EMPIRICAL AND NUMERICAL ANALYSIS OF DIAMER BASHA DIVERSION TUNNELS

DEPARTMENT OFGEOLOGICAL ENGINEERING

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BALOCHISTAN UNIVERSITY OF INFORMATION TECHNOLOGY ENGINEERING & MANAGEMENT SCIENCES

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BALOCHISTAN UNIVERSITY OF INFORMATION TECHNOLOGY ENGINEERING AND MANAGEMENT SCIENCES

For the partial fulfillment of the requirements for the degree of

BACHELOR OF SCIENCE (BS)

In

GEOLOGICAL ENGINEERING

By

ATHER HUSSAIN 51756

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DECLARTION OF THE AUTHORS

It is hereby declared by Athar, Haris that the topic of our BS project/thesis "2D Tunnel Support Analysis of diversion tunnels "Diamer Basha with Phase2 software", is our own spectacular work and it has only been presented for a BS degree in Geological Engineering at Baluchistan University of Information Technology, Engineering & Management Sciences, Quetta.

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CERTIFICATE OF APPROVAL

The proposed thesis entitled and presented **"Empirical and numerical Analysis of Diamer Basha Diversion Tunnels"**

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Students of BS Geological Engineering Department were evaluated and recommended by Final Year Design Project (FYDP) in its meeting held on ________________________

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ABSTRACT

In this Research, the rock masses along the axis of diversion tunnels at Diamer Basha Dam, Pakistan were characterized for evaluation of rock mass behavior. Geological and geotechnical studies were performed to classify the rock masses along the axis of diversion tunnel. The core specimens retrieved were tested in laboratory and physical and mechanical properties of the rock masses were determined. Detailed discontinuity survey was carried out to determine the type and orientation of discontinuities with respect to direction of tunnel drive. The rock mass was classified using Rock Mass Rating (RMR) and Q-System. Based on rock mass classification results, the rock mass along the tunnel alignment was divided into three Geotechnical units, where the combined effects of the engineering-geological conditions, the initial stress situation and the ground water conditions were predicted to present consistent tunneling conditions.

It was concluded from the analysis of rock mass behavior that conventional drill and blast excavation method is most suitable to be used for smooth blasting by skilled workers. The rock mass classification systems e.g RMR and Q-system clearly shows that the rock mass in the project area is competent and require minimum support. Keeping in view the qualitative and quantitative values of the RMR and Q-system optimum support system is designed to stabilize the tunnel.

The overburden stresses were estimated using empirical equations. The Hook & Brown and M-C parameters were also calculated using RocLab software

Key words: Rock mass classification, Failure Criteria, Support Analyzing using Phase2 Software.

CHAPTER 1

INTRODUCTION

Introduction

1.1 Background

The design and construction of unground structures involve certain potential risks due to the nature and characteristics of their spatial variation, rock mass behavior, 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 (Rasouli 9). Limited information about subsurface geology, ground hydrology, strength & stiffness of rock mass, and response or behavior of rock mass to excavation is available in the early stages of execution of any underground civil and mining project.(Hussain, Ur Rahman et al. 2018). Empirical design methods have success stories in the de-sign of underground structures, both soft and hard rocks (Ur Rehman, Mohammad et al. 2019). Empirical design techniques, in particular rock mass classification systems, can resolve rock engineering issues during the first stages of tunneling projects. The widely recognized design methodologies Rock Mass Rating (RMR) and Q-system, which are frequently employed in the tunneling industry, are among these classification systems (Bieniawski 1989).

Although the empirical approaches offer an appropriate design for underground structures, these methods do not thoroughly assess how excavation will respond, how rock masses will behave, and how well the support system will work. Due to the anisotropy, heterogeneity, non-elasticity, and nonlinearity of rock masses as well as the need for high-quality input data, modeling rock masses for an empirical analysis of tunnels in rocks might be difficult. Additionally, design factors like tunnel shape, size, exaction, and support sequence add to the complexity of the modeling (Akram and Zeeshan 2018).

Numerical approaches have received increased attention in civil engineering and rock engineering for the solution of complicated geometries in tunnels and rock situations because to their low cost, time-efficient nature, convenience, and availability of user-friendly codes. Additionally, the addition of numerical analysis reduces the design's risk uncertainty. However, choosing a method from the available numerical methods depends on a number of variables, including the nature of the problem, a method's ability to solve the problem, and the ease of the available codes. Based on engineering expertise and the behavior of rock masses, numerical methods provide the best mathematical solution to a problem (Moldovan and Popa 2012).

1.2 Problem Statement

Tunnel support by empirical methods does not provide evaluation of response of excavation, rock mass behavior modeling of rock masses is challenging due to anisotropy, heterogeneity,nonelastic behavior and nonlinear nature of rock mass. Therefor evaluation of support structures, stress redistribution, and stress deformation around tunnels, empirical design methods are aided by numerical methods. To produce more viable, authentic, safe and economical design of excavation.

1.3 Aim and Objectives

- To analyse the rock mass of the dam site and quality inputs to both empirical and numerical methods.
- To analyse the tunnel support through different empirical approaches.
- To analyse the performance of the design support system and response to ground excavation with the help of phase 2 software
- To assess the behavior of diversion tunnels through numerical simulation.

1.4 Location and Geology of Diamer Basha project.

There are two Hydroelectric Power Schemes, one on each bank of the Indus River, that make up the Diamer Basha Dam Project. The dam itself is 270 metres in height and is made of Roller Compacted Concrete (RCC). Power caverns, transformer and switchgear caverns, headrace and tailrace tunnels, surge tanks, access and diversion tunnels, and so on are all part of the enormous and complicated network of subterranean structures that make up both power projects. The Pakistan Water and Power Development Authority (WAPDA) contracted with the Diamer Basha Consultants (DBC), managed by Lahmeyer International GmbH, Germany, to analyse the Feasibility Report, develop the Engineering Design with Tender Drawings, and issue Tender Documents for the Project. The current Tender Design research is the fourth and final research phase for the Diamer Basha Dam Project, which began with a feasibility study and included site and laboratory studies. The previous twelve-volume report was the Basha Diamer Dam Project Feasibility Study that was developed by NEAC Consultants and delivered in August 2004. The conclusions of the Feasibility Study and the earlier stages of the project, beginning with the preliminary feasibility study by Monenco (Montreal Engineering Company, Canada), are detailed in Volume IV on Geology and

Geo-Engineering, which includes twelve appendices. The next chapter discusses the engineering geological evaluation of the river diversion constructions. The plan calls for two tunnels on the right bank and a canal that runs parallel to the RB to be diverted. The southern tunnel will be used to cleanse the RB intakes in the future; it is now marked as Diversion Tunnel no. 1. After DT 2 in the North has served its purpose, it must be sealed off. Dwg. No. CW-01-002 depicts the overall plan for the project. Dwg. No. CW-01-001 depicts the building operations with the many phases that must be performed during the dry and wet seasons.

1.5 Engineering Geology of Diversion Canal and Tunnels

The surface geology was mapped during GBM preparation (1:2500) as part of the data collection for the study of the river diversion constructions. Dwg. No. GEO-01-001 (1: 7500) provides a summary of the surface geology of the dam site. The GBM's 1: 2500 scale Dwg. Nos. GEO-01-003, -004, -005, and -007 cover the regions affected by the diversion system. In addition, two sheets of 1:500 scale detailed geological mapping (Dwg. Nos. GEO-01-601 and -602, respectively) cover the combined power/diversion intake. Only the structures at the inlets and outlets of the diversion tunnels had seen any significant exploratory drilling. Besides the u/s section of the tunnels is sufficiently covered with boreholes. Boreholes for the diversion tunnels are outlined in detail in Drawing No. GEO-06-103; for their precise positions, please refer to Drawing No. GEO06-101.

In addition, three boreholes were sunk below ground level for the main dam construction between the expected flushing gate chamber and diversion tunnel plug 2. Each hole is designated by a number, such as BDR-8, BDR-9, or BDR-10. Only three drill holes have been made into the diversion canal's d/s section, where it crosses the alluvial flood plain d/s of the RB rock saddle. The locations of these craters are marked on Drawing No. GEO-06105. Alluvium was used for the drilling, with some CPT and SPT performed on the overburden. Other holes, bored for the main dam construction, either completely or partially conceal the diversion canal. The BDR-6, -11, -17, and 20 are only a few examples. Every one of the aforementioned boreholes had undergone coring and logging. Split spoon sampling was used to recover a small sample of the overburden material on the alluvial flood plain. Only wash samples were collected from the remaining overburden. DVD 1 of the Factual Report Geology includes logs and images from the core boxes. In order to determine the best canal alignment, a few test holes were drilled as part of the feasibility study. The diversion canal was drilled on the d/s side, and this was DSCD-1. The u/s section of the canal inside the RB rock saddle also has the NDR-1, NDR-2, NDR-4, and NDR-5 wells. The factual report includes records and images.

In this investigation, a single packer was used to measure water pressure in increments of five metres down the boreholes. The WPT was performed when the core barrel was collected from the first 5 metres of coring. The time gap was shortened if the stratum was very porous. The results of the tests were analysed visually, and their Lugeon values were calculated. DVD 2 of the Factual Report Geology includes the supplementary graphics. Overburden has not been subjected to any permeability testing.

In 8 of the aforementioned boreholes, the borehole scanner system (ETIBS®) was deployed. Due to obstructions and subsequent sealing of boreholes, not all of the holes were scannable. In conjunction with the programme WellCAD it was feasible to get orientation data from discontinuities below surface (Factual Report Geology). Overburden-penetrating boreholes were not analysed.

The DT intake region and the RB rock saddle region were surveyed for surface joints. Joint orientations and characteristics were used to analyse the survey data. Both current and historical geophysical investigation data has been analysed. It was interpreted with a primary emphasis on bedrock depths around the alluvial floodplain, DT exits, and intakes. Hand specimens and core samples were tested in a laboratory to determine the whole rock's characteristics. Engineering geological evaluation of dam uses the same data sources and test findings as the previous chapter.

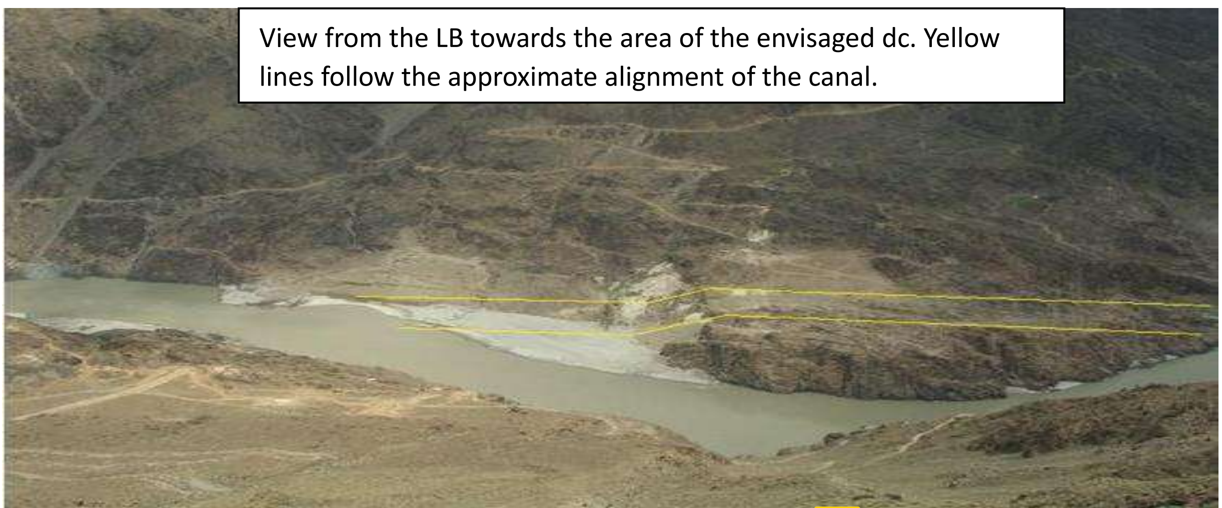


Figure 1 View of diversion tunnels

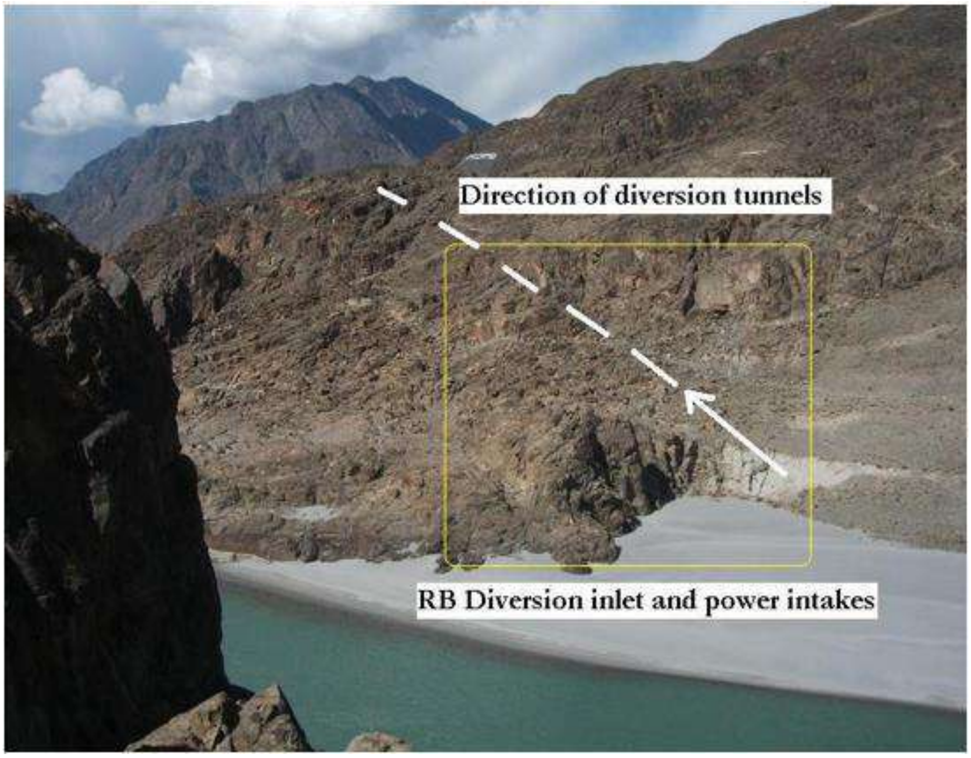


Figure 2 View of the inlet portal of diversion Tunnels.

CHAPTER 2
LITERATURE REVIEW

Literature Review

2.1 Bilecik-Istanbul Roadway

The goal of the research conducted by Sari et al. was to analyses a tunnel's support system using computational and empirical methods. The location chosen for that study was a tunnel built on the Turkish highway connecting Bilecik and Istanbul. Through empirical methods, they were able to determine the characteristics of rock mass and make design support recommendations. These characteristics were then employed as input parameters for the numerical analysis. In terms of support design, the outcomes from numerical and empirical methodologies were examined. It was demonstrated that empirically based numerical analysis results were the most rational and trustworthy (Sari, Gunhan Pasamehmetoglu et al. 2008).

2.2 Youfangping Tunnel

Engineering has long focused on improving the support system utilized for building tunnels through soft surrounding rock. That thesis suggests a support system that involves weakening the anchor bolts while boosting the rigidity and strength of the major supports in order to regulate the deformation of soft rock and assure construction safety. This was accomplished by combining the significant deformations that frequently took place during the construction of the Gucheng-Zhuxi expressway's Youfangping tunnel. The plan was also examined and contrasted with the original idea and another that called for weakening the anchor bolts. Additionally, numerical simulation was used to analyses the displacement deformations, force conditions on the anchor bolts, formation of plastic zones, stress on the shotcrete, and force conditions on the secondary lining structure (Gao, Chen et al. 2016).

2.3 Cankurtaran Tunnel Project

This study's primary goals are to evaluate the geotechnical properties of the rock masses and to suggest a support design for the Cankurtaran Tunnel project, which is located in northeastern Turkey. To identify the characteristics of rock masses made up mostly of volcanic and sedimentary rocks, a thorough engineering geological research was conducted. The 15 segments that make up the tunnel path were chosen based on their lithological and structural characteristics. The quality of rock masses and final tunnel lining support were assessed using the rock masses rating (RMR) and Rock Mass Quality Index (Q) systems. Utilizing the convergence-confinement (CC) technique, the analytical performance of the proposed support units was evaluated. Numerical finite element method (FEM) modelling in 2D and 3D was used to establish the support design, plastic zone size, and deformation. The total displacement and dimension of the plastic zone were lowered by the empirical support system presented in this work.

An accurate assessment and the prevention of portable dangers are vital in his research activity, which is conducted in accordance with underground construction development and its expensive procedure. A variety of techniques have been developed to evaluate underground structures. A new soft computing model was created as a result of the research to assess tunnel support systems (Kaya and Bulut 2019).

2.4 Melbourne metro tunnel

In his research, they made the crucial prediction about settlement that was necessary to prevent the ground's current buildings from collapsing. However, the support they were giving and the subterranean circumstances might result in major settlement problems and tunnel collapse. They used the Rock Science 2D and RockSience3D software to simulate the excavation condition and evaluate the impact of various factors. As a result, they employed the two-dimensional and threedimensional finite element methods. The tri-arch State Library Station and twin tunnels beneath Melbourne Formations served as the backdrop for the study area, which was the Melbourne Metro Tunnel. According to their research, a library station's maximum settlement is roughly 6.4 millimeters. It was discovered that techniques like sturdy supporting systems with rock bolts, segment lining, and columns, as well as an optimized excavation sequence and avoiding the construction of new structures close to the station, can lessen settling on the ground surface and in tunnels. They were developing a settlement prediction and assessment of factors on the tunnel project that not only offered settlement prediction and recommendations of factors on settlement control for the tunnel project practically, but also displayed a thorough analysis of settlement prediction that taking potential factors into consideration (Liu and Zhang).

2.5 Horemheb Tomb (Kv57)

Using the phase two software, the intricate underground structure of Horemheb's tomb (KV57) in Luxor, Egypt, is analyses. The failure loads, which are derived from a series of laboratory tests, are applied, and the deformation that takes place in the underground structure's body after application are detected. The structure is then accurately two-dimensionally analyzed for stability and deformation in complex geotechnical engineering and rock mechanics using finite element code. The soil properties derived from laboratory tests are necessary for modelling. The elastic-plastic Mohr-Coulomb material model is employed in the analysis. It requires five variables: Young's modulus (E), poison's ratio (v), friction angle (), cohesion (c), and soil plasticity (c) (Oke, Vlachopoulos et al. 2016).

2.6 Niayesh urban road tunnel

Sequential excavation method (SEM) was used for tunnel construction because of the soft ground (SD) tunnel building technologies were put forth at this phase, and the most suitable option was chosen based on its capacity to reduce surface settlements. The best excavation sequence was then chosen after planning and modelling several excavation sequences utilizing the side wall drift approach in three dimensions. The ideal distance with the least amount of surface settling was found after a numerical analysis of the trailing distance between various excavation phases (Gao, Chen et al. 2016).

2.7 Diamer Bhasha Dam

Sedimentation in reservoirs is a worldwide problem that poses a danger to reservoir durability and productivity. Diamer Bhasha Dam and other big reservoirs in Pakistan have lost 33 percent of their capacity due to sedimentation, demonstrating the need of sediment control. The annual sediment inflow into the planned dam, with a capacity of 10 BCM, is estimated to be 196.91 million tonnes. The research recommends sediment flushing as a viable management approach that might increase the reservoir's longevity to more than 140 years. To finance the construction of the DiamerBhasha and Mohmand dams in 2018, Pakistan's then-Chief Justice, Mian Saqib Nisar, launched a crowdfunding effort. Diamer-Bhasha dam has become a symbol of Pakistan's prosperity despite initial scepticism over the project's viability (Tareen, 2022). Tareen concludes that the campaign's success may be traced back to the sympathetic but optimistic portrayal of Pakistani Muslims that it presents.

From 1962 to 2016, Hussain, Shahab, and tibinger (2020) analysed the river discharge patterns in the Upper Indus River Watershed. Using a number of different types of analysis, they found that there were noticeable changes in the flow of water both before and after the Bhasha dam was built. Positive effects on the river environment and the prospect of hydroelectric developments downstream are highlighted in the study, along with the dam's other ecological and infrastructure advantages.

2.8 Challenges in Diamer Basha Diversion Tunnel Modeling

An important water resource project that aims to supply Pakistan with water for cultivation and hydropower generation includes the Diamer Basha diversion tunnels. Many studies studying the application of real-life values in numerical modeling have been carried out in order to guarantee the efficient building and maintenance of these tunnels. To demonstrate the use of empirical data in improving the modeling of the Diamer Basha diversion tunnels, the study explores the body of existing research related to this matter. The diversion tunnels at Diamer Basha have special difficulties that are needed for careful numerical modeling. The region's hydrological and geological features are the main causes of these difficulties. The area including the Diamer Basha diversion tunnels is distinguished by complex geological structures. These formations are made up of different kinds of fault zones, geological features, and rock strata. The study of Heidarzadeh et al.'s (2021) shows that the area's complicated geology poses difficulties for stability evaluations and excavation techniques. It is important to accurately characterize geological conditions using empirical values because this has a direct impact on tunnel design, building methods, and long-term performance. Substantial differences in the qualities of the rock mass, such as its strength, deformation features, and fracture density, are frequently shown by geological data. Stress concentrations and differential tunnel deformation may result from these features' non-uniformity inside the tunnel alignment.

Due to these differences, an in-depth understanding of the experimental information unique to the Diamer Basha region is required in order to reduce the likelihood of engineering difficulties and improve the accuracy of tunnel modeling. The hydrological dynamics of the area are intimately linked to the diversion tunnels' activity. Important elements affecting tunnel performance are groundwater interactions, pressure gradients, and water flow rates. The study conducted by Lv et al. (2020), exhibits historical flow rate data was important in anticipating how the diversion tunnels would react to shifting hydrological circumstances. Optimizing the tunnels' design and operation requires that the actual values for these hydrological parameters be accurately represented in the numerical models. Rockfalls and landslides are two geohazards that can occur in the Diamer Basha area. These occurrences might seriously compromise the operating safety and integrity of the tunnel. Real information about historical geohazard events and their effects must be included in modeling to properly estimate and manage risk. Accurate modeling ought to consider the possible outcomes of these geohazards as well as the effectiveness of mitigating actions.

2.9 Empirical Methods of Design

Using statistical analysis of subsurface measurements, such as engineering rock mass classifications, these techniques determine whether or not a mine or tunnel is safe to enter. The empirical method compares what has been learned from past initiatives to what is expected at a future location. An empirical design may make better use of the invaluable practical knowledge obtained at numerous projects if it is supported by a systematic approach to ground categorization. The role of the design engineers is not to calculate exactly but to assess soundly, and this is especially significant since a good engineering design is a balanced design in which all the aspects which interact are taken into consideration, even those which cannot be measured. The fundamental component of empirical design methodologies, rock mass classifications are widely employed for rock tunnels.

At the moment, a system of organisation is used in the majority of tunnels dug in the USA. The most popular categorization system is the Terzaghi system, which was first proposed almost 40 years ago. In reality, the use of rock mass classifications has been widely adopted over the globe with great success (Bieniawski, 1990). Other engineering evaluation and design approaches may be utilised in tandem with the empirical methods of design (Ali, 2014). In the early phases of a project, when only limited data on the behaviour of rock mass, stress conditions, and hydrological parameters are available, these methodologies are crucial and helpful for the design.

2.10 Rock Mass Classification Systems

Designing excavations using a rock mass categorization method is an example of an Empirical technique. Thus, it is best described as a "trial-and-error" process. Since Ritter (1879) sought to formalise an empirical approach to tunnel construction, particularly for estimating support needs, there has been an evolution in the methods used to categorise rock masses. The most common and widely-used empirical approaches to the building of rock mass categorization systems RMR, QSystem, RQD, RSR, and GSI are all methods for categorising rock masses. The following are some of the reasons why rock mass categorization techniques are used in tunnel construction (Bieniawski, 1990):

- Clusters of rocks exhibiting the same characteristics
- Helps us grasp the foundational features of discrete communities.
- Provides quantitative data for the design of intricate engineering challenges; facilitates rock excavation planning and design.
- An agreed-upon plan for everyone involved in the project to learn and grow together.

Terzaghi (1946), Laufffer (1958), Deere (1964), Wickham, Tiedemann, and Skinner (1972), Bieniawski (1973), and Barton, Lien, and Lunde (1974) are only few of the authors who have developed several methods for classifying rock masses (Bieniawski, 1990).

Rock Mass Classification System	Originator	Country of Origin	Application Areas
Rock Load	Terzaghi, 1946	USA	Tunnels with steel Support
Stand-up time	Lauffer, 1958	Australia	Tunneling
New Austrian Tunneling Method (NATM)	Pacher et al., 1964	Austria	Tunneling
Rock Quality Designation (RQD)	Deere et al., 1967	USA	Core logging, tunneling
Rock Structure Rating (RSR)	Wickham et al., 1972	USA	Tunneling
Rock Mass Rating (RMR)	Bieniawski, 1973 (last modification 1989-USA)	South Africa	Tunnels, mines, (slopes, foundations)
Modified Rock Mass Rating (M-RMR)	Ünal and Özkan, 1990	Turkey	Mining
Rock Mass Quality (Q)	Barton et al., 1974 (last modification 2002)	Norway	Tunnels, mines, foundations
Strength-Block size	Franklin, 1975	Canada	Tunneling
Basic Geotechnical Classification	ISRM, 1981	International	General
Rock Mass Strength (RMS)	Stille et al., 1982	Sweden	Metal mining
Unified Rock Mass Classification System (URCS)	Williamson, 1984	USA	General
Communication Weakening Coefficient System (WCS)	Singh, 1986	India	Coal mining
Rock Mass Index (RMi)	Palmström, 1996	Sweden	Tunneling
Geological Strength Index (GSI)	Hoek and Brown, 1997	Canada	All underground excavations

Table 1 most widely used classification system

2.10.1 Objectives of rock mass classification system

Figure out what Factors significantly affect how a rock mass behaves.

Describe the distinguishing features of various types of rock masses.

Make connection between the rock conditions you have seen in one location and those.

2.10.2 Benefits of rock mass classifications

Insisting on even the barest minimum of input data as classification criteria has the potential to vastly improve the quality of site investigation.

Quantitative data for design consideration.

Facilating more informed engineering decisions and clearer project wide dialogues'.

2.11 List of Rock Mass classifications

Different classification systems place different emphases on the various parameters, and it is recommended that at least two methods be used at any site during the early stages of a project:

Rock Quality Designation index (RQD)

Rock Structure Rating (RSR)

Geomechanics Classification or the Rock Mass Rating (RMR) system

Tunnelling Quality Index (Q system)

Terzaghi's rock mass classification

2.11.1 Rock Quality Designation Index (RQD)

Deere created it with his colleagues in 1967.This method uses data from drill cores to provide quantitative assessments of rock composition. If the core is NX size (54mm in diameter), RQD is the percentage of the overall length that is made up of complete pieces with a length more than 10cm. Methods for accurately gauging core sample sizes and assigning a rough Rock Quality Designation.

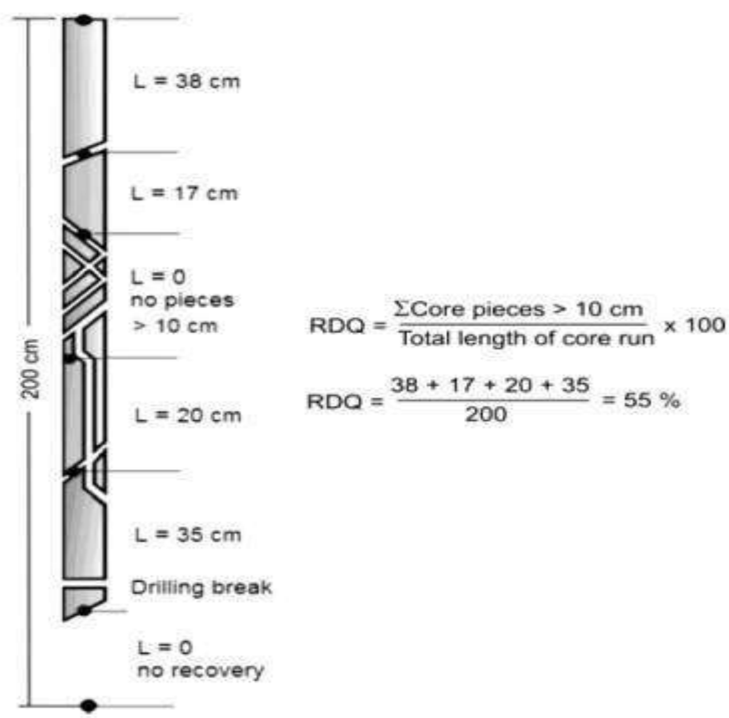


Figure 3 Procedure for measurement and calculation of RQD

In 1982, Plastron proposed that the RQD might be determined from the number of discontinuities per unit volume in cases when core was unavailable but discontinuity traces were apparent in surface disclosure or exploration adits. Equation 2.2 provides the proposed connection for clay-free masses.

RQD = 115-3.33 Jv ----------------Equation (2.2)

Where RQD represents the Rock Quality Designation Index and Jv represents the volumetric joint count of all joints per unit length across all joint (discontinuity) sets.

2.11.2 Rock Structure Rating (RSR)

Wickham et al (1972) proposed a quantitative approach for characterising the quality of a rock mass and for choosing adequate support on the basis of their Rock Structure Rating (RSR) classification.

The RSR system's innovation was to provide a numerical value to each of the following factors based on their ratings (Design methods, 2019):

RSR = A + B + C.

a. Geology Parameter:

Overall evaluation of geological structure based on: Igneous, Metamorphic, and Sedimentary Rock Provenance. Decomposition, medium hardness, hardness, and softness of rocks. Structure of the Earth's underlying rock (huge, moderately faulted/folded, severely faulted/folded, and barely faulted/folded).

:

	Basic Rock Type				Geological Structure			
	Hard	Medium	Soft	Decomposed	Massive	Slightly Folded or Faulted	Moderately Folded or Faulted	Intensively Folded or Faulted
Igneous	1	2	3	4				
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4				
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

Table 2 Geology

Condition B: Geometry Based on the effect of the discontinuity pattern on the tunnel's driving direction:

- Joint separation.
- Strike and dip are types of joint alignment.
- Tunnelling Direction

Average joint spacing	Strike ⊥ to Axis					Strike || to Axis		
	Direction of Drive					Direction of Drive		
	Both	With Dip		Against Dip		Either direction		
	Dip of Prominent Joints [a]					Dip of Prominent Joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed, < 2 in	9	11	13	10	12	9	9	7
2. Closely jointed, 2-6 in	13	16	19	15	17	14	14	11
3. Moderately jointed, 6-12 in	23	24	28	19	22	23	23	19
4. Moderate to blocky, 1-2 ft	30	32	36	25	28	30	28	24
5. Blocky to massive, 2-4 ft	36	38	40	33	35	36	24	28
6. Massive, > 4 ft	40	43	45	37	40	40	38	34

Table 3 Geometry

C Parameter: Joint Condition and Groundwater Inflow Based on:

- Overall rock mass quality on the basis of A and B combined.
- Classification of joint health: excellent, mediocre, or bad.
- Quantity of water entering the tunnel (in gallons per minute for each one thousand feet).

Anticipated water inflow gpm/1000 ft of tunnel	Sum of Parameters *A* + *B*					
	13 - 44			45 - 75		
	Joint Condition [b]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight, < 200 gpm	19	15	9	23	19	14
Moderate, 200-1000 gpm	15	22	7	21	16	12
Heavy, > 1000 gp	10	8	6	18	14	10

Table 4 Sum of Parameters

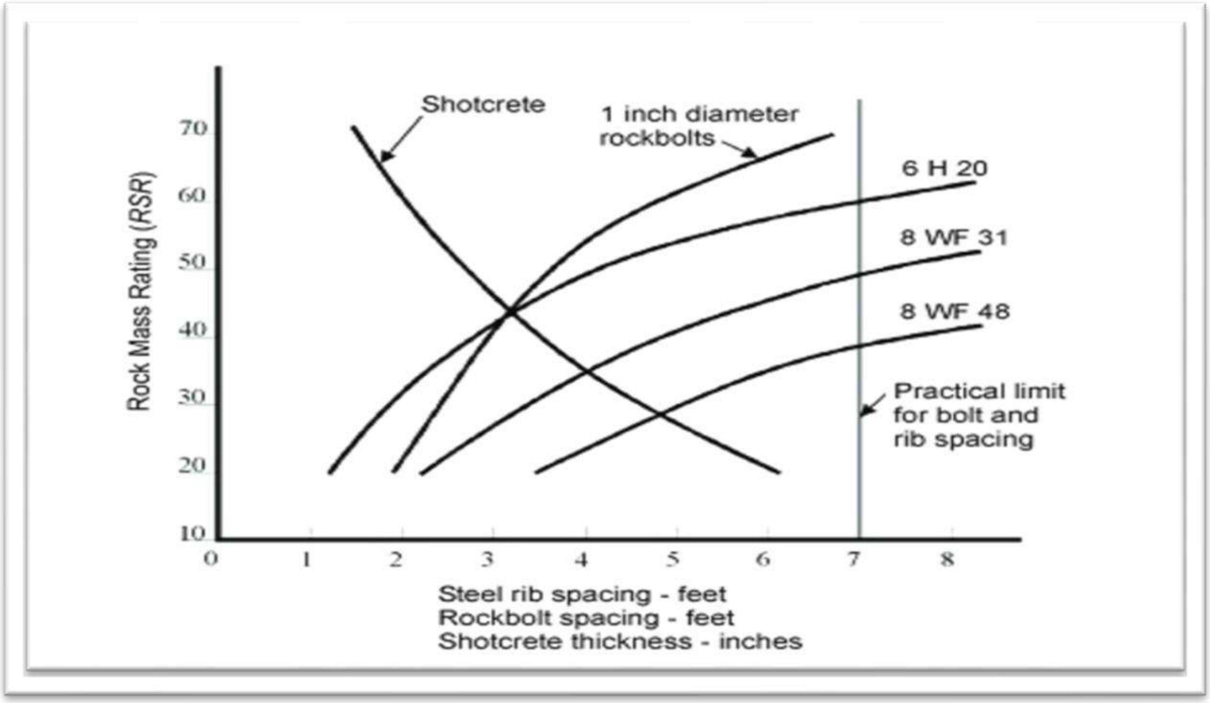


Figure 4 RSR Support estimation

2.11.3 Rock Mass rating system (RMR System)

Biniawski developed the rock mass rating system in 1976; it is also known as the geomechanics categorization system. It was created with the unique case studies in structural design in mind. Changes were made to the system in 1974, 1976, 1979, and 1989 as new contextual analyses were recognised in relation to tunnels, mines, chambers, slopes, and foundations (Z.T.Biniawski, 1989). The Geo-mechanics classification system has a comprehensive applicability in many rock engineering disciplines such as mining, hydro power projects, tunnelling and hill slope stability. (Kumar S. S., 2012). Six parameters calculable on-site and from cores are included into the geomechanics classification (Ali, 2014).

- Rock quality index (RQI) is a measure of the rock's uniaxial compressive strength.
- Discontinuity Spacing
- Existence of breaks in continuity
- Water table status
- Distribution of breaks in continuity

Using this technique, geologists may categorise rock masses into different structural zones. The regions are separated into their own categories (HOEK, 2016).Table 4 below illustrates how these six criteria are being evaluated with respect to a variety of geological and geotechnical factors. The project site's suggested support systems are determined by the RMR score, which is based on the aforementioned criteria. The table below provides the recommended support depending on the RMR value.

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values				
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa 1 - 5 MPa < 1 MPa
	Rating		15	12	7	4	2 1 0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%
	Rating		20	17	13	8	3
3	Spacing of		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm
	Rating		20	15	10	8	5
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous
	Rating		30	25	20	10	0
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al (1972).

Table 5 Rock mass Rating System.

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock RMR: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock RMR: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock RMR: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock RMR: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock RMR: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Table 6 Guidelines for excavation and support of 10 m span rock tunnels in accordance with RMR system

The six rock mass metrics listed below are used in the Bieniawski engineering categorization system, which was created in 1973.

1. The intact rock's uniaxial compressive strength.
2. The RQD, or rock quality designation.
3. Discontinuity spacing.
4. The existence of discontinuities, as indicated by 4a Length and Persistence
   - Separation 4b
   - Smoothness, 4c
   - Fourth Infill
   - Change or weathering (4e)

4) The state of the groundwater.
5) Discontinuity orientation.

These can all be measured in the field and also found in borehole data. Each of these characteristics' ratings is added up to produce the RMR value. Every parameter is quantifiable in the field, and some of them can also be found in borehole data (Bieniawski, 1993 #24).

2.11.4 Q-System

The Norwegian Geotechnical Institute (NGI) developed this method of rock mass categorization based on 212 case histories in order to facilitate the building of tunnels. (Barton, Lien, & Lunde, 1974). The rock mass classification method is one of the best classification systems for tunnel design (Kumar, 2002), having been employed in roughly 1260 different projects throughout the globe. The highest and lowest Q-System ratings indicate the highest and lowest possible rock quality, respectively. Q-index is measured on a logarithmic scale, with a minimum value of 0.001 and a maximum value of 10000. In this taxonomy, Q is defined as a function of six free parameters using equation 2.3.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad \text{---(2.3)}$$

Where RQD is the Rock Quality Designation index, Jn is the Joint Set Number, Jr is the Estimated Joint Roughness for the Worst Joint Set in the Tunnel, and Ja is the Estimated Joint Alteration Number for the Worst Joint Set in the Tunnel. SRF is made up of people who think about how in-situ stress conditions affect Rock's overall quality. Barton et al. (1974) give the following clarifications to help you understand the characteristics that are taken into account while determining Q's worth. The first quotient (RQD/Jn) provides an approximate estimate of the block size and reveals the structure of the rock mass. The roughness and slickness of the joint walls or infill materials are conveyed by the Second quotient (Jr/Ja). This action is done in favour of direct interaction between joints that are unequal and unmodified. When thin clay mineral coatings and fillings are present in rock joints, the strength is drastically diminished. It characterises the rock mass's inter-block shear strength.

The third quotient (Jw/SRF) includes two characteristics associated with stress. Rock force (RF) is the sum of three components: (1) untying load; (2) rock stress; and (3) squeezing loads in plastic weak rock masses, as experienced by an excavation as it moves through clay-bearing rock and shear zones. It's also useful as a metric of overall stress. The shear strength of joints is negatively impacted by the Jw parameter, which measures the amount of water pressure, since it decreases the effective normal stress. Clay infilling the joints might soften in the presence of water, which can lead to outwash. The active stress component is often shown, and this value is usually established by trial and error. Tables 7, 8, 9, 10, 11, and 12 provide a thorough and detailed system for determining the values of the Q-System parameters (Rock quality designation [RQD], Number of joints [Jn], Roughness number for joint [Jr], Joint alteration [Ja], Joint water [Jw], Surface reduction factor [SRF]). A high value indicates high quality rock, whereas a low one indicates low quality.

1	Rock quality designation (RQD)		RQD
A	Very poor	>27 joints per m^3	0-25
B	Poor	20-27 joints per m^3	25-50
C	Fair	13-19 joints per m^3	50-75
D	Good	8-12 joints per m^3	75-90
E	Excellent	0-7 joints per m^3	90-100
Note: i) Where RQD is reported as ≤ 10 (including zero) the value 10 is used to assess the Q-value ii) RQD-intervals of 5 are adequately accurate			

Table 7 Rock quality designation (RQD) and volumetric jointing

2	Jn values	Jn
A	Massive, no or few joints	0.5-0.1
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four joint sets, random, heavily jointed, " sugar cube ", etc.	15
I	Crushed rock, earth like	20
Note: i) For tunnel intersection, use 3*Jn ii) For portals, use 2*Jn		

Table 8 Joint set number (Jn) Value

3	Jr values	Jr
a) Rock- wall contact and b) Rock-wall contact before 10 cm shear movement		
A	Discontinuous joints	4
B	Rough or irregular undulating	3
C	Smooth undulating	2
D	Slickensides, undulating	1.5
E	Rough irregular planar	1.5
F	Smooth planar	1
G	Slickensides planar	0.5
Note: i) description refer to small scale features and intermediate scale features, in that order		
c) No-rock wall contact when sheared		
H	Zones containing clay minerals thick enough to prevent rock wall contact	1
I	Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1
Note: ii) 1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. iii) Jr = 0.5 can be used for planar, slickensides joints having lineation, provided that the lineation are oriented for minimum strength.		

Table 9 Joint roughness number (Jr)

4 Ja values		φ_r approx.	Ja
a) Rock-wall contact (no filling, just coatings)			
A	Hard impermeable filling firmly healed hard such as epidolite/quartz		0.75
B	Only surface staining with unaffected joint walls.	25-35°	1
C	A little altered joint-walls with Non-softening mineral coatings; sandy particles/ clay free fractured rock, etc.	25-30°	2
D	Silty/sandy clay coatings. Small clay fraction.	20-25°	3
E	Mineral coatings with clay of low friction, such as Mica/Kaolinite etc.	8-16°	4
b) Rock-wall contact before 10 cm shear with a slim mineral filling			
F	Clay-free fragmented rock, sandy particles	25-30°	4
G	Strongly over-consolidated, non-softening, clay mineral fillings (less than 5mm Continuous thickness).	16-24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (less than 5mm continuous thickness).	12-16°	8
I	Swilling clay fillings, i.e., montmorillonite (less than 5mm continuous thickness).	6-12°	8-12
c) No rock-wall contact due to thick mineral filling even after shear			
J	Zones or bands of crushed rock. Medium or low over-consolidation.	16-24°	6
K	Zones of clay, disintegrated rock. Medium or low over-consolidation.	12-16°	8
L	Zones of clay, disintegrated rock. Joint alteration depends on the percentage of swelling clay-size particles.	6-12°	8-12
M	Thick continuous zones of clay or band of clay. Strongly over consolidated	12-16°	10
N	Thick continuous zones of clay. Joint alteration depends on the percentage of welling clay-size particles.	12-16°	13
O	Thick and continuous clay zones. Joint alteration depends on the percentage of swelling clay-size particles.	6-12°	13-20

Table 10 Joint alteration (Ja) values

5 Jw values		Jw
A	Dry excavation or minor inflow (humid or a few drips)	1.0
B	Medium inflow, infrequent outwash of joint filling (many drips/"rain")	0.66
C	Jet inflow or higher pressure in competent rock with unfilled joints	0.5
D	Large inflow or higher pressure, considerable outwash of joint fillings	0.33
E	Exceptionally high inflow continuing without perceptible decay. Causes outwash of material and possibly cave in	0.2-0.1
F	Exceptionally high inflow continuing without perceptible decay. Causes outwash of material and possibly cave in	0.1-0.05

Table 11 Joint water reduction factor (Jw) values

6	SRF values			SRF
	a) Weak zones crossing the underground excavation, which may cause loosening of rock mass			
A	Multiple occurrences of weak zones within a short section containing clay or chemically disturbed very loose surrounding rock at any depth, or long section with incompetent rock.			10
B	Multiple shear zones within a short section in competent clay-free rock with weak surrounding rock at any depth.			7.5
C	Single weak zone with or without clay or chemical disintegrated rock with depth less than or equal to 50m.			5
D	Loose, open joints, heavily jointed at any depth			5
E	Single weak zones with or without clay or chemical disintegrated rock with depth greater than 50m			2.5
	Note: i) Reduce these values of SRF by 25-50% if the weak zones but do not intersect the underground opening			
	b) Competent massive rock with stress problems	$\sigma c / \sigma 1$	$\sigma \Theta / \sigma c$	SRF
F	Low stress, near surface, open joints	>200	<0.01	2.5
G	Medium stress, favorable stress condition	200-10	0.01- 0.3	1
H	High stress, very tight structure. Usually good for stability. Depending on stress orientation it may be unfavorable to stability.	10-5	0.3-0.4	0.5-2 2-5*
I	Moderate spalling land/slabbing after greater than one hour in massive rock	5-3	0.5- 0.65	5-50
J	Spalling or rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
K	Heavy rock burst and instant active deformation in massive rock	<2	>1	200-400
	Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma 1 / \sigma 3 \leq 10$ reduce σc to 0.8 σc, and $\sigma \Theta$ to 0.8 $\sigma \Theta$, when $\sigma 1 / \sigma 3 > 10$ reduce σc to 0.5 σc, and $\sigma \Theta$ to 0.5 $\sigma \Theta$. iii) Few case records available where depth of crown below surface is less than span width Suggest SRF increase from 2.5 to 5 for such cases (see H).			
	c) Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure		$\sigma \Theta / \sigma c$	SRF
L	Mild squeezing rock pressure		1-5	5-10
M	Heavy squeezing rock pressure		>5	10-20
	d) Swelling rock: chemical swelling activity depending on the presence of water			SRF
N	Mild swelling rock pressure			5-10
O	Heavy swelling rock pressure			10-15

Table 12 Stress reduction factor (SRF) values

The Q-system value is calculated from the results obtained for the various parameters using the aforementioned tables. Bortan et al. (1974) divide rock quality into nine categories, as illustrated in table 12, based on the Value of Q System.

Q-System values range	Group	Classes of rock mass
0.001 – 0.01	3	Exceptionally Poor
0.01 – 0.1		Extremely Poor
0.1 – 1	2	Very Poor
1 – 4		Poor
4 – 10		Fair
10 – 40	1	Good
40 – 100		Very Good
100 – 400		Extremely Good
400 - 1000		Exceptionally Good

Table 13 Rock mass classification on the basis of Q-system

The practise of estimating the values of the parameters for this method of categorization requires expert manipulators. This classification system's shortcoming is that inexperienced professionals may have difficulty estimating the parameters' scores, leading them to estimate a lower value for the Q-System (D. Milne, 1998). The kind of rock mass under the surface is a major factor in determining the underground excavation's breadth and height. When the width or height is increased or decreased, it immediately affects the stability. For further emphasis on the duty to ensure safety, Bortan et al. propose a new metric for Q-System called the excavation support ratio (ESR). The lower value of ESR symbolises the requirement of large level firmness and vice versa. The ESR is used in estimating the stability-maintaining system that can be installed, in conjunction with the projected usage of excavation. Table 2.14 compiles a variety of scenarios in which ESR values may be found. By using the following expression (NGI), 2019), ESR demonstrates the Equivalent dimension based on the width and depth of the subterranean excavation.

De = (width or altitude in m)/ESR

7	**Excavation types**	**ESR values**
A	Temporary mine openings	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high Pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge Chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, Portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public Facilities, factories.	0.8

Table 14 Excavation support ratio (ESR)

Figure 3 depicts the suggested support chart by Bortan et al. for subterranean excavations, which is based on the Q-system ratings and equivalent dimension. This table gives the energy absorption of fibre reinforced sprayed concrete and a general framework based on empirical data for deciding what sort of support system is advised for various combinations of rock bolt centre to centre spacing and sprayed concrete thickness.

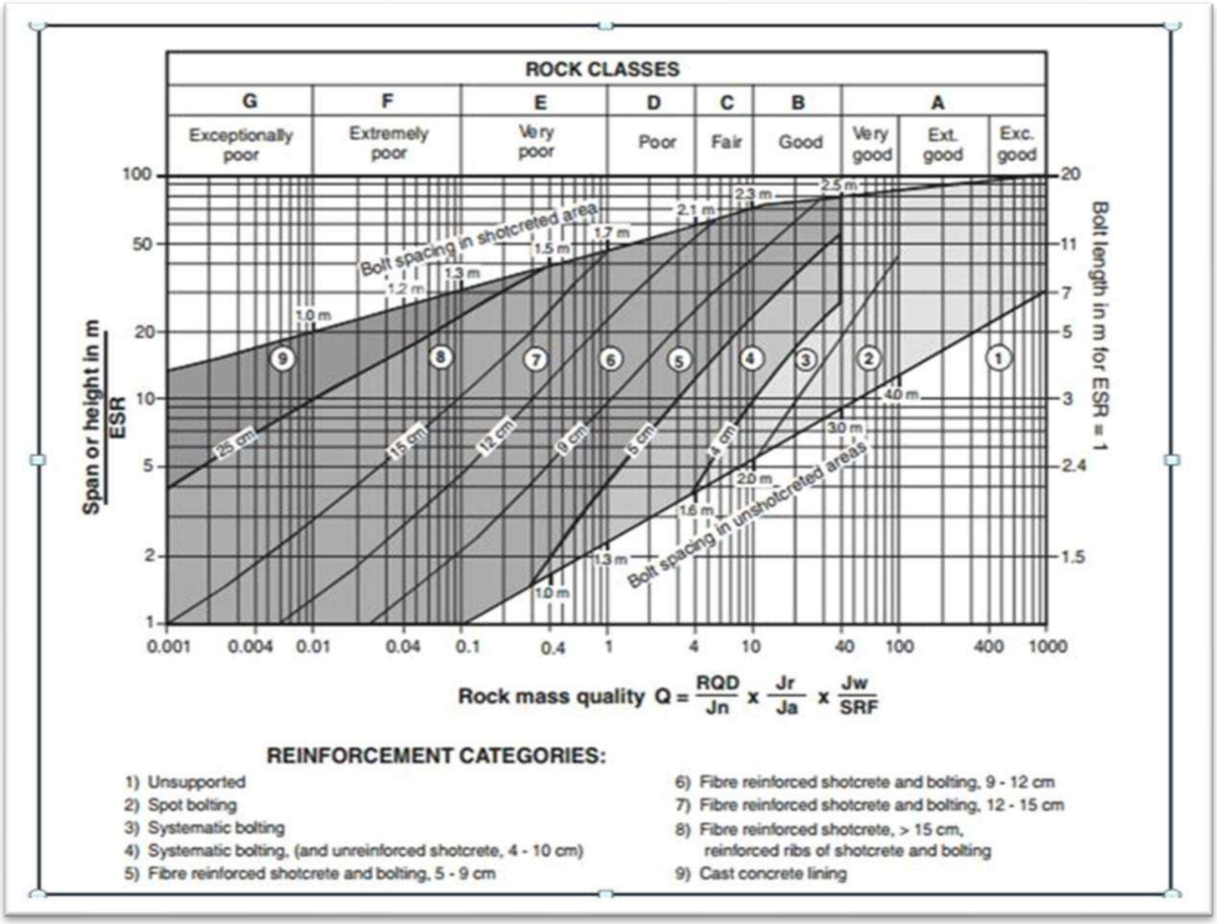


Figure 5 Permanent support system recommendation chart for Q-system

A characterization tool for assessing engineering properties of rock mass (2019) describes the Geological Strength Index (GSI) as a system of rock-mass characterization developed in engineering rock mechanics to meet the need for reliable input data related to rock-mass proper-ties required as input for numerical analysis or closed form solutions in the design of tunnels, slopes, or foundations. For input data into the continuum numerical analysis codes and closed form solutions based on the Hoek-Brown failure criterion (see, for example, Marinos & Hoek (2000) and Marinos et al. (2007)), the Geological Strength Index (GSI) is currently the most popular engineering index for classifying rock mass quality. Accurately calculating the failure envelope or the deformation moduli of the rock mass relies heavily on this number.

Figure 6 Geological strength index chart

The comprehensive practice for estimation of input parameters for numerical analysis of stress condition and the remedial measures is presented in figure 2.6 (Hoek, 2013).

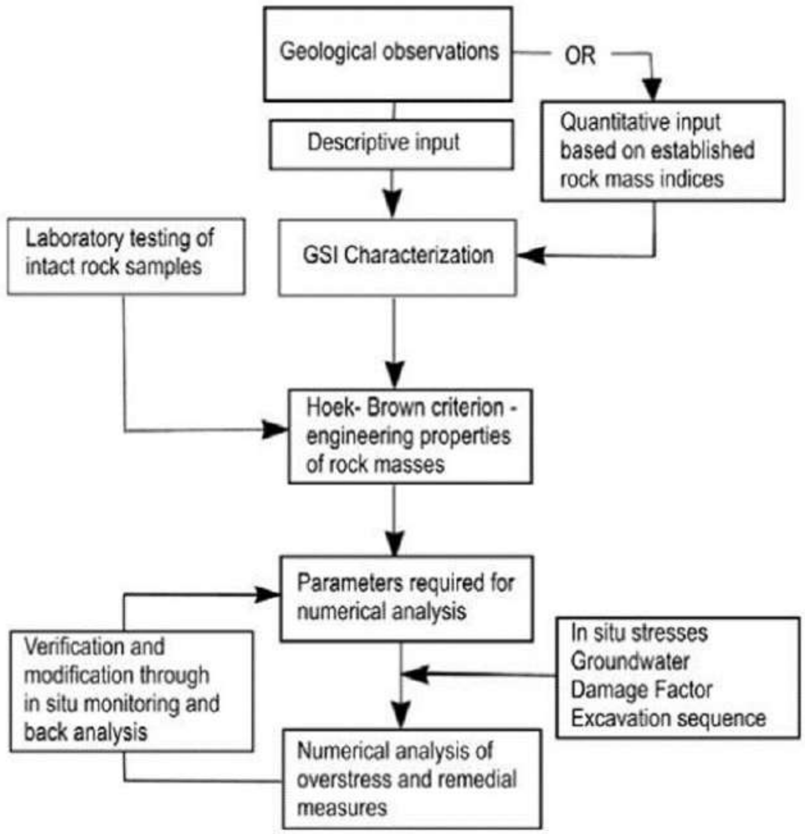


Figure 7 GSI and Hoek and Brown failure criteria for estimation of input parameters for numerical investigation

Following additional rock mass evaluation techniques, the GSI index may be determined.

1. Method A: Skilled geologists or mining engineers use data obtained (observational data) onsite to estimate GSI, which is then reviewed using a chart to determine its value (Mahmoud Hashemi, 2010).
2. When only partial data is available, we may use Method B, in which the GSI index is approximated using other categorization methods such as RQD and RMR etc. Some examples of well-established associations from which the GSI may be estimated are provided below (Mahmoud Hashemi, 2010) and (Hoek, 2013).

Method B: Sonmez and Ulusay approximated the GSI value using ratings for the structure and the surface, respectively (Harun Sonmez, 2002). Approximating the GSI using block volume (Vb) and joint surface condition factor (Jc) was done by Cai et al. (2004). Block volume with the highest joint set count is:

$$Vb = S1 * S2 * S3 \qquad \text{where S is joint spacing}$$

Joint spacing is denoted by S. The formula for calculating the joint surface condition factor from joint roughness, weathering, and infilling is as follows:

$$Jc = Jw * Js/Ja$$

To properly measure the GSI value, we employ the Vb and Jc (Mahmoud Hashemi, 2010). The quantitative chart for estimate of GSI given by sonmez and Ulusay is displayed in Figure 5.

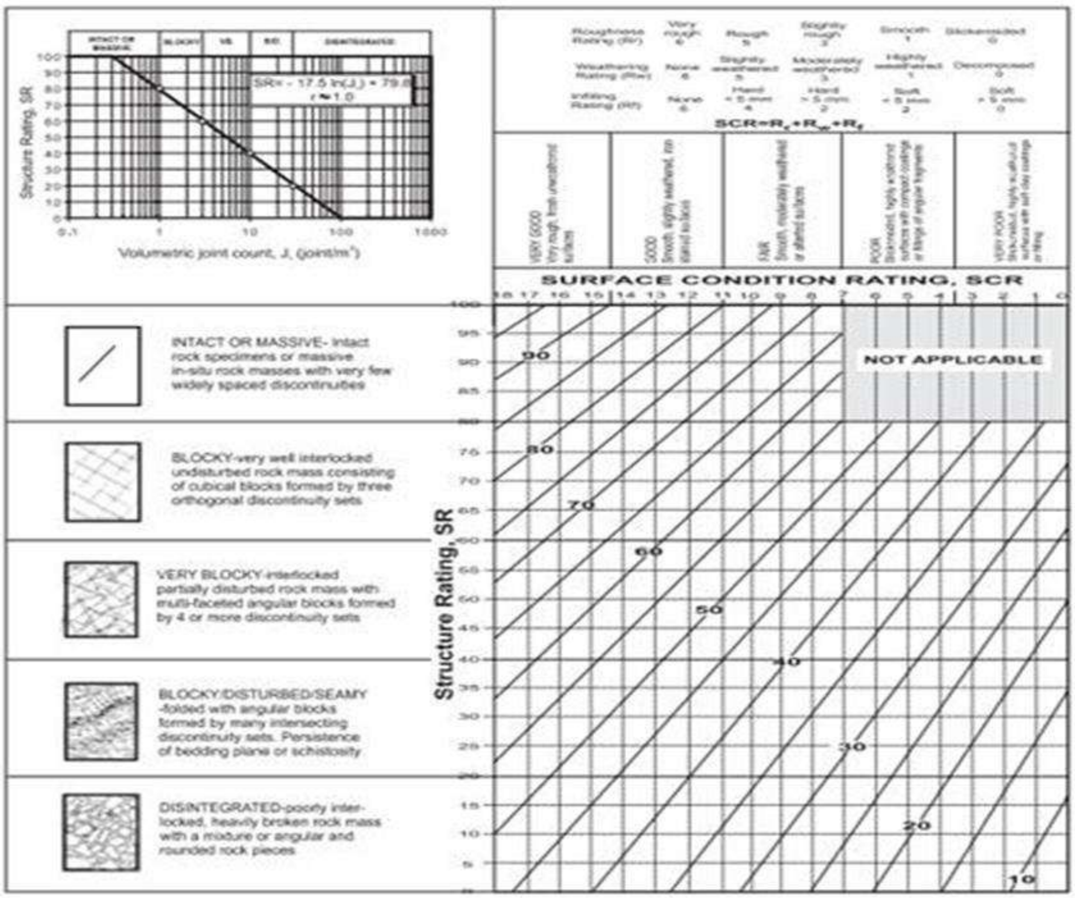


Figure 8 quantitatively estimation of GSI chart

2.12 Neelum Jhelum tunnel project

The Water and Power Development Authority of Pakistan recently completed the NJHEP, a hydroelectric project that is situated in the Muzaffarabad region of Kashmir in northeastern Pakistan. The first turbine began producing power in April 2018 after construction started in late 2008. The project is intended to produce 969 MW of electricity from 283 m3/s of water, with a gross hydraulic head of 420 m. This water is redirected from the Neelum River to the Jhelum River through tunnels. A diversion dam, an intake system, headrace tunnels, an underground powerhouse complex, and a tailrace tunnel are among the project's principal buildings. More than 80% of the construction involved was underground excavation work.

The single (34%) and twin (66%) circular and horseshoe-shaped tunnels that make up the 28.6 km long headrace tunnels. The cross-section of the excavation for the single and twin headrace tunnels, which are oriented northeast-southwest, respectively, spans ranges of 10.7-11.8 m and 7.758.53 m. 0.9 km from the intake portal, a single tunnel with a modified horseshoe-shaped crosssectional area and a hydraulic span of 9.6 m is divided into twin headrace tunnels (Rehman, 2021).

The occurrence of intense rockburst in deep tunnels is inevitable when geological structures are present in deep massive rock mass, and it normally has dynamic characteristics. In such conditions, the stability of the underground excavation is critical. In previous work, static numerical modeling using FLAC was done to find only the influence of the shear zone on rockburst. The dynamic effect of the shear zone on rockburst occurrence both near the boundary of a tunnel and on its support system is still unclear. In this paper, a FLAC 2D dynamic numerical modeling has been done to study the mechanism of a rockburst at great depth. The actual field measured parameters have been used during simulation. It is believed that the most intense rockburst event of 31 May 2015 in NJHEP was due to a slip along a shear zone, which resulted in severe damage to the excavation boundary due to its dynamic impact. We have numerically investigated the mechanism of rockburst in the headrace tunnels of the NJHEP, which have been subjected to dynamic loading, and we have also evaluated the dynamic impact of rockburst on the installed support in the adjacent tunnel.

Rockburst is a dynamic phenomenon that involves the unstable failure of rock. The risk of severe rockburst is high when a geological structural plane is present near any tunnel along with change in equilibrium status of the area due to tunnel excavations. Different numerical studies have been done to explain the effect of these structures on rockburst occurrence. Zhang et al. [35] numerically evaluated the failure of a rock mass in the Jinping-II hydropower station. The blocking effect of the fault caused intense stress concentration, which resulted in increased shear strain energy near the fault which, in turn, caused severe seismic activity and energy release. Zhang et al. [36] have used the failure approach index (FAI) during numerical simulation and determined that structural planes led to local stress concentration that caused rock mass failure in the hanging wall. In the current study, the FLAC 2D numerical simulation has been used to evaluated the influence of the shear zone on tunnel stability, and its possible effect on the rockburst failure mechanism around Tunnel 696 and on the support system in the adjacent Tunnel 697 subjected to static and dynamic loading. (Naji, A.M; 2019)

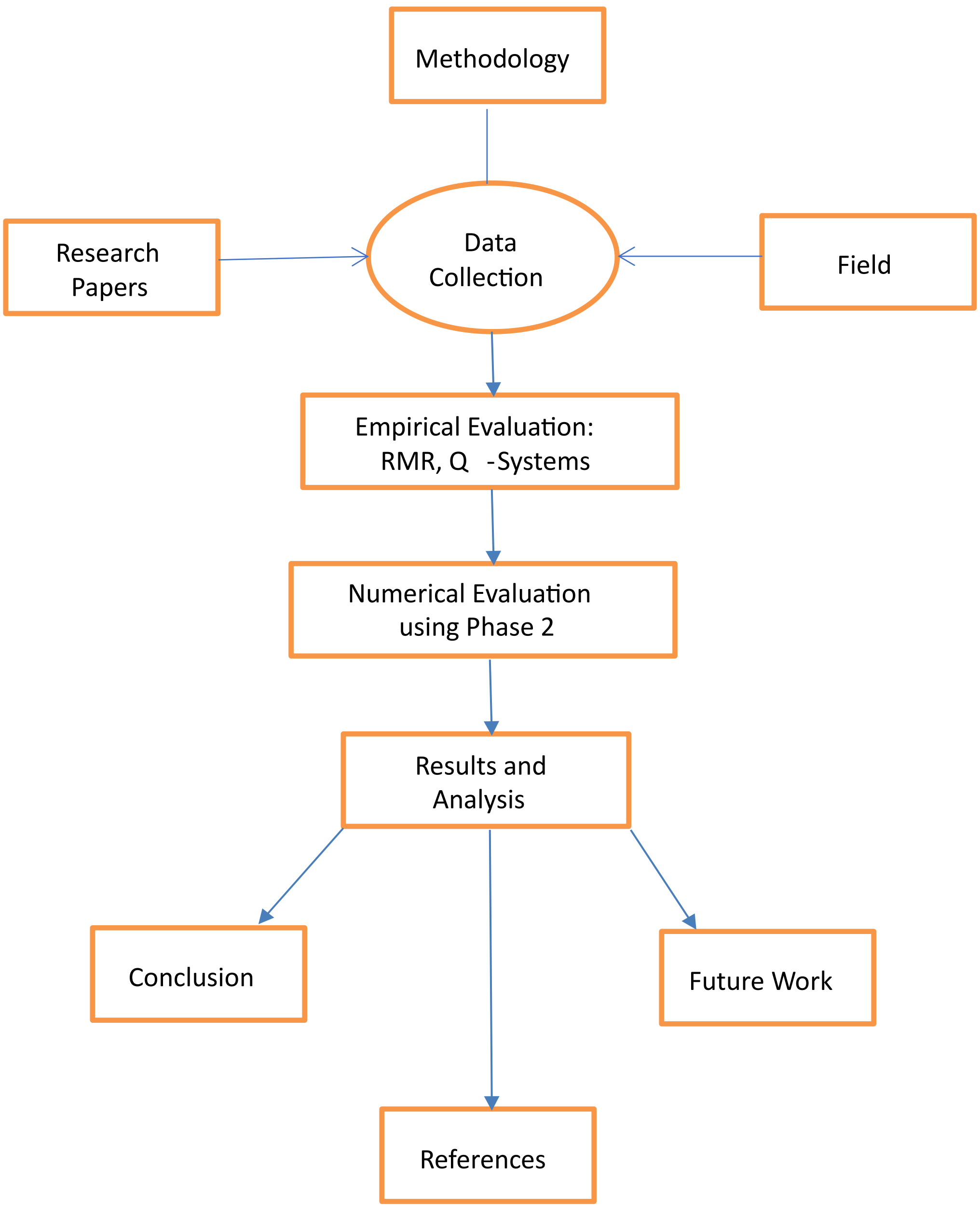
CHAPTER 3
METHODOLOGY

Methodology

3.1 Introduction

This section describes the methodology adopted to achieve the above objectives. Numerical modeling of diamer basha diversion tunnels with the help of phase two software. Support installation process is done on the bases of empirical values RMR and Q support Chart, while 2D finite element program for calculating stresses and estimating support around underground excavations.

3.2 Flow Chart



3.3 Sequential Excavation Method

The proposed tunnel location is first divided into segments, which are then mined using an Excavator and road header in a sequential manner by providing supports. As soil from each section is removed, pressurized concrete known as shotcrete is sprayed on the ceiling, walls, and sides. Additional structural support is provided by installing lattice girders. In sequential excavation method we divide tunnel face into four steps.

3.3.1 Stress Calculation

The following formula may be used to calculate the stress for underground excavations, represented by:

$$\sigma = A/F \quad \text{Where:}$$

F: The pressure or weight that has been placed on the excavation.

A: The excavation's cross-sectional area.

3.3.2 Sequential Excavation Method

The required volume of shotcrete, V, for each excavated section of the tunnel is determined by: $V=A\times t$ Where:

The area that corresponds to the excavated section is designated by the letter A.

The applied shotcrete layer's thickness, denoted by the letter t.

3.4 Step No. 1

The first step is to do project setting.

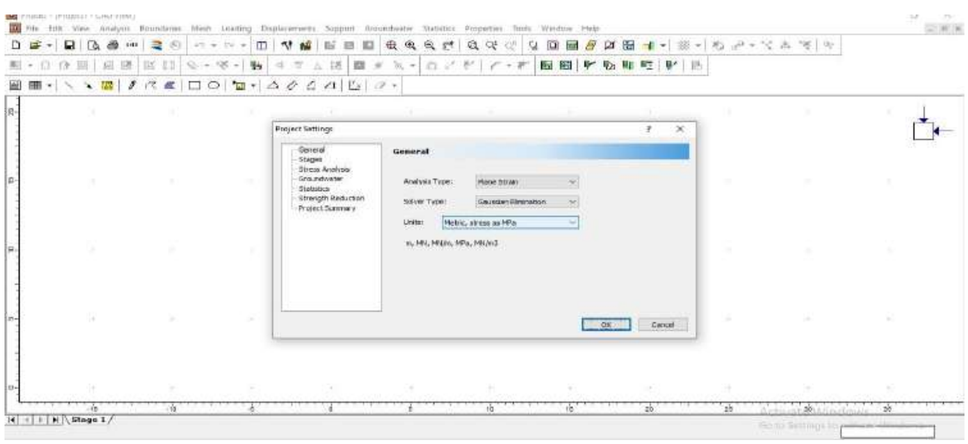


Figure 9 Project setting

3.4.1 Step No. 2

Now I will draw two D shape tunnel with center to center distance of 50m.

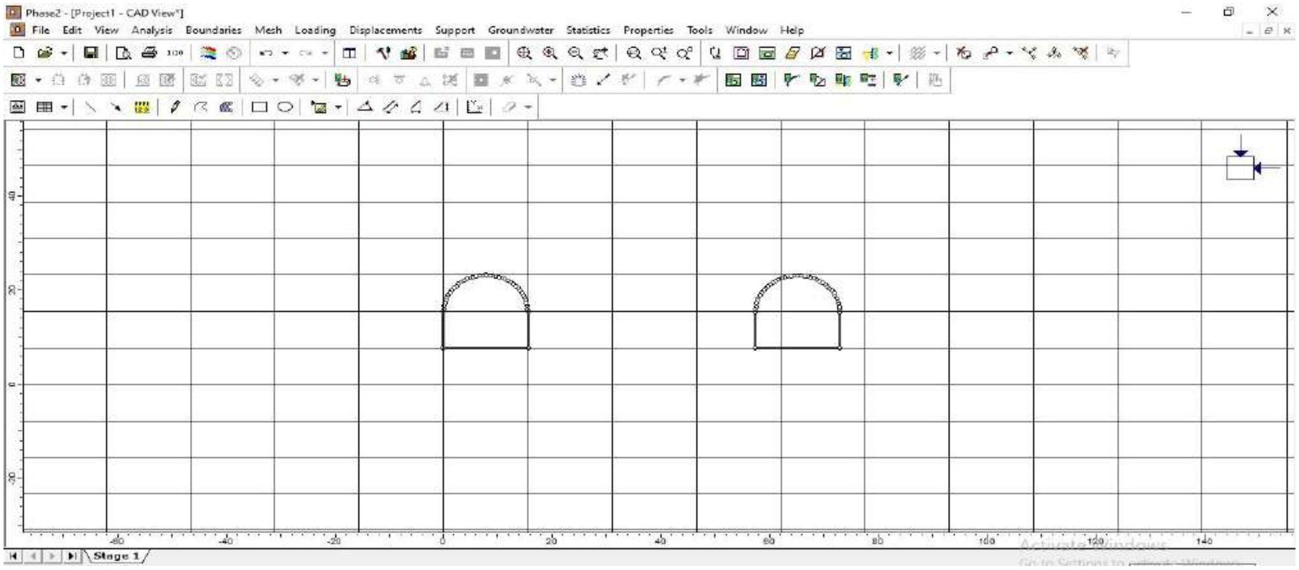


Figure 10 Using grid to draw the model.

3.4.2 Step No. 3

Now I am going to select the stages of excavation.

Stage 1: it's the inside condition.

Stage 2: it is the stage of heading excavation.

Stage 3: it's the stage of bench excavation.

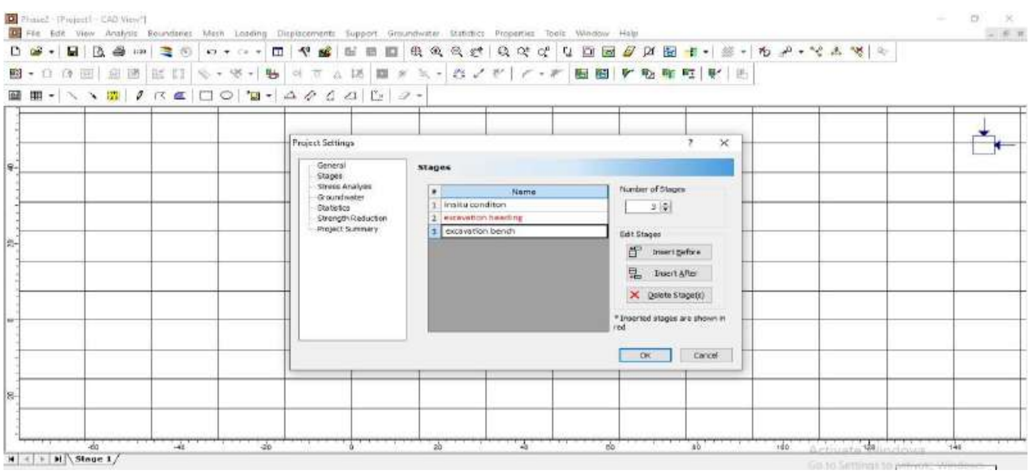


Figure 11 Applying stage boundaries

3.4.3 Step No. 4

All the values related to material properties mainly gabbronorite as mention in the above tables 4 and 5 should be implemented on the phase two software.

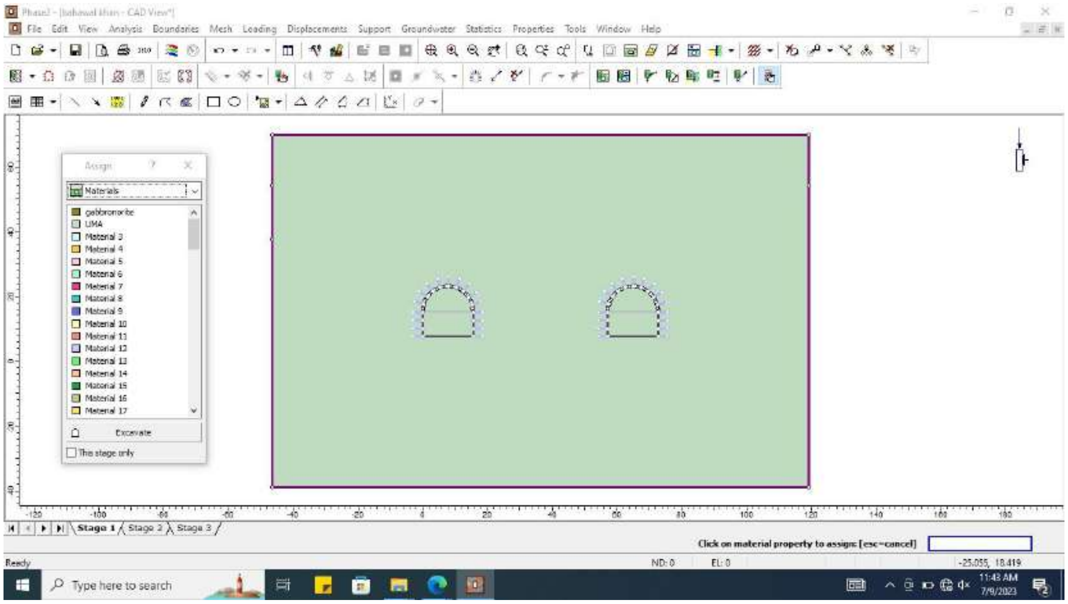


Figure 12 Assigning material properties

3.4.4 Step No. 5

The following factors may be taken into account during the meshing process in this Step:

Mesh Size:

Mesh Size = Smallest feature size in the model/Desired resolution

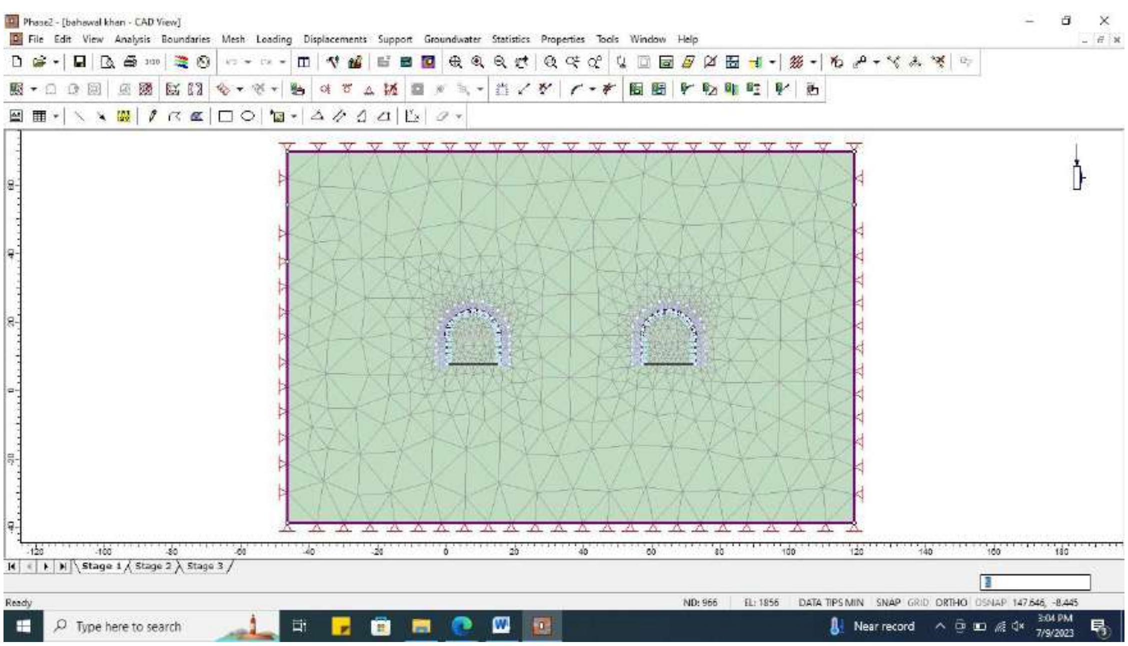


Figure 13 Using finite element analysis

3.5 Rock Mass Parameter Calculation

Using Rock mass classification RMR Parameters and finding the rock mass quality also support recommendation on the basis of RMR and Q as mention in table1, 2, and 3.

Adjusted RMR Calculation:

After taking into account the following adjustment variables, the Adjusted RMR can be derived from the Average RMR:

Adjusted RMR = Average RMR + Correction Factors Using the data shown in Table 1, for instance:

Adjusted RMR = 42 - 5 = 3

RMR parameters	**Description of RMR Parameters**	**Parameter rating**
Uniaxial compressive strength	Uniaxial strength is 100 to 50	7
RQD	25 to 50 percent	8
Spacing of discontinuity	Most occurring 0.2 to 0.06	7
Condition of discontinuity	There are open joints having length 1 to 3m. Slightly rough, soft material filled and moderately	13
	weathered.	
Ground water	Wet to damp	7
Joint orientation	fair	-5
Average RMR	RMR	42
Adjusted RMR	Adjusted RMR	37

Table 15 Rock Mass classification base on RMR

3.6 Q System Calculation

The Q-value, derived from the RMR, is given by:

$$Q= \exp (RMR-44)/9$$

As an illustration, using values from Table 2:

$$Q= \exp (3744)/9 \approx 0.46$$

Classification system	System Rating	Rock mass quality
RMR system	37	Poor
Q System (Q=exp (RMR-44)/9	0.46	Very poor

Table 16 Rock mass quality based on RMR and Q system

Classification system	**Rock quality mass**	**Excavation Method**	**Support system**	
RMR	Poor	Full face	Systematic rock bolts 3m with spacing 2.5.	50mm Shotcrete In crown and walls
Q System	Very poor	Full face	Systematic rock bolts of 5cm with spacing 2.5cm	40 to 100 mm shotcrete in crown and walls

Table 17 Support recommendation Based on RMR and Q

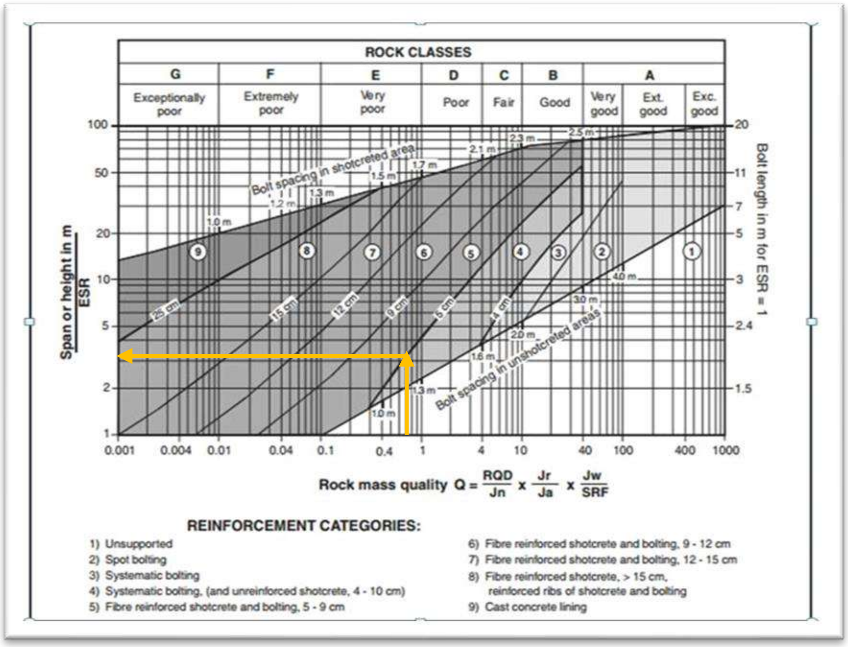


Figure 14 Support analysis based on Q value.

3.7 Required Input Parameter Tables Obtain from Field used in Numerical Modeling

In this step we Design input parameters required for Numerical modeling as mentioned in below tables.

Uniaxial compressive strength	Hoek and brown constants	Unit weight (KN/M2)	Poisson Ratio	Young modulus (GPA)	Vertical stress	Horizontal stress
124	mb=5.813	31.65	0.25	60	5.01	5.96
	s=0.0084					
	a=0.504					

Table 18 Design Input Parameters For numerical modeling

Gabbronorite	Length	Over Burden (m)	Uniaxial compressive strength (MPA)	Tensile strength (MPA)	Shear modulus (GPA)	Deformation modulus
GTU 1	893m	190.4	124	58.69	24.00	21.51

Table 19 Strength properties of rock mass along tunnel axis.

CHAPTER 4
RESULTS AND DISCUSSION

Results and Discussion

4.1 Principle total stress before excavation

Values of priciple total stress before the excavation of tunnel having a minimum range of 2.55 to maximum range of 6.15 showing the decrease of stresses before excavation.

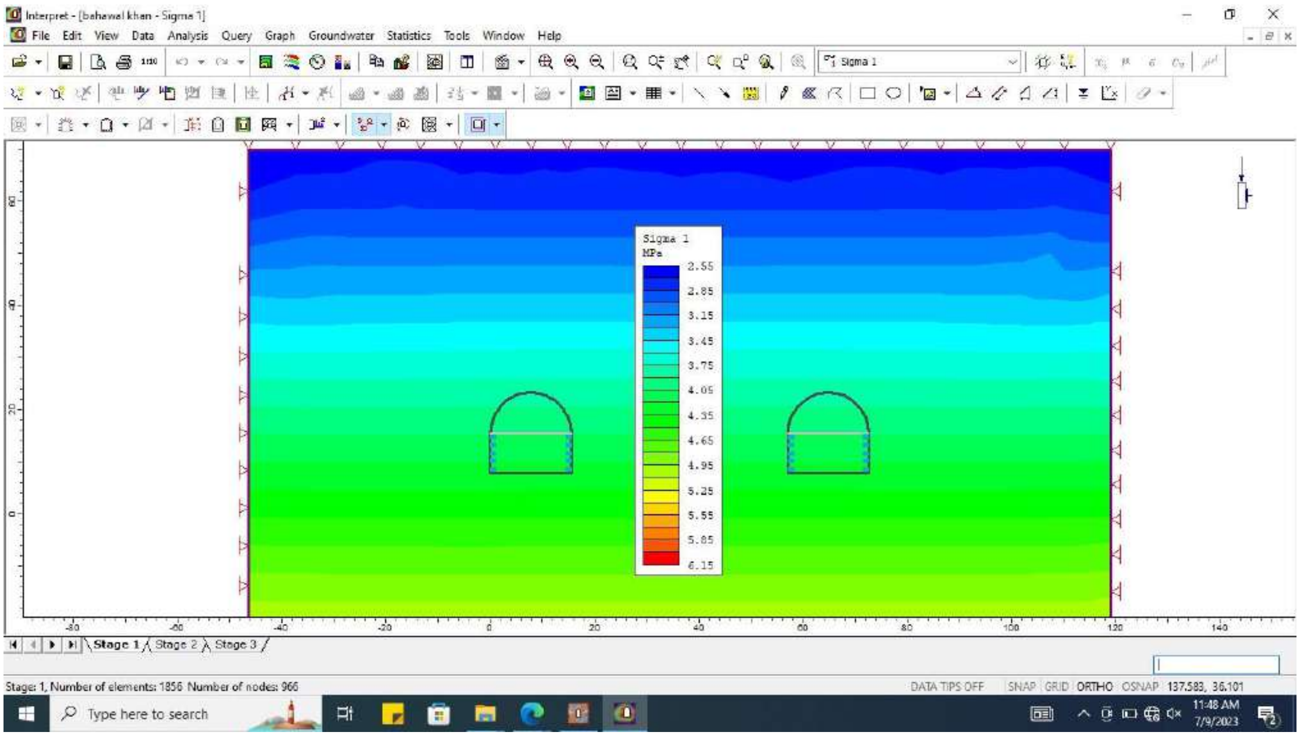


Figure 15 Sigma 1 results before excavation phase 2.

4.1.1 Sigma 1 Value after Top Heading Excavation

After the excavation of top heading the sigma 1 values increases to a maximum range of 15.60.

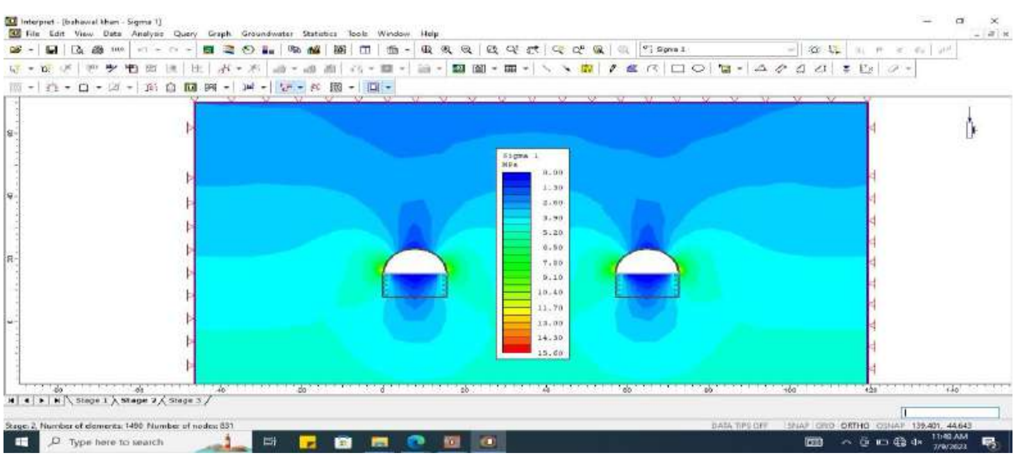


Figure 16 Sigma 1 Value after Top Heading Excavtion

4.1.2 Sigma 1 Value after full face Excavation

This step is performed on the basis of RMR support chart installation of rock bolts and shotcrete.we can see the decrease in value of sigma 1.

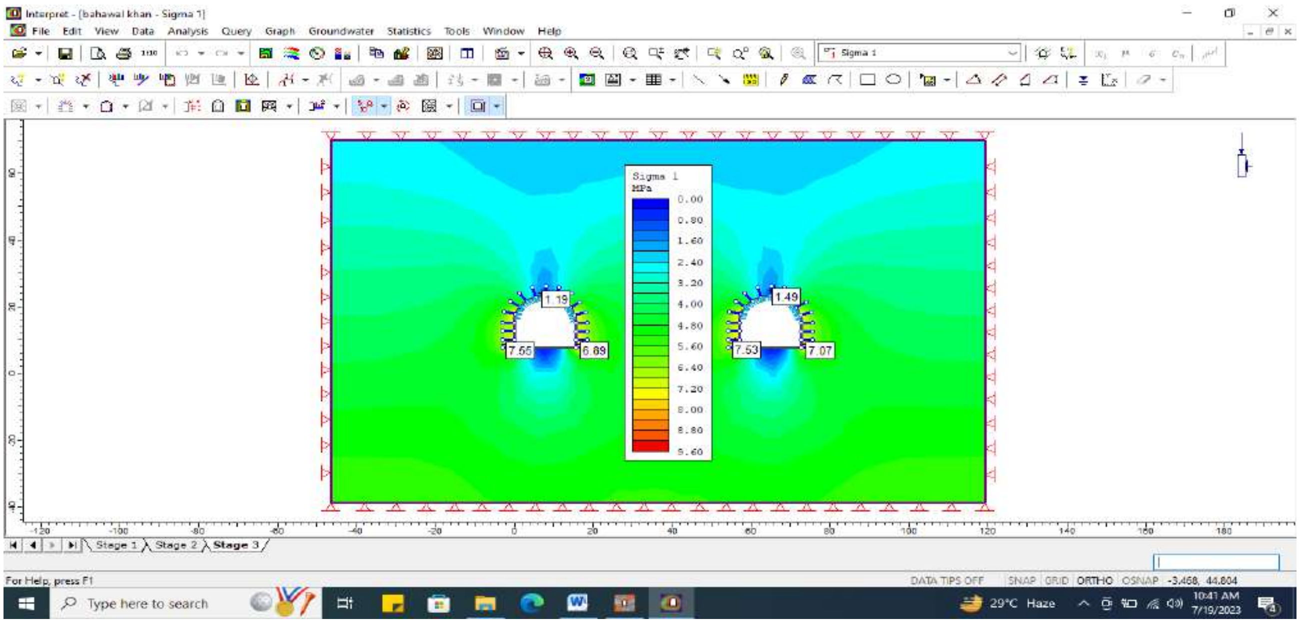


Figure 17 Sigma 1 value after full phase excavation and support installation based on RMR

4.1.3 Sigma 3 values Before Excavation

Below model shows the values sigma 3 before the excavation of tunnels values Ranging from 0.50 up to a maximum value of 1.81 sigma 3 is also minimum before excavation.

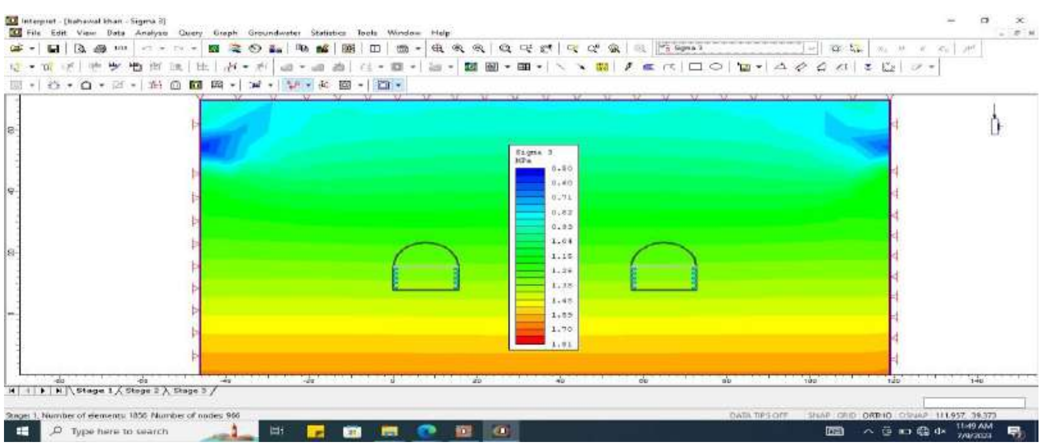


Figure 18 Sigma 3 values before excavation.

4.1.4 Sigma 3 Values after Top Heading Excavation

This figure shows the value of sigma 3 after excavation of top heading having maximum value of 3.00 and minimum value of -1.80.

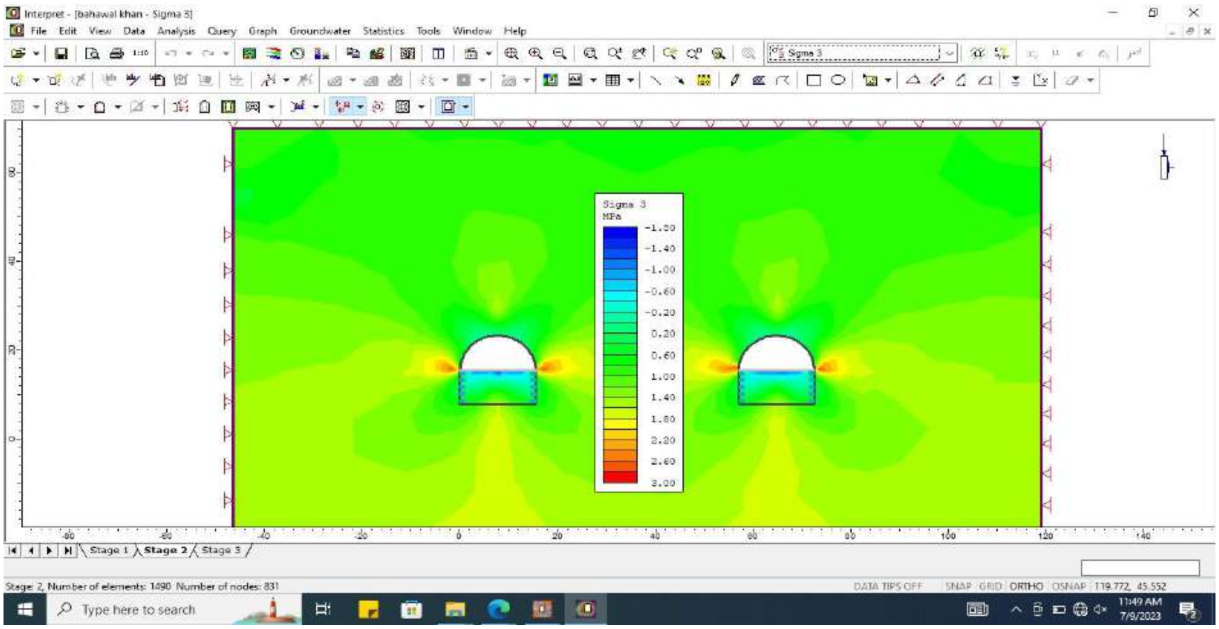


Figure 19 Sigma 3 values after top heading excavation.

4.1.5 Sigma 3 Values after Support installation on the basis of RMR bench excavation

The model shows the value of sigma 3 after installing supports that's inculde rock bolt and shotcrete on the bases of RMR chart as mention in table 3.

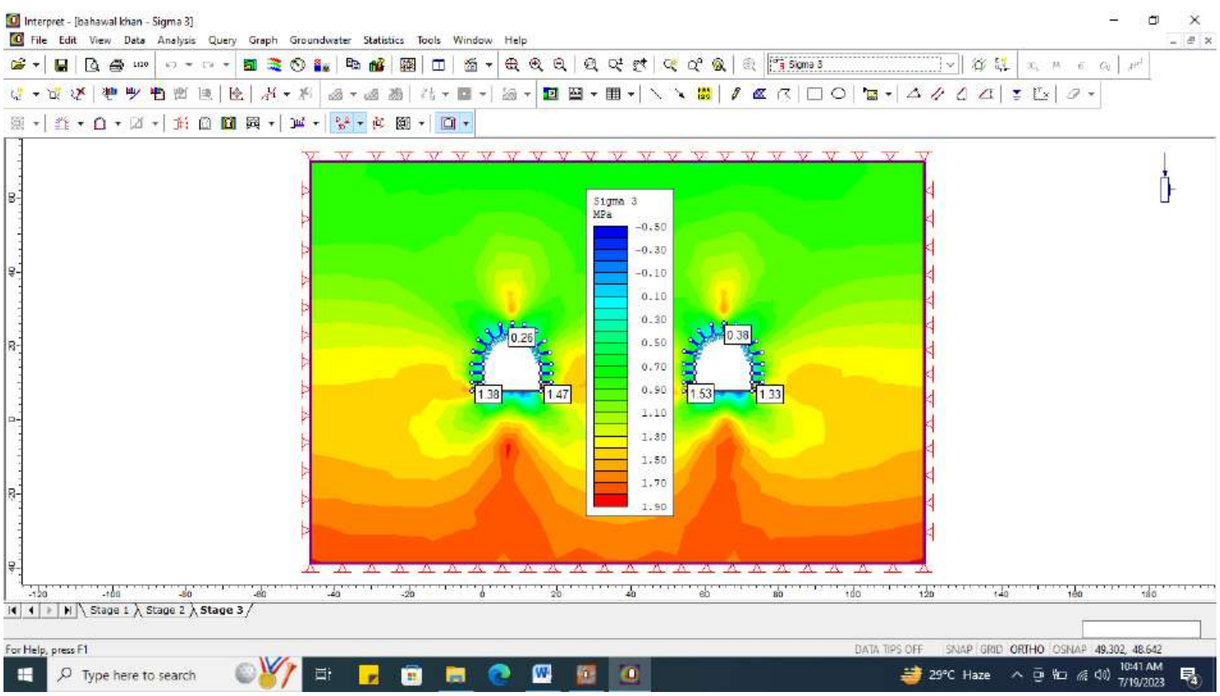


Figure 20 Sigma 3 values after support installation on the basis of RMR

4.1.6 Total Displacement before Excavation

The values of total displacement before the excavation of tunnels.As we know the model shows there is a slight displacements before excavation.

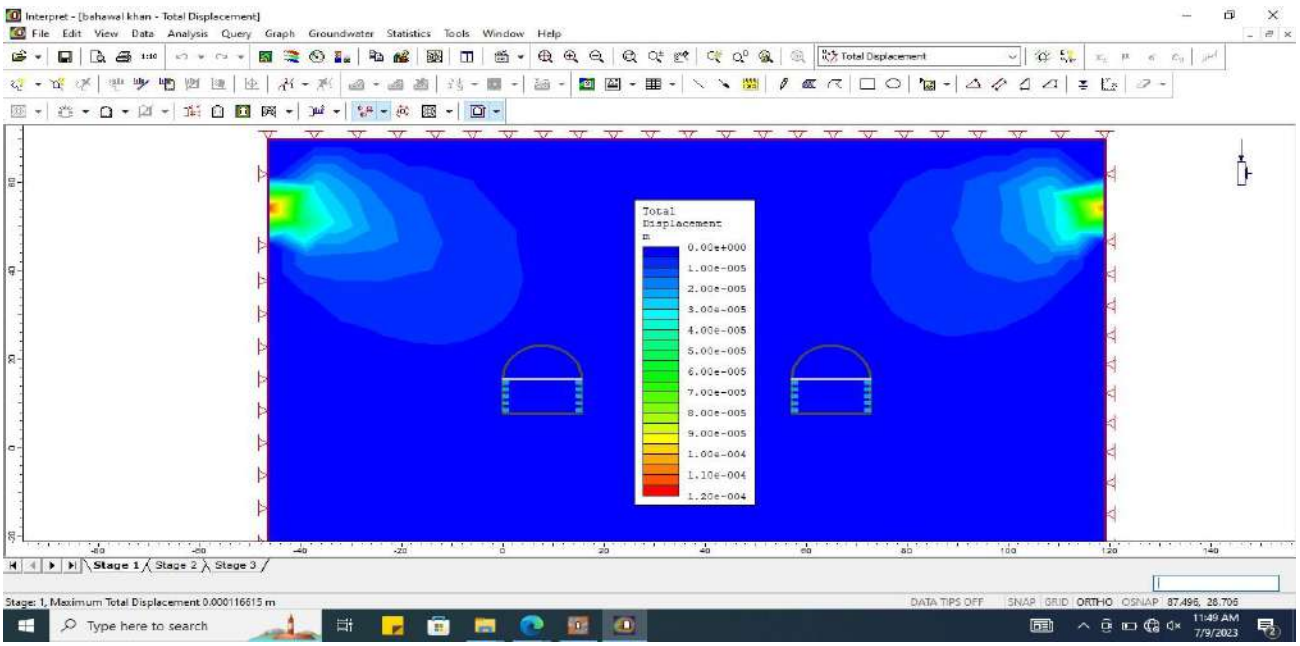


Figure 21 Total Displacement before Excavation

4.1.7 Total Displacement after Top Heading Excavation

The results of total displacement after excavation of Upper heading of tunnel the values od displacements goes on increasing around different portions of tunnel.

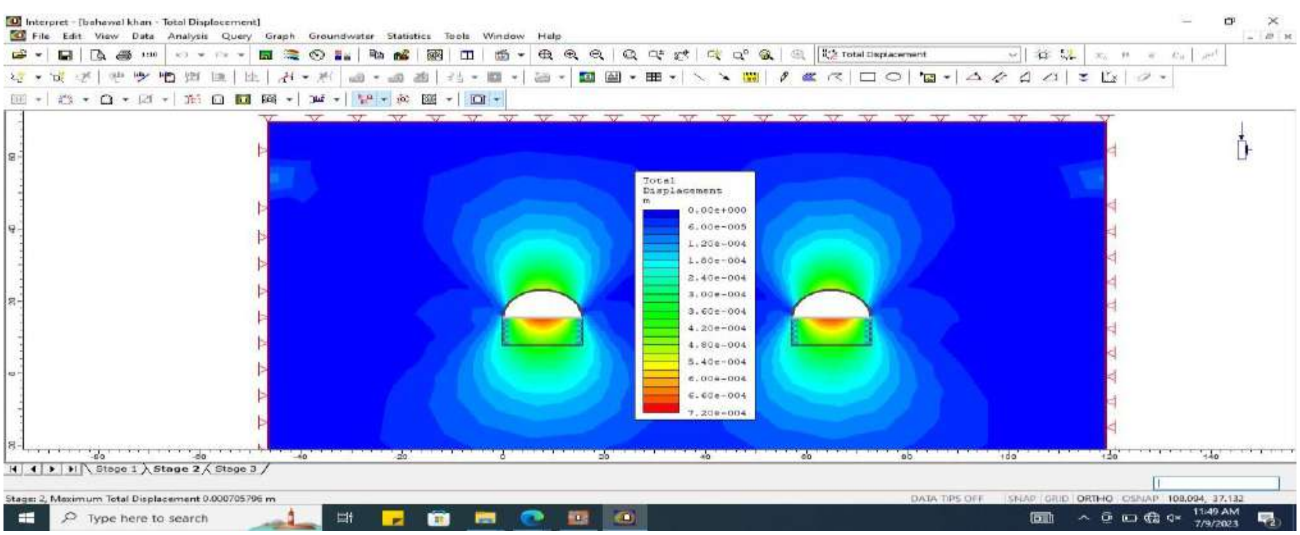


Figure 22 Total displacement after top heading excavation.

4.1.8 Total displacements after RMR support system

The below given model shows the values of total displacement after full face excavation and support installation based on RMR.

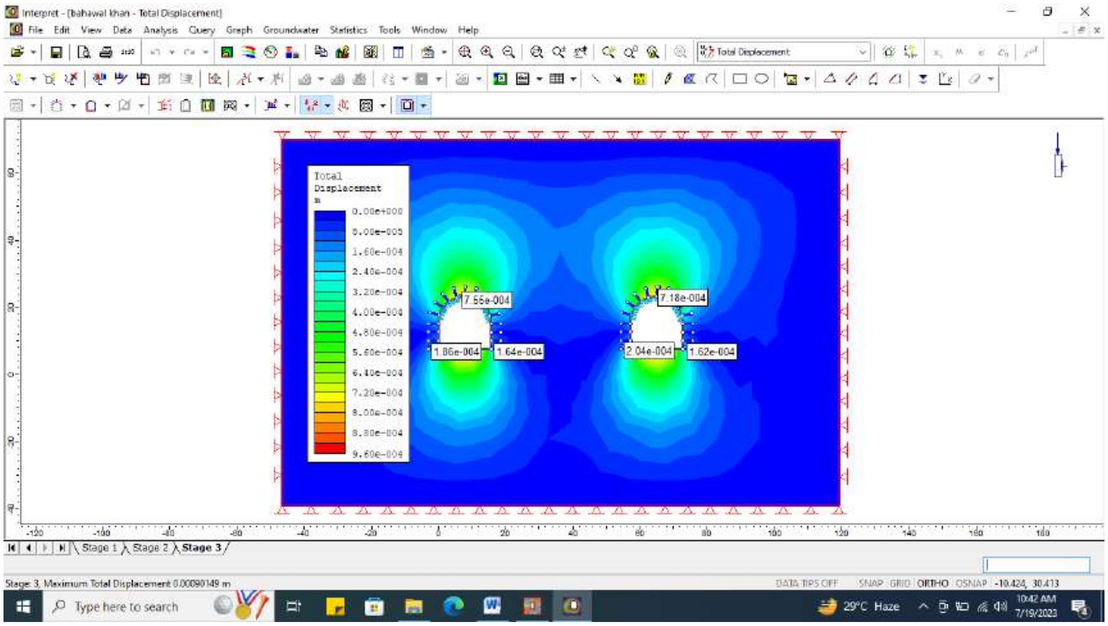


Figure 23 RMR support installation and Total displacements

4.1.9 Sigma 1 values After Installation support On The basis of Q

Values of sigma 1 on the basis of Q support system as we seen there is a same stress value for both the systems.

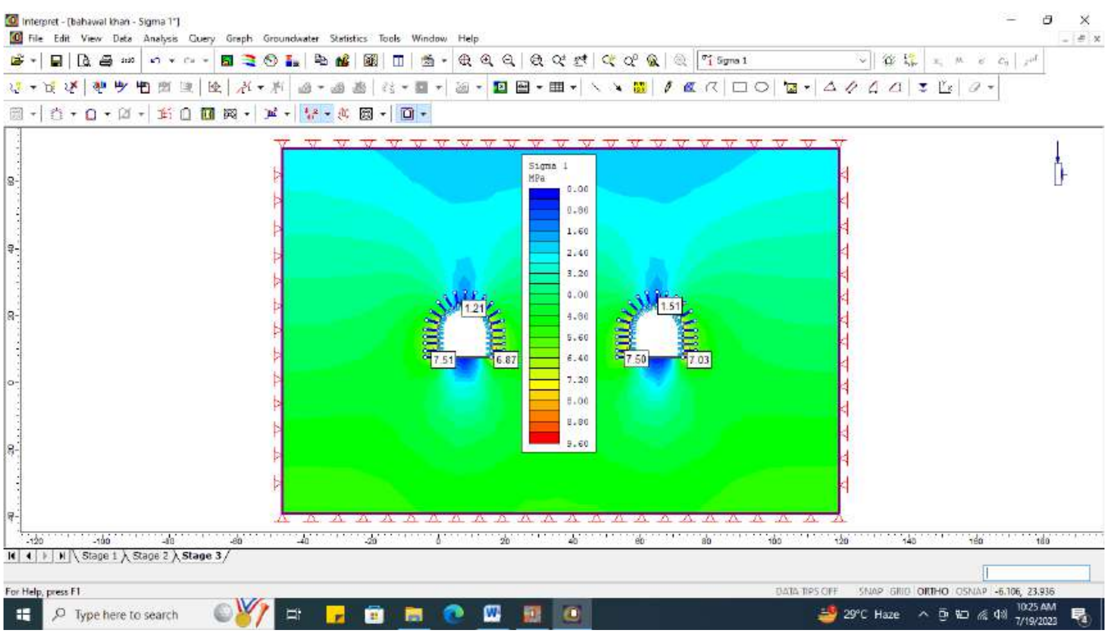


Figure 24 Q support results of Sigma 3

4.1.10 Sigma 3 values based on Q support.

Q support also shows the same value of sigma 3 as shown in RMR support as both the methods..

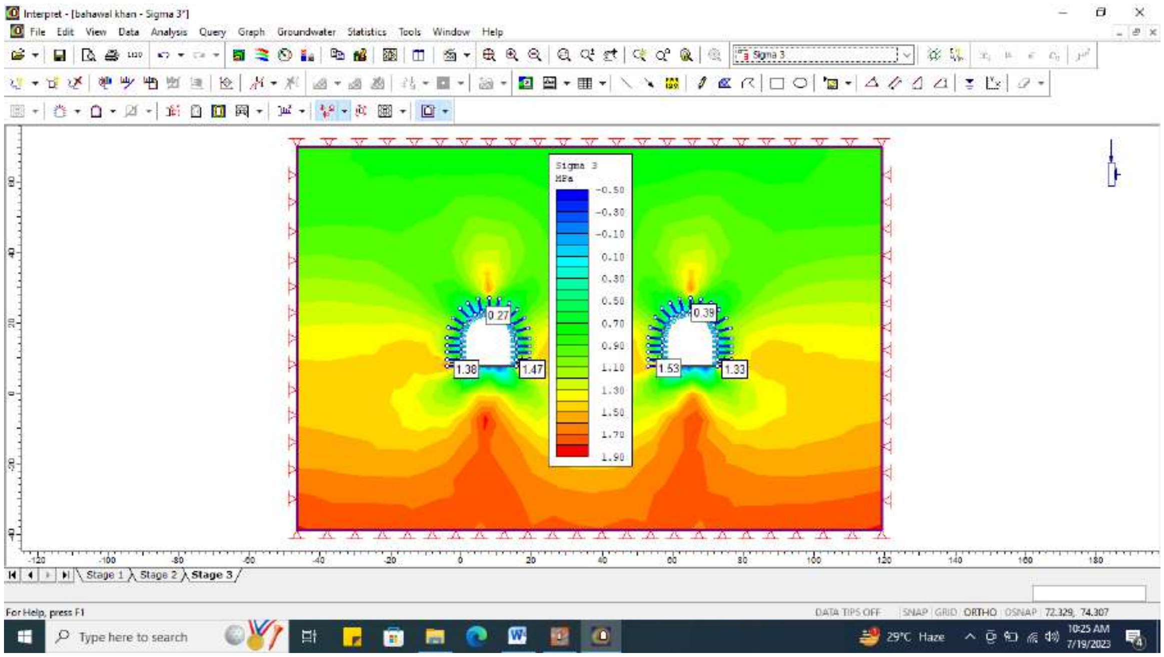


Figure 25 Sigma 3 values on Q support.

4.1.11 Total Displacements after Q support

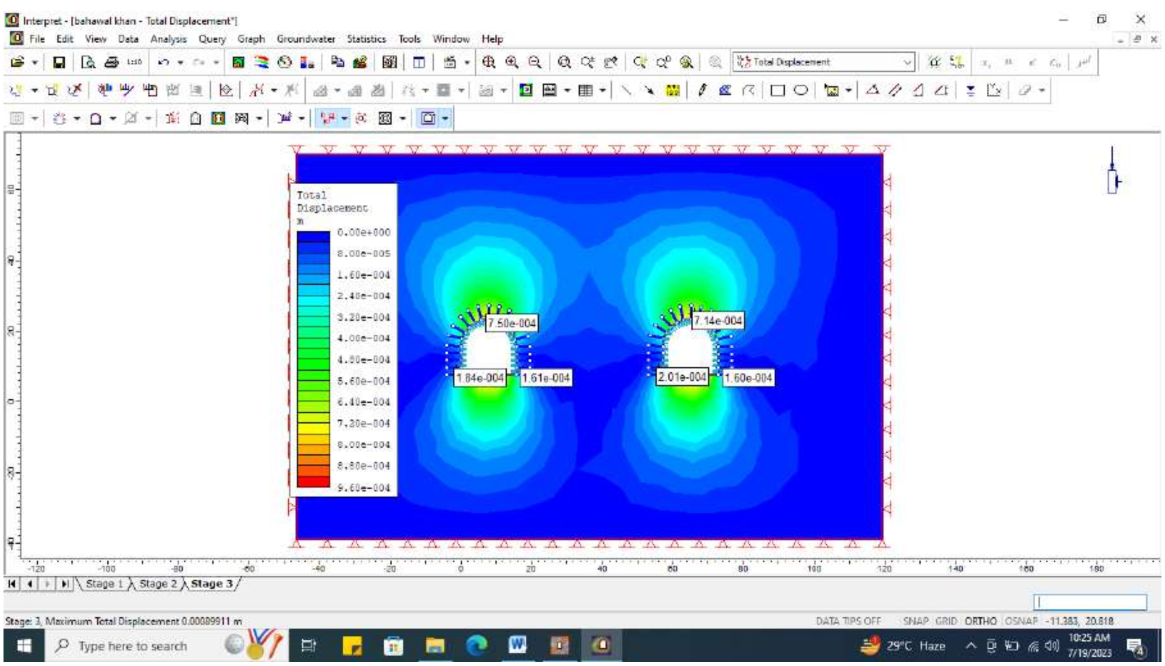


Figure 26 Total displacements results based on Q support.

4.2 Discussion

The findings of this research, together with those of Sari et al. (2008), emphasise the value of using empirical methodologies to evaluate tunnel support systems. Their research into the tunnel on the Turkish highway between Bilecik and Istanbul relied mainly on empirical approaches to determine the properties of the rock mass and to provide design support suggestions. The computational and empirical methods emphasised by Sari et al. are consistent with the current study's focus on the values of principal total stress before to excavation and the observation of changes to these values post excavation.

The difficulty of tunnel construction through soft rock has been highlighted by Gao, Chen et al. (2016), who offer support systems that limit the displacement of the surrounding rock. During building of the Youfangping tunnel, they encountered considerable deformations. Figures illustrating Sigma 1 and Sigma 3 values from the present research similarly shed light on the need of dependable back-up plans. When compared with the existing literature, our findings highlight the need of iteratively improving support design for tunnels.

The significance of assessing the geotechnical qualities of rock masses was also emphasised by Kaya and Bulut (2019), who examined the Cankurtaran Tunnel project. They investigated the complexities of rock formations mostly made up of volcanic and sedimentary materials. These conclusions are supported by the data presented here, especially the overall displacement values before and after excavation. Results from this work are linked to the larger literature via a focus on convergence-confinement (CC) methods and finite element method (FEM) modelling.

Liu and Zhang's investigation of the Melbourne Metro Tunnel demonstrated the value of settlement prediction in avoiding catastrophic building failures. This is a critical point that agrees with Chapter 4's findings, particularly when looking at overall displacement values after RMR and Q-based support system installations. Both papers stress the need for solid infrastructure to help prevent any future conflicts from becoming permanent settlements. Intriguingly, the literature proposes the Sequential excavation technique (SEM) for tunnel construction in the Niayesh urban road tunnel owing to the soft ground (Gao, Chen et al., 2016). The findings of this research discreetly highlight the significance of selecting excavation techniques according to the geological conditions. Similarities between the present study's focus on the Rock Mass Quality Index (Q) systems and rock mass rating (RMR) and the Bieniawski engineering categorization system (Bieniawski, 1993) can also be seen in the classification of rock masses.

CHAPTER 5
CONCLUSION AND RECOMMENDATIONS

Conclusion and Recommendations

5.1 Conclusions

Based on analysis of results following conclusions are derived:

1) The rock mass along tunnel axis is divided into three geotechnical units based on the Rock types and classes. i.e GU-1, GU-2, GU-3
2) The RMR and Q-system used as empirical methods reveals that the rock mass along the tunnel axis is competent and none of the rock mass unit fall into immediate collapse region.
3) The joint sets present in the rock mass along the tunnel axis are mostly favorable and have less adverse effect on the stability of tunnel.
4) The support systems recommended by both RMR and Q-system are efficient to be used for stabilizing the tunnel under the given rock mass conditions.
5) The sigma1 value before excavation is 6.15(MPA), after full face excavation this values rises to 15.69 (MPA) but after support analyses on the bases RMR these stresses reduces to a value of 9.60 (MPA).
6) The support recommends by RMR and Q system are applicable in case of tunnel support analysis as both the system have same value of stresses around tunnel.
7) Overall the rock mass characterization shows that the rock masses along the tunnel's axis are good and requires minimum support for stability.

5.2 Recommendations

Based on analysis of results following recommendations are made:

The support systems recommended by both RMR system and Q-System should be installed for stability of the diversion tunnels at Diamer basha dam project.

The parameters used in empirical design techniques should be optimized using different statistical tools.

The support system recommended for the said project should be evaluated through numerical modelling.

The empirical and numerical methods should be used together for efficient and stable design of any underground structure within the rock mass environment.

The results will be used as a reference for safe and stable designing of tunnel in other areas.

5.3 Future Implications

There are a number of implications for the future of tunnelling that can be drawn from the study's findings. In order to better comprehend rock masses and navigate them, it is expected that more sophisticated geological surveys will become available as time goes on. Accurate geological mapping and analysis should improve the strength of future Tunneling. There is need for improvement in rating systems that take into account a wide range of geological circumstances; although the Rock Mass Quality Index (Q) and rock mass rating (RMR) systems have offered useful frameworks, they might be improved upon. Future tunnel projects may gain efficiency and accuracy in modelling and forecasting tunnel behaviours from novel numerical approaches thanks to the fast improvements in computer capabilities, with the use of AI and 3D modeling. Use of discrete numerical modeling, UDEC/3DEC we can improve the stability of future tunnel projects resulting in economical and safe environments.

REFERENCES

Akram, M. S. and M. Zeeshan (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: 221-226.

Ali, W., et al. (2014). "Rock mass characterization for diversion tunnels at Diamer Basha Dam, Pakistan-a design perspective." International Journal of Scientific Engineering and Technology 3(10): 1292-1296.

Bieniawski, Z. T. (1989). Engineering rock mass classifications: a complete manual for engineers and geologists in mining, civil, and petroleum engineering, John Wiley & Sons.

Gao, S.-m., et al. (2016). "Structure optimization for the support system in soft rock tunnel based on numerical analysis and field monitoring." Geotechnical and Geological Engineering 34: 1089 1099.

Hussain, S., et al. (2018). "Numerical modeling for engineering analysis and designing of optimum support systems for headrace tunnel." Advances in Civil Engineering 2018.

Kaya, A. and F. Bulut (2019). "Geotechnical studies and primary support design for a highway tunnel: a case study in Turkey." Bulletin of Engineering Geology and the Environment 78: 6311 6334.

Liu, Y. and Q. Zhang "Numerical Modelling of the Stability of Metro Tunnel Stations."

Moldovan, A. R. and A. Popa (2012). "Finite element modelling for tunneling excavation." Civil Engineering & Architecture 55(1): 98-113.

Oke, J., et al. (2016). "Semi-analytical model for umbrella arch systems employed in squeezing ground conditions." Tunnelling and underground space technology 56: 136-156.

Rasouli, M. (2009). "Engineering geological studies of the diversion tunnel, focusing on stabilization analysis and support design, Iran." Engineering Geology 108(3-4): 208-224.

Sari, Y. D., et al. (2008). "Numerical analysis of a tunnel support design in conjunction with empirical methods." International journal of geomechanics 8(1): 74-81.

Naji, A.M.; Emad, M.Z.; Rehman, H.; Yoo, H. Geological and geomechanical heterogeneity in deep hydropower tunnels: A rock burst failure case study. Tunn. Undergr. Space Technol. 2019, 84, 507–521.

Ur Rehman, Z., et al. (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): 54-60.

Javed, W. and Tingsanchali, T., 2016, November. Sediment flushing strategy for reservoir of proposed Bhasha Dam, Pakistan. In 2nd World Irrigation Forum (WIF2) (pp. 6-8).

Sabir, M. and Torre, A., 2017. Different Proximities and Conflicts Related to the Setting of Big Infrastructures: The Case of Diamer Bhasha Dam in Pakistan. Regional Cooperation in South Asia: Socio-economic, Spatial, Ecological and Institutional Aspects, pp.30532.

Tareen, S., 2022. Crowdfunding Nayā Pakistan: Development, Nationalism, and the Diamer Bhasha Dam. The Muslim World, 112(1), pp.151-166.

Hussain, K., Shahab, M. and Štibinger, J., 2020. Detection of trend change in long-term flow series of Upper Indus River Watershed, Pakistan. International Journal of Energy and Water Resources, 4, pp.109-118.

Lv, Y., Jiang, Y., Hu, W., Cao, M. and Mao, Y., 2020. A review of the effects of tunnel excavation on the hydrology, ecology, and environment in karst areas: Current status, challenges, and perspectives. Journal of Hydrology, 586, p.124891.

Heidarzadeh, S., Saeidi, A. and Rouleau, A., 2021. The damage-failure criteria for numerical stability analysis of underground excavations: A review. Tunnelling and Underground Space Technology, 107, p.103633.

Naji, A.M.; Rehman, H.; Emad, M.Z.; Ahmad, S.; Kim, J.-j.; Yoo, H. Static and Dynamic Influence of the Shear Zone on Rock burst Occurrence in the Headrace Tunnel of the Neelum Jhelum Hydropower Project, Pakistan. Energies 2019,

Ali, E. W. (2014). M.sc thesis, rock mass characterization for diversion tunnel at diamer basha dam, Pakistan-a design perspective. Peshawar: University of Engineering and Technology, Peshawar, Pakistan.

Terzaghi, K., 1946. Rock defects and loads on tunnel supports. Rock tunneling with steel supports.

Lauffer, H., 1958. Gebirgsklassifizierung fur den stol lenbau. Geologie und Bauwesen, 24(1), pp.46-51.

Deere, D.U., 1964. Technical description of rock cores for engineering purpose. Rock Mechanics and Enginee-ring Geology,, 1(1), pp.17-22.

Wickham, G.E., Tiedemann, H.R. and Skinner, E.H., 1972, June. Support determinations based on geologic predictions. In N Am Rapid Excav & Tunnelling Conf Proc (Vol. 1).

Barton, N., Lien, R. and Lunde, J.J.R.M., 1974. Engineering classification of rock masses for the design of tunnel support. Rock mechanics, 6, pp.189-236.

Bieniawski, Z.T., 1973. Engineering classification of jointed rock masses. Civil Engineering= Siviele Ingenieurswese, 1973(12), pp.335-343.