Chapter

Design Techniques in Rock and Soil Engineering

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Abstract

At the initial stage of tunnel design, the tunnel stability can be assessed by different design techniques which are broadly classified into three categories i.e. Mathematical Analysis, Empirical Methods and Numerical Analysis. Mathematical methods or closed form solutions are more precise methods; however, its use is limited to simple geometries and almost impossible for complex geometries due to complex and tedious calculations involved. In practice, Empirical and Numerical Methods are usually used for stability analysis of tunnels. It should be noted that it is not the replacement of final design. Empirical design methods use information about the structural geology and other rock mass properties as input that can be easily obtained at the initial stage of a project. Numerical Methods commonly require mechanical properties, especially strength and deformation of rocks. Numerical methods are also considered as precise due to provision of allowance for variable inputs and geometry and having ability for sensitivity analysis. It is good practice to evaluate the stability of tunnels using at least two Empirical methods and validated through Numerical methods.

Keywords: tunnel design, design techniques, stability, sensitivity, RMR

1. Introduction

The process of engineering design comprises of devising a scheme/module, or process to achieve the required goal or target. It can also be defined as an assessment –making procedure, which utilized the knowledge of basic sciences, mathematics and engineering sciences to convert resources optimally to meet quantified objectives. In other words, engineering design is the procedure of formulating framework, segment, or procedure to address desired problems [1]. General goal of engineering design is to develop a solution (the design) to a known problem. However, there is no single solution, and depends upon the approach used by different engineers resulting different solution. Among the solution obtained some will work well than others, but it is necessary that all solutions should 'work'. The reason behind the fact that solutions to engineering design problem are not unique is perhaps due to very broader spectrum of the concerns encountered in design [2].

2. The design process

Each and every engineering problem/task passes through a design process. According to Hill (1983), as discussed by Biniawski (1988), the design process is:

a) logical development of design inside organization of actions and b) a work plan process for planning the design program. For satisfactory design results, a define process can work as agenda of activities. The defined process or methodology can be considered as a form of quality control that ensures that all aspects that should be considered in the design are considered [2]. Response to a complex engineering problem does not shortly seem in a vacuum. Well-meaning description of engineering problem needs exercise or approach. Design processes generally depend upon the number of engineers analyzing design. The process described here is general, and one can adapt it to the problem, they are trying to solve [3]. Following are the different stages of design process [1] illustrated in **Figure 1**.

1. Recognition of need or a problem
2. Statement of the problem
3. Collection of information
4. Analysis of solution component
5. Synthesis to create a detailed solution
6. Evaluation of ideas and solutions
7. Optimization
8. Recommendation
9. Communication
10. Implementation

2.1 Recognition of need or a problem

Engineering design activity always occurs in response to a human need [3]. Before attempting any solution for design, the presence and nature of a problem must be. This is not an easy task. It needs the rather rare skill of inquiring the right kind of question and call for a clear identification of the problem to be solved. In design it involves the recognition of a genuine social need want or opportunity.

2.2 Statement of the problem

If there is any problem involves, it is then necessary to clearly define it. This may involve a list of specification or criteria. These must be stated clearly and concisely. A poorly recognized and expressed problem cannot be anticipated to result in a good solution. In rock mechanics design, this means to set design objectives in terms of economy, safety and stability.

2.3 Collection of information

This stage comprises the collecting, investigation, processing and analyzing of information to obtain the explicit nature of the targeted problem. In rock engineering collection of information include site investigations, conducting in-situ and

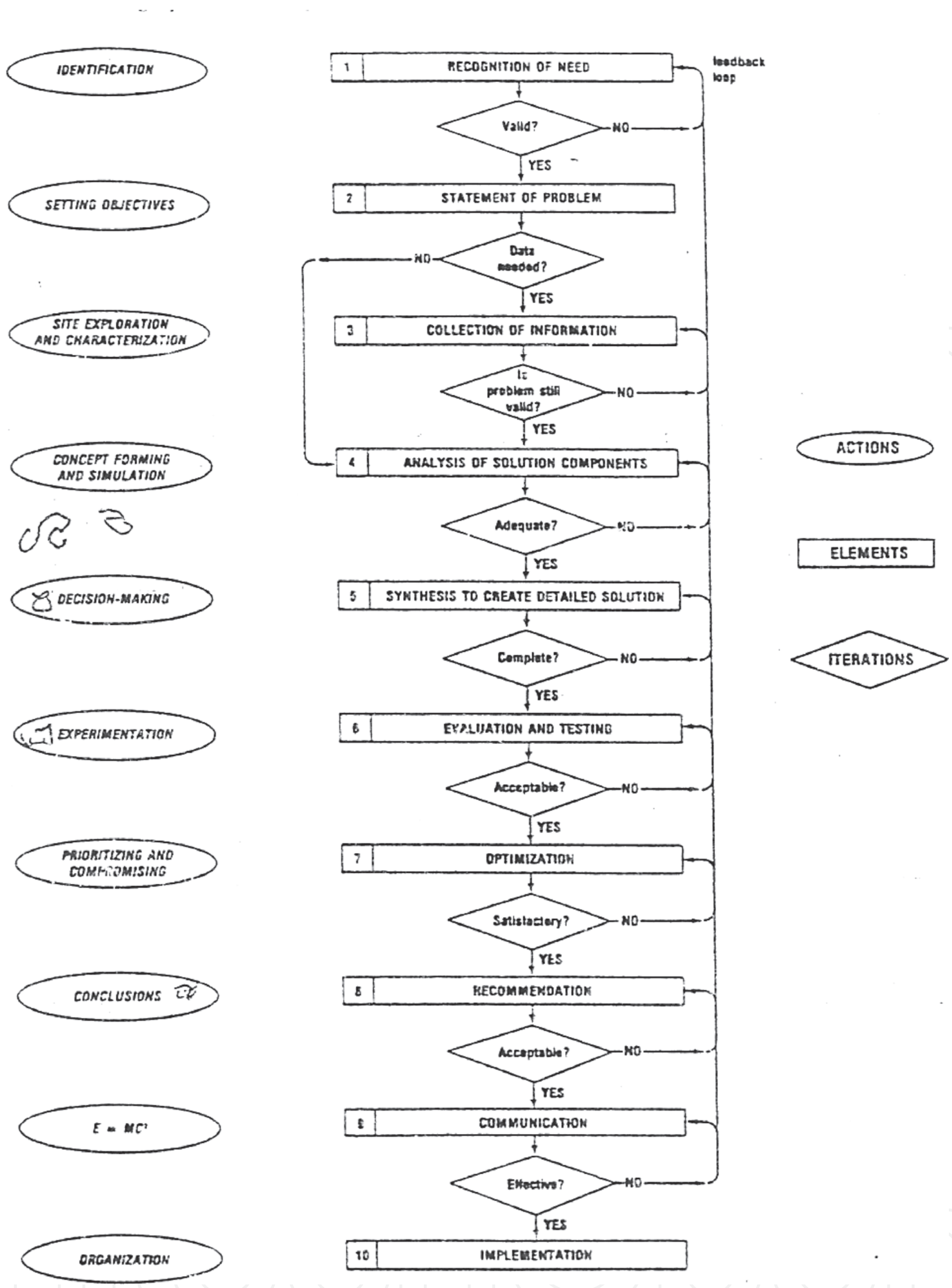

Figure 1.
The engineering design process [1].

laboratory tests to determine the characteristics of the rock strata and assessment of applied loads and field stresses.

2.4 Analysis of solution component

The selection of approach to either search for the most promising method of solution or certain hypothesis is selected or conceived depends upon the nature of the problem. Design approaches at this phase involve numerical analysis and mathematical, physical modal studies, observation and monitoring or the empirical analyses based on experience.

2.5 Synthesis to create a detailed solution

On the basis of analysis of the individual solution component, all design is focused to furnish comprehensive alternative solutions. In this phase of design, calculations, specifications, performance predictions, cost estimates, scheduling procedures and the experimentation are involved.

2.6 Evaluation of ideas and solutions

In this phase the solution is interpreted and compare with the original hypothesis, specification, facts assumptions, requirements or constraints. This demand for a clear understanding of the all relevant interacting factors that's needed for the engineering judgments. The solution for engineering problems should be balanced involving all the factors with interact.

2.7 Optimization

Optimization is the assortment of a best solution (with regard to some criteria) from some set of available alternative solutions [4]. There are always multiple solutions available to any engineering problem. Refinement and modification of a solution may then be required to reach a practicable agreement between the generally contradictory constraint and assets. The effectiveness of an optimization process mostly depends upon simplicity and clarity with which problem and solution are specified.

2.8 Recommendation

Recommendation is the principle of the whole Engineering design process. It provides a refined endorsement of the solution to problem, point out limitations and shows the trend to be followed in applying the solution.

2.9 Communication

The conclusive aim of the all design stages is the creation or instigation of a progression accomplishment. In order to achieve the objective requires the engineer must communicate the finding effectively. Effective communication means that all relevant aspects should be appropriately presented. If a mathematician were to sum up these thoughts, he might well do so by the Eq. (1).

$$E = MC^2 \tag{1}$$

Where,

E means effectiveness of the subject, M mean the mastery of the subject matter and C means the communication.

So for effective communication one should have sound knowledge of the subject matter and good communication skills. The design engineer must have the capability to communicate views and ideas concisely and clearly and to convey technical knowledge effectively.

2.10 Implementation

This is the final stage of design procedures. The finding or results communicated are applied under the given circumstance and proper monitoring is carried out for further refining the result or design that has been recommended for action. The main objective of the design is to ensure that a desire goal and quality will achieved within the time frame and the budget allocated.

2.11 Feed back

After implementation of the design, its performance is monitored and recorded. Remedial measurements are suggested for more improvement of the performance the solution design.

3. Design techniques in soil and rock engineering

There are different significant design techniques in rock engineering. They are classified into three groups which are Analytical, Empirical and Observational. Rock masses having more complex in nature. Due to the very complex nature of rock masses and the difficulties encountered with their characterization, the analytical approach is the least used in the present engineering practice. Due to this reason, it does not lie in the analytical techniques themselves, since some have been developed to a high degree of sophistication, but in the inability to furnish the necessary input data as the ground conditions are adequately explored. Consequently, such analytical techniques as the finite element method, the boundary element method, closed form mathematical solutions, photo-elasticity or analogue simulation are mainly useful for assessing the influence of the various parameters or processes and for comparing alternative design schemes; they are the methods of the future not as yet acceptable as the practical engineering means for the design of rock tunnels [5]. Empirical methods of design are commonly applied as these are built on earlier practices derived from creation of rock structures owning alike physical characteristics [6]. It is a good practice to evaluate the stability of tunnels using at least two Empirical methods and validate through Numerical methods. Therefore, these two groups of tunnel design methods will be discussed in detail [7].

4. Empirical methods of design

The empirical approach relates the experience encountered at previous projects to the conditions anticipated at a proposed site. If an empirical design is backed by a systematic approach to ground classification, it can effectively utilize the valuable practical experience gained at many projects, which is so helpful to exercising one's engineering judgment. This is particularly important since, a good engineering design is a balanced design in which all the factors which interact, even those which cannot be quantified, are taken into account; the responsibility of the design engineers is not to compute accurately but to judge soundly. Rock mass classifications, which the main part of the empirical design methods, are extensively used tunnels within rock. At present, most of the tunnels excavated in the United States make use of some classification system. Terzaghi classification which was presented over 40 years ago is the most broadly used. In fact, rock mass classifications have been successfully applied throughout the world [5].

The empirical methods of design may be used in association with other engineering assessment and design Techniques [6]. These methods are very essential and beneficial for the design in the earlier stages of the project, when minimum evidence about the behavior of rock mass, stress conditions and hydrological characteristics are obtainable [8].

4.1 Rock mass classification systems

Rock mass classification is a tool for the assessment of the rock behavior and performance based on the essential inherent and structural parameters [9]. Rock mass classification systems are the most and widely used empirical methods of design Different rock mass classification systems are RMR, Q-System, RQD, RSR, GSI etc. [6]. Rocks have been classified on the basis of origin, mineralogical compositions and distinct physical properties and ground condition. Rock Classification provides a mutual basis of communication to recognize rock mass in a category having same and well define characterization and basic input parameters for rock engineering design. For designing purposes in several attempts were made to classify rock based on rock and site characterization. Such simplified classification systems have served to understand the upper bound response of the rocks [10]. Rock mass classification systems effectively combined the results comes observation, experience and other engineering judgment for providing a quantitative evaluation of rock mass situations. Rock mass classification systems has the below mentioned purposes in tunneling design [5].

1. Group rock masses having similar behaviors.

2. Provides the root for understanding the characteristics of independent groups.

3. Helps in planning and designing of excavation in rock and provide quantifiable data for the design of complex engineering complications.

4. A common understanding agenda for all the related people in the project.

Up till now different rock mass classification systems have been proposed by Terzaghi (1946), Lauffer (1958), Deere (1964), Wickham, Tiedemann, and Skinner (1972), Bieniawski (1973), and Barton, Lien, and Lunde (1974), (Bieniawski Z. T. 1990). The different classification systems used for the design purposes are assembled in **Table 1**.

4.1.1 Terzaghi's rock mass classification

A well-known classification system for support of tunnels. This explanatory system was developed in the U.S.A in 1946. Terzaghi's (1946) formulate the first rational method of evaluating the rock loads suitable to the design of steel sets. This classification is appropriate for the estimating rock loads for steel arch supported tunnels. It is not so suitable for modern tunneling methods using shotcrete and rock bolts [5].

Terzaghi's classify rocks as under [11]:

1. ***Intact Rock***: Rocks that's having no joints and cracks, it breaks crossways a sound rock or loose block may drops off the top for many hours and days due to blasting. It is called sapling condition.

   Stratified rock: that rock composed those distinct sections having slightly or no confrontation to parting beside the margins stuck between the strata. In such rock the spalling condition is generally happened.

2. ***Moderately jointed rock:*** That rock having joints and hair cracks, but the blocks among joints are locally developed collectively or so closely joined that perpendicular walls do not need on the sides support. In this type of rock, both spalling and popping conditions may be happened.

S. No	Rock mass classification system	Originator	Origin country	Application areas
1	Rock Load	Terzaghi, 1946	USA	Tunnels with steel support
2	Stand-up time	Lauffer, 1958	Australia	Tunneling
3	New Austrian Tunneling Method (NATM)	Pacher et al., 1964	Austria	Tunneling
4	Rock Quality Designation (RQD)	Deer et al., 1967	USA	Core logging, Tunneling
5	Rock Structure Rating (RSR)	Wickham et al., 1972	USA	Tunneling
6	Rock Mass Rating (RMR) Modified Rock Mass Rating (M-RMR)	Bieniawski 1973 (List modified, 1989-USA) Özkan and Ünal, 1990	South Africa Turkey	Tunnels, Mines, (Slopes, Foundations) Mining
	Rock Mass Quality (Q)	Barton et al., 1974 (Last modified 2002)	Norway	Tunnels, Mines, Foundations
8	Strength- Block Size	Franklin, 1975	Canada	Tunneling
9	Rock Mass Strength (RMS)	Stille et al., 1982	Sweden	Metal Mining
10	Unified Rock Mass Classification System (URMC)	Williamson, 1984	USA	General Communication
11	Weakening Coefficient System (WCS)	Singh, 1986	India	Coal Mining
12	Basic Geotechnical Classification	ISRM, 1981	International	General
13	Geological strength index (GSI)	Hoek et al. 1995		Mines and Tunnels

Table 1.
Most widely used rock mass classification systems [6, 10].

3. Blocky and seamy rock: Such rocks consist of chemically intact or almost intact rock fragments which are totally detached from each other and erroneously joined. In such rock, vertical walls may need sides support.

4. Crushed rock: such rocks are chemically intact rock but have the characteristic of crusher outing. If maximum or completely all the fragments are as small as fine sand particles and no cementation has taken place, crushed rock below the water table demonstrate the properties of water-bearing sand.

5. Squeezing rock: Squeezing rock gradually progresses into the tunnel without noticeable increase in volume. An obligation for squeeze is a high percentage of microscopic and sub-microscopic elements of micaceous minerals or clay minerals with a low swelling capability.

6. Swelling rock: Such rock moves inside the tunnel mainly because of expansion. The capability to swell seems to be insufficient to those rocks that have clay minerals such as montmorillonite, with a high swelling capability.

4.1.2 Classifications containing stand-up time

Lauffer (1958) anticipated that stand up time for an excavation span is associated with the quality of rock mass in which the width is mined. The Unsupported span may be defined as the width of the tunnel or the distance between the face and the adjacent support, if such is grater that the tunnels width. Laufer's (1958) advanced classification has been improved by various researchers especially Pacher et al., (1974) and currently formulae the part of the worldwide tunneling attitude so called the New Austrian Tunneling Method (NTAM). The importance of the standup time is to increase in the tunnel width results in a substantial decrease in the period available for the fixing of support. The NATM comprises numerous systems for workable, safe and stable excavation in rock situations where the stand-up time is restricted before collapse occurred. These systems are:

- The use of small headings and benching
- The use of several small drifts to form a reinforced ring inside which the unpackaged of the tunnel can be mined

As described by Terzaghi (1946), these practices are appropriate to apply in squeezing soft rock mass i.e. shale's, phyllites and mudstones. The practices are also appropriate when tunneling in exceptionally jointed rock, but needs excessive attention to apply these practices to underground excavations designed in hard rocks having dissimilar failure mechanisms. For hard rock excavation support design, it is practical to accept the assumption that the stability of the rock mass adjacent to the underground excavation is not time-dependent. A defined wedge visible in the roof of an excavation will fall as soon as after excavation. This can happen after blasting or during the succeeding scaling process. Early support is demanded do keep such a wedge in place, or to improve the limit of safety preferably before the rock supporting the full wedge is removed. On the other hand, in a highly stressed rock condition, failure will generally be induced by some change in the stress condition adjoining the excavation. The failure may occur gradually and apparent it as spalling or it may occur rapidly in the form of a rock burst. In either case, the support system design must take into account the modification in the stress condition rather than the 'stand-up' time of the excavation.

4.1.3 Rock quality designation index (RQD)

It is developed by Deere et al., (1967). Such system provides the quantities estimation of rock mass quality from the drill core logs. RQD is defined as the percentage sum of all intact core pieces having length more than 10 cm in the total length of the core provided that the core should be of NX size (54 mm in diameter). The precise practices for the estimation of the size of core portions and the approximation of Rock Quality Designation Index are summarized as shown in **Figure 2** [11].

In 1982, Plastron suggested that when core is not available and discontinuity traces are visible in surface disclosure or exploratory adits, the RQD may be calculated from the number of discontinuities per unit volume. The suggested relationship is for clay free masses and is given below by Eq. (2).

$$RQD = 115 - 3.33\,J_v \tag{2}$$

Where,
RQD is the Rock Quality Designation Index,
J_v is the number of all joints per unit length for all joint (discontinuity) sets, so called volumetric joint count.

4.1.4 Rock structure rating

Wickham et al. (1972) established another quantitatively rock mass classification system termed as Rock Structure Rating (RSR). RSR is used to describe and measure

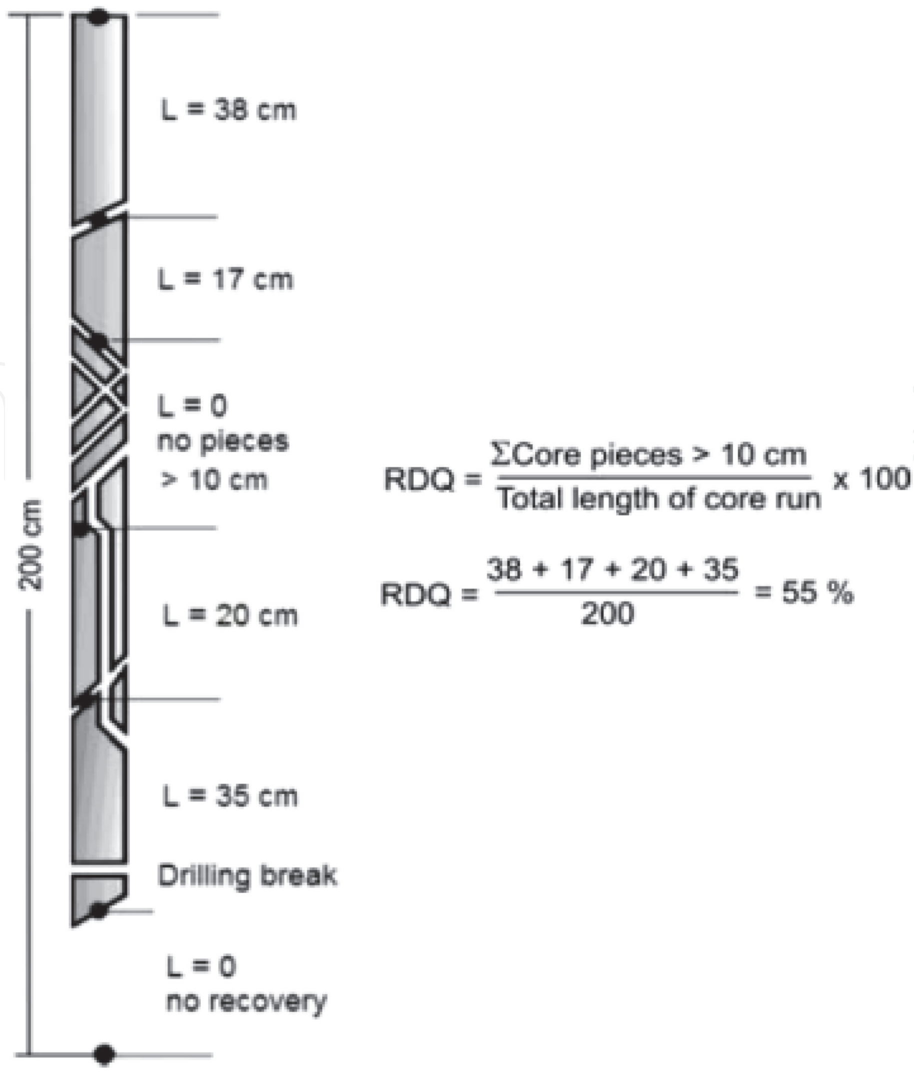

Figure 2.
Procedure for measurement and calculation of RQD [11].

the quality of rock mass for selecting of appropriate support and reinforced system. Such classification system not applied generally as compared to other classification systems, but it has its important role in the emergent of other empirical classification schemes. Many investigators advised that for good, reliable and suitable results for planning of excavation more than one rock mass classification systems should be used at initial stage of the project. The significance of the rock structure rating, in the context of this conversation, is to bring forward the idea of assessment of each of the constituents recorded below to calculate a mathematical value of RSR = A + B + C.

Where,

Factor A: Area Geology: It includes Common evaluation of geological structure based on:

- Rock type Origin (sedimentary, metamorphic and igneous).
- Rock Hardness (it means hard, medium, soft and decomposed).
- Geologic structure (immense, marginally faulted/folded, reasonably faulted/folded, extremely faulted/folded).

Factor B: Geometry of the geological structures: it consists of effect of disjointedness arrangement with consideration to the tunnel alignment on the basis of:

- Joint spaces.
- Orientation of joints (dip and strike).
- Direction of tunnel drive.

Factor C: it includes influence of groundwater inrush and joint situation on the basis of:

- Whole rock mass class based previous parameter combined (A and B).
- Situation of Joint (poor, fair and good).
- Quantity of water flow (gallons/minute/1000 feet of tunnel).

The following tables are used for the calculation of RSR (maximum RSR is 100) [9] (**Tables 2–4**).

	Basic Rock Type				**Geological Structure**			
	Hard	**Medium**	**Soft**	**Decomposed**				
Igneous	1	2	3	4		Slightly	Moderately	Intensively
Metamorphic	1	2	3	4		Folded or	Folded or	Folded or
Sedimentary	2	3	4	4	Massive	Faulted	Faulted	Faulted
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.
Rock structure rating, parameter a: General area geology [9].

	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		
Average joint spacing	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	33	34

[a] *Dip: flat: 0–20°; dipping: 20–50°; and vertical: 50–90°.*

Table 3.
Rock structure rating, parameter B: Joint pattern, direction of drive [9].

	Sum of Parameters $A + B$					
	13–44			45–75		
Anticipated water inflow gpm/1000 ft. or tunnel	Joint Condition[a]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	26	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

[a]Joint condition: good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open.

Table 4.
Rock structure rating, parameter C: Groundwater, joint condition [11].

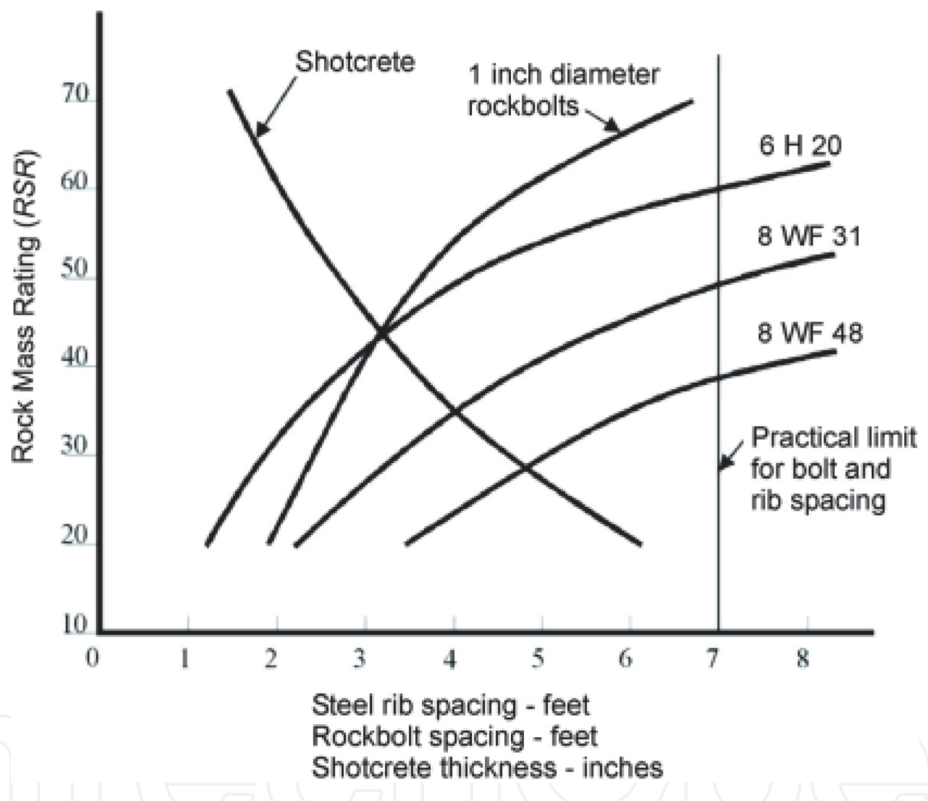

Figure 3.
RSR support recommendation chart [9].

The RSR value calculated for the above tables are then used for the calculation support system recommendation. The support recommendation chart for the RSR value is given in **Figure 3**.

4.1.5 Rock mass rating system (RMR system)

The rock mass rating system was produced by Biniawski in 1976; it is sometimes also called geo-mechanics classification system. It was developed taking into account the distinctive case histories in the field of structural designing This classification system was altered in 1974, 1976, 1979 and 1989, because of considering of more contextual analyses identified related to tunnels, mines, chambers, slopes and foundations [1]. The Geo-mechanics classification system has a widespread application in different rock engineering fields such as mining, hydro power projects, tunneling and hill slope stability (Kumar S. S., 2012). The geo-mechanics classification incorporates the following 6 parameters that are computable in the site and from cores [6]:

1. Uniaxial compressive strength

2. Rock quality designation (RQD)

3. Spacing of discontinuities

4. Condition of discontinuities

5. Ground water condition

6. Orientation of discontinuities

While using this classification system, the rock masses are divided into a number of structural regions. Each region is classified independently [12]. These six parameters are being given different rating based on different geological and geotechnical condition as shown in **Table 5**.

Based on the overall rating of RMR calculated form above mentioned parameters support systems are being recommended for the project site. Support recommendation based on RMR value is given in **Table 6**.

4.1.6 Q-system

This system of rock mass classification was devised by Barton et al., (1979) in Norwegian Geotechnical Institute (NGI), explicitly for the design of tunnel established on 212 case histories. The rock mass classification system is generally used for tunnel design throughout the world and has been used in approximately 1260 various projects and considered as one of the best classification systems for design of tunnels (Kumar N., 2002). The extreme ratings of Q-System shows good quality of rock mass and the lowest ratings designate poor quality of rock mass. The minimum and maximum of Q-index ranges from 0.001 to 10000 on logarithmic scale. According to this classification system Q is the function of six independent parameters as defined by Eq. (3).

$$\mathrm{Q} = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF} \tag{3}$$

Where,

RQD Rock Quality designation index, *Jn* shows joint set number, *Jr* shows number of joint roughness estimated for the set of joint that is most terrible and dangerous to alignment of tunnel, *Ja* show joint alteration number estimated for the most dangerous and unfavorable set of joint along the alignment of tunnel, *Jw* is joint water condition which shows the water reduction factor, Stress Reduction Factor, SRF is comprised to consider the consequence of in-situ stress condition on the whole quality of Rock. The following comments are offered by Barton et al. (1974) for explaining the meaning of the parameters used to decide the value of Q.

The first quotient $\left(\frac{RQD}{Jn}\right)$ demonstrating the organization of the rock mass, is a rough measure of the block size.

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 - unlaxial compressive test is preferred		
		Unlaxial 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 (Mm)	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 favorable	Favorable	Fair	Unfavorable	Very Unfavorable
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)		<1 m	1–3 m	3–10 m	10–20 m	>20 m
Rating		6	4	2	1	0
Separation (aperture)		None	<0.1 mm	0.1–1.0 mm	1–5 mm	>5 mm
Rating		6	5	4	1	0
Roughness		Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating		6	5	3	1	0

Infilling (gouge)	None	Hard filling < 5 mm	Hard filling >5 mm	Soft filling <5 mm	Soft filling >5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	6	5	3	1	0
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING[**]					
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 favorable	Favorable			Very unfavorable	Fair
Drive against dip - Dip 45–90°	Drive against dip - Dip 20–45°			Dip 0–20 - Irrespective of strike°	
Fair	Unfavorable			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 [5].

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 boiling.		
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 wi1n 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 lop 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 the RMR system [1, 6].

The second quotient $\frac{Jr}{Ja}$ communicates the unevenness and frictional features of the joint walls or infill materials. This measure is taken in favor of uneven, unchanged joints in direct interacted. The strength is reduced significantly in case where rock joints have coating of thin clay mineral and fillings. It defines the inter – block shear strength of rock mass.

The third quotient $\frac{Jw}{SRF}$ incorporates two stress related parameters. SRF is a degree of 1) untying load when the excavation passes through clay bearing rock and shear zones, 2) rock stress when the excavation is within competent rock, and 3) squeezing loads in plastic weak rock masses. It is also as a total stress parameter. The Jw parameter is amount of water pressure, adversely affect the shear strength of joints as it reduces the effective normal stress. In addition, presence of water may create softening and ultimately the possibility of outwash when clay infill the joints. It generally shows the active stress component and that is determined empirically. The comprehensive and detail system of determining the values of the Q-System parameters (Rock quality designation *(RQD)*, Number of joints *(Jn)*, Roughness number for joint *(Jr)*, Joint alteration number *(Ja)*, Joint water reduction factor *(Jw)*, Surface reduction factor *(SRF)* are given in **Tables 7**–**12**. The extreme value exemplifies good class of rock and the inferior value signifies poor class of rock.

The values achieved for the different parameters using the above cited tables are then used for the determination of the value of the Q- system. Based on the Value of Q-System the Bortan et al. (1974) classify the quality of rock into nine different groups as shown in **Table 13**.

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 [13].

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. Far portals, use 2 Jn.

Table 8.
Joint set numbers (Jn) values [13].

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

3	Jr values	Jr
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) values [13].

4	Ja values	φ_T 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 5 mm Continuous thickness).	16–24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (less than 5 mm continuous thickness).	12–16°	8
I	Swilling clay fillings, i.e., montmorillonite (less than 5 mm 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 [13].

High professionalism is required for estimation of the values of parameter used in this system. The poor professional users may face trouble while approximating the score of the parameters and may approximate the lesser value for Q-System, which is considered the weakness of this classification system [14].

The width and altitude of the underground excavations mainly depend on the class of rock mass and considered as significant elements in design of underground excavations. The facet of width or altitude directly disturbs the stability when amplified or declined. To highlight the safety obligation, Bortan et al. (1974) further

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 [13].

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 day-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 50 m.			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 50 m			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		σc / σ1	σΘ / σc	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 ≤ σ1 / σ3 ≤ 10 reduce σc to 0.8 σc, and σΘ to 0.8 σΘ, when σ1 / σ3 > 10 reduce σc to 0.5 σc, and σΘ to 0.5 σΘ. 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			σΘ / σc	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	

6	SRF values	SRF
N	Mild swelling rock pressure	5–10
O	Heavy swelling rock pressure	10–15

Table 12.
Stress reduction factor (SRF) values [13].

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 based on Q-system [13].

carry the addition of a fresh parameter to Q-System named as excavation support ratio (ESR). The lower value of ESR symbolizes the necessity of great level firmness and vice versa. The ESR is used for the estimation of support system that can be set up to sustain the stability and also associated to the anticipated use of excavation. Incorporating various conditions, different values of ESR are summarized in **Table 14**. Based on the width and altitude of underground excavation, ESR shows the Equivalent dimension that is achieved by means of the Eq. (4) [13].

$$De = \frac{\text{width or altitude in meter}}{\text{ESR}} \tag{4}$$

The support chart proposed by Bortan et al. (1974) as shown in **Figure 4**, is based on the Q-system ratings and equivalent dimension for the endorsement of permanent support system for underground excavations. This chart provides a

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) [13].

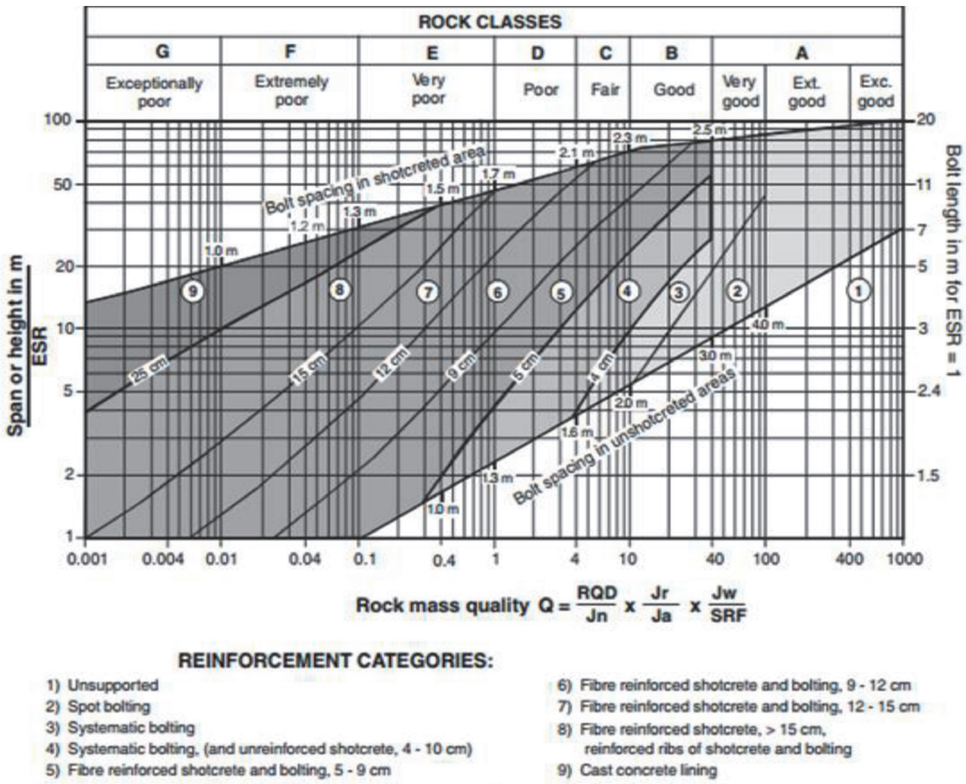

Figure 4.
Permanent support system recommendation chart for Q-system [13].

wide-ranging framework established on the empirical data that what kind of support system is recommended in case of rock bolt's center to center spacing and the thickness sprayed concrete, and also give the energy absorption of fiber strengthened sprayed concrete.

4.2 Geological strength index (GSI)

This classification system established and improved by Hoek and other researchers including the block size and its shear strength in order to estimate value of GSI quantitatively. The GSI index value for any rock mass is depend on the estimation techniques, expertise and reliability of these two input parameters. Sonmez and Ulusay developed the arithmetical basis for GSI value calculation and present quantitatively GSI chart as given in **Figure 5** [16]. Further research were carried out for quantification of GSI value by (Cai, et al.,2004), they present the assessment method for block size, joint and joints wall condition for GSI value quantification.

GSI system should not be considered as the replacement for other classification systems like RMR and Q-System, as this system cannot recommend any support system for stability of rock mass. This system can only be used in estimation of rock mass properties and input parameters for numerical modeling [15]. The comprehensive practice for estimation of input parameters for numerical analysis of stress condition and the remedial measures is presented in **Figure 5** (Hoek, 2013).

The GSI index may be estimated by subsequent various methods used for assessment of rock mass.

Method A: Using this method the GSI is estimated by skilled geologist or mining engineers from the data collected (observational data) at site and then the value of GSI is evaluated from chart [17].

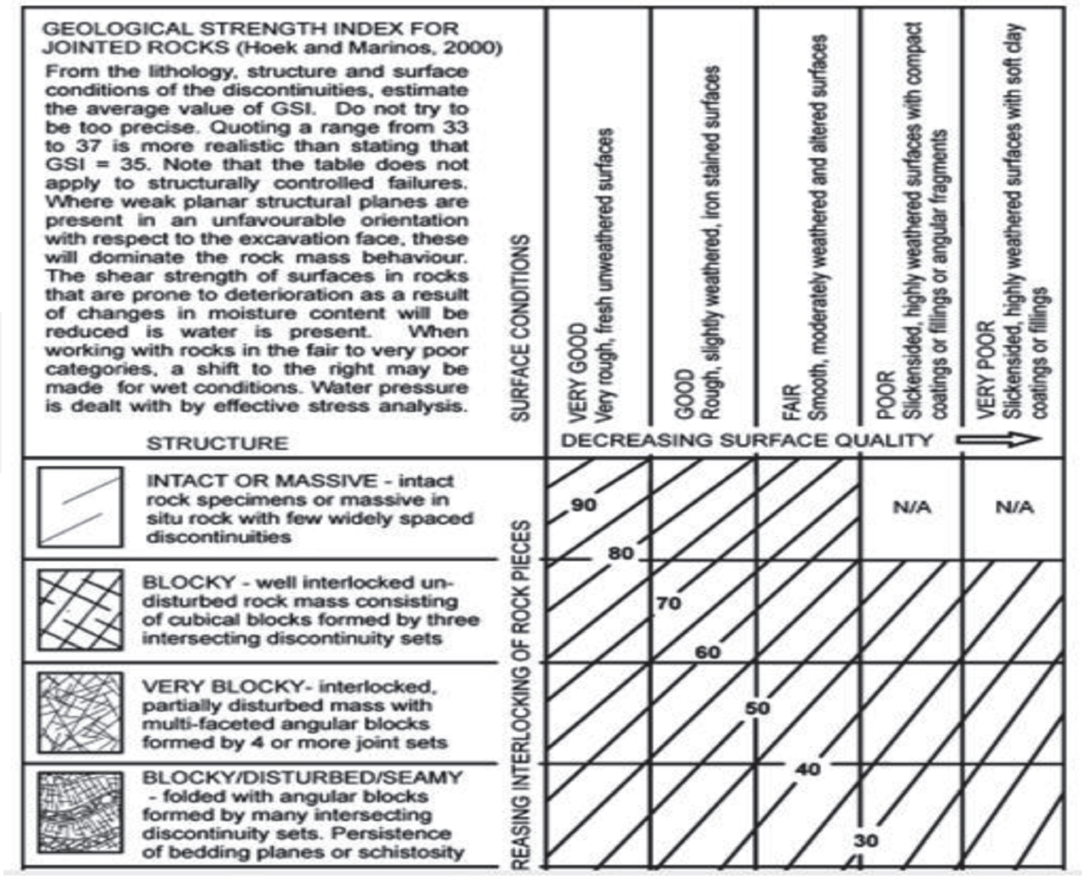

Figure 5.
Geological strength index chart [15].

Method B: In this method the GSI index is estimated by using other classification systems like RQD and RMR etc. when limited data is available. The GSI can be estimate from the well-known relationship presented by various researchers [17].

Method C: The sonmez and Ulusay considered structure rating (SR) and surface condition rating (SCR) for approximation of GSI value [17].

The Cai et al. (2004) used block volume (V_b) and joint surface condition factor (J_c) to approximation the GSI. The block volume having greater number of joint sets indicated as:

$$Vb = \mathrm{S1} \times \mathrm{S2} \times \mathrm{S3} \quad (5)$$

where, S is joint spacing.

The Jc defined by the roughness of joint, weathering and infilling, these are used to measure the joint surface condition factor by using the Eq. (6).

$$Jc = \mathrm{Jw} \times \mathrm{Js}/\mathrm{Ja} \quad (6)$$

The V_b and Jc are used to precisely quantify the GSI value [17]. The quantitative chart for estimation of GSI suggested by sonmez and Ulusay [1999] is shown in **Figure 6**.

5. Numerical methods of design

The empirical methods of design do not estimate accurately the reliability supports, redistribution of stresses, rock mass deformation [18]. These parameters are very important in designing and analysis of any excavation therefore, numerical

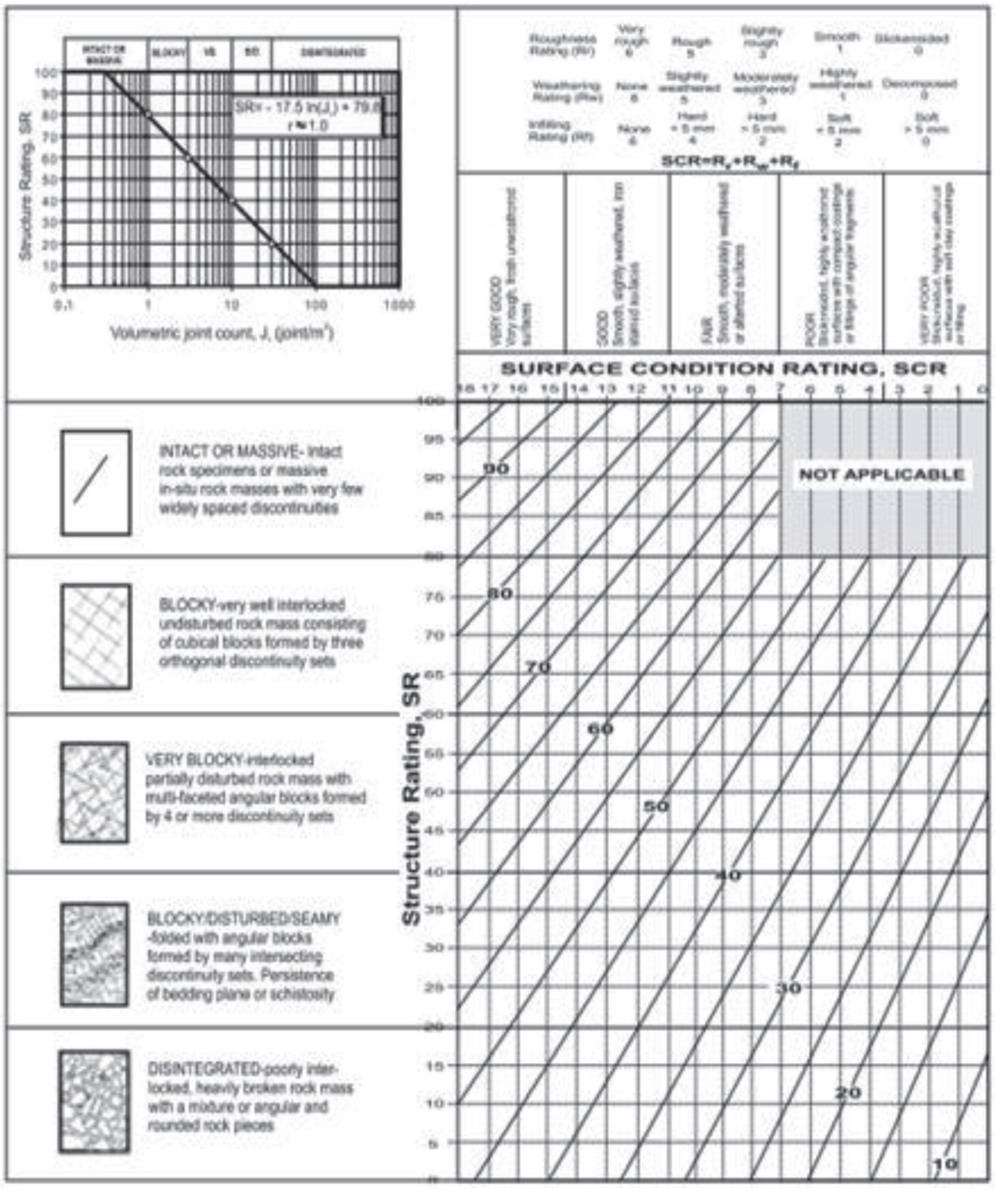

Figure 6.
Quantitative estimation of GSI chart [15].

analysis should be carried out for appropriate designing. The numerical methods are considered very useful to estimate the above parameters precisely and in minimum time as compared to other methods of design. Numerical methods used physical and strength properties of rock as input for analysis. For efficient and viable design the numerical and empirical methods are used in parallel [19–23].

Different researchers developed and present various numerical methods and models. These are divided into eight classes on the basis of four methods and two levels as shown in **Figure 7** [24, 25].

5.1 Numerical methods of modeling for rock/soil engineering

The numerical methods of design uses in rock/soil engineering are grouped into three classes for modeling in rock mechanics as discussed above.

5.1.1 Continuum methods

The different continuum methods of design are as under.

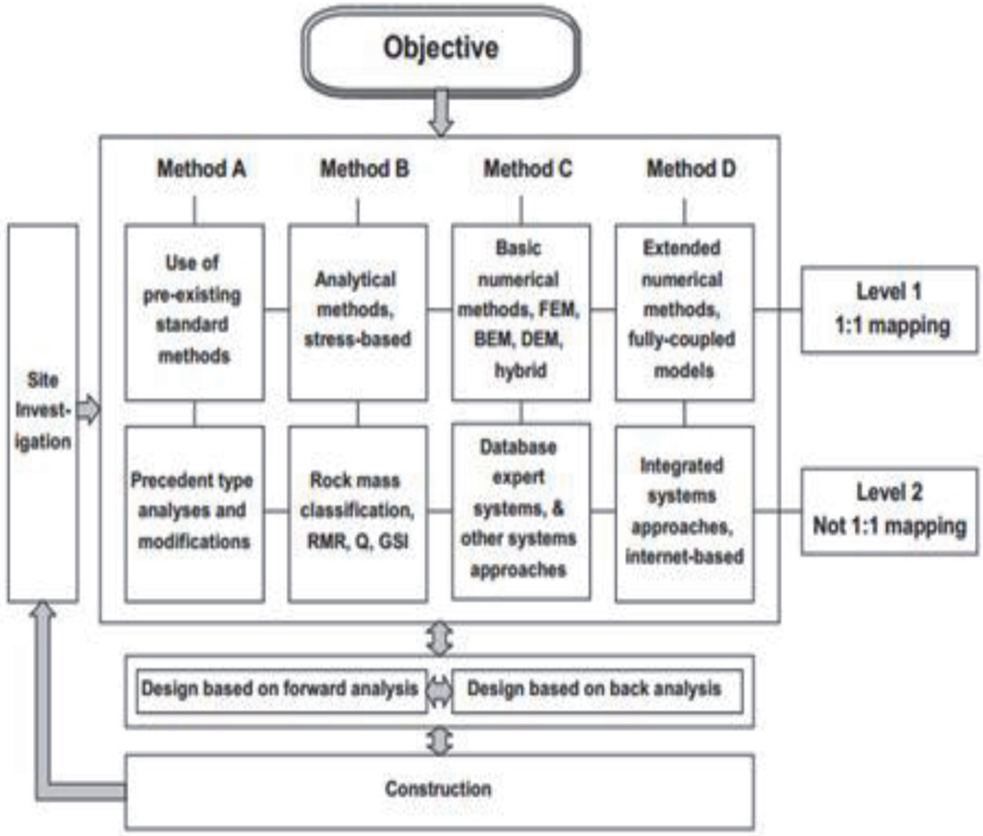

Figure 7.
Division of numerical models and methods [24, 25].

1. Finite Difference Method (FDM)

2. Finite Element Method (FEM)

3. Boundary Element Method (BEM)

Finite Difference Method (FDM).

The Finite difference method (FDM) is the direct calculation of PDEs and transmitted the creative PDEs in term of unknown at grid point into a system of algebraic equations by interchange the fractional derivatives with difference at irregular or regular grid forced over problem areas. This system is solved due to establishing the required initial and boundary condition. This method is old but widely applied in the numerical modeling in rock mechanics. This method is based for explicit approach of discreet element method (DEM) [26].

Finite Element Method (FEM).

The Finite element method (FEM) splits the problem into sub-elements of smaller sizes and shapes with fitting the number of nodes at the vertices and at the side of discretization. FEM is mostly used to estimate the behavior of PDEs at elemental level and for signifying the behavior of elements; it produces the local algebraic equation. After creating the local equation the FEM gathered it according to topographic relation of node and elements and further put it into worldwide system of algebraic equation for receiving the required information after establishing the definite initial and boundary situations.

Boundary Element Method (BEM).

The Boundary element method is the precise method then FEM and FDM because of its easiness. This method involves the discretization of solution areas at boundary and thus decreases the problem dimension by simplifying the design input parameters. This method computes separately the essential information in the solution domains from the information at the boundary, which is achieved by the solution of boundary integral equation rather than direct solution of PDEs [26].

5.1.2 Discontinuum methods

The different discontinuum methods of design are given below.

1. Discrete Element Method (DEM)

2. Discrete Fracture Network (DFN)

5.1.3 Hybrid continuum/Discontinuum

Following are the different Hybrid continuum/discontinuum methods of design:

1. Hybrid FEM/BEM methods

2. Hybrid DEM/DEM methods

3. Hybrid FEM/DEM methods

4. Other hybrid method/models

6. Finite element method (FEM)

This method of design was developed by Clough et al., (1950). Due to wide application of this method in mining engineering especially tunneling, it get more attention for solving mining problems and popularity in this field [19]. The FEM divide problem into small parts and connect these parts at a point/nodes at the apexes and at the boundaries of meshing/discretization. The FEM has many applications in modeling in rock engineering design due to dealing with nonlinearity, boundary conditions and heterogeneity problems [26, 27].

The unidentified function over each element in FEM estimated through test function having its nodal values of anonymous system (in polynomial form). This practice is the fundamental supposition of FEM. For experimental function, it is mandatory to satisfy the principal of PDFs. In this research the FEM based software Phase2 was used for analysis of stresses and total displacement around tunnel. For experimental function it must be satisfied the principal of PDFs, which is given in Eq. (7).

$$\mathrm{u}_i^e = \sum_{j=1}^{M} N_{ij} \mathrm{u}_i^e \tag{7}$$

Where,

Nij is the shape function or interpolation function; this must be defined into inherent coordinates for use of Gaussian quadratic integration, M is the element order.

Using shape function the problem original PDFs can be substituted by the arithmetical equation as given below.

$$\sum_{j=1}^{N} \left[K_{ij}^e\right] \left\{\mathrm{u}_i^e\right\} = \sum_{j=1}^{N} f_i^e \quad or \;\; Ku = F \tag{8}$$

Where,

Keij is the coefficient matrix, u_i^e vector is the nodal value vector having unidentified variables, f_i^e is consist of body force contribution and initial boundary condition, K is the global stiffness matrix.

Keij is also called the element stiffness matrix in term of elasticity problem which is given by Eq. (9).

$$\left[K_{ij}^e = \int_{\Omega i} ([B_i][N_i])^T [D_i][B_i] d\Omega \right. \tag{9}$$

Where,

D_i is the elasticity matrix; B_i is the geometry matrix which is determined from the relation between displacement and strain.

In FEM the material properties of different materials can easily feed into FEM by assigning different properties to different elements distinctly.

6.1 Finite elements

The element may be in numerous forms i.e. one dimensional, two dimensional and three dimensional elements. One dimensional element having cross-sectional area and usually denoted by line sections or segment. Two dimensional element fields consist of triangle and quadrilateral. Three dimensional element field described by tetrahedron and parallelepiped. Some element shapes and node position used in two dimensional element fields [28] (**Figure 8**).

6.2 Shape function

It is the displacement within the element at any point when related to the displacement of the nodes. For instance the displacement of u and v within the quadrilateral element at any point represented by Eq. (10).

$$\begin{vmatrix} u \\ v \end{vmatrix} = \begin{bmatrix} N1 & 0 & N2 & 0 & N3 & 0 & N4 & 0 \\ 0 & N1 & 0 & N2 & 0 & N3 & 0 & N4 \end{bmatrix} \begin{bmatrix} u1 \\ vi \\ u2 \\ v2 \\ u3 \\ v3 \\ u4 \\ v4 \end{bmatrix} \tag{10}$$

Where,

u1, v1… .*u4, v4* are nodal displacement and *N1-N4* are shape function and that are connected with the nodes 1–4 correspondingly.

6.3 Coordinate transformation

The shape function is additionally used for coordinate's alteration of element in order to simplify the integration for calculation of stiffness matrix of some quantities for element. The coordinates (x, y, z), within the element of a point represented by Eq. (11) [28].

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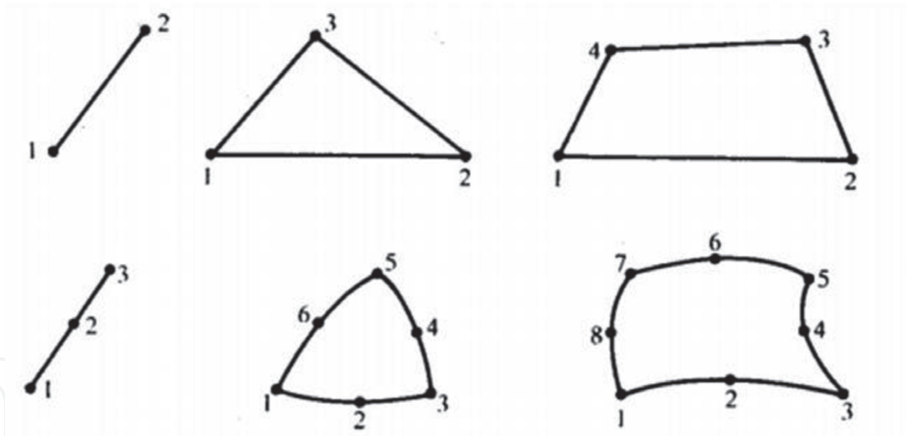

Figure 8.
Some element forms and node position used in two dimensional [28].

$$x = \sum_{i-1}^{n} N_i x_i$$
$$y = \sum_{i-1}^{n} N_i y_i \qquad (11)$$
$$z = \sum_{i-1}^{n} N_i z_i$$

6.4 Relation between strain and displacement

For two dimensional element domain the relation between strain and displacement represent by Eq. (12) [28].

$$\varepsilon = \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \\ \varepsilon_z \end{bmatrix} = B \begin{bmatrix} u1 \\ v1 \\ u2 \\ v2 \\ \dots \\ un \\ vn \end{bmatrix} \qquad (12)$$

6.5 Relation between stress and strain

It may express as:

$$\Delta\sigma = D_T \Delta\varepsilon \qquad (13)$$

Where,

$\Delta\sigma$ is the vector of stress components, $\Delta\varepsilon$ represents corresponding components of strains and DT is a square matrix that is constant in the elastic case.

6.6 Global stiffness matrix

It is formed when added the stiffness matrices of all elements. The equation for global stiffness is given as:

$$K\Delta\delta = \Delta R \tag{14}$$

Where.

$\Delta\delta$ is unknown vector having increments of nodal displacement due to increment force ΔR.

For material linear elastic material behavior the equation may by write as (Scheldt, 2002).

$$K\delta = R \tag{15}$$

6.7 Finite element based software's

Following are finite element based software's.

1. Displacement Analyzers Finite Element program (DIANA) software is developed by TNO Building and Construction Research, Netherlands. It is a flexible software and used in solving of linear and nonlinear structural engineering in 2D and 3D [28].

2. Phase2 developed by rock science for solving 2D non-linear problems like analysis of displacements and stresses around underground openings, in the field of mining and civil engineering [29].

3. ABAQUS software is developed by Hibbitt et al. (1978) in USA. It is used for linear and non-linear, problems and analyzes the stresses of any structure in 3D [28].

4. ANSYS software is developed for solving both linear and non-linear problems for isotropic and non-isotropic properties of materials [28].

7. Conclusion

Engineering design is the valuation using knowledge of basic sciences, mathematics and engineering sciences to convert resources optimally to meet quantified objectives. Its goal is to develop a solution to a known problem. There are different stages of design process; one can adapt it to the particular problem for solving it. We have variety of design techniques in rock engineering. They are classified in to three groups i.e. are Analytical, Empirical and Observational. Among, these empirical approaches can effectively be used for engineering judgment. Rock mass classification is one of the widely used empirical methods for the assessment of the rock mass behavior. The empirical methods of design do not estimate accurately the reliability of support systems, redistribution of stresses and rock mass deformation. Numerical methods are considered very useful to be used for estimate these parameters precisely and in short time as compared to other methods of design. So it is recommended that for efficient and viable design the numerical and empirical methods should be used in parallel for the assessment of soil/rock mass behavior to design any underground structure.

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Conflict of interest

We have no conflict of interest.

Notes/thanks/other declarations

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References

[1] https://www.google.com.pk/search?q=optimization&oq=optimization&aqs=chrome..69i57j69i59j69i60l3j69i61.2975j0j7&sourceid=chrome&es_sm=93&ie=UTF-8

[2] Barton, N., Lien, R., & Lunde, J. (1974). Engineering classification of rock masses for design of rock support. *Rock Mechanics, 189–236*.

[3] Bieniawski, Z. (1984). *Rock Mechanics Design in Mining and Tunneling* . Rotterdam, Netherlands: A. A.Balkema.

[4] Stacey, T. (2004). The link between the design process in rock engineering and the code of practice to combat rock fall and rockburst accidents. *The Journal of The South African Institute of Mining and Metallurgy*, 29–34.

[5] Khandani, S. (2005). *Engineering Design Process.* Saylor.org.

[6] Bieniawski, Z. T. (1990). *Tunnel Design By Rock Mass classifications.* Washington: Department of the army, US army Corpss of Engineer Washington, DC 20314–1000.

[7] Ali, E. W. (2014). *M.Sc thesis, rock mass charecterization for diversion tunnel at diamer basha dam, pakistan-a design perspective.* peshawar: University of Engineering and Technology, Peshawar, Pakistan.

[8] *www. google.com.* (2016, March Thursday). Retrieved 2016 Thursday, 2016, from www. google.com: https://www.rocscience.com/documents/hoek/corner/04_Rock_mass_classification.pdf

[9] Muhammad Tahir. (2014). Prediction performance and Generalization of the emeprical Estimation of Rock mass Deformation Modulus Based on Rockmass Classification Systems. *International Journal of Scientific Engineering and Technology*, 1488–1495.

[10] *www.google.com*. (2015). Retrieved from www.google.com

[11] Karahan, E. (2010). *Design of Excavation and Support System for the Cubukbili Tunnel in Antalya.*

[12] E.Hoek, P. K. (1993). *Support of Underground Excavations in Hard Rock.*

[13] E.HOEK, P. K. (2016, March Wednesday). *www.google.com*. Retrieved March 2016, from www.google.com: http://web.mst.edu/~rogersda/umrcourses/ge341/Rock%20Mass%20Rating.pdf.

[14] (NGI), N. g. (2013). *Using the Q-System.* Sweden and Norway: NGI.

[15] D. Milne, J. H. (1998). Rock mass characterization for underground hard rock mines. *Tunnelling and Underground Space Technology*, 383–391.

[16] Pantelidis, L. (2009). Rock Slope Stability assestment through rock mass calsification systems. *International Journal of Rock Mechanics and Mining Science*, 315–325.

[17] Sonmez, H., & Ulusay, R. (1999). Modifications to the geological strength index (GSI) and their applicability to stability of slopes. *Int J Rock Mech Min Sci., 36*, 763–760.

[18] V. Marinos, P. E. (2005). Gelogical strength index: applications and

[19] Mahmoud Hashemi, S. M. (2010). Application of rock mass charecterization for determining the mechanical properties of rock mass: a comparitive study. *Rock Mechanics, Rock Engineering*, 305–320

[20] H. Basarir, A. O. (2005). Analysis of support requirements for a shallow

diversion tunnel at Guledar dam site, turkey. *Engineering Geology*, 131-[

[21] Bobet, A. (2010). Numerical methods in geomechanics. *The Arabian Journal for Science and Engineering, 35*, Number 1B.

[22] Zulfu Gurocak, P. S. (2007). Empirical and numerical analyses of support requirements for a diversion tunnel at the Boztepe dam site, eastern Turkey. *Engineering Geology, 91*, 194–208.

[23] M. Genis, H. B. (2007). Engineering geological appraisal of the rock masse s and preliminary support design, Dorukhan Tunnel, Zonguldak, Turkey. *Engineering Geology, 92*, 14–26.

[24] Aydin Ozsan, H. B. (2006). Engineering geological investigations along the Ankara subway extension. *IAEG*, paper no. 586.

[25] Rasouli, M. (2009). Engineering geological studies of the diversion t

[26] John A. Hudson, X.-T. F. (2010). Technical auditing of rock mechanics modelling and rock engineering design. *International Journal of Rock Mechanics & Mining Sciences, 47*, 877–886.

[27] L. Jing, J. H. (2002). Numerical methods in rock mechanics. *International Journal of Rock Mechanics & Mining Sciences, 39*, 409–427.

[28] Jing, L. (2013). A reviewof techniques, advances and outstanding issues in numerical modelling for rock mechanics and rock engineering. *International Journal of Rock Mechanics & Mining Sciences, 40*, 283–353.

[29] M. Caia, P. K. (2004). Estimation of rock mass deformati on modulus and strength of jointed hard rock masses using the GSI system. *International Journal of Rock Mechanics & Mining Sciences, 41*, 3–19.