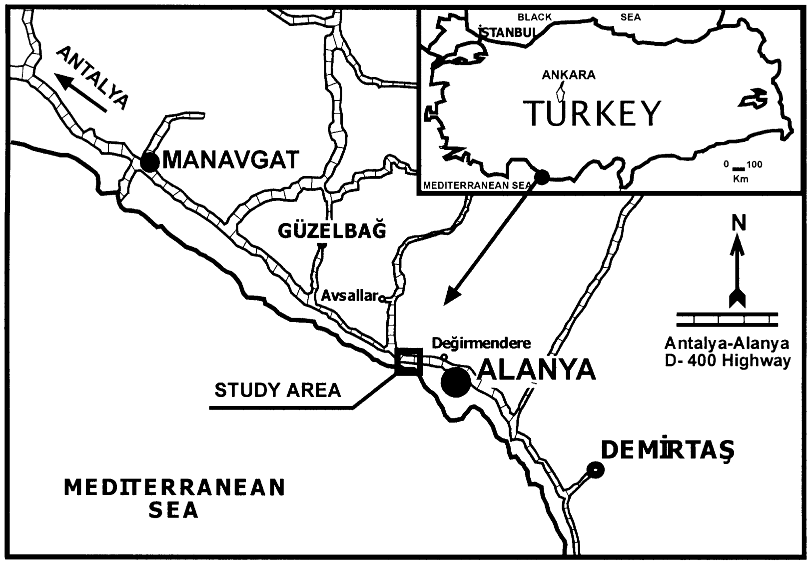


Fig. 2. Location map of the project area (Scale: 1/5000).

cross-section preparation from boring data, selection of representative rock core samples for geomechanics laboratory testing, determination of rock material and rock mass characteristics, determination of RQD from boring data, and determination of discontinuity characteristics through scan-line survey. Laboratory tests were performed to determine the geomechanical parameters of good quality rock masses (i.e., regularly jointed, recrystallized limestone). For poor quality rock masses (i.e., phyllite, calc schist, pelitic schist and intercalation of these lithologies), the Hoek–Brown criterion was used to obtain the relevant geomechanical parameters since it was almost impossible to recover representative core samples for laboratory testing.

The tunnel grounds were classified according to the Q-system, RMR method and NATM. Empirical tunnel support types and categories were selected for each of the three classification systems. The shear strength parameters and geomechanical properties of the rock masses at each borehole location were obtained by using the geological strength index (GSI). Back analysis was performed on a failed rock slope to perform a check on the validity of the shear strength parameters obtained by the GSI method.

The tunnel grounds were divided into sections according to their rock mass classes. By using the appropriate geotechnical parameters (i.e., shear strength, deformation modulus, Poisson's ratio, dilation angle, etc.), deformations and stress concentrations around each tunnel section were investigated and the interactions of the empirical support systems with the rock masses were analyzed by using the Phase^2 finite element software. The regularly jointed rock masses (i.e., recrystallized limestone) were modeled to be anisotropic, whereas irregularly jointed, highly foliated and very deformable soil-like lithologies (i.e., phyllite, calc schist, pelitic schist and intercalation of these lithologies) were modeled to be isotropic.

In order to decide on the most suitable geometry and determine the stability of the portal, side or cut slope sections, slope stability analyses were performed. Initially, kinematic analyses were performed for the regularly bedded rock masses. Later, limit equilibrium analyses were performed for the kinematically failed rock slopes incorporating the effect of water pressure. Slope stability analyses of irregularly jointed, highly foliated and laminated weak lithologies were analyzed and compared by two different softwares (Slope/W and PLAXIS 7.2). Following the slope stability analyses, recommendations were made regarding the required support systems or appropriate slope remediation measures.

1.2. Study area

The study area is located 7 km west of Alanya on the Antalya–Alanya D-400 highway along the coast of the Mediterranean Sea (Fig. 2) and is included in the 1/25,000 scale topographic map of the General Directorate of Highways in the Alanya Section. The study area is in the close proximity of the General Directorate of Highways Recreation Park which lies in between the Avsallar and Değirmendere villages.

2. Geology of the region

The study area is overlain by the Alanya Massif which is the name given to a large area of metamorphic rocks situated towards the east of Antalya Bay in the Eastern

Mediterranean region [1]. The Mesozoic continental margin type lithologies of the Antalya unit crop out beneath the Alanya Nappes in a large tectonic window. In the east of the Antalya Bay between Alanya and Anamur, rocks of the Antalya unit are in turn tectonically overlain by the metamorphic rocks of the Alanya Massif. This is made up of three superimposed, relatively flat lying crystalline nappes [2]. The Alanya Massif consists of the structurally lowest part of Mahmutlar Nappe, the intermediate part of the Sugözü Nappe, and the structurally highest unit of the Yumrudağ Nappe. In the project area, the units observed belong to the Yumrudağ Nappe. In post-Maastrichtian times the Alanya Nappes, which were by then welded into one unit, were thrust over the sedimentary rocks of the Alanya Unit. The final thrusting of the Alanya Nappes and the underlying Antalya Unit over the Tauride carbonate platform occurred before the Middle Eocene [2].

Yumrudağ Nappe is the structurally highest member of the Alanya Nappes and constitutes the bulk of the Alanya Massif. It consists of schists overlain by a thick sequence of recrystallized limestone. The passage from the schists to the overlying carbonates is gradational with schist and carbonate bands several meters thick at the contact. Pelitic (weak chloritic to talcic) schists, calc schists, phyllites, meta-dolomites and recrystallized limestone bands are the major lithologies of the schist unit. Due to tectonic activity (thrusting, intense folding and shearing), rock mass can be highly heterogeneous in some of the lithologies of the schist unit.

The overlying Permian carbonate unit forms the thick carapace of the Alanya Massif. Several hundred meters of generally flat-lying, gray, massively bedded, monotonous recrystallized limestones are the characteristic lithology. There are occasional calc-schist bands and local meta-bauxite horizons. The deceptively flat-lying structure of the carbonate unit hides strong isoclinal folding prominent in the lower levels [2].

3. Engineering geological assessment of the rock masses in the project area

A detailed geological and engineering geological study was carried out in the project area. In order to constitute the geological model and to determine the engineering geological properties of the tunnel grounds, a total of 302 m of drilling was performed within eight boreholes along the Ilıksu 1 and Ilıksu 2 tunnels. A geological map with a scale of 1/2000 showing the borehole locations, and a geological cross-section along the Ilıksu 1 and 2 tunnel axes with a 1/2000 horizontal scale and a 1/500 vertical scale are presented in Fig. 3. A detailed discontinuity survey was carried out for each geologic unit and at the portals of the Ilıksu tunnels. Approximately 598 discontinuity data (joint data and bedding plane data) were measured in order to perform kinematic analysis.

The main rock types observed in the project area include recrystallized limestone, calc schist, pelitic schist, phyllite and intercalation of these units. The two tunnels in the project area, which are the Ilıksu 1 and 2 tunnels, are 529 and 165 m in length, respectively. An 81 m long cut slope section lies in between these two tunnels.

Pelitic schist, which is partly intercalated with calc schists, will be cut along the tunnel route at the exit (outlet) portal of the Ilıksu 1 tunnel and at the entrance portal of the Ilıksu 2 tunnel (Figs. 4A–C). It varies from weak chloritic schist to talcic schist in composition. This lithology is green to greenish gray, moderately to highly weathered, and possesses weak to moderately weak strength. It is easily separated along the foliation planes, which are highly persistent. The joint walls are slickensided and undulating according to the classification of ISRM [3]. Apertures are 0.20–0.25 mm wide. Average spacing of joints in the pelitic schist ranges between 10 and 60 mm. The uniaxial compressive strength (UCS) is classified as very low, with a measured range of 3–29 MPa. The rock quality designation (RQD) of the pelitic schist ranges between 0% and 81%.

Phyllite, which is mostly intercalated with calc schists and limestone bands, will be cut in the central part of the Ilıksu 1 tunnel and also along most part of the Ilıksu 2 tunnel. Phyllite is the most problematic unit of the study area. This lithology is blackish to dark gray, possesses weak to moderately weak strength, and is moderately to highly weathered. In phyllites, discontinuity planes are generally very well developed and rock masses are highly heterogeneous (Figs. 4D and E). Spacing of the foliation planes is generally fine (<6 mm). Foliation planes generally have 0.50–2.5 mm wide apertures. Discontinuity surfaces are slickensided with occasional calcite and clay infilling and possess medium persistence according to ISRM [3]. The joint systems are not well developed. Therefore, continuity of phyllite is generally limited to the foliation planes. As a result of intense foliation and the low resistance mineralogical content of phyllite, it is likely to posses less resistance against natural effects, especially water. Near the surface, it is highly sheared, deformed and has very poor quality rock mass properties. The UCS of the rock is classified as low, with a measured range of 16–50 MPa. The RQD of phyllite ranges between 0% and 62%.

Calc schist, which is observed to be intercalated with limestone bands and phyllite (Figs. 4A and C), will be cut along most parts of the Ilıksu 2 tunnel (km: 128 + 155–128 + 368). Calc schist is light to dark gray and almost always includes calcite veins. The foliation planes and rare joint systems control the discontinuous nature of the rock mass. It is slightly to moderately weathered and has moderate to high strength. Although

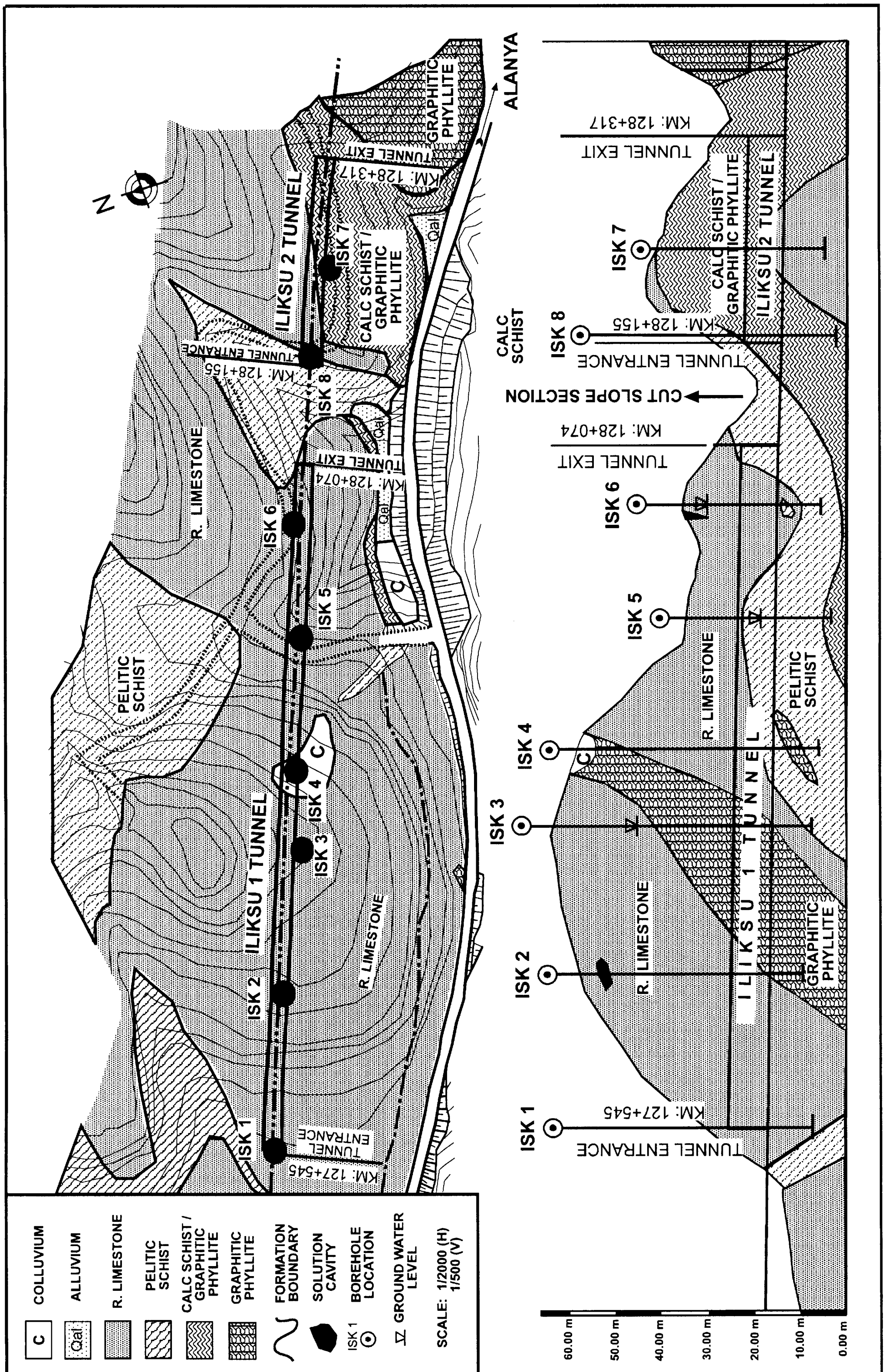


Fig 3. Geological map and cross section of the Ilıksu tunnels.

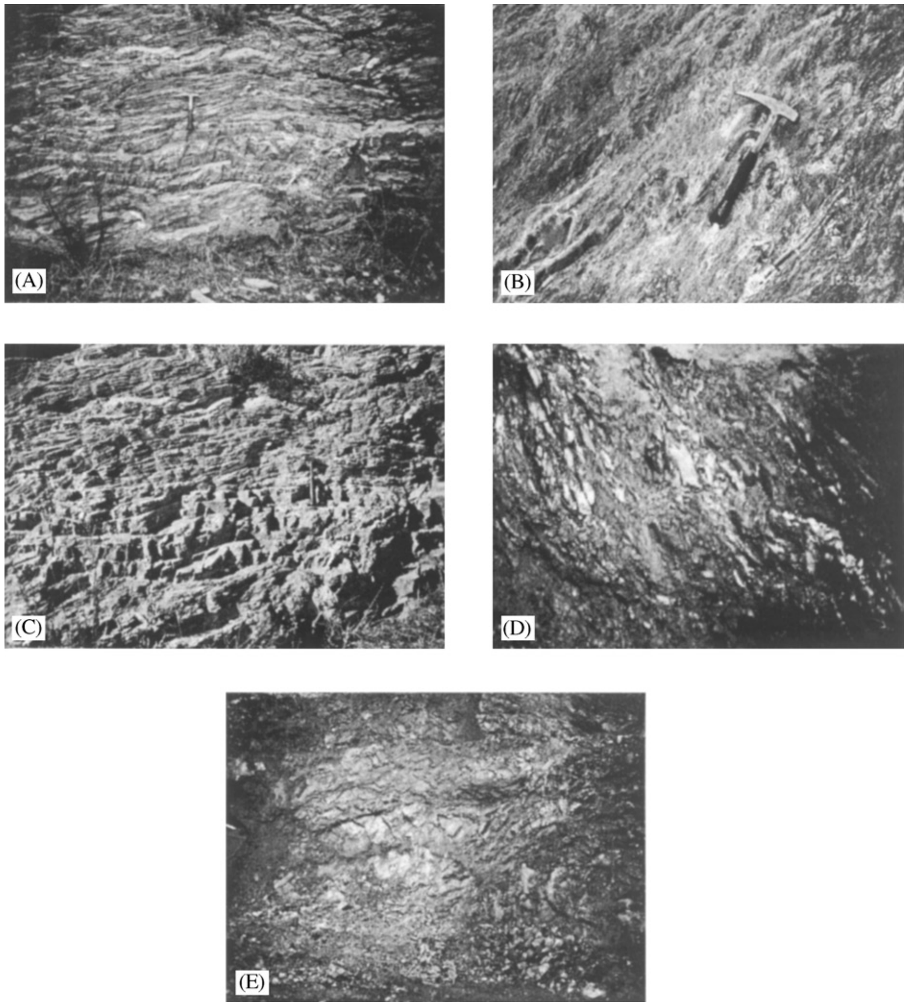


Fig. 4. Major lithologies of the schist unit. (A) Intercalation of the pelitic schist and thin calc schist layers by intense folding and tectonism (note that they consist of lensified hard rock bodies). (B) Tectonically deformed, folded, laminated and foliated pelitic schist unit. (C) Thin pelitic schist and thick calc schist layers of different competence that are differentially sheared and folded (note that the highly heterogeneous rock masses also consist of lensified hard rock bodies). (D, E) Graphitic phyllite. A heterogeneous rock mass comprising of highly foliated, sheared, and very deformable soil-like lithologies (note that it consists of floating lensified hard rock bodies).

the strength, durability and RQD results of the calc schists are generally high, they give low rock quality results due to intercalation with pelitic schists and phyllites. The UCS is measured to range between 62 and 92 MPa. Joint walls are open, irregular and undulating [3] and contain phyllite infillings. Spacing of the calc schist joint walls range from 20 to 60 cm. The RQD of the calc schist ranges between 0% and 20%.

Recrystallized limestone is exposed in most parts of the project area and generally overlies all of the units. It will be crossed along the tunnel route at the entry part of the Ilıksu 1 tunnel and will continue partly through the outlet portal of Ilıksu 1. It is also exposed at the central part of the Ilıksu 2 tunnel (Fig. 5). Karstic features are locally observed in limestone. Solution features such as sinkholes, rifts and solution cavities range from 1 cm up to a few meters (Fig. 5). In the recrystallized limestone unit, rock masses are often very blocky. However, at some locations along Ilıksu 1, the UCS and the quality of the rock masses decrease due to intense folding, karstic nature of the limestone and the presence of schistosity bands. The UCS of the recrystallized limestone is high, with a range of 30–117 MPa. The RQD values range between 61% and 73%. Recrystallized limestone is gray to light gray, strong, and slightly weathered. The joint planes have spacing ranging between 60 and 200 cm which is classified as wide according to ISRM [3].

To determine the necessary geomechanical parameters for tunnel design, rock mechanics testing was performed on samples obtained from the 8 rotary core borings drilled in the study area. Approximately 37–38 good quality core samples were obtained from limestone (i.e., karstic, intraformational, etc.). Although many core samples were obtained from phyllite, pelitic schist and calc schist, only a few of them were of intact core

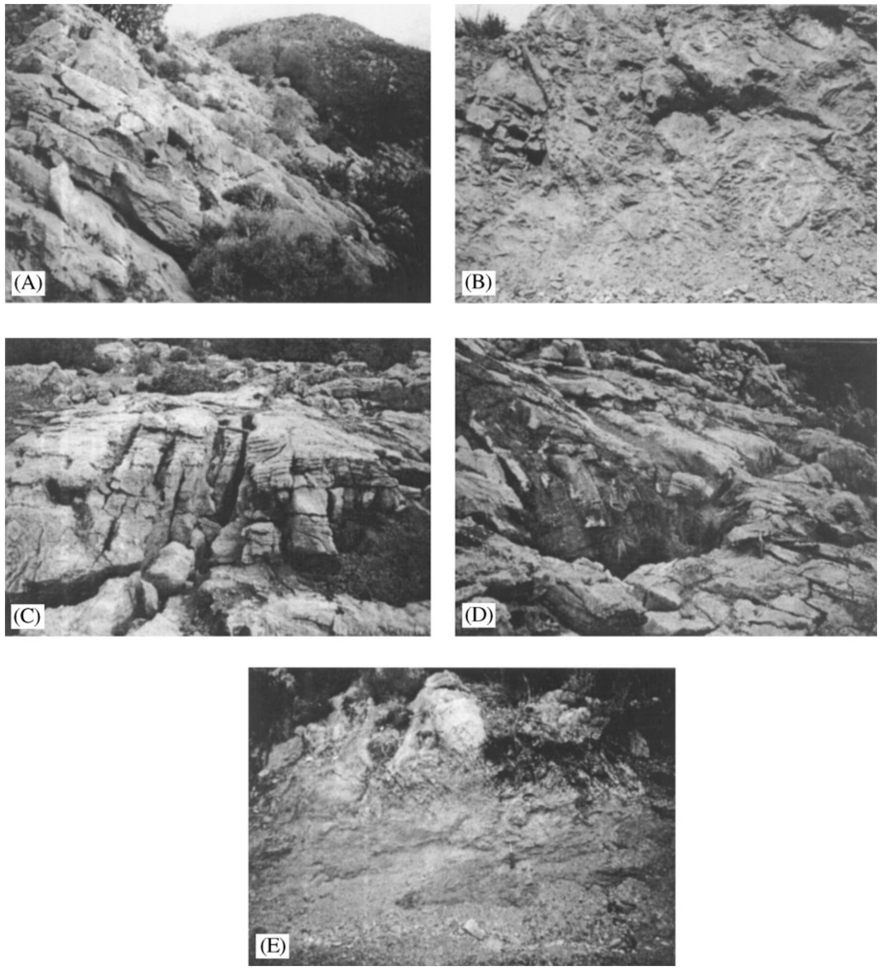


Fig. 5. Recrystallized limestone unit. (A) Blocky rock mass structure of recrystallized limestone. Rock mass quality of the blocky limestone decreases by folding (B), karstic nature (C, D) and the presence of schistosity bands (E).

quality (length of core in pieces > 100 mm) for performing UCS testing in the laboratory. The appearance of such core samples is given in Fig. 6. In order to overcome the difficulties in laboratory testing, rock mass classification systems, and in particular the GSI was used to assess the required geomechanical parameters. Table 1 presents the results of the geomechanics tests that were performed by the Turkish General Directorate of Highways Technical Research Department, Soil Mechanics and Tunnel Section.

4. Rock mass classification and empirical underground support design for the Ilıksu tunnels

The geotechnical properties of the units comprising the area were assessed using three empirical rock mass classification systems, namely the Q-system [4], the rock mass rating (RMR) method [5] as well as New Austrian Tunneling Method (NATM) by using correlations with the RMR and Q-systems according to the procedure given by the Turkish General Directorate of Highways [6]. Each classification system has been applied to classify the rock mass at the individual boring locations (i.e., boring numbers ISK 1–8). A summary of the results of the rock mass classifications at the individual boring locations along the Ilıksu tunnels is presented in Table 2.

A summary of the empirical temporary support systems (including rock bolt, shotcrete, wiremesh and steel sets) according to RMR, Q-system and NATM are briefly summarized in Table 3. In this table, support requirements are simply assembled within three categories of the rock mass: Fair Quality/B1 (Q range: 10–4), Fair Quality/B2 (Q range: 4–1) and Poor Quality/B3 (Q range: 1–0.1).

5. Determination of the rock mass strength with the Hoek–Brown failure criterion

The Hoek–Brown failure criterion was first introduced to estimate the strength of hard rock masses for the design of underground excavations [7]. Due to the

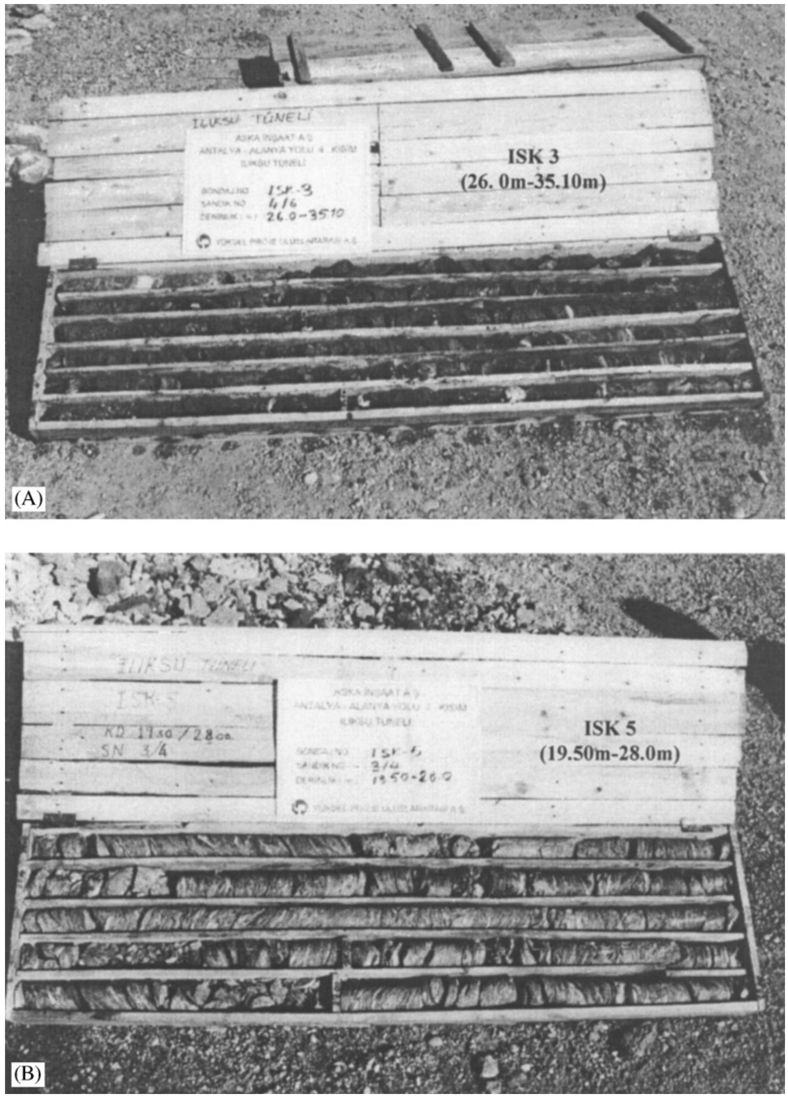


Fig. 6. A view of core samples that were obtained from graphitic phyllite (A), pelitic schist (B). Note that almost none of the cores are of intact quality to be tested in laboratory.

limited applicable alternatives, the original criterion has been changed and modified over the years, and has been applied to a variety of rock masses including very poor quality rock masses [8]. A new classification technique named as the GSI was also introduced into the criterion [9,10] which was used in concordant with the Geomechanics Classification System [5]. Determination of the strength of closely jointed, foliated and heterogeneous weak rock masses is hardly possible because it is not always possible to recover representative core samples that are large enough to be tested in the laboratory (Fig. 6). In order to overcome these difficulties, GSI has been extended for very poor quality of rock masses [11], extremely poor quality schistose rock masses and phyllites [12] as well as heterogeneous rock masses [13]. The GSI is a practical system and depends on the visual impression of the rock structure to estimate the strength of the rock mass for various geological conditions by using field inspection.

After Hoek and Brown updated and modified some aspects of the practical applications of the criterion, the criterion basically depended on three input parameters to estimate or measure the strength and deformability of the rock masses. These are the UCS of the intact rock pieces in the rock mass (σ_{ci}), value of the Hoek–Brown constant for these intact rock pieces (m_i) and the GSI of the rock mass [11].

In the project area, schist units and a thick sequence of recrystallized limestone, which are the members of the Yumrudağ Nappe, are observed. The major lithologies of the very poor quality schist units are weak, laminated, foliated, tectonically deformed, highly heterogeneous and differentially sheared pelitic schists, calc schists and intercalation of these lithologies (pelitic schist/calc schist

Table 1
Laboratory test results along with sample location and sample depth at the project area

Borehole no.[a]	Sample depth (m)	Uniaxial compressive strength, UCS (MPa)	Modulus of elasticity, E (GPa)	Poisson's ratio, υ	Unit weight, γ_t (kN/m^3)	Sample description
ISK 1	4.60–4.75	116	—	—	26.38	R. Limestone
ISK 2	15.80–16.10	68	—	—	26.48	R. Limestone
ISK 2	23.35–23.50	30	23	—	26.58	R. Limestone
ISK 2	43.40–43.63	87	—	—	—	Phyllite
ISK 2	44.45–44.70	65	—	—	26.58	Phyllite
ISK 3	16.00–16.20	117	—	—	—	R. Limestone
ISK 3	24.45–24.60	104	40	0.42	26.77	Phyllite
ISK 3	33.35–33.50	41	—	—	—	Phyllite
ISK 3	33.85–34.00	84	—	—	—	Phyllite
ISK 3	35.90–36.10	72	—	—	26.18	Phyllite
ISK 3	36.40–36.50	49	37	0.07	—	Phyllite
ISK 4	20.05–20.25	28	—	—	—	R. Limestone
ISK 4	23.75–23.90	42	—	—	—	R. Limestone
ISK 4	28.95–29.20	62	30	0.07	—	R. Limestone
ISK 4	32.75–33.00	64	32	0.22	—	R. Limestone
ISK 4	33.75–34.00	112	—	—	25.79	R. Limestone
ISK 4	40.00–40.15	29	—	—	—	Pelitic schist
ISK 4	41.50–41.65	6	—	—	—	Pelitic schist
ISK 4	43.50–43.85	3	—	—	—	Pelitic schist
ISK 5	8.75–9.00	90	61	0.43	26.28	R. Limestone
ISK 5	28.30–28.45	3.5	1.0	—	—	Pelitic schist
ISK 7	16.70–16.90	92	88	0.40	26.18	Calc schist
ISK 7	18.80–19.05	62	—	—	—	Calc schist
ISK 7	24.00–24.20	65	—	—	—	R. Limestone
ISK 7	27.20–27.45	85	—	—	26.28	R. Limestone
ISK 8	15.25–15.50	16	—	—	—	Phyllite
ISK 8	17.00–17.25	50	26.6	0.05	27.07	Phyllite

[a] Borehole locations are presented in Fig. 3.

and graphitic phyllite/calc schist) along with graphitic phyllites and pelitic schists comprising highly foliated, sheared, heterogeneous and very deformable soil-like lithologies (Fig. 4). The recrystallized limestone rock masses are generally very blocky (Fig. 5). At some locations along the Ilıksu 1, the rock mass quality of the blocky limestone decreases due to deformation and disturbance caused by intense folding, due to the karstic nature of limestone, or due to the presence of schistosity bands (Fig. 5). The GSI values of these lithologies corresponding to the rock mass quality at each borehole location (through using the extended GSI table [14,15]) are denoted by circles in Table 4. According to this table, the rock mass at borehole locations ISK 1, 2 and 4 is classified as fair quality, at ISK 6 as poor quality and at ISK 3, 5, 7 and the exit portal of the Ilıksu 2 tunnel as very poor quality.

The Hoek–Brown input parameters σ_{ci}, m_i and the ranges of GSI values corresponding to the rock mass quality at each borehole section along with the rock mass disturbance factor, D [16], an estimate of the geomechanical properties (i.e., modulus of deformation, rock mass strength, etc.) and post-failure behavior of the rock masses are tabulated in Table 5. By using the relationship between the Hoek–Brown and Mohr–Coulomb criteria [11], the shear strength parameters of the rock mass at each borehole location were obtained and are presented in Table 5. To determine the necessary rock mass strength parameters based on the generalized Hoek–Brown failure criterion, the RocLab [17] software with the tunnel application option was used. Tunnel depths and average unit weights were estimated from Fig. 3 and Table 1, respectively.

The post-failure characteristics of fair quality rock masses may correspond to an assumed strain-softening behavior. Typical geomechanical properties of the fair quality rock mass observed at borehole locations ISK 1, 2 and 4 along the Ilıksu 1 tunnel route are presented in Table 5. Fair quality rock masses are very well interlocked and are typically characterized as blocky rock masses with three joint sets plus random. These are structurally the highest members of the Asmaca formation, which have been classified as very blocky according to the GSI (Fig. 5A). They include slightly weathered, medium strong to strong rock types such as a thick sequence of recrystallized limestone and

Table 2
Summary of the rock mass classification results in the project area

Tunnel location	Tunnel section (km)	Lithology	Borehole no.	Section length (m)	Q-system	RMR	NATM
	127 + 545 – 127 + 585	R. Limestone	ISK 1 (Entrance portal)	40	2.107	53	B2
	127 + 585 – 127 + 675	R. Limestone	ISK 2	90	7.305	59	B1
	127 + 675 – 127 + 685	R. Limestone/Phyllite	[a]Transition zone	10	7.305	59	B2
Ilıksu 1	127 + 685 – 127 + 790	Phyllite	ISK 3	105	0.234	31	B3
	127 + 790 – 127 + 800	Phyllite/R. Limestone	[a]Transition zone	10	0.234	31	B2
	127 + 800 – 127 + 860	R. Limestone/Pelitic schist	ISK 4	60	5.693	58	B1
	127 + 860 – 127 + 870	Pelitic schist/R. Limestone	[a]Transition zone	10	5.693	58	B2
	127 + 870 – 127 + 990	R. Limestone/Pelitic schist	ISK 5	120	0.971	31	B3
	127 + 990 – 128 + 074	R. Limestone/Pelitic schist	ISK 6 (Exit portal)	84	0.571	46	B3
Cut slope Section	128 + 074 – 128 + 155	Pelitic schist	ISK 6	81	0.571	46	B3
	128 + 155 – 128 + 195	Phyllite/Calc/Pelitic schist/ R. Limestone	ISK 8 (Entrance portal)	40	0.900	31	B3
Ilıksu 2	128 + 195 – 128 + 275	Phyllite /Calc/Pelitic schist	ISK 7	80	0.125	31	B3
	128 + 275 – 128 + 317	Phyllite/Calc schist	Exit portal	42	0.804	31	B3

[a] During the NATM operations of the Ilıksu 1 tunnel, 10 m B2 type support class is proposed to constitute a transition zone between the B1 and B3 support system of the rock mass classes. This support is applied to reduce installation difficulties between support types.

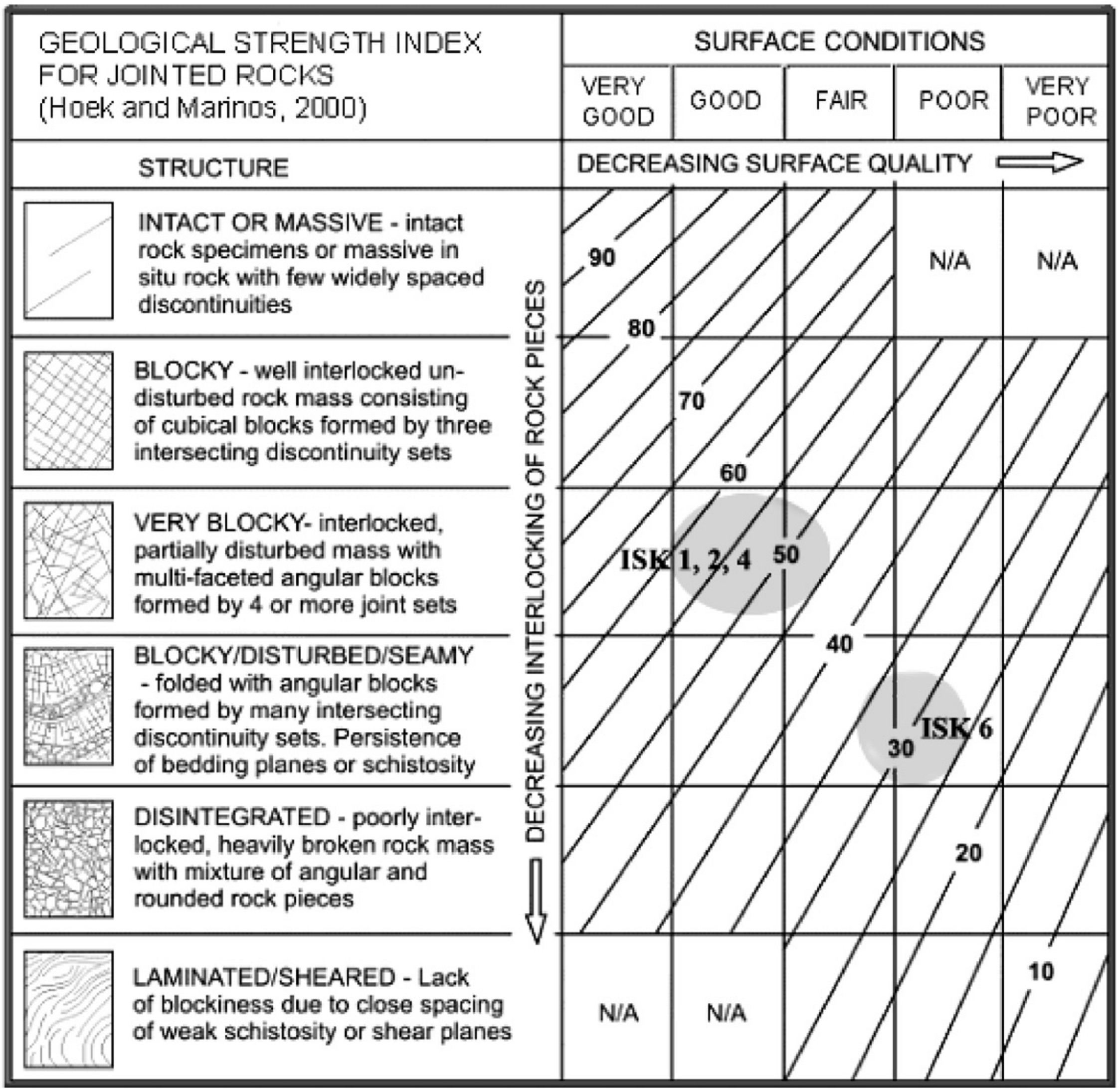
Table 3
Summary of the empirical support type and related support requirements for the Ilıksu tunnels project

Support type	B1/4 < Q < 10/fair rock	B2/1 < Q < 4/fair rock	B3/0.1 < Q < 1/poor rock
Examined portion	Ilıksu 1 (31%)	Ilıksu 1 (13%)	Ilıksu 1 (56%), Ilıksu 2 (100%)
Construction phase	Top heading and bench	Top heading and bench	Top heading and bench
Excavation method	Drill and blast	Smooth blasting, roadheaders if required	Smooth blasting, roadheaders if required
Round length	Top heading (2.0–3.0 m) and bench (4.0 m)	Top heading (1.5–2.5 m) and bench (3.5 m)	Top heading (1.5–2.0 m) and bench (2.5 m)
Stand-up time	2–4 days	5–10 h	2 h
Support time	Commence support after each blast	Commence support after each blast	Commence support after each blast
NATM, RMR and Q-system support requirements	Shotcrete (100 mm) + wiremesh (1 layer) + systematic grouted bolting SN type ($\Phi = 26$ mm, $L = 4$ m, spacing 2.0×2.5 m) + spot SN type bolting (if required in lower half)	shotcrete (150 mm) + wiremesh (1 layer) + systematic grouted bolting SN type ($\Phi = 26$ mm, $L = 4$ m, spacing 2.0×2.0 m) + spot SN type bolting (if required in lower half)	shotcrete (250 mm) + wiremesh (2 layer) + steel ribs (spacing1.0×1.0 m) + systematic grouted bolting SN type ($\Phi = 26$ mm, $L = 4$ m, spacing 1.0×2.0 m) + forepoling ($L = 4$ m, spacing 0.5×0.5 m, every two ribs)

intraformational conglomerate and a few meters thick meta-dolomite at the contact.

The poor quality rock mass in between km: 127 + 990 – 128 + 074 is a highly weathered rock mass with numerous karstic zones. The rock mass observed at borehole location ISK 6 might be described as blocky/disturbed, which ranges from fair to poor range of categories (Table 4a and Fig. 5). This category was assigned to rock masses comprising karstic limestone. Within this unit, schistosity bands and intraformational

Table 4
Estimated GSI values for each borehole location of the project area [14,15]
(a)



(b)

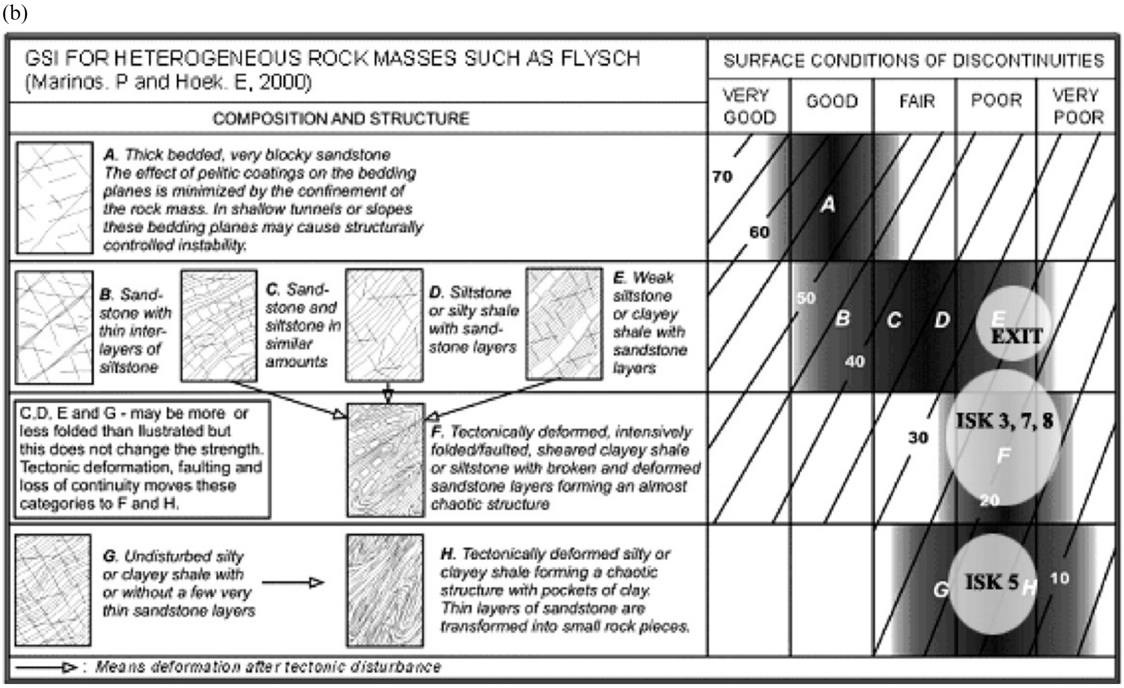


Table 5
Geomechanical properties of fair quality, poor quality and very poor quality rock mass for each borehole section at the project area as determined by RocLab [17] (sample depths are given in Table 1)

Geomechanical properties	ISK 1	ISK 2	ISK 3	ISK 4	ISK 5	ISK 6	ISK 8	ISK 7	Ilıksu 2 exit portal
Intact rock strength (σ_{ci}) (MPa) (Average results from Table 1 and after [11])	75	63	15	58	8	45	35	5	15
Hoek–Brown constant (m_i)	10	10	6	9	8	8	7	6	7
Geological strength index (GSI) (from Table 4)	50±5	55±5	21±3	50±5	18±5	30±5	22±5	18±3	21±5
Disturbance factor, D	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Hoek–Brown constant (m_b)	0.925	1.173	0.139	0.832	0.161	0.285	0.171	0.121	0.163
Hoek–Brown constant (s)	0.0013	0.0025	2.66e–5	0.0013	1.79e–5	0.0001	3.04e–5	1.79e–5	2.66e–5
Constant (a)	0.506	0.504	0.541	0.506	0.550	0.5220	0.538	0.550	0.541
Friction angle (ϕ')	57°	53°	23°	51°	24°	42°	33°	20°	28°
Cohesive strength (c'), MPa	0.357	0.487	0.067	0.363	0.036	0.121	0.085	0.023	0.049
Global strength (σ_{cm}), MPa	9.553	9.209	0.504	7.019	0.312	2.832	1.520	0.167	0.624
Rock mass tensile strength (σ_{tm}), MPa	0.103	0.133	0.003	0.089	0.001	0.014	0.006	0.001	0.002
Uniaxial compressive strength (σ_c), MPa	2.575	3.061	0.050	1.991	0.020	0.344	0.130	0.012	0.050
Deformation modulus (E_m), MPa	6495	7938	547	5712	336	1591	885	266	547
Poisson's ratio (v) (estimated from Table 1)	0.25	0.25	0.30	0.25	0.30	0.27	0.30	0.30	0.30
Dilation angle ($\varphi°$)[a]	7°	6.6°	0°	6.3°	0°	2.6°	1°	0°	0°
Post-failure behavior of the rock masses	Fair quality	Fair quality	Poor to very poor quality	Fair quality	Poor to very poor quality	Poor quality	Poor to very poor quality	Poor to very poor quality	Poor to very poor quality

[a] The post-peak or post failure characteristics of the rock masses are determined by using experience in numerical analysis on a variety of practical problems [11].

conglomerate levels were also observed. Post-failure behavior of this section may be assumed to correspond to an elasto-plastic behavior (Table 5).

The post-failure characteristics of the very poor quality rock masses may be assumed to be perfectly plastic since this rock mass category possesses weak to moderately weak strength due to its moderately to highly weathered state. Typical geotechnical properties for this quality rock mass observed in borehole locations ISK 3 and ISK 5 of the Ilıksu 1 tunnel location, and ISK 7, 8 and the exit portal of the Ilıksu 2 tunnel route are presented in Table 5. Examples of such very poor quality rock masses are calc schist, pelitic schist and graphitic phyllite of the Asmaca formation which show intense shearing and deformation along the lamination or foliation planes. Graphitic phyllites and pelitic schists are also foliated, laminated and sheared chaotic rock masses consisting of floating lensified hard rock bodies in a soil-like environment. Alternations of the pelitic schist/calc schist and graphitic phyllite/calc schist lithologies form heterogeneous, differentially sheared, folded and laminated weak rock masses of variable competence. These rock mass groups have been categorized according to the recently compiled GSI table for heterogeneous rock masses [14,15] which is presented in Table 4b. Photos of these group of rock masses are presented in Fig. 4.

6. Slope stability analyses of portal, side or cut slopes

6.1. Theoretical background

The most important parameter in the stability analysis of slopes is to determine safe slope geometries with minimum support, excavation, benching, deformation and/or modification of the natural topography. In order to decide on the most suitable geometry and to determine the stability of the portal, side or cut slope sections, slope stability analyses were performed. This section of the paper consists of a detailed geotechnical evaluation of the entrance and exit portal structures of the Ilıksu 1 and Ilıksu 2 tunnels, analysis of the stability of the cut slope section in between the Ilıksu 1 and Ilıksu 2 tunnels and determination of the required support systems of all cut slopes. The gradients of the cut slopes above and along the sides of the portal locations were proposed by the Contractor as 1/5 (h:V) and 1/3 (h:V),

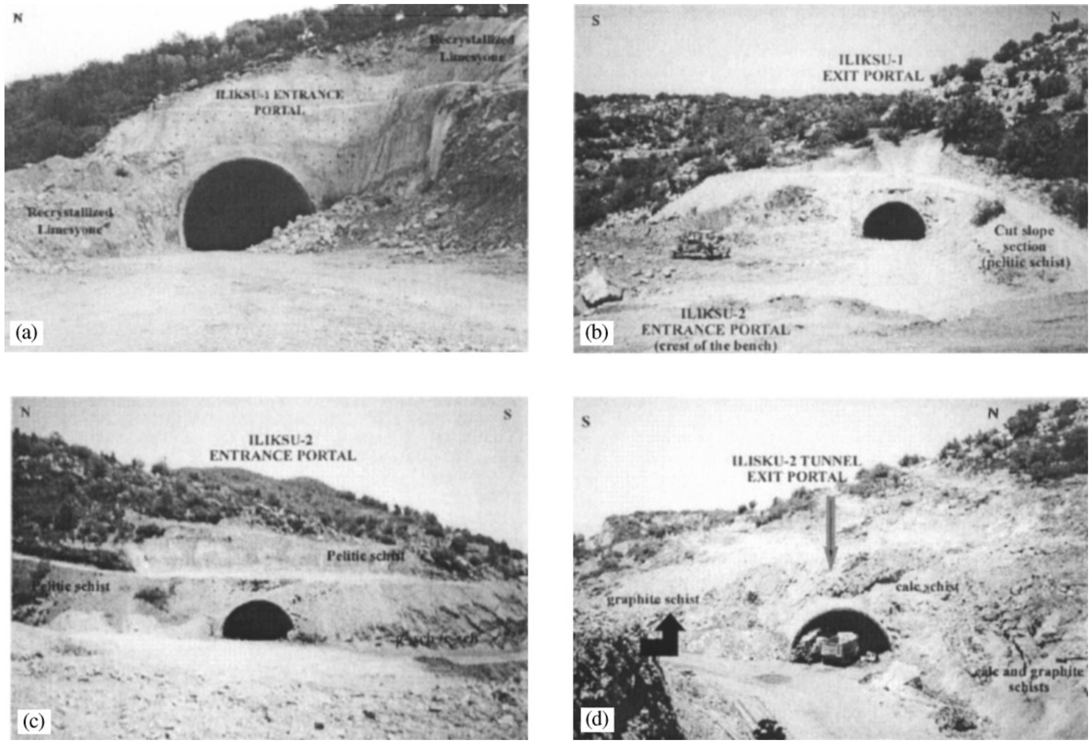


Fig. 7. General view of the portals: (a) entrance portal of the Ilıksu 1 tunnel (km: 127 + 545), (b) exit portal of the Ilıksu 1 tunnel (km: 128 + 074), (c) entrance portal of the Ilıksu 2 tunnel (km: 128 + 155) and (d) exit portal of the Ilıksu 2 tunnel (km: 128 + 317).

respectively. Fig. 7 presents a general view of the portals. In order to check the stability of the proposed cut slopes consisting of regularly jointed rock masses, kinematic analysis was initially performed to determine the type and possibility of the occurrence of any kinematic failure. After that, limit equilibrium analyses were performed on the regularly jointed rock slopes in order to pinpoint any instability problems with the incorporation of water pressure. The shear strength parameters (cohesion, c' and, internal friction angle, ϕ'; from Table 5) and the unit weight of the rock masses (from Table 1) were used to perform these analyses.

6.2. Slope stability analyses of portals comprising regularly jointed rock masses

Along the entrance and exit portals of Ilıksu 1, kinematical analyses were only applied to regularly jointed and structurally controlled recrystallized limestone (Figs. 7a and b). The results of the kinematic analyses (Fig. 8) showed that only wedge failure was possible on the north cut slope of the Ilıksu 1 tunnel entrance portal as determined by the DIPS Software [18]. Planar failure was possible on the north cut slope of the Ilıksu 1 tunnel entrance portal. In addition, the south cut slope also showed planar failure as determined by DIPS (Figs. 8 and 9).

Hoek and Bray [19] present a limit equilibrium procedure for wedge failure analysis that takes into account the effects of cohesion and water pressure. For limit equilibrium analyses of critical slopes adjacent to highway roads and critical engineering excavations (i.e., tunnels, open mine pits, etc.), a factor of safety (F) of 1.50 is usually preferred [19,20]. The factor of safety is calculated from the following equation:

$$\begin{aligned} F = {} & 3/\gamma H(c_A X + c_B Y) + (A - (\gamma_\mathrm{w}/2\gamma)X)\ \mathrm{Tan}\ \phi_A \\ & + (B - (\gamma_\mathrm{w}/2\gamma)Y)\ \mathrm{Tan}\ \phi_B, \end{aligned} \tag{1}$$

where c_A and c_B are the cohesive strengths of planes A and B along the base of the wedge; ϕ_A and ϕ_B are the angles of internal friction on planes A and B along the base of the wedge; γ is the unit weight of the rock mass; γ_w is the unit weight of water; H is the height of the wedge; X, Y, A, and B are dimensionless factors and depend on the geometry of the wedge and the slope; and, the angles ψ_a, ψ_b, ψ_5 required for calculating the coefficients X, Y, A and B are measured from the stereoplot according to Hoek and Bray [19].

Hoek and Bray [19] present a limit equilibrium procedure for plane failure analysis that accounts for the effects of cohesion (c'), angle of internal friction (ϕ') and water pressures (U, V; Fig. 10) for a slope with a

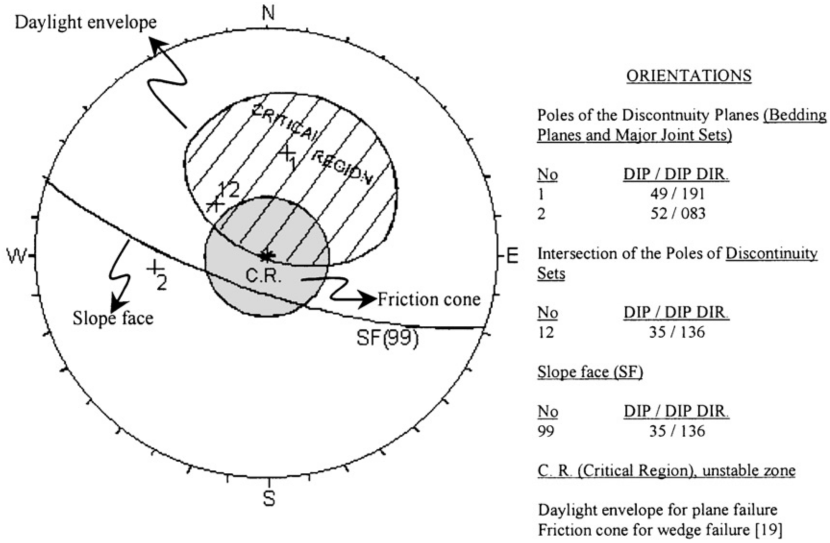


Fig. 8. Kinematic demonstration of planar and wedge failure possibility on the north cut slope of the Ilıksu 1 tunnel entrance portal using DIPS software.

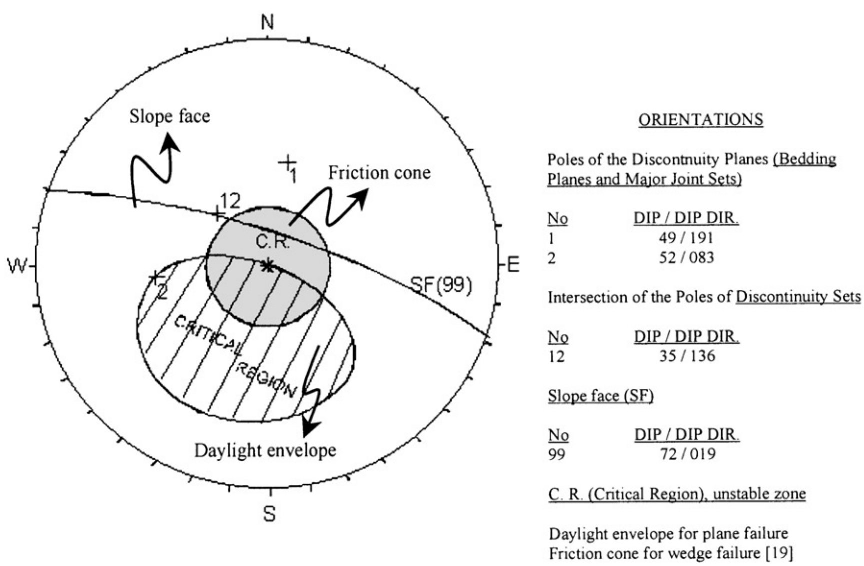


Fig. 9. Kinematic demonstration of planar failure possibility on the south cut slope of the Ilıksu 1 tunnel entrance portal using DIPS software.

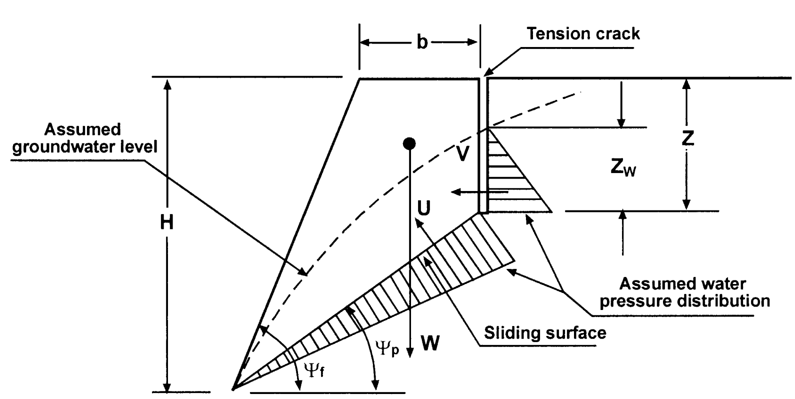


Fig. 10. Geometry of a slope with tension crack [19].

Table 6
Summary of the stability results for the portal faces of the Ilıksu tunnels consisting of weak rock masses

Section	Lithology	Soil properties (from Tables 1 and 5)	FS with SLOPE/W (simplified Bishop)	F.S. with PLAXIS 7.2 (Phi-c reduction by using Standard Coulomb)	Stability condition
Ilıksu 1 exit portal	Pelitic schist	$\gamma = 26.65\,\mathrm{kN/m^3}$, $\phi = 23°$, $c = 78\,\mathrm{kPa}$	1.34	1.31	Unstable
Ilıksu 2 entrance portal	Phyllite, calc schist, pelitic schist	$\gamma = 25.0\,\mathrm{kN/m^3}$, $\phi = 18°$, $c = 42\,\mathrm{kPa}$	0.940	0.930	Unstable
Ilıksu 2 exit portal	Calc schist and phyllite	$\gamma = 25.0\,\mathrm{kN/m^3}$, $\phi = 22°$, $c = 70\,\mathrm{kPa}$	1.28	1.23	Unstable

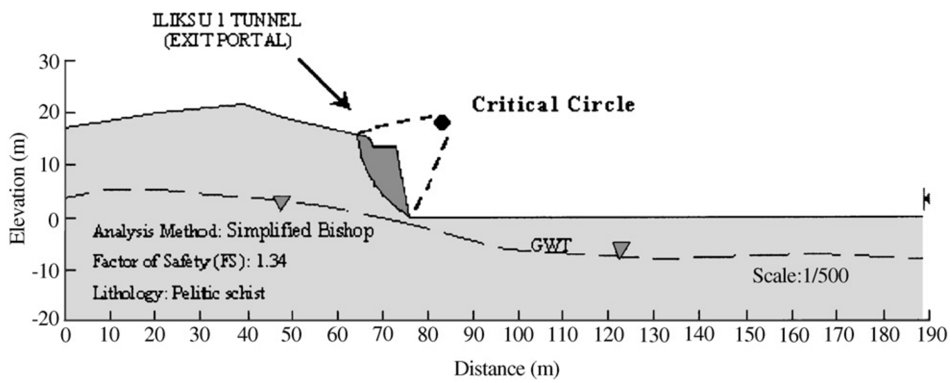


Fig. 11. Stability analysis of the exit portal slope face of the Ilıksu 1 tunnel using Slope/W.

tension crack through the following equation:

$$F = \frac{c'A + [W \operatorname{Cos} \Psi_p - U - V \operatorname{Sin} \Psi_p] \operatorname{Tan} \phi'}{W \operatorname{Sin} \Psi_p + V \operatorname{Cos} \Psi_p}, \tag{2}$$

where from Fig. 10; A is the length of the discontinuity plane; H the height of the block; z the depth of the tension crack; Ψ_f the dip of the slope face; Ψ_p the dip of the discontinuity plane; W the weight of the planar block/unit width; z_w the depth of water in the tension crack; V the lateral water pressure in the tension crack/unit width; U' the uplift water pressure/unit width; c' the cohesion; and, ϕ the internal friction angle.

The results of the side slope wedge limit equilibrium analyses for saturated conditions (using Eq. (1)) showed that the factor of safety (F) was determined to be greater than 1.50 ($F = 3.1$). Hence, wedge failure is not expected for the north cut entrance portal slope of the Ilıksu 1 tunnel and a slope of 1/3 (h:V) is safe even under saturated conditions. Upon performing the limit equilibrium analyses for plane failure using Eq. (2), the factor of safety was determined to be greater than 1.5 for saturated conditions ($F = 1.7$ and 1.8 on the north and south cut slopes, respectively). Therefore, for the north and south cut portal slopes of Ilıksu 1, planar failure is not expected. For conditions where the water table is at the bottom of the tension crack (i.e., for $V = 0$, but $U \neq 0$), the factor of safety for the north and south cut slopes increases to 2.0 and 2.1, respectively, which indicates that a slope of 1/3 (h:V) is safe for the side cut rock slopes.

6.3. Slope stability analyses of the portals and the cut slope section comprising weak rock masses

Slope stability analyses of irregularly jointed, highly foliated and very deformable soil-like lithologies (i.e., pelitic schists, graphite schists, phyllites, etc.) were analyzed by the slope stability software Slope/W [21] using Bishop's simplified method, and by the Phi-c reduction option of the finite element analysis software package PLAXIS 7.2 [22] assuming Standard Coulomb Condition, for which a global safety factor was evaluated for each portion of the slope. The rock mass properties of the lithologic units, obtained from borehole locations ISK 5, 6, 7 and ISK 8 are presented in Table 5.

The results of the analysis shows that slope stability problems are expected on the exit portal face of the Ilıksu 1 tunnel (Fig. 7b) and the entrance and exit portal faces of the Ilıksu 2 tunnel (Figs. 7c and d). The results are summarized in Table 6. Figs. 11 and 12 give examples of slope stability analyses of the Ilıksu 1 tunnel exit portal face. The results of the analysis shows that support systems should be provided at the Ilıksu 1 tunnel exit portal face and at the Ilıksu 2 tunnel entrance

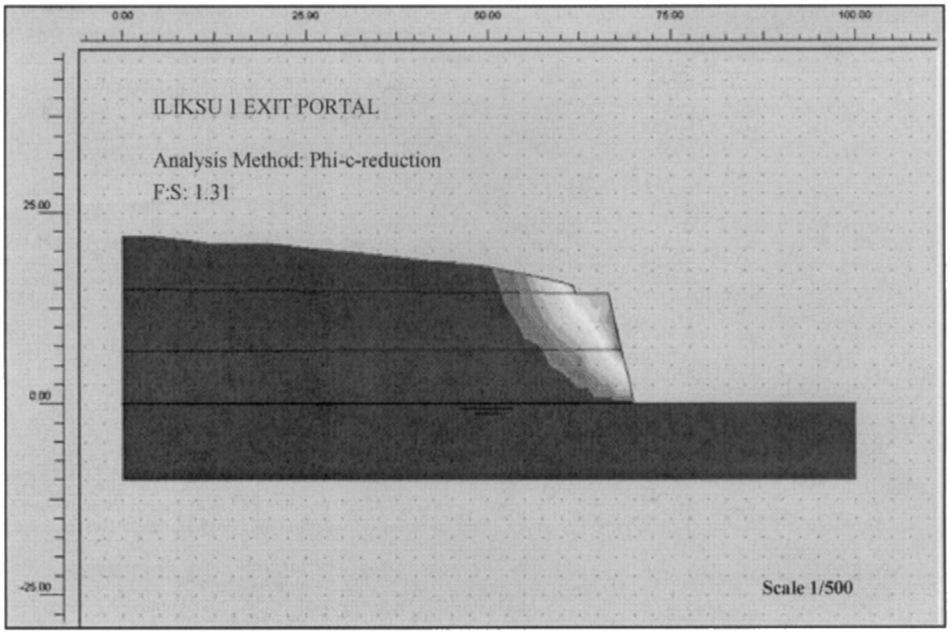


Fig. 12. Stability analysis of the exit portal slope face of the Ilıksu 1 tunnel according to the Phi-c-reduction analysis method of PLAXIS 7.2.

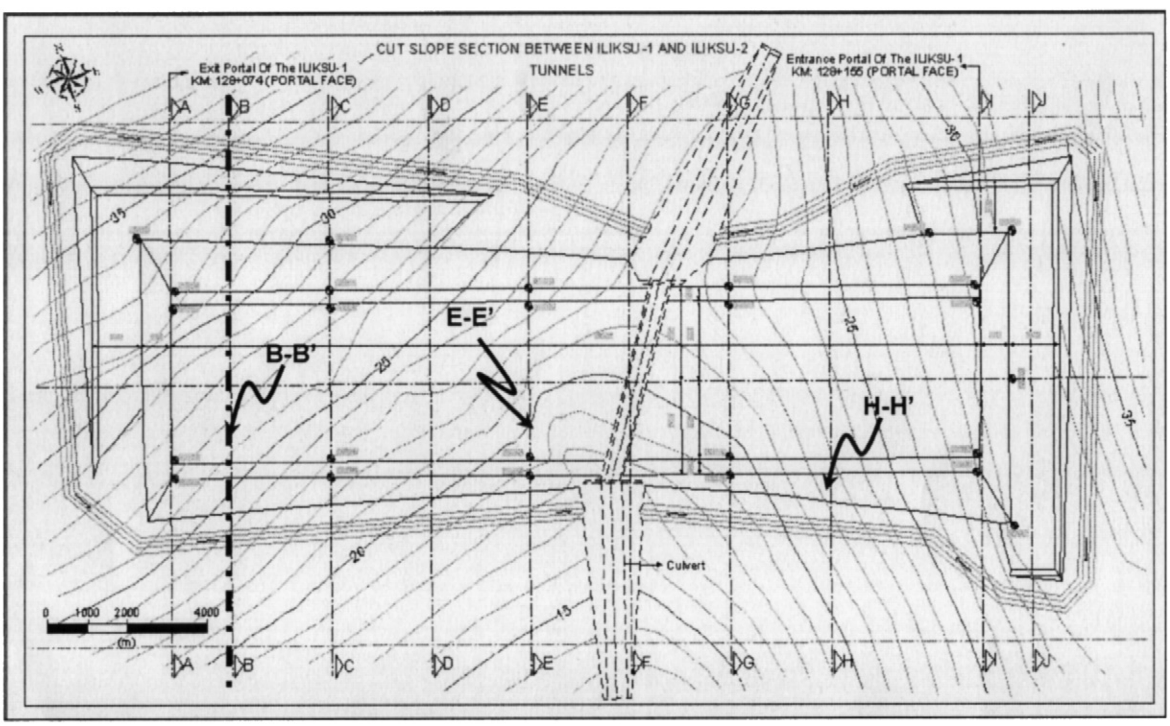


Fig. 13. Cut slope section in between the Ilıksu 1 and Ilıksu 2 tunnels comprising pelitic schist.

and exit portal faces. The support recommendations are discussed in the latter section.

Since the Ilıksu 1 exit portal, Ilıksu 2 entrance and exit portals are not stable ($F < 1.5$; Table 6), support systems should be applied to prevent circular failure at these locations. The lithology of the cut slope section in between the Ilıksu 1 and Ilıksu 2 tunnels consists of pelitic schist. According to the Contractor's excavation plan, there is no bench proposed at the entrance portal of Ilıksu 2 but one bench is proposed to be constructed at the north cut and portal face slopes of Ilıksu 1. In between the two tunnels, one bench is proposed to be constructed at the north cut slope section.

Table 7
Summary of the stability results for each cross-section along the cut slope section (sections B–B′, E–E′ and H–H′)

Cross-section	FS with SLOPE/W (simplified Bishop)	FS with PLAXIS 7.2 (Phi–c reduction by using Standard Coulomb)	Stability condition ($F>1.5$)
B–B′	1.25	1.23	Unstable
E–E′	1.49	1.45	Unstable
H–H′	1.75	1.71	Stable

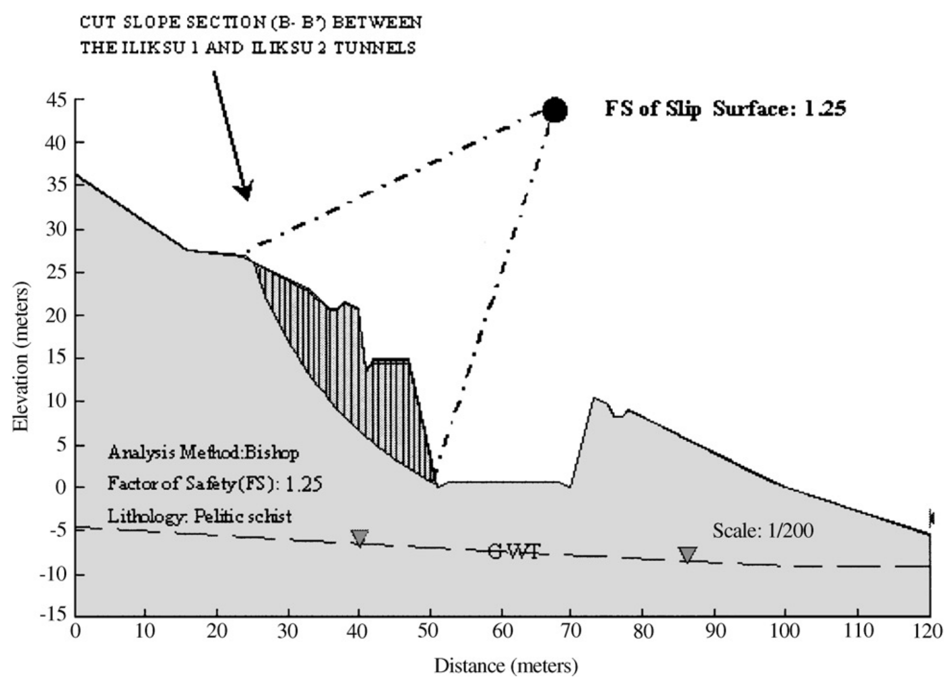


Fig. 14. Stability analysis of the cut slope section B–B′ between the Ilıksu 1 and Ilıksu 2 tunnels according to Slope/W.

The entire cut slope section is planned to be constructed in between km: 128 + 074 and 128 + 155. A plan view of the designed cut slope is given by Fig. 13. Stability of the cut slope section is analyzed by Slope/W and PLAXIS 7.2. In order to check the stability, three cross sections B–B′ (km: 128 + 080), E–E′ (km: 128 + 110) and H–H′ (km: 128 + 140) along the cut slope from west to east were analyzed. Table 5 (borehole locations ISK 5 and ISK 8) gives the rock mass properties of the pelitic schist that forms the lithology of the cut slope portion of the project area. The results of the analyses show that different stability conditions exist along each cross section studied. A summary of the stability results is tabulated in Table 7. Figs. 14 and 15 give examples of slope stability analyses of section B–B′.

Table 7 shows that each of the two methods gives very similar factor of safety results. According to the results presented in Table 7, stable and unstable zones exist along the 81 m long cut slope section. From east to west, the stability of the cut slope section decreases and two out of three sections considered shows instability ($F<1.5$). Therefore, support systems should be provided on the western part of the cut slope section to prevent slope instability. The following section discusses the support recommendations.

6.4. Recommended supports for the portal, side and cut slopes

The slope faces at the entrance and exit portals of the Ilıksu 1 and Ilıksu 2 tunnels along with the north and south side cut slopes of the portals should be supported for stability. The slopes of the portal faces are designed as 1/5 (h:V) and the cut (side) slopes of the portals are designed as 1/3 (h:V), aiming minimum excavation and least deformation of the natural topography. Recommended support types at the slope faces along with the north and south side cut slopes of the portals according to Turkish General Directory of Highways [6] are as follows:

- 10 cm shotcrete (apply as 5 + 5 cm layers) and wiremesh (one layer),

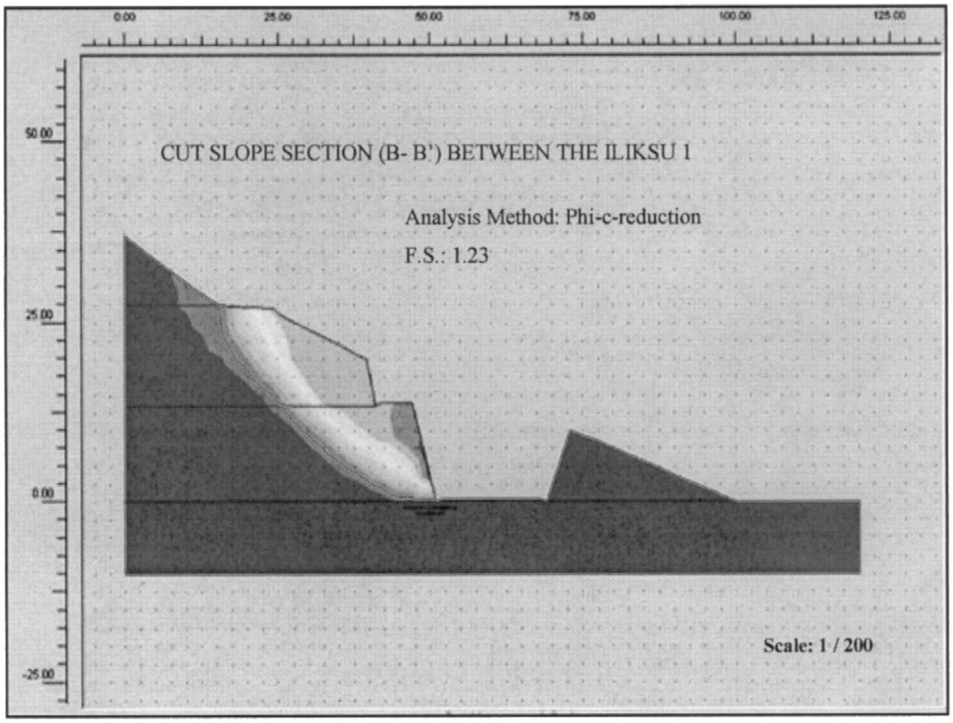


Fig. 15. Stability analysis of the cut slope section B–B′ between the Ilıksu 1 and Ilıksu 2 tunnels according to PLAXIS 7.2.

- rock bolting ($\Phi = 26$ mm, $L = 6.0$ m long, spacing 1.5×1.5 m).

As stated earlier, the eastern part of the 81 m cut slope section is analyzed to be stable. The recommended support types on the unstable western cut slope section are as follows:

- 10–15 cm shotcrete (apply as 5 + 5 or 5 + 10 cm layers) and wiremesh (one layer),
- rock bolting ($\Phi = 26$ mm, $L = 6.0$ m long, spacing 1.5×1.5 m).

6.5. Determination of the shear strength parameters of calc schist by using back analysis and comparison of the results with the GSI method

Approximately 1 km away from the Ilıksu 1 tunnel, the Çandırtepe tunnel (between km: 126 + 449 and km: 126 + 700) has been constructed. This tunnel is included in the Alanya–Antalya road (Section IV) construction project. Towards the west cut slope of the Çandırtepe tunnel portal entrance (km: 126 + 449), a failure surface has developed most probably due to insufficient support and stress relief during portal excavation (Fig. 16). The lithology of the Çandırtepe tunnel portal and its surroundings is calc schist. Calc schist is regularly bedded and does not show intense intercalation with highly foliated, laminated, sheared and very deformable soil-like lithologies (i.e., graphitic phyllite, pelitic schist). Two wedge types of failures that occur within each other were observed at the entrance portal slope of the Çandırtepe tunnel. One of them controlled the entire failure surface and the other one was locally developed within the larger wedge. Back analysis was applied to these wedges in order to determine the shear strength parameters (i.e., angle of internal friction (ϕ') and cohesion (c')). The major orientations of the slope face, upper slope face and discontinuity sets are given in Table 8.

The shear strength parameters (c' and ϕ') can be estimated by back analyses calculations from Eq. (1) as described by Hoek and Bray [19]. The results of the back analyses are plotted in Fig. 17. The curves originate from plotting c, ϕ pairs that satisfy limit equilibrium condition (i.e., $FS = 1.0$). The intersection points of the two graphs give $\phi' = 25^\circ$ and $c' = 58$ kPa (Fig. 17).

In the Ilıksu tunnel project, calc schist is generally intercalated with graphite schist and pelitic schist that are highly and irregularly foliated and possess very deformable soil-like lithologies. Although the RQD, strength and durability values of calc schists are relatively high, they give low rock quality results due to intercalation with pelitic schists and graphitic phyllite. Accordingly, the values of the shear strength parameters are low and very close to the shear strength

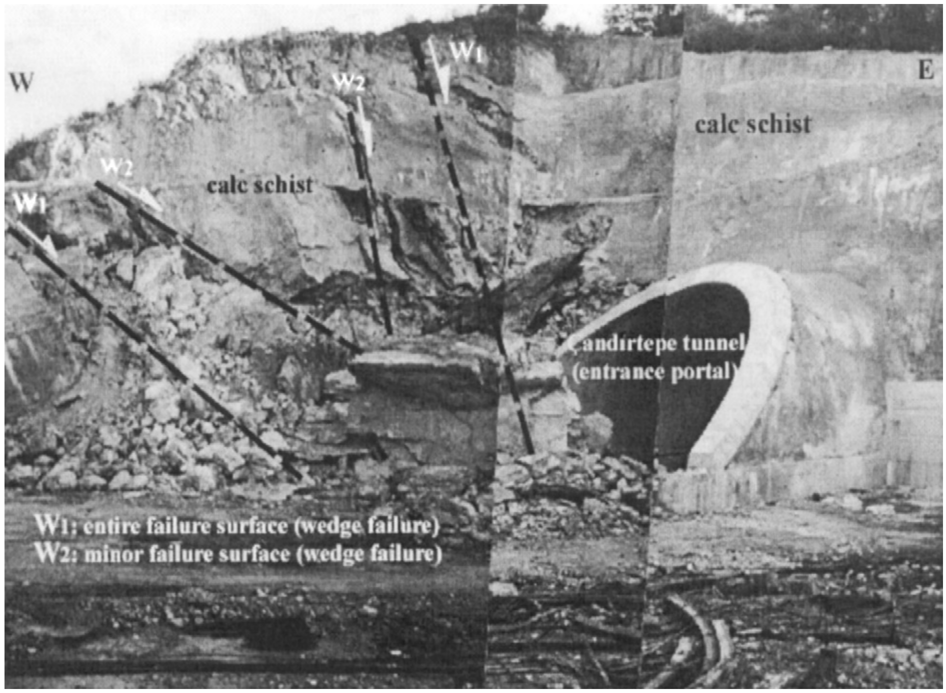


Fig. 16. General view of the failure surfaces (wedge failures) towards the west cut slope of the entrance portal of the Çandırtepe tunnel.

Table 8
The major orientations of the slope face, upper slope face and discontinuity sets of the two wedge types of failures that occur within each other at the entrance portal slope of the Çandırtepe tunnel

	Wedge 1 (calc schist) dip/dip direction	Wedge 2 (calc schist) dip/dip direction
Slope face	70°/220°	70°/220°
Upper slope face	24°/210°	24°/210°
Joint set 1	70°/165°	70°/160°
Joint set 2	72°/265°	75°/290°

parameters of the graphitic phyllite. To determine the shear strength parameters based on the generalized Hoek-Brown failure criterion, the RocLab [17] software with the slope application option was used. The height of the slope is 16 m and the average unit weight (γ_t) of calc schist is 26.20 kN/m^3. According to the GSI results (from Table 5), the range of the shear strength parameters of calc schist are as follows: $c = 36-67$ kPa; $\phi = 20-25°$.

There is a very good agreement between the shear strength parameters obtained through the GSI method and the back-calculated shear strength parameters. The back calculated c' and ϕ' parameters give a slightly higher value (closer to the upper bound) than the GSI method most probably because the calc schist at the Çandırtepe location is not intensely intercalated with the other schist units. The results of the back calculated shear strength results show that the GSI method yields satisfactory shear strength results for very poor quality rock masses.

7. Stability of the Ilıksu tunnels

7.1. General

This part of the paper deals with the stability assessment of the Ilıksu tunnels and with the necessary reinforcement measures for those unstable tunnel sections where large stress changes and deformations are induced when the underground openings are excavated. The finite element software package Phase^2 [23] was used to determine the induced stresses and deformations developed around the Ilıksu tunnels and to investigate the interaction of the proposed support systems with the tunnel ground. The tunnel grounds were divided into sections with respect to the rock mass classes that were evaluated at the borehole locations along the tunnel route and at the exit portal of the Ilıksu 2 tunnel (i.e., the geotechnical parameters tabulated in Table 5 were used as input for the Phase^2 hybrid-finite element model).

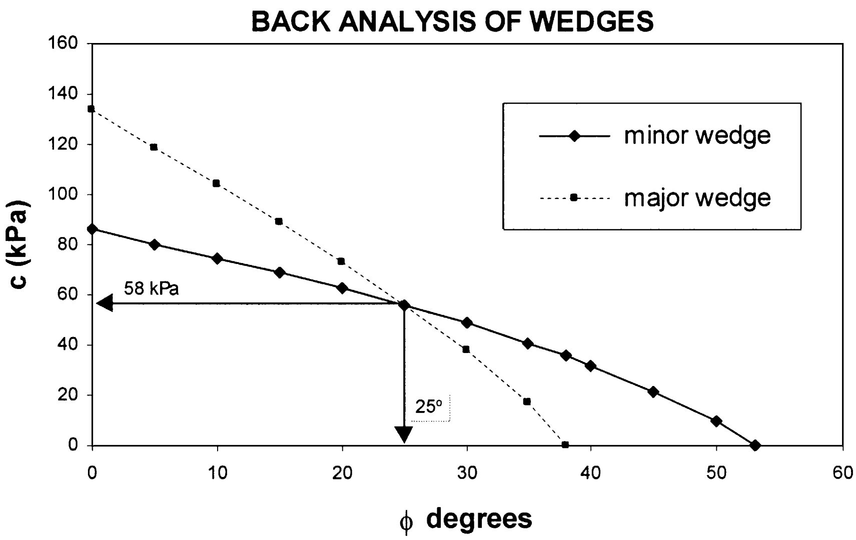


Fig. 17. Back analysis plot. The intersection point of the two-failed wedge $c-\phi$ pairs give $\phi = 25^\circ$ and $c = 58$ kPa for calc schist.

The tunnels are planned to be constructed as single tube flute shaped tunnels with a 10 m span and 7.5 m height. The tunnels lie at a relatively shallow depth (the depth to the ground surface elevation from the design level of the tunnels varies from approximately 20–50 m). Since the very blocky rock masses of recrystallized limestone are regularly bedded and possess clay coated and slickensided bedding surfaces, they were modeled to be anisotropic, whereas highly jointed, foliated, laminated, sheared and very deformable soil-like rock masses (i.e., graphitic phyllites, pelitic schists and intercalation of these rock masses) without any preferred failure directions (i.e., those lacking continuity of the joint surfaces) were modeled to be isotropic by Phase^2.

The objective of Phase^2 was to check the validity of the empirical temporary tunnel support requirements given in Table 3, using the top heading and bench method of excavation. Modeling with Phase^2 consisted of four stages. In the first stage, in situ stress distributions (gravity loading due to the thickness of the overburden at the design elevation) were examined. In the following two stages, the principal stress distributions (σ_1, σ_3), the safety factor, yield points (shear, tension) and the induced displacements developed around the tunnels were analyzed using top heading followed by excavating the entire tunnel. In the final stage, the effectiveness of the temporary support systems (i.e., shotcrete and rock bolting) was investigated. An example of these four stages are illustrated in Fig. 18.

7.2. *Stability of the entrance and exit portals of the Ilıksu 2 tunnel (km: 128+155–128+317)*

The most unsuitable conditions (poor to very poor quality rock mass) that could be faced during tunneling were taken into consideration in modeling this tunnel section [24]. Accordingly, the rock mass strength parameters at borehole location ISK 8 (Table 5) were used for the analysis. The post-failure behavior of the rock mass was considered to be perfectly plastic according to the recommendation of Hoek and Brown [11].

In the tunnel section (km: 128 + 155–128 + 317), the support systems that were empirically determined in Table 3 were used. The support used for the 10 m span of the tunnel was fully bonded, 4 m long and 32 mm diameter untensioned grouted rock bolt with a 250 mm thick shell of reinforced shotcrete. Queen Cable's bolt model was used as pattern bolting for predicting the behavior of cable bolt reinforcement in tunneling.

The results of the numerical stress analysis of the tunnel section (km: 128 + 155–128 + 317), the total displacements in the rock mass as a result of the four stages and the extent of yield zones (shear and tension) with and without support are shown in Figs. 19 and 20, respectively. The reader should note the difference in the extent of the failure zone and the magnitude of the induced displacements after support installation. Compared with the unsupported excavation, displacements have been drastically reduced. After installation of the support system, the total displacement is nearly reduced by three-folds with respect to the induced displacement without support. Therefore, it is concluded that the support recommendations given for the Ilıksu 2 tunnel in Table 3 are satisfactory.

A summary of the total induced displacements before and after support are as follows:

- Total induced displacement δ_t before installation of the supports: 125 mm,
- total induced displacement δ_t after installation of the supports: 43 mm.

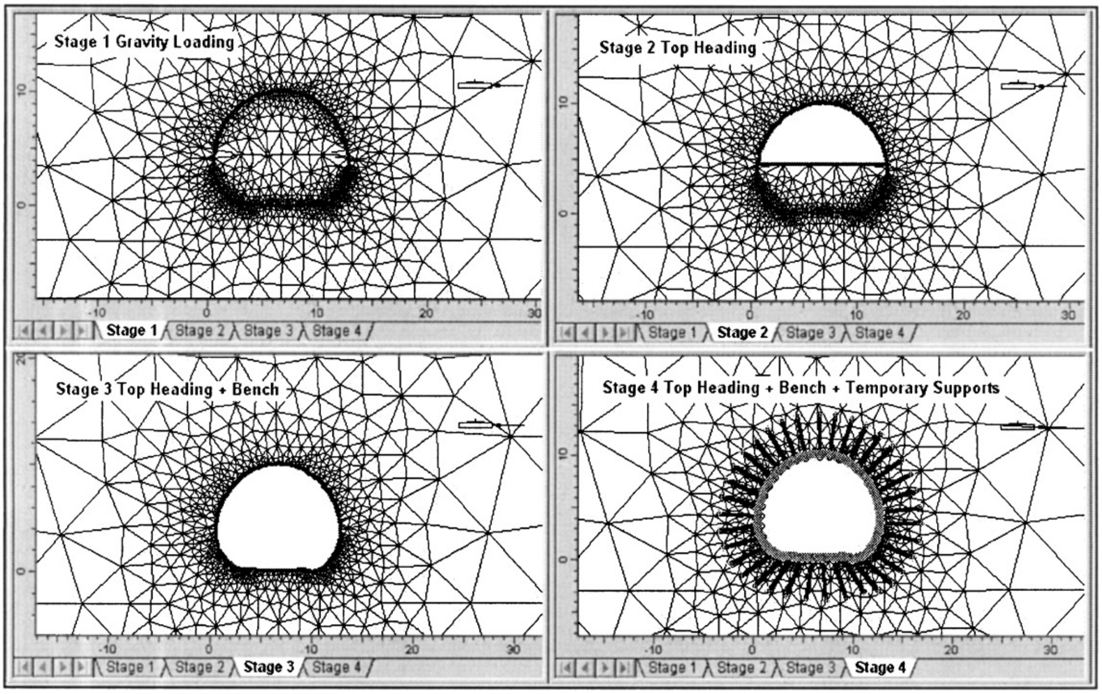


Fig. 18. Numerical modeling of the four stages, km: 128 + 155–128 + 317 of the Ilıksu 2 tunnel.

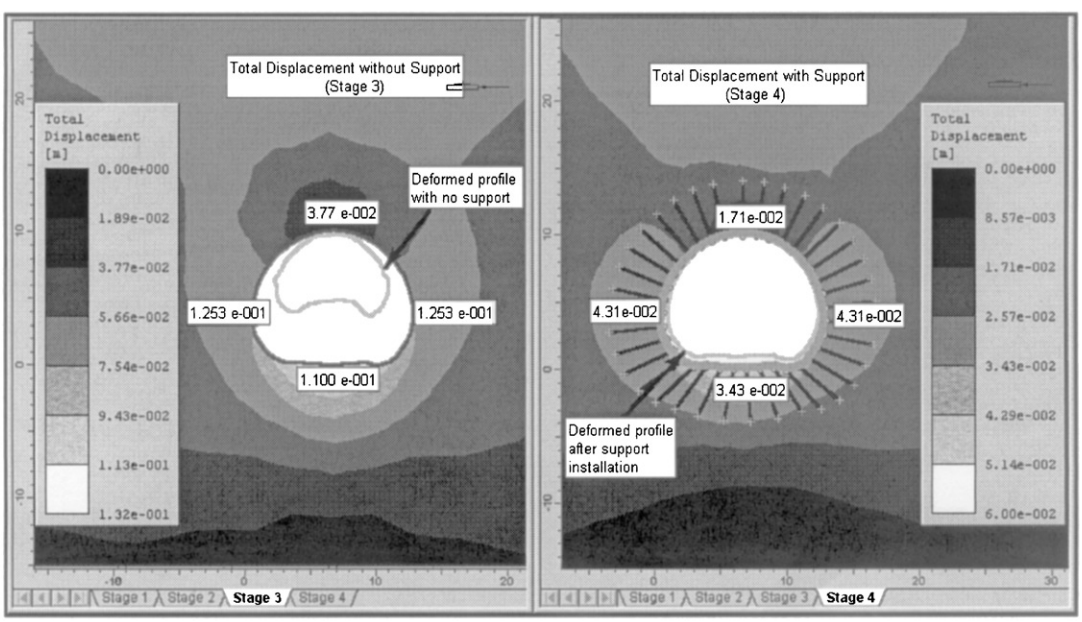


Fig. 19. Total induced displacement contours and their deformed mesh at the Ilıksu 2 tunnel portal locations (km: 128 + 155–128 + 317).

The Phase^2 model applies supports immediately after the excavation but in real life some deformation is allowed to occur and installation of support systems takes time. The support system used appears to be very effective in stopping further deformations that could occur with time. The delayed installation of the support is simulated by allowing the field stress induced load to be split between stages. This means that some deformation is allowed to take place in the first three unsupported stages, followed by support installation in the fourth stage. In the Ilıksu 2 tunnel, the maximum stress concentration (σ_h/σ_v) develops at the roofs and the maximum induced displacement occurs along the sidewalls of the tunnels. Failure is usually expected in the form of compression at the roof and tension at the wall of the tunnels. The maximum extent of the yield

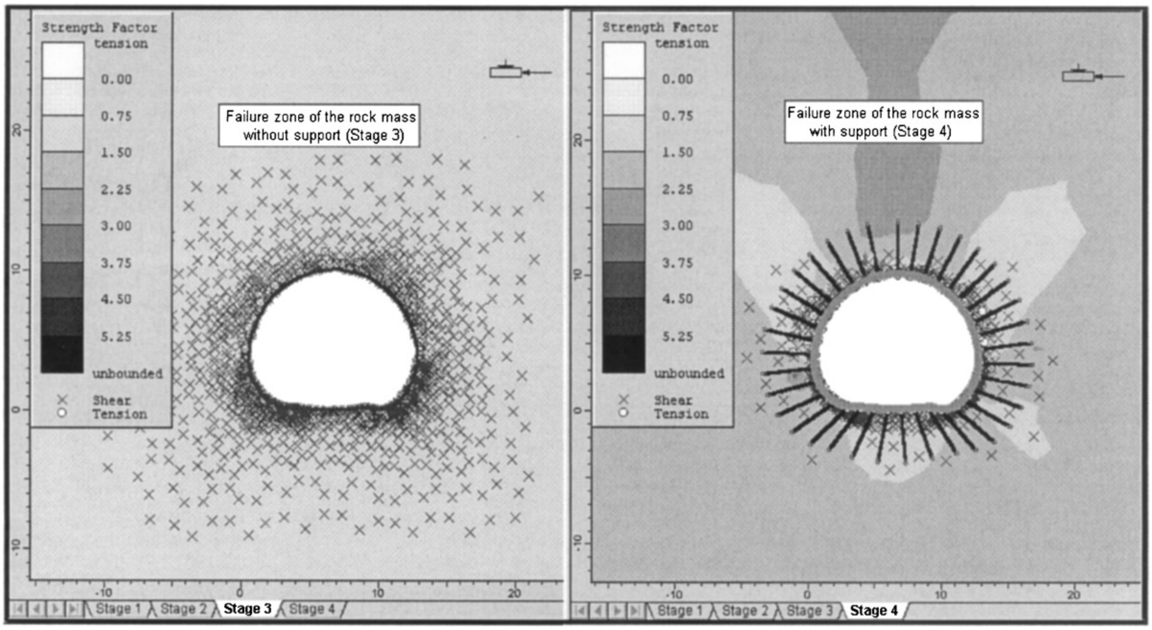


Fig. 20. The failure zone of the rock mass and the extent of the yield zone (x: shear failure, o: tensile failure) without support and with support at the portals of the Ilıksu 2 tunnel (km: $128+155-128+317$).

zones in terms of shear and tension is observed along the highly foliated, laminated, sheared and irregularly jointed surfaces along both portals of the Ilıksu 2 tunnel, and the exit portal of the Ilıksu 1 tunnel.

8. Summary and conclusions

8.1. Summary and conclusions relating to the methodology

The purpose of this study is to present a methodology for tunnel and support design in mixed limestone, schist and phyllite conditions through investigating two tunnel case studies in southern Turkey. The main lithologies of the project area are regularly jointed, recrystallized limestone and the weak lithologies of the schist unit (i.e., pelitic schist, calc schist, graphitic phyllite and alternations of these lithologies). The study started with literature review on the geology and geotechnical characteristics of the project site, a preliminary site reconnaissance visit to identify major lithologies, structural features and to decide on possible borehole locations. Detailed geological and geotechnical field investigations in the project area covered geological mapping and geological cross-section preparation from boring data, selection of representative rock core samples for geomechanics laboratory testing, determination of rock material and rock mass characteristics, determination of RQD from boring data, and determination of discontinuity characteristics through scan-line survey. Laboratory tests were performed to determine the geomechanical parameters of good quality rock masses (i.e., regularly jointed, recrystallized limestone). For poor quality rock masses (i.e., phyllite, calc schist, pelitic schist and intercalation of these units), the Hoek–Brown criterion was used to obtain the relevant geomechanical parameters (i.e., shear strength parameters, deformation moduli and post-failure behavior) since it was almost impossible to recover representative core samples for laboratory testing.

The tunnel ground rock mass classes and empirical support types/categories were determined according to the Q-system, RMR method and NATM. The Hoek–Brown failure criterion was used for the rock masses at each borehole location to make a reliable estimate of the geomechanical properties of the rock masses.

Slope stability analyses were performed at the portal, side or cut slope sections of the tunnels. Initially, kinematic analyses were performed for the regularly jointed rock masses (i.e., recrystallized limestone). Later, limit equilibrium analyses were performed for the kinematically failed rock slopes incorporating the effect of water pressure. Circular failure analogy was used for the slope stability analyses of the irregularly jointed, highly foliated, laminated and very deformable soil-like lithologies (i.e., phyllite, calc schist, pelitic schist and intercalation of these lithologies). A back analysis on a failed slope was performed to check the validity of the shear strength parameters obtained by the GSI method. Following the slope stability analyses, recommendations were made regarding the required support systems or appropriate slope remediation measures.

The tunnel grounds were divided into sections according to their rock mass classes. The deformations and stress concentrations around each tunnel section were investigated and the interactions of the empirical support systems with the rock masses were analyzed by using the Phase^2 finite element software. The regularly jointed rock masses were modeled to be anisotropic, whereas the irregularly jointed, highly foliated and very deformable soil-like lithologies were modeled to be isotropic in the tunnel finite element analyses.

8.2. Summary and conclusions relating to the Ilıksu tunnels

As stated above, the methodology for tunnel and support design in mixed limestone, schist and phyllite conditions was assessed through two tunnel case studies in southern Turkey which involved investigating the engineering geological and geotechnical characteristics of the rock material and rock mass of the tunnel grounds, and suggesting appropriate support and stabilization techniques. The tunnels which are named as Ilıksu 1 and Ilıksu 2 are located along the 4th division route of the Antalya–Alanya autoroad.

In order to constitute the geological model and to determine the geotechnical properties of the ground, a total of 302 m of drilling was performed with eight boreholes along the Ilıksu 1 and Ilıksu 2 tunnels. In addition to the boring operations, a detailed geological and geotechnical study (scan-line survey, discontinuity measurements, kinematic analyses, etc.) was carried out in the project area and the engineering geological characteristics of the rock masses were determined. Geomechanics tests were performed on 62 core samples obtained from the 8 boreholes in the project area.

The results of the kinematic analyses on regularly jointed, recrystallized limestone showed that only wedge failure was possible on the north cut slope of the Ilıksu 1 tunnel entrance portal. After performing a wedge limit equilibrium analyses for saturated conditions, it was determined that wedge failure was not expected for the north cut entrance portal slope of the Ilıksu 1 tunnel. Planar failure was kinematically possible for both the north cut and south cut slopes along the Ilıksu 1 tunnel entrance portal. After performing a plane failure limit equilibrium analyses for saturated conditions, no plane failure was expected along the north and south cut portal slopes of the Ilıksu 1 tunnel.

Slope stability analyses of irregularly jointed, highly foliated and very deformable soil-like lithologies (i.e., pelitic schists, graphite schists, etc.) were conducted with the slope stability software Slope/W and the finite element analyses software PLAXIS 7.2. The results of the analysis showed that slope stability problems were expected along the exit portal face of the Ilıksu 1 tunnel and along the entrance and exit portal faces of the Ilıksu 2 tunnel. The 81 m long cut slope section was also analyzed by Slope/W and PLAXIS 7.2 which gave very similar factor of safety results. The results of the slope stability analyses led to a recommendation to provide support at the portals of the tunnels and along the cut slope section. The recommended support types are presented in the paper.

A back analysis performed within calc schist led to shear strength parameters of $c = 58\,\text{kPa}$ and $\phi = 25^\circ$. These results compared very well with the shear strength parameters obtained with the GSI method.

The finite element software package Phase^2 was used to determine the induced stresses and deformations developed around the Ilıksu tunnels and to investigate the interaction of the proposed empirical support systems with the tunnel ground. The post-failure behavior of rock masses were estimated and applied to Phase^2 for tunnel design. Upon installation of the support systems, the total displacement and the extent of the failure zone was drastically reduced. Therefore, it was concluded that the empirical support recommendations given for tunnel design were satisfactory.

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