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Preliminary Support Design for a Diversion Tunnel at Daroongar Dam Site, Ne Iran

S. Tarigh Azali, M. Ghafoori and G. R. Lashkaripour

Department of Geology and Civil Eng., Faculty of Sciences and Engineering,
Ferdowsi University of Mashhad, Mashhad, 91775-1436, Iran

Abstract: This paper presents the results of engineering characteristics of the rock masses and tunnel support design studies, carried out at the Daroongar dam site, northeast of Iran. The diversion tunnel will be driven in rock mass, consisting of sandy limestone and limy marl of Upper Cretaceous age. Studies were carried out both in field and laboratory. Field studies include discontinuity surveying, core drilling and sampling for laboratory testing. Laboratory studies were performed on core specimens taken from the boreholes, in order to determine physical and mechanical properties of the main lithotypes. Empirical and numerical methods were used for tunnel design. The tunnel grounds were divided into sections with respect to the rock mass geological engineering properties. Rock masses were characterized using Rock mass rating (RMR), Rock mass quality (Q) and Geological strength index (GSI) system and the tunnel ground support types and categories were determined according to the RMR and Q-system. The strength and modulus of elasticity of rock masses are determined using the Hoek–Brown empirical strength criterion. The interaction of the recommended support systems by empirical methods with the rock masses were analyzed by two dimensional universal distinct element code (UDEC). It was seen that empirical and numerical approaches showed similar results. This indicates that more reliable support design could be achieved by using the distinct element method together with the empirical methods.

Key words: Engineering characteristics % diversion tunnel % GSI % empirical % support

INTRODUCTION

The Daroongar Dam has been designed as an earthfill dam with a total storage capacity of about 60 million m^3. The dam will be built across Daroongar River, about 79 km of northeast of Quchan city in the northeast of Iran (Fig 1). The purpose of the dam is to control and store water for domestic and irrigation of the Darreh-Gaz area, that is, located about 35 km northeast of dam site (Fig. 1). The diversion tunnel will be driven along the right bank of the dam that has a maximum overburden of about 60 m. The tunnel would have an excavation span of 6.2 m, a height of 8.2 m and a length of 200 m. The water level is lower than the tunnel axis.

Empirical and numerical methods are commonly used methods when underground engineering structures are designed [1-3]. In order to collect the geotechnical parameters and to predict the behavior of the rock masses surrounding the diversion tunnel, a detailed engineering geological investigation (for example, discontinuity measurements, core drilling, laboratory testing, etc.) has been performed in the rock masses surrounding the diversion tunnel. The rock mass was classified according to the Q-system, RMR method and GSI. Amongst them, the Q system and RMR method were used for rock support system design and the GSI system was used for estimating design parameters. The RocLab software was used to determine the shear strength parameters and the geotechnical properties of the rock mass according to the GSI method. In order to investigate the tunnel section using the empirical and numerical methods, the tunnel grounds is divided into three sections with respect to the rock mass engineering geological properties that are evaluated at the borehole locations along the tunnel route.

In this paper, the preliminary support design of a diversion tunnel will be described using both empirical and numerical approach which is to be driven in sandy limestone of the Atamir formation, limy marl of the Abderaz formation of Upper Cretaceous age and a faulted zone. In order to verify and fortify stability and support recommendations from rock mass classification system, the distinct element method (DEM) called UDEC [4] was used in numerical analysis. Input parameters for the analyses were derived from field investigations, experimental data, rock joint characterization and from geo-mechanical properties of the intact rock.

Corresponding Author: Dr. M. Ghafoori, Department of Geology and Civil Eng., Faculty of Sciences and Engineering, Ferdowsi University of Mashhad, Mashhad, 91775-1436, Iran, E-mail: ghafoori@ferdowsi.um.ac.ir

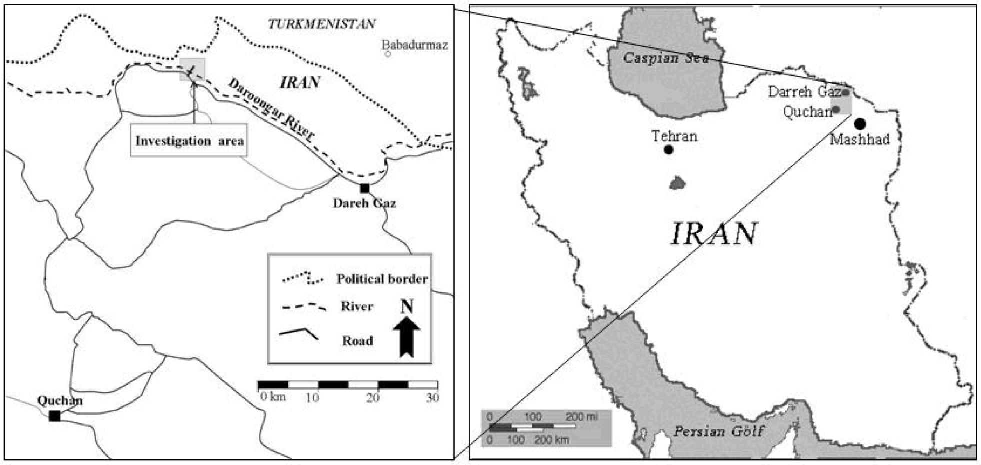

Fig. 1: Location map of the Daroongar dam site

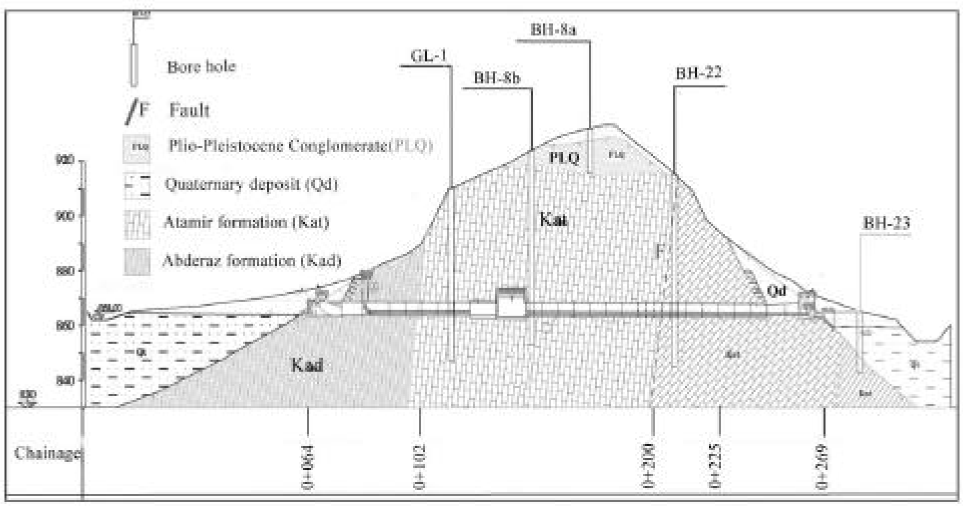

Fig. 2: Geological cross-section of the diversion tunnel alignment

Geology of the Tunnel Rout: The study area is a part of the Kopet–Dagh sedimentary zone [5]. The diversion tunnel will be driven in the sedimentary rocks of the Cretaceous age, Quaternary deposits and a fault zone. The oldest geologic unit along the diversion tunnel alignment is the Atamir formation (*Kat*) of beginning Upper Cretaceous age that is composed mainly of sandy limestone (Fig. 2). Overlying this unit is the Abderaz formation (*Kad*) of Upper Cretaceous age, comprised of limey marl. Also, the Quaternary deposits are observed at the slope of the hill and inlet and outlet portals of the tunnel cross-section (Fig. 2). During drilling works and field investigation, a fault zone with a width of about 89 m was determined (Fig. 2). The fault zone is in the NW–SE direction.

Engineering Geological Investigations: A detailed engineering geological study (core drilling, discontinuity measurements, laboratory testing, etc) was carried out in the project area and the engineering geological characteristics of the rock masses were determined. In order to precise recognition of geotechnical assessment and to verify subsurface of the tunnel rock mass conditions, a total of 285 m of drilling was performed within five boreholes along the diversion tunnel route which their location shown at geological section (Fig. 2).

A detailed discontinuity survey was carried out at the diversion tunnel route. Approximately 185 discontinuity data (joint data and bedding plane data) were measured in all sections in order to determine engineering characteristics of the rock mass.

Table 1: Joint parameters from engineering geological investigations

Sections	Rock type	Joint type	Orientation Dip dir./dip	Spacing [m]	Length [m]	Opening [mm]	Roughness
Section1	(Kat)	Main	201/81	2	5-10	2.5-10	Mod.Rough
		Main	291/85	2.5	2-5	0.5-2.5	Mod.Rough
		Main	280/42	1.5	2-5	2.5-10	Smooth
		random	077/12	5	1-3	2.5-10	Mod.Rough
Section2	(Kad)	Main	201/81	2	5-10	2.5-10	Mod.Rough
		Main	291/85	2.5	2-5	0.5-2.5	Rough
		Main	280/42	1.5	2-5	2.5-10	Rough
		random	077/12	5	1-3	2.5-10	Rough
Section3	(Kad)	Main	200/49	1	1-3	2.5-10	Mod.Rough
		Main	053/71	0.5	2-4	2.5-10	Smooth
		Main	306/74	0.4	2-4	0.5-2.5	Rough

Table 2: Summary of the laboratory testing results

Parameters		Kad rock unit Min.	Max.	Average	Kat rock unit Min.	Max.	Average
Unit weight (KN/m3)		24.1	26.2	25.7	24.9	25.6	25.1
Uniaxial compressive strength, UCS (MPa)		20.2	54.9	33.3	34.4	61.4	46.3
Modulus of elasticity, E (GPa)		5.42	9.43	7.00	10.0	12.9	11.03
Poisson's ratio (u)		0.26	0.34	0.3	0.24	0.32	0.30
Triaxialtest results	Cohesion (Mpa)	1.3	8.3	7.3	3	8.3	5.6
	Internal friction angle (F)	37.4	62.9	54.7	44.8	62.9	53.8

The following parameters were measured according to the methods suggested by the International Society of Rock Mechanics [6]: orientation, spacing, persistence and roughness. In Table 1, the main characteristics of joint sets for all sections are summarized.

Detailed laboratory studies were performed on core specimens taken from the boreholes, in order to determine physical and mechanical properties of the main lithotypes. The density, uniaxial compressive strength (UCS), deformability and triaxial tests were conducted according to ISRM [7] standards. The deformability parameters, Poisson's ratio (õ) and modulus of elasticity (E) were also obtained from deformability test. The results are given in Table 2.

Rock Mass Classification Systems: The geotechnical properties of the rock mass surrounding the diversion tunnel were assessed by using three empirical rock mass classification systems, namely the Q-system [8,9], the rock mass rating (RMR) [10] method and Geological Strength Index (GSI) [11,12].

In this study, in order to investigate the tunnel rout using the rock mass classification system, the tunnel length is divided into three sections along its axis. The test results of core specimens taken from the boreholes sections and site investigations were used as the input parameters for the assessment of all three rock mass classification systems. The results will be compared and discussed in the following sections.

Geological Strength Index (GSI) classification of rocks: The Geological Strength Index (GSI) is a system of rock-mass characterization that has been developed in engineering rock mechanics to meet the need for reliable input data, particularly those related to rock-mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks [13]. The GSI system is the only rock mass classification system that is directly linked to engineering parameters such as Mohr–Coulomb, Hoek–Brown strength parameters or rock mass modulus [14]. In this study, The Geological Strength Index (GSI) has been applied to classify the rock mass at individual section location.

In the project area, limy marl units of Abderaz formation and a thick sequence of sandy limestone of the Atamir formation are observed. The GSI values of these lithologies corresponding to the rock mass quality at each section (through using the extended GSI table [11]) are denoted by circles in Fig 3. According to this figure, the rock mass at sections 1 (limy marl unit), 2 (sandy limestone) is classified as fair to poor quality and at section 3 (faulted zone) of diversion tunnel as poor quality. The rock masses observed at section 1 and 2 might be described as blocky to very blocky, which ranges from fair to poor range of categories (Fig. 3). In section 3, Most of rock masses are described by the term very blocky to blocky/disturbed since the rock masses are often broken with angular blocks formed by many intersecting discontinuity sets.

RMR and Q classification systems: The summary of the results of the rock mass classifications for all three sections along the diversion tunnel, according to the RMR and Q systems, are presented in Table 3. As it is shown in Table 3, the rock masses in section 1 and 2 according to the RMR system, are classified as fair rock quality (class II) while section1 according to the Q system are classified as poor rock quality. The rock masses in the sections 3 (at faulted zone), according to the RMR and Q systems, are classified as poor rock masses.

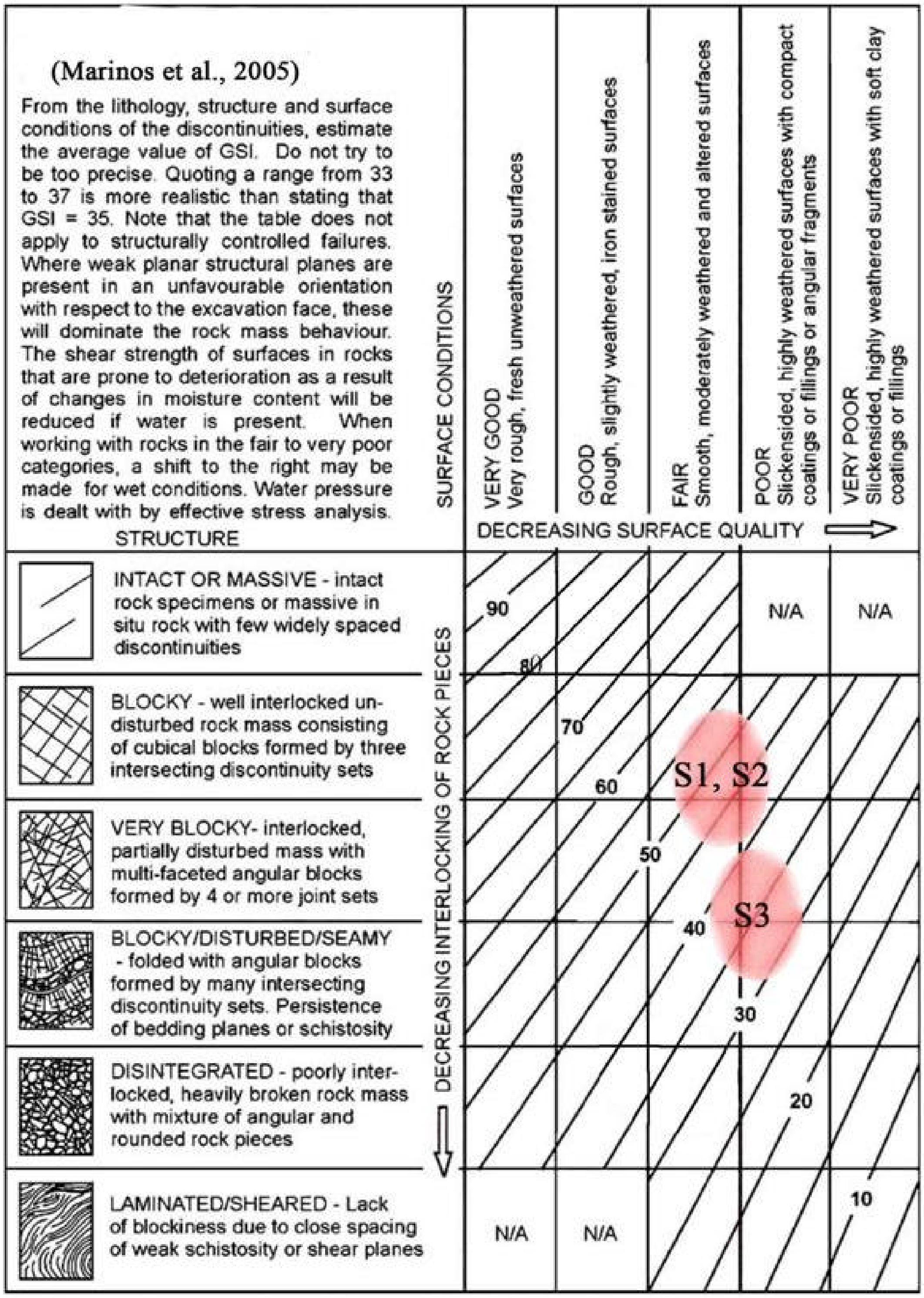

Fig. 3: Estimated GSI values for each section of the diversion tunnel (S1: section 1; S2: section 2; S3: section 3)

Table 3: Summary of the rock mass classification results

Section	Chainage (km)	Lithology	Borehole No.	Section length (m)	Q	RMR
1	0+064-0+102	Limey marl (Kad)	entrance portal	38	3.3(poor)	48(fair)
2	0+102-0+200	Sandy limestone (Kat)	GL1, BH8a & BH8b	98	6.6(fair)	49(fair)
3	0+200-0+269	Sandy limestone (Kat)	BH22	89	1(poor)	39(poor)

Table 4: Summary of the Q and RMR support recommendations

	Support recommendations			
Sections	Q	RMR	Support time (RMR)	Stand-up time (RMR)
1	Systematic bolting with 40-100 mm unreinforced shotcrete	Systematic bolt 4 m long, spaced 1.5– 2.0 m in crown and walls with wire mesh in crown, Shotcrete 0.05–0.1 m in crown and 0.03 m in sides	Commence support after each blast	85 hour
2	Systematic bolting with 40-100 mm unreinforced shotcrete	Systematic bolt 4 m long, spaced 1.5– 2.0 m in crown and walls with wire mesh in crown, Shotcrete 0.05–0.1 m in crown and 0.03 m in sides	Commence support after each blast	90 hour
3	Fibre reinforced shotcrete, *50* - 90 mm and bolting	Systematic bolts, 4– 5 m long, spaced 1.0– 1.5 m in crown and walls with wire mesh and light ribs steel sets spaced 1.5 m where required Shotcrete 0.1–0.15 m in crown and 0.1 m in sides	Install support concurrently with excavation	Immediate collapse

From these analyses, the support measures were defined in accordance with the recommendations of both systems. A summary of the empirical temporary support systems (including rock bolt, shotcrete, wiremesh and steel sets) and related support requirements according to RMR [10] and Q-system [9] are briefly summarized in Table 4.

Rock Mass Characterization: In order to estimate the rock-mass parameters, the Hoek and Brown [16] strength criterion, updated by Hoek *et al*. [15], was adopted:

$$s_1 = s_3 + s_{ci}\left(m_b \frac{s_3}{s_{ci}} + s\right)^a$$

where F_1 and F_3 are the maximum and minimum effective stresses at failure in Mpa, m_b is the value of the Hoek- Brown constant m for the rock mass, s and a are constants which depend upon the rock-mass characteristics, $ó_{ci}$ is the uniaxial compressive strength of the intact rock in MPa.

The Hoek–Brown failure criterion was first introduced to provide input data for the analyses required for the design of underground excavations in hard rock [16]. Due to the 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 [17]. A new classification technique, GSI, was also introduced into the criterion [18,19] which was used in concordant with the Geomechanics Classification System [10]. 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 [20]. In order to overcome these difficulties, GSI has been extended for very poor quality of rock masses [21], extremely poor quality schistose rock masses [22] as well as heterogeneous rock masses [23].

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 (F_{ci}), value of the Hoek–Brown constant for these intact rock pieces (m_i) and the GSI of the rock mass [21].

The Hoek–Brown input parameters F_{ci}; m_i and the ranges of GSI values corresponding to the rock mass quality at each section along with the rock mass disturbance factor, D [15], an estimate of the geomechanical properties (i.e., modulus of deformation, rock mass strength, etc.) of the rock masses are tabulated in Table 5.

Since most geotechnical software is still written in terms of the Mohr-Coulomb failure criterion, it is necessary to determine equivalent angles of friction

Table 5: Summary of the Geomechanical properties of the rock masses

Geomechanical properties		Section 1	Section 2	Section 3
Hoek–Brown constant (mi)		11	9	9
Hoek–Brown constant (m_b)		0.914	1.218	0.731
Hoek–Brown constant (s)		0.0017	0.00198	0.0006
Constant (")		0.509	0.509	0.518
Mohr-Coulomb parameters	Friction angle- M (degrees)	49	50	46
	Cohesive strength (c), MPa	0.247	0.377	0.259
Global strength (F_{cm}), MPa		4.14	6.676	4.861
Uniaxial compressive strength (F_{ci}), MPa		1.311	1.941	0.970
Deformation modulus (Em), Mpa		3839.34	4801.52	2549.05

Table 6: Joint parameters for each set of discontinuities

Sections	Joint set	Normal stiffness (Gpa/m)	Shear stiffness (GPa/m)	Friction angle (degrees)
Sections 1 and 2	201/81	1.7	1	36
	291/85	1.5	0.9	30
	280/42	1.8	1	39
	077/12	2	1	40
Section 3	200/49	1.5	1	28
	053/71	0.4	0.2	23
	306/74	0.5	0.3	24

and cohesive strengths for each rock mass. By using the relationship between the Hoek–Brown and Mohr–Coulomb criteria [15], the shear strength parameters of the rock mass at each section 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 [24] software was used. Tunnel depths and unit weights were estimated from Fig. 2 and Table 2, respectively.

Numerical Modelling of the Rock Mass Around Diversion Tunnel: The distinct element software package UDEC (v. 3.1) [4] was used to determine the deformations developed around the diversion tunnel and to investigate the interaction of the proposed support systems with the tunnel ground.

The Universal Distinct Element Code (UDEC) is a two dimensional numerical program based on the distinct element method for discontinuum modeling. Distinct element discontinuum models have been developed for deformation and stability analyses of multiply jointed rock masses, for instance around underground excavations [25]. UDEC simulates the response of discontinuous media (such as a jointed rock mass) subjected to either static or dynamic loading. In UDEC, rock mass is represented by an assemblage of discrete blocks, discontinuities are viewed as interfaces between these blocks. Blocks are allowed to move, rotate and deform and interfaces between these blocks can be compressed, opened and slipped at each other [26].

The tunnel is planned to be constructed a mouth-shaped tunnel with a 6.2-m span and 8.2-m height. The tunnel grounds were divided into three sections with respect to the rock mass engineering geological properties that are evaluated at the borehole locations along the tunnel route. The input data for the numerical modelling have been derived from field investigations, laboratory studies and from rock-mass classifications. The rock mass around the tunnel was modelled using the fully deformable block assumption. Intact rock in the present model is considered as an elastic–perfectly plastic material that follows Mohr–Coulomb failure criterion, while all joints satisfy the Coulomb slip model with the properties summarized in Table 6. Hydrostatic stress field is assumed.

Modeling with UDEC consisted of tow stages. In the first stage, the conditions for the excavation without any support were examined. In this stage, the model was first run to equilibrium without any use of reinforcements and the model was cycled to an equilibrium state for stress redistribution. The objective of UDEC was to check the validity of the empirical temporary tunnel support requirements given in Table 4. In the final stage, the effectiveness of the temporary support systems was investigated. Results from UDEC showing deformations around tunnel for all sections before and after the support installation are given in Figs. 4 and 5, respectively.

As shown in Fig 4, in the first (km: 0+064-0+102) and second (km: 0+102-0+200) sections of the tunnel, the rock blocks in the roof of the excavation has become detached from the surrounding blocks and is falling into the excavation. In the first (km: 0+064-0+102) and second (km: 0+102-0+200) sections of the diversion tunnel, the support systems that were empirically determined in Table 4 were used. The support used for this sections was 3-m long and 12.5-mm diameter grouted rock bolt with a 50-mm-thick and 30-mm-thick shell of shotcrete for roof and walls, respectively. As shown in Fig. 5, after installation of the support system, the instability is controlled and total displacement is reduced to 6.4-mm and 5.7-mm for first and second sections, respectively. It can be seen from Fig. 4 that the extent of instability, when

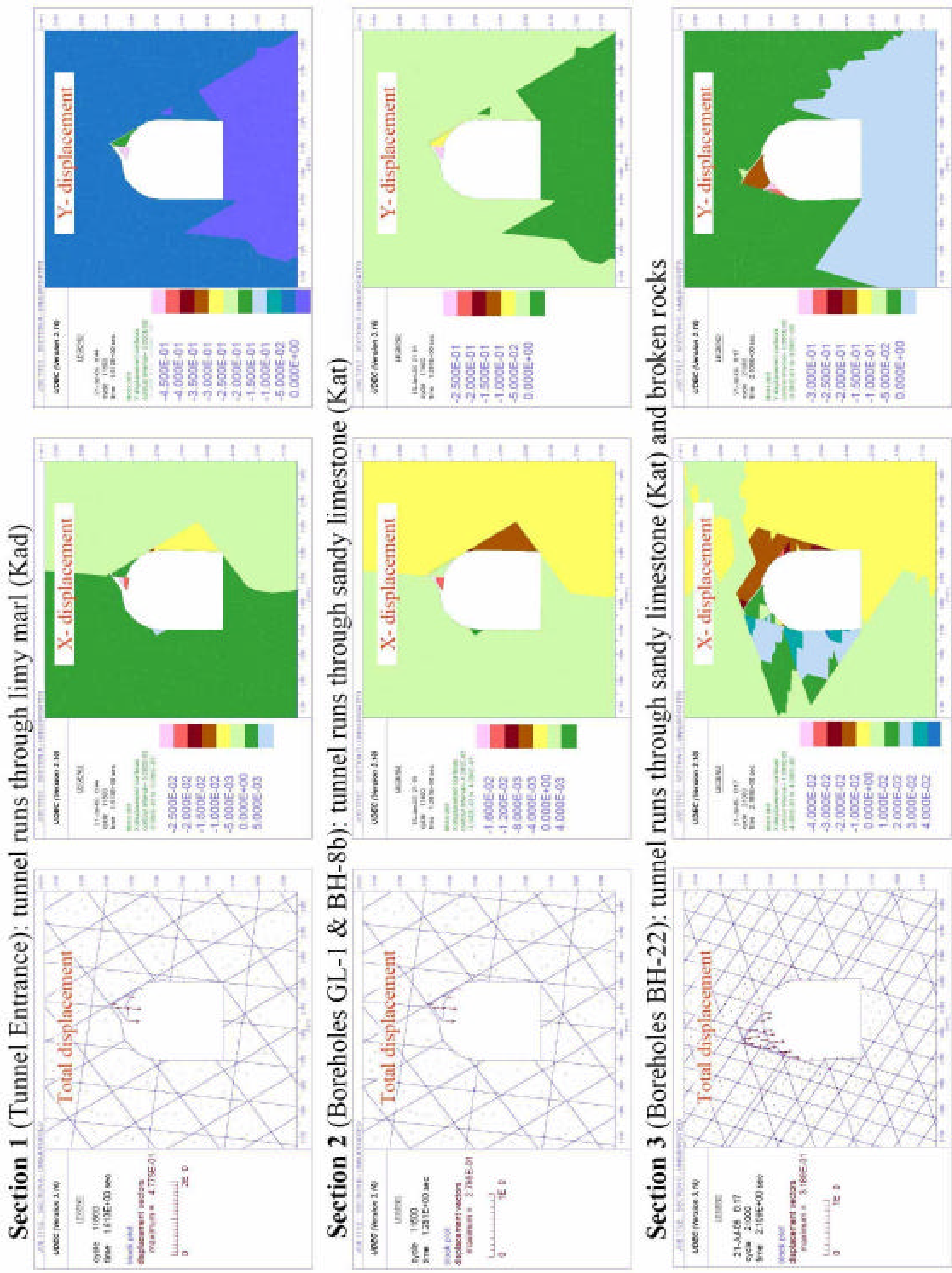

Fig. 4: The displacement behavior around the tunnel before support

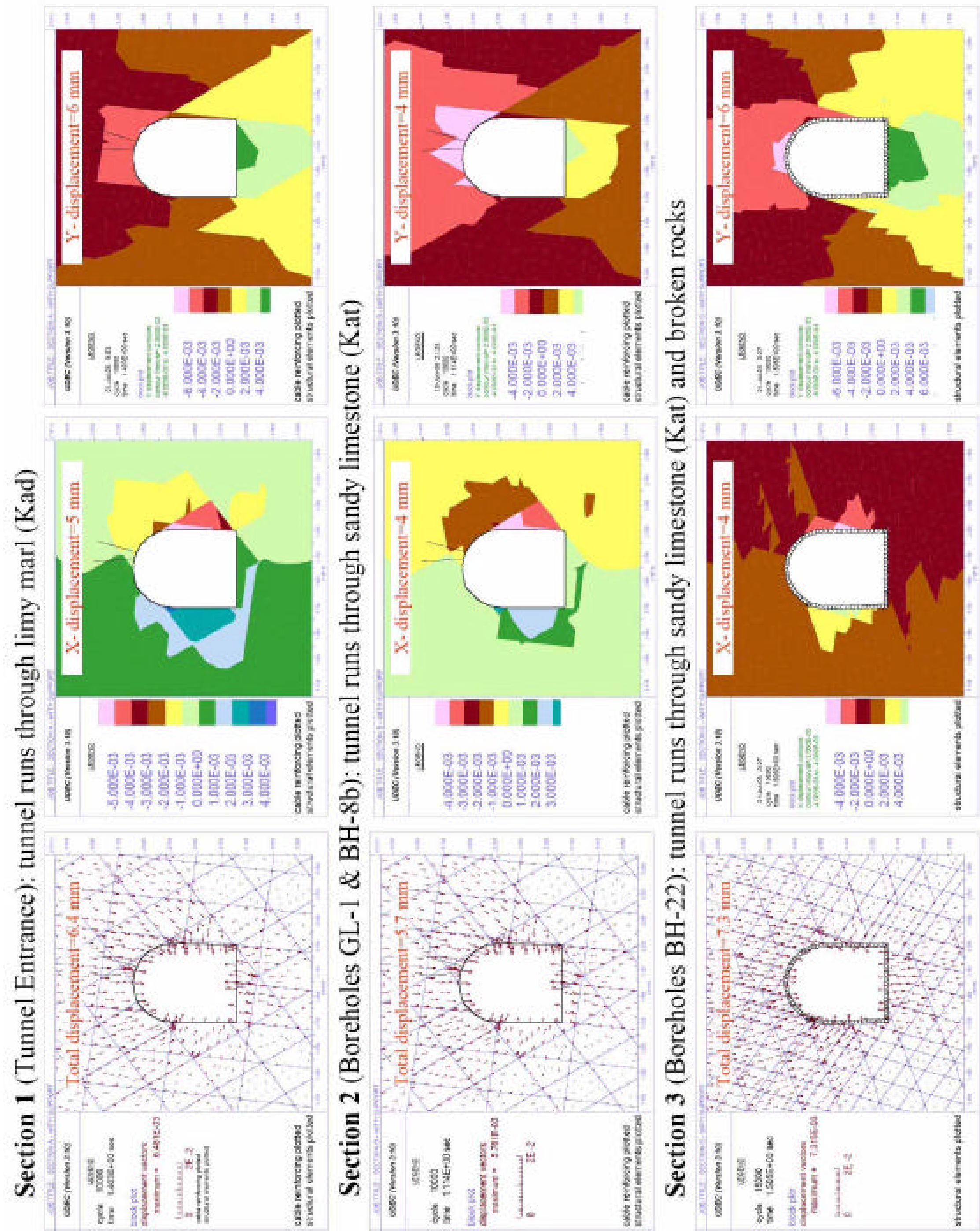

Fig. 5: The displacement behavior around the tunnel after support

there is no reinforcement on the tunnel, for the third section (km: 0+200-0+269) is more than all the other sections. The support used for this section was steel sets with an H-section and 200-mm Flange width. As shown in Fig. 5, after installation of the support system, the instability is controlled and total displacement is reduced to 7.3-mm for third section. Therefore, it is concluded that the support recommendations given for the diversion tunnel in Table 4 are satisfactory.

CONCLUSIONS

In this paper, empirical methods were used to assess the rock mass quality and support techniques along the diversion tunnel. A detailed engineering geological study (discontinuity measurements, core drilling, laboratory testing, etc.) was carried out in the project area for input data in numerical modeling studies. The rock mass systems approaches (RMR, Q-system and GSI) have been applied extensively to predict and evaluate appropriate rock reinforcement requirements for the tunnel. The distinct element software package UDEC was used to determine deformations developed around the diversion tunnel and to investigate the interaction of the proposed support systems with the tunnel ground. The empirical methods recommend the utilization of bolt and shotcrete as support elements for first and second sections. Numerical methods showed that sliding and falling of rock blocks occur around the tunnel. When the recommended support systems by the empirical methods were applied, these instabilities disappeared in distinct element analysis. The empirical methods indicate that substantial support (steel set) was necessary for third section of the tunnel and numerical method shown that the instability is more than all the other sections. However, after installation of support elements recommended by the empirical method, the distinct element analysis showed that there is not any instability around the tunnel. Upon installation of the support system, for all sections, the total displacement and instability was reduced drastically. It was concluded that the empirical support recommendations given were satisfactory. Consequently, the empirical and numerical results fit each other. The validity of proposed support systems, recommended by both approaches, should be checked with the observational method.

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