Comparison of predicted and actual behaviour and engineering geological characterization of the Kallat tunnel, Iran

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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 marl 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. 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 (PHASES) 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.

Résumé: Des observations de caractérisation et de performance d'études géologiques ont été effectuées sur le site du tunnel de Kallat dans le nord-est de l'Iran. Les zones étudiées sont constituées de grés calcaire, de calcaire, et de marne recouvert d'une épaisse série de calcaire. Les études comportent des mesures de discontinuité et des essais en laboratoire ayant pour objectif de déterminer les propriétés geomechaniques des roches du site du tunnel ainsi que de ses environs. La force et le module d'élasticité de masse des roches ont été déterminés en utilisant le critère de force empirique Hoek-Brun. Les classifications de masse des roches (RMR, système-Q, et GSI) ont été combinées avec deux modèles numériques pour étudier la stabilité globale de l'excavation et pour prévoir le comportement de déformation du tunnel de Kallat. Deux modèles basés, respectivement, sur un code fini d'élément (PHASES) et sur un code distinct d'élément (UDEC) ont été définis. L'applicabilité et la validité du procédé proposé a été vérifiée en comparant les prévisions aux observations réelles. On a pu constater que les déformations réelles sont assez comparables à celles prévues par la méthode de "Distinct Element".

Keywords: engineering geology, tunnels, discontinuities, numerical models, deformation.

INTRODUCTION

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

Empirical rock-mass classification systems are 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 (Grimstad and Barton, 1993; Barton, Lien & Lunde, 1974). 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 (Bhasin et al., 1995, 1996; Bhasin and Høeg, 1998). 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 (Cundall and Hart, 1993).

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 (PHASES) and a Distinct Element Code (UDEC), have been applied for predicting the behaviour of the rock mass around the tunnel. 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.
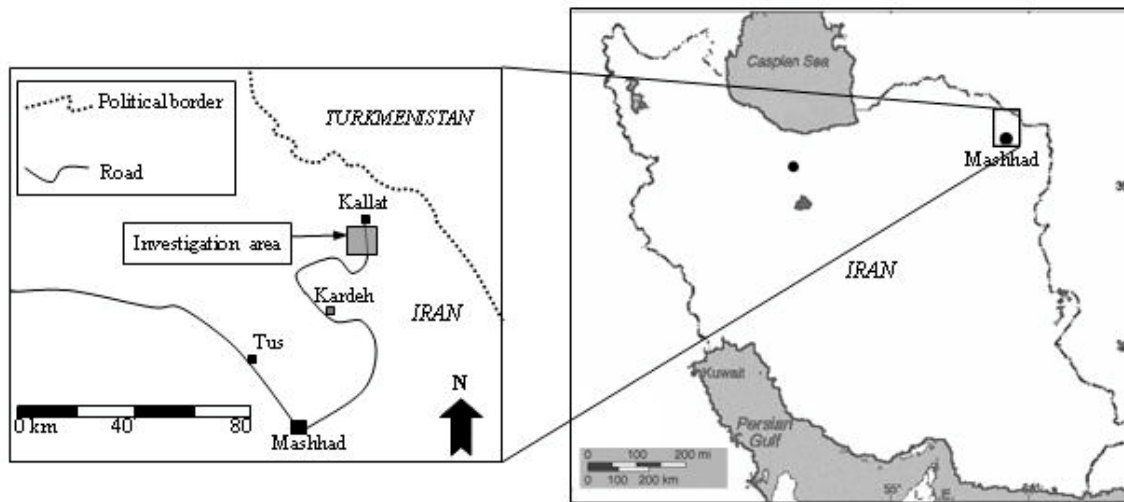
Figure 1. Location map of the Kallat tunnel

GEOLOGY OF THE TUNNEL ROUTE

Kallat area is geologically located in the Kopet-Dagh Basin (Afshar-Harb, 1994). The Kopet-Dogh Basin formed as an intercontinental basin in NE Iran and SW Turkmenistan (Berberian & King, 1981; Alavi et al, 1997), and contains more than 6000 m of Mesozoic and Cenozoic sediments. 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 (Figure 2). The Kallat Formation of Cretaceous age consists of alternations of limestone and marl (Figure 2). Limestone and marl 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 4). 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 2 and 3 m. The fault zones and overthrust zones consist of clay soil with tectonically deformed limestone and marl.
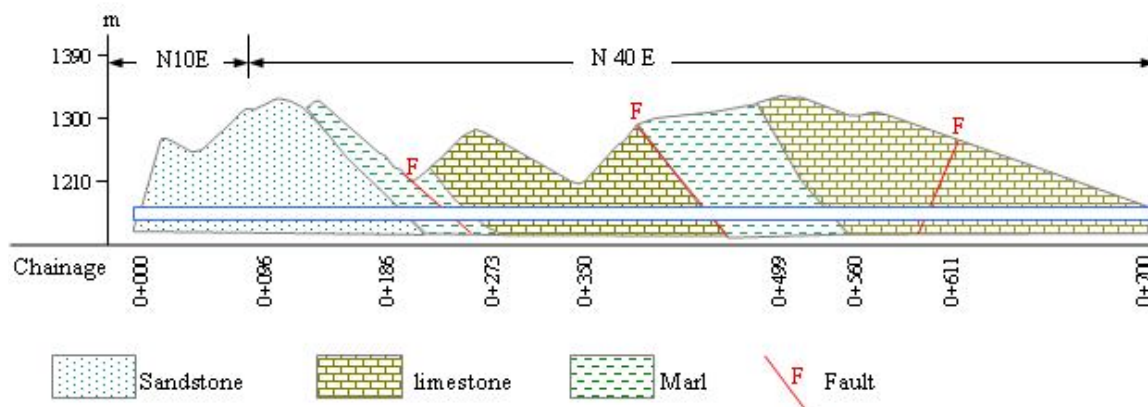
Figure 2. Geological cross section of tunnel route

LABORATORY TESTS

In order to determine physical and mechanical properties of the main rock types, laboratory studies were performed on core specimens taken from the boreholes. Density, uniaxial compressive strength (UCS) and deformability tests were conducted according to ISRM (1981) standard. The results are given in Table 1.

ROCK MASS CHARACTERIZATION

In order to collect 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.

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

Parametes	sandstone			limestone			marl		
	Min.	Max.	Average	Min.	Max.	Average	Min.	Max.	Average
Unit weight (KN/m3)	24.1	26.2	25.5	24.5	2.8	27	25	27	26.5
Uniaxial compressive strength, UCS (MPa)	52.5	60	55	42.8	50	45	31.6	39.2	35.1
Modulus of elasticity, E (GPa)	9	20.2	18	10.1	23.1	19	9	14	12
Poisson's ratio (u)	0.26	0.32	0.3	0.21	0.34	0.31	0.22	0.36	0.32

Both RMR and Q classification systems (Bieniawski, 1989; Grimstad and Barton, 1993) were used to assign a rock mass quality designation (Table 2). The Geological Strength Index (GSI) was calculated (Hoek, Marinos & Benissi, 1998) from the field observations in order to derive the m and s 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.

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
marl	40	1.5	35

In order to estimate the rock mass parameters, the Hoek and Brown (1980) rock-mass strength criterion, updated by Hoek, Marinos & Benissi (2002), was adopted:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b\frac{\sigma_3}{\sigma_{ci}} + s\right)^a$$

where σ_1 and σ_3 are the maximum and minimum effective stresses at failure in MPa, mb, s, and a are the values 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.

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

rocks type	Hoek Brown parameters			σ_t (MPa)	σ_c (MPa)	E_m (GPa)	Mohr-Coulomb parameters	
	m_b	A	s				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
marl	0.1	0.5	0.0001	0.013	0.22	1.496	0.12	31

NUMERICAL PREDICTIONS

Two numerical modelling codes were used to predict the rock mass 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: nonlinear Finite Element Code (PHASES) developed by (Rocscience, 2000) and a Distinct Element Code (UDEC) (Cundall and Hart, 1993; ITASCA, 2001).

Predictive study by Finite Element Code

The computer program PHASES finite element program (Rocscience, 2000) 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 for low quality material and the Hoek– Brown failure criterion for better quality rock mass (Hoek and Brown, 1998). The required parameters and their numerical values are given in Table 4. The failure criteria used by the program are the Hoek–Brown criteria and a hydrostatic stress field has been assumed. With the PHASES 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 marl, 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 marl.

   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 (PHASES) 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 Figure 3. Displacements are very small and the total displacement induced by the tunnel excavation in the surrounding rock mass resulted less than 1 cm in all models. The extent of failure zone around the tunnel for marl (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, Figure 3).
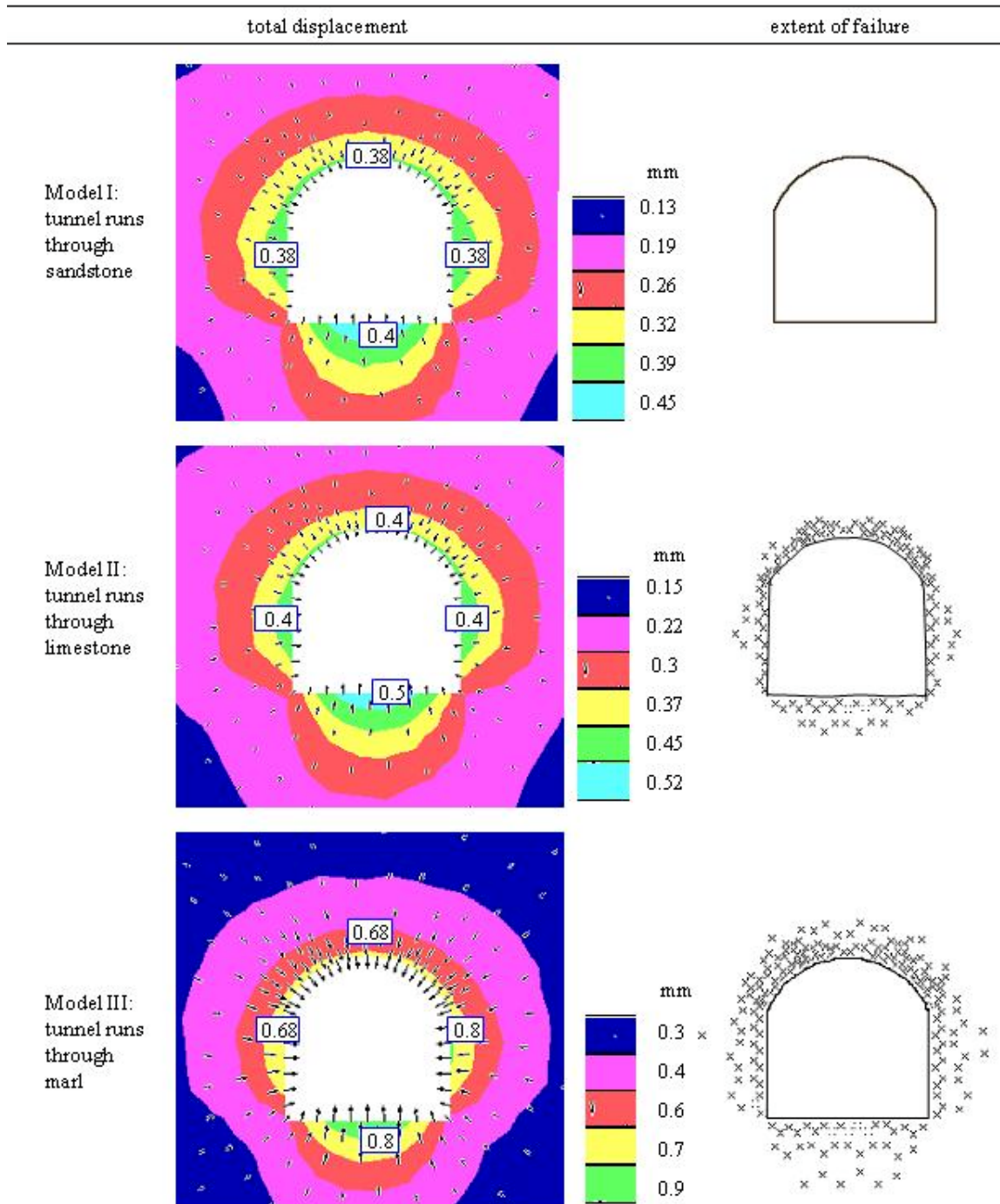

Figure 3. The displacement behaviour and extent of failure around the tunnel (finite element analysis)

Predictive study by Distinct Element Code

The Universal Distinct Element Code: UDEC model (Cundall and Hart, 1993; ITASCA, 2001) 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 (Hao and Azzam, 2005).

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 (Bhasin et al., 1996). The input data for numerical modelling have been derived from field investigations, rock joint characterization and from rock mass classifications (see Table 3).

To simulate excavation of tunnels in limestone, sandstone and marl, 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 marl.

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.

Table 4. Properties of rock joints

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

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 Figures 4 and 5, respectively. As shown in Figure 4, 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 marl (Model III) is much larger than all the other sections (Figure 5).

COMPARISON OF PREDICTIONS AND OBSERVATIONS

The Kallat tunnel was excavated by drill and 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 (chainage 0+000 to 0+095). The upper unit consists of sandstone, more thinly bedded (0.4 to 1.5 m) (chainage 0+095 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 marl (chainage 0+197 to 0+477). The upper unit is consists of limestone (Kallat limestone) with irregular bedding (chainage 0+550 to 0+725). Rock falls and slides were frequently observed in these rock units.

Marl 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 3 shows the results of the finite element analysis and figures 4, 5 show the results of the distinct element analysis. Comparing the results derived from the DEM, 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 (Cundall and Hart, 1993).
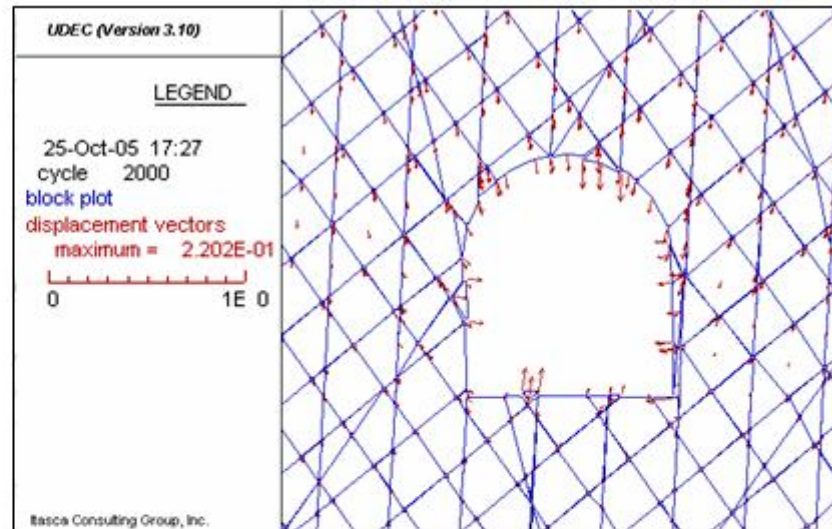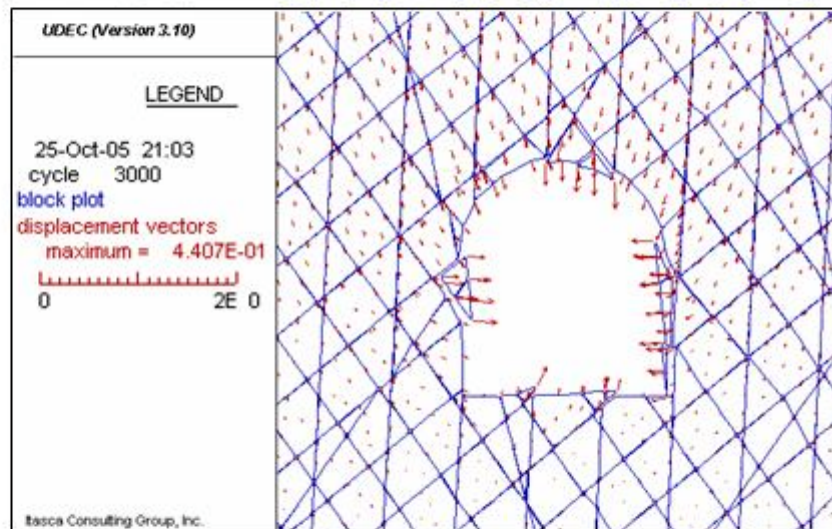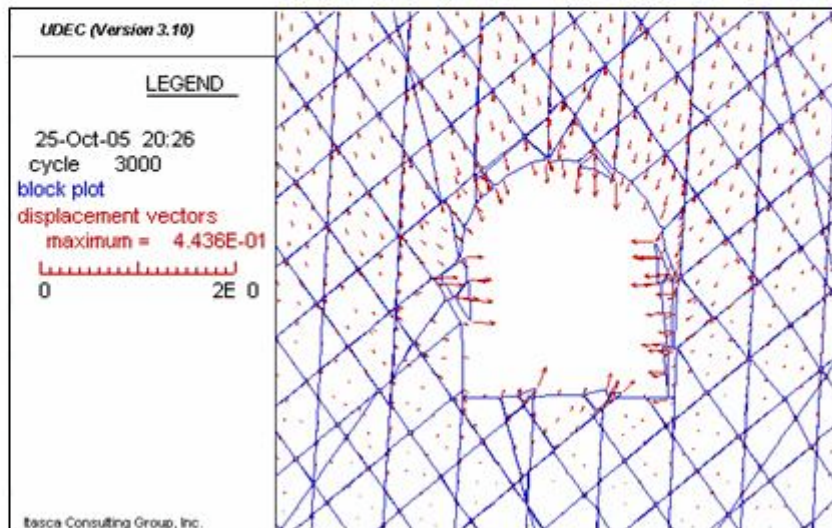
Figure 4. The displacement behaviour around the tunnel (distinct element analysis)
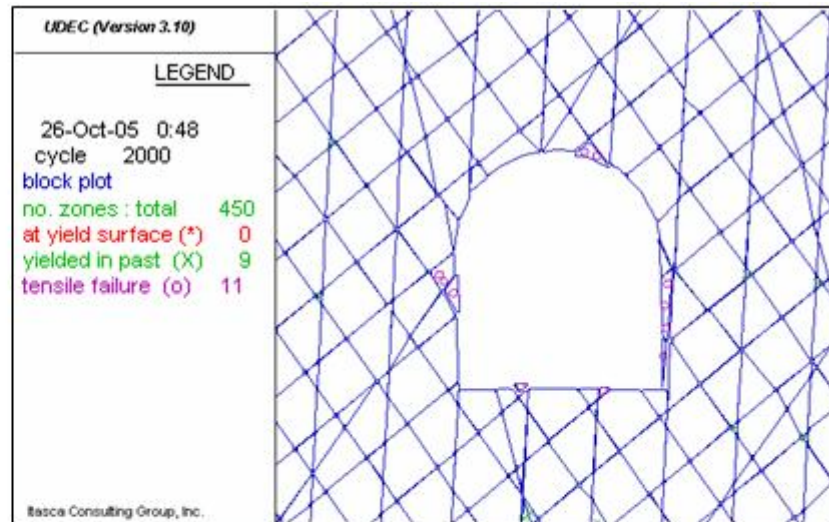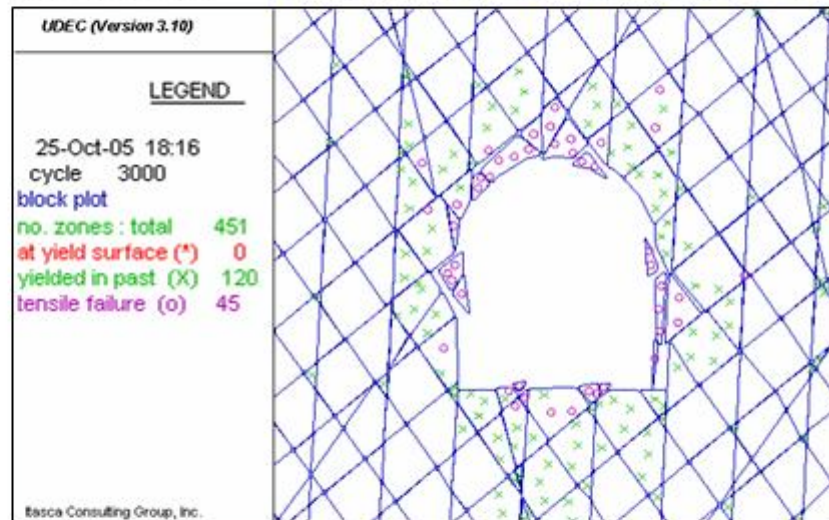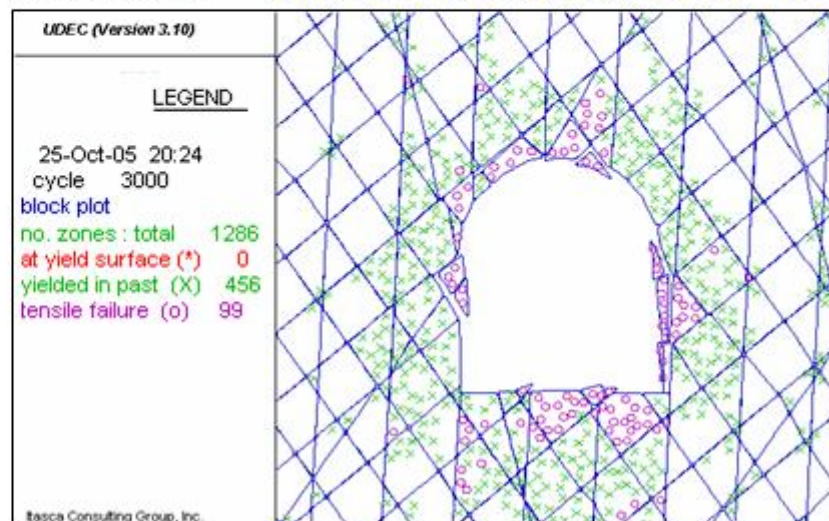
Figure 5. The extent of failure around the tunnel (distinct element analysis)

CONCLUSIONS

A comprehensive engineering geological assessment of sedimentary rocks has been carried out at the site of the Kallat tunnel in northeast of Iran. The geomechanical properties of these rocks have been carefully assessed based on laboratory and field investigations as input data for numerical modelling studies. The rock mass characterization approaches (RMR, Q-system and GSI) have been applied extensively to predict and evaluate appropriate rock reinforcement requirements for the tunnel. 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 than 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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