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Numerical modelling for geotechnical assessment of rock mass behaviour and performance of support system for diversion tunnels using optimized Hoek-Brown parameters

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Abstract

Purpose. Empirical and numerical methods play a vital role in assessing rock mass behaviour quantitatively and qualitatively to design underground structures/caverns and support systems. This research aims to assess and evaluate the rock mass be-haviour for safe, stable, efficient, and economical design of support system for underground structures especially tunnels in diverse rock mass conditions.

Methods. In this research, such empirical design methods as Rock Mass Rating (RMR), Q-system and GSI were used to characterize and classify the rock mass environment along the tunnel for the preliminary design of twin tunnels and support systems. The geomechanical parameters, Hoek-Brown failure criterion, and its variants for assessing rock mass behaviour were optimized using multiple regression of Stewart, generalized and globalized variant of nonlinear regression method. The rock mass was classified for the selected section A-A. The excavation method and support system for the said section were designed based on the results obtained from empirical modelling. 2D elasto-plastic finite element method (FEM) was used for numerical analysis of rock mass behaviour and performance of the designed supports in section A-A.

Findings. The major rock type encountered in the diversion scheme comprises gabbronorite (GN) and Ultramafic Association (UMA). Based on the quantification of RMR, Q-system, and GSI, section A-A's rock mass ranges from very poor to poor. From the numerical analysis for the said rock mass environment both RMR and Q system support recommendations are equally efficient to support the rock mass surrounding the tunnel. However, keeping in view the yield zone, especially in the crown, the rock bolt's length should not be less than 5 meters. Based on the analysis of results, both the tunnels are at a safe distance from each other.

Originality. In this research, the design input parameters for numerical modeling were optimized by using different techniques to eliminate the chances of error in evaluating rock mass behaviour and designing an optimum support system in the said rock mass environment.

Practical implications. The assessment of rock mass behaviour and the design of optimum support systems in heterogenous conditions is quite challenging and requires thorough investigation through different design techniques. This research provides a refined meth-od to be used for the safe, stable, and economical design of tunnels.

Keywords: rock mass, RMR, Q-system, UMA, GN, FEM

1. Introduction

The design and construction of unground structures involve certain potential risks due to the nature and characteristics of their spatial variation, rock mass behaviour, and level of knowledge. The success of an underground project can be achieved through advance and effective geotechnical investigation, adoption of the effective design method, effective ground stabilization, and monitoring techniques [1]-[3]. In the preliminary stage of execution of any under-ground civil and mining project, limited data about subsurface geology, ground hydrology, strength & stiffness of rock mass, and response or behaviour of rock mass to excavation is available [4]-[6].

Empirical design methods have success stories in the design of underground structures, both soft and hard rocks [7]-[13]. At the primary stage of tunnelling projects, empirical design methods, especially rock mass classification systems, can solve rock engineering problems [14]-[16]. Among these classification systems, Rock Mass Rating (RMR) and Q-system are internationally accepted design methods commonly used in the field of tunneling [4], [6], [17]. Although the empirical methods provide acceptable design for underground structures, these methods do not evaluate the response of excavation, rock mass behaviour, and effectiveness of support system in detail. An empirical analysis of tunnel in rocks, modeling of rock masses is challenging due

to anisotropy, heterogeneity, non-elastic and nonlinear nature of rock mass, and requirement of quality input data [18]-[20]. Furthermore, the design aspects i.e. shape and size of tunnel/exaction and support sequence, make the modeling more complex [21]-[23]. To evaluate the performance of support structures, stress redistribution, and stress deformation around tunnels, the empirical design methods are aided by numerical methods to produce more viable, authentic, safe, and economical design for excavation and supports [24]-[26].

Due to the low cost, time-efficient nature, convenience, and availability of user-friendly codes, numerical methods have got more attention in civil engineering and rock engineering for the solution of complex geometries tunnels and rock conditions [27]-[29]. Moreover, the addition of numerical analysis minimizes the risk uncertainties in the design. However, selecting a method out from available numerical methods depends on many factors including the nature of the problem, the capability of a method to solve a problem, and the simplicity of the codes available. The numerical methods give an optimum mathematical solution to a problem based on engineering judgment and rock mass behavior.

2. Materials and methods

In this study, the geological and geotechnical data of the Diamer Basha Dam site, especially the diversion scheme, is acquired from different investigative reports and literature. The ground conditions are established by collecting geological, hydrological, and mechanical data obtained from different reports and research papers related to the site. To characterize the rock mass of the dam site and obtain quality inputs to both empirical and numerical methods, the data is scrutinized and analyzed through statistical tools. RMR and Q- systems are used as design tools to design excavation and supports for diversion tunnels, whereas GSI is used to obtain inputs for the numerical tool. 2D Finite Element tool Phase^2 is used to evaluate the performance of the design support system and the response of ground to excavation.

2.1. Diversion scheme

The project consists of two parallel diversion tunnels with a centre-to-centre distance of 50 meters on the Right Bank (RB) of the dam and a diversion canal (DC), passing on the RB. Diversion Tunnel No. 1 (DT-1) is proposed to be used later as a flushing tunnel for the RB intake. The Diversion Tunnel No. 2 (DT-2) shall be plugged after its operation is finished. In this study, only tunnels of the diversion scheme will be analyzed. The DT-1 is 782 m long, while the DT-2 is 911 m long. These are D-shaped tunnels with 15.5 m width and 15.5 m height.

2.2. Location and geology

Diamir Basha Dam is proposed on River Indus, between the Khyber Paktunkhwa province and Gilgit Baltistan in Pakistan. The dam site is approximately 315 km upstream from the existing Tarbela Dam, 165 km downstream from Gilgit city, and 40 km from Chilas as shown in Figure 1.

Monenco in 1984, conducted geological mapping, surface mapping, and geotechnical investigations for preliminary feasibility. During 2001-2002, Water & Power Development Authority (WAPDA) extended the investigations through topographical surveys, detail geological mapping, and exploratory boreholes at the dam site and the second option of dam axis C2.

Figure 1. Location map of Diamer Basha Dam site

For feasibility, NEAC Consultant performed a comprehensive range of geotechnical studies, including exploration drill holes, geological mapping, geo-physical survey, two exploratory adits. The Diamer Basha Dam project is situated within the Ju-rassic-Cretaceous island arc in northern Pakistan, known as the Kohistan Arc. Due to the Indo-Pakistan Plate subduction below the Eurasian Plate is mainly produced between > 130 to 55 Ma [30].

The arc of Kohistan is an ancient island arc, created along with the arc of Ladakh as an eastern continuation, divided by the Nanga Parbat's syntax. They were sutured along with MKT (Main Karakorum Thrust, Shyok Suture) to the Eurasian Plate's previously accreted Karakoram block (80 ~ 90 Ma), and along with MMT in the south and east with the Indian Plate.

The major rock type encountered an area where the proposed diversion scheme comprises Gabbronorite (GN) and Ultramafic Association (UMA). The Ultramafic magmas have been injected into the Gabbronorite while they were still in molten/partially molten form. The rock body is made up of intermixing of the two types of magma.

2.3. Physical & geotechnical properties of the rocks

At the outcrops, the GN seems to be fresh and hardly weathered. Some part of the rock is just discoloured and mechanically weathered up to a depth of few centimetres. The intensity of this weathering seems to be proportional to the content of mafic minerals. These parts' surfaces show a light brown to light orange colour from iron oxides, which is derived from the weathering of Fe-bearing minerals. Below that, the rock shows occasional slightly discoloured pyroxenes and is otherwise fresh and hardly weakened. The ave-r-age unit weight of the rock is 29.2 kN/m^3.

Weathering and alteration affect the UMA rocks more than GN due to their minerals' less chemical and physical resistance. Usually, the classic pyroxenites and websterites are showing a red to rust-coloured staining on exposed surfaces. The grain boundaries are sometimes weakened to the extent that the rock can be easily crushed by hand. However, this zone of decomposition is diminishing after a few centimeters or more in some cases. The UMA is heavier than GN, with an average unit weight of 32.3 kN/m^3.

Seven boreholes in the area along the axis of diversion tunnels grouped the rock mass along the tunnel into three distinct geotechnical units. Data from boreholes and core logging suggest that the rock mass with high RQD values and mild to no weathering is of good quality massive rock. GN rock was observed in the area from the inlet up to 482 m along the tunnel axis in geotechnical unit –1. According to

the results from the other two boreholes, the rock mass from 482 to 630 m along the alignment is expected. The region of joint GN is located sub-parallel to the tunnel at and around the elevation of the diversion and attack. Two boreholes are drilled in Geotechnical unit 2. Ultramafic rock is located at 633 m from the Diversion Tunnels inlet. With moderate to strongly weathered rocks, the rock mass is more intensively joined together. The rock mass mostly consists of joints and therefore shows low RQD. Geotechnical Unit-3 starts from chainage 800 m; inlet up to the outlet of the tunnel goes in GN rock. As both the tunnels are parallel to each other and hence due to similarities in geological conditions, the discus-sion is on only Diversion Tunnel-1. Great variations have been observed in mechanical properties within the same geotechnical unit, especially strength, Poisson's ratio, and elastic modulus. To investigate the variation in the data and to obtain more realistic values of the parameters, the collected data is re-analyzed.

The geotechnical data was statistically analyzed along the tunnel axis in detail. Analysis of the results is presented in Table 1. The results show that the average value of water absorption and porosity for UMA is 0.7% and 2.22 respectively, while for GN Their value is 0.22% and 0.65%, which shows that UMA is more porous and permeable than SGN.

Table 1. Summary of physio-mechanical properties rock mass

Sr. No.	Strength Parameter	UMA				GN			
		Min.	Max.	Avg.	St. Deviation	Min.	Max.	Avg.	St. Deviation
1	Sp. gravity (g/cc)	2.84	3.54	3.29	0.17	2.87	3.01	2.94	0.03
2	Unit weight (kN/m^3)	28.3	34.80	32.30	0.17	28.6	29.8	29.2	0.03
3	Water absorption (%)	0.11	3.51	0.70	0.64	0.05	0.43	0.22	0.10
4	Porosity (%)	0.35	10.00	2.22	1.88	0.14	1.27	0.65	0.29
5	UCS (MPa)	14.14	138.00	85.80	32.92	71.00	203.00	124.10	34.10
6	UTS (MPa)	4.30	7.50	5.90	1.02	5.54	11.50	8.20	1.79

Similarly, for UMA, the average uniaxial compressive intensity with a standard deviation of 32.92 is 85.80 MPa. The mean value of Brazilian tensile strength for UMA's is 5.60 MPa with a standard deviation (SD) of 1.02, and for GN, the mean value of compressive and tensile strength is 124.10 and 8.20 MPa with an SD of 34.10 and 1.79, respectively, which demonstrates that GN rocks are highly resistant compared to UMA. To obtain the behaviour of rock at different stress levels and estimating rock and rock mass parameters, Hoek-Brown failure criterion and its variants, the Multiple regression of Stewart, Generalized and Globalized variant was applied to laboratory tests data. The failure criterion parameters are optimized using Excel add-in "Solver" as shown in Figures 2 and 3, respectively.
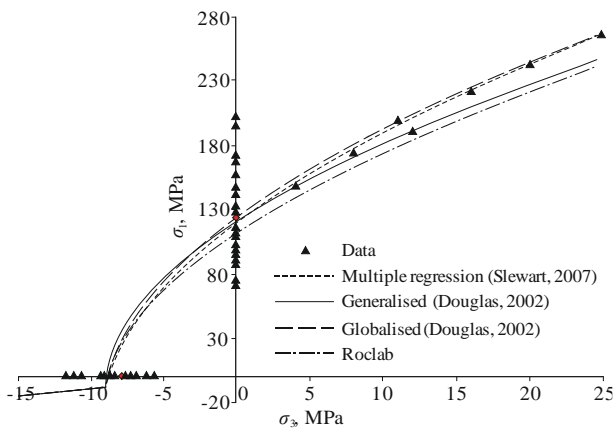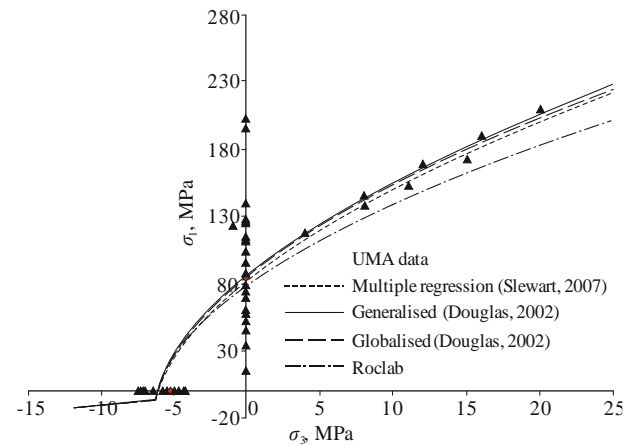

Figure 3. Curve fitting of Hoek & Brown failure criterion and its variants for data to intact UMA

The comparison of various variants is carried out based on residual. It is observed from the analysis that the globalized variant optimally described the behaviour of both the intact GN and UMA rock at different stress levels.

The various parameters gained from different fitting techniques are presented in Table 2 and 3.

After analysing data, the most probable values of physical and mechanical properties are summarized in Table 4 and Table 5, respectively.

Figure 2. Curve fitting of Hoek & Brown failure criterion and its variants for data about intact GN

Table 2. Statistics of Hoek & Brown failure criterion for GN

	Parameters	GN intact rock			
		RocLab analysis	Multiple regression developed by Stewart (2007)	The generalized variant of HB failure criterion	The globalized variant of HB failure criterion
1	σ_t (UTS) MPa	−11.22	−9.26	−9.70	−8.69
2	σ_c (UCS) MPa	117.24	122.77	120.51	124.73
3	α	0.50	0.50	0.56	0.48
4	m_i	10.55	13.26	12.42	14.36
	Sum of square difference (residual)	50450	42734	42370	37527

Table 3. Statistics of Hoek & Brown failure criterion for UMA

Parameters		GN intact rock			
		RocLab analysis	Multiple regression developed by Stewart (2007)	The generalized variant of HB failure criterion	The globalized variant of HB failure criterion
1	σ_t (UTS) MPa	−5.86	−5.76	−5.83	-5.64
2	σ_c (UCS) MPa	77.67	85.46	84.29	85.74
3	α	0.50	0.50	0.53	0.50
4	m_i	13.25	14.83	14.47	15.21
	Sum of square difference (residual)	50450	36420	33010	32907

Table 4. Summary of laboratory test results of GN rock

Parameter	Gabbronorite (GN)		
	Min	Max	Most occurring
Unit weight, γ (g/cm^3)	2.83	3.48	3.23
UCS, σ_c (MPa)	71.00	203.00	124.73
Uniaxial tensile strength, σ_t (MPa)	5.54	11.50	8.69
Modulus of elasticity, E (GPa)	20.00	120.00*	60.00
Poisson's ratio, υ	0.05	0.45	0.25
Hoek-Brown constant, m_i		14.36	

*The values seem abnormal

Table 5. Summary of laboratory test results of UMA rock

Parameter	Gabbronorite (GN)		
	Min	Max	Most occurring
Unit weight, γ (g/cm^3)	2.86	2.98	2.92
UCS, σ_c (MPa)	14.14	138.00	85.74
Uniaxial tensile strength, σ_t (MPa)	4.30	7.50	5.64
Modulus of elasticity, E (GPa)	20.00	140.00*	80.00
Poisson's ratio, υ	0.05	0.4	0.26
Hoek-Brown constant, m_i		15.21	

*The values seem abnormal

3. Results and discussion

3.1. Rock mass classes long tunnel axis

The geological section along the alignment of DT1 is shown in Figure 4. Based on the analysis of geological and geotechnical data, the rock mass along the axis of the diversion tunnel was classified into three zones, based on the dominating range of RMR values. The rock mass from inlet up to 482 m (Ch-00 to 742) is a massive rock of good quality, with medium to widely spaced joints GN. According to the RMR system, "Good" rock mass is the dominating class in this geotechnical unit. From 482 m (Ch-742) up to 633 m (Ch-893) the rock mass is of "Fair" to "Poor" quality with weak and closely jointed zones at places. From 633 m (Changes 893) up to outlet (Ch-1143), the tunnel passes through UMA, and the quality of the rock mass decreases as the rock shows many fractured zones and weathered joints with a 15 m thick zone of weakness, cutting the tunnel at a low angle and "Poor" rock class is predominant in this zone.

For numerical analysis, section A-A is selected along the alignment of the tunnel. The RMR value of rock mass is calculated from geological and geotechnical, and borehole data as presented in Table 6.
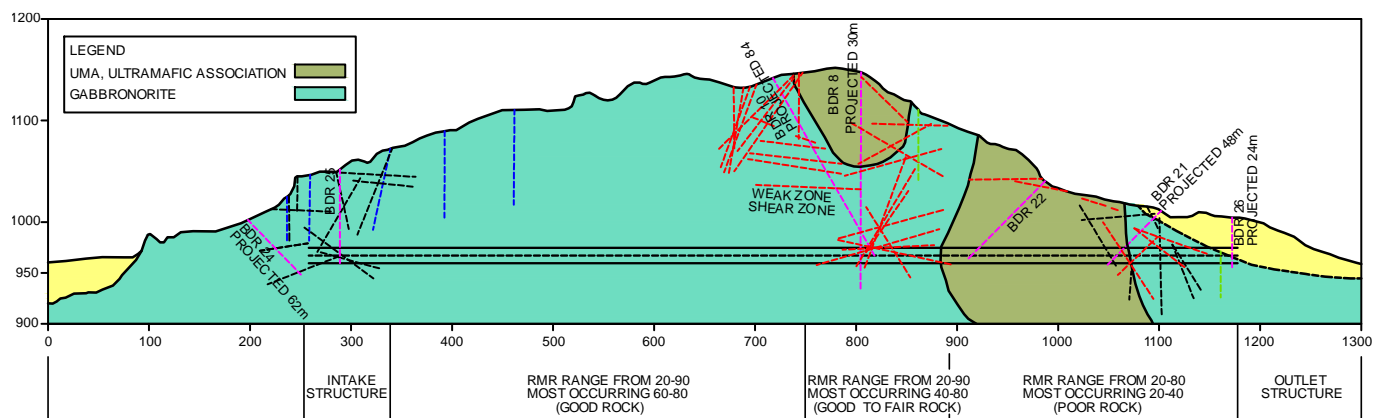

Figure 4. Geological section along the DT1 alignment

Table 6. Rock mass classification base on RMR Parameters for sections A-A

RMR parameters	Section A-A	
	Description of RMR parameters	Parameter rating
Uniaxial compressive strength	The value of uniaxial compressive strength is 100-50	7
RQD %	RQD 25%-50%	8
Spacing of discontinuity	Most occurrence is 0.2-0.06 m	7
Condition of discontinuity	There are open joints having length 1-3 m, slightly rough, soft material filled, and moderately weathered	13
Ground water	Wet to damp	7
Joint orientation for tunnels	Fair	−5
Average basic RMR	42	
Adjusted RMR	37	

GSI and Q values were estimated from RMR values. Classification of rock mass for the selected section based on RMR, GSI, and Q – the system is given in Table 7.

Table 7. Rock mass quality based on RMR, Q-system, and GSI for the selected sections A-A

Classification system	Section A-A	
	System rating	Rock mass quality
RMR-system	37	Poor
Q-system (Q = exp((RMR-44) / 9)	0.46	Very poor
GSI (GSI = RMR + 5)	42	Poor

Table 8. Support system recommendation based on RMR and Q-system for the selected sections A-A

Classification system	Rock mass quality	Excavation method	Support system		
			Rock bolting	Shotcrete	Steel sets
RMR	Poor	Excavation: top heading and bench, 1.0-1.5 m advance in the top heading	Systematic rock bolts 4-5 m long, spacing 1-1.5 m in the crown and sidewalls with wire mesh	100-150 mm shotcrete thickness in the crown and 100 mm in walls	Light to medium ribs at spaced 1.5 m where required
Q-system	Very Poor	Excavation: top heading and bench, 1.0-1.5 m advance in the top heading	Systematic rock bolts 3.5-4 m long, spacing 1.3 m in the crown and sidewalls	100-120 mm fiber-reinforced shotcrete in crown and walls	Light to medium ribs at spacing 1.5 m where required

Table 9. Rock mass geotechnical parameters

Rock	Hoek & Brown constants				
	σ_{crm} (MPa)	σ_{trm} (MPa)	m_b	s	a
UMA (weak) (section A-A')	1.128	0.025	1.20	0.0006	0.510

In design, the deformation module is the most representative input parameter, especially in numerical analysis. Two pre-existing models as proposed by [31] and [32] are used to estimate the deformation modulus of the rock mass along the tunnel's alignment. The deformation modulus values obtained from these models are given in Table 10.

Table 10. Rock mass geotechnical parameters

Rock	Deformation modulus (GPa)		
	(Hoek & Diederich, 2006)	(Tahir & Mohammad, 2014)	Avg.
UMA (weak) (section A-A')	4.045	1.573	2.809

3.3. In-situ stresses

In this research, the following Equation 1 was used for the estimation of vertical stresses:

$$\sigma_v = \gamma H , \qquad (1)$$

where:

γ – the unit weight, MPa;

H – the height of overburden, m.

The ratio between the horizontal stress and vertical stress is called constant represent by K and mainly depends on the depth of overburden. Determining the value of this ratio by calculation is more practical. The theoretical approach to determining horizontal stress from vertical stress is, however, easy to use. Horizontal stress is shown to depend on the constant elasticity of the rock mass [7]. The following Equation 2 is used for horizontal tension stresses:

$$\sigma_h =\left(\frac{\upsilon}{1-\upsilon}\right)\sigma_v + \frac{\beta E_{rm}G}{1-v}\left(H+100\right), \qquad (2)$$

where:

υ – poison ratio;

β – indicate the coefficient of thermal expansion, and mostly its value for rocks is $8 \cdot 10^{-6}$/°C;

E_{rm} – the rock young modulus, MPa;

G – the rock thermal gradient, °C/m.

However, the below-stated relationship is adopted in this research for estimation of the horizontal stress as:

$$\sigma_h =\left(\frac{\upsilon}{1-\upsilon}\right)\sigma_v. \qquad (3)$$

The overburden stresses along the tunnel axis are summarized in Table 11.

Table 11. Overburden stress along the tunnel axis

S.No.	Geotechnical units	Vertical stress (MPa)	Horizontal stress (MPa)	K-value
1	GTU-1	5.72	1.91	0.33
2	GTU-2 (section A-A)	3.17	1.11	0.35
3	GTU-3	1.23	0.41	0.33

3.4. Numerical analysis of support and stability

Based on geoengineering interpretation and consideration, together with the subsurface investigation and tunnel face observation, the rock mass along the tunnel is classified into three distinct zones varying from good to poor rock conditions. In the zone where the predominant rock mass belongs to the poor class, section A-A (Fig. 4) is selected for numerical analysis to analyze the stability of the tunnel and validate the support systems designed empirically.

Software Phase² is used to analyze the applicability of empirical design methods, i.e. RMR and Q, to determine the induced deformation around different sections and investigate rock and recommended support interaction.

The parallel diversion tunnels DT1 and DT2 are modelled in five stages. In stage-1, the virgin field conditions are validated, and in the stage, the rock mass behaviour was studied after excavation and support installation. The model is simulated using an elasto-plastic constitutive model with Generalized Hoek-Brown failure criteria for RMR and Q support systems. Figures 5, 6, 7, 8 and 9 show the numerical simulation results using Q support systems in Phase[2].

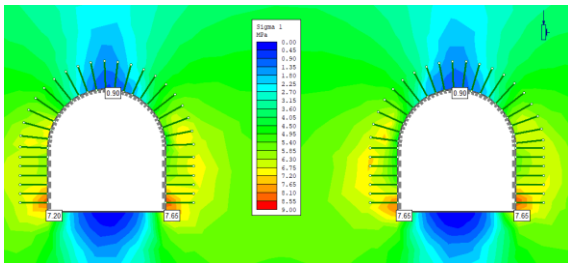*Figure 5. Phase[2] model of tunnel*

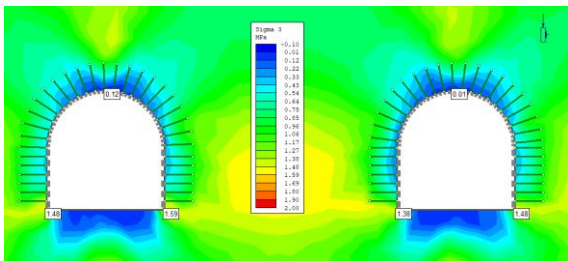*Figure 6. Major principal stress at stage-5 (Q support)*

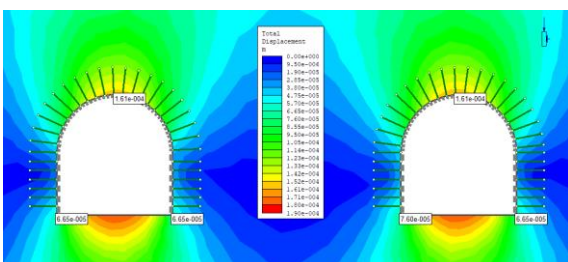*Figure 7. Minor principal stress at stage-5 (Q support)*

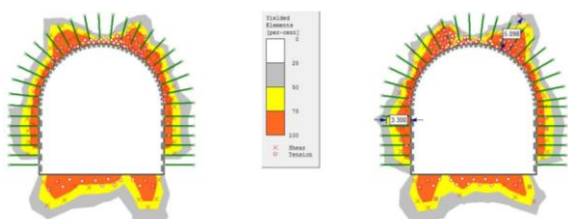*Figure 8. Total displacement at stage-5 (Q support)*

Figure 9. Yield zone and elements at stage-5 (Q Support)

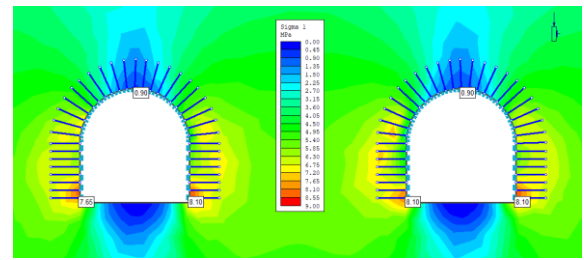Similarly, Figures 10, 11, 12 and 13 show the numerical simulation results using RMR support systems in Phase[2].

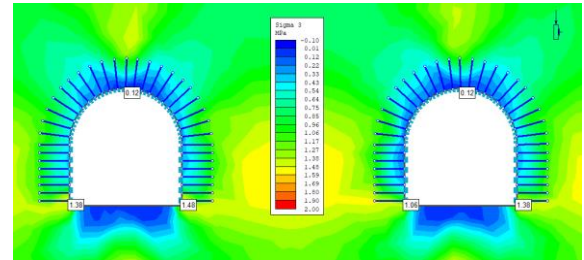*Figure 10. Major principal stress at stage-5 (RMR support)*

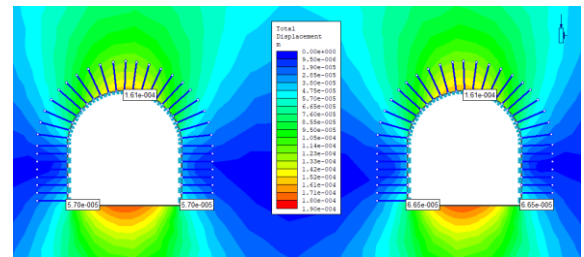*Figure 11. Minor principal stress at stage-5 (RMR support)*

Figure 12. Total displacement at stage-5 (RMR support)
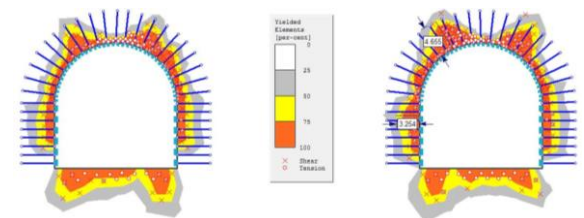
Figure 13. Yield zone and elements at stage-5 (RMR Support)

The Results from both RMR and Q support from simulated models are presented in Table 12 and 13 respectively.

The numerical analysis results validate that both the RMR and Q systems suggest a safe support system for the given rock mass condition. No yield elements are observed in rock bolts as well as shotcrete as recommended by RMR and Q system. The RMR support model (stage 4) shows that the yield zone in the crown is 4.6 m, while for the Q Support model, the Yield zone in the crown is 5.1 m.

4. Conclusions

The major rock type encountered an area where the diversion scheme is proposed, comprised of Gabbronorite (GN) and Ultramafic Association (UMA). The geological and geotechnical data indicate that in GN rock mass, the variation in the mechanical properties is less than UMA rocks, probably due to the inertness to chemical weathering of GN rock.

Table 12. Results from RMR support models

Sr. No.	Description	Stage -2		Stage-3		Stage-4		Stage-5	
		DT1	DT2	DT1	DT2	DT1	DT2	DT1	DT2
	The model with RMR support								
1	Total displacement in crown ($\cdot 10^{-5}$m)	14.40	0.90	15.2	0.95	16.10	14.20	16.10	16.10
2	Total displacement in wall ($\cdot 10^{-5}$m)	6.30	0.90	6.65	0	6.65	5.70	6.65	5.70
3	Max major principal stress (MPa)	4.90	4.90	8.10	4.50	8.25	5.25	8.10	8.10
4	Max minor principal stress (MPa)	0.45	1.50	1.33	1.43	1.35	0.60	1.38	1.48

Table 13. Results from Q support models

Sr. No.	Description	Stage -2		Stage-3		Stage-4		Stage-5	
		DT1	DT2	DT1	DT2	DT1	DT2	DT1	DT2
	The model with RMR support								
1	Total displacement in crown ($\cdot 10^{-5}$m)	14.20	0.95	15.20	0.95	15.20	15.20	16.10	16.10
2	Total displacement in wall ($\cdot 10^{-5}$m)	5.70	0.95	7.60	0	7.60	5.70	6.65	6.65
3	Max major principal stress (MPa)	5.20	4.55	7.65	4.50	7.50	4.50	7.65	7.65
4	Max minor principal stress (MPa)	0.60	1.50	1.52	1.43	1.50	0.60	1.48	1.59

Near the surface and at depth below the water table, the intact UMA rock pieces have low strength and stiffness properties.

Based on the quantification of RMR, Q-system, and GSI the rock mass quality at section A-A ranges from very poor to poor. For the said rock mass environment, both RMR and Q system support recommendations are equally efficient to support the rock mass surrounding the tunnel safely. No yield elements are observed in rock bolts as well as shotcrete as recommended by RMR and Q system. The RMR support model (stage 4) shows that the yield zone in the crown is 4.6 m, while for the Q support model, the yield zone in the crown is 5.1 m. However, keeping in view the yield zone, especially in the crown, the rock bolt's length should not be less than 5 meters. Based on the analysis of results, both tunnels are at safe distances from each other. This research provides a refined method to be used for the safe, stable, and economical design of tunnels.

Acknowledgements

References

[1] Rasouli, M. (2009). Engineering geological studies of the diversion tunnel, focusing on stabilization analysis and support design. *Engineering Geology*, 108(3), 208-224. https://doi.org/10.1016/j.enggeo.2009.07.007

[2] Osgoui, E., & Ünal, R. (2005). Rock reinforcement design for unstable tunnels originally excavated in very poor rock mass. *Underground Space Use: Analysis of the Past and Lessons for the Future*, 291-296.

[3] Rehman, H., Ali, W., Naji, A., Kim, J., Abdullah, R., & Yoo, H. (2018). Review of rock-mass rating and tunneling quality index systems for tunnel design: Development, refinement, application and limitation. *Applied Sciences*, 8(8), 1250. https://doi.org/10.3390/app8081250

[4] Hussain, S., Ur Rehman, Z., Mohammad, N., Tahir, M., Shahzada, K., Wali Khan, S., & Gul, A. (2018). Numerical modeling for engineering analysis and designing of optimum support systems for headrace tunnel. *Advances in Civil Engineering*, (2018), 1-10. https://doi.org/10.1155/2018/7159873

[5] Gurocak, Z., Solanki, P., & Zaman, M. (2007). Empirical and numerical analyses of support requirements for a diversion tunnel at the Boztepe dam site, eastern Turkey. *Engineering Geology*, 91(2), 194-208. https://doi.org/10.1016/j.enggeo.2007.01.010

[6] Barton, N., Lien, R., & Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics Felsmechanik Mecanique des Roches*, 6(4), 189-236. https://doi.org/10.1007/BF01239496

[7] Rehman, Z., Mohammad, N., Hussain, S., & Tahir, M. (2019). Numerical modeling for the engineering analysis of rock mass behaviour due to sequential enlargement of Lowari tunnel Chitral Khyber Pakhtunkhwa, Pakistan. *International Journal of Geotechnical Engineering*, 13(1), 1-7. https://doi.org/10.1080/19386362.2017.1319615

[8] Genis, M., Basarir, H., Ozarslan, A., Bilir, E., & Balaban, E. (2007). Engineering geological appraisal of the rock masses and preliminary support design, Dorukhan Tunnel, Zonguldak, Turkey. *Engineering Geology*, 92(1), 14-26. https://doi.org/10.1016/j.enggeo.2007.02.005

[9] Hussain, S., Mohammad, N., Tahir, M., Rehman, Z., & Mohammad, N. (2016). Rock mass characterization along the tunnel axis for Golen Gol hydropower project Chitral, Pakistan. *Journal of Himalayan Earth Sciences*, 49(2), 75-83.

[10] Basarir, H., Ozsan, A., & Karakus, M. (2005). Analysis of support requirements for a shallow diversion tunnel at Guledar dam site, Turkey. *Engineering Geology*, 81(2), 131-145. https://doi.org/10.1016/j.enggeo.2005.07.010

[11] Hussain, S., Mohammad, N., Khan, M., Rehman, Z., & Tahir, M., (2016). Comparative analysis of rock mass rating prediction using different inductive modeling techniques. *International Journal of Mining Engineering and Mineral Processing*. 9-15, https://doi.org/10.5923/j.mining.20160501.02

[12] Hussain, S., Khan, M., Rehman, Z., Mohammad, N., Raza, S., Tahir, M., Ahmad, I., Sherin, S., & Khan, N.M. (2018). Evaluating the predicting performance of indirect methods for estimation of rock mass deformation modulus using inductive modelling techniques. *Journal of Himalayan Earth Sciences*, 51(1), 61-74.

[13] Wang, X., Kulatilake, W., & Song, W.D. (2012). Stability investigations around a mine tunnel through three-dimensional discontinuum and continuum stress analyses. *Tunnelling and Underground Space Technology*, (32), 98-112. https://doi.org/10.1016/j.tust.2012.06.003

[14] Bieniawski, Z. (1989). *Engineering rock mass classifications: A complete manual for engineers and geologists in mining, civil, and petroleum engineering*. Hoboken, United States: Wiley, 272 p.

[15] Andriani, G.F., & Parise, M., (2017). Applying rock mass classifications to carbonate rocks for engineering purposes with a new approach using the rock engineering system. *Journal of Rock Mechanics and Geotechnical Engineering*, 9(2), 364-369. https://doi.org/10.1016/j.jrmge.2016.12.001

[16] Biniawski, Z. (1993). Classification of rock masses for engineering: The RMR system and future trends. *Rock Testing and Site Characterization*, 553-573. https://doi.org/10.1016/B978-0-08-042066-0.50028-8

[17] Aydan, Ö., Ulusay, R., & Tokashiki, N. (2015). Rock mass quality rating (RMQR) system and its application to the estimation of geomechanical characteristics of rock masses. *Engineering Geology for Society and Territory: Applied Geology for Major Engineering Projects*, (6), 769-772. https://doi.org/10.1007/978-3-319-09060-3_137

[18] Hashemi, M., Moghaddas, S., & Ajalloeian, R. (2010). Application of rock mass characterization for determining the mechanical properties of rock mass: A comparative study. *Rock Mechanics and Rock Engineering*, (43), 305-320. https://doi.org/10.1007/s00603-009-0048-y

[19] Singh, J., & Tamrakar, N.K., (2013). Rock Mass Rating and Geological Strength Index of rock masses of Thopal-Malekhu River areas, Central Nepal Lesser Himalaya. *Bulletin of the Department of Geology*, (16), 29-42. https://doi.org/10.3126/bdg.v16i0.8882

[20] Rehman, H., Naji, A.M., Kim, J.J., & Yoo, H.K. (2018). Empirical evaluation of rock mass rating and tunneling quality index system for tunnel support design. *Applied Sciences*, 8(5), 782. https://doi.org/10.3390/app8050782

[21] Akram, M., & Zeeshan, M. (2018). Rock mass characterization and support assessment along power tunnel of hydropower in Kohistan area, KPK, Pakistan. *Journal of the Geological Society of India*, 91(2), 221-226. https://doi.org/10.1007/s12594-018-0839-4

[22] Hussian, S., Mohammad, N., Rehman, Z., Khan, N.M., Shahzada, K., Ali, S., Tahir, M., Raza, S., & Sherin, S. (2020). Review of the geological strength index (GSI) as an empirical classification and rock mass property estimation tool: Origination, modifications, applications, and limitations. *Advances in Civil Engineering*, (2020), 6471837. https://doi.org/10.1155/2020/6471837

[23] Bobet, A. (2010). Numerical methods in geomechanics. *Arabian Journal for Science and Engineering*, *35*(1B), 27-48. https://doi.org/10.1016/0378-3839(84)90026-7

[24] Jing, L. (2003). A review of techniques, advances and outstanding issues in numerical modelling for rock mechanics and rock engineering. *International Journal of Rock Mechanics and Mining Sciences*, *40*(3), 283-353. https://doi.org/10.1016/S1365-1609(03)00013-3

[25] Mohammadi, H., & Rahmannejad, R. (2010). The estimation of rock mass deformation modulus using regression and artificial neural notworks analysis. *Arabian Journal for Science and Engineering*, *35*(1), 205-217.

[26] Jing, L., & Hudson, J.A. (2002). Numerical methods in rock mechanics. *International Journal of Rock Mechanics and Mining Sciences*, (39), 409-427. https://doi.org/10.1016/S1365-1609(02)00065-5

[27] Aksoy, C.O., Geniş, M., Uyar Aldaş, G., Özacar, V., Özer, S.C., & Yilmaz, Ö. (2012). A comparative study of the determination of rock mass deformation modulus by using different empirical approaches. *Engineering Geology*, (131), 19-28. https://doi.org/110.1016/j.enggeo.2012.01.009

[28] Moldovan, A.R., & Popa, A. (2012). Finite element modelling for tunneling excavation. *Acta Technica Napocensis: Civil Engineering & Architecture*, *55*(1), 335.

[29] Moldovan, A.R., & Popa, A. (2012). Finite element modelling for tunneling excavation Rezumat. *Acta Technica Napocensis: Civil Engineering & Architecture*, *55*(1), 98-113.

[30] Ali, W., Mohammad, N., & Tahir, M. (2014). Rock mass characterization for diversion tunnels at diamer basha dam, Pakistan – A design perspective. *International Journal of Engineering, Science and Technology*, *3*(10), 1292-1296.

[31] Hoek, E., & Diederichs, M.S. (2005). Empirical estimation of rock mass modulus. *International Journal of Rock Mechanics and Mining Sciences*, (43), 203-215. https://doi.org/10.1016/j.ijrmms.2005.06.005

[32] Tahir, M., & Mohammad, N. (2014). Prediction performance and generalization of the empirical estimation of rockmass deformation modulus based on rockmass classification systems. *International Journal of Engineering, Science and Technology*, *3*(12), 1488-1495.

Чисельне моделювання геотехнічної оцінки поведінки гірничого масиву та ефективності кріплення дериваційних тунелів з використанням оптимізованих параметрів критерію Хоека-Брауна

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Мета. Оцінка поведінки гірського масиву для безпечної, стабільної, ефективної та економічної конструкції кріплення підземних споруд, особливо тунелів у різних умовах гірського масиву на основі емпіричних та чисельних методів дослідження.

Методика. Використані емпіричні методи проектування – рейтингова система класифікації гірських порід (RMR), Q-система та індекс геологічної міцності GSI – для характеристики та класифікації середовища гірського масиву вздовж тунелю та для попереднього проектування подвійних тунелів і кріплення. Геомеханічні параметри, критерій руйнації Хоека-Брауна та його варіанти оптимізовані із використанням множинної регресії Стюарта, узагальненого та глобального варіанта методу нелінійної регресії. Метод виймання та кріплення для зазначеної ділянки визначено на основі емпіричного моделювання. Двовимірна еластопластична реалізація методу скінчених елементів (МСЕ) була використана для чисельного аналізу поведінки масиву гірських порід та характеристик кріплення на ділянці А-А.

Результати. Встановлено, що основний тип породи, що зустрічається в зоні передбачуваного відведення, включає габронорит та ультрамафічну асоціацію. Кількісна оцінка за RMR, Q-системою та GSI дозволяє класифікувати гірську масу на розрізі А-А як слабку або дуже слабку. На основі чисельного аналізу гірського масиву розроблені рекомендації за допомогою систем RMR та Q, які однаково ефективні для проектування кріплення гірського масиву, що оточує тунель. Встановлено з урахуванням зони деформованості у покрівлі, що довжина штанги має бути не менше 5 м. Аналіз показав, що обидва тунелі знаходяться на безпечній відстані один від одного.

Наукова новизна. Вхідні конструктивні параметри для чисельного моделювання були оптимізовані із використанням комплексу методів, щоб унеможливити помилки при оцінці поведінки гірського масиву та проектуванні оптимального кріплення в його середовищі.

Практична значимість. Дане дослідження пропонує вдосконалений метод, який можна використовувати для безпечного, стабільного та економічного проектування тунелів.

Ключові слова: гірський масив, рейтингова система, Q-система, метод скінчених елементів, кріплення