Engineering geological investigations along the Ankara subway extension

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Abstract: The purpose of this paper is to assess the engineering geological characteristics of rock mass and to recommend appropriate support for a tunnel section between Kuyubasi and Dutluk stations in the Kecioren metro line. The Kecioren metro line is part of the Ankara subway tunnel project, planned by the Greater Municipality of Ankara, Turkey. Laboratory experiments and field studies were conducted for the study area. Field studies consisted of geological mapping, discontinuity surveying, core drilling and sampling for laboratory studies. Uniaxial compressive strength tests, triaxial compressive strength tests and deformability tests were conducted in the laboratory. The support types required for the tunneling, the categories of the rock masses and the rock mass strength parameters were determined based on the Q and RMR rock mass classification systems. Two dimensional finite element analyses were utilized to analyse the interactions of the tunnel support with the rock mass surrounding the tunnel.

Résumé: Le but de cet article est de determiner les caracteres de géologie de l'ingénieur materiaux et de recomenderer une section converable du tunnel pour la ligne du metro de Kecioren entre Kuyuba ı et Dutluk stations. Le metro de Keçiören fait partie du Metro d'Ankara que a été planifié par le Service de Transport ferroviers de ville d'Ankara. Les études de laboratoire et de terrain ont été realisées. Les études du terain consiste a' faire des cartes geologiques, surveiller des discontinuités, prendre des carottes et des echantillons pour des examinations dans le laboratoire. Les tests de stresses compressives uniaxiaux et triaxiaux ont été realisés en laboratoire. Les types du support et des categories ont été déterminés selon les Q et RMR classifications des masses rocheuses. Les parameters des forces des masses sont evalués par Q et RMR sytems de classification des roches. Les elements finits en deux dimensions ont été utilisés pour analyser les intersections des supports du tunnel avec les masses rocheuses autur du tunnel.

Keywords: Engineering geology; finite element method

INTRODUCTION

Designing safe and economic tunnel support systems is the main goal of a designer. Rock mass classification systems such as Q and RMR have been successfully applied to many construction designs. In addition, numerical analysis techniques are widely used today in order to model and estimate the stresses and strains around tunnel. Reliable estimates of the strength and deformation characteristics of the rock masses are required for numerical approaches. Rock mass classifications such as RMR, Q and GSI are commonly used for obtaining for rock mass strength parameters.

The increase in the population of Ankara and the number of daily commuters to the city centre have considerably increased the travel congestion and created the need for further development of the transport infrastructure. The Municipality of Ankara aiming to ease the travel congestion has decided to extend the existing subway tunnels. Keciören Subway tunnel is one of the proposed extension tunnels and is located in the north part of Ankara (Figure 1).

This article presents the engineering geological investigation along the tunnel route and the preliminary support design for the subway extension tunnel between Kuyubasi and Dutluk stations, a distance of about 1.65km. The depth of the tunnel ranges from 12 to 30 m from the ground surface. The index and design properties of the intact rocks were determined and samples were collected for laboratory testing. The uniaxial compressive strength (σ_c), the Young's modulus (E), the Poisson's ratio (ν), the tensile strength (σ_t), the internal friction angle (ϕ), the cohesion (c) and the bulk unit weight (γ) were the parameters determined from testing. The rock masses along tunnel route were classified according to Q, RMR and GSI. Preliminary support design of this tunnel used both empirical and numerical approaches. Thus, this study reveals the advantage of using both approaches simultaneously. Empirical tunnel support types and categories were selected for each classification system. The strength properties of the rock masses were determined by means of RMR, Q and GSI. The interaction between the proposed support systems and ground were analysed by means of numerical modelling.

GEOLOGY OF THE SITE

Kecioren subway tunnel mainly runs through volcanic rocks consisting of tuff, andesite, dacite and agglomerate (Figure 2). These volcanic rocks are all overlaid by artificial infill.

Tuff develops a very smooth exposure, which is creamy-white and greyish to white in colour. Tuff has high porosity and it contains feldspar, quartz and mostly biotite minerals and also volcanic rock pieces such as andesite and basalt.

Agglomerates are composed of grains of andesite and basalt rocks (sand to gravel to cobble size) ranging from few millimetres to about a meter in size. The matrix of the agglomerates consists mainly of tuff and is white, grey and red in colour. Weathering was observed in this rock unit.

Andesite, which is pinkish to dark grey in colour, is mainly composed of quartz, plagioclase and orthoclase, with less abundant chloride, sericite and opaque minerals. Dacite rock unit is greyish-white in colour and includes the same minerals as andesite.

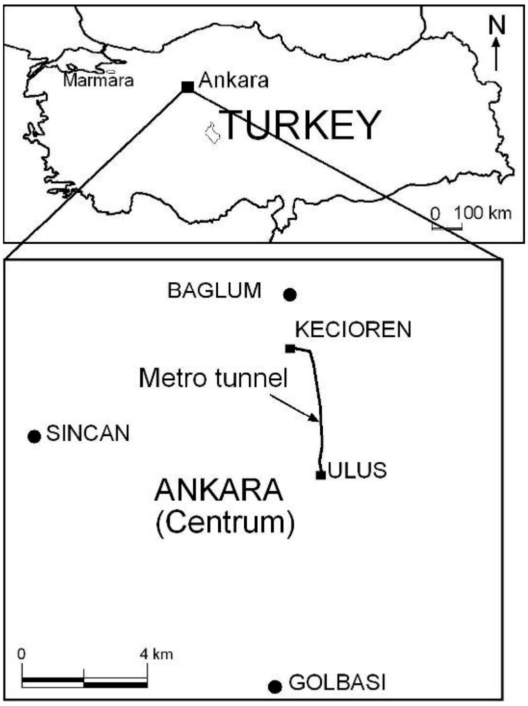

Figure 1. Location map of the project area

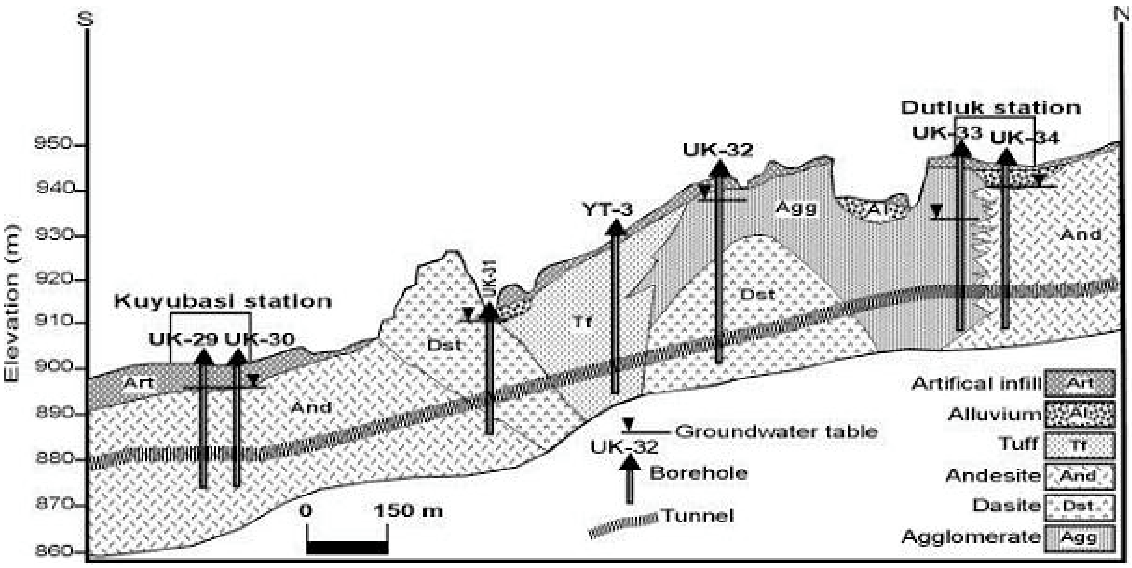

Figure 2. Geological cross-section between Kuyubasi and Dutluk subway stations of The Kecioren metro line.

ENGINEERING PROPERTIES OF THE ROCK MASSES

The engineering geological investigation consisted of field observations, drilling of boreholes and laboratory testing undertaken on the samples collected from the areas of interest. Quantitative description of rock discontinuities such as orientation, persistence, roughness, filling and aperture were determined in the field in accordance with the ISRM suggested methods (ISRM 1981).

The tunnel alignment was divided into four different zones, based on the different geological units identified. These zones are the tuff, agglomerate, dacite and andesite areas respectively. The Tuff zone was characterised as "highly and partially moderately weathered" rock faces of brown to brownish-yellow in colour. Joint sets did not appear to be well developed.

The Agglomerate zone was recorded as moderately to highly weathered. The joint spacing was mostly < 20mm and ranged from 20 to 600 mm and with low persistence (1-3 m). The joint aperture ranged from 2.5 to 10 mm. The infilling material was predominantly clay.

Andesite and dacite were slightly to moderately weathered and the joint spacing was mainly < 20mm. The persistence was medium (1- 3 m), whereas the joint aperture ranged from 2.5 to 10 mm and was mostly infilled by clay and tuffite. Weathered and fractured zones were also observed during examination of the borehole cores. Details of joint surveys are shown in Table 1.

Laboratory experiments were carried out to determine physical and mechanical properties of the andesite, basalt and the tuffs including their unit weight, porosity, uniaxial and tensile strength. Triaxial compressive strength tests were conducted on core specimens to determine cohesion, c, internal friction angle, ϕ and material constants, m_i and s_i. Deformability or stress-strain tests were conducted to determine Young's modulus (E) and Poisson's ratio (ν). Test results are presented in Table 2. All laboratory tests were carried out in accordance with the ISRM Suggested methods (ISRM 1981).

ROCK MASS CLASSIFICATIONS OF THE KECIOREN METRO TUNNEL

Rock mass classification is used to evaluate the quality and expected behaviour of rock masses in a consistent manner, and is based on the most important parameters that influence the rock mass. This has led to the development of many empirical design systems involving rock masses. Tunnel support design, pillar design and rock slope design are all examples of empirical design systems. Although there are several rock mass classification systems available, the most widely used systems are the RMR, Q and GSI which are also used in this research. It is believed that, one or more rock mass classification schemes should be used to build up a picture of the composition and characteristics of a rock mass and in order to provide initial estimates of support requirements using estimates of the strength and deformation properties of the rock mass (Basarir et al. 2005)

Table 1. Quantitative descriptions and statistical distribution of joints of the rock units at the Kecioren metro tunnel.

	Range	Description	Distribution (%)		
			Agglomerate	Andesite	Dacite
Spacing (mm)	< 20	Extremely close	49	63	46
	20-60	Very close	31	23	40
	60-200	Close	10	8	7
	200-600	Moderate	10	6	7
Persistence (m)	1-3	Low	87	88	82
	3-10	Medium	13	12	18
	10-20	High	-	-	
Aperture* (mm)	0.25-0.5	Partly open	7	11	9
	0.5-2.5	Open	11	21	17
	2.5-10	Moderately wide	82	68	74
Roughness	1†	0-2‡	25	23	15
	2	2-4	33	36	23
	3	4-6	19	25	36
	4	6-8	13	8	25
	5	8-10	10	8	8

* Aperture of discontinuities contain mostly limonite, hematite and clay infilling materials.
† Roughness profile numbers ; ‡ JRC values

The RMR rock mass classification system was initially developed at the South African Council of Scientific and Industrial Research (CSIR) by Bieniawski (1974) on the basis of his experiences in shallow tunnels in sedimentary rocks. Classification parameters were reduced from eight to six in 1974. Recommended support systems and adjustment of the rating were introduced in 1975. Class boundaries were modified in 1976 and ISRM rock mass descriptions were adopted in 1979. Uniaxial compressive strength (UCS), rock quality designation (RQD), joint or discontinuity spacing, discontinuity conditions, ground water condition and joint orientation are the utilized parameters. In order to apply RMR, the site should be divided into a number of geological structural units in such a way that each type of rock mass is represented as a separate geotechnical structural unit. In this paper the 1989 version of RMR_{89} (Bieniawski, 1989) ratings for granite, diorite and tuff formations were used.

Table 2. Laboratory test results.

Rock Unit	Unit Weight (KN/m^3)	Uniaxial compressive strength (MPa)	Modulus of elasticity (GPa)	Poisson Ratio	Cohesion (MPa)	Internal friction angle (ϕ^o)
Tuff	19.77	6.31	1.67	0.16	9.29	36.77
Andesite	22.86	38.6	4.8	0.10	9.72	53.21
Dacite	23.17	87	16.6	0.19	9.6	47
Agglomerate	22.5	40.5	14.2	0.16	-	-

Barton et al. (1974) at the Norwegian Geotechnical Institute (NGI) originally proposed the Q system of rock mass classification based on 200 case studies of tunnels and caverns. In 1993, the Q system was updated to include 1000 cases (Grimstad and Barton, 1993). RQD, joint set number (j_n), joint roughness (j_r), joint alteration (j_a), joint water reduction factor (j_w) and stress reduction factor (SRF) are utilized to calculate Q value as given in Eq. 1.

$$Q = \frac{RQD}{J_n}\frac{J_r}{J_a}\frac{J_w}{SRF} \tag{1}$$

A stress free form of Q was defined later by Goel et. al. (1995) as Q_N. In order to calculate Q_N, SRF is taken 1, which is given in Eq. 2:

$$Q_N = \left(\frac{RQD}{J_N}\right)\left(\frac{J_r}{J_a}\right)J_w \tag{2}$$

In 2002, Q system was re-compiled to improve correlation between engineering parameters and a new parameter Q_c has been defined by Barton (2002) as below:

$$Q_c = Q\frac{\sigma_{ci}}{100} \tag{3}$$

Both RMR and Q systems incorporate geological, geometric and design parameters in arriving at a quantitative value of the rock mass quality. Both systems utilize very similar parameters in calculating the final rock mass quality rating. The main difference is the different weighting given to similar parameters.

Compressive strength is used in the RMR system directly, whereas the Q system only considers strength as it relates to in situ stress in competent rock. Both the RMR and Q systems use the geology and the geometry of rock mass, in slightly different ways. Ground water is considered and some component of rock strength is included in both systems. The lack of a stress parameter in the RMR system is the greatest difference between the two systems.

The geological strength index, GSI, was developed by Hoek et. al. (1995). Intact rock and jointing properties are used to estimate rock mass deformability and strength. GSI is based on the appearance of rock mass (e.g. very good, good) and the structure of the rock mass (e.g. blocky, disturbed and disintegrated). The 1989 version of Bieniawski's RMR classification can be used to estimate GSI. In order to obtain GSI values, five rating points were obtained from RMR_{89} values.

RMR, Q, GSI, Q_N and Q_c values for tuff, andesite, dacite and agglomerate are presented in Table 3.

Table 3. RMR, Q and GSI values of different zones.

Formation	RMR		Q		GSI	Q_N	Q_c
Formation	Value	Class	Value	Class			
Tuff	44	Fair	1.350	Poor	39	3.38	0.09
Andesite	30	Poor	0.242	V. Poor	25	0.61	0.09
Dacite	53	Fair	2.332	Poor	48	5.83	2.03
Agglomerate	56	Fair	4.266	Fair	51	10.67	1.73

EMPIRICAL SUPPORT DESIGN

As discussed above, empirical design methods Q and RMR formed the basis for the design of the temporary tunnel support during the early stages of the project. These methods were also used later in the design phase in conjunction with numerical methods to develop the final design for tunnel support. Q and RMR systems were used here, because they were more applicable to large span openings and they provided design guidelines for rock bolts and fibre reinforced shotcrete including rock bolt, shotcrete and steel sets, which were the predominant temporary support measures used for the subway tunnels. The support recommendations for the Q (Barton 2002) and RMR (Bieniawski 1989) systems together with the RMR excavation guides are given in Table 4.

ROCK MASS STRENGTH PARAMETERS

Rock mass strength parameters are necessary input data for numerical modelling. The rock mass strength parameters such as the deformation modulus (E_{mass}), uniaxial compressive strength of rock mass (σ_{cmass}) and m and s (Hoek-Brown constants) were calculated by means of empirical equations based on the Q, RMR and GSI systems.

Strength of rock mass

Goel (1994) suggested Eq. 4 for calculating σ_{cmass} based on Q_N, tunnel width (B), the unit weight of rock mass (γ), and strength of intact rock (σ_{ci}):

$$\sigma_{cmass} = \frac{5.5\gamma Q_N^{1/3}}{\sigma_{ci} B^{0.1}} \text{ (MPa)} \tag{4}$$

Bhasin and Grimstad (1996) suggested an equation for hard rocks (Q>10) as follows:

$$\sigma_{cmass} = \left(\frac{\sigma_{ci}}{100}\right) 7\gamma Q^{1/3} \tag{5}$$

Ramamurthy (1985) proposed the utilization of RMR and σ_{ci} for calculating σ_{cmass}:

$$\sigma_{cmass} = \sigma_{ci} e^{\left(\frac{RMR-100}{18.75}\right)} \tag{6}$$

Sheorey (2001) used RMR in his equation to calculate the strength of rock mass as follows;

$$\sigma_{cmass} = \sigma_{ci} e^{\left(\frac{RMR-100}{20}\right)} \text{ (MPa)} \tag{7}$$

Later, Q_c based improvement, using the normalization of Q values, has been made and σ_{cmass} of rock mass has been expressed as below (Barton, 2002);

$$\sigma_{cmass} = 5\gamma Q_c^{1/3} \tag{8}$$

Table 4. Support systems and excavation guides proposed by RMR and Q systems.

Formation	Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
Tuff	RMR=56 Fair rock	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	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.	None.
	Q=1.35 Poor rock		Systematic bolting 3 m long, spaced 1.7 m.	50-90 mm fibre reinforced	None
Andesite	RMR=30 Poor rock	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	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.
	Q=0.24 Very poor		Systematic bolting 3 m long, spaced 1.4 m.	120-150 mm fibre reinforced	None
Dacite	RMR=53 Fair rock	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face	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.	None.
	Q=2.33 Poor rock		Systematic bolting 3 m long, spaced 1.8 m.	50-90 mm fibre reinforced	None
Agglomerate	RMR=44 Fair rock	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	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.	None.
	Q=4.27 Fair rock		Systematic bolting 3 m long, spaced 2.3 m.	40-100 mm fibre reinforced	

Rock mass strength was also calculated by using Rock-Lab program developed by Rocscience (Rocscience, 2002). Rock-Lab calculates the uniaxial compressive strength of rock mass based on σ_{ci}, s and a:

$$\sigma_{cmass} = \sigma_{ci} s^{a} \text{ (MPa)} \tag{9}$$

The calculated σ_{cmass} values are given in Table 5.

Table 5. Calculated rock mass strength parameters.

Parameter	Eq. No	Tuff	Andesite	Dacite	Agglomerate
σ_{cmass}, MPa	4	2.23	0.25	0.22	0.56
	5	0.97	3.85	18.71	10.35
	6	0.60	0.92	7.09	2.04
	7	0.70	1.17	8.30	2.46
	8	4.36	5.19	14.67	13.50
	9	4.48	2.73	3.73	4.73
	Average	2.22	2.35	8.79	5.61
E_{mass}, GPa	10	0.99	0.99	9.08	5.77
	11	0.42	0.88	1.32	0.90
	12	4.40	4.54	12.66	12.00
	Average	1.94	2.13	7.69	6.22
m constant	14	2.63	2.85	6.07	5.65
	15	0.334	0.229	0.229	0.229
	Average	1.48	1.54	3.15	2.93
s constant	13	0.0068	0.0012	0.0112	0.0213
	16	0.0001	0.0001	0.0001	0.0001
	Average	0.0034	0.0007	0.0059	0.0107

Deformation modulus of rock mass

Different researchers have proposed different equations to calculate the deformation modulus of rock mass. Mitri et. al. (1994) have proposed an equation in which E_i is the elasticity modulus of intact rock:

$$E_{mass} = E_i \left[0.5 \left(1 - \left\{ Cos \pi \frac{RMR}{100} \right\} \right) \right] \text{(GPa)} \tag{10}$$

For poor rock, σ_{ci}<100 MPa, Hoek and Brown (1998) have found a correlation between E_{mass} and GSI:

$$E_{mass} = \sqrt{\frac{\sigma_{ci}}{100}} 10^{\left(\frac{GSI-10}{40}\right)} \text{ (GPa)} \tag{11}$$

Later, Q_c based improvement, using the normalization of Q values, has been made and E_{mass} has been expressed as follows (Barton, 2002);

$$E_{mass} = 10 Q_c^{1/3} \tag{12}$$

The calculated E_{mass} values are presented in Table 5.

Hoek-Brown constants of rock mass

The Hoek-Brown failure criterion for rock masses uses m_m and s_m constants. Singh et al. (1997) made the following approximations to calculate m_m and s_m constants for tunnels.

$$s_m = 0.002 Q_N \tag{13}$$

$$\frac{m_m}{m_i} = 0.135 Q_N^{1/3} \tag{14}$$

Hoek et. al. (2002) suggested some relationships between m_m, s_m and GSI as:

$$\frac{m_m}{m_i} = e^{\left(\frac{GSI-100}{28-14D}\right)} \tag{15}$$

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \tag{16}$$

where, D is the disturbance factor that depends on the amount of disturbance in the rock mass associated with the method of excavation (e.g. smoothness of blasting). In this study it was assumed that blasting quality was excellent and controlled blasting techniques were applied and thus the value of D was considered to be zero.

The calculated m_m and s_m values are tabulated in Table 5.

NUMERICAL MODELLING

To check the performances of the proposed support systems from rock mass classification system, a two dimensional finite element program called PHASE^2 (Rocscience 1988) was used in numerical analysis. Two-dimensional method is sufficient for this modelling, since in the cross-section at which the stress analysis is carried out, the dimension of the rock surrounding the tunnel is much smaller than the tunnel length. Strength and yield zone of the rock mass was estimated by the Hoek–Brown failure criterion. In this analysis, strain softening and elastic prefectly plastic post-failure strength parameters were used for all type of rock masses. The residual strength parameters were taken as half of the peak values used. Necessary rock mass strength properties were taken from the estimated values given in Table 5 above. The average values were used for the analysis of all types of rock masses.

The loading conditions for vertical stress are taken as an increasing trend with depth due to its overburden weight and is estimated by

$$\sigma_v = \gamma H \tag{17}$$

H is the depth of overburden in meters. Since the tunnel lies at a relatively shallow depth and for a conservative approach, the ratio of the in situ horizontal stress to the in situ vertical stress ($\sigma_h/\sigma_v = k$) is assumed as 2.

This model represents a tunnel of about 7-meter span, to be excavated in rock mass structure. The numerical analyses were carried out for all rock masses, which were analysed by the empirical methods. By means of the modelling with Phase^2 principal stresses, total displacements and the yielded elements around tunnels were analysed using top heading followed by excavating the entire unsupported tunnel. Then, similar support systems to those proposed by empirical methods were applied, and the performances of the temporary support systems (i.e., shotcrete and rock bolting) were investigated. Maximum horizontal (U_{xx}), vertical (U_{yy}), total displacements (U_t) and thickness of plastic zone (R_{pl}) around unsupported and supported tunnel are presented in Table 6.

For the unsupported tunnel cases, principal stresses and yielded elements around the tunnel excavated by top heading and benching are shown in Figure 3. For the supported cases, deformed zone, total displacements and yielded elements are shown in Figure 4.

Table 6. Maximum horizontal, vertical, total displacements and thickness of plastic zone around supported and unsupported tunnel.

	Supported?				Unsupported?			
	Maximum displacements			Thickness of plastic zone	Maximum displacements			Thickness of plastic zone
	U_{xx}, mm	U_{yy}, mm	U_t, mm	R_{pl}, m	U_{xx}, mm	U_{yy}, mm	U_t, mm	R_{pl}, m
Tuff	3.26	3.90	3.91	4.15	1.03	0.69	1.05	0.48
Andesite	3.52	4.16	4.16	1.95	0.90	0.50	0.92	0.15
Dacite	0.46	0.22	0.46	0.53	0.40	0.18	0.40	0.00
Agglomerate	0.63	0.36	0.63	1.06	0.42	0.18	0.42	0.00

CONCLUSIONS

In this study, empirical rock mass classification systems were used in conjunction with numerical method to estimate the stability and preliminary support design of Kecioren subway metro tunnel. Both of the rock mass classification systems, RMR and Q, were used to determine parameters for input into the numerical model. Hoek–Brown parameters and support measure recommendations from the empirical results were analysed using the numerical model. The strength parameters were estimated from the empirical analysis, carried out over the last two decades and they were compiled for use in the numerical modelling. Empirical and numerical results were generally found to be close to each other. Both of the approaches showed that the Tuff and Andesite masses could create serious stability problems from the point of failure and yielding plastic zones. The other sections of the tunnel excavated in different rock masses showed no significant stability problems. The stability problems of these sections can be overcome by installation of lighter support systems.

It is seen that usage of empirical indexes in numerical analysis gave more realistic results. As a result, it is suggested that rock mass classification systems should be used in tandem with numerical tools. Consequently, the empirical and numerical results appear to be similar to each other. The validity of proposed support systems, recommended by both approaches, should be verified by comparing predictions with actual measurements during construction.

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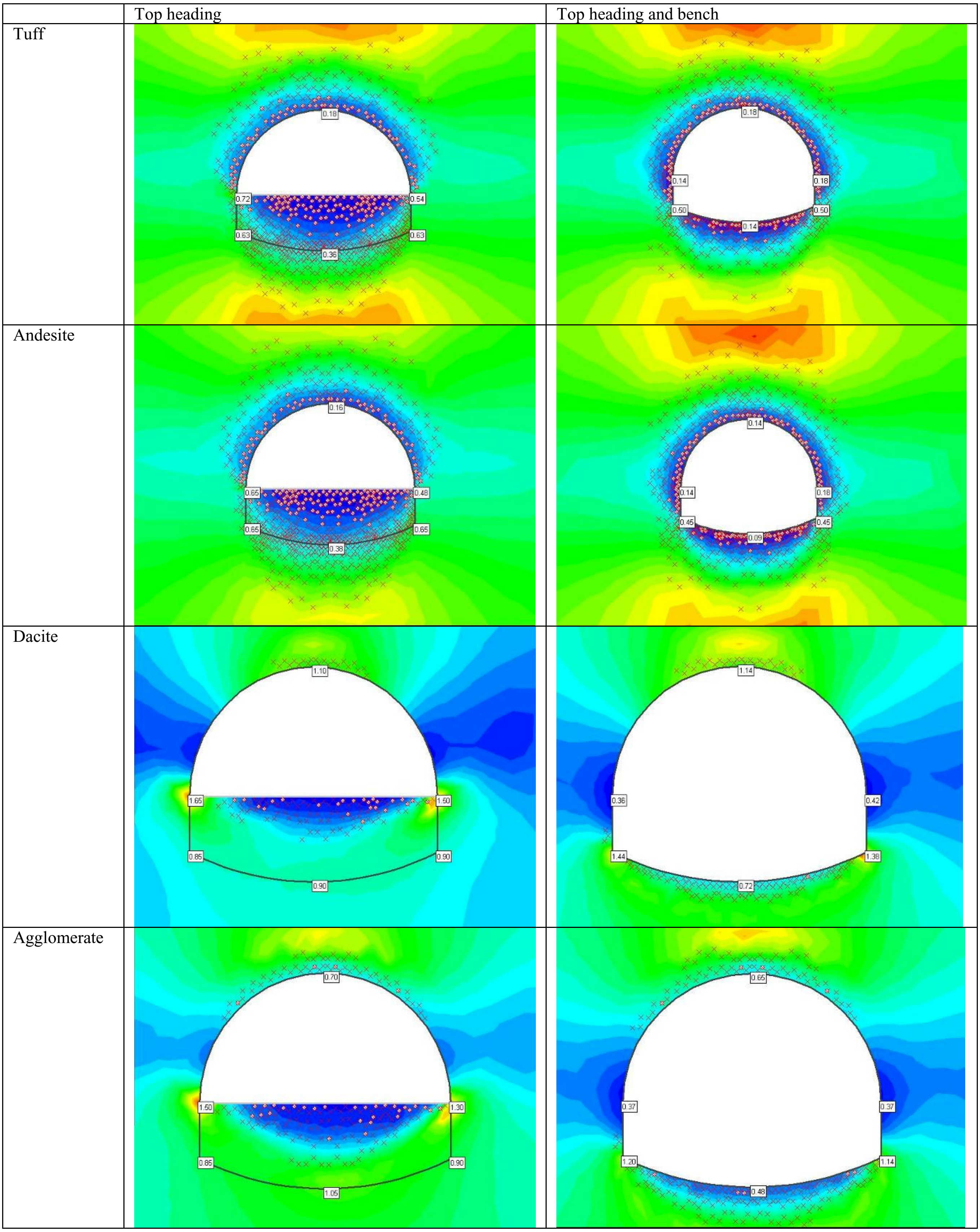


Figure 3. Stresses and yielded elements around unsupported tunnel.

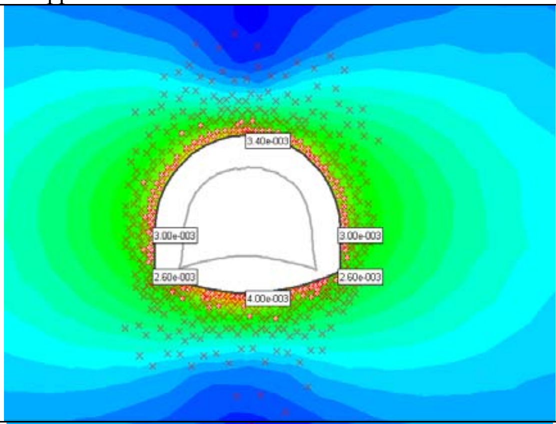
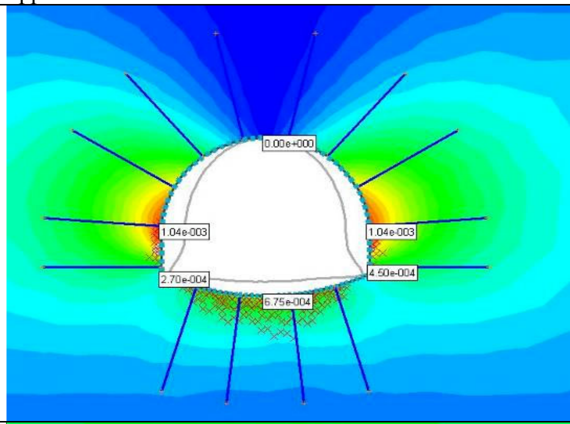
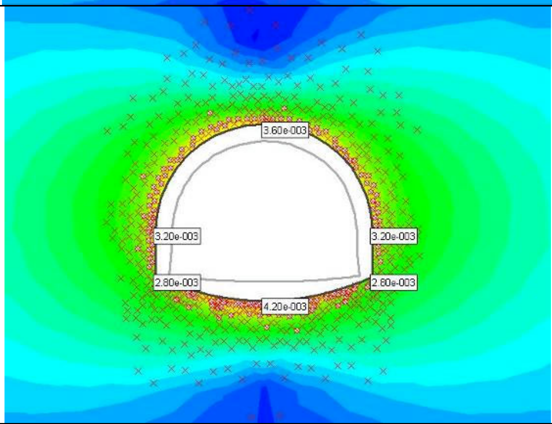
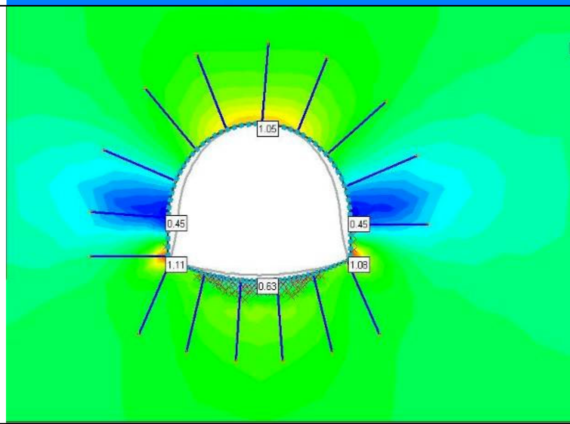
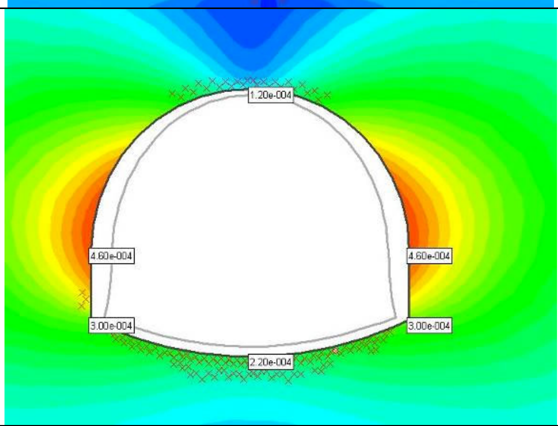
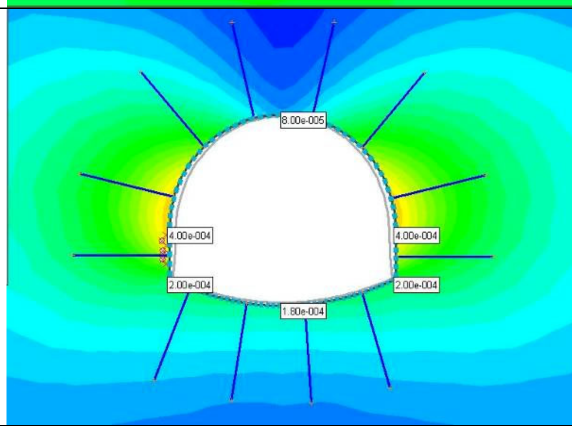
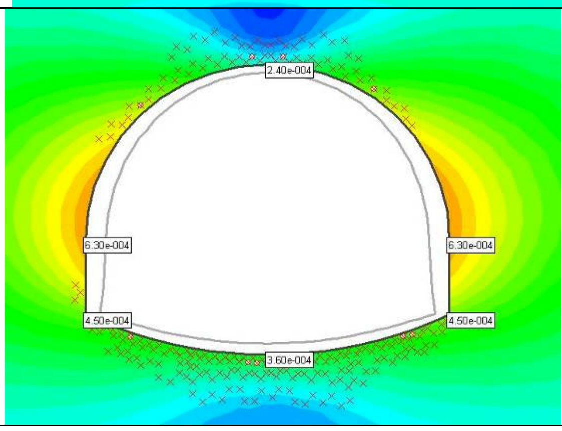
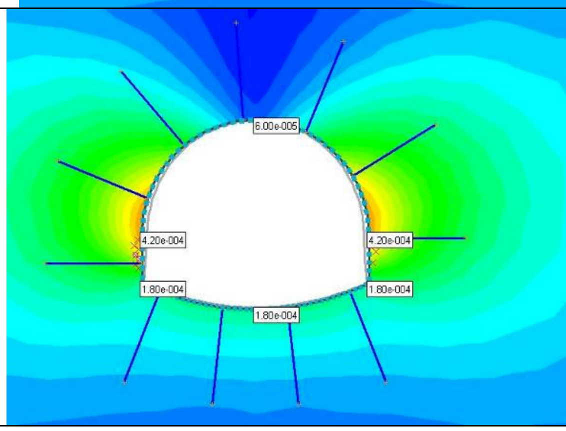
	Unsupported	Supported
Tuff		
Andecite		
Dacite		
Agglomerate		

Figure 4. Deformed zone, total displacements and yielded elements for the supported cases.

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