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Geotechnical-Geological studies and tunnel support design at Rudbar-Lorestan dam site, Iran

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Abstract
This paper presents the results of engineering geological investigations and tunnel support design studies, carried out at the Rudbar-Lorestan dam site, center of Iran. The Rudbar-Lorestan dam is to be constructed inorder to convey water for hydropower purposes. Studies were carried out both in the field and the laboratory. Field studies include engineering geological mapping, intensive discontinuity surveying and sampling for laboratory testing. Based on the results of the mapping carried out, the transfer tunnel path passes through argillaceous limestone and dolomitic limestone type of soil/rock.Empirical, analytical and numerical methods were combined for safe tunnel design. Rock mass rating (RMR), Rock mass quality (Q) and Geological strength index (GSI) systems were used for empirical rock mass quality determination, site characterization and support design. The convergence-confinement method was used as analytical method and software calledPhase[2], a 2D finite element program, was utilized as numerical method.The support system, suggested by empirical methods, was applied and the performance of suggested support system was evaluated by means of numerical modelling. It was concluded that the suggested support systems were adequate, since after applying the suggested support system to weak rock masses, tunnel deformation and the yielded elements around the tunnel decreased significantly. Thus, it is suggested that for more reliable support design empirical, numerical and analytical methods should be combined.

Keywords: Engineering geology, Tunnel support design, Empirical methods,Convergence-confinement method, Numerical method.

1. Introduction

The RudbarLorestan power plant and dam site is located within the Zagros fold and thrust belt at south of Aligudarz city (center of Iran) (Fig. 1).The height of the dam is about 150 m, and its reservoir is about 2×108m3. Also, mean annual rainfall and mean annual temperature show 650 mm and 7.7 °C, respectively.The access to the area especially the dam site for tectonically mapping is rather difficult. The dam site is close to two main structures. Main Zagros Reverse fault to the Northeast of dam and a segment of the Main Zagros Recent Fault (Saravand-Baznavid) on the Southwest of the dam site. Dolomites of Dalan formation (Late Permian) have formed both of walls [1,2].

The study area is located in the High Zagros seismo-tectonic province, an active folded-thrust belt between the convergent Arabian and Eurasian plates (Fig. 2).

Rock mass classifications were performed according to RMR, Q and GSI systems for the diversion tunnel. The properties of rock mass surrounding the tunnel, tunnel diameter, tunnel depth, geometry are the basic input parameters for a safe tunnel design [4,5]. Therefore, the rock mass properties of the site were determined by using the rock mass classification systems. Q system was also utilized to define the support requirements.

For analyzing the stability of the tunnel and for determination of necessary support systems, the convergenceconfinement method, described by Carranza-Torres and Fairhurst (2000), was used as an analyti-

Figure 1. The locationoftheRudbarLorestandam site.

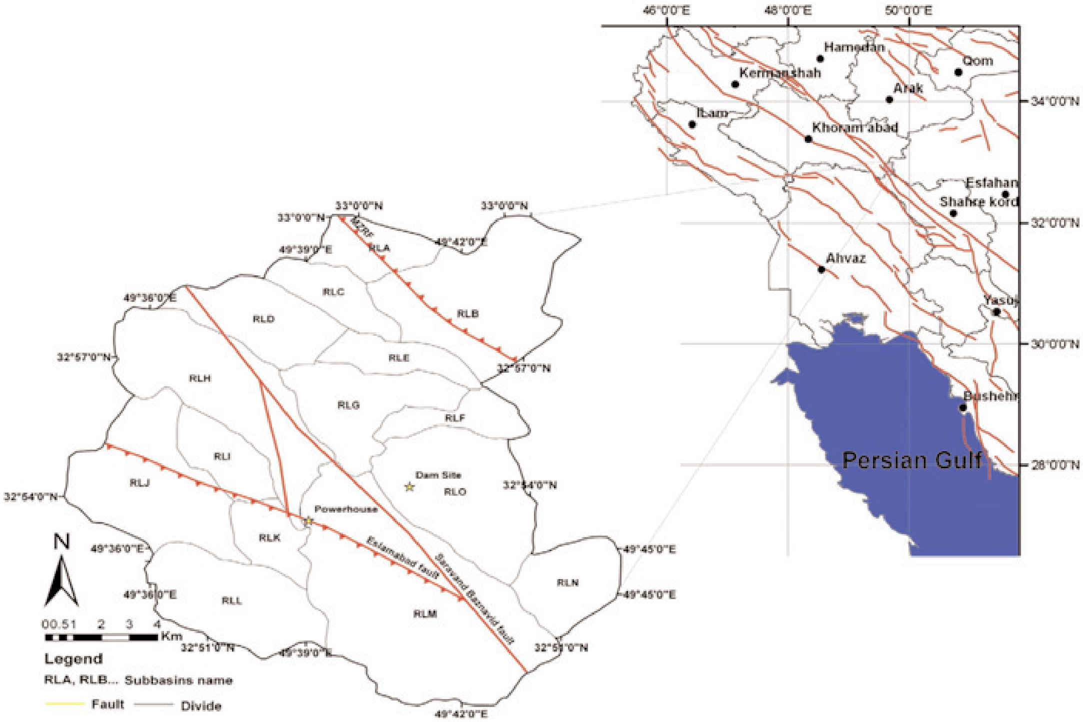

Figure 2. The tectonicsettingoftheRudbarLorestandam site [3].

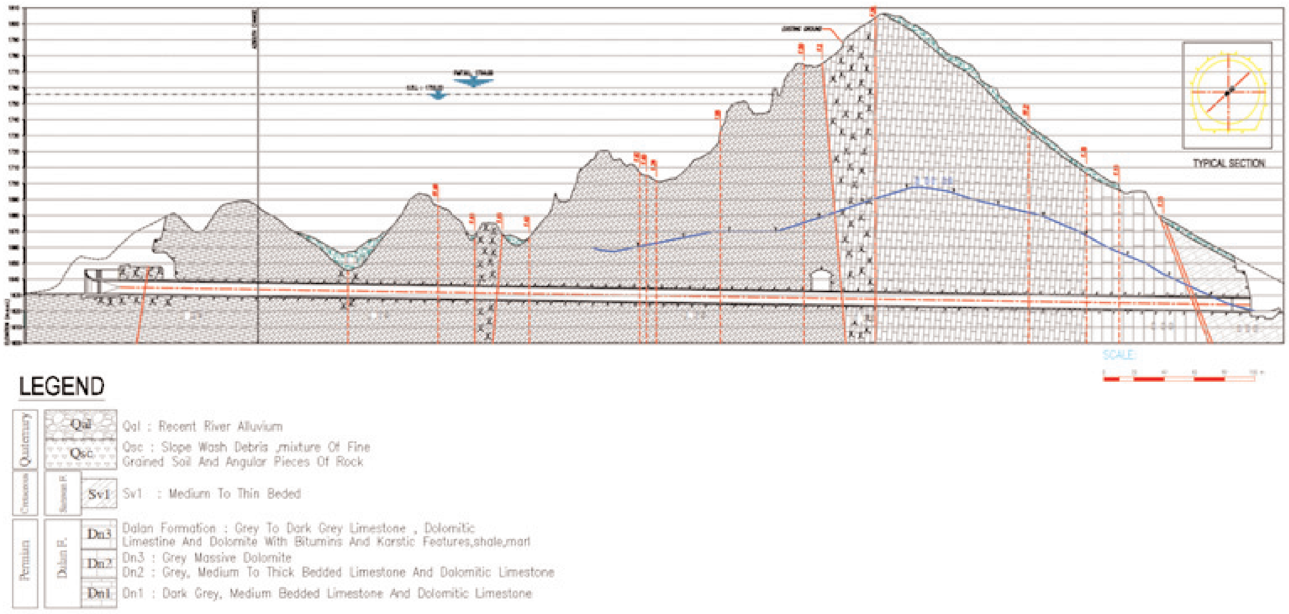

Figure 3. Engineering geological profiles of the excavated sections of the diversion tunnel [2].

cal method.

By means of numerical analysis, more realistic tunnel and ground conditions can be simulated and it is possible to obtain deformations, stresses and the thickness of plastic zone, around tunnel. Therefore, a finite element program called Phase2 (Rocscience 1998) was used in this research to analyse the stability of tunnel and the support performance. For Phase2, the strength properties of rock mass, the depth of excavation, tunnel geometry are the critical input parameters. In this study, the strength properties of rock mass were obtained by means of the empirical equations, proposed by different researchers, based on RMR, Q and GSI values.

2. Engineering geological studies

Units of studied tunnel have been distinguished on the basis of some characteristics such as lithology of layers, differences of structural features and geotechnical characteristics. In general, by considering the repeated units in different parts of the tunnel route, 5 engineering geological units are divisible. Meanwhile, in the critical section, 2 units are located. Based on the results of the mapping carried out, the transfer tunnel path passes through argillaceous limestone and dolomitic limestone type of soil/rock as shown in Fig.3.

Table 1. General parameters of the tunnel path [2].

Parameters	Properties
Length of tunnel path	800 m
Inlet tunnel free water surface	1641 m
Outlet tunnel free water surface	1630 m
Diameter of tunnel	8 m
Maximum overburden	170 m
Capacity	350 m3/s
Dip	-1.5%

The inlet and outlet portals of the tunnel are 1641 m and 1630 m higher respectively than the free water level and the maximum overburden point of this tunnel is 170 m. The geometry of the tunnel and the parameters of tunnel properties have been shown in Table 1.

The types of lithology identified are shown in Table 2. The boundaries of types of lithography are according to the stratigraphy and in many cases for the geomechanical features; the lithography was the main factor in separation and classification.The category classification of massif regional characteristics in geomechanical features is illustrated in Table 3.As is evident from Table 2, the rock mass, along the tunnel path, varies from very weak, thinly bedded, crushed

Table 2. Lithology of rock mass along tunnel.

Geology	Stability state	
	Description	Type
FL	Very weak, crushed, unstable	**C**
DL-LM	Moderately strong, thick bedding, little crushing, stable	**A**
FZ	Weak, crushed, almost unstable	**C**
DL-LT	Moderately strong, medium bedding, little crushing, stable	**A**
AL-MA	Weak to moderately strong, thin bedding, crushed almost unstable	**B**

Table 3.Category classification of massif regional characteristics and geomechanical features.

Type	Geomechanical features
A	Very strong, massive, the average distance between discontinuities being significantly more than half a meter
B	Semi-solid to solid, medium to thick layers, the average distance between discontinuity being significantly less than half a meter
C	Semi-solid to weak, thin to medium layer, the average distance of discontinuity significantly less than 0.2 m
D	Poor, crushed

and unstable to moderately strong, thick bedding and stable.

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 physical and mechanical properties of the rock units and Geotechnical properties of bedding and effective joint setsalong tunnel alignment are presented in Table 4 and table 5, respectively.

Discontinuity orientations were processed by utilizing a computer software, called DIPS 5.1 (Rocscience, 2002), based on equal-area stereographic projection and dominant discontinuity sets have been distinguished on the inlet and outlet portals of the tunnel. The determined dominant discontinuity sets on the inlet and outlet portals of the tunnel are illustrated in Fig. 4.

2.1. Engineering classification of rock masses

Some rock mass classifications suchasrock massrating (RMR),quality system (Q) and geological

Table 4. Physical and geotechnical properties of the rock along tunnel alignment [2].

Geology	UCS*(MPa)	Modulus of deformation (GPa)	mi Constant	Dry density (g/cm3)	Permeability (cm/s)	Weathering in surfaces
FL	25	1.5	6	2.4-2.5	10^{-3}-10^{-4}	Highly weathered
DL-LM	50-100 (75)	12	9	2.5-2.7	10^{-4}-10^{-5}	Slightly weathered
FZ	20	1.5	6	2.4-2.5	10^{-3}-10^{-4}	Highly weathered
DL-LT	50-80 (70)	12	9	2.5-2.7	10^{-4}-10^{-5}	Slightly weathered
AL-MA	25-50 (35)	4.5	7	2.4-2.6	10^{-5}-10^{-6}	Moderately weathered

Table 5. Geotechnical properties of bedding and effective joint sets along tunnel alignment[2].

Geology	Bedding				Discontinuities			
	D/DD*	Cohesion (KPa)	Friction angle (Degree)	Layer Thickness (cm)	D/DD	Cohesion (KPa)	Friction angle (Degree)	Spacing (cm)
FL	-	-	-	-	-	-	-	<60
DL-LM	54/064	100	34	20-100	J1=82/056 J2=81/157 J3=67/229	0	30	60-200
FZ		-	-	-	-	-	-	<60
DL-LT	83/032	100	34	50-200	J1=57/301 J2=65/131 J3=49/345	0	30	60-200
AL-MA	68/045	0	30	10-100	J1=85/280 J2=80/100 J3=30/030	0	30	60-200

* Dip/Dip Direction

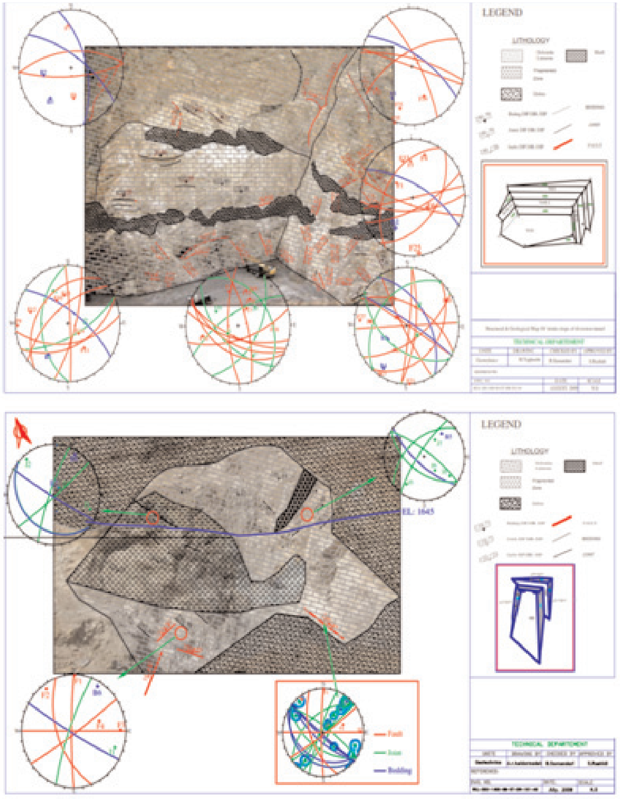

Figure 4. Dominant joint sets in inlet and outlet portals of the diversion tunnel [2].

strength index (GSI) systems have been performed on the engineering geological units of the diversion tunnel. The rock mass properties were determined using these system results.

Rock mass rating (RMR) systemwas initially developed by Bieniawski (1974) on the basis of his experiences in shallow tunnels. In this research the version 1989 of RMR (Bieniawski, 1989) has been used.

The geological strength index (GSI), is a new rock mass classification system that was developed by Hoek (1994).

The Q-system was developed as a rock tunneling quality index by the Norwegian Geotechnical Institute (NGI) (Barton et al. 1974) and the last update was released in 2002. The Q-value can be calculated as follows:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (1)$$

This classification system includes six parameters of rock quality as following:

1.Rock quality designation (RQD).

2.Number of joint sets (J_n).

3.Joint surface roughness (J_r).

4. Degree of joint weathering and alteration (J_a).

5. Joint water reduction factor (J_w).

6. Stress reduction factor (SRF)

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

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times J_w \qquad (2)$$

Hoek et al. (1995) proposed the modified Tunneling Quality Index, Q', calculated in the same way as the standard Q rock mass classification, except that the stress reduction factor (SRF) and joint water reduction factor (J_w) was set to 1.00.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \qquad (3)$$

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

$$Q_c = Q \frac{\sigma_{ci}}{100} \qquad (4)$$

Where σ_{ci} is the strength of intact rock in MPa.

These classi cation systems are used in order to estimate rock mass parameters along the critical sections. The results are presented in Table 6.

3. Rock mass properties

Rock mass properties such as deformation modulus of the rock mass and uniaxial compressive strength and Hoek-Brown constants of the rock mass have been determined using empirical equations based on Q_n, Q, RMR and GSI systems.

Table 6. The estimated rock mass classification systems.

Formations	RQD (%)	RMR	Q	GSI	Q_n	Q	Q_c
FL	<25	<35	0.1	30	0.28	0.42	0.03
DL-LM	50	50-60	3	45-55	16.25	16.25	1.25
FZ	10-25	35	3	30	7.50	7.50	0.63
DL-LT	55-65	60	1-4	50-55	15.00	15.00	1.38
AL-MA	25-50	35-50	0.1-1	30-45	0.83	1.67	0.07

It should be noted that each of experimental relations includes a part of characteristics of rock masses (based on the classification system of rock applied in that equation). Therefore, as the different relationships are averaged, this error is reduced in calculating the characteristics of rock masses [17, 18]. Of course, standard deviation was used with the effort to minimize this value as least as possible so that a real average value is obtained. (In some cases, this aimwas achievedby removing maximum and minimum).

3.1. Strength of rock mass

Different researchers have proposed different empirical equations to calculate the strength of rock mass(s) based on rock mass classi?cation systems. The most widely used equations are tabulated in Table 7. The calculated σ_{cmass} values are given in Table 8.

3.2.Deformation modulus of rock mass

In-situ determination of the deformation modulus of rock mass (E_{mass}) is costly and often very dif cult. Thus, empirical methods are generally used in estimating E methods, E_{mass}. By means of the empirical can be easily obtained. The proposed equaticfis by differentresearchers are presented in Table 9.

The calculated E_{mass}values have been given in Table 10. The $E_{mass}$was calculated using various relationships as mentioned above and it was observed that it varies from a low of 0.60GPa to a high of 21.60GPa.

3.3. Constants of rock mass

Hoek-Brown failure criterion for rock masses is based on m_m and S_m constants. Some suggested equations based on the empirical methods are used to calculate these constants. These equations are presented in Table 11. The calculated $m_m$and S_m constants are given in Table 12.

4. Tunnel support design

Although rock mass classification systems are generally applied to carry out the support design of tunnels, these systems cannot give a quantitative

Table 7. The proposed empirical equations for calculation of σ_{cmass}

Researcher	Equation No.	Equation	Notes
Kalamaris and Bieniawski (1995)	(5)	$\sigma_{cmass} = \sigma_{ci} \exp\left[\frac{(RMR-100)}{24}\right](MPa)$	For Q<10
Singh et al. (1997)	(6)	$\sigma_{cmass} = 7\gamma Q^{\frac{1}{3}}(MPa)$	
Aydan et al. (1997)	(7)	$\sigma_{cmass} = 0.0016RMR^{2.5}(MPa)$	
Sheorey (1997)	(8)	$\sigma_{cmass} = \sigma_{ci} \exp\left[\frac{(RMR-100)}{20}\right](MPa)$	
Trueman (1998)	(9)	$\sigma_{cmass} = 0.5\exp(0.06RMR)(MPa)$	
Aydan and Dalgic (1998)	(10)	$\sigma_{cmas} = \frac{RMR}{RMR+6(100-RMR)}\sigma_{ci}(MPa)$	
Barton (2000)	(11)	$\sigma_{cmass} = 5\gamma(Q\frac{\sigma_c}{100})^{\frac{1}{3}}(MPa)$	γ is the density of rock mass (t/m3)

Table 8. Strength values (σ_{cmass})obtained from different equations.

Equation no.	(5)	(6)	(7)	(8)	(9)	(10)	(11)	Average	Std. dev.
FL	1.35	6.53	7.89	0.75	3.02	1.67	2.93	3.45	2.72
DL-LM	11.50	26.95	35.89	7.90	13.56	12.69	17.50	18.00	9.94
FZ	0.88	19.63	5.00	0.47	2.24	1.05	8.20	5.35	6.88
DL-LT	13.22	26.24	44.62	9.47	18.30	14.00	16.65	20.36	11.91
AL-MA	2.87	9.64	16.19	1.74	5.51	3.50	4.85	6.33	5.03

Table 9. The proposed empirical equations for calculation of E_{mass}

Researcher	Equation No.	Equation	Notes
Bieniawski (1978)	(12)	$E_{mass}(GPa) = 2RMR - 100$	For RMR>50
Sefarim and Pereira (1983)	(13)	$E_{mass}(GPa) = 10(\frac{RMR-10}{40})$	For RMR<50
Grimstad and Barton (1993)	(14)	$E_{mass}(GPa) = 25Log.Q$	For Q>1
Aydan et al. (1997)	(15)	$E_{mass}(MPa) = 0.0097 \times RMR^{3.54}$	
Read et al. (1999)	(16)	$E_{mass}(GPa) = 0.1\left(\frac{RMR}{10}\right)^{3}$	
Barton (2002)	(17)	$E_{mass}(GPa) = 10\left(Q\frac{\sigma_{ci}}{100}\right)^{\frac{1}{3}}$	
Hoek et al. (2002)	(18)	$E_{mass}(GPa) = \left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}}.10^{(GSI-10/40)}$	

Table 10. calculatedEmassvalues from empirical methods for different rock units.

Equation no.	(12)	(13)	(14)	(15)	(16)	(17)	(18)	Average	Std. dev.
FL	-	3.16	-	1.64	2.70	2.40	0.89	2.16	0.90
DL-LM	10.00	-	9.95	14.05	16.64	12.33	6.50	11.58	3.56
FZ	-	2.37	2.42	0.86	1.56	6.30	0.60	2.35	2.07
DL-LT	20.00	-	10.98	19.12	21.60	12.44	7.25	15.23	5.80
AL-MA	-	5.62	-	4.55	6.40	3.71	2.16	4.49	1.66

Table 11. The proposed empirical equations for calculation of m and s constants of rock mass.

Researcher	Equation No.	Equation	Notes
Hoek et al. (1995)	(19)	$\frac{m}{m_i} = 0.135(Q)^{1/3}$	
Hoek et al. (1995)	(20)	$S = 0.002Q$	
Palmstrom (2000)	(21)	$S = j_p^{2}$	
Palmstrom (2000)	(22)	$m_{mass} = m_i \times j_p^{0.64}$	J_p= jointing parameter for undisturbed rocks
Palmstrom (2000)	(23)	$m_{mass} = m_i \times j_p^{0.875}$	D= disturbance factor
Hoek et al. (2002)	(24)	$S = \exp(\frac{GSI-100}{9-3D})$	
Hoek et al. (2002)	(25)	$\frac{m}{mi} = \exp(\frac{GSI-100}{28-14D})$	
Hoek et al. (2002)	(26)	$a = \frac{1}{2} + \frac{1}{6}\left[\exp(-\frac{GSI}{15}) - \exp(-\frac{20}{3})\right]$	

Table 12. Calculated constants of rock mass from empirical methods for different engineering geological units.

Equation no.	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	Average	Std. dev.
									m	s
FL	0.605	0.001	0.001	0.603	0.259	0.028	0.169	0.531	0.409	0.010
DL-LM	2.820	0.025	0.002	1.264	0.615	0.092	0.832	0.506	1.383	0.040
FZ	1.492	0.013	0.003	0.895	0.445	0.022	0.133	0.544	0.741	0.012
DL-LT	2.911	0.028	0.003	1.449	0.741	0.104	0.937	0.505	1.510	0.045
AL-MA	1.072	0.003	0.001	0.829	0.379	0.051	0.357	0.513	0.659	0.018

description to a specific rock mass and they fail to predict interaction between the surrounding rock mass and supporting system, thus fail to give descriptions on the developments of the support and behavior of supported structures such as tunnel deformation and stress redistribution [5]. Thus in this study for the safe tunnel support design empirical, analytical and numerical methods were employed.

4.1. Empirical methods

The support measures were defined in accordance with the recommendations of Q systems, which are written below:

In relating the Q index with the stability and support requirements of underground excavations, Barton et al. (1974) have defined an additional parameter, which is called the Equivalent Dimension, D_e, of excavation. This dimension is obtained by dividing the span, diameter or wall height of excavation by a quantity called the Excavation Support Ratio, ESR. Hence:

$$D_e = \frac{Excavationspan; diameterorheight(m)}{ExcavationSupportratioESR} \quad (27)$$

The value of ESR is related to the intended use of the excavation and to the degree of security, which is demanded of the support system, installed to maintain the stability of the excavation. For the diversion tunnel, the excavation support ratio, ESR is defined as 1.6. Hence, for an excavation span of 8 m, the equivalent dimension, $D_e$is 5. The equivalent dimension, D_e, plotted against the value of Q, is used to define a number of support categories in a chart published in the original paper by Barton et al. (1974). Grimstad and Barton (1993) have recently updated this chart to reflect the increasing use of steel fiber reinforced shotcrete in underground excavation support.

For FL, D_e value of 5 and Q value of 0.06 indicates that a pattern of 1.5 m long rock bolts, spaced1.2 m and 12 cm of steel fiber reinforced shotcrete are required.

For FZ,D_e and Q values are 5 and 0.15 respectively, thus the suggested support system is composed of 1.5 mlong and 1.3 m spaced pattern rock bolts and10 cm thick steel fiber reinforced shotcrete.

4.2. Analytical method

In order to define support requirements analytically, the convergence-confinement method was utilized. Even though, this method has been known since the paper by Fenner (1938), the term convergence-confinement was developed in 196's and 70's. Convergence-confinement is a procedure that allows the load imposed on a support installed behind the face of tunnel to be estimated. The application of the method requires knowledge of the deformation characteristics of the ground and of the support.

In this paper, to apply the convergence-confinement method, a methodology, described by Carranza-Torres and Fairhurst (2000), suitable for rock masses satisfying Hoek-Brown criterion was followed.

A cylindrical tunnel of radius R subjected to uniform far-field stress, σ_0 and internal pressure pi is considered. The rock mass, in which the tunnel is excavated, is assumed to satisfy Hoek-Brown failure criterion. The critical support pressure, is defined as (Carranza- Torres and Fairhurst, 2000);

$$p_i^{cr} = \left[p_i^{cr} - \frac{s}{m_b^2} \right] . m_b . \sigma_c \quad (28)$$

where σ_c is the unconfined compressive strength, m_b and s are the rock mass parameters and is the scaled critical pressure given by the following expression.

$$p_i^{cr} = \frac{1}{16}\left[- \sqrt{1 + 16 s_o} \right] \quad (29)$$

In the equation above, S_0 is the scaled far-field stress calculated by:

$$S_0 = \frac{\sigma_0}{m_b \sigma_{ci}} + \frac{s}{m_b^2} \quad (30)$$

If the internal support pressure p_i is greater than this critical pressure p_i^{cr}, no failure will occur. In this case, the behavior of surrounding rock mass is elastic and the inward elastic displacement of tunnel wall u_r^{el} is given by;

$$u_r^{el} = \frac{\sigma_0 - p_i}{2G_{rm}}.R \tag{31}$$

Where$G_{rm}$is the shear modulus of rock mass.

If the internal support pressure pi is less than critical support pressure, p_i^{cr} failure occurs. Then the radius of broken zone, R_{pl} is defined by:

$$R_{pl} = R.\exp\left[2\sqrt{p_i^{cr}} = \sqrt{p_1}\right] \tag{32}$$

Hoek and Brown (1997) suggest that in some cases the assumption of no plastic volume-change for the rock mass may be more appropriate. For the case of non-dilating rock masses the total inward plastic deformation is calculated as follow;

$$u_\gamma^{pl} = \frac{(\sigma 0 - p_i^{cr})}{2RG_{rm}}\left[\frac{1-2\upsilon}{2}\frac{\sqrt{p_i^{cr}}}{S0 - p_i^{cr}}+1\right]\left(\frac{R_{pl}}{R}\right)+\frac{1-2\upsilon}{4(S0-p_i^{cr}}\left[1n(\frac{Rpl}{R})\right]2-\frac{1-2\upsilon}{2}\frac{\sqrt{p_i^{cr}}}{S0-p_i^{cr}} \left[21n(\frac{Rpl}{R})+1\right] \tag{33}$$

In this paper, the spreadsheet given in the paper by Carranza-Torres and Fairhurst (2000) was utilized for the calculation of above defined parameters. The necessary input parameters for the spreadsheet are tunnel radius, R, in-situ stress σ_0, support pressure and rock properties. Tunnel radius and in-situ stress, were the same for all formations and they were taken as 4m and 1.35MPa, respectively. Whereas, the necessary rock properties were changed according to the formation and they were used as input. Initially internal support pressure, P_{io} was assumed to be zero for unsupported tunnel cases in all formations. Then, support pressure was increased progressively and corresponding deformations were calculated. The ground reaction curves, demonstrating the relations between displacement and support pressure, were constructed for FL and FZ zones (Fig. 5).

Maximum deformations for unsupported tunnel in FL and FZ formations were found to be 18.13 and 40.99 mm respectively. The external radius of plastic zone for FL and FZ were 5.64 and 11.71 m respectively. Hoek and Marinos (2000) have found some relationships between strain, geotechnical issues and support types as presented in Table 14.InTable 13, strain is defined as a percentage of the ratio of tunnel closure to tunnel diameter. The strain values, the radius of plastic zone and maximum deformations for the FL and FZ formations are tabulated in Table 14. In the present study, the strain values for FL and FZ formations were calculated as 0.45% and 1.02%, respectively, as shown in Table 14. Table 13 shows that for the formations with strain values less than 1%, few stability problems are expected and the application of the support systems proposed by rock mass classifications is recommended. But for the formations with strain values less between 1 to 2.5% stability problems are expected and the application of the support systems proposed by rock mass classifications with by light steel sets or lattice girders are added for additional security is recommended.

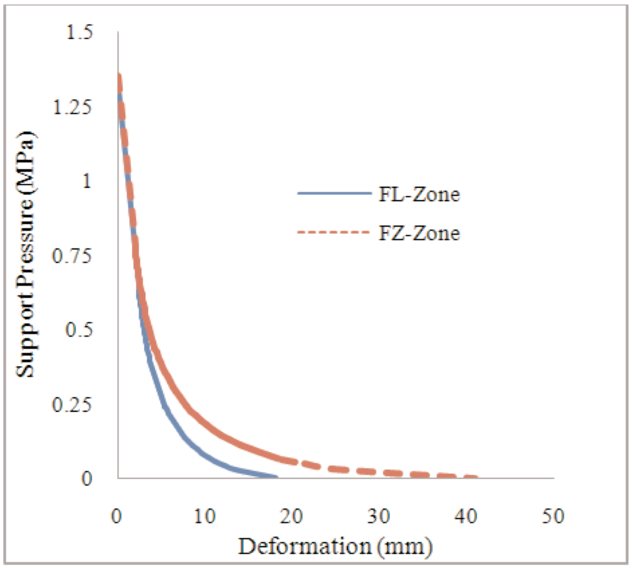

Figure 5. Relationship between support and displacement around a tunnel opening excavated in FL and FZ formations.

4.3. Numerical method

The computer software Phase2, a Finite Element Program developed by Rocscience (1998), was used for calculating stresses, deformations and developed plastic zone around tunnel. Software permits two-dimensional study of the non-linear deformations of rocks using Hoek-Brown failure criterion. The input parameters are uniaxial compressive strength, Young's modulus, Poisson's ratio and m and s Hoek-Brown constants. In this program, automatic mesh around the tunnel is generated and based on the elas-to-plastic analysis, deformations and stresses are computed. Several iterations of the program with

Table 13. Relationships between strain, geotechnical issues and support types (Hoek and Marinos, 2000)

Strain ε%		Geotechnical issues	Support types
A	Less than 1	Few stability problems and very simple tunnel support design methods can be used. Tunnel support recommendations based upon rock mass classifications provide an adequate basis for design.	Very simple tunneling conditions, with rockbolts and shotcrete typically used for support
B	1 to 2.5	Convergence confinement methods are used to predict the formation of a 'plastic' zone in the rock mass surrounding a tunnel and of the interaction between the progressive development of this zone and different types of support.	Minor squeezing problems which are generally dealt with by rockbolts and shotcrete; sometimes light steel sets or lattice girders are added for additional security.
C	2.5 to 5	Two-dimensional finite element analysis, incorporating support elements and excavation sequence, are normally used for this type of problem. Face stability is generally not a major problem.	Severe squeezing problems requiring rapid installation of support and careful control of construction quality. Heavy steel sets embedded in shotcrete are generally required.
D	5 to 10	The design of the tunnel is dominated by face stability issues and, while two-dimensional finite analyses are generally carried out, some estimates of the effects of forepoling and face reinforcement are required.	Very severe squeezing and face stability problems. Forepoling and face reinforcementwith steel sets embedded inshotcrete are usually necessary.
E	More than 10	Severe face instability as well as squeezing of the tunnel make this an extremely difficult three-dimensional problem for which no effective design methods are currently available. Most solutions are based on experience.	Extreme squeezing problems. Forepoling, face reinforcement are usually applied and yielding support may be required in extreme cases.

Table 14. Critical support pressure, radius of plastic zone, maximum deformations and strain values obtained from the convergence-confinement method

Formation	Critical support pressure, P_{cr}, MPa	Radius of plastic zone, R_{pl}, m	Maximum deformation, u_{rmax}, mm	Strain, %
FL	0.72	5.64	18.13	0.45
FZ	0.84	11.71	40.99	1.02

more appropriate support parameters lead to a final reasonable estimate of tunnel convergence.

The strength and the yield zone of rock mass were estimated by the Hoek-Brown failure criterion. It is assumed that material behaves as elastic-perfectly plastic. The necessary strength properties of rock mass were calculated in FL and FZ formations.

It is more difficult to estimate undisturbed horizontal stress, σ_h. It is known that they are variable at shallow depth, tending to a hydrostatic state in deep environment (Hoek and Brown, 1978). In this research, for want of any better information σ_h are assumed to be equal to σ_v as suggested by Hoek (2003). In case of large topographic relief or where large tectonic forces have been active, some modification to the lateral stresses assumption may be required. Thus, stresses in all directions are equal and calculated as;

$$\sigma v = \gamma H \tag{34}$$

Where, γ is the unit weight of rock mass in MN/m^3 and H is the tunnel depth in m. The lithostatic pressure was calculated to be 1.35MPa for studied tunnel. In order to describe the underground structure, ground section was divided into more than 2300 triangular finite elements. The analysis includes two models; the first model was used to examine the conditions excavation without any support and the second model consist of support application to the excavation boundary. Phase^2 model applies support immediately after the excavation, however in real cases some deformation is allowed to occur and installation time of support system takes time, in this time rock mass around tunnel has already shown a certain reduction of stress state. To simulate delayed support installation load splitting phenomenon was used in the second model.

During the analysis, the thickness of plastic zone, tunnel deformation and stresses for unsupported and supported cases were computed. For unsupported cases stresses, deformations and yielded elements around the tunnel, excavated in the FL and FZformations, are shown in Fig. 6.

For the tunnel excavated in FL, some yielded elements are observed and the radius of plastic zone is around 7.76 m. The number of yielded elements increased and the thickness of plastic zone enlarged to 13.12 m for the tunnel driven in FZ formation. Used support elements were rock bolts and shotcrete as proposed by the empirical method. The properties of support elements, such as length, pattern of bolts and the thickness of shotcrete were similar to those proposed by Q system. Thus, for the tunnel section excavated in the FL, the recommended support system was composed of 1.5 m long rock bolts with 1.2 m spacing and 120 mm thick shotcrete. In order to obtain tunnel stability for FZ, 1.5 m long rock bolts with 1.3 m spacing and100 mm thick shotcrete were used as support elements.After support installation, not only the number of yielded elements but also the extent of plastic zone decreased as shown in Fig. 7. This indicates that the applied support systems are adequate to obtain tunnel stability.

The radius of plastic zone and maximum total displacements obtained from Phase^2 forunsupported

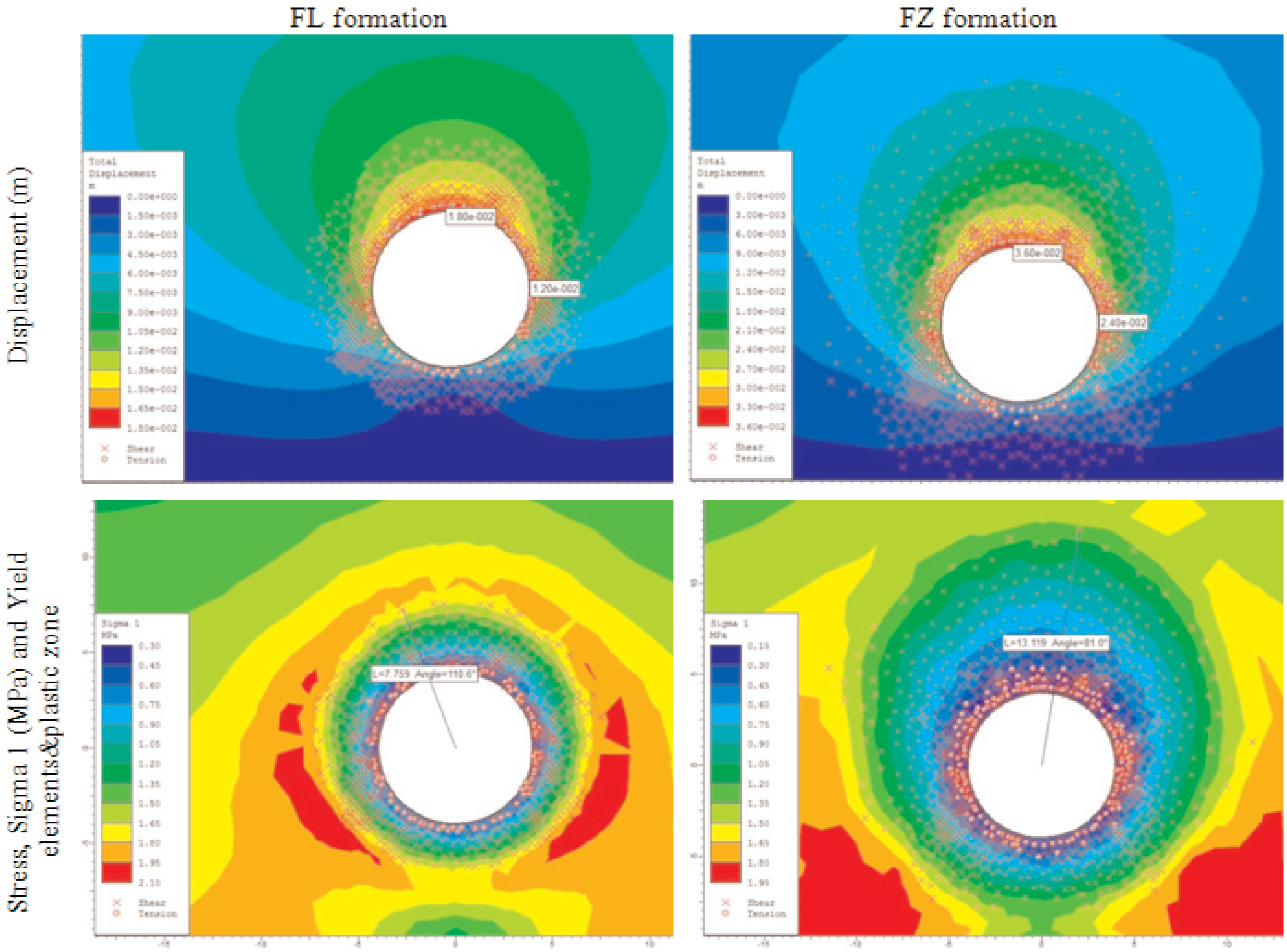

Figure 6. Stresses, displacements and yielded elements for unsupported tunnel in FL and FZ formations.

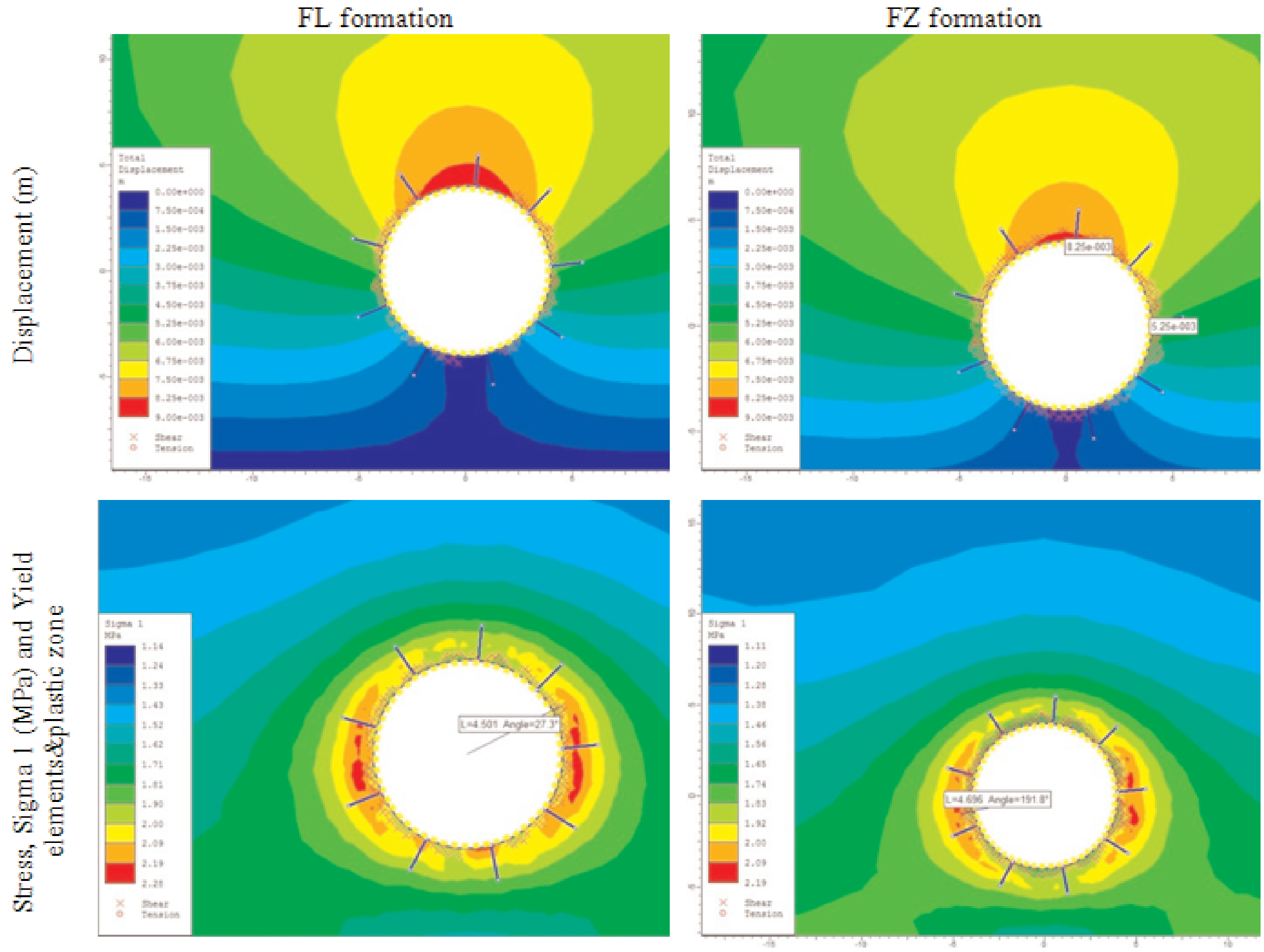

Figure 7. Stresses, displacements and yielded elements for supported tunnel in FL and FZ formations

Table 15. Radius of plastic zone, maximum total deformations obtained from phase2

Formation	Radius of plastic zone, Rpl, m		Maximum deformation, urmax, mm	
	Unsupported	Supported	Unsupported	Supported
FL	0.72	5.64	18.13	0.45
FZ	0.84	11.71	40.99	1.02

and supported cases are presented in Table 15.

As it is seen from Tables 14 and 15, the results obtained from convergence-confinement and numerical modellingare similar. It is thought that, the small differences are caused by a finite discretization in numerical model.The flow chart given in Fig. 8 shows the followed tunnel support design procedure in this research.

5. Conclusions

Knowledge of ground conditions is a key factor in adopting an excavation method and designing a support system for underground openings. A comprehensive engineering geological assessment of rock masses has been carried out at the RudbarLorestandam site in center of Iran. The geotechnical properties of these rocks have been carefully assessed based on laboratory and ?eld investigations for assessing stability problem along the tunnel. In mediums where the rock masses are tectonically disturbed, the rock masses are already under stress before an underground opening is ever excavated. Therefore, prediction of stability problems along the tunnel route has been done by empirical, analytical and numerical

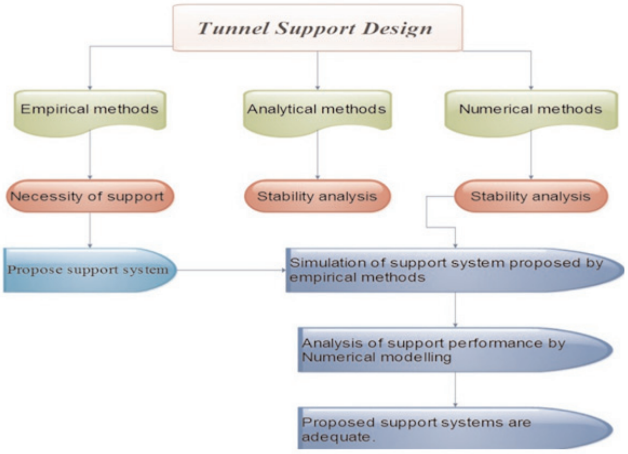

Figure 8. The followed tunnel support design procedure.

methods.According to the results acquired from the empirical, analytical and numerical modeling, there were some stability problems both for FL and FZ formations. The empirical method recommends the utilization of bolt and shotcrete as support elements for FL and FZ. Analytical and numerical methods showed that some deformations occurred and plastic zone developed around the unsupported tunnel. Numerical modeling was utilized to evaluate the performance of recommended support system. When the recommended support systems were applied, number of yielded elements and displacements were reduced significantly in numerical analysis. Empirical method indicated that substantial support was necessary for weak formations and both analytical and numerical methods agreed that the size of the plastic zone and deformations were increased. However, after installation of the support elements recommended by Q system, numerical method showed that there was a sharp decrease in both number of yielded element and the size of plastic zone around the tunnel. The results proved that the empirical, analytical and numerical methods are agreed with each other. However, the measurements carried out during construction can be used to check the validity of the proposed support system or to adapt the design of support system.

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