Western Australia School of Mines:
Minerals, Energy and Chemical Engineering

Ground Support and Reinforcement Design in Deep Underground Excavation Based on Ground Condition with Emphasis on Mining

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DECLARATION

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgment has been made.

This thesis contains no material, which has been accepted for the award of any other degree or diploma in any university.

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PUBLICATIONS INCORPORATED INTO THIS THESIS

1- Journal Papers

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ABSTRACT

Development of deep underground mining projects that results in safe, high-efficiency and cost-effective conditions is crucial for optimum extraction of mineral deposits. The main challenges at great depth are high-field stresses, seismic events, large-scale deformation, occurrence of sudden failures, and high temperatures that cause abrupt and unpredictable instability and collapse over a large scale. Ground behaviour can vary even for the same rock mass characteristic with the same groundwater condition, stress field and an equal rating in a classification system. Therefore, identification of ground behaviour modes, identifying failure mechanism, and risk assessment are essential steps in the design of deep underground mines. Hence, ground behaviour modes, failure mechanism, ground control management, and monitoring system are discussed in a comprehensive literature review.

In this thesis at first, design of ground support system at great depth is taken account into characterising ground conditions and presenting a new classification in deep and hard rocks. Major geological structures, ground loading factor, hydrology, static/dynamic loading, and key features of in-depth underground mining that are the most effective factors on the diagnosis of rock mass behaviour modes, are evaluated. The result was a developed categorisation including stable, massive rock failure, intact/structural failure, structural failure, and water effect modes. Identification and assessment of failure mechanisms is necessary for ground support design. In some cases, only one failure mechanism is dominant, but in many cases may failure starts with one mechanism and then followed by other mechanism or combination of mechanisms. Rock mass failure mechanisms at great depth are classified into three groups: structural failure, induced stress/seismic failure, and operational failure mechanism. Based on three factors including maximum principal stress, rock mass strength, and continuity factor, a flowchart is proposed for identifying the failure mechanism so-called Flowchart GB-FN. Also, the progress of various failure modes in rock masses over time and at depth is presented.

Second, a new procedure for the design of ground support systems to incorporate deep underground factors is proposed. The main criterion in this classification are static/dynamic loading types and sources, determination of loading factor, characterisation of ground condition and the effects of major geological structures with

underground excavations, distinguishing primary-secondary failure modes, utilising appropriate design analysis methods, estimation of static/dynamic ground demand, and selection of proper surface and reinforcement support elements.

Third, the performance of the ground support system and real ground behaviours are considered by field observational measurements and monitoring systems. The results are used for the optimisation of design parameters. Furthermore, in the thesis a new methodology “Comprehensive Underground Excavation Design” so-called CUED method is presented with emphasis on diagnosis of ground behaviour(s) and failure mechanism(s) in deep and hard rock conditions. According to the methodology, a procedure is defined for each step by determination of input data, processing data and output data that is named IPO. IPO is applied to determine parameters of the CUED method in each step.

To verify the proposed design procedure, a comprehensive database from several underground mines with extensive fieldworks has been used, and some of the case examples are presented in the related chapters. The reliability of the design procedure for great depth and hard rock conditions was justified. The presented principles in this thesis is an innovative methodology for geotechnical design of deep underground mines. The thesis results demonstrated that proposed methods efficiently increase the safety and optimise the project’s cost and time.

Comparison of examined cases studies with proposed design procedure shows high capability of presented method in accomodation deep underground mining factors. Therfore, application of the presented method could significantly improve safety and economy in deep underground mines.

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CHAPTER 1: INTRODUCTION

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1.1. Background

In recent decades, an increase in prices for most mineral commodities and high demand from industries has encouraged the development of deep underground mining projects. Underground mine development requires a plan and design strategy to tackle operational challenges and geotechnical problems like instability.

Underground excavation design methodologies are usually associated with developing solutions for instability in underground mining operations. Many design methods for practical underground structures have been proposed. In situ stress, geological structures, rock mass properties and underground water conditions are evaluated in the design of underground openings, and then design parameters are modified during construction. Hoek and Brown (1980) presented one of the earliest guidelines for the design process in underground excavations. Bieniawski (1984) proposed a systematic design procedure regarding constraints, objective, input data, design method, and determination of output parameters. Stille and Palmström (2003) proposed an approach for rock engineering design based on ground behaviour. Hudson and Feng (2007) outlined an updated design flowchart for rock engineering practices regarding overall assessment of projects such as identify rock mass structures and project conditions, initial design, and final design phases. Stacey (2015) developed a design procedure based on defining the design and executing the design steps with emphasis on key thinking and making a decision during all steps of the design procedure.

Deep underground openings with different geometries are excavated in different types of ground conditions, in situ stress, underground water and overburden that add complexity to the estimation of some rock engineering properties and uncertainty. A rock mass description offers a variety of challenges like rock mass strength and ground failure modes in underground excavations. Potential failure and ground behaviour are used as crucial parameters for developing geotechnical designs (Goricki, 2013).

Stille and Palmström (2008) presented a classification for ground mode conditions in an underground excavation based on the main reasons for instability, which are gravity, stress and underground water condition. Rock burst damage mechanism and its severity, as well as required support functions, have been proposed by Cai (2013). The process of rock failure is determined by crack initiation, crack propagation and coalescence and plays a significant role in considering the micromechanics of rock in long-term strength (Shao and Li, 2015). Analysis results of case studies in geotechnical fields indicate that damage and progressive failure in rock masses have warning signs as indicators and precursors (Szwedzicki, 2003). Development of a deep underground mine leads to induced stresses in the rock mass and may cause rocks to fail violently or a rock burst (seismic events) because of a sudden release of stored energy (Kaiser and Cai, 2012). Sudden and intensive failure of rock is called rockburst damage. This phenomenon is more related to hard rocks and geological structures like dykes and faults and in mining projects associated with high extraction ratios and mining methods that create unsuitable stress conditions.

Ground control or management is a process to predict and manage rock behaviour and the failure condition in underground openings. Ground management is primarily related to the ground condition and any instability issues (Lang, 1995). Singh and Goel (2011) presented a proposal for the classification of the ground condition and a guideline for excavation methods, and the requirement of support systems according to ground behaviour.

Hoek (2006) proposed a guideline for the support required based on rock mass description and behaviour. It is a guideline to describe the rock mass, its behaviour type and also the support system requirement to create a safe condition in underground openings. To choose an appropriate support system in underground excavations, a classification was suggested by MARINOS (2012). The classification shows the principles of rock mass behaviour and associated mode for containing and controlling the specific mechanism of failures. It is noteworthy that rock mass behaviour is an essential consideration in the development of ground support systems.

1.2. Statement of the problem

The development of underground mining projects is resulting in a serious geotechnical challenge related to safety and cost-effectiveness in operations. The main challenges at depth are related to instability, the risk of rock failures, ore dilution, and ore loss. Regarding high stresses in deep mining, a complex ground behaviour and failure mechanism occur during mining activities. Sudden failure and large deformation are some of the important challenges at great depth.

The origins of these challenges pertain to geological structures, discontinuities conditions, site hydrology, uncertainty in the estimation of rock engineering properties and stress components, and mining induced stress/seismicity that influence the behaviour of rock mass structures and may cause instability in underground mining excavations. Additionally, deep mining operations generally deal with high-stress, which can result in sudden deformation and failure of rock mass structures. Unstable rock failure may lead to a delay in production, high-cost in rehabilitation, damage support and mining equipment, loss of ore reserves, injury and fatalities of personnel. Therefore, the design of ground support systems in deep underground mining excavations based on ground behaviour and failure mechanism is essential.

Ground behaviour is described based on the characterisation of rock mass structures, and the knowledge and understanding of rock mass behaviour play a significant role in selecting appropriate techniques and approaches to stabilise rock mass structures. Ground behaviour can vary even for the same rock mass characteristics, groundwater, stress field and same rating in a classification system. Therefore, diagnose of ground behaviour, failure mechanism and hazard recognition is a necessity in the process of design.

The stabilisation of rock zones in mining excavations is associated with firstly, considering excavation design approaches and secondly, selecting appropriate ground support elements, which are all called ground control and management strategies. In the case of excavation design approaches, the orientation of underground openings, excavation method, and sequence and blasting control will be necessary. Ground support systems should be designed for both dynamic and static loading conditions and from ground behaviour and failure modes at great depth.

1.3. Aims and objectives

Understanding the rock mass behaviour provides appropriate design parameters to use in the development of underground mining excavations. Wayne Gretzky, one of the famous hockey players, said, "Skate to where the puck is going, not where it is and if you cannot predict, you will never become a good hockey player". Similarly, a good rock engineer needs to forecast; otherwise, it is not possible to be a good ground engineer (Cai, 2013).

Knowledge, understanding and estimation of rock engineering properties is a fundamental step in the geotechnical design approach. The result of site investigations and laboratory/field tests supply information for characterisation of rock mass structures based on intact rock properties, geological conditions, discontinuity characteristics, in situ stresses, and hydrological conditions. Then, rock mass structures are characterised from massive to disintegrated/crushed/soil like materials in five classes.

Effective factors influencing ground behaviour are active stress factor, construction conditions and geological structures. A practical approach is proposed for the diagnosis of ground behaviour modes and failure mechanism. A fundamental part of the modern design of underground excavations is predicting instability and rock failure before occurrence in the rock mass, and managing of ground conditions.

The purpose of this study is to develop a new design method in deep underground excavations with the focus of identification of ground behaviour and failure mechanisms. The methodology is to be more reliable and useful to reduce geotechnical challenges, increase productivity and reduce costs in deep level underground works. Also, some of the specific objectives of the thesis are to below:

- Clear understanding characterisation of rock mass at great depth
- Reliable prediction of failure modes
- Suitable design approaches in deep mining
- Monitoring and observation methods in deep mining operations
- Design optimisation

1.4. Significance of the study

Estimation of rock engineering properties, state of stress condition, groundwater, and design analysis of any instability in the openings are the necessary steps in design methods. A significant challenge of geotechnical engineers in underground construction is rock stability and how it affects the productivity and operating costs. Different geometry (size and shape), in situ stress state, geological structure and rock mass composition, and groundwater condition are some of the essential factors that may influence various ground behaviour modes during construction. Also, groundwater pressure, residual stress, seismic events, tectonized stress and induced stress lead to a complex and multiple loading from rocks at a micro scale to a large scale surrounding an excavation.

This research provides a dynamic design methodology so-called "CUED methodology". The methodology is established based on input data, data processing and out data in each step, which is named IPO approach. The proposed method is subjected to high-stress levels, seismic events, sudden deformation and related to the failure of hard rock mass structures. Also, a collection of comprehensive data, diagnosis of rock hazards and failure mechanisms, design analysis, and selection of stabilisation methods are conducted in the design phase. Appropriate ground support techniques and strategies in unstable zones surrounding an excavation are evaluated in the design, implementation and monitoring stages. Specialising design methodology to develop deep underground mines and hard rock conditions considers mining factors. Also, the method provides new insight into ground characterisation, ground behaviour, ground control management.

1.5. Methodology of research

This thesis deals with the design of ground support systems in deep and hard rock underground mining excavations concerning consideration and evaluation of the behaviour of rock masses during operational activities.

For this purpose, comprehensive data from in-depth underground excavation projects were collected by site investigation, some laboratory tests and observational methods undertaken in field works.

Concerning information from various cases, a hard–rock mass classification is proposed incorporating rock types, geological conditions, in situ stress effects, and intact rock and strength characteristics.

Collected data related to ground behaviour and failure mechanism in deep, hard rock mining excavations is used to present a practical and reliable model for diagnosing ground behaviour and failure mechanism based on the most effective factors.

Based collected cases data, knowledge and experiences, the design methodology is developed to deep underground mining conditions. Ground support design in deep underground mines is evaluated for ground management by proposing a new design methodology for selecting appropriate ground support elements for both dynamic and static loads. Also, geotechnical monitoring and update of the design are undertaken as part of the design procedure.

Several case examples in deep underground excavations are studied by collecting data, interpreting, analysing and verifying the design parameters implemented. This provided reliability of the proposed methodology under the thesis objectives.

1.6. Scope and limitations

The presented methodology provides a comprehensive reference to involved factors in Geomechanical instability and may partly use at different stages of mining design phases such as conceptual, pre-feasibility, feasibility and detail design construction, and during operation. Obviously, with increasing design process the focus is more from top to bottom of the presented design methodology.

The research program was conducted in deep underground mines in hard rock conditions. Collected data was mostly based on site observations, fieldwork and laboratory tests. Limit access and permission in some mining project did not allow to consider rock mass behaviour and failure modes.

Also, deep underground mining projects do not employ monitoring system during operational activities, so there was not any recorded data to consider monitoring system in the mining projects in this research.

1.7. Thesis structure

The thesis consists of ten chapters, which is shown in Figure 1.1 and summarised below:

- Chapter 1 presents a brief examination of the research field, primary research goals and scope, and a description of the methodology of the research program.
- Chapter 2 presents an extensive review of ground conditions, underground mining project conditions, rock mass behaviour modes in underground excavations, a

review of typical failure mechanisms in rock mass structures, and review of common strategies in ground control management in deep and hard rock mines.

- Chapter 3 describes the characterisation of rock mass structures in deep and hard rock conditions based on practical features, including intact rock, discontinuity characteristics, geological condition, in situ stress, and hydrology as the input data. Processing of rock mass composition is considered about conventional empirical methods, standard guidelines and engineering judgments. Then, a classification of hard rock mass composition at great depth is proposed.
- Chapter 4 discusses the diagnosis of ground behaviour in deep-hard rock mines. Influencing factors on ground behaviour, which are major geological structures, active stress, critical features of underground mining excavations, hydrological condition, and static and/or dynamic loading conditions are considered. Based on the collected data from fieldwork in several case studies, a developed model is proposed for identifying rock mass behaviour modes at great depth.
- Chapter 5 describes deep hard rock mass failure mechanisms in two parts: the intact rock failure mechanism, and rock mass failure mechanism. Collected data from mine sites are analysed and interpreted through site observational methods, and a model is developed for rock mass failure mechanisms in hard and deep mines.
- Chapter 6 presents a developed strategy for ground control management in deep mining excavations. Also, a new methodology is proposed for ground support design by both static and dynamic loading, major geological condition, loading factor and failure mechanism.
- Chapter 7 considers the operations and construction approaches in deep mines. Rock engineering strategies in underground mining excavations concerning geotechnical aspects, which are excavation methods, excavation sequences, extraction ratio, and quality control of materials, are discussed.
- Chapter 8 presents geotechnical monitoring and design updates in deep mines. For this purpose, demonstration of instrumentation, monitoring systems, back – analysis approaches, and a design update is provided for practical conditions and case studies in deep-hard rock mines.

- Chapter 9 proposes a new comprehensive underground excavation design (CUED) methodology in deep and hard rock conditions.
- Chapter 10 provides a summary of the critical findings of the thesis and presents a recommendation for further work in the future.

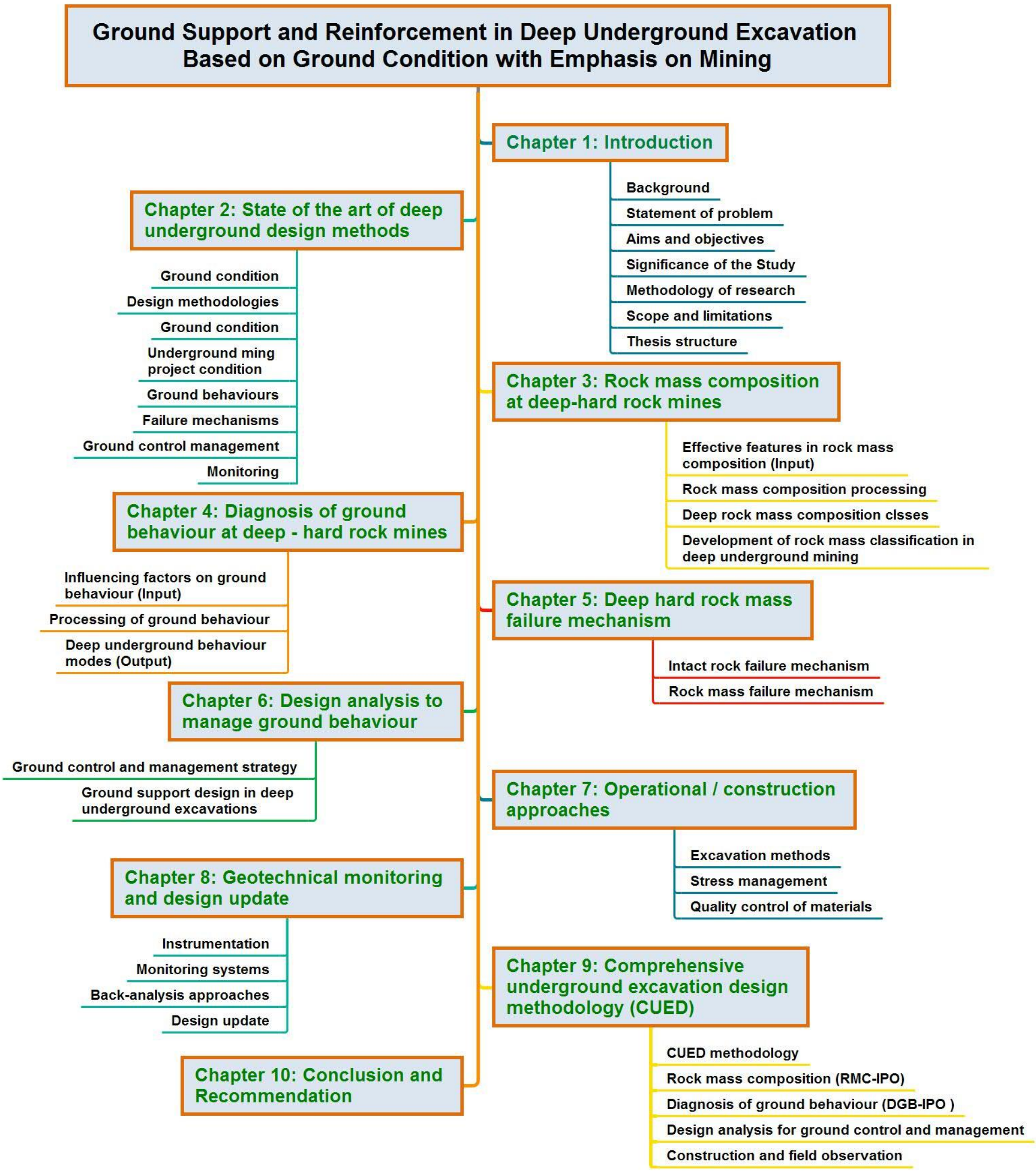

Figure 1.1. Thesis structure

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CHAPTER 2: STATE OF THE ART OF DEEP UNDERGROUND DESIGN METHODS

CHAPTER 2: CONTENTS

2.1. Introduction

In the recent decades, an increase in prices for most mineral commodities and high demand in the industry has brought developing of underground mining projects at great depths. The significant role of the geotechnical field in the development of underground openings is inevitable. A trend towards great depth causes an increase in the problems related to stability, a reduction in labour efficiency and increasing costs. Unstable and failure rock zones may lead to injuries and fatalities as well as damage to equipment. Also, balancing financial expenses and assessment of risks is a challenge for rock engineers. Deep underground openings with different geometries are excavated in different types of ground, in situ stress, groundwater and overburden, which involve complexity and uncertainty in rock engineering properties. Rock mass descriptions offer a variety of challenges, such as rock mass strength and failure modes, for deep underground construction.

There are different types of design methods such as empirical methods, analytical methods, numerical methods and observational methods, in rock engineering projects. The process of design has been considered in several papers and books, for example, Hoek and Brown (1980b), Bieniawski (1984), (Stille and Palmstrom, 2003), (Hudson and Feng, 2007), and(Stacey, 2015). Understanding rock mass behaviour provides appropriate design parameters to use in numerical methods.

The knowledge and comprehension of rock behaviour and failure, which involve all of the processes and procedures for dealing with geotechnical hazards in rock underground works while the rock mass is not supported after excavation, considerably depends on the property of the rock mass. Considering ground behaviours, geotechnical hazards and the process of the failure mechanism are a necessity in the modern design. The sequence of failure progress in rocks is specified as crack initiation, crack propagation and coalescence and plays an important role to assess the micromechanics of rock on long-term strength (Shao and Li, 2015).

Identifying pre-failure and indicator warnings in rock masses enables the recognition of rock failure before a collapse occurs. Consideration of geological structure behaviour and rock failure establishes an appropriate strategy for the management of the ground condition. Ground control is initially concerned with ground conditions and instability issues (Lang, 1995). Knowledge of the ground condition plays a considerable role in pillar design, support design for underground mines, slope stability, dam design, instrumentation of mining operations, roof controls and geomechanics and so on. Installing a ground support system is a typical action for stabilising a rock mass structure. Although several rock support classifications such as Hoek and Wood (1987), Goricki et al. (2006) and Singh (2011) have been presented, almost all of them lack a relationship between ground behaviour and a failure mechanism, and ground support systems in an underground excavation.

This chapter considers the background of design methodology in underground excavations. The main steps in design underground mining methods including ground condition, failure mechanism, ground control and management, and monitoring are discussed. Also, most essential phases in the modern design of underground excavations are presented.

2.2. Design methodologies

In recent decades many design procedures have been developed to use in underground excavations. Design procedures focused on a reduction of challenges and problems in rock engineering projects by using appropriate approaches to determine rock mass properties and improve the stability of openings.

Hoek and Brown (1980b) presented one of the earliest guidelines for the design process in underground excavations, as shown in Figure 2.1. The main features of this methodology include site investigation and characterisation of data, design analysis, and monitoring of the performance of the rock mass during and after construction. Bieniawski (1984) proposed a systematic design process for rock engineering (Figure 2.2). Rock underground design procedures are usually related to location, size and shape, layout, excavation process, support system and monitoring. According to this method, required input data are collected from geological structures, in situ stress field, and groundwater based on the objectives of projects. Empirical methods, observational methods and analytical methods are employed to determine output specifications such as ground support systems.

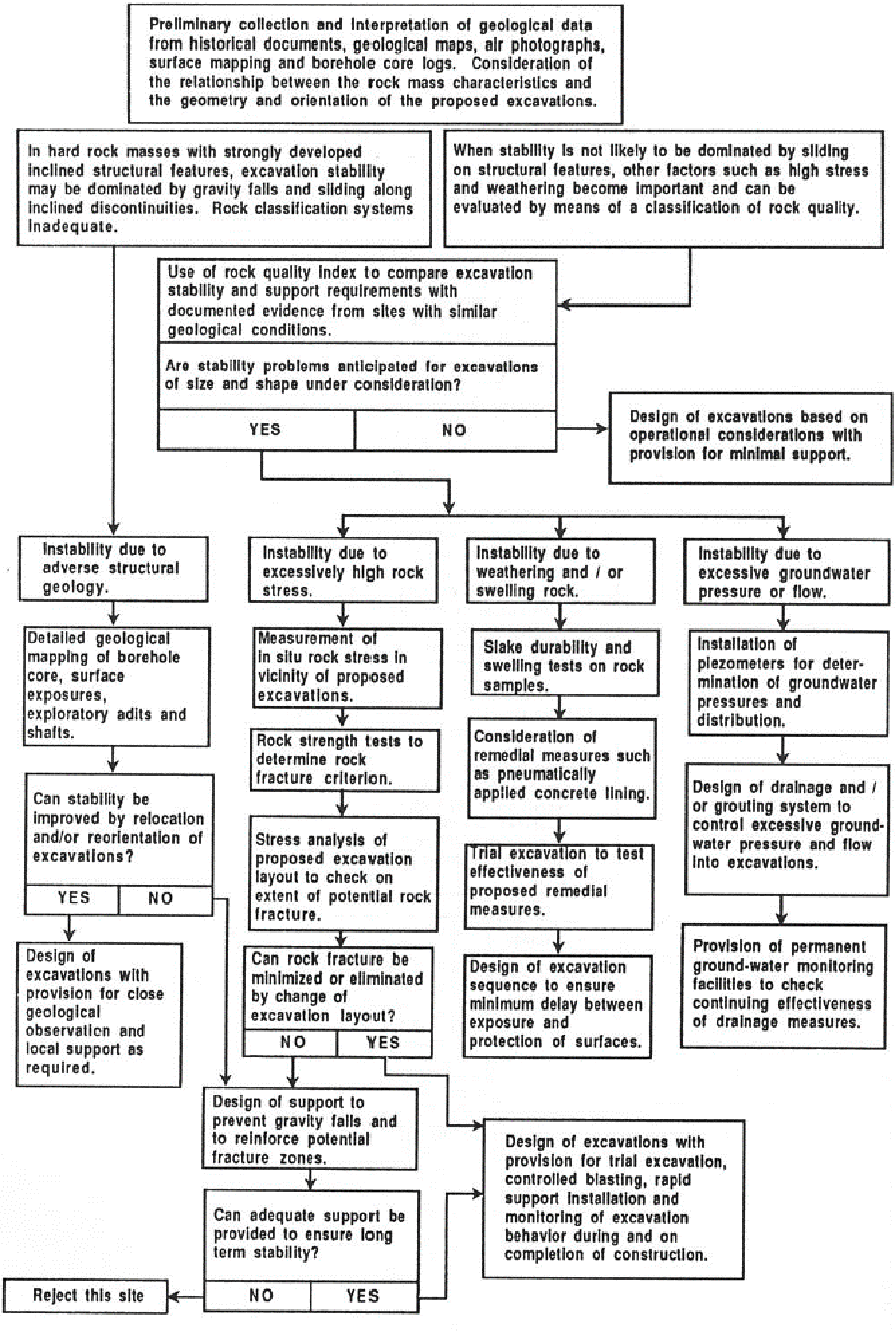

Figure 2.1. Design procedure in rock underground excavations (Hoek and Brown, 1980a)

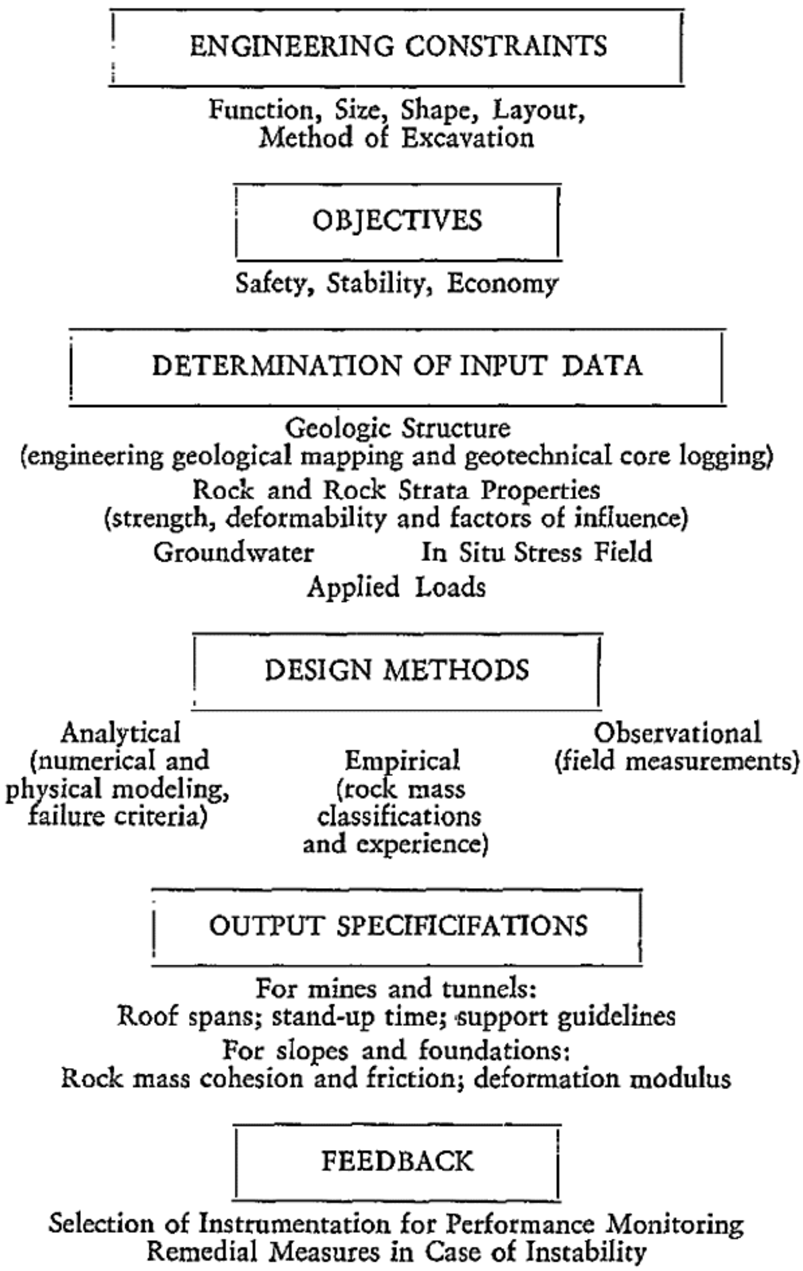

Figure 2.2. The design process in rock engineering practices (Bieniawski, 1984)

Brady and Brown (2006) presented a design methodology to use in underground mines as shown in Figure 2.3. The methodology was established in five steps including site characterisation, mine model formulation, design analysis, rock performance monitoring, and retrospective analysis. The mechanical properties of host rock and ore body are estimated in the site investigation phase. Geomechanical features of the mine site such as strength and deformation properties are modelled in the second step. Mining excavations are designed and analysed based on mathematical and computational methods. The response of rock mass and load-deformation behaviour of rocks during mining activities are monitored and reviewed.

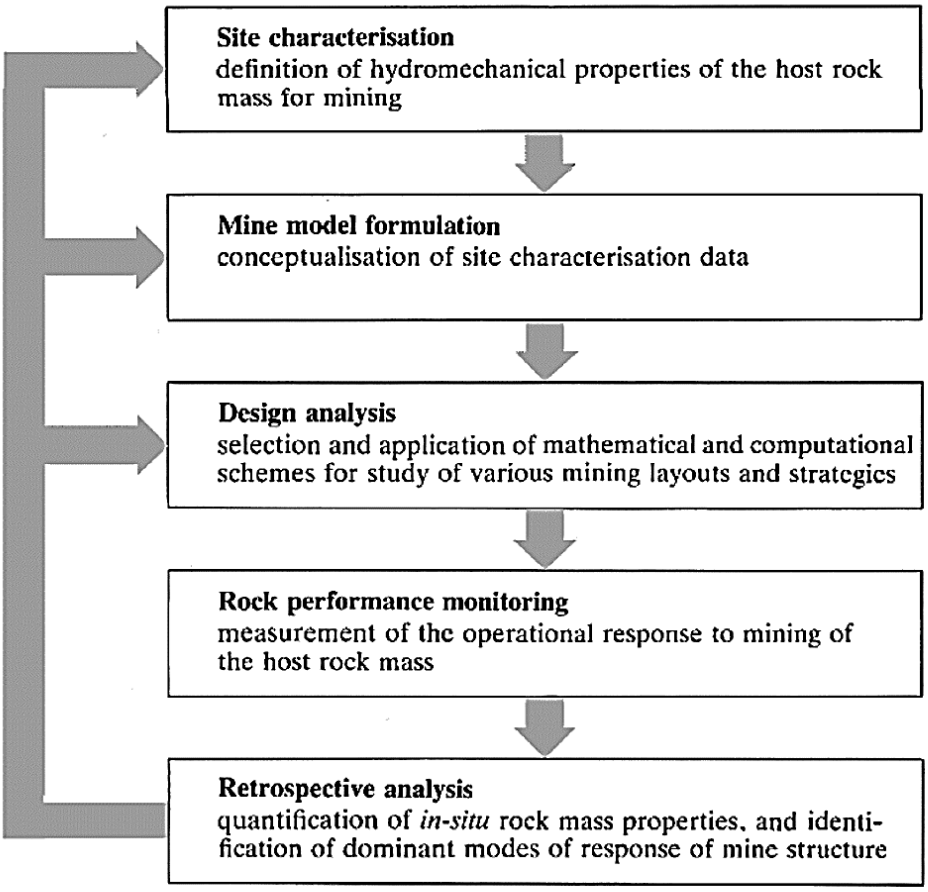

Figure 2.3. Rock engineering design in underground mines (Brady and Brown, 2006, Brady and Brown, 1985)

Stille and Palmström (2003) outlined an approach for rock engineering design based on ground behaviour in Figure 2.4. According to the design method, rock mass characterisation is performed based on geological judgement and field observations which provide the information required to identify rock mass features and possible ground behaviour types. Hence, the suitable design tool(s) such as classification systems, numerical methods and observation methods are selected to design rock engineering practices. Also, considering the degree of the jointed rock mass and weak zones such as faults is essential in the design process of underground excavations.

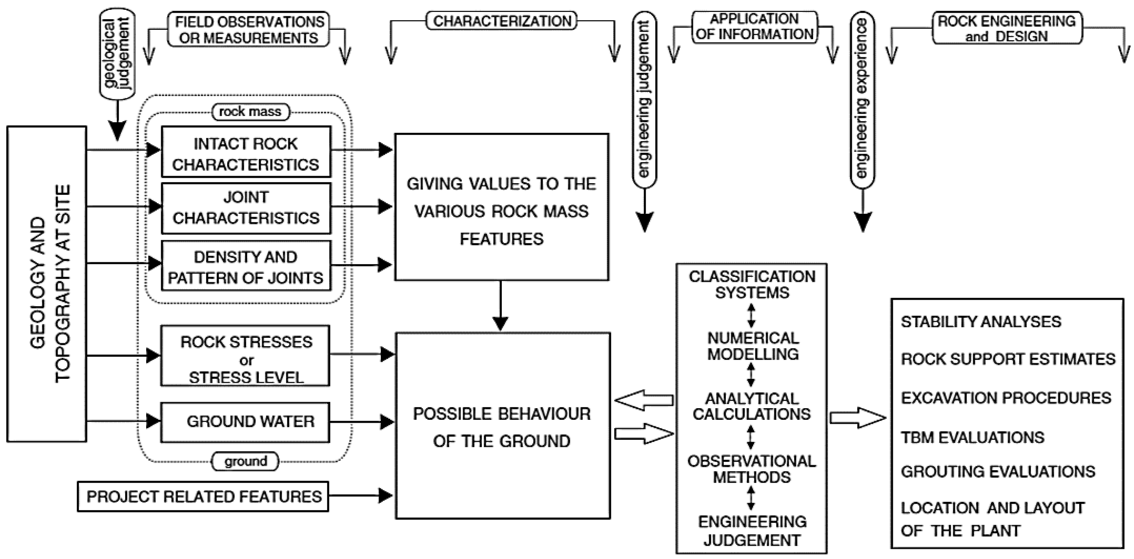

Figure 2.4. The principle of rock engineering practices based on ground behaviour (Stille and Palmström, 2003)

Hudson and Feng (2007) developed an updated design flowchart for rock engineering practices regarding overall assessment of projects such as identify rock mass structures and project conditions, initial design, and final design phases.

Fundamental thinking and making a decision are essential issues during the stages of planning a project and a design procedure. Stacey (2015) proposed a process shown in Figure 2.6 as a design methodology. The procedure provides a work plan in design programs. The work plan is to consider problems and constraints in site investigations and available input data in rock engineering projects, and set out objectives for those projects and then define a budget. The first four stages in this figure are collectively called Defining the Design. The essential step in this procedure is the formulation of the model. Also, the other stages are the implementation of the design at different levels.

Prior design procedures cover the collecting required data for site investigation with a definition of rock engineering properties and geological information. Design analysis methods are generally but not especially explained through the procedure. Also, the relations between various factors have been presented in the design methods. Analysis methods resulted in a guideline for selecting ground support systems and excavation methods. Ground behaviour modes and failure mechanisms in in-depth underground mining projects have not discussed and considered in the current design procedures that is an essential step for design analysis and ground management. The size, geometry and excavation methods in mining engineering fields are different from civil engineering projects, which lead to different ground behaviour modes and failure mechanism and required appropriate ground supports and monitoring systems.

Design of underground mining projects can be summarised in the following steps:

1. Describe rock mass structures, determine rock engineering properties, estimate in situ stresses, and assess groundwater condition
2. Identify a layout of the underground excavations such as stopes and ore drive access
3. Interpret and distinguish ground conditions surrounding openings
4. Design analysis of instability
5. Field observation and monitoring

Design of underground mining projects can employ an instability condition as a problem to model, using suitable methods such as analytical and empirical methods; then the installation of an appropriate support system, and monitoring and optimisation.

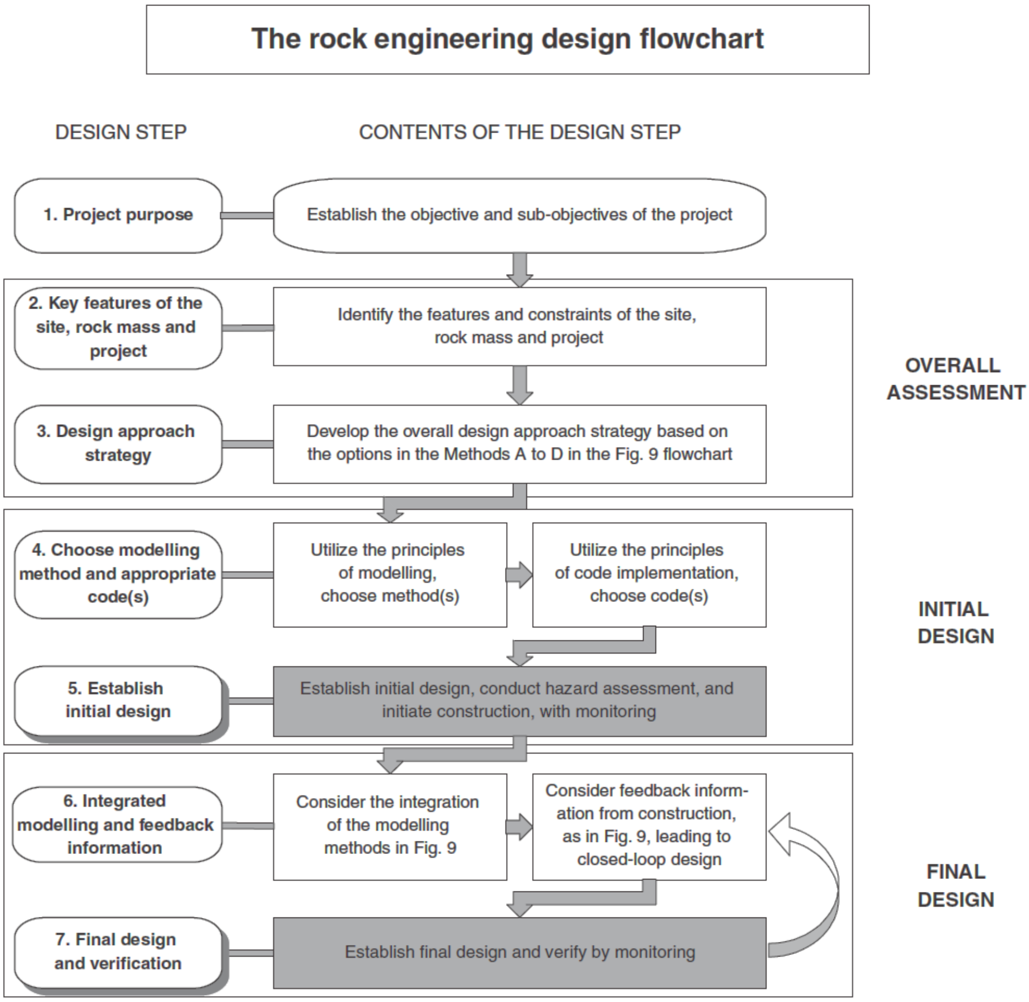

Figure 2.5. The procedure of rock engineering excavations and designs (Hudson and Feng, 2007)

2.3. Ground condition

Rock is a natural and solid material, which formed from a combination of one or more than one type of minerals. The dominant rock types include igneous, sedimentary and metamorphic rocks. Melting, crystallisation, weathering, transportation, lithification and metamorphism are some of the essential processes in the formation of rocks.

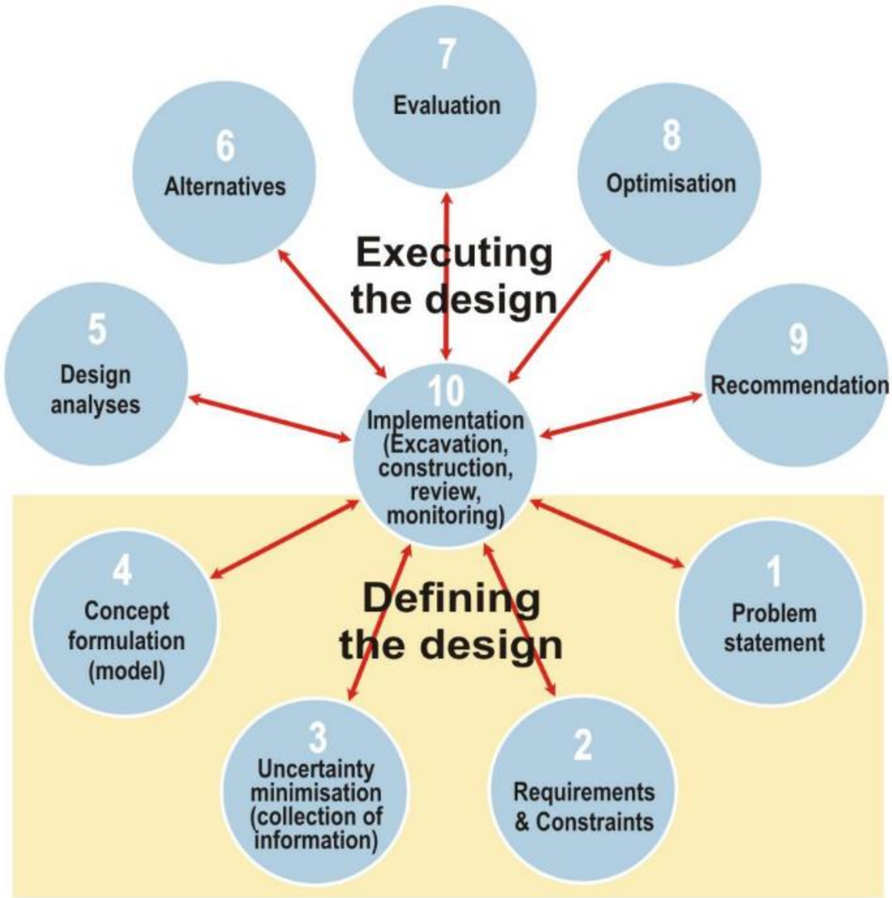

Figure 2.6. Design procedure in engineering projects (Stacey, 2015)

The first step in forming rock is called induration and consists of a hardening process and metamorphism of rocks and sediments. Then, crystallisation of materials is the consolidation of materials in cold solution or hot melt conditions. Also, the grade of induration can vary from poorly indurated like compacted sediment, to highly indurated, which results from recrystallisation to metamorphic rocks. Hence, based on geological history, the properties of sedimentary rock and its equivalent metamorphic rock may vary over a wide range. In contrast, ongoing disaggregation via mechanical and chemical phases breaks down intact rocks into an accumulation of small solids and slack sediments (Adler, 1973). Mechanical activity is common during the initial phase of separation of rock material due to various factors such as tectonics, wind and temperature. Weathering and water are the two major components of chemical alteration. Hydrothermal activity may produce residual clays or cavities, especially in rocks contacting mineral clays. Chemical processes are critical in engineering projects because they reduce the strength of the rock.

Intact rock (at laboratory sizes) is representative of a rock mass without any joints or discontinuities. Intact rock properties usually vary over a wide range of values in geological structures. Determination of intact rock properties is an essential issue in most rock engineering projects.

The geological approach for engineering purposes is to investigate the original rock formation, weathering, tectonic activities, geomorphology, erosion, the nature of various strata, hydrogeological conditions and other features. Consideration of the three main types of geological rocks, igneous, sedimentary and metamorphic, is to characterise rock structures for engineering purposes. Igneous rocks are formed through cooling magma and growing minerals below the surface of the earth. The volcanic rocks usually have large crystals due to slow cooling magma and their crystalline texture. The common minerals in igneous rocks are feldspar, quartz, muscovite, olivine, pyroxene and amphibole. The minerals containing iron like biotite, magnetite and hornblende are scarcer in these types of rocks. The strength of igneous rocks is often high in the fresh state (Hencher, 2012). Quick cooling magma often makes finer-grain size rocks like basalt because of the fast crystallisation. The main groups of sedimentary deposits are clastic soils (fragmentary materials from other rocks), clastic rock, volcanoclastic rock (such as tuffs), and chemical & biochemical rock. Metamorphic rocks are derived from other rocks (igneous, sedimentary or metamorphic) due to the temperature and pressure conditions, and consequently may physically deform and chemically change. For example, marble is formed from limestone through heat and pressure in metamorphic zones. The calcite in the limestone is metamorphosed into calcite crystals.

Structures in ground conditions are divided into three groups as below(Figure 2.7):

1. Primary structures that are related to genetics, formation and bedding in rocks.

2. Secondary structures, which are created by tectonic activities such as fault, folding, and shear zones
3. Mechanical structures or structures formed by engineering activities such as blasting

Evaluation of rock structure condition is associated with specifying the inherent properties of the rock mass that involves measurement of the intact rock strength, natural fracture and discontinuities and their situation, to provide a context for rock mass classification and the design procedure (Barsanti and Basson, 2015). Figure 2.8 illustrates the main features describing rock mass structures. Various types of minerals including sheet minerals such as mica and chlorite, hard minerals for example quartz, swelling clay minerals, and soluble carbonate minerals influence mechanical properties of the rock (Palmstrom and Stille, 2015). Meanwhile, weathering, discontinuity conditions and weak zones, groundwater and rock type provide information to classify the structure of a rock mass.

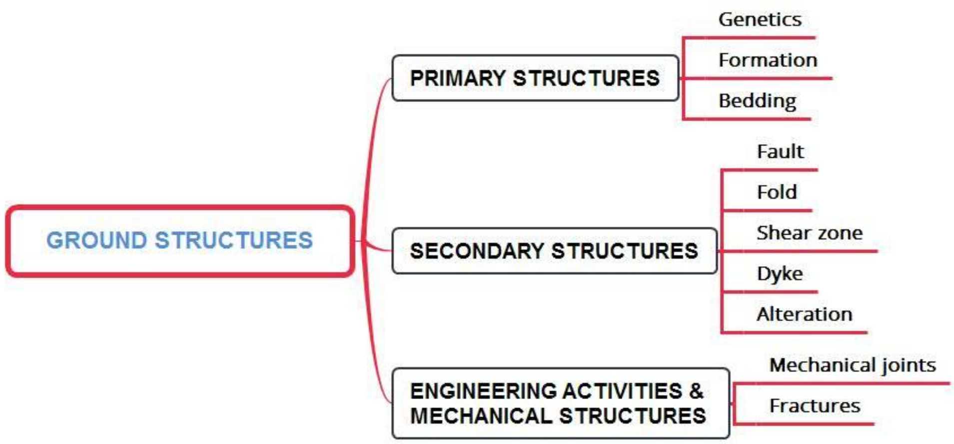

Figure 2.7. Different types of structures in the ground

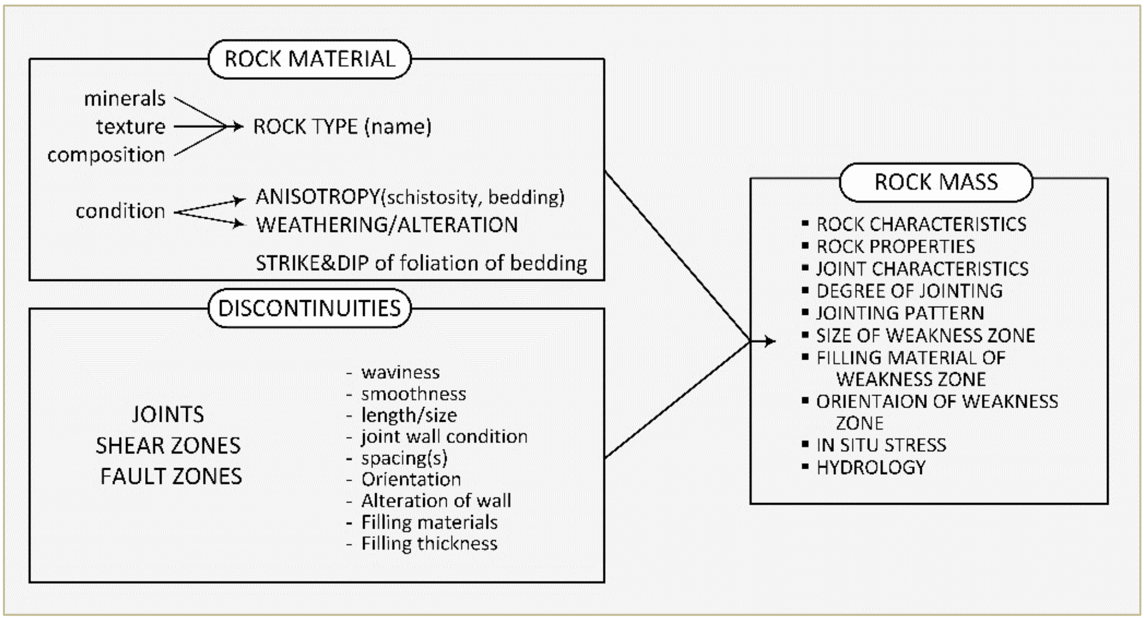

Figure 2.8. The main features for characterising rock mass structure (Modified after Palmstrom and Stille (2015))

Geological structures such as faults often create severe problems for safety and economics. Estimation of orientation and magnitude of the principal stress and probability of deformation mechanisms of fault zones may lead to prediction and forewarning of failure initiation, brittle or plastic behaviour in rock mass structures (Carter, 2015). Figure 2.9 shows the mechanism of plate movement and the formation of major fault types in geological structures. Spreading or divergence of movement of surface plates creates normal faults. Compression of plates (convergence) forms thrust faults (reverse faults). Meanwhile, strike-slip faults are associated with lateral slip movement of plate surfaces.

Geological conditions, such as tectonic activity, influence the quality and structure of a rock mass. Tectonic areas involve fractured and jointed rocks, and the intact rock strength is often reduced due to shearing.

Weathering and alteration affect the strength and deformability of intact rock, parameters that are very important for engineering purposes. The degree of weathering is related to the depth of rocks. Surface rocks are usually highly weathered in comparison with the rock structure in deep underground mines.

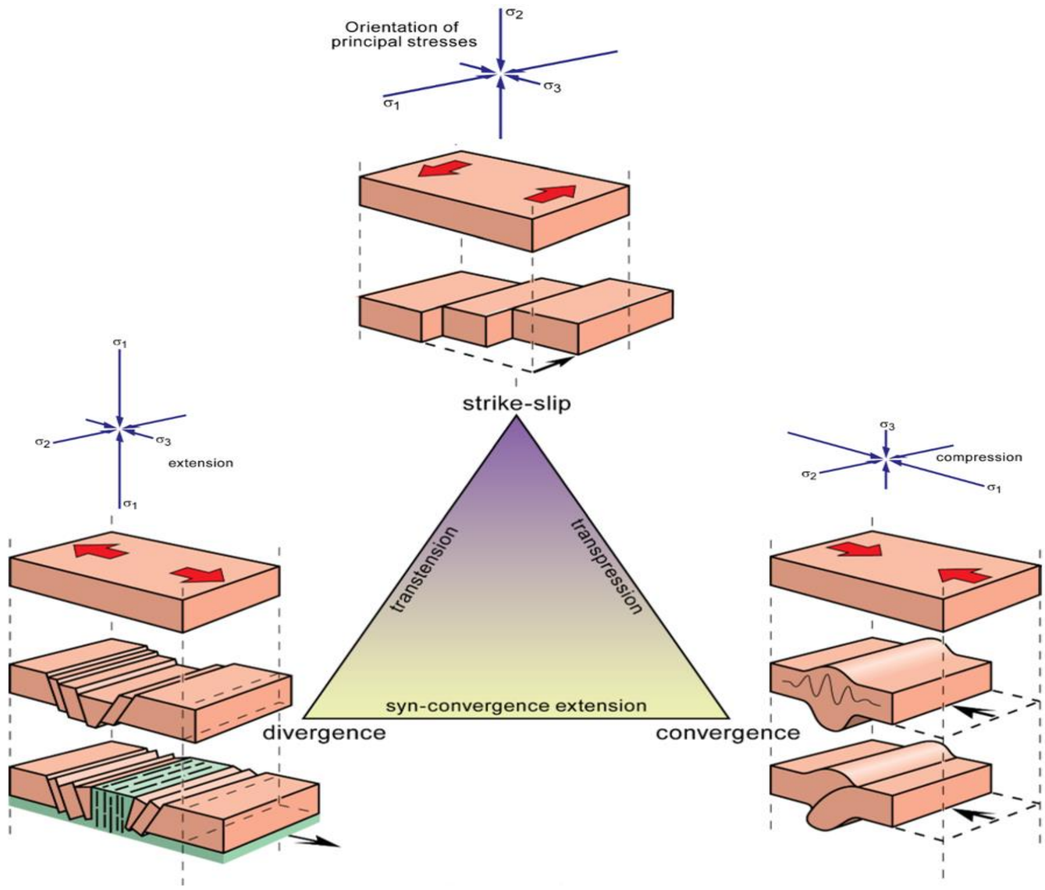

Figure 2.9. Mechanism plate movement and ternary diagram of major fault types (Modified after Carter (2015))

Generally, initial stresses result from gravitational stress, topographic stress, tectonic stress and residual stress. Gravitational stress is due to the gravity factor. Vertical stress in the field may be equal to the gravitational vertical factor. Surface topography considerably affects local rock stress conditions near valleys or mountain ranges. The orientation of the minor stress component is almost perpendicular with the surface slope of topography, while it is parallel with the topographic surface slope for the major stress(Stille and Palmstrom, 2008). Tectonic stress is a result of tectonic events such as faults, plate tectonics and folds. The relationship between the vertical and horizontal stress conditions may change in a very vast range in tectonic rock masses. Meanwhile, in many cases, the total horizontal stress state derived from the gravity state is less than the tectonic state. Residual stress is related to remanent stresses from previous stages of the geological history of the rock like the cooling process of magma and consolidation of rock. Geological structures such as faults and folds influence in situ stress condition. Stress field varies with depth, tectonic activities and underground excavations.

Ground conditions in all site projects are not at the same level of complexity: while some grounds are simple, the others may be complex. Suitable methods, influenced by the purpose of the projects, should be selected to determine rock engineering properties during site investigations. Identification of the essential geological information and critical parameters should be carried out during the ground investigation. Meanwhile, the cost of the site investigation is a small part of the overall project cost (about a few percents) (Hencher, 2012). An engineering geological survey and data collection from site provide information to make a preliminary ground model, which is linked to basic engineering design. In situ and laboratory tests can be carried out to determine detailed ground information. The quality of site investigation for rock engineering projects depends on the complexity of ground conditions, the nature of projects, the existence of information from previous projects, and cost.

A site investigation from the aspect of engineering geology requires detailed and comprehensive information of environmental factors, rock ground condition and natural hazards.

Rock mass characterisation is based on quantitative and qualitative observation and is independent of the design procedure. Consideration of rock mass structure and characterisation is used to estimate ore body geometry and rock mass properties which play vital roles in stope design, stope dilution and the requirements of ground support (Forster et al., 2012). Data collection techniques with geological and geotechnical mapping, and core logging methods are applied to link information to describe the rock mass composition. Geophysical borehole logging is a useful technique to characterise the rock mass around a borehole that developed from use in petroleum exploration, civil engineering and mining engineering and provides information on geotechnical properties and geological structures' correlations.

Management of groundwater utilises the evaluation of drainage of rainwater, location of water streams and storage, and also the potential for recharge of groundwater in the feasibility study of mining projects (Brown and Rosengren, 2000). Employment of the strategy prevents or reduces water inflow into a mining excavation and affections the surrounding rock mass.

The initial state of stresses affects the deformation of underground excavations and behaviour of the surrounding rock mass. This may cause instability of an underground opening and the necessity for support and reinforcement of the rock mass.

The most important process to assess a rock mass structure consist of the following stages (Brown and Rosengren, 2000):

- Drilling and core logging
- Geotechnical scanline mapping
- Rock mechanics laboratory/field testing
- Borehole logging
- Analysing data and determining quantitative geometry of the rock mass
- Simulating rock mass geometry by 3D statistical models
- Defining rock mass classification based on engineering parameters and the particular application of rock mass

Rock mass structures should be characterised based up the estimation of rock engineering properties, field stress components, major discontinuities conditions, geological condition and tectonic activities and hydrological condition.

2.4. Underground mining projects

The excavations of underground mining projects are influenced by rock mass condition; and shape, geometry, size and stability of orebodies. Underground works require some access for stopes, extraction of mineral resources, transport of ore and waste materials, water/power supply, ventilation of main and temporary accesses, drainage, transport of personnel and equipment. Typical underground mining access is shown in Figure 2.10. The lifespan of underground openings can be divided into three groups based on their service purposes and uses:

1- Short – service life (less than six months); for example, mine stops and temporary access
2- Medium-service life (more than six months and less than three years) such as ore drive access and exploration tunnel
3- Long-service life (more than three years) like decline, road tunnel and underground cavern

Mining projects at great depth are developed using various excavations such as vertical shafts, inclined ramps, horizontal drifts, fuel stores, explosive magazines, mining stopes, fuel stores and pump houses. Transportation of personnel, equipment, and ore and waste materials is commonly achieved by vertical shafts or declines, which have access to the main levels of drifts and ore passes. The shape of shafts can be circular, rectangular or elliptical. The dimensions of drifts and ramps are selected based on equipment, ventilation, walkways and other facilities. The dimension can change from 2.2 m to 6.0 m, or 5.0 m^2 to 25.0 m^2. Additionally, the typical grade of ramps is between 1:10 and 1:5. The typical ramp radius is designed to be about 15 m and in a spiral shape (COPCO, 2007).

The common excavation techniques in the mining field are drilling and blasting, and mechanical methods are utilising road headers or earthmoving equipment like bulldozers (Dobinson and Bowen, 1997). The excavation cycle involves drilling of holes, charging, igniting and blasting, ventilating, scaling, loading, hauling, installing reinforcement and surface support systems.

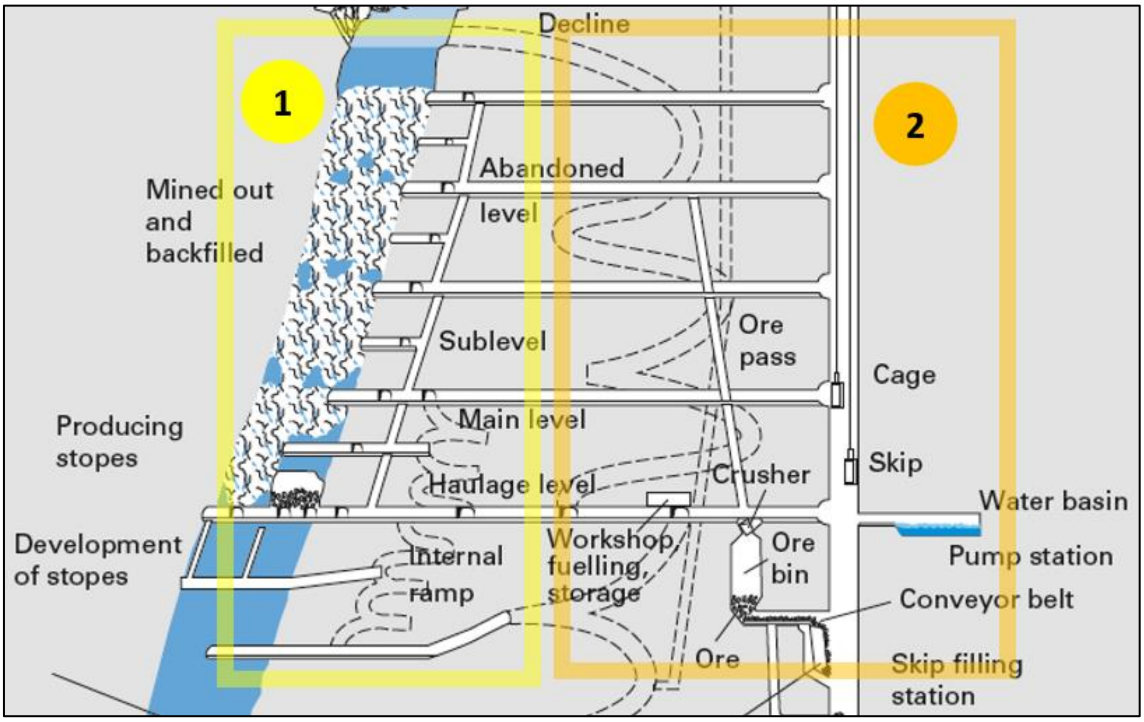

Figure 2.10. Typical underground mining access: (1) high-stress concentration area with short-medium term life, (2) low-stress concentration area with medium-long term life (Modified after COPCO (2007))

2.5. Ground behaviour

Deep underground openings with different geometries are excavated in a ground that can vary concerning geological units, in situ stress, underground water and overburden, and with uncertainties in the parameters, all or some of which may be problematic to the rock engineer. Rock mass behaviour in underground mines provides a first iteration for selecting appropriate design parameters. The design method in underground openings can be different for tension, pressure or torsion states due to the different type's ground behaviour. Additionally, potential failure and ground behaviour are used as critical parameters for developing geotechnical designs (Goricki, 2013). The choice of support system in an underground space project should not be based only on the rock mass classification rating; an understanding of the ground condition and failure mechanism in the rock mass structure is also required.

Investigation of geology in rock engineering should not only involve detail examination of drill cores but also consider information from the overall geological environment. Faults are one type of large geological structure that commonly formed by shear action because of significant movements from a few centimetres to tens or hundreds of kilometres, and can be horizontal, vertical or inclined at any angle. Site investigation is a part of the preliminary design of an underground opening and has a crucial role in the accuracy of the design.

An instability condition in a rock mass as a complex material causes failure in an underground opening. It depends on a variety of factors such as the effect of stress, underground water condition and rock mass composition. Stille and Palmström (2008) represented ground modes in underground excavations, as shown in Table 2.1.

Generally, the main reasons for instability in underground openings are gravity, stress and underground water condition. Block falls, cave-in and ravelling ground can occur due to the gravity condition. Also, stress may be the main reason for a rock burst occurring and flowing ground is probably because of an underground water influence. Some minerals and rock materials have a crucial influence on the ground behaviour of underground openings. The parallel orientation of sheet minerals such as mica, chlorite, and amphiboles is usually detected in sedimentary rocks, and these weakness planes can influence rock properties and behave strangely. The swelling clay minerals like montmorillonite can cause alteration or weathering of rocks and declining shear strength in a rock mass due to water being rich in oxygen (Palmstrom and Stille, 2010). Therefore, investigation of special minerals in the surrounding rock mass is a necessity for the prediction of ground modes such as swelling.

Table 2.1. Ground behaviour modes in underground openings (Stille and Palmström, 2008)

Ground behaviour type	Definition	Comments
Type 1: Gravity driven		
a. Stable	The surrounding ground will stand unsupported for several days or longer	Massive, durable rocks at low and moderate depths
b. Block fall(s)		
Of single blocks	Stable with potential fall of individual blocks	Discontinuity controlled failure
Of several blocks	Stable with potential fall of several blocks (slide volume < 10 m^3)	
c. Cave-in	Inward, quick movement of larger volumes (>10 m^3) of rock fragments or pieces	Encountered in highly jointed or crushed rock
d. Running ground	A particulate material quickly invades the tunnel until a stable slope is formed at the face. Stand-up time is zero or nearly zero	Examples are clean medium to coarse sands and gravels above groundwater level
Type 2: Stress induced		
Brittle behaviour		
e. Buckling	Breaking out of fragments in tunnel surface	Occurs in anisotropic, hard, brittle rock under sufficiently high load due to deflection of the rock structure
f. Rupturing from stresses	Gradually breaking up into pieces, flakes, or fragments in the tunnel surface	The time dependent effect of slabbing or rock burst from redistribution of stresses
g. Slabbing	Sudden, violent detachment of thin rock slabs from sides or roof	Moderate to high overstressing of massive hard, brittle rock Includes popping or spalling[a]
h. Rock burst	Much more violent than slabbing and involves considerably larger volumes (Heavy rock bursting often registers as a seismic event)	Very high overstressing of massive hard, brittle rock
i. Plastic behaviour (initial)	Initial deformations caused by shear failures in combination with discontinuity and caused by overstressing	Takes place in plastic (deformable) rock from overstressing. Often the start of squeezing
Plastic behaviour		
j. Squeezing	Time dependent deformation, essentially associated with creep caused by overstressing. Deformations may terminate during construction or continue over a long period	Overstressed plastic, massive rocks and materials with a high percentage of micaceous minerals or of clay minerals with a low swelling capacity
Type 3: Water influenced		
Hydratization		
k. Ravelling from slaking	Ground breaks gradually up into pieces, flakes, or fragments.	Disintegration (slaking) of some moderately coherent and friable materials. Examples: mudstones and stiff, fissured clays
Swelling minerals		
l. Swelling		
Of certain rocks	Advance of surrounding ground into the tunnel due to expansion caused by water adsorption. The process may sometimes be mistaken for squeezing	Occurs in swelling of rocks, in which anhydrite, halite (rock salt) and swelling clay minerals, such as smectite (montmorillonite) constitute a significant portion
Of certain clay seams or fillings	Swelling of clay seams caused by adsorption of water. This leads to loosening of blocks and reduced shear strength of clay	The swelling takes place in seams having fillings of swelling clay minerals (smectite, montmorillonite)
m. Flowing ground	A mixture of water and solids quickly invades the tunnel from all sides, including the invert	May occur in tunnels below groundwater table in particulate materials with little or no coherence (and clay)
n. Water ingress	Pressurized water invades the excavation through channels or openings in rocks	May occur in porous and soluble rocks, or along significant openings or channels in fractures or joints

[a] This term was often used by Terzaghi (1946) as synonymous with the falling out of individual blocks, primarily as a result of damage during excavation.

Determination of the main aspects related to the development of underground opening projects is necessary. The useful parameters are the size and shape of the openings, access possibilities, topography, groundwater condition, durability (related to installed rock support and maintenance) and serviceability (requirements the owner imposes on the project), behaviour types of failure or instability, safety level, lifetime of the project and the cost.

In order to distinguish behaviour modes, evaluation of rock mass composition is an essential step. The evaluation of rock mass behaviour is done by using two classification and characterisation methods in underground openings. MARINOS (2012) proposed a diagram to determine rock behaviour based on rock mass structures, and this is reproduced in Figure 2.11. Rock mass structure is divided into intact/massive, blocky, disturbed and foliated in this classification. Also, consideration and identification of failure mechanisms such as squeezing and unravelling at different depths and intact rock strength conditions (low σ_c and high stress σ_s) provide a determination of engineering geological behaviour. Moreover, the case studies on which this diagram is based were less than 500 m in depth and had a rock strength to about 100 MPa. It should be mentioned that these case studies cannot be used for brittle failures such as spalling and rock bursts at very considerable depths. However, complex properties of the ground condition cannot be appropriately demonstrated by a single expression and, it is necessary to group items with the same features or properties into one class or category and define classification criteria. Geological features such as foliation and faults in underground spaces affect ground behaviour (Schubert, 2013). Weak structure zones can influence the severity of ground modes such as squeezing and rock bursts. Consequently, distinguishing and planning to manage effective factors in severe failure modes is necessary for rock engineering.

Increasing deep grounds in mining and underground spaces may lead to violent failures in the form of rock bursts due to releasing a large amount of seismic energy. Excavation-induced stresses in deep underground structures commonly exceed the rock mass strength particularly surrounding the excavation and result in brittle failure by extension fracturing (Kaiser et al., 2015). Rock burst failure is classified into five types: buckling, strain burst, face crush/pillar burst, fault-slip burst and shear rupture. Strain burst can usually be mining-induced because of changing static stress near mining operations or increasing dynamic stress by a remote seismic event. Rock bulking can occur due to remote seismic events and the bursting event itself (Cai, 2013). Deep underground mining methods in highly stressed grounds can generate new fractures and instability behaviour near excavation of openings. Rock behaviour is directly associated with brittle rock failure and a fracturing process. Numerical simulation is a beneficial method for studying rock failure mechanism by using several numerical codes such as UDEC (Noorani and Cai, 2015). In order to use numerical methods in rock engineering, the accuracy of input data is vital.

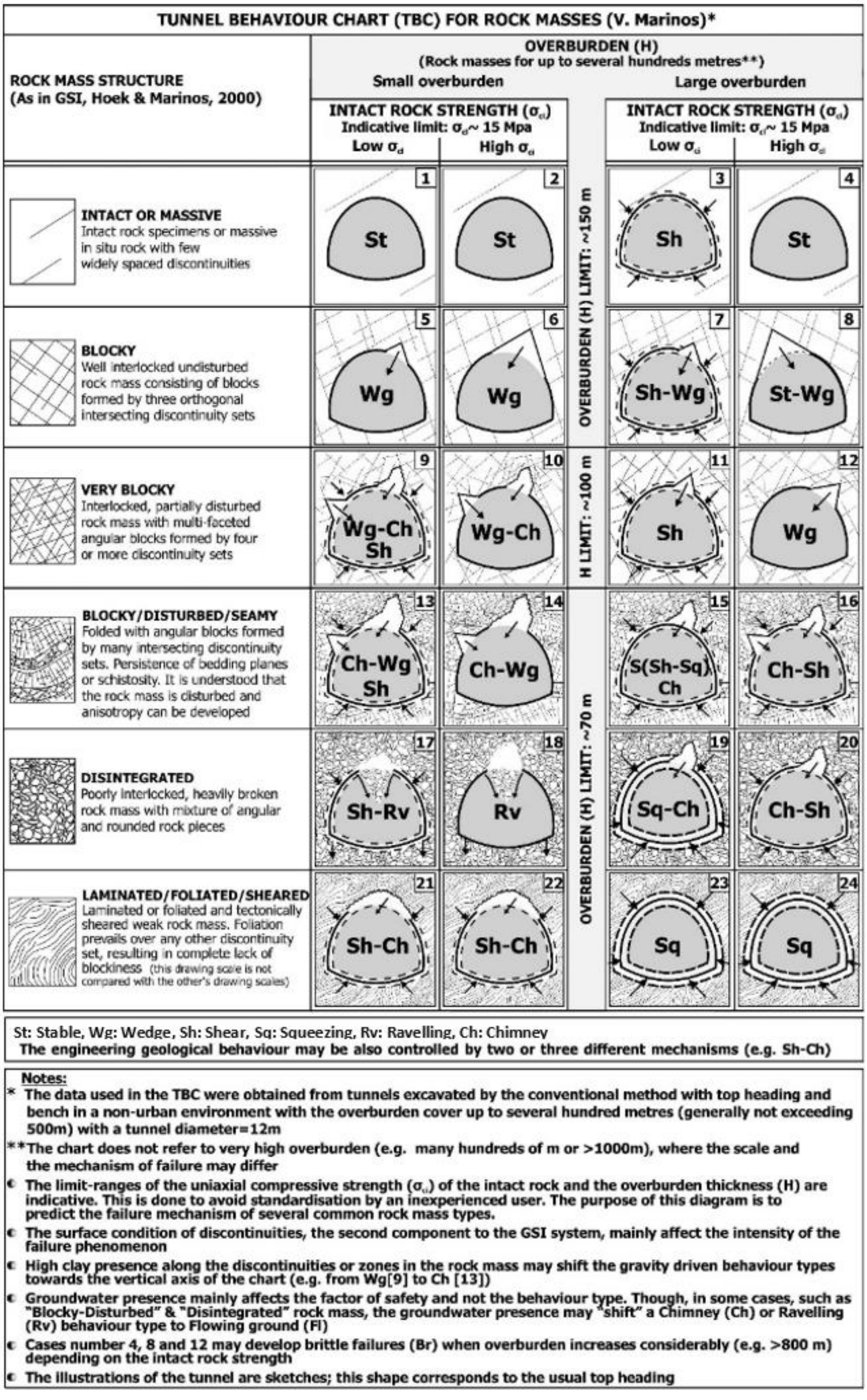

Figure 2.11. Distinguish of ground behaviour in underground openings (Modified after MARINOS (2012))

The intensity of rockburst damage is minor, moderate and major, and is often categorised by the depth of failure and the qualitative types of rock burst. Support devices for different modes of rockburst damage can be selected based on their reinforcement, retaining and holding functions. Application of reinforcement, like rock bolts and cables, to the rock mass is required to avoid rock falls. Wire mesh, strap, reinforcement shotcrete, cast concrete and steel arches are used as retaining elements, and tie retaining elements of the support system in the rock mass are defined as a holding function. The rock burst damage mechanisms and their severity and required support functions has been presented in Figure 2.12. Fracture, seismic and stress are some of the significant reasons for rockburst damage in underground excavations.

Rock materials are involved complex and uncertainty parameters, and sometimes make an estimation of ground behaviour difficult. However, the most critical factors are discontinuity properties, weakness zones such as fault and shear zones, rock mass strength, underground water condition and in situ stress, and the size and shape of excavation, which influence ground mode behaviour.

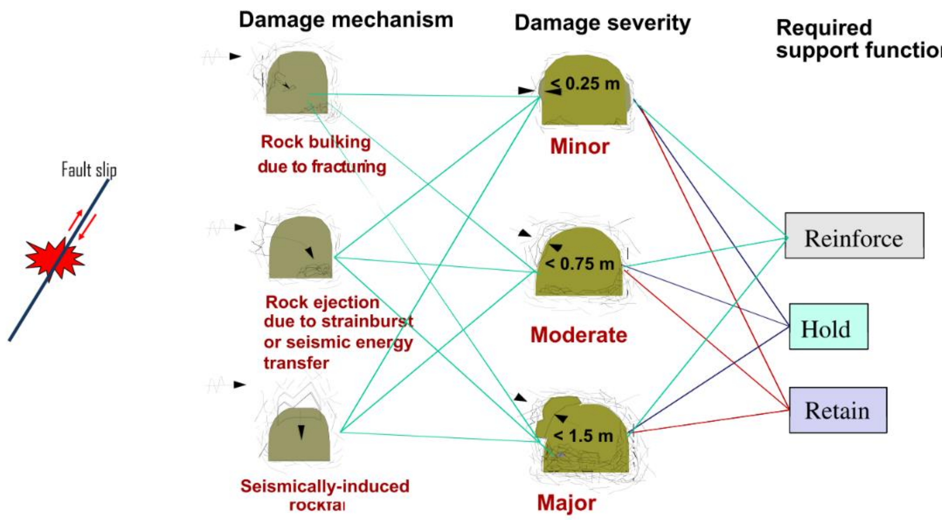

Figure 2.12. Different types of rockburst damage, severity and necessity support functions (Cai, 2013)

2.6. Failure mechanism

The excavation of underground openings changes stress and water conditions around rock mass structures, and the ground mode may alter the rock mass structure and its failure. The process of failure in rock is determined by crack initiation, crack propagation and coalescence and plays a key role in considering micromechanics of rock in long-term strength (Shao and Li, 2015). The behaviour of brittle rock is usually divided into five stages in the stress-strain curves: (1) crack closure, (2) linear elastic deformation, (3) crack initiation and its stable growth, (4) release of critical energy, and unstable crack and deformation growth and (5) failure and post-peak behaviour (Shao & Li, 2015). Different processes of rock mass failure may be involved around underground openings under different ground behaviour conditions.

Rock mass behaviour is relatively complex and lack of enough knowledge about in situ stress distribution, properties of a jointed rock mass, effects of time, blasting damage and water increase the level of uncertainty in underground openings (Szwedzicki, 2003). Australian Mining and Civil (AMC) proposed a classification for damage due to stress in Australian mines, as shown in Figure 2.13. In this classification, damage levels are divided into six classes from S0 (no visible damage) to S5 (extreme damage to excavation). It defines depth of damage, area of damage, and ground control methods (such as support systems) for every level, and can be a guideline in considering ground behaviour in an underground opening.

The instability and failure condition in hard rock can be different from that in soft rock. Figure 2.14 represents a categorisation of instability modes in hard rock at shallow to deep depth in underground openings (Kaiser et al., 2000). To evaluate an instability condition, the rock mass rating has been divided into three parts: massive with RMR > 75, moderately fractured where 50 > RMR > 75, and highly fractured with RMR < 50. Also, instability types have been predicted based on the ratio between maximum far-field stress (σ_1) and unconfined compressive strength (σ_c).

Rock failure is due to the creation, growth and accumulating of microcracks. The conventional approaches for modelling of rock failure are the linear Mohr-Coulomb failure criterion and the nonlinear Hoek-Brown failure criterion, both of which assume simultaneous cohesion and normal-stress dependent friction mobilisation. Figure 2.15 indicates a schematic compression test with cohesion loss and frictional mobilisation components. The growth of microcracks and change to the shear plane lead to mobilisation of frictional strength and the initial cohesion (ci) lowering to the residual value (cr). The stages of the failure process are: (I) the closure of microcracks and linear elastic deformation, (II) crack initiation, (III) peak strength, and (IV) failure. ε_c^p and ε_f^p are the plastic strain components in the ultimate state of the frictional and cohesive strength components. At the early stage of brittle failure, the cohesion component is dominant, but cohesion loss is the prevailing factor of weakening in the failure process. The tensile cracking gradually leads to destruction of the cohesion component of strength (Hajiabdolmajid et al., 2002).

Controlling displacement and rock failure in the rock mass surrounding an underground opening is a necessity. Microcracks inside rock are initiated by elevated stress levels at depth and behaviour of the rock mass during the failure process can change from a continuum condition (intact to moderately fractured) to a discontinuous state, such as slabbing or spalling ground mode (Hajiabdolmajid and Kaiser, 2003). The process of brittle failure in rocks from a geomechanical view is indicated in Figure 2.16. Loading or stress on the rock causes the destruction of bonds and cohesive strength between grains, extension of microcracks, mobilisation of frictional strength and consequently creation of a new surface (shearing plane) inside the rock. Figure 2.16(a) indicates that tensile cracks can occur in the rock even with confining stress. The ε_c^p and ε_f^p parameters are the plasttic strain for cohesion loss and friction components, respectively.

Damage Level	General Description	Rock Mass/Tunnel Damage	Examples
S0	No visible damage (Low stress conditions)	**Description:** No stress-induced damage visible **Depth of Damage (m):** 0m indicated depth of damage **Area of Damage (% of drive profile):** 0% of drive profile affected **Ground Control:** Easily controlled with minimal support eg split sets and mesh	
S1	Minor damage (spalling)	**Description:** Superficial damage only, easily scaled back to good rock **Depth of Damage (m):** 0m to 0.2m indicated depth of damage **Area of Damage (% of drive profile):** <10% of drive profile affected **Ground Control:** Easily controlled with minimal support eg split sets and mesh	
S2	Moderate damage (or spalling)	**Description:** Spalling clearly developed and more widespread in walls and backs **Depth of Damage (m):** Indications of damage/loosening to up to 0.5m depth into walls or backs (≈10% of wall or back span*) **Area of Damage (% of drive profile):** 10% to 50% of profile affected **Ground Control** Minor rehabilitation required in high utilisation excavations	
S3	Significant damage to excavations.	**Description:** Damage evident in all excavation surfaces. 'Bagging' in the mesh clearly developed; Shearing on foliation/bedding clearly indicated, isolated split set head failures **Depth of Damage (m):** Indications of damage/loosening to a depth up to 1.5m (≈30% of wall or back span*) **Area of Damage (% of drive profile):** >50% of profile affected **Ground Control:** Significant rehabilitation effort required to maintain safe access	
S4	Severe damage to excavations.	**Description:** Severe damage with up to 2m wall or backs to floor convergence and/or significant floor heave. Passable on foot, with extreme caution, but serviceability significantly reduced. Many bolts broken in shear, mesh severely bagged, some local rockfalls **Depth of Damage (m):** Indications of damage/loosening to a depth of greater than 1.5m but less than 4.0m(≈50% of wall or back span*) **Area of Damage (% of drive profile):** >80% of profile affected **Ground Control:** Limit of rehabilitation with conventional support	
S5	Extreme damage to excavations. Opening collapsed	**Description:** Widespread support failure and large rockfalls (>1000 tonnes) and in some cases complete or nearly complete drive closure. **Depth of Damage (m):** Indications of damage/loosening to a depth greater than 4m (≈100% of wall or back span*) **Area of Damage (% of drive profile):** 100% of profile affected **Ground Control:** Access not advisable, Beyond rehabilitation	

Figure 2.13. Classification of damage due to stress in an underground opening, by AMC in Australian (Sandy et al., 2010)

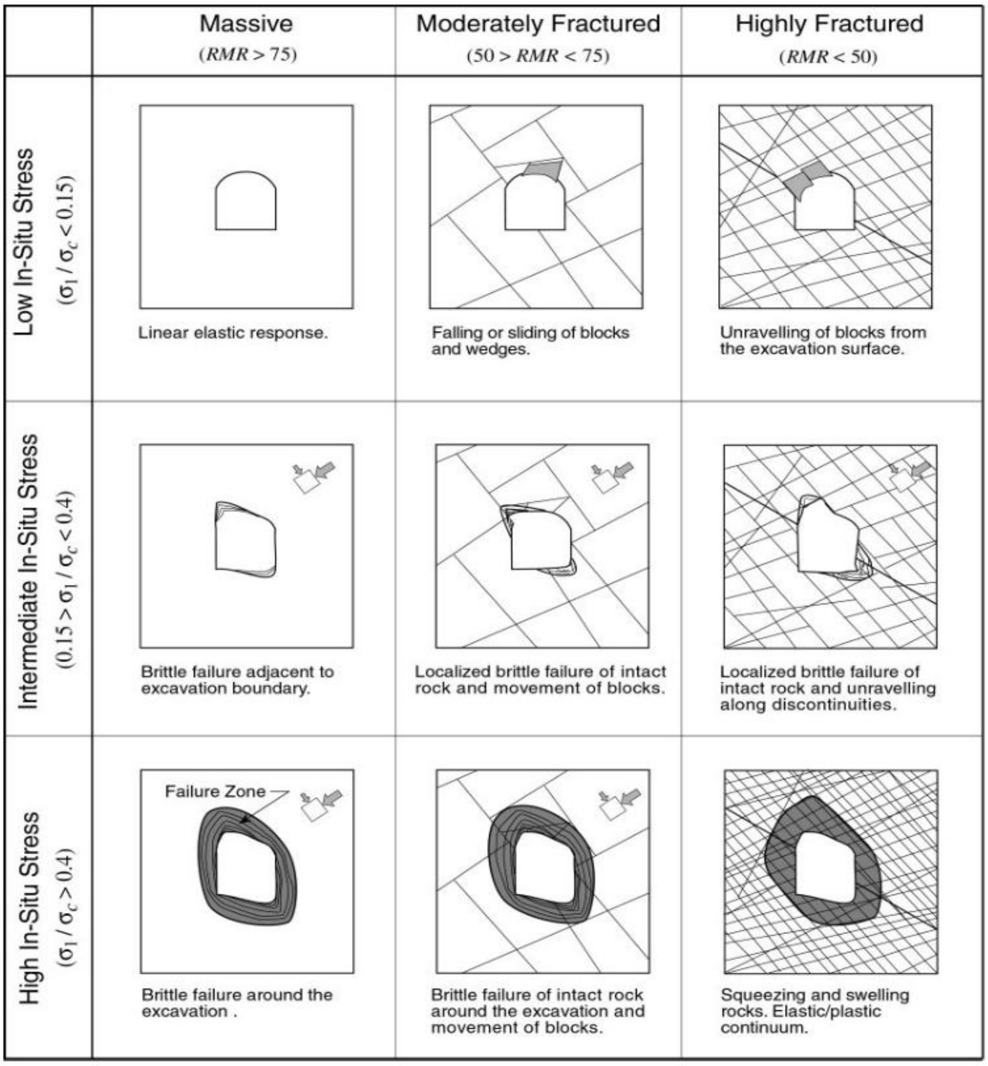

Figure 2.14. Different modes of rock failures in an underground opening (Kaiser et al., 2000)

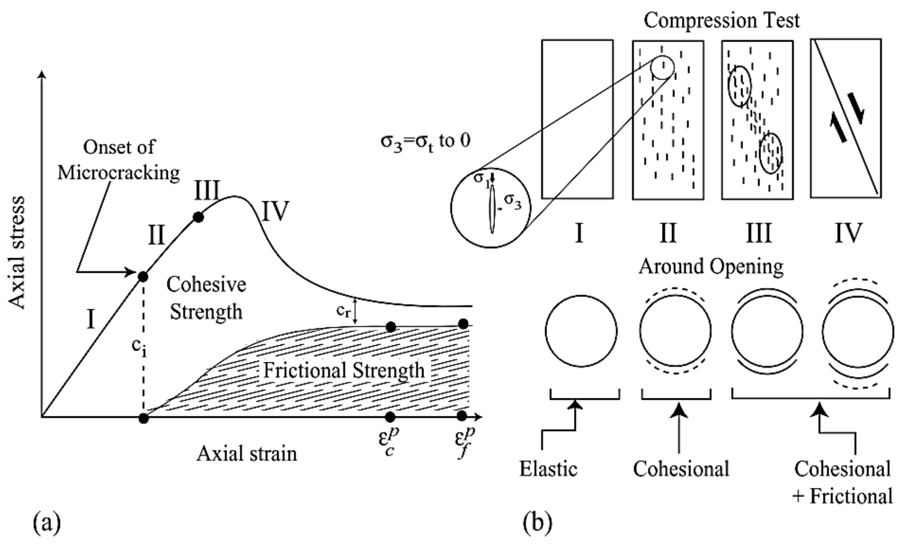

Figure 2.15. Strength component in CWFS model: (a) laboratory test, (b) around the underground structure (Hajiabdolmajid et al., 2002)

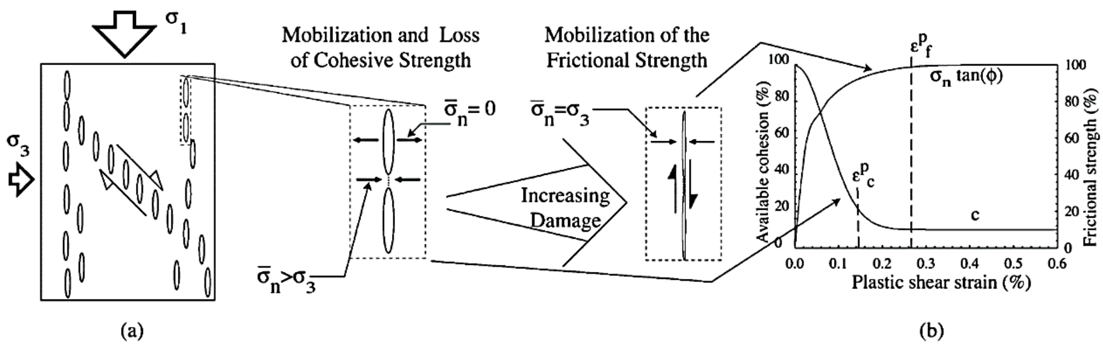

Figure 2.16. The mechanism of failure process in the rock (Hajiabdolmajid and Kaiser, 2003)

There are different kinds of failure modes in deep underground mines because of different geomechanical factors such as weakness of roof layers, in situ stress and hydrology. Some of the most important failure modes in deep underground mining are considered in follow.

2.6.1. Rock fall/ground fall

Rock fall is one of the common failures in most underground mines. The significant factors affecting rock fall failure in underground openings include the following (Corbett et al., 2014):

1. Changing in situ stress conditions and stress environment due to depth, geological composition and mining operations.
2. Naturally varying strata strength because of changing geological structure and stratigraphy, etc.
3. Changing the installation of strata support regarding quantity or quality.

Rock falls in underground coal mines, where only rock bolts are used as the means of roof reinforcement, occur because of cavities of different sizes and shapes in the roof. This type of failure is divided into two types: skin falls, that means failure of rock strata is between two adjacent bolts, and entry falls that are related to cavities higher than the bolting horizon (Peng, 2007). Figure 2.17 and Figure 2.18 present examples of this type of failure. Generally, the shape of this failure is a notch, and if it is not predicted and controlled, the depth of damage failure or collapse will increase.

Figure 2.17. Roof fall in a mine (Peng, 2007)

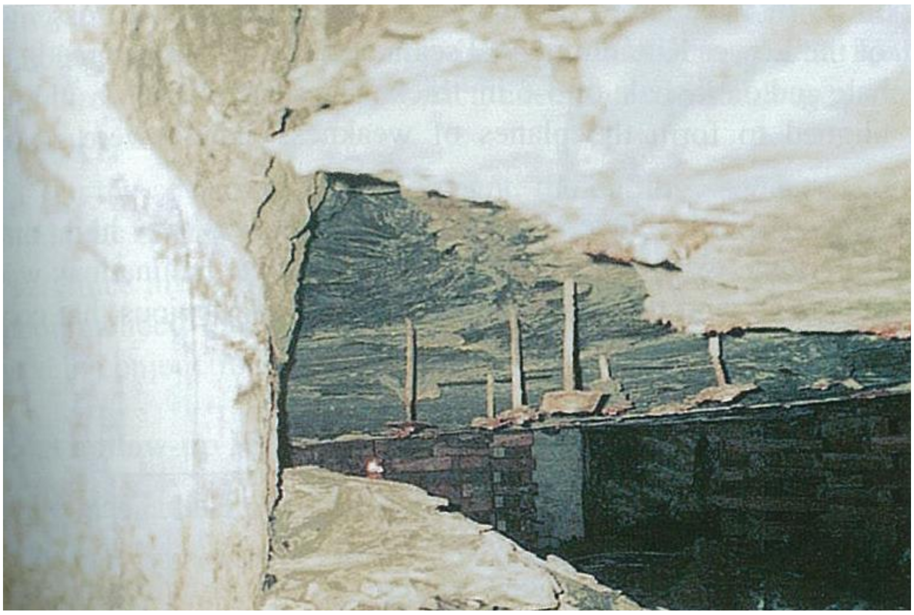

Figure 2.18. View of a roof fall (Peng, 2007)

2.6.2. Rockburst

The trend to deep underground mines (in-depth mining projects) leads to induced stresses in the rock mass and may cause the rock to fail violently or from a rock burst (seismic event) because of a sudden release of its stored energy. Sudden and intensive failure of rock is called rockburst damage. This phenomenon is closely related to hard rocks and geological structures like dykes and faults and also in mining projects associated with high extraction ratios and mining methods that create unsuitable stress conditions (Kaiser and Cai, 2012). The choice of suitable mining methods and support design diminish rock burst risk. However, there are uncertainties in rock mass properties and surrounding ground conditions such as fault zone distribution and in situ stress that justify rock engineers relying on ground control measures to design proper rock support and reinforcement to ensure workplace or stope safety. In burst-prone ground, suitable support systems should be installed before developing an excavation (Kaiser and Cai, 2012). Designing rock support in burst conditions is different from traditional rock support that mainly targets the control of gravity of rock falls and shallow zones of loose rock. In burst ground conditions, it is necessary to endure dynamic loads and large deformations because of rock dilation (bulking) during intensive failure of rock.

Rock burst damage and its intensity can be induced by many factors such as geology and geotechnical issues. There are a variety of factors associated with rock bursts such as rock mass strength, excavation method, the shape of an opening, earthquake, rock blasting, geological structures of a rock mass, field stress condition and groundwater condition (Liu et al., 2013). Figure 2.19 illustrates some significant factors of effective rockburst damage. This figure has categorised the factors into four groups namely, seismic, geology, geotechnical and mining. The intensity of dynamic loading is related to seismic events and geotechnical and mining factors are determined by rock mass properties and mining activities (Kaiser and Cai, 2012). Mining factors such as extraction ratio, excavation span and support system have an impact on the rock burst failure. Thus, the estimation of rock engineering parameters and selecting appropriate mining methods regarding the orientation of excavation and geometric parameters may result in diminished failure.

Figure 2.20 shows the different mechanics of rockburst failure. Rock bulking due to fracturing, rock ejection because of seismic energy release and rock falls derived by seismic shaking are significant factors of rockburst damage. The more significant challenges in a rock burst condition are to identify relevance and the mechanism of potential failure in a rock structure and energy release, especially in fault and weakness zones.

Seismic event	Geology	Geotechnical	Mining
•Event magnitude •Rate of seismic energy release •Distance to seismic source	•In situ stress •Rock type •Bedding •Geological structures (dykes, faults, and shears •Orientation o major geological structures	•Rock strenght •Joint fabric •Rock brittleness •Loading factor (rock mass strength/maximum principal stress) •Groundwater ocndition •Temperature	•Mining induced static and dynamic stresses •Excavation span •Extraction ratio •Mining stiffness •Excavation sequence (stress-path), blsting •Installed rock support system •Backfill •Production rate

Figure 2.19. Essential factors in rockburst behaviour (Modified after Kaiser and Cai (2012))

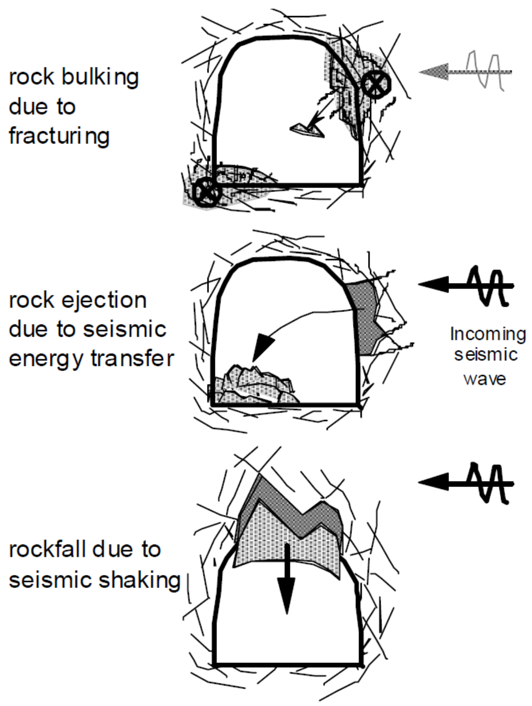

Figure 2.20. Rock burst behaviour (Kaiser et al., 1996)

2.6.3. Squeezing and large deformation

An underground opening under challenging conditions such as high stress is usually associated with squeezing and swelling ground modes. Squeezing ground behaviour is identified by large-scale deformation not only in a weak rock mass condition, but it is also observed in hard rock with high stress. This is a significant concern during the construction and conservation of underground mining openings (Mercier-Langevin and Hadjigeorgiou, 2011). The deformation of span openings reach tens of centimetres in squeezing ground conditions. In this type of ground condition, it is not generally acceptable to stop the deformation, and control and management of ground movement should be concentrated on (Woolley and Andrews, 2015). The evaluation of rock mass stability is the primary concern with squeezing problems due to deformations of rock masses over a long time.

The examination of squeezing phenomena in mines shows that most of them are associated with geological features such as shear zones and faults, high-stress conditions, fracture sets or joint sets, and severe foliation. Also, intact rock strength in the host rock type is relatively weak due to the presence of localised alteration minerals such as chlorite and mica in this type of ground behaviour mode. Some geological structures such as bedding and foliation and its thickness layer are related to the orientation of the large-scale deformation in squeezing behaviour (Mercier-Langevin and Hadjigeorgiou, 2011). A classification of squeezing ground conditions in underground openings based on foliation thickness has been presented in Table 2.2. Heavy and moderate squeezing categories have been presented for two different ground conditions. One of the most important factors in this classification is the difference in uniaxial compressive strength.

Table 2.2. Classification of squeezing ground mode based on thickness of foliation (Mercier-Langevin and Hadjigeorgiou, 2011)

Category	Category 1	Category 2
Squeezing level	Heavy squeezing	Moderate squeezing
Rock layers	Thin (10 s of millimetres)	Thick (10 s of centimetres)
Uniaxial compressive strength	Approximately 10 MPa	Approximately 50 MPa
Maximum rate of convergence	100 s of mm per month	10 s mm per month
Depth of broken rock mass	Up to 6 m	Up to 3 m

2.7. Ground control management

Ground control or management is a process to predict and manage rock mass behaviour and failure conditions in underground openings. Ground control is initially concerned with ground conditions and instability issues. Underground fatality statistics in Western Australia show that 37 percent of all fatalities between 1980 and 1994 were caused by rock falls (Lang, 1995).

In the past 30 years, the position of ground control or management has been established in modern mine design, and it is the sole significant factor in assessing the success or failure of a mine operation (Peng, 2007). Knowledge of the ground condition plays a considerable role in pillar design, support design for underground mines, slope stability, dam design, instrumentation of mining operations, roof controls and so on.

Ground control methods can be considered in both initial design and modification of design during mining operations. The three main approaches to this issue (ground control design) are experience-based design, management based design and technically based design. Experience-based design is defined using the previous empirical experience of ground control. The term management is related to mine management and the use of empirical knowledge of ground control with economic considerations. Using professional rock mass principles in ground control is called technically based design (Swindells, 1992). There is a significant difference between the application of ground control designs and validation of models at the laboratory scale, and real mine operation conditions due to coverage of wide areas, varying site conditions, an uncertainty of influencing factors and their value. It should be mentioned that traditional methods for evaluation of ground control models were used far more in the past. For example, back analysis has been a popular method in recent years and one or more parameters such as entry convergence, which has already been measured at a specific location in a mine and a certain time, were selected to validate the design or model. In the latter case, if the output of the model accommodates the measured value, the model is applicable. The results of real and practical projects during the past 30 years have shown that the back analysis method is still undoubtedly the best method for validating ground control design and models. However, there are very complex multiple factors that influence ground stability and mining operations.

The performance of the ground support system around an excavation surface is measured in order to evaluate the ground control design.

Furthermore, Lang (1995) illustrated five main processes for ground control management in underground openings:

1. Collection of data
2. Analysis and design, including probabilistic design
3. Implementation

4. Quality control
5. Instrumentation, monitoring and review

Geotechnical issues and ground control management should be considered during the whole life of underground opening projects from the feasibility study stage to the final closure of a mine. Geological structures usually influence ground control. Therefore, mining engineers, supervisors and underground workforces should have a good understanding of this on a local scale or even a smaller scale from less than a metre to some tens of metres in stopes. Regarding groundwater conditions, geotechnical characterisation of the geological structure by systematic and standard methods can be useful for this purpose. Changing geological structures during the development of an underground opening need to be distinguished earlier and the rock support and reinforcement modified if required. Using appropriate excavation methods, sequences and rates, the damage zone in a rock mass can be reduced. Shape, size and orientation of underground openings influence the potential instability. It is necessary to design rock support and reinforcement for an underground structure based on the ground condition.

2.7.1. Grond support classifications

Understanding potential hazards provide necessary information for the design of support and reinforcement in underground mining projects. Several empirical rock support classifications have been proposed based on ground type behaviour. Suitable methods for excavation and support systems have been given in Figure 2.21 for ground type modes. Understanding the probability of failure helps to avoid damage and collapse in the rock mass surrounding an underground construction. Sometimes, it is not possible to predict and identify the failure before its occurrence in an underground opening due to a high level of uncertainty in the rock mass parameters. Therefore, it may collapse due to a variety of reasons such as shear failure, landslides and fault.

Knowledge of stress conditions and rock mass strength provide critical factors to estimate types of ground behaviour in underground openings. Designing and selecting an appropriate type of rock support system is one of the essential tasks of rock engineers.

S. No.	Ground conditions	Excavation method	Type of support	Precautions
1	Self-supporting/ competent	TBM or full face drill and controlled blast	No support or spot bolting with a thin layer of shotcrete to prevent widening of joints	Look out for localized wedge/shear zone; past experience discourages use of TBM if geological conditions change frequently
2	Non-squeezing/ incompetent	Full face drill and controlled blast by boomers	Flexible support; shotcrete and pre-tensioned rock bolt supports of required capacity; steel fiber reinforced shotcrete (SFRS) may or may not be required	First layer of shotcrete should be applied after some delay but within the stand-up time to release the strain energy of rock mass
3	Raveling	Heading and bench; drill and blast manually	Steel support with struts/pre-tensioned rock bolts with SFRS	Expect heavy loads including side pressure
4	Minor squeezing	Heading and bench; drill and blast	Full column grouted rock anchors and SFRS; floor to be shotcreted to complete a support ring	Install support after each blast; circular shape is ideal; side pressure is expected; do not have a long heading, which delays completion of support ring
5	Severe squeezing	Heading and bench; drill and blast	Flexible support; full-column grouted highly ductile rock anchors and SFRS; floor bolting to avoid floor heaving and to develop a reinforced rock frame; in case of steel ribs, these should be installed and embedded in shotcrete to withstand high support pressure	Install support after each blast; increase the tunnel diameter to absorb desirable closure; circular shape is ideal; side pressure is expected; instrumentation is essential

Figure 2.21. Guideline for excavation method and rock support system based on ground behaviour in underground openings (Singh, 2011)

6	Very severe squeezing and extreme squeezing	Heading and bench in small tunnels and multiple drift method in large tunnels; use forepoling if stand-up time is low	Very flexible support; full-column grouted highly ductile rock anchors and thick SFRS; yielding steel ribs with struts when shotcrete fails repeatedly; steel ribs may be used to supplement shotcrete to withstand high support pressure; close ring by erecting invert support; encase steel ribs in shotcrete, floor bolting to avoid floor heaving; sometimes steel ribs with loose backfill are also used to release the strain energy in a controlled manner (tunnel closure of more than 4% will not be permitted)	Increase the tunnel diameter to absorb desirable closure; provide invert support as early as possible to mobilize full support capacity; long-term instrumentation is essential; circular shape is ideal
7	Swelling	Full face or heading and bench; drill and blast	Full-column grouted rock anchors with SFRS shall be used all around the tunnel; increase 30% thickness of shotcrete due to weak bond of the shotcrete with rock mass; erect invert strut; the first layer of shotcrete is sprayed immediately to prevent ingress of moisture into rock mass	Increase the tunnel diameter to absorb the expected closure; prevent exposure of swelling minerals to moisture, monitor tunnel closure
8	Running and flowing	Multiple drift with forepoles; grouting of the ground is essential; shield tunneling may be used in soil conditions; realign the tunnel	Full-column grouted rock anchors and SFRS; concrete lining up to face, steel liner in exceptional cases with shield tunneling; use probe hole to discharge ground-water; face should also be grouted, bolted, and shotcreted	Progress is very slow; trained crew should be deployed; in reach of sudden flooding, the tunnel is realigned by-passing the same, if ground is not groutable; monitor rate of flow of seepage
9	Rock burst	Full face drill and blast	Fiber reinforced shotcrete with full-column resin anchors immediately after excavation	Micro-seismic monitoring is essential

Figure 2.21. Guideline for excavation method and rock support system based on ground behaviour in underground openings (Singh, 2011)

Hoek (2006) proposed a guideline for support requirements based on rock mass description and behaviour, as shown in Figure 2.22. This is a basis to describe a rock mass, its type of behaviour and also the requirements for the support system to create safe conditions in an underground opening.

Rock mass description	*Rock mass behaviour*	*Support requirements*	*Shotcrete application*
Massive metamorphic or igneous rock . Low stress conditions.	No spalling, slabbing or failure.	None.	None.
Massive sedimentary rock. Low stress conditions.	Surfaces of some shales, siltstones, or claystones may slake as a result of moisture content change.	Sealing surface to prevent slaking.	Apply 25 mm thickness of plain shotcrete to permanent surfaces as soon as possible after excavation. Repair shotcrete damage due to blasting.
Massive rock with single wide fault or shear zone.	Fault gouge may be weak and erodible and may cause stability problems in adjacent jointed rock.	Provision of support and surface sealing in vicinity of weak fault of shear zone.	Remove weak material to a depth equal to width of fault or shear zone and grout rebar into adjacent sound rock. Weldmesh can be used if required to provide temporary rockfall support. Fill void with plain shotcrete. Extend steel fibre reinforced shotcrete laterally for at least width of gouge zone.
Massive metamorphic or igneous rock. High stress conditions.	Surface slabbing, spalling and possible rockburst damage.	Retention of broken rock and control of rock mass dilation.	Apply 50 mm shotcrete over weldmesh anchored behind bolt faceplates, or apply 50 mm of steel fibre reinforced shotcrete on rock and install rockbolts with faceplates; then apply second 25 mm shotcrete layer. Extend shotcrete application down sidewalls where required.
Massive sedimentary rock. High stress conditions.	Surface slabbing, spalling and possible squeezing in shales and soft rocks.	Retention of broken rock and control of squeezing.	Apply 75 mm layer of fibre reinforced shotcrete directly on clean rock. Rockbolts or dowels are also needed for additional support.
Metamorphic or igneous rock with a few widely spaced joints. Low stress conditions.	Potential for wedges or blocks to fall or slide due to gravity loading.	Provision of support in addition to that available from rockbolts or cables.	Apply 50 mm of steel fibre reinforced shotcrete to rock surfaces on which joint traces are exposed.
Sedimentary rock with a few widely spaced bedding planes and joints. Low stress conditions.	Potential for wedges or blocks to fall or slide due to gravity loading. Bedding plane exposures may deteriorate in time.	Provision of support in addition to that available from rockbolts or cables. Sealing of weak bedding plane exposures.	Apply 50 mm of steel fibre reinforced shotcrete on rock surface on which discontinuity traces are exposed, with particular attention to bedding plane traces.
Jointed metamorphic or igneous rock. High stress conditions.	Combined structural and stress controlled failures around opening boundary.	Retention of broken rock and control of rock mass dilation.	Apply 75 mm plain shotcrete over weldmesh anchored behind bolt faceplates or apply 75 mm of steel fibre reinforced shotcrete on rock, install rockbolts with faceplates and then apply second 25 mm shotcrete layer Thicker shotcrete layers may be required at high stress concentrations.

Figure 2.22. A guideline for support requirement in different ground condition (Hoek, 2006)

Bedded and jointed weak sedimentary rock. High stress conditions.	Slabbing, spalling and possibly squeezing.	Control of rock mass failure and squeezing.	Apply 75 mm of steel fibre reinforced shotcrete to clean rock surfaces as soon as possible, install rockbolts, with faceplates, through shotcrete, apply second 75 mm shotcrete layer.
Highly jointed metamorphic or igneous rock. Low stress conditions.	Ravelling of small wedges and blocks defined by intersecting joints.	Prevention of progressive ravelling.	Apply 50 mm of steel fibre reinforced shotcrete on clean rock surface in roof of excavation. Rockbolts or dowels may be needed for additional support for large blocks.
Highly jointed and bedded sedimentary rock. Low stress conditions.	Bed separation in wide span excavations and ravelling of bedding traces in inclined faces.	Control of bed separation and ravelling.	Rockbolts or dowels required to control bed separation. Apply 75 mm of fibre reinforced shotcrete to bedding plane traces before bolting.
Heavily jointed igneous or metamorphic rock, conglomerates or cemented rockfill. High stress conditions.	Squeezing and 'plastic' flow of rock mass around opening.	Control of rock mass failure and dilation.	Apply 100 mm of steel fibre reinforced shotcrete as soon as possible and install rockbolts, with face-plates, through shotcrete. Apply additional 50 mm of shotcrete if required. Extend support down sidewalls if necessary.
Heavily jointed sedimentary rock with clay coated surfaces. High stress conditions.	Squeezing and 'plastic' flow of rock mass around opening. Clay rich rocks may swell.	Control of rock mass failure and dilation.	Apply 50 mm of steel fibre reinforced shotcrete as soon as possible, install lattice girders or light steel sets, with invert struts where required, then more steel fibre reinforced shotcrete to cover sets or girders. Forepoling or spiling may be required to stabilise face ahead of excavation. Gaps may be left in final shotcrete to allow for movement resulting from squeezing or swelling. Gap should be closed once opening is stable.
Mild rockburst conditions in massive rock subjected to high stress conditions.	Spalling, slabbing and mild rockbursts.	Retention of broken rock and control of failure propagation.	Apply 50 to 100 mm of shotcrete over mesh or cable lacing which is firmly attached to the rock surface by means of yielding rockbolts or cablebolts.

Figure 2.22. A guideline for support requirement in different ground condition (Hoek, 2006)

The stability of excavation of an underground opening in very weak rock is associated with faults and shear zones and requires using an appropriate support system such as rock bolts, shotcrete, and fore-poles. Early prediction of unstable locations in fault and shear zones can play a crucial role in the overall process of support design. Fault zones in underground space projects cause changes to ground behaviour and groundwater conditions frequently. It is clear that the rock mass and its behaviour is diagnosed during the design stage. Then, the excavation method and the types of support or reinforcement for the rock mass are evaluated (Hoek, 2006). The significant parameters for underground structures in fault zones with squeezing potential are rock mass strength, initial stress conditions and deformation characteristics. Based on these criteria, a schematic diagram of the sequence of support decision on a site with squeezing behaviour has been shown in Figure 2.23. Generally, there are four steps to the support decision as shown in this diagram: (1) modify UCS based on joint orientation, (2) adjust the UCS using underground water condition, (3) determine stress factor with regard to overburden, and (4) take support decision based on behaviour type and support category.

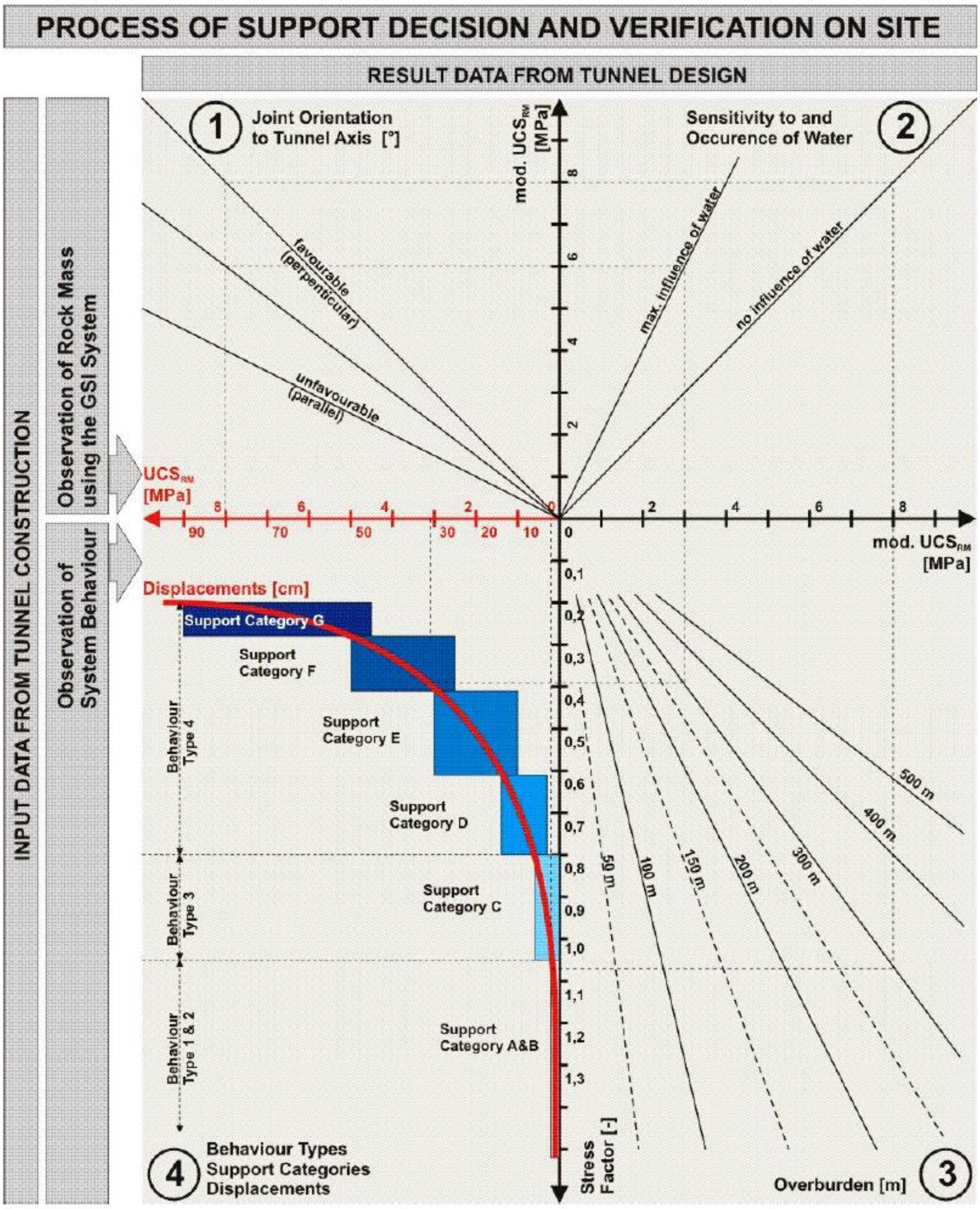

Figure 2.23. Schematic process of the sequence support decision in underground openings (Hoek, 2006)

The Q-System of the rock mass classification was proposed in 1974 as below (Barton et al., 1974):

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (2.1)$$

Where, RQD: Deere's Rock Quality Designation≥10cm, Jn: Joint set number, Jr: Joint roughness number, Ja: joint alteration number, Jw: joint water reduction factor, and SRF: stress reduction factor.

The Q system is used for characterisation of rock masses, and suggestion of a preliminary ground support system in tunnel designs (see Figure 2.24).

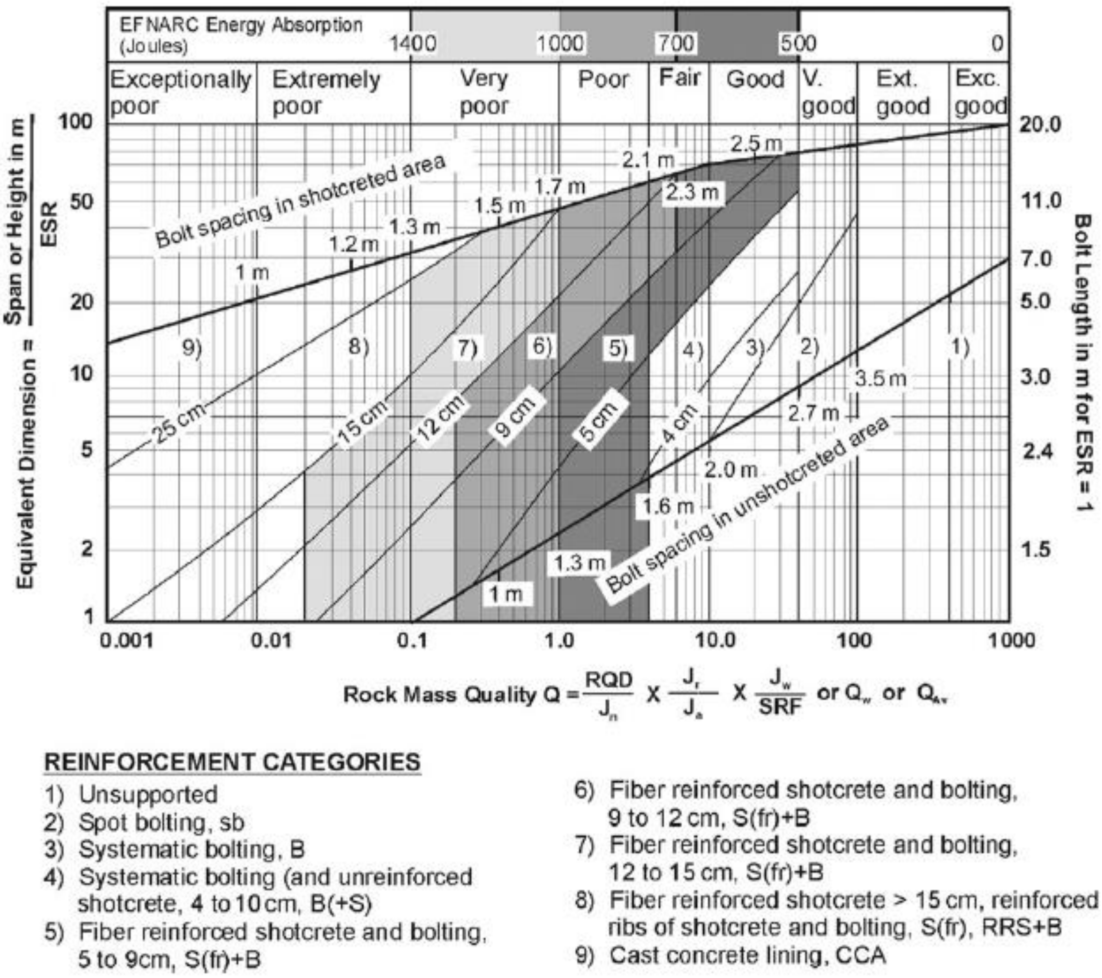

Figure 2.24. Preliminary suggested support system based on the Q system for underground excavations (Grimstad, 1993)

Classification methods are usually quick and straightforward tools for the estimation of the required rock support system based on ground conditions. Failure mode, field stress condition and time, which are significant factors in the design of a rock support pattern, are not considered quantitatively. However, all of them are based on previous projects and are useful to rock engineers as checks in the design process.

2.7.2. Rock support and reinforcement system in underground mines

The main principles of rock support are to provide a steady persistence in an unstable rock mass and also to reduce rock deformation by a certain amount in order to prevent immature failure. A support system in underground space projects should be able to assist the rock mass in supporting itself and to improve ground conditions by building a unified ground structure (Cai et al., 2015). The requirement for the properties of a support system in high-stress conditions consists of strength, elastic stiffness but plastic deformability, and reliable anchors. Strong support can resist a high load to restrict deformation of a rock mass. Quick reaction and deformability to control further rock deformation is a fundamental characteristic of an elastically stiff but plastically deformable support system, and reliable anchorage requires an integration of the support system and the rock mass under harsh conditions. Energy absorption capacity can be a better parameter than strength in the evaluation of the efficiency of a support system. At present, a tendency is to develop an energy-absorbent factor having strong and deformable parameters in support devices (Li, 2015). A variety of supports like concrete lining, mesh, shotcrete, rock bolts and cables are used in underground spaces in mining and civil engineering.

Ground support systems are specified based on ground behaviour modes and stress conditions in underground excavations. For example, in low-stress rock masses, which are potentially unstable, shotcrete and mesh are used for support, while in high-stress rock masses under dynamic loading and seismic events that may lead to the release of strain energy and sudden failure, yielding rock bolts and steel sets are used to provide stable conditions. Therefore, by considering the performance of ground support systems based on the load-deformation factor, the appropriate devices are selected for the rock mass zone around an excavation. Having supports which have high strength, high deformability as well as being energy–absorbent, makes them useful and beneficial to deal with stress-induced rock instabilities (Li, 2015). Yielding rock bolts are developed by engineers in the mining field, while in the civil engineering, particularly in tunnelling projects, more focus is given to improving yielding support surface devices. Therefore, with regards to aims and types of projects, the requirement of a support system can differ even in the same ground conditions in underground openings.

Rock bolts are a kind of support device which is grouped under rock reinforcement systems. Support usually provides a confining surface for the ground by installing structural elements, while reinforcement enhances the behaviour of the ground by the installation of structural elements within the ground. Rock reinforcement systems are flexible and useful in a variety of rock conditions. Reinforcement devices can be classified into three groups based on transferring element loads (Bobet and Einstein, 2011):

- Continuously Mechanically Coupled (CMC)
- Continuously Frictionally Coupled (CFC)
- Discretely Mechanically or Frictionally Coupled (DMFC)

A yield bolt is a type of support which consists of a smooth steel bar, and one or several anchors in the borehole and its performance can be represented by the static and dynamic load-displacement curves. In the mining industry, yielding rock bolts like cone bolt (Figure 2.25), the Garford bolt (Figure 2.26), D-bolt (Figure 2.27) and Yield-bolt (Figure 2.28) are used to deal with rockburst conditions (Li, 2015). Bolts and anchors are efficient support systems to control rockburst conditions in underground openings (He and Sousa, 2014). In high-stress conditions, to create a stable and safe rock mass, it is required to integrate rock support patterns and reinforcement systems in underground openings.

The response of a rock mass in high-stress conditions can be a form of rock burst in hard rock or squeezing in moderate or weak rock. Therefore, support devices can be deformable to avoid immature failure. There are many examples in underground projects in overstress conditions where external devices (supports) such as stiff steel sets failed, and internal devices (reinforcements) like rock bolts failed (Li, 2015).

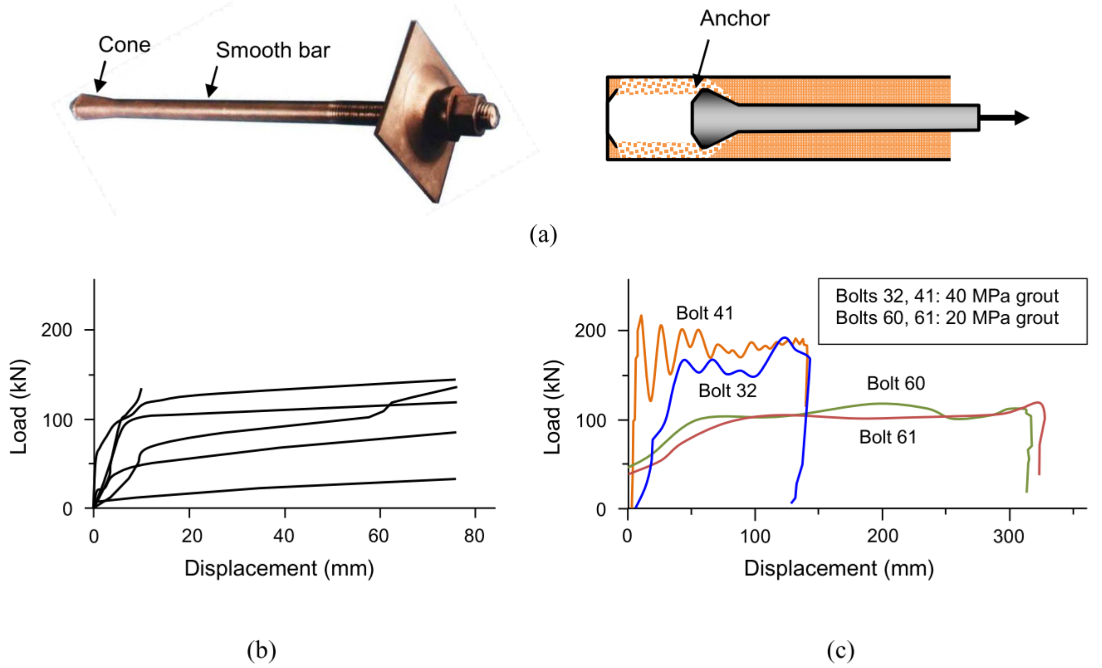

Figure 2.25. (a) Cone bolt, (b) statistic pull test results, (C) dynamic test result (Li, 2015)

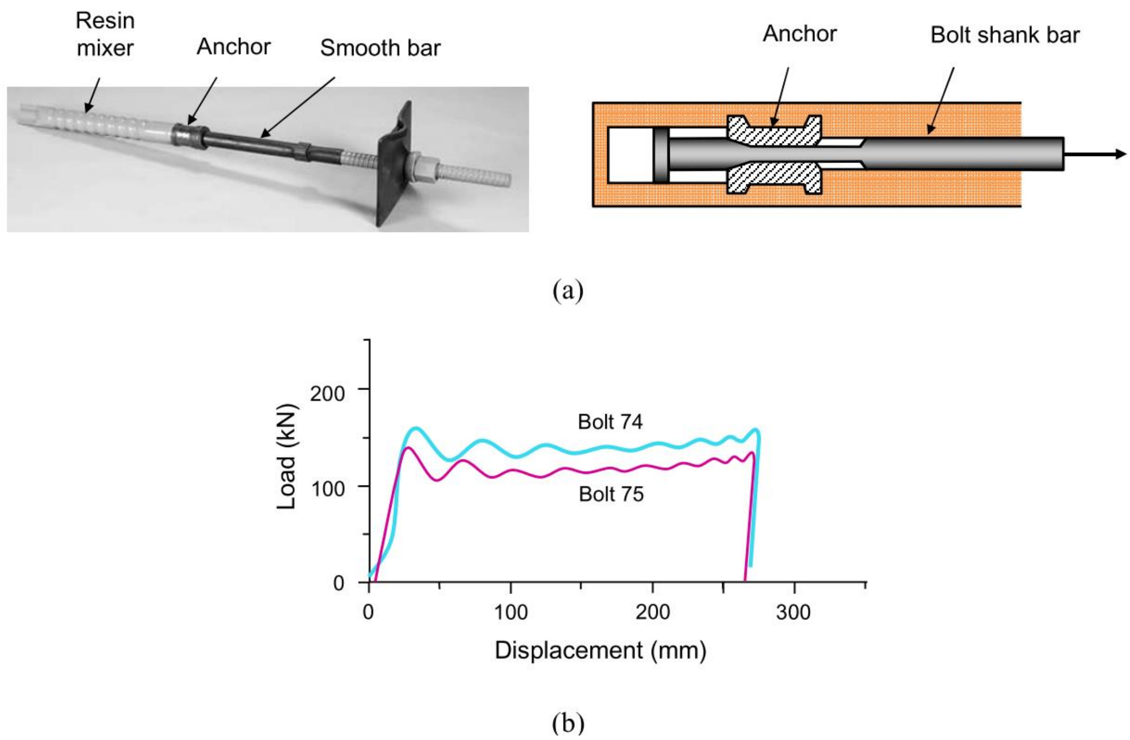

Figure 2.26. (a) Garford bolt, (b) dynamic test result (Li, 2015)

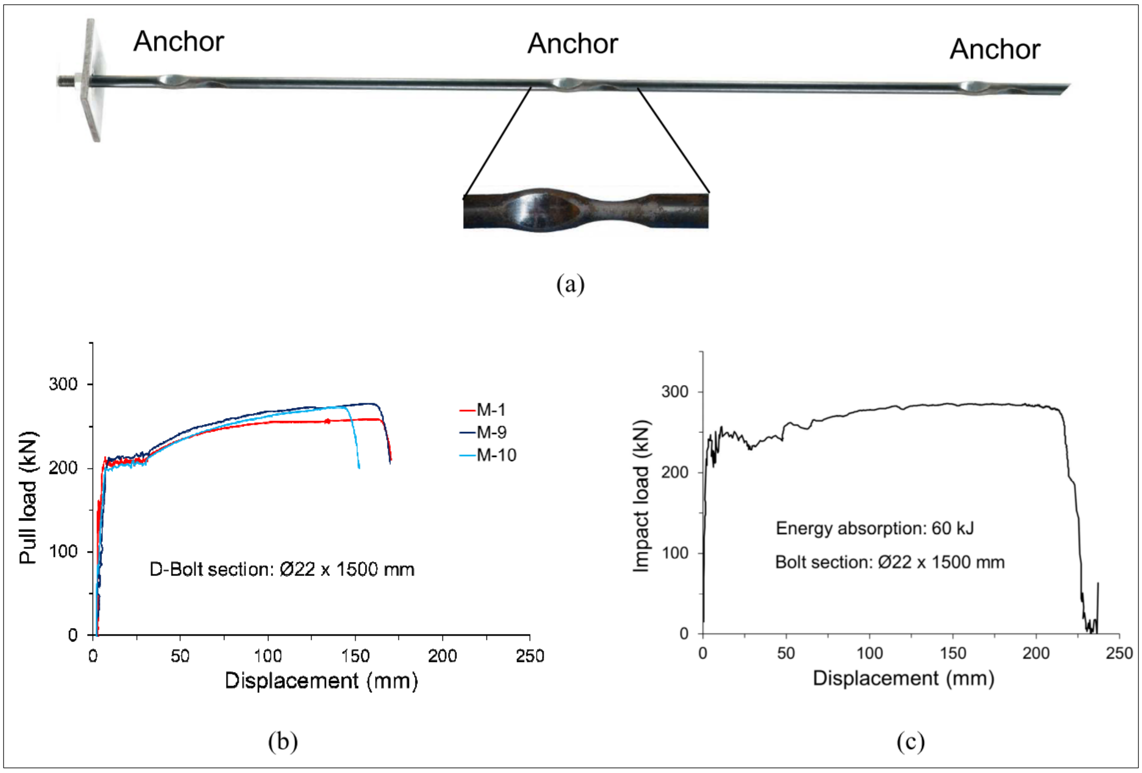

Figure 2.27. (a) D bolt, (b) statistic pull test result, (c) dynamic test result (Li, 2015)

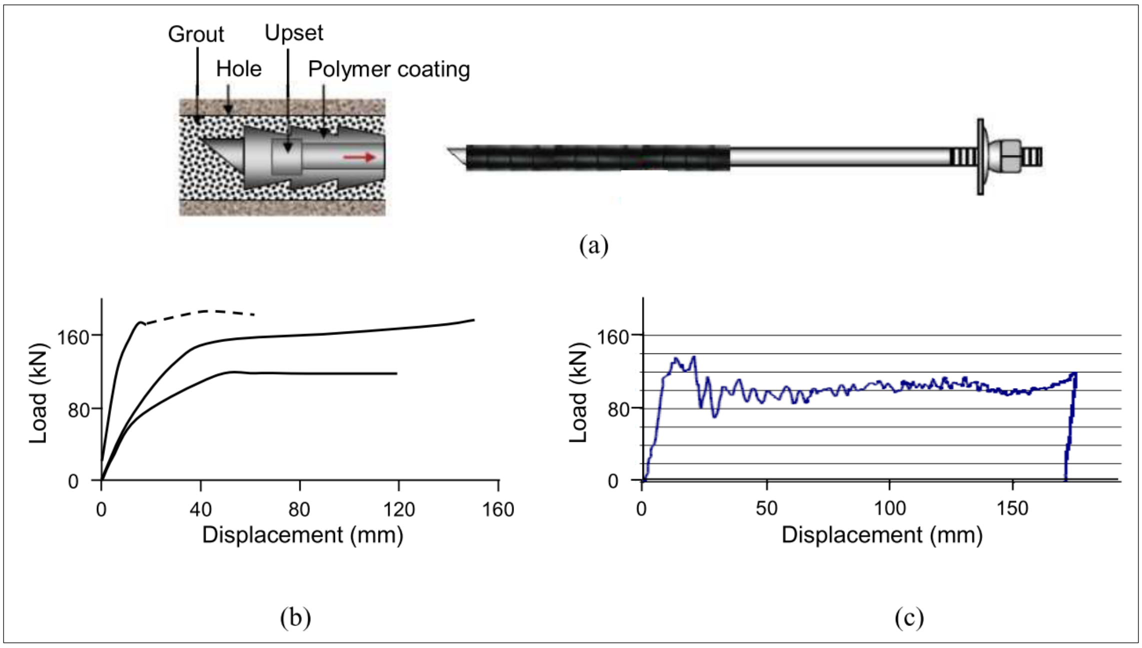

Figure 2.28. (a) Yield bolt, (b) a statistic pull test result, (c) a dynamic drop test result (Li, 2015)

Surface support can play a pivotal role to decrease rock fall failure in deep underground mines. Some parameters such as energy absorption, availability, capacity, installation cycle and costs need to be considered in designing and selecting from different types of surface support systems. The ability to resist high stresses, and flexibility to allow large displacements of excavation walls are the most critical factors (Louchnikov et al., 2014).

Rock bolts are used as reinforcement tools in rock mass structures to enhance strength, increasing stiffness of the rock mass against tension or shear forces (Srivastava and Singh, 2013). Stiff response for the elastic stage of loading and deformability during plastic conditions can be appropriate ground support system characteristics. Furthermore, seismic events and capability for energy absorption should be considered in dynamic loading states (Louchnikov et al., 2014). Surface supports can be divided into primary and secondary groups. Primary supports such as mesh or shotcrete maintain loose rocks and secondary supports, such as straps and cable, function to transfer load between rock bolts.

The potential for instability in the surrounding underground opening is essential for both human safety and equipment damage considerations. It is important to select suitable rock support and reinforcement to diminish risks. Since there is a wide range of rock engineering parameters and also uncertainty in ground conditions, which make it difficult for engineers doing the ground support design, diagnosing ground behaviour and failure mechanisms is an essential step to overcome this challenge during the selection of appropriate support tools.

2.7.3. Ground support design and methodologies

The typical ground mode behaviour in deep underground mines with hard rock and high-stress conditions is rock bursting. Therefore, rockburst support design and methodologies are focussed on in this part. However, this procedure is also able to take other types of ground conditions into account in the design of a support system and reinforcement. Three tactics are employed for the mechanics of rock support (Kaiser & Cai, 2012):

1. Strengthen and control bulking or rock mass by reinforcement
2. Prohibit fractured block failure and unravelling by retaining broken rock
3. For stable ground, hold fractured blocks and securely tie back the retaining element(s).

These principles are shown in Figure 2.29 as a schematic. Reinforcement of a rock mass enhances its strength and ability to support itself(Kaiser and Cai, 2012)(Kaiser and Cai, 2012).

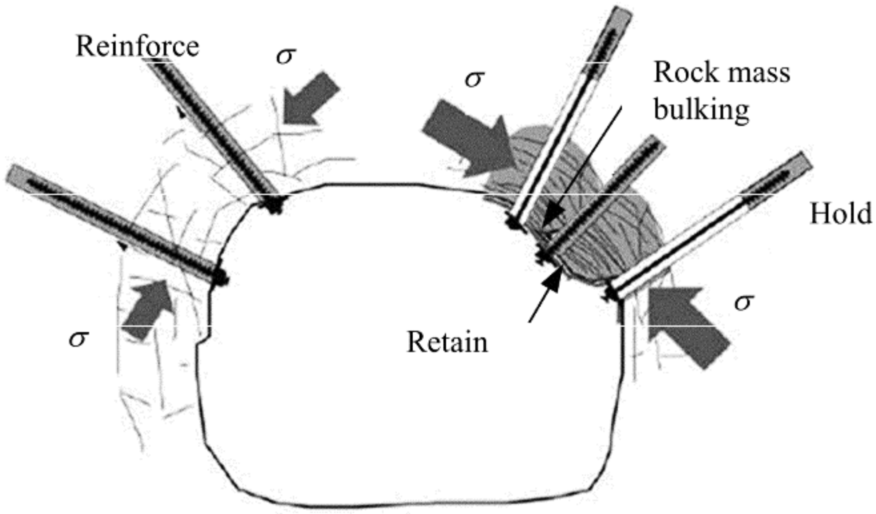

Figure 2.29. Reinforce, retain and hold are three critical functions of rock support (Kaiser and Cai, 2012)

Brittle rock fall is usually related to large rock mass bulking. A seismic event can influence rock via a significant release of energy. Therefore, the installation of a rock support system not only must be capable of absorbing dynamic energy but also to adapt to sizeable sudden rock deformation because of rock failure with bulking events. Tying retaining elements of a rock support system and providing a stable ground condition is related to the holding function. Yielding hold elements like high capacity friction bolts and cone bolts are used for rockburst damage cases (Kaiser and Cai, 2012). Knowledge and experience of using different tools in underground mine projects are essential to provide stable and safe conditions at the lowest cost. The principles of rock support design for rockburst conditions has been summarized in Figure 2.30:

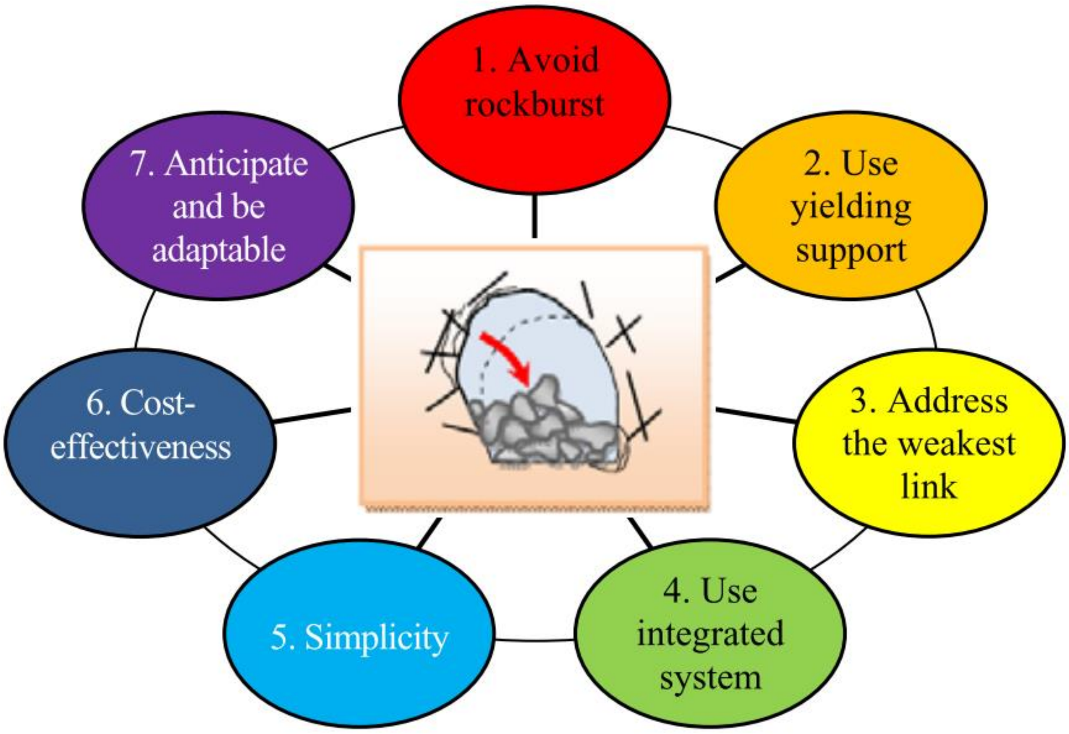

Figure 2.30. Principles of rock support design for rockburst behaviour (Kaiser and Cai, 2012)

The first point is to prevent rockburst conditions. Changing the underground opening location, using different excavation shapes, modifying the sequence of mining operations and methods, and changing the stope size and/or shape are some methods to avoid rock burst risks. The second principle is using a rock support system to allow deformability and absorb dynamic energy because brittle rock failure is usually associated with significant impact energy and rock dilation. The retaining elements are often the weakest links in traditional support systems, and the connection between them usually fails to result in considerable rockburst damage. Unfortunately, attention is given to only load and dissipation energy capacities of a rock bolt in rock support design procedures. Therefore, it is essential to match the strength of surface support elements and the capacity of the bolts. Rock support systems for holding need the combination of reinforcement elements, like rebars, and surface elements, such as mesh and shotcrete. It is essential that different support elements should be integrated. Simplicity, ease of production, installation and maintenance are the most critical factors for rock support elements. Mining companies seek to decrease costs in mining operations. Where required, support against rock burst events is expensive. Rock burst damage may reduce mine production for an extended period of time. Therefore, avoiding damage and controlling burst conditions can be cost-effective (Kaiser and Cai, 2012). The ability for prediction and adaptation is the last main factor in designing rock supports for burst conditions. This kind of ground behaviour has the potential to change regularly. Therefore, it is irrational to have a fixed design for rock support, and the design should be responsive to different ground conditions.

Avoiding dynamic loading and large rock bulking is an essential issue for rock support design in burst ground conditions, and it alters the type of design from traditional rock support. The capacity of rock support in this condition should be enough regarding load, displacement and energy dissipation. Generally, four criteria are assessed in rockburst support designs (Kaiser and Cai, 2012):

1- Force criteria: This covers the design of rock support for both dynamic and static loads. A deformability characteristic in rock support is used to dissipate some of the energy demand until the static requirement declines to below the support load capacity. The load factor of safety (FS_{Load}) is defined in equation (2.2):

$$\mathbf{FS_{Load}} = \frac{\textbf{Support load capacity}}{\textbf{Load demand}} \qquad \textbf{(2.2)}$$

2- Displacement criteria: stress exceeding the rock mass strength causes rock fracturing. The rock support system cannot prevent this event, and as a result, this may lead to bulking deformations. Therefore, the rock support system should have an adequate displacement capacity to encounter or exceed the required displacement. Equation (2.3) shows the displacement factor of safety:

$$\mathbf{FS_{Disp}} = \frac{\textbf{Support displacement capacity}}{\textbf{displacement demand}} \qquad \textbf{(2.3)}$$

3- Energy Criterion: During the occurrence of a rock fall, kinetic energy is released, potential energy changes and energy demand is increased. Therefore, the rock support's capacity to absorb energy should be at least as much as the energy released from the seismic events. The energy factor of safety (FS_{Energy}) has been defined in Equation (2.4):

$$\mathbf{FS_{Energy}} = \frac{\textbf{Support energy capacity}}{\textbf{Energy demand}} \qquad \textbf{(2.4)}$$

The ability of rock support to absorb kinetic energy in a rock containing mass (m), which is ejected from the roof with velocity (v_e) and capacity of displacement (ds), is calculated by equation (2. 5):

$$E = \frac{1}{2} m v_e^2 + m g d_s \qquad (2.5)$$

Where, g: gravitational acceleration.

4- System compatibility criterion: The previous three design criteria are aimed at designing reinforcement and support holding elements. A suitable support system is compatible with the rock deformation and load. Since it is difficult to calculate the requirement of surface support elements, empirical methods are usually applied. However, it is necessary to ensure compatibility of displacement, load and energy capacity of surface support with reinforcement/holding elements.

Therefore, it seems that the modern design of rock supports must evaluate the capability to absorb dynamic energy especially in seismic events in underground openings. It also needs to consider the integrity of different kinds of rock mass stabilisation. Also, the simplicity of installation, ability to adapt to changing ground conditions and optimisation cost are the other important factors for the assessment and selection of appropriate tools in underground excavations.

2.8. Monitoring

Instrumentation and monitoring are used to compare natural values against variable design parameters in rock mass structures, confirming the validity of assumptions in design procedures, checking the responses of rock masses, verifying the performance of a ground support system, and ensuring the safety of an underground excavation during and after construction. Monitoring of rock engineering parameters can be measured directly, such as displacements and the load in support devices, or indirectly, for example, seismic emission, stress at a point and a velocity of wave propagation.

The monitoring system should be selected regarding budget, reliability, ease of installation, measurement accuracy, suitability with environmental conditions, for example, humidity and corrosive condition, ease of reading and user-friendliness, and insignificant interference during the construction stage.

Instrumentation systems can be categorised into four groups (SIMRAC, 2002):

1. Optical systems: a type of photogrammetric surveying methods, which are simple and traditional methods for quick measurements of underground excavations by

borehole cameras and petroscopes to provide profiles of excavations and movements on boundaries.

2. Mechanical systems: include rod, wire, cable and tapes for measuring displacements. The method is simple, cheap and more reliable.
3. Hydraulic/pneumatic systems: based on the fluid pressure inside a flexible metal or plastic chamber to measure support loads and normal stress components.
4. Electrical devices: used for measuring strain and stress components, seismic events and displacements between two or more points. Harsh conditions in underground construction may cause electrical systems to fail.

The accuracy of the read parameter for monitoring systems is acceptable if it is within 1% of the actual value (SIMRAC, 2002). Also, there may be various reasons for some errors in the measurements recorded. Gross, systematic, conformance, environmental, observation, sampling and random are the usual error types in measurements. Inexperience, misreading, loss of calibration, weather, temperature, noise and environmental effects are some source of errors in the monitoring system. Some typical solutions to ameliorate the measurement errors are:

- Training
- Recalibration
- Multiple reading
- Usage standards
- Application of proper instrument devices
- Duplicate reading by observers

Achieving efficient operations in underground excavations requires optimal ground support systems to establish stability in ground conditions. Performance of ground support devices in static and dynamic loading, the field stress condition and seismic events are assessed by monitoring systems. A good monitoring system is associated with using all available information from seismic event sources, seismic loading, small and large-scale deformation in rock mass structures, and the induced stress field in field instrumentation. Installing different types of instruments at great depth and high-stress levels, where there is more potential for damage of these devices because of seismic events, multiple measurement systems like excavation deformation and seismic events are utilised for the evaluation of ground support performance.

On-site monitoring in underground mines is to assess ground conditions and performance of ground support components based on rock mass deformation, mining-induced stress changes, seismic events and activities. The critical process elements for field instrumentation and monitoring in high stress and in-depth underground mining projects are listed below (Zhang et al., 2016):

- Selection of a field site(s): selection of a suitable area (to satisfy installation of instruments to provide enough data) is carried out based on being protected from seismic mining operation activities for example blasting, being far from mechanical noise sources (for example mine ventilation fans), getting access to services like electricity, and access to recently developed underground openings to gather as much data as possible.
- Multiple-point borehole extensometers: can be used for deformation monitoring in rock mass structures surrounding excavations, especially ground deformation before and after seismic events.
- Laser-scanning: is a quick and straightforward monitoring method for surface deformation. After running an initial scan from a surface excavation, an appropriate time for subsequent scanning is after deformation of more than 10 mm is detected by multiple-point borehole extensometers.
- Instrumented rock bolts: are used for monitoring of the development of load-deformation along with rock bolts.
- Borehole observation and monitoring: surveying boreholes by a camera to distinguish discontinuity conditions and rock types during the mining activities.
- Local seismic systems: installation of seismic sensors near an excavation for evaluation of ground motion due to seismic events.
- Damage mapping: when a seismic event occurs, damage mapping should be carried out to assess immediate effects and assess possible solutions of rock mass responses to seismicity.
- Numerical modelling: detail support information for a local area can be evaluated with numerical modelling of static and dynamic loading, and also an unloading condition.

In underground mining projects with seismic activities, local seismic events are unforeseeable in time and space some typical methods like laboratory tests on cores, drop tests and simulated by numerical methods. In order to the achievement of a realistic assessment of ground support performance, an integrated site monitoring system is employed in mines (Zhang et al., 2016). Site monitoring program in mining projects is defined based on the rock mass deformation, mining-induced change and seismic activities. Laser scanning, local seismic monitoring system, damage mapping, surveying, and numerical modelling are some of the typical methods for quick measurements in underground mines. Monitoring data is collected in a regular time from active mining areas like access levels and ore drives. Then, the information is processed, interpreted and analysed. For example, an increase in the number of seismic events surrounding underground mining excavations is an important failure indicator. Back analysis method is used to estimate a critical deformation or induced stresses in rock masses for update design parameters and ground support systems.

Observation and monitoring methods can be used during the early stage or during the development of underground mining projects to acquire real ground behaviour and modify design parameters. Some of the benefits of monitoring and site observations are control of design uncertainties, achieving value-cost/time, reducing failure risk in rock mass structures and improving ground support systems.

2.9. Summary

Knowledge and understanding of rock mass properties and composition is a fundamental prerequisite of rock engineering projects to evaluate and design an underground excavation. Investigation and measurements on the site provide information of the intact rock, discontinuities, geology and in-situ conditions, which are used to describe the rock mass composition. Classification of rock mass composition into simple classes helps to identify the kind of behaviour of the rock mass surrounding an excavation. A systematisation of rock mass composition has been proposed as follows:

- Intact/massive rock: a few joints or almost no joints
- Jointed/blocky rock/bedded: few joints or very wide joint spacing
- Blocky/folded rock: slightly to strongly jointed

- Disintegrated/crushed/soil like: heavily jointed or crushed rock, fragments with no or little cohesion or bonding
- Special mineral: minerals in rocks with special properties, such as clay rocks

Nowadays, modern design in rock engineering fields is not possible without the evaluation of ground behaviour in underground openings. In other words, one of the main tasks in the design procedure is considering and identifying the behaviour of the ground surrounding underground openings. Evaluation of active stress factor (ratio between rock mass strength and maximum stress), underground water condition, excavation methods/underground mining methods, extraction ratio, and size and shape of the opening are the most important factors to recognise ground behaviour. Generally, there are three types of ground behaviours: (1) water influenced such as swelling, (2) gravity driven like block falls and (3) stress-induced, for example, rockburst. Also, failure mode and instability in rock engineering is related to ground behaviour. Therefore, it is a necessity to predict hazardous conditions and evaluate the failure mechanism in every type of ground mode. The sequence of the failure mechanism is divided into six phases:

1. Stable/elastic
2. Failure indicators
3. Ground movement
4. Primary precursors
5. Secondary precursors
6. Local damage / regional failure

In a stable or elastic condition, there is no serious potential for failure. This phase can be in hard rock conditions without any major discontinuities and faults in the site, and it can be favourable for rock engineering purposes. The second phase of failure is failure indicators. There are some warning signs and unfavourable engineering geology features such as faults, folds and joints, moisture, and unstable shapes that assist in distinguishing the potential of failure modes. The possible failures in this step can be plastic and brittle types. Also, the ground movement is also used as a hazard and failure indicator. Some important signs are surface cracking, crack opening, shear movement, vertical and horizontal displacement at the periphery of an opening. Tensile cracking, splitting, slabbing, shearing and buckling may happen in this state. If these failure symptoms were not observed, the initial process of failure could be witnessed during the primary precursors' phase, for example, seeing surface subsidence, or small rock falls. This step can occur months before collapse or local damage in an underground opening. It is clear that during the initial failure in rock masses, the intensity increases and this stage is called the secondary precursors' phase. Rock noises, flowing ground, changes in water flow and pillar yielding are some symptoms in a rock mass, which can be used to diagnose the risk of failure. The last phase of the failure mechanism is local damage or regional failure and is seen as a rock fall or rock burst or collapse.

Consideration and identification of the failure mechanism provide useful information to analyse the design for ground control and management by different methods such as empirical, numerical and soft computation methods. The design procedure in rock engineering determines the project layout, excavation or stope sequences and excavation rate, and the design procedure results in the selection of ground reinforcement and support system and the time-cost model. In the next step, it is necessary to start the construction project, which involves the process of excavation, depressurisation, quality control of material and installation of ground reinforcement and support devices. The last phase of the modern design procedure for underground openings is field measurement to monitor and update the design parameters.

In this research, a statistical analysis was done for comparison of the design procedure between previous underground mining projects. Seventy-seven real projects between 1973 and 2015 have been considered. All steps and phases were encoded as presented in Table 2.3. In total, 7 phases and 37 steps are defined in the modern design procedure.

The results of this comparison have been summarised in Figure 2.31. The minimum and the maximum number of stages used in the seventy-seven projects are 4 and 16, respectively. Also, a trend for the existing projects has been plotted. On average, the number of design stages in rock engineering projects has increased from 6 in 1973 to 12 in 2015. Therefore, progress in science and technology has resulted in more phases and stages in real underground opening projects in recent years. Furthermore, the result of statistical analysis for the number of phases used in underground openings has been given in Figure 2.32. According to this figure, no project used all seven phases in geomechanical procedures although three, four, five and six phases have been used.

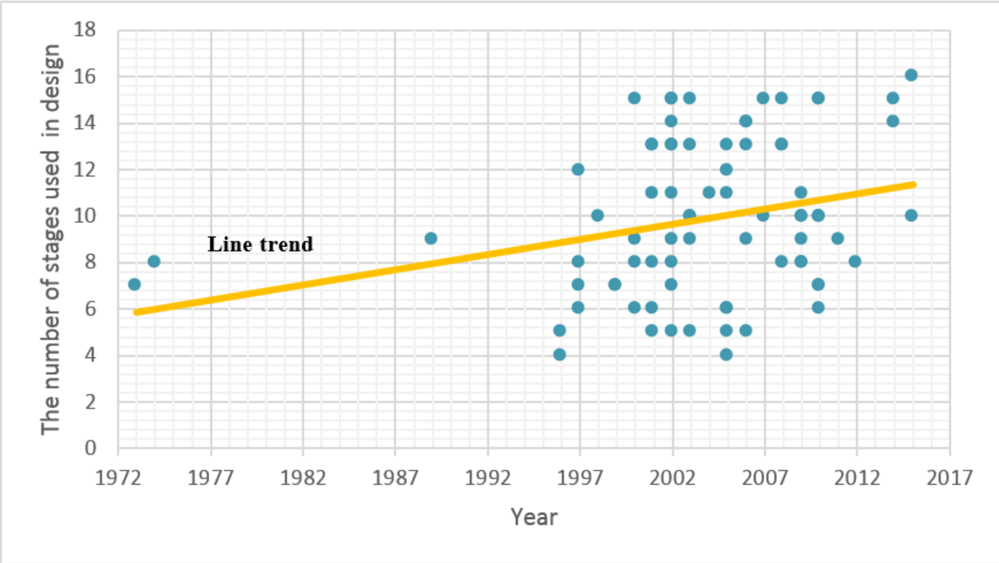

Figure 2.31. The number of stages used in design procedures in the rock engineering projects between 1973 and 2015.

Also, Figure 2.33 shows the result of the number of stages that were used in design procedures between the years 1973 and 2015. Comparison of the numbers in this figure indicates that in previous projects, there is no consideration of hazard recognition nor identification of failure mechanism phases. However, there are 11 related to stage 4.6 (local damage/regional failure) that were evaluated, only after construction and the occurrence of failure and collapse, through monitoring and field measurement. Therefore, there is a necessity to develop a fourth phase in the modern design procedure.

Table 2.3.Coding of Phases and steps of modern design procedure in underground opening

1.Rock Mass Composition /Rock Mass Description	1.1. Intact Rock
	1.2. Discontinuity Characteristics
	1.3. Geological condition
	1.4. In-situ Stress condition
	1.5. Intact / Massive rock
	1.6. Jointed / Blocky rock/Bedded
	1.7. Blocky / Folded rock
	1.8. Disintegrated / Crushed / Soil like
	1.9. Special Minerals
2.Diagnosis of Ground Behaviour	2.1. Active Stress
	2.2. Hydrogeological condition
	2.3. Critical features of an underground opening project
3.Ground description and Behaviour of Underground Openings	3.1. Water influenced
	3.2. Gravity-driven & Stress-induced
4.Hazard Recognition / Geotechnical Failure / Failure Mechanism	4.1. Stable / Elastic
	4.2. Failure indicators
	4.3. Ground movement
	4.4. Primary Precursors
	4.5. Secondary precursors
	4.6. Local damage / Regional failure
5.Design Analysis to Manage Ground Behaviour	5.1. Empirical methods
	5.2. Analytical methods
	5.3. Numerical methods
	5.4. Observation methods
	5.5. Engineering judgment
	5.6. Other methods- expert system, soft computation
	5.7. Project positioning
	5.8. Excavation / stopes sequences
	5.9. Excavation rate
	5.10. Ground reinforcement and support pattern
	5.11. Time-cost model/estimation of time and cost
6.Construction	6.1. Blasting, excavation and depletion phases
	6.2. Loosening/depressurization
	6.3. Quality control of material
	6.4. Ground reinforcement and support pattern
7.Field Measurement	7.1. Rapid Change: Direct application to construction
	7.2. Moderate Change: Data interpretation, Back Analysis

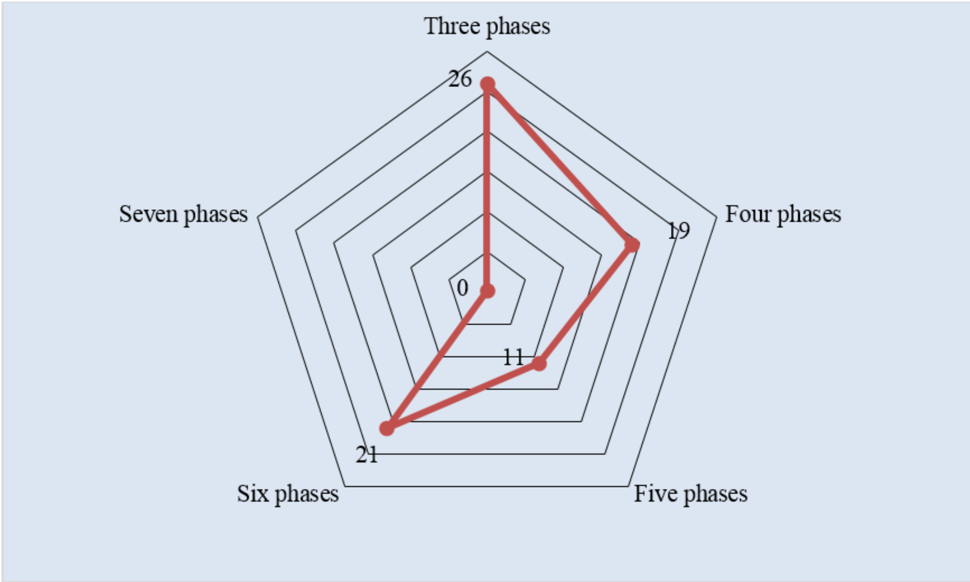

Figure 2.32. A comparison of the number of phases used in underground opening projects

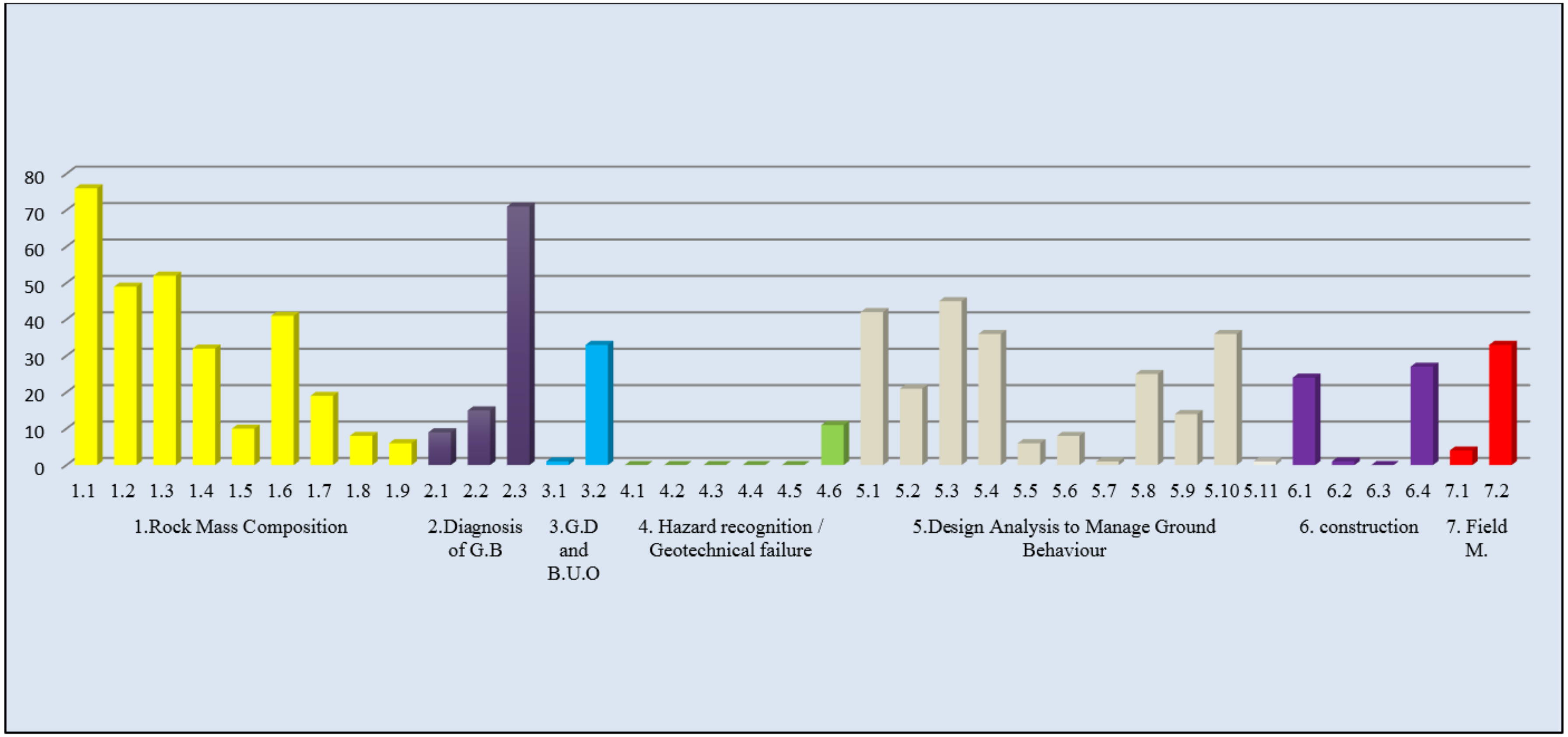

Figure 2.33. The number of stages used in design procedures between 1989 and 2015

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CHAPTER 3: GROUND CHARACTERISATION AT DEEP–HARD ROCKS MINES

CHAPTER 3: CONTENTS

3.1. Introduction

The ground characterisation is used to specify the inherent properties of the rock mass, which are measurements of the intact rock strength, natural fractures and discontinuities and their condition, to provide a context for rock mass classification. Rock mass characterisation provides an estimation of ore body geometry, rock mass properties which play a crucial role in stope design, stope dilution and indirectly ground support requirements.

Data collection techniques with geological and geotechnical mapping and core logging can be used to link information to describe rock mass composition. The characterisation is an attempt to describe geological features and rock engineering parameters. The most important parameters of rocks are strength, deformability, weathering, durability, groundwater condition, permeability, in situ stress condition (magnitude and orientation), and condition of discontinuities (persistence, spacing, orientation, aperture, roughness, infilling etc.).

This chapter presents a procedure of ground characterisation in deep underground mines in three main steps including input data, data processing and output (Figure 3.1). The classification results in different classes in rock mass structures.

3.2. Effective features in rock mass composition (Input)

Rock mass structure appraisal is an essential step in many underground projects. It is associated with determining inherent rock properties such as intact rock strength and providing a context for rock mass classification in design procedures. The important primary parameters for rock mass description for engineering purposes are colour, weathering, discontinuities, stratigraphic and rock type. Additionally, rock type, topographical and geological structures, and in situ stress are some of the main input parameters for describing rock masses.

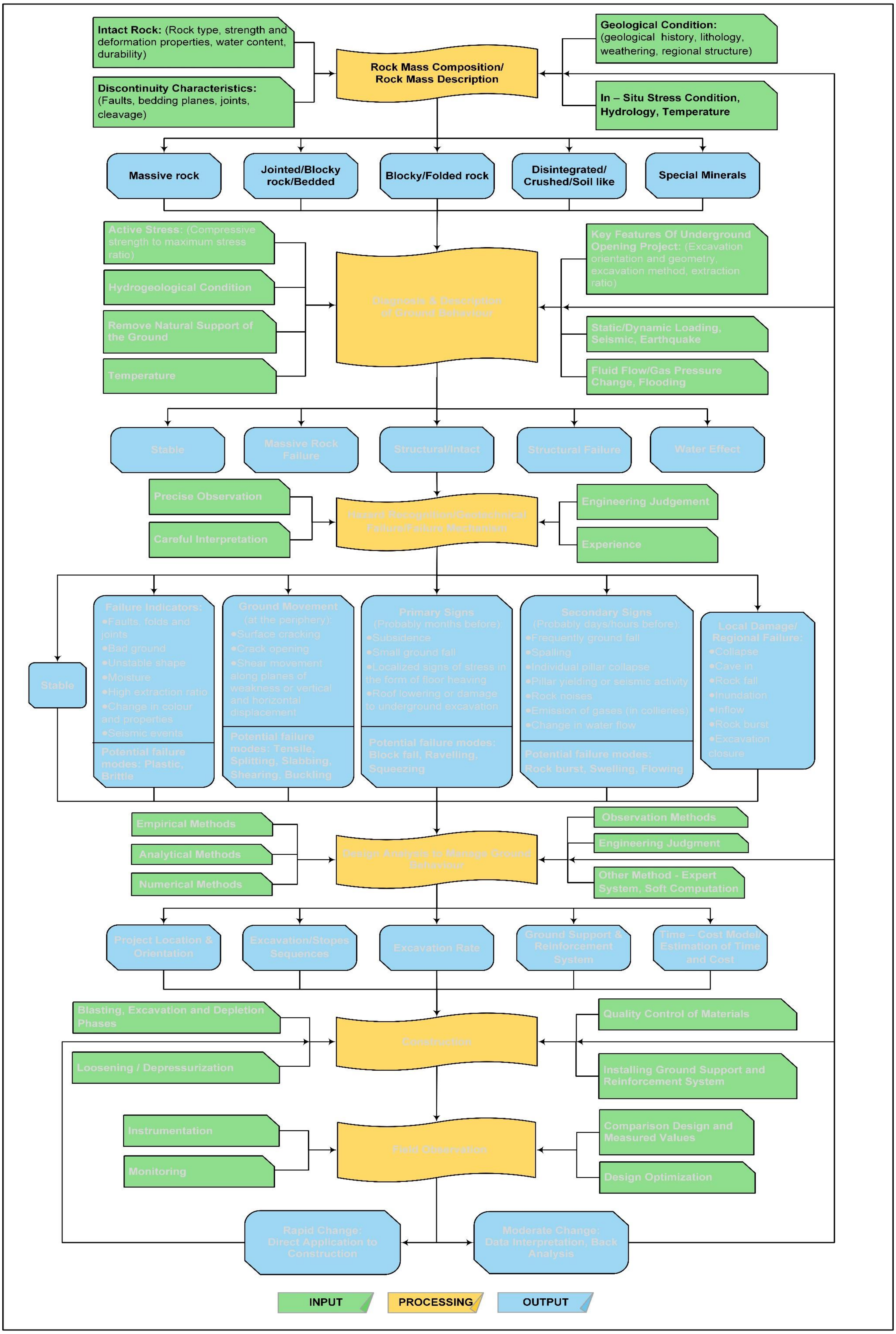


Figure 3.1. The process of rock mass composition in deep underground mining

The characterisation is an essential part of rock engineering practice for estimating large scale rock mass properties. Rock mass characterisation is based on quantitative and qualitative site investigation and laboratory tests. The initial data for characterisation of rock structures in underground projects in a greenfield study is usually obtained from engineering geological survey and drilling boreholes. Geological field observation is used to determine geological structures such as faults, rock type, and lithological contacts. Engineering geological mapping and laboratory test results provide rock strength, the location of weakness zones and discontinuity properties (Palmstrom and Stille, 2015). Differences in lithological rock units may be encountered in mining operations. It is better to divide rock masses into a series of structural zones with almost homogenised structures and rock mass condition to characterise each of those zones.

The input data for ground characterisation of rocks are intact rock, discontinuity characteristics, geological conditions, in situ stress, and hydrology. In the following paragraphs, these features are considered.

3.2.1. Intact rock

In the earth's natural process, rocks usually convert between sedimentary rocks, igneous rocks, metamorphic rocks, and soil. Figure 3.2 illustrates a cycle of rock material and soil in the earth and conversions to each other, for geological and rock engineering purposes. The rock cycle in nature has a variety and has a complicated process. In the consolidation step, three main rock types are formed by crystallisation, compaction and cementation at high temperature and high pressure. Some types of rocks' response to environmental conditions, such as weathering and erosion, is either load-deformation or time-related. This may lead to disaggregation of rocks by mechanical and/or chemical processes.

Figure 3.3 shows the main groups of rocks in geological processes. Igneous rocks are formed through the cooling and consolidation of magma. Generally, this type of rock (like granite and andesite) has high strength. In the case of sedimentary rocks, deposition and cementation of minerals are the main processes in the rock formation. The strength of sedimentary rocks is commonly lower than igneous types. Metamorphic rocks arise from changing minerals and texture in igneous and sedimentary rocks due to high temperature and pressure conditions.

The properties of intact rocks are used for the description of rock type and determination of rock engineering parameters. Petrological name, density, strength, mineral composition, porosity, hardness, weathering, and deformation modulus are some of the critical parameters related to the characteristics of rocks. Additionally, grain size and texture of rock materials are related to their strength.

Figure 3.4 illustrates the most important factors and features related to the properties and composition of rock materials. Considering some unique properties of minerals such as swelling or slaking can provide information to identify the mechanical properties of rocks. Shape, size and interlocking between grains, and cementation can change rock porosity.

The strength of intact rock can be determined in laboratory tests. However, in some cases, it is not possible to ascertain intact rock strength directly in laboratories because of time, budget and problems in recovery of samples due to schistose, tectonic or weak quality conditions. Figure 3.5 shows typical values of rock strength. The quality of cementation, grain size and type of minerals affect the strength of rocks. For example, a sandstone with low cement and high porosity are weaker than one with low porosity and strong quartz cement.

Approximate identification of rock type in a site investigation could be sufficient for rock engineering purposes. Rock outcrops are used for distinguishing between igneous, sedimentary and metamorphic rock types in the field. Figure 3.6 shows a guideline for recognition of rock types in rock engineering practices.

To characterisation of intact rock, the type of rock is identified and mineral content, texture, grain size and colour, and alteration are described. Estimation of engineering properties of intact rocks such as unconfined rock strength (UCS), Young’s Modulus (E) and weathering factor (WF), density, Poisson's ratio are necessary for the design of deep underground minings.

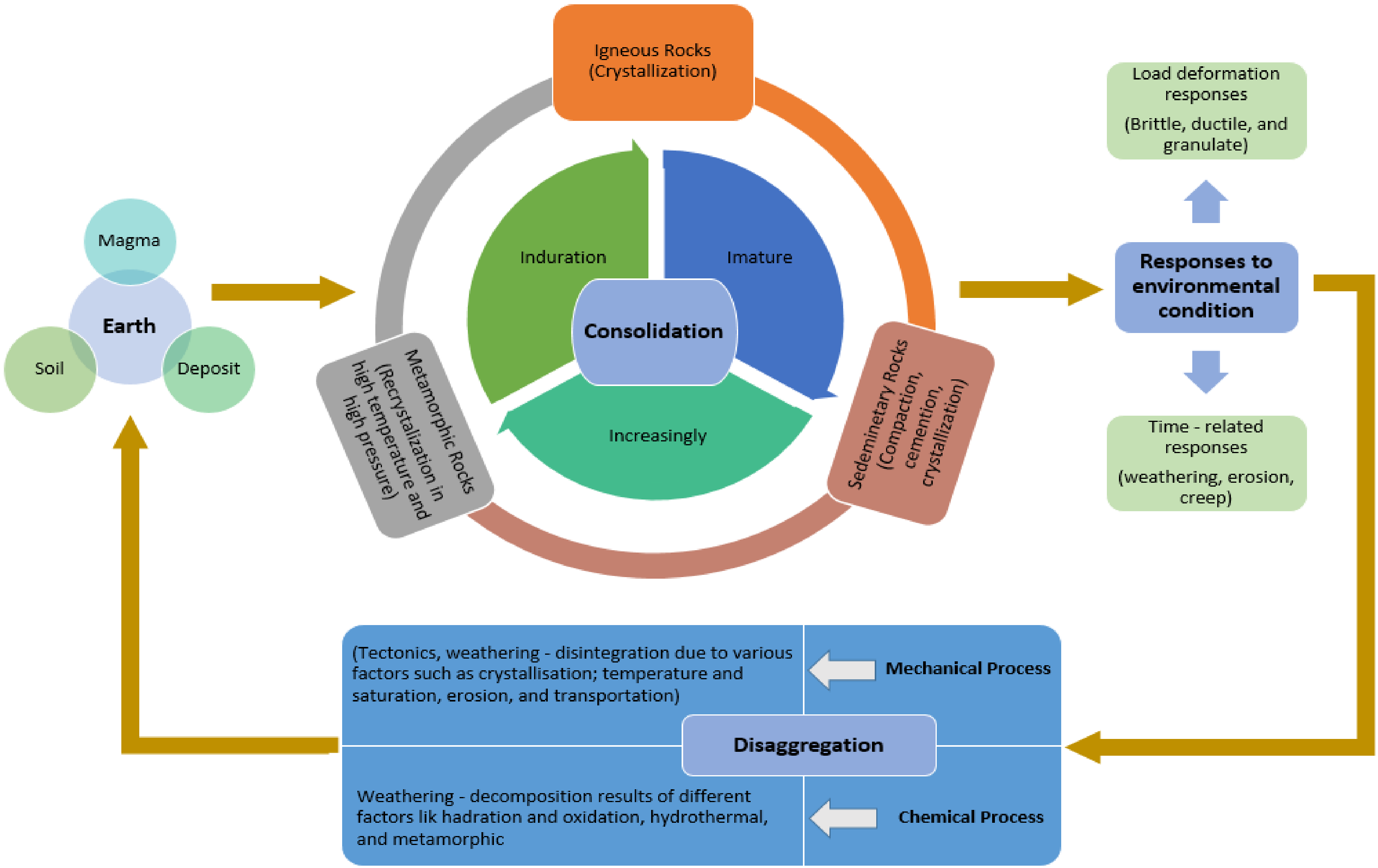


Figure 3.2. The natural rock cycle processing in the aspect of geological and rock engineering condition

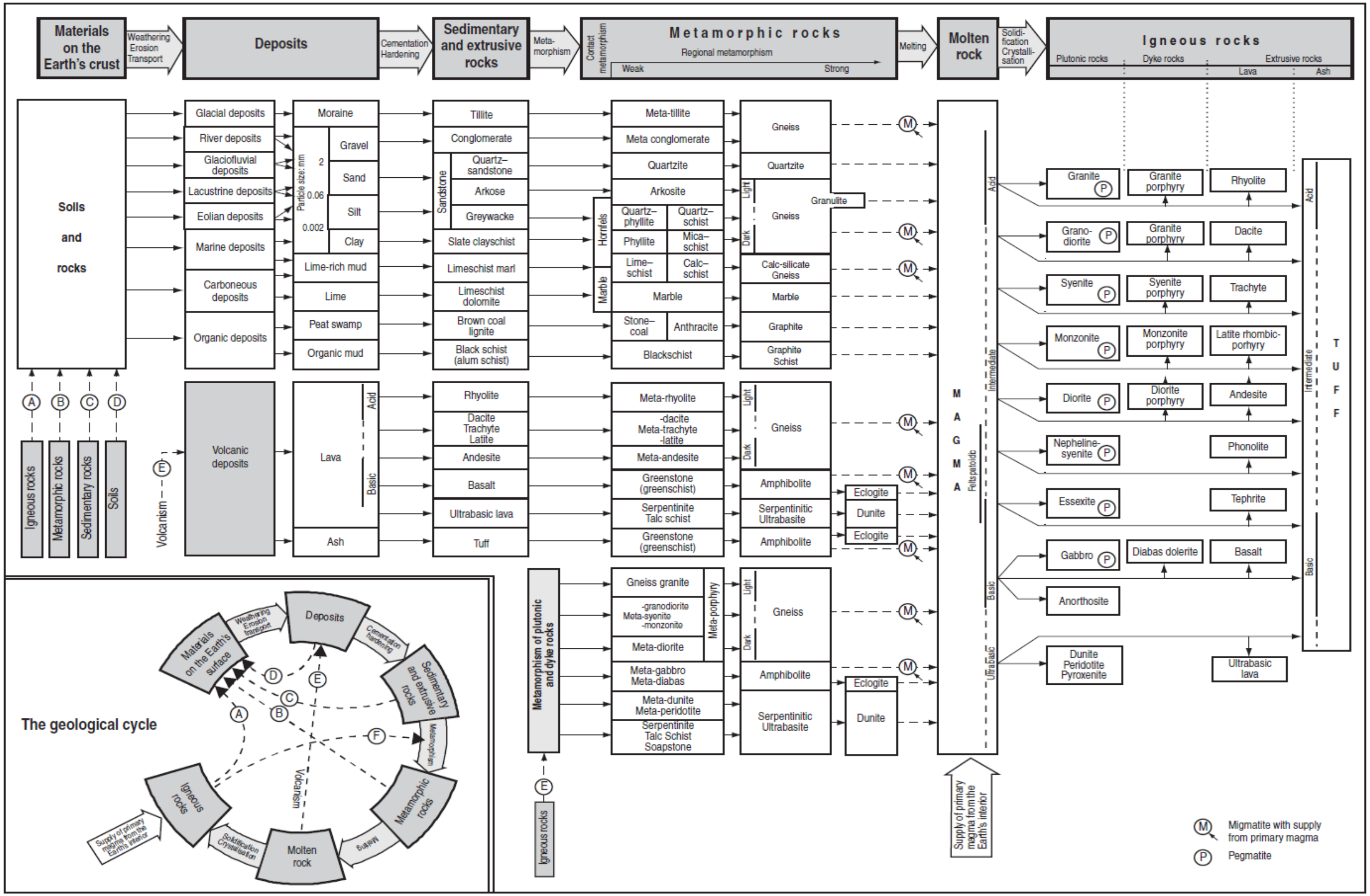


Figure 3.3. Main types of geological rocks (Palmstrom and Stille, 2015)

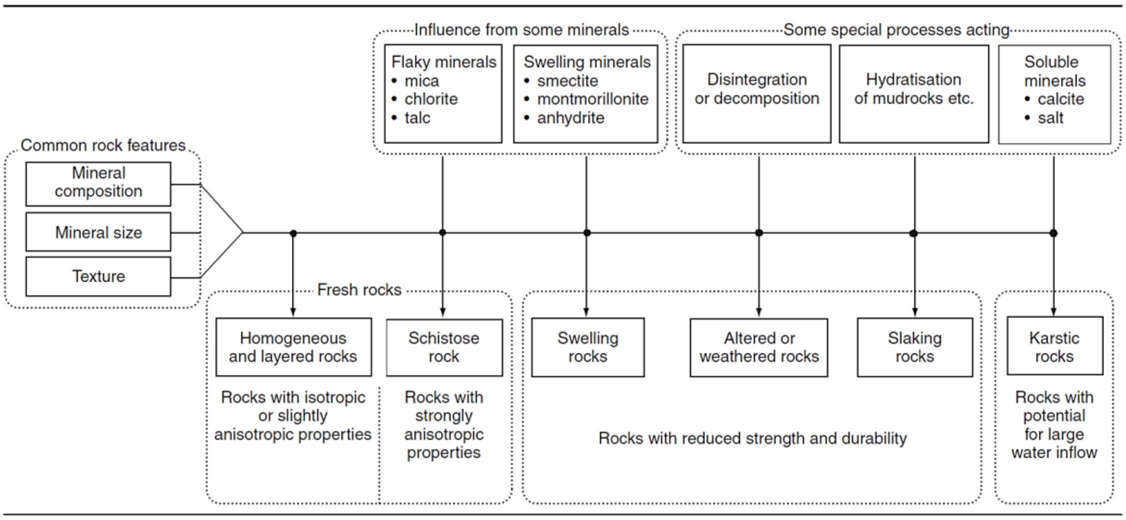


Figure 3.4. The most important factors/features influencing rock properties and behaviour (Palmstrom and Stille, 2015)

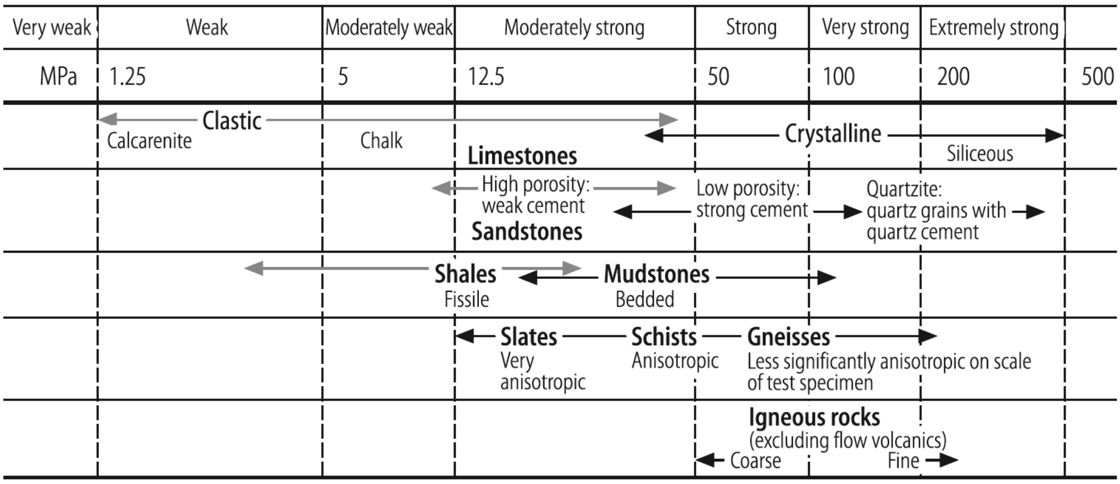


Figure 3.5. Strength range of typical rocks (Price, 2009)

3.2.2. Discontinuity characteristics

A weakness plane (not necessarily with a separation) across a rock mass is defined as a discontinuity. The impact of tension or shear stresses or even a combination of them on a rock mass may be the formation of joints. Surface joints formed by shearing are usually smooth and are evidenced by detritus in the wall. Also, joints caused by extensile conditions are clean and contain little detritus on the surfaces. Some joints are related to compressional structures and residual stress such as joints formed by folds. Furthermore, in the case of igneous and volcanic rocks, joints usually develop during the cooling process, while in wet sedimentary deposits, this happens in the drying process.

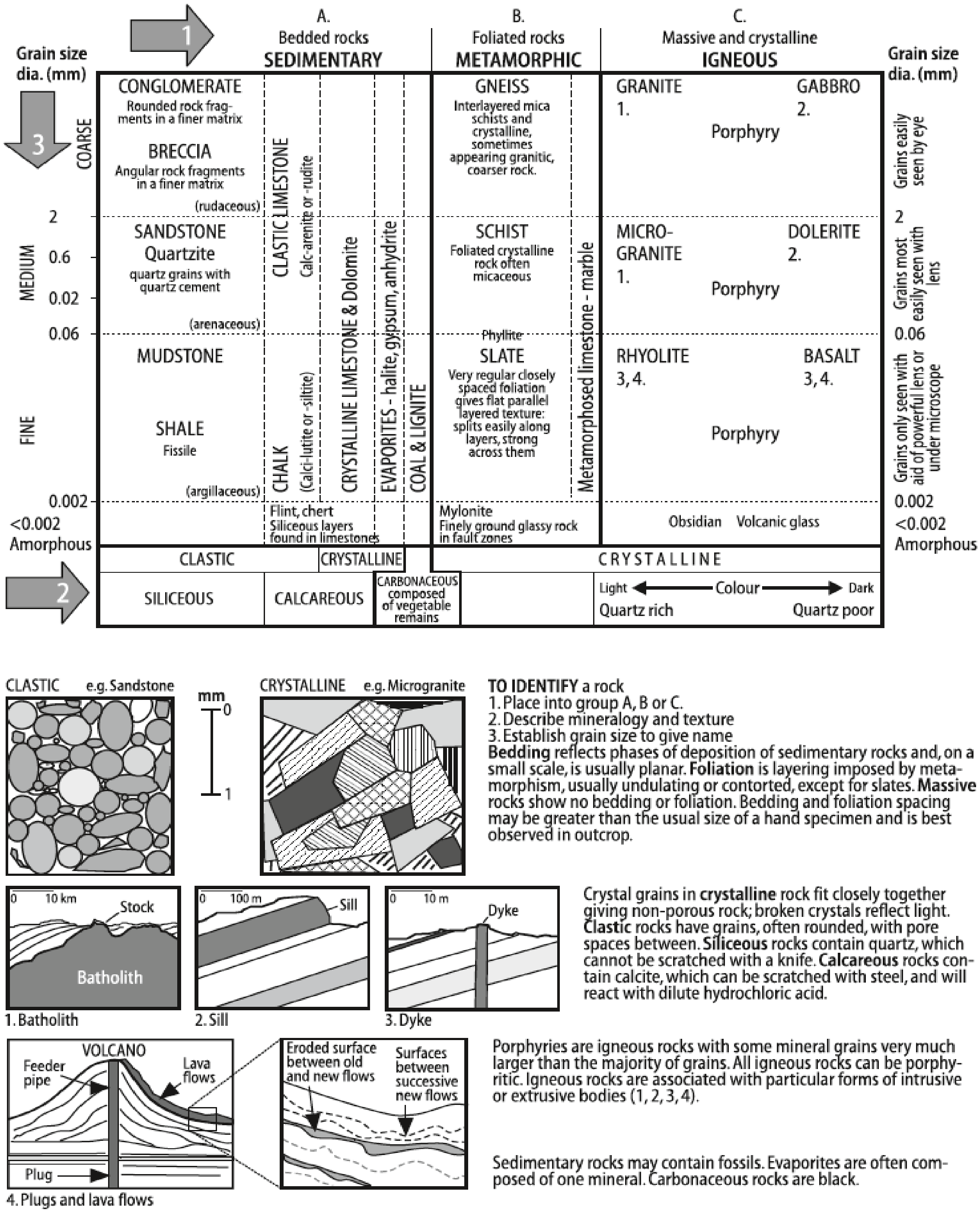


Figure 3.6. Identification of rock types in field-works for engineering purposes (Price, 2009)

The size of discontinuities varies from a small fissure to enormous faults. The most important discontinuities involve fissures, joints, bedding planes, cleavage and schistosity planes, and faults. The types of discontinuities based on the length is presented in Figure 3.7. Rock discontinuities in geotechnical engineering are composed of rock defects (less than 1 cm in length) such as microcracks, joints (1 cm < joint lengths < 100 m) such as cracks, and weakness zones (lengths > 100 m) such as faults.

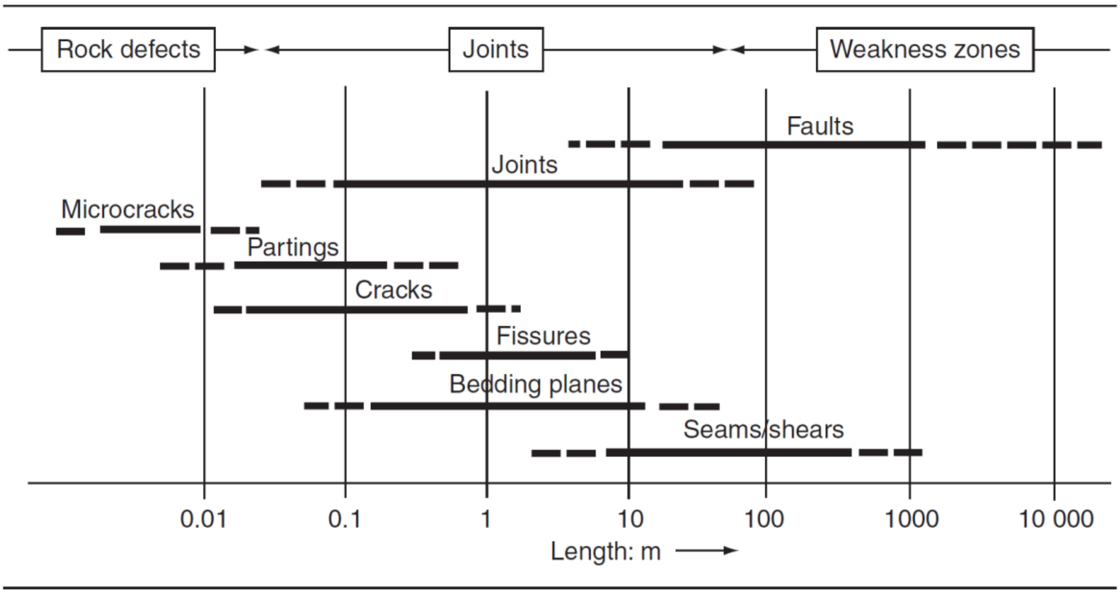


Figure 3.7. Different types of discontinuities in rock masses (Palmstrom and Stille, 2015)

Additionally, discontinuities based on their type can be classified into (Price, 2009):

1) **Integral discontinuities:** There is no opening due to weathering processes or shear movements. This type of discontinuity involves tensile strength and cohesion. For example, foliation, bedding planes and cemented joints are of this type.

2) **Mechanical discontinuities:** Opening because of weathering or stress condition and low tensile strength are their main features. The main groups of mechanical discontinuities are:

    - ❖ Cleavage, Foliation/fabric and bedding planes, which are formed by mineralisation or changes in the material in the rock structure.
    - ❖ Joints, which is related to the strain of stress condition or tectonic activities.
    - ❖ Fractures, caused by strains from induced stress because of human activities such as blasting, or strains from geomorphological conditions like landslides.
    - ❖ Faults and shear zones, which is associated with tectonic and human activities with shear movements on surfaces.

Some discontinuities such as bedding planes, schistosity and foliation planes usually occur as group features in rock masses with regular distance and the same orientation. These are called a set or a family (for example, one joint set or one family of joints, and a bedding plane family). Joints based on the size are divided into (Bell, 2007):

- Master joints: persistence of hundreds of metres
- Major joints: smaller than master joints
- Minor joints: not exceeding bedding planes
- Minute fractures: very small, occasionally in bedded sediments
- Micro – joints: only a few millimetres

Geological structures and discontinuity conditions influence mechanical properties of a rock mass. For geological purposes, the types of discontinuities like bedding and dykes, faults, and different types of joints and fractures are considered to be at a minor scale (Hencher, 2012). Interfaces and boundaries are encountered in geological structures, which can be interpreted by an engineering geological survey, drilling boreholes and observational results.

Local stress conditions usually cause the creation of natural fractures in rock structures. Engineering processes can cause the formation of fractures in shear, extension and tensile modes. Different states of natural fractures in geological structures involve tectonic, regional, contraction and surface fractures. Table 3.1 presents a classification of natural fractures. Shear fractures are due to displacement parallel to discontinuity planes. The form of this type of fracture usually makes an acute angle with the direction of major principal stress ($\boldsymbol{\sigma_1}$) and an obtuse angle with the orientation of minor principal stress ($\boldsymbol{\sigma_3}$) (Figure 3.8–B and C). For the extension fracture mode, the planes are parallel with the major ($\boldsymbol{\sigma_1}$) and intermediate ($\boldsymbol{\sigma_2}$) principal stresses, and perpendicular with the minor principal stress state (Figure 3.8–A).

The acute angle between shear fracture planes and principal stresses (called conjugate angle), depends on the following factors (Nelson et al., 2001):

1- Rock mechanical properties,

2- The absolute value of the minor principal stress (σ_3), 3- The magnitude of the intermediate principal stress (σ_2) which is related to σ_1 and σ_3

Table 3.1. Typical classification of natural fractures (Nelson et al., 2001)

Experimental Fracture Classification
1. Shear fractures
2. Extension fractures
3. Tensile fractures
Naturally Occurring Fracture Classification
1. Tectonic fractures (due to surface forces)
2. Regional fractures (due to surface forces or body forces)
3. Contractional fractures (due to body forces)
4. Surface-related fractures (due to body forces)

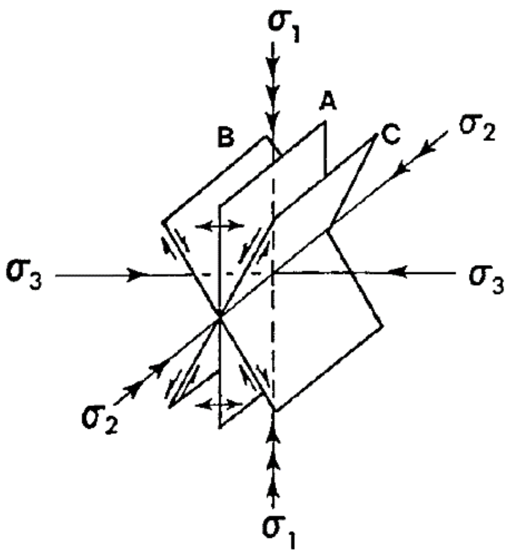


Figure 3.8. Formation of fracture planes with orientation of principal stress state, (A) Extension fracture, (B & C) shear fractures (Nelson et al., 2001)

Tension fracture is associated with displacements of planes parallel to σ_1 and σ_2, while at least one of the principal stress components (σ_3) should be negative or tensile. In addition, the principal stress components should be compressive or positive in the extension fracture type.

Faults occur due to shear displacements, from a few millimetre movements to many kilometres in the earth's crust. The behaviour of faults is plastic at great depth with high temperature and stress conditions (Hencher, 2012). Joints are a type of fracture with indiscernible displacement and created in overstressed rock materials. Joints can be systematic (as roughly parallel series) or non – systematic or random. Local stress conditions affect joint properties like their orientation, persistence and roughness and may cause joints to develop with changes in temperature and water pressure. Joint types are divided into primary (related to rock formations), secondary (due to tectonic activities and gravity stress fields), and tertiary (influenced by weathering and geomorphological process) (Hencher, 2012). Development of joints is mainly associated with extensional fractures parallel to the major compressive stress (σ_1) (Hencher, 2012).

Veins in rock structures usually indicate that a rock mass is weak and highly jointed in the data collection and interpretation process (Bewick and Kaiser, 2016). As a result, the veins may lead to inappropriate characterisation and prediction of rock mass behaviours. The recording and interpretation of veins need to be performed separately from that of the open joints, and also the estimation of rock mass strength along veins should be carried out, for characterisation of veined rocks.

The orientation of discontinuities on geological surfaces can be representative of the direction of stresses. Also, continuity of joints (persistence) in different directions affects the size and shape of rock blocks which is essential, especially in sliding and falling blocks in engineering projects. Measurements of dip and strike of discontinuities are required for engineering geological purposes.

Discontinuity deformation behaviour is related to openness or joint aperture. It also affects the strength and seepage characteristics in rock structures. Furthermore, the type of materials and thickness of filling in joints is important in discontinuity characteristics. The filling materials such as chlorite, gypsum and quartz, should be identified for estimating physical properties of fractures. Meanwhile, it may be required to consider weathering, hardness and healing of fracture infill for some engineering purposes. Discontinuities may be healed during a mineralisation or precipitation process.

Discontinuity conditions in rock mass structures should be characterised based on the type, major sets and orientations, continuity factor, infilling materials, active behaviour with groundwater, and explanation of the effects of tectonic activities on fractures.

3.2.3. Geological condition

Investigation of geological features of large-scale rocks includes lithology of rocks, and tectonic activities and effected features such as faults, geological structures, for example, folds, and weathering conditions. Table 3.2 shows some of the essential parameters during rock mass considerations. Discontinuities usually influence strength, deformability and mechanical behaviour of rock masses. Faults and shear zones may be associated with a high degree of jointed rock, permeability, and geological disasters such as earthquake activity.

Table 3.2. The main factors of geological consideration of rock masses (Hencher, 2012)

FACTOR	CONSIDERATIONS	EXAMPLES OF ROCK TYPES SITUATIONS
lithological heterogeneity	difficulty in establishing engineering properties, construction problems (plant and methodology)	colluvium, un-engineered fill, interbedded strong and weak strata, soft ground with hard corestones
joints/natural fractures	sliding or toppling of blocks, deformation, water inflows, leakage/migration of radioactive fluids	slopes, foundations, tunnels and reservoirs, nuclear repository
faults	as joints, sudden changes in conditions, displacement, dynamic loads	tunnels, foundations, seismically active areas
structural boundaries, folds, intrusions	heterogeneity, local stress concentrations, changes in permeability – water inflows	all rocks/soils
weathering (mass scale)	mass weakening; heterogeneity (hard in soft matrix), local water inflow, unloading fractures	all rocks and soils close to Earth's surface, especially in tropical zones; ravelling in disintegrated rock masses
hydrothermal alteration	as weathering, low strength and prone to collapse especially below water table	generally for igneous rocks especially near contacts

The history of fracture systems and its extension in folds are complicated. Figure 3.9 shows a typical geometry of fractures in a fold. The location and intensity of fracture zones can differ with the type of fold and the state of tectonic events. Rock exposure faces display orientation and geometric elements of fractures in rocks, which is useful to describe geometry and type of fracture, and local major principal stress condition.

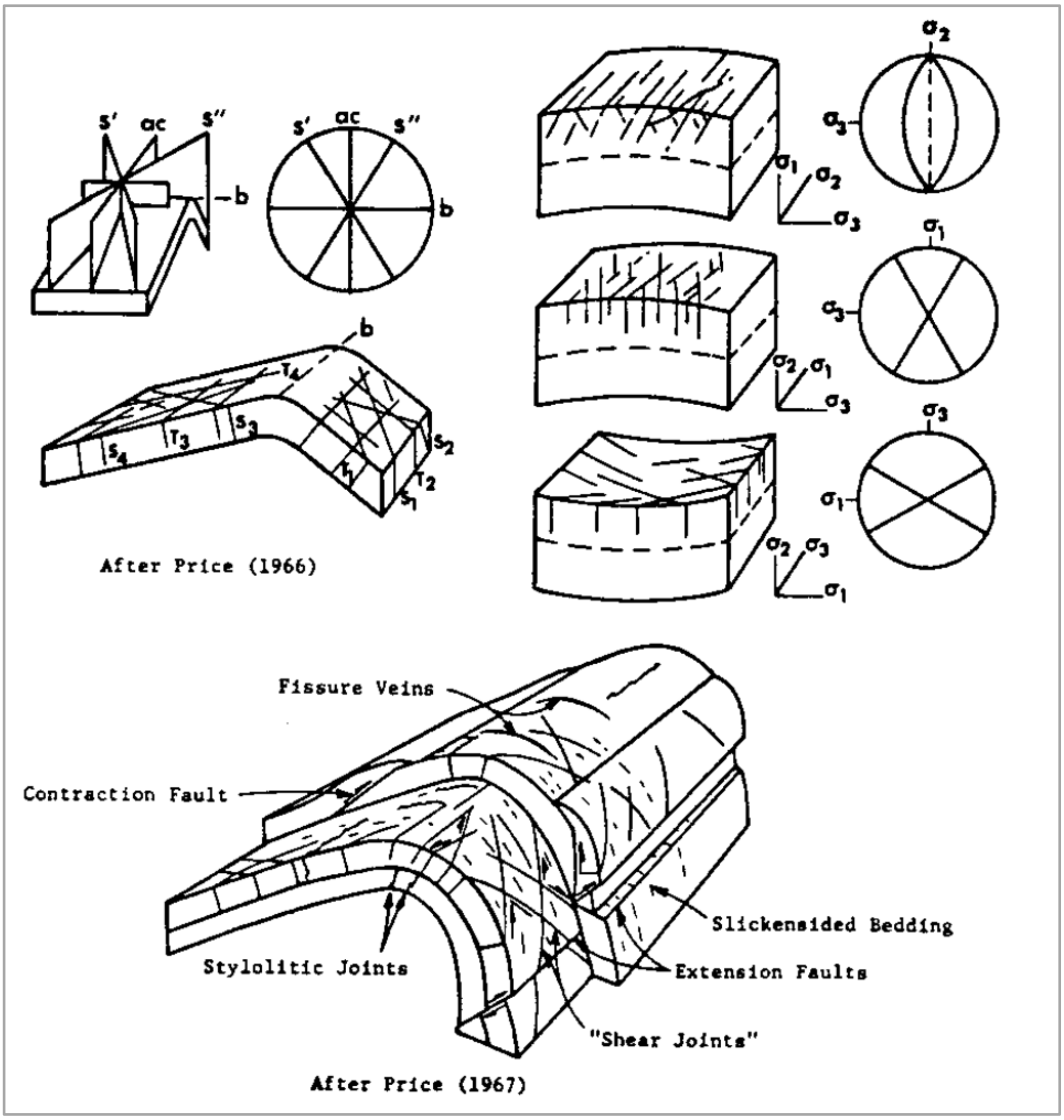


Figure 3.9. The typical geometry of fracture systems in folds (Nelson et al., 2001)

Faults are a shear movement of planes in geological structures. Faults contain many fracture zones with shear modes parallel and conjugate to faults, or extension fractures between shear types at acute angles. Movement planes of faults and fracture zones are related to the direction of principal stresses; they can be used to determine the primary condition of stress in the field.

Figure 3.10 displays a definition for major faults based on principal stress states in geological structures. In this figure, ϕ is the stress ellipsoid–shape ratio and is defined as follows:

$$\boldsymbol{\phi} = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \quad , \quad \mathbf{0 \le \boldsymbol{\phi} \le 1} \qquad \textbf{(3.1)}$$

Based on the ϕ ratio, five different states are described for stress regimes in major faults. The colour in this diagram is encoded as red for normal faults, blue for reverse faults, and green for strike-slip faults. Collecting data from drilling boreholes and site investigations for rock mass characterization provide information to interpret the tectonic structure surrounding rock engineering projects. Early predictions at tectonic fault localities in deep underground excavations allow pre-planning and assessing of proper support requirements.

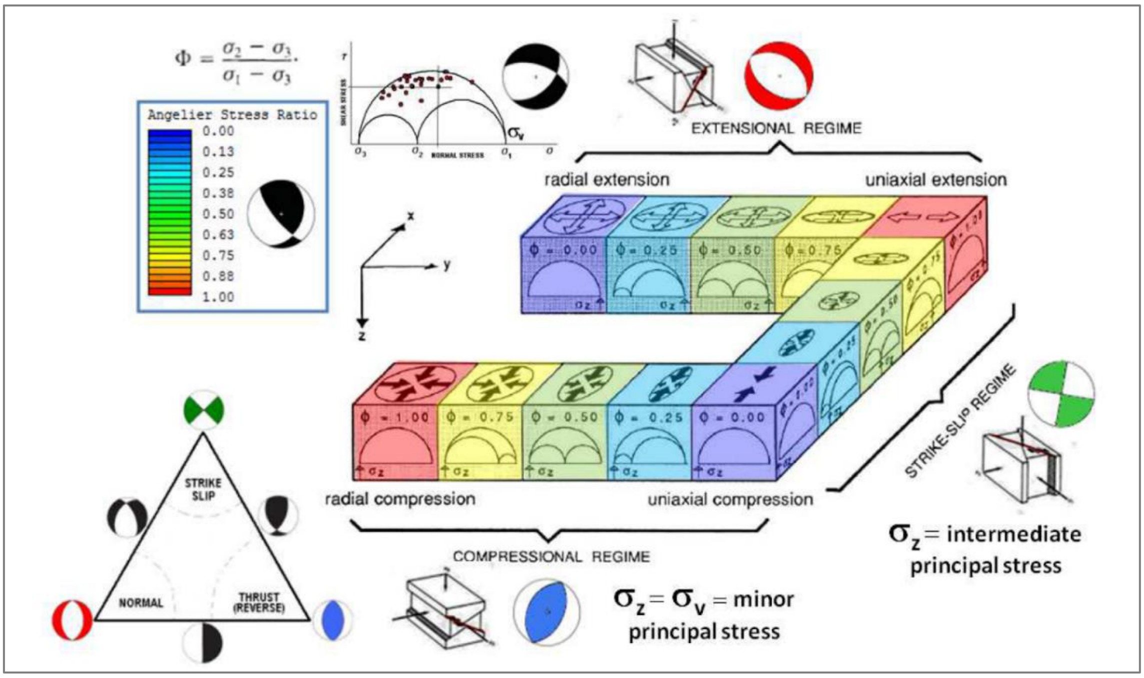


Figure 3.10. The categorisation of tectonic faults based on principal stresses in geological structures (Carter, 2015)

Generally, fractures created by tectonic activities are larger than regional and surface fractures in rock mass structures. Table 3.3 shows the conceptual scale of natural fractures in the geological scale. Application of surface forces caused by local tectonic activity develops tectonic fractures in the environment.

Regional fractures are mostly generated in a large area with simple geometry and large spacing, which is different compared to tectonic fractures. These type of fractures, which are also called systematic joints, include the following parameters:

1. Orientation ranges between 15 and 20^{o},
2. Spacing varies from 0.3 – 6m,
3. Develop consistency over a large area

Table 3.3. The general pervasiveness of natural fractures in geological structures (Nelson et al., 2001)

Orders of Magnitude in Size Spanned	
Tectonic Fractures	9–10 Orders
Regional Fractures	5
Contractional Fractures	2
Surface-Related Fractures	4–5

Thermal processing in a rock formation from hot conditions to cooling and vice versa induces tensile stresses leading to propagation of thermal contractional fractures. Depth, thermal gradient and mineral type of rock composition affect these types of fractures (Nelson et al., 2001). Changing mineral phase in rock composition during, for example, calcite to dolomite, which may be related to reducing volume in the geometry of rock, causes the creation of tension or extension fractures and are called mineral phase – change fracture systems.

Weathering changes physical and chemical properties of the rock. Weathering of rock can alter the colour, mineral composition, cementing materials and strength of rocks. Table 3.4 shows a classification for the description of qualitative and quantitative rock weathering in engineering purposes. The opening of discontinuities, discolouration of the surface rocks and joints, decomposition of rock materials, and fractures of mineral grains could occur due to a weathering phenomenon.

Table 3.4. Qualitative and quantitative classification of rock weathering

Category	Weathering Factor (WF)	Description
Fresh Rock (I)	$0.8 \leq \sigma_{ci}/\sigma_{cfresh} \leq 1$	No visible sign of weathering, may slight discoloration along discontinuities, make a ringing sound by hitting hammer
Slightly Weathered (II)	$0.6 \leq \sigma_{ci}/\sigma_{cfresh} < 0.8$	kept original rock structure, small amount soil from rock along discontinuities, surface weathering of rock material and discontinuities, ringing sound by a hammer
Moderately Weathered (III)	$0.35 \leq \sigma_{ci}/\sigma_{cfresh} < 0.6$	Evidence of original structure in rocks, degraded of almost half of rock materials, a drumming sound by hammer, discolouration of rock material and discontinuities, present of fresh/discoloured rock
Highly Weathered (V)	$0.1 \leq \sigma_{ci}/\sigma_{cfresh} < 0.35$	Visible of original mass structure, Disintegrated or decomposed more than half of rock materials, friable of rock material, a dull sound by hitting hammer
Completely Decomposed / Residual Soil (IV)	$0.1 < \sigma_{ci}/\sigma_{cfresh}$	Destroyed original rock structure, all rock material decomposed and/or disintegrated to rock and soil, crumbled by hand

Collected information and data related to geological condition is utilised for creating a geological model of sites based on the lithology, geometry and type of natural fractures/joints at rock mass scale, major tectonic structures and their boundary, and weathering and alteration conditions at mass scale. Geological conditions and models provide a conceptual view of rock engineering projects.

3.2.4. In situ stress

In situ state of stress in rock is due to the weight of rock mass in the overburden, and tectonic activities. Initial stresses could be caused by gravity, rock displacements, geological structures, topography, tectonic plates and activities, and properties of rocks (Sheorey, 1994). The effect of in situ stress conditions on rock mass structures are (Singh, 2011):

- Fracturing and rock failure in high-stress levels
- Increase in situ stress level with the depth
- Reduction of the stress concentration with deformation in rock
- Permeability of rock mass under stress conditions
- Effect on discontinuity condition in rock masses and changing their properties
- Effect on geometry and underground excavation method

In rock engineering projects, in situ stress can be considered over five scales including tectonic scale, site scale, excavation scale, borehole scale, and microscopic scale (

Figure 3.12). The tectonic scale of in situ stress is associated with large scale areas like hundreds or thousands of kilometres, and relevant plate movements need to be considered within situ stress and its principal states. Moving and colliding tectonic plates can be regarded as the main reason for regional in situ stress. Figure 3.11 shows a global stress map. The map displays the orientation of maximum horizontal crustal stress in the upper 40 km by various methods. The symbols in the map are NF (normal-fault), SS (strike-slip), TF (thrust-fault) and U (unknown). Understanding in situ stress conditions in large scale helps to evaluate plate tectonic activities in rock engineering projects.

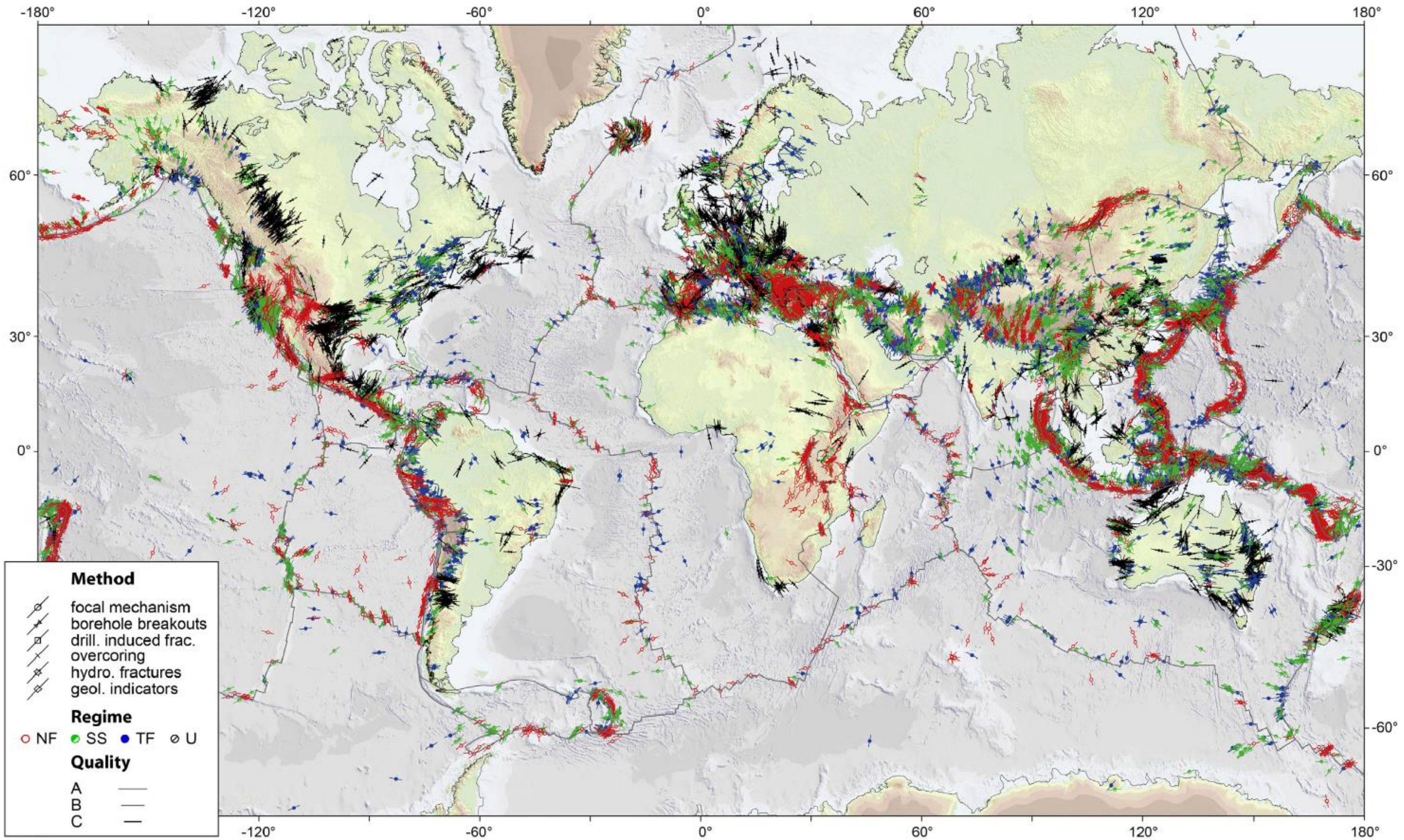


Figure 3.11. World Stress Map (WSM): Symbol display NF (Normal-Fault), SS (Strike-Slip), TF (Thrust Fault) and U (Unknown) (Heidbach et al., 2018)

Estimation of in situ stress at the site scale is appropriate for rock engineering purposes. In some special projects such as hydropower caverns, the stress state in the local area around the excavation may be necessary, so the excavation scale of stress condition is considered. The borehole scale of in situ stress is used for interpreting measurements on the scale of around 0.1 m (Hudson and Feng, 2015). In the case of microscopic scales, the grain size scale of rock material is considered.

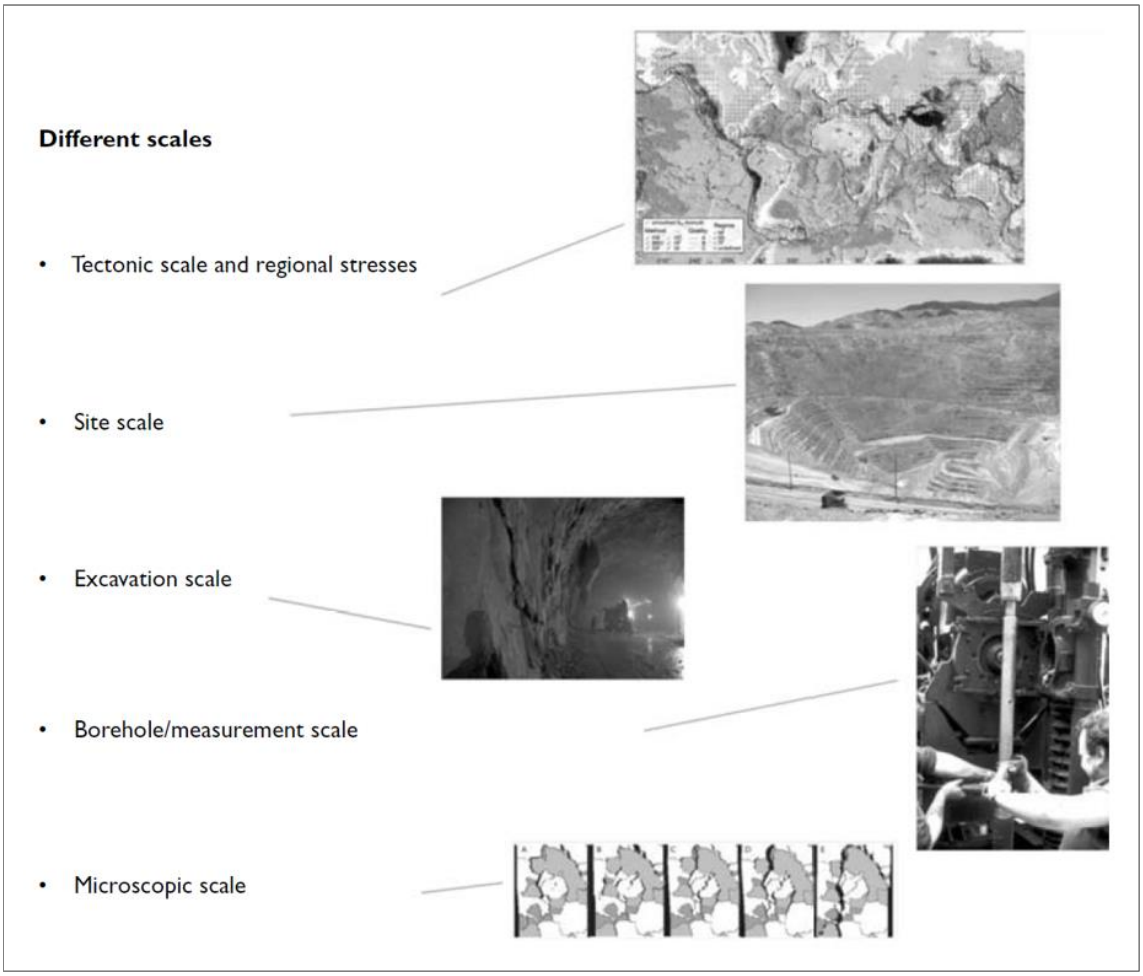


Figure 3.12. Considering in situ stress in different scales for engineering purposes(Feng and Hudson, 2011)

Inhomogeneity, anisotropy, the presence of discontinuities, and free surfaces are some of the critical factors which affect disturbance of in situ stresses (Hudson and Feng, 2015). Figure 3.13 shows the four main factors of disturbance of in situ stress conditions in rock. The stress state varies in the regular layered rock due to the difference in characteristics (such as minerals, colour, shape, texture and density) of each layer.

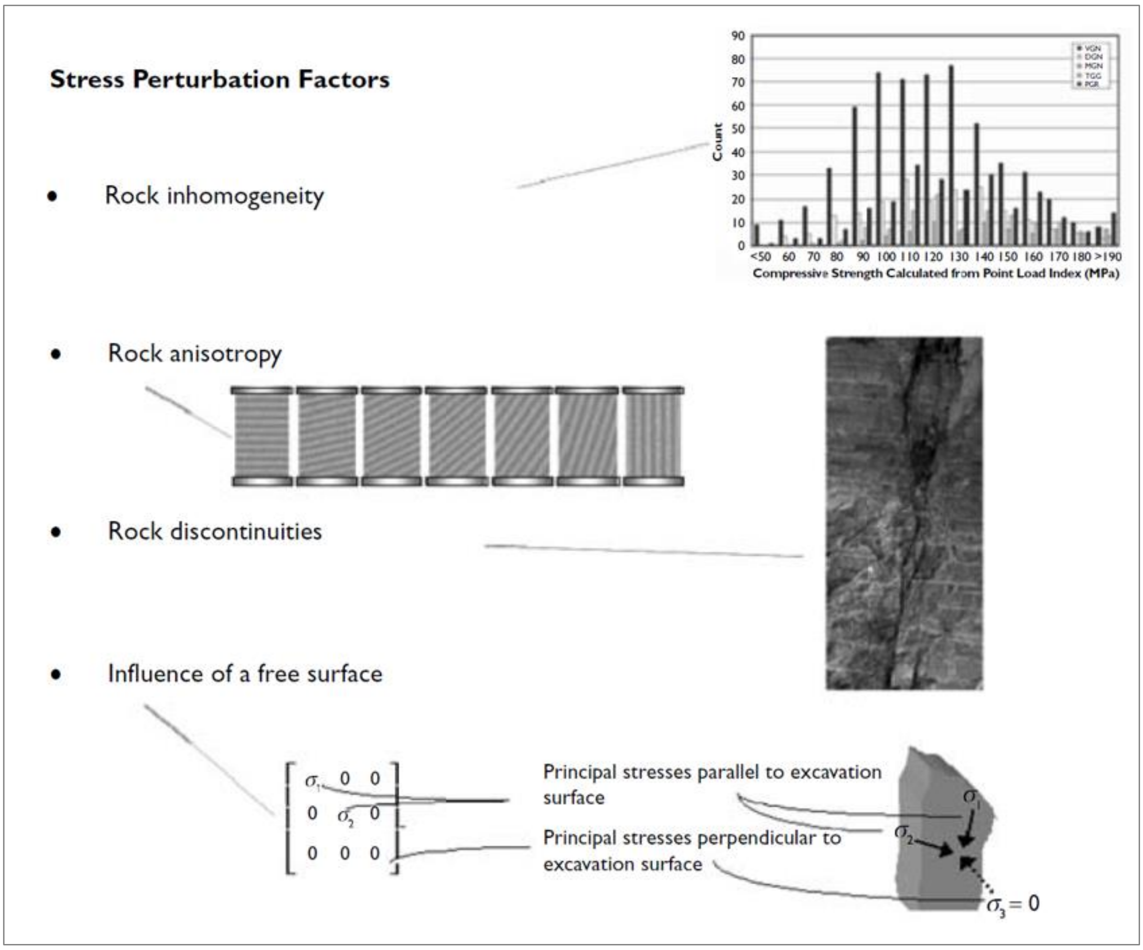


Figure 3.13. The essential factors in disturbance of initial stress condition (Hudson and Feng, 2015)

Natural fractures and discontinuities in rock from the microscale (like grain size defects in rocks) to the large scale (such as faults and shear zones) influence the initial stress state in the rock structure. In natural free rock surfaces, there cannot be normal or shear stresses. Therefore, one of the principal stress components is in the plane of the free surface. Typically, the principal stresses are assumed perpendicular and parallel with underground excavation surfaces. The result of this concept affects the displacement of the rock mass in the periphery of the opening, excavation design and ground support design.

Figure 3.14 presents methods of in situ stress measurement and estimation in deep underground excavations. At a regional scale, the fundamental estimation for orientation can be derived from the topography and geological structure and in situ tectonic stress. Then, the in situ stress component can be analysed based on the information from multiple sources such as regime, orientation and in situ stress measurements.

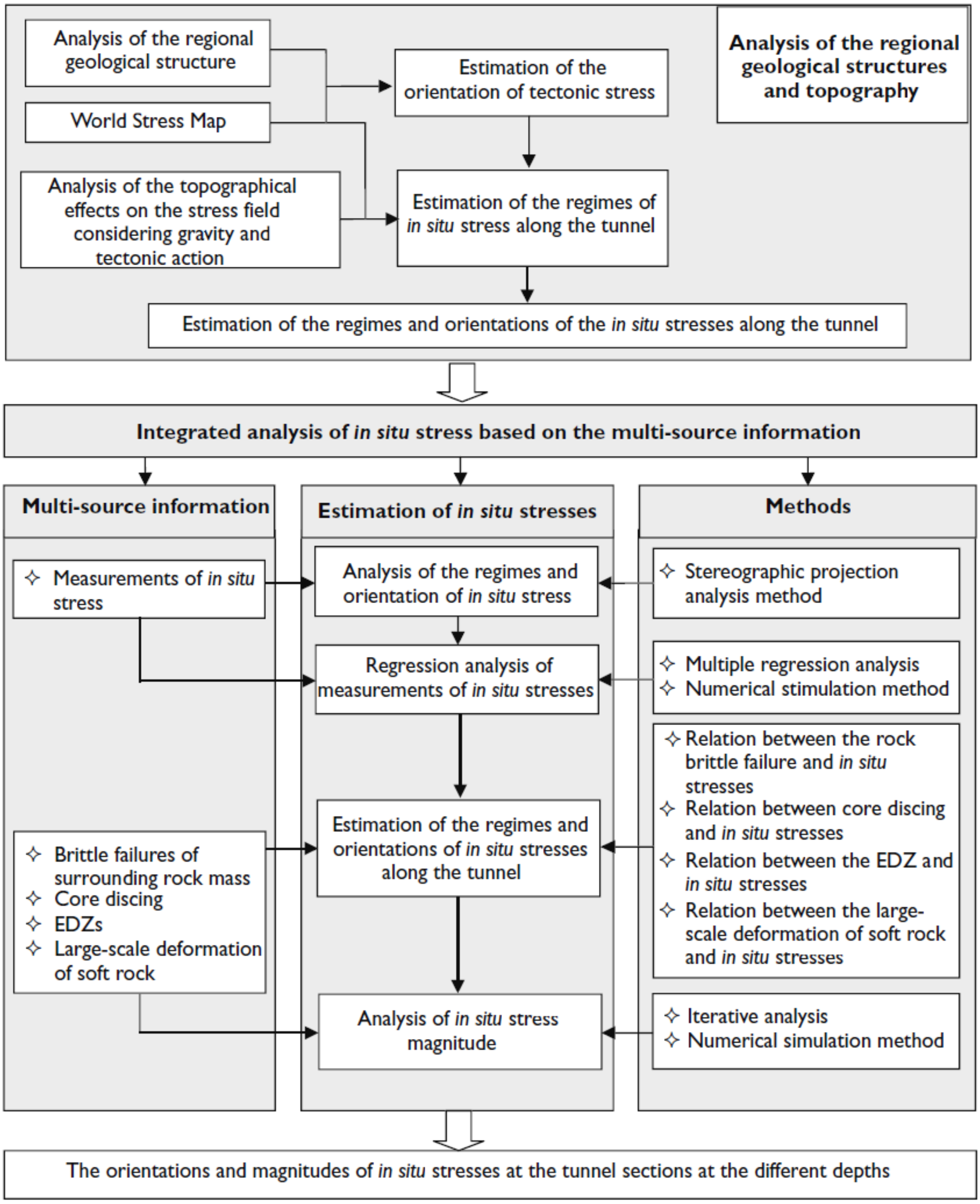


Figure 3.14. Selecting appropriate in situ stress measurement methods in deep underground excavations (Hudson and Feng, 2015)

The proposed methodologies for estimation of in situ stress from preliminary to construction phases of projects is given in Figure 3.15. In the preliminary phase considering regional and local geology, geological maps and the world stress map can be used to estimate stress condition. In the detailed design phase, analysis of faults, rock mass fracturing model, major geological features (such as faults, folds, permeability), and overburden of the opening are considered. Geomechanical field measurements, under-coring, hydraulic fracturing, and borehole breakout are the conventional methods to estimate the in situ stress state in the design phase of rock underground works (Madirolas and Perucho, 2014). In the construction stage, back analysis, overcoring and flat jack tests are useful to evaluate the accuracy of estimation of in situ stress state in the rock masses.

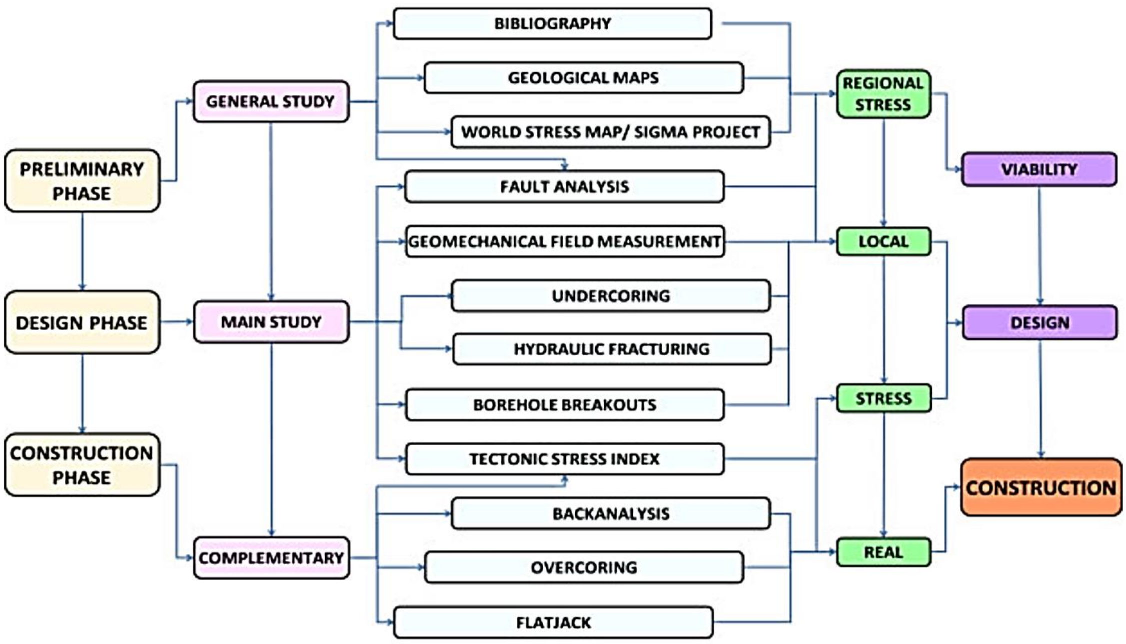


Figure 3.15. The suggested methods to estimate in situ stress in different phase a project (Madirolas and Perucho, 2014)

In situ stresses or field stress are one of the main factors influencing rock mass structures. Initial stresses are originally derived from gravitation, tectonic activities, residual stress, topography, thermal, and residual stress in rocks. Discontinuities, anisotropy, inhomogeneity, free surfaces during engineering activities in underground mining lead to disturbance of stresses in rocks. There are a variety of methods to estimate stress components. In the preliminary design stage, geological structures and empirical methods are useful methods for this issue. In the detailed design stage, field tests such as undercoring and hydraulic fracturing are employed for calculating in situ stress conditions. In the construction stage, back analysis methods can be used to determine a reliable value of the stress field.

3.2.5. Hydrology

Groundwater influences the physical and chemical properties of rock materials. The interaction between rock and water may lead to degradation and changing volume in rocks. Porosity is a common feature in many rocks. The network between individual pores and discontinuities affects the rate of groundwater inflow, which is called hydraulic conductivity. Figure 3.16 shows the typical ranges of hydraulic conductivity rates in rocks and soils. Fractures are the main pathways for water in rock structures and high degree jointed rocks usually have high inflow rates. Tectonized geological features like shear and fault zones could have a high inflow rate as well.

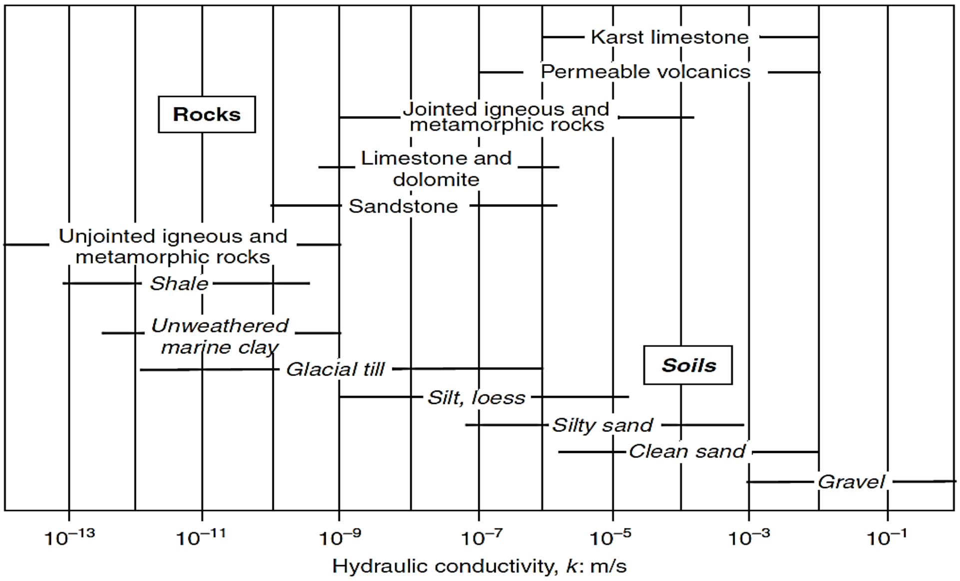


Figure 3.16. Hydraulic conductivity rates in rocks and soils (Palmstrom and Stille, 2015)

Permeability and fluid flow in intact rocks are usually very low compared with rock masses which have discontinuities. Evaluation of water flow and pore pressure in rock materials are essential in rock underground engineering projects. The nature of fluid flow in jointed rock masses can change during underground mining operations and excavation constructions. Water flowing may commence into a mining excavation when the pore pressure is higher than the rock strength.

Groundwater effects mechanical and chemical properties, and also water pressure in rock materials (Stille and Palmstrom, 2008). Some minerals like calcite and salt dissolve with water in a chemical process and cause reducing rock strength and increasing fluid flow in rock mass structures. Also, clay minerals such as montmorillonite absorb water and result in increasing volume and water pressure in rocks. Furthermore, the existence of water may cause weathering and alteration of discontinuity and rock surfaces.

The strength of some rocks significantly reduces with increasing moisture content, like clay shales. Typically, the reduction of strength is about 30 – 100% due to deteriorating rock materials during chemical processes with water and moisture conditions (Hoek and Brown, 1997). The study of groundwater effects on rock engineering properties is necessary for underground mines.

3.3. Rock mass composition processing

Rock mass properties for engineering purposes varies in a wide range of values due to uncertainty in ground conditions. Therefore, it is necessary to analyse collected data and acquire a reliable estimation of parameters to describe rock masses. Data processing is accomplished according to ground characteristics and project conditions. Properties of intact rocks and discontinuity conditions are the basic parameters for ground characteristics. Project conditions such as depth are considered for the evaluation of in-situ stress conditions. Statistical analysis, geological modelling, empirical methods, standards guides, and engineering software (such as Dips) are used to accomplish data processing. Figure 3.17 shows the common data processing methods in the characterisation of rock mass structures.

Figure 3.17. Typical methods of data processing for rock mass composition

Through data processing, the ground condition is characterised in the following sequence, as shown in Figure 3.18:

- Intact rock characteristics
- Discontinuity characteristics
- Vein and foliation characteristics
- Rock mass characteristics

Characterisation of intact rocks involves the description of appearance properties and estimation of mechanical properties. The common appearance properties are rock type, minerals, grain size, texture, weathering, alteration, hardness and colour. Also, mechanical properties of intact rocks such as strength and elastic modulus should be estimated to characterise rock mass structures.

The distribution of rock type and its parameters, location of major weakness zones and faults, and discontinuities' condition such as spacing, roughness, joint sets, frictional properties and so on can be determined by engineering geological mapping.

Characterisation of weakness zones in geological structures, which are fault, shear zones, vein, foliations, dykes, anticline, and syncline are critical for rock engineering purposes. In veined rock mass structures, joint conditions and veins in rock blocks should be described separately. Veins usually create small rock blocks and reduce the strength of rock masses, so rock falls are more prone to occur in rock fractured by stress conditions or blasting (Bewick and Kaiser, 2016). Therefore, it is required to define density and orientation, thickness and mineral types of veinlets.

Data processing for rock mass characterisation involves estimating geometry of rock blocks, in situ stress condition, water content and permeability, and intact rock and strength characteristics. The result of data processing for ground characterisation is utilised for classification of rock mass structures in rock underground excavation projects.

3.4. Deep-rock mass classification in underground mining

Deep mining is associated with geotechnical challenges related to sudden failure and large deformation in rock mass structures. The dominant factor of the failure mechanism in deep mining is high–induced stress/seismic events. Generally, depths more than 600 m are referred to as deep mining.

The process of collecting qualitative and quantitative data and information from rock masses that describes rock mass geometric and mechanical properties, is rock mass characterisation. Rock mass composition is usually evaluated based on observational methods and laboratory/field test results. Knowledge, experience and engineering judgement are the significant factors that affect the performance and quality of interpretation for estimating ground condition.

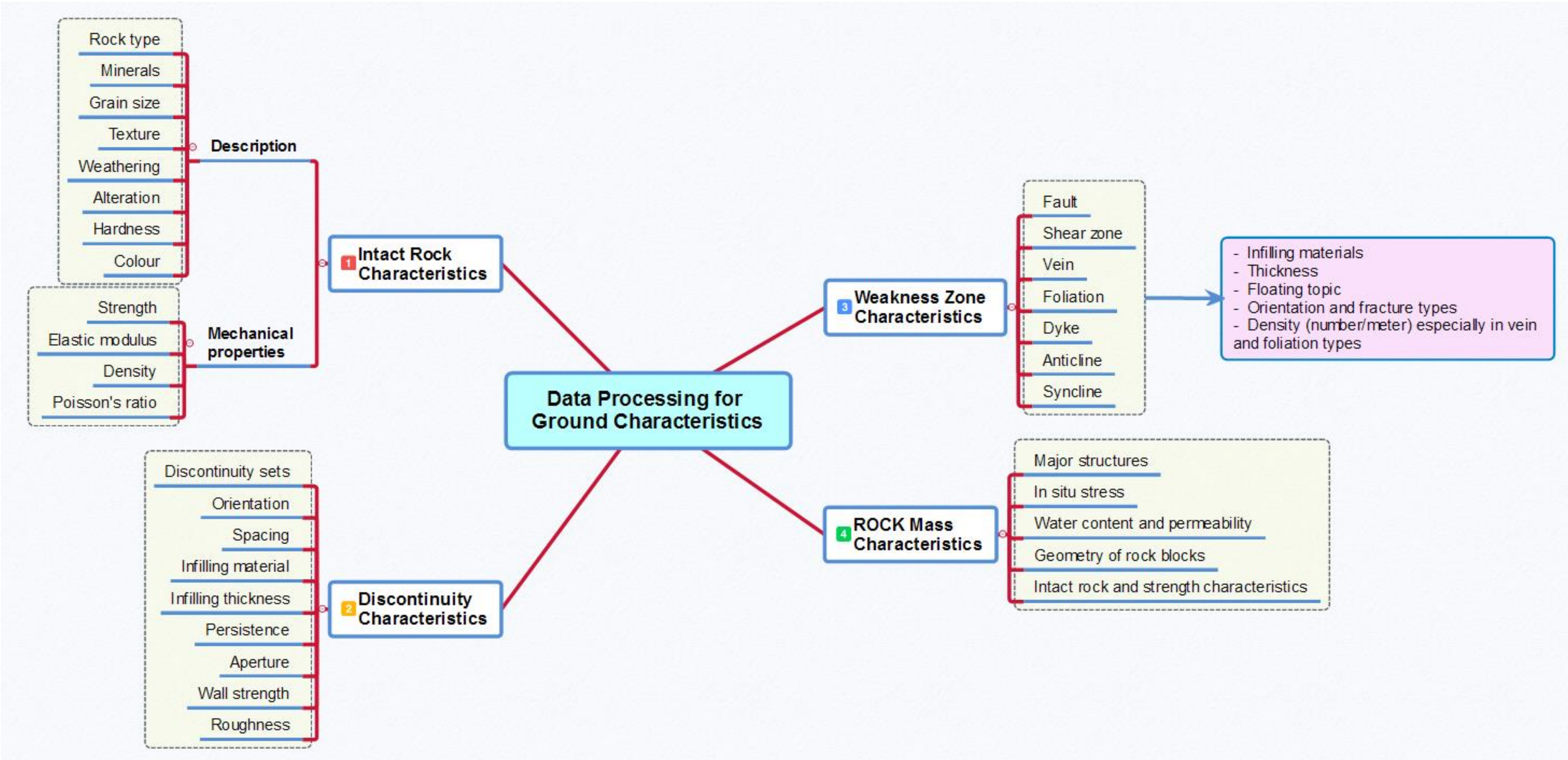


Figure 3.18. Data processing procedure for ground characteristics

By integrating input data from the field with data processing for the ground characterisation, rock mass structures are classified into five classes (as shown in Figure 3.19):

- Massive rock
- Jointed/blocky/bedded rock
- Blocky/folded rock
- Disintegrated/ crushed/soil rock
- Special materials

Figure 3.19. Classification of rock mass composition (Rahimi and Sharifzadeh, 2017)

Table 3.5 presents a developed classification of rock mass composition in underground mining. The classification is associated with the description of geological conditions, characteristics of discontinuity conditions, assessment of in situ stress effect, and determination of intact rock and strength characterisation. Geological condition considers the lithology, rock type, major structures, weakness zones, texture, grain size of rocks, minerals and other descriptive parameters of rocks. The major structures in a rock mass are considered, and the history of geological and tectonic activities are identified for mine sites. Meanwhile, the type of fractures in weakness zones, such as faults and shear zones, are investigated, and the probable orientation of principal stress is anticipated.

Discontinuity conditions are related to joint sets, the boundary between lithologies or beddings/layers, and any weakness surfaces in the rocks. Discontinuities are characterised by determination of joint sets, orientation, spacing, infilling materials, persistence, aperture, and roughness, which describe the discontinuity in the rock mass. Also, the geometry of rock blocks including the shape and size of rock blocks are required to be estimated by geological engineering mapping.

In situ stress at depth is derived from gravity, tectonic activities, local stress concentration, residual stress, and surface stresses in rocks. In rock engineering projects, in situ stress can be considered over five scales: tectonic scale, site scale, excavation scale, borehole scale, and microscopic scale. Natural fractures and discontinuities in rock from the microscale (like grain size defects in rocks) to the large scale (such as faults and shear zones) influence the initial stress state in the rock structure. The orientation of principal stresses is identified about the formation of major geological structures.

Furthermore, the rock mass structure is characterised through intact rock strength, weathering factor, and strength scaling factor.

Strength characterisation of a rock mass is calculated by the following equations:

$$\boldsymbol{Weathering\ Factor} = \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} \tag{3.2}$$

$$\boldsymbol{Strength\ Scaling\ Factor} = \frac{\sigma_{ci}}{\sigma_{cm}} \tag{3.3}$$

Where,

σ_{ci}: Uniaxial compressive strength of intact rock

$\sigma_{ci(fresh)}$: Uniaxial compressive strength of intact rock at fresh state

σ_{cm}: Rock mass strength

The proposed classification is applicable for characterising rock mass structures in underground mining. The classes of the proposed classification are described in the following sections.

Table 3.5. Rock mass composition classes and its attributed characteristics in deep underground mining

Rock mass composition classes	Geological condition	Discontinutiy condition	In situ stress effect	Intact rock and strength characteristics
Massive	Massive; homogenous; no or a few discontinuities; schistose; igneous rocks with massive [light(quartz rich) to dark colure (quartz poor)] and porphyry structures and crystalline texture, metamorphic massive rocks such as marble; mineral grain similar size or composition; coarse grain size like granodiorite; medium grain size such as dolerite; fine grain size e.g.basalt	• Surface and regional fractures; one or two discontinuitiy sets (partially to well interconnected); widely spaced discontinuties; discontinuties have less effect on rock mass properties; • spacing > 2m • volumetric joint count (joints/m³) < 1 • block volume > 8m³	More affected by gravitaitonal stress	$\sigma_{ci}(MPa) > 50$ $0.4 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $1 < \frac{\sigma_{ci}}{\sigma_{cm}} < 2$
Jointed / Blocky / Bedded	Igneous and metamorphic jointed rocks; homogenous foliated and bedded rocks; jointed seam and blocky ground; cemented minrals with inter-granular matrix mateiral in sedimentary rocks; usually anisotropy in physical properties; layered rocks; massive rock with some joint sets; coarse grain size like conglomerate; medium grain size for example tuff; fine grain size such as siltstone	• Three or more discontinuity sets(partially to well interconnected); discontinuous microfractures may be present near parallel to the layering; regional, surface and sometimes tectonic fractures; bedding plane; • 0.6m < spacing < 2m • 1 < volumetric joint count (joints/m³) < 3 • 0.1m³ < block volume < 8m³	More affected by gravitational and tectonic streses	$40 < \sigma_{ci}(MPa) < 250$ $0.3 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $2 < \frac{\sigma_{ci}}{\sigma_{cm}} < 10$
Blocky / Folded	Jointed blocky rocks and structrues like foliations and veins; foliated rocks; foliation in gneisses; sliding or toppling of blocks; clastic; crystalline; slightly to moderately decomposition of minerals shearing during folding; tectonically deformed; less or moderately folded-broken and deformed; anisotropy; schistosity; flaky minerals in rocks; thin bedded rocks and folding	• Four or more discontinuity sets (poor to partilally interconnected); folded with angular blocks; main structures are freacture, joints, bedding plane and minor faults; tectonic, contraction (extension or tension joints) and surface(shearing) fractures; schistosity and folation planes; • 0.1m < spacing < 0.6m • 3 < volumetric joint count (joints/m³) < 30 • 0.1dm³ < Block volume < 100dm³	Local stress concentration; mostly affected by tectonic, residual and surface stress	$25 < \sigma_{ci}(MPa) < 250$ $0.2 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $10 < \frac{\sigma_{ci}}{\sigma_{cm}} < 40$
Disintegrated / Crushed / Soil like	Shearing during folding or faulting; some minerals may be altered or decomposed; boundaries commonly slickensided; crushed zone composed of disoriented; usually angular fragments of the host rock substances; tectonically deformed; laminated; heavily jointed; tectonized rocks; highly jointed or crushed rocks; poorly cemented rock materials; hetergeneity; change of minerals; cracked massive rocks; loose structure and composition of the rocks from alteration and/or chemical weatheirng; rock material similar soil with low friction properties; rock fragments with few contacts; foliated shears; heaviliy jointed or breciated dykes or layers	• More than four discontinuity sets (poor interconnected); mostly tectonic, contraction and surface fractures; lowest shear strength in direction of slikensides in plane parallel to boundaries; • spacing < 0.1m • volumetric joint count (joints/m³) > 30 • block volume < 0.1dm³	More affected by tectonic stresses	$\sigma_{ci}(MPa) < 25$ $0 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $\frac{\sigma_{ci}}{\sigma_{cm}} > 40$
Special Materials	Minerals or rocks with special properties; clay minerals; swelling minerals; soluble minerals; thin layer or forming a chaotic structure with pockets of clay; layers or lense of clay; clacite	• May discontinuities filled with special materials and in presence of water affected by alteration and/ or hydrothermal actions; clacite containing in weakness zones; development of cavities in limstone	Gravitaitonal and topographical stress	$1 < \sigma_{ci}(MPa) < 100$
σ_{ci}: Uniaxial compressive strength of intact rock $\sigma_{ci(fresh)}$: Uniaxial compressive strength of intact rock at fresh state σ_{cm}: Rock mass strength		$Weathering\ Factor = \frac{\sigma_{ci}}{\sigma_{ci(fresh)}}$ $Strength\ Scaling\ Factor = \frac{\sigma_{ci}}{\sigma_{cm}}$		

3.4.1. Massive rocks

A massive rock could be a type of igneous, sedimentary or metamorphic rocks. Also, igneous rocks are typically massive. Massive rocks usually consist of crystalline texture with a wide range of grain size from less than 0.002 mm such as obsidian and dolerite to more than 6mm like granite and marble. Massive rocks may contain a hydrothermal alteration in near contact boundaries.

Massive rocks are described as rock masses with few discontinuities or with a wide range of spacing (Stille and Palmstrom, 2008). Discontinuities do not have a considerable effect on the properties of rock masses in comparison to intact rock properties (Marinos et al., 2005). The strength of massive rock can vary from weak to strong. Weak rocks such as salt have elastic to deformable or plastic properties, while strong rocks like granite mostly behave in a brittle manner.

Major structures in massive rocks are joints and fractures with a spacing of more than 2 m. Primary estimation of block size in massive rock is more than 8 m^3. Fractures in massive rocks are commonly surface and regional types.

Massive structures are generally affected by topographic, gravitational and residual stress. Gravity and thermal stress in igneous rock are due to the crystallisation process in rocks. Compaction, cementation and crystallisation processes cause gravity stress in sedimentary rocks. Tectonized and thermal stresses in metamorphic rocks could be created under high pressure and temperature conditions.

Strength characterisation of massive rock is estimated as below:

$$\textbf{Strength Characteristics of Massive Rocks} = \begin{cases} \mathbf{0.4 \leq Weathering\ Factor \leq 1} \\ \mathbf{1 \leq Strength\ Scaling\ Factor \leq 2} \end{cases}$$

3.4.2. Jointed/blocky/bedded rock

Jointed, blocky, or bedded rocks are usually defined as rock mass structures intersected by several discontinuities, which form rock blocks. Layered and bedded rocks are mostly sedimentary, and their minerals are usually not interlocked but cemented together with anisotropy properties (Price, 2009). The spacing and orientation of discontinuities in rock structures influence the shape and size of blocks. The degree of jointing in rock masses can be measured by joint spacing, joint density, block volume and rock quality designation (RQD) from observation surfaces, scanline mapping and drill cores

(Palmstrom and Stille, 2015). The quality of joints in a rock mass is described by the shape and size of rock blocks generated by intersecting joint sets.

Figure 3.20 illustrates the common types of rock blocks concerning joint spacing and pattern. A brief definition of each one is:

- Polyhedral blocks: Irregular discontinuities with small persistence create the rock blocks
- Tabular blocks: One predominant discontinuity set intersects with minor parallel joints
- Equidimensional blocks: Three dominant and orthogonal discontinuity sets intersect and create rock blocks
- Rhombohedral blocks: Blocks are produced by the intersection of three or more dominant and indirect discontinuity sets
- Prismatic blocks: Two predominant, parallel and orthogonal joint sets with minor joints make prismatic blocks, and the block thickness is less than the block width or length
- Columnar blocks: They are generated by more than three joint sets, which are parallel and irregular, where block lengths are bigger than the width and thickness

In this class, discontinuities are the main feature of the rock mass and influence the behaviour of rock materials. The orientation and degree of joints in rocks make the type and quality of rock blocks. Bedding, weathering, infilling material of joints and humidity affect the blocky ground. Discontinuity spacing varies between 0.6 m and 2 m. Also, the rock block volume is estimated to be between 0.1 m^3 and 8 m^3 (Figure 3.21). Geological structures are usually affected by gravitational and tectonic stresses in jointed and blocky rocks.

The strength characteristics of jointed and blocky rocks are estimated to be between 0.3-1 and 2-10 for the weathering factor and the strength scaling factor, respectively.

3.4.3. Blocky/folded rocks

Blocky and folded rocks are associated with rock masses affected by tectonic activities and discontinuities. The major geological structures including veins, foliation, drakes, anticline and syncline appear with jointed rocks. Some metamorphic rocks such as gneisses consist of foliation structures. Additionally, minerals in sedimentary and metamorphic rocks are clastic, crystalline, and slightly to moderately decomposed.

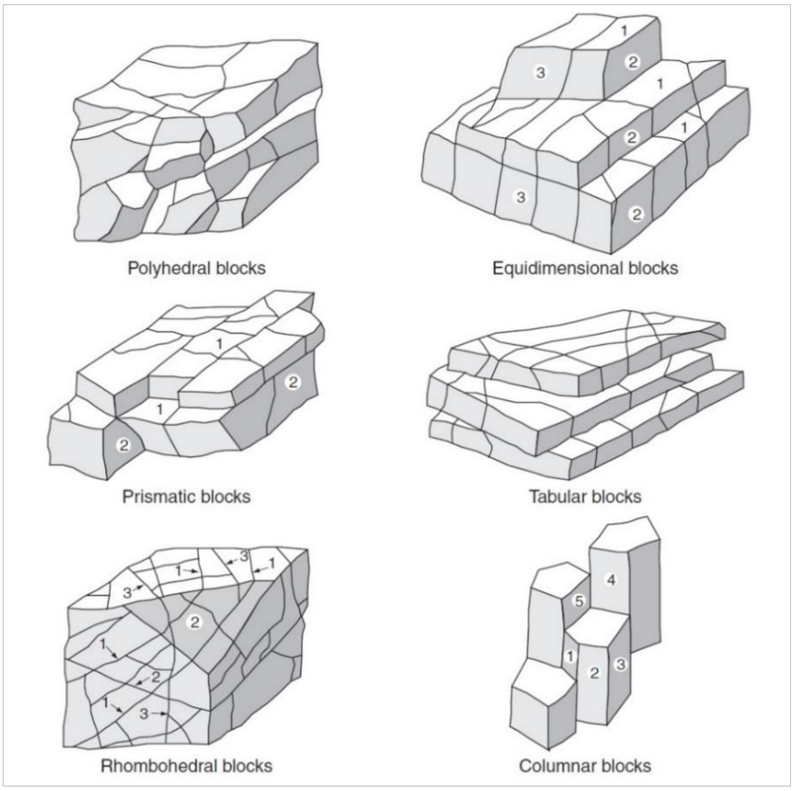
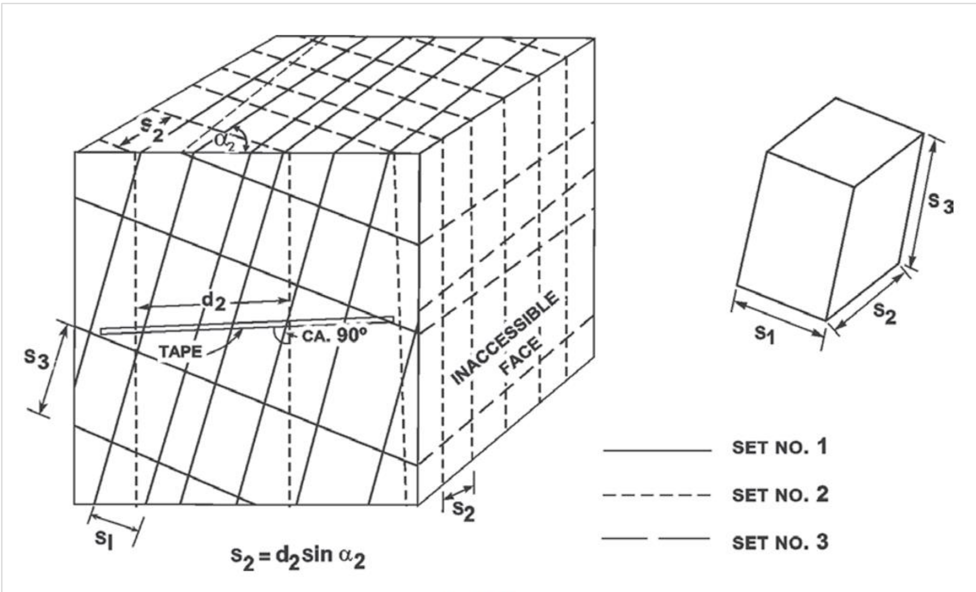


Figure 3.20. Typical geometry of rock blocks (Dearman, 1991)



$$V_b^0 = \frac{S_1 S_2 S_3}{\sin \gamma_1 \sin \gamma_2 \sin \gamma_3}$$

Figure 3.21. Estimation of rock block volume in a jointed rock mass (ISRM, 2014)

Local stress concentration, tectonic activities and residual stress cause deformation, broken and folded rocks. Tectonic activities result in tension, shearing and extension fractures. In this class, folded rocks contain angular rock blocks, schistosity and foliation planes typically created by more than three joint sets. The properties of blocky/folded rocks are described as:

- Spacing: 0.1 m – 0.6 m
- Block volume: 0.1 dm3 – 100 dm3
- Weathering Factor: 0.2 – 1
- Strength Scaling Factor: 10 – 40.

3.4.4. Disintegrated/crushed/soil like materials

Disintegration is a physical process by weathering or tectonic activities that causes the breaking down of rocks into smaller pieces of minerals or particles. The disintegration of some rocks like mud rocks includes hybridisation and oxidisation which is due to changing humidity and temperature in the environment (Stille and Palmstrom, 2008). The essential characteristics of this type of rock material include highly jointed, crushed rock with low bonding rock/soil material. The quality of interlocking between rock blocks, the interaction between grains and blocks, shape and size of grains, and cementation of grains controls the behaviour of this type of rock structure. Meanwhile, disintegrated, crushed/soil materials are often found in faults or weakness zones.

The mechanical properties of many shear and fault zones are lower than those of surrounding rock structures. This may be due to faults' movements; crushed, infilling material of rocks with special properties like soluble minerals; and hydrothermal fluid activities. Figure 3.22 shows typical sketches showing filled discontinuities in faults and weakness zones. The degree of fracturing, the condition of wall rocks in shear zones, dimension of crushed zones, lithological contacts, and properties of infilling material should be described for the characterisation of rock structures.

Disintegrated rocks are tectonically deformed, and consist of intensively folded, faulted, laminated, brecciated dykes or layers, and are heavily jointed. The composition of minerals sometimes changes due to rock alteration and chemical weathering. Soil like materials usually has low friction properties.

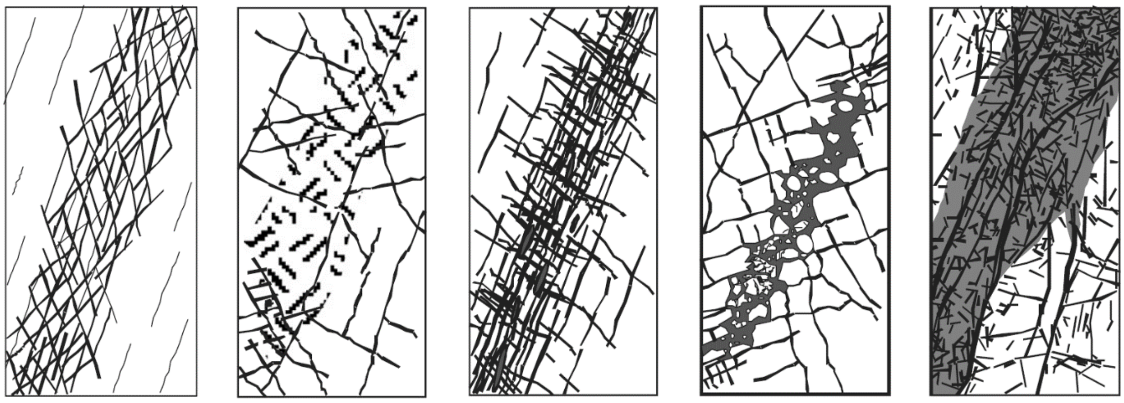


Figure 3.22. A typical scheme of filled discontinuities in complex crushed zones (ISRM, 1979)

Discontinuity spacing and rock block volume are usually less than 10 cm and 0.1 dm^3, respectively. Meanwhile, strength characterisation is defined as:

$$\textbf{Strength Characteristics of Disintegrated rocks} = \begin{cases} \mathbf{0 \leq Weathering\ Factor \leq 1} \\ \mathbf{Strength\ Scaling\ Factor \geq 40} \end{cases}$$

3.4.5. Special materials

Some rocks contain special materials or minerals, which influence their physical and engineering properties in particular conditions. Some minerals like quartz and topaz have high Moh's scale of hardness and present unfavourable conditions for drilling and blasting of excavations. Swelling clay minerals such as montmorillonite and bentonite absorb water and cause alteration of infilling discontinuities, weakness zones and bedding that lead to the reduction of shear strength in rock materials (Stille and Palmstrom, 2008). This group of rock structures is often soft and weak. The proportion of minerals and mineral properties considerably influence rock mass structures.

Furthermore, Table 3.6 presents a description of special properties of some minerals and rocks and their influence on ground behaviour. Alteration and weathering influence the strength and deformation properties of the rock. Frequent changes in temperature and humidity may affect slaking rocks by hydration, oxidisation and disintegration. Some minerals such as mica and pyroxene have anisotropic and elastic properties. The strength of rocks containing these type of minerals reduces in the direction of cleavage surfaces. Meanwhile, soluble minerals, such as calcite in limestone, may lead to permeability problems especially in weakness zones in underground structures. Considering the infilling materials of discontinuities, which have soluble minerals, is essential to characterise rock structures.

In the following sections, the method is applied to several examples in underground excavation projects.

3.5. Case studies of rock mass classification

Based on the proposed process for rock mass structures in underground excavations, some examples from mining projects are considered in the following sections.

3.5.1. Case example A

This gold mine deposit is located in the Norseman-Wiluna greenstone belt in Western Australia. Figure 3.23 shows information related to geological formation and lithology of the mine. The eastern part of the geological formation is dominated by coarse-grained monzonite and porphyritic monzonite.

Table 3.6. Description of special properties of some minerals and rocks (Palmstrom and Stille, 2015)

Type of minerals and/or rocks	Description
Clay minerals	Occurs either as a rock constituent (e.g. in highly weathered or altered rocks) or as a filling and coating material in seams and filled joints as a result of alteration and/or hydrothermal actions. A special group of clays are the smectites (swelling clay) mentioned below
Swelling minerals (smectite, anhydrite)	A change in the moisture content in swelling minerals of the smectite (montmorillonite) group can cause significant instability problems related to high swelling pressures. These minerals, occurring either as a rock constituent or as infilling or alteration products in seams or faults, have, in addition to expansion, a low shear strength, which may contribute to rock falls and, in some cases, slides in underground openings and cuttings. Swelling rocks with clay are montmorillonitic shales, altered or weathered basalts, tuffs or other crystalline rocks. Anhydrite is one of the major minerals in evaporite deposits; it is also present in dolomites and limestones, and as a gangue mineral in ore veins
Porous rocks	Such rocks may be porous sandstones and some deteriorated or decomposed rocks where parts of the minerals or mineral bonds have been dissolved. Such rocks may contain abundant water and cause flowing ground or other water inflow problems to tunnels
Slaking rocks	Some rocks may slake (hydrate or 'swell', oxidise), disintegrate or otherwise weather in response to the change in humidity and temperature consequent on excavation
Anisotropic minerals	Minerals such as mica, chlorite, amphiboles and pyroxenes may significantly influence the mechanical properties of the rocks in which they occur. Also, other minerals such as serpentine, talc and graphite reduce the strength of the rockmass due to easier sliding along the cleavage or coated joint surfaces *Continuous* layers of mica and chlorite may, in addition, cause drilling and blasting problems. Typical rocks are mica schists and some phyllites. Both show strong anisotropic properties
Friable rocks	Loosely cemented, mostly sedimentary rocks
Soluble minerals (calcite, salt)	Calcite is susceptible to being dissolved by groundwater. This process may cause permeable regions in the calcite-containing weakness zone, as well as the development of cavities in limestones (karstification) Karst cavities can form large drainage systems, causing water problems during tunnelling when encountered
High quartz content[a]	A high content of, especially, fine-grained interlocking quartz minerals in some rocks often causes high drill bit wear and tunnel-boring machine cutter wear

[a] Has generally little impact on the ground behaviour in excavations

The central part of the deposit is formed of volcanoclastic sediments and intruded dykes of monzonite and lamprophyre. The geological formation in the western part consists of fine to coarse-grained sandstone and siltstone. The main feature of the geological structure involves the hanging wall fault with a northwest strike which separates the eastern part and the central part of the mineralised zone. Similarly, the central and footwall sedimentary sequence is separated by the footwall fault and consists of several metres of chlorite alteration.

Site investigation methods, geological engineering mapping and laboratory tests were conducted for the description of rock mass structures on the mine site. Intact rock samples from different lithology were tested in a laboratory test for estimation of the rock strength. Uniaxial compressive strength varies from 50 to 100 MPa for the rock types encountered. The discontinuitiy condition was studied by engineering geological survey in the main access decline and some access levels in the mine site. The results of engineering geological mapping indicated several joint sets in the mine area with a range of dip/dip direction of (30–70°)/(50–300°). Trace length of joints varies between 2 m and 10 m. Rock types have mostly been discoloured due to weathering action.

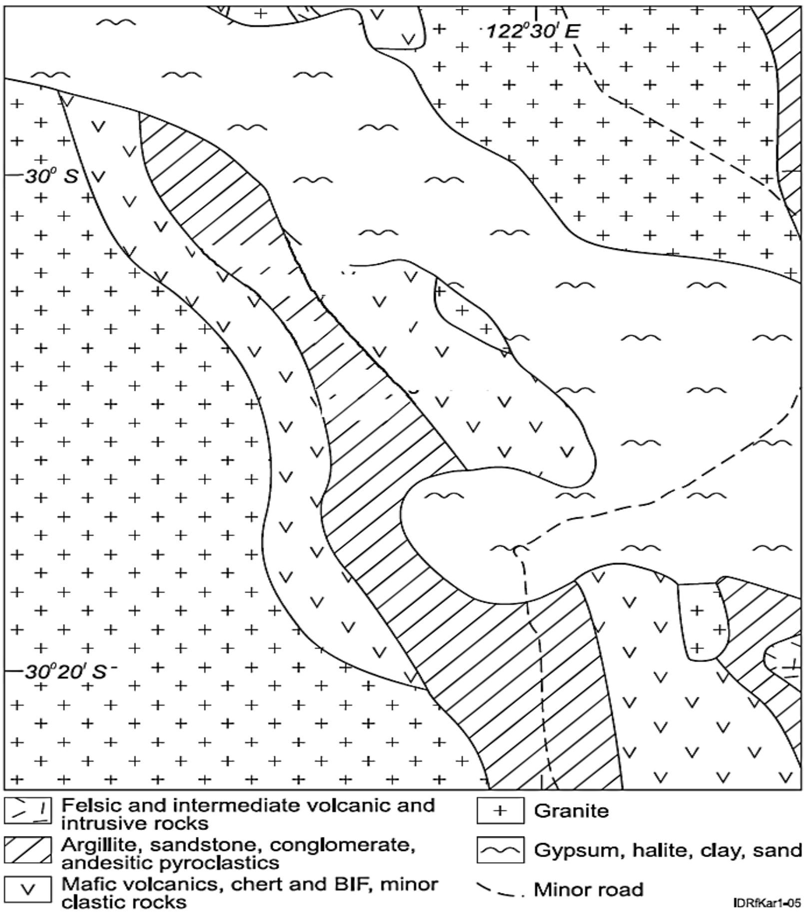


Figure 3.23. Geological information and lithology of the deposit (Gray et al., 2005)

Figure 3.24 shows structural features in rock masses in the mine site. Rock blocks have been formed by intersecting joints distributed such that the interlocking is determined as the medium to good. The Stereonets discontinuity mapping from the mine site is shown in Figure 3.25. The joint roughness is mostly classified as slightly rough-to-rough. Also, surfaces between joints are often clean. Weathering is recognised as a discoloured surface rock. Furthermore, sodic alteration, which is in direct contact with sodic minerals and groundwater, is locally visible in the rock zones of the mine.

Based on the collected data from the mine site and processing data, rock mass structures are characterised as Jointed/blocky/bedded, and detailed information and the characterisation process is shown in Figure 3.27.

3.5.2. Case example B

This gold deposit is hosted in the meta-sedimentary rocks in the Laverton tectonic zone in Western Australia. Figure 3.26 shows geological information of the mining district. The deposit consists of a series of shallow dipping ore mineralisations in mafic and conglomerate structures. The main rock types are sandstone, granodiorite, basalt, siltstone, shale and sandstone. Alteration fluids in the conglomerate produce magnetite – pyrite, and a collection of dolomite, albite, pyrite, quartz and gold.

Major structures in the mine site are joint sets, faults, shear zones and veins. The joint sets are oriented as below:

- Joint set 1 (dip/dip direction): $86^{o}/25^{o}$
- Joint set 2: $2^{o}/55^{o}$
- Joint set 3: $83^{o}/105^{o}$
- Joint set 4: $49^{o}/342^{o}$

Site investigation methods indicated that rock structures consist of tectonic fracture including shear and tensile type rock surfaces. Vein structures appear in rock structures in the mine site and also drilling cores with thicknesses about 2 – 10 cm, and contain chlorite, calcite, quartz and clay. Persistence of joint sets is in the range of 1 – 15 m. The nature of joint surfaces is mostly undulated to rough. Joint spacing in rock mass structures is between 0.5 and 4 m. The fractures are mostly dry to humid.

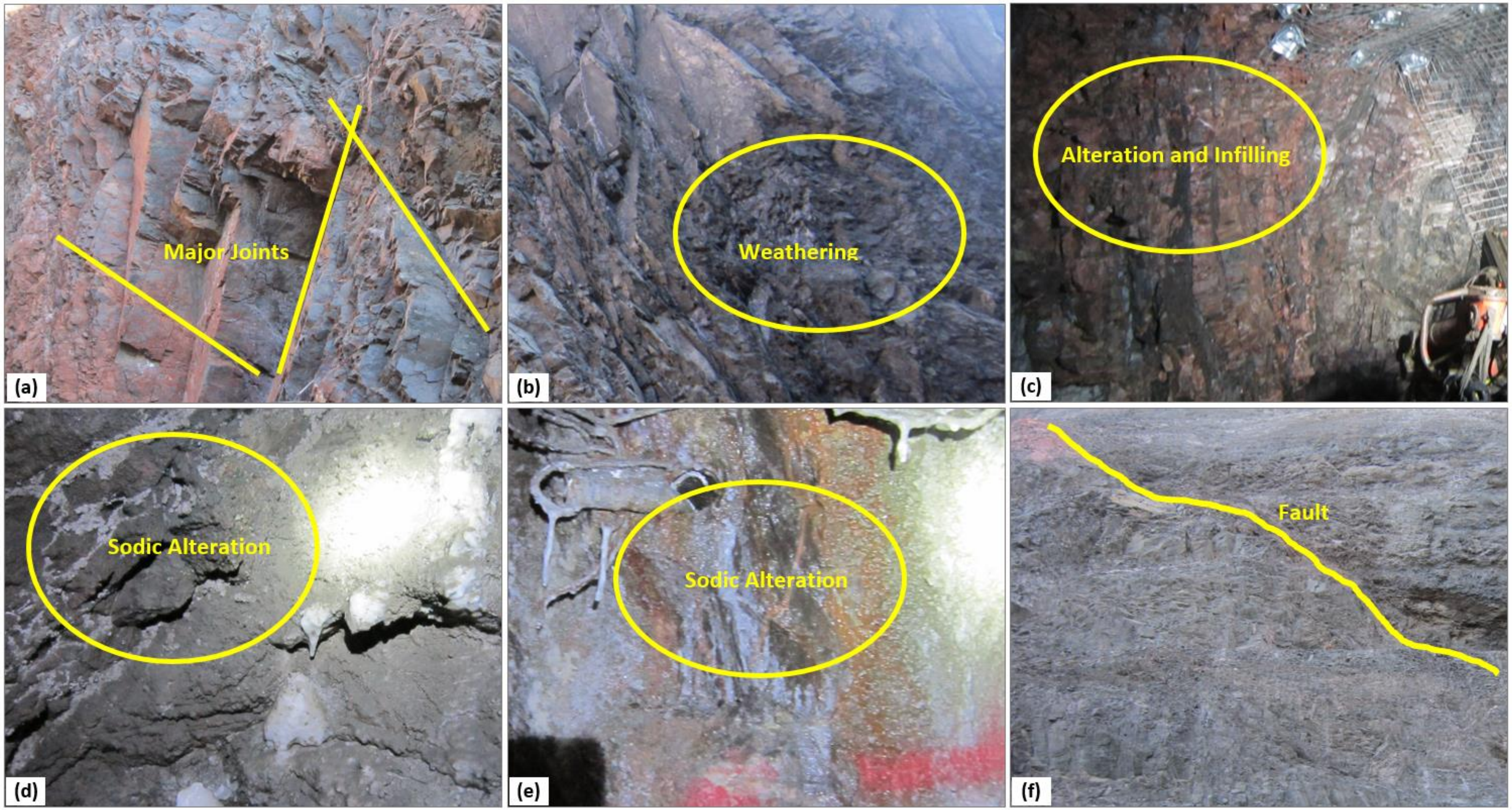


Figure 3.24. Structural features in the rock mass in the mine site; (a) Major joint sets, (b) Weathering, (c) Alteration and infilling, (d) and (e) Sodic alteration, (f) Fault.

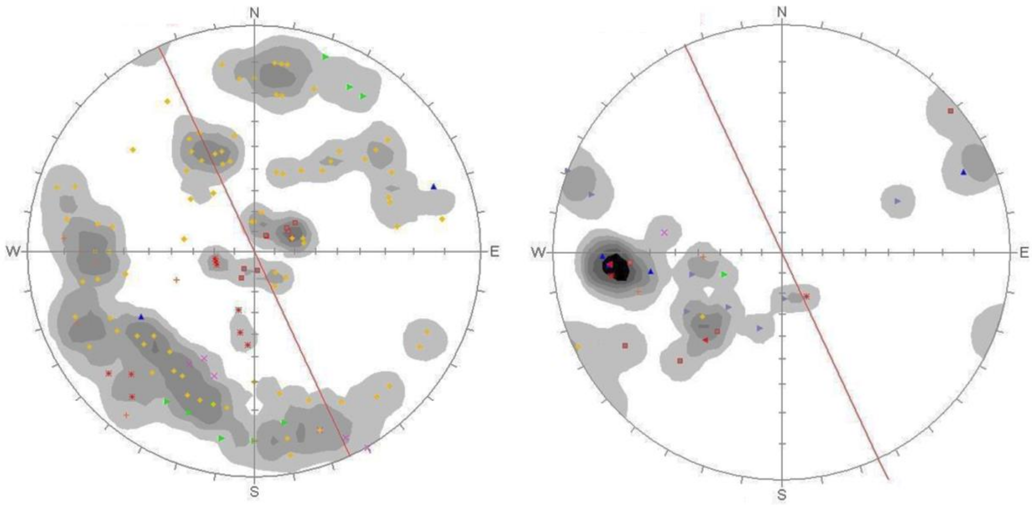


Figure 3.25. Stereonets discontinuity mapping

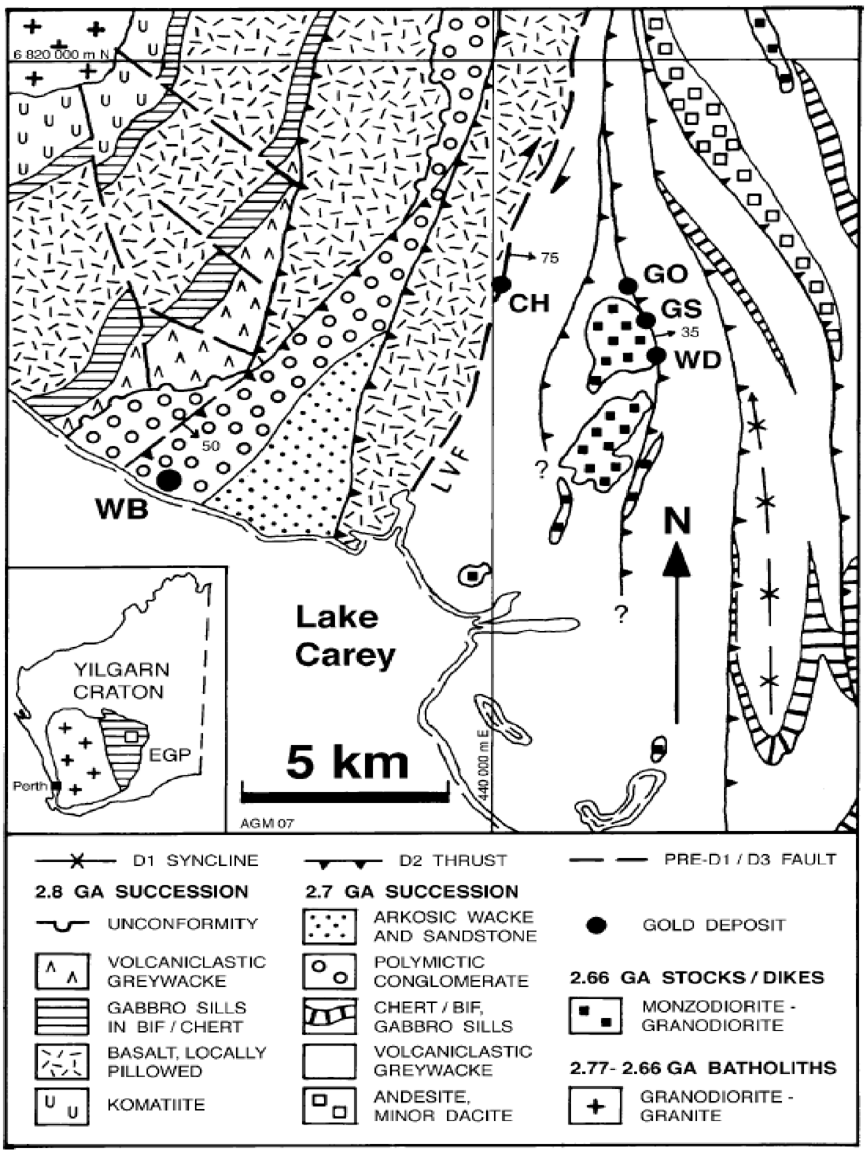


Figure 3.26. Geological map of the mining district (Mueller et al., 2008)

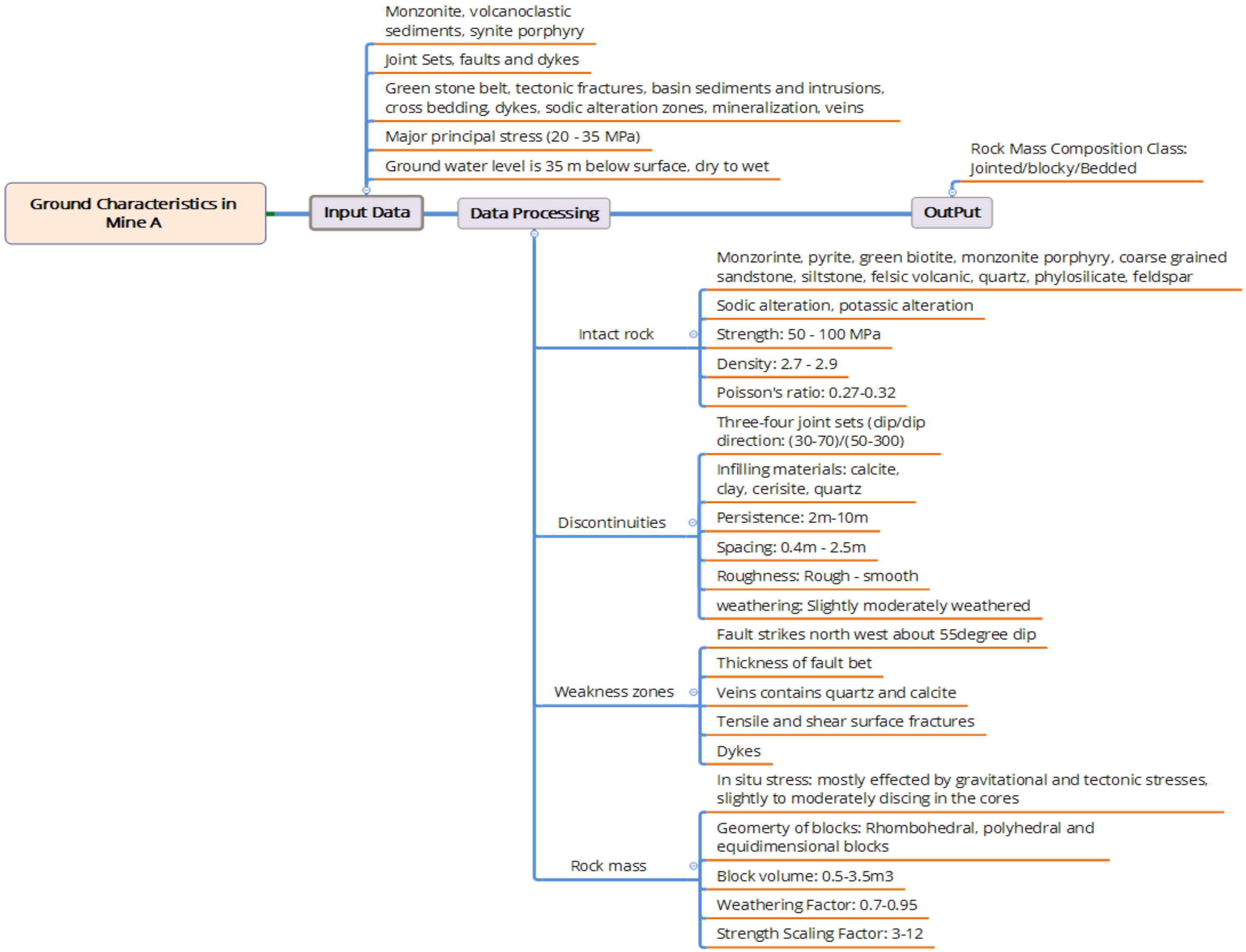


Figure 3.27. Ground characteristics of the mine site

Also, fault zones with a north-west strike consist of 2-3 m calcite alteration. The properties of intact rock from laboratory tests were estimated to be:

- Intact rock strength: 140 – 190 MPa
- Young's modulus: 55 – 75 GPa
- Poisson's ratio: 0.28 – 0.35

In the case of hydrology, the groundwater table is about a few metres below the natural ground surface. Groundwater causes weathering and alteration inside rocks and sometimes causes corrosion of ground support equipment. Figure 3.28 shows some features of rock structures at the mine site.

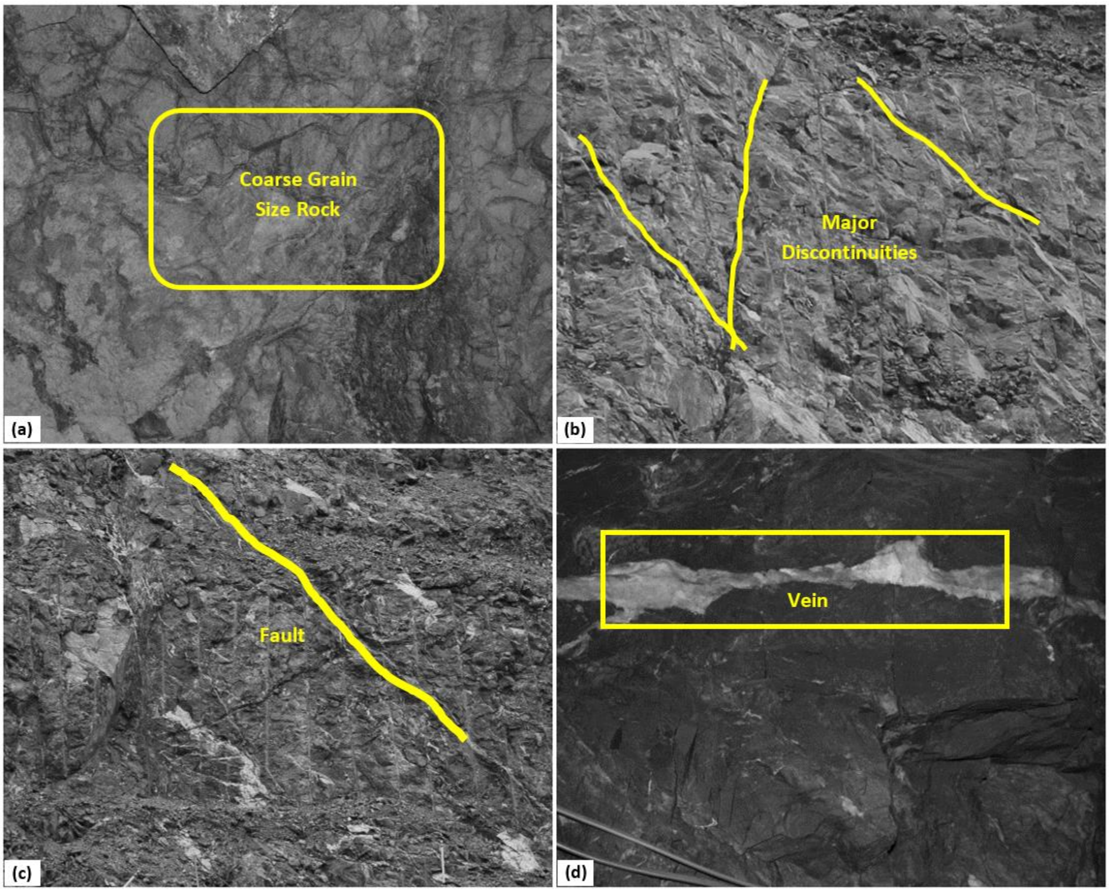


Figure 3.28. The features of rock masses at the mine site: (a) coarse grain size rock, (b) orientation of major discontinuities, (c) fault zone, (d) vein in the rock

In situ stress was estimated by empirical methods and the magnitude of major stresses vary between 18 MPa and 43 MPa.

Characterisation of rock mass structures at the mine site is presented in Figure 3.29. The composition of rock mass structures is categorised as being in the jointed/blocky/bedded and blocky/folded classes.

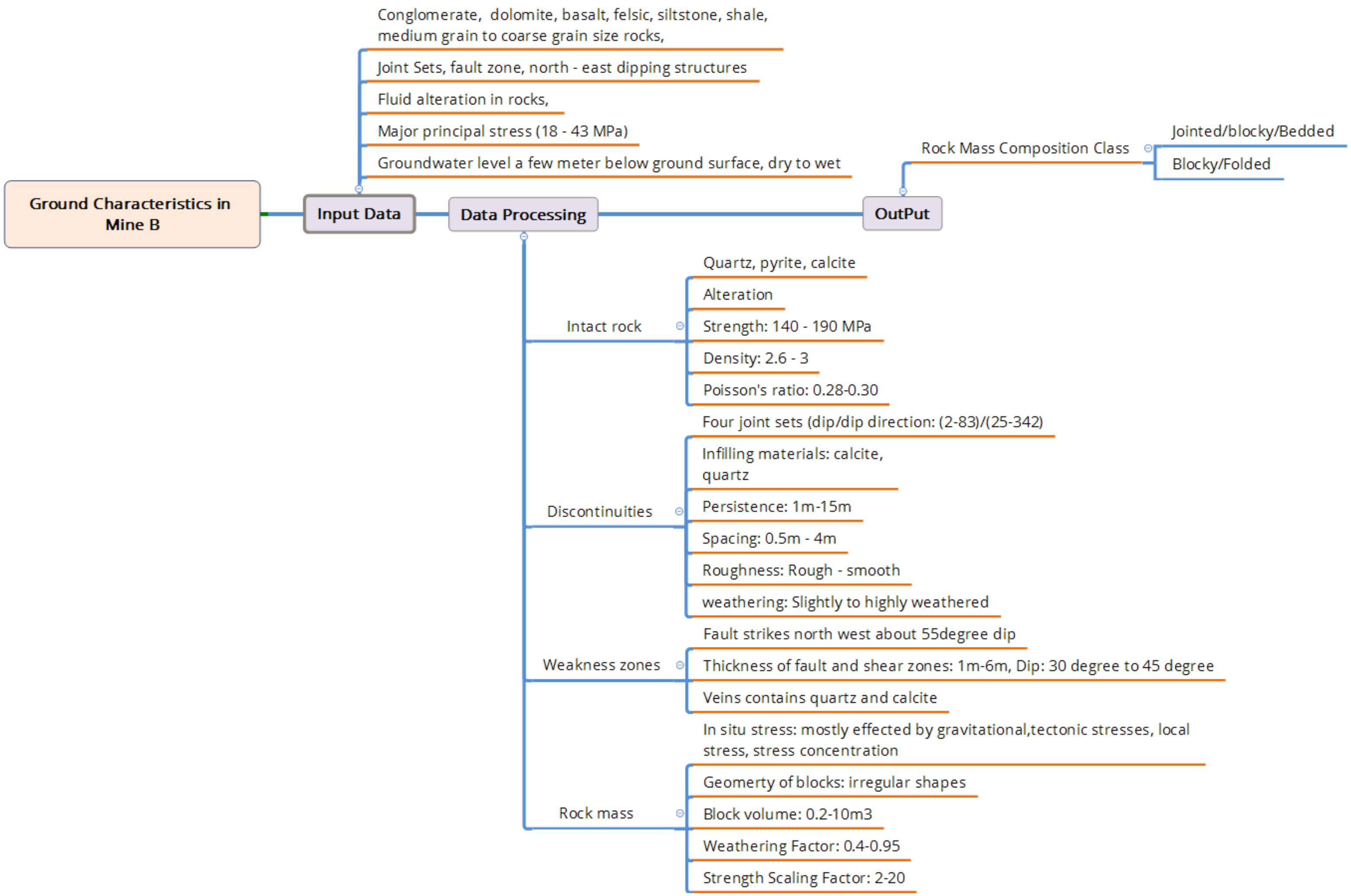


Figure 3.29. Classification of rock mass structures at the mine site

3.5.3. Case example C

The case study is a hydropower project, which is a type of an in-depth underground excavation project (Jiang et al., 2010, Duan et al., 2017). Figure 3.30 shows a layout of the cross-section of geological formations and underground excavations. A series of excavated tunnels at shallow depth has been shown in Figure 3.30–(b). The project consists of different geometry of underground caverns and tunnels from 350m to 2500m depths (Figure 3.30–(a) and (d)).

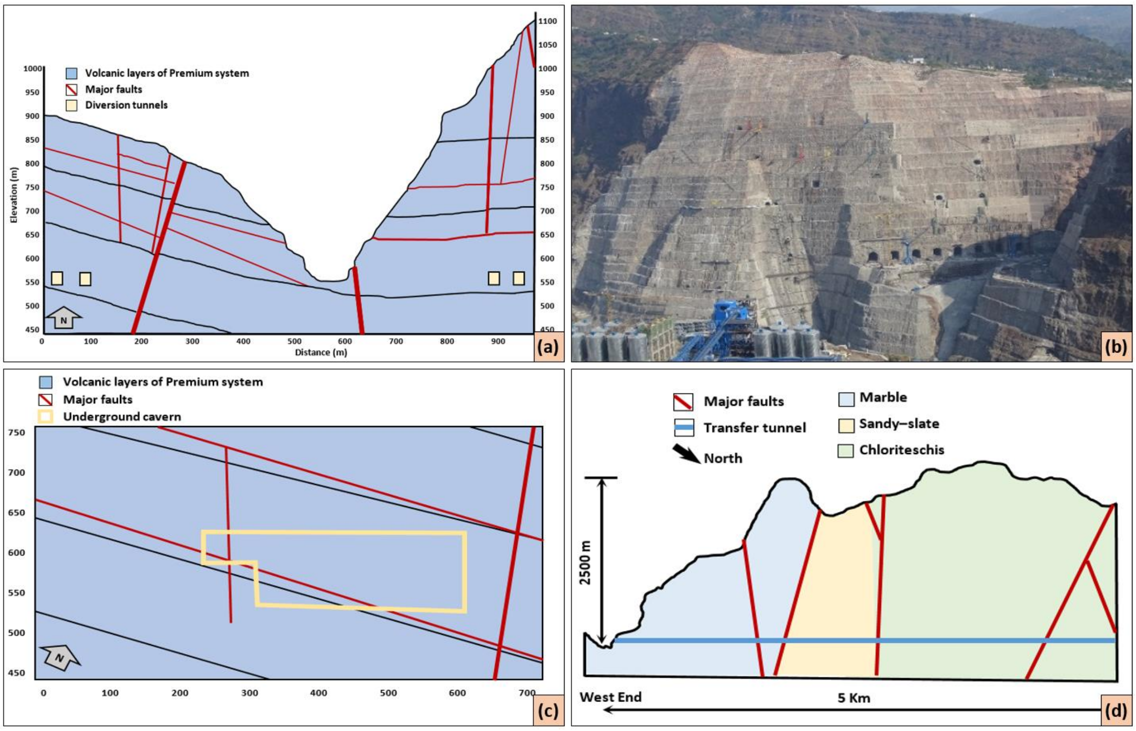


Figure 3.30. The layout of the hydropower project; (a) cross-section of geological formations, (b) a layout of tunnels, (c) a cross-section of an underground cavern in 500m depth, (d) longitudinal cross-section of geological structures of transfer tunnel in 2500m depth

Geological formation mainly consists of basaltic layers from volcanic eruptions and lead to the stratigraphic column, which belongs to Premium system at Emei mountain group with N30°-50°E trend and dip about 15°-25°SE dip (Duan et al., 2017). Devonite basalt, amygdaloidal basalt, Aphanitic basalt, breccia lava and cryptocrystal basalt are the main rock types in underground excavations as shown in Figure 3.30–(a) and (c). The basalt layers is a kind of columnar joints in rock structures, which is formed by a thermally induced cooling process in the lava. Figure 3.30–(d) shows a geological formation of the transfer tunnel at 2500m depth. The main lithologies along the tunnel are marble, sandy–slate, and Chloriteschis. Several faults and shear zones/interlayer weakness zones were observed in geological formations at the field study. The major geological structures in underground excavations have been shown in Figure 3.30–(a), (b) and (d). Weakness zones mostly have loose rock structures with weak mechanical properties and variable thickness among main rock units. The thickness of rock zones affected by faults/shear zones is between 0.5m and 8m. Also, interlayer weakness zones containing soil/rock interfaces vary from 5cm to 30cm.

Figure 3.31 shows the structure of rock mass condition at the project. The characteristic features in the rock structures are fracturing and jointing, bedding, shear and fault zones, and columnar joint sets, which affect the quality and behaviour of the ground condition in the underground excavation. Engineering geological mapping and site observation indicate the main dominant discontinuities condition with dip/dip direction as to follow:

1- Joint set1: $(80^o\text{-}90^o)/(40^o\text{-}65^o)$
2- Joint set2: $(55^o\text{-}70^o)/(15^o\text{-}35^o)$
3- Joint set3: $(55^o\text{-}70^o)/(15^o\text{-}35^o)$
4- Fault and shear zones: $(15^o\text{-}20^o)/(45^o\text{-}50^o)$

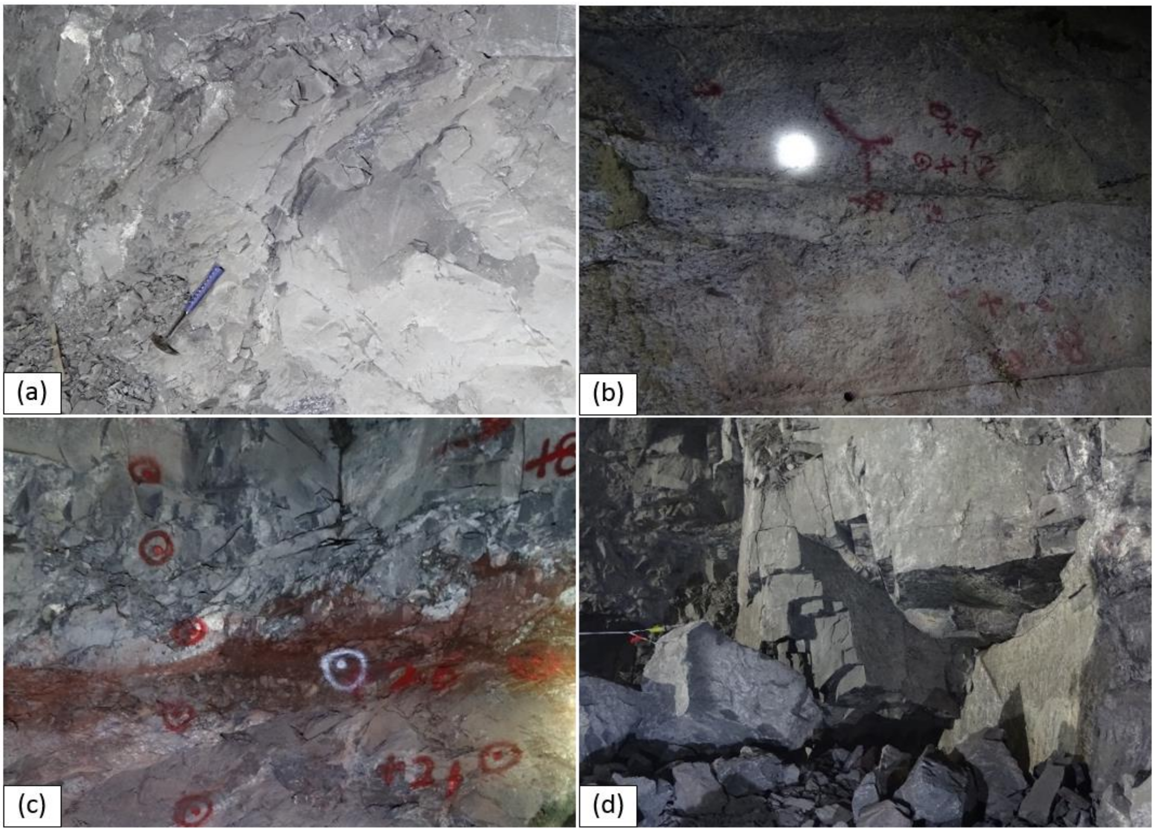


Figure 3.31. Rock mass structures in the main underground caverns; (a) fracturing, (b) bedding, (c) shear zone, (d) columnar joint sets

Also, joint spacing varies between 0.5 – 5m. Generally, the roughness of joints was determined as the rough and undulated condition. The main infilling materials were carbonate and calcite. Joint persistence varies from 1m to 30m based on site investigation and geological engineering surveys. The properties of intact rocks were estimated from laboratory tests as Uniaxial compressive strength: 120-170MPa, Young's modulus: 15–40GPa, Tensile strength: 5–15MPa and Poisson's ratio: 0.25–0.27. Principal field stresses in underground caverns were estimated between 13 and 33MPa. The maximum principal stress was about 60MPa at 2500m depth. The stress field is affected by regional tectonic structures and local topography.

Figure 3.32 shows detail information of ground characterisation in the main underground caverns. According to this flowchart, rock mass composition is classified into jointed/blocky/bedded class in main unit rocks and blocky/folded in weakness zones.

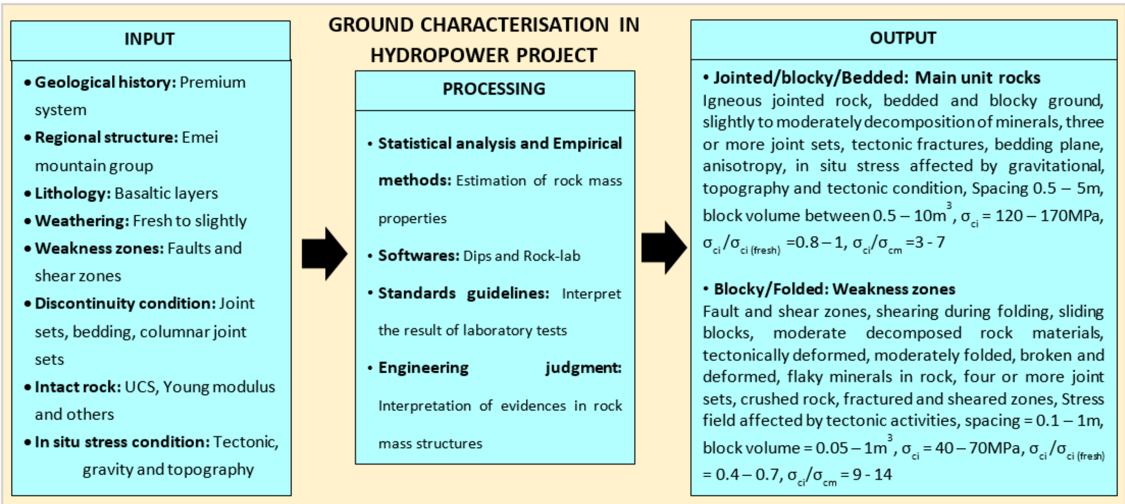


Figure 3.32. The rock mass composition of main underground caverns

3.5.4. Case example D

This case study focuses on a road tunnel that runs through the Alborz Mountain Range. The planned length of the tunnel is about 5 km and horseshoe-shaped with 12 m width and 9 m height. Figure 3.33 shows the longitudinal geological section along the tunnel. The geological formation mainly consists of Eocene Karaj tuff rocks, which contains volcano-sedimentary units and mostly consists of andesite and tuff. The site investigation results indicate that alteration has happened in rock materials containing clay minerals such as chlorite.

Based on the XRD analysis, the andesite consists of feldspar, chlorite, carbonates, pyroxene and opaque minerals. The tuff is composed of silica & fine quartz grain, chlorite and other clay minerals, iron oxide, carbonates and other opaque minerals.

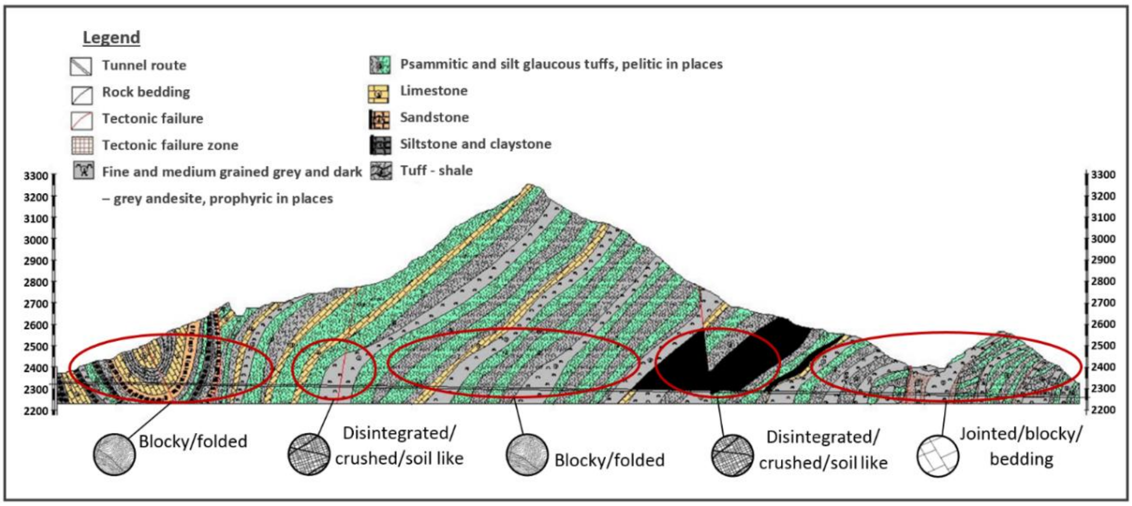


Figure 3.33. Longitudinal geological profile of the tunnel

The number of 471 joints has been recorded by engineering geological survey to determine discontinuities condition. Also, the mechanical properties of rock material were determined by laboratory tests. Based on empirical methods, in situ stress condition estimate in the range of 7-10 MPa. Collected data has been processed by statistical analysis (to determine the reliable value of rock engineering properties), empirical methods (to estimate in situ stress condition), and Dips software (to the analysis of discontinuities condition).

Table 3.7 presents the mechanical properties of rock type and dip/dip direction of discontinuities in the tunnel. Rock types are classified into three domains: andesite, tuff and faulted tuff. The andesite is mainly fresh to slightly altered rock, and there are some interfaces with moderately altered tuff. The tuff rocks are in contact with weakness zones like faults and shear zones with heavily broken rock containing angular blocks and folded rock originating from tectonic activities.

Figure 3.34 shows an example of different types of rocks along the tunnel. The spacing of discontinuities varies from less than 10 cm in altered tuff rocks to 2m in andesite–

basalt rock. The groundwater condition observed is dripping in the faulted tuff rock and has caused highly altered and weathered rock zones.

Table 3.7. The results of physical and mechanical properties of the main rock units rocks along the tunnel route

Parameter	Andesite	Faulted Tuff	Tuff
Dip/Dip Direction of joints	(50-70)/(55-270)	(30-85)/(65-260)	(25-85)/(200-275)
$\sigma_{ci}(MPa)$	100 – 140	20 – 40	85 – 110
RQD	70	33	62
$Density\ (g/cm^3)$	2.72	2.3	2.5
E (MPa)	21	8	12
ϑ	0.24	0.36	0.3

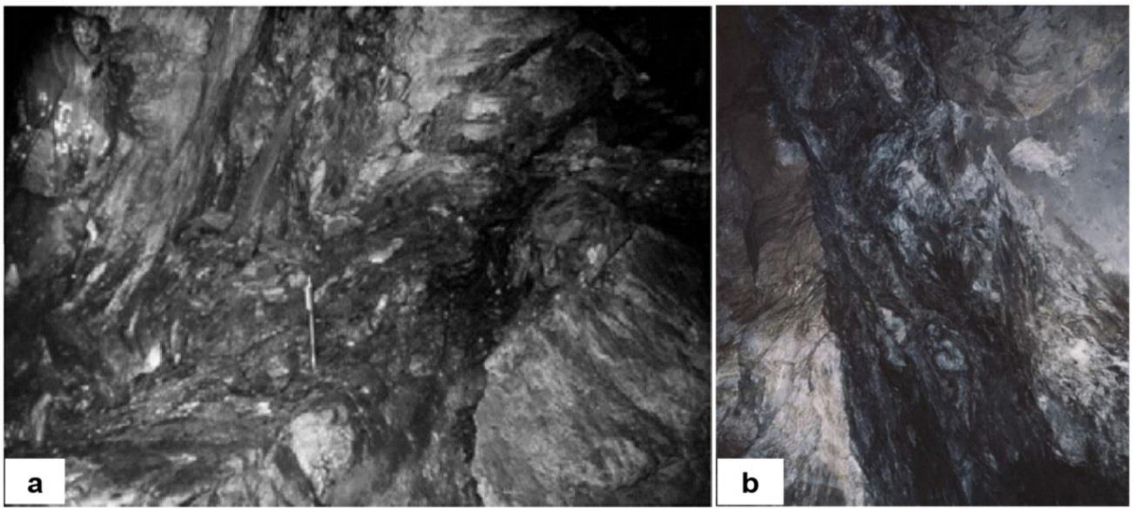


Figure 3.34. (a) Altered and foliated tuff rock, (b) A contact zone of andesite – basalt and tuff rocks

Figure 3.35 illustrates the summary of rock mass classification in the tunnel. Different classes of rock mass composition along the tunnel and for different lithology has been presented in Figure 3.33. Different type of rock mass composition in the tunnel indicates the complexity of geological structure in the projects. Characterisation of rock structure can be used to predict ground behaviour and failure modes that is an essential step for design analysis in the projects.

3.6. Conclusion

A developed classification of rock mass composition for deep underground hard- rock mining presented. The classification was established in three main steps: considering effective features in rock mass composition (input data); processing data which involve analysis and interpretation of data; and rock mass classes (output data).

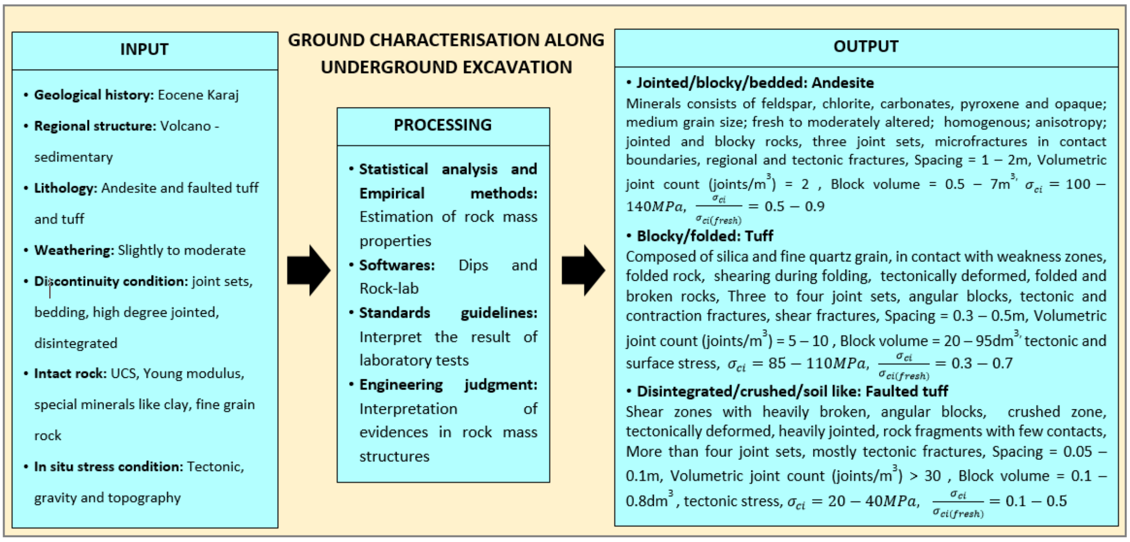


Figure 3.35. Rock mass classification along the underground excavation

The input data is a determination of rock engineering properties such as rock strength, discontinuity conditions, geological history, regional geological structure, and in situ stress state. Uniaxial compressive strength, point load strength, tensile strength, density, porosity, durability and elastic constants are the standard laboratory tests that are used for measurements of geological and mechanical properties of intact rocks. The most important discontinuity types are fissures, joints, bedding planes, cleavage and schistosity planes, and faults. In situ stress condition results from gravitational stress, topographic stress, tectonic stress and residual stress. Groundwater usually change mechanical and chemical properties of some minerals such as clay. Qualitative and quantitative collected data are used for description of rock mass structures.

Data processing is related to organisation, analysis and interpretation of input data. This step deals with intact rock characteristics, discontinuity characteristics, weakness zone characteristics, and rock mass characteristics. To this purpose, geological maps and profiles, statistical analysis, geological modelling, standards guidelines, and empirical methods are utilised as well as knowledge and experience.

The results of data processing are defined in the output step with different groups of rock mass compositions, which are massive rocks, jointed/blocky/bedded, blocky/folded, disintegrated/crushed/soil like materials, and special materials.

A developed rock mass classification was proposed and applied in several case studies in this chapter. The results of case examples confirmed the reliability aspects of rock mass classification in the new method.

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CHAPTER 4: DIAGNOSIS OF GROUND BEHAVIOUR AT DEEP–HARD ROCK MINES

CHAPTER 4: CONTENTS

4.1. Introduction

Ground behaviour in deep underground mining is diagnosed through three steps: firstly considering effective factors on ground condition that are employed as input data; secondly organising, interpretation, analysing collected data in processing step; and thirdly identify ground behaviour modes. Figure 4.1 shows a process of diagnosis of ground behaviour in deep underground mining.

Input data for identifying the responses of rocks during mining operations are rock mass structures, orientation, excavation methods, geometry of openings, active stress factor, static and/or dynamic loading surrounding excavations, and hydrological condition. Design procedure in the underground opening is associated with assessing ground behaviour surrounding excavations. The responses of the ground in the excavation is derived from change loading, drawback natural support, and fluid flow (Price, 2009).

Data processing consider the determination of parameters related to distinguishing the responses of rock mass with mining activities. To this purpose, the relations between major geological structures and excavation methods is assessed with principle stress components and rock mass strength. Also, making a precise observation and careful interpretation of existing evidence in rock mass structures and environmental conditions is the first principle in the diagnosis of ground behaviour (Kaiser and Kim, 2008).

Figure 4.2 illustrates ground reactions in underground excavation due to engineering activity in the regional/local scale. The possible responses of the loading ground elastic/plastic deformation, consolidation settlement and failure. Ground movement, fracturing, filling voids and cave in and collapse are the typical results of removing natural support in the ground. Also, some engineering/natural process in the ground such as mineral extraction and change groundwater level due to precipitation may lead to fluid flow/ gas pressure change.

The output for the description of rock mass behaviour is a developed classification as stable, intact rock failure, structural/intact, structural failure, and water effect classes. The process for diagnosis of ground behaviour at great depth is discussed in this chapter.

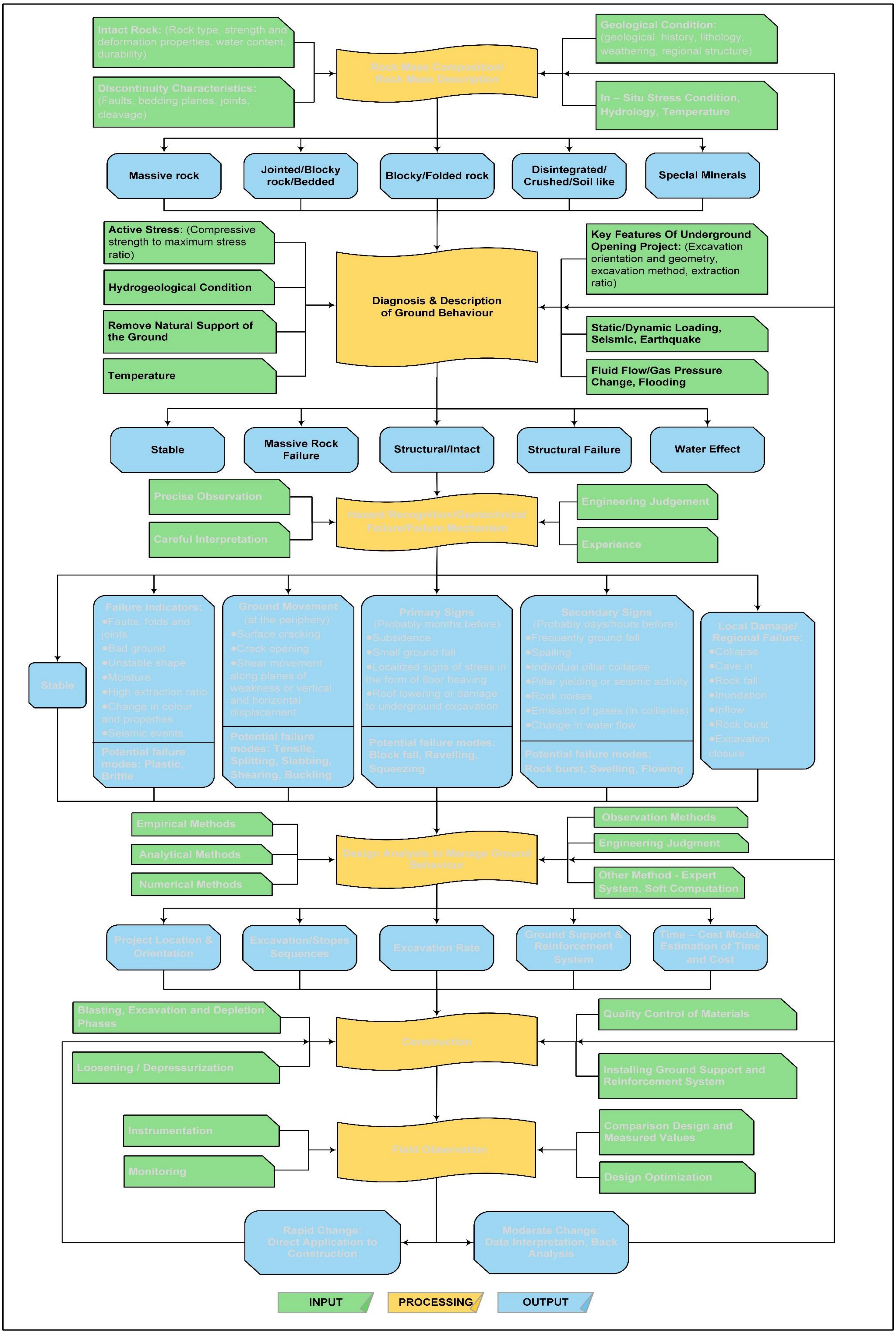

Figure 4.1. The process of diagnosis of ground behaviour in deep underground mining

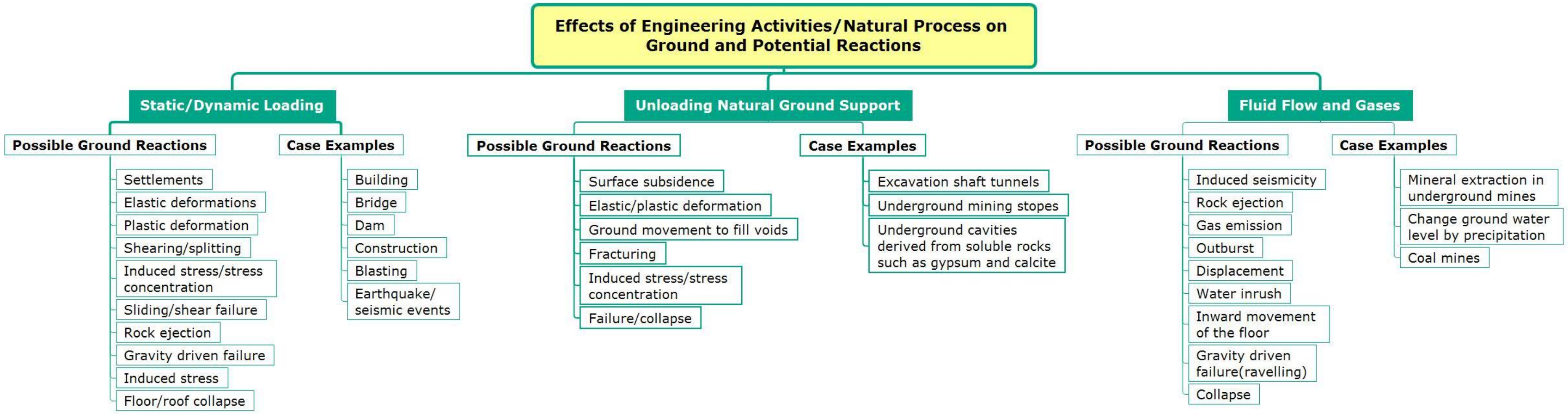

Figure 4.2. Possible ground reactions due to engineering activities/natural environmental conditions in the regional or local scale

4.2. Factors influencing ground behaviour (Input data)

In situ rock mass structures' response to the natural environmental condition and human activities are called ground behaviours. Construction of different types of rock engineering projects may lead to changing ground conditions in three forms: loading ground (such as the weight of a dam), removal of natural ground support (like excavation of a tunnel), and gas pressure / fluid flow changes in the ground (for example, pumping water from a well) (Price, 2009). The underground excavation or underground mining causes the loss of natural support ground by removing soil and rock materials, which may have the following results:

- Deformation of the rock mass surrounding underground excavations,
- Heave of ground towards the centre of the opening,
- Sliding failure in rock slopes, and
- Ground movement and subsidence.

Additionally, groundwater level can be changed by the drainage of an artesian aquifer, precipitation and floods. Pumping water through a well or excavation of ground in underground projects disturbs the fluid/gas pressure balance and affects the stress condition in a rock structure, and may cause local seismicity and failure (Price, 2009). Most rock engineering projects influence ground conditions by more than one of these functions (loading, support withdrawal, and disturbance of fluid/gas pressure). Good understanding of rock mass properties and environmental conditions is necessary to determine appropriate responses. The most effective factors on ground behaviour in underground excavations are shown in Figure 4.3. Major geological structures, active stress factor, hydrological condition, static/dynamic loading conditions are the typical natural environmental factors, which may change and influence the ground behaviour during construction of underground excavations. Meanwhile, engineering activities, including drilling and blasting, usually induce a change of ground condition, as well as the extraction of mineral resources in underground mining.

It is not possible to assess a ground support system without a clear understanding of the ground condition before, during and even after the construction stage in underground works. The behaviour of a rock mass structure can be affected by the state of stress, groundwater condition, the geometry of the opening and also installation of a ground support system. Available data related to natural factors in the ground and human activities in the ground (based on the factors listed in Figure 4.3) is collected, interpreted, analysed and evaluated for the diagnosis of ground behaviour in underground mining projects.

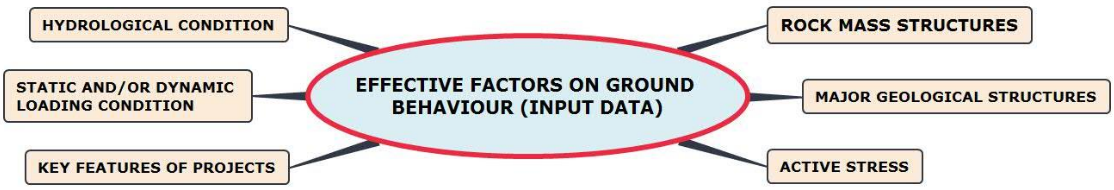

Figure 4.3. Influencing factors on ground behaviours in underground excavations (Input data)

4.2.1. Major geological structures

The typical geological features in rock mass structures, which impact rock mass behaviours are:

- Rock mass compositions
- Special materials: special clay minerals and soluble minerals
- Major discontinuity sets
- Weakness zones including thickness, infilling materials, and fracture types

The geological formation based on major structures in the rocks is divided into several zones related to different particular properties and orientations. Figure 4.4 shows schematic domains of geological formations for the description of ground behaviour based on deformability. Rock mass formations are grouped into different zones based on rock mass characteristics, discontinuity conditions and the orientation of major discontinuity sets. Then each domain is described based on the lithology, rock type, minerals and strength properties. The ground behavioural effects of different zones concerning each other should be considered and interpreted.

Some specific rock structures like foliation give rise to anisotropy and affect fracture orientations and rock strength. In this case, it is better to create one domain zone for foliation and assess responses of rock masses regarding to the intensity and orientation of the foliation in the domain.

Geological structures in fault and shear zones are examined in the subdivision domain attributed to fracture density, infilling minerals, and alteration.

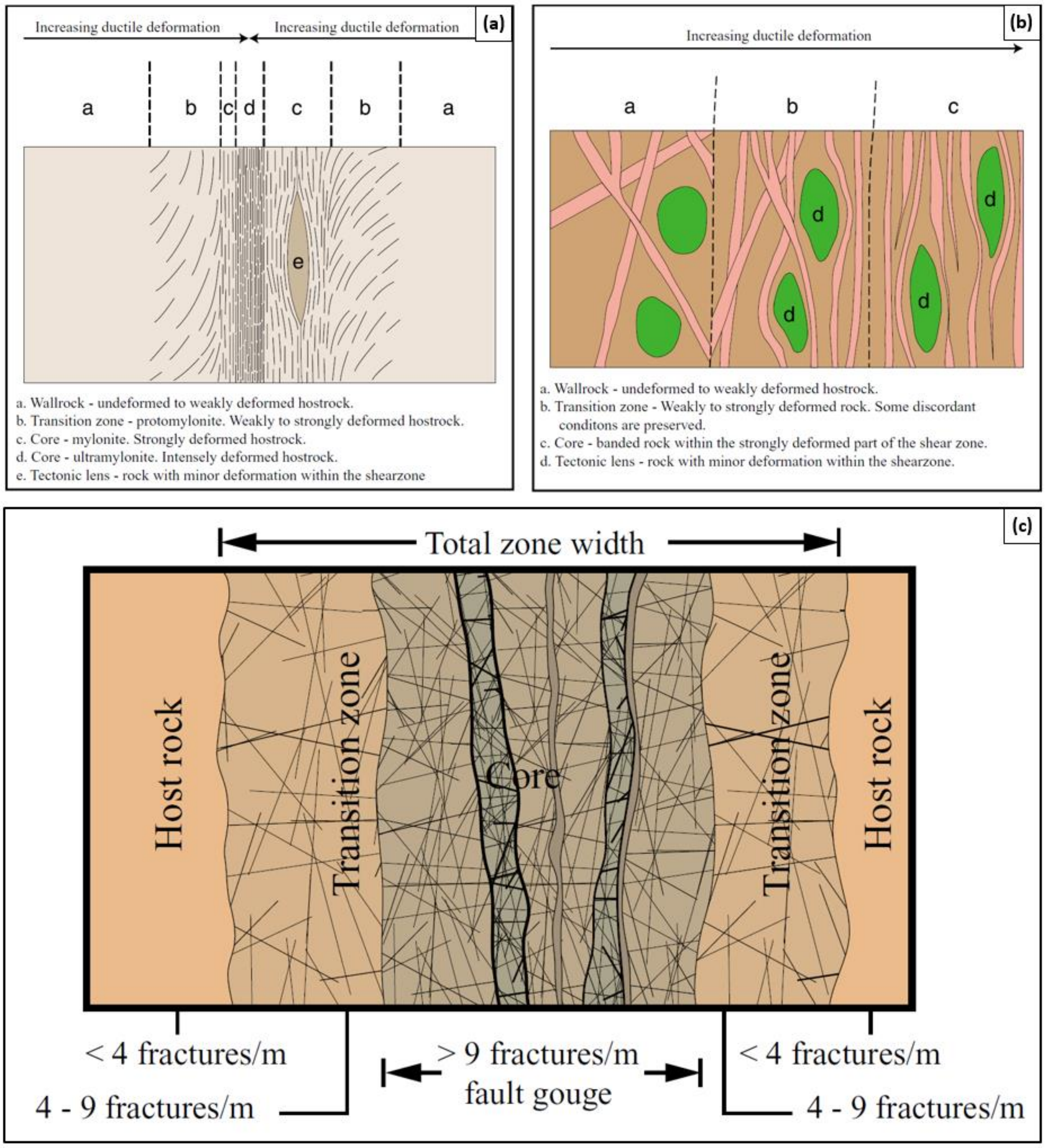

Figure 4.4. Major geological structures and domain/zones for the description of ground behaviour based on deformability; (a) a ductile shear zone, (b) homogenous rock, (c) heterogeneous rock (Modified after (Munier et al., 2003))

It is difficult to visualise, interpret and assess the real orientation of rock structures from direct observations and prepare a geological model. Therefore, uncertainty and confidence is assumed in the delineation of rock features in the geological model used for the description of ground behaviour. Figure 4.5 presents different types of uncertainty and confidence in geological structures, which influence the description of ground behaviour. Typically, uncertainty in dip usually appears in mapping from the surface and when making a geological model. It is required to estimate uncertainty and confidence in the position and structure at depth. Interpretation and description of ground behaviour at depth include the geometry of structures, which are observed information from boreholes and should be justified and correlated for all parts of the zone.

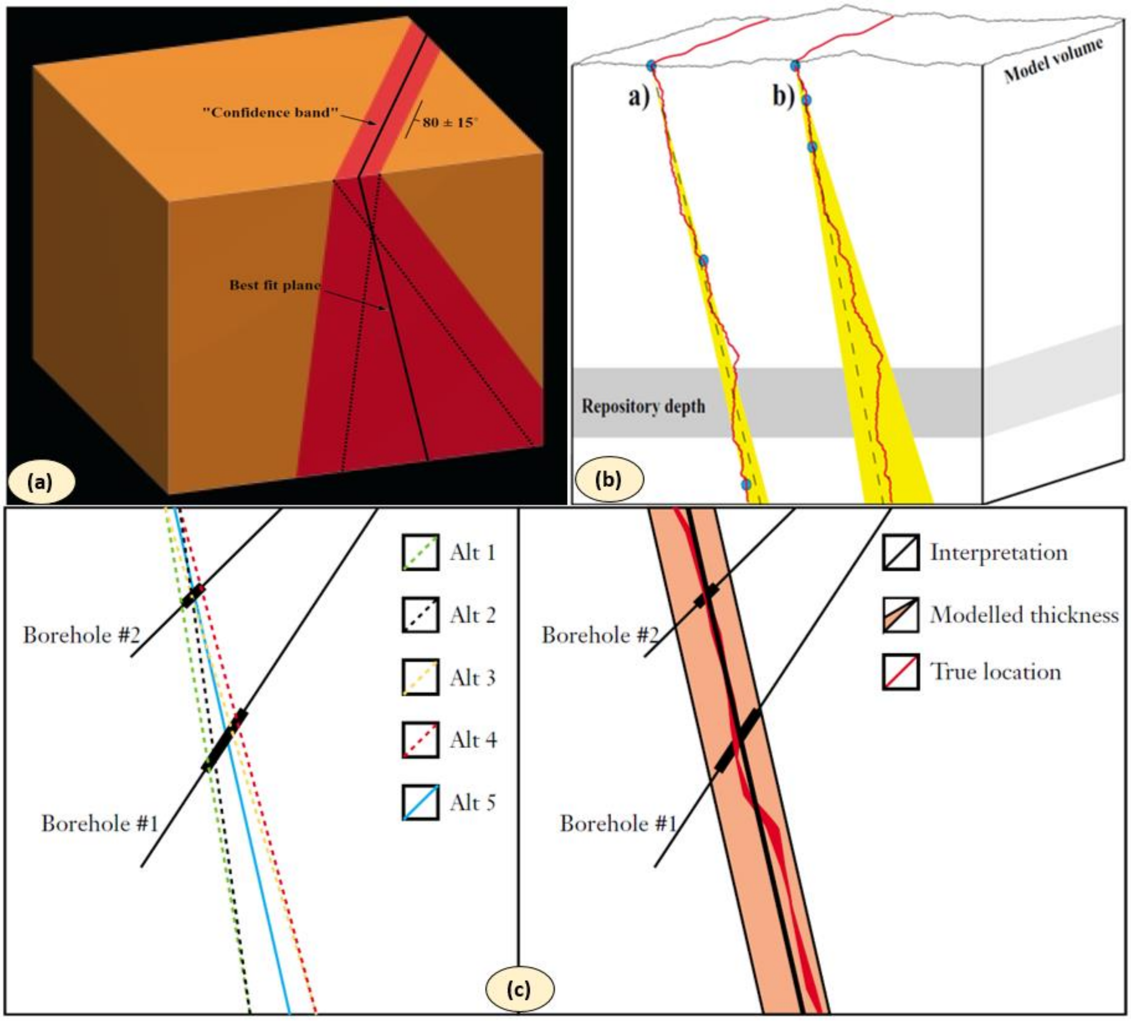

Figure 4.5. The uncertainty and confident on major structures for the description of ground behaviour; (a) uncertainty range in dips, (b) uncertainty in the geometry of formation, (c) uncertainty in thickness (Modified after Munier et al. (2003))

Major geological structures cause construction problems during excavation. After characterisation of rock mass structures, the key parameters, which should be considered for assessing ground behaviour, are:

- Determine the geotechnical domain in the geological formation
- Describe the properties of rock types and minerals

- Identify conditions and locations of weakness zones
- Determine geometry conditions of discontinuity sets and infilling materials
- Interpret and diagnose ground behaviour

Understanding the structures and complexity of rock masses is the primary and most significant step to identify rock mass behaviour in rock underground mining projects. Engineering geological knowledge, experience and engineering judgment are required to achieve a reliable outcome for rock engineering purposes.

4.2.2. Ground loading factor

Strength factor and stress conditions are two significant parameters to distinguish, assess and evaluate the responses of rock mass structures in underground mining. A wide range of ground behaviour can be expected based on loading condition and rock mass composition. Stress-strain behaviour of intact rock and rock mass is presented in Figure 4.6. Typically, the behaviour of rocks in loading and under stress conditions consists of three steps: elastic behaviour, ductile behaviour and failure. Also, post ductile/plastic behaviour of rocks can be described as strain – hardening, perfect plastic, strain – softening, and brittle types.

Loading behaviour in an intact rock depends on rock type, texture and structure, physical characteristics such as density and porosity, loading condition, temperature, confining stress, and saturation. Stress concentration during and after an underground excavation influences the properties of rock masses and may lead to various ground behaviours occurring.

Ground loading factor is the relation between rock mass strength to major principal stress and is calculated as below:

$$\boldsymbol{Loading\ based\ on\ Safety\ Factor\ (LF) = \frac{Rock\ mass\ Strength\ (\sigma_{cm})}{Major\ Principal\ Stress\ (\sigma_1)}} \quad \textbf{(4.1)}$$

The primary behaviours of ground-based on loading factor are defined in equation (4.2).

$$\boldsymbol{Primary\ Ground\ Behaviour = \begin{cases} LF \geq 2, & Stable \\ 1 < LF < 2, & Potential\ Unstable \\ LF \leq 1, & Unstable \end{cases}} \quad \textbf{(4.2)}$$

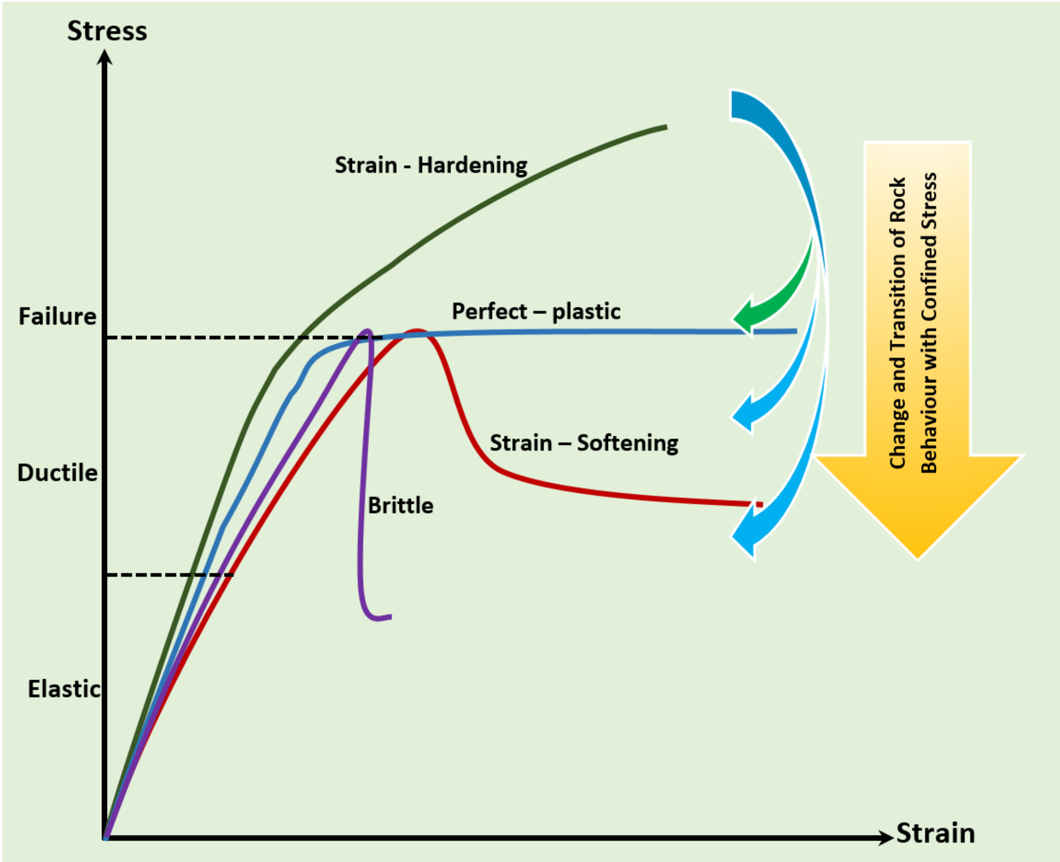

Figure 4.6. Stress-strain behaviour of intact rocks and rock masses

Figure 4.7 to Figure 4.11 show a primary assessment of ground behaviour condition based on loading factor and rock mass composition classes. Massive rocks typically are in stable conditions when loading factor is more than 2. There is a potential problem in ground conditions for LF between 1 and 2, such as fractures in rocks. For LF less than 1, serious problems could be encountered in the ground, such as failure. Ground behaviour in the rock of the jointed/blocky/bedded and blocky/folded classes is influenced by the existence of structures in rocks, and the intensity of problems may increase.

Additionally, the behaviour of disintegrated/crushed/soil like materials is slight to severe over different ranges of loading factors. The composition of rock materials considerably affects the level of problems and challenges in ground behaviour types. Ground behaviour for special material classes is assessed similarly to massive rocks regarding considering only the loading factor.

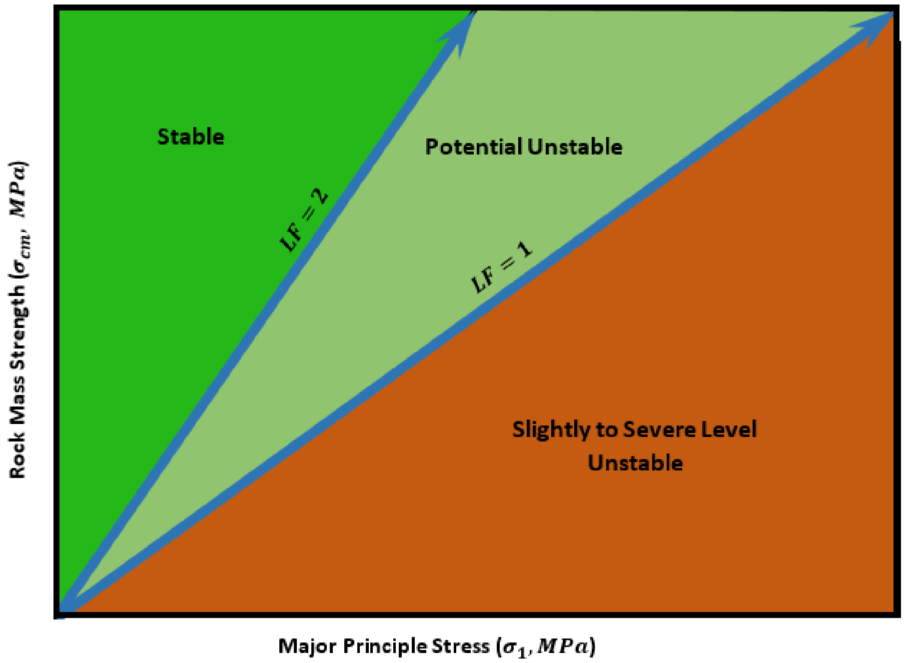

Figure 4.7. The primary assessment of ground condition in massive rocks based on loading factor

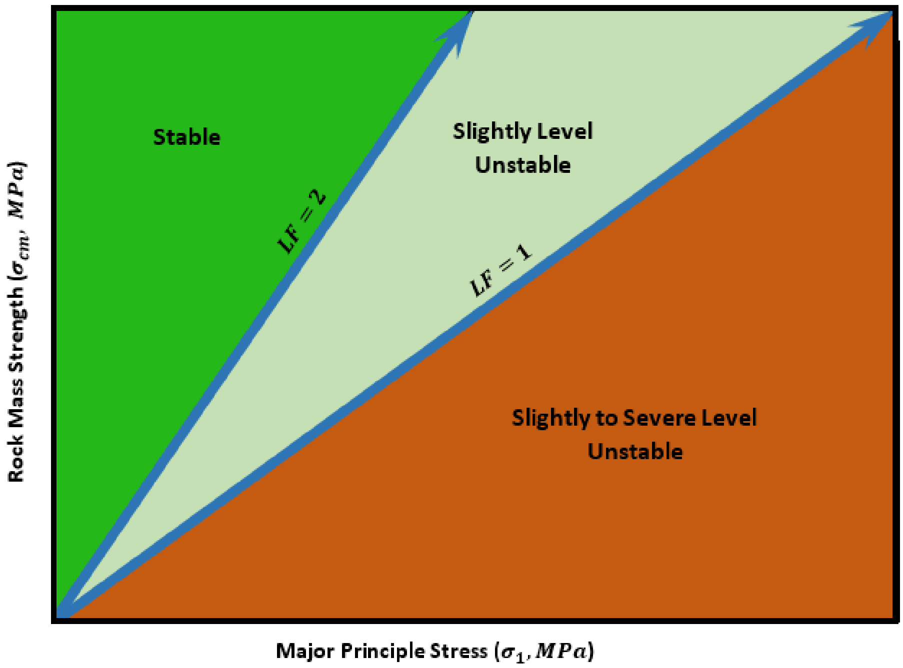

Figure 4.8. The primary assessment of ground condition in jointed/blocky/bedded rocks based on loading factor

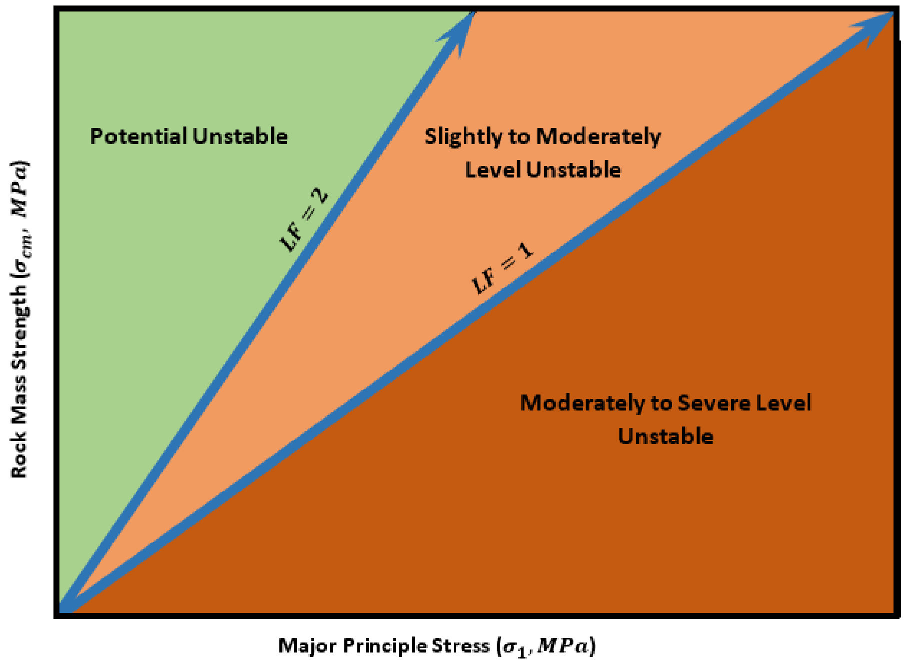

Figure 4.9. The primary assessment of ground condition in blocky/folded rocks based on loading factor

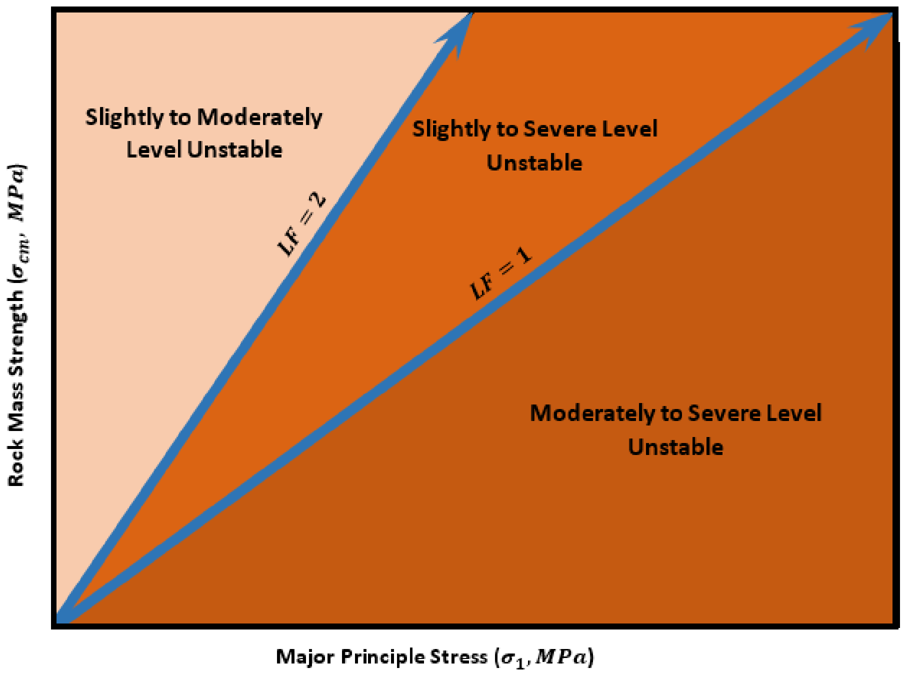

Figure 4.10. The primary assessment of ground condition in disintegrated/crushed/soil like materials rocks based on loading factor

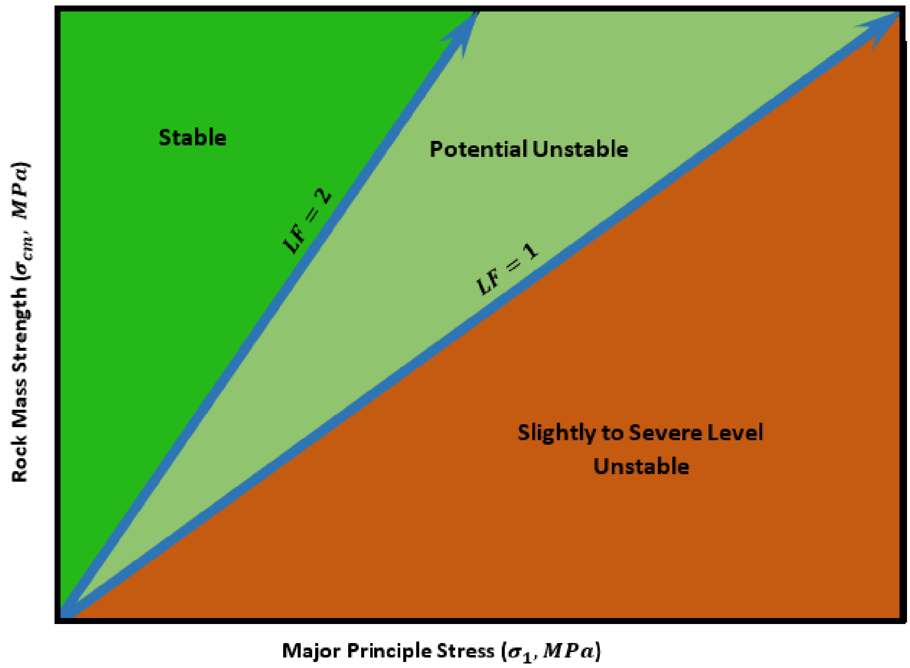

Figure 4.11. The primary assessment of ground condition in special materials based on loading factor

4.2.3. Hydrological condition

Hydrology influences most of the rock engineering works, which are below the groundwater level. Seepage, fluid flow into underground excavations, and effectiveness on rock mass properties are some examples relevant to the hydrological condition. In order to acquire appropriate information related to ground responses in rock engineering practice, consideration of the role of groundwater is required.

Pore fluid pressure refers to the pressure of groundwater within soils or rocks. It may change due to several actions such as groundwater level, earthquakes and induced stress. An underground excavation below the groundwater table will cause water inflow in the surrounding area. Estimation of the flow rate in the fractured rock mass is essential for drainage and stability of engineering structures.

Reliable evaluation of groundwater conditions can provide accurate results for predicting ground behaviour modes and design parameters and construction of underground excavations. Rock material properties, depth, permeability, fracture properties and network, and geological conditions such as faults control groundwater inflow into rock structures. Figure 4.12 presents a primary assessment of ground behaviour affected by the hydrological condition. The first step is determining intact rock properties such as minerals, rock types and porosity. Then, effective factors of discontinuities, for example infilling materials, discontinuity network, and thickness of infilling, are considered and measured. Flow networks of the rocks depending on fracture systems, rock type, bedding planes and tectonic conditions. The usual pattern of conduit systems in rocks is given in Figure 4.13. In step 3, the effect of groundwater condition on physical and chemical properties of minerals and rocks are considered. Also, it is required to estimate permeability and fluid flow rate in the ground. Finally, the potential problems related to hydrology is assessed and diagnosed for rock mass composition classes.

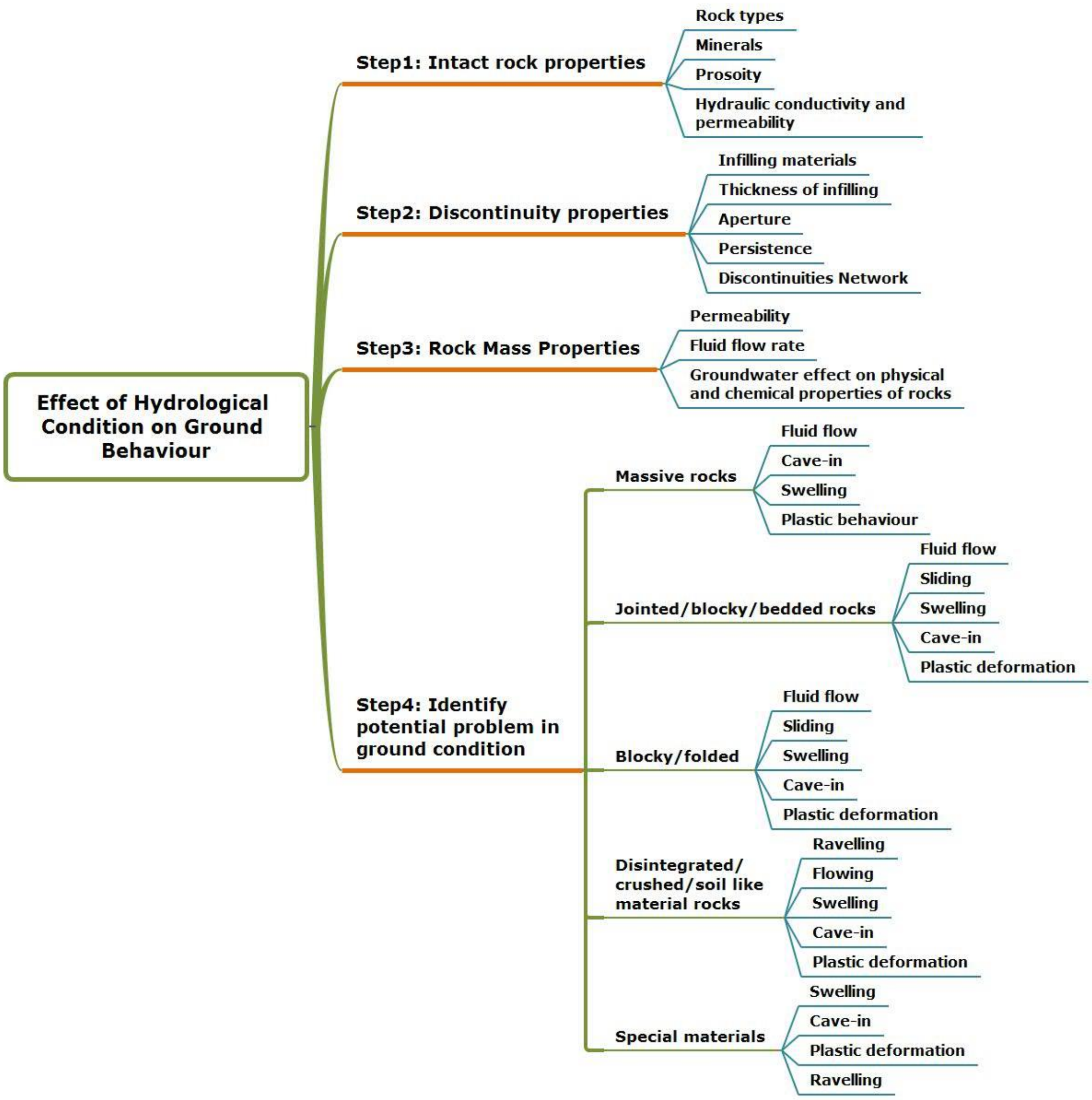

Figure 4.12. The primary assessment of hydrological condition on ground behaviour

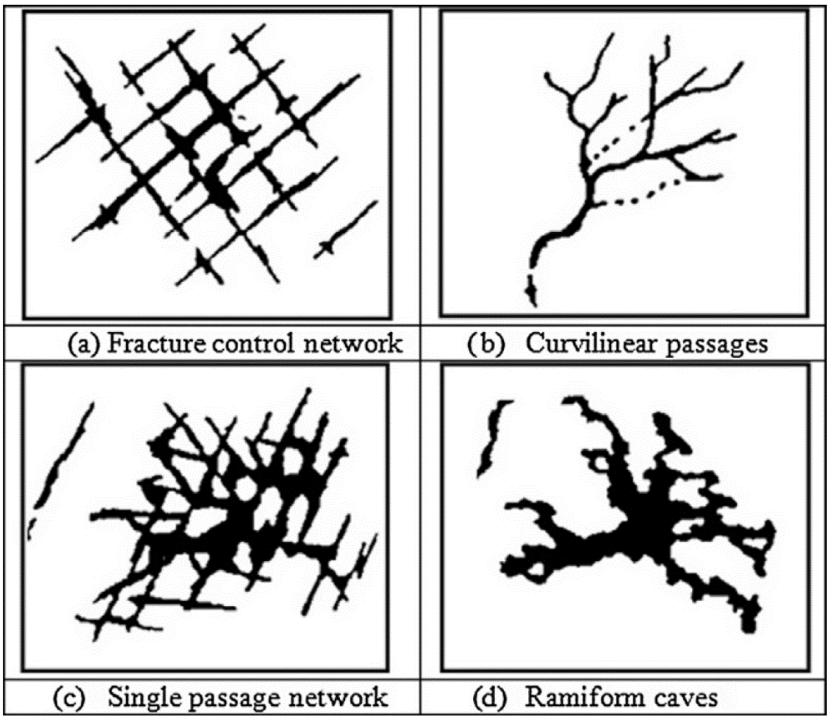

Figure 4.13. Some of the usual conduit systems in rock structures (Zarei et al., 2013)

Palmstrom and Stille (2015) presented a classification of groundwater inflow rate per 10 m tunnel length:

- Dry condition (inflow rate: < 0.1 litres/min)
- Seepage (inflow rate: 0.1 – 1 litres/min)
- Dripping (inflow rate: 1 – 10 litres/min)
- Flowing (inflow rate: 10 – 500 litres/min)
- Heavily flowing (inflow rate: 0.5 – 5 m3/min)
- Water in burst (inflow rate: > 5m3/h)

The following steps describe assessing water inflow in an underground excavation before a construction stage:

1- Collecting data including geological structures, permeability, rock quality and groundwater condition,

2- Determining rock mass permeability by laboratory and field tests, estimating groundwater level by monitoring during the different seasons and dividing the underground opening into a number of sections for assessing water inflow,

3- Evaluating groundwater inflow based on geological information, rock mass structure, permeability and groundwater level,
4- Estimating water inflow in different sections and considering the effect of a high quantity on the excavation.

4.2.4. Static and dynamic loading condition

Ground loading occurs due to natural processes in environments and engineering activities, in the act of static and dynamic loading. Geological formations such as sedimentary deposits produce static loading through geological periods. An example of dynamic loading is earthquake events resulting from a natural process. Figure 4.14 illustrates the ground reaction on loading conditions and the potential ground behaviours. The origins of static loading are gravity, changing groundwater level and gas pressure, and temperature. The response of the ground due to natural processes in environments are gravitational loading, change of groundwater level and gas pressure, temperature, and tectonic activities. The problems in the rock mass structures in these situations could be settlements, subsidence, fracturing, water inrush, displacement and creep. In the case of engineering activities, the typical ground behaviours are sliding, fracturing, flowing, ravelling, ground movement, sliding and displacements.

4.2.5. Key features of underground excavation projects

Construction factors such as excavation method, excavation sequence and advance rate influence rock mass behaviours. The disturbance of a rock mass structure due to the mechanical excavation method is less than due to the blasting method. Blasting mainly creates and develops cracks and joints, and leads to loosening of the rock mass surrounding an excavation. Deformation of the rock mass in a large opening is increased, because of reducing rock mass strength, in comparison to small size openings. Figure 4.15 shows the effect of excavation size, sequence and shape on rock mass structures. The rock mass can behave as an intact or blocky or high degree jointed/blocky and folded rock depending on the dimension of the excavation. Excavation sequences influence the rate of disturbance of rock surrounding an excavation and have an impact on deformation and induced stress. Meanwhile, excavation shapes with high curvature are more stable than rectangular shapes.

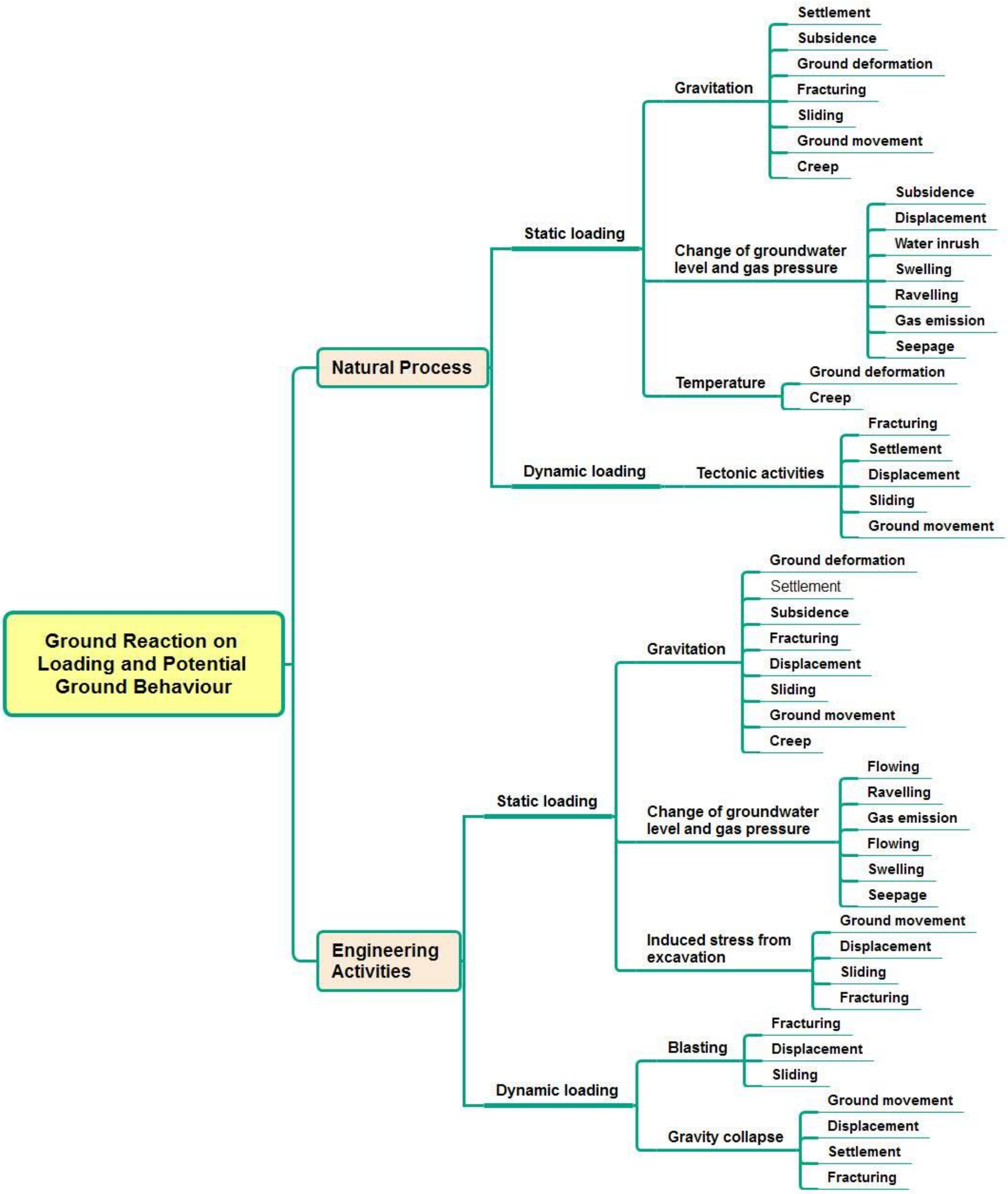

Figure 4.14. The primary assessment of ground behaviour based on loading condition and the related potential problems

The relation between the degree of jointed rock mass and size of excavation is defined by the ground continuity factor, which is used for considering the behaviour of the rock mass surrounding an opening (Palmstrom and Stille, 2010). Table 4.1 displays the continuity and discontinuity ground condition for the prediction of block size and volume in rock engineering works. Block size and volume can be used to determine sliding blocks and wedge failure in the rock surrounding an opening.

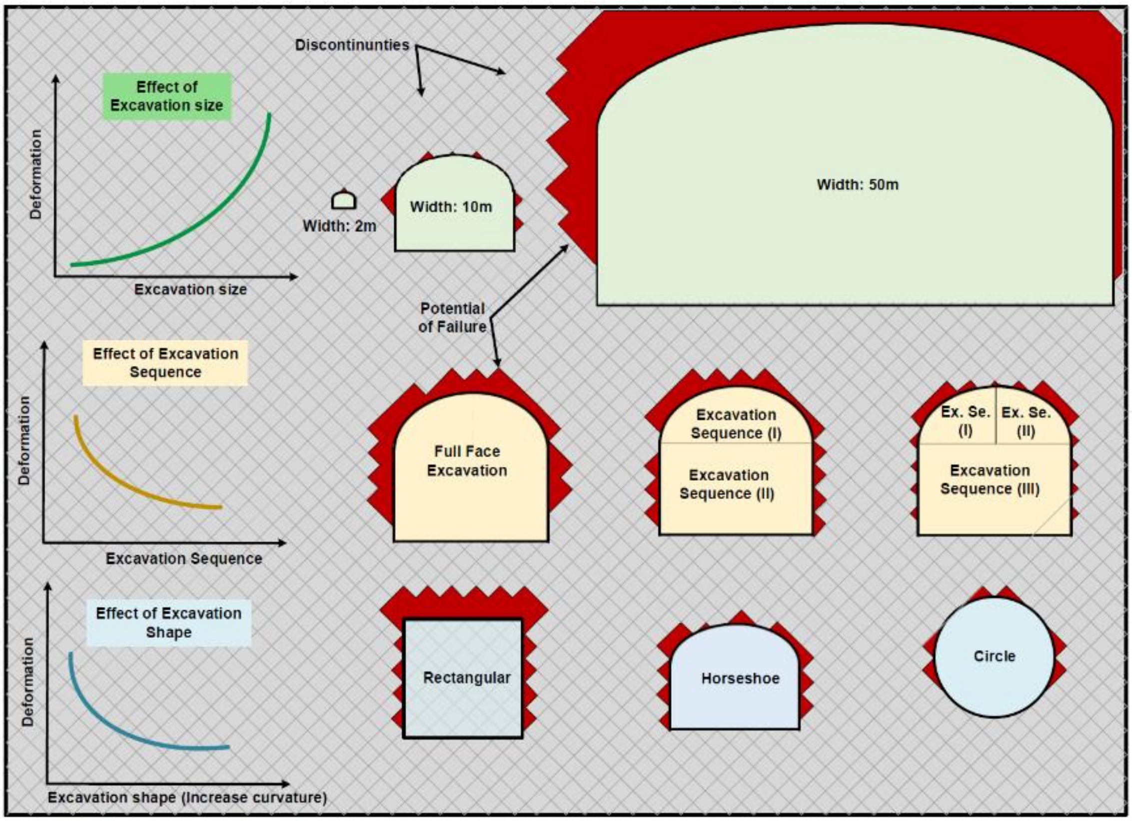

Figure 4.15. The relation between excavation size, sequence and shape based on the rock mass behaviour

Table 4.1. Estimation of block size and volume based on ground continuity factor (Palmstrom and Stille, 2015)

Ground continuity (CF)[a]	CF = D_t/D_b	Variation in block volume: V_b		
		$D_t = 2$ m	$D_t = 10$ m	$D_t = 50$ m
Continuous (bulky)	>50*	<0.064 dm^3	<0.008 m^3	<1 m^3
Discontinuous-continuous (blocky-bulky)	20–50	0.064–1 dm^3	0.008–0.13 m^3	1–16 m^3
Discontinuous (blocky)	6–20	0.001–0.04 m^3 (1–40 dm^3)	0.13–5 m^3	16–600 m^3
Continuous-discontinuous (intact-blocky)	3–6	0.04–0.3 m^3	5–40 m^3	600–5000 m^3
Continuous (intact)	<3	>0.3 m^3	>40 m^3	>5000 m^3

D_t = tunnel span, D_b = block diameter, V_b = block volume
[a] The boundary between continuous and discontinuous ground may be a matter of discussion

Discontinuity properties, joint sets, thickness and location of weakness zones provide useful information to evaluate blocky behaviour of rock and an unstable condition. Presence of moisture or groundwater may change the rock mass and discontinuity properties. It is an important issue, especially for rock consisting of minerals with special properties. Groundwater causes a reduction in rock mass strength and friction of discontinuities. Hence, when an excavation is encountered with disintegrated or crushed rock mass, the presence of groundwater along discontinuities may lead to flowing ground. At great depth, the rock mass is commonly less jointed than at shallow depth. Therefore, the presence of groundwater and joint water pressure is significant for evaluation in order to diminish the risk of rock failure.

The time delay before installing supports may affect the loading on rock structures and even change the ground behaviour. Stand-up time of a rock mass can be used for the time limit to install a ground support system before problems occurring. Precise observation and careful interpretation are critical points for the determination of the actual ground behaviour mode before and during construction.

Rock mass composition, ground loading factor, hydrology, and key features of underground excavations such as geometry and orientation are essential factors to diagnose ground behaviour modes. Domain/zone in rock masses is selected based on lithology and discontinuity conditions. The geotechnical information and potential responses of the ground for each domain are recorded such as potential deformation in rocks. Ground loading factor is employed to determine a primary assessment of stability in the ground. Also, groundwater inflow and the effect of mechanical and chemical properties of minerals should be assessed. Meanwhile, excavation methods, geometry and orientation of openings consider based on the rock mass structures surrounding underground excavations. All this information are collected and used as input data for the description of rock mass behaviours.

4.3. Data processing for diagnosis of ground behaviour

Possible ground behaviour can be identified from a site investigation, rock mass structures, discontinuity conditions, groundwater and the induced stress condition. Figure 4.16 illustrates a procedure of data processing for the diagnosis of ground behaviour in a rock underground excavation. Diagnosis of rock mass behaviour is associated with precise observation, careful interpretation of available evidence in the environment, engineering judgment and experience. Collecting data from site investigations and the result of laboratory/field tests are used to diagnose ground behaviour modes before the excavation stages. Precise and accurate observation and also the interpretation of available evidence of the ground condition is a preliminary prediction of the ground behaviour in underground excavations (Rahimi and Sharifzadeh, 2017). Knowledge, experience, and engineering judgement are the main principles during the processing of input data to foresee the behaviour in the ground condition.

Ground behaviour, on the one hand, is the result of rock mass composition classes, which vary from massive rock to heavily jointed and soil like, and on the other hand, depends on the underground excavation condition and its consequent stress changes. Diagnosis of ground behaviours has prime importance in underground design and defined in the following expression (Sharifzadeh et al., 2017):

Diagnosis of ground behaviour (DGB) = Rock mass composition (RMC) + Underground excavation Condition (UEC) + Environmental Condition (EC) (4.3)

Rock mass composition has a significant impact on various ground behaviours. Considering minerals with special properties, the degree of the jointed rock mass and orientation of discontinuities relative to excavation alignment are necessary for predicting ground behaviour. The intersecting joint sets and created rock blocks may cause sliding blocks or wedge failure.

Hydrogeology on its own is an essential and broad subject. It influences most of the rock engineering works, which are below the groundwater level. Seepage, fluid flow into an underground excavation, and effects on rock mass properties are some examples relevant to hydrological conditions. In order to achieve acceptable outcomes of ground responses in rock engineering practices, the reaction of the ground condition to groundwater and its role should be considered (Price, 2009). The deformation of a rock mass increases with increasing underground opening size. This is because rock mass strength reduces on a large scale compared with a small scale under the same condition. Concerning excavation methods, mechanical methods cause less disturbance compared to blasting methods.

Ground loading factor is used to identify the different types of ground behaviour. Rock mass structures behave unstably under loading factors less than one, while the ground condition is commonly stable under a loading factor more than two.

Ground responses in engineering projects are associated with natural processes and engineering activities. Identification of rock mass behaviour is accomplished by determination of the geometry of excavation, continuity factor, excavation method, excavation sequence, main types of loading, and orientation of the underground excavation.

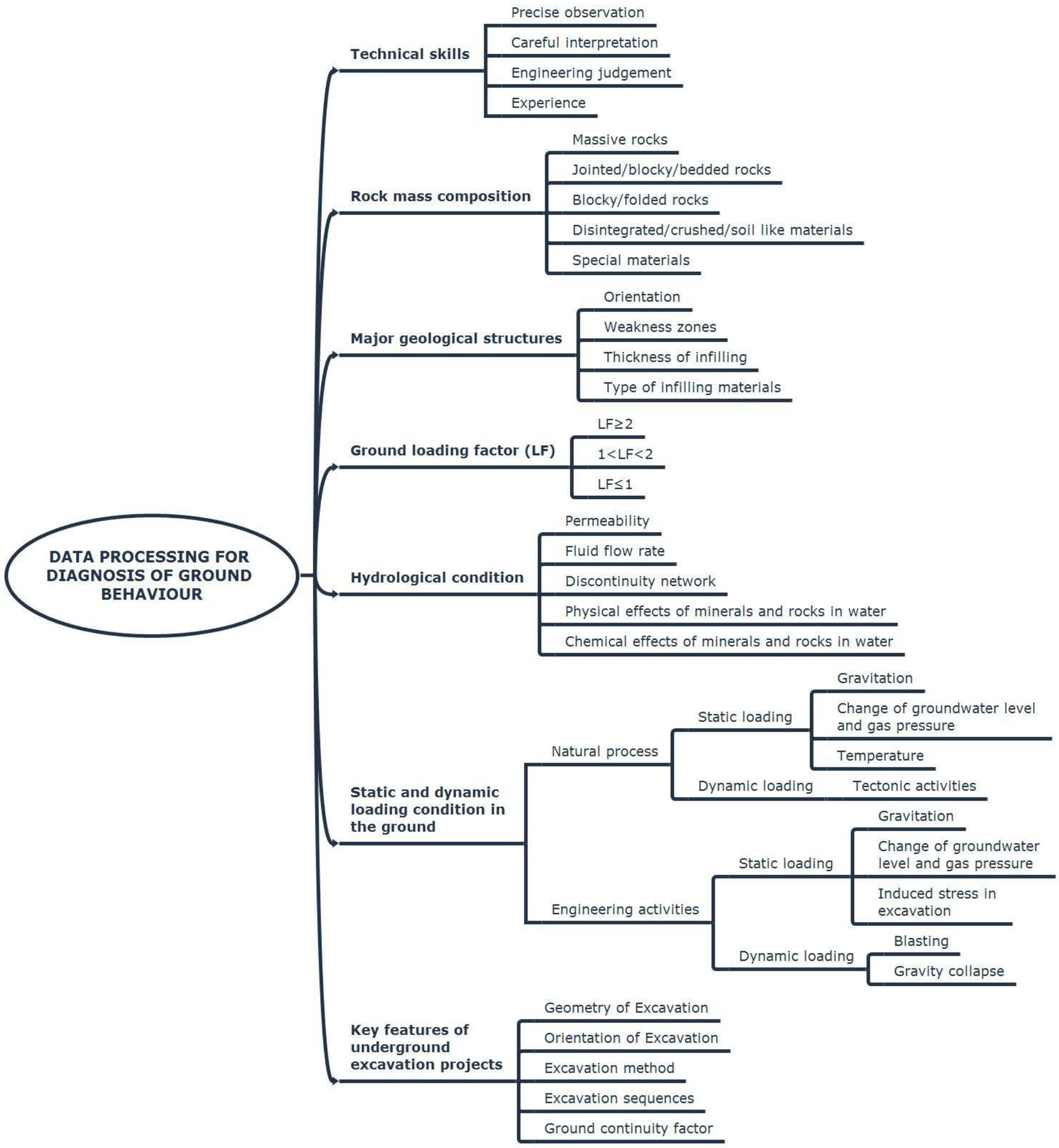

Figure 4.16. Data processing for diagnosis of ground behaviour in rock underground mining projects

4.4. Ground behaviours in deep underground mining

Rock masses at great depth have a complex structure and describing ground behaviour requires a fundamental level of knowledge, experience and engineering judgement. Weakness zones such as faults and shear zones have a major influence on ground behaviour in deep underground excavations. At great depth, when the ground reaction is not predicted or distinguished, rock mass may behave in unforeseen ways, and sometimes the condition of good ground decreases in quality due to a variety of factors such as blasting quality. Meanwhile, misunderstood or incorrect prediction of ground behaviour may lead to using improper numerical modelling and consequently assessing implausible stability conditions for rock underground structures.

Figure 4.17 presents a classification of ground behaviour modes in deep underground excavations. Considering rock structure type, excavation condition and stress concentrations, rock behaviour around excavations could be classified into stable, massive rock failure, intact/structural, structural failure, and water effect. In a real ground condition at depth, rock engineers commonly encounter more than one type of rock behaviour. Therefore, evaluation of several/combination of ground behaviours in one project is essential to favourable control of rock failure hazards. These classes are described in the following sections.

4.4.1. Stable

Rock mass behaviour and its related behaviour is stable when the strength is enough. A rock mass structure surrounding an underground excavation is stable without using supports for some days or even longer. Rock masses may have a few discontinuities or joints. Figure 4.18 presents stable behaviour of ground in different rock mass compositions and accordance with ground loading factor, continuity factor and static/dynamic loading condition. The figure indicates that a massive rock at depth is stable with continuity factor less than three and loading factor more than two. Likewise, jointed/blocky/bedded rock is stable with continuity factor less than three and loading factor more than two. However, the span of the underground opening should be smaller than in the massive rock. The same condition prevails for blocky/folded rocks. In stable conditions, the dominant loading is static. The origin of static loading could be gravity and induced stress in the rock mass formation.

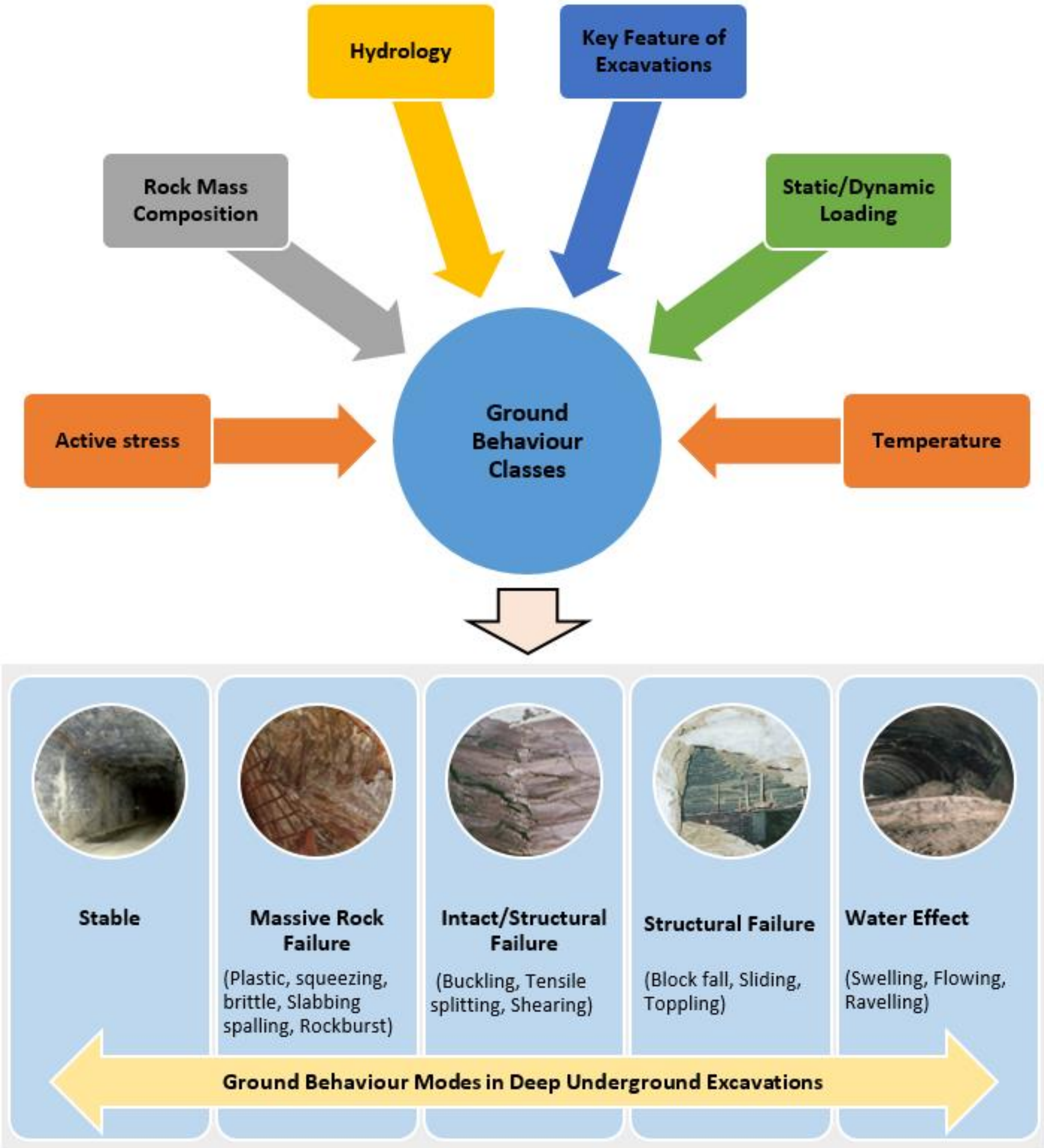

Figure 4.17. Ground behaviour modes in deep underground excavations

4.4.2. Massive rock failure

Massive rock failure is mostly associated with a rock mass consisting of a few joints so the behaviour of intact rocks is mostly related to the properties of intact rock and discontinuities. Figure 4.19 shows the area of massive rock failure in different types of rock mass compositions.

Based on the construction condition and geological structures, massive rock failure mostly occurs in the continuous and discontinuous regions with continuity factor less than six. In this state, the ground loading factor is usually less than two. A range of static and dynamic loading conditions influence ground conditions. For a static condition, induced stresses have a considerable impact on ground behaviour and may cause brittle, plastic behaviour and squeezing in underground mining. The most effective dynamic loading is usually caused by seismic events and blasting damage in rocks. The behaviour of massive rock failure in dynamic loading leads to slabbing and rockburst failure.

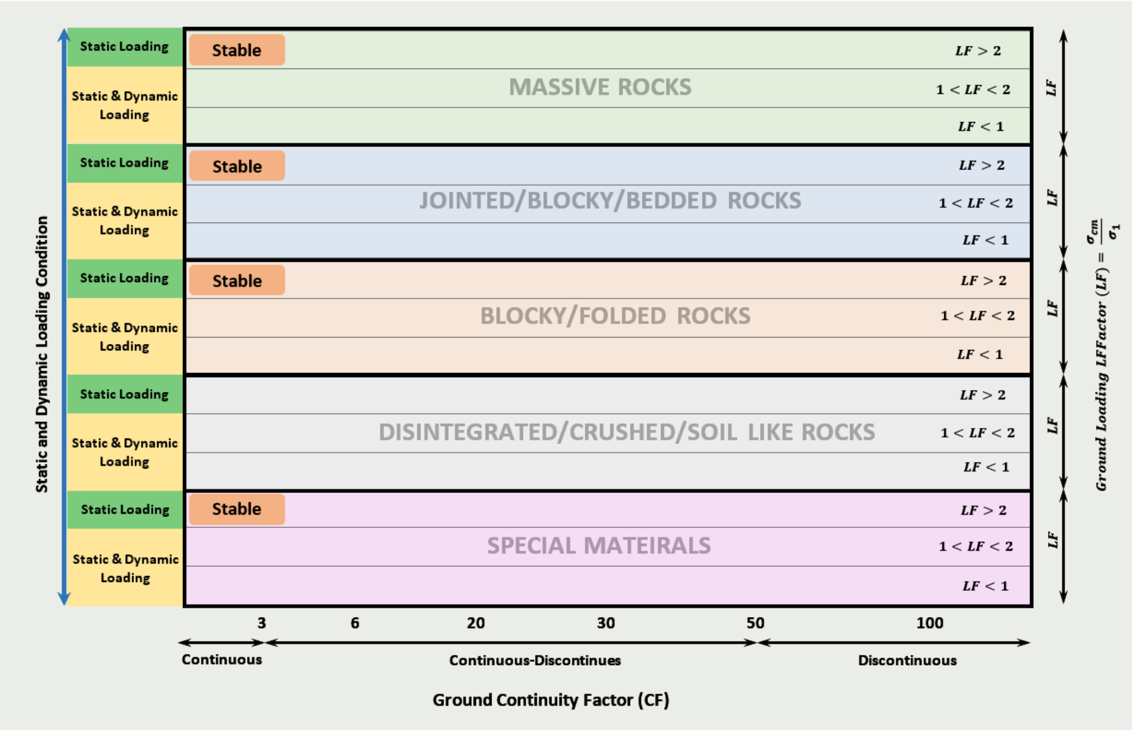

Figure 4.18. Primary assessment and diagnosis of stable behaviour in different types of rock mass compositions in deep underground excavations

4.4.3. Intact/structural failure

The moderately jointed rock contains some discontinuity sets and different geometric dimensions of rock blocks. Interactions between discontinuities and intact rocks during construction associated with static/dynamic loading conditions lead to the occurrence of a combination of intact failure and structural failure in underground mining and are therefore called intact/structural failure. Figure 4.20 shows the extent of intact/structural failure based on continuity factor and loading condition in the ground. Rock mass structures surrounding underground openings usually behave as continuous-discontinuous ground. Intersecting discontinuities in the roof and wall cause block falls and sliding under low-medium stresses, especially in blocky and folded rocks. Under high-stress levels with loading factor less than two, shearing, splitting and buckling may occur in underground mines.

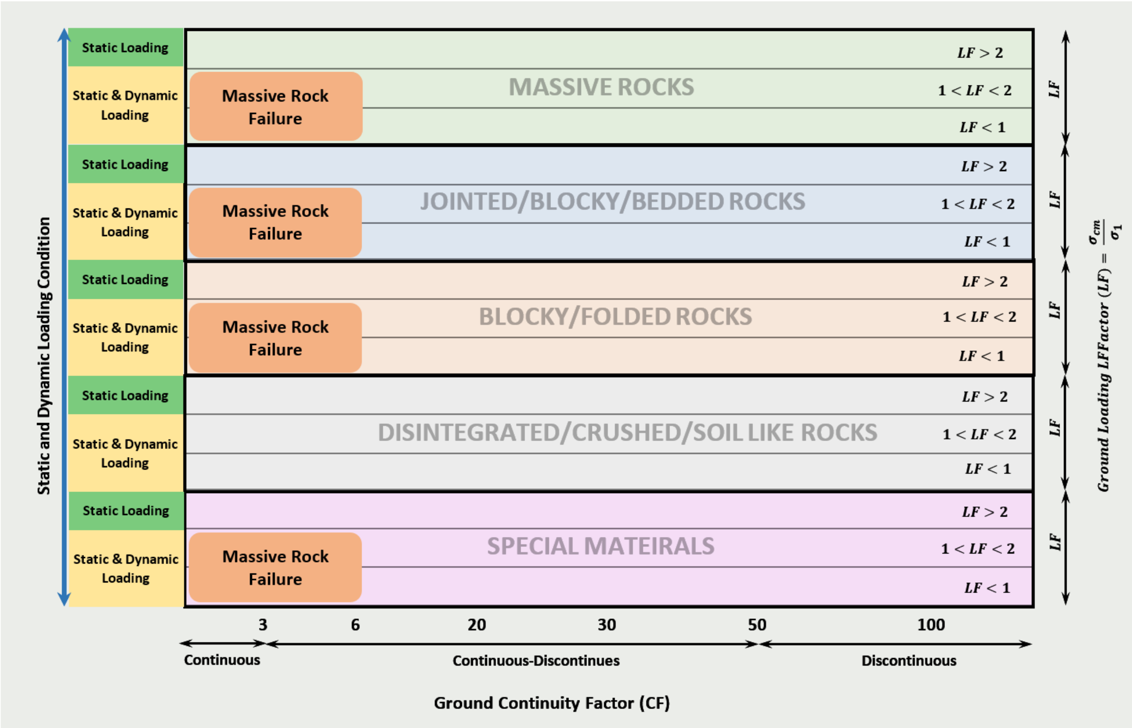

Figure 4.19. Diagnosis of massive rock failure in rock mass classes in deep underground excavations

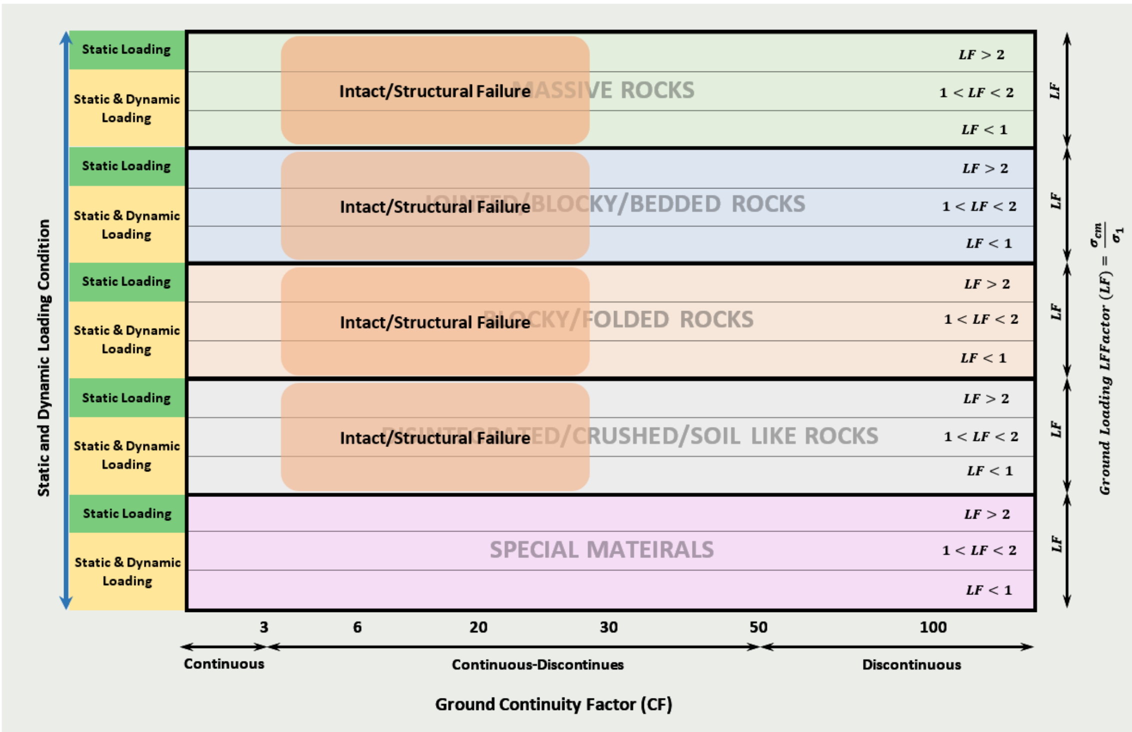

Figure 4.20. Diagnosis of intact/structural failure in different rock mass compositions in deep underground excavations

4.4.4. Structural failure

The behaviour of structural failure is related to high degree jointed rocks or high ground continuity factor. Structural failure happens in large scale underground mines with moderately–highly jointed rocks. Figure 4.21 shows diagnoses of structural failure in rock mass classes relying on loading factor, ground continuity factor, and static/dynamic loading condition. The composition of rocks and tectonic structures is effective in this type of ground behaviour. In low-stress conditions, the ground problem can appear as ground fall, toppling, sliding, chimney failure and plastic deformation. Ground response under high-stress levels is large deformation and squeezing behaviour. The severity of structural failure is affected by static and dynamic loading. For example, ground fall and wedge failure may occur in static loading conditions under low-stress level, and the squeezing and ravelling may occur in high-stress conditions and with seismic events.

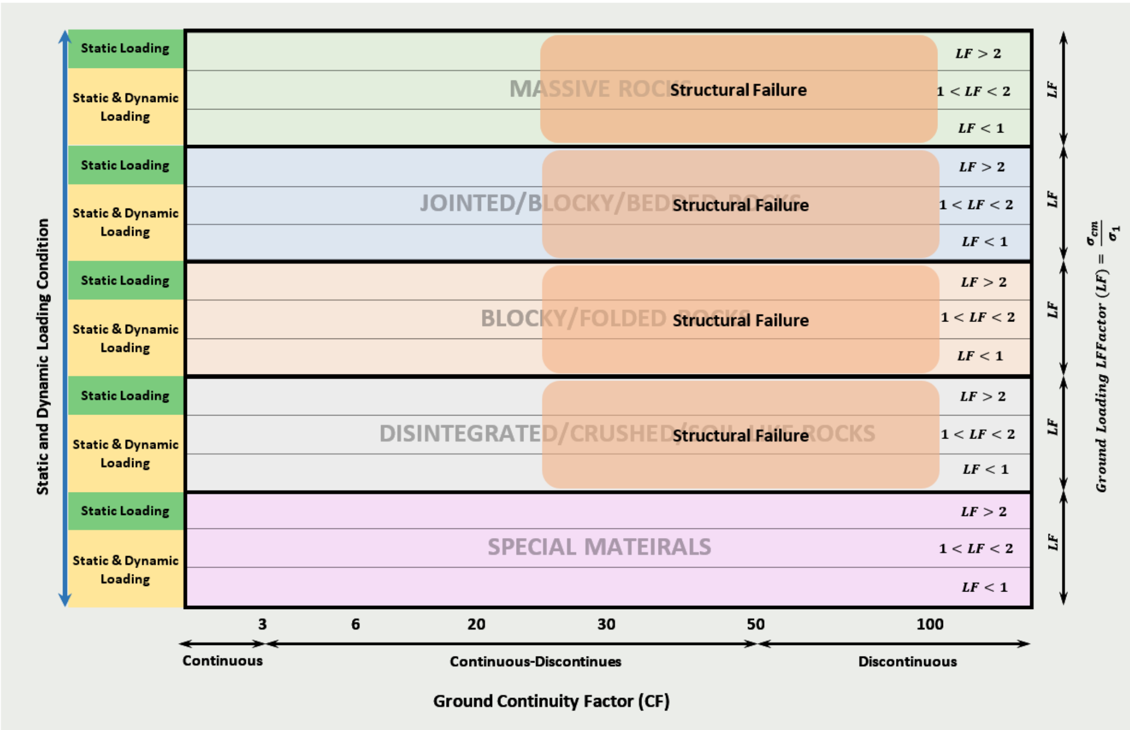

Figure 4.21. Diagnosis of structural failure in different rock mass compositions in deep underground excavations

4.4.5. Water effect

Groundwater condition may have an impact on the mechanical, chemical, hydraulic and physical properties of rock materials. The water effect is usually encountered with special minerals in rocks such as clay and soluble minerals. Figure 4.22 shows the area of water effect behaviour in rock mass structures. The behaviour is related to changing properties of the rock materials and excessive water pressure. The typical problems derived from water effect are flowing, swelling, ravelling, plastic behaviour and squeezing.

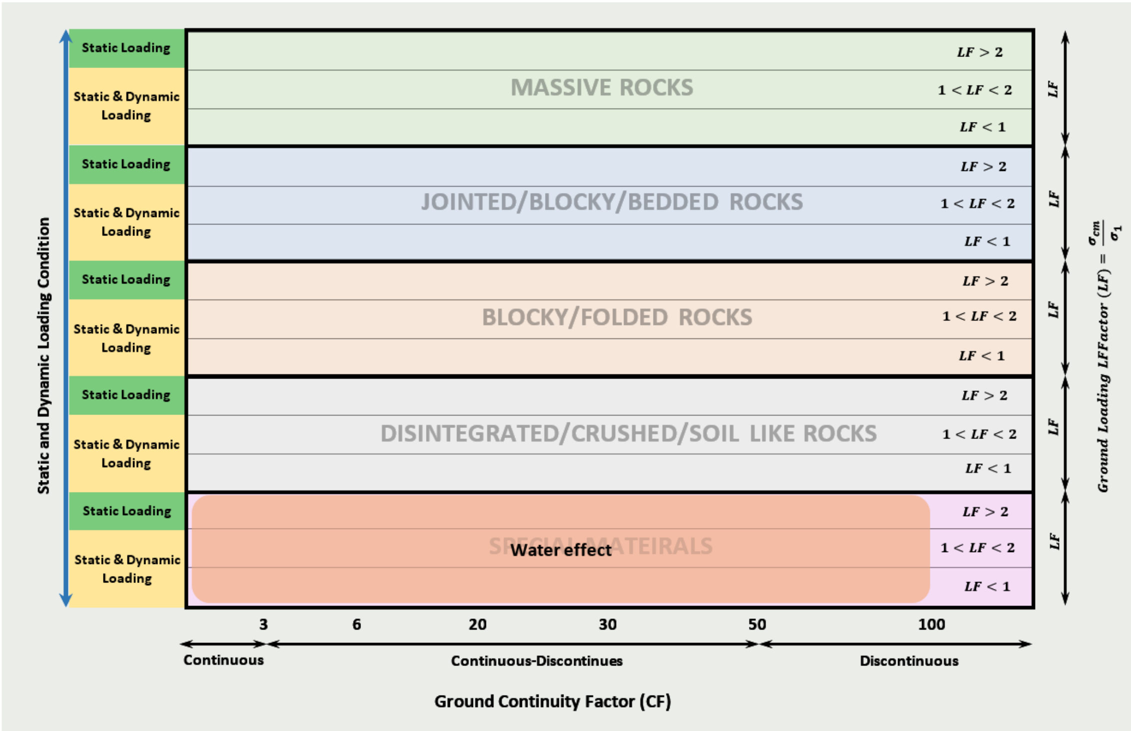

Figure 4.22. Diagnosis of water effect behaviour in rock mass structures

The swelling ground is defined as time-dependent ground behaviour which leads to increased volume of ground due to water absorption, stress state or a combination of these factors. Rock materials containing clay minerals in contact with water causing swelling phenomena.

Sharifzadeh et al. (2002) presented a classification of water effects on rocks' behaviour in four classes: hard and competent rocks (HR), medium and structurally weak rocks (MR), sensitive rocks (SR) and evaporite rocks (ER), as shown in Figure 4.23. The characteristics of hard rocks usually involve high strength, low porosity and compact texture. The water may cause strength reduction of these types of rocks due to pore pressure based on the water saturation level. The medium and structurally weak rocks group is described as medium strength, small to coarse grain size, internal weak structures, the existence of interfaces between weak structures and sometimes minerals or crystals, alteration, and medium to high porosity. Absorbing water may result in a decrease of cohesion and strength and the deformability modulus.

Water content makes a difference between the HR and MR classes in the classification. Sensitive rocks are associated with rocks with special minerals, like clay minerals, which are accompanied by water absorption of minerals in a physiochemical process, and swelling, slaking and squeezing behaviour. Evaporite rocks like salt and gypsum absorb water, which weakens the chemical bonds between molecules and minerals. This group of rocks is associated with time-dependent behaviour modes. Evaluation of the groundwater condition on rock mass structures is associated with identifying water-related behaviours in order to decrease risks and increase safety.

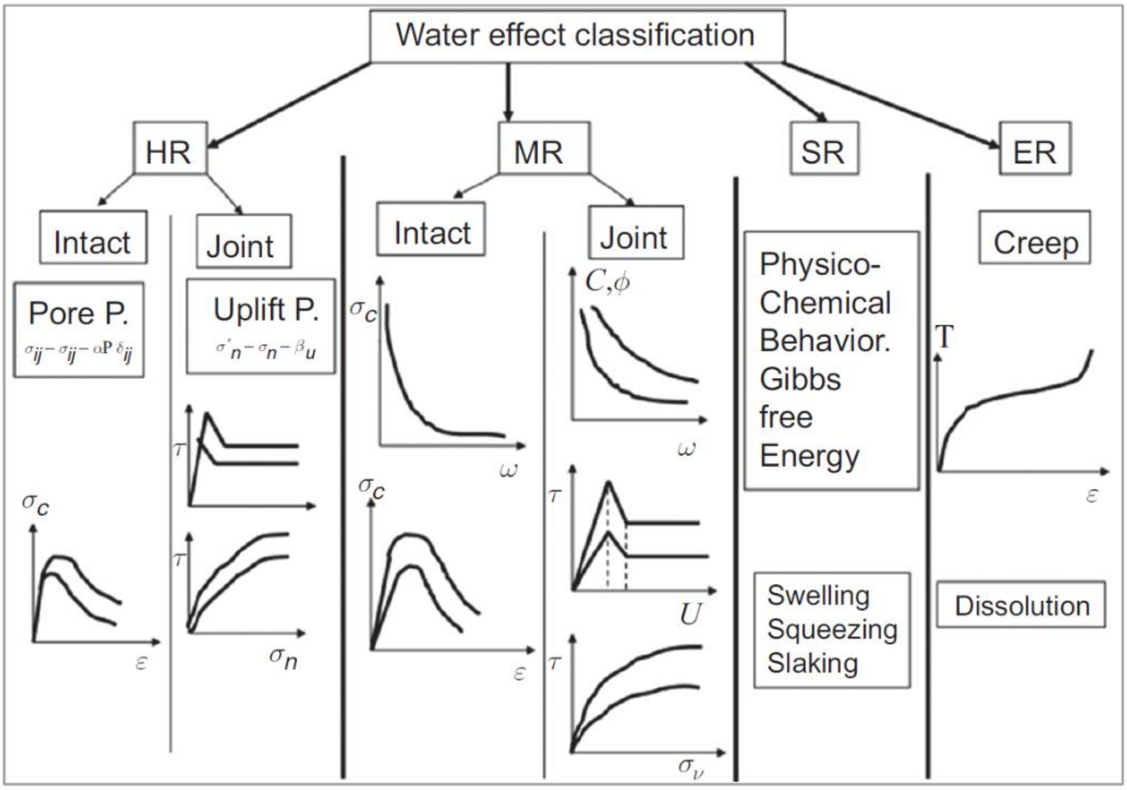

Figure 4.23. The classification of water effects on rock behaviour (HR: hard and competent rocks, MR: medium and structurally weak rocks, SR: sensitive rocks, ER: evaporate rocks) (Sharifzadeh et al., 2002)

4.5. Case studies of diagnosis of ground behaviours

Diagnosis of ground behaviour is taken into account in several case studies in deep underground excavations in the following section. Practical factors, data processing and output, are applied to identifying main types of ground behaviours.

4.5.1. Case example A

This case example studies the diagnosis of ground behaviour in case example A in Chapter 3. Rock mass composition for the mine site was classified as jointed/blocky/bedded. Ground behaviour condition is considered in the main access decline in the underground mine.

The major geological structures are fault zones and joint sets. The thickness of fault zones is less than one metre near several metres of chlorite alterations. Site investigation methods found a foliation rock in the fault zones. The orientation of discontinuity sets and decline was between 25 and 60 degrees, and the range of discontinuities' dip was recorded between 30 and 70 degrees. Typically, orientation between discontinuities was in a fair condition. Figure 4.24 and Figure 4.25 show geological structures and geometry of rock blocks at the mine site. Block volume was estimated to be between 0.5 m^3 and 3.5 m^3.

The decline as main access for the mine is nominally 5.5 m wide and 5.7 m high, and the excavation is by the drilling and blasting method. Figure 4.26 shows a view of the decline and installed support system and the ground problems encountered.

Intact rock strength varies from 50 MPa to 100 MPa. In situ stress was estimated between 14 MPa and 30 MPa. Hence, the ground loading factor varies from 0.7 to 1.5. The hydrological condition causes an alteration in discontinuity surfaces and faults and also leads to cavities in soluble rocks in weathering profiles.

Figure 4.27 presents the detailed process for diagnosis of ground behaviour at the mine site. Ground behaviour conditions at the decline of mine A are classified as massive rock failure, intact/structural failure, water effect.

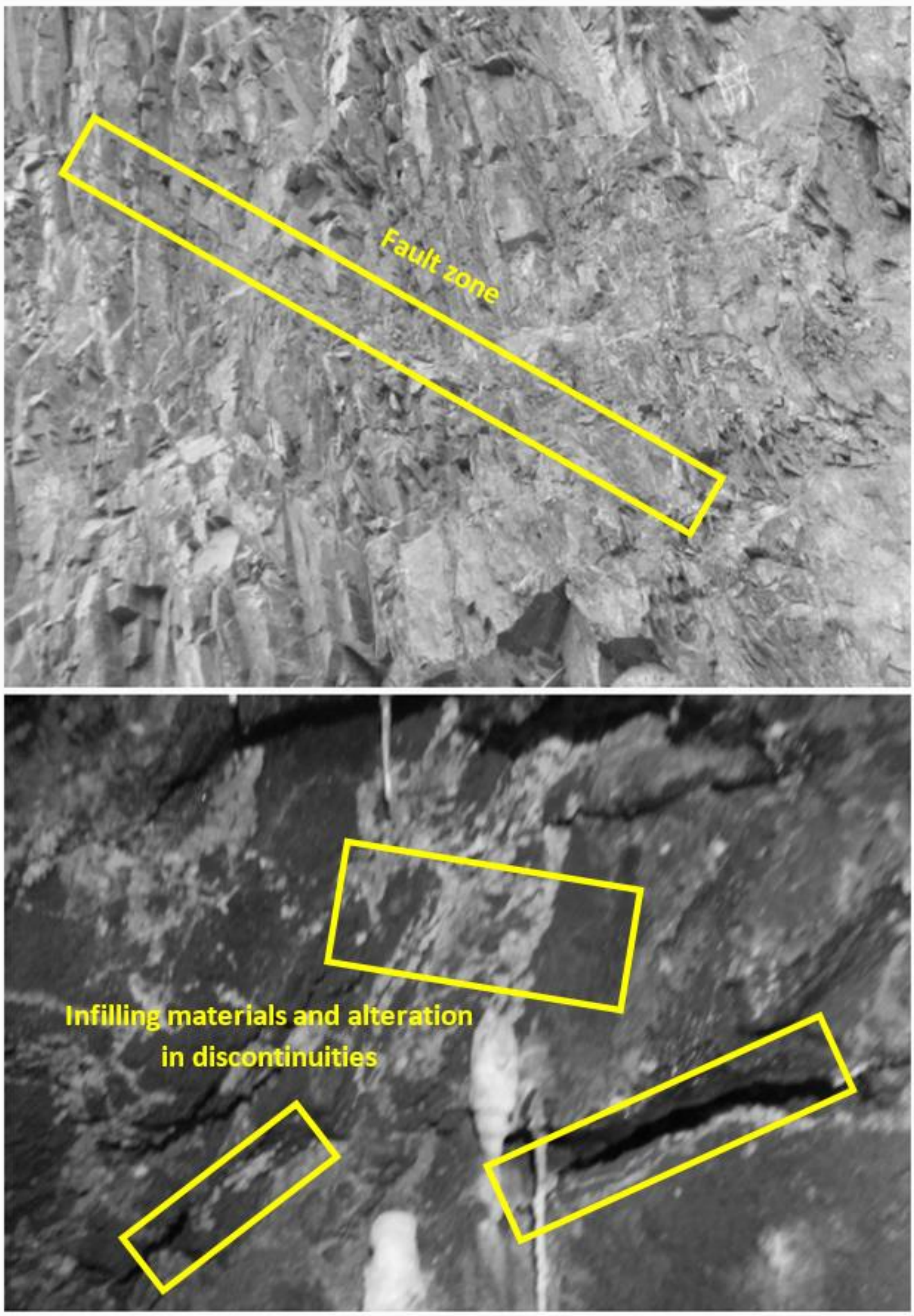

Figure 4.24. Major geological structures and discontinuity condition at the mine site

Figure 4.25. Major discontinuity sets and geometry of discontinuities

Figure 4.26. Ground problems in the decline and installed support devices

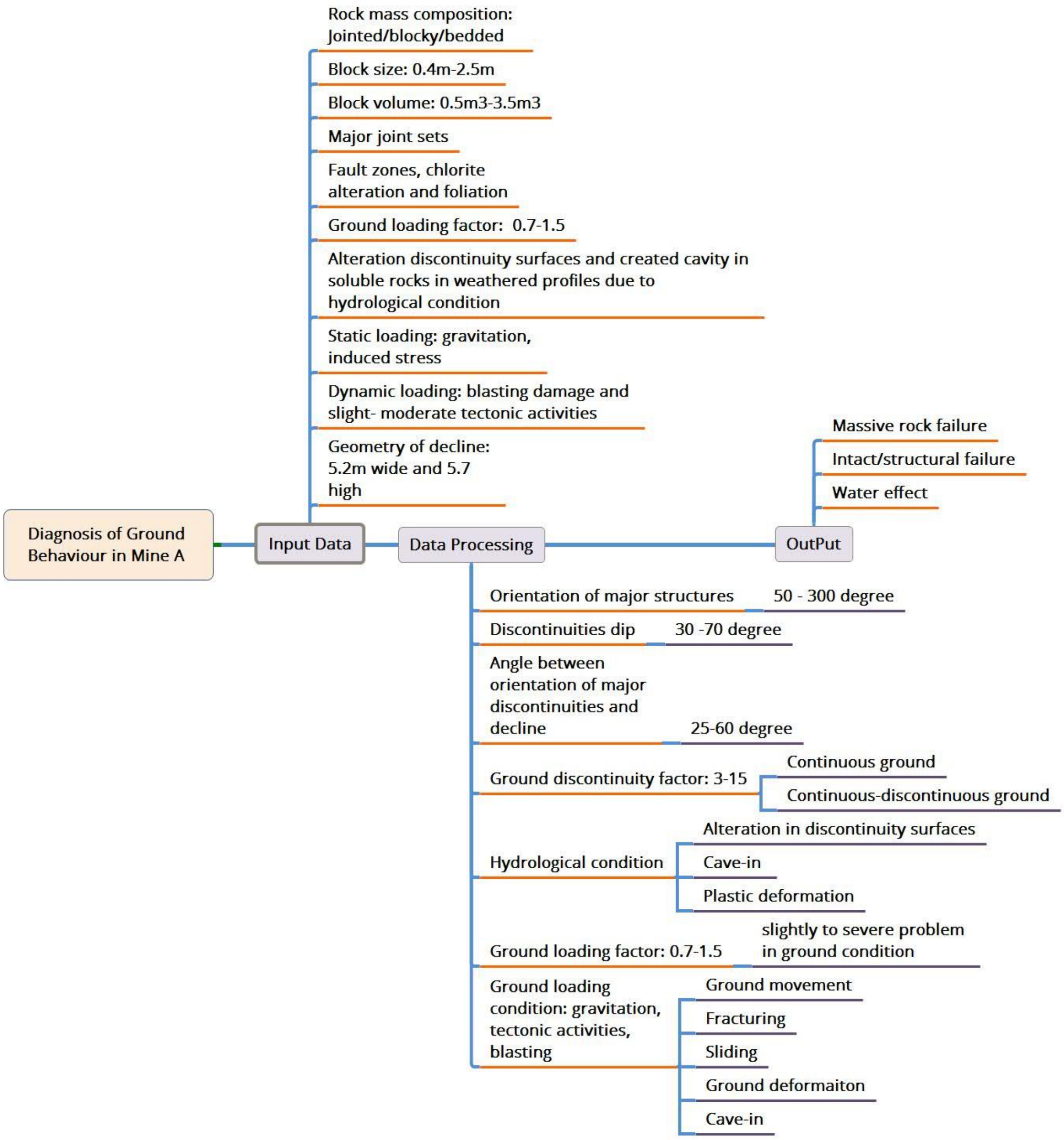

Figure 4.27. Diagnosis of ground behaviour in the decline of mine A

4.5.2. Case example B

The case example B from Chapter 3 is considered here, and the states of ground behaviour modes are assessed. The effective geological structures on ground behaviours are weakness zones, joint sets, faults and veins. According to collected data from site observations, veins and shear fractures were observed mostly in weakness zones. The infilling materials contain chlorite, quartz, clay and sericite. The alteration and weathering in the rocks and discontinuities were slight to moderate. Shear zones with high dip and containing silica were found near the fault zone. Figure 4.28 and Figure 4.29

show geological structures in the rock.

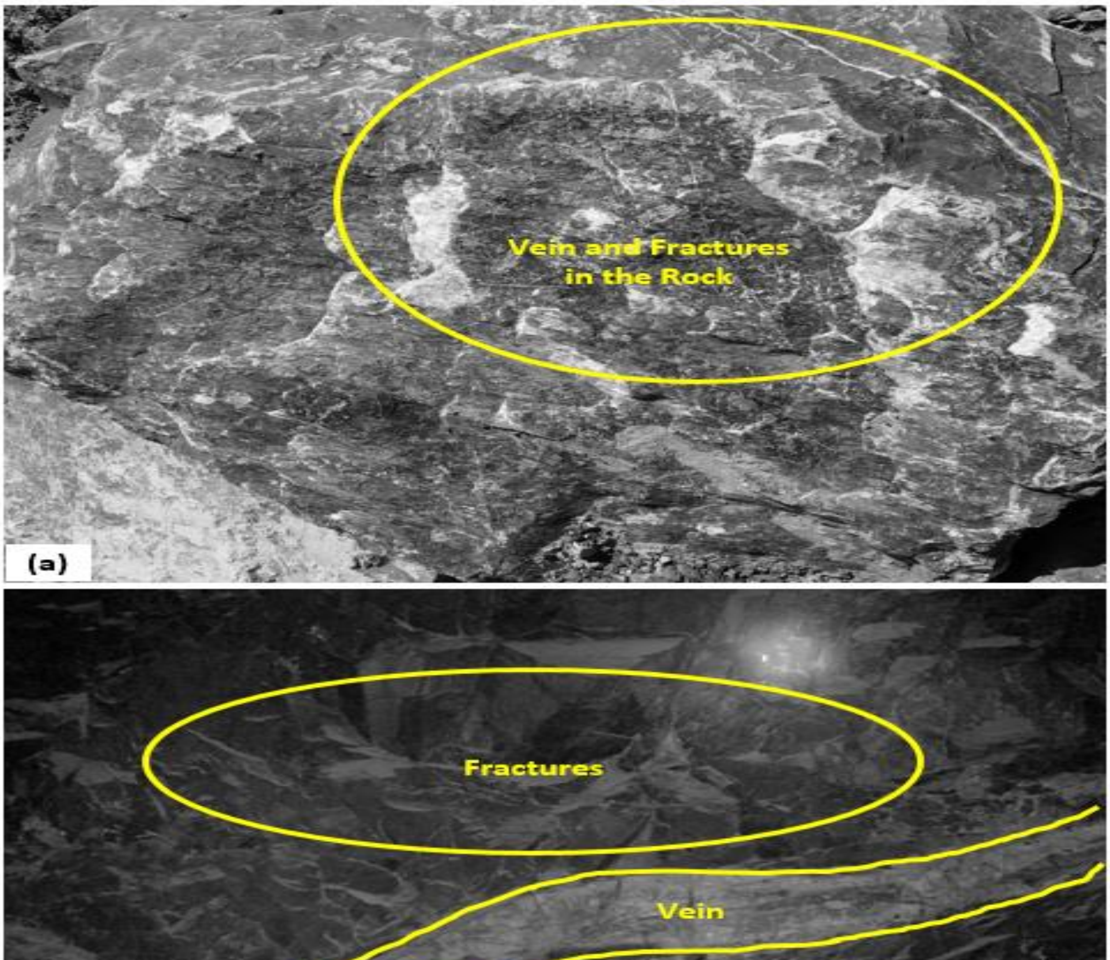

Figure 4.28. Some rock features at the mine site: (a) and (b) vein and fractures, (c) alteration

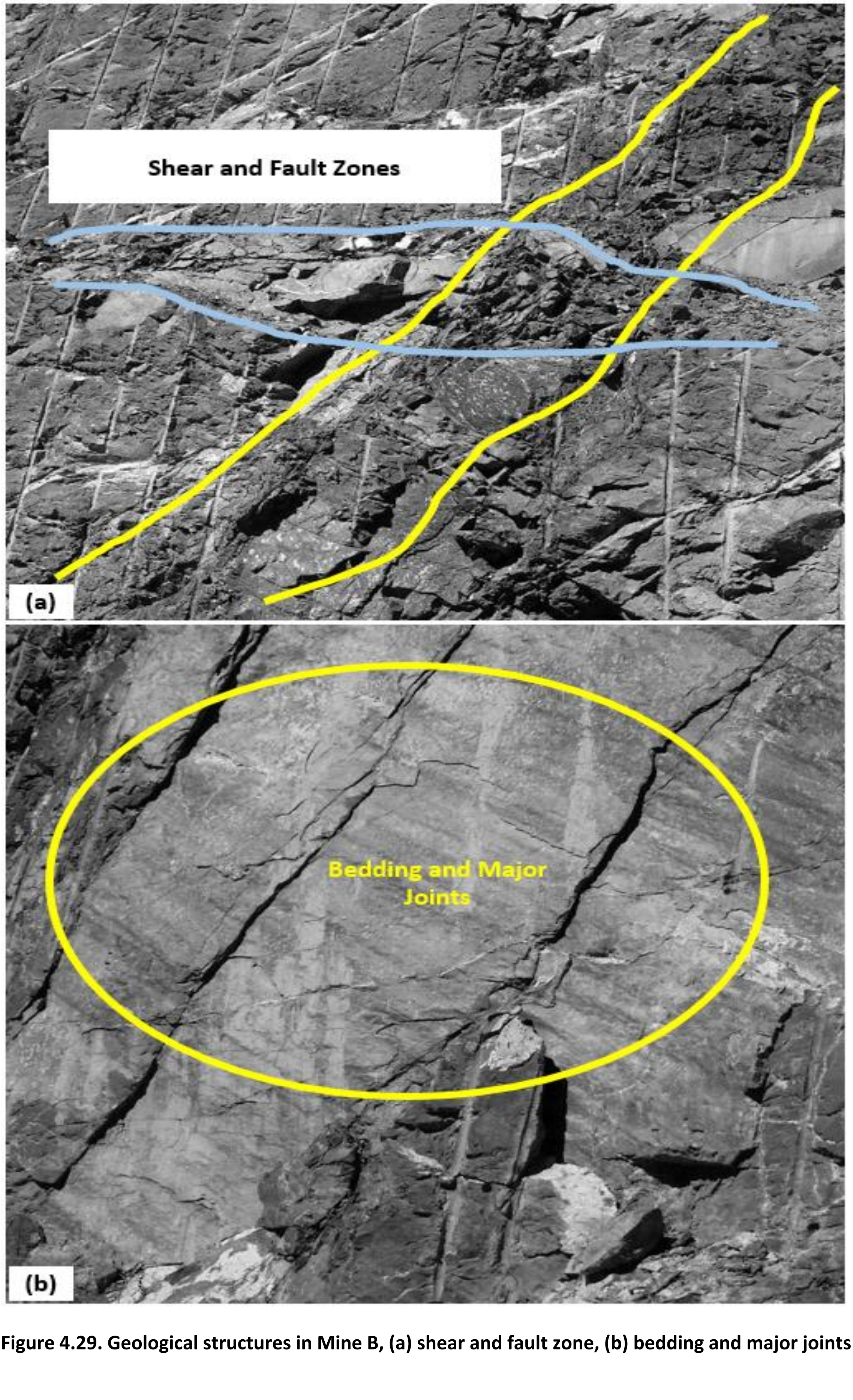

Figure 4.29. Geological structures in Mine B, (a) shear and fault zone, (b) bedding and major joints

The rock masses in the footfall and hangingwall are relatively homogenised. Intact rock strength has been obtained in the range of 140 MPa, and 190 MPa from diamond drilled cores and the results of laboratory tests. Principal stresses were measured to be from 18 MPa to 43 MPa. The azimuth of the in situ principal stresses is between 130 and 260 degrees. Ground loading factor is calculated below:

$$Ground\ Loading\ Factor = \frac{Rock\ Strength\ Mass}{Principal\ Stress} = 0.5 - 2.2$$

The ground behaviour problem related to loading factor is the intact failure and structural failure. The quality of rock mass based on Q classification obtained is between 2 and 17.

Groundwater condition caused corrosion in ground support devices at the mine site. Hydrology caused a reduction in strength in surface discontinuities and rock blocks.

An ore drive access with 4.6 m width and 4.6 m height is a part of the mine site, and is analysed for ground behaviour types. Figure 4.30 shows a schematic of the geological structure orientation compared with the orientation of the ore drive. The angle between the two orientations is in an unfavourable condition. The potential problem in this situation is intact and structural failure. Figure 4.31 illustrates the assessment of the ground behaviour condition in the primary access ore drive in mine B. The ground behaviour modes in the ore drive access of mine B are assessed as being in the intact failure and intact/structural failure classes.

4.5.3. Case example C

The ground condition in the main underground caverns of example A is considered for the identification of ground behaviour modes. For this purpose, three main factors including rock mass structures, stress concentration, and construction condition are evaluated based on site observations and available evidence the rock mass structures. Rock mass composition was classified as jointed/blocky/bedded in the main unit rocks and block/folded in weakness zones. The size of the underground excavations is 368–438 m in length, 21–41 m in width and 39–88 m in height (Figure 4.32).

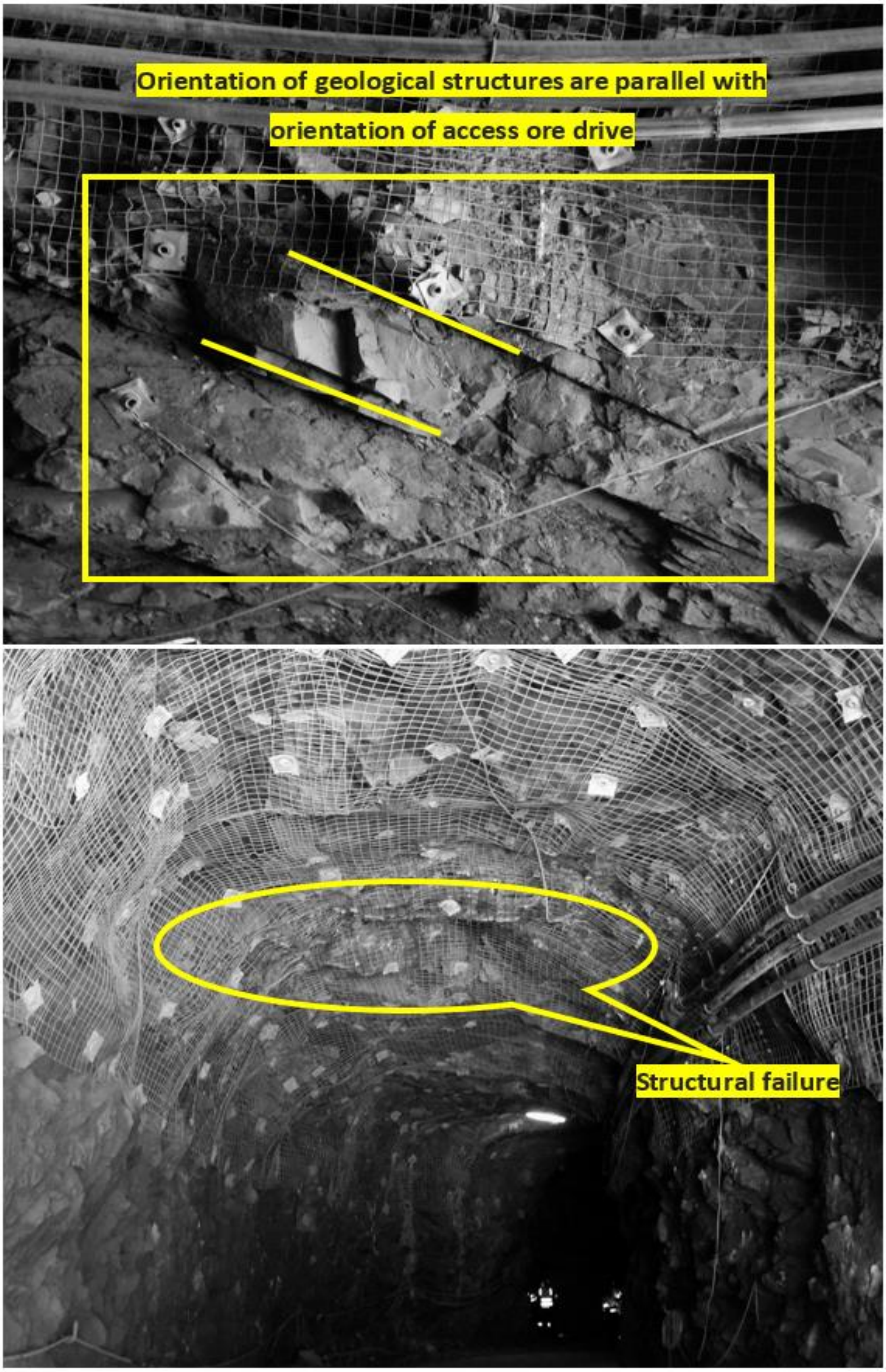

Figure 4.30. A state of geological structural orientation in the access ore drive and related ground problems

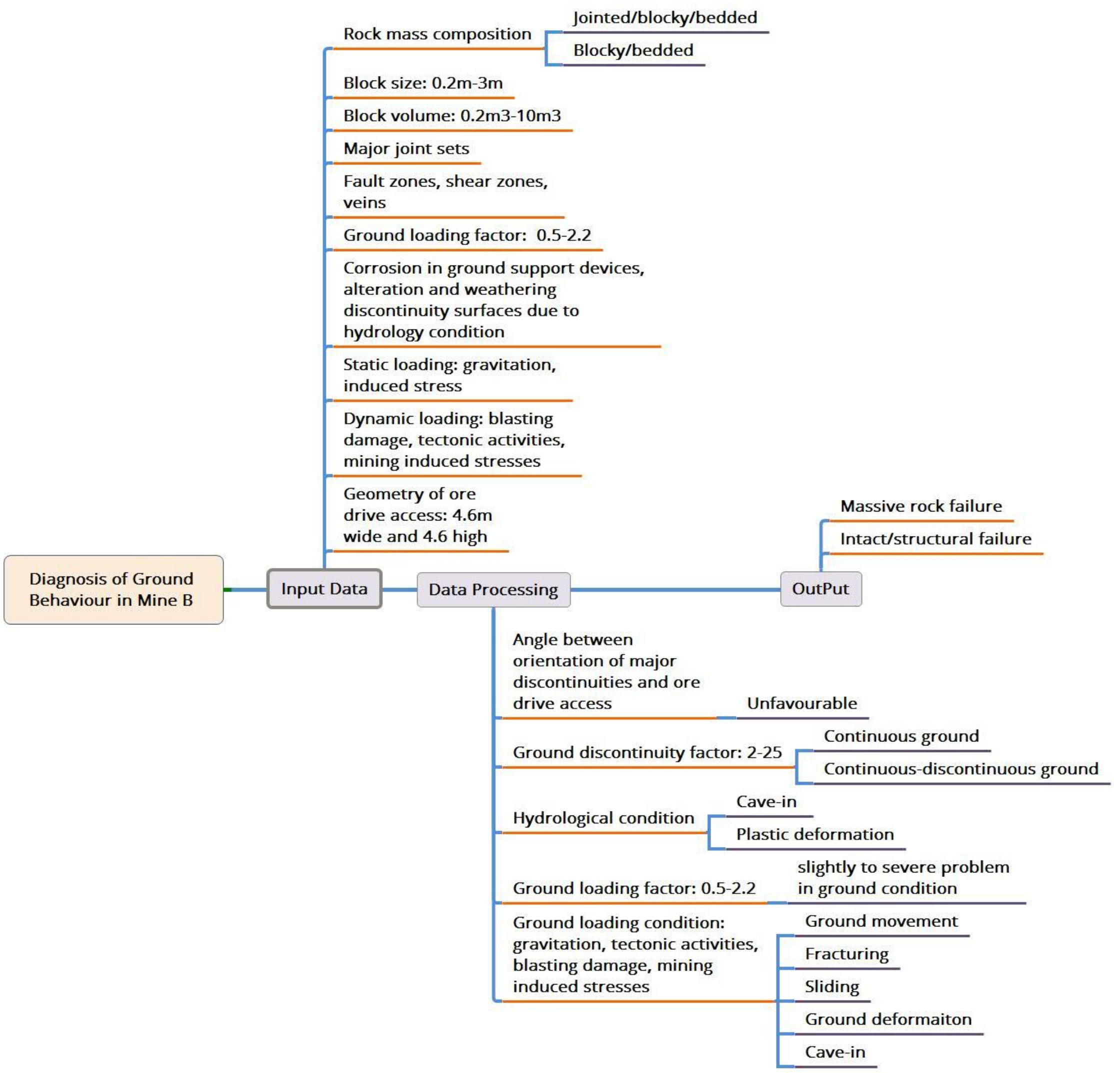

Figure 4.31. Diagnosis of ground behaviour in ore drive access of mine B

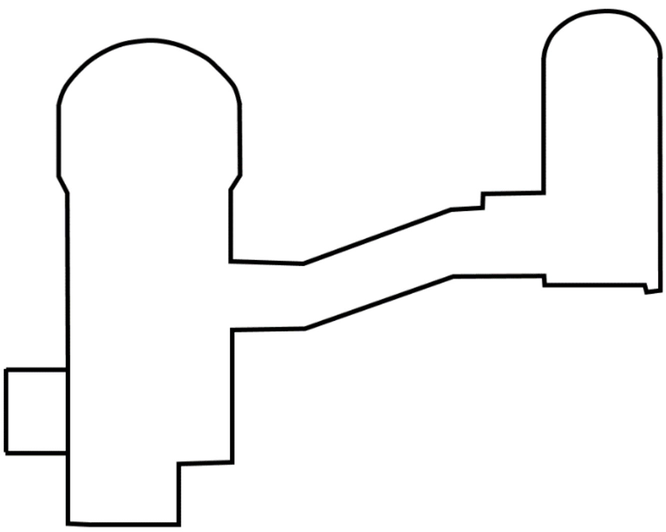

Figure 4.32. The layout of the main underground caverns (Not in scale)

Basaltic layers consist of columnar joint sets in the field, which affect engineering properties and behaviour of the rock mass. Main joints and beddings in the underground excavation have low tensile and shear strength that causes large deformation and failure due to static/dynamic loading during blasting, stress concentration and seismic events. Figure 4.33 and Figure 4.34 show rock features of the geological formation in the main cavern. The high-stress condition, large span underground excavation and geological structures have created a complex ground behaviour at the site. The geometry of an underground excavation and intact rock failure is shown in Figure 4.35.

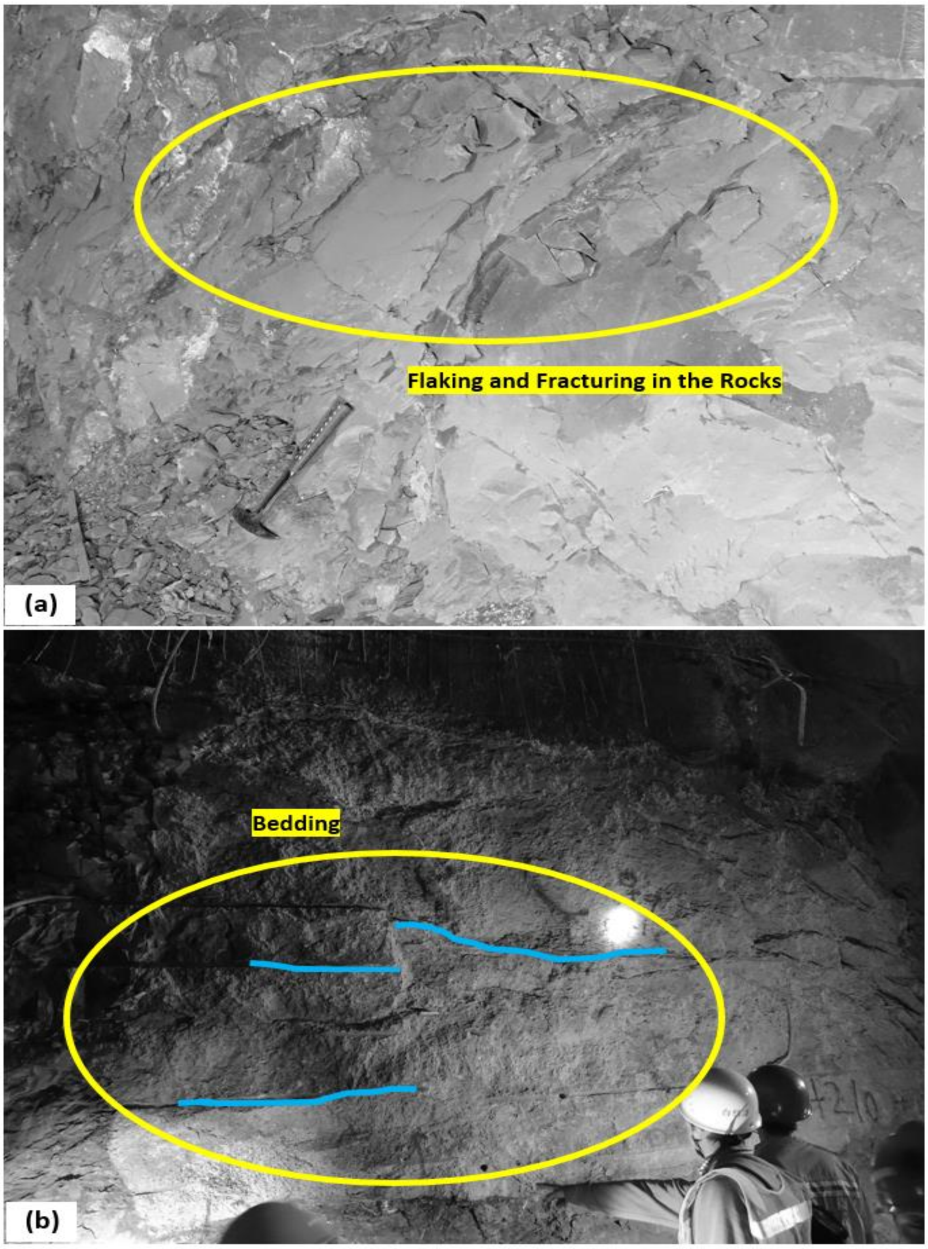

Figure 4.33. Geological structures in the main caverns: (a) flaking and fracturing in the rocks, (b) bedding

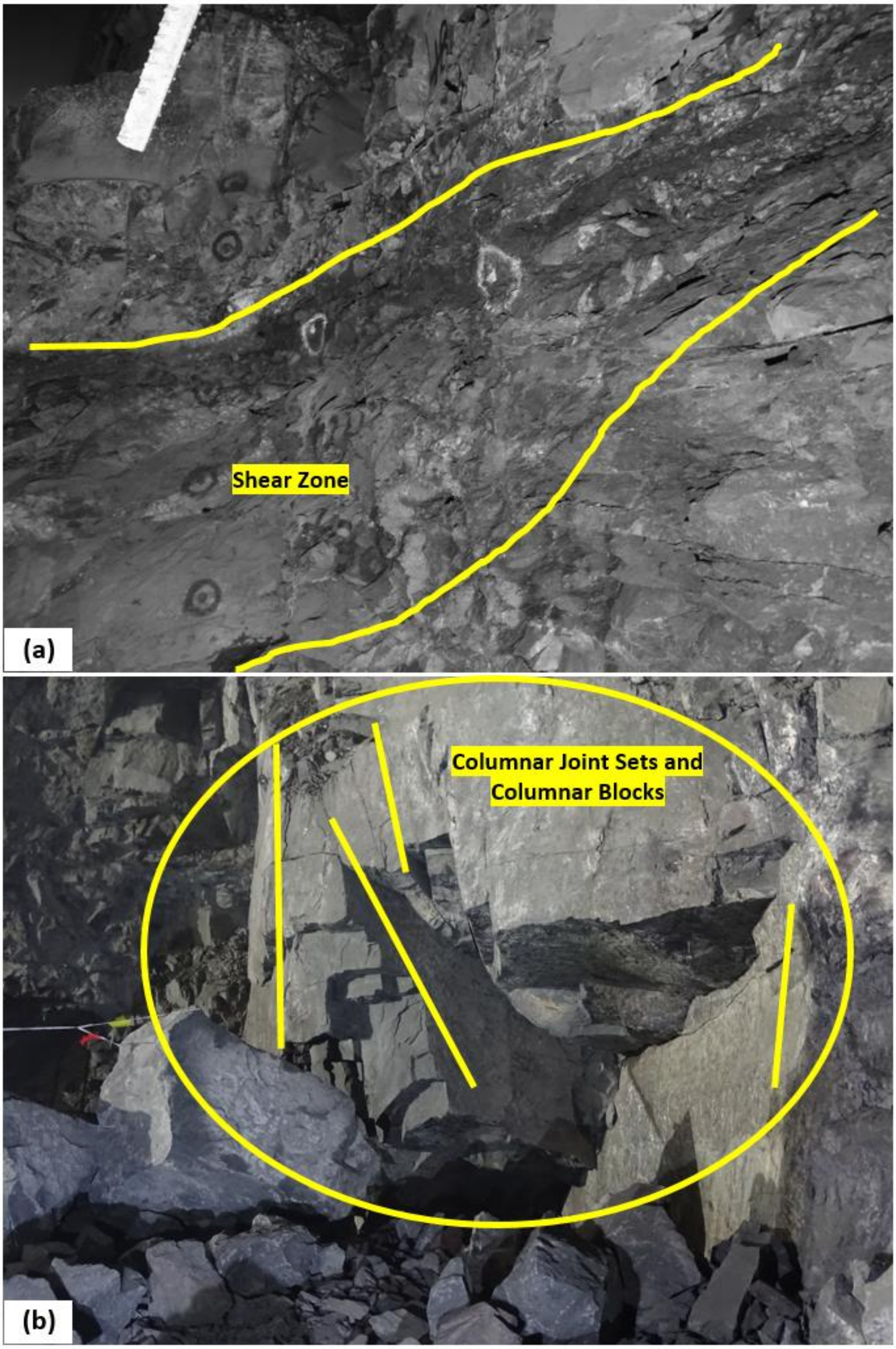

Figure 4.34. Major rock structures in the main caverns: (a) shear zone, (b) Columnar joint sets and columnar blocks

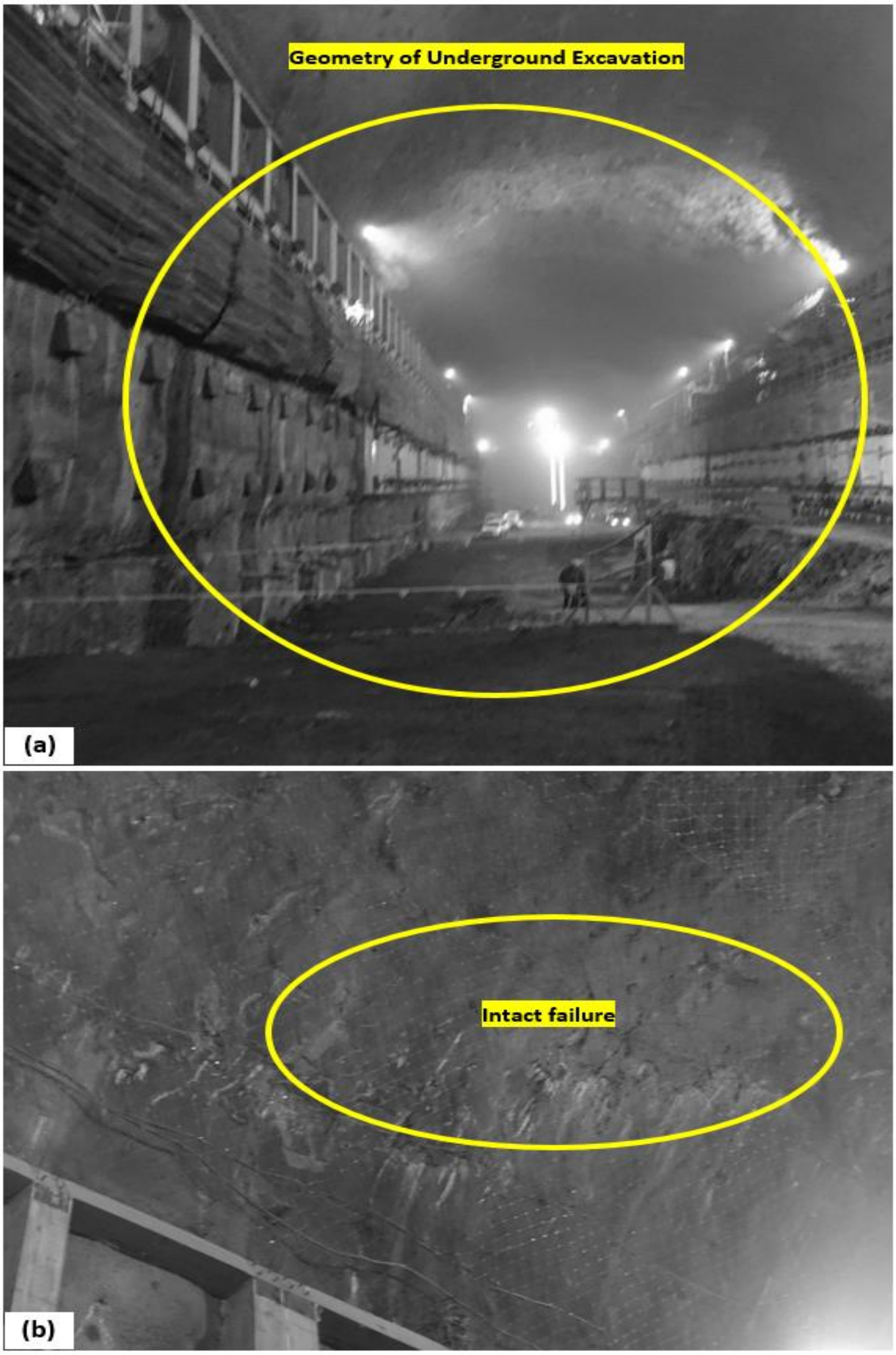

Figure 4.35. (a) The geometry of underground cavern excavation, (b) Intact rock failure

Figure 4.36 illustrates the assessment of ground behaviour condition in rock underground cavern excavations. Rock mass characteristics, active stress parameter, construction condition, hydrological condition, and the effects of seismic events and blasting were the primary and essential factors to diagnose ground behaviours. The main types of ground behaviour were identified as massive rock failure, structural/intact and structural failure.

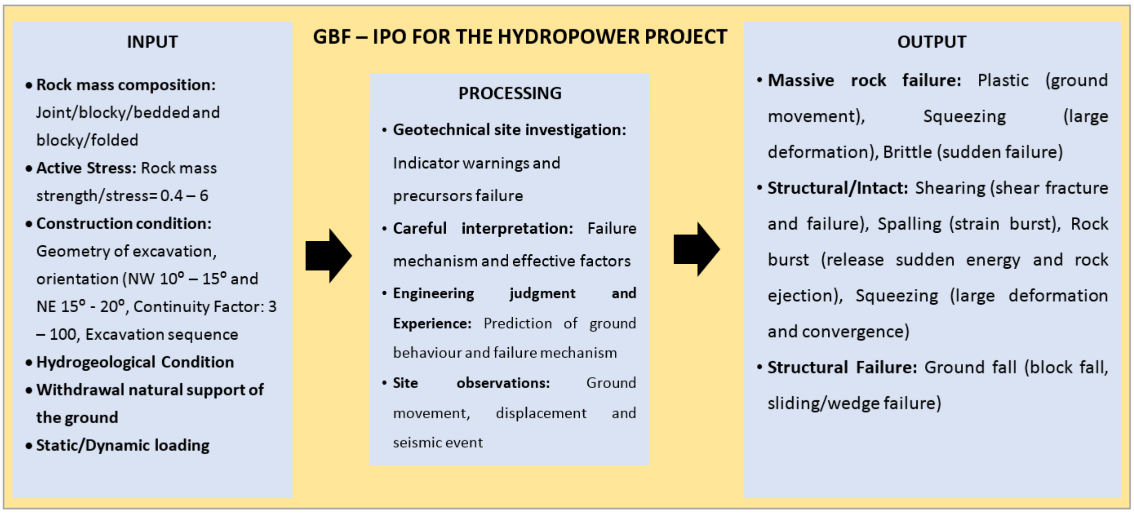

Figure 4.36. The flowchart for the diagnosis of ground behaviour (DGB–IPO) in the main underground caverns

4.5.4. Case example D

This case example is a deep underground mine in Western Australia. The mine is a massive nickel deposit and consists of a lower olivine ortho to mesocumulate unit, of variable thickness along strike, and overlain by thin spinifex textured komatiite flow units (Perring, 2015). Three main geological rock units are: (1) the immediate hangingwall that includes ultramafic and talc magnesite ultramafic, with minor foliation and schistosity, and major talc content, (2) ore, sulphides of 0.1 m to 8 m in width, and (3) immediate footwall is a mafic to intermediate fine-grained rock, jointed or foliated close to the mineralisation. Figure 4.37 shows a cross-section of the geological structure and mine layout. Two major structures at the mine site are shear zone and thrust fault, which are important in considering ground behaviour in underground stopes. The intact rock strength is in the range of 70 MPa to 130 MPa, and elastic modulus properties change between 50 and 65 GPa. The depth of underground stopes is about 700 m from surface. In situ stress estimates vary from 10 MPa to 45 MPa. The rock mass structure was classified as Jointed/blocky/bedded and blocky/folded. Figure 4.38 shows rock mass structures in the mine site.

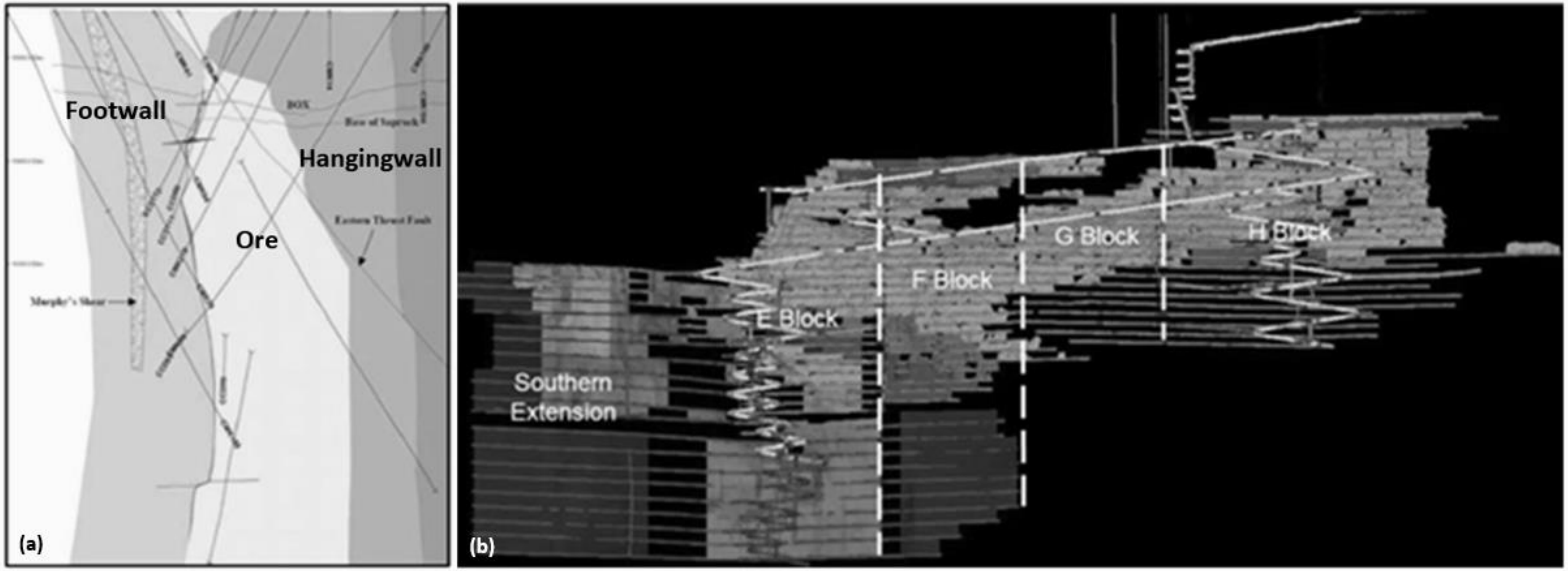

Figure 4.37: (a) Geological structure and (b) Mine layout cross section

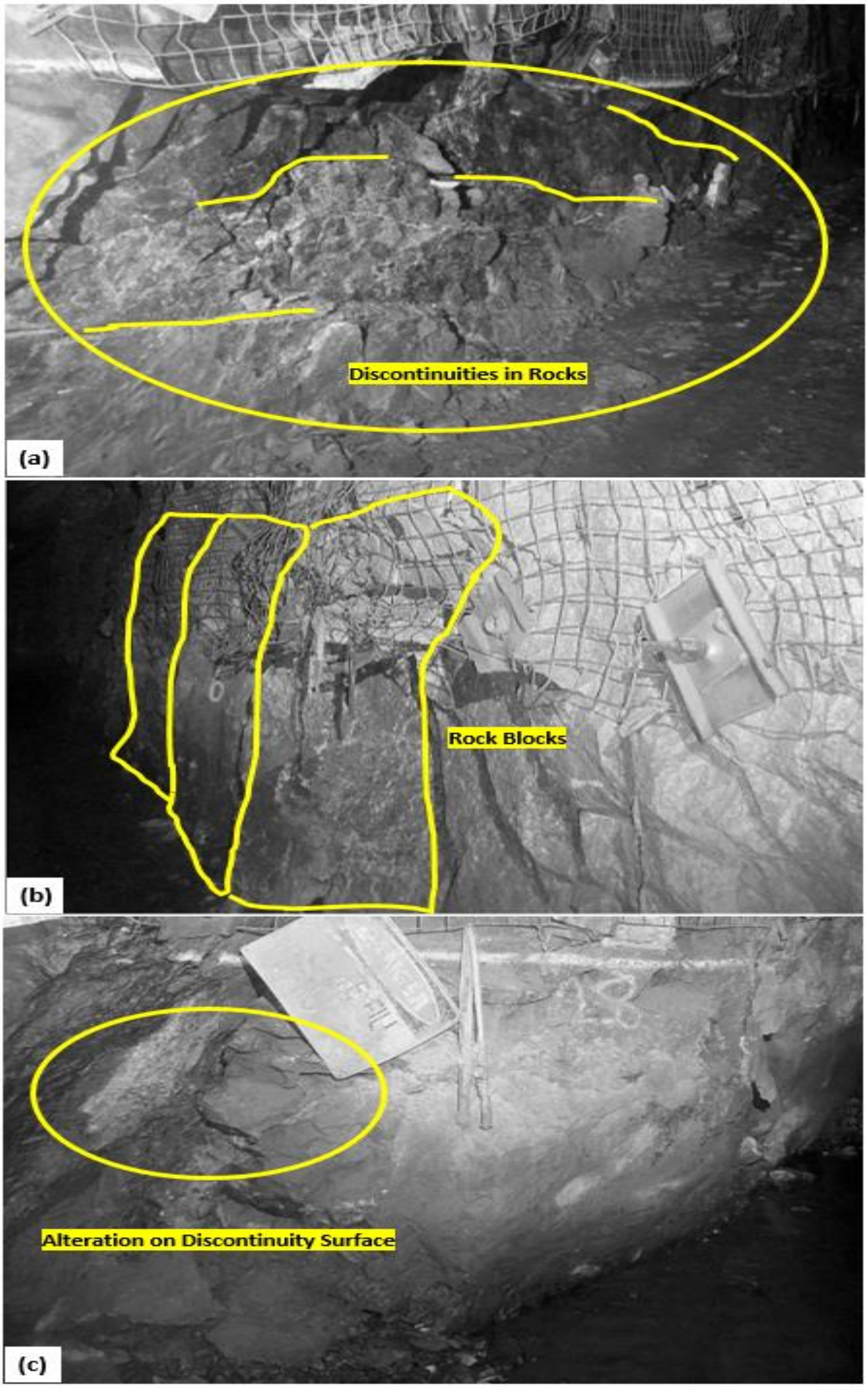

Figure 4.38. Rock mass structure in the mine site; (a) discontinuities in the rocks, (b) rock blocks, (c) alteration on discontinuity surface

Ground behaviour is assessed in an ore drive access to underground stopes and shown in Figure 4.39. The dimension of the ore drive is 5.2 m width and 5.7 m height (Figure 4.40-a). Ground loading factor was estimated between 0.3 and 1.6. This shows that there is a potential for a moderately – severe ground problem during mining operations. The main static and dynamic loading were identified as gravity, mining-induced stress, blasting damage, and tectonic activities. Ground behaviour modes are identified as massive rock failure, and intact/structural failure modes at mine D. Figure 4.40 shows an example of rock mass behaviours in the ore drive.

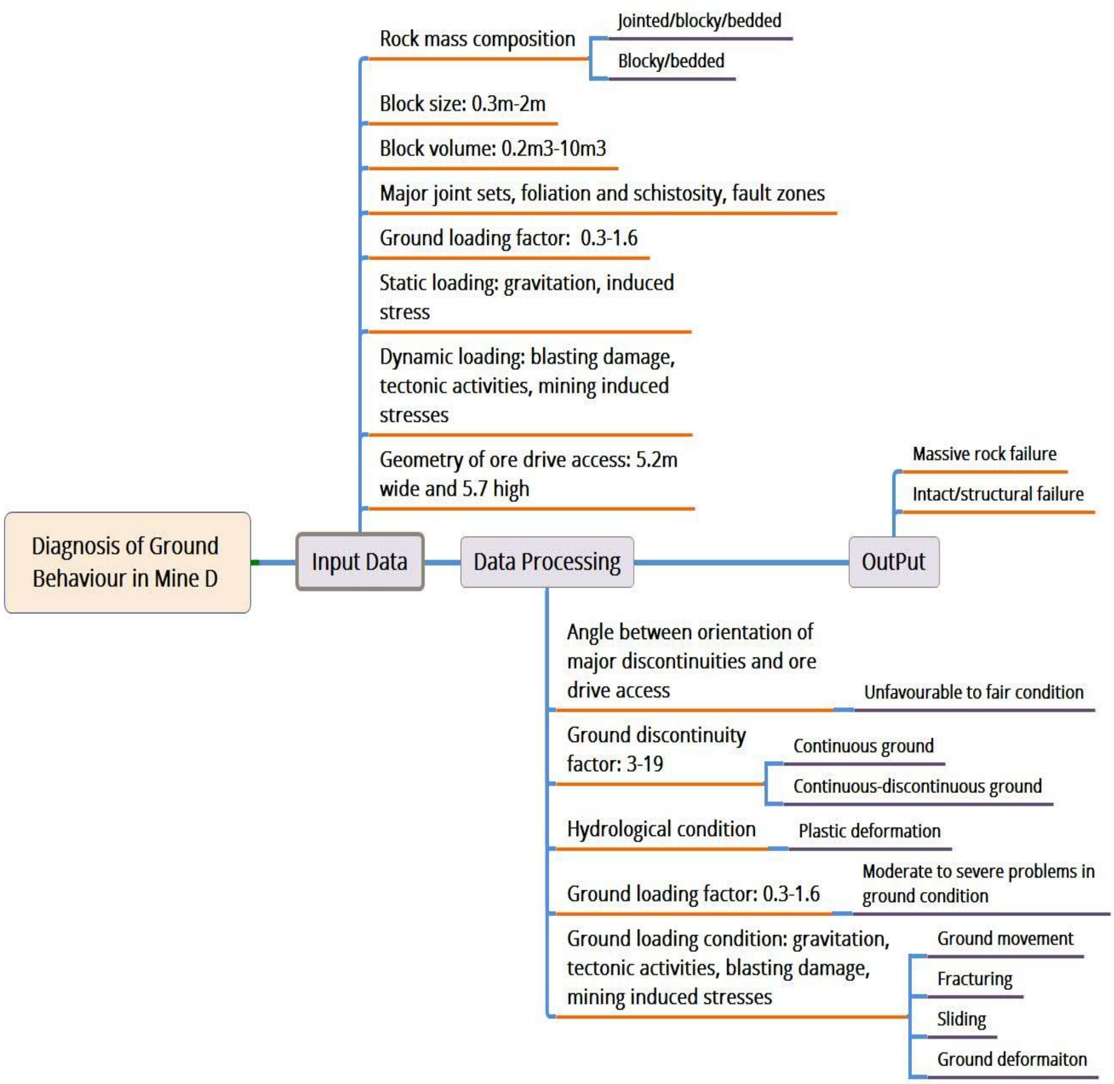

Figure 4.39. Diagnosis of ground behaviour in the ore drive of mine D

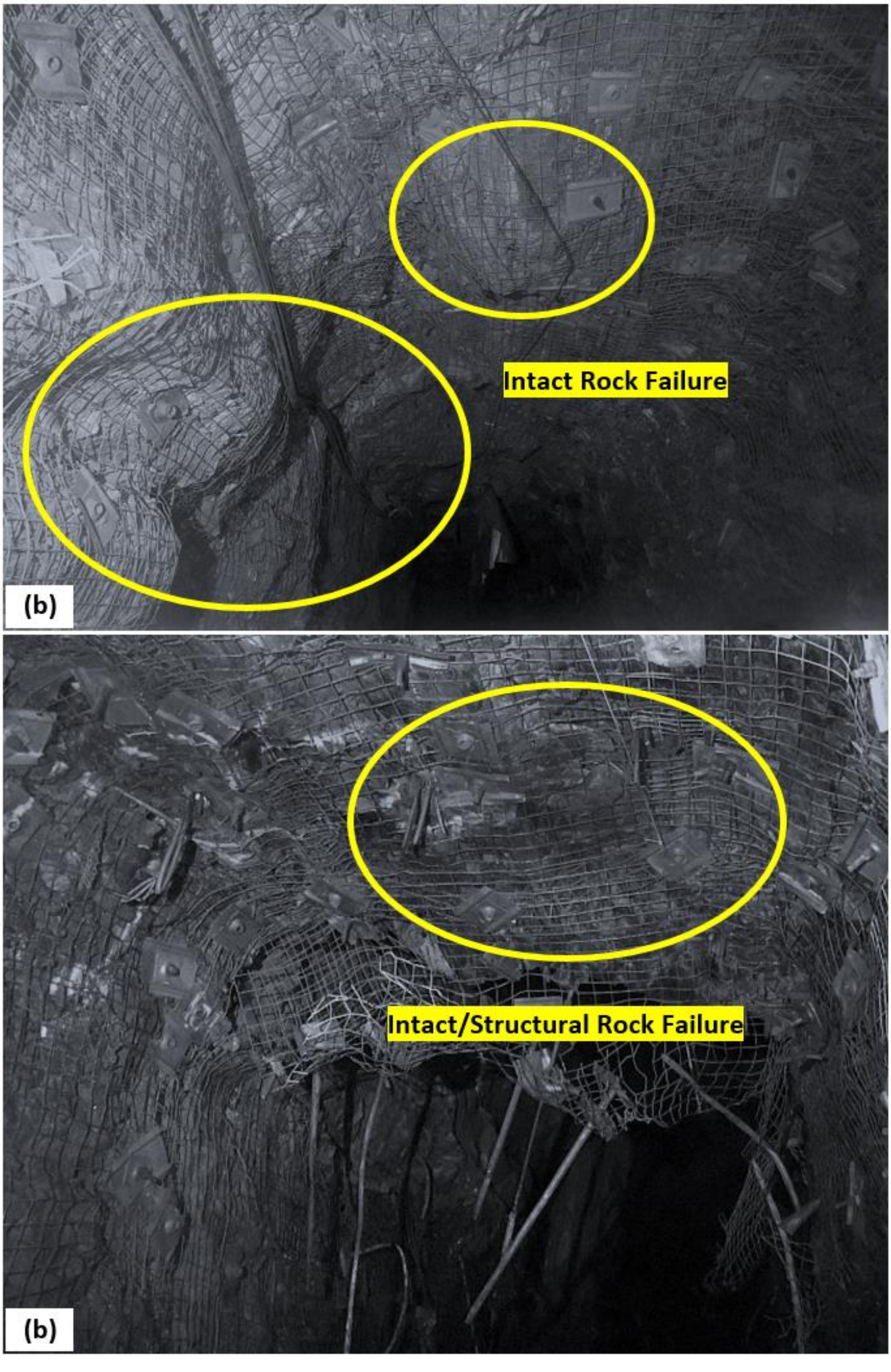

Figure 4.40. Ground behaviour modes in ore drive (a) intact rock failure, (b) intact/structural failure

4.6. Conclusion

Natural environmental conditions and engineering activities cause ground behaviour modes in deep underground mines. Major geological structures, rock mass composition, rock mass properties, stress states in rock masses, hydrology, static and dynamic loading are effective geotechnical factors in the projects. Weak geological formations like faults, shear zone and foliation lead to an anisotropy behaviour and a reduction in strength. Deformability of rock is increased in weakness zones. The geometry of geological structures at depth is associated with uncertainty in their thickness and dip, and judgment and correlation of all parts of rock zones are required to interpret and describe ground behaviour.

The relation between rock mass strength and major principal stress at great depth is a simple and efficacious factor to define the potential ground behaviour modes. Ground behaviour modes in intact rock and rock masses vary from brittle to hardening as confined stress condition changes. Hydrology and groundwater may reduce the strength of discontinuity surfaces. The mechanical and chemical properties of some rocks and minerals like clay and calcite become different. Water pressure in rock is a typical cause for challenges in rock engineering projects. Swelling, fluid flow, cave-in, plastic deformation and ravelling are some of the ground behaviour modes caused by groundwater.

Two types of loading are encountered through the construction stages and mining operations: static and dynamic. The origin of static loading is gravity, changes in groundwater level, and temperature. Dynamic loading is derived from tectonic activities, blasting damage and gravity collapse in underground mining. Fracturing, ground displacement, sliding, flowing and gas emission are some of the common ground reactions to loading.

The geometry of underground excavation, excavation method, excavation sequence, and the orientation of the underground opening with major discontinuities cause ground problems. Deformability of rock mass structures in a large span opening is more than that in a small span under similar conditions. Blasting method generate a damage zone in the rock mass surrounding excavations.

Influencing factors on ground behaviour were used as input data. Rock mass behaviour at great depth was assessed and evaluated by determination of geometry of excavation, continuity factor, excavation method, excavation sequence, main types of loading, and orientation of underground excavation. The classification of ground behaviour modes was presented as stable, massive rock failure, intact/structural failure, structural failure, and water effect. There is a frequently changing ground behaviour in deep underground mines due to a wide range of stresses, deformation and static/dynamic loading types in the ground.

The proposed classification was applied in several case studies in deep underground excavations. The reliability of the proposed method was proved in the projects by recording evidence of rock mass behaviours and site observation methods.

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CHAPTER 5: DEEP HARD ROCK MASS FAILURE MECHANISM

CHAPTER 5: CONTENTS

5.1. Introduction

Deep underground mining activities cause induced stress and change rock mass behaviours, which may lead to rock failures. Sudden and violent failure mechanisms from the intact rock scale to large rock mass structures is not always recognized. High-stress conditions, seismic events and interlayered weakness zones in rock mass structures are some of the important, relevant factors in failure mechanisms at great depth. Identifying a failure mechanism in multiple ground behaviour modes from stable rocks to progressive rock failures is a trigger to manage ground hazards.

The typical failure modes in rock in underground construction include rock fall, brittle failure, rockburst, swelling, and squeezing, which are associated with discontinuities' condition, in-situ stress, rock mass properties and project environment. Rock mass behaviour and any changes in rock mass behaviour are not always recognisable as warnings of failure. A procedure to perceive pre-failure of a rock mass can be useful for rock engineers in the prediction of geotechnical failure and collapse, in order to avoid a major loss. Analysis of case studies in geotechnical fields indicates that damage and progressive failure in a rock mass have warning signs as indicators and precursors (Szwedzicki, 2003). Geotechnical indicators such as faults and folds signal that a rock mass has the potential for failure.

This Chapter is focused on the evaluation of failure mechanisms in intact rocks and rock mass structures (Figure 5.1). Initiation cracks, propagation fractures and increasing deformation of rock, especially in weak geological structures under high stresses, are the critical indication warnings before the occurrence of failure and collapse. Failure mechanisms in hard rocks progress with tensile cracking and extension fracturing and then the occurrence of the sudden failure. Weakness zones at great depth are more prone to plastic and time-dependent behaviour. Failure precursors appear as deformation under a long-term stress condition in the rock mass structures containing various defects such as discontinuities, veins, shear zones and faults.

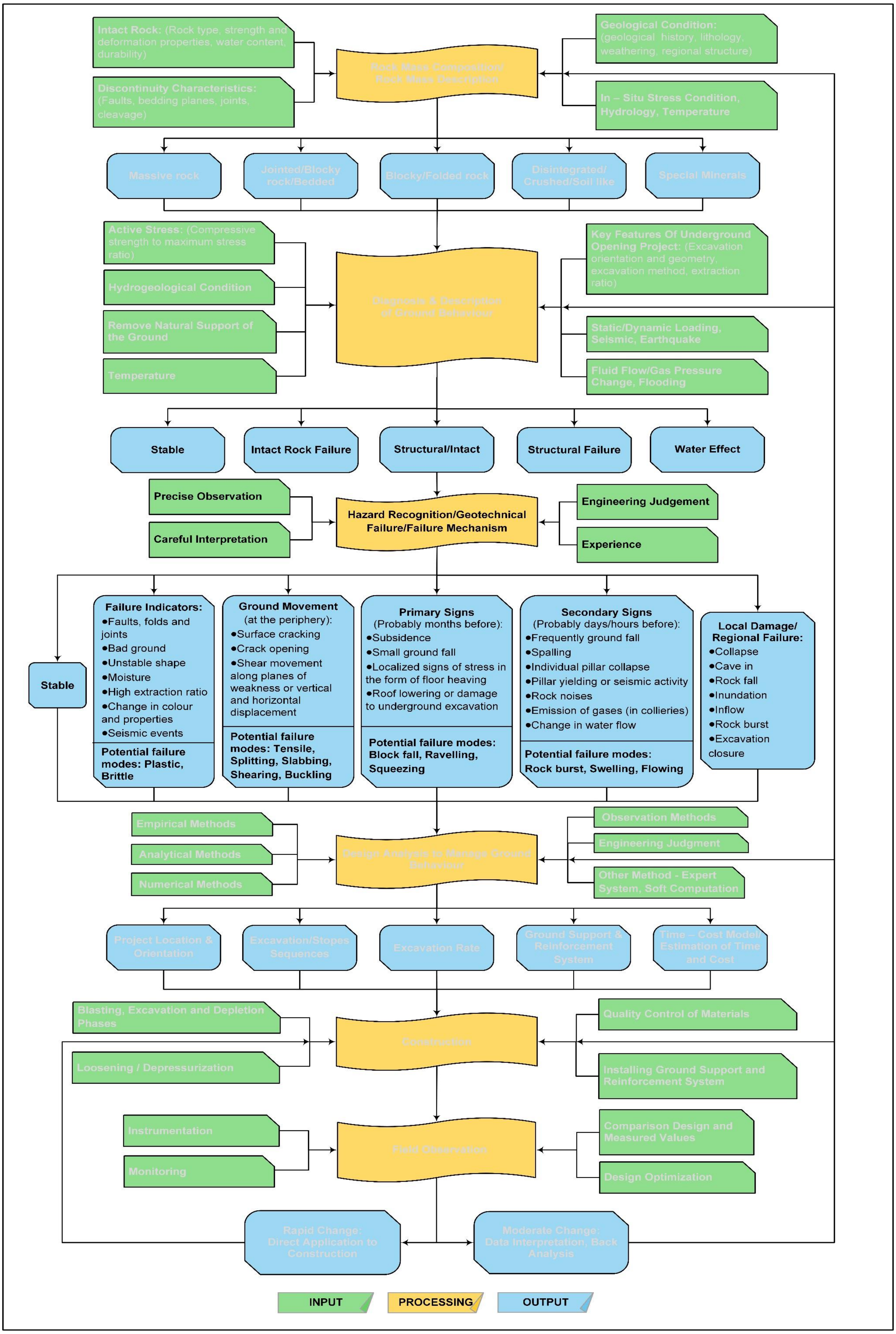


Figure 5.1. Identifying failure mechanism in deep underground mining

5.2. Intact rock failure mechanism

The behaviour of intact rock depends on its physical characteristics (such as minerals, texture, density and porosity) and environmental condition (such as confining stresses, temperature and saturation) (Figure 5.2).

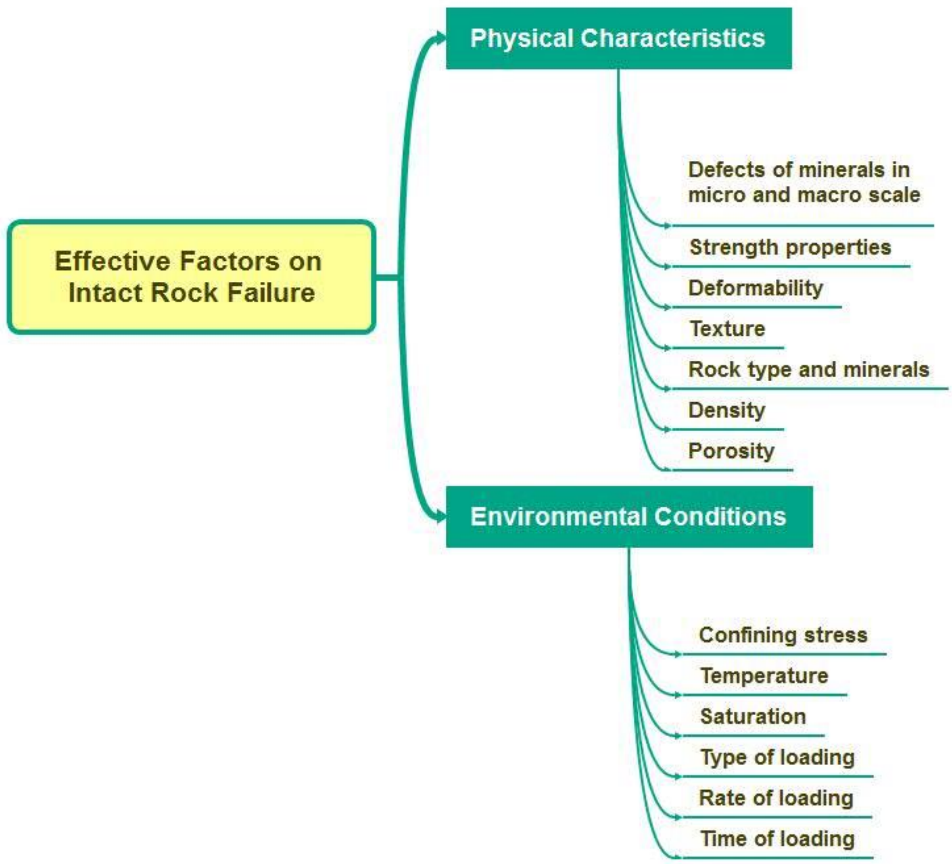


Figure 5.2. Effective factors on intact rock failure

Typical intact rock behaviours are elastic behaviour, brittle behaviour, and ductile/plastic behaviour. Most rock materials have an elastic limit, which is related to the maximum amount of stress from which there is a capability to recover its original shape. Rocks usually have a variety of behaviours in different conditions. Some rocks display softening behaviour and avoid abrupt loss of rock strength after weakening of the rock, due to low confining pressure. The stress-strain behaviour of intact rocks is shown in Figure 5.3. According to this figure, elastic–stable microcracking, stable–unstable microcracking, unstable microcracking – brittle failure, and brittle failure–residual strength are the main stage of the failure process in intact rocks.

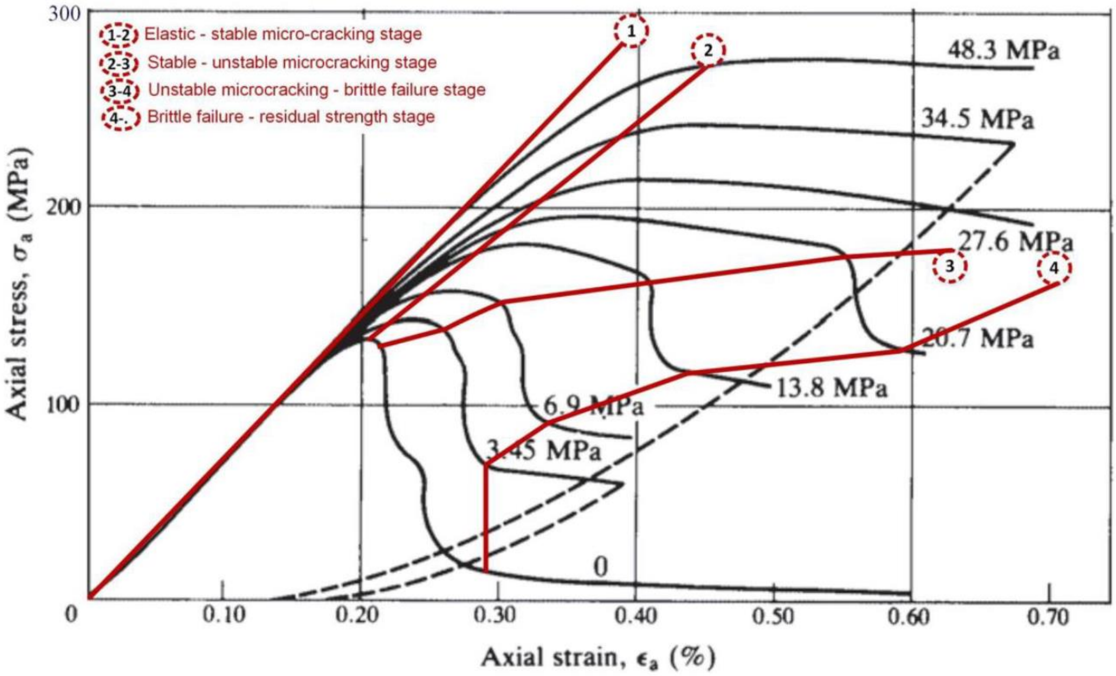


Figure 5.3. Stress-strain behaviour on intact rocks (Sharifzadeh et al., 2017)

The mechanism of failure in an intact rock is presented in Figure 5.4. In phases A and B, the rock has an elastic behaviour. In phases C and D, microcrack initiation and growth occur inside the rock. Propagation of the micro-crack with an increase of the stress leads to macro-crack growth in phase E. Phases F and G are related to failure progression in the rock such that cohesion strength reduces and the frictional component of strength increases.

The strength of intact rocks is derived from friction between mineral grains and cohesion controlled by cementation bonds. The primary friction angle of minerals in lower bound conditions maybe 10 degrees or less (Hencher, 2012). It is usually more than 30 degrees for planar rock joints. Figure 5.5 is a schematic concept of cohesion and friction of rock material under different shear and normal stresses. The frictional component can be reduced by polishing of the surfaces. At a very high-stress level, dilation of the rock is restricted, and all the asperities undergo shearing.

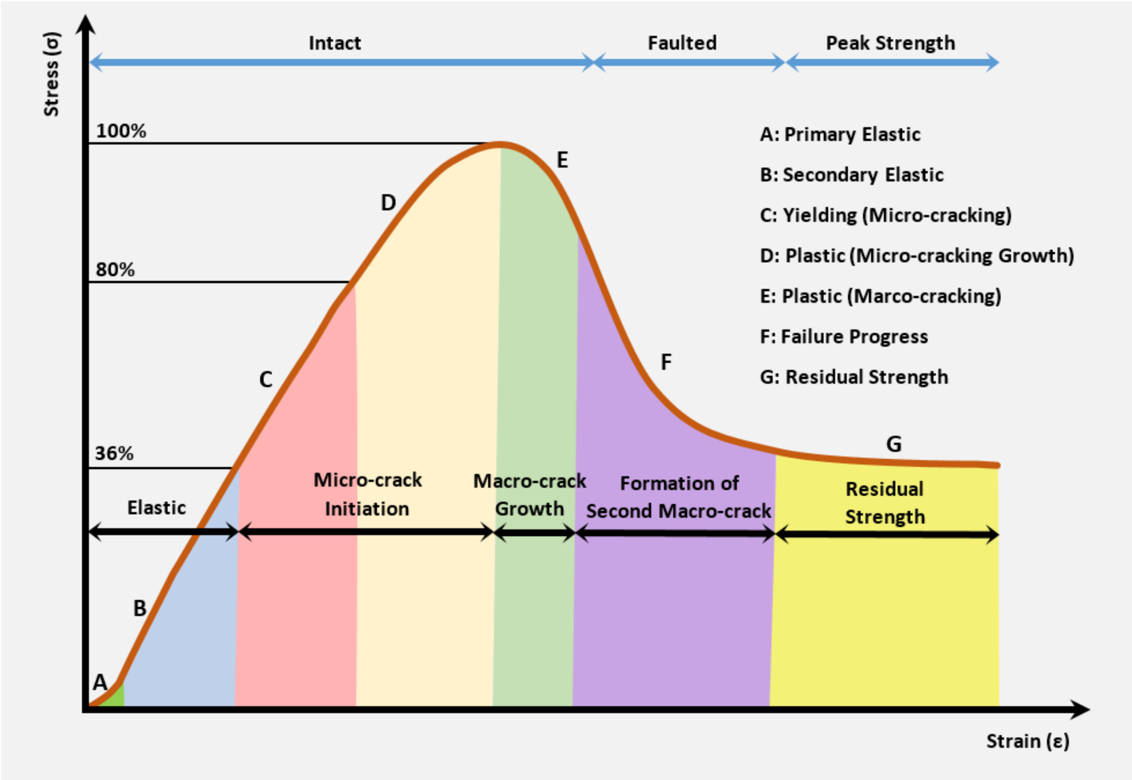


Figure 5.4. Failure mechanism in an intact rock (Modified after Balideh and Joseph (2012))

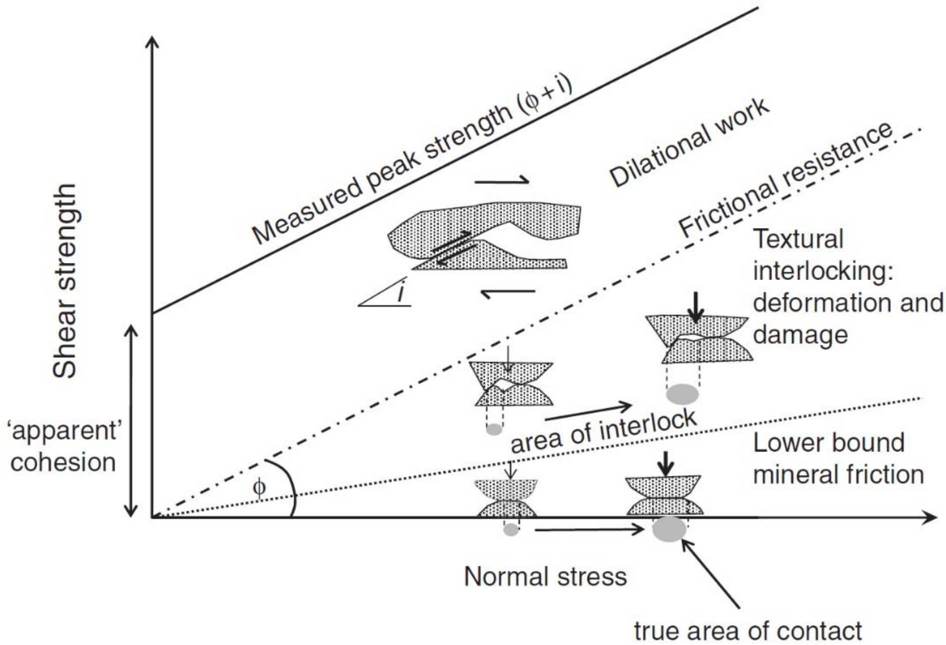


Figure 5.5. A schematic concept of cohesion and friction in rock materials (Hencher, 2012)

Post-peak behaviour of rocks is affected by material properties, and it is described by the brittleness index based on aggregated elastic energy in the rock during loading. Extension of failure in the post-peak state is caused by a portion of this energy. The brittleness index is calculated by equation (5.1) (Esmaieli et al., 2013):

$$k = \frac{dW_r}{dW_e} = \frac{\frac{d\sigma^2(E-M)}{2EM}}{\frac{d\sigma^2}{2E}} = \frac{E-M}{M} \qquad \textbf{(5.1)}$$

Where; M: post – peak modulus and E: Unloading elastic modulus

Parameters in equation (5.1) are obtained from the stress-strain curves.

Figure 5.6 shows the brittleness index variations corresponding to characteristic shapes of stress-strain curves. According to equation (5.1), the brittleness index varies between $-\infty < k < 0$. This index increases from shapes on the left to shapes on the right. This suggests that changing and reducing rock properties can cause the rock to be more ductile.

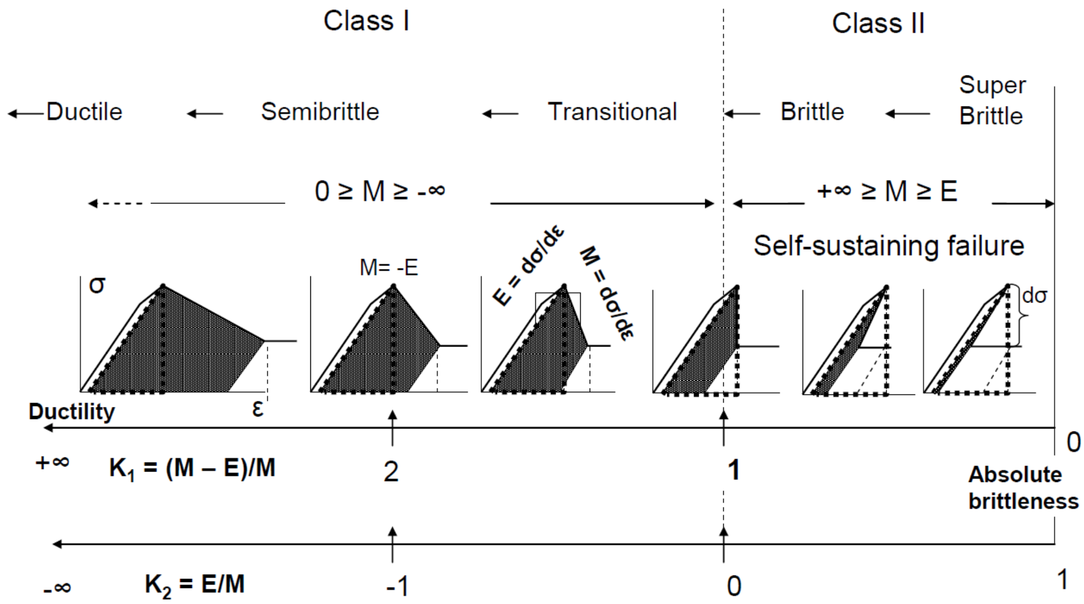


Figure 5.6. The variation of Brittleness Index (Tarasov and Potvin, 2012)

Failure mechanism of intact rock at great depth is presented in Figure 5.7. The fracturing and occurrence of different types of failures are associated with the properties of intact rocks, excavation geometry and stress condition. The failure process of hard rock at great depth is associated with initiation and propagation of cracks in brittle fractures. The primary form of damage in hard rock is by extensile cracking instead of shear cracking, even under compression load. Extensile crack propagation causes spalling ground mode under a low level of stress confinement. At a high-stress level, sudden failure such as rock burst and rock ejection occurs in deep underground mines. In medium and soft rocks, cracking and fracturing is through shearing. Squeezing, shear failure and large deformation are the typical failures in these type of rocks.

5.3. Rock mass failure mechanism

Rock mass properties are influenced by a variety of factors such as discontinuity conditions and direction, persistence, frequency of joints and fractures and their surface characteristics. A rock mass contains a variety of rock materials, with each rock type having its variability of behaviour and characteristics. The variability is assessed using laboratory or field testing and site investigation, and this assessment is an essential function. However, sometimes it is not sufficient, especially about common geological features such as joints, faults, interface surfaces between different rock materials, bedding and shear zones that may have a different and unexpected behaviour mode.

Since the geological features are relatively complex, it is almost impossible to describe their structure and behaviour as well as man-made structures such as aircraft. Uniform rock masses may have entirely different behaviours at distinct layers of depth due to various confining stress states. The behaviours of rock mass components affect the mechanism of instability in underground structures (Stacey, 2003).

A variety of factors are used for the identification of geotechnical hazards in underground mining, listed below (Simmons and Simpson, 2006):

1. Overburden characteristics related to a geological structure
2. Frequent failure occurrence, during a particular phase of the mining cycle
3. Previous knowledge, utilising the previous experience and results of monitoring and mining operations
4. Lithological and bedding structure, in order to detect mobilisation of composite mechanisms
5. Geometry of underground stopes used for assessing and evaluating the scale of the failure process and its mechanism in the mining operations
6. Understanding the nature of the failure mechanism by considering post failure observations and also trying to diagnose the failure in the rock mass

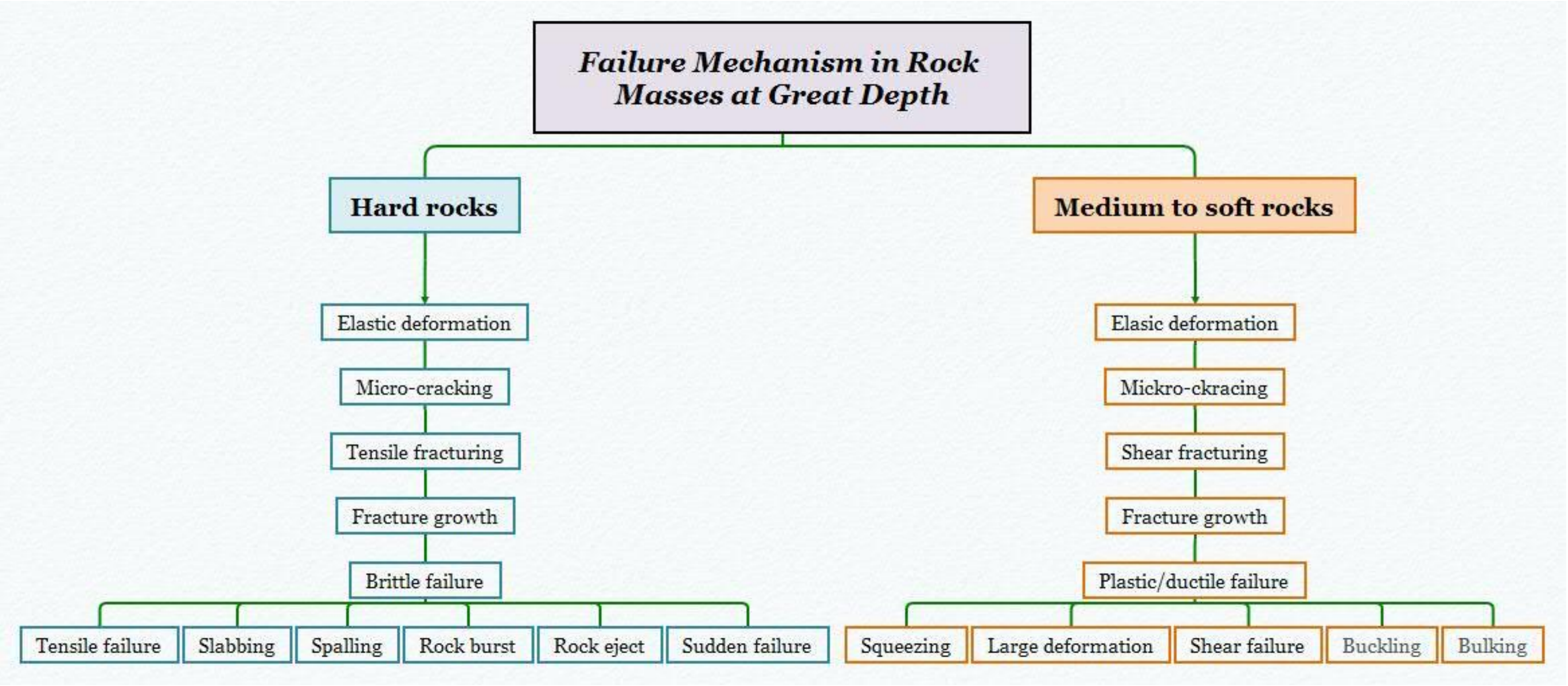


Figure 5.7. Failure process and mechanisms of intact rocks at great depth

Deformation of a rock mass may occur in multiple sequences due to a variety of tectonic activities. Figure 5.8 shows a matrix of possible deformation and failure modes in rock masses. The five basic deformation modes include tensile fracture, shear fracture, brittle shear zone, ductile shear zone, and pervasive ductile fabric which has been shown in the figure. The possibility of multiple sequences of rock deformation can be explored by following in a clockwise direction, where one mode of deformation leads to another type.

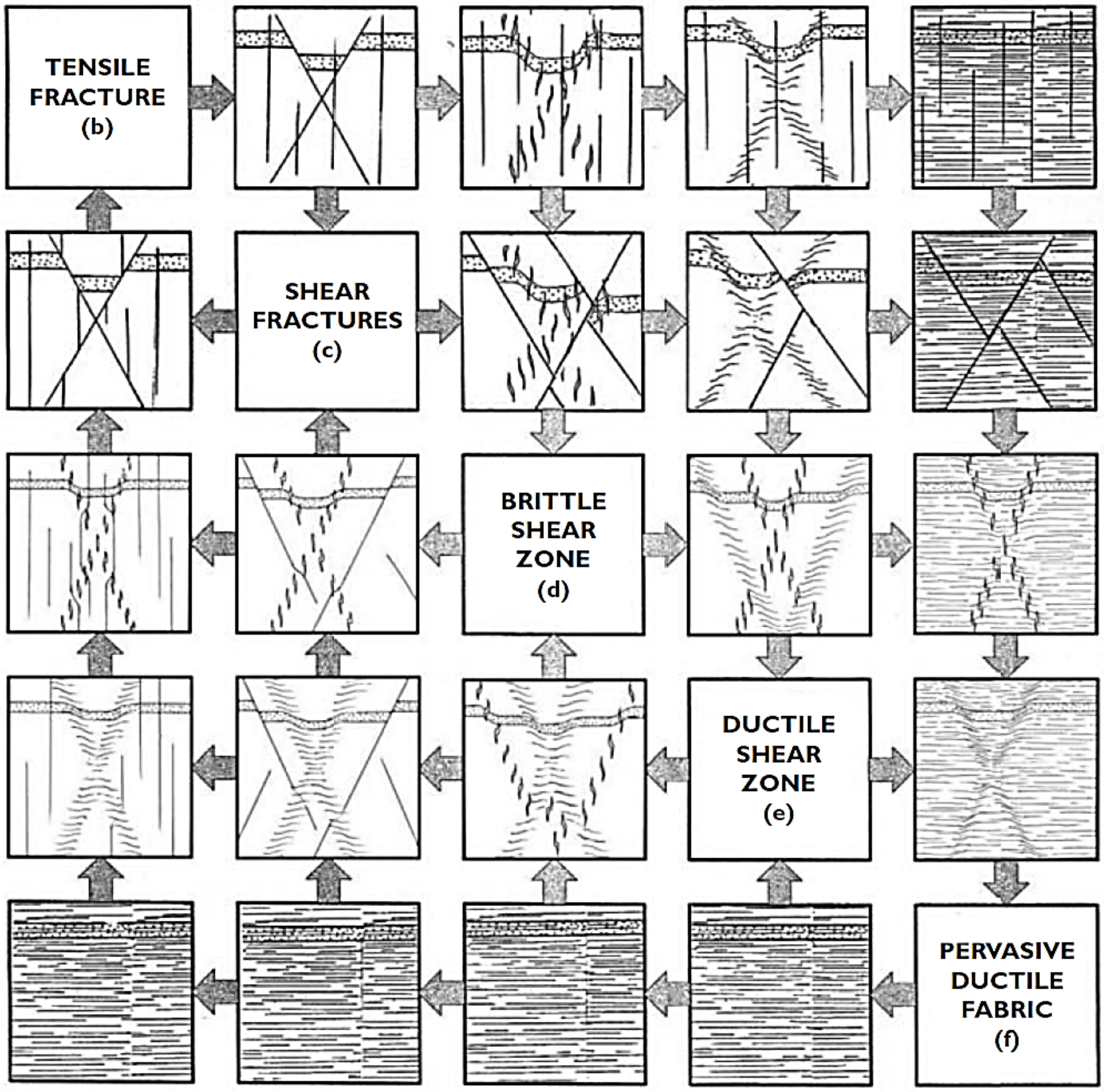


Figure 5.8. The interaction between multiple sequences of matrix deformation in rocks (Hudson and Feng, 2015)

The different post-failure behaviour of rock masses based on the GSI system, including elastic-brittle, strain – softening, and elastic-perfectly plastic, is shown in Figure 5.9. The brittle mode behaviour is more suitable for good quality rock structures with GSI > 75. In the case of jointed rock mass with GSI between 25 and 75, a strain-softening behaviour mode is dominant. Weak rock structures mostly behave in a perfect-plastic mode.

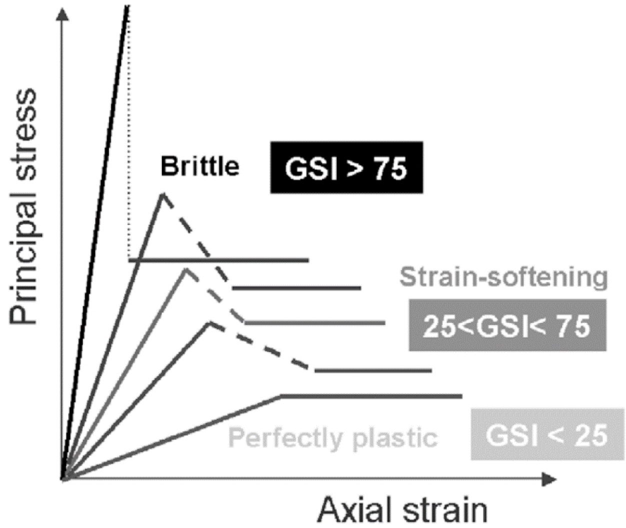


Figure 5.9. Post-failure behaviour modes of rock mass based on the Geological Strength Index(GSI) system (Alejano et al., 2009)

The strain-softening behaviour is described as a gradual transition failure from peak strength to residual strength. The failure criterion for strain-softening rock behaviour for this model is defined in the following equation (Alejano et al., 2009):

$$\boldsymbol{f(\sigma_r, \sigma_\theta, \eta) = 0} \qquad \textbf{(5.2)}$$

Where,

η: softening parameter

Figure 5.10 shows strain-softening behaviour in a confined compressive test. In this figure, η^* is a controlling softening parameter between softening and residual modes. Whilst η varies between 0 and η^* $(0 < \eta < \eta^*)$, softening behaviour can occur in a rock. Residual mode is accomplished for $\eta > \eta^*$(Alejano et al., 2009).

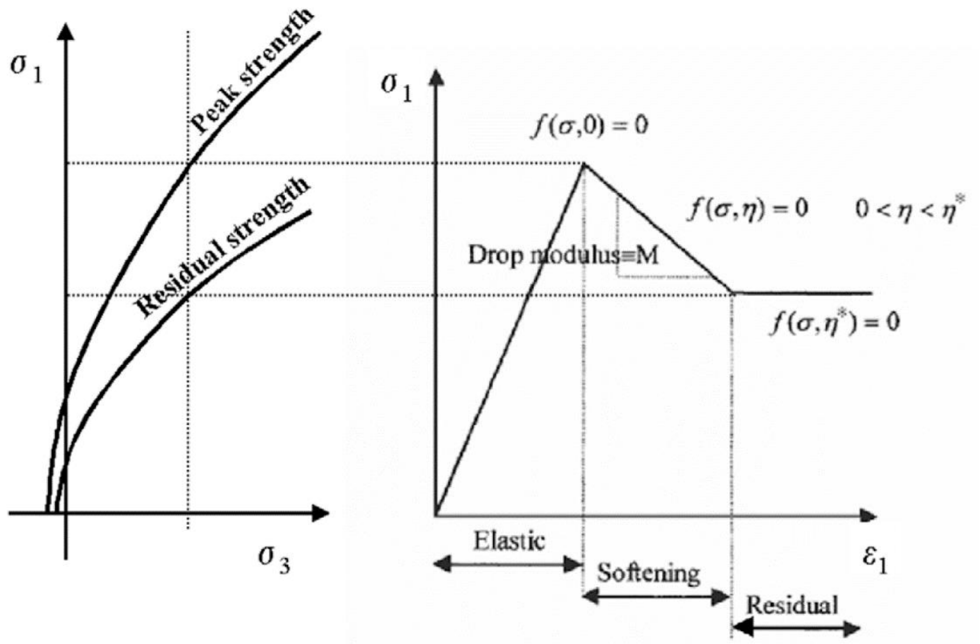


Figure 5.10. Stress-strain curve for softening behaviour in a confined compression test (Alejano et al., 2009)

The plastic strain increments for softening behaviour in a constitutive equation can be calculated as in the following equation:

$$\boldsymbol{\varepsilon_r^p \bullet = \lambda \bullet \frac{\partial g}{\partial \sigma_r} \; and \; \varepsilon_\theta^p \bullet = \lambda \bullet \frac{\partial g}{\partial \sigma_\theta}} \quad \textbf{(5.3)}$$

Where

$g(\sigma_r, \sigma_\theta, \eta)$: Plastic potential function

$\lambda\bullet$: Plastic multiplier

Plastic strain increments are controlled by a fractional time factor (τ) and calculated as:

$$\boldsymbol{\varepsilon_r^p \bullet = \frac{\partial \varepsilon_r^p}{\partial \tau} \; and \; \varepsilon_\theta^p \bullet = \frac{\partial \varepsilon_\theta^p}{\partial \tau}} \quad \textbf{(5.4)}$$

The strain - softening behaviour model based on the Mohr-Coulomb yield criterion is based on the following equations:

$$\boldsymbol{f(\sigma_\theta, \sigma_r, \eta) = \sigma_\theta - K_\phi(\eta)\sigma_r - 2C(\eta)\sqrt{K_\phi(\eta)}} \quad \textbf{(5.5)}$$

The potential plastic state is defined by:

$$\boldsymbol{g(\sigma_\theta, \sigma_r, \eta) = \sigma_\theta - K_\psi \sigma_r} \quad \textbf{(5.6)}$$

Where:

K_ψ: Dilation coefficient

$$K_{\psi} = \frac{1+sin\psi}{1-sin\psi} \quad \textbf{(5.7)}$$

Figure 5.11 shows linear plastic parameters, including cohesion and friction angle functions of strain-softening behaviour, related to the Mohr-Coulomb yield criterion. The parameters are:

$C(\eta)$: Cohesion function, $\phi(\eta)$: Friction angle, ϕ^{peak}: Friction peak, C^{peak}: Cohesion peak, ϕ^{res}: Friction residual, and c^{res}: Cohesion residual

The plastic parameter is calculated by (Alejano et al., 2009):

$$\boldsymbol{\eta = \varepsilon_1^p - \varepsilon_3^p = \varepsilon_\theta^p - \varepsilon_r^p = \gamma^p} \quad \textbf{(5.8)}$$

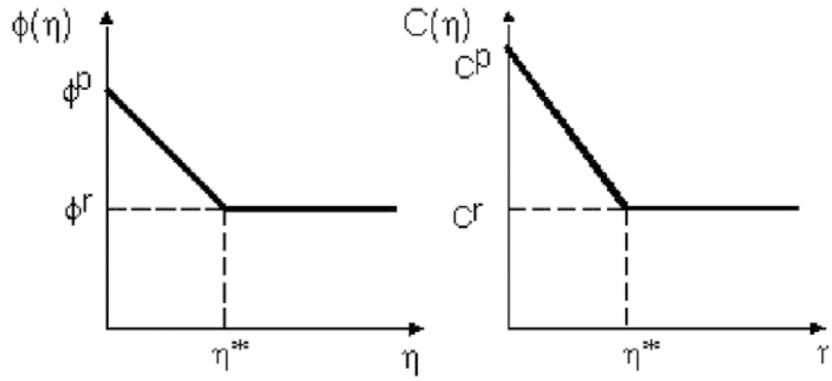


Figure 5.11. Linear plastic parameters in Mohr-Coulomb strain softening (Alejano et al., 2009)

Strain-softening behaviour of material based on the Hoek-Brown failure criterion used in a wide range of rock engineering projects is calculated by (Alejano et al., 2010):

$$\boldsymbol{f(\sigma_\theta, \sigma_r, \eta) = \sigma_\theta - \sigma_r - \sqrt{m(\eta)\sigma_r\sigma_{ci} + s(\sigma)\sigma_{ci}^2}} \quad \textbf{(5.9)}$$

Where: $g(\sigma_\theta, \sigma_r, \eta)$: Plastic potential function, σ_{ci}: Intact compressive strength, η: Plastic parameter, ψ: Dilatancy, $m(\eta), s(\sigma)$: Material parameters

$$\boldsymbol{g(\sigma_\theta, \sigma_r, \eta) = \sigma_\theta - K(\eta)\sigma_r} \quad \textbf{(5.10)}$$

$$K_{\psi} = \frac{1+sin\psi(\eta)}{1-sin\psi(\eta)} \quad \textbf{(5.11)}$$

$$\eta = \gamma^p = \varepsilon_\theta^p - \varepsilon_r^p$$

$$\boldsymbol{m(\eta)} = \begin{cases} \boldsymbol{m^{peak} - \frac{m^{peak} - m^{res}}{\eta^*}\eta,} & \boldsymbol{0 < \eta < \eta^*,} \\ \boldsymbol{m^{res}} & \boldsymbol{\eta > \eta^*,} \end{cases} \quad \textbf{(5.12)}$$

$$\boldsymbol{m(\eta)} = \begin{cases} \boldsymbol{s^{peak} - \frac{s^{peak} - s^{res}}{\eta^*}\eta,} & \boldsymbol{0 < \eta < \eta^*,} \\ \boldsymbol{s^{res}} & \boldsymbol{\eta > \eta^*,} \end{cases} \quad \textbf{(5.13)}$$

Failure mechanism of rock mass structures at great depth is illustrated in Figure 5.12. Rock mass structures surrounding underground excavations have conditions that range from stable to collapsed in six steps, which are stable, warning indicator, ground movement, initial signs of failure, secondary signs of failure, and local damage/regional failure. Rock mass behaviour and its change are not always recognisable as warnings of failure. A procedure to recognise the pre-failure of a rock mass can be useful for rock engineers in the prediction of geotechnical failure and collapse, in order to avoid a major loss. Analysis of case studies in geotechnical fields indicates that damage and progressive failure in a rock mass have warning signs as indicators and precursors (Szwedzicki, 2003). Geotechnical indicators such as faults and folds signal that a rock mass has the potential for failure, while geotechnical precursors suggest the occurrence of damage and failure. Evaluation of indicators of failure and precursors of failure play a considerable role in predicting and avoid local damage and collapse and consequently, avoid expensive operations in a mining project.

Figure 5.13 shows the effect of stress – concentration in rock mass behaviours. To begin with, the rock mass surrounding an excavation has an elastic deformation due to induced stress. Then, plastic behaviour is dominant, and stress concentration is reduced because of the increasing deformation of rock materials. Exceeding plastic deformation may lead to the initiation of failure that progresses to failure precursors as deformation intensifies. Finally, the process may cause the occurrence of the failure in the opening.

Collected data from several in-depth underground mining projects were analysed and used for evaluation of failure modes in rock mass structures, and the results are summarised in below.

The behaviour of massive/ intact rocks with concentrated stress levels is defined as:

$\boldsymbol{1 - Hard\ rock}$:

$$\rightarrow \begin{cases} \boldsymbol{Stable} & \boldsymbol{\sigma_c(MPa) \geq 100\ , \frac{\sigma_{cm}}{\sigma_{max}} \geq 5} \\ \boldsymbol{Brittle} & \boldsymbol{\sigma_c(MPa) \geq 100\ , 2 \leq \frac{\sigma_{cm}}{\sigma_{max}} \leq 5} \\ \boldsymbol{Spalling\ and\ Rock\ burst} & \boldsymbol{\sigma_c(MPa) \geq 100\ , \frac{\sigma_{cm}}{\sigma_{max}} \leq 2} \end{cases} \quad (5.14)$$

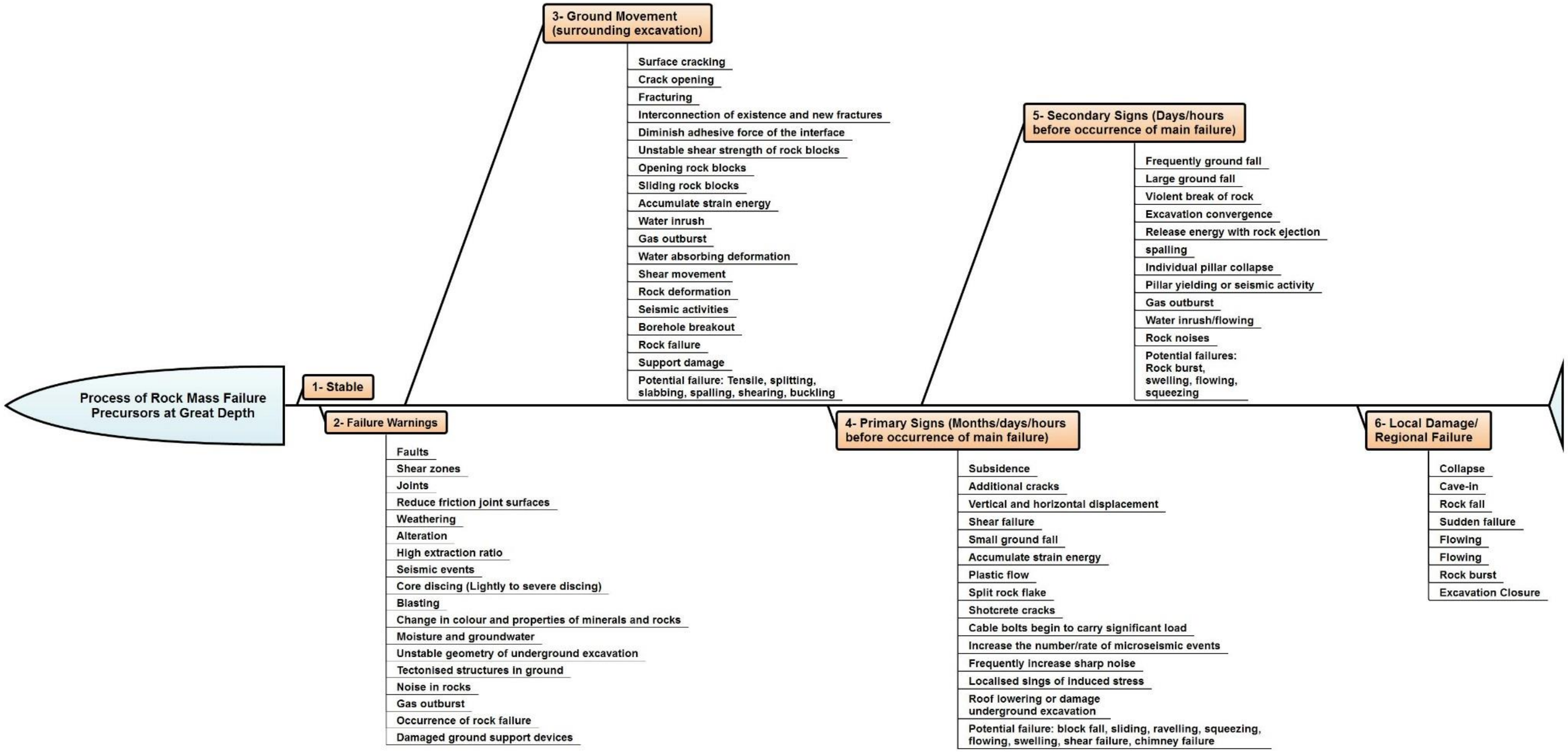


Figure 5.12. The process of rock mass failure precursors at great depth

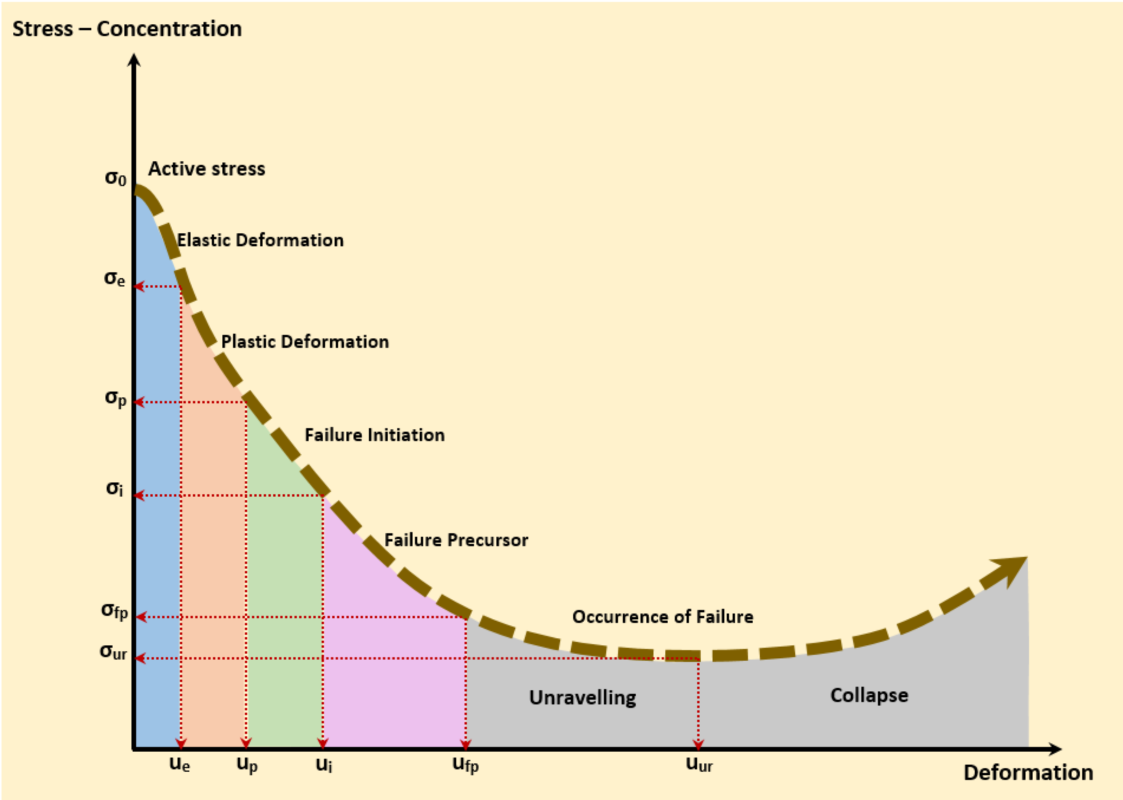


Figure 5.13. Rock mass behaviour surrounding excavation due to stress concentration during and after excavation In situ stress before excavation

$\mathbf{2 - \boldsymbol{Medium\ rock}}$:

$$\rightarrow \begin{cases} \boldsymbol{Stable/Elastic} & \mathbf{50 \leq \ \boldsymbol{\sigma_c(MPa)} \leq 100, \frac{\boldsymbol{\sigma_{cm}}}{\boldsymbol{\sigma_{max}}} \geq 5} \\ \boldsymbol{Ductile} & \mathbf{50 \leq \ \boldsymbol{\sigma_c(MPa)} \leq 100, 2 \leq \frac{\boldsymbol{\sigma_{cm}}}{\boldsymbol{\sigma_{max}}} \leq 5} \\ \boldsymbol{Failure} & \mathbf{50 \leq \ \boldsymbol{\sigma_c(MPa)} \leq 100, \frac{\boldsymbol{\sigma_{cm}}}{\boldsymbol{\sigma_{max}}} \leq 2} \end{cases} \qquad (5.15)$$

$\mathbf{3 - \boldsymbol{Soft\ rock}}$:

$$\rightarrow \begin{cases} \boldsymbol{Stable/elastic} & \mathbf{\boldsymbol{\sigma_c(MPa)} \leq 50, \frac{\boldsymbol{\sigma_{cm}}}{\boldsymbol{\sigma_{max}}} \geq 5} \\ \boldsymbol{Plastic/shearing} & \mathbf{\boldsymbol{\sigma_c(MPa)} \leq 50, 2 \leq \frac{\boldsymbol{\sigma_{cm}}}{\boldsymbol{\sigma_{max}}} \leq 5} \\ \boldsymbol{Squeezing} & \mathbf{\boldsymbol{\sigma_c(MPa)} \leq 50, \frac{\boldsymbol{\sigma_{cm}}}{\boldsymbol{\sigma_{max}}} \leq 2} \end{cases} \qquad (5.16)$$

Massive rock failure is associated with low degree jointed rock mass. The usual failure modes for medium and soft rocks are plastic deformation and squeezing under high-stress levels. Whereas for hard rock conditions, the failure modes can be brittle, slabbing, spalling and rock burst. The typical failure modes of the structural/intact condition are buckling, tensile, splitting and shearing. Buckling is more related to layered rock structures in high-stress conditions. A moderate degree jointed rock mass in medium and high-stress conditions may lead to sliding and shear failures in underground openings. Structural failure modes can be described based on the continuity factor of ground (5.17) (Palmstrom and Stille, 2015) and stress concentration as follows:

$$\boldsymbol{Continuity\ Factor(CF)} = \frac{\boldsymbol{Equvalent\ Dimension\ of\ Excvation}}{\boldsymbol{Equavalent\ Dimention\ of\ Block\ Rock}} \qquad 5\mathbf{5}.17)$$

1) For $CF > 30$, $\frac{\sigma_{cm}}{\sigma_{max}} \geq 5$; block fall, cave in, skin fall, wedge failure
2) For $3 < CF < 30$, $2 \leq \frac{\sigma_{cm}}{\sigma_{max}} < 5$; Joint or bedding plane opening and slipping, cave in, block fall
3) For $CF < 3$, $\frac{\sigma_{cm}}{\sigma_{max}} < 2$; Toppling, fault sliding, chimney type failure, unravelling, anisotropic strains (in schistose and stratified rock condition)

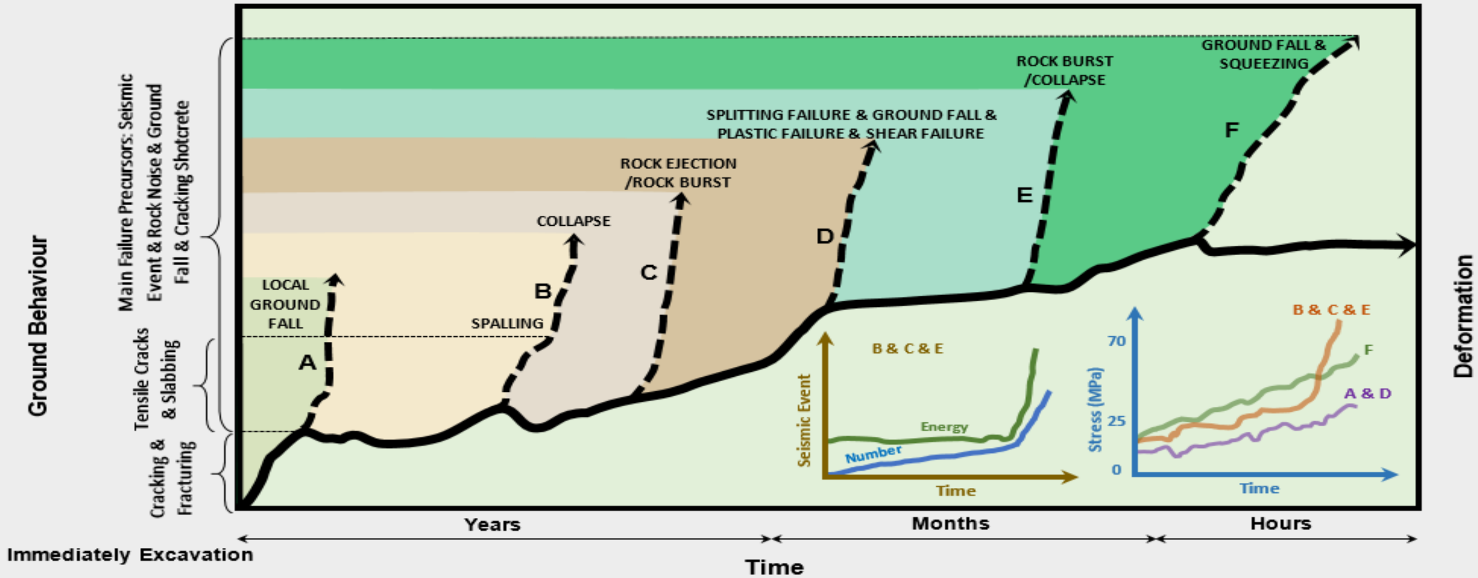
The presence of groundwater in rock mass structures causes reduction of the rock strength and friction of discontinuities. When an underground excavation is encountered with disintegrated or crushed rock mass, the presence of groundwater along discontinuities may lead to flowing ground. Also, minerals with special properties such as clay and montmorillonite absorb water and lead to the reduced shear strength of rock material, and consequently may lead to swelling phenomena.

Figure 5.14 presents a combination of different types of ground behaviour and failure modes against time, during and after processing of the excavation stage. The figure is the result of site monitoring, geotechnical site investigation, interpretations and engineering judgements from several underground excavation projects. According to the figure, cracking and fracturing happened after completing excavation, due to high-stress conditions. Intersecting major joint sets appeared at the excavation surface in jointed/blocky rock structures. Therefore, ground fall and wedge/sliding failure will occur in an underground excavation. In situ and induced stresses caused slabbing and spalling in the rock structures. Seismic events and the small ground fall on the wall, and side crown in an excavation was recorded before the occurrence of large-scale failure and collapse during the period of years to hours before the collapse. Spalling and rock burst failure frequently occurred during excavation and after installation of a ground support system in underground openings. The failures were accompanied by cracking, fracturing, increased rate/number of seismic events, and sometimes local ground falls before a sudden release of energy and ejection of rock. The primary failure precursors in rock mass structures were noticed years, months and even hours before the final failure/collapse. Typical failure modes in underground excavations with weak geological structures were a tensile failure, block fall, and shear/slip failure, plastic squeezing, and structural stress induced failure/collapse. Plastic squeezing failure is associated with time-dependent ground behaviour. Failure precursors were completed by progressive deformation under long-term stress conditions in the rock mass structures containing various defects such as discontinuities, veins, shear zones and faults.

The flowchart for diagnosing the ground behaviour and failure mechanism in rock mass structures surrounding deep underground excavations is given the name Flowchart GB-FM and is shown in Figure 5.15. The flowchart was developed by geotechnical site observation, monitoring, engineering judgement and empirical methods based on active stress (σ_{cm}/σ_1) and ground continuity factor (CF = Equivalent diameter of underground opening/ rock block diameter). The continuity factor of rock mass structure was divided into five class based on continuous and discontinuous ground behaviour according to Palmstrom and Stille (2015):

1) CF < 6
2) 6 < CF < 20
3) 20 < CF < 50
4) 50 < CF < 100
5) CF > 100

Ground condition is stable for active stress greater than 5 and continuity factor less than 6. At active stress between 2 and 5, failure mechanism is estimated as plastic, splitting, shearing and ground fall. Jointed rock structures, weakness zones and relatively high-stress condition affect happening failures, which is mostly accompanied by fracturing and tensile cracking, shear factoring and plastic flow surrounding underground excavation.



Description:

A. Appear intersecting discontinuities (immediate excavation) → loading/unloading ground (from a few hours to years) → incremental loosening and distorting major joints → reduce friction joint surfaces → unstable shear strength of blocks → opening rock blocks → sliding blocks / ground fall (from a few hours to years)

B & C & E. Initiation of cracks (immediate excavation) → propagation fractures → tensile cracking / extension parallel slabbing (from a few days to years) → accumulate strain energy → split rock flake → shotcrete cracks/cable bolts begin to carry significant load (from a few hours to a few days) → small ground fall → increase the number / rate of microseismic events → frequently increase sharp noise (hours – days before occurring failure) → large ground fall / violent break of rock → release energy with rock ejection / spalling / rock burst (from a few minutes to hours, and 5 – 30m far from stopes) → collapse

D. Appealing cracks on the surface → fracturing wall and crown of excavation → ground movement → joint dilation → shear fractures / tensile fractures → extrusion of joint infilling → excavation convergence → local ground fall → failure / collapse (from a few hours to years)

F. Generation new cracks → growth fractures → fracturing at ground surface of excavation → interconnection of existence and new fractures → diminish adhesive force on the interface → ground movement → plastic flow → additional cracks → vertical and horizontal displacement → shear failure → excavation convergence → large scale of ground fall → collapse (from months to years)

Figure 5.14. Ground behaviour modes and failure mechanisms over time at great depth

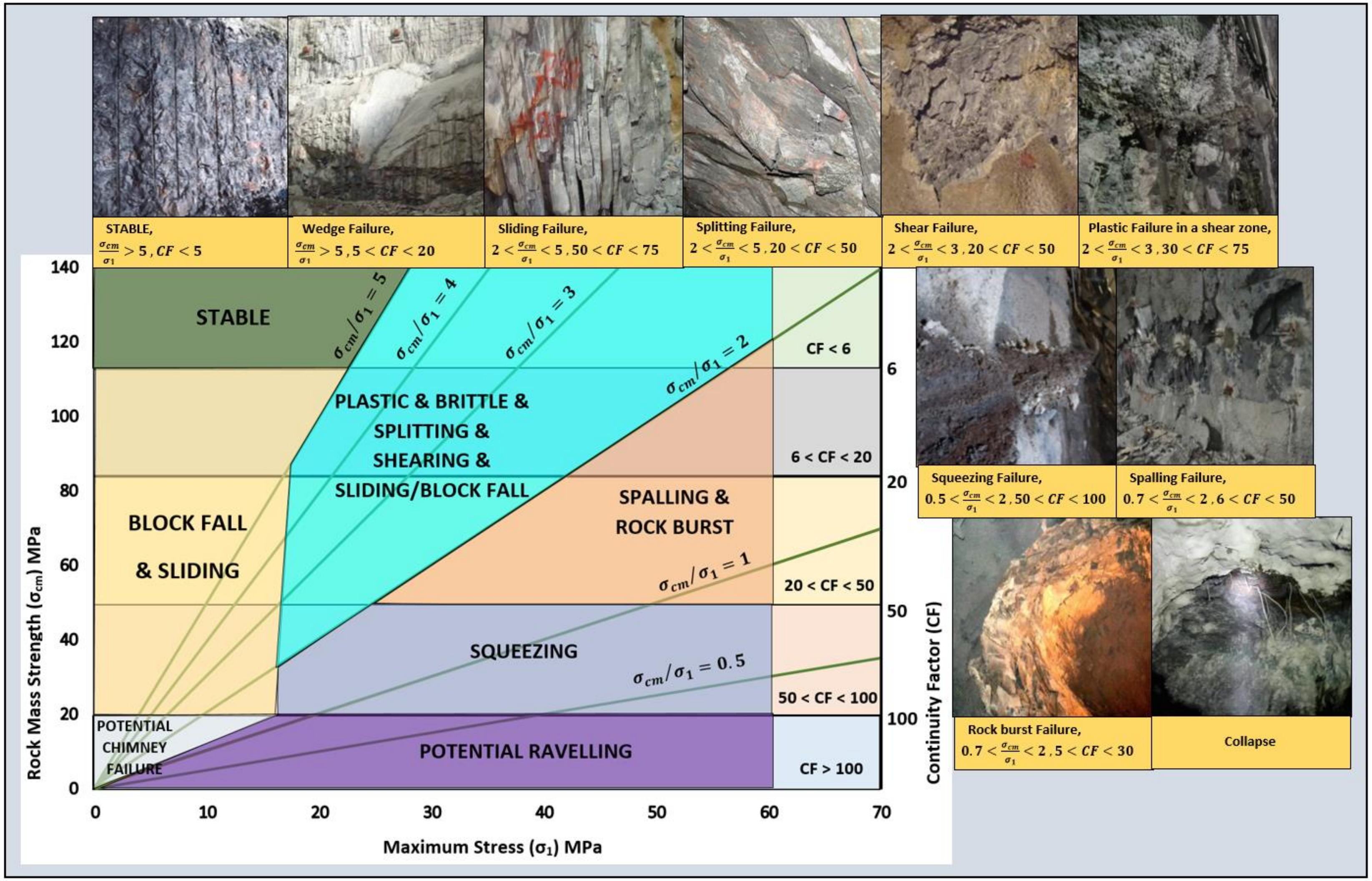


Figure 5.15. Diagnosis of ground behaviour and failure mechanism according to rock mass strength, maximum principal stress and continuity factor (Flowchart GB-FM)

Shear failure is initially deformed in combination with discontinuities and high-stress condition. The rocks consist of a high percentage of clay minerals are more potential for happening this type of failure. Block falls are controlled by the direction and the number of sets in the ground condition. The shape of rock blocks, derived from discontinuities, lead to the severity of ground fall and sliding failure. Brittle failure occurs in hard rock and gradually break into pieces and fragments rocks. Brittle failure in high stress and the seismic condition can be a type of rock burst failure. Moderate to high level of field stress in the massive and blocky rocks (CF < 30) make a sudden and violent rock slab from sides and roof in underground excavations. Rock burst failure is a sudden and violent failure of hard, massive and blocky rock, high-stress condition and seismic events. Squeezing behaviour is a type of time-dependent behaviour in the moderate to hard rocks with the high-stress condition. Squeezing ground observed a type of blocky and folded classes typically. Weak interlayer zones in hard rock and overstress lead to squeezing failure during or after construction period. The percentage of clay minerals and groundwater condition influence the rate of deformation in squeezing behaviour. A high degree of jointed rocks usually has a low scale of intact rock strength. Disintegration, brecciated and foliated rock masses with low cohesion may cause rock mass ravelling. The type of infilling material of discontinuities, groundwater condition and stress level have an impact on the severity of ravelling ground.

In deep underground mining with hard rock conditions, in some cases only one failure mechanism is dominant, but in many cases, a failure may start with one mechanism and then followed by other mechanism or combination of other mechanisms. This makes the design more complicated but should be considered in design and construction. Typically, failure mechanisms at great depth could be classified into three groups: structural failure, induced stress/seismic failure, and operational failure mechanism. Structural failure usually occurs in moderate to high jointed rocks or rock masses contains interlayer shear/fault weakness zones. Large deformation is a common ground behaviour mode in highly jointed rocks. Mining-induced stress and seismic events are the major failure modes at great depth, which usually occur as a kind of sudden failure. Generally, seismic events with the magnitude more than 1ML, beside of the rock failure due to release absorbed energy in rocks can cause damage in ground support devices too. Operational failure at great depth is associated with a rock damage zone surrounding mining excavations due to blasting effectiveness. However, rock failure,in reality, is a combination of structure, stress and operational effects. Here, the dominant factors of induced rock failures are discussed.

5.3.1. Structural failure mechanism

Structural failure is related to major geological structures and discontinuities in rock masses. The intersection of discontinuities create rock blocks and may lead to failure due to gravity and sliding between discontinuity surfaces. The strength properties of discontinuity surfaces between blocks govern initiation of failure in rock masses. In a highly jointed and disintegrated rock mass, rock failure is usually initiated in a part of the rock mass with low cohesion and friction properties. The thickness, properties of infilling material of discontinuity surfaces, and their shear strength are critical parameters to assess structural failure derived from gravity or sliding. Figure 5.16 shows some typical structural failures in deep underground mines.

Structural failure mechanisms in deep underground mining are presented in

Figure 5.17. Scat failure occurs in areas with no or inadequate support system. The mechanism of ground fall/wedge failure/sliding failure is explained as:

- Opening of joints or discontinuities
- Sliding and moving of rock blocks along one or more undesirable directions of the discontinuity planes
- Mostly occurs where there is a rapid change in strength properties of discontinuity surfaces due to temperature, groundwater, induces stress, and mining operations

Instability of blocks in rock masses occurs when the strength along discontinuity surfaces such as joints and bedding surfaces cannot hold the rock blocks and, therefore, failure happens as blocks slide or block falls around excavations (Blyth and Freitas, 1984). Increased tension or reduction of compression stresses in discontinuity surfaces leads to reduced friction between surfaces. As a result, the shear strength between rock blocks is unable to provide a stable condition and rock falls occur in rock masses surrounding underground openings.

Jointed/blocky/bedded rocks with moderate to good quality rocks may lead to ground fall, wedge failure, sliding failure and scat failure. A rock mass with a high degree of jointing and with low-quality discontinuity surfaces and boundaries (low friction and low cohesion) cause chimney failure and ravelling failure. Stope failure is associated with failure in large span underground stopes during the extraction of minerals and development of paste materials in the stopes.

Corrosion failure usually occurs in old areas of underground mines where ground support devices have been corroded, or rock structures have been altered/weathered. Structural failure of support devices can happen in this situation. The existence of weakness zones such as faults and shear zones with weak properties can lead to a collapse at the free face in underground excavations.

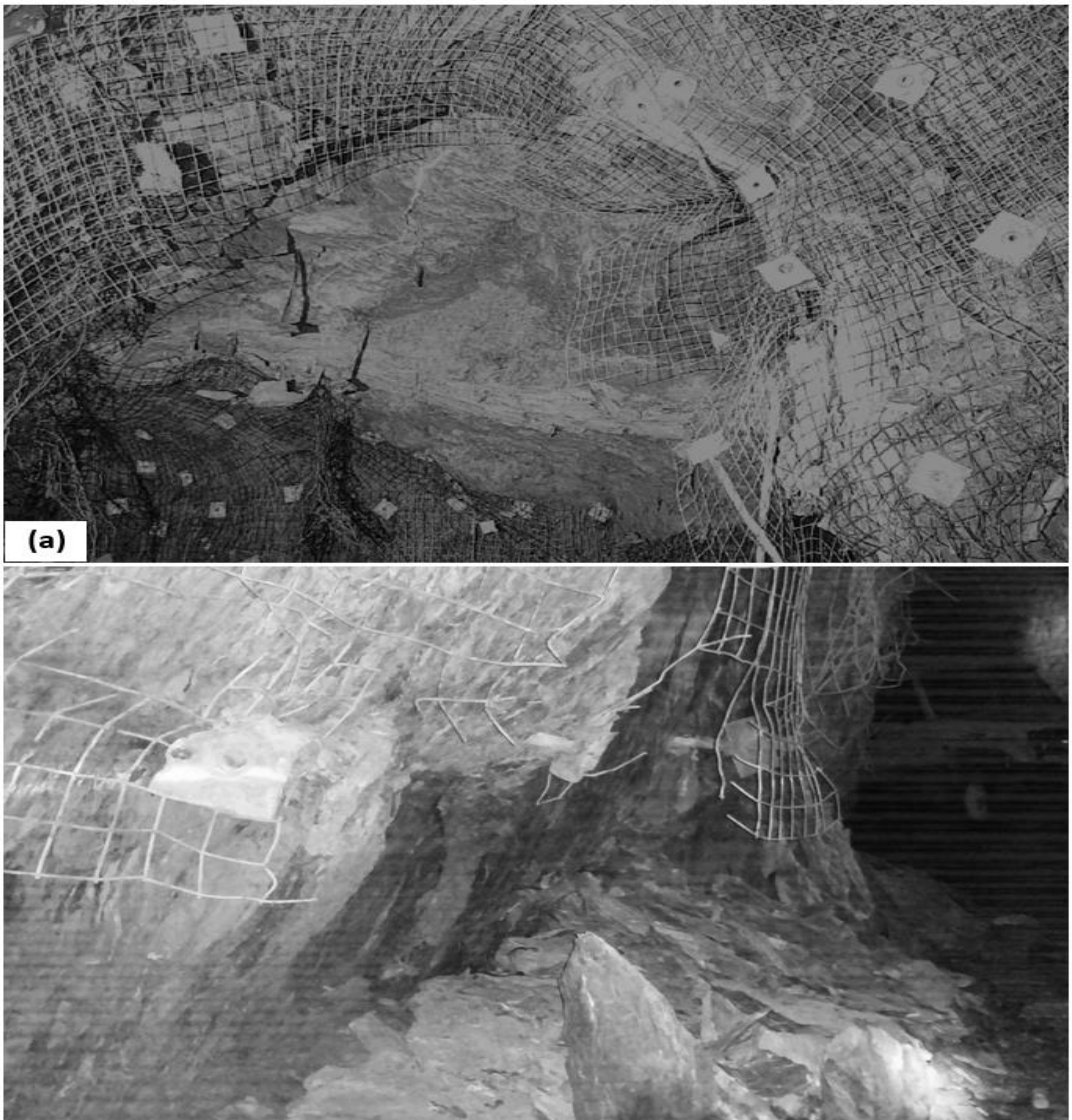


Figure 5.16. Structural failures: (a) ground fall, (b) sidewall fall

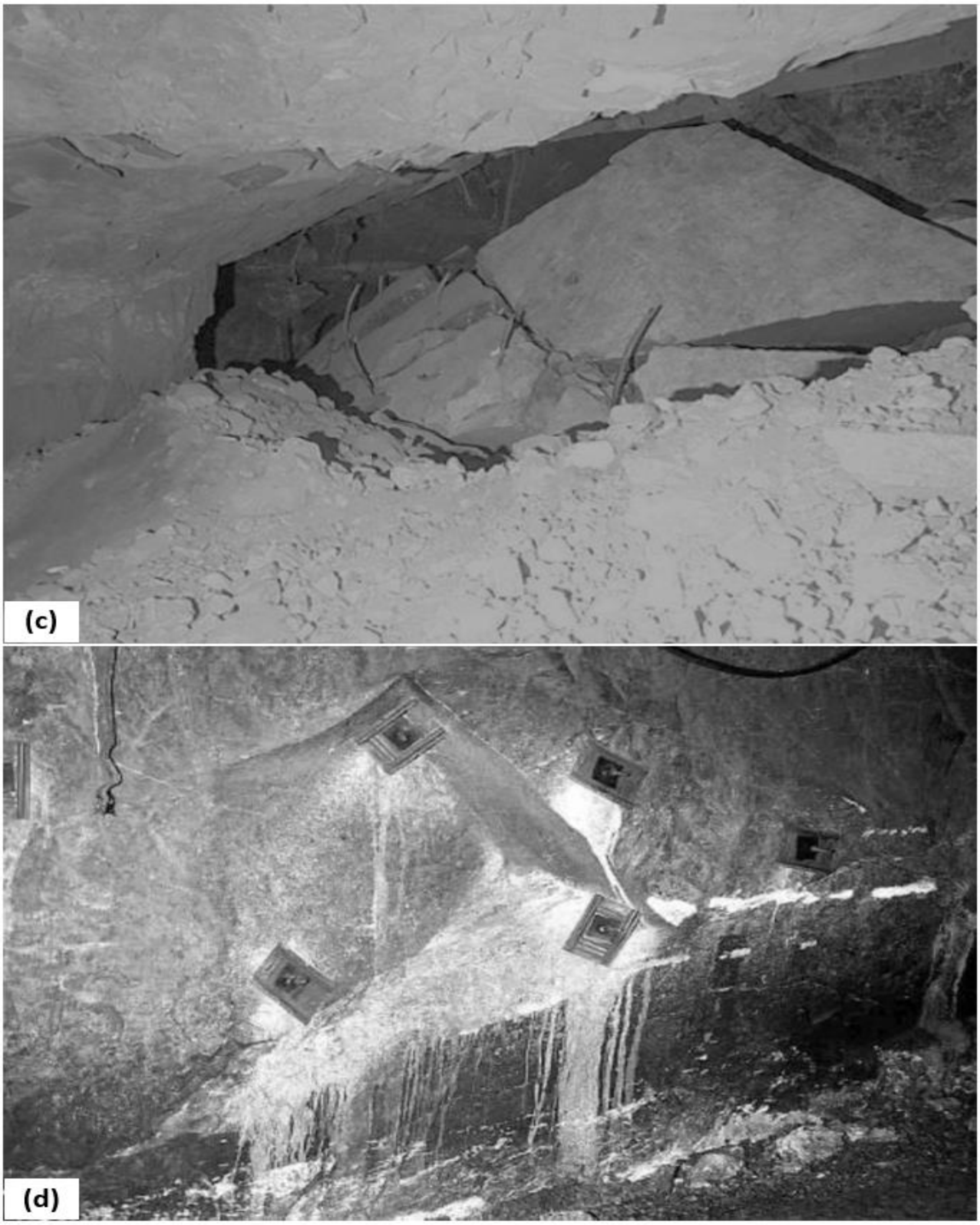


Figure 5.16. Structural failures: (c) block fall, (d) wedge failure

The most affected area for structural failure mechanisms on the flowchart GB-FM is presented in Figure 5.18. According to the figure, structural failure usually occurs in continuous and discontinuous conditions with low-stress conditions. However, the dominant factor on failure mechanisms in heavily broken rock is mostly controlled by pieces of rock materials, and high-stress conditions influence the severity of rock failure.

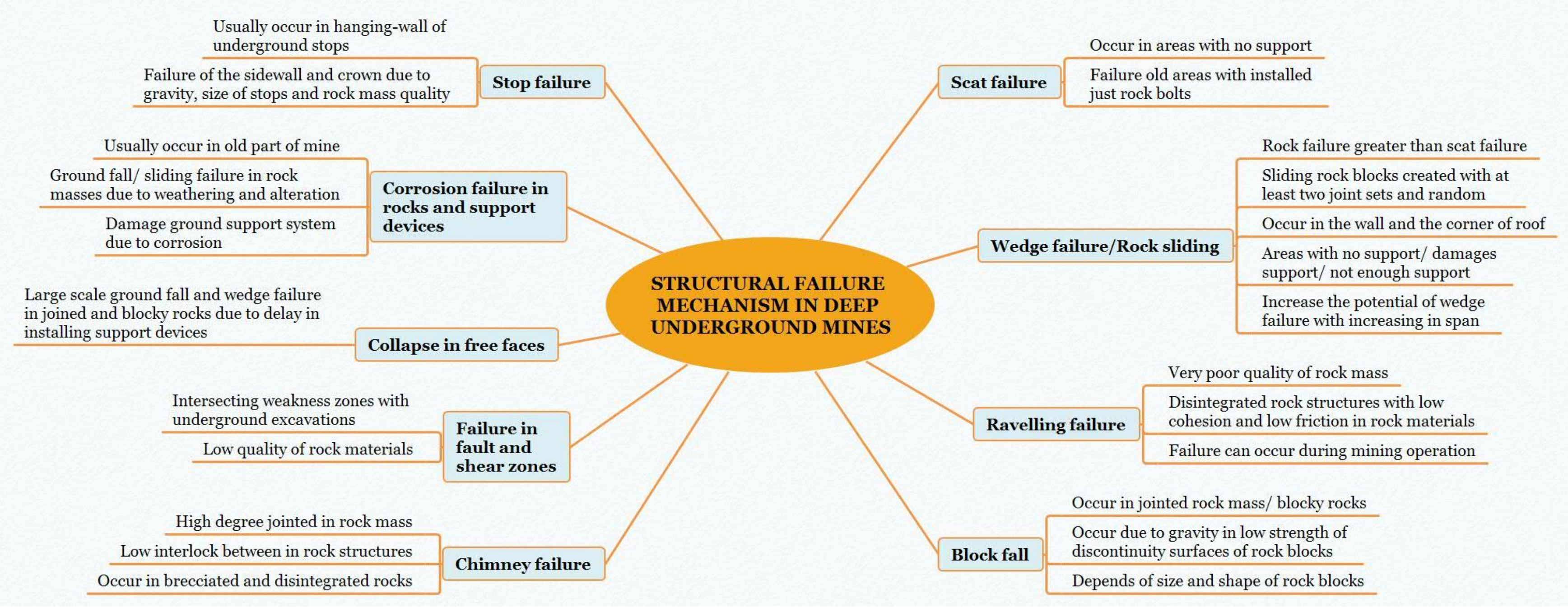


Figure 5.17. Structural failure mechanisms in deep underground mines

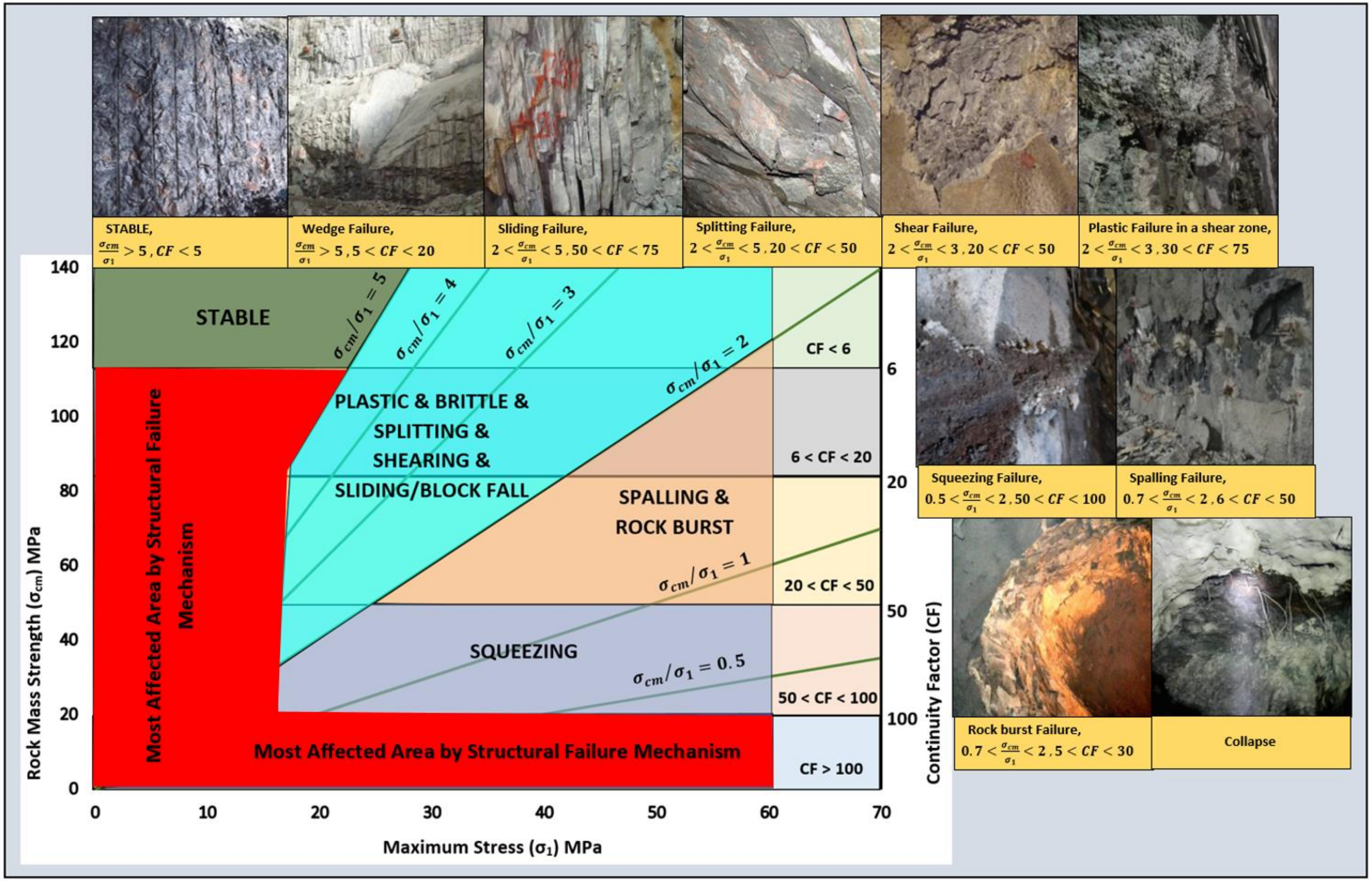


Figure 5.18. Most affected area based on structural failure mechanism in the Flowchart GB-FM

5.3.2. Induced stress/seismic failure mechanism

Stress influences rock properties and behaviour in a variety of ground conditions. Figure 5.19 illustrates the relation between rock behaviour and stress in different geological structures. Stress state, lithology, temperature, rock properties, discontinuities and groundwater are associated with deformation and failure occurrence. Figure 5.19 (a) is a schematic of isotropic and homogenous rock, which deform due to the stress condition. In blocks (b) and (c), tensile fractures in the direction of the principal stress may lead to brittle deformation or brittle fracture. In blocks (d) and (e), deformation of rock progresses from a brittle to ductile state. Block (f) shows ductile deformation of a rock mass under anisotropy conditions. Stress condition and loading change a rock's properties from homogenous to anisotropic because of deformation and fracturing in the rock.

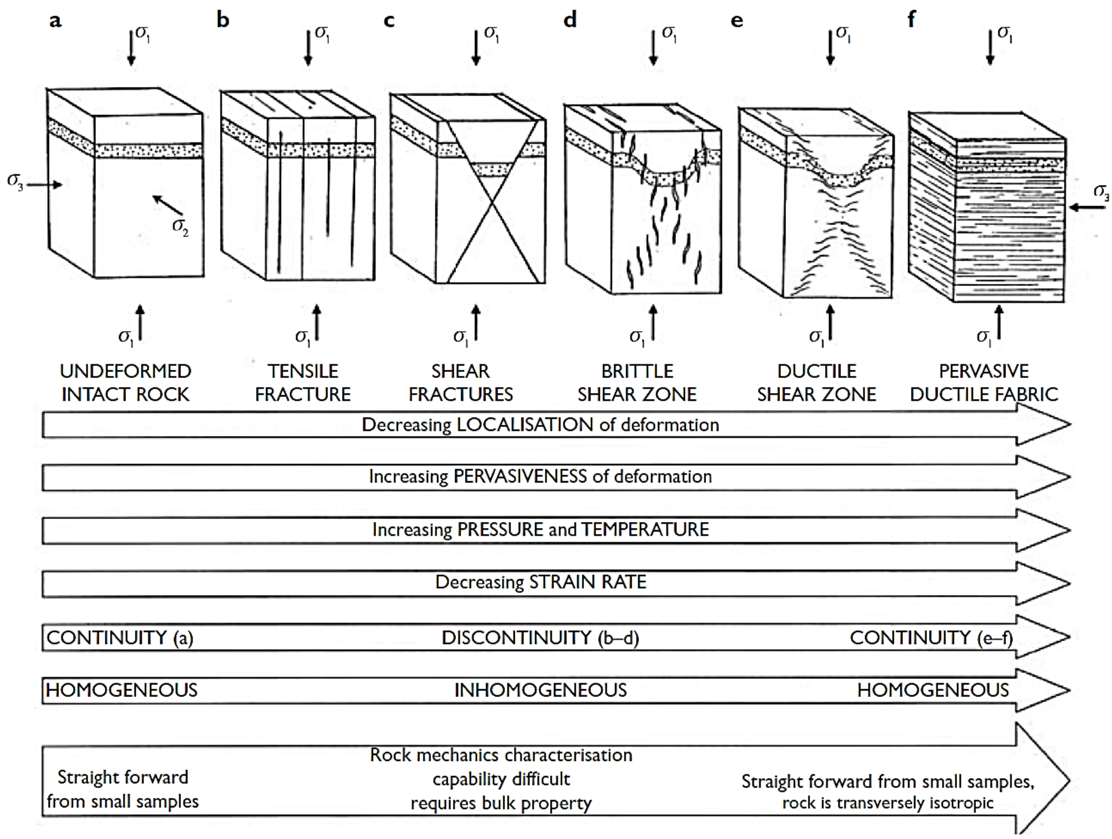


Figure 5.19. The behaviour of rock structure and stress in different geological condition (Hudson and Feng, 2015)

Failure modes differ in hard rock compared to soft rock, and also at shallow depth compared with great depth. Rock failure is due to the creation, growth, accumulation and interaction of microcracks. Microcracks inside rocks are initiated by excessive stress levels at depth and behaviour of a rock mass during the failure process changes from a continuum condition (intact to moderately fractured) to a discontinuous state, such as slabbing or spalling ground mode (Rahimi and Sharifzadeh, 2017). Experience indicates that initiation of rock failure can occur in a stress condition within the range of 30%-50% of the uniaxial compressive strength of the intact rock (Kaiser et al., 1996). For less than 25% of the strength, rock is stable under static conditions, while a failure occurs during an excavation stage where the stress is more than 50% of the intact rock strength.

In high-stress conditions, brittle spalling and slabbing failure is more likely. Intersecting rock discontinuities create rock blocks, and where there is a low interlocking, there is a probable dislocation or rotation of individual blocks in a rock structure. Massive rock with a few joints is more related to ductile behaviour for low or moderate ranges of strength. Brittle failure occurs in anisotropic rock mass structures like folded, bedded or laminated rock. Figure 5.20 presents typical relationships of induced stress/seismic failure mechanisms in deep underground mines. Spalling behaviour is associated with deep underground excavations, hard rock and high-stress conditions. Extension fractures and deformation in spalling failure can lead to rock burst (Diederichs, 2007). Thin cracks and slabs in spalling behaviour are due to the release of energy from seismic events and strain burst. Figure 5.21 shows an example of spalling failure in an underground excavation. The local induced stresses and seismic events caused spalling zones and moderate brittle failure. Mechanical behaviour of spalling is intrinsically brittle with rock strength reduced across the cracks and fractures. Increased dynamic loading causes the expansion of fractures and depth of parallel slabs in spalling failure. Meanwhile, fractures in the spalling process are parallel to a maximum compressive stress direction.

The depth of failure in brittle rock conditions in underground excavations is calculated by the following empirical equation (Liu et al., 2017):

$$\frac{\boldsymbol{D_f}}{\boldsymbol{a}} = \mathbf{1.25}\frac{\boldsymbol{\sigma_{max}}}{\boldsymbol{\sigma_c}} - \mathbf{0.51}(\pm\mathbf{0.1}) \qquad \textbf{(5.18)}$$

Where, D_f: Failure depth, a: tunnel radius/ effective radius, σ_{max}: maximum tangential stress, σ_c: uniaxial compressive strength.

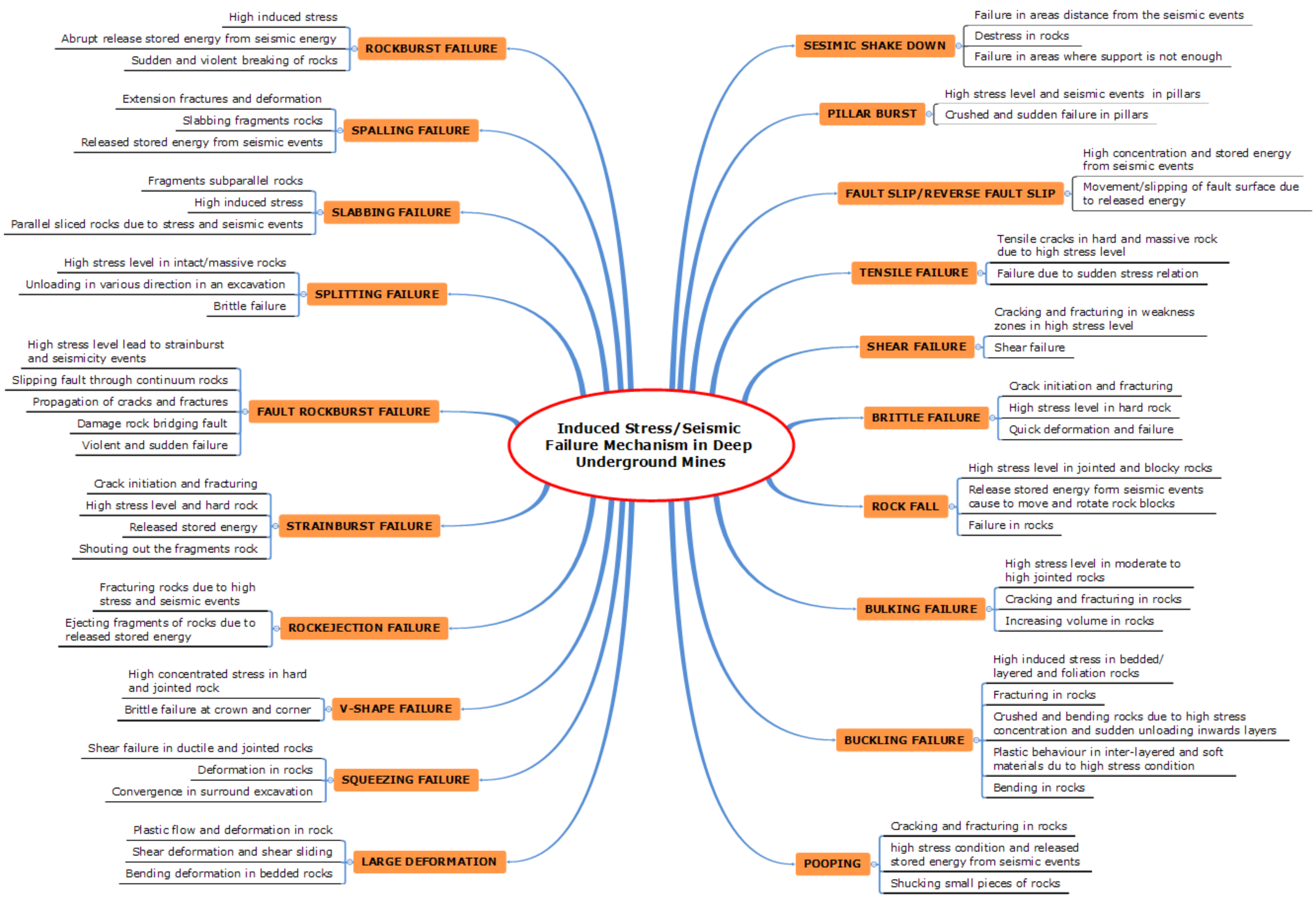


Figure 5.20. Induced stress/seismicity failure mechanism in deep underground mines

Figure 5.21. An example of spalling failure in deep underground excavation

Rockburst failure is the sudden or violent breaking of rocks due to seismic sources. The stress usually exceeds the rock mass strength surrounding underground excavations, and if the strain energy is not dissipated, sudden failure may occur.

Various factors affect the severity of rockburst damage, like stress level, rock strength, geological structures, depth, opening geometry, the lateral extent of rock around an opening, seismic-induced stress, ground support efficiency, and stiffness of mines or openings. Damage levels under rockburst failure are divided into minor, moderate and major damage. Figure 5.22 shows a classification of rockburst damage in an underground excavation.

Minor damage is usually defined as a shallow skin of fracture and spalling. The depth of damage is less than 0.25 m, and the weight of failing rock in this situation is less than 7 kN/m^2 (Cai and Kaiser, 2018). This type of damage frequently occurs in a high-stress condition, during the initial development of an underground opening at great depth, moderately jointed rock and far from seismic sources. The thickness of damage under moderate level varies between 0.25 m and 0.75 m. In this situation, the rock mass is heavily jointed. Support elements such as shotcrete may be fractured, and rock bolts may be compromised. Deep fracturing and loosened rock (more than 0.75 m) in underground excavations are defined as major damage. This phenomenon mainly leads to damaged or broken ground support elements. Some examples of rockburst failure in deep underground mines are shown in Figure 5.23.

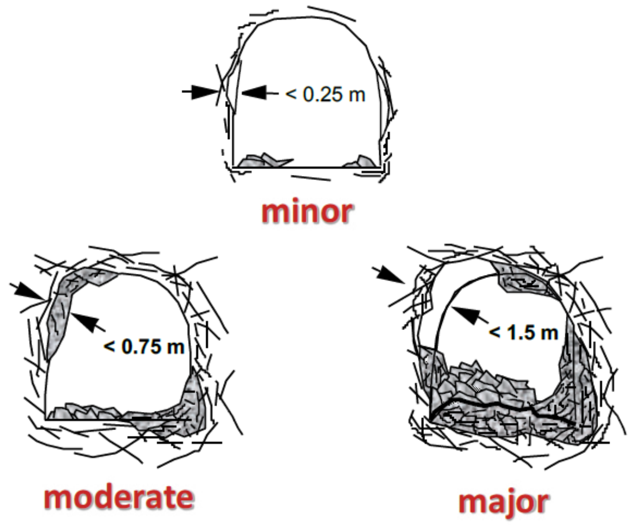


Figure 5.22. The intensity of rock burst failure (Cai and Kaiser, 2018)

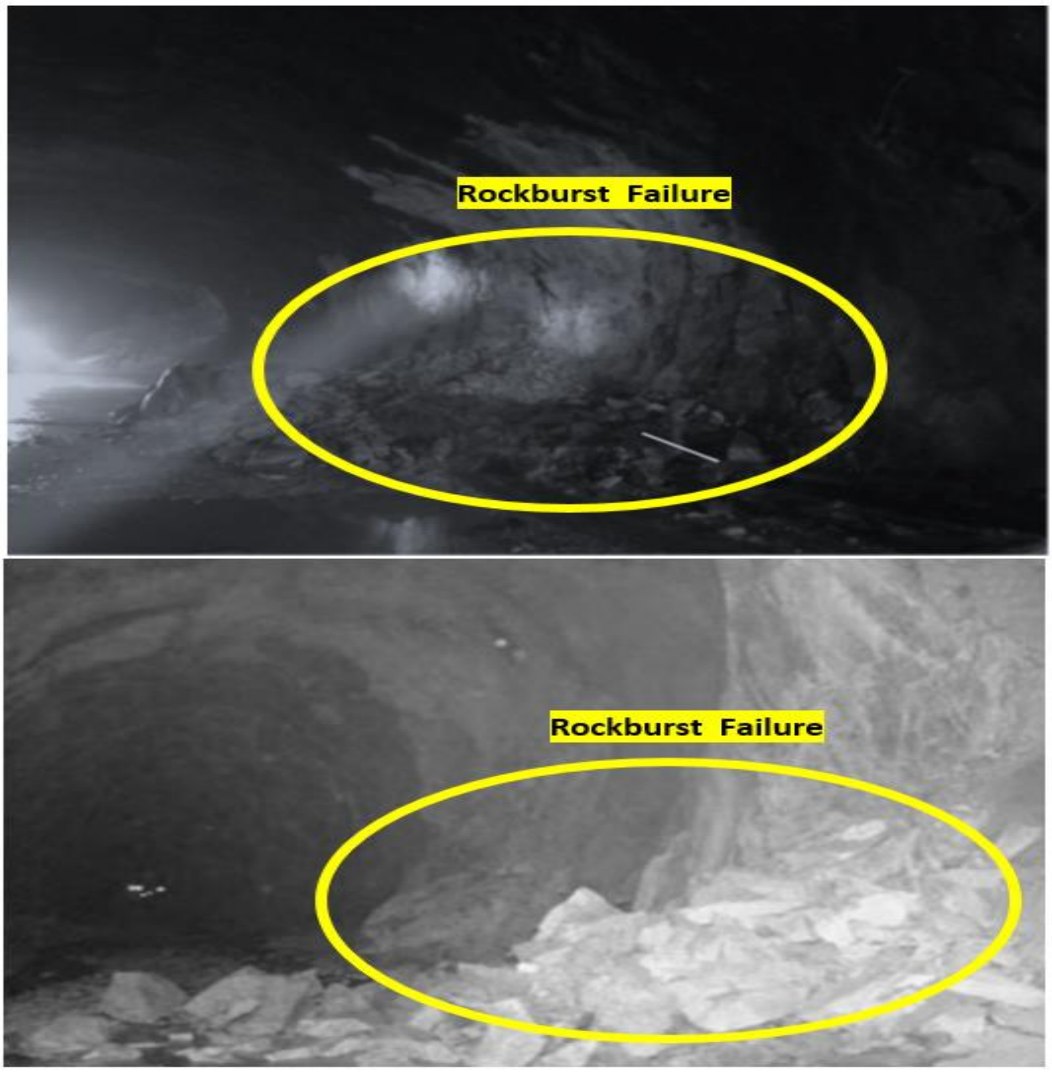


Figure 5.23. Rockburst failure in deep underground excavations

Different types of damage mechanisms in rockburst conditions have been summarised in Table 5.1. The main reasons for rockburst damage are stored strain energy, high-stress levels, seismic energy, jointed rock mass, and insufficient strength of the rock mass which may lead to bulking, rock ejection and rock fall failure. Also, the weight of failed rock per square metre of surface (kN/m2) in an underground opening has been estimated for minor, moderate and significant damage severity. Damage mechanism and severity, thickness, closure and kinetic energy provide information to design and apply desired ground support elements.

Table 5.1. Damage mechanism types in rockburst ground condition (Kaiser et al., 1996)

Damage mechanism	Damage severity	Cause of rockburst damage	Thickness [m]	Weight [kN/m^2]	Closure* [mm]	v_e [m/s]	Energy [kJ/m^2]
Bulking without ejection	Minor	highly stressed rock	< 0.25	< 7	15	< 1.5	not critical
	Moderate	with little excess	< 0.75	< 20	30	< 1.5	not critical
	Major	stored strain energy	< 1.5	< 50	60	< 1.5	not critical
Bulking causing ejection	Minor	highly stressed rock	< 0.25	< 7	50	1.5 to 3	not critical
	Moderate	with significant	< 0.75	< 20	150	1.5 to 3	2 to 10
	Major	excess strain energy	< 1.5	< 50	300	1.5 to 3	5 to 25
Ejection by remote seismic event	Minor	seismic energy	< 0.25	< 7	< 150	> 3	3 to 10
	Moderate	transfer to	< 0.75	< 20	< 300	> 3	10 to 20
	Major	jointed or broken rock	< 1.5	< 50	> 300	> 3	20 to 50
Rockfall	Minor	inadequate strength,	< 0.25	$< 7g/(a+g)$	na	na	na
	Moderate	forces increased	< 0.75	$<20g/(a+g)$	na	na	na
	Major	by seismic acceleration	< 1.5	$<50g/(a+g)$	na	na	na

v_e is the velocity of displaced or ejected rock; a and g are seismic and gravitational accelerations
* closure expected with an effective support system

Fault burst mechanisms are accompanied with released energy and lead to rockburst damage in deep underground excavations. Figure 5.24 shows a conceptual mechanism of rockburst failure in fault slips in geological structures. High-stress level conditions lead to strain burst and seismic events in geological structures in underground excavations. This phenomenon causes damaged rock bridges and then the destruction of intact rock or rock mass along a fault.

Rockburst behaviour in a deep underground excavation is assessed by empirical methods, intelligent methods (for example, Artificial Neural Networks), and numerical methods. Rock mass properties, geological structures, project environment and excavation methods are some important factors to control rockburst. Intelligent methods are associated with information and data collected from previous experiences to identify rockburst failures in the ground. Numerical methods are used for evaluating stress conditions, released energy, strain bursts and ground support systems.

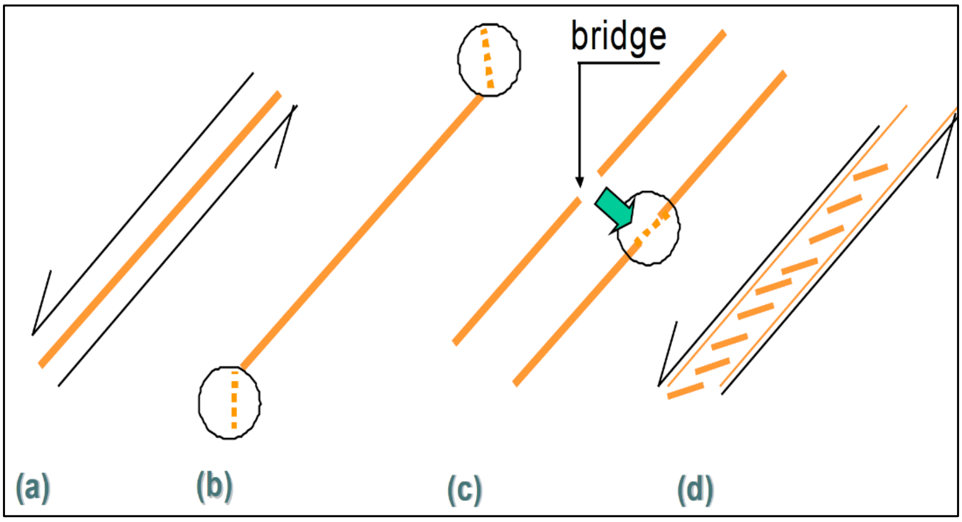


Figure 5.24. A concept of rockburst mechanism damage in fault slip: (a) slipping of fault through continuum rock (b) propagation of cracks and fractures, (c) damage of rock bridging fault, (d) en echelon failure of rock bridge along the fault (TRIFU and SUORINENI, 2009)

Two parameters, LERR (Local Energy Release Rate) and ERE (Elastic Release Energy), are used to assess potential rockburst occurrence in deep underground excavations (Hudson and Feng, 2015). The LERR index is calculated by the following equations (Hudson and Feng, 2015):

$$\boldsymbol{LERR_i = U_{i\,max} - U_{i\,min}} \qquad \textbf{(5.19)}$$

Where,

$U_{i\,max}$: Peak strain energy

$U_{i\,min}$: Minimum strain energy

$$\boldsymbol{U_{i\,max} = [\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\vartheta(\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_1\sigma_3)]/2E} \qquad \textbf{(5.20)}$$

$$\boldsymbol{U_{i\,min} = \left[\sigma'^2_1 + \sigma'^2_2 + \sigma'^2_3 - 2\vartheta(\sigma'_1\sigma'_2 + \sigma'_2\sigma'_3 + \sigma'_1\sigma'_3)\right]/2E} \qquad \textbf{(5.21)}$$

σ_1, σ_2 and σ_3: Principal stresses

ϑ: Poisson ratio

E: Young's modulus

The LERR index is employed for rockburst assessment in a deep underground opening during the excavation process under high in-situ stress states (Hudson and Feng, 2015).

The risk of failure in burst prone ground can be mitigated by three steps (Hudson and Feng, 2015):

1. Reduction of concentrated energy; by a variety of methods such as modifying shape, size and excavation method of underground openings.
2. Destressing and transferring process; This strategy is more associated with high-stress conditions and may be augmented by drilling holes and blasting. The modification of parameters such as pattern blasting and location of holes is required during the design process or mining operations.
3. Energy absorption by using yielding ground support devices.

The mechanism of rock burst failure is complex in deep underground mines. Engineering judgment, decisions based on the encountered ground condition and also using the art of rock support can help avoid failure in burst prone ground.

At great depth and high-stress conditions, seismic hazard changes with mining and excavation sequence. Assessment of seismic events and risks can be carried out by collecting data from spatial seismic event clusters, magnitude – frequency of events, history of apparent stress – time, focal mechanism, estimating peak particle velocity, and decay rates of post-firing event methods (Knobben, 2017). These methods need a lot of knowledge, experience and training in the seismic field. Table 5.2 presents a Seismic Hazard Scale (SHS) for mines of Western Australia. This parameter considers quantification of seismic events recorded in geological structures, underground stopes and mining operations like blasting, based on the rate of magnitude events and b-value parameter from the Gutenburg-Richter relation (Hudyma, 2004). SHS is applicable for reflected seismic events, up to about Richter magnitude +3, from failures of rock structures at the mine site.

Table 5.3 presents a classification for the mechanism of seismic events in underground mining projects. The severity of failure derived from seismicity depends on excavation size, geological structures, brittleness of rock and far-field PPV (Potvin & Wesseloo, 2013). Shear rupture and fault slip happen with increasing confining stress and large magnitude seismic events.

Table 5.2. Seismic hazard scale (SHS) in the mines of Western Australia (Hudyma, 2004)

Seismic Hazard Scale (SHS) defined from daily rates of seismic events and qualitative event descriptions						
		Mine seismicity frequency per day				
Qualitative description		Felt locally	Felt in a few parts of a mine, like a secondary blast	Often felt on surface, or like a development blast	Felt like a production mass blast	Detected by regional earthquake network
Approx. Richter magnitude		M_L > = -2	M_L > = -1	M_L > = 0	M_L > = +1	M_L > = +2
		Seismic hazard scale and qualitative description				
-2	Nil	>0.001 (once every few years)	0 (has never occurred)	0 (has never occurred)	0 (has never occurred)	0 (has never occurred)
-1	Very low	> 0.01 (a few times per year)	>0.001 (once every few years)	0 (has never occurred)	0 (has never occurred)	0 (has never occurred)
0	Low	>0.1 (at least weekly)	>0.01 (a few times per year)	>0.001 (once every few years)	0 (has never occurred)	0 (has never occurred)
0.5	Low to moderate	>0.3 (a few times per week)	>0.03 (monthly)	>0.003 (yearly)	<0.001 (may have happened once)	0 (has never occurred)
1	Moderate	>1 (at least daily)	>0.1 (at least weekly)	>0.01 (a few times a year)	>0.001 (once every few years	0 (has never occurred)
1.5	Moderate to high	>3 (a few a day)	>0.3 (a few times a week)	>0.03 (monthly)	>0.003 (yearly)	<0.001 (may have happened once)
2	High	>10 (more than 10 a day)	>1 (at least daily)	>0.1 (at least weekly)	>0.01 (a few times a year)	>0.001 (once every few years)
2.5	High to very high	>30 (more than 30 a day)	>3 (a few a day)	>0.3 (a few times a week)	>0.03 (monthly)	>0.003 (yearly)
3	Very high	>100 (more than 100 a day)	>10 (more than 10 a day)	>1 (at least daily)	>0.1 (at least weekly)	>0.01 (a few times ayear)
3.5	Very high to extreme	>300 (more than 300 a day)	>30 (more than 30 a day)	>3 (a few aday)	>0.3 (a few times a week)	>0.03 (monthly)
4	Extreme	>1000 (more than 1000 a day)	>100 (more than 100 aday)	>10 (more than 10 per day)	>1 (at least daily)	>0.1 (at least weekly)

Table 5.3. The seismic event category in underground mining projects (Potvin and Wesseloo, 2013)

Confining Stress	Seismic Event	Postulated Source Mechanism	First Motion from Seismic Records	Richter Magnitude M_L
Increasing confining stress	Strain-burst	Superficial spalling with violent ejection of fragments	Usually undetected, could be implosive	-0.2–0
	Buckling	Outward explosion of large slabs pre-existing parallel to surface of opening	Implosive	0–1.5
	Face crush / pillar burst	Violent explosion of rock from stope face or pillar sides	Mostly implosive, complex	1.0–2.5
	Shear rupture	Violent propagation of shear fracture through intact rock mass	Double-couple shear	2.0–3.52
	Fault-slip	Violent renewed movement on existing fault or dyke contact	Double-couple shear	2.5–5.0

According to Figure 5.20, large deformation is associated with weak rock structures and high-stress conditions. The large deformation behaviour in rock masses is separated into the following steps:

1) Plastic flow/deformation in weak rocks due to high-stress level,
2) Shear deformation due to the induced stress overcoming shear strength of rocks
3) Bending deformation in layered rocks
4) Large deformation in weak rock structures

Figure 5.25 shows the mechanisms of large deformation in rock structures. Shear stress is usually concentrated in the rock mass surrounding an excavation and causes a large deformation or squeezing behaviour.

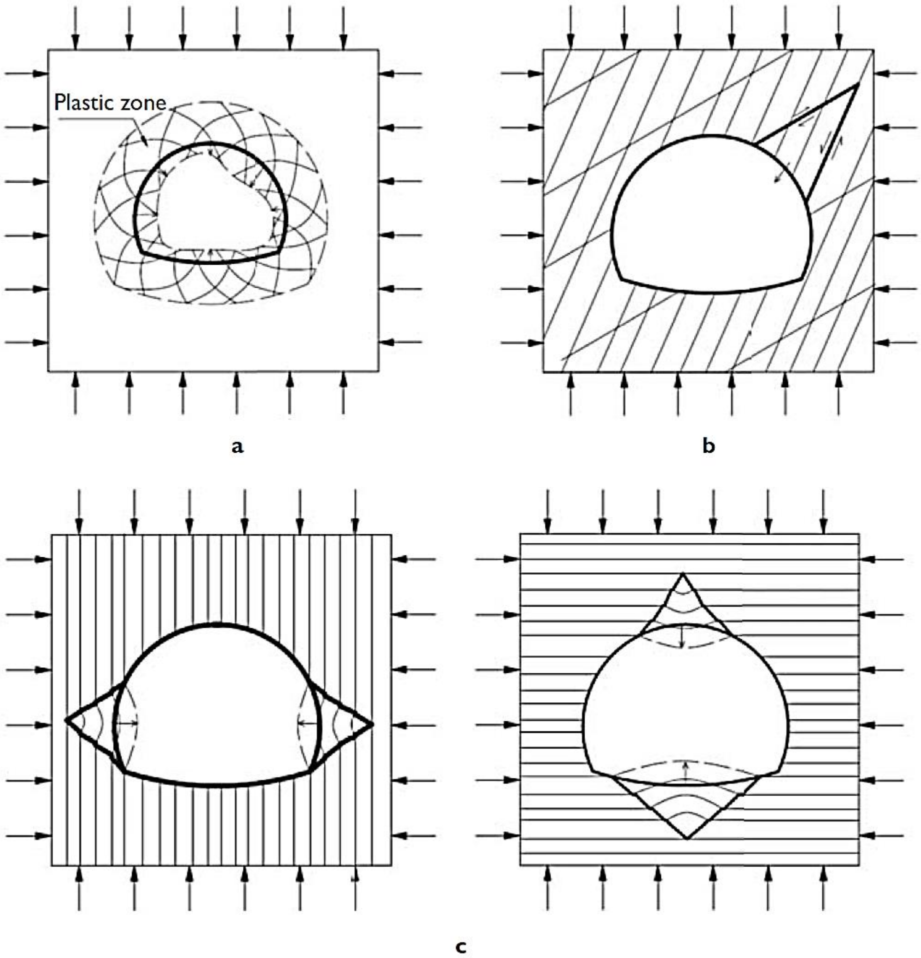


Figure 5.25. Large deformation behaviour mechanisms in rock structures: (a) plastic flow/deformation behaviour, (a) shear and sliding deformation, (c) bending deformation behaviour in layered rock strata (Hudson and Feng, 2015)

Squeezing failure is one of the typical phenomena in high-stress conditions in moderate to high jointed hard rock and medium to soft rocks. Squeezing behaviour occurs when there is a significant rock movement. This phenomenon is related to the stress exceeding the critical shear stress in a rock. The mechanisms of squeezing phenomena in hard rock are described as (Hadjigeorgiou and Karampinos, 2017):

1. Shear failure in a ductile and jointed rock mass
2. Buckling failure in rock zones consisting of thinly – layered rocks
3. Shearing of intact rocks and sliding along bedding planes in thick-bedded sedimentary rocks

High degree of fracturing, foliation, weakness and shear zones and high-stress conditions result in high deformation of rock structures and buckling failure. Figure 5.26 shows a schematic after complete shear failure and buckling failure in squeezing ground conditions. Uniform shear failure occurs in isotropic rock structures. Buckling failure mode can occur in thin layers of the rock mass. Buckling behaviour in sidewalls causes the reduction of the critical load due to dilation. Shearing failure at the top and bottom of an excavation boundary occur early during squeezing events.

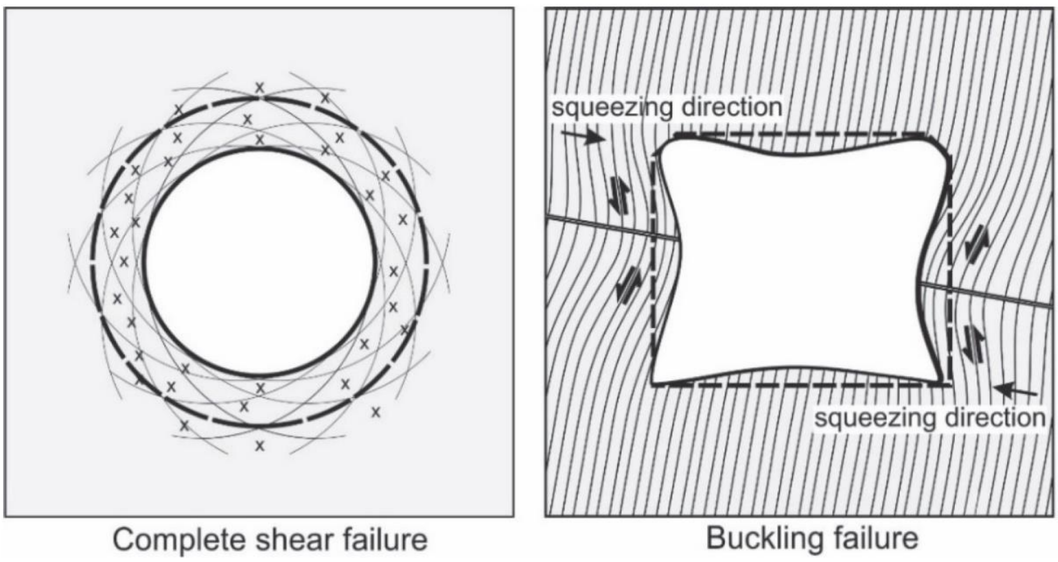


Figure 5.26. A schematic of the mechanism of squeezing behaviour by complete shear failure and buckling failure in an underground excavation (Hadjigeorgiou and Karampinos, 2017)

The potential of squeezing behaviour is identified by the Hard Rock Squeezing Index, which is based on the spacing of foliation and the stress/strength ratio and shown in Figure 5.27. The index is in the range of less than 1 percent strain for no squeezing conditions to more than 10 percent strain for extreme squeezing, which is more probable in foliation with low spacing (less than 10 mm) and stress/strength ratio of more than 0.7. Meanwhile, the orientation of foliation in underground openings control the deformation of rock mass structures. Figure 5.28 presents the squeezing index based on the angle of foliation with the excavation. Induced stress level and degree of rock alteration may result in extreme squeezing failure during excavation or even after the construction stage.

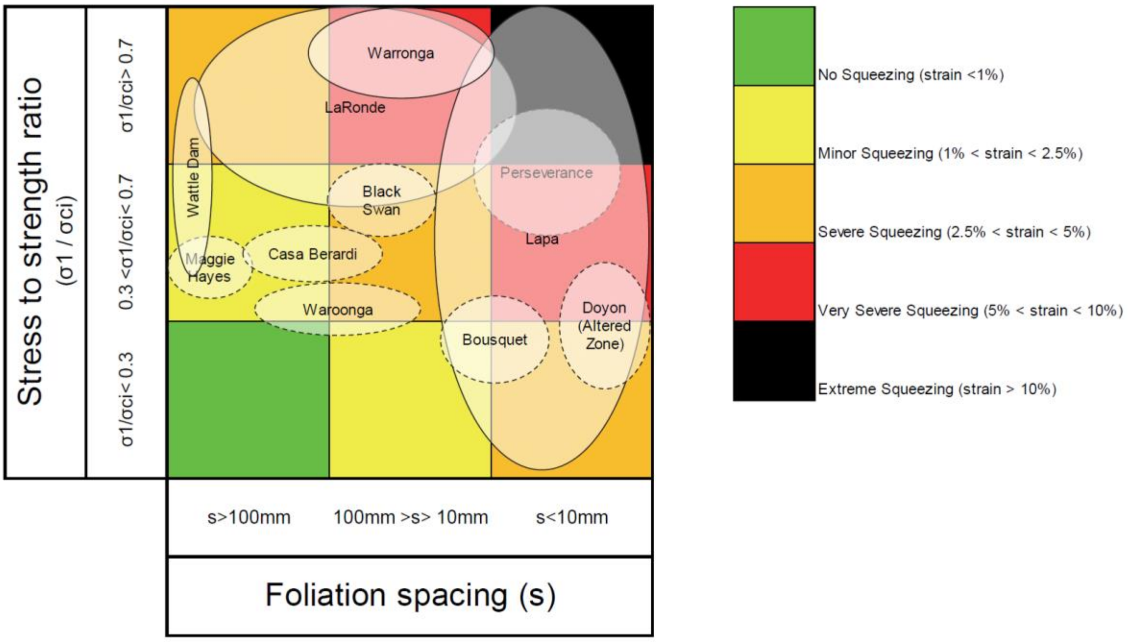


Figure 5.27. Prediction of squeezing condition based on hard rock squeezing index (Hadjigeorgiou and Karampinos, 2017)

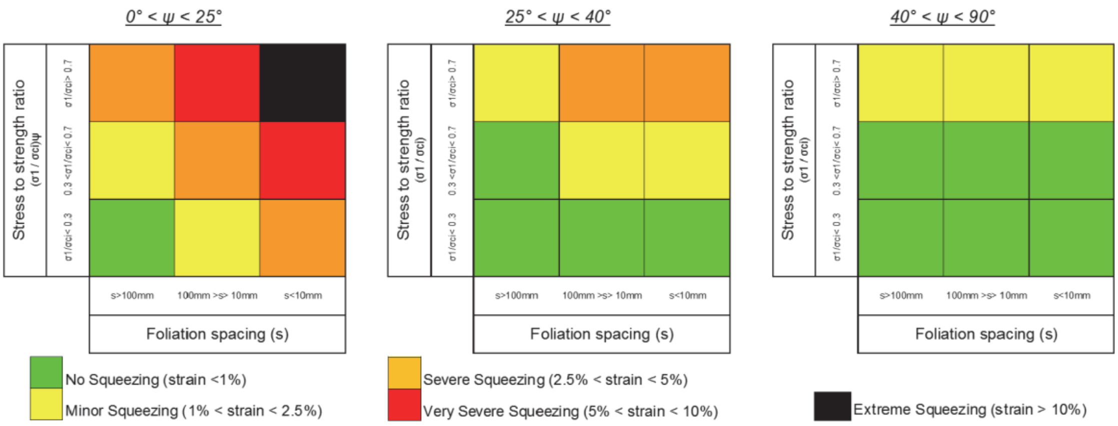


Figure 5.28. Squeezing index based on the angle of interception (ψ) (the angle between the normal to sidewalls and normal to foliation) (Hadjigeorgiou and Karampinos, 2017)

Estimation of strain is an essential parameter for the design process of ground support systems in mining projects. Table 5.4 shows a classification of closure strain in mining projects. Low strain range (less than 2%) in temporary access to mining projects is acceptable by using less ground support systems. Large-scale deformation in underground mining excavations is related to squeezing ground behaviour. Strain in such openings sometimes reaches 40%. Application of yielding bolts and fibre-reinforced shotcrete may provide stable conditions in rock mass structures. Meanwhile, failed ground support devices are rehabilitated to control potential failure.

Table 5.4. The classification of strain closure in underground mines (Roache, 2016)

Drive Strain (ε_t)	Mine Classification	Description
$0 < \varepsilon_t < 2\%$	Few Support Problems	Generally no significant deterioration of the drive.
$2\% < \varepsilon_t < 5\%$	Light Squeezing	Squeezing is now noticeable at the shoulder. Ground support (with no stiff liner) requires no significant rehabilitation.
$5\% < \varepsilon_t < 10\%$	Fair Squeezing	Minor requirement for replacement of failed wall friction bolts (less than 5% of wall bolts). No deterioration of the roof observed.
$10\% < \varepsilon_t < 20\%$	Stripping Zone	Generally reaching the limit in terms of reliable bolt performance, wall stability and width requirements for machinery access. Stripping required.
$20\% < \varepsilon_t < 40\%$	Post Stripping Zone	This essentially means that the drive has been made safe with the removal of unstable material, and re-installation of pattern ground support. The roof requires possible re-support as the material begins to break up close to the excavation boundary. Bolt warping and breakage common in the shoulders and possibly in the roof.

Figure 5.29 shows some typical failure derived from induced stress/seismicity in deep underground mines. High-stress level and seismicity events are the main reasons for the occurrence of these failures in hard rock and weakness zones in deep underground mines. The effect of stress concentration and seismic event on the failure mechanism in the Flowchart GB-FM is presented in Figure 5.30. Induced stress mostly results in brittle failure and sudden failures in hard rock in massive and moderated jointed rock structures with a loading factor less than one. Seismic events may cause a variety of failure modes in a complex mechanism even with loading factors of more than two. The combination of stress concentration and seismic events make it difficult to comprehend failure mechanism. Therefore, site observational methods and engineering judgments are important methods to assess the behaviour of rock masses at depth.

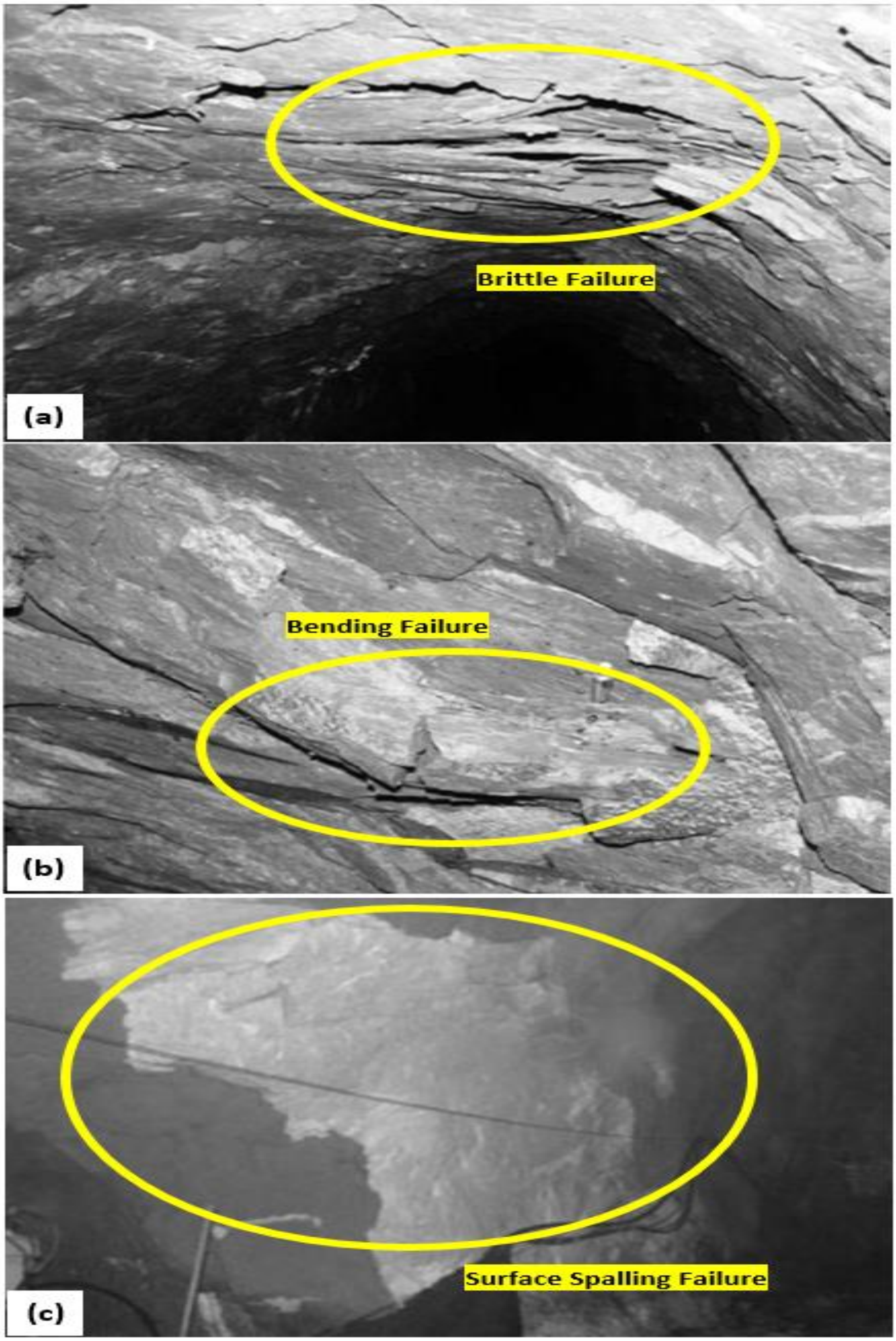


Figure 5.29. Typical failure modes derived from induces stress/seismicity in deep underground mines; (a) Brittle failure in high stress, (b) Bending failure in high stress, (c) surface spalling failure in high stress and seismic events

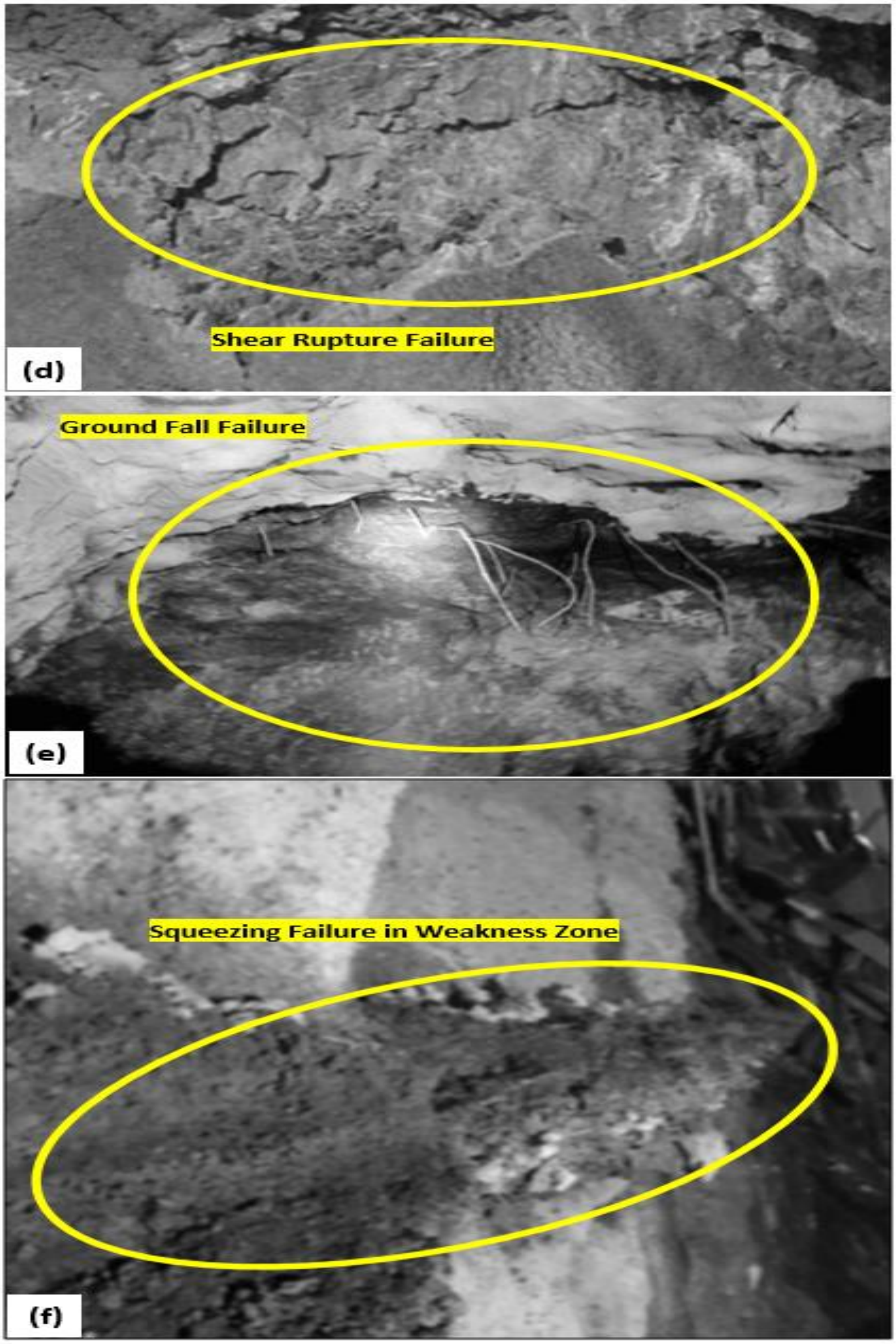


Figure 5.29. Typical failure modes derived from induces stress/seismicity in deep underground mines; (d) Shear rupture failure in weakness zone in high stress, (e) ground fall in seismic events, (f) squeezing failure in shear zones

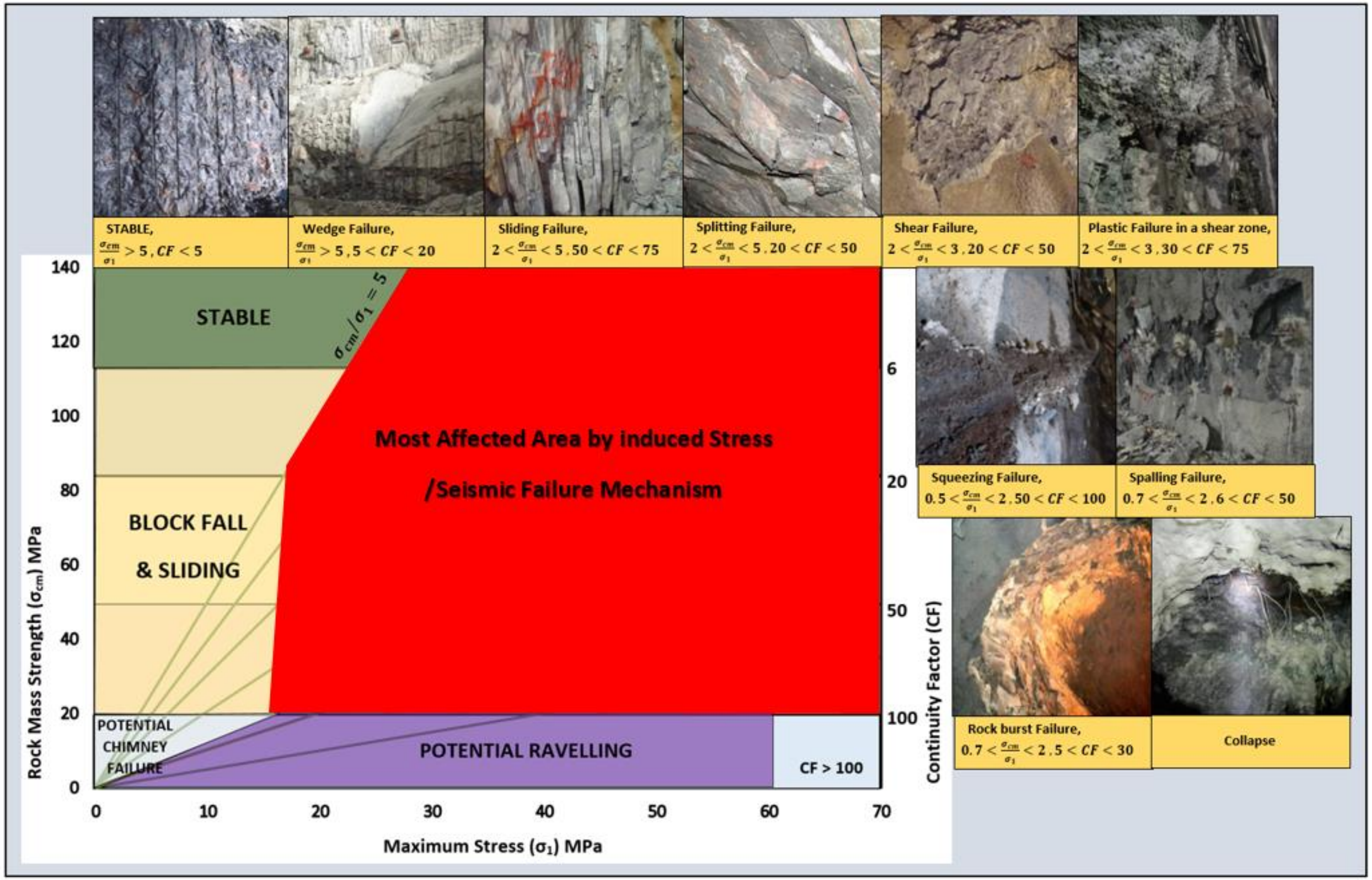


Figure 5.30. Most affected area according to induced stress/seismic failure mechanism in the Flowchart GB-FM

5.3.3. Operational failure mechanism

Operational failures in deep underground mines are relevant to mining operations and engineering activities. Drilling, blasting, scaling, installing ground support systems and filling paste materials in underground stopes are some of the typical mining operations may cause failure in broken rocks. Figure 5.31 shows the operational failure mechanism in deep underground mines. Drilling and blasting usually generate a damaged zone in rock mass structures-vibration and shaking derived from blasting, moves and rotates fractured and broken rocks in underground stopes. Additionally, blasting causes a rapid loading in rock masses and sometimes causes strain bursts and seismicity in the pillar or other geological structures and leads to a failure in rocks.

Scaling is usually performed in underground mines to remove deteriorated rocks by mechanical rock failure. The poor quality of rock masses in underground stopes' hangingwalls or footwalls may lead to rock failure. The poor quality could be natural, due to blasting effect or the geometry of underground stopes. The extracted stopes in some deep underground mining methods such as long-hole stopping methods are filled with paste materials as a support system. Filling of stopes with paste materials is sometimes followed by paste failure due to seismic events, vibration from blasting and quality of paste materials.

Figure 5.31. Operational failure mechanism in deep underground mines

5.4. Case studies of identifying failure mechanism

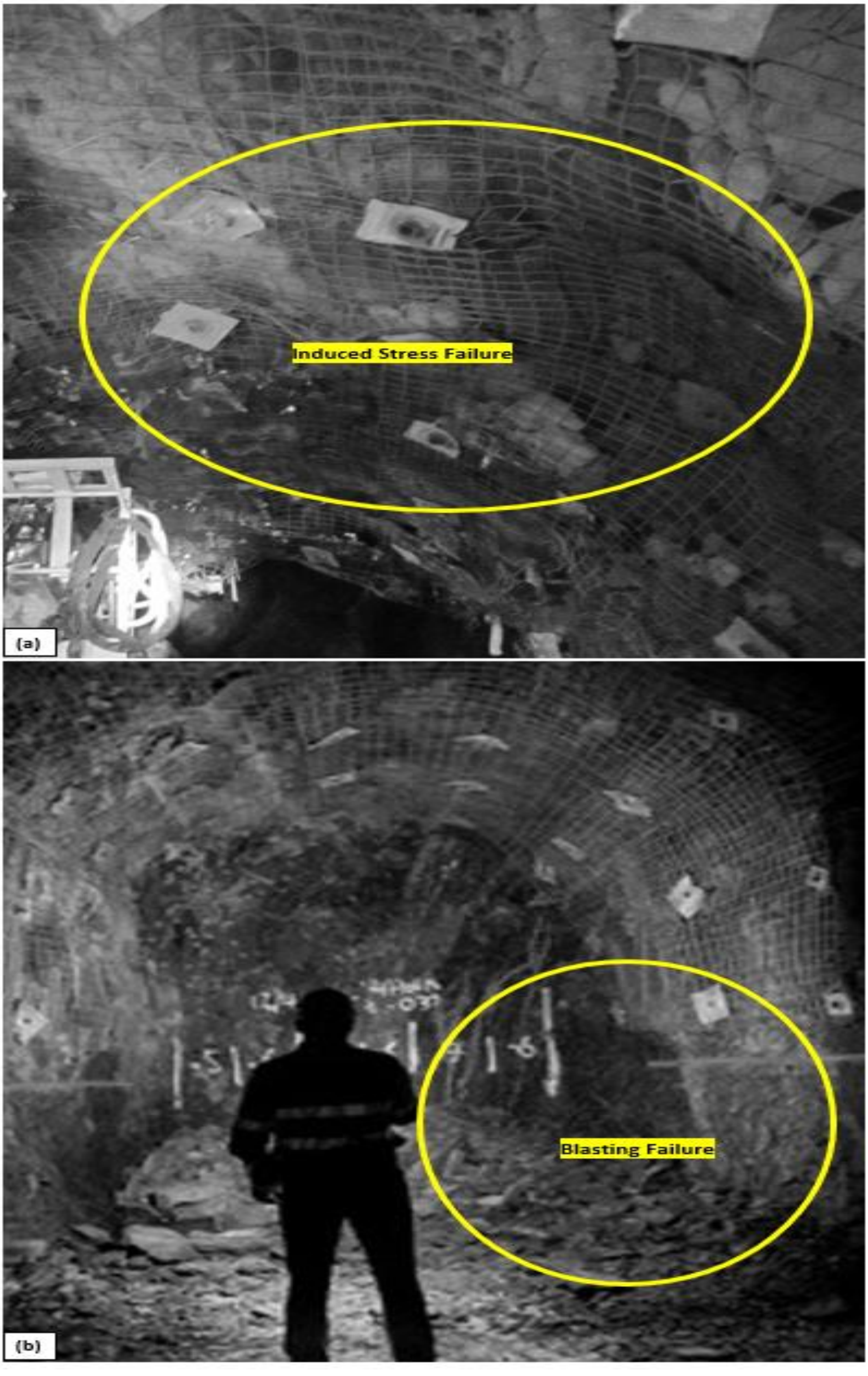
Failure mechanisms in deep underground excavations and mines are studied from examples in Chapter 3, where the ground behaviour modes have already been identified.

5.4.1. Case Example A

Rock mass composition in Case example A in Chapter 3 was classified as being in the jointed/blocky/bedded class. Ground behaviour in decline, as main access at the mine site, has been identified as massive rock failure, intact/structural failure, and water effect.

Block fall occurred due to loosening of rock blocks in the roof and wall. Groundwater, alteration and degree of jointing influenced the volume of rock fall. Since the rock mass was blocky, wedge sliding or gravity failure frequently occurred, especially after blasting. The mechanical and geometrical properties of joints mainly affected the stability of falling or sliding blocks. The quality of the rock mass in shear zones was highly fractured and brecciated. Ravelling failure was expected. However, ground hazards were managed and controlled by installing wire mesh and shotcrete. Site observational methods demonstrated the occurrence of different types of induced stress failures such as brittle failure, tensile failure, splitting failure and shear failure. Mining operations caused blasting failure and scaling failure in decline.

Failure mechanisms at the mine site are presented on the Flowchart GB-FM in Figure 5.32. Based on the proposed method and site observational methods, rock failure was effectively in the area of the chart covered by structural and induced stress.



5.22. Some failure modes at the mine sites (a) Induced stress failure, (b) blasting failure

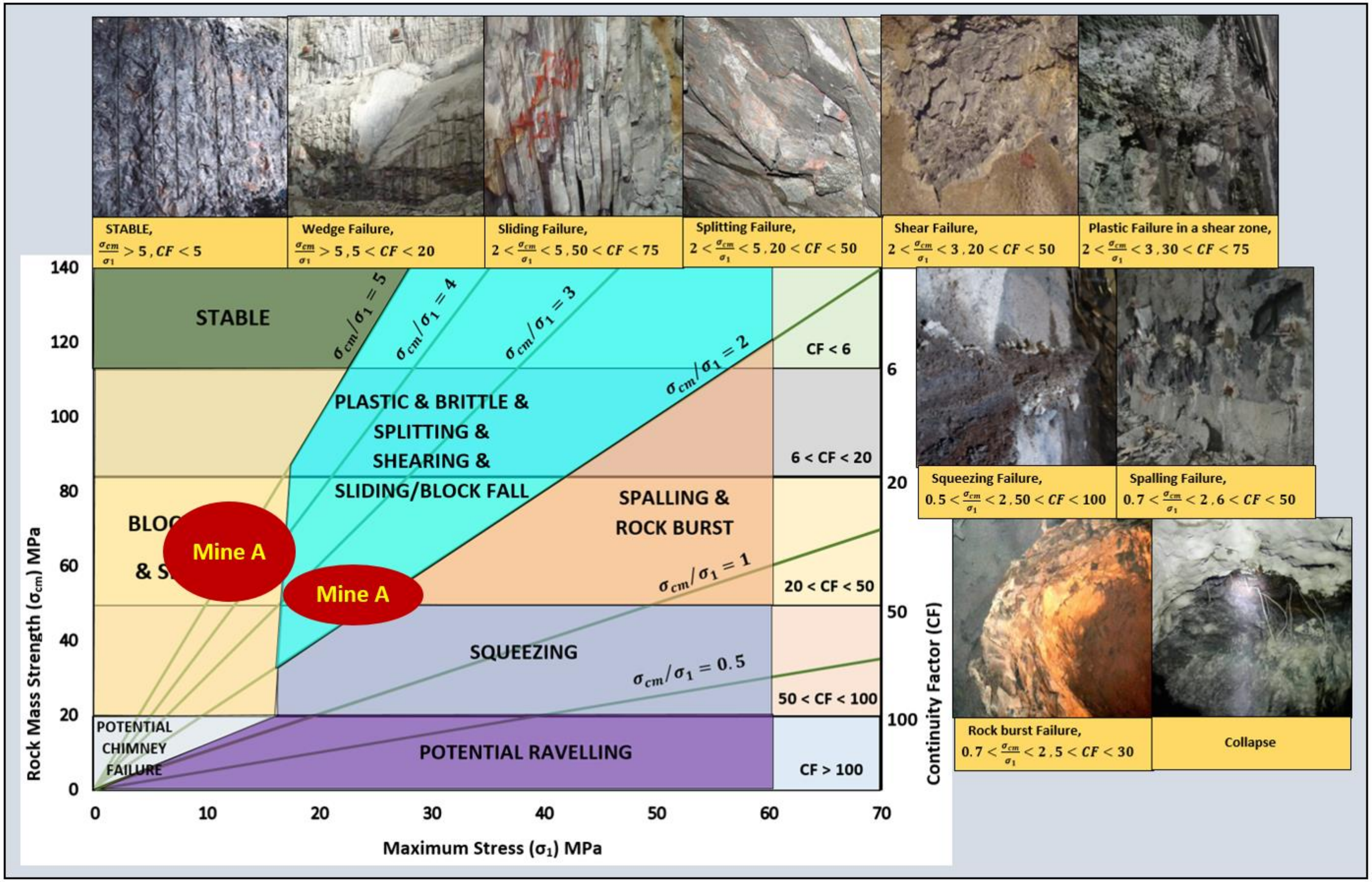


Figure 5.32. Failure mechanism area of mine A on the flowchart GB – FM modified text

5.4.2. Case Example B

Case mine B from Chapter 3 was considered for classification of rock mass composition and identification of ground behaviour modes. Rock mass structure was characterized as being in the jointed/blocky/bedded and blocky/folded classes. Evaluation of ground behaviour modes resulted in massive rock failure and intact/structural failure classifications.

Failure at the mine site was caused by structural failure, induced/seismic failure and operational failure modes. Blocky rocks contain joints and rock blocks. Block fall, sliding and wedge failure occurred due to gravity and low shear strength on discontinuity surfaces.

Stress concentration in mining operations leads to the creation of cracks and fractures in the surface of excavations. Stress/seismic driven failures were due to high-stress environments and seismic events at depth. Figure 5.33 shows some of the observed failure modes at the mine site. Bulking failure in the wall occurred with the increased volume of rock due to the high stress level. The effect of stress concentration was visible even after installing ground support systems. Slabbing failure was due to the creation of tensile cracks and parallel slicing pieces of rocks in the wall following the release of stored energy in the rock mass because of seismic events. Also, an intensive sudden failure of rock burst type occurred. The failure mechanism was identified by recording an increase in the rate of seismic events before failure. The loading factor in this area was about 0.6. Ground fall and sliding failure were observed after blasting and scaling in mining operations. Figure 5.34 presents the failure mechanism area in mine B in flowchart GB – FM. The observed failure modes occurred in the stress range between 15 MPa and 45 MPa. The continuity factor was estimated in the range of 4 to 50 in the rock failure area.

5.4.3. Case Example C

Rock mass structures of case example C were characterised as jointed/blocky/bedded and blocky/folded. Ground behaviour was assessed based on rock mass composition, major geological structures, loading factor, static/dynamic loading, hydrology, and critical features of underground excavations. According to this, massive rock failure, intact/structural failure, and structural failure were identified as the main ground behaviour modes.

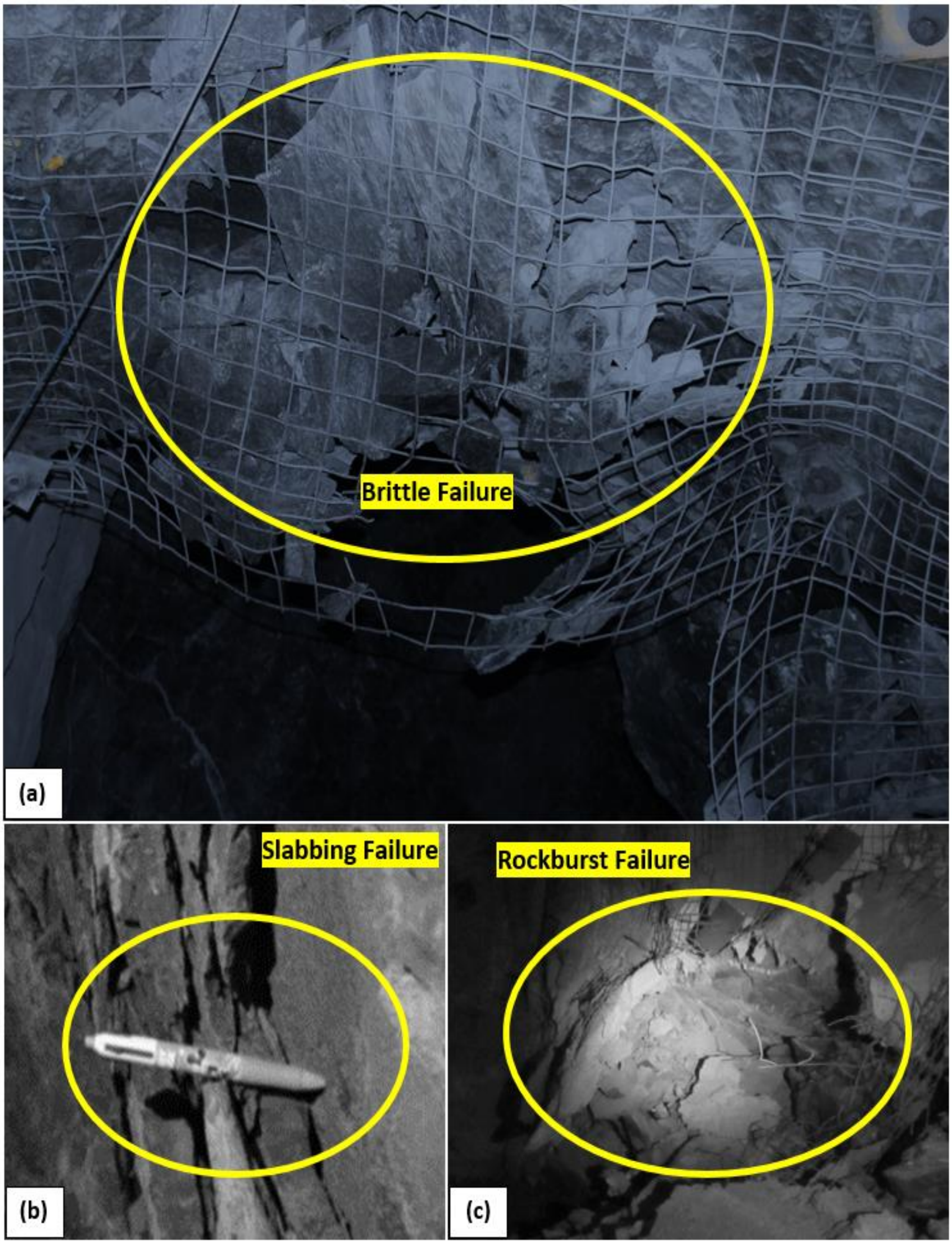


Figure 5.33. Observed failure modes at the mine; (a) brittle failure, (b) slabbing failure, (c) Rockburst failure

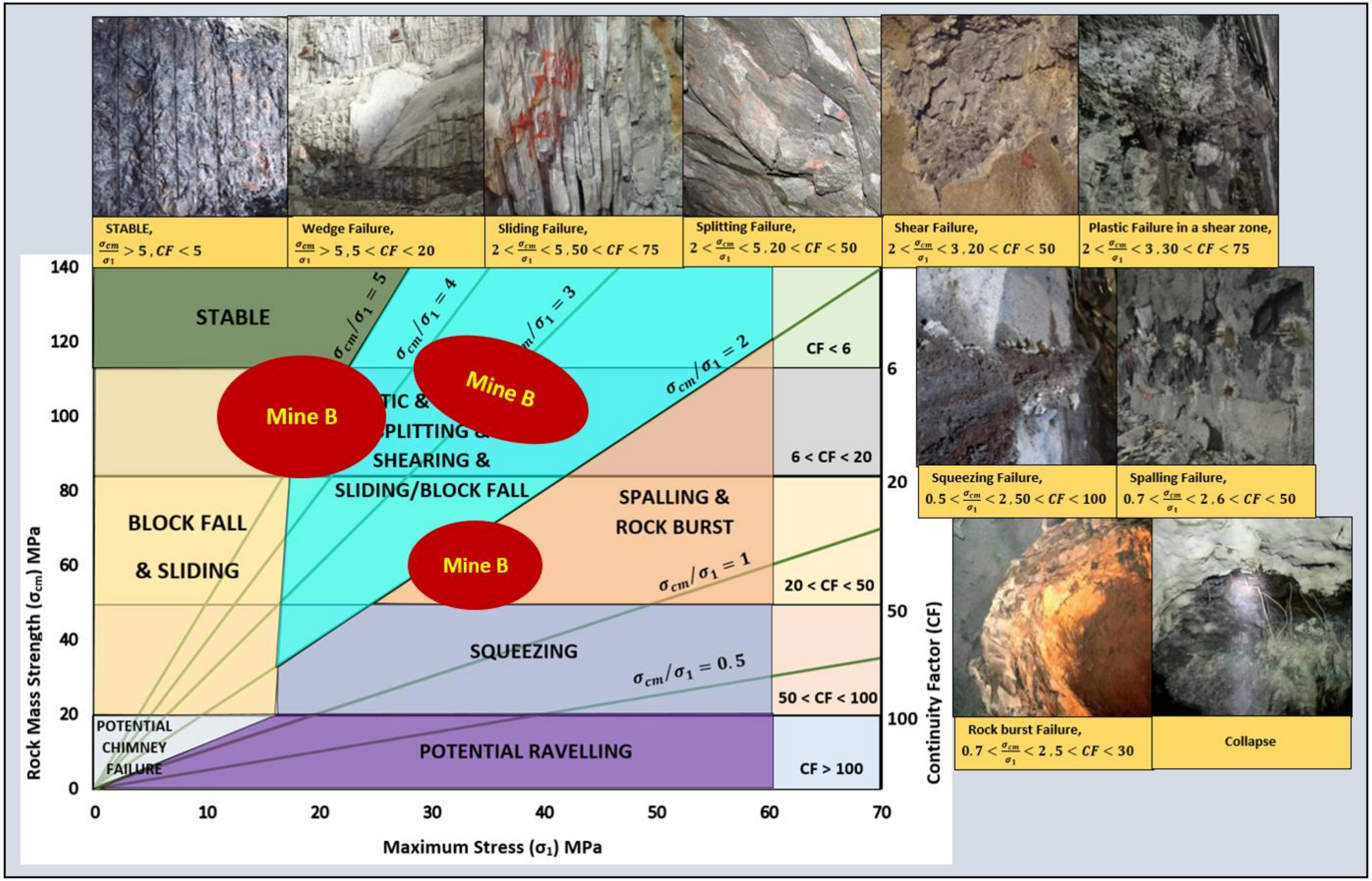


Figure 5.34. Failure mechanism area of the mine B in the flowchart GB – FM

Failure modes in main underground caverns were the structural failure, induced/seismic failure and operational failures. Structural failure were block fall, wedge failure and sliding failure modes. Intersecting of discontinuities in rock mass led to block fall in the roof and wall. Wedge failure and gravity failure frequently occurred, especially after blasting. Stress concentration cause was creating cracks and fractures in the surface of excavations. The effect of stress concentration was visible even after installing ground support systems. Slabbing failure was due to the creation of tensile cracks and parallel slicing pieces of rocks in the wall following the release of stored energy in the rock mass because of seismic events.

Some of the observed failure modes in the project are shown in Figure 5.35. The mechanism of sudden failure in the underground excavation was identified as below:

- Tensile crack initiation due to stress concentration
- Propagation fractures in the walls and crowns
- Extension parallel slabbing over a few months to days
- Split rock flaking
- Cracking in the shotcrete
- Cable bolts carrying significant loading
- Small ground fall
- Released stored energy from seismic events with rock ejection
- Increased rate/number of seismic events
- Frequently increasing sharp noise (hours before failure)
- Rock burst

Failure mechanisms in weakness zones during excavation were observed and recorded as:

- Creation of shear cracks and fractures
- Interconnection of existing and new fractures
- Reducing joint friction on surfaces and adhesive force on interfaces
- Rock deformation and ground movement
- Plastic flow in rocks
- Shear failure
- Small ground fall
- Large deformation over time in rocks surrounding an excavation

- Frequent ground fall
- Collapse

Identifying failure mechanism of the underground excavations is presented in the flowchart GB-FM in Figure 5.36. The observed failures in the excavations were a combination of massive rock failures, induced stress/seismic failure, and operational failure. The collected data from site observations and monitoring methods were used for verification of the flowchart GB – FM.

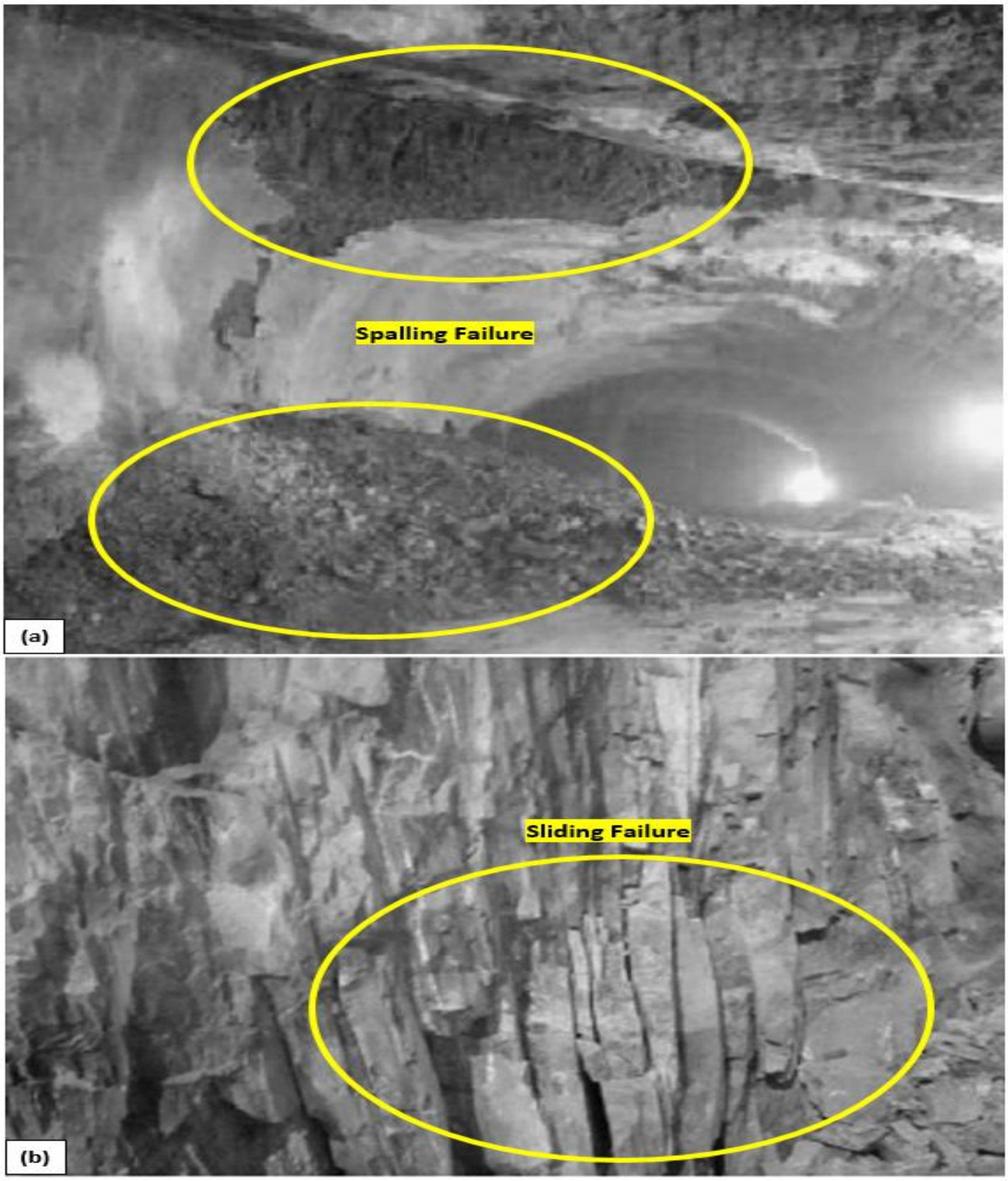


Figure 5.35. Failure modes observed in the deep underground excavation; (a) spalling failure, (b) sliding failure

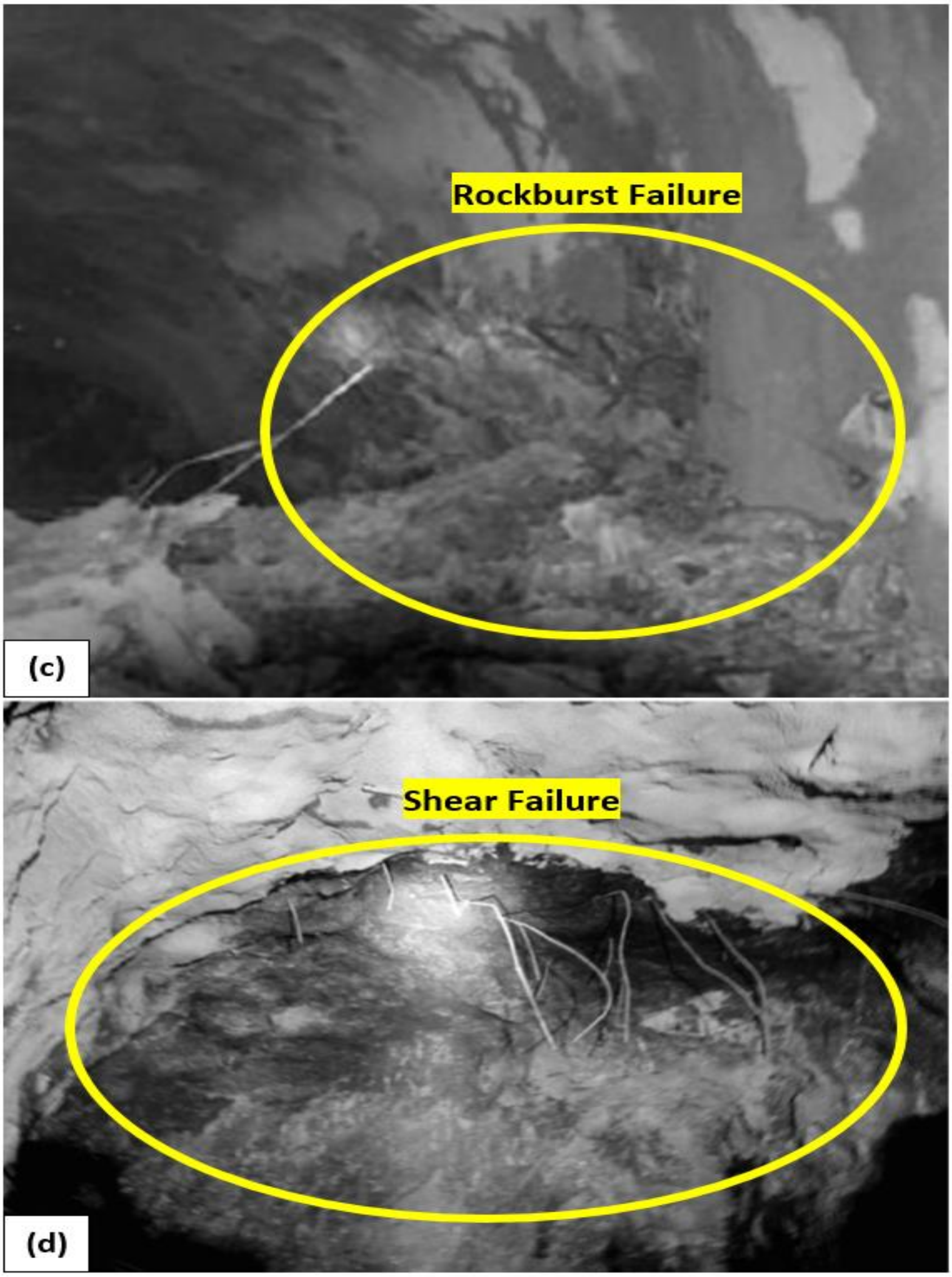


Figure 5.35. Failure modes observed in the deep underground excavation; (c) rockburst failure, (d) shear failure

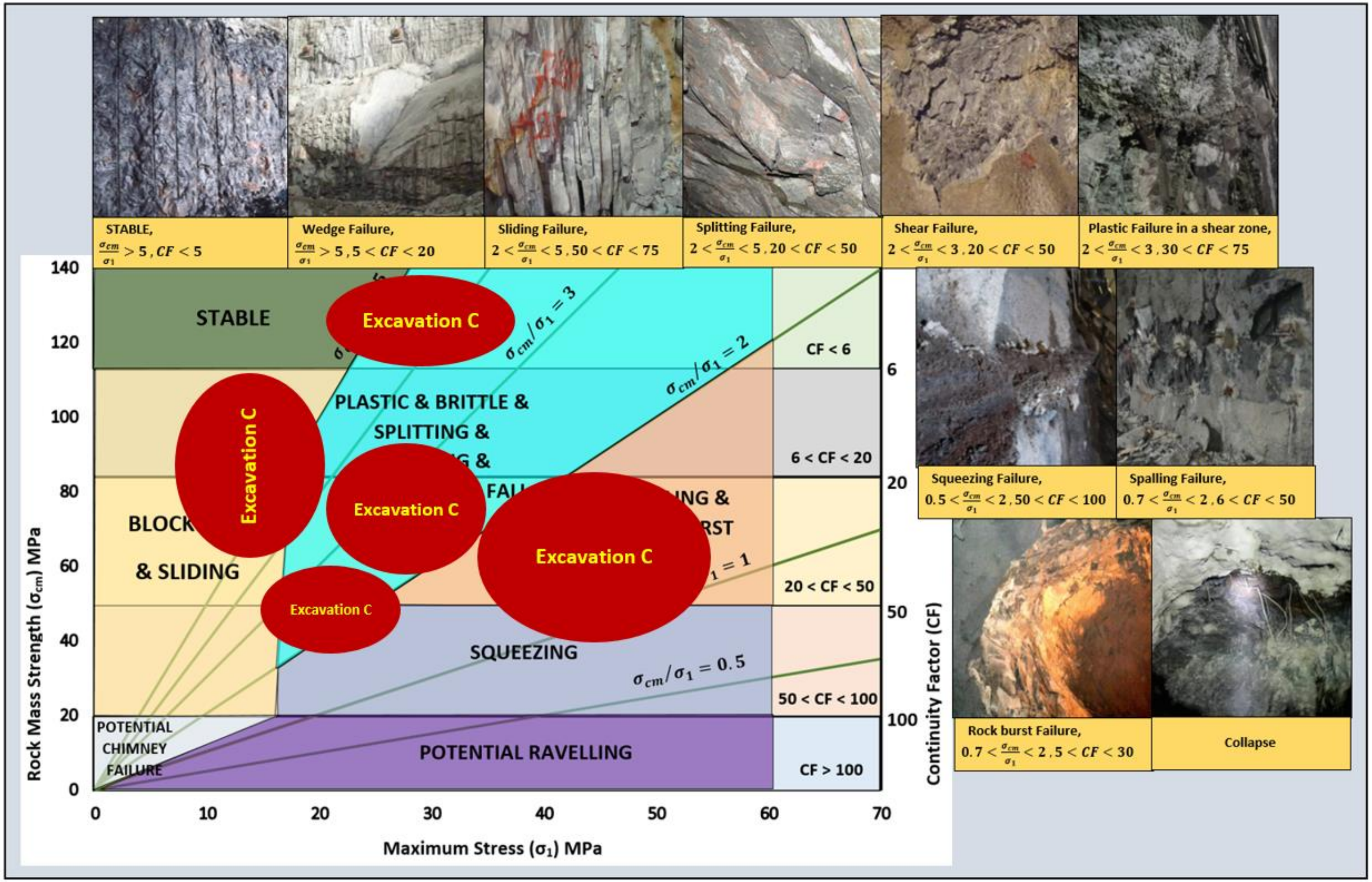


Figure 5.36. Failure mechanism area of the underground excavation C in the flowchart GB–FM

5.4.4. Case Example D

The deep underground mine in case example D from Chapter 4 is considered for assessment of failure mechanism in a rock mass structure. Characterisation of the rock mass structures results in jointed/blocky/bedded and blocky/folded group classifications. Also, rock mass behaviour was identified as massive rock failure and intact/structural failure modes. This type of failure mostly occurred at a depth of 300 – 700 m with the stress value of 10 – 39 MPa. Figure 5.37 shows some observed failure modes in the underground mine. Pillar movement and vertical squeezing were the forms of the failure mode in the massive rock failure class. Vertical squeezing on pillars was accompanied by lateral separation. Squeezing failure was apparently visible in a situation where fibrecrete had been applied. Stress had become concentrated (probably more than 25 MPa) in thin pillars and caused a crack in fibrecrete. The pillars are located in an access drive to underground stopes at levels between 550 m and 700 m, and the stress condition was more than 25 MPa.

According to structural/intact type, sidewall bulking occurred, with the depth of failure of about 1m as a visible movement. The behaviour and failure modes were observed in the ore drive near stopes between 600 and 700 m levels. Undercutting failure mode occurred at unsupported wall sections below the lowest line of bolts in the underground opening, which can be considered as a type of structural failure at the mine site.

Based on the site observations, empirical methods and results of design analysis from numerical modelling methods, different types of failure modes in the mine site is shown in Figure 5.38. The figure displays two graphs (intact rock and rock mass) which are divided into stable and unstable conditions based on ${}^{\sigma_1}/_{\sigma_c}$ and ${}^{\sigma_3}/_{\sigma_c}$ (σ_1: maximum stress, σ_c: rock strength, σ_3: minimum stress). The ratio of stress/strength is a key parameter to predict failure modes. For stress/strength less than one, rock mass structures are mostly stable. For ratios between 0.5 to 1, unstable conditions led to a type of rock fall and undercutting. In this condition and for the state of σ_3 equal to zero, brittle and sliding failure is more likely to occur. With an increase in stress/strength ratio (more than 1) in the underground structure, the rock mass is more prone to spalling and buckling, especially in sidewalls in stopes. Geotechnical precursors indicate that rock burst and squeezing may occur under high levels of stress (the ratio being more than 2) and highly jointed rock structures.

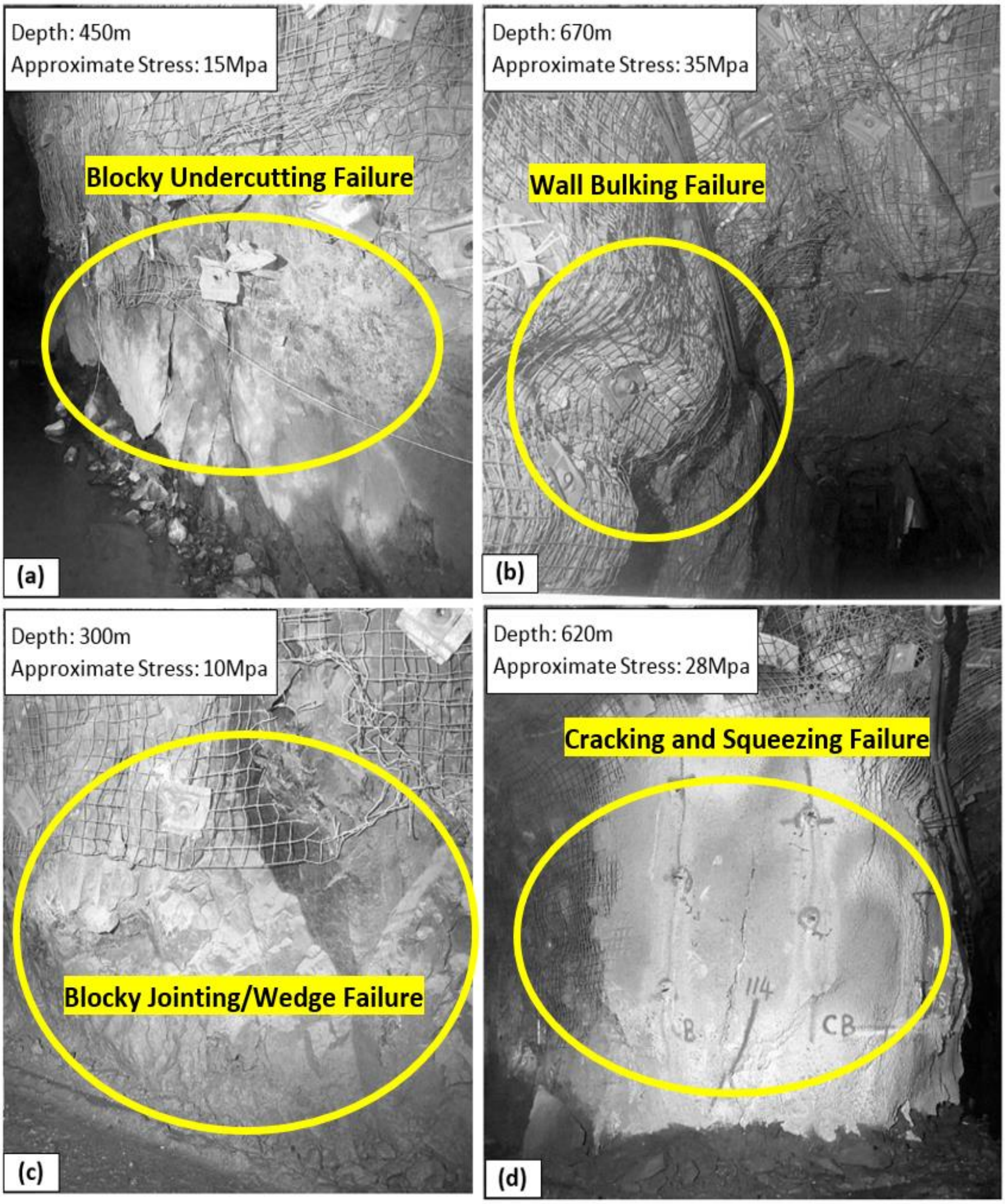


Figure 5.37: Failure mechanisms in the case deep UG mine (a) Blocky undercutting failure, (b) Wall bulking, (c) Blocky jointing (wedge failure), (d) Vertical loading and squeezing in pillar nose with cracks in fibrecrete

Also, the range of failure mechanisms at the mine site based on flowchart GB-FM is presented in Figure 5.37. The flowchart has predicted the potential failure mechanisms at the mine site. Gravity failure and induced/seismic failure were the common failure modes in the underground stopes.

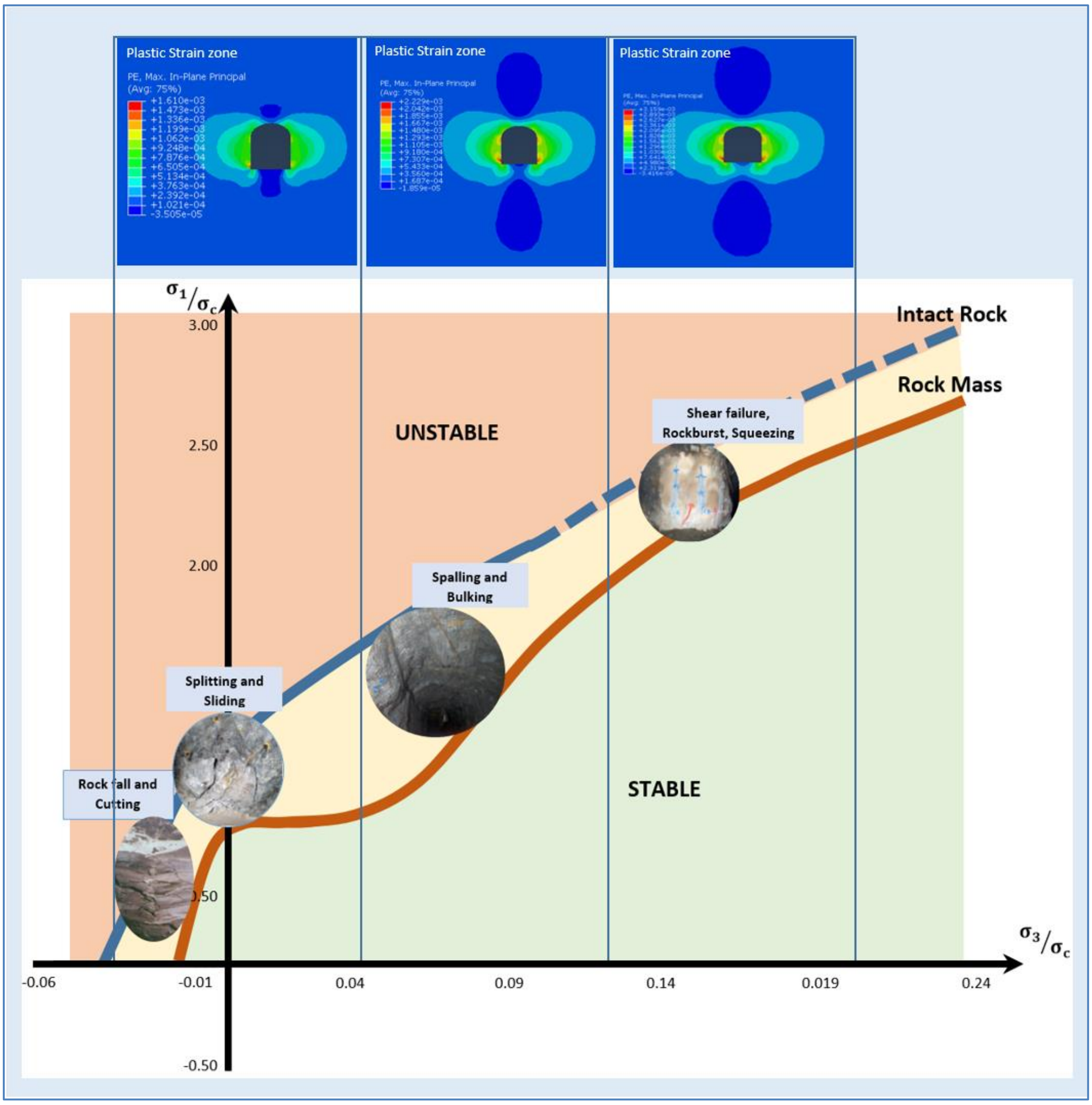


Figure 5.38: Analysis of rock failure mechanism and identify different types in rock mass structures at the mine D

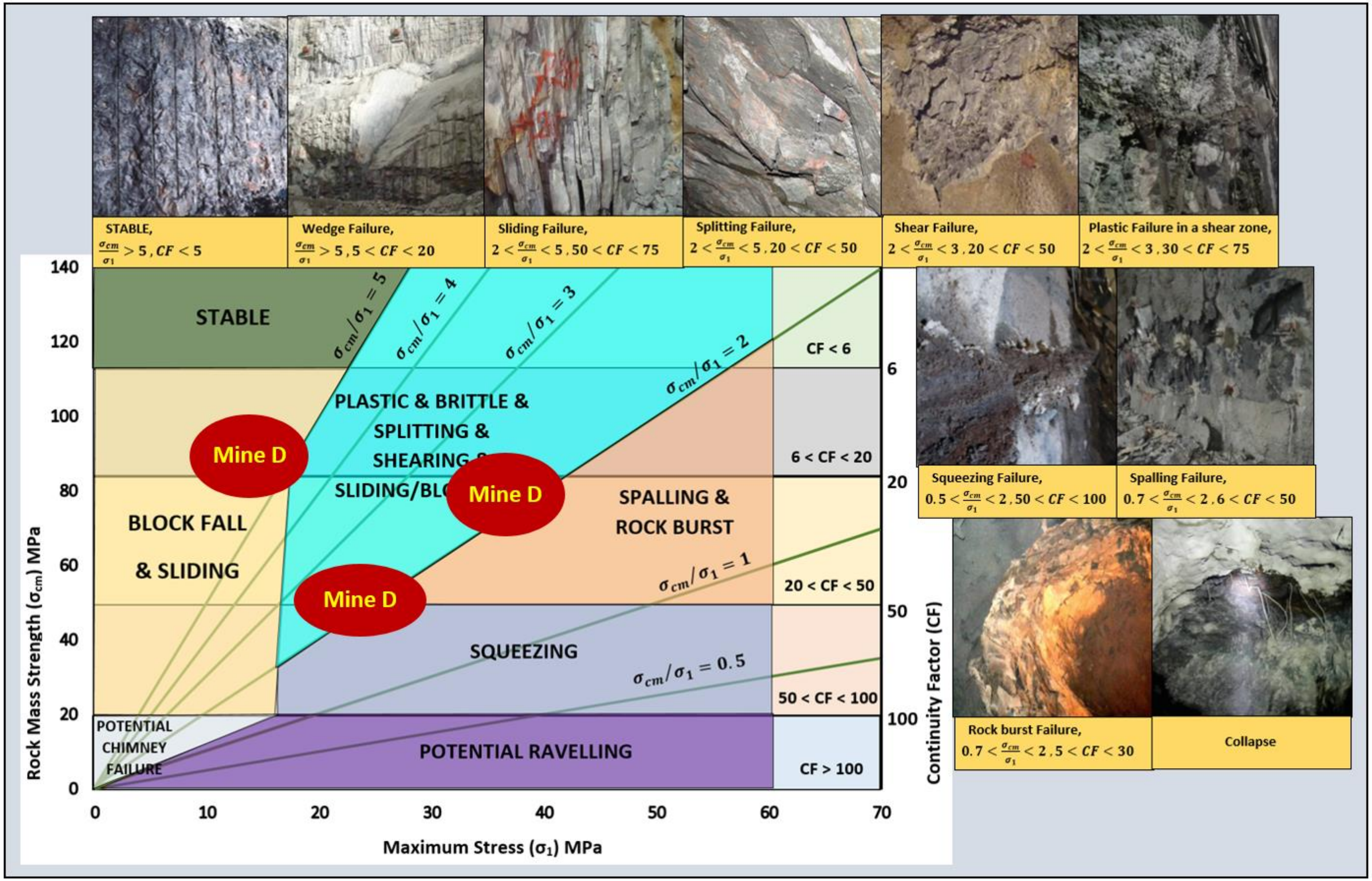


Figure 5.39. Failure mechanism area of the mine D in the flowchart GB – FM

5.5. Conclusion

Deep underground mining activities cause mining-induced stress and change rock mass behaviours that lead to rock failures. Sudden and violent failure mechanisms from the intact rock scale to large rock mass structures are not always recognized. High-stress conditions, seismic events and interlayered weakness zones in rock mass structures are some of the important, relevant factors for failure mechanisms at great depth. Identifying a failure mechanism in complex ground behaviour modes from stable rocks to progressive rock failures is a trigger to assess and control geotechnical hazards.

Intact rock failure depends on physical characteristics and environmental conditions. The failure in an intact rock is initiated with exceeding the elastic limit and microcrack growth inside rocks. Through increasing the stress level, fracture propagation increases and leads to failure in rocks. Failure mechanisms in intact and hard rock at great depth were identified as elastic deformation, micro-cracking, the creation of tensile fractures, propagation of fractures and brittle failures such as slabbing and spalling. The important differences of failure mechanisms in medium and soft rocks are the creation of shear fractures and plastic failure modes.

The mechanism and progress of failure in rock mass structures were presented below:

1- Stable

2- Failure warnings such as joints, shear zones, weathering and seismic events

3- Ground movement such as fracturing, sliding rock blocks, and rock deformation

4- Primary signs before the occurrence of the failure, for example, small ground fall, plastic flow, shotcrete cracks, and split rock flaking

5- Secondary signs before the occurrence of the failure, in particular, the frequent ground falls, the violent breaking of rocks, rock noise

6- Local damage/regional failure

A hard rock mass under high-stress conditions behaves in the form of brittle and sudden failure when the loading factor is less than one. The failure mechanism for interlayered weakness zones is plastic behaviour and large deformation, where there are high stresses and the loading factor is less than one. Ground behaviour modes and failure mechanisms were presented based on the progress of failure with time. Site observational methods and the records of seismic events from several deep rock underground excavations were utilised for validation of this graph. Also, identification of the failure mechanism at great depth was evaluated and presented based on rock mass strength, maximum principal stress, and continuity factor in the flowchart GB – FM. For example, sudden failure was established where the loading factor is less than 2 and where the continuity factor was less than 50.

Failure mechanisms in rock mass structures were categorised into three groups: structural failure mechanism, induced stress/seismic failure mechanism, and operational failure mechanism. Structural failure mechanism was driven from a discontinuity condition in a rock mass by gravity. The typical structural failures are scat failure, wedge failure, block fall, failure in fault and shear zones. Stress concentration and seismic events at great depth are the main causes of induced stress/seismic failure modes. Some of the common failure modes in this category are pillar burst, seismic shakedown, bulking failure, popping, fault burst failure, and spalling failure. Operational failure was considered as failures caused by mining operations and engineering activities such as underground stope failure, and blasting failure.

Several case studies were applied for considering failure mechanisms at great depth and different ground behaviour conditions. The proposed flowchart and methods were applied in case examples where they demonstrated their applicability at great depth and high-stress conditions.

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CHAPTER 6: DESIGN ANALYSIS TO MANAGE GROUND BEHAVIOUR

CHAPTER 6: CONTENTS

6.1. Introduction

Ground management strategy is to identify ground hazards, evaluate and monitor rock mass structures during the whole life of an underground excavation. The typical activities for ground control in mining operations are collecting data from site observations, identifying rock failure, designing blasting patterns, installing ground support systems, and geotechnical monitoring.

Designs of ground support systems using traditional methods were mostly based on restraining the gravity of rock blocks surrounding the excavation, but in modern design, support elements should endure static and/or dynamic loading and large deformations in rock mass structures during the whole life of excavations (Rahimi and Sharifzadeh, 2017). Ground support demand for stabilising rock mass structures in hard rock and high stresses require an estimation of energy demand of the rock and energy dissipation of support elements, especially in dynamic loading conditions (Feng and Hudson, 2011). Ground control and management deal with all geotechnical activities related to hazard recognition, understanding of failure mechanisms, and design of ground support systems to provide a safe environment economically in rock underground engineering projects.

The purpose of the chapter is to propose a practical geotechnical strategy for ground management in deep and hard rock conditions during the design, construction and serviceability stages of underground mining projects. Critical geotechnical steps for mitigation of risks and stabilisation of rock masses in deep underground excavations are (as shown in Figure 6.1):

1. Optimise layout of openings based on major geological structures and orientation of principal stresses
2. Modify sequential excavation and extraction rate
3. Define ground control and management strategies for small/large deformation based on potential failure modes
4. Design natural ground as a local support system, such as pillars in underground mining methods
5. Design and utilise backfilling methods as a regional support system in mines
6. Design and apply surface and reinforcement support devices for unstable rocks

Additionally, a practical methodology for the design of ground support systems in deep, hard rock and high-stress conditions is proposed with regard to geological structural conditions, loading conditions (static and/or dynamic types), loading factor (the ratio rock mass strength: major field stress), and primary/secondary failure modes. Several deep underground excavations will be studied, and the proposed methods will be examined for some case examples.

6.2. Ground control and management strategy (Mid-Long term)

Ground control is a process used to solve geotechnical problems related to mining operations, rock mass behaviour and instability. Additionally, it is defined as all applicable processes and methods that are used, firstly, to identify potential failures in rock mass structures, secondly, to implement a solution in ground instability cases and, thirdly, to manage the applied solution through the lifetime of the project. The techniques include a plan, the design and a method of operations to avoid workplace injuries and equipment damage due to the risk of rock failure.

Diagnosis of failure modes and their mechanisms is the fundamental step in ground control planning. Collected data from site investigations, engineering geological surveys and laboratory/field tests are used for characterisation of rock mass structures, and then the failure mechanism is diagnosed based on in situ rock stresses, and hydrological and project conditions.

The geotechnical aims of ground control and management plan in underground mining stopes are listed below:

1. To define a hazard control program by evaluating, designing and monitoring rock mass structures
2. To extract mineral resources in a safe and economical manner
3. To develop a process for hazard identification and failure mechanism diagnosis supported by a training program for personnel

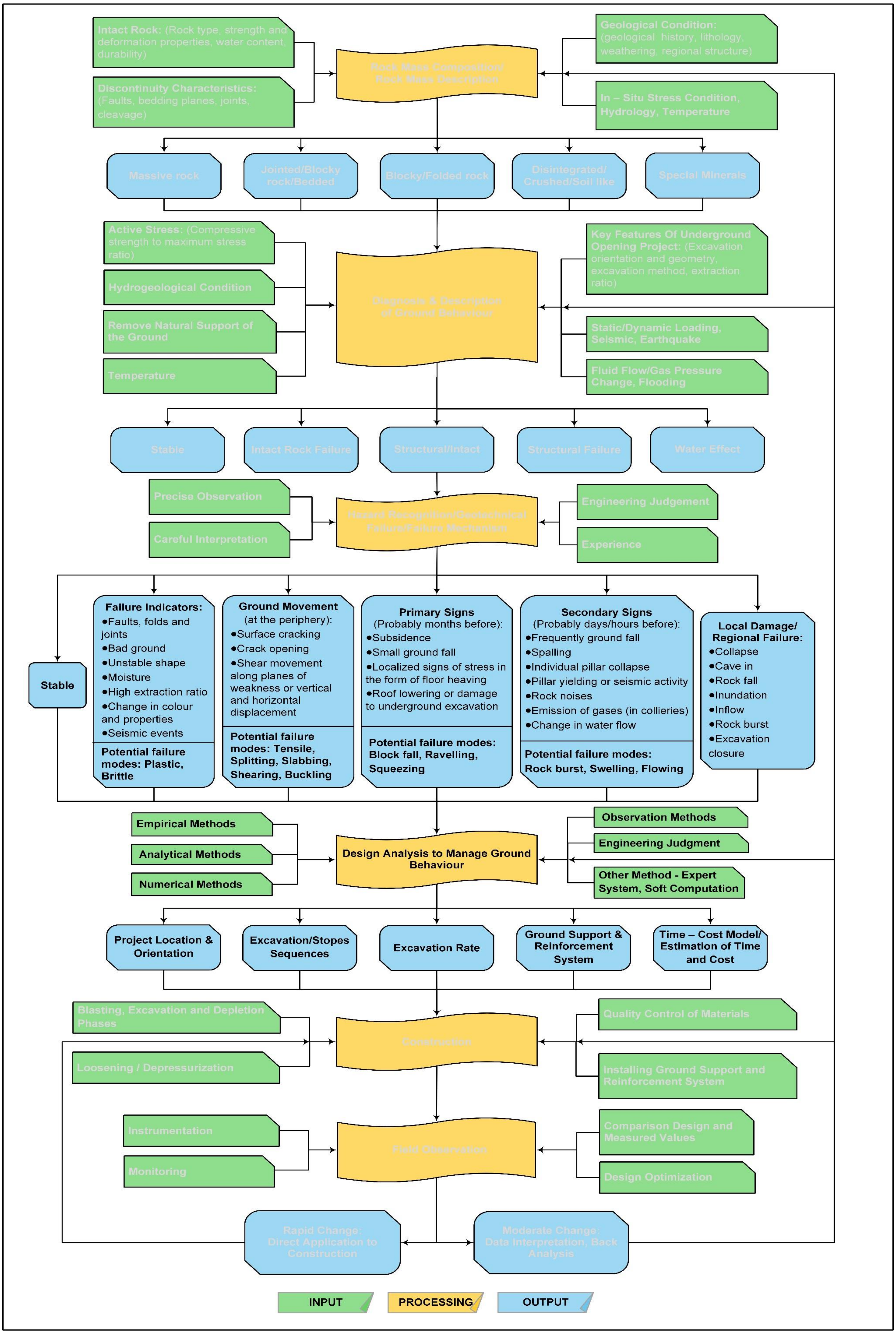


Figure 6.1. Ground control and management strategies in deep underground mining

Ground control methods can be considered in both initial design and modification of design during the whole life of projects. The main three stages for this issue are experience-based design, management based design and technically based design. The experience-based design is defined using the previous empirical experience of ground control. The term management is related to using appropriate ground control with economic and cost-effective methods. Application of the main principles of rock mass structures like time-dependent behaviour in ground control is called technically based design (Swindells, 1992). Changing geological structure during development of an underground opening needs to be detected in advance and the rock support and reinforcement modified if required. Using appropriate methods for drilling, blasting pattern and excavation can reduce the intensity of damage in the rock mass. Shape, size and orientation of underground openings influence the potential instability. Also, useful parameters are projected location, project orientation, excavation/stopes sequence, excavation rate and application of ground support and reinforcement systems.

Figure 6.2 presents a ground management strategy in-depth underground mining projects. The main steps of the scheme are the design, construction and serviceability. The design step of ground management is associated with input geological and geotechnical data from site investigations, engineering geological mapping and results of laboratory/field tests. Design analysis of an underground excavation is carried out based on ground behaviour, failure mechanisms and project conditions, and results in location and project orientation, excavation method, sequential excavation, extraction rate, and selecting ground support systems. The practical approaches of ground control and management during the construction stage are a determination of standard procedures for geotechnical activities, provision of required equipment with competent personnel, quality control of materials, identification of geotechnical hazards, safety analysis before ground failure occurs, and inspection/monitoring of ground support performance. The appropriate approach for the projects during serviceability are maintenance and rehabilitation of ground support failure, load deformation measurements and preparation of a contingency plan.

Deep underground mining projects are designed and developed in three stages:

1. Strategic design: This is a type of primary design and preparing a broad plan for the mines site such as the location of access and underground stopes.

2. Tactical design: Tactical object is to provide the detail design of projects for example stability analysis of rock mass in underground excavations and selecting ground support system before the operational stage at the mines
3. Operational design: This is related to monitor and update design parameters through observational methods and monitoring system.

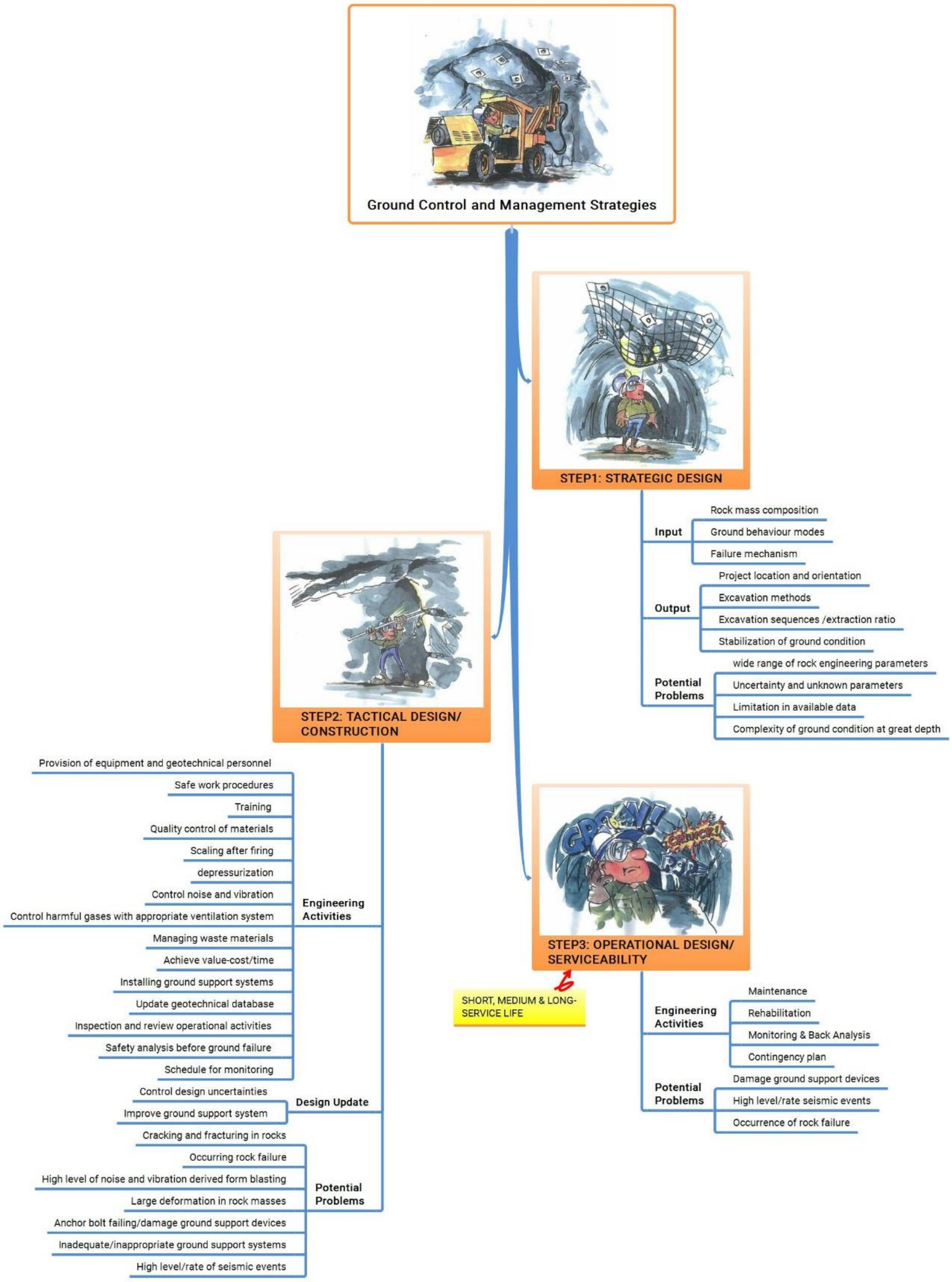


Figure 6.2. Ground management strategies in deep underground excavations

A wide range of parameters in rock mass compositions, ground behaviours modes, failure mechanisms and in situ stresses make complexity and uncertainty in the estimation of rock engineering properties especially in seismically active mines at great depth. In the design phase, visualisation, interpretation and assessment of the real orientation and geometry of rock mass structures are difficult from direct observations to prepare the geological and geotechnical model. Therefore, uncertainty and confidence in the characterisation of rock mass structures, diagnosis of ground behaviour, failure mechanism, and ground support design are assumed. The possible engineering disaster from design phase encountered in construction stage could be a complex failure mechanism such as sudden failure and large deformation, inadequate and inappropriate ground support systems. Hence, ground control and management strategies should be accomplished following knowledge, experience and management to overcome challenges and problems in mining operations. In the serviceability stage, seismic events, stress concentration, and corrosion of ground support systems may lead to damage support devices and rock failures. A contingency plan with a monitoring system is required for the evaluation of ground problems.

The major steps for ground control and management at great depth are listed below:

1. Collecting data from available evidence, observed features, and seismic events
2. Identify potential geotechnical hazards
3. Design analysis of hazards for ground management, and determine appropriate strategies such as smooth blasting method, and installing ground support system
4. Evaluation of the effectiveness of multi-factors on ground conditions, especially time
5. Implementation of ground management strategies in the hazard area
6. Geotechnical monitoring and review the ground responses
7. Update strategies

Geotechnical issues and ground control management should be considered during the whole life of underground opening projects from the feasibility study stage to the final closure of a mine.

For the duration of the design phase of ground control and management, four approaches, namely, project location and orientation, blasting control, sequential excavation/excavation method/extraction rate, and ground support selection method, significantly affect the type of ground behaviours and failure mechanisms in underground excavations. These approaches are discussed in the following sections.

6.2.1. Project location and orientation (Layout)

The layout of project location and orientation is chosen based on principal stress orientations, major structural defects in rock masses, excavation geometry, the location of mineral resources, availability and accessibility of equipment, objective and purpose of projects, and the location of mineral resources in mining projects. The angle between the orientation of an opening and major structures of rock masses influence the type of failure and mechanisms in underground mining activities (Figure 6.3).

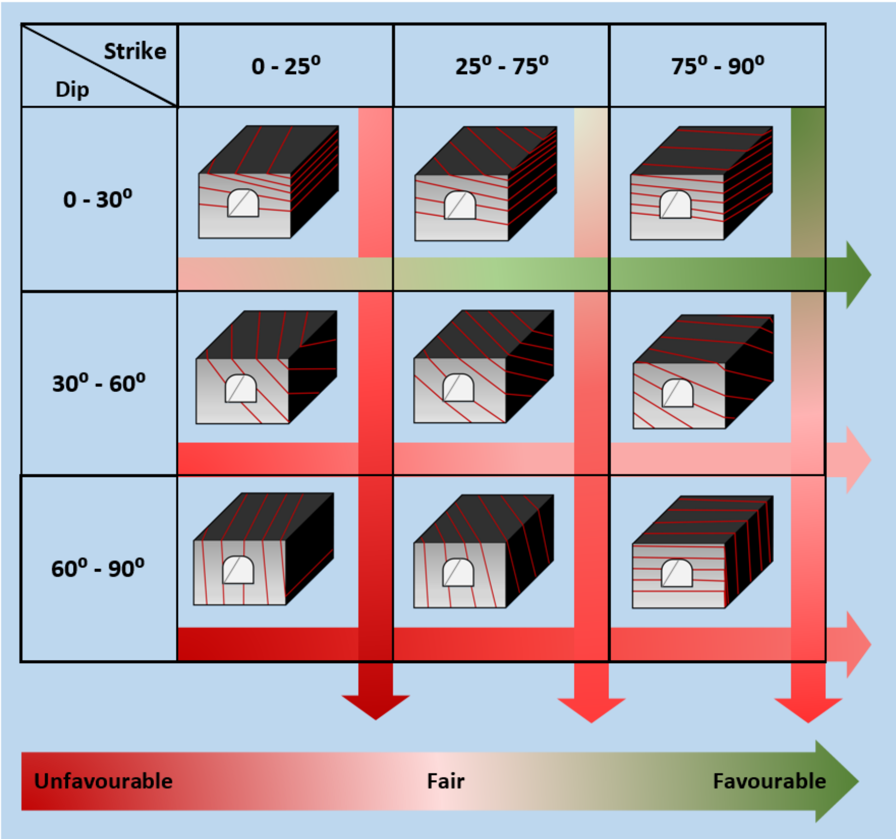


Figure 6.3. The influence of discontinuity orientations and dips with the axis of excavations

Theoretical results and practical implementations indicate that the perpendicular and parallel orientation of an opening with significant structures are the most favourable and unfavourable in underground mining projects, respectively. Simple failure mechanisms, like tensile fracturing and shear failure, may combine and produce complex ground behaviour at different orientations of excavations. The unfavourable orientation of discontinuities surrounding openings reduces the bearing capacity of rock blocks and may lead to ground falls or sliding failure (Sharifzadeh et al., 2017).

The axis of underground excavations also influence discontinuities inside rock masses and may affect fluid channels and flowrates in openings. Fluid flow can cause different types of ground behaviour and failure modes, for example, flowing and swelling phenomena.

The appropriate layout of location and orientation of excavations about the orientation of dominant structures and principal stresses can reduce structural failure modes and consequently, the ground support system required for stabilising. As a result, an underground mining project is forecast to run at a low cost and have a high performance in such a situation.

6.2.2. Blasting control

In deep underground mining, the blasting pattern and procedure are designed specially to provide a satisfactory outcome. Drill–hole parameters, spacing, burden, pattern and diameter of empty holes, charging method, and the target of fragmentation size are some of the key parameters for blasting control. Suitable drill and blast design parameters reduce the damaged rock zone surrounding an excavation and lead to suitable fragmentation size, cost–reduction in production and ground support equipment (Szwedzicki, 2003). Poor quality of blasting in an underground mine can cause considerable damage in rock mass structures.

Blasting excavation methods usually create three zones around a rock mass with over break, new fractures and propagation of existing discontinuities (see Figure 6.4). The distribution of damage in the rock mass following the drilling and blasting method is called the “Excavation Disturbed Zone” (EDZ). The over break zone is approximately 10% of the excavation profile, but sometimes this damage zone reaches 60-70% in roofs (Suorineni, 2009). A control blasting method can be employed to decrease the thickness of the excavation damage zone.

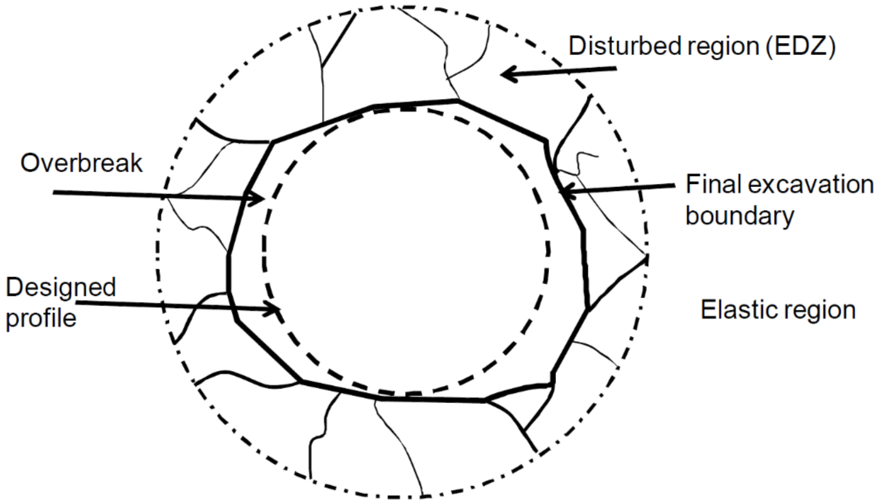


Figure 6.4. Blast damage zones around and underground excavations (Suorineni, 2009)

Typically, three factors control the performance of blasting and intensity of damage in mines (Singh, 2018):

1- Quality of rock mass such as discontinuities, strength, weathering, hydrology and density
2- Explosion characteristics, which are the velocity of detonation, powder factor, and borehole factor
3- Blasting pattern and procedure, for example, drill hole deviation, spacing, burden, and amount of explosive in holes

The quality of drilling and blasting is influenced by ten percent design and ninety percent of practice and implementation of design parameters (Singh, 2018). Operation of blasting is mostly related to preparing the drilling face, face mark – up, accurate drilling, and loading of holes.

A practical method for optimisation of blast design is conducting a delay interval between holes to create a lower level of ground vibration at the mine site. Figure 6.5 shows an optimised blasting pattern with delay sequences in an underground opening. Ground vibration from blasting can cause a severe problem to ground instability. Blast vibration is due to three parameters: peak particle velocity, duration and frequency. These parameters are evaluated by explosion characteristics in the design of the blasting pattern.

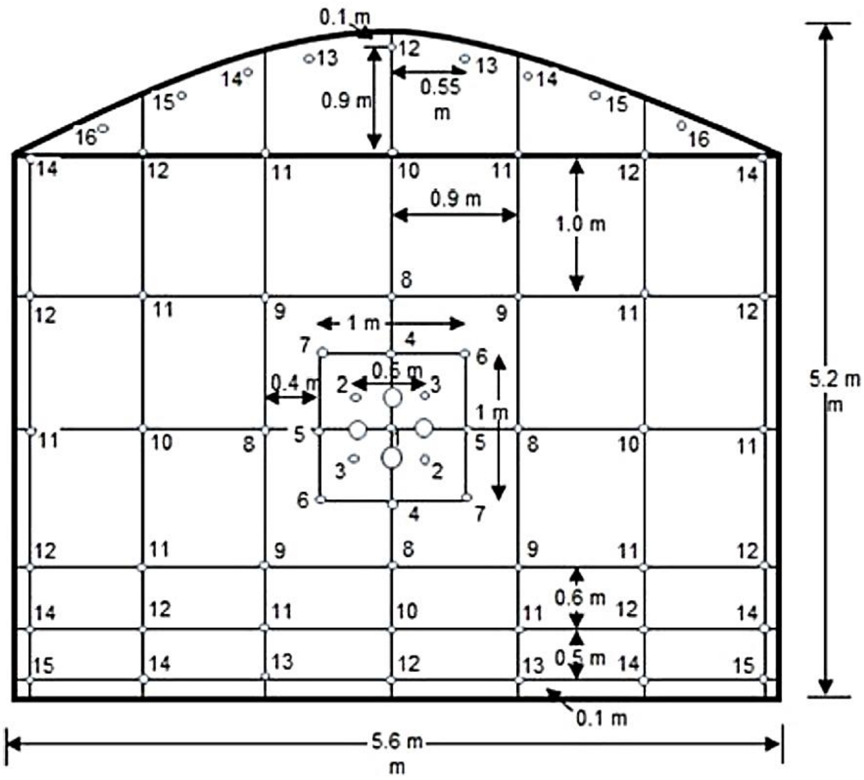


Figure 6.5. An optimised basting pattern with delay sequences in underground openings (Roy et al., 2016)

A practical technique to control sudden failure under high stresses at great depth is using the deep-hole pre-cracking blasting (DHPB) technique in mines (Ouyang et al., 2015). The technique is based on drilling deep holes to created mechanical fractures in the rock mass to release stress concentrations surrounding excavations.

Controlling the blast damage zone, vibration and stress concentration in the design and implementation of the blasting procedure are essential for the improvement of safety, economy and productivity in deep underground mines.

6.2.3. Excavation sequence/extraction ratio

The excavation method has a significant influence on the engineering behaviour of rock masses. For example, drilling and blasting methods can provide safe environments compared to mechanical excavation methods at great depth and in hard rock conditions, because of a destressing effect and dissipation of stress concentrations in a fractured rock mass following blasting (Mazaira and Konicek, 2015).

Excavation sequence in a mining operation is described by the extraction of the orebody in an underground mining operation in a particular order to achieve a high extraction rate of the orebody with minimal ground problems. Post-excavation stress can be reduced by applying an appropriate excavation method, sequence and extraction rate in underground openings (Sharifzadeh et al., 2013). A series of individual stopes is excavated safely and economically. Sequences in underground operations can be divided into primary, secondary and tertiary priorities. The primary priorities of sequential panels or stopes are usually designed to be–in high-grade regions of the orebody, considering target products in the mine plan, field stresses, the stability of the rock masses, the dimension of the stopes, and backfilling methods. The primary panels or stopes are excavated and then filled with backfill materials for at least two vertical lifts before extracting the secondary and tertiary priority stopes. Figure 6.6 is a schematic view of sequential excavations in underground stopping. Generally, the excavation sequence dimension varies between 5 and 30 m width, 15 and 50 m length and 15 and 100 m height, in Australia. Sequential excavation in mining operations can be developed as top-down, bottom-up, centre- out and abutment-centre (Ghasemi, 2012). The dimension of stopes in sequential excavation affects mining operation costs, instability of rock masses and failure mechanisms.

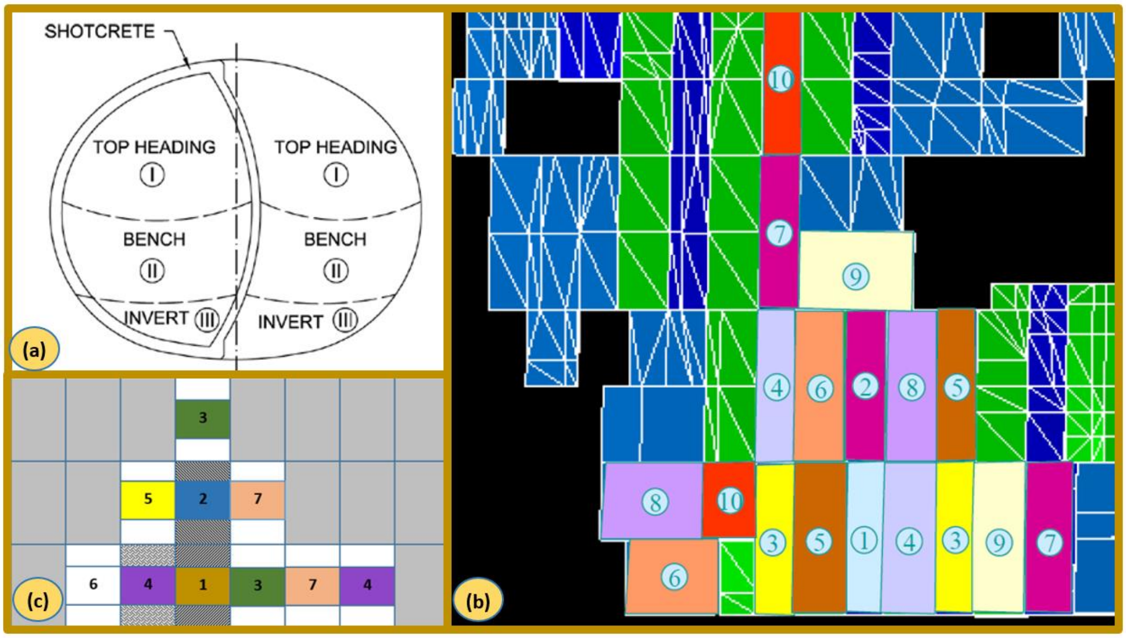


Figure 6.6. Sequential excavation in; (a) a tunnel, (b) an underground mine with bottom-up and centre-out method, (c) an underground mine with bottom-up sequences method (The number shows the sequences of excavations) (modified after Ghasemi (2012)).

Figure 6.7 shows sequential excavations in underground mines. Geological condition, dip, dimension, grade, mining methods, rock mechanical properties, resources of fill materials, production planning, and operational schedule influence mining sequence and extraction ratio.

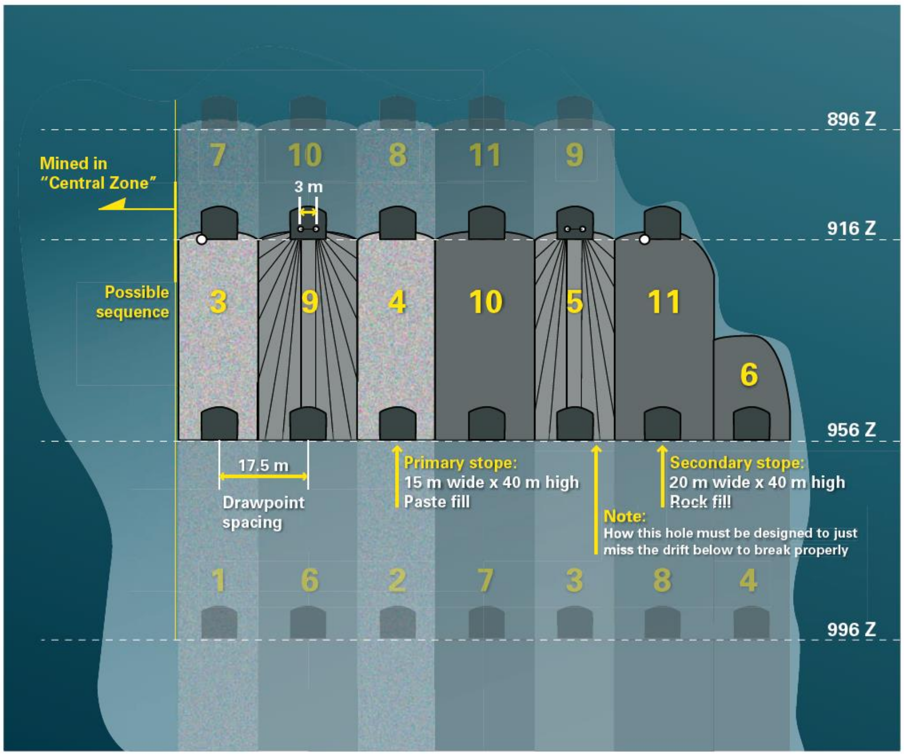


Figure 6.7. Mining sequences in sublevel stopping methods (COPCO, 2007)

Mining sequences are used to achieve target products in a safe and economic manner. The main concern in selecting a mining sequence is the locations with high induced stress, especially in permanent pillars. Sequential methods are required to achieve a high ore recovery in high stresses and seismically active mines at great depth. Some key technical methods in sequential mining to achieve high recovery are to below (Beck and Sandy, 2003):

1- Managing access to ore body: Development of a mine require more access in reserves. If a part of the development lost due to bad ground conditions, the inventory of development can be used. Also, a rehabilitation plan is employed to reduce hazards in bad ground conditions.

2- Managing fault displacement and damage on major structures: Development of sequencing stopes intersected by faults or shear zones may lead to large seismic events, large deformation, fault slip or pillar failure. Mining away from fault zones and careful blasting methods may lead to release energy in smaller increments.

3- Prioritisation areas for mining extraction: Selecting the initial starting of mining, the location of final pillars and the end of life prioritisation are necessary for mining sequences.

4- Managing induced damage with support and utilisation of the rehabilitation: Mining-induced stress and seismicity make a change in the quality of ground conditions and may lead to damage or ground support system as well. A rehabilitation plan is required to install an extra support system and reduce the risk of rock failure.

In mining sequences, the stability of ground condition, recovery and cost should be considered to select appropriate mining sequences at great depth and high-stress conditions.

6.2.4. Ground support techniques

Ground supports provide an strong zone in unstable rocks and reduce rock deformation by a certain amount to avoid immature failure. The stabilisation of the ground in underground works can be accomplished by natural or artificial ground support methods. Natural ground support approaches like room and pillar methods are useful in medium–hard rock conditions, low–medium stress levels and short-medium term life in excavations. Artificial ground support devices are mainly divided into surface rock support and rock reinforcement elements. Surface support tools are applied on the surface and external parts of rock mass structures. Rock reinforcements are installed in the internal part of rock masses. The usual surface and reinforcement devices used in underground mining projects are rock bolts, cable bolts, shotcrete, concrete lining, strapping, mesh, timber sets, steel sets, hydraulic props, yielding sets and mesh (MOSHAB, 1999). Backfilling methods is a practical technique for sublevel stoping as a local support system in large-scale openings in mining projects. Stress level, density, particle size, porosity, strain level and the proportion of cementation are assessed to design backfill materials.

Instability of rock masses is derived from geotechnical structural defects in rocks and static/dynamic loading conditions due to stress concentration, seismic events and released energy, drilling and blasting, gravity, groundwater and temperature. The stabilisation process for rock mass structures in underground openings is as follows:

- Determine project conditions and purposes
- Identify major geotechnical defects and failure mechanisms in rocks
- Identify the main types of loading (static/dynamic) surrounding excavations and estimate their intensities
- Analyse ground condition and estimate the rock mass deformations
- Select the type of ground support approaches: natural ground support and/or artificial ground support systems
- Select the types of surface and reinforcement support devices
- Control the ground – support performance

The application of ground support and reinforcement systems has to provide stable conditions in rock mass structures through reinforcing, holding and retaining functions (Kaiser et al., 1996b). Figure 6.8 shows the use of support devices in different parts when encountering failure zones in an underground excavation. Installing rock bolts in a small damage zone may provide stability in the excavation. Large-scale damage zones require the use of different layers of support systems as described below (dependent on the loading condition and failure modes) (Li, 2017):

Part 1: installing rock bolts to reinforce and strengthen fractured rock, by forcing rock blocks together

Part 2: using inner support systems (such as shotcrete, mesh) for a retaining function

Part 3: cable bolting to provide an adequate holding function in loosened blocks

Part 4: implementation of external surface support devices like steel sets and cast concrete, which is more applicable for long-term life excavations.

Ground support systems at great depth and high-stress conditions are evaluated and designed by practical, numerical and observational methods.

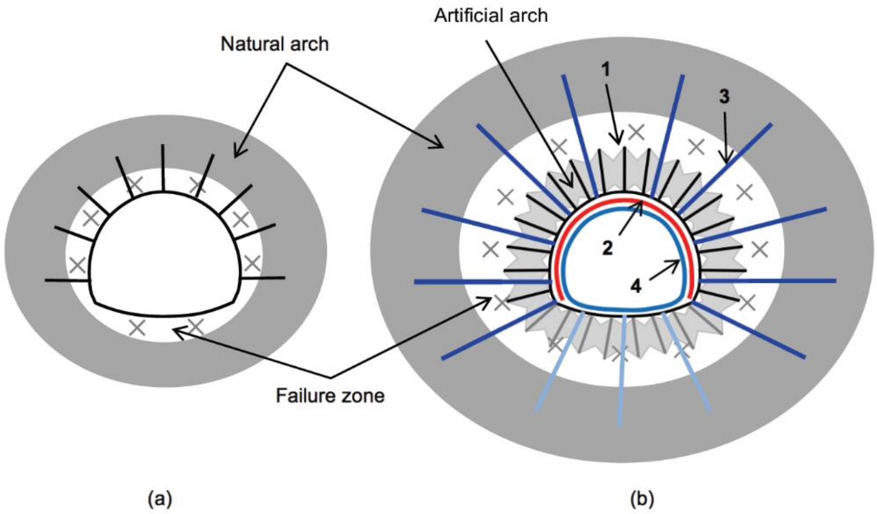


Figure 6.8. Different types of ground support devices in a failure zone; (a) rock bolting in small damaged zone; (b) large damage zone with (1) rock bolting, (2)retaining by inner surface support devices, (3) cable bolting, and (4) outer surface support devices (Li, 2017)

A variety of types of external and internal devices may be used together as ground support. A satisfactory situation is the compatibility of deformation between them in rock masses. Ensuring compatibility of the support elements with the reaction of the host rock mass structure enhances safety. Figure 6.9 is a schematic of incompatibility and compatibility of deformation in external and internal support tools in underground excavations. An incompatibility condition of a support system may lead to failure of rock bolts and therefore expose external device support systems to an overstress condition. Rock support demands can be supplied by deformable, reliable and strong devices in high-stress conditions. The control of design parameters in the construction phase is carried out by field measurements to verify the agreement between the disturbed rock mass structure and the design parameters and then allow the parameters in the design process to be modified.

Figure 6.10 presents a flowchart of design analysis for ground control and management in deep underground excavations. Diagnosis of ground behaviour, identification of failure mechanism and geometry of excavation are used as input data for design analysis. The conventional design analysis methods to manage ground behaviour consist of empirical methods, analytical methods, numerical methods and observation methods, neural network and expert system. Ground control and management are established in excavation strategies, design analysis strategies and support strategies. In stables condition, full face and half face excavation method can be used for underground construction. Massive and blocky rocks are more competent to be stable. Generally, the ground condition is a self-supporting but may need to use some local light support device to prevent small rock/wedge failure derived from intersecting discontinuities. Plastic, brittle and tensile are some typical failures of intact rocks. Full face and half face are the proper methods for excavation in this condition.

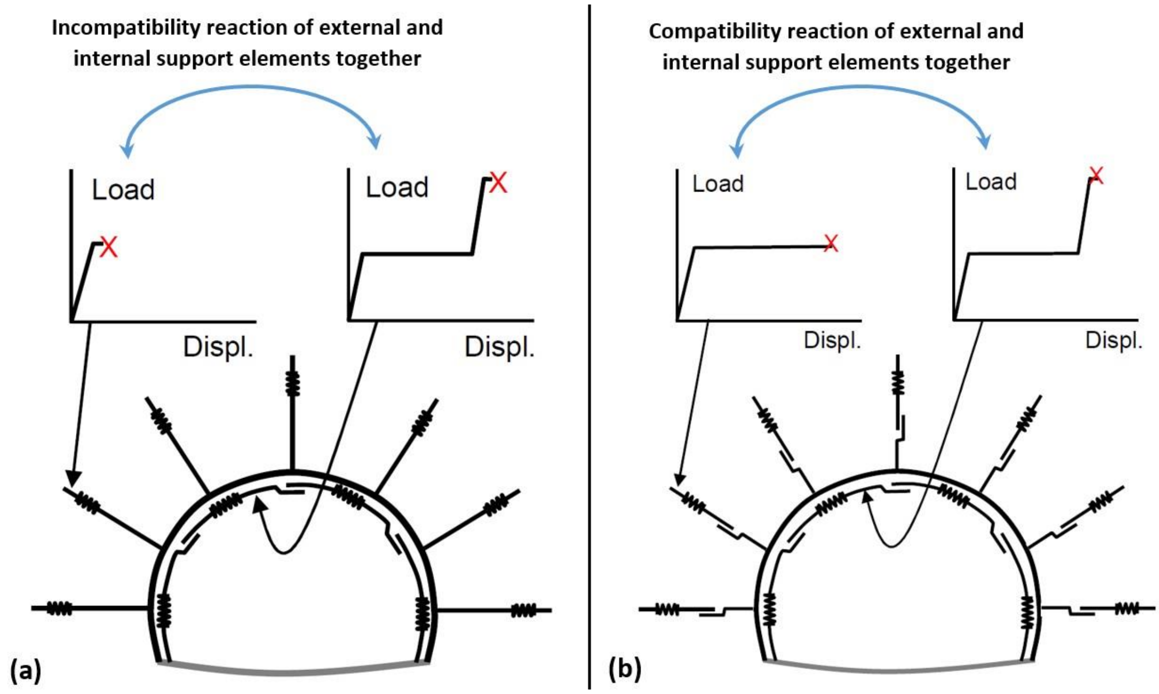


Figure 6.9. A schematic of (a) incompatibility; and (b) compatibility of deformation between external and internal support elements (Modified after Li (2015a))

Design analysis strategies can be carried out by analytical methods, shear stress analysis, neural networks, discontinuity deformation analysis and so on. Support devices should be selected based on deformation control, unify zone of failure and sewing layers to each other. In the case of structural failures, which block fall, sliding, buckling, shearing and toppling are frequent failures, sequential method for excavation and key block theory discontinuity deformation analysis, analytical method and shear stress analysis are the most important strategies to utilize support devices for deformation control, reduce stress concentration and unify the zone of failures. In a high level of stress field, ground failure modes are slabbing, spalling, rock burst and squeezing. Application of pilot tunnel and multi-sequence methods can be used for excavation methods. Instability of ground condition should be evaluated based on energy release rate, rock burst tendency index, discontinuity deformation analysis and observational methods. Meanwhile, reduce stress concentration, stress release and deformation control are the main points for selecting appropriate support devices. Furthermore, water effect failure modes like flowing and swelling should be managed regarding deformation control by using proper support systems, for example, grouted rock anchors.

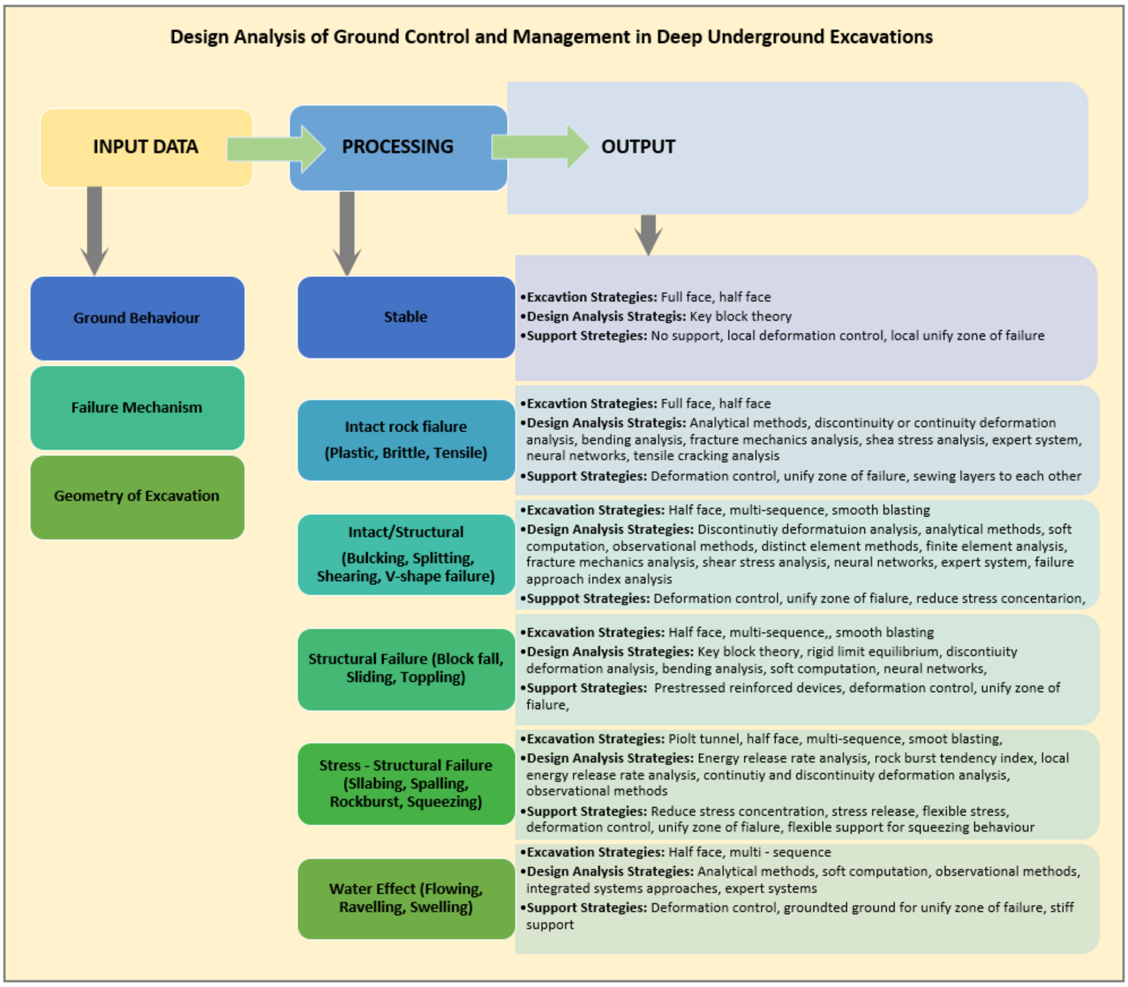


Figure 6.10. Design analysis, excavation and ground management strategies in deep underground excavations

6.3. Ground support design in deep underground excavations

Design analysis to ground management determine required ground support and reinforcement system, excavation method, sequential excavation, extraction rate in underground stopes and a time – cost model for the project. The main principle of a ground support system is to provide a steady persistence in unstable conditions to prevent immature failure. Strong, elastically stiff but plastically deforming, and reliable anchorage are important properties of proper support devices at high-stress conditions (Li, 2015b). Design of ground support system under dynamic loading is required an understanding of the failure mechanism, loading mechanism and assessing energy absorption of ground support.

Analytical methods are more applicable for evaluation of unstable circular shape in a closed form solution, and rock mass structures are assumed to behave in an isotropic and homogenous condition. Empirical support design methods are based on rock mass classification systems, for example, RMR, and Q system. Numerical modelling such as finite element method, discontinuous deformation analysis is helpful for analysis of wide range of ground behaviour and failure mechanism. However, there is not proper for design support in burst-prone ground condition(Cai and Kaiser, 2018). The approach of the observational method is employed in complex ground condition especially in a static and dynamic loading condition.

The modern design of rock support and reinforcement system evaluates the capacity of energy absorption in high-stress conditions and seismic events in underground practices (Kaiser et al., 1996b). Simplicity, availability, quick installation, flexibility in different conditions, integrity, and cost effectiveness are some critical parameters related to ground support design. The change of the geological structure and rock mass behaviour during a construction stage should be identified earlier and use rock support and reinforcement tools if it is required.

The techniques for ground improvement by support elements are sewing rock blocks together, unifying the zone of failure, avoiding fracturing progression, controlling deformation and strengthening rock mass structures. A number of factors including availability, capacity, simplicity, cost-effectiveness, installation method and energy absorption should be considered in the design.

Different loading conditions surrounding an excavation require different types of ground support systems (Rahimi et al., 2014). In general, the active loads on a surface of excavation are static, dynamic and a combination of both of them. The origins of static loading are gravity, in situ/induced stress, tectonic activities, groundwater, residual stresses and temperature. Seismic events, strain burst, fault slip, pillar slip, gravity collapse, loading/unloading rate, and blasting are the main sources of dynamic loading in underground openings.

Support capacity depends on loading mode, loading rate, the share of loads between different support elements, displacement of the support system and energy absorbing capacity (Kaiser and Cai, 2012). The capacity of ground support system is evaluated concerning availability and combination of support elements to act as an integrated system, the type and amount of loads, displacements and energy demand, especially in dynamic loading conditions (Cai and Kaiser, 2018). Figure 6.13 shows the design procedure of the ground support system in in-depth underground mining projects. The main factors in the design are an estimation of depth failure and fracturing, demand ground support in the static and dynamic condition, and evaluation of rock support system capacity based on the load, displacement and energy absorption factors.

Ground support design based on static loading conditions is used in underground mining projects where the risk of seismic events is low. The typical ground support devices for static loading conditions are fibre-reinforced shotcrete, rock bolts and cable bolts (Jacobsson et al., 2013). Ground support design in ground with dynamic loading conditions should include an absorbing kinetic energy factor derived from seismic events (Kaiser et al., 1996a). The results of drop–weight tests indicate that about 25% and 75% of absorption of energy demand, respectively, belong to surface support and rock bolt devices in hard rock conditions. In soft rock conditions, this proportion is divided into 30% for rock bolts and 70% for surface support systems (Louchnikov and Sandy, 2017). Transferring the load from the surface to reinforcement ground devices is not critical in static conditions, while this point is a fundamental requirement in dynamic conditions to ensure the performance of ground support systems.

Reinforcement system design in underground mining excavation is based on required ground demand and available reinforcement capacity. Ground demand can be estimated by empirical, numerical and analytical methods. Reinforcement capacity is associated with the loading capacity, pattern, stiffness, and load displacement capacity.

Key parameters in ground demand assessment in underground mining excavations are rock mass structures, rock mass strength, field stress changes, static and dynamic loading surrounding excavations, stiffness that lead to identifying failure modes. Ground demand is divided into five main components (see Figure 6.11) and explained in below

(Hutchison, 1996):

1- Dilation control: Joint and fractures surfaces are held and interlocked together with stiff reinforcement to maintain rock mass strength.

2- Displacement: large rock mass displacement can occur where high mining induced stresses exist in highly jointed rocks or weakness zones.

3- Gravity loading: This factor is related to deadload rock blocks created from intersecting discontinuities.

4- Surface ravelling: Ground ravelling can occur when the pattern of reinforcement system is not compatible and suitable with the spacing of joint sets in block size, or bolts have inadequate bond strength in rock masses.

5- Service life: short term and long term lifespan of mining excavations influence in selecting reinforcement systems and ground demand. Corrosion and stress conditions and seismic events are some of the essential factors in considering ground demand in long term life.

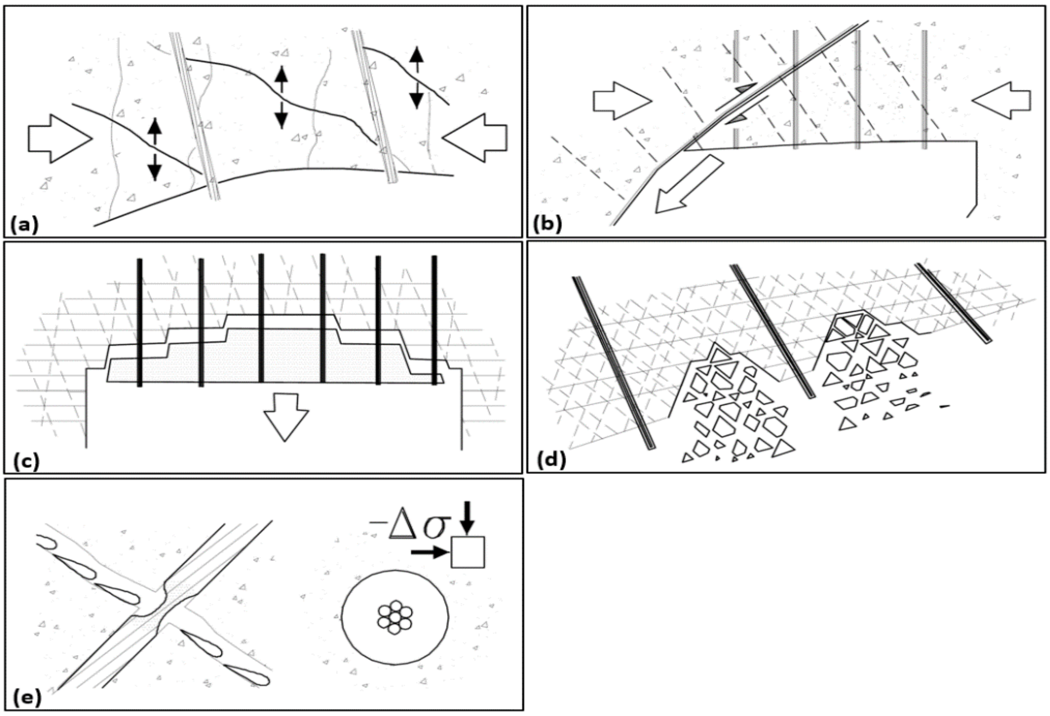


Figure 6.11. Main components of ground demand considerations in reinforcement deisgn (Hutchison, 1996)

Reinforcement capacity depends on the properties of steel, a bond interface between bolt and grout, quality of grout, load transfer between rock masses and bolts, pattern of bolts, orientation and length. The capacity of reinforcement is expressed as the following factors:

1- Immediate stiffness: As shown in Figure 6.12 (a), this factor is a relationship between initial loading and associated displacements in reinforcement systems.
2- Ultimate ductility: The amount of maximum displacement or ductility capacity is an important factor in considering high-stress conditions and seismic activities (Figure 6.12 (b)).
3- Ultimate load capacity: the maximum bearable load in static/dynamic loading before failing should be taken into account of reinforcement design (Figure 6.12 (c)).
4- Surface retention: It is required for the local integrity of the rock faces and personal safety (Figure 6.12 (d)).
5- Longevity and sensitivity: These parameters should be evaluated in local high stresses, corrosive environment in rock masses, and bad quality of blasting that can make deterioration ground support systems (Figure 6.12 (e)).

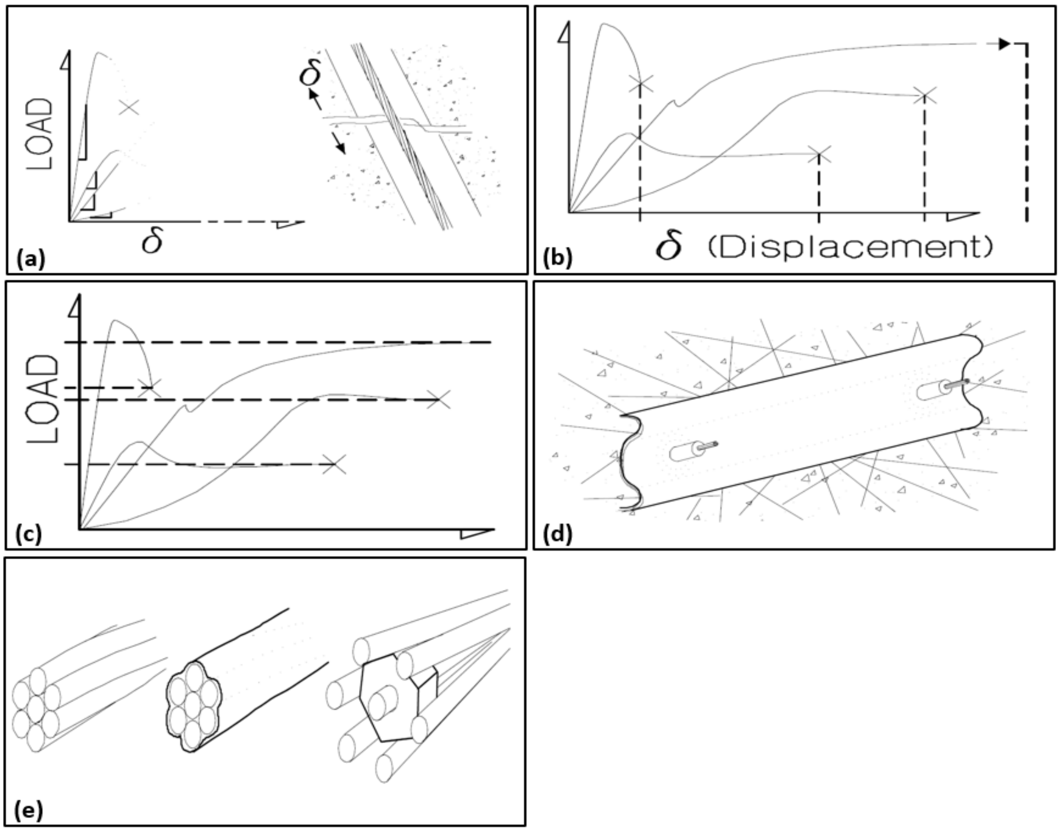


Figure 6.12. Main components in considerations of reinforcement system capacity (Hutchison, 1996)

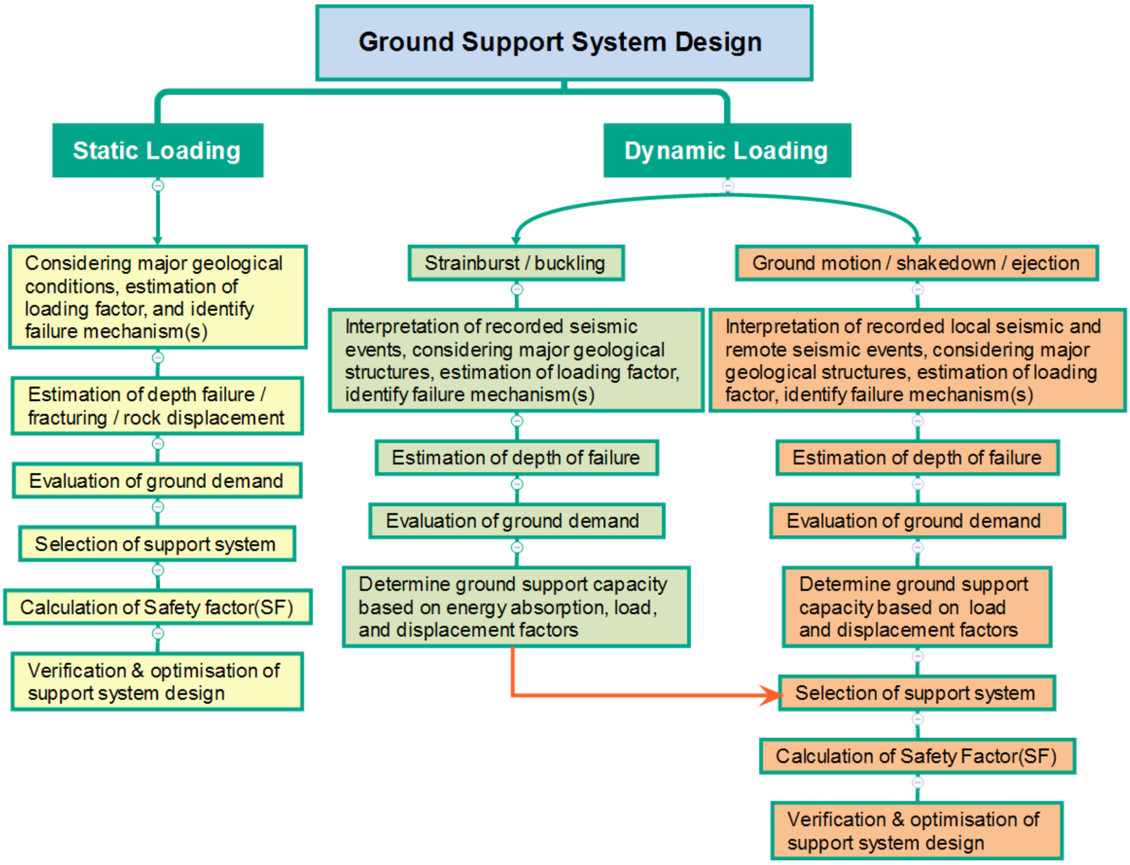


Figure 6.13. Ground support design in deep underground mines

6.3.1. Ground demand in static conditions

In static loading, the ground condition is associated with deformation and fracturing in rock mass structures, for example, plastic behaviour. The appropriate ground support can be surface and reinforcement types about the intensity of instability. Ground support design based on static loading condition is used in underground mining projects where the risk of seismic hazard is low. The typical ground support devices for static loading conditions are fibre–reinforced shotcrete, rock bolts and cable bolts (Jacobsson et al., 2013).

Ground support demand in static conditions is determined based on dead weight and stress concentration in rock masses surrounding excavations and is estimated by Eq. (6.1)(Cai and Kaiser, 2018). Support elements increase the frictional forces of rock blocks, resistance to deformation of the fractured rock mass and the support of the dead – weight surrounding an excavation.

$$\textbf{Ground Demand (static condition)} = \boldsymbol{\rho} \times \mathbf{g} \times \mathbf{d_f} \qquad \textbf{(6.1)}$$

Where; Static support demand: (kJ); ρ: Density of rock (Tonnes/m^3); g: Gravity of earth (m/s^2); and d_f: Displacement of rock/depth of failure (m).

6.3.2. Ground demand in dynamic conditions

Dynamic loading usually makes sudden failure in hard rock. Ground support devices in this condition should be able to absorb energy derived from seismic events. The specific system can be yielding bolts in the underground excavation.

Dynamic support demand stabilises a rock mass under dynamic loading conditions and dynamic failure mechanisms, and is estimated as below (Kaiser et al., 1996a):

$$\textbf{Ground Demand}(\textbf{dynamic condition}) = \frac{1}{2}\mathbf{mv^2} + \mathbf{qmgd} \qquad \textbf{(6.2)}$$

Where; Dynamic support demand: (kJ); m: mass of ejected rock materials (tons); v: velocity (m/s); q: constant factor for the effect of gravity on the ejected rock materials (m/s) (-1: floor, 0: wall, and 1: back); d: distance of ejected rock blocks (m); and g: gravity of earth (m/s^2).

The velocity (v) can be estimated from numerical modelling or seismicity event data and using equation (6.3) (Potvin et al., 2010):

$$\mathbf{ppv} = \frac{\mathbf{C \cdot 10^{\frac{1}{2}(m_L+1.5)}}}{\mathbf{R+R_0}} \qquad \textbf{(6.3)}$$

Where; C: 0.2 – 0.3 for design purposes; R: distance to the source; $\mathrm{R_0} = \alpha.\,10^{\frac{1}{3}(m_L+1.5)}$; and α: 0.53 – 1.14.

Figure 6.14 shows an estimation of failure depth in the dynamic rupture mechanism based on empirical data from previous projects. In the figure, CI is crack initiation threshold stress in rocks and is determined from laboratory tests. CI is about 0.4 – 0.5 for crystalline rocks (Diederichs, 2017).

The depth of failure where there is spalling behaviour and for a circular tunnel is estimated by the following equations (Diederichs, 2017):

$$\textbf{Depth of Failure } (\mathbf{D_f}) = \left[\mathbf{1} + (\mathbf{0.4}\boldsymbol{K}^{-\mathbf{0.27}} \times (\frac{\mathbf{3\sigma_1-\sigma_3}}{\mathbf{CI}} - \mathbf{1})^{\mathbf{0.65}\boldsymbol{K}^{\mathbf{0.14}}}\right] \times \mathbf{R} \qquad \textbf{(6.4)}$$

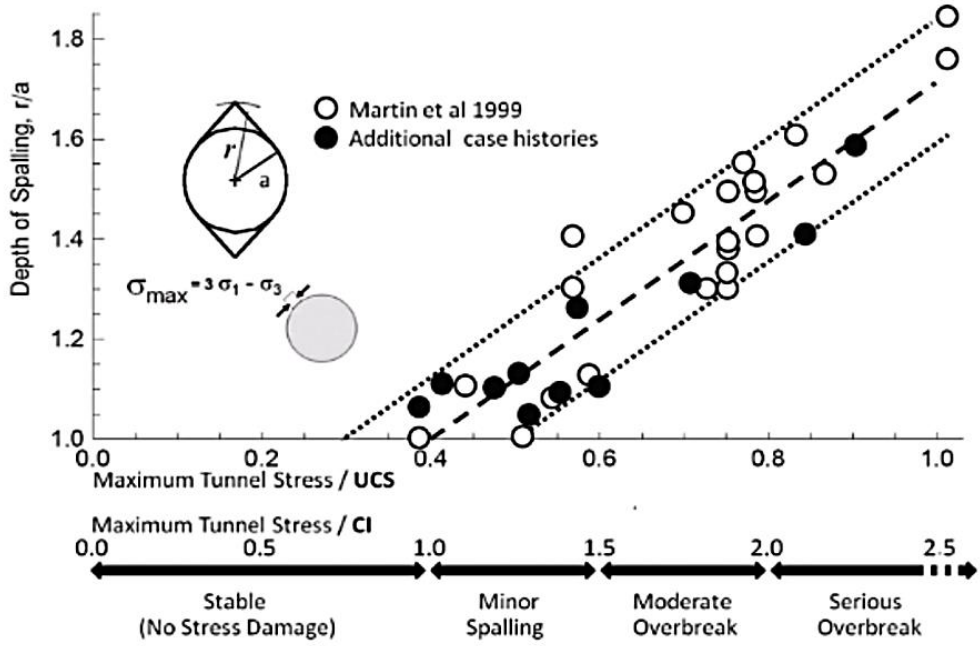


Figure 6.14. The estimation of depth failure in a dynamic loading condition (Diederichs and Martin, 2010)

Where; K: Stress ratio; CI: Crack initiation stress (for the case where there is no data available, use 0.5*UCS); R: Radius or half-span of an underground excavation.

The capacity for energy absorption of various surface and reinforcement support devices are shown in Table 6.1 and Figure 6.15, respectively. The implication is that yielding support devices such as Durbar, Cone bolt, Garford bolt and D-bolt is effective in dynamic and tensile loading, rockburst, and squeezing behaviour. Installing further rock bolts at an acute angle (less than 30^{o}) to the orientation of discontinuities is a solution for reducing shear failure (Stacey, 2016).

The result of drop–weight test indicates that about 25% and 75% of absorption of energy demand belong to surface support and rock bolt devices in hard rock condition respectively. In soft rock condition, this proportion is divided into 30% for rock bolts and 70% for surface support systems. Transferring load from the surface to reinforcement ground devices is not critical in the static condition, while this point is a fundamental requirement in dynamic condition to ensure about the performance of ground support systems (Louchnikov and Sandy, 2017).

Table 6.1. The capacity for energy absorption of different surface support elements (Louchnikov and Sandy, 2017)

Surface support	Energy absorption per unit area (kJ/m^2)	Maximum displacement at failure (mm)
FRS 60 mm, synthetic fibre	0.8	60
FRS 80 mm, synthetic fibre	2.2	80
FRS 110 mm, steel fibre and weld mesh embedded	7.0	120
Weld mesh 100 × 100 mm (5.6 mm wire)	1.3	210
FRS 60 mm + weld mesh over	2.1	210
FRS 80 mm + weld mesh over	3.5	210
M85/2.7 mesh (Minax high-tensile chain-link)	2.4	200
G80/4 mesh (Tecco high-tensile chain-link)	6.5	300
FRS 60 mm + M85/2.7	3.2	200
FRS 60 mm + G80/4	7.3	300
FRS 80 mm + M85/2.7	4.6	200
FRS 80 mm + G80/4	8.7	300
Woven mesh (6 mm wire) with welded double-wire on perimeter	2.0	300
HEA mesh	11.8	800
Woven mesh (10 mm wire)	22.5	600

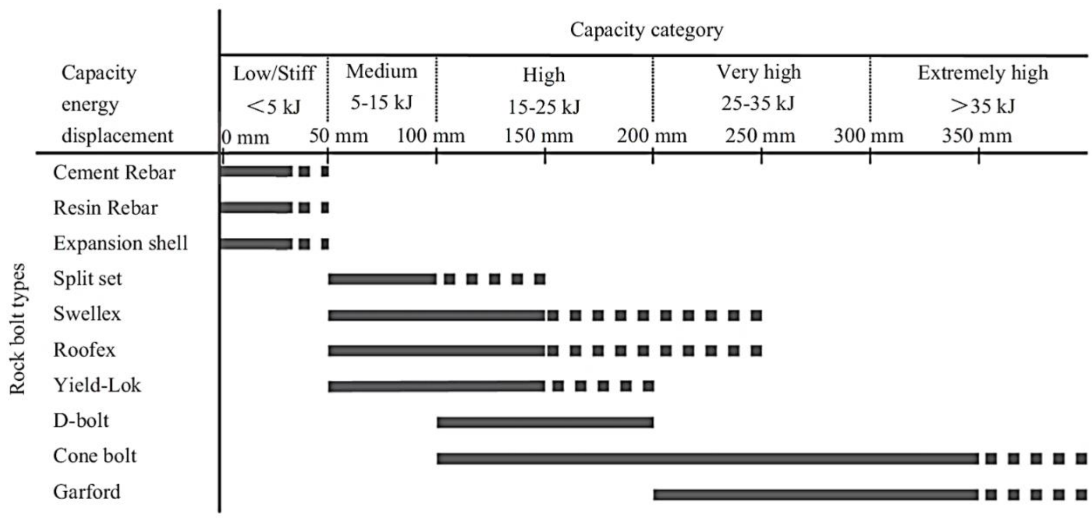


Figure 6.15. The energy absorption capacity of various reinforcement devices (Masoudi and Sharifzadeh, 2018)

The safety factor is a crucial criterion for stability analysis and design of ground support surrounding a rock mass in underground structures. This parameter estimates the load capacity of support devices under static and dynamic loading conditions. The factor of safety is estimated by Eq. (6.5):

$$\textbf{Safety Factor} = \left\{ \begin{array}{c} \dfrac{\textbf{Loading capacity of ground support system}}{\textbf{Total effective static loads on the ground surrounding excavation}} \\ \boldsymbol{or} \\ \dfrac{\textbf{Critical discplacement } (\mathbf{u_r})}{\textbf{Rock displacement at equilibirum } (\mathbf{u_{eq}})} \\ \boldsymbol{or} \\ \boldsymbol{min}\left(\dfrac{\boldsymbol{nE_{ab}}}{\boldsymbol{E_{ej}}}, \dfrac{\boldsymbol{u_{max}}}{\boldsymbol{u_{eq}}}, \dfrac{\boldsymbol{u_{ult}}}{\boldsymbol{u_{eq}}}\right) \end{array} \right\} > \mathbf{1} \qquad \textbf{(6.5)}$$

Where; $\mathrm{E_{ab}}(\text{Absorption energy capacity of ground support}) = \frac{1}{2}\mathrm{mv}^2$; n: the number of rock bolts; E_{ej}: kinetic energy from ejected rocks; u_{eq}: equilibrium displacement; u_{max}: maximum allowable displacement; and u_{ult}: ultimate displacement.

The required factor of safety for a long-term lifespan is between 1.5 and 3. At high-stress levels and soft – medium rock strength conditions, squeezing behaviour may happen with high-stress deformation. Critical displacement (u_c) is a suitable parameter to calculate the safety factor where there is squeezing behaviour. Also, under dynamic loading conditions, the capacity of ground support devices for absorbing energy should be higher than the ejected kinetic energy of rock masses. The ratio of energy absorption capacity by ground support devices to the kinetic energy of ejected rocks in a dynamic loading condition is used as the safety factor in burst-prone rocks (Li, 2017).

A safety factor of more than one may provide stability under dynamic loading conditions. However, ground control and management should be accompanied by field measurements to update ground support systems with any significant changes in the ground condition like the rate of seismic events.

6.3.3. Classification of ground support design in deep underground hard rocks

Table 6.2 presents design principles and a procedure for ground support and reinforcement in deep and hard rock conditions. The most effective steps in the design of ground support systems are:

1- Identification of the loading types
    - Static loading
    - Dynamic loading
2- Determination of the primary source of loading

- Origin of static loading: gravity, in situ /induced stress, tectonic activities, groundwater, residual stresses and temperature
- Origin of dynamic loading: Seismic events, strain burst, fault slip, pillar burst, gravity collapse, loading/unloading rate, blasting and earthquake

3- Geological structural condition
   - Description of the majority of the geological structure: massive rock, moderately jointed/blocky/folded rock, highly jointed/disintegrated rock
   - Favourability and unfavourability of significant structures in openings
   - Estimation of the block size surrounding openings
   - Determination of Continuity Factor (CF) in the ground

4- Estimation of the loading factor ($LF = \frac{Rock\ Mass\ Strength\ (\sigma_{cm})}{Major\ Principal\ Stress\ (\sigma_1)}$)
   - $LF > 2$; (Low level)
   - $1 < LF < 2$; (Medium level)
   - $LF < 1$; (High level)

5- Identification of potential failure based on loading type, loading source, major geological structural condition and loading factor
   - Primary failures
   - Secondary failures

6- Use of appropriate analysis and design methods based on failure modes in static and/or dynamic conditions

7- Selection of ground support systems (natural ground and/or artificial devices) under the required life term of excavations

The ground support and reinforcement system should be selected about durability and service life of underground excavations. Temporary support systems or natural ground are suitable for short-term, and permanent support systems are used in medium or long-term life.

In deep and hard rock conditions where there is frequently changing behaviour, rapid variations of stress and deformation, energy accumulation in rock masses, and application of fibrecrete, yielding rock bolts, cable bolts and mesh are necessary to stabilise openings. Seismic and deformation monitoring could be a useful strategy to control ground behaviour during mining operations.

It should be mentioned that the proposed method in Table 6.2 is more applicable for the strategic and tactical design of underground mine projects. Verification and optimisation of design parameters should be accomplished in the operational design stage.

6.4. Case studies of ground control and management strategies

Design analysis for managing the ground condition following the proposed methods are examined in deep underground excavations.

6.4.1. Case example C

The example analyses ground behaviour and failure mechanism in case example C from Chapter 5. Design analysis for ground control and management was considered for main caverns of the hydropower project. Rock mass behaviour and failure mechanism have assessed a type of complex ground condition. The typical failure modes were ground fall, spalling, rock burst and squeezing. Also, the dimension of underground excavations is a type of large span. The Q classification system was used for the estimation of ground support system in underground openings. The main rock units and weakness zones classified Q: 10–40 very good and Q: 0.5-2 poor respectively. The blasting method and smooth blasting method for the final surface to diminish damage rock zones are used for excavation of projects. Figure 6.16 shows a summary of ground control and management flowchart in the project. Sprayed shotcrete on the surface after excavation is a useful method to control cracking and fracturing rock mass. Rock bolts and concrete is used to avoid loosening rock structures and ground fall at crown and walls. Meanwhile, pre-stress rock bolts, grouted bolts and cables are installed to reinforce weakness zones.

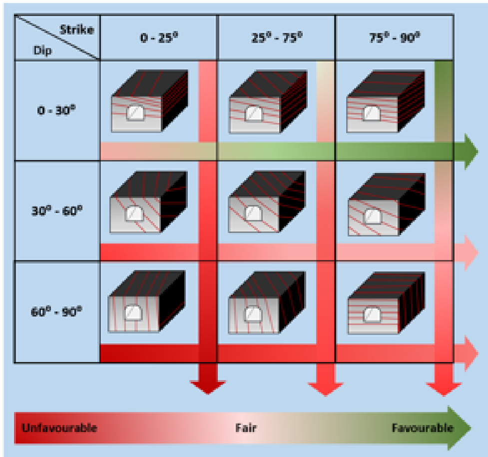
Table 6.2. Design principles and procedure of ground support and reinforcement systems in deep and hard rock conditions (1)

Loading Types	Origin of Loading	Geological Structural Condition	Load Factor	Potential Failures		Appropriate Analysis and Design Methods	Suggested Ground Support System
				Preliminary	Secondary		
Static	• Gravitation • In situ/Induced stress • Tectonic activities • Groundwater • Residual • Temperature	Massive Rock GSI > 70 Q' > 40 B_s (Block Size) > $10m^3$ CF < 3	$\sigma_{cm}/\sigma_1 > 2$	Stable	Local block fall, sliding fall	Rigid limit equilibrium method, Key block theory	Stable, sealing surface with spot bolting and mesh, if needed
			$1 < \sigma_{cm}/\sigma_1 < 2$	Stress induced failure, progressive failure,	Block fall, minor slabbing, spalling, bulking failure, popping / shucking small rock fragments, strain burst	Analytical methods, tensile cracking analysis, combination of limit equilibrium and energy release method, continuity deformation analysis, failure approach index analysis, energy release rate analysis, observational method	Unify zones of failure with mesh and bolting in appropriate spacing and length (at least 1/3 the span of excavation), deformation control with pre – tensioned rockbolts and retained with shotcrete, use D-bolts/Cone bolt/Garford dynamic bolt, face and pillars in ore drives retained with mesh
			$\sigma_{cm}/\sigma_1 < 1$	Stress induced failure ,fracturing and brittle failure, severity sudden failure	Damage microseismic, block fall, brittle failure, tensile failure, popping/shucking rock fragments, strain burst, pillar burst	Energy release rate analysis, deformation analysis, rock burst tendency index, local energy release rate analysis, continuity deformation analysis, observational methods, expert system, integrated systems approaches	Reduce stress concentration, using back fill as a regional support in mining sequences, Adjustment of pillar size in underground mining methods, scale first and then install support elements, retain ejected rock with fibrecrete and mesh, using yielding devices to absorb released energy, yielding steel sets, flexible devices to absorb shocks from seismic events, split sets are good which are be able to slip under dynamic loading, use D-bolt/Garford dynamic bolt/Kinloc bolt/Cone bolt/cable bolts, using steel sets, seismic monitoring
		Moderated Jointed/Blocky/ Folded Rock Favourability and unfavourability of major structures[1] 45 < GSI < 70 4 < Q' < 40 $100dm^3 < B_s < 10m^3$ 3 < CF < 35	$\sigma_{cm}/\sigma_1 > 2$	Structure induced failure, block fall	Toppling, block fall, sliding failure, wedge failure	Continuity – discontinuity deformation analysis, key block theory, rigid limit equilibrium method, analytical methods, bending analysis, fracture mechanics analysis, finite element methods, failure approach index analysis	Flexible support, sewing layers to each other, pre – tensile rock bolts, application of split sets for small scale of failure, holding rock mass with mechanical rock bolts, seal surface with shotcrete to prevent failure, support devices should be installed before ground movement, fibrecrete and rockbolts/cable bolts, the length of rockbolts should be at least 1/3 of the span, using cement rebar/resin rebar/split set/swellex/friction bolt with 2m or less spacing with mesh and straps for unify zone of failure, face bolted and meshed
			$1 < \sigma_{cm}/\sigma_1 < 2$	Stress/structure induced failure, block fall, shear failure, tensile failure	Block fall, progressive shear failure, shallow stress induced brittle and shear failure, flaking rock mass/ splitting failure, tensile failure, buckling failure, toppling failure, bending failure	Analytical methods, observational methods, shear stress analysis, expert system, failure approach index analysis, discontinuity deformation analysis, finite element analysis, distinct element methods, soft computation, neural networks, integrated approach systems, fracture mechanics analysis	Prestressed reinforced devices, unify zone of failure, scale rock mass surrounding excavation before installing support tools to unify rock zone, use straps and wire mesh across bedding planes and joints to prevent skin failure between rockbolts, reinforce rock mass with rockbolts to limit displacement and buckling, grouted rock anchors, , pre-tensioned rockbolts, sometimes need to use steel support to control shear failure/plastic behaviour/large deformation, shotcrete with mesh and straps for permanent support system, stabilizing pillars with mesh and rock bolts, leaving extra pillars, friction bolt/cemented/grouted bolt/resin bolts/split sets/cable bolts with spacing less than 1.5m, survive large rock deformations, using back fill as regional support in mining sequences
			$\sigma_{cm}/\sigma_1 < 1$	Stress/structure induced failure, shear failure, large deformation, block fall	Crushing and splitting of rock blocks, tensile failure, strain burst, pillar burst, buckling failure	Discontinuity deformation analysis, analytical methods, bending analysis, finite element analysis, fracture mechanics analysis, failure approach index analysis, expert system, observational methods, observation methods	Stiff support, unify zone of failure, scaling well and then install support devices, stiff support, full column ground rock anchors, thick steel fibre reinforced shotcrete, yielding steel ribs, cable bolts, provide maximum holding capacity, split sets, using back fill as regional support in mining sequences, expansion rebar/split set/Roofex/Yield-Lok/D-bolt with spacing less than 1m with mesh and straps, flexible steel sets, survive ground movement and large deformation
		Highly Jointed / Disintegrated Rock GSI < 45 Q' < 4 $B_s < 100dm^3$ CF > 35	$\sigma_{cm}/\sigma_1 > 2$	Structure induced failure, ground fall, wedge failure,	Cave in, block fall, wedge failure, chimney failure, notch failure, cave in	Discontinuity deformation analysis, shear stress analysis, failure approach index analysis, finite element analysis, distinct element analysis, bending analysis, neural networks	Unify zone of failure, grouted ground for unify zone of failure, pattern support with grouted rock bolts such as split sets, retaining rock mass with shotcrete and mesh, steel support with struts, swellex/roofex/d-bolt/hybrid bolts/friction bolts with spacing about 2m, survive ground movements
			$1 < \sigma_{cm}/\sigma_1 < 2$	Structure/stress induced failure, large deformation failure, shear failure	Ground fall, plastic failure, chimney type failure, ground movement, shear failure, buckling, tensile failure, ravelling, flowing	Discontinuity deformation analysis, shear stress analysis, expert system, observation methods, failure approach index analysis, soft computing, finite element analysis, distinct element analysis, integrated approach systems approaches, expert system	Grouted ground for unify zone of failure, steel support with pre-tensioned rock bolts, rockbolts or cable to control a separation, flexible steel sets, survive large scale displacements in rock masses, fibrecrete plus mesh and straps, friction bolt/hybrid bolts/grouted bolts with spacing less than 2m, reinforced pillars with fibrecrete and mesh, monitoring ground deformation
			$\sigma_{cm}/\sigma_1 < 1$	Stress/structure induced failure, large deformation failure, ravelling and flowing ground in brecciated and disintegrated ground,	Chimney type failure, ground fall, buckling failure, splitting failure, shear failure, large strains, floor heave and sidewall closure	Discontinuity deformation analysis, shear stress analysis, expert system, observation methods, failure approach index analysis, soft computing, finite element analysis, distinct element analysis, local energy release rate, integrated approach systems approaches	Grouted rock anchor, steel support with pre-tensioned rock bolts, grouted highly ductile rock anchor and steel fiber reinforced shotcrete, for swelling condition: full-column grouted rock anchors with fibre reinforced shotcrete, For ravelling condition: steel support with struts, pre tensioned rock bolts with fiber reinforced shotcrete, steel sets are required for long – term support, using back fill as regional support in mining sequences, reinforced pillars with fibrecrete and mesh, resin bolt/expansion shell/split sets/swellex/grouted cable bolts with spacing about 1m, monitoring rock mass deformation

Table 6.2. Design principles and procedure of ground support and reinforcement systems in deep and hard rock conditions (2)

Loading Types	Origin of Loading	Geological Structural Condition	Load Factor	Potential Failures: Preliminary	Potential Failures: Secondary	Appropriate Analysis and Design Methods	Suggested Ground Support System
	• Seismic events	Massive Rock GSI > 70 Q' > 40 $B_s > 10m^3$ CF < 3	$\sigma_{cm}/\sigma_1 > 2$	Seismicity damage, strain burst, tensile failure,	Block fall, sliding failure, brittle failure, blast damage, rock ejection, shear failure, pooping/shucking rock, sudden failure, pillar burst	Observational methods, engineering judgment, finite element methods, distinct element methods, fuzzy logic, energy release rate analysis, rock burst tendency index, local energy release rate, integrate system approaches, soft computations	Retaining rock mass with wire mesh, reinforced with strong yielding rockbolts and grouted rebar, steel fibre reinforced shotcrete, split sets in minor dynamic loading, high density cable bolting in high level of seismic events, flexible steel sets to absorb released energy, stabilizing pillars with dynamic bolts and mesh, leaving extra pillars, fibrecrete plus multi-layer mesh and dynamic bolts (D-bolt/Garfod bolt/Cone bolt with spacing less than 1-2m), face and ore-drive meshed and used bolts/cable bolts, survive seismic and displacement monitoring
	• Strain burst		$1 < \sigma_{cm}/\sigma_1 < 2$	Seismicity damage, brittle failure, spalling, slabbing popping/shucking rock, sudden failure	Block fall, sliding failure, bulking failure, blast damage, splitting failure, tensile failure, strain burst, pillar burst		
	• Fault slip		$\sigma_{cm}/\sigma_1 < 1$	Seismicity damage, brittle failure, server rock burst, rock ejection	Block fall, brittle failure, shear failure, blast damage, splitting, pillar burst, strain burst		
	• Pillar burst	Moderated Jointed / Blocky/ Folded Rock Favourability and unfavourability of major structures[(1)] 45 < GSI < 70 4 < Q' < 40 $100dm^3 < B_s < 10m^3$ 3 < CF < 35	$\sigma_{cm}/\sigma_1 > 2$	Structure/seismicity induced failure, block fall, shear failure,	Toppling, block fall, sliding failure, wedge failure, blast damage, shear failure, splitting failure, large deformation, pillar failure,	Observational methods, engineering judgement, finite element methods, distinct element methods, fuzzy logic, failure approach index analysis , local energy release rate, integrate system approaches, soft computations, discontinuities deformation analysis, distinct elements methods	Retaining rock mass with fibrecrete plus chain link mesh, reinforced with strong yielding rockbolts and grouted rebar, split sets in minor dynamic loading, high density cable bolting in high level of seismic events, flexible steel sets to absorb released energy and control deformation, using back fill as regional support, grouted rock bolts and cable bolts, Seismic monitoring and deformation control, reinforced pillars with fibrecrete and mesh
Dynamic	• Gravity collapse • Loading/Unloading rate		$1 < \sigma_{cm}/\sigma_1 < 2$	Stress/structure/seismicity induced failure, block fall, shear failure, tensile failure,	Block fall, progressive shear failure, brittle and shear failure, tensile failure, buckling failure, toppling failure, bending failure, pillar failure, cave in, large deformation		
	• Blasting		$\sigma_{cm}/\sigma_1 < 1$	Stress/structure/seismicity induced failure, shear failure, large deformation, block fall	Crushing and splitting of rock blocks, tensile failure, blast damage, , strain burst, pillar burst, buckling failure, cave in, ravelling, flowing, pillar failure, large scale collapse		
	• Earthquake	Highly Jointed / Disintegrated Rock GSI < 45 Q' < 4 $B_s < 100dm^3$ CF > 35	$\sigma_{cm}/\sigma_1 > 2$	Structure/stress/seismicity induced failure, ground fall, large deformation failure, shear failure ravelling, flowing	Cave in, ground fall, chimney failure, notch failure, blast damage ground movement, shear failure splitting failure, shear failure, large strains, floor heave and sidewall closure, ravelling, flowing	Observational methods, engineering judgment, finite element methods, distinct element methods, energy integrate system approaches, soft computations, discontinuity deformation analysis, shear stress analysis, finite element methods, soft computing, failure approach index analysis	Reinforced rock mass with resin bolt/expansion shell/split sets/swellex/grouted cable bolts with spacing about 1m, steel support with pre-tensioned rock bolts, for swelling condition: full-column grouted rock anchors with fibre reinforced shotcrete, for ravelling condition: steel support with struts, pre tensioned rock bolts with fiber reinforced shotcrete, steel sets are required for long – term support, using back fill as regional support in mining sequences, reinforced pillars with fibrecrete and mesh, monitoring rock mass deformation
			$1 < \sigma_{cm}/\sigma_1 < 2$				
			$\sigma_{cm}/\sigma_1 < 1$				

- $Loading\ Factor\ (LF) = \frac{Rock\ Mass\ Strength\ (\sigma_{cm})}{Major\ Principal\ Stress\ (\sigma_1)}$
- GSI: Geological Strength Index;
- $Q' = \frac{RQD}{J_n} \times \frac{J_r}{J_a}$
- Continuity Factor $(CF) = \frac{Dimension\ of\ Excavation}{Dimension\ of\ Block}$
- B_s: Block Size



[(1)]Favourability and Unfavourability of Major Geological Structures in Underground Excavations

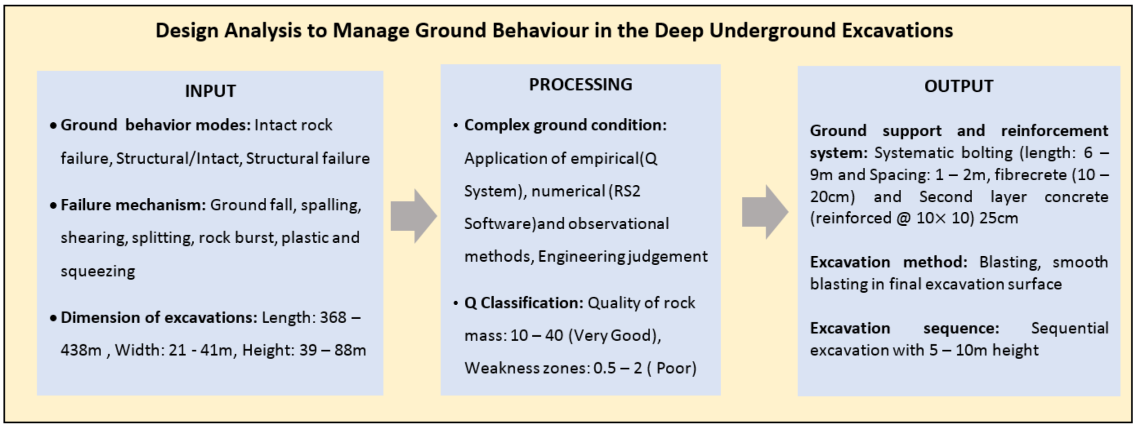


Figure 6.16. Applied ground control and management strategies in underground excavations of case example A

The RS2 software was used for stability analysis of or rock mass surrounding excavation. This software is a powerful 2D finite element program for numerical analysis of rock projects in a wide range of rock engineering projects including excavation design, ground support design, slope stability, dynamic analysis, groundwater seepage and so on. One of the major features of RS2 is creating a multistage model for the analysis of progressive failure, support interaction and stability (Rocscience, 2019).

The ratio of σ_{ci}/σ_{cm} was estimated between 5 and 10 for the blocky rock mass structures. Major principal stress measured 33 MPa, and the ratio of strength/stress was determined in a range of 0.4 to 6. This ratio shows the potential sudden failure modes surrounding excavations. Ground loading factor identifies a potential of sudden failure in the underground excavations. The Numerical modelling was analysed by the selection of input parameters as intact rock strength: 145MPa, Young's Modulus: 27GPa, Poisson's ratio: 0.27, and major principal stress: 33MPa. Figure 6.17 shows the result of the numerical modelling of stability analysis of the rock mass structures surrounding excavation. Based on the numerical simulation, there is a potential failure at crown and wall of excavation which has been shown some examples of occurring failures during excavation and after installing the first layer of shotcrete and rock bolts in Figure 6.17 (c), (d) and (e). The results of numerical modelling and observational methods demonstrated occurrence sudden failure modes perpendicular to principal stresses in the crown and sidewall. Also, the ratio of rock mass strength to stress by numerical modelling estimated between 0.35 to 1.6 surrounding excavation. 10-20 cm fibrecrete and 6–9m length systematic bolting with spacing 1–2m was selected as the ground support system. Application of all ground support devices provides stability in the rocks. However, site monitoring and observational methods are required to manage and control ground behaviour.

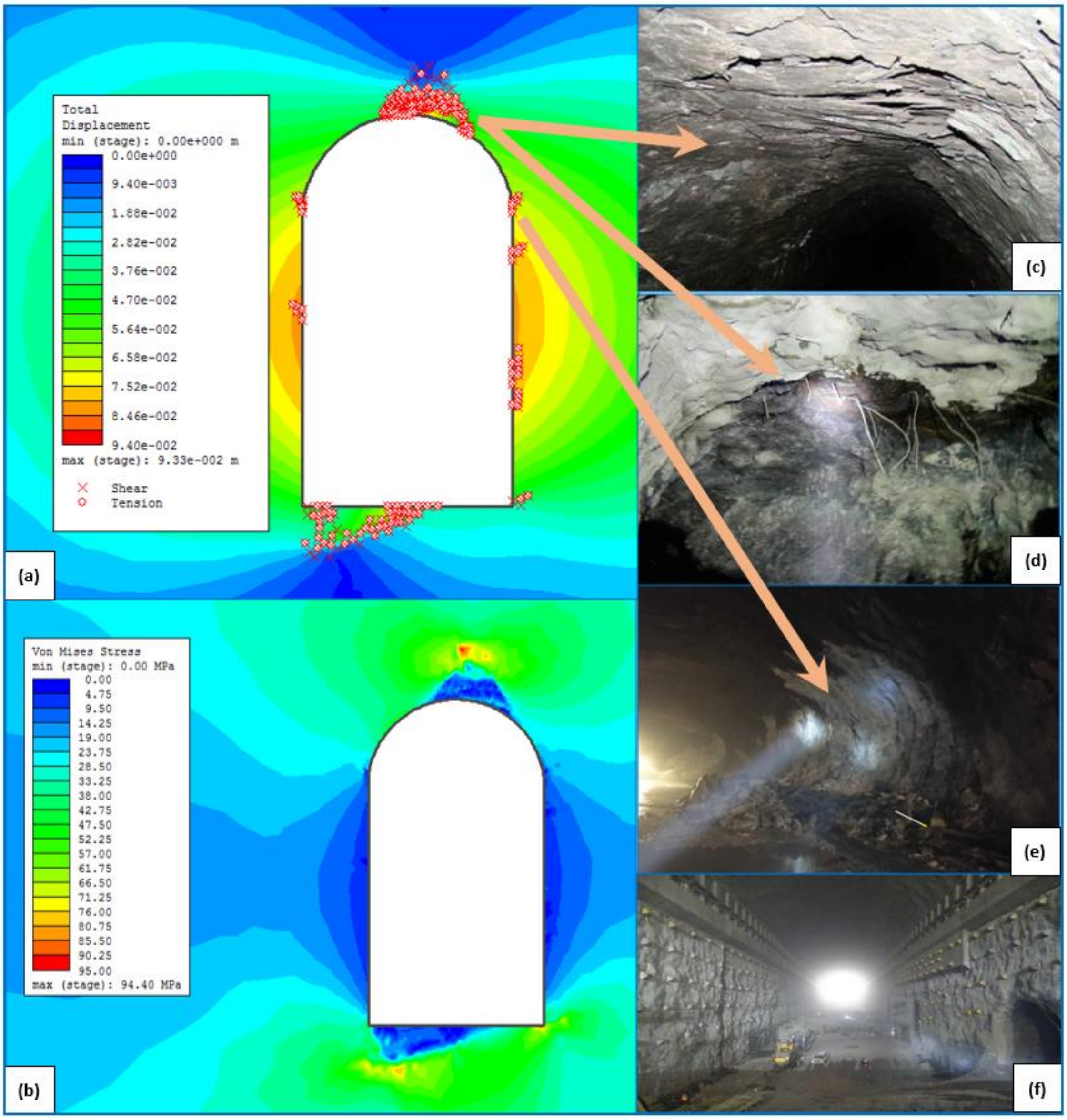


Figure 6.17. Design analysis for ground management by numerical modelling; (a) displacement of rock mass (b) distribution of stress surrounding excavation, (c) failure at crown before installing first layer of shotcrete and rock bolts, (d) failure at crow after installing ground support, (e) failure at a side crown, (f) underground excavation after installing ground support system.

6.4.2. Case example E (Deep underground mines in Western Australia)

Mine (I): The mine geology consists of mafic to volcanic and volcanoclastic sedimentary, shale and conglomerate rocks. Major geological structures are western shear zone and eastern, horizontal fault and thrust fault. Sublevel stopping method is used for the extraction of mineral resources. Stope dimensions in the mine site are typically 30 m long and 20 m high. Pillars as natural ground support are implemented in low grade and uneconomic zones. Typical failure modes in the mine site are structural failures (ground fall and wedge failure) and stress-induced failure types (slabbing, pillar failure and squeezing failure).

Mine (II): The gold mine hosted in Devonian carbonaceous metasediments units. The mineralisation consists of pyrrhotite, arsenopyrite and chalcopyrite. The orebody is mined using the underground long – hole open stoping method. The main challenges associated with the mining operations are a high degree of jointing in the rock mass, several existing shear zones, instability of the rock mass and dilution of the ore body in stopes. The record of seismic events indicates that the mine area is in low to moderate levels of seismicity. The most failure modes are structural and induced stress failure modes.

Mine (III): The gold mine deposit is hosted in mafic stratigraphical units, which are coarse grain and massive basalt units. Gold mineralisation is related to sphalerite, galena and scheelite mineralisation, and it is mostly hosted in laminated quartz veins. Three geotechnical domains at the mine site are hangingwall basalt, the ore body (dolerite, basalts and shear zones) and footwall basalts. The Q-value of the rock masses was estimated to be in the range of 4 to 30. Failure mechanisms in the rock masses include mining induced stress, gravity and blasting, and cause wedge failure, ground fall, slabbing, shear slip and pillar failure. The seismicity of the mine site is low to moderate.

Mine (IV): Nickel ore as the main resource is hosted in nickel-rich lava rivers and nickel placer deposits. The nickel ore contains a band of massive sulphide, overlain by matrix ore, overlain by disseminated ore. The main rock types are basalt, talc-chlorite ultramafic, antigorite ultramafic, porphyry-felsic and porphyry– intermediate. Mineral resources at the mine site are extracted by the long hole and cut & fill mining methods. There are several faults, shear zones and porphyry dykes in the mine area. There is a low rate of groundwater inflow (3 to 5 m^3/day) from the ore surface and hanging walls during the rainy seasons. Seismic events caused a sudden fracture, creating new joints and failures in rock zones surrounding excavations. Strain burst, pillar burst, fault slip, shear failure, floor heave, stress-induced failure and squeezing failure occurred during engineering operations.

Mine (V): The gold deposit consists of multiple shallow dipping ore zones of gold mineralisation and hosted by mafic and conglomerate. The main rock types are basalt, conglomerate, siltstone, sandstone and shale. Major structures at the mine site are discontinuity sets, fault zones and weakness zones. The quality of rock mass, based on the Q system, was estimated to be between 2 and 19. Rock noise was recorded in underground stops in some cases before the occurrence of rock failure. During mining operation, several rocks falls, and rock bursts occurred. Failure modes at the mine site were classified into gravity, induced stress, and seismicity types. In some cases, unravelling happened in rock zones with high degrees of jointing. Also, seismic events caused slabbing, strain bursts, rock bursts and ground falls.

The summary of geological information and geotechnical properties in the mines sites are presented in Table 6.3 and Table 6.4. Also, some typical failure modes that occurred in deep underground mines in Western Australia are shown in Figure 6.18.

The design of ground support systems for the case studies was evaluated based on the proposed method in Table 6.2. The main source of loading at the mine site was identified as gravity, tectonic activities, seismic events, fault slip, strain burst and blasting damage. The geological structural condition was mainly moderately jointed/blocky rocks, and the GSI and Q-value was estimated in the range of 30-80 and 1-48, respectively. The results of the design of the ground support system at some deep underground mines in Western Australia are summarised in Table 6.5. The loading factor was between 0.9 and 2.3. Therefore, the rock mass structures have the potential to suddenly failure. Site investigations and observational methods indicated that the primary failure modes were mostly block fall, wedge failure, induced stress failure, shear failure, slabbing and rock burst failure modes. Also, during the mining operation, secondary failure modes like squeezing failure and pillar failure occurred due to seismic events, induced stresses and blasting damage in rock zones surrounding excavations.

Table 6.3. The summary of geological information of Western Australian's mines

Mine site	Mineral Resources	Lithology	Mining method	Major geological structures
Mine (I)	Nickel	Mafic to felsic volcanic rocks, volcanoclastic sedimentary rocks, conglomerate, ultramafic rocks, massive sulphide mineralization	Downhole bench stopping method	Fault, shear zone, discontinuity sets
Mine (II)	Gold	Pyrite, arsenopyrite, chalcopyrite, coarse crystalline arsenopyrite	Long hole open stopping method	Discrete striking and dipping fault structures, shear zones
Mine (III)	Gold	Basalt, laminated quartz, sphalerite, galena,	Sublevel stop mining method	Anticline, faults, ductile structures, foliated zones
Mine (IV)	Nickel	Basalt, talc – chlorite, ultramafic, antigorite, porphyry – felsic, and porphyry intermediate	Long hole and cut & fill mining method	Fault, shear zone, porphyry dykes, joint sets
Mine (V)	Gold	Basalt, conglomerate, siltstone, sandstone, and shale	Long hole open stoping method	Discontinuity sets, faults, shear zones,

Table 6.4. Rock engineering properties at deep underground mine, case studies in Western Australia

Mine Site	Depth (m)	UCS (MPa)	E (GPa)	v	Joint Set1 (Dip/Dip direction)	Joint Set2 (Dip/Dip direction)	Joint Set3 (Dip/Dip direction)	σ_1 (MPa)	σ_2 (MPa)	σ_3 (MPa)
Mine (I)	650	120-160	50-70	0.30	65 / 85	80/175	51/263	40	32	7
Mine (II)	950	70-100	30-40	0.28	49/50	55/001	82/182	55	39	18
Mine (III)	1300	135-170	55-75	0.32	35/37	13/339	48/225	70	56	25
Mine (IV)	800	130-220	45-80	0.34	67/304	74/140	83/86	56	37	21
Mine (V)	780	160-240	63-80	0.33	49/350	4/57	81/109	51	31	19

Ground support elements were selected based on the estimation of static and dynamic ground support demand in each mine site. Fibrecrete with mesh as a surface support system, and friction bolt split sets, grouted rebars and cable bolts as reinforcement tools were selected as ground support systems for stabilising rock mass structures. Figure 6.19 shows the results of numerical modelling of the main decline access with 5.2 m width and 5.7 m height in Mine (III). Figure 6.19–b is the estimation of the plastic zone (failure zone) surrounding excavation, which is about 1.5m.

Figure 6.18. Some typical failures in deep underground mines in Western Australia: (a) ground fall, (b) rock burst, (c) wedge failure, (d) blocky undercutting, (e) bulking, (f) pillar burst

Table 6.5. The result of the design of the ground support system in five deep underground mines in Western Australia

Mine Site	Source of Loading	Geological Structural Condition	Load Factor	Potential Failures		Depth of Failure/Fracturing (m)		PPV Estimation	Estimation of Ground Support Demand		Ground Support System
				Primary	Secondary	Estimation	Observation		Static	Dynamic	
Mine (I)	Gravitation, field stress, tectonic activities, seismic events, pillar burst, blasting	Moderated jointed/blocky/folded rock 53< GSI<77 4.5<Q'<27 $0.5<B_s<9$ 4<CF<10	$1 < \sigma_{cm}/\sigma_1 < 1.3$	Wedge failure, block fall, induced stress failure, tensile failure	Pillar failure, slabbing, brittle failure, unravelling, squeezing failure	0.6-0.9	0.5-1.5	1.1m/s	41KN/m²	11KJ/m²	50mm Fibrecrete with mesh, 2.4m Friction bolts (1.2m×1.2m), 2.4 Resin bolts (1m×1m), 6-9m Cable bolts (2m×2m)(where required), face meshed for drives
Mine (II)	Gravitation, induced stress, tectonic activities, fault slip, pillar burst, blasting	Moderated – Highly jointed 30< GSI<77 1<Q'<10 $0.1<B_s<7$ 6<CF<19	$0.6 < \sigma_{cm}/\sigma_1 < 1$	Shear failure, block fall, tensile failure, large deformation failure, unravelling, pillar failure	Plastic failure, squeezing failure, splitting failure, chimney failure	0.8-1.3	0.5-1.7	0.7m/s	47KN/m²	10.6KJ/m²	75-100mm Fibrecrete with mesh, 3m Resin bolts (1.3m×1.3m), 2.4m D-bolts (1.3m×1.3m), 8m Cable bolts (where required), applied mesh and fibrecrete for pillars
Mine (III)	Gravitation, induced stress, tectonic activities, fault slip, blasting damage, strain burst	Massive – Moderated jointed/blocky rock 58< GSI<83 14<Q'<48 $3.7<B_s<12.5$ 2<CF<6	$0.9 < \sigma_{cm}/\sigma_1 < 1.2$	Large wedge failure, shear slip, slabbing, rock burst	Pillar burst, brittle failure, ground fall, popping rock fragments, strain burst	1-1.5m	0.5-1.2	1.5m/s	36KN/m²	11.5KJ/m²	2.4m Galvanised Friction bolt (1.2m ×1.2m), 2.4m Grouted Split sets (1-1.3m×1-1.3m), 9m Garford cable (2m×2m) and mesh, face bolted and mesh
Mine (IV)	Strain burst, pillar burst, seismic events, gravitation, stress induced, ground water	Moderated jointed/blocky rock 47< GSI<75 3<Q'<27 $1<B_s<9$ 5<CF<17	$1.1 < \sigma_{cm}/\sigma_1 < 2$	Stress induced failure, wedge gravity failure, strain burst, shear failure, ground movement,	Floor heave failure, squeezing failure, crown pillar failure, blast damage rock, fault slip	0.7-1.1	0.3-1.4	1.8m/s	39KN/m²	14.4KJ/m²	2.4m Grouted rebar (1.5m ×1.5m), 50mm Fibrecrete with mesh, 3m and 2.4m Grouted Split sets (1.5m ×1.5m), 6m Cable bolts (1.5m ×1.5m), mesh
Mine (V)	Seismic events, stress induced, gravitation, tectonic activities	Moderated jointed/blocky rock 43< GSI<65 13<Q'<35 $2<B_s<12$ 3<CF<14	$1.5 < \sigma_{cm}/\sigma_1 < 2.3$	Gravity driven large wedge failure, shallow dipping wedge failure, slabbing, rock burst,	Ground fall, unravelling, strain burst cracking, seismically induced wedge failure, buckling, blast damage	0.5-0.9	0.4-1.3	1.4m/s	37KN/m²	11.2KJ/m²	50-100mm Fibrecrete, mesh, 2.4m Resin bolts (1.1m ×1.1m), 2.4m Friction bolt (1.5m ×1.5m), 2.4m split set(1.4m ×1.4m) where required, 5-8m Cable bolt(2m×2m) where required)

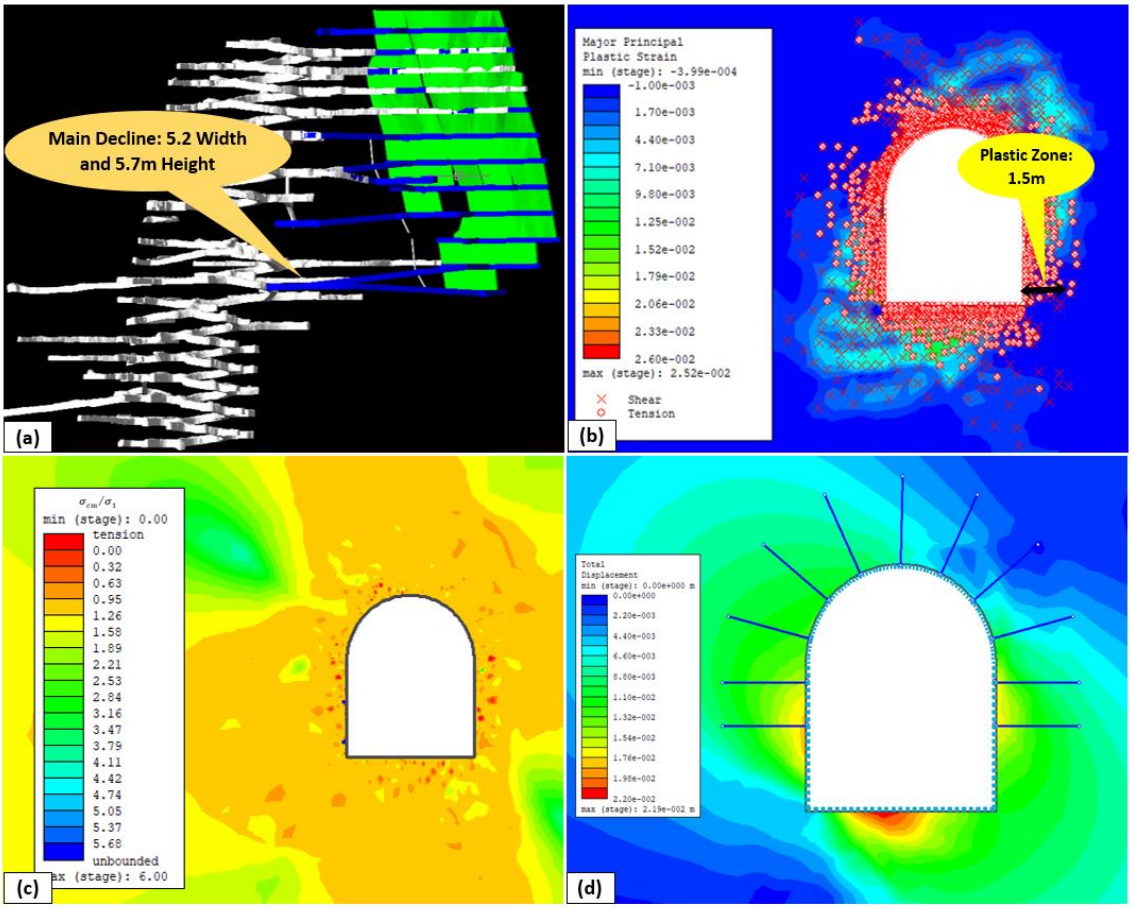


Figure 6.19. Numerical modelling of main decline of Mine C; (a) main decline access, (b) plastic zone, (c)loading factor (σ_{cm}/σ_1), and (d) total displacement after installing ground support system

The result of numerical modelling demonstrates the reliability estimation of failure depth compared with empirical methods (1 m–1.5 m) and observational methods (0.5 m–1.2m). Also, the ratio of safety factor/loading factor (σ_{cm}/σ_1) is presented in Figure 6.19–c. Maximum displacement of rocks surrounding excavation was estimated 2.2 cm after installing ground support system (Figure 6.19–d). The results of numerical modelling demonstrated the stability of rock masses surrounding excavation after installation ground support systems.

6.5. Conclusion

Deep rock underground excavations are usually associated with a high-stress environment and seismic events. Severe damage in rock mass structures and ground support systems may occur due to large magnitude seismic events, defects in rock mass structures, stress concentration, blasting damage and tectonic activities such as strike-slip faults. Utilisation of proper ground control and management strategy avoids the risk of failure. Ground control and amendment strategy of deep hard rock were proposed regarding the design, construction and serviceability stages of works. Collecting comprehensive data, diagnosis of hazardous conditions and failure mechanisms, design analysis, and selecting stabilisation methods are conducted in the design phase. Determination of safe work procedures, training personnel, identify hazard conditions, quality control and quality assurance of materials, and safety analysis before ground failure is essential in the construction stage. Control of the ground condition during serviceability (short, medium and long-term) is focused on monitoring (seismic events and load–deformations), maintenance, rehabilitation, seismic monitoring and contingency planning.

The critical factors in the design stage of in-depth underground mining projects are to establish suitable location and layout of openings; determine suitable excavation method, sequential excavation and extraction ratio; and selection of proper ground support equipment for small and/or large-scale deformation. Microseismic and blast monitoring throughout the mining operations are required to control sudden failures. Sequential excavation for mining purposes utilises the top – down, bottom – up, centre – out and abutment – centre methods to deal with stress concentration and instability in large-scale mine stopes.

Also, a procedure for ground support design in deep and hard rock was presented. The main principles in the proposed method were

- ground loading types and sources,
- characterisation of the major geological structural condition,
- determination of ground load factor,
- identification of primary and secondary potential failure,
- selection of appropriate design analysis for static and/or dynamic loading conditions,
- estimation of static and/or dynamic support demand, and
- selection of surface and reinforcement support elements based on their capacity for energy absorption and safety factor.

At low-stress levels, the dominant loading source is the gravitational force and ground support elements should be selected based on their capacity for energy dissipation. The behaviour of rock masses and the failure mechanism is complex in high rock stresses and dynamic loading conditions due to released strain energy from seismic events, strain burst, fault slip and pillar burst. The support elements are selected by their capacity for energy absorption factor in rock mass structures.

A number of deep underground excavations were studied. The mine sites have hard rock and high field stress. For ground support design, the geological structures were characterised and the potential failure modes were identified. Wedge failure, block fall, squeezing, rock burst, ravelling, pillar burst, slabbing and blast damage are the common types of failure at the mine sites. Also, the depth of failure based on observational methods, empirical methods and numerical methods were estimated in the range of 0.3 m to 1.7 m in the main decline access with 5.2 m width and 5.7 m height in Western Australian underground mines. Static and dynamic ground support demand was calculated to be about 40 kN/m2 and 11 kJ/m^2, respectively. Fibrecrete with mesh was selected as a surface support system, and cable bolt, split sets, friction bolt and D-bolt were selected as reinforcement systems in the rock masses. The applied ground support systems at the mine sites provided stable ground and a safe environment during mining operations.

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CHAPTER 7: OPERATIONAL/CONSTRUCTION APPROACHES IN DEEP UNDERGROUND MINES

CHAPTER 7: CONTENTS

7.1. Introduction

Design procedure and parameters are implemented in the construction phase of a project. Site location and underground layout specify the actual position for exaction stage. Drilling and blasting is a typical method in underground mining projects. The most important operational and construction approaches in deep underground mines are excavation, scaling, depressurisation/stress management, managing waste materials, control harmful gasses with an appropriate ventilation system, supplies required energy and water, quality control of material, and installing ground support system (Figure 7.1).

Figure 7.1. The typical operational and construction activities in deep underground mines

Water use and quality control of materials, wastes, noise and vibrations, and energy use should be controlled during construction. The ventilation is vital to provide fresh air, remove explosive and harmful gases such as CO, CO_2 and dust. Quality control of material is associated with engineering properties of ground support devices (for example compressive strength of shotcrete) to ensure compatibility of parameters with design conditions.

The most operational approaches in deep underground mining are excavation methods, stress management and quality control of materials as shown in Figure 7.2 and are discussed in following sections in this chapter.

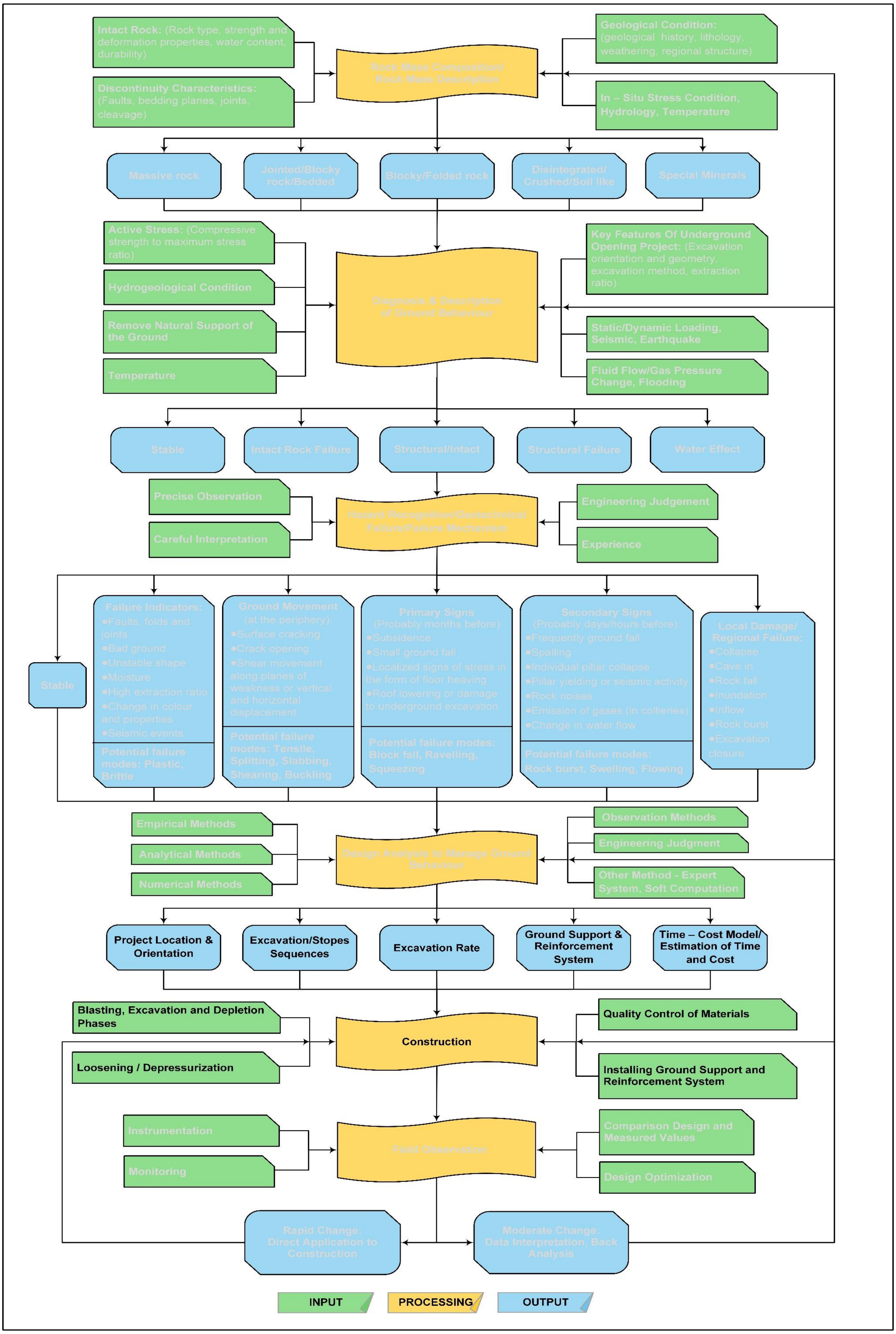

Figure 7.2. Operational and construction approaches in deep underground mines

7.2. Excavation methods

Rock underground construction is associated with the process of the excavation in rock mass structures and making stable in the ground environment by implementing the design parameters. The typical excavation methods are drilling and blasting and mechanical excavations like road header and TBM. Mechanical methods are usually used in the road and tunnel construction projects. Drilling and blasting method is a useful method in hard rocks, rock structures with varying properties, large span underground excavations for example cavern, mining projects. The typical cycle of excavation by blasting is performed in drilling holes, charging, ignition, ventilation, scaling, loading and hauling steps.

The discontinuity condition, rock mass strength, weathering and groundwater condition affect the quality of blasting and the damage zone. The results of the damage zone usually change the properties of the rock mass and performance of the rock structure and probably ground behaviour surrounding the underground opening. In seismically active mines, blasting may cause realising stored energy in rocks derived from seismic events. This phenomenon results in redistribute of induced stress and broken rock mass zone(Singh and Narendrula, 2007). Drilling and blasting excavation methods often extend existing discontinuities and fractures in the rock mass around an opening. On the other hand, mechanical excavation methods have low impacts on rock mass disturbance. The new fractures due to excavation should be considered in the characterisation of the rock mass and support system.

Blasting usually makes three zones; overbreak, creating new cracks and fracture, and growth pre-existing fractures. The overbreak zone is approximately 10% of the excavation profile, but sometimes this damage zone reaches 60-70% in roofs (Suorineni, 2009). Explosions from rock blasting have some negative impacts in mining operations such as:

- Ore dilution and reduction in mineral grade
- Damaged ground problems and increased cost in-demand support and reinforcement
- Reduction of rock mass strength
- Reduction in stand up time of opening

Damage zones in an underground opening are divided into four zones including excavation influence zone (EIZ), excavation damage zone (EDZ), highly damaged zone (HDZ) and construction damage zone (CDZ) shown in Figure 7.3. Construction damage zone is controlled by the excavation method and blasting quality especially in deep underground mines. Excavation influence zone is related to disturbed rock zone surrounding underground openings. According to the figure, strengthening and weakening paths present the cohesion loss and friction mobilisation factors in the graph, respectively. The concept is used to estimate the depth of damage zones.

In rock mass with a high degree of jointing, modifying the pattern of blasting and also using smooth blasting methods reduce the blast damage zone and avoid loosening interlock between rock blocks. Experience in underground opening projects indicates that smooth blasting provides lower excavation cost as well as fewer supports compared with the poor quality of blasting (Palmstrom and Stille, 2015). Scaling is carried out to bring down unstable rock block around the surface excavation. Meanwhile, the rock surface could be clean and prepare for installing shotcrete and rock bolts.

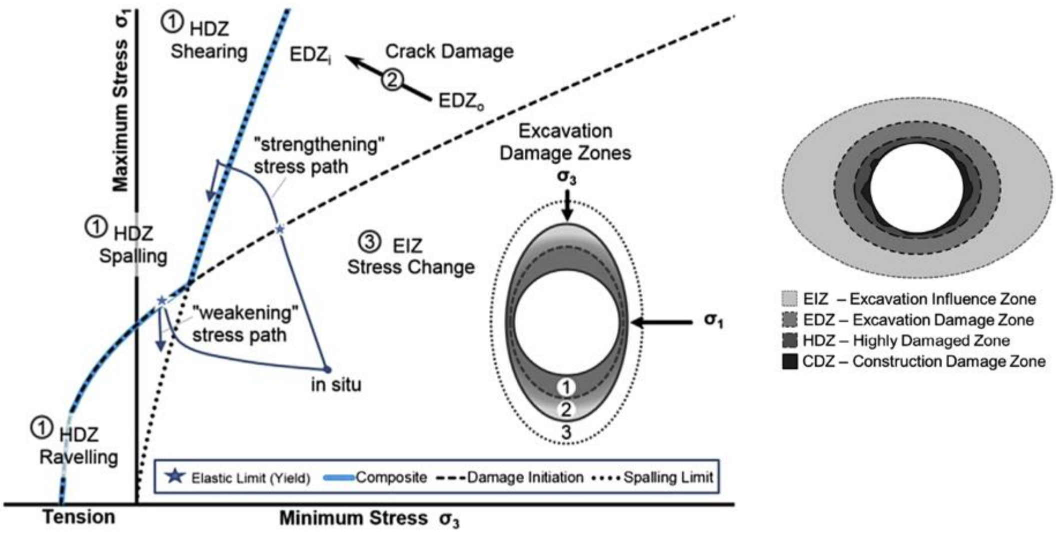

Figure 7.3. Damage zones in an underground excavation (Perras and Diederichs, 2016)

Figure 7.4 is a schematic of extra support demand due to the overbreak zone. It is clear that over break of rock increases construction cost, large deformation or even failure and collapse. Therefore, a proper blasting or other excavation methods should be planned and designed to moderate overbreak thickness in the disturbed zone.

Excavation phase of the project can divide into multi excavation drifts into small sections to redistribute in situ stress condition to the improper effect on stability and ground behaviour.

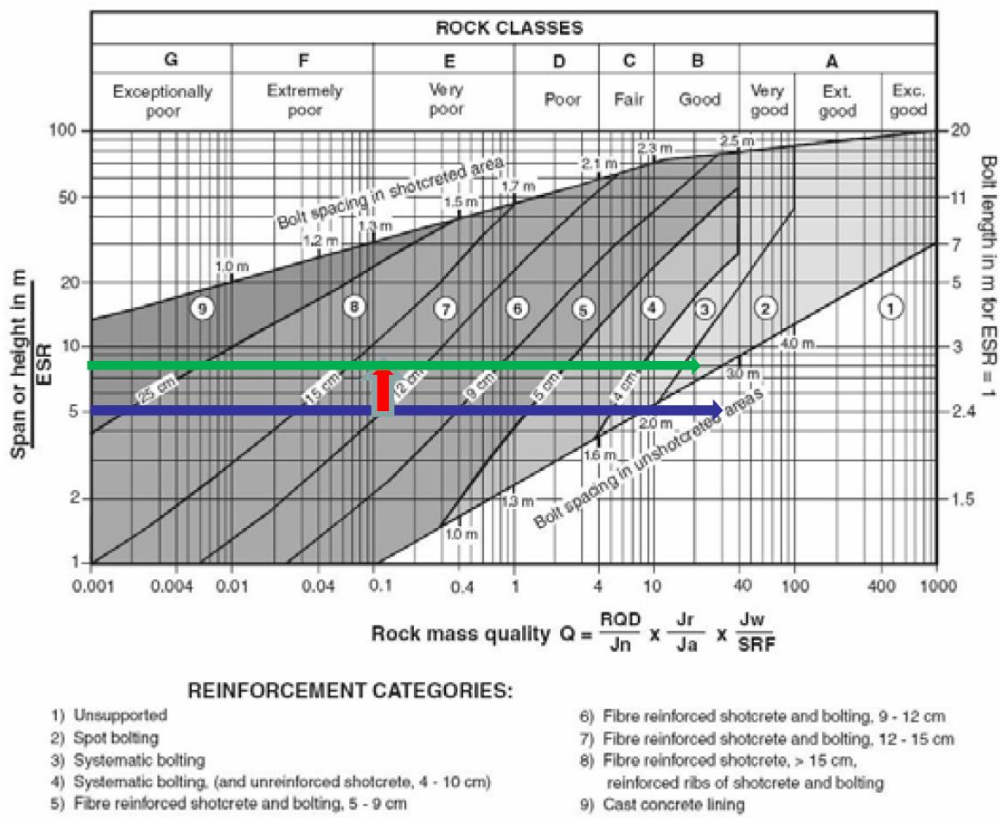

Figure 7.4. Extra support demand due to overbreak zone on excavation profile (Suorineni, 2009)

The sequence of excavation influence the disturbance of in situ stress components and rock mass structures and result in the unstable condition. Disturbed zone surrounding excavation can be diminished by selecting appropriate excavation method and sequences and extraction ratio, especially in underground mining projects.

In hard rock with the high-stress condition where there is the potential of sudden failure in rock zones around the excavation, distress of rock mass can be carried out by drilling some more hole with longer than blasting holes and less charging to release store energy in rocks.

Inspection of faces and stopes before and after excavation is necessary to consider weak structures before the blasting stage, and excavation damage zone after blasting. This may cause changing rock failure modes compared with expected design analysis and may need modification of ground support pattern or the type of devices.

7.3. Stress management

Depressurisation or destressing is a typical method to control rock failures in deep and high-stress conditions. Ground stress and seismic events are inevitable in underground mining operations and may cause various failures at great depth, such as rock burst (Rahimi and Sharifzadeh, 2017). Figure 7.5 shows different methods for the reduction of rock failure due to excessive stresses. Destress blasting is used for fracturing rock zones to dissipate stored strain energy from rock masses in mining operations and underground constructions. The method is used to reduce the level of stress concentration, by creating fractures in the rock mass that cause a reduction in the elastic modulus of the rock mass, and enable the rock to carry high-stress conditions.

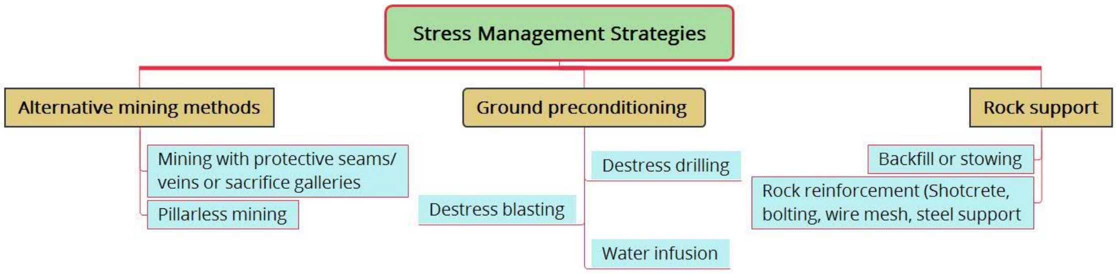

Figure 7.5. Excessive stress management methods in damaged rock zone around excavation (Modified after Saharan and Mitri (2011))

Figure 7.6 shows relocation of the stress concentration level by the destress blasting method surrounding an excavation. The effect of the destress blasting method can be evaluated by measuring some rock engineering parameters such as deformation of the rock mass, stress magnitude changes, seismic effects, and changes in the elastic modulus. The technique is applied to manage rock hazards derived from high-stress conditions such as strain burst and rock ejection.

7.4. Quality control of materials

Quality control and assessment of materials are determined by the necessary quality level and quality grade. The quality level is described as the difference between the required geotechnical techniques including specifications and actual implanting work. Quality grade is the difference between standards and specifications required of companies and the quality of manufactured products. Quality control is assessed through a systematic examination and quality assurance from geotechnical activities to achieve planned objectives.

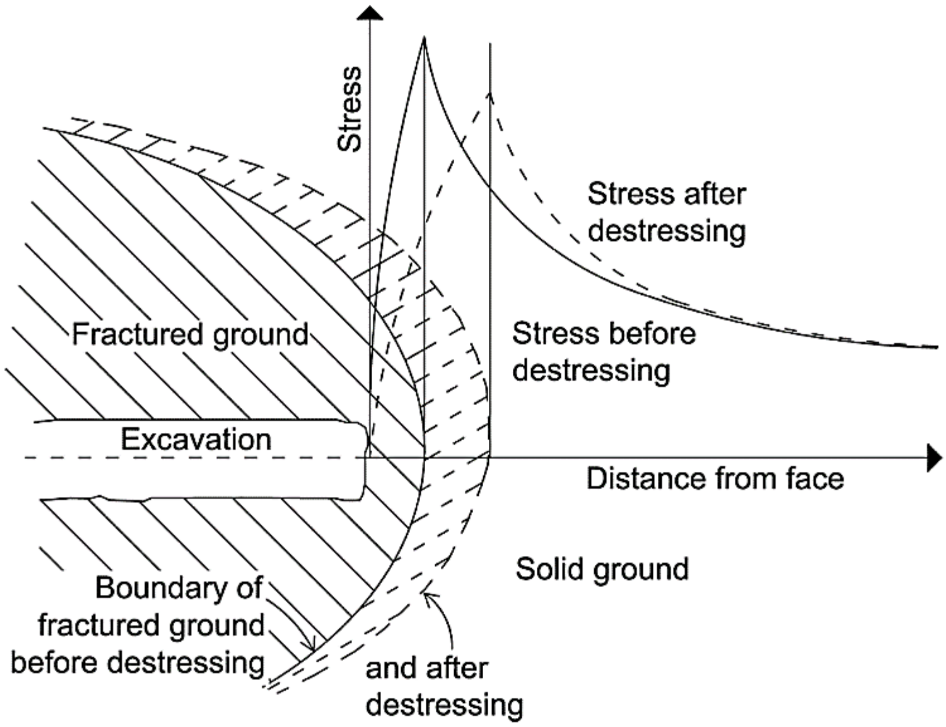

Figure 7.6. The effect of destressing blasting method on rock zone surrounding an excavation (Mazaira and Konicek, 2015)

Quality assurance of ground control management in underground mining projects includes verifying that the construction is being done by the design, checking the availability of equipment, personnel facilities and general resources, and can be summarised by the following tasks (Szwedzicki, 2003):

- Discussion with managers about related activities for ground control
- Inspection of geotechnical activities in underground mines
- Review of procedures for operational activities, standards, documents and critical tasks
- Consideration, discussion and review of geotechnical record and input data on the design

- Observation and monitoring of drilling, blasting, rock mass behaviour and failure modes
- Discussion with supervisors and operators about the identified issues and development activities

The common problems during the shotcreting process in underground mining projects are difficulty in achieving correct consistency (especially W/C ratio), sprayability, proper storage and utilisation of admixtures, and use of the correct nozzle distance by operators (Talbot and Burke, 2013). Training of operators and supervisors is required to address these problems in projects. Also, there is a concern in using grouted rock bolts to fill pores in rock zones where there is groundwater which would lower the rock bolts' performance in the ground. Using recent technology, reflected ultrasonic wave signals indicate any voids and the quality of rock bolt installations can be improved (Yokota et al., 2013). Geotechnical quality control should be undertaken before installation to ensure that they are following the design parameters.

Backfill materials as local support in an underground mining project are selected based on excavation size, distance from the face to backfill, the proportion of filled area, porosity and quality. The high porosity of backfill materials makes more and shrinkage and containment problems. Also, the distance from the face to backfill should be kept less than 6m. The porosity factor determines the stiffness and resistance of support quality. For the backfill as local support, sufficient resistance of the materials is about 0.2Mpa in the stopes with burst-prone ground (SIMRAC, 2002). Unconfined and confined compressive strength (saturated and drained condition), particle size distribution, PH level, mineralogy of backfill materials, density are the typical tests that are used to specify properties of backfill/paste fill materials to use in deep underground mines.

7.5. Case study of operational approaches in a deep underground excavation

Operational and construction approaches of case example from chapter 6 is considered. The excavation phase of main caverns of the project was implemented by drilling and blasting method with sequential steps within 5m-10m. Figure 7.7 shows the construction stage an underground excavation the Project. Drilling and blasting method was carried out for excavation. The primary ground support and reinforcement systems including wire mesh, rock bolts and shotcrete were installed after sequential excavation. The concrete and cables devices were used as a secondary ground support system at the underground excavation.

Technical problems during construction stages were rock cracking, anchor bolt failing, shotcrete cracking, large deformation, rock burst during excavation and ground fall (Figure 7.8, a and b). The potential of unstable rock mass structures was analysed by displacement measurements on the surface excavation from monitoring system by installing multiple position extensometers (Figure 7.8, c). Data collection from field measurements was used to modify geotechnical design parameters and ground support patterns and devices.

7.6. Conclusion

Acquired parameters in design analysis to manage ground control are implemented in operational and construction approached in deep underground mines. The essential approaches in mining operations are excavation, scaling, quality control of materials, supply required energy and water, control noise and vibration, managing water materials, control harmful gases with an appropriate ventilation system, depressurisation/stress management, and installing ground support system.

Two typical excavation methods in deep underground excavations are mechanical methods such as the mechanical hammer, and blasting methods. Extraction minerals in underground stops are usually proceeded by drilling and blasting. Overbreak, creating new cracks and fractures, and growth pre-existing fractures are three main zones as a consequence of blasting damage surrounding underground excavations. Optimisation of blasting pattern, type of charging, design of blasting with regard to orientation of discontinuities, and smooth blasting methods can reduce the thickness of the damage zone. Scaling and cleaning surface of excavations after blasting is required to diminish mechanical failures during mining operations.

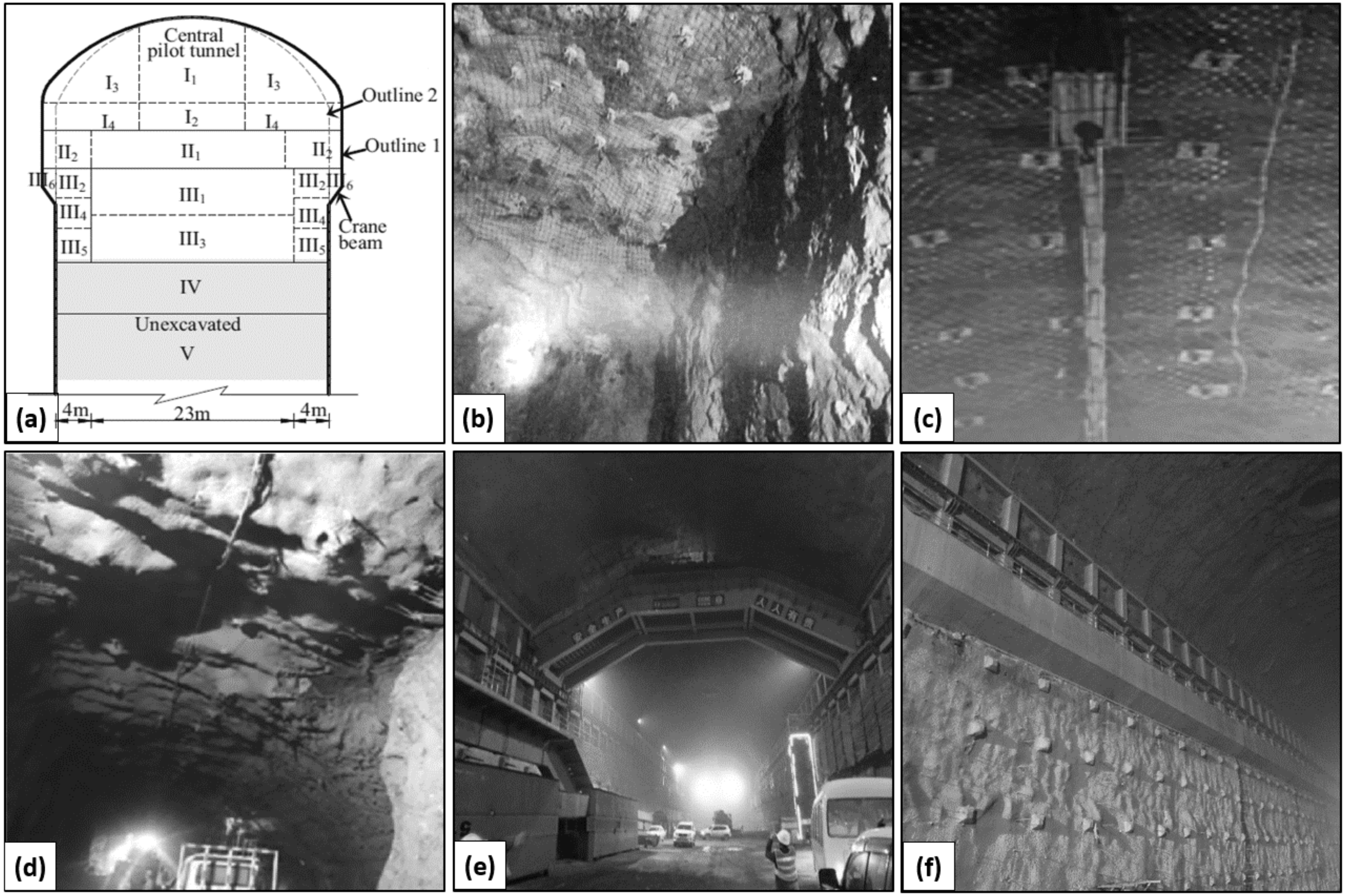

Figure 7.7. Construction Stage at the main caverns of the project, (a) sequential excavation, (b) installing wire mesh, (c) installing rock bolt, (d) sprayed shotcrete on the surface excavation, (e) installing concrete, (f) completed construction

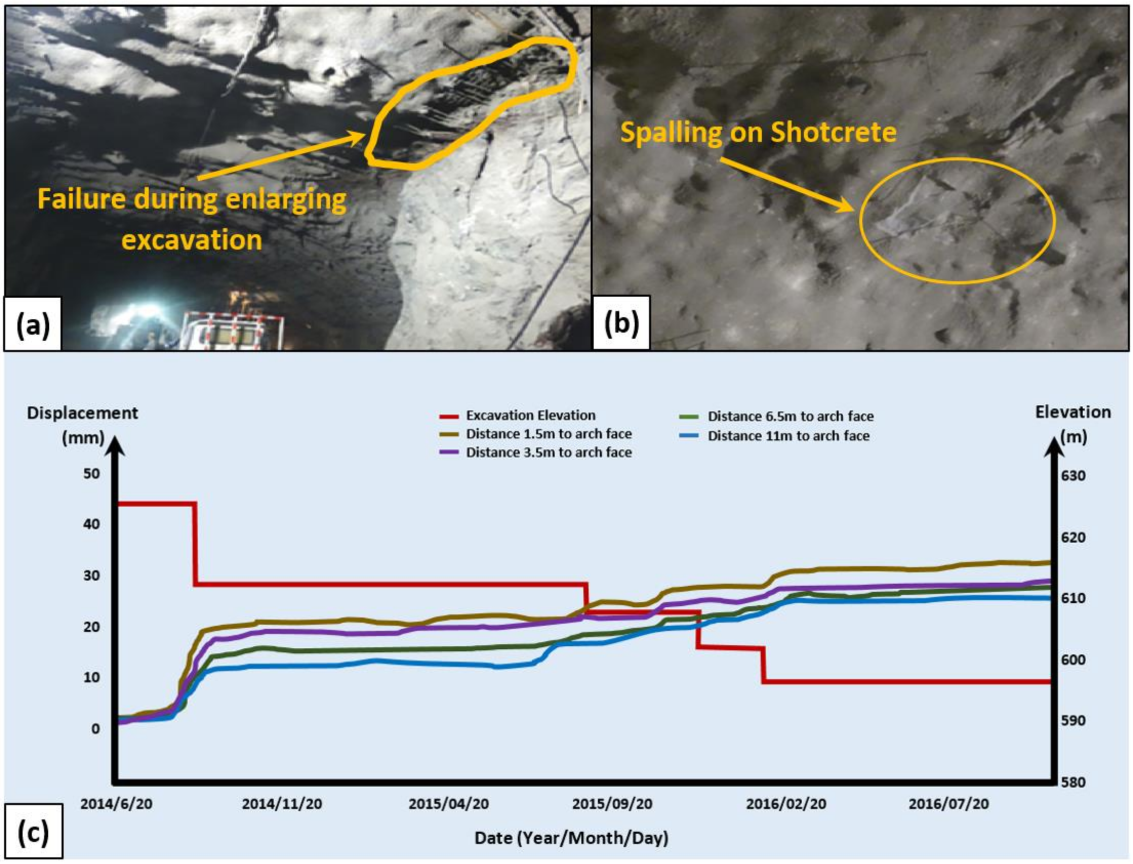

Figure 7.8. Technical problems in the construction stage, (a) occurring failure through enlargement excavation, (b) cracking and spalling on the shotcrete, (C) displacement monitoring in an underground excavation

Mining operations at great depth are associated with high stresses and release stored energy from seismic events that makes severe rock failures. Destress blasting/stress management is employed for creating mechanical cracks and fractures in the rock mass to dissipate stored strain energy to reduce potential sudden failure in underground stopes. Also, other stress management in underground operations is using alternative mining methods and ground support systems.

Quality control of materials is an essential part of mining activities for examination and quality assurance of support elements. Regular inspection of geotechnical activities, update geotechnical procedures and standards, laboratory tests from shotcrete and rock bolts and past materials are used to specify quality assurance at works.

The operational and construction approaches in deep underground excavations were considered. Shotcrete cracking, rock burst, anchor bolt failing, and ground fall were some of the construction problems observed in the project. Collected data form site observational methods and field measurements were used to modify the problems.

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CHAPTER 8: GEOTECHNICAL MONITORING AND DESIGN UPDATE

CHAPTER 8: CONTENTS

8.1. Introduction

The aim of field measurements is usually to recognise relevance between disturbed rock structure condition and design parameters on the ground in order to reduce risks and problems. Monitoring measurements provide high precision data and temporal resolution in a real-time in geotechnical projects, and some of the highlight points are (Ghorbani et al., 2012):

1- Disclose unknown parameters in the design process
2- Verify design parameters and assumptions
3- Apply observational methods during or after the construction phase
4- Optimise design parameters and procedure to enhance safety condition
5- Ensure safety of adjacent structures due to construction

The monitoring program is defined as installing sensors for during a particular time; observation, recording data and checking the progress quality; and modify design parameters. Geomechanical monitoring through responses to rock mass structure is practised to considering real ground behaviour and improve safety in underground excavations. The design of the monitoring plan deals with project conditions and the geotechnical objectives. The mechanism of behavioural control of support elements and ground conditions determines the type of instrument devices and the location of their installation.

Field measurements can reduce risks and problems by measurements data and applied at mine sites. The design of the monitoring system in underground openings usually consists of four steps (Vargas, 2014):

1. Define monitoring program during the design process,
2. Estimate period of time for monitoring based on project objectives,
3. Observe, record, interpret and analyse data, and
4. Modify the design parameters.

Step three is usually applied to provide trustworthy and accurate data during the monitoring program.

Back analysis is utilised to determine reliable rock engineering parameters for evaluation of the actual ground condition and adapt ground support systems.

Figure 8.1 shows the benefits of the application of monitoring and observational methods in underground mining. Monitoring methods can be used during the early stage of development of underground mining projects to acquire real ground behaviour and modify design parameters. Some of the benefits of monitoring and site observations are reduction uncertainties of design parameters, achieving value–cost/time, minimising hazards and improving ground support systems.

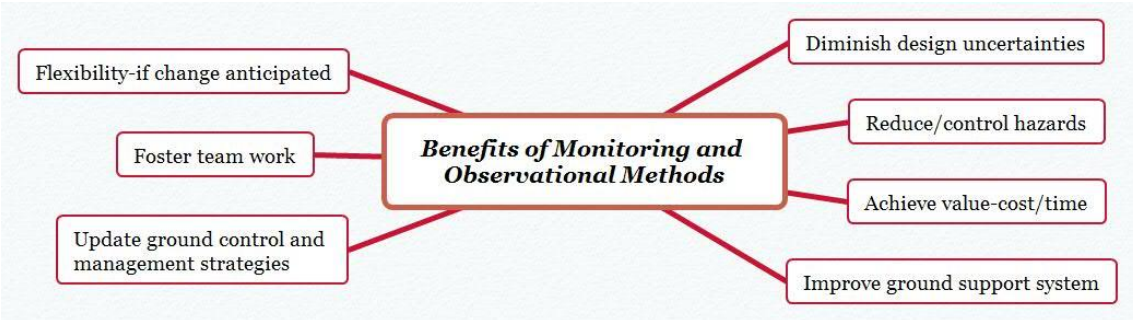


Figure 8.1. The advantages of applying monitoring and observational methods in underground mining (Modified after Rahimi and Sharifzadeh (2017))

The main components of geotechnical monitoring and design update are instrumentation, monitoring and back analysis, as shown in Figure 8.2 and Figure 8.3. In the following section in this chapter, these components are described.

8.2. Instrumentation

A good monitoring system is associated with using all available information from seismic sources, seismic loading, small and large-scale deformation in rock mass structures, and induced stress field in field instrumentation. Installing different types of instruments at great depth and high-stress level, where is more potential for damage of device because of seismic events, handle different measurements like excavation deformation and seismic events to use evaluation of ground support performance.

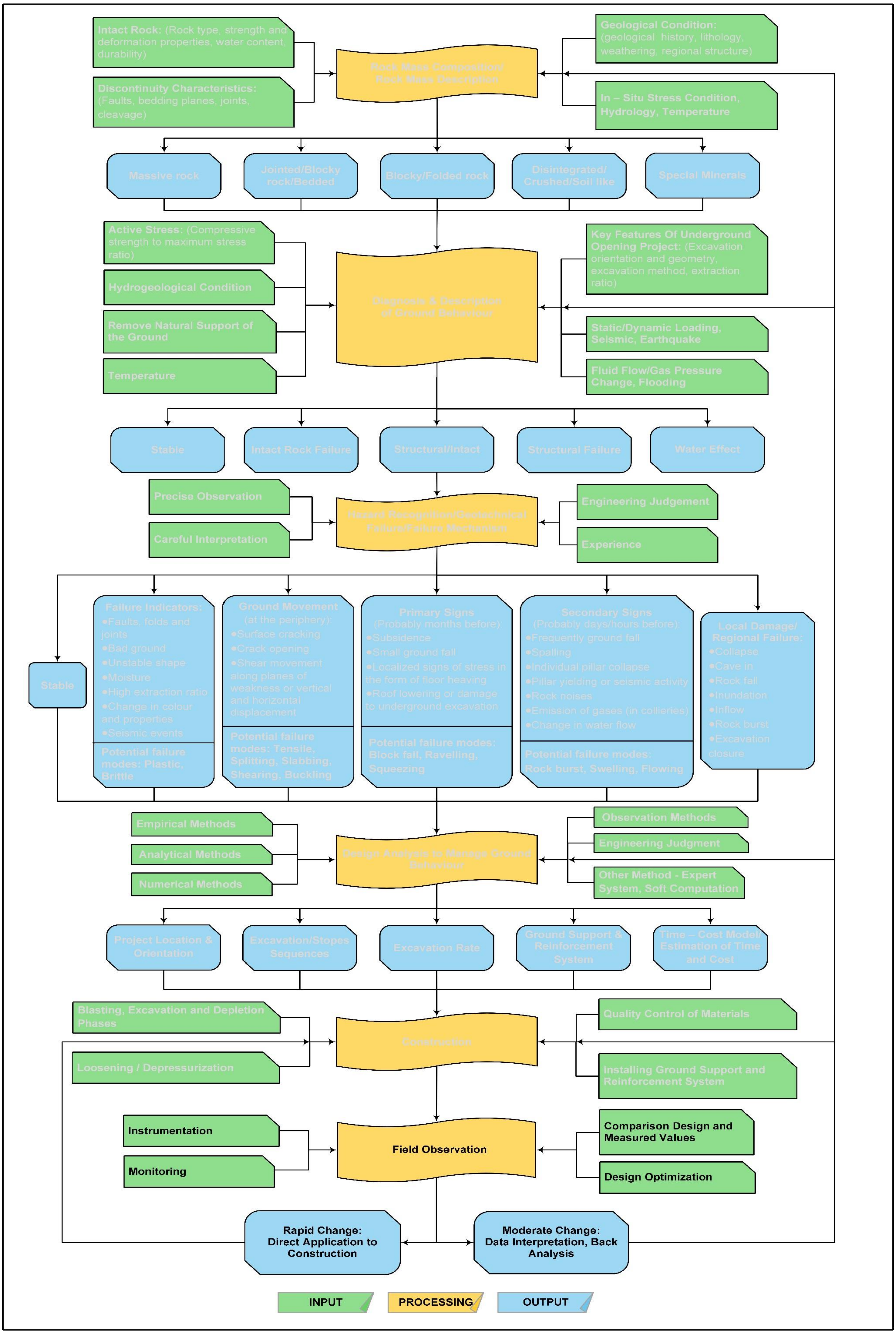


Figure 8.2. The process of field observation in deep underground mining

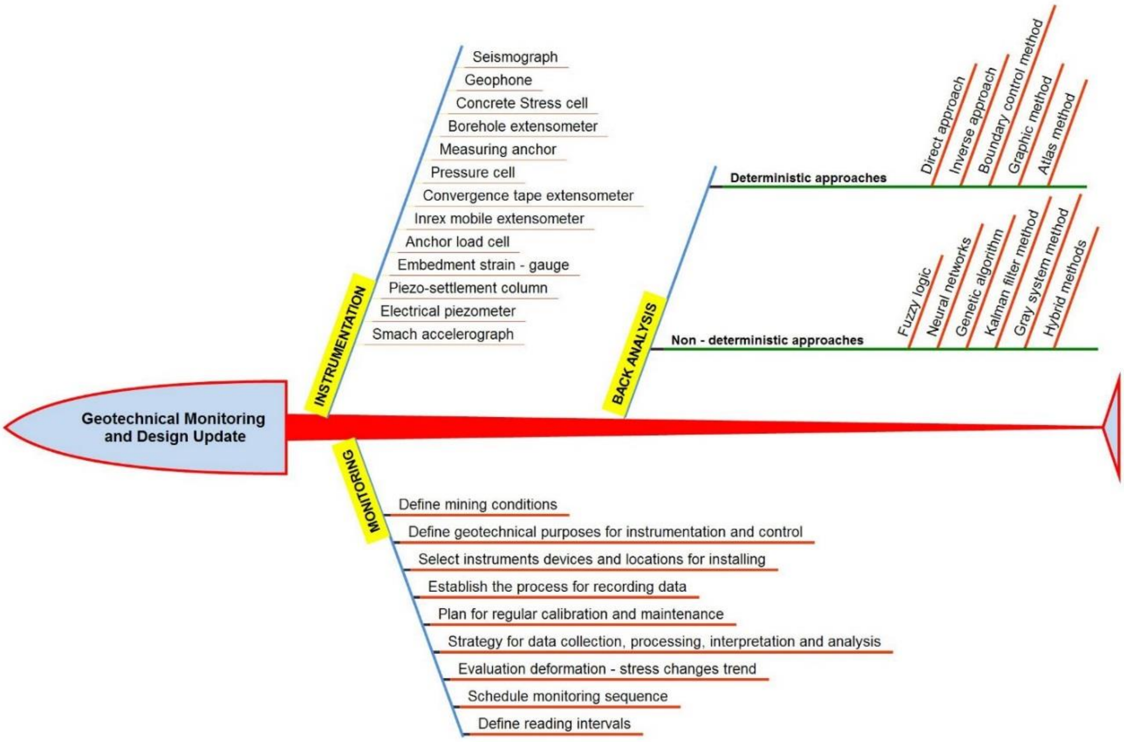


Figure 8.3. Geotechnical monitoring and design update procedure in underground excavations (Modified after Sharifzadeh et al. (2017))

Instrumentation systems can be categorised into four groups (SIMRAC, 2002):

- Optical systems; is a type of photogrammetric surveying methods which are simple and traditional methods for quick measurements of underground excavations by borehole cameras and petroscops to provide a profile of excavations and movements on boundaries.
- Mechanical systems; including rod, wire, cable and tapes to measure displacements. The method is simple, cheap and more reliable.
- Hydraulic/pneumatic systems; which is based on an acting fluid pressure inside a flexible metal or plastic to measure support loads and normal stress components.
- Electrical devices; this category instruments are used for measuring strain and stress components, seismic events and displacements between two or more points. The harsh condition in underground construction may cause failing electrical systems.

Observation methods are accomplished by instrumentation and monitoring methods and by various types of instruments that are shown in Figure 8.4. There are various types of instrument devices like extensometer, pressure cell, electrical piezometer that are used for measuring the performance of support devices and ground parameters. Measurements of deformations and forces are more common in monitoring systems. Design monitoring plan is deal with project conditions and the objective of geotechnical purposes. The mechanism for control behaviour of support elements and ground condition determine the type of instrument devices and the location of their installation.

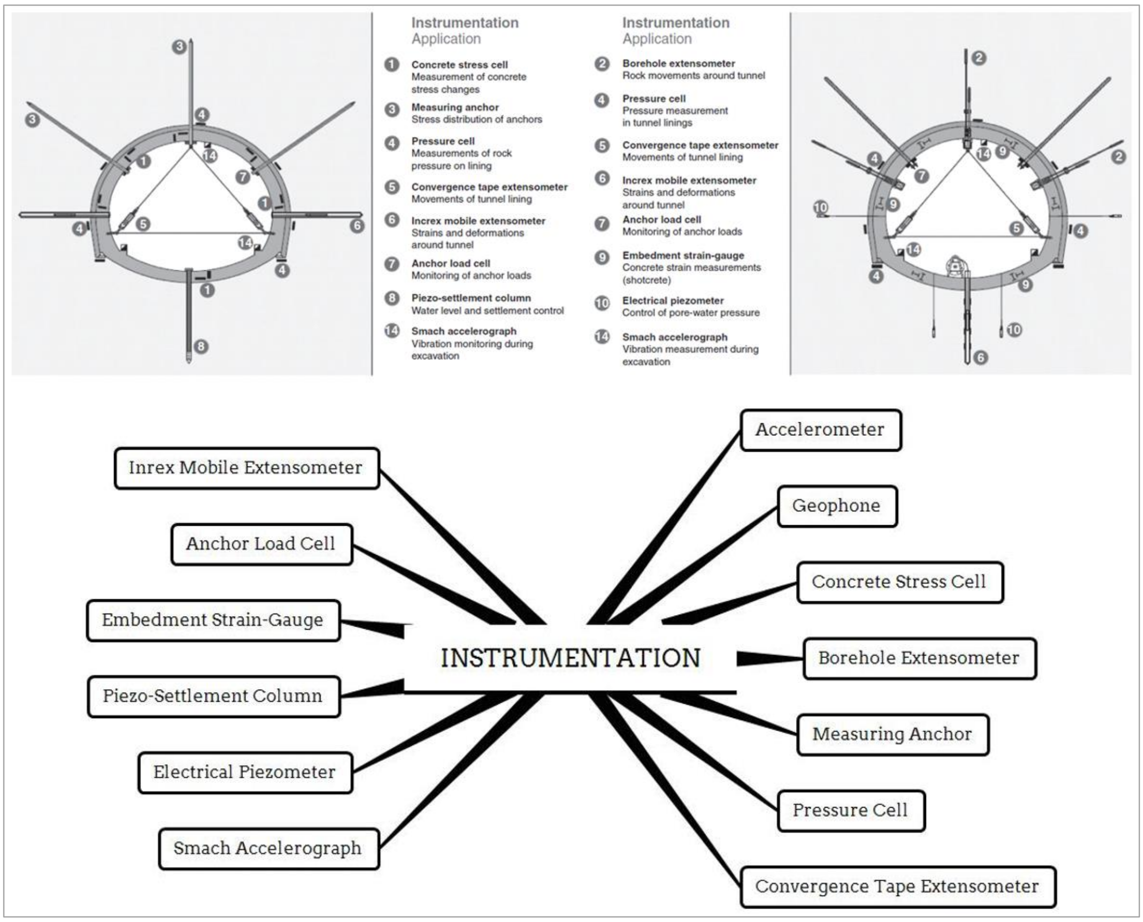


Figure 8.4. Instrumentation methods in underground excavations (Modified after Rahimi and Sharifzadeh (2017))

Figure 8.5 shows the typical digital panoramic borehole camera system for monitoring. A digital camera system is used in boreholes before/during/after excavation in underground openings to control weakness zones. For this purpose, a series of boreholes are drilled surrounding excavations to install these tools. The multipoint extensometer is a simple and reliable device, which can be installed easily in different directions in boreholes to monitor rock mass deformations (Duan et al., 2017). Monitoring stress condition in rock mass structures can be carried out by application of cable dynamometers and bolt stress meters, especially near to weakness zones. The stability of the rock mass is evaluated following the change of loading and unloading rates. The monitoring of loading in weak rock mass structures can be accomplished by preinstalling a dynamometer on prestressed anchor cables. The in situ observation process to monitor rock mass failure should be undertaken by at least one or two different devices and methods simultaneously in underground excavations.

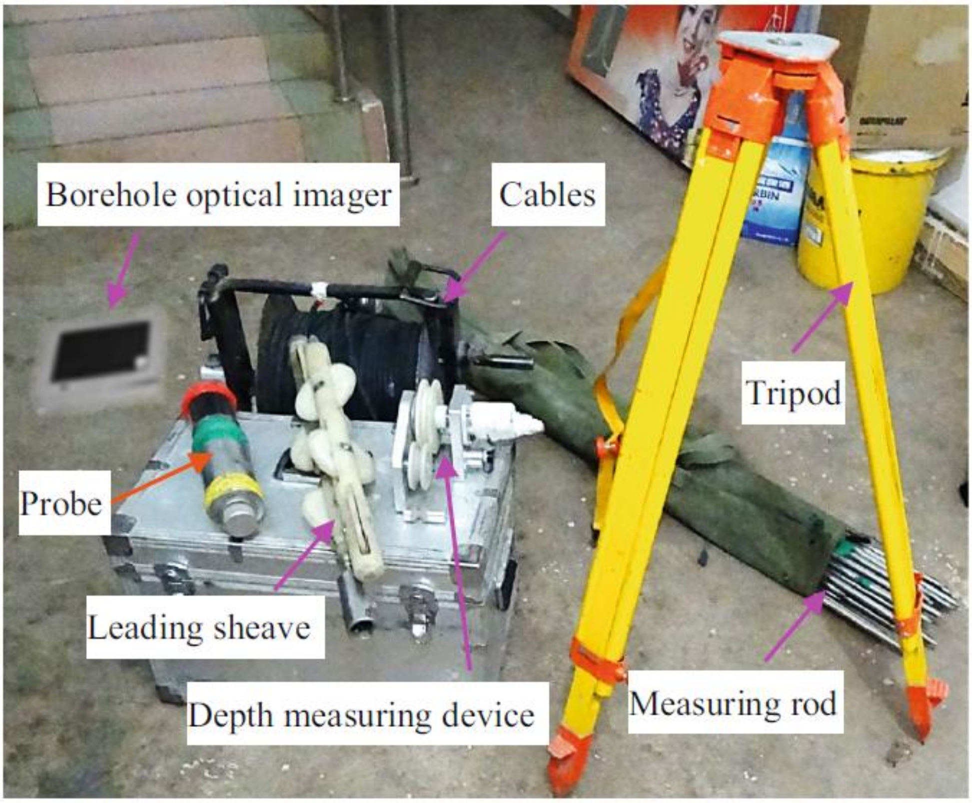


Figure 8.5. A typical digital camera system tools for monitoring process of failure in underground excavations (Duan et al., 2017)

Mining operations at great depths are encountered with high stresses and seismic events. In seismically active mines, microseismic monitoring is an appropriate system to manage the ground condition in seismic events. Microseismic monitoring at great depth and high stresses supply useful data and information on changing ground behaviour and failure mechanism under stress concentration and seismic events. The principal components of the microseismic monitoring system are (Wu et al., 2012):

- Recording ground motions with seismic sensors such as geophones by making ground velocity records or accelerometers by recording acceleration in ground condition
- Converting analogue seismic signal to digital with means recorders
- Synchronisation of time in server and receivers
- Making data communication from sensors to recorders and recorders to central workstation
- Processing, interpretation, visualisation and reporting seismic data

The main equipment for seismic monitoring systems are sensors, data acquisition instruments, data transferring units, and a centre server, as shown in Figure 8.6. Data transfer unit can be a type of cable, optical fibre and wireless system mines. Figure 8.7 shows different types of sensors used in seismic monitoring systems. Typically, sensors are two types: geophone and accelerometer. Also, these types can be used for uniaxial and triaxial wave recording (Xiao et al., 2016).

Figure 8.6. The main components of the seismic monitoring system (Xiao et al., 2016)

The layout of sensors depends on monitoring objects, installation conditions, and monitoring environments in deep underground excavations. Figure 8.8 shows three types of design for sensor arrays. The typical relation between the sensor array and seismic events are inside, edge, outside. When seismic events are inside of sensor array, the results can be the high accuracy compared with other types. It should be mentioned that the number of sensors is increased in a critical location such as the high level of seismic events near underground stopes.

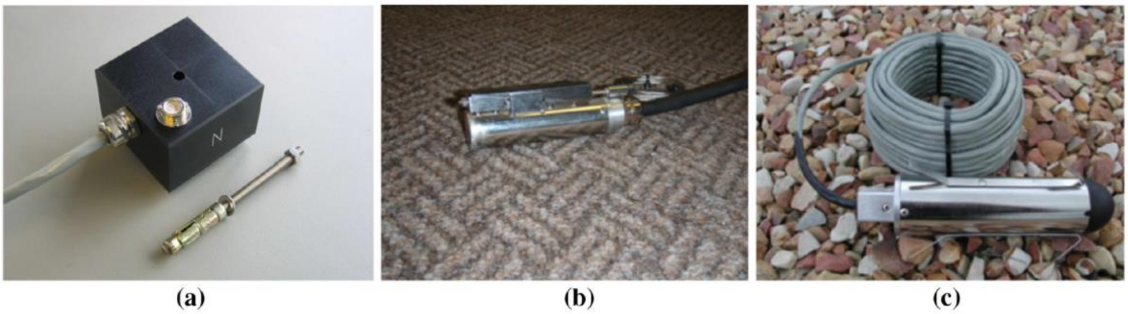


Figure 8.7. Different types of sensors in seismic monitoring systems; (a) surface type, (b) uniaxial geophone types for boreholes, (c) triaxial accelerometer types for boreholes (Xiao et al., 2016)

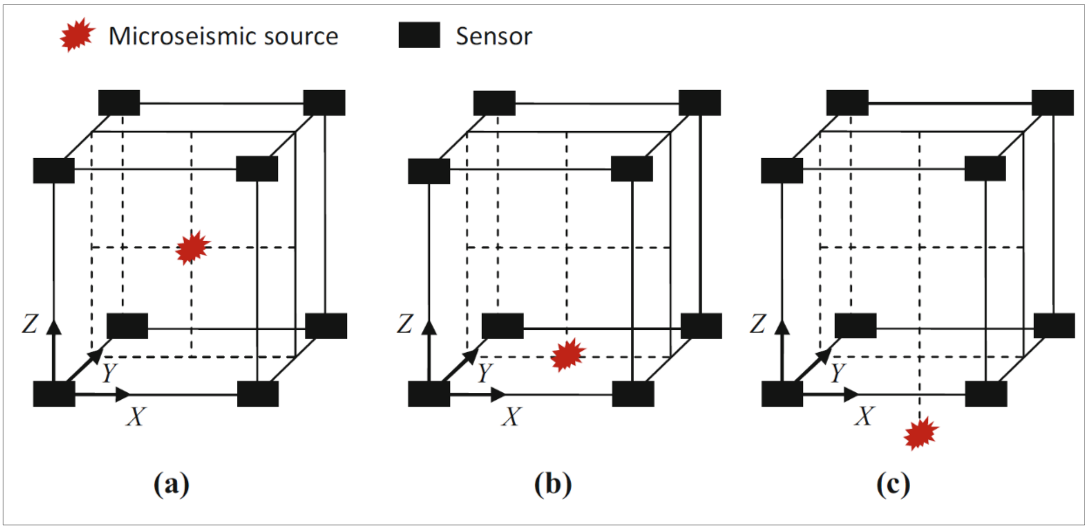


Figure 8.8. Typical design in array sensors; (a) seismic source inside sensor array, (b) seismic source at the edge sensor array, (c) seismic source outside sensor array (Xiao et al., 2016)

Figure 8.9 shows some examples of sensor locations in an underground mine. For large–scale seismic monitoring, sensors are installed in main accesses excavated in different levels (Figure 8.9–a). Underground mining operations are associated with sequence excavations in stopes, so monitoring can be carried out by installing sensors before excavation underground stopes (Figure 8.9–b) and during mining operations (Figure 8.9–c). Also, a layout of locations of the installed sensor has been shown in Figure 8.9–d.

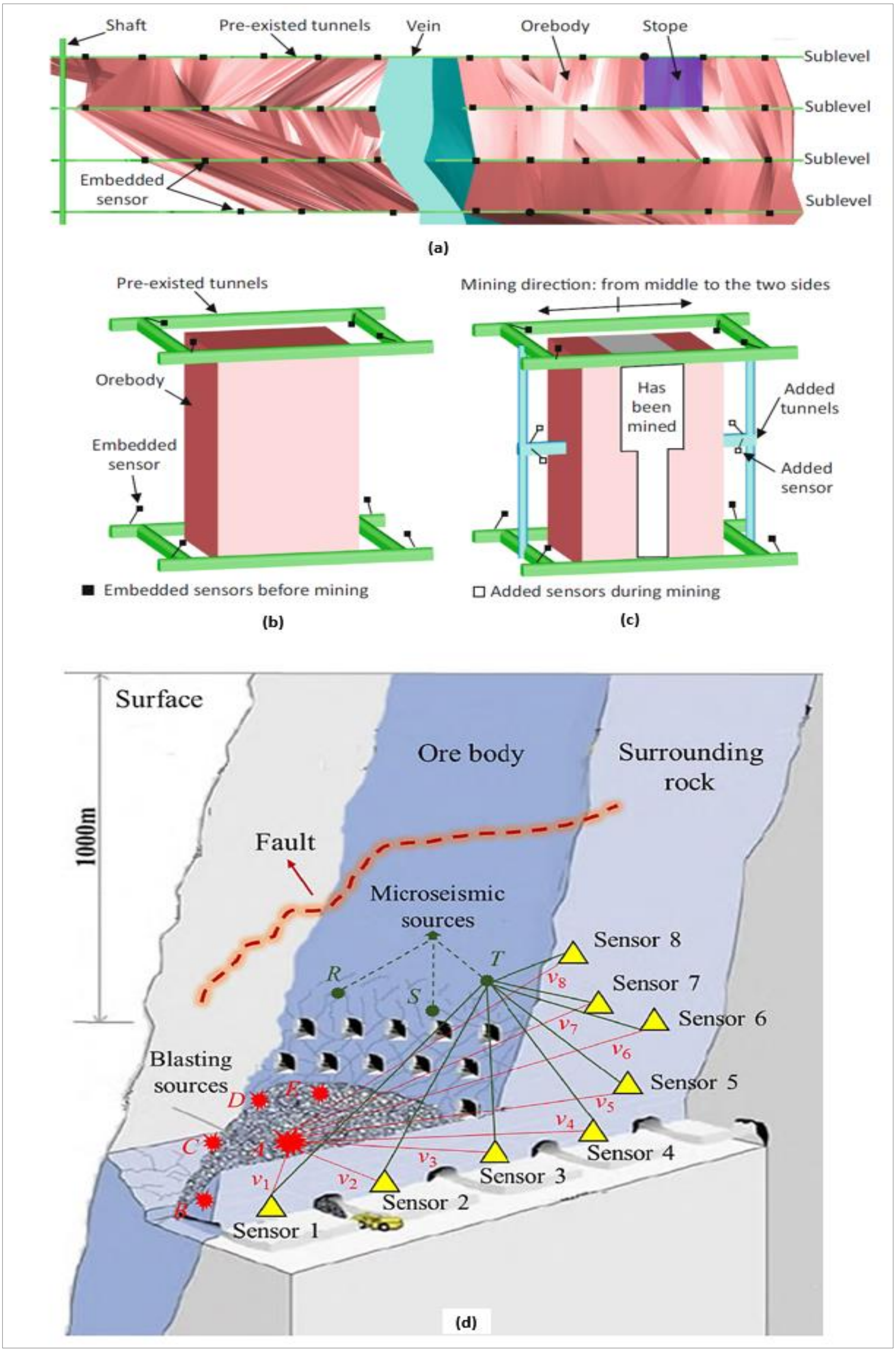


Figure 8.9. Examples of sensor layouts in deep underground mine; (a) installed sensors in main accesses in sublevel stoping , (b) installed sensors before underground mining stopes, (c) added sensors during mining operations, (d) location of sensors in a mining zone at great depth (Xiao et al., 2016, Dong et al., 2017)

8.3. Monitoring systems

Geotechnical monitoring through responses of rock mass structure is used to consider real ground behaviour and improved risk management in underground excavations (Widzyk-Capehart et al., 2016). The monitoring is performed by collecting data, processing, interpretation and analysis. Collected data form monitoring is used in two ways. In the first step, for abnormal status, for example, an excessive deformation, immediate action may require to prevent risk failure. Secondly, data analysis and interpretation is undertake to find the reliable value of design parameters. Figure 8.10 presents the main process of monitoring systems in underground mines. The condition of mine projects such as seismicity active mine, geotechnical purposes, limitations, and the period of monitoring influence the monitoring plans.

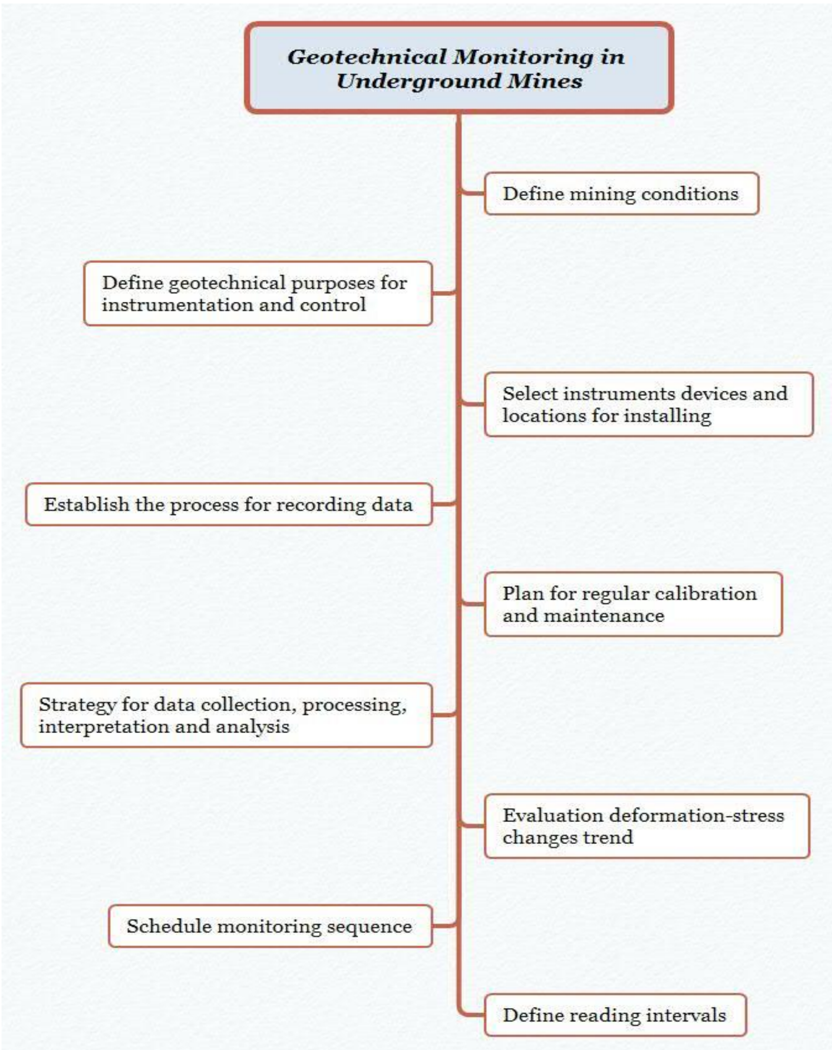


Figure 8.10. The main process of geotechnical monitoring in underground mines (Modified after Sharifzadeh et al. (2017))

8.3.1. Conventional monitoring

The critical process for field instrumentation and monitoring in high stress and in-depth underground mining projects are listed below (Zhang et al., 2016):

- Selection field sites: Location of instrumentation is selected based on being in safe areas, being far from mechanical noise areas, availability of underground excavations in the mine sites, and ability to collect appropriate and enough data
- Multiple–point borehole extensometers, which can be used for deformation monitoring in rock mass structures surrounding excavations, especially ground deformation before and after seismic events.
- Laser– scanning, is a quick and straightforward monitoring method for surface deformation. After running an initial scan form surface excavation, an appropriate time for second scanning is detecting deformation more than 10mm by multiple – point borehole extensometers.
- Instrumented rock bolts, which is used for monitoring load–deformation along rock bots.
- Borehole observation and monitoring: Surviving boreholes by a camera to distinguish discontinuity conditions and rock types during the mining activities.
- Local seismic systems: installation of seismic sensors near excavation for evaluation of ground motion due to seismic events.
- Damage mapping: when seismic event makes happen, damage mapping should be carried out to assess immediate effects and accomplishment the possible solutions on the response of a rock mass to seismicity.
- Numerical modelling: detail support information for a local area can be evaluated with numerical modelling encounter static and dynamic loading, and also unloading condition.

Field instruments and measurements can be classified into two main groups based on the type of underground excavation, environmental condition and accurate measurements (Ghorbani et al., 2012):

1- Geodetic surveys, which consist of photogrammetric methods, digital photogrammetry and aerial photogrammetry, conventional or traditional methods, for example, measurement of displacement by total station theodolite and other particular methods.

2- Geotechnical field measurements; the conventional techniques are extensometers, joints meters, strain meters, inclinometers, etc.

Application of these methods is dependent on ground condition, critical zone of rock structures such as shear zones, and availability of instrument devices.

In the case of monitoring of weakness zones, the first step is the estimation of the location with the potential of failure occurrence in an underground opening during/after the excavation process. Then, rock deformation, stress-induced and rock structure damage zones surrounding excavations can be evaluated by application of different tools and methods like acoustic velocity testing, bolt stress meters, extensometers, microseismic monitoring system and digital panoramic borehole camera system. For example, microseismic system and shear dislocation distance methods are useful for monitoring rock mass structures with the potential of shear slip failure. Also, digital camera system, extensometers, cable dynamometers and bolt stress meters is used to control plastic squeezing and stress-induced failure/collapse.

8.3.2. Seismic monitoring

The main components of the seismic monitoring system in an underground mine are (Essrich, 2005):

1. Monitoring objects: The object of the monitoring system is defined based on required source parameters to quantify, locations/areas of seismicity, analysis method, and requirements and budget.
2. Seismic system/instrumentation: The seismic system is the selection of all network hardware and software that are required for collection data, data processing and output. Physics of oscillation, sensor types, network types, interfaces, and wave propagation are selected in this step.
3. Data collection: Recording data and data processing are performed for determination seismic events. The key points in this step are good understanding of functions related to system downtime, station downtime, location accuracy, ratio accepted/rejected events, and sensitivity.
4. Data analysis: The methods and tools are applied for analysis of seismic data. Data analysis results in the estimation of location in space and time, stress – drop, peak

particle velocity/acceleration, seismic hazard quantification, seismic risk quantification, and slip – burst type.

5. Risk reduction: The final step in the seismic monitoring system is considering the methods for reduction of risks at the mine sites. Some of the techniques are bounding seismically active areas, limitation of mining operations in locations with high seismic levels, supporting rock mass with the potential of sudden failure, classification of underground mining stopes based on levels of seismic events and hazards, and continuous observation of seismic events in short-medium term.

Table 8.1 presents the seismic monitoring system in mining projects. Seismic data collection, analysis of seismic data, and interpretation are the main steps in the monitoring system.

Table 8.1. The process of seismic monitoring system (Essrich, 2005)

Part A Seismic Data Collection	1. Seismic Systems • Monitoring objectives • Principal concepts/solutions • Sensors • Data transfer • Network design • Limitations	Outcome Ability to choose monitoring equipment best suited to defined needs; knowledge of technical and financial requirements of monitoring; knowledge of physical limitations of technology;
	2. Data Processing • Automatic processing • Manual processing • Basic parameters • Spectral domain • First motion • Moment Tensor	Outcome Knowledge of hard- and software requirements for seismic data processing; awareness of issues related to quality of seismic raw data; knowledge of seismic source quantification methods;
	3. Theoretical Seismology • Source types • Elastic waves • Radiation pattern • Oscillatory motion • Wave propagation • Interfaces • Real and idealised media	Outcome Knowledge of theoretical concepts of seismic sources, initiation and termination, strain energy storage and release; knowledge of simplified models of vibration and their mathematical description; properties of waves and physical principles of their propagation simplified media;
Part B Seismic Data Analysis	1. Descriptive Statistics • Classification • Histograms • Location • Variation • Correlation	Outcome Knowledge of statistical methods of data analysis and awareness of their limitations;
	2. Data manipulation • Smoothing • Compression • Regression • Probabilities	Outcome Knowledge of methods of data manipulation and their benefits in terms of trend and pattern identification;

Table 8.1. The process of seismic monitoring system (Essrich, 2005)

	3. Seismological Tools • Time history • Gutenberg-Richter relation • Maximum size estimates • Recurrence time • Predictions & success rate • Energy-Moment Graph • Energy/Moment Index • Ground-Motion Relation • Seismic hazard assessment	Outcome Ability to choose from a wide selection of tools to create desired types of information; ability to direct service provider w.r.t. deliverables; ability to formulate relevant questions and seek comprehensive answers; familiarity with limitations of methods and common pitfalls;
Part C Seismic Data Interpretation	1. Application I: Single event • Source location • Source parameters • Failure mechanism • Damage mechanism • Zone of influence	Outcome Interpretation of basic source parameters as quantified by seismic monitoring equipment; physical interpretation of parameters and their significance for source interpretation; correlation between underground observations and source parameters (location, source dimension, source mechanism);
	2. Application II: Event clusters • Trends over time • Patterns in space • Seismically active • Rockburst prone • Instability concept • Mine design parameters	Outcome Ability to identify trends and trend changes in seismic data; ability to recognise patterns; correlation of observed changes with mining and geotechnical parameters; classification of working places and geotechnical areas into various seismic risk categories;
	3. Application III: Losses • Rockburst reports • Damage classification • Ground motion vs. damage • Threshold of relevance • Local rockburst scenario • (In)direct costs • Contingency planning	Outcome Ability to document rockbursts and associated damage; ability to estimate ground motion characteristics for support design and hazard classification; ability to quantify rockburst related losses and their financial implication; ability to utilise knowledge gained from rockburst report analysis to reduce seismic hazard and mitigate its consequences;
	4. Administration • Contract law • Quality management • Auditing • Mine Health & Safety Act • Other legal requirements	Outcome Knowledge of contract law; ability to evaluate data quality and aspects of seismic system performance; awareness of legal requirements in the field of health and safety.

The basic principles of seismic monitoring are illustrated in Figure 8.11. Released energy in rock mass structures due to seismic events as P–waves and S–waves are recorded in sensors as analogue signals. Then, these data are transferred and digitised in data acquisition instruments. Electric signals through data transfer unit are transferred to the centre server. Seismograph can be displayed in a software and source parameters of seismic events can be analysed.

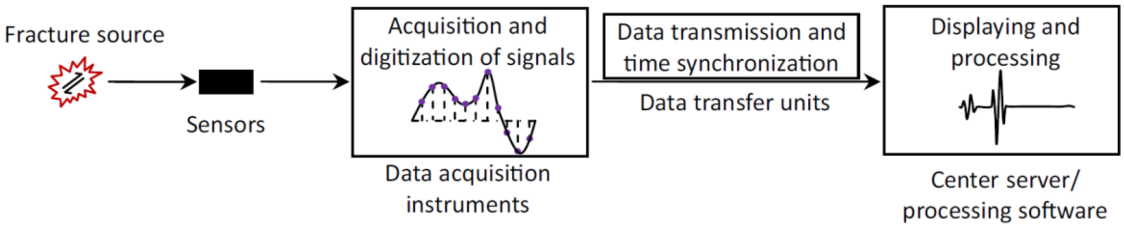
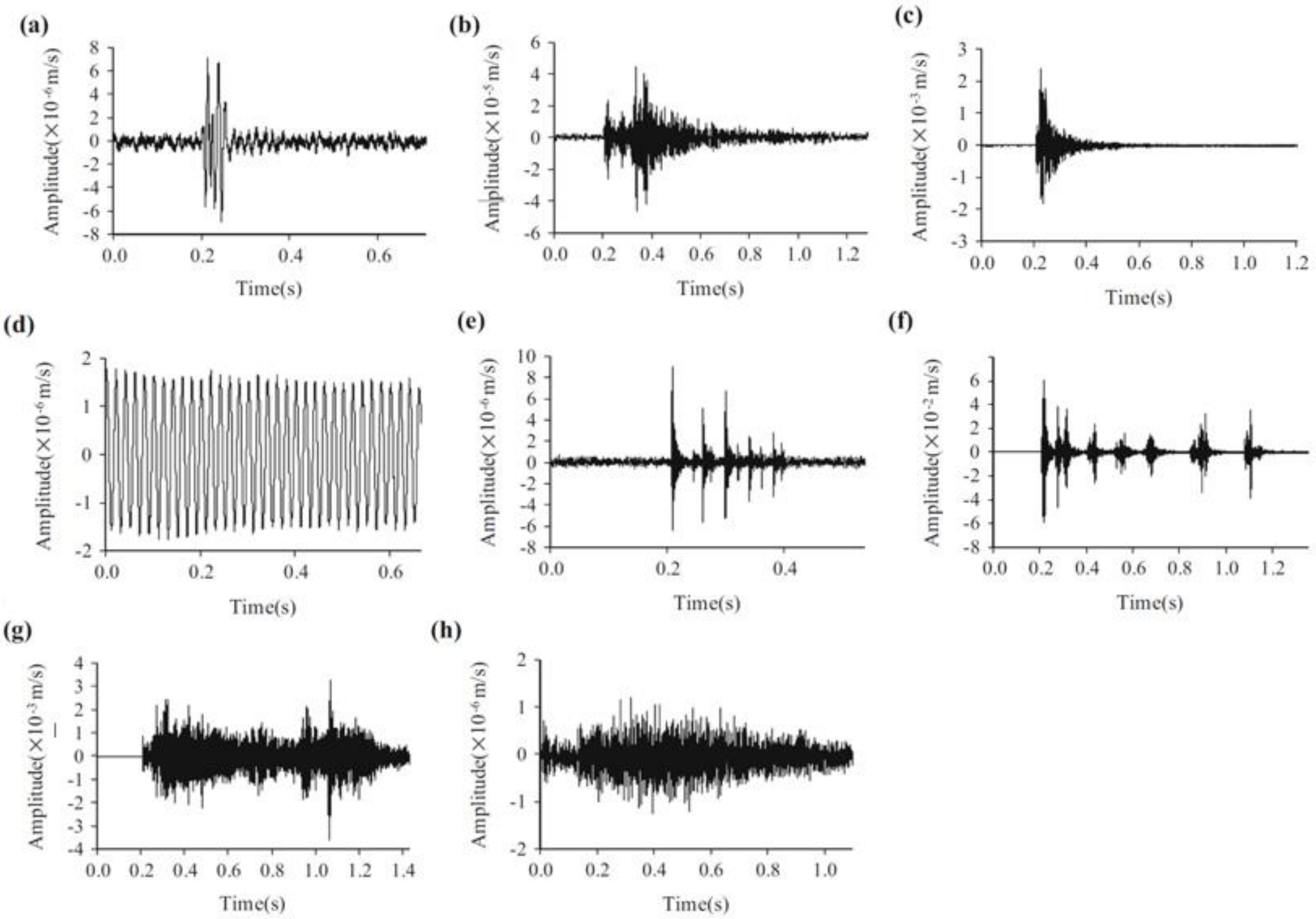


Figure 8.11. Seismic monitoring principles in underground mines (Xiao et al., 2016)

Continuous monitoring and periodic inspection form data transferring units are necessary during the monitoring process. The typical waveform signals in rock mass structures are shown in Figure 8.12. The sources of singles are fracturing in rock masses, electrical noise, drilling, blasting, mechanical vibration, and unknown such as construction vehicles. The description and characterises of each signal have been illustrated in Figure 8.12.

Seismic monitoring in deep underground excavations are classified into two scales: construction region from several hundred meters to kilometres and operational areas like underground mine stopes in the range of up to several hundred meters. The frequency of monitoring in construction region varies from a few Hertz to several hundred Hertz and geophone is an appropriate element in this scale. In the operational scale, main monitoring frequency is in the range of several hundred Hertz to thousands of Hertz. Accelerometers are used for seismic monitoring of rock masses in operational scale.

Data acquisition instruments convert amplified analogue signals into digital signals. These instruments are namely three parts: preamplifier, analogue to digital convertor or A/D convertor, and data acquisition computer (DAC) (Xiao et al., 2016). Figure 8.13 shows an example of a frequency spectrum microseismic analysis from fracturing events in rock masses. In the figure, the main frequency band [f_1, f_2] of fracturing waveform in a single rock mass is between 0.707 times the maximum amplitude.



Signals types	Characteristic description
Rock mass facture	The fracturing signal of a rock mass generally has a duration of less than 1 s. The rock mass fracturing signal has a wide range of frequency, mainly concentrated in the range of 10–3000 Hz, with an amplitude magnitude of 10^{-2}–10^{-7} m/s. The waveforms of rock mass fracturing signals are various, as shown in (a – c)
Electrical	Electrical signals are mainly generated by the improper operation and connection of various electrical components, as well as ineffective cable grounding. The electrical signal generated due to ineffective cable grounding has very similar characteristics to the local AC power signal, which is the resonance wave with the same amplitude, very long duration, and a frequency of 50 Hz, as shown in (d)
Drilling	The drilling signal is mainly generated by the drilling of blast boreholes and rock bolt boreholes. This type of signal has a notable characteristic, i.e., its multiple wave nature. The waveform within the same signal has a clear periodicity with an occurrence period. The signal has a frequency mainly concentrated in the range of 100–2000 Hz, with an amplitude magnitude of 10^{-5}–10^{-6} m/s. The waveform characteristics of typical signals are shown in (e)
Blasting	The blast signal generally has a duration of more than 1 s, and the waveform in the same blast signal has a clear periodicity of 0.1– 0.2 s, which is longer than that of a drilling signal. The blast signal received by the geophone has a frequency mainly concentrated in the range of 100–500 Hz, with an amplitude magnitude of 10^{-2}–10^{-3} m/s. The waveform characteristics of typical blast signals are shown in (f)
Mechanical vibration	The mechanical vibration signals are mainly generated by the operation of construction equipment, such as TBM movement or heavy vehicles passing. The amplitude of this signal depends on the vibration strength, as shown in (g and h)
Unknown	In addition, there may be other signals with different waveform characteristics, but for which no clear signal sources have been found on site—which may be a result of the superimposition of various ambient noises, so these signals require further analysis

Figure 8.12. Typical waveform seismic signals in rock masses, and the characteristics and descriptions of seismic signals in monitoring (Xiao et al., 2016)

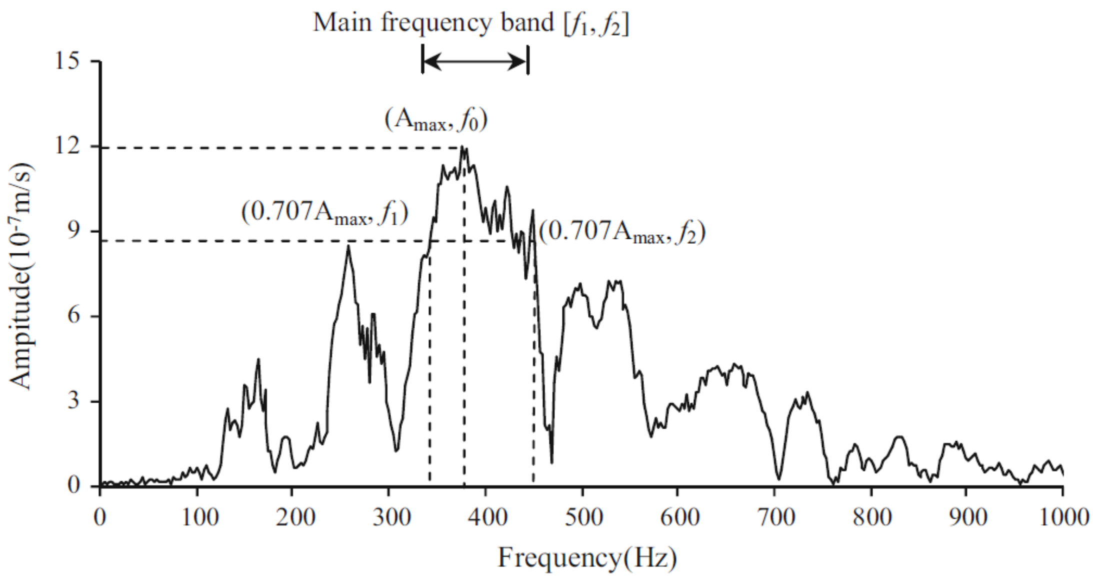


Figure 8.13. An example of data acquisition for the microseismic event from fracturing events in rock masses (Xiao et al., 2016)

Seismic data is transferred from data acquisition instruments to a central computer for processing and analysis. Figure 8.14 shows typical data transfer units is seismic monitoring. The main parts of data transfer units are sensors to data acquisition transfer, data acquisition instrument to the centre server, and centre server to data processing and analysis.

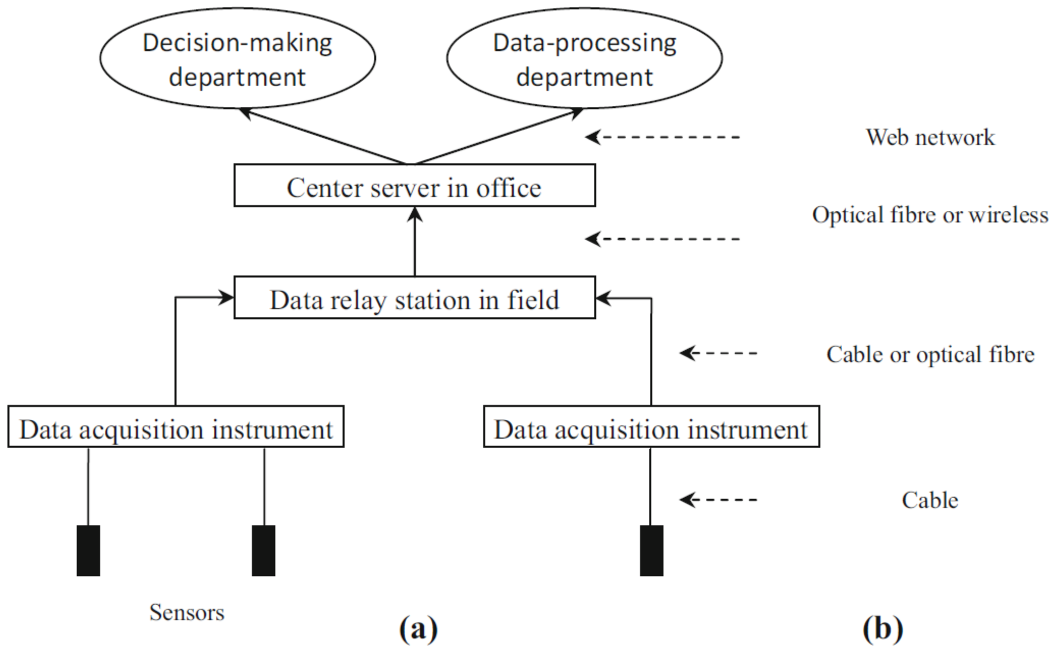


Figure 8.14. Typical units transferring data in seismic monitoring; (a) transferring between each component, (b) transfer in the main units (Xiao et al., 2016)

8.4. Back-analysis approaches

Rock underground engineering design sometimes requires in situ rock engineering parameters that are identified through back calculations or back analysis. Generally, back analysis solutions are performed by two tools (Rahimi and Sharifzadeh, 2017):

1- Determine stress, strain and displacement by stress analysis techniques such as analytical and numerical methods
2- Optimise/minimise differences between measured field data and obtained data from results of stress analysis.

In rock underground excavations strain, stress and displacement responses in rock masses and ground support elements are used to back analysis methods. Strain and stress values are very different from place to place, and a wide range of measurements require to represent appropriate values. The measurement of the displacement of rock masses surrounding excavations are easy and reliable, displacement-based back analysis has been more attention in the projects.

Figure 8.15 presents typical back analysis techniques in rock underground excavations. Generally, back analysis techniques use two approaches: deterministic and non-deterministic. Deterministic methods such as the direct approach, inverse approach and graphic methods, are based on the difference between system and model to minimise variability of the (deterministic) signal between them. Non–deterministic methods like probabilistic methods and genetic algorithms, are based on the discrepancy between model and systems, considered as a non-deterministic signal (Sharifzadeh et al., 2017).

Input material properties and modelling are checked and compared with the results of field measurements in back analysis methods. Three approaches for identification of parameters are to follow:

1- Calculation of displacements, stresses, and strain in a forward analysis by using the values of mechanical parameters in an assumed mechanical model
2- Determination of values of mechanical properties in an identification procedure from measured strains, stresses and displacements in as assumed mechanical model
3- Calculation of values of mechanical parameters in a back analysis by using measured stresses, strains and displacements in a model

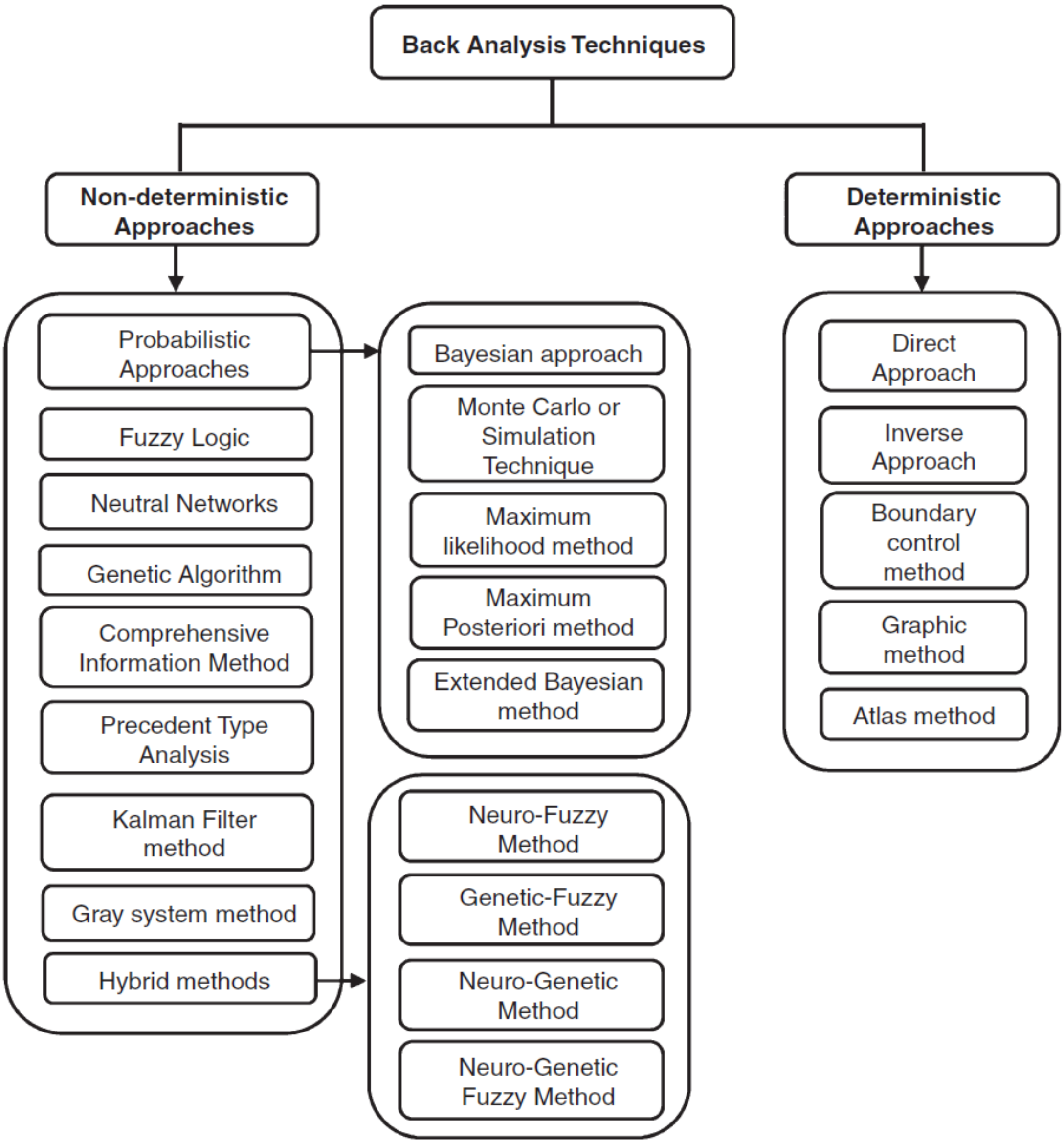


Figure 8.15. Back analysis methods in rock underground designs (Sharifzadeh et al., 2017)

Selecting parameters to use in back analysis methods should be based on their importance on stability, ability to estimate enough precise value by other methods and a reduction in a number of unknown parameters to be determined as much as possible.

8.5. Design update

In order to minimise geotechnical challenges and problems in underground mine, optimisation of design parameters are necessary for deep underground mines. Geological conditions, seismicity, depth, mineral price, and economic condition change over time. Also, uncertainty and a wide range of rock engineering parameters are in most of the design parameters. The behaviour of the rock mass is checked and controlled by monitoring. Reassessment of design is necessary for unexpected behaviours of rock masses during mining operations. A real investigation, utilisation of a comprehensive monitoring system, applying observational methods are used to minimise problems and uncertainties through optimisation phase.

The art of rock engineering is to use up-to-date analysis and methods to optimise design parameters. Implementation of various processes such as excavation access drives improves constructability during the design procedure and may enable design optimisation (Stacey, 2003). Monitoring methods used during the mine's progress provide the required data via field measurements and observation methods to optimise a mine, and this can affect safety, productivity and mining operation cost.

8.6. Conclusion

Geotechnical monitoring and field observational methods provide new data from ground condition to apply for assessing real behaviour of rock masses and ground support systems. Rock engineering properties and the initial state of field stresses are given as input data in the design analysis. Real behaviour of the ground in sophisticated structures often varies from predicted by numerical modelling. To solve this problem, design parameters should be controlled, checked and modified by geotechnical monitoring systems.

Instrumentation, monitoring, and back analysis are the main parts of field measurements systems. Instrumentation is a process of installing instrument devices in specific locations of underground mines to record and transfer data. There are various types of instrument devices like extensometer, pressure cell, electrical piezometer, accelerometer, and geophone that are used for measuring the performance of support devices and ground parameters. Generally, instrumentation systems are categorized into four groups: optical systems, mechanical systems, hydraulic/pneumatic systems, and electrical devices. Project conditions, geotechnical purposes, strategy for data collection–processing–interpretation-analysis, budget, the period of time for monitoring, and available devices are some of the essential parameters that are used for selecting instruments in mining projects. In seismicity active mines, geophone and accelerometers are installed to locations of mine site where cover microseismic sources.

The primary process for monitoring systems in deep underground mines and high-stress conditions are selecting field sites, multiple-point borehole extensometers, laser-scanning, instrumented rock bolts, borehole observation and monitoring, local seismic systems, damage mapping, and numerical modelling. Rock deformation, induced stresses, and rock structure damage zones are monitored by different methods and tools like acoustic velocity testing, bolt stress meters, extensometers, microseismic system, and digital panoramic borehole camera system. Seismic monitoring systems are associated with defining monitoring objects, selecting instrument devices, determine the appropriate strategy for data collection-processing-data analysis, and considering the methods for reduction of risks.

Back analysis methods are used to determine the accurate values of ground parameters to apply for a design update. Optimisation of design parameters is to minimise geotechnical challenges and problems in underground mines. Minimising hazards, reducing uncertainties of design parameters, obtaining value-cost/time, and improvement ground support systems are some of the crucial benefits of utilisation of monitoring systems and observational methods in deep underground mines.

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CHAPTER 9: COMPREHENSIVE UNDERGROUND EXCAVATION DESIGN (CUED) METHODOLOGY IN DEEP AND HARD ROCK CONDITIONS[1]

CHAPTER 9: CONTENTS

[1] This chapter is a summary of the submitted paper in *Journal of Rock Mechanics and Geotechnical Engineering:*
RAHIMI, B., SHARIFZADEH, M & Feng, X.-T. 2019. A New Comprehensive Underground Excavation Design (CUED) Methodology in Deep and Hard Rock Conditions. *Journal of Rock Mechanics and Geotechnical Engineering*, Reviewing

9.1. Introduction

Geotechnical design probe methods for solving a problem in practical projects. Design of underground structures usually includes the use of location, size and shape, layout, excavation process, support system and monitoring (Bieniawski, 1992). Hoek and Brown (1980) developed a design methodology for rock underground practices based on geological data, rock mass structures, groundwater condition, weathering, state of stress condition, and excavation factors. A guideline for the geomechanical design of underground construction in the Austrian society for geomechanics (OGG) published for excavation and support design (Schubert et al., 2003). Hudson and Feng (2007) updated the flowcharts for rock engineering design based on rock mechanic modelling. Modern design methodology for underground excavation projects should be updated continuously.

Diagnosis of ground behaviour and failure mechanism is not discussed in the existing design methods. Ground behaviour can vary even for the same quality of rock masses, underground water conditions, and stress field. Therefore, diagnose of ground behaviour and identification of failure mechanism/hazard recognition are crucial steps required to consider in modern design procedure. This paper aims to present the CUED method based on rock mass characterisation, ground behaviour and failure modes, design analysis to manage the ground condition, and construction and field observation. The first step in the design of an underground excavation is describing rock mass conditions that are obtained from the results of site investigation and laboratory/field tests. Characterisation of rock mass structures, critical features of projects such as geometry and orientation, field stresses, hydrology, active stress factor (rock mass strength/maximum principal stresses) are some of the essential parameters that are used for diagnosis of ground behaviour. Groundwater pressure, residual stress, seismic events, tectonized stress and induced stress make a complex and multiple loading from micro to the large scale of rocks surrounding excavation (Sharifzadeh et al., 2017). Data collection from site observations, knowledge and engineering judgment help to a good understanding of ground conditions that are required to identify the behaviour of rocks in the specific conditions (Rahimi et al., 2017). Based on diagnosed ground behaviour and identified failure mechanism, the proper approaches of ground control and management are utilised to evaluate project location, excavation sequences/extraction ratio, stability conditions and selecting ground support systems. Optimisation of design parameters is carried out by collected new data from field observational methods.

The CUED method developed based on several years' experiences of authors in underground excavations in mining and civil engineering fields and some case examples are examined in this research.

9.2. CUED Methodology

The CUED method for geotechnical design purposes is illustrated in Figure 9.1. This method has been established based on the six main stages. These stages are evaluated through three steps: Input data-processing data-output data, which is called IPO:

1- Input data, which is consists of collecting data and information from site investigation and laboratory/field tests.
2- Processing data involve collection, organisation, analysis and interpretation of data.
3- Output data that is defined as the results of a processing system in a simple group or class and be able to use in problem-solving approaches in rock engineering works.

In fact, IPOs are essential elements of CUED method in the modern design of rock underground engineering projects, which handles each step as a systematic approach. An unstable condition can be appraised as the main problem in geotechnical projects, and ground support system design and monitoring as a solution to that problem. To this purpose, rock mass structures should be characterised to estimate inherent engineering properties of rock masses. Then, instability is assessed based on ground conditions, indicator warnings such as joints and faults, and also failure precursors like fracturing and ground movement surrounding excavations. Regarding identifying failure mechanism(s), appropriate design methods (s) such as numerical or analytical methods are used for selecting required ground support systems to control risk failures and provide safety for personnel and equipment.

Acquired parameters in design analysis to manage ground control are implemented in construction stages. Underground operations at great depth are usually associated with a high-stress level in rock mass and seismic events that make severe rock failure.

Therefore, appropriate excavation methods or sequential exaction in underground mining stopes and stress management methods such as destress blasting should be employed to reduce potential sudden failure during mining operation or construction.

Also, quality control of materials is carried out to examine geotechnical quality assurance of equipment and support devices. Geotechnical monitoring and field observational methods provide new data for assessing ground-support responses. Rock engineering properties and an initial state of field stresses are given as input data in the design analysis. Real behaviour of the ground in sophisticated rock structures at great depth sometimes varies from predicted in strategic and tactical design stages. To solve this problem, design parameters should be controlled, checked and optimised by geotechnical monitoring systems. Minimising hazards, reducing uncertainties of design parameters, obtaining value-cost/time, and improvement ground support systems are some of the important benefits of utilisation of monitoring and observational methods.

The CUED method is based on a dynamic approach to use in the geotechnical engineering field for strategic, tactical, and operational design during the projects lifetime. The proposed method is more applicable at great depth and in hard rock condition, which rock masses are mostly encountered with high-stress conditions, frequently changing behaviour and potential multi-failure mechanisms. Application of dynamic approach in design methodology can provide stability of rock mass structures in underground excavations.

9.3. Rock mass composition (RMC-IPO)

Rock mass composition indicates rock masses in situ conditions before any engineering activities. Rock mass materials are composed of all unit elements of rock blocks and discontinuities. The rock materials are described by rock type, discontinuities, colour, a degree of jointing, block size and shape, and estimation of rock properties such as rock strength and deformation modulus.

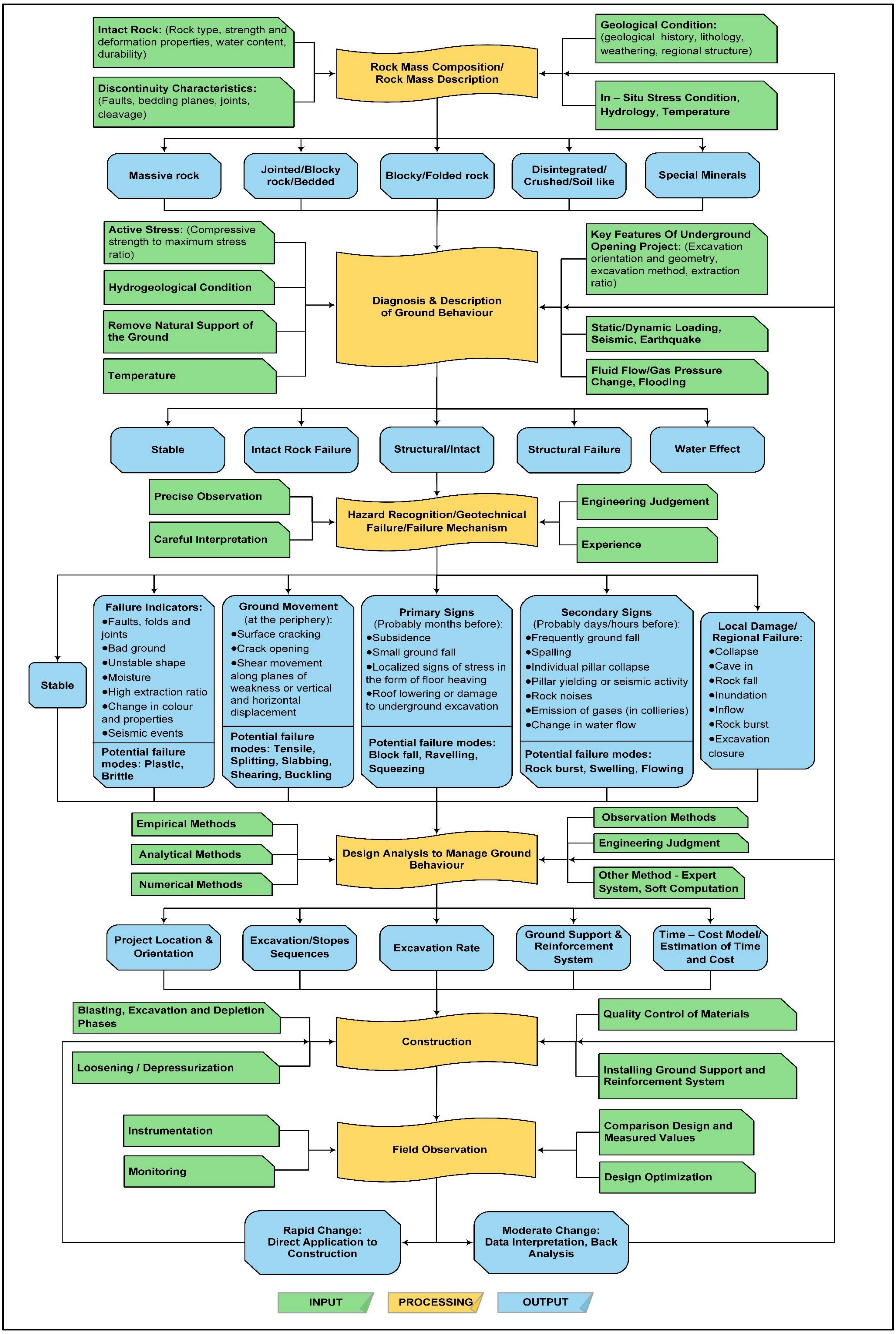


Figure 9.1. The Comprehensive Underground Excavation Design (CEUD) procedure in deep and hard rock condition

Rock mass composition is evaluated based on the rock mass type and consequent tectonics, stress, hydrological variations, and over geological time. The geological approach for engineering purposes is to investigate original rock formation, weathering, tectonic activities, geomorphology, erosion, nature of various strata, hydrogeological condition, and other features. The IPO for rock mass composition is considered in the following paragraphs. The characterisation is to estimate the qualitative and quantitative parameters of the ground. Figure 9.2 illustrates rock mass composition-(input data/ data processing/output data) approach so-called RMC–IPO in rock engineering projects based on the CUED method.

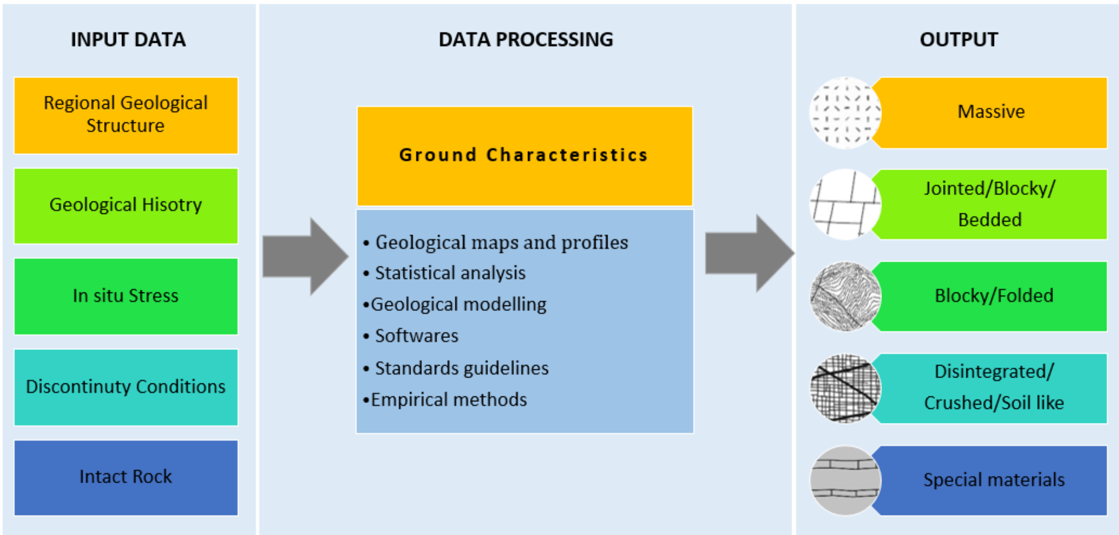


Figure 9.2. Determination of rock mass composition using IPO (Input–Processing–Output data) approach

Table 9.1 presents the classification of rock mass composition in deep and hard rock conditions. Massive rocks are described with few discontinuities or a wide range of discontinuity spacing, and homogeneous in composition. The jointed or blocky or bedded group contains rocks with few joint sets in the rock structure resulting in blocks that are very well interlocked together. The orientation and degree of joints in rocks determine the type and quality of rock blocks. In the case of the blocky or folded class, several intersecting discontinuity sets create interlocked and angular blocks. Geological folds or faults occur by tectonic activities. Disintegrated or crushed or soil ground type contains heavily broken rock mass, poorly interlocked, a mixture of rounded and angular rock fragments, and tectonically sheared weak rocks. Some minerals in rock structures affect the engineering properties of rock and control its behaviour. Swelling clay minerals such as montmorillonite and bentonite absorb water and cause an alteration or infilling discontinuities. Soluble materials such as calcite in limestone may lead to high permeability and groundwater inflow problems, especially in weakness zones in underground structures.

Characterisation of rock mass structure help to identify possible ground behaviour. The knowledge of rock mass composition and ground behaviour is used to analyse potential failure in underground excavation, which is an essential step in design procedure.

The quality of interlocking between rock blocks, the interaction between grains and blocks, shape and size of grains, and cementation of grains influence composition of the rock structures. Some minerals in rock structures affect the engineering properties of rock and control its behaviour. Swelling clay minerals such as montmorillonite and bentonite absorb water and cause an alteration or infilling discontinuities, weakness zones, and bedding that lead to a reduction of shear strength in rock materials (Stille and Palmstrom, 2008). Soluble materials such as calcite in limestone may lead to high permeability and groundwater inflow problems, especially in weakness zones in underground structures.

Characterisation of rock mass structure help to identify possible ground behaviour. The knowledge of rock mass composition and ground behaviour is used to analyse potential failure in underground excavation, which is an essential step in design procedure.

9.4. Diagnosis of ground behaviour (DGB-IPO)

Diagnosis of ground behaviours has the prime importance of underground design and define as the following expression(Sharifzadeh et al., 2017):

Diagnosis of ground behaviour (DGB) = Rock mass composition (RMC) + Underground excavation Condition (UEC) + Environmental condition (EC)

Description of rock materials and joint characteristics in rock mass structure provide sufficient information to predict expected ground behaviour. Major geological structures, active stress factor, hydrological condition, static/dynamic loading conditions in the ground are the typical natural environmental factors, which may change and influence the ground behaviour during construction.

Table 9.1. Rock mass composition classes and its attributed characteristics utilised in the CUED method

Rock mass composition classes	Geological condition	Discontinutiy condition	In situ stress effect	Intact rock and strength characteristics
Massive	Massive; homogenous; no or a few discontinuities; schistose; igneous rocks with massive [light(quartz rich) to dark colure (quartz poor)] and porphyry structures and crystalline texture, metamorphic massive rocks such as marble; mineral grain similar size or composition; coarse grain size like granodiorite; medium grain size such as dolerite; fine grain size e.g.basalt	• Surface and regional fractures; one or two discontinuitiy sets (partially to well interconnected); widely spaced discontinuties; discontinuties have less effect on rock mass properties; • spacing > 2m • volumetric joint count (joints/m^3) < 1 • block volume > 8m^3	More affected by gravitaitonal stress	$\sigma_{ci}(MPa) > 50$ $0.4 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $1 < \frac{\sigma_{ci}}{\sigma_{cm}} < 2$
Jointed / Blocky / Bedded	Igneous and metamorphic jointed rocks; homogenous foliated and bedded rocks; jointed seam and blocky ground; cemented minrals with inter-granular matrix mateiral in sedimentary rocks; usually anisotropy in physical properties; layered rocks; massive rock with some joint sets; coarse grain size like conglomerate; medium grain size for example tuff; fine grain size such as siltstone	• Three or more discontinuity sets(partially to well interconnected); discontinuous microfractures may be present near parallel to the layering; regional, surface and sometimes tectonic fractures; bedding plane; • 0.6m < spacing < 2m • 1 < volumetric joint count (joints/m^3) < 3 • 0.1m^3 < block volume < 8m^3	More affected by gravitational and tectonic streses	$40 < \sigma_{ci}(MPa) < 250$ $0.3 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $2 < \frac{\sigma_{ci}}{\sigma_{cm}} < 10$
Blocky / Folded	Jointed blocky rocks and structrues like foliations and veins; foliated rocks; foliation in gneisses; sliding or toppling of blocks; clastic; crystalline; slightly to moderately decomposition of minerals shearing during folding; tectonically deformed; less or moderately folded-broken and deformed; anisotropy; schistosity; flaky minerals in rocks; thin bedded rocks and folding	• Four or more discontinuity sets (poor to partilally interconnected); folded with angular blocks; main structures are freacture, joints, bedding plane and minor faults; tectonic, contraction (extension or tension joints) and surface(shearing) fractures; schistosity and folation planes; • 0.1m < spacing < 0.6m • 3 < volumetric joint count (joints/m^3) < 30 • 0.1dm^3 < Block volume < 100dm^3	Local stress concentration; mostly affected by tectonic, residual and surface stress	$25 < \sigma_{ci}(MPa) < 250$ $0.2 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $10 < \frac{\sigma_{ci}}{\sigma_{cm}} < 40$
Disintegrated / Crushed / Soil like	Shearing during folding or faulting; some minerals may be altered or decomposed; boundaries commonly slickensided; crushed zone composed of disoriented; usually angular fragments of the host rock substances; tectonically deformed; laminated; heavily jointed; tectonized rocks; highly jointed or crushed rocks; poorly cemented rock materials; hetergeneity; change of minerals; cracked massive rocks; loose structure and composition of the rocks from alteration and/or chemical weatheirng; rock material similar soil with low friction properties; rock fragments with few contacts; foliated shears; heaviliy jointed or breciated dykes or layers	• More than four discontinuity sets (poor interconnected); mostly tectonic, contraction and surface fractures; lowest shear strength in direction of slikensides in plane parallel to boundaries; • spacing < 0.1m • volumetric joint count (joints/m^3) > 30 • block volume < 0.1dm^3	More affected by tectonic stresses	$\sigma_{ci}(MPa) < 25$ $0 < \frac{\sigma_{ci}}{\sigma_{ci(fresh)}} < 1$ $\frac{\sigma_{ci}}{\sigma_{cm}} > 40$
Special Materials	Minerals or rocks with special properties; clay minerals; swelling minerals; soluble minerals; thin layer or forming a chaotic structure with pockets of clay; layers or lense of clay; clacite	• May discontinuities filled with special materials and in presence of water affected by alteration and/ or hydrothermal actions; clacite containing in weakness zones; development of cavities in limstone	Gravitaitonal and topographical stress	$1 < \sigma_{ci}(MPa) < 100$

σ_{ci}: Uniaxial compressive strength of intact rock
$\sigma_{ci(fresh)}$: Uniaxial compressive strength of intact rock at fresh state
σ_{cm}: Rock mass strength

$$Weathering\ Factor = \frac{\sigma_{ci}}{\sigma_{ci(fresh)}}$$
$$Strength\ Scaling\ Factor = \frac{\sigma_{ci}}{\sigma_{cm}}$$

Figure 9.3 shows ground reactions in underground excavation due to engineering activity and natural environmental conditions. The responses of ground in loading can be elastic/plastic deformation, consolidation settlement and failure. Ground movement, fracturing, filling voids and cave–in, and collapse are the typical results of removing natural support in the ground. Also, some engineering/natural process in the ground such as mineral extraction and change groundwater level due to precipitation may lead to fluid flow/ gas pressure change.

Rock masses at great depth have a complex structure and describing ground behaviour requires a fundamental level of knowledge, experience and engineering judgement. Figure 9.4 shows the IPO approach for diagnosis of ground behaviour, so-called (DGB–IPO). Input data in the flowchart are the characterisation of rock masses, geometry and orientation of excavations, excavation methods/sequential excavations/extraction ratio, field stresses, and groundwater. These parameters are used for determination of rock mass behaviour.

Knowledge, experience, and engineering judgement are the main principles during the processing of input data to foreseen target behaviour in ground condition. Identification of the main reasons for rock mass behaviour assists to distinguish failure modes. At great depth, when a failure in ground conditions is not predicted or distinguished, rock mass may behave in unforeseen ways, and sometimes the condition of good ground decreases in quality due to a variety of factors such as blasting quality (Rahimi and Sharifzadeh, 2017). The behaviour of the ground condition may change during or after the construction stage and with time. Hence, the evaluation of short and long-term rock behaviour is required in excavations.

9.5. Identify rock failure mechanism (GB-FM)

Typically, a failure mechanism in rock materials consists of three steps: elastic behaviour, ductile behaviour and failure. Also, post ductile/plastic behaviour of rocks can be described as strain-hardening, perfect plastic, strain-softening, and brittle types (Figure 9.5). Failure mechanism in an intact rock depends on rock type, texture and structure, physical characteristics such as density and porosity, loading condition, temperature, confining stress, and saturation. Stress concentration during and after an underground excavation influences the properties of rock masses and may lead to happening various ground behaviour and failure modes.

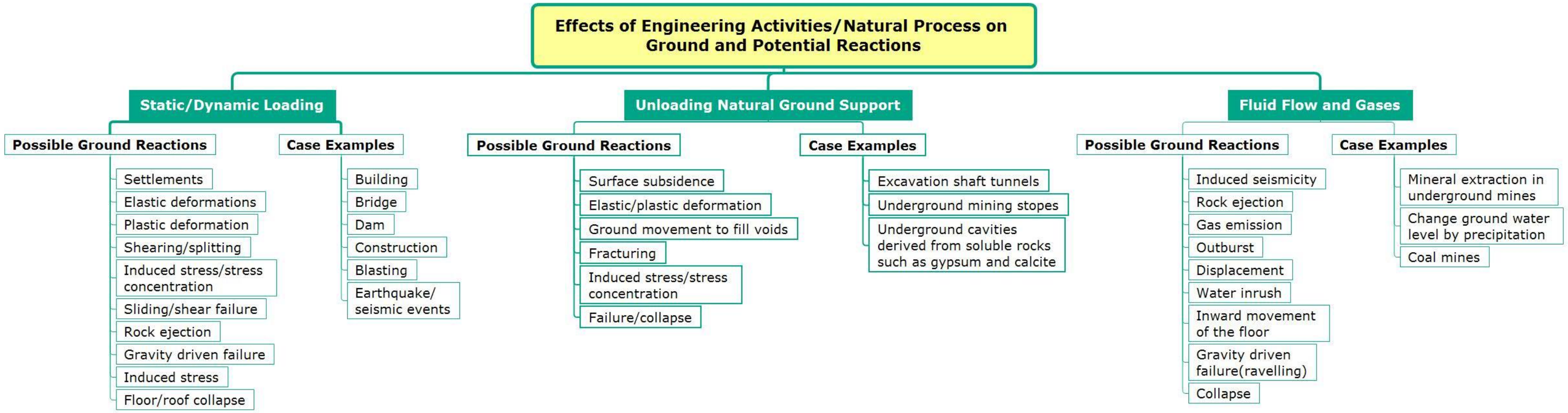


Figure 9.3. Possible ground reactions due to engineering activities/natural environmental conditions

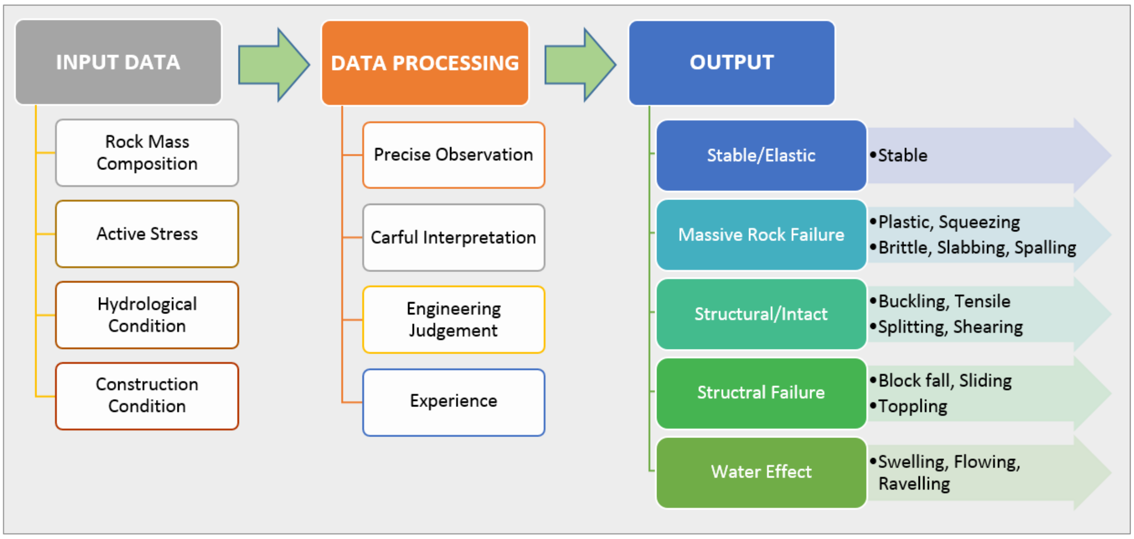


Figure 9.4. The flowchart for the diagnosis of ground behaviour (DGB–IPO) in underground excavations based on the CUED method

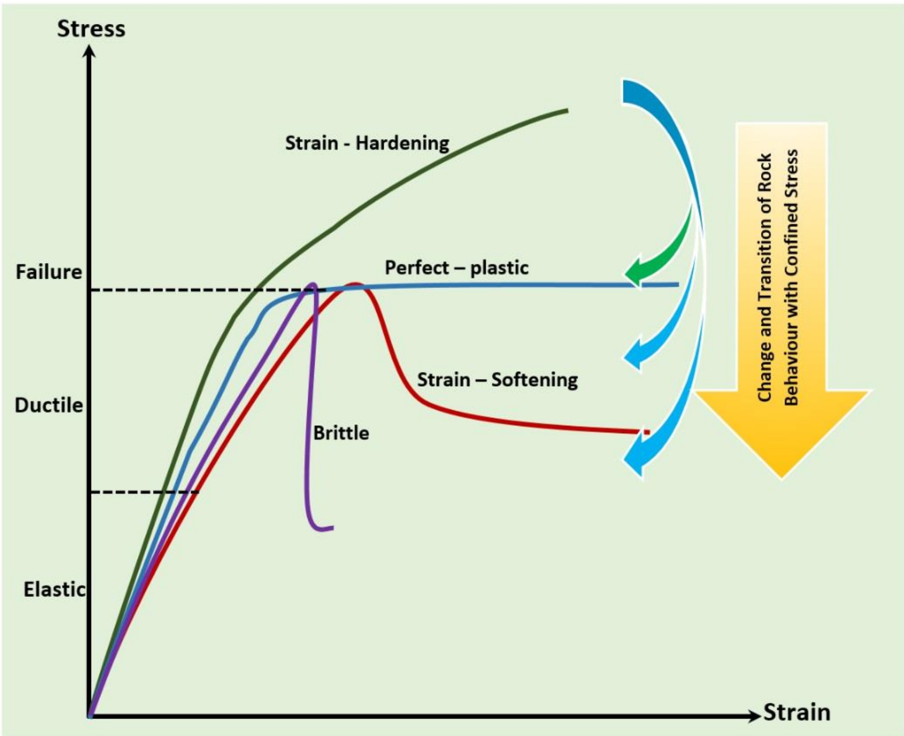


Figure 9.5. Different types of behaviour and failure mechanisms of intact rocks

Figure 9.6 shows the effect of stress–concentration in rock mass behaviours. In the first step, rock mass surrounding excavation has an elastic deformation due to induced stress. Then, plastic behaviour is dominant, and stress concentration is reduced because of the increasing deformation of rock materials. Exceeding plastic deformation may lead to imitation of failure that can be progressed by failure precursors during deformation intensity. Finally, the process may cause occurring failure in the opening.

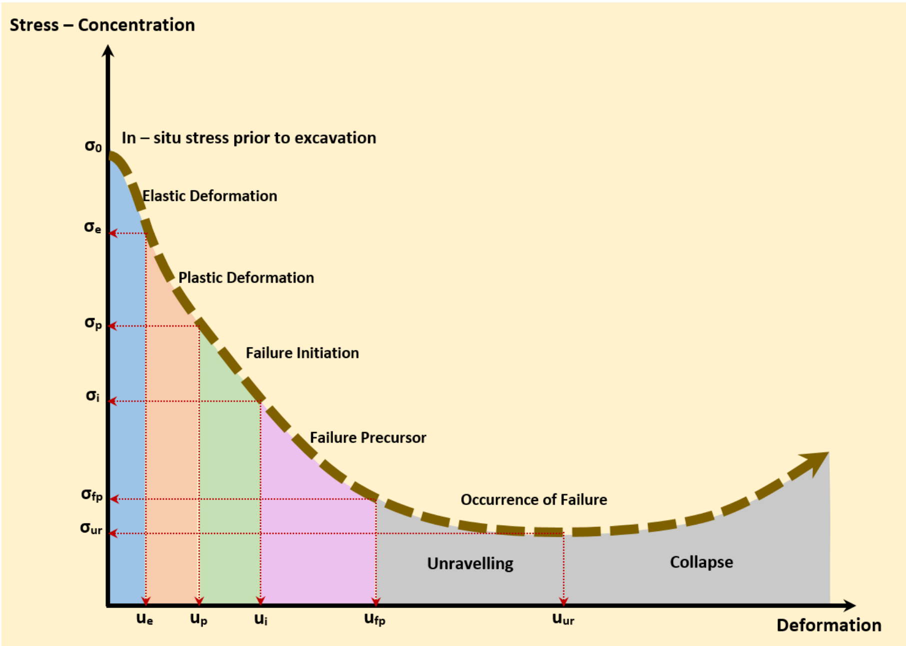
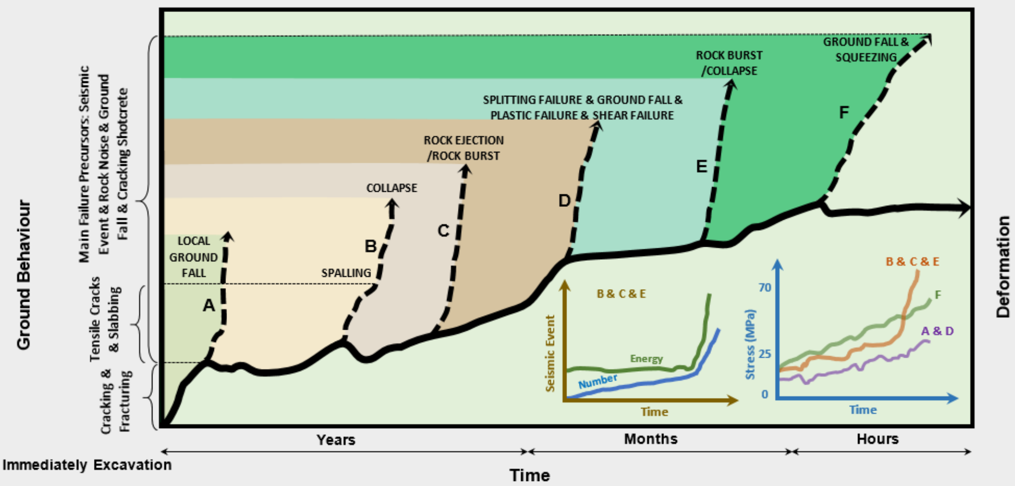


Figure 9.6. Rock mass behaviour surrounding excavation due to stress concentration during and after excavation

Figure 9.7 presents a combination of different types of ground behaviour and failure modes against time, during and after processing of the excavation stage. According to the figure, cracking and fracturing happened after completing excavation, due to high-stress conditions. Intersecting major joint sets appeared in the excavation surface in jointed/blocky rock structures. Therefore, ground fall and wedge/sliding failure occurred. Spalling and rock burst failure frequently occurred during excavation and after installation of a ground support system in underground openings. The failures were accompanied by cracking, fracturing, increased rate/number of seismic events, and sometimes local ground falls before a sudden release of energy and ejection of rock. The main failure precursors in rock mass structures were noticed years, months and even hours before the final failure/collapse. Typical failure modes in underground excavations with weak geological structures are tensile failure, block fall, and shear/slip failure, plastic squeezing, and structural stress induced failure/collapse.



Description:

A. Appear intersecting discontinuities (immediate excavation) → loading/unloading ground (from a few hours to years) → incremental loosening and distorting major joints → reduce friction joint surfaces → unstable shear strength of blocks → opening rock blocks → sliding blocks / ground fall (from a few hours to years)

B & C & E. Initiation of cracks (immediate excavation) → propagation fractures → tensile cracking / extension parallel slabbing (from a few days to years) → accumulate strain energy → split rock flake → shotcrete cracks/cable bolts begin to carry significant load (from a few hours to a few days) → small ground fall → increase the number / rate of microseismic events → frequently increase sharp noise (hours – days before occurring failure) → large ground fall / violent break of rock → release energy with rock ejection / spalling / rock burst (from a few minutes to hours, and 5 – 30m far from stopes) → collapse

D. Appealing cracks on the surface → fracturing wall and crown of excavation → ground movement → joint dilation → shear fractures / tensile fractures → extrusion of joint infilling → excavation convergence → local ground fall → failure / collapse (from a few hours to years)

F. Generation new cracks → growth fractures → fracturing at ground surface of excavation → interconnection of existence and new fractures → diminish adhesive force on the interface → ground movement → plastic flow → additional cracks → vertical and horizontal displacement → shear failure → excavation convergence → large scale of ground fall → collapse (from months to years)

Figure 9.7. Ground behaviour modes and failure mechanisms over time at great depth

The flowchart for identifying ground behaviour and failure mechanism in rock mass structures surrounding underground excavations at great depth is given the name Flowchart GB–FM and shown in Figure 9.8. Ground condition is stable for active stress greater than 5 and continuity factor less than 6. At active stress between 2 and 5, failure mechanism is estimated as plastic, splitting, shearing and ground fall. Moderate to high level of field stress in the massive and blocky rocks (CF < 30) make a sudden and violent rock slab from sides and roof in underground excavations. Rock burst failure is a sudden and violent failure of hard, massive and blocky rock, high-stress condition and seismic events. Squeezing behaviour is a type of time-dependent behaviour in the moderate to hard rocks with a high-stress condition. Squeezing ground observed a type of blocky and folded classes typically. Weak interlayer zones in hard rock and overstress lead to squeezing failure during or after construction period. In some cases only one failure mechanism is dominant, but in many cases, a failure may start with one mechanism and then followed by other mechanism or combination of other mechanisms. This makes the design more complicated but should be considered in design and construction.

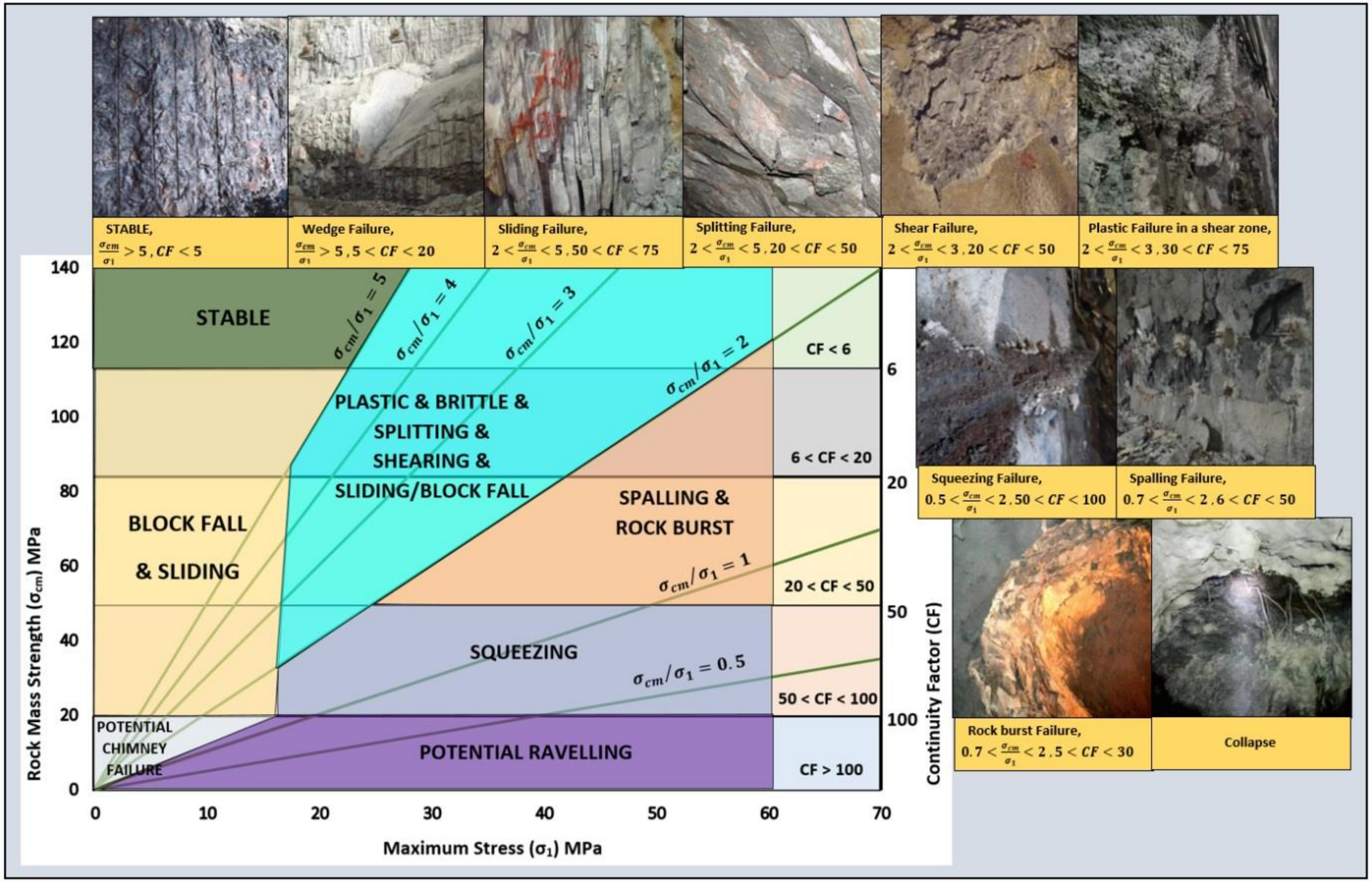


Figure 9.8. The flowchart for identifying ground behaviour and failure mechanism in deep underground excavations (Flowchart GB-FM)

9.6. Design analysis for ground control and management

Ground control and management are associated with a process to manage and control ground behaviour and failure modes in underground openings. Ground control methods can be considered in both initial design and modification of design during the whole life of projects. The main three stages for this issue are experience-based design, management based design and technically based design. The experience-based design is defined using the previous empirical experience of ground control. The term management is related to using appropriate ground control with economic and cost-effective methods. Using an appropriate method for drilling, blasting pattern and excavation can reduce intensity damage in the rock mass. Shape, size and orientation of the underground opening influence of potential instability.

Figure 9.9 shows a flowchart of design analysis for ground control and management-input/processing/output data approach based on the CUED method, and that is named GCM–IPO. Diagnosis of ground behaviour, identification of failure mechanism and geometry of excavation are used as input data for design analysis. The conventional design analysis methods to manage ground behaviour consist of empirical methods, analytical methods, numerical methods and observation methods, neural network and expert system. Ground control and management in the CUED are established in excavation strategies, design analysis strategies and support strategies. In stables condition, full face and half face excavation method can be used for underground construction. Massive and blocky rocks are more competent to be stable. In the case of structural failures, which block fall, sliding, buckling, shearing and toppling are common failures, sequential method for excavation and key block theory discontinuity deformation analysis, analytical method and shear stress analysis are the most important strategies to utilize support devices for deformation control, reduce stress concentration and unify the zone of failures. In a high level of stress field, ground failure modes are slabbing, spalling, rock burst and squeezing. Instability of ground condition should be evaluated based on energy release rate, rock burst tendency index, discontinuity deformation analysis and observational methods.

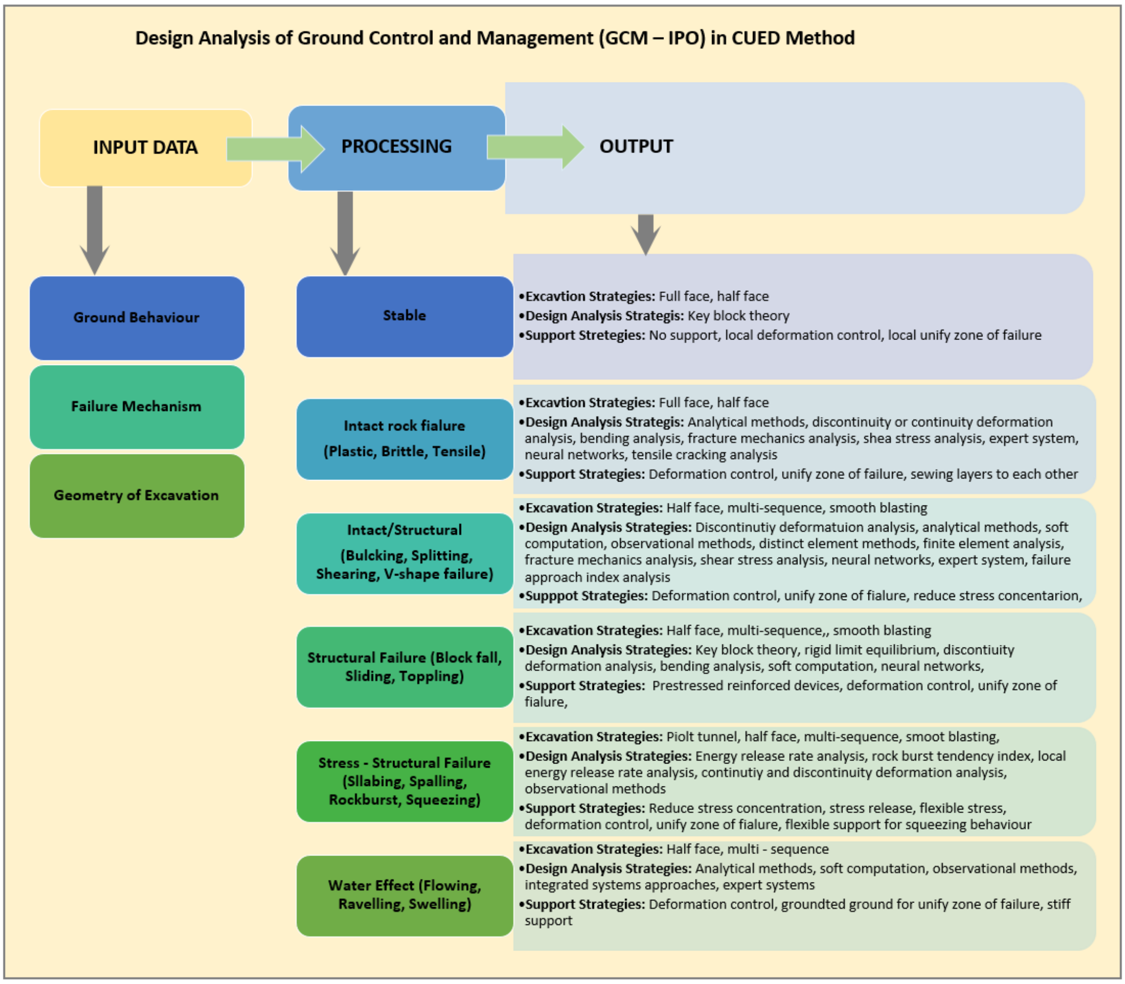


Figure 9.9. Design analysis, excavation and ground management strategies in the CUED method

Design analysis to ground management determine required ground support and reinforcement system, excavation method, sequential excavation, extraction rate in underground stopes and a time–cost model for the project. Design of ground support system under dynamic loading is required an understanding of the failure mechanism, loading mechanism and assessing energy absorption of ground support.

9.7. Construction and field observation

Acquired parameters in design analysis to manage ground control are implemented in the operational stage in underground excavations. The construction is associated with the process of the excavation in rock mass structures and stabilising ground in unstable conditions. Field observation and monitoring system is to provide new measurements data to determine the performance of a ground-support system and optimise design parameters. A summary of the construction and monitoring stages based on the CUED method is presented in Figure 9.10.

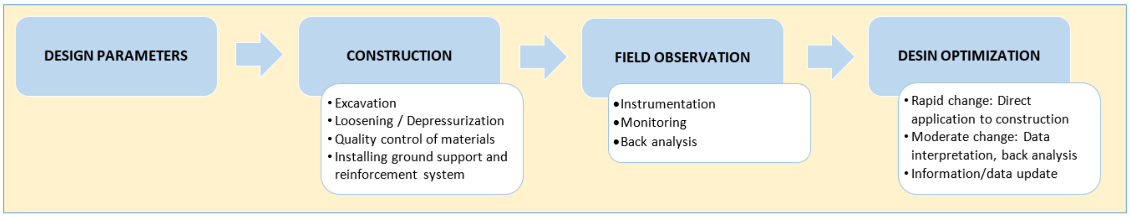


Figure 9.10. The main principles of construction and monitoring stages in CUED method

The most important operational and construction approaches in deep underground openings are excavation, scaling, depressurisation/stress management, managing waste materials, control harmful gasses with an appropriate ventilation system, supplies required energy and water, quality control of material, and installing ground support system, which is shown in Figure 9.11.

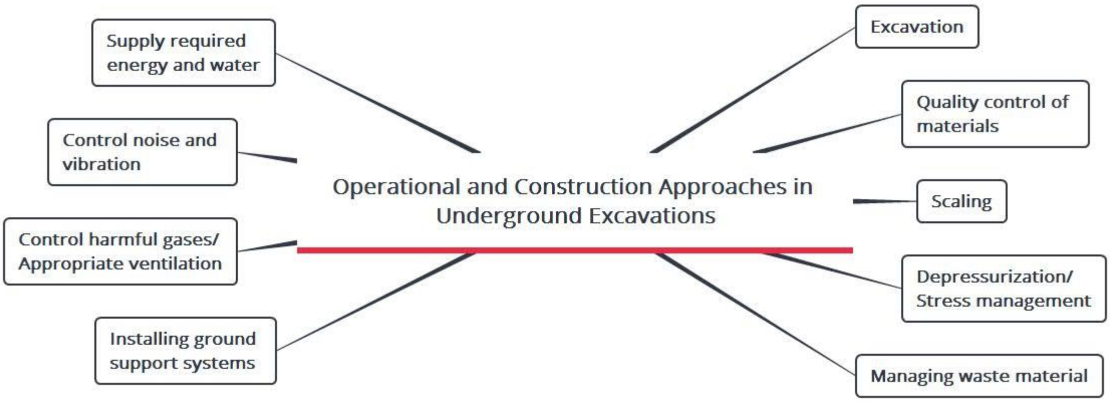


Figure 9.11. Operational and construction approaches in underground excavations

The aim of field measurements is usually to recognise relevance between disturbed rock structure condition and design parameters on the ground to reduce risks and problems. Monitoring measurements provide high precision data and temporal resolution in real time in geotechnical projects, and some of the highlight points are (Ghorbani et al., 2012):

6- Disclose unknown parameters in the design process
7- Verification of design parameters and assumptions
8- Be able to apply observational methods during or after the construction phase
9- Optimisation of design parameters and procedure to enhance safety condition, and as a result save money and time

10- Enable to Ensure safety of adjacent structures due to construction

The monitoring program can be defined as; installing sensors for during a particular time; observe, record data and check the progress quality; and modify design parameters. Geomechanical monitoring through responses to rock mass structure is used to considering real ground behaviour and improve safety in underground excavations.

9.8. Discussion and Conclusion

The CUED method presented based on evaluation and determination of geotechnical design parameters in IPO approach. Different methods are used in data processing. The complexity of ground condition at great depth, availability of data, design phases (strategic design, tactical design, and operational design), project purposes, life-term of excavation are some of the critical parameters related to selecting appropriate data processing methods during the design procedure. The advantage of CUED method is as a dynamic method and applicable for all type of ground condition and any underground excavation from shallow to great depth, low stress to high-stress level. The proposed methodology has been justified for several underground mining excavations at great depth with high stress and hard rock conditions.

The fundamental approach of CUED methodology is predicting instability condition and potential failure before occurrence in underground excavations. Based on the diagnosis of ground behaviour and failure mechanism, appropriate tools or methods for design analysis concerning the complexity of the ground condition and project phase are selected. Design methods such as empirical methods, numerical methods, and analytical methods are used to analyse excavation methods, the design of rock support and reinforcement system. In complex ground condition, observational methods and engineering judgement are more suitable to evaluate the unstable condition.

Design parameters are usually associated with a number of uncertainties and unknown parameters in the ground condition. Optimisation of design parameters is used by field observations to modify and update rock engineering properties and ground support measurements.

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CHAPTER 10: CONCLUSION AND RECOMMENDATIONS

CHAPTER 10: CONTENTS

10.1. Summary

The thesis has dealt with the design of ground support and reinforcement systems in deep underground mines based on ground conditions and failure mechanisms. In recent decades, there is a tendency for excavations at great depth to put up a variety of challenges for rock engineers related to instability issues. Developments in modern rock engineering attempt to reduce problems via increase of productivity and cost reduction in projects. A variety of factors influences underground excavation design. The most critical parameters are geological conditions, rock mass structures, construction condition, stress concentration and hydrological condition. Due to various effective factors, the complexity of ground condition, and uncertainties in each project, the procedure of design should be specialised for each project concerning specific factors and conditions. A comprehensive database from several underground mines with extensive fieldwork was collected, analysed and used for developed methods of ground characterisation, diagnosis of ground behaviour, identify failure mechanism, ground support design. Some of the case studies have been presented in the related chapters. This has provided a data-rich and has made it possible for the objective of the research program to be achieved.

An extensive literature review was described in Chapter 3, and the importance of ground behaviour modes and failure mechanism in the design of ground support system in deep underground mines was discussed and explained.

The effective features of rock structures were considered for ground characterisation at great depth and hard rock conditions. The characterisation process was established in three steps: input data, data processing, and output data. The input data was the collecting data and information from intact rocks, discontinuities, geological conditions, and hydrology. Data processing for ground characteristics is intact rock characteristics, discontinuity characteristics, weakness zone characteristics, and rock mass characteristics. Output data of the characterisation process resulted in a new developed classification for deep underground mines. The classification is associated with the description of geological conditions, characteristics of discontinuity conditions, assessment of in situ stress effects, and determination of intact rock and strength characterisation. The main classes in proposed classification are massive rocks, jointed/blocky/bedded rocks, blocky/folded rocks, disintegrated/crushed/soil like materials, and special materials.

Natural environmental conditions and engineering activities cause a change in the behaviour of rock mass structures in deep underground mines. The main steps for diagnosis of ground behaviour were established below:

1- Considering influence factors in ground behaviours (Input data) that are major geological structures, ground loading factor, hydrological condition, static and dynamic loading condition, and critical features of projects
2- Data processing, which is calculation, interpretation and analysis of input data
3- Deep ground behaviour modes (output)

According to these steps, ground behaviour modes in deep underground mines are diagnosed with stable, massive rock failure, intact/structural failure, structural failure, and water effect classes.

Identification of failure mechanism in intact and rock mass scales are necessary for the design of ground support systems. Failure mechanisms in intact and hard rocks are identified as elastic deformation, microcracking, the creation of tensile fractures, propagation of fractures and brittle failures such as slabbing and spalling. The progress of failure in rock mass structures are stable, the failure warning such as sliding rock blocks, primary signs, for example, small ground fall, secondary signs like the violent breaking of rocks, and local damage/regional failure. Failure mechanisms at great depth were classified into three groups: structural failure, induced stress/seismic failure, and operational failure mechanism. According to maximum stress, rock mass strength, and continuity factor, a flowchart for identification of failure mechanism at great depth was proposed in Flowchart GB–FN. Also, the progress of failures in rock mass structures over time at great depth was presented.

A ground control and management strategy were presented corresponding to the three stages of projects: strategic design, tactical design, and operational design. Strategic design results in preparing a broad plan and primary design for mining excavations. The tactical design is to provide detail design such as stabilisation methods. The operational design stage is related to monitoring and update design parameters. Additionally, a new procedure for the design of ground support systems for deep and hard rock was proposed. The main principles are: static and/or dynamic loading types, determination of loading sources, characterization of geological conditions and the effects of orientation of major structures with openings, estimation of ground loading factor, identification of potential primary and secondary failures, utilization of appropriate design analysis methods, estimation of depth failure, calculation of the static and/or dynamic demand ground support capacity, and selection of surface and reinforcement elements. Gravitational force is the dominant loading force in low–level stresses. In high-stress level, failure mechanism becomes more complex in rock mass structures. In this condition, a variety of factors such as released stored energy due to seismic events, stress concentration, and major structures influence ground behaviour, and judgement is very complicated. The key rock engineering schemes to minimise the risk of failures in high-stress levels at great depth involve depressurisation and quality control of materials and are considered throughout the underground construction and mining operations phases. Microseismic and blast monitoring throughout the mining operations are required to control sudden failures. Proper excavation sequences in underground stopes based on top-down, bottom-up, centre-out, and abutment-centre are discussed.

Geotechnical monitoring and field observational methods are used for providing new data to evaluate the real behaviour of rock masses and ground support systems. Field measurements consist of instrumentation, and back analysis methods. There are various types of instrument devices like extensometer, pressure cell, electrical piezometer, accelerometer, and geophone that are used for measuring the performance of support devices and ground parameters. Rock deformation, induced stresses, and rock structure damage zones are monitored by different methods and tools like acoustic velocity testing, bolt stress meters, extensometers, microseismic system, and digital panoramic borehole camera system. Seismic monitoring systems is an appropriate method for seismically active mines that are associated with recording – interpretation-analysis seismic events, and considering the methods for reduction of risks. The results of field monitoring and observational methods are used for the optimization of design parameters.

Furthermore, a new methodology “Comprehensive Underground Excavation Design” and called CUED method with emphasis on diagnosis of ground behaviour and failure mechanism(s) in deep and hard rock conditions. The CUED method proposed in six steps including rock mass composition, diagnosis of ground behaviour, failure mechanism, design analysis to manage ground behaviour, construction, field measurements and modification. A procedure has been defined for each step by determination of input data, processing data and output data, so-called IPO. IPO is applied to determine parameters of the CUED method in each step.

To verify the proposed design procedure, several case studies from in-depth underground mining projects have been studied, and some typical cases were presented in the thesis. The reliability of the design procedure for deep and hard rock conditions was justified. The thesis results indicated that proposed methods efficiently increase the safety and optimise the project's cost and time.

The key points and results of the thesis are listed as below:

- Developed a new classification for ground characterisation in deep–hard underground mines following the geological condition, discontinuity characteristics, in situ – stress effect, and intact rock and strength characteristics
- Proposed a methodology and procedure for the diagnosis of ground behaviour modes in deep underground mines concerning rock mass structures, stress field conditions, and critical features of the projects
- Evaluated intact rock and rock mass failure mechanisms in deep underground mines and proposed a new flowchart for identifying failure mechanism based on maximum principal stress, rock mass strength, and continuity factors in underground excavations
- Proposed a ground control and management strategy based on strategic design, tactical design, and operational design
- Proposed a procedure and classification for ground support design in deep and hard rock condition according to ground loading types and sources, characterisation of major geological structures, determination of ground loading factor, identification of primary and secondary potential failure, and estimation of static/dynamic support demand
- Assessed operational/construction approaches in deep underground mines with emphasis on excavation methods, stress management, and quality control of materials
- Considered geotechnical monitoring and design update based on instrumentation systems, monitoring design, and back analysis methods

- Proposed a new comprehensive underground excavation design (CUED)
- Applied, analysed, evaluated, and justified proposed methods on several case examples in deep underground mines and excavations

10.2. Recommendations for future work

The primary objective of this research program was to design ground support and reinforcement system in deep underground mines based on ground conditions. The application of developed and proposed methods for ground support design is mostly for hard rock condition. The approaches would be used and updated for medium and soft rocks in underground mines.

There was a limitation of available data in in-depth mining projects to use the results of the monitoring system for quantifying the loading types, preliminary-secondary failure modes in proposed classification of ground support system in this thesis.

Appendix: Co-author attribution statement

RAHIMI, B., SHARIFZADEH, M & Feng, X.-T. 2019. Ground Behaviour Analysis, Support System Design and Construction Strategies in Deep Hard Rock Mining–Justified in Western Australian's Mines. *Journal of Rock Mechanics and Geotechnical Engineering,* Accepted

	Conception and design	**Acquisition of data & method**	**Data conditioning & manipulation**	**Analysis & statistical method**	**Interpretation & discussion**	**Final Approval**
Behrooz Rahimi	65%	65%	65%	65%	65%	65%
Mostafa Sharifzadeh	30%	30%	30%	30%	30%	30%
Xia-Ting Feng	5%	5%	5%	5%	5%	5%

RAHIMI, B., SHARIFZADEH, M & Feng, X.-T. 2019. A New Comprehensive Underground Excavation Design (CUED) Methodology in Deep and Hard Rock Conditions. *Journal of Rock Mechanics and Geotechnical Engineering*, Reviewing

	Conception and design	**Acquisition of data & method**	**Data conditioning & manipulation**	**Analysis & statistical method**	**Interpretation & discussion**	**Final Approval**
Behrooz Rahimi	60%	60%	60%	60%	60%	
Mostafa Sharifzadeh	30%	30%	30%	30%	30%	
Xia-Ting Feng	10%	10%	10%	10%	10%	

RAHIMI, B., SHARIFZADEH, M & Feng, X.-T. 2019. An Updated CUED (Comprehensive Underground Excavation Design) Methodology in Deep Underground Hard-Rock Mining. (In Progress).

	Conception and design	**Acquisition of data & method**	**Data conditioning & manipulation**	**Analysis & statistical method**	**Interpretation & discussion**	**Final Approval**
Behrooz Rahimi	65%	65%	65%	65%	65%	
Mostafa Sharifzadeh	30%	30%	30%	30%	30%	
Xia-Ting Feng	5%	5%	5%	5%	5%	