ROCK MASS STRENGTH AND DEFORMABILITY
BEHAVIOUR OF SABALOKA IGNEOUS
COMPLEX

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A Thesis Submitted to the University of Khartoum for the Requirements of Ph.D. Degree in Mining Engineering

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May 2010
Acknowledgement

The author of this thesis wishes to acknowledge his appreciation to his supervisor Associate Prof. Dr. Mohamed Ahmed Osman for giving his time freely to supervise the work of this thesis and for his generous guidance, advices and motivation through out this research, I would like to thank him very much for his support to my field tests work.

I would like also to thank Prof. Dr. Amin A. Abdelrahman for his numerous helpful, discussions, corrections of the manuscript of this thesis.

Thanks go to the geologist Mr. Mohamed Gasmala who showed me how to study the geology in the field.

The author also wishes to express his thanks to the ministry of higher education for financial support to complete the practical work at Assuit University Egypt.

The author is grateful to the faculty of engineering Khartoum University and the Building and Road Research Institute who provided assistant in preparing this thesis.

Thanks go to the staff at the department of mining Engineering Khartoum University.

Finally, I would like to thank every body who was important to the successful realization of this thesis, as well as expressing my apology that I could not mention personally one by one.
ABSTRACT

Sabaloka plateau is located, 80 km north of Khartoum particularly between latitudes 16° 10' 0" and 16° 20' 0" N and longitudes 32° 42' 0" and 32° 4' 0" E. Study of the geological history of this area indicated that the repeated changes in the volcanic activity, alternated with destructive events, caldera collapses, produced a very complex system of basaltic and rhyolite rock masses. The variation in lithology, in the degree of tectonization and disturbance determined a wide spectrum of geotechnical materials, ranging from hard lavas to poorly welded pyroclastic deposits. Quarries, tunnels and other infrastructures were constructed upon these rock masses. This thesis is intended to investigate theoretically and experimentally the rock mass strength and deformability of these rocks in Sabaloka, in order to characterize them for engineering purposes. The estimate of rock mass strength and deformability is reasonably predicted through the use of empirical failure criteria such as the Hoek–Brown failure criterion which has gained broad acceptance in the rock mechanics community, and in situ tests and empirical expressions to predict deformability. The rock mass properties and modulus of deformations of these rocks have been carefully assessed based on laboratory tests (uniaxial compressive, tensile test, triaxial test), and field investigations. The rock mass characterization approaches, Rock Mass Rating (RMR), Rock Mass Quality (Q) and Geological Strength Index (GSI) systems have been applied in this thesis to predict and evaluate the rock mass properties and support design. Numerical modeling studies (RocLab and Examine 2D programs) based on field and laboratory data, have been used to evaluate the performance of these rock masses. Tunnel stability problems were expected in both trachy basalt and vesicular basalt rock masses, hence, the support system was evaluated by means of Q-system which is the most proper one for support design of tunnels. The field and laboratory test results were analyzed, weighed and compiled together to reveal the engineering performance, of these different rock masses in term of strength and rock mass deformation modulus. The analyses of the results have shown that the investigated rock masses would be classified into three categories. Category I possesses very good strength with deformation modulus of 44115 MPa, uniaxial compressive strength of 164 MPa, Geologic Strength Index 72, Hoek constant mI, of 14.5, internal angle of friction $\phi$ of 40°, cohesion c 13 MPa, tensile strength of 1.37 MPa and rock mass strength of 56 MPa. Category II is of good quality with deformation modulus of 7515 MPa, uniaxial compressive strength of 96 MPa, Geologic Strength Index 45, Hoek constant mI, of 12.5, internal angle of friction $\phi$ of 30°, cohesion c 4.74 MPa, tensile strength of 0.121 MPa and rock mass strength of 17 MPa. However, category III is of poor quality with deformation modulus of 586 MPa, uniaxial compressive strength of 48 MPa, Geologic Strength Index 30, Hoek constant mI, of 8.8, internal angle of friction $\phi$ of 24°, cohesion c 1.6 MPa, tensile strength of 0.028 MPa and rock mass strength of 5 MPa.
ناحیه صخره‌ای با ۱۲.۵ MPa نیروی شدید واقع شده است. ناحیه شدید در نتایج آزمایش‌ها به وقوع پیوسته واقع شد. این ناحیه شدید در نتایج آزمایش‌ها به وقوع پیوسته واقع شد. این ناحیه شدید در نتایج آزمایش‌ها به وقوع پیوسته واقع شد.
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\( \sigma_1 \) = major principal stress
\( \sigma_2 \) = intermediate principal stress
\( \sigma_3 \) = minor principal stress
\( \sigma_n \) = normal stress
\( \sigma_c \) = uniaxial compressive strength of intact rock
\( \sigma_{ci} \) = uniaxial compressive strength of intact rock in the Hoek-Brown failure criterion.
\( \sigma_{cm} \) = uniaxial compressive strength of the rock mass
\( \sigma_t \) = uniaxial tensile strength of intact rock
\( \sigma_{tm} \) = uniaxial tensile strength of intact rock
\( \tau \) = shear stress.
\( c \) = cohesion of intact rock or rock mass.
\( c_j \) = cohesion of joint or discontinuity
\( \Phi \) = friction angle of intact rock or rock mass.
\( \rho \) = rock density in kg/m³ or t/m³
\( \gamma \) = unit weight in N/m³
\( E_i \) = intact rock modulus
\( E_m \) = rock mass modulus of deformation
NGI = Norwegian Geotechnical Institute Index (rock mass classification)
RQD = Rock Quality Designation (rock mass classification)
RMR = Rock Mass Rating (rock mass classification)
RMR basic = Rock Mass Rating Basic value (RMR for dry condition and no adjustment for joint orientation).
Q = the rock mass Quality systems (rock mass classification)
GSI = Geological Strength Index (rock mass classification)
N = rock mass Number (rock mass classification)
k = slope of regression line
\( \lambda \) = joint per meter
\( m \) = material constant in the Hoek-Brown failure criterion
\(m_b\) = material constant for broken rock in the Hoek-Brown failure criterion
\(m_i\) = material constant for intact rock in Hoek–Brown criterion
\(s\) = material constant in the Hoek–Brown failure criterion
\(\alpha\) = material constant for broken rock in the Hoek-Brown criterion
\(D\) = Disturbance factor in the Hoek–Brown failure criterion
\(J_n\) = joint set number (parameter in the NGI–index)
\(J_r\) = joint roughness number (parameter in the NGI–index)
\(J_V\) = number of joints / discontinuities per unit length (parameter in the NGI–index)
\(J_a\) = joint alteration number (of least favorable discontinuity parameter in the NGI–index)
\(J_W\) = joint water factor (parameter in the NGI–index)
\(SRF\) = Stress Reduction Factor (parameter in the NGI–index)
\(JRC\) = Joint Roughness Coefficient
\(JCS\) = Joint wall Compressive Strength
\(PMT\) = Pressure meter Test
\(\sigma_{rr}\) = radial stress
\(P_L\) = limit pressure
\(P_y\) = yield pressure
\(\sigma_\theta\) = circumferential stress
\(U_r\) = radial displacement
\(V\) = total volume
\(P\) = corresponding pressure at the end point
\(R_0\) = initial probe radius
\(\Delta R\) = change in probe radius
\(\mu\) = Poisson’s ratio
\(ISRM\) = International Society for Rock Mechanics

Examine 2D = Two-dimensional displacement discontinuity program

PLSI = Point Load Strength Index
CHAPTER ONE

Introduction

1. Background

This is an introductory chapter that intended to delineate the objectives, methodology and conclusions adopted in this research program. It is well known that for any engineering structure constructed within or founded upon a rock mass, the behavior of the rock mass under the applied loads will have important implications upon both the construction costs and operation of the structure. The behavior of the rock mass will be largely dependent on the network of discontinuities within the rock mass. The effect of single discontinuity on rock mass strength and deformation will depend upon the scale of the planned structure in relation to the geometrical properties of the discontinuity.

The probability and consequence of failure may be the same whether the failure surface consists of either one large persistence discontinuity or a number of short interconnected discontinuities. The latter condition is much more difficult to characterize, assess and analyse than the former. Therefore, the difficulties in assessing the condition of a jointed rock mass make it very difficult to estimate the potential strength and deformation properties for use in design. As the demand for infrastructure increases, the frequency of projects built upon jointed rock masses is likely to increase. This will lead to either an increase in the risk of failure or the cost of construction. Greater understanding of the behavior of jointed rock masses is therefore required.

Basalt and rhyolite rock masses are distributed throughout Sabaloka region located 80 km north of Khartoum and are expected to form foundations for various types of structures such as tunnels, roads, and light rise buildings. The data of rock mass properties of these rocks will be essential in order to evaluate the rock mass strength of these rocks and this is will be the subject of this thesis.
1.1 Objective
One of the major problems of designing in rock masses is that of estimating the strength and deformation properties of the in-situ rock mass. Understanding of the behavior of rock mass requires study of intact rock material and of individual continuity surface which go together to make up the system.

In this thesis, initial focus is placed on the strength and deformability of trachy basalt, vesicular basalt and rhyolite intact rock material and joint properties before broadening to the behavior of rock mass. This is typical of many approaches to investigate the strength and deformability properties of rock mass.

1.2 Methodology
Evaluation of rock mass properties can be achieved by use of rock mass classification systems (RMR, Q, GSI etc…) and Hoek-Brown failure criterion, and these three methods will be used in this thesis. Hoek-Brown empirical failure criterion for rock masses is mainly based on triaxial testing of small rock samples (intact rock) and has been verified against test data for rock masses. An extensive program will be launched to determine the in situ and laboratory properties of the different rock types at the study area. Numerical modeling will be applied to evaluate theoretically the behavior of various rock types.

1.2.1 Rock Mass Classification Systems
Empirical rock mass classification systems are commonly recognized as useful tools for the evaluation and prediction of rock masses and the choice of support requirements on the basis of experience in similar geological conditions. Rock mass properties derived from these empirical systems are commonly used, in the preliminary phase, as data input for modeling rock mass behavior. Rock mass classification systems constitute an integral part of empirical underground and surface excavation design. They traditionally used to group areas of similar geomechanical characteristics, to provide guidelines of stability performance and to select appropriate support. In more recent years classification systems have often been used in tandem with an analytical and numerical tools. There has been a proliferation of work linking classification indices to material properties such as modulus of deformation, (m) and (s) for the Hoek –Brown failure criterion. These values are then been used as input parameters for the numerical models. Consequently the importance of rock mass characterization has increased over time.
The primary objective of all classification systems is to quantify the intrinsic properties of the rock mass based on past experience. The second objective is to investigate how external loading conditions acting on a rock mass influence its behavior. An understanding of these processes can lead to the successful prediction of rock mass behavior for different loading conditions. Three classification systems are used for underground and surface excavations. The first system is the Rock Quality Designation (RQD) proposed by Deere et al. (1967). Quite often this is the only information readily available at construction sites. The other two widely used classification systems are the Norwegian Geotechnical Institutes Q system, Barton et al. (1974) and the various versions of the Rock Mass Rating system (RMR), originally proposed by Bieniawski (1973). Interestingly, both systems trace their origin in tunneling. Furthermore both systems use RQD as one of these constitutive parameters. A summary of the roles that rock mass classification systems play is presented in Figure 1.1. The focus of this study is on the use of the rock mass classification systems RMR, Q, and GSI to estimate the properties of rock masses.

Figure 1.1 Application of rock mass classification systems in rock mechanics and rock engineering (After P.K. Kaiser, 2004).
1.2.2 Hoek – Brown failure criterion

In order to evaluate the rock mass parameters, the Hoek and Brown (1980) rock mass strength criterion, updated by Hoek, Marinos and Beniss (2002), was adopted.

\[
\sigma'_1 = \sigma'_3 + \sigma_{ci} \left( m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a
\]

Where \(\sigma_1\) and \(\sigma_3\) are the maximum and the minimum effective principal stresses at failure, \(m_b\), \(s\) and \(\alpha\) are the values of Hoek-Brown constants for the rock mass, \(\sigma_{ci}\) is the uniaxial compressive strength of the intact rock.

This criterion is a widely accepted method by the rock mechanics community for analysis of rock failure criterion, and is primary a modification of the proven Mohr Coulomb failure criterion, using empirical relationship derived from extensive field research. The derivation has been updated by Hoek et al in (2002) edition, proceeding further research. As the Hoek-Brown criterion modifies the Mohr Coulomb theory according to the properties of the rock mass through which the excavation is made, the values of \(m_b\), \(s\) and \(\alpha\) are determined according to a rock classification index (GSI) developed for characterizing the rock mass. The geological strength index (GSI) was developed to provide for local rock properties by Hoek and Marinos (2000) replacing the rock mass rating (RMR). \(m_b\) is the modification of the existing Hoek – Brown constant \(m_i\) and \(\sigma_{ci}\) are obtained from undrained triaxial testing using standard laboratory equipment. For consistency with research conducted by Hoek and Brown resulting in the derivation of the empirical constants, the triaxial tests should be conducted a minimum of five times over a range of \(0 < \sigma_3 < 0.5 \sigma_{ci}\). The values of \(\sigma_{ci}\) and \(m_i\) may then be determined in the method detailed by Hoek (2000). To apply the Hoek-Brown failure criterion the value of \(m_i\) can be used to determine the modified values of \(m_b\), \(s\) and \(\alpha\) as given by the equations of Hoek (2000).
In the above equations the factor $D$ is known as a disturbance parameter where in an undisturbed rock mass, $D$ approaches zero and in extremely disturbed ground approaches one. It should be noted from the equations that as the value of GSI approaches 100 and the value of $D$, approaches zero, the strength of the rock mass $\sigma_{cm}$ approaches the uniaxial compressive strength of intact rock $\sigma_{ci}$. Using the classification systems and failure criterion, the stability of the rock mass can thus be determined.

1.2.3 Sampling and field tests
The site investigation program was designed to provide detailed information to estimate rock mass properties. Geo mechanical data determined in the field include the following:
- The Schmidt hammer test
- Fracture characteristics- Fracture orientation and data were collected along scan lines in order to develop a model for fracturing in Sabaloka igneous complex. Information of fracture sets is essential for rock mass classification and is necessary for developing a geo mechanical model of the rock mass.
- Data on fracture roughness and infilling, and other parameters needed for input to rock mass classification were collected along scan lines.

The field work program included several visits to Sabaloka site. Drilling of boreholes was carried out. During drilling core boring Nx size is performed, and samples were collected for laboratory tests.

1.2.4 Laboratory tests
A number of laboratory tests were carried out on the rhyolite and basaltic rock cores, including unit weight ($\gamma$), compressive strength (UCS) evaluated by three different test
methods (1) Unconfined compressive strength test (2) By correlation with the Schmidt hammer rebound value, and (3) By point load test following the standard procedure of (ASTM D5731 (2001)). Triaxial tests were carried out on the rhyolite and basaltic rock from Sabaloka. One of the most convenient methods of determining triaxial strength of rock is to apply a uniform hydraulic pressure to the curved surface of a cylindrical specimen and then to apply a compressive axial load to the rock specimen until failure occurs. Tests were done on series of samples under different confining pressures to study the strength properties of the sample under various stress conditions which simulate natural or arranged levels. Test data were used to determine the strength and elastic properties of rock, the angle of shearing resistance, cohesion and deformation modulus. The cell illustrated in figure 1.2 was used for rock strength estimates.

![Figure 1.2 cutaway view of triaxial cell](image)

1.3 Thesis Outline

This chapter presents a general introduction objective and the methodology of the study. Chapter 2 described the geology of the study area. This is followed in chapter 3 by a literature review discussing the estimation of the rock masses, the factors affecting the rock masses, evaluation of the intact rock strength, review of rock mass classification and review of Hoek-Brown failure criterion. Chapter 4 covers the methodology of the study
including field data collection and laboratory testing and the results of field and laboratory strength measurements and how the measurement of rock mass strength parameters provide the foundation for the geotechnical characterization of the rock unit present within the study area. In chapter 5 the analysis of representative rock mass strength was addressed through the use of the Hoek-Brown failure criterion in conjunction with rock mass classification. Finally in chapter 6 conclusions and recommendations for further research are presented. References are located at the end of this thesis. This thesis includes also appendices with more detailed information on rock mass classification systems, and Hoek-Brown rock mass failure criterion, detailed information from the tests, and detailed information of the rock mass strength estimation performed by the author.
Chapter Two
Geology of the study area

2.1 Introduction

The following literature review concerning the Sabaloka igneous complex is based on previous studies presented by Almond D.C., 1977 and Almond D.C., and F.M. Ahmed.1993. Sabaloka is one of the large group of anorogenic igneous complexes in sudan characterized by predominance of acidic rocks, shallow level of emplacement, the common present of ring structures, and a tendency to include perakaline variants. These complexes are conveniently referred to “Younger Granites”. Sabaloka is one of the most accessible since it lies astride the Nile at the Six Cataract, only 80 km north of Khartoum. The complex is noteworthy for the wide array of volcanic rocks preserved within a cauldron subsidence. Table 2.1 summarizes the geological sequence of Sabaloka area.

2.2 Location of the study area

In Sabaloka region the study area is located at the northeast part of the plateau which called the (northeast scrap). This area provides a useful profile through the volcanic rocks in the middle section of the cauldron complex. The cauldron volcanic rock consists of trachy basalt and flow-banded lava. The northeast scarp of the sabaloka plateau is 6 km long and up to 500 m above mean sea level. It exposed a total thickness of 200m of volcanic rocks below the base of the plateau ignimbrite, resting on a penplained surface of basement rocks. The almost straight course of the scarp suggests a fault line, but no confirmatory evidence of displacement has been found .Extensive alluvial fans sweep down from the foot of the scarp towards the Nile, skirting the small Nubian hill of jebel Umm Marahik. Basement gneisses underlie the fans, while clean section of the volcanic rocks can be found through the length of the scarp. These expose, in sequence upwards:

- Trachybasaltic lava:- Black. Black, shiny –weathering and platy –jointed at outcrop, this unit is about 20m in thickness and includes two or three lava flows, resting unconformably on basement gneiss.
- Agglomerate and tuff of the cataract formation:- Cobble - and boulder – agglomerates compose the greater part of this formation and are poorly sorted and crudely bedded. Inter-beds of graded tuff and lapillistone are common, and in the middle of the scarp make up about one quarter of the total thickness. Both large
and small clasts are nearly all of rhyolite, but there are occasional pieces of weakly welded ignimbrite and of trachybasalt. It is likely that these volcanic sediments were deposited from water-rich mudflows in mass-flows activity related to early ash-flow eruptions.

- Pisolithic tuff: Immediately overlying catastact formation is a layer of green, pisolithic tuff marking the change from epiclastic agglomerate and tuff to ash flow deposits of the scarp ignimbrite. The accretion lapilli which characterized this kind of tuff may have formed as moist aggregates of ash in eruption clouds or by accretion within standing pools of water, and they are particularly common as products of hydroclastic eruption.

- Scarp ignimbrite: Rising above the craggy features which mark out ledges of agglomerate, the outcrop of this ignimbrite is expressed as the less steep terraced slope ascending to the edge of the plateau. Around Khor Sada, four or five individual ashflows occur within a total of 50-100m. Rhyolite-rubbley layers mark the base of some of the flows and welding is present through much of the section.

- Plateau ignimbrite: This formation is typically black in the strongly welded zone near the base, but become grey above. Columnar jointing is well developed, specially in the black zone. Dips on eutaxitic foliation suggest that the sheet dip in this area is about 20° SW.

The study area can be divided to many localities according to Almond and Ahmed (1993) as follows.

2.2.1 Locality1.

Sections through the scarp around this point expose sequences beginning at the base with few meters of platy jointed trachybasalt. This is overlain by about 2m of graded tuffs an lapillistone, which in turn pass upwards into crudely bedded agglomerates containing thin tuff layers and totaling about 50m. Megaclasts are nearly all of rhyolite lava (both flow-banded and massive varieties), and the larger fragments of this very poorly sorted rock are subrounded and grain supported. In places, however, cobble-sized clasts are supported in a matrix of coarse tuff.
Above the cliffs of agglomerate come terraced slopes of scarp ignimbrite. These rocks are grey to pink in colour, locally contain abundant rhyolite fragments, and are visibly welded in most places. At the base, a 2m band of pisolitic tuff is probably water-laid, and forms a good marker horizon along most of the scarp section. On the top of the scarp ignimbrite come darker-coloured, tougher welded tuffs with a larger proportion of crystals. These rocks occupy the highest ground and are part of the plateau ignimbrite.

2.2.2 Locality2.
An eastward –thinning tongue of flow-banded rhyolite inter-vens between the top of the upper most agglomerate layer and the overlying scarp ignimbrite. These lavas thicken to the northwest but are absent along most of the scarp. Their position here suggests that extrusion of at least the upper part of the great mass of rhyolite lava in the Cauldron post dated the deposition of the cataract clastic rocks, but occurred before the major ignimbrite eruptions. There are no rhyolite lavas known to be younger than these ignimbrites.

2.2.3 Locality3.
Above the trachybasalt at base of this section it can be seen that the upper and lower units of agglomerate separated by a few meters of platy, bedded tuff. The upper agglomerate leaf dies out a short distance to the southeast, but its position in the sequence continues to be marked by a 2 m bed of hard dark-coloured lapillstone known as the (Black Band).

2.2.4 Locality4.
In exposures around the point were the point where Khor Soda emerges from the hills, two or three varieties of trachybasaltic lava can be distinguished within the 20 m section through the rocks. The varieties differ in size and quantity of the feldspar phenocrysts and in the ground mass texture. It is likely, that more than one lava flow is present within the trachybasaltic unit. Above the trachbasalt is a thin agglomerate, but at this locality much of the cataract formation consist of unwelded and weakly welded ignimbrite with a prominent rubbly layer in the middle marking the base of flow. For the geology of this area see Figure 2.1 which shows the elevation model image for Sabaloka igneous complex with elevation contour lines. The north east
scrap and adjacent section of Sabaloka plateau, with part of the Nile gorge to the east are shown in Figure 2.2. The almost flat plateau surface is composed of ignimbrite and traversed by rectilinear, close-spaced vertical joints, predominantly in sets trending north east and northwest. Both the Nile and the scarp are joint-controlled.

Figure 2.1 Geological map of Sabaloka area (Al Khidir, S.O.H, 2010)
Figure 2.2 the geological map of the study area (After Malik I.A.2008)
2.3 Extrusive rocks

2.3.1 basic lavas

A thin formation of basic lavas is to be seen wherever the base of the volcanic succession is exposed, and identical material is widespread as fragments within ignimbrites and the ring dyke. A part from a few basaltic dykes intrusive into the basement these are the only basic rocks in the complex, but they have a wider distribution and more constant character than any other group. The formation is well exposed in two areas. At the first, about 10 to 20 m of lavas rest on pre volcanic sediments northwest of the plateau. A comparable thickness of lavas is exposed along the northeastern scarp of the plateau and here includes three flows, of which the central member is distinguished by relatively coarse grain and the presence of feldspar phenocrysts. At another locality, in the Grarben the lavas are overlain by 15 m of well bedded tuffs, (Almond, 1977).

It therefore seems that the centralized rhyolitic volcanicity which formed the main phase of activity at Sabaloka was superimposed on a thin but extensive field of basic lavas flows with occasional pyroclastics cones.
Table 2.1 Geological succession of the sabaloka area (after D.C. Almond, 1977)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Age</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Superficial deposits</em></td>
<td></td>
</tr>
<tr>
<td>rocks in the complex, but they have a wider distribution and more constant character than</td>
<td></td>
</tr>
<tr>
<td>any other group. Nile silts, alluvial fanes, Aeolian, sands, lag gravels, wash-zone sands</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cainozoic, largely Quaternary</td>
</tr>
<tr>
<td>Unconformity</td>
<td></td>
</tr>
<tr>
<td><em>Nubian sediments</em></td>
<td></td>
</tr>
<tr>
<td>Alluvial sandstone, conglomerate, siltstone, locally ferruginized</td>
<td>late cretaceous/early Tertiary</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Unconformity</td>
<td></td>
</tr>
<tr>
<td><em>Sileitat Es – Sufur Igneous complex</em></td>
<td></td>
</tr>
<tr>
<td>Peralkaline granite, quartz syenite and rhyolite</td>
<td>Middle Jurassic</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Disconformity</td>
<td></td>
</tr>
<tr>
<td><em>Sabaloka Cauldron complex</em></td>
<td></td>
</tr>
<tr>
<td>Trachybasaltic lava, agglomerate and tuffs, rhyolitic lava and ignimbrite, boss of</td>
<td></td>
</tr>
<tr>
<td>mica granite, ring dyke of prophyritic microgranite</td>
<td>Lower Devonian</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Subvolcanic sediments of the Cauldron complex</em></td>
<td></td>
</tr>
<tr>
<td>Conglomerate, sandstone and shale</td>
<td>Lower Devonian</td>
</tr>
<tr>
<td></td>
<td>Unconformity</td>
</tr>
<tr>
<td><em>Tuleih Igneous complex</em></td>
<td></td>
</tr>
<tr>
<td>Quartz syenite, microgranite and a range of dyke-rocks</td>
<td>Lower Ordovician</td>
</tr>
<tr>
<td></td>
<td>Disconformity</td>
</tr>
<tr>
<td><em>Basment</em></td>
<td></td>
</tr>
<tr>
<td>Late-tectonic granitoid batholiths – predominantly adamellitic and mostly porphyroblastic</td>
<td></td>
</tr>
<tr>
<td>Late tectonic Gabbro</td>
<td>Precambrian</td>
</tr>
<tr>
<td>Gneiss complex</td>
<td></td>
</tr>
</tbody>
</table>

2.3.2 Petrography of basic lavas

The lavas are platy-jointed rocks, black to dark-green in colour. Aphyric varieties contain flow-oriented plagioclase, about 30% augite, opaque oxides and in some cases up to 10% pseudomorphed olivine. The primary minerals have been more or less extensively altered to chloride, carbonate and albite, but lavas metamorphosed by the mica granite are not affected in this way so it seems that alteration occurred at an early date. In porphyritic
lavas the phenocrysts are of weakly zoned labradorite while the groundmass laths are more sodic than in most aphyric lavas, and augite is less abundant. Such rocks are transitional towards trachyte.

2.3.3 Rhyolite lavas
Thoroughly acidic lavas make up over half the bulk of the volcanic rocks presented within the cauldron and are particularly well exposed in the southwest of the plateau, where they give rise to the broken topography of pale-coloured hills in distinct contrast within the less dissected ground and more sombre hues of the ignimbrite outcrops. Arcuate features reveal the regular dip of the flows towards the centre of the plateau, orange-coloured features picking out massive lavas whereas autobrecciated flows show up grey. In the lower part of the succession some flows are brecciated throughout their thickness. The flows are separated in places by bedded tuffs and lenses of volcanics conglomerate, the most conspicuous of these horizons being up to 9 m thick and traceable for 4 km. Mapping suggests that the main volcanioclastic horizon in Khor Musa may correlate with the Cataract formation, separating aphyric Lower rhyolites from upper rhyolites with phenocrysts. If so, the distribution of these two formations implies that early acidic volcanicity centred in which in now is the southeast of the plateau whereas later activity enlarged the field in a northerly direction. At no time did rhyolite lavas reach into the northeast of the area.

2.3.4 Petrography of rhyolite lavas
Pale-gray where least altered, the lava are generally mottled with orange oxidation products which in autobrecciated rocks concentrates in the matrix. Thin sections reveal little, the most obvious variations being in the phenocrysts. These are entirely absent from many of the lower rhyolites though a few section contain rare quartz or kaolinized feldspars. All the upper rhyolites contain spares microphenocrysts, the quartz often conspicuously rounded, while feldspars may be euhedral or have rounded edges. Both potassium feldspar and oligoclase are normally present. Ground mass texture is often evenly cryptocrystalline, but micropoikilitic enclosure of feldspar needles in quartz often gives a patchy appearance and spherulitic and nodular variants also are founded. Some rocks are further diversified by nests of relatively coarse quartz and fluorite. Chlorite, clay minerals and carbonate are common alteration products.
Chapter Three
LITERATURE REVIEW

3.1 Introduction

This chapter is intended to review all literature pertaining to the subject of this research. This will include the rock investigation procedures, the rock classification systems, the failure criteria proposed by various investigators, the determination of rock mass modulus by various methods.

3.1.1 Rock Mass Strength

Variation of engineering activities has great relation to rock masses, such as:

- mines,
- underground excavations,
- dams, and
- bridges

They are constructed within or upon rock masses. Rock masses exert an important influence upon the behavior of the structure and determine both the methodology and the form of construction and the likely operational requirement during the structure’s design life.

Two essential geotechnical considerations in the design of any structure in rock are:

1. The maximum load that can be placed on (or removed from in the case of underground structures) the supporting rock mass without catastrophic failure or loss of integrity of the rock mass, and
2. The likely movement of the rock mass under the application (or removal) of loads.

The maximum load that a rock mass can support is normally expressed in terms of stress as the failure strength. The failure strength is usually defined as the maximum stress that causes failure of a given rock mass specimen subjected to a certain confining stress. The movement or deformation of rock mass under a certain stress can also be estimated from
the deformation modulus. The deformation modulus is defined as the stress acting upon the rock mass divided by the strain within the rock mass volume of interest. Estimating both the failure strength and deformation modulus are two of the most difficult problems facing designers of structures in rock masses. Compared to a jointed rock mass, intact rock is relatively simple material to estimate the strength and deformability parameters for; but in practice, for a large scale engineering projects of interest, jointed rock masses are the norm.

Before rock mass strength can be made, an appreciation of the rock mass structure and material properties must be established. One of the main aims of the geotechnical site investigation is to evaluate the essential factors influencing the strength of the rock mass in order to find quantitative estimates of strength and deformability for design. One of the most common ways of estimating the rock mass strength is by using the failure criterion. The existing rock mass failure criteria are stress dependent and include several parameters that describe the rock mass properties. These parameters are often based on classification or characterization systems. A comprehensive literature review of existing classification systems and rock mass failure criteria with special attention to the Hoek and Brown failure criterion has been performed.

3.1.2 Geotechnical investigations
The design of structure within a given rock mass depends upon such factors as:

- Type of rock,
- The likely design loading, and
- The operational requirements of the final structure.

The designer must work within these bounds to meet acceptable levels of safety and economy (Hoek, 2000). There is a wide range of typical rock mechanics problems involving construction and stabilization of rock slopes, dams, foundations, tunnels, large caverns, and mining excavations. Each of these problem areas requires consideration of various failure mechanisms. For example, circular failures in soil and jointed rock, wedges failures, toppling failures and rock falls, all are problems involving rock slopes but all involve different failure mechanisms that must be considered during the design process.
Only some of these analyses require an estimates of the strength along persistent planar discontinuities traversing the rock mass but nearly all require reliable estimates of rock mass strength and deformability. The first stage of any geotechnical investigation is to visualize the problem, i.e. what is the basic shape and form of the proposed structure. Once this has been decided, then a site investigation must be undertaken. The site investigation consists initially of preliminary site appraisal (Attewell, 1993). This involves a review of available information on the site, for example topography and geological map, aerial photograph, and a visit to the site to inspect surface exposures and morphology. The information collected from the preliminary site appraisal will then be used to plan the preliminary ground investigation.

The aim of the preliminary site investigation is to provide reasonable confirmation of the design outline and indication of likely construction requirement and costs. A preliminary ground investigation usually consists of a limited number of boreholes and mapping of the discontinuity structure is undertaken.

Following the preliminary ground investigations, the main ground investigation typically consists of both in situ and laboratory tests in and around the areas identified from the preliminary ground investigation as requiring careful inspection.

The geological database obtained from the site investigations is then used to estimate the properties of the rock mass. Usually the rock mass geometry is so complex and the actual material properties significantly separate from idealized values that the design of rational and consistent procedures is impossible.

### 3.1.3 Estimating strength and deformability of rock masses

Evaluation of the strength and deformation properties of jointed rock masses presents formidable theoretical and experimental problems. The range of techniques used to evaluate the effect of discontinuities upon rock mass strength includes analytical, numerical and empirical methods with the results from these analyses verified through laboratory and field testing results.

The methods of collecting geological data have not changed substantially within the last years, although analysis methods in the form of computer software have advanced greatly. Unfortunately, because these analysis methods are only as good as the available
input data, estimating the characteristics of a typical rock mass at a suitable scale is still a major problem.

Laboratory testing of rock masses still plays a large role in the determination of strength of rock masses. Hoek (2000) suggested only 10 to 20 percent of a balanced rock mechanics investigation should be allocated towards laboratory testing. Laboratory tests can usually only be carried out on intact rock of small sample size due to the limited size and loading capacity of the testing equipment. Therefore, the lab test specimens will be much smaller than the scale of interest for a typical engineering project. The results will then be representative of the extreme end of the strength values for a jointed rock mass and provide very little consideration of the influence of the discontinuity network on the strength of the rock mass.

One method introduced to account for the factors that influence the strength of jointed rock masses was the rock mass classification (Bieniawski, 1989, and Barton, Lien and Lunde, 1974). Originally developed to determine support systems for tunnels, these classifications were developed based on practical experience. A database of geological properties and performance of the support systems are used in the previous underground engineering projects. Information such as intact rock strength, ground water flow, in situ stress and the number, spacing, inclination and interface properties of discontinuities were recorded. These classification systems represent the first systematic method to examine a jointed rock mass for design. For application of these classifications, a rating was obtained, which would then be used to determine the appropriate support method.

While these systems were useful in their intended areas of application, estimating the strength has still proven difficult for other areas. Many attempts have been made to formulate both theoretical and empirical failure criteria to estimate the strength of a given rock mass. Most of these criteria use an estimate of the strength of the intact rock altered by other factors based on either experiments on jointed rock masses or block models to account for the reduction in strength due to the discontinuity network. Hoek and Brown (1980) recognized the value of the rock mass classification systems to assess the effect of the discontinuities in the rock mass and use them to calculate reduction factor to apply to intact strength prediction from their failure criterion. Therefore before evaluating any
attempt at predicting the strength of rock masses, it is therefore necessary to firstly understand what factors influence the behavior of jointed rock masses.

3.2 Factors Affecting Strength of Rock masses

A jointed rock mass is an in situ rock material which has been made discontinuous by weakness planes (generally of natural origin, e.g. joints, faults and bedding planes), which may be broadly revered to as discontinuities. Discontinuities occur either in sets (e.g. joints cleavages bedding planes) or unique, (e.g. faults). The former is often treated by statistics and the latter by separate analysis. Anon (1977) describes a discontinuity as “a plane of weakness that has zero or low tensile strength or tensile strength lower than stress levels generally applicable in engineering applications”. Therefore a discontinuity is not necessarily a plane of separation, but rather a plane of structural weakness.

There are many factors that influence the strength of jointed rock masses. As mentioned in section 2.1.3, the factors of the greatest significant were incorporated into rock mass classification schemes (Bieniawski 1989, Barton et. al, 1974). While these schemes were mainly concerned with under ground excavations, others have noted that similar factors, feature strongly in the analysis of other problems in rock mechanics, such as rock slope stability (Romana, 1993). They were therefore assumed to be of some uses in other applications.

Depending on the aim of the analysis, the factors considered to influence the strength will vary. The author believes the following list captures the most important factors to consider, when estimating the strength of jointed rock masses:

- Intact rock strength
- Discontinuity spacing
- Orientation of discontinuity
- Persistence and extent of the discontinuity
- Infilling between discontinuity
- Degree of roughness and waviness of the surface
- Ground water conditions
- In situ stress
- Geological history
Discontinuity of rock masses is generally heterogeneous and anisotropic. The intact rock strength indicates the ability of the jointed rock mass to resist shearing failure through the intact pieces of rock. Each discontinuity has a different degree of strength along its length. Therefore, any acceptable solution to a rock mass model should consider both the anisotropy of the rock mass and the discontinuities that govern the stability of the rock mass.

### 3.2.1 In situ stress

The application of loading condition for a jointed rock masses changes the in situ stress field within the rock mass. In rock masses, local stress redistributions can occur around pre-existing discontinuities, and in areas following local failure and yielding of rock material. The state of stress within a rock mass is usually the result of the locked in strains resulting from previous geological processes acting upon the rock mass. While the change in stress can be calculated following the application or removal of a load, the total stress is difficult to determine because the in situ stress state is rarely known.

The vertical stress at a given point within the rock mass is usually equal to the weight of the overburden above the point of interest (Brown and Hoek, 1978).

\[
\sigma_v = \gamma z \quad \text{------------------------ (3.1)}
\]

where:
- \( \sigma_v \) = vertical stress
- \( \gamma \) = unit weight of the founding rock
- \( z \) = the depth below the surface

The scatter involved in this relation is large, especially within 1000 m of the earth’s surface as shown in Figure 3.1. Differences between the calculated overburden stress and measured vertical stress are generally due to problems in measurement (Richard, 2004). While equation 3.1 is usually assumed it is better to measure the in situ stress to be sure (Hudson and Harrison, 1997).
Horizontal stresses are too much difficult to determine. Brown and Hoek (1978) state that the horizontal stresses may be large even at shallow depths. High horizontal stresses are usually associated with changes in the history of the site (e.g. sites may have previously experienced high overburden stresses). Terzaghi and Richart (1952) determined by elasticity theory that the horizontal stress is equal to $\nu / (1-\nu)$ multiplied by the vertical stress, where $\nu$ is the Poisson’s ratio. This relation was used traditionally but has proved to be unrealistic, because of the assumption that no lateral strain is allowed under application of gravitational forces (Hoek, 2000). Studies by Brown and Hoek (1978) and Shoerey (1994) have found that the horizontal stress / vertical stress ratio is higher at shallow depth and reducing with increasing depth. However, Hudson and Harrison (1997) note that, the use of such a ratio should be regarded with caution as the vertical stress does approach zero at the surface and will significantly influence the value of the horizontal stress as a ratio of vertical stress.

The horizontal stresses within the rock mass are difficult to determine and are caused by a number of factors such as topography, erosion, tectonic activity, rock anisotropy and discontinuities (Hudson and Harrison, 1997). Therefore the horizontal stress must usually be confirmed on site by in situ tests, if it has strong influence on the design.
Jointed rock masses have already been addressed to the failure in the past and with the exceptional of the influence of larger scale structures located close to the rock mass, the in situ stress field is usually assumed to be uniform at the scale of interest. In situ stress has an important effect upon the behavior of rock mass especially when considering the effect of joints upon the rock mass strength. If the region of interest of the rock mass is close to the surface, the influence of the joints will dominate the mechanical behavior of the rock mass, because under low confining pressure, failure usually occurs via sliding along existing defects rather than shear through intact blocks which is more common under higher confining pressure (Singh et al., 2002).

3.2.2 Groundwater and effective stress

The stress state within a rock mass depends upon the groundwater pressures within the rock mass. In jointed rock masses, groundwater pressure will act to separate apart opposing discontinuity surface, reducing the effective normal stress between these surfaces. Therefore, the frictional shear resistance acting across the interface. Previous work (Serafim 1968; Brace and Martin, 1968; Jaeger and Cook, 1976; Hoek and Brown, 1980; and Hoek, 1983) have shown that there is some debate as to whether the principle of effective stress is applicable to rock masses. Concern appear to be centre on the effect of pore pressure on intact rock masses, where micro cracking of intact rock does not immediately allow the intrusion of pore water to reduce effective normal stresses. For rock masses with significantly higher intact strengths than applied loads intact rock fracture is unlikely to be a significant failure mechanism and therefore the principle of effective stress $\sigma' = \sigma - u$ (where $u$ is the pore water pressure) should be in closely jointed rock masses. For intact rock, Hoek (1983) stated that the principle of effective stress is satisfactory provided that the pore structure of the rock which is sufficiently interconnected and the loading is applied at a slow rate to allow internal pressure to equalize during testing. Lade and de Boer (1997) concluded that the principle of the effective stress was suitable for most geotechnical applications, but there were significant deviators at higher stress levels. Therefore, it is anticipated that for the magnitude of loads typically applied to jointed rock masses, pore water dissipation will occur rapidly after application of load.
In certain types of rock masses, the intact material can react and reduce the strength of the intact material. The impact of groundwater is therefore particularly important when dealing with shales, siltstone, and similar rocks with the strength susceptible to changes in moisture content (Hoek and Marinose, 2000). Hoek (1983) cites studies stating tests on sandstone specimens ranging from oven dried to saturation reduced the strength by a factor of 2. Similar results were found by Broch (1974) but within engineering limits (air dry – to saturation) it is likely this reduction will be only 20-30% (Barton, 1976). Hoek and Brown (1980) plotted dimensionless plots of Broch’s (1974) results and found that the fracture characteristics did not significantly change with moisture content and attributed the strength reduction to the uniaxial compressive strength. Barton (1973) reviewed the effect of water on the shear strength of rock joints and found that for low to medium stress levels, the shear strength of planer surface is largely unaffected if it is wet for rough undulating joint surfaces from 5% up to 30% (Barton, 1973). Barton (1973) and Broch (1974) showed that the tensile strength, compressive strength and frictional strength are strongly affected by the moisture content. It is likely that rough joints will be more affected by moisture than smooth joints because of the adverse effect of moisture upon the tensile strength (Byerlee, 1967).

### 3.2.3 Rock mass structure

The strength of jointed rock masses, depends more upon the strength and deformability of the discontinuities rather than on the strength of intact rock material. The rock mass strength of the jointed rock is then has direct relation with the residual strength, which is governed by the interlock between the rock blocks making up the rock mass. Similarly, the rock mass deformability is governed by the displacements of the rock blocks within the rock mass. The rock mass structure is typically described by the discontinuity properties listed in section 3.2. The spacing and orientation of the discontinuities determine the ability of the rock mass to deform or fail without fracturing of the intact rock pieces (Anon, 1977). These factors also determine the shape of the rock pieces. Anon (1977) classifies intact rock blocks into three separate groups:

- Blocky where all three dimensions are similar;
- Tabular, where one dimension is greater than the other two; and
- Columnar, where two dimensions are similar and greater than the third.

These definitions suggest that the discontinuities are orthogonal to each other, but this is not always the case in the field. The spacing and orientation also determine whether the failure mechanism will be along a single discontinuity or path through several discontinuities, or involve the rock mass as the whole, i.e. combination of fracturing of rock pieces and sliding along the discontinuities.

The spacing and orientation of discontinuities will have an important influence upon whether the rock mass can be treated as continuum. The strength along the discontinuities will also have important influence. Figure 3.2, shows how consideration of the rock mass at different scales can affect the analysis approach. A jointed rock mass will still act as anisotropic rock mass, if one of the discontinuity sets is vastly weaker than the others.

![Figure 3.2: Influence of scale on analysis of rock masses (from Hoek, 2000)](image_url)
There is some debate as to exactly how many discontinuity sets are required before isotropic behavior of rock masses can be assumed. Hoek and Brown (1980) assume that four sets are usually needed, where Sing, et al. (2000) suggest as many as six. Clearly then the scale of the problem is important in determining the behavior of a rock mass. By definition, the dimension of the structure supported on jointed rock masses, will be much greater than the spacing between discontinuities such that the rock mass can be assume to be isotropic.

The strength of discontinuity is determined by the nature of the surface along the discontinuity and the infilling material between the discontinuity surfaces (Barton, 1976). The nature of the discontinuity considers the effect of the surface roughness and wavelength of the discontinuity surface. The hardness and consistency of the discontinuity infilling will influence the shear strength and stiffness along the discontinuity and the normal stiffness of the discontinuity.

The persistence is the measure of the length of discontinuity. A joint of long persistence oriented at unfavorable angle to the direction of the applied load will maximize the risk of the failure, however, a series of discontinuities of short persistence that are aligned in a similar direction with respect to the applied load but offset short distance from each other, may also contribute to failure. The persistence is typically estimated by the length of the discontinuity trace observed on a rock exposure.

Knowledge about discontinuities within the rock mass can be improved by analysis of borehole cores but the core recovery in closely jointed rock masses is poor and it is difficult to assess the true joint orientation (Priest, 1993). Traditionally a jointed rock mass is assumed to be anisotropic and homogenous continuum for engineering analysis.

### 3.2.4 Intact rock strength

Intact rock refers to un-fractured blocks between structural discontinuities. Most early research upon estimating rock strength has concentrated on the strength of intact rock because of ease of obtaining and testing of laboratory specimens and the existing theoretical background available in the discipline of solid mechanics (Hudson and Harrison, 1997). Intact rock strength is most commonly represented as the unconfined compressive strength, $\sigma_c$. The intact strength is easy to obtain in such test as uniaxial test, triaxial test, point load test, Schmidt hammer test (ISRM, 1985) and visual indices (Brown, 1981). The intact strength is important both for the determination of the shear strength of discontinuities and for intact rock masses.
The accuracy of the measurement of rock strength is determined by many various factors such as specimen shape (Obert and Duvall, 1967) and size (Bieniawski, 1968; Jaeger and Cook, 1976), rate of loading, presence of water, temperature, anisotropy (Jaeger and Cook, 1976), and stiffness of the testing machine (Hudson and Harrison, 1997). Generally the lab tests are carried out on constant length: diameter ratio of 2:1 and of constant diameter (typically 50 mm).

For intact rocks, Hoek and Brown (1980) found strength decreases with increasing sample size up to limit. Hoek and Brown (1997) suggested that this was due to the greater opportunity for failure around grains as the number of grains increase in the test sample.

Hoek and Brown (1980) compared data on specimen size and uniaxial compressive strength $\sigma_c$ as shown in figure 3.3. The uniaxial compressive strengths were normalized against 50 mm specimens. This enabled a better comparison of results by eliminating differences due to variations in such environmental factors as moisture content, specimen shape, loading rate since these factors were generally the same for given data set.

After analyzing the uniaxial compressive strength of various samples of intact rock with the diameter of sample tested, Hoek and Brown (1980) proposed the following relationship between uniaxial compressive strength and sample size.

$$\sigma_{ed} = \sigma_{c50}\left(\frac{50}{d}\right)^{0.18}$$

(3.2)

Where

$\sigma_{ed}$ = uniaxial compressive strength in sample of given diameter, d.

$\sigma_{c50}$ = equivalent uniaxial compressive strength in 50 mm diameter sample.

where all samples are in a length diameter ratio of 2:1. Hendron (1968) stated that the 2:1 length diameter ratio is necessary to insure both that a fairly uniform stress distribution occurs throughout the sample during loading and the failure surface is free to form throughout the sample without intersecting the sample head.
3.3 Review of Rock Mass Classification

Rock mass classifications relate practical experience gained on previous projects to the conditions anticipated at a proposed site. They are particularly useful in the planning and preliminary design stages of a rock engineering project but, in some cases, they also serve as the main practical bases for the design of complex underground structures. According to Bieniawski (1989) modern rock mass classification was developed to create some order out of the chaos in site investigation procedures and to provide the desperately needed design aids. According to Bieniawski (1989), the objectives of rock mass classifications are: -
- To identify the most significant parameters influencing the behavior of rock mass.
- To divide a particular rock mass formation into areas of similar behavior, that is, rock mass classes of varying quality.
- To provide a basis for understanding the characteristics of each rock mass class.
- To relate the experience of rock conditions at one site to the conditions and experience encountered at others.
- To derive quantitative data and guidelines for design engineering.
- To provide common basis for communication between engineers and geologist.

Terzaghi (1946) used a rock mass classification system for the design of tunnel support in which rock loads carried by steel sets were estimated based on descriptive classifications. This is considered the first rational classification system in rock engineering. In this descriptive system, he highlighted the characteristics that dominate the behavior of the rock mass. He included clear concise descriptions with practical comments that presents the engineering geological information most useful to design engineer.

Stink (1950) is considered as the father of the (Australian school) of tunneling and rock mechanics. He emphasized the importance of structural defects in rock masses.

Lauffer (1958) proposed that the stand – uptime for an unsupported span is related to the quality of the rock mass in which the tunnel was excavated. Lauffer”s work has been modified by Pacher et al (1974) and now forms part of the New Austrian Tunneling Method (NATM). The rock quality designation index (RQD) was developed by Deere et al (1967) to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100mm (4 inches) to the total length of core run and is calculated as follows:

\[
RQD = \left( \frac{\text{Length of core pieces } > 10 \text{ cm}}{\text{Total length of core run}} \right) \times 100 \quad \text{(3.3)}
\]

Palmstrom(1982) suggested that, when no core is available but discontinuity traces are visible in surface exposure or exploration adits, the RQD may be estimated from the number of discontinuities per unit volume as follows:

\[
RQD = 115 - 3.3 J_v \quad \text{-------------------------(3.4)}
\]

Where
$J_v$ is the volumetric joint count or the sum of the number of joints per unit length for all joint sets.

Priest et al., (1976) used the following equation when no cores are available:

$$RQD = -3.68 \lambda + 110.4 \quad \text{------------------------ (3.5)}$$

Where $\lambda$ = the number of joints per meter.

Cording and Deere (1972), Merritt (1972) and Deere (1988) have attempted to relate RQD to Terzaghi rock load factors and to rockbolt requirements in tunnels. The most important use of RQD is as a component of the RMR and Q rock mass classification.

RQD index is significantly influenced by the orientation of the borehole, and the value can vary significantly for the same rock mass depending on the borehole orientation. The RQD is the measure of drill core quality or fracture frequency, and disregards the influence of joint tightness, orientation, continuity, and infilling. Consequently, the RQD does not fully describe a rock mass.

Wickham et al (1972) described a quantitative method for describing the quality of rock mass and for selecting appropriate support on the basis of their Rock Structure Rating (RSR) classification. Most of the case histories, used in the development of this system, were for relatively small tunnels supported by means of steel sets. The significance of the RSR system, is that it introduced the concept of rating each of the components listed below to arrive at a numerical value.

$$\text{RSR} = A + B + C \quad \text{------------------------ (3.6)}$$

Where:

A - Refers to the rock type (based on origin) and strength of the rock mass (rock hardness and geological structure).

B - Refers to the influence of the discontinuity pattern with regard to the direction of drive (based on joint spacing, joint orientation and direction of tunnel).

C - Refers to the influence of ground water and joint condition on the rock mass (based on overall rock mass quality, joint condition and amount of water inflow).
This system makes very crude estimates of support requirements particularly in terms of rock bolts and shotcrete, as they are based on very simplistic theoretical arguments and few historical cases.

Bieniawski (1973) introduced the geo mechanics classification also named the Rock Mass Rating (RMR), at the South African Council of Scientific and Industrial Research. The rating system was based on case histories drawn largely from civil engineering. Consequently, the mining industry tends to regard the classification as somewhat conservative and several modifications have been proposed in order to make the classification more relevant to mining application. Bieniawski’s (1989) rock mass rating classification (RMR89) is the system that is most frequently used today. The RMR is based on 351 case studies (Bieniawski, 1989) and is assessed on the following six parameters:

- Uniaxial compressive strength (UCS) of rock material tunnels.
- Rock Quality Designation (RQD).
- Spacing of discontinuities.
- Condition of discontinuities.
- Ground water conditions.
- Orientation of discontinuities.

The rock mass classification chart for the RMR is shown in Table A1:1 to A1:5 in Appendix 1:1.

Several modifications to these systems have been proposed. Laubscher (1977, 1984, 1990, and 1993), Laubscher and Taylor (1976), and Laubscher and Page (1990) have described a Mining Rock Mass Rating System (MRMR), which modifies the basic rock mass description for joint orientation, blast damage, mining induced stress and rate of weathering. In using Laubscher’s MRMR system, it should be borne in mind that the system was originally developed for block cave mining.

Bieniawski (1989) published a set of guidelines for the selection of support in tunnels for rock masses in which the RMR had been determined. These rating relate specifically to a horseshoe shaped tunnel with maximum span of 10 m at a maximum depth of 900 m.
On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al (1974) of the Norwegian Geotechnical Institute (NGI) proposed a Tunneling Index (Q) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1000 and is defined by:

\[
Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}
\]

Where

- **RQD** = the Rock Quality Designation
- **J\_n** = the joint set number
- **J\_r** = the joint roughness number
- **J\_a** = the joint alteration number
- **J\_w** = the water reduction factor
- **SRF** = the stress reduction factor

Each of these parameters is defined by a parameter obtained from Table A1: 6 in Appendix 1.2.

The first quotient, RQD/ J\_n represents the structure of the rock mass and is a crude measure of the block or particle size. The second quotient, J\_r / J\_a , represents the roughness and frictional characteristics of the joint walls or filling materials and is the crude measure of the inter- block shear strength. The third quotient J\_w / SRF , is a crude measure of the active stress.

Grimstad and Barton (1993) using a Q system estimated support categories in terms of equivalent dimension (D\_e). D\_e is an additional parameter defined by Barton et al (1974) where:

\[
D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation support ratio (ESR)}}
\]

Loset (1992) suggests that, for rock with 4 < Q < 30, blasting damage will result in the creation of new joints with a consequent local reduction in the value of Q for the rock surrounding the excavation. He suggests that this can be accounted for by reducing the RQD value for the blast damage zone.
Figure 3.4: Tunnel support requirements as function of rock mass quality Q, span or height, and ESR, the effective span ratio (Grimstad and Barton, 1993).
3.4 Uses of Rock Mass Classification Systems

The two most widely used rock mass classification systems are Bieniawski’s RMR (1976, 1989) and Barton et al. Q (1974), and a relationship has been found between RMR and Q as follows.

\[ \text{RMR} = 9 \ln Q + 44 \quad \text{---------------- (3.9)} \]

Barton (1995) developed the following correlation between Q and RMR.

\[ \text{RMR} = 15 \log Q + 50 \quad \text{---------------- (3.10)} \]

Hudson and Harrison (1997) have used a rock engineering systems approach to demonstrate that a classification system for one structure would not necessarily be the same as for another.

The main significance of the rock mass classification was that it provides a systematic basis upon which to evaluate a given rock mass for design and therefore a solid practical basis upon which to make engineering judgments.

Pells, (2000) and Watson, (2004) reported that the reliability of the classification systems is questionable under certain conditions. The reason for this is that, although this classification systems consider similar parameters in calculating the final rock mass rating, different systems apply different weighting to similar parameters and some include distinct parameters that influence the final rock mass quality rating. They concluded that when dealing with extremely weak ground, the RMR classification system is difficult to apply. This is because it was developed for the hard rock environment.

Hoek and Brown (1997) concluded that the use of the rock mass classification systems is coupled with uncertainties. Neither system can be said to fully characterize the rock mass and several of the factors included are very subjective and difficult to assess quantitatively, which might also influence the strength estimates. It is important to understand that the use of rock mass classification scheme does not and can not replace the more elaborate design procedures. However, the use of these design procedures requires access to relatively detailed information on in situ stresses, rock mass properties and planned excavation sequence, none of which may be available at an early stage in
the project. As this information becomes available, the use of rock mass classification schemes should be updated and used in conjunction with site specific analysis.

### 3.5 Principal Methods of Determining the Rock Mass Strength

Krauland et al. (1989) listed four principal ways of determining the rock mass strength:

1. mathematical modeling
2. rock mass classifications
3. large scale testing
4. back analysis of failure.

Empirically derived failure criterion for rock masses often used in conjunction with rock mass classifications can be added to this list.

A mathematical model requires determination of a large numbers of parameters and is often based on simplified assumptions.

Classifications are often used in the primary stage of a project to predict the rock mass quality and the possible need for support. The results is an estimate of the stability quantified in subjective terms such as bad, acceptable, good or very good rock conditions. The value obtained by some of the classification systems is used to estimate or calculate the rock mass strength using a failure criterion.

Large-scale tests provide data on the true strength of the rock mass at the actual scale of the construction, and, indirectly, a measure of the scale effect that most rock exhibit. As large-scale tests are neither practical nor economically feasible; most researchers have studied the scale dependency of rock mass strength in laboratory environment.

Back-analysis of previous failures is attractive, as it allows more representative strength parameters to be determined. Obviously failure must have occurred and the mode of failure must be reasonably established. There are little data available on rock mass failure that can be used for back-analysis.

#### 3.5.1 Empirical rock failure criteria

Empirical failure equations have been proposed based on laboratory tests of intact rock specimens. Some of these have also been developed as rock mass failure criteria; with suitable adjustments related to rock mass classification index (such as RMR, GSI classification and Hoek – Brown criterion). The expressions for well known rock mechanicals empirical failure criteria for intact rock are given in Table 3.1. The criteria...
are formulated in terms of $\sigma_1$ and $\sigma_3$ without any consideration of the intermediate principal stress.

**Table 3.1: Failure criteria for intact rock (after Edelbro, 2003)**

<table>
<thead>
<tr>
<th>Failure equation:</th>
<th>Development / comments</th>
<th>Author, criterion first published</th>
</tr>
</thead>
<tbody>
<tr>
<td>$(\sigma_1 - \sigma_3)^2 = a + b(\sigma_1 + \sigma_3)$</td>
<td>An empirical generalisation of Griffith theory of intact rock.</td>
<td>Fairhurst (1964)</td>
</tr>
<tr>
<td>$\sigma_1 - \sigma_c + \sigma_3 + F\sigma_3^f$</td>
<td>Empirical test data fitting for intact rock.</td>
<td>Hobbs (1964)</td>
</tr>
<tr>
<td>$\sigma_1 - \sigma_c + a\sigma_3^b$</td>
<td></td>
<td>Murrel (1965)</td>
</tr>
<tr>
<td>$\frac{\tau_m - \tau_a}{\sigma_c} = D \frac{\sigma_m}{\sigma_c}$</td>
<td>Empirical curve fitting for intact rock.</td>
<td>Hoek (1968)</td>
</tr>
<tr>
<td>$\sigma_1 - \sigma_c + a\sigma_3$</td>
<td>Triaxial test on soft rock</td>
<td>Bodonyi (1970)</td>
</tr>
<tr>
<td>$\sigma_1 - \sigma_3 + \sigma_c + b(\sigma_1 + \sigma_3)^8$</td>
<td>Empirical curve fitting for 300 rock specimens.</td>
<td>Franklin (1971)</td>
</tr>
<tr>
<td>$\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^2)^{1/3}$</td>
<td>Application of Griffith theory and empirical curve fitting. Both for intact and heavily jointed rock masses</td>
<td>Hoek &amp; Brown (1980)</td>
</tr>
<tr>
<td>$\frac{\sigma_1}{\sigma_c} = a + b\left(\frac{\sigma_3}{\sigma_c}\right)^{n}$</td>
<td>Empirical curve fitting for 700 rock specimens. Both for intact and heavily jointed rock masses</td>
<td>Baniawalid (1974), modified by Yudhib et al. (1983)</td>
</tr>
<tr>
<td>$\sigma_1 - \sigma_3 + a\sigma_3(\frac{\sigma_1}{\sigma_3})^{1/8}$</td>
<td>Applied to 80 rock samples.</td>
<td>Ramamurthy et al. (1985)</td>
</tr>
</tbody>
</table>

| $\sigma_1 = \left(\frac{M}{B} \sigma_3 + 1\right)^{b}$ | Empirical curve fitting for both soil and rock specimens. | Johnston (1985) |
| $\sigma_1 = \sigma_c(1 + \frac{\sigma_3}{\sigma_c})^{b}$ | Both for intact and heavily jointed rock masses | Belmer (1952), Sheerey et al (1989) |
| $\sigma_1 = \sigma_3 + A\sigma_e \left(\frac{\sigma_1}{\sigma_c} - S\right)^{1/8}$ | $A$, $B$ and $S$ are strength parameters | Yoshida (1990) |

where: $\tau_m = (\sigma_1 - \sigma_3)/2$ and $\tau_a = (\sigma_1 + \sigma_3)/2$

- $\sigma_1 = \text{Major principal stress}$
- $\sigma_c = \text{Minor principal stress}$
- $\sigma_{1e} = \text{Major normalized effective principal stress}$
- $\sigma_{3e} = \text{Minor normalized effective principal stress}$
- $\tau_0 = \text{Uniaxial tensile strength}$

and $a$, $b$, $F$, $f$, $G$, $D$, $B$, $M$ and $\sigma$ are constants.
The failure criteria for rock masses are based on large scale and laboratory testing experience. The most widely referred rock mass criteria are presented in Table 3.2. They are all formulated in terms of $\sigma_1$ and $\sigma_3$ and are similar to the criteria for intact rock independent of $\sigma_2$. These criteria were derived from triaxial testing of small rock samples. Each of the four rock mass failure criteria are related to rock mass property parameters that describe the rock mass behaviors.

These criteria use classification systems to extract the difference between intact rock and the rock mass properties. Among these failure criteria Hoek – Brown failure criterion is the most widely accepted failure criterion for estimating the rock mass strength. The rock mechanics community has gladly adopted the equations suggested by Hoek and his co-workers.

Hoek (1994) stated that he originally only intended the criterion to be used for initial and preliminary estimates of rock mass strength. Hoek should also be complemented for his attempts at providing a tool for predicting the triaxial strength of rock masses. It was probably the apparent lack of such a criterion that led to the large acceptance of the criterion. The wide spread use of the Hoek – Brown failure criterion has not been complemented by equally increasing efforts to verify the same. There are very few reported cases in which the application of the Hoek – Brown failure criterion has been verified against actual observations of failure. Some verifications of the criterion was provided in a recent study by Helgstedt (1997) That compared predicted strength with back calculated values from dam foundations and large scale natural slope , as well as from tests on rock fill.

Helgstedt (1997) concluded that the Hoek – Brown criterion consistently predicted too high shear strength for these cases. All these cases were rock masses of poor to medium quality with GSI in the range of 22 – 55.
Table 3.2: Rock mass failure criteria (after Edelbro, 2003)

<table>
<thead>
<tr>
<th>Failure equation:</th>
<th>Comments:</th>
<th>Author, criterion first published:</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1^* = \sigma_3^* + \sigma_{ct} \left( m_b \frac{\sigma_3^*}{\sigma_{ct}} + s \right)^a$</td>
<td>2002 version</td>
<td>Hoek and Brown, 1980</td>
</tr>
<tr>
<td>$\sigma_1 = A \sigma_{cl} + B \sigma_{ct} \left( \frac{\sigma_3}{\sigma_{ct}} \right)^a$</td>
<td>A is a dimensionless parameter and B is a rock material constant, $a$ is suggested=0.65</td>
<td>Yudhhbir et al (1983)</td>
</tr>
<tr>
<td>$\sigma_1 = \sigma_{cm} \left( 1 + \frac{\sigma_3}{\sigma_m} \right)^{b_m}$</td>
<td>Use RMRm value</td>
<td>Sheorey et al. (1989)</td>
</tr>
<tr>
<td>$\sigma_1 = \sigma_3^* + \sigma_3^* \cdot B_j \left( \frac{\sigma_{cl}}{\sigma_3} \right) \epsilon_j \epsilon_j$</td>
<td>2001 version</td>
<td>Ramamurthy, (1995)</td>
</tr>
</tbody>
</table>

3.6  A brief History of the Development of the Hoek - Brown Failure Criterion.

3.6.1  The original Hoek - Brown failure criterion.

The original failure criterion was developed during the preparation of the book, titled "Underground Excavation in Rock", by Hoek, E. and Brown, E.T 1980. The criterion was required in order to provide input information for the design of underground excavations. Since no suitable methods for estimating rock mass strength appeared to be available at that time, the efforts were focused on developing a dimensionless equation that could be scaled in relation to geological information. The original Hoek- Brown equation was neither new nor unique since an identical equation had been used for describing the failure of concrete as early as 1936. The significant contribution that Hoek and Brown made was to link the equation to geological observations in the form of Bieniawski’s Rock Mass Rating and later to the Geological Strength Index.

The original criterion was conceived for use under the confined conditions surrounding underground excavations. The data upon which some of the original relationships have been based came from tests on rock mass samples from the Bougainville mine in Papua New Guinea. The rock mass here is very strong andesite (uniaxial compressive strength about 270 MPa) with numerous clean, rough, unfilled joints. One of the most important sets of data was from a series of triaxial tests carried out by J. Jaeger (1960) at the
Australian National University in Canberra. These tests were on 150 mm diameter samples of heavily jointed andesite from one of the exploring adits at Bougainville. The original criterion, which is based towards hard rock, was based upon the assumption that rock mass failure is controlled by translation and rotation of individual rock pieces, separated by numerous joint surfaces. Failure of intact rock was assumed to play no significant role in the overall failure process and it was assumed that the joint pattern was chaotic so that there are no preferred failure directions and the rock mass can be treated as isotropic. The original Hoek–Brown failure criterion is defined as:

\[
\sigma_1 = \sigma_3 + \sigma_{ci} \left( \frac{m}{\sigma_{ci}} + s \right)^{0.5}
\]

where

- \( m \) = constant depending on the characteristics of the rock mass
- \( s \) = constant depending on the characteristics of the rock mass
- \( \sigma_{ci} \) = uniaxial compressive strength of the intact rock material
- \( \sigma_1 \) = major principal stress at failure, and
- \( \sigma_3 \) = minor principal stress at failure.

One of the issues that had been troublesome throughout the development of the criterion has been the relationship between Hoek-Brown criterion, with the non-linear parameters \( m \) and \( s \), and the Mohr-Coulomb criterion, with the parameters \( c \) and \( \Phi \). Practically all software for soil and rock mechanics is written in terms of the Mohr-Coulomb criterion and it was necessary to define the relation between \( m \) and \( s \) and \( c \) and \( \Phi \) in order to allow the criterion to be used to provide input for this software.

An exact theoretical solution to this problem was developed by Dr J. W. Bray (1967) at the imperial College of Science and Technology and this solution was first published in the 1983 Rankine lecture. This publication also expanded on some of the concepts published by Hoek and Brown in 1980 and it represents the most comprehensive discussion on the original Hoek–Brown criterion.
3.6.2 The Updated Hoek – Brown Failure criterion

In 1988, Hoek and Brown presented an update to the original Hoek – Brown criterion. In this paper they defined a method of using Bieniawski’s RMR classification for estimating the input parameters. In order to avoid double counting and the effects of ground water and joint orientation, it was suggested that the rating for groundwater should always be set at 10 (completely dry) and the rating for joint orientation should always be set to zero (very favorable). In 1990 Hoek, E. estimated Mohr-Coulomb friction and cohesion values from the Hoek- Brown failure criterion.

3.6.3 The Modified Hoek-Brown Failure Criterion

Based upon work carried out in 1992 by Sandip Shah in his Ph.D thesis at the University of Toronto, a modified criterion was proposed. This criterion contained a new parameter ($\alpha$) that provided the means for changing the curvature of the failure envelope, particularly in the very low normal stress range. Basically, the modified Hoek –Brown criterion forces the failure envelope to produce zero tensile strength.

3.6.4 The Generalized Hoek Brown Failure Criterion.

It soon became evident that the modified criterion was too conservative when used for better quality rock masses and a ‘generalized’ failure criterion was proposed in two publications by Hoek, E. 1994 and Hoek, E., Kaiser, P.K. and Bawden. W.F. 1995. The generalized criterion incorporated both the original and the modified criteria with a ‘switch’ at an RMR value of approximately 25. Hence, for excellent to fair quality rock masses the original Hoek- Brown criterion was used while, for poor and extremely poor rock masses the modified criterion (published in 1992) with zero tensile strength is used. These two publications also introduced the concept of the geological strength index (GSI) as a replacement for Bieniawski’s RMR. It had become increasingly obvious that Bieniawski’s RMR is difficult to apply to very poor quality rock masses and also the relation between RMR and ($m$) and ($s$) is no longer linear in these very low ranges. The Generalized Hoek – Brown failure criterion for jointed rock mass is defined by:

\[
\sigma'_1 = \sigma'_3 + \sigma_{ci} \left( m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a
\]

Where

\[ (3.12) \]
$m_b$ is the value of the Hoek–Brown constant $m$ for the rock mass and $\alpha$ is the constant which depends upon the rock mass characteristics.

The most comprehensive paper was published by Hoek, E. and Brown, E. T. 1997 and it incorporated all of the refinements described above. In addition a method for estimating the equivalent Mohr Coulomb cohesion and friction angle was introduced. In this method the Hoek Brown criterion is used to generate a series of values relating axial strength to confining pressure (or shear strength to normal stress) and these are treated as hypothetical large scale triaxial or shear test.

A linear regression method is used to fined the average slope and intercept and these are then transformed into cohesive strength $c$ and friction angle $\Phi$. The most important aspect of this curve fitting process is to decide upon the stress range over which the hypothetical in situ tests should be carried out. This was determined experimentally by carrying out a large number of comparative theoretical studies in which the results of both surface and underground excavation stability analyses, using both the Hoek Brown and Mohr Coulomb parameters.

Hoek, E. Marinos, P. and Benissi, M. (1998) discussed the applicability of the geological strength index (GSI) classification for very weak and sheared rock masses. This paper extends the range of geological strength index (GSI) down to 5 to include extremely poor quality schistose rock masses such as the schist encountered in the excavations for the Athens metro and the graphitic phyllites encountered in some of the tunnels in Venezuela. This extension to GSI is based largely on the work of Maria Benissi on the Athens Metro. There are now two GSI charts. The first of these, for better quality masses published in 1994 and the new chart for very poor quality rock published in this paper.

Hoek, E. and Marinos, P. (2000) published a paper in which they introduced an important application of the Hoek Brown criterion in the prediction of the condition for tunnel squeezing, utilizing a critical strain concept proposed by Sakurai in 1983.

Hoek, E. and Karzulovic, A. (2000) published a paper, in this paper they repeated most of the material contained in Hoek and Brown, 1997 but added a discussion on blast damage.
The paper published by Marinos, P. and Hoek, E. (2001) on estimating the geotechnical properties of heterogeneous rock masses such as flyash, this paper does not add anything significant to the fundamental concepts of the Hoek – Brown criterion but they demonstrates how to choose appropriate range of GSI for different rock mass type.

The paper published by Hoek, E., Carranza-Torres, C. and Corkum, B. (2002) represents a major re-examination of the entire Hoek-Brown criterion and includes new derivations of the relationships between \( m, s, \alpha \) and GSI. A new parameter D is introduced to deal with blast damage. The relationship between Mohr Coulmb and the Hoek brown criteria is examined for slopes and underground excavations and a set of equations linking the two is presented.

A significant paper in which a new GSI chart for molassic rock masses is introduced, by Hoek, E., Marinos and Marinos V. (2004). The paper discusses the difference between these rock masses and the flyash type rocks which have been severely disturbed by organic processes.

The discussion on the range of the application and the limitations of GSI and the general guidelines for the use of GSI are given in the paper by V. Marinos, P. Marinos and E. Hoek (2005). Marinos, P. Hoek, E., Marinos, V. (2006) presented the geological model in which the ophiolitic complexes develop, their various petrographic types and their tectonic deformation, mainly due to over thrust also a GSI chart for ophiolitic rock masses is presented. The paper on empirical estimation of rock mass modulus was published by Hoek, E. and Diederichs, M.S (2006). While this paper is not directly related to the Hoek – Brown failure criterion, the deformation modulus of a rock mass is an important input parameter in any analysis of rock mass behavior that include deformations. Several authors have proposed empirical relationships for estimating the value of rock mass deformation modulus on the basis of classification schemes. These relationships are reviewed and their limitations are discussed on this paper. Also based on a data from a large number of in situ measurements from China and Taiwan, a new relationship between the deformation modulus and GSI is proposed. A summary of all the changes to the equations for estimating the shear strength of rock masses using the Hoek – Brown criterion was presented by Hoek (2002) and is reproduced here as Table (3.5) below.
<table>
<thead>
<tr>
<th>Publication</th>
<th>Coverage</th>
<th>Equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hoek &amp; Brown 1980</td>
<td>Original criterion for heavily jointed rock masses with no fines. Mohr envelope was obtained by statistical curve fitting to a number of ((\sigma_n, \tau)) pairs calculated by the method published by Balmer [28]. (\sigma_1', \sigma_3') are major and minor effective principal stresses at failure, respectively. (\sigma_n, \tau) are effective normal and shear stresses, respectively.</td>
<td>(\sigma_1' = \sigma_2' + \sigma_3' \sqrt{\frac{m \sigma_2'}{\sigma_3'}} + s)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\sigma_1 = \frac{\sigma_1'}{2} \left[ \frac{m - \sqrt{m^2 + 4s}}{2} \right] + \frac{\sigma_1'}{2})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\tau = \sigma_1 \left( (\sigma_n - \sigma_1) / \sigma_3 \right)^B)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\sigma_n' = \sigma_3' + \left( (\sigma_1' - \sigma_3') / (1 + \partial \sigma_1' / \partial \sigma_3') \right))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\tau = (\sigma_n - \sigma_3) \sqrt{\frac{\partial \sigma_1}{\partial \sigma_3}})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\partial \sigma_1' / \partial \sigma_3' = m \sigma_3 / 2(\sigma_1' - \sigma_3'))</td>
</tr>
<tr>
<td>Hoek 1983</td>
<td>Original criterion for heavily jointed rock masses with no fines with a discussion on anisotropic failure and an exact solution for the Mohr envelope by Dr J.W. Bray.</td>
<td>(\sigma_1' = \sigma_3' + \sigma_3' \sqrt{\frac{m \sigma_2'}{\sigma_3'}} + s)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\tau = \frac{\sigma_1' - \sigma_3'}{2} - \frac{\sigma_3'}{2} \left[ \frac{m - \sqrt{m^2 + 4s}}{2} \right] + \frac{\sigma_3'}{2})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\phi_i = \arctan \left[ \frac{1}{\sqrt{4h \cos^2 \theta - 1}} \right])</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\theta = \arctan (1 / \sqrt{h^3 - 1}) / 3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(h = 1 + \left( 16 \sigma_1' + s \sigma_3' \right) / (3m^2 \sigma_3'))</td>
</tr>
<tr>
<td>Hoek &amp; Brown 1988</td>
<td>As for Hoek 1983 but with the addition of relationships between constants (m) and (n) and a modified form of RMR in which the Groundwater rating was assigned a fixed value of 10 and the Adjustments for Joint Orientation were set at 0. Also a distinction between disturbed and undisturbed rock masses was introduced together with means of estimating deformation modulus (E) (after Semnfin and Pereira).</td>
<td>(n_h / n_i = \exp((RMR - 100) / 14))</td>
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<td></td>
<td></td>
<td>(s = \exp((RMR - 100) / 6))</td>
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<tr>
<td></td>
<td></td>
<td>DISTURBED rock masses:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(n_h / n_i = \exp((RMR - 100) / 28))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(s = \exp((RMR - 100) / 9))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(E = 10(RMR-100).40)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(n_h, n_i) are for broken and intact rock, respectively.</td>
</tr>
<tr>
<td>Hoek, Wood &amp; Shah 1992</td>
<td>Modified criterion to account for the fact the heavily jointed rock masses have zero tensile strength. Balmer's technique for calculating shear and normal stress pairs was utilised.</td>
<td>(\sigma_1' = \sigma_3' + \sigma_3' \left( m \sigma_3' / \sigma_3 \right))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\sigma_n' = \sigma_3' + \left( (\sigma_1' - \sigma_3') / (1 + \partial \sigma_1' / \partial \sigma_3') \right))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\tau = (\sigma_n - \sigma_3) \sqrt{\frac{\partial \sigma_1}{\partial \sigma_3}})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\partial \sigma_1' / \partial \sigma_3' = 1 + am_\sigma_1' (\sigma_3' / \sigma_3) (\alpha - 1))</td>
</tr>
<tr>
<td>Hoek, Kaiser &amp; Bawden 1995</td>
<td>Introduction of the Generalised Hoek-Brown criterion, incorporating both the original criterion for fair to very poor quality rock masses and the modified criterion for very poor quality rock masses with increasing fines content. The Geological Strength Index GSI was introduced to overcome the deficiencies in Bieniawski's RMR for very poor quality rock masses. The distinction between disturbed and undisturbed rock masses was dropped on the basis that disturbance is generally induced by engineering activities and should be allowed for by downgrading the value of GSI.</td>
<td>(\sigma_1' = \sigma_3' + \sigma_3' \left( m \sigma_3' / \sigma_3 \right))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>for GSI &gt; 25 (n_h / n_i = \exp((GSI - 100) / 28))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(s = \exp((GSI - 100) / 9))</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a = 0.5)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>for GSI &lt; 25 (s = 0)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(a = 0.65 - \frac{GSI}{200})</td>
</tr>
</tbody>
</table>
Figure 3.5: A summary of all the changes to the equations for estimating the shear strength of rock masses using the Hoek – Brown failure criterion. (Aft Hoek and Brown 2006)
3.7 Fitting the Intact Hoek – Brown Failure Criterion to Test Data.

Hoek and Brown (1997) proposed linear regression of the triaxial data for determining the uniaxial compressive strength, $\sigma_{ci}$ and intact material constant, $m_i$. This method expresses the intact Hoek – Brown failure criterion in a linear form as follows:

\[
Y = m \sigma_c x + \sigma_c^2 \tag{3.13}
\]

Where,\[y = (\sigma_1 - \sigma_3) \tag{3.14}\]

And \[x = \sigma_3 \tag{3.15}\]

For these analysis Hoek and Brown (1997) recommended that $0 < \sigma_3 < 0.5 \sigma_{ci}$ and the number of data points $n \geq 5$.

\[
\sigma_{ci} = \sqrt{\frac{\sum y}{n} - \left(\frac{\sum x y - \frac{\sum x \sum y}{n}}{\frac{\sum x^2 - (\sum x)^2}{n}}\right)} \tag{3.16}
\]

This value for uniaxial compressive strength is substituted into equation (3.17) below to find $m_i$.

\[
m_i = \frac{1}{\sigma_c} \left[\frac{\sum x y - \frac{\sum x \sum y}{n}}{\sum x^2 - (\sum x)^2} \right] \tag{3.17}
\]

The fit of the empirical criterion to the data will be quantified by the coefficient of determination $r^2$. 

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The problem in deriving the parameters from the up linear regression method is that the Hoek – Brown criterion is non-linear.

Mostyn and Douglas (2000) found that significant errors existed when fitting the intact Hoek – Brown failure criterion by linear regression to a set of intact strength data especially within the tensile region of the criterion. The region where \( \sigma_3 < 0 \) on the principal stress axis represents the region of greatest curvature of the Hoek – Brown failure criterion and therefore two points at equal distance in \( \sigma_1 \) plane either side of the Hoek – Brown failure criterion in this region will not be at equal perpendicular distances to the criterion. This situation occurs because the least squares procedure only considers the error between the measured \( \sigma_1 \) and the predicted \( \sigma_1 \), i.e. in \( \sigma_1 \) plane only.

To ensure that the Hoek – Brown criterion was defined for the full range of \( \sigma_3 \), Mostyn and Douglas (2000) proposed the following alteration to the Hoek – Brown equation,

\[
\sigma_1 = \sigma_3 + \sigma_t \left( m_i \frac{\sigma_3}{\sigma_t} + 1 \right)^{0.5} \quad \text{for } \sigma_3 > \frac{-\sigma_t}{m_i} \\
\sigma_1 = \sigma_3 \quad \text{for } \sigma_3 \leq \frac{-\sigma_t}{m_i}
\]

Also because the Hoek – Brown failure criterion is not defined for values lower than the fitted tensile strength, many methods will be forced to select the lowest measured tensile strength to the curve (Mostyn and Douglas, 2000). Clearly the curve should be fitted to the average of the tensile strength data. This is important as Lade (1993) found that tensile data offered a good control on the failure envelopes over the low stress range. Mostyn and Douglas (2000) proposed a modification to the least squares procedure by defining the least squares error as;
In this way the minor principal stress $\sigma_3$ values are used to fit the Hoek – Brown criterion in the tensile region. However, because differences between $\sigma_3$ values are small compared to differences between $\sigma_1$ values, using $\sigma_3$ differences to calculate errors will create artificially small errors. The regression procedure will then place more emphasis on minimizing errors between $\sigma_1$ differences compared to errors in $\sigma_3$ differences. To make $\sigma_3$ differences more comparable, Mostyn and Douglas (2000) suggested scaling of the $\sigma_3$ differences by multiplying them by $m_i$ as shown in equation (3.20). Equations (3.19) and (3.20) were found by Mostyn and Douglas (2000) to give superior fits at low confining stresses to a large number of data sets compared to the traditional Hoek – Brown curve fitting techniques.

3.7.1 Uniaxial compressive strength $\sigma_{ci}$ of intact rock.

The uniaxial compressive strength $\sigma_{ci}$ was chosen as input parameter for the Hoek Brown failure criterion because it was the most widely available parameter in the rock mechanics literature (Hoek and Brown, 1980). It was also used as a scale factor whereby dividing the principal stresses by $\sigma_{ci}$, to make the criterion dimensionless so that it could be scaled easily to available geological information on the desired rock type (Hoek, 2002). The uniaxial compressive strength, $\sigma_{ci}$, is therefore widely accepted as an essential parameter within a failure criterion to provide an appropriate scale to a given set of data. For this reason, most failure criteria include the uniaxial compressive strength somewhere in the criteria formulation. Douglas (2002), Richards’s et al. (2001) and Mostyn and Douglas (2000) have demonstrated that the use of the uniaxial compressive strength as independent parameter is justified.

The uniaxial compressive strength tests results are usually either generated from uniaxial compressive tests or, if no suitable sized cores exist for compression testing, calculated using correlations from point load or Schmidt hammer test results. There is usually more variability in calculating UCS values from point load and Schmidt hammer tests so direct determination of the uniaxial compressive strength in the testing machine is preferred. However suitable sized samples are not always available, especially in jointed rock masses. In which case, point load tests offer a less preferable alternative. Hoek and
Brown (1997) outlined a process using linear regression to calculate $\sigma_{ci}$ from series of triaxial tests (see equation 3.16). If there are no available test results upon which to base an estimates of $\sigma_{ci}$, Table (3.3) can be used to estimate the unconfined compression test from field observations (Brown, 1981). This value will therefore be highly subjective and the criterion should be checked to assess the sensitivity based on this estimate.

**Table 3.3: Field values of uniaxial compressive strength (from Hoek, 2000).**

<table>
<thead>
<tr>
<th>Grade*</th>
<th>Term</th>
<th>Uniaxial Comp. Strength (MPa)</th>
<th>Point Load Index (MPa)</th>
<th>Field estimate of strength</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>R6</td>
<td>Extremely strong</td>
<td>&gt; 250</td>
<td>&gt;10</td>
<td>Specimen can only be chipped with a geological hammer</td>
<td>Fresh basalt, chert, diabase, gneiss, granite, quartzite</td>
</tr>
<tr>
<td>R5</td>
<td>Very strong</td>
<td>100 – 250</td>
<td>4 – 10</td>
<td>Specimen requires many blows of a geological hammer to fracture it</td>
<td>Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff</td>
</tr>
<tr>
<td>R4</td>
<td>Strong</td>
<td>50 – 100</td>
<td>2 – 4</td>
<td>Specimen requires more than one blow of a geological hammer to fracture it</td>
<td>Limestone, marble, phyllite, sandstone, schist, shale</td>
</tr>
<tr>
<td>R3</td>
<td>Medium strong</td>
<td>25 – 50</td>
<td>1 – 2</td>
<td>Cannot be scarped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer</td>
<td>Claystone, coal, concrete, schist, shale, siltstone</td>
</tr>
<tr>
<td>R2</td>
<td>Weak</td>
<td>5 – 25</td>
<td>**</td>
<td>Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer</td>
<td>Chalk, rocksalt, potash</td>
</tr>
<tr>
<td>R1</td>
<td>Very weak</td>
<td>1 – 5</td>
<td>**</td>
<td>Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife</td>
<td>Highly weathered or altered rock</td>
</tr>
<tr>
<td>R0</td>
<td>Extremely weak</td>
<td>0.25 – 1</td>
<td>**</td>
<td>Indented by thumbnail</td>
<td>Stiff fault gouge</td>
</tr>
</tbody>
</table>

* Grade according to Brown (1981).

** Point load tests on rocks with a uniaxial compressive strength below 25MPa are likely to yield highly ambiguous results.

Hoek (1999) suggested that the ratio of the uniaxial compressive strengths in the field and in the laboratory can be estimated from the following equation which involves the GSI.

$$\left( \frac{\sigma_{GSI}}{\sigma_{ci}} \right) = 0.022e^{0.086\text{GSI}}$$

------------------------ (3.21)

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3.7.2 **Intact material constant \( m_i \)**

Hoek and Brown, (1980) stated that \( m_i \) is dependent upon the initiation and propagation of fracture and determination of the curvature of the Hoek –Brown failure envelope. Hoek (1983) stated that \( m_i \) is influenced by such properties as the mineral composition, grain packing pattern, nature of cementing matrix, degree of interlock between particles, grain size and angularity. If these properties in a certain rock type are quite variable, it follows that there will be similar variability within \( m_i \) values for that rock type.

**Table 3.4: Values of the constant \( m_i \) for intact rock (from Hoek and Marinos, 2000).**

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Class</th>
<th>Group</th>
<th>Texture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Clastic</td>
<td></td>
<td>Coarse</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Conglomerate (22)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Greywacke (18)</td>
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<tr>
<td></td>
<td></td>
<td>Non-Clastic</td>
<td>Breccia (20)</td>
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<tr>
<td></td>
<td></td>
<td>Chemical</td>
<td>Gyptstone 16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SEDIMENTARY</td>
<td></td>
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</tbody>
</table>

While determination of \( m_i \) by regression of triaxial tests is the preferred method, Hoek and Brown (1980) provided tables for the selection of \( m_i \) based on the rock type. These tables have been progressively updated and expanded throughout the development of the Hoek – Brown failure criterion. The latest table (Hoek and Marinos, 2000) is reproduced in Table 3.4.
Note that for intact rock, when $\sigma_1 = \sigma_3 = \sigma_t$

$$m_i = - \frac{\sigma_{ci}}{\sigma_t}$$

Mostyn and Douglas (2000) have demonstrated that if the value for $\sigma_t$ can be well defined, then this offers a good control on the failure envelope at low stress ranges. This is in accordance with recommendations of Lade (1993) that the tensile strength should be one of the three independent characteristics of any failure criterion. However, while the estimation of the tensile strength in the lab is reasonably simple, Lade (1993) noted that the ratio of uniaxial compressive strength to tensile strength can vary widely. Johnston (1985) noted that the $\sigma_c / \sigma_t$ ratio can vary from 2.5 up to 30 and appears to be a function of the compressive strength and rock type. Hoek and Brown (1980) preferred to use $m_i$ as an empirical curve fitting parameter instead of $\sigma_t$ because of the difficulty to use $\sigma_t$ as the fundamental rock property.

Figure 3.6 shows the effect of $m_i$ upon the Hoek–Brown failure criterion. For large values of $m_i$ (15 - 25), Mohr envelopes are steeply inclined with high instantaneous (tangent) friction angles at low normal stress. These envelopes are indicative of typical failure envelopes for brittle igneous and metamorphic rock masses such as andesite, gneiss and granites. Lower $m_i$ values (3 - 7) were typical of the failure envelope to rocks such as more ductile carbonates such as limestone and dolomite.

Figure 3.6 shows that lowering the value of $m_i$ increases the curvature of the failure envelope. Lade (1993) noted that the curvature (of a failure criterion) is related to interaction between dilation at low confining pressures and crushing at high confining pressure.

**Figure 3.6: Effect of increasing $m_i$ on intact Hoek–Brown envelopes.**
Mostyn and Douglas (2000) plotted Hoek-Brown envelopes from published values as shown in Figure 3.7 and Figure 3.8. These figures feature a variety of rock types within published ranges. The principal stress axes have been normalized by dividing the principal stresses by $\sigma_c$ to make them directly comparable. They show that the criterion under predicted failure strengths when published were low and over predicted when published were high at both low and high confining stresses. Therefore some concern is warranted when using published values of $m_i$ to estimate the strength of intact rock masses.

Figure 3.7: Principal stress plots of Hoek-Brown fits to different rock types using $m_i$ values for high confining stresses (from Mostyn and Douglas, 2000)
Figure 3.8: Principal stress plots of Hoek-Brown fits to different rock types using m_i values for low confining stresses (from Mostyn and Douglas, 2000)

Using a database of 475 data sets of triaxial data on intact rock, Mostyn and Douglas (2000) calculated m_i values from these datasets using the procedure outlined in Hoek and Brown (1980) and compared these results with published m_i values found in Hoek et al., (1995) and Hoek and Brown (1997).

Mostyn and Douglas (2000) found a weak correlation ($r^2 = 16.4\%$) between published m_i and m_i derived from tests. The results published by Mostyn and Douglas (2000) showed very large range in values of m_i. Mostyn and Douglas (2000) concluded that m_i could not be easily predicted from rock type alone.

Richards et al. (2001), using triaxial data for sandstones, showed that the m_i values for sandstone have the potential to vary considerably beyond the limit given in the latest table of m_i in Hoek and Marinos (2000). For a total of 46 sets of triaxial data for sandstone, Richards et al. (2001) found an average value m_i value of 14 within arrange of 4 to 35.
This analysis supports the preference of Hoek and Brown (1997) to select \( m_i \) values on the basis of triaxial tests rather than from the table given in Marinos and Hoek (2000). However, it also raises questions as to how reliable Table 3.4 is to predict the correct value of \( m_i \). The results of the analysis by Richards et al. (2001) shows that if there is a lack of test data and \( m_i \) is selected from tables, a sensitivity analysis should be undertaken to check the impact that an extreme \( m_i \) would have upon the estimate provided by a rock mass strength envelope. Richards (2004) notes that reported values for \( m_i \), especially in weaker materials could be affected by pore pressure if they were derived from tests conducted in Hoek cells where the drainage and pore pressure measurement are not easy. The influence of the value of \( m_i \) on the rock mass strength is significantly less than that of either GSI or \( \sigma_{ci}\). Marinos P. Marinos and E. Hoek (2005)

3.7.3 The Hoek – Brown parameter Geological Strength Index (GSI).

The GSI provides a system for estimating the reduction in rock mass strength for different geological conditions. Values of GSI are related to both the degree of fracturing and the condition of fracture surface, as shown in Table (3.5) below. The strength of jointed rock mass depends on the properties of intact rock piece, as well as the freedom of the rock piece to slide and rotate under difference stress conditions. This freedom is controlled by the geometrical shape of the intact rock piece and the condition of the surface separating the pieces. Angular rock piece with clean, rough surface will result in much stronger rock mass than one that contains rounded particles by weathered and altered material.

This index was first introduced by Hoek et al. (1992) and further developed Hoek, (1994) and Hoek, Kaiser and Bawden (1995). This index was subsequently extended for weak rock masses in series of papers by Hoek et al. (1998), Marinos and Hoek (2000, 2001).

Table 3.5: blocks sizes and discontinuity space for rock mass structure terms.

<table>
<thead>
<tr>
<th>Structure category</th>
<th>Rock mass structure (Hoek et al., 1992)</th>
<th>Dimension</th>
<th>Equivalent defect spacing</th>
<th>Structure category (Hoek et al., 1998)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blocky</td>
<td>Very large</td>
<td>(&gt;2 m)</td>
<td>Extremely wide</td>
<td>Blocky</td>
</tr>
<tr>
<td></td>
<td>Large</td>
<td>(600 mm - 2 m)</td>
<td>Very wide</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>(200 - 600 mm)</td>
<td>Wide</td>
<td>Very blocky</td>
</tr>
<tr>
<td>Blocky/seamy</td>
<td>Small</td>
<td>(60 - 200 mm)</td>
<td>Moderately wide</td>
<td>Blocky/disturbwed</td>
</tr>
<tr>
<td>Crushed</td>
<td>Very small</td>
<td>(&lt;50 mm)</td>
<td>&lt; Moderately wide</td>
<td>Disintegrated</td>
</tr>
<tr>
<td></td>
<td>(assume &lt;20 mm)</td>
<td></td>
<td></td>
<td>Foliated/laminated/ sheared</td>
</tr>
</tbody>
</table>
GSI is similar to RMR but incorporates also newer versions of abaieniawski's original system (Bieniawski, 1976, 1989). Hence, the following relations were developed (Hoek, Kaiser and Bawden, 1995).

For RMR \(_{76} > 18\):

\[
\text{GSI} = \text{RMR}_{76}. \quad \text{------------------------ (3.23)}
\]

For RMR\(_{89} \geq 23\):

\[
\text{GSI} = \text{RMR}_{89} - 5 \quad \text{------------------------ (3.24)}
\]

For both versions, dry conditions should be assumed, i.e. assigning a rating of 10 in RMR\(_{76}\), and a rating of 15 in RMR\(_{89}\), for ground water category in each classification system. Also, no adjustments for joint orientation should be made.

For rock mass with RMR\(_{76} < 18\) and RMR\(_{89} < 23\), the RMR system cannot be used, since these are the minimum values that can be obtained in each of this versions, respectively. For these cases, i.e. very poor quality rock masses, the NGI – index (Barton, Lien and Lunde, 1974) should be used instead. In using this classification system to estimate GSI, the joint water reduction factor (Jw) and the stress reduction factor (SRF) should be set to 1.
Table 3.6: Geological strength index for blocky jointed rock masses, (after Heok-Brown, 1997)

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>SURFACE CONDITIONS</th>
<th>DECREASING SURFACE QUALITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities</td>
<td>VERY GOOD - very rough, fresh unweathered surfaces</td>
<td>N/A</td>
</tr>
<tr>
<td>BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets</td>
<td>GOOD - slightly weathered, iron stained surfaces</td>
<td>N/A</td>
</tr>
<tr>
<td>VERY BLOCKY - interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets</td>
<td>FAIR - smooth, moderately weathered and altered surfaces</td>
<td></td>
</tr>
<tr>
<td>BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity</td>
<td>POOR - slickensided, highly weathered surfaces with compact coatings or fillings</td>
<td></td>
</tr>
<tr>
<td>DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces</td>
<td>VERY POOR - slickensided, highly weathered surfaces with soft clay coatings or fillings</td>
<td></td>
</tr>
<tr>
<td>LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
3.7.4 The Hoek – Brown parameter disturbance factor D.

Experience in the design of excavations in open pit mines has shown that the Hoek – Brown criterion for undisturbed in situ rock masses \( (D = 0) \) results in rock mass properties that are too optimistic (Pierce et al., 2001; Sjoberg et al., 2001).

The effect of heavy blast damage as well as stress relief due to removal of the over burden results in disturbance of the rock mass (Hoek and Brown, 1988).

A number of other studies to assess the degree of disturbance of the rock mass have been carried out by observing the surface and underground excavations. For example, Lorig and Varona (2001) showed that factors such as the lateral confinement produced by different radii of curvature of slopes (in plan) as compared with their height also have an influence on the degree of disturbance. Also, Sonmez and Ulusay (1999) back analysis five slope failures in open pit mines in Turkey and attempted to assign disturbance factors to each rock mass based upon their assessment of the rock mass properties predicted by the Hoek-Brown criterion.

Cheng and Liu (1990) report the results of very careful back analysis of deformations measurements, from extensometers placed before the commencement of excavations, in the Mingtan power cavern in Taiwan. It was found that a zone of blast damage extended for a distance of approximately 2 m around all large excavations. The back calculated strength and deformation properties of the damage rock mass give an equivalent disturbance factor \( D = 0.7 \).

From these references it is clear that a large number of factors can influence the degree of disturbance in the rock mass surrounding an excavation, and that it may never be possible to quantify these factors precisely. However, based on experience and on the analysis of the detailed contained in these papers, Hoek et al. (2002) have drawn up a set of guidelines for estimating the factor \( D \), as shown in Figure (3.9) below.

3.8 The mechanical properties of rock mass:

Having defined the parameters \( \sigma_{ci}, m_i, GSI \) and \( D \) as described above, the next step is to estimate the mechanical properties of the rock mass. The procedures for making this estimate has been described in detail by Hoek – Brown (1997).

3.8.1 Hoek – Brown parameter, \( m_b \):

This parameter (which may be considered similar to the friction angle in the Mohr Coulomb criterion) and which is the reduced value of the material constant \( m_i \). For undisturbed rock masses this parameter can be determined by the following equation:

\[
m_b = m_i \exp\left(\frac{GSI - 100}{28}\right)
\]

------------------ (3.25)
For disturbed rock masses the following equation can be used:

\[ m_b = m_i \exp \left( \frac{GSI - 100}{28 - 14D} \right) \]

(3.26)

The parameter \( m_b \) in Eqn. (2.14) and Eqn. (2.15) depends on both the intact rock parameter \( m_i \) and the GSI value.

<table>
<thead>
<tr>
<th>Appearance of rock mass</th>
<th>Description of rock mass</th>
<th>Suggested value of ( D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.</td>
<td>( D = 0 )</td>
<td></td>
</tr>
<tr>
<td>Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.</td>
<td>( D = 0 )</td>
<td></td>
</tr>
<tr>
<td>Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.</td>
<td>( D = 0.8 )</td>
<td></td>
</tr>
<tr>
<td>Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.</td>
<td>( D = 0.7 ) Good blasting</td>
<td></td>
</tr>
<tr>
<td>Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.</td>
<td>( D = 1.0 ) Production blasting</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.9: Guidelines for estimating disturbance factor \( D \) (Hoek et al. 2002)
3.8.2 **Hoek – Brown parameters (s) and (a).**

The first parameter is the cohesive component of the Generalized Hoek – Brown failure criterion and is already small number. The (a) parameter essentially controls the curvature of the Generalized Hoek – Brown failure envelope, especially at low confining stresses.

These parameters can be given by the following equation for undisturbed rock masses with GSI >25, i.e. rock mass of good to reasonable quality.

\[ s = \exp\left(\frac{GSI - 100}{9}\right) \]

\[ \text{and} \]

\[ a = 0.5 \]

For disturbed rock masses the following equation is used.

\[ s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \]

For GSI < 25, i.e. rock masses of very poor quality the Hoek brown criterion applies with.

\[ S = 0 \]

\[ \text{and} \]

\[ a = 0.65 - \frac{GSI}{200} \]

Hoek – Brown (2002) suggests the following equation for all values of GSI to determine the parameter a.

\[ a = \frac{1}{2} + \frac{1}{6} \left( e^{-GSI/15} - e^{-20/3} \right) \]

Estimation of Hoek – Brown parameters before and after 2002 is given in Table 3.7. The maximum and minimum values of Hoek Brown parameters are shown in Table 3.8.
Table 3.8 Maximum and minimum values of Hoek Brown parameters (Hoek Brown, 1997)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_b / m_i$</td>
<td>0.034</td>
<td>0.49</td>
</tr>
<tr>
<td>$s$</td>
<td>0.000026</td>
<td>0.11</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>0.50</td>
<td>0.62</td>
</tr>
</tbody>
</table>

From the above equations it is clear that the rock mass strength parameters are sensitive to the GSI value. Hoek and Brown, 1997 indicated that the choice of GSI = 25 for the switch between the original and modified criteria was purely arbitrary, and it could be argued that GSI = 30 provide a continuous transition in the value of $\alpha$. Also as stated by the same investigators, extensive trials have shown that the exact location of this switch GSI value has negligible practical significance, and the predicted strength using the original relations had proved to be too conservative (giving too low strengths) for most applications. From this it was concluded that the specimens of Angina andesite in reality probably were disturbed and the particles interlocking destroyed (Hoek and Brown 1988). The previously suggested relations between GSI and values for $m$, $s$, and $\alpha$ (Hoek and Brown 1980) were therefore viewed as representative of a disturbed rock mass (equations 3.14, 3.16 and 3.20).

3.8.3 Indirect estimates of rock mass modulus

The deformation modulus of rock mass is an important input parameter in any analysis of rock mass behavior that includes deformations. For example in designing the primary support and final lining for a tunnel, the deformation of a rock mass surrounding the tunnel are important and a numerical analysis of these deformation requires an estimate of the rock mass deformation
modulus. Similarly to the problem of predicting the rock mass strength, it is also very difficult and expensive to determine in the field. Predicting the rock mass deformability is complicated by the lack of suitable method to determine the effect of discontinuity network upon deformability of rock mass. Similarly to the strength of jointed rock, the modulus of jointed rock varies significantly depending on the proportion of the intact rock in a rock mass. Heuze (1980) found that the deformation modulus of rock masses vary between 20-60% of the intact modulus, E.

In the 1960’s several attempts were made to use Deere’s RQD for estimating in situ deformation modulus, but this approach is seldom used today (Deere, 1988). Some theoretical expressions have been derived for simple joint geometries (Amadei and goodman, 1981, Gerrad, 1982). Fossum (1985) derived a mathematical procedure to calculate the deformation modulus of an equivalent continuum for a randomly jointed rock mass. These simple expressions are inadequate as they assume the discontinuities are dispersed in a regular manner that is not realistic for practical purposes.

Bieniawski (1978) analyzed a number of case histories and proposed the following relationship for estimating the in situ deformation modulus, \( E_m \) from RMR.

\[
E_m = 2RMR - 100 \text{ (GPa) for } E_m < 50
\]

\[ \text{(3.33)} \]

Barton et al. (1985) have found good agreement between measured displacements and predictions from numerical analyses using in situ deformation modulus values estimated from

\[
10\log_{10} Q < E < 40\log_{10} Q \quad E_{\text{max}} = 25\log_{10} Q.
\]

\[ \text{(3.34)} \]

This relationship was modified to produce an equation relating deformation modulus to the rock mass rating (Serafim and Pereira, 1983) as follows;

\[
E = 10^{\left(\frac{RMR-10}{40}\right)} \text{ (GPa)}
\]

\[ \text{(3.35)} \]

This relationship, based on back analysis of dam foundations was found to work well for good quality rock masses but predicted higher values for poor quality rock masses. Wyllie (1999) stated that the equation by Serafim and Pereira (1983) on rock masses with poor to very good qualities indicates that modulus is related to the rock mass rating over the range 20-85. Also , the influence of discontinuity orientation should be taken into account in the settlement and stability analysis, where the favorable orientation for settlement ( i.e. parallel to loading ) would be unfavorable for sliding.
Ramamurthy (1993) has proposed relationships for the modulus by relating the modulus of jointed rock in an unconfined state to that of the intact rock and the joint factor, $J_f$ as follows;

$$E_{\text{jointed}} = E_{\text{intact}} \exp\left[-1.15 \times 10^{-2} J_f\right] \quad (3.36)$$

where the joint factor, $J_f = \frac{J}{n}$

and $J$ is the joint frequency (number of joints/meter), $n$ is an inclination parameter and $r$ is the joint strength parameter.

Hoek and Brown (1997) modified equation (3.35) noting that GSI > 25 are approximately equal to RMR values, on the bases of practical observations and back analysis of excavation behavior in poor rock quality masses to give for rock masses with $\sigma_{ci} < 100$ MPa.

$$E_m = \sqrt{\frac{\sigma_{ci}}{100}} \left(\frac{GSI-10}{40}\right) \quad (GPa) \quad (3.37)$$

where $\sigma_{ci} < 100$ MPa.

Hoek and Brown (1997) included the reduction factor $\sqrt{(\sigma_{ch} / 100)}$ to account for the degradation in rock strength with poor quality rock masses.

Hoek et al. (2002) proposed a further modification to allow for existing damage for exposure surface, by introducing the factor, $D$ as follows;

$$E_m = \left(1 - \frac{D}{2}\right) \sqrt{\frac{\sigma_{ci}}{100}} \left(\frac{GSI-10}{40}\right) \quad (3.38)$$

Where $E_m$ is calculated in GPA.

Clearly, the deformability of jointed rock mass should not exceed the deformability of the intact rock material. Read et al. (1999) calculated the rock mass deformability of intact rock (RMR = 100) using the equation proposed by Serafim and Pereira (1983) and Hoek and Brown (1997). He found that the predicted values were greater than the actual by a factor of three. They proposed a new expression relating rock mass deformability ($E_m$) and the rock mass rating (RMR) as follows;
This expression gave the more realistic values for \( E_m = 100 \) GPA at \( RMR = 100 \). In addition they stated that the predicted rock mass modulus should be normalized to ensure that rock mass deformability does not exceed the intact rock modulus as follows:

\[
E_m(\text{norm}) = \frac{E_m}{E_{100}}
\]  

These expressions are based on the tangent modulus of the rock mass. Duncan and Chang (1970) have suggested relationships between elastic modulus, axial strain and confining pressure which while originally derived for soils have been adapted to rocks by Kula (1975). These expressions have been modified to predict the deformability of tectonic rock masses (Habimana et al., 2002) and regularly rock masses (Sridevi and Sitharam, 2000).

Projects in China and Taiwan, Hoek and Diederichs (2005) proposed the following equation for determining the rock mass deformability modulus \( E \).

\[
E = E (0.02 + 1 - D/2)/1+\exp (60+15D -GSI)/11)
\]  
The simplified Hoek and Diederichs equation for rock mass modulus, (Hoek and Diederichs (2006) only requires GSI and D as input parameters. The modulus is calculated as follows.

\[
E_{m}(\text{MPa}) = 100,000 \frac{1 - D/2}{\left(1 + e^{0.75+2D-(80)/11)}\right)}
\]  

3.8.4 In-Situ Measurements of Deformation Modulus

All in situ deformation tests are expensive and difficult to conduct. Initial preparations at each test site are particularly time consuming. The following types of in situ tests are mostly used to determine the modulus of deformation:

- Plate jacking tests (PJT).
- Plate loading tests (PLT)
- R adial jacking tests (Goodman jack test)
- Flat jack tests
- Cable jacking tests
- Dilatometer tests
- Pressure meter tests
The effect of Poisson’s ratio is one of the parameters used for the calculation of modulus value in an in situ test. Sharma and Singh (1989) found that there is not much variation in the values of the deformation modulus if the value of the Poisson’s ratio is between 0.1 and 0.35.

3.9 Mohr Coulomb Criterion Applied to Rock Masses.

Nearly all software developed for soil and rock mechanics analysis provides the Mohr Coulomb constitutive model for simulation of plastic behavior. However, there is no generally accepted direct method to estimate the Mohr Coulomb parameters (C & $\Phi$) for rock mass. It has, therefore, become interesting to evaluate these parameters using a linear regression analysis of the non-linear Hoek-Brown failure envelope. This would allow the Hoek-Brown criterion to be used as input to numerical analysis. The latest version of fitting an average linear relationship to the curve generated by the generalized Hoek–Brown criterion was done in 2002, (Hoek et al., 2002). Hoek et al., (2002) presented the equations for the effective angle of friction $\Phi$ and effective Cohesive strength $C$ as.

\[
\phi' = \sin^{-1} \left[ \frac{6am_b(s + m_b\sigma'_{3n})^{\alpha-1}}{2(1+\alpha)(2+\alpha) + 6am_b(s + m_b\sigma'_{3n})^{\alpha-1}} \right]
\]

\[
c' = \frac{\sigma_{ci}(1+2\alpha)s + (1-\alpha)m_b\sigma'_{3n} + (s + m_b\sigma'_{3n})^{\alpha-1}}{(1+\alpha)(2+\alpha)\sqrt{1 + \left(\frac{6am_b(s + m_b\sigma'_{3n})^{\alpha-1}}{(1+\alpha)(2+\alpha)}\right)^2}}
\]

Based on balancing the areas above and below the Mohr Coulomb plot, where $\sigma_{3n} = \sigma_{3\text{max}} / \sigma_{ci}$

The parameters $m_b$, $s$, and $\alpha$ are defined in this chapter before. The upper limit of confining stress ($\sigma_{3\text{max}}$) over which the relationship between Mohr-Coulomb and Hoek–Brown criteria can be seen in Figure 3.10.
For underground excavation the relation between $\sigma_{3 \text{ max}}$ and $\sigma_{cm}$ for equivalent Mohr–Coulomb and Hoek–Brown parameters is suggested to be (Hoek et al., 2002).

$$\frac{\sigma'_{3 \text{ max}}}{\sigma'_{cm}} = 0.47 \left( \frac{\sigma'_{cm}}{\gamma H} \right)^{-0.84} \tag{3.45}$$

Where $\gamma H$ is the maximum virgin or primary stress acting perpendicular to the tunnel axis and $\sigma_c$ is the effective rock mass strength. If the vertical stress is the maximum stress, then $\sigma_{\text{m in situ}} = \gamma H$, where $\gamma$ is the unit weight of the rock and $H$ is the depth of tunnel below surface.

This expression is applicable for deep tunnels and also shallow tunnels. And for slopes;

$$\sigma'_{3 \text{ max}} = 0.72\sigma'_{cm} \left( \frac{\sigma'_{cm}}{\gamma H} \right)^{-0.91} \tag{3.46}$$

The linear Mohr–Coulomb criterion in terms of principal stresses is given by (Hoek et al., 2002);
\[ \sigma'_1 = \sigma'_c + k \sigma'_3 \]  

where \( k = \frac{1 + \sin \phi'}{1 - \sin \phi'} \)

Hoek et al., (2002) introduced a concept of a global rock mass strength for estimating the overall rock mass strength. The global rock mass strength can be estimated as follows:

\[ \sigma'_c = \frac{2c' \cos \varphi'}{1 - \sin \varphi'} = \sigma'_{ci} \left( \frac{m_s + 4s - a[m_s - 8s]}{2(1 + a)(2 + a)(4 + s)^{\varphi - 1}} \right) \]

\( \sigma'_c \) is the uniaxial global rock mass strength when \( \sigma_3 = 0 \)

3.9.1 None-linear envelope.

Often there is a preference for the non linear Hoek – Brown failure criterion to be expressed in terms of normal and shear stresses. To convert major and minor principal stresses into normal and shear stresses (Balmar, 1952) the expression for the Mohr circle as shown below is differentiated with respect to \( \sigma_3 \) to find the relationship between normal stress and the principal stresses (Hoek et al., 2002) and can be used to convert a set of principal stresses to normal and shear stresses:

\[ (\sigma - \frac{1}{3}(\sigma'_1 + \sigma'_3))^2 + \tau^2 = \frac{1}{4}(\sigma'_1 - \sigma'_3)^2 \]

\[ \sigma'_i = \frac{\sigma'_1 + \sigma'_3}{2} - \frac{\sigma'_1 - \sigma'_3}{2} \frac{d \sigma'_i}{d \sigma'_3} - 1 \]

\[ \tau = (\sigma'_1 - \sigma'_3) \sqrt{\frac{d \sigma'_i}{d \sigma'_3}} \]

Where for the Hoek – Brown failure criterion;
Note that a curved shear strength envelope implies that the cohesion increases with normal stress as result of greater of confinement of the rock mass.

It is impossible to find a closed form solution for the curved Mohr envelope corresponding to Hoek – Brown failure criterion, therefore the nonlinear relationship proposed by Hoek and Brown (1980) is used:

$$\sigma' = a + m_b \left( m_b \frac{\sigma_3}{\sigma_c} + s \right)^{a-1}$$ \hspace{1cm} (3.53)

Where \( A \) and \( B \) are empirical constants. \( A \) and \( B \) are calculated by linear regression as follows:

$$y = Bx + \log A$$ \hspace{1cm} (3.55)

where \( y = \log \left( \frac{\tau}{\sigma_{ci}} \right) \) \hspace{1cm} (3.56)

and \( x = \log \left( \frac{\sigma - \sigma_i}{\sigma_{ci}} \right) \). \hspace{1cm} (3.57)

And \( A \) and \( B \) are calculated from the following expressions:

$$B = \frac{\sum_{i=1}^{n} y_i x_i - \sum_{i=1}^{n} x_i y_i}{\sum_{i=1}^{n} x_i^2 - \left( \sum_{i=1}^{n} x_i \right)^2}$$ \hspace{1cm} (3.58)

and \( A = e^{\exp \left( \frac{\sum_{i=1}^{n} y_i - B \sum_{i=1}^{n} x_i}{n} \right)} \). \hspace{1cm} (3.59)
Because this Mohr – Coulomb envelope is non–linear, the friction angle will be instantaneous value at a given normal stress. The instantaneous friction angle is found by finding the gradient of the curve \( \frac{d \tau}{d \sigma} = \tan \Phi_i \) by differentiating equation 6.38 and solving for \( \Phi_i \) as follows:

\[
\Phi_i' = \arctan \left( A \beta \left( \frac{\sigma_y - \sigma_t}{\sigma_c} \right)^{b-1} \right)
\]

\[----------------------- (3.60)\]

### 3.10 Preliminary Stability Estimates:

Possible concerns for instability in constructing tunnels and caverns at depth include rock bursting and rock squeezing. Peck (1969) developed a quick method for estimating stability of excavations in clays in terms of a stability number relating the total vertical pressure at depth and the un-drained shear strength of the clay. This method was later adapted for hard rocks by Bhasin (1994). The degree of potential squeezing is given by stability number \( N_t \)

\[
N_t = \frac{2 \sigma_v}{UCS}
\]

Where \( \sigma_v \) is the total vertical stress at depth obtained by summing overburden weights above the point, and UCS is the unconfined compressive strength.

Table (3.9) gives criteria for the degree of squeezing on the basis of \( N_t \). Squeezing is possible for \( N_t > 1 \) and the severity of squeezing increases with increasing value of \( N_t \).

**Table 3.9: criteria for degree of squeezing based on stability number \( N_t \) (Bhasin 1994)**.

<table>
<thead>
<tr>
<th>( N_t )</th>
<th>Degree of squeezing</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1</td>
<td>Non-squeezing</td>
</tr>
<tr>
<td>1–5</td>
<td>Mild to moderate squeezing</td>
</tr>
<tr>
<td>&gt;5</td>
<td>Highly squeezing</td>
</tr>
</tbody>
</table>

Singh et al (1993) have identified a fairly clear demarcation or boundary, based on the \( Q \) values, between tunnels that suffer squeezing in poor conditions or at great depth. Their criterion is given in terms of depth \( H \) where squeezing conditions are expected:

\[
H \geq 350 \ Q^{1/3} \ ------------------------ (3.62)
\]
Where $H$ is the tunnel depth in meter depth larger than the value given above will lead to potential squeezing.

3.11 Conclusions

The following conclusions can be drawn from the literature review: -

- Rock mass classification is based on case histories and hence tends to perpetuate conservative practice.
- Most rock mass classification systems reviewed were oriented towards the prediction of support requirements for tunnels and permanent structures. Also, the support recommendations proposed by the classification systems are general and have to be modified as new conditions are exposed in developing excavations.
- Local knowledge is based on feel and experience, and different engineers may have different judgment, producing non-comparable assessments for the same geotechnical area.
- Rock mass classification is not a rigorous analytical method, as is often assumed by users.
- Rock mass classifications represent only one type of design method, an empirical one, which needs to be used in conjunction with other design methods.
- The reliability of the main classifications systems is questionable under certain conditions (Pells, 2000 and Watson, 2004). The reason for this is that, although the main classification systems consider similar parameters in calculating the final rock mass rating, different systems apply different weighting to similar parameters and some include distinct parameters that influence the final rock mass quality rating.
- The absence of the intact rock strength in the Barton system (except for a low intact rock strength / environment strength ratio).

Intact rock strength

- The intact rock strength is defined in most classification systems, as the strength of rock materials between the discontinuities. Strength used values are often from the laboratory unconfined compressive strength (UCS) tests. The difficulties caused by the definition of intact rock strength and using strength values based on UCS laboratory tests are:
  - The UCS test sample is most often about 10 cm long and if the discontinuity spacing is less than 10 cm the core may include discontinuities.
• Samples tested in the laboratory tend to be of better quality than the average rock because poor rock is often disregarded when drilled cores or samples breaks and can not be tested.

• The intact rock strength measured depends on the samples orientations if the intact rock exhibits anisotropy.

• UCS is not a valid parameter because, in reality most rock will be stressed under circumstances resembling conditions of triaxial tests rather than UCS test conditions.

• Some classification systems use the point load test. The same problems apply when using the UCS test also apply to point load test. The inclusions of discontinuities in the rock will cause PLS value tested parallel to this continuity to be considerably lower than if tested perpendicular to the bedding plane. This effect is stronger for the PLS test than for a UCS test, as the PLS test is basically a splitting test.

• The disadvantage of using Schmidt hammer for estimation of intact rock strength is the influence of discontinuities behind the tested surface. Schmidt hammer values may be influenced by a large and unquantifiable loss of rebound if a discontinuity was present inside the rock behind the tested surface.

• Many authors have commented on RQD measurements (RD Terzaghi 1965). Some major problems with RQD measurements are:

  • The value of 10 cm (4 in) unbroken rock is arbitrary.
  • The value of 10 cm for unbroken pieces of rock core is an abrupt boundary. A rock mass with discontinuity spacing of 9 cm perpendicular to the borehole axes will results in RQD value of 0% while a discontinuity of 11 cm will results in an RQD of 100%.
  • The RQD is biased through orientation with respect to discontinuity orientation.
  • The RQD value is influenced by drilling equipment, drilling operators and core handling.
  • The equipment and specially the core barrels used for geotechnical rock drilling are not standard. ISRM recommends measuring RQD on cores drilled with a double-tube core barrel only.
  • Estimation of the mechanical behavior of closely jointed rock masses is one of the fundamental problems in rock mechanics since the size representative specimens is too large for laboratory testing. Among the empirical strength criteria suggested for intact rocks and rock masses, the Hoek – Brown criterion has become highly popular. Since its introduction in 1980, the criterion has been refined and expanded over the years.
• The original approach used by Hoek and Brown (1980) is more practical. In this approach equation (3.9) is used to generate a series of triaxial tests values simulating full scale field test, and statistical curve fitting process is used to drive an equivalent Mohr envelope.

• The Hoek – Brown failure criterion is only applicable to intact rock or to heavily jointed rock masses which can be considered homogenous and isotropic. In other words the properties of these materials are the same in all directions, Hoek et al (1995).

• In order to use the Hoek – Brown criterion for estimating the strength an deformability of jointed rock masses three properties of rock mass have to be estimated. These are:
  1. The uniaxial compressive strength $\sigma_{ci}$ of the intact rock pieces in the rock mass.
  2. The value of the Hoek –Brown constant $m_i$ for these intact rock pieces, and
  3. The value of the Geological Strength Index GSI for the rock mass.

The following topics will be discussed further in this thesis:

• The rock mass strength estimated from the rock mass classification Q, and RMR-Systems

• The deformability and rock mass strength estimated from Hoek –Brown criterion and the equivalent from Mohr Coulomb criterion.

• The difficulties related to the Hoek –Brown failure criterion parameters for jointed rock masses.

• The effect of GSI on $s$, $\alpha$ and $m_b$ parameters.
Chapter four
Field and Laboratory Test Results

4 Introduction
A series of field and laboratory tests were conducted on the rock mass at Sabaloka. The objective of these tests is to characterize the geo mechanical properties of Sabaloka rock mass. This chapter will give summary of the different tests and results obtained in the field and in the laboratory.

4.1 Field tests
The field visits to collect rock mass information have been made at Sabaloka site. The scanline surveys consisted of the establishment of scanline along which the following rock mass measurements were taken.
- Nature of discontinuity
- Orientation of discontinuity
- Spacing of discontinuity
- Rock Quality Designation
- Openness of discontinuity
- Infilling of discontinuity
- Roughness of discontinuity surface
- Persistence of discontinuity
- Present of ground water (if any)
- Schmidt hammer reading

Three boreholes were drilled and pressure meter tests were carried out to measure the modulus of deformation.

4.1.1 Fracture characteristics:-
Fracture orientation and frequency data were collected along scan lines at area of quarries and tunnel in order to develop a model for fracturing in the study area. Information about fracture sets is essential for rock mass classification (Q and RMR) and is necessary for developing a geo mechanical model of the rock mass. In addition, data on fracture roughness and infilling, and other parameters needed for input to the Q system were
collected along the scan lines. In common with all rock mass, the rocks at the study area exhibit fractures. The fracture at Sabaloka igneous rock rhyolite and basalt are systematic and can be organized into fracture sets. In order to estimate fracture characteristics, fracture data were collected at several locations in the vicinity of the study area. As with the geo mechanical properties obtained from testing of outcrop samples, fracture data collected at or near the surface will tend to show inferior conditions than will be encountered at depth.

In addition to this, High Tech. Mining company had already excavated a tunnel to a depth of 40m below the surface. Samples from different depths were obtained from this excavation. Three boreholes were excavated to obtain core samples in a vesicular basalt formation. Many geo mechanical tests were conducted on these samples. Scan line for quarry (1) is shown in Figure 4.1.

Other fracture data (in addition to orientation) such as fracture intensity as expressed by RQD and the fracture diameters needed for the Q-and RMR rock classifications have also been collected.
Figure 4.1 Scan line location for quarry (1)

4. 1.2 Geotechnical samples and data collection sites:

Fracture data and samples for geotechnical testing were collected from sites listed in Table 4-1 and map in Figure 4.2. These sites represent the rock formation that will be encountered at the study area. In particular the rhyolite, trachy basalt and vesicular basalt all are well exposed in abandoned quarries and road cuts. Data were collected from surface locations primarily quarries, and road cuts. The purpose of geotechnical testing was to estimate the properties of these rocks.

The geomechanical properties of the rock specimens obtained from surface location are expected to be inferior to the properties of the same rock units at depth, as results of near surface weathering and development of micro cracks due to stress relief.

Table 4.1 data collection sites

<table>
<thead>
<tr>
<th>location</th>
<th>Rock type</th>
<th>No. of scan lines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q- 1</td>
<td>Quarry number (1)</td>
<td>Trachy basalt</td>
</tr>
<tr>
<td>Q- 2</td>
<td>Quarry number (2)</td>
<td>Vesicular basalt</td>
</tr>
<tr>
<td>OC -</td>
<td>Outcrop</td>
<td>Trachy basalt</td>
</tr>
<tr>
<td>RC -</td>
<td>Road cut</td>
<td>Rhyolite</td>
</tr>
<tr>
<td>TU- 1</td>
<td>Tunnel number(1)</td>
<td>Rhyolite</td>
</tr>
</tbody>
</table>

Notes: Prefixes indicate the following: Q = quarry, RC = road cut, OC = outcrop, TU = Tunnel
Figure 4.2 map showing the locations of the studied area sites

4.1.3 Discontinuity Spacing

The perpendicular distance between adjacent discontinuities is referring to the discontinuity spacing. Spacing of discontinuity determines the dimensions of blocks in the slope or excavation which in turn influences the scale of potential rock falls. Discontinuity spacing was measured as the difference between successive ‘distance’ measurements where the discontinuities intersect the scanline. Spacing terms are defined in Table 4.2.
Table 4.2 Terms describing discontinuity spacing. Adopted from Burns et al. (2005)

<table>
<thead>
<tr>
<th>Term</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely wide spaced</td>
<td>&gt; 6 m</td>
</tr>
<tr>
<td>Very widely spaced</td>
<td>2 – 6 m</td>
</tr>
<tr>
<td>Widely spaced</td>
<td>2m – 600 mm</td>
</tr>
<tr>
<td>Moderately wide spaced</td>
<td>600 – 200 mm</td>
</tr>
<tr>
<td>Closely spaced</td>
<td>200 – 60 mm</td>
</tr>
<tr>
<td>Very closely spaced</td>
<td>60 – 20 mm</td>
</tr>
<tr>
<td>Extremely closely spaced</td>
<td>&lt; 20 mm.</td>
</tr>
</tbody>
</table>

The spacing distance at the study area was found to be closely spaced in trachy basalt and moderately wide spaced in rhyolite rocks.

**4.1.4 Discontinuity Orientation**

The orientation of discontinuities is the primary geological factor influencing excavation and slope stability (Selby 1993, Wyllie and Mah, 2004). The discontinuity dip and dip direction were measured in the field using a geological compass. The dip of the joints of trachy basalt which measured from the horizontal of the plan of the joint is 45°, 90° and 40°. The dip of the joints of rhyolite is 65° and 30°. Discontinuity dip is defined as the maximum inclination of a discontinuity to the horizontal in degrees. The dip direction of the discontinuity is defined as direction of the horizontal trace of the line of dip measured clockwise from north (Wyllie and Mah 2004).

**4.1.5 Discontinuity Infill**

Infill is the term given to the material which separates adjacent walls of discontinuities. If the discontinuity contains infilling, the shear strength properties are often modified by the thickness and properties of the infill (Wyllie and Mah 2004). In this study, discontinuity surfaces were typically clean, so that measuring shear strength is derived from the rock material.

**4.1.6 Discontinuity Persistence**

Persistence describes the trace length of discontinuity to its termination in solid rock or adjacent other discontinuities, as observed in an exposure. This parameter essentially
defines the size of blocks and the length of potential sliding surface. The description of persistence follows the scheme of Brown (1980) Table 4.3.

Table 4.3 Term describing discontinuity persistence adopted from Brown (1980).

<table>
<thead>
<tr>
<th>Term</th>
<th>Persistence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low persistence</td>
<td>&lt; 1 m</td>
</tr>
<tr>
<td>Low persistence</td>
<td>1 – 3 m</td>
</tr>
<tr>
<td>Medium persistence</td>
<td>3 – 10 m</td>
</tr>
<tr>
<td>High persistence</td>
<td>10 – 20 m</td>
</tr>
<tr>
<td>Very high persistence</td>
<td>&gt; 20 m</td>
</tr>
</tbody>
</table>

4.1.7 Discontinuity Roughness

The roughness of discontinuity is an important component of the shear strength, particularly where the discontinuity is not filled. The profile gauge of the rock layers of the study area was compared against roughness grades (Figure 4.3) and found to be rough and irregular, undulating.

Figure 4.3 Roughness grades after Bradshaw (2004)
4.1.8 RQD

An estimate of RQD is often needed in areas where line mapping or area mapping has been conducted. In these areas it’s not necessary to use core since a better picture of the rock mass can be obtained from scanline data. Priest and Hudson (1976) found that an estimate of RQD could be obtained from discontinuity spacing measurements made on an exposure using the following equation.

\[ RQD = 100 e^{0.1\lambda} (0.1\lambda + 1) \]  \hspace{1cm} (4.1)

For values of \( \lambda \) in the range 6 to 16 per meter, a good approximation to measure RQD values was found to be given by the linear relation.

\[ RQD = -3.68\lambda + 110.4 \]  \hspace{1cm} (4.2)

Where \( \lambda \) = the number of joints per meter

It should be noted, however, that RQD measured from drill core can be an unreliable predictor of discontinuity frequency because:

- It relies on the ability of the logger to discriminate between natural fractures and those caused by blasting or drilling.

- It may be influenced by the strength of rock material being drilled.

- Good core recovery depends on the drilling practice used.

- RQD is not a good measure of the better rock mass conditions. If a rock mass has one uniformly spaced discontinuity set with a spacing of either 0.1 m or 5 m, the RQD will be 100 in both cases, and

- In anisotropic rock mass, the measured RQD will be influenced by drilling orientation.
Table 4.4 RQD Field Scanline Test Results

<table>
<thead>
<tr>
<th>Site</th>
<th>Scanline</th>
<th>RQD</th>
<th>Rock type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q-1</td>
<td>A1</td>
<td>74</td>
<td>Trachy basalt</td>
</tr>
<tr>
<td>Q-2</td>
<td>B1</td>
<td>78</td>
<td>-</td>
</tr>
<tr>
<td>OC</td>
<td>C1</td>
<td>60</td>
<td>-</td>
</tr>
<tr>
<td>RC</td>
<td>D1</td>
<td>92</td>
<td>Rhyolite</td>
</tr>
</tbody>
</table>

4.1.9 Compressive strength based on Schmidt hammer tests:-

A type –L Schmidt hammer was used to measure rebound hardness of rock faces. Measurements were taken on exposed rock fracture and exposed rock faces adjacent to the fractures. The Schmidt hammer was used in the horizontal position, (the rock face were vertical). The Schmidt hammer rebound values for each scan line are presented in Table 4.5. Following standard procedure (Barton and Choubey, 1976) ten readings (in different locations) were taken at each face, to obtain (R). A standard chart Figure 4.4 was used to estimate the uniaxial compressive strength with the results summarized in Table 4.6. The rhyolite was classified as category I based on the hardness of the rebound hammer with an average value of R approximately 52 and trachy basalt rebound hardness was about 37 classified as category II. The vesicular basalt samples were extracted from borehole coring which is classified as category- III
Figure 4.4 Standard chart for estimating UCS from Schmidt hammer (type L) rebound values (after Deere and Miller, 1966)
Table 4.5 Schmidt hammer rebound values.

<table>
<thead>
<tr>
<th>Site</th>
<th>Sample No.</th>
<th>Schmidt hammer rebound values R</th>
<th>Avg. R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quarry (1)</td>
<td>1</td>
<td>40 39 42 43 38 36 32 44 39 35</td>
<td>38.8</td>
</tr>
<tr>
<td>Trachy basalt</td>
<td>2</td>
<td>35 36 44 46 40 31 38 43 41 42</td>
<td>39.6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>37 39 40 41 45 40 38 36 41 45</td>
<td>40.2</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>41 43 36 45 35 42 40 39 38 40</td>
<td>39.9</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>43 45 38 40 41 32 46 40 44 40</td>
<td>40.9</td>
</tr>
<tr>
<td>Tunnel (1)</td>
<td>1</td>
<td>54 49 52 50 51 53 54 52 54 51</td>
<td>53.4</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>2</td>
<td>50 51 49 53 52 52 55 49 50 53</td>
<td>52.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>53 51 53 49 49 51 53 50 51 49</td>
<td>52.2</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>50 48 50 50 47 43 49 51 46 53</td>
<td>50.8</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>53 57 52 49 51 48 46 50 53 45</td>
<td>53.2</td>
</tr>
</tbody>
</table>

Table 4.6 Unconfined compressive strength based on Schmidt hammer

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Rock type</th>
<th>Site</th>
<th>Unit weight (KN/m3)</th>
<th>Schmidt hammer ®</th>
<th>Unconfined compressive strength(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Trachy basalt</td>
<td>Quarry (1)</td>
<td>26.96</td>
<td>38.8</td>
<td>88</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td>39.6</td>
<td>89</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td>40.2</td>
<td>91</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td>39.9</td>
<td>89</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>40.9</td>
<td>91</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td>34.9</td>
<td>70</td>
</tr>
<tr>
<td>1</td>
<td>Rhyolite</td>
<td>Tunnel (1)</td>
<td>25.45</td>
<td>53.4</td>
<td>160</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td>52.2</td>
<td>150</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td>52.2</td>
<td>150</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td>50.8</td>
<td>140</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>53.2</td>
<td>160</td>
</tr>
</tbody>
</table>
4.1.10 Pressure meter Test

The pressure meter test (PMT) provides an in situ measurement of both strength and stiffness. After placing a flexible, cylindrical probe into the ground, and then expands it in the lateral direction as shown in Figure 4.5. The stress field created around the probe has a vertical axis of symmetry, and a probe of sufficient length creates an approximately plane strain condition perpendicular to its axis. This relatively unique combination provides boundary conditions that simplify the analysis of the test data. The lateral pressure applied by the probe, \( p \) (radial stress, \( \sigma_{rr} \) at the cavity wall) plotted versus the relative increase in probe radius, \( \Delta R_0 / R_0 \) (at the cavity wall) provides an in situ stress-strain curve that may be interpret to obtain the lateral in situ stress, stiffness, and strength. From tests performed at various depths, the engineer may develop a depth profile of these parameters for design use. There are four main types of pressure meter probes, these are:

- pre- boring pressure meter
- self- boring pressure meter
- driven cone pressure meter
- pushed shelby tube pressure meter

The pressure meters used in rock generally have thicker, stronger membranes to apply higher pressure than those used in soil. Because of the extra strength, they also have less sensitivity, less expansion capability, and reduced accuracy for testing softer materials. The operator must carefully control the diameter of the borehole to avoid over-expansion and bursting of the membrane.

![Figure 4.5 PMT in borehole](image)
4.1.10.1 Pressure meter Test Data
Pressure meter tests generally include test increments during which either the pressure or the volume must be held constant for a specified period of time, usually (30 to 60 seconds during this research) to provide a measurement of creep. The measured pressure and volume from water filled probe, required correction for:
- volume lost to the expansion of the tubing in the control unit and leading to the probe, compression of the probe membrane, and compressibility of the system fluid due to trapped air.
- pressure added to overcome the inertia of the membrane
- the difference in probe fluid pressure due to the elevation difference between the pressure gage and the saturated probe

Figure 4.6 shows a typical S-shaped plot of the corrected volume and pressure measurements at the same elapsed time for each increment of a pressure meter test, either volume-or pressure-controlled. The initial curvature between points A and B bring the probe into solid contact with the boring sidewall and at point B replaces the in situ horizontal stress \( p_0 = \sigma_h \) removed during preparation of the borehole. The linear portion of the pressure meter curve, from point B to the yield pressure at point C \( p_y = \sigma_y \) represent the linear stiffness of the geomaterial. The final curved portion from point C to D, results from plastic failure, and after applying sufficient pressure, it reaches a maximum resistance at the limit pressure, \( P_L \). The pressure meter curve provides several test results that may be correlated with the properties of the rock mass, or used directly in various design problems. These properties include the deformation modulus of rock mass \( E_m \), yield pressure \( P_y \) the limit pressure \( P_L \).
4.1.10.2 Modulus of deformation from pressure meter in-situ tests

The pressure meter probe expansion is axi symmetrical about the central (vertical) axis and plane strain in any plane perpendicular to the central axis. With these initial assumptions, only the behavior of the geo material significantly affects the radial expansion of the probe. As shown in Figure 4.7 using cylindrical coordinates, the radial stress, $\sigma_r$, and the circumferential stress, $\sigma_\theta$, in the stress field perpendicular to the probe axis are both principal stresses. For analysis purpose at low strain levels, many interpretation methods model the geo material as isotropic and linear elastic. These assumptions seem reasonable over the pressure range starting at the in situ horizontal stress, $\sigma_h$, and ending at the yield pressure, $\sigma_y$, at which point plastic deformation begins at the cavity wall and progresses into the material as indicated on the volume –pressure curve by the departure from linearity.
P = cavity pressure
r_0 = cavity radius
σ_r = radial stress at radius r
σ_θ = circumferential stress at radius r
U_r = radial displacement at radius r

**Figure 4.7 pressure meter stress field after Haberfield, 1987.**

The above assumptions also result in a correlation for the pressure meter modulus of the rock mass $E_m$, calculated from the linear portion of the test curve (B to C) using either the volume or relative change in probe radius.

Where $V_1, V_2 =$ total volume (initial + injected) at the end points

$P_1, P_2 =$ corresponding pressures at the end point

$R_0 =$ the initial probe radius

$\Delta R =$ change in probe radius (from $R_0$)

$\mu =$ Poisson’s ratio, assumed =0.33 for hard rock

The pressure meter field method was used for measuring the modulus of deformation for category III. The pressure meter tests provided an estimate of the in-situ lateral pressure, strength, and deformation properties of the rock.
4.1.10.3 Test Method and Results
The pressure meter probe is placed in the ground by lowering it into a predrilled borehole. Once the probe is placed at the desired depth, the pressure in the probe is increased in equal increments and the associated increase on the probe volume is recorded. The test is terminated if yielding in the rock became large. This procedure is repeated at the desired depth intervals. A pressure - volume curve is plotted and pressure meter modulus is calculated as shown in Figures 4.8 to 4.11.

Figure 4.8 pressure – volume curve for vesicular basalt borehole (1) depth 3m

Figure 4.9 pressure – volume curve for vesicular basalt borehole (1) depth 6
Figure 4.10 pressure–volume curve for vesicular basalt borehole (1) depth 9 m

Figure 4.11 pressure–volume curve for vesicular basalt borehole (2) depth 6 m
The limit pressure as measured with pressure meter, ranges from 1400 kPa to 2200 kPa. Creep pressure range between 1000 to1600 kPa. Modulus of deformation values varied from about 19100 kPa to 50500 kPa. These indicate that the obtained results by the pressure meter in agreement with that determined by the other methods i.e the vesicular basalt is weak rock.

4.2 Laboratory Tests

Laboratory tests are performed on selected samples from blocks and borings to determine the pertinent engineering properties of the rock in the study area. The test results are presented in a clear manner including table graphs and pictures of samples. Laboratory tests performed included.

- Unit weight
- Point load index
- Unconfined compressive strength
- Tensile strength
- Triaxial compressive strength (Hoek cell)

All tests are done in general accordance with ASTM specifications.

4.2.1 -Unit weight:- Unit weight ($\gamma$) was determined for the purpose of estimating vertical stresses at depth, and for calibrating estimates based on the Schmidt hammer tests.

Unit weight measurements were carried out using the test procedure described in ASTM D 5731(2001). Representative samples were chosen to perform the unit weight test. The tests consist of measuring the weight of the specimens and it’s bulk volume ISRM (1981). The test results, with averages for each site, are given in Table 4.7. The trachy basalt unit weights were measured on samples obtained from surface exposures. Because of the prolonged exposure at the surface, they have been subjected to weathering and development of micro cracks, both of which reduce unit weight. Unit weights at depth are expected to be slightly higher.
Table 4.7 Rock unit weights.

<table>
<thead>
<tr>
<th>Site</th>
<th>Sample No.</th>
<th>Rock type</th>
<th>Rock mass (g)</th>
<th>Rock volume(cm$^3$)</th>
<th>Density g/cm$^3$</th>
<th>Unit weight kN/m$^3$</th>
<th>Avg. unit weight kN/m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel (1)</td>
<td>1</td>
<td>Rhyolite</td>
<td>580.5</td>
<td>229.022</td>
<td>2.53</td>
<td>25.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>_</td>
<td>580</td>
<td>229</td>
<td>2.53</td>
<td>25.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>_</td>
<td>560.0</td>
<td>219.861</td>
<td>2.55</td>
<td>25.5</td>
<td></td>
</tr>
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<td></td>
<td>4</td>
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<td>568</td>
<td>224.442</td>
<td>2.53</td>
<td>25.3</td>
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<td></td>
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<td>595.5</td>
<td>229.022</td>
<td>2.6</td>
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<td></td>
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<tr>
<td></td>
<td>6</td>
<td>_</td>
<td>254.94</td>
<td>100.5</td>
<td>2.54</td>
<td>25.5</td>
<td></td>
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<td></td>
<td>7</td>
<td>_</td>
<td>253.64</td>
<td>100.4</td>
<td>2.52</td>
<td>25.2</td>
<td></td>
</tr>
<tr>
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4.2.2 Unconfined compressive strength:- The unconfined compressive strength (UCS) is a basic strength parameter for rock, and an input parameter for both the Q and the RMR rock mass classification system. The (UCS) is also used to quantify the potential for squeezing ground in tunneling. More generally, the (UCS) is used to determine the capacity for rocks to support applied stresses. The compressive strength of Sabaloka rhyolite and basalt was evaluated by three different test methods: (1) unconfined compressive strength tests (2) by correlations with Schmidt hammer rebound value, and (3) by point load tests. Of these methods, the unconfined compressive strength test is...
generally superior. However, sample preparations for the unconfined strength test is time consuming and expensive. Use of the point load test and Schmidt hammer allowed testing of a large number of samples and give good indication of a range of results.

4.2.2.1 Methods of sampling
Block samples of rhyolite and trachy basalt were extracted from Sabaloka igneous complex rock. The cylindrically cored specimens for uniaxial compression tests were prepared from block samples with the length to diameter ratio approximately 2.0. The poor quality core of category- III is shown in Figure 4.12. The photos of rock samples of categories I, II and III were shown in Figures 4.13, 4.14 and 4.15 respectively. The compressive strength tests were carried out on rhyolite, vesicular basalt and trachy basalt and the results are shown in Table 4.8.

Figure 4.12 The poor quality core with very much discontinuities from heavily jointed vesicular basalt
Figure (4.13) 54.7 mm diameter by 100 mm long specimens of rhyolite from Sabaloka igneous complex.

Figure (4.14) 54.7mm diameter by 100 mm long specimens of vesicular basalt from Sabaloka igneous complex.
Figure (4.15) 54.7 mm diameter by 100 mm long specimens of trachy basalt from Sabaloka igneous complex.
Table 4.8  Unconfined Compressive Strength Test Results.

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The stress-strain curves plots of trachy basalt and rhyolite with brittle failure are shown in Figures 4.16 and 4.17 respectively.

Figure 4.16 stress strain plots to failure for uniaxial compressive strength of trachy basalt

Figure 4.17 stress strain plots to failure for uniaxial compressive strength of rhyolite
4.2.3 Point load test results:

The point load test is an index test for strength classification of rock (A description of the testing and calculation procedure can be found in ASTM D5731 (2001), ISRM (1985) and Broch and Franklin (1972). Figure 4.18 shows the point load apparatus used to perform the tests. All the samples tested met the requirements for test specimens (Figure 4.19). Results of the point load test for determination of unconfined compressive strengths are given in Table (4.9). It is important to note that the trachy basalt samples were from quarries and highly weathered outcrop and are not representative of the trachy basalt at depth, which is expected to have significantly higher strength.

Figure 4.18 the point load apparatus used to perform the tests
Figure 4.19 Test methods and critical dimensions for point load test (ISRM, 1985)
Figure 4.9 Data and results of point load test

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The data obtained on unconfined compressive strength from all test methods are presented in Table 4.10 and three sets of strengths are reported one for the trachy basalt which is based on measurements in quarries and outcrop and the other sets based on measurements on samples collected from tunnels and boreholes.

Table 4.10. Summary for unconfined compressive strength results in (MPa) by several test methods

<table>
<thead>
<tr>
<th>Test method</th>
<th>UCS test (MPa)</th>
<th>Schmidt hammer (MPa)</th>
<th>Point load (MPa)</th>
<th>Average (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trachy basalt</td>
<td>96</td>
<td>86.3</td>
<td>81.6</td>
<td>87.9</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>164.5</td>
<td>152</td>
<td>153</td>
<td>156.5</td>
</tr>
<tr>
<td>Vesicular basalt</td>
<td>53.2</td>
<td>_</td>
<td>57</td>
<td>55</td>
</tr>
</tbody>
</table>

The results from these three independent techniques (1) unconfined compressive strength tests (2) by correlations with Schmidt hammer rebound value, and (3) by point load tests, were in good agreement with each other. The slight difference between compressive strength from uniaxial test results, 96 MPa for trachy basalt, and point load irregular lump samples 81.6 MPa, is attributed to the better quality samples used for uniaxial tests whereas the irregular lump samples were taken from a wider range of rock masses therefore including more disturbed materials.

4.2.4 Brazilian tension test:-
Brazilian tensile tests were carried out on trachy basalt and rhyolite intact rock. A total of eight tests for each rock type were conducted and the results are shown in Table 4.11.
Table 4.11 Tensile strength results of trachy basalt and rhyolite intact rock

<table>
<thead>
<tr>
<th>Site</th>
<th>Rock type</th>
<th>Test No.</th>
<th>Tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q-1</td>
<td>Trachy basalt</td>
<td>1</td>
<td>18.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>9.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>22.58</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>15.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>6.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>12.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>3.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>7.34</td>
</tr>
<tr>
<td>Tunnel -1</td>
<td>Rhyolite</td>
<td>1</td>
<td>15.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>17.14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>16.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>12.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>26.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>25.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>13.15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>19.89</td>
</tr>
</tbody>
</table>

4.2.5 Triaxial compression test

Triaxial compressive tests were carried out using a 70 MPa capacity triaxial cell. The triaxial cell is the Hoek-Franklin cell for specimens of 54.74 mm diameter. Five different confining pressures were applied during the triaxial tests. The range of confining pressures of \(0 > \sigma_3 > 0.5 \sigma_{ci}\) proposed by Hoek (1980) were used. The specimens were first subjected to the required confining pressure and then the axial load was applied until the specimen fail. The results of triaxial tests on intact rock of 70 samples of Sabaloka igneous complex can be found in Table 4.12 and Appendix 1:3 to 1:5. The test results in Figure 4.20 and 4.21 indicate a Mohr Coulomb strength of \(c = 4.7\) MPa and \(\Phi = 31^0\) for trachy basalt and \(c = 13\) MPa and \(\Phi = 40^0\) for rhyolite. Summary of the laboratory test results is shown in Table 4.13.
### Table 4.12 Results of triaxial compression test

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>Test No.</th>
<th>$\sigma_{ci}$ (MPa)</th>
<th>$m_i$</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trachy Basalt</td>
<td>1</td>
<td>96.01</td>
<td>11.59</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>49</td>
<td>18</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>124.8</td>
<td>15.46</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>112.98</td>
<td>8.9</td>
<td>0.99</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>1</td>
<td>182</td>
<td>11.3</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>143</td>
<td>8.2</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>167</td>
<td>12.08</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>156</td>
<td>16.13</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>126</td>
<td>19.19</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>151</td>
<td>17.19</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>160</td>
<td>15.5</td>
<td>0.99</td>
</tr>
<tr>
<td>Vesicular basalt</td>
<td>1</td>
<td>48.05</td>
<td>10.05</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>51.5</td>
<td>11.6</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>48</td>
<td>10</td>
<td>0.98</td>
</tr>
</tbody>
</table>

### Table 4.13 Summary of the laboratory results of the three rock types in the study area.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Rhyolite</th>
<th>Trachy basalt</th>
<th>Vesicular basalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kNm$^{-3}$)</td>
<td>25.45</td>
<td>26.96</td>
<td>27</td>
</tr>
<tr>
<td>Uniaxial compressive strength, UCS (MPa)</td>
<td>164</td>
<td>96</td>
<td>53</td>
</tr>
<tr>
<td>Modulus of elasticity E (GPa)</td>
<td>57.4</td>
<td>33.6</td>
<td>7.2</td>
</tr>
<tr>
<td>Poisson ratio ($\mu$)</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>
Figure 4.20  Results of triaxial testing of category II (Trachy basalt)
Figure 4.21 Results of triaxial testing of category I (Rhyolite)

Conclusion:
- **Trachy basalt:**
  A highly fractured trachy basalt rock mass containing 4 joint sets and many random fractures, having average RQD of 69%, and an average joint spacing of 0.06m. Most of the joints were found to have a rough to smooth surface and closed with persistence of few meters. The aperture generally is less than 20 mm. Average intact rock compressive strength 96 MPa. The triaxial test results of trachy basalta indicate Mohr Coulomb strength of $c = 4.7$ MPa and $\Phi = 31$.

- **Rhyolite:**
  Rhyolite rock mass containing 2 joint sets and many hair cracks, average RQD 95%, average joint spacing 0.25m, joint surfaces are generally rough and unweathered. A tunnel is excavated in rhyolite at 40m below the ground level and the excavation surface
is dry. A fracture zone is observed at the entrance of the tunnel, joint surfaces are slickenside and undulating and are highly weathered. Joints are separated by about 0.2 to 0.35 m filled with clay. The triaxial test results of rhyolite indicate Mohr Coulomb strength of $c = 13$ MPa and $\Phi = 40^\circ$. 
Chapter Five
Analysis and Discussion of the Rock Mass Data

5. Analyses of the rock mass data
5.1 Introduction

The data has been mainly analyzed using rock mass classification systems and Hoek-Brown criterion with the advanced software RocLab and Examine2D programs. These programs were used to analyze the rock mass strength of the three categories of Sabaloka igneous complex.

5.2 Analysis of the results of point load index and compressive strength.

A linear regression between the $I_s (50)$ and UCS values determined for 30 tests of trachy basalt and rhyolite are shown in figure (5.1), trachy basalt and rhyolite is consider as hard rock. The zero intercept regression equation obtained from the entire data sets is:
\[
\text{UCS} = 23.77 I_s (50) \quad \text{------------------------ (5.1)}
\]

![Figure (5.1) Regression between uniaxial compressive strength and point load index (trachy basalt and rhyolite)](image)
The correlation coefficient ($R^2$) obtained from equation (5.1) is 0.835 which is very good for rock mechanics, and the equation yield a conversion factor of 23.77 which is within the average for hard rocks.

5.3 Rock mass classification analysis

5.3.1 Introduction

Rock mass classification is the mean of assessing a numeric rating to the quality and performance of rock mass, based on easily measurable parameters. Rock mass classification is not usually considered a sufficient basis for the design of underground excavation, but it can be a starting point, and is the useful device for comparing rock masses. Four rock mass classification systems were used to estimate the rock quality of sabaloka igneous rock (trachy, vesicular basalt and rhyolite). The systems used were the RQD Deere (1967), Q system (Barton, Lien, and Lunde, 1974), and the RMR (Rock Mass Rating) (Bieniawski1974) and the GSI Hoek and Brown, (1997) which are the most widely used rock mass classification systems. The Q and RMR systems are briefly described, and then Q and RMR estimates for the study area are reported. In all cases dry water condition is assumed. The GSI system will be used later in this chapter with Hoek-Brown failure criterion.

5.4 Q system

The Q system was developed by Barton, Lien and Lunde (1974), at the Norwegian Geotechnical Institute (NGI) and was recently updated by Barton and Grimstad (1994). The Q-value gives a description of the stability of rock mass such that high values indicate good stability and low values indicate poor stability. Based on six parameters the Q value calculated using the following formula:-

\[
Q = \frac{RQD}{J_a} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}
\]

Numerical values are assigned to each parameter of the Q system according to detailed descriptions to be found in Barton et al (1974). Table 5.1 assigns qualitative classes to the rock mass according to the overall value of Q.
Values used as an input for the Q rating are presented in Table 5.2. Q values were obtained from mapping done from quarries, outcrops, road cuts and tunnel (1) as shown in Table 5.3.

### Table 5.1 Q system rock classes (after Barton, lien, and Lunde, 1974)

<table>
<thead>
<tr>
<th>Q</th>
<th>Rock mass quality for tunneling</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.01</td>
<td>Exceptionally poor</td>
</tr>
<tr>
<td>0.01 – 0.1</td>
<td>Extremely poor</td>
</tr>
<tr>
<td>0.1 – 1.0</td>
<td>Very poor</td>
</tr>
<tr>
<td>1.0 – 4.0</td>
<td>poor</td>
</tr>
<tr>
<td>4.0 – 10.0</td>
<td>Fair</td>
</tr>
<tr>
<td>10.0 – 40.0</td>
<td>Good</td>
</tr>
<tr>
<td>40.0 – 100.0</td>
<td>Very good</td>
</tr>
<tr>
<td>100 - 400</td>
<td>Extremely good</td>
</tr>
<tr>
<td>&gt; 400</td>
<td>Exceptionally good</td>
</tr>
</tbody>
</table>

### Table 5.2 Field input parameters for determination of Q.

<table>
<thead>
<tr>
<th>site</th>
<th>scanline</th>
<th>RQD</th>
<th>No. of sets (Jn)</th>
<th>Roughness (Jr)</th>
<th>Alteration (Ja)</th>
<th>Rock type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q-1</td>
<td>A1</td>
<td>74</td>
<td>15</td>
<td>3</td>
<td>0.75</td>
<td>Trachy basalt</td>
</tr>
<tr>
<td>Q-2</td>
<td>B1</td>
<td>78</td>
<td>15</td>
<td>3</td>
<td>1</td>
<td>Trachy basalt</td>
</tr>
<tr>
<td>OC</td>
<td>C1</td>
<td>60</td>
<td>15</td>
<td>3</td>
<td>1</td>
<td>Trachy basalt</td>
</tr>
<tr>
<td>RC</td>
<td>D1</td>
<td>92</td>
<td>6</td>
<td>3</td>
<td>0.75</td>
<td>Rhyolite</td>
</tr>
<tr>
<td>TU-1</td>
<td>E1</td>
<td>98</td>
<td>4</td>
<td>3</td>
<td>0.75</td>
<td>Rhyolite</td>
</tr>
<tr>
<td>TU-1 (fracture zone)</td>
<td>F1</td>
<td>35</td>
<td>20</td>
<td>1</td>
<td>4</td>
<td>Rhyolite</td>
</tr>
</tbody>
</table>
Table 5.3 Q values for rock types in the study area

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Q(typical min)</th>
<th>Q(typical max)</th>
<th>Q(mean value)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trachy basalt</td>
<td>12</td>
<td>19.7</td>
<td>15.6</td>
</tr>
<tr>
<td>rhyolite</td>
<td>61.3</td>
<td>98</td>
<td>79.5</td>
</tr>
<tr>
<td>Rhyolite fracture</td>
<td>0.44</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>zone</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The Q system tunnel support chart Figure 5.3 is used to arrive at preliminary estimates of the support requirements for tunnel at the study area. This chart was originally developed by Barton et al. (1974), and was later updated by Grimstad and Barton (1993). Using this system there are nine classes of tunnel support ranging from no support required for rocks with high Q rating, to cast concrete lining for exceptionally poor rocks. In addition to specifying the type of support, the chart provides specifications on dimensions of support elements (e.g., shotcrete thickness, rock bolt spacing, etc.). The support requirements shown depend on the Q-value and effective span or height (whichever is the larger), divided by the effective span ratio (ESR).
Figure 5.3 Tunnel support requirements as function of rock mass quality $Q$, span or height, and ESR, the effective span ratio (Grimstad and Barton, 1993).
5.4.1 Discussion of RQD and Q -Rating:

RQD values obtained from scanlines, which range from 60 to 78% for trachy basalt show very frequent fracturing. The frequent fracturing in the case of trachy basalt made it difficult to prepare a core sample for uniaxial test as shown in Figure 5.2. Values of RQD for rhyolite which is ranging from 90 to 98% show large fracturing space. Values of the parameters averaged for all scanlines at each location are given in Table 5.3. The typical value of $J_n$ of 15 corresponds four joints sets (bedding planes plus three sub-vertical joint sets) in the case of trachy basalt. Rock joints are typically rough, irregular and undulating. Inside the tunnel mainly dry conditions are observed. Very favorable SRF values are indicated by existing excavation showing no major stress problems such as spalling, squeezing and breakouts. In the case of the favorable stress condition SRF is assumed to be 1. An exception is for fracture zone encountered at the entrance of tunnel (1) where SRF = 7.5 is given. As can be seen from Table 5.3, Q values range from minimum of 12 to maximum of 20 with the most common mean value of 16 for trachy basalt and minimum of 61 to maximum of 98 with the most common mean value of 79.5 for rhyolite. The most common mean value of 16 in case of trachy basalt indicates “Good” rock rating. The most mean common value of 79.5 for rhyolite indicates “Very good” rock rating. The minimum of 0.44 corresponds to localized high fracture intensity measured at the entrance of the tunnel. It is reasonable to assume that most rocks away from fracture zone will have “Very good” or better rating.
For Sabaloka igneous complex ESR =1.6 is recommended for access tunnels. The use of ESR is equivalent to applying a factor of safety. For category – I if a tunnel diameter of 10 m is assumed with Q rating of “Very Good” (rhyolite rocks), the access tunnel will require no support. The adequacy of using no support for rhyolite rocks is consistent with experience in the current tunnels in the study area, where no support has been required for large excavation at width of 12 m. In the case of category II (Trachy basalt) the same diameter 10 m is assumed with Q rating of “Good”, the tunnel in this rocks will require systematic bolting.

5.5 Analysis RMR (Rock Mass Rating)

Rock Mass Rating (RMR) (Bieniawski, 1974), increases with rock quality from 0 to 100. The RMR is based on five parameters: unconfined compressive strength of the rock, rock quality designation (RQD), groundwater conditions, joint and fracture spacing, and joint characteristics. A six parameter is an orientation of joints which is used for specific applications in tunneling, mining, and foundations. Sub values for each parameter are summed to determine RMR.
- Unconfined compressive strength (UCS) of the intact rock can be evaluated by means of the point load test, by correlation of Schmidt hammer rebound value, or by unconfined compressive strength tests (values from all three methods were reported).
- RQD is evaluated as described by Deere (1963). RQD is essentially the percent of core run or scanline length in segments longer than 100 mm (4 inches).
- Joint spacing is evaluated from drill core or scanline data. The rock mass rating for joint spacing increases as the space of joints increases.
- Joint and fracture condition is examined with respect to the fracture sets influence the work. In general, the description of joint surface roughness and coating materials are weighted toward the smoothest and weakest joint set.
- Groundwater can strongly influence rock mass behavior. The groundwater rating varies according to the conditions encountered (dry, damp, wet, dripping, or flowing), with the higher rating for a dried rock mass.
- The orientation of joints relative to an excavation face can have an influence on the behavior of the rock. For this reason Bieniawski recommends adjusting the sum of the five rating numbers to account for a favorable or unfavorable orientation. The final RMR value is determined as the sum of the rating from the six classes defined in Table 5.4.

### Table 5.4 Classification of rock masses based on RMR (Goodman, 1989)

<table>
<thead>
<tr>
<th>Class</th>
<th>Description of rock mass</th>
<th>RMR</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Very good rock</td>
<td>81-100</td>
</tr>
<tr>
<td>II</td>
<td>Good rock</td>
<td>61-80</td>
</tr>
<tr>
<td>III</td>
<td>Fair rock</td>
<td>41-60</td>
</tr>
<tr>
<td>IV</td>
<td>Poor rock</td>
<td>21-40</td>
</tr>
<tr>
<td>V</td>
<td>Very poor rock</td>
<td>0-20</td>
</tr>
</tbody>
</table>

Values of RMR obtained from several sites in the vicinity of the study area are shown in Table 5.5. The RMR sub scores are summed to obtain the overall RMR that obtained from the parameter values by means of charts (Bieniawski 1976). For example, RQD in the range 90-100 has score of 20 (Appendix 1:1 Table A1:2). A rating adjustment of -3
(favorable/ fair) for discontinuity orientation is assumed. Scanline data from all scanlines at each site were combined for determining the RMR values for that site. The results based on fractures measured along scanlines at quarries and outcrop give an average RMR of around 60 right at the lower end of the “Good rock” range (Class II). Data from road cut and tunnel (1) give an average RMR of around 82, at the lower end of “Very good rock” range (Class I).

**Table 5.5 Values of RMR for sites in the vicinity of the study area**

<table>
<thead>
<tr>
<th>Site</th>
<th>Strength</th>
<th>RQD</th>
<th>Spacing</th>
<th>Fracture condition</th>
<th>Groundwater</th>
<th>Adjustment</th>
<th>RMR</th>
<th>Rock type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q-1</td>
<td>7</td>
<td>13</td>
<td>10</td>
<td>20</td>
<td>10</td>
<td>-3</td>
<td>57</td>
<td>Trachy basalt</td>
</tr>
<tr>
<td>Q-2</td>
<td>7</td>
<td>17</td>
<td>10</td>
<td>20</td>
<td>10</td>
<td>-3</td>
<td>61</td>
<td>-</td>
</tr>
<tr>
<td>OC</td>
<td>7</td>
<td>13</td>
<td>10</td>
<td>20</td>
<td>10</td>
<td>-3</td>
<td>57</td>
<td>-</td>
</tr>
<tr>
<td>RC</td>
<td>12</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>10</td>
<td>-3</td>
<td>79</td>
<td>Rhyolite</td>
</tr>
<tr>
<td>Tu-1</td>
<td>12</td>
<td>20</td>
<td>20</td>
<td>25</td>
<td>10</td>
<td>-3</td>
<td>84</td>
<td>-</td>
</tr>
</tbody>
</table>

**Correlation with Q:** Barton (1995) developed the following correlation between Q and RMR:

\[
RMR = 15\log Q + 50 \quad \quad \quad \quad \quad \quad \quad \quad (5.3)
\]

Using this correlation for trachy basalt gives a range of Q-values of 12 to 20 for RMR of 66 to 70. From the RMR – Q correlation, in the case of trachybasalt both Q values and RMR indicate a range of “Good” rock quality. For the rhyolite, this correlation gives a range of Q-values of 61 to 98 for RMR of 77 to 80, indicating a rating of (Very Good) rock quality.

**5.6 Analysis of stability numbers for the main rock types**

Table 5.6 gives estimated stability numbers for the three rock types at Sabaloka. The estimated unconfined compressive strengths are based on the results of the Schmidt hammer, point load, and unconfined compressive tests. The vertical stress corresponds to
the maximum depth of the rock unit and assumes dry conditions. Stability numbers increase with depth, and higher stability numbers are detrimental to stability. As shown in Table 5.6, the stability numbers are generally less than 1.0, indicating no potential squeezing problems see Table 3.9 (after Bhasin 1994). An exception is the weaker vesicular basalt formation where the potential for squeezing is expected at depth on the order of 1 km.

Table 5.6 Estimated stability numbers for the three types of rock at Sabaloka.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>UCS (MPa)</th>
<th>Depth (m)</th>
<th>Vertical stress(MPa)</th>
<th>Stability number $N_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rhyolite</td>
<td>164</td>
<td>1500</td>
<td>38.1</td>
<td>0.46</td>
</tr>
<tr>
<td>Trachybasalt</td>
<td>96</td>
<td>1500</td>
<td>40.5</td>
<td>0.84</td>
</tr>
<tr>
<td>Vesicular basalt</td>
<td>53</td>
<td>1500</td>
<td>39</td>
<td>1.47</td>
</tr>
</tbody>
</table>

5.7 Analysis of the results using Hoek –Brown criterion

5.7.1 Introduction

In order to design, and construct the surface and underground excavations safely and economically, it is important to know the rock mass properties thoroughly. The properties of rock mass differ considerably from that of intact rock. In the laboratory intact rock can be tested and the mechanical and strength properties can be known easily, but it is very difficult to evaluate strength properties for rock mass as a whole using the laboratory tests. However, it is possible to estimate strength properties of rock mass from laboratory tests of intact rock. Detailed rock mass properties are important to develop methods and technologies leading to practically useful means for design of surface and underground excavations. The rock mass deformation modulus and strength are used as inputs to analyze the rock mass behavior by numerical models for any surface or underground structures. The determination of the overall mechanical properties of a jointed rock mass is one of the most difficult works in rock mechanics it is generally very difficult task to develop a way that can be used in any practical purpose to predict the strength of the rock mass, since there are so many parameters that affect the deformability and strength of rock mass.
There are several tests for analysis of deformation and strength parameters, in the laboratory and in situ. The in situ tests can only be performed when the exploration adits are excavated and the cost of conducting in situ tests is relatively high. Many attempts have been made to develop methods to estimate the deformability and strength parameters indirectly. The Geological Strength Index (GSI), developed by Hoek et al. (1995), is one of them and used widely. GSI is largely based on experiences from a number of field observations (like block nature and size, rock recovery, etc.) of rock and its jointing natures (like number of joints, joints alterations, fillings in joints, nature of joint surfaces, etc.). Hence, GSI values can be estimated from the geological description of the rock mass. Generally, the GSI system depends on the description of two factors these are the structure and block surface conditions of rock mass. The GSI system is the only system, provides a set of mechanical properties without the effort in the field and laboratory. The properties that can be estimated from GSI include Hoek –Brown strength parameters \( m_b, s \) and \( \alpha \), or the equivalent Mohr-Coulomb strength parameters \( c \) and \( \Phi \) as well as elastic modulus \( E_m \) for design purpose. Although it has been used widely in many countries, its applicability to the rock masses in Sudan has not been tested yet. GSI system can be used in Sabaloka igneous complex rocks to a better understanding of strength behavior of rhyolite and basalt rock mass for its future development of the structure built in or upon these rocks.

The rock mass of the study area has been categorized as category – I (for slightly fractured rock like rhyolite ), category-II fractured rock like trachy basalt), and category-III (for highly fractured rock like vesicular basalt). Rock mass belongs to category-I is of very good to good quality with rough and un-weathered joint surfaces. Rock mass of category-II is characterized by slightly weathered joint surfaces, whereas, rock mass of category-III is weathered with very poor quality.

The strength and deformation related parameters of rock mass of these categories were evaluated from the intact rock strength tests carried out in the laboratory. In the present study, an attempt has been made to estimate the deformation modulus and strength parameters of rock mass of sabaloka igneous complex with the direct information from GSI system. Only for category-III the modulus of deformation was determined by pressure- meter in- situ test.
The information for quantifying the GSI values is obtained from block volume and joint condition factor as well as from site construction and field mapping data. GSI values for the three categories of rock mass were estimated from widely used and well known classification chart developed by Hoek and Brown (1997). Hoek and Brown strength properties for intact rock, equivalent Mohr-Coulomb strength parameters and elastic modulus of the jointed rock mass were calculated using the resulting GSI values. Triaxial, uniaxial and tensile tests on intact rock samples were carried out in the laboratory for all categories.

5.7.2 Estimation of Geological Strength Index-GSI

Several rock mass classification systems has been proposed and used in practice, such as those mention in chapter four part one. A rock mass classification system can be used to estimate mechanical properties at a preliminary design stage. The GSI system seems to be the best choice for design because it can provide a complete suite of input parameters for numerical analysis of surface and underground excavations. In a design process that employs numerical analysis, rock mass deformation modulus and strength are the only required input parameters. The GSI system developed by Hoek et al. (1997) is the chart suggesting definite numerical values and organized with the ‘blocks’ and respective ‘joint condition’ of the jointed rock mass. The value of GSI for the Sabaloka igneous rock mass has been estimated from such chart as shown in Table 5.7.

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### Table 5.7 Chart for Geological Strength Index, GSI (after Hoek & Brown 1997)

<table>
<thead>
<tr>
<th>GSI</th>
<th>Very good, very rough, fresh unweathered surfaces</th>
<th>Good, rough, slightly weathered, iron-stained surfaces</th>
<th>Fair, moderately weathered and altered surfaces</th>
<th>Poor, slickenside, highly weathered surfaces with compact coatings or fillings of angular fragments</th>
<th>Very poor, slickenside, highly weathered surfaces with soft clay coatings or fillings</th>
</tr>
</thead>
<tbody>
<tr>
<td>BLOKY</td>
<td>Very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets.</td>
<td>80</td>
<td>70</td>
<td>65</td>
<td>60</td>
</tr>
<tr>
<td>VERY BLOCKY</td>
<td>Interlocked, partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets.</td>
<td>60</td>
<td>50</td>
<td>45</td>
<td>40</td>
</tr>
<tr>
<td>BLOCKY/DISTURBED</td>
<td>Folded and/or faulted blocks formed by many intersecting discontinuity sets.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DISINTEGRATED</td>
<td>Poorly interlocked heavily broken rock mass with a mixture of angular and rounded rock pieces.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.7.3 Determination of rock mass strength

The strength of a jointed rock mass depends on the strength of the intact rocks and the joint condition. Mohr – Coulomb criterion is expressed in terms of major principal stress ($\sigma_1$) and minor principal stress ($\sigma_3$) as,

$$\sigma_1 = \frac{2c \cos \phi}{1 - \sin \phi} + \frac{1 + \sin \phi}{1 - \sin \phi} \sigma_3$$

Where, $c$ and $\phi$ are the cohesive strength and angle of friction of rock mass respectively.

In rock mechanics, rock mass strength is generally represented by Hoek –Brown strength equation as,

$$\sigma_1 = \sigma_3 + \sigma_{ci} \left[ m_b \frac{\sigma_3}{\sigma_{ci}} + s \right]^\alpha$$

Where $m_b$, $s$, and $\alpha$ are constants for the rock mass and $\sigma_{ci}$ is uniaxial compressive strength of the intact rock. The compressive strength of Sabaloka igneous complex is found from the laboratory tests carried out in this chapter part one. The parameters $m_b$, $s$ and $\alpha$ can be found from the laboratory triaxial tests carried out in this thesis using the following set of equations after Hoek et al. (2002) with the known values of GSI.

$$m_b = m_i \exp \left( \frac{GSI - 100}{28 - 14D} \right)$$

$$s = \exp \left( \frac{GSI - 100}{9 - 3D} \right)$$
Here $D$ is the disturbance factor of rock mass due to blast damage and stress relaxation and can be found from Table 4.15. It varies from zero for undisturbed in-situ rock masses to unity for very disturbed rock masses. Prior to excavation, the only tools available to the designer are boreholes and possibly surface exposures. The model that is developed can therefore only be considered as preliminary. However, the author does not believe that parameter $D$ is appropriate. Different type of rock masses with different stress histories will be affected differently when subjected to blasting and distressing affects. Good quality blasting should not affect a large region of rock mass. If it is considered that this is not the case or that de-stressing will affect the rock mass the GSI can be reduced accordingly. The GSI is considered the appropriate parameter as it is meant to be a representation of the degree of interlocking of the rock mass Hoek et al. (1995). The author has not used the disturbance factor, $D$ in this thesis, instead, the GSI is used. The disturbance factor, is assumed to be zero for the three categories of the rock masses. The value of $m_i$, Hoek constant for intact rock of Sabaloka igneous complex can be found from triaxial tests done by the author (section 4.11.2) using the following equation.

\[
m_i = \frac{1}{\sigma_{ci}} \left[ \frac{\sum xy - (\sum x \sum y/n)}{\sum x^2 - ((\sum x)^2/n)} \right]
\]

(5.9)

The compressive strength of rock mass is obtained by setting $\sigma_3 = 0$ in equation 5.5 to give equation 5.10.

\[
\sigma_c = \sigma_{ci} s^a
\]

(5.10)

The tensile stress of rock mass can be found as,
\[ \sigma_t = -s \sigma_{ci} / m_b \]  

(5.11)

Hoek et al. (2002) suggested in their recent update of strength criterion to obtain the maximum confining stress, \(\sigma_{3\text{max}}\) for tunnels and slopes from the following equations.

\[ \frac{\sigma_{3\text{max}}}{\sigma_{cm}} = 0.47 \left( \frac{\sigma_{cm}}{\gamma H} \right)^{-0.94} \]  

(5.12)

\[ \frac{\sigma_{3\text{max}}}{\sigma_{cm}} = 0.72 \left( \frac{\sigma_{cm}}{\gamma H} \right)^{-0.91} \]  

(5.13)

Where \(\sigma_{cm}\) is rock mass strength, \(\gamma\) is the unit weight of rock mass and \(H\) is the overburden depth of the tunnel.

Normal (\(\sigma_n\)) and shear (\(\tau\)) strength are calculated according to Balmer (1952) as,

\[ \sigma_n = \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2} \frac{d\sigma_1}{d\sigma_3} \left( 1 - \frac{\sigma_1}{d\sigma_3} \right) \]  

(5.14)

\[ \tau = \frac{d\sigma_1}{d\sigma_3} \left( \frac{d\sigma_1}{d\sigma_3} \right)^{-1} \]  

(5.15)

Where

\[ \frac{d\sigma_1}{d\sigma_3} = 1 + am_b \left( m_b \sigma_3 / \sigma_{ci} + s \right)^{a-1} \]  

(5.16)

The rock mass properties relating to the rock strength of the rock mass of Sabaloka igneous complex have been calculated and estimated from above mentioned equations and classification chart. The results are shown in Table 5.8.

5.7.4 The variation of the Hoek – Brown parameters with GSI

Figure 5.4 shows the variation of the Hoek-Brown parameters \(m_b / m, \alpha\) and \(s\) with GSI based on Hoek et al. (2002) and \(D = 0\). It should be noted that if the Hoek (1997) Table 5.7 is used then the minimum and maximum values of GSI are 10 and 80 respectively. \(m_b / m\), which mainly accounts for friction, varies gradually from unity as could be
expected for rock mass. The value of $s$ (which mainly accounts for cohesion) diminishes rapidly with a reduction in GSI thus, indicating a rapid reduction in compressive strength and tensile strength as the quality of the rock mass decreases. The value of $\alpha$ remains relatively constant and has a maximum value of 0.62.

![Figure 5.4 Variation of $\alpha$, $s$ and $m_b/m_i$ with GSI.](image)

Where $\alpha$, $s$ and $m_b/m_i$ are parameter controls the curvature of the Heok-Brown failure envelope, the cohesive component of the Heok-Brown failure criterion and the Heok constant for intact rock and rock mass respectively.
5.7.5 Modulus of deformation from laboratory tests:
The deformation modulus of rock mass is not often easily measured parameter. Several researchers have proposed empirical methods for estimating the rock mass deformation modulus on the base of classification schemes. These methods are based on GSI rock mass classification scheme and the generalized Hoek – Brown failure criterion. According to Hoek et al. (2002), modulus of deformation relating to GSI is given by the following equations,

\[ E_m = \left( 1 - \frac{D}{2} \right) \sqrt{\frac{\sigma_{ci}}{100}} 10^{\frac{(GSI-10)}{45}} \]

for \( \sigma_{ci} \leq 100 \text{ MPa} \)  

\[ E_m = (1 - D/2). 10^{(GSI-10)/40} \]

for \( \sigma_{ci} \geq 100 \text{ MPa} \)

The modulus is calculated in GPa.

5.8 Numerical Analysis
5.8.1 Introduction
One of the major obstacles which were encountered in the field of numerical modeling for rock mechanics, is the problem of data input for rock mass properties. The usefulness of elaborate constitutive models, and powerful numerical analysis programs, is greatly limited, if the analyst does not have reliable input data for rock mass properties.

5.8.2 RocLab Program
RocLab is the software program for determining rock mass strength parameters, based on the latest version of the generalized Hoek-Brown failure criterion. RocLab provides a simple and intuitive implementation of the Hoek-Brown criterion, allowing users to easily obtain reliable estimates of rock mass properties and modulus of deformation. The task of determining rock mass properties is not usually an end in itself. It carried out in order provide input for numerical analysis program, which required material properties in order to perform stability or stress analysis. The rock mass properties determined by roc
Lab can be used as input for numerical analysis program such as Examine 2D. The following tasks can be accomplished with rocLab.

1- Determining Strength Parameters

Determining the generalized Hoek-Brown strength parameters of rock mass \( (m_b, s \text{ and } \alpha) \), based on the following input:
- unconfined compressive strength of intact rock \( \sigma_{ci} \)
- the intact parameter \( m_i \)
- the geological strength GSI.
- the disturbance factor D.

2- Determine Rock Mass Deformation Modulus.

Determine rock mass deformation modulus based on the following input data:
- the intact deformation modulus \( E_i \).

3- Plot Failure Envelopes

Plot the Hoek-Brown failure envelope in principal / or shear normal stress space.

4- Triaxial Lab Test Data

Triaxial lab test data on intact rock should be used to determine \( \sigma_{ci} \), and \( m_i \), using the Marquardt –levenberg fitting technique.
- The triaxial data can be imported from Microsoft Excel program.
- The data can also be entered in the program using a built-in spreadsheet.

5- Equivalent Mohr Coulomb Parameters

Calculation of equivalent Mohr Coulomb strength parameters (cohesion and friction angle)
- The best fit Mohr Coulomb strength envelope is determined over stress range that you can define based on your application (i.e. tunneling or slope stability).
- Plot the equivalent Mohr Coulomb failure envelope in principal and/ or shear normal stress space.

5.8.3 Analysis using RocLab

All the strength parameters and deformation modulus of intact rock and rock mass of category –I, -II and-III of Sabaloka igneous rock was calculated using RocLab see Figure 5.5 to Figure 5.7.
From the analysis of rock mass parameters and deformation modulus of the Hoek – Brown failure criterion, using the RocLab program the present study shows the rock mass of category-I of the Sabaloka igneous complex is more competent in terms of strength than category-II and –III. Hoek –Brown and Mohr-Coulomb strength parameters, uniaxial compressive strength, overall strength and tensile strength of rock mass and failure envelope range of different rock categories of Sabaloka igneous complex are calculated using this program. This values signify that the rock mass belongs to category –I possesses good strength with deformation modulus of about 44115 MPa. Rock mass of category – II is of fair quality and the deformation modulus is 7514.6 MPa, whereas the quality of category – III is very poor with a deformation modulus of 585.96 MPa. The uniaxial compressive strength of rock mass of Sabaloka igneous complex range from34.49 to 0.826 MPa, whereas the tensile strength ranges from −1.37 to -0.028 MPa depending on the rock categories. The cohesion ranges from 13.036 to 1.613 MPa. The shear angle ranges from 40 ° to 24°. These strength parameters and deformation modulus for different categories may help designers for developing surface and underground excavations.
Table 5.9 Strength and deformation parameters of rock mass of Sabaloka.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Category – I</th>
<th>Category – II</th>
<th>Category – III</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \sigma_{ci} ) (MPa)</td>
<td>164</td>
<td>96</td>
<td>48</td>
</tr>
<tr>
<td>GSI</td>
<td>72</td>
<td>45</td>
<td>30</td>
</tr>
<tr>
<td>( m_i )</td>
<td>14.5</td>
<td>12.5</td>
<td>8.8</td>
</tr>
<tr>
<td>D</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Output</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hoek-Brown criterion</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( m_b )</td>
<td>5.334</td>
<td>1.753</td>
<td>0.722</td>
</tr>
<tr>
<td>( s )</td>
<td>0.044</td>
<td>0.0022</td>
<td>0.0004</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>0.501</td>
<td>0.508</td>
<td>0.522</td>
</tr>
<tr>
<td>Failure envelope range for tunnels</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \sigma_{3 \text{ max}} ) (MPa)</td>
<td>0.62</td>
<td>0.594</td>
<td>0.536</td>
</tr>
<tr>
<td>Mohr –Coulomb fit</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C (MPa)</td>
<td>13.03</td>
<td>4.737</td>
<td>1.6</td>
</tr>
<tr>
<td>( \Phi (\theta) )</td>
<td>40.2</td>
<td>31</td>
<td>23.6</td>
</tr>
<tr>
<td>Rock mass parameters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \sigma_{t} ) (MPa)</td>
<td>-1.37</td>
<td>-0.121</td>
<td>-0.028</td>
</tr>
<tr>
<td>( \sigma_{c} ) (MPa)</td>
<td>34.49</td>
<td>4.303</td>
<td>0.826</td>
</tr>
<tr>
<td>( \sigma_{cm} ) (MPa)</td>
<td>56.163</td>
<td>16.72</td>
<td>4.927</td>
</tr>
<tr>
<td>Modulus of deformation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( E_m ) (MPa)</td>
<td>44114.93</td>
<td>7514.64</td>
<td>585.96</td>
</tr>
</tbody>
</table>

( Tunnel depth is 40 m and average unit weight for rock mass is 0.026 MN/m³)
Figure 5.5 minor vs. major principle stress and normal vs. shear stress plot for rock mass of category - I
Figure 5.6 minor vs. major principle stress and normal vs. shear stress plot for rock mass of category - II
Figure 5.7  minor vs. major principle stress and normal vs. shear stress plot for rock mass of category - III
5.8.4 Examine 2D Finite Element program

5.8.5 Introduction

Examine 2D is designed to be a simple to use parametric analysis tool for investigating the influence of geometry and in situ stress variability on the stress changes in rock due to excavations. The induced stress in the plane of the analysis can be viewed by means of stress contours patterns in the regions surrounding the excavations. As a tool for interpreting the amount of deviatoric over stress (principal stress differences) a round openings, strength factor contours gives a quantitative measured of (strength)/(induced stress) according to the defined failure criterion for the rock mass. The elastic boundary element analysis in Examine 2D dictates that the material being modeled is assumed to be:

- homogeneous
- isotropic
- linearly elastic

Obviously, most of the rock masses which will be modeled possess none of these properties. In summary Examine 2D has the following analysis and modeling capabilities:

- Single material with isotropic properties.
- Elastic stress / displacement analysis.
- Strength factor (over stress based on elastic results) using Mohr Coulomb or generalized Hoek Brown failure criterion.
- Constant or gravitational in situ stress.
- Interactive modeling and parametric analysis.
- Uniform pressure loading

5.8.6 Analysis using Examine 2D

In order to confirm the results of the empirical analysis, numerical modeling code two dimensional element model Examine 2 finite program was used to predict the rock masses behavior based on field data investigations, and laboratory tests. Examine 2D program computed the yield zone in rock mass surrounding the tunnel and induced displacements around the excavation. The results of the finite element analysis of the tunnel, the maximum total displacements at walls, roof and floor of the tunnel and the
extent of the yield zones for different rock types in Sabaloka area are shown in Figure 5.8, 5.9 and 5.10. Displacements are very small in the case of the tunnel runs in rhyolite rocks and the total displacement induced by the tunnel excavation in the surrounding rock mass resulted less than 4 cm at rhyolite model. While in the case of basaltic rocks the displacement induced is much larger. The application of the program Examine 2D to compare the Strength Factor contours and deformed excavation profiles in the three types of tunnels running in different rock excavations was determined and illustrated in Figure 5.11 to 5.13. This is indicates that the rock mass strength is greater than the induced stress, and the excavation is safety in the present conditions in the case of category І and ІІ. The numerical results from the finite element analysis by Examine 2D software are shown through the maximum and minimum value of displacement and strength factor in Table 5.10.

**Table 5.10 Numerical results from Examine 2 program**

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Parameters</th>
<th>Minimum value</th>
<th>Maximum value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rhyolite</td>
<td>Displacement (mm)</td>
<td>6</td>
<td>42</td>
</tr>
<tr>
<td>Trachy basalt</td>
<td>-</td>
<td>25</td>
<td>153</td>
</tr>
<tr>
<td>Vesicular basalt</td>
<td>-</td>
<td>165</td>
<td>990</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>Strength Factor</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Trachy basalt</td>
<td>-</td>
<td>0.2</td>
<td>2.6</td>
</tr>
<tr>
<td>Vesicular basalt</td>
<td>-</td>
<td>0.2</td>
<td>1.4</td>
</tr>
</tbody>
</table>
Figure 5.8 Total displacement model of tunnel runs through rhyolite rocks

Figure 5.9 Total displacement model of tunnel runs through trachy basalt rocks
Figure 5.10 Total displacement model of tunnel runs through vesicular basalt rocks

Figure 5.11 Strength Factor model of tunnel runs through rhyolite rocks
Figure 5.12 Strength Factor model of tunnel runs through trachy basalt rocks

Figure 5.13 Strength Factor model of tunnel runs through vesicular basalt rocks
5.8.7 Discussion of rock mass classifications and Numerical Analysis Results.

The range of RMR-Q correlation for trachy basalt and rhyolite is the same as that obtained from the field mapping. Both Q and RMR rock mass classification systems assign the description (Good) to the rock mass quality of trachy basalt and (Very Good) rock mass quality for rhyolite. For the RMR system, “Good” is the second lowest of the five categories. Overall the range of Q and RMR are mutually consistent, both indicating “Good” rock quality for trachy basalt and “Very Good” rock quality for rhyolite. Numerous tunnels and underground excavations have been successfully built in rock mass classified as “Good”. Hoek (2000) gives several examples as the Mingtan pumped storage project. This can be found in Hoek, E. (2000), http://www.rocscience.com/hoek/PracticalRockEngineering.asp

The three categories of rock mass of Sabaloka igneous complex are studied for estimation of strength and deformation modulus. All the strength parameters and deformation modulus of intact rock and rock mass of categories –І, -ІІ and- III of Sabaloka are calculated using RocLab program and the results are given in Table 5.8.

The rock mass belongs to category –І shows uniaxial compressive strength ($\sigma_c$) of 34.49 MPa with tensile strength of -1.37 MPa and modulus of deformation of 44115 MPa see Figure 5.5. The value of ($\sigma_c$) for the rock mass of category –ІІ is 4.3 MPa and the tensile strength is -0.121 MPa with the modulus of deformation of 7514.7 MPa see Figure 5.6.

Category –ІІІ shows uniaxial compressive strength ($\sigma_c$) of 0.826 MPa and the tensile strength is – 0.028 MPa with the modulus of deformation of 585.96 MPa see Figure 5.7.

This implies that the rock mass of category –І is more competent than categories –ІІ and –ІІІ in terms of strength. Category –ІІ Figure 5.6 shows a moderate strength properties and category–ІІІ shows a very poor strength properties. The input values of intact uniaxial compressive strength ($\sigma_{ci}$) were 164 MPa, 96 MPa and 45 MPa for category –І, -ІІ and –ІІІ respectively and those were laboratory tests results. The input parameters of $m_i$ and $\sigma_{ci}$ were obtained from spreadsheets of triaxial tests carried out in the laboratory (see appendix 1: 3 to 1: 5). The GSI values were selected from a chart complied by Hoek and Brown (1997) from a large number of case histories. In the present thesis, for example, the value of $m_i$ was found to be 8.8 for vesicular basalt and 12.5 for trachy basalt but in the classification chart of Hoek and Marinos (2000) it was...
assigned as 17 for basaltic rocks, and the value of m, for rhyolite is found to be 14.5, but in the actual classification chart it was assigned as 16. The modulus of deformation of category – III evaluated from pressure meter tests was found to be different from that evaluated from Hoek – Brown criterion but both of them classified category –III as soft rock. It’s stated by several authors, field modulus of deformation measurement gives values which varies from laboratory values by significant amounts due to jointing in rock masses. For instances Farmer and Kemeny (1992) found that the deformation modulus on intact rock samples is in the order 5 to 20 times higher than in situ values .The variation in block-ness or degree of jointing on rock masses may often cause a major part of this large variation. The vesicular basalt in the study area has heavily fractured nature. The different strength parameters and deformation modulus found in this study can be used for a better as well as future development of surface and underground constructions at the studied area.

5.8.8 Conclusions
The analysis of rock mass classifications shows that:-

A- Sabaloka rhyolite rocks are characterized by high strength rocks and the overall rock mass quality assessment of very good.

B- Sabaloka trachy basalt rocks are characterized by medium strength rocks and the overall rock mass quality assessment of good.

C- Sabaloka vesicular basalt rocks are characterized by low strength rocks and the overall rock mass quality assessment of poor.

D- The relation between uniaxial compressive strength UCS, and the point load strength, PLS, determined for rhyolite and trachy basalt are UCS =23.7 PLS which are similar to the value of 24 recommended by many researchers for hard rocks.

E- The stability numbers are generally less than 1.0, indicating no potential for squeezing problems in all type of tested rocks.

From the analysis of the rock mass parameters and deformation modulus of the hoek brown criterion, using RocLab program it could be concluded that:-

A- The rock mass of rhyolite of Sabaloka igneous complex possess very good strength with deformation modulus of 44115 Mpa, uniaxial compressive
strength of 164 Mpa, Geological Strength Index 72, Hoek constant $m_i$ of 14.5, internal angle $\Phi$ of $40^0$, cohesion $c = 13$ Mpa, tensile strength of 1.37 Mpa, and rock mass strength of 56 Mpa.

B- The rock mass of trachy basalt of Sabaloka igneous complex is of good quality with deformation modulus of 7515 Mpa, uniaxial compressive strength of 96 Mpa, Geological Strength Index 75, Hoek constant $m_i$ of 12.5, internal angle $\Phi$ of $30^0$, cohesion $c = 4.74$ Mpa, tensile strength of 0.121 Mpa, and rock mass strength of 17 Mpa.

C- The rock mass of category III of Sabaloka igneous complex is of poor quality with deformation modulus of 586 Mpa, uniaxial compressive strength of 48 Mpa, Geological Strength Index 30, Hoek constant $m_i$ of $24^0$, cohesion $c = 1.6$ Mpa, tensile strength of 0.028 Mpa, and rock mass strength of 5 Mpa.

From the analysis of rock mass parameters and deformation modulus of the Hoek-Brown criterion, using Examine 2D program it could be concluded that:

A- The maximum displacement around the tunnel excavated in the three rock categories 42 mm in the case of rhyolite, 153 mm in the case of trachy basalt, and 990 mm in the case of Vesicular basalt, considering only the weight of the vertical column of rock resting on an element at a depth of 40 m below the ground surface.

B- The Strength Factors resulting from the three rock types are 1 to 3 in the case of rhyolite, 0.2 to 2.6 in the case of trachy basalt and 0.2 to 1.4 in the case of vesicular basalt.
Chapter Six
Conclusions and Recommendations

6.1 Conclusions
This chapter summarizes the conclusions reached in the various chapters of this study and the conclusions derived are presented below.

6.1.1 Conclusions from literature review
The following conclusions can be drawn from the literature review:

- Rock mass classification is based on case histories and hence tends to perpetuate conservative practice.
- Most rock mass classification systems reviewed were oriented towards the prediction of support requirements for tunnels and permanent structures. Also, the support recommendations proposed by the classification systems are general and have to be modified as new conditions are exposed in developing excavations.
- Rock mass classification is not a rigorous analytical method, as is often assumed by some users.
- Rock mass classifications represent only one type of design method, an empirical one, which needs to be used in conjunctions with other design methods, like failure criteria.
- The reliability of the main classifications systems (RMR and Q) is questionable under certain conditions (Pells, 2000 and Watson, 2004). The reason for this is that, although the main classification systems consider similar parameters in calculating the final rock mass rating, different systems apply different weighting to similar parameters and some include distinct parameters that influence the final rock mass quality rating.
- Q system is based on six parameters and the disadvantage of this system is the absence of discontinuity spacing and intact rock strength, these two parameters are not taken into consideration.

RQD measurement
Many authors have commented on RQD measurements (RD Terzaghi 1965). Some major problems with RQD measurements are:

- The value of 10 cm for unbroken pieces of rock core is an abrupt boundary. A rock mass with discontinuity spacing of 9 cm perpendicular to the borehole axes will result in RQD value of 0% while a discontinuity of 11 cm will result in an RQD of 100% because RQD is calculated for the core length more than 10 cm.
b- The RQD is biased through orientation with respect to discontinuity orientation.
c- The RQD value is influenced by drilling equipment, drilling operators and core handling
d- The equipment and specially the core barrels used for geotechnical rock drilling are not standard. ISRM recommends measuring RQD on cores drilled with a double-tube core barrel only.

**Intact rock strength**
- The intact rock strength is defined in most classification systems, as the strength of rock materials between the discontinuities. Strength used values are often from the laboratory unconfined compressive strength (UCS) tests. The difficulties caused by the definition of intact rock strength and using strength values based on UCS laboratory tests are:
  a- The uniaxial compressive strength (UCS) tests conducted on samples of about 10 cm long and if the discontinuity spacing is less than 10 cm the core may include discontinuities and this will be rock mass rather than intact rock strength.
  b- Samples tested in the laboratory tend to be of better quality than the average rock because poor rock is often disregarded when drilled cores or samples breaks and can not be tested. The intact rock strength measured depends on the samples orientations if the intact rock exhibits anisotropy.
  c- UCS is not a valid parameter because, in reality most rock will be stressed under circumstances resembling conditions of triaxial tests rather than UCS test conditions.
  d- Some classification systems use the point load test. The same problems applying to use the UCS test also apply to point load test. The inclusions of discontinuities in the rock will cause PLS value tested parallel to this continuity to be considerably lower than if tested perpendicularly. This effect is stronger for the PLS test than for a UCS test, as the PLS test is basically a splitting test.

**Schmidt hammer test**:
- The advantage of using Schmidt hammer for estimation of intact rock strength is that it is portable, cost-effective instrument, capable of estimating intact rock strength with distinct advantage over traditional laboratory testing Schmidt (1951). Laboratory tests are time consuming, expensive. Conversely, a large number of nondestructive Schmidt hammer tests can be performed quickly and efficiently in either the laboratory or the field. The disadvantage of this test is the influence of discontinuities behind the tested surface. Schmidt hammer values may be influenced by a large and unquantifiable loss of rebound if a discontinuity is present inside the rock behind the tested surface.

**Hoek-Brown failure criterion**
Estimation of the mechanical behavior of closely jointed rock masses is one of the fundamental problems in rock mechanics since the size representative specimens is too large for laboratory testing. Among the empirical strength criteria suggested for intact rocks and rock masses, the Hoek – Brown criterion has become highly popular. Since its introduction in 1980, the criterion has been refined and expanded over the years.

The original approach used by Hoek and Brown (1980) is practical. In this approach a generalized Hoek –Brown criterion for jointed rock masses was used to generate a series of triaxial tests values simulating full scale field test, and statistical curve fitting process is used to drive an equivalent Mohr envelope.

The Hoek – Brown failure criterion is only applicable to intact rock or to the heavily jointed rock masses which can be considered homogenous and isotropic.

In order to use the Hoek – Brown criterion for estimating the strength and deformability of jointed rock masses, three properties of rock mass have to be estimated. These are :

4. The uniaxial compressive strength $\sigma_{ci}$ of the intact rock pieces in the rock mass.
5. The value of the Hoek –Brown constant $m_i$ for these intact rock pieces , and
6. The value of the Geological Strength Index GSI for the rock mass.

**6.1.2 Conclusions from Field and Laboratory Test results.**

The following conclusions can be drawn from the field and laboratory test results.

- Rhyolite rock mass containing 2 joint sets and many hair cracks, having average RQD of 95%, and an average joint spacing of about 0.25m, joint surfaces are generally rough and unweathered. A tunnel is excavated in rhyolite at 40m below the ground level and the excavation surface is dry. A fracture zone is observed at the entrance of the tunnel, joint surfaces are slickenside and undulating and are highly weathered. Joints are separated by about 0.2 to 0.35 m filled with clay. The average unit weight is 25.5 KN/ m³. The average unconfined compressive strength based on Schmidt hammer is 152 MPa, point load is 153MPa and uniaxial compressive strength is 164 MPa. The stress strain curve of rhyolite rocks indicated brittle failure. The average tensile strength obtained from Brazilian tension tests is 19 MPa. The triaxial test results of rhyolite indicate Mohr Coulomb strength of $c = 13$ MPa and $\Phi = 40^0$. 
• The trachy basalt rocks show intensive jointing system and severe alteration as well as the fragmentation of the rock mass into the sugar cube-like block. Four joint sets are observed, three sub-vertical and one horizontal beside random joints, having average RQD of 69% and an average joint spacing of 0.06 m. Most of the joints were found to have a rough to smooth surface and closed with persistence of few meters. The aperture generally is less than 20 mm. Average intact rock compressive strength 96 MPa. The average unit weight is 27 KN/m. The average unconfined compressive strength based on Schmidt hammer is 86 MPa, point load is 82 MPa and uniaxial compressive strength is 96 MPa. The stress-strain curve of trachy basalt rocks indicated brittle failure. The average tensile strength obtained from Brazilian tension tests is 11 MPa. The triaxial test results of trachy basalt indicate Mohr-Coulomb strength of c = 4.7 MPa and $\phi = 31^\circ$.

• The vesicular basalt rocks are heavily jointed with white spots filled with quartz. The pressure meter test conducted on this rock in the field gives limit pressure, ranges from 1400 kPa to 2200 kPa creep pressure range from 1000 to 1600 kPa. Modulus of deformation values varied from about 19100 kPa to 50500 kPa. These results indicate that, the vesicular basalt is a soft rock.

6.1.3 Conclusions from the analyses of the rock mass data
• The relationships between the uniaxial compressive strength, UCS, and the point load strength, PLS, determined for rhyolite and trachy basalt are UCS = 23.7 PLS which are similar to the value of 24 recommended by many researchers for hard rocks.

• Both Q and RMR rock mass classification systems assign the description (Good) for the rock mass quality of trachy basalt and (Very Good) rock mass quality for rhyolite.

• The evaluation of rock mass quality predicted from both RMR and Q systems for each category are in good agreement with each others.

• Rock mass description of the tested rock mass categories I, II and III of Sabaloka leads to a GSI in the order of 72, 45 and 30 for each one respectively.
Based on the Q values calculated here and on the computational procedure revised by Grimstad and Barton (1993) support type were determined for the 10 m diameter tunnel, the obtained Q value is (16) ; the trachy basalt falls into the (Strong rock) group. The tunnel in this rock will require systematic bolting.

Based on the Q values calculated here and on the computational procedure revised by Grimstad and Barton (1993) support type were determined for the 10 m diameter tunnel, the obtained Q value is (79) for rhyolite ; the rhyolite falls into the (Very Strong rock) group. No support system recommended for the rock mass in this group.

The stability numbers are generally less than 1.0 indicating no potential squeezing problems. An exception is the weaker vesicular basalt where the potential for squeezing is expected at depth on the order of 1 km.

Fitting the Hoek – Brown failure criterion to the triaxial test data by using RocLab program gives constant $\alpha$ and $s$ with values of 0.5 and 0.0022 respectively which are inside normal values in Hoek-Brown (1997), see Table 3.8.

Laboratory testing of intact material to determine Hoek-Brown constant $m_i$, shows that the most notable results is the typical of 14.5 for category I, and 12.5 for category II which is slightly lower than the values suggested in Hoek and Brown (1997).

The basaltic rock is strong to medium strong with uniaxial compressive strength from 50 to 100 MPa and modulus of elasticity from 33 GPa to 7.2 GPa for intact rock. The strength and moduli values vary according to the different lithologies. The trachy basalt is stronger than the vesicular basalt which is containing more joints.

From the analysis of rock mass parameters and deformation modulus of Hoek – Brown failure criterion, using RocLab program it could be concluded that:-

- The rock mass of rhyolite of Sabaloka igneous complex possesses very good strength with deformation modulus of 44115 MPa, uniaxial compressive strength of 164 MPa, Geological Strength Index 72, Hoek constant $m_i$ of 14.5, internal angle of friction $\Phi$ of 40°, cohesion $c = 13$ MPa, tensile strength of 1.37 MPa, and rock mass strength of 56 MPa.
B - The rock mass of trachy basalt of Sabaloka igneous complex is of good quality with deformation modulus of 7515 MPa, uniaxial compressive strength of 96 MPa, Geological Strength Index 45, Hoek constant $m_i$ of 12.5, internal angle of friction $\Phi$ of $30^\circ$, cohesion $c = 4.74$ MPa, tensile strength of 0.121 MPa, and rock mass strength of 17 MPa.

C - The rock mass of category III of Sabaloka igneous complex is of poor quality with deformation modulus of 586 MPa, uniaxial compressive strength of 48 MPa, Geological Strength Index 30, Hoek constant $m_i$ of 8.8, internal angle of friction $\Phi$ of $24^\circ$, cohesion $c = 1.6$ MPa, tensile strength of 0.028 MPa, and rock mass strength of 5 MPa.

- One of the most important finding of the present study is the marked reduction in strength parameters for basaltic rock mass relative to the intact material under the same conditions. For, example tensile strength of the rock mass is reduced to nearly zero, uniaxial compressive strength is reduced by an order of 15 % and modulus of deformation by a factor of 25 % relative to intact basalt.

- Prediction of maximum displacement around the tunnel excavated in the three rock categories were made using a modeling program examine 2D and the displacement in the case of category I is less than 4 cm, 15 cm in the case of trachy basalt and 99 cm in the case of vesicular basalt.

- The rock mass strength based on Hoek –Brown criterion was obtained also in stress analysis with Examine 2D program. We examined the Strength Factor SF in each category when applied a gravitational in situ field stress using Examine 2D program and in category I very good results were obtained. The SF represents the ratio of rock mass strength to induced stress at a given point. In this case the SF obtained is greater than 1 this indicates that the underground excavation is safe. The Strength Factors in category III is less than 1 indicates that the underground excavation in these types of rocks will require supports.

- Fitting the Hoek – Brown failure criterion to the triaxial test data by using RocLab program gives constant $\alpha$ and $s$ with values of 0.5 and 0.0022 respectively which are inside normal values in Hoek-Brown (1997), see Table 3.8.

- The variation of strength and deformability of Sabaloka rock mass categories due to the presence of variation of joint space was shown to be significant in uniaxial
conditions. This can have a major effect in the working strength values, especially in shallow foundations and surface excavations.

6.2 Recommendations

Clearly there is need for more work to be conducted with Sabaloka igneous rock complex to further refine and study rocks mass strength and deformability of the other rocks. The following issues are suggested.

1- More work is needed to examine the strength of disturbed rock masses like vesicular basalt.

2- Further advanced numerical modeling program should be used to assess rock data obtained from triaxial tests.

3- More work is needed on the structural geology of the study area to simplify the reading of GSI.

4- To reduce the disturbance factor D, controlled blasting techniques should be used and that will minimize instability and improve aesthetics of the exposed excavation. The principal behind controlled blasting is that closely spaced holes drilled on the final face are loaded with a relatively light charge to produce a reasonably uniform distribution of the charge on the face.
References


