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Engineering Geological Characterization of Kallat Tunnel, Ne Iran

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

Departmet of Geology, University of Ferdowsi Mashhad, Iran

Abstract: Detailed engineering geological characterization and performance observations were carried out at the site of the Kallat tunnel in the north east of Iran. The study area consists of calcareous sandstone, limestone and marlstone overlain by a thick sequence of limestone. The studies include discontinuity measurements and laboratory testing to determine the geomechanical properties of the rocks for the tunnel site as well as the surrounding area. The strength and modulus of elasticity of rock masses were determined using the Hoek-Brown empirical strength criterion. Rock Mass Rating (RMR), Rock mass Quality (Q) and Geological Strength Index (GSI) systems were used for empirical rock mass quality determination and site characterization. The rock mass classifications (RMR, Q-system and GSI) were combined with two numerical models to investigate the overall stability of the excavation and to predict the deformation behaviour of the Kallat tunnel. Two models based, respectively, on a Finite Element Code (Phase^2) and on a Distinct Element Code (UDEC) were defined. The applicability and validity of the proposed procedure has been checked by comparing the predictions with actual observations. It was found that the actual deformations are reasonably close to those predicted through the Distinct Element method.

Key words: Rock mass · tunnel · discontinuity · numerical models · deformation

INTRODUCTION

Kallat tunnel is located at the end of Mashhad-Kalat road (northeast of Iran) (Fig. 1). The 725 m long Kallat tunnel, has two cross section types, an A-type cross section (tow ways rout, chainage 0+000-0+100) and a B-type cross section (one way rout, chainage 0+100 to +725) with cross-sectional areas of 64 m^2 and 36 m^2, respectively. In this study, rock mass behaviour and properties of the B-type cross section were investigated.

Empirical rock-mass classification systems commonly recognized as useful tools for the prediction of rock masses and the choice of support requirements on the basis of experience in similar geologic conditions [1, 2]. Rock mass properties derived from these empirical systems are commonly used, in the preliminary phase, as data input for modelling rock mass behaviour around an excavation. Numerical models can be used to compute the redistributed stress field around the excavation, to examine the mechanical response of a jointed rock mass around an unsupported or supported tunnel and to predict the expected deformation caused by each excavation phase [3-5]. Numerical models simulate the response of discontinuous media involved in tunnelling, but their results are strongly influenced both by the selection of rock and rock joint input parameters and by the joint constitutive models [6].

In this paper, field and laboratory data and numerical modelling of the rock mass around the Kallat tunnel are presented and compared. The peculiar characteristic of the rock mass is the geological complexity due to the presence of the faults and sedimentary rock at the tunnel site. Two models, a nonlinear Finite Element Code (Phase^2) and a Distinct Element Code (UDEC), have been applied for predicting the behaviour of the rock mass around the tunnel. The nonlinear Finite Element Code (PHASES) has been used to model tunnel design [7-11]. The discrete element program (UDEC) has been widely used to model static problems in jointed rock mass e.g. [12-18].

Input parameters for this study were derived from field investigations, experimental data, rock joint characterization and from geomechanical properties of the intact rock and joints. The results of the simulations, in terms of expected deformation of the rock mass have been compared to actual observations.

Suppose to be here Fig. 1.

Corresponding Author: Dr. M. Ghafoori, Departmet of Geology, University of Ferdowsi Mashhad, Iran

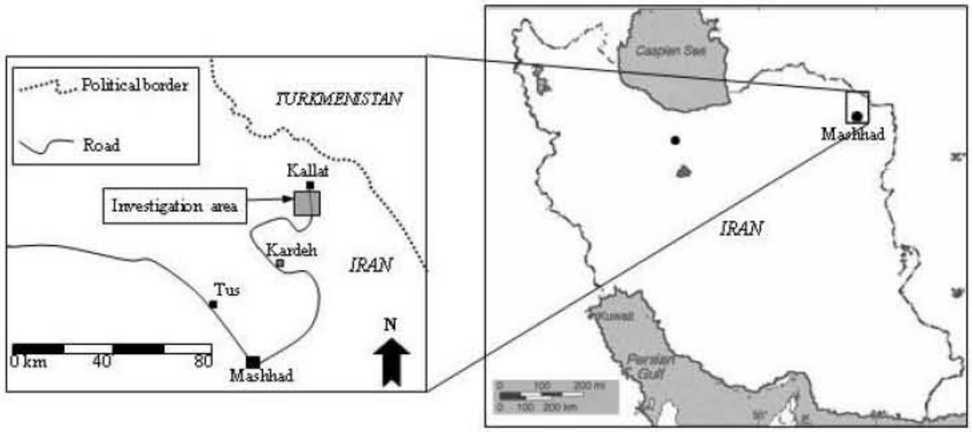

Fig. 1: Location map of the Kallat tunnel

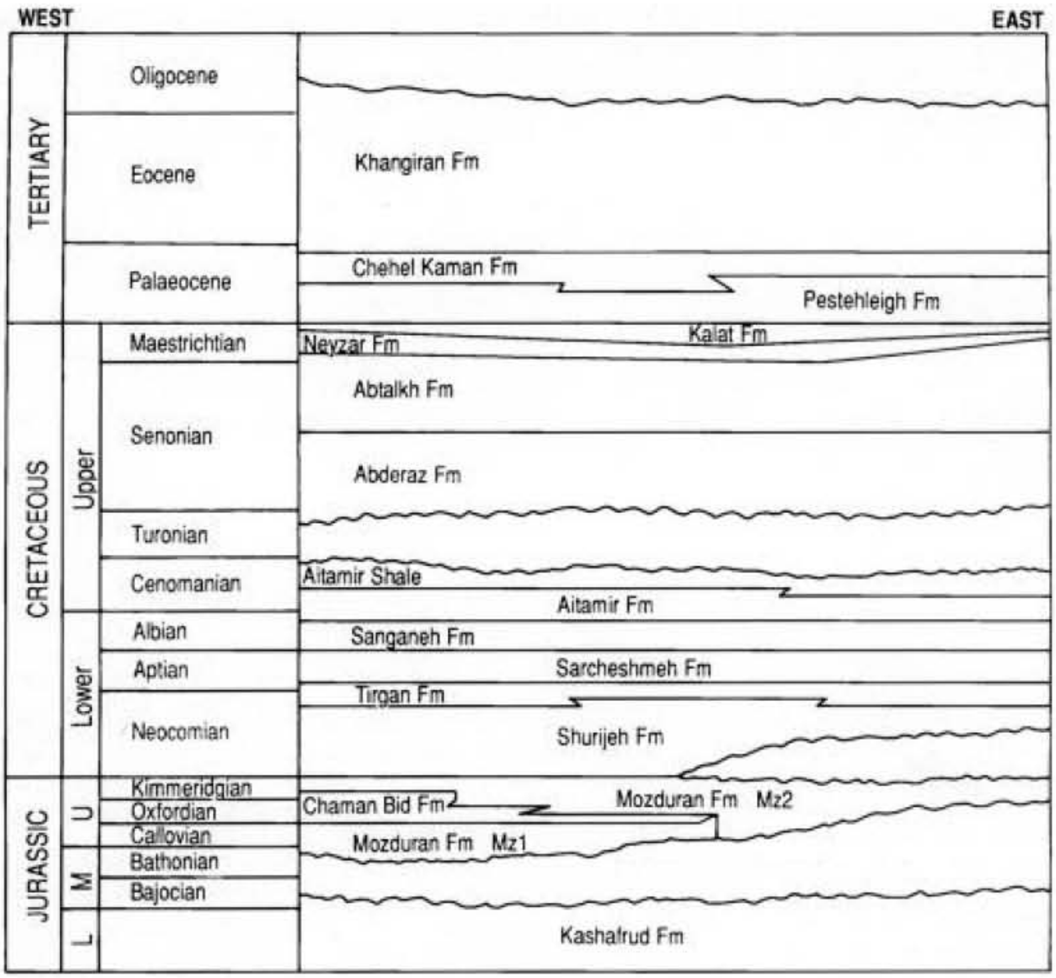

Fig. 2: Stratigraphical column for the Kopet-Dogh basin, NE Iran (after Afshar-Harb 1979)

Table 1: Summary of the laboratory testing results of the three main rock types

	Sandstone			Limestone			Marlstone		
Parametes	Min.	Max.	Average	Min.	Max.	Average	Min.	Max.	Average
Unit weight (KN m^{-3})	24.1	26.2	25.5	24.50	2.8	27.0	25.0	27.0	26.5
Uniaxial compressive strength, UCS (MPa)	52.5	60.0	55.0	42.80	50.0	45.0	31.6	39.2	35.1
Modulus of elasticity, E (GPa)	9.0	20.2	18.0	10.10	23.1	19.0	9.0	14.0	12.0
Poisson's ratio (u)	0.26	0.32	0.3	0.21	0.34	0.31	0.22	0.36	0.32

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

Rocks type	RMR	Q	GSI
Sandstone	53	4.1	48
Limestone	46	2.2	41
Marlstone	40	1.5	35

Table 3: Geomechanical parameters of the rock mass surrounding the tunnel

	Hoek brown parameters						Mohr-coulomb parameters	
Rocks type	m_b	A	S	σ_t (MPa)	σ_c (MPa)	E_m (GPa)	c (MPa)	Φ (degrees)
Sandstone	0.8	0.5	0.0004	0.027	1.02	3.965	0.29	48
Limestone	0.3	0.5	0.00013	0.02	0.47	2.397	0.19	39
Marlstone	0.1	0.5	0.0001	0.013	0.22	1.496	0.12	31

Geology of the tunnel route: Kallat area is geologically located in the Kopet-Dagh basin [19]. The Kopet-Dogh basin formed as an intercontinental basin in NE Iran and SW Turkmenistan [20, 21] and contains more than 6000 m of Mesozoic and Cenozoic sediments. The general stratigraphy of the Kopet-Dogh basin comprises 15 formations from mid-jurassic to Oligocene age (Fig. 2). The Kallat Tunnel was excavated in the Neyzar and Kallat Formations. Neyzar formation consists of sandstone and limestone in the upper part of the Kopet-Dagh Mesozoic sequence (Fig. 2). The Kallat Formation of Cretaceous age consists of alternations of Limestone and marlstone (Fig. 3). Limestone and marlstone in some areas occur as separate units but in other areas are interbedded.

Three strike-slip faults were detected during the construction of the Kallat Tunnel (Fig. 3). The dips and dip directions of these faults are 45/202, 60/002 and 83/297. The average thickness of fault zones is generally between 5and 10 m. The fault zones and overthrust zones consist of clay soil with tectonically deformed limestone and marlstone (brecciated fill) are common in this type of fault zone and flowing conditions in areas affected by groundwater are encountered.

Laboratory tests: In order to determine physical and mechanical properties of the main rock types, laboratory tests were performed on the specimens prepared from block samples to determine the geomechanical parameters of the intact rock. Density, uniaxial compressive strength (UCS) and deformability tests were conducted according to ISRM [22] standard. The results are given in Table 1.

Rock mass characterization: In order to collect the geomechanical parameters and predict the behaviour of the rock mass under excavation, a detailed engineering geological survey, including a joint study, has been performed in the rock masses surrounding the Kallat tunnel. Dominant discontinuity sets for the surrounding rock masses are as follows;

Set 1: 83/297
Set 2: 45/202
Set 3: 60/002

Based on the geological cross-sections, the tunnel alignment was divided into three different zones with different engineering geological characteristics.

Both RMR and Q classification systems [1, 23] were used to assign a rock mass quality designation (Table 2). The Geological Strength Index (GSI) was calculated [24] from the field observations in order to derive the m and s of Hoek- Brown parameters. The summary of the rock mass classifications parameters for each rock type along the Kallat tunnel, according to the RMR, Q and GSI systems, are presented in Table 2.

In order to estimate the rock mass parameters, the Hoek and Brown [25] rock-mass strength criterion, updated by Hoek *et al.* [26] was adopted:

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

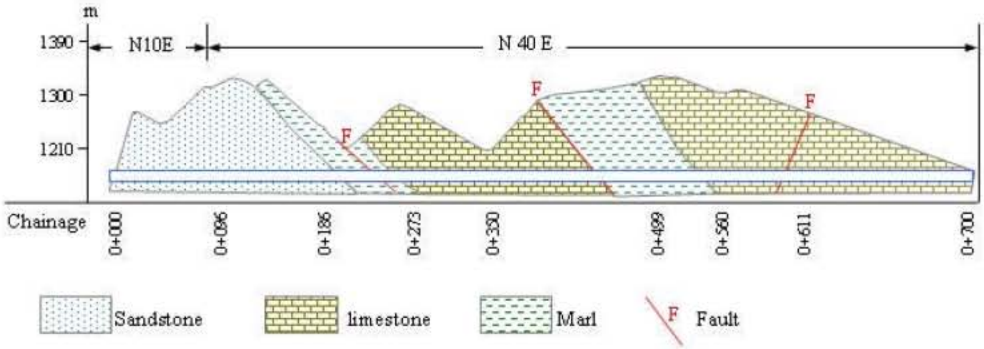

Fig. 3: Geological cross section of tunnel route

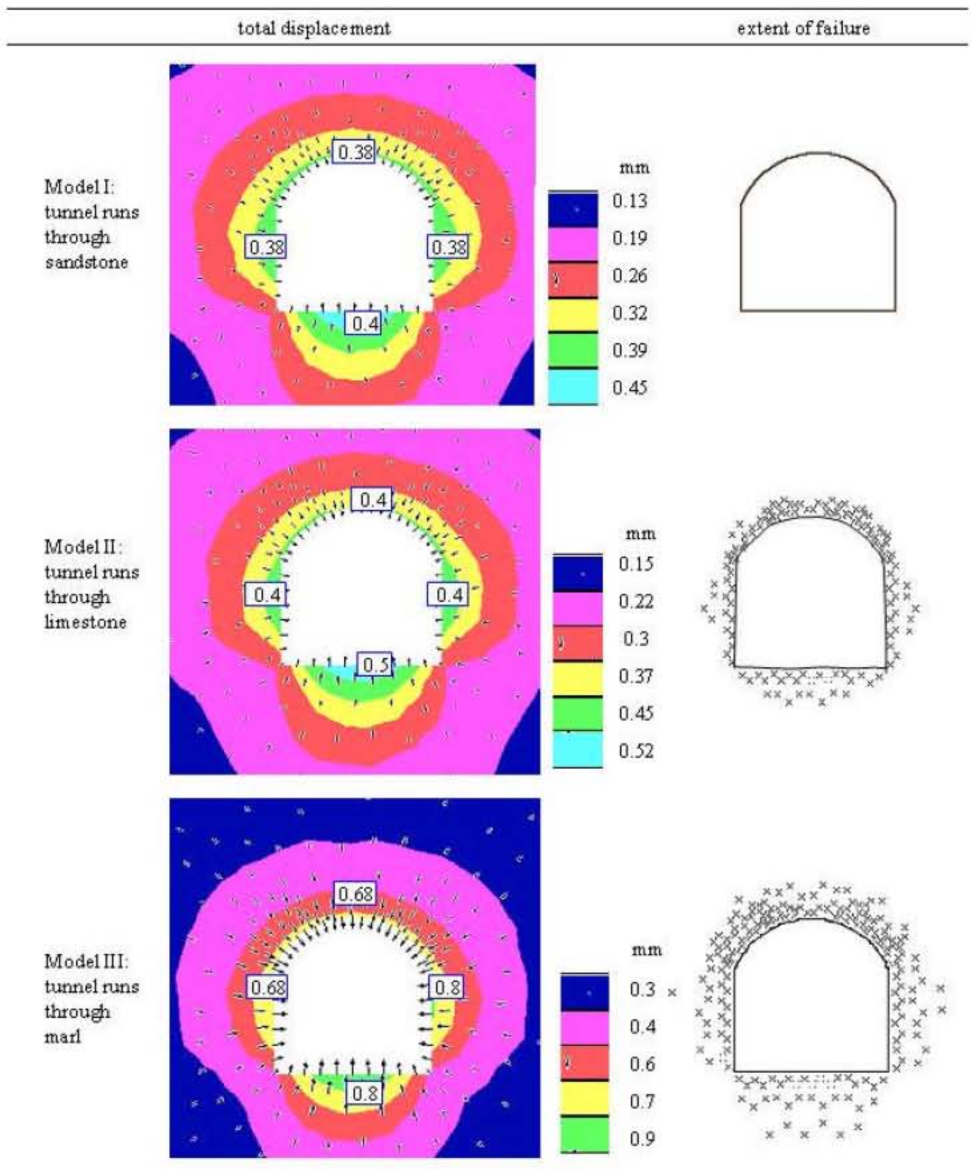

Fig. 4: The displacement behaviour and extent of failure around the tunnel (finite element analysis)

Table 4: Properties of rock joints

Joint set	Normal stiffness (GPa m^{-1})	Shear stiffness (GPa m^{-1})	Friction angle (degrees)
Set 1	0.25	0.2	35
Set 2	0.33	0.2	33
Set 3	0.33	0.1	32

where σ_1 and σ_3 are the maximum and minimum effective stresses at failure in MPa, mb, s and a are the value of the Hoek-Brown constants for the rock mass, σ_c is the uniaxial compressive strength of the intact rock in MPa. The Hoek-Brown input parameters σ_{ci} ; mi and the ranges of GSI values corresponding to the rock mass quality at each section are tabulated in Table 3.

Numerical predictions: In order to confirm the results of the empirical analyses, Two numerical modelling codes, a two dimentional hybrid element model called Phase^2 finite element program [27] and a Distinct Element program called UDEC [6, 28] were used to predict the rock masse behaviour based on field investigations, experimental data, rock joint characterization and geomechanical parameters. Rock mass displacements, measured by field observations, were compared to those predicted by Phase2 and UDEC programs.

Predictive study by finite element code: The computer program Phase^2 finite element program was used for calculating stresses and deformations of the tunnel. This software permits a 2-D study of non-linear deformation of rocks, using both the Mohr-Coulomb criterion for low quality material and the Hoek-Brown failure criterion for better quality rock mass [29]. The required parameters and their numerical values are given in Table 4. The failure criteria used by the program are Hoek-Brown criteria and a hydrostatic stress field has been assumed. With the Phase2 program, an automatic mesh is generated around the tunnel section and deformation is computed on the basis of an elastoplastic analysis for static loads. To simulate excavation of tunnels in sandstone, limestone and marlstone, three numerical models were generated using same mesh and tunnel geometry and different material properties. These models are as follows:

Model I: tunnel runs through sandstone
Model II: tunnel runs through limestone
Model III: tunnel runs through marlstone.

A numerical analysis was carried out for all three models. In order to describe the underground structure, the ground is divided into more than 396 triangular finite elements with at least 230 nodes for a single tunnel section.

In order to compare the subsequent displacements observed at the site, the numerical model (Phase^2) computed the yield zone in rock mass surrounding the tunnel and induced displacements around the excavation. The results of the finite element analysis of the tunnel, the maximum total displacements at walls, roof and floor of the tunnel and the extent of yield zones for different rock types are shown in Fig. 4. Displacements are very small and the total displacement induced by the tunnel excavation in the surrounding rock mass resulted less than 1 cm at all models. The extent of failure zone around the tunnel for marlstone (model III) is much larger than all the other sections. There is no yielded element nor any plastic zone developed around the excavation for sandstone (Model I, Fig. 4).

Predictive study by distinct element code: The Universal Distinct Element Code: UDEC model [6, 28] was used to predict deformation and perform stability analyses of multiple jointed rock masses around the underground excavations. The UDEC code enables shear and normal displacements along the joints together with deformations of the intact rock material. Discontinuities are modelled explicitly and block caving or joint slip and/or separation induced by excavation may be simulated. In UDEC, the 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 [30].

The UDEC Code is used to predict the displacements of the rock mass surrounding the tunnel during excavation. The results are compared with actual deformations [4]. The input data for numerical modelling have been derived from field investigations, rock joint characterization and from rock mass classifications (Table 3).

To simulate excavation of tunnels in limestone, sandstone and marlstone, three numerical models were generated using the same mesh and tunnel geometry and different material properties. These models are as follows:

Model I: tunnel runs through sandstone
Model II: tunnel runs through limestone
Model III: tunnel runs through marlstone.

Model I: tunnel runs through sandstone

Model II: tunnel runs through limestone

Model III: tunnel runs through marl

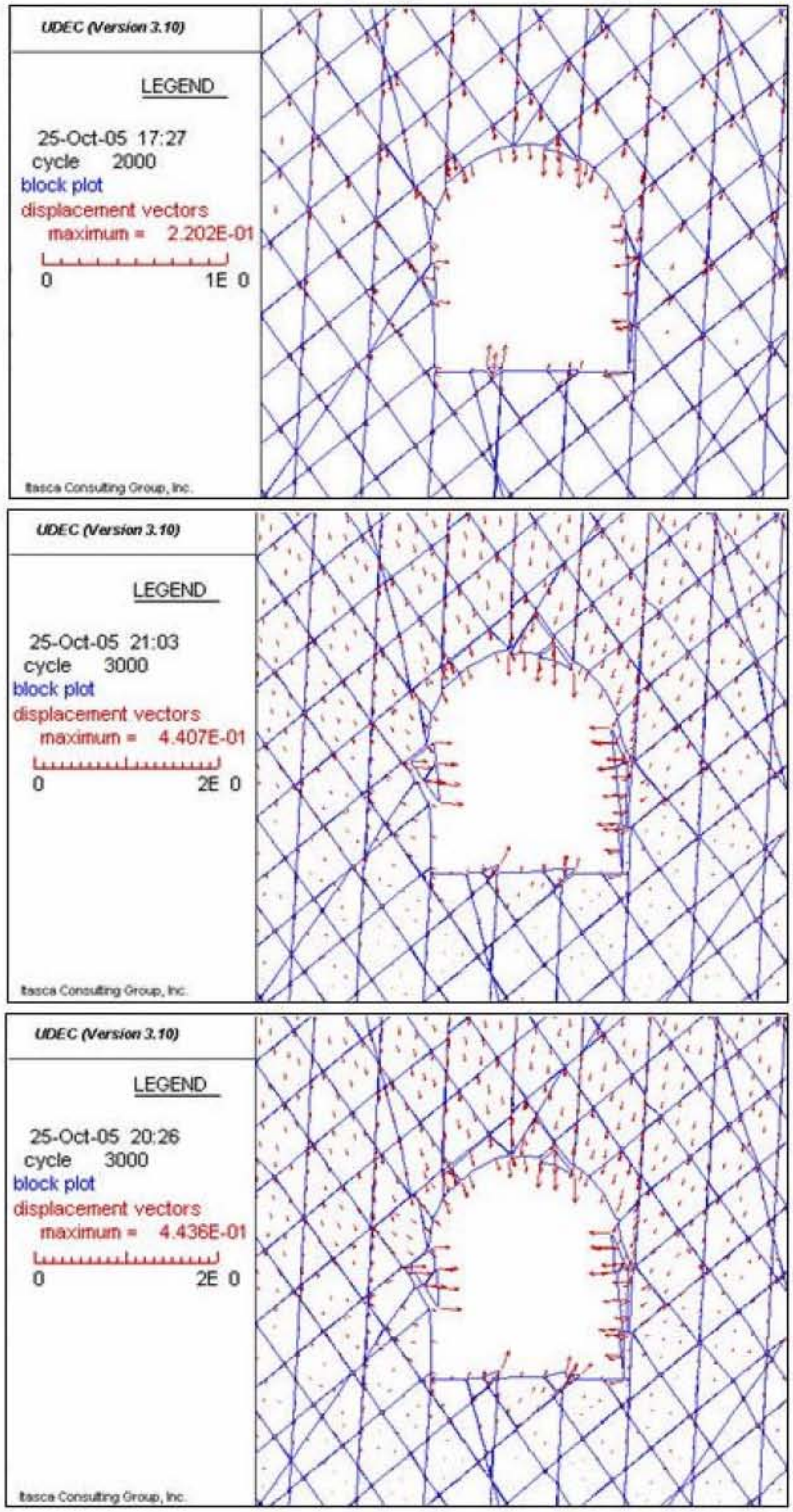

Fig. 5: The displacement behaviour around the tunnel (distinct element analysis)

Model I: tunnel runs through sandstone

Model II: tunnel runs through limestone

Model III: tunnel runs through marl

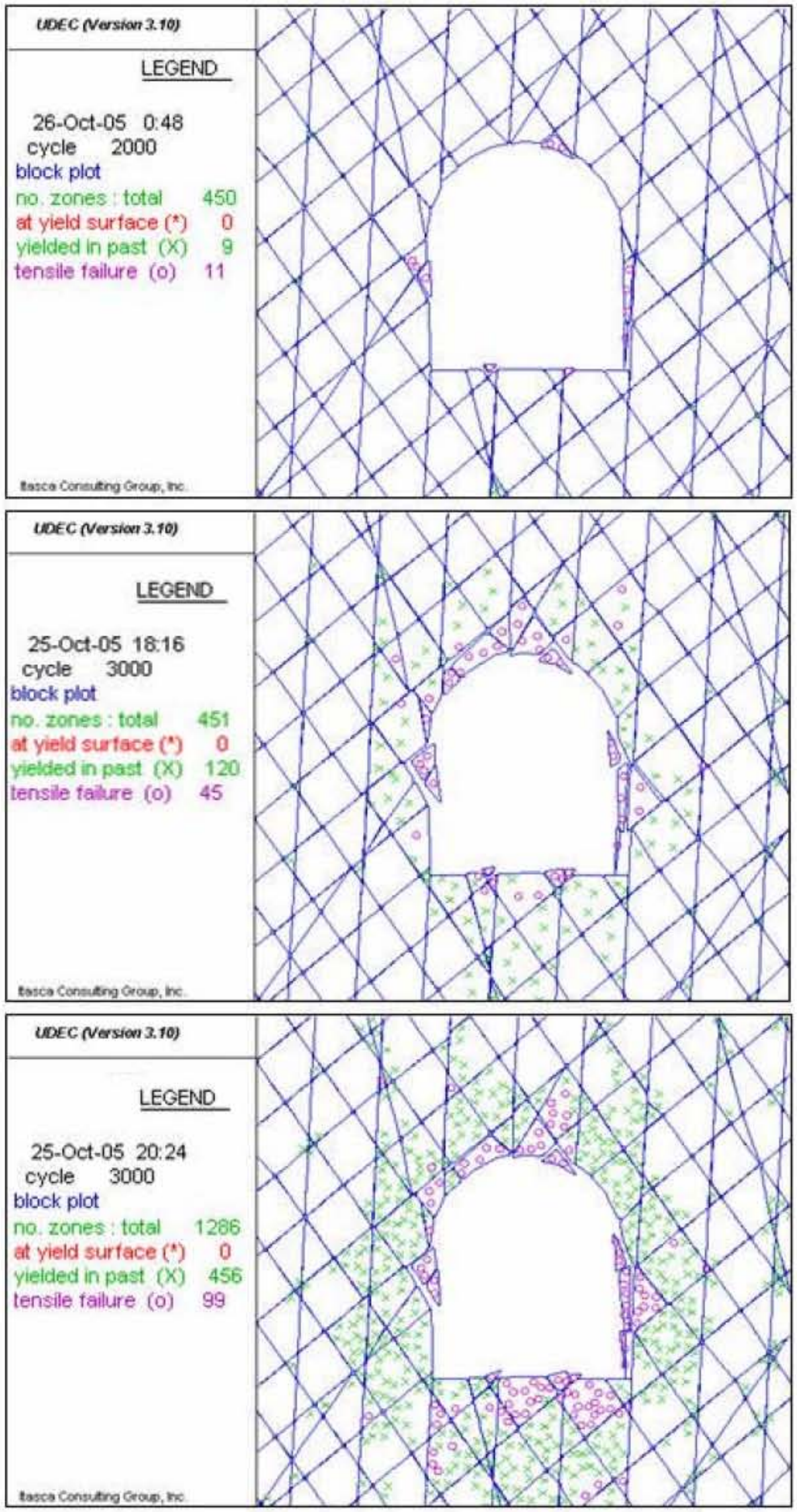

Fig. 6: The extent of failure around the tunnel (distinct element analysis)

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 4.

In order to compare the subsequent displacements observed at the site, the numerical model (UDEC) computed the yield zone in the rock mass surrounding the tunnel and induced displacements around the excavation. The results of the distinct element analysis of the tunnel, the maximum total displacements at rock mass of the tunnel and the extent of yield zones for different rock types are shown in Fig. 5 and 6, respectively. As shown in Fig. 5, the sliding and falling of rock blocks is concentrated in the walls and roof of the tunnel, respectively. Therefore, this result explains the possibility of instability when there is no reinforcement on the tunnel. The extent of the failure zone around the tunnel for marlstone (model III) is much larger than all the other sections (Fig. 6).

DISCOSSION

The Kallat tunnel was excavated by blast. During and shortly after the tunnel excavation, instabilities occurred in the rock masses surrounding the tunnel. Deformation within the tunnel was controlled using the symmetrical bolts, shotcrete and wire mesh.

The sandstone in the Kallat excavation is composed of two units. The lower unit is thickly-bedded (1-5 m), consisting of greenish grey sandstone. The upper unit consists of sandstone, more thinly bedded (0.4-1.5 m) (chainage 0+100 to 0+197). The most common types of failure in these rock units were those involving small wedges falling from the roof or sliding out of the sidewalls of the tunnel. These wedges were formed by intersecting structural features, such as bedding planes and joints, which separate the rock mass into discrete but interlocked pieces.

Limestone in the Kallat excavation consists of two units. The lower unit (Neyzar limestone) is white in colour and consists of medium bedded limestone (0.8 m) with interlayers of thinly-bedded marlstone (chainage 0+270 to 0+477). The upper unit consists of limestone (Kallat limestone) with irregular bedding (chainage 0+550 to 0+725). Rock falls and slides frequently observed in these rock units.

Marlstone in the Kallat excavation was observed in chainage 0+197 to 0+270 and chainage 0+477 to 0+550. In this area two collapses occurred and large deformations were observed during excavation.

Predictions of maximum displacement and the extent of failure around the tunnel, during or shortly after tunnel excavation, were made using Finite Element Modelling (FEM) and the Distinct Element Modelling (DEM). The results from DEM were found to be in good agreement with observations of convergence collected during tunnel excavation.

Figure 4 shows the results of the finite element analysis and Fig. 5 and 6 show the results of the distinct element analysis. The displacements in the FEM shows less than 1 cm in all the models. This result did not coincide with the field observation. The presence of multiple joint sets, which control the mechanism of failure, which cannot be defined in the FEM is one of the reasons that results in a difference in the numerical simulations, as pointed out by [6].

CONCLUSIONS

In this study, empirical methods were used to estimate the rock mass quality and support elements for rock masses in Kalat tunnel specially marlstonestone. The geomechanical properties of these rocks have been carefully assessed based on laboratory and field investigations as input data for numerical modeling studies. The rock mass characterization approaches (RMR, Q-system and GSI) have been applied extensively to predict and evaluate the rock mass properties. Numerical modeling studies (FEM and DEM) based on mapped field data and laboratory data, have used to evaluate the performance of rock mass prior to the tunnel construction. These predictive studies have been then compared with field observation. The DEM and the FEM were applied to the same section of the rock masses to compare their applicability. The DEM model rather the FEM model proved to generate more realistic results because the DEM simulates the non-linear behavior of the multiple joint sets which control the mechanism of failure since the multiple joint sets in the FEM cannot be simulated.

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