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Numerical Evaluation of Empirically Suggested Support System for the Diversion Tunnels at Diamer Basha Dam Project, Pakistan

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Abstract

During the design of the tunnel in rocks, inadequate geological and geotechnical data of the construction site is a way of life. Due to this insufficiency of data, the empirical systems for rock–mass are still required for the underground excavation design. The characterization of rock mass followed by classification is an essential part of the initial support design using RMR and Q systems. In this research, these systems are used for the classification of rock mass along the diversion tunnel of a hydropower project in Pakistan. The GSI system is used for the calculation of well-known failure criteria constants. Rock mass is characterized and classified into three geological units along the tunnel route, based on seven number of bore holes, drilled in this route. These rock mass quality values are used for the deformation modulus calculation. The in situ stresses are also calculated statistically, using the available empirical equations. The empirical support determined from the two systems is evaluated numerically, using FLAC2D. The results of numerical modelling indicate that the support suggested by empirical approaches are appropriate. The results are shown in term of critical strain, thrust–bending moment interaction diagram, and axial stresses in rock bolts. Although the highest critical strain value is 0.00058 for geotechnical unit 2, however, this deformation is within the control range.

Keywords: Characterization, classification, support, diversion tunnel, numerical modelling.

1. Introduction

In tunnel design, stability analysis is the primary concern. For this purpose, different approaches such as; empirical classification, numerical modelling, analytical and observational methods are used. In hydropower projects, diversion tunnel being the first underground structure helps in estimating optimum support measures for the rest of underground structures in the vicinity. Empirical approaches like Rock Mass Rating (RMR) system (Bieniawski, 1989; Celada et al., 2014), Tunnel Quality Index (Q) system (Barton, 2002; Barton et al., 1974) and Geological Strength Index (GSI) system (Marinos and Hoek, 2000) are used worldwide in the field of tunneling. However, these systems are impotent to provide the information about the stress redistribution, support performance and deformation around the tunnel under different circumstances. To observe these features, numerical modelling techniques are used.

Based on rock mass characterization, the geological and geotechnical condition changes significantly from project to project and even within a single project, but experience in the nearby project always has a significant contribution in tunneling. Himalaya is the toughest mountain range in tunneling prospective (Carter, 2011; Lee et al., 2018; Naji et al., 2018; Naji et al., 2019b; Naji et al., 2019c). Due to the rock mass uncertainties, tunneling in Himalaya is more difficult particularly in those areas where experience is zero, which requires the empirical evaluation method by some mean for the optimize tunnel design. The best way to design underground structure is to apply empirical methods and their evaluation through numerical modelling (Basarir et al., 2005; Sari and Pasamehmetoglu, 2004). Numerical assessment of designed tunnel is the best approach in the preliminary design stage, and their interactive practice assures thorough engineering judgment.

The diversion tunnels selected for this study are part of Diamer Bahsa dam (DBC), Pakistan. The dam is planned to have an installed power capacity of 4500MW. The project site is around 315 km (upstream) from Tarbela dam and 180 km below Gilgit city at the boundary line of Gilgit–Baltistan and Khyber–Pakhtunkhwa, Pakistan (Fig. 1). The length of the proposed 15.4 m diameter "D" shape diversion tunnels DT–1 and DT–2 are 887 m and 1016 m, respectively. The bearing of the DT–1 and DT–2 is not parallel to each other as they diverge towards downstream.

In this research, RMR, Q, and GSI systems have been used to categorize the rock mass along the diversion tunnels. The first two systems have been employed to assess the rock mass class and support requirement of these tunnels, considering the latest changes made in these systems. Characterization of rock mass has been carried out from geotechnical exploration data of seven boreholes, drilled along the alignment of the planned tunnels. Field exploration and laboratory test data are used, and rock mass properties are determined from empirical approaches. The input data for numerical modelling is obtained through statistical analysis. Through numerical analysis, tunnel deformation in term of critical strain, shotcrete and rock bolts stability is evaluated.

2. Geotechnical studies

The DBD project is situated within Kohistan Arc which was developed as a consequence of the subduction of Indo–Pakistan Plate below the Eurasian Plate . The prevailing rock within the project periphery is mafic intrusive Gabbronorite (GN). Amphiboles, pyroxene and plagioclases are the dominant minerals present in GN. In addition to GN, ultramapic rock is also existing in the reservoir area, known as UMA (ultramafic association).

Fig. 1. Location of the Project (without scale).

To find the rock mass engineering features of DBD area, a detailed geological study that comprises; laboratory testing, core drilling, discontinuity survey, etc., is accomplished. Out of the 62 boreholes, the exploration data of boreholes No. BDR–26, 25, 24, 22, 21, 10, and 08 is evaluated in this study to inspect the in-situ environment along the alignment of DT–1 and Dt–2.

The rock mass is separated into three geotechnical units (GU) along the orientation of DT–1 and DT–2 i.e., GU–1, GU–2 and GU–3 based on geological data from drill boreholes and outcrops. The analysis is limited to DT–1 due to the similar size, shape, and the small distance between the diversion tunnels.

According to borehole BDR–24 and BDR–25, in GU–1, the tunnel alignment up to chainage 0 + 480 from inlet is massive GN. Due to shear zone and local faults, the tunnel alignment from chainage 0 + 480 till 0 + 633 is in fractured and closely jointed GN, according to borehole BDR–8 and BDR–10.

In GU–2, the DT–1 alignment touches UMA at chainage 0 + 633. In this unit, the rock mass is more jointed comparatively in BDR–21 than BDR–22. In BDR–22, the rock mass is more jointed and indicates low rock quality designation (RQD) at the tunnel alignment level. The GU–3 starts from chainage 0 + 800 and the tunnel alignment in this unit is frequently passing through GN rock. The borehole number BDR–26 represent this section and BDR–21 shows the boundary of UMA and GN. The BDR–21 is drilled along DT–2 alignment and displays the contact between GN and UMA at a depth of 75 m.

The invert elevation at inlet and outlet of DT–1 is 960 m and 958 m, and that of DT–2 is 951 m and 948 m with 0.225 % and 0.295 % gradient respectively. The tunnel depth along DT–1 varies from 75 m to 175 m in GU–1. This variation for GU–2 is from 45 m to 115 m, and the average depth of the crown for GU–3 is 35 m.

Along with exploration, thorough laboratory testing is accomplished and their details are summarized in Table 1 (Munir, 2013).

In this study, the intact rock's mechanical and physical properties are selected from the tests conducted on rock sample from the chosen boreholes and are summarized in Table 2.

Table 1. Intact rock properties in project area.

Rock Type	PLSI (MPa)		UCS (MPa)		Modulus of Elasticity (GPa)		Poison's Ratio (v)		Specific Gravity	
	Min–max	mean	Min–max	mean	Min–max	mean	Min–max	mean	Min–max	mean
GN	1.71–14	5.2	29–203	100	3.7–250	60	0.017–.952	0.25	2.84–3.45	2.94
UMA	1.29–12.6	4.8	15–138	80	13.2–340	100	.022–0.887	0.26	2.82–3.54	3.29

Table 2. Intact rock properties for the three geological units along the tunnel route.

Parameter	Geotechnical Unit–3			Geotechnical Unit–2			Geotechnical Unit–1		
	Min. value	Max. value	Mean	Min. value	Max. value	Mean	Min. value	Max. value	Mean
ɣ (KN/m^3)	28.0	29.5	28.9	29.6	34.6	32.1	28.0	29.5	28.9
UCS (MPa)	50	210	110	33	101	80	46	203	110
E (GPa)	18.7	78.7	43.1	12.3	37.8	23.6	17.2	76.1	41.2
v	0.07	0.39	0.199	0.01	0.354	0.152	0.02	0.38	0.195

3. Rock mass classification systems

3.1. RMR system

From tunneling database, Bieniawski (1973) formulated the RMR system and has undergone extensive changes (Rehman et al., 2018a; Rehman et al., 2019). In the present research, 1989 version called RMR_{89} is used. To rate the intact rock strength, joint spacing and RQD, the continuous rating approach (Rehman et al., 2018b) is used for the calculation of RMR_{89}.

The detailed characterization and rock mass classes are presented in Table 3. The details of rating for joint spacing, RQD, joint roughness and joint orientation are shown in Figure 2. Statistical analysis indicates that RMR changes from 42.3 to 77.1 with an average of 63 for GU–1. These values are 44.3 to 71 with an average of 60 and 55.8 to 73.8 with an average of 64 for geotechnical unit 2 and 3. The geotechnical unit–1 and 3 are of good quality as compared to geotechnical unit–2, which is the fair rock mass. Figure 3 shows detailed distribution of rock mass quality for these three units.

3.2. Q–system

This system was established for the categorization of the rock mass environment for underground excavations (Barton et al., 1974). The system is specified in terms of RQD, joint alteration (J_a), Joint roughness (J_r), the joint set number (J_n), joint water reduction (J_w), and stress reduction factor (SRF) and Q–value can be computed using Equation 1.

$$Q_c = \left(\frac{RQD}{J_n}\right)\cdot\left(\frac{J_r}{J_a}\right)\cdot\left(\frac{J_w}{SRF}\right)\cdot\left(\frac{\sigma_c}{100}\right) \qquad (1)$$

Equation 1 was modified by introducing intact rock strength as a normalized factor, as shown in Equation 2 (Barton, 2002) .

$$Q = \left(\frac{RQD}{J_n}\right)\cdot\left(\frac{J_r}{J_a}\right)\left(\frac{J_w}{SRF}\right) \qquad (2)$$

The distribution of J_n for the seven boreholes is shown in Figure 4. This distribution reveals that one joint set is dominant in BDR-25 and three sets are in BDR–10. All the boreholes show dry rock mass condition. Shear zones are observed during the exploration. As the Q–system has no provision for multiple shear zones for tunnel deeper than 50 meters, however, the style of the table for the SRF characterization for weakness zone in Q-system indicates that SRF=5 will be appropriate in similar conditions (Barton et al., 1974). Therefore, SRF values of 5, 2.5, 1.25, and 1 are selected according to a particular situation.

A summary of the ratings of Q-system parameters is presented in Table 4. These values are in the range of 0.33 to 82 with an average value of 21.4 for geotechnical unit-1. For geotechnical unit-2 and 3, the Q value ranges from 0.22 to 35 and 3.67 to 78.4 with an average of 8.8 and 17.5, respectively. The rock mass quality along geotechnical unit–1 and 3 is good and geotechnical unit–2 rock mass is of fair quality.

3.3. GSI, Mohr–Coulomb parameters and Hoek–Brown constants for rock–mass

Hoek et al. (2000) formulated GSI in 1995 considering the structure of rock mass and its appearance which was modified later for heterogeneous weak rock masses (Marinos and Hoek, 2000) (Marinos and Hoek, 2001). Further, they applied GSI chart for rock mass properties determination. The m_i value for GN and UMA for this project is 23 and 25, respectively (Munir, 2013).

To calculate rock mass constants, Roclab software is used and the results are summarized in Table 5.

3.4. Recommended support system

Rock bolt spacing only depends on RMR value (Equation 3) and their length is dependent on underground excavation size and rock quality value (Equation 4) (Bieniawski, 1989) (Lowson and Bieniawski, 2013). This length is dependent on ESR and tunnel span (Equation 5) in Q–system (Barton, 2002) (NGI, 2015). Comparing the length proposed by Equation 5 with the actual length of installed rock bolts in hydropower tunnels in Pakistan indicates ESR=1 for this project (Rehman et al., 2017).

The shotcrete thickness is also dependent on excavation size and rock quality (NGI, 2015) (Grimstad and Barton, 1993; Kim et al., 2019; Rehman et al., 2019). Although empirical classification approaches are the tools for support design, but engineering judgment should also be applied as a check or verification tool (Palmstrom and Stille, 2007).

$$S_b(m) = 2.50 \times \left(\frac{RMR - 20.0}{65.0} \right) + 0.50 \quad (3)$$

$$(85 \geq RMR > 20)$$

$$Span = \frac{[L + 2.50]^{\left(\frac{25.0 + RMR}{52} \right)}}{3.6} \quad (4)$$

$$L = 2 + \frac{(0.15 \times B)}{ESR} \quad (5)$$

The stand–up time concept of RMR classification and maximum unsupported excavation span of Q–system (Equation 6) reveal top heading and bench sequence for the tunnel construction in this project (Barton et al., 1980; Bieniawski, 1989).

$$Max.Span = 2.0 \times ESR \times Q^{0.40} \quad (6)$$

Table 6 summarizes the recommended support system for DT–1 and DT–2 based on the two systems.

Table 3. RMR_{89} rating for the three geological units along the tunnel route.

Parameter	**Rating**						
	GU3	**GU–2**		**GU–1**			
	DBR–26	**DBR–22**	**DBR–21**	**DBR–08**	**DBR–10**	**DBR–24**	**DBR–25**
UCS	9	7.33	7.33 and 9	7.33 and 9	9		
RQD	2–19.8	0–17.8	0–18.4	0–19.4	0.8–18.4	9.2–20	4.8–20
Joint Spacing	7.07–20	6–16.75	2–17.28	4–18.8	6–20	6–18.8	6–20
Persistence	2						
Aperture	4						
Roughness	3–6	2–6	2–6	2–5	1–5	3–6	3–6
Filling	4						
Weathering	6	5–6	6				
Ground water	15						
Joint Orientation	–10–0		–12–0				
RMR (min–max)	55.8–73.8	44.3–69.93	45–71	42.3–70.6	48.8–70.6	64–75	56.97–77.1
Avg. RMR	64	60.17	59.9	59.73	59.6	70.5	69
Average RMR	Good Rock (64)	Fair 60		Good Rock 42.3–77.1 (63)			
Shear Strength Parameters	c=0.3–0.4 MPa, ϕ =35–45°	c=0.2–0.3 MPa, ϕ =25–35°		c=0.3–0.4 MPa, ϕ =35–45°			

Table 4. Q system rating for the three geological units along the tunnel route.

Parameter	**Rating**						
	GU–3	**GU–2**		**GU–1**			
	DBR						
	26	**22**	**21**	**08**	**10**	**24**	**25**
RQD	10–100	10–90	10–90	10–95	10–90	45–100	25–100
J_n	2,4 and 9	4 and 9	2, 4 and 9				
J_r	2 and 3		3	2 and 3	1,2 and 3	2 and 3	
J_a	2						
J_w	1						
SRF	1	1and5		1, 2.5, 5		1	1,1.25,2.5
Q (min–max)	3.67–78.4	0.22–34	0.75–35	0.33–25	0.4–25	10–82	4–82
Avg. Q	17.5	9	8.66	5.3	5.4	39	36
Rock Mass Quality (Avg. Q)	Good (17.5)	Fair (8.8)		Good (21.4)			
Avg. Q_c	19.25	7.04		23.54			

Table 5. Mohr–Coulomb parameters and Hoek–Broun constants along with GSI value.

GSI, H–B and M–C Constant	GU3	GU–2		GU–1			
	DBR–26	DBR–22	DBR–21	DBR–08	DBR–10	DBR–24	DBR–25
GSI	70–80 (73.4)	50–70 (57.1)	45–75 (59)	50–80 (63.5)	45–75 (60.7)	65–85 (74.3)	65–85 (74.3)
m_i	23	25	23 and 25	23 and 25	23	23	23
m_b	8.895	5.402	5.55	6.517	5.652	9.186	9.186
s	0.052	0.0085	0.0105	0.0173	0.0127	0.0575	0.0575
a	0.501	0.503	0.503	0.502	0.503	0.501	0.501
c (MPa)	2.446	1.102	1.168	1.89	1.715	3.012	3.012
ϕ(Degree)	66.65	60.54	60.61	61.27	60.49	62.38	62.38
Average Values	GSI=73.4, mi=23, mb=8.895, s=.052, a=0.501, c=2.446, ϕ=66.65	GSI=58.05, mi=25, mb=5.588, s=0.0095, a=0.503, c=1.134 ϕ=60.72		GSI=68.2, mi=23, mb=7.387, s=0.0292, a=0.502, c=2.82, ϕ=61.67			

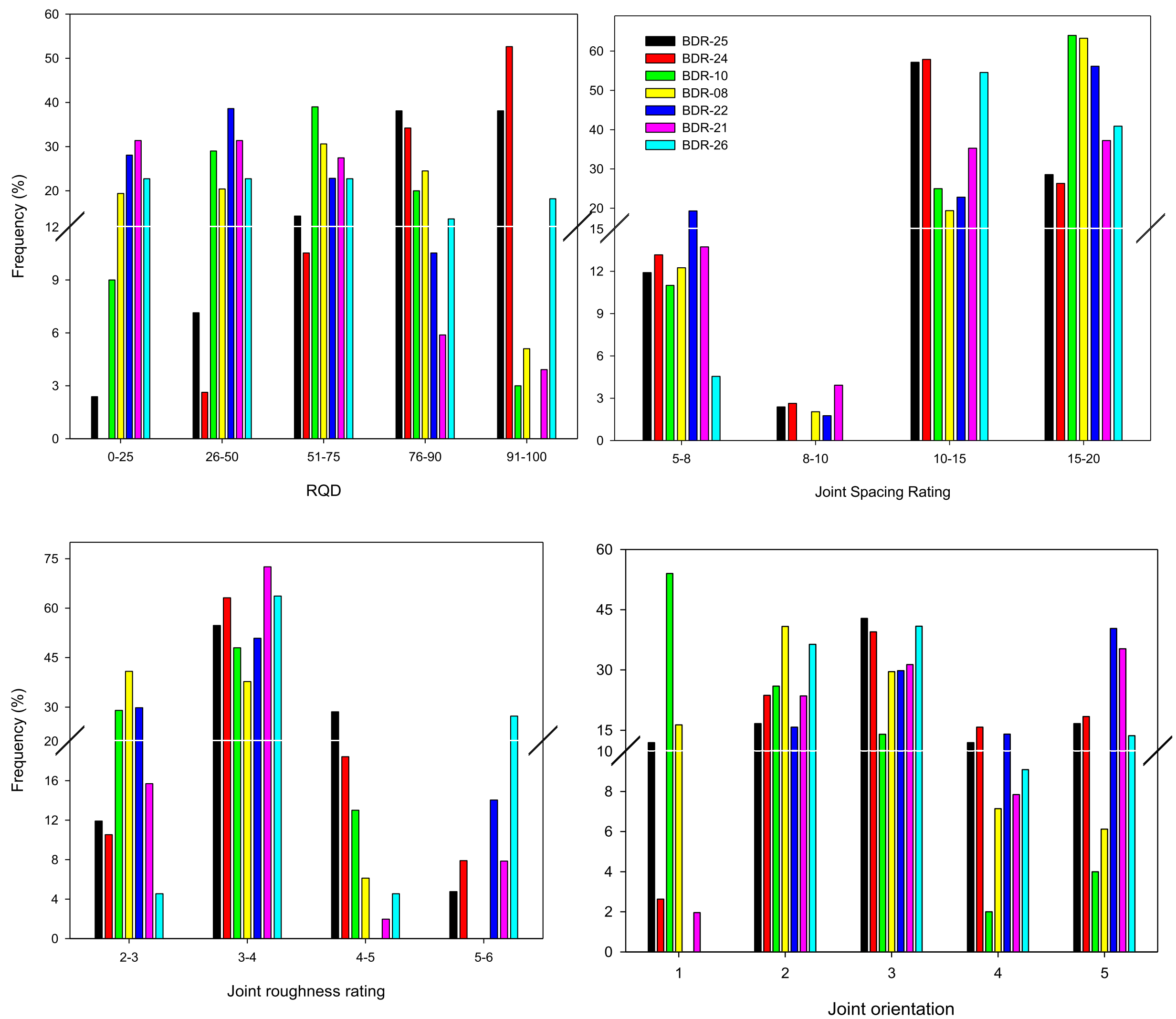


Fig. 2. Percentage frequency of RQD rating, joint spacing rating, joint roughness rating and joint orientation rating with respect to tunnel orientation

Table 6. Empirical support design for the tunnel in different geological units.

GU	3		2		1	
Classification System	RMR	Q_c	RMR	Q_c	RMR	Q_c
Rating	64.0	19.25	60.0	7.04	63.0	23.54
Rock Bolt Length	06 m	4.3 m	6 m	4.3 m	6 m	4.3 m
Rock Bolt Spacing	2.2 m	2.5 m	2.04 m	2.2 m	2.2 m	2.5 m
Shotcrete Thickness	10 cm	5 to 6 cm	12 cm	6 cm	10 cm	5 to 6 cm
Recommended Support	L (m)=4–6 S_b (m)= 2.2 in crown S_b (m)= 2.5 m in sidewall 8cm thick shotcrete		L=4–6m S_b=2.0m in crown S_b= 2.5 m in sidewall 9cm thick shotcrete		L=4–6m S_b=2.15m in crown S_b=2.5 m in sidewall 8 cm thick shotcrete	

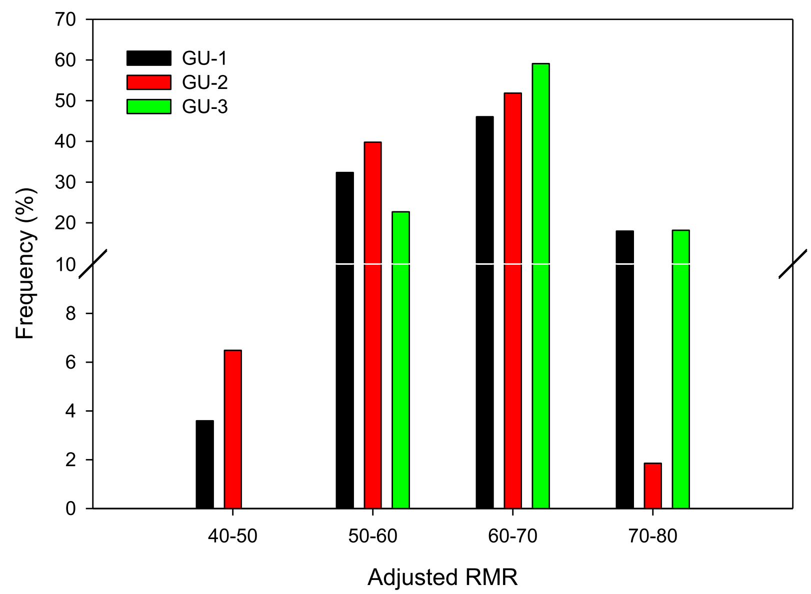
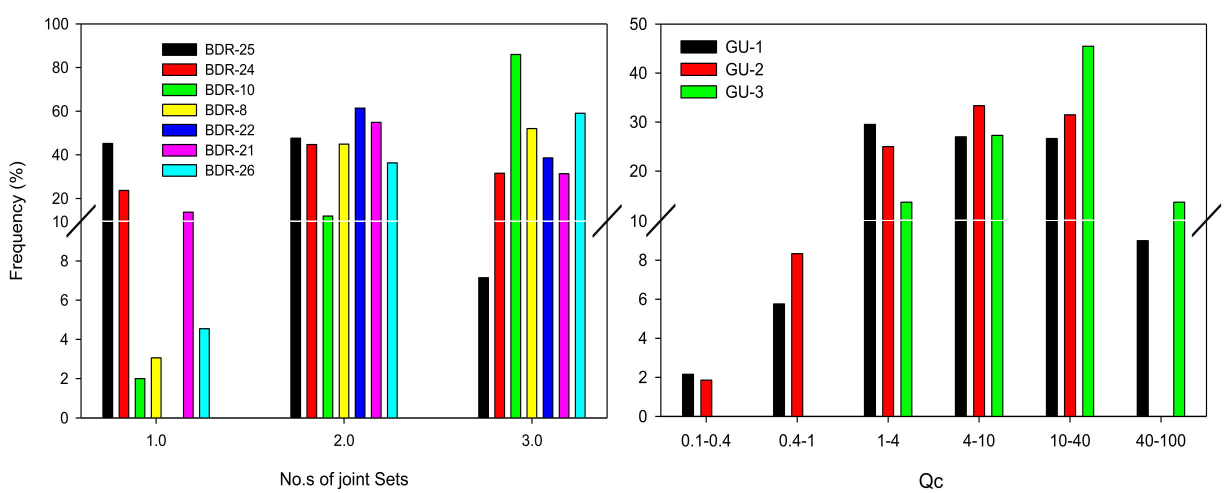


Fig. 3. Frequency distribution of RMR_{89} in three geological units.



4. Rock mass deformation modulus (E_{rm})

Large scale in–situ test, back analysis and empirical approaches are used for calculating the E_{rm}. In–situ test for calculating E_{rm} are costly and difficult (Hoek and Diederichs, 2006). Back analysis cannot be conducted before construction. Water and Power Development Authority (WAPDA) performed a plate load test on the left abutment in Adit–4 for the same project to calculate the E_{rm}. A correlation of deformation modulus was developed with RMR and Q value (Munir, 2013) at test location which is given in Equation 7 and 8 respectively;

$$E_{rm} = 1.35 \times e^{0.047RMR} \quad (R^2 = 0.82) \qquad (7)$$

$$E_{rm} = 10.22 \times Q^{0.324} \quad (R^2 = 0.736) \qquad (8)$$

Different empirical equations, as presented in Table 7, were used for the determination of deformation modulus. These deformation modulus values (mean), obtained from Table 7, are matched with the resultant values obtained from Equation 7 and 8, and hence the empirical calculated values are reliable for the current study.

5. Virgin stresses

The role of virgin (in–situ) stresses is significant for the design and construction of tunnel. Knowledge of these stresses is mandatory to design the underground structure. There are numerous approaches presented for the calculation of these stresses — and always needed to assess it in the best possible way. In-situ stress measurement is a costly venture; therefore; alternative approaches such as experience from nearby underground structures or empirical approaches are used for its estimation. Numerous empirical relationships have been recommended by the research community for the calculation of in–situ stresses. In this section, some of the commonly used equations are discussed.

The believed perception about the vertical stress (σ_v) is shown in Equation 16 which reveal that σ_v increases with depth (H).

$$\sigma_v = \gamma \times H \qquad (16)$$

Where ɣ is the rock unit weight.

The horizontal stress (σ_h) calculation in the in-situ environment is considerably more challenging than σ_v. The ratio of σ_h and σ_v is denoted by K. Sheorey (1994) suggested the influence of tectonic forces for in-situ stresses which was noted by Hoek (2007) for calculating K.

$$K = 7 \times E_h \left(0.001 + \frac{1}{z}\right) + 0.25 \quad (17)$$

where z (m) and Eh (GPa) are the overburden and the average deformation modulus, respectively.

The following relation (Equation 18) between σ_h and σ_v was developed based on hydraulic fracturing tests data results (Stephansson, 1993).

$$\sigma_h = 2.8 + 1.48 \times \sigma_v \qquad (H < 1000m) \quad (18)$$

Equation 19 was developed for variation in horizontal stress with depth in Himalayan region (depths below 400m).

$$\sigma_h = 1.5 + 1.2 \times \sigma_v . \qquad (19)$$

Sheorey and his co-researchers suggested Equation 20 for computing σ_h (Sheorey et al., 2001),

$$\sigma_h = \frac{\nu}{1-\nu}\sigma_v + \frac{\beta . E_{rm} . G}{1-\nu}(H + 1000), \quad (20)$$

where the geothermal gradient is G=0.024 0C/m, the linear thermal expansion coefficient is β=8*10–6 /0C, and Poisson's ratio is ν.

The average values of σ_v, σ_h and K for GU–1 are 3.6125, 5.38 and 1.49, for GU–2 are 2.568, 4.01 and 1.56 and for GU–3 1.0115, 4.15 and 3.28 respectively, as shown in Table 8.

6. Numerical modelling

To evaluate the performance of the empirical support, an explicit finite difference formulation software, called FLAC2D Version 7.0, is used in this study which is suitable for the sequential excavation. The top heading and bench stages for excavation are simulated using three construction steps. To exclude the boundary influence, tunnel periphery was modelled at enough distance from the sides. The Modified Hoek-Brown Model was used for the analysis. Around the tunnel periphery, the fine mesh was simulated. The model was fixed at sides and bottom, and the in-situ stress environment was created using gravity and FISH function. The input rock mass properties such as physical and mechanical properties are used from the estimated values presented in the previous sections. The empirically calculated average in-situ stresses values (Table 8) are used. The Q and RMR systems guidelines were adopted for excavation and support.

To analyze tunnel stability and empirical support performance in geotechnical units, three numerical models were generated as shown in Figure 5, showing tunnel geometry and mesh.

After the application of support, 2.72 mm, 4.45mm and 2.96 mm are the maximum total displacements for GU–1, 2 and 3 respectively. To evaluate the tunnel stability, critical strain concept (Park and Park, 2014) is used. The resultant values are 0.000353, 0.00058 and 0.00038 Which shows that the deformation is within the control range.

To evaluate the performance of sprayed shotcrete, thrust–bending moment interaction diagram, which is the graphical illustration of the liner/shotcrete failure, is used for the sprayed shotcrete in the tunnel roof (Carranza-Torres and Diederichs, 2009). The factor of safety (FOS) used for the evaluation of shotcrete are 1.5, 2.0, and 2.5 as shown in Figure 6. The resulting values obtained are plotted in thrust bending moment interaction diagram. Figure 6 (a) and (b) is for shotcrete with a thickness 8cm and 9cm respectively based on Table 6 results. The results show that the shotcrete is stable in compression and show a low factor of safety in tensile failure. This low factor of safety value is the interaction point of the tunnel wall and roof.

The maximum axial stresses in rock bolt are depicted in Table 9 at the end of top heading and bench modelling. Due to the topography, as demonstrated in Figure 5, the maximum axial stresses are in the right half of the tunnel roof in all three geotechnical units. These values are 0.0092 MPa, 0.0406 MPa, and .0263 MPa for GU–1, 2 and 3, respectively.

Table 7. Estimation of rock mass deformation modulus along the tunnel route using empirical equations.

Eq. No.	Equation	Reference	E_{rm} (GPa)		
			GU–3	GU–2	GU–1
9	$E_{rm} = 2RMR - 100 \quad (RMR > 50)$	(Bieniawski, 1978)	28	20	26
10	$E_{rm} = 0.1 \times \left(\frac{RMR}{10}\right)^3$	(Read et al., 1999)	26.2	21.6	25
11	$E_{rm} = 10 \times Q_c^{1/3}$	(Barton, 2002)	26.8	19.16	28.66
12	$E_{rm} = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}}.10^{\frac{GSI-10}{40}}$ for $(\sigma_{ci} \le 100)$ $E_{rm} = \left(1 - \frac{D}{2}\right).10^{\frac{GSI-10}{40}}$ for $(\sigma_{ci} \ge 100)$	(Hoek et al., 2002)	38.46	14.22	28.5
13	$E_{rm} = E_i.\exp(-0.0035(5(100 - RMR)))$	(Ramamurthy, 2004)	22.96	11.72	21.56
14	$E_{rm} = E_i.\exp(-0.0035(250(1 - 0.3\log Q)))$	(Ramamurthy, 2004)	24.9	12.61	24.35
15	$E_{rm} = E_i\left(0.02 + \frac{1}{1 + e^{(60+15D-GSI)/11}}\right)$	(Hoek and Diederichs, 2006)	34.12	11.23	28.76
Mean (± Standard Deviation)			28.78 (5.51)	15.79 (4.33)	26.12 (2.72)

Table 8. Estimation of in situ stresses using empirical equations.

Equation No.	$\sigma_h = k.\sigma_v$								
	GU–3			**GU–2**			**GU–1**		
	σ_v	σ_h	k	σ_v	σ_h	k	σ_v	σ_h	k
17		6.03	5.96		4.40	1.72		6.68	1.85
18		4.30	4.25	(1.44–3.69) 2.57	6.6	2.57	(2.17–5.06) 3.61	8.147	2.26
19	1.01	2.71	2.68		4.58	1.78		5.84	1.62
20		0.26	0.26		0.46	0.18		0.88	0.24
Average		**3.32**	**3.28**	**2.568**	**4.01**	**1.56**	**3.6125**	**5.38**	**1.49**

Table 9. Axial stresses in rock bolts at the end of different sequence of excavation (all units are in Pa).

Rock bolts location	GU–3		GU–2		GU–1	
	Top heading excavation	Bench excavation	Top heading excavation	Bench excavation	Top heading excavation	Bench excavation
Right wall	–	1.913	–	3.256E4	–	4.069E3
Roof (right half)	1.430E4	2.630E4	9.833E03	4.058E4	9.042E3	9.243E3
Roof (left half)	8.749E3	1.035E4	8.250E3	1.899E4	8.965E3	9.154E3
Left wall	–	2.140E4	–	2.846E4	–	4.774E3

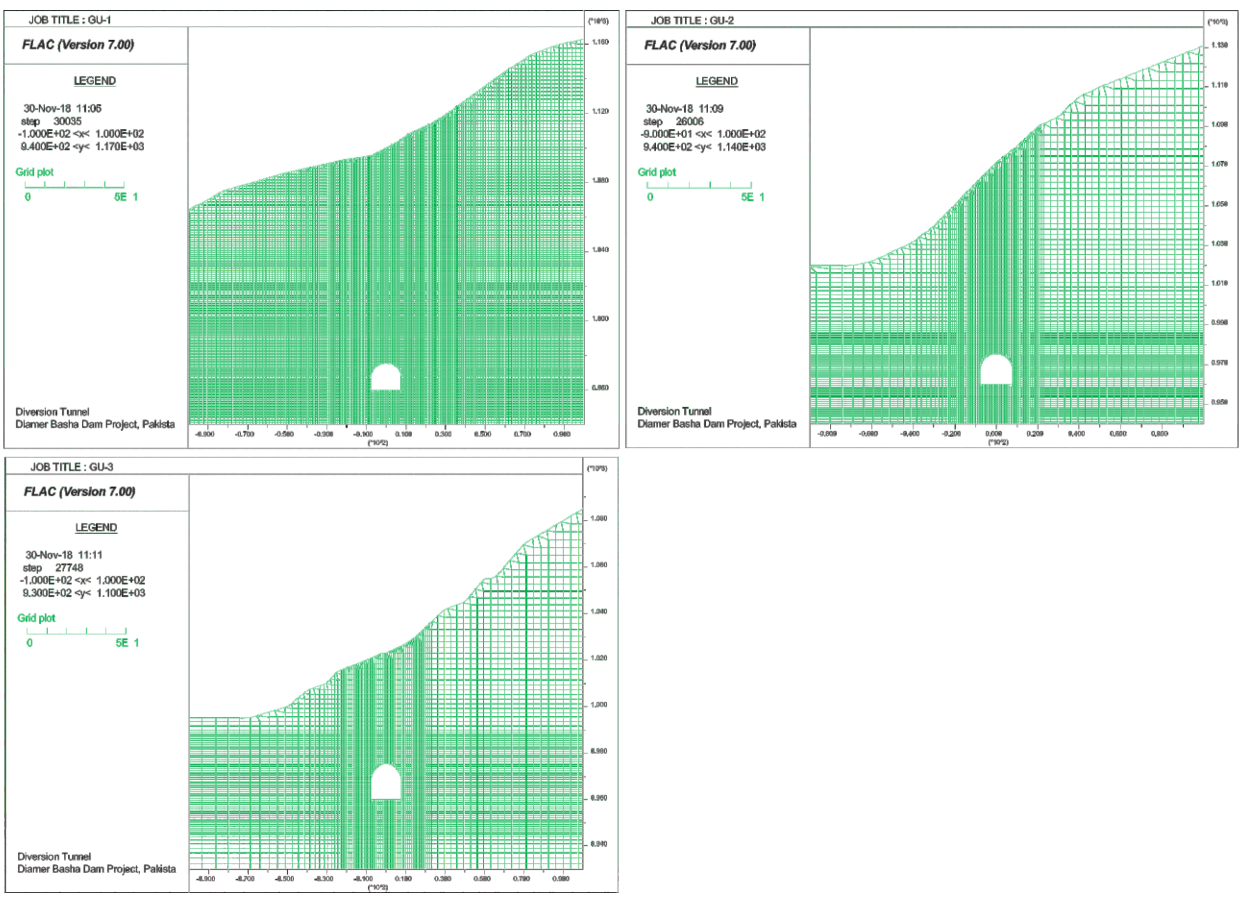
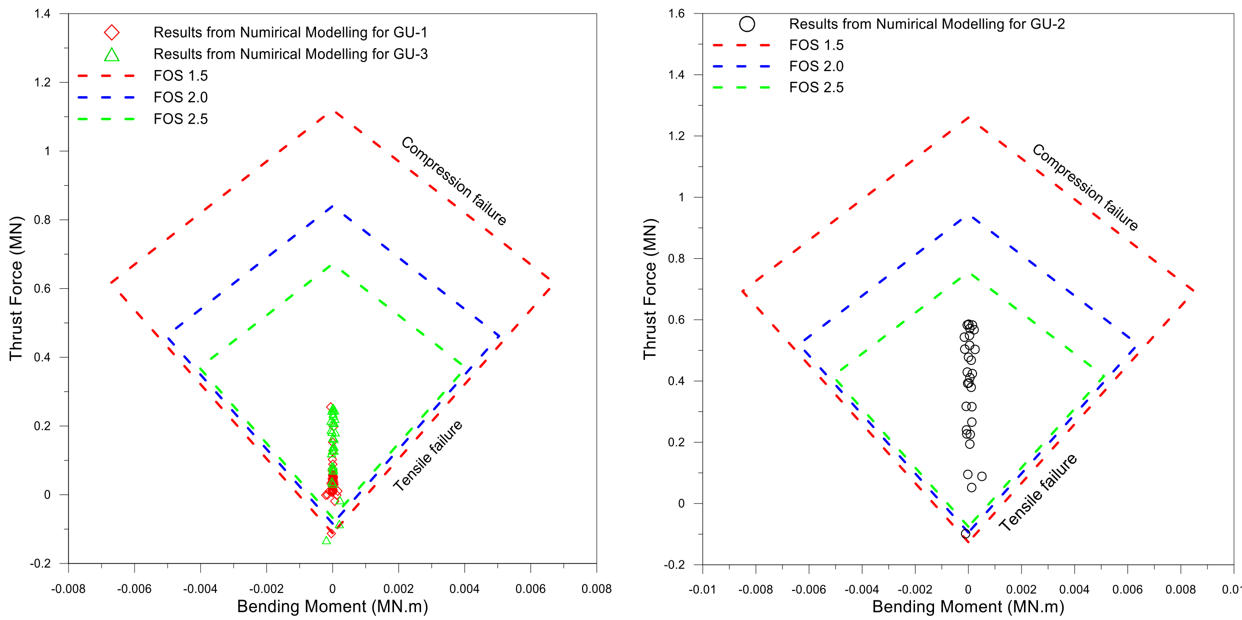


Fig. 5. Mesh generated for the three geological units GU–1 (top left), GU–2 (top right), and GU–3 (bottom).



a) 08 cm thick shotcrete b) 09 cm thick shotcrete

Fig. 6. Thrust–Bending moment interaction diagram for the three geotechnical units.

7. Conclusion

The conclusions acquired from this research work are as below:

1. The characterization and classification reveal that there are three geotechnical units along the tunnel route, namely GU–1, 2, and 3.

2. The average value of deformation modulus obtained from empirical equations gives acceptable values as the resultant mean value is matching with the deformation values obtained from project databased empirical equation for the purpose.

3. Comparing the resultant support from the two systems reveal that RMR suggested support is heavier than the Q system support.

4. The tunnel deformation in term of critical strain, and support evaluation in terms of axial stresses in rock bolt and thrust–bending moment interaction diagram for shotcrete reveals that the empirical support system is appropriate for the stability of tunnel.

Authors' Contribution

Hafeezur Rehman and Wahid Ali proposed the main concept and involved in write up. Hafeezur Rehman performed the numerical modelling. Abdul Muntaqim Naji contributed to reviewed the final paper and made important suggestions and recommendations for paper revision. Wahid Ali and Hafiz Muhammad Mukhtar helped in data collection. Saeed ullah Jan Mandokhail helped in writing and re-checking the paper technically as well as grammatically.

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