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ORIGINAL PAPER

Analyses of stability and support design for a diversion tunnel at the Kapikaya dam site, Turkey

Zulfu Gurocak

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Abstract This paper presents the engineering geological properties and support design for a diversion tunnel through diabase at the Kapikaya dam site, eastern Turkey. The rock mass rating and rock mass index were used to determine the support requirements, which were also analyzed using commercial software based on the finite element method. The parameters calculated by the empirical methods were used as input parameters for the FEM analysis and the results from the two methods were compared. It was found that the optimum solution was obtained by using a combination of both empirical and numerical approaches.

Keywords Finite element method · Hoek–Brown failure criterion · Kapikaya dam · RMR · RMi

Résumé L'article présente les propriétés géotechniques des terrains et la conception d'un soutènement pour un tunnel de dérivation creusé dans des diabases sur le site du barrage de Kapikaya (Turquie de l'est). Les indices RMR et RMi ont été utilisés pour définir les renforcements nécessaires, utilisant par ailleurs des logiciels commerciaux basés sur la méthode des éléments finis (MEF). Les paramètres calculés par la méthode empirique ont été utilisés comme données d'entrée pour l'analyse MEF et les résultats des deux méthodes ont été comparés. La méthode optimale a été obtenue par une combinaison des approches empiriques et numériques.

Z. Gurocak (✉)
Geological Engineering Department,
Firat University, 23119 Elazig, Turkey
e-mail: zgurocak@firat.edu.tr; zgurocak@gmail.com

Mots clés Méthode des éléments finis · Critère de rupture de Hoek et Brown · Barrage de Kapikaya · RMR · RMi

Introduction

Determination of the safest and most economical support system for underground structures is very important hence the design usually involves the use of both empirical and numerical approaches. Empirical methods are generally preferred by engineers and engineering geologists; the rock mass rating (RMR), rock mass index (RMi) and Q rock mass classification systems have been employed by many workers and have gained a universal acceptance (Barton 2002; Ramamurthy 2004; Hoek and Diederichs 2006). These rock mass classification systems were originally obtained from many tunneling case studies. However, they do not provide the stress distributions and deformations around the tunnel. These aspects therefore need special attention when empirical methods are used and the appropriate information must be obtained from field observations. Numerical approaches are dependent on the strength parameters of the rock masses. Consequently the stability analysis of a tunnel is likely to result in a safer and more economical design if a combination of empirical and numerical approaches is used.

This study area is 30 km east of Malatya, eastern Turkey (Fig. 1) where construction of the Kapikaya dam on the Mamikan stream has begun. The dam project was designed to regulate drainage and irrigate the agricultural areas of the Kale plain, under the supervision of General Directorate of State Hydraulic Works (1991), the Ministry of Energy and Natural Resources in Turkey.

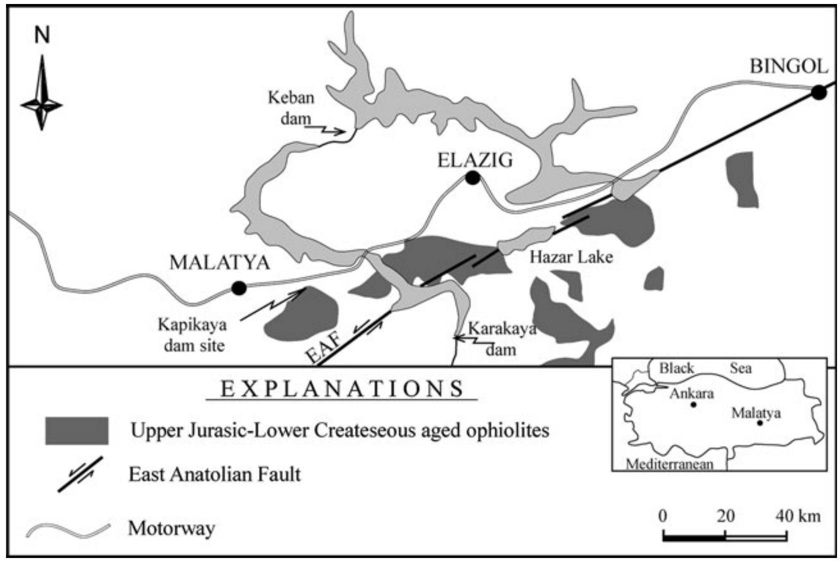

Fig. 1 Location map of the study area

The diversion tunnel of the Kapikaya dam has a horseshoe geometry, with an excavation width and height of 4.40 m (Fig. 2) and a length of 413.69 m. The tunnel will have a maximum overburden of about 78 m. In this paper, the preliminary support design of this tunnel will be described using both empirical and numerical approaches, illustrating the advantage of using both approaches simultaneously.

The dam site is located within the Ispendere Ophiolites, which is composed of diabases including subsequent diabase dykes. Geological mapping and geotechnical descriptions were conducted in the field. The physical, mechanical and elastic properties of the rocks under consideration were determined from laboratory testing on intact rock samples. These tests included an evaluation of uniaxial compressive strength (σ_c), Young's modulus (E), Poisson's ratio (v) and unit weight (γ). The rock mass properties of the dam site were determined using the RMR and RMi rock mass classification systems.

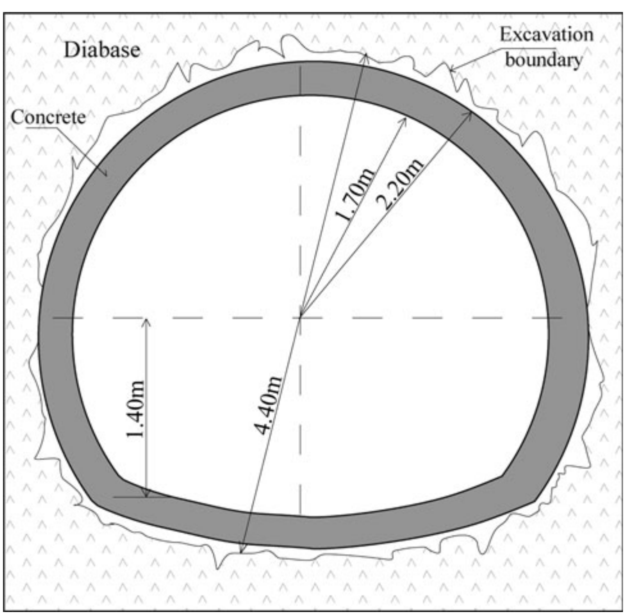

Fig. 2 Cross-section of the diversion tunnel

Geology, field and laboratory studies

Strata of various ages, from the Upper Jurassic to the Quaternary, outcrop in the region. Upper Jurassic–Lower Cretaceous ophiolitic rocks are exposed at the Kapikaya dam site (Fig. 3). These rocks are a part of the extensive Jurassic–Cretaceous aged ophiolitic complex in the Southeast Anatolian Thrust Zone. They are found as allochthonous bodies in the Eastern Taurus (Yazgan 1984). The diabases are composed mainly of fine grained plagioclase and clinopyroxcene crystals; some carbonatization of plagioclases and cloritization of clinopyroxcene can be detected. They are yellowish-grey in colour, well jointed and moderately to slightly weathered.

Overlying the mainly Upper Jurassic–Lower Cretaceous deposits are talus and alluvial materials. Talus displays a wide distribution in the study area (Fig. 3). From the drillings conducted by the General Directorate of State Hydraulic Works (1991) at the dam site, the thicknesses of talus deposits were found to vary between 0.5 and 8 m. These deposits are coarse blocky, pebbly, sandy and clayey in nature. The alluvium is observed in the Mamikan stream bed (Fig. 3). From the drilling data, it consists of blocks, pebbles, sand and silt, with a thickness of 1–6 m.

When undertaking the engineering geological mapping of the Kapikaya dam site, the orientation, persistence, spacing, opening, and roughness, degree of weathering and filling of discontinuities in the diabases was recorded. The

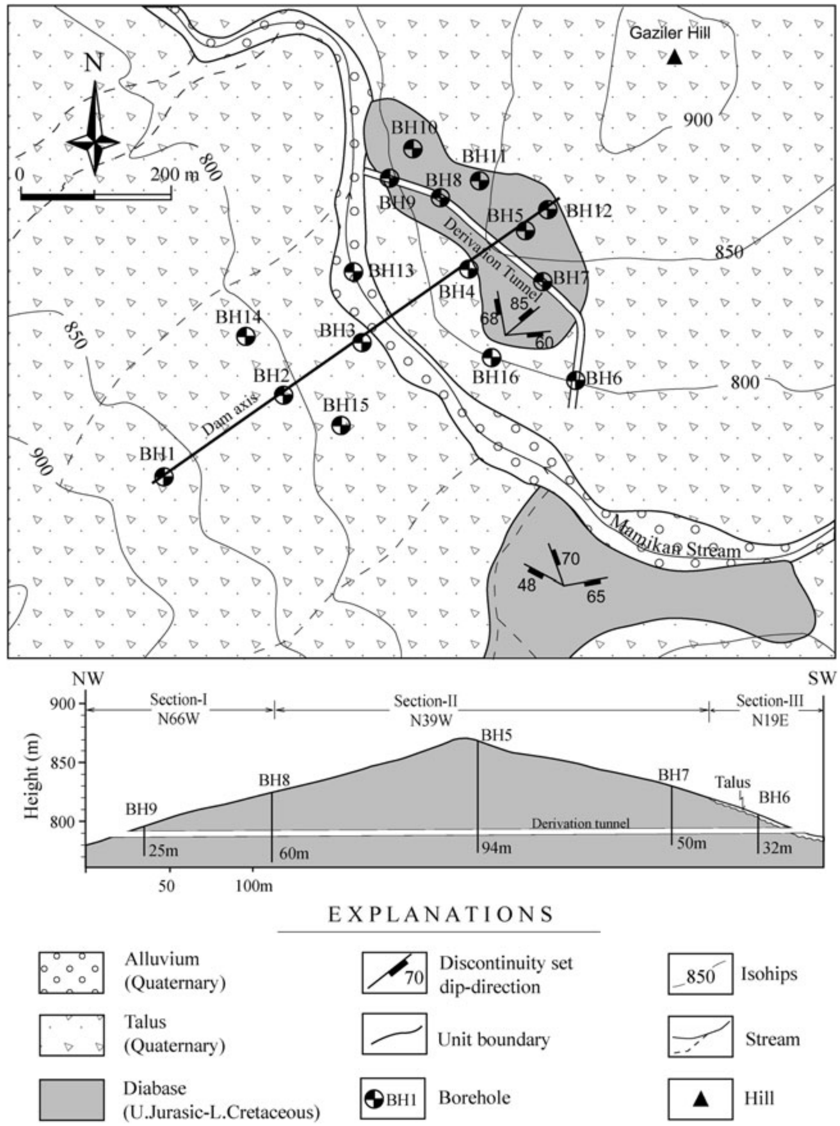

Fig. 3 Geological map of the study area

intrusive investigation undertaken by the General Directorate of State Hydraulic Works (1991) included 16 boreholes and a total of 722 m of core from 16 boreholes. The RQD values determined following Deere (1964) are given in Table 1.

Uniaxial compressive strength (σ_c), Young's modulus of intact rock (E), Poisson's ratio (v) and unit weight (γ) were determined in accordance with ISRM (1981).

Table 1 The percentage distribution of RQD values of diabases

%	Rock quality	Distribution (%)
0–25	Very poor	3
25–50	Poor	10
50–75	Fair	30
75–90	Good	40
90–100	Excellent	17

As the study area is located in a tectonically active region, the diabases exposed around the Kapikaya dam site contain systematic joint sets. These were described using the scan-line survey method following ISRM (1981) description criteria (Table 2).

The degree of weathering of the discontinuity surfaces was assessed using the Schmidt hammer and the weathering index was calculated from the equation proposed by Singh and Gahrooee (1989):

$$W_c = \frac{\sigma_c}{\mathrm{JCS}} \tag{1}$$

where σ_c = uniaxial compressive strength of fresh rock (MPa), and JCS = strength of discontinuity surface (MPa).

JCS was calculated from the following equation:

$$\mathrm{LogJCS} = 0.00088\gamma R + 1.01 \tag{2}$$

where γ = bulk volume weight (kN/m^3), and R = hardness value from rebounding of the Schmidt hammer.

Table 2 Quantitative description of joints in the diabase

Property	
Joint set number	Three joint sets plus random
Spacing of discontinuities (mm)	18–197 (average 120)
Persistency of discontinuities (m)	Generally 1–3 m, however occasionally <1 and 10–20 m
Aperture of discontinuities (mm)	Generally >5 mm, however occasionally 0.1–0.5 mm
Roughness of discontinuities	Generally rough, however occasionally smooth
Infilling	Generally calcite having a thickness of >5 mm, however occasionally <5 mm calcite
Weathering degree	Generally slightly weathered, occasionally moderately weathered
Groundwater conditions	Generally dry occasionally damp

A total of 846 joint measurements were taken from the diabases. Discontinuity orientations were based on equal-area stereographic projection obtained using the DIPS 5.0 (1999a) software and three major joint sets were distinguished for the diabases: Joint set 1: 68/75 Joint set 2: 78/185 and Joint set 3: 85/143 (see Fig. 4).

According to ISRM (1981), the joint sets in the diabases are very closely spaced, with low persistence, open, rough-planar and slightly to moderately weathered.

The tunnel route was divided into three section and laboratory tests, such as uniaxial compressive strength (σ_c), Young's modulus of intact rock (E), Poisson's ratio (v) and unit weight (γ), were conducted in accordance with the ISRM suggested methods (1981) on samples collected from each sections. Pertinent results are summarized in Table 3.

Empirical analyses

A reliable stability analysis and prediction of the support are some of the most difficult tasks in rock engineering hence in the current study several methods were used.

RMR classification system

Bieniawski (1974) initially developed the RMR system based on experience in tunnel projects in South Africa. Since then, this classification system has undergone significant changes, mainly involving ratings added for ground water, joint condition and joint spacing. In order to use this system, the uniaxial compressive strength of the intact rock, RQD, joint spacing, joint condition, joint orientation and ground water conditions must be known. The results (based on Bieniawski 1989) are summarized for the three sections in Table 4.

RMi classification system

Rock mass index was proposed by Palmström (1995) for the general characterisation of rock mass strength and rock mass deformation, design, calculation of the constants in the Hoek–Brown failure criterion for rock masses and TBM progress. In 2000, some significant changes and adjustments for estimating preliminary rock support by using block volume and tunnel diameter were made. The RMi is a volumetric parameter indicating the approximate uniaxial compressive strength of a rock mass. It is expressed for jointed rock as (Palmström 2000):

$$\mathrm{RMi} = \sigma_c \mathrm{JP} = \sigma_c 0.2\sqrt{\mathrm{jC}}\,\mathrm{Vb}^D \quad (D = 0.37\mathrm{jC}^{-0.2}) \qquad (3)$$

where σ_c = the uniaxial compressive strength of intact rock, jC = the joint factor, which is a combined measure

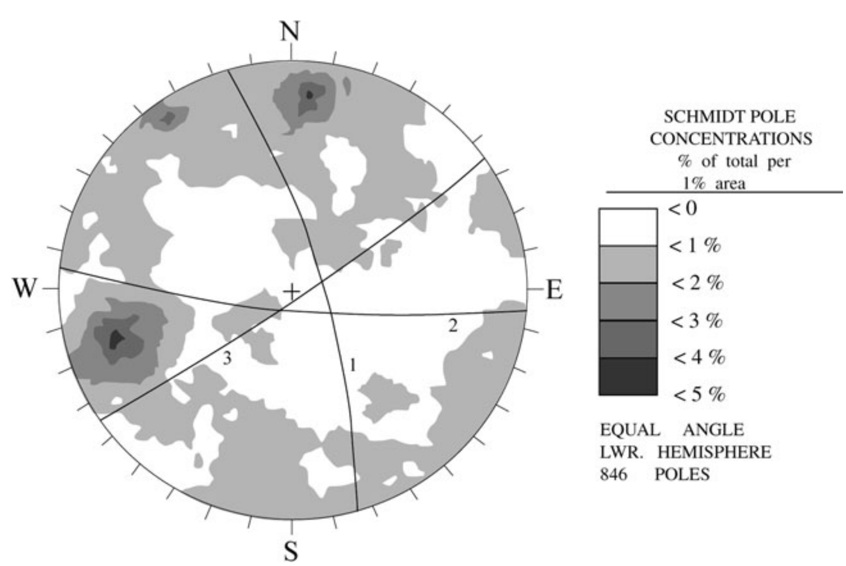

Fig. 4 Stereographic projection of joint sets in diabases

Table 3 Physical, mechanical and elastic properties of diabases for sections shown in Fig. 3

Property	Section I	Section II	Section III
Uniaxial compressive strength (σ_c, MPa)	38.74–214.51 (average 73.40)	57.66–286.90 (average 97.42)	41.50–162.10 (average 81.22)
Young's modulus of intact rock (E, GPa)	12.36–66.12 (average 18.44)	16.82–87.38 (average 25.63)	7.64–58.46 (average 21.30)
Poisson's ratio (v)	0.221–0.278 (average 0.259)	0.230–0.286 (average 0.261)	0.228–0.282 (average 0.251)
Unit weight (γ, kN/m^3)	28.10–30.85 (average 28.92)	28.55–31.66 (average 29.08)	27.71–29.90 (average 28.56)
RQD (%)	38–70 (average 59)	84–96 (average 90)	63–74 (average 66)

Table 4 Tunnel support categories for the diabases for sections shown in Fig. 3

Classification system		Section I	Section II	Section III
RMR_{89}	Basic RMR value	52.5	59.8	53.7
	Rating adjustment for joint orientation	−12	−12	−5
	RMR value	40.5	47.8	48.7
	Support	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and walls with wire mesh. 100–150 mm in crown and 100 mm in sides. Light to medium ribs spaced 1.5 m where required	Systematic bolts 4 m long spaced 1.5–2 m in crown and walls with wire mesh in crown. 50–100 mm in crown and 30 mm in sides	
RMi	RMi value	0.43	1.94	0.37
	Support	2.5 m length rock bolts spaced 1.5 m. 50–70 mm thick fiber reinforced shotcrete in roof	2.5 m length rock bolts spaced 2 m in roof	2.5 m length rock bolts spaced 1.25 m. 50–70 mm thick fiber reinforced shotcrete in roof
		2 m length rock bolts spaced 1.5 m in walls	2 m length rock bolts spaced 3 m in walls	2 m length rock bolts spaced 2 m 40–50 mm thick shotcrete in walls

for the joint size (jL), joint roughness (jR), and joint alteration (jA), given as

$$\mathrm{jC} = \mathrm{jL}\frac{\mathrm{jR}}{\mathrm{jA}} \tag{4}$$

Vb = the block volume, measured in m^3, JP = the jointing parameter, which incorporates the main joint features in the rock mass. It can be found from the following equation;

$$\mathrm{JP} = 0.2\sqrt{\mathrm{jC}}\mathrm{Vb}^{D} \tag{5}$$

The block volume (Vb) was calculated using the following equation (Palmström 2000):

$$\mathrm{Vb} = \beta \mathrm{Jv}^{-3} \tag{6}$$

where β is block shape factor, and Jv is volumetric joint count. Volumetric joint count (Jv) can be calculated using RQD values below (after Palmström 2000):

$$\mathrm{Jv} = 35 - \frac{\mathrm{RQD}}{3.3} \tag{7}$$

In this study, the diversion tunnel route was divided in three sections by considering the strike of the tunnel axis (Fig. 3). RMR_{89} and RMi classification systems were followed and the support systems determined for each section using data obtained from the field surveys, boreholes and laboratory tests. The results are summarized in Table 4.

Estimation of rock mass properties

The rock mass properties such as geological strength index (GSI), Hoek–Brown constants, deformation modulus (E_{mass}) and uniaxial compressive strength of rock mass (σ_{cmass}) were calculated by means of empirical equations suggested by different researchers.

GSI and Hoek–Brown parameters

The GSI was developed by Hoek et al. (1995) based on the appearance of a rock mass and its structure. Marinos and

Table 5 Calculated GSI and Hoek–Brown parameters for the sections shown in Fig. 3

Parameters	GSI	Constant			
		m_i	m_b	s	a
Section I	44	15	2.030	0.0020	0.509
Section II	52	15	2.701	0.0048	0.505
Section III	47	15	2.260	0.0028	0.507

Hoek (2001) used additional geological properties in the Hoek–Brown failure criterion and introduced a new GSI chart for heterogeneous weak rock masses. The value of GSI was obtained from the quantitative GSI chart proposed by Marinos and Hoek (2000).

The Hoek and Brown (1997) failure criterion was used for determining the rock mass properties at the dam site. Hoek et al. (2002) suggested the following equations for calculating rock mass constants (i.e., m_b, s and a):

$$m_b = m_i \exp\left(\frac{\mathrm{GSI} - 100}{28 - 14D}\right) \quad (8)$$

$$s = \exp\left(\frac{\mathrm{GSI} - 100}{9 - 3D}\right) \quad (9)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\mathrm{GSI}/15} - e^{-20/3}\right) \quad (10)$$

where D is a factor that depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. In this study the value of D is considered zero. The calculated GSI and Hoek–Brown constants are listed in Table 5.

Strength and modulus of rock mass from RMR, RMi and GSI

Several empirical equations have been suggested by different researchers for estimating the strength and modulus of rock masses based on the RMR, RMi and GSI values. In this study, the strength of the rock mass was calculated from the following equation suggested by Hoek et al. (2002):

$$\sigma_{\mathrm{cmass}} = \sigma_{\mathrm{ci}} \frac{(m_b + 4s - a(m_b - 8s))(m_b/4 + s)^{as-1}}{2(1+a)(2+a)}, \quad (11)$$

where σ_{ci} is the uniaxial compressive strength of the intact rock, m_b, s and a are rock mass constants. The strength of rock masses for basalt and tuffite were considered appropriate and determined for Section I, Section II and Section III as 13.69, 21.52 and 5.43 MPa, respectively.

The deformation modulus of rock masses was calculated as suggested by different researchers based on RMR, RMi and GSI values; see Table 6. The results are summarized in Table 7.

Numerical analysis

In order to verify the results of the empirical analyses, a 2D hybrid element model, Phase[2] 6.00 Finite Element Program (Rocscience 2005), was used in a numerical analysis. The rock mass properties assumed in this analysis were obtained from the estimated values discussed above.

The vertical stress was assumed to increase linearly with depth due to the overburden weight, as follows:

$$\sigma_v = \gamma H \quad (22)$$

where γ is the unit weight of the rock mass in MN/m^3, and H is the depth of overburden in metres.

The horizontal stress was determined from the following equation suggested by Sheorey et al. (2001):

$$\sigma_h = \frac{v}{1-v}\sigma_v + \frac{\beta E_{\mathrm{mass}} G}{1-v}(H + 100) \quad (23)$$

Table 6 Selected equations for estimating deformation modulus of rock mass

Author	Equations	Equation number
Bieniawski (1978)	$E_{\mathrm{mass}} = 2\mathrm{RMR} - 100 \text{ for RMR} > 50 (\mathrm{GPa})$	(12)
Read et al. (1999)	$E_{\mathrm{mass}} = 0.1\left(\frac{\mathrm{RMR}}{10}\right)^3 (\mathrm{GPa})$	(13)
Palmström and Singh (2001)	$E_{\mathrm{mass}} = 5.6\mathrm{RMi}^{0.375} \text{ for RMi} > 0.1 (\mathrm{GPa})$	(14)
Palmström and Singh (2001)	$E_{\mathrm{mass}} = 7\mathrm{RMi}^{0.4} \text{ for } 1 < \mathrm{RMi} < 30 (\mathrm{GPa})$	(15)
Ramamurthy (2001)	$E_{\mathrm{mass}} = E_i \exp[(\mathrm{RMR} - 100)/17.4] (\mathrm{GPa})$	(16)
Hoek et al. (2002)	$E_{\mathrm{mass}} = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{\mathrm{ci}}}{100}}10^{\frac{\mathrm{GSI}-10}{40}} (\mathrm{GPa}) \text{ for } \sigma_c < 100 \text{ MPa}$	(17)
Ramamurthy (2004)	$E_{\mathrm{mass}} = E_i \exp -0.0035\,[5(100 - \mathrm{RMR})] (\mathrm{GPa})$	(18)
Hoek and Diederichs (2006)	$E_{\mathrm{mass}} = E_i\left(0.02 + \frac{1-(D/2)}{1+e^{(60+15D-\mathrm{GSI})/11}}\right) (\mathrm{GPa})$	(19)
Hoek and Diederichs (2006)	$E_{\mathrm{mass}} = 100,000\left(\frac{1-(D/2)}{1+e^{(75+25D-\mathrm{GSI})/11}}\right) (\mathrm{MPa})$	(20)
Sonmez et al. (2006)	$E_{\mathrm{mass}} = E_i 10^{[((\mathrm{RMR}-100)(100-\mathrm{RMR})/4,000\exp(-\mathrm{RMR}/100))]} (\mathrm{GPa})$	(21)

Table 7 Calculated values of deformation modulus of rock mass for sections shown in Fig. 3

Deformation modulus of rock mass (E_{mass}, GPa)			
Equation number	Section I	Section II	Section III
Eq. 12	5.00	19.60	7.40
Eq. 13	14.47	21.38	15.49
Eq. 14	4.08	7.18	3.86
Eq. 15	–	9.12	–
Eq. 16	1.20	2.54	1.49
Eq. 17	6.07	11.07	7.58
Eq. 18	8.04	12.68	9.48
Eq. 19	3.84	8.86	5.43
Eq. 20	5.63	11.00	7.27
Eq. 21	2.05	4.72	2.58
Average	5.60	10.82	6.73
Standard deviation	3.914	5.943	4.189

Table 8 Material properties of diabases for numerical model for sections shown in Fig. 3

Property	Section I	Section II	Section III
Material type	Isotropic	Isotropic	Isotropic
Young's modulus (GPa)	5.60	10.82	6.73
Poisson's ratio	0.259	0.261	0.251
Compressive strength (MPa)	13.69	21.52	16.44
Vertical stress (MPa)	0.925	2.270	0.714
Horizontal stress (MPa)	0.323	0.800	0.240
m Parameter	2.030	2.701	2.260
s Parameter	0.0020	0.0048	0.0028
Material type	Plastic	Plastic	Plastic
Dilation parameter	0°	0°	0°
m Residual	1.015	1.351	1.130
s Residual	0.0010	0.0024	0.0014

where $\beta = 8 \times 10^{-6}$/°C (coefficient of linear thermal expansion), $G = 0.024$°C/m (geothermal gradient), v = Poisson's ratio, and E_{mass} = deformation modulus of rock mass, MPa.

The Hoek–Brown failure criterion was used to identify elements undergoing yielding and plastic behaviour in the rock masses in the vicinity of the tunnel. Plastic post-failure strength parameters were used in this analysis and residual parameters were assumed as half of the peak strength parameters (Table 8).

To simulate excavation of tunnels in diabases, a finite element model was generated using the same mesh and tunnel geometry. The outer model boundary was set at a distance of six times the tunnel radius. A total of 6,532 three-noded triangular elements were used in the finite element mesh.

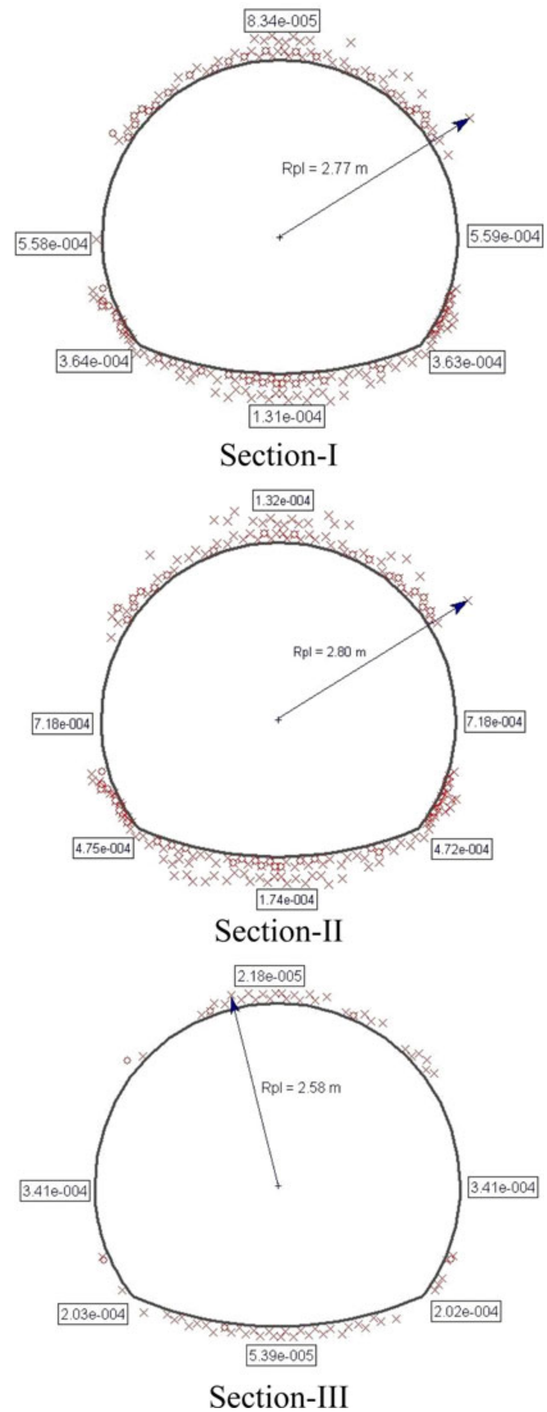

Fig. 5 Radius of plastic zone and maximum total displacements for unsupported cases

As a first step, the numerical analyses were undertaken for unsupported cases (Fig. 5) and the radius of the plastic zone and maximum total displacement for each section determined (Table 9).

Table 9 Radius of plastic zone and maximum total displacements obtained from Phase[2] for unsupported case

	Radius of plastic zone, R_{pl} (m)	Maximum total displacement (m)
Section I	2.77	5.59e-004
Section II	2.80	7.18e-004
Section III	2.58	3.41e-004

It can be seen from Table 9 that the maximum total displacement values for each section are very small. However, the extent of the plastic zone and elements undergoing yielding suggest that there would be stability problems for the tunnel.

As second step, the support elements obtained from the RMR and RMi classification systems were evaluated for each section (Figs. 6, 7, and 8). The radius of the plastic zone and maximum total displacement determined are given in Tables 10 and 11.

It can be seen from Tables 10 and 11 that the radius of the plastic zone and maximum total displacement decreased after rock bolt installation. The values obtained using RMR are very similar to those obtained using RMi. However, it can be seen from Fig. 6 that the thickness of

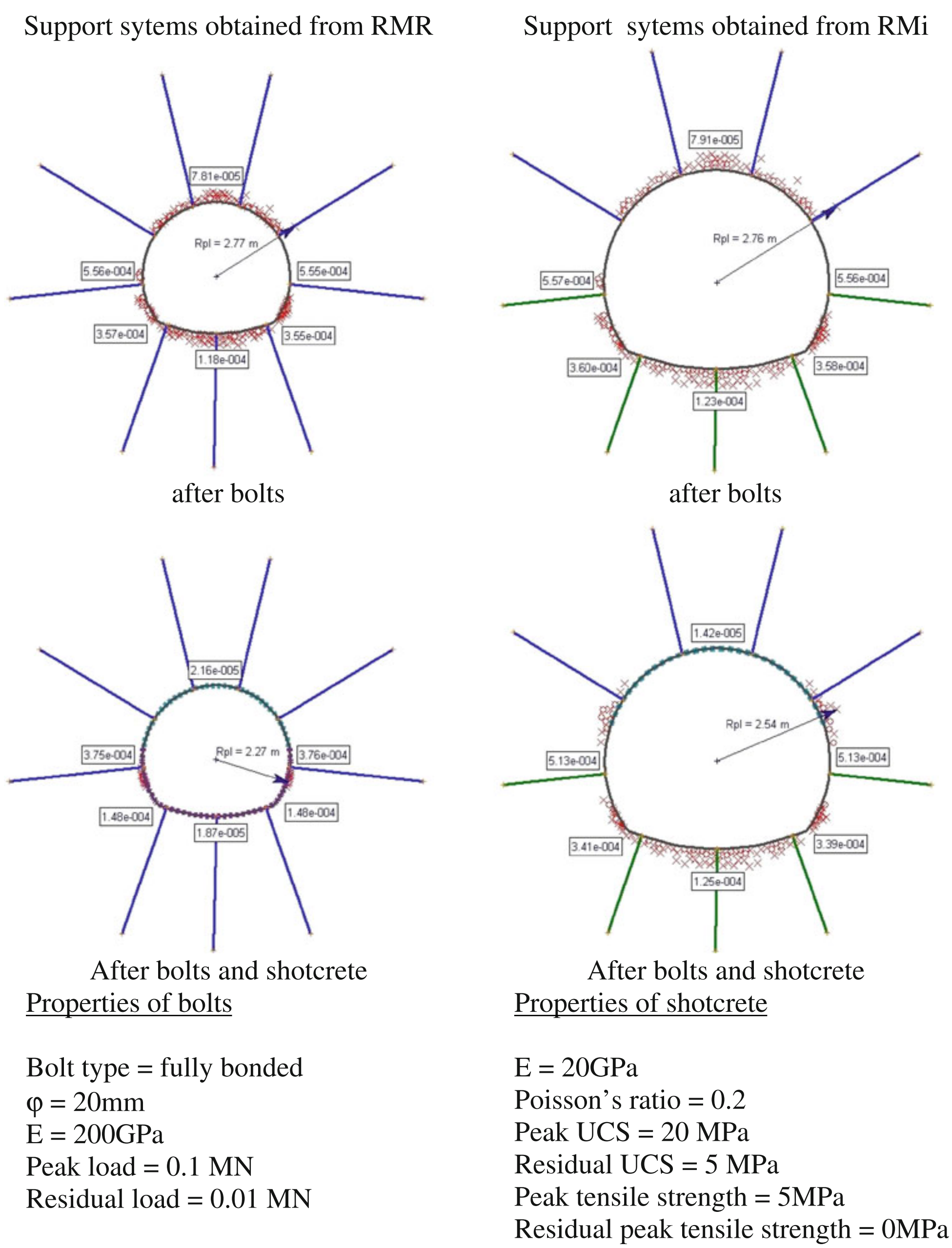

Fig. 6 Radius of plastic zone and maximum total displacements for Section I

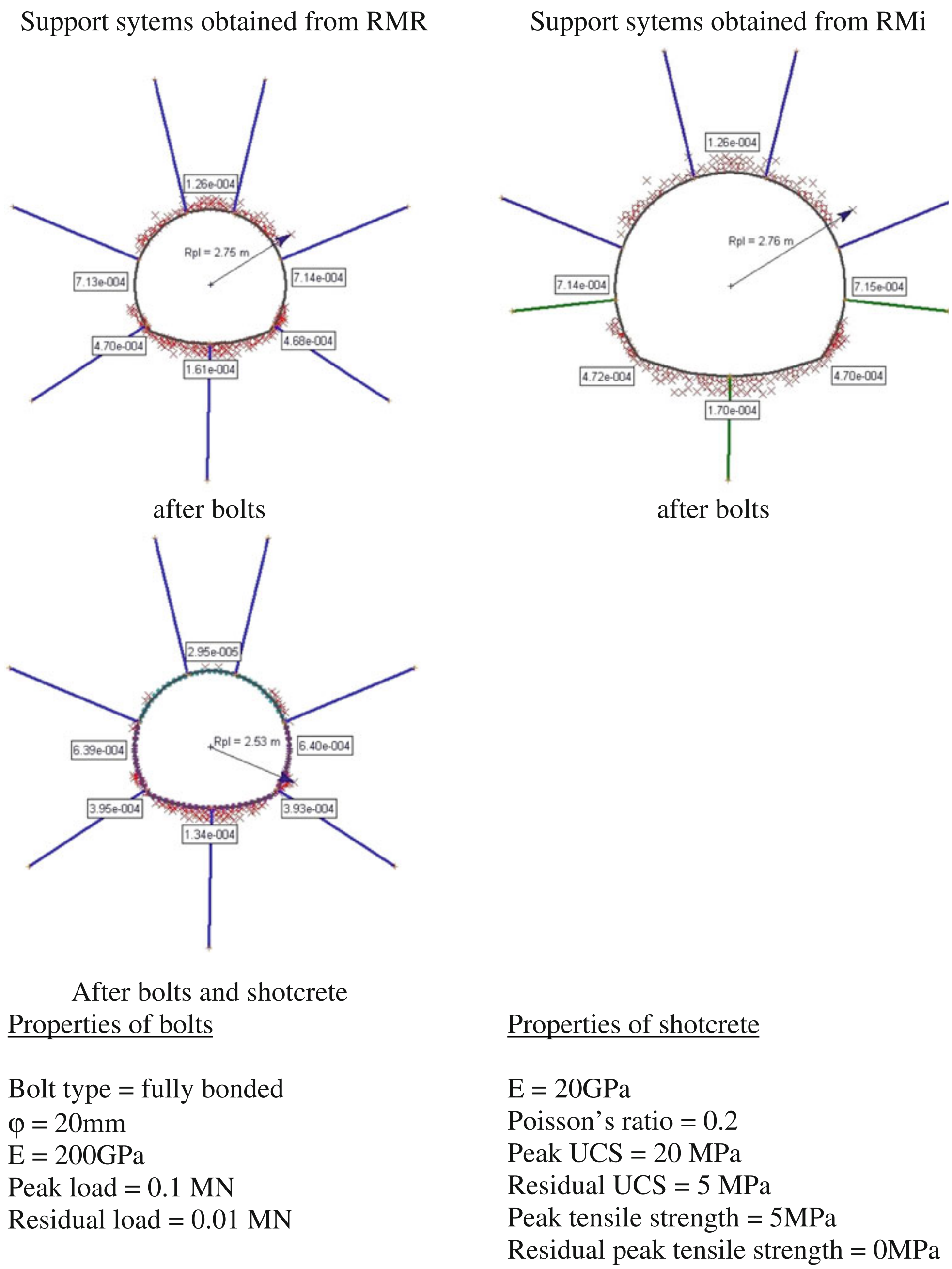

Fig. 7 Radius of plastic zone and maximum total displacements for Section II

shotcrete suggested by RMi is not enough to ensure stability in Section I and there are still some yielding elements around the tunnel.

Eventually, the support systems given in Table 12 were suggested and analyzed for each section by using the Finite Element Method (Fig. 9).

It can be seen that after support installation, not only the number of yielded elements but also the extent of the plastic zone decreased substantially for each section, as shown in Fig. 9. The maximum total displacement values decreased to 4.11e-004, 5.57e-004 and 3.08e-004 m for each section, respectively.

Conclusions

In this study, empirical methods were used to estimate rock mass quality and support elements for diabases in the

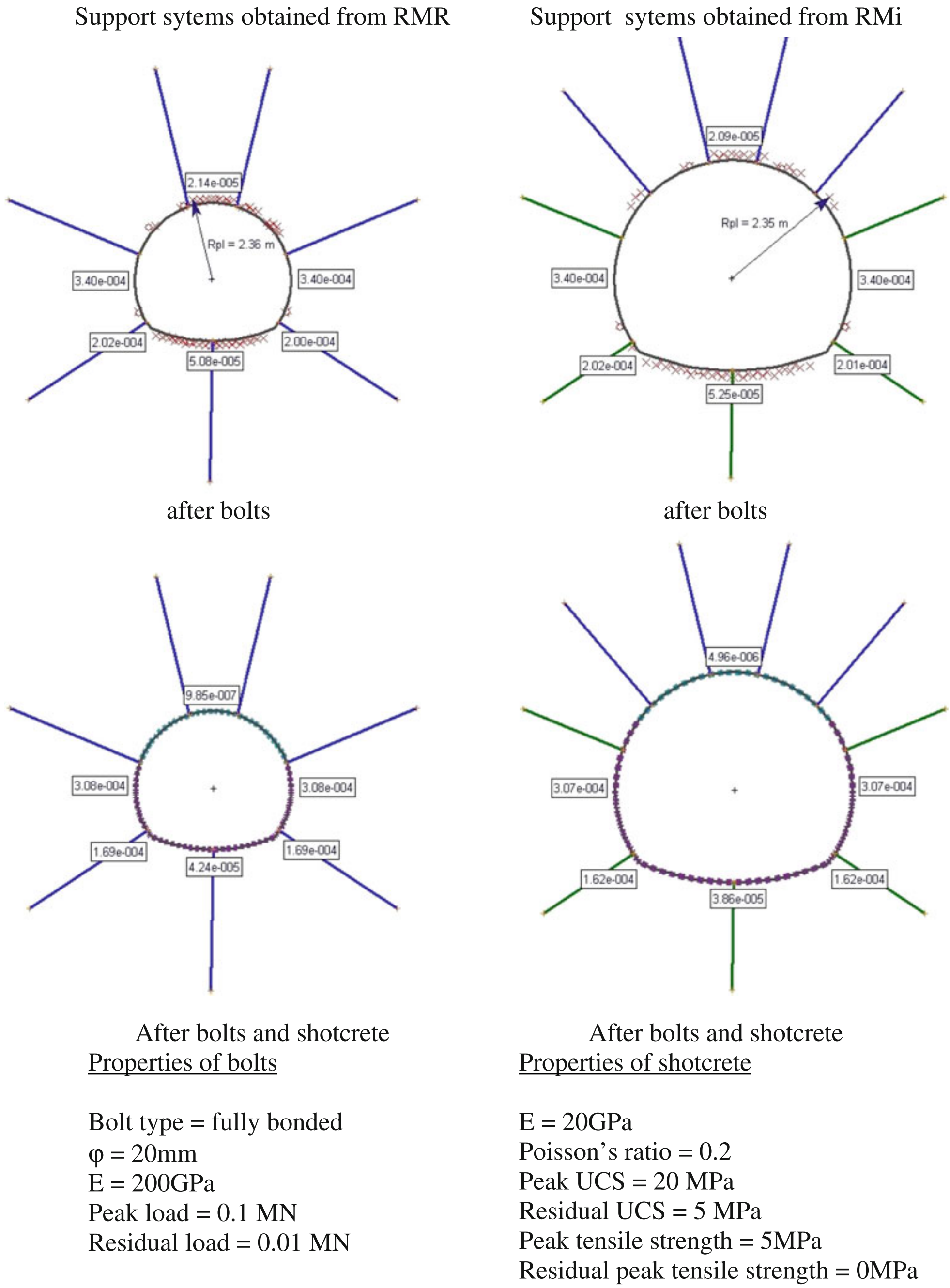

Fig. 8 Radius of plastic zone and maximum total displacements for Section III

Table 10 Radius of plastic zone (R_{pl}, m) obtained from Phase^2

	For RMR		For RMi	
	After rock bolts	After rock bolts and shotcrete	After rock bolts	After rock bolts and shotcrete
Section I	2.77	2.27	2.76	2.54
Section II	2.75	2.53	2.76	–
Section III	2.36	2.20	2.35	2.20

Table 11 Maximum total displacements (m) obtained from Phase^2

	For RMR		For RMi	
	After rock bolts	After rock bolts and shotcrete	After rock bolts	After rock bolts and shotcrete
Section I	5.56e-004	3.76e-004	5.56e-004	5.13e-004
Section II	7.14e-004	6.40e-004	7.15e-004	–
Section III	3.40e-004	3.08e-004	3.40e-004	3.07e-004

Table 12 Suggested supports, radius of plastic zone and maximum total displacement for each section

Sections	Section I	Section II	Section III
Suggested support systems	Systematic bolts 2.5 m long, spaced at 1.5 m and 200 mm thick fibre reinforced shotcrete in crown and walls	Systematic bolts 2 m long, spaced at 2 m and 150 mm thick fibre reinforced shotcrete in crown and walls	
Radius of plastic zone, R_{pl} (m)	4.20	4.20	4.20
Maximum total displacement (m)	4.11e-004	5.57e-004	3.08e-004

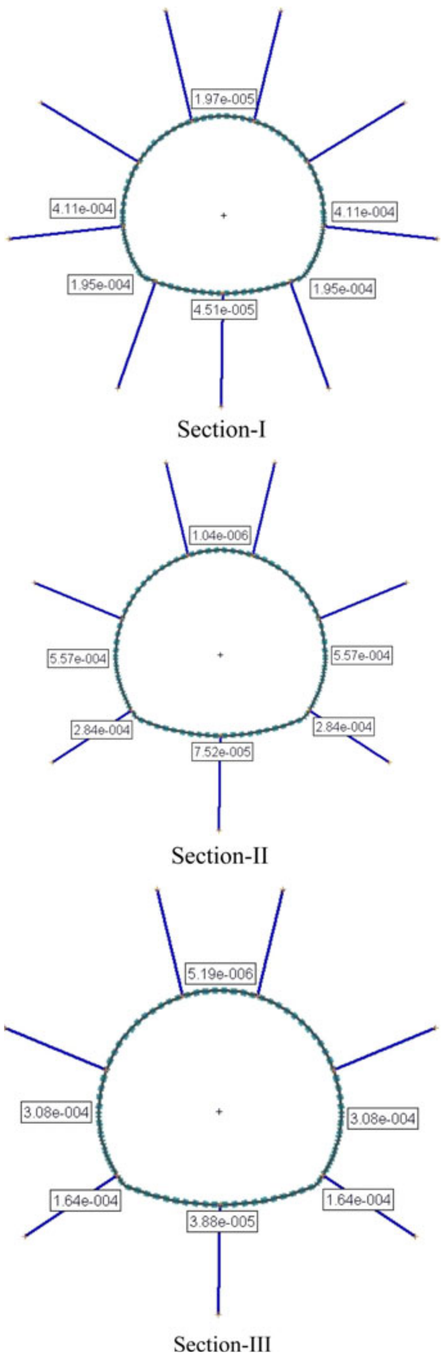

Fig. 9 Radius of plastic zone and maximum total displacements for each section

diversion tunnel at the Kapikaya dam site. Based on the information collected in the field and the laboratory, the RMR and RMi classification systems were used to characterize the rock masses. These classification systems were also employed to estimate the support requirements for the diversion tunnel. The Hoek–Brown parameters and support measure recommendations from the empirical results were used as input in the numerical analyses.

According to the results obtained from the empirical and numerical analyses, there were some stability problems with the diabases. The empirical methods recommend the utilization of bolt and shotcrete as support elements. The results of the numerical method show that the diabases would be expected to experience some deformation. Numerical modeling was used to evaluate the performance of the recommended support system. When this was applied, displacements were reduced significantly in the numerical analysis. The results obtained from the empirical and numerical approaches were fairly comparable. However, the validity of the proposed support systems should be checked by comparing the results obtained by a combination of empirical and numerical methods with the measurements carried out during construction.

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