AN ASSESSMENT OF THE CLASSIFICATION OF LIMESTONE ROCK SLOPES IN SURFACE MINING: A CASE STUDY IN SOUTH AFRICA

by

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SUMMARY

The prime necessity for excavation in the mining industry is to extract precious commodities buried below overburden material. Stable slopes during excavation occur when the resisting force is greater than shear stress. Engineers and scientists in the mining industry consider several factors that are important for slope stability during excavation meanwhile considering the cost of mining operations. There are different methods that have been developed to assess slope stability. The most common methods used in the evaluation of rock mass slope stability are kinematic analysis, limit equilibrium and rock mass classification. However, the rock mass classification was mainly established for hard (durable) rock and may not be applicable to soft rock. Hard rock is considered resistant to deterioration over time whereas, soft rock, such as carbonaceous rocks, weathers (decays) owing to changes over time in both rock moisture and temperature. As a result, slopes comprising soft rock (carbonaceous material) are likely to experience instability over time. To ensure safe and cost-effective mining operations, it is crucial to evaluate which rock mass categorisation systems are adequate for sloe stability assessments.

ABSTRACT

Slopes in surface mines are usually designed from the geotechnical data. The important geotechnical parameters include geology, rock strength and groundwater. In designing the most optimum slopes, each geotechnical parameter is evaluated individually before restructuring them in their appropriate spatial association with each other. To examine each parameter's input to the rate, magnitude and direction of slope movement during the commodity extraction process, it is possible to simulate the results of the individual parameters. In surface mining, the frequency and magnitude of slope failure analysis is critical if it occurs in the area of an entree ramp or above an area of higher ore grade. These areas of potential failure require regular monitoring and suitable remedial measures to be implemented without delay. Mining of limestone in South Africa is important in the cement industry, however, surface mining of carbonate rock may result in slope failures. Limestone is carbonaceous, which reacts when exposed to changes in water and temperature. Slope stability methods have primarily been designed for surface mining using a hard rock datum. These slope stability methods have ignored the environmental factors which are important aspects for the steadiness of carbonaceous rock. The drive of this study is to use the designed rock slope stability methods on soft rock to assess which methods can be used to ensure safe and economic mining operations. Using rock mass classification schemes is helpful in determining the slope steadiness in surface mining but limitations exist. The systems require additional parameters that look at support structures for slopes taking into consideration slope material that is susceptible to solubility when exposed to environmental agents. In utilising rock mass categorisation, one is able to find rock mass properties such has cohesion and internal frictional angel in a safe working condition with limited laboratory work. Rock mass properties help determine the slope condition with regards to stability and safety.

Key words

Rock slope, slope stability, carbonaceous rock, rock mass classification, mining, South Africa

Zulu

Imithambeka ezimayini zomhlaba yakhiwe ngokwe mininingwane ye-geotechnical. Imingcele ebalulekile ye-geotechnical ifaka i-geology (isayensi ebhekene nokwakheka komhlaba, umlando wawo, nezinqubo ezisebenza kuwo), namandla edwala namanzi angaphansi komhlaba. Ekwakheni imithambeka engcono kakhulu, ipharamitha ngayinye ye-geotechnical ihlolwa ngokuhlukana ngaphambi kokuyakha kabusha ebudlelwaneni bayo bendawo efanele. Imiphumela yepharamitha ngayinye ingenzelwa ukuthola ukusebenza kwayo okumayelana nobukhulu, isilinganiso kanye nokuqondiswa kokufuduka kwemithambeka ngesikhathi sokukhishwa kwempahla. Ezimayini zomhlaba, isilinganiso nobukhulu bokuhlaziywa kokuhluleka kwemithambeka kubalulekile uma kwenzeka eduze kommango wokungena noma ngaphezulu kwendawo esebangeni eliphakeme. Lezi zindawo ezibuye zibe nokuhluleka zidinga ukuqashwa njalo kanti futhi izinyathelo ezifanele zokulungisa zingathathwa ngaphandle kokubambezeleka. Ukumbiwa kwamatshe eNingizimu Afrika kubalulekile embonini kasimende, kodwa-ke, ukumbiwa komhlaba okungaphezulu kwedwala le-carbonate kungaholela ekuhlulekeni kwemithambeka. I-limestone ibhekwa njenge mpahla e-carbonaceous, eshintshayo uma ivezwa kuzinguquko ezisemanzini nasezingeni lokushisa. Izindlela zokuqina kwemithambeka zenzelwe kakhulu ukumba phansi kusetshenziswa ucezu lolwazi ngedwala eliqinile. Lezi zindlela zokuzinza kwemithambeka azizinakanga izici zemvelo okuyizinto ezibucayi ekusimameni kwedwala lekhaboni. Inhloso yalolu cwaningo ukusebenzisa izindlela eziklanyelwe ukuqina kwemithambeka yedwala elithambile ukuze kuhlolwe ukuthi yiziphi izindlela ezingasetshenziswa ukuqinisekisa ukusebenza kwezimayini eziphephile nezomnotho.

Xhosa

Amathambeka kumphezulu wemigodi ayilwe ngokubanzi ngokwe datha yofundo ngezobugcisa. Ukubaluleka kweparemeters zemfundo ngezobugcisa kuquka ukwakheka komhlaba, amandla eliwa kunye namanzi aphantsi komhlaba. Ekuyilweni kwelona lilungileyo ithambeka iparemeter yofundo ngezobugcisa nganye iyavavanywa ngokwahlukana phambi kokuba zakhiwe kabutsha umphandle wazo ofanelekileyo ngokusondelelana kwazo, enye kwenye. Iziphumo zeparemeter nganye zinokuboniswa ukuze kumiselwe ukusebenza ngokujoliswe kubukhulu,ixabiso nesikhokelo sokufuduswa kwethambeka ngexesha lokumbiwa kwezorhwebo emigodini. Kulwambiwo lwemigodi rhoqo nobukhulu bethambeka uhlalutyo lokuwa lubalulekile xa kunokuthi kwenzeke kwindawo yokungena okanye ngaphezulu komgangatho wexabiso ophezulu. Ezi ndawo zamathuba amakhulu okuwa kwethambeka kudinga ingqwalasela kunye namanyathelo osasazo afanelekileyo, anokuthi asetyenziswe ngaphandle kolibaziseko. Ukumbiwa kwelitye lekalika eMzansi Afrika kubalulekile kwicandelo lesamente nangona kunjalo ukumbiwa kwelitye lekhabhonathi kungabangela ukuwa kwethambeka. Ilitye lekalika lithathwa njengezinye zezinto ze-corbonaceous, zivakalelwa xa zithe zadibana nenguqulelo yamanzi namaqondo obushushu. Iindlela zokuzinziswa kwethambeka ziyilwe kwasekuqaleni kulwambiwo lwemigodi kusetyenziswa idwala elinzima kwaye eliluqilima. Ezi ndlela zokuzinzisa ithambeka azizange ziyijonge imeko yendalo esingqongileyo nekuzizo izinto ezibalulekilyo kuzinzo lweliwa leCarbonaceous. Injongo yolufundon kuku sebenzisa iindlela eziyiliweyo zoqilimo kwidwala lethambeka. Kumadwala athambileyo ukujonga ukuba zeziphi iindlela ezinokusetyenziswa ukuqinisekisa ukuphepha kuqoqosho nemisebenzi yezemigodi

DECLARATION

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An assessment of the classification of limestone rock slopes in surface mining: A case study in South Africa

I certify that the aforementioned dissertation or thesis is entirely my own work and that all the references that I have utilised or cited have been indicated and properly recognised by means of comprehensive references.

SIGNATURE

April 2024

DATE

ABBREVIATIONS

C	Cohesion
DWAF	Department of water affairs and forestry
GSI	Geological Strength Index
GSI_m	Modified Geological Strength Index
IRMR	In-situ Rock Mass Rating
RMR	Rock Mass Rating
RQD	Rock quality designation
SMR	Slope Mass Rating
S	Represents the constants that depend on the rock-mass characteristics
UCS	Uniaxial Compressive Strength
Φ	Angle of internal friction
σ'1	Maximum effective stress at failure
σ'3	Minimum effective stress at failure
σci	Uniaxial compressive strength of the intact rock pieces

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Firstly, I would like to acknowledge the gift of life from the living God I serve. The strength and ability to conduct and finish this work is through his mercy and grace. It would not have been possible had it not for him.

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CHAPTER 1: INTRODUCTION

1.1. BACKGROUND

The design of a rock slope in surface mining (open-pit) for the extraction of any precious commodity requires stabilising the ground. Stable slopes play an important part in open-pit mining operations as rock slope failure may hinder movement on roads along benches and, at times, present a (safety) geohazard to the miners which then will result in the loss of production (Kolapo *et al*., 2022). Analysis of naturally stable slopes is mainly done using the principles of rock mechanics. In rock mechanics when the slope's resisting forces have a lower value than the strength of the rock mass then the end result will be stable (Wyllie & Mah, 2004). Natural physical processes control rock slope failure and deformation but these can be accelerated by anthropogenic activities. In the mining industry, stable slopes are analysed using rock engineering principles. In rock engineering, the design of stable slopes is guided by information derived from the collection of geotechnical data (Wyllie & Mah, 2004).

Geotechnical factors that are considered to have an influence on rock slope stability during mining, specifically excavation include geology, rock strength, groundwater, effects of blasting, mining method and equipment used (Kolapo *et al*., 2022). The factors that trigger slope failure in slopes are categorised as internal and external factors. Factors such as geology, rock strength and groundwater are considered as internal factors. Effects from blasting, mining method and equipment used are considered as external factors and are caused by human decision and activities (Kolapo *et al*., 2022).

Within geology, the most significant feature to consider for slope stability is structural geology. Data on structural geology is collected from mapping any discontinuities. A discontinuity is a break or surface that symbolises a variation in physical or chemical features in a soil or rock mass. A discontinuity could be in the form of a joint, bedding plane, cleavage schistosity, crack, foliation, fracture, fissure or fault plane (Kliche, 1999). Discontinuities like joints and cracks in a rock mass (geological structures) are planes of weakness with no visible displacement but may directly or indirectly influence the steadiness of the rock mass (Gumede & Stacey, 2007). Discontinuities may arise numerous times with approximately the same mechanical features in a discontinuity set or may be a single discontinuity. The occurrence of discontinuities like cracks and joints in a rock mass (geological structures) has negative effect as the real strength of the in-situ rock mass being reduced than the strength of intact rock (Sjoberg, 1997). The impact of discontinuities on the shear strength of a rock mass is dependent upon the orientation, roughness, thickness of infillings or gouge material and moisture content of the rock.

Rock strength factors that are applied in determination of slope stability are predominantly the shear strength of discontinuities, shear strength of the rock mass, weathering characteristics of the rock, and, to a minor level, the compressive strength of complete rock (Wyllie & Mah, 2004). In the shear strength of discontinuities, the rock is considered to be a Coulomb material when the friction angle (φ) and the cohesion (c) are used to is demonstrate the shear strength of the slithering surface (Coulomb, 1773). Data collection for cohesion and frictional angle is acquired through triaxial testing (Kolapo *et al*., 2022). Triaxial data can be collected through a laboratory index test and/or through an empirical method. The shear strength of the rock mass can also be defined by back-analysis of slope failures or determined by means of an empirical method established by Hoek & Brown (1980) where the shear strength is expressed as a curved Mohr envelope. The compressive strength of rock can be appraised on core samples, or from using index tests onto outcrops in the field. The capability of rock to deteriorate (weather or break down) can also be calculated in the laboratory or measured by field index tests (Wyllie & Mah, 2004).

The occurrence of groundwater in an open-pit is mostly from precipitation or recharge from adjacent rivers, tailing dams, reservoirs, etc. The detrimental effect of groundwater on slope stability is the water pressure within discontinuities, moisture content within the rock mass, surface water erosion and excavation cost if working below the water Table (Wyllie & Mah, 2004). The greatest influence of groundwater on a rock material is the reduction in steadiness due to water pressure within the discontinuities (Beale & Read, 2013; Wyllie & Mah, 2004). Permeability (hydraulic conductivity) is used to decide on the movement of groundwater and the supply of water pressure. The flow of groundwater through intact rock is known as primary permeability and when groundwater flows through discontinuities it is recognised as secondary permeability.

The primary purpose of excavation in the mining industry is to access a precious commodity that is covered by overburden material. Slopes are developed as transportation routes to access the orebody and results in numerous high rock cuts in the open-pit to facilitate mining production. Slopes are planned using the collected geotechnical information. The geotechnical information is applied to appraise the rock mass properties. The rock mass properties are used to determine the best blasting practice and the mining method. Inadequate blasting in mining negatively impacts the stability of rock slopes, primarily because they can fracture and loosen the rock behind the slope face, in addition to causing vibrations from the blasting process (Kolapo *et al*., 2022). During blasting operations, natural cracks and fractures in a rock mass structure are extended by additional stresses induced by the blasting, and therefore the shear strength of the rock mass is significantly reduced, thereby causing instability of the rock mass.

In mining operations, the surrounding rock mass around excavation develops deformation due to changes in the *in situ* stress field conditions. During excavation, natural slopes may face deformation as a result of the reduction in shear strength, which can lead to slope failure. The rock mass movement may continue if no solutions are implemented on the cut slope. The overall slope stability must be considered when selecting mining methods and equipment usage (Kolapo *et al*., 2022).

The most common systems used in the determination of slope stability are (1) kinematic analysis, (2) the limit equilibrium method and (3) rock mass classification. Kinematic analysis is used to analyse the probability of a rock slope failure owing to the presence of discontinuities. The limit equilibrium method is used to evaluate the sensitivity of a possible slope failure condition based on the slope geometry and rock mass parameters. The rock mass classification system compares the practical experience encountered at a previous site with the current site to give an estimation of slope stability. In the mining industry, rock mass classification was initially used in underground mining for tunnel support in hard rock (Gundewer, 2014). Abbas & Konietzky (2017) note that recently, numerous classification systems have been established for underground and surface stability evaluation (e.g. Rock Mass Rating, Q-tunnelling and Geological Strength Index). These classification systems are commonly used today as a checklist of items that should be considered in the process of assessing the characteristics of the rock masses which are to be mined and in which slopes are to be constructed (Hustrulid *et al*., 2000). The advantage of using rock classification is the ability to correlate the relationship from one classification to another. Somodi *et al*. (2021) worked on the relationship between RMR_{89} and GSI where one is able correlate data from each classification system .Celada *et al*. (2014) worked on the RMR_{89} to RMR_{14} which can be correlated using an equation.

Mining of limestone in South Africa is critical for the cement industry. The demand for minerals in the 21st century requires mines to expand operations which result in bigger and deeper excavation sites (Kolapo *et al*., 2022). However, carbonaceous material is known to be associated with karst processes within the Transvaal Supergroup in South Africa. Limestone is a non-clastic carbonate rock (Strahler & Strahler, 1973) composed principally of calcium carbonate (calcite). A carbonate rock mass is considered by fundamentally anisotropic properties (physico-mechanical, hydraulic, dynamic, thermal) (Andriani & Parise, 2017). Limestone is usually considered as a physically strong rock; however, it can be totally removed by solution during weathering (Waltham, 2001). According to Waltham (2001), precipitated water and earth water reacts with the limestone on the ground and can disintegrate (dissolve) the rock, particularly where it percolates down cracks and bedding planes thereby making extensive fractures and caves. When limestone is in contact with water, the calcium carbonate reacts with carbon dioxide and water (equation 2.1) to form calcium bicarbonate which is highly water-soluble (Lurie, 2008). The formation of calcium bicarbonate quickens during cooler temperatures because colder water holds more dissolved carbon dioxide gas (Strahler & Strahler, 1973). See the equation below.

$$CaCO_3(s) + CO_2(g) + H_2O(l) \rightarrow Ca(HCO_3)_2(aq) \qquad \text{EQ 1.1}$$

When limestone is exposed to heat it releases calcium dioxide gas and leaves behind lime (equation 2.2). Lime has a crumblier texture than the original limestone and it reacts with water to produce slaked lime (calcium hydroxide).

$$CaCO_3(s) \rightarrow CaO(s) + CO_2(g) \qquad \text{EQ 1.2}$$

1.2. Problem Statement

Valuation of the steadiness of slopes in open-pit mines at diverse phases of excavation is significant for harmless and cost-effective mining production. Despite the analysis of slope stability using conventional techniques, mines are still experiencing slope failures, which have proven to be catastrophic and expensive (Kolapo *et al.,* 2022). Methods for the assessment of slope stability are mainly based on the collected geotechnical data (Read & Stacey, 2009). However, assessment of slope stability in surface mines for carbonaceous rock has received relatively little attention. Carbonate rock is a soft rock and surface mining of this rock may result in decreased slope stability and accelerated slope failure.

A dolomitic limestone open-pit mine located approximately halfway between Brits and Thabazimbi in the province of Limpopo found in South Africa has been identified for this study. Geologically, this area has undergone refolding, faulting and is intruded by the Bushveld complex. Due to folding, faulting and igneous intrusions in the past, the mining area is heavily jointed. As much as limestone is a hard rock and does not easily react with water, the study area is highly fractured which may allow water to percolate within the fracture openings. Occurrence of water within discontinuities may eventually dissolve away the rock and leave cavities, which will grow over time. These processes will weaken the whole steadiness of the rock and may decrease slope stability.

1.3. Motivation

Valuation of the steadiness of slopes in open-pit mines at diverse phases of excavation is significant for harmless and cost-effective mining production. Slope failures are common in open-pit mines below is a list of existing failed slopes around the globe Table 1.1. Stable slopes are essential for safe mining operations as failure may hinder movement along the benches and, at times, create geohazards to the miners which then can have a negative outcome such as decrease in production or worse the loss of life. Consideration of the behaviour of carbonaceous rock when exposed to the environment will assist in modifying the current rock mass classification in surface mining.

1.4. Study Area

The area of study for this research will be conducted from a limestone open-pit mine located approximately halfway between Thabazimbi and Brits in the Waterberg District of the Limpopo Province (Figure 1.1). Part of the open-pit mine is on farm Buffelskraal 545 KQ (Portion 1), farm Nooitgedacht 136 JQ (portion 1) and farm Krokodilkraal 545 KQ (Portions 3, 4, 5 and 6). These farms are found in the Thabazimbi Local Municipality which form part of the Waterberg District in the province of Limpopo (Durand, 2013).

Table 1.1: Global list of failed slopes (Kolapo *et al*., 2022).

Country	Location	Names of Mining	Mode of Failure	Causes of Failure
Botswana	Central District	Letlhakane Mine	Toppling	Presence of tension crack formation, crack widening and extension
Canada	British Columbia	Afton Mine	Wedge, toppling and circular failure	Multiple failures occurred as a result of intersection of discontinuities
		Brenda Mine	Toppling	Intersection of joint sets
		Cassiar Mine	Toppling	Presence of shear zones, faults and sets of discontinuities
		Highland Valley, Copper	Toppling	Steeply dipping joints, increase in groundwater pressure and melting of snow
		Lonex Pit at Highland Valley	Toppling	Groundwater condition, steeply dipping faults
		Highmont	Planar	Structural discontinuities, precipitation, run off, poor quality and low strength rock
		Nickel Plate Mine	Wedge	Steeply dipping joint sets and faults
	Vancouver	Island Copper	Wedge and toppling	Large fault zone passing through a weaker rock mass
	Quebec	Jeffrey Mine, Asbestos	Wedge and planar	Intersection of several thick shear zones and smaller scale discontinuities
China	Mongolia	Changshanhao open-pit	Wedge and toppling	Presence of faults and joints
	Shazhenxi	Qianjiangping	Planar	Increase in water level, poor geological structure and continuous rainfall
Norway	Hange I Dalane	Tellness Dagbrudd	Wedge	Heavy rainfall
Mexico	Calama, Antofagasta	Chiquicamata	Toppling	Presence of fault zones

Spain	Seville	Aznacollar Mine	Complex	Presence of tension cracks, heavy rainfall, groundwater pressure
Sweden	Kiruna	Kirunavaara	Complex	Presence of tension cracks
United States of America	Utah	Bingham Canyon Mine	Rotational and planar	Rise in water table, fractured rock mass with minor joints and larger
	Nevada	Carlin Trend	None	Presence of wider fault zones and clay
		Liberty Pit	Wedge	Intersection of joint sets
		Veteran—Tripp Pit	Wedge	Intersection of faults, presence of clay gouge in fault zones
		Kimbley pit	Wedge	Presence of flat sipping fault, high water pressure
	Arizona	Cyprus Bagdad and Sierrita	Toppling	Presence of steeply joint sets
		Twin Butes	Toppling	Numerous faults and several joints
South Africa	Limpopo	Palabora Mine	Wedge	Presence of faults and set of joints
	Mokopane	Sandsloot Open Pit	Planar and wedge	Presence of set of joints
Zambia	Chingola	Nchanga Open-pit	Wedge	Intersection of joint sets, abnormally rainfall, weathering

FIGURE 1.1: LOCATION MAP OF THE CONTINENTAL RESOURCES OPEN-PIT MINE (GOOGLE EARTH PRO SATELLITE IMAGE, 2018).

1.4.1. GEOLOGY

The geology at the mine comprises of Dolomitic Limestone of the Oaktree Formation from the Transvaal Supergroup (Durand, 2013). The Transvaal Supergroup consist of numerous clastic, volcanic and chemical formations. The Supergroup has been exposed to several distortion events which consist of the disturbance of the approximately 2060 Ma Bushveld Complex (Walraven and Martini, 1995). This Supergroup contains one of the world's initial carbonate platform successions (Beukes, 1987; Altermann & Wotherspoon, 1995). The stratigraphy of the Transvaal Sequence in the Western fragment of the Supergroup (Table 1.2) consists of the Wachteenbeetje Formation as the base formation. The sediments in the Wachteenbeetje Formation were due to marine transgressions which caused bottom aquatic carbonaceous clay nuggets to overlay siltstones and dolomites dumped in an aquatic shelf setting.

The Wachteenbeetje Formation is overlain by the Black Reef Formation. The sediments in the Black Reef Formation were deposited when the transgressional cycle was interrupted by uplift of the basin and subsequent erosion. On top of the Black Reef Formation is a chemical and clastic sediments of the Malmani Subgroup. The Malmani Subgroup consists of five formations, each formation differs on the amount of chert and the appearance and nonappearance, as well as the diversity of stromatolite structures (SACS, 1980). The five Formations consist of the (1) Oaktree Formation, (2) Monte Christo Formation, (3) Littleton Formation, (4) Eccles Formation and the (5) Frisco Formation.

The sediments in the Oaktree Formation were deposited in the intermediate precinct from the subordinate Black Reef siliciclastic sedimentation. The Oaktree Formation entails dark-grey dolomite that has no chert presence. It is also associated with big stromatolitic arches. According to Roux (1984), the dolomite in the Oaktree Formation consists of a higher percentage (>1%) of manganese and has resulted in the growth occurrence of wad and manganocrete. The wad at the bottom of the Oaktree Formation is unexpectedly rigid with an elastic modulus of 20 MPa and samples display no fragmentation in water (Day, 1991).

On top of the Malmani Subgroup is the Penge Formation. The Malmani Subgroup together with the Penge Formation make up the Chuniespoort Group. On top of the Chuniespoort Group is the Timeball Hill and Rooihoogte Formations of the Pretoria Group. The mine is situated within the Crocodile River fragment area. It is an inlier of intensely deformed rocks of the Transvaal Sequence surrounded by acidic and basic rocks of the Bushveld Complex (Walraven & Martini, 1995). On the west side of the Transvaal Basin is the Crocodile River fragment and the Rooiberg inlier. The Marble Hall, Dennilton and Stavoren inliers are also found in the western side of the Transvaal Basin (Hartzer, 1995) (see Figure 1.2).

TABLE 1.2: SEDIMENTATION PROCESSES OF THE TRANSVAAL SEQUENCE IN THE CROCODILE RIVER FRAGMENT (ADAPTED FROM HARTZER, 1989).

Supergroup	Group	Sub-group	Formation	Sedimentary structures and features	Depositional environment
Transvaal	Pretoria		Silverton	Laminated claystone and siltstone	Deep basin
			Daspoort	Ripple and low-angle, cross-bedded quartz, arenite	Beach
			Strubenkop	Laminated claystone and siltstone	Shelf
			Hekpoort	Lava	Volcanism
			Timeball Hill	Laminated claystone and siltstone Ripple and low-angle, cross-bedded quartz, arenite	Deep basin Delta
			Rooihoogte	Conglomerate, quartzite, shale	Delta
	Chuniespoort		Penge	Banded Ironstone	Deep basin
		Malmani	Frisco	Clastic laminated, carbonate, megadomes, shale, chert	Subtidal
			Eccles	Fenestral dolomite, chert domal and columnar	Intertidal
			Lyttelton	Megadomal dolomite and limestone, shale	Subtidal
			Monte Christo	Fenestral dolomite, chert domal and columnar	Intertidal
			Oaktree	Megadomal dolomite and limestone, shale	Subtidal
			Black Reef	Conglomerate, argillite, ripple and low-angle, cross-bedded quartz, arenite	Beach Fluvial
			Wachteenbeetje	Claystone, laminated, Siltstone/claystone, Dolomite, low-angle cross-bedded quartz, arenite	Deep basin Shallow marine

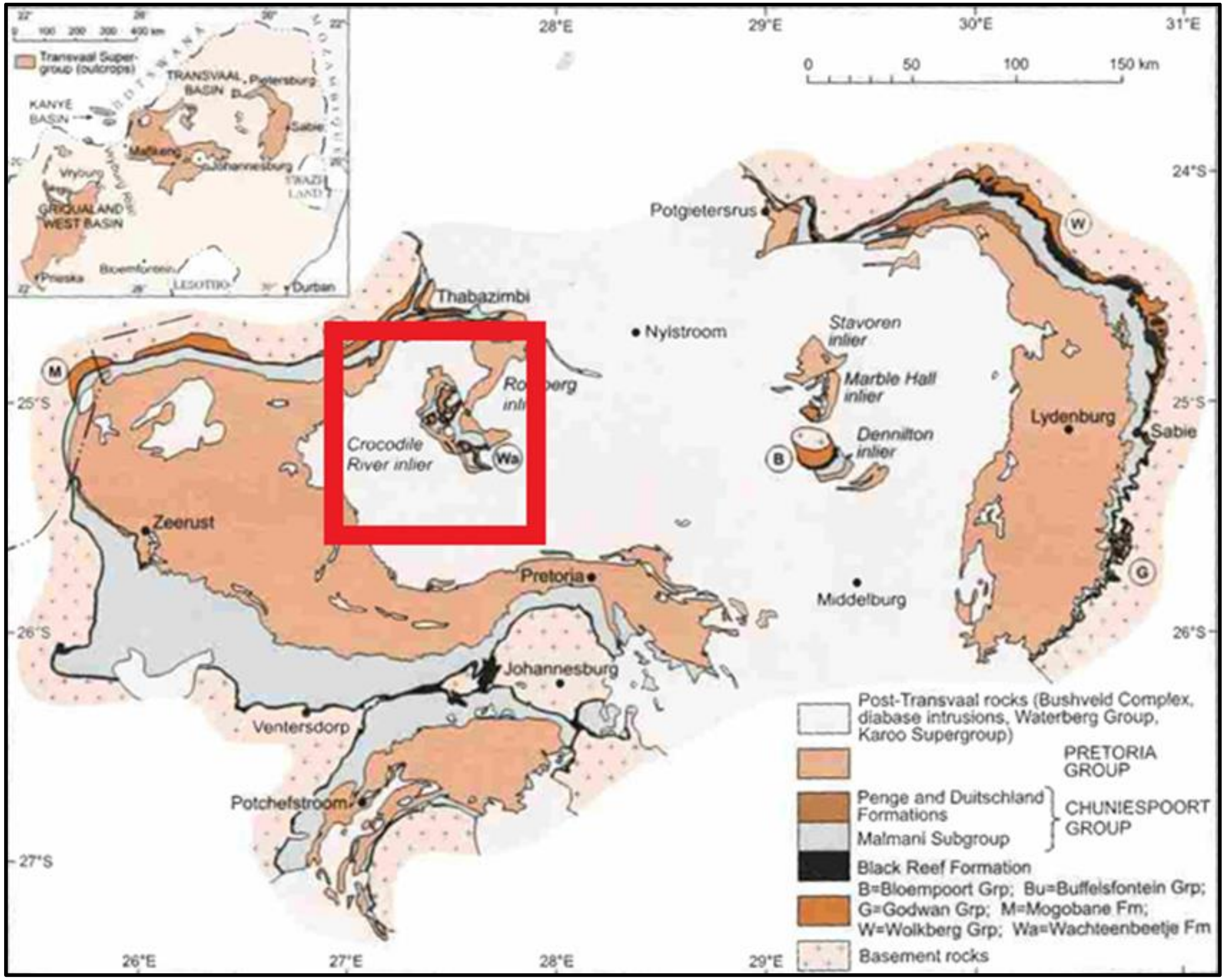


FIGURE 1.2: THE TRANSVAAL BASIN WITH DEFORMED INLIERS (HARTZER, 1995).

1.4.2. Mining Environment

The area in which the mine is situated is flat to slightly undulating and is covered by Bushveld vegetation. The Bushveld vegetation is underlain by 0.3 to 0.5 m thick gravel and soil overburden. Beneath the overburden material is the 2500 m strike length and 360 m thick dolomitic limestone (Judeel, 2017). The surface area slopes gently towards the east in the bearing of the Crocodile River (Durand, 2013). The elevation of the area fluctuates between 940 m.a.s.l. near the Crocodile River and 1000 m.a.s.l. to the west where hilly terrain occurs. According to seismic data of the Council for Geosciences, the study site (open-pit mine) is not set in a seismically-active or hot-spot region (Judeel, 2017).

The Dolomitic limestone is mined through surface mining making use of benches and slope faces at the pit. Access to the pit is mainly through the Northwest (Figure 1.3 c). The Northern, Western and Southern side of the pit has clear slopes that one can study the geotechnical condition. A 12 to 14 m wide haul road is constructed at a gradient of 9% to access the bottom level of the quarry. Currently the mine consists of two bench levels which can be seen on Figure 1.3 b. The current slope faces are having heights in the order of 8 to 10 m and slope angles in the order of 80°. The East side of the pit has mostly soil slopes.

The mining phase of the open-pit operation consists of cleaning/clearing of the surface, drilling, blasting, loading, hauling, crushing, screening and processing of the ore. Front-end loaders and excavators are used for some cleaning and loading purposes. Drilling and blasting are carried out by a specialized drill and blasting contractor. Large rocks are fragmented by using a mechanized rock breaker. Mined material is transported to the crushing and screening plant on site. The mine’s processing area is situated on the premises close to the offices (Figure 1.4).

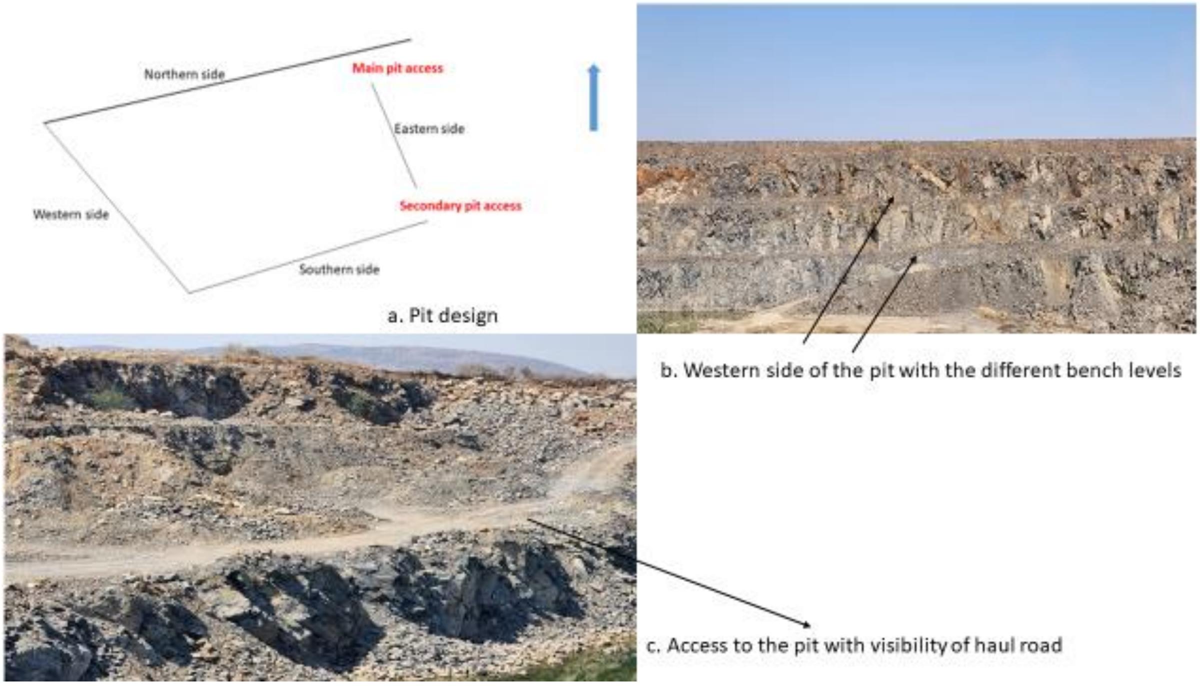


FIGURE 1.3: SURFACE MINING AT THE PIT (STUDY SITE).

FIGURE 1.4: MINE PLAN (DURAND, 2013).

1.4.3. Regional Hydrology

Groundwater is usually the key source of water distribution in the region although surface water is also used conjunctively where it is available. The quarry is part of the Crocodile River catchment area. According to Boonzaaier (2008), the hydrology of the Crocodile River fragment forms part of quaternary catchments A23K, A24A and A24B in the Lower Crocodile River Sub-area (Figure 1.5). In the study area, the dolomite forms fairly plane ground with extensive farming and irrigation. Borehole production in the area can be greater than 10 l/s, particularly from the main alluvial water-bearing rock. Groundwater heights in the alluvial aquifer are between 5 and 10 m beneath ground level. The water level of the dolomitic aquifer is up to 20 m beneath ground level. The groundwater condition is good, but the aquifers in the area are susceptible to contamination. Aquifers in the dolomite are constrained to structural features and areas of bigger weathering and karst development.

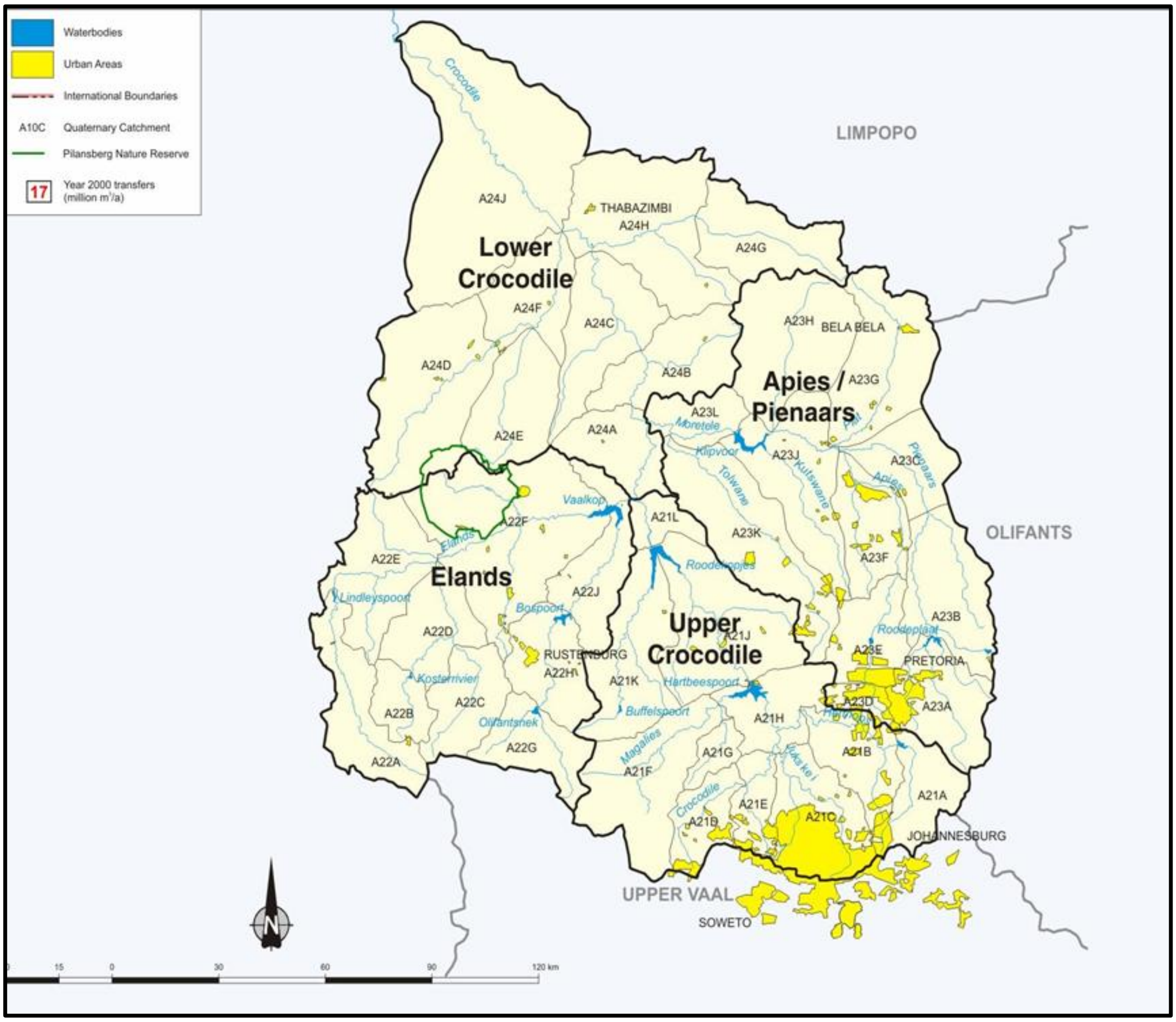


FIGURE 1.5: CROCODILE RIVER CATCHMENT (BOONZAAIER, 2008).

1.4.4. CLIMATOLOGY

The study site falls under the Thabazimbi Local Municipality of the Waterberg District in the Limpopo Province of South Africa. In the Thabazimbi region, everyday temperatures are mild to hot, with a day-to-day extreme average of 27°C to 33°C, but can go as high as 45°C. The regular minimum average differs between 8°C and 12°C (Thabazimbi Local Municipality, 2018/19). The average annual rainfall is approximately 650 mm in this region and the maximum rainfall total in 24 hours is 150 mm (Judeel, 2017). Rainfall is extremely periodic, with peak rainfall happening as thunderstorms in the summer season from October to April (Thabazimbi Local Municipality, 2018/19).

1.5. Research Questions

The following set of questions provided the thrust for the researcher to complete the objectives of the study:

- Can existing rock mass classification schemes be applied on soft rock slopes in surface mining operations?
- Are the rock mass classification schemes applied in hard rock suitable for carbonaceous rock?
- Can neglected hard rock properties cause changes in rock strength for carbonaceous rock?

1.6. Aim and Objectives

1.6.1. Aim

Evaluating the most applicable rock mass classification schemes on rock slopes in limestone surface mining operations.

1.6.2. Objectives

- Application of rock mass classification schemes in surface mining to determine rock mass properties
- Determine which existing rock mass classification scheme is most suitable for surface mining of carbonaceous rock.
- Assess which characteristics of carbonaceous rock affect the use of existing rock mass classification schemes.
- Describe limitations of existing rock mass classification schemes that are used on rock slopes in limestone surface mining operations.

CHAPTER 2: LITERATURE REVIEW

2.1. SLOPE STABILITY IN OPEN-PIT MINING

In order for mining operations to be both safe and cost-effective, slope stability in open-pit mines must be evaluated at various stages of excavation (Fleurisson, 2012). Stable slopes are essential for safe mining operations as slope failures may hinder movement along the benches and, at times, create geohazards to the miners which then has repercussion of a loss of production or worse death of mining staff. During the pre-feasibility study, probability and developing mining stages, exploration boreholes are drilled to provide ore reserve data (Stacey, 2001). These drilled core data are also used to obtain geotechnical information through the rapid geotechnical core logging technique (Stacey, 2001). The geotechnical core logging technique provides information such as drilled record, recovery, geotechnical interval, rock type, rock competence and joint surface condition (Stacey, 2001). The geotechnical information assist with characterising the rock mass of the area, identify potential deformation and failure within the mine. This information is then used to decide on the slope design, monitoring and support systems (Fleurisson, 2012).

The drive for the excavation regulates the configuration and size (Stacey, 2001) of the mining area. The mining abstraction excavation configuration is determined by the orebody outline and the preferred mining method. The mining method also takes into consideration the access route to the orebody such as haulage and crosscut developments for transportation of the ore and type of equipment to use (Poxleitner, 2016). Best blueprint of pit and quarry slopes also encompasses the profitable issues, for instance the selection of mining and the selection of mining equipment (Stacey, 2001). The selectivity of mining depends on the value of the ore and size of the deposit. Thus, it can be extracted at different bench pinnacles due to the dissemination of the ore grade. The choice of the mining tools will be dependent on the magnitude of mining such as the day-to-day extraction requirements may require a 15- to 30-meter bench elevation in order to yield the necessary tonnage of ore (Stacey, 2001).

Steep slopes are favourable in terms of the economics of surface mines, while low slope angles favour slope stability. Thus, geotechnical engineers at open-pit mines are tasked with steepening slope angles so as to lessen unwanted stripping costs meanwhile sustaining safe high-walls in the pit (Bye & Bell, 2001). The overall pit slope is vertiginous in competent rocks than in frail rocks, at times with fluctuating slope angles around the same open-pit mine.

2.2. Failure Criterion

Hoek-Brown (non-linear) and Mohr-Coulomb (linear) failure criterions are the conventionally-used rock failure conditions in slope stability analyses (Aksoy *et al*., 2016). Most geotechnical software is developed using the Mohr-Coulomb failure criterion for slope stability analysis (Hoek *et al*., 2002). The homogeneous material conditions are described by a series of linear equations in principal stress space, where any effect from the intermediate principal stress $\sigma 2$ is disregarded (Aksoy *et al*., 2016). The Mohr-Coulomb failure criterion can be demonstrated as a function of principal stresses or normal stress (σ), major ($\sigma 1$) and minor ($\sigma 3$) and shear stress (τ) on the failure plane (Jaeger & Cook, 1979; from Aksoy *et al.,* 2016). Mohr-Coulomb suggested the relationship as;

$$\tau = c + \sigma tan\varphi \text{ EQ 2.1}$$

Where:

- c is cohesion; and
- φ *is* internal frictional angle.

The most important constraints in the Mohr-Coulomb failure criterion are cohesion (*c*) and internal frictional angle (*Φ*) (Coulomb, 1773 from Wyllie, 2004)**.** The cohesion and internal frictional angle are applied to determine the shear strength of the rock mass of a slope (Karman *et al*., 2013). Cohesion and internal frictional angle datum of a rock mass is collected from triaxial testing. Triaxial testing determines the shear strength of a rock mass from cohesion and internal frictional angle and also determines the rigidity of the rock mass. Data collection for triaxial testing is difficult due to several factors such as sample disturbance and equipment size limitation. The initial Hoek-Brown failure criterion (Hoek & Brown, 1980) was established to make means of appraising the strength of jointed rock masses. A researched imitative association known as the Hoek-Brown failure criterion describes a non-linear increase in isotropic rock's peak strength with increasing confining stress (Aksoy *et al*., 2016). In place of intact rock, the original Hoek-Brown failure criterion can be written in the succeeding equation:

$$\sigma'1 = \sigma'3 + \sigma ci(mb\ \sigma'3/\sigma ci + S)^{a} \text{ EQ 2.2}$$

where*:*

- σ′1 and σ′3 are the maximum and minimum effective stresses at failure;
- Mb is the value of the Hoek–Brown constant m for the rock mass;
- s and a represent the constants that depend on the rock-mass characteristics; and
- σci is the uniaxial compressive strength of the intact rock pieces.

The uniaxial compressive strength can be determined either from laboratory or be predictable from available Tables (Hoek *et al*., 1998). The shear strength of the rock mass stipulated by angle of internal friction and cohesion can be predicted from the published curves plotted here as Figure 2.1 A and B.

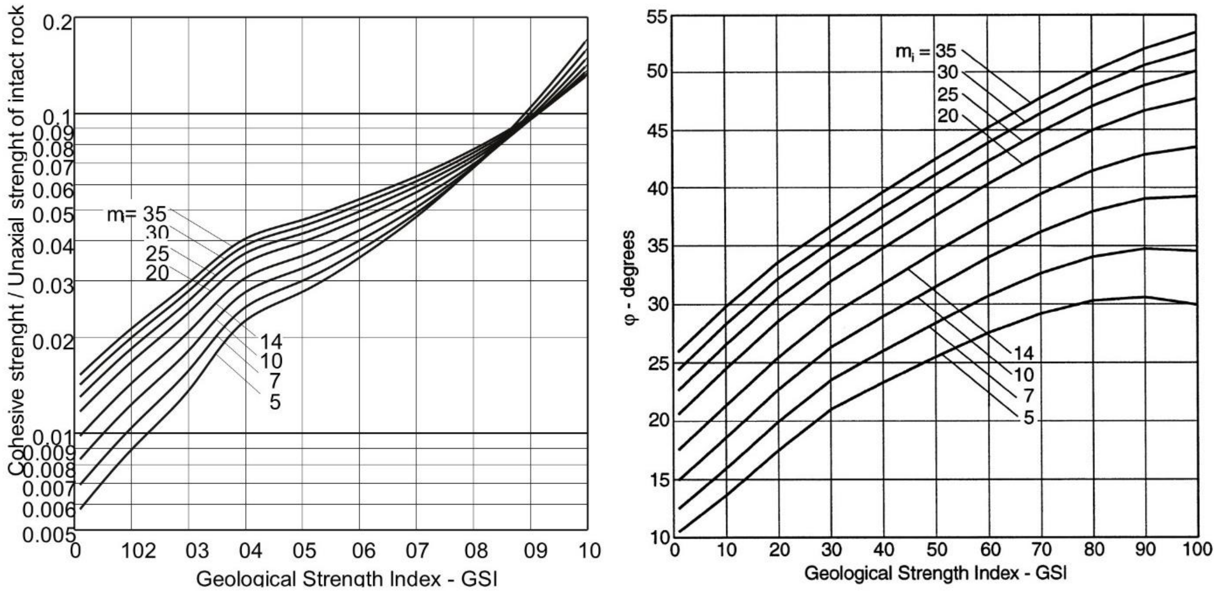


Fig. 2.1A **Fig. 2.1B**

FIGURE 2.1: A AND B PUBLISHED CURVES PLOT FOR COHESION STRENGTH AND ANGLE OF INTERNAL FRICTION (HOEK *ET AL*., 1998).

2.3. Potential Slope Failure (Mine Design)

Most slope failures are designed based on two categories which are structural controlled and non-structural controlled failures (Hustrulid *et al*., 2000). Structural controlled failures include toppling failure, plane failure and wedge failure. Non-structural controlled failures include weakening of the rock mass from second-order structural attributes such as bedding planes and joints. The possible failure can then be utilised to define the best slope stability design for mining operations.

A plane failure is a structural controlled failure (Kliche, 1999). It happens when a rock block slides on a single plane that dips out of face (Figure 2.2). The geometrical circumstances for plane failure to transpire are:

1. The plane that slip must strike the slope face parallel or almost parallel (within about ±20°)
2. It is compulsory for the slipping plane to “daylight” in the slope face, which implies that its dip must be smaller than the slope face’s dip
3. The dip of the slipping plane must be bigger than the angle of friction of this plane

4. The greater end of the slipping surface either crosses the upper slope, or end in a tension crack.
5. Release surfaces that offer insignificant resistance to sliding must exist in the rock mass to express the adjacent boundaries of the slide. On the other hand, failure can transpire on a sliding plane traversing through the convex “nose” of a slope.

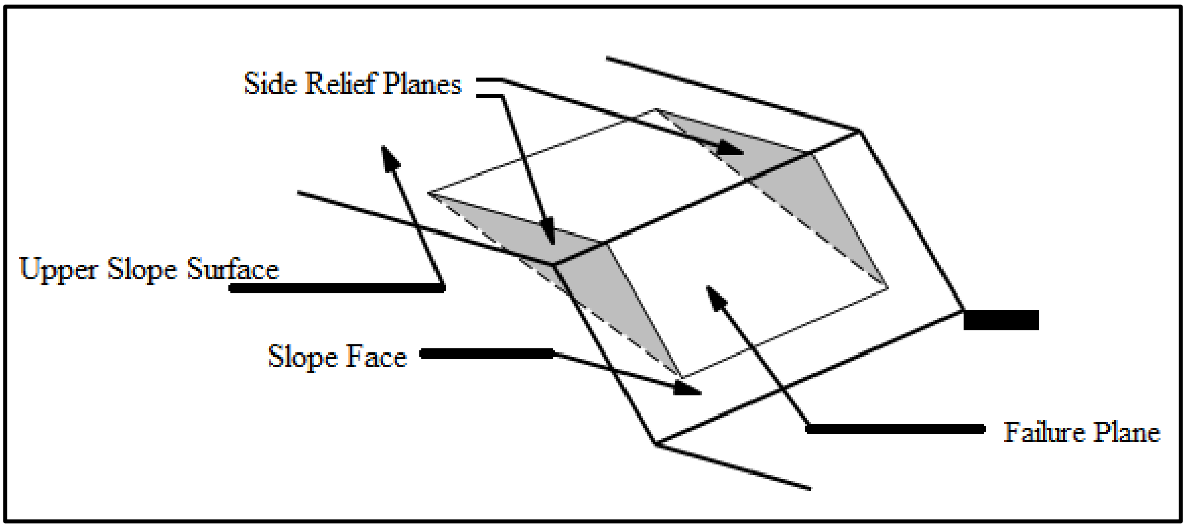


FIGURE 2.2: DIAGRAM OF A PLANE FAILURE (ADAPTED FROM KLICHE, 1999).

When two planar (continuous) discontinuities and the lines of intersection of the two planes daylights just at the bottom of the rock face a wedge failure takes place.

The configuration of the wedge for scrutinising the basic mechanism of sliding is distinct in Figure 2.3. According on this configuration, the common settings for a wedge failure are as follows:

1. The two planes will at all times cross in a line.
2. The descent of the line of intersection has to be flatter than the dip of the face, and steeper than the regular friction angle of the two slide planes. The inclination of the slope face is calculated in the view at 90 degrees to the line of intersection.
3. The line of intersection s to dip in a direction out of the face for gliding to be possible.

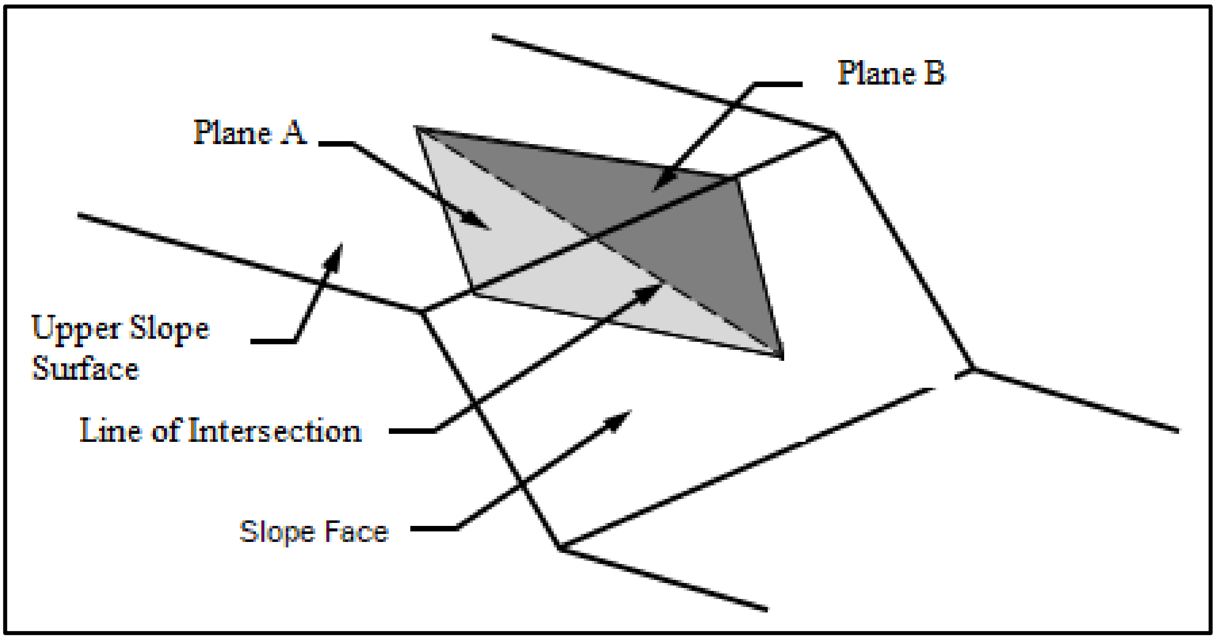


FIGURE 2.3: DIAGRAM OF A WEDGE FAILURE (ADAPTED FROM KLICHE, 1999).

A circular failure commonly happens in a highly intensified fractured or heavily weathered rock. In a highly intensified fractured or heavily weathered rock the intensely distinct structural pattern no longer occurs, and the glide surface is able to find the line of lessor resistance from the slope. Slope failure in such materials propose that this slide surface normally takes the form of a circle. The circumstances below which circular failure will happen ascends when the distinct particles in a soil or rock mass are much smaller as compared to the size of the slope. Thus, fragmented rock in a fill will tend to perform as a “soil” and decline in a circular mode when the slope magnitude is largely bigger than the magnitude of the rock pieces (Figure 2.4).

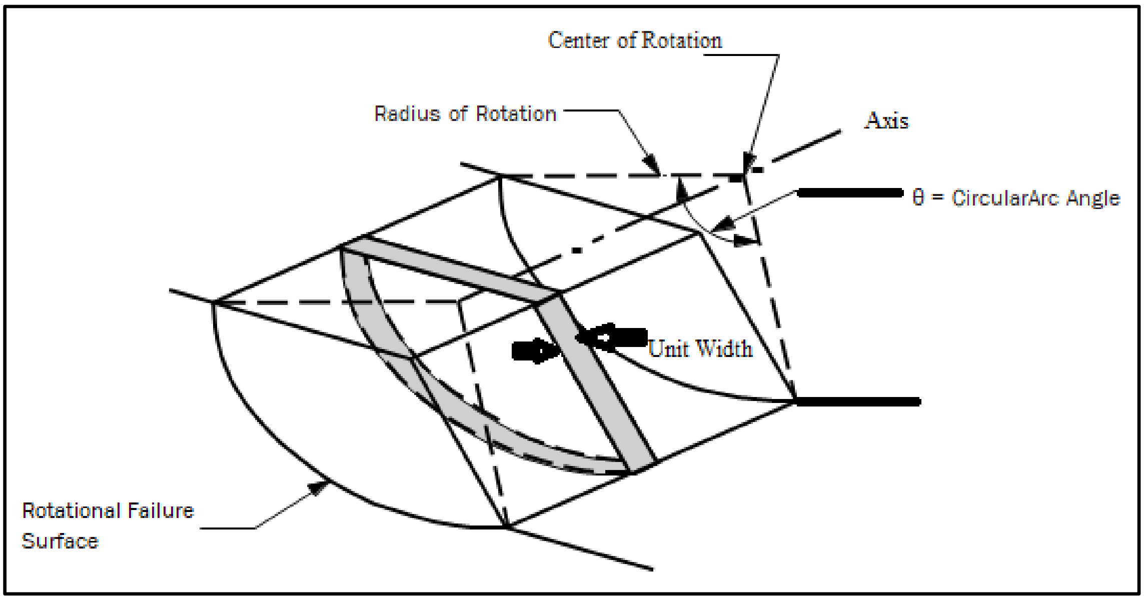


FIGURE 2.4: DIAGRAM OF A CIRCULAR FAILURE (ADAPTED FROM KLICHE, 1999).

A toppling failure comprises of turning of pillars or blocks of rock around a immobile base. Alike to the plane and wedge failures, the steadiness examination of toppling failures includes, first, conducting a kinematic analysis for the structural geology to recognise the possible toppling (Figure 2.5).

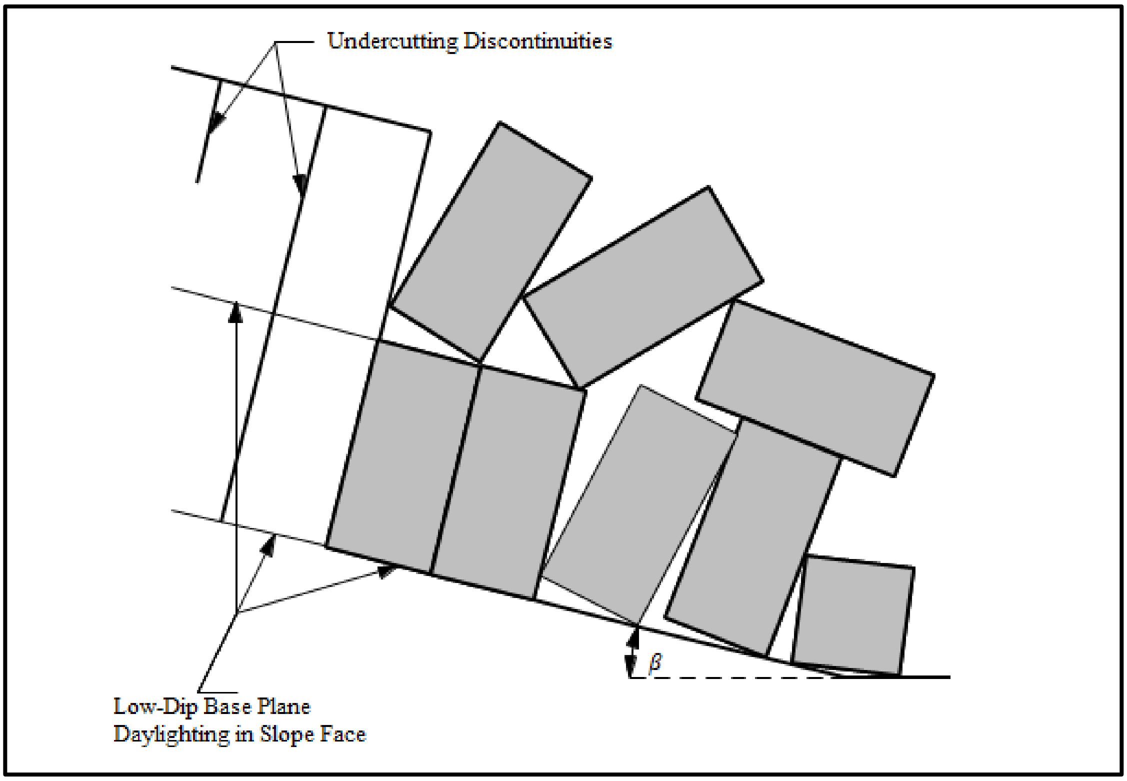


FIGURE 2.5: TOPPLING FAILURE (ADOPTED FROM KLICHE, 1999).

2.4. Slope Stability Methods

In evaluating the steadiness of a rock slope, the utmost significant aspect to be considered is the configuration of the rock mass beyond the face (Wyllie & Mah, 2004). There are various direct and indirect approaches for defining the elastic deformability modulus of fractured rock masses and elastic strength (Khani *et al*., 2013). The best frequently applied indirect approaches in the valuation of slope steadiness being rock mass classification, limit equilibrium, kinematic analysis and numerical modelling (Karaman *et al*., 2013). The direct method of determining strength and deformability is laboratory testing.

2.4.1. Kinematic Analysis

Kinematic analysis is a technique applied to examine the possibility for different modes of rock slope failure owing to the presence of adverse oriented discontinuities. It looks at the connection amongst the orientation of the discontinuities and the face of the slope (Wyllie & Mah, 2004). Kinematic analysis is conducted using stereographic representation of the rock slope and is constructed from Markland's test (Markland, 1972). The Markland test is an important instrument for recognising discontinuities that can show wedge, planar or toppling failure modes in a rock mass (Zaki *et al*., 2012). A discontinuity is a plane or surface that notes a variation in the chemical or physical appearance in a rock mass or soil. A discontinuity could be in the form of a foliation, bedding plane, schistosity, fracture, joint, crack, cleavage, fault plane or fissure (Kliche, 1999). In Markland's test, if a discontinuity dips in the same path as the slope face, at an angle below (within 20°) the slope angle, however, bigger than the friction angle alongside the failure surface then a plane failure may transpire (Figure 2.6.a) (Hoek & Bray, 1981). A wedge failure may transpire if the line of intersection of two discontinuities, making a wedge-shaped block, dips in the same path as the slope face and the plunge angle is smaller than the slope angle but bigger than the friction angle along the planes of failure (Hoek & Bray, 1981) (Figure 2.6.b). When a precipitously dipping discontinuity is running alongside the slope face (within 30^0) and dips into it then a toppling failure may transpire (Hoek & Bray, 1981) (Figure 2.6.c). Circular failure transpires in highly fractured rock with randomly oriented discontinuities, rock fill or very weak rock. (Wyllie & Mah, 2005) (Figure 2.6.d).

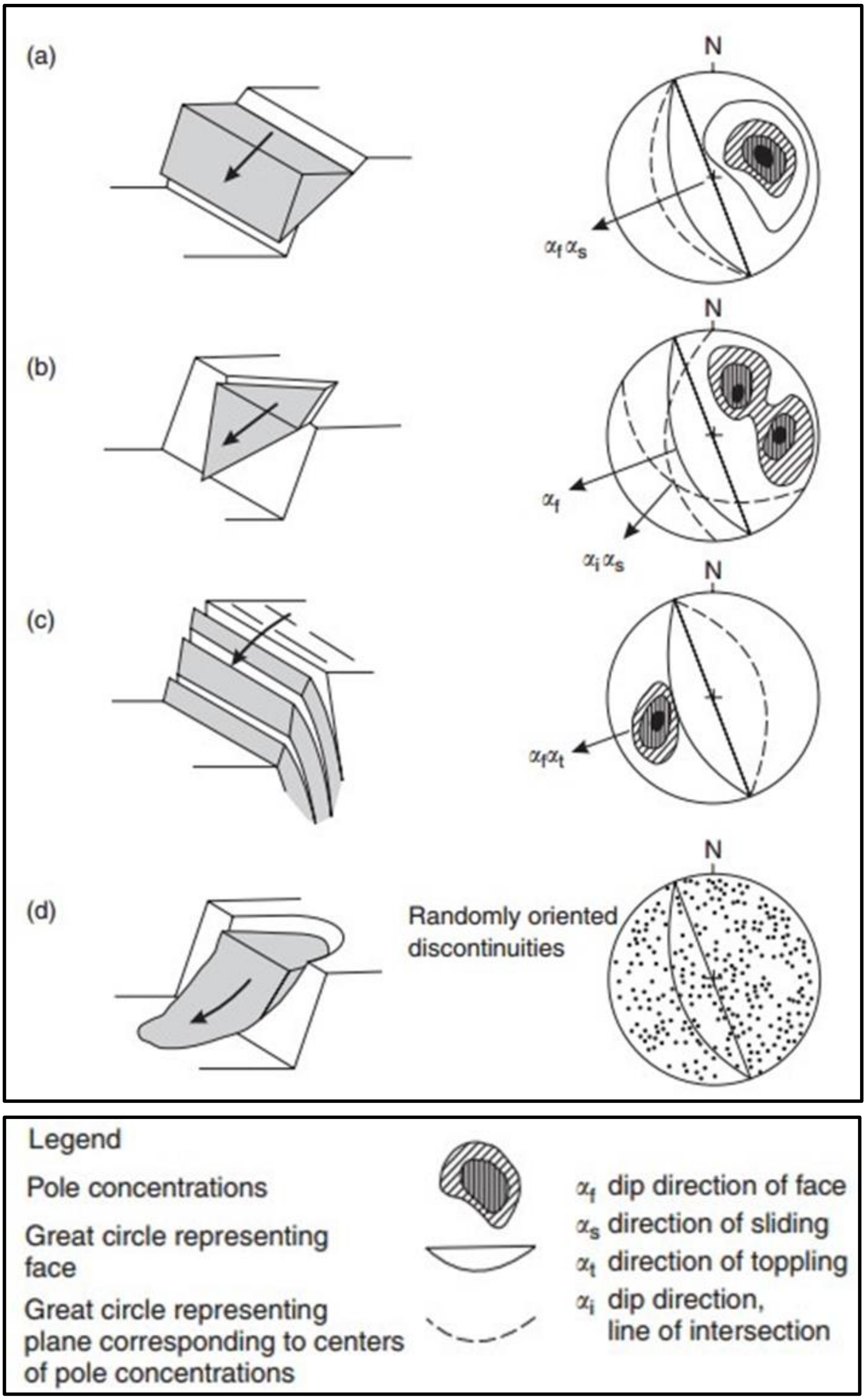


FIGURE 2.6: MARK'S TEST (ADAPTED FROM HOEK & BRAY, 1981).

2.4.2. Limit Equilibrium

The limit equilibrium technique is applied to regulate the sensitivity of a possible slope failure condition based on the slope geometry and rock mass parameters (Kliche, 1999). It is a traditional method applied in slope stability assessments. The end products from the limit equilibrium technique are the probability of failure (POF) and factor of safety (FOS) calculations (Kanda & Stacey, 2016). The factor of safety is the proportion of the repelling forces (shear strength) that tend to resist the slope motion from the energetic forces (shear stress) that tend to activate the motion along a plane of discontinuity. The equation for FOS is:

$$FOS = (c + s \tan f)/t \qquad \text{EQ 2.3}$$

Where:

FOS = factor of safety

c = cohesion

f = angle of internal friction

s = normal stress on slip surface

t = shear stress

In accordance to limit equilibrium, if a slope has a safety factor of 1.0 or more is it considered as a stable slope and when the safety factor is less than 1.0 then it is unsteady (Kliche, 1999). Probability of failure (POF) is a different technique to FOS in limit equilibrium (Chiwaye & Stacey, 2010). In POF, contributing constraints are defined as probability distributions and are combined with a deterministic system applied to compute the FOS (Chiwaye & Stacey, 2010). Figure 2.7 shows the graphic representation of POF and its association with FOS in accordance with hesitation measure (Tapia *et al*., 2007).

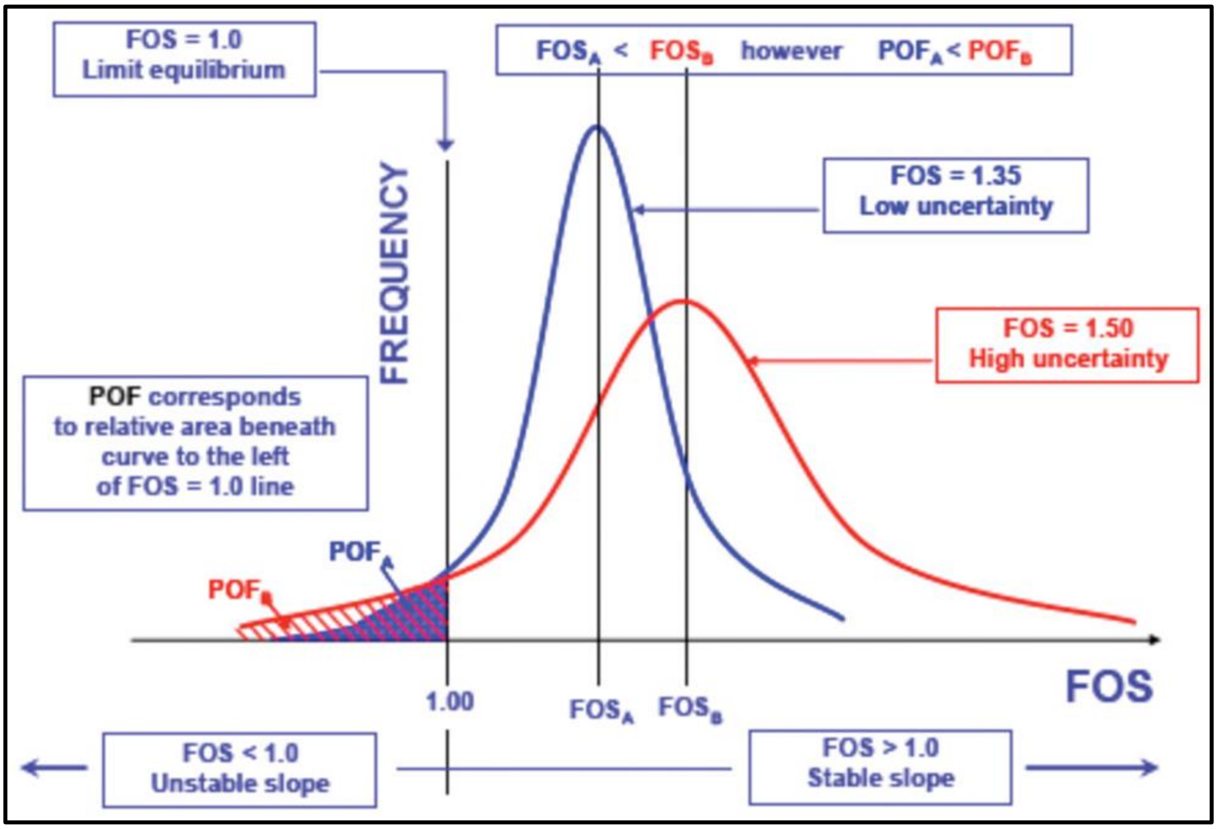


FIGURE 2.7: GRAPHIC REPRESENTATION OF POF AND ITS ASSOCIATION WITH FOS (TAPIA *ET AL*., 2007 IN CHIWAYE & STACEY, 2010).

2.4.3. Numerical Modelling

Numerical modelling attempts to signify the mechanical reaction of a rock mass exposed to a series of preliminary environment such as water levels and *in situ* stresses, marginally conditions and induced variations such as slope digging. Hudson & Harrison (1992) considered that modelling for slope stability in rock engineering is necessary as it does not solitary consider the discrete factors of the system but also how these factors all interrelate jointly. Recognition of all the pertinent factors of the system, equivalent to the physical variables, and the connecting of mechanisms is vital, and their joint operation must be taken into consideration (Hudson & Harrison, 1992).

2.4.4. Empirical Method

Slope stability analysis uses the rock mass classification as an empirical technique. It was initially used in civil engineering and all engineering parameters affecting rock mass were included (Gundewer, 2014). In the mining trade, the rock mass classification was initially used in underground mining for tunnel support in hard rock (Gundewer, 2014). Parameters such as weathering, water pressure and the influence of water in hard rock was usually insignificant thus ignored. The rock mass classification system matches up the practical experience exposed at a preceding site with the current site to give an estimation of slope stability. Several classification systems have been established in the past for surface and underground stability evaluation. The most commonly used rock mass classification methods include Rock Mass Rating, Q-tunnelling and Geological Strength Index (Abbas & Konietzky, 2017). The application of a rock mass classification method offers a line-up of items that must be considered in the process of assessing the characteristics of the rock masses in which slopes are to be mined (Hustrulid *et al*., 2000).

2.5. Rock Mass Classification

Rock mass classification forms the backbone of the empirical draft technique in mining and is broadly used in rock engineering. The use of a rock mass classification provides a line-up of items that must be considered in the process of assessing the characteristics of the rock masses in which slopes are to be mined (Hustrulid *et al*., 2000). The rock mass classification derives shear strength constraints from practical experience encountered at previous site to the current site with similar characteristics.

A rock mass is a rock substance together with discontinuities. The steadiness in a mine digging in a jointed rock mass is controlled by many factors such as strength of occurrence of water, rock material, joint strength, regularity of jointing and confining stress (Stacey, 2001). In accordance with Bieniawski (1993), the intentions of rock mass characterisation and classification are:

- To recognise the most important parameters driving the performance of a rock mass.
- To split a specific rock mass development into a number of rock mass sections of different quality.
- To offer a foundation for understanding the features of each rock mass class
- To develop measureable data for engineering design.
- To endorse support strategies for tunnels and mines.
- To offer a mutual base for communication amongst engineers and geologists.
- To relate the knowledge on rock settings at one location to the conditions experienced gained at other.

Rock mass classification systems have extensively been applied with great success in South Africa, Austria, Europe, the United States of America and India for the following reasons:

1. It improves better communication amongst designers, geologists, planners, engineers and contractors.
2. An engineer's view, knowledge, and decision are correlated and merged more efficiently by an engineering (assessable) classification system.
3. Engineers favour numbers as opposed to long explanation; thus, an engineering classification system has substantial application in a general evaluation of the rock quality.
4. The classification method assistances in the grouping of knowledge and is incredibly successful.
5. A perfect application of engineering rock mass classification takes place in the forecasting of tunnels, hydroelectric projects, silos, caverns, bridges, hill roads, building complexes, rail tunnels, and so forth (Stacey, 2001).

The engineering characteristics of the rock material and discontinuities should be taken into account in any engineering design for rock mass stability (Abbas & Konietzky, 2017). Several constraints must be measured in order to define a rock mass acceptably for guaranteeing rock mass stability. The several significant constraints applied for depiction and classification of rock mass (Bieniawski, 1993) are:

- the intensity of the intact rock material (compressive strength, modulus of elasticity)
- the rock quality designation (RQD) which is the estimation of the drill core quality or intensity of breaking
- factors of rock joints such as orientation, layout, and form (surface roughness, aperture, weathering and infilling)
- flow and pressure of groundwater
- in situ stress
- main geological structures (folds and faults).

Rock mass classification systems originated from Ritter (1879) when he developed an empirical technique for tunnel projects and to provide the requisite support (Hoek, 2007). Ever since then, various rock mass classification systems have been established for surface and underground stability evaluation. Rock mass classification systems applied in surface slope stability evaluations includes:

- Bieniawski's Rock Mass Rating
- Hoek-Brown Geological Strength Index

- Slope Mass Rating
- Laubsher's Rock Mass Rating
- Engineering classification of karst

In all engineering classification schemes, the lowest rating is referred to as the 'poor rock mass' and the highest rating is called "excellent rock mass" (Goel & Singh, 2012). At present, rock mass classification schemes are also utilised in combination with numerical simulations (Abbas & Konietzky, 2017). Constraints such as intensity and deformation can be assumed and used in numerical simulations to define stability, factor of safety, failure pattern, etc. (Abbas & Konietzky, 2017).

2.5.1. Bieniawski Rock Mass Rating (RMR)

The Rock Mass Rating was established by Z.T. Bieniawski in 1974 at the Council of Scientific and Industrial Research (CSIR) in South Africa (Zhang *et al*., 2019). It was established to evaluate the stability and support requisite of tunnels. The classification has gone through numerous substantial evolutions. In 1974, dropping of the classification constraints from 8 to 6; in 1975 modification of ratings and reducing of the suggested support requisite. In 1976, further modifications were made to the class confines to even multipliers of 20; and in 1979, the ISRM (1978) rock mass narrative was approved (Goel & Singh, 2012). In applying this classification system, the rock mass is divided into several structural areas and each area is categorised individually. The confines of the structural areas usually concur with the main structural features such as a change in rock type or fault. In some instances, substantial variations in discontinuity spacing or features, within the same rock type, may require the partition of the rock mass into smaller structural areas. The six parameters that are utilised to categorise a rock mass applying the RMR system are as follows: Uniaxial compressive strength of rock material, rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions and orientation of discontinuities (Goel & Singh, 2012). The Rock Mass Rating system is seen in Table 2.1, showing the scoring for each of the six parameters recorded below. These scorings are combined to provide a value of RMR.

TABLE 2.1: BIENIAWSKI ROCK MASS RATING CLASSIFICATION SYSTEM (BIENIAWSKI, 1993)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

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

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

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

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100-81	80-51	50-41	40-21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

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

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

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

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

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

2.5.2. Geological Strength Index (GSI)

The geological strength index (GSI) is a classification for predicting the loss in rock mass intensity for diverse geological environments as recognised by field viewing (Hoek *et al*., 1998). The system was presented to control the shortcomings in RMR for extremely poor quality rock masses (Zhang*, et al*. 2019). The system is straight-forward and it is dependent on the visual observation of the surface conditions of the discontinuities (joint modification and roughness) and rock structure (blockiness). The visual observation of the two parameters produces a practical foundation for defining a wide range of rock mass forms with broad rock structure ranging from extremely compacted interlocked strong pieces to heavily crumpled rock masses (Hoek *et al*., 1998). Depending on the rock mass narrative the value of GSI is predicted from published Table, reproduced here as Table 2.2 The uniaxial compressive strength (σci) and the material constant (*m*i) are determined by laboratory testing or estimated from published Tables, reproduced here as Tables 2.3 and Table 2.4. In a jointed rock mass Hoek-Brown developed a guide on which condition should GSI be applied in study area. Figure 2.8 shows under which joint conditions should GSI be applied.

Marinos (2010) worked on the geotechnical classification of complex and weak and rock masses. The complexity and weak rock masses under reflection is often heterogeneous, consisting of rocks with very low strength and have in most situations experienced high tectonic disruption causing the destruction of their original structure. The geotechnical types and their classification of the rock masses that can be applied in molasses, flysch, ophiolites, fragmented limestone and disturbed or weathered gneiss were studied. The emphasis is largely on the fragmented limestone and the proposed description of Geological Strength Index (GSI) for limestone (Table 2.5).

TABLE 2.2: HOEK-BROWN GEOLOGICAL STRENGTH INDEX (HOEK *ET AL*, 1998).

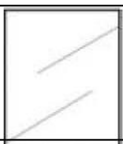
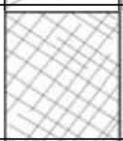
GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000) From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis. SURFACE CONDITIONS STRUCTURE DECREASING INTERLOCKING OF ROCK PIECES	VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
 INTACT OR MASSIVE—intact rock specimens or massive *in situ* rock with few widely spaced discontinuities	90 80			N/A	N/A
 BLOCKY—well interlocked un-disturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		70 60			
 VERY BLOCKY—interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets			50 40		
 BLOCKY/DISTURBED/SEAMY folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity				30	
 DISINTEGRATED—poorly inter-locked, heavily broken rock mass with mixture of angular and rounded rock pieces					20 10
 LAMINATED/SHEARED—lack of blockiness due to close spacing of weak schistosity or shear planes	N/A	N/A			

Table 2.3: Field estimates of the uniaxial compressive strength of intact rock pieces (Hoek *et al*., 1998).

Grade[a]	**Term**	**Uniaxial compressive strength (MPa)**	**Point load index (MPa)**	**Field estimate of strength**	**Examples**
R6	Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 – 250	4 - 10	Specimen requires many blows of a geological hammer	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff
R4	strong	50 - 100	2- 4	Specimen requires more than one blow of a geological hammer to fractured	Limestone, marble, phyllite, sandstone, schist, shale
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Claystone, coal, concrete, schist, shale, siltstone
R2	Weak	5 - 25	[b]	Can be peeled with a pocket knife difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, rocksalt, potash
R1	Very weak	1 – 5	[b]	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock
R0	Extremely weak	0.25 - 1	[b]	Intended by thumbnail	Stiff fault gouge

[a] Grade according to Brown (1981)

[b] Point load tests on rocks with a uniaxial compressive strength below 25 MPa with a possibility to yield vague results

TABLE 2.4: VALUES OF THE CONSTANT MI FOR INTACT ROCK, BY ROCK GROUP (HOEK ET AL., 1998).

Rock type	Class	Group	Texture			
			Coarse	**Medium**	**Fine**	**Very fine**
Sedimentary	Clastic		Conglomerate (22)	Sandstone (19)	Siltstone (9)	Claystone (4)
	Non-clastic	Organic		Greywacke (18)		
				Chalk (7)		
				Coal (8 – 21)		
		Carbonate	Breccia (20)	Sparitic limestone (10)	Micritic limestone (8)	
		Chemical		Gypstone (16)	Anhydrite (13)	
Metamorphic	Non-foliated		Marble (9)	Hornfels (19)	Quartzite (24)	
	Slightly foliated		Migmatite (30)	Amphibolite (25 – 31)	Mylonites (6)	
	Foliated		Gneiss (33)	Schists (4 – 8)	Phyllites (6)	Slate (9)
Igneous	Light		Granite (33)		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
			Diorite (28)			
	Dark		Gabbro (27)	Dolerite (19)	Basalt (17)	
			Norite (22)			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

[a]*These values are for intact rock specimens tested normal to bedding or foliation.*

The value of mi will be significantly different if failure occurs along a weakness plane.

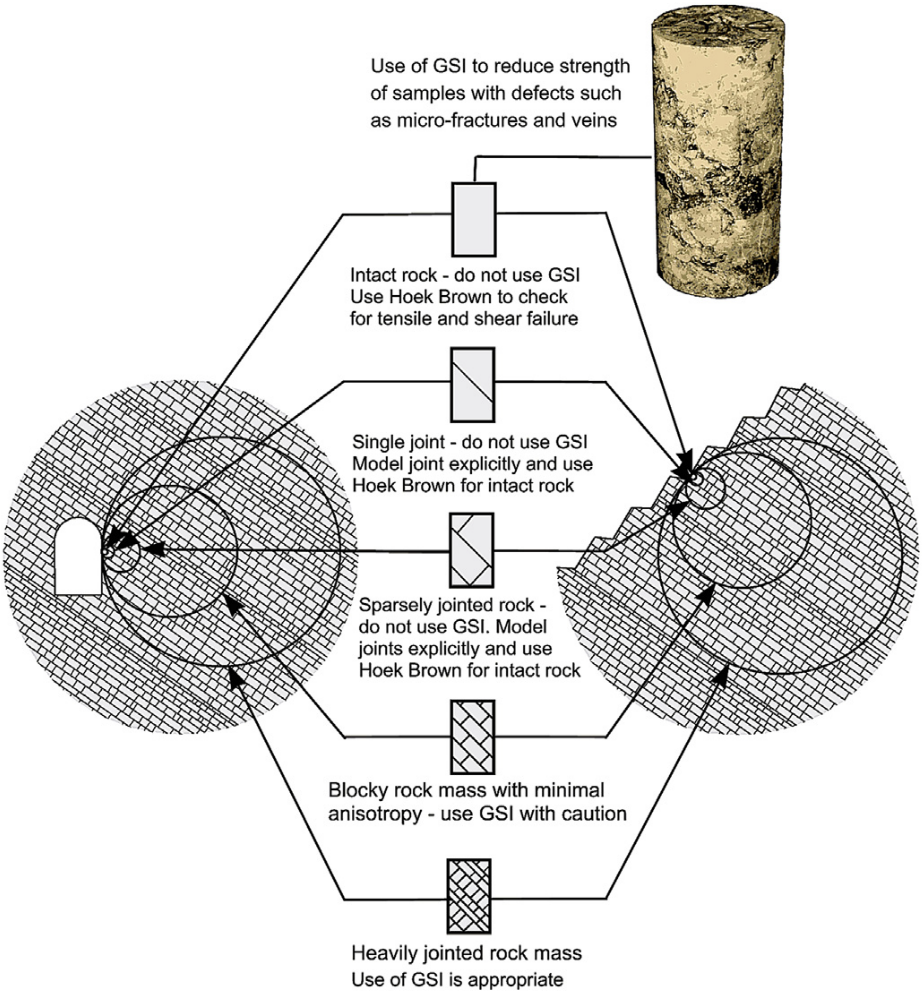


FIGURE 2.8: JOINT CONDITION FOR APPLICATION OF GSI (HOEK-BROWN, 1980).

TABLE 2.5: GSI FOR LIMESTONE (MARINOS, 2010).

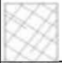
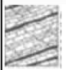
GEOLOGICAL STRENGTH INDEX FOR LIMESTONE ROCK MASS

Based on the description of the lithology, structure and surface conditions of discontinuities (particularly of the bedding planes), choose a box in the chart. Locate the position in the box that corresponds to the conditions and estimate the average value GSI from the contours. Quoting a range from 33 to 37 is more realistic than stating that GSI-35. The determination of the structure and the condition of discontinuities may range between two adjacent fields. Note that the Hoek – Brown criterion does not apply to structurally controlled failures. Where unfavourably oriented continuous weak planar discontinuities (like bedding planes) are present, these will dominate the behaviour of the rock mass (attention therefore at types B and C). The strength of some rock masses is reduced by the presence of groundwater and this can be allowed for by a slight shift to the right in the columns for fail, poor and very poor conditions. Water pressure does not change the value of GSI and it is dealt with by using effective stress analysis

Surface conditions of discontinuities (Predominately bedding planes)

DECREASING SURFACE QUALITY ⇨

DECREASING INTERLOCKING OF ROCK PIECES ⇩

STRUCTURE	**VERY GOOD** Very rough, fresh unweathered surfaces	**GOOD** Rough, slightly weathered, iron stained surfaces	**FAIR** Smooth, moderately weathered and altered surfaces	**POOR** Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	**VERY POOR** Slickensided, highly weathered surfaces with soft clay coatings or fillings
 Type-A undisturbed thick bedded to non-bedded limestone, well interlocked consisting of cubical blocks formed by three intersecting discontinuity sets.	80				
 Type B- undisturbed thin to medium-bedded limestone, with tight well interlocked structure consisting of cubical blocks form three intersecting discontinuity sets. Bedding thickness is of several cm to few dm					
 Type C- undisturbed thin to medium bedded limestone with claystone or siltstone or chert alternations, with tighten-well interlocked structure consisting of cubical blocks formed by three intersecting discontinuity sets. Bedding thickness is of several c to few dm	70				
 Type D- Very blocky limestone, well interlocked mass with multi-facted angular blocks formed by 4 or more joints.		60 50			
 Type E – folded-highly disturbed thin bedded limestone with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes.			40		
 Type F- folded-highly disturbed thin bedded limestone with claystone or siltstone or chert alternations, with angular blocks formed by many intersecting discontinuity sets. Loose and open structure due to the poor contact of the blocks with different deformational characteristics. Bedding planes are difficult maintaining their parallelism.			30		
 Type G- Heaily broken, disintegrated limestone. Poorly interlocked with mixture of angular and rounded pieces.				20	
 Type H- Heavily broken, disintegrated limestone with high clay presence along–_the joints. Limestone blocks are not in contact and have very poor interlocking (the scale of this figure is not comparable with the others).					10

2.5.3. Slope Mass Rating (SMR)

The Slope Mass Rating was established by Romana (1985). It is a modified RMR system for slopes. The SMR is by utilising four correction factors from the basic RMR (Bieniawski, 1989). These factors relay on the prevailing relationship amongst discontinuities disturbing the rock mass and the slope, and the slope excavation method. It is obtained using the expression:

$$SMR = RMR + F1F2F3 + F4 \quad (1)$$

where*:*

RMR - is the simple RMR index coming from Bieniawski's rock mass classification scheme

F1 – subject to the parallelism (A in SMR Table 2.6) amongst discontinuity dip direction, αj, and slope dip, αs, (RMR Table)

F2 - is connected to the probability of discontinuity shear strength (Romana, 1993) and is governed by on the discontinuity dip, $B=\beta j$, in the case of planar failure (SMR Table 2.6). For toppling failure, this parameter adopts the value 1.0

F3 - based on the relationship (C in SMR Table 2.6) amongst slope, βs, and discontinuity, βj, dips. This parameter is the original Bieniawski Adjustment Factor (from 0 to -60 points) and expresses the probability of the discontinuity to outcrop on the slope face (Romana, 1993) for planar failure.

F4 - is a correction factor that depends on the excavation method (Table 2.7).

Adjustment factor F4 for the method of excavation

Since the publication of SMR in 1985 a lot of authors have modified the SMR to their own needs, modifying the methodology or the considered parameters (Romana *et al*., 2015). The modified RMR include continuous functions by Tomas (2007), Chinese Slope mass rating (CSMR) and Graphical approach by Tomas (2012).

TABLE 2.6: SLOPE MASS RATING (SMR) TABLE (ROMANA, 1985).

	Type of failure	Very favourable	Favourable	Normal	Unfavourable	Very unfavourable
A	P | αj−αs|	>30°	30–20°	20–10°	10–5°	5°
	T |αj−αs−180|					
	W |αi−αs					
	F1	0.15	0.40	0.70	0.85	1.00
B	P/W |βj|ó|βi|	20°	20–30°	30–35°	35–45°	>45°
	F2	0.15	0.40	0.70	0.85	1.00
C	P βj−βs	>10°	10–0°	0°	0–(−10°)	(−10°)
	P/T/W F3	0	−6	−25	−50	−60
P: planar failure; T: toppling failure; W: wedge failure. αj: dip direction of the discontinuity; αs: dip direction of the slope; αi: dip direction of the intersection line of two sets of discontinuities; βj: discontinuity dip; βi: angle of plunge of the intersection line of two sets of discontinuities; βs: slope dip.						

TABLE 2.7: RMR ADJUSTMENT FACTOR F4 FOR THE METHOD OF EXCAVATION (ROMANA, 1985).

Method of excavation F4 value	**Method of excavation F4 value**
Natural slope	15
Pre-splitting	10
Smooth blasting	8
Normal blasting or mechanical excavation	0
Deficient blasting	-8

TABLE 2.8: DESCRIPTION OF SMR CLASSES (ROMANA, 1985).

Classes	V	IV	III	II	I
SMR	0-20	21-40	41-60	61-80	81-100
Description	Very bad	Bad	Normal	Good	Very good
Stability	Completely unstable	Unstable	Partially stable	Stable	Completely stable
Failures	Big planar or soil-like	wedges	many wedges	some blocks	none
Failure probability	0.9	0.6	0.4	0.2	0

<table>
<tr><th>Slope support guidelines based on SMR</th><th>0</th><th>10</th><th>15</th><th>20</th><th>30</th><th>40</th><th>45</th><th>50</th><th>55</th><th>60</th><th>65</th><th>70</th><th>75</th><th>80</th><th>90</th><th>100</th></tr>
<tr><td>Re-excavation</td><td></td><td colspan="4">Re-excavation walls</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Drainage</td><td></td><td colspan="5">Surface Drainage
Deep drainage</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Concrete</td><td></td><td></td><td></td><td colspan="7">Shotcrete
Dental concrete
Ribs and/or beams
Toe wall</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Reinforcement</td><td></td><td></td><td></td><td></td><td></td><td colspan="9">Bolts
Anchors</td><td></td><td></td></tr>
<tr><td>Protection</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td colspan="5">Toe ditch
Toe or slope fences
nets</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Protection</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td colspan="6">Scaling
None</td></tr>
</table>

2.5.4. LAUBSCHER IRMR AND MRMR

Mining Rock Mass Rating (MRMR) and the Laubscher's In-situ Rock Mass Rating (IRMR) were advanced as extensions of Bieniawski's RMR system for mining usage (Read & Stacey, 2009). The IRMR consists of four simple parameters:

1. The intact rock strength (IRS) is determined as the unconfined compressive strength (UCS).
2. The rock strength (RBS) determined as the strength of the rock blocks contained within the rock mass.

3. The blockiness of the rock mass which is controlled by the number of joint sets and their spacing (JS).
4. The joint condition defined in terms of a geotechnical description of joints contained within the rock mass (JC). The IRMR value is established by adding the JS and JC values to the RBS value. Once the IRMR rating has been established the MRMR value is determined by adjusting the IRMR value to account for the effects of weathering, joint orientation, mining-induced stresses, blasting and water (Figure 2.9; Read & Stacey, 2009).

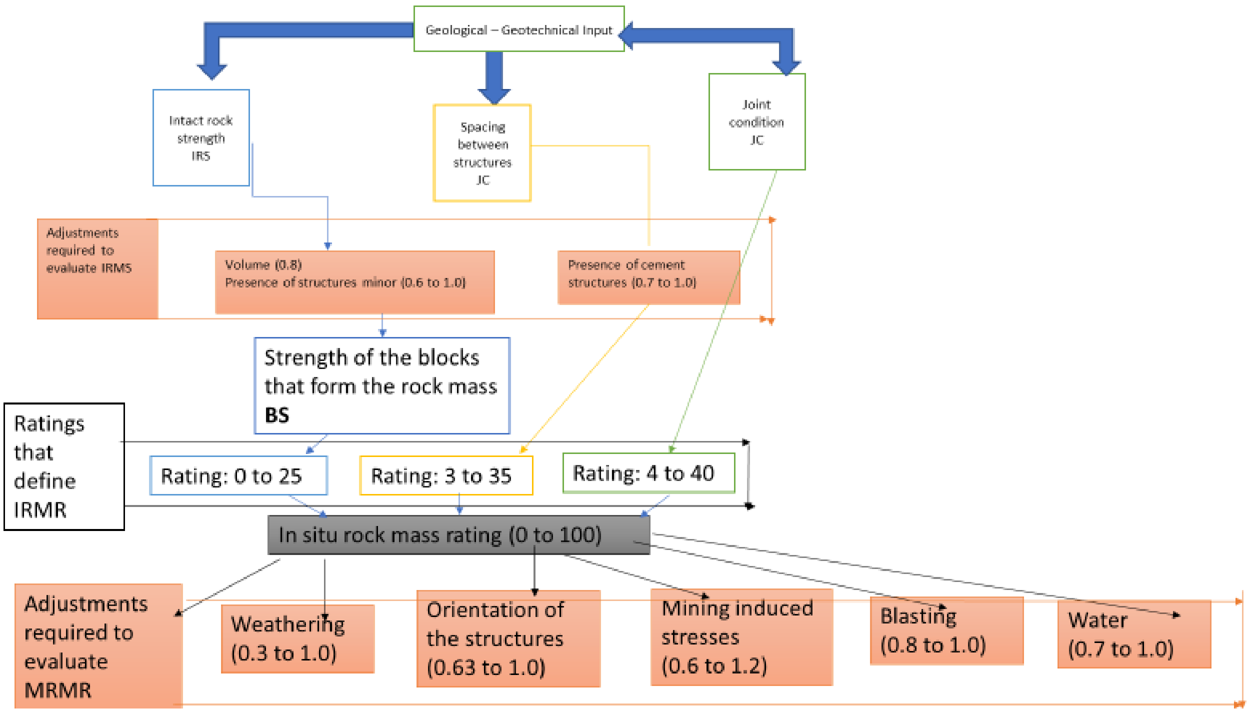


FIGURE 2.9: SCHEMATIC OF THE MINING ROCK MASS RATING (MRMR) (ADOPTED FROM READ & STACEY, 2009).

2.5.5. ENGINEERING CLASSIFICATION OF KARST GROUND CONDITIONS

An engineering classification of karst ground conditions was developed by Walton & Fookes (2003) for weak rock associated with karst conditions. Karst takes place mainly on limestone and dolomite, ground cavities and dissolutional landforms develop best on competent, fragmented rocks whose undamaged unconfined compressive strength is generally 30–100 MPa (Walton & Fookes, 2005). Features associated with karst and dissolution of rocks consist of surface micro-features, surface macro-features, subsoil features and sinkholes. Sinkholes are depressions formed when the ground surface has been eroded around an internal drainage point into the underlying limestone. Figure 2.10 shows the classification of sinkholes by Walton & Fookes (2005), with respect to the mechanisms of the ground failure and the nature of the material which fails and subsides.

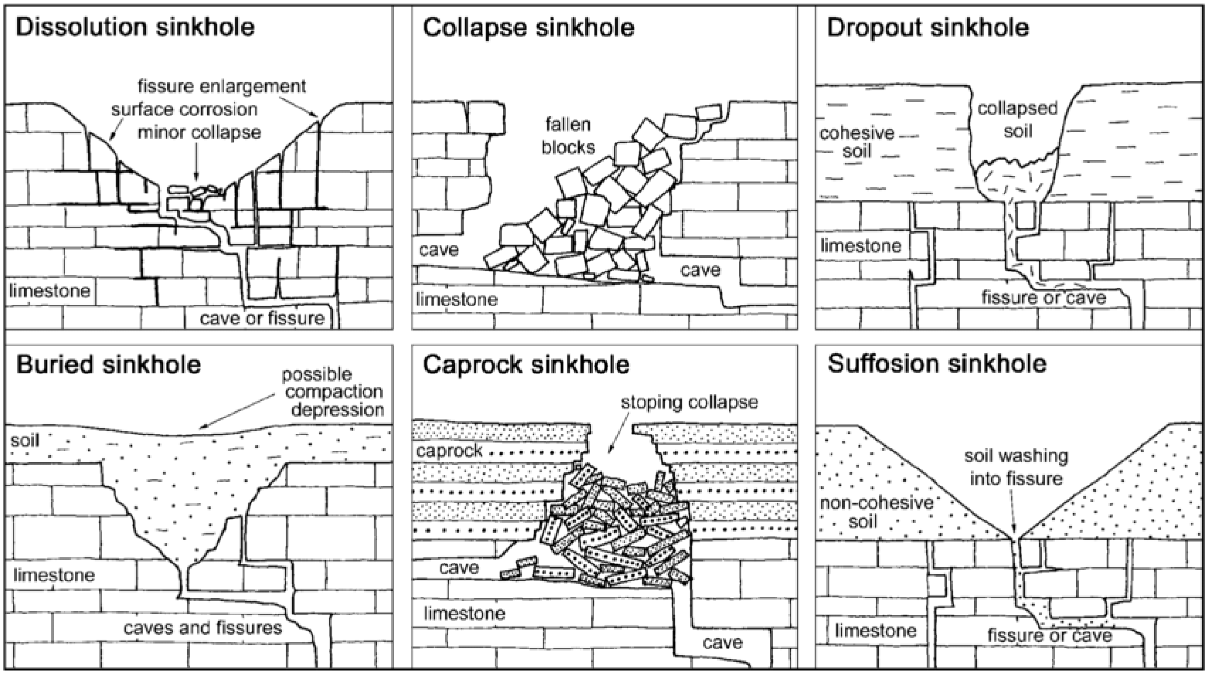


FIGURE 2.10: CLASSIFICATION OF SINKHOLE (WALTON & FOOKES, 2005).

CHAPTER 3: RESEARCH METHODOLOGY

3.1. INTRODUCTION

Chapter three provides a detailed account of the methods, materials and apparatuses applied in the gathering of information and analysis of data to better understand the application of rock mass classification in slope steadiness analysis for our study. The guidelines used to gather geotechnical information was adopted from Palleske (2017) which is outlined in Table 3.1. In Table 3.1 all data types (first column) from geological model to geological characterisation were used. Geological model and structural model (major features) from Table 3.1 were used to collect aerial literature. Structural model (fabric), intact strength, strength of structural defeats and geological characterisation was used to populate the rock mass condition of the mine. The concept of how to collect data was adopted from Dunn (2011) (see Table 3.2). In Table 3.2 guidance as to how much data to collect, when to collected and the level of data collected was used.

3.2. GEOTECHNICAL DATA

Geotechnical data collection consists of rock mass and discontinuity data. The normal geotechnical information collected consists of rock quality designation (RQD), core recovery weathering, rock strength and variation of the rock mass (Human & Jupp, 2016). A rock mass consists of rock material together with discontinuities. The assets of rock mass essentially are influenced by on the performance of discontinuities under different natural and induced stresses (Dev, 2018). The most common discontinuities in a rock mass are bedding plans, joints, faults and other types of cracks (Kemeny *et al*., 2002). The orientation of the joints, together with the frequency of the joints and the roughness of the joints play an important role on slope stability (Dev, 2018).

There are varies direct and indirect approaches for defining the strength and deformability behaviour of fissured rock masses (Khani *et al*., 2013). The direct system of defining the strength and deformability behaviour of fractured rock mass is field work (Khani *et al*., 2013). Field work includes observation of the slopes from the rock face, borehole drilling and collection of samples for laboratory verification. One of the field observation techniques was the usage of rock mass classification. The rock mass classification derives shear strength parameters from practical knowledge faced at preceding site to the current site with similar characteristics.

Borehole drilling was conducted on site with the help of specialised engineers and scientist. During drilling adequate time and planning is required. This takes into account a broad plan conversed to the workers involved preceding to starting; adequate resources, amenities and tools to finish the logging; and a suitable logging system for the recording of the essential data in a manner for stress-free participation into the geotechnical database (Human & Jupp, 2016). After drilling a logging datum is developed to populate geotechnical core logging datum (Human & Jupp, 2016).

Rock samples were taken from the rock face and from the discontinuity infilling material for laboratory testing. Rock samples from the rock face was used to determine the rock type. Rock samples collected from the discontinuities were used to determine the filling material. This infilling material plays significant role within the discontinuities as it widens the discontinuity and allows reactions of minerals within the walls which then result in weathering of the rock mass. The infilling material from the discontinuities were analysed using a microscope.

TABLE 3.1 GEOTECHNICAL GUIDELINES AND THE REQUIRED QUANTITY FOR SURFACE MINING DATUM COLLECTION (STACEY, 2009).

Data type	Conceptual	Project stage			
		Pre-feasibility	Feasibility	Design/ construction	Operations
Geological model	Regional literature; advanced exploration mapping and core logging; database established; initial country rock model	Mine scale outcrop mapping and core logging; enhance geological database; initial 3D geological model	Infill drilling and mapping; further enhance geological database and 3D model	Targeting drilling and mapping; refine geological database and 3D model	Pit mapping and drilling; further refine geological database and 3D model
Structural model (major features)	Aerial photos and initial ground proofing	Mine scale outcrop mapping; targeted oriented drilling; initial structural model	Trench mapping infill-oriented drilling; 3D structural model	Refined interpretation of 3D structural model	Structural mapping on all pit benches; further refine 3D model
Structural model (fabric)	Regional outcrop mapping	Mine scale outcrop mapping; targeted oriented drilling; database established; initial stereographic assessment of fabric data; initial structural domains established	Infill trench mapping and oriented drilling; enhance database; advanced Stereographic assessment of fabric data; confirmation of structural domains	Refined interp. of fabric data and structural domains	Structural mapping on all pit benches; further refine fabric data and structural domains
Intact rock strength	Literature values supplemented by index tests on core from geological drilling	Index and lab testing on samples from targeted mine scale drilling; database established; initial assessment of lithological domains	Targeted drilling and detailed sampling and lab testing; enhance database; detailed assessment and establishment of geotech. units for 3D geotech. model	Infill drilling; sampling and lab testing; refine database and 3D geotech. model	Maintenance of database and 3D geotech. model
Strength of structural defects	Literature values supplemented by index tests on core from geological drilling	Lab direct shear tests of saw cut and defect samples from targeted mine scale drillholes and outcrops; database established; assessment of defect strength within initial structural domains	Targeted sampling and lab testing; enhance database; detailed assessment and establish defect strengths within structural domains	Selected sampling and lab testing and refine database	Maintenance of database
Geological characterisation	Pertinent regional information; geotech. assessment of advanced exploration data	Assess and compile all new mine scale geotech. data; enhance geotech. database and 3D model	Assess and compile all new mine scale geotech. data; enhance geotech. database and 3D model	Refine geotech. database and 3D model	

TABLE 3.2: SUMMARISED ADDITIONAL STRATEGIES THAT CAN BE APLLIED TO DECIDE HOW MUCH GEOTECHNICAL INFORMATION (DUNN, 2011).

Description	
Data density	How much data to collect? Which data collection program to conduct such as strength testing, hydrogeological monitoring?
Data collection timing	When to collect geotechnical data? Should data be collected with the early stages (scoping or pre-feasibility stage) or with resource drilling program?
Level of data collection	Stage 1 (Exploration and scoping studies) - basic geotechnical data (rock quality designation (RQD), fracture frequency, field strength estimates, weathering and alteration) Stage 2 (Pre-feasibility) - detailed geotechnical domain or rock mass characterisation (RMC) Stage 3 (Feasibility) - detailed data on individual discontinuities and orientated core Stage 4 (Detailed design) - dedicated geotechnical holes to assess specific pit walls Stage 5 - implementation and construction
Data collection standards and management	Most often mines adopt the data collection procedures and standards of the consultancy conducting the study

3.3. Shear Strength

In slope stability studies, shear strength parameters are determined either from discontinuities or from the rock mass (Wyllie & Mah, 2004). The parameters of the discontinuities is examined in the field, from the boreholes, digging, and the natural outcrops. Discontinuity parameters examined from fieldwork comprise of length, roughness, persistence, spacing, termination, aperture, filling, orientation, and other properties (Priest, 1993).

Techniques for attaining discontinuity information in the field in a structured way includes usage of handheld compass and measuring tape (Guta, 2017). Characterization of rock mass discontinuities by using field techniques such as cell mapping and scanline survey (Priest & Hudson, 1981; Priest, 1993). In using scanline survey, fracture evidence is taken alongside a streak at a rock face. In using cell mapping, the main structures (faults, joint sets, bedding planes, etc.) on the rock face are recorded first and the secondary information (length, orientation, spacing, etc.) is then determined for each structure. The challenge of using traditional methods of discontinuity data is that is it often difficult to carry out and prone to errors. Incorporation of rock mass classification tables with scanline mapping assist in gathering more information from the slope face from a safe working spacing. Rock mass classifications utilize engineering observation and experience from past case studies with similar conditions (Zhang *et.al*., 2019).

3.4. Instruments Used

3.4.1. Rock Mass Classification Tables

The rock mass classification constitutes a dynamic mine design instrument when used correctly (Dyke, 2008). It provides a hands-on tool for design and has been fruitfully applied in Canada, South Africa, Chile, the Philippines, the United States of America, Australia, Europe and India (Laubscher, 1990). In South Africa, the commonly used rock mass classification systems are the Laubsher's Mining Rock Mass Rating (MRMR), Bieniawski's Rock Mass Rating, Norwegian Geotechnical institute's Q system (Dyke, 2008) and the GSI system (Potvin *et al*., 2012). The rock mass classification derives shear strength parameters from practical experience encountered at previous site to the current site with similar characteristics. Rock mass classification Tables that will be used for this study includes:

- Bieniawski's Rock Mass Rating
- Hoek-Brown Geological Strength Index
- Slope Mass Rating
- Laubsher's Rock Mass Rating
- Engineering classification scheme of karst

The Bieniawski's Rock Mass Rating is used in this study because it looks at the parameters needed for geotechnical data collection (i.e. rock strength UCS and RQD), discontinuity (spacing, orientation and condition) and the groundwater condition. The Hoek-Brown Geological Strength index was used because it looks at surface conditions of the discontinuities (joint modification and roughness) and rock structure (blockiness) of the rock mass which are significant parameters to consider to jointed rock mass. The Slope mass rating was used to determine more information on the behaviour of discontinuities and the possibilities of slope failure. Engineering classification is used to observe the karst ground condition in carbonaceous slopes.

3.4.2. Rock and Mineral Identification

Rock and mineral identification forms part of the rock material properties that are described from either hand specimen or tested in the laboratory (NRCS, 2012). Hand description involves the usage petrological properties to determine the name of the minerals in a rock. Laboratory description involves the usage of microscopes to determine the minerals. The properties under microscope include relief, colour, cleavage and cross polarization properties (*Interference colour, extinction and* twinning) (Cox *et al*., 1967).

In our study rock and mineral were collected to determine the lithology. Usage of hand specimen together with laboratory will help enhance the analysis of the lithology. Rock samples were collected from the face of the slope for each different rock unit found in the mine. Infilling material found in the discontinuity was also sampled to determine the infilling material.

The Nikon SMZ74ST electron microscope (Figure 3.1) was used for thin section and hand specimen analysis. This microscope belongs to Advanced Laboratory Solutions; a company based in Johannesburg, South Africa. A 2x lens will be used to analyse the samples with a 0.06mm scale from the microscope.

FIGURE 3.1: NIKON SMZ74ST ELECTRON MICROSCOPE.

CHAPTER 4: RESULTS

Chapter four reports the outcomes of the mapping done in the open-pit mine. The mapping includes using the rock mass classification tables and structural geology of the area. Slope stability is determined by using rock mass classification tables. Slope failure is determined by the structural geology data.

4.1. ROCK MASS CLASSIFICATION TABLES

4.1.1. ROCK MASS RATING (RMR)

In using the Rock Mass Rating (RMR) table, the recorded fieldwork is highlighted in red (Appendix Table S.1a, b, c and d). A summarised results table is presented below (Table 4.1) from the application of RMR on site. The application of the RMR Table Section A gives a description of the rock mass strength which focus on firstly the strength of intact rock material (1) and secondly the core quality (2) from drilled Rock Quality Design (RQD). The third (3) and fourth (4) aspect is the discontinuity description which one must use information from Section E. The last aspect (5) is the groundwater condition. These five aspects give information on the rock mass condition. Section B from the RMR table gives a description of the discontinuity orientation. Information from section A and B determines the rock mass classes (Section C). Section D describes the meaning of rock class using information from section C together with the cohesion and frictional angle of rock mass. Section E provides more information on the discontinuity conditions and section F provides information of the discontinuity orientation in a tunnel thus section F will not be applicable for this study.

Table 4.1: Summarised Rock Mass Rating.

Site	**A- Classification parameters and their rating**					**B-Discontinuity orientation**	**C-Rock mass class**	**D-Meaning of rock mass class**
	1	2	3	4	5			
Northern	12	3	15	25	0	-5	fair rock	c- 200 – 300 Φ-25-35
Western	12	3	15	25	0	-5	fair rock	c- 200 – 300 Φ-25-35
Southern	12	3	10	25	0	-5	fair rock	c- 200 – 300 Φ-25-35
Eastern	12	3	10	25	0	-5	fair rock	c- 200 – 300 Φ-25-35

4.1.1.1. Strength of the unharmed rock material and RQD

The overall strength of the unexcavated limestone in the pit (Table 4.1 section A, 1) is found to be rating of 12 which means strong rock. The unexcavated rock samples are tested (see Table 2.3) where the rock needs a lot of hammering from the geological hammer for it to breakdown. However, looking at the overall rock mass which is affected by discontinuities the overall drill core quality has a rating of 3 which means less than 25% (Table 4.1 section A, 2). A 25% drill core quality means poor quality (Appendix Table S.1A, B, C and D, section A, 2).

4.1.1.2. Discontinuity spacing

The discontinuity spacing differs but mostly within 0.6-2 m (Table 4.1, section A, 3) for the Northern and Western side of the pit. It is noted that on the Southern and Eastern side of the pit, the discontinuity spacing varies from 200-600 mm. Thus, the Northern and Western side of the pit have wider spacing of discontinuities as opposed to the Southern and Eastern part of the pit. This means the stability on the Southern and Eastern side of the pit are influenced by structure control.

4.1.1.3. Discontinuity condition

The discontinuity orientation varied across the pit. Discontinuities found in the pit consists of faults, joints, bedding planes and veins. The length of the faults varied compared to the joints or bedding planes observed on the pit. Faulted strata are the longest discontinuity observed in the pit. It is visible from the Northern, Western and Southern side of the pit. On the Western part of the mine pit (Figure 4.1 a) a major fault is observed transecting across two benches. The faulting on the western part of the open-pit was mainly dipping to the North and striking in an East-West direction (Figure 4.1). The thickness of the fault zone varied from 0.3-0.8 m. On the Northern part of the pit (Figure 4.1 b), two faults have been found intersecting each other with thickness varying from 0.3-0.8 m.

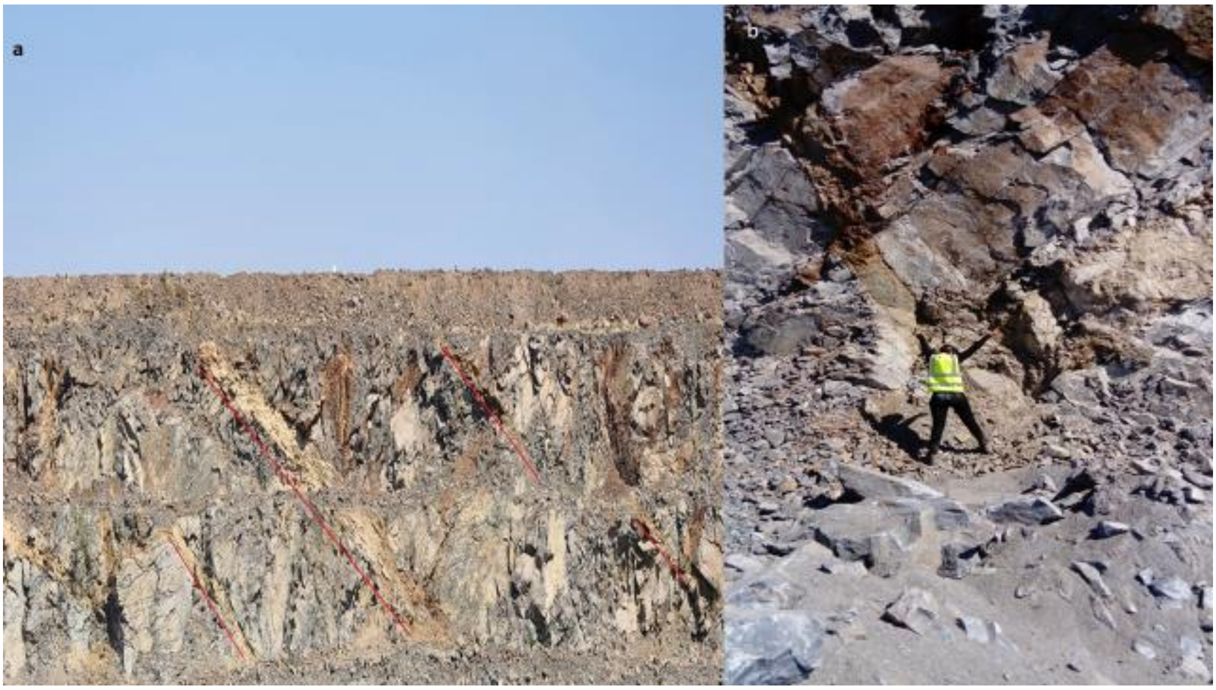


FIGURE 4.1: FAULTING OBSERVED ON THE (A) WESTERN SLOPE FACE AND (B) NORTHERN SLOPE FACE.

Joints observed in the pit were mainly dipping in the same direction from all sides of the pit. The joints were vertical joints, dipping mostly between 80-90° (Figure 4.2). The joints were mostly clean with smooth surface (Figure 4.2 b). Some joints cut through the limestone in such a way that it may be mistaken for the slope face (Figure 4.2 a and c). Most of the openly cut joints exposed the limestone to atmospheric agents which enabled the weathering of these slopes.

FIGURE 4.2: JOINT ORIENTATIONS FOUND AT THE OPEN-PIT MINE. (A) OPEN JOINTS THAT LOOK LIKE SLOPE FACE ON THE WESTERN PART OF THE OPEN MINE PIT, (B) 3 SET OF JOINTS ON THE WESTERN PART OF THE OPEN MINE PIT AND (C) OPEN AND CLEAN JOINTS ON THE NORTHERN SIDE OF THE PIT.

Veins filled with calcite have been observed throughout the pit. The veins have no preferential direction or orientation. In some rock units you find two or three sets of veins with calcite infilling. Figure 4.3 and 4.4 shows the veins infilling with calcite observed at the pit. In some veins, the infilling material (calcite) is subjected to weathering at the contact between vein and rock unit (Figure 4.3 b).

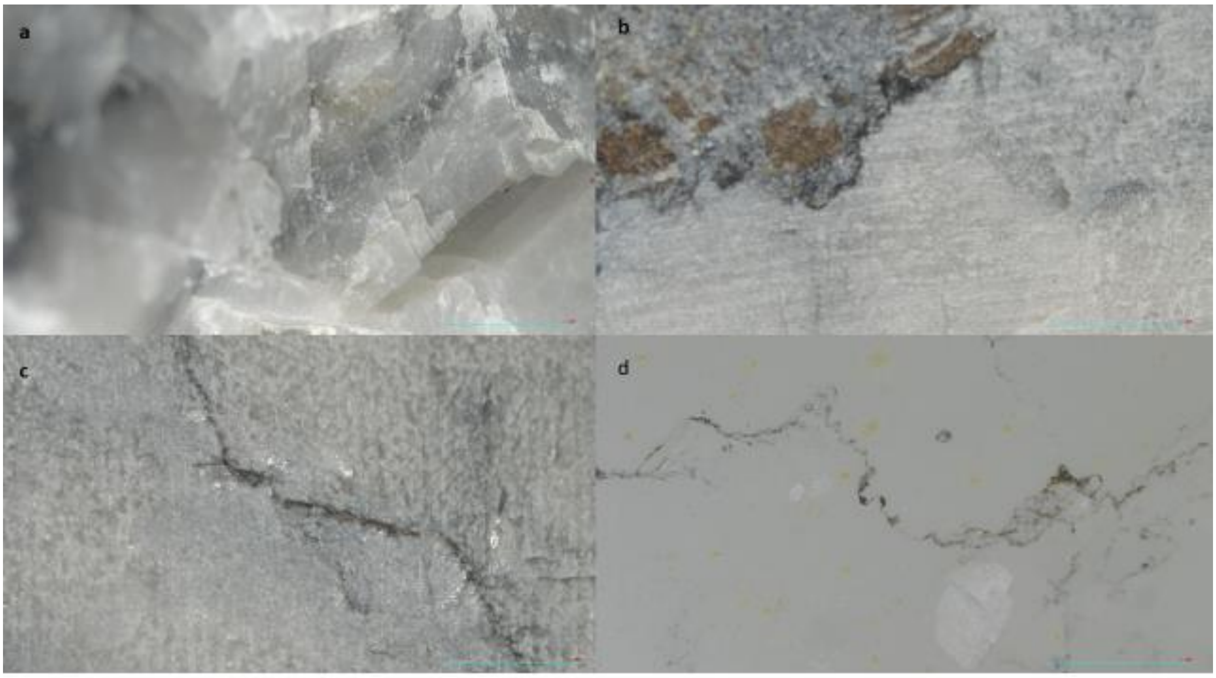


FIGURE 4.3: CALCITE VEIN. (A) HAND SPECIMEN OF CALCITE VEIN, (B) HAND SPECIMEN OF CALCITE VEIN WITH WEATHERING ON THE CONTACT LIMESTONE CONTACT, (C) HAND SPECIMEN PF CALCITE VEIN WITH MINOR JOINT AND (D) THIN SECTION OF CALCITE VEIN WITH MINOR JOINT.

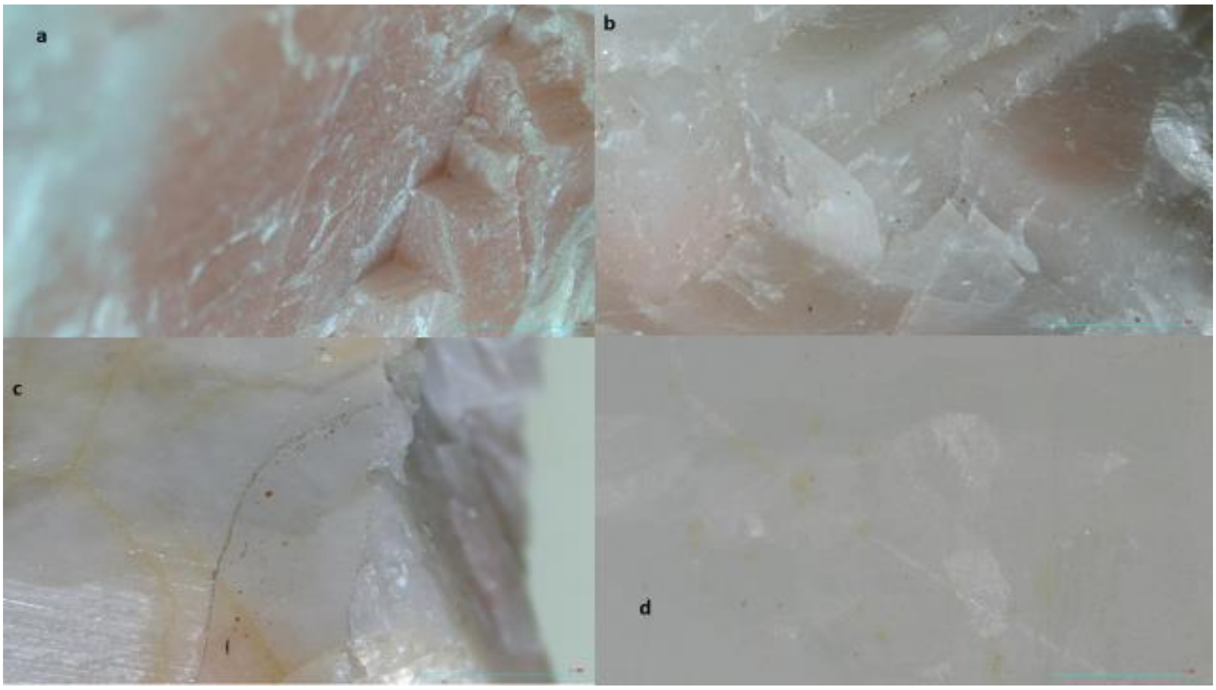


FIGURE 4.4: CALCITE VEIN WITH PINK IMPURITY FOUND ON THE NORTHERN PART OF THE OPEN MINE PIT. (A) HAND SPECIMEN OF CALCITE VEIN WITH PINK IMPURITY, (B) HAND SPECIMEN OF CALCITE VEIN WITH CLEAVAGE, (C) HAND SPECIMEN OF CALCITE VEIN WITH MINOR JOINTS AND (D) THIN SECTION OF CALCITE VEIN WITH MINOR JOINTS.

The fault zone on both the western and northern sides of the pit consists of soft infilling material that one can break with only fingernails or crumbling through your hand. According to Mohs scale, samples that one can be broken with fingernails or crumbled through the hand have a scale value of 2.5. Minerals that have a Mohs scale hardness value of 2.5 falls between that of gypsum and calcite. When viewed under a microscope, samples from the fault zone displayed instances of weathered calcite (Figure 4.5).

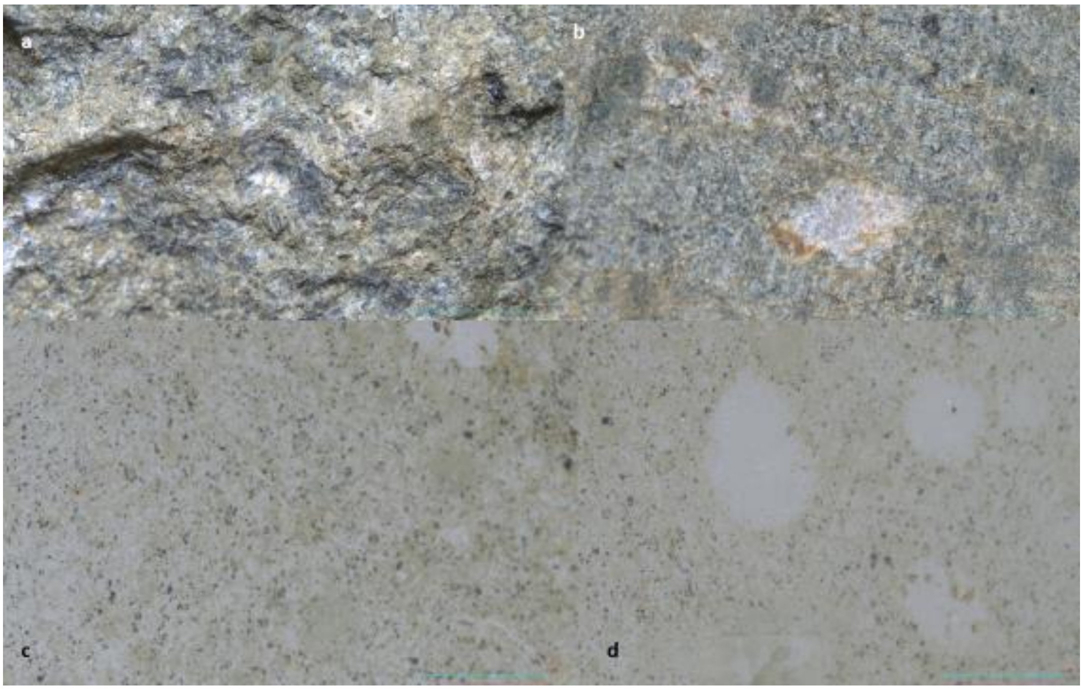


FIGURE 4.5: WEATHERED CALCITE UNDER MICROSCOPE. (A) HAND SPECIMEN UNDER MICROSCOPE, (B) HAND SPECIMEN WITH INCLUSION UNDER MICROSCOPE 2X LENS, (C) THIN SECTION UNDER MICROSCOPE AND (D) THIN SECTION WITH INCLUSION UNDER MICROSCOPE.

Guidelines for discontinuity condition was from Table 2.1, section E. The discontinuity length (persistence) varied with joints being less than 1 m and faults with 1-2 m thus persistence rating score was between 6 and 4. The separation from each discontinuity also varied less than 0.1 mm to more than 5 mm thus rating score was from 0-5. The roughness of discontinuities was mainly rough to slightly rough thus rating score of 3-5. The infilling material found within the discontinuities was in some no infilling and in some soft material thus rating score was 0 – 6. The weathering observed within the discontinuities walls was in some fresh material and in some slightly weathered. Thus, the total rating score was within 23-25 which meant our discontinuity condition was slightly rough surface with separation less than mm and discontinuity walls slightly weathered.

4.1.1.4. Groundwater condition

The groundwater condition in the pit gave a rating of zero with flowing water condition. The flowing groundwater condition is observed from the visible water-table in Figure 4.6. Groundwater is usually the main provenance of water distribution to rural and agricultural societies. During the visit to the mine on 29 August 2022, water from the water-table had risen to a point where no mining activity could take place (Figure 4.6) and the mine had to be closed temporarily while the water was pumped out.

FIGURE 4.6: CURRENT GROUNDWATER LEVEL AT THE PIT (29 AUGUST 2022).

4.1.2. Slope Orientation

The orientation of the slope is perpendicular to the rock mass. Thus, the overall score for RMR in the pit is 40-60 which means a fair rock. According to Section C from the RMR tables, a fair rock has average stand up time of one week for 5 m span. This means rock mass may stand without failure for short period (a week) but where failure occurs support will be required.

4.1.3. Geological Strength Index (GSI)

The fieldwork for GSI was only done on the Northern, Western and some of the Southern part of the pit due to rock mass condition of the slope (Figure 2.8). Figure 2.8 determines the kind of rock mass condition in which GSI can be applied. The Eastern side of the pit did not meet the requirements for GSI application thus any GSI discussed in the study excludes the Eastern part of the open-pit mine. The observations of the slope rock mass condition are highlighted in red from Appendix Table S.2 A, B, and C. A summarised results table presented below (Table 4.2) for the application of GSI on site. According to Table 4.2 the structure of the pit on all three sides was found to be very blocky. A very blocky structure means the discontinuities are interlocked and the rock mass is partially disturbed with multi-faceted angular blocks formed by 4 or more joint sets. The surface condition for Northern side of the pit was found to be good while the Southern and Western side of the pit being fair. A good surface condition according to GSI means a rough surface with slightly weathered and iron stained surfaces. Meanwhile a fair surface condition means a smooth surface with moderately weathered and altered surfaces. The GSI recording for the three pit sides gave a score of 40–60 indicating a very blocky rock structure with a good – fair surface condition.

Table 4.2: Summarised GSI results from the pit.

Site	Structure	Surface condition	GSI value
Northern	very blocky	good	60
Southern	very blocky	fair	40 - 50
Western	very blocky	fair	40 - 50

4.1.4. Geological Strength Index for Limestone

The application of GSI for limestone was also only applied on the Northern, Western and some of the Southern part of the pit due to rock mass condition of the slope (Figure 2.8). The recorded fieldwork done is highlighted in red in Appendix Table S.3 A, B and C. A summarised table in Table 4.3. According to Table 4.3 the structure of the pit on all three sides was found to be very blocky. A very blocky structure means well interlocked mass is with multi-faceted angular blocks formed by 4 or more joint sets. The surface condition for all three sides of the pit was found to be good. A good surface condition according to GSI Limestone means a rough surface with slightly weathered and iron stained surfaces. The GSI recording for the three pit sides gave a score of 50–60 indicating a very blocky rock structure with a good surface condition.

Table 4.3: Summarised GSI limestone results from the pit.

Site	Structure	Surface condition	GSI value
Northern	very blocky	good	50 - 60
Southern	very blocky	good	50 - 60
Western	very blocky	good	50 - 60

4.1.5. Slope Mass Rating

The Western side of the pit (Figure 4.1 a) was used to determine SMR as it was the only safe side available to take reading on the face. The recorded values from the pit are highlighted in red from appendix Table 4.1(RMR has been previously calculated from Appendix Table S.1 A, B, C and D and gave a total value of 42 – 47). The description of the results are given in Table 4.4 with red highlighted results for our study site.

Table 4.4: SMR outcomes collected at the study site (Romana, 1985).

	Type of failure	***Very favourable***	***Favourable***	***Normal***	***Unfavourable***	***Very unfavourable***
A	*P* *| αj−αs|*	***>30°***	*30–20°*	*20–10°*	*10–5°*	*5°*
	T *|αj−αs−180|*					
	W |αi−αs					
	F1	***0.15***	*0.40*	*0.70*	*0.85*	*1.00*
B	*P/W* *|βj|ó|βi|*	*20°*	*20–30°*	*30–35°*	***35–45°***	*>45°*
	F2	*0.15*	*0.40*	*0.70*	***0.85***	*1.00*
C	*P* *βj−βs*	***>10°***	*10–0°*	*0°*	*0–(−10°)*	*(−10°)*
	P/T/W *F3*	***0***	*−6*	*−25*	*−50*	*−60*
P: planar failure; T: toppling failure; W: wedge failure. αj: dip direction of the discontinuity; αs: dip direction of the slope; αi: dip direction of the intersection line of two sets of discontinuities; βj: discontinuity dip; βi: angle of plunge of the intersection line of two sets of discontinuities; βs: slope dip.						

TABLE 4.5: DESCRIPTION OF SMR CLASSES (ROMANA, 1985).

Classes □	**V**	**IV**	**III**	**II**	**I**
SMR	0-20	21-40	**41-60**	61-80	81-100
Description	Very bad	Bad	**Normal**	Good	Very good
Stability	Completely unstable	Unstable	**Partially stable**	Stable	Completely stable
Failures	Big planar or soil like or circular failure	planar or big wedge	**planar along some joints and many wedge failure**	some block failure	None
Failure probability	0.9	0.6	**0.4**	0.2	0

4.1.6. Engineering Classification of Karst Ground

The engineering classification of karst ground is used in the study area. Evidence of a dissolution sinkhole (Figure 4.7) and buried sinkholes were observed (Figure 4.8). Dissolution sinkholes occur due to joints being filled by soil matrix at toe on Figure 4.7. The fissures were filled with soil which resulted in a rock mass being more of soil texture instead of intact rock mass. Minor collapse is seen at the toe. Buried sinkhole occur due to soil cover from bench top filling up the cavities.

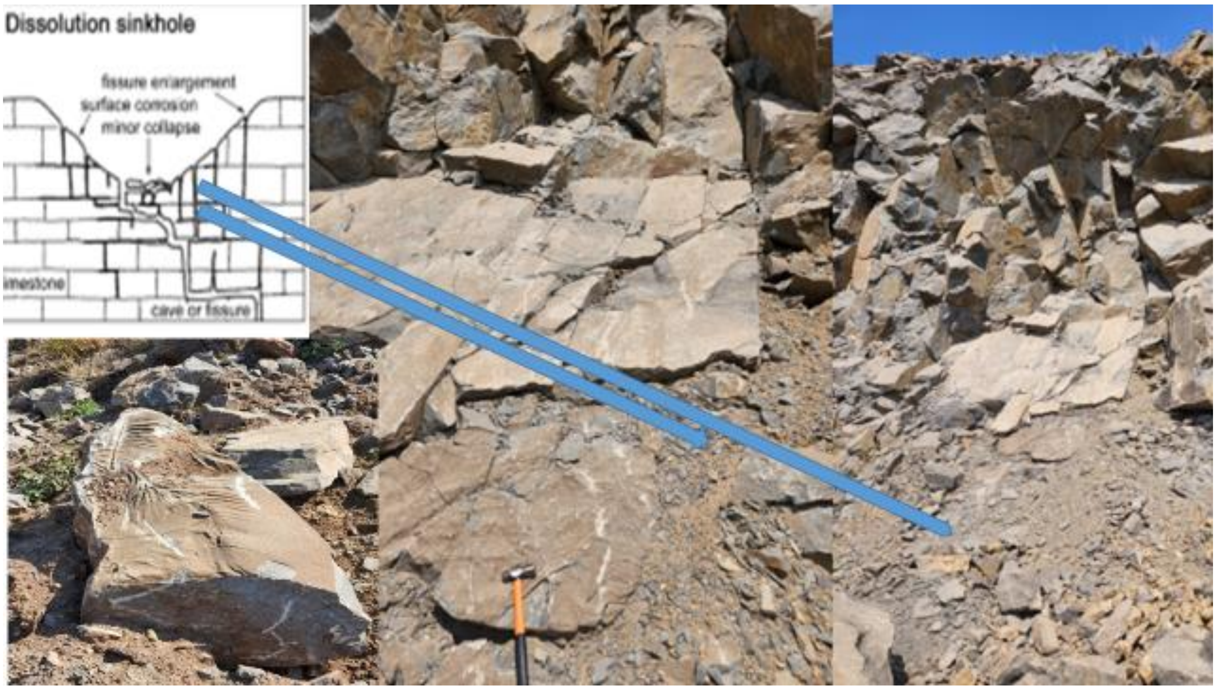


FIGURE 4.7: DISSOLUTION SINKHOLE ON THE WESTERN SIDE OF THE PIT.

FIGURE 4.8: BURIED SINKHOLE ON THE NORTHERN SIDE OF THE PIT WITH SOIL COVER FILLING UP THE CAVITIES.

4.2. Structural Geology

The structural geological datum is collected by means of scanline mapping. A measuring tape was stretched alongside the slope face and all discontinuity that traverses on the streak was mapped. The most often found discontinuities at the mine were faults, joints, veins and bedding planes (Table 4.6).

The limestone bedding plane found on the southern (Figure 4.9 b) and southern side of the pit is dipping in the direction of the West while on the eastern (Figure 4.9 a) and western side of the pit it is dipping toward the South (Figure 4.8). Limestone observed in the pit is associated with lamination, in some areas thick lamination is present mostly on the northern part of the open mine pit (Figure 4.9 c). While thin lamination is observed on the western part of the open mine pit (Figure 4.9 d). The bedding planes are mostly clean with loose soil infilling. On the eastern part of the open-pit mine, bedding planes had soil matrix infilling (Figure 4.10).

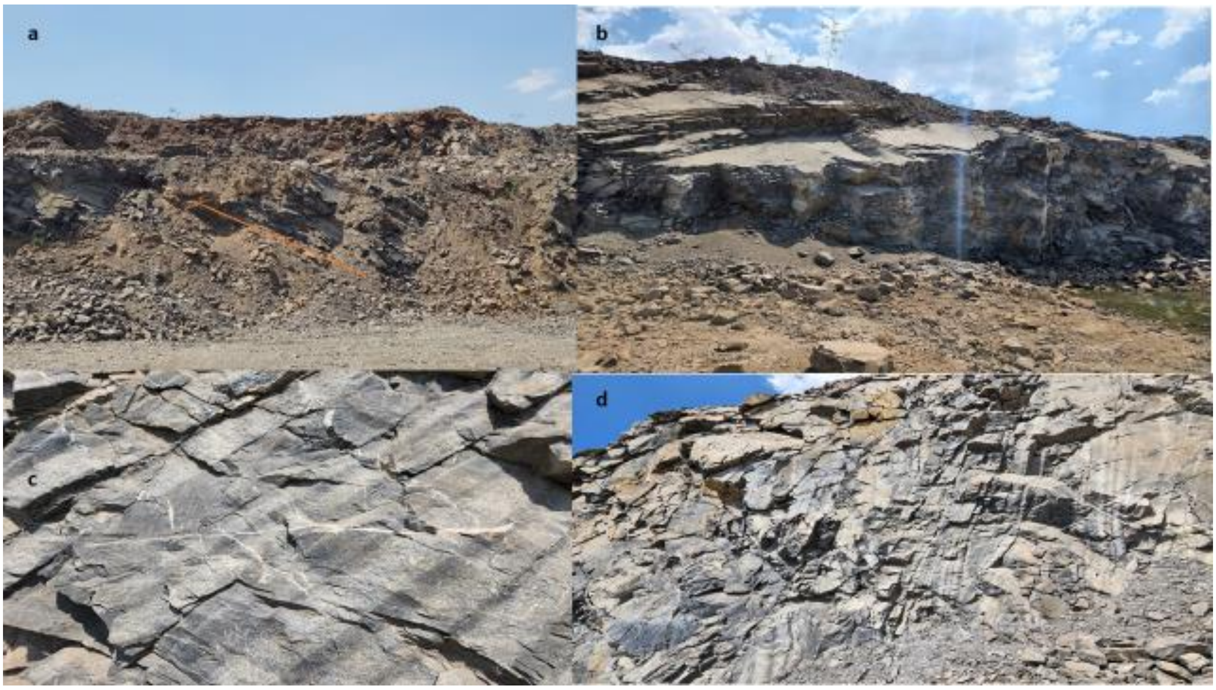


FIGURE 4.9: BEDDING ORIENTATION AT THE OPEN MINING PIT. (A) BEDDING ON THE EAST WITH SMALL PLANT (0.4M TALL) ON THE TOP BENCH USED AS SCALE, (B) THICK LAMINATION ON THE NORTH WITH SMALL PLANT (0.4M TALL) ON THE TOP BENCH USED AS SCALE, (C) BEDDING ON THE NORTH AND (D) THIN LAMINATION ON THE WEST.

TABLE 4.6: RESULTS FROM SCANLINE MAPPING.

Station	Type	Dip	Strike	Persistence and termination	Aperture/ width	Nature of filling	Surface roughness	Surface shape	Water flow	Degree of weathering	Comments
North	Bedding	West	North-South	Very high - neither end visible	Very wide	Clean with a bit of soil sipping in	Smooth	Stepped	Bedding is dry	Fresh	Minor lamination, evenly spaced (0.4-0.6 m), veins infilling
North	Fault	West	North	Very high - neither end visible	Very wide	Calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.4m thickness
North	Joints	80-90	East-West	Very high - neither end visible	Thin opening	Clean	Smooth	Planar	Joint is dry	Fresh	Clean joints
North	Fault	South	East-West	Very high - neither end visible	Very wide	Calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.2 m thickness
North	Bedding	West	North-South	Very high - neither end visible	Very wide	Clean with a bit of soil sipping in	Smooth	Stepped	Bedding is dry	Fresh	Heavily laminated strata, veins infilling
North	Joints	80-90	East-West	Very high - neither end visible	Open	Clean	Smooth	Planar	Joint is dry	Moderate	When joint is open rusty/weathered surface is observed
West	Bedding	South	East-West	Very high - neither end visible	Very wide	Clean	Smooth	Stepped	Bedding is dry	Fresh	Minor lamination

West	Fault	North	East-West	Very high - neither end visible	Very wide	Weathered calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.4 m thickness
West	Joints	80-90	North-South	Very high - neither end visible	Thin opening	Clean	Smooth	Planar	Joint is dry	Fresh	Clean joint
West	Bedding	South	East-West	Very high - neither end visible	Very wide	Clean	Smooth	Stepped	Bedding is dry	Fresh	little plant growing on the bedding
West	Fault	North	East-west	Very high - neither end visible	Very wide	Weathered calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.6 m thickness
West	Joints	80-90	North-South	Very high - neither end visible	Open	Clean	Smooth	Planar	Joint is dry	Moderate	When joint is open rusty/weathered surface is observed
West	Bedding	South	East-west	Very high - neither end visible	Very wide	Clean	Smooth	Stepped	Bedding is dry	Fresh	Minor lamination, veins infilling
West	Fault	North	East-west	Very high - neither end visible	Very wide	Weathered calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.8 m thickness
West	Joints	80-90	North-South	Very high - neither end visible	Thin opening	Clean	Smooth	Planar	Joint is dry	Fresh	Clean joint
West	Joints	80-90	North-South	Very high - neither end visible	Open	Clean	Smooth	Planar	Joint is dry	Moderate	When joint is open rusty/weathered surface is observed

West	Bedding	South	East-west	Very high - neither end visible	Very wide	Clean	Smooth	Stepped	Bedding is dry	Fresh	Minor lamination, veins infilling
West	Fault	North	East-west	Very high - neither end visible	Very wide	Weathered calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.8 m thickness
West	Joints	80-90	North-South	Very high - neither end visible	Thin opening	Clean	Smooth	Planar	Joint is dry	Fresh	Clean joint
South	Bedding	West	North-South	Very high - neither end visible	Very wide	Clean with a bit of soil sipping in	Smooth	Stepped	Bedding is dry	Fresh	Veins infilling
South	Fault	80-90	East-West	Very high - neither end visible	Very wide	Calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.4 m thickness
South	Joints	80-90	East-West	Very high - neither end visible	Thin opening	Clean	Smooth	Planar	Joint is dry	Fresh	clean joint
South	Bedding	West	North-South	Very high - neither end visible	Very wide	Clean with a bit of soil sipping in	Smooth	Stepped	Bedding is dry	Fresh	Veins infilling
South	Fault	west	North-South	Very high - neither end visible	Very wide	Calcite	Smooth	Stepped	Fault zone is dry	Moderate	0.4 m thickness
South	Joints	80-90	East-West	Very high - neither end visible	Thin opening	Clean	Smooth	Planar	Joint is dry	Fresh	Clean joint

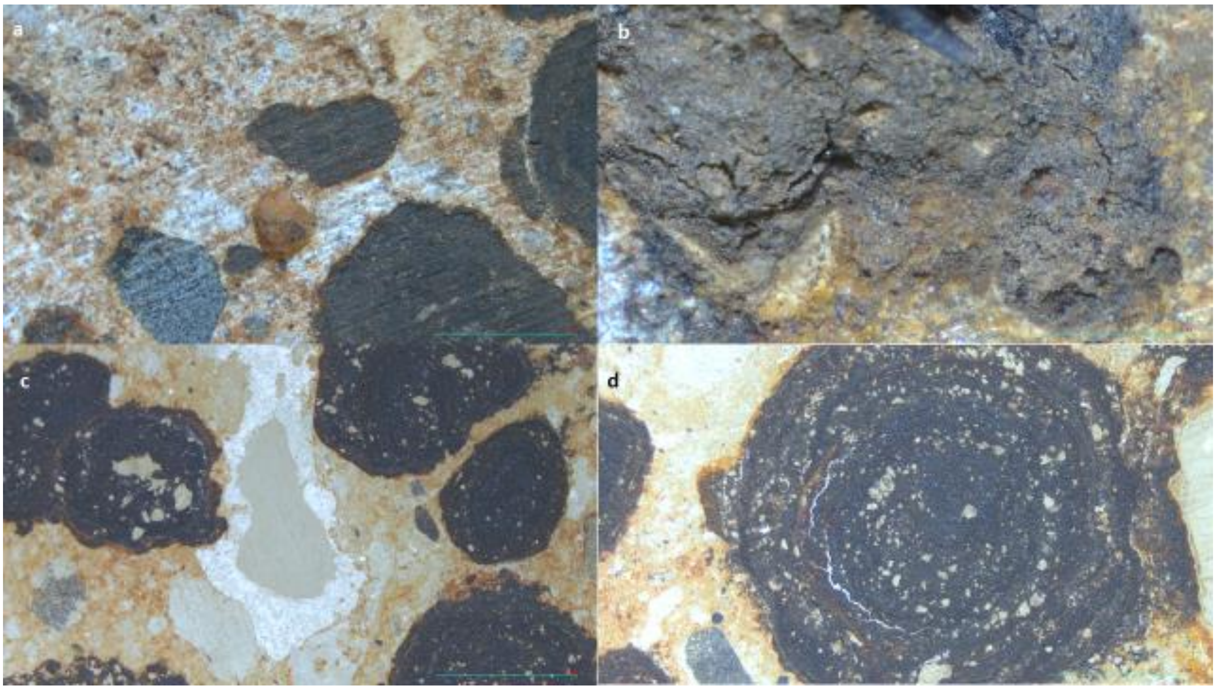


FIGURE 4.10: SUBSTRATE MATRIX FOUND ON THE EASTERN PART OF THE OPEN MINE PIT. (A) HAND SPECIMEN OF SOIL MATRIX, (B) HAND SPECIMEN OF SOIL MATRIX ON 3X MAGNIFYING LENS, (C) THIN SECTION OF SOIL MATRIX AND (D) THIN SECTION OF SOIL MATRIX.

CHAPTER 5: DISCUSSION

5.1. ROCK CLASSIFICATION SCHEMES

The initial purpose of the study was to use rock mass classification tables to determine the shear strength parameters for slope stability analysis in surface limestone mining. The shear strength parameters are usually determined from rock mass classification, limit equilibrium, kinematic analysis and numerical modelling. In this study the rock mass classification system was applied to determine the shear strength parameters. The rock mass classification systems chosen were Bieniawski's Rock Mass Rating, Geological Strength Index, Romana Slope Mass Rating and the Engineering Classification of karst ground conditions.

5.1.1. BIENIAWSKI'S ROCK MASS RATING (RMR)

The RMR classification has been used widely in different types of engineering ventures such as mines, foundations, tunnels, and mines but little in slopes. As RMR was developed in 1973 but has gone through changes and revision in 1974, 1976, 1979 and 1989. In this study the 1989 version was used. Originally 49 historical cases were used in the evolution and authentication of the RMR in 1973, accompanied by 62 coal mining cases (historical cases that were included in 1984) and an additional 78 historical tunnelling and mining cases gathered by 1987. In 1989, the RMR system had been applied in 351 historical cases (Bieniawski, 1989). Celada *et al*. (2014) improved RMR_{89} to RMR_{14} which can be correlated using equation 5.1. In this study the RMR_{89} was used.

$$RMR_{14} = 1.1\ RMR_{89} + 2 \qquad \text{EQ 5.1}$$

The tools applied to get the RMR value was the published strength values from point loader, tapes for measuring spacing from discontinuities and to determine RQD. The groundwater values were observed from visual view. The limestone at the site was heavily jointed thus making it different to get intact samples for triaxial testing thus the use of Table 2.3 became helpful as Hoek had already tested similar samples and published values. In using the RMR table on the open-pit mine gave a score of 42-47 which according to Section C from the RMR tables (Table 4.1.) and Table S.1 A, B, C and D from appendix means a ‘fair rock with average stand up time of 1 week for 5m span.” In Section C a fair rock mass has cohesion of between 0.2-0.3 KPA while the angle of internal friction changes from 25 to 35°. Apart from using RMR table for stability analysis one could further get internal frictional angle and cohesion constraints values from this classification system. Cohesion is mainly used to detect the connector between rock particles within a rock. The angle of internal friction is applied to observe the internal friction produced along the shear surface. Cohesion and angle of internal friction are one of the two important mechanical strength constraints applied to describe the material shear strength (Shahani *et al*., 2022). In mechanical strength cohesion and angle of internal friction help determine the deformation and stability of the rock mass. One is able to obtain cohesion and angle of internal friction from direct shear strength test such as triaxial test. The challenge with using triaxial test is that it is a lengthily process and expensive. Also samples required for triaxial testing are problematic in a highly jointed mass (Shahani *et al*., 2022). The rock mass becomes too fragile to get intact sample without any discontinuity. Comparing the RMR cohesion and internal frictional angle to GSI plotted curves (Figure 2.1) one can observe the same cohesion and internal frictional angle values for our study area. Looking at the work done by Somodi *et al*., (2021) where the relationship between RMR and GSI from granite, siltstone, sandstone and quartzite, one can agree that the two rock mass classification schemes can be correlated.

A fair RMR means at the beginning stage of design and operation, it is wise to make use of the support recommended for fair rock. As the mining operation progresses smoothly for fair rock slope with no steadiness complications, and the support is acting out very well, then it ought to be possible to slowly lessen the support request. Also, if the digging is needed to be steady for a small duration, then it is sensible to consider the less costly and extensive support recommended for fair rock. But, if the rock mass encircling the digging is required to undertake huge mining induced pressure deviations, then additional considerable support suitable for fair rock should be placed. These available guidelines for RMR support structures are mostly used for tunnel constructions and underground conditions but little evidence for surface slopes.

Mehrotra (1992) conducted widespread block shear tests in order to determine the shear strength factors of rock mas using RMR. The results from the study gave the following:

1. The rock mass rating (RMR) scheme may be utilised to determine the shear strength constraints c and Φ of the weathered and waterlogged rock masses. It was detected that the cohesion (c) and the angle of internal friction Φ rise when RMR upsurges.
2. The consequence of waterlogged on shear strength constraints has been established to be noteworthy. For lowly saturated (wet) rock masses, an extreme lessening of 70% has been detected in cohesion (c), meanwhile the decline in angle of internal friction Φ is of the order of 35% when equated to those for the arid rock masses.
3. There is a non-linear discrepancy of the angle of internal friction with RMR for arid rock masses.

Looking at the results from RMR and Mehrotra testing one can agree that RMR may be utilised to define the rock mass strength properties. The stronger the rock to resist failure (RMR Table) the more strength it has thus c and Φ increases (Mehrotra, 1992). A saturated rock mass will reduce cohesion due to the chemical reaction that may take place and result in a weak rock. In carbonaceous rocks (equation 1.1 and 1.2) we have seen chemical reaction does result in dissolution of calcium carbonate. In our study area we have seen evidence of dissolution. The foot of the pit is currently filled with water from the water table rise, once pumping is done a reduction in cohesion is expected and research on the condition of limestone will be needed.

Considering the challenge mine inspectors face when inspecting the environmental condition of a mine, one may consider using RMR as a tool to examine the safety condition of the mine as it looks at important aspects of slope stability. The inspectors may require the RMR tables filed as per the Department of Minerals and Energy requirement and use as an easy background information.

5.1.2. Geological Strength Index

The GSI system was applied as the second rock mass classification in the pit. The GSI is useful geotechnical classification for complex and weak rock masses. It is a useful scheme to apply to jointed rock masses where laboratory samples are not demonstrative of the rock mass owing to the disruption, jointing and the multiplicity of most developments (Marinos, 2010). The system is utilised to determine the constraints ‘s’, ‘a’ and ‘mb’ in the Hoek-Brown criterion, applying empirical equations (Verma, 2011). The GSI system was presented to master the deficits in RMR for below par standard rock masses (Verma, 2011). The deficits of RMR are structure (blockiness) and the state of the joints which are two aspects deliberated to have imperative impact on the mechanical properties of a rock mass (Hoek & Brown, 2018). The GSI places more stress on the simple geological surveillance of rock mass appearances, imitate the material, its structure and its geological history and would be established explicitly for the estimation of rock mass properties (Marinos *et al*., 2004).

In our study area we have blocky jointed rock mass (Appendix Table 4.2 and Table 4.2 A, B, and C from appendix) thus usage of GSI is acceptable. The GSI-system gave a score of 40-60 which means a very block interlocked, moderately disturbed mass with multi-faceted angular blocks formed by four or more sets of joints. Using the published curve plots for cohesion strength and angle of internal friction from Hoek *et al.* (1998) a GSI score of 40 correlates with a cohesion value of 0.03 and the friction angle is roughly 26° (Figure 2.1). A GSI score of 60 correlates with a cohesion value of 0.048 where the degree angle is roughly 31° (Figure 2.1). Values for s’, ‘a’ and ‘mb’ can be calculated using the formulas below or using the published GSI values from Hoek *et al*. (1995).

$$S = \exp\frac{GSI-100}{9} \quad \text{EQ 5.1}$$

$$a = 0.65\frac{GSI}{200} \quad \text{EQ 5.2}$$

$$mb = mi\exp\frac{GSI-100}{28} \quad \text{EQ 5.3}$$

The GSI-system gives details of the freedom of interlocking angular blocks which play significant role in slope failure process where the block may slide or rotate without a great deal of intact rock failure. The liberty is governed by the geometrical form of the intact rock fragments as well as the state of the surfaces diving the pieces. Unlike Hoek-Brown failure criterion which presume that intact rock is at liberty from defects other than microcracks and flow (Hoek & Brown, 2018). In handling the tectonically fragmentary rock mass from our study area the GSI system is adequate for predicting the rock strength and observe the potential slope failure. Apart from the strength of intact rock it is also imperative to define the deformation modulus of the rock mass on a slope. In GSI system data one is able to define the deformation modulus by using the equation below or published values from Hoek *et al*. (1995).

$$Em = \frac{\sqrt{\sigma ci}}{100}.10\frac{(GSI-10)}{40} \quad \text{EQ 5.4}$$

The RMR_{R4} which was developed by Caleb *et al*. (2014) can also be used to correlate GSI (Zhang *et al.*, 2019). The correlation was done from the use of tunnel datum and can be used to determine GSI value from RMR_{14}.

The GSI limestone gave a value of 50-60 (Table 4.3) with a description of very blocky limestone, well interlocked mass with multi-faceted angular blocks formed by 4 or more joints. Comparing to Table 4.2 using both GSI systems common description is “very blocky rock mass with multi-faceted angular blocks.”

Durability of a rock mass such as carbonaceous rocks changes over time due to the presence and fluctuations of both water and temperature (Wyllie & Mah, 2004). In carbonaceous rocks, the strength parameters that are likely to be affected by the presence of water and temperature are cohesion and frictional angle (Wyllie & Mah, 2004). According to Onyango *et al*. (2022), laboratory testing on spongy (porous) rock have revealed that intact rock strength lessens with a rock of higher porosity percentage. Porosity in a rock act as podium for fluids to leach in and be absorbed rock. The benefit of porosity in a rock is to performance as source for water concentration and storage however the challenge comes when water reacts with minerals within the rock thus affecting durability of the rock mass. According Walthma (2001), when water reacts with limestone it can dissolve away the rock especially if it seeps down the cracks.

5.1.3. Slope Mass Rating

The third rock mass classification system utilised at the pit was the slope mass rating. The slope mass rating is a commonly used tool to classify slopes and understand the rock mass behaviour of slopes in surface mines (Verma, 2011). The Romana slope mass rating gave a score of 50-58 which means it's a stable slope with partial stability according to Table 4.6. The second part of Table 2.8 looks at the slope support guidelines based on SMR values. According to the 50-58 rating from Table 4.6 in our field of study the suitable support stable slopes options one may consider are concrete (toe wall), protection (toe ditch) and reinforcement (bolts and anchors). Looking at the current bench conditions at the mine pit, toe wall and toe ditch (protection) is most likely to be used. Discontinuity datum collected for slope mass rating can be applied in kinematic slope stability using stereonet plots (Verma, 2011). Kinematic slope stability is an easy tool to analyse the planar and wedge failure in a rock slope (Verma, 2011). The structural data is geometrically plotted in an equal are net to establish the mode and probability of failure.

5.1.4. Engineering Classification of Karst Ground

The Engineering classification of karst ground conditions was utilised as the last rock mass classification on the pit. The Engineering classification of karst ground conditions showed evidence of dissolution (Figure 4.7) and a buried sinkhole (Figure 4.8) from discontinuities filled with soil. The prime engineering danger is the descending leaching of soil into ancient and steady discontinuities to create slope failures. Karst is a unique topography formed on soluble rock with terrain associated to adequate underground drainage (Walton & Fookes, 2005). Karst forms mainly in limestone (and dolomites). Carbonaceous material is known to be associated with karst processes within the Transvaal Supergroup in South Africa. Limestone is a non-clastic carbonate rock (Strahler & Strahler, 1973) composed principally of calcium carbonate (calcite). The diverse karst landscapes connect to each other, but the local hydrological, geological and climatic conditions produce suites of karstic landscapes with just about endless variety (Ford & Williams, 1989).

The disintegration of calcium carbonate in water is predominantly dependant on the accessibility of biogenic carbon dioxide, which arises at the maximum absorptions in deep soils and in tropical regions where decay of organic matter is quick. Dissolution of rock takes place on visible outcrops, at the rockhead underneath soil, and alongside underground cracks (Walton & Fookes, 2005). A palaeontological study (Durand, 2013) was conducted in 2013 around our study area. The purpose of the research was to find any fossils, or any noteworthy stromatolites preserved in the limestone. The research produced evidence of dissolutional karstification on the surface (Durand, 2013). Thus, the mining of limestone in the study site requires engineers and scientists, accountable for slope stability, to also consider natural processes that may weaken the shear strength of the rock mass. As much as the limestone is strong and does not easily react with water, the study site is extremely fractured which will allow water to stay within the fracture openings over time. Occurrence of water within the limestone will eventually dissolve away the rock and leave cavities which will grow over time, and can it result to the cave formation.

5.2. Discontinuities Found Within Rock Masses

Deformability, strength and permeability factors of a rock-mass are intensely influenced by its discontinuities. The effect of discontinuities on the shear strength of a rock mass is dependent upon the roughness, variation of the fractures, the moisture content, the thickness of infillings or the gouge material, etc. A discontinuity set consists of a number of discontinuities that share many of the same mechanical characters or a single discontinuity. When the fractures are clean, the compressive strength and roughness of the joint walls are significant, while with filled fractures it is important to consider the mineralogical and physical properties of the gouge material dividing the joints wall. (Beale & Read, 2013). Mapping of discontinuity assist in assessing the weak planes in a rock mass. The weak planes orientation and frequency play both a positive and negative role in excavation. The negative role is the reduction in rock mass strength. The positive role is that it eases the excavation process for hard rock where less blasting material required.

The previous work done on the study site suggest intense deformed rocks from the Crocodile River fragment and are surrounded by Bushveld complex acidic and basic rocks (Walraven & Martini, 1995). The Crocodile River fragment has been subjected to refolding and faulting thus resulting in a complex geological structure. The complex geological structure can be seen from Figure 4.1 (faulting observed), Figure 4.2 (joints orientation), Figure 4.6 (groundwater condition showing the Western side of the pit with clear geological structures), Figure 4.7 (the dissolution sinkhole features seen from the geological structures filled with soil), Figure 4.9 (bedding orientation) and the scanline mapping from Table 4.6. The strength, deformability and permeability of this site will be affected by the complex geological structure for surrounding areas. Reduction in rock mass strength can be seen from rock mass rating tables (Appendix 4.1 A, B, C and D) under section A for intact rock strength. The rock compressive strength is relatively high but the more discontinuities have reduced the overall strength of the rock mass. Deformability can be seen through the dissolution sinkhole features (Figure 4.7) where the geological structures are filled with soil. Infilling of geological structures means greater opening of the structures. Apart from widening the geological structures but infilling material brings in chemical weathering where the infilling material reacts with environmental agents and host rock. Evidence is seen in the vein infilling material which has rusty texture indicating chemical weathering (Figure 4.3, Figure 4.4 and Figure 4.5). In South Africa 90% of aquifers are a result of fractured rock mass whereby the discontinuities result in network of voids that can allow groundwater to travel and be stored (Lin, *et.al.* 2015). Looking at the discontinuity condition on the pit Figure 4.1 (faulting observed), Figure 4.2 (joints orientation), Figure 4.6 (groundwater condition showing the Western side of the pit with clear geological structures), one may observe no water trapped within the discontinuities, or evidence of water movement however Figure 4.3 and Figure 4.5 shows weathering product within the discontinuities.

In order to comprehend the performance of jointed rock masses, it is needed to start with the mechanisms which work together to make up the system - the intact rock material and distinct discontinuity planes (Hoek, 1983). Intact rock speaks to the uninterrupted blocks which are found in-between structural discontinuities in a normal rock mass. These fragments may vary from a few millimetres to a number of metres in size and their performance is normally elastic and isotropic. Their failure can be categorised as brittle which suggests an instant decrease in strength when a restrictive stress level is exceeded. Depending on the orientation, number, and condition of the discontinuities, the intact rock fragments will rotate, crush or translate in response to stresses enforced upon the rock mass. The performance of discontinuities is influenced by the geometry and strength of discontinuities. As can be seen from our study site, the area has heavily jointed rock mass with the presences of faults, bedding plans and joints (Table 4.6). The faulting observed on site is one of the distinctive discontinuity structures which has control on the sliding and rotation of the rock mass.

The bedding planes are associated with soil normally on the upper benches of the pit while clean bedding plans on the lower benches. The occurrence of soil within the bedding plans act as an tool to further open up the bedding planes thus greater planes for movement (sliding) of rock mass. Joints are clean while some consist of calcite infilling.

Brekke & Howard (1972, in Hoek & Brown, 1980) issued seven clusters of discontinuity infillings or gouges that have important role on the engineering behaviour of rock masses.

1. Through rainfall from solutions of quartz or calcite, the joints, seams and seldom the minor faults can be repaired. In this situation, the discontinuity can be fused together. However, it is possible that these discontinuities may have re-fractured, creating new surfaces. It should

be noted that, quartz and calcite might exist in the discontinuity but may not always be repairing it.

2. Clean discontinuities are the one that do not have coatings or filings. The character of several of the rough joints or partings is favourable. Near the surface, it is crucial to differentiate between clean discontinuities and empty discontinuities that have had their infill material percolated or eroded away from surface weathering.
3. When calcite fillings are porous or flaky, they are more likely to dissolve throughout the course of an underground opening. Then, their contribution to the rock mass's strength is lost. This long-term stability (and occasionally fluid flow) issue is one that is easily missed during design and construction. Fillings made of gypsum might act similarly.
4. Talc, chlorite and graphite coatings or filings make a lot of slick joints, seams or flaws (which is low strength) as a result of the reduced cohesiveness, especially when wet.
5. Clay that is inactive and present in seams and flaws is a very weak substance that is prone to squeezing or washing out.
6. From free swelling and the results of loss of strength, or through notable increased swelling pressure when constrained by a tunnel lining, swelling clay gouge may result in serious issues.
7. After digging, material that has modified to become more cohesionless (sand-like) may run or flow into the tunnel

According to Brekke & Howard (1972) all seven groups of discontinuity infilling in our study area have been observed on the slope which then means it is imperative factor to scrutinize in slope stability. Goodman (1970) piloted a number of tests on joints with gouge material. The findings showed that shear strength decreased as filler material thickness increased, indicating that the filling material's strength controls the joint's strength once it exceeds the amplitude of surface projections. Considering the findings of Goodman (1970) results for the study area would mean poor strength of the joints filled with calcite (Figure 4.4 and 4.7), fault filled with weathered calcite (Figure 4.1 and 4.5) and soil matrix found in bedding planes (Figure 4.10). Observation of calcite veins have also been found on the pit (Figure 4.3 and 4.4).

5.3. Slope Failure

The angular particles or blocks of solid, brittle material that makes up the jointed rock mass are divided by discontinuity surfaces that may or may not be covered with weaker material (Hoek, 1983). The strength of such rock mass is only as strong as their unbroken portion is strong and as mobile as they can, which in turn depends on the direction, spacing, quantity and the shear strength of the discontinuity. Since there are a large number of possible groupings of block forms and dimensions, it is clearly required to find any behavioural tendencies which are mutual to all of these groupings. The formation of such mutual tendencies is the most significant to determine slope failure.

The configuration of the rock mass beyond the face of a slope is imperative. Slope failure is associated with collapse, topples, slumps or slide out of rocks from the face. The danger in slope failure in the mining environment is not only for safety but it has implications of forcing a mine to implement some sort of out-of-schedule evacuation of the pit which would then disturb production and will have a negative effect on revenue. Different from the other engineers, mining staff cannot pick their material, but they are required to be strategic and slice their slopes in the rocks and soils that are existing to entree the mineral resource that they are obligatory to abstract (Karparov & Handley, 2009).

A slope's sliding surface could be a single plane that runs through the entire length of the slope or it could be a complicated surface made up of discontinuities and cracks running through solid rock. The slope failure at the pit is structurally controlled with all possibilities of slope failure. Planar failure is possible through bedding planning on the Northern side of the pit. Planar happens when a block of rock glides on a single plane dipping out of the face and when bedding plane strike sub-parallel to the slope and dip out of the slope. Planar failures expected to be limited to benches and regions of the pit or quarry which have contrary configuration, or where there are liberate structures which strike across the slope face and therefore allowing a planar slide to follow (Stacey, 2001). In our study area the steeply dipping strata will allow for planar failure along the bedding plane. Wedge failure is possible through bedding plan being intersected by joints or fault (Figure 4.1). Topping failure is possible in slopes with plenty of joints. Rotational failure is possible mostly on the Eastern part of the open mine pit where it is mostly soil exposure (Figure 4.9 bedding on the East).

Looking at the slope failure that occurred at Sandsloot open-pit mine (platinum mine) a similar situation is expected for the limestone mine. The Sandsloot mine experienced several slope failures due to presence of set of joints (Table 1.1). Several scanline mapping and kinematic datum was collected to analyse the different slope failures (Bye & Bell, 2001). The failure zones identified were planar, wedge, toppling and circular failure zones, as well as a ''nose'' zone (Bye & Bell, 2001). The modes of failure were predominantly structurally controlled thus similar to the limestone mine being structurally controlled. Thus, to overcome the constant slope failure in the Sandsloot mine constant monitoring was implemented, and the slope design had to be tailored to suit the geological condition (Bye & Bell, 2001).

In the Letlhakane mine (Botswana) where slope failure was expected from slope monitoring systems (Kayesa, 2005). The mine was known to have problem of tension crack formation, crack widening and extension (Table 1.1) thus measures put in place to avoid exposing personnel and mining equipment (Kayesa, 2005). The monitoring systems included daily visual walkover inspection of the slope by geotechnical engineer, the use of Geomos survey technique and slope monitoring report. Thus, it was easier to work on safety measures.

5.4. The Impact of Water on the Region

Water in the mining industry is important required natural resource. Water in surface mining can be used to reduce air pollution by regular spraying of water from water tankers on the ground. It is also used in cooling off machines and washing of minerals during separation. The source of water can be from precipitation, recharge from adjacent rivers, groundwater, tailing dams, reservoirs etc. The study area is close to Crocodile River in the Limpopo Province of South Africa. The Crocodile River is surrounded by agriculture activity and a few mines.

The presence of ground water in a rock slope can have a detrimental effect upon stability. Wyllie & Mah (2004) summarised the effects of groundwater on slope stability as follows:

- Water pressure lessons the steadiness of the slopes by reducing the shear strength of probable failure surfaces. By boosting the forces that cause sliding, water pressure in tension cracks or other similar near vertical fissures decreases steadiness.
- Fluctuations in moisture content of some rocks can result in quicker weathering and a reduction in shear strength.
- Icing of ground water may result in wedging in water-filled cracks due to temperature reliant capacity deviations in the ice. Also, icing of surface water on slopes can obstruct drainage pathways causing in an accumulation of water pressure in the slope with a consequential decrease in stability.
- Where the toe of a slope is undermined or a block of rock is loosed, erosion of weathered rock by surface water and of low strength infillings by ground water may cause local unsteadiness.

The flow pattern of groundwater within a slope is governed by the water repository within the slope, configuration of the slope, recharge from the surrounding rock mass, permeability characteristics of the slope material, etc. (Beale & Read, 2013). The greatest significant impact of groundwater in a rock mass is the weakening of stability due to water pressure within the discontinuities (Beale & Read, 2013). Usually, a slope with no water seepage on the slope face is mistakenly to assume that groundwater is not present. Geotechnical data collection during site investigations typically takes place over a relatively short period of time and often during good weather conditions. Research on the long-term observation of the rock mass strength has received relatively little attention (Andriani & Parise, 2017). Hard rock is normally considered to be resistant to deterioration over short time periods (i.e. the lifespan of a surface mine) (Mišcevic & Vlastelica, 2014) but does the same apply to surface mining operations in carbonaceous rock?

During the early stages of mining no water-table was intersected on the pit (Figure 1.3). However, in 2020 the mine intersected the water-table which affected the mining operations at the pit (Figure 4.6). Currently water is being pumped out daily at the pit.

Water has the ability to percolate and recharge the ground water-table but, in most cases, it causes transient water pressure within the discontinuities that weakens shear strength. Permeability (hydraulic conductivity) is a critical parameter used to determine the flow of groundwater and the dispersal of water pressure. The flow of groundwater through intact rock is known as primary permeability and when groundwater flows through discontinuities it is known as secondary permeability. The rock mass in our study area is extremely splintered making it susceptible to secondary permeability. Also, the study area has faults with softer infilling material (calcite). The occurrence of water within carbonaceous rock is of great concern as it may weaken the shear strength of the rock mass. Water weakens the shear strength by reducing the effective normal stress acting on the surface (Beale & Read, 2013).

5.5. Effects of Slope Stability on Local Surroundings

The local geomorphology is known to change as a result of mineral extraction. Changes in the local geomorphology caused by mineral extraction includes habitat loss, land use, vibrations of the ground from blasting and dust from the open-pit mine (Ayuningrum & Purnaweni, 2018). The vibrations from blasting have a negative impact on surrounding rock mass as the rocks behind the slope face are fragmented and loosened (Kolapo *et al*., 2022). The fragmented rock mass that is also highly fractured means a more slope failures. Slope stability is a crucial consideration in the management of mining operations as slope failure compromises the economical and safety aspect of production. In limestone extraction changes also includes modification of groundwater flow pattern and quality (Ganapathi & Phukan, 2020). To control excessive dust in our study area water truck mounted with spray system is used within the mine. The truck spray water along the road to reduce air pollution. As much as dust is being reduced but the wet gravel road leads to erosion thus negative impact to surrounding society. Also, constant truck driving on the road means damage to road which is also used by surrounding farmers.

Mineral extraction requires expansion of project, which can be further deeper mining or horizontal extension. Deeper extraction requires the additional development of benches and slope faces thus operation will move from small scale mining to much bigger and deeper mining environment (more than 100 meters deep). Thus, more benches mean complicated stability conditions and groundwater challenges. Horizontal extension means removal of vegetation surrounding the mine. In our mine deeper mining resulted in the interception of watertable thus constant water pumping is required for mineral operation to continue. Horizontal expansion resulted in the removal of vegetation. Vegetation cover not only does it affect surrounding farming activity as top rich soil is removed but vegetation is also used as ground stability option. Vegetation avoids run-off water during rainy season, it regulates the atmospheric conditions of an area and it also prevents landslides as the roots withhold the ground.

Limestone mining on Nusakambangan Island near Australia has resulted in environmental damage during the removal of vegetation to access the mineral (Ayuningrum & Purnaweni, 2018). The deforestation led to a reduction in slope stability, and the development of karst in the area. The absence of a buffer in the form of trees makes the area very vulnerable to the movement of slope material, especially in the form of landslide hazards. In addition, other environmental problems that may arise are decreases in soil productivity, and increases in erosion and sedimentation, as well as the disturbance to flora and fauna that have habitats in karst areas (Ayuningrum & Purnaweni, 2018).

According to Stacey (2001), slopes in open-pits and quarries can represent a safety hazard for the following reasons:

- a huge failure of slopes is a threat for workers working surrounding the pit or quarry;
- bench failure are a local threat for workers within the neighbourhood;
- localised failure may result in consequent bouncing of big rocks down into the pit or quarry;
- decline of surface soil border or weathered material may interrupt other parts and may result to mud flows and the build-up of mud in the bottom of the pit or quarry.

Slope failure not only has a negative impact of the safety and economy of the mining environment, but it also affects the surrounding environment. The surrounding environment of the mine includes agricultural farming. Failure of slopes on a larger scale will affect the current geological structure of the area. Geologically, this area forms part of the major Transvaal Supergroup inliers, which is the Crocodile River inlier. This inlier has been ben brutally distorted and experienced low-grade to medium grade metamorphism (Hartzer, 1995). The deformation includes refolding, faulting and is intruded by the Bushveld complex. Due to folding, faulting and igneous intrusions in the past, the mining area is heavily jointed. Heavily jointed area that will result slope failure mostly from planar and wedge failure. Planar failure or wedge failure from a faulted structure will result in extended areal failure which will have negative impact to agricultural farming surrounded.

The Transvaal dolomitic limestone from the Malmani Subgroup is known for karst development (Johnson *et al*., 2006). The surface morphology of karstic rock is typically manifested by karren features. With respect to hydrology, the development of karst is characterized by slow phreatic groundwater movement (Johnson *et al*., 2006). Cave development in a phreatic environment is characterized by large, flat discoidal chambers and complex mazes of passages controlled by local joint patterns. About 750 caves representing 80% of the caves in South Africa are known to be from the Transvaal Basin. The geology of the Thabazimbi District (Malmani dolomite) is known to have several caves (Caincross *et al*., 2018).

As observed on the studied open-pit mine, groundwater is at a shallow level which requires continuous pumping for mining operations to continue. In 1962, a dewatering program from Transvaal dolomitic limestone led to a large sinkhole development under a crushing plant which disappeared, and 29 lives lost (Lurie, 2008). The constant pumping has negative impacts for local farmers needing water as the water table level will drop. Moreover, the constant pumping exposes this area to sinkhole and cave development because it occurs in Transvaal dolomitic limestone which has a history of sinkhole formation.

The current history of Thabazimbi (caves) together with Transvaal karst development gives makes mining in this region difficult. Limestone is an essential ingredient in building or construction materials but the extraction of this product comes with great concern. Also mining in general is known to be a harmful activity to the general environment as it affects the geomorphology and hydrology of the area. In particular, regular pumping of water from the mine also has negative impacts on the current water resource of surrounding area. The water levels drop due to daily pumping thus farmers and surrounding communities will need to drill deeper for water. Pumping of water in a highly jointed rock mass from carbonaceous material will give rise to more karst and caves.

CHAPTER 6: CONCLUSION

6.1. SUMMARY

This chapter describes the research's theoretical background as well as the study's goals and objectives. The summary of the research contribution, the challenges faced during the data collection and the research, as well as the activities that can be predicted moving forward as a result of the research, come next.

This study stated by deliberating the contextual of this research work, the statement of the problem, the research questions intended to be retorted, the research objectives and the importance of the study. It also addresses the comprehensive concern prepared in this study such as the effect of surface mining of carbonaceous rock and unstable slopes due to environmental impact associated with soluble material. It went on show a review of literature which provided an overview of the different slope failure conditions expected in surface mining. This discussion led to the proposal of using rock mass classification to define the rock strength for surface mining slopes. The application of rock mass classification was to estimate the rock mass properties which can be used to determine slope stability.

Economically, in mining design it is important to steepen the slope angles in order to eliminate the minor soil and rock cover as possible, meanwhile at the equivalent time the angles should be flat so as to lessen the probability of failure. Sketching a pit slope in order to strike a balance between economics and safety in open-pit mines is important (Karparov & Handley, 2009). In every slope, the rock or soil is susceptible to slope failure over time. Geotechnical data required to understand the mine pit include structural geology, lithological information and hydrogeological information.

6.2. FEEDBACK ON THE RESEARCH QUESTIONS

6.2.1. CAN EXISTING ROCK MASS CLASSIFICATION SCHEMES BE APPLIED ON ROCK SLOPES IN SURFACE MINING PROCEDURES?

Using rock mass classification has been helpful in determining the slope steadiness in surface mining. According to the work conducted in our study area yes, some of the existing rock mass classifications can be utilised on rock slope to define slope stability for surface mining. The most commonly applied rock mass classification systems are GSI (Hoek & Brown, 1998), RMR system (Bieniawski, 1989), SMR (Romana, 1985), Q-system (Barton *et al.*, 1974), MRMR (Laubsher, 1974). As much as the Q-system and RMR were mainly established for tunnel construction we were able to use the RMR system and not the Q-system. The data collected was able to define the rock mass strength for slope stability analysis. The Bieniawski's Rock

Mass Rating (RMR) gave a score of 42-47 which means fair rock with average stand up time of 1 week for 5m span. RMR incorporates geometric, geological and engineering constraints in getting to a numerical value of their rock mass class. RMR utilises compressive strength directly. RMR deals with the configuration and geology of the rock mass, but in a somewhat different way. It considers groundwater and consider some elements of rock material strength.

The Romana slope mass rating gave a score of 50-58 which means it's a stable slope with partial stability. SMR is more focused on the discontinuities orientation and direction thus one is able to apply the information in kinematic analysis. The advantage of using slope mass rating is the support guidelines one may consider. Each score provides support guidelines that one can consider for support.

The GSI classification system is applied to define the rock mass strength and deformability constraints using the rock mass structure and discontinuity surface setting. The GSI-system gave a score of 40-60 which means an extremely blocky interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets. Incorporates geometric, geological and engineering parameters in getting a quantitative value of their rock mass quality. The GSI system not only does it define the rock mass strength but it can be used as a teaching tool for non-scientist and non-engineers working on the mine. It gives a visual presentation of the situation at the mine, one may use the visuals to have rough approximation of the expected steadiness condition. The engineering classification of karst ground conditions showed evidence of dissolution and buried sinkhole from discontinuities filled with soil. This classification is more of visual comparison.

6.2.2. Are the rock mass classification schemes applied in hard rock suitable for carbonaceous rock?

According to Marinos (2010), limestone is a weak or complex formation, if it is highly disintegrated by thrusting and tectonic rugs, unstable masses can be produced. In the study area the current rock mass classifications are suitable for carbonaceous rock but with some limitations. One is able to use RMR to determine the slope condition but is unable to determine the support conditions for the slopes. According to our study area we have fair RMR which means in the early phases of design and mining, it is desirable to use the support recommended for fair rock. When the mining progresses successfully with no stability challenges, and the support is performing very well, then it could likely slower the support requirements. Also, if the digging is needed to be stable for a short moment of time, then it is worthwhile to attempt the less costly and wide support suggested for fair rock. However, if the rock mass neighbouring the digging is required to undergo significant mining-induced stress variations, then additional substantial support suitable for fair rock should be installed. These available guidelines for RMR support structures are only for tunnel constructions and underground conditions and nothing for surface slopes nor carbonaceous material.

The SMR is applied in both hard and soft rock. It does not give specific attention to the rock type but more on the discontinuities found in the rock mass. The information on discontinuities is mainly concerned with the orientation and direction.

The GSI application is reliant on the type of rock mass condition such as the joint conditions found on the rock mass. GSI also has a specific classification for limestone which is more applicable to carbonaceous material.

The engineering classification of karst ground conditions is suitable for carbonaceous slopes as it looks at the conditions associated with carbonaceous material such as caves and sinkholes.

6.2.3. Can Neglected Hard Rock Properties Cause Changes in Rock Strength for Soft Rock?

In hard rock the impact of deformation is considered when looking at the stress applied on a rock mass (Bieniawski, 1989). Increases in load pressure result in the increase of deformation. Carbonaceous rock typically comprises weak rock or complex formations. The solubility of rock is affected when exposed to environmental agents such as water and temperature expressed equation 2.1 and 2.2. Thus, deformation in carbonaceous material is not only based on the load pressure applied but the impact of environmental agents. Normally, environmental agents cause weathering of a rock mass which influences porosity. Evidence of weathering has been observed in the study area from both the rock mass (e.g. dissolution) and from the in-filling material found in the discontinuities. Also, evidence of porosity has been observed through the engineering classification of karst ground conditions were discontinuities act as secondary zones for water to permeate through the rock.

6.3. Challenges Encountered

The challenges encountered in this study was the limitation to laboratory testing of rock strength. Collection of soft material (crumbled) and being unable to test in the laboratory other than considering the material as soil material. This study is focused on soft rock and not soil thus using soil mechanic equipment was not part of scope. However, the microscope work was conducted through Advanced Laboratory Solutions.

6.4. Future Work

RMR is useful in slope stability analysis. One could look at the guidelines for support structures for slopes as currently only guidelines for tunnel support structures is available. The GSI classification for limestone is useful in slope stability but as carbonaceous material but it would be interesting to see factors such as weathering and porosity being included. Integration of unmanned aerial vehicle data into rock mass classification. The visual observation together with laboratory and scanning technology could possibly produce quicker data for mines when analysing slope stability.

A modified GSI with empirical equation is proposed by Onyango *et al*. (2022) whereby the GSI, as performed on site, is altered by a natural log of the amount of porosity, giving rise to a modified GSI (GSI_m). The GSI_m combined with laboratory properties of rock can be used to define the rock mass properties.

6.5. Conclusion

Slope stability of open-pit mines is significantly controlled by the occurrence of structural features in the rock mass. Evaluation of the slope stability in open-pit mines at diverse levels of mining is imperative for continuous safety and cost-effective mining operations. Slopes are normally assessed using the geotechnical data and physio-mechanical properties of rock/soil. After the geotechnical data, the rock mass quality is measured, and from this the rock mass properties are determined. Comparing the published results from RMR and GSI with regards to cohesion and the angle of frictional angle is valuable when one has limited resources in a mine or when it is a small-scale mining operation.

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APPENDIX

TABLE S.1A: OVERALL RESULTS FOR RMR AT THE NORTHERN SIDE OF THE PIT.

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS							
	Parameter		Range of values				
1	Strength of intact rock material	Point-load strength index	>10 MPa	**4 - 10 MPa**	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred
		Uniaxial comp. strength	>250 MPa	**100 - 250 MPa**	50 - 100 MPa	25 - 50 MPa	5-25 MPa; 1 - 5 MPa; < 1 MPa
		Rating	15	**12**	7	4	2; 1; 0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	**< 25%**
	Rating		20	17	13	8	**3**
3	Spacing of discontinuities		> 2 m	**0.6 - 2 . m**	200 - 600 mm	60 - 200 mm	< 60 mm
	Rating		20	**15**	10	8	5
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	**Slightly rough surfaces Separation < 1 mm Slightly weathered walls**	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous
	Rating		30	**25**	20	10	0
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10-25	25 - 125	> 125
		(Joint water press)/ (Major principal)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5
		General conditions	Completely dry	Damp	Wet	Dripping	**Flowing**
		Rating	15	10	7	4	**0**
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)							
Strike and dip orientations			Very favourable	**Favourable**	Fair	Unfavourable	Very Unfavourable
Ratings		Tunnels & mines	0	-2	-5	-10	-12
		Foundations	0	-2	-7	-15	-25
		Slopes	0	**-5**	-25	-50	
C. ROCKMASS CLASSES DETERMINED FROM TOTAL RATINGS							
Rating			100 - 81	80 - 61	**60 - 41**	40 -21	< 21
Class number			I	II	**III**	IV	V
Description			Very good rock	Good rock	**Fair rock**	Poor rock	Very poor rock
D. MEANING OF ROCK CLASSES							
Class number			I	II	**III**	IV	V
Average stand-up time			20 yrs for 15 m span	1 year for 10 m span	**1 week for 5 m span**	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)			> 400	300 - 400	**200 - 300**	100 - 200	< 100
Friction angle of rock mass (deg)			> 45	35-45	**25-35**	15-25	< 15

TABLE S.1B: OVERALL RESULTS FOR RMR AT THE WESTERN SIDE OF THE PIT.

<table>
<tr><td colspan="12">A. CLASSIFICATION PARAMETERS AND THEIR RATINGS</td></tr>
<tr><td colspan="4">Parameter</td><td colspan="8">Range of values</td></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Strength of intact rock material</td><td colspan="2">Point-load strength index</td><td>>10 MPa</td><td>4 - 10 MPa</td><td>2 - 4 MPa</td><td>1 - 2 MPa</td><td colspan="4">For this low range - uniaxial compressive test is preferred</td></tr>
<tr><td colspan="2">Uniaxial comp. strength</td><td>>250 MPa</td><td>100 - 250 MPa</td><td>50 - 100 MPa</td><td>25 - 50 MPa</td><td colspan="2">5-25 MPa</td><td>1 - 5 MPa</td><td>< 1 MPa</td></tr>
<tr><td colspan="3">Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td colspan="2">2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="3">Drill core Quality RQD</td><td>90% - 100%</td><td>75% - 90%</td><td>50% - 75%</td><td>25% - 50%</td><td colspan="4">< 25%</td></tr>
<tr><td colspan="3">Rating</td><td>20</td><td>17</td><td>13</td><td>8</td><td colspan="4">3</td></tr>
<tr><td rowspan="2">3</td><td colspan="3">Spacing of discontinuities</td><td>> 2 m</td><td>0.6 - 2 . m</td><td>200 - 600 mm</td><td>60 - 200 mm</td><td colspan="4">< 60 mm</td></tr>
<tr><td colspan="3">Rating</td><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="4">5</td></tr>
<tr><td rowspan="2">4</td><td colspan="3">Condition of discontinuities (See E)</td><td>Very rough surfaces Not continuous No separation Unweathered wall rock</td><td>Slightly rough surfaces Separation < 1 mm Slightly weathered walls</td><td>Slightly rough surfaces Separation < 1 mm Highly weathered walls</td><td>Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous</td><td colspan="4">Soft gouge >5 mm thick or Separation > 5 mm Continuous</td></tr>
<tr><td colspan="3">Rating</td><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="4">0</td></tr>
<tr><td rowspan="4">5</td><td rowspan="3">Groundwater</td><td colspan="2">Inflow per 10 m tunnel length (l/m)</td><td>None</td><td>< 10</td><td>10-25</td><td>25 - 125</td><td colspan="4">> 125</td></tr>
<tr><td colspan="2">(Joint water press)/ (Major principal)</td><td>0</td><td>< 0.1</td><td>0.1, - 0.2</td><td>0.2 - 0.5</td><td colspan="4">> 0.5</td></tr>
<tr><td colspan="2">General conditions</td><td>Completely dry</td><td>Damp</td><td>Wet</td><td>Dripping</td><td colspan="4">Flowing</td></tr>
<tr><td colspan="3">Rating</td><td>15</td><td>10</td><td>7</td><td>4</td><td colspan="4">0</td></tr>
<tr><td colspan="12">B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)</td></tr>
<tr><td colspan="4">Strike and dip orientations</td><td>Very favourable</td><td>Favourable</td><td>Fair</td><td>Unfavourable</td><td colspan="4">Very Unfavourable</td></tr>
<tr><td colspan="2" rowspan="3">Ratings</td><td colspan="2">Tunnels & mines</td><td>0</td><td>-2</td><td>-5</td><td>-10</td><td colspan="4">-12</td></tr>
<tr><td colspan="2">Foundations</td><td>0</td><td>-2</td><td>-7</td><td>-15</td><td colspan="4">-25</td></tr>
<tr><td colspan="2">Slopes</td><td>0</td><td>-5</td><td>-25</td><td>-50</td><td colspan="4"></td></tr>
<tr><td colspan="12">C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS</td></tr>
<tr><td colspan="4">Rating</td><td>100 - 81</td><td>80 - 61</td><td>60 - 41</td><td>40 -21</td><td colspan="4">< 21</td></tr>
<tr><td colspan="4">Class number</td><td>I</td><td>II</td><td>III</td><td>IV</td><td colspan="4">V</td></tr>
<tr><td colspan="4">Description</td><td>Very good rock</td><td>Good rock</td><td>Fair rock</td><td>Poor rock</td><td colspan="4">Very poor rock</td></tr>
<tr><td colspan="12">D. MEANING OF ROCK CLASSES</td></tr>
<tr><td colspan="4">Class number</td><td>I</td><td>II</td><td>III</td><td>IV</td><td colspan="4">V</td></tr>
<tr><td colspan="4">Average stand-up time</td><td>20 yrs for 15 m span</td><td>1 year for 10 m span</td><td>1 week for 5 m span</td><td>10 hrs for 2.5 m span</td><td colspan="4">30 min for 1 m span</td></tr>
<tr><td colspan="4">Cohesion of rock mass (kPa)</td><td>> 400</td><td>300 - 400</td><td>200 - 300</td><td>100 - 200</td><td colspan="4">< 100</td></tr>
<tr><td colspan="4">Friction angle of rock mass (deg)</td><td>> 45</td><td>35-45</td><td>25-35</td><td>15-25</td><td colspan="4">< 15</td></tr>
</table>

TABLE S.1C: OVERALL RESULTS FOR RMR AT THE SOUTHERN SIDE OF THE PIT.

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

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

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

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

C. ROCKCLASSES MASS DETERMINED FROM TOTAL RATINGS

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

D. MEANING OF ROCK CLASSES

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

TABLE S.1D: OVERALL RESULTS FOR RMR AT THE EASTERN SIDE OF THE PIT.

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

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

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

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

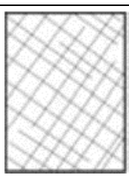
C. ROCKCLASSES MASS DETERMINED FROM TOTAL RATINGS

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

D. MEANING OF ROCK CLASSES

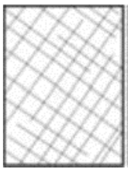
Class number	I	II	**III**	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	**1 week for 5 m span**	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	**200 - 300**	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35-45	**25-35**	15-25	< 15

TABLE S.2A: OVERALL RESULTS FOR GSI AT THE NORTHERN SIDE OF THE PIT.

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek & Marinos, 2000) From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis. SURFACE CONDITIONS STRUCTURE DECREASING INTERLOCKING OF ROCK PIECES	VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
 INTACT OR MASSIVE—intact rock specimens or massive *in situ* rock with few widely spaced discontinuities	90 80			N/A	N/A
 BLOCKY—well interlocked un-disturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		70 **60**			
 **VERY BLOCKY—interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets**			50 40		

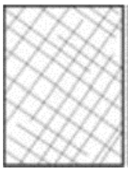
 BLOCKY/DISTURBED/SEAMY folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity				30	
 DISINTEGRATED—poorly inter-locked, heavily broken rock mass with mixture of angular and rounded rock pieces					20 10
 LAMINATED/SHEARED—lack of blockiness due to close spacing of weak schistosity or shear planes	N/A	N/A			

TABLE S.2B: OVERALL RESULTS FOR GSI AT THE WESTERN SIDE OF THE PIT.

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek & Marinos, 2000) From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis. SURFACE CONDITIONS STRUCTURE DECREASING INTERLOCKING OF ROCK PIECES	VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
 INTACT OR MASSIVE—intact rock specimens or massive *in situ* rock with few widely spaced discontinuities	90 80			N/A	N/A
 BLOCKY—well interlocked un-disturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		70 60			
 **VERY BLOCKY—interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets**			**50** **40**		

 BLOCKY/DISTURBED/SEAMY folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity				30	
 DISINTEGRATED—poorly inter-locked, heavily broken rock mass with mixture of angular and rounded rock pieces					20 10
 LAMINATED/SHEARED—lack of blockiness due to close spacing of weak schistosity or shear planes	N/A	N/A			

TABLE S.2C: OVERALL RESULTS FOR GSI AT THE SOUTHERN SIDE OF THE PIT.

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek & Marinos, 2000) From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis. SURFACE CONDITIONS STRUCTURE DECREASING INTERLOCKING OF ROCK PIECES	VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
 INTACT OR MASSIVE—intact rock specimens or massive *in situ* rock with few widely spaced discontinuities	90 80			N/A	N/A
 BLOCKY—well interlocked un-disturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets		70 60			
 **VERY BLOCKY—interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets**			**50** **40**		
 BLOCKY/DISTURBED/SEAMY folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or				30	

schistosity					
 DISINTEGRATED—poorly inter-locked, heavily broken rock mass with mixture of angular and rounded rock pieces					20 10
 LAMINATED/SHEARED—lack of blockiness due to close spacing of weak schistosity or shear planes	N/A	N/A			

Table S.3A: GSI for limestone on the Northern side of the pit (Marinos, 2010).

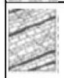
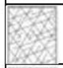
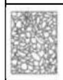
GEOLOGICAL STRENGTH INDEX FOR LIMESTONE ROCK MASS Based on the description of the lithology, structure and surface conditions of discontinuities (particularly of the bedding planes), choose a box in the chart. Locate the position in the box that corresponds to the conditions and estimate the average value GSI from the contours. Quoting a range from 33 to 37 is more realistic than stating that GSI-35. The determination of the structure and the condition of discontinuities may range between two adjacent fields. Note that the Hoek – Brown criterion does not apply to structurally controlled failures. Where unfavourably oriented continuous weak planar discontinuities (like bedding planes) are present, these will dominate the behaviour of the rock mass (attention therefore at types B and C). The strength of some rock masses is reduced by the presence of groundwater and this can be allowed for by a slight shift to the right in the columns for fail, poor and very poor conditions. Water pressure does not change the value of GSI and it is dealt with by using effective stress analysis STRUCTURE	Surface conditions of discontinuities (Predominately bedding planes)	**VERY GOOD** Very rough, fresh unweathered surfaces	**GOOD** Rough, slightly weathered, iron stained surfaces	**FAIR** Smooth, moderately weathered and altered surfaces	**POOR** Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	**VERY POOR** Slickensided, highly weathered surfaces with soft clay coatings or fillings
		DECREASING SURFACE QUALITY ⇨				
 Type-A undisturbed thick bedded to non-bedded limestone, well interlocked consisting of cubical blocks formed by three intersecting discontinuity sets.	DECREASING INTERLOCKING OF ROCK PIECES ⇩	80				
 Type B- undisturbed thin to medium-bedded limestone, with tight well interlocked structure consisting of cubical blocks form three intersecting discontinuity sets. Bedding thickness is of several cm to few dm						
 Type C- undisturbed thin to medium bedded limestone with claystone or siltstone or chert alternations, with tighten-well interlocked structure consisting of cubical blocks formed by three intersecting discontinuity sets. Bedding thickness is of several c to few dm		70				
 **Type D- Very blocky limestone, well interlocked mass with multi-facted angular blocks formed by 4 or more joints.**			**60** **50**			
 Type E – folded-highly disturbed thin bedded limestone with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes.				40		
 Type F- folded-highly disturbed thin bedded limestone with claystone or siltstone or chert alternations, with angular blocks formed by many intersecting discontinuity sets. Loose and open structure due to the poor contact of the blocks with different deformational characteristics. Bedding planes are difficult maintaining their parallelism.				30		
 Type G- Heaily broken, disintegrated limestone. Poorly interlocked with mixture of angular and rounded pieces.					20	
 Type H- Heavily broken, disintegrated limestone with high clay presence along–_the joints. Limestone blocks are not in contact and have very poor interlocking (the scale of this figure is not comparable with the others).						10

Table S.3B: GSI for limestone on the Western side of the pit (Marinos, 2010).

GEOLOGICAL STRENGTH INDEX FOR LIMESTONE ROCK MASS

Based on the description of the lithology, structure and surface conditions of discontinuities (particularly of the bedding planes), choose a box in the chart. Locate the position in the box that corresponds to the conditions and estimate the average value GSI from the contours. Quoting a range from 33 to 37 is more realistic than stating that GSI-35. The determination of the structure and the condition of discontinuities may range between two adjacent fields. Note that the Hoek – Brown criterion does not apply to structurally controlled failures. Where unfavourably oriented continuous weak planar discontinuities (like bedding planes) are present, these will dominate the behaviour of the rock mass (attention therefore at types B and C). The strength of some rock masses is reduced by the presence of groundwater and this can be allowed for by a slight shift to the right in the columns for fail, poor and very poor conditions. Water pressure does not change the value of GSI and it is dealt with by using effective stress analysis

Surface conditions of discontinuities (Predominately bedding planes)

DECREASING SURFACE QUALITY ⇒

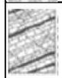
DECREASING INTERLOCKING OF ROCK PIECES ⇓

STRUCTURE	**VERY GOOD** Very rough, fresh unweathered surfaces	**GOOD** Rough, slightly weathered, iron stained surfaces	**FAIR** Smooth, moderately weathered and altered surfaces	**POOR** Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	**VERY POOR** Slickensided, highly weathered surfaces with soft clay coatings or fillings
 Type-A undisturbed thick bedded to non-bedded limestone, well interlocked consisting of cubical blocks formed by three intersecting discontinuity sets.	80				
 Type B- undisturbed thin to medium-bedded limestone, with tight well interlocked structure consisting of cubical blocks form three intersecting discontinuity sets. Bedding thickness is of several cm to few dm					
 Type C- undisturbed thin to medium bedded limestone with claystone or siltstone or chert alternations, with tighten-well interlocked structure consisting of cubical blocks formed by three intersecting discontinuity sets. Bedding thickness is of several c to few dm	70				
 **Type D- Very blocky limestone, well interlocked mass with multi-facted angular blocks formed by 4 or more joints.**		**60** **50**			
 Type E – folded-highly disturbed thin bedded limestone with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes.			40		
 Type F- folded-highly disturbed thin bedded limestone with claystone or siltstone or chert alternations, with angular blocks formed by many intersecting discontinuity sets. Loose and open structure due to the poor contact of the blocks with different deformational characteristics. Bedding planes are difficult maintaining their parallelism.			30		
 Type G- Heaily broken, disintegrated limestone. Poorly interlocked with mixture of angular and rounded pieces.				20	
 Type H- Heavily broken, disintegrated limestone with high clay presence along–the joints. Limestone blocks are not in contact and have very poor interlocking (the scale of this figure is not comparable with the others).					10

Table S.3C: GSI for limestone on the Southern side of the pit (Marinos, 2010).

GEOLOGICAL STRENGTH INDEX FOR LIMESTONE ROCK MASS

Based on the description of the lithology, structure and surface conditions of discontinuities (particularly of the bedding planes), choose a box in the chart. Locate the position in the box that corresponds to the conditions and estimate the average value GSI from the contours. Quoting a range from 33 to 37 is more realistic than stating that GSI-35. The determination of the structure and the condition of discontinuities may range between two adjacent fields. Note that the Hoek – Brown criterion does not apply to structurally controlled failures. Where unfavourably oriented continuous weak planar discontinuities (like bedding planes) are present, these will dominate the behaviour of the rock mass (attention therefore at types B and C). The strength of some rock masses is reduced by the presence of groundwater and this can be allowed for by a slight shift to the right in the columns for fail, poor and very poor conditions. Water pressure does not change the value of GSI and it is dealt with by using effective stress analysis

STRUCTURE		Surface conditions of discontinuities (Predominately bedding planes)	**VERY GOOD** Very rough, fresh unweathered surfaces	**GOOD** Rough, slightly weathered, iron stained surfaces	**FAIR** Smooth, moderately weathered and altered surfaces	**POOR** Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	**VERY POOR** Slickensided, highly weathered surfaces with soft clay coatings or fillings
			DECREASING SURFACE QUALITY ⇒				
	Type-A undisturbed thick bedded to non-bedded limestone, well interlocked consisting of cubical blocks formed by three intersecting discontinuity sets.	DECREASING INTERLOCKING OF ROCK PIECES ⇓	80				
	Type B- undisturbed thin to medium-bedded limestone, with tight well interlocked structure consisting of cubical blocks form three intersecting discontinuity sets. Bedding thickness is of several cm to few dm						
	Type C- undisturbed thin to medium bedded limestone with claystone or siltstone or chert alternations, with tighten-well interlocked structure consisting of cubical blocks formed by three intersecting discontinuity sets. Bedding thickness is of several c to few dm		70				
	**Type D- Very blocky limestone, well interlocked mass with multi-facted angular blocks formed by 4 or more joints.**			**60** **50**			
	Type E – folded-highly disturbed thin bedded limestone with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes.				40		
	Type F- folded-highly disturbed thin bedded limestone with claystone or siltstone or chert alternations, with angular blocks formed by many intersecting discontinuity sets. Loose and open structure due to the poor contact of the blocks with different deformational characteristics. Bedding planes are difficult maintaining their parallelism.				30		
	Type G- Heaily broken, disintegrated limestone. Poorly interlocked with mixture of angular and rounded pieces.					20	
	Type H- Heavily broken, disintegrated limestone with high clay presence along–_the joints. Limestone blocks are not in contact and have very poor interlocking (the scale of this figure is not comparable with the others).						10