CERTIFICATE IN ROCK MECHANICS

LEARNING GUIDE FOR:
PART 2 : BASIC ROCK MECHANICS PRACTICE

SOUTH AFRICAN NATIONAL INSTITUTE OF ROCK ENGINEERING

CHAMBER OF MINES OF SOUTH AFRICA

MINING QUALIFICATIONS AUTHORITY

PREPARED BY:
MIDDINDI CONSULTING (PTY) LTD
(In cooperation with SIM Mining Consultants, D Arnold, N Ndeweni)

EDITED BY:
Dr MKC Roberts

LAYOUT & DESIGN BY:
The Image Foundry

CHAPTER 1

PAPER 2 LEARNING GUIDE: BASIC ROCK MECHANICS PRACTICE

The learning material presented in the following chapters have been developed to assist the persons studying for the Chamber of Mines Rock Mechanics Certificate, currently the only qualification that deems a rock engineering practitioner in the South African Mining Industry as 'competent'.

The material provided in the following chapters:

- is not meant to be rock engineering handbook, but a learning guide;
- is not be a complete re-write of information already captured in technical handbooks or papers but will include the minimum information to ensure that understanding can be created;
- is referenced to outcomes in the syllabi to assist the learner to ensure that all topics are covered;
- is referenced to other rock engineering handbooks that should be used by the learner to ensure complete coverage of topics;
- provide practical examples, photographs and other important or interesting information where deemed appropriate or required;
- is enhanced by exercises and case studies to illustrate the principles presented in the main text;
- animations and interactive tools on selected topics can be utilised to assist in understanding and applying specific concepts;
- includes questions for self-testing;
- is referenced to a guide in basic concepts such as units, definitions, trigonometry, physics, etc.

PAPER 2 SYLLABUS

This section contains the syllabus for Part 2 of the COM Certificate in Rock Engineering is provided, as prepared by the South African National Institute for Rock Engineering. The syllabus describes the levels of minimum knowledge and understanding and is not to be viewed as a complete list of rock engineering knowledge.

PREAMBLE

1. TOPICS COVERED

This is a general paper covering basic rock mechanics practice applicable in all types of mining environment. The rock engineering knowledge required here is thus of a fundamental nature, and is not specific to any particular type of mining.

2. CRITICAL OUTCOMES

The examination is aimed at testing the candidate's abilities in the six cognitive levels: knowledge, comprehension, application, analysis, synthesis

and evaluation. Thus, when being examined on the topics detailed in this syllabus candidates must demonstrate their capacity for:

- Comprehending and understanding the general rock engineering principles covered in this syllabus and applying these to solve real world mining problems
- Applying fundamental scientific knowledge, comprehension and understanding to predict the behaviour of rock materials in real world mining environments
- Performing creative procedural design and synthesis of mine layouts and support systems to control and influence rock behaviour and rock failure processes
- Using engineering methods and understanding of the uses of computer packages for the computation, modelling, simulation, and evaluation of mining layouts
- Communicating, explaining and discussing the reasoning, methodology, results and ramifications of all the above aspects in a professional manner at all levels.

3. PRIOR LEARNING

This portion of the syllabus assumes that candidates have prior learning and good understanding of :

- The field of fundamental mechanics appropriate to this part of the syllabus
- The application and manipulation of formulae appropriate to this part of the syllabus as outlined in the relevant sections of this document
- The terms, definitions and conventions appropriate to this part of the syllabus as outlined in the relevant sections of this document.

4. STUDY MATERIAL

This portion of the syllabus assumes that candidates have studied widely and have good knowledge and understanding of :

- The reference material appropriate to this part of the syllabus as outlined in the relevant sections of this document
- Other texts that are appropriate to this part of the syllabus but that may not be specifically referenced in this document
- Information appropriate to this part of the syllabus published in journals, proceedings and documents of local mining, technical and research organisations.

SYLLABUS

This section contains the syllabus for Part 2 of the COM Certificate in Rock Engineering is provided, as prepared by the South African National Institute for Rock Engineering. The syllabus describes the levels of minimum knowledge and understanding and is not to be viewed as a complete list of rock engineering knowledge.

NOTE

This syllabus is available in PDF format on the SANIRE webpage

2. GEOTECHNICAL CHARACTERISTICSSTRESS AND STRAIN COMPONENTS

2.1. GEOLOGY

2.1.1. ROCK TYPES

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Identify, describe and differentiate between the fundamental classes of rock :
  - Igneous rocks
  - Sedimentary rocks
  - Metamorphic rocks
- Identify, describe and differentiate between fundamental types of rock :
  - Igneous rock types commonly occurring in South African mining operations, eg. dolerite, kimberlite, pyroxenite, granite, etc
  - Sedimentary rock types commonly occurring in South African mining operations, eg. coal, sandstone, shale, dolomite, etc
  - Metamorphic rock types commonly occurring in South African mining operations, eg. quartzite, conglomerate, schist, gneiss, mylonite, etc
- State the origins of the rock types above
- State the strengths of the rock types above.

2.1.1. GEOLOGICAL SEQUENCES

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe the general geological sequences and principal rock types associated with :
  - The Karoo sedimentary period
  - The Witwatersrand Supergroup
  - The Ventersdorp Supergroup
  - The Transvaal Sequence
  - The Bushveld Igneous Complex
- Describe in particular the rock types associated with relevant ore horizons
- Describe and explain the geotechnical significance of the rock types associated with the relevant horizons.

2.1.1. STRUCTURAL GEOLOGY

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Construct and apply geological sections and projections from borehole information
- Construct and apply geological sections and projections from plans
- Define, describe and explain the significance of the following terms in terms of mining operations :
  - Bedding, Cleavage
  - Slips, Joints
  - Faults

- Describe and explain normal faulting and the stress environments in which they are formed
- Describe and explain reverse faulting and the stress environment in which they are formed
- Describe and explain the following geological features and terms :
  - Dykes, Sills,
  - Folding
  - Dip, Strike,
  - Striation, Slicken side, Comminution, Mylonite,
  - Erosion channels, Depositional channels, Wash-outs, Replacement zones
  - Potholes, Domes
  - Devolatisation, Calorific value, Ash content, Yield, Select height.

2.1.1. HYDRO-GEOLOGY

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Define, describe and explain the significance of the following terms in terms of mining operations :
  - Aquifer, Aquifuge, Water table
  - Porosity, Permeability
- Describe and explain the effect of pore water pressure on rock mass strength
- Describe and explain the effect of pore water pressure on the frictional strength of joints.

2. ROCK AND ROCKMASS BEHAVIOUR

2.1. DATA COLLECTION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Define and describe the standard terminology for core logging
- Describe, explain and evaluate the problems associated with characterising weathered rock
- Describe, explain and evaluate the implications and impact of the orientation of the survey line on the interpretation of recorded data
- Describe, explain and interpret data presented in the form of stereo nets
- Identify joint trends, failure mechanisms, and other features presented in the form of stereo nets
- Describe, explain and apply core logging techniques
- Describe, explain and apply scan line mapping techniques
- Describe, explain and apply stereo net analyses
- Describe, explain and apply stereo pair analyses.

2.2. ROCKMASS CLASSIFICATION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Describe and explain how rock quality designation (RQD) may be determined from borehole core
- Describe and explain how rock quality designation (RQD) may be determined from the in-situ rockmass
- Describe and explain the formulation and components of Barton's Q system of rockmass classifcation
- Describe and explain the formulation and components of Bieniawski's RMR system of rockmass classifcation
- Describe and explain the formulation and components of Laubscher's MRMR system of rockmass classifcation
- Compare and contrast these three rockmass classification systems and their respective applications
- Describe and explain what modifications are necessary to apply rockmass classification systems to local conditions
- Apply Barton's Q system to classify a rockmass
- Apply Bieniawski's RMR system to classify a rockmass
- Apply Laubscher's MRMR system to classify a rockmass
- Describe, discuss and apply rockmass classification techniques for the selection of mining methods
- Apply rockmass classification results to determine the stability of unsupported spans
- Apply rockmass classification results to determine the stability of unsupported rockslopes
- Apply rockmass classification results to determine support requirements for various situations
- Determine rockmass 'm' and 's' parameters for the Hoek and Brown criterion based upon rockmass classification results
- Determine rockmass deformability from joint stiffness and rockmass classification results.

2.3. MINE SEISMICITY: PRINCIPLES OF SEISMICITY

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Identify, describe and explain the conditions required for a rockburst to occur
- Identify, describe and explain the processes that give rise to rupture in rock
- Identify, describe and explain the rupture processes in solid rock
- Identify, describe and explain the rupture processes along a pre-existing fault plane
- Describe and explain the phenomenon of foreshocks and aftershocks
- Describe and explain the phenomenon of seismic body waves
- Describe, explain and distinguish between different types of seismic body waves
- Describe and explain the phenomenon of seismic surface waves
- Describe and explain how seismic surface waves are generated

- Describe and explain how seismic surface waves are related to seismic body waves
- Describe and explain the differences between seismic body waves and seismic surface waves
- Identify and describe the factors affecting the velocity of propagation of seismic waves
- Describe and explain how the rockmass environment affects the velocity of P-waves and S-waves
- State typical velocities of P-waves and S-waves in hard rock and soft rock environments
- Describe and explain how this information may be used in locating the sources of seismic events
- Describe and explain how the energy of a seismic body wave is attenuated as it propagates through elastic rock
- Describe and explain how the energy of a seismic body wave is attenuated as it propagates through non-elastic rock
- Describe and explain how seismic waves and the energy of seismic waves causes damage in excavations
- Describe and explain the phenomena of reflection and refraction of seismic body waves
- Describe and explain how seismic wave reflection and refraction causes damage in excavations
- Distinguish between seismic source mechanism and seismic damage mechanism
- Identify, describe and explain the common methods used to locate seismic events
- Describe the simplifying assumptions commonly used in determining seismic event location
- Explain the difference between the terms hypo-centre and epicentre used in describing the location of seismic events
- Describe, explain and contrast the following primary source parameters in terms of their physical meaning and the data required to determine them :
  - Seismic energy, Seismic moment, Richter magnitude, Local magnitude, Source dimension, Corner frequency, Energy index
- Identify, describe and explain the basic steps that are involved in carrying out a rockburst site investigation.

4. FUNDAMENTAL PRINCIPLES OF MINING LAYOUTS AND MINING LAYOUT DESIGNROCK STRENGTH

4.1 SURVEY & ECONOMICS

4.1.1 MINE PLANS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the meanings of the following terms :
  - Reef width, Internal waste, Channel width, Stope width
  - Mine call factor, Block factor
- Interpret and apply information contained in survey plans
- Construct sections from survey plans
- Interpret and apply information contained in sections drawn from survey plans.

4.1.2 MINING ECONOMICS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the reasons for attempting to mine an ore reserve at a constant grade
- Identify, describe and evaluate the implications of mining unpayable ore against leaving ore pillars in mining situations.

4.2 MINE DESIGN

4.2.1 MINING LAYOUTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain for underground mining operations :
  - How materials (support units, explosives, etc.) are transported into a section
  - What services (electricity, water, etc.) are required in a section
  - How services (electricity, water, etc.) are provided in a section
  - How broken ore is removed from a stope.

4.2.2 SHAFT LAYOUTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the layout associated with a vertical shaft, showing the method of handling the broken ore
- Describe and explain the layout associated with an inclined shaft, showing the method of handling the broken ore
- Describe and explain how vertical shafts are protected from the effects of mining induced stresses and displacements
- Describe and explain how inclined shafts are protected from the effects of mining induced stresses and displacements
- Describe and explain how the ore around a vertical shaft may be mined early to avoid the necessity of leaving a shaft pillar
- Describe and explain methods of avoiding shaft pillars
- Describe and explain the implications of the above methods on mine layouts
- Describe, explain and design shaft layouts for the extraction of particular blocks of ground.

4.2.3 MINE VENTILATION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the purpose of ventilating a mine
- List, describe and explain the main sources of heat in a mine
- List and describe the most common gasses found in a mine and their associated hazards
- Describe and explain in principle how ventilation is brought into and exhausted from a mine
- Describe and explain how the flow of ventilation is controlled on the different mining levels

- Describe and explain how the flow of ventilation is controlled in development ends
- Describe and explain how the flow of ventilation is controlled in stopes
- Describe and explain in principle how refrigeration is provided in mine workings using:
  - Cooled air, Chilled water, Ice plants.

4.3 AUDITING FOR BEST PRACTICE

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain geotechnical investigation/design/monitoring/evaluation loop processes
- Describe and explain process of geotechnical hazard and risk evaluation and assessment.

5. MINING SUPPORT AND MINING SUPPORT DESIGN

5.1 SUPPORT TYPES AND SPECIFICATIONS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the concept of active support
- Describe and explain the concept of passive support
- Describe and explain the differences between active and passive support
- Describe and explain the functions of skin support
- Describe and explain the functions of areal support
- Describe and explain the principles of rock reinforcement
- Describe, characterise and discuss the following tunnel support types :
  - Mechanically anchored bolts, Cable bolts, Friction bolts (split sets, hydro-bolts)
  - Cement grouted bolts, Resin bonded bolts
  - Full-column grouted/bonded bolts
  - Yielding bolts (cone bolts)
  - Multiple rod bolts, Multiple strand cable anchors
  - Prestressed tendons
  - Steel arches, Massive concrete linings
  - Shotcrete, Gunite, Thin sprayed linings
  - Wire mesh, Rope lacing, Tendon straps
  - Characterise the following aspects of these support types :
  - Their principles of operation
  - Their technical specifications
  - Their load-deformation characteristics
  - The methods of ensuring support unit quality
  - Their installation/application procedures
  - The methods of ensuring their installed quality

- Determine appropriate support types, support sizes and support patterns given tunnel layouts, rockmass conditions and stress regimes
- Describe, characterise and discuss specifications for the variety of support types
- Describe, characterise and discuss specifications for the variety of support sizes
- Describe, characterise and discuss specifications for the variety of support patterns
- Describe, characterise and discuss quality control requirements for support units from fabrication to installation.

5.2 SUPPORT INSTALLATION METHODS AND PROCEDURES

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- • Sketch, describe and explain the methods and procedures for installing common excavation support units
- • Determine appropriate support installation methods and procedures for given tunnel layouts, rockmass conditions and stress regimes
- • Describe, characterise and discuss specifications for the variety of support installation procedures
- • Describe, characterise and discuss quality control requirements for support installation methods and procedures.

5.3 BACKFILL SYSTEMS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the functions of backfill as local support
- Describe and explain the functions of backfill as regional support
- Sketch, describe and explain the different backfill preparation methods and preparation systems for :
  - Full plant tailings fill
  - Classified tailings fill
  - Paste fill
- Describe and explain the uses of cementitious binders in backfill
- Sketch, describe and explain in principle how backfill is transported from preparation plant to underground stope
- Sketch and describe the methods of containing backfill in stopes
- Describe and explain the methods of controlling the quality of backfill
- Sketch, describe and explain the features of backfill systems to fill particular blocks of ground.

6. INVESTIGATION AND EVALUATION

6.1 ROCKMASS CHARACTERISATION
6.1.1 FUNDAMENTALS OF ROCK PROPERTY DETERMINATION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Sketch, describe and explain how uniaxial compressive strength tests are carried out
- Sketch, describe and explain the test equipment required for uniaxial compressive strength tests
- Describe and explain the test procedure for uniaxial compressive strength tests
- Sketch, describe and explain how triaxial compressive strength tests are carried out
- Sketch, describe and explain the test equipment required for triaxial compressive strength tests
- Describe and explain the test procedure for triaxial compressive strength tests
- Describe and explain how samples may be collected for compressive strength tests
- Describe and explain how samples are prepared for compressive strength tests
- Describe and explain how load measurements are made during compressive strength tests
- Describe and explain how strain measurements are made during compressive strength tests
- Sketch, describe and explain how discontinuity shear strength tests are carried out
- Sketch, describe and explain the test equipment required for discontinuity shear strength tests
- Describe and explain the test procedure for discontinuity shear strength tests
- Describe and explain how samples may be collected for shear strength determinations
- Describe and explain how samples are prepared for shear strength determinations
- Sketch, describe and explain how Brazilian indirect tensile strength tests are carried out
- Sketch, describe and explain the test equipment required for Brazilian indirect tensile strength tests
- Describe and explain the test procedure for Brazilian indirect shear strength tests
- Describe and explain how samples may be collected for Brazilian indirect shear strength determinations
- Describe and explain how samples are prepared for Brazilian indirect shear strength determinations
- Describe the method of calculating strength for Brazilian indirect tensile strength tests
- Sketch, describe and explain how point load index tests are carried out
- Sketch, describe and explain the test equipment required for point load index tests
- Describe and explain the test procedure for point load index tests
- Describe and explain how samples may be collected for point load index determinations
- Describe and explain how samples are prepared for point load index determinations
- Describe the method of calculating strength for point load index tests

- Evaluate the shortcomings of each of the above tests as indicators of the large scale strength of rock
- Describe and explain how the following variables may affect the indicated strength of rock samples tested in uniaxial compression :
  - Specimen end-effects
  - Volume of rock tested
  - Test machine stiffness
  - Loading rate
- Sketch and describe the complete stress-strain graph for rock in uniaxial compression, indicating :
  - Hysteresis in the post peak portion of the curve
  - The associated radial strain
  - The associated volumetric strain
  - The brittle-ductile transition
  - The residual strength
- Sketch and describe the complete stress-strain graph for rock in triaxial compression, indicating :
  - Hysteresis in the post peak portion of the curve
  - The associated radial strain
  - The associated volumetric strain
  - The brittle-ductile transition
  - The residual strength.

6.2 F NDAMENTALS OF MONITORING ROCKMASS BEHAVIOUR

6.2.1 OBJECTIVES OF MONITORING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe the general objectives of a monitoring system
- Describe and explain the observational method of design in rock engineering
- Describe and explain what may be learned from monitoring
- Describe and explain how the results learned from monitoring may be used in design
- Describe the rock parameters which may be measured in a monitoring program
- Describe the rock response which may be measured in a monitoring program.

6.2.2 PRINCIPLES OF MONITORING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the general features of a monitoring system
- Describe and explain measuring instruments in terms of :
  - Accuracy
  - Error
  - Sensitivity

- Precision
- Describe and explain how one would measure :
- The stress tensor in a rock mass
- The depth of fracturing in tunnel walls
- Convergence
- Tunnel wall dilation
- Stress changes in rock
- Pressure changes in backfill
- Height of caving of hangingwall.

6.2.3 MONITORING INSTRUMENTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe the following instruments and explain their purpose, installation and operation :
  - Hydraulic pressure cells
  - Tapes and tape extensometers
  - Doorstopper stress cell
  - Triaxial stress cell
  - Closure meter
  - Peizometer
  - Inclinometer
  - Multiple Point Borehole Extensometers
  - Hydraulic Pressure Cells
  - Vibrating Wire extensometers and stressmeters
  - Petroscopes
  - Brunton Compass
- Interpret results from the above instruments
- Interpret rockmass response indicated by results from the above instruments
- Describe and explain the principles of operation of strain gauges.

6.3 FUNDAMENTALS OF SEISMIC MONITORING SYSTEMS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Identify, describe and explain the objectives of seismic monitoring in mines
- Identify, describe and explain the features of a seismic event that are used to locate the source of the event
- Describe and explain the differences between geophones and accelerometers
- Identify, describe and discuss typical applications of geophones
- Identify, describe and discuss typical applications of accelerometers
- Describe, explain and discuss the functions and physical attributes of the following components of a seismic system :
  - Seismometer, Amplifier

- Analogue to Digital Converter
- Trigger Unit, Storage Unit
- Data transmission to surface
- Data processing, Data base

- Describe and explain the factors that need to be considered in the placement of seismometers
- Describe and explain how placement could affect event location accuracy
- Describe and explain how placement could affect the sensitivity of a seismic system
- Describe and explain possible reasons for differences between ideal seismic network layouts and actual installed seismic network layouts
- Describe and explain the physical limitations of seismic network performance in terms of the accuracy of source parameters
- Identify, describe and explain the various factors that may influence the quality of seismic data
- Describe the factors affecting hardware performance
- Describe the factors affecting the quality of seismic data processing
- Describe the factors affecting data storage and data analysis
- Describe the controls that may be instituted to minimise the above effects
- Estimate approximate installation and operating costs associated with various size seismic networks.

6.4 FUNDAMENTALS OF NUMERICAL MODELLING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the principles of the following numerical modelling techniques :
  - Finite Element Methods
  - Finite Difference Methods
  - Boundary Element Methods
  - Distinct Element Methods
  - Keyblock Methods
- Describe and explain for which rock engineering applications the above numerical modelling techniques are suitable
- Describe and explain what the limitations of the above numerical modelling techniques are in terms of the following aspects :
  - When that method should be used
  - Why that particular method should be used
  - How the results should be interpreted
  - How the results should be correlated with actual conditions.

7. DRILLING AND BLASTING

7.1 EXPLOSIVES

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Describe and explain the main types of explosives used in mining
- Describe the ingredients of the main types of explosives used in mining
- Describe and explain the types of initiating explosives used in mining
- Describe and explain the mechanism of rock breaking by blasting
- Describe and explain the effect of shock energy on rock fragmentation
- Describe and explain the effect of gas pressure on rock fragmentation
- Describe and explain the function of stemming
- Apply knowledge of explosives and their characteristics to select appropriate explosives for a given task.

7.2 DRILLING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the mechanism of rock breakage by chisel bit
- Describe and explain the mechanism of rock breakage by button bit
- Describe the types of equipment used by the following drilling methods :
- Describe, explain and discuss the use of the following drilling methods :
  - Percussion drilling
  - Rotary drilling
  - Diamond drilling
  - Raise / Tunnel boring
- Describe and explain how cuttings are removed from the hole for each of the above drilling methods.

7.3 BLASTING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe, explain and discuss the effect of the following parameters on blast damage :
  - Explosive type
  - Initiation method
  - Initiation sequence
  - Hole orientation
- Describe, explain and discuss the following cushion blasting techniques :
  - Smooth blasting
  - Post-splitting
  - Pre-splitting
- Describe typical applications of each
- Describe, explain and discuss the objectives of de-coupling explosives

- Describe, explain and discuss the effects of de-coupling explosives
- Describe, explain and discuss the methods by which de-coupling of explosives is achieved.

8. LEGAL COMPLIANCE AND GENERAL MANAGEMENT

8.1 MINE HEALTH AND SAFETY ACT AND REGULATIONS

8.1.1 THE MINE HEALTH AND SAFETY ACT, ACT 29 of 1996

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to describe and explain the requirements of the Act in terms of :

- Owner's responsibilities and duties
- Manager's responsibilities and duties
- Employee's rights and duties
- Suspension of working due to inadequate safety
- Manufacturer's and Supplier's duties
- Manager's duty to consult a Rock Engineer
- Rock Engineer's responsibilities and duties
- Rock-related codes of practice requirements
- Hazard identification and risk assessment
- Negotiation and consultation forum's function, duties and powers
- Tripartite institution's functions and duties
- Inspector' functions and powers
- Requirements for enquiries into accidents
- Minister's abilities and powers
- Scope of safety-related regulations.

8.1.2 REGULATIONS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to describe and explain the requirements of regulations formulated in terms of the Act under the following headings :

- Appointments and Administration
- Duties and Responsibilities
- Health and Safety Representatives and Committees
- Inspectorate of Mine Health and Safety
- Miscellaneous and General Provisions
- Outlets, Ladderways and Travellingways
- Protection of the Surface and in the Workings
- Qualifications and Certificates of Competency
- Surveying and Mine Plans
- Tripartite Institutions
- Definitions.

8.2 CODES OF PRACTICE

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to describe and explain the

fundamental requirements of codes of practice to combat rock-related accidents as contemplated in terms of the Act and the DME Guideline for the Compilation of a Mandatory Code of Practice for :

- Tabular Metalliferous Mines
- Underground Coal Mines
- Massive Mining Operations
- Surface Mines.

8.3 BASIC RISK ASSESSMENT

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe and explain the process and procedure of geotechnical hazard and risk evaluation and assessment.

8.4 GENERAL MANAGEMENT
8.4.1 REPORTING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe the requirements and format of a formal technical report
- Compile clear technical reports to senior mine officials on subjects covered in this syllabus
- Compile clear technical reports to head office personnel on subjects covered in this syllabus.

8.4.2 MINE ACCIDENTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- State, describe and explain the requirements of investigations into rockfall and rockburst accidents
- State, describe and explain the main causes of rock-related accidents in various mining operations in various mining districts.

8.4.3 ROCK ENGINEERING FUNCTIONS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to :

- Describe, explain and discuss the functions of the rock engineering discipline on various types of mines
- Describe, explain and discuss the functions of various staff categories encountered in a rock mechanics department.

REFERENCES

2. GEOTECHNICAL CHARACTERISTICS
2.1. GEOLOGY
2.1.1. ROCK TYPES

- Lurie J 1987 South African Geology for Mining, Metallurgical, Hydrological and Civil Engineering Lexicon Publishers Jhb Chapters 3, 5, 7

2.1.1. GEOLOGICAL SEQUENCES

- Lurie J 1987 South African Geology for Mining, Metallurgical, Hydrological and Civil Engineering Lexicon Publishers Jhb Chapter 9

2.1.1. STRUCTURAL GEOLOGY

- Lurie J 1987 South African Geology for Mining, Metallurgical, Hydrological and Civil Engineering Lexicon Publishers Jhb Chapters 6, 23, 24, 25

2.1.1. HYDRO-GEOLOGY

- Lurie J 1987 South African Geology for Mining, Metallurgical, Hydrological and Civil Engineering Lexicon Publishers Jhb Chapter 20
- Hoek E & Bray JW 1977 Rock Slope Engineering IMM London Chapter 6
- Fauconnier CJ & Kersten RWO (ed) 1982 Increased Underground Extraction of Coal SAIMM Jhb Chapter 3

3. ROCK AND ROCKMASS BEHAVIOUR
3.1. DATA COLLECTION

- Stacey TR 2001 Best Practice Rock Engineering Handbook for 'Other' Mines SIMRAC Jhb Chapter 2
- Hoek E & Brown ET 1980 Underground Excavations in Rock IMM London Chapters 3, 4
- Hoek E & Bray JW 1977 Rock Slope Engineering IMM London Chapters 3, 4

3.2. ROCKMASS CLASSIFICATION

- Stacey TR 2001 Best Practice Rock Engineering Handbook for 'Other' Mines SIMRAC Jhb Chapter 2
- Hoek E & Brown ET 1980 Underground Excavations in Rock IMM London Chapter 2

3.3. MINE SEISMICITY: PRINCIPLES OF SEISMICITY

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 5
- Mendecki AJ (ed) 1997 Seismic Monitoring in Mines Chapman & Hall London Chapters 1, 2, 5, 7, 8

4. FUNDAMENTAL PRINCIPLES OF MINING LAYOUTS AND MINING LAYOUT DESIGN
4.1. SURVEY AND ECONOMICS

- Local knowledge

4.1.1. MINE PLANS

- Local knowledge

4.1.1. MINING ECONOMICS

4.2. MINE DESIGN

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 12, 13, 14, 15, 16
- Atlas Copco Website http://www.atlascop-co.com/Websites%5CRDE%
- 5Cwebsite.nsf/$All/751129377B17207341 2567440058CC3F?OpenDocument

4.2.1. MINING LAYOUTS

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 12, 13, 14, 15, 16
- Atlas Copco Website http://www.atlascop-co.com/Websites%5CRDE%
- 5Cwebsite.nsf/$All/751129377B172073412567440058CC3F?OpenDocument

4.2.1. SHAFT LAYOUTS
4.2.1. MINE VENTILATION

4.3. AUDITING FOR BEST PRACTICE

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 10

5. MINING SUPPORT AND MINING SUPPORT DESIGN
5.1. SUPPORT TYPES AND SPECIFICATIONS

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 11
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Stacey TR 2001 Best Practice Rock Engineering Handbook for 'Other' Mines SIMRAC Jhb Chapter 4

5.2. SUPPORT INSTALLATION METHODS AND PROCEDURES

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 11
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7

5.3. BACKFILL SYSTEMS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 14
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 11
- Stacey TR 2001 Best Practice Rock Engineering Handbook for 'Other' Mines SIMRAC Jhb Chapter 4

6. INVESTIGATION AND EVALUATION

6.1. ROCKMASS CHARACTERISATION

6.1.1. FUNDAMENTALS OF ROCK PROPERTY DETERMINATION

- Hoek E & Brown ET 1980 Underground Excavations in Rock IMM London Chapters 2, 3, 4
- Stacey TR 2001 Best Practice Rock Engineering Handbook for 'Other' Mines SIMRAC Jhb Chapter 2
-

6.2. FUNDAMENTALS OF MONITORING ROCKMASS BEHAVIOUR

6.2.1. OBJECTIVES OF MONITORING

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 9
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 10
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 18

6.2.1. PRINCIPLES OF MONITORING

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 9
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 10
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 18

6.2.1. MONITORING INSTRUMENTS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 9
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 10
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 18
- Obert L & Duvall WI 1967 Rock Mechanics and the Design of Structures in Rock John Wiley & Sons New York Chapter 9

6.3. FUNDAMENTALS OF SEISMIC MONITORING SYSTEMS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 5
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 9

6.4. FUNDAMENTALS OF NUMERICAL MODELLING

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 8
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 11
- Lightfoot N & Maccelari MJ 1998 Numerical Modelling of Mine Workings SIMRAC Jhb Chapters 1-11

7. DRILLING AND BLASTING

7.1. EXPLOSIVES

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 17

7.2. DRILLING

- General knowledge of standards, practices and procedures concerning drilling on mines

7.3. BLASTING

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 17

8. LEGAL COMPLIANCE AND GENERAL MANAGEMENT

8.1. MINES HEALTH AND SAFETY ACT AND REGULATIONS

8.1.1. THE MINES HEALTH AND SAFETY ACT

- Act of Parliament 1996 Mine Health and Safety Act - Act 29 of 1996 Kivell Law Publishing Homestead

8.1.1. REGULATIONS

- Ministerial Act Latest Mine Health and Safety Act - Regulations Kivell Law Publishing Homestead

8.2. CODES OF PRACTICE

- Chief Inspector of Mines Latest Guideline for the Compilation of a Mandatory Code of Practice to Combat Rock Fall and Rock Burst Accidents in Tabular Metalliferous Mines - Ref.No. DME 16/3/2/1-A3 DME
- Chief Inspector of Mines Latest Guideline for the Compilation of a Mandatory Code of Practice to Combat Roof Fall Accidents in Underground Coal Mines - Ref.No. DME 16/3/2/1-A4 DME
- Chief Inspector of Mines Latest Guideline for the Compilation of a Mandatory Code of Practice to Combat Rock Fall Accidents in Massive Mining Operations - Ref.No. DME 16/3/2/1-A5 DME
- Chief Inspector of Mines Latest Guideline for the Compilation of a Mandatory Code of Practice to Combat Rock Fall and Slope Instability Related Accidents in Surface Mines - Ref.No. DME 7/4/118-AB4 DME

8.3. BASIC RISK ASSESSMENT

8.4. GENERAL MANAGEMENT

8.4.1. REPORTING

- General knowledge of report writing practice

8.4.1. MINE ACCIDENTS

- Act of Parliament 1996 Mine Health and Safety Act - Act 29 of 1996 Kivell Law Publishing Homestead

8.4.1. ROCK ENGINEERING FUNCTIONS (ROCK ENGINEERING AS PART OF A MINING OPERATION)

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 12

CHAPTER

2

GEOTECHNICAL CHARACTERISTICS STRESS AND STRAIN COMPONENTS

INTRODUCTION

Chapter 2 deals with Geology, which forms the cornerstone of any rock engineering design as it describes the material that is being engineered. Knowledge and understanding of geology issues, such as those described in this chapter,arecritical to ensure the design takes cognisance of all the material's properties and that the final design is appropriate to the environment, optimal and safe.
In this chapter, the following is addressed:

- Typical rock types found in South Africa
- Typical geological sequences in South Africa
- Structural geology
- Hydrogeology.

LEARNING OUTCOMES

2. GEOLOGY
2.1. ROCK TYPES
From the syllabus provided, rock engineers must be able to demonstrate knowledge and understanding of:

2.1.1 Identify, describe and differentiate between the fundamental classes of rock :

- Igneous rocks
- Sedimentary rocks
- Metamorphic rocks

2.1.2 Identify, describe and differentiate between fundamental types of rock :

- Igneous rock types commonly occurring in South African mining operations, e.g. dolerite, kimberlite, pyroxenite, granite, etc
- Sedimentary rock types commonly occurring in South African mining operations, e.g. coal, sandstone, shale, dolomite, etc
- Metamorphic rock types commonly occurring in South African mining operations, e.g. quartzite, conglomerate, schist, gneiss, mylonite, etc

2.1.3 State the origins of the rock types above
2.1.4 State the strengths of the rock types above.

2.2. GEOLOGICAL SEQUENCES

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

2.2.1 Describe the general geological sequences and principal rock types associated with:

- The Karoo sedimentary period
- The Witwatersrand Supergroup
- The Ventersdorp Supergroup
- The Transvaal Sequence
- The Bushveld Igneous Complex

2.2.2 Describe in particular the rock types associated with relevant ore horizons

2.2.3 Describe and explain the geotechnical significance of the rock types associated with the relevant horizons.

2.3. STRUCTURAL GEOLOGY

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

2.3.1 Construct and apply geological sections and projections from borehole information

2.3.2 Construct and apply geological sections and projections from plans

2.3.3 Define, describe and explain the significance of the following terms in terms of mining operations:

- Bedding, Cleavage
- Slips, Joints
- Faults

2.3.4 Describe and explain normal faulting and the stress environments in which they are formed

2.3.5 Describe and explain reverse faulting and the stress environment in which they are formed

2.3.6 Describe and explain the following geological features and terms:

- Dykes, Sills,
- Folding
- Dip, Strike,
- Striation, Slicken side, Comminution, Mylonite,
- Erosion channels, Depositional channels, Wash-outs, Replacement zones
- Potholes, Domes
- Devolatisation, Calorific value, Ash content, Yield, Select height.

2.4. HYDRO-GEOLOGY

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

2.4.1 Define, describe and explain the significance of the following terms in terms of mining operations:

- Aquifer, Aquifuge, Water table
- Porosity, Permeability

2.4.2 Describe and explain the effect of pore water pressure on rock mass strength

2.4.3 Describe and explain the effect of pore water pressure on the frictional strength of joints.

LEARNING OUTCOME 2.1.1

IDENTIFY, DESCRIBE AND DIFFERENTIATE BETWEEN THE FUNDAMENTAL CLASSES OF ROCK:

- Igneous rocks
- Sedimentary rocks
- Metamorphic rocks

Igneous rocks

Igneous rocks are typically made up of crystallised hot melt (magma or lava) and make up four-fifths of the earth crust. The hot melt is made up of different chemical elements and different types of minerals; therefore, when it crystallises it results in different types of igneous rock. Holt melt may crystallise below the earth surface, forming intrusive igneous rocks, e.g. granite and pyroxenite. Some may crystallise above the earth surface, forming extrusive igneous rocks, e.g. basalt, obsidian etc. Intrusive are differentiated from extrusive by grain size, where the latter is normally characterised by very fine grain size compared to the intrusive, and is a function of the rate of cooling. Fast cooling rates result in fine grained materials and slow rates in coarse grained materials. Classification of igneous materials can be done based on:

- Depth of formation;
- Colour index;
- Chemical content;
- Mineral content.

A simple classification is shown in Table 1 below.

	SiO2 (%)	Extrusive	Intrusive		Essential minerals	Depth (km)
			Hypabyssal	Abyssal		
Acid	63	Rhyolite Obsidian	Granophyre	Granite	Quartz & Alkali Feldspars	2.6
Intermediate	52	Trachyte	Syenite-porphyry	Syenite	Alkali Feldspars	
		Andesite	Diorite-porphyry	Diorite	Horn-blende & Plagioclase	
Basic	45	Basalt	Dolerite	Gabbro	Augite & Plagioclase	
Ultra basic		Kimberlite	Kimberlite	Peridotite Dunite	Olivine	3.3
Grain size →						

Table 1: Simple igneous rock classification (Lurie, 1987)

NOTE

For a detailed description of the classification in Table 1, refer to Page 18, Lurie, J. 1987. Geology.

A tabular, approximately parallel-sided intrusive is normally referred to as a sheet, where a dyke is a sheet that cuts across bedding and a sill a sheet that intrudes between beddings.

Metamorphic rocks

Metamorphic rocks are formed when the existing rock undergoes transformation either physically or chemically under the influence of

back

temperature, pressure or active fluids (mainly water). Since all rocks undergo some changes over time (e.g. Compaction), it becomes difficult to separate metamorphic and non-metamorphic rocks. In its simplest definition, metamorphic rocks are those rocks that that show characteristics distinct from sedimentary and igneous rock characteristics.
The process of metamorphism changes the character of the pre-existing rocks. Examples of metamorphic rocks include:

- quartzite (recrystallised sandstone),
- marble (recrystallised limestone),
- slate (low grade dynamic metamorphosed argillaceous rocks),
- gneiss (regionally metamorphosed granite), etc.

Sedimentary rocks
Sedimentary rocks are formed by the breakaway and weathering of rocks, its transportation and deposition in built-up layers. Settling of the particles can be a chemical, physical or a biological process. Sedimentary material is grouped by the method of deposition:

- Material not transported but settled out where it was formed is called residual deposits;
- wind deposited material Aeolian;
- water deposited material fluviatile (rivers) and lacustrine (lakes);
- glacial deposits are formed by continental and marine conditions; and
- deltaic deposits where rivers deposit material into lakes or oceans.

Sedimentary rocks are classified according to the locality of the deposition, climatic conditions during deposition or the dominant formative agent with the simplest classification separating them into the following:

- mechanically formed (e.g. boulders, pebbles),
- organically formed (e.g. Chalk, coal) ,
- chemically formed (e.g. limestone, banded ironstone, calcrete) and
- residual (e.g. Soils)

Sedimentary rocks are further categorised in Table 2.

Essential composition	Mechanically formed	Organically formed	Chemically formed	Residual
Siliceous	Sandstone Quartzite	Radiolarian chert	Siliceous sinter Silcrete Chert Flint	Sandy soil
Calcareous	Shelly limestone	Shelly limestone Coral limestone	Limestone Dolomite Calcrete Travertine Tufa	
Ferruginous		Bog iron ore	Banded ironstone Ferricrete	
Aluminous	Mudstone Shale			Laterite Black turf
Phosphatic		Bone bed Guano		
Sulphates and chlorides			Gypsum beds Rock salt	
Heterogeneous	Conglomerate Breccia			

back

	Arkose Tillite Agglomerate Tuff	

Table 2: Classification of sedimentary rock (Lurie, 1987)

- Pages 14, 25, 35, Lurie, J. 1987. Geology.

REFERENCE

LEARNING OUTCOME 2.1.2

IDENTIFY, DESCRIBE AND DIFFERENTIATE BETWEEN FUNDAMENTAL TYPES OF ROCK:

- Igneous rock types commonly occurring in South African mining operations, e.g. dolerite, kimberlite, pyroxenite, granite, etc
- Sedimentary rock types commonly occurring in South African mining operations, e.g. coal, sandstone, shale, dolomite, etc
- Metamorphic rock types commonly occurring in South African mining operations, e.g. quartzite, conglomerate, schist, gneiss, mylonite, etc

CONNECTION 1

Refer to "LEARNING OUTCOME 2.1.1" on page 24

Pyroxenite (Figure 1) is an intrusive igneous rock normally found in the Bushveld Complex and is composed out of pyroxene and plagioclase. It normally has a low RQD, indicating that significant jointing could be expected. Grain sizes are large, indicating slow cooling rates.

Figure 1: Pyroxenite

Anorthosite is an intrusive igneous rock normally found in the Bushveld Complex. Spotted Anorthosite: White feldspar with dark pyroxene speckles; Mottled Anorthosite: Grey feldspar with dark pyroxene mineral blotches.

Figure 2: Anorthosite

Norite is an intrusive igneous rock normally found in the Bushveld Complex that contains calcium rich feldspar minerals.

Figure 3: Norite

Dolerite is a fine-grained intrusive igneous rock composed of pyroxene, plagioclase feldspar and miner olivine. Very often (but not always) dykes or sills encountered in the South African mining environment consist of dolerite (Figure 4).

Figure 4: Dolerite

Syenite intrusion (Figure 5) is a very strong igneous rock similar to granite, but with low or absent quartz content. Fractures are very rough and have large apertures. The rock mass probably has a 'sugar cube' type appearance and may have a high degree of mechanical interlocking.

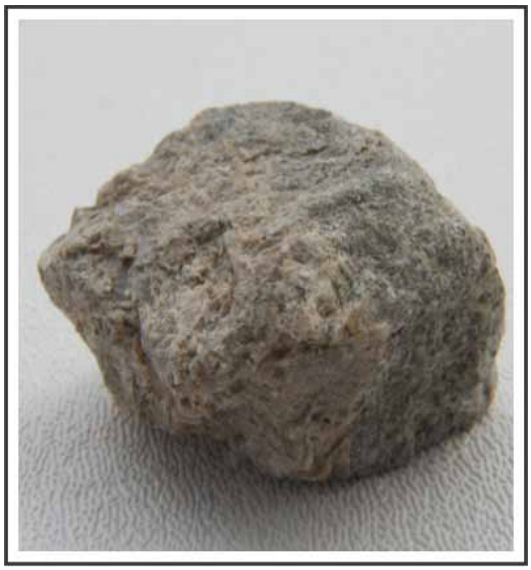


Figure 5: Syenite

Kimberlite is a volcanic rock that is rich in olivine and other magnesium and iron-bearing minerals and may contain micas (Figure 6).

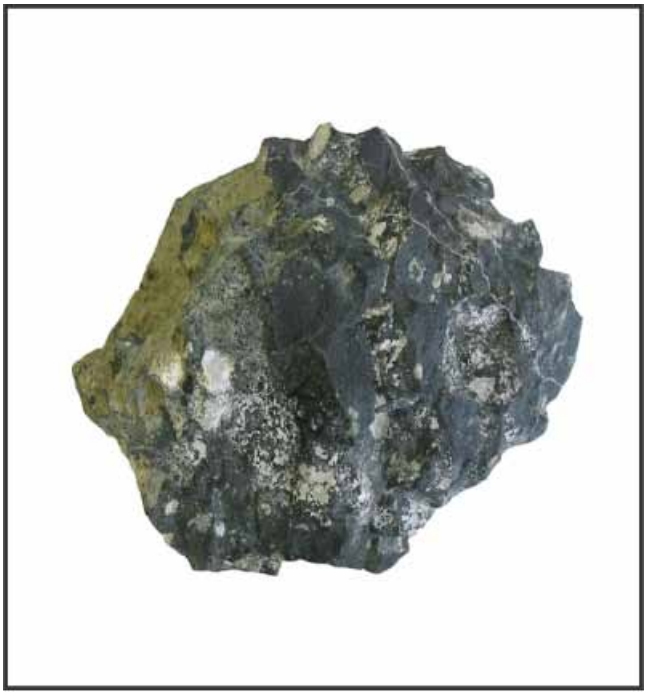


Figure 6: Kimberlite

Granite is a coarse-grained intrusive igneous rock, rich in quartz and feldspar and other minerals include mica and hornblende. It is normally a structurally homogeneous rock (Figure 7).

Figure 7: Granite

Shale is a sedimentary rock and tends to form distinct bedding and laminations that are broken into blocks by joints (Figure 8).

Figure 8: Shale

back

Sandstone: A clastic sedimentary rock; strong, often massive, but also with extensive bedding. Most sandstones are composed of quartz and feldspar (Figure 9).

Figure 9: Sandstone

Mudstone is a sedimentary rock that is very weak, but with no evidence of extensive bedding planes (i.e. massive). Mudstone contains clay and mud and softens when in contact with water (Figure 10).

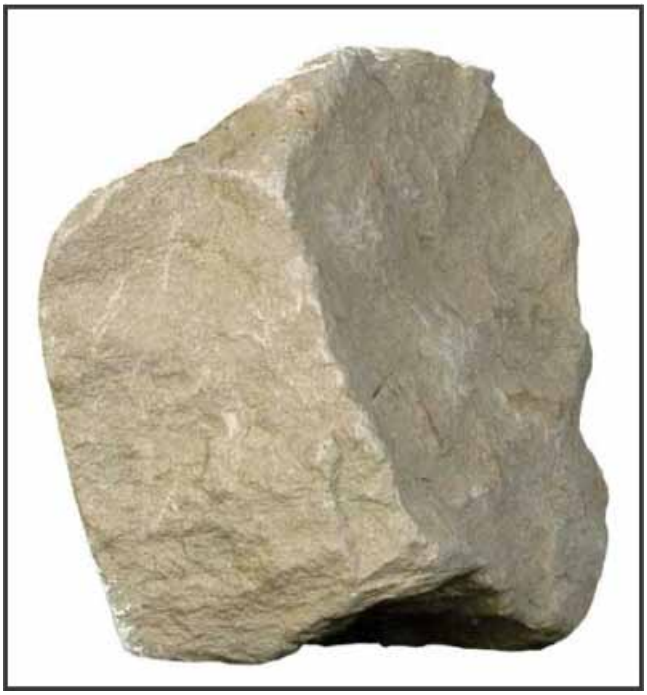


Figure 10: Mudstone

back

back

Coal is a combustible carbon-rich sedimentary rock that is organically formed. It is black in colour and it can also be termed a metamorphic rock, depending on the level of pressure and temperature conditions that it was derived from (Figure 11).

Figure 11: Coal

Siltstone is a clastic sedimentary rock that is made of sediment between sand and clay, is finer grained than sandstone, but coarser than shale and forms from silt-size particles (between 1/256 and 1/16 millimetre diameter).

Figure 12: Siltstone

back

back

Dolomite is a sedimentary rock made up of a calcium-magnesium carbonate ($CaMg(CO3)2$) and is highly water-soluble. This leads to the formation of underground caves and sinkholes (Figure 13).

Figure 13: Dolomite

A conglomerate is a sedimentary rock that consists of pebbles cemented together by finer material. Pebbles are round, indicating that they have been transported for long distances (Figure 14).

Figure 14: Conglomerate

back

Quartzite is a metamorphic rock derived from sandstone. It contains minerals such as quartz, feldspar and some mica. Quarts weather very slowly, hence forming hills and mountains (Figure 15).

Figure 15: Quartzite

Schist is metamorphic rock that is formed from regional metamorphism. It is characterised by finer foliated material (Figure 16).

Figure 16: Schist

Gneiss is formed by regional metamorphism of rock with granitic texture (Figure 17).

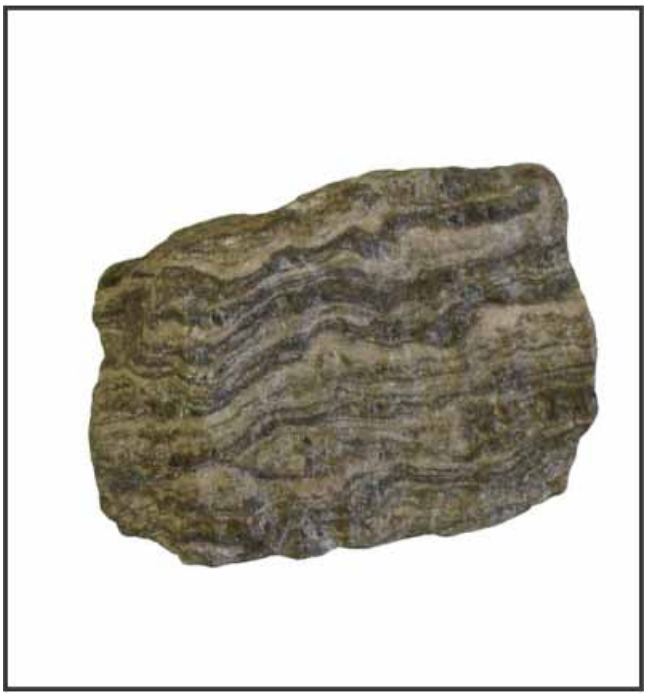


Figure 17: Gneiss

Mylonite is a fine-grained metamorphic rock produced by dynamic re-crystallisation of constituent minerals resulting in a reduction of the grain size of the rock. They are formed by an accumulation of large shear strain in ductile fault zones (Figure 18).

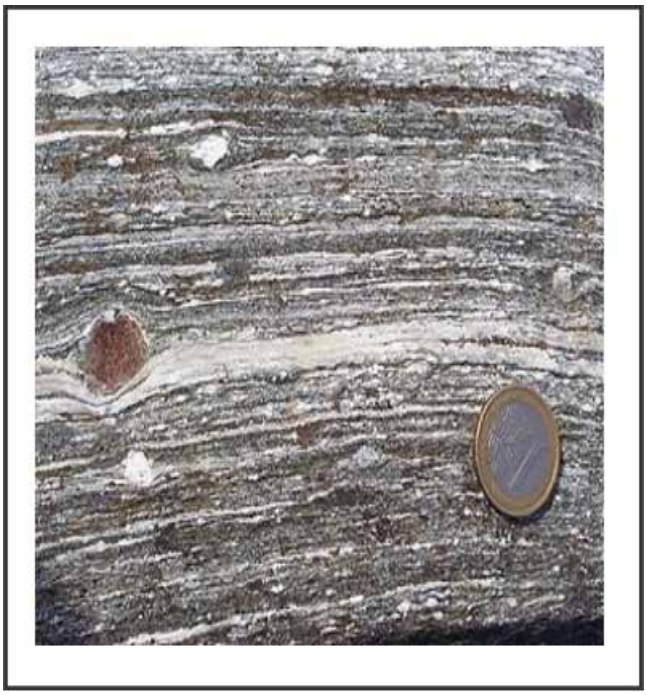


Figure 18: Mylonite

back

CONNECTION 2

Refer to "LEARNING OUTCOME 2.1.1" on page 24 and "LEARNING OUTCOME 2.1.2" on page 26

LEARNING OUTCOME 2.1.3

STATE THE ORIGINS OF THE ROCK TYPES ABOVE

LEARNING OUTCOME 2.1.4

STATE THE STRENGTHS OF THE ROCK TYPES ABOVE

CONNECTION 3

Refer to "Learning Outcome "4.1.1" on page 35 in Paper 1 and "LEARNING OUTCOME 6.1.1.1" on page 391 in Paper 2

The strength of a rock material is usually measured by determining the uni-axial compressive strength (UCS). UCS tests are conducted on core samples (Outcomes 4.1 in paper 1 and Outcome 6.1.1 in Paper 2) and are reported in Pascal units. In most cases, strengths fall within the MPa range, but rock types do exist that have UCS values in the KPa range.

NOTE

Other methods such as impact splitting (Outcome 6.1.1), point load index tests (Outcome 6.1.1) and tri-axial (Outcome 6.1.1) and other methods are also used.

From literature, typical rock strength values for the most common rock types were sourced. However, it is critical to understand that most rock types are not homogeneous or isotropic and that strengths quoted in this outcome may not be similar to strengths for the same materials in vastly different mining areas.

BRIDGING GAP 1

"Glossary" on page 3

Refer to definitions in bridging gap file for homogeneous and isotropic.

Ensure that tests on local rock samples have been conducted or apply these values given with caution. Many other sources exist that suggest appropriate values and could also be used to augment the values given below.

Material	Source*	No of specimens	Uni-axial compressive strength & MPa
Granite	A	17	159-296
Limestone	A	46	34-262
Marl	A	15	69-193
Sandstone	A	48	34-234
Shale	A	18	76-234
Diorite	B	-	176-294
Basalt	B	-	147-294
Dolomite	B	-	79-245
Coal	B	-	5-49
Halite	C	13	19-39
Potash	C	21	25-39
Anhydrite	C	14	25-106
Quartzite (Glassy)	-	25	165-190
Lava (Ventersdorp)	-	11	123-295
Dolerite	-	8	160-190
Sources A. Obert and Duvall (1967), B. Farmer (1968), C. Patchet (1970)			

Table 3: Typical Uni-axial Compressive Strengths (UCS) for various rock materials

back

REFERENCE

- Chapter 2, Ryder J.A. and Jager A.J., A textbook on rock mechanics for tabular hard rock mines, SIMRAC, 2002
- Page 16, Jager A.J. and Ryder J.A., A handbook on rock engineering practice for tabular hard rock mines, SIMRAC, 1999.

LEARNING OUTCOME 2.2.1

DESCRIBE THE GENERAL GEOLOGICAL SEQUENCES AND PRINCIPAL ROCK TYPES ASSOCIATED WITH:

- The Karoo sedimentary period
- The Witwatersrand Supergroup
- The Ventersdorp Supergroup
- The Transvaal Sequence
- The Bushveld Complex

The geological sequences mentioned above describe the main rock environments within which the South African Mining Industry operates. This outcome will discuss the sequences, but will also provide some variations within these sequences, while it also includes short discussions on the sequences, geological environments or general information on less known commodities.

In South Africa, compared to Great Britain, the stratigraphy is unique due to the presence of easily identifiable rock units that have allowed geologists to group them into systems, series, etc. Stratigraphic classification can be made by following one of three methods:

- Lithostratigraphic – character of the rock
  - Group – two or more related formations
  - Formation – the fundamental unit
  - Member – entity within a formation
  - Bed – distinctive stratum within a member of formation
- Biostratigraphic – fossil content
- Chronostratigraphic – age.

Mining in South Africa has been widespread and has taken place for many years. This has allowed for the accurate construction of the stratigraphy as it is found in several of the mining areas. Figure 17 shows the location of the main geological groups across South Africa.

NOTE

Maps can be downloaded from the Council for Geoscience (CGS) website (www.geoscience.org.za) and is only provided in this material for discussion purposes. If more accurate information is sought, the website should be consulted.

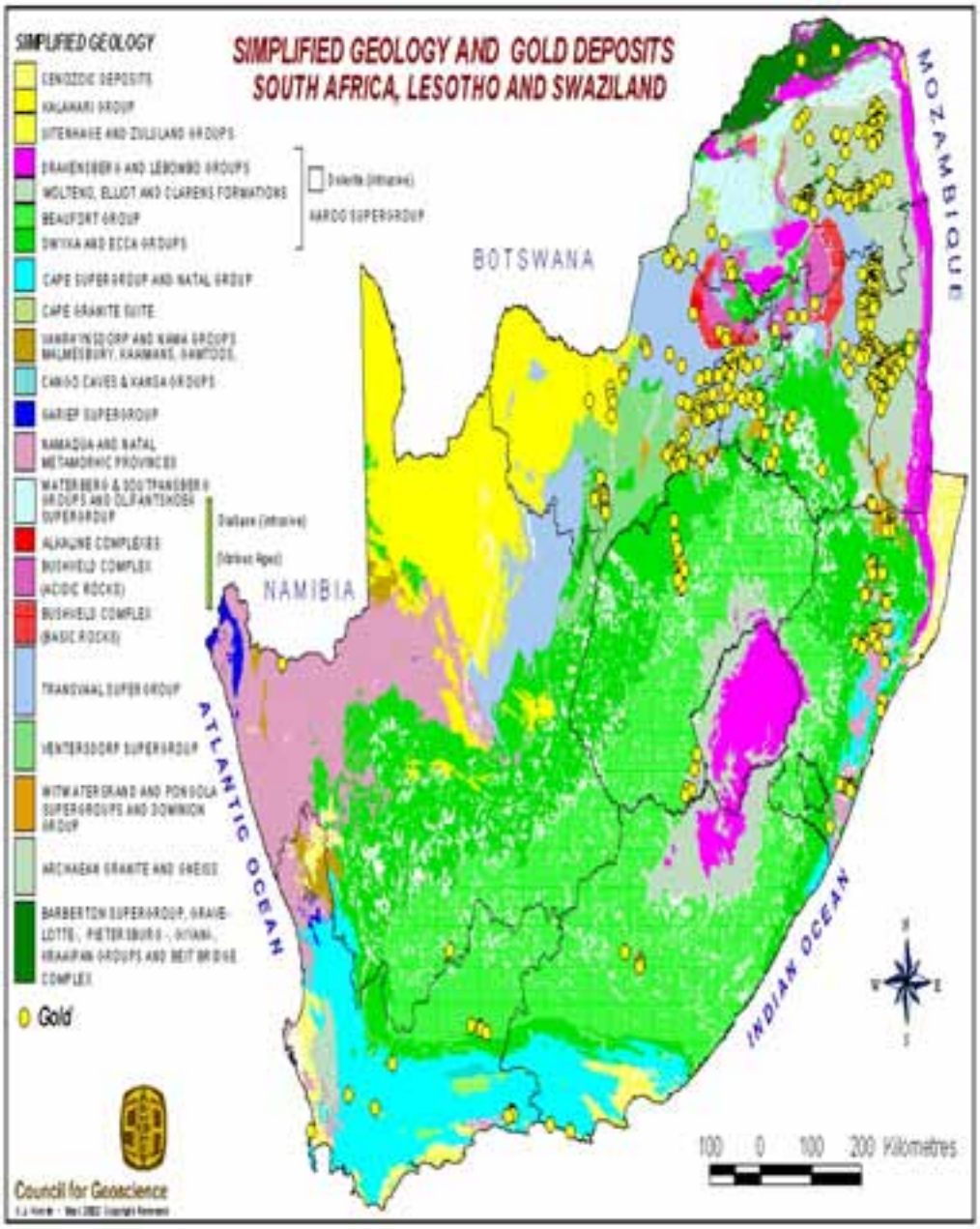


Figure 19: Simplified geology and gold deposits (Council for Geoscience)

Karoo Sedimentary Sequence

This sequence underlies approximately 50% of the total area of South Africa. In the southern parts of the country, the full sequence is present (approximately 8000m thick), but it thins out towards the north (Figure 21) overlapping pre-Karoo material.

back

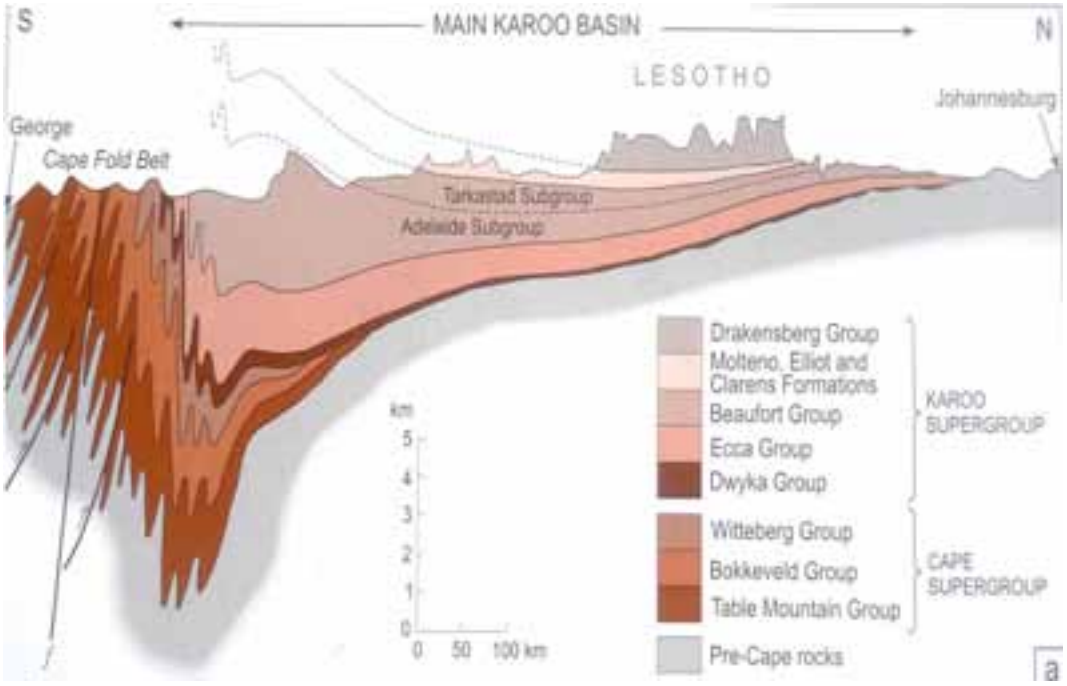


Figure 20: Main Karoo Basin sequence (Johnson et al, 2006)

The lithology is described below and is also shown in Table 5.
The Dwyka formation forms the base of the sequence and is present across the whole country. The Dwyka Tillite (glacial) has an argillaceous matrix and angular fragments.

The Ecca group is also present across the whole country and is divided into Southern, Western, Central and Northern facies. The Southern Ecca is limited to the southern Cape and consists of shales and sandstones. The Western and Central Eccas consist mainly of shales, while the Northern Ecca is composed of shale, sandstone and coal seams.

The Beaufort Group is made up of mudstones and interbedded sandstones and is subdivided into the Upper Tarkastad and Lower Adelaide Groups.

The Molteno formation consists of coarse sandstone and shales, the Elliot Formation of 'red' argillaceous succession, the Clarens Formation of sandstone (Aeolian) and the Drakensberg Formation of lavas from numerous flows.

Group	Subgroup	Formation
Lebombo		Drakensberg Jozzini
	Clarens, Elliot, Molteno	
Beaufort	Tarkastad	Burgersdorp, Katberg
	Adelaide	Balfour, Middleton
Ecca		Collingham, Volksrust Shale, Whitehill Shale, Prince Albert shale, Vryheid, Pietermaritzburg Shale.
		Dwyka

Table 4: Subdivision of the Karoo sequence (Lurie, 1987)

back

back

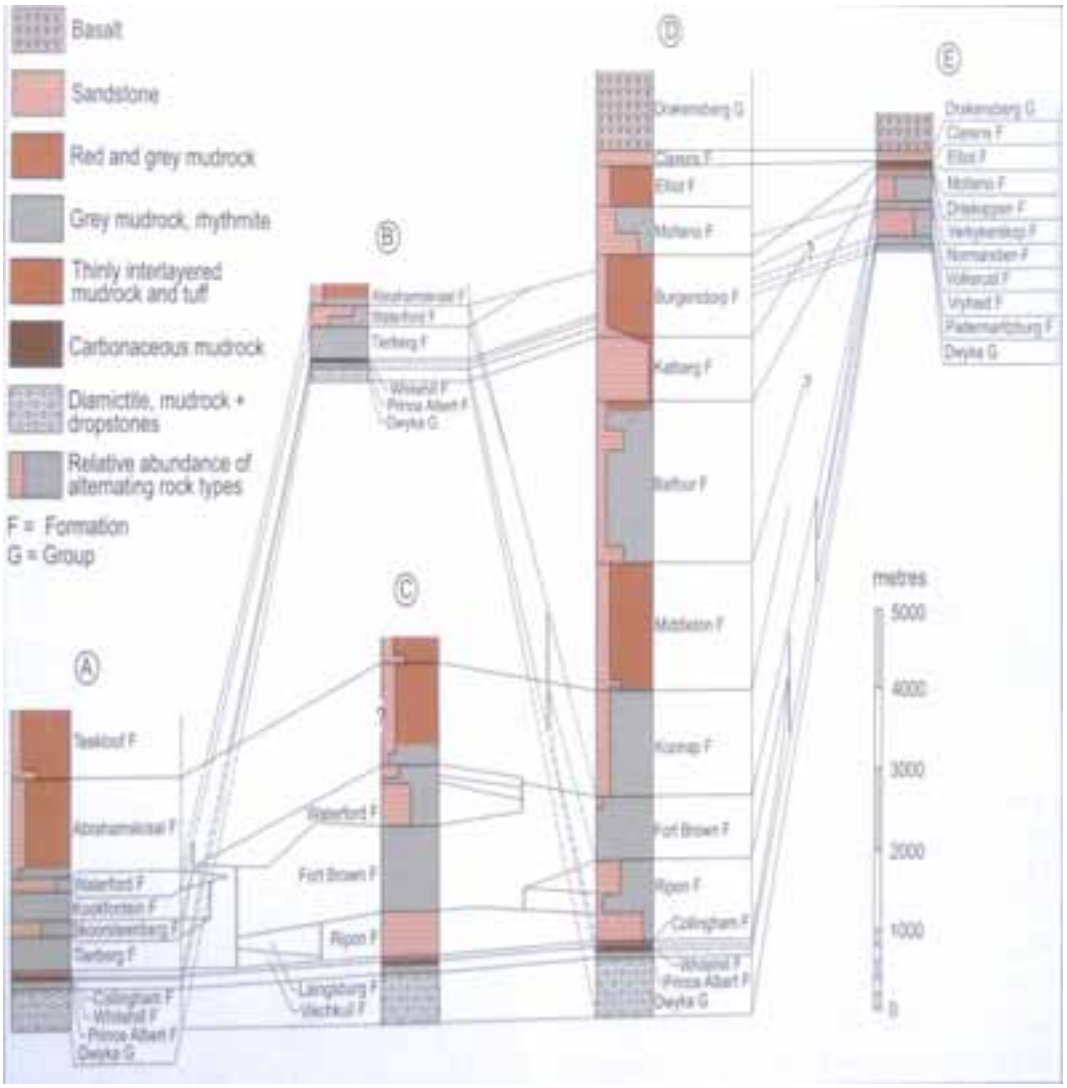


Figure 21: Generalised stratigraphy of the Karoo Supergroup in the Main Karoo Basin (Johnson et al., 2006)

The Karoo Sequence hosts the coal deposits that are present in the Beaufort and Ecca Groups, some of which are thin, discontinuous seams, but several that are thick and continuous and are mined. Figure 22 shows the simplified geology for South Africa, but also indicates the locations of active Coal Mines, while Figure 24 provides a zoomed-in view of the Coal Fields of South Africa.

back

back

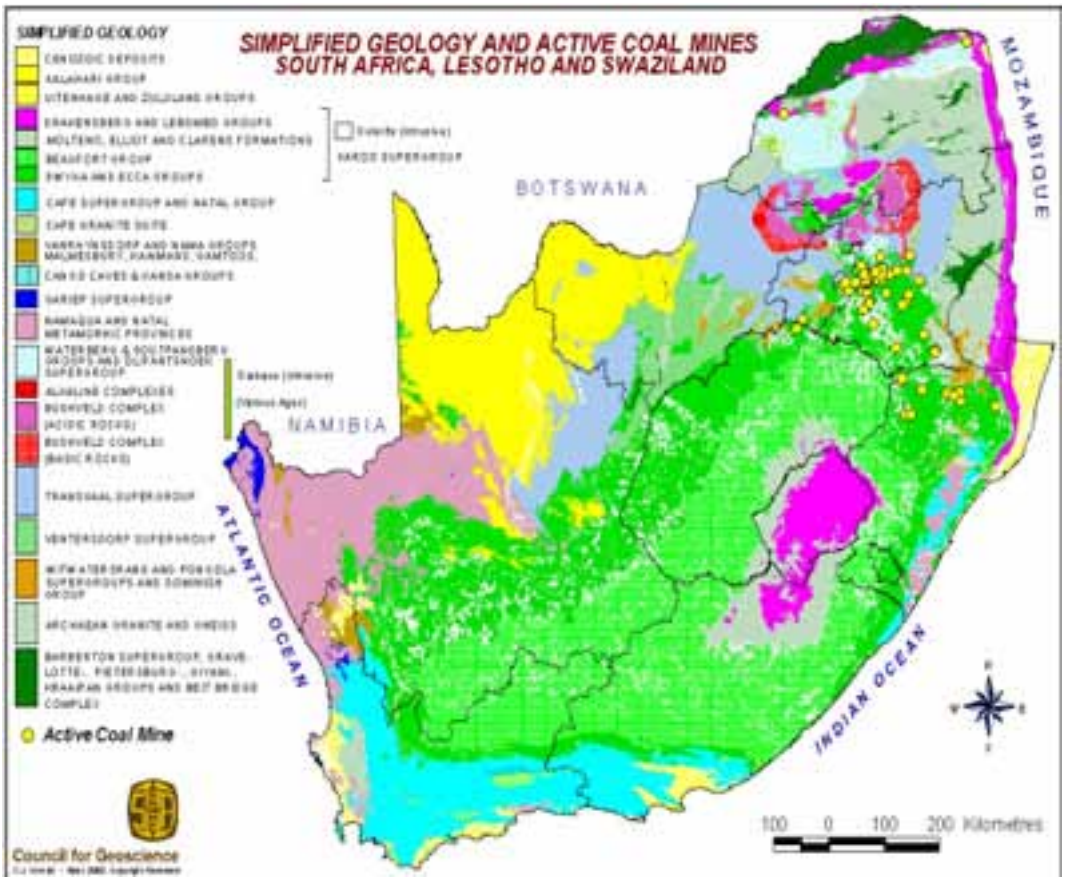


Figure 22: Geology for the Coal mines (CGS)

back

back

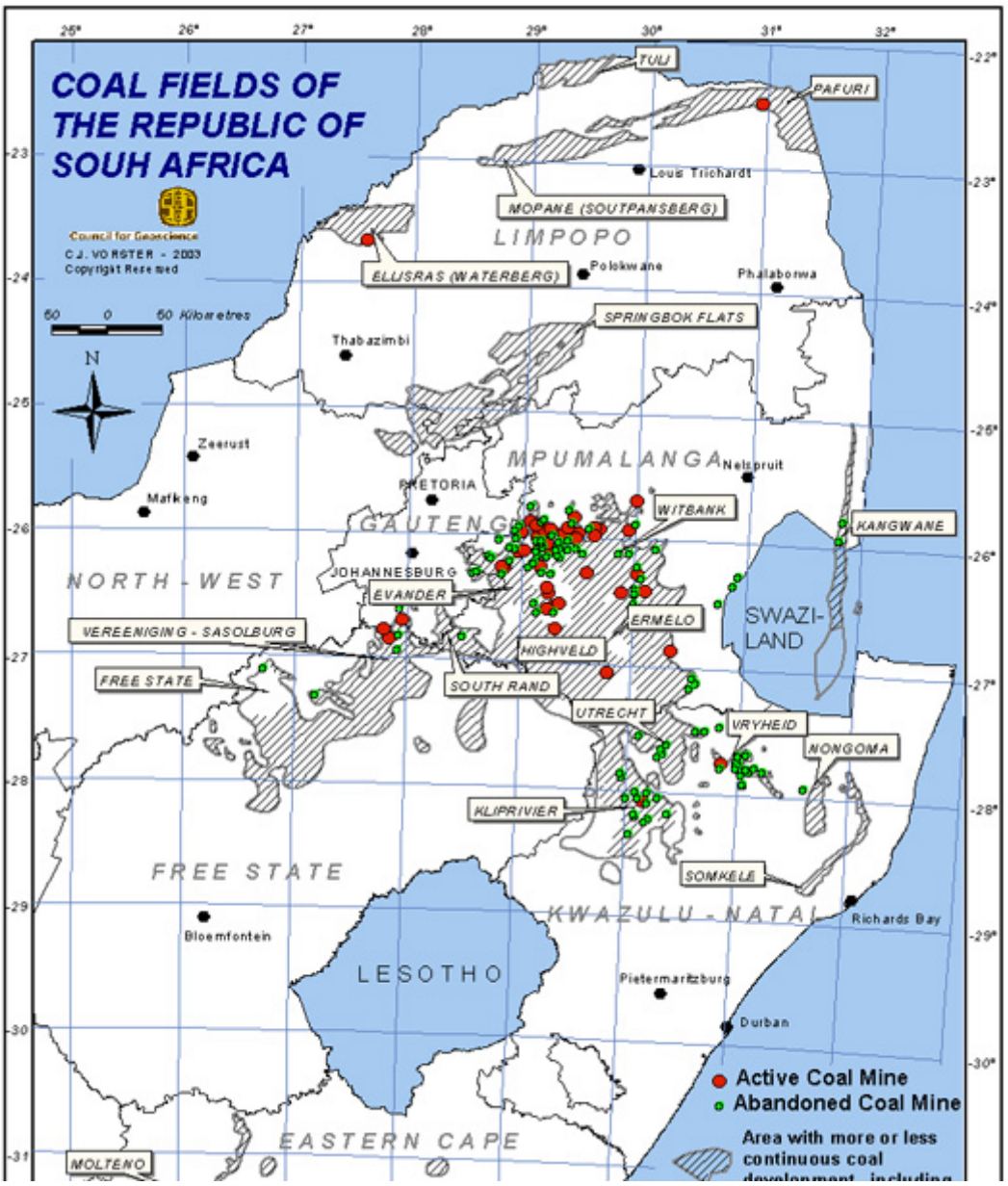


Figure 23: Coal Fields of South Africa (CGS)

Witwatersrand Supergroup

The Witwatersrand Supergroup is, due to extensive gold mining in RSA, very well known. The basin underlies the Free State and Southern Gauteng areas (Figure 26) and hosts many gold mines as shown in Figure 27.

back

back

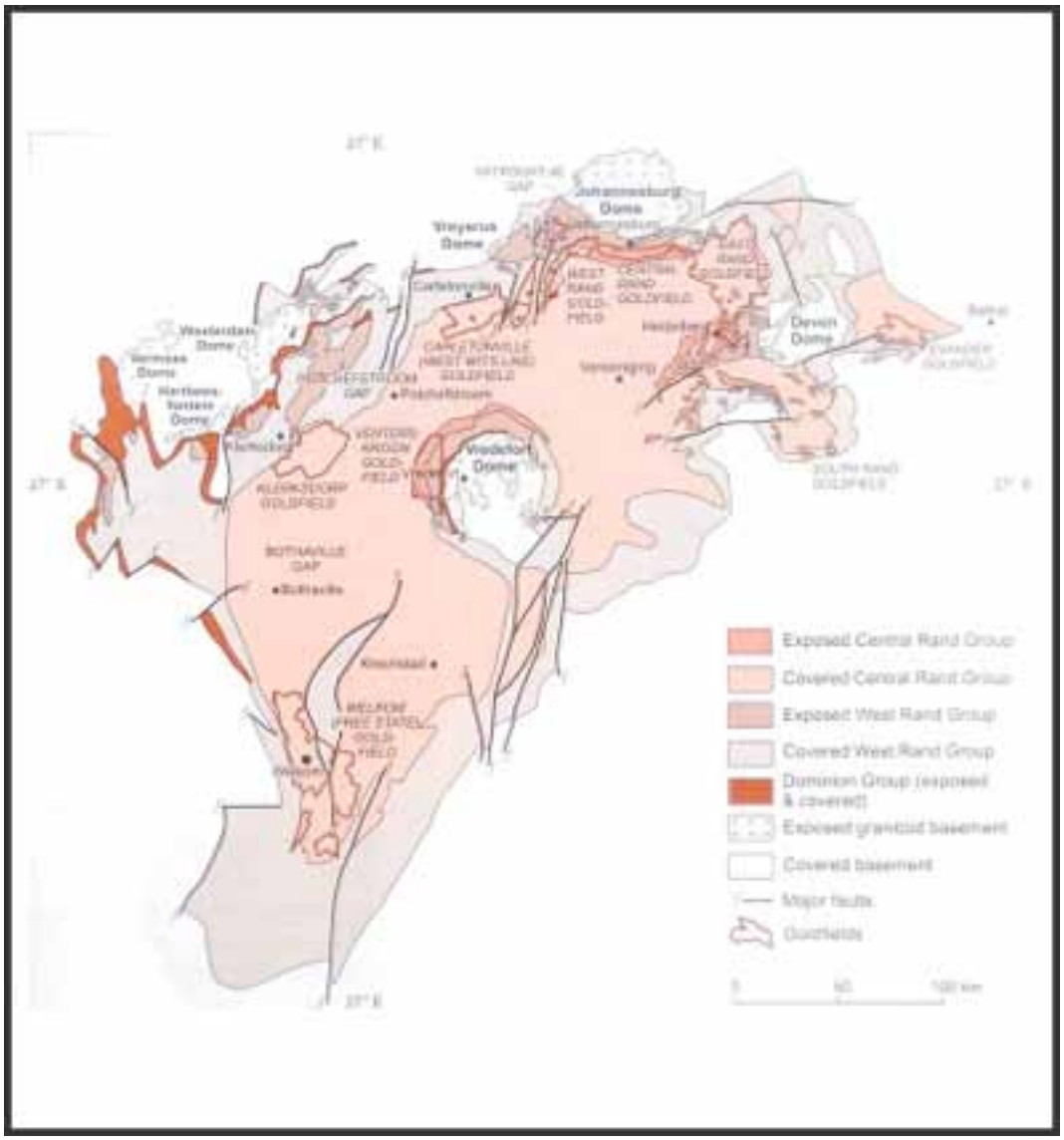


Figure 24: Witwatersrand Basin (Johnson et al., 2006)

back

back

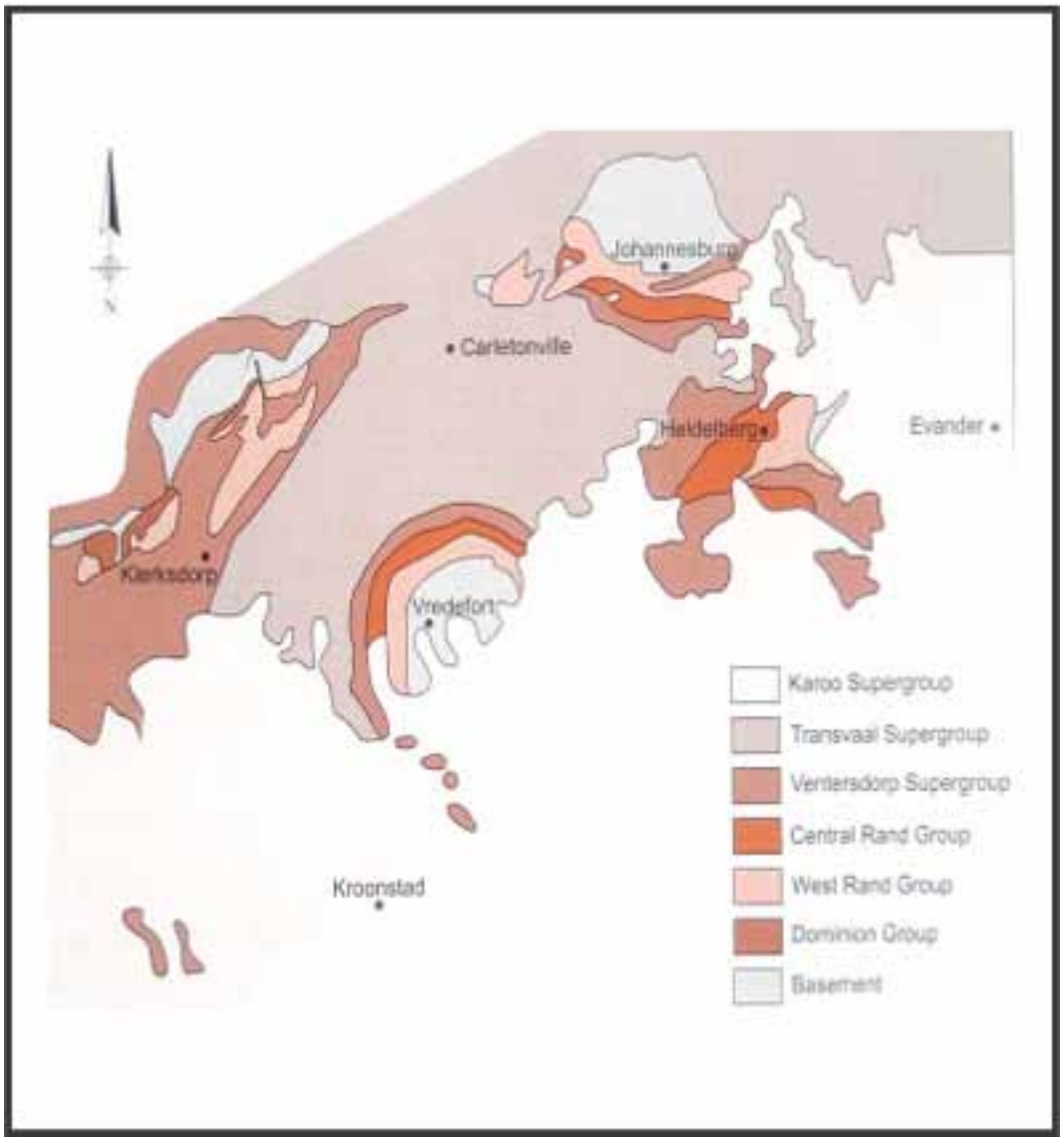


Figure 25: Outcrops of the West Rand and Central Rand Groups (Johnson et al., 2006)

back

back

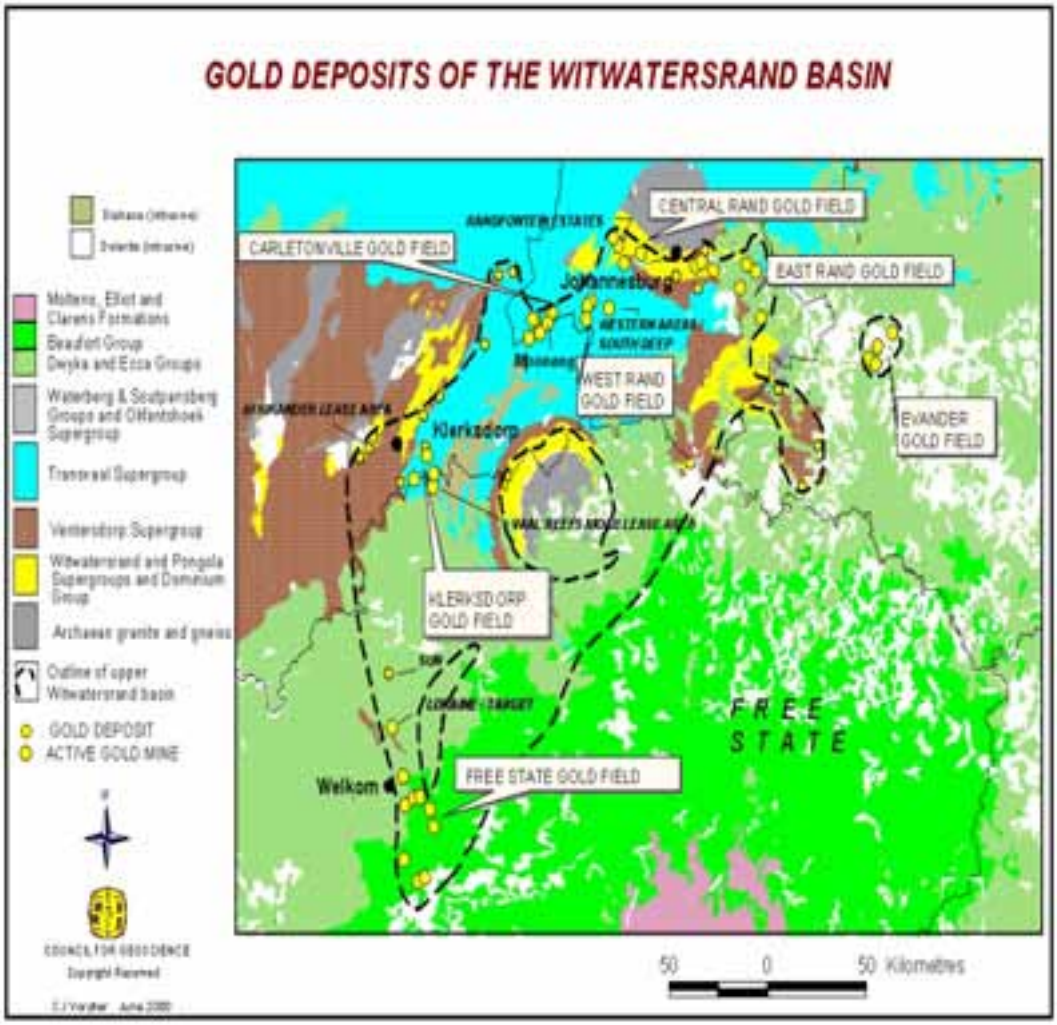


Figure 26: Gold deposits in the Wits Basin (CGS)

back

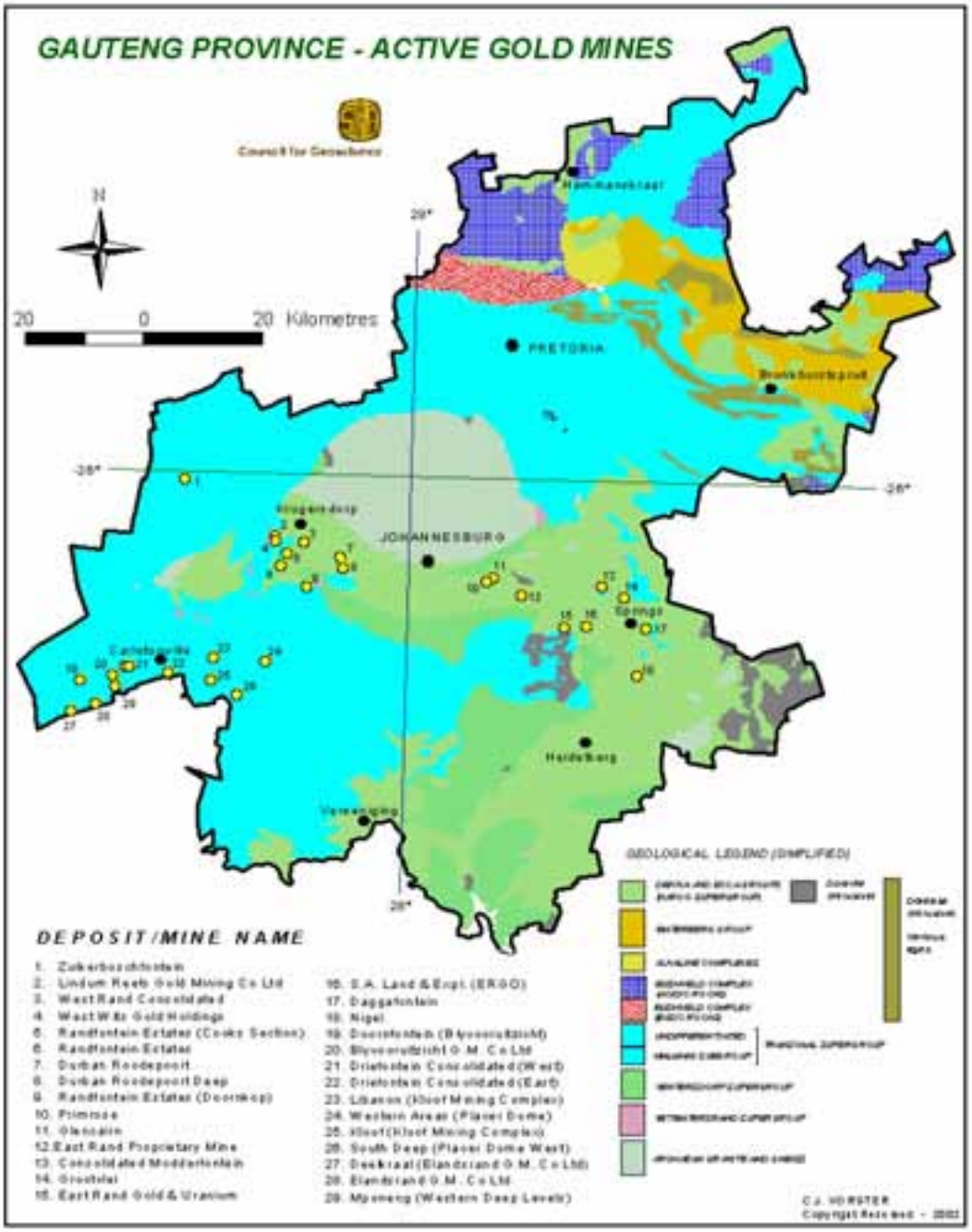


Figure 27: Active gold mines in Gauteng (part of Wits Basin) as in 2002 (CGS)

Mine names may have changed since 2002.

Large faults are known to exist at different throws and orientations, from Roodepoort to the Free State, throughout the whole Supergroup.

The Supergroup is divided into:

- West Rand Group (lower Wits system)
  - Hospital Hill Subgroup
  - Government Subgroup
  - Jeppestown Subgroup
- Central Rand Group (upper Wits system)
  - Johannesburg Subgroup
  - Turffontein Subgroup.

Mining of gold within these groups is practised in several Goldfields, shown in Figure 27 and Figure 29 and include, inter alia, the following main mineable gold reefs:

- Welkom (Free State): A , B , Basal and Leader Reefs
- Klerksdorp: VCR, C and Vaal Reefs
- Carletonville: VCR and Carbon Leader
- West Rand: Elsburg, South and Main Reefs
- Central Rand: Kimberley, South, Main and North Reefs
- East Rand: Kimberley and Bird Reefs
- South Rand: VCR, Kimberley, Bird and South Reefs
- Evander: VCR, Kimberley and Bird Reefs

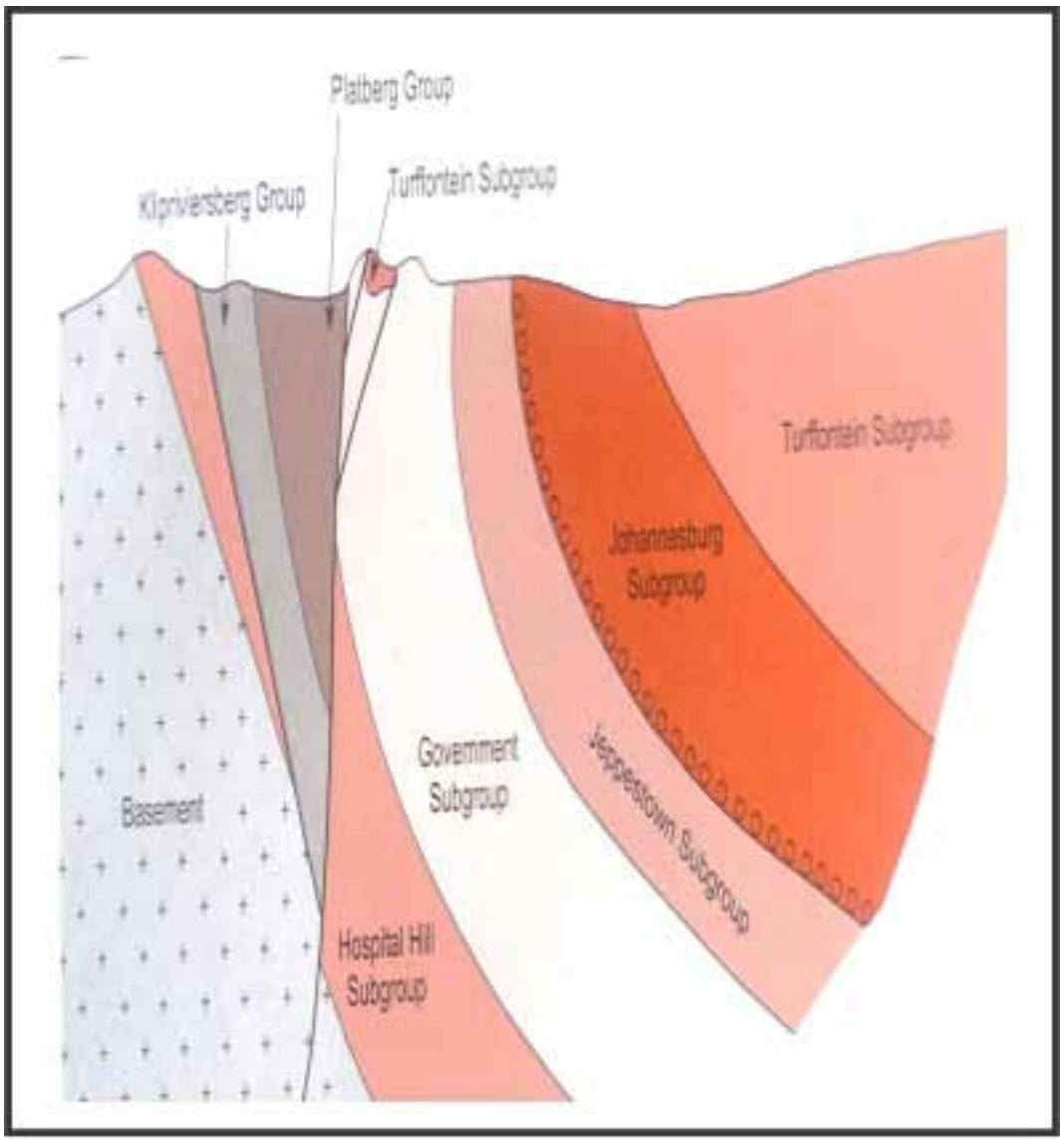


Figure 28: Section through the Central Rand (Johnson et al., 2006)

back

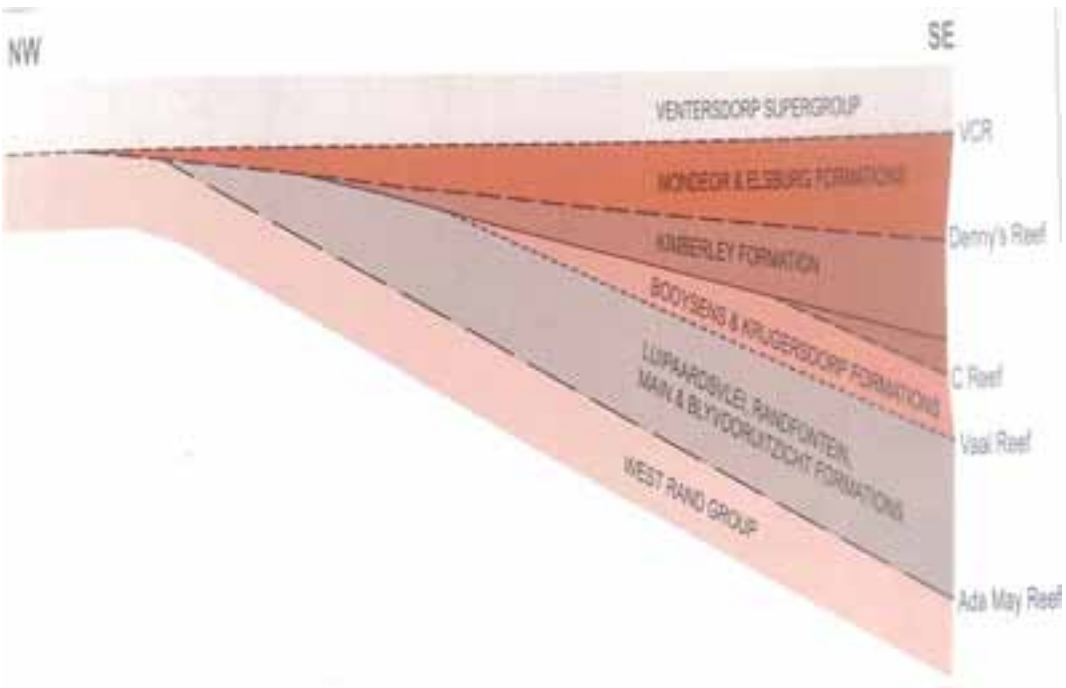


Figure 29: Section through the Central Rand towards the Klerksdorp Goldfield (Johnson et al., 2006)

back

back

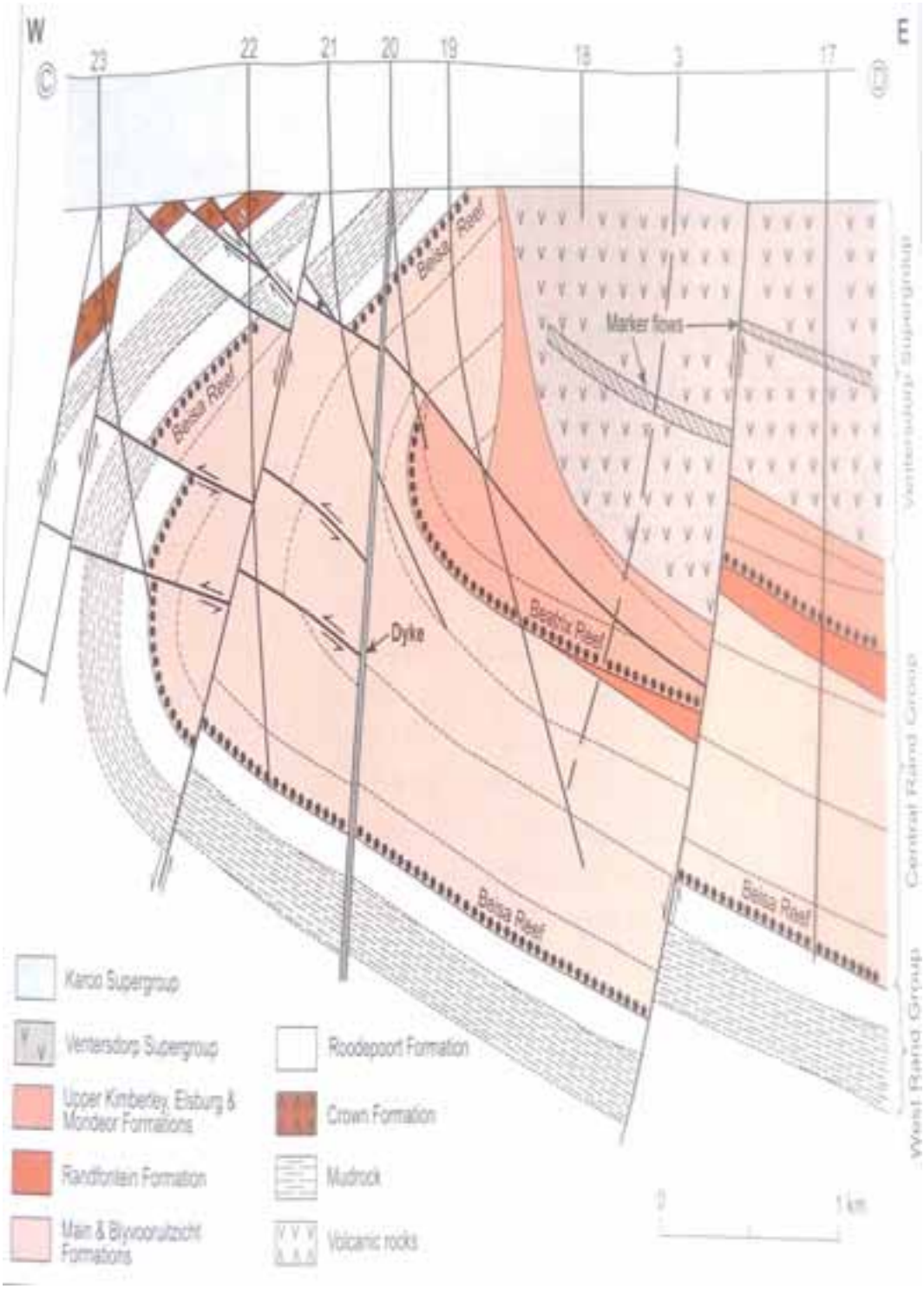


Figure 30: Section through the Welkom (Free State) Goldfield (Johnson et al., 2006)

Ventersdorp Supergroup

The outcrop of the Ventersdorp Supergroup occurs in an area between Barkley West, Vryburg, Mafikeng, Ventersdorp and Klerksdorp. The Supergroup is divided into:

- Lower Klipriviersberg Group
  - Consists mainly of volcanics
- Middle Platberg Group consists of:
  - Amygdaloidal and porphyritic lavas
  - Pyroclastics and
  - Sedimentary rocks.

back

back

In some areas, where this Supergroup overlies the Witwatersrand Supergroup, a conglomerate exists along its base that has been derived from erosion products of the Witwatersrand Supergroup and contains gold. This reef is called the Ventersdorp Contact Reef.

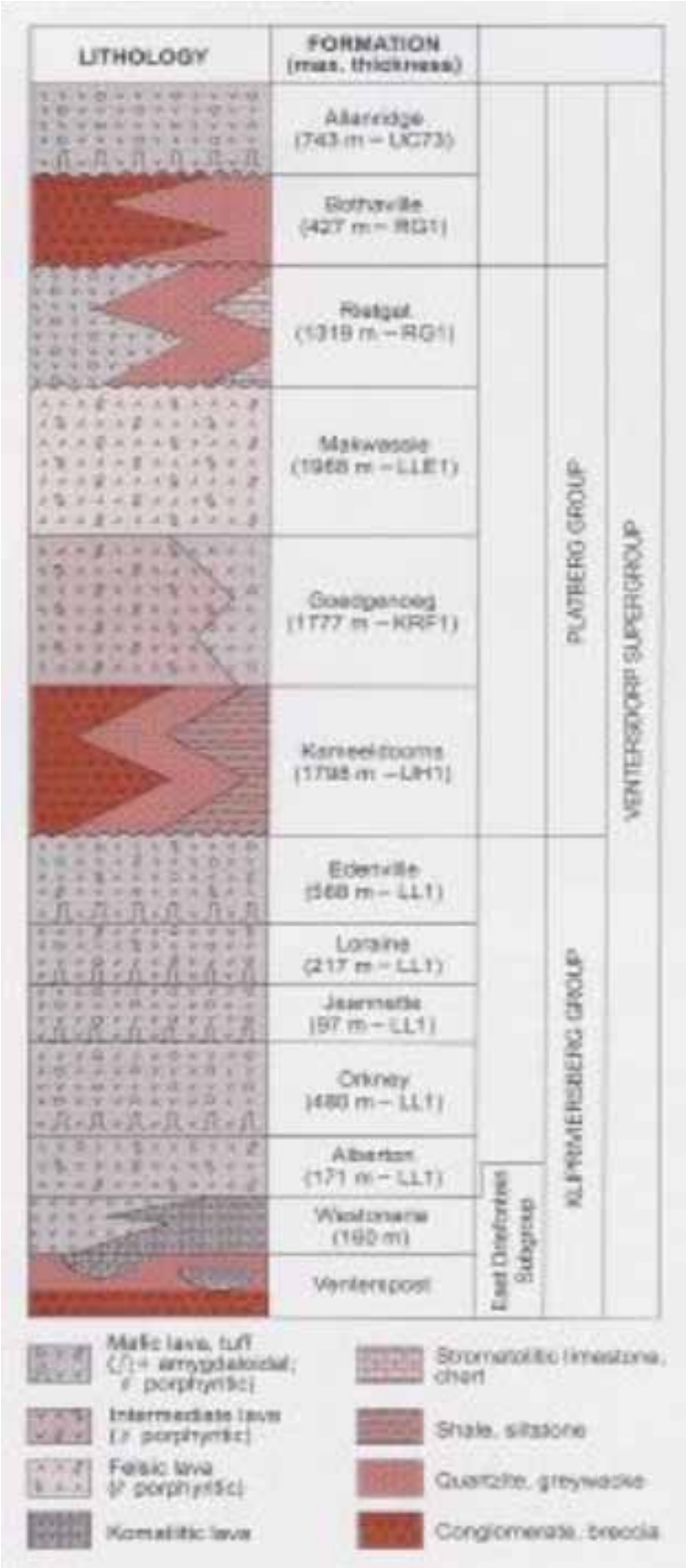


Figure 31: Ventersdorp Supergroup stratigraphy

back

back

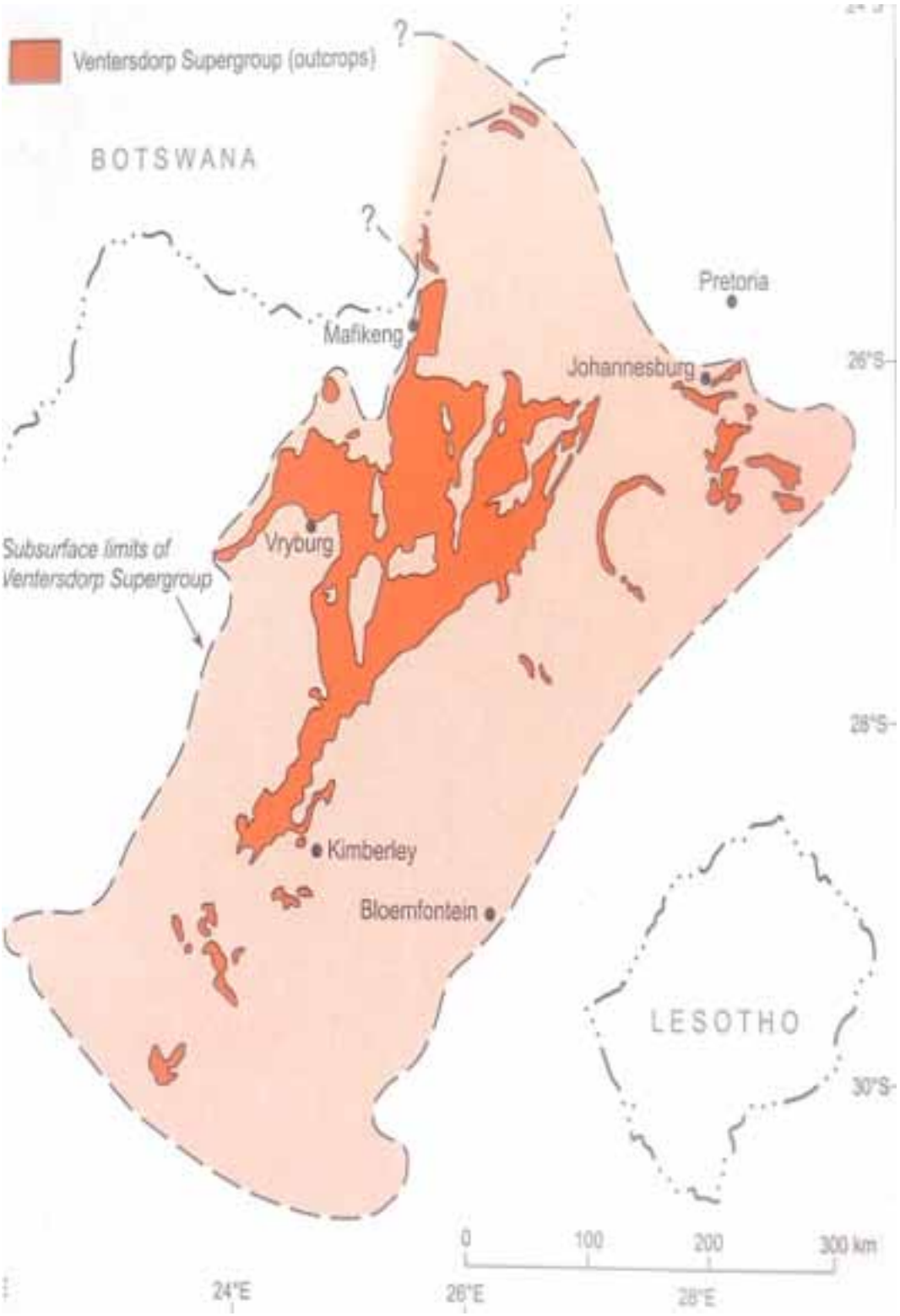


Figure 32: Distribution of Ventersdorp Supergroup (Johnson et al., 2006)

back

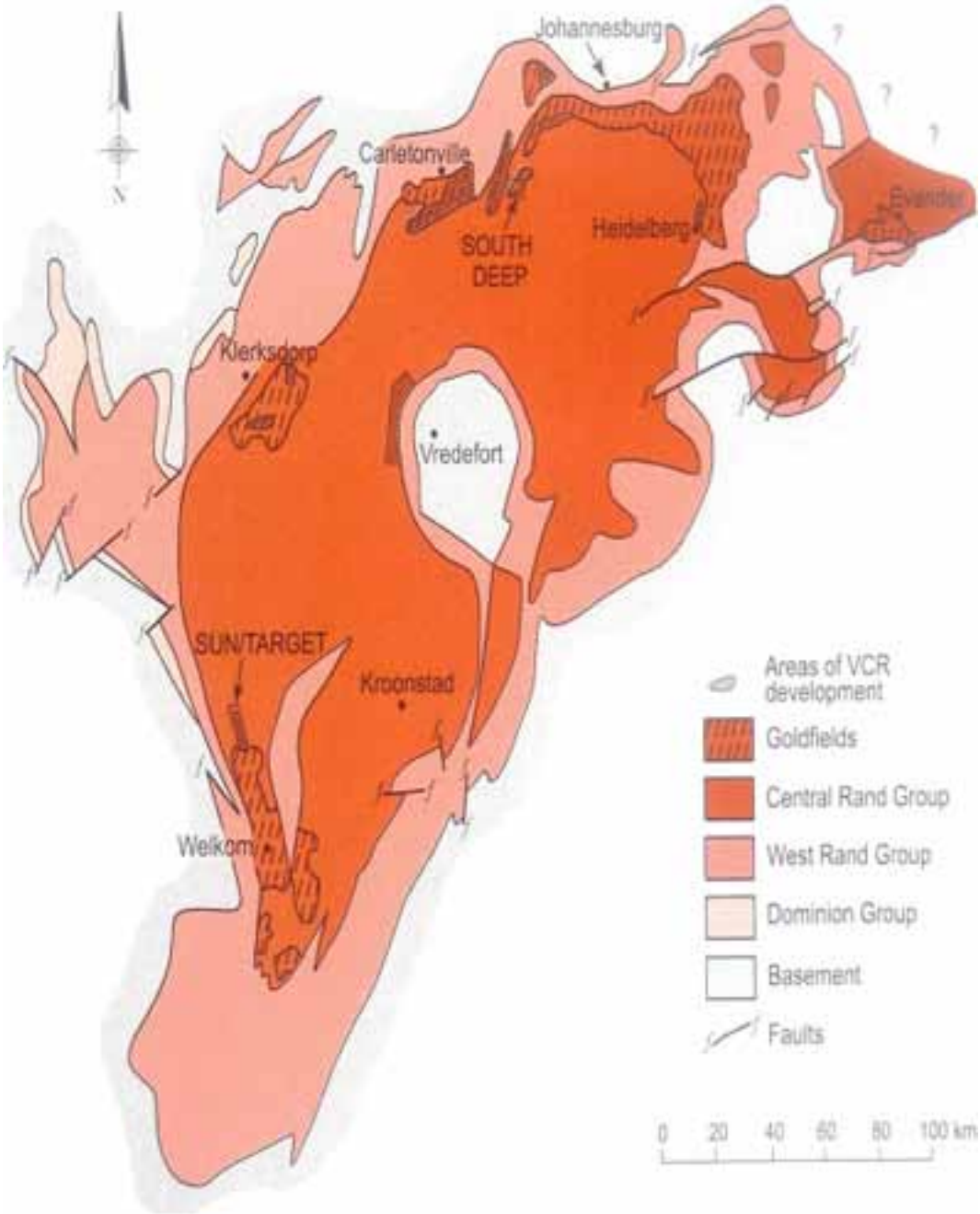


Figure 33: Known Ventersdorp Formation (VCR) occurrences (Johnson et al., 2006)

TRANSVAAL SEQUENCE

The Transvaal Sequence overlies the Ventersdorp Supergroup and includes outcrops from Botswana, across Gauteng and Mpumalanga, towards Swaziland. The Transvaal Sequence includes the basin-like Bushveld Complex, while it is caught up in the Witwatersrand Basin towards the south. The Transvaal Sequence is subdivided into:

- Wolkberg Group
  - Consists of quartzites and minor lava bands
  - Black Reef Formation consisting of quartzites, conglomerate (dark coloured) and shale thickens from to 800m in the eastern parts.

- Chuniespoort Group
  - Consists mainly of the Malmani Subgroup dolomites ranging from 40m to 2300m,

- Iron formations of the Penge formation and
- Carbonate and clastic rocks of the Duitschland formation.

- Pretoria Group
  - Consists of alternating quartzites and shale at a thickness of approximately 3700m.
  - Includes, inter alia, the following Formations:
    i. Rayton (sandstone shale, volcanic rocks)
    ii. Magaliesberg (sandstone)
    iii. Silverton (shale, lava)
    iv. Daspoort (sandstone)
    v. Strubenskop (shale)
    vi. Dwaalheuwel (sandstone)
    vii. Hekpoort (andesite)
    viii. Boshoek (conglomerate, sandstone)
    ix. Timeball Hill (shale, sandstone, lava)
    x. Rooihoogte ((conglomerate, sandstone)

- Rooiberg Group
  - This volcanic group overlies the Pretoria group, forms part of the cover (‘roof’) of the Bushveld Complex and is often seen as part of the Transvaal Sequence.
  - The Group is divided into the following Formations
    i. Dullstroom (1500m thick) is at the base with mafic to intermediate volcanic rocks such as basals;
    i. Damwal (1500m thick), Kwaggasnek (1000m) and Schrikkloof (1000m) is a thick sequence of siliceous extrusive rocks.

back

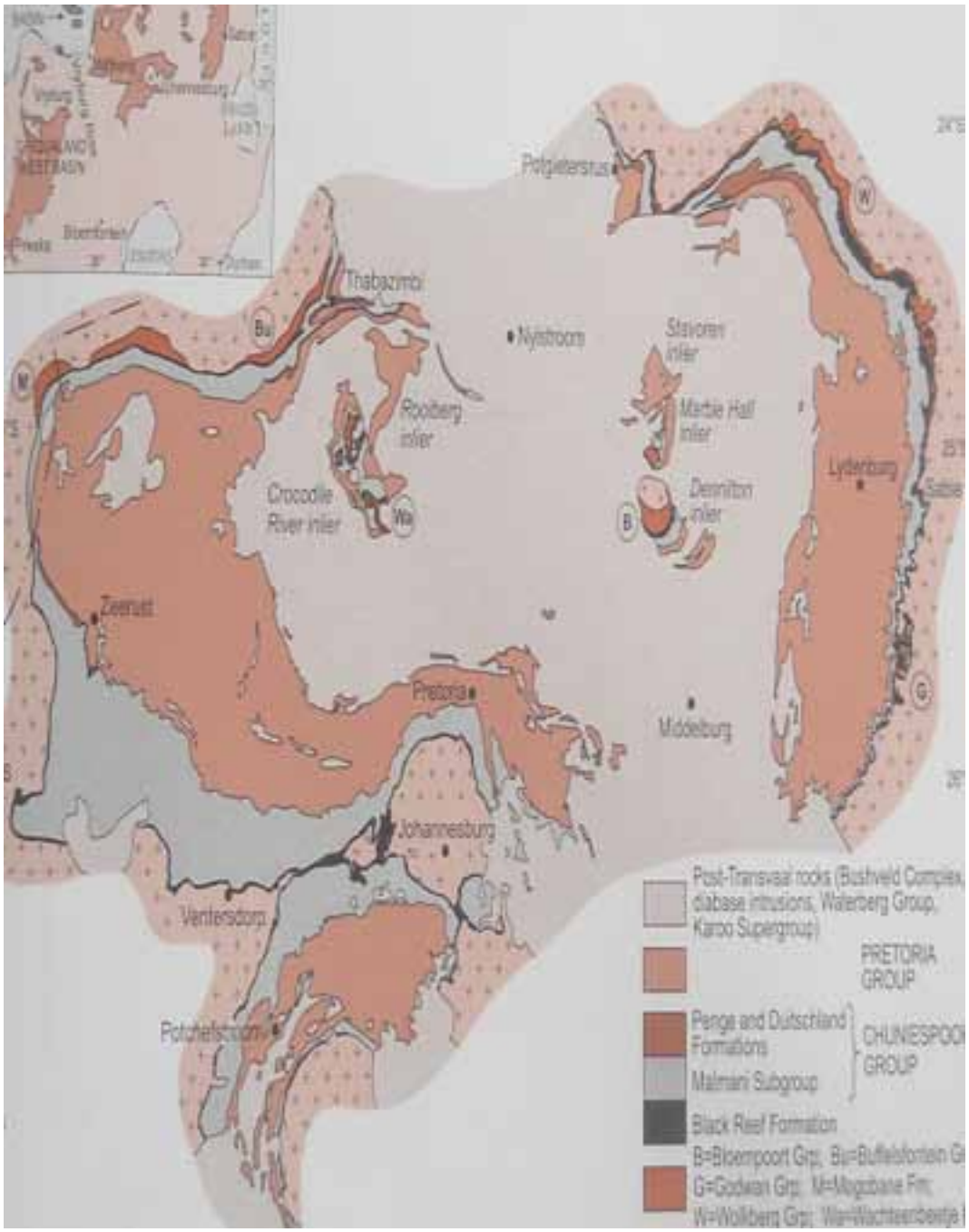


Figure 34: Transvaal Basin (Johnson et al., 2006)

back

Figure 35: Transvaal Basin (Johnson et al., 2006)

Bushveld Complex

The complex consists of various igneous rocks over a large area stretching from Rustenburg in the west to Lydenburg in the east, a distance of approximately 500km. This sequence was possibly created by several phases of volcanic action:

- A basic volcanic (extrusive) phase as the Transvaal Supergroup was formed, resulting in interbedded lavas;
- A basic sill (intrusive) phase during which diabase sills were formed in the Pretoria Group Shales;
- An acid colcanic (extrusive) phase creating the Rooiberg Group forming the 'roof' of the Bushveld Complex;
- A basic plutonic phase (intrusive) during which a layered sequence of ultrabasic and basic rocks intruded into the Upper Pretoria Group;
- An acid plutonic (intrusive) phase where magma intruded on top of the basic rocks and consolidated.

back

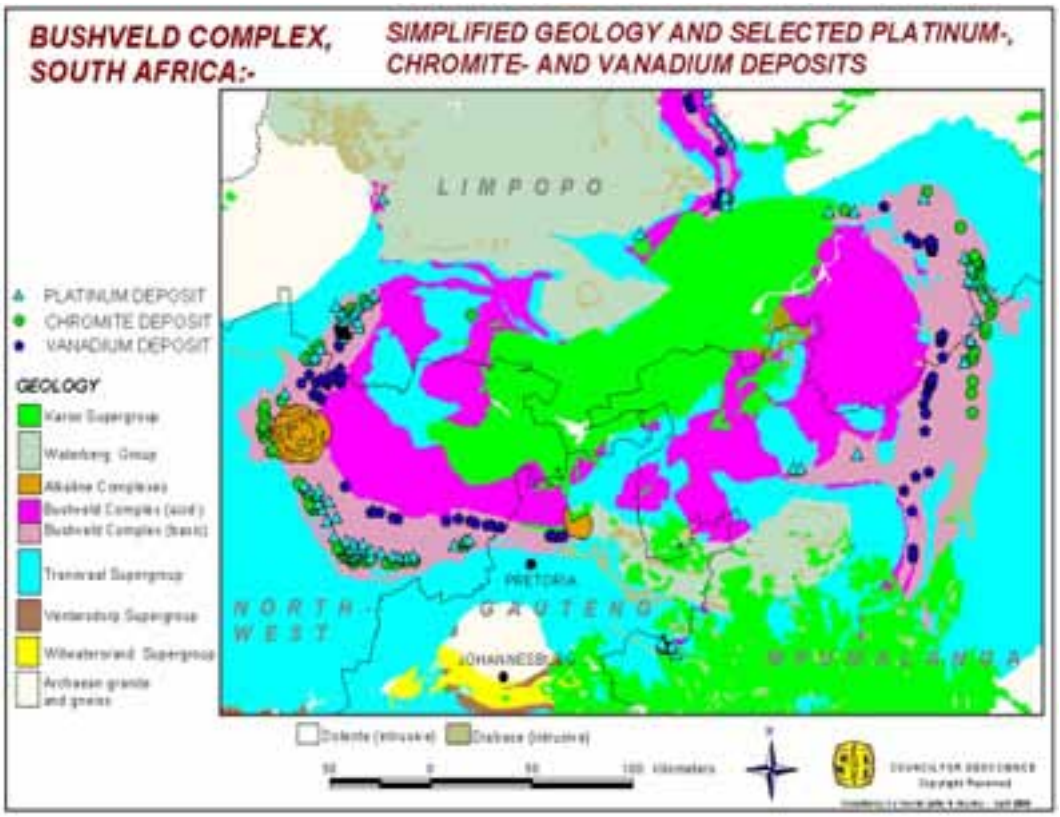


Figure 36: Bushveld complex geology and deposits (CGS)

Formal Subdivision				Informal Subdivision	
BUSHVELD COMPLEX	Lebowa Granite Suite				
	Rashoop Granophyre Suite				
	Rustenburg Layered Suite	**Eastern Bushveld** Luipershoek Olivine Diorite Ironstone Mag-netite Gabbro Magnite Heights Gabbro-Norite	**Western Bushveld** Bierkraal Magnetite Gabbro	Upper Zone	Subzone C Subzone B Subzone A
		Mappoch Gabbro-Norite Leolo Mountain Gabbro-Norite Winnarshoek Norite-Anorthosite	Pyramid Gabbro-Norite	Main Zone	Subzone C Subzone B Subzone A
		Winterveld Norite-Anorthosite Mooihoek Pyroxinite	Mathlagame Norite-Anorthosite Ruighoek Pyroxinite	Critical Zone	Upper Subzone Lower Subzone
		Serokol Bronzitite Jagdlust Harzburgite Rostock Bronzitite Clapham Bronzitite	Tweelaagte Bronzitite Groenfontein Harzburgite Makgope Bronzitite Eerlyk Bronzitite	Lowe Zone	Upper Bronzi-tite Subzone Hazburgite Subzone Lower Broni-tite Subzone Basal Subzone
		Shelter Norite	Kolobeng Norite	Marginal Zone	

Table 5: Lithology of the Bushveld Complex (Lurie, 1987)

back

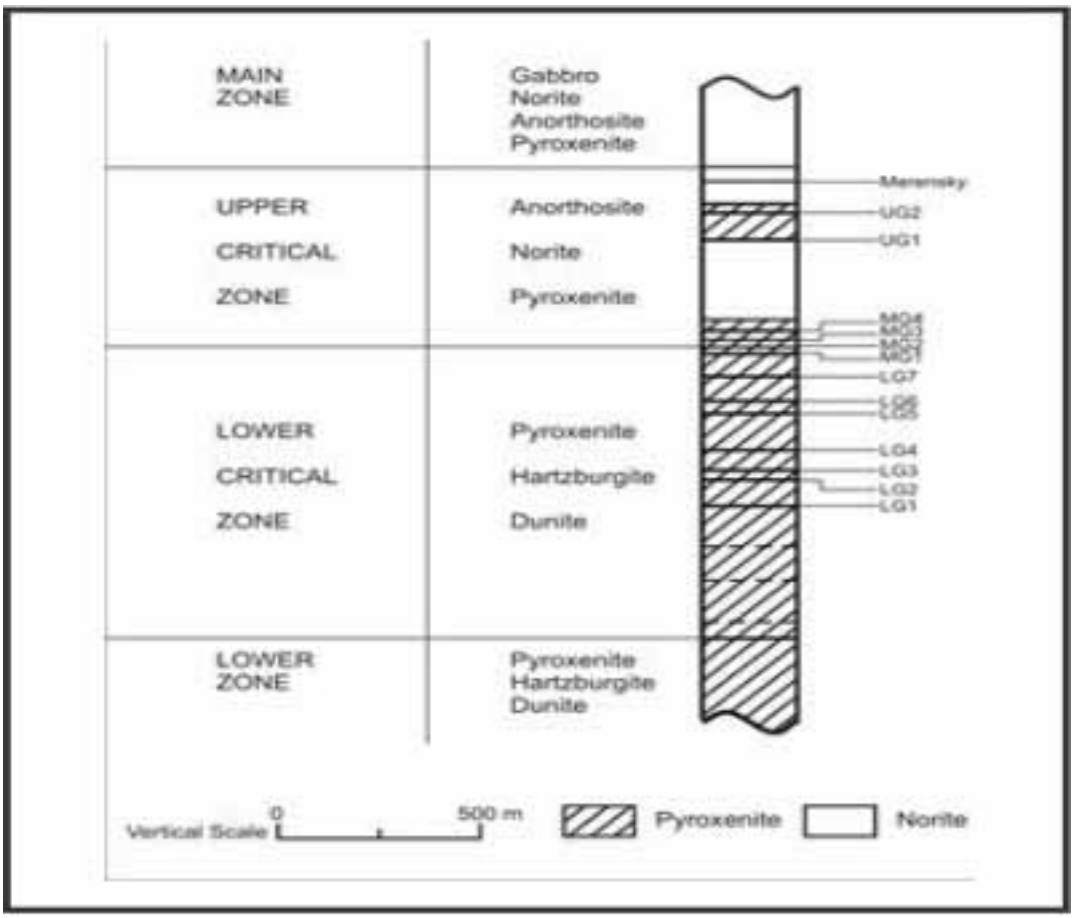


Figure 37: Bushveld stratigraphy (Jager and Ryder, 1997)

REFERENCE

- Johnson, M.R., Anhauser, C.R., Thomas, R.J. The Geology of South Africa, Council for Geoscience, 2006.
- Chapter 9, Page 44, Lurie, J. South African Geology, 1987.
- Wilson, M.G.C., Anhauser, C.R. The mineral resources of South Africa, Council for Geoscience, 1998.

LEARNING OUTCOME 2.2.2

DESCRIBE IN PARTICULAR THE ROCK TYPES ASSOCIATED WITH RELEVANT ORE HORIZONS

The geotechnical importance of the rock types listed below in Table 6 is by no means viewed as complete and does not include site-specific alterations or changes in the immediate area of the ore bodies. The conditions therefore include those 'typically' associated with the different ore bodies and should be appended with more detail information by the reader, from the references given.

Ore body	Rock types associated with the immediate ore body	Geotechnical importance
Coal	Sandstone, Siltstone, Shale, Mudstone	Massive sandstone bridges across roadways, laminated shale affect roof stability when present. Strengths of materials affect surface impacts if caving occurs. Weathered material can assist caving and even result in 'plug' failures between pillars. Jointed rock mass affects roof displacement.
Gold	Quartzite, Shale, Lava	Siliceous quartzite presents a risk to violent failure. Argillaceous quartzite increases closures into mined out voids. Crossbedding causes stability problems. Rock mass is laminated, requiring span controls to ensure stability. Shale exposure results in collapses of the hangingwall unless small spans are maintained. Shale presence increases potential for bed separation and instability. Lava is jointed and in some areas of very poor quality. Several different lava types exist and each is associated with different behaviours, such as western area formation (WAF), which has poor strength and requires small spans to maintain stability.
Manganese	Banded ironstone	Extremely laminated but with strong cohesion between beddings. Span control is critical to prevent unravelling. Jointing is present in the rock and provides surface that allows freedom of movement in excessive spans.
Diamonds	Dolomite, Quartzite, Lava, Shale	Dolomite is in essence massive, but jointed and very competent. Quartzite is often interbedded with shale and provides a well-laminated rock mass with little cohesion on bedding planes. Joints provide the freedom for displacements if uncontrolled. The lava is jointed but is in most instances very competent.
Copper	Granite, gneiss	
Platinum	Pyroxenite, Norite, Anorthosite	Normally very competent material with a large number of joints that traverse the rock. Presence of chromite layers within the material provides low cohesion surfaces that become unstable if not adequately supported. Stress fracturing of Norite materials appears to be very different to the other materials and care should be taken to limit Norite exposure in the hangingwall.
Chrome	Pyroxenite, Norite	

Table 6: Rock types and geotechnical importance of ore horizons

Figure 38: Crossbedding in sedimentary environments

Some geological and deposit information regarding the diamond, manganese and copper deposits are also included below.

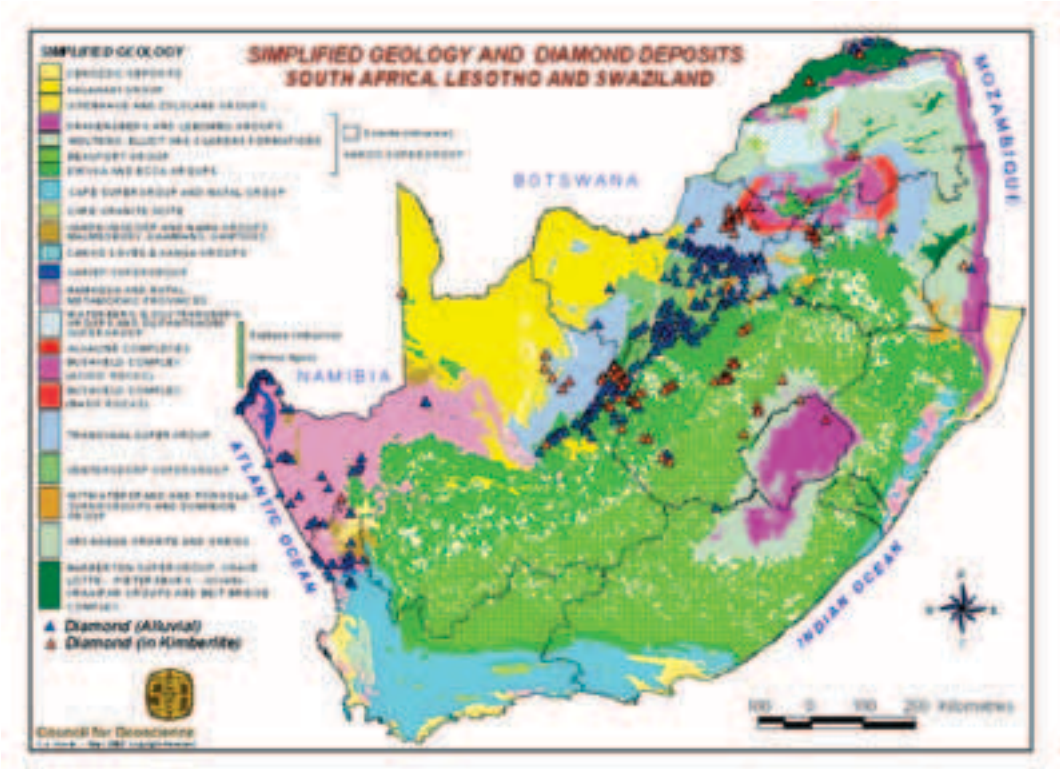


Figure 39: Diamond deposits (CGS)

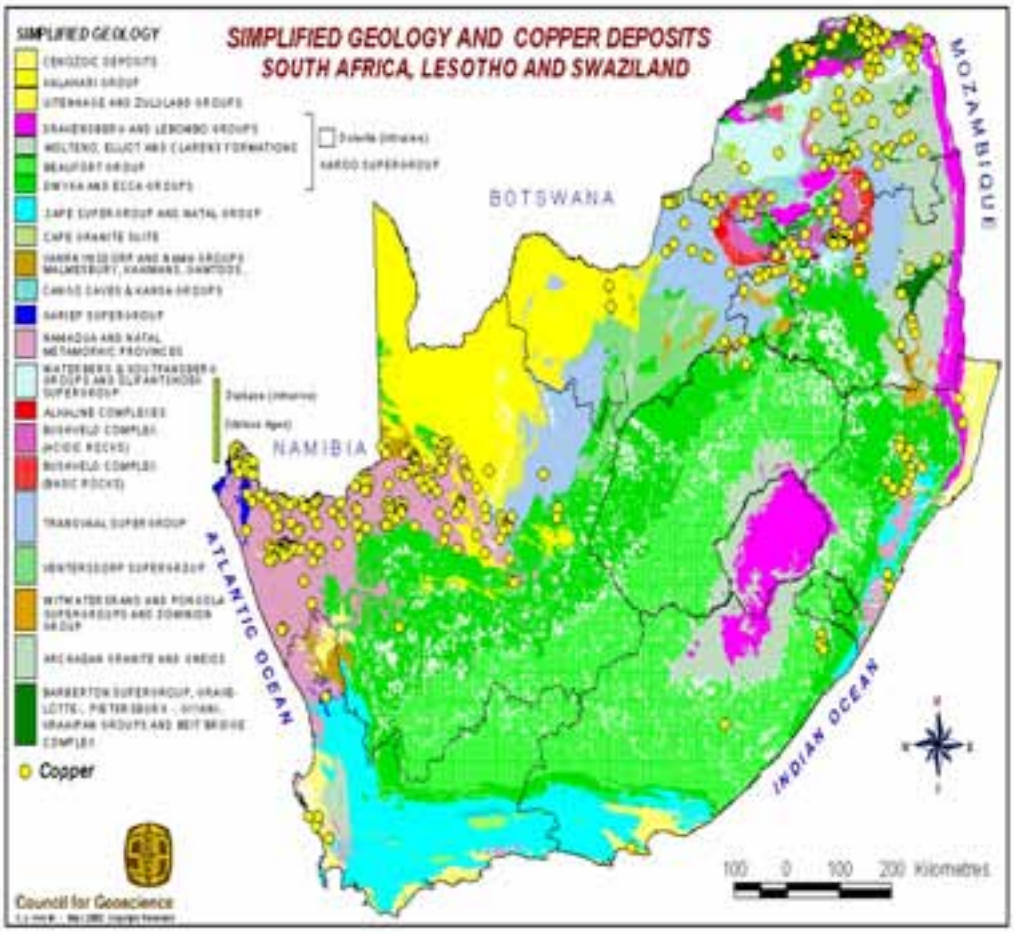


Figure 40: Copper deposits (CGS)

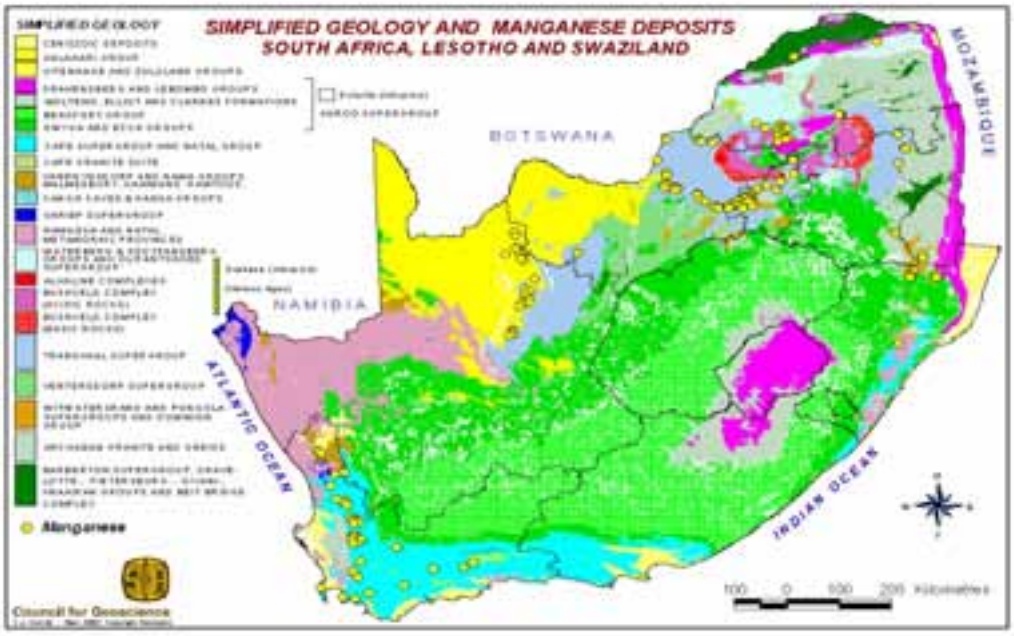


Figure 41: Manganese deposits (CGS)

REFERENCE

Council for Geoscience website www.geoscience.org.za

CONNECTION 4

Refer to "LEARNING OUTCOME 2.2.2" on page 56

LEARNING OUTCOME 2.2.3

DESCRIBE AND EXPLAIN THE GEOTECHNICAL SIGNIFICANCE OF THE ROCK TYPES ASSOCIATED WITH THE RELEVANT HORIZONS

LEARNING OUTCOME 2.3.1

CONSTRUCT AND APPLY GEOLOGICAL SECTIONS AND PROJECTIONS FROM BOREHOLE INFORMATION

Sections and projections can be drawn from borehole data, where the minimum information necessary to draw appropriate sections and projections is three boreholes, while multiple borehole data can be used to draw more accurate sections and projections showing more detail.

- Three-borehole data. With data from three boreholes, information is limited to the elevation of the ore body. Using the depths at which the boreholes intersected, the ore body, the ore body dip and strike directions can be determined. However, it is required that the boreholes are not located in a straight line. The application of this data is described using an example given by Lurie (1987), page 197.

		Borehole A	Borehole B	Borehole C	Borehole D
Data from boreholes	Collar elevation	190m	200m	170m	180m
	Depth of coal intersection	40m	65m	30m	Unknown
Calculated seam depth	Actual seam elevation above sea level	190-40=150m	200-65=135m	170-30=140m	

Table 7: Three borehole data

NOTE

Collar elevation means the depth of the top of the hole (i.e. where it enters the ground) and is usually measured to a regional Datum elevation (such as 'above sea level').

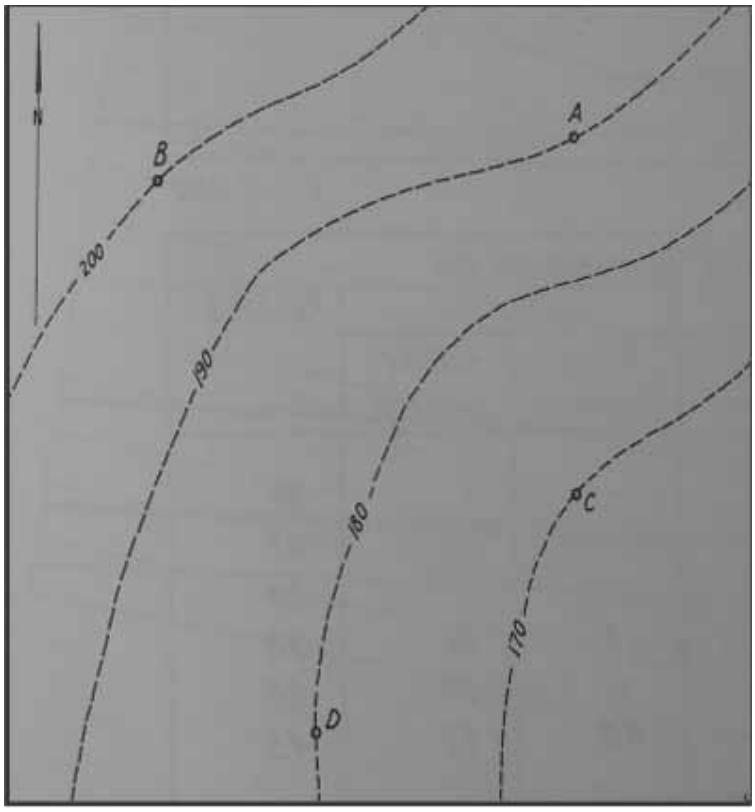


Figure 42: Borehole plan for three boreholes (Lurie, 1987)

The first step is to determine the actual elevation of the coal seams, since the information given in Table 6 only indicates the distance between the collar elevation and the coal seam intersection. The actual elevations can be determined by SUBTRACTING the distance below the collar, from the collar elevation (Table 6).

NOTE

Subtracting is required with this example since elevations are 'metres above sea level'. The elevations therefore become 'mathematically larger' above the collar elevations and 'mathematically smaller' below the collar elevations. If collar elevations were given below a different regional Datum line, collar elevations would be 'negative' and to get the coal seam intersection depths, the distance would again be subtracted from the 'negative collar elevations', resulting in a 'larger negative elevation', indicating the seam elevation depth below the Datum line (Figure 42).

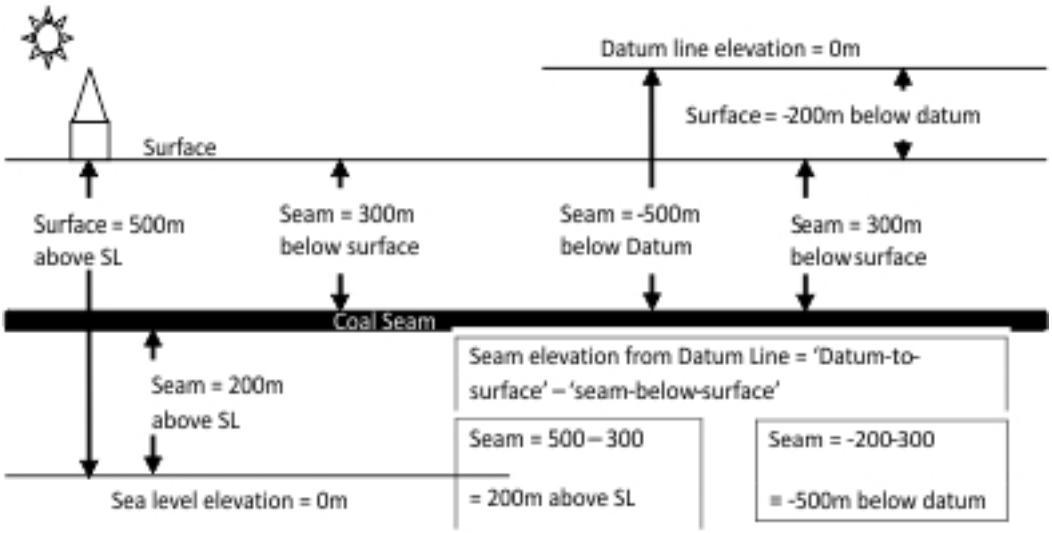


Figure 43: Datum line illustration

On the mine plan:

- Connect two of the boreholes (e.g. A and B, Figure 46) with a line;
- Subtract the depths of the coal seams at these two boreholes from each other (e.g. 150m-135m = 15m);
- Divide AB into three portions each representing 5m intervals (5m x 3 = 15m);
- Connect the 140m mark with Borehole C, which intersected the coal seam at 140m. This line represents the strike of the seam as all points along this line will be at 140m above sea level;
- By drawing lines parallel to the one above and through the 145m and Borehole A (150m) positions, strike lines can be drawn across the mining plan;
- Use these lines to indicate the true dip direction, which is 90° to the strike lines;
- Along the true dip direction, draw more strike lines at similar intervals along the true dip line until the strike lines intersect Borehole D. The elevation of this line is also the elevation at which the coal seam should be intersected in this hole.

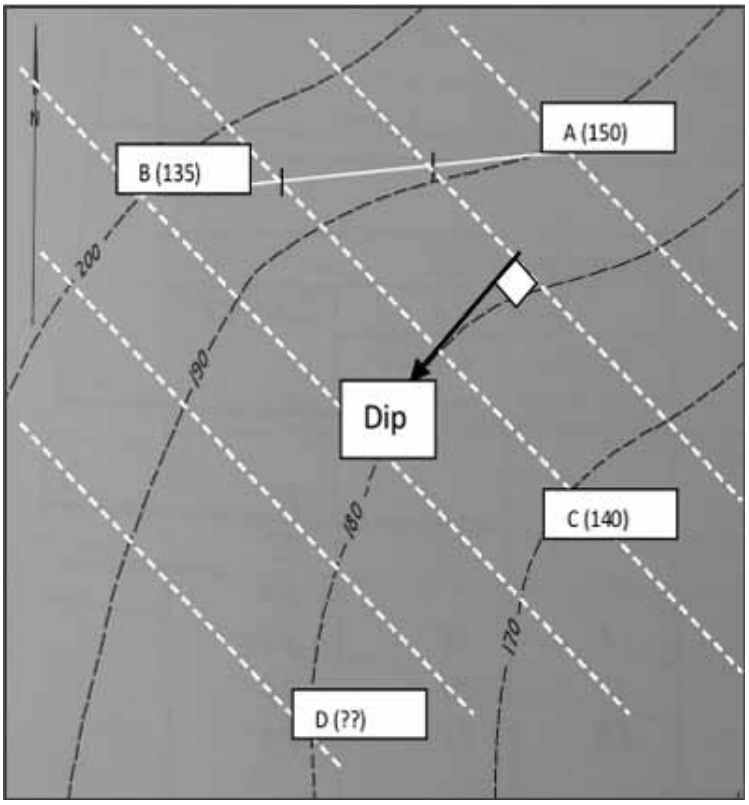


Figure 44: Three borehole data evaluation (Lurie, 1987)

- Multiple borehole data. The application of this data is described using an example given by Lurie (1987), page 198. The following data is also provided in Table 7 and the assumption is that all structure dips are constant. In this example, the following must be found:
  - Strike, dip and thickness of the dolerite;
  - The dolerite outcrop;
  - Dip of the coal seams;
  - Throw on the dolerite; and
  - An accurate section along line XY (Figure 48 and Figure 49).

back

Boreholes	Collar elevation (m)	Depth of intersection				
		Coal		Dolerite		Tillite
		No1 seam	No 2 seam	Upper surface	Lower surface	
B2	1088	23	33	-	-	38
B4	1092	27	37	-	-	42
C1	1089	19	29	-	-	34
D2	1096	-	31	11	26	36
D4	1094	-	29	9	24	34
E3	1093	28	38	48	63	43

Table 8: Multiple borehole data

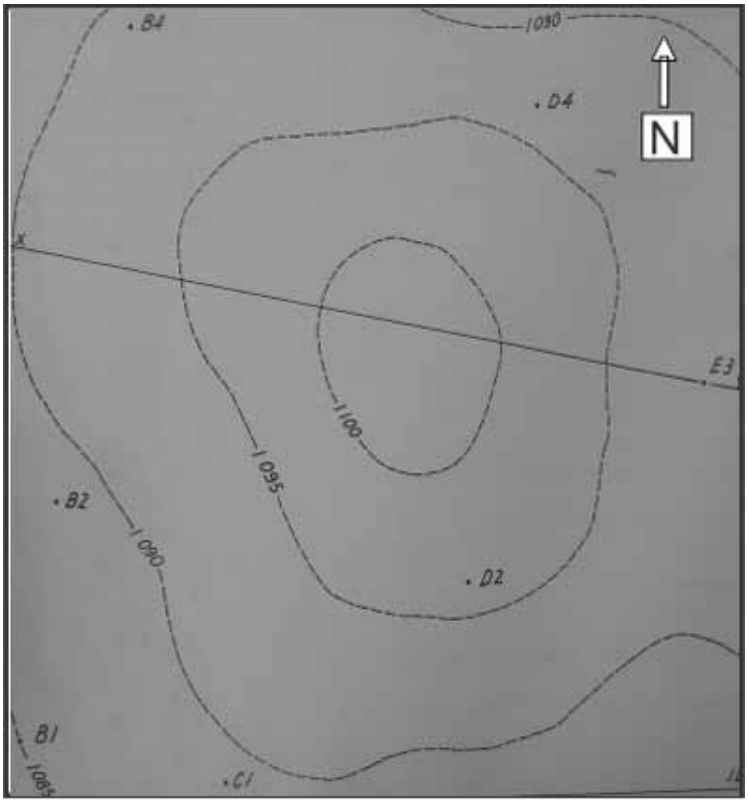


Figure 45: Multiple borehole information 1:1000 (Lurie, 1987)

a. Strike, dip and thickness of the dolerite

Actual elevations of all data in Table 7 are shown in the table below (see previous example).

Boreholes	Collar elevation (m)	Depth of intersection				
		Coal		Dolerite		Tillite
		No1 seam	No 2 seam	Upper surface	Lower surface	
B2	1088	1065	1055	-	-	1050
B4	1092	1065	1055	-	-	1050
C1	1089	1070	1060	-	-	1055
D2	1096	-	1065	1085	1070	1060
D4	1094	-	1065	1085	1070	1060
E3	1093	1065	1055	1045	1030	1050

Table 9: Calculated elevations for multiple borehole example

back

back

It follows from Table 8 that only Boreholes D2, D4 and E3 intersected the dolerite and provide data that can be used. From Table 8 it is clear that the dolerite was intersected by holes D2 and D4 at exactly the same elevation (1085m for upper and 1077m for the lower contact surfaces). A line connecting these boreholes would therefore suggest the strike of the dolerite (- - - -). The true dip is therefore in a direction 90° to the strike direction and is shown to indicate that E3 is situated along this true dip line at the point shown below.

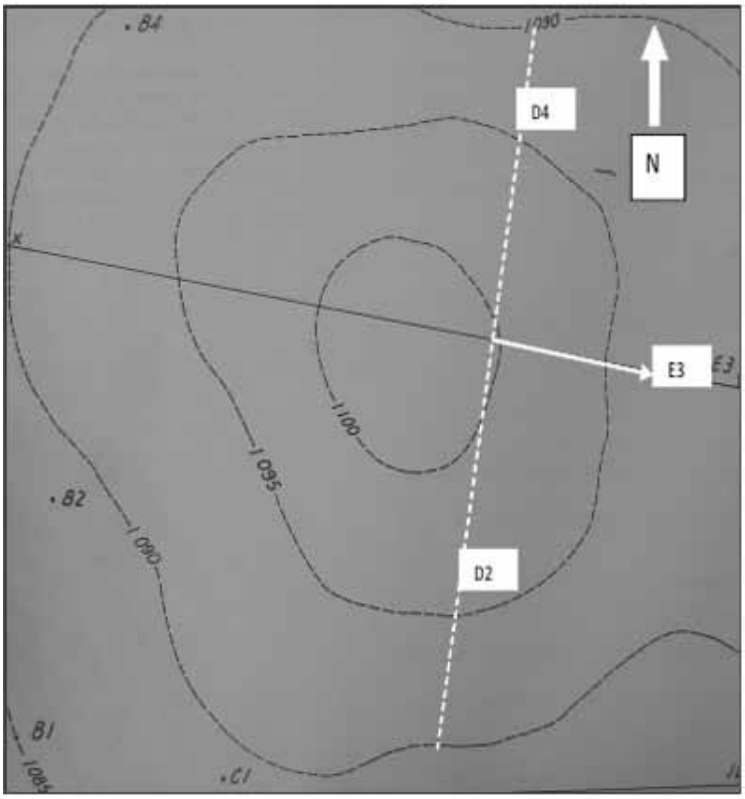


Figure 46

If E3 is situated along the true dip from the dolerite strike (D4/D2) line, as shown in Figure 46, it follows from trigonometry that:

The dolerite dip can be determined using Figure 46 and the following calculations:

$$\begin{aligned} \text{Tan } \theta &= (\text{Elevation difference/ Plan distance}) \\ &= (40\text{m} / 40\text{m}) \\ &= 1 \end{aligned}$$

$$\begin{aligned} \Theta &= \text{ATan } (1) \\ &= 45° \text{ east} \end{aligned}$$

The dolerite dip is at 45° from the strike line towards the E3 borehole position (east) and since it was assumed that all structure dips are constant, the dolerite dips at 45° east in this area.

back

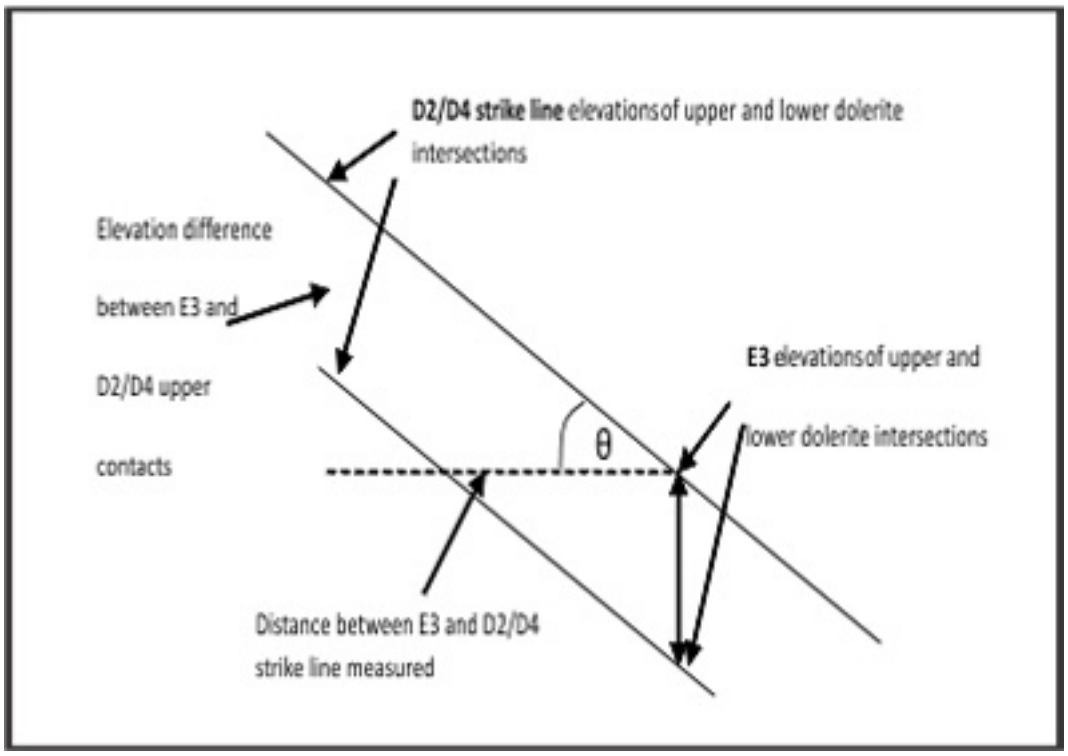


Figure 47

The dolerite thickness can be determined from Figure 48 below and trigonometry as follows:

The true thickness t can be determined from the triangle drawn below where the dolerite dip angle of 45° can be applied to show that:

$$\cos 45° = (t / \text{elevation difference})$$
$$= t / 15\text{m}$$

It follows that

$$t = 15\text{m}. \cos 45°$$
$$= \underline{10.6\text{m}}.$$

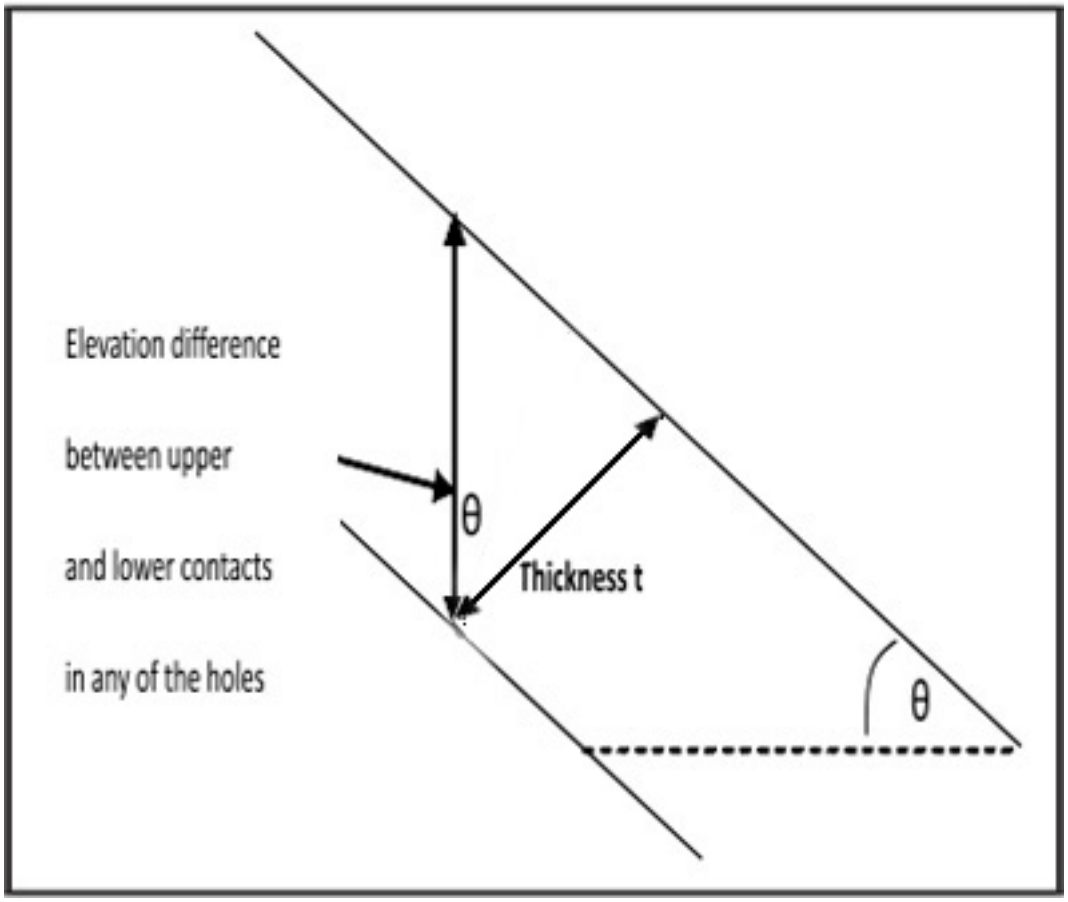


Figure 48: Dolerite thickness

b. The dolerite outcrop

The dolerite elevations on the strike line between D2/D4 are 1085m for the upper and 1070m for the lower contact. By drawing parallel strike lines at shallower (higher metres above sea level) elevations, the dolerite elevations will at some stage become equal to the surface contours, see **Figure 49**

NOTE

The dolerite contact elevations are given in the form of (upper contact elevation / lower contact elevation).

The elevations at which the dolerite contact and the surface become equal will show the surface outcrop positions of the dolerite.

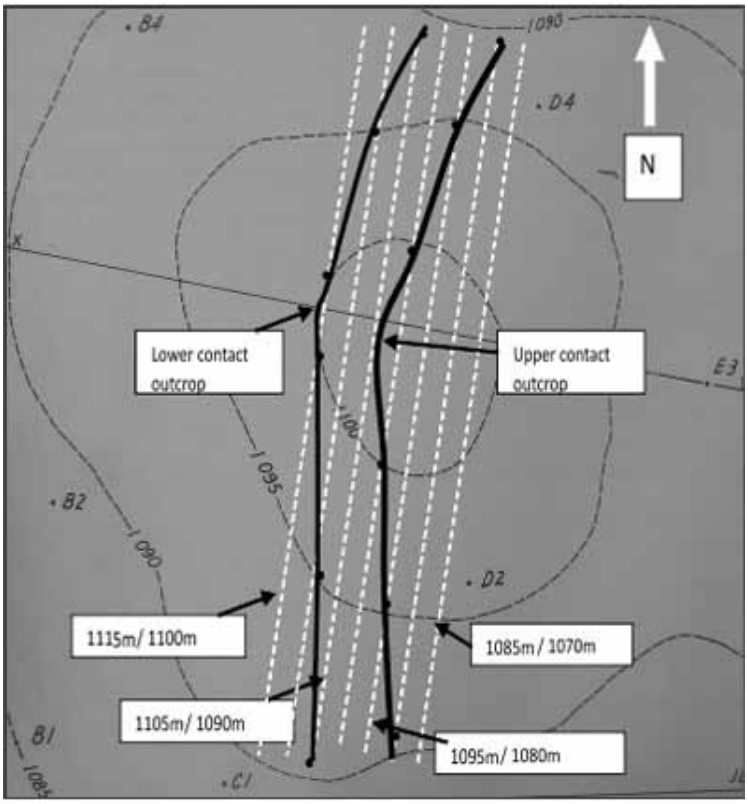


Figure 49: Plan view

c. Dip of the coal seams

Using Table 8, coal intersections for No1 and 2 seams are at the same elevations in B2, B4 and E3. Connect these boreholes with lines that indicate the coal seam strike and indicate true dip direction at 90° to the strike line, see Figure 50. It is assumed that since the dolerite may have a throw (next question), the two boreholes on the same side of the dolerite must indicate coal strike. The dip of the coal seams can be determined as in (a) for the dolerite, but using the B2/B4 and C1 elevations and positions.

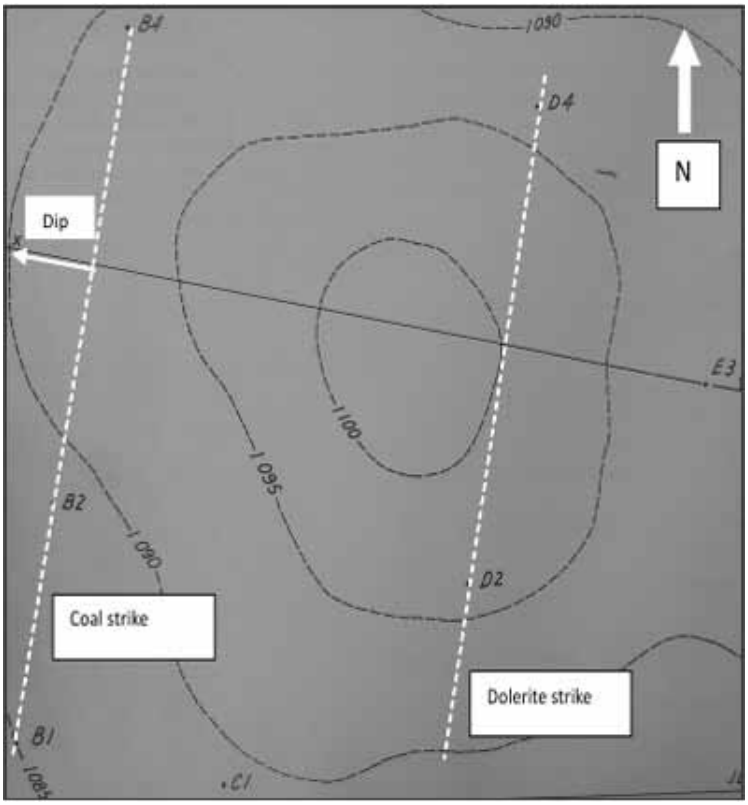


Figure 50: Strike directions

The coal seam dip can be determined using the following calculations:

Tan θ = (Elevation difference/ Plan distance)
= (5m /40m)
= 0.125

Θ = ATan (0.125)
= 7.13° west
(C1 elevation higher than B2/B4 elevation)

The coal seams dip at 7° from the strike line towards west and since it was assumed that all structure dips are constant, the coal dips at 7° west in this area.

d. Throw on the dolerite

Assuming that the strikes and dips of the two seams are the same in this whole area, the throw can be estimated by determining the middling between the same seam if elevations are extrapolated from west to east across the dolerite intersection.

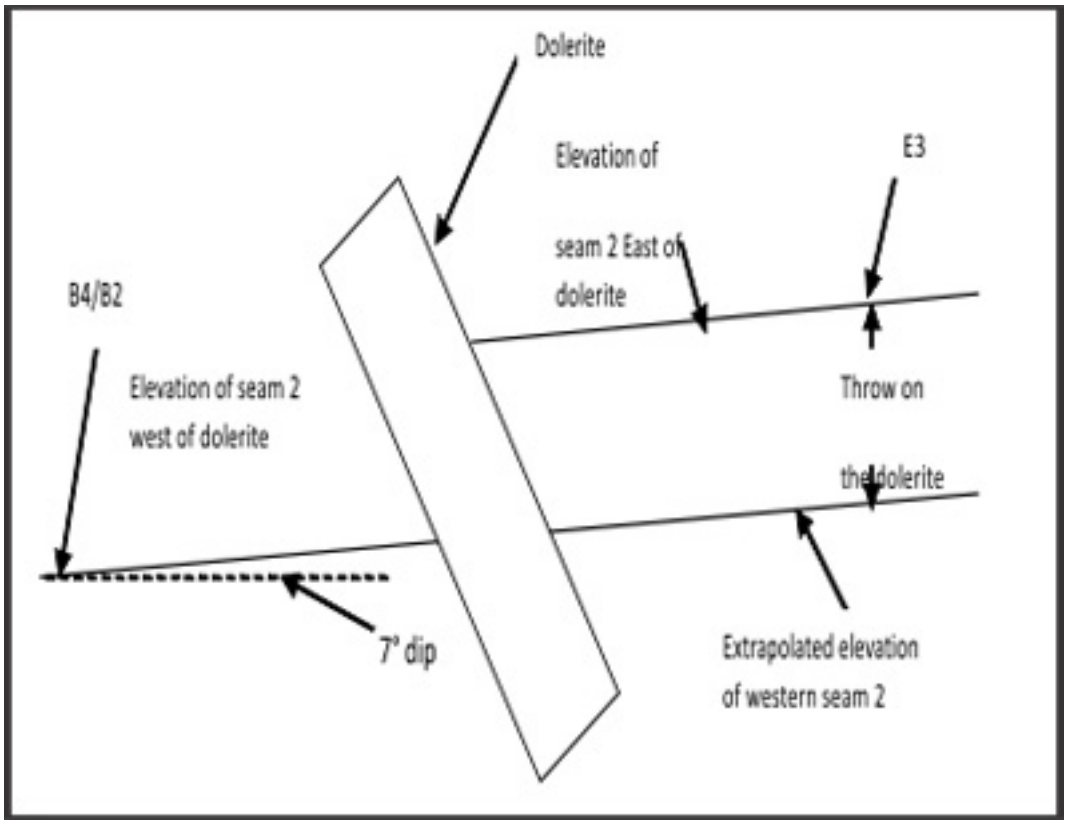


Figure 51: Section

Using the plan distance (measured from 1:1000 plan) between the B2/B4 strike line and the E3 borehole, and using a 7° coal seam dip, the elevation of the coal seam at E3 can be calculated if no throw on the dolerite existed, see Figure 51. By subtracting the extrapolated seam elevation from the actual seam elevation at E3, the throw can be determined. Using the No2 coal seam elevations, the following resulted:

- 112m plan distance between B2/B4 strike line and E3;
- Coal seam elevation of = B2/B4 elevation + change in evaluation to E3, and is determined by:

Tan (7°) = (Change in elevation / plan distance)
= Change in elevation / 112m

Thus
Change in elevation
= 112m. Tan (7°)
= <u>13.75m</u>

The elevation of the coal seam at E3 = 1055m -13.75m = 1041.25m, while the actual coal No 2 seam elevation at E3 is 1055. That indicates a throw on the dolerite of:

Throw = Actual elevation – extrapolated elevation
= 1055 – 1041.25
= <u>13.75m.</u>

e. An accurate section along line XY

Using the work described in the next outcome (Outcome 2.1.3.), this portion of the multiple borehole example can also be executed.

back

CONNECTION 5

It is suggested that "LEARNING OUTCOME 2.3.1" on page 60 is studied prior to this outcome as it forms the basis of section drawing.

The section must be drawn along the XY line on Figure 49. Follow the steps in Outcome 2.1.3. to mark intersections of the surface, dolerite and coal seams with the XY line. The outcrop of the dolerite previously determined has also been shown on Figure 49. The following is provided for ease of evaluating and practicing the section drawing in Figure 52:

- Lines that indicate coal elevations are drawn with 'dash-dot' lines from boreholes B2, B4 and C1;
- Lines that indicate surface elevations are drawn in continuous 'thin' lines;
- Lines that indicate the dolerite outcrop were drawn in 'dash-dash' lines;
- Lines that indicate coal seam, dolerite intersections and surface elevations are drawn in continuous 'thick' line (the actual lines are very close together and have been drawn as a single line to simplify the sketch).

back

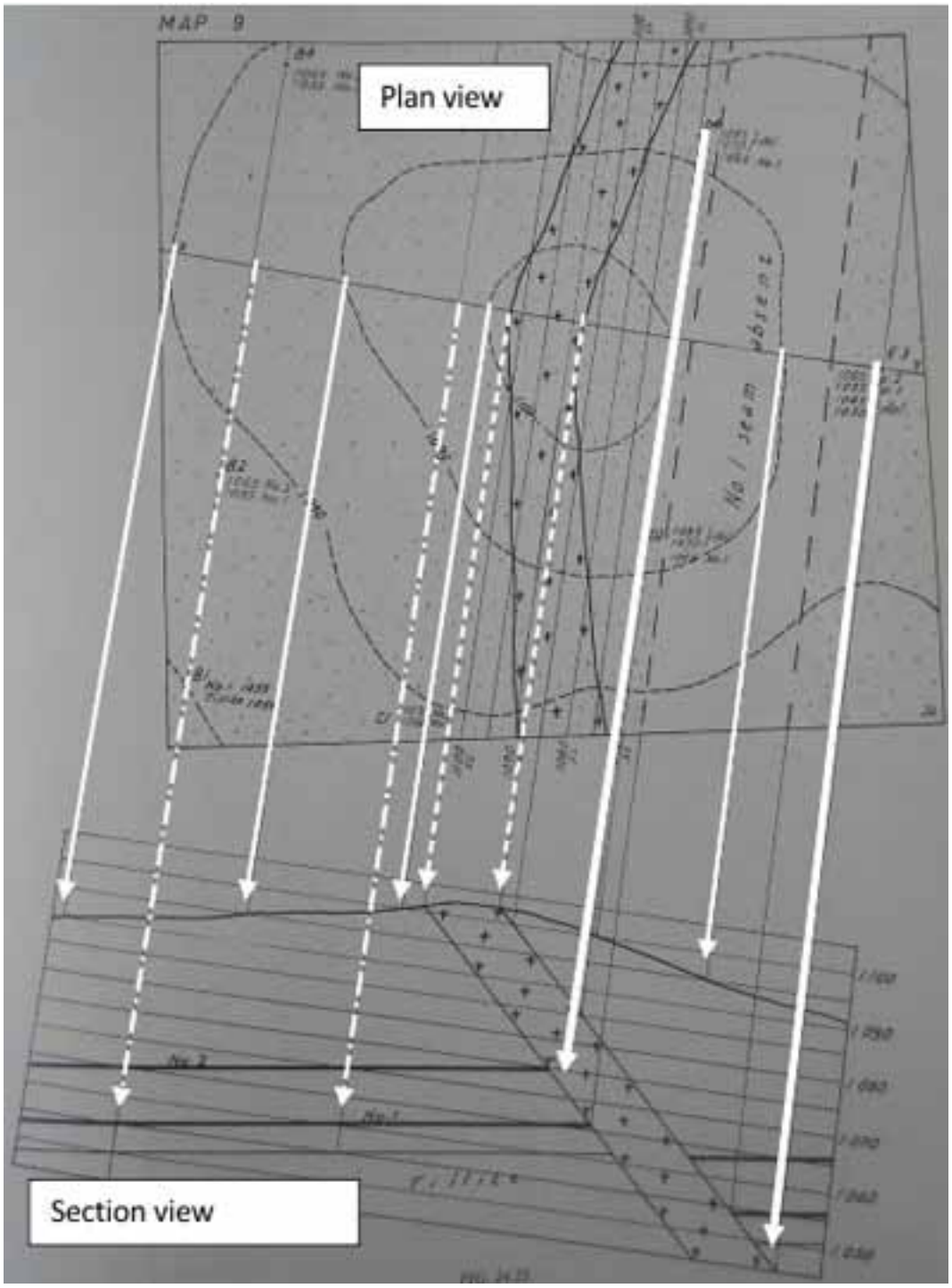


Figure 52: Multiple borehole solution (Lurie, 1987)

REFERENCE

- Page 190, 212 Lurie, J. 1987. Geology.

LEARNING OUTCOME 2.3.2

CONSTRUCT AND APPLY GEOLOGICAL SECTIONS AND PROJECTIONS FROM PLANS

Drawing sections is essential when a better idea is required of the possible effect that stoping might have on development, or geology might have on stability. The process below spells out the steps in drawing good sections.

back

EXAMPLE 1

Drawing a section using a mine plan, geology and peg elevations.
This is illustrated in Figure 53.

Step 1: Choose the line on the plan along which the section must be drawn and draw in the line on the plan (…….) – called the section line. Name the line AA' (Note a 2nd line would be named BB', etc). Write the name of the section line on the plan.

Step 2: Place a piece of paper along the section line and plot the intersection of mining abutments, survey pegs and geological structures (on plan) with this line, on the paper. Name each intersection, e.g. Abutment 1 is A1, Fault 1 is F1, Peg number, Crosscut 1 is CC1 etc. Extend lines from the intersection point downwards on the paper (- - - - -)

Step 3: On the paper below the numbered intersections (± 5-10cm): Choose an elevation or Datum line for the section, from which all depths will be measured (Datum = - 1490m).

Step 4: Plot the known elevation points (peg elevations) along the dotted lines drawn from the peg positions (- - - -) by measuring the elevation from the selected Datum line.

Step 5: Connect the elevation points by drawing lines through these points (_____) and then draw in the mined-out areas (▭) between the abutment intersection lines (- - -).

Step 6: Draw in the fault by drawing in a line (_____) between the intersection points of the abutments with the fault (F1 and F2) intersection lines (- - -).

Step 7: Draw in the crosscut at its correct elevation.

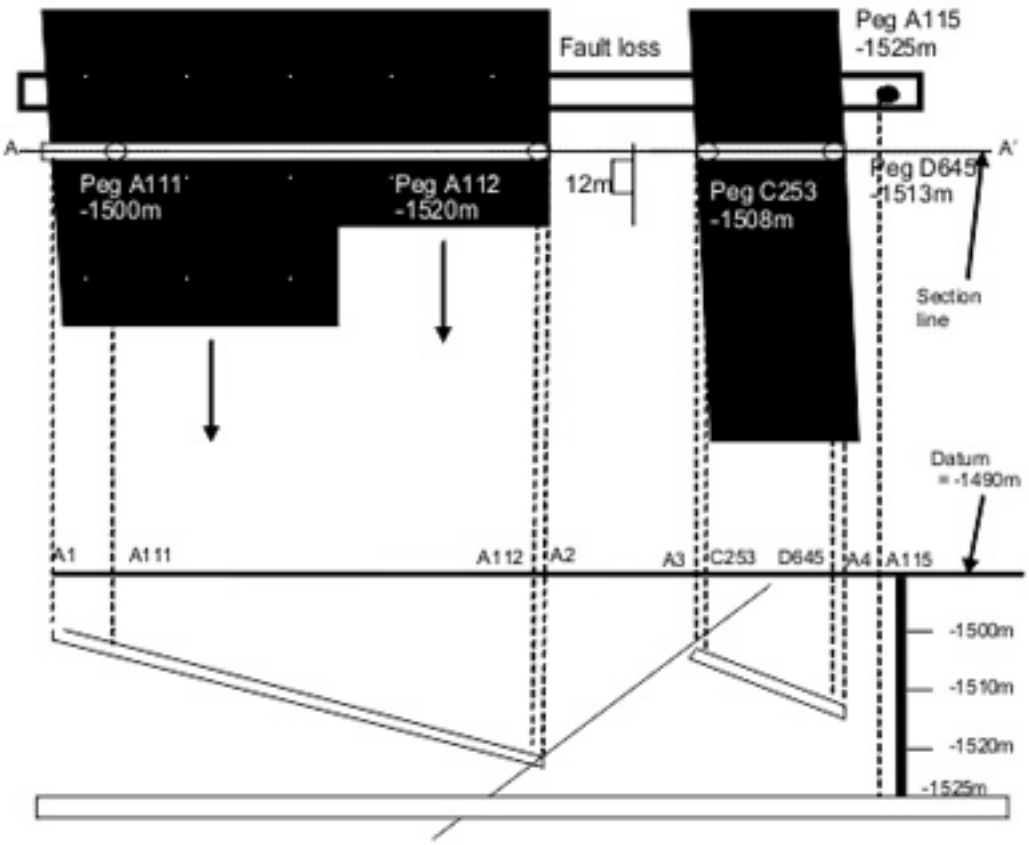


Figure 53: Section drawing from plan views (black areas represent mined out panels on plan)

back

EXAMPLE 2

Draw a section from a 1:1000 mine plan

See Figure 54 and Figure 55

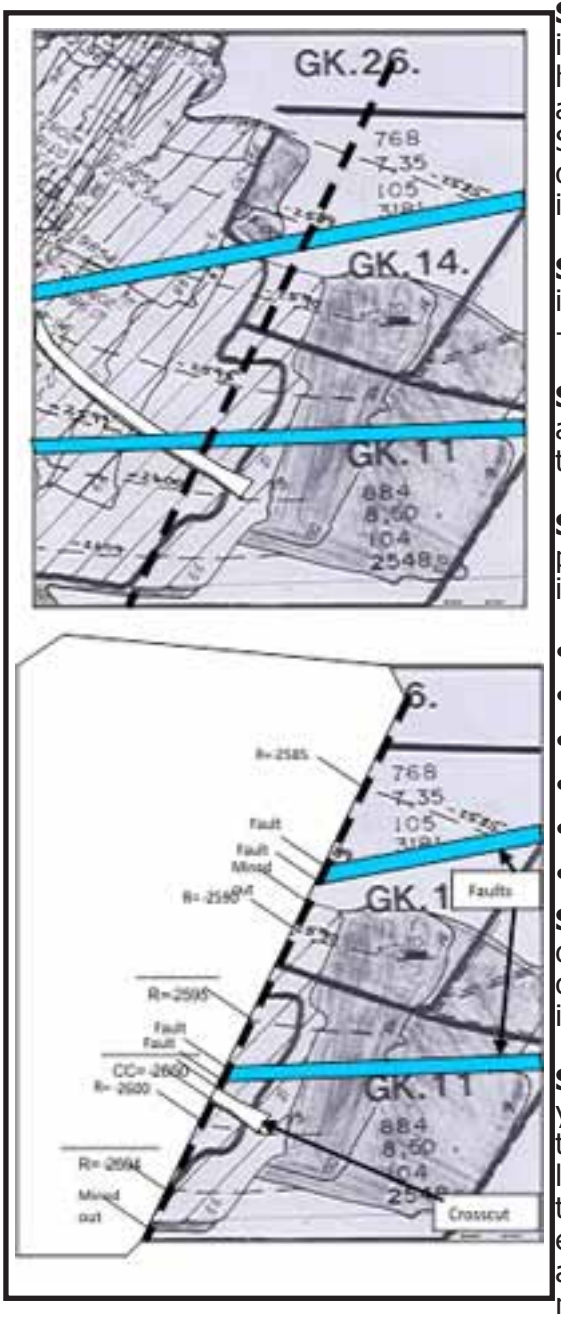


Figure 54: Mine plan

Step 1: Decide where the section is to be drawn by deciding what hazard is to be investigated (example: middling reef to crosscut). Sections should preferably be drawn perpendicular to the hazard in question.

Step 2: Draw a line on the plan indicating the line of the section (- - - - -).

Step 3: Take a clean paper sheet and line up the paper edge up along the section line drawn on the plan.

Step 4: Make a pencil mark on the paper edge of the prints of all the intersection of the following issues:

- Reef elevations
- Mined out areas
- Solid areas
- Geological structures
- Tunnels
- Stope face positions

Step 5: Read from the plan the depth of each of the marks made on the edge of the paper and write it down next to the mark.

Step 6: Decide on a "datum" for your section. That is the depth of the top edge of the paper or of a line called the datum line. Take the correct scale (mine plans are either 1:200, 1:1000 or 1:2500) and scale the depths of each of the marks on the paper.

Step 7: Now connect the necessary points for the mined and unmined reef areas and draw in geological structures and tunnels.

The section is now available to evaluate the layout.

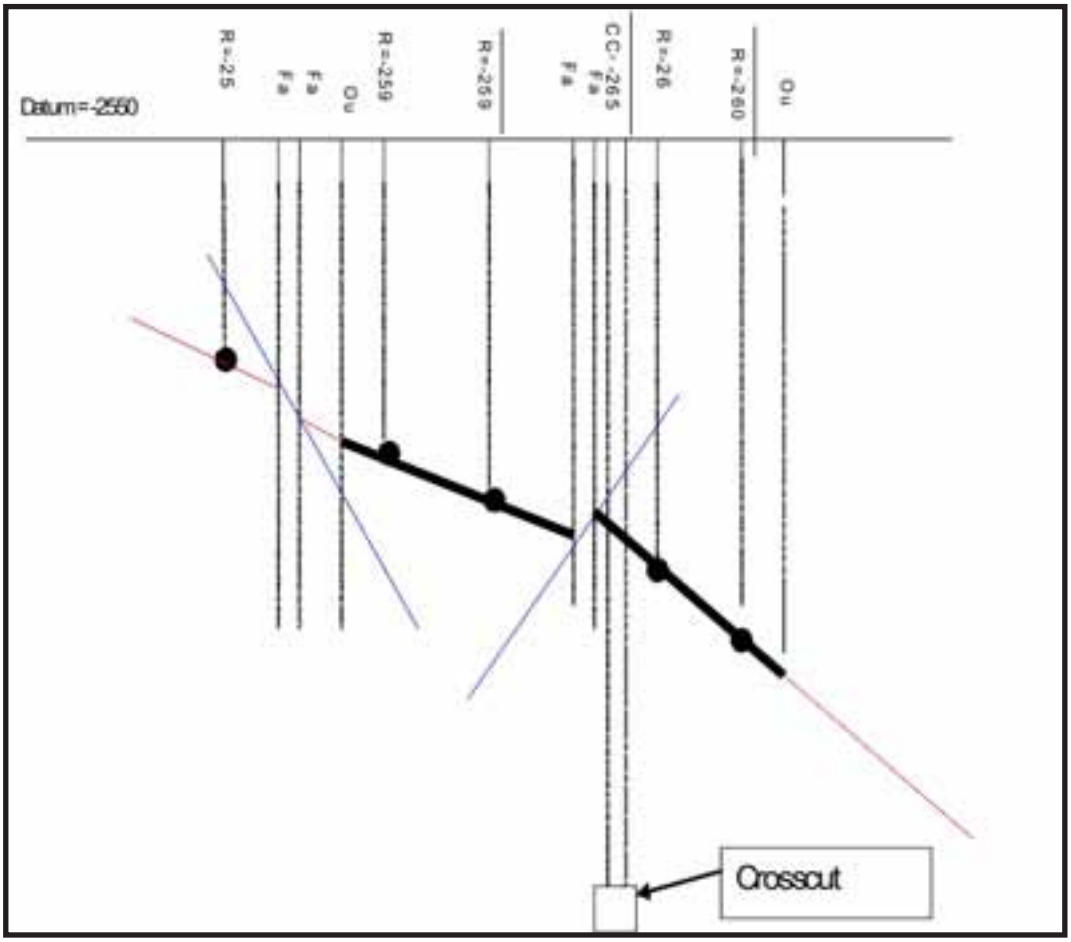


Figure 55: Final section

LEARNING OUTCOME 2.3.3

DEFINE, DESCRIBE AND EXPLAIN THE SIGNIFICANCE OF THE FOLLOWING TERMS IN TERMS OF MINING OPERATIONS:

- Bedding, Cleavage
- Slips, Joints
- Faults

Bedding

Bedding is associated with sedimentary processes (e.g. mud seam in sandstone), and bedding planes that separate bedding are very often prominent weaknesses. Such features are often very persistent and can be traced over hundreds of meters. Normal sedimentary bedding often constitutes an important discontinuity set within the rock mass. Figure 56 shows some examples. Other forms of depositional structures that fall within this category include primary igneous flow banding.

back

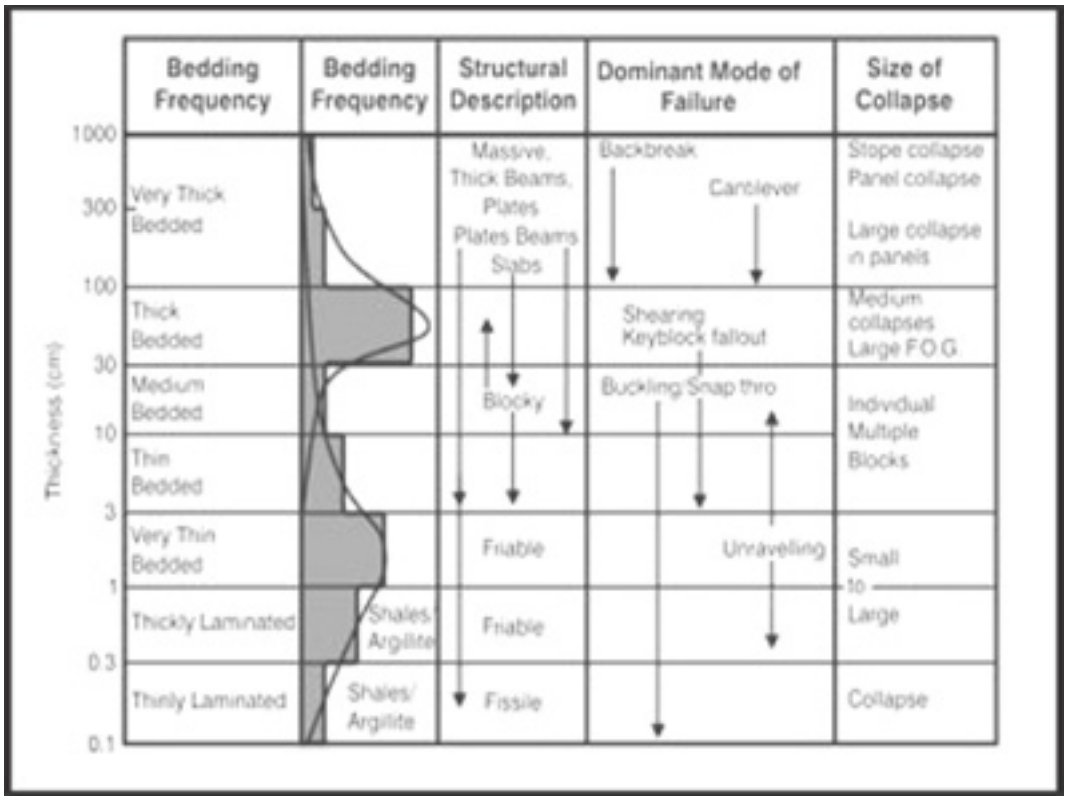


Figure 56: Bedding (Ryder and Jager, 2002)

Cleavage

Cleavage is a result of rock splitting along regular planes that are not necessarily parallel to bedding, e.g. slate cleavage, see Figure 57. Cleavage is normally parallel or sub-parallel to the axial surface of folds. It is defined by the quality of the break (perfect, good, fair and poor).

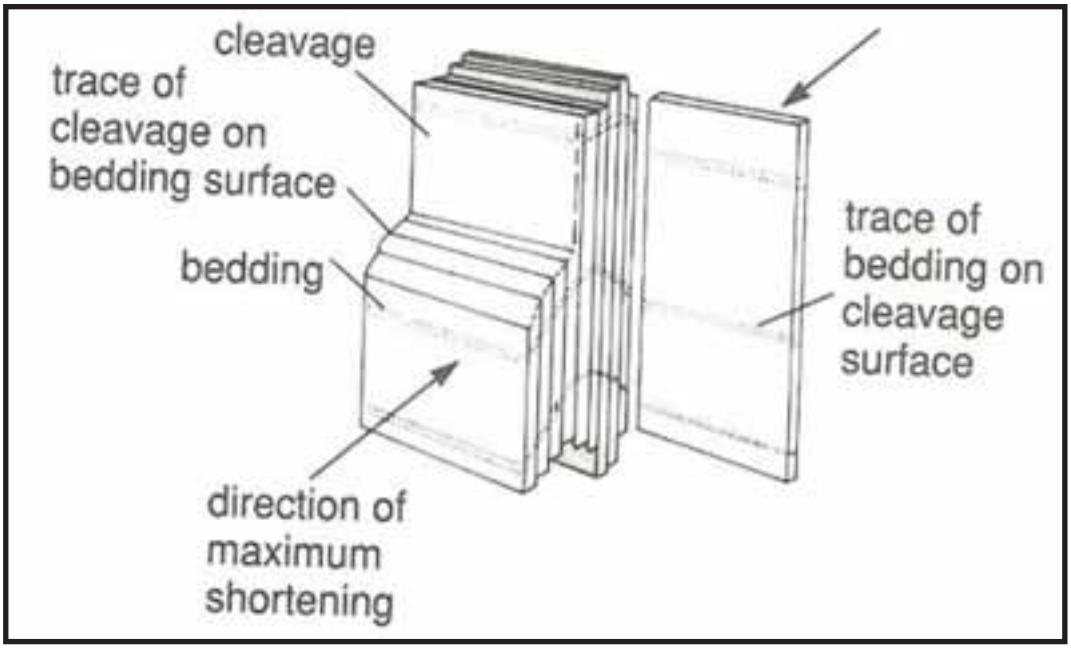


Figure 57: Cleavage that forms at right angles to direction of maximum compression (Ramsay and Huber, 1987)

Slips

Slips refer to movement along geological feature planes where slip direction can often be indicated by striation marks on the plane. The slip can be:

back

- dip-slip (e.g. in reverse and normal faults) (Figure 58);
- strike-slip (e.g. in sinistral and dextral faults) (Figure 58);
- oblique-slip (which is a combination of both dip-slip and strike-slip)

In the mining environment, the term slip is often used in 'informal' ways to refer to displacement or dislodging of rock, the presence of 'faults' or 'joints'.

Joints

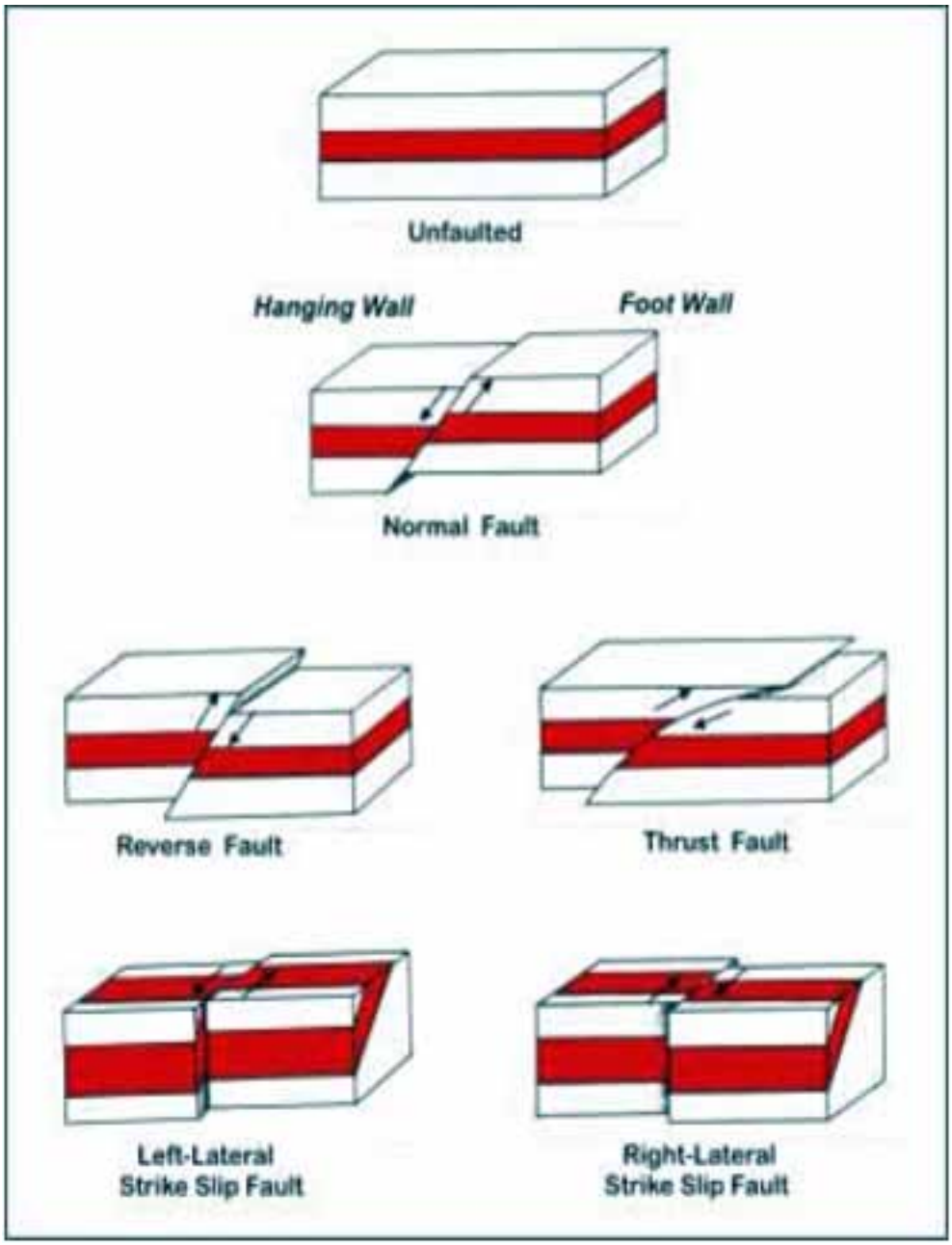
Joints are natural discontinuities, generally not formed parallel to lithological variations, and that show no signs of shear displacement, see Figure 58. Joints can be open, filled or healed. Joints formed during the igneous process are a result of the cooling of the lava, when cooling cracks (joints) are formed. Joints can also be created through other processes, such as bending of the rock mass. In essence, a joint is a weak plane, cutting through the rock mass (which could create blocky conditions), but not displacing rock layers. When vertical dipping joints intersect, low angle dipping joints (typically associated with ramp structures), wedge creation and failure are common.

Figure 58: Joints

Faults

Faults refer to discontinuities across which there has been substantial movement or displacement, see Figure 59. This displacement can be demonstrated by the relative displacement of marker horizons or by the presence of slickenside, or gouge on the fault plane. The magnitude and direction of movement may, or may not, be measurable.

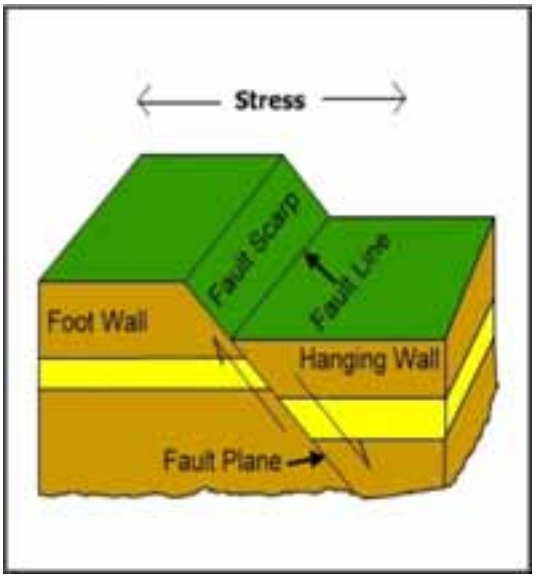
back


Unfaulted
Hanging Wall
Foot Wall
Normal Fault
Reverse Fault
Thrust Fault
Left-Lateral
Strike Slip Fault
Right-Lateral
Strike Slip Fault

back

Figure 59: (a) model showing different types of faults; (b) a fault plane underground

LEARNING OUTCOME 2.3.4



DESCRIBE AND EXPLAIN NORMAL FAULTING AND THE STRESS ENVIRONMENTS IN WHICH THEY ARE FORMED

Normal faults

Normal faults are caused by extensional stresses (pulling apart) and are often also called extensional faults. The hanging wall side of the structure moves downward relative to the footwall side, as indicative in Figure 60 below.

Figure 60: Normal faults

LEARNING OUTCOME 2.3.5

DESCRIBE AND EXPLAIN REVERSE FAULTING AND THE STRESS ENVIRONMENT IN WHICH THEY ARE FORMED

Reverse faults result due to compressional stresses resulting in the hanging wall side of the structure being displaced upwards relative to the footwall side, see Figure 61.

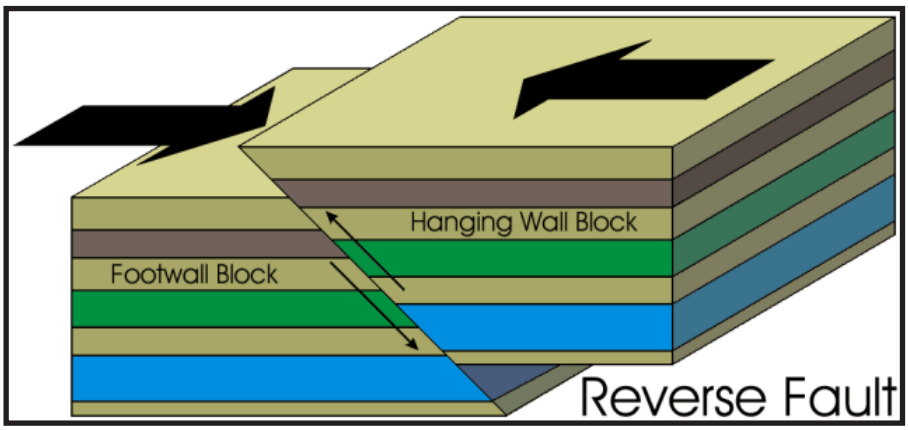


Figure 61: Reverse fault

LEARNING OUTCOME 2.3.6

DESCRIBE AND EXPLAIN THE FOLLOWING GEOLOGICAL FEATURES AND TERMS:

- Dykes, Sills,
- Folding
- Dip, Strike,
- Striation, Slicken side, Comminution, Mylonite,
- Erosion channels, Depositional channels, Wash-outs, Replacement zones
- Potholes, Domes
- Devolatisation, Calorific value, Ash content, Yield, Select height

Dykes and sills

Dykes and sills are formed when lava is forced into cracks (e.g. joints or fault planes), cools down and solidifies. If the magmatic intrusion is concordant (corresponding in direction with the planes of adjacent or underlying strata), it is known as a sill, see Figure 62 and if it is discordant (intrudes at a high angle to the bedding), it is called a dyke, see Figure 62.

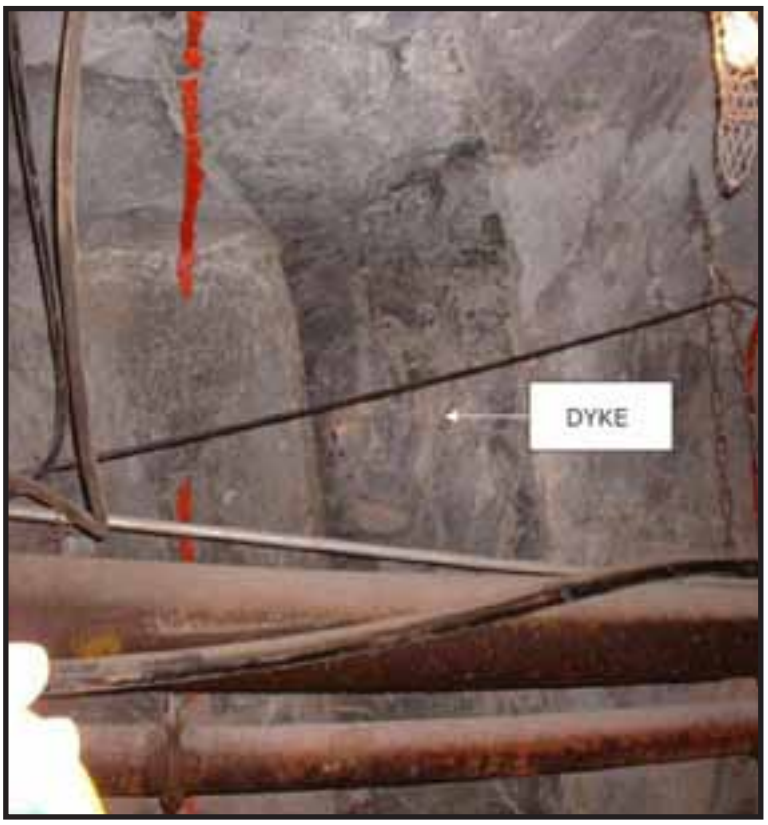


Figure 62: Dyke intrusion

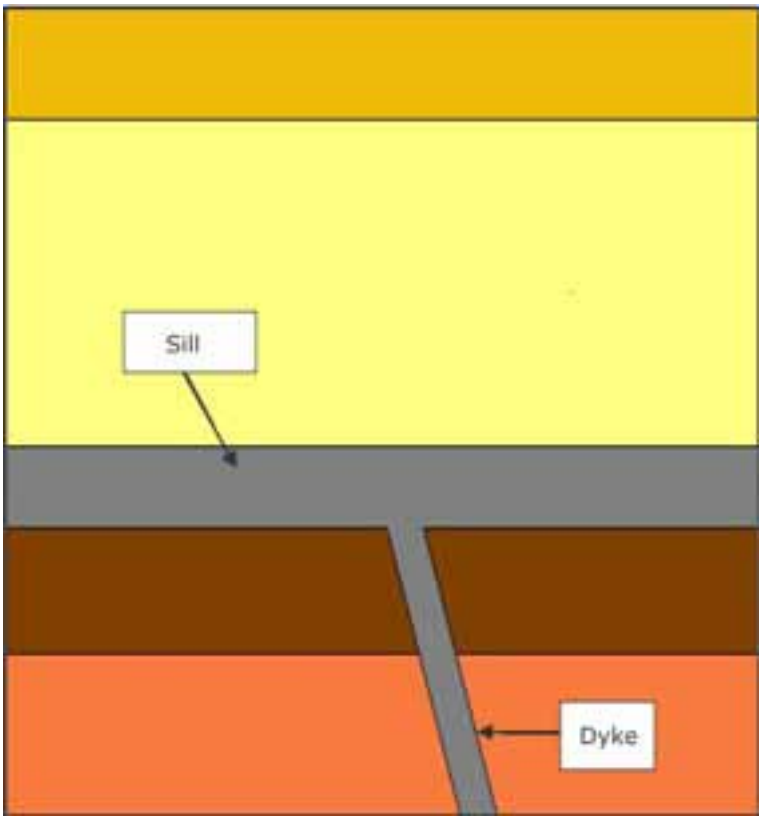


Figure 63: Relationship between a dyke and sill

Folding

Folding is referred to when rocks deform by plastic deformation and result in permanent curved structures, forming either syncline (fold limbs dips towards each other) or anticline (fold limbs dips away from each other) folds, see Figure 64.

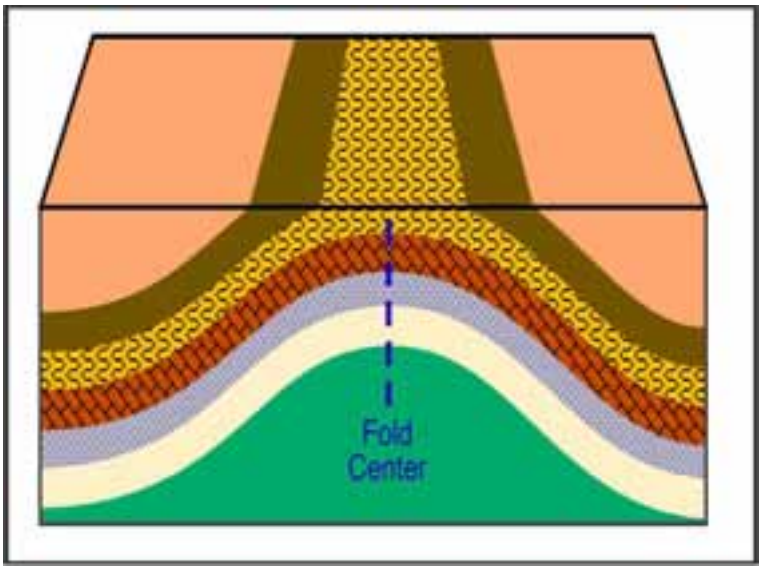


Figure 64: (a) Anticline: where two limbs dip away from each other,

Figure 64: (b) Anticline fold,

Figure 64: (c) Syncline fold

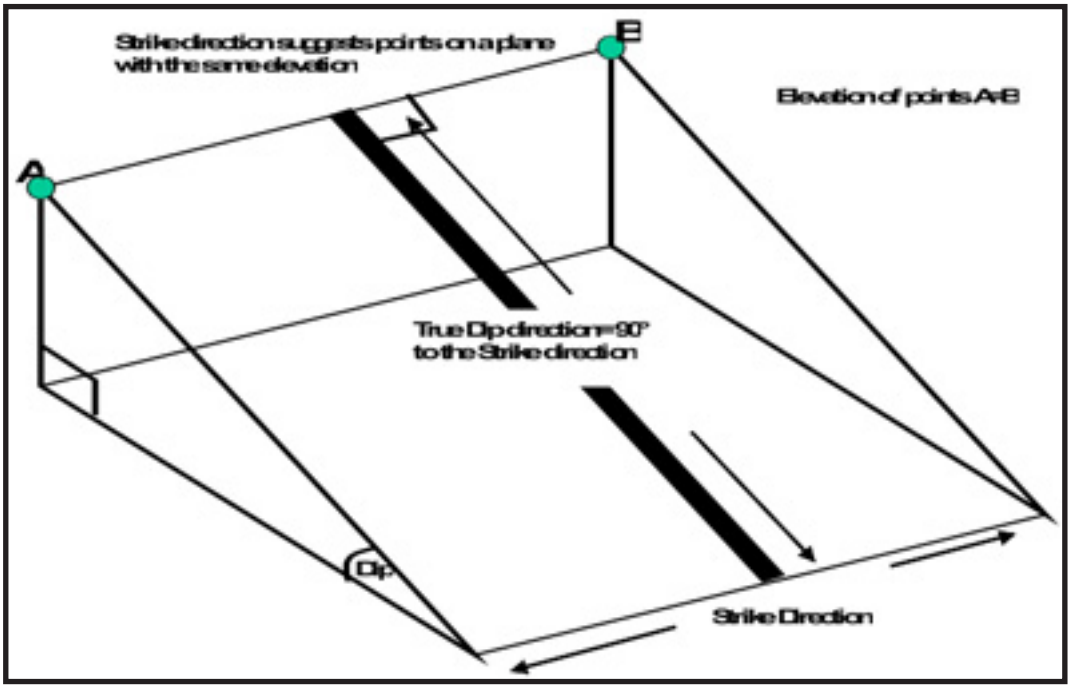


Figure 65: Strike and dip directions

Strike

Strike is the direction of a line formed by the intersection of a horizontal plane with an inclined plane such as a rock layer and suggests that all points along this intersection line will be at the same elevation. The strike is always perpendicular to the dip direction and is often referred to as the 'trend' of the fault or plane. The strike orientation can be represented as follows: N45E indicating a strike direction of 45° between north and east.

Dip

Dip is the measure of the maximum inclination of a plane from a horizontal surface. Dip is always measured perpendicular to the strike direction, varies from 0 to 90°, and can be represented by 60°SE, indicating a dip of 60° in the south-eastern direction. The general relationship between these two terms is shown in Figure 65 and 66

NOTE

Dip direction is also often used instead of the Strike orientation and is normally indicated by a combination of the dip direction and plane dip as follows: 050/60, indicating a dip of 60° in a direction 50° from north, measured in a clockwise direction. Strike orientation would then form a 90° angle with the dip direction.

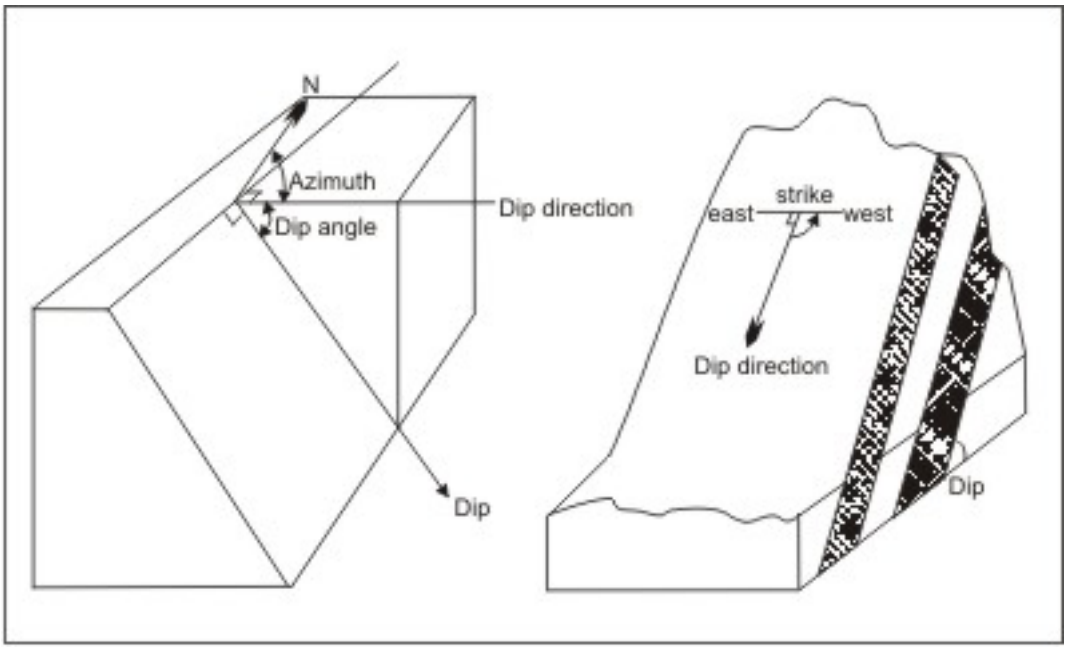


Figure 66: Definition of dip and dip directions (Ramsay and Huber, 1987)

Striations

Striations are linear farrows or grooves that result from movement on a fault or of a glacier on its base, see Figure 67. Striation marks are therefore good indicators of the direction of movement.

Figure 67: Striations

Slickenside

Slickenside is a surface that is smoothly polished as a result of frictional movement between two closely-spaced bodies of rock and as a result the surface may be striated. The slickenside is common along fault or shear zones.

back

Comminution
Comminution is the process by which solid materials are reduced in size by crushing, grinding and other processes. It occurs naturally during faulting in the upper part of the crust and is an important operation in mineral processing.

Mylonite
Mylonite is a chert-like rock without cleavage, but with a banded or streaky structure produced by extreme shearing of rocks that have been pulverised and rolled during intense dynamic metamorphism.

Erosional channels
Erosional channels indicate the erosion of a relatively soft surface by water and are often referred to where mineralised zones were eroded by water and no longer exist.

Depositional channels
Cross-sections allow us to see the shape of the different sedimentary units beneath the ground. Each sediment type that makes up these sedimentary units is deposited under particular environmental conditions, known as the "depositional environment". Figure 68 illustrates:

At the base of the cross-section below, there is limestone bedrock formed from the shells of tiny sea creatures. A wide, deep channel is evident in the surface of this limestone bedrock. A thick layer of sand and gravel has been deposited in this bedrock channel. Gravel is transported and deposited by water (e.g. rivers) and ice (glaciers). Sand is transported and deposited by wind (e.g. in sand dunes) and water (rivers and the sea). This can occur either within rivers (the "river channel") or on river banks during flood events. Two more, smaller, channels are evident in the surface of the sand and gravel. It is likely that these channels were formed by rivers or streams. An ancient soil ("palaeosol") has formed on top of the sand and gravel between these two channels. One of the river/stream channels has cut through the palaeosol.

back

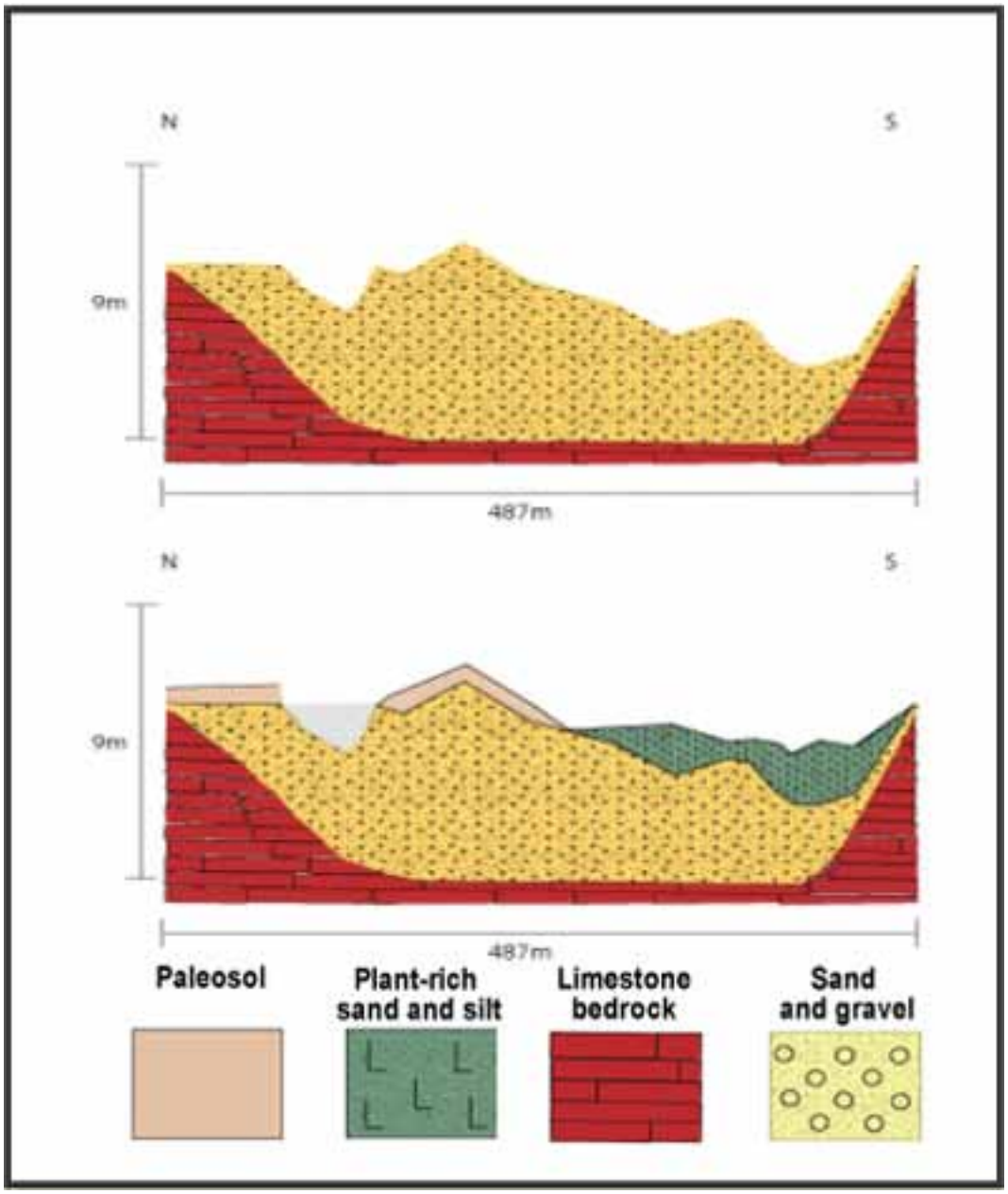


Figure 68: Depositional channels

http://www.seillevalley.com/InterpretingSediments.htm

Wash-outs

Wash-outs are the sudden erosion of soft soil or other support surfaces by water, usually occurring during heavy rain or a flash flood. A severe washout can result in a landslide or cause a dam break.

Replacement zones

Replacement zones refer to the dissolving or disintegration of one material followed by precipitation of a new material in its place.

Potholes

Potholes are formed when huge volumes of magma erode the floor, and successively the magma will cool down, filling up the eroded surface, see Figure 69. It may result in a fault surface at the interface of a pothole.

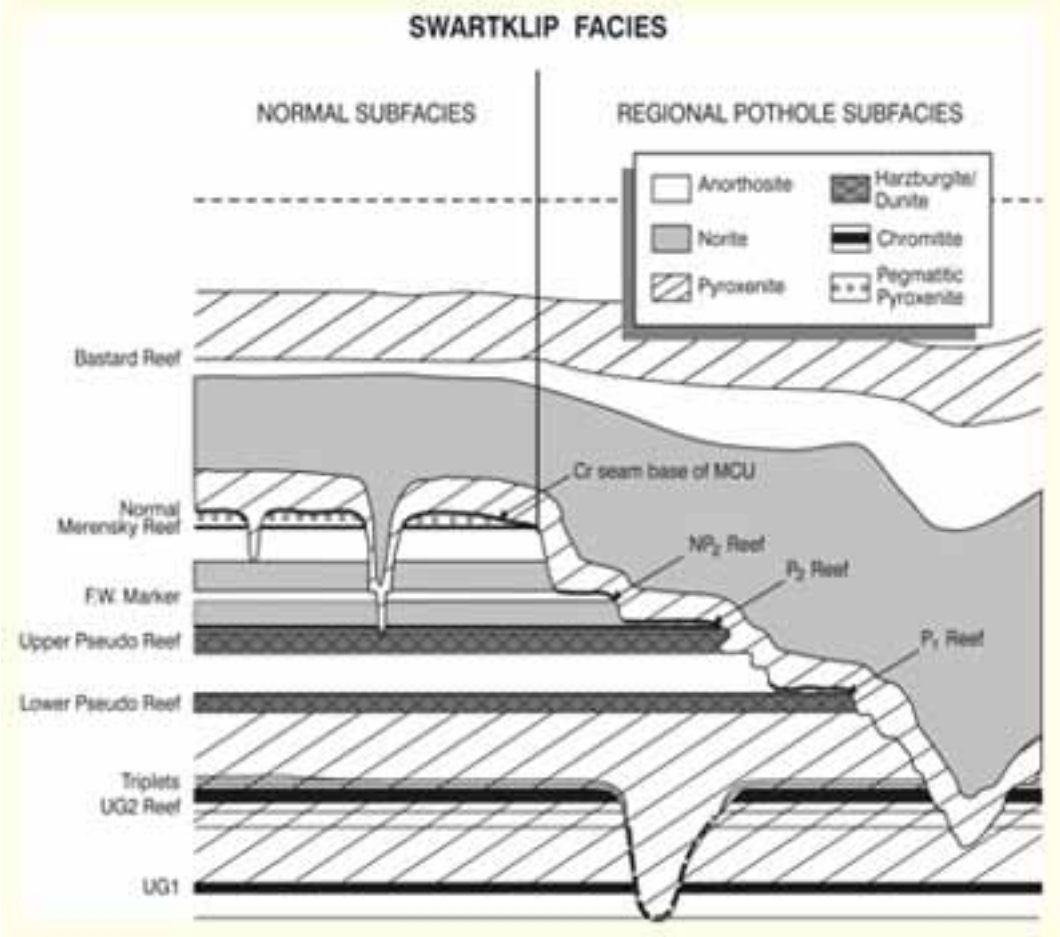


Figure 69: Section showing potholes

Domes

Domes are ramp faults with curved slip planes, see Figure 70. Geometric variations include duplex ramps contained within floor and roof layer-parallel faults and linking ramps connecting separate layer-parallel faults hidden in the footwall and hangingwall.

- Perrit , Ramp structures in bushveld

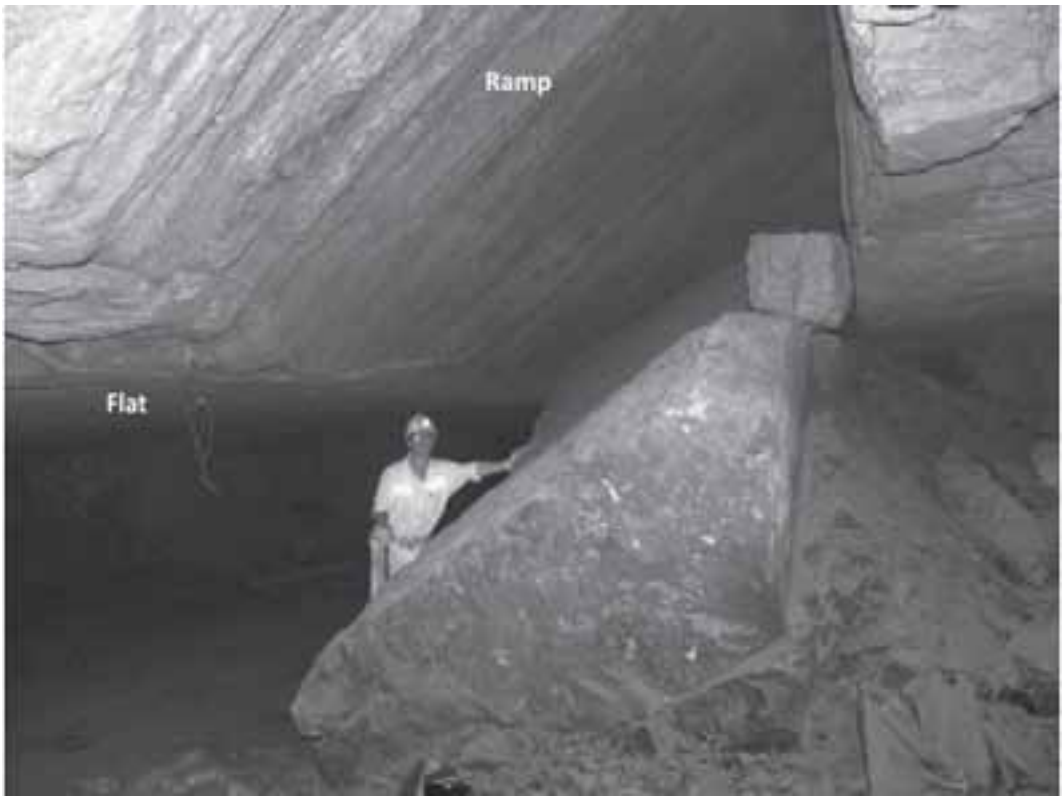


Figure 70: Domes

Devolatilisation
Devolatilisation is the release of gases and tars when a solid fuel is heated. The process in which coal is transformed at elevated temperatures to produce gases, tar and char is therefore also called devolatilisation. Gas formation can be related to the thermal decomposition of specific functional groups in the coal. Tar formation is more complicated and includes the rupture of weaker bridges in the coal macromolecule to release smaller fragments called metaplasts, repolymerisation of metaplast molecules, the transport of lighter molecules to the surface of the coal particles by diffusion in the pores of non-softening coals and liquid phase or bubble transport in softening coals and the transport of lighter molecules away from the surface of the coal particles by combined vaporisation and diffusion. Char is formed from the unreleased or recondensed fragments. Varying amounts of loosely bound "guest" molecules, usually associated with the extractable material, are also released during devolatilisation.

Calorific value
Calorific value is the heat that results from the combustion of a unit quantity of coal in a bomb calorimeter. It is a direct indication of a heat content of the coal, and therefore it represents the total quantity of the combined heat of carbon, hydrogen, nitrogen and sulphur present in coal. It is expressed in RSA as a number measured in megajoules per kilogram (MJ/kg). The combustion process generates water vapour and certain techniques may be used to recover the quantity of heat contained in this water vapour by condensing it. The Higher Calorific Value supposes that the water of combustion is entirely condensed and that the heat contained in the water vapour is recovered, Table 10 show some examples. The Lower Calorific Value (or Net Calorific Value (NCV)) supposes that the products of combustion contain the water vapour and that the heat in the water vapour is not recovered.

Fuel	Higher Calorific value MJ/kg
Anthracite	32.5 – 34.0
Bituminous coal	17.0 – 23.25
Carbon	34.1
Charcoal	29.6
Coal	15.0 – 27.0
Coke	28.0 – 31.0
Peat	13.8 – 20.5
Semi-anthracite	26.7 – 32.5

Table 10: Calorific Value

NOTE

Coal is a readily combustible rock containing more than 50 percent by weight of carbonaceous material formed from compaction and indurations of variously altered plant remains similar to those in peat. After a considerable amount of time, heat and burial pressure, it is metamorphosed from peat to lignite. Lignite is considered to be "immature" coal at this stage of development, because it is still somewhat light in colour and it remains soft.

- *Lignite increases in maturity by becoming darker and harder and is then classified as sub-bituminous coal. After a continuous process of burial and alteration, chemical and physical changes occur until the coal is classified as bituminous – dark and hard coal.*
- *Bituminous coal ignites easily and burns long with a relatively long flame. If improperly fired, bituminous coal is characterised by excess smoke and soot.*
- *Anthracite coal is the last classification – the ultimate maturation. Anthracite coal is very hard and shiny.*

Ash content

Ash content is the solid residue that remains after the complete combustion of coal. The ash content is always less than the original mineral matter content of coal due to the loss of carbon dioxide from carbonate minerals, sulphur from sulphide minerals and moisture from the constitution of clay minerals. Typical Ash Content in Coal:

- Anthracite Coal : 9.7 - 20.2 weight %
- Bituminous Coal : 3.3-11.7 weight %
- Lignite Coal : 4.2 weight %

Yield

Yield is the quantity of water, coal, gold or other resource that can be produced from a deposit. Yield can be looked at in two ways:

- how much of the required metal is there in one ton of ore (e.g. gram/ton), which allows the estimation of processing costs per unit weight of metal or other product (e.g. diamonds) to see if this is economically viable;
- as an estimate of the total quantity of product that the mine will produce in its estimated life.

REFERENCE

- Pages 1422-1429, S. Perritt and M.K.C. Roberts "Flexural-slip structures in the Bushveld Complex, South Africa?" Journal of Structural Geology, Volume 29, Issue 9, September 2007,

- Page 190, 212 Lurie, J. 1987. Geology.
- Chapter 1, Ryder J.A. and Jager A.J., A textbook on rock mechanics for tabular hard rock mines, SIMRAC, 2002
- Page 10, Jager A.J. and Ryder J.A., A handbook on rock engineering practice for tabular hard rock mines, SIMRAC, 1999.

LEARNING OUTCOME 2.4.1

DEFINE, DESCRIBE AND EXPLAIN THE SIGNIFICANCE OF THE FOLLOWING TERMS IN TERMS OF MINING OPERATIONS:

- Aquifer, Aquifuge, Water table
- Porosity, Permeability

NOTE

Geohydrology is a synonym for hydrogeology, a study of the flow of water through the earth without consideration of the effects of the geology, while hydrology is the study of the occurrence, distribution and chemistry of all waters of the earth

Aquifer
Aquifer is a rock or sediment in a formation, group of formations, or part of a formation that is saturated and sufficiently permeable to transmit economic quantities of water to wells and springs. Different types of aquifers include:

- Artesian: a confined aquifer that is under enough pressure to cause the water level in a drilled hole to rise past the level of the ground surface;
- Confined: an aquifer that is overlain by a confining bed. The confining bed has a significantly lower hydraulic conductivity than the aquifer;
- Perched: a region in the unsaturated zone where the soil may be locally saturated because it overlies a low-permeability unit;
- Semi-confined: an aquifer confined by a low-permeability layer that permits water to slowly flow through it;
- Unconfined: an aquifer in which there are no confining beds between the zone of saturation and the surface. There will be a water table in an unconfined aquifer.

Aquifuge
Aquifuge is an absolutely impermeable unit that will neither store nor transmit water. Related terms include aquitard, which is a bed of low permeability along an aquifer, and aquiclude, which is a solid, impermeable area.

Water table
The surface in an unconfined aquifer or confining bed at which the pore water pressure is atmospheric. It can be measured by installing shallow wells extending a few feet into the zone of saturation and then measuring the water level in those wells.

Porosity
Porosity the ratio of the volume of void spaces in a rock or sediment to the total volume of the rock or sediment and can include the following:

- effective porosity: the volume of the void spaces through which water or other fluids can travel in a rock or sediment divided by the total volume of the rock or sediment;

- primary porosity: the porosity that represents the original pore openings where a rock or sediment formed;
- secondary porosity: the porosity that has been caused by fractures or weathering in a rock or sediment after it has been formed.

Intrinsic permeability
Intrinsic permeability pertains to the relative ease with which a porous medium can transmit a liquid under a hydraulic or potential gradient. It is a property of the porous medium and is independent of the nature of the liquid or the potential field.

DEFINITION

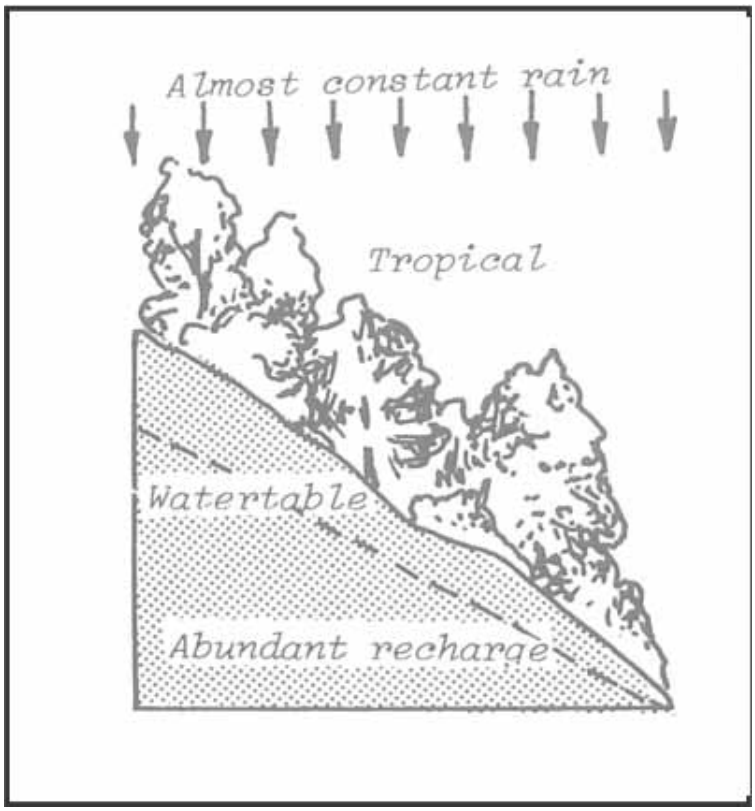
The following definitions are provided to assist with an increased understanding of hydrology:

***Total hydraulic** head is the sum of the elevation head, the pressure head, and the velocity head at a given point in an aquifer, equal to the total mechanical energy per unit weight.*

***Hydrologic cycle** is the circulation of water from the oceans through the atmosphere to the land and ultimately back to the ocean, see Figure 70.*

***Viscosity** is the property of a fluid describing its resistance to flow. Units of viscosity are Newton-seconds per meter squared or Pascal-seconds. Viscosity is also known as dynamic viscosity.*

***Transmissivity** is the rate at which water of a prevailing density and viscosity is transmitted through a unit width of an aquifer or confining bed under a unit hydraulic gradient. It is a function of properties of the liquid, the porous media and the thickness of the porous media*



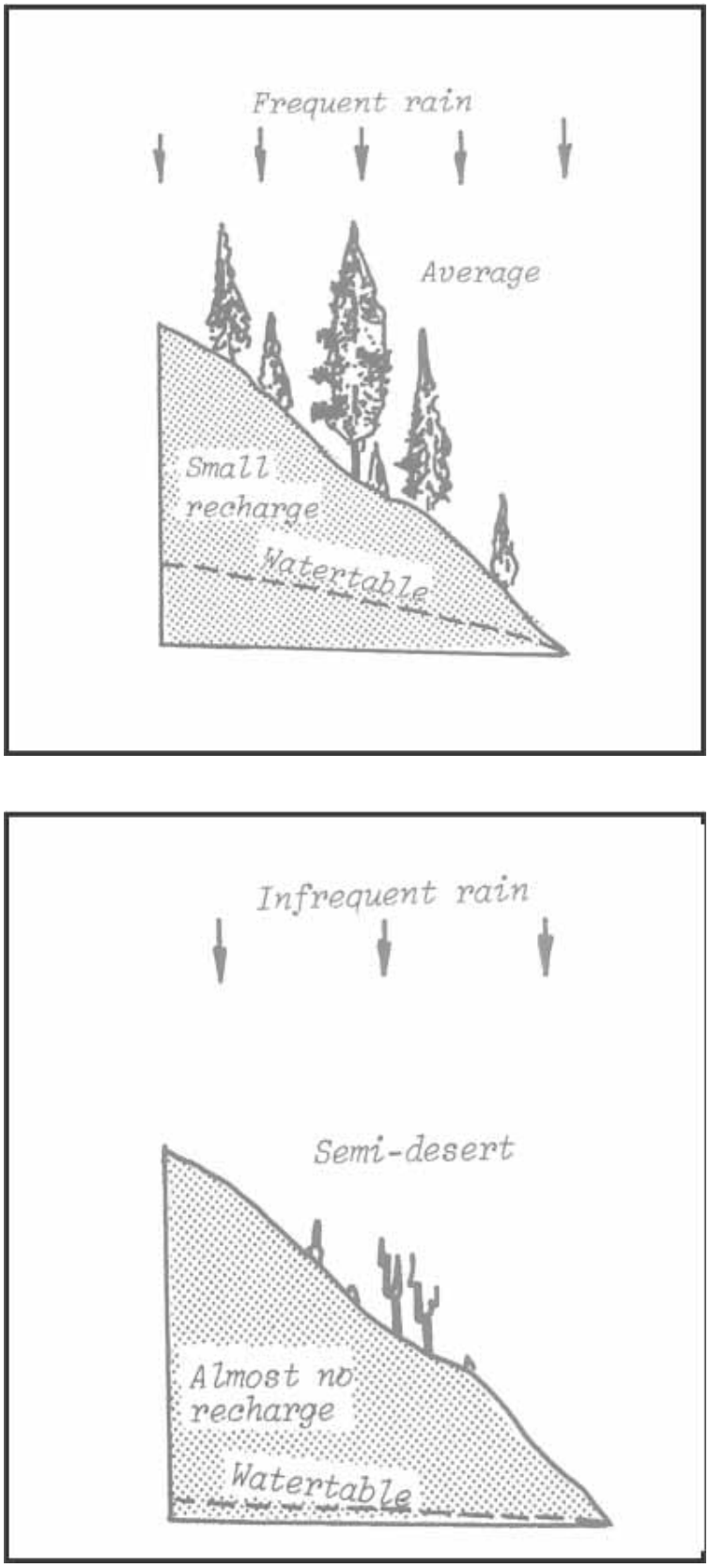


Figure 70: Water table and hydrologic cycle

Reference for more definitions regarding hydrology

- pg 127-147, Hoek and Bray, Rock Slope Engineering, IMM, 2005.
- Page 160, Lurie J, South African geology, 1987.

LEARNING OUTCOME 2.4.2

CONNECTION 6

Refer to outcomes in PAPER 1: "LEARNING OUTCOME 4.1.16" on page 124

"LEARNING OUTCOME 4.2.4" on page 133
and

"Learning Outcome 6.1.6" on page 198

DESCRIBE AND EXPLAIN THE EFFECT OF PORE WATER PRESSURE ON ROCK MASS STRENGTH.

LEARNING OUTCOME 2.4.3

CONNECTION 7

Refer to outcomes in PAPER 1: "LEARNING OUTCOME 4.1.16" on page 124

"LEARNING OUTCOME 4.2.4" on page 133

and

"Learning Outcome 6.1.6" on page 198

DESCRIBE AND EXPLAIN THE EFFECT OF PORE WATER PRESSURE ON THE FRICTIONAL STRENGTH OF JOINTS.

CHAPTER

3

ROCK AND ROCK MASS BEHAVIOUR

INTRODUCTION

Chapter 3 deals with rock and rock mass behaviour. The collection of rock type and rock mass-related data and the analysis of the data into rock mass quality groups allows the rock engineer to develop a good understanding of the environment for which he is designing and makes available to him a large number of empirical methods to estimate stability and support requirements.

Knowledge and understanding of mine seismicity are critical to rock engineers across the mining spectrum as the monitoring of these events requires understanding of the conditions under which they are created, but provides trends that allow some estimation of risk.

To ensure that final designs are appropriate to the environment, optimal and safe, this chapter provides the rock engineer with a good basic understanding of the mining environment and how it behaves.

In this chapter, the following is addressed:

- Data collection
- Rock mass classification
- Mine seismicity

LEARNING OUTCOMES

3.1. ROCKMASS CLASSIFICATION

3.1.1 DATA COLLECTION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

3.1.1.1 Define and describe the standard terminology for core logging.

3.1.1.2 Describe, explain and evaluate the problems associated with characterising weathered rock

3.1.1.3 Describe, explain and evaluate the implications and impact of the orientation of the survey line on the interpretation of recorded data

3.1.1.4 Describe, explain and interpret data presented in the form of stereonets

3.1.1.5 Identify joint trends, failure mechanisms, and other features presented in the form of stereonets

3.1.1.6 Describe, explain and apply core logging techniques

3.1.1.7 Describe, explain and apply scan line mapping techniques

3.1.1.8 Describe, explain and apply stereonet analyses

3.1.1.9 Describe, explain and apply stereo pair analyses.

back

3.1.2 ROCKMASS CLASSIFICATION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

3.1.2.1 Describe and explain how rock quality designation (RQD) may be determined from borehole core

3.1.2.2 Describe and explain how rock quality designation (RQD) may be determined from the in-situ rockmass

3.1.2.3 Describe and explain the formulation and components of Barton's Q system of rockmass classification

3.1.2.4 Describe and explain the formulation and components of Bieniawski's RMR system of rockmass classification

3.1.2.5 Describe and explain the formulation and components of Laubscher's MRMR system of rockmass classification

3.1.2.6 Compare and contrast these three rockmass classification systems and their respective applications

3.1.2.7 Describe and explain what modifications are necessary to apply rockmass classification systems to local conditions

3.1.2.8 Apply Barton's Q system to classify a rockmass

3.1.2.9 Apply Bieniawski's RMR system to classify a rockmass

3.1.2.10 Apply Laubscher's MRMR system to classify a rockmass

3.1.2.11 Describe, discuss and apply rockmass classification techniques for the selection of mining methods

3.1.2.12 Apply rockmass classification results to determine the stability of unsupported spans

3.1.2.13 Apply rockmass classification results to determine the stability of unsupported rock slopes

3.1.2.14 Apply rockmass classification results to determine support requirements for various situations

3.1.2.15 Determine rockmass 'm' and 's' parameters for the Hoek and Brown criterion based upon rockmass classification results

3.1.2.16 Determine rockmass deformability from joint stiffness and rockmass classification results.

3.2. MINE SEISMICITY

3.2.1 PRINCIPLES OF SEISMICITY

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

3.2.1.1 Identify, describe and explain the conditions required for a rockburst to occur

3.2.1.2 Identify, describe and explain the processes that give rise to rupture in rock

3.2.1.3 Identify, describe and explain the rupture processes in solid rock

3.2.1.4 Identify, describe and explain the rupture processes along a pre-existing fault plane

3.2.1.5 Describe and explain the phenomena of foreshocks and aftershocks

3.2.1.6 Describe and explain the phenomenon of seismic body waves

3.2.1.7 Describe, explain and distinguish between different types of seismic body waves

3.2.1.8 3Describe and explain the phenomenon of seismic surface waves

back

back

3.2.1.9 Describe and explain how seismic surface waves are generated

3.2.1.10 Describe and explain how seismic surface waves are related to seismic body waves

3.2.1.11 Describe and explain the differences between seismic body waves and seismic surface waves

3.2.1.12 Identify and describe the factors affecting the velocity of propagation of seismic waves

3.2.1.13 Describe and explain how the rockmass environment affects the velocity of P-waves and S-waves

3.2.1.14 State typical velocities of P-waves and S-waves in hard rock and soft rock environments

3.2.1.15 Describe and explain how this information may be used in locating the sources of seismic events

3.2.1.16 Describe and explain how the energy of a seismic body wave is attenuated as it propagates through elastic rock

3.2.1.17 Describe and explain how the energy of a seismic body wave is attenuated as it propagates through non-elastic rock

3.2.1.18 Describe and explain how seismic waves and the energy of seismic waves cause damage in excavations

3.2.1.19 Describe and explain the phenomena of reflection and refraction of seismic body waves

3.2.1.20 Describe and explain how seismic wave reflection and refraction cause damage in excavations

3.2.1.21 Distinguish between seismic source mechanism and seismic damage mechanism

3.2.1.22 Identify, describe and explain the common methods used to locate seismic events

3.2.1.23 Describe the simplifying assumptions commonly used in determining seismic event location

3.2.1.24 Explain the difference between the terms hypocentre and epicentre used in describing the location of seismic events

3.2.1.25 Describe, explain and contrast the following primary source parameters in terms of their physical meaning and the data required to determine them:

- Seismic energy
- Seismic moment
- Richter magnitude
- Local magnitude
- Source dimension
- Corner frequency
- Energy index

3.2.1.26 Identify, describe and explain the basic steps that are involved in carrying out a rockburst site investigation.

LEARNING OUTCOME 3.1.1

DEFINE AND DESCRIBE THE STANDARD TERMINOLOGY FOR CORE LOGGING

Core logging is a simple process, with the key aspect being identification and recording. It is therefore important to note 'what you see' when inspecting the core and to describe it well. To be able to do this, strict processes are normally followed to ensure that nothing is missed during the logging. In describing your observations, you should use uniform terms so that users of the data understand what you saw and recorded.

back

back

Always describe what you see in as simple terms as possible. Draw diagrams if necessary and take pictures using a scale for size and dimension. Always be thorough in your methods and meticulous in your descriptions. This information provides the backbone for future designs.
The following features will enable a better understanding of the terminology usually involved in describing the material during geotechnical core logging:

- Rock appearance
- Colour
- Broad rock type description
- Weaknesses
- Fractures
- Joints
- Frequency
- Description of fracture and joint physical appearance
- Signs of the presence of water

Rock appearance

Rock appearance is the easiest criterion in core logging as you describe what you see in the core. These features may include depth of the core drilled, the colour, its grain size, it texture, the presence of faults, etc. It is basically an overview of the core tray prior to an in-depth log being done.

By looking at the shape of the grains, we can tell what its original environment was, in other words, where it came from. Some information to assist you in Grain shape identification is described below.

Grain shape is important because:

- it is an indication of the textural maturity and
- different shapes of pebbles are considered to be typical of different environments of deposition – Proximal or distal, sub-aerial, alluvial or marine. The surface texture of the larger cobbles and boulders is also of interest.

The clast roundness and sphericity must be described by visual comparison with Figure 1. Note that ROUNDNESS refers to the sharpness of the corners and edges of the grain. SPHERICITY measures the degree to which a particle approaches a spherical shape.

back

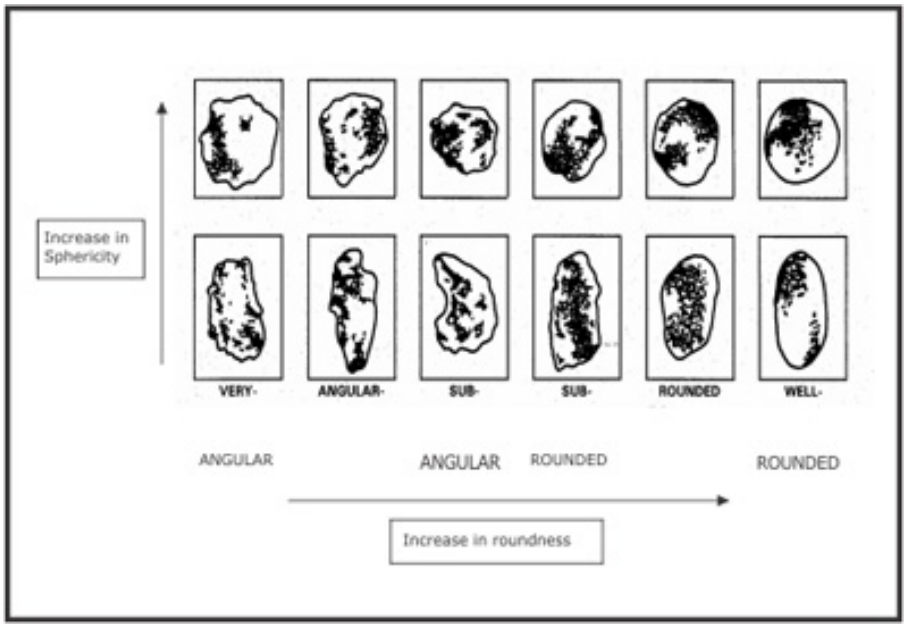


Figure 1: Simple predicted scale: Photographic images of grains used for visual determination of grain roundness (after Powers 1953)

Rock colour

Rock colour is difficult to describe. In fact, it is even more difficult to see underground in artificial and poor light conditions. The correct technique is to compare the rock to the colours on the Geological Society Rock Colour Chart, shown in Figure 2 (e.g. the Mansell Charts). Always compare a fresh wet surface with the chart.

When photographs are taken of exposures or core, a colour chart should be included in the frame to enable the colours to be correctly reproduced by the processing laboratory. If the colour chart is not available, be as precise as possible in your description of the colour. Do not use terms meaningful to yourself only. Avoid vague terms like "BLUEISH OR GREENISH". Use phrases like "SLIGHTLY BLUEISH LIGHT GREY OR SLIGHTLY GREENISH MEDIUM GREY" that someone else can understand.

Figure 2:Colour chart used in core logging (source: Corstor handout)

Rock type

Rock type description should be broadly identified by a few words e.g. orthoquartzite; shale, bedded non-magnetic; lava, amygdaloidal; intrusive; etc. and followed by a more detailed description that would include grain size, shape texture, colour.

This can fall into the three categories of rocks, i.e. igneous, sedimentary or metamorphic. Once having a grasp of this, it can be further broken down into their different types. A stratigraphic column is useful to use in this stage as most mines already have an understanding of the expected stratigraphic sequence they are going through. This will be useful to have with you when logging the core.

The correlation of the rock should also be indicated, either by the identification within the log of a sequence of sediments in which the specific

back

log interval is included (into formations or members) or specifically in the description itself if the interval being logged is a unit or significant intersection, e.g. Basal Reef, Karoo Dolerite, Green Bar, Purple Zone. Normally, the formation, group or other major unit to which the rocks belong will have been indicated elsewhere in the log and it is only members, beds and relatively thin rock sequences that will be identified. When there is a major change in the lithology (rock type), e.g. when the hole passes into a new formation or group or through a fault, it should be noted.

Weaknesses

Weaknesses are any planes that act as pathways for movement, e.g. joints, faults, cracks, fissures and bedding surfaces. The common weakness planes are outlined in paragraphs (e) and (f) below.

Fractures

Fractures are the way in which a mineral breaks, other than along its planes of cleavage. A fracture can be described as uneven, conchoidal or hackly.

- Conchoidal is a smooth curved fracture, which looks like a shell.
- Hackly is a rough or jagged fracture.

Cleavage

The physical property of some minerals to break along planes related to the definite crystal directions of the mineral. Cleavage is defined by the quality of the break (perfect, good, fair and poor).

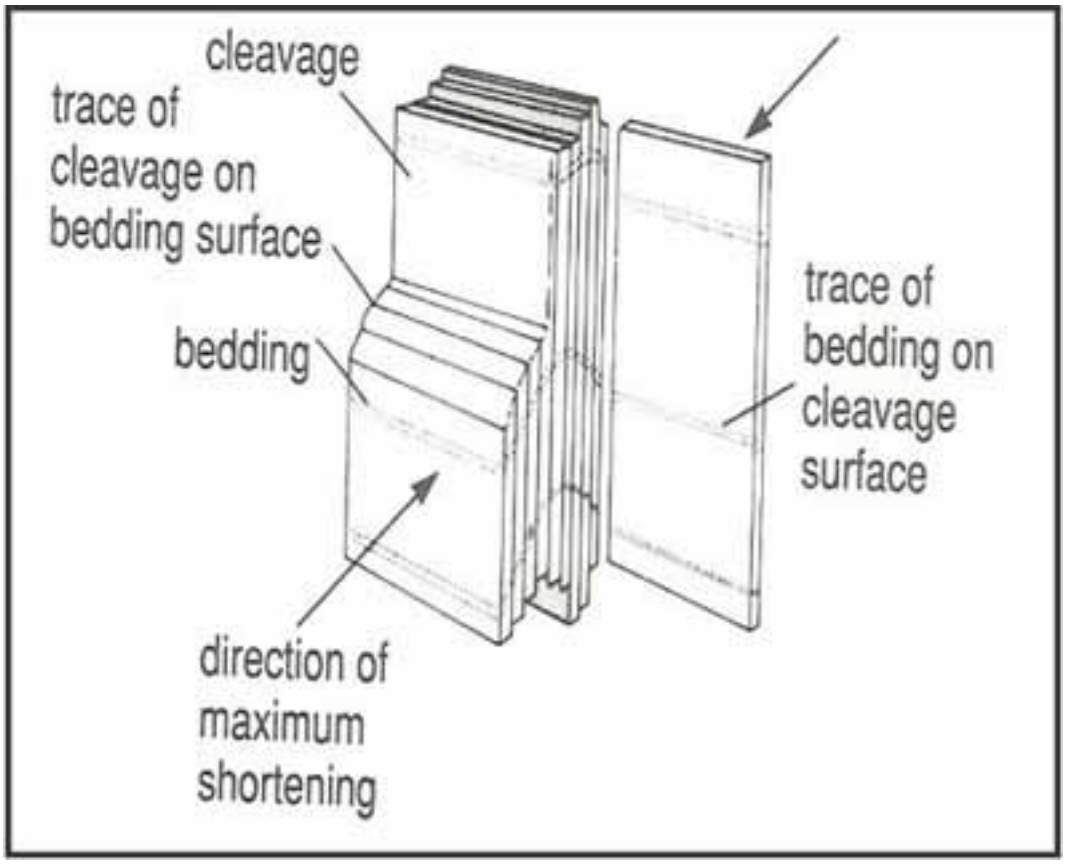


Figure 3: Cleavage that forms at right angles to direction of maximum compression (Ramsay and Huber, 1987)

Joints

A joint is a natural fracture having little or no displacement. Joints may be classified according to their mechanism of origin (tension, shear etc) or their geometric relationship to a specified major structure (cross joints, parallel joints etc). Any natural fractures that cannot be classified

back

back

according to any other nomenclature tend to be referred to loosely as jointing.

Frequency

Frequency is the number of times a geological structure recurs in the length of core that is examined, e.g. the number of joints in one metre of core might be equal to 5. As you develop your methodology, it will become easier to recognise the different parameters and notice their repetition in the core sequence.

Description of fracture and joint physical appearance

Faults and joints are often associated with shear zones of varying thickness, gouge infilling and slickensides. Faults can be classified as normal, reverse, thrust or strike – slip faults (Refer to paragraphs e and f above for more information) based on their origin, throw or slip direction, but can be further classified according to infilling material, surface roughness or even whether the fault surface is open or closed.

Signs of the presence of water

The influence of water on the stability of excavations in rocks or soils is of great importance. The presence and volume of water should be noted and described. When water is noted, either by calcite or whitish infilling in the core, it means that there is a possible plane of weakness in the rock and this should be noted for support design purposes. Water can assist instabilities, in that planes of weakness are washed clean of material that could increase friction against slip, while water can affect the competency of some rocks such as shale.
Usually, the presence of water is described as a rate (see rock mass classification system tables) in terms of litres/minute or m3/hour, but most often this information is not available to the logger, but only from the driller's logs. When logging, the logger should note dampness or wetness.

To allow description of the terms usually utilised during logging, the logging process will be described as part of the example below. By working through the example given below, a good understanding of the terms used will be created.

CONNECTION 8

Refer to outcomes "LEARNING OUTCOME 3.1.2.3" on page 145 and "LEARNING OUTCOME 3.1.2.4" on page 149

EXAMPLE 3

he process followed to log borehole core for the purpose of determining MRMR values for the core is described below and refers to the data input sheet (core logging sheet) shown in Figure 4 below. The terms discussed below include:

- Intervals
- Depth of core
- Core recovery
- Rock Quality Designation (RQD)
- Rock type
- Rock competence
- Weathering
- Hardness
- Joint orientations
- Fracture frequency
- Joint surface condition

Core is always logged with a final purpose in mind and that is to describe the rock mass at the hand of the data gathered. Many methods and systems exist to allow this description to get engineering importance, such as rock mass ratings, joint set analysis, joint property analyses, etc.

Intervals

Before geotechnical core logging can commence, the borehole should be divided into geotechnical intervals. Even if the intervals are changed later on, it is best to have a good idea of the geological and geotechnical aspects of the borehole. Dividing the borehole into geotechnical intervals is based on:

- the geology (change in rock type)
- the joint sets (total number of joints, their spacing and number of joint sets) and fracturing
- the joint condition
- the presence of veining, shearing fault/weakness zones
- the strength of the rock
- the weathering

Generally, if there is a potential interval that is less than 50cm in length, it should be added to the most geotechnically representative interval above or below it.

back

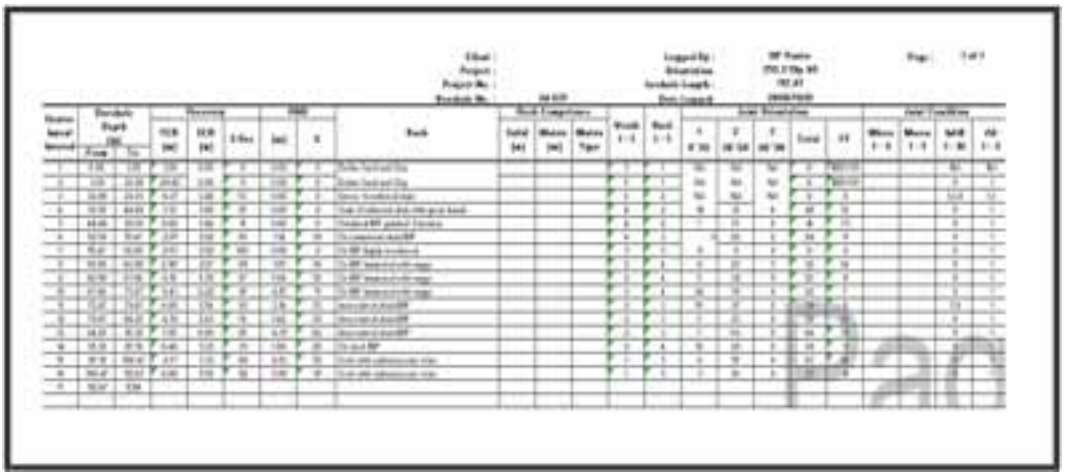


Figure 4: Core logging sheet

Depth From and Depth To

Start by measuring the depth at the top of the interval to the bottom of the interval. It is important to ensure that you cross check your measurements by:

- measuring forwards from the nearest core block at the top of the interval
- measuring backwards from the core block at the bottom of the interval
- ensuring that core loss has been accounted for.

This exercise takes a great deal of practice, but is much easier if the metre markings have been measured with a high level of accuracy, as you can then go by the metre markings rather than the core blocks to save time.

Total Core Recovery/Drilled Length (TCR)

The Total Core Recovery (TCR) is simply:

Drilled Length or Total Core Recovery (TCR)
= Depth To – Depth From Solid Core Recovery (SCR)

The Solid Core Recovery (SCR) is measured by estimating the intact core length if there is any broken material in the geotechnical interval. The SCR should be checked with the TCR as it should not be more. When measuring the SCR, it will become apparent how much core was lost. Accurate metre markings will also enable you to work this out.

The difference between TCR and SCR
The main difference between the TCR and the SCR is that the TCR is the total amount of core drilled, regardless of whether or not the material is there. The SCR is a measurement of what core is actually physically present.

Percentage Core Recovery (%CR)

The percentage Core Recovery (%CR) is simply:
%CR = (SCR / TCR) x 100

Rock Quality Designation as a percentage (RQD)

The Rock Quality Designation (RQD) is the sum of lengths of complete cylindrical core that are greater than 10cm, i.e. the smallest distance between fractures should be 10cm or greater. Only natural fractures should be considered in the measurement, not drilling, handling or induced

back

fractures. The RQD must be less than or equal to the SCR.
The RQD gives a general idea of the quality of rock, but is used mainly as a component of the RMR and Q rock mass classification systems. To determine RQD, rock lengths bounded by natural fractures such as joints or shears result in surfaces of separation and longer than 100mm are measured and the cumulative length determined (Figure 5).

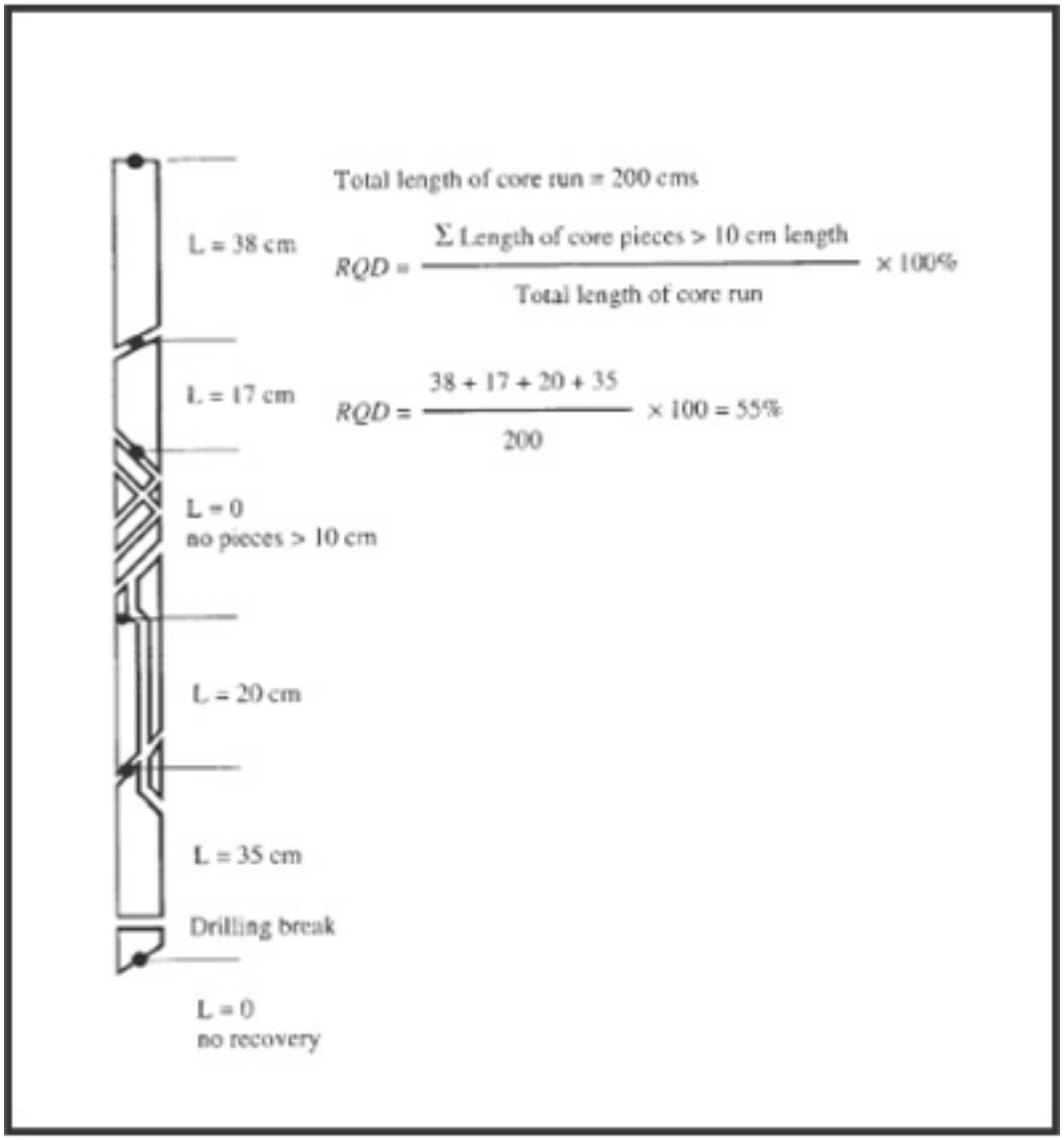


Figure 5: Procedure for the calculation of RQD (after Deere, 1989)

Rock Quality Designation Percentage (%RQD)
The Rock Quality Designation Percentage (%RQD) is calculated as follows:

%RQD = (The Sum of Core > 10cm) / TCR x100

Rock type
Either a specific rock type or a description of the rock type such as weathered, faulted, sheared, interbedded, intercalated etc can be included.

Rock competence – Solid
This is the length of the solid cylindrical core in the geotechnical interval. It is more accurate to measure the solid cylindrical core and subtract this from the SCR.
Rock competence – Matrix
This is the length of the matrix material in the geotechnical interval.
The rock matrix needs to be defined by one or more of the following terms:

back

back

- M1 Fault
- M2 Shears
- M3 Intense fracturing
- M4 Intense mineralisation
- M5 Deformable material

To add to the level of detail of the log, the matrix sections can be measured and recorded along with the matrix type.
It is important to note that the rock competence measurements should add up to the SCR – 'what is physically there,' not the TCR, which accounts for core loss.

Weathering

This should be estimated using one or more of the following terms.
Unweathered: No visible signs of alteration in the rock material, but discontinuity planes may be stained.
Slightly: Discontinuities are stained or discoloured and may contain a thin filling of altered material. Unweathered rock colour generally preserved. Discolouration may extend into the rock from the discontinuities.
Moderately: Slight discolouration extends from discontinuities for a distance greater than 20% of their spacing (i.e. generally greater part of the rock). Discontinuities may contain filling of altered material. The surface of the core is not friable (except in the case of poorly cemented sedimentary rocks) and the original fabric of the rock has been preserved. Partial opening of grain boundaries may be observed.
Highly: Discolouration extends throughout the core. The surface of the core is friable and usually pitted due to washing out of highly altered minerals by water. The original fabric of the rock has mainly been preserved, but separation of grains has occurred. Not easily indented with a knife, does not slake in water.
Completely: Residual soil. The core is totally discoloured though internally the rock fabric is partly preserved, but grains have completely separated. Easily indented with a knife, and slakes in water.

Hardness

This should be estimated using the one or more of the following terms.

- Very soft (1-34MPa)
- Soft (35-84MPa)
- Hard (85-104MPa)
- Very hard (145-165MPa)
- Extremely hard (165- +185MPa)

Joint orientation

If joint orientation is not critical to stability, or if only the alpha angle can be measured, joint orientations can be divided into three categories:

- 0-30 0
- 30-600
- 60-900

Each set should be totalled and added for the total number of joints within the geotechnical interval.

However, higher quality data is recorded when the alpha and beta angles are recorded and reported. This is also very appropriate for 'orientated core'. During core drilling, runs of core (commonly approximately 3 metres long) are extracted from the core barrel and placed in core boxes. The extraction process rotates the core randomly so that once the core

back

is laid out in core boxes, its original orientation is lost, although the orientation of the core axis is generally still known. Unorientated core (only alpha angle can be measured) provides information of rock mass quality, but cannot adequately assist to provide geotechnical analysis of discontinuity orientations (dip direction is unknown) and identification of dominant discontinuity sets.

Orientated core provides information on rock mass quality and sufficient information to allow analysis of discontinuity orientations. The drilling and logging process is more complex and requires attention to many facets not required for unorientated core. The need for the orientation data and the complexity of the drilling and logging processes make it essential that the orientation line is drawn accurately on the core. Failure to extend this line accurately could render efforts in the drilling and logging processes useless.

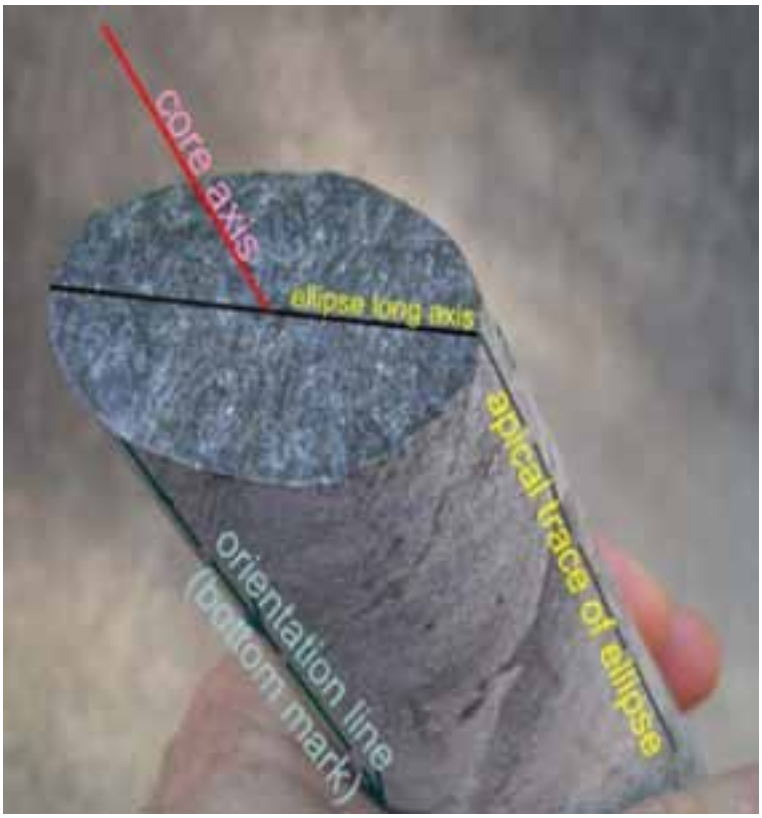


Figure 6: Orientation line and core axis shown on orientated core

Measurement conventions used are:

- Alpha angle (α-angle): the acute angle (Figure 7) between the core axis and the long axis of the ellipse (0-90°);
- Beta angle (β-angle): the angle (Figure 7) between a reference line along the core and the ellipse apical trace measured in a clockwise sense (0-360°). In 'oriented core', the reference line is the 'orientation mark' or 'bottom mark' and the beta angle of the apical trace of the ellipse is measured clockwise from this line.

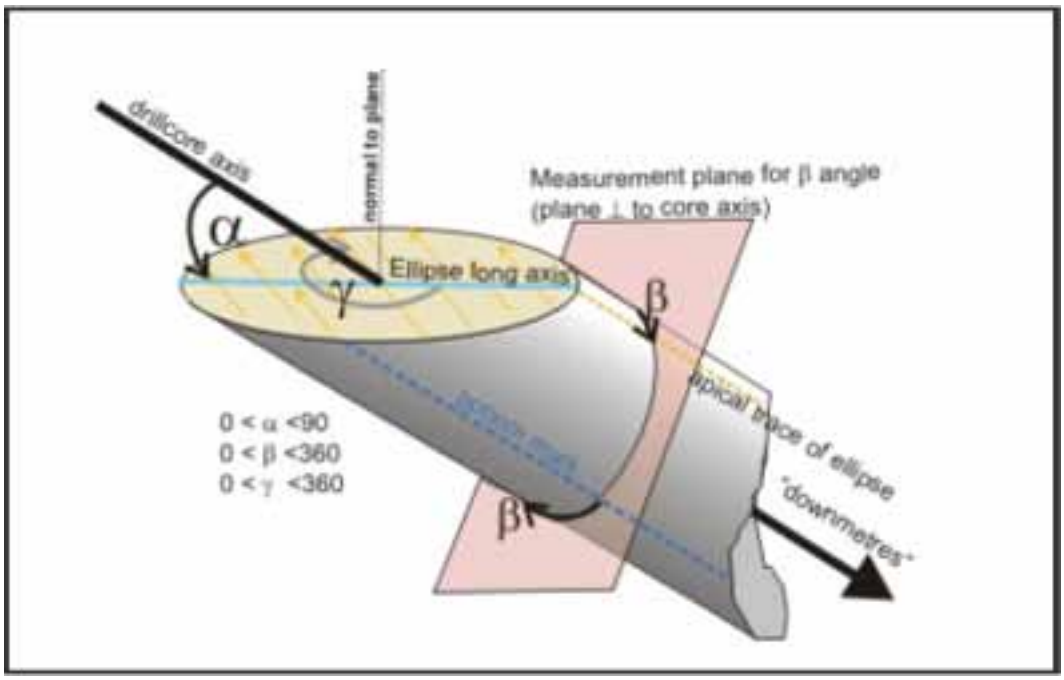


Figure 7: Alpha and beta angles

Fracture Frequency (FF)

The Fracture Frequency (FF) does not recognise core recovery; the FF/m must be increased if there is core loss, which will occur in the weaker sections of the core. The adjustment is done by dividing the FF/m by the core recovery and multiplying the quotient by 100.
FF = ((Total Number of Joints / SCR) / CR) x100

Joint Surface Condition (JC)

Asses the micro-joint roughness (less than 10cm scale) with one or more of the following ratings:

- Rough stepped/Irregular
- Smooth stepped
- Slickensided stepped
- Rough undulating
- Smooth undulating
- Slickensided undulating
- Rough planar
- Smooth planar
- Polished

These terms are illustrated graphically on Figure 8. The 'rough' and 'smooth' prefixes generally refer to the grain size of the host rock, but characterise the nature of the joint surface between the 'steps' or 'undulations'.

back

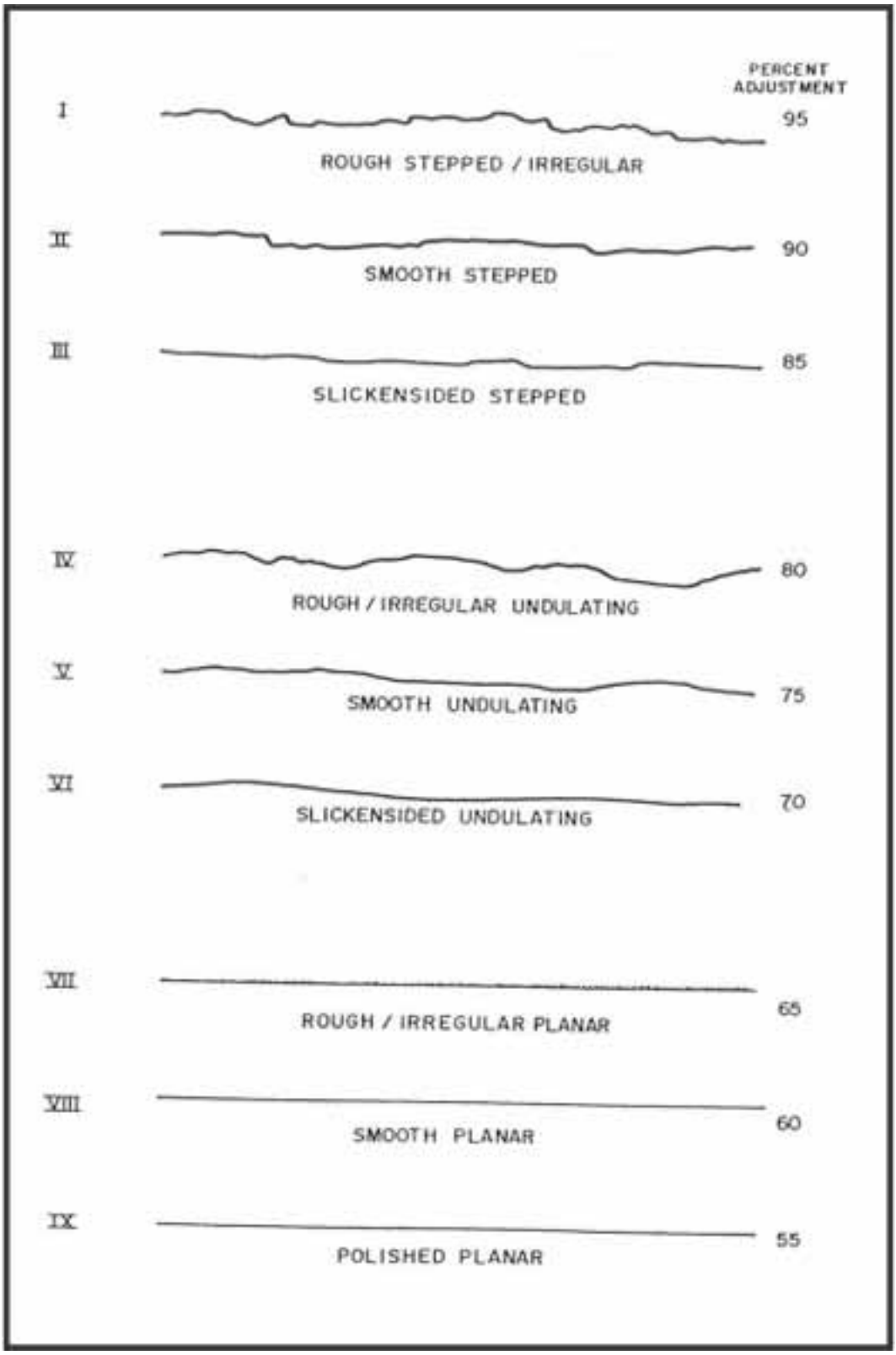


Figure 8: Joint roughness profiles.

The macro-joint roughness (1m scale) is difficult to determine from core fracture surfaces, due to the limited exposure. A value is required and estimation is made from the core from one of the following ratings:

- Planar
- Undulating
- Rough planar
- Curved
- Irregular

back

Infill type

The fill on the fracture surfaces must be characterised with one or more of the following ratings:

- Gouge thickness > Amplitude of irregularities (if more than 3mm this should be added as a parameter on the MRM calculations)
- Gouge thickness < Amplitude of irregularities
- Soft-sheared material – Fine
- Soft-sheared material – Medium
- Soft-sheared material – Coarse
- Non-softening material – Fine
- Non-softening material – Medium
- Non-softening material – Coarse
- Clean

The 'Gouge' for the 1 and 2 ratings implies soft-sheared material and not a solid vein. 'Soft-sheared' implies friable material that can be broken between the fingers. 'Non-softening' implies a non-friable material such as calcite, quartz etc that would typically have closed the joint – it is softer then the host rock.

Wall alteration

This is an indication of the fill material on the joint surfaces. One of the following ratings should be chosen:

- Wall = Rock hard
- Wall > Rock hard
- Wall < Rock hard

REFERENCE

- Simple predicted scale photographic images of grains used for visual determination of grain roundness (Powers, 1953).
- Colour chart used in core logging (Corstor training handouts).
- Cleavage that forms at right angles to direction of maximum compression (Ramsay and Huber, 1987).
- Chapter 3, Rock mass classifications – Hoek's notes.
- A geomechanics classification system for the rating of rock mass in mine design – DH Laubscher
- Page 7, Hoek E., Practical rock engineering, 2006.
- Page 14, Stacey T.R., Best practice rock engineering for Other Mines, 2001.
- Chapter 3, Hoek E., Brown E.T., Underground excavations in rock, 1980.

LEARNING OUTCOME 3.1.1.2

DESCRIBE, EXPLAIN AND EVALUATE THE PROBLEMS ASSOCIATED WITH CHARACTERISING WEATHERED ROCK

It is essential to evaluate the degree of weathering using the same descriptions so as to avoid confusion and misinterpretations. In rock mass rating systems used in the mining industry, the degree of weathering suggested below is commonly used.

back

Weathering

The degree of weathering of the rock is rated on a scale from 1 to 5 as follows:

1. Un-weathered
2. Slightly weathered
3. Moderately weathered
4. Highly weathered
5. Completely weathered

Adjustments for weathering as used in the MRMR system are shown below. A weathering adjustment is relevant when rock types occur that are susceptible to deterioration over time. The adjustment percentages are given in the table shown in Figure 3-6 below (Laubscher, 1993).

Rate of weathering and adjustments (%)					
Description of weathering extent	6 months	1 year	2 years	3 years	4 + years
Fresh	100	100	100	100	100
Slightly	88	90	92	94	96
Moderately	82	84	86	88	90
Highly	70	72	74	76	78
Completely	54	56	58	60	62
Residual soil	30	32	34	36	38

Table 1: Weathering adjustments (Laubscher 1993)

In explanation of the adjustments given in Table 1, if the rock will weather to a residual soil in six months, the adjustment is 30%; if it will be only slightly weathered after three years, the adjustment is 94%. In Figure 9 below, a weathering classification used in core logging is shown.

NOTE

In all these tables for the selection of the state of weathering of the material, decisions are based on experience and knowledge of the material and state of weathering and may differ greatly between loggers. The single biggest concern is that weathering ratings are subjective and contrary to most other parameters rated, cannot be accurately measured.

Term	Symbol	Description	Discoloration extent	Fracture condition	Surface charateristics
Fresh (FW)	W1	No visible sign of rock material weathering	None	Closed or discolored	Unchanged
Slightly weathered (SW)	W2	Discoloration indicates weathering of rock material on discontinuity surfaces. Less than 5% of rock mass altered	<20% of fracture spacing on both sides of fracture	Discoloured, may contain thin filling	Partial discoloration
Moderately weathered (MW)	W3	Less than 50% of the rock material is decomposed and/ or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuos framework or as corestones	>20% of fracture spacing on both sides of fracture	Discoloured, may contain thick filling	Partial to complete discoloration, not friable except poorly cemented rocks

back

Highly weathered (HW)	W4	More than 50% of the rock material is decomposed and/ or disintegrated to a soil. Fresh or dis-colored rock is present either as a discon-tinuos framework or as corestones	Throughout	Filled with al-teration minerals	Friable and possibly pitted
Completely weathered (CW)	W5	100% of the rock mate-rial is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.	Throughout	Filled with al-teration minerals	Resem-bles soil
Residual soil	W6	All rock meterial is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume but the soil has not been signifi-cantly transported	Throughout	N/a	Resem-bles soil

Figure 9: Table indicating weathering classification used during core logging

REFERENCE

- Laubscher D.H., Planning mass mining operations, 1993

LEARNING OUTCOME 3.1.1.3

DESCRIBE, EXPLAIN AND EVALUATE THE IMPLICATIONS AND IMPACT OF THE ORIENTATION OF THE SURVEY LINE ON THE INTERPRETATION OF RECORDED DATA

The survey line used for mapping of discontinuity surfaces is normally referred to as a 'scan line' or 'sampling line'. It is imperative that the survey scan line is orientated in such a way that the interpretation of the recorded data is easy. For instance: In the case of layering, the scan line should preferably cross the exposed layer as close as possible to 90°. In Figure 10, 'scan line 1' mapping will not record the layering, while 'scan line 2' will intersect layering, provide suitable data and is therefore better orientated.

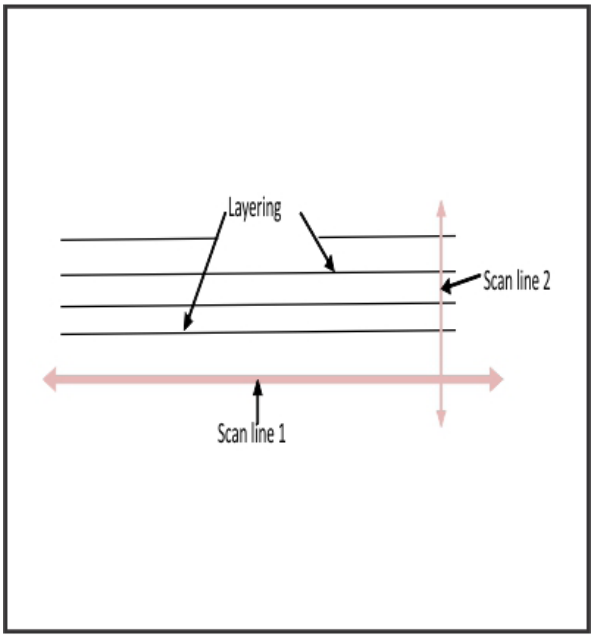


Figure 10: Sketch illustrating best orientations of a scan line

back

It is not always possible to cross all the discontinuities at 90°, but it is essential to try and intersect most of the discontinuities with the scan line shown in Figure 11 below.

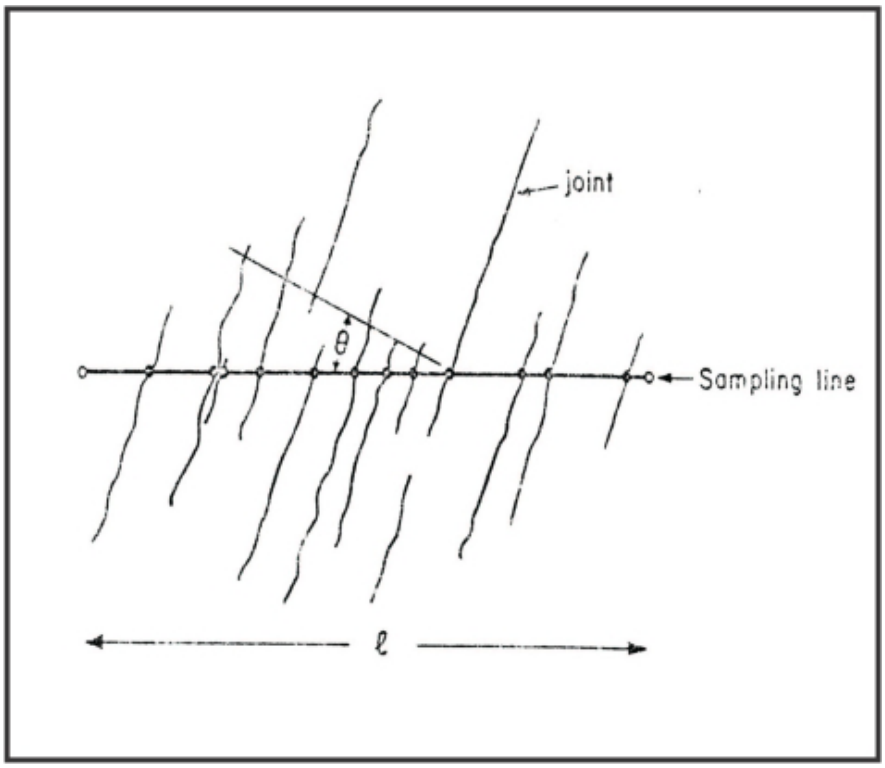


Figure 11: Sketch illustrating position of a scan line to map joints

If a survey scan line is poorly orientated, the data recorded may have missed important discontinuity surfaces and will therefore be wrongly interpreted. In rock engineering designs, as well as in rating the rock mass, it is imperative that all the observed discontinuity surfaces be recorded from the mapping scan line. Should the data recorded be poor, wrong designs, support measures and mining strategies may result.

All mapping techniques suffer from the same problem that structures parallel to an exposed rock surface will not intersect the scan line and will therefore not be recorded. Terzaghi developed a bias and correction factor to correct mapping data for structures that are not recorded. However, other opinions exist that suggest correction is unnecessary when the walls being mapped were created by normal blasting and where the discontinuities result in irregular rock wall surfaces.

The spacing of discontinuities determines block dimensions and therefore also support designs. The relative orientation of the face being mapped and the discontinuities are an important factor that should be considered when interpreting mapping data. The orientation causes some bias in the number and spacing of discontinuities, where the spacing is less than the apparent spacing and the number of discontinuities larger than the number mapped.

The bias in the spacing can be corrected by using:

$$S = S_{app} \sin\theta$$

where S is the true spacing,
Sapp the spacing mapped and
θ the angle between the face and the strike of the mapped discontinuities.

back

back

The number of discontinuities can be adjusted by:

$$N = Napp/Sin\theta,$$

where N is the true number,
Napp is the mapped number and
θ is the angle between the face and the strike of the mapped discontinuities.

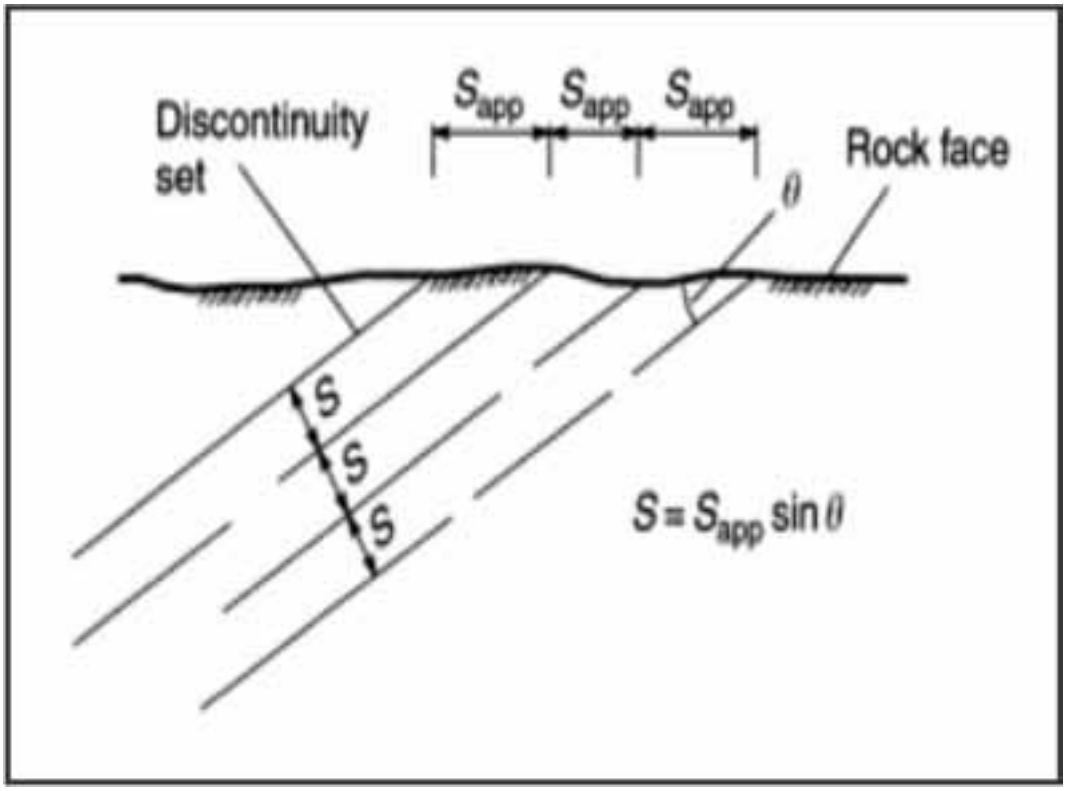


Figure 12: Terzaghi correction for joint mapping

EXAMPLE 4

In the title of the plot in Figure 23 below, the words "unweighted contour plot" are displayed. Explain the significance of this statement.

Solution:
As a result of directional bias of the mapping data, corrections have to be made to the frequency and spacing of discontinuities mapped along the highwall. A method of 'weighting' or 'correcting' the data would be through the Terzaghi corrections.

back

back

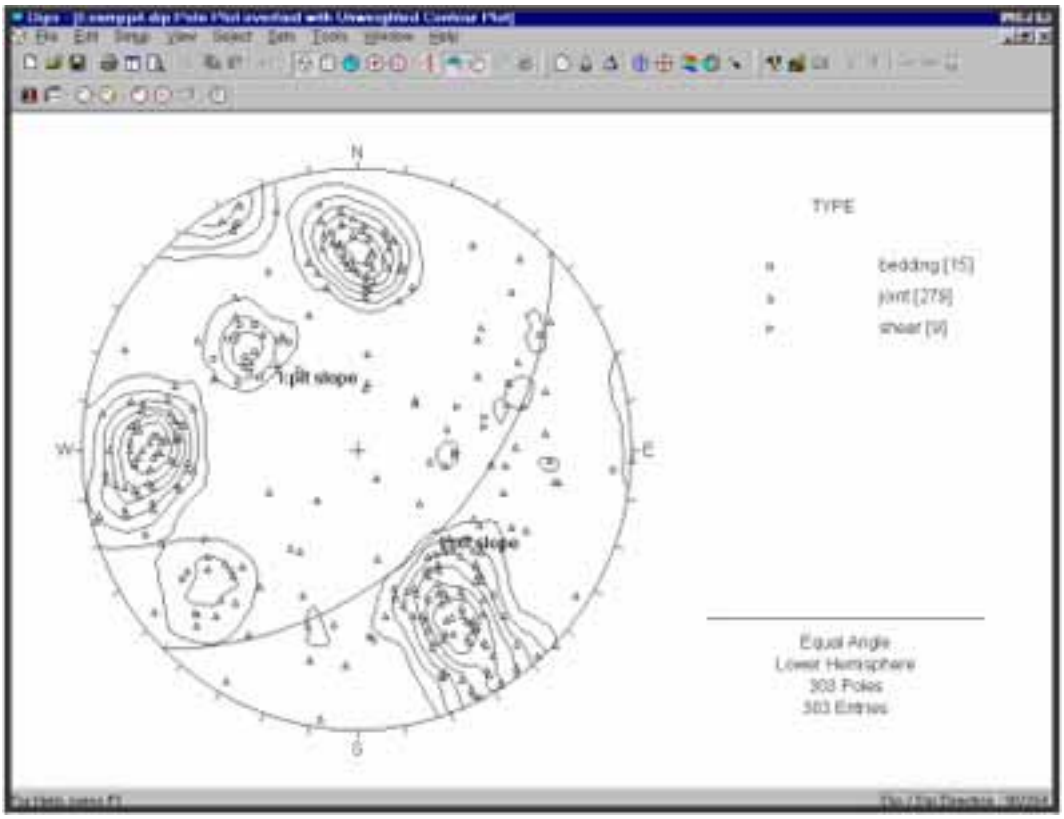


Figure 13: Stereonet plot of poles for collected data from a slope face

REFERENCE

- Wylie, D.C., Mah, C.W., Hoek, E. Rock slope engineering-Civil and Mining, 2004.
- Chapters 3 and 4, Hoek, W., Bray, J.W. Rock slope engineering, 2005.
- Chapter 1, Hoek E., Practical rock engineering, 2006.
- Chapter 3, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.

EXAMPLE 5

List problems associated with data mapping and describe why these will be a problem with the adequate interpretation of data in determining modes of failure of the slope.

Comments regarding

- Dimensionality – Needs more than one dimension as it causes preferential sampling leading to inability to distinguish true spacings of structures running parallel to face
- Apparent spacings – Correction of data to true dip and dip direction is required (Terzaghi's correction) to achieve actual number of intersections and spacings
- Directional bias of structures – Since it is physically impossible to measure the length of structures, it is difficult to build a picture of 3D block sizes and shapes
- Length censoring – It is difficult to establish continuity of length.

REFERENCE

- Paper 3-4 May 2010 Model answers.

back

LEARNING OUTCOME 3.1.1.4

DESCRIBE, EXPLAIN AND INTERPRET DATA PRESENTED IN THE FORM OF STEREONETS

DEFINITION

The stereographic projection is a particular mapping tool that projects a sphere onto a plane. The stereographic projection is therefore a way of picturing the sphere as the plane, with some inevitable compromises. In practice, the projection is carried out by a computer or by hand using a special kind of graph paper called a stereonet or Wulff net.

Why do we need stereographic projections?

- Estimate the orientation of discontinuities based on a population of measurements
- Estimate the number of sets of discontinuities based on a population of measurements
- Estimate from the friction angle (cones and daylight envelopes) the potential for instability of rock blocks
- Define the type of rock mass we are dealing with, i.e. continuous, terraced, blocky, ravelled, rifled
- Calculate dip, apparent dip, angles of intersection, dip of lines of intersections etc,.
- Derive dominant joint generations using rosettes

There are two types of stereonets, namely equal angle and equal area. The projection methods of the grids result in these different grid shapes. Figure 14 below shows an example of an equal area and Figure 15 of an equal angle stereonet grid. The constructions carried out on these different net grids are very similar and their use is often dependent on the preference of the user.

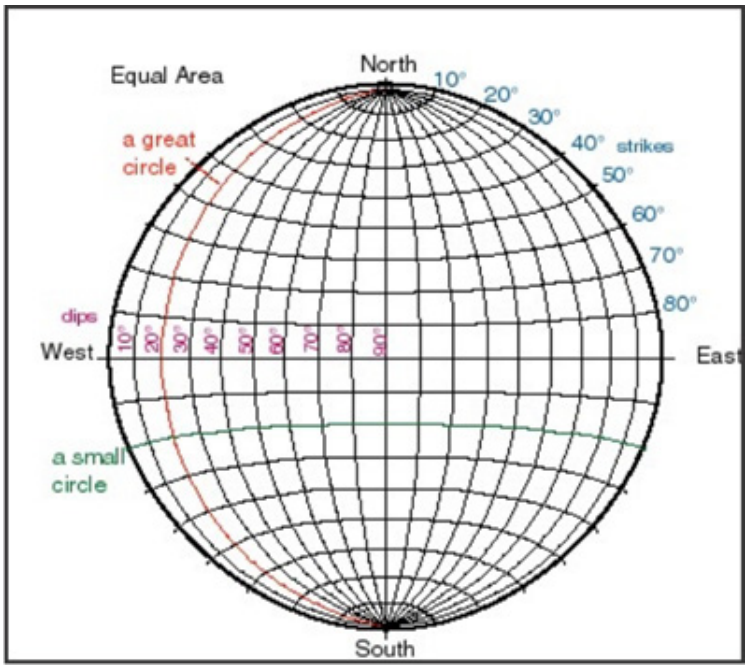


Figure 14: Equal area stereonet

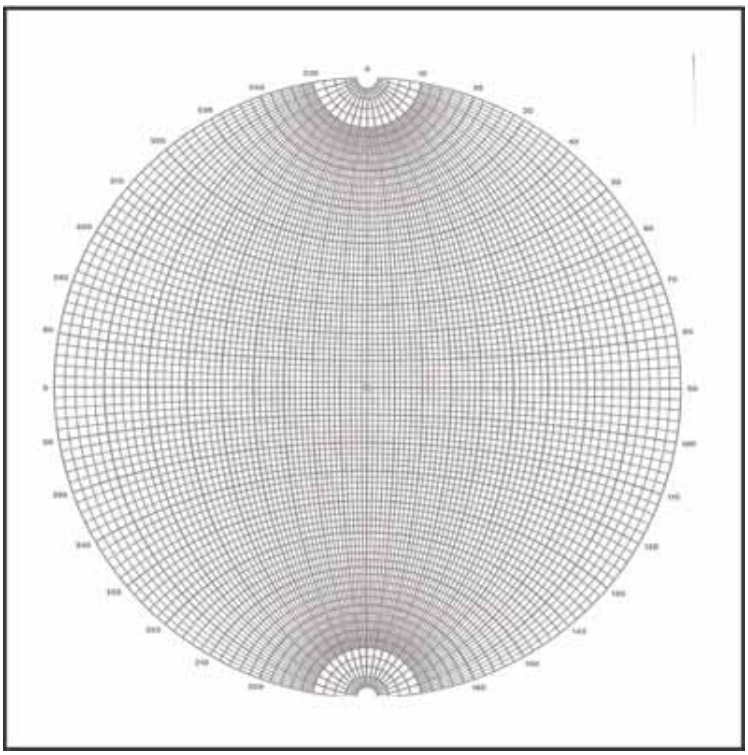


Figure 15: Equal angle stereonet

Plotting of a plane on a stereonet by drawing the representative great circle:

- Prepare the stereonet underlay that shows the following:
  - The zero point (north)
  - 10° gradations from 0° to 360°
  - East-west lines from the outside-in in 10° intervals
- Prepare tracing paper, place it over the stereonet underlay and secure it with a drawing pin through the centre of the net;
  - Mark north on the tracing paper
  - Mark the dip direction on the stereonet by counting the dip direction along the 10° gradations from the north / zero point;
  - Move this dip direction mark to the east-west line on the stereonet;
  - Count and mark the dip from the outside of the circle towards the centre along the 10° intervals;
  - Draw the great circle from the zero point, through the dip mark, along the circles on the underlay;
  - Move the tracing paper so that the north point is back on the zero point;
  - Count 90° from the great circle along EW line and mark the plane's pole position.
- By repeating this exercise for many planes (or dip/dip direction measurements), several poles can be plotted on the stereonet.
- These poles can be grouped based on their positions on the stereonet and a single plane can then be identified that represents grouped poles (Figure 16).

back

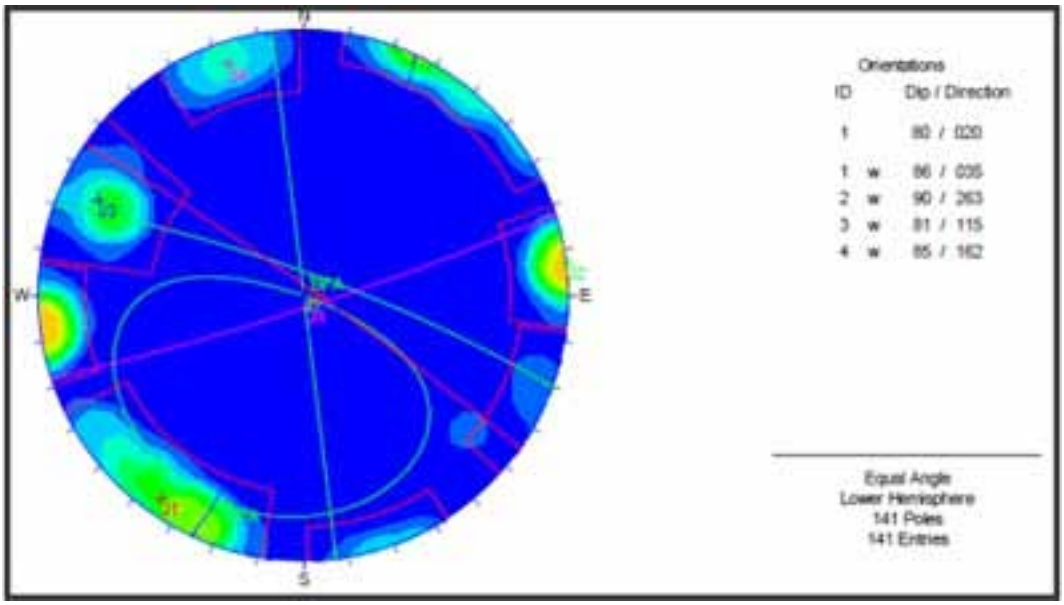


Figure 16: Planes plotted from poles for a great variety of discontinuities

EXAMPLE 6

Orientation measurements were taken at random locations on the surface of a large undulating discontinuity surface in order to estimate its mean orientation. The following values were recorded (dip direction/dip): Plot the poles to the planes, see Figure 17.

074/13	095/35	113/23	117/42	119/33	123/30	125/17
131/26	132/24	135/32	136/27	137/25	138/39	139/28
146/33	150/35	150/44	154/22	161/34	174/29	

Solution:

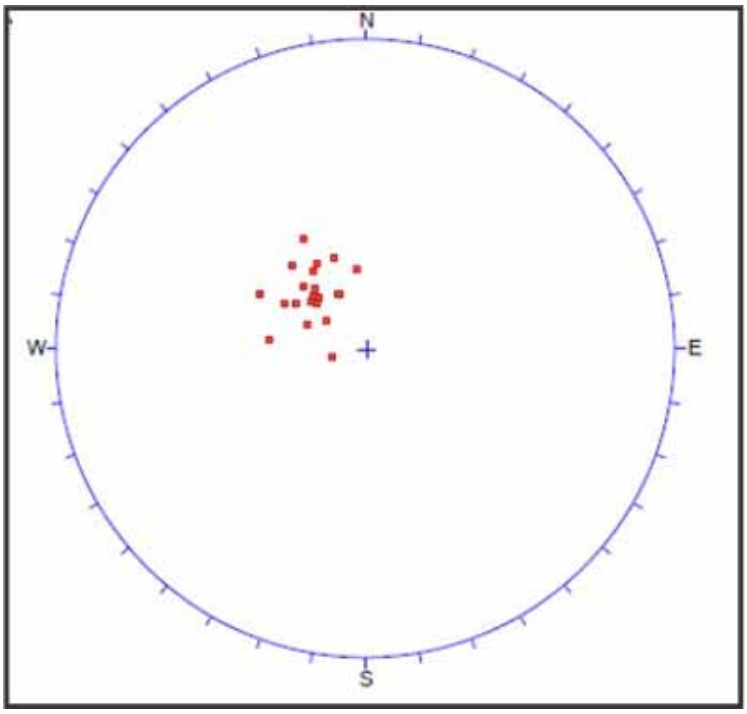


Figure 17: Pole plot for all recorded values

back

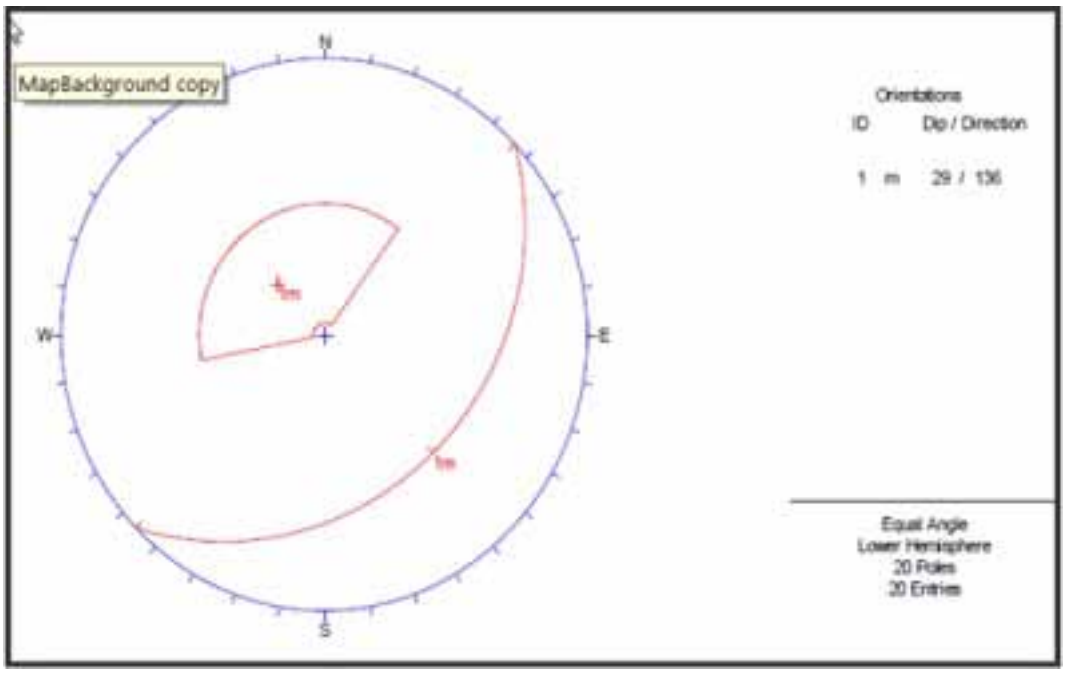


Figure 18: Modal orientation of the representative plane

The poles of the values given were plotted as previously suggested and a representative plane identified (Figure 18) using appropriate software packages (such as Rocscience DIPS), as the plane with dip 29° and dip direction 136°. Figure 3-12 below shows how planes are plotted from poles.

NOTE

Care must be taken in selecting the poles to group, as large variations in the pole plot positions reduce the appropriateness of the representative plane identified by the software.

EXAMPLE 7

Determine the line of intersection between two planes, with orientations 50/130 and 30/250 respectively.

Solution:

- See Figure 19 below. Draw the great circles that represent the two planes on an equal area stereonet as described in the section above;
- Turn the tracing paper until the intersection between the two great circles lies along the east-west axis of the stereonet;
- Measure the plunge (dip of a line) of the intersection line by counting the degrees from the outside of the stereonet to the intersection point – 20.5°;
- Return the tracing so that the north and zero points are aligned again;
- Draw a line from the centre of the stereonet, through the intersection point of the two great circles, until it intersects the outer circle of the stereonet;
- Read of the trend (dip direction for a line) of the line of intersection from the north point clockwise to the intersection – 200.5°.

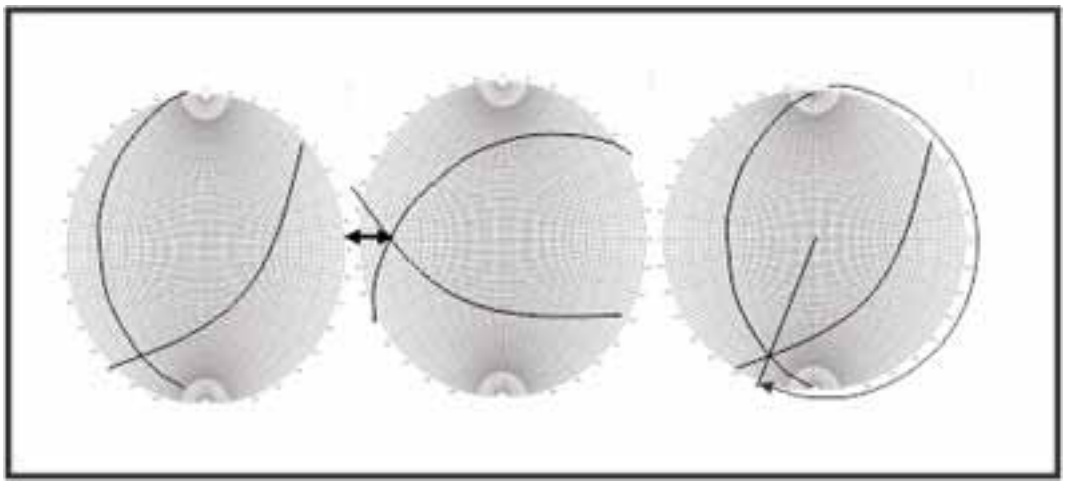


Figure 19: (a) Great Circles, (b) plunge and (c) trend of intersection line

REFERENCE

- Chapter 1, Hoek E., Practical rock engineering, 2006.
- Chapter 3, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.
- Chapters 3 and 4, Hoek, W., Bray, J.W. Rock slope engineering, 2005.

LEARNING OUTCOME 3.1.1.5

IDENTIFY JOINT TRENDS, FAILURE MECHANISMS, AND OTHER FEATURES PRESENTED IN THE FORM OF STEREONETS

By plotting various joints from scan line or borehole data, using stereonets and contouring the data, joint trends, sets and their average dip and dip directions can be easily obtained from stereonet plots. Figure 20 below shows a contour plot of poles using Dips (Rocscience).

CONNECTION 9

Refer to outcome "LEARNING OUTCOME 3.1.1.4" on page 113

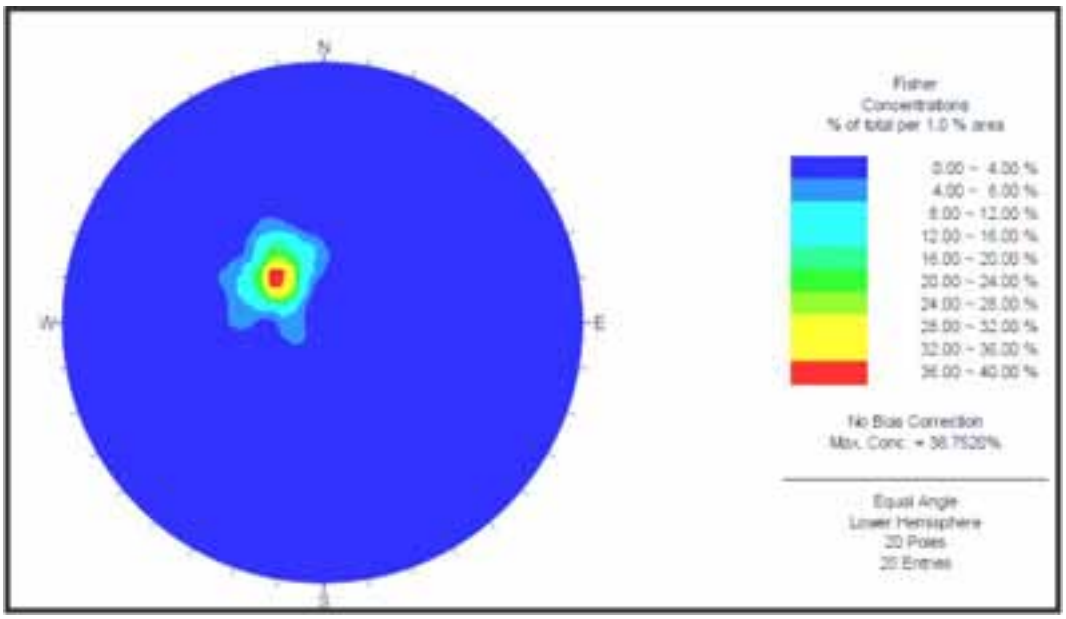


Figure 20: Contour plot of the poles representing a number of planes

back

Stereonets can be useful to determine the failure mechanisms described in Outcome 3.1.1.5. Kinematic (stability) analysis can be carried out using geotechnical data plotted on stereonets. Modern methods mostly include the use of software packages such as Dips (Rocscience), but it is advised to practice these manually, using stereonet underlays and tracing paper. The Dips training manual explains how these stability analyses are carried out and should be available to software owners.

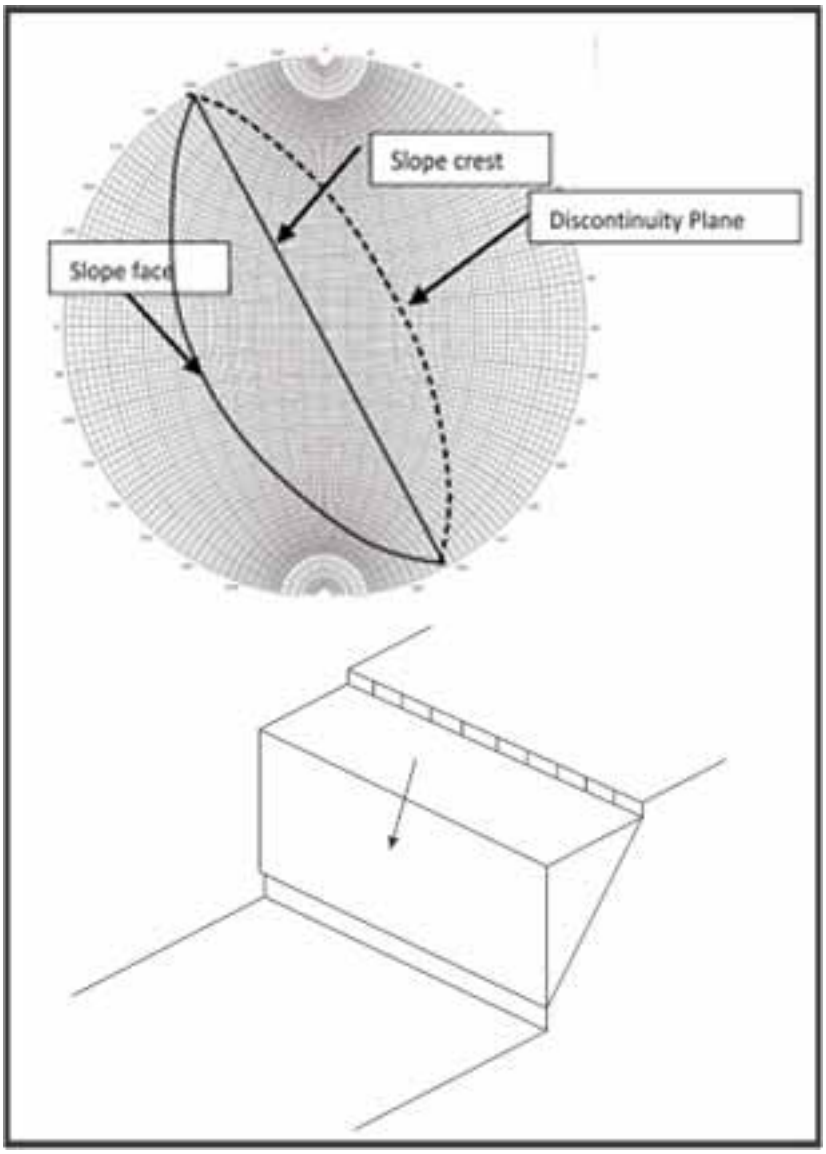


Figure 21: Kinematic analysis of planar failure of an open pit slope

Figure 21 shows the great circles for the slope crest, slope face and discontinuity plane plotted on the stereonet. Planar failure is possible when the discontinity strike is near parallel to the slope face strike. When the discontinuity plane dips at an angle less than the slope face (i.e. the great circle lies outside the great circle of the slope face), failure is possible as the discontinuity plane daylights within the slope face (assumes zero friction on the plane).

Figure 22 below shows a Kinematic analysis for toppling failure. This failure mechanism becomes possible when the discontinuity striek is similar that of the slope face, but the discontinuities dip in the opposite direction to the slope face or are steeply dipping.

back

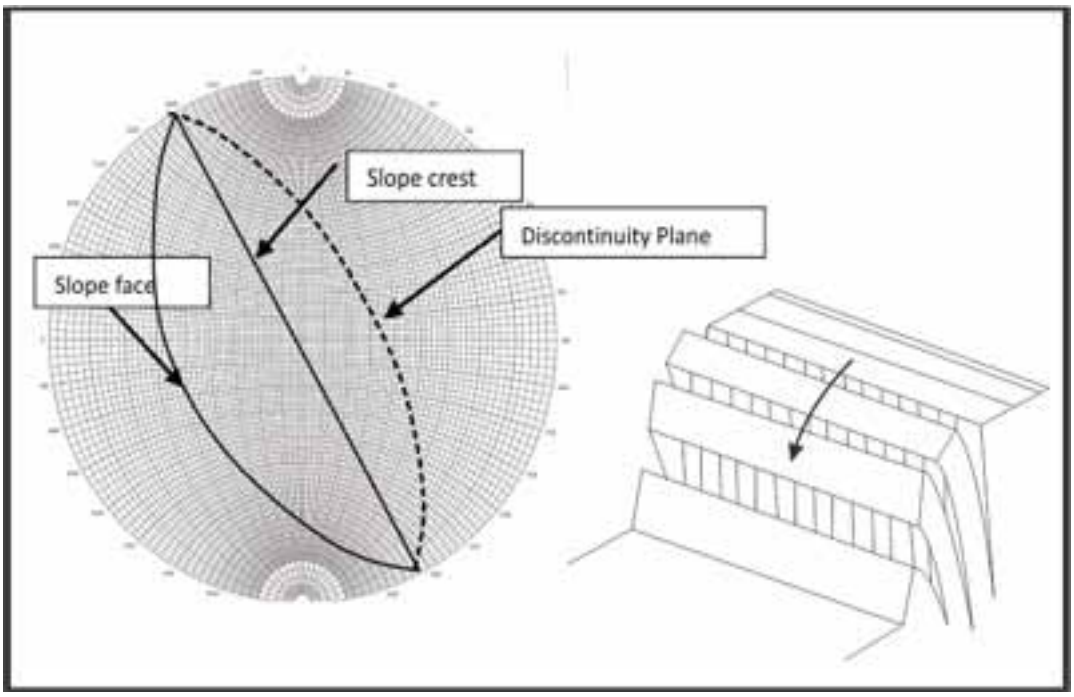


Figure 22: Kinematic analysis of toppling failure of an open pit slope

Figure 23 represents conditions under which a kinematic analysis suggest wedge failure due to two intersecting discontinuities. This failure becomes possible when the line of intersection between two planes plunges at a lower angle than the slope face (intersection plunge falls outside the slope face great circle) and the direction of sliding (indicated by the arrow) is in the same direction as the plunge of the slope face (assumes zero friction on the plane).

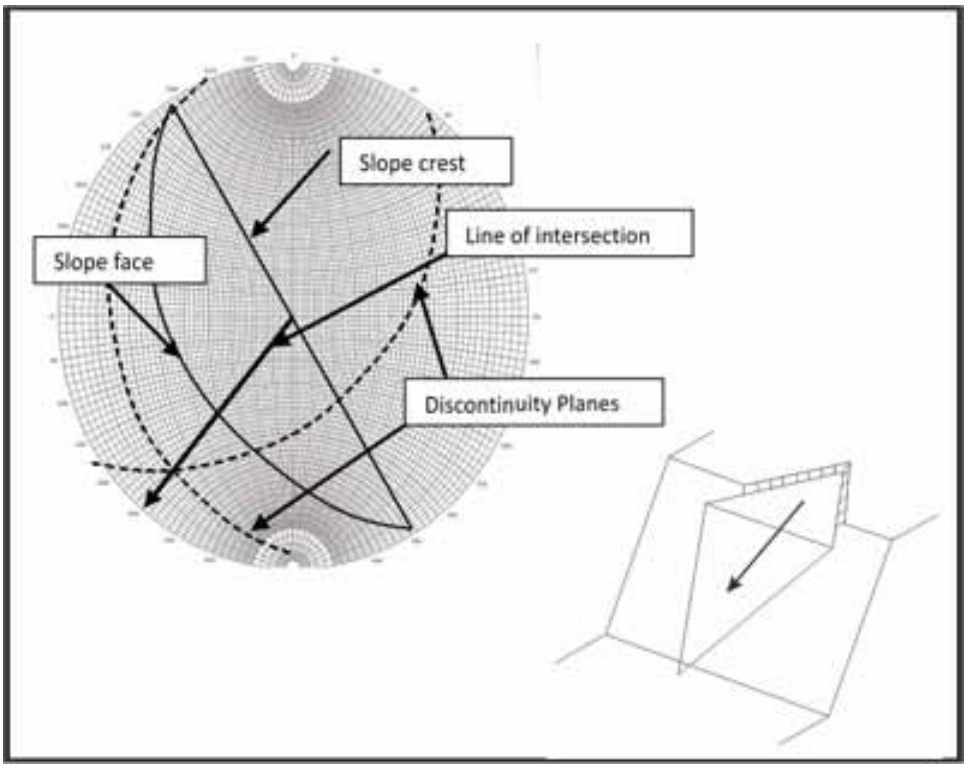


Figure 23: Kinematic analysis of wedge failure of an open pit slope

EXAMPLE 8

Describe what the term 'plane friction cone' represents.

Solution:
It is the stereographical representation of the angle of friction on the plane, and is drawn onto the stereonet as a circle around the centre of the stereonet, at the appropriate angle measured from the outside of the stereonet towards the centre (Figure 24).

EXAMPLE 9

Describe what the term 'sliding zone' represents.

Solution:
The term 'wedge sliding zone' represents the area on a stereonet where, if plane intersections occur inside this area, wedge sliding may occur (Figure 24).

EXAMPLE 10

In Figure 24, data has been presented on a stereonet. Suggest the probable failure mechanism for this highwall.

Solution:
The friction angle on the discontinuities is presented by the circle 'cone' drawn at 35°, while the discontinuities are represented by the great circles. From the data, the following can be observed:

- Wedge failures on discontinuities that intersect:
  - A&B: Direction of failure is into the highwall (opposite dip direction to the slope face) and no failure is therefore possible
  - B&C: Plunge of intersection line is less than the friction angle of the plane and wedge failure will not occur along the intersection line.
  - D&C: Wedge failure is not possible as the plunge of intersection line is more than friction angle and the slope face dip.
  - A&D: Plunge of intersection line is steeper than the slope face, so failure is not possible.
  - A&C: Wedge failure is not possible as plunge of intersection line is more than friction angle and the slope face dip.
  - B&D: Direction of failure is into the highwall (opposite dip direction to the slope face) and no failure is therefore possible
- For plane failure to occur, the planes' strike directions must be near parallel to the slope face and dip at angles more than the friction angle and less than the slope face. The only planes with near similar strike directions are:
  - Plane B: Dip into the highwall so no plane failure is possible;
  - Plane C: Plane failure on C is not possible as the plane dips at steeper level than both the friction angle and the slope face.
- Toppling failure is possible if the discontinuity strikes near parallel to the slope face, but dips steeply in the opposite direction to the slope face:
  - Plane B: Toppling failure is possible, but will be limited due to the difference in the strike directions of the discontinuity and the slope face;
  - Plane C: Toppling failure on C will not be the preferred mechanism due to similar dip direction to the slope face.

back

Figure 24 shows a zone between the slope face great circle and fraction angle 'cone'. For failure to be possible, discontinuity intersections must fall within this 'wedge sliding zone' as the line of intersection between the discontinuities will then dip steeper than the friction angle and shallower than the slope face.

Based on the analysis, only partial toppling failure should occur to plane B.

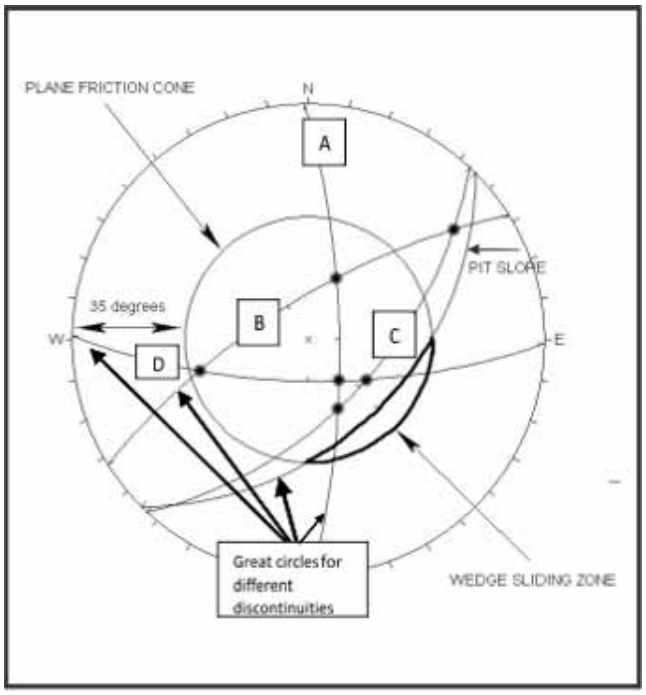


Figure 24: Failure analysis

EXAMPLE 11

If the angle of friction in the abovementioned example was reduced to 20°, would the potential for wedge failure increase?

Solution:
No. No intersections of planes will fall inside the zone. At 20° friction angle, no 'day-lighting' of intersections occurs. For failure to become possible, the pit slope would need to be steepened.

REFERENCE

- Paper 3-4 May 2010 Model answers.
- Chapters 3 and 4, Hoek, W., Bray, J.W. Rock slope engineering, 2005.
- Chapter 3, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.
- Chapter 1, Hoek E. , Practical rock engineering, 2006.

LEARNING OUTCOME 3.1.1.6

DESCRIBE, EXPLAIN AND APPLY CORE LOGGING TECHNIQUES

Construction of geotechnical borehole logs

Geotechnical borehole logs are normally constructed based on the geotechnical data assimilated from the geotechnical logging of the drill cores and will comprise an assessment of the following parameters:

back

- Drilling interval
- Core recovery (Total and Solid)
- Rock mass designation (RQD)
- Lithological unit
- Rock competence
- Weathering regime
- Intact rock strength (IRS)
- Joint distribution
- Fracture frequency (FF)
- Joint surface condition.

If jointing in the rock mass is judged to be such that excavation behaviour will be dictated significantly by the joint orientations and other joint characteristics, then specific mapping of the joint parameters will be required. If, however, the behaviour will be of a homogeneously jointed rock mass, then rock mass classification mapping will be appropriate.

Joint mapping

A detailed description of methods of joint mapping is not appropriate for this handbook, and it is considered that such detailed mapping will not normally be necessary. To determine the potential for the formation of blocks and wedges (the geometric possibility of occurrence thereof), it is necessary to know the following parameters and their variability for each joint set:

- the orientation (dip and dip direction);
- the joint spacing; and
- the joint length.

From studies of jointing in different parts of the world, typical statistical distributions of these parameters have been reliably established. These distributions are:

- joint dip angle – normal distribution;
- joint dip direction – normal distribution;
- joint spacing – log normal;
- joint length – negative exponential.

Page 38 of Best practice handbook for other mines shows an example of a geotechnical core logging sheet.

The methods to log core are explained below.

Borehole details

The following log indicates an example of parameters recorded for a surface borehole. Borehole data summary sheet used and the final state of the borehole are recorded as shown in Figure 26

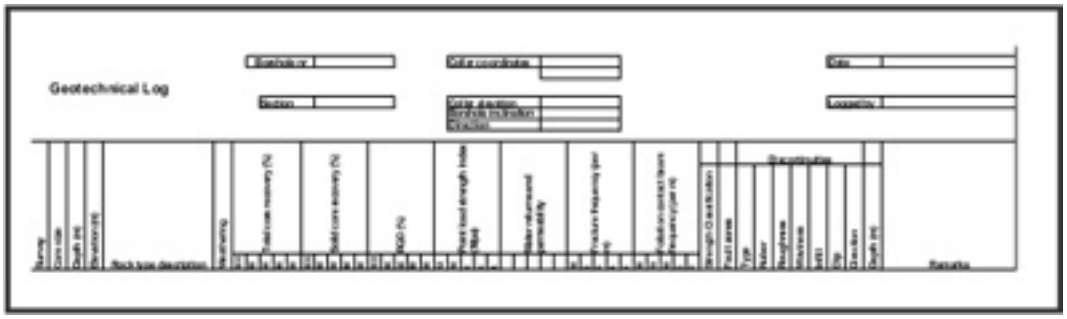


Figure 25: Components in a geotechnical log

A variety of measurements is ususally filled in by the drilling crew and lists possible problems they have encountered at a specific site. These comments should be made as drilling progresses and should be recorded (Figure 26) in a Drillers' log. These parameters have no absolute physical meaning, but with a moderate amount of care and without special instrumentation, they can give useful information about the strength and consistency of the rock and guidance in the selection of mining or construction equipment. All the following measurements on core are made as part of the drilling operation.

It is essential that a clear and accurate log is kept at all times of the Depth of the geological units in the borehole. Large cumulative errors can develop in long boreholes and inconsistencies between core length, borehole depth and core recovery percentage, as measured by the drillers, occur frequently. The drillers must be made aware of the requirement for accuracy in geotechnical boreholes.
Drilling rate is the rate of penetration of the drill and should be recorded (for a constant specified hydraulic feed pressure) at regular intervals as drilling progresses. It is recommended that this should be done by timing the advance over a marked distance. Delays such as slowdowns due to caving of the hole or machine breakdowns should not be featured as a slow drilling rate.

The make and type of Drill bits used should be specified and data relating to bit wear should be collated with a view to determining average bit life in different rock types or regions.

GENERAL DRILLING DATA

SIZE OF CASING	DEPTH (metres)	DATE OF INSERTION	RECOVERED?

CORE SIZE	DEPTH (metres)	
		Core stored at:
		No. of Core Trays:
		Core Recovery Statistics:

HOLE	DATE OF COMPLETION	DEPTH (metres)
ORIGINAL		
1ST DEFLECTION		
2ND DEFLECTION		
3RD DEFLECTION		
4TH DEFLECTION		
5TH DEFLECTION		
6TH DEFLECTION		
7TH DEFLECTION		
8TH DEFLECTION		
9TH DEFLECTION		
10TH DEFLECTION		
11TH DEFLECTION		
12TH DEFLECTION		
13TH DEFLECTION		
14TH DEFLECTION		
15TH DEFLECTION		

Figure 26: General drilling data information recorded (Drillers' log)

Core length recovered versus borehole length is the actual core that is salvaged from the hole as compared to the total length of core that should have been drilled.

Core losses are measured using a tape measure, the cumulative length of all core recovered should be expressed as a percentage of the length of the hole drilled (identified from the drillers recorded depths) over the given logging interval and entered as a histogram of 'total core recovery'. Similarly, the cumulative length of core recovered as solid intact pieces of rock, expressed as a percentage of the drilling run, should be entered as 'solid core recovery'.

When the core has been recovered, it should be withdrawn with minimum disturbance, laid out in core trays of standard lengths (1.5 or 2 metres is standard practice), cleaned and photographed as soon as possible afterwards. The core is removed from the core trays and laid out. It is then washed down and cleaned with a cleaning agent to remove grease and grime from the drilling process. It is important to pay attention to what cleaning agent you use, because some agents affect the mineral composition of the rock.

The core orientation (plunge and azimuth) of any borehole used should be surveyed using the most accurate method available. Specific borehole

parameters should be measured in order to orientate the discontinuities intersection with the core and these are illustrated in Figure 28.

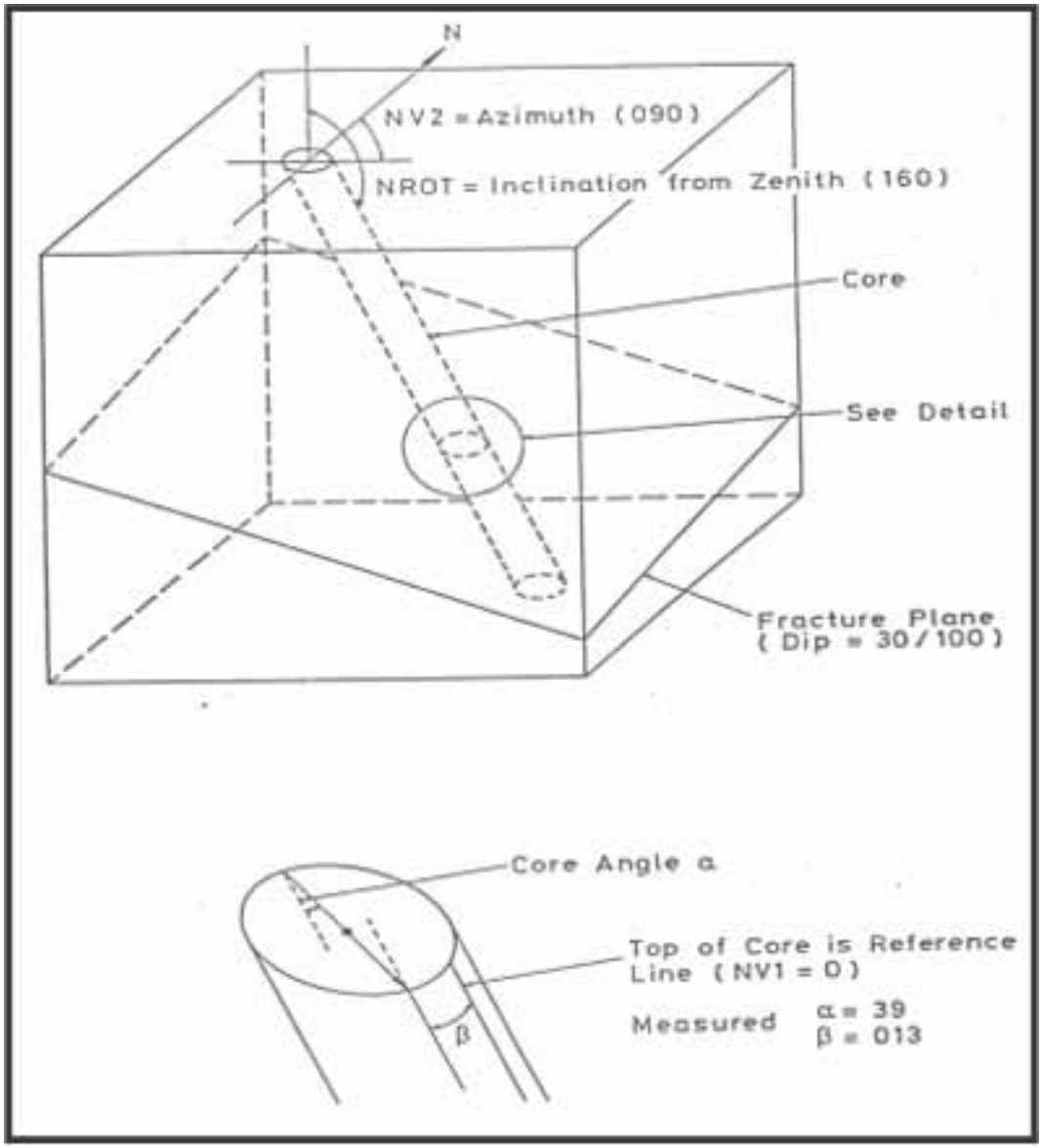


Figure 27: Measured parameters for core orientation (Ramsay and Huber, 1987)

From Figure 27, the angle NV1 is the core reference angle measured from the "top" surface of the core along the axial direction, around to the reference line (either the Craelius orientation marker or the dip direction of the geological marker). The angle NROT is the angle of inclination (plunge) of the borehole measured from zero vertically upwards, to 180 degrees vertically downwards. NV2 is the angle of the azimuth of the borehole measured from North.

Discontinuity orientation is such an important parameter that it is often necessary to use orientation apparatus when carrying out diamond drilling. The Craelius core orientation device is appropriate for use during the site investigations. This is by no means the only core orientation apparatus available, but it is the most robust and simple. The Craelius orientator is a mechanical device that fits inside the appropriate core barrel, takes the profile of the core stub and recedes back into the barrel. The various stages in the operation are illustrated in Figure 28.

back

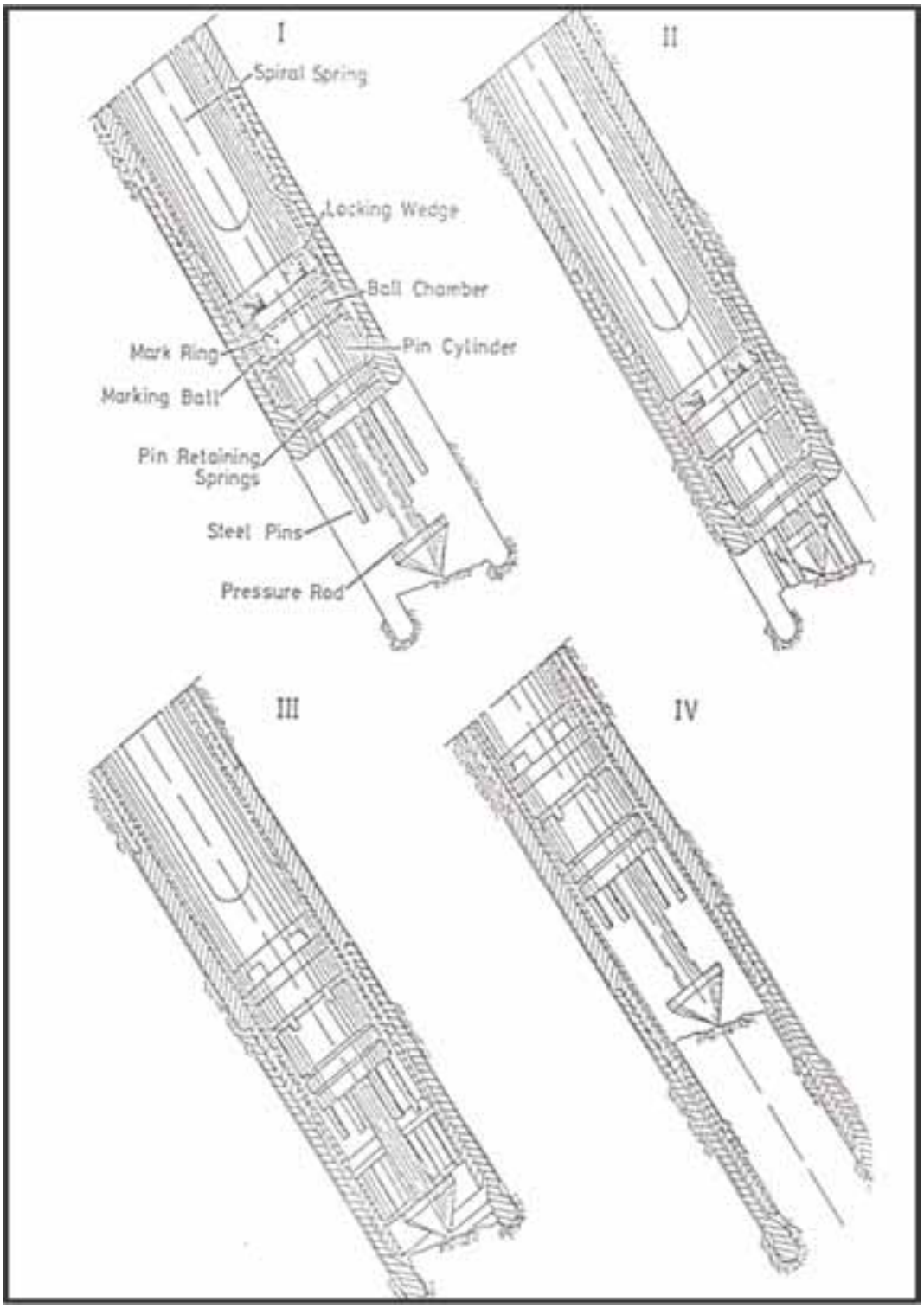


Figure 28: Craelius orientator

The apparatus is shown with its working parts labelled (diagram 1, Figure 28). It consists of six self-locking steel pins that take up the profile of the core stub at the base of the borehole (diagram 2). A free-moving ball bearing within an internal chamber locates itself at the gravitational base of the device. When pressure is transformed to a spring-loaded plunger via the pressure rod, the chamber with the steel ball is compressed, trapping the ball bearing and impressing it against an aluminium disc. At the same time, the profiling pins are locked in position and the locking wedges are released (diagram 3). During the core run, the orientator recedes into the barrel and is recovered with the core (diagram 4). On removal of the core, the top stub is placed in a cradle and the face of the rock stub is rotated until the profile conforms to the positioning of the

back

back

steel pins on the orientator. The gravitational low point is then identified from the indented aluminium plate and the orientator reference line is inscribed on the core stub. Thereafter, the successive core segments are aligned by matching the opposing fracture surfaces.

There are three major constraints on the use of this orientation device:

- The borehole must be inclined 10 degrees or more from the vertical and accurately surveyed to prove this.
- It can only be used in relatively competent ground conditions where the pressure rod can be displaced and the segments of core matched sequentially.
- The rock stubs must not be completely flat (if there is a dominant sub-perpendicular discontinuity, the borehole should be re-orientated)

These constraints require that considerable care must be exercised by the drillers.
The orientation device is not manufactured to all standard diameters and a special core
barrel may have to be purchased for the operation. Advanced planning of the drilling programme is required to ensure inter-compatibility of the drilling equipment.

Core sequence is a continuous section of the stratigraphic unit (Figure 29) that is obtained by drilling via a hollow cylinder for the purpose of obtaining geological information. If the sequence has all the lithologies in the core, you have a complete core sequence. Most mines usually have a core sequence that is kept for training purposes and to compare to samples they take from underground.

Figure 29: Section of a core sequence

Photography is an important part of geotechnical logging, as it sometimes captures what the eye cannot see, and may be referred to after the core has been discarded. An apparatus, such as the one illustrated in Figure 30, ensures that photographs are taken at a constant scale.

back

A scale should always be used when taking photographs, as shown in Figure 29 with the clinorule, as it gives an idea of size when the photograph is distorted. Before taking a picture, ensure that the core is watered down to obtain the maximum information from the core. This is absolutely necessary, whether you are taking pictures or logging the core.

Wet the core regularly as it dries out and creates differences in colours on different photographs that make analysis difficult at a later stage; however, be careful not to wet rock types that may react with water and result in changes to the competency of the specific rock type as it makes logging very difficult.

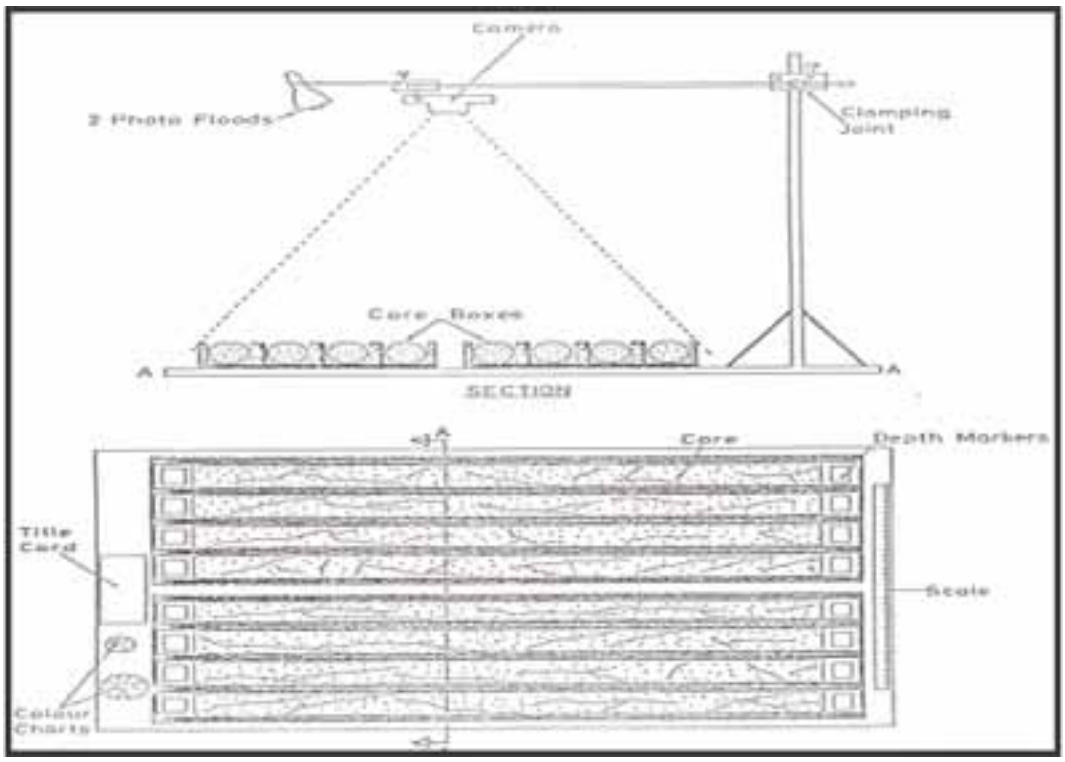


Figure 30: Photographs taken at a constant scale using a core photography unit

Sampling

Sampling is the process whereby a number of small portions or samples of a drilled core are taken for tests that will be used to estimate the quality of the rock mass.

The sizes of the samples taken are usually governed by the number that is available, but at least five samples should be used (ISRM Standards on tests) for each of the specific areas in the stratigraphy that is of interest to you.

Samples should also be handled, stored and prepared according to industry-accepted norms (ISRM Standards on tests).

Protection against weathering is very important, as oxidation is a weathering process that is common to core samples. Oxidation is a process that changes the colour and composition of the core, which is why core is always logged as soon as possible after drilling. Logging core after a period of time could result in incorrect values. E.g. if the samples were left outside in the rain, some rock types will weather to a point that they crumble when touched and cannot be used to gather information. The core should be wrapped and packed in the core trays and stacked in a

safe environment when it is brought from underground and also when logging is completed.

- Stacey, T.R. Best practice handbook for other mines, SIMRAC, 2001.
- Roc-G 025 Learning guide.

LEARNING OUTCOME 3.1.1.7

DESCRIBE, EXPLAIN AND APPLY SCAN LINE MAPPING TECHNIQUES

This outcome is described by providing an example of mapping and analysis of data as performed at a specific site and for a specific purpose. The aim of this example is to show the process and techniques followed in this exercise and does not indicate 'rules for how it must always be done'. It is advised that other examples be sought and studied to broaden exposure to alternative processes.

Joint mapping was carried-out in stopes using the scan-line survey technique. A total of more than 150 discontinuities were mapped in stopes over a total length of 450m. Stress induced fractures parallel to the face were not mapped in detail as they were numerous and ubiquitous (parallel and closely spaced). A single measurement was taken and was considered representative enough for the set (Figure 31).

Figure 31: Ubiquitous (parallel and at constant spacing) stress fracturing

Discontinuity Survey Data Sheet											
Project		Develop-ment mapping		Face				Level			
Tunnel Orientation				Scanline Orienta-tion				File Name			
From POS		A		To POS		A2		Date	2004/05/28		
Msc 2 Mapping				Logged by		H.G					
Discontinuity Type	Location		Rock			Orientation		Continuity			Re-marks
	Distance (m)		Type	Hardness	Sur-face	Dip	Dip Direc-tion	Dip	Strike		
								Ends	Length (m)	Ends	
B/P (Bed-ding Plane)	7.7	2.2	Elsburg			33	193			Into RM	No Filling
B/P	1.1	3.3	Elsburg			33	179			Into RM	No Filling
B/P	1.35	3.8	Elsburg			25	189			Into RM	No Filling
B/P	5.7	8.3	Elsburg			25	153			Into RM	No Filling
B/P	6.3	7.2	Elsburg			29	158			Into RM	No Filling
B/P	8.7	11.1	Elsburg			30	180			Into RM	No Filling

Table 2 : Scanline mapping data sheet

A 30m measuring tape was held straight and tight between two strike gullies for stope mapping. To avoid tape sag, the tape was attached with wire to timber packs or props during stope mapping (Figure 32). In stope mapping, the tape was run from one gulley to another. Once the tape was fixed, every discontinuity intersecting the tape was measured and recorded. The scanline mapping data sheet used is shown in Table 1.

The mapping procedure involved recording the following data onto a mapping data sheet shown as Table 2:

- The distance along the scanline (measuring tape);
- The point where the discontinuity intersects the line;
- Discontinuity type column, i.e. whether the discontinuity was a joint, fault etc;
- Dip and dip directions were then measured using a compass;
- Termination of joint planes was noted as to whether they were terminating into or out of the rock mass;
- The distance from the discontinuity-scanline intersection to joint termination;
- Any interesting feature of the discontinuity was recorded, such as filling, roughness, and, most importantly, the average orientation of neighbouring stress-induced fractures;
- Calculation of the dip and dip directions for the stope was done using geo-survey maps.

Figure 32: Attaching the scan line tape

Measured orientation data (dip and dip direction) from the mapping data sheet was transferred to the DIPS software. The software plots joint positions in space making it possible to visually delineate different joint sets. Once joint sets are identified, joint spacings are then calculated and, together with other properties (e.g. lengths, filling etc), are allocated to their corresponding sets. For analysis of statistical distributions, commercially available statistical software (JMP 4) was employed. In the analysis of results, data from excavations with common orientations were combined. In instances where the mapped data is considered to be limited and not appropriate for acceptable statistical analysis in individual stopes, the data for various stopes were combined and analysed.

An illustration of joint orientation results for the stope hangingwalls is shown in Figure 33 (as taken from DIPS, Rocscience).

back

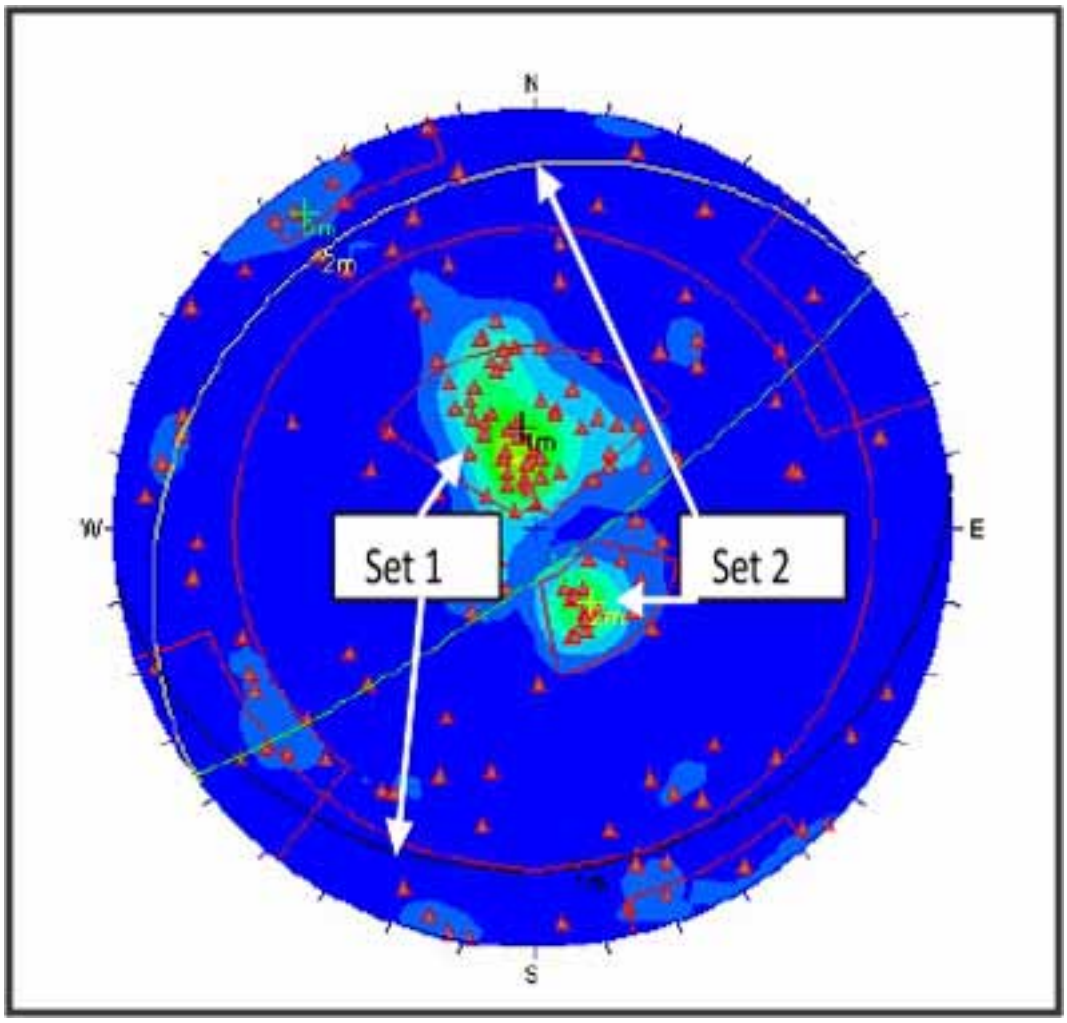


Figure 33: Overall joint orientation in hangingwall stopes

Plotting the poles for the mapped joint data on a stereonet, clear groupings of poles became apparent. Using these groupings, two joint sets were identified, where the dominant set is termed Set 1 and the lesser set is termed Set 2, with representative orientations of 20°/167° and 19°/322° (dip/dip direction) using DIPS (Rocscience). It is clear that these joints are mostly shallow dipping. The remaining joints mapped are random joints.

Most of the joints in stopes were unfilled (data recorded). However, the few filled joints were steeply dipping and had rough quartz as the filling material, with a thickness ranging between 2 and 5cm.

Stress fractures were observed to follow the direction of these quartz-filled joints. Statistical distributions of joint orientations were observed to be normal (Figure 35). The results correspond well with what most researchers have concluded when analysing joint orientation distributions.

back

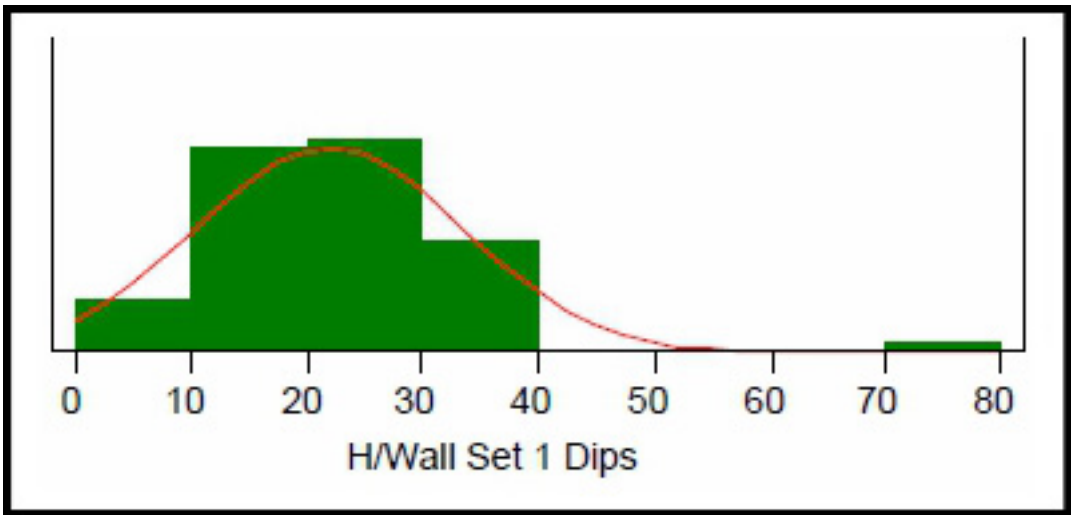


Figure 34: Set 1 hangingwall dip distribution

Joint data from a mapping data sheet was entered into an Excel spreadsheet and allocated the joint set numbers identified with the DIPS software. Joint spacings are then re-calculated taking cognisance of the corrections suggested by Terzaghi (Figure 35).

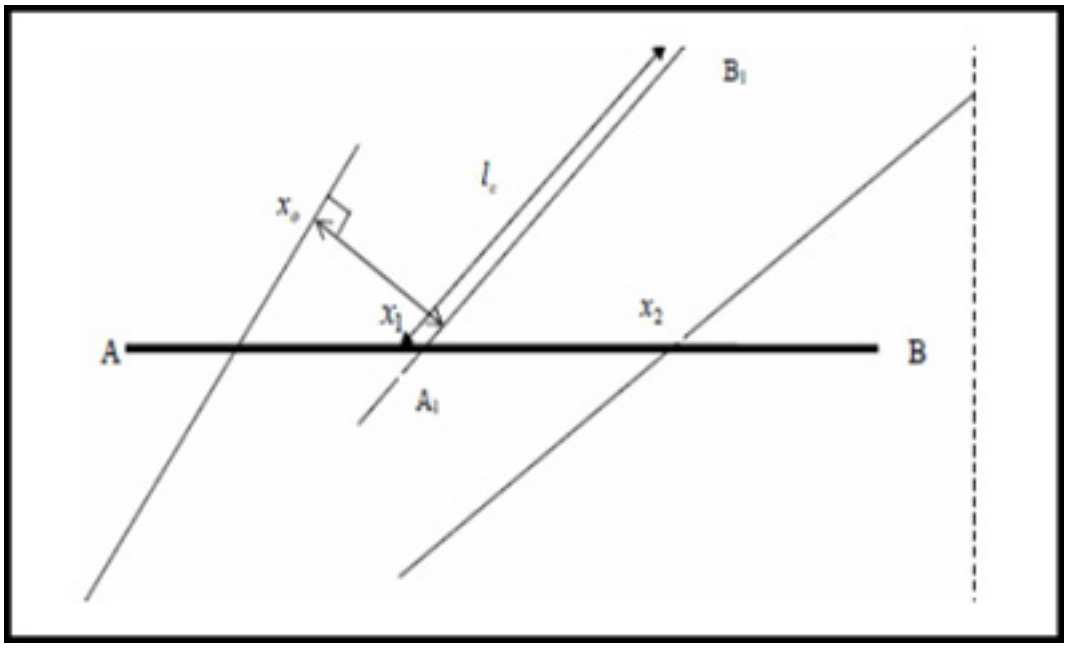


Figure 35: Illustration of joint spacing and length measurement along scanline

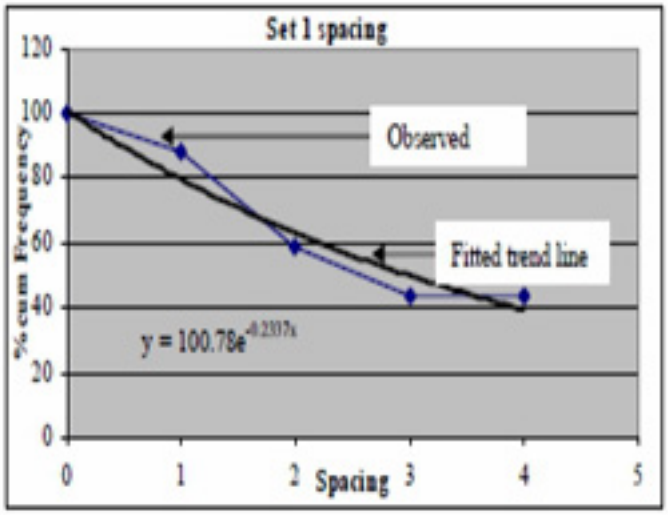
CONNECTION 10

Refer to outcomes "LEARNING OUTCOME 3.1.1.3" on page 109

The half trace-length (lc in Figure 35) was recorded as the distance between A1 and B1. The spacing and length calculations described above are performed easily on an Excel spreadsheet for numerous joint data points. Analysis of the statistical distributions of joint spacings was done using an Excel spreadsheet (Figure 36).

back

Set 1 Spacing				
Spacing (m)	Frequency (m)	Spacing greater than: (m)	Cumulative frequency	% Cumulative frequency
0 - 1	4	0	4	100
1 - 2.	10	1	14	88.2
2 - 3.	5	2	19	58.8
3 - 4.	0	3	19	44
>4	15	4	34	44



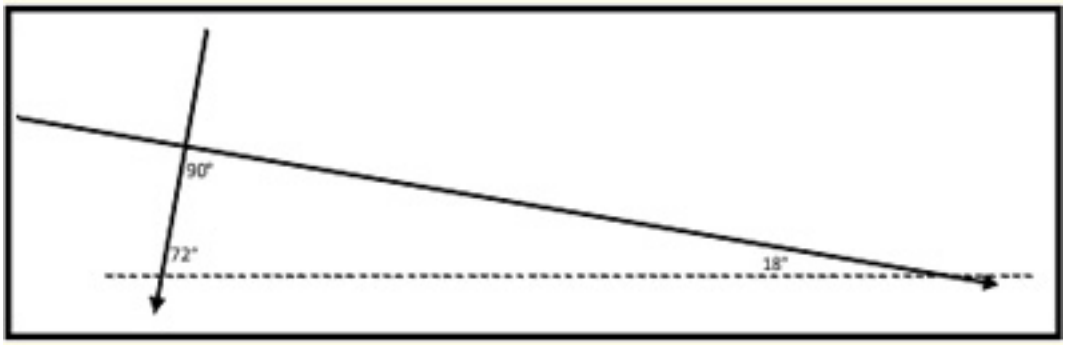


Figure 36: Set 1 Cumulative frequency

A statistical analysis of hangingwall joint properties confirms a normal distribution for orientation, and a negative exponential distribution for trace lengths.

Two joint sets are found in stope hangingwalls. Both sets are shallow dipping, but they have opposing dip directions. Stress-induced fractures in the hangingwall were more closely spaced (5-15cm) than footwall fractures (30-40cm). Combinations of steeply dipping stress induced fractures and the shallow dipping hangingwall joints are the main cause of rockfalls in stopes.

back

REFERENCE

- Page 56, Hoek, W., Bray, J.W. Rock slope engineering, 2005.

LEARNING OUTCOME 3.1.1.8

CONNECTION 11

Some of these stability analyses are explained under "LEARNING OUTCOME 3.1.1.4" on page 113 and "LEARNING OUTCOME 3.1.1.5" on page 117

DESCRIBE, EXPLAIN AND APPLY STEREONET ANALYSES

An important technique used in open pit rock engineering is the ability to use stereonet analysis to determine the probability of slope failures.

EXAMPLE 12

Define the "plunge" of a line, in 3D space, with reference to angle size, plane of measurement and directional sense.

Solution:
-90°< β <+90°. The acute angle measured in a vertical plane between a given line and the horizontal.

EXAMPLE 13

Define the "trend" of a line, in 3 D space, with reference to azimuth, direction of measurement and where measured from.

Solution:
0°< α <360°. The geographical azimuth measured in a clockwise rotation from north to the vertical plane containing the given line of plunge.

EXAMPLE 14

Define the word "strike" of a plane.

Solution:
The geographical azimuth of an imaginary horizontal line in a given inclined plane. Usually only one end is specified, e.g. 50° South-East.

EXAMPLE 15

A plane has a dip direction and dip of 352/72. What is the dip and dip direction of its normal?

Solution:
The dip must be 18° so that the one will be normal to the other (90° - 72°) and the strikes must be similar, but the dip directions must differ by 180° (352° - 180° = 172°). Therefore, the normal will be 172/18.

EXAMPLE 16

A plane has a strike of 133°and an angle of dip of 27°. What are the possible orientations of its normal?

Solution:
The strikes must be similar or 180° apart, therefore

133° or (133°+180°)
= 313°,

while the dips must be the difference between 90° and the dip of plane 1, i.e.

(90°-27°) = 63°.

EXAMPLE 17

The normal to a plane has a trend / plunge of 116/64. What is the strike of the plane?

Therefore, the normal to the plane will have orientations:

133/63 or 313/63.

Solution:
The trend is the dip direction of a line. The strike of the line would therefore be 90° from the trend. This means the strike must be (116 ± 90°) = 206° or 26°.

REFERENCE

- RRMC Paper 3-4 October 2008 Model answers.
- Chapters 3 and 4, Hoek, W., Bray, J.W. Rock slope engineering, 2005.
- Chapter 3, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.
- Chapter 1, Hoek E., Practical rock engineering, 2006.

LEARNING OUTCOME 3.1.1.9

DESCRIBE, EXPLAIN AND APPLY STEREO PAIR ANALYSES.

Rock engineers / geotechnical engineers often start a site investigation by evaluating drill cores and forget that options are available to assess the regional geology prior to 'zooming' into the detailed rock types and quality. Structural discontinuities that may affect stability of slopes are often related to large scale, regional structures and knowledge of such regional structures may be worthwhile prior to looking at local detail. Stereoscopic examination of adjacent pairs of air photographs can allow detection of surface features that could indicate the presence of underground structures.

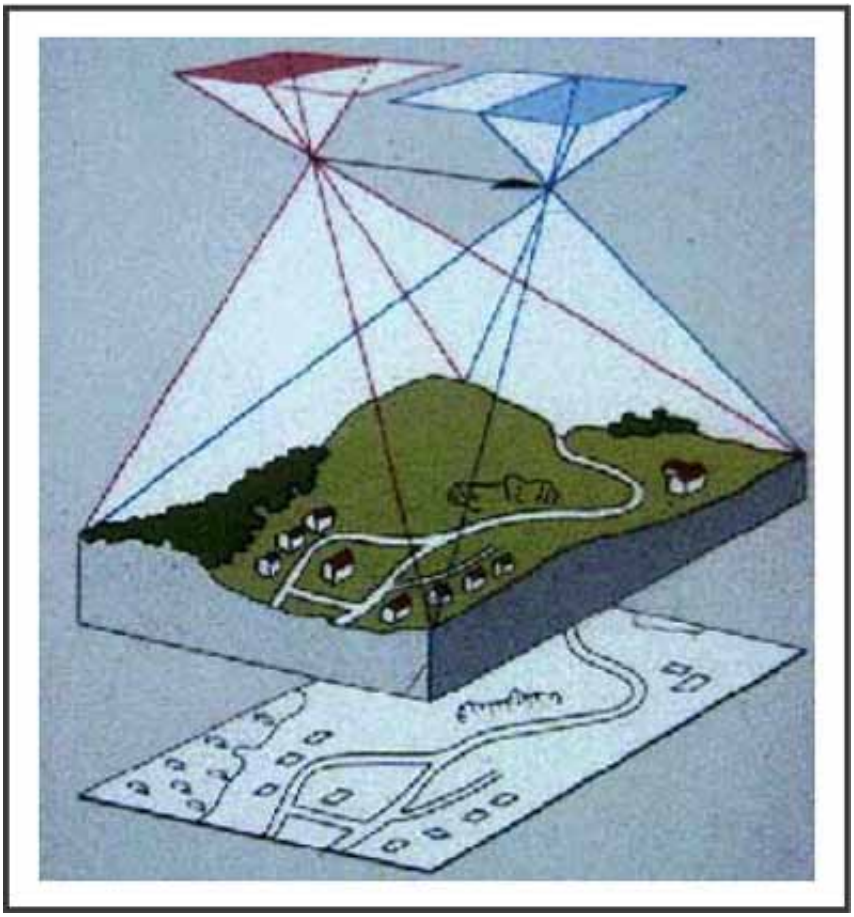


Figure 37: Principle of stereoviewing

back

The same process can be followed when photographing exposed surfaces and viewing the pairs of photographs to assist with structural mapping. A phototheodolite is used to take photographs of an exposed rock surface, which are then viewed with a stereocomparator or stereoviewer. The resulting three-dimensional model of the rock surface allows X, Y and Z measurements of co-ordinates of certain points to a high degree of accuracy (information suggest a 3cm accuracy when photographs were taken at a distance of 1.5km from the face).

A method to orientate a pair of aerial photographs for best three-dimensional viewing is outlined below:

- Arrange the selected pair of photos in such a way that the shadows on them generally appear to fall toward the viewer. It is also desirable that the light source enters the side away from the observer during the study of the photographs.

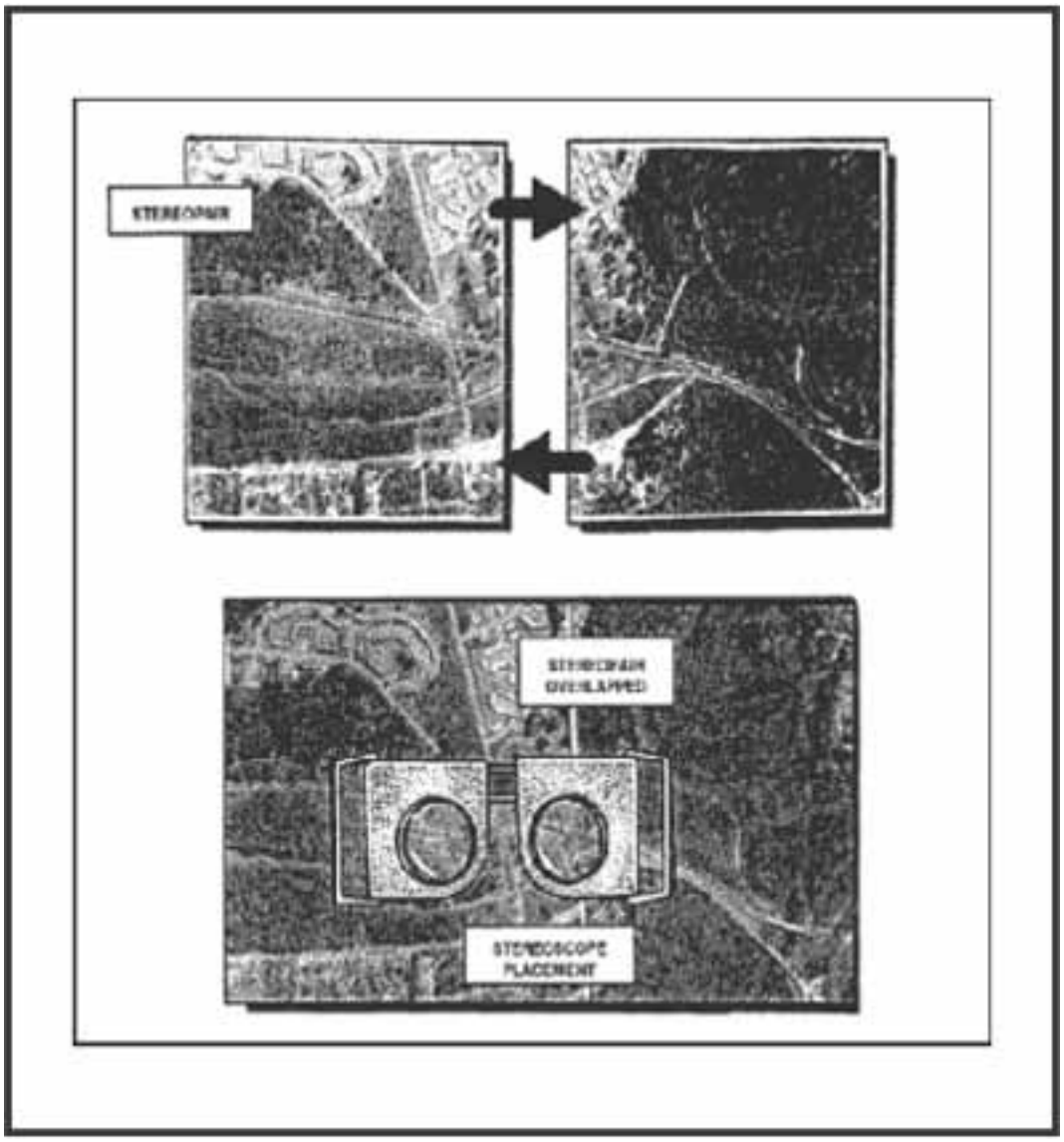


Figure 38: Placement of stereoscope over stereopair

- Place the pair of photographs on a flat surface so that the detail on one photograph is directly over the same detail on the other photograph.
- Place the stereoscope over the photographs so that the left lens is over the left photograph and the right lens is over the right photograph.

back

back

- Separate the photographs along the line of flight until a piece of detail appearing in the overlap area of the left photograph is directly under the left lens and the same piece of detail on the right photo is directly under the right lens.
- With the photograph and stereoscope in this position, a three-dimensional image should be seen. A few minor adjustments may be necessary, such as adjusting the aerial photographs of the stereoscope to obtain the correct position for your eyes. The hills appear to rise and the valleys sink so that there is the impression of being in an aircraft looking down at the ground, see Figure 38.
- The identification of features on photographs is much easier and more accurate with this three-dimensional view.

When applied to a slope face, photographs can be taken as shown in Figure 39, ensuring overlapping of photographs as also discussed above. The same process is followed to align and view the overlapping photographs using a stereoviewer.

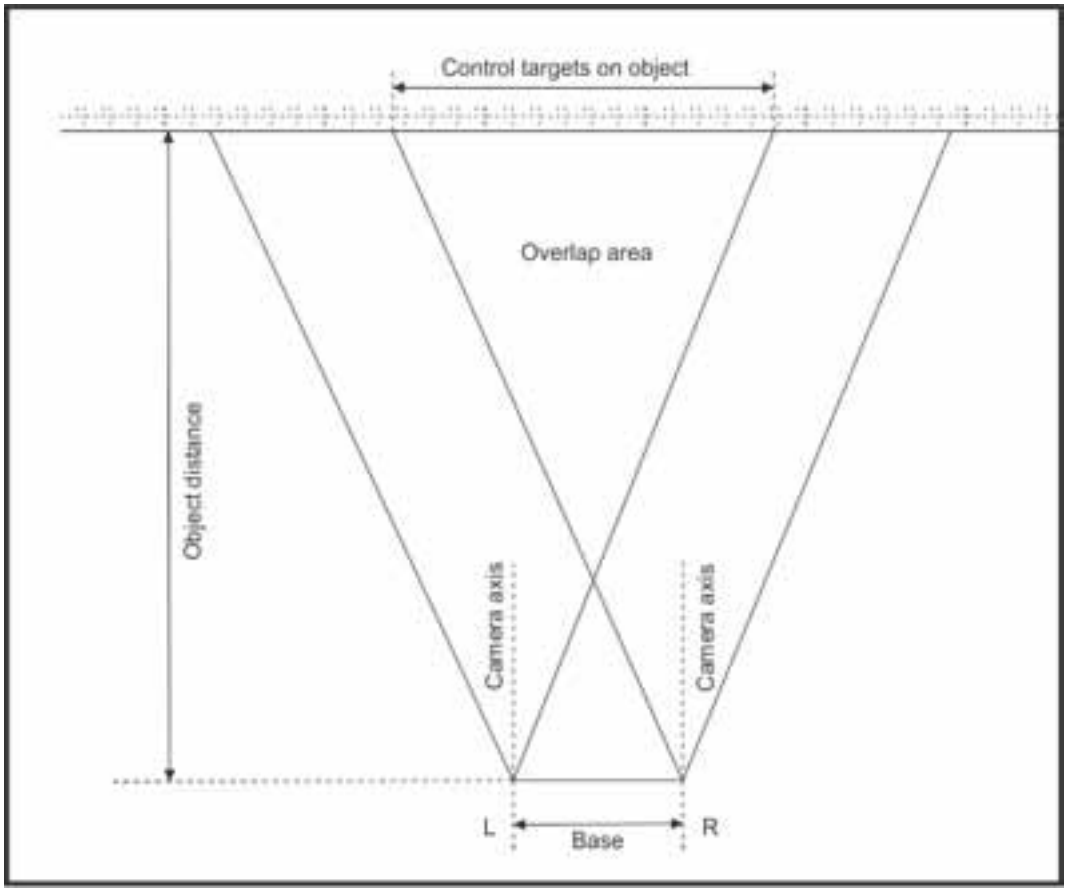


Figure 39: Layout for photographs for stereoviewing

The identification of features on a photograph is not difficult if the following facts are remembered. On the view that is presented by the photograph, objects do not look familiar and may even appear distorted. Before a photograph can be studied or used for identification of features, it must be orientated. Orientation for study consists of rotating the photograph so that the shadows on the photograph point towards the user so that he faces a source of light. This places the source of light, the object and its shadow in a natural relationship. Failure to orient a photograph properly may cause the height or depth of an object to appear reversed. For example, a mine or quarry may appear to be a hill instead of a depression.

back

back

The requirement for stereovision can be satisfied by overlapping photographs if one eye sees the object on one photograph and the other eye sees the same object on another photograph. While this can be done after practice with the eyes alone, it is much easier if an optical aid is used. These optical aids are known as stereoscopes. There are many types of stereoscopes, including:

- Pocket stereoscope, see Figure 40. The pocket stereoscope, sometimes known as a lens stereoscope, consists of two magnifying lenses mounted in a metal frame. Because of its simplicity and ease of carrying, it is the type used most frequently by military personnel.

Figure 40: Pocket stereoscope

- Mirror stereoscope, see Figure 41. The mirror stereoscope is larger, heavier, and more subject to damage than the pocket stereoscope. It consists of four mirrors mounted in a metal frame.

back

back

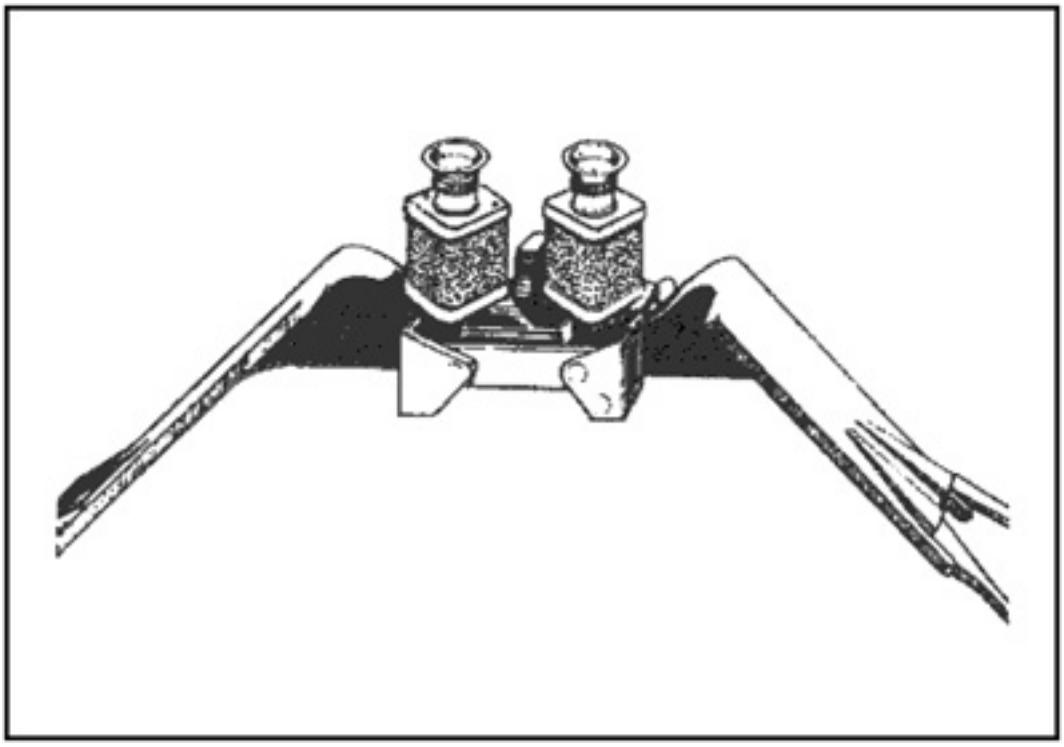


Figure 41: Mirror stereoscope

INTERESTING INFO

The practice of stereoviewing by geotechnical engineers is based on a number of applications in the military, agriculture and town planning fields. Aerial photography and photogrammetry are used for mapping and other purposes in the following situations:

- making topographic maps (most topographic maps are made by photogrammetry)
- large-scale maps for subdivision design
- mapping of the land component associated with hydrographic surveys
- long construction projects to make detailed maps or digital terrain models to enable engineering design
- collecting data for Geographic Information Systems
- environmental mapping and monitoring
- providing evidence in court on what the situation was on the ground at a point in the past
- calculating volumes of pits, quarries and stockpiles using the digital terrain models
- recording and monitoring natural phenomena such as floods, volcanoes, and the like
- recording and monitoring farming and other activity

These users make use of aerial photographs taken with a very precise camera mounted in an aircraft. The photographs are taken at intervals along a run so that the photographs overlap by about two thirds. The precise locations of control points visible in the photographs are measured on the ground so that the photographs can be oriented and scaled in relation to the true dimensions on the ground. In some modern cameras, a precise Global Positioning Systems (GPS) unit is integrated with the shutter to give the three-dimensional coordinates of control points in the photographs themselves without ground survey.

back

back

Other adjustments have to be made to the photographs to account for the tilts of the aircraft as it flies along and the blurring that would occur due to the forward motion of the aircraft at low altitudes.

When the control has been computed and sufficient points of known co-ordinates are available, two overlapping photographs are placed in a photogrammetric plotter. Through the use of complex optics, the area where the overlapping portions are superimposed on each other can be studied in three dimensions.
The principal advantage of aerial survey and photogrammetry include:

- the ability to cover large areas at a time and take advantage of economies of scale
- as well as maps, the process produces photographs that are easy for laymen to interpret
- it is possible to map areas that are inaccessible from the ground
- if appropriate historical photography is available, it is possible to map the terrain as it was at a particular time, even though it is now changed

In taking aerial photographs, the aircraft flies over the area to be photographed taking a series of pictures, each of which overlaps the photograph preceding it and the photograph following it so that an unbroken coverage of the area is obtained, see Figure 42 and 43

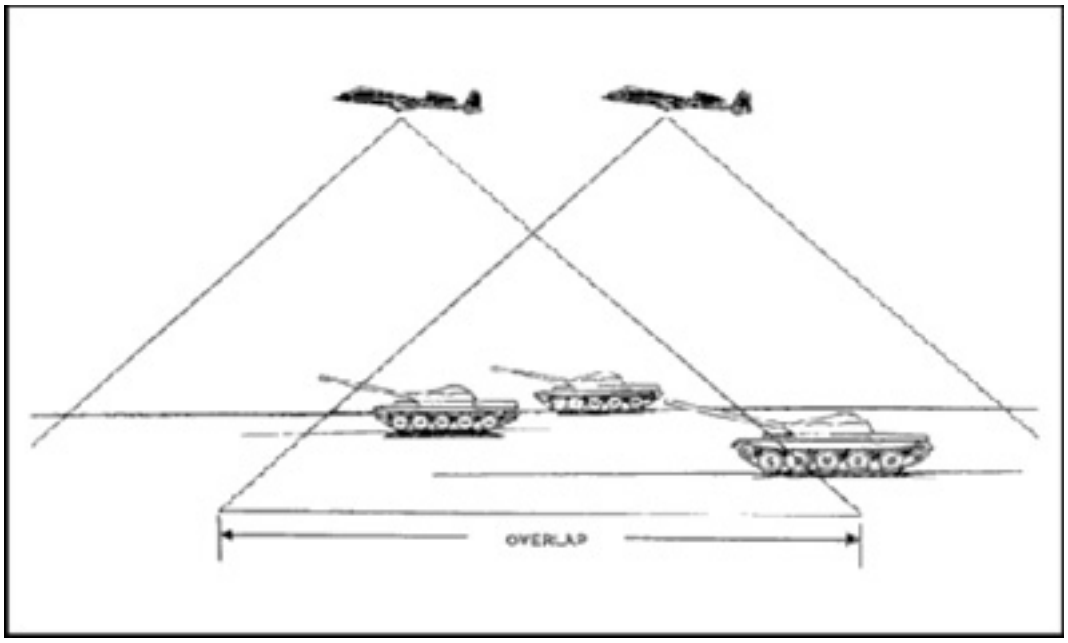


Figure 42: Photographic overlap

back

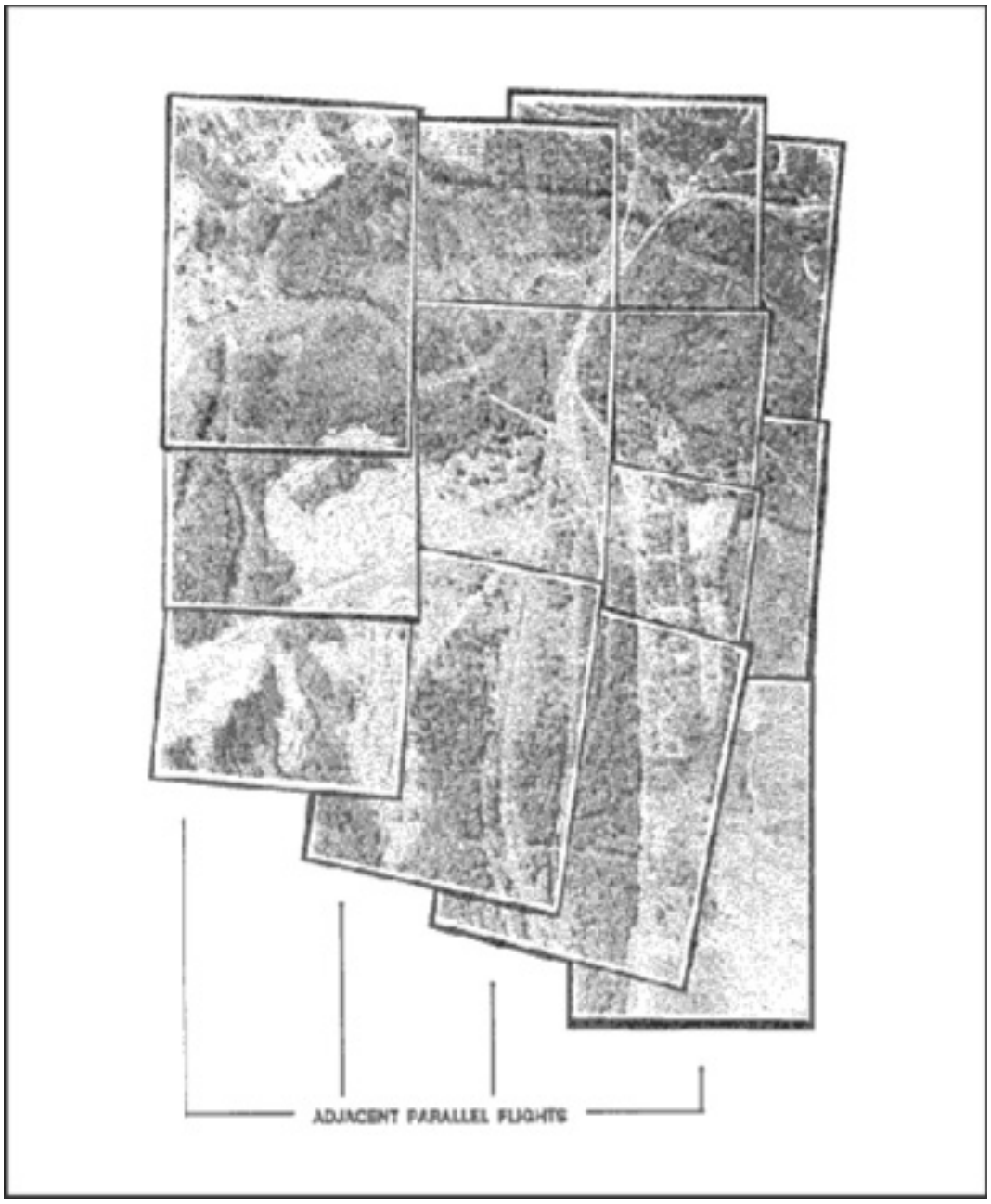


Figure 43: Side lap

REFERENCE

- http://www.map-reading.com/apident.php
- Hoek, W., Bray, J.W. Rock slope engineering, IMM, 2005.

LEARNING OUTCOME 3.1.2.1

ROCK MASS CLASSIFICATION

DESCRIBE AND EXPLAIN HOW ROCK QUALITY DESIGNATION (RQD) MAY BE DETERMINED FROM BOREHOLE CORE

The standard rock quality designation is recorded, both in meters and as a percentage, which is calculated as follows:

RQD – Rock Quality Designation

back

RQD is defined as the ratio of the cumulative length of sticks of NX size core more than 100mm in length in a drill run to the total length of the drill run, see Figure 44:

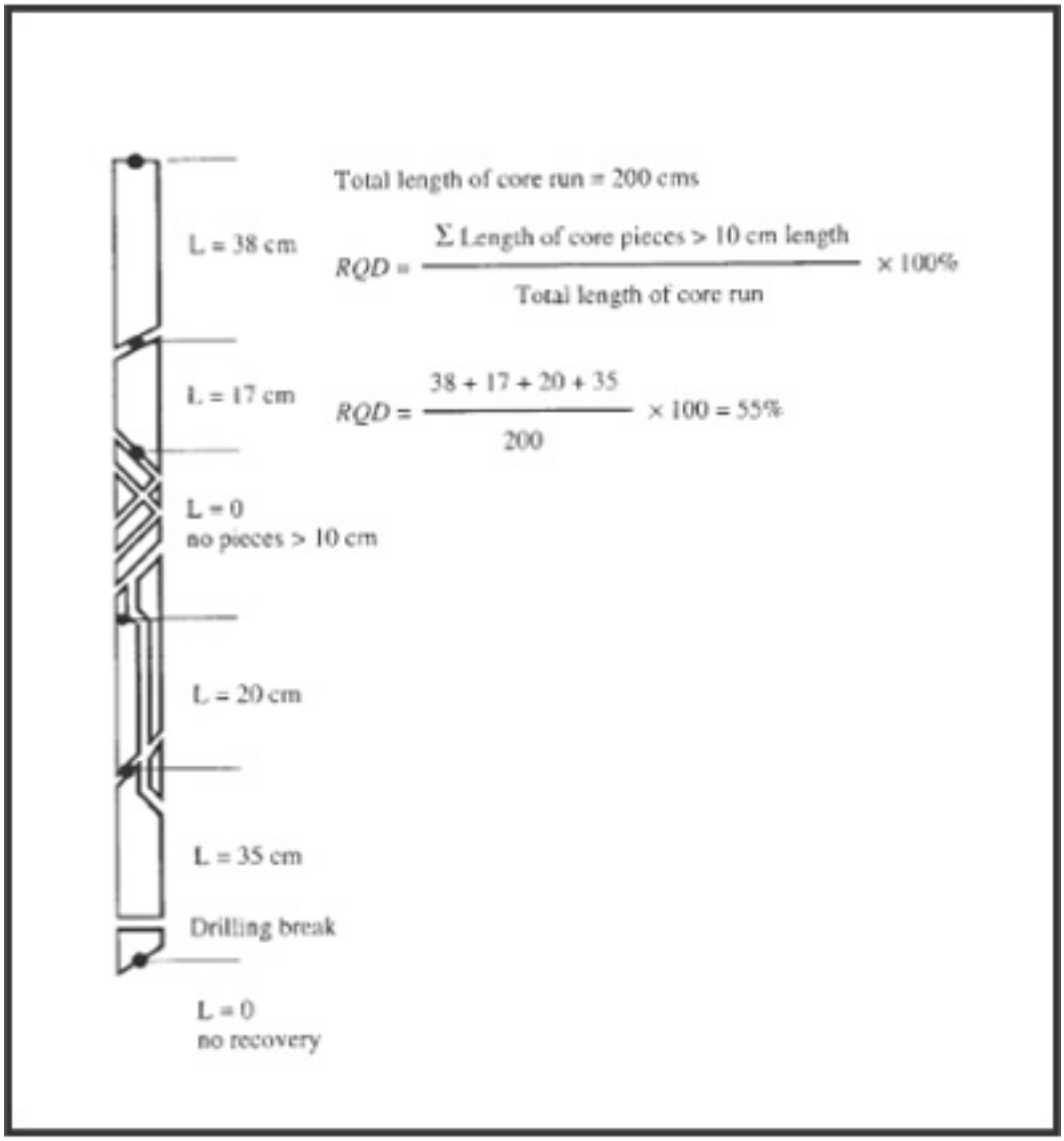


Figure 44: RQD calculation

$$RQD\% = \frac{\text{Total Length of core} \geq 100\text{mm} \times 100}{\text{Length of core run}}$$

Judgement must be exercised for poorly-orientated boreholes. For example, holes parallel to bedding in a sedimentary deposit may indicate very high values of RQD, whereas holes across bedding in the same rock may indicate much lower RQDs.

Deere suggested that the rock quality could be categorised as follows:

RQD	Rock quality
< 25%	Very poor
25-50%	Poor
50-75%	Fair
75-90%	Good
90-100%	Very good

Table 3: Deere's rock quality categories

back

INTERESTING INFO

For RQD determination, the ISRM recommends a core size of at least NX (54.7mm). Deere (1988) suggested that other core sizes are applicable providing proper drilling techniques are utilised that do not cause excess core breakage or poor recovery. The recommended run length was suggested by Deere (1988) to be no greater than 1.5m.

REFERENCE

- Page 18, Hoek, E., Brown, E.T. Underground excavations in rock, 1980.
- Page 54, 69, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.
- Page 52, Budavari S., Rock mechanics in Mining Practice, 1983.

LEARNING OUTCOME 3.1.2.2

DESCRIBE AND EXPLAIN HOW ROCK QUALITY DESIGNATION (RQD) MAY BE DETERMINED FROM THE IN-SITU ROCKMASS

RQD can be estimated from inspection of exposed rock surfaces by determining the number of unhealed joint planes per m^3 of rock. This may be done by counting the relevant number of joint planes (excluding blast fractures) that cross a 2 to 3m length of tape held against the excavated wall. The number of joint planes divided by the relevant sample length gives the number of joints per meter. This process is then repeated for two additional orientations of the scan line. The sum of these three values gives Jv, the number of joints per m^3, and hence the RQD from the equation (Barton et al 1974) can be approximated by:

BRIDGING GAP 2

∑ refer to "Signs & Symbols" on page 2

$$RQD = 115 - 3.3\,Jv$$

Where Jv= ∑ joints / m for each set present

Priest and Hudson (Brady and Brown, 2006) estimated the RQD from discontinuity spacings measured from core logging or exposed surface mapping. They suggested the relationship to be:

$$RQD = 100e^{-0.1\lambda}(0.1\lambda + 1)$$

where λ = frequency of discontinuities (joints/m)

= 1/average spacing

NOTE

Where RQD is reported or measured as less than 10, a nominal value of 10 is used to evaluate Q. RQD intervals of 5, giving 100, 95, 90 … 10 are sufficiently accurate.

REFERENCE

- Page 18, Hoek, E., Brown, E.T. Underground excavations in rock, 1980.
- Page 54, 69, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.
- Page 52, Budavari S., Rock mechanics in Mining Practice, 1983.
- Page 182, Hutchinson D.J., Diederichs M.S., Cable bolting in underground mines, 1996.

back

LEARNING OUTCOME 3.1.2.3

DESCRIBE AND EXPLAIN THE FORMULATION AND COMPONENTS OF BARTON'S Q SYSTEM OF ROCKMASS CLASSIFICATION

Barton et al. (1974)) developed this tunnelling quality index system from civil engineering case histories in which all the components of the character of the rock mass were included. The classification system and support recommendations made by them were made from 212 case studies. The original parameters in the Q rating system have not been changed, but the stress reduction factor (SRF) has been altered by Grimstad and Barton in 1993.

The original parameters in the Q system consist of:

- RQD
- Number of joint sets
- Joint alteration
- Joint water conditions
- Stress reduction factor.

The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

$$Q = RQD/J_n \times J_r/J_a \times J_w/SRF$$

where RQD is the Rock Quality Designation,
J_n the joint set number,
J_r the roughness number,
J_a the alteration number,
J_w the water reduction factor and
SRF a stress reduction factor.

The rock tunnelling quality index Q can therefore be considered to be a function of only three parameters that are measures of:

- Block size (RQD/Jn)
- Inter-block shear strength (Jr/ Ja)
- Active stress (Jw/SRF)

and where

- the first quotient (RQD/Jn) represents the structure of the rock mass and therefore a measure of the block or particle size.
- The second quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or filling materials. In the case where joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Where no rock wall contact exists, the conditions are extremely unfavourable to tunnel stability.
- The third quotient (Jw/SRF) consists of two stress parameters and can be regarded as a total stress parameter describing the 'active stress'.

SRF is a measure of:

- loosening load in the case of an excavation through shear zones and clay-bearing rock,
- rock stress in competent rock, and
- squeezing loads in plastic incompetent rocks.

back

back

The parameter Jw is a measure of water pressure, which can affect the shear strength of joints due to a reduction in effective normal stress.

Table 4 shows Barton's clasification for various Q values

Q	Group	Classification
10-40 40-100 100-400 400-1000	1	Good Very Good Extremely Good Exceptionally Good
0.10-1.0 1.0-4.0 4.0-10.0	2	Very Poor poor Fair
0.001 - 0.01 0.01 - 1.0	3	Exceptionally poor Extremely Poor

Table 4: Classification of rock mass based on Q-values (Barton et al. 1974)

Table 5 (Barton et al. 1974) provides the guidelines for the classification of individual parameters used to obtain the Tunnelling Quality Index Q for a specific rock mass.

Description	Value	Notes
1. ROCK DESIGNATION A. Very poor B. Poor C. Fair D. Good. E. Excellent.	**QUALITY** 0 - 25. 25 - 50 50 - 75 75 - 90 90-100	1. WhereRQD is reported or measured as ≤ 10 (including 0) a nominal value of 10 is used to evaluate Q. 2. RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently 90 - 100 accurate
2. JOINT SET NUMBER A. Massive, no or few joints B. One joint set C. One joint set plus random D. Two joint sets E. Two joint sets plus random F. Three joint sets G. Three joint sets plus random H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc. J. Crushed rock, earthlike	**J_n** 0.5 - 1.0 2 3 4 6 9 12 15 20	1. For intersections use ($3.0 \times J_n$) 2. For portals use ($2.0 \times J_n$)
3. JOINT ROUGHNESS NUMBER *a. Rock wall contact* *b. Rock wall contact before 10 cm shear* A. Discontinuous joints B. Rough and irregular, undulating C. Smooth undulating D. Slickensided undulating E. Rough or irregular, planar F. Smooth, planar G. Slickensided, planar *c. No rock wall contact when sheared* H. Zones containing clay minerals thick enough to prevent rock wall contact J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	**J_r** 4 3 2 1.5 1.5 1.0 0.5 1.0 (nominal) 1.0 (nominal)	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. 2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.
Description	**Value**	**Notes**

back

back

4. JOINT ALTERATION NUMBER	J_a	**φr degrees (approx.)**	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75	25 - 35	1. Values of φr, the residual friction angle,are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 30	
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	20 - 25	
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	8 - 16	
E. Softening or low-friction clay mineral coatings i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	25 - 30	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	16 - 24	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	12 - 16	
H. Medium or low over-consolidation, softeningclay mineral fillings (continuous < 5 mm thick)	8.0	6 - 12	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0-12.0		
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay conditions)	8.0		
M.	8.0-2.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, smalL clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0-3.0		
P. & R. (see G.H and J for clay conditions)	6.0-24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rockwith unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates;increase Jw if drainage installed.
D. Large inflow or high pressure0.332.5 - 10.0			
E. Exceptionally high inflow or pressure at blasting,decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formationare not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
Description		**Value**	**Notes**

back

back

6. STRESS REDUCTION FACTOR a. *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated*	**SRF**			
A. Multiple occurrences of weakness zones containing clay orchemically disintegrated rock, very loose surrounding rock any depth)	10.0			1. Reduce these values of SRF by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50 m)	5.0			
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m)	2.5 7.5			
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	5.0			
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)	2.5			
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)	5.0			
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)				
6. STRESS REDUCTION FACTOR b. Competent rock, rock stress problems	σ_c/σ_1	$\sigma_t\sigma_1$	**SRF**	2. For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to 0.6 σ_c and $0.6\sigma_t$, where
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	σ_c = unconfined compressive strength
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10-20	σ_t = tensile strength (point load) and
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				σ_3 are the major and minor principal stresses.
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10-20	3. Few case records available where depth of crown below surface is less than span width. SuggestSRF increase from 2.5 to 5 for such cases (see H).
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10-15	

7. ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (Q), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of jointsper metre for each joint set are added. A simple relationship can be used to convert this number to RQD for the case of clay freerock masses:
   RQD = 115 - 3.3Jv (approx.), where
   J_v = total number of joints per m^3 (0 <RQD < 100 for 35 >J_v > 4.5).

2. The parameterJn representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.

3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.

4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.

5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in-situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Table 5: Classification of parameters for the Tunnelling Quality Index Q (Barton et al. 1974)

back

REFERENCE

- Page 27, Hoek, E., Brown, E.T. Underground excavations in rock, 1980.
- Page 53, Budavari S., Rock mechanics in Mining Practice, 1983.
- Page 69, Ryder J.A., Jager A.J., A Textbook on rock mechanics for tabular hard rock mines, 2002.
- Page 218, 314, Jager A.J, Ryder J.A., A Handbook on rock engineering practice on hard rock tabular mines, 1999.
- Page 191, Hutchinson D.J., Diederichs M.S., Cable bolting in underground mines, 1996.

LEARNING OUTCOME 3.1.2.4

DESCRIBE AND EXPLAIN THE FORMULATION AND COMPONENTS OF BIENIAWSKI'S RMR SYSTEM OF ROCKMASS CLASSIFICATION

Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification for jointed rock mass or the Rock Mass Rating (RMR) system.

The following parameters are used to classify a rock mass using the RMR (1989) system:

- Uni-axial compressive strength of rock material
  - UCS values are normally used, but when applying the Point Load Index values, care must be taken when working with very low strength rocks;
- Rock Quality Designation (RQD)
  - Deere's RQD rating as normally applied;
- Spacing of discontinuities
  - The term joints includes all weakness planes such as bedding, joints and faults;
- Condition of discontinuities
  - accounts for separation, roughness, continuity, wall condition and infilling;
- Groundwater conditions
  - Account for water inflow and the impact of water on rock mass competency.

The rating obtained by summing the ratings for the abovementioned five parameters is then adjusted to account for the orientation of the joints.

The Rock Mass Rating system guidelines are presented in Table 4, on page 61 of Hoek's notes, giving the ratings for each of the parameters listed above. At the same time, guidelines to categorise the quality of the rock mass and some assistance in applying the results (such as stand-up time, cohesion and friction angle estimates) are also available from Table 4.

A relationship has been found between RMR and Q as follows (Bieniawski, 1989):

$$RMR = 9 \ln Q + 44$$

NOTE

Since updated versions of the rock mass ratings are continuously provided, it is critical to note which version is used, while it is advised to remain aware of the latest alterations to current systems.

REFERENCE

- Chapter 3 Rock mass classifications – Hoek's notes.
- Page 218, 314, Jager A.J. Ryder J.A., A Handbook on rock engineering practice on hard rock tabular mines, 1999.
- Page 66, 366, Ryder J.A., Jager A.J., A Textbook on rock mechanics for tabular hard rock mines, 2002.
- Page 78, 83, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.
- Page 14, Hoek, E., Brown, E.T. Underground excavations in rock, 1980.
- Page 11, Van der Merwe J.N, Madden B.J., Rock engineering for coal mining, 2002.
- Page 27, Stacey T.R., Best practice rock engineering for Other Mines, 2001.
- Chapter 3, Hoek E., Practical rock engineering, 2006.
- Page 186, Hutchinson D.J., Diederichs M.S., Cable bolting in underground mines, 1996.

LEARNING OUTCOME 3.1.2.5

DESCRIBE AND EXPLAIN THE FORMULATION AND COMPONENTS OF LAUBSCHER'S MRMR SYSTEM OF ROCKMASS CLASSIFICATION

Laubscher developed the Mining Rock Mass Rating (MRMR) system in 1975, which was modified by Laubscher and Taylor (1976). The intention of this 'modified' or 'mining' rock mass rating was to cater for the differences in underground mining and civil excavations. This system takes into account the same parameters as the Geomechanics system, but combines the groundwater and joint conditions, resulting in the following four parameters prior to modification due to weathering, the stress environment, excavation orientation and excavation method:

- rock material strength (UCS)
- RQD
- joint spacing
- joint condition and ground water.

Rating values for each of these parameters are given in Table 7 below. Adjustments for the joint condition and groundwater parameter in Table 8 are cumulative.

back

EXAMPLE 18

A dry straight joint with a smooth surface and fine soft-sheared joint filling would have a minimum rating of 40 x 70% x 60% x 60% = 10.1

The mining rock mass rating MRMR value is obtained by summing the individual four parameter ratings and must lie between zero and 100.

Parameter		Range of values										
1	RQD	100-97		96-84	83-71	70-56	55-44	43-31	30-17	16-4	3-0	
	Rating (=RQDx15/100)	15		14	12	10	8	6	4	2	0	
2	UCS (MPa)	185	1 8 4 - 165	1 8 4 - 165	1 6 4 - 145	1 4 4 - 125	1 2 4 - 105	104-85	84-65	44-25	24-5	4-0
	Rating	20	18	16	14	12	10	8	6	4	2	0
3	Joint spacing	Refer to Figure 46										
	Rating	25						0				
4	Joint condition including groundwater	Refer to Table 8										
	Rating	40						0				

Table 7: MRMR rating system (Stacey, 2001)

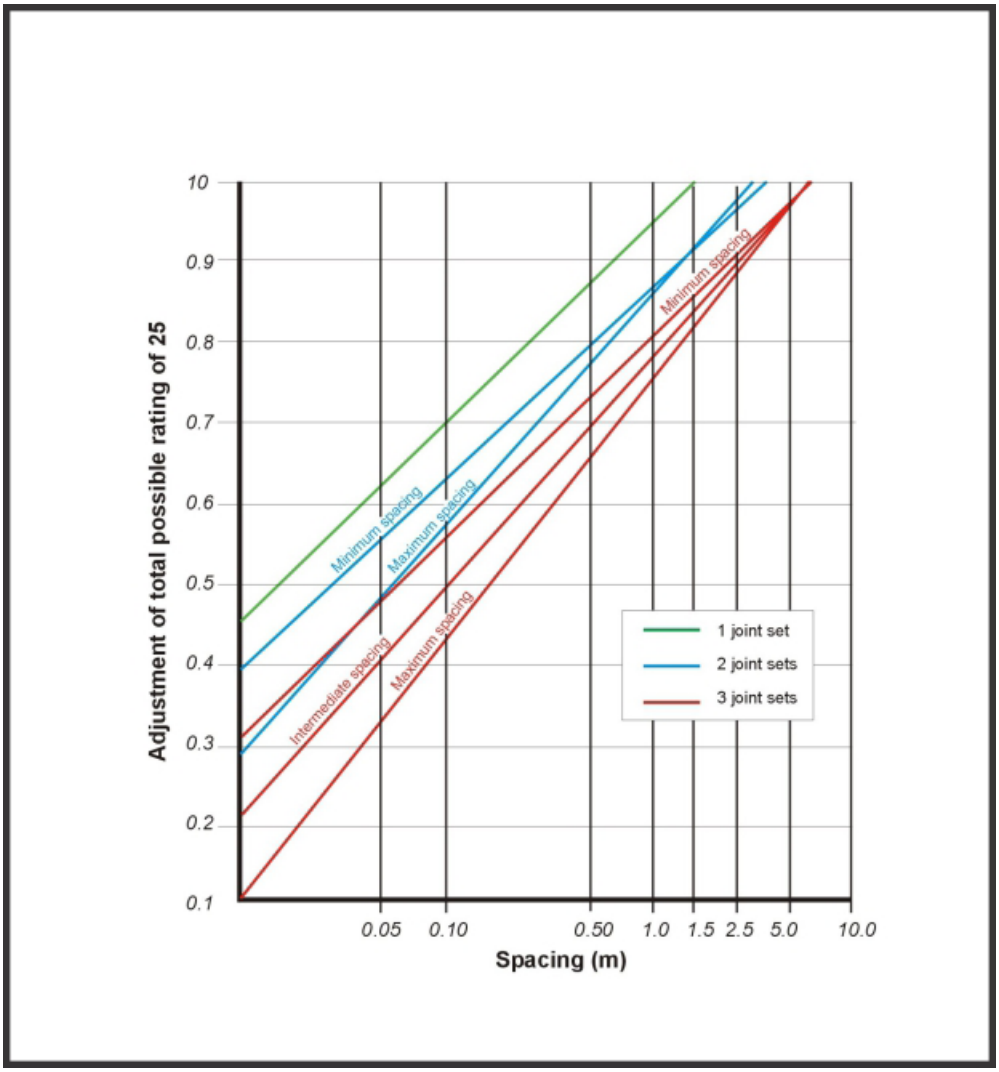


Figure 45: Joint spacing rating for MRMR (Stacey, 2001)

back

Parameter	Description		Dry conditions	Wet conditions		
				Moist	Moderate pressure 25-125 1/mm	Severe pressure >125 1/min
A Joint expression (large-scale irregularities)	Wavy	Multi-directional	100	100	100	95
		Uni-directional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint expression (small-scale irregularities or roughess	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
C Joint wall alteration zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D. Joint filling	No fill – surface staining only		100	100	100	100
	Non softening and sheared material (clay or talc free	Coarse sheared	95	90	70	50
		Medium sheared	90	85	65	45
		Fine sheared	85	80	60	40
	Soft sheared material (e.g. talc)	Coarse sheared	70	65	40	20
		Medium sheared	65	60	35	15
		Fine sheared	60	55	30	10
	Gouge thickness <amplitude of irregularity		40	30	10	
	Gouge thickness <amplitude of irregularity		20	10	Flowing material 5	

Table 8: Joint condition and water rating for MRMR (Stacey, 2001)

Adjustments can then be applied to the MRMR value obtained above to take account of weathering of the rock mass, joint orientation relative to the excavation, mining-induced stresses and blasting effects:

- A weathering adjustment is relevant when rock types occur that are susceptible to deterioration over time (Table 9).

Rate of weathering and adjustments (%)					
Description of weathering extent	6 months	1 year	2 years	3 years	4 + years
Fresh	100	100	100	100	100
Slightly	88	90	92	94	96
Moderately	82	84	86	88	90
Completely	54	56	58	60	62
Residual soil	30	32	34	36	38

Table 9: MRMR weathering adjustment (Stacey, 2001)

back

EXAMPLE 19

If the rock will weather to a residual soil in six months, the adjustment is 30%; if it will be only slightly weathered after three years, the adjustment is 94%.

- The orientations of the joints have a significant effect on excavation stability.
  - The joint orientation adjustment depends on the orientations of the joints with respect to the vertical axis of the block (Table 10).
  - A shear zone or a fault can have a significant influence on stability and adjustments with respect to the dip of the feature relative to the development, are

    0 to 15 – 76%;
    15 to 45 – 84%;
    45 to 75 – 92%.

EXAMPLE 20

If two of the four joints that form the block have orientations away from the vertical (less than 90°), the adjustment to the rating is 85%.

Number of joints defining the block	**Adjustment (%)** Number of faces inclined away from the vertical				
	70	75	80	85	90
3	3	-	2	-	-
4	4	3	-	2	-
5	5	4	3	2	1
6	6	5	4	3	2 or 1

Table 10: MRMR joint orientation adjustment (Stacey, 2001)

EXAMPLE 21

Average confinement is subjectively selected to suggest an adjustment of 100%.

- The stresses acting around the mining excavations have an effect on the stability. The suggested adjustments for mining-induced stresses are based on judgement where:
  - Good confinement enhances stability and the maximum positive adjustment is 120%;
  - Poor confinement, when numerous, closely-spaced joint sets are present, does not promote stability, and the maximum negative adjustment is 60%.

EXAMPLE 22

If blasting practices are viewed to be poor, the rating is downgraded by 80%.

- The quality of blasting has an influence on the fracturing and loosening of the rock walls and adjustments take cognisance of the excavation method as in Table 11.

Excavation technique	**Adjustment (%)**
Boring	100
Smooth wall blasting	97
Good conventional blasting	94
Poor blasting	80

Table 11: MRMR excavation technique adjustment (Stacey, 2001)

The above adjustments are all cumulative and are applied as multipliers to the MRMR calculated from Table 7, Table 8 and Figure 48.

EXAMPLE 23

For example, if the MRMR value was 72, adjustments of 94% for weathering, 85% for joint orientation, 100% for mining-induced stresses, and 80% for blasting effects, the adjusted MRMR is now given by:

Adjusted MRMR

$$= 72 \times 0.94 \times 0.85 \times 1.0 \times 0.8$$
$$= \underline{46}$$

The MRMR rating can now be utilised to estimate the rock mass strength (RMS). The RMS is determined from the UCS of the rock and the MRMR of the rock mass, using the relationship:

back

RMS = UCS x (MRMR – RUCS)/100

where RUCS is the rating value in Table 7 that corresponds to the appropriate UCS of the rock material.

The design rock-mass strength (DRMS) is the strength of the unconfined rock mass in a specific mining environment where the mining exposes the rock surface and the stability of the zone that surrounds the excavation is under scrutiny. The extent of this zone depends on the size of the excavation and any instability that might occur will commence on the exposed rock surface. Adjustments, which relate to that mining environment, are applied to the RMS to give the DRMS and since excavation surface instability is suggested, it should be adjusted for weathering, orientation, and blasting. As the DRMS is in megapascals, it can be related to the mining-induced stresses:

EXAMPLE 24

If weathering = 85%,
orientation = 75%,
blasting = 90%,

gives a total adjustment of

(0.85*0.75*0.9) 57%

and the

RMS = 50 (as determined from the MRMR), the

DRMS = 50 x 57%

= 29 MPa.

Therefore, the rock mass has an unconfined compressive strength of 29 MPa.

REFERENCE

- Page 31, Stacey T.R., Best practice rock engineering for Other Mines, 2001.
- Page 48, Hoek E., Practical rock engineering, 2006.
- Laubscher, D.A. A geomechanics classification system for the rating of rock mass in mine design, SAIMM, 1990.

LEARNING OUTCOME 3.1.2.6

COMPARE AND CONTRAST THESE THREE ROCKMASS CLASSIFICATION SYSTEMS AND THEIR RESPECTIVE APPLICATIONS

In Table 12 below, a comparison of the parameters used as input to various rock mass classification systems is shown (original table was adapted to include the adjusted MRMR). The comparison between the input parameters suggests that:

- Systems evaluate very similar parameters, even though the method of evaluation varies and
- That some correlation between the rating systems seems to be possible.

back

Parameters	RMR	MRMR	Adjusted MRMR	Q
Uni-axial compressive strength	X	X	X	X
Number of joint sets				X
Joint spacing	X	X	X	X
Joint strength	X	X	X	X
State of stress		X	X	X
Groundwater	X	X	X	X
Blast damage			X	

Table 12: Comparison of input parameters used in various rockmass rating systems (Edelbro)

Based on case studies, several authors have suggested that a correlation between the RMR system and the Q system can be expressed in the following forms;

- RMR = 9.0 ln Q + 44 (Bienawski, 1976)
- RMR = 5.9 ln Q + 43 (Rutledge, 1987)
- RMR = 5.4 ln Q + 55.2 (Moreno, 1980)
- RMR = 5.0 ln Q + 60.8 (Cameron, 1981)
- RMR = 10.5 ln Q + 41.8 (Abad, 1984)

These equations and their relation to each other are shown in Figure 47. It is evident that the equations above do not provide a unique correlation between RMR and Q. Depending on the overall intact rock and discontinuity properties and spacing, different relationships between Q and RMR can be expected. Another difference between RMR and Q is evident in the assessment of joint spacing. If three or more joint sets are present and the joints are widely spaced, it is difficult to get the Q system to reflect the competent nature of a rock mass, as can be seen at low Q rating values in Figure 46.

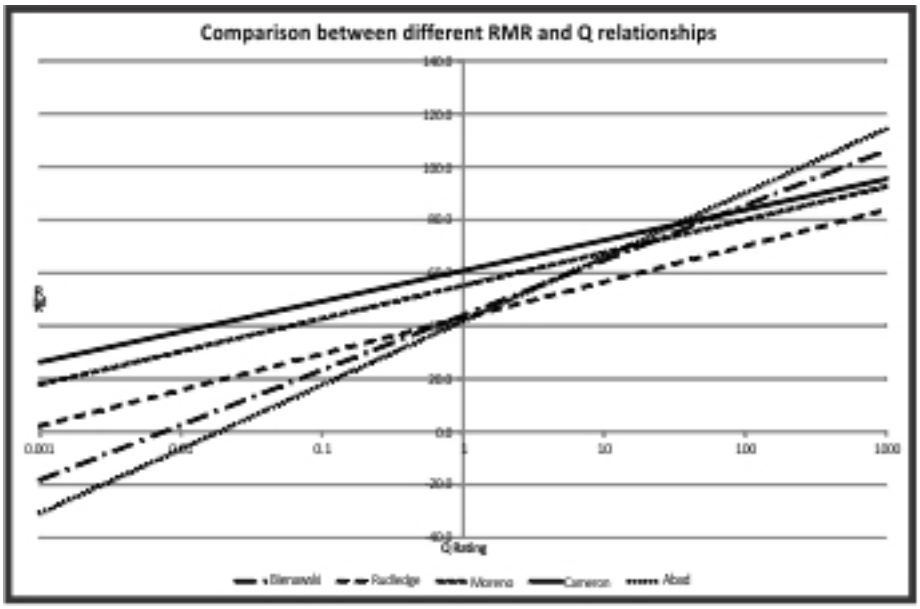


Figure 46: Comparison of different RMR and Q relationships

This relationship is also significantly dependent on logging quality and experience of the logger in selecting the most appropriate ratings. It is suggested that application of this relationship should be with care and cognizance of the impact of the logging exercise. This concern is highlighted when Figure 47 indicates that the relationships suggested above

back

back

fall apart at very low Q ratings or poor rock mass conditions (huge variation in RMR for same Q values) and very high Q ratings or good rock mass conditions (RMR > 100). However, if transformation from one system to another is required, it appears as if Bienawski's relationship is favoured.

The following also serves as comparison of the application of these systems:

- The Q system:
  - was developed from civil engineering case histories;
  - relates to rock masses where stability is governed mainly by jointing, and
  - was originally applicable to low stress tunnel conditions, but
  - has included ratings under SRF that also address higher stress conditions;
  - is used for span and tunnel support design;
  - is used to determine the Modified Stability Number (N') used in stable mining span or caving potential estimation

- The RMR system:
  - was based on shallow civil construction tunnels in sedimentary rocks and was based on 49 case studies, but was modified after a further 64 case studies;
  - does not include a stress-related parameter;
  - is to a large extent not used extensively in the RSA underground mines, but
  - is used often in open pit slope designs or stand-up time, support and spans in tunnels;
  - drawpoint design;

- The MRMR system:
  - was developed from the RMR system and made it applicable to underground mining excavations and support;
  - caters for mining-related issues such as weathering, blasting, excavation orientation w.r.t. jointing and stress;
  - applies very well to the mining environment in general and has been applied to various mining methods, especially massive and cave mining methods;
  - is often used for caving potential estimation;
  - is used in unsupported span estimation;
  - adjusted MRMR is used for initial slope design;
  - drawpoint support design when used to determine the DRMS;

One of the fundamental differences between tunnel and mine design approaches to rock mass classification is the large variation in the engineered openings in mining applications. For tunnel projects, tunnel orientation, depth and stress conditions are usually constant for significant portions of a project. In mining, none of these conditions can be assumed to be constant. Due to the relatively constant engineered conditions in tunnelling, the stress condition has been included in the Q classification system and the relative orientation between the tunnel and critical joint set has been included in the RMR system. This approach has not been widely adopted in the mining industry, because it would result in the same rock mass having dozens of classification values throughout the mine, depending on drift orientation, mining level and the excavation

back

history. This would lead to significant confusion and render the rock classification values useless.

Rock mass classification is one of the only approaches for estimating large-scale rock mass properties. In the mining industry, the Q and RMR classification system forms the basis of many empirical design methods, as well as the basis of failure criteria used in many numerical modelling programmes. Classification systems have evolved as engineers have attempted to apply rock classification to a wider range of engineering problems.

NOTE

Practitioners should be aware that classification and design systems are evolving and that old versions of classification systems are not always compatible with new design approaches.

INTERESTING INFO

Some of the problems that can be encountered are outlined below:

- More than one relationship has been suggested for relating joint spacing to RQD. These approaches do not all agree and the users should use more than one method. An estimate within 5% is more than adequate for RQD.
- Practitioners sometimes estimate one classification and then derive a second classification from empirical relationships. Relating Q and RMR makes for an interesting comparison between classifications and may improve our understanding of the rock mass; however, the two systems should always be derived independently. There are many published relationships between Q and RMR; however, it is likely that no one relationship would work for all rock mass conditions. Nevertheless, there is no reason why the Q and RMR systems should be directly related.
- Care must be taken when using classification systems with empirical design methods. The user must be sure that the classification system used matches the approach taken for the development of the empirical design method. A design method based on RMR76 cannot be expected to give the same results as RMR89. Under these circumstances, for purposes of continuity, it is sometimes necessary to continue using an earlier version. Design methods that do not rely on case histories or past experience, do not have the same constraints.
- Mining applications of the Q and RMR systems have tended to simplify classification systems to only include factors dependent on the rock mass, ignoring environmental and loading conditions. This has resulted in the Q' and RMR' that ignore factors such as stress and joint orientation. This approach to classification is warranted in complex mining situations. Serious errors can result if these simplified classification systems are applied to the empirical civil tunnel design approaches such as the Q support graph.

Despite their limitations, the reviewed classification systems are still in use as they provide an invaluable reference to past experience.

REFERENCE

- Page 198, Hutchinson D.J., Diederichs M.S., Cable bolting in underground mines, 1996.

LEARNING OUTCOME 3.1.2.7

DESCRIBE AND EXPLAIN WHAT MODIFICATIONS ARE NECESSARY TO APPLY ROCKMASS CLASSIFICATION SYSTEMS TO LOCAL CONDITIONS.

Most classification systems have their origin in shallow workings where geological structures dominate the stability of excavations. In the intermediate to deep mining scenario, the importance of geology in the stability of excavations is secondary to the stress conditions that prevail. It is therefore essential that Rock Engineers have a good feel as well as test results (for backing this up) on the properties of their host rocks. Classification systems enable the conditions encountered at a particular site to be compared qualitatively with preceding experience, thereby helping in the selection of support requirements. Modelling parameters can also be derived and/or verified from rock mass rating values.
No single index value gives an adequate description of the complex behaviour of a rock mass. Therefore, classification systems have been derived to describe the relative contributions of various parameters to bulk properties. Such systems have been generally developed to quantify rock mass properties for the design of underground openings.

Classification systems in current use show varying degrees of complexity. The two most widely-spread systems used in South Africa are the CSIR classification and the NGI Tunneling Quality Index (Q-system). These systems may be used in their own right or adapted according to specific requirements identified on site. Two general approaches have been taken to allow these classification systems to be applied to mining conditions. The first approach was to try and create a complete design method from the classification systems by including other engineering and loading condition factors.

One example of this is Laubscher's MRMR system (Mining Rock Mass Rating), which added factors such as blasting, weathering, multiple joint orientations and stress to obtain a term called the design rock mass strength. This term represents the unconfined strength of the rock mass in a specific mining environment. The assessment of these added factors is quite subjective and requires an experienced practitioner.

The second approach consists of simplifying the classification system to only include factors dependent on the rock mass and to ignore environmental considerations such as stress and drift orientation. The resulting rating is solely dependent on the rock mass, and will give the same assessment for the same rock conditions at different depths and drift orientations within a mine. This simplified rock classification approach has been applied to both the RMR and Q systems. Q' is the modified Q classification with SRF = 1 and RMR' drops the joint orientation factor.

The Q' and RMR' classification values have been used in many different mining situations. With these design approaches, factors such as stress and the influence of joint orientation have been added as steps in the design process, which do not influence the classification values. Some of these design methods are mentioned in the following sections.

Many mine design approaches have been developed from the Q' and RMR' classification systems. In many cases, these design approaches account for the influence of stress and joint orientation in the design steps and it would be incorrect to assess these factors twice by including them in the rock mass classification.

REFERENCE

- Chapter 3 Rock mass classifications – Hoek's notes
- Page 58, Hoek E., Practical rock engineering, 2006.

LEARNING OUTCOME 3.1.2.8

APPLY BARTON'S Q SYSTEM TO CLASSIFY A ROCKMASS

EXAMPLE 25

A 15m wide workshop for an underground mine is to be excavated in quartzite at a depth of 2,500m below surface. Two sets of rough, planar and unaltered joints traverse the workshop. Laboratory tests on core samples of intact rock give an average uni-axial compressive strength of 210MPa. The principal stress directions are approximately vertical and horizontal and the magnitude of the horizontal principal stress is approximately 0.5 times that of the vertical principal stress. The rock mass is moist, but there is no evidence of flowing water. RQD values obtained from core logging were calculated as 90%.

Determining input values for the Q Index calculation

- The numerical value of RQD is used directly in the calculation of Q and, for this rock mass, the value of 90% will be used.

  RQD =90%
- From Table 5 part 2: Two joint sets for rough or irregular joints that are undulating, are allocated a joint set number of

  Jn = 4
- Table 5 part 3: Allocates a joint roughness number of

  Jr = 1.5
- Table 5 part 4: Allocates a joint alteration number for unaltered joint walls with surface staining only of

  Ja= 1.0
- Table 5 part 5: For an excavation with minor water inflow, the joint water reduction factor is

  Jw = 1.0
- For a depth below surface of 2,500m the overburden stress will be approximately 66MPa and, in this case, the major principal stress σ_1 = 66MPa. Since the uni-axial compressive strength of the quartzite is given as 210MPa, this gives a ratio of σ_c/σ_1 of 3.18. Table 5 part 6 shows that, for competent rock with rock stress problems, this value of σ_c/σ_1 can be expected to produce mild rock burst conditions and that the value of SRF lies between 5 and 10.

  SRF = 6.4 will be used for this calculation (see below).

Using these values gives:

$$Q = RQD/J_n \times J_r/J_a \times J_w/SRF$$

Then follows that for the values given above

$$Q = (90/4)(1.5/1.0)(1.0/6.4) = 5.3$$

NOTE

In areas where allocated values fall within a spread of values, the following reasoning can be applied. Using the σ_c/σ_1 of 3.18 from the example above, the rating can be determined by using the ratio of the actual σ_c/σ_1 value to the lower category limit for the σ_c/σ_1 ratio and then multiplying this ratio to the lower rating limit. For example:

Appropriate SRF rating
= (3.18 / 2.5) * 5
= 6.4

where 2.5 is the lower limit for the σ_c/σ_1 ratio and 5 is the lower limit of the allocated rating from Table 5.

LEARNING OUTCOME 3.1.2.9

APPLY BIENIAWSKI'S RMR SYSTEM TO CLASSIFY A ROCKMASS

EXAMPLE 26

A tunnel is to be driven through slightly weathered granite with a dominant joint set dipping at 60° against the direction of the drive. Index testing and logging of diamond-drilled core give typical Point-load strength index values of 8MPa and average RQD values of 70%. The slightly rough and slightly weathered joints with a separation of < 1mm, are spaced at 300mm. Tunnelling conditions are anticipated to be wet.

The RMR value is determined as from Table 6, which provide the following ratings when the example conditions are applied (refer to notes applicable to the ratings below):

Table	Item	Value given	Rating
(a1)	Point load index	8MPa	12
(a2)	RQD	70%	13
(a3)	Discontinuity spacing	300mm	10
(a4)	Discontinuity condition	Note 1	22
(a5)	Groundwater	Wet	7
(b)	Adjustment for orientation	Note 2	-5
		Rock mass rating (RMR)	59

NOTE

Note 1: *For slightly rough and altered discontinuity surfaces with a separation of < 1mm, Table 6 (A4) gives a rating of 25. When more detailed information is available, Table 6(E) can be used to obtain a more refined rating. Therefore, in this case, the rating is the sum of: 4 (1-3m discontinuity length), 4 (separation 0.1-1.0mm), 3 (slightly rough), 6 (no infilling) and 5 (slightly weathered) = 22.*

Note 2: *Table 6(F) gives a description of 'Fair' for the conditions assumed where the tunnel is to be driven against the dip of a set of joints dipping at 60°. Using this description for 'Tunnels and Mines' in Table 6(B) suggests an adjustment rating of -5.*

LEARNING OUTCOME 3.1.2.10

APPLY LAUBSCHER'S MRMR SYSTEM TO CLASSIFY A ROCKMASS

EXAMPLE 27

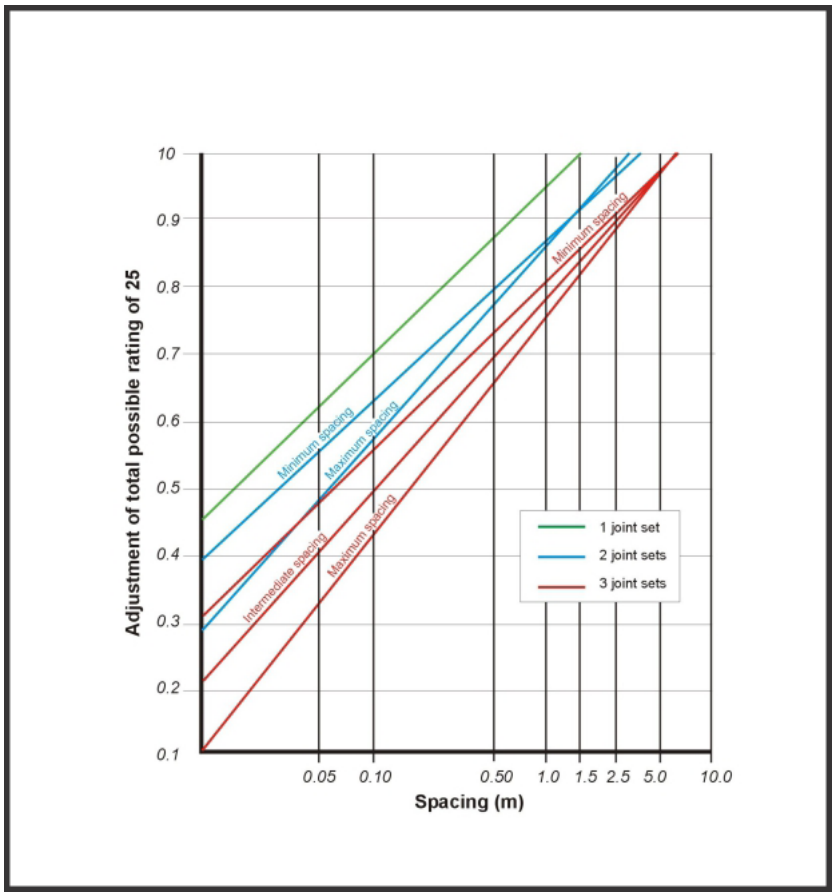
A rock mass with a RQD of 50% and consisting of two sets of moist, curving joints with rough surfaces and fine soft-sheared joint filling, spaced on average approximately 0.3m apart, is planned for development. The rock will moderately weather over a period of two years and has a UCS of 60MPa. Two of the three joints that form the blocks in the excavation walls have orientations away from the vertical (less than 90°) and expected stress levels are very low. It is being considered to conventionally blast the excavation making use of local workers. Determine the MRMR and adjusted MRMR.

From Table 7, Table 8 and Figure 45, the MRMR can be determined as stated below:

Information	Item	Value given	Rating
Table 8	RQD	50%	9.0
Table 8	UCS	60MPa	11.0
Figure 46	Joint spacing	Note 1	17.8
Table 8	Joint condition and water	Note 2	16.2
		Mining Rock Mass rating (MRMR)	54
Table 9	Adjustment for weathering		86%
Note 3	Adjustment for stress		70%
Table 10	Adjustment for joint orientations		80%
Table 11	Adjustment for excavation technique		80%
		Adjusted MRMR	21

NOTE

Note 1: *From Figure 45 (also see below), for two joint sets at average 0.3m spacings, a rating of 0.71 x 25 = 17.8 results.*



Note 2: *From Table 8 it follows that ratings for curved joints are 80%, rough surfaces 92%, fine, soft infilling 55% and alteration 100%. This provides a joint condition and water rating of 0.8*0.92*0.55*1.0*40 = 16.2.*

Note 3: *Due to low stress levels, confinement is expected to be low and may result in si*dewall scaling. A rating of 70% is selected.

LEARNING OUTCOME 3.1.2.11

DESCRIBE, DISCUSS AND APPLY ROCKMASS CLASSIFICATION TECHNIQUES FOR THE SELECTION OF MINING METHODS

Mining methods are selected through a process where a large number of different factors such as mining scale, production rate, selectivity, flexibility, ore body and host rock characteristics, etc are evaluated. All of these factors are important in the final selection of a mining method and should be kept in mind when making the decision.

During the selection of the most appropriate mining method, the geotechnical engineer evaluates those factors pertaining to his field of expertise . Such a process implies that any method is a prospective mining method, but that some methods would be more beneficial to a specific environment. The following broad mining method options are available:

- Methods with naturally supported stopes: Stability is maintained with pillars. These methods ensure stability as small spans are maintained, but results in a low extraction ratio.
- Methods with artificially supported stopes: Stability is maintained using artificial support such as timber-based units or backfill.

- Methods where stopes are supported by a combination of natural and artificial material: Pillars are left to ensure separation of the mining area into compartments, and backfill is utilised to limit the spans as mining progresses.
- Methods where no support is placed and the mined-out area is allowed to collapse after a period of time that is sufficient to allow the extraction of the ore.

The following issues directly affect the possibility to implement specific mining methods:

Mining method	Geomechanical conditions	Use of rock mass quality
Open stoping	Stable stope walls	RMR and MRMR, as well as N' (determined from Q) are used to determine stable spans and therefore the stability of the host rock.
	Stable crown / sill pillars	Q is used to estimate crown pillar dimensions.
Caving	Stability of the host rock.	RMR and MRMR, as well as N' (determined from Q) are used to determine stable spans and therefore the stability of the host rock.
	Caving of the ore body	MRMR is used to assess the potential for ore body caving, whilst DRMS (calculated from MRMR) is used during numerical modelling to assess the possibility to induced failure that will enhance caving.
Board-and-pillar	Stable pillars	RMR, MRMR and RMS can be utilised establish the pillar strengths to allow pillar designs.
	Stable roof	Q and RMR can be utilised to assess unsupported spans, while support requirements can be determined using the same parameters.
Backfill supported methods	Unstable host rock	Spans are limited to remain stable using RMR and MRMR, as well as N' (determined from Q).
Shrinkage	Stable host rock that will not unravel during extraction	RMR and MRMR, as well as N' (determined from Q) are used to determine stable spans and therefore the stability of the host rock.

Table 13: Impact of rock mass classifications on mining method selection

Although the rock mass ratings produced by the relevant classification systems do not directly determine the mining method, the quality of the rock mass determine the conditions to which the mining method is subjected and which in itself allows or excludes a particular mining method from being appropriate.
A weighted mining method selection process, as shown inTable 14, is an acceptable method as it allows the inclusion of all the critical parameters in the selection process.

Candidate Mining Method	Shape	Thickness	Dip	Depth	Grade Distribution	Strength	Contacts	Weighted Ratings
	Regular	Wide	Steep	Shallow	Could be erratic	Moderate	Not well defined	
Weighting	3	1	2	1	2	3	2	
Cut & Fill	4	3	3	4	1	4	4	47
Shrinkage Stoping	4	4	5	4	2	4	4	54
Sub-level Open Stoping	4	4	5	4	3	4	4	56
Sub-level Caving	4	3	5	4	2	1	1	38

Table 14: Example: Ore body and host rock characteristics in mining method selection

REFERENCE

- Chapter 12, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.

LEARNING OUTCOME 3.1.2.12

APPLY ROCKMASS CLASSIFICATION RESULTS TO DETERMINE THE STABILITY OF UNSUPPORTED SPANS

Unsupported spans are intuitively related to the quality of the rock material forming the sidewall, hangingwall / roof of the excavation. It is suggested that poor rock material should become unstable at smaller spans than good material, purely due to the higher probability that the poorer rock material has more joints, smaller block sizes, poorer joint strengths, etc.

Rockmass classifications has therefore been utilised to determine the comparitive stability of unsupported mining spans.

RMR or Q Index results are presented in Figure 47 below to indicate the maximum unsupported span and approximated stand-up time. It is therefore possible to derive probable unsupported spans and the period that it should remain stable before support would be required. It is clear that good quality material (high Q and RMR) may remain stable when unsupported for many years, as has been observed in many instances, while low ratings suggest immediate collapses if left unsupported. Ratings achieved through the logging or mapping process are simply plotted in Figure 47, and the results utilised in the design of the spans.

NOTE

- These relationships were derived empirically and indicate probable spans and stand-up time.
- It is suggested that ranges are used rather than using an average rock mass quality value.
- Back analysis to ensure appropriate designs and monitoring during implementation of the design remains essential due to the empirical nature of this relationship.

back

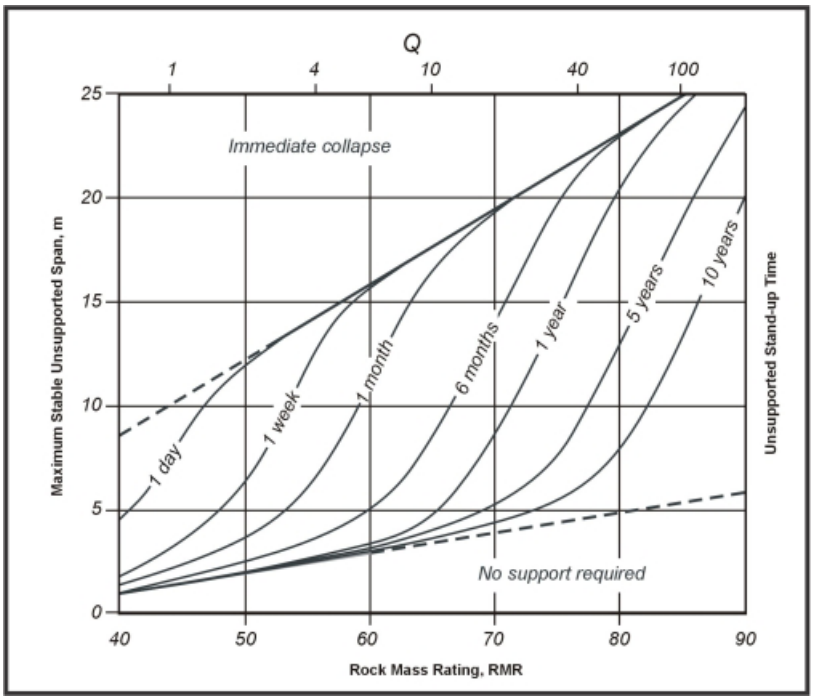


Figure 47: Maximum unsupported span and stand-up time of excavation based on RMR and Q Index ratings (Stacey, 2001)

The span design process for open stopes in steep dipping ore bodies is based on the understanding that one of or a combination of the ore body (roof), hangingwall and footwall, could collapse. By drafting the relationship between the rock mass quality and mining span, the critical stable span for a specific rock mass quality can be determined. Potvin has provided such a relationship and has indicated the zones of stability, instability and transitions areas, based on the data sample used at the time. **A rock mass quality parameter N**', called the Modified Stability Number and the Hydraulic Radius concept, is used to determine the stability of mining spans.

The Hydraulic Radius is calculated from

HR=(Length x Width)/(2 x Length+2 x Width)

and suggests that the two-dimensional dimensions of a stope (length and width) can be represented by a single number (HR).

In determining the modified stability number N':

- The Q' parameter is taken from the **rock mass quality Index Q**, as developed by Barton, but replaced the 'stress'-related input parameters in the Q Index with other parameters.
- The Q Index is determined from $Q=RQD/J_n .J_r/J_a .J_w/SRF$ while the J_w/SRF is removed in determining the Q' Index : $Q^{\wedge}'=RQD/J_n .J_r/J_a$
- Q' is then multiplied with the Potvin factors a, b and c. $N^{\wedge}'=a.b.c.Q'$.
- Potvin 'a' factor: This factor accounts for the influence of high stresses that reduce rock mass stability and is determined by the ratio of unconfined compressive strength of intact rock to maximum-induced stress parallel to the opening surface. When no stress impact is expected, it is set to 1.0, but stress analysis to determine the stress ratio above is promoted. It is set to 1.0 if intact rock strength is 10 or more times induced stress, which indicates that high stress is not a problem. It is set to 0.1 if rock strength is two times induced stress or less, which indicates that high stress significantly reduces opening stability.

back

back

- Potvin 'b' factor: Looks at the influence of the orientation of discontinuities with respect to the surface analysed.

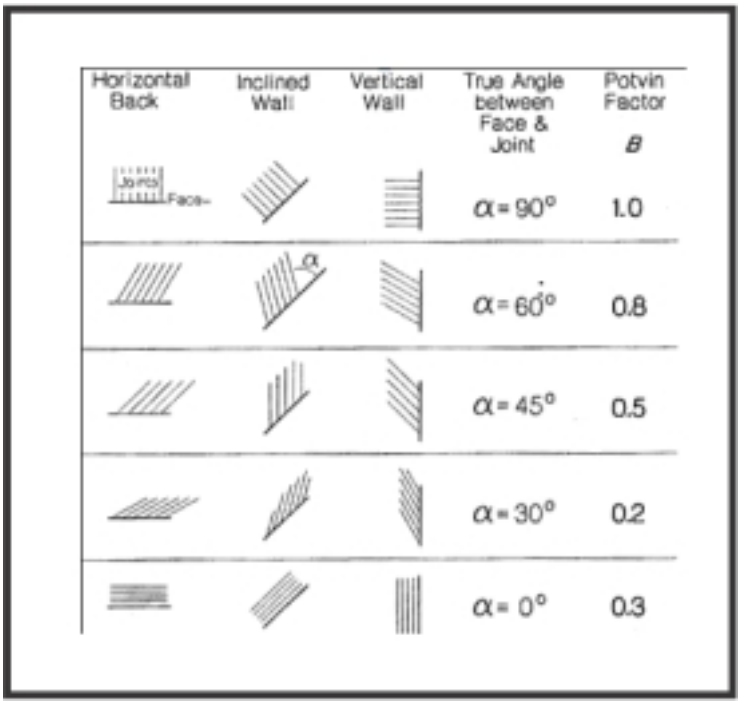


Figure 48: Potvin B Factor

- Potvin 'c' factor: Considers orientation of the surface being analysed and reflects the inherently more stable nature of vertical walls compared to a horizontal back.

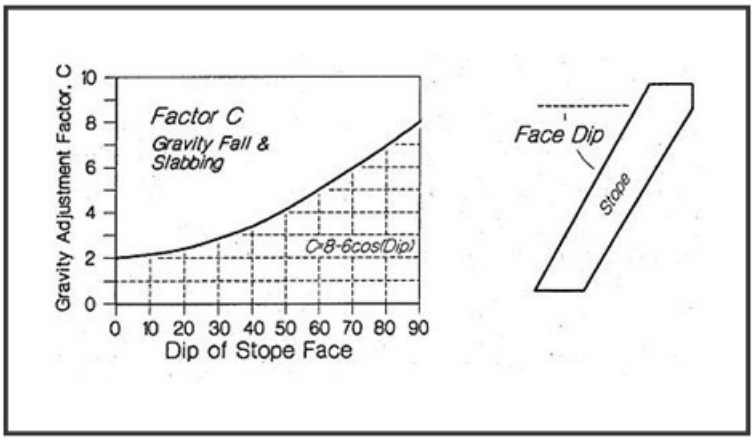


Figure 49: Potvin C Factor

Figure 50 provides the relationship between the Modified stability number (N') calculated using the Q' values (as calculated from the Q Index) and the Potvin factors, with the dimensions (HR) of a number of stable and unstable stopes. A line that separates the stable and unstable stopes provides a proposed relationship between stope dimensions (hydraulic radius) and rock mass quality (N') for the specific mine. The relationship presented in Figure 50 suggests that a poor rock mass quality (low N') will require a smaller hydraulic radius (stope dimension) to remain stable, than a good rock mass (high N').

back

back

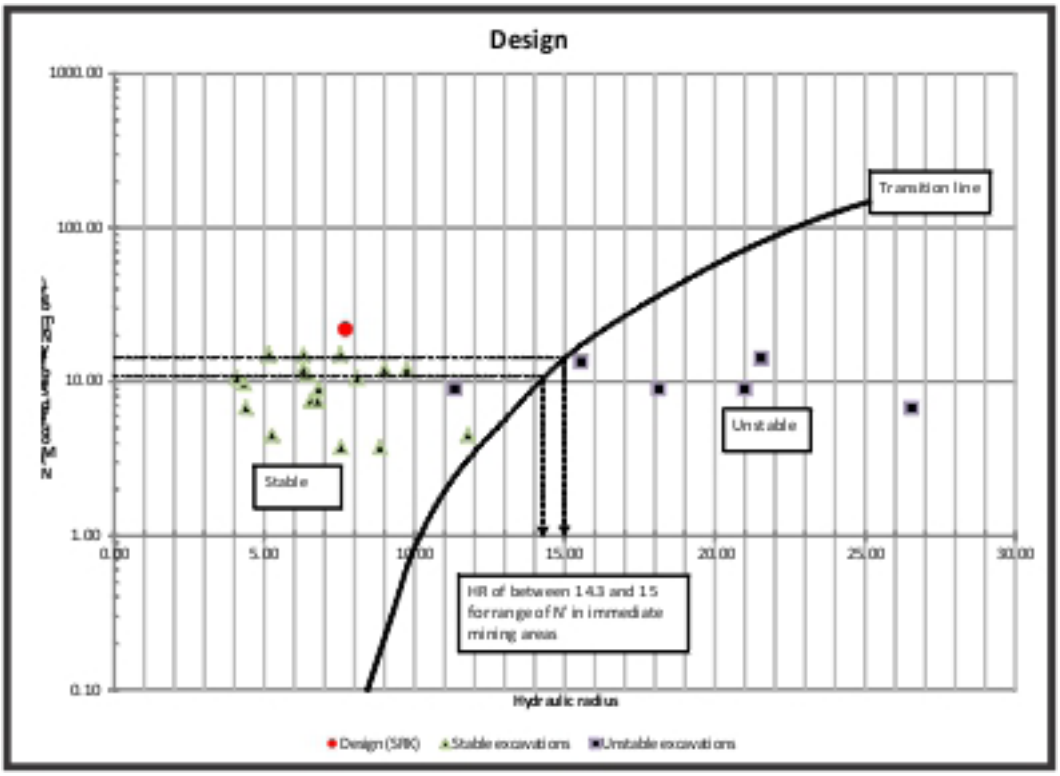


Figure 50: Example: Modified stability number vs Hydraulic radius

INTERESTING INFO

Sometimes, the stopes being analysed and represented as in Figure 51, do not have square or rectangular dimensions. A concept was developed by Milne and Pakalnis (Pakalnis et al.) for irregular mining boundary shapes (Radius Factor), to allow the calculation of a single number that would represent the dimensions of the stope. The Radius Factor was derived through mathematical manipulation of the standard Hydraulic Radius concept (ratio of area to circumference) and suggests that the open span can be represented by a radius calculated from Radius factor=$0.5/(1/n\ \Sigma^{1}_{n}\ 1/r_{\theta})$. For simple mining span boundary shapes, the Radius Factor and Hydraulic Radius provide the same solution, but the Radius Factor credits pillars left in-situ for the 'span breaking' role they perform. Utilising this factor, it could be shown that a stub pillar left within a stope significantly reduces the mining spans and indicates that the mining has greater potential to remain stable with a stub pillar. In Figure 52 a number of dotted lines on the mining layout indicate the radii measured to calculate the Radius Factor that would represent the complete stope. The line graph indicates a Radius Factor that is lower than the Hydraulic Radius if the central 'stub' pillar is not left within the stope, suggesting that pillars left in-situ have span controlling benefits, allowing the larger stope dimensions to increase.

back

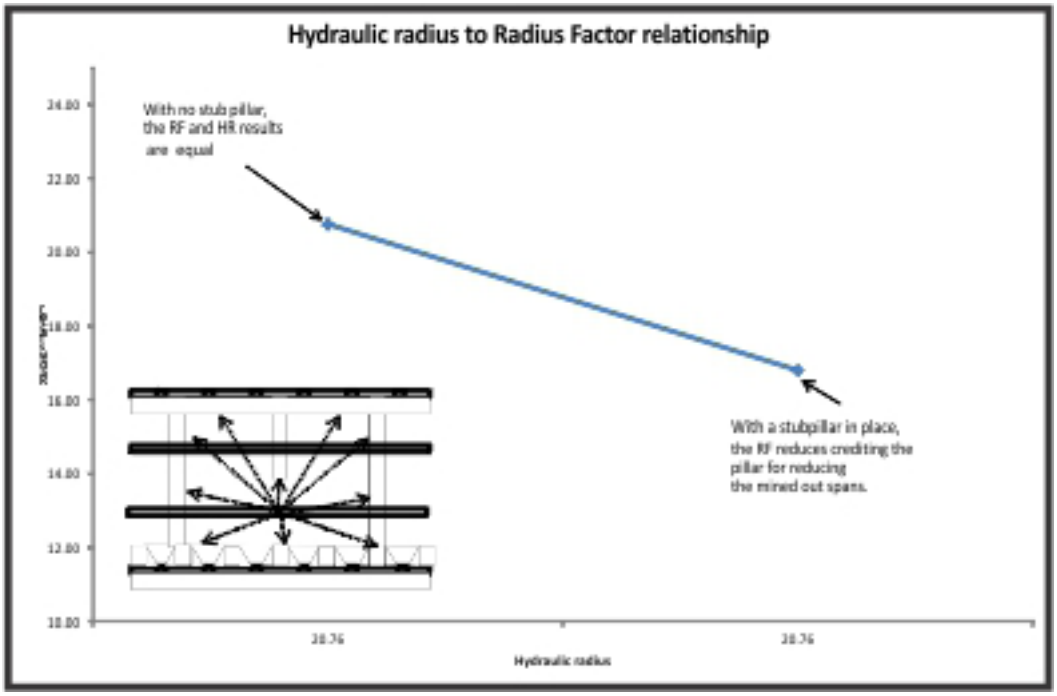


Figure 51: Impact of stub pillar on Radius Factor (Hydraulic radius)

REFERENCE

- Chapter 3, Stacey T.R., Best practice rock engineering for Other Mines, 2001.
- Page 266, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.
- Page 197, 221, Hutchinson D.J., Diederichs M.S., Cable bolting in underground mines, 1996.

LEARNING OUTCOME 3.1.2.13

APPLY ROCKMASS CLASSIFICATION RESULTS TO DETERMINE THE STABILITY OF UNSUPPORTED ROCK SLOPES

Rock slope failure mechanisms traditionally include simple planar, wedge, circular and toppling failure modes. In analysing the planar and wedge failure mechanisms, the quality of the rock mass forming the slope is not considered due to the nature of the mechanism, which is shear displacement. Even though the shear displacament on a plane or intersection surface between planes is affected by issues normally included into rock mass ratings (water, infilling, continuity, stress conditions, etc), the failure is not affected by a specific rock quality parameter.

Circular failure though suggests that the failure is not governed by a specific plane, but more by the composition of the material itself and that a failure plane is formed within the rock material at the point of least resistance. In analysing this mechanism, it is intuitively felt that a very poor quality, broken up rock mass (low quality rating) would more easily result in failure than a good quality (high quality rating), massive rock mass. This is confirmed when one realises that soil, the ultimate material to yield circular failures, would report an extremely low rock mass quality if it could be rated. Weathered rock material would also tend to fail according to this mechanism and it is known that weathered material usually results in very low rock mass quality ratings. However, circular failure is governed by the shear strength of the material and since large rock mass slopes may contain a large number of structures, an accurate

back

method of evaluating the stability using simple shear failure on strctures is impossible. This resulted in the decsription of the rock mass strength in a manner that was failure mode independent, namely the Hoek-Brown failure criteria, where the rock mass is viewed to be so broekn up that it behaves as a continuum. In determining the Hoek-Brown strength of the rock mass, the quality parameters m and s can be calculated using the RMR or GSI ratings, both systems that are rock mass quality rating systems.

CONNECTION 12

Paper 1 "Learning Outcome 6.2.2" and
Paper 2 "LEARNING OUTCOME 6.4" on page 497

Rock mass classification is a strutured and more comprehensive form of empirical slope design. The RMR system was modfied into a slope rock mass quality system (SMR), especially for rock slopes where the RMR rating is modified according to four adjustment factors accounting for joint and slope geometry and blasting methods. As an example, Table 15 shows how this rock mass classification is also used to determine the stability of slopes.

Class	SMR	Description	Stability	Failures	Support
I	81-100	Very Good	Completely stable.	No failures	None
II	61-80	Good	Stable	Some blocks	Occasional
III	41-60	Normal	Partially stable	Planar failure in some joints and many wedge failures	Systematic
IV	21-40	Bad	Unstable	Planar failures in may joints or big wedge failures.	Important / corrective
V	0-20	Very Bad	Completely unstable	Big planar or soil-like	Re-excavation

Table 15: SMR classes (Romana, 1993)

Haines and TerBrugge based their slope stability charts on the MRMR, as developed by Laubscher. In this relationship, the slope height and MRMR ratings are used to estimate the stable slope angle.

Adjusted MRMR rating	100	90	80	70	60	50	40	30	20	10	0
Overall Slope Angle	>75°	75°	70°	65°	60°	55°	50°	45°	40°	35°	<35°

Table 16: Correlation between adjusted mining rock mass rating (MRMR) values and overall rock slope angles

Table 16 is best suited to the preliminary determination of overall slope angles, while the design chart shown in Figure 52 below is well suited to the selection of inter-ramp or bench stack angles.

back

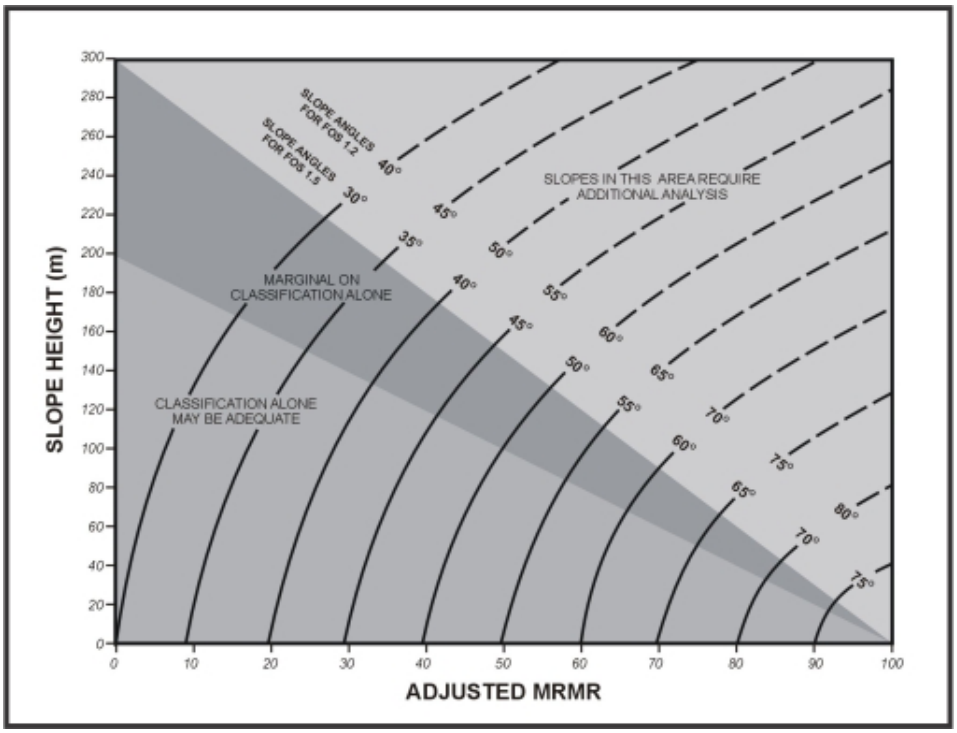


Figure 52: Empirical slope design chart (Haines and Terbrugge, 1991)

All classification systems show the same weakness in that they are not precise enough for the design of final slopes in open pit mining, but are applicable to small-scale slopes and first estimates of stable slopes in low accuracy investigations.

REFERENCE

- Chapter 3 Rock mass classifications – Hoek's notes.
- Chapter 3, Stacey T.R., Best practice rock engineering for Other Mines, 2001.

LEARNING OUTCOME 3.1.2.14

APPLY ROCKMASS CLASSIFICATION RESULTS TO DETERMINE SUPPORT REQUIREMENTS FOR VARIOUS SITUATIONS

EXAMPLE 28

A tunnel is to be driven through slightly weathered granite with a dominant joint set dipping at 60o against the direction of the drive. Index testing and logging of diamond drilled core give typical point-load strength index values of 8MPa and average RQD values of 70%. The slightly rough and slightly weathered joints with a separation of < 1mm are spaced at 300 mm. Tunnelling conditions are anticipated to be wet. Suggest the support requirements for this tunnel.

As for slope and span designs, it stands to reason that poor quality rock material will require substantially more and possibly different support systems to maintain stability. It is therefore logical that empirical support design methods would exist that relate rock mass quality ratings with support requirements.
In describing how rock mass quality ratings can be used to estimate support requirements, examples will be provided.

Bienawski's rock mass rating (RMR)

Support of tunnels using the RMR ratings

Based on the RMR (98) tables described in "LEARNING OUTCOME 3.1.2.5" and "LEARNING OUTCOME 3.1.2.9", the following RMR rating results:

Table	Item	Value given	Rating
(a1)	Point load index	8MPa	12
(a2)	RQD	70%	13
(a3)	Discontinuity spacing	300mm	10
(a4)	Discontinuity condition	Note 1	22
(a5)	Groundwater	Wet	7
(b)	Adjustment for orientation	Note 2	-5
		Rock Mass rating (RMR)	59

Bieniawski (1989) published a set of guidelines for the selection of support in tunnels in rock for which the value of RMR has been determined (Table 17).

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

Table 17: RMR: Support requirements (Bieniawski 1989)

NOTE

These guidelines have been published for a 10m span horseshoe shaped tunnel, constructed using drill and blast methods, in a rock mass subjected to a vertical stress < 25MPa (equivalent to a depth below surface of <900m).

- For the case considered earlier, with RMR = 59, Table 17 suggests that:
- The tunnel could be excavated by top heading and bench, with a 1.5 to 3m advance in the top heading;
- Support should be installed after each blast and the support should be placed at a maximum distance of 10m from the face;
- Systematic rock bolting, using 4m long, 20mm diameter fully-grouted bolts spaced at 1.5 to 2m in the crown and walls;
- Wire mesh, with 50 to 100mm of shotcrete for the crown and 30mm of shotcrete for the walls.

NOTE

The value of RMR of 59 indicates that the rock mass is on the boundary between the 'Fair rock' and 'Good rock' categories. In the initial stages of design and construction, it is advisable to utilise the support suggested for fair rock. If the construction is progressing well with no stability problems, and the support is performing very well, then it should be possible

back

to gradually reduce the support requirements to those indicated for a good rock mass. In addition, if the excavation is required to be stable for a short period of time, then it is advisable to try the less expensive and extensive support suggested for good rock. However, if the rock mass surrounding the excavation is expected to undergo large mining-induced stress changes, then more substantial support appropriate for fair rock should be installed. This example indicates that a great deal of judgement is needed in the application of rock mass classification to support design.

It should be noted that Table 17 has not had a major revision since 1973. In many mining and civil engineering applications, steel fibre reinforced shotcrete may be considered in place of wire mesh and shotcrete.

Support of shafts using the RMR ratings

The method of evaluating the stability of shafts (Stacey, 2001) was based on the Q rock mass classification system, but using an acceptable conversion to RMR resulted in the drafting of the design charts in Figure 53 and Figure 54. In this case, the diameter, or largest dimension of the shaft, should be considered as the span on the vertical axis. The implication from this is that, as a guide, a 6m shaft will not require support if the rock mass quality exceeds an RMR of about 50. Figure 53 expresses the rock bolt requirements as m2/unit, while Figure 55 indicates the required shotcrete thickness in mm.

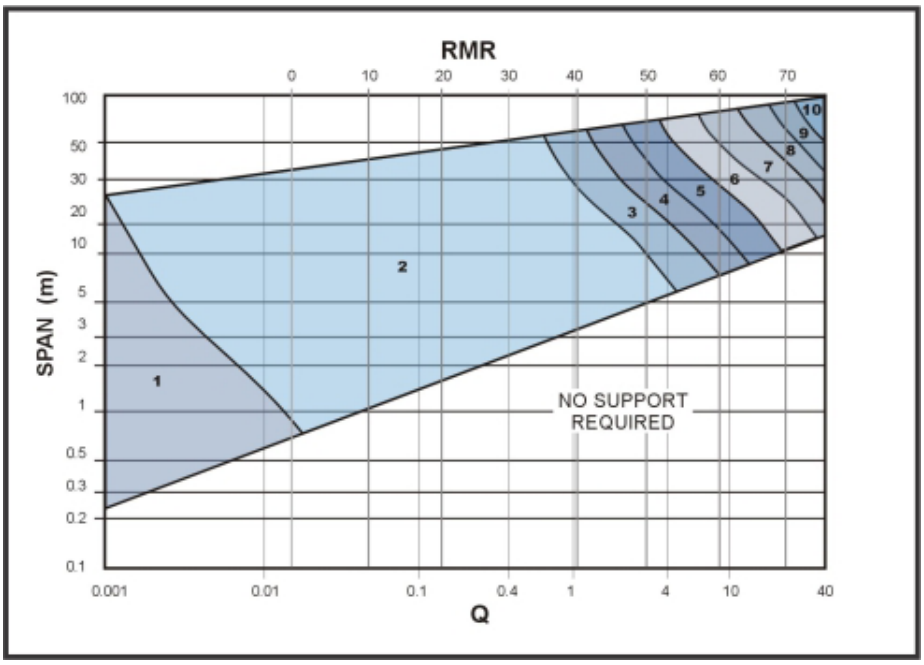


Figure 53: Rock bolt support design chart for shafts using RMR (Stacey, 2001)

back

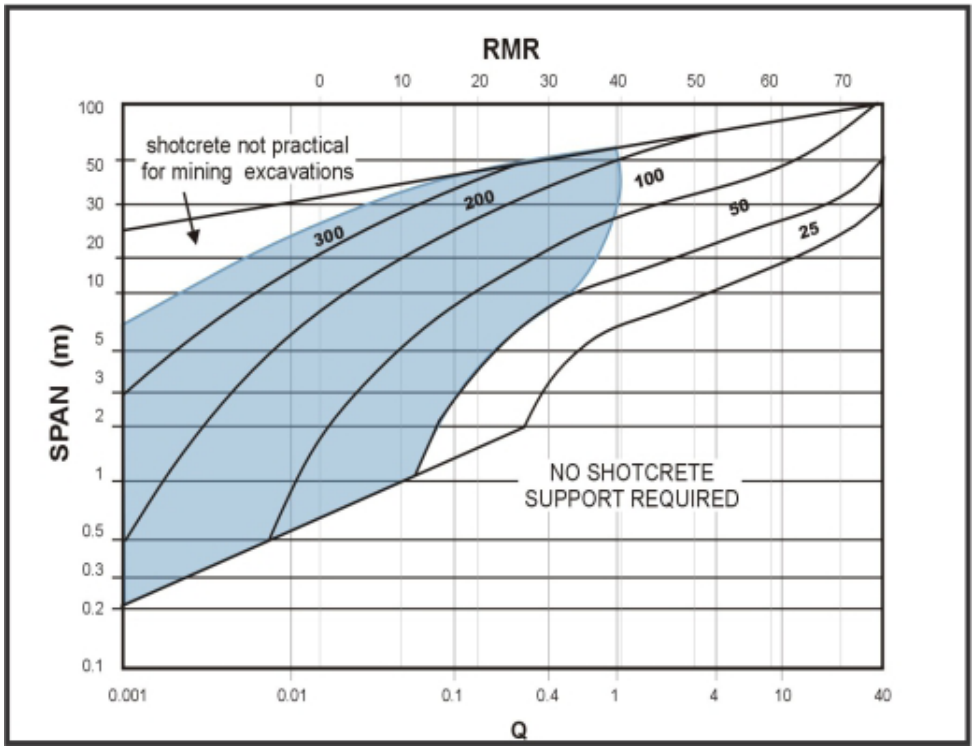


Figure 54: Shotcrete design chart for shafts using RMR (Stacey, 2001)

Barton's Q Index

Support of tunnels using the Q Index

The Q classification system also caters for rock mass compositions that necessitate systematic support systems. For excavation spans that are indicated to be self-supporting (Outcome 2.1.2.12), consideration must still be given to the support of potentially unstable blocks defined by jointing. At spans in excess of those indicated to be self-supporting, support recommendations are provided in Figure 56 and Figure 56.

Figure 55 expresses the relationship between Q and the dimension of the tunnel, which is normalised by the ESR. Barton et al. (1974) defined a parameter Equivalent Dimension (De) of an excavation by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR (Table 18).

The value of ESR is related to the intended use of the excavation and to the degree of security that the installed support must provide against instability (Table 18).

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio ESR}}$$

Type of Excavation		ESR
A	Temporary mine openings, etc.	2 - 5
B	Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers.	1.6 - 2.0
C	Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels.	1.2 - 1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections.	0.9 - 1.1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels.	0.5 - 0.8

Table 18: Suggested ESR values (Ryder and Jager, 2002)

back

In using Figure 55, it is suggested to use the following approach:

- Determine the De of the excavation and plot on the vertical axis to the left;
- Determine the appropriate Q Index, or range of Q Index values, and plot on the bottom horizontal axis;
- Draw lines horizontally from the De and vertically from the Q Index until they intersect;
- Read off the following support requirements:
- Shotcrete thickness from the diagonal lines;
- Bolt length on the vertical axis to the right; and
- Bolt spacing to the horizontal axis at the top.

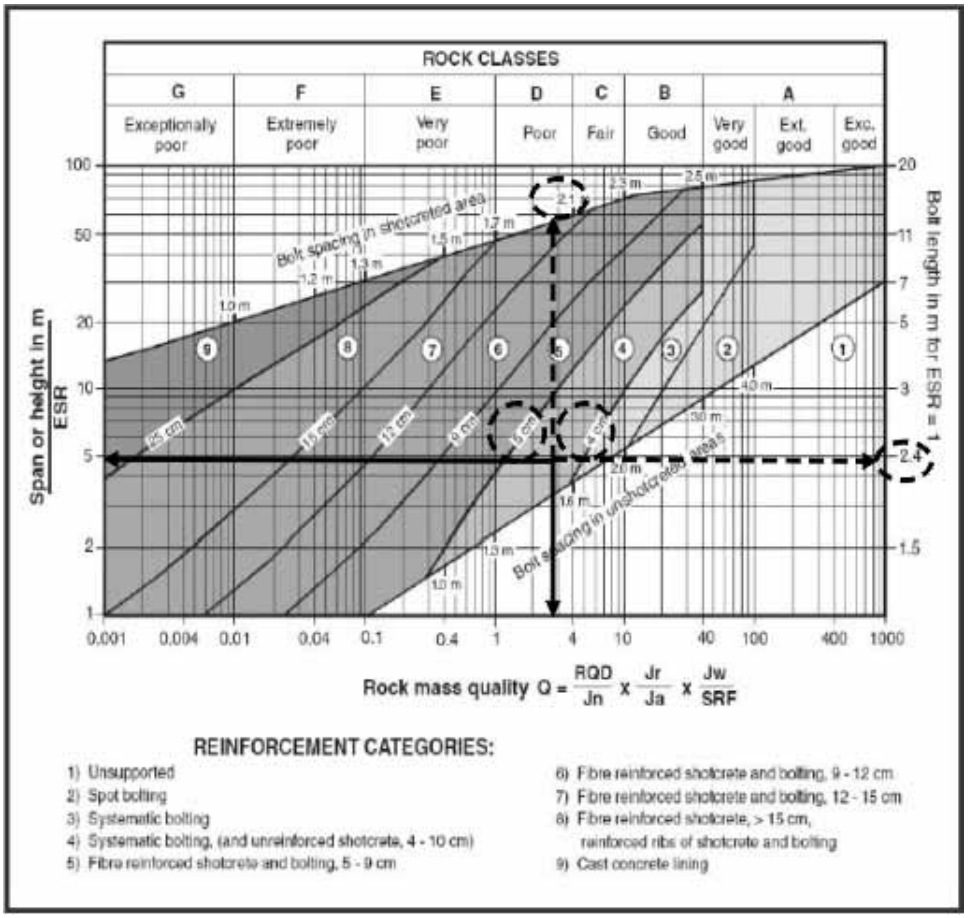


Figure 55: Support for tunnels using Q Index (Ryder and Jager, 2002)

In the Q system, the length of the rock bolts is purely a function of the relative size of the excavation, but the spacing of the rock bolts and the necessity for fabric support, is a function of the rock mass quality (Figure 55).

The required support pressure can also be viewed as a function of the Q value and the roughness of the discontinuity on which it is likely to have a potential instability. Figure 56 relates the required support pressure (Note: in kg/cm2) to the Q Index and joint roughness number (from the Q Index system).

In using Figure 56, it is suggested to use the following approach:

- Determine the appropriate Q Index, or range of Q Index values, and plot on the bottom horizontal axis;

back

back

- Draw a vertical line from the Q Index until it intersects the appropriate Joint Roughness Number diagonal (see vertical axis on right); then draw a horizontal line towards the vertical axis on the left;
- Read off the support pressure requirement and convert to more familiar units such as kN/m2 (kPa) or MPa (as used in Table 19).

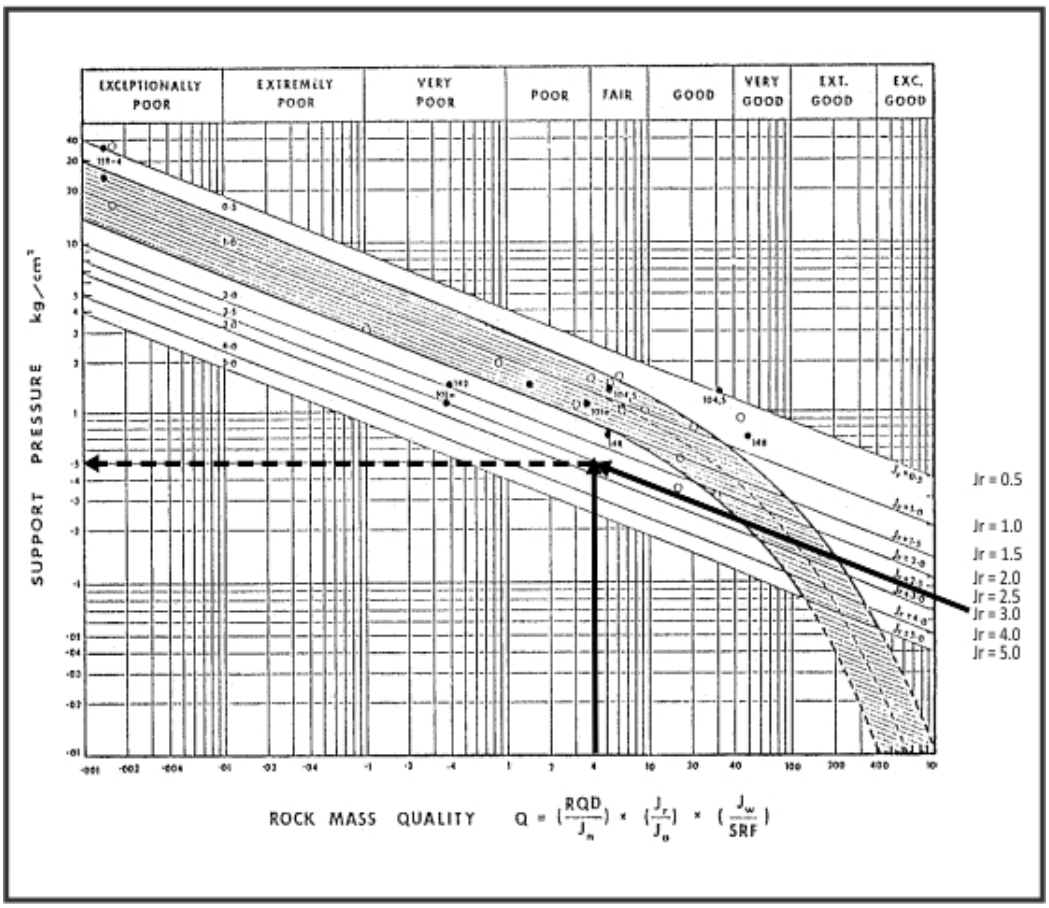


Figure 56: Support pressure required using Q Index (Ryder and Jager, 2002)

Conversion from kg/cm2 to N/m2:

- *Convert the kg to N by multiplying with 9.81m.s-2;*
- *Convert 'per cm2 ' to 'per m2 ' by multiplying the number from the step above by 10,000;*
- *E.g. from Figure 58 follows that a support pressure of 0.5kg/cm2 is required. Following the steps above yields (0.5*9.81*10000 = 49.05kN/m2 = 0.049MPa) where 1MN/m2 = 1MPa.*

Guidelines for the design of primary support systems were also proposed by Barton, Lien and Lunde (1974) based on the Q system, as shown in Table 19.

back

back

Rock mass jointing	Support pressure	Length and spacing of reinforcement
If the number of discontinuity sets > 2	$P_{roof} = (0.2\ Q^{-1/3}) / J_r$	Bolts $L_{roof} = (2 + 0.15B)/ESR$ Anchors $L_{roof} = (0.4B)/ESR$ $s = (C / P_{roof})^{1/2}$
If the number of discontinuity sets ≤ 2	$P_{roof} = (0.2.J_n^{-1/2}\ Q^{-1/3})/3J_r$	
	Pwall : calculated with the same formulae as Proof, by replacing Q with Q': Q' = 5Q if Q > 10 Q' = 2.5Q if 0.1 ≤ Q' 10 Q' = Q if Q < 0.1	Bolts $L_{wall} = (2 + 0.15H)/ESR$ Anchors $L_{wall} = (0.35H)/ESR$ $s = (C / P_{roof})^{1/2}$

Table 19: Estimations of support pressure, length and spacing of primary support using the Q system (Ryder and Jager, 2002)

where

J_r = joint roughness number,
J_n = joint set number,
P = support pressure (MPa),
B = span of excavation (m),
H = height of excavation (m),
ESR = excavation support ratio,
L = length of reinforcement (m),
s = spacing of reinforcement (m),
C = yield strength of bolt (MN).

EXAMPLE 29

For the purpose of tunnel stability and support design in low stress environments, the rock mass classification Q-System (Barton et al. 1974) can be used to assign rock quality (Q) values to the underground access excavations. Q-rating values for the ore body and its footwall, as determined from the drill core logging data, were shown to be between 1.9 in the ore body and between 2.1 in the immediate footwall material. If it is assumed that all tunnel development will be placed within the footwall material, suggest primary support requirements for the following excavations, making use of the provided ESR values to the right:

- Decline (maximum width m);
- Footwall drive (maximum width m);
- Drill drive (maximum width m).

ESR values per excavation type were provided and are shown below.

Type of Excavation	ESR
Long-term excavations: Permanent excavations (Declines, chambers, workshops etc.)	1.6
Medium-term excavations: Footwall drives, muck storage chambers etc.	2
Temporary mine openings: Drill drives, drawpoints etc.	5

The length of the tendon support for tunnel stability is purely a function of the size of the excavation, but the spacing and the necessity for fabric (or areal coverage) support, is a function of the rock mass quality. For the design of tendon length and spacing, the following are used:

$$\text{Bolts } (L)=[(2+0.15B)/ESR]*FOS$$

$$P_{roof}=((0.2Q^{-0.33}))/J_r$$

$$\text{and} \quad s=(C/P_{roof})^{0.5}$$

where:

- B is the span of excavation (m),
- ESR as per information provided,
- FOS is the Factor of Safety for tendon design (taken as 1.2),
- P_{roof} is the support pressure (MPa) required for the support installed,
- Q is the Q Index and
- C is the yield strength or capacity of the bolt (MN).

back

back

For the support recommendations specific to this environment, the Q-ratings and ESR values provided, equate to the following support requirements after completing the calculations suggested above:

Excavation	ESR	Span (m)	Q Index	Life of excavation	Tendon length (m)	Support unit yield strength (MN)	Support pressure (MN/m2)	Spacing (m)
Decline	1.6	5	2.1	Long term	2.1	0.196	0.104	1.4
Footwall drive	2	5	2.1	Medium term	1.7		0.104	1.4
Drill drive	5	5	1.9	Short term	0.7		0.108	1.3

NOTE

It is advised that a spreadsheet is created with the relevent formuli entered and these calculations performed.

Support of shafts using the Q Index

Stacey (2001) indicated a method to estimate shaft support requirements, based on the Q rock mass classification system. For the vertical shaft walls he noted that:

- $Q_{wall} = 5 \times Q_{roof}$ when $Q > 10$,
- $Q_{wall} = 2.5 \times Q_{roof}$ when $0.1 < Q < 10$ and
- $Q_{wall} = Q_{roof}$ when $Q < 0.1$.

Using the value of Q_{wall} determined for the rock mass through which the shaft is passing, the charts in Figure 55 and Figure 56 can be used to estimate support requirements. In this case, the diameter, or largest dimension of the shaft, should be considered as the span on the ordinate axis. The implication from this is that, as a guide, a 6m shaft will not require support if the rock mass quality exceeds a Q value of 3.

REFERENCE

- McCracken, A and Stacey, T.R., 1989. Geotechnical Risk Assessment of Large Diameter Raisebored Shafts. Shaft Engineering, Inst Min Met, pp 309 – 316
- Page 364, Ryder J.A., Jager A.J., A Textbook on rock mechanics for tabular hard rock mines, 2002.
- Page 207, Hutchinson D.J., Diederichs M.S., Cable bolting in underground mines, 1996.

LEARNING OUTCOME 3.1.2.15

CONNECTION 13

Refer to paper 1 "Learning Outcome 6.2.5" on page 218

DETERMINE ROCKMASS 'M' AND 'S' PARAMETERS FOR THE HOEK AND BROWN CRITERION BASED UPON ROCKMASS CLASSIFICATION RESULTS

LEARNING OUTCOME 3.1.2.16

CONNECTION 14

Refer to paper 1 "Learning Outcome 6.2.5" on page 218

DETERMINE ROCKMASS DEFORMABILITY FROM JOINT STIFFNESS AND ROCKMASS CLASSIFICATION RESULTS.

back

PRINCIPLES OF SEISMICITY

This section of the material guides was not created in the same order as the outcomes listed in the syllabus, as it was deemed more important to present the necessary information in a logical order. At the start of each part of this section, the outcomes discussed in that part are listed for reference to the syllabus.

LEARNING OUTCOMES

SEISMICITY PART 1

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Identify, describe and explain the processes that give rise to rupture in rock
- Identify, describe and explain the rupture processes in solid rock
- Identify, describe and explain the rupture processes along a pre-existing fault plane
- Describe and explain the phenomena of foreshocks and aftershocks
- Identify, describe and explain the features of a seismic event that are used to locate the source of the event

CONCEPT OUTCOMES 1-5:

Describe the material types that can store significant amounts of strain energy under load (strong), fail suddenly and violently (brittle) and release strain energy violently (such as biscuits).

Define a seismic event; explain source mechanism vs. failure mechanism. Describe initiation and termination of failure and a range of accepted source models (from micro-fracturing to fault slip). Explain the preparation phase and the principal of stress adjustment over distances and events directly triggered by ground-motion.

- Durrheim, R.J., Anderson, R.L., Cichowicz, A., Ebrahim-Trollope, R., Hubert, G., Kijko, A., McGarr, A., Ortlepp, W.D. & van der Merwe, N. (2006). Investigation into the risks to miners, mines, and the public associated with large seismic events in gold mining districts, Dep of Minerals and Energy, RSA
- Ortlepp, W.D. (1997) Rock Fracture and Rockbursts: an illustrative study. Monograph series M9. South African Inst. Min. Metall., 220pp.
- Richardson, E., Jordan, T.H. (2002) Seismicity in Deep Gold Mines in South Africa: Implications for tectonic earthquakes, BSSA Vol. 92, No.5
- Hofmann, G.F. (2007) Coulomb stress triggering for seismic hazard assessment of geological structures in underground mines, SANIRE Symposium proceedings
- Press, F., Siever, R. (1986) Earth, Freeman and Co.
- Mendecki, A.J. (ed. 1997) Seismic monitoring in mines, Chapman & Hall 1997.

back

INTRODUCTION

CONNECTION 15

"Findings of the Investigation"into the risks to miners, mines, and the public..." at the end of this section

The foundations of mine seismology lie within earthquake seismology, the study of tectonic movements, strain accumulation on plate boundaries and the generation of seismic waves when these highly-strained rock masses fail suddenly and violently. Compared to natural earthquakes, mining-related tremors are limited in size due to the processes that generate stress concentrations (mining), and the geological features that accumulate strain, which are regional fault systems in the most severe cases. The magnitude associated with mining-related tremors has historically been limited to around 5.3, while the largest tectonic earthquakes ever recorded are above M=9. Among the largest mine tremors was the Stilfontein event on 9 March 2005, which became subject of a detailed investigation initiated by the Chief Inspector Mines (Durrheim et al., 2006).

Before delving too far into mining-induced seismicity, it is appropriate to visit some of the principles of seismology as they are equally valid for the mining environment. Earthquakes are the result of tectonic stresses forming mostly on the boundaries of tectonic plates that form the hard layer of the Earth's crust. The crust, with a thickness of 20-40km, is merely the skin of the much larger Earth measuring approximately 6 170km from surface to centre. Unlike the human skin, the crust consists of rigid plates that move at speeds of up to 20cm per year, in some places drifting apart, in others colliding with each other (Figure 57).

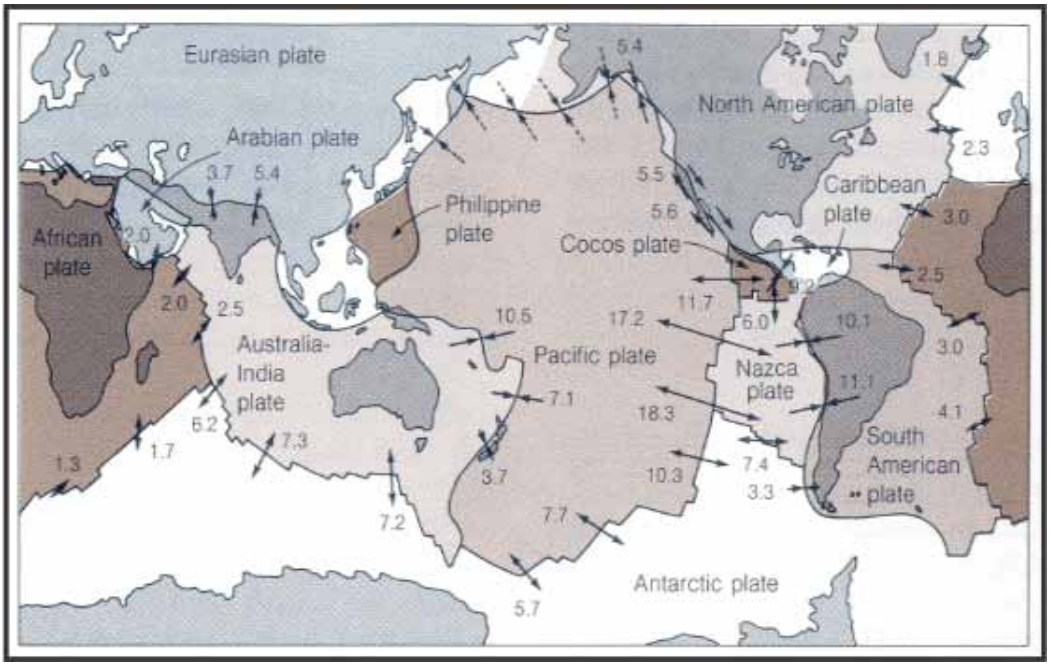


Figure 57: Drift velocities and directions of movement of major tectonic plates (Press & Siever, 1986)

back

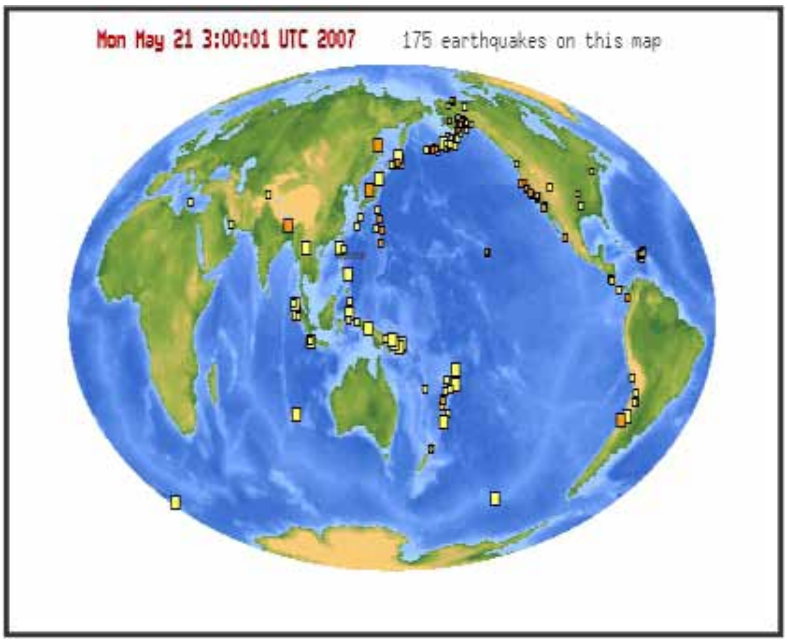


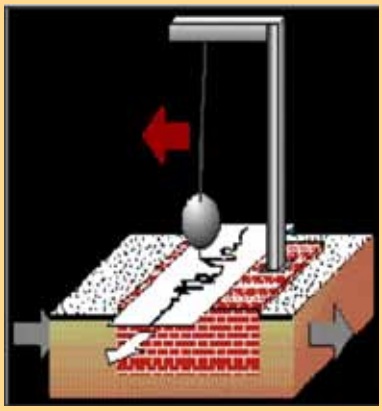
Figure 58: Roughly 48 hours of world-wide earthquake activity (National Earthquake Information Centre of the US, http://neic.gov)

The geographical correlation between plate boundaries and daily earthquake activity as recorded by worldwide seismic networks is evident (Figure 59) and leads to the conclusion that processes internal to the crust, although driven by processes deeper within the Earth, are the likely causes of natural seismicity. There appears to be a definite concentration of seismic sources along plate boundaries and a similarly clear absence of activity from the interior regions of most continents. Judging from the snapshot in Figure 59, Africa is void of seismic sources, which is not entirely correct as will be shown further below.

Earthquake strength is usually measured either in intensity (Mercalli scale) or in magnitude (Richter scale). The Mercalli scale developed in 1902 quantifies damage severity resulting from an earthquake and therefore depends on the distance from the source. It is not based on a measurement, which makes it less objective, but allows the comparison of historic quakes from before the time of seismographs. The Mercalli intensity scale ranges from I to XII, with XII being total destruction. Table 1 quantifies intensities and offers comparisons to the magnitude scale. The table suggests that the 2005 Stilfontein tremor would have been classified as an intensity 5 to 6 event on the Mercalli scale.

back

INTERESTING INFO



Simple seismograph (Source: http://earthquake.usgs.gov/learning/faq.php**)**

Giuseppe Mercalli (www.geo.mtu.edu/UPSeis/Mercalli.html**)**

Mercalli Intensity (at epicentre)	Magnitude	Witness Observations
I	1 to 2	Felt by very few people; barely noticeable.
II	2 to 3	Felt by a few people, especially on upper floors.
III	3 to 4	Noticeable indoors, especially on upper floors, but may not be recognised as an earthquake.
IV	4	Felt by many indoors, few outdoors. May feel like heavy truck passing by.
V	4 to 5	Felt by almost everyone, some people awakened. Small objects moved. Trees and poles may shake.
VI	5 to 6	Felt by everyone. Difficult to stand. Some heavy furniture moved, some plaster falls. Chimneys may be damaged.
VII	6	Slight to moderate damage in well-built, ordinary structures. Considerable damage to poorly-built structures. Some walls may fall.
VIII	6 to 7	Little damage in specially-built structures. Considerable damage to ordinary buildings, severe damage to poorly-built structures. Some walls collapse.
IX	7	Considerable damage to specially-built structures, buildings shifted off foundations. Ground cracked noticeably. Wholesale destruction. Landslides.
X	7 to 8	Most masonry and frame structures and their foundations destroyed. Ground badly cracked. Landslides. Wholesale destruction.
XI	8	Total damage. Few, if any, structures standing. Bridges destroyed. Wide cracks in ground. Waves seen on ground.
XII	8 or greater	Total damage. Waves seen on ground. Objects thrown up into air.

Table 20: Modified Mercalli Intensity Scale
http://www.geo.mtu.edu/UPSeis/index.html

back

back

Contrary to Mercalli, Richter devised his magnitude scale in 1935 to be based on actual measurements using a specific type of seismograph. Magnitude M was to represent energy release by being proportional to the logarithm of the maximum recorded wave amplitude. This also became known as the local magnitude, a term that is still in use today.

Charles Richter (www.geo.mtu.edu/UPSeis/intensity.html**)**

Later, techniques were devised to estimate the diameter of the source area As and the slip distance ds at the source, which allowed an estimate of seismic moment . Moment magnitude is now the most widely used strength measure used by seismologists. The mine magnitude ML, also sometimes called 'local mine magnitude' and denoted M, is a measure of both seismic energy release and seismic moment and is calculated from the spectral content of recorded seismograms. It represents both energy release, i.e. damage severity, and source size, i.e. extent of damage, and can be modified and calibrated to local conditions. Mine magnitudes do not relate to a fixed benchmark and are relative rather than absolute measures of source strength.

The final note in this introduction relates to the richness of information that can be gained from seismic wave recordings. Location and time are only the beginning of seismic data analysis, as waves carry much information not only about their origin, but also about the medium they travel in. Waves only travel on straight paths where the medium is undisturbed and homogenous. Analysing reflected and refracted waves allows seismologists to develop models of rock mass properties with great detail of layering, gradual changes in horizontal and vertical directions and sudden variations in material properties such as faults and dykes, ground water saturation and mineralisation (Figure 59). Vibroseis methodology uses artificially generated waves to explore the upper crust for mineral deposits, to delineate ore reserves and estimate their depth. Without the help of seismologists, the Earth's interior would be largely inaccessible and its structure only subject of speculation.

back

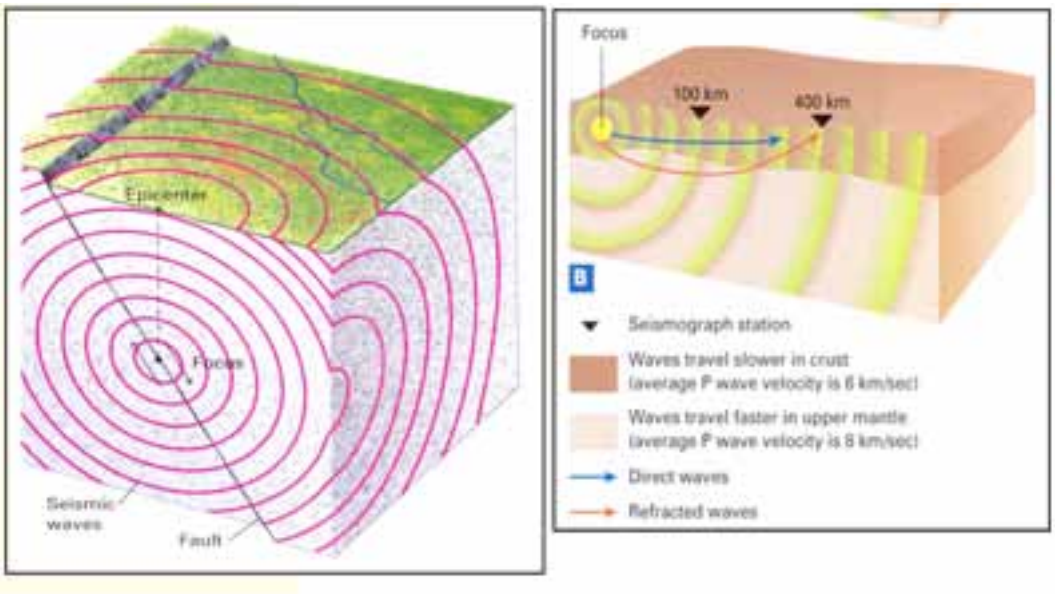


Figure 59: Seismic waves radiated from the earthquake focus travel outwards in all directions (left); refracted waves penetrate deeper rock mass layers and arrive first, carrying information about the boundaries between layers and the velocity differences (right) (Thompson & Turk, 2007)

In the mining environment, instead of tectonic movement driving stress accumulation and strain release, the creation of excavations, their position and shape are seen as the causes of seismic failure. One finds a multitude of mining layouts and geological conditions that are associated with seismic events. Most common are mining faces and abutments, pillars and geological structures. These features occur in different geotechnical areas and under varying stress conditions. It is not surprising that seismic response to mining activity varies greatly and that seismic events have characteristics that reflect the conditions under which they occur.

Seismicity in Southern Africa is recorded by the South African National Seismological Network (SANSN) operated by the Council for Geoscience in Pretoria. A plot of source locations during a three-month period in early 2009 shows some degree of clustering along the African rift system and a number of event groups within the boundaries of South Africa. All of these are tectonic earthquakes associated with minor intra-plate movements of the African continent (Figure 60(a)). Where the SANSN's sensitivity is sufficient and the tectonic processes are excluded, mining-related seismicity is recorded (West to east Rand, Figure 60(b)). It is estimated that approximately 90% of seismic activity in South Africa is mining related.

back

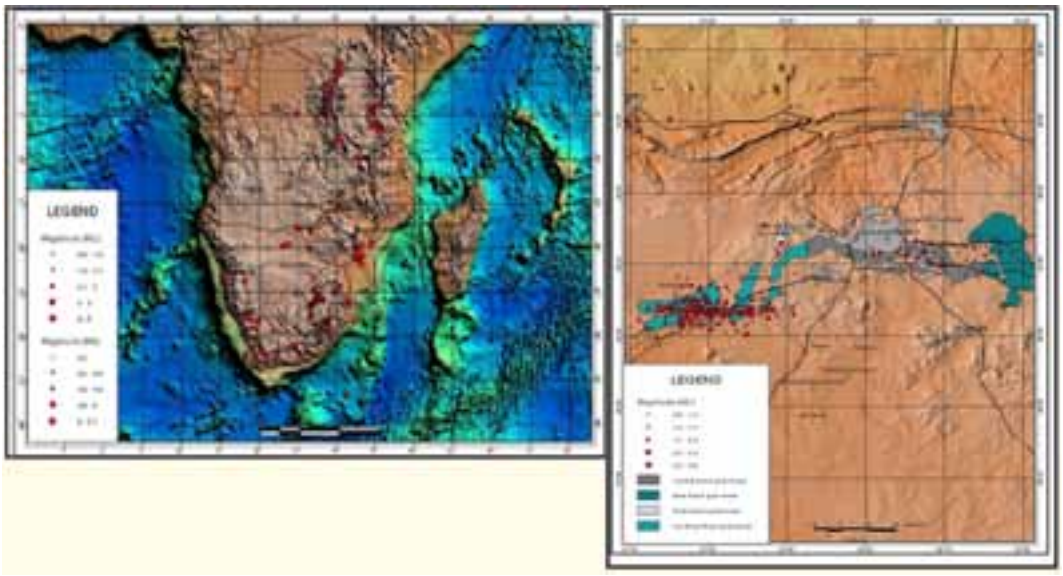


Figure 60: Seismicity recorded by the national seismic network in Southern Africa (Left) and around past and current gold mining areas (right).

The subsequent chapters introduce the principles of seismicity, the techniques applied to collect seismic data and the methodologies of analysing and interpreting these data. To be of use to the South African rock engineer, the analysis and evaluation have to relate directly to the local mining practice and rock mass environment, a prerogative for the authors of this text.

RUPTURE IN ROCK

Witness to the close relationship between mining and seismicity is the distribution of seismic events recorded during blasting time, which usually shows rapid activity increase with the onset of production blasting in the afternoon (Figure 61). It is evident that smaller events are more clustered around mining faces and occur mostly during and directly after blasting time. The larger events occur more randomly throughout the day and at greater distances to active mining.

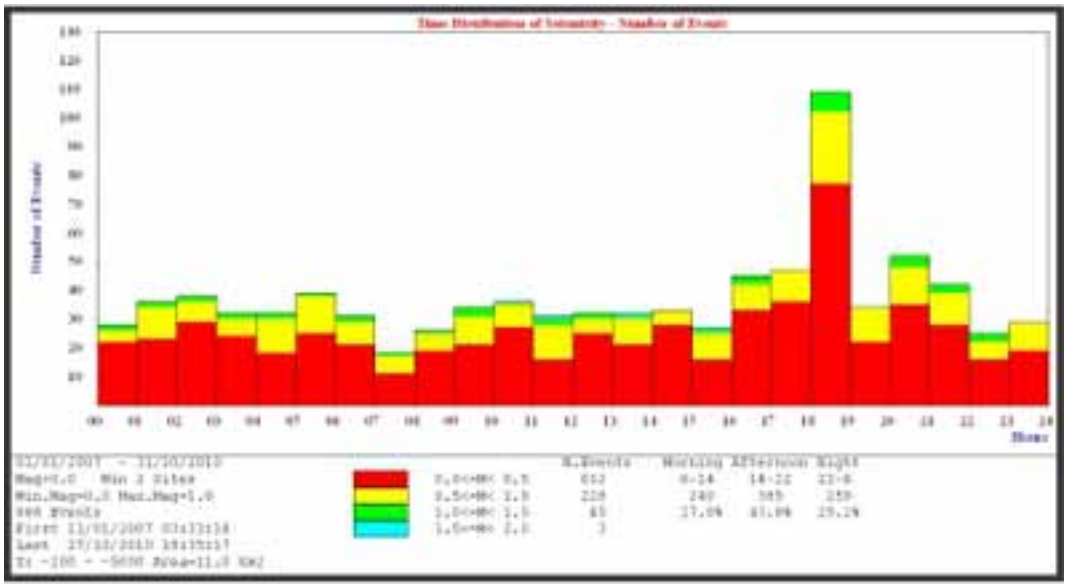


Figure 61: Time/day plot

back

back

Any deep-level mining creates stress values in excess of the strength of the rock. In intermediate to deep-level mining environments, we find a multitude of layouts and geological conditions that can be associated with seismic events such as mining faces, abutments, pillars or remnants and geological structures. In massive mines, crown pillars and ore slender pillars may attract seismicity, in tabular mines abutments and stabilising pillars may be seismically active. But first, let us define the term 'seismic event':

DEFINITION

Seismic event = Sudden release of strain energy due to stresses exceeding the strength of the rock mass.

Any deep-level mining creates stress values in excess of the strength of the rock. This is particularly true of narrow tabular excavations such as stopes in deep South African gold and platinum mines.

The above definition is clear about the suddenness of failure and the imbalance between load and strength. Therefore, either stresses must be too high for the intact and flawless rock mass to withstand or there must be a local weakness in the rockmass where failure initiates. In addition, if failure occurs slowly (creep), it is not considered seismic and no seismic waves will be radiated. Such a slow failure can therefore not be detected and located by means of a seismic system. Short source duration, the time during which the source emits wave energy, is essential for successful location.

Common to the various rupture types is a preparation phase in which stresses accumulate and deformation increases at a slow rate. This is followed by the failure phase when the rate of deformation suddenly rises and stresses increase beyond the material's strength (or the strength drops below the applied stress). Rapid failure leads to load shedding and the release of stored strain energy, which results in permanent deformation in the rock volume and is the source of elastic waves being generated. Fundamentally, two types of shear failures can be distinguished, which are distinct from crushing or buckling:

- The fracture of a previously intact rock volume, which fails once stresses exceed the strength of the rock mass (Ortlepp, 1997)
- Sudden failure along an existing fault plane or other pre-existing discontinuity by shear forces exceeding frictional strength created by clamping forces on the plane.

The first type, fracturing of previously intact rock, is the result of stresses exceeding the rock strength. Whether the initial microscopic failure continues to grow into a large fracture zone or not is a matter of energy balance: If the strain energy released by the fracturing process is sufficient to break additional bonds between neighbouring particles, the fracture zone will continue to grow.

The second type, slip on a pre-existing discontinuity such as a fault or a bedding plane, is initiated when the stress field induced by mining activities disturbs the equilibrium of tectonic forces and 'unlocks' the fault. The stability of discontinuities depends on cohesion, frictional properties and the clamping forces. Changes to any of these three may lead to failure. Slip is usually initiated at a small patch of the fault plane where clamping forces are low due to weak infilling or the presence of water. The slip area grows only if it is beneficial from an energy balance point of view, i.e. when the energy released from the contact area exceeds the amount of energy required to overcome friction.

back

back

Figure 62 illustrates a rock engineer's understanding of brittle failure where rising stress levels result in controlled elastic deformation up to a point of sudden failure when the rock loses part of its carrying ability. The failure phase is accompanied by structural damage to the rock mass. The failure zone retains some amount of residual strength and is able to continue to deform at reduced load levels.

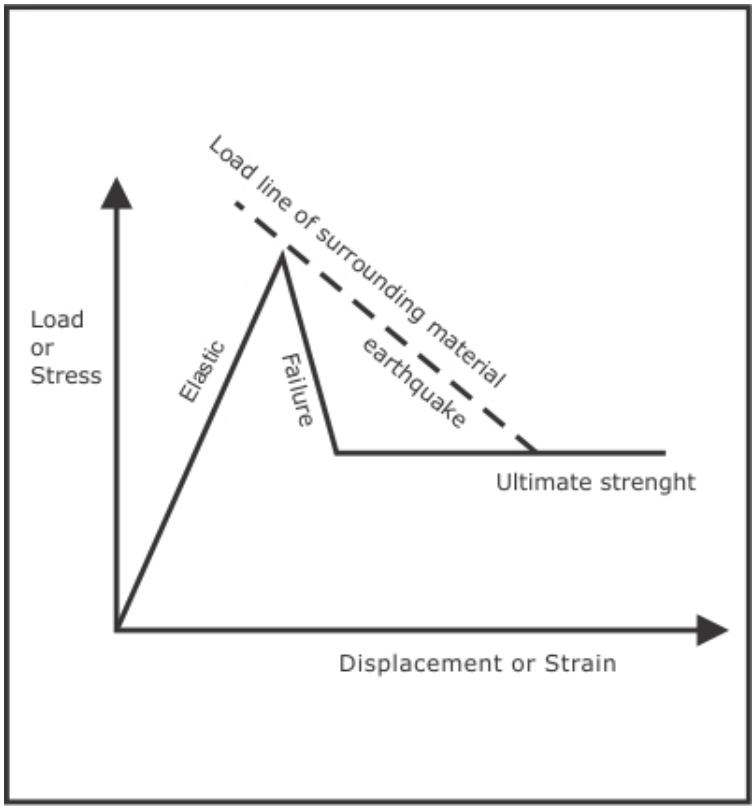


Figure 62: Schematic illustration of stress-strain behaviour showing brittle failure and resulting in an earthquake, as seen by Rock Engineers

These two types can be referred to as 'bursts' and 'slips'. Fracture type events tend to be limited in magnitude to the medium range, while shear slips are seen as responsible even for very large regional events with a magnitude in excess of 5 (Richardson & Jordan). For a more detailed discussion of seismic failure mechanisms refer to the study materials for Paper 3.1. As an illustration of the complexity of seismic sources, consider Figure 63 suggesting seismic sources that are neither burst nor slip type, but rather a mixture of both (Mendecki et al., 1997).

back

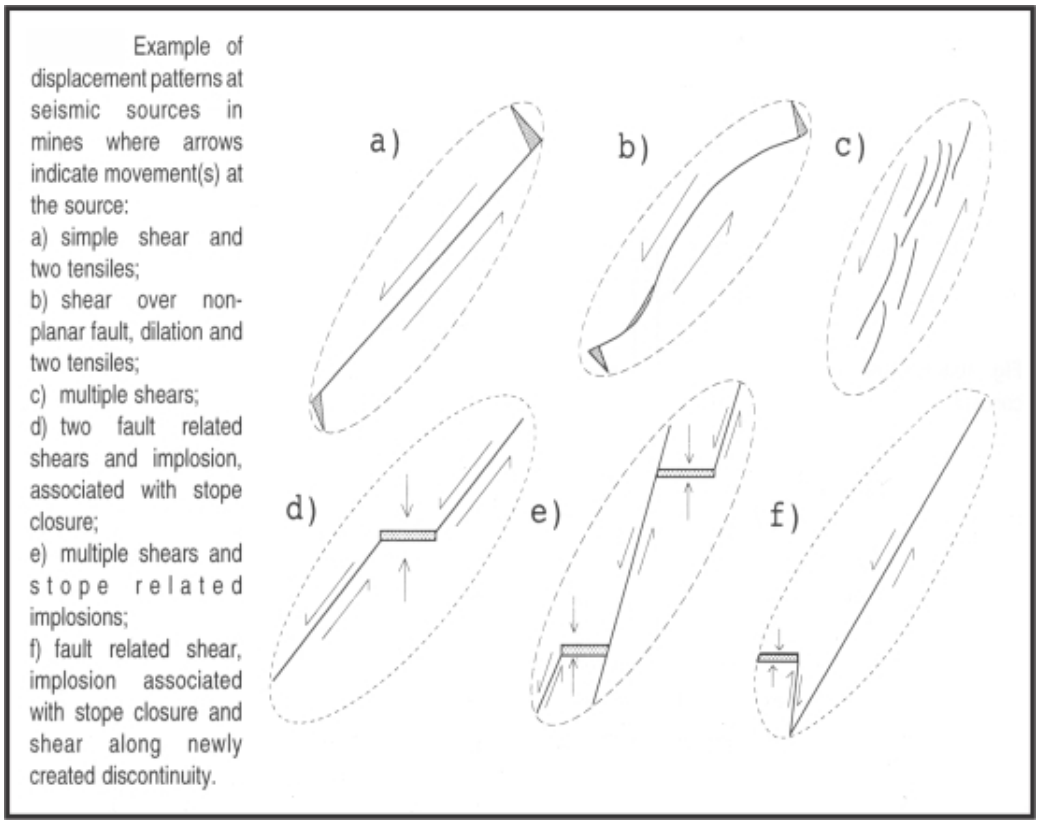


Figure 63: Seismic events associated with mining that have relatively small sources (a-c) and larger sources (d-f). (Mendecki et al., 1997)

It should be noted that basic physical considerations prevent large events from taking place on objects of small dimensions, simply because the amount of stored strain energy in, for example, a 10m pillar is insufficient to generate an ML=3.0 event. For the same reason, a 3x3m development end is unlikely to be associated with an ML=2.0 event, unless it negotiates a geological structure. As a rough guideline, Table 21 lists various emission sources and the magnitude range and source sizes associated with them.

	Tunnel development	Mining face	Dip pillar remnant	Abutment strike pillar	Geological structure
Magnitude	-0.5 … 0.0	-0.5 … 1.5	1.0 … 2.5	1.5 … 3.5	2.0 … 5.0
Source radius	5-8m	5-80m	50 - 120m	80 - 200m	100 - 300m

Table 21: Magnitude range and source sizes associated with different source types

This rule of size limitation is especially true for geological structures: Minor faults that daylight in one panel only and are associated with a minor throw cannot be the source of large magnitude events. When searching for possible candidates for an M=2 event, one should look for structures that extend over several panels and have a traceable length of several ten metres.

Large seismic events can under certain circumstances be triggered by smaller events. This could happen, for instance, because of waves dynamically interacting with a rock mass close to failure or because of stress redistribution resulting from seismic failure. The shaking associated with a passing wave may be sufficient to destabilise a fault contact. There are documented examples of two faults interacting over a period of

back

days before coming to rest in a deep level mine (Essrich, unpublished). In the case of smaller events preceding large events, these are termed 'foreshocks'. The efforts to predict large events in mines are often based on the assumption of a preparation phase in which a major instability is preceded by small failures that are detectable by sensitive seismic monitoring equipment.

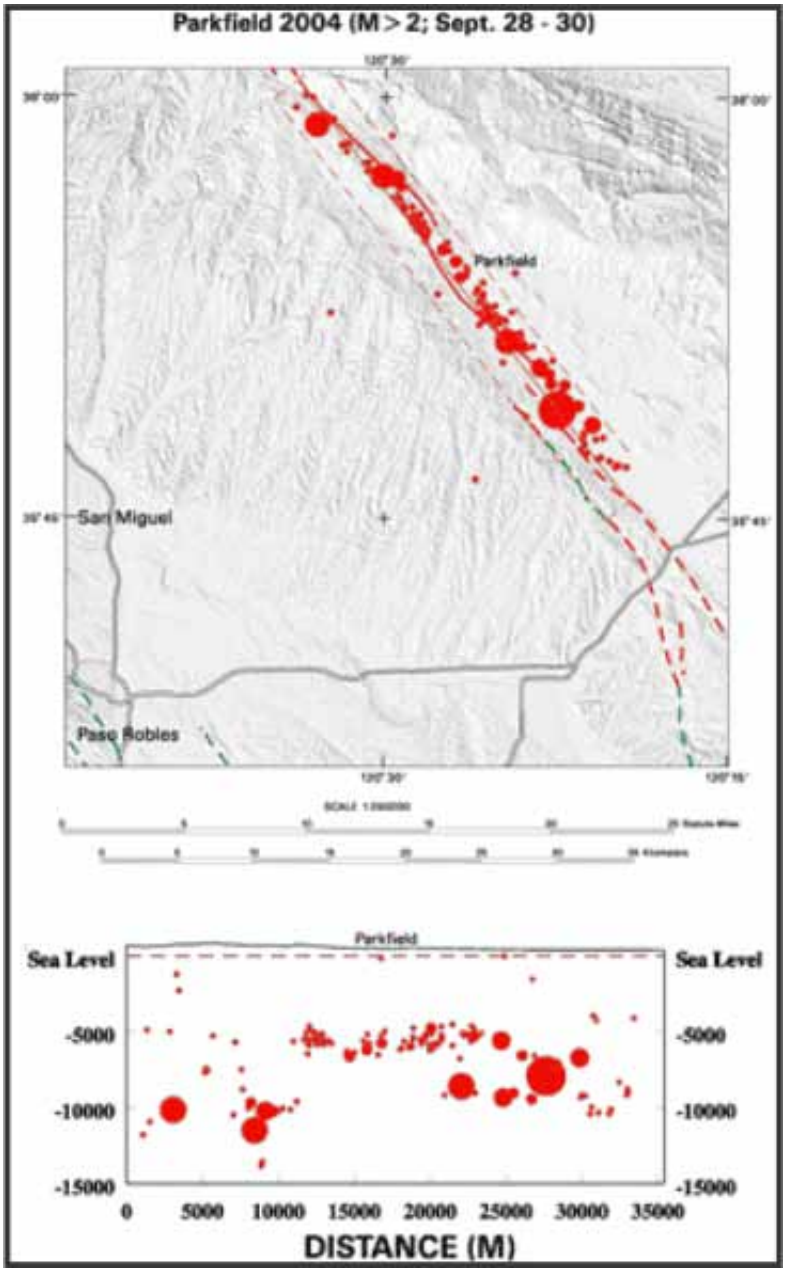


Figure 64: Map showing M>2 aftershocks following M6.0 earthquake in Parkfield, CA. Below the map is a cross-section showing earthquake depth along the fault within the dashed red box (USGS)

CONNECTION 16

Read more about event prediction in the study materials for Paper 3.1.

A second type of associated activity is events following a main event – so-called 'aftershocks'.

DEFINITION

For a seismic event to be considered an aftershock, it must:

- occur before the regional seismicity level returns to long-term average;
- occur within one rupture length of the main event;
- be smaller in magnitude than the main event.

back

back

In a wider sense and for useful hazard assessments, the first criterion is commonly used.
In mining, as much as in tectonically-induced seismicity, event sequences often involve more than one geological feature. The main event may have occurred on Fault A, but the subsequent events of the sequence are located on Fault B or some other seismically unstable feature. Figure 65 shows a plan and a section view of closely clustered aftershocks that followed within three days after an M=6 earthquake near Parkfield, California in 2004. In this case, it seems, only one fault region was involved.

The decay in aftershock frequency is described by Omori's law that relates the rate of aftershocks n to the time t after the main shock:

$$n(t) = K/(c+t)^p$$

c and p are parameters that are close to 0 and 1 respectively. n(t) is a hyperbolic function that approaches zero for large values of t. Within the context of mining-induced seismicity, n(t) is useful to estimate safe re-entry periods after large events. Parameters c and p are calibrated using recorded aftershocks subsequent to main shocks of magnitude 1, 2 etc.

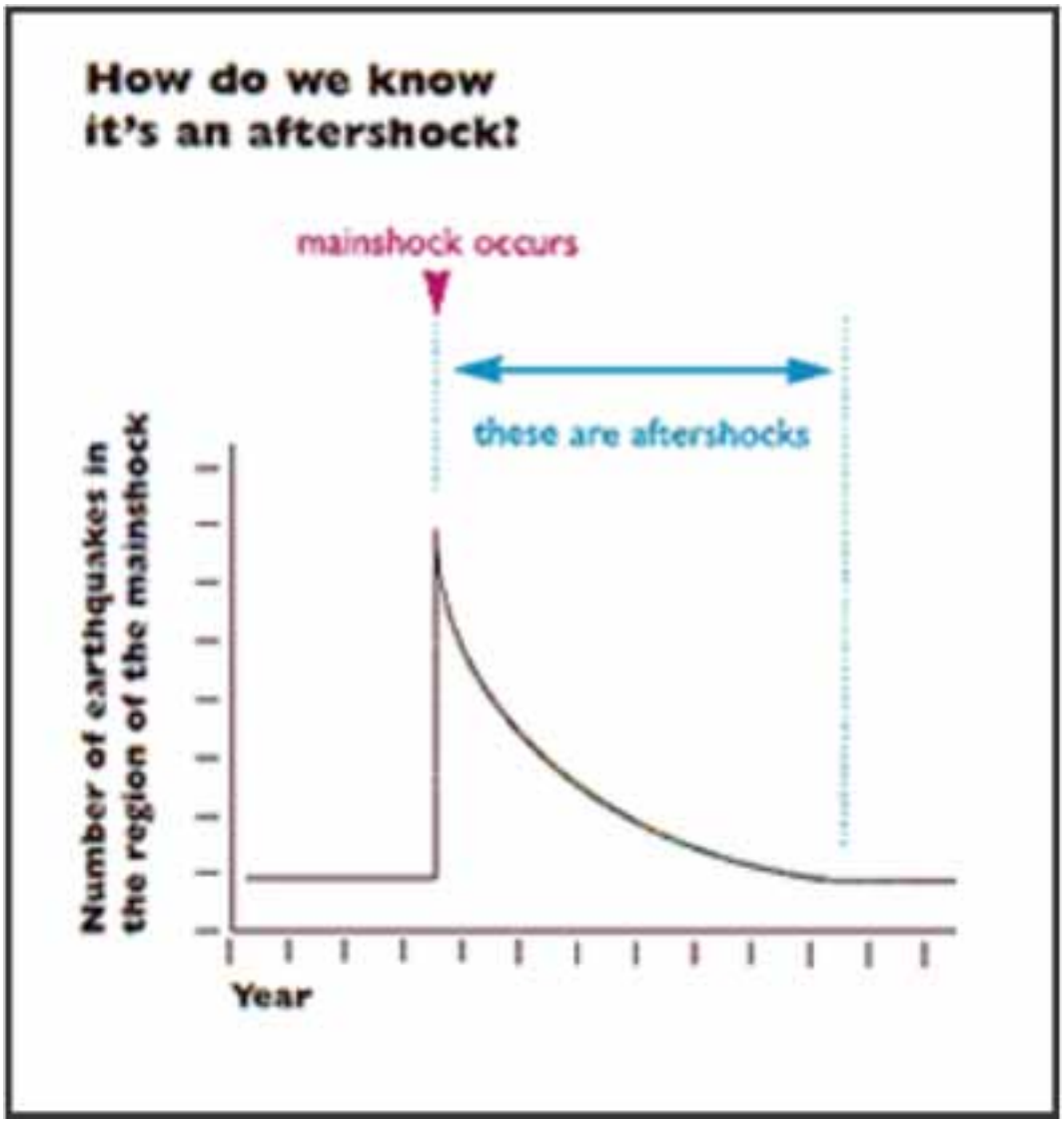


Figure 64: Number of aftershocks in region of main shock (http://earthquake.usgs.gov/learn/glossary**)**

back

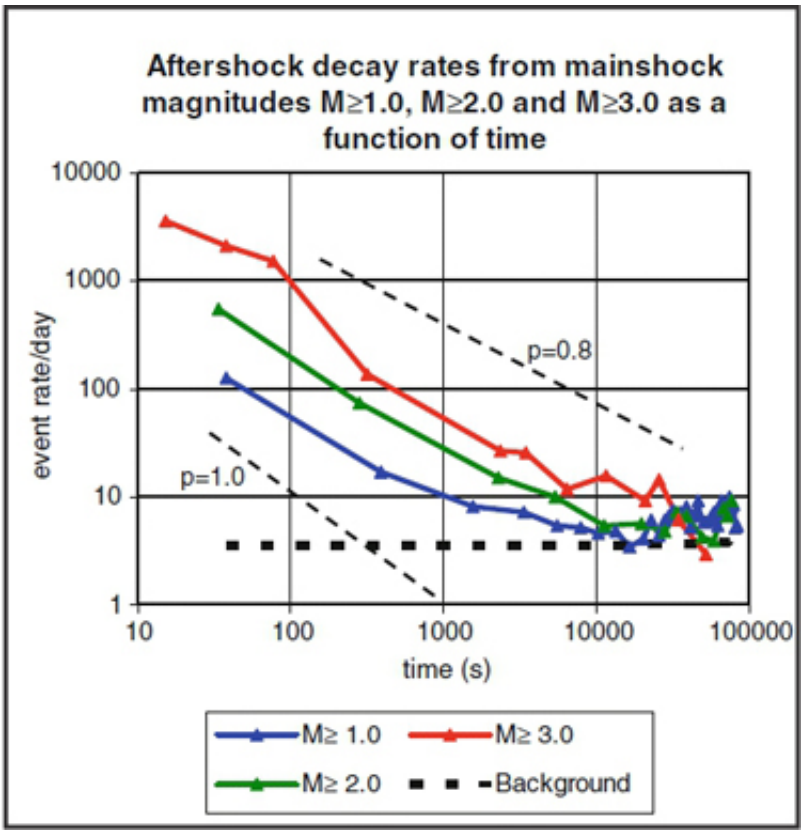


Figure 65: Actual aftershock activity compared to the activity predicted by the modified Omori law (colour by main shock magnitude, within 1000m of the main event; Kgarume et al., 2010)

- Durrheim, R.J., Anderson, R.L., Cichowicz, A., Ebrahim-Trollope, R., Hubert, G., Kijko, A., McGarr, A., Ortlepp, W.D. & van der Merwe, N. (2006). Investigation into the risks to miners, mines, and the public associated with large seismic events in gold mining districts, Dep of Minerals and Energy, RSA
- Ortlepp, W.D. (1997) Rock Fracture and Rockbursts: an illustrative study. Monograph series M9. South African Inst. Min. Metall., 220pp.
- Richardson, E., Jordan, T.H. (2002) Seismicity in Deep Gold Mines in South Africa: Implications for tectonic earthquakes, BSSA Vol. 92, No.5
- Hofmann, G.F. (2007) Coulomb stress triggering for seismic hazard assessment of geological structures in underground mines, SANIRE Symposium proceedings
- Press, F., Siever, R. (1986) Earth, Freeman and Co.
- Mendecki, A.J. (ed. 1997) Seismic monitoring in mines, Chapman & Hall 1997.
- Kgarume, T.E., Spottiswoode, S.M. & Durrheim, R.J. (2010) Deterministic properties of mine tremor aftershocks, Deep and High stress conference, Santiago, Chile, October 2010.

Findings of the Investigation into the risks to miners, mines, and the public associated with large seismic events in gold mining districts, Durrheim et al., DME & CSIR 2006:

1. *The magnitude 5.3 seismic event that occurred at DRDGOLD's North West Operations on 9 March 2005 and its aftershocks can be ascribed to past mining.*

2. *There are distinct differences between the mining districts with respect to the maximum magnitude of events. The Klerksdorp and Free State districts tend to experience larger events than the East, Central, West and Far West Rand. These differences are ascribed to differences in geological structure and mining layouts.*

3. *Seismic events are likely to be triggered as the water level rises in mines that have been closed and allowed to flood. It is unlikely that any event triggered by a rising water table will have a greater magnitude than the events that occurred during mining.*

4. *If the flooding of old workings adjacent to current operations is being contemplated, a survey of the mining geometry and geology must be conducted to ensure the integrity of the rock mass comprising the water barrier pillar.*

5. *A range of technologies, ranging from stoping layout and sequencing strategies to local support methods, is available to mitigate the risks of underground damage resulting from large seismic events.*

6. *There is no merit in placing slimes or backfill in areas that have been mined out in the past for the purposes of reducing the risks of large seismic events.*

7. *Some buildings in gold mining districts are considered to be vulnerable to damage, and even collapse during seismic events, posing safety and financial risks.*

LEARNING OUTCOMES 3.2.

SEISMICITY PART 2

This part describes seismic waves generated by seismic events, how they propagate in rock and what determines their velocity of propagation.

- Describe and explain the phenomenon of seismic body waves
- Describe, explain and distinguish between different types of seismic body waves
- Describe and explain the phenomenon of seismic surface waves
- Describe and explain how seismic surface waves are generated
- Describe and explain how seismic surface waves are related to seismic body waves
- Describe and explain the differences between seismic body waves and seismic surface waves
- Identify and describe the factors affecting the velocity of propagation of seismic waves
- Describe and explain how the rockmass environment affects the velocity of P-waves and S-waves
- State typical velocities of P-waves and S-waves in hard rock and soft rock environments
- Describe and explain how this information may be used in locating the sources of seismic events

Concept outcomes 5-14:
Introduce simple source models such as single force acting at a point resulting in 3D radiation patterns for P- and S-waves (temporary disturbances); describe propagation in solid materials, reflection and refraction

back

on interfaces incl. free surfaces, conversion to surface waves. Explain simple location methodology (triangulation based on measured travel times and assumed wave velocities). State examples for actual measured elastic wave velocities.

REFERENCE

- Al-Sadi, H.N. (1980) Seismic exploration technique and processing, Birkhäuser Verlag.
- Bolt, B.A. 1978. Earthquakes: a primer, W.H. Freeman and Company, San Francisco. 241 p.
- Pujol & Herrman, 1990
- Reid, H.F. (1910) On mass movements in tectonic earthquakes and the depth of the focus, Gerlands Beitraege zur Geophysic 10, 318-351.
- Telford, W.M., Geldert, L.P., Sherriff, R.E., Keys, D.A. (1976) Applied geophysics, Cambridge University Press.
- Milev, A.M., Spottiswoode, S.M., Noble, B.R., Linzer, L.M., van Zyl, M., Daehnke, A. & Acheampong E. (2002) The meaningful use of peak particle velocities at excavation surfaces for the optimisation of the rockburst criteria for tunnels and stopes, GAP 709 final report.
- Russel, D. 1999. Acoustic Animations, Kettering University, Applied Physics. (add web site)

Seismic waves

The previous chapter explained the conditions for dynamic rock mass failure to occur and set out the main characteristics of seismic events in a mining environment. If the failure is sudden, a small portion of available strain energy (usually 1-5%) is converted to elastic wave energy. These waves carry information not only about the time and location of failure, but also about the source dimension, the mechanism of rupture and the source strength. For this reason, seismic waves are of particular interest to seismologists who analyse seismic data with the aim to reduce seismic hazard on mines.

INTERESTING INFO

The position of a seismologist is comparable to a road accident investigator who only has sound recordings of a vehicle collision, no pictures or other material evidence

Irrespective of the source mechanism, two principal waves are generated at the source: A compressional wave similar to a sound wave, and a transversal wave similar to what can be seen on water surfaces. Their propagation velocities differ: Because of their higher speeds, longitudinal waves are also called the primary or P-waves (they arrive first at a seismometer). They consist of a package of compressed and dilated or stretched zones that travel together. The particle motion associated with the P-wave front is parallel to the direction of propagation as seen in Figure 66. In contrast, transversal waves propagate with lower velocity (secondary or S-wave). Their particle motion is in a plane normal to the direction of propagation and the deformation caused by the wave front is a shear movement (Figure 67).

back

back

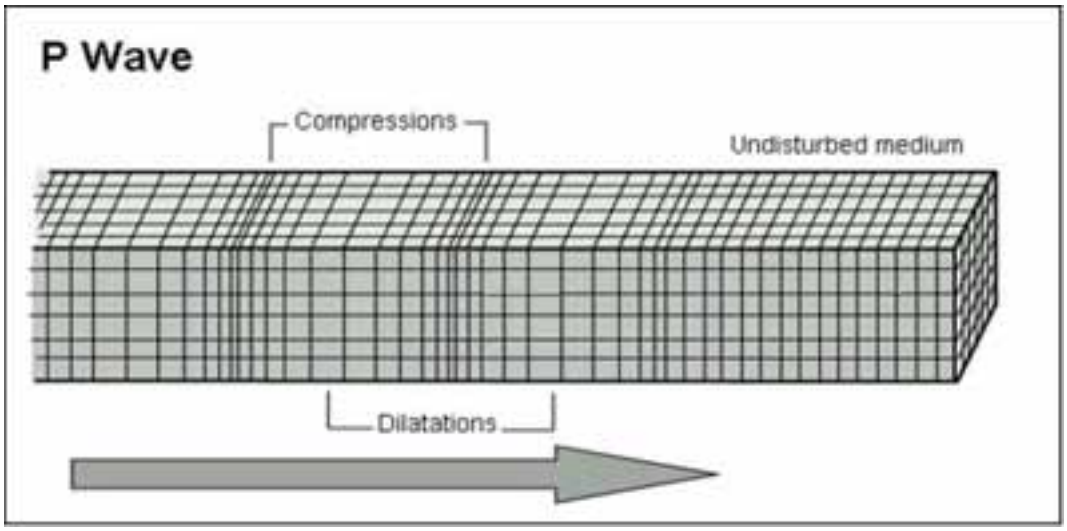


Figure 66: Particle motion associated with primary (or compressional) waves; from Bolt (1978)

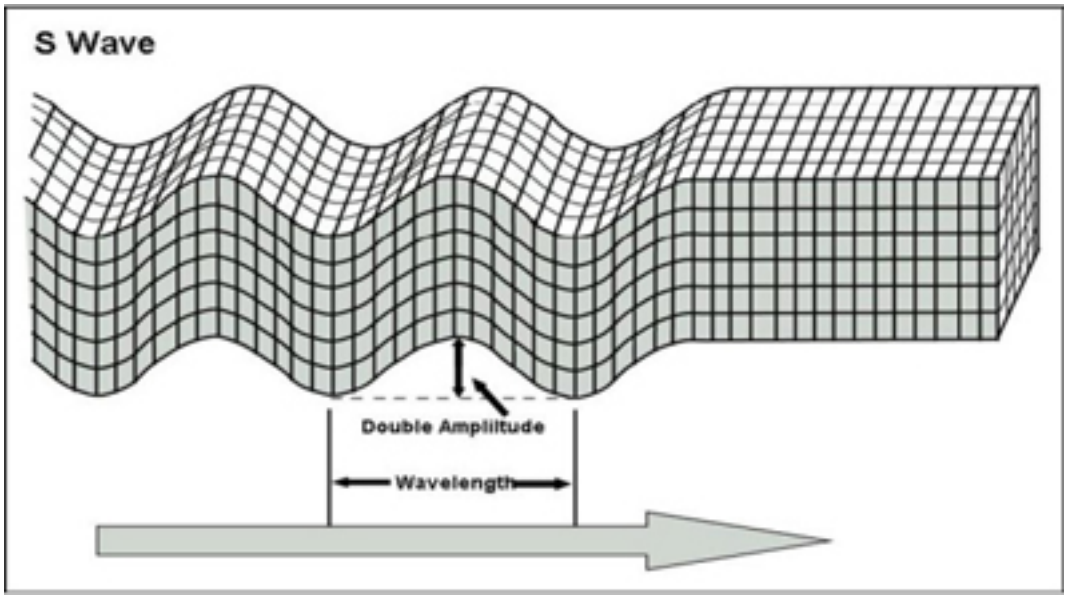


Figure 67: Particle motion associated with secondary (or transversal) waves; from Bolt (1978)

BRIDGING GAP 2

Refer to "vector" on page 17 to describe physical parameters with magnitude & direction

These two wave types are always generated by an event and the question could be asked, why? How does one explain that under all the different conditions under which seismic failure occurs, there is always a portion of the released energy converted into each of these two waves? For an answer, it is instructive to picture a single, directed force acting at a point in the rockmass to represent the force acting at the source.

back

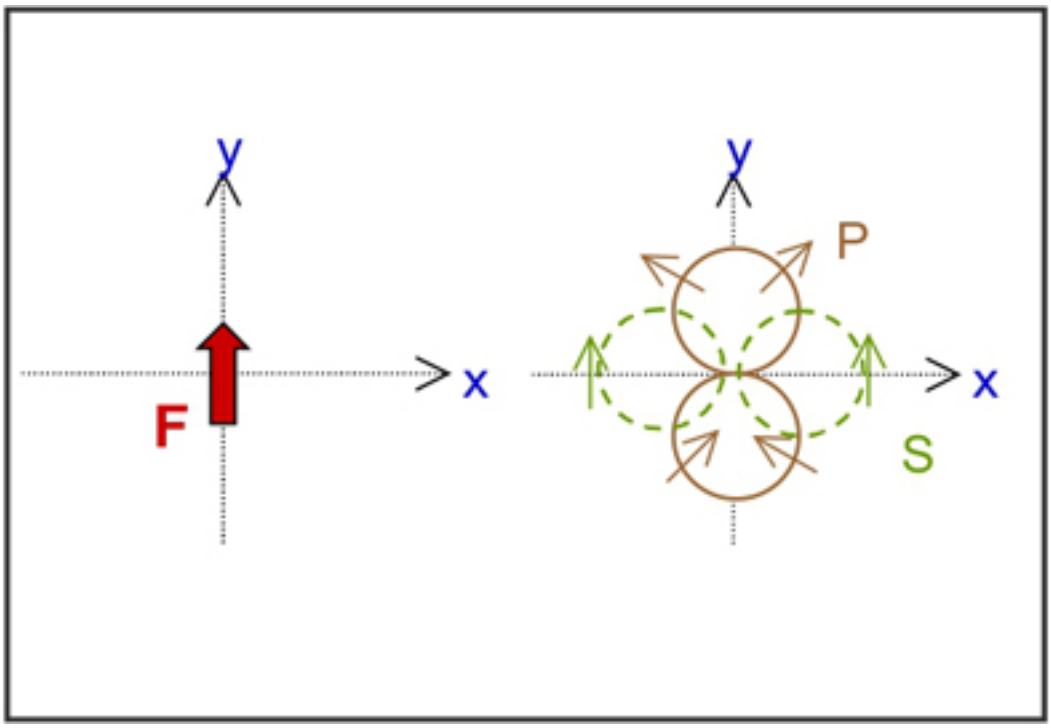


Figure 68: Single force F applied to origin of a 2D coordinate system (left) and resultant radiation pattern of P and S waves (right)

The time during which the force acts must be short in order to result in a rapid spike-shaped velocity increase leading to a short pulse of deformation in the rock. As a consequence of the deformation, elastic waves emitted from this source have a certain radiation pattern, as depicted in Figure 69 (two dimensions only).

INTERESTING INFO

The two wave fields of the P- and S-wave consist of particle motion with varying intensity depending on direction. This is known as the radiation pattern of the source and results in differing amounts of energy per unit area depending on the angle of emission. Figure 71 shows the displacement the vector field associated with the single force in Figure 70 (After Pujol & Herrman, 1990), as usual only for isotropic, elastic and homogenous media.

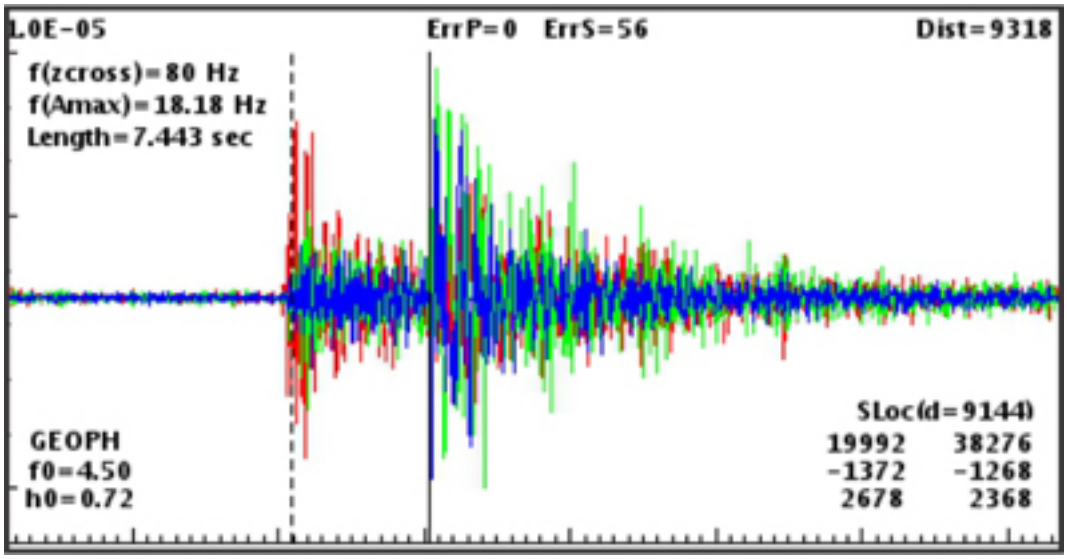


Figure 69: Sample seismogram (tri-axial sensor)

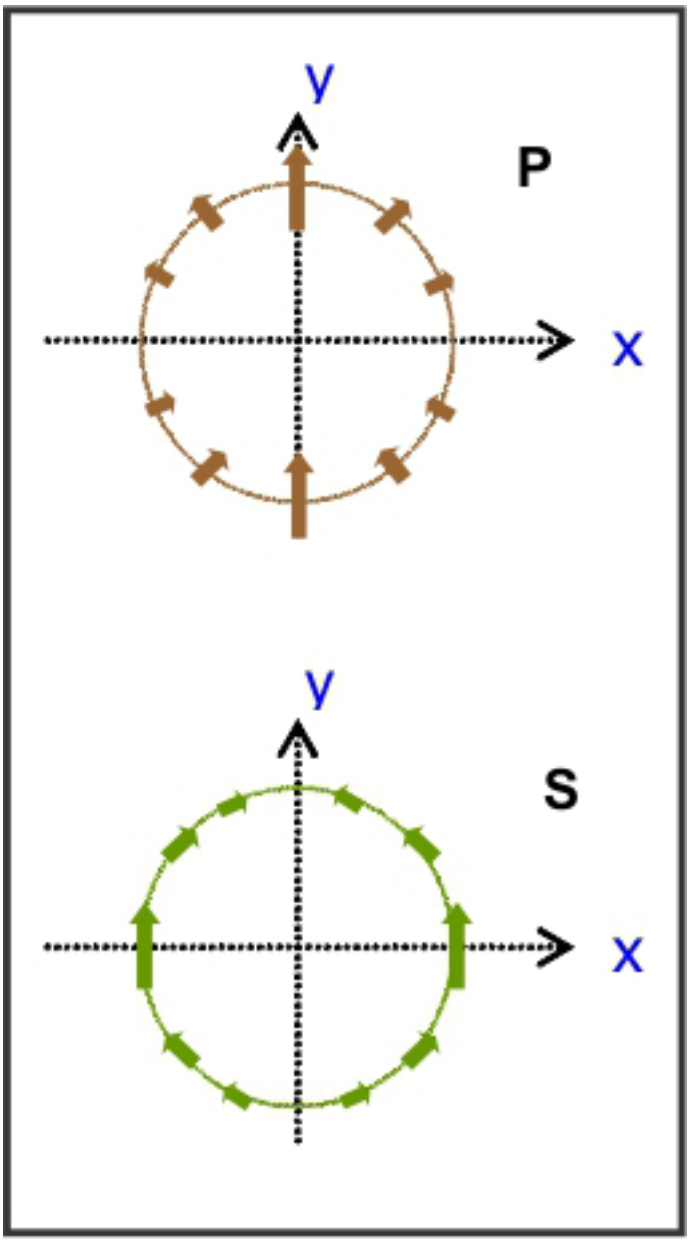


Figure 70: Wave vector field of P- and S-wave emitted from single force source

Each wave field has two lobes and the direction of particle motion and its amplitude depend on the angle of emission. The compressional wave field has a zone of movement away from the source (above F) and a zone of movement towards the source (below F). The amplitude is at a maximum parallel to F and a minimal normal to the direction of F. The shear wave field has highest amplitudes in the plane normal to F, and is weak parallel to F. All S-vectors are aligned with the applied force.

Seismic body waves always travel outwards and away from the source. Their wave fronts expand continuously until all of their energy is attenuated. Should this wave front encounter a free surface, part of its elastic energy can be converted into surface waves, such as Rayleigh and Love waves.

The reason for the conversion lies with the principle of energy conservation: Wave energy cannot be transmitted into air and has to stay in the solid body, where it is partly reflected and partly converted to surface waves. Therefore, surface waves are generated by incident body waves or by rapid energy release on or near a surface, such as an impact or an explosion.

back

Under favourable conditions, surface waves can travel considerable distances before being absorbed. The particle motion in a passing Rayleigh wave is elliptical on a vertical plane as shown in Figure 71. Love waves travel faster having velocities intermediate between shallow and deeper S-wave velocity. Particle motion is that of a horizontal shear wave (Figure 72).

Because of their complex polarisation pattern (path on which particles move when disturbed by the wave front), and because of possible amplification of amplitudes, surface waves can be more destructive than body waves (Milev et al., 2002).

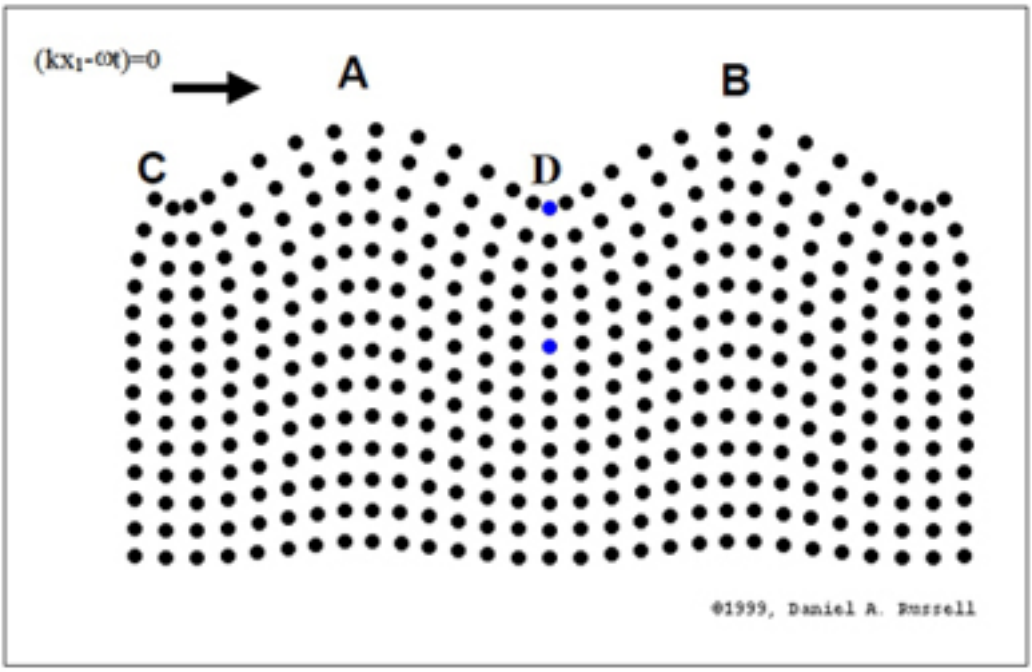


Figure 71: Sectional view of a Rayleigh wave front travelling from left to right. Note the maximum lift at points A and B, the maximum downward displacement at D. (http://paws.kettering.edu/~drussell/demos.html)

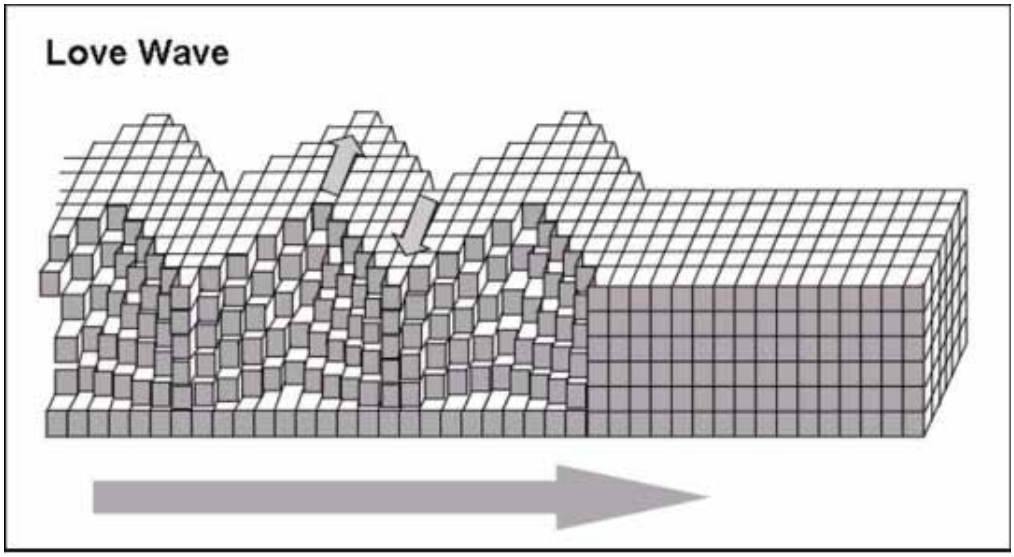


Figure 72: Love waves require at least a two-layered medium with the upper layer having a low velocity (Bolt, 1978)

BRIDGING GAP 3

"Glossary" on page 3
Waves

back

back

ANIMATION

http://www.kettering.edu/physics/drussell/

The characteristic differences between the various wave types relate to the propagation velocity and the particle motion associated with each wave type.

Wave type	Velocity	Particle motion
Longitudinal (P)	highest	parallel to direction of propagation
Transverse (S)	0.7 VP	normal to direction of propagation
Rayleigh	0.9 VS	elliptical retrograde
Love	below VS	parallel to surface, normal to direction of propagation, only in low-velocity layer beneath surface

Rayleigh wave amplitudes decay exponentially with depth and travel at velocities lower than S-waves depending on material properties: When the Poisson ratio is 0.25, VR is approximately 0.9VS. The Love wave velocity approaches VS in the surface layer (Telford et al., 1976). Because of the retrograde ground motion associated with Rayleigh waves, they are considered more damaging to buildings than Love waves and there is some evidence that Rayleigh waves can be damaging to the stope hanging wall (Milev et al., 2002)

After leaving the source volume of the seismic event, waves propagate at a certain speed, they have dominant frequencies and wave length and all of these variables depend on the elastic properties of the medium. For an elastic medium that is homogenous and isotropic on the scale of seismic waves (meters and tens of meters), Hooke's Law can be expressed in terms of the two Lamé constants, λ and μ. The latter is also known as 'shear modulus'. The wave velocities of P- and S-waves respectively are functions of these two constants:

$$Vp = \sqrt{\frac{\lambda + 2\mu}{\rho}} \qquad Vs = \sqrt{\frac{\mu}{\rho}}$$

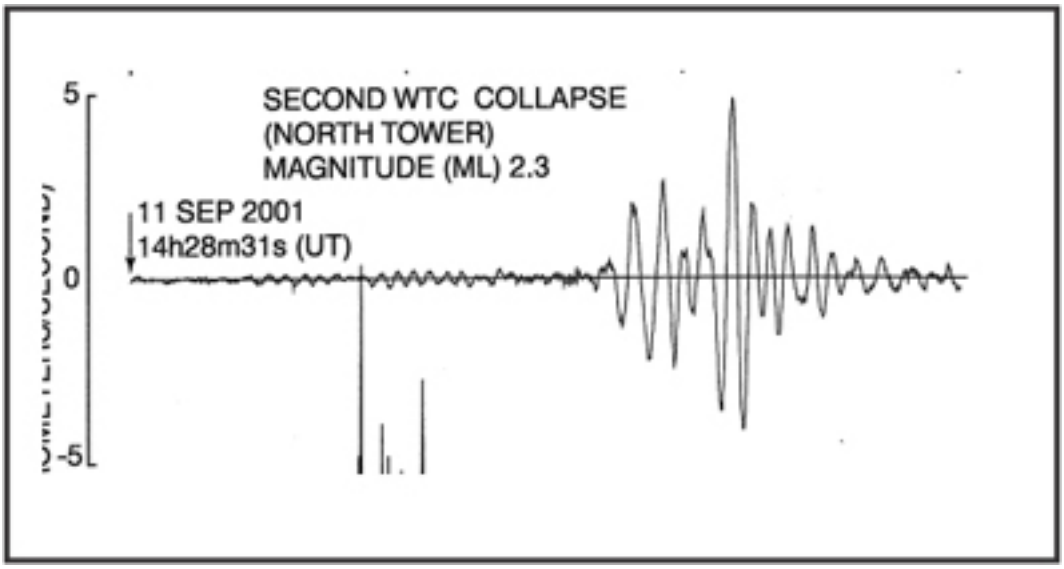
BRIDGING GAP 4

"Glossary" on page 3
E, σ, λ, μ=G

where ρ is density

INTERESTING INFO

Seismic waves generated by aircraft impacts and building collapses at World Trade Centre, New York City



Won-Young Kim in EOS, Vol 8, 20/11/2001, P.565 "112001 kim – seismic waves.pdf"

There are simple relations explaining the Lamé constants in terms of

back

back

Young's modulus, Poisson ratio, and bulk modulus (see References). For materials with a Poisson ratio of 0.25, Vp = √3 * Vs, i.e. P-velocity is approximately 1.73 times higher than S-velocity. Note from above relations that the wave velocity is constant for a certain material – it is not a function of the seismic source (such as amplitude and corner frequency), i.e. strong earthquakes do not radiate faster waves than small quakes do; the waves merely have higher amplitudes.
Because of the inability of liquids and gases to transmit shear movement, S-waves do not propagate in these materials. Table 22 lists a number of materials and their respective sound-wave velocities.

Solids at 20°C	Liquids at 25° C	Gases at 0°C and at atmospheric pressure
Granite 6000	Fresh water 1493	Oxygen 317
Quartzite 5700	Salt water 1533	Air 331
Copper 3750	Kerosene 1315	Nitrogen 339
Iron 5130	Mercury 1450	Steam (100 C) 405
Lead 1230		Hydrogen 1270

Table 22: Longitudinal P-wave velocities in m/s (after Al-Sadi, 1980)

These velocities, especially where they refer to rock types, are subject to wide variation. The common range of elastic wave velocities encountered in deep-level gold mines and intermediate level platinum mines is usually in the order of 4 000 to 6 500m/s for P-, and 2 400 to 3 800m/s for S-waves. A natural body of rock differs from the elastic, homogenous and isotropic materials that were assumed in order to derive the above expressions for Vp and Vs. It is usually layered and features some of the following:

- zones of intense fracturing,
- crushed zones,
- geological discontinuities at various orientations,
- zones of differing mineralization,
- highly stressed zones and de-stressed zones,
- excavations of varying size,
- backfilled or partially collapsed excavations.

Generally, an incompetent rock mass will allow waves to pass at lower speeds than a competent rock mass. Layering absorbs additional energy from wave fronts and forces waves to change direction, which will be elaborated on in more detail in the next section. These factors, the effect of the medium on wave propagation, contribute to inaccuracies in location and source quantification of seismic events (see Chapter 5.1 on seismic monitoring and data processing reference this more fully).

back

back

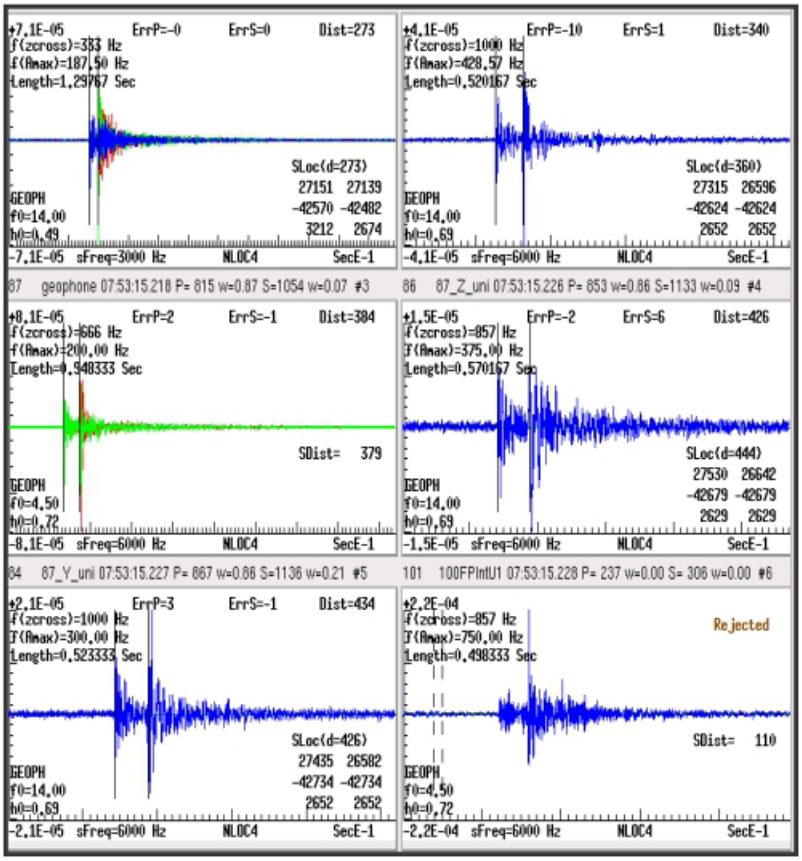


Figure 73: Set of seismograms recorded at increasing distance from the event.

In a seismic network, the vibrations resulting from a passing wave front are converted into electric signals by geophones and accelerometers. They are the basis for location and source parameter calculation. Recorded wave forms have, as we know, rather complicated shapes that differ not only from event to event, but also from sensor to sensor position. We also note that the mechanical process of seismic failure is a rapid one, taking perhaps a few milliseconds, but the length of seismograms is normally in the range of ten or hundred milliseconds. We conclude that the wave package from the seismic source, by the time it passes through the seismic station, is longer than the process that generated these waves. The reasons for this 'stretching' of the pulse, illustrated in Figure 74, are related to the medium through which waves travel and are discussed in detail below.

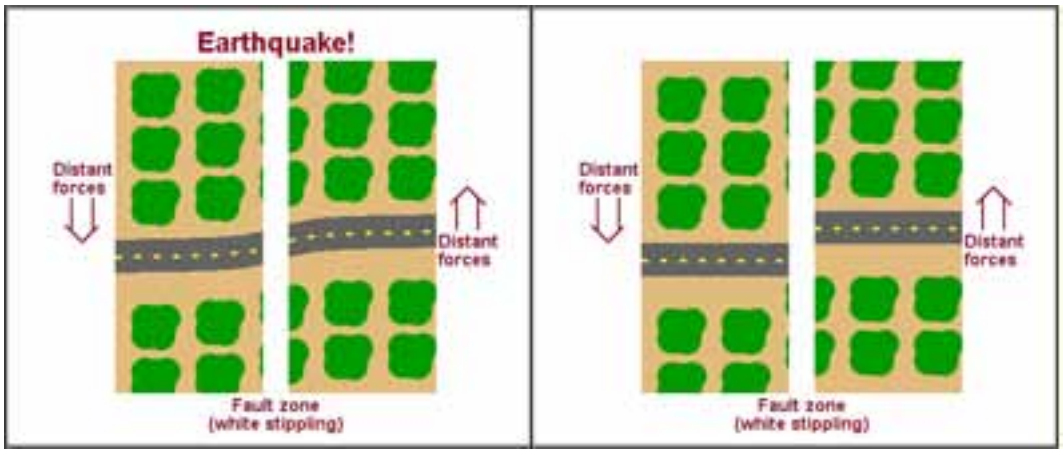


Figure 74: The elastic rebound theory (Reid, 2010) explains the build-up of stresses and the sudden release of strain energy which result in an earthquake (ElasticReboundDemo.mov, courtesy http://projects.crustal.ucsb.edu/understanding/elastic/rebound.html).

back

back

Generally, during dynamic failure, stored strain energy is converted into other forms of energy with a few percent being released as elastic energy in the form of elastic waves radiated from the source region. What leaves the seismic source is a very short pulse with a mixture of frequencies and amplitudes, a complex wave package with a warped and distorted three-dimensional surface.

Depending on the shape of this source and on its confinement (slip on a fault, crushing of a pillar, explosive charge etc.), the wave front will resemble a flat disk, a sphere, or a cylindrical rod or a mixture of these. Figure 75 shows three principal types of wave fronts, including a planar wave. At large distances from the source (10-15 source radii), wave fronts can be approximated as planar waves irrespective of the failure type. Planar wave fronts are easier to describe than curved fronts, as the disturbance is constant on a plane normal to the direction of propagation. We define the 'ray' or 'ray path' as the line perpendicular to the progressing wave front. Often it is more convenient to describe ray paths than wave fronts, as we will see below.

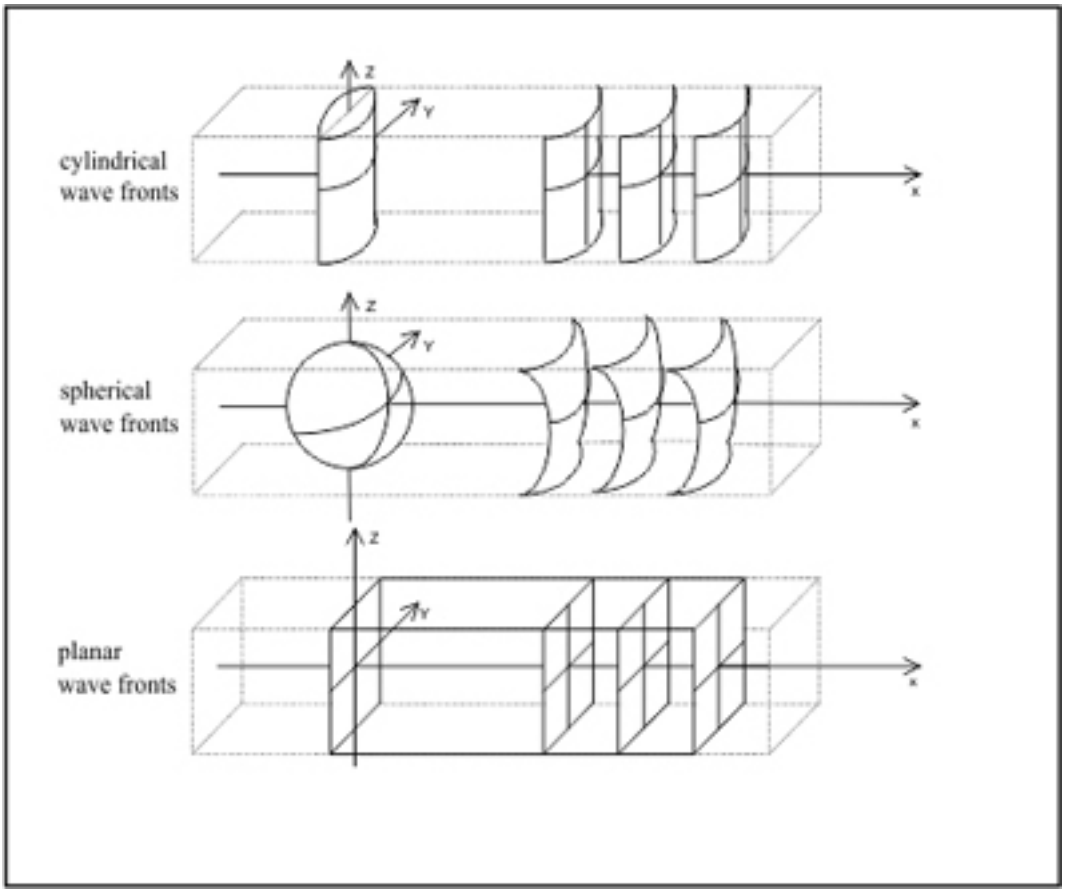


Figure 75: Cylindrical, spherical and planar wave fronts as simplified examples of seismic wave fields

Figure 75 displays three basic types of wave fronts, their shape being dependant on the shape of the source. For example, a cylindrical wave front could be associated with a blast hole, reflecting the shape of the charge. Spherical waves are a model for seismic waves close to a burst-type source. Flat or planar wave fronts are most appropriate for greater distances from sources, where the initial spherical cylindrical shape has flattened enough to be considered plane.

Fermat's principle may seem trivial, but it is an important aspect of wave radiation: A wave travelling between two points, for example seismic source and seismic station, will always take the quickest path, i.e. the path with minimum travel time. In a homogenous medium, the ray

back

back

follows a straight line. The calculation of event locations relies entirely on this principle, whether it is based on a homogenous or a layered earth model as in the case of ray-tracing algorithms.

BRIDGING GAP 4

Mine co-ordinate systems, LO27, R- vs. L-handed, orthogonal vs. others under "Co-ordinate Systems" on page 9

Seismic system software contains tools to allow location methods based on more complex geological models, which may deliver more accurate locations where the relevant stratigraphic data is available. But in general, locating a seismic source is a matter of triangulation using the arrival times recorded at a number of sensors and determining the point in space that matches the recorded times. Other, more advanced location algorithms are discussed in details below.

Locating an event in 3D requires the solution of a set of equations to determine four variables: Event source time t_0, and co-ordinates x_0, y_0 and z_0, usually in the mine's co-ordinate system. In any seismic network, the sensor co-ordinates are known, the P- and S-wave velocities are assumed, and the wave arrival times are measured by the network. In case of tri-axial sensor sets, the directions of the wave fronts as they enter the sensor sets can also be determined. Therefore, wave velocities play a central role in finding accurate locations and efforts are sometimes well placed to determine these accurately, for instance through calibration blasts.

REFERENCE

- Al-Sadi, H.N. (1980) Seismic exploration technique and processing, Birkhäuser Verlag.
- Bolt, B.A. 1978. Earthquakes: a primer, W.H. Freeman and Company, San Francisco. 241 p.
- Pujol & Herrman, 1990
- Reid, H.F. (1910) On mass movements in tectonic earthquakes and the depth of the focus, Gerlands Beitraege zur Geophysic 10, 318-351.
- Telford, W.M., Geldert, L.P., Sherriff, R.E., Keys, D.A. (1976) Applied geophysics, Cambridge University Press.
- Milev, A.M., Spottiswoode, S.M., Noble, B.R., Linzer, L.M., van Zyl, M., Daehnke, A. & Acheampong E. (2002) The meaningful use of peak particle velocities at excavation surfaces for the optimisation of the rockburst criteria for tunnels and stopes, GAP 709 final report.
- Russel, D. 1999. Acoustic Animations, Kettering University, Applied Physics.

LEARNING OUTCOMES

SEISMICITY PART 3

This part looks at the way waves are affected by the medium they traverse, gradual and sudden changes in rock type such interfaces that separate stratigraphic layers with different elastic properties.

- Describe and explain how the energy of a seismic body wave is attenuated as it propagates through elastic rock
- Describe and explain how the energy of a seismic body wave is attenuated as it propagates through non-elastic rock
- Describe and explain the phenomena of reflection and refraction of seismic body waves
- Describe and explain how seismic waves and the energy of seismic waves cause damage in excavations
- Describe and explain how seismic wave reflection and refraction cause damage in excavations

back

- Distinguish between seismic source mechanism and seismic damage mechanism
- Identify, describe and explain the conditions required for a rockburst to occur
- Identify, describe and explain the basic steps that are involved in carrying out a rockburst site investigation.

Concept outcomes 15-22:
Explain absorption and geometrical spreading resulting in attenuation; describe energy conversion in non-elastic materials; describe 'energy loss' due to reflection and refraction. Describe interaction of waves with free surfaces (reflection, amplification). Define common rockburst types and damage mechanisms. Explain rockburst site investigation techniques.

- Milev, A.M., Spottiswoode, S.M., Noble, B.R., Linzer, L.M., van Zyl, M., Daehnke, A. & Acheampong, E. (2002) The meaningful use of peak particle velocities at excavation surfaces for the optimisation of the rockburst criteria for tunnels and stopes, GAP 709 final report.
- Ortlepp, W.D. (2005) RaSiM comes of age – a review of the contribution, SAIMM RaSiM.
- Ortlepp, W.D. (1997) Rock fracture and rockbursts – an illustrative study, SAIMM Johannesburg.

Influence of the medium
The creation of elastic waves at the seismic source is, literally, only the beginning. The wave radiation patterns described in the previous chapter, the properties of the rock mass, layering, gradual and sudden changes in density and other parameters all impact on the way wave pulses propagate through rock: Rock mass properties affect the wave shape and decide how far a wave can travel.

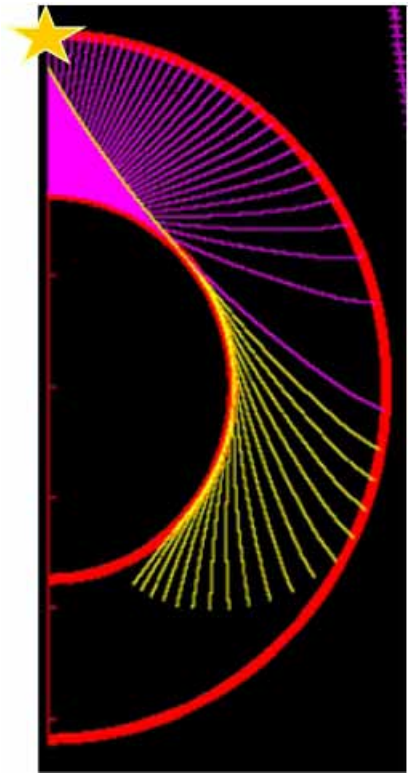


Figure 76: Earthquake waves travel along curved paths due to density increasing with depth

back

It is a common observation that waves weaken with growing distance from their origin. In the simple case of elastic and homogenous rock, wave amplitudes are weakened simply because the wave front grows as the wave expands in all directions. The amount of elastic energy released by the source has to supply a larger wave field surface as the distance from the source grows. If the source is a perfect sphere, its surface grows as d2, where d is the source distance and therefore the energy density decays as 1/d2. This effect is known as geometrical spreading, but is only one of several mechanisms attenuating waves. The following mechanisms contribute to the reduction of wave amplitudes in elastic rock:

- Absorption: The consumption of elastic wave energy to overcome elastic forces resulting in temporary displacement.
- Geometrical spreading: A decrease in energy density in the wave surface area as it spreads in three dimensions.
- Reflection: The partial loss of wave energy at interfaces.

Because of these mechanisms, wave amplitude generally decreases with distance from the source. They also suffer the loss of high frequencies first: A short, sharp pulse with high frequency content turns into a longer low-amplitude, low-frequency wave packet after travelling through rock. The attenuation function is exponential, amplitude depends on distance (or travel time) and the quality factor Q of the medium:

$$A(r) = e^{\frac{-\omega r}{2QV}}$$

Q can be determined from seismic data and is an adjustable variable in current seismic networks to allow for different rock characteristics. To know the way in which a medium affects wave amplitudes is essential for the process of 'source inversion', i.e. compensating for distance travelled and estimating properties of the source rather than the point in the rock at which the recordings were made. Attenuation is effectively a low-pass filter as depicted in Figure 78 for a 20Hz and a 200Hz P-wave in quartz-ite. The dimensionless quality factor in this case is 300.

back

back

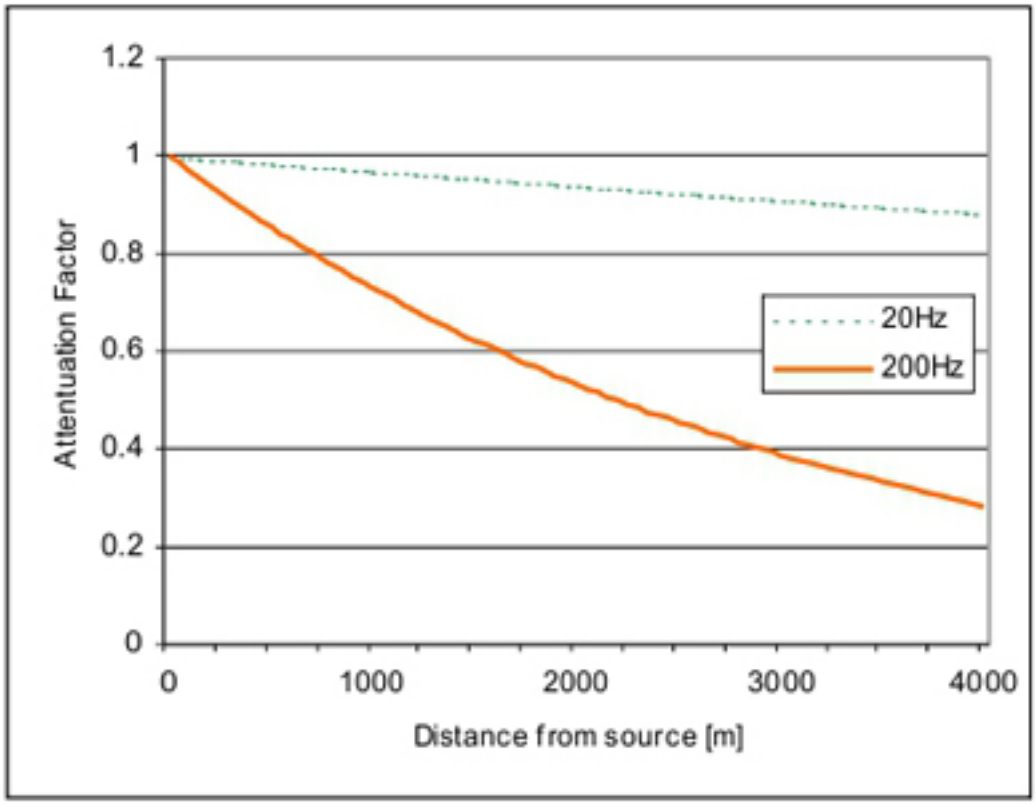


Figure 77: Attenuation function for two P-waves in quartzite for different dominant frequencies

Because of its filtering effect, attenuation leads to a distortion of the wave form by removing high-frequency content. Sharp rises are removed from the signal and the pulse becomes longer, a common observation in seismograms is recorded at growing distances from an event. The fewer high frequencies are contained in a signal the longer it is stretched in the time domain.

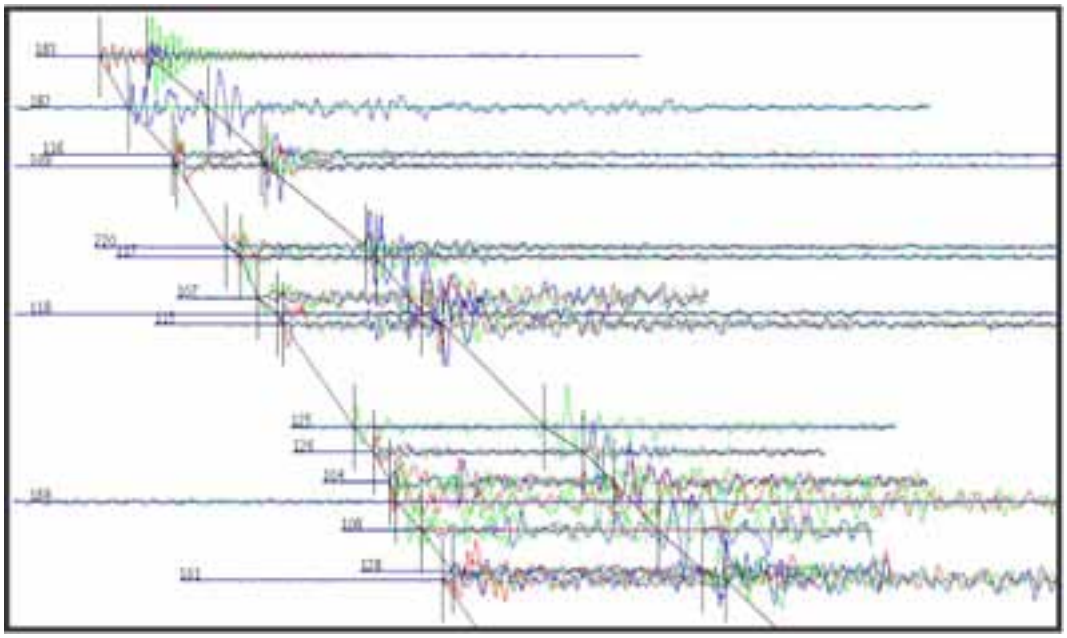


Figure 78: Increasing P-S separation and length of the wave package with growing distance from the source (top to bottom, wave amplitudes not to scale)

back

back

A further influence of the medium on signal shape can be attributed to dispersion. Dispersion is caused by a medium's property to transmit waves of different frequency with different velocities. A material shows normal dispersion when low frequencies travel faster than high frequencies; inverse dispersion describes the opposite. Dispersion leads to a distortion of the signal: A wave form changes its shape the longer it travels in a material with this property.

Non-elastic rock types:
In addition to the above mechanisms, in non-elastic rock types wave energy is converted to permanent deformation when particles cannot return to their original position due to broken molecular bonds. Part of the consumed energy is therefore used to overcome friction, turned into heat and into permanent deformation.

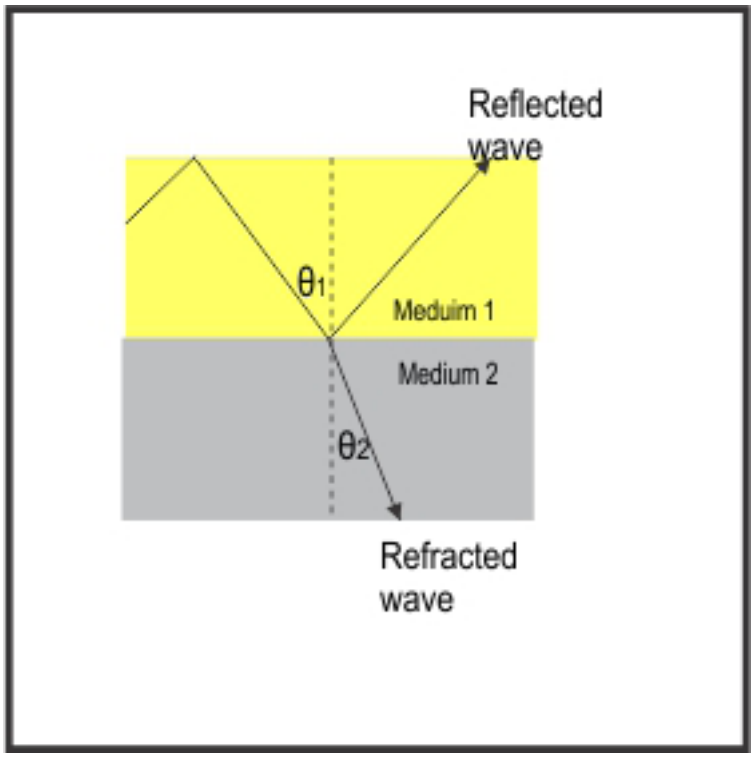
The third of the attenuating processes, reflection is associated with layer boundaries and excavation skins and deserves a more detailed discussion. All materials other than isotropic, homogenous, fully-elastic materials potentially alter the direction of ray paths. A stratified rock mass is likely to consist of rock types with varying velocity and density and, as a result, elastic waves change their characteristics when crossing an interface. Affected are angle of propagation, energy content, velocity, and spectral composition of the wave package.

Reflection occurs when both velocity and density of the material change on an interface. The reflection coefficient CR, the ratio of amplitudes of the incident to the reflected wave, is a function of the impedance $Z=\rho V$ on both sides of the contact:

$$C_R = (Z_2-Z_1)/(Z_2+Z_1)$$

If the product of density and velocity is greater in the second than in first medium, the reflection coefficient is positive. A phase change of π occurs when R is negative. A special case is the contact between a solid material and gas. For a P-wave hitting the earth's surface, $Z_2=0$ and C_R becomes -1, i.e. the wave's energy is reflected completely and the reflected wave phase is shifted by 180° to the incident wave.

back



Refracted and reflected wave ray
Question: Based on the angle θ2 , which medium has the lower velocity, top or bottom? http://paws.kettering.edu/~drussell/demos.html

Except in the case of total reflection, transmission occurs on the contact, i.e. partial penetration of wave energy into the second medium. Because of the need for energy conservation, transmission and reflection coefficients are related.

$$C_r = 1 - C_R^2 = \frac{Z_1 Z_2}{(Z_2 + Z_1)^2}$$

The transmission coefficient C_T (also called the efficiency of transmission) depends on the angle of incident, but no phase change is observed on the transmitted wave with respect to the incident wave. Two principal cases must be distinguished: Incident normal to the interface (no change in direction) and incident at an inclined angle when the transmitted wave leaves the contact at an angle other than the incoming wave, which is referred to as refraction. Snell's law determines the angle of incident and refraction and how they relate to the velocity contrast:

The latter case is called refraction: A change in direction of the wave ray when passing through an interface with velocity change. The velocities Vi on either side of the contact and the angles of incident α and refraction β are related to each other by Snell's law:

$\sin \alpha / \sin \beta = V_1/V_2$

with incident angle α, exit angle β

It is obvious that bending away from the normal occurs where the velocity in the second layer is larger than in the first layer. There is an angle of incident called critical angle αc at which no wave energy is transmitted into the second medium and the reflection becomes total.

An important principle applies to the relation between wave length and size of the object encountered by the wave: A change in energy content, phase or direction of propagation will only occur if the thickness of the

penetrated layer is comparable to the wave length of the elastic wave. In practical terms, a P-wave with a wave length of 300m will not 'see' and interact with a 1.5m thick layer of shales and will hardly be deflected by a 5m wide dyke. Wave length and object must be in the same size range for the two to interact with each other effectively.
C

EXAMPLE 30

Calculate the wave length of a P-wave in quartzite at f_c=150Hz

Worked example: Assuming a quartzite for rock type, P-wave velocity is in the range of 5 700 to 6 000m/s. Since frequency f, velocity V and wave length λ are related by V=f* λ, one can solve for wave length and insert the corresponding values for V and f (assume V to be 5 800m/s):

$$V = f\lambda \rightarrow \lambda = \frac{V}{f} = \frac{5800}{150}\frac{m/s}{1/s} = 39m$$

Why is the wave length of elastic waves relevant? Because, when waves interact with physical objects, their effect is most pronounced when both are in the same size range. This is one of the reasons why underground excavations with sizes of tens to hundreds of metres are mostly affected by low frequency (read: large magnitude) events. The other reason is that these large events also generate waves with higher amplitudes, which carry more energy.

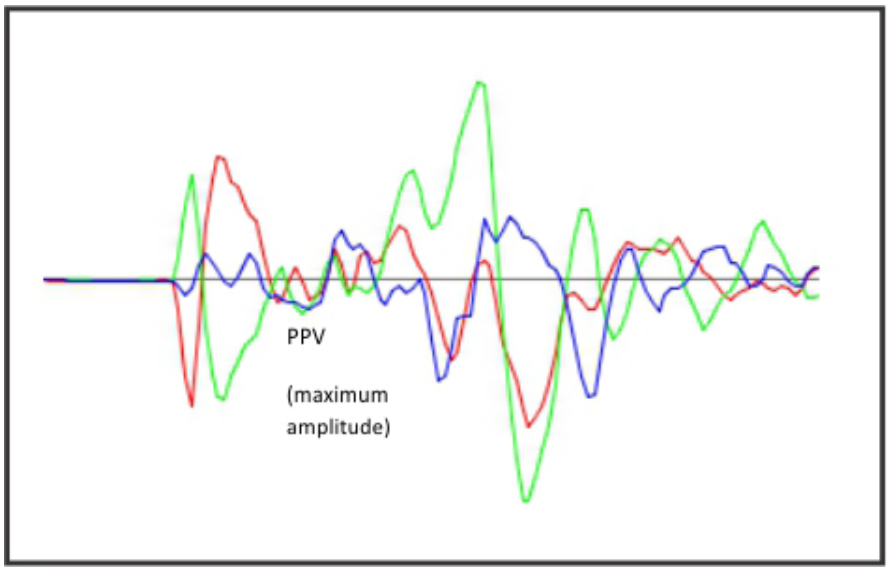
Waves and excavations

The above principles describe elastic waves being generated by seismic events and travelling through a homogenous or layered rock mass. The next step on the journey from source to stope is to investigate how waves interact with tunnels and other mining excavations.

Damage caused by seismic waves interacting with excavations is primarily the result of two mechanisms: Forced oscillation causes fracturing due to high stresses and high ground velocities lead to ejections despite existing support resistance leading to permanent deformation. Wave energy forces excavation surfaces to vibrate at the frequency of the incoming wave, which can result in rocks being dislodged from their position. The greater the wave energy, the greater the amplitude and the maximum velocity of the vibration. Within the context of seismic damage, an often used term is peak particle velocity:

DEFINITION

Peak Particle Velocity (PPV) = Maximum ground motion amplitude experienced by a particle subject to ground vibration.



Being a velocity, PPV is measured in m/s or mm/s. It can be estimated from so-called ground-motion relations that relate magnitude and distance to PPV, or it may be measured directly and quantified with the help of ground motion sensors that are installed on the surface of an excavation (Milev et al., 2002).

Permanent displacement may occur where the forced oscillations dislodge rock from its original position and support measures are inadequate to prevent the failure. The areal extent of damage and its severity is a function mainly of PPV, rock face integrity and the proper functioning of support. Inelastic closure is the result of large rock volumes being fractured and permanently separated from their position, resulting in a bulking effect.
As is evident from rockburst site investigations, destruction of underground service infrastructure is often localised and may vary within a few ten metres of a tunnel or stope from severe to essentially unaffected. This is understood to be partly due to localised geological conditions such as jointing and faulting, but may also be related to the complexity of the wave field interacting with the excavation surface. Wave fields that experienced reflection and refraction along their path are more complex than those travelling through largely undisturbed rocks. The original and the secondary fields can interact in a process called interference and lead to localised amplification or destruction (constructive or destructive interference).

In order to discuss rockbursts in more detail, the term should be defined properly:

DEFINITION

Rockburst = a seismic event that results in damage to excavations

Rockbursts are a sub-set of seismic events in that they include only those events that result in damage to excavations or injury to people. For seismic events to take place, usually some of the following have to be present: A moderate to high (shear) stress level, brittle rock types, and a weak shear zone with insufficient cohesion or pillars left too small. Once dynamic failure has occurred and the released elastic strain energy has resulted in damage, one can speak of a 'rockburst'.

To bring order into the different failure modes, Table 23 offers a classification of rockburst types in five categories and the magnitude range they occupy (Ortlepp, 2005). The table describes the source of energy release and the physical process characterising the failure, also known as the failure mechanism. Pillar foundation failure describes a failure

back

mechanism, as does shear rupture or buckling. Because of the obvious separation of source and damage – the same failure mechanism can lead to a range of different damage types – one needs to describe the damage mechanism separately and independently. Shake-down, crushing, footwall heave, etc. are examples of damage mechanisms. A detailed rockburst site investigation should cover both failure mode and damage mechanism to openings and the link between the two.

Rockburst type	Postulated source mechanism	First motion from seismic records	Local Magnitude M_L
Strain bursting	Superficial spalling with violent ejection of fragments	Usually undetected, could be implosive	-0.2 to 0
Buckling	Outwards expulsion of larger slabs pre-existing parallel to surface of opening	Probably implosive	0 to 1.5
Pillar or face crush	Sudden collapse of stope pillar or violent expulsion of large rock volume from tabular stope face or tunnel face	Possibly complex implosive or shear	1.0 to 2.5
Shear rupture	Violent propagation of shear fracture through intact rock mass	Double-couple shear	2.0 to 3.5
Fault slip	Sudden movement along existing fault	Double-couple shear	2.5 to 5.0

Table 23: Classification of rockburst types and their approximate magnitude range (Ortlepp, 2005).

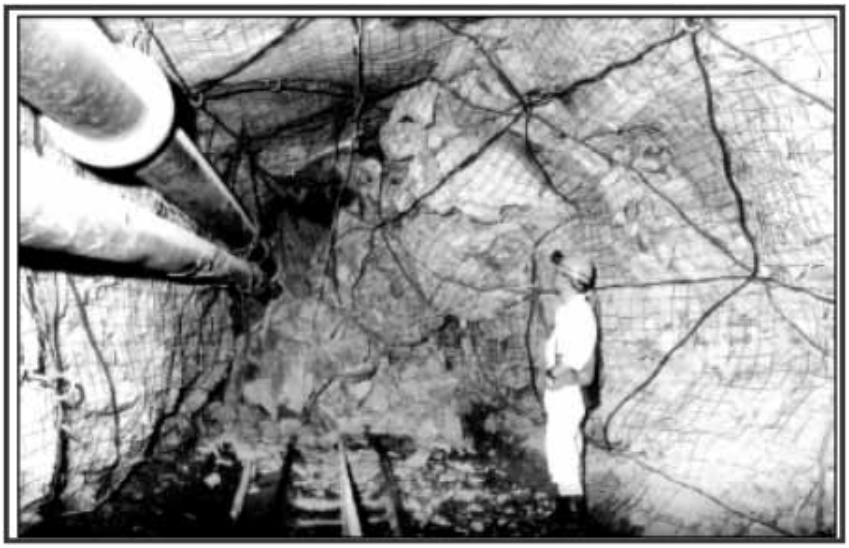


Photo: WD Ortlepp

REFERENCE

For details on the past 100 years of rockburst research in RSA see:

- Durrheim, R.J. 2010. Mitigating the risk of rockbursts in the deep hard rock mines of South African: 100 years of research. In Extracting the Science: a century of mining research, J. Brune (editor), Society for Mining, Metallurgy, and Exploration, Inc., ISBN 978-0-87335-322-9, pp.156-171.

Because of the range of factors that contribute to a rockburst, it is essential that a rockburst investigation covers the following issues, documents the findings and integrates the information into a coherent understanding of the incident. Where appropriate, rock mass modelling is recommended in cases where the results can shed light on the source mechanism:

back

back

- Seismic event details and source parameters of the event(s) considered responsible for the damage;
- Documentation of the seismic history of the area, including previous large events, rockbursts and short-term seismic trend analysis;
- Site visit to the affected area to record the extent, position and intensity of damage, geological detail that may explain the event source and its effects, support details (design and installation), evidence of high stresses and their effects and eye witness reports, where possible;
- Investigation of the behaviour of rock mass and support units and any geological detail that may explain the damage;
- Recovery of rock-samples and support elements from the site, where appropriate;
- Review of relevant rock engineering reports and mining strategy for the area.

The overall objective of a rockburst investigation is to establish the root causes of the seismic event, identify contributing factors (for instance historic mining in the area), and develop an understanding of the reasons for resulting damage in order to prevent recurrence in the future. Examples of recommendations from rockburst incident evaluations may include a different mining strategy, intensified support efforts or, in the worst case, abandonment of the area because the rockburst risk appears too high to be tolerated.

With reference to quantifying seismic hazard on mines, a possible approach could be to distinguish between 'seismically active' and 'rockburst prone' areas, i.e. experiencing seismicity as opposed to suffering damage from seismicity. An area that has experienced rockbursts in the past should receive a higher risk rating than one exposed to seismicity only.

Hazard quantification relies on a range of seismic indicators and analysis methodologies that will be discussed in detail in the materials for paper 3.1, together with the relationship between seismicity and mining strategy, depth, different rock types and other geology-related factors. For the requirements of paper two, knowledge of basic source parameters is required, which is presented in the following section.

REFERENCE

- Milev, A.M., Spottiswoode, S.M., Noble, B.R., Linzer, L.M., van Zyl, M., Daehnke, A. & Acheampong, E. (2002) The meaningful use of peak particle velocities at excavation surfaces for the optimisation of the rockburst criteria for tunnels and stopes, GAP 709 final report.
- Ortlepp, W.D. (2005) RaSiM comes of age – a review of the contribution, SAIMM RaSiM.
- Ortlepp, W.D. (1997) Rock fracture and rockbursts – an illustrative study, SAIMM Johannesburg.

LEARNING OUTCOMES

SEISMICITY PART 4

Hazard quantification relies on a range of seismic indicators and analysis methodologies that will be discussed in detail in the materials for Paper 3.1, together with the relationship between seismicity and mining strategy, depth, different rock types and other geology-related factors.

For the requirements of paper two, knowledge of basic source parameters is required, which is presented in the following section.

back

- Identify, describe and explain the common methods used to locate seismic events
- Describe the simplifying assumptions commonly used in determining seismic event location
- Explain the difference between the terms hypocentre and epicentre used in describing the location of seismic events
- Describe, explain and contrast the following primary source parameters in terms of their physical meaning and the data required to determine them:
- Seismic energy, Seismic moment, Richter magnitude, Local magnitude
- Source dimension, Corner frequency, Energy index

Concept Outcomes 23-26:
Describe different location algorithms and their underlying assumptions, point out limitations.
Define epi- and hypocentre and continue to other basic source parameters.
Define energy, moment, magnitude i.t.o. various input parameters, explain physical meaning of these.
Introduce basic physics of oscillation and wave propagation.
Explain which part of seismograms is used to calculate parameters in the time and the spectral domain.

REFERENCE

- Aki, K., Richards, P.G. (1980) Quantitative seismology. Theory and methods, WH Freeman Publishers, San Francisco.
- Brune, J.N. (1970) Tectonic stress and the spectra of seismic shear waves from earthquakes, J Geophys. Res. 75.
- Gibowicz, R.J., Kijko, A. (1994) An introduction to mining seismology, Academic Press, San Diego, Cal.
- Hanks, T.C. (1981) The corner frequency shift, earthquake source models, and Q. Bull. Seismo. Soc. Am., pp 597-612.
- Spottiswoode, S.M. (1993) Seismic attenuation in deep-level mines, 3rd Intl Symposium on Rockbursts and Seismicity in mines, Balkema, pp409-414.

Basic seismic source parameters

As a recipient of seismic services, a rock engineer is frequently in the position to relate seismic information to the mining environment, rock conditions or geological information. Usually after rockbursts or rockburst-related injuries, the practitioner is required to develop an overall understanding of the failure process and to evaluate key parameters that may be useful in developing a strategy to prevent future losses. A set of seismic parameters should be considered in such instances as they can contribute to a physical understanding of the process leading to the seismic event.

It may be instructive to reiterate the complexity of the task of quantifying a seismic event, which consists of a description of a remote, physical process of which no direct observations exist, simply on the basis of sound recordings of this process. The simplest requirements would consist of determining at least the accurate time, 3D-location and a measure of size of the event that would allow the recipient to identify the area on the mine that may have been affected and some initial estimate of the possible consequences. All of this detail must be gained from recordings of ground motion that were taken hundreds or thousands of metres away from this process by application of suitable analysis methods.

Apart from source time, the hypocentre location is therefore considered the single most important seismic parameter when it comes to seismic

back

event interpretation. Modern seismic networks usually have sufficient location accuracy – at least in plan – to allow an association between event and geological feature, pillar, abutment or other potential sources. When considering possible sources and provided that the damage report and underground observations do not clearly identify the source – one should consider magnitude and location together.

The following definitions of primary source parameters are provided:

DEFINITION

hypocentre = the point in space where the rupture took place (3D co-ordinates required)
epicentre = projection of the hypo-centre onto the surface (2D co-ordinates suffice)
magnitude = a measure of source strength.

Magnitude, especially, can be defined and calculated in a number of different ways, which results in a range of different interpretations of this parameter. As examples, consider Richter magnitude, moment magnitude, duration and body wave magnitude and others. Of central importance in mine seismology is the site-dependant 'local magnitude' ML introduced earlier and discussed in detail further below.

Locating a seismic source is a matter of triangulation using the arrival times recorded at a number of sensors and determining the point in space that matches the recorded times. The following simplifying assumptions (Figure 80(a)) are made for the standard location techniques:

- Waves travel on a straight path
- Waves travel with constant velocity,
- The seismic source is a point source.

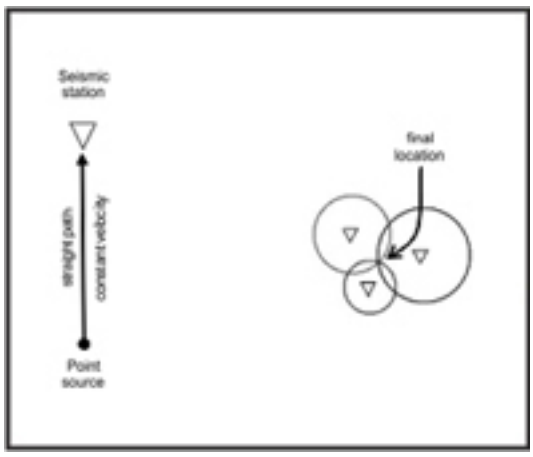


Figure 79: (a) Straight path assumption and (b) Location

Quite often, these conditions are not met in a natural rock mass consisting of layers of rock, embedded excavations and their fracture zones. Assuming that an event of, say, magnitude 2 originated from a single point in the rock mass is equally incorrect and the user of seismic information should always beware of the assumption when evaluating seismic information.

As far as the location algorithms are concerned, seismic systems on South African mines follow these steps as the standard location technique: Processing of seismic waveforms yields the time of arrival of seismic waves at each of the seismic stations (marked by Δ in Figure 79 (b)). One arrival time each is determined for the P-wave and for the S-wave. When picking the arrival time of the direct wave, the distance between source and station can be determined by using an estimated wave velocity. Therefore, each seismic station delivers a P- and an S-wave sphere

back

back

on which the source centre should have been located (in the 2D example marked by a circle). The location algorithm then determines that point where all spheres intersect and finds the likely solution by minimising the sum of residual errors between the single-station estimates and the compromise location.

Once the hypocentre has been determined, the source time (time of initiation) can be calculated as the final unknown in a set of equations. This is followed by the description of the source i.t.o. of strength, physical source size, moment, energy release and other aspects. The methodology is based on the physics of oscillations, considerations such as the properties and behaviour of swinging systems, elastic wave propagation and our understanding of rupture in solid rock or along pre-existing discontinuities.

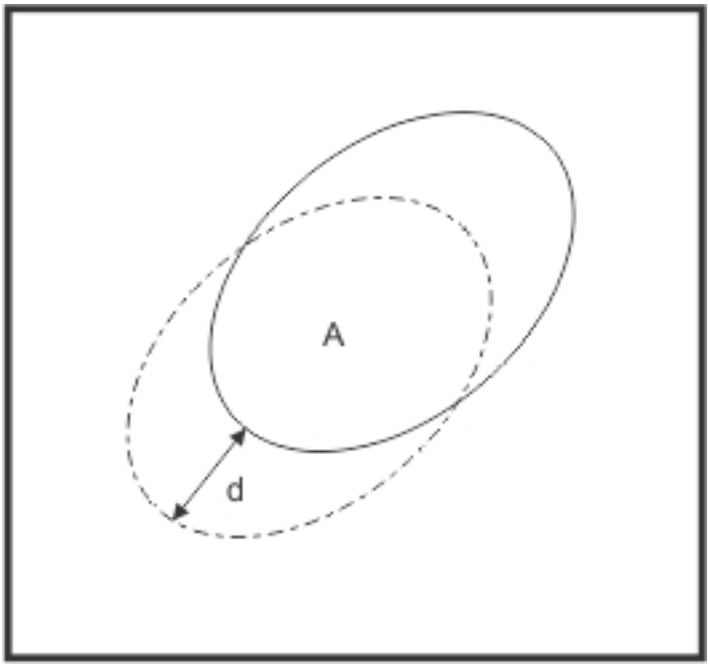


Figure 80: Slip distance d and slip area A (Brune model)

A simplified model proposed by Brune in 1970 describes the seismic source as flat and circular with an even slip distribution across the failure plane. To simplify the inversion, it is assumed that the source is seen from a large distance, effectively becoming a point source. The basis for seismic moment Mo calculation is the source area A, the average displacement across the slip area ū, and the rigidity G at the source (e.g. Aki & Richards, 1980). The unit of moment is Newton*metre [Nm):

$$Mo = \overline{GuA}$$

In most cases, the source area is inaccessible to take measurements and these dimensions are not known. In approximation, Mo can be estimated from seismograms (Gibowicz & Kijko, 1994). Relevant is the displacement spectrum of a wave form recorded in the far-field, usually obtained by integrating a geophone recording of ground velocity. Using the low-frequency plateau of the spectrum Ω0, rock density ρ, and wave velocity V, moment is calculated as:

$$Mo = 4\pi\rho\Omega_0 V^3 \frac{R}{F_1}$$

where V is the P- or S-wave velocity at the source respectively, R is the distance between sensor and source, and F1 a factor compensating for radiation pattern, free-surface amplification and the recorder site

back

back

response. Moment is calculated separately for S- and P-waves and subsequently averaged. Mo is a scalar entity and is different from the Moment Tensor that describes the three-dimensional strain distribution within the source area. The full 3D tensor of moments can be determined from the first arrival of P-waves, but requires good network coverage and quality seismograms for reliable results.

Scalar moment expresses source size and forces acting at the source. Physically large slip areas in soft rock masses have high moment values. Note that Mo depends on Ω0 which measures the displacement at the source due to the lowest frequencies contained in the signal. This value is taken as an approximation of the permanent deformation that remains after the dynamic failure process is complete. Mo of an event is calculated as the average between S- and P-wave moment.

BRIDGING GAP 5

Integration, spectrum under "Glossary" on page 3

Suitable for estimates of radiated energy is the velocity spectrum of P- and S-waves. By integrating over-all frequencies of the squared velocity spectrum, we sum up energy released throughout the entire frequency range. The resulting E_{v2} is independent of the source model and radiation patterns (Mendecki, 1997). Seismic energy E_n then becomes:

$$E_n = 4\pi\rho V E_{v2}$$

with density ρ.

The unit of energy is Joule [J]. Total elastic radiated energy E_n is not the average, but the sum of energies in the P- and the S-wave. Seismic energy can be understood much like kinetic energy, as it is related to the speed of failure. Seismic energy release is influenced by the state of stress and the rock mass strength or, in case of geological features, the clamping forces that had to be overcome to initiate slip.

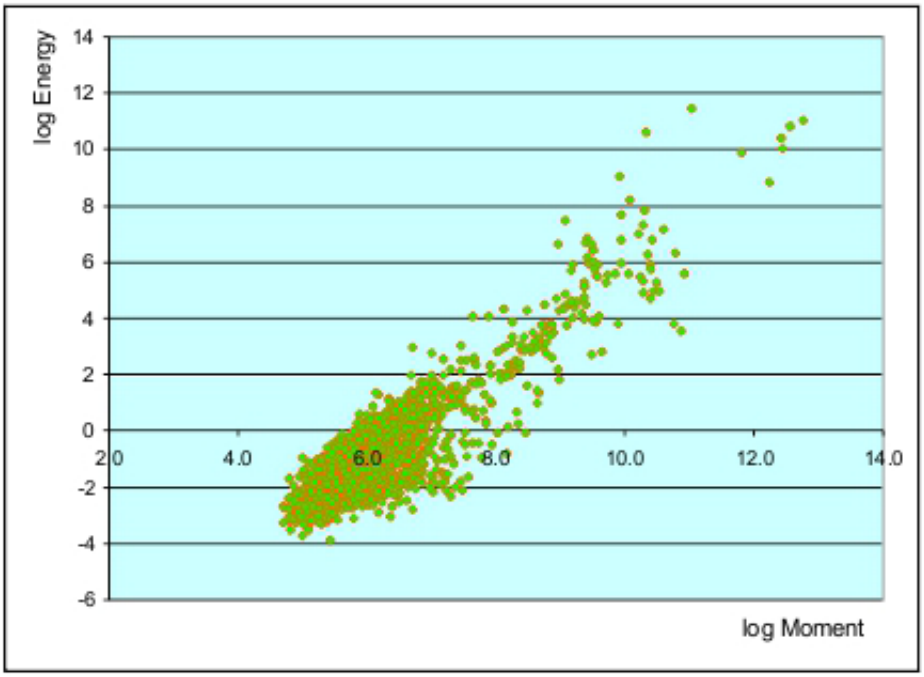


Figure 81: Quasi-linear relationship between energy and moment

BRIDGING GAP 6

Introduce ""Tensor" on page 17" to describe physical parameters with location & directional components: tensors)

On its own, E_n is not a suitable measure of stress levels, since it increases with source size, i.e. magnitude. Large events generally have larger moment and larger energy outputs, as Figure 82 illustrates. Plotting moment and energy of a group of events shows the roughly linear

back

back

relationship between these two parameters. Recording a high-energy event is not necessarily evidence of high stress or brittle rock types; it may simply be a large event. Put differently, E_n is an absolute measure of source energy, not a relative one. For the purpose of comparison, a relative measure is needed.

Designed for this purpose, Energy Index EI compares an event x with E_n^x to the typical energy released by events of the same size E_n^{avg}:

$$EI=E_n^x/E_n^{avg}$$

The average, expected energy is given by a linear regression based on an energy-moment plot (Figure 83). The relation events directly on the regression line have an Energy Index value of 1 (point A), those above >1 (B), those below <1 (C).

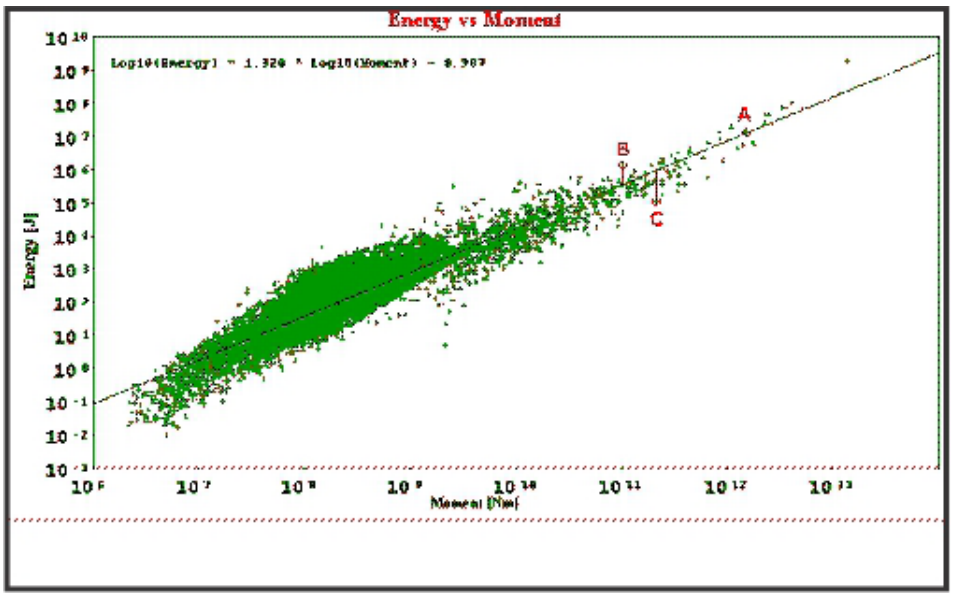


Figure 82: Linear regression relating energy and moment of recorded events

Events with EI>>1 are high stress drop events; those with EI<<1 are high deformation events, originating from a softer rock mass at lower stress levels. The unit of EI is 1, being the ratio of two energy values. Due to EI values covering a wide range of values, logarithmic value log(EI) is often reported rather than EI itself.

BRIDGING GAP 7

Refer to Linear regression in "Linear Functions" on page 11

As a measure of the stress drop at the source, Brune proposed the apparent stress to be calculated from seismic energy En and moment Mo. It is proportional to the ratio of energy over moment; its unit is Pascal [Pa]:

$$\sigma_a=G(En/Mo)$$

with shear modulus G

Apparent stress is not a measure of stress change, but rather a measure for the average between static shear stresses before and after failure. It can be expected not to vary too much in a limited mining area, as long as rock type and field stresses do not vary greatly.

A parameter of importance for seismic event interpretation is the source radius, an estimate of the physical dimensions of the source in metres (m). To calculate the source radius, Brune (1970) proposed the following formula based on corner frequency f_c and wave velocity V_S for shear waves:

back

back

BRIDGING GAP 8

"logarithmic function" on page 14

$$R_s = 2.34\frac{V_s}{2\pi f_c^s}$$

This accounts for the shear part of the seismic source and assumes a planar failure mechanism. In practice, where access to the seismic source is possible and a site investigation can produce an independent measure of the source radius, R is found to overestimate the real radius by a factor of 2-3. The source diameter D=2R is inversely proportional to the corner frequency, which confirms our general experience that large objects tend to move and vibrate slowly.

NOTE

The larger the source volume the lower are the wave frequencies emitted from it, the more energy they carry and the further they travel.
Local mine magnitude M_L, a general earthquake strength parameter, is based on both seismic energy and moment and is strictly speaking not a primary source parameter. M_L combines the two main aspects – deformation and energy – into one parameter that can provide an initial indication of the overall strength of an event:

$$M_L = A*\log(M_o)+B*\log(E_n)+C$$

Typical values are approximately 0.5 for A, 0.3 for B and -5 for C. By modifying the three coefficients, the local magnitude scale may be adjusted to match other magnitude scales, such as the one used by the Council for Geoscience to report large tremors. It is clear from the magnitude relation that events can have the same magnitude whether their respective energy is high and moment is low or vice versa. This reduces the usefulness of magnitude somewhat as it leaves doubts as to what contributed to the source strength: slip area or slip velocity. ML should be seen as a general measure of size of other parameters providing clarity on the details of source.

Because of the uncertainty regarding the role of energy and moment in the mixed magnitude relation, some networks are configured to use either a pure moment or pure energy-based magnitude. The trade-off is that only one aspect of the event is measured and expressed in each case, either physical size or energy release.

EXAMPLE 31

Determine the Magnitudes for two events with logMo/logE equal to 10.96/4.96 and 10.57/5.62

Magnitude calculation:
Calculate M_L for two events with different energy and moment release using the above magnitude relation. For A, B and C, choose settings implemented on some local mines: 0.516, 0.344 and -6.572 respectively, so that the magnitude relation takes the form:

$$M_L = 0.516*\log(M_o)+0.344*\log(E_n)+6.572$$

also known as the 'Stankiewicz magnitude'.

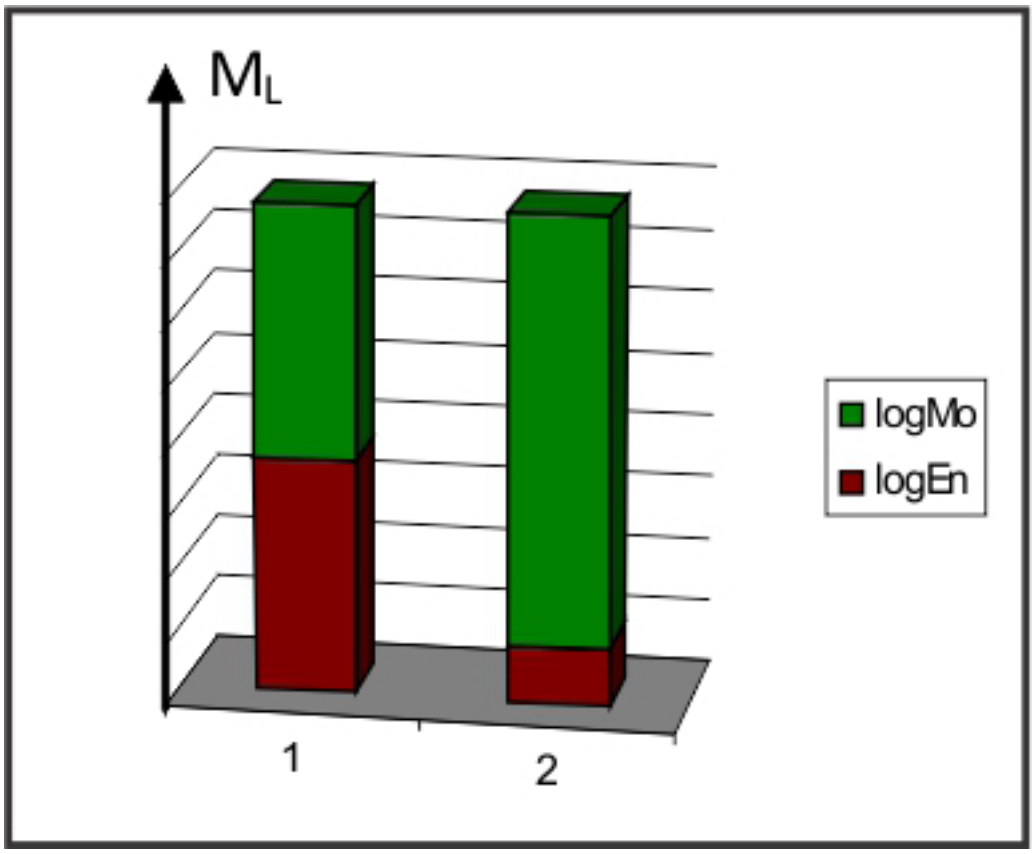
The following table provides the corresponding logarithmic values for E_n and M_o:

	Seismic Energy En	log (En)	Seismic Moment Mo	log(Mo)
Event 1	91.201kJ	4.96	9.12*1010Nm	10.96
Event 2	416.869kJ	5.62	3.715*1010Nm	10.57

back

back

Inserting the respective values yields M_L=0.78 for event 1 and M_L=0.81 for event 2. Within the accuracy limits of the methods used to determine En and M_o from seismograms, both events can be considered M_L=0.8 events.



Two events with equal magnitude but different E_n/M_o ratios

This is therefore a case of two events of equal magnitude, but with different characteristics, as event 1 released significantly less energy than 2, while event 2 had the smaller source size (corresponds to moment). One could wonder which one of the two events represents the higher hazard.

- Aki, K., Richards, P.G. (1980) Quantitative seismology. Theory and methods, WH Freeman Publishers, San Francisco.
- Brune, J.N. (1970) Tectonic stress and the spectra of seismic shear waves from earthquakes, J Geophys. Res. 75.
- Gibowicz, R.J., Kijko, A. (1994) An introduction to mining seismology, Academic Press, San Diego, Cal.
- Hanks, T.C. (1981) The corner frequency shift, earthquake source models, and Q. Bull. Seismo. Soc. Am., pp 597-612.
- Spottiswoode, S.M. (1993) Seismic attenuation in deep-level mines, 3rd Intl Symposium on Rockbursts and Seismicity in mines, Balkema, pp409-414.

back

CHAPTER

4

FUNDAMENTAL PRINCIPLES OF MINING LAYOUTS AND MINING LAYOUT DESIGN

INTRODUCTION

This chapter assumes that the rock engineer has knowledge and understanding of the environment in which he will design the mine and which was dealt with in the previous chapters. However, mine design for rock engineers is not something that stands separate from other disciplines on the mine. There is therefore a need to understand what requirements other disciplines may have of the mine to be designed so that the rock engineer can include those requirements into his final design.

This chapter will deal with:

- Survey and economics
- Mining layouts
- Ventilation of mining excavations
- Auditing for best practice

LEARNING OUTCOMES

4.1. SURVEY AND ECONOMICS
4.1.1 MINE PLANS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

4.1.1.1 Describe and explain the meanings of the following terms:

- Reef width,
- Internal waste,
- Channel width,
- Stope width,
- Mine call factor,
- Block factor

4.1.1.2 Interpret and apply information contained in survey plans

4.1.1.3 Construct sections from survey plans

4.1.1.4 Interpret and apply information contained in sections drawn from survey plans.

4.1.2 4.1.2 MINING ECONOMICS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

back

4.1.2.1 Describe and explain the reasons for attempting to mine an ore reserve at a

- constant grade
- Identify, describe and evaluate the implications of mining unpayable ore against leaving ore pillars in mining situations.

4.2. MINE DESIGN

4.2.1 MINING LAYOUTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

4.2.1.1 Describe and explain, for underground mining operations:

- How materials (support units, explosives, etc.) are transported into a section
- What services (electricity, water, etc.) are required in a section
- How services (electricity, water, etc.) are provided in a section
- How broken ore is removed from a stope.

4.2.2 SHAFT LAYOUTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

4.2.2.1 Describe and explain the layout associated with a vertical shaft, showing the method of handling the broken ore

4.2.2.2 Describe and explain the layout associated with an inclined shaft, showing the method of handling the broken ore

4.2.2.3 Describe and explain how vertical shafts are protected from the effects of mining-induced stresses and displacements

4.2.2.4 Describe and explain how inclined shafts are protected from the effects of mining-induced stresses and displacements

4.2.2.5 Describe and explain how the ore around a vertical shaft may be mined early to avoid the necessity of leaving a shaft pillar

4.2.2.6 Describe and explain methods of avoiding shaft pillars

4.2.2.7 Describe and explain the implications of the above methods on mine layouts

4.2.2.8 Describe, explain and design shaft layouts for the extraction of particular blocks of ground.

4.3. MINE VENTILATION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

4.3.1 Describe and explain the purpose of ventilating a mine

4.3.2 List, describe and explain the main sources of heat in a mine

4.3.3 List and describe the most common gases found in a mine and their associated hazards

4.3.4 Describe and explain in principle how ventilation is brought into and exhausted from a mine

4.3.5 Describe and explain how the flow of ventilation is controlled on the different mining levels

back

back

4.3.6 Describe and explain how the flow of ventilation is controlled in development ends

4.3.7 Describe and explain how the flow of ventilation is controlled in stopes

4.3.8 Describe and explain in principle how refrigeration is provided in mine workings using cooled air, chilled water and ice plants.

4.4. AUDITING FOR BEST PRACTICE

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

4.4.1 Describe and explain geotechnical investigation/design/monitoring/evaluation loop processes

4.4.2 Describe and explain process of geotechnical hazard and risk evaluation and assessment.

4.1 SURVEY AND ECONOMICS

4.1.1 MINE PLANS

LEARNING OUTCOME 4.1.1.1

DESCRIBE AND EXPLAIN THE MEANINGS OF THE FOLLOWING TERMS:

Reef width

Reef width is the thickness of the reef or ore body, usually measured perpendicular to the contacts of the reef / ore body (Figure 1). The 'reef width' therefore indicates the 'true width' of the mineralised zone, independent of whether the reef / ore body dips at any angle between 0 and 90°. However, it is a term most often used in tabular ore bodies where the reef has a relatively constant dip, shape and thickness.

Internal waste

Internal waste is the waste rock material between two or more reef bands (Figure 1).

It is sometimes necessary to mine the internal waste with the reef bands, resulting in the dilution of the reef grade, as waste tonnes are added to the total tonnes being processed while yielding a constant amount of the commodity. The total grade (measured in e.g. g/tonne) therefore reduces.

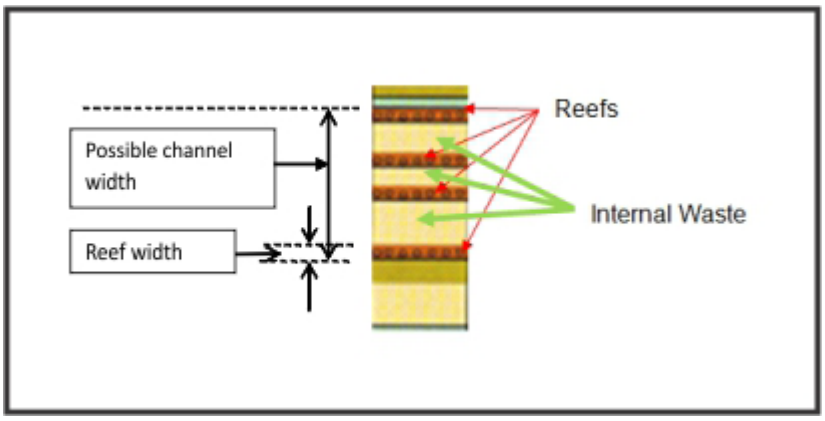


Figure 1: Reef width, internal waste and channel width

back

Channel width

Channel width normally refers to the width of the channel that is mined, which includes the internal waste (Figure 1). The mining area may be 1.5 meters high, but the channel width is only measured from the top contact of the top reef to the bottom contact of the bottom reef where they are mined simultaneously.

Stope width

Stope width is the actual, full width of the mining slot mined, measured from the hanging wall to the footwall of a stope (Figure 2).

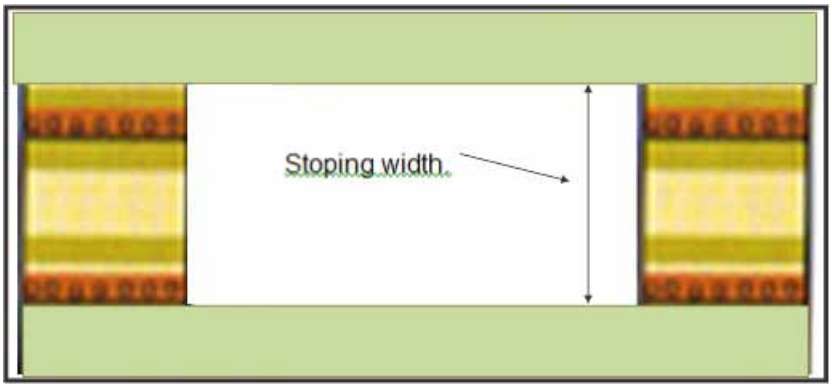


Figure 2: Stope width

Mine call factor

Mine call factor is a factor used on mines to determine the effectiveness of their mining process and is given as a percentage (%). It is a measure of the actual product recovered from the processing plant to the product contained in the ore body.

In a gold mine, the MCF would be given by:

MCF = grams gold recovered / grams gold available in ore body x 100/1

In many mines, a large amount of ore is not recovered due to poor ore cleaning of the stopes and wasting / losing of ore during loading and transporting phases.

NOTE

The gold available from the ore body is a function of the in-situ grade and mined tonnage planned while the gold recovered is a function of the mining and processing quality. A plant call factor (PCF) or recovery factor is also often applied, which measures the processing effectiveness by the ratio between actual gold recovered during processing and gold sent into the plant. To estimate the PCF, a measurement of the ore grade delivered to the plant is required and is often referred to as the 'head grade' as it was measured at the 'head of the shaft', i.e. on the surface.

Block factor

Ore reserves are usually divided into blocks, where each block is assigned a mineral value, as determined through the sampling of core and statistical projections. A Block Factor is a factor that represents the actual grades exposed by mining operations to the planned grade in each (or all) of the reserve blocks and is expressed as a percentage (%).

LEARNING OUTCOME 4.1.1.2

INTERPRET AND APPLY INFORMATION CONTAINED IN SURVEY PLANS

Grid lines and coordinates

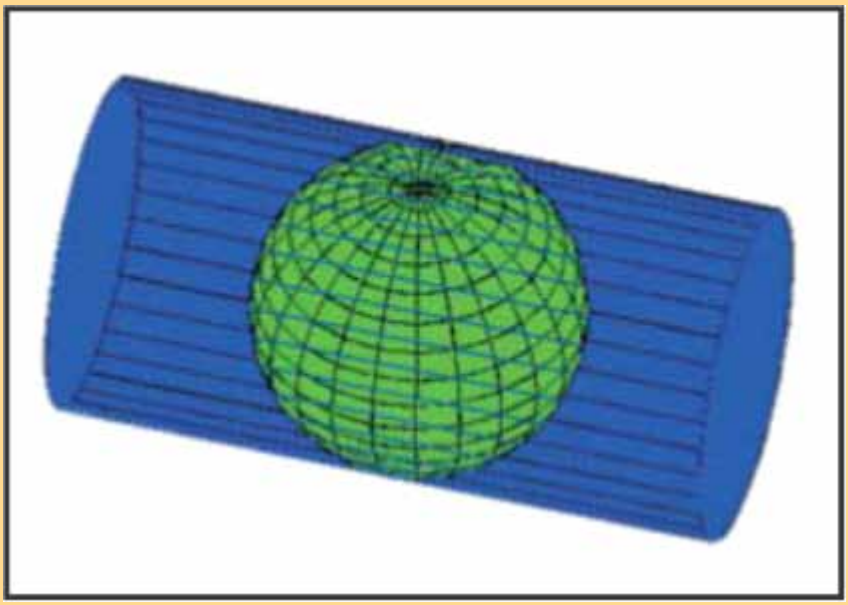
Grid lines are lines drawn across the mining plan that each represents a certain x- or y-coordinate. The x-coordinate lines are drawn parallel to the equator and the y-coordinate lines parallel to the 'line of longitude' in which the survey system is operating. The x- and y-grid lines intersect each other at 90°.

back

INTERESTING INFO

Mercator projected the Earth based on a cylindrical projection where a cylinder is put around the Earth and all information is projected onto the cylinder. In doing this, parallels and meridians are perpendicular to each other and run in straight lines, as seen below.

This projection was used to calculate x- and y-coordinates, known as the LO (local orientation) coordinate system. Distortion increases to the poles (north-south) and South Africa stretches in a north-south direction. Therefore, to minimise distortion in South Africa, we turn the cylinder 90°, resulting in a transverse projection. We call this the Transverse Mercator Projection, because we use Mercator's projection and turn (or transform) the cylinder through 90°.

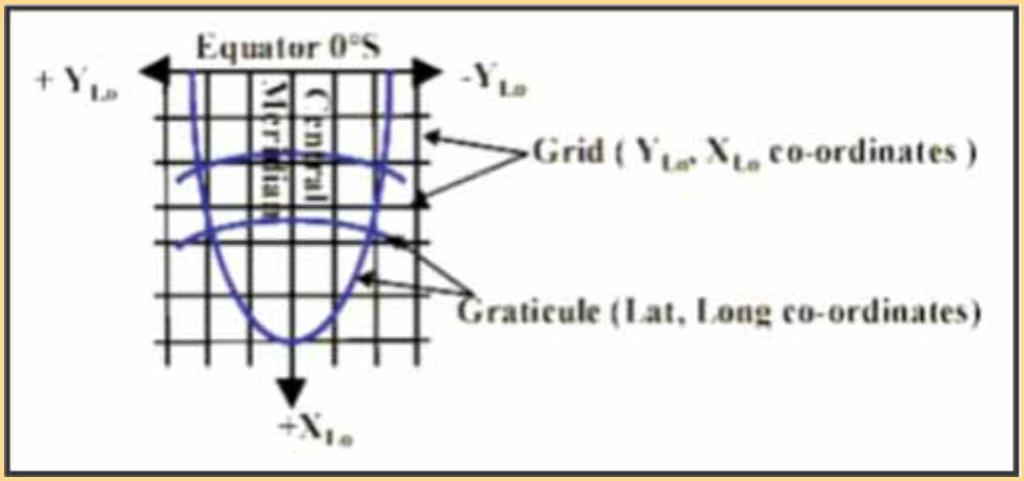


In South Africa, we use the LO coordinate system quite extensively. Only the area within one degree of longitude on either side of the central meridian is projected. The width of each segment, often referred to as a belt, is therefore two degrees of longitude and is referred to as the central meridian (CM) of that belt. Each zone is named after the longitude of origin, i.e. LO 17°, LO 19°, LO 21° etc.

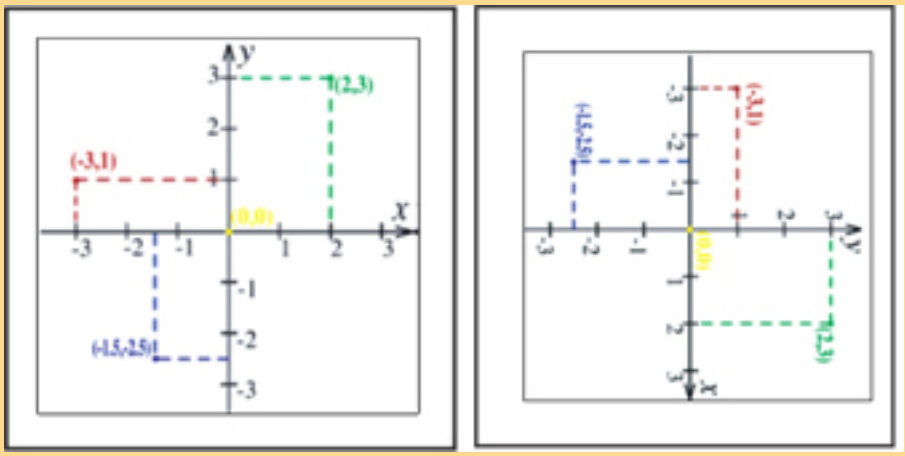
back

LOs are always uneven, because they are in bands that are 1° east and 1° west, starting from 0°. This means that the central line will be 1°, resulting in LOs being uneven.

In the figure below, you can see the difference between the geographic coordinate system and the Mercator projection. You will see that the co-ordinates for the geographic coordinate system (blue) are based on the shape of the Earth itself, while the Mercator projection (black) shows the latitudes and longitudes as straight, perpendicular lines.



So, where do x-coordinates and y-coordinates fit in? These are referred to as Cartesian coordinates and measure the distance from the centre of the Earth. Remember, up until now, latitudes and longitudes were lines measured from the equator and the central meridian, meaning that they were measured from lines of zero on the surface of the Earth. In the case of Cartesian coordinates, they are measured from the actual centre of the Earth, the midpoint of the Earth. Therefore, they also include a z-value, which is height or depth. For this same reason, Cartesian coordinates are always portrayed in metres, as measured from the midpoint of the Earth. Measurement of the x- and y-coordinates is based on the mathematical calculation of x- and y-values, as portrayed below (left-hand side).



Remember when we discussed projections, we mentioned that in South Africa, we turn the projection 90°? When we work in Cartesian coordinates, we do the same, because
we still have to project, resulting in the system on the right-hand side, where South Africa will now have a positive x- and a positive or negative y-value, depending on which longitude was used as the central meridian.

back

If we work with the LO-system, we will, for instance, use longitude 27 east as our tangent line (central meridian), meaning that east of LO27 our y-coordinate will be negative, and west of LO27 our y-coordinate will be positive.

Two LO-coordinate systems used on the mines are the LO27 and LO29 system, where the origin falls on the 27 or 29 lines.

In our mines, the x-lines are always positive, but the y-lines can be either positive or negative, depending on whether you are east or west of the origin (Figure 3).

Regulation 12.5.2.10 requires coordinate lines of a prescribed plan to be lines not more than 200mm apart, with numerical values indicated on both ends of the line on the plan.
Each survey peg or position underground can be related to the coordinate system with an x-coordinate and a y-coordinate. Elevations are reflected as z-coordinates.

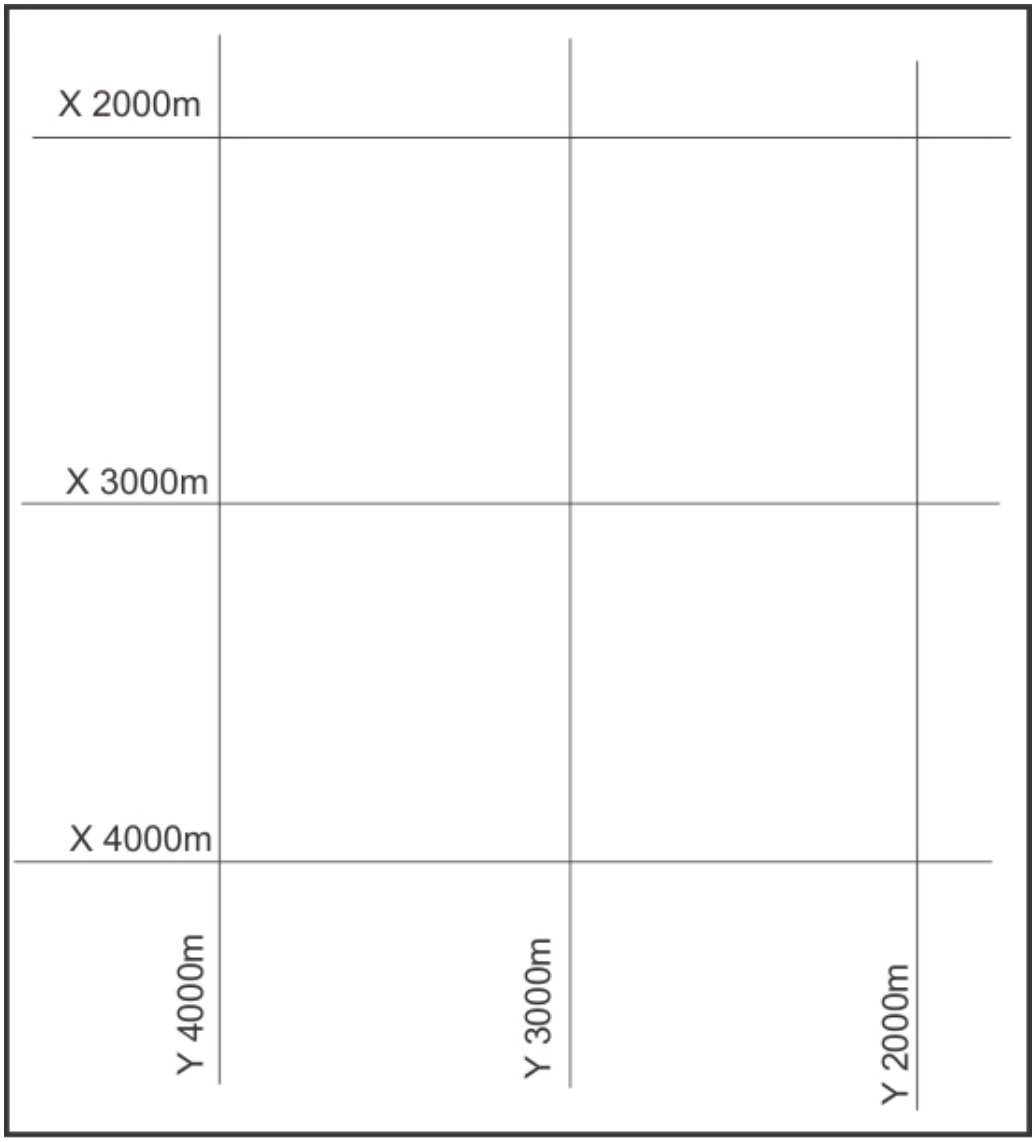


Figure 3: Example of coordinate lines

Because x is the distance as measured from the equator, it will always be a large number. So if you receive any data, and there is no indication of which one is the x- and which one the y-coordinate, you can safely assume that the largest coordinate is your x-coordinate.

back

back

In some cases, mapmakers make use of a constant, which is the first part of
the x-coordinate, repeated in all the coordinates, because the x-coordinate becomes too long to use easily. The constant is indicated on the map, as a constant, and left out of the actual written x-coordinate, e.g.:

- Actual coordinates are x1 = 2 789 456 and x2 = 2 789 573,
- Constant is 2 789 and written coordinates are x1 = 456 and x2 = 573

Scale

The scale of a plan must always be indicated on the plan. The use of the correct scale, as indicated on Figure 4 is critical when performing any measurements.

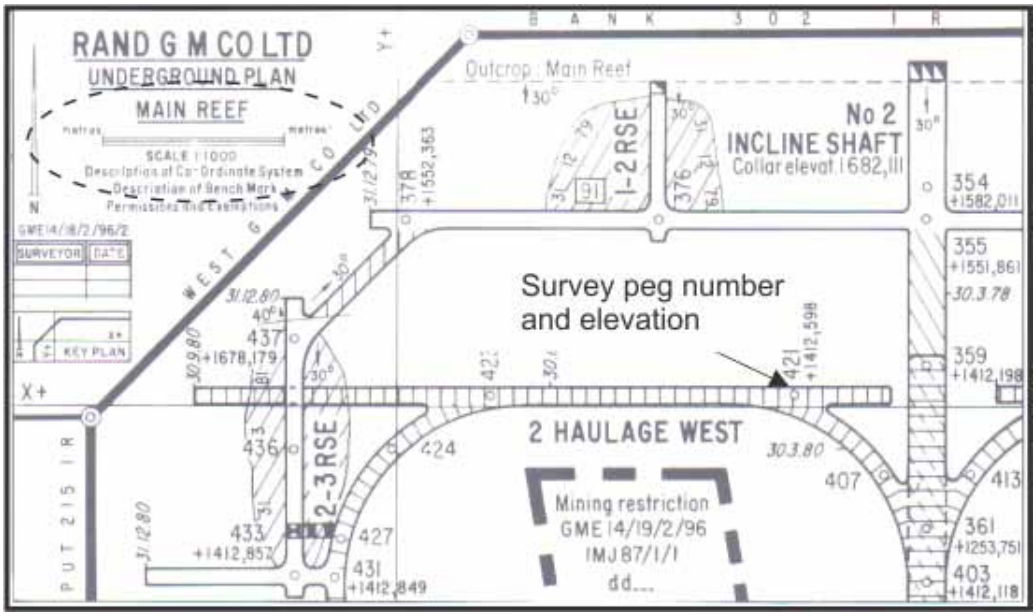


Figure 4: Plan drawn on a 1: 1000scale

Scale rulers are available with standard scales that may include 1:200, 1:500, 1:1000 or 1:2500, and should be used on the appropriately scaled plans. If only a normal ruler (mm) is available Table 1 shows transformations that can be made to measurements:

Actual distance to be scaled	Distance (mm) measured with a normal ruler when plan scale is:			
	1:200	1:500	1:1000	1:2500
1m	5mm	2mm	1mm	0.4mm

Table 1: Conversions due to different scales

Datum

A datum is an imaginary horizontal line from where depths (z-coordinates) are measured, i.e. the z-coordinate of a datum line is zero. This line can be 'at sea level' or a 'local datum' line.

When 'sea level' is used as the datum for mining operations, most of the mines in South Africa will show positive z-coordinates, where the coordinates increase upwards, i.e. shallow levels in an underground mine will have larger z-coordinates than deeper levels. This is because deeper levels will be closer to the 'sea level'.

When a local Datum line is used and this line is, for example, 1800m above sea level, which is also 200m above surface in this particular area,

back

coordinates will be measured from this line downward to the underground levels, and will be reported as -500m or 500m below Datum for an excavation situated 300m below surface. This is illustrated in Figure 5.

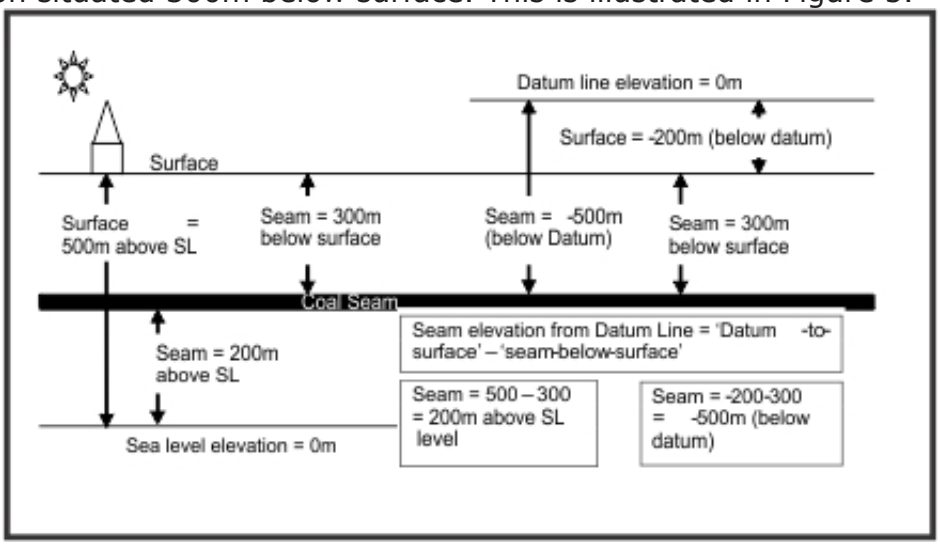


Figure 5: Section showing the use of a datum line

Legend for existing information

The legend on the plan must clearly indicate what information has been recorded on the plan.

Since the data indicated in plans can include many issues, it is not covered in more detail. However, the user of a plan should interrogate the creator of the plans until he has been assured of the meaning of all measurements and plots on the plans. These can include, but should not be limited to:

- *Survey: Coordinates, grids, scale, etc;*
- *Mining: Mined out areas, unmined areas, marking of 'footwall' and 'hangingwall' tunnels, shafts, planned mining operations, etc;*
- *Geology: faults, dykes, potholes, iron replacements, jointing, reef dip, etc;*
- *Geotechnical: Ground Control Districts, seismic events, support installations in tunnels, etc.*

Outlines of excavations relevant to the purpose of the plan

The outlines of excavations relevant to the purpose of the plan or section must be shown so that precise positions, middlings, etc can be interrogated by measurement. Since rock engineering is normally concerned with the interaction of stress levels with excavations, the size, shape and location of excavations should be accurately known, as shown in Figure 6. Failure to accurately indicate excavation outlines will result in uncertainty and the risk of experiencing instability.

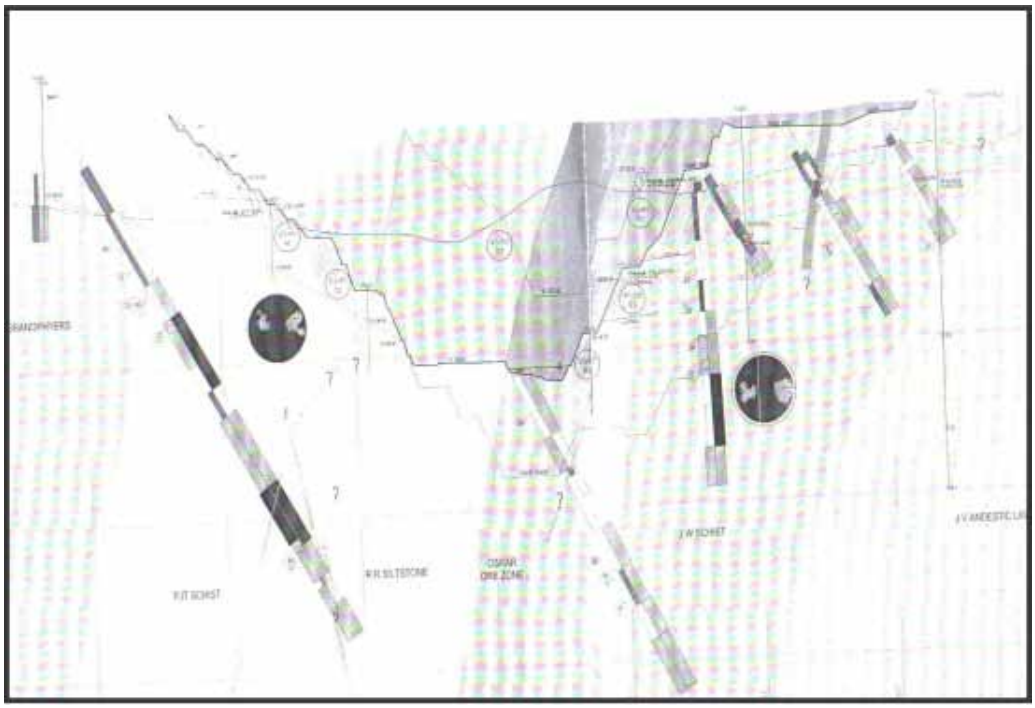


Figure 6: Geotechnical section showing the outline of an open pit

Position and reference numbers of survey pegs

Survey pegs are used to manage underground excavations by ensuring that each peg's position is well known, accurately determined and recorded in the x-, y- and z-orientations.

These coordinates can be confirmed by performing 'check' surveys and it is a good practice to confirm peg positions, and therefore excavation positions, especially when 'old' mining areas are entered again. An example of this is shown on Figure 7.

By referencing peg numbers, it is ensured that ambiguity is prevented by not having pegs of similar numbers situated at different locations in the mine.

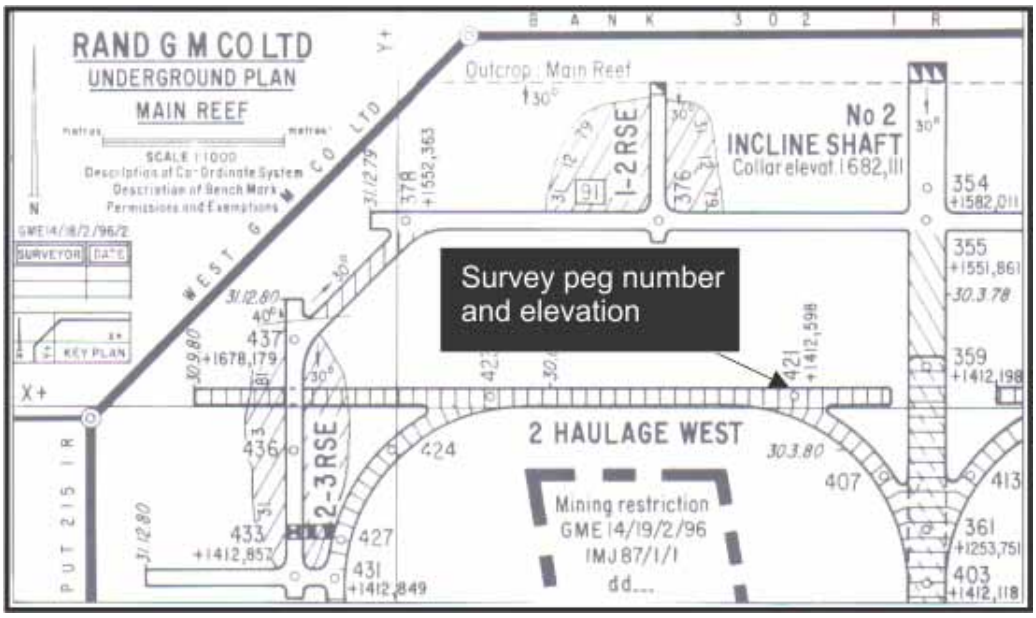


Figure 7: Plan showing a survey peg number and its position.

Contours of ore body elevations

Contours are imaginary lines where all points along a specific line are at the same elevation. In underground plans, reef contours are marked, which connect points where the reef elevations are the same and so indicate the 'strike' direction of the reef (Figure 8). If the contour elevations remain constant, lines spaced closer indicate steeper reef dips, while lines further apart indicate shallower dipping reefs.

NOTE

Reef contours are utilised when sections are drawn (Outcomes 2.1.3.1 and 2.1.3.2)

Slopes can also be plotted using contour lines very similar to those used for presenting the topography (surface).

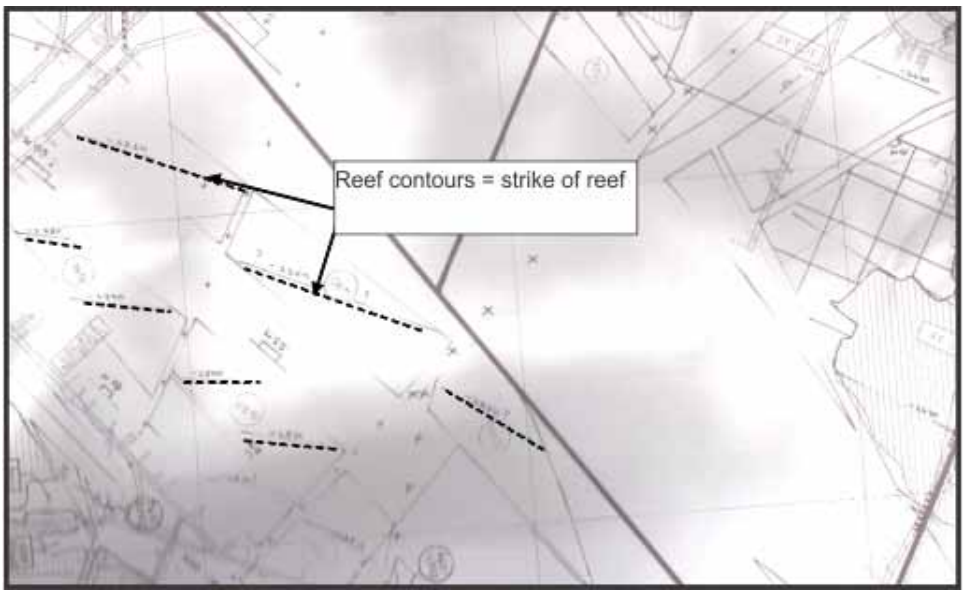


Figure 8: Sketch illustrating underground reef contours

Annotation of true or magnetic north direction

All mine plans must have a north direction indicated on it in order to assist in the orientation of the plan (Figure 9). This is also essential for orientation when section views are to be drawn or evaluated for specific layouts.

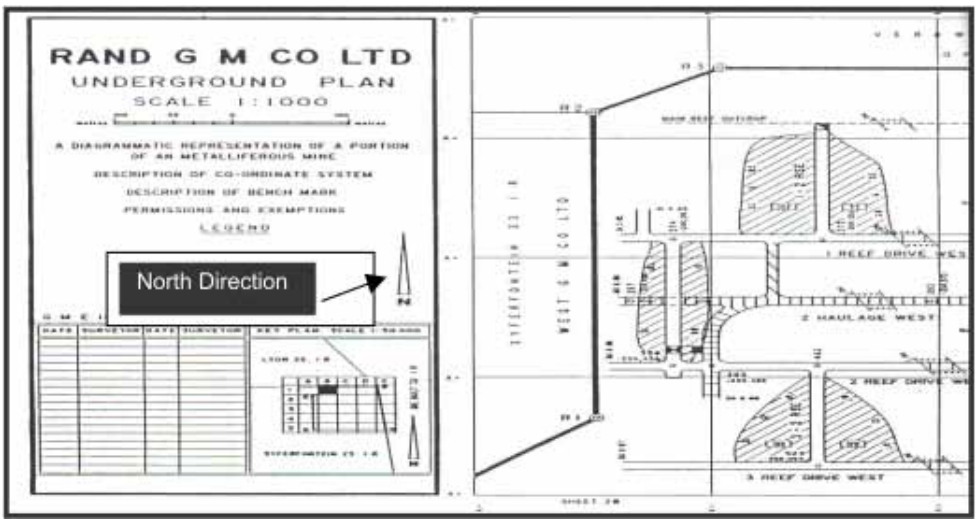


Figure 9: Plan indicating northern direction

Nomenclature

Nomenclature is a system of names used in a particular subject, such as abbreviations to indicate stratigraphy, as shown in Table 2.

Bird reef	Bd R	Crown lava	Cr lava
Black reef	B R	Erosion channel	E Ch
Kimberley reef	K R	Hangingwall marker	H W Mkr
Cobble reef	Cbl R	Jeppestown shale	J Sh
Dummy reef	D R	Hangingwall of green bar	H W G B
Lower Elsburg	LE	Booysens shale	B Sh

Table 2: Example of a list of abbreviations for stratigraphy

Symbols

The symbols illustrated below (Figure 10) are in common use on most of the mines and may not include site-specific symbols.

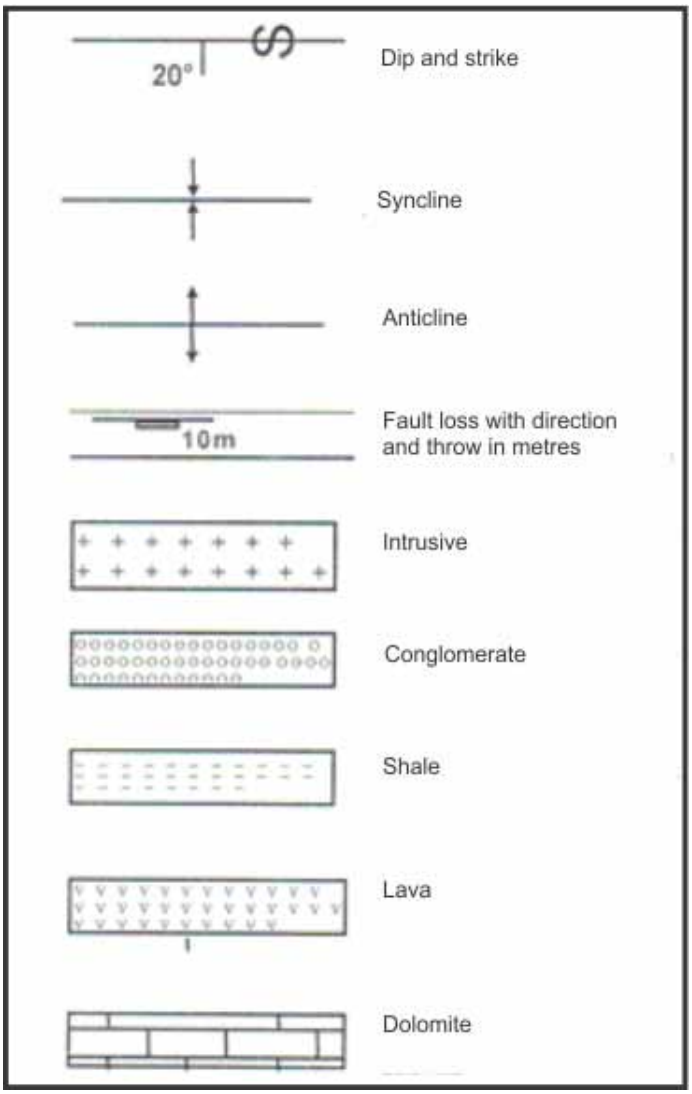


Figure 10: Example of some symbols used on geotechnical plans

REFERENCE

- Page 157, Page 11, Van der Merwe J.N, Madden B.J., Rock engineering for coal mining, 2002.

LEARNING OUTCOME 4.1.1.3

CONNECTION 17

See paper 2 "LEARNING OUTCOME 2.3.1" on page 60 and onwards

CONSTRUCT SECTIONS FROM SURVEY PLANS

LEARNING OUTCOME 4.1.1.4

Interpret and apply information contained in sections drawn from survey plans.

Sections provide an additional tool to the rock engineer to evaluate geological settings or mining layouts and should be used whenever layouts are interrogated. The ability to see in various dimensions when looking at two-dimensional plan views is often not well developed and the drafting of sections to assist in the evaluation is strongly suggested.
Sections drawn from mine plans can provide additional information or assist with analyses. Examples of information / analyses possible are shown in Table 3, but it must be realised that this list is not complete and should be used to guide and not limit the use of sections.

Information provided	Analyses possible (examples)
Reef and excavation elevations	Impact (stress) of mining on footwall tunnels Over-stoping distances required Layouts for access to reef Reef intersections in footwall tunnels High stress abutment positions Middling between mining and footwall tunnels
Reef dip (tabular), ore body thickness (massive)	Layouts for access to reef Reef intersections in footwall tunnels Variations in ore body dimensions (massive) Area for possible mining method alterations
Geological structure positions and displacement on geological structures	Seismic risk (faults) Impact on hangingwall stability if structure dips over stope Intersections with excavations
Thickness / losses caused by structures	High stress abutments Access tunnel stability in 'loss' area Seismic risk (dykes)
Middling to surface	Middling stability Possible impact on surface structures

Table 3: Assistance from sections

4.1.2 MINING ECONOMICS

LEARNING OUTCOME 4.1.2.1

DESCRIBE AND EXPLAIN THE REASONS FOR ATTEMPTING TO MINE AN ORE RESERVE AT A CONSTANT GRADE

Ore bodies situated within South Africa are national assets and mining of these assets must be managed in such a way that the asset is optimally exploited. Optimal exploitation is a balance between maximising annual profits and extending acceptable profit margins for as long as possible into the future, as well as extending the life of the mine. It is in finding this balance that the optimum solutions for each mine / mining group lie.

back

It must be accepted as a statistical truth that all ore bodies have variations in product grade across the mining lease area. This variation in grade must be managed to ensure that the maximum benefit can be obtained. Usually, extensive feasibility studies are conducted where income and cost relationships are iterated until the maximum benefit for profit margins and 'life of mine' is found. Once the optimum grade for the operation is identified, the key to success lies in the management of mining in individual mining areas at different grades so that the designed grade is provided to the plant.

If this is not managed well and mining is driven by maximum short-term profit only, high grade areas will be mined out first and only lower grade areas will remain after a period of time.

Only lower grade product will mean that the expected income will reduce unless there is a sharp increase in tonnages produced by the same personnel. Other solutions to this problem include the reduction in employee numbers, changes in mining methods, limited use of resources and equipment – all in an attempt to reduce costs to increase profits for the investor.

NOTE

Changes in the price of the product may affect the optimum mining grades and must also be included in the design and management of the optimum grade.

LEARNING OUTCOME 4.1.2.2

IDENTIFY, DESCRIBE AND EVALUATE THE IMPLICATIONS OF MINING UN-PAYABLE ORE AGAINST LEAVING ORE PILLARS IN MINING SITUATIONS

A few possible scenarios exist in looking at the mining of un-payable ore or waste material:

- The mining of waste is essential to ensure continued production of higher grade ore or to ensure the safety of personnel:
  - In some mining situations, over-/under-mining of critical excavations is necessary to protect the remaining excavations against high stress levels and stress / seismic damage. In such cases, it is not always necessary to dilute the mining grade by adding the waste to the reef, but rather to tram and hoist it separately or fill mined-out areas with waste. When waste mining has to be carried out, it can be done on the smallest possible stoping width, while not all the broken rock needs to be removed from the mining slot, reducing the tonnages generated by this practice.
- Mining of un-payable areas in an unplanned / uncontrolled manner:
  - This practice is a concern as un-payable means that the mining operation itself is working at a loss. Since mines are investment vehicles, mining at a loss is not easily acceptable.
- Leaving of unpaid areas is planned:
  - However, if for some reason mining personnel is not allocated to a payable mining area and cannot be released from employment, it is possible to limit the overall loss by planning to mine unpayable areas, as some income will be generated, while having employees sitting on surface remains a dead loss. However, this evaluation is not a simple process and all possible impacts should be evaluated.

back

- If a mining layout requires in-situ pillars as part of the mine design, it is logical to attempt to leave the pillars in unpaid areas. This includes regional pillars and bracket pillars, but may not be appropriate to in-stope support pillars. The main reason is that in-stop support pillars must usually be left at specific dimensions and positions to ensure stability and can therefore not be changed or moved at will. Regional pillars, however, are left intact to limit deformations, to reduce stress levels, to compartmentalise etc and its exact position is not always required. There is therefore potential to move regional pillars, as long as the design requirements such as maximum spans or dimensions are maintained, to include unpaid areas.
- If a mining method can be adapted to a pillar-based mining method instead of using artificial support systems without affecting stability or safety, while improving profit margins, it may also be a good solution to leave lower grade pillars rather than placing high cost support such as backfill or cement packs.

4.2 MINE DESIGN
4.2.1 MINING LAYOUTS

LEARNING OUTCOME 4.2.1.1

DESCRIBE AND EXPLAIN FOR UNDERGROUND MINING OPERATIONS:

How materials (support units, explosives, etc.) are transported into a section

In most mining methods, material, support units and explosives are lowered down the shaft (vertical or inclined) with suitable material cars, then removed from the shaft at the designated level and horizontally transported along designated tramming ways, by means of rail-bound locomotives or trackless vehicles to the section where it will be used. In completely trackless mines, vehicles will transport materials from the surface, down the decline systems to the section where it is required.

There are some legal considerations as discussed below, when transporting explosives:

- Transporting of prepared charges/explosives
  - Regulation 9.20.2 states that any person may in the underground workings of any mine convey explosives in an unopened case or container.
  - Regulation 9.20.3 states no person shall convey or cause or permit to be conveyed, any explosives other than safety fuse or lighting torches unless the explosives are contained in an unopened case or container, or in a receptacle of a design and construction approved in writing for the purpose by the Regional Director.
  - Regulation 9.20.5 states that blasting cartridges, ammonium nitrate blasting agent and detonating fuse may be conveyed together but shall not be conveyed in the same conveyance or receptacle with any other explosives.
  - Regulation 9.20.6 states that detonators, safety fuse, capped fuse, igniter cord, electrical initiators, fuse igniters and lighting torches may be conveyed together but shall not be conveyed in the same conveyance or receptacle with any other explosives.

Figure 11 below shows a typical horizontal tunnel / footwall drift / haulage used in the gold mining industry to transport material using rail-bound electric, battery or diesel locomotives pulling material cars (Figure 12 and Figure 13).

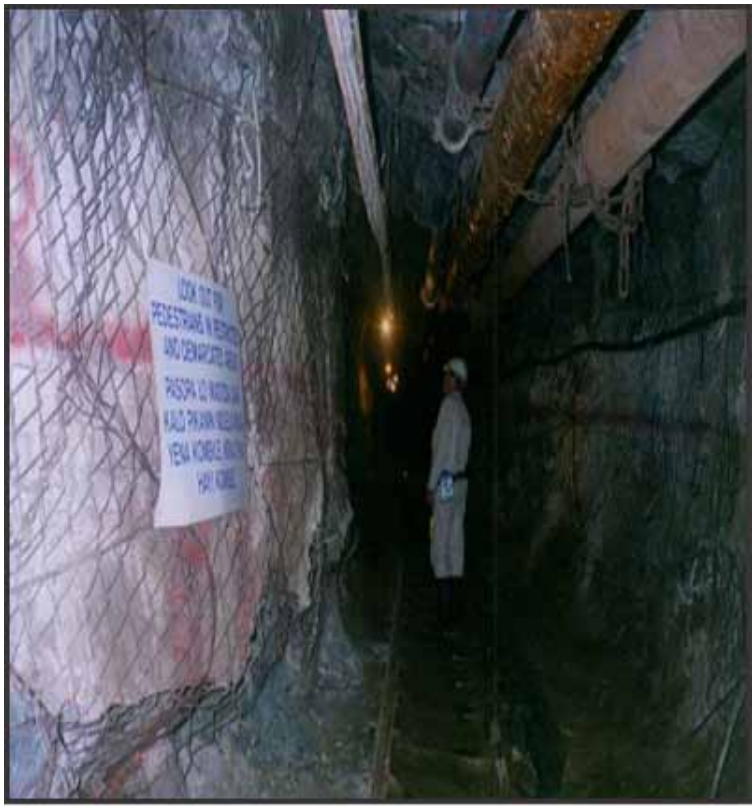


Figure 11: A typical haulage / horizontal footwall drift where rail bound equipment transport material

Figure 12: Battery operated locomotive used in tramming of material into sections

Figure 13: Overhead electric locomotive used in tramming of material into sections

What services (electricity, water, etc.) are required in a section?

The more important services required for underground mining operations include:

- electricity to drive winches, provide lighting, operate water pumps and ventilation fans (or drilling equipment in some mines);
- water to allow drilling, cleaning of rock surfaces, cooling (spray chambers) (or hydropower for drilling in some mines);
- clean water for drinking purposes;
- compressed air for drilling purposes, protection against smoke infiltration in refuge chambers;
- ventilation (Outcome 4.3 onwards).

How services (electricity, water, etc.) are provided in a section?

In a typical underground mining operation, all the services come down the vertical or inclined shafts and out on the various working levels, along haulages (horizontal drifts) and crosscuts towards the mining face. Different mines use different services, but some typical layouts are shown in the figures below.

- Compressed air: In Table 4, an example of a mine's compressed air reticulation is shown.

Columns diameter installed in the DIP MATERIAL WAY	150mm	6 inches
Columns diameter installed in the STRIKE MATERIAL WAY	100mm	4 inches
Strike material way column distance to the face	not greater than 15m	
Manifold distance from Strike Material Column	not greater than 15m	
Manifold distance from face	not greater than 30m	
'Boesman' Hose length	not greater than 15m	
25mm Mining hose length	not greater than 30m	
Number of Rockdrills operating from one manifold	not greater than 4	

Table 4: Compressed air reticulation standard

Compressed air and water columns are rooted from the surface compressor room and surface dams or rand water board connections, down the shaft in columns suspended on brackets attached to the shaft walls and

back

out on the various levels along horizontal tunnels towards the stoping sections.

The sizes of the columns will depend on the requirements of the various levels. See Table 5 below indicating an example from a mine's standard on dimensions of water and air columns used on the mine.

Excavation	Height (m)	Width (m)	Air pipe diameter (mm)	Water pipe diameter (mm)
Haulage	3.7	3.0	200	150
Crosscut	3.5	3.0	150	100
Travelling way towards reef	2.0	2.4	150	100
Raise on reef	2.7	1.5	50	100
Box hole / Ore pass	1.2	2.0	50	50

Table 5: Example of dimensions and grade of water columns used in various sections on a mine

Along the levels, water and air columns are suspended along the tunnel hangingwall and when air flow (ventilation) is controlled in the tunnel by installing ventilation doors, the service pipes have to go through the ventilation door construction (Figure 14).

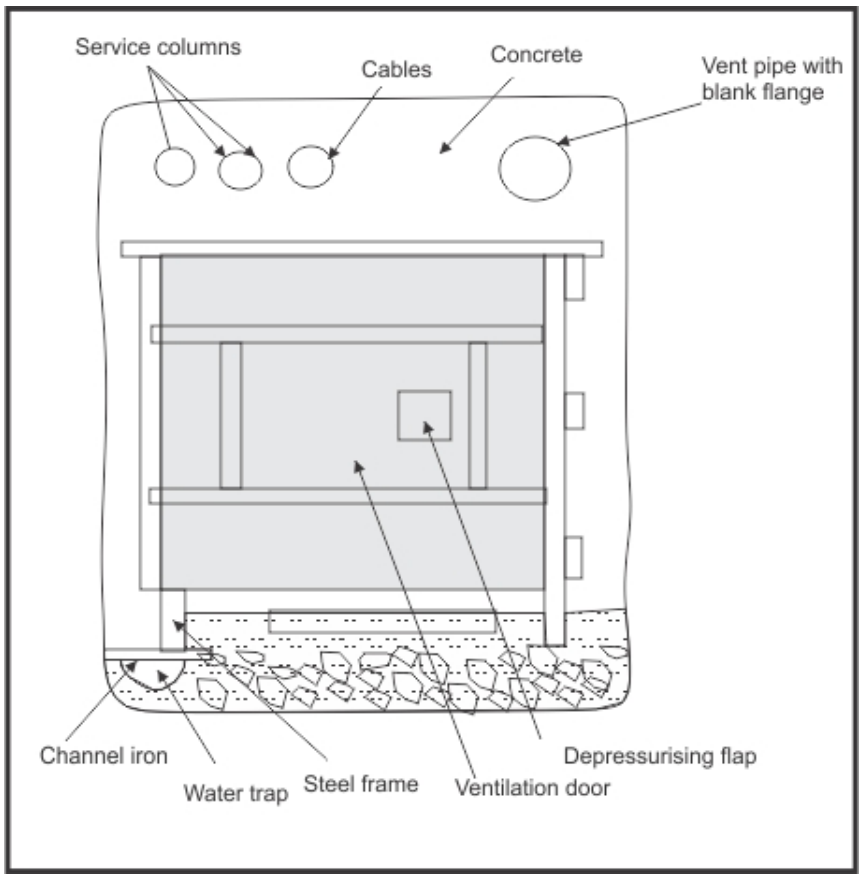


Figure 14: Example of a ventilation door installation showing the position of the service pipes (section view)

In the cases of leakages from the pipes, columns need to be closed at various positions to isolate the columns and, for this reason, valves are installed at designated positions (Figure 15).

back

back

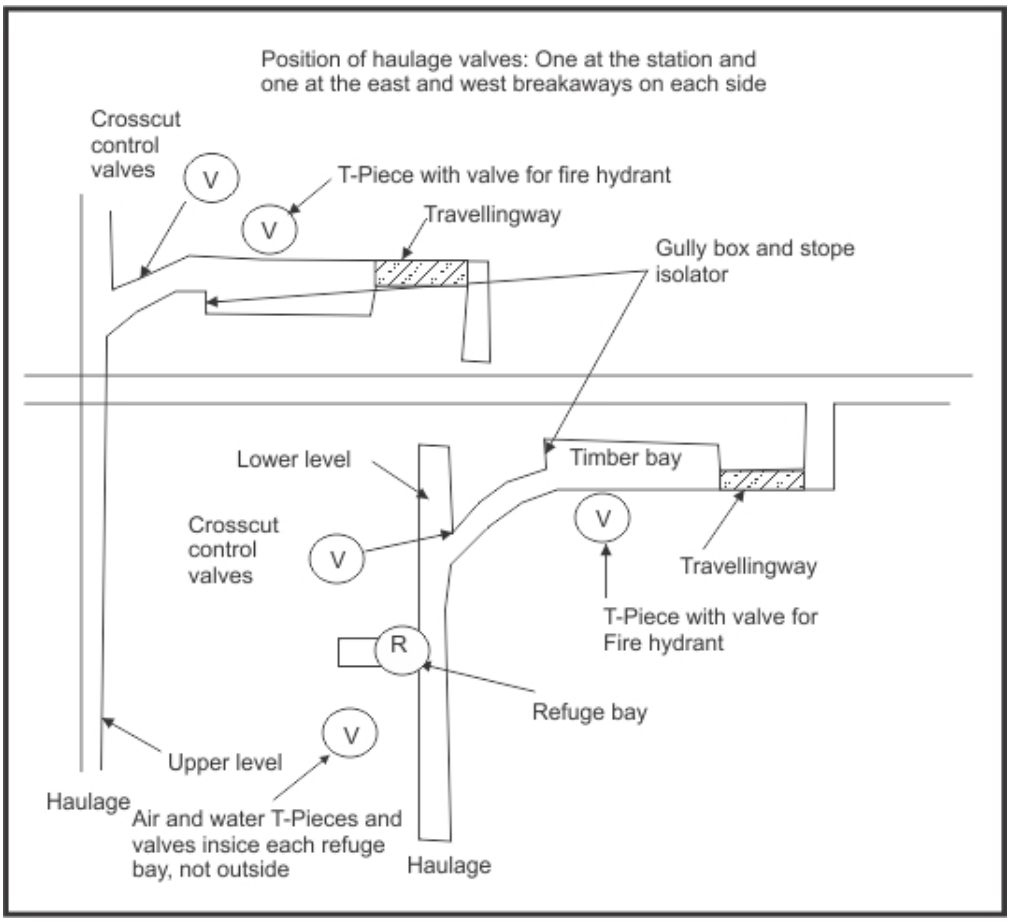


Figure 15: Example of valve positions as per a mines standard (plan view)

Inside the stope, pipe installations are as shown in Figure 16, using pig-tail eyebolts and chains. The installations are made from the crosscut, through the travelling way and into the raise (on-reef inclined excavation).

back

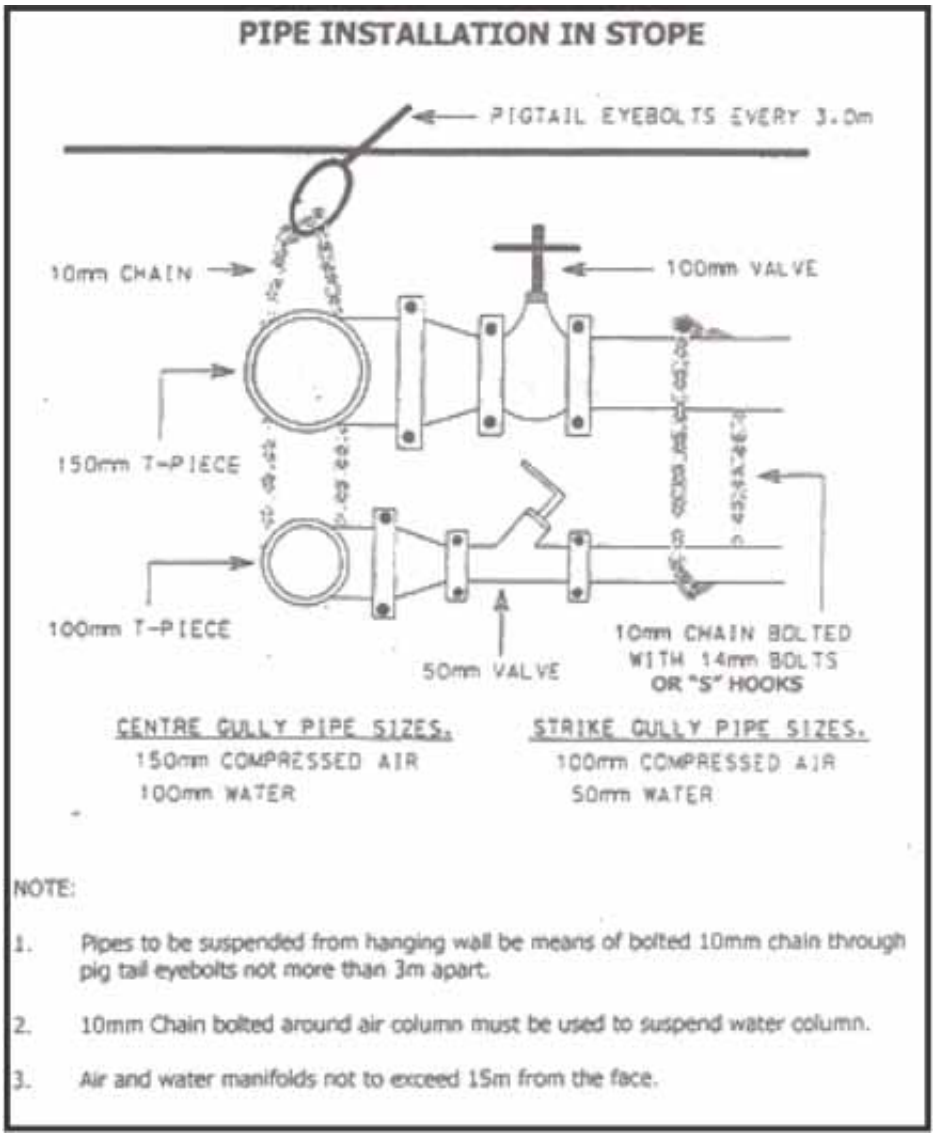


Figure 16: Example of air and water pipe suspension in a stope (section view)

Figure 17 shows an example of an electrical cable installation where cables are routed from the surface substations, down the shaft in cable racks, suspended on brackets attached to the shafts walls and out on the various levels. Along the levels it is routed to underground substations, from where it is distributed to the different mining sections. The cables are routed along the haulages, into crosscuts, through travelling ways and into the raises towards gully distribution boxes. Electricity in stoping sections is mainly used for lighting and to power electric winches.

back

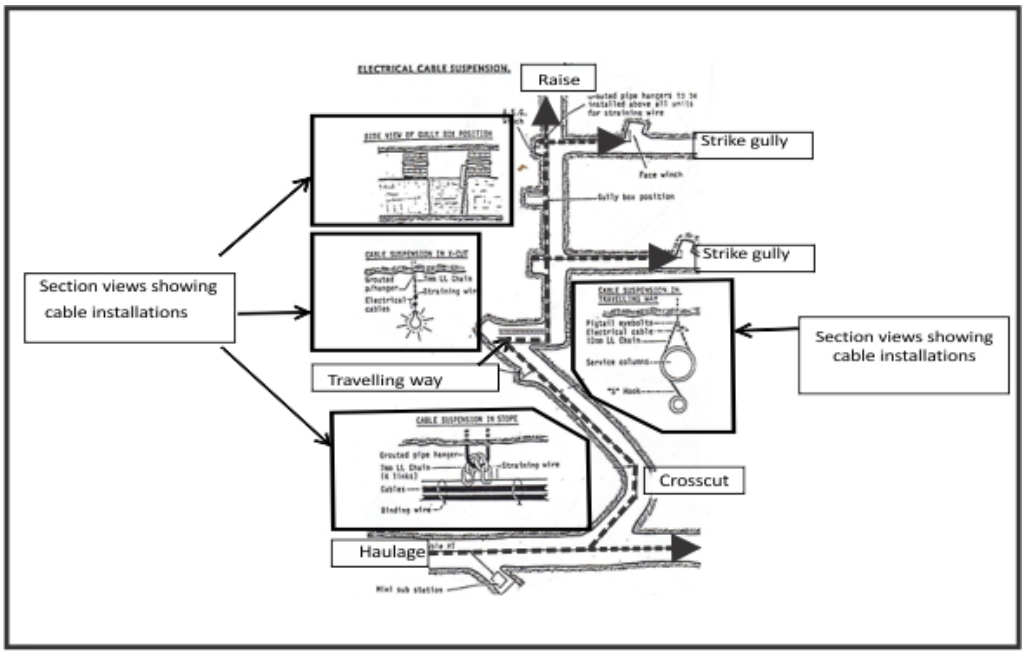


Figure 17: Example of the electrical cable suspension from the haulage to the stoping section (plan and section views)

Figure 18 is an example of a ventilation (air) standard in development ends where a haulage, return airway (RAW), connecting crosscut (between the haulage and RAW) and a crosscut (towards reef intersection) are being developed. The air flow directions are indicated with arrows. Note that fresh air is picked up with fans and pulled into ventilation pipes to be discharged on mining faces in the crosscut, haulage and RAW. Air then returns via the excavations, through the connecting crosscut to the RAW, from where it returns to the main exhaust to the surface.

The ventilation flow in a typical tabular stope is shown in Figure 19, using arrows to indicate the air flow directions. Note the presence of ventilation curtains along the gullies to ensure that air flows onto the mining faces.

back

back

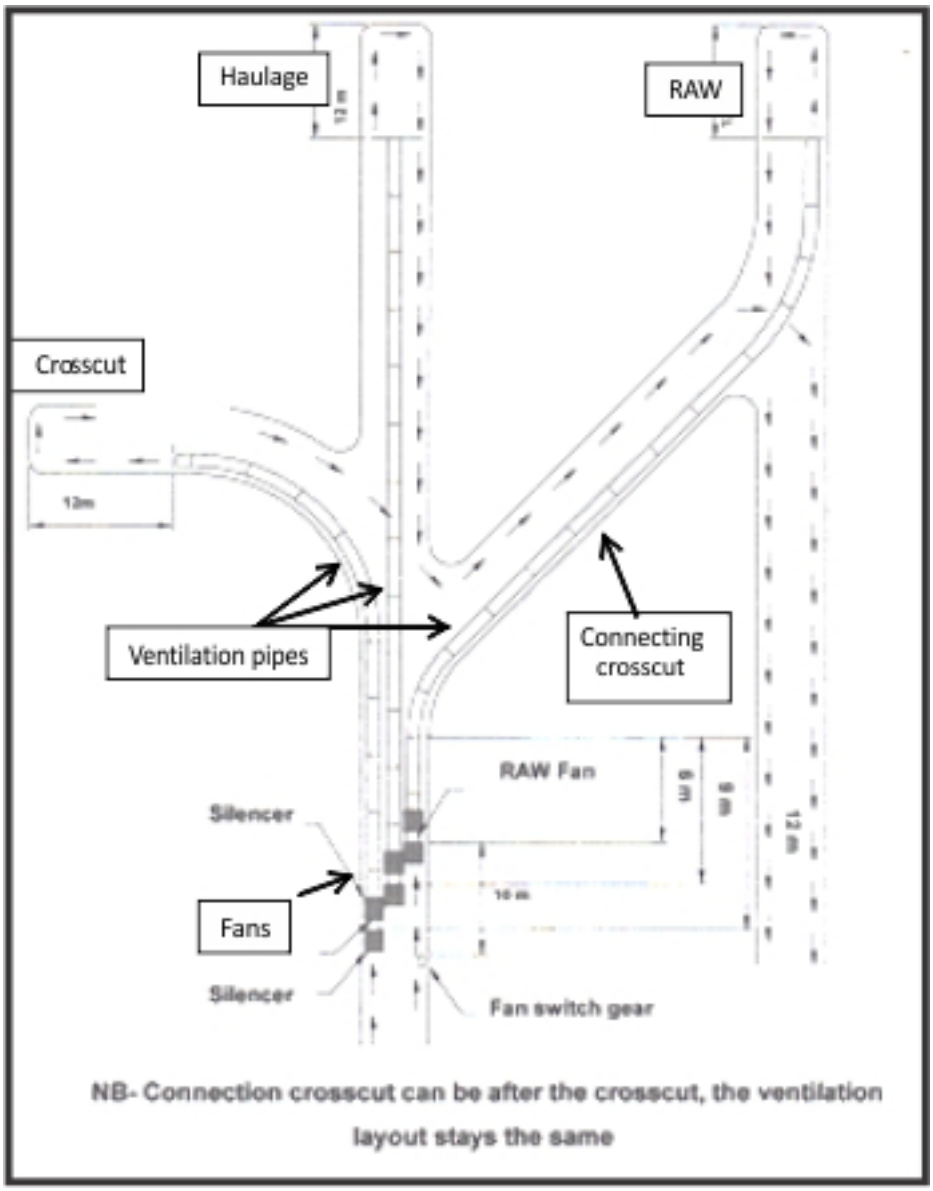


Figure 18: Example of a ventilation layout for haulages, crosscuts and a return airway (plan view)

back

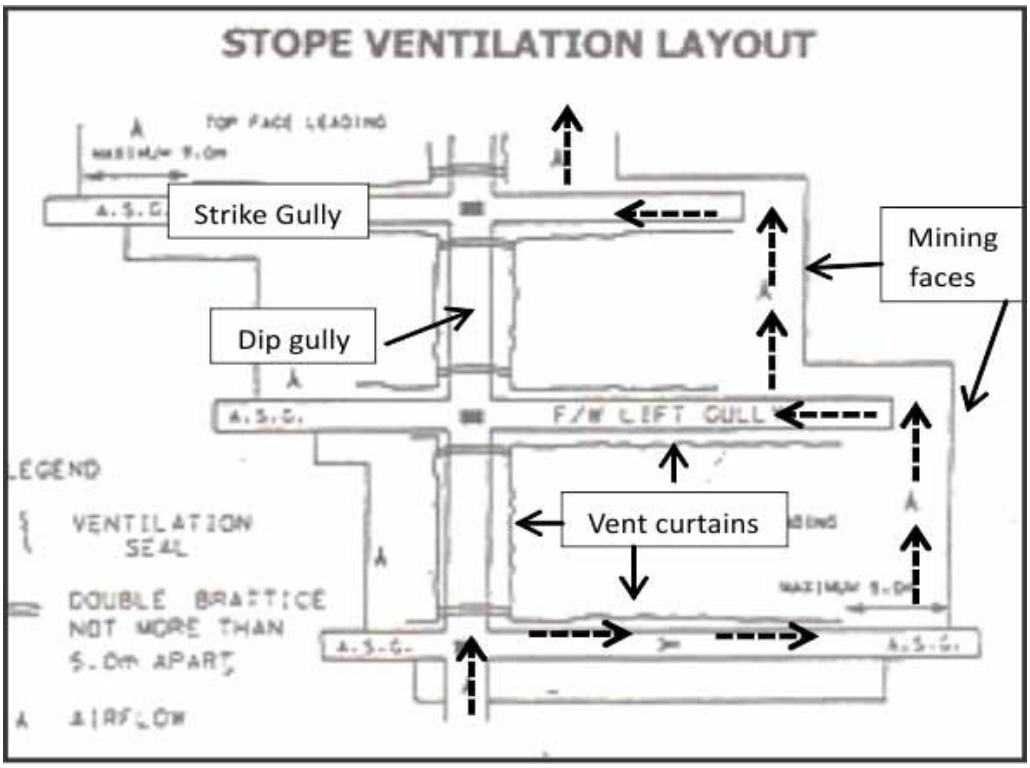


Figure 19: Example of a ventilation layout of a stoping section (plan view)

- Harmony mines standards and procedures

How broken ore is removed from a stope?

The removal of broken ore from stopes is dependent on the mining method implemented. Some examples of ore handling are discussed below and are not meant to a complete list, but merely examples of systems currently in place:

- Open stoping and caving methods: Ore is drawn through drawpoints and loaded by loaders into trucks, transported via the drives and decline shafts to the surface or to ore passes from where it is dumped onto conveyor belts and taken out of the mine (Figure 21 and Figure 23);
- Shrinkage: Ore is drawn through drawpoints and loaded onto trucks with loaders or through ore passes equipped with box fronts (engineered steel construction that allows control of the ore flow) into hoppers (track-bound container) from where it is pulled to the main ore pass systems with a locomotive, then tipped onto a conveyor belt system or 'skips' (container that moves up and down a vertical shaft) that remove the material via declines or vertical shafts respectively;
- Coal mining: Ore is either loaded by loaders onto trucks and then via conveyor belt systems out of the mine (Figure 20), or, when face shearers are used, ore is moved along the face on a conveyor and then dumped onto another conveyor system to be taken out of the mine (Figure 22);
- Tabular ore body layouts:
  - Track-bound: Ore is scraped along the mining face and into the strike gullies; then it is scraped along the strike gully towards the raise / centre gully and into ore passes. In the footwall tunnels, ore is drawn through box-front construction into hoppers, taken to main ore pass systems with locomotives and then removed from the mine either by 'skips' in a vertical shaft or conveyor belt system via an incline shaft;

back

- Trackless: Ore is loaded onto trucks using loaders and transported to ore passes or conveyor belts for transport either by 'skips' in a vertical shaft or a conveyor belt system via an incline shaft;
- Hybrid: Ore is scraped along the mining face and into the strike gullies; then it is scraped along the strike gully towards the raise / centre gully and into ore passes. Below the ore passes, the ore is loaded onto trucks using loaders and transported to ore passes or conveyor belts for transport either by 'skips' in a vertical shaft or a conveyor belt system via an incline shaft.

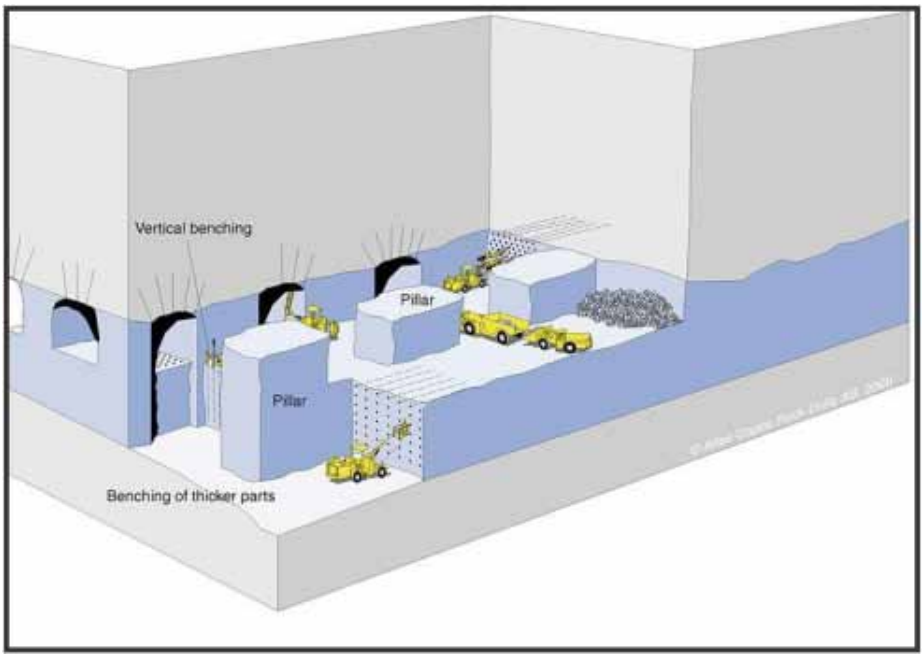


Figure 20: Removal or ore from board-and-pillar stopes

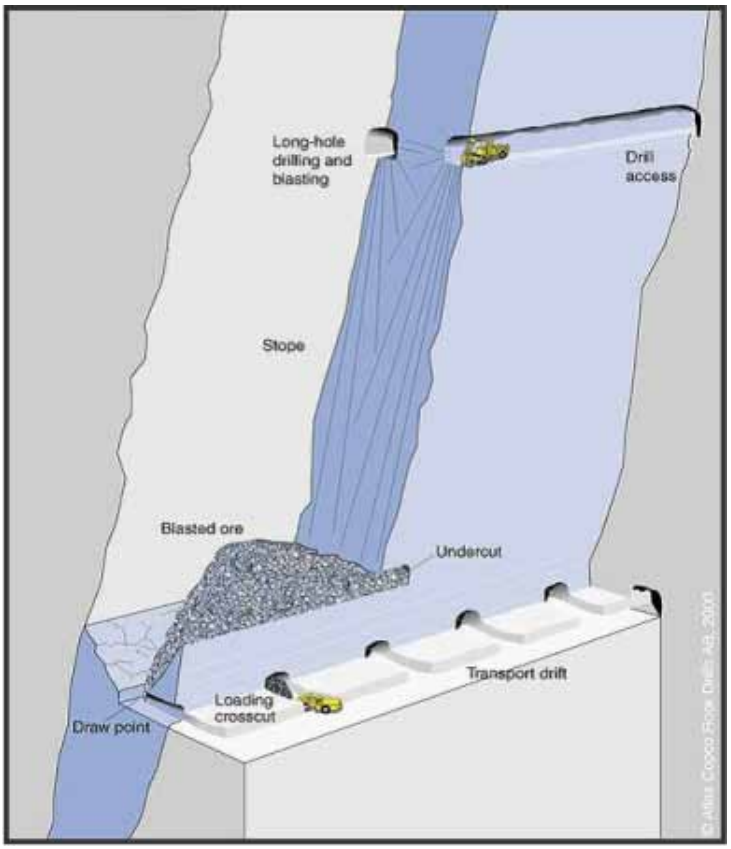


Figure 21: Removal of ore from an open stope

back

back

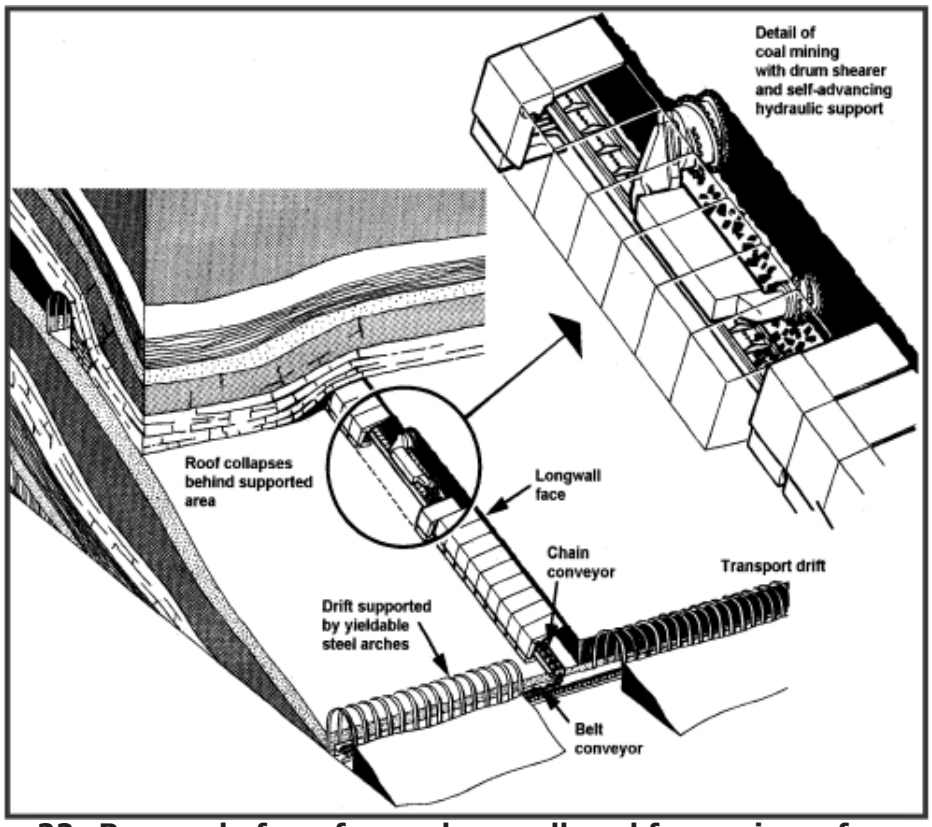


Figure 22: Removal of ore from a longwall coal face using a face shearer

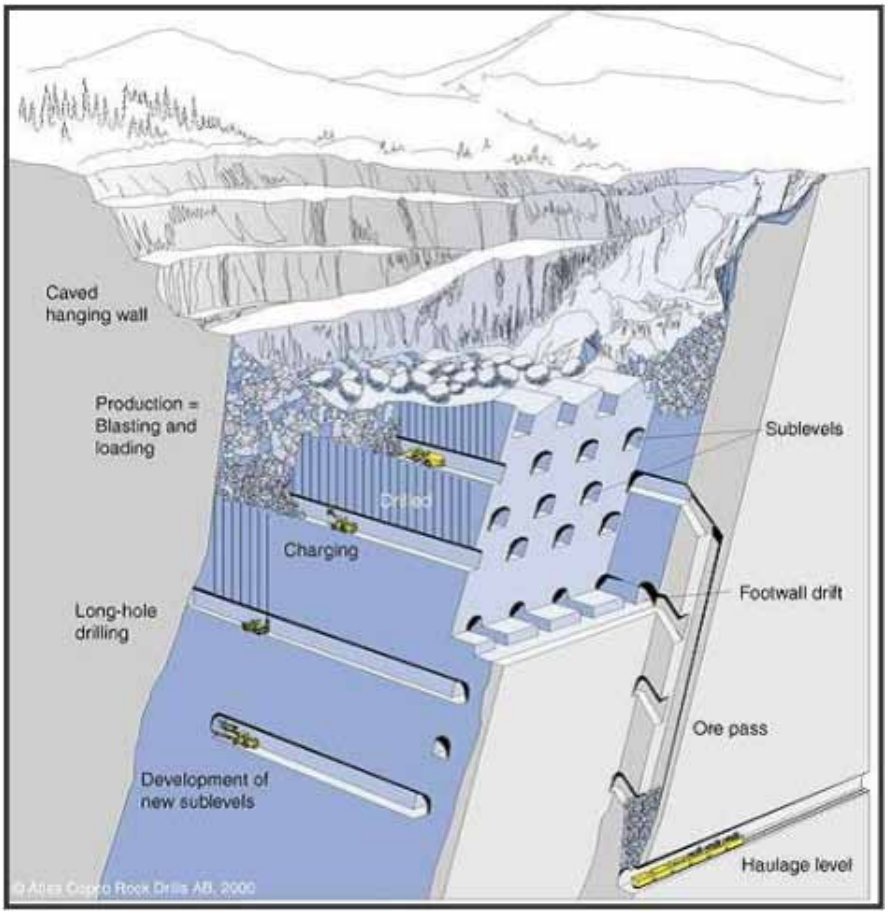


Figure 23: Removal of ore from a caving stope

back

- Various websites
- Various mine procedures and systems

4.2.2 SHAFT LAYOUTS

LEARNING OUTCOME 4.2.2.1

DESCRIBE AND EXPLAIN THE LAYOUT ASSOCIATED WITH A VERTICAL SHAFT, SHOWING THE METHOD OF HANDLING THE BROKEN ORE

In the case of vertical shafts, near vertical shaft ore pass systems are developed close to the shafts and normally consist of reef and waste systems. These are developed to interlink between the levels, which means that ore or waste can be thrown down the ore pass at any level and will eventually land up at the main loading box below the bottom working level, see Figure 24.

From this main loading box (Figure 25), skips are loaded in the shaft in which the ore is then vertically transported up to surface. On the surface, the ore is dumped onto a conveyer belt where the reef is transported to the reduction plant for processing and the waste rock is transported to waste rock dumps.

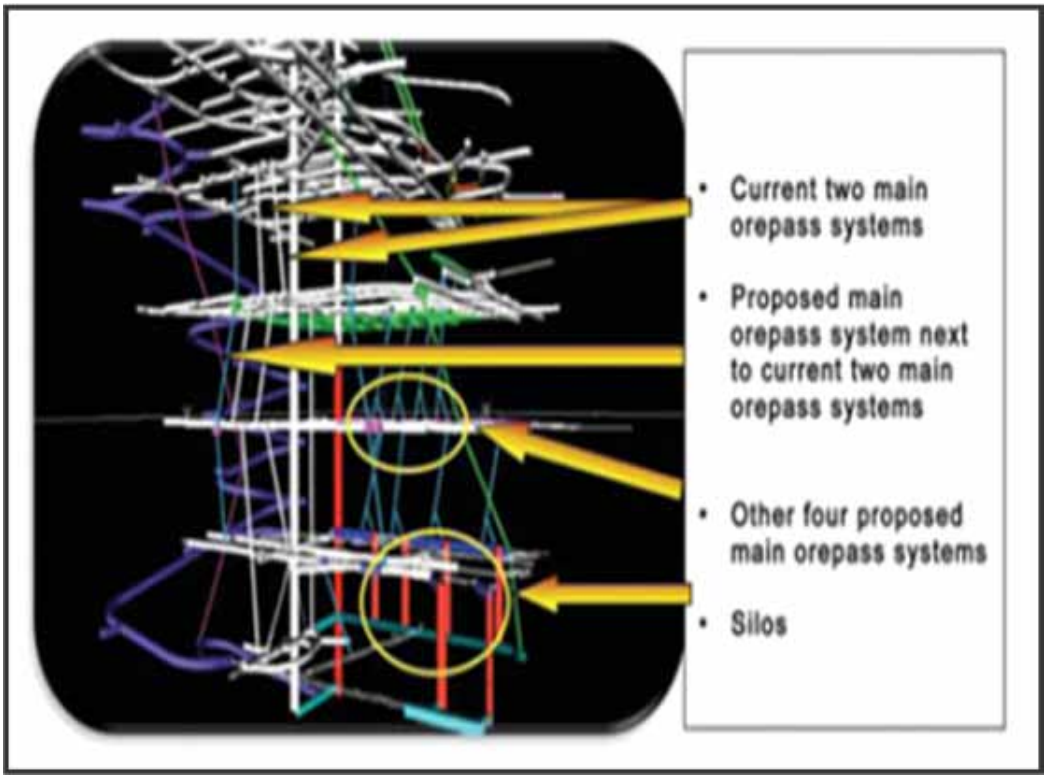


Figure 24: Section of the south Deep, Twin shafts showing the ore pass systems (Ore pass best practice at South Deep JA Maree) www.saimm.co.za/Journal/v111n04p257.pdf

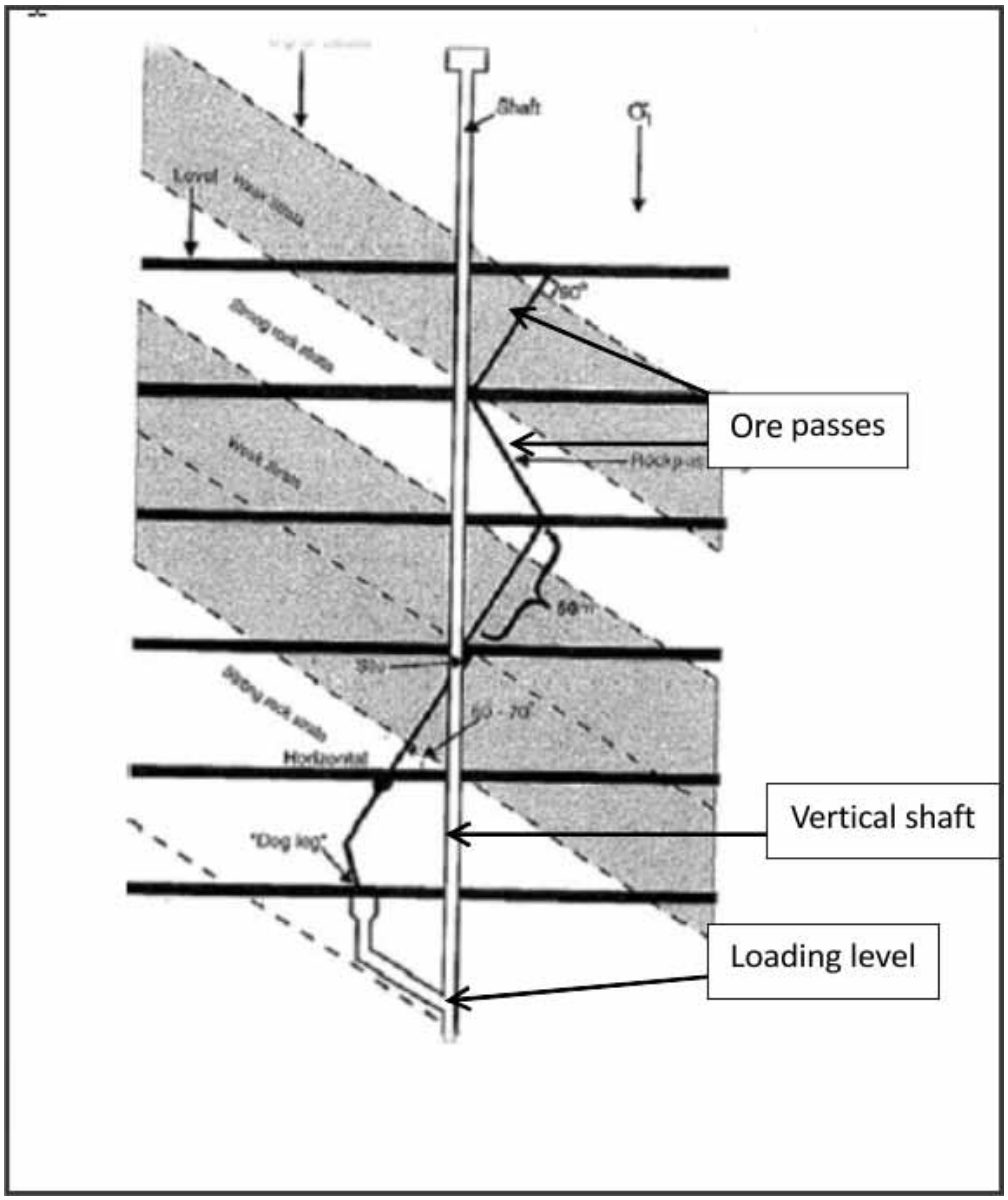


Figure 25: Section showing an orientation of an ore pass system near a vertical shaft (Ore pass best practice at South Deep JA Maree)

REFERENCE

- Ore pass best practice at South Deep JA Maree. www.saimm.co.za/Journal/v111n04p257.pdf

LEARNING OUTCOME 4.2.2.2

DESCRIBE AND EXPLAIN THE LAYOUT ASSOCIATED WITH AN INCLINED SHAFT, SHOWING THE METHOD OF HANDLING THE BROKEN ORE

In the case of incline shafts, broken ore can either be transported up the incline along a conveyor belt, as used in the coal and shallow platinum mines, or by means of an incline shaft ore pass system, as shown in Figure 26.

In the case of an incline shaft ore pass system, the ore is transported to inter-level ore pass systems where it is dumped to eventually land up in

back

the main loading station box. From the main loading station, it is loaded into skips by means of opening chutes where it is hoisted to surface via the decline shaft.

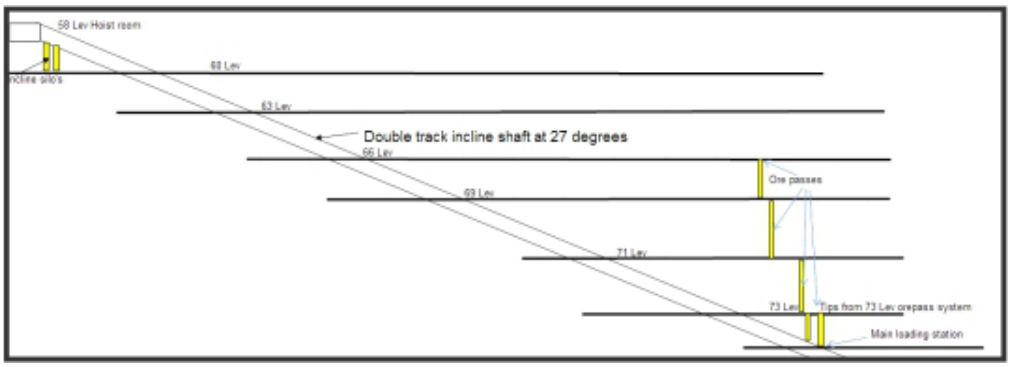


Figure 26: Sketch illustrating a typical ore pass system used in incline shafts

LEARNING OUTCOME 4.2.2.3

DESCRIBE AND EXPLAIN HOW VERTICAL SHAFTS ARE PROTECTED FROM THE EFFECTS OF MINING INDUCED STRESSES AND DISPLACEMENTS

Vertical shafts provide the access to the ore body, allowing men, material and ore to be transported into and out of the mine and are critical to the ventilation of the mine. It is therefore critical that the shaft barrel must be adequately protected against anything that might cause damage to it. Since rock damage is mostly related to rock mass strength, field stress levels and the subsequent failure and displacement of failed rock into the barrel, protection against excessive stress levels is very important (the occurrence of seismic activity close to the barrel that causes damage is a dynamic form of stress-related damage and will also be briefly mentioned).

To prevent excessive stress levels, all mining practices should be kept away from the barrel (including issues such as stoping and development).

NOTE

It has been known for development to also impact on the stability of shaft barrels, especially when excavations are developed too close to the barrel (large chambers) or where years of scaling in ore passes developed too close to the barrel originally, eventually resulted in stress interaction between the barrel and the scaled out ore pass with substantial subsequent damage to the shaft barrel.

For protection of the shaft barrel, shaft pillars are designed to keep stress levels around the shaft barrel as low as possible. By leaving these substantial areas of unmined ground around the barrel as a shaft pillar, stress levels on large and essential service excavations normally situated around the barrel are also reduced. These excavations may include crusher chambers, workshops, cooling chambers, etc and are all critical to the successful operation of a mine. The shaft pillar abutment stresses will be significantly larger than the stresses in the centre of the pillar and service excavations will need to be sited away from these abutments (Figure 27).

back

back

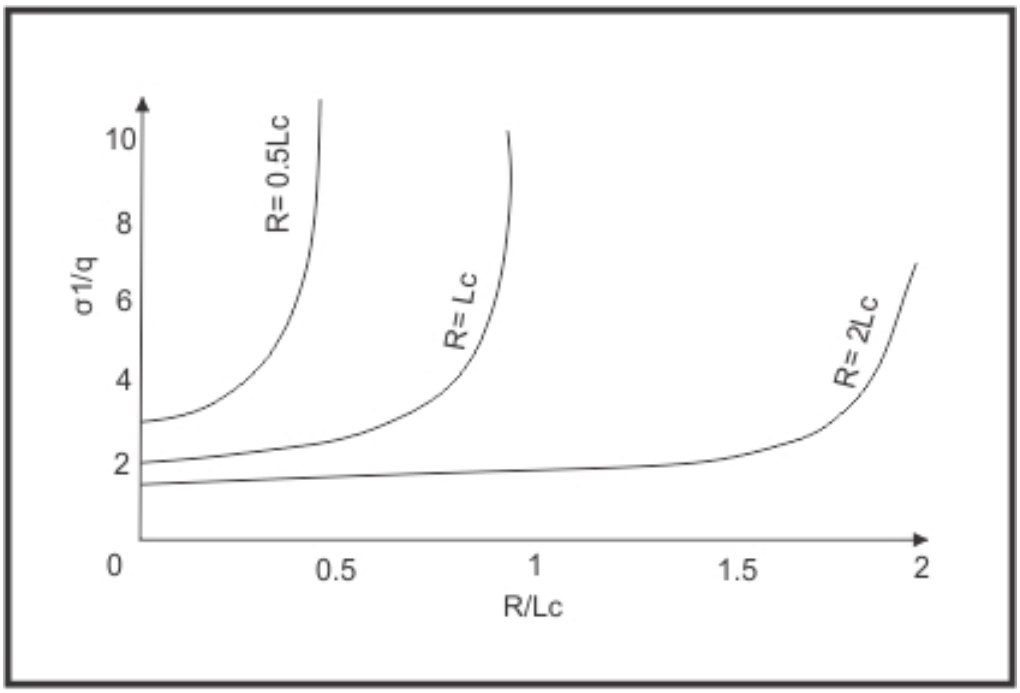


Figure 27: Stress levels across shaft pillars at various sizes (Budavari, 1983)

Total stress levels

Industry-accepted vertical shaft protection criteria stipulate that the major principal stress (σ_1) at the position of the barrel in the centre of the shaft pillar, should be limited to a critical stress level, which is equal to $\sigma_1 < \sigma_c/3$ where σ_c is the uni-axial compressive strength of the rock material.

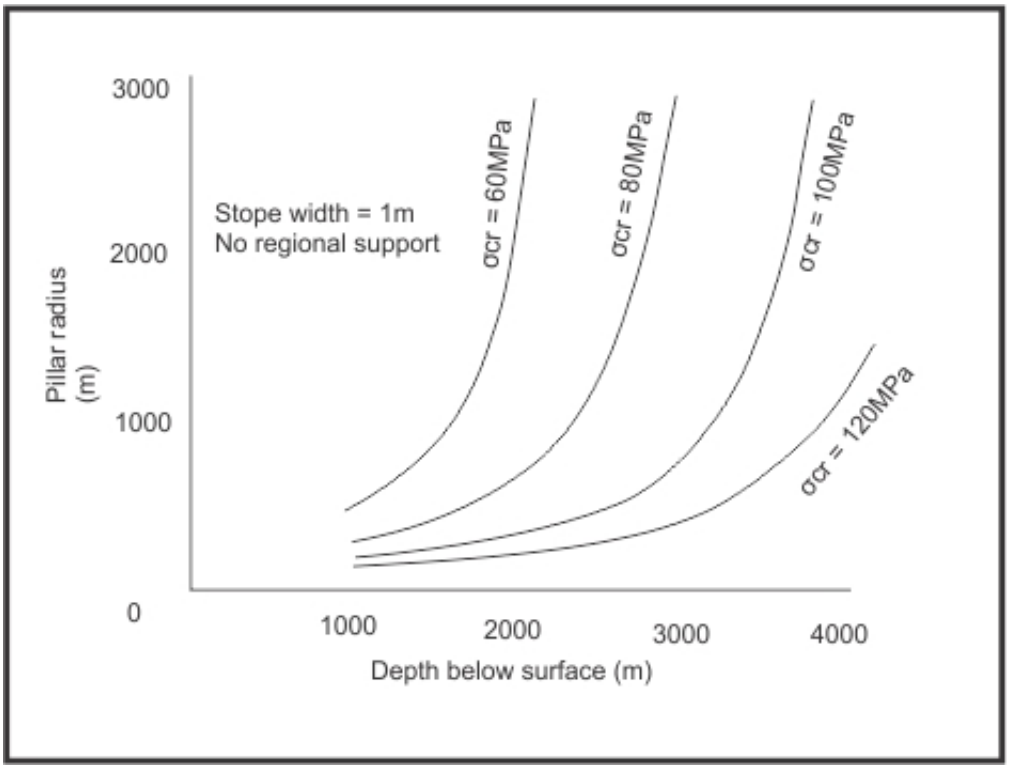


Figure 28: Shaft pillar design chart using critical stress levels (Budavari, 1983)

back

back

Induced strain (deformation) and tilt levels

The induced strain and tilt need to be limited to prevent damage to the steelwork and concrete lining. Damage caused by rock movement around the barrel can be the result of three different processes, illustrated in Figure 29:

- Straining of the rock mass due to high stress levels (elastic behaviour) in a vertical direction: The result when a shaft pillar is designed with no mining allowed through the shaft barrel;
- Excessive downward displacement of the shaft barrel: The result of vertical movement caused by hangingwall and footwall closure when the shaft pillar is mined out around the shaft barrel;
- Displacement of the rock around the barrel in a semi-horizontal direction (tilt): The result of non-concentric mining around the barrel causing closure into the mined-out excavations and rock mass movement in a semi-horizontal direction.
- Of these processes, only the first is possible during the periods when a shaft pillar is still in place (assuming the shaft pillar is large enough not to allow tilt), the other two processes occur once mining out of the shaft pillar commences. This means that a shaft pillar must prevent excessive vertical straining and should be large enough to ensure that mining beyond the pillar will not induce tilt on the barrel.

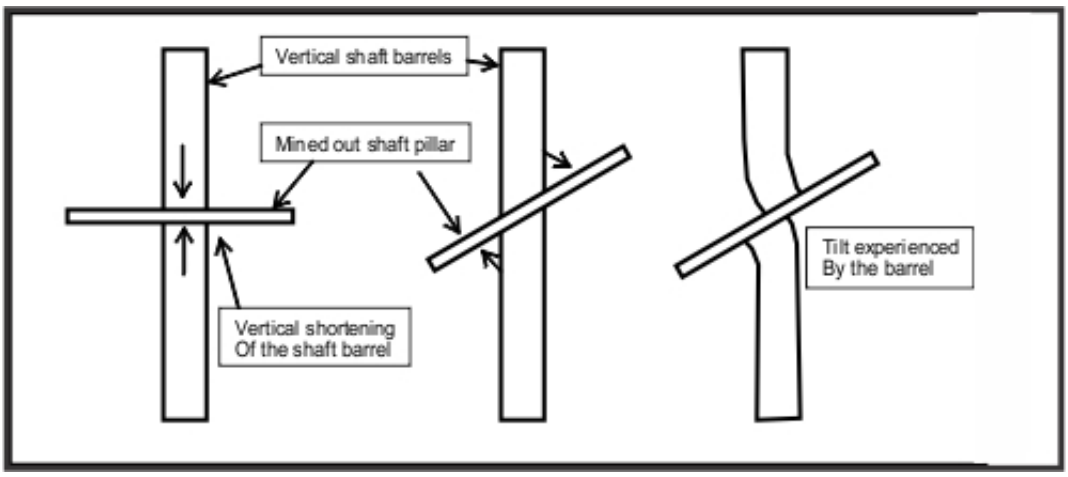


Figure 29: Explanation of barrel shortening and tilt

Rules of thumb are available to estimate strain and tilt level, but it is suggested that the modern rock engineers will rather use numerical models to design the size of shaft protection pillars.

NOTE

Rules of thumb for maximum strain and tilt values have been provided by Budavari et al. (1983) and suggest that εzmax = 0.28 (sm/R) and tmax=0.13 (sm/R) where sm is the effective stoping width and R the radius of the shaft pillar (provided the depth of the shaft pillar is such that no surface effects are possible).

When designing shaft pillars, the maximum mining stage or worst case is analysed and the use of values obtained from the rules of thumb above as maximum limits is acceptable, even though experience has shown that vertical strains and tilts should be limited to 1.0 x10-3 at a maximum stoping width of 1.0m.

As experience in shaft pillar design and extraction grew, several criteria were developed, resulting in Table 6 below (Stacey, 2001). Numerical modelling and the application of a conservative induced strain criterion of (0.2 x 10-3) allowed the creation of Figure 30 (Stacey, 2001), and can assist in ensuring good, but low accuracy level estimations of the required shaft pillars at certain mining depths.

back

Type of damage	Criterion	Reference
Unspecified damage	$\sigma_z' < 17MPa$	Wilson (1971)
Unspecified damage criterion based on back analysis	$\varepsilon_z' < 1x10^{-3}$	Salaman (1974)
	$\varepsilon_z' < 1x10^{-3}$	Wagner (1985)
	$\varepsilon_z' < 1x10^{-3}$	Daeman (1972)
	$t' < 1x10^{-3}$	Wagner (1985)
Steelwork damage and increased shaft maintenance	$\varepsilon_z' < 0,2x10^{-3}$	Wilson (1971)
	$\varepsilon_z' < -0,4x10^{-3}$	Van Emmenis & More O'Ferrall (1971)
	$\varepsilon_z' < 0,4x10^{-3}$	Esterhuizen (1980)
Tensile fracture of concrete lining	$\varepsilon_z' < 0,51x10^{-3}$ - $\varepsilon_z' < 0,3x10^{-3}$	Kratzch (1983)
	$\varepsilon_z' < 0,2x10^{-3}$	Budavari (1986)
Compressive fracture of concrete lining	$\varepsilon_z' < 0,7x10^{-3}$	Jager & Ryder (1999)

Table 6: Damage criteria for shafts (Stacey, 2001)

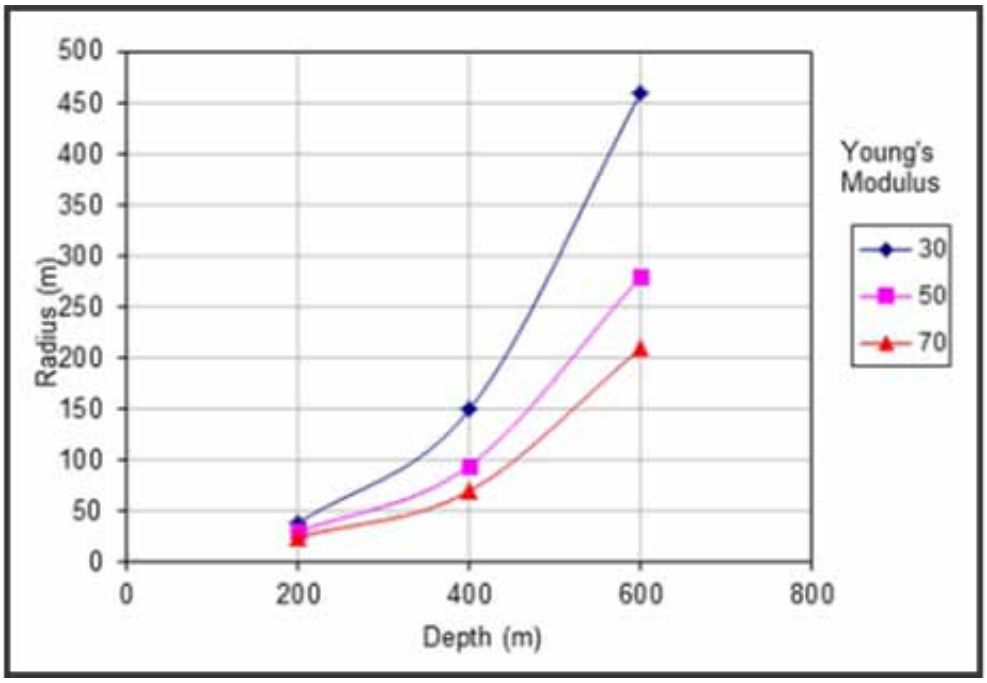


Figure 30: Shaft pillar radius vs depth for varying rock mass stiffness (Stacey, 2001)

NOTE

Other graphs are also available from Stacey (2001) and should be utilised for low-level feasibility estimates of the shaft pillar sizes.

REFERENCE

- Chapter 10, Budavari S., Rock mechanics in Mining Practice, 1983.
- Page 121, Stacey T.R., Best practice rock engineering for Other Mines, 2001.

LEARNING OUTCOME 4.2.2.4

DESCRIBE AND EXPLAIN HOW INCLINED SHAFTS ARE PROTECTED FROM THE EFFECTS OF MINING-INDUCED STRESSES AND DISPLACEMENTS

When inclined shafts are sunk in the ore body, it is also essential to leave a pillar to protect the shaft and a similar method to the vertical shaft pillar design is used where the resulting major principal stress must be less than the critical stress for the rock in which it is sited ($\sigma_1 < \sigma_{critical}$). Required pillar widths at various depths can be estimated using Figure 31.

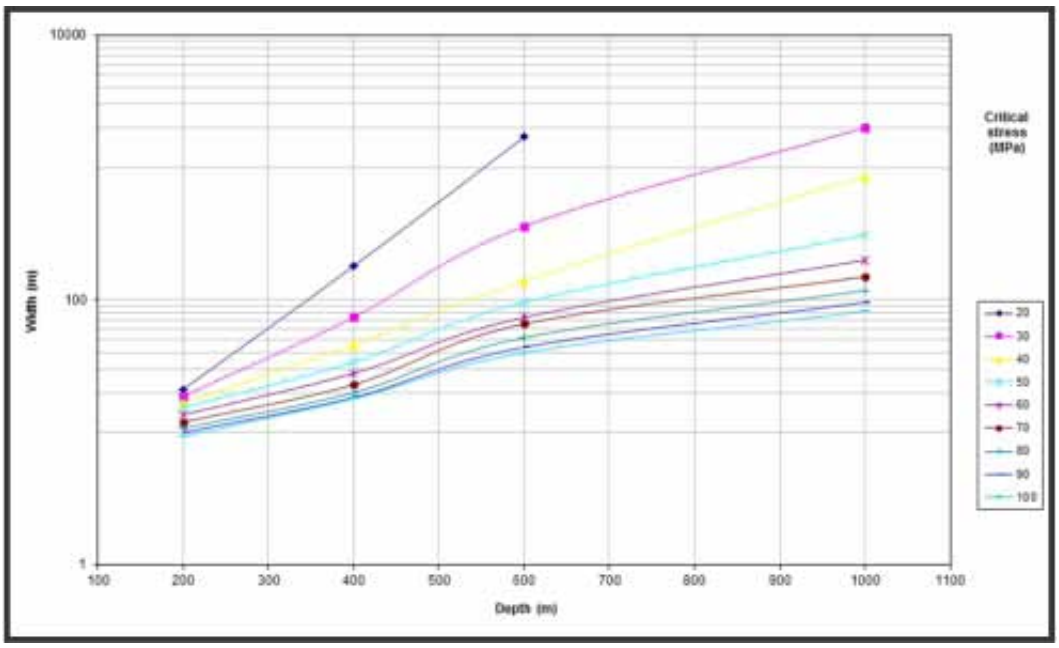


Figure 31: Incline shaft pillar dimensions (Stacey, 2001)

When the shaft is not sited on, but below the ore body, larger pillars will be required unless the shaft is placed very close to the reef. De-stressing the shaft barrel by over-stoping it with mining early in the life of the shaft should be considered to limit induced stress levels on the shaft and so also limit potential for damage.

Since the field stress in a decline shaft pillar will be orientated vertically, stress damage and the subsequent increase in displacements into the shaft barrel will mostly be limited to sidewalls of the shaft, while rails, pipes etc are usually placed along the footwall or suspended from the hangingwall. This means that substantially more displacement can be absorbed by the barrel before engineering works are affected, compared to the vertical shaft, where displacement of walls will immediately affect the steelwork and concrete lining.

NOTE

*Areas may exist where substantially higher k-ratios are present and where some hangingwall and footwall displacements may result. It is essential to know the virgin stress field when numerical modelling is conducted to d*etermine the most appropriate shaft pillar size.

REFERENCE

- Chapter 10, Budavari S., Rock mechanics in Mining Practice, 1983.
- Page 121, Stacey T.R., Best practice rock engineering for Other Mines, 2001.

LEARNING OUTCOME 4.2.2.5

DESCRIBE AND EXPLAIN HOW THE ORE AROUND A VERTICAL SHAFT MAY BE MINED EARLY TO AVOID THE NECESSITY OF LEAVING A SHAFT PILLAR

Early extraction of the ore around a shaft barrel could include the following possible scenarios:

- The ore body is mined prior to the sinking of the shaft: If access to the ore body is possible from another shaft, the ore body can be extracted before the shaft is sunk. This allows the stress levels to change to very low levels prior to sinking (mining de-stresses the barrel), while displacements will be allowed to occur and will terminate before the barrel is developed. It is critical that the mined-out area is sufficiently large to ensure that stress-level increases at the position of the barrel are prevented.
- The ore body is mined soon after the shaft sinking has been completed: Removal of the shaft pillar at this stage will limit stress levels to the absolute minimum, but will still expose the barrel, steelwork and lining to displacements. To limit the impact of these displacements so early in the life of the shaft, the lining around the ore body intersection may be left out (only mesh and shotcrete or fibre reinforced applied to the walls) and the steelwork may be suspended or not installed until after the extraction of the shaft pillar. Each of the options has disadvantages, but conditions will be better than extraction at the end of the life of the mine.

REFERENCE

- Chapter 10, Budavari S., Rock mechanics in Mining Practice, 1983.

LEARNING OUTCOME 4.2.2.6

DESCRIBE AND EXPLAIN METHODS OF AVOIDING SHAFT PILLARS

Shaft pillar can only be avoided if the barrels do not intersect the ore body and there are a number of possible layouts that could be investigated should the avoidance of locking up ore be a major concern and early extraction is not possible.

- Sinking beyond the sub-outcrop (Figure 32):
  - Mining has limited potential to affect the stability of the shaft barrel if placed far enough from the sub-outcrop
  - Long crosscutting tunnels are required and travelling and tramming distances and time become excessive;
  - Distances between the main and sub-shaft may become large and will require tramming between shafts of ore from one shaft to the other over large distances;
  - All development are within the ore body footwall and will not require the ore body to be traversed by any tunnels;
- Crosscutting through the reef horizon from the main shaft and continuing with a sub-vertical shaft (Figure 33):
  - Shafts can be placed well away from the ore body to prevent any impact of mining on the barrels;
  - Tunnels will be placed in the ore body hangingwall and a number of protection pillars will be required on those haulages

that have to go through the ore body. In the end, the pillars left intact to protect each of these haulages for the life of the mine must be compared to that left with a normal shaft pillar;

- Distances between the main and sub-shaft may become large and will require tramming between shafts of ore from one shaft to the other over large distances;

- Site the shaft's position within a large fault loss or dyke (Figure 34):
  - Shaft barrels will not go through the ore body, but will intersect a major fault plane. If seismic activity is expected on this mine, this option could place the barrel at severe risk to seismic damage.
  - A number of pillars will also be required to protect haulages that have to go through the ore body, as some of the tunnels will be placed in the ore body hangingwall;
  - The fault loss must be large enough to ensure that mining-induced impacts on the barrel are prevented;

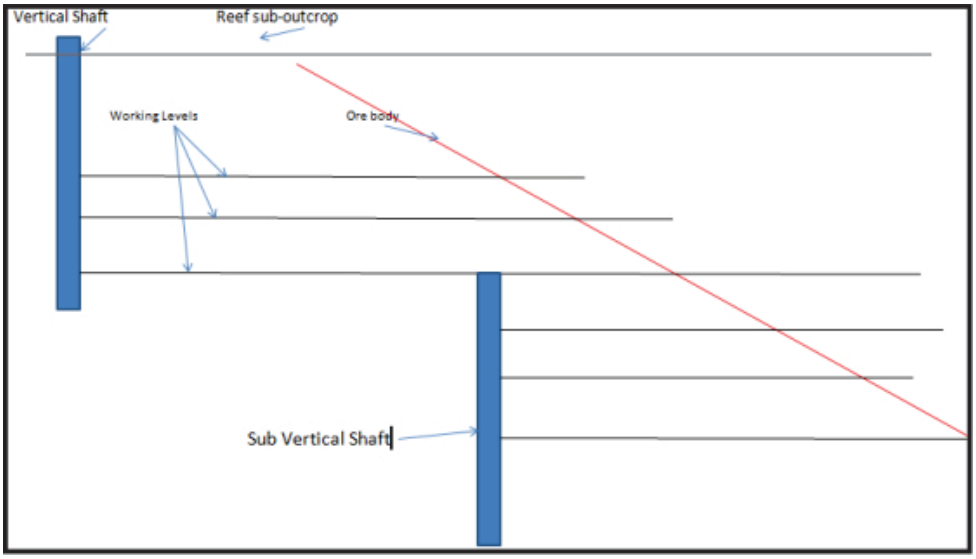


Figure 32: Avoidance of a shaft pillar by sinking it beyond the sub-outcrop

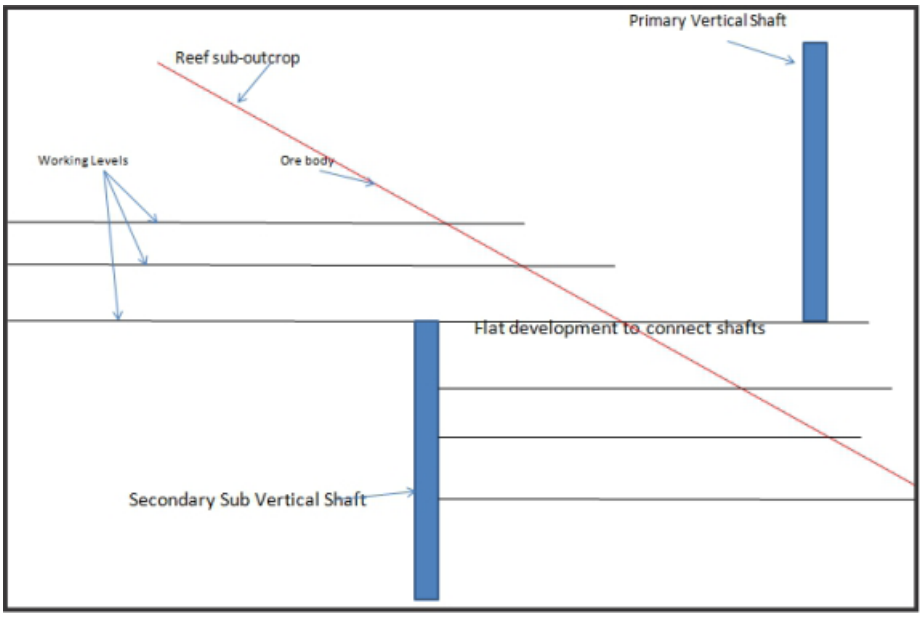


Figure 33: Avoiding a shaft pillar by stopping the primary shaft short

back

of the reef and starting a secondary shaft in the footwall of the reef

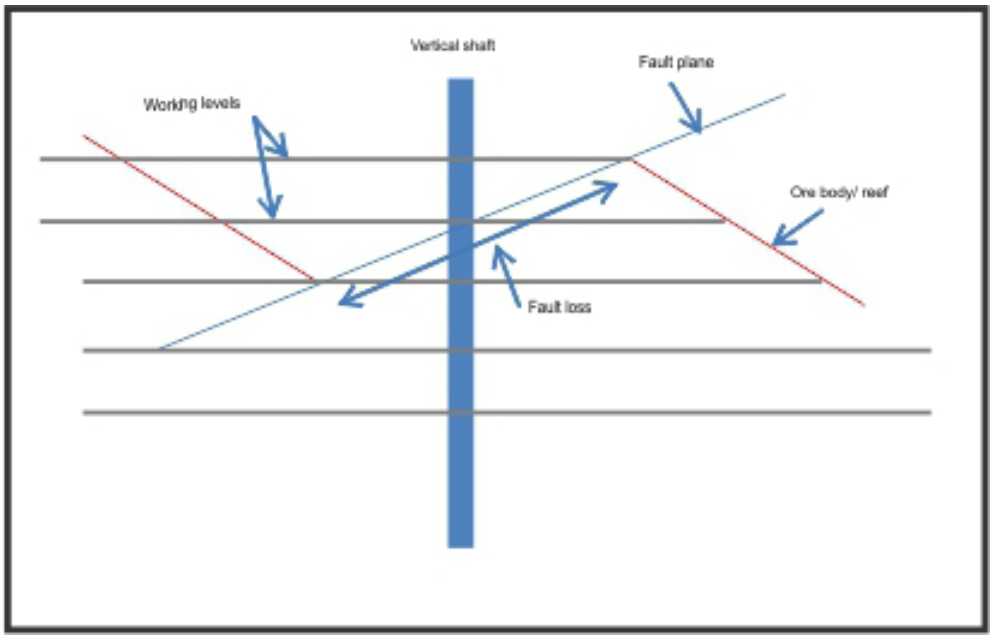


Figure 34: Avoiding a shaft pillar by siting it in a fault loss

REFERENCE

- Chapter 10, Budavari S., Rock mechanics in Mining Practice, 1983.

LEARNING OUTCOME 4.2.2.7

CONNECTION 18

"LEARNING OUTCOME 4.2.2.6" on page 250

DESCRIBE AND EXPLAIN THE IMPLICATIONS OF THE ABOVE METHODS ON MINE LAYOUTS

LEARNING OUTCOME 4.2.2.8

DESCRIBE, EXPLAIN AND DESIGN SHAFT LAYOUTS FOR THE EXTRACTION OF PARTICULAR BLOCKS OF GROUND.

To design a shaft, with or without a shaft pillar, at least the following issues are critical to evaluate:

- Barrel must be stable for the life of the mine:
  - Optimum distance away from the mining to limit any mining-induced impact (apply numerical modelling to ensure that stress, stain and tilt levels are acceptable);
  - The need for a shaft pillar if the barrel intersects the ore body or the implementation of pre-extraction;
  - The lithology of the planned barrel position so that adequate support precautions can be drafted for poor strength horizons;
  - The presence of water-bearing structures or layers that might be intersected to plan support and dewatering precautions;
  - Adequate lining design to sustain any loading that might be exerted onto the concrete (especially in weak material types);
- Must be able to access all or most of the ore body for which it is intended:
  - Levels must be possible that will allow extraction of ore, i.e. they will have to end up in the ore body footwall;

back

- Consider haulage protection pillars for tunnels that must traverse the ore body to get into the footwall, and limit the number of intersections;
- Attempt to consider future mining of ore body blocks that have not been confirmed, but could possibly exist, so that the final depth of the shaft is correct (does not require later deepening) or plan inclined shafts to deeper levels from the planned shaft (even if this is done in concept, consider possibility);

- Keep tramming and travelling distances and time to the absolute minimum:
  - Production levels are heavily dependent on the provision of 'men' and material to the mining face as well as the removal of ore from the mine. Keep this in mind in selecting the shaft site and tunnel elevations and positions;
  - Keep distances as small as possible by placing the shaft in the centre of the area to be mined (or as close to the centre as possible). Placing the shaft to one side means that mining near the end of the life of a mine will be the furthest from the shaft, and long travelling times, tramming distances, many excavations to maintain, etc. will all increase costs and may result in profit loss;
- Make extraction of the shaft pillar (early or at the end of the life of the mine) as easy as possible:
  - Plan the extraction of the shaft already at the point of designing it;
  - If pre-extract:

  1. planning should be done in detail and all layouts adapted to suite;
  2. issues of concern should include the position of the shaft barrel;
  3. area to be mined out;
  4. sequencing;
  5. support practices;
  6. backfilling of the mined-out area;
  7. placement of footwall tunnels to allow fact extraction of the area, etc.

  - If post-extract:

  1. planning should be done in concept, BUT level elevations should be planned that will prevent tunnels from being too close to ore body elevation as it is prone to be seriously damaged during extraction;
  2. the shape of the shaft pillar should make mining easier to extract later (square rather than circular);
  3. pre-adaptation of steelwork and lining in the barrel could be considered;
  4. creation of stations at most appropriate levels to be considered, even if no further development occurs on those levels until extraction is executed;
  5. ore passes for ore removal from stopes, storage and transportation to surface must be considered (i.e. will shaft ore passes be used to transfer ore to shaft bottom and then load or will a loading facility be required closer to the shaft pillar evaluation?);
  6. impact of seismic activity on excavations, shaft barrel to be considered;
  7. support systems, such as backfill, should already be considered and planned if not available, etc.

- Allowing ventilation flow into and out of working sections must be considered:
  - Heat pick-up is a function of distance and this requires the shortest possible tunnel lengths to the working areas;
  - Dimensions of the shaft barrel and tunnels (final dimensions) must all consider the amount of ventilation required;
  - The development and support of major air return systems should be planned and monitored;
  - The creation of cooling station / chamber excavations must be planned at positions where they will be stable for the life that it is required and where it will not be affected by normal mining, or shaft pillar extraction if planned for the end of life of the mine.
- Allow the placement of service excavations that would remain stable for the life of the mine:
  - All service excavations should be considered for placement depending on its life expectancy. These may include crusher chambers, ore pass systems, workshops, underground offices and control rooms, storage areas, pumping stations, dams, etc.

4.3. MINE VENTILATION

LEARNING OUTCOME 4.3.1

DESCRIBE AND EXPLAIN THE PURPOSE OF VENTILATING A MINE

Air flowing through a mine is as essential to the mine as the bloodstream is to the human body, where the blood carries oxygen to all parts of the body and regulates the body's temperature. Ventilation (the flow or air) provides a cool working environment (reducing working temperatures to acceptable limits) in all parts of the mine where personnel work. The cool air flowing through an excavation absorbs the heat in the working place and exits as hot air, which is then cooled down again before entering other working places or is exhausted out of the mine. When underground heat is excessive, the air heats up quickly and loses its ability to 'cool' the working places. Good management of the flow of air is therefore important to make sure air does not follow a path where it picks up heat in unnecessary areas and then enters working face at higher temperatures, not being able to cool it down.

The mining regulations stipulate acceptable wet and dry bulb temperatures that must be maintained at working faces so that work can continue without the risk of humans overheating.

NOTE

Heat exhaustion is a life-threatening condition and must be avoided by good ventilation systems and controls.
www.mayoclinic.com/health/first-aid-heat-exhaustion/FA00020

Ventilation is also required to clear toxic fumes from blasting and removing exhaust fumes from diesel equipment.

Mines are usually divided into sections or ventilation districts, and the main air supply has to be "split" between the various districts. To ensure sufficient air for each district, regulators and doors are installed in these districts, or booster fans are installed in districts receiving insufficient air.

Even though an adequate air volume may enter a stope, the air needs to

back

be directed to the place where it is required. Air will always flow along the path of least resistance, often bypassing the actual working faces. Therefore, travelling ways and worked-out areas in a stope must be suitably sealed off to force as much of the air as possible to flow along the working faces. Air should not be allowed to leak through worked-out areas, as this reduces the air available to cool the working faces.

For satisfactory ventilation, the following are essential:

- Adequate shaft systems
- Large airways
- Adequate fan power
- Proper splitting of available air
- Effective air control (reduction in leakage of air)
- Cooling plants (if heat conditions become too severe).

LEARNING OUTCOME 4.3.2

LIST, DESCRIBE AND EXPLAIN THE MAIN SOURCES OF HEAT IN A MINE

Ventilating air must be kept as cool as possible, i.e. heat absorbed by the air on its way to and in the stope itself, should be reduced to a minimum.

Sources of heat in an underground environment are typically:

- Heat from electrical and diesel driven machinery;
- Blasting practices;
- Geothermal gradient (increase in virgin rock temperature (VRT) due to depth below surface);
- Heat built up in worked-out areas that are not sealed off;
- Humans give off heat;
- Scraping and friction;
- Dry downcast shafts, dry intake haulages, dry inclined shafts and strict water control are essential to limit heat pick-up by the air flowing through the excavations.

LEARNING OUTCOME 4.3.3

LIST AND DESCRIBE THE MOST COMMON GASSES FOUND IN A MINE AND THEIR ASSOCIATED HAZARDS

Overleaf, in Table 7 is a list and description of the most common gases found underground.

back

back

Name	Symbol	Colour	Smell	Taste	RD	Poisonous	Explode / Burns	Soluble in water	Practical test	Occurrence	Effect on human	Max quantity
Carbon dioxide	CO_2	None	None	None	1.52	No	No	Slightly	Headache	Breathing, fires and explosions, diesel equipment. Poorly ventilated areas	Heavy breathing, dizziness, palpitation, unconsciousness	5000ppm
Carbon monoxide	CO	None	None	None	0.97	Yes	Explodes (12-72%)	Slightly	Early warning device	Fires, explosions, blasting, diesel equipment	Paralyses muscle, death	100ppm
Methane	CH_4	None	None	None	0.55	No	Explodes (5-15%)	No	Methanometer	Rotting organic material. Enters excavation through faults, dykes etc.	Burns, suffocation	1.4%
Nitrous fumes	NO_2	Red brown	Gun powder	None	1.6	Yes	No	Very	Irritating eyes, nose and throat	Blasting, diesel equipment, welding	Water blisters on lungs, drowning when blisters burst	5ppm
Hydrogen sulphide	H_2S	None	Rotten eggs	None	1.2	Yes	Explodes (4-44%)	Very	Headache and irritation to eyes	Chemical when water runs over iron pyrites	Burns lungs, numbs smelling sense of nose	20ppm
Hydrogen	H_2	None	None	None	0.07	No	Explodes (4-74%)	Slightly	Drager	Battery charging bays when acid water reacts with zinc / lead	Burns, causes oxygen deficiency	1.4%
Hydrogen cyanide	HCN	None	Bitter almonds	Sweet	0.94	Yes	Burns (5.6-40%)	No	Drager	Chemical when water reacts with backfill (material from cyanide processing plants)	Absorb through skin, paralyses nerve system	50ppm

Table 7: Gasses commonly found underground

back

LEARNING OUTCOME 4.3.4

DESCRIBE AND EXPLAIN IN PRINCIPLE HOW VENTILATION IS BROUGHT INTO AND EXHAUSTED FROM A MINE

The fresh air ventilating through a mine, enters at the down-cast shafts, is drawn through the working places where it becomes contaminated and is removed from the mine via up-cast shafts.

A typical deep level mine has:

- downcast shaft systems where the fresh air from surface enters the mine,
- intake airways through which the air flows to the workings, various connections between the workings and
- return airways through which the air passes from the working to up cast shafts.
- Fans are used to force air through the mine (Figure 35b), since natural ventilation is normally inadequate and unreliable.
- Once the air has travelled through the mine, the contaminated air and blasting fumes are removed from the mine via return airways and up-cast ventilation shafts (Figure 35a). On some mines, a single vertical shaft may be divided into downcast fresh air and up-cast (contaminated hot air) parts by means of dividing the shaft with brattice walls.

Figure 35: (a) Surface fans exhausting air form a shaft and

Figure 35: (b) underground fan

The ventilation flow diagrams presented in Figure 36 for a specific salt mine and Figure 37 for a coal mine are typical of ventilations flow in most mines, where:

- Air is pulled from atmosphere into the mine via a downcast shaft;
- Air is distributed at various levels into the mine;
- Air flows through excavations / working places and
- Returns to surface via an outlet or ventilation shaft;
- Attempts are made to limit the re-use of air.

back

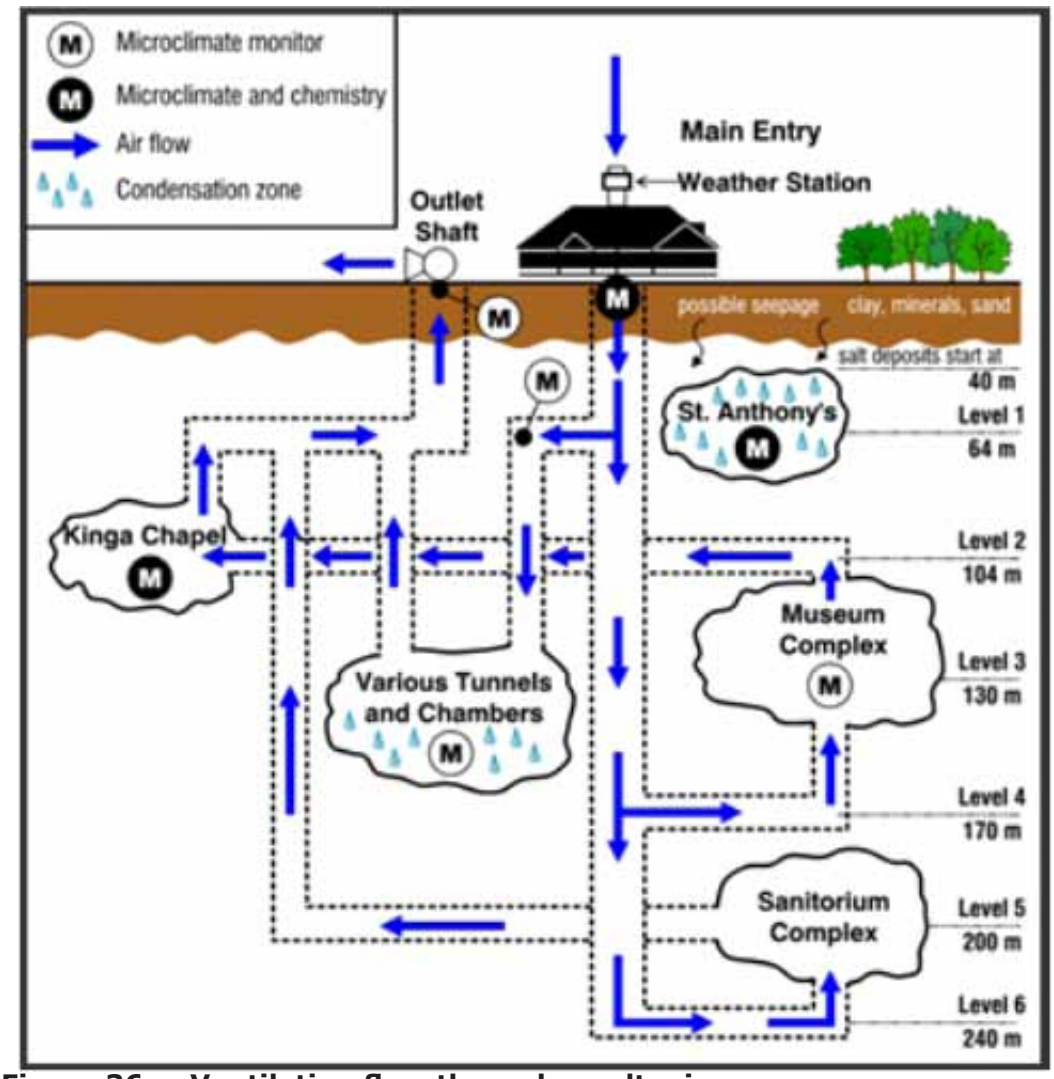


Figure 36: Ventilation flow through a salt mine

back

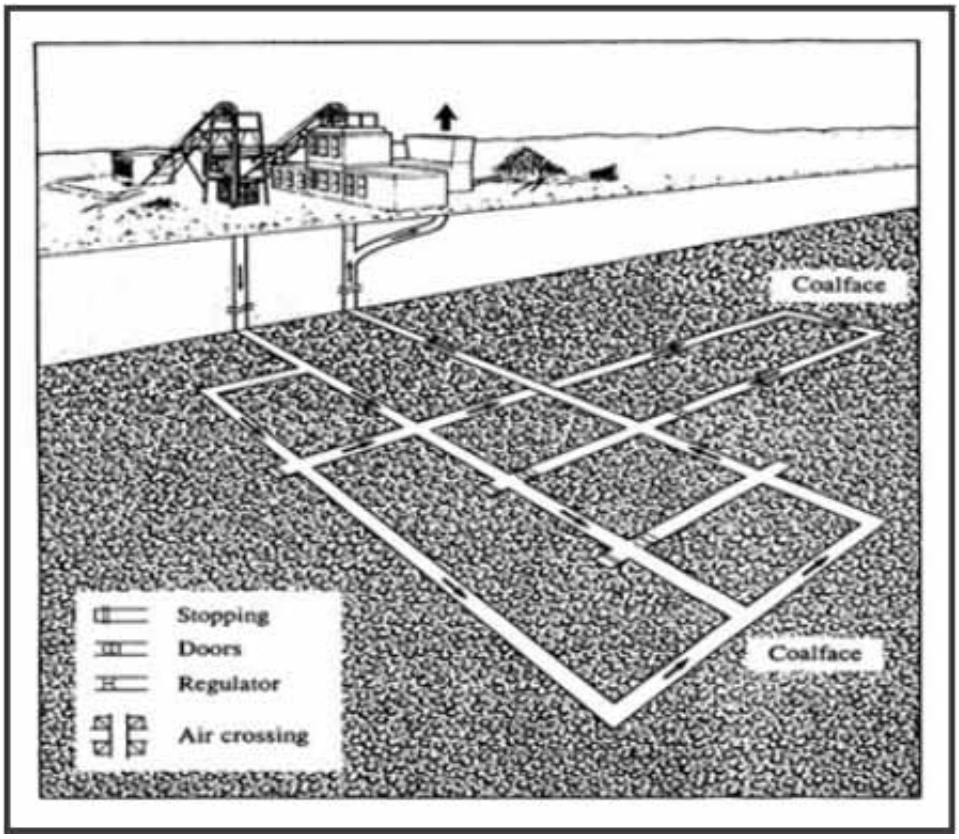


Figure 37: Ventilation flow through a coal mine

LEARNING OUTCOME 4.3.5

DESCRIBE AND EXPLAIN HOW THE FLOW OF VENTILATION IS CONTROLLED ON THE DIFFERENT MINING LEVELS

On the different mining levels, ventilation flow is controlled by means of ventilation doors and the sealing off of abandoned excavations. The use of doors is simply to resist air flow into a certain direction and to force the air to flow where it was designed to flow.

Ventilation Doors:

- Should not leak excessively and any damage to it must be fixed immediately;
- Must be kept closed and, when opened, air flow should be controlled as shown in Figure 39b (Regulations require a door to be self-closing or operated by an attendant (Figure 38));
- The ideal door should be efficient, economical and requires the minimum maintenance (Figure 39a).

back

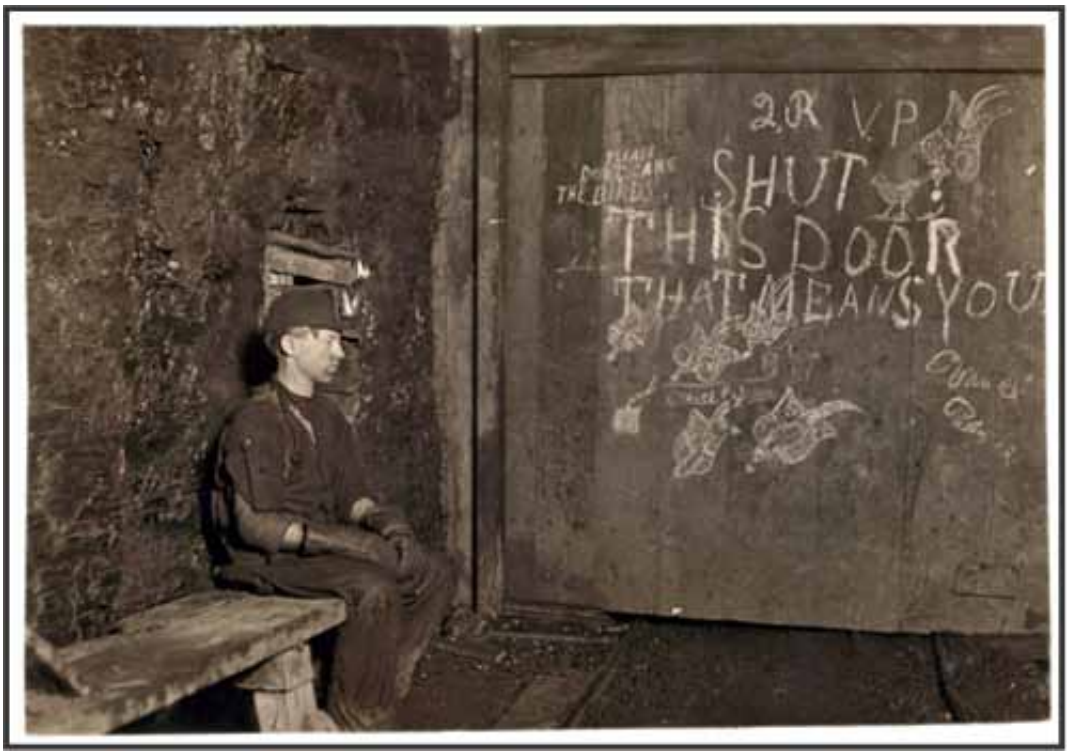


Figure 38: Ventilation door and attendant

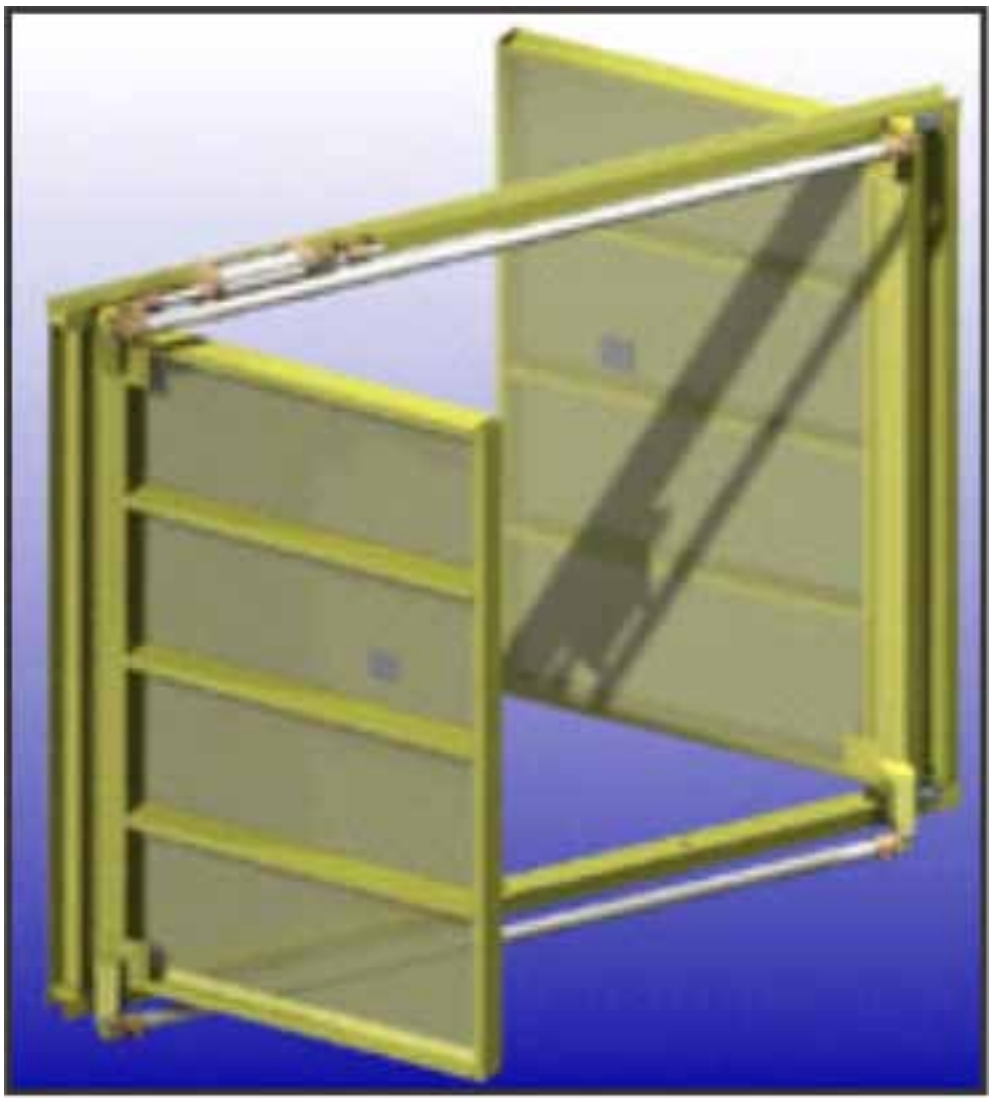


Figure 39: (a) Automatic ventilation doors and

back

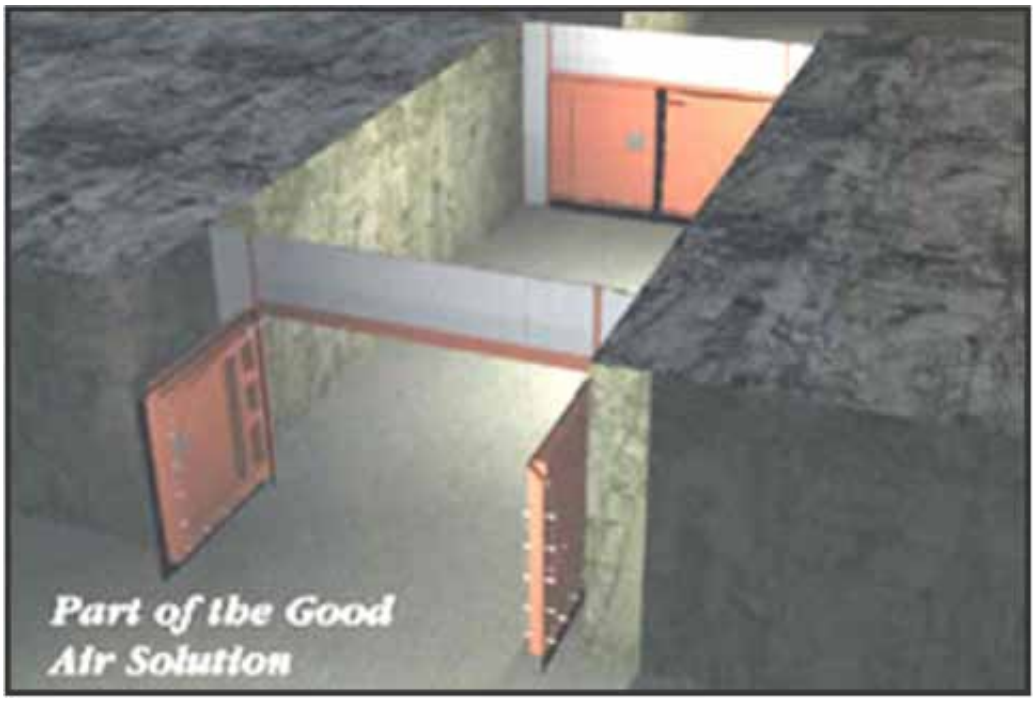


Figure 39: (b) vent door systems utilised to control air flow during opening and closing of doors

LEARNING OUTCOME 4.3.6

CONNECTION 19

"LEARNING OUTCOME 4.2.1.1" on page 232

DESCRIBE AND EXPLAIN HOW THE FLOW OF VENTILATION IS CONTROLLED IN DEVELOPMENT ENDS

Development ends are in actual fact 'dead ends' since through-flow of air is not possible, because the end is only now being developed. In an attempt to force through-flow of air, fans and ventilation pipes are used (Figure 18) to get the air onto the faces.

Purpose of fans for ventilating development ends

- Fans are used to ventilate areas of the mine where there is no natural flow of ventilation;
- Fans must be installed as per the mine standard and in such a manner that recirculation of air cannot take place. Recirculation is when the same ventilation is used over and over; you will normally suspect the ventilation is re-circulating when there is an increase in dust, gases or temperatures;

Distance of ventilation columns / pipes from the development working face

- As per mine standard
- So as to ensure that all harmful dust, smoke, fumes and gases are effectively expelled by the flow of air;

NOTE

Change in ventilation conditions

- *Should the ventilation in a working place weaken, stop or reverse (flow in opposite direction), all work should be stopped, the workers withdrawn, electric currents to winches and other electrical equipment switched off, and the condition reported to the supervisor ;*
- *As soon as the ventilation returns to normal, a full examination should be done.*

LEARNING OUTCOME 4.3.7

DESCRIBE AND EXPLAIN HOW THE FLOW OF VENTILATION IS CONTROLLED IN STOPES

CONNECTION 20

"LEARNING OUTCOME 4.2.1.1" on page 232

Stopes are the most important working places in a gold mine as this is where the gold-bearing reef is mined. More workers are employed in a stope than in any other type of working place and it is therefore essential that adequate quantities of air are provided and controlled to maintain safe and healthy conditions. The major problem in the stope ventilation is to ensure that as much available air as possible is directed and kept in the areas where men work and as little air as possible should be allowed to leak into worked-out areas (Figure 19).

Methods used to direct air into the stope faces:

- Strike walls or curtains should be kept as close as possible to the face – normally within 9.5 metres from the face (Figure 19);
- Ventilation brattices or doors should be installed at selected positions in dip-gullies and travelling ways and should be kept closed (Figure 19);
- Accumulation of rock and dirt restricting the airflow should be prevented;
- Dip gullies, which are no longer required, should be sealed off (Figure 19).

LEARNING OUTCOME 4.3.8

DESCRIBE AND EXPLAIN IN PRINCIPLE HOW REFRIGERATION IS PROVIDED IN MINE WORKINGS USING COOLED AIR, CHILLED WATER, ICE PLANTS

When the ventilation air's temperature becomes excessively high (due to heat pick-up as it flows through the working places), it will lose its ability to reduce temperatures in working places and it is required to cool the air down again, using chilled water or ice plants (Figure 40), cooling chambers or heat exchangers.

Figure 40: Fridge plant

The concept of cooling down the air using cold or chilled water is the following:

- Cold water is sprayed into the excavation or in a heat exchanger through which the warm air is forced (Figure 41);
- The water, when in contact with the air, absorbs the heat from the air, thereby cooling the air down;
- The heated up water now flows back to fridge (Figure 40) or ice plants, where it is cooled down again for re-use.

Figure 41: Underground cooling car (heat exchanger)

4.4 AUDITING FOR BEST PRACTICE

LEARNING OUTCOME 4.4.1

DESCRIBE AND EXPLAIN GEOTECHNICAL INVESTIGATION / DESIGN / MONITORING / EVALUATION LOOP PROCESSES

The geotechnical investigation / design / monitoring / evaluation process was designed to 'close the loop' to effectively attempt to prevent accidents from occurring underground. 'Close the loop' refers to a process where the geotechnical investigation starts by gathering data to evaluate an issue and go through the whole process to derive solutions, implement and monitor the results to ensure that the issue has been sufficiently addressed. This process can change in detail content from investigation to investigation, but in essence the major parts of the process remain the same as the generic process given in Figure 41.

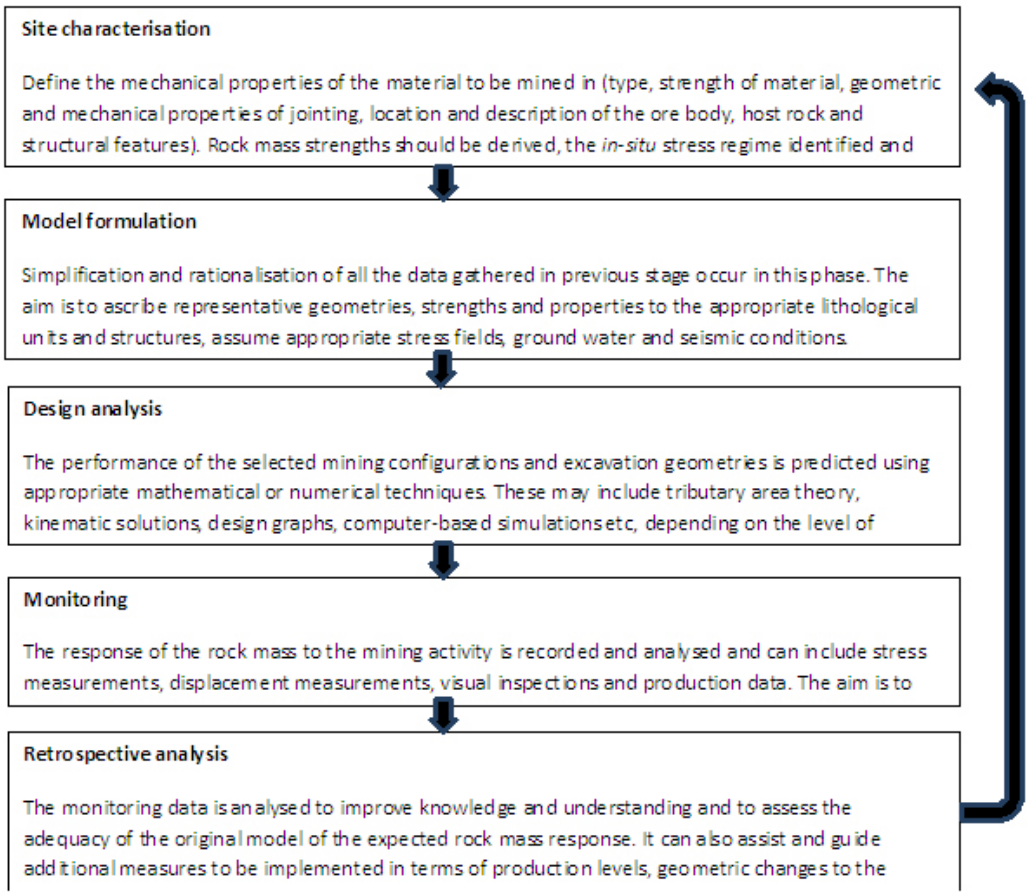


Figure 42: Showing the geotechnical investigation flowchart (Brady and Brown, 2006)

REFERENCE

- Page 13, Brady B.H.G., Brown E.T., Rock mechanics for underground mining, 2006.

LEARNING OUTCOME 4.4.2

DESCRIBE AND EXPLAIN THE PROCESS OF GEOTECHNICAL HAZARD AND RISK EVALUATION AND ASSESSMENT

CONNECTION 21

"Chapter 8" on page 572" deals comprehensively with risk assessments and the process and will therefore not be described in full in this chapter.

The Mine Health and Safety Act requires that a rock engineering risk assessment be carried out to identify significant risks. The primary purpose of identifying geotechnical hazards and assessing geotechnical risks is to eliminate and/or reduce these hazards and risks, and thereby to prevent accidents and thus ensure the safety of employees while they are at work.

In the past, risk management in the mining industry was mainly focused on reducing financial risk. The Leon Commission recommended that the mining industry should also provide for hazard identification and risk assessment processes in the management of occupational health and safety. This recommendation was included as one of the corner-stones of the Mine Health and Safety Act No.29 of 1996.

CHAPTER

5

MINING SUPPORT AND MINING SUPPORT DESIGN

INTRODUCTION

Once all the different disciplines' requirements are catered for in the mine design, certain conditions may exist that require the excavations to be supported to maintain or ensure stability. This is an integral part of mine design and a thorough understanding of rock mass characteristics and rock mass behaviour as mining progresses is essential.

Similarly, a thorough knowledge and understanding of support materials, support units and support systems is required as the design of mining support comprises of selecting appropriate support systems that are capable of controlling the expected rock mass deformation sufficiently to maintain the stability of mine excavations with various purposes and life (time) requirements.

In practice, support system design is about:

- determining the support demand for each type of excavation in its specific rock mass and stress environment and
- finding / selecting a support system satisfying this demand in a cost-effective manner.

In deep level mines, support demand is strongly controlled by the high stress and seismically active environment (even though other factors are also important), while in shallower depth workings, the rock mass characteristics, excavation size, excavation shape and orientation become the major support demanding issues.

The performance of the designed system is strongly dependent upon the quality of the support units and the correct installation thereof and hence should receive the necessary attention by rock engineering and operating staff.

The following are addressed in this chapter:

- Support types and specifications
- Support installation methods and procedures
- Backfill systems

LEARNING OUTCOMES

5.1. SUPPORT TYPES AND SPECIFICATIONS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

5.1.1 Describe and explain the concept of active support

5.1.2 Describe and explain the concept of passive support

5.1.3 Describe and explain the differences between active and passive support

5.1.4 Describe and explain the functions of skin support

5.1.5 Describe and explain the functions of areal support

5.1.6 Describe and explain the principles of rock reinforcement

5.1.7 Describe, characterise and discuss the following tunnel support types:

- Mechanically anchored bolts, Cable bolts, Friction bolts (split sets, hydro-bolts)
- Cement grouted bolts, Resin bonded bolts
- Full-column grouted/bonded bolts
- Yielding bolts (cone bolts)
- Multiple rod bolts, Multiple strand cable anchors
- Pre-stressed tendons
- Steel arches, Massive concrete linings
- Shotcrete, Gunite, Thin sprayed linings
- Wire mesh, Rope lacing, Tendon straps

5.1.8 Characterise the following aspects of these support types:

- Their principles of operation
- Their technical specifications
- Their load-deformation characteristics
- The methods of ensuring support unit quality
- Their installation/application procedures
- The methods of ensuring their installed quality

5.1.9 Determine appropriate support types, support sizes and support patterns given tunnel layouts, rockmass conditions and stress regimes

5.1.10 Describe, characterise and discuss specifications for the variety of support types

5.1.11 Describe, characterise and discuss specifications for the variety of support sizes

5.1.12 Describe, characterise and discuss specifications for the variety of support patterns

5.1.13 Describe, characterise and discuss quality control requirements for support units from fabrication to installation.

5.2. SUPPORT INSTALLATION METHODS AND PROCEDURES

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

5.2.1 Sketch, describe and explain the methods and procedures for installing common excavation support units

5.2.2 Determine appropriate support installation methods and procedures for given tunnel layouts, rockmass conditions and stress regimes

5.2.3 Describe, characterise and discuss specifications for the variety of support installation procedures

5.2.4 5.2.4Describe, characterise and discuss quality control requirements for support installation methods and procedures.

5.3. BACKFILL SYSTEMS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

5.3.1 Describe and explain the functions of backfill as local support

5.3.2 Describe and explain the functions of backfill as regional support

5.3.3 Sketch, describe and explain the different backfill preparation methods and preparation systems for:

- Full plant tailings fill
- Classified tailings fill
- Paste fill

5.3.4 Describe and explain the uses of cementitious binders in backfill

5.3.5 Sketch, describe and explain in principle how backfill is transported from preparation plant to underground stope

5.3.6 Sketch and describe the methods of containing backfill in stopes

5.3.7 Describe and explain the methods of controlling the quality of backfill

5.3.8 Sketch, describe and explain the features of backfill systems to fill particular blocks of ground.

LEARNING OUTCOME 5.1.1

DESCRIBE AND EXPLAIN THE CONCEPT OF ACTIVE SUPPORT

Support capable of exerting a positive support force 'upon installation' (Figure 1: Dotted line) is considered to be active support. Examples are tensioned end-anchored rock bolts, pre-stressed cable bolts and pre-stressed elongates.

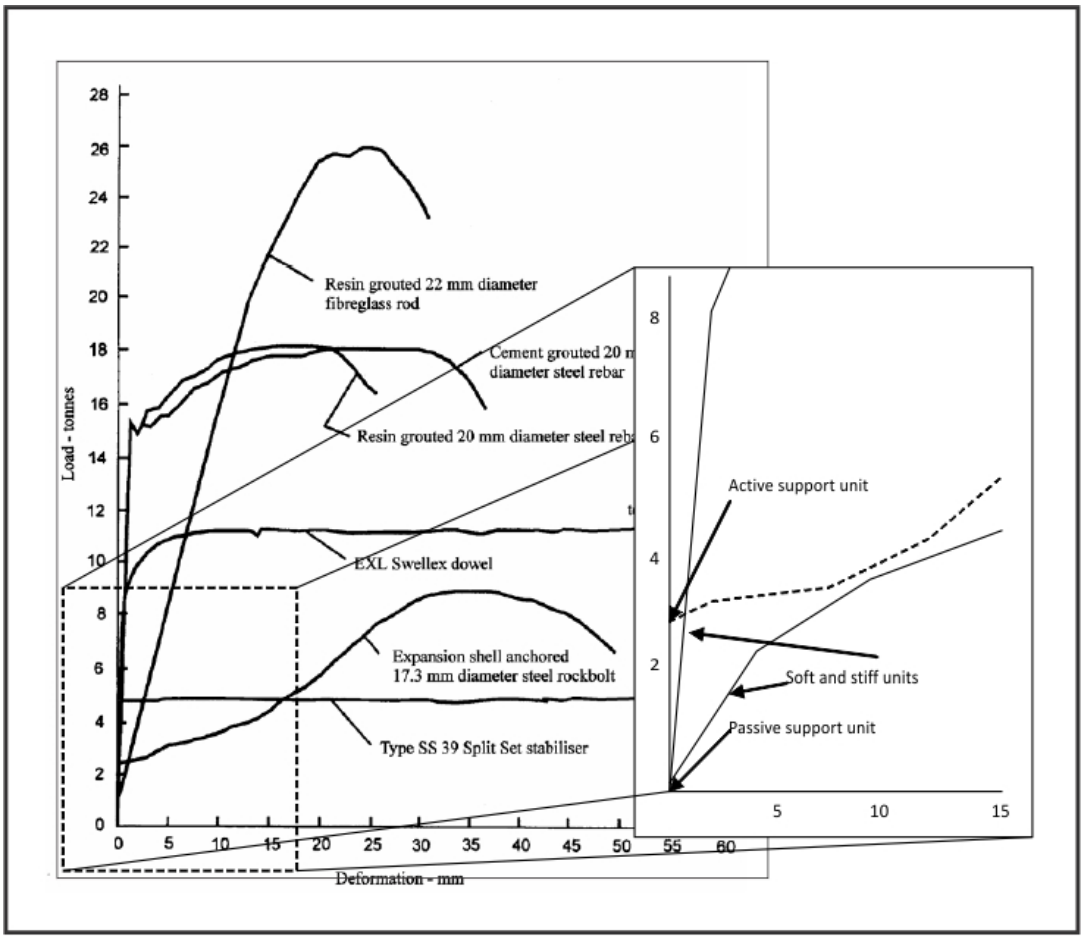


Figure 1: Load (Force) – deformation curves for tendon support units

LEARNING OUTCOME 5.1.2

DESCRIBE AND EXPLAIN THE CONCEPT OF PASSIVE SUPPORT

Support units and/or systems requiring rock deformation to generate support reaction forces, such as stope closure and tunnel wall dilation, are considered to be passive. Examples are grouted tendons (Figure 1: Solid line), mat packs and wire mesh.

LEARNING OUTCOME 5.1.3

DESCRIBE AND EXPLAIN THE DIFFERENCES BETWEEN ACTIVE AND PASSIVE SUPPORT

CONNECTION 22

Refer to "LEARNING OUTCOME 5.1.1" and "LEARNING OUTCOME 5.1.2" on page 270

Active support applies to support elements that exert a positive thrust against the rock surface immediately on installation and requires no movement of the rock to generate the support reaction. Active support is used where rock fragments are to be kept in place by the immediate support force exerted by the unit. It is important to note that pre-stressing of the unit takes up some of its available yield range and that any additional deformation could result in premature failure of the unit if this was not taken into account in the design.

In contrast, passive support elements are brought into contact with the rock surface with little or no positive thrust against that surface. The support will only begin to function after the rock has moved and some reaction has been induced in the support unit. Passive support units permit enough movement that continued development of fracturing in the sidewalls can proceed without restraint and the units then act to control the displacing rock and maintain integrity. Passive support can have a low or high initial stiffness that allows early force generation in the support unit (stiff) or slow force generation (soft) when rock movement takes place (Figure 1).

REFERENCE

- Page 87 Budavari (et al), Rock Mechanics in Mining Practice, 1983

LEARNING OUTCOME 5.1.4

DESCRIBE AND EXPLAIN THE FUNCTIONS OF SKIN SUPPORT

Skin support comprises of material applied to the skin of the rock surface, covering all of it. The purpose of it is to:

- strengthen the skin of the rock mass by bonding with the rock;
- penetrate into cracks and inhibit block movement, thereby securing fragments of rock that may otherwise dislodge;
- assist to reduce weathering.

NOTE

Examples include thin sprayed liners and relatively thin layers of shotcrete.

- *Thick layers of shotcrete, of sufficient flexural strength coupled with systematic rock bolting can be considered structural support.*
- *Wire mesh and rope lacing are generally referred to as containment support as they form no bond with the rock strata but merely contain rock fragments between support units such as rock studs.*
- *Collectively, skin support and containment support are often referred to as fabric support.*

Figure 2: Thin sprayed liner being applied underground (courtesy Meyco)

Shotcrete is probably the most widely used skin support in the South African mining industry, but thin skin liners are fast making progress to become a very useful skin support product.

An important area of shotcrete application in underground mining is in the support of 'permanent' openings such as ramps, haulages, shaft stations and crusher chambers. Rehabilitation of conventional rockbolt and mesh support can be very disruptive and expensive. Increasing numbers of these excavations are being shotcreted immediately after excavation. The incorporation of steel fibre reinforcement into the shotcrete is an important factor in this escalating use, since it minimises the labour intensive process of mesh installation.

INTERESTING INFO

Dry mix shotcrete: The dry shotcrete components, which may be slightly pre-dampened to reduce dust, are fed into a hopper with continuous agitation. Compressed air is introduced through a rotating barrel or feed bowl to convey the materials in a continuous stream through the delivery hose. Water is added to the mix at the nozzle. Gunite, a proprietary name for dry-sprayed mortar used in the early1900s, has fallen into disuse in favour of the more general term shotcrete.

Wet mix shotcrete: Shotcrete components and the water are mixed (usually in a truck mounted mixer) before delivery into a positive displacement pumping unit, which then delivers the mix hydraulically to the nozzle where air is added to project the material onto the rock surface.

The final product of either the dry or wet shotcrete process is very similar. The dry mix system tends to be more widely used in mining, because of inaccessibility for large transit mix trucks and because it generally uses smaller and more compact equipment. This can be moved around relatively easily in an underground mining environment. The wet mix system is ideal for high production applications in mining and civil engineering, where a deep shaft or long tunnel is being driven and where access allows the application equipment and delivery trucks to operate on a more or less continuous basis. Decisions to use the dry or wet mix shotcrete process are usually made on a site-by-site basis.

back

Fibre reinforced shotcrete: Of the many developments in shotcrete technology in recent years, two of the most significant were the introduction of silica fume, used as a cementitious admixture and steel or polypropylene fibre reinforcement. The result is an extremely strong, impermeable and durable shotcrete. Other benefits include reduced rebound, improved flexural strength, improved bond with the rockmass and the ability to place layers of up to 200mm thick in a single pass because of the shotcrete 'stickiness'.

Steel fibre reinforced shotcrete was introduced in the 1970s and has since gained world-wide acceptance as a replacement for traditional wire mesh reinforced plain shotcrete. The main role that reinforcement plays in shotcrete is to impart ductility to an otherwise brittle material.
Mesh reinforced shotcrete: While steel fibre reinforced shotcrete has been widely accepted in both civil and mining engineering, mesh reinforced shotcrete is still widely used and is preferred in some applications. In very poor quality, loose rock masses, where adhesion of the shotcrete to the rock surface is poor, the mesh provides a significant amount of reinforcement, even without shotcrete. Weld mesh is frequently used to stabilise the surface or to provide reinforcement. In such cases, plain shotcrete is applied later to provide additional support and to protect the mesh against corrosion. Chain link mesh, used in many underground mining excavations to support loose rock, is not usually suitable for shotcrete reinforcement. This is because penetration of the shotcrete is inhibited by the twisted joints.

A summary of recommended shotcrete applications in underground mining, for different rock mass conditions, is noted in the table below.

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.

back

back

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.
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, con glomerates 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.

Table 1: Summary of recommended shotcrete applications in underground mining (www.rocscience.com/hoek/pdf/16_Shotcrete_support.pdf)

back

INTERESTING INFO

Take note of all the issues being considered in the design of shotcrete for a specific application:

1. Wetcrete mix to be used due to the following reasons:
   - Far less rebound. A loss of 5-10% is normal with the use of the correct equipment and trained personnel;
   - Better working environment; dust problem reduced;
   - Thicker layers because of effective use of the admixing materials;
   - Controlled water dosage;
   - Improved bonding;
   - Higher compressive strength and very little variation in the results; and
   - Much larger production and consequently improved total economy.
2. Shotcrete classification

   a. Compressive strength (28 Day)

   Base mix, tested by cube. The strength classes are to be as in the table below.

Compressive strength class	C45
Minimum characteristic cube compressive strength, (MPa) for cast specimen.	35
Minimum compressive strength (MPa) for drilled cores, Height / diameter ratio 2:1	30

   It is important that the required compressive strength is achieved by the mix design that gives minimum shrinkage. There are two reasons for this:
   - Low shrinkage improves adhesion; and
   - Low shrinkage reduces cracking and improves durability.

   b. Energy absorption class

Toughness classification	Plate	RDP
II	600J (min)	280J (min)

   c. Bond (Adhesion)

   The following values shall apply:

Type of bond	Minimum bond strength to rock (MPa)
Structural	1.0

   d. Tensile strength

   The tensile strength with regard to this application is not deemed important.

   e. Shotcrete performance requirements (Summary)

Material	Test	Requirement
Shotcrete soon after application	• Penetrometer	0.5 MPa at 1.5 hours if specified
Hardened Shotcrete (test panels)	• Cores • EA (Plate/RDP)	30 MPa (Min) 600/280J (Min)
Hardened Shotcrete (In-situ)	• Cores	30 MPa (Min) 600/280J (Min) and check minimum thickness / bond
Bond	• In-situ and Cores	1.0 MPa

f. Mix design

Example of mix designs from various applications

Mixtures		MOAB Khotsong		Midmar tunnel		Perserverence Mine, Australia	
Product	Group	Quantity (Kg/m³)	Percentage (%)	Quantity (Kg/m³)	Percentage (%)	Quantity (Kg/m³)	Percentage (%)
Pit Sand	Fine aggregate	320	9.72	320	13.76		0.00
Crusher sand	Fine aggregate	905	27.49	865	37.19	400	16.92
Stone (5-10mm)	Coarse aggregate	1400	42.52	470	20.21	1300	54.98
Cement	Concrete base	450	13.67	450	19.35	400	16.92
Silica fume	Hydration	40	1.21	50	2.15	40	1.69
TCC 735	Curing agent	5	0.15		0.00	1.2	0.05
TCC 766	Activator	15	0.46		0.00	3.2	0.14
Water	Pumpability	157.5	4.78	171	7.35	220	9.30
Water/Cement ratio		0.35		0.38		0.55	
TOTAL		3292.5	100	2326	100	2364.4	100

In most cases, the sprayed concrete must meet a 35MPa strength class according to a normal national standard test procedure. In other countries, tunnels and roadways this requirement is now 45 MPa.

g. Application thickness

The thickness of the concrete layer is determined by the application being designed for. Shotcrete layers of greater than 30mm give more than 30MPa compressive strengths and thicker than 100mm give in excess of 35MPa.

h. Reinforcement

The traditional reinforcement in sprayed concrete is steel wire (welded) mesh (normally 3 to 6kg/m² and square openings of 100 to 150mm).

i. Fibres

The main reason for using steel fibres in sprayed concrete is to increase the ductility of the material. As a secondary effect, steel fibres improve the final flexural strength of sprayed concrete. Due to the installation of mesh, fibres will not be necessary in the shotcrete design.

LEARNING OUTCOME 5.1.5

DESCRIBE AND EXPLAIN THE FUNCTIONS OF AREAL SUPPORT

In general, stope support design is based upon providing, with a factor of safety, sufficient support resistance to prevent rock fall out up to the height of the most likely horizon where hangingwall separation may occur, such as weak bedding contacts.

NOTE

Usually fall out heights on operating mines are determined for each geotechnical area by means of recording of such over a meaningful period of time; in the absence of such records, weak geological horizons are identified in the stratigraphic hangingwall column.

In seismically active mines, support systems are also required to have some energy absorbing capacity. Stope support units are selected and spaced to provide the calculated support resistance and, if necessary, a calculated energy absorbing capacity. In the 1980's and 1990's Roberts used fallout thickness and ejection thickness from fatalities to determine support resistance criteria and energy absorption criteria for the design

back

of stope support systems. This provided robust criteria that support systems are required to meet.

In spite of these design criteria being met, it is possible that in stress-fractured and/or jointed rock, smaller rock fragments may dislodge if the support unit spacing does not provide sufficient interaction between them and it is often necessary to space units closer and/or to choose units with higher direct contact areas with the rock surface. It is then said that increased 'areal coverage' is required with the basic measure being the percentage that the contact area of the support units makes to the total area of hangingwall to be supported.

In stopes, backfill is an excellent back-area areal support system with areal coverage ideally in excess of 60%.

In the case of tunnels, the potentially unstable rock between tendons needs to be contained by areal coverage support systems such as rope lacing, wire mesh (Figure 3) and/or shotcrete.

Figure 3: Welded and chain link/diamond wire mesh

INTERESTING INFO

- Wire mesh, reinforced with wire rope lacing, is widely used as a method to control fractured rock walls of tunnels. Combined with grouted tendons, it becomes an effective way of controlling the effects of changing stresses and rockbursts on excavation stability.
- Shotcrete (Gunite) typically consists of applying a concrete mixture to the rock walls by means of compressed air. Two types of shotcreting methods are available, namely dry shotcrete, where water is only added to the mixture at the application nozzle, and wet shotcrete, where the water is mixed with the material in the batch mixer before application.
- Shotcrete can be used for:
  - Consolidation of fractured ground before drilling support holes.
  - A support medium in low stress change environments.
  - Protection of the mesh and lace against corrosion.
- Timber and steel sets – In instances where sets are used because rock deterioration has advanced so far that drilling cannot be done, or where a dead weight situation occur (reef intersections), sets are installed. Where sets are installed, it is important that:
  - Sets are cribbed to the hangingwall to ensure contact.
  - Cribbing should be such that the weight is transferred onto the legs of the sets and not the crossbars.
  - Crowns (legs) should be clear (300-500mm) of the sidewalls to allow for some sidewall movement without jeopardising the stability of the sets.

back

REFERENCE

- Player JR (et al.) (2008) 6th International Symposium on Ground Support in Mining and Civil Engineering Construction
- Roberts MKC 'The Design of Stope Support in South African Gold and Platinum Mines.' PhD. Mining Engineering.1999. University of the Witwatersrand, South Africa.

LEARNING OUTCOME 5.1.6

DESCRIBE AND EXPLAIN THE PRINCIPLES OF ROCK REINFORCEMENT

CONNECTION 23

Refer to the slide show on Bolting Principles

Rock reinforcement can be divided into two categories, where the first refers to the reinforcement of broken (jointed and/or fractured) rock around excavations and the second is where laminated rock layers are reinforced.

Reinforcement of 'broken' material

Often, and in particular in deep level mining, the rock mass around tunnels is blocky or in a failed state where it is not possible to support it by tying it back to solid rock by means of tendons. In such cases, a tendon support system, usually consisting of tendons installed at close spacing and supplemented by fabric support, is required to maintain the geometric integrity of the rock mass and to redistribute the stresses for the rock mass to support itself.

The reinforcement acts to increase the shear resistance of the fracture surfaces or joints between blocks in this broken rock mass and a so-called 'reinforced shell' is created in the periphery of the excavation. This 'reinforced shell' is therefore strengthened by the support units and can therefore succeed in maintaining the integrity of the excavation walls at higher induced stress levels.

Reinforcement of laminated layers

The 'clamping' of laminated layers follows the same principle in that the tensioning of the support units exerts a normal force onto layer contacts, which increases the shear strength of these contact planes. For separation on the layer contacts to occur, shear displacement must be induced along the contact plane and it is this displacement that is prevented by the strengthening of the contact shear strength with support units.

Support tendons that consist of an end anchor, a rod or rope, a washer or nut are normally used to reinforce the rock (Figure 4). They are installed by anchoring the rod at the back of the hole with the end anchor and then using the nut and washer, the tendon is tensioned. This support is also referred to as 'active' support. Where scaling of the rock wall is expected, and where increased shear resistance is required, grouting of the installed tendon can be carried out.

back

INTERESTING INFO

A 'rock stud', by definition, is a specific tendon type with a nut that can be tensioned and where turning of the nut results in movement of the nut down the thread. A rock bolt is also a specific tendon type where the nut is fixed to the tendon and turning the nut results in the rotation of the whole tendon.
Often the terms 'tendon' and 'rock bolt' are used as if being exactly the same, but the term 'tendon' is a generic term that refers to all forms of support that can be installed 'into' a rock surface. The term 'rock bolt' is often used on the mines as a colloquial generic term or 'slang' for any tendon that may be used, but is in fact a specific tendon type, called the 'rock bolt'.

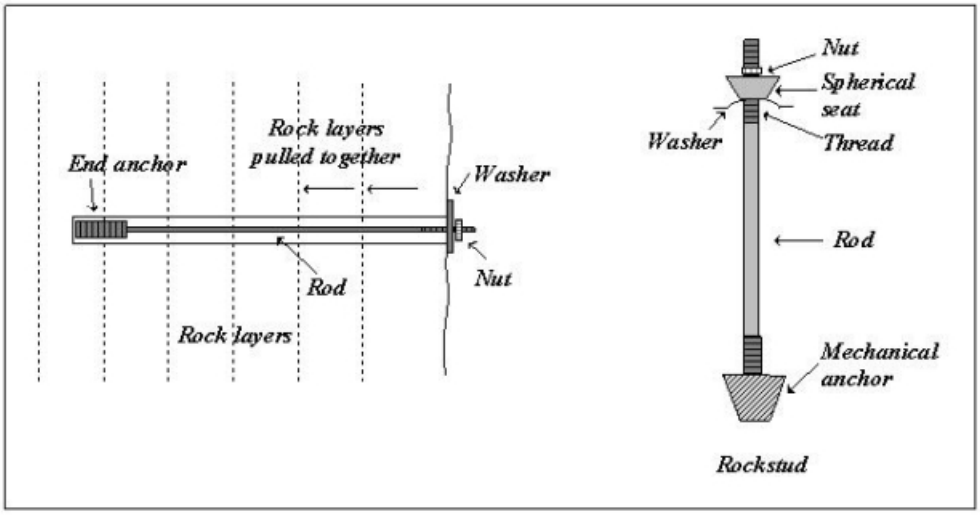


Figure 4: End anchored rock stud

REFERENCE

- Page 358, Ryder J.A. and Jager A.J., A textbook on rock mechanics for tabular hard rock mines, 2002
- Chapter 8, Hoek E, and Brown E.T., Underground excavations in rock, 1980
- Chapter 8 and Chapter 11, Brady H.G.B. and Brown E.T., Rock mechanics for underground mining, 2006

LEARNING OUTCOME 5.1.7

DESCRIBE, CHARACTERISE AND DISCUSS LISTED TUNNEL SUPPORT TYPES

The following tunnel support types will be discussed briefly in this outcome:

- Tendons
  - Mechanically anchored tendons
  - Cable bolts
  - Friction anchors
  - Grouted (cement and resin) tendons
- Skin support
  - Shotcrete
  - Thin skin liners
- Areal coverage
  - Mesh and lace
  - Straps
- Deadweight support units
  - Sets
  - Arches

NOTE

This list will not include all the available units and it is suggested that the different manufacturers are contacted for a more detailed list of products within the support types discussed.

back

back

- http://www.dsigroundsupport.com/products/plates-and-mesh/straps-mats.html
- http://www.duraset.com/welcome-to-aveng-manufacturing-duraset
- http://www.fabchem.co.za/index_files/Page582.htm

Tendons

Mechanically anchored bolts are constructed of solid steel bars fitted with expanding shells at the one end and a threaded portion of the bar at the other (Figure 5a). The expanding shell unit consists of a wedge surrounded by a number of steel "leaves" (Figure 5b). A nut, spherical seat and bearing plate are used at the threaded end to enable the bolt to be tensioned by tightening the nut, thereby pulling the wedge into the shell leaves of the expanding shell and pushing it against the end of the drill hole.

Because of the long free length of the steel tendon, mechanical anchor bolts can elongate when load is applied, hence it is considered to have some yield range. In cases where the rock tends to fritter around the bearing plate, such as in high stress conditions, it loses its tension and needs constant re-tensioning. The disadvantages of using mechanically anchored bolts under such conditions are overcome by fitting it with grout tubes and cement grouting the units post tension.

Due to the tendency to un-tension and its exposure to corrosion (especially if not grouted), it became unpopular and was largely replaced by fully grouted rock bolts such as "shepherd's crooks".

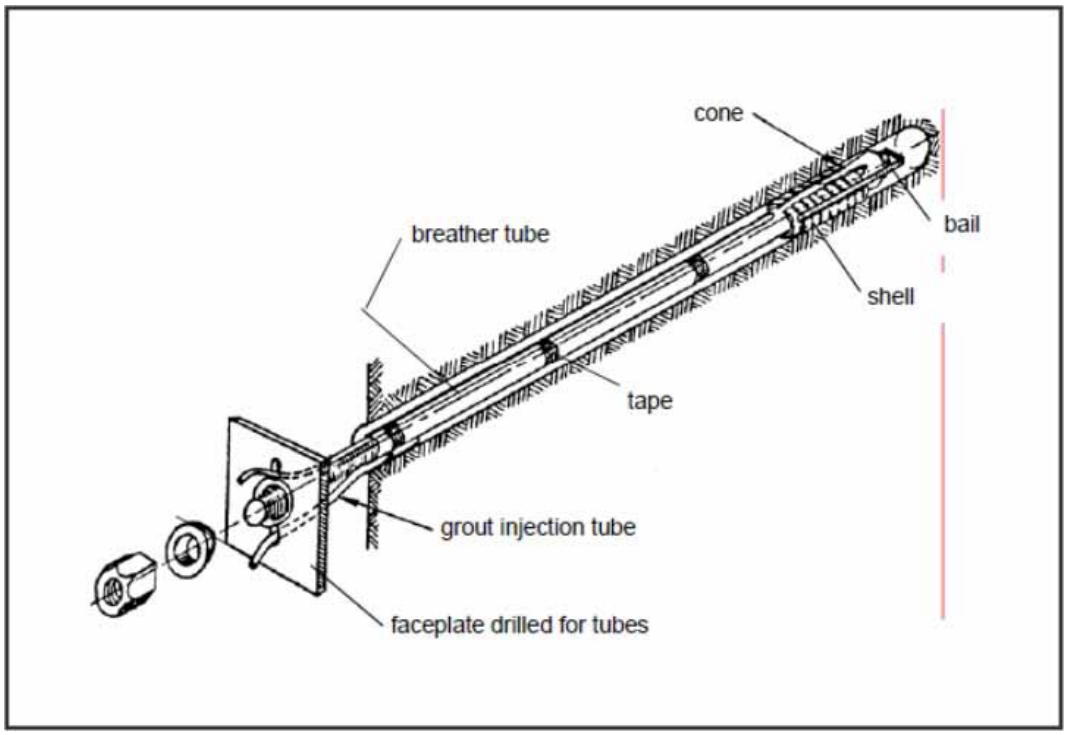


Figure 5(a): Mechanical end-anchor unit and range of (b) expansion shells (Hoek, 2006)

back

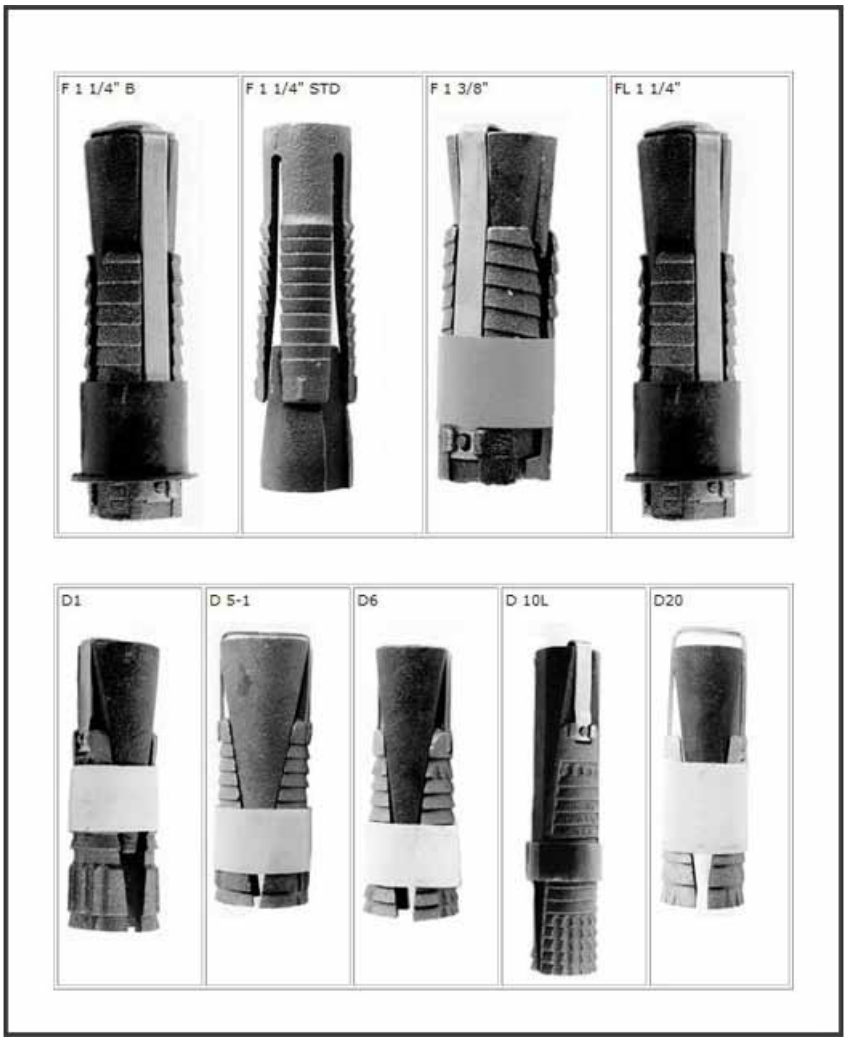


Figure 5(b): range of (b) expansion shells (Hoek, 2006)

Cable bolts are flexible units specifically constructed to be flexible to allow transportation of long support units for installation in large excavations. A unit usually consists of a steel strand or set of multiple strands (Table 2, a strand comprises a number of wires spun together). Cable bolts are used as full column grouted units (Figure 6). Often, the bolt is modified by constructing/creating bulbs, birdcages and nutcages at specific positions along the length of the unit to increase resistance against pulling through the grout (Table 2). The relatively short cable anchor units are usually fitted with an end-anchoring device (Figure 5b) and can therefore be pre-tensioned before grouting and are called cable anchors. Very long cable bolts are not fitted with end-anchoring devices, but are full column grouted.

NOTE

Strictly speaking, cable bolts should rather be referred to as cable tendons.

Occasionally, bolts similar to multiple strand cable bolts are constructed by using multiple rods in place of strands; they are less flexible but have the advantage of allowing less corrosion to occur, multiple wire strands expose more surface area to corrosion attack.

back

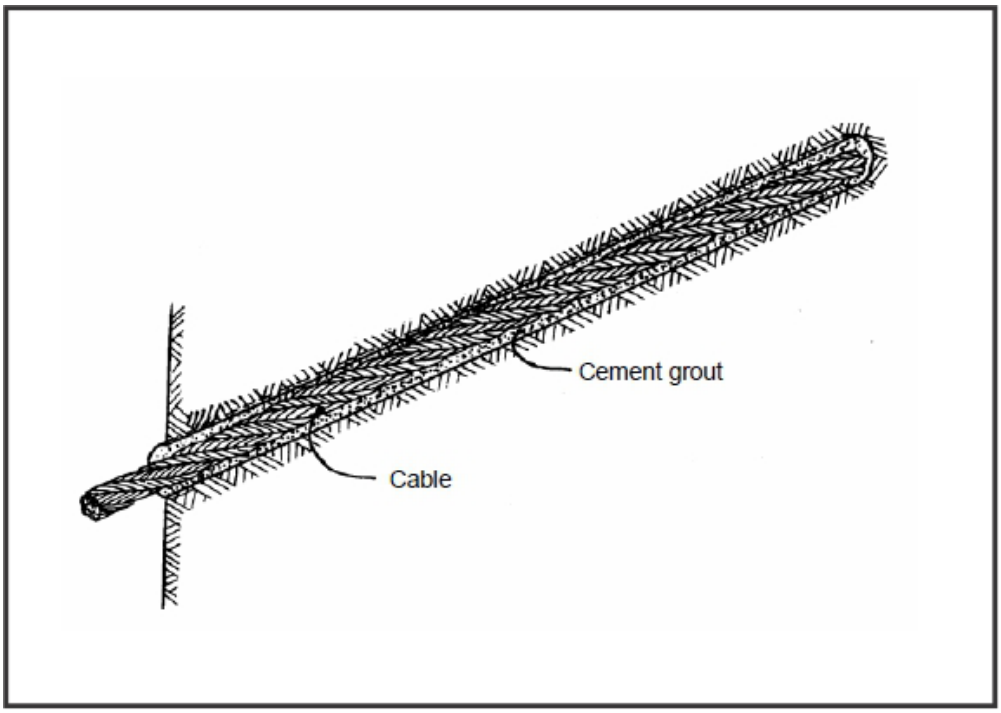


Figure 6: (a) Cable tendon unit and

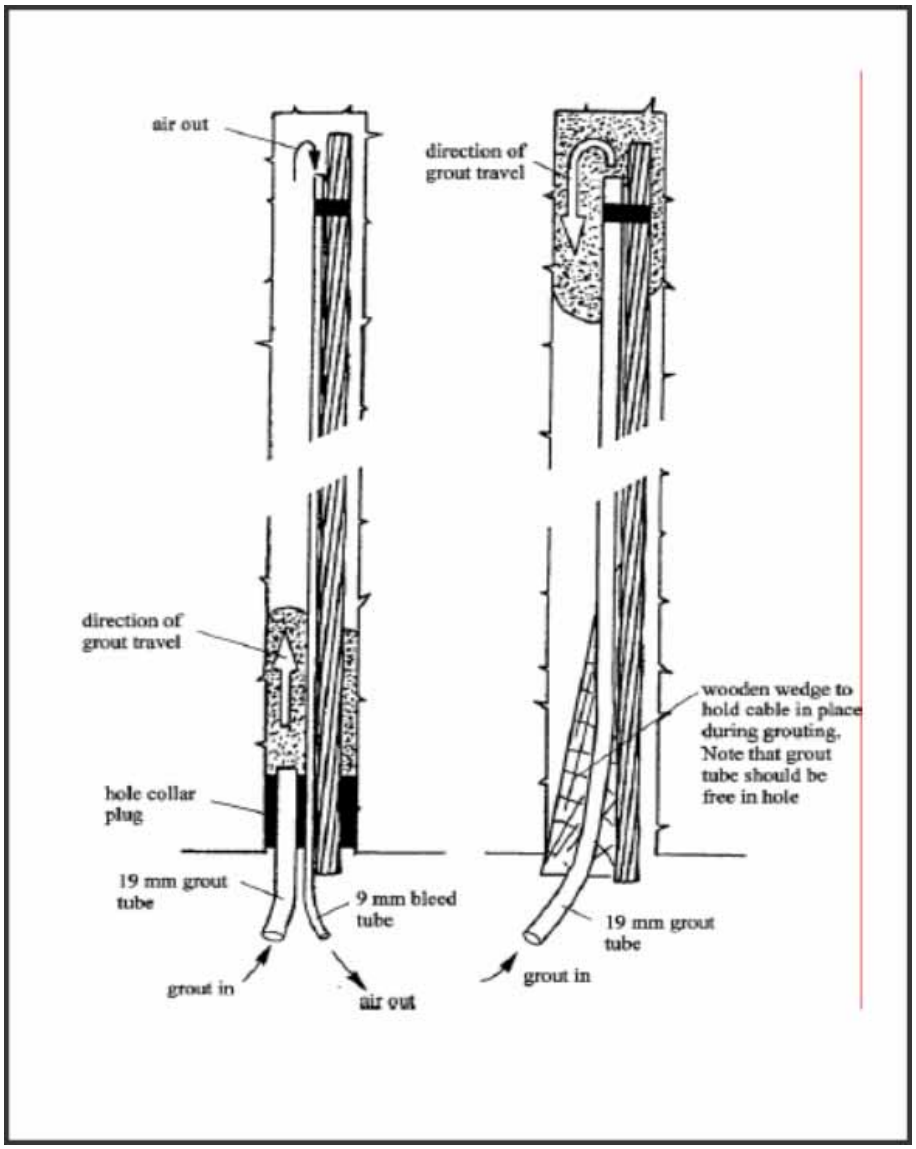


Figure 6: (b) grouting methods (Hoek, 2006)

back

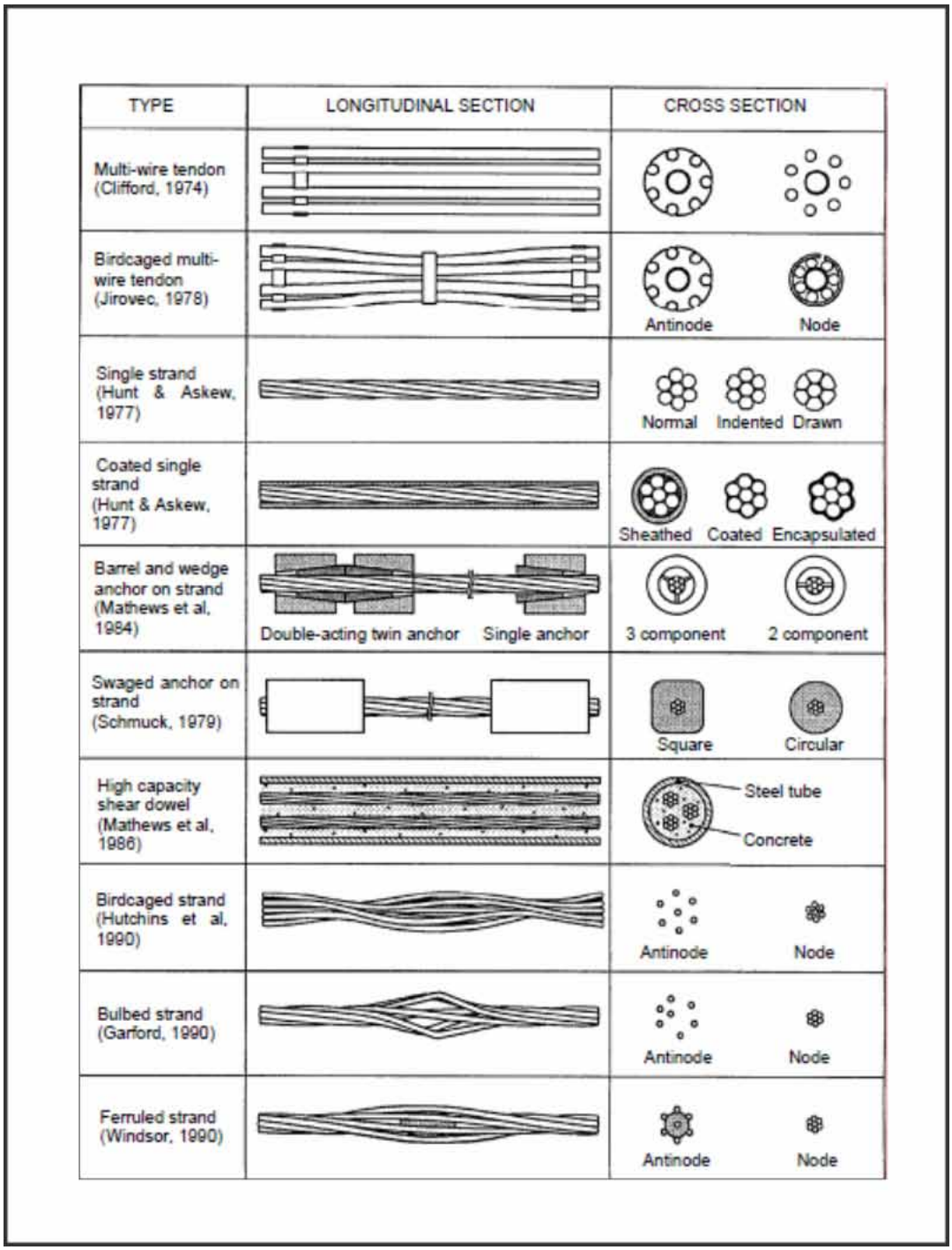


Table 2: Types of cables for cable support units (Hoek, 2006)

Friction anchors are tubular thin-walled units inserted in the support drill holes and where a radial force is applied against the rock, thereby generating a frictional resistance to sliding between the rock and the steel.

Two common types are available:

- Splitset: A split tube of slightly larger diameter than the drill hole is forced into the hole and where the spring action provides the radial force against the rock (Figure 7a),
- Swellex: A tubular construction is installed in the hole and water under pressure injected into the tube expanding it to make contact with the rock surface along the whole length of the hole (Figure 7b). Well-known products in South Africa that fall under this category are the Hydrabolt (water remains in the unit) and Xpandabolt (water is drained from the unit).

back

Friction anchors are popular with operating staff as they are easy to install particularly where mechanised mining is practiced and roof-bolters are used. However, concerns in the use of these units include corrosion, need for control of drill hole diameter and good quality installation practices.

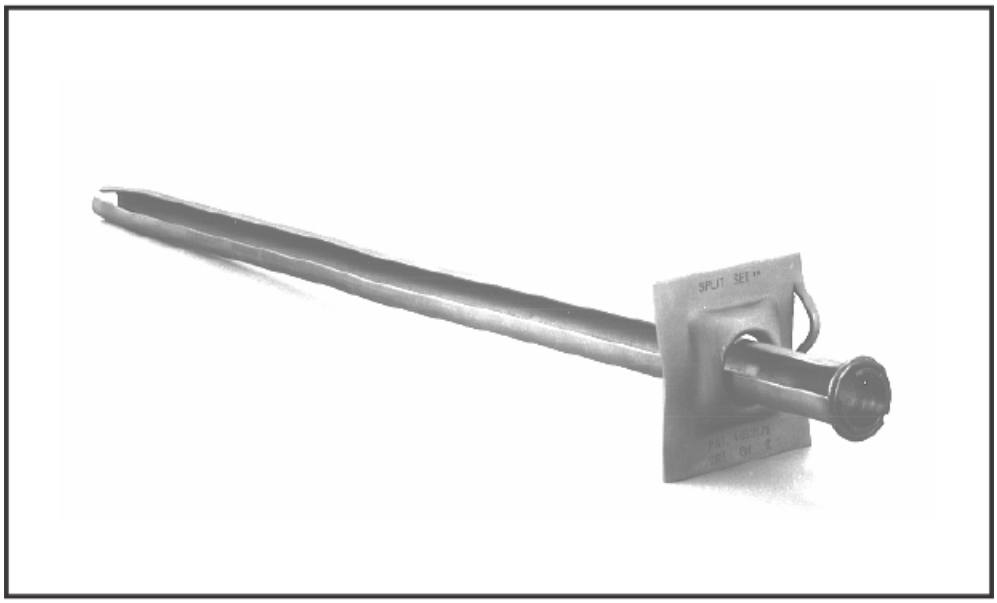


Figure 7(a): Splitset and

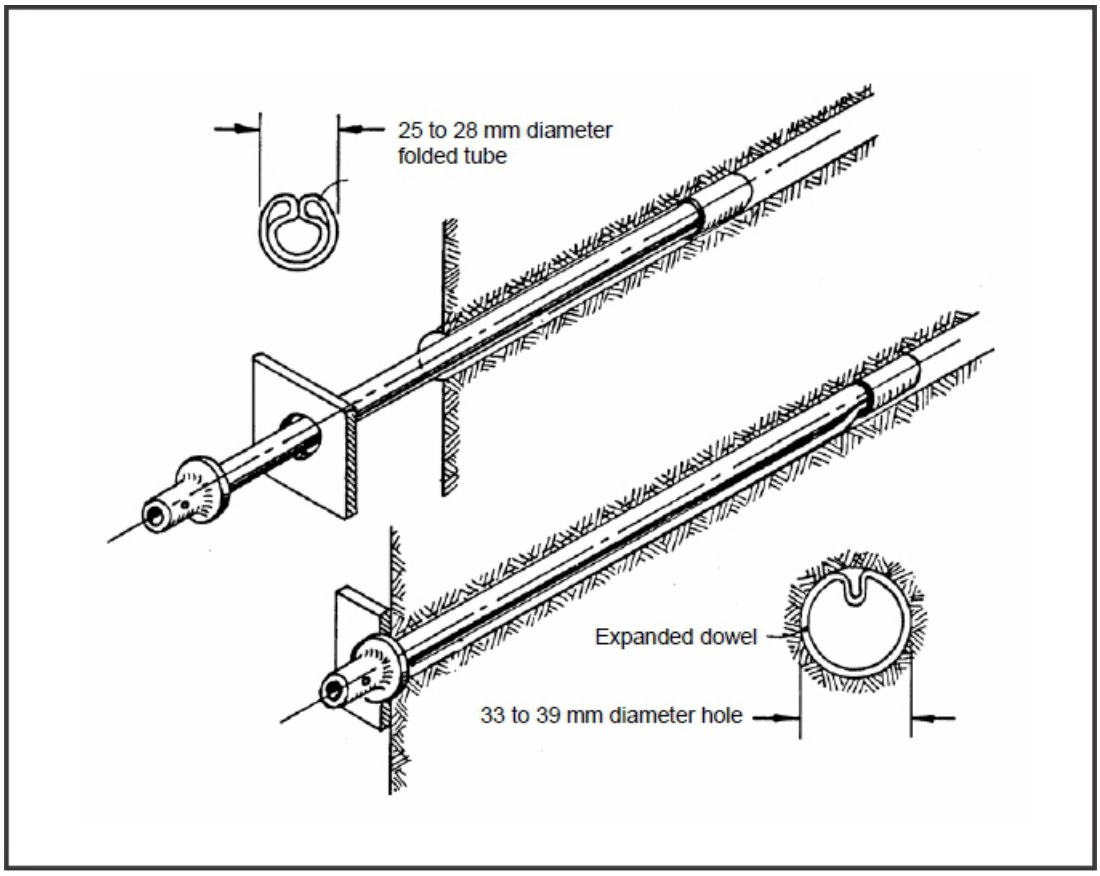


Figure 7(b): Swellex units (Hoek, 2006)

back

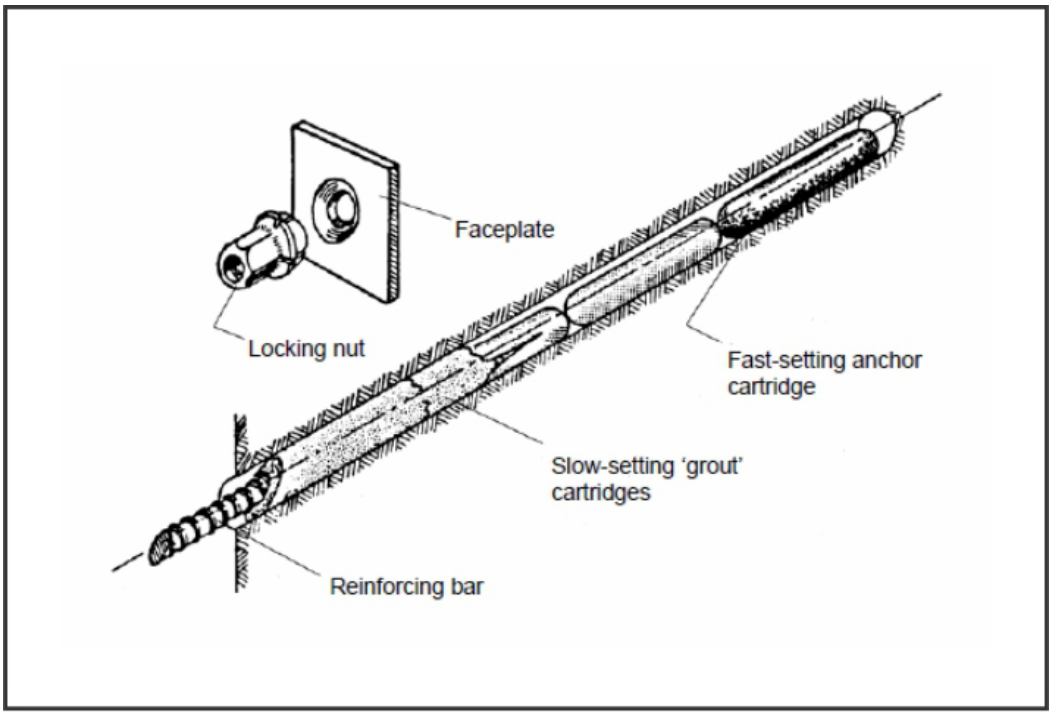


Figure 8: Fast setting resin used as 'anchor' to allow pre-stressing of the unit (Hoek, 2006)

INTERESTING INFO

Grout and resin can be used as end or point anchors instead of mechanical end anchors where bolts are required to be tensioned. Grout or resin capsules are installed in the back of a hole of correct length to ensure that a sufficient length of bolt is bonded to the rock surface of the drill hole to provide bond strength at least equal to the ultimate strength of the bolt. Once the grout or resin is set, the bolt is tensioned to its desired load. It is of course necessary to allow sufficient time for the grout or resin to set so as not to destroy the bond while the bolt is tensioned. Resin is often the favoured bonding agent since it can be manufactured to set at a faster rate. Where slipping of mechanical anchors occurs in "soft" rock, grout or resin anchors are usually used in preference to mechanical anchors, as the anchor length can easily be increased where required.

Full column bonded bolts are stiff units since they are bonded to the rock over the entire length of the hole, thereby allowing very little displacement to take place once they are installed before resisting displacements. Bolts can be bonded with either grout or resin. In deep level mines, full column grouted shepherd crooks (smooth bars) are used extensively, largely due to the "crooks" being convenient for lacing ropes through them as fabric support in conjunction with wire mesh.

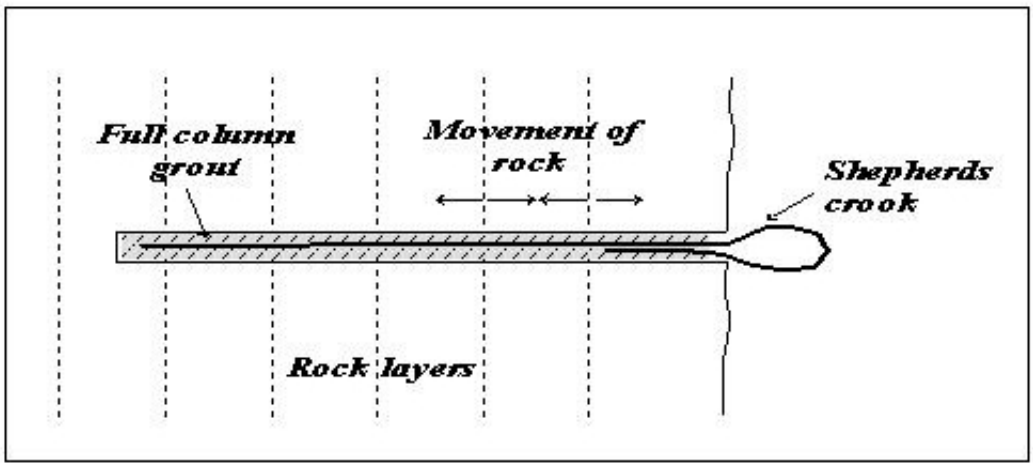


Figure 9: "Stiff passive" Shepherds crooks (Hoek, 2006)

Neither the mechanical end-anchored nor the full column grouted tendons have the capacity to yield without failure under dynamic loads imposed upon them when rock bursts occur. As a result, yielding rock bolts were developed, such as the cone bolt. The principal of construction and operation is that a steel cone attached to the back end of the bolt is allowed to be pulled through grout of a designed length and strength to allow for a specified yield distance while maintaining a specified load and thereby preventing the bolt to fail under dynamic loading. A specified front end length of the bolt is de-coupled from the grout by means of a de-bonding material.

back

INTERESTING INFO

The concept 'critical bond length' refers to that length of the tendon that must be grouted to ensure that the tendon will fail before the tendon / grout bond is broken. This concept can be mathematically described as follows:

Tensile strength (N) = area of grouted contact along tendon (m^2) x tendon / grout bond (Pa)

OR

Tensile strength = $(2\pi r)(L)$ x Bond strength

Where r = radius of the tendon (m), L is the grouted length of the tendon (m) and $(2\pi r)$ is therefore the circumference (m) of the tendon.

EXAMPLE

Determine the critical bond length for a 16mm tendon, when the tendon tensile strength is 120kN and the tendon / grout bond strength is 2 MPa.

120kN =
(2 x π x 0.016m)(L)(2MPa)

120 000 N =
0.1005 x L x 2 000 000Pa

L =
120 000 N / (0.1005
x 2 000 000Pa)
= 0.59m
= 59cm

The minimum or critical length of tendon that must be grouted is therefore 59cm (based on the selected input parameters).

Figure 10: Cone bolt (Aveng Manufacturing Duraset)

Another South African designed yielding bolt that provides support in underground excavations under stress and is prone to seismic activity is the 'Durabar'. It is an engineered yieldable steel tendon and comprises a steel bar with a designed length being distorted in a "wave" profile. The bar is coated with a de-bonding material and is grouted into the support hole. As the forces bearing on the rock bolt reach the designed yield load, the bolt pulls through the grout. The mechanical work done on the steel as it moves through the wave-path is the energy dissipation mechanism.

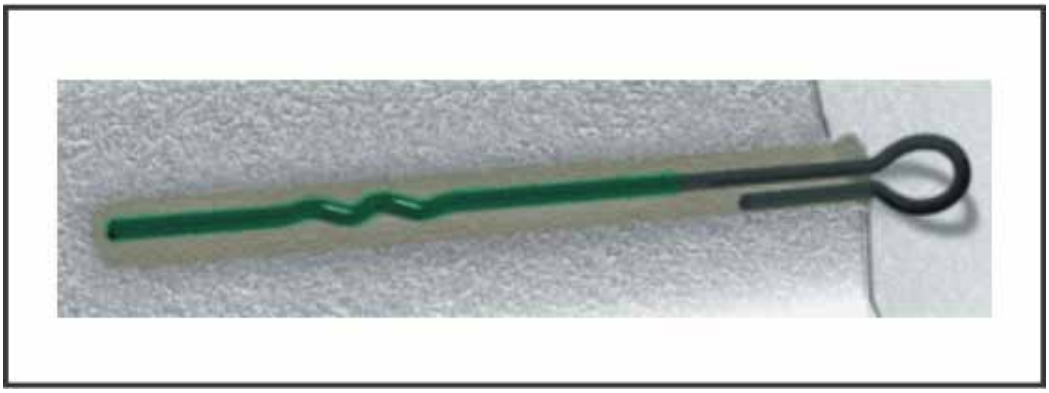


Figure 11: Durabar (Aveng Manufacturing Duraset)

REFERENCE

http://www.duraset.com/welcome-to-aveng-manufacturing-duraset

Skin support

Shotcrete is a mixture of cement, aggregate and water that is pumped pneumatically through a nozzle onto the walls of an excavation to form a bonded coherent layer and strengthens the skin of the rock mass. Shotcrete prevents or minimises rock displacement by filling large open joints and fractures, transferring the rock load to adjacent stable rock, and sealing the rock face to prevent ravelling and sloughing. It may contain admixtures, additives and fibres or a combination of these to improve tensile, flexural and shear stress resistance to cracking and increase energy absorption capability. It is usually applied as a relatively thin layer and the support mechanism is not of a structural arch nature.

INTERESTING INFO

Gunite differs from shotcrete only in that smaller size "aggregate" is used in the form of river or crusher sand.

Thin spray-on liners (TSLs) are polymer-based products applied to the rock with thickness that can be as low as 3 to 4mm. TSLs can be classified into reactive or non-reactive liners. Non-reactive TSLs, once applied to the excavation surface, gain strength over time as a result

back

back

of hydration. On the other hand, reactive liners gain strength rapidly due to an exothermic reaction brought on by a catalyst. Thin spray-on liners, sometimes referred to as super skins, have been developed in an attempt to provide replacements to the traditional forms of containment support. Thin spray-on linings have the advantage of low volume, rapid application and rapid curing. All forms of sprayed liner support are largely dependent on the bonding between the sprayed liner and the rock interface, but are also capable of penetrating cracks.

NOTE

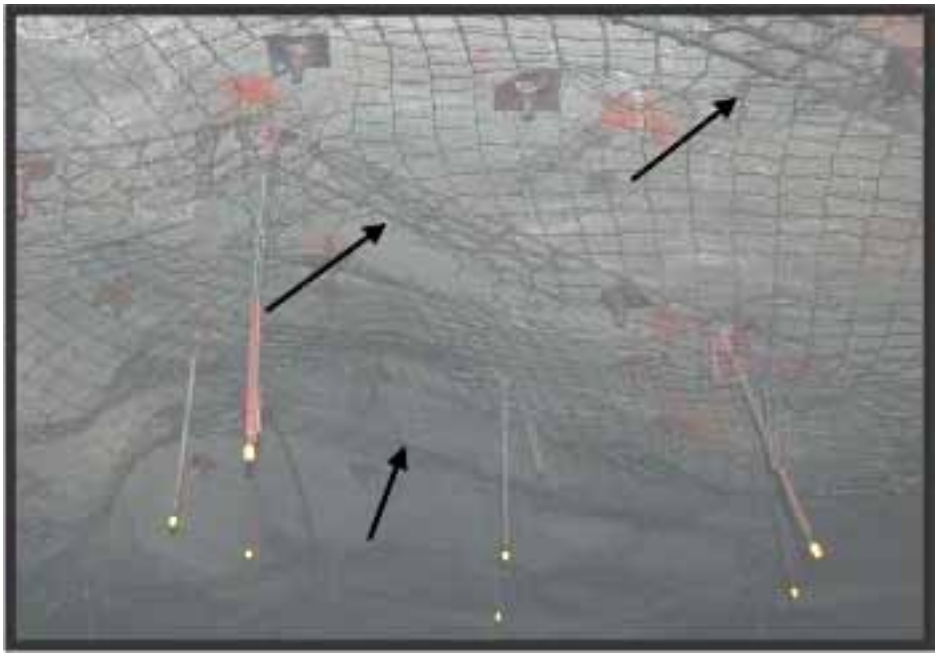
Concrete linings do not accommodate the stress-induced deformations in tunnels, and are therefore seldom used in modern mines. Concrete linings are still used in shafts, but are being replaced by reinforced shotcrete in most mine designs.

Areal coverage support

Tendon straps are rigid forms of fabric support, the best known of which is the OSRO (Figure 12a) and W-straps (Figure 12b), where the Osro strap comprises a square pattern of very large diameter mild steel "rods" welded together at their crossing points, and the W-strap a metal plate.

Osro straps are generally of 30cm width and come in various lengths and are bolted to the rock surface by tendons fitted with relatively large bearing plates to hold them in place.

In the coal mining industry, W-straps are used in very friable or highly jointed ground. They are steel plates of 30cm width and any length with an elongated "w" cross section. They are provided with holes to fit over roof bolts, and are bolted to the roof. Both of them provide good areal coverage, although not total as with wire mesh, and increase the support system stiffness.



back

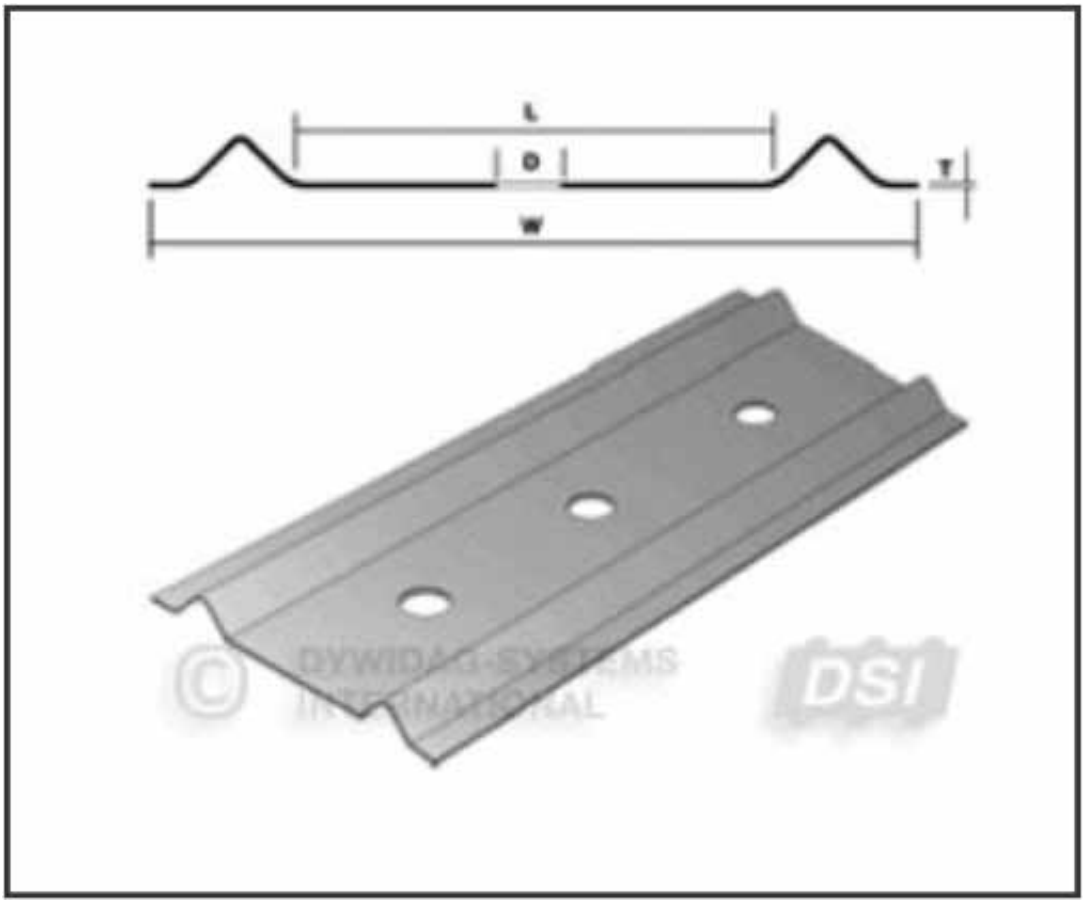


Figure 12(a):Osro straps (Fabchem) with mesh and (b) W-strap (DSI)

Wire mesh and lacing are used extensively as fabric support in deep-level tabular mines where the rock mass and stress environment are such that rock fragments can dislodge between bolts. The support system consists of tendon support installed to a certain pattern mesh that is placed against the rock surfaces and lacing (steel ropes) that is threaded through the tendons (loops are available on tendons) to keep the mesh in place. The mesh and lace keep small fragments of rock from scaling into excavations and causing injury, but provide its ultimate support during a rockburst. This support method has been hugely successful in maintaining integrity of excavations subjected to seismic activity, by containing the broken rock (Figure 13) and preventing total excavation closure. This success has been mainly due to the support system's ability to absorb large displacements without completely failing.

NOTE

Meshing and lacing are extremely susceptible to corrosion over time.

Figure 13: Meshing and lacing containing loose rock after a rockburst (Ortlepp, 1997)

Trusses

Trusses (Figure 14a) consist of two steel cables that are grouted into the rock at two points and then connected and tensioned using couplers such the 'dogbone' coupler (Figure 14b). These units are usually utilised to support brows or 'bullnoses' where rock is kept in place by the trusses in conjunction with mesh, but has also proven successful in supporting roadways in coal mines.

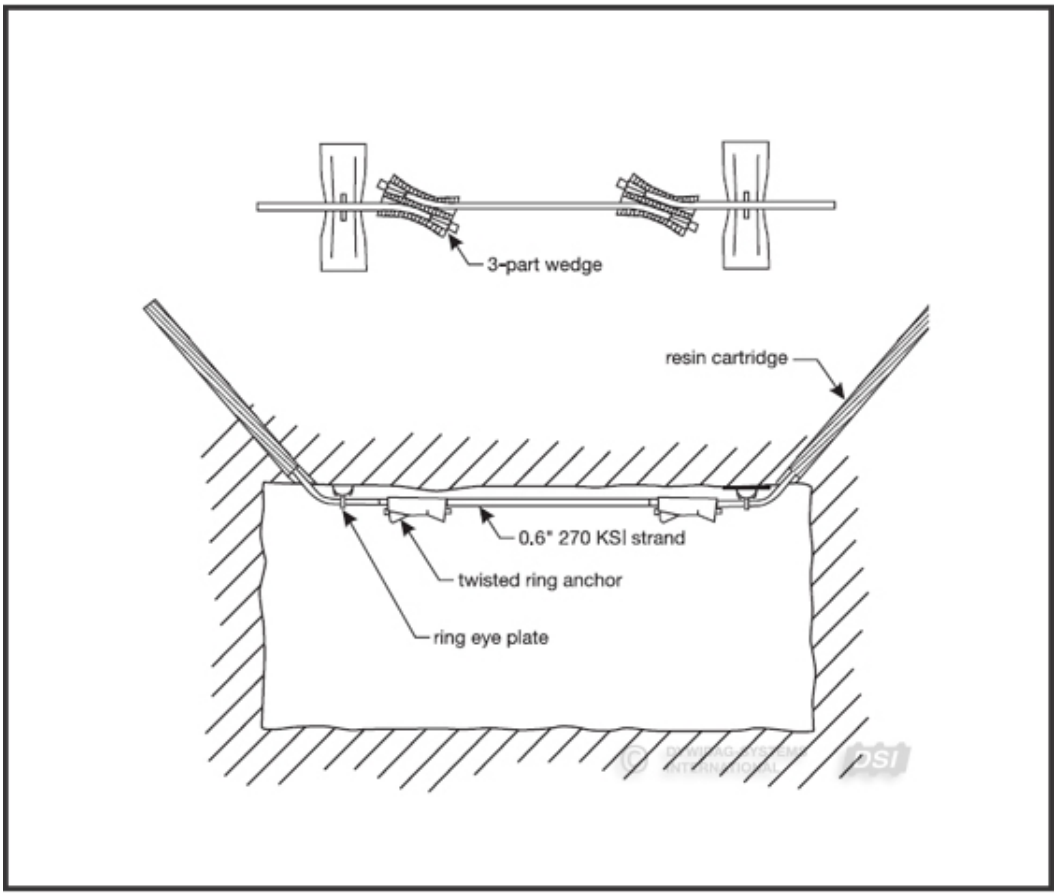


Figure 14 (a): Truss support units tensioned using

Figure 14 (b): 'dogbone' coupler

Deadweight support

Sets or arches are installed where rock conditions have deteriorated to such an extent that drilling cannot be done or where large amounts of rock (dead weight) need to be controlled (reef intersections). Using arches in traversing very weak rock layers such as clay and lamprophyre dykes (running) has proven successful.

Sets are usually constructed of timber or timber and steel combinations and consist of a structure with vertical legs and horizontal cross beams between legs in all directions along the tunnel. This support method is historically very common, but recent developments in other support practices have reduced the use of timber sets significantly.

NOTE

Timber sets in major ventilation in-take tunnels are a significant fire hazard and disastrous accidents have occurred in the past when these units have caught alight. It should be avoided as far as possible.

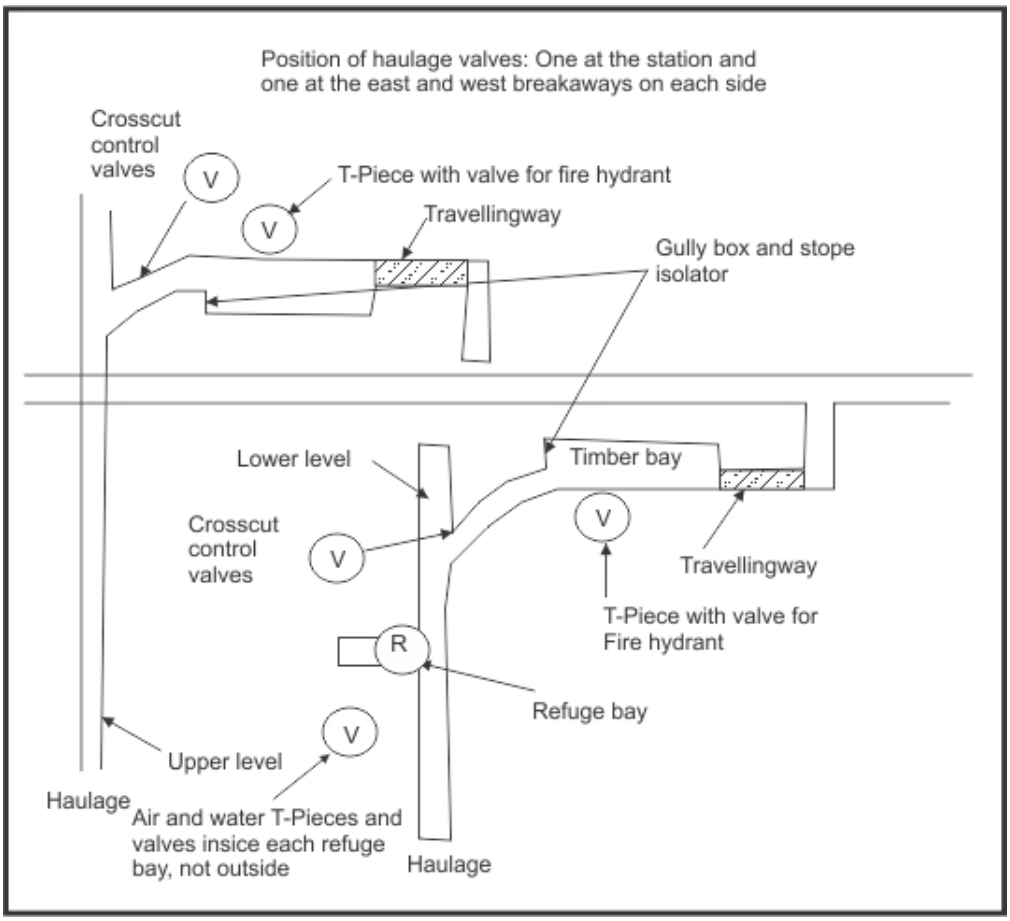


Figure 15: Timber sets with timber plank (gumplank) cribbing on hangingwall and sidewalls

Steel arches are largely used where high load-carrying capacity elements are required such as tunnels developed through running dykes or where tunnels are rehabilitated and ground conditions are such that it is not possible to drill into the rock mass to install tendon and fabric support. They usually consist of various shaped segments of profiled steel beams and when clamped together form a horse-shoe (Figure 16a) or circular (Figure 16b) shaped arch with the legs on the footwall. They can be rigidly bolted together or clamped together with a clamp joint with some yielding capability. Sometimes more rigidity is required in which case circular arches are constructed. Arches are installed at spacing determined by the load-carrying capacity of the arch and the tributary rock load each arch is required to carry. They are usually linked with steel bars and the remaining spaces filled with timber lagging, steel mesh or even

back

steel plates. However, since it is a form of passive support, some form of blocking is required to transmit loads from the rock to the steel set; foam cement or "void filler" is often pumped into the void to perform this function.

Figure 16: Steel arches installed in tunnels (a) horseshoe and (b) circular shaped

NOTE

When sets or arches are installed, it is important that:

- *Sets are cribbed to the hanging wall (using cement foam is a modern solution to ensure contact between the sets and the rock, and limits damage to the sets once rock displacements do occur).*
- *Cribbing above sets should be such that the weight is transferred onto the legs (the cross beams are usually not strong enough*

back

to support large amounts of rock). This concern has been negated by using arches as they are designed to distribute load throughout the steel construction, thereby limiting stress raisers at specific points in the structure.

- *Crowns, caps and legs should be clear (300-500mm) of the rock walls to allow for movement (if rock movement is expected).*

REFERENCE

- Ryder JA & Jager AJ (1999) A Handbook on Rock Mechanics for Tabular Hard Rock Mines SIMRAC Chapter 6
- Practical rock engineering. Hoek E, 2006
- (See Brady and Brown, Second edition, pages 319&320).
- These fabric supports are very well described and characterised on pages 362 and 363 in the: "Handbook on Rock Engineering Practice for Tabular Hard Rock Mines"; Editors: A.J. Jager & J.A. Ryder.

LEARNING OUTCOME 5.1.8

CHARACTERISE THE FOLLOWING ASPECTS OF THE COMMONLY USED SUPPORT TYPES DESCRIBED IN 5.1.7 ABOVE, AS APPLICABLE:

- principle of operation
- typical technical specifications
- load-deformation characteristics
- methods of ensuring support unit quality control
- installation/application procedure and
- methods ensuring their installed quality

CONNECTION 24

The principles of operation were dealt with in "LEARNING OUTCOME 5.1.7" on page 279

Due to the vast number of commonly used support products, only the following commonly used support units will be discussed in this outcome:

- Mechanical end-anchored rock stud
- Cable anchors
- Split sets
- Hydrabolts
- Shotcrete
- Mesh
- Resin
- Grout

NOTE

It is advised that specifications, characteristics, quality control, installation procedures etc for all the support units at a specific operation will be sourced by the relevant personnel that design the support system for the mine if the information is not given in this outcome.

Technical specifications

In general, rock engineers set performance specifications for a support system based on the support demand of the specific excavation to be supported, typically referring to information that is readily available from support unit load-deformation graphs and from which the following can be read:

- pre-loading (kN);
- yield load / ultimate strength (kN);
- stiffness (kN/mm); and
- yield range before failing (mm).

EXAMPLE

If it can be established that a support system should provide tendon support with:

- a 50kN active installation load
- no more than 100kN as the ultimate strength of the unit
- a very high degree of stiffness is not required and
- the support is required for a short period of time, then

the conclusion that a mechanical end-anchored tendon would suffice is probable and an appropriate bolt on offer by the suppliers would be sought.

Manufacturers use performance specifications to develop and manufacture support types of varying capacity to satisfy the varying demands of performance specifications from the large variations in conditions found in the mining industry. They translate the performance specification of each support type through the use of appropriate materials and manufacturing processes into products with technical specifications, ending in a vast number of available specifications. Due to this large amount of data available, it would be sufficient to provide some technical specifications of the most commonly used support elements.

Load deformation characteristics

The load-deformation graph of tendon support units that is usually inspected by an engineer, describes its behaviour when subjected to tensile forces along its axis. This graph indicates how the load (force) in the unit changes as it is subjected to tensile forces along its axis. These graphs do not, however, describe the unit's behaviour when subjected to shear forces acting across its diameter or when placed in axial compression. These behaviours should be investigated if it is suspected that units may behave in such a way that it affects the support system's ability to maintain excavation stability. In this outcome, only the tensile behaviour of tendons will be discussed.

Support unit quality control

Quality control of support units delivered to the mine is critical and substandard materials will behave in substandard ways and could make all efforts to ensure excavation stability by installing the support useless.

Installation procedure

Installation procedures vary from product to product, but very generic procedures will be discussed. It is the responsibility of the engineer to ensure that any product used on the mine is accompanied by training material that will include installation procedures.

Methods to ensure installation quality

Quality control of installed support units is also complex, but certain generic issues will be discussed in this outcome. It is the responsibility of the engineer to ensure that any product used on the mine is accompanied by training material that will include quality control practices required.

MECHANICAL END-ANCHORED ROCK STUD

Technical specifications

Expansion shells hold in any rock strata that are sufficiently competent to provide an adequate anchorage. They are designed to anchor in soft ground or hard rock. In good strata, the anchorage exceeds the ultimate strength of the steel bolt. All expansion shells require competent strata in the anchor zone. The suitability of the anchorage and the specific expansion shell to be used is best determined by physical load testing (Table 3).

DSI South Africa expansion shells feature parallel contact with the hole wall with sharp serrations on the outside leaves ensuring positive holding power (Figure 17).

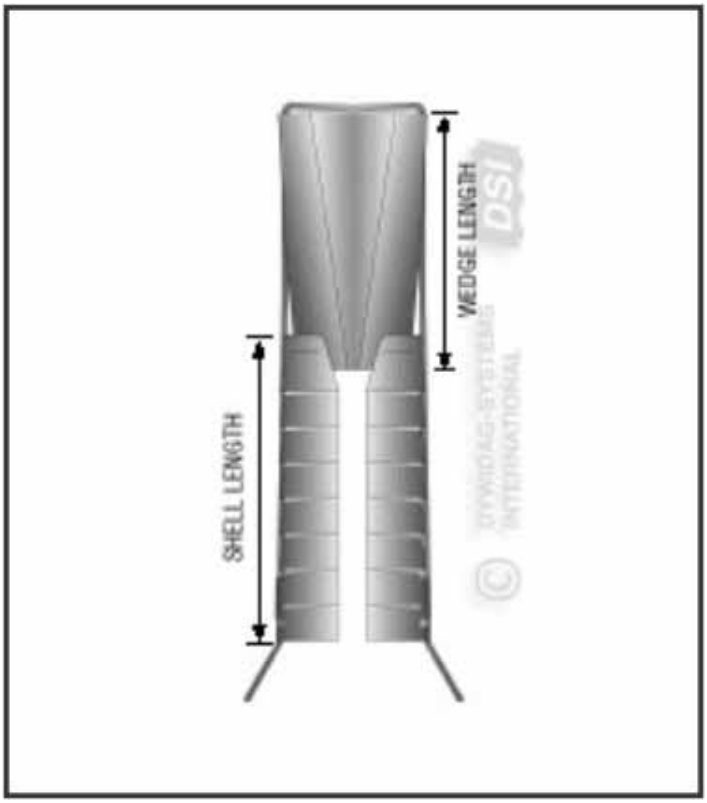


Figure 17: Expansion shell (rock studs) http://www.dywidag-sa.com/

The expansion shell assembly is securely held by a strong protective sleeve to save installation time and prevent loss of parts while handling. The sleeve must be removed prior to installation.

Physical Properties					
Bolt Diameter	Thread Form	Nominal Hole Diameter	Shell Length	Wedge Length	Net Weight
mm		mm	mm	mm	kg
16	DIM405-18mm	26	74.5	33.5	0.30
16	DIM405-18mm	30	74.5	33.5	0.37

Table 3: Physical properties of the rock stud (Courtesy of DSI Systems (Pty) Ltd)

CONNECTION 25

DSI products

http://www.dywidag-sa.com/products/

Load deformation characteristics

Load deformation graphs indicate the relationship between the load generated and the deformation undergone by a tendon. This relationship allows the comparison of different tendons by evaluating the following (refer to Figure 18):

- Pre-loading: Intersection of the graph with the y-axis;
- Stiffness: Slope of the initial parts of the graphs;
- Yield range: Deformation undergone until tendon fails;
- Yield load: Load at which tendon starts to yield;
- Yield point: Point at which tendon yielding commences (point at which slope of load-deformation graph reduces significantly)
- Post-yield behaviour: Type of behaviour (brittle, ductile, etc) after yielding commences.

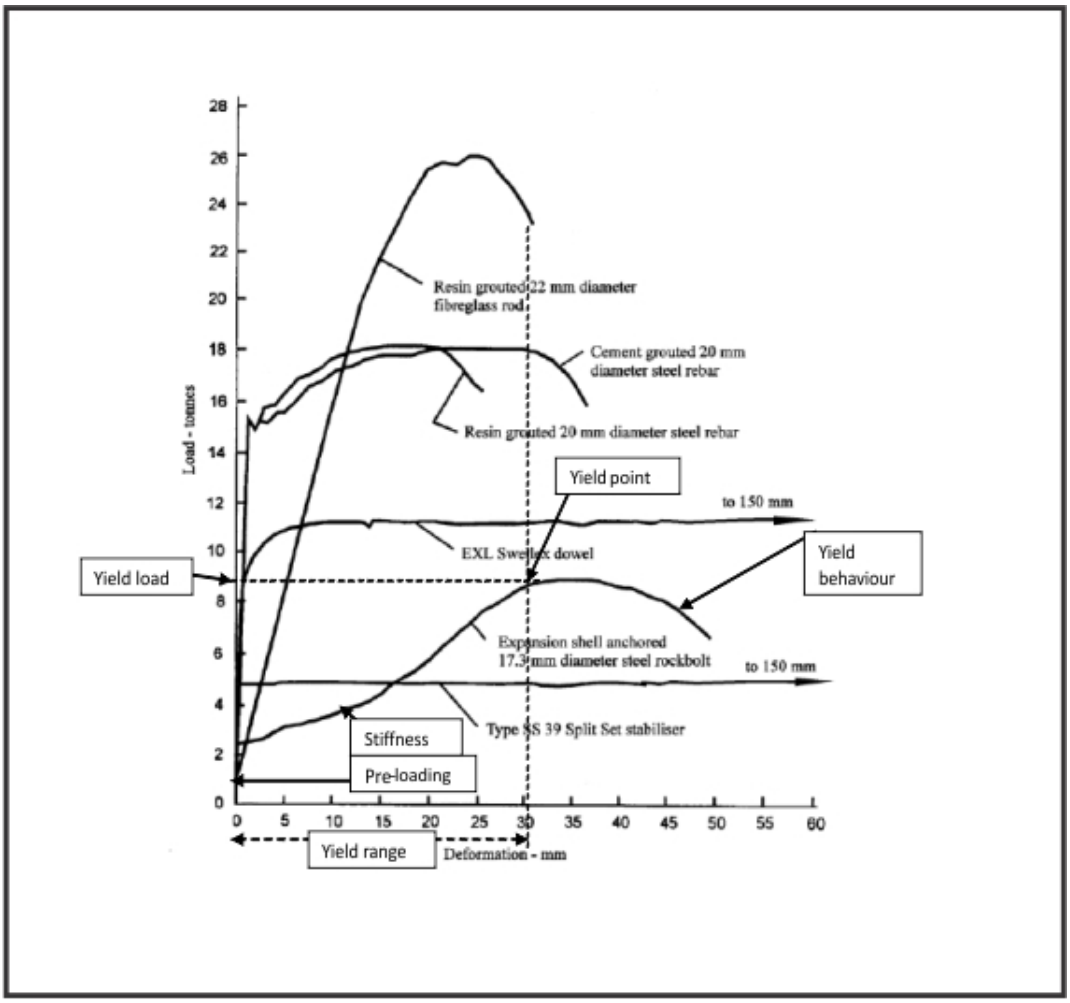


Figure 18: Load deformation graphs for tendons

Support unit quality control

Manufacturers test results on bar steel strength, expansion shell and face plate. Corrosion levels and changes (visual inspection) that require covered storage to protect units against corrosion.
Selection of samples by the mine to perform tensile strength tests in addition to that performed by the supplier.

Installation procedure

Drilling

Use standard drilling equipment and ensure an accurate hole diameter and a straight hole. The depth should be over drilled to ensure the bolt can be pre-stressed and not protrude too far. Clean the drill hole by blowing air to the full depth to remove debris. Efforts should be made to prepare the collar area with a flat surface and as perpendicular to the bolt axis as possible.

Bolt placement

Place the nut, cone and washer plate on the rock bolt and push the bolt into the hole to the correct embedment depth.

Setting the anchor

Torque the bolt to the required pre-loading with an impact gun, pneumatic or hydraulic torque wrench. This action migrates the cone into the shell, thereby expanding the mechanical anchor into the rock. Final torque can be checked and adjusted with a manual or hydraulic torque wrench. Prepare collar area with fast setting grout to ensure full bearing under the plate

back

Methods to ensure installation quality

Visual inspection to ensure that units are tensioned.
Conducting pull tests on a sample of installed units to ensure adequate friction between expansion shell and rock (no slippage).

CONNECTION 25

Refer to SIMRAC paper by Steffen et al "Dynamic Testing of Tunnel Support"

CABLE ANCHORS

Technical specifications

Flexible cable anchors are manufactured from high tensile steel strands and often incorporate a threaded live end used to pre-tension the anchor. The ultimate tensile strength of most cable anchors varies between 100 and 380kN, depending on the strand diameter (Figure 19 and Figure 20). Mechanical cable anchors are flexible high-tensile steel strands of specific lengths, anchored inside a drilled hole with an innovative locking-off unit at the collar. The advantage of using this locking-off device is that once the required pre-stressed load is achieved, the shear ring shears on the barrel into a positive visual indicator.
The cable anchors are made to requirements as specified by the end user. These anchors were designed as an immediate support and are normally post-grouted after being pre-stressed.

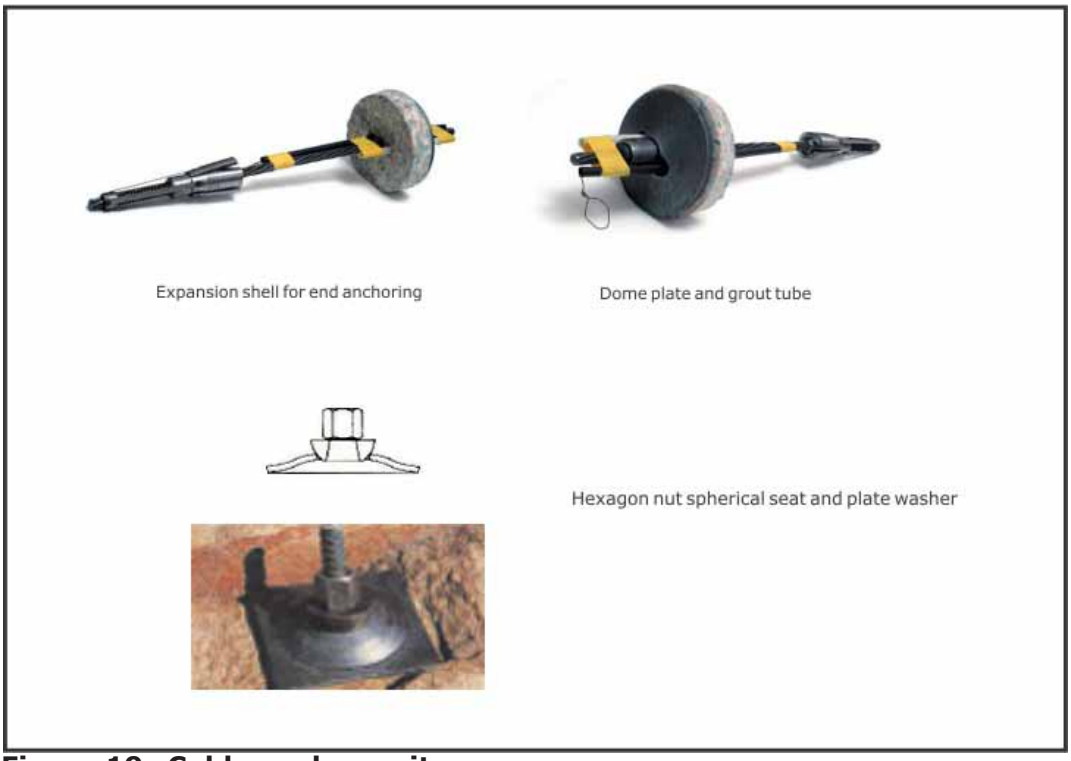


Figure 19: Cable anchor units

Figure 20: Barrel and wedge of mechanical anchor

back

STRAND DIAM-ETER (MM)	ULTIMATE TENSILE STRENGTH (TONS)	WORKING LOAD (TONS)	HOLE DIAM-ETER (MM)
12.5	18.0	10.0	38.0 – 42.0
15.24	25.0	15.0	38.0 – 42.0
18.00	35.0	20.0	48.0 – 51.0
22.00	50.0	20.0	48.0 – 51.0

Table 4: Specifications of Mechanical cable anchors

CONNECTION 26

Paper on Load deformation

Load deformation characteristics

Support unit quality control

Manufacturers test results on steel strength, Corrosion (visual).
Covered storage facility is required to protect units against corrosion.
Select sample to perform own tensile strength tests

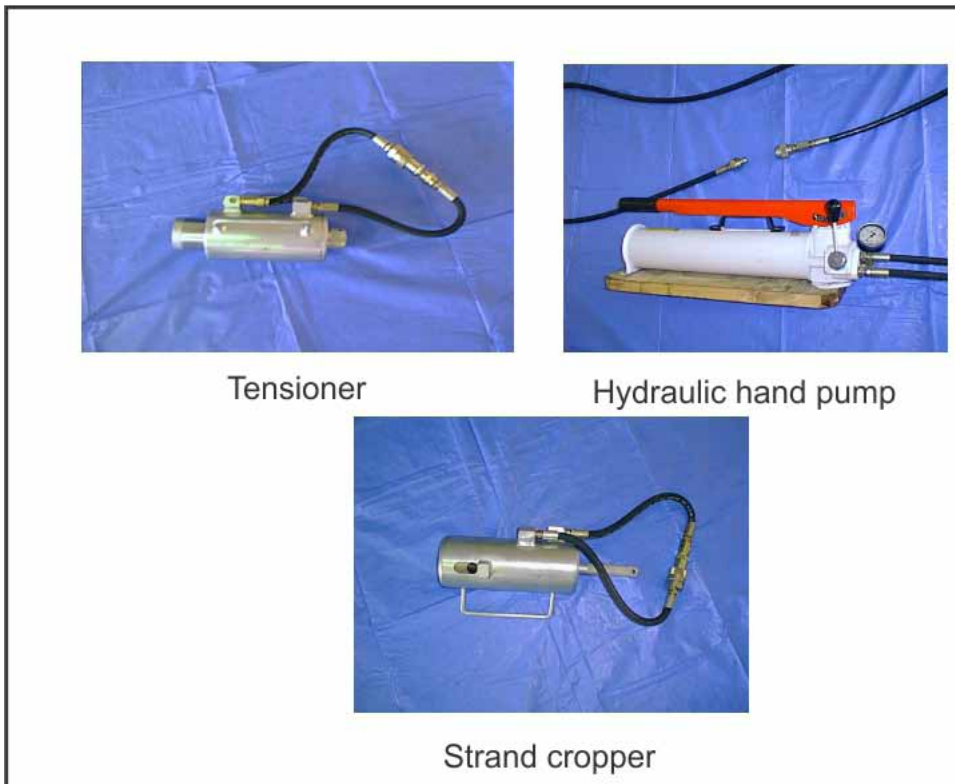


Figure 21: Equipment required for installation

Installation procedure

Pre-check lists

- Examine the drill machine and drill steel for:
  - Correct machine size for function being performed.
  - Wear or damage to drill bit and drill steel.
  - Air and water connections secured.
  - Adequate lubrication.
- Check the equipment and the anchor to be installed:
- Check the 25 ton tensioner as per the daily maintenance list
- Check the hydraulic pump as per the daily maintenance list

back

- Check the strand cropper as per the daily maintenance list
- Check the grout pump with hoses and correct fittings
- Check the type of anchor – the length and that the other material is available, i.e. the bearing plate with the barrel and wedge.
- Before installing the mechanical cable anchor, check the following:
  - The length of the anchors, always add 0.5m for the tensioner
  - The type of anchor, i.e. the mechanical anchor
- Invert the anchor and push it to the back of the drilled hole, if 0.5m of strand is left protruding from the hole, then the depth of the drilled hole is correct.
- Check the diameter of the hole, if it is not to the correct specifications, the anchor could either slip inside the hole, if the hole is oversize, or undersize, which results in the anchor not being able to enter the hole.
- Remove the tape at the expansion shell and push the large poly pipe down the expansion shell (but not all the way).
- Insert the expansion shell carefully into the collar of the hole (holding the large poly pipe and preventing such from entering into the hole) and gently push the anchor into the hole until it strikes the back of the drilled hole.
- Pull the protruding strand of the anchor until the expansion shell kicks open and locks firmly inside the hole.
- Cut a short piece off the poly pipe and insert into the collar of the hole; two pieces of poly pipe will now protrude from the drilled hole, namely the grout tube (short cut piece) and the breather tube (with wire protruding from inside).
- Seal the collar of the drilled hole with quick-setting grout to prevent grout from escaping during grouting operation.
- Clean the protruding strand with a clean cloth.
- Place the anchor bearing plate over the strand through the centre until it is flush against the collar of the hole (the two grout tubes should protrude through the two smaller side holes).
- Push the barrel and the wedge over the length of the protruding strand until flush against the bearing plate.

Tensioning procedure

- Connect the hydraulic tensioner to the hydraulic pump.
- Push the tensioner with the jack nose over the protruding strand until flush up against the barrel (always ensure that the grout tubes will not be pinched closed during the pre-stressing operation).
- Operate the hydraulic pump until the shear ring of the barrel shears.
- Never let anyone stand directly behind the tensioner while in operation.
- The anchor is now pre-stressed to the specified load.
- Reverse the control of the hydraulic pump and operate until the tensioner is loose on the protruding strand.
- Remove the tensioner and connect the strand cropper to the hydraulic pump and cut the protruding strand approximately 100mm from the barrel.
- Prepare the grout and connect the grout pipe from the grout pump to the short grout tube and pump the grout into the hole until grout starts to escape from the breather tube.
- Close off the breather tube and pump once or twice until hole is completely full.

back

back

- Remove pump grout hose and close off the grout pipe.
- The anchor installation is now complete. Proceed to the next hole.

Monitoring of installation

- Tendon support must be installed within the correct distance to the face in accordance with specified requirements.
- Each row must be spaced according to specified requirements.
- The distance between tendons must not exceed the specified distances.
- Cable anchors are intended to be installed perpendicular to the rock surface or strata. Deviations can result in bending stresses, thereby reducing the ultimate load carrying capacity of the anchor.
- Wherever the rock strata are horizontal, install cable anchors at an angle of not less than 60º to strata.
- Pull tests must be conducted regularly.

Methods to ensure installation quality

Units are tensioned and not loose. Grout tubes are filled with grout indicating full column grout. Pull tests to ensure adequate installation. Grout cubes filled for compressive strength tests. Load indicators on units to ensure correct pre-loading.

Testing procedures

- Install the nut in a 500 tons tensile testing machine.
- Apply a gradually increasing tensile load until the required load is attained.
- Release the load and inspect the specimen for any deformation.
- Re-load to the second proof load, release the load and inspect the specimen for any deformation.
- Apply a gradually increasing tensile load until failure occurs.

Quality control of installation

- Support holes must be drilled to the required diameter by using the correct drilling bit size.
- Support holes must be drilled to the correct direction, at least a right angle to the strata. This can be measured by using clino-rule.
- Support spacing must conform to the standard set by the rock engineer. A measuring tape can be used to measure the distances between the support units.
- Cable anchors must be fully grouted. Normally, grout overflows immediately when the support hole is full.
- Regular pull tests must be conducted on the cable anchors.
- Cable anchors must be tensioned to the required load that is specified on the hydraulic hand pump gauge.

CONNECTION 27

See 'Hutchinson & Diedericks: Cable Bolting in Underground Mines'

and also DSI_cable bolts

back

SPLIT SETS
Technical specifications

	Kitsbolt 33	Kitsbolt 39	Kitsbolt 42	Kitsbolt 46
Outside diameter	33.0-34.5mm	38.0-39.6mm	42.0-43.6mm	46.0-47.6mm
Taper lead diameter	27mm	33mm	37mm	41mm
Nominal outside diameter of retaining ring	49mm	54mm	58mm	62mm
Nominal outside diameter of retaining flange		70mm		
Steel thickness	2.3mm	2.3mm	2.3mm	3.2mm
Nominal yield load (tension)	90 kN	90 kN	90 kN	
Nominal ultimate load (tension)	110 kN	110 kN	110 kN	
Slot width	145-165mm			

Table 5: Splitset specifications

Technical information above is courtesy of RSC Ekusasa Mining

Load deformation characteristics
Figure 18 shows the load deformation relationship for an SS39 Split set, while also providing information on the relationship between bit diameter and holding tons for a FS46 Split set.

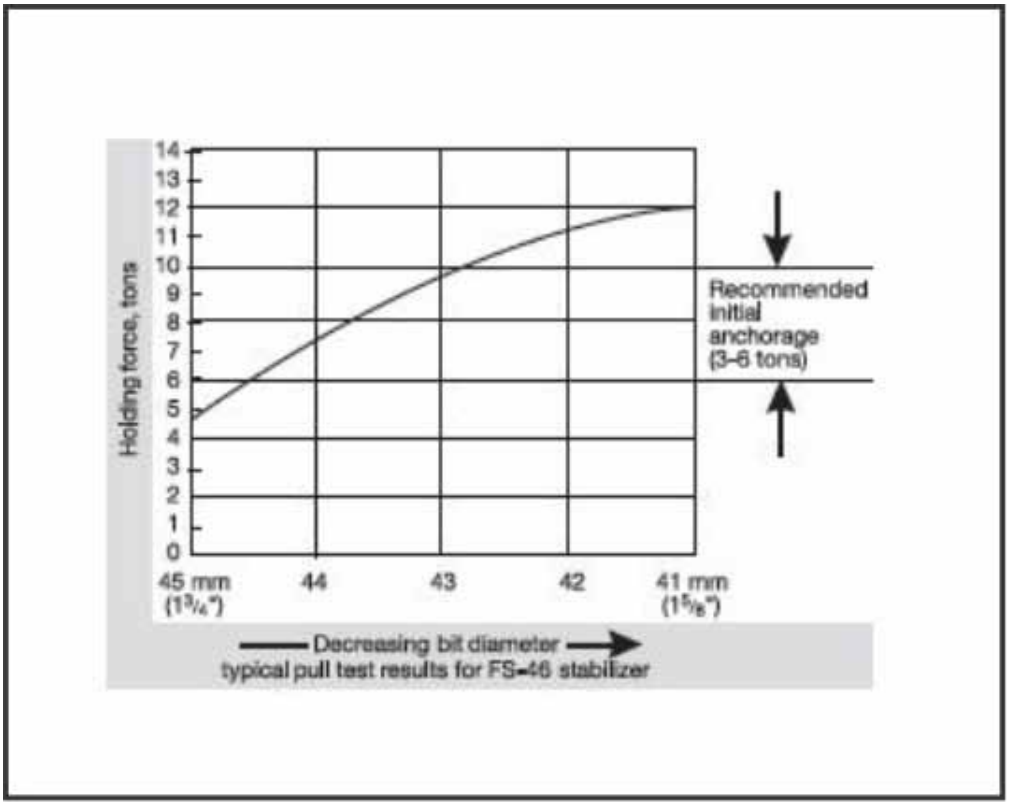


Figure 22: Bit diameter and holding ton relationship for a split set

Support unit quality control
Corrosion levels through visual inspection.

Installation procedure

The general installation process is such that these units are easy to install and all that are required are the appropriate driving tool and a percussion type drill (jackhammer, jackdrill, stopper etc). Drill a hole 1 to 4mm smaller in diameter than the split set to be installed. Place the driving tool into the chuck of the drill. Place the bearing plate over the tapered end of the friction rock anchor. Place the tapered end of the friction rock anchor into the collar of the drill hole and align. Start the drill and drive the friction rock anchor home until the bearing plate is hard up against the rock face.

Detail installation procedure

- Drill a hole slightly smaller in diameter than the Split Set™ stabiliser. The recommended bit diameter for SS39 Split Set™ bolts is around 35 to 38mm;
- Ensure that the hole is drilled slightly deeper than the complete length of the bolt. This will allow the unit to be driven in again in areas where the collar may spall away;
- Attach the appropriate driver tool. Check the condition of the driver tool; excessively worn driver tools can damage the unit during installation;
- Align the unit with the drilled hole and drive it in;
- Check the time that it takes to completely drive the unit into the hole. The time taken to drive the unit in will be a guide as to the load that the fixture will be holding. A driving time of more than 25-40 seconds will generally result in a fixture holding around 7-8 tonnes;
- In soft ground, it could be necessary to reduce the bit diameter to increase the driving time, thereby increasing the load the bolt will hold;
- Drive times of less than 10 seconds will usually result in a fixture that will hold less than the recommended load.

CONNECTION 28

See RSC Friction Stabilizers.pdf

Methods to ensure installation quality

The following guidelines with regards to split sets have been provided by DSI (link to DSI website and Friction stabiliser_paper 2 pdf):

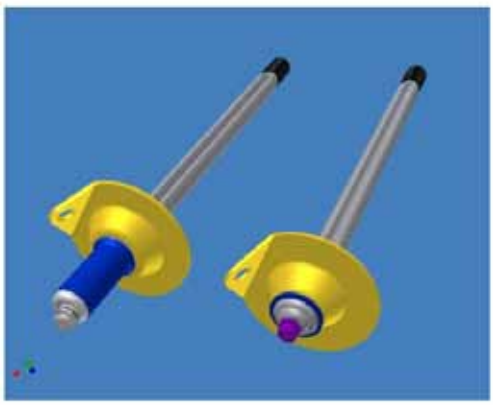
- Type of ground: Soft material requires longer unit lengths to ensure appropriate pull-out force. Soft ground also increases potential for hole over-break;
- Hole condition: Drilled hole must be cleaned to ensure smooth installation.
- Hole length: Holes drilled too short will prevent contact between faceplate and rock, as a part of the split set will stick out of the hole. Damage to the unit occurs when it is forced into the short hole. Drill hole longer than the required split set length;
- Oversized holes: Hole size is the most critical aspect of split set installation. The support provided by the split set is dependent on the fact that the hole is slightly smaller than the split set. Bent steel, soft rock and incorrect bit size cause oversized holes;
- Undersized holes: Becomes difficult to install if the hole is significantly smaller than the split set and damage to the unit occurs when forced. Too small or worn bits cause undersized holes;
- Drive time: For a 1.5-1.8m split set, the 'drive time' (time to install unit) must be between 8 and 15 seconds to provide proper initial anchorage. Faster installation means a too large hole and longer installation time means too small holes.

- Bit selection: Button bits are usually 2.5mm larger than their reported sizes when new, resulting in a 39.5mm hole for a new 37mm button bit. They do, however, wear quickly, resulting in increases in anchorage. 'Cross' bits are more accurate in the hole diameter provided and are preferred to ensure good installation.
- Perpendicular installation: Perpendicular installation ensures that the welded ring is in contact along its whole circumference and prevents point loading on parts of the unit.
- Monitoring: Installation must be monitored (drive times, installation direction, etc) and pull tests conducted to test initial anchorage.

Example of split set pull test procedure

- Ensure that the area to be worked in is safe from all hazards, e.g. loose rock.
- Ensure that all personal are wearing the appropriate PPE.
- The Pull Tester consists of two assemblies: the hydraulic portion, which includes the cylinder, gauge, pump, hose and adapter; and the mechanical portion, which includes the claw, housing, spindle and nut. Check that all are in good working order.
- Attach the pull claw over the top of the pull collar with the gap of the claw facing down. This prevents the claw from dislodging from the pull collar. The pull collar has to be attached to the split set before installation.
- Place the housing and hydraulic cylinder over the spindle and claw, and hand tighten the spindle nut until all portions are held in place. The piston end of the hydraulic cylinder should be facing away from the rock face.
- Place the pump (with gauge attached) in an area that does not expose personnel to potential hazards, e.g. rock falls. Please note that no personnel should stand directly behind the pull tester while testing is taking place.
- The pump is then actuated to raise cylinder pressure, until a predetermined loading is reached, or the split set slips.
- The maximum gauge reading at slip or the pre-determined limit indicates the holding force of the split set.

HYDRABOLTS
Technical specifications



Hydrabolt

Hydrabolt pump

Figure 23: Hydrabolt equipment

back

The hydrabolt is a fabricated steel tube, sealed at both ends. At one end, a hydraulic nipple, with integral non-return valve, allows connection to a hydrabolt pump. The principal of operation is that having drilled a hole in rock to the required diameter and depth, the bolt is installed and a pump used to expand the tubular bolt radially with water (or other incompressible liquid) at high pressure – typically around 25Mpa. Frictional resistance between the wall of the hole and hydrabolt provides outstanding pull-out resistance. The input pressure will determine the final pull-out force of the tendon.

The hydrabolt incorporates a check valve to prevent the fluid from being released once the inflating nozzle is removed. As the installed hydrabolt is sealed, it does not continue to corrode internally.

The hydrabolt has been designed to facilitate ease of installation, clamping inside the hole, adapting to the shape, size and irregularities of the hole and distributing the clamping force evenly throughout the length of the hole providing the user with immediate support and safe working environment.

Specifications of hydrabolts

Diameters	: 26mm expanding to 36mm
	: 29mm expanding to 40mm
Lengths	: 900mm to 3000mm

back

Load deformation characteristics

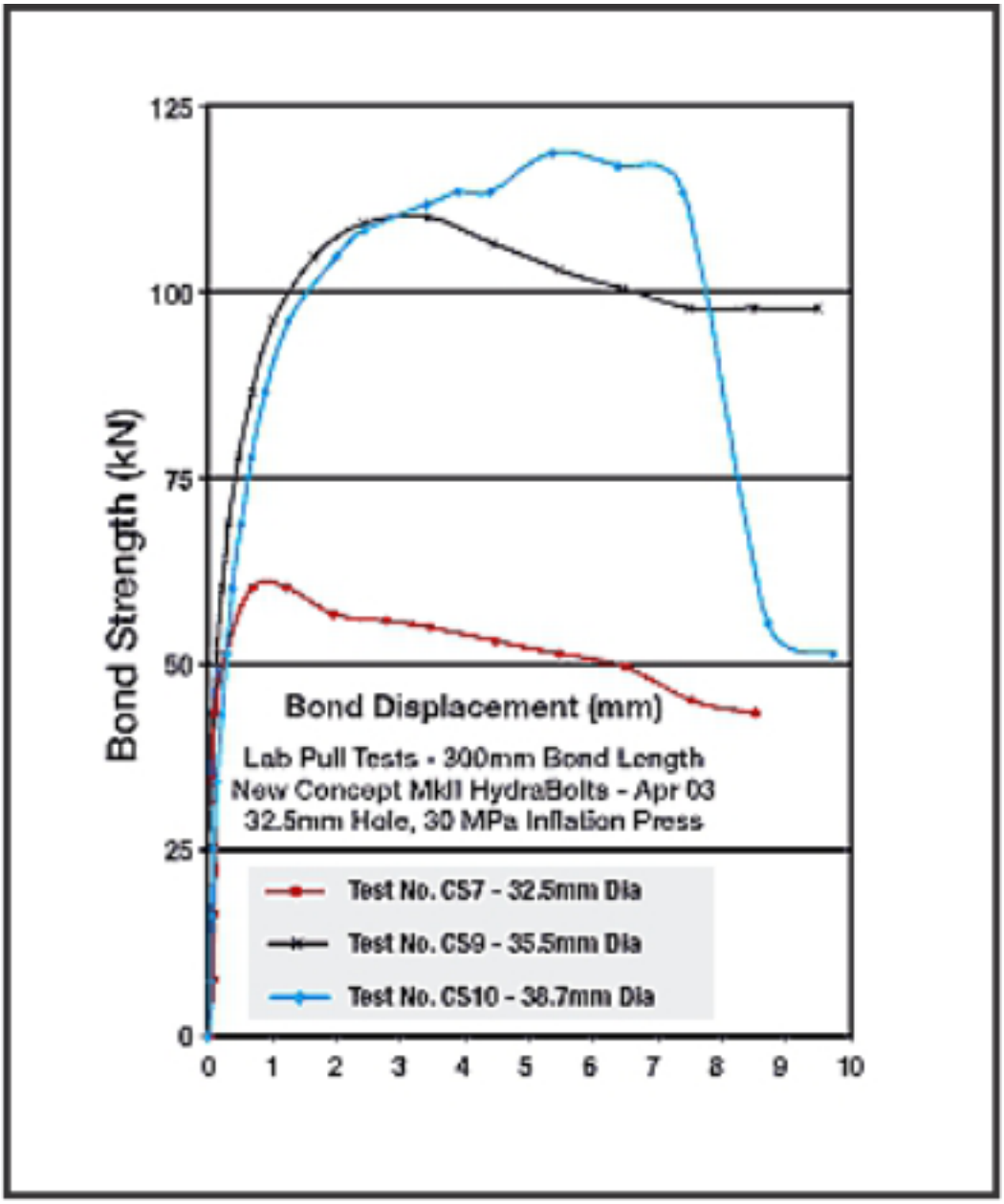


Figure 24: Hydrabolt load-deformation graph

Support unit quality control

Manufacturers test results on steel strength.
Covered storage is required to protect units against corrosion.
Select samples to perform own tensile strength tests.

Installation procedure

High Press A-PUMP NC8 c/w
Safety Pistol NC8 + Hose
Hose 10m – NC8 Both Ends
High Press AP Conversion Kit

Drill rig

Drill steel

Nozzle

Figure 25: Equipment required for Hydrabolt installation

Personal protective equipment

- Eye-protection needs goggles.
- Hearing protection needs ear-plugs.
- Head protection needs helmets.
- Hands protection needs abrasion resistant gloves.
- Body protection needs clothing (Overall).
- Foot protection needs rubber gumboots.
- Fall from heights protection needs safety belt.

Installation procedures

- Examine and make safe the area where the installation will take place.
- Install temporary support as per mine standards.
- Prepare the hanging wall to drill the support holes.
- Mark off and drill the required holes in the hanging wall according to your mine standards. As close to 90° as possible to the strata.
- Check the hydrabolt for any damage to the tube and the valve.
- Assemble the washer onto the hydrabolt.
- Insert the hydrabolt into the hole until the washer is tight against the hanging wall. Hold in place if necessary with hand.
- Remove the red plastic cap from the valve.
- Check that the valve is clean and free of dirt; clean with water if required.
- Fit the hydrabolt nozzle onto the valve.
- Compress the handle on the safety pistol and start pumping water into the hydrabolt.
- As soon as the hydrabolt starts expanding, move away and pressurise the hydrabolt from a safe position.
- Continue pumping until the safety pistol squirts water out the relief valve (20Mpa).
- Remove the nozzle from the hydrabolt and check that the load indicator is visible.
- Prepare for the next installation.

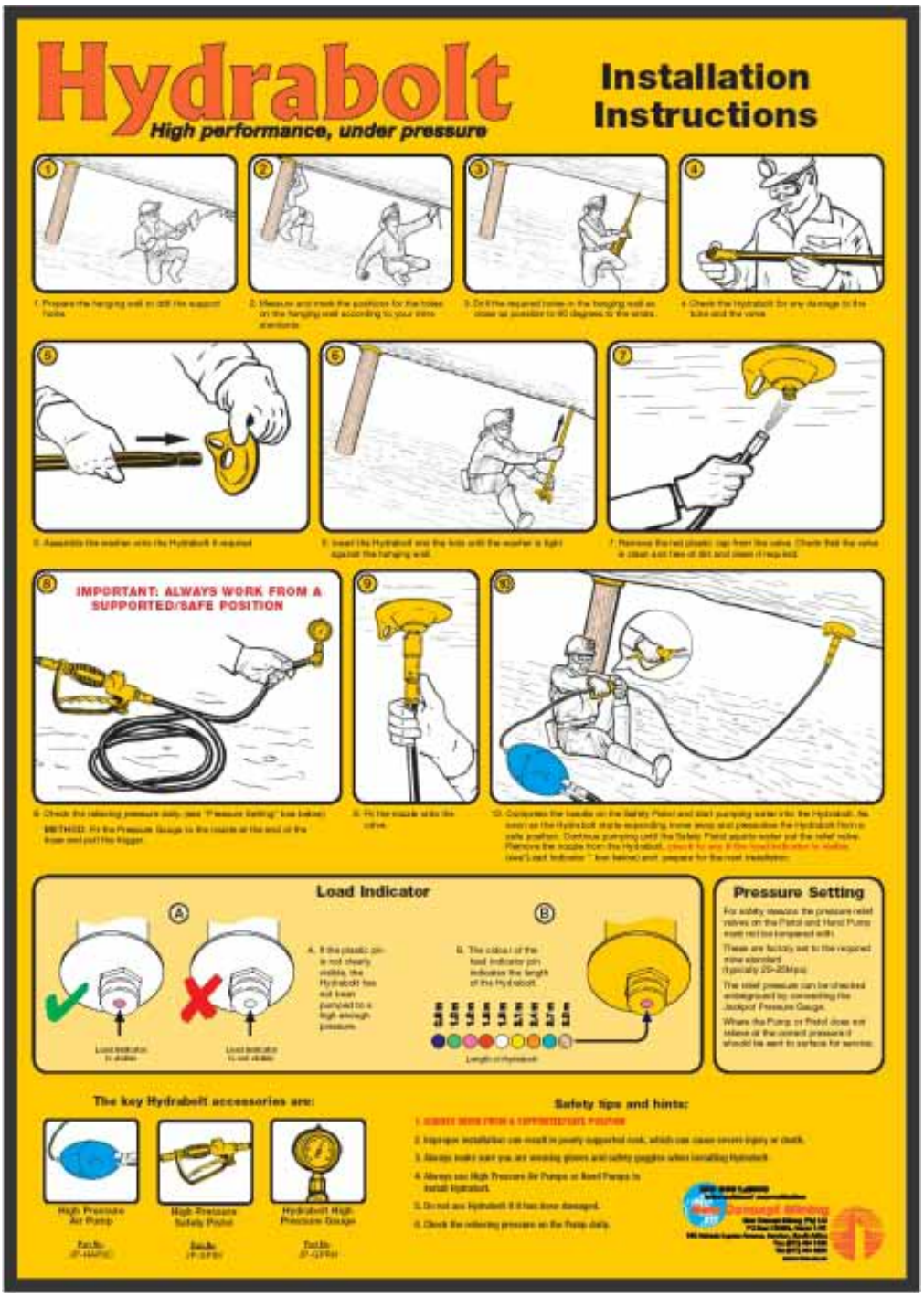


Figure 26: Hydrabolt installation instructions (Courtesy of New Concept Mining)

Methods to ensure installation quality

Units do not protrude the hole (too short hole) or are installed too easy (hole diameter too large).
Pull tests to ensure pull-out force exceeds design.

Testing procedures

- Load – deformation laboratory test.
- Pull test after hydrabolt support installation.

Quality control of installation

- Drill support holes to the required correct length and size.
- Make sure that the hole is free of loose rocks and install the hydrabolt to its full length.

- Appropriate storage conditions recommended by supplier.
- In case where the hydrabolt is not pre-loaded to the correct pressure, the load indicator will not be visible.
- Where valve fails after pre-stressing and results in loss of load, the unit pressure must be tested prior to departure.

Monitoring of installation

- Number of hydrabolts that were blasted out per line.
- Hydrabolts that are necking out due to falls of rock in the surrounding.
- Falls of ground between the supports.
- Tensile failure of the bolts.
- Detect corrosion in the installed bolts.
- Failure of blocks of rock along the support units.
- Conduct regular pull tests on the bolts.

SHOTCRETE
Technical specifications

The following specifications should be considered together with any mine-specific specifications that might exist.

SABS 827	Sampling of aggregates
SABS 829	Sieve analysis, fines content and dust content of aggregates
SABS 830	Chloride content of aggregates
SABS 832	Organic impurities in fine aggregates (limit test)
SABS 833	Detection of sugar in fine aggregates
SABS 834	Soluble deleterious impurities in fine aggregate (limit test)
SABS 836	Effect of fine and coarse aggregate on the shrinkage and expansion of cement: aggregate mixes (mortar prism method)
SABS 838	Sand equivalent value of fine aggregate
SABS 839	Soundness of aggregates (magnesium sulphate method)
SABS 841	Aggregate crushing value of coarse aggregates
SABS 842	Fact value (10% fines aggregate crushing value) of coarse aggregate
SABS 843	Water absorption of aggregates
SABS 847	Flakiness index of coarse aggregates
SABS 860	Concrete tests – Dimensions, tolerances and uses of cast test specimens (15)
SABS 861-1	Concrete tests – Mixing fresh concrete in the laboratory
SABS 861-2	Concrete tests – Sampling of freshly mixed concrete
SABS 861-3	Concrete tests – Making and curing of tests specimens
SABS 863	Concrete tests – Compressive strength of hardened concrete
SABS 864	Concrete tests – Flexural strength of hardened concrete
SABS 865	Concrete tests – The drilling, preparation, and testing for compressive strength of cores taken from hardened concrete
	Aggregates from natural sources - Aggregates for concrete
SABS 1083	Deleterious clay content of the fines in aggregate (methylene blue absorption indicator test)

SABS 1243	Particles of diameter
	Determination of energy absorption capacity of slab specimens
SABS 1244	Cement standards and selection
SNCF 40	Sprayed Concrete
SABS Env 197-1	Sprayed Concrete Guidelines for Specifiers and Contractors
European specifications	

DEFINITION

- Sprayed concrete is a mixture of cement, aggregate and water projected pneumatically from a nozzle into place to produce a dense homogeneous mass. Sprayed concrete normally incorporates admixtures and may also contain the addition of fibres or a combination of these.
- Fibre reinforced sprayed concrete is primarily made of cements, aggregates and discrete reinforcing fibres. Fibres suitable for reinforcing concrete and mortar have been produced from steel and organic polymers. Glass and carbon fibres have also been used in mortar matrices.
- Dry process is a technique in which cement and aggregate are batched, mixed and fed into a purpose-made machine wherein the mixture is pressurised, metered into a compressed air stream and conveyed through hoses or pipes to a nozzle where water is introduced as a spray to wet the mixture, which is then projected continuously into place. The mixture may also incorporate admixtures, cement additions or fibres, or a combination of these.
- Rebound losses are part of the overall production losses and consist of material that, having been sprayed through the nozzle and struck the surface, does not adhere.
- Nozzle is the equipment through which the mix is discharged; it consists of a pipe with a mixing unit into which constituents are injected. With the dry process, water and any liquid admixtures are added.
- Binder is the total amount of cement and cementitious addition in the sprayed concrete.
- Cement additions, also known as cement extenders, are materials that exhibit a pozzolanic reaction or materials where the hydration process is activated by the presence of a high alkaline environment.
- Coarse aggregate shall be aggregate that is retained on a sieve with an aperture size of 4.75mm, when sieved through a standard nest of sieves.
- Fine aggregate shall be aggregate that passes a sieve with an aperture size of 4.75mm, when sieved through a standard nest of sieves.
- Blended aggregate shall be aggregate, either naturally occurring or crushed, coarse or fine, which is combined to form a graded aggregate.

Materials and design criteria

Aggregate

- The materials used for the concrete shall comply with requirements of SABS 1083:1994 and such amendments thereto.
- Aggregate shall be clean and free of deleterious organic materials. All aggregates shall be checked for potential chemical reactions like alkali silica reaction, and should any potential problems be determined, the aggregate shall be deemed unacceptable.

- Should no other source of aggregate be available, then the aggregate must be approved by a responsible engineer after the evaluation of its influence on the quality and durability of the sprayed concrete.
- The grading analysis of the combined aggregate shall fall within the EFNARC envelope for sprayed concrete (European Specification for Sprayed Concrete Clause 5.4). Irrespective of this fact, the aggregates gradation curve shall be compatible with the requirements for compressive strength, durability and energy absorption when combined with the cementitious material and fibre.
- Aggregates based on crushed materials shall be selected with care to ensure that their angularity and shape do not adversely influence the rebound losses of the fresh concrete.
- The portion of aggregate larger than 8mm in size shall not exceed 5% when expressed as a percentage of the dry aggregate, and no aggregate shall exceed 12mm. The maximum in-situ moisture content of the delivered aggregate shall not exceed 5% when expressed as a percentage of the dry aggregate.

Cement

- Cement shall comply with the requirements of SABS ENV 197-1.
- The type and content of cement shall be selected to meet the specified requirements for concrete compressive strength, durability and energy absorption. Any material proposed as a binder that is not included in SABS ENV 197-1 must first be tested for suitability and should comply with the minimum requirements for traditional cement.
- The cement content shall be between 350 and 450kg/m3 of concrete for the dry concrete process.
- The initial setting time shall be long enough to give workability, but not so prolonged as to prevent sufficient early strength development to provide safe support.
- The cement performance shall be evaluated by preliminary tests together with other concrete components intended to be used. Its compatibility and reactivity with admixtures shall be checked during preliminary tests.
- Where there is a risk of sulphate attack or of alkali silica reaction, special cement types and/or cement additions shall be used.
- The cement shall be contained in heavy duty, robust plastic waterproof containers or bags and packed within the aggregate bag in such a manner that the cement bag cannot come into intimate contact with the container or bag in which the fibre is packed.

Cement additions

- Only cement additions with established suitability should be used. Cement additions shall be incorporated with the cement and shall be homogeneously mixed.
- The compatibility of additions with admixtures and accelerators must be verified by preliminary testing.
- Additions that have chemical reactions with cement can partially substitute the cement content. The type and amount of an addition shall be evaluated in preliminary tests.
- Approved cement additions are indicated hereunder:
  - Pulverised fuel ash (fly ash)
  - Ground granulated blast-furnace slag (ggbs)
  - Silica fume

back

Cementitious material	Maximum addition
Silica Fume	15% of Portland cement
Fly Ash	30% of Portland cement
	15% of Portland/Fly ash cement
	20% of Portland blast furnace slag cement
GGBS	30% of Portland cement

Table 6: Maximum level of additions to shotcrete (by weight)

Admixtures

- Admixtures may consist of powders or liquids and shall be used at the manufacturer's recommended dosages.
- The use of admixtures will generally not be allowed and may only be used on the written authority of the employer. The supplier shall be required to provide full and sufficient documentation to support the use of such admixtures, together with the effect such admixtures have when combined with the various constituent parts of the combined concrete mix.
- The supplier shall provide a full method statement to the employer with respect to the method by which the admixture has been, or shall be added to the concrete in order to achieve consistency and uniformity of addition.
- The supplier shall supply details of the effect of water on the admixtures used, together with any stability problems that may be associated with high or low ambient temperatures.
- The supplier shall provide full details on the impact of the admixtures on the health and safety of the operatives.
- Admixtures that are required to be added at the time of mixing shall be contained in waterproof containers or robust plastic bags and incorporated into the bag containing the constituent parts of the mix in such a manner that no damage occurs to the packaging.

Accelerators

- Accelerators may consist of powders or liquids and shall be used at the manufacturer's recommended dosages.
- The use of accelerators will generally not be allowed and may only be used on the written authority of the employer. The supplier shall be required to provide full and sufficient documentation to support the use of such accelerators, together with the effect such accelerators have when combined with the various constituent parts of the combined concrete mix.
- All accelerators shall be used at the manufacturer's recommended dosages and shall comply with the European Specifications for Sprayed Concrete Guidelines for Specifiers and Contractors 1999 clauses G4.6 and G5.5 with the following exceptions:
- Alkali-free accelerators shall comply with the following:
- The maximum Na2O-equivalent of the accelerator is limited to 1.0% by mass.
- The dosage (by mass of the binder) is preferably:
- 6.1 powder accelerators: 4-8%
- 6.1 liquid accelerators: 4-10%
- The pH value of liquid accelerators shall be between 2.5 and 8. Alkali-containing accelerators shall not be allowed.

back

back

- The decrease in compressive strength (at 7 or 28 days) of the accelerated sprayed concrete compared with sprayed concrete without the addition of an accelerating agent shall not exceed 15% for both powder and liquid accelerators.
- Accelerators that are required to be added at the time of mixing shall be contained in waterproof containers or robust plastic bags and incorporated into the bag containing the constituent parts of the mix in such a manner that no damage occurs to the packaging.
- Accelerators that are required to be added at the nozzle shall be supplied in separate waterproof containers of a capacity suitable to provide uninterrupted spraying. The size of the container and quantity contained therein shall be agreed with the employer.
- In the event that an accelerator is required, the supplier shall carry out preliminary spray trials to determine the quantity of accelerator necessary to accelerate the mix under normal working conditions, and shall supply the appropriate quantity.

Fibre

- The employer shall specify the base material from which the fibre shall be made.
- The length of the fibre shall not exceed 66% of the proposed layer thickness and shall be of such a length to promote good spraying characteristics and when tested in terms of the procedures specified for EFNARC panel testing, give the required energy absorption values.
- The quantity of fibre included in the sprayed concrete material shall be sufficient to give the required energy absorption values when tested in terms of the procedures specified for EFNARC panel testing.
- The type and quantity of fibre incorporated in the sprayed concrete mix shall be verified in preliminary tests.
- Steel fibre shall be contained in sealed robust plastic containers to protect them from corrosion, oil, grease, chlorides and deleterious materials that will reduce the efficiency of mixing or spraying processes or that will reduce the bond between the fibres and the sprayed concrete.
- Fibres that tend to form a fibre ball during batching and mixing shall not be used.

Mix compositions

- The employer shall specify the required performance of the sprayed concrete and the supplier shall be responsible for selecting the mix proportions to produce the specified performance.
- The supplier shall submit his design mix together with sufficient bagged material to an independent laboratory, approved by the employer, to confirm the compressive strength and energy absorption values under laboratory conditions; in addition to these values, the supplier shall submit sufficient samples (10 bags) of his product to determine the following criteria:
  - Grading analysis
  - Relative density of the aggregate
  - In-situ moisture content of the aggregate
  - Mass of the aggregate
  - Mass of the fibre
  - Mass of the cementitious material

back

- The supplier shall supply samples selected at random by an independent laboratory approved by the employer at monthly or such other intervals as indicated in the project specifications for quality control testing.

Bags

- Bags containing the aggregate and individually packed constituents shall be manufactured from strong durable ultraviolet light stabilised plastic or other approved material and shall be of sufficient size to contain the contents without being over stressed.
- Each constituent shall be contained in individual bags or containers, packed inside the aggregate bag and shall be clearly identified.
- Bags containing the aggregate shall be of sufficient size to hold a total mass of between 20 and 30 kilograms. The bags shall be silk screen or marked in another approved manner with the following information:
  - Suppliers name
  - Compressive strength of mix
  - Total water to obtain specified compressive strength
  - Energy absorption value at total water content
  - Mass and type of fibre per bag

Water

- The employer shall supply a full chemical analysis of the water proposed for use in the sprayed concrete.
- The supplier shall take cognisance of the water intended for use when designing the sprayed concrete mix.

back

Load deformation characteristics

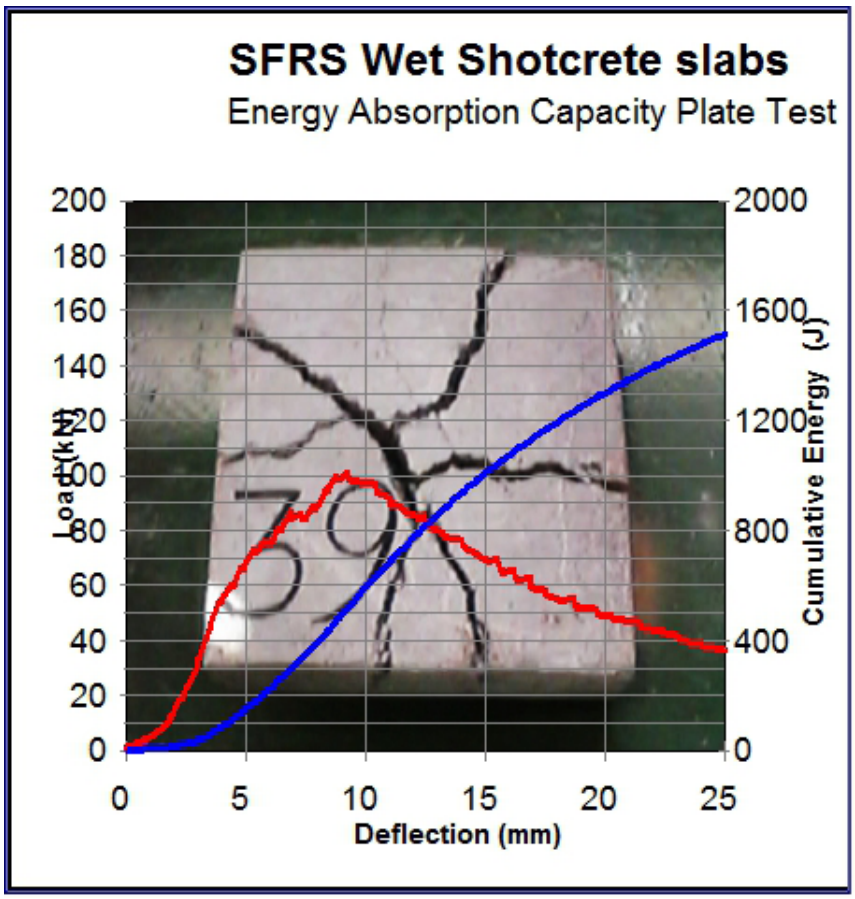


Figure 27: Shotcrete load-deformation and energy absorption graph

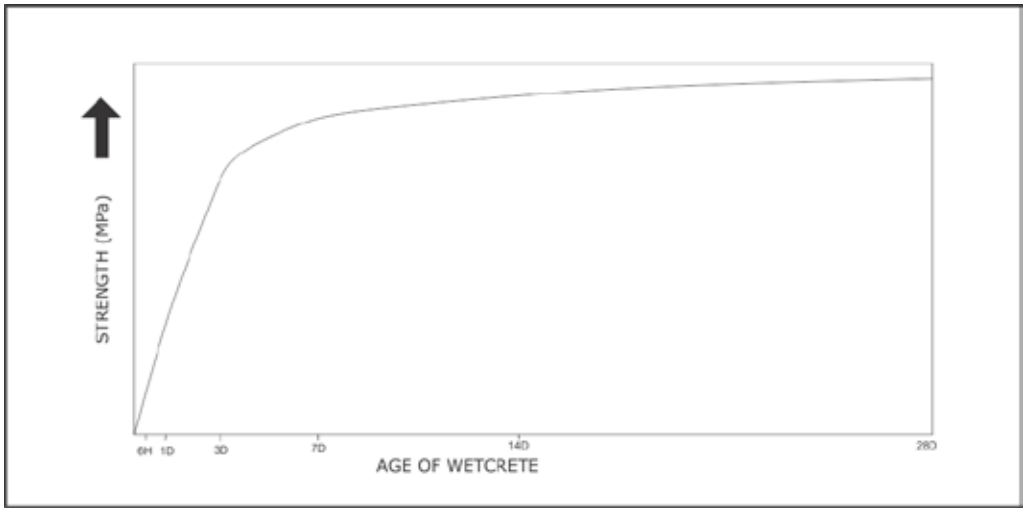


Figure 28: Increase in shotcrete strength with time

The figures above show typical results of load deformation tests conducted on shotcrete samples. Figure 27 shows the results of a 'slab' test, where the red line indicates the load-deformation behaviour and the blue line refers to the energy absorption capacity of the material. The peak flexural strength of the shotcrete is shown to be 100kN (for this specific mix design) at a deflection of 10mm. However, the ability of the shotcrete to sustain load does not disappear immediately after reaching this peak, as shown by the continued load generation (to more than 40kN at 25mm deflection) and increasing energy absorption capacity.

back

back

Figure 28 shows the results of core sample tests and indicate a fast increase in strength within the first 3 days after application and only slight increases after 7 days.

Support unit quality control
Manufacturer's test results on compressive strength, deflection.
Storage is required to protect material against heat.
Select samples to perform own strength tests.

Installation (application) procedure
In the application of shotcrete for ground support, as in any other field project, good planning is essential for efficient and safe working conditions. Planning starts with proper site setup to facilitate mobilisation of equipment and personnel to perform the shotcrete operations in an effective manner. One key requirement in the proper site setup is that the shotcrete equipment should be close to the work. Doing so will minimise hose lengths to be used and will require smaller volumes of air to move the material.

Crew organisation and personnel selection on the job are also important for the success of shotcrete application. The minimum size of a shotcrete crew should consist of a foreman, nozzleman, finisher, assistant nozzleman, blowpipe operator, gunman or pump operator, mix operator and labourers.

Scaffolds or other means of lifting are required where these will permit the nozzleman to move closer to the work in order to properly cover the rock material. Such equipment should allow the nozzleman to move around freely, not pose any obstruction to the progress of the works, and be stable.

Prior to the application of shotcrete, it is crucial to properly prepare the receiving surface. Even correctly applied shotcrete may be compromised if the rock is not properly prepared. Surface preparation involves removing any loose material or debris, removing any cracked or unsound rocks, sandblasting or hydroblasting if necessary, followed by bringing the surface to a saturated surface dry condition. If there is any doubt, it is generally better to err on the dry side rather than having too damp a surface. The objective is to achieve a stable, sound, rough, and non-absorptive surface that will enhance the bond of the applied shotcrete.

Batching, mixing and the supply of the shotcrete mixture can be done in several ways depending on the particular method of application (wet- or dry mix) and the location of the project. There is a growing tendency towards the use of pre-blended materials, particularly for dry-mix shotcrete or for remote locations. The use of pre-blended materials allows for convenient supply of shotcrete to the site as well as excellent quality control of the shotcrete mix proportions.

The quality of the applied shotcrete is dependent on the skill and application technique of the nozzleman. In the case of dry-mix shotcrete, this is particularly critical, as the nozzleman controls the amount of water to be introduced to the shotcrete stream at the nozzle.

PROCEDURE EXAMPLE

- Once the area to be shotcreted has been identified, the holder of a blasting certificate must examine and make the area safe.
- The holder of a blasting certificate/competent person must ensure that the workplace is properly ventilated and should test for dangerous gases.
- A certified and appointed operator of the relevant machinery must use the approved vehicles to move the equipment to the workplace to be shotcreted.

back

back

- Ensure that the equipment is in good working order and pre-checked.
- The area to be shotcreted must be properly washed down, using a 25mm hose and service water. It is essential that the area to be shotcreted is clean and dust free as this has a huge impact on the bonding of the shotcrete to the hanging wall.
- After completion of the washing, the area must again be examined and barred down.
- A scaffold or appropriate vehicles can be used to do work in areas that are too high to do proper application or examination. The blasting certificate holder / competent person must visually examine the scaffold for any defects. He must also supervise the proper erection and stability of the scaffold. Safety belts must be worn at all times during the erection and use of scaffolds.
- All compressed air hose fittings must be secured with a safety sling to prevent injuries in the event of a hose-fitting failure.
- If the shotcrete machine is electrically driven, the holder of the blasting certificate / competent person must inspect all electrical connections visually. If any defects are observed, or if in any doubt, the certified electrician must be called to rectify the problem before using the machine. All electricity to the machine must be isolated immediately at the code box.
- Once the holder of the blasting certificate / competent person is satisfied that all machines to be used are in good working order, the mixing and application of the shotcreted can be done.
- Mixing of the shotcrete material must be done in the mixing machine as per training. No other materials must be added, and the machine is not intended for any other material application other than shotcrete.
- Once the certified operators are satisfied pertaining to the quality of the mix, the compressed air can be opened to the desired flow and the mixed shotcrete can be let into the feed tank.
- The operator of the nozzle must give the instruction for the compressed air and shotcrete mix to be opened.
- The operator of the nozzle must ensure that he is standing in a safe position and with the nozzle pointed in a safe direction.
- The operator must ensure that the compressed air pressure is not set too high or low, as this will affect the amount of rebound from the rock face. A slow and constant turning motion must be maintained to give a consistent application.
- Each layer of shotcrete must be applied from a different angle to ensure consistency in thickness and to ensure that the rear of outstanding rocks is also covered.
- Special attention must be given on the first layer to apply a thicker layer of shotcrete in cracks and small cavities. This is the area where the different rocks meet and the bonding is most important.
- After application, the holder of a blasting certificate / competent person or supervisor in charge can inspect the area systematically for consistency and correct application thickness.
- The operator must try to get a constant thickness. To ensure the thickness is sufficiently thick, a probe stick, with a washer welded onto it, must be used.
- The velocity of ventilation must be monitored at all times by the holder of a blasting certificate / competent person to ensure a safe and healthy working environment.
- The application crew must take great care in cleaning the machines properly at the end of a shifts work, as the shotcrete sets and equipment can be lost/damaged.

back

- After completion of a working place or section thereof, the application crew must call for the responsible supervisor to personally inspect the application in the workplace for quality and thickness.
- The application crew must not remove any of the equipment before the inspection by the supervisor has been done.
- Once the responsible supervisor has inspected the workplace as per procedure and is satisfied with the quality and thickness of application, he must sign off the workplace as being satisfactory on the document provided.

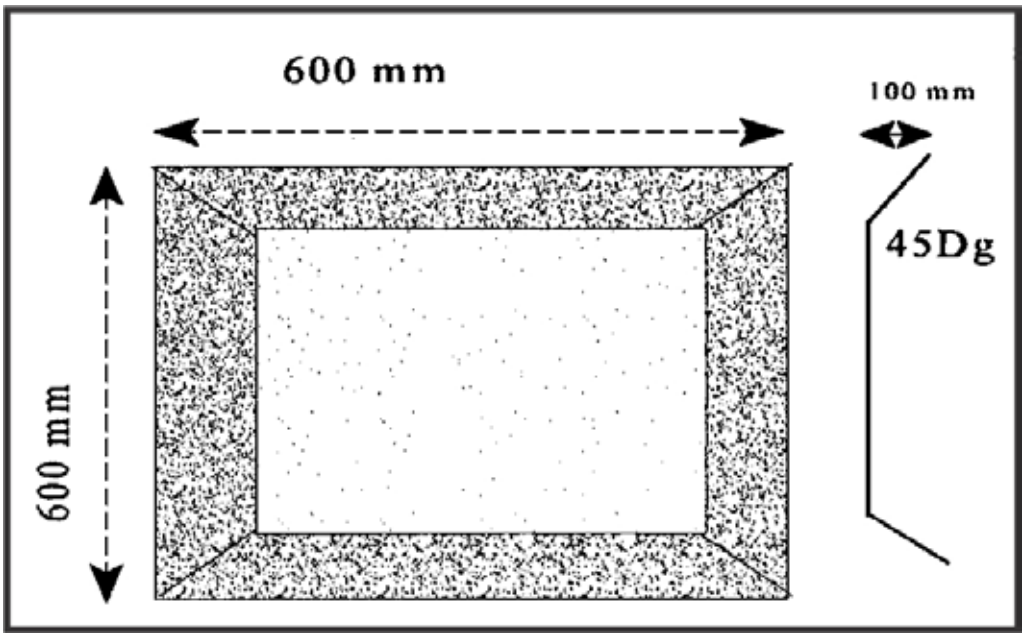
Methods to ensure installation quality

The main objective of the preliminary quality control is to test the initial mix design against the required specification. The focus of the testing is to determine the energy absorption characteristics of the mix design along with its compressive strength. The method of taking the samples and the testing of such is described below.

Energy absorption

The purpose of this test is to determine as close as possible the actual placed shotcrete's energy absorption characteristics.

- Three trays are placed against the sidewall of the excavation to be sprayed. At a point during the spraying process, the plates are sprayed full with the shotcrete product. As soon as they are full, a knife is drawn across the tray to level off the contents. These trays are then left in-situ for a period of 28, after which they must be carefully transported to the surface. On arrival at the surface, the shotcrete plates are removed from the trays, and are then cut with the diamond saw (core saw) to a size of 600mm x 600mm.



- The plates are then dispatched to the lab selected for testing. At the lab, the plate is placed in a frame and loaded with a point load, and two readings are taken.
- They are, force applied versus vertical displacement of the plate. The results are then plotted on a graph. From this graph it is then possible to calculate the energy absorption characteristics of the shotcrete. Should there be a high degree of difference between actual and design specification, then a re-examination of the mix may be required (usually the fibre content as this directly relates to energy absorption).

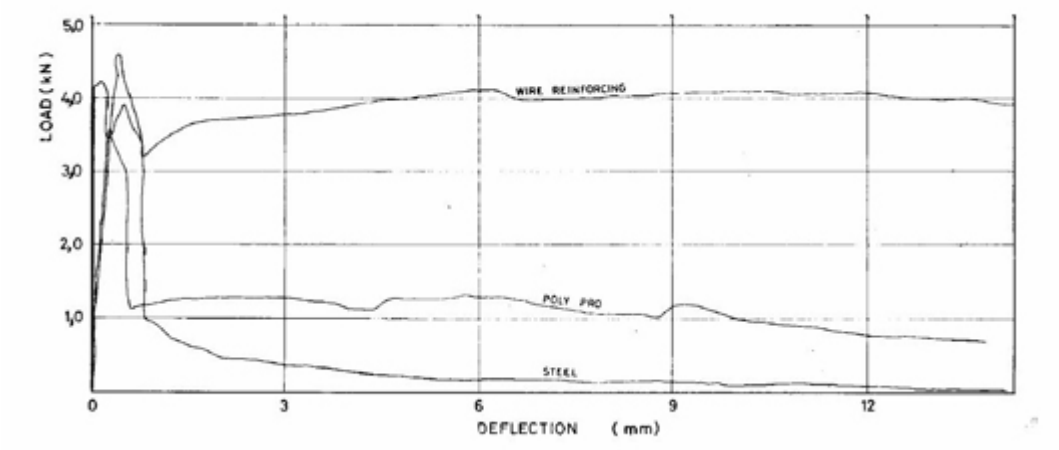


Figure 29: Load-deflection graph for shotcrete

Compressive strength
The purpose of this test is to ensure that a quality test can be done on the shotcrete's compressive strength.

- When spraying the three trays discussed above, it will also be necessary at the same time to spray a fourth tray from the same batch. This is necessary so that we can relate the compressive strength results directly to the energy absorption results. This tray, once sprayed, is also allowed to cure in-situ for 28 days. At the end of this period, the tray must be drilled for six cores, all 100mm long and 50mm in diameter. Note that the coring machine is a specialised one as it must give core with true cylindrical characteristics.
- Once the core is drilled, it must be carefully transported to the surface and dispatched to the lab. These six cores will then undergo a compressive load test and from there an average compressive strength can be determined.

NOTE

When these trays are being sprayed, all of the following continuous quality control tests must also be carried out. In doing this, it is then much easier to relate the continuous quality control test results to the key design parameters of energy absorption and compressive strength obtained in the above testing.

Continuous quality control and testing
Preliminary quality control and testing deals with determining whether the shotcrete and its methods of application meet with the design specifications. This section deals with methods used on a daily basis to ensure that the shotcrete applied is maintained against those specifications. The type of test and frequency of such will vary and will very much depend on production rates.

Penetration needle test
This test, as the name suggests, consists of a needle that is pushed into the mix sprayed on to the rock excavation. The resistance encountered causes movement on a scale (pushing against a spring). A reading can be taken, which relates back to an early compressive strength. The test must be done within one (1) hour of the mix being sprayed and for every batch sprayed. The result obtained should be in the vicinity of 0.5 MPa; if it fails to meet this requirement, spraying must be stopped and the manager must be informed so that the problem can be investigated and rectified (See Photo 8).

Thickness test
By using indicators (plastic pointers attached to the excavation wall), the operator can control the thickness sprayed to ensure that the design specification is met. On a weekly basis, a quality controller will visit the spray site underground and will on a random basis drill holes into the placed shotcrete, throughout the area sprayed, to check that the applied thickness is to specification. An arrangement must be made so that he can drill the hanging wall, as this is usually the most difficult area to build up to the required thickness. Should the results be below specification, further spraying will be required.

24-hour cube test
At the surface batch plant, 3 x 24-hour cubes are poured for each spray period (day). They are allowed to cure for 24 hours and are then subjected to a compressive strength test. The reason for this test is to ensure that the shotcrete is on the 28-day strength curve; if not, then the quality of the mix must be investigated and corrective action taken – this may even result in having to re-spray an area.

Seven-day cube test
Also at the surface batch plant, after every seventh set of 24 hour cubes, 3 x 7-day cubes are poured. These cubes are allowed to cure in a bath for period of seven days. After this period has elapsed, they are then subjected to a compressive strength test. A figure will be obtained which, in junction with the 24-hour cube test results, can be plotted on a graph of strength versus days (see Figure 16) to determine whether the shotcrete mix is on its ultimate 28-day compressive strength curve.

Bonding and core test
On a basis that is very much site dependent, a bonding and core test must be conducted.

The excavation sidewall is built up to a required thickness in excess of 120mm with the use of the thickness gauges over an area of 1.0m square. A core drill is mounted on the sidewall and 3 x 50mm diameter cores are drilled (the drill is pinned in one position and rotated) to a depth of 120mm. Note that the cores must be left in position. A dolly is bonded to each of the cores and in return they are attached to a puller and put under tensile load until they break free from the rock face. The readings obtained indicate the bond strength of the in-situ shotcrete. Therefore, using all three readings, an average bond strength can be determined. A minimum bond strength of 0.5MPa is a suggested requirement.

The same three cores are then weighed and also placed in a measuring jug full of water to determine the volume; using the weight and volume of the three cores an average in-situ density can be determined.
Finally, these cores are dispatched to the surface for lab testing to determine the in-situ compressive strength that can then be checked against design specifications.

NOTE

It is important that each built-up area is marked with a date to ensure that the area of the shotcrete tested is known. The core must be marked with this same date for test result identification purposes.

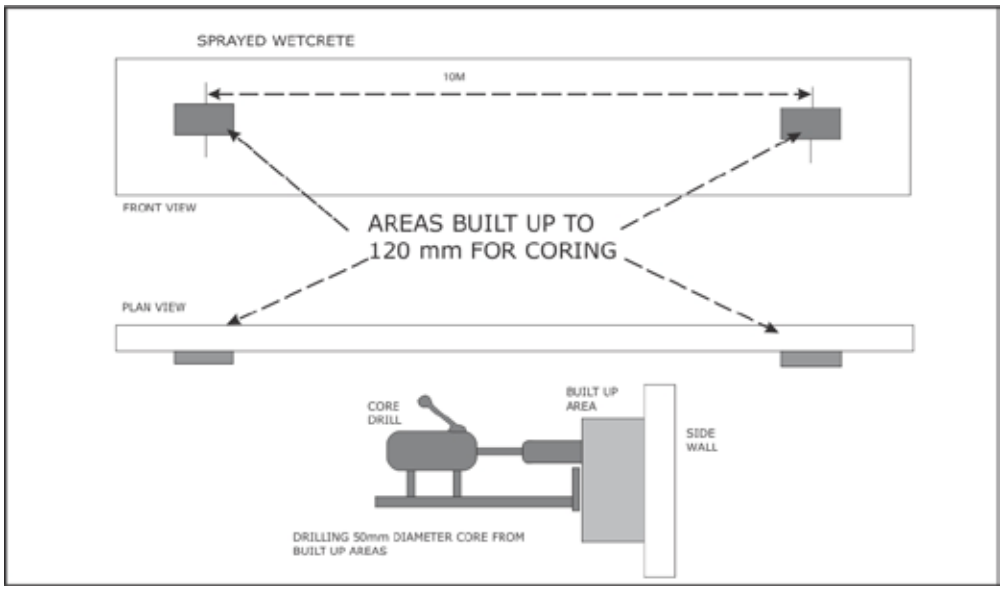


Figure 30: Shotcrete bond strength tests

Fibre content test

The purpose of this test is to determine fibre content within the applied shotcrete. On a weekly visit to the spray site, the quality controller should carry out the following procedure:

- an area recently sprayed must be picked and a grab sample of the shotcrete (including the fibres) taken;
- this sample must be weighed;
- using an approximate density of concrete (±2200kg/m3), calculate the volume of the sample taken (weight measured divided by density);
- this sample is then placed in a sieve and washed of all fines;
- a magnet is used to pick out metal fibres and are then placed on the scale and weighed;
- using this weight of the fibres and the calculated volume sampled, the 'kg fibre per m3 of shotcrete' can be determined (weight of fibre divided by calculated volume of shotcrete sample);
- Compare this number to the fibre content specification.

NOTE

This procedure is for metal fibres. If fibres consist of non-metal materials, the shotcrete material (sand and cement) should be washed from the sample to ensure that only fibres remain behind to be weighed. The rest of the procedure remains appropriate.

The less fibre, the poorer the energy absorption characteristics of the shotcrete.

INTERESTING INFO

Tools for a quality controller

- Portable drill – must have own power source and able to drill to a depth of 200 mm
- Core drill – stand mounted for ease of attaching to rock face and able to cut core up to 50mm diameter to a depth of 150mm
- Scales – Portable and marked in grams up to a limit of 2.5kg
- Strong magnet – to lift steel fibres
- Steel sieve – to wash out fines from mix

back

Testing frequency
Preliminary testing, proof of mix design

- In order to prove the mix design, the following tests should be carried out by an independent laboratory:
- Three compressive strength tests (three cube per test) at seven days and three compressive strength test (3 cube per test) at 28 days.
- Three EFNARC slab test results.
- The above test results shall be derived from three different lots.
- Ten bags of product for the determination of other parameters, as indicated in the table below.

Test description	Number of tests
Grading analysis	5
Relative density of aggregate	3
In-situ moisture content of aggregate	10
Mass of aggregate	10
Mass of fibre	10
Mass of cementitious material	10

Tolerances and variations

The permissible variations and tolerances applicable to any bagged product are indicated in table below.

Material	Variations and tolerances
Constituent aggregates	2.5% of specified mass
Blended aggregate	2.5% of specified mass
Cement	2% of specified mass
Cementitious additions	2% of specified mass
Total cementitious material	2% of specified mass
Admixture	10% of specified mass
Steel fibre	5% of specified mass
Polypropylene fibre	10% of specified mass
Compressive strength	-0 to +8.5MPa of specified
In-situ core strength	67% of the specified compressive strength determined under laboratory conditions by cubes
Energy absorption	Not less than that specified

MESH
Technical specifications

The following technical specifications are dependent on the products evaluated and are provided as indications only.

- Welded wire mesh
  - Approximate tensile strength (dependent on product) – 330kN
  - Weld shear strength – 155kN x wire cross sectional area (m2)
  - Ends cut flush for safe handling

back

- Galvanized wire available
- T (thickness) – variable and dependent on strength required;
- W (sheet width) – approximate 1.2m to 2.4m;
- L (sheet length) – approximately 1.2m to 7m, or even in rolls of 30m;
- S (gird size or wire spacing) – variable (as shown).

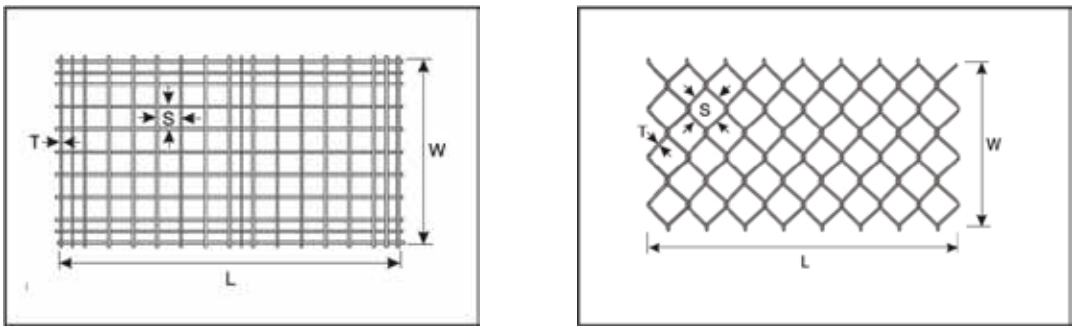


Figure 31: (a) Welded and chainlink (b) mesh specifications

- Chainlink mesh
  - Galvanised wire available
  - T (thickness) – variable and dependent on strength required;
  - W (sheet width) – approximate 1.2 to 3.0m;
  - L (sheet length) – in rolls of 7.5 to 30m;
  - S (grid size or wire spacing) – variable, but approximately 5 to 7.5cm.

CONNECTION 29

See DSI Mesh Brochure

Support unit quality control
Manufacturers test results on steel strength.
Corrosion (visual). Weld integrity on welded mesh.
Covered storage to protect against corrosion.

Installation procedure

- The meshing should cover the hanging and the sidewalls of the tunnel.
- The mesh should be cut in a length of the circumference of the tunnel and should be installed perpendicular to the direction of the tunnel, and not as a length in the direction of the tunnel.
- No holes or gaps in the meshing are acceptable.
- The mesh must overlap 20cm, or at least two grids of mesh (S in Figure 31), below the lowest line of lacing wire, on the sidewalls and in the direction of the tunnel.

CONNECTION 30

See DSI Welded Mesh Brochure

Methods to ensure installation quality
Visual inspection to ensure contact with rock surfaces and overlapping of mesh grids.

RESIN
Technical specifications

Technical specifications are dependent on the products, but the following are provided as an indication of general resin capsule specifications:

- Aging: Cured resin is resistant to acids and alkalis and no significant weakening with time occurs, even when completely submerged in water.

- Heat and cold resistance: Cured resin is completely resistant to cold, but heat only to 100°C.
- Strength: In general, the cured resin is stronger than most rocks, but custom strengths are available to the user;
- The products are available in custom lengths and diameters;
- Products, in general, consist of resin grout and a catalyst, separated by a thin plastic film until mixed (Figure 32).

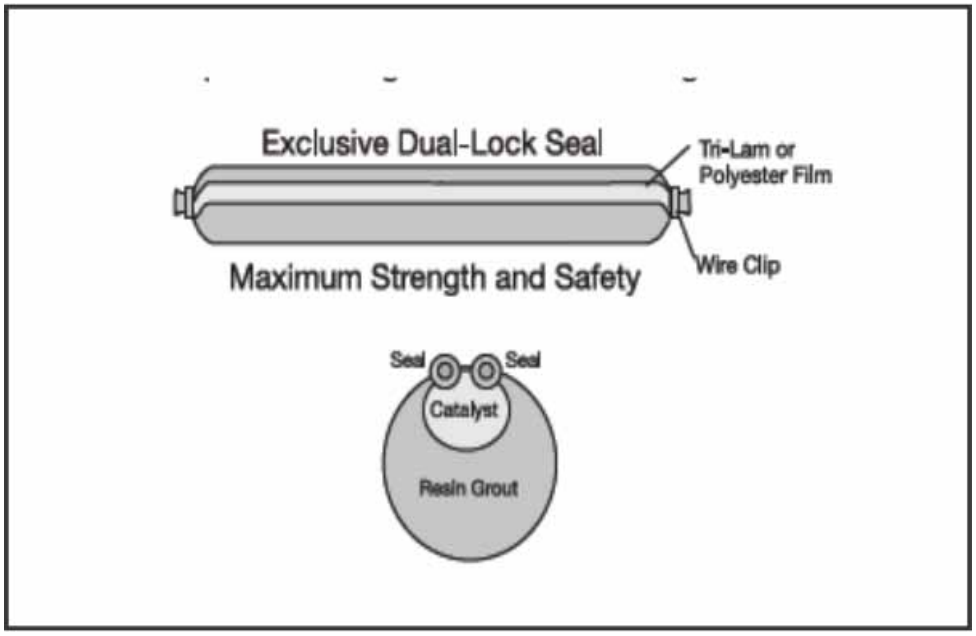


Figure 32: Resin capsule

Diameters (mm)		Cap. Dia	Cap. Length	Cap. Volume	Cap. / box
Hole	Bolt	(mm)	(mm)	(cm3)	
22-24	16	19	380	108	60
22-24	16	19	500	142	60
23-26	18	21	500	173	50
25-28	20	23	500	208	50
27-32	20	25	400	196	40
28-32	20	25	450	221	40
29-32	20	25	500	245	40
30-936	25	28	500	308	28
34-40	25	32	400	321	25
37-42	25	35	400	384	20
37-45	25	35	400	384	20

Table 7: Resin capsule information

NOTE

All resin capsules used must conform to SABS 1534:1991, the SABS Quality certificate. The expiry date shall be clearly marked on each box of resin. Time expired resin shall not be used under any circumstances.

Load deformation characteristics

The strength of resin products is described using information available for a certain product. The user must be advised that results for different products could be different, but that the trends indicated below should be very similar.

Resin strength (compressive) increases as curing time increases (Figure 33), indicating that full design strength may only be reached after some time has elapsed after installation. This must be taken into account when choosing and installing resin capsules.

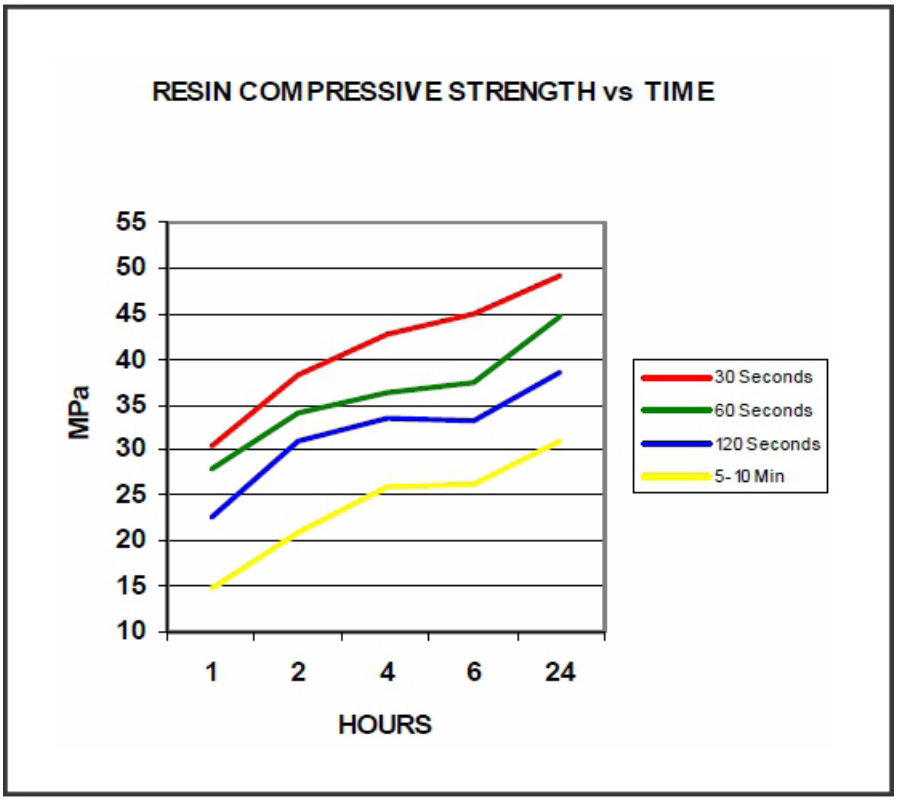


Figure 33: Resin strength (Courtesy of Minova)

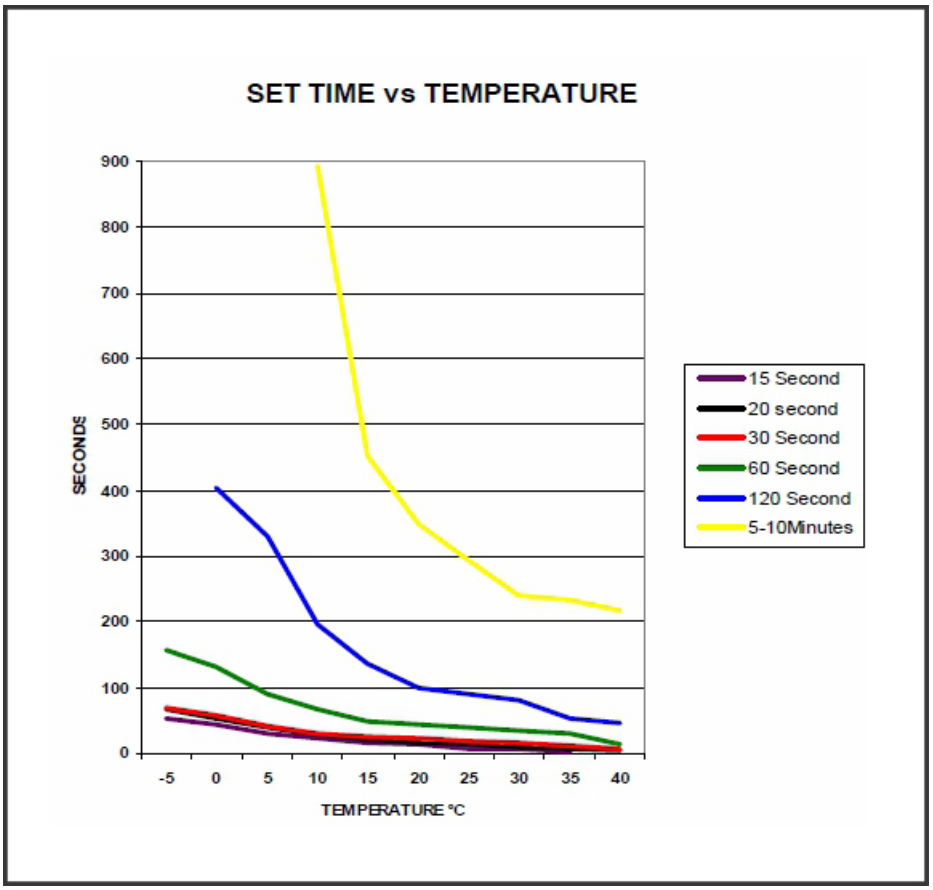


Figure 34: Setting time as a function of temperature (Courtesy of Minova)

Setting time is known to be affected by the temperature in the area of installation (including air and rock temperatures). It is clear that higher

back

temperatures reduce the setting time significantly (Figure 34) and this must be taken into account during the design. Setting prior to full installation will reduce the bond strengths.

Support unit quality control

Manufacturers test results on compressive strength in relation to curing time.
Storage to protect against heat.
Samples are selected for independent strength tests.

Installation procedure

The installation/application instructions of resin capsules are:

- Insert the necessary resin capsules into the hole.
- Push the bolt up the hole to at least 300mm from the collar (Rotation of the bolt during this step is optional).
- Rotary drills: rotate the bolt rapidly (250-750rpm) for the specified mixing time, while pushing the bolt home. Percussion drills: rotate the bolt 40-60 revolutions.
- Hold for the required time (note effect of temperature on set times).
- Do not rotate again as damage to partially gelled resin may result. The period of strength development following setting will vary, depending on set time and product temperature. The bolt may be tensioned after the required hold time.

NOTE

For optimum results, it is important to ensure careful control of: hole diameter, i.e. bit size; hole length, i.e. length of drill rod; use of sufficient resin to anchor the desired length of the bolt to the surrounding strata; and a minimum of 500mm of resin bond is recommended to ensure long-term anchor strength of end-anchored bolts.

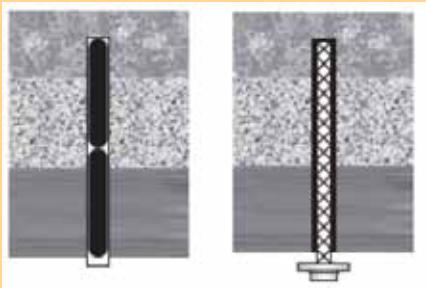
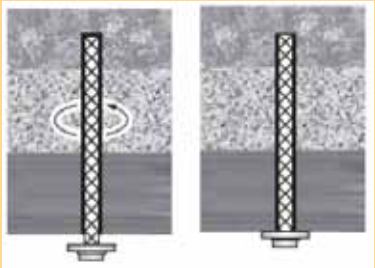
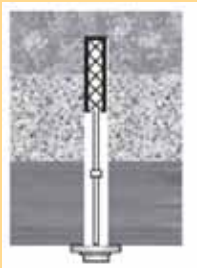
INTERESTING INFO

Setting times and mixing times are suggested in the table below (for Minova products – Courtesy of Minova) and can be used as a rough guide to design installations.

Colour code	Set time (at 20° C)	Mixing time (at 15 - 21° C)
Purple	15 s	10 s
Red	30 s	10 s
Green	60 s	15 s
Blue	120 s	20 s
Yellow	5-10 min	20 s

Table 8: Installation of a resin bolt – Courtesy of DSI

back

	1. Insert cartridge into the hole. 2. Push the bolt into the hole to a point just below the roof. Slow rotation of the bolt during insertion is optional.
	3. Rotate the bolt as indicated.
	4. Fully grouted bolts: Push the bolt upward with the maximum thrust available from the machine and hold until resin hardens. Do not rotate after step 3, as damage to partially set resin may result. 5. Tensioned and combination bolts: Hold the bolt until the resin has hardened. When hardened, rotate the nut until the desired torque is developed.

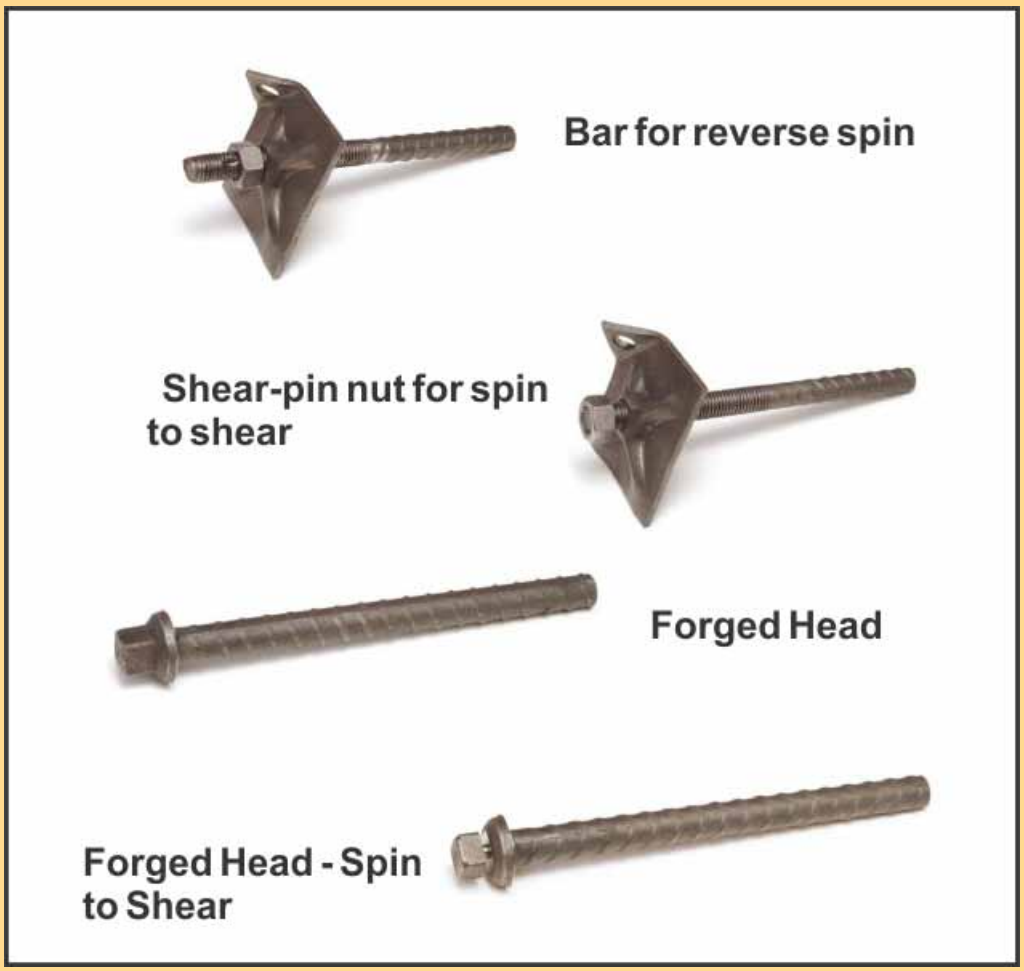
Methods to ensure installation quality

One in each ten bolts should be supplied with a device capable of visually indicating that an installation has been pre-tensioned to the required levels.

Resin setting and mixing times can be confirmed through on-site visual inspections.

Pull tests can be conducted to ensure sufficient anchorage is obtained.

Resin bolts are rolled bars formed to give superior support performance when used in conjunction with the latest resin bolting technology. The bar is designed in such a way that the nominal diameter of the bar approximates the pitch diameter of the thread, thereby allowing for maximum thread strength, in relation to nominal bar diameter.



Link to Safety in Mines Research Advisory Committee website www.mhsc.org.za to download the following report (users are advised that costs may be involved with downloading the report if the user is not part of a MHSC registered company):
Project Summary: SIM 02 0 205
Project title: An investigation into the support systems in South African collieries (214 p.).
Author(s): I. Canbulat, A. Wilkinson*, G. Prohaska*, M. Mnisi** and N. Singh***

GROUT
Technical specifications

One of the primary causes of rock bolt failure is corrosion and this can be counteracted by filling the gap between the bolt and the drill hole wall with grout. Grouting should be considered where the ground-water is likely to induce corrosion or where the bolts are required to perform a 'permanent' support function.

Although they are used to reduce corrosion, its primary function is to anchor tendons installed to support the rock around a mining excavation. Grout is available in pumpable or cartridge form and the type implemented is often the decision of the rock engineering practitioner. However, grout cartridges have the advantage that they are convenient to handle,

back

can easily include fast stetting ingredients and have better controlled setting times than pumpable grout. The grout cartridges come in variable lengths and diameters and can have the following specifications:

- Compressive strength:
  - 2 Hours: 12MPa
  - 4 Hours: 14MPa
  - 24 Hours: 21MPa
  - 3 Days: 35MPa
  - 7 Days: 43MPa
- Anchorage:
  - 2 Hours: 90kg/cm
  - 4 Hours: 320kg/cm
  - 12 Hours: 375kg/cm
  - 24 Hours: 540kg/cm
- Shelf life: 12 Months in a dry environment.

The specifications quoted are relevant to a specific product and information regarding the product used by the user must be gathered from the suppliers.

CONNECTION 31

See DSI Paper on Resin and grout

NOTE

CONNECTION 32

Refer to "Figure 18:" on page 296

Load deformation characteristics

Support unit quality control

Manufacturers test results on compressive strength in relation to curing time (if appropriate). Packing to protect against rain.
Installation procedure

With the use of pumpable grout, the traditional method of grouting up-hole rock bolts is to use a short grout tube to feed the grout into the hole and a smaller diameter breather tube, extending to the end of the hole, to bleed the air from the hole. The breather tube is generally taped to the bolt shank and this tends to cause problems because this tube and its attachments can be damaged during transportation or insertion into the hole. In addition, the faceplate has to be drilled to accommodate the two tubes.

The following procedure can be applied:

- Pre-use checks must be done on all critical items and it must be recorded. The contractor is responsible for ensuring that forms are created and available for all the relevant equipment. This includes, i.e. tensioning equipment.
- Grout must be a cement / water mixture with a consistency that will allow a handful of grout to remain sticking to an inverted-cupped hand for 10 seconds.
- The grout must not flow out of hangingwall holes.
- All holes must be completely filled with grout. When the tendon is installed, some grout must be forced from the hole; if not, the hole must be washed out and re-grouted.
- Tendons must be installed immediately after the holes have been grouted, and must all be installed perpendicular to the direction of the excavation.
- Anchor eyes (if tendons have eyes for lacing installation) must not protrude more than 100mm from the collar of the hole.

INTERESTING INFO

Sealing the system for grout injection can be a problem. Many of these difficulties are overcome by using a hollow core bolt. While more expensive than conventional bolts, these hollow bolts make the grouting process much more reliable and should be considered wherever permanent rock bolt installations are required. The grout should be injected through a short grout tube inserted into the collar of the hole and the central hole in the bolt should be used as a breather tube. When installing these bolts in down holes, the grout should be fed through the bolt to the end of the hole and the short tube used as a breather tube.

The grout should be readily pumpable without being too fluid and a typical water/cement ratio of 0.4 to 0.5 is a good starting point for a grout mix for this application. It is most important to ensure that the annular space between the bolt and the drill hole wall is completely filled with grout. Pumping should be continued until there is a clear indication that the air has stopped bleeding through the breather tube or that grout is seen to return through this tube

back

back

- Tendons that stick before reaching the end of the hole must be withdrawn. The hole must be washed out and cleared of any obstruction.
- The temporary support that has been installed must not be removed before the cement has set, i.e. after 24 hours, under the immediate supervision of the appointed ganger.
- The Lategan compressed air grout pump is used to place OPC grout into support holes.
- All grouting pumps and hoses must be cleaned thoroughly at the end of the shift. Pumps must be serviced six monthly.
- If holes are not full column grouted, tendons cannot function as a support unit and can therefore not pass quality control for payment.

When grout capsules are used.

- These capsules must be stored in a dry place until required.
- Only the required amount of grout capsules must be immersed in water to avoid wastage.
- Capsules must be immersed in clean water in a separate bucket provided for this purpose.
- When the bubbling has stopped, the capsules must be removed and used for grouting purposes immediately.
- Holes must be grouted with equipment supplied and approved by management.
- Grouting operators must wear eye protection while grouting holes.
- The support holes must be filled completely with grout (full column grouting).

CONNECTION 33

Also refer to "Chapter 8" on page 572"

Methods to ensure installation quality

Samples selected for independent strength tests.
Pull tests are conducted.

INTERESTING INFO

The MHSA states that:

1. Any person who:

   a. designs, manufactures, repairs. imports or supplies any article for use at a mine must ensure, as far as reasonably practicable--

   i. that the article is safe and without risk to health and safety when used properly; and

   ii. that it complies with all the requirements in terms of this Act;

   b. erects or installs any article for use at a mine must ensure, as far as reasonably practicable, that nothing about the manner in which it is erected or installed makes it unsafe or creates a risk to health and safety when used properly; or

   c. designs, manufactures, erects or installs any article for use at a mine must ensure, as far as reasonably practicable, that ergonomic principles are considered and implemented during design, manufacture, erection or installation.

2. Any person who bears a duty in terms of subsection (1) is relieved of that duty to the extent that is reasonable in the circumstances, if:

   a. that person designs, manufactures, repairs, imports or supplies an article for or to another person; and

back

b. that other person provides a written undertaking to take specified steps sufficient to ensure, as far as reasonably practicable, that the article will be safe and without risk to health and safety when used properly and that it complies with all prescribed requirements.

3. Any person who designs or constructs a building or structure, including a temporary structure, for use at a mine must ensure, as far as reasonably practicable, that the design or construction is safe and without risk to health and safety when used properly.

Link to Guideline Procedure for support quality assurance and tests on Elongates, Packs, tendons, mechanical props etc

REFERENCE

- Brady BHG & Brown ET (1993) Rock Mechanics for Underground Mining
- Chapman & Hall Chapter 11
- Kuijpers JS, Rangasamy T, Sellers EJ Performance and Design of Thin Spray Liners as Rock Support in Mine Tunnels SIMRAC *
- Minova RSA Product information
- Ryder JA &Jager AJ (1999) A Handbook on Rock Mechanics for Tabular Hard Rock Mines SIMRAC Chapter 6
- Ryder JA &Jager AJ (2002) A Textbook on Rock Mechanics for Tabular Hard Rock Mines SIMRAC Chapter 7
- www.rocscience.com/hoek/pdf/16_Shotcrete_support.pdf

INTERESTING INFO

Also see
Support Catalogue

LEARNING OUTCOME 5.1.9

DETERMINE APPROPRIATE SUPPORT TYPES, SUPPORT SIZES AND SUPPORT PATTERNS GIVEN TUNNEL LAYOUTS, ROCKMASS CONDITIONS AND STRESS REGIMES

The aim of the design of support for tunnels and service excavations is to create a zone of stable rock around the excavation. Stable means that the amount deformation is limited by the support and the deformation of the rock takes place in a controlled way. The zone of stable rock is created by increasing or maintaining the friction between the rock fragments in the zone.

The length of the reinforcing support must be sufficient to create a thick enough zone in relation to the size of the excavation and the spacing between the supports small enough to ensure the integrity of the zone. The spacing is also a function of the mode rock fragment size.

Where the rock fragment sizes are small enough, or the rock so weak that fallout between reinforcing support is still likely, areal support such as shotcrete or wire mesh is added.

The type and intensity of the support designed also depend on the functional importance of the excavation and the number of people at risk. The safety factor in the design is therefore varied according to the risk factor.

Empirical design methods are often applied, i.e. the use of designs that have proved to be effective in similar conditions in the past.

Four approaches to the evaluation of support requirements are given below:

- Precedent practice: the determination of support requirements in a mine, based on what has worked well in the past, is a very pragmatic approach. It also has the advantage that mine personnel are familiar with the support elements and the methods of support installation. It is advisable that the support is reviewed, however, to ensure that it is appropriate, that safety is not compromised, and conversely that the rock is not being over-supported. Re-evaluation of the support methods is particularly applicable if geotechnical conditions change, and if new or changed mining methods are used.
- Rock mass classification: the use of rock mass classification techniques to determine rock support requirements has now been practiced for many years and is a sound approach. It is important that the following points be noted in this regard:
  - rock mass classification techniques are applicable to rock mass quality. In their standard form, they have limited application to massive rock and to stratified rock with limited cross jointing;
  - since they are based on observed quality, the support derived from them in their standard form is conservative. This is satisfactory for civil engineering applications, but not for mining applications;
  - since they are applicable to general rock mass quality, they do not take into account particular mechanisms of rock instability. It is therefore important that possible instability mechanisms are well understood to ensure that the support indicated is appropriate.
  - Notwithstanding these reservations, rock mass classification provides a very quick and convenient method of determining support requirements.
- Kinematic analysis: the determination of support requirements for blocks and wedges follows directly from the evaluation of stability of such blocks and wedges.
- Stress analysis: Excellent computer programmes for stress analysis are commercially available. Two that are very commonly used for geotechnical problems are FLAC (Itasca, 1999) and Phase2 (Rocscience, 1998), and both packages can handle the modelling of rock support

CONNECTION 34

Also refer to "LEARNING OUTCOME6.4" on page 497

In general, the design of a support system must cater for the anticipated mechanism of rock mass instability and the interaction of the elements within a support system with the rock mass. The interaction will be controlled by how the rock mass behaves and the loading conditions. Each tunnel should be considered as an individual design problem and treated accordingly.

The **structure of the rock mass** and the conditions of the discontinuities will determine the extent of instability around the excavation (determines choice of tendon length), and the interaction of the tendons within the rock mass (spacing). The structure of the rock mass is determined by the relative spacing and orientation of the natural discontinuities (joints and bedding) and any superimposed fracture patterns, which together create the blocky nature of the rock mass. The poorer the integrity of the rock mass, the greater the potential of instability and therefore the poorer the interaction between support tendons and the rock mass. Therefore, the presence of abnormally poor ground conditions will necessitate a careful evaluation of the design of the support system.

The **orientation** of the tendon installation relative to the orientation of the discontinuity within the rock mass will also influence the extent of interaction between the elements of the support system. Tendons installed sub-parallel to the axis of a discontinuity greatly reduce the extent of the support interaction. This will increase the instability between the tendons and the rock mass will not be adequately reinforced. In order to compensate for this, a reduction in the tendon spacing in order to increase the areal coverage is an important requisite. However, in highly discontinuous rock masses where there is a tendency for the rock mass to unravel between the support elements, there is a need for an aerial coverage (fabric) support system to be designed to minimise the effects of the unravelling process and to maintain the integrity of the rock mass.

The **stress environment** will determine if the stability of an excavation will be controlled by the geotechnical structures or by stress-induced fracturing. In low-stress environments, such as shallow depth excavations or those developed above or below a mined-out area, the low clamping stresses may result in the loosening of the rock mass in the immediate walls of the excavation. A tendon system can therefore be designed based on the static analysis of the support capacity in relation to the unstable rock mass. The key block analysis of the immediate rock mass will be required in order to determine the design procedure with respect to a suitable factor of safety. In many cases, the use of very stiff or active support elements is required.

A change in the stress field will also influence the stability. This may result from an increase in the extent of fracturing around a tunnel due to an increase in stress, or induced shearing or dilation within the excavation walls due to a large reduction in one of the stress components. A general unravelling of the rock mass may be induced rather than individual key block fall outs. Where the anchorage of the tendon elements is in excess of the depth of potential instability, the support design may be based on the uniform confinement of the unstable rock mass volume.

The influence of **seismicity** on the tunnel support design will be dependent on the mechanism of interaction of the dynamic stresses with the tunnel peripheral rock mass. This influence varies from seismically-induced falls of ground to dynamic expulsion of blocks to the initiation of failure within the peripheral rock mass. Design of the support system should consider the extent of interaction of the support elements with the rock mass and their load-deformation capacity as well as taking into account the degree of dynamic loading and deformation to be expected as well as the overall rock mass condition.

Where an abnormal (non-average) ground condition is encountered, or where an excavation has unique dimensions, site-specific support layouts are designed by the rock engineering practitioner. These are normally done on planning of a mining layout for a specific excavation.

Various **rock mass classification schemes** have been identified with respect to support designs.

Schemes such as those developed by Barton et al. and Bieniawski were developed as methods of estimating support requirements for underground excavations.

Using precedent practice, based on their **tunnelling quality index** (Q) and the excavation support ratio, ESR, a chart was established that categorises various support requirement categories. Figure 35 is a support chart derived by Barton et al. that allows decision-making on the type of support, spacings and lengths.

back

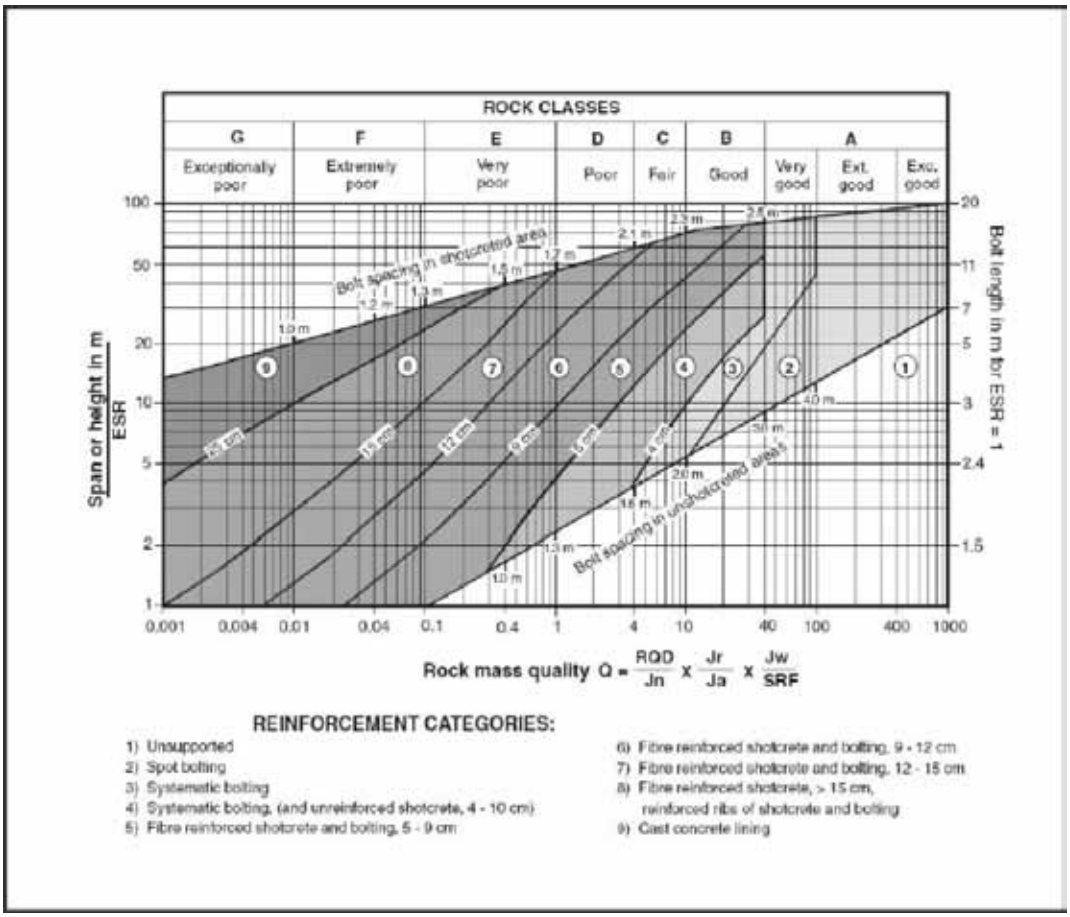


Figure 35: Estimated support categories based on the tunnelling quality index Q (After Grimstad and Barton, 1993, reproduced from Palmstrom and Broch, 2006)

The length L of rock bolts can also be estimated from the excavation width B and the Excavation Support Ratio ESR: L = (2 + 0.15B) / ESR.

The maximum unsupported span can be estimated from:

Maximum span (unsupported) = 2.ESR. $Q^{0.4}$

Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure (Proof) is estimated from:

$$\text{Proof} = (0.2\ Q^{-0.33}) / J_r$$

while the support spacing can be estimated from:

$$S = (C/\text{Proof})^{0.5}$$

where C is the maximum loadbearing capacity of the tendon (MN).

EXAMPLE

For the support recommendations specific to production excavations, the Q-ratings and ESR values quoted in Table 9, subjected to conditions in Table 10, equate to support requirements shown in Table 12 and Table 13.

back

Excavation Information			
Nr	Excavation	ESR	Excavation width (mm)
1	Decline / ramp	1.6	5
2	Footwall Drive	2	5
3	Drill Drive	5	5
4	Drawpoint crosscut	5	5
5	Drawpoint brow	5	5
6	Drill Drive in backfill	5	5

Table 9: Production excavations

Excavation Information		Condtions					
Nr	Excavation	Q Rating	Life of Excavation	Orientation	Water	Rock Type	Wedge Creation
1	Decline / ramp	2.1	Long Term	Yes	Limited	FW	Yes
2	Footwall Drive	2.1	Medium Term	Shear Parallel	No	FW	Yes
3	Drill Drive	1.9	Short Term	Shear Perpen-dicular	No	Ore	Yes
4	Drawpoint crosscut	2.1	Short Term	Shear Parallel	No	FW	No
5	Drawpoint brow	2.1	Short Term	Shear Parallel	No	FW	No
6	Drill Drive in backfill	1.9	Short Term	Shear Parallel	No	Ore / Fill	No

Table 10: Expected conditions in production excavations

Primary support

Primary support is installed to maintain excavation perimeter stability in excavations during the development phase, but may also be the only support installed in some excavations forming the permanent support for the life of the specific excavation. Primary support requirements are based on excavation dimensions, rock mass quality and expected conditions, as well as the possibility of encountering wedges, created by joints.

The installation of the following primary support is generally required in production excavations:

- a maximum split set spacing of 1.4m apart in all host rock excavations and 1.3m in excavations situated within the ore body;
- 2.4m split set lengths in declines and ramps with 2.0m lengths in all other excavations;
- All split sets in medium- and long-term excavations should be galvanised and grouted;
- Mesh should be used as areal coverage support in medium- to long-term excavations;
- Drill drives developed within paste filled stopes will have ore body material roofs, but sidewalls will consist of footwall material and cemented paste fill. Support similar to the ore body drives are maintained for the whole excavation as it is assumed that the paste fill will have no cold joints and will therefore be massive. If scaling of the walls occurs during blasting, the use of mesh to contain the walls is suggested.

The presence of wedges created by jointing in the excavation roofs suggests that tendons of sufficient length to be able to support these wedges are essential. An analysis of wedges created by jointing in north-south

back

striking drives in the ore and footwall material was executed using the Unwedge software package and yielded the following results:

- 5m ore and footwall drives are prone to form wedges with apex heights of maximum 0.5m
- Wider (>10m) drives have the potential to form wedges of 1.9m apex heights.

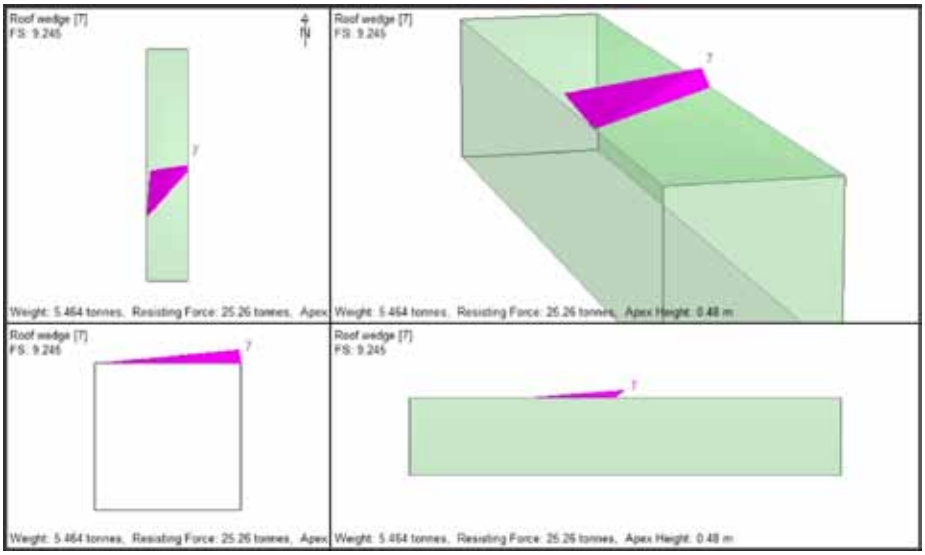


Figure 36: Hangingwall wedge dimensions

The installed 2.0m long tendons have historically maintained stability of the wedges in standard 5m wide tunnels and have sufficient bond length beyond the possible wedge heights. The suggested support pattern should also be utilised in the excavation sidewalls to limit sidewall scaling and should be installed at least as low as the grade line.

Life ERx-pect-ancy of Exca-vation	Foot-wall or ore material	ESR	FOS	Q-index range	Exca-vation Dimen-sion	Tendon Length	Tendon Capac-ity	Support Pres-sure	Spacing (m)	
					m	m	MN	mPa	Square Pattern	Rectangu-lar Pattern
Tempo-rary	Ore Body	5	1.2	1.9 - 2.5	5	0.7	0.160	0.108	1.2	1.0 x 1.5
Me-dium Term Exca-vaions	Footwall	2	1.2	2.1 - 2.7	5	1.7	0.160	0.104	1.2	1.0 x 1.5
Long Term Exca-vations	Footwall	1.6	1.2	2.1 - 2.7	5	2.1	0.160	0.104	1.2	1.0 x 1.5

Table 11: Calculations utilising the Q Index as per Barton et al.

back

Excavation Information		Primary Support								
Nr	Exca-vation	Sup-port Unit Capacity (Strength)	Support Pressure (MN/m^2)	Spacing (mm)	Tendon Length mini-mum (m)	Recom-mended std length	Aerial Coverage	Pre-load-ing	Corrosion Resistant	Type
1	Decline / ramp	0.196	0.104	1.4	2.1	2,4	No	No	Yes	ABCF
2	Foot-wall Drive	0.196	0.104	1.4	1.7	2.0	Yes	No	Yes	ABCF
3	Drill Drive	0.196	0.104	1.3	0.7	2.0	No	No	No	C
4	Draw-point crosscut	0.196	0.108	1.4	0.7	2.0	Limited	No	No	C
5	Draw-point brow	NA	NA	NA	NA	NA	Yes	No	No	BCF
6	Drill Drive in backfill	0.196	0.108	1.3	0.7	2.0	No	No	No	C

A	Galva-nised
B	Grouted
C	Splitset
D	Resin Bar
E	Ripple Bar
F	Mesh

Table 12: Primary support per excavation

Secondary support

Secondary support installation is essential in many of the footwall access excavations due to expected life span, excessive excavation widths, wedge creation in the excavation roofs and poor, blocky ground conditions. Cable anchors are suggested to cater for the creation of joint bounded wedges in the larger span breakaway areas. At the same time, the use of shotcrete as an areal coverage fabric support is suggested for some permanent excavations (Table 13).

Excavation Information		Secondary Support					
Nr	Excavation	Aerial Coverage	Pre-loading	Corrosion Resistant	Type 1	Type 2	Additional Comments
1	Decline / ramp	No	No	Yes	GH	ACDEJ	GH in poor rock mass conditions ACDEJ 6m <excavation width<8m Sidewall>8m
2	Footwall Drive	Yes	No	Yes	KI		
3	Drill Drive	No	No	No	K	L	L in poor rock mass conditions where areal coverage is required
4	Drawpoint crosscut	Limited	No	No	K		
5	Drawpoint brow	Yes	No	No	DK		Last 4m of Drawpont crosscut only
6	Drill Drive in backfill	No	No	No	KI		Mesh alongside wall if fill scaling occurs
					A	4.5mm	
					B	6.0mm	
					C	End_anchored	
					D	grouted	
					E	cable	
					F	25mm	
					G	50mm	
					H	Shotcrete	
					I	Mesh	
					J	38 tonnes	
					K	Double up pattern splitsets	
					L	Superskinliner	

Table 13: Detailed secondary support requirements

Bienawski utilised the Rock Mass Rating (RMR), indicating the quality of the rock mass, to suggest support requirements (Table 14) for underground excavations in which support types, lengths and spacings are suggested for specific excavation types.

back

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

Table 14: Guidelines for excavation and support of 10 m span rock tunnels in accordance with the RMR system (After Bieniawski 1989)

An additional method of determining the support requirements for tunnels underground is by means of evaluating the Rockwall Condition Factor (RCF). This method is based on the quality assessment of the rock mass as well as stress levels to which the excavation is subjected. It has been calibrated for Witwatersrand gold mines. It is not calibrated for the Bushveld or other mining districts. It is often erroneously used outside the Witwatersrand gold mines.

In evaluating the proposed development, the σ1 critical criterion is widely used and is based on a critical stress value. Should this critical value be exceeded, the sustainability of the excavation cannot be guaranteed and the support installation will be significant. In evaluating the stability of tunnels, the Rockwall Condition Factor (RCF) is commonly used (Jager and Ryder, 1999).

$$RCF=Q_{\theta max}/F\sigma_c$$

where $Q_{\theta max}$ is the maximum tangential stress or stress concentration on an excavation,
F is an empirical factor representing the condition of the rock mass and
σ_c is the intact rock strength of the rock material.

The value of $Q_{\theta max}$ is calculated for an excavation with a circular cross-sectional shape as follows:

$$Q_{\theta max}= 3\sigma_{1i}-\sigma_{3i}$$

where $\sigma 1_i$ and is $\sigma 3_i$ are the major and minor principal stresses in the plane perpendicular to the axis of the circular excavation.

This is not truly representative of the more typical rectangular cross-sections applied in practice, but this method has been used empirically to determine support requirements for hard rock tunnels in deep mines, particularly within Witwatersrand rocks. Jager and Ryder state that the

back

back

value of F does not differ significantly from unity for strong quartzitic rock and can be as low as 0.5 for a highly discontinuous rock mass.

A set of empirical support recommendations is suggested based on the RCF value obtained (Jager and Ryder, 1999). In general, it was found that for:

- RCF < 0.7 good ground conditions prevail with minimum support (Table 15);
- 0.7 > RCF < 1.4 average conditions prevail with typical support systems (Table 16) and for
- RCF > 1.4 poor ground conditions prevail with special support requirements (Table 17).

Case	Primary support (typical requirements)	Secondary support (typical requirements)
Static stress conditions	'Spot' support where necessary (Friction bolt, split sets, rock studs. Etc)	'Spot' support where necessary (Friction bolts, split sets, rock studs. Etc)
Stress changes anticipated	'Spot' support where necessary (Friction bolts, split sets, rock studs. Etc)	Fully grouted tendons/friction bolts >1.5m in length installed on a basic 2m pattern. Support resistance: 30-50kN/m2. Welded mesh (sidewall only)
Seismic activity anticipated	Friction bolts or tendons > 1.2m in length, installed as close to the face as possible, on a basic 2m or 1.5m pattern. Support resistance: 30-50kN/m^2.	Fully grouted (pref. yield) tendons/friction bolts >1.5m in length installed on a basic 2m pattern. Support resistance: 50kN/m^2. Welded mesh (hangingwall and sidewalls).

Table 15: Support recommendations for good ground conditions (RCF<0.7) (by Ryder JA & Jager AJ)

Case	Primary support (typical requirements)	Secondary support (typical requirements)
Static stress conditions	Friction bolts or tendons > 1.5m in length, installed as close to the face as possible, on a basic 2m or 1.5m pattern. Support resistance: 30-50kN/m^2.v	Welded mesh integrated with primary support tendons/friction bolts.
Stress changes anticipated	Fully grouted tendons or steel ropes >1.8m in length installed as close to the face as possible, on a basic 2m or 1.5m pattern. Support resistance: 40-60kN/m^2	Welded mesh integrated with primary support tendons/friction bolts.
Seismic activity anticipated	Fully grouted (pref. yield) tendons or steel ropes >1.8m in length installed as close to the face as possible, on a 1.5m or double 2m pattern. Support resistance: 80-100kN/m^2.	Welded mesh integrated with primary support tendons/friction bolts, plus optional shotcrete.

Table 16: Support recommendations for average ground conditions (0.7<RCF<1.4) (by Ryder JA & Jager AJ)

back

Case	Primary support (typical requirements)	Secondary support (typical requirements)
Static stress conditions	(If necessary) shotcrete to face, then fully grouted tendons/friction bolts or steel ropes, length > 1.8m on a basic 1.5m pattern, as close to the face as possible. Support resistance: 80-110kN/m^2.	Support resistance: 80-110kN/m2.
Stress changes anticipated	(If necessary) shotcrete to face, then fully grouted steel ropes or yielding tendons /friction bolts, length >1.8m on a basic 1m or double 2m pattern, as close to the face as possible. Support resistance: 120-230kN/m^2.	Welded mesh integrated with primary support tendons/ friction bolts. Add integral shotcrete in long-life tunnels. If necessary, additional hangingwall support comprising grouted steel ropes.
Seismic activity anticipated	(If necessary) shotcrete to face, then fully grouted steel ropes or yielding tendons / friction bolts, length >2.3m on a basic 1m pattern, as close to the face as possible. Support resistance: 80-100kN/m^2.	Welded mesh integrated with primary support tendons/ friction bolts. Add integral shotcrete in long life tunnels. If necessary, additional hangingwall support comprising grouted steel ropes.

Table 17: Support recommendations for poor ground conditions (RCF>1.4) (by Ryder JA & Jager AJ)

REFERENCE

- Brady BHG & Brown ET (1993) Rock Mechanics for Underground Mining
- Chapman & Hall Chapter 11
- Ryder JA &Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines SIMRAC Chapter 6
- Stacey TR (2001) Best Practice Rock Engineering Handbook for "Other" Mines

LEARNING OUTCOME 5.1.10

DESCRIBE, CHARACTERISE AND DISCUSS SPECIFICATIONS FOR THE VARIETY OF SUPPORT TYPES.

When considering support requirements for different excavations, knowledge of the types of support elements commonly used in the industry is important to be able to measure their appropriateness to the conditions found/expected in the excavations.

CONNECTION 35

The following should be read in conjunction with information given in "LEARNING OUTCOME 5.1.7" on page 279 and "LEARNING OUTCOME 5.1.8" on page 293

- Mechanical, end-anchored rock bolt: Active support. Quick to install. Prone to anchor slip, particularly due to blast vibrations and in weaker rocks. Allows shear movements on block surfaces to take place. This, with anchor slip and hence loss of tension, allows rock mass to loosen.
- Grout (resin or cement) end-anchored rock bolt: Active support if preloaded, passive if not, but in general stiff support. As for above, except that anchor slip is less likely. Tensioning will reduce the yield capacity (yield range).
- Fully-grouted rock bolt: Stiff support. Well-grouted bolts will have limited extensibility in tension and shear if movement takes place on joints. Smooth bar has much improved yield performance. Usually untensioned. Friction rock bolts: usually quickly installed. Performance dependent on hole size. Owing to thinness of steel section, can be prone to corrosion. Connecting arrangements are weak.

back

- Yielding rock bolt: Stiff support. Fully grouted, provides good performance in both tension and shear. Maintains load capacity for significant yield (say 300mm).
- Tensioned cables: Active support and stiff when fully grouted. Used when significant support pressures are required.
- Diamond mesh (chain link mesh): Passive, soft support. Flexible surface support. Prone to unravel when one strand is broken. Not easy to install due to its floppiness. Because of this, easy to shape to a variable rock surface, but conversely, difficult to keep against the rock surface. Floppiness and the fact that it often is not up against the rock inhibit shotcreting quality (vibration of mesh, air pockets behind mesh), particularly for aperture less than 100mm. Good reinforcement for shotcrete (tough and yielding).
- Weld mesh: Passive, stiff support. Reasonably easily handled, but due to its stiffness, difficult to mould to the rock surface. Conversely, does not flop away from rock surface. Good containment capability, but strands and welds are prone to "brittle" failure. Allows higher quality shotcreting.
- Tendon straps: Passive, most often stiff support. Flexible, easily installed surface support, with good yield capability. Additional straps can be added as required if suitable rock bolts have been installed.
- Wire rope lacing: Passive, soft support. Laced over wire mesh, lacing is a proven surface support capable of containing major yielding of rock mass.
- Shotcrete: Passive, stiff support. Good surface support when used with mesh or fibre reinforcement. Shotcrete will crack after small deformations and further performance is dependent on the reinforcement. Diamond mesh performs better than weld mesh as reinforcement i.t.o. yieldability. Steel and monofilament polypropylene fibres can both perform well – the longer the fibres, the greater the yield capability. The performance of steel fibres in corrosive conditions is suspect. Shotcrete provides protection to other support elements against mechanical damage.
- Steel arches: Stiff, passive support applicable in particular circumstances such as rehabilitation. Allows rock mass to loosen, particularly if arches are not well blocked. Quality of arch installation is often poor, with arch legs not being satisfactorily founded or supported.

REFERENCE

- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines Chapter 6
- Stacey TR (2001) Best Practice Rock Engineering Handbook for "Other" Mines SIMRAC Chapter 4

LEARNING OUTCOME 5.1.11

DESCRIBE, CHARACTERISE AND DISCUSS SPECIFICATIONS FOR THE VARIETY OF SUPPORT SIZES

The support tendon length will determine the extent of interaction with the rockmass. In moderately stressed environments, the support design is generally based on the concept of 'anchoring' the potentially unstable rockmass around a tunnel to the deeper, more competent, stable rockmass. The length of the tendon must be sufficient to accommodate the depth of instability and provide suitable anchorage in excess of this depth.

back

The required length of rock bolts is usually a function of the dimensions of the opening. As a rule of thumb, for reasonable rock conditions, the length of bolts should be 0.33 times the span or the wall height. For very good rock mass conditions, this multiplier could be as low as 0.25, and for a poor quality rock mass, could be as high as 0.5 (or even higher). Long cable units may be required in larger excavations, and typical lengths would be 0.4 times the span or the wall height.

When the hangingwall geometry results from stratification, its stability will depend on the stability of the beam of hangingwall rock. The length of rock bolts and cables used for stabilisation should ensure that the thickness of the stable beam is achieved. This length must be sufficient to ensure adequate anchorage in the stable rock above the defined beam thickness.

All tendon support types are available in lengths that can be determined by the rock engineering practitioner, even though standard lengths are more easily available. Designed lengths can either be enforced by the practitioner or the next standard length (longer than the designed length) can be selected.

At the same time, the tendon diameter mostly determines the tendon's tensile strength. As the diameter increases, so does the tensile strength, assuming constant steel strength is used in the manufacturing (refer to Table 4 for same effect in cable bolts).

REFERENCE

- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines
- Hutchinson & Diedericks Cable bolting in underground mines

LEARNING OUTCOME 5.1.12

DESCRIBE, CHARACTERISE AND DISCUSS SPECIFICATIONS FOR THE VARIETY OF SUPPORT PATTERNS

Support patterns are derived from the comparison of support 'needed' to the support 'provided'. In essence, this means that if a certain support resistance is required to maintain stability, the pattern and support unit capacity determines how much support resistance can be provided by the pattern of tendons.

DEFINITION

Support resistance is the load or force, expressed in Newton (N), generated by a support unit or system per tributary area, which is normally taken as 1m^2

NOTE

Any pattern of tendons installed in an excavation can be expressed as force (load) / m2. Based on this, a pattern of tendons installed 2.0m apart in all directions (Figure 37a), and each generating a force of 100kN, will provide a support resistance of 100kN / (2 x 2)m2 = 25 kN/m^2.

back

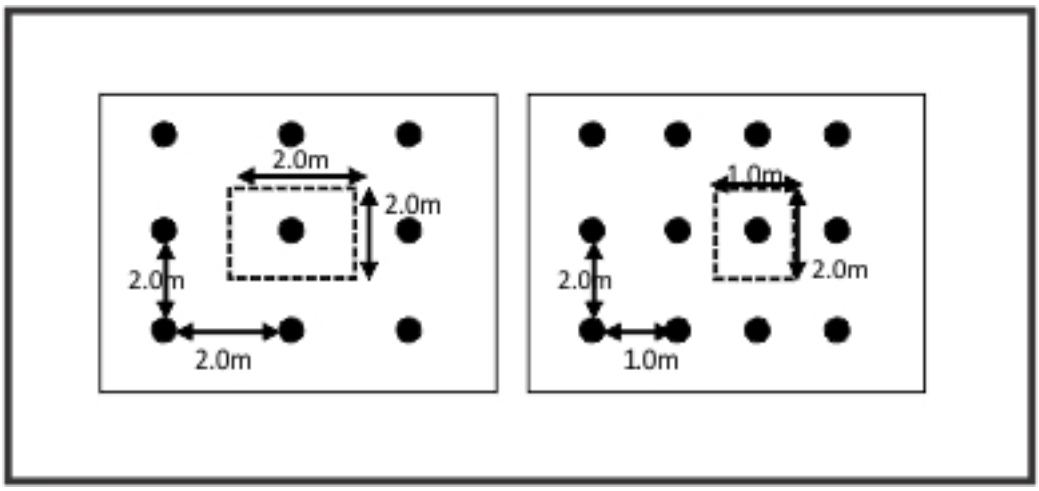


Figure 37: Support patterns (a) square and (b) rectangular

This means that a pattern can be derived if the support unit capacity and the required support resistance to maintain stability are known. For instance: If 140kN tendons are used and a support resistance of 75kN/m2 is required to maintain stability, the area to be supported by the tendon that will provide this support resistance can be calculated from: Capacity / Support resistance required or 140kN / 75kN/m2 = 1.86m2. Using this area supported per unit, support unit spacings within a pattern can be derived as follows:

- Square pattern: Square root of 1.86m^2 = 1.36m x 1.36m;
- Rectangular pattern: 1.2m spacing in one direction: Spacing in other direction is 1.86m^2 / 1.2m = 1.55m.

The length and spacing of the tendons (support pattern) together will determine their interaction with the rockmass.

Different tendon reinforcing and lacing patterns result in different degrees of support resistance and areal coverage (mesh present, but not illustrated) are illustrated in Figure 38.

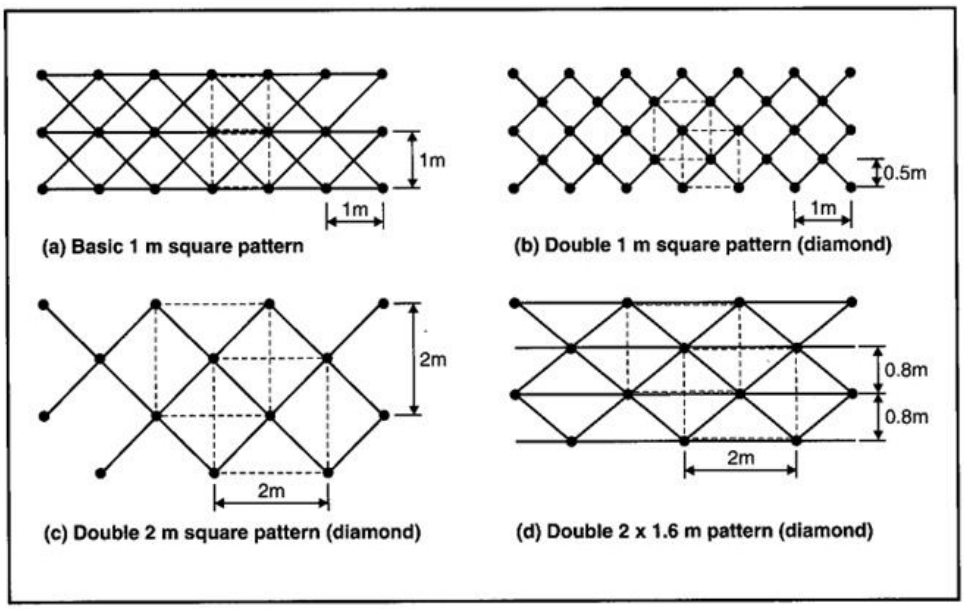


Figure 38: Different tendon reinforcing and lacing patterns (by Ryder JA & Jager AJ)

back

back

NOTE

The dash lines in Figure 38 indicate the basis of the terminology where a basic pattern shows non-overlapping dashed lines that connect the support units into squares and double patterns show overlapping dashed line areas. It must be noted that lacing patterns are not restricted to the patterns illustrated below.

REFERENCE

- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines SIMRACChapter 6

LEARNING OUTCOME 5.1.13

DESCRIBE, CHARACTERISE AND DISCUSS QUALITY CONTROL REQUIREMENTS FOR SUPPORT UNITS FROM FABRICATION TO INSTALLATION.

CONNECTION 36

Refer to "LEARNING OUTCOME 5.1.8" on page 293

The design of a support system is only as good as its installed performance. The design process should consider the applicability of the technology, the installation procedure sensitivity and the systems available to control the quality of the installation process and ultimately the performance of the support system.

Initial laboratory tests should be conducted on the elements of any new support system, under conditions that simulate their interaction with the rock mass and their anticipated in-situ loading.

- Axial and shear testing of tendons under static and dynamic loading, as individual units and with associated grouting systems, should be conducted.
- Static and dynamic load tests of fabric support systems with linkage to any tendon anchorage and interaction with the rockmass should be done.
- An evaluation of the adhesion of membrane or shotcrete systems should also be conducted.

This type of testing programme will assist in determining the expected performance of a support system and is useful in the setting of appropriate factors of safety in the design process.
Quality control systems should also include the appropriate education of the personnel involved with the installation of the support systems. Certain components of the support systems, such as grouts, resins, etc, have a restricted shelf life, which must be considered in the ordering, supply and utilisation of the products.

In-situ testing is a vital component of the quality control process, as it is the only true reflection of the performance characteristics of the elements of the support system installed in the rock mass. These tests should be conducted regularly and be appropriate to the mechanism of interaction between the support element and the rock mass. Pull tests may be conducted on certain tendons in situ and shotcrete adhesion and thickness measured in situ. Shotcrete samples may be taken for further compressive laboratory tests.

CONNECTION 37

For more information on shotcrete testing see "LEARNING OUTCOME 5.1.8" on page 293

Visual inspections should be also conducted on the quality of the support elements prior to and during the installation process. These also include planned task observations, pre-load indicators, checking the tension often on support by spanner or torque wrench and conducting work place audits.

back

The performance of the support systems can additionally be monitored with extensometers and closure pegs. This is useful especially with respect to verification of assumptions made during the design process.

The scale of the quality control programme and monitoring requirements should be a function of the criticality of the excavations and safety of the personnel. The sensitivity and degree of uncertainty of the performance of the support system must also be taken into consideration. Typical quality control programmes may test anywhere between 1 and 10% of the total number of support elements.

CONNECTION 38

For more information refer to SIM 04 02 05 "Final Project Report - Development of a practical guideline for support quality assurance for different systems used in tunnels and stopes" by DF Malan, PS Piper, T Clements, SJ Hayes.

SIMRAC has put together a comprehensive report SIM 04 02 05 "Final Project Report - Development of a practical guideline for support quality assurance for different systems used in tunnels and stopes" by DF Malan, PS Piper, T Clements, SJ Hayes.

NOTE

The following paragraphs are all extracts from the report that have some relevance to this outcome. Ideally, the report should be overviewed in its entirety to have an overall understanding on all the facets of quality assurance for support systems underground.

The requirements for underground evaluation of rock reinforcement systems differ from the other main support groups in two major respects. Firstly, the tendon can only operate as part of a system which includes the correct type of hole to accommodate the tendon and the appropriate type of anchoring system. Secondly, the performance of the system is difficult to evaluate through inspection methods since it is situated inside the rock rather than between the hangingwall and footwall. These differences mean that quality assurance of the system components prior to and during installation is much more critical than in the case of the other main support groups.

In the hard rock mines of South Africa, Jager and Ryder (1999), estimated that 800 km of new tunnels are developed each year in addition to the 10 000 km that is currently in use. With such a large number of tendons being installed, it is vital that an efficient quality assurance testing procedure is available to the mining industry. The focus is only on tendons manufactured from steel and a small variability is expected in the laboratory performance characteristics of these units, provided adequate quality assurance procedures are adhered to in the manufacturing process.

As a large percentage of these tendons are resin bolts or grouted, adequate quality assurance of the grout and resin is also required to ensure that safe support systems are maintained in the mining industry. This is especially important in applications where mechanical anchoring of bolts is not used and sole reliance is placed on resin or grout to provide the required anchorage. Anchors, which may appear to be properly grouted, but with grout not performing to specifications might lead to large sections of development being essentially unsupported.

The laboratory tests for new tendons are provided below in Table 18.

PARAMETER	TESTING/PROCEDURES	REMARKS
Types of tests	**Mechanical tendons** (i) Tensile strength tests for the tendon. (ii) Dynamic tests on the system if applicable. (iii) Shear testing and bending tests. (iv) Laboratory pullout tests on the system. **Friction tendons** (i) Tensile strength tests (ii) Laboratory pullout tests iii) Inflation pressure for water-pressurised friction tendons. (iv) Dynamic tests on the system if applicable. (v) Shear testing and bending tests. **Grouted tendons** (i) Tensile strength tests for tendon. (ii) Dynamic tests on the system if applicable. (iii) Shear testing. (iv) Pullout tests on the system (SEPT). (yielding tendons will also fall under this section)	1) The SABS 1408-2002 specifications for testing must be adhered to. **For rock bolts and studs** this implies: i) Minimum ultimate tensile strength of 570 MPa. ii) 0.2 % yield proof stress of 360 MPa. iii) Minimum elongation after fracture to be 22 %. **For bearing plates**: i) Withstand a force of at least equal to the 0.2% proof stress without the nut, rock bolt-head or spherical seat being pulled through the hole of the bearing plate with a maximum deformation of 15 mm. 2) **For laboratory pullout tests**, the SABS 1408-2002 specifications for testing must be adhered to. 3) **For water-pressurised friction tendons**, a maximum inflation pressure of 30 MPa must be reached.
Sample size	10 units of each product type for each type of test.	

Table 18: Laboratory tests for tendons (SIM 04 02 05)

Analysis of results

For the tensile strength tests and the pull-out tests, the load-deformation graphs listed below (reporting of results) should be calculated from the test results, using common data points from each individual test graph at intervals of deformation of not greater than 1mm. The calculations should assume the support system consists of one unit (therefore n=1) rather than a system consisting of more than one unit. The calculations should be performed using the standard statistical functions available in the Microsoft Office Excel programme. The mean of all the tests samples as well as acceptable confidence levels should be obtained.

Reporting of results

The results from the tests should be graphed as follows:

- All individual tensile strength tests on one load-deformation graph.
- All individual laboratory pullout tests on one graph
- Mean of the tensile strength tests and of the pullout tests on different graphs.
- The mean together with the acceptable confidence levels for tensile strength tests.
- The mean together with the acceptable confidence level for pull-out tests.

The laboratory tests for existing tendons are given below in the table below.

PARAMETER	PROCEDURE	REMARKS
Types of tests	**Mechanical tendons** (i) Tensile strength tests for the tendon. (ii) Dynamic tests on the system if applicable. (iii) Shear testing and bending tests. (iv) Laboratory pullout tests on the system. **Friction tendons** (i) Tensile strength tests (ii) Laboratory pullout tests iii) Inflation pressure for water-pressurised friction tendons. (iv) Dynamic tests on the system if applicable. (v) Shear testing and bending tests. **Grouted tendons** (i) Tensile strength tests for tendon. (ii) Dynamic tests on the system if applicable. (iii) Shear testing. (iv) Pullout tests on the system (SEPT). (yielding tendons will also fall under this section)	1) The SABS 1408-2002 specifications for testing must be adhered. **For rock bolts and studs** this implies: i) Minimum ultimate tensile strength of 570 MPa. ii) 0.2 % yield proof stress of 360 MPa. iii) Minimum elongation after fracture to be 22 %. **For bearing plates:** i) Withstand a force of at least equal to the 0.2% proof stress without the nut, rock bolt-head or spherical seat being pulled through the hole of the bearing plate with a maximum deformation of 15 mm. 2) **For laboratory pullout tests**, the SABS 1408-2002 specifications for testing must be adhered. 3) **For water-pressurised friction tendons**, a maximum inflation pressure of 30 MPa must be reached.
Sample size	3 units of each product type for each type of test.	

Table 19: Tests for existing tendons (SIM 04 02 05)

The analysis and reporting of results are to be done as noted for new tendons.

The testing procedure for new resin products is noted in Table 20 and for new grout products in Table 21 below.

PARAMETER	TESTING/PROCEDURES	REMARKS
Types of tests	i) Sheath (for capsules only) ii) Dimensions (for capsules only) iii) Wetted volume yield iv) Working life v) Pull strength vi) Grout mix contraction vii) Compressive strength viii) Storage stability	The sheath, dimensions, wetted volume yield, working life, pull strength and grout mix contraction should be tested and conform to the procedures given in SANS 1745:2003.
Sample size	A minimum of 5 samples for each of the tests described.	

Table 20: Laboratory testing procedure – new resin products (SIM 04 02 05)

PARAMETER	TESTING/PROCEDURES	REMARKS
Types of tests	i) Sheath ii) Dimensions iii) Gel and setting time iv) Bond strength and system stiffness v) Uniaxial compressive strength vi) Elastic modulus vii) Creep viii) Storage stability	The sheath, dimensions, gel and setting time, bond strength, UCS elastic modulus and grout mix should be tested and conform to Anglo American Sepcification 212/2.
Sample size	A minimum of 5 samples for each of the tests described.	

Table 21: Laboratory testing procedure – new grout products (SIM 04 02 05)

From the minimum of five samples tested for each type of test, it will be beneficial to do the statistical calculations. The results of the tests can be documented by photographs and data tables.

Table 22 and Table 23 indicate the types of tests that should be conducted on existing resin and grout products respectively.

PARAMETER	TESTING/PROCEDURES	REMARKS
Types of tests	i) Sheath ii) Dimensions iii) Gel and setting time iv) Bond strength and system stiffness v) Uniaxial compressive strength vi) Elastic modulus vii) Creep viii) Storage stability	The sheath, dimensions, gel and setting time, bond strength, UCS elastic modulus and grout mix should be tested and conform to Anglo American Sepcification 212/2.
Frequency	Each batch	
Sample size	A minimum of 5 samples for each of the tests described.	

Table 22: Laboratory testing procedure – existing resin products (SIM 04 02 05)

PARAMETER	TESTING/PROCEDURES	REMARKS
Types of tests	i) Sheath (for capsules only) ii) Dimensions (for capsules only) iii) Wetted volume yield iv) Working life v) Pull strength vi) Grout mix contraction vii) Compressive strength viii) Storage stability	The sheath, dimensions, wetted volume yield, working life, pull strength and grout mix contraction should be tested and conform to the procedures given in SANS 1745:2003.
Frequency	Each batch	
Sample size	A minimum of 5 samples for each of the tests described.	

Table 23: Laboratory testing procedure – existing grout products (SIM 04 02 05)

Analysis and reporting should be conducted in a manner identical to new resin and grout products.

It is recommended that, after successful laboratory testing of tendons, the underground evaluation is performed in three phases, i.e. underground testing, underground trials and routine underground assessments of the performance of the rock reinforcement system. It is very important that the different phases run sequentially.

The process commences with acceptable results from standard laboratory tests and it assumes that individual performance of the tendon and of the anchoring system meets the required specifications. The initial underground testing provides an opportunity to evaluate the rock reinforcement system using the customer's ground conditions and their installation equipment.

During the underground testing phase, the focus should be on the in-situ performance of the system and should also compare the performance to the rock reinforcement system that is currently in use. The system should be installed according to the exact procedures agreed upon between the supplier and customer. The installation equipment must be in good working order to ensure that the sub-standard equipment does not influence the performance of the reinforcement system.

With underground trials, a much larger quantity of tendons is installed under a wider variety of conditions and, more importantly, the tendons are installed by mine workers and exposed to the normal mining practices. The results from the earlier underground tests will provide a benchmark against which the results of underground trials can be compared.

The final phase is routine underground evaluation of the rock reinforcement system after the new rock reinforcement system is in regular use. During this phase, the installation procedures will be audited and selective testing of tendons will be performed on a regular basis.

The main purpose of underground testing is to quantify the rock reinforcement system using the customer's ground conditions and their installation equipment. The focus should be on the system performance when the rock reinforcement is installed in the customer's rock mass using the customer's installation equipment. It is important that the system is installed according to the exact procedures agreed upon between the supplier and customer. While this may not be representative of the performance of future installations, it provides the best possible in-situ performance. If the system is unable to provide the required performance under these conditions, then it is unlikely to be suitable for general use.

The installation equipment must be in good working order to ensure that the sub-standard equipment does not influence the performance of the reinforcement system. The tendons should be installed in addition to the normal pattern of support required by mine standards. The emphasis of the underground testing should be placed on the performance relative to the existing support units in use as well as laboratory test results. As many variables as possible should be eliminated at this stage of testing. Those that cannot be eliminated should be recorded.

The main purpose of underground trials is to obtain further confidence in the performance of the new product as its use progresses and to answer questions that have not been adequately addressed. The underground trialling also provides an opportunity to evaluate non-performance-related factors such as condition after underground storage, ease of installation and worker acceptability. In the case of rock reinforcement systems, the most critical additional variables are those relating to installation by trained operators. The correct procedures may not always be used and the installation equipment may not be in perfect working order for the entire duration of the trials. As a result of the introduction of these additional factors, the rock reinforcement system performance may not be as good as that achieved during underground testing. Should the system performance deteriorate, it is important to quantify the change and to identify the causes so that they can be rectified

back

Many factors contribute to achieving the required performance from a rock reinforcement system. The five main ones are the tendon, the anchoring system, the hole, the installation procedures and the installation equipment. Since a change in any one of the factors, a number of which are beyond the control of the supplier, can lead to reduced system performance, it is critical that routine evaluations are conducted on a regular basis. The importance of routine evaluation is increased since it is difficult to quantify the system performance through inspection methods as most of the rock reinforcement is hidden inside the rock.

The following routine evaluation procedures are proposed:

- Quarterly sampling and laboratory testing of the system components (especially resins and grouts after underground storage) and visual inspection of other components.
- Quarterly testing of rock-bolters in coal mines for conformance to torque, thrust and speed specifications.
- Quarterly auditing of installation procedures.
- Quarterly pull tests or torque tests on at least three tendons, selected randomly, or each application at each customer.
- On-going monitoring through fall of ground observations and tendon measurements using load cells and extensometers to quantify the effectiveness of the system for a wide range of conditions.

REFERENCE

- DF Malan, PS Piper, T Clements, SJ Hayes (2005) Development of a practical guideline for support quality assurance for different systems used in tunnels and stopes – Final Project Report
- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines

5.2 SUPPORT INSTALLATION METHODS AND PROCEDURES

LEARNING OUTCOME 5.2.1

SKETCH, DESCRIBE AND EXPLAIN THE METHODS AND PROCEDURES FOR INSTALLING COMMON EXCAVATION SUPPORT UNITS

The timing of the installation of the support element has an important influence on the displacement control. A general set of principles has been proposed to guide the general support installation method and procedures.

- Install the support elements close to the face soon after the excavation has been developed.
- There should be good contact between the rock mass and the support element.
- The deformability of the support element should be such that it conforms to the displacements of the excavation surfaces.
- The support system should help prevent the deterioration in the mechanical properties of the rock mass with time due to weathering.
- Repeated removal and replacement of support elements should be avoided.

back

- The support system should be easily adaptable to the changing rock mass conditions and excavation cross-section.
- The support system should provide minimal obstruction of the working face.
- The rock mass surrounding the excavation should be disturbed as little as possible during the excavation process.

CONNECTION 39

Refer to "LEARNING OUTCOME 5.1.8" on page293forspecificprocedures from additional sources to these shown below. The user is advised to read both sections and apply those appropriate to the local mine.

Mechanical support cables (40 ton, pre-stressed, full column grouted)
Method using a long hole drilling machine (conventional)

- Holes to be drilled in the hangingwall without the use of a drill rig require the installation of at least four temporary supports (mechanical props or timber poles), around the position of the hole to be drilled. These supports must not be further than 1.5m from the collar of the hole. In friable ground conditions, it might be necessary to use a section of wire mesh against the hanging, supported with headboards on the temporary support units.
- Drill the hole for the cable 50cm longer than the length of the cable to be installed. Ensure that the diameter of the hole is suitable for the anchor diameter. Horizontal holes should be angled five degrees up to ensure that the water and sludge runs out.
- Insert the cable into the hole until 0.8m of the cable remains protruding from the hole.
- Activate the mechanical anchor with the attached wire and ensure that the anchor is gripping properly by pulling the cable.
- Insert the short grout tube into the hole and seal the collar of the hole with paper and/or cotton waste.
- Install the sponge provided, followed by the base plate, ensuring that both the grout tube and breather tube will not be pinched or become obstructed. Voids between the base plate and the rock wall and the collar of the hole are filled with a thick consistency grout or the sponge provided, to ensure a good seal and to ensure proper seating of the base.
- Slide the wedge box and crocodile wedges over the protruding end of the cable into the position at the collar of the hole.
- Fit the tensioner onto the cable and tension the cable to 20 tons. Stop pumping at 20 tons, and check the reading of the gauge for 2 minutes to see whether it stays at that reading to make sure that the anchor is not slipping. If it is slipping, tension again to 20 tons and repeat the check. If the cable is equipped with a shear ring device that shears at the correct tension, the cable must be tensioned until the shear ring snaps, driving the wedges home.
- Mix cement and water to form a thick but pumpable grout. Pump the grout through the grout tube into the hole until thick grout comes out of the breather tube.
- Close the breather tube off by bending and tying it back.
- Carry on pumping for 30 seconds, and then close the grout tube off in a similar fashion.
- On successful installation, and no further tensioning required, the protruding cable can be cut shorter (if it is in the way), but not closer than 150mm from the wedge box. An angle grinder must be used for this and not a cutting torch, as it will change the temper and therefore the strength of the cable.

Sequence of installation when using a drill rig for drilling the holes

1. Mark off the position of the support cables to ensure the correct spacing and distances from the hanging and footwall as specified by the official support layout.

back

2. Position the drill rig such that the hole can be drilled at the correct position and direction and that the cabin of the machine is located in a safe and supported area.
3. Drill the hole for the six-meter cable to a depth of 6.5m (50cm longer than the length of the cable). Ensure that the diameter of the hole is suitable for the anchor diameter. The hole diameter should not be more than 51mm for the standard cable supplied at present.
4. Horizontal holes should be angled a few degrees up to allow the water and sludge to run out. Down holes must be flushed properly after drilling.
5. While drilling, nobody should be allowed to come within 2.0 meters of the collar of the hole as the vibrations caused by the drilling can induce rockfalls. A second person is required to assist in loading the extra rods to complete the hole to a depth of 6,5m. He may not enter before drilling has stopped. Drilling may only be re-started when he has left the "danger area". Use must be made of a ladder and safety harness if the installation of the next rods is above normal working height.
6. Before installing the cable, inspect the cable anchor for possible damage and check whether the attached breather tube is unobstructed.
7. Insert the cable into the hole until approximately 0.8m of the cable remains protruding from the collar of the hole.
8. Activate the mechanical anchor by pulling the activating wire and ensure that the anchor is gripping properly by jerking the cable.
9. Insert the short grout tube into the hole and seal the collar of the hole with suitable material, e.g. paper or a sponge.
10. Insert the base plate, ensuring that both the grout tube and breather tube will not be pinched or become obstructed. Voids between the base plate, the rock wall and the collar of the hole must be filled with thick consistency grout to ensure a good seal and to ensure proper seating of the base plate.
11. Slide wedge box and crocodile wedges over the protruding end of the cable into position at the collar of the hole.
12. Fit the tensioner onto the cable and tension the cable to 20 tons, or until the indicator ring on the wedge box snaps, to allow for creep. Stop tensioning at 20 tons and check the reading of the gauge for two minutes to see whether it stays the same and is not slipping. If it is slipping, tension again to 20 tons and repeat the check. Note that the maximum load of the cable is 40 tons and anchors are normally tensioned to 0,5 x the capacity of the cable. Ensure that the shear ring snaps.
13. Mix cements and water to form a thick but pumpable grout. Pump the grout through the grout tube (short tube without the wire) into the hole until thick grout comes out of the breather tube. Close the breather tube off by bending and tying it back. Carry on pumping for 30 seconds or until the pump stops. Close the grout tube off by bending and tying it back.
14. If necessary, cut the protruding end of the cable a minimum distance of 150mm below the wedge box. Always use an angle grinder or a mechanical cutter for this and not a cutting torch, as the heat from the cutting torch will change the temper and therefore the strength of the cable.

Mechanical roof bolt

- Examine the area to be drilled, bar and make safe.
- Install temporary support as per mine standard requirements.
- Mark off the area as per the required pattern.
- Drill a hole in the wall as close to 90° to the wall as possible.
- Commence drilling with a starter jumper and followed by the

back

length of the steel relative to the length of rod to be grouted.

- Clean the drilled holes by means of compressed air and water.
- Slide the plastic sleeve of the expansion shell.
- Push the mechanical roof bolt into the hole until the washer touches the hangingwall.
- Allow 18-20mm space between the wall and the washer tensioning a crimp nut setup.
- Tension the installation using either an impact wrench or sliding "T" spanner.

Procedure for grouting of drilled holes, meshing and lacing

- Examine the area to be grouted, bar and make safe.
- Install temporary support as per mine standard requirements.
- Mark off the area as per the required pattern.
- Commence drilling with a starter jumper and followed by the length of the steel relative to the length of rod to be grouted.
- Clean the drilled holes by means of compressed air and water.
- When using a cement grout pump
  - Mix the cement and water to the right consistency, i.e. should be stiff enough to stick to your hand when it is inverted.
  - Fill the grout pump and close the lid. Ensure the butterfly hold-down nuts are secure.
  - Close air release port of the pump and gradually open the feed, forcing the cement out through the hose into the hole to be grouted.
  - When the hole is full of cement, close the compressed air feed then open the air release port to reduce the pressure in order to stop the grouting procedure.
- When using cementitious capsules
  - Lay the capsules in water until it stops bubbling.
  - The damp capsules can either be inserted into the hole by sliding them in and tamping them home by the charging stick method or by using a pneumatic gun to shoot them into the holes.
  - Capsules are inserted until the hole is full of cement.
  - Wet the support tendons with water and insert it into the hole allowing for ±20cm of protrusion (or as per the relevant mine standard).
  - When the grout has cured in the holes for ±48 hours, the lacing can commence.
- Apply the diamond mesh where necessary as a skin against the rock face, holding it in position by allowing the tendon's eyes to protrude through the apertures of the mesh (a minimum overlap of mesh is required).
- Feed the lacing rope through the tendon's eyes as per standard pattern. If the rope is too short, knot another piece of rope onto the rope and fasten back ends with clamps.
- Tension the rope, using an air hoist. Clamp off the tensioned rope with an overlap and clamps (as per relevant mine standard). Cut off the loose ends.

back

Shotcrete

Shotcrete should not be applied directly to a dry, dusty or frozen rock surface. The work area is usually sprayed with an air-water jet to remove loose rock and dust from the surface to be shot. The damp rock will create a good surface on which to bond the initial layer of shotcrete paste. The nozzle-man commonly starts low on the wall and moves the nozzle in small circles working his way up towards the back or roof. Care must be taken to avoid applying fresh materials on top of rebound or over-sprayed shotcrete. It is essential that the air supply is consistent and has sufficient capacity to ensure the delivery of a steady stream of high velocity shotcrete to the rock face.

Shooting distances are ideally about 1 to 1.5 metres. Holding the nozzle further from the rock face will result in a lower velocity flow of materials, which leads to poor compaction and a higher proportion of rebound.

A well-trained operator can produce excellent quality shotcrete manually, when the work area is well lit and well ventilated, and when the crew members are in good communication with each other using prescribed hand signals or voice activated FM-radio headsets. However, this is a very tiring and uncomfortable job, especially for overhead shooting, and compact robotic systems are increasingly being used to permit the operator to control the nozzle remotely.

When shotcrete is applied to rock masses with well-defined water-bearing joints, it is important to provide drainage through the shotcrete layer in order to relieve high-water pressures. Drain holes, fitted with plastic pipes, are commonly used for this purpose. Where the water inflow is not restricted to a few specific features, a porous fibre mat can be attached to the rock surface before the shotcrete layer is applied. When practical to do so, the water from these drains should be collected and directed into a drainage ditch or sump.

Shotcrete spraying procedure

- Rock face preparation
  - All loose rocks must be barred from the area to be shotcreted. The area must then be cleaned by washing it down with water to remove all dust. Shotcreting may only start when the area is no longer wet, but merely damp.
- Lighting
  - Only locked sealed lights, approved by the Chief Inspector of Mines, may be used in all working places not in thorough ventilation, i.e. development ends, sinking shafts, back stopes, etc. Lighting must be adequate at all times, and if needed more lights must be installed.
- Protective clothing
  - Adequate protective clothing consisting of hardhat, safety goggles and gloves must be worn at all times by the operator while spraying. Dust respirators must be worn by all personnel involved in the spraying, as well as all people doing other work on the return ventilation side of the shotcrete.
  - Only respirators as recommended by the SHE Department and approved by the Manager must be used.
- Spraying
  - The shotcrete must be sprayed to the minimum thickness prescribed.
  - All cracks and nooks must be filled to obtain a smooth finished effect.

back

- To allow proper filling of the cracks, the nozzle must be aimed from the direction in which the development of the excavation was done backwards.
- Forming of slump cracks in the shotcrete must be avoided by not building up a too thick a layer in a single position at once.
- The original survey peg positions must be indicated on the shotcrete, using spray paint.
- The air pressure used when spraying must be at least 500kPa (5Bar), but preferably 600kPa.

REFERENCE

- Evander Gold Mine Support Procedures
- Harmony-Target Mine Support Procedures
- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines

LEARNING OUTCOME 5.2.2

DETERMINE APPROPRIATE SUPPORT INSTALLATION METHODS AND PROCEDURES FOR GIVEN TUNNEL LAYOUTS, ROCKMASS CONDITIONS AND STRESS REGIMES

High or wide mining areas may be very susceptible to changes in the rock mass and must therefore be adequately supported. The high exposed faces/sidewalls may traverse various discontinuities that make it very susceptible to falls of ground if not made safe properly and well supported. In supporting high excavations, a rule of thumb is used that states that the support along the sidewall/faces must be at least half the height of the excavation.

NOTE

This is only a rule of thumb that must be ignored if adverse conditions that may affect stability exist. If the expected stress levels acting on the high excavation are large, more intensive support is required. Some high excavations also accommodate major infrastructures such as hoists or pump chambers and in such cases its stability must be ensured for the life of the excavation.

In different geotechnical areas, support lengths, spacing or densities, strengths, characteristics and even installation angles may differ significantly. Support installation should preferably occur prior to the creation of the high or wide excavation walls. Excavations where this is possible, include:

- Large chambers where the excavation method is one where the final height and width are created through several cuts and lifts and allow the installation of final support systems during each cut and lift, i.e. long before the height or width it was designed for is created.

Example:

Excavation	Comments
Crusher chamber	The crusher chamber will be developed in lava, some 14m below the BIS / Lava intersection. The development method allows the creation of the crusher chamber in three vertical slices of roughly similar dimensions. The top slice will be mined first to allow the installation of support in the hangingwall of the chamber and the top 5m of the sidewalls that will be exposed in this slice. The subsequent slices will allow completion of support installation in the crusher sidewalls as the sidewalls are increased in height to the full height of the chamber. On completion of the 3rd slice, the chamber will be completely supported, allowing all mining to occur under already supported hangingwall and sidewalls.

- Timber bays in scattered mining environments: Timber bays are created close to travelling ways that lead to stopes to act as storing areas for the in-stope timber support. Usually it entails the sliping of the existing crosscut sidewall, normally 3m wide, to a width of 5 to 6m. This means that the support originally installed in the crosscut is not sufficient for the final width. Support to cater for the final designed width can be installed prior to sliping and can be updated after each sliping round to ensure complete and adequate support installed by the time the final width is reached (Figure 39a).
- Breakaway areas from tunnels: When a secondary tunnel is developed from the main or current tunnel, the intersection area increases in width to approximately 10m (dependent on the tunnel width dimensions). Normal tunnel support is not sufficient to keep this high width area stable. By installing some longer support units in a tunnel, around the point where the secondary tunnel is to break away, before the breakaway is created, the wider areas can be supported prior to the actual creation and will enhance stability (Figure 39b).

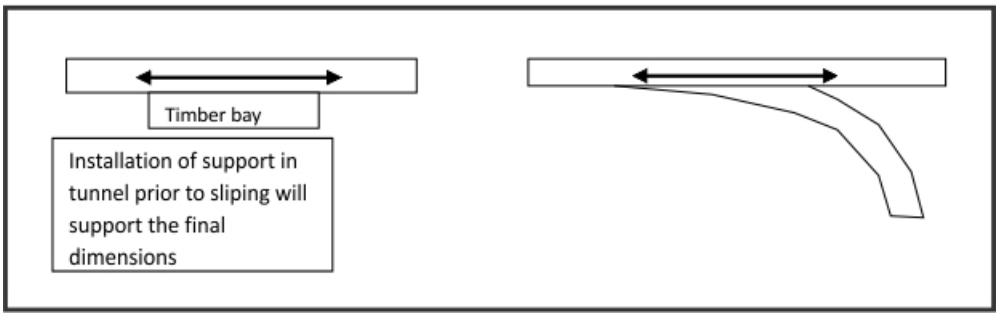


Figure 39: Supporting of large areas (a) timber bay and (b) tunnel breakaway areas

Under **highly stressed conditions,** tunnelling in hard-rock mines becomes slow and difficult mainly due to support installation constraints. The following is a successful support installation procedure for highly stressed tunnels:

- Primary shotcrete (50 to 100mm thick) is applied soon after the development blast to prevent initial deterioration and also weathering of the rock (if applicable).
- Cable bolts (4.0m long) are installed in each support ring.
- Adequately robust, large face plates are placed on the cable bolts.
- Secondary shotcrete is applied at some convenient distance from the advancing tunnel face. The reasons for its installation distance from the face are as follows: it will not unduly hinder the production cycle, while some of the tunnel dilation will have already occurred and hence the extent of the expected failure of the secondary shotcrete will be reduced and it can be used to repair the primary shotcrete that can be expected to be severely damaged.
- The secondary shotcrete is reinforced with steel fibre to improve the shear and flexural strength of the material. The use of mesh is also acceptable, but it may prove too labour intensive.
- The final thickness of the shotcrete is in excess of 200mm.
- During the useful life of the tunnel, remedial measures may still be necessary to repair the permanent support. This will mainly include the re-application of portions of the shotcrete (particularly after over-stoping or major seismicity in the vicinity).

back

Squeezing rock refers to time-dependent deformation that occurs around the excavation, and is essentially associated with creep caused by the in-situ stress field. Deformation may terminate during construction or continue over a long period. In the early twentieth century, thick masonry linings were often installed in the tunnels to control the deformation. More recently, steel sets in combination with thick concrete linings have been used as support. The low deformability of steel, concrete and shotcrete in these conditions can lead to buckling of the steel units and shearing of the linings. Other support installation approaches exist that attempt to accommodate the large deformations without the support collapsing. This approach may include yielding rock bolts that allow significant deformation without failure, over-excavation where the tunnel is excavated to a larger size allowing for significant closure to take place, and delayed installation of shotcrete to allow the squeezing energy to be dissipated first by another passive yielding support system.

IMPORTANT

The stiffness of support installed and the rock mass stiffness must be matched to ensure that the support is not over-deformed and fails, but also that support is not installed after all deformation has occurred (support is not loaded). The concept applied is the one of Ground Reaction Curve (Figure 40). This curve indicates the pressures required to control rock mass deformation (ABFH and ACEG in Figure 40) and allows the plotting of the support load-deformation curves (DEF in Figure 40). The application of these curves is briefly explained as follows:

- *If support softer than curve DEF is installed (1), most deformation in the tunnel walls may already have occurred, limiting the load to be generated in the support unit and therefore its ability to maintain stability;*
- *If support much stiffer than curve DEF is installed (2), it may be subjected to too much rock mass deformation and may fail while its supporting action is still required;*
- *Time of installation is indicated by the start of the support reaction curve (D). Later installation (3) at similar stiffness will reduce the impact of support and earlier installation (4) at the same stiffness may subject the support to too much deformation.*
- *It is therefore critical to be able to determine the ground reaction curves and compare these with support reaction curves to determine the optimum support installation time and stiffness.*

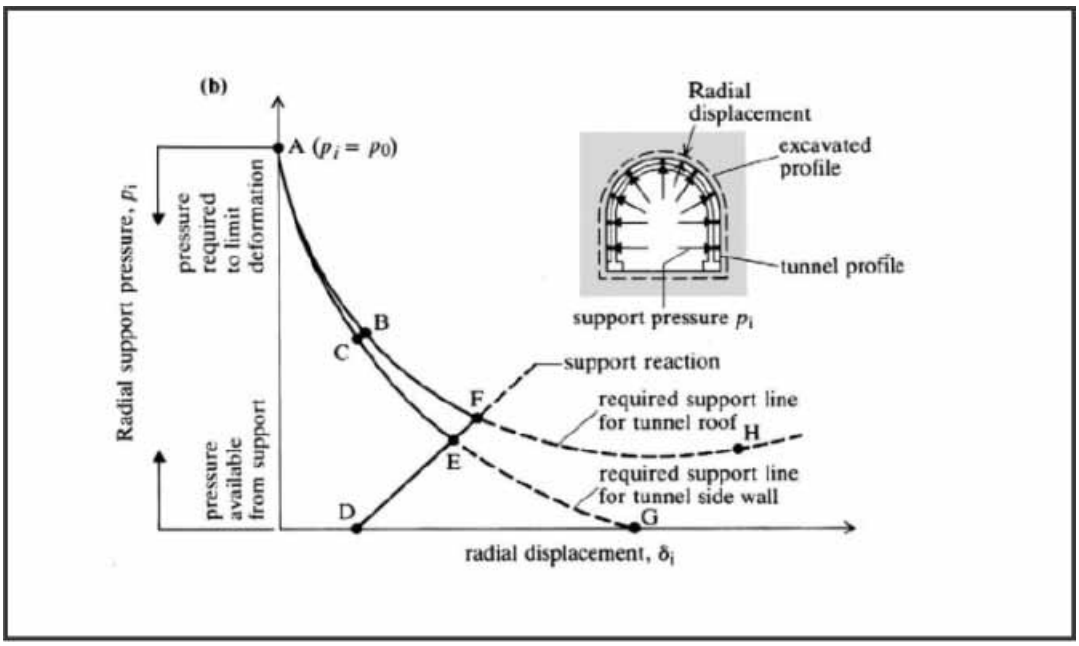


Figure 40: Ground reaction curve example (Brady and Brown).

back

Poor rock mass conditions also affect support installation and rock mass rating systems (Q, RMR and RCF) are used to determine the type and application of various support types, lengths and patterns.

REFERENCE

- Bosman JD, Malan DF, Drescher K (2000) Time-dependant Deformation at Hartbeesfontein Mine
- RoC Learning Guide RoC-G065
- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines

LEARNING OUTCOME 5.2.3

CONNECTION 40

Refer to "LEARNING OUTCOME 5.2.1" on page 351.

DESCRIBE, CHARACTERISE AND DISCUSS SPECIFICATIONS FOR THE VARIETY OF SUPPORT INSTALLATION PROCEDURES

REFERENCE

- Evander Gold Mine Support Procedures
- Harmony-Target Mine Support Procedures
- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines

LEARNING OUTCOME 5.2.4

DESCRIBE, CHARACTERISE AND DISCUSS QUALITY CONTROL REQUIREMENTS FOR SUPPORT INSTALLATION METHODS AND PROCEDURES

Monitoring is the final step in the cycle of the support installation process. It is a continuous process, and should consist of three parts:

- monitoring of the support material prior to installation,
- monitoring of the suitability of the overall support system underground and
- monitoring of the quality of installations.

The pre-installation monitoring should be done in three phases:

- Prior to delivery in the factory: Unannounced factory visits should be made periodically and samples collected for testing and checks on the dimensions of the support units. Likewise, at the resin/grout factory, the quality certificates and delivery dates of the raw materials and storage of the finished products should be checked. In both cases, the quality control procedures of the manufacturing process and quality control reports should be inspected.
- After delivery but prior to dispatch underground on the mine: A pre-determined number of support units should be randomly collected for the testing of material quality. Dimensions of the support units should be checked with a Vernier scale. Support units should be visually inspected for straightness where required. Resin/grout capsules should be checked by weighing selected samples, measuring the dimensions, visually inspecting the colour markings, performing the stiffness test (i.e. a capsule held at 45° with 30cm protruding should not buckle), visually inspecting for leakage, and testing the setting times by hand mixing approximately 5cm of a capsule's length and prodding to check when the mixture solidifies. The expiry dates on the boxes should be checked. Stacking of boxes should be checked and they should not be stored outdoors.

- After dispatch underground but prior to installation in the section: The threads on bolts and nuts should be checked for damage and the length of support units should be measured and visually compared with the rest for deviations. Every box of resin/grout capsules should be visually examined for leakage of the capsules. Upon delivery, the expiry dates should be checked. Old resin/grout must not be used.

Monitoring support system suitability underground:
Support suitability is shown by the reaction of the rock mass and/or support units installed in a particular area:

- Open tensile cracks forming approximately in the centre of a tunnel and running along the length of the tunnel may indicate that support units are too short, tendon spacings are too wide, a resin/grout problem exists or the support capacity is insufficient.
- If the support units fail in tension, this may indicate that the steel units are not strong enough / insufficient diameter or the support density is insufficient.
- Shear failure of the units indicates shear displacement in the rock mass that will require higher shear resistant support units.
- 'Pulling' of support units into the rock indicates high load bearing and good anchorage of the support units.
- Cracking of shotcrete indicates high levels of deformation and insufficient yieldability of the product. However, if fibre reinforced shotcrete is applied, even cracked shotcrete will still generate significant confinement stresses on the rock mass. Complete disintegration of shotcrete will indicate failure.
- Levels of corrosion must be checked as strengths of corroded units are significantly lower than 'fresh' units.
- A very important element of monitoring is therefore aimed at being continuously aware of the rockmass composition and how the installed support performs.

Monitoring of the quality of support installations:

CONNECTION 41

Refer to "LEARNING OUTCOME 5.1.8" on page 293 and "LEARNING OUTCOME 5.2.1" on page 351.

Perhaps the most important element of monitoring the quality of support installations is to observe the installation process. In this regard, over-drilling of holes and deviations in spinning and holding times of resin/grout capsules are the most common errors. Incorrect hole lengths and diameters also occur frequently – the most practical way to ensure correct hole lengths is to place a short length of steel pipe over the drill rods so that the correct length of rod protrudes. The correct way to determine the hole length is to assemble a tendon with nut and washer and to measure the free length of the support unit. Another common error is jerky operations, e.g. stopping to reposition the bolter/drill machine halfway through the support unit insertion or spinning cycles. This invariably results in disturbing the resin/grout during the holding period, or insufficient mixing.

REFERENCE

- Van der Merwe JN (1998) Simple User's Guide on Roof Support, Installation and Evaluation, SIMRAC: COL504. Final Report

5.3 BACKFILL SYSTEMS

LEARNING OUTCOME 5.3.1

DESCRIBE AND EXPLAIN THE FUNCTIONS OF BACKFILL AS LOCAL SUPPORT

In South Africa, especially in deep stopes, backfill is playing an important role as a local support medium. Where backfill of adequate quality has been placed, hangingwall conditions and stability can improve dramatically, to the extent that the frequency and severity of rock falls and rock bursts are significantly reduced. Apart from the safety-related benefits, backfill can improve face advance rates, improve face and gully conditions, provide better stope width control and dilution control. The use of backfill together with good conventional face area prop support is a preferred medium of local support in deep mining and in high stope width mining. An important provision is that to realise the potential considerable benefits, the backfill has to be kept close to the face at all times (preferably < 6m). Fairly stringent quality controls, particularly of density and porosity, which contribute to shrinkage control, have to be exercised. In general, the criteria used when backfill is being selected for local support purposes include: stope closure rates, stope width, strain rates, freestanding ability, water drainage and the required stiffness.

NOTE

Work done by D Hemp on the impact of backfill on hangingwall vibrations during a seismic event is interesting and should be studied for additional impact on local support impacts.

REFERENCE

- Goldbach & Hemp, Hangingwall behaviour in seismic events backfill
- Kirsten & Stacey, Hangingwall behaviour in seismic events
- Piper & Ryder, Selection of backfill as local support
- Palarski, Backfill in polish coal mines
- Fly ash as backfill in coal mines
- Ryder and Jager page 279 for a table with typical values.

CONNECTION 42

Refer to the discussion on backfill types, methods, applications in Chapter 4.

"LEARNING OUTCOME 4.2.1.1" on page 232

NOTE

The decision on the use of backfill is very similar to the selection of support for any other application in that the conditions (rock mass behaviour) and support characteristics determine the appropriateness of the support. With backfill, the local and/or regional behaviour and the characteristics of the backfill types determine whether the type is appropriate. The characteristics of the backfill are described by the stress-strain relationships (Figure 46) compared to the load-deformation relationships usually used for other support types. The main reasons for this are:

- *Fill is viewed as a complete areal coverage support medium and that load (N) is generated per m2, resulting in a Pa application (N/m2), where support spacing plays no role;*
- *Deformation of the backfill is better described in strain rather than displacement (mm) compressed as it clearly indicates the amount of deformation undergone to generate the suggested stress levels, a concept easily understood by users.*

Many graphs will therefore show % compression of fill along the bottom axis, which is nothing else than strain expressed as a percentage (i.e. (compressed amount or change in dimensions / original fill height) x 100).

Two theoretical and general, stress–strain relationships are promoted for backfill types, namely the hyperbolic and quadratic relationships (Figure 41).

NOTE

It has been known that the quadratic relationship is more appropriate to the general backfill types utilised in mining methods.

These relationships can be used to determine the backfill properties when designing backfill for an application, namely the 'a' and 'b' values, that are used to describe the backfill behaviour. By selecting the stress to be generated at a specific level of deformation (strain) by the backfill and the maximum strain that can be allowed ('b' value), the most appropriate backfill 'a' value can be derived from the given equations.

DEFINITION

'a' value is the stress generated by backfill at 50% of the maximum strain ('b value) undergone by the backfill.

REFERENCE

- Refer to Ryder and Jager page 279 for a table with typical values.

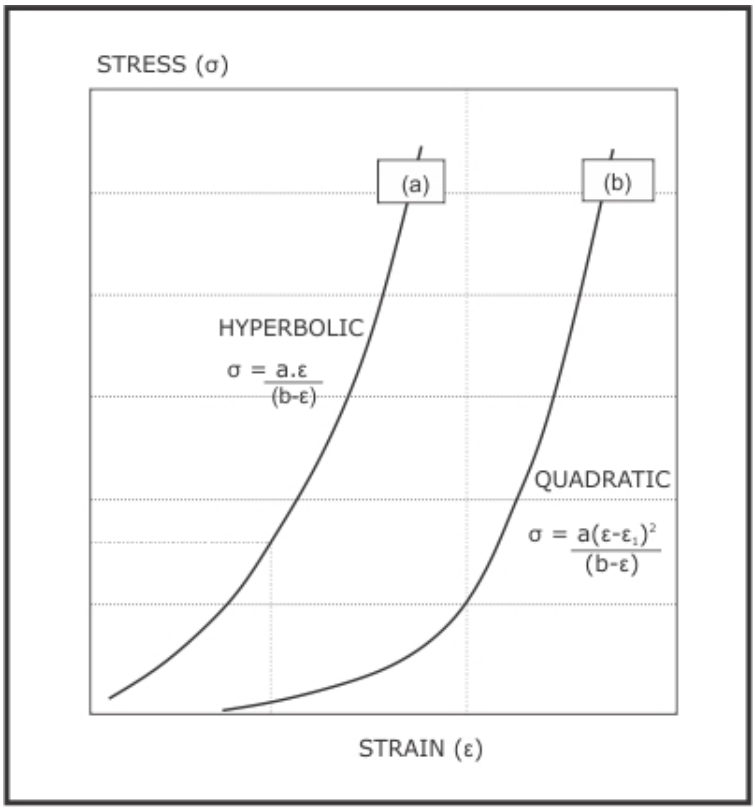


Figure 41: Backfill stress-strain relationships (a) Hyperbolic and (b) Quadratic

REFERENCE

- Budavari S (ed) (1983) Rock Mechanics in Mining Practise
- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines

LEARNING OUTCOME 5.3.2

DESCRIBE AND EXPLAIN THE FUNCTIONS OF BACKFILL AS REGIONAL SUPPORT

CONNECTION 43

Refer to the discussion on backfill types, methods, applications in Chapter 4, from "LEARNING OUTCOME 4.2.2.1" on page 243 to "LEARNING OUTCOME 4.2.2.3"

As regional support, backfill is required to reduce the volumetric convergence in the mined-out areas and the high stresses on the face, so that the occurrence of seismic events can be reduced by achieving lower levels of energy release rate (ERR) and excess shear stress (ESS). The percentage extraction of the reef can also be increased when backfill is used as regional support in place of or to supplement stabilising pillars. In addition, backfill used as regional support assists in shaft protection and middling stabilisation in multi-reef extraction situations.

REFERENCE

- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practice for Tabular Hard Rock Mines
- Kirsten & Stacey, Hangingwall behaviour in seismic events
- Piper & Ryder, Selection of backfill as local support
- Palarski, Backfill in polish coal mines
- Fly ash as backfill in coal mines

LEARNING OUTCOME 5.3.4

SKETCH, DESCRIBE AND EXPLAIN THE DIFFERENT BACKFILL PREPARATION METHODS AND PREPARATION SYSTEMS FOR:

- Full plant tailings fill
- Classified tailings fill
- Paste fill

Full plant tailings fill

Full plant tailings fill (FPT) or slurry backfill consists of tailings 'as is' and is pumped directly from the plant to storage bins, or most often to the tailings dams. On the tailings dams, water is drained from the tailings over time. These tailings include particles of all dimensions and may have a large amount of fine or coarser material, depending on the 'grind' in the plant.

NOTE

'Grind' in the plant refers to the fineness to which rock is grinded in the plant to allow maximum recovery of the mineral.

This material is used for backfill purposes without affecting or changing the particle dimensions (Outcome 4.2.2.1.4) at all. Water is added to the full plant tailings on the tailings dam (often called 'monitoring') and the water and tailings product is pumped to storage bins (often called 'pachukas'), from where it is transported to the workings as backfill. In many cases, the product in the bins is first filtered to remove water and then water is added under controlled circumstances to ensure the required density is obtained before it is sent underground.

Backfill additives can be added to the full plant tailings to enhance its performance and can include additives to:

- improve water separation from the tailings, called 'flocculants';
- achieve higher early strength with minimal shrinkage, often

cement based, but trials with gypsum and other chemicals have also been done in the past. Chemicals to slow down or speed up cement setting times can be added as well.

- can be transported over last distances, called 'plasticisers'.

These additives can be added at various points:

on surface:

- this includes plasticisers, as pumping or flow over large distances is often required;
- cement addition at this point would also require the addition of 'sleeper' additives that postpone setting of the cemented fill, but are obviously high risk as failure of the pumping system could result in the large-scale loss of pipes if the setting eventually commences;

at the underground site:

- Flocculant to improve filling quality as the water is drained easier and faster from the backfill;
- Cement to improve early strength and reduce water run-off and slumping, but control of this addition is often difficult as measurement systems (to ensure correct % cement addition) are most often crude and not necessarily trustworthy;
- Chemicals to speed up cement setting are often added at the place of fill placement, especially in steep dipping ore bodies where backfill 'flow' must be controlled to limit the risk of uncontrolled backfill flow to gullies, etc.

Systems to manage the manufacturing and provision of this fill include:

Plant :

- Creates the tailings after grinding and extraction of the mineral.

Tailings dam:

- The first storing facility for the fill.
- Equipment for monitoring is required and may include water jets, piping and pumps to transport the tailings and water mix to the backfill plant.

Backfill plant:

- This plant may exist of simple storage bins with large 'agitators' maintaining a good mix and preventing settling out of the solid material in the bins.
- 'Agitators' are simply large propellers within the bins that keep on mixing the material, much like when mixing dough when baking, but have been known to also include systems where compressed air is forced through the tailings and the bubbles ensure mixing of the material.
- Piping to receive and transport tailings down the mine, with pumps when pumping is required and free-flow using gravity is not the driving force in the transport method.
- Filters to remove water and allow density controls to be put in place.

Underground:

- Storage bins at various levels if the transport distances are large with agitators to maintain a good material mix.
- Piping to transport the material.
- Storage bins for the fill at the filling site if cement is added at the delivery point.
- Storage bins and pumps for any additives that are added at the delivery point.

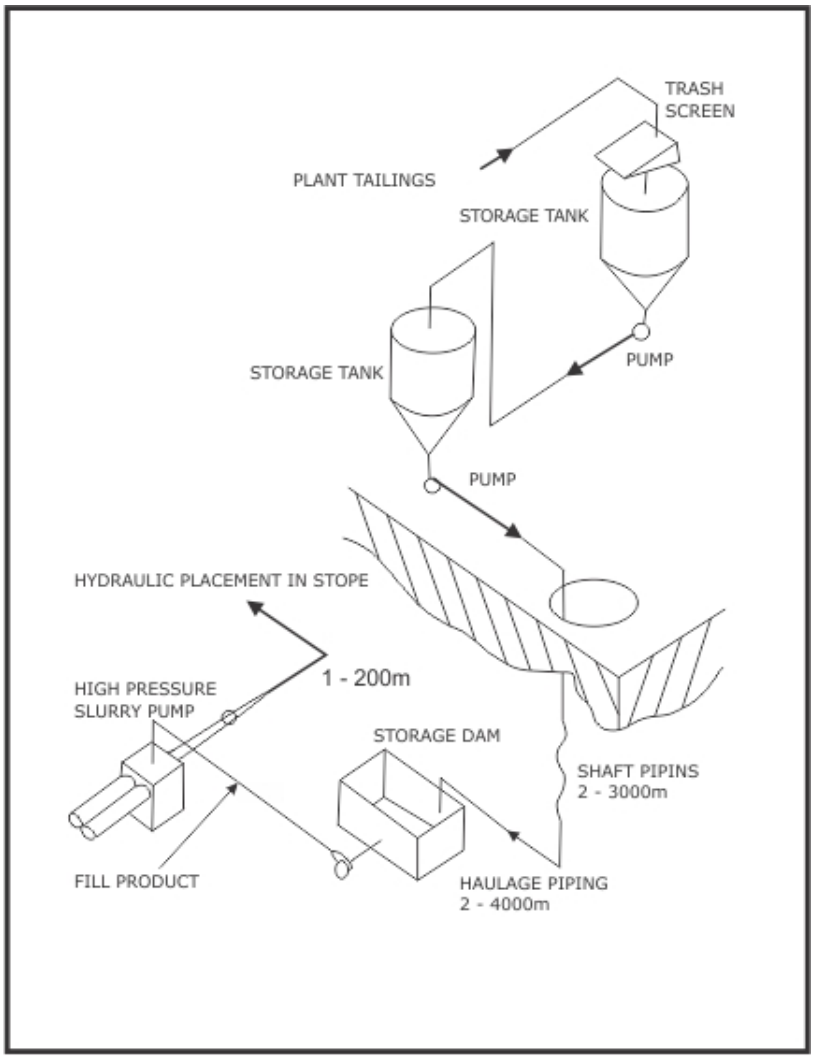


Figure 42: Typical PFT system

Classified tailings fill

Classified cycloned tailings (CCT), sometimes called sand fill, are prepared from concentrator tailings by hydrocyclone treatment to remove the slimes or clay-sized fractions. In most cases, the highest proportion of the fill product lies in the 40-150µm range of particle size and the proportion of -10µm material is less than 4%. The -10µm fraction is most critical in terms of percolation and must be removed by hydrocycloning.

'Classification' or 'cycloning' therefore refers to the fact that the material particle dimensions are controlled and that smaller particles are removed from the original tailings to provide the fill material type required for the specific application. Cyclones separate the material according to dimension by applying the fact that different particle dimensions behave differently when subjected to centrifugal forces within the cyclone. The cycloning process also allows the creation of a particle size controlled product and well-graded materials ("LEARNING OUTCOME 4.2.2.4") can be provided on a constant basis. By removing especially the smaller particles form the material, drainage is improved through higher porosity, but strength can also be reduced by this porosity increase. However, fines are most often responsible for clogging of pipes and paddock construction material and its removal from the tailings reduces transport and placement concerns.

Additives are similar to those discussed for the FPT above and can be applied in a similar manner.

back

Systems to manage the manufacturing and provision of this fill include:

Plant:

- Creates the tailings after grinding and extraction of the mineral-Tailings dam:
- The first storing facility for the fill.
- Equipment for monitoring is required and may include water jets, piping and pumps to transport the tailings and water mix to the backfill plant.

Backfill plant:

- Cycloning equipment to separate the fines from the material with bins and pipes to manage the cyclone final product before transporting it underground as well as to remove the fines to an alternative tailings dam.
- This plant may exist of simple storage bins with large 'agitators' maintaining a good mix and preventing settling out of the solid material in the bins.
- Piping to receive and transport tailings down the mine, with pumps when pumping is required and free-flow using gravity is not the driving force in the transport method.
- Filters to remove water and allow density controls to be put in place.

NOTE

Agitators can, if material is kept in bins for too long, start to break down the material particle dimensions again and may negate the whole cycloning process. Care must be taken to ensure that material provided is utilised for filling as soon as possible.

back

back

INTERESTING INFO

The following flow sheet for comminuted waste filling systems is presented for interest.

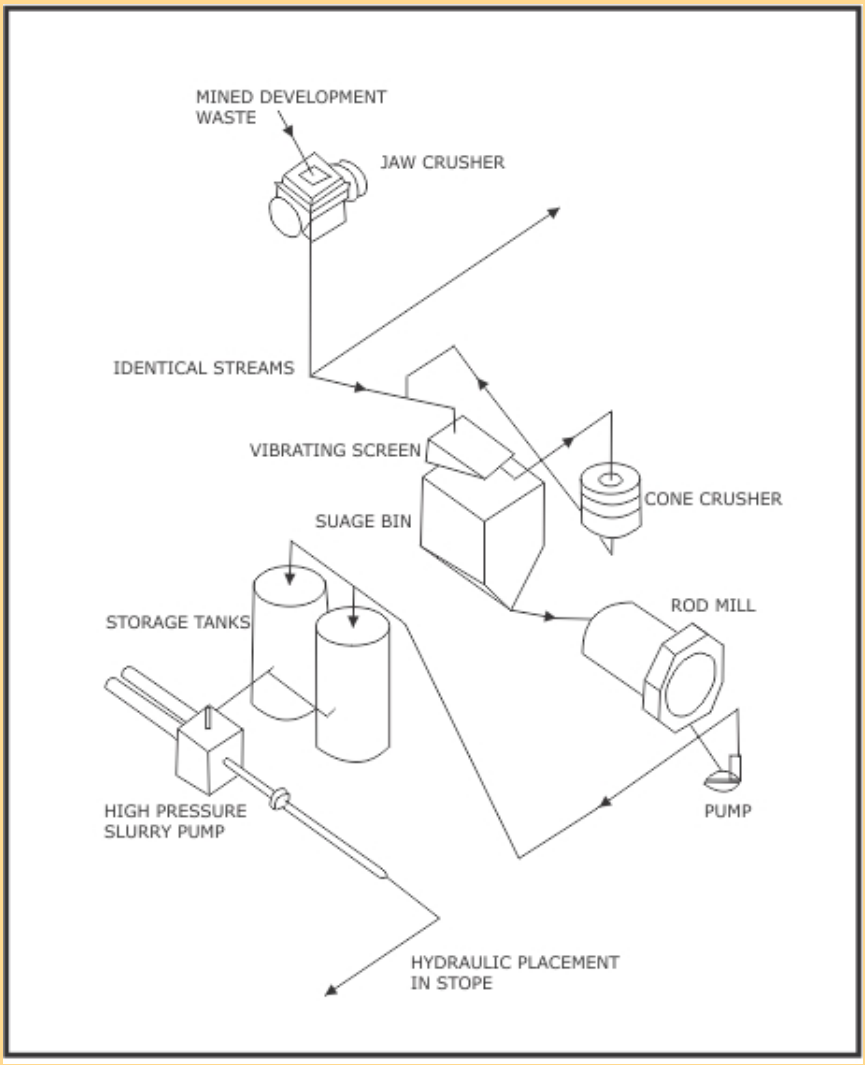


Figure 44: A typical waste fill system

Additional information on the:

- particle size distributions
- stress-strain relationships and
- advantages / disadvantages

of the different fill systems is provided in Figure 45, Figure 46 and Table 24.

back

back

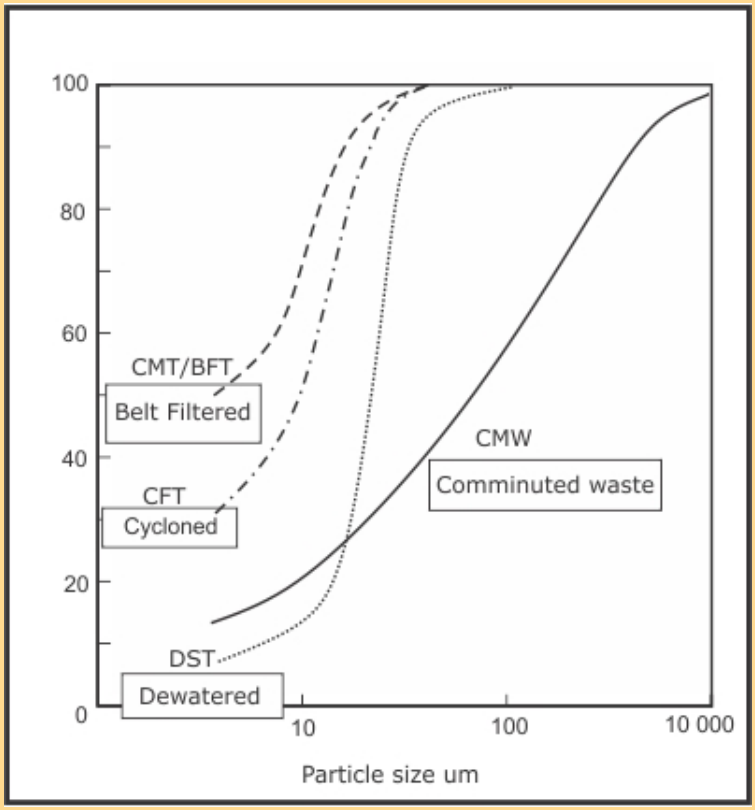


Figure 45: Particle size distributions for different fill types (typical)

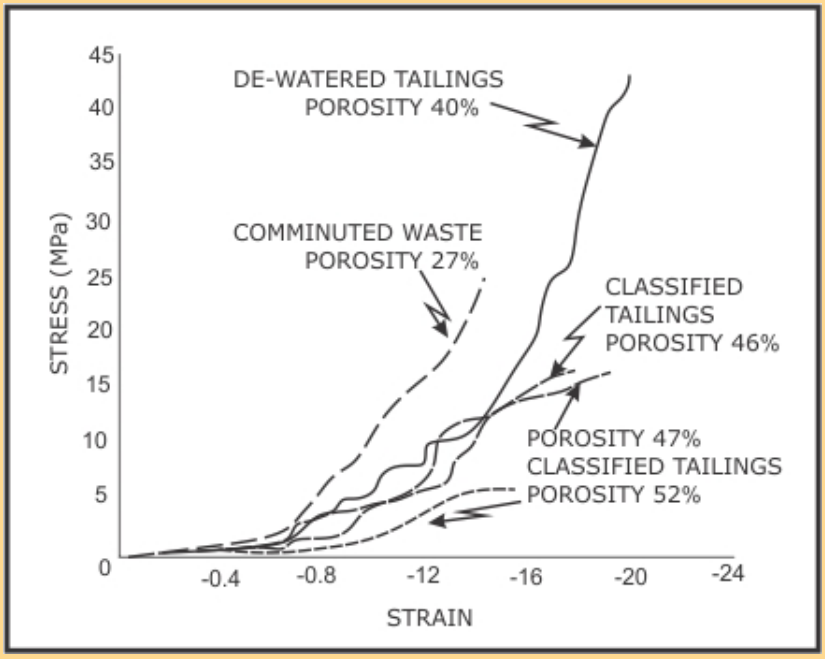


Figure 46: Stress-strain relationships (typical) for various fill types

System	Advantages	Disadvantages	Remarks
Classified Cyclone Tailings (CTT)	Simple method of conveying large volumes of fill from surface directly into areas to be filled Has good drainage characteristics	Large volumes of excess water have to be handled underground Use of only the coarser fraction, leaving the finer fraction for tailings disposal creates potential problems with the building of slimes dams More difficult to pump than full plant tailings	Sufficient backfill material must be available

back

Cemented Full Plant Tailings	Simple method of conveying large volumes of fill from surface directly into areas to be filled Maximum use of tailings product Very good pumping characteristics	P{oor drainage characteristics High moisture content requires greater quantities of binder	The binder material assists by hydrating the excess water retained by the fines.
Slurry Fill	Very little run-off water Can be used with full plant tailings The total cost of this system could be less than cemented CTT	Requires the availability of a special binder Accelerator is costly and requires accurate dosaging	Technically this system is good
Past Fill	Relatively small amounts of cement (3-5%) produce stiff backfill (1.5 -3.5MPa) Reduced tailings impoundment requirements Good support properties Reduced spills underground The entire tailings stream can be placed underground	Costly system which requires expensive pipes, pumps and dewatering equipment Conveying distances limited due to high pressure gradients (appr 1km horizontally) Good quality control is necessary	Pumping difficulties can be alleviated by pumping the fill underground as a slurry and then dewatering it close to the stopes.

Table 24: Advantages and Disadvantages of Fill systems (by Stacey TR)

Underground:

- Storage bins at various levels if the transport distances are large with agitators to maintain a good material mix.
- Piping to transport the material.
- Storage bins for the fill at the filling site if cement is added at the delivery point.
- Storage bins and pumps for any additives that are added at the delivery point.

The following impact of cycloning can be achieved:

Pipe wear:

- Wear reduces with increased % of -10µm particles;

Placement:

- More effective placement with decrease in % of -10µm particles;

Compression:

- Local support: Negligible impact on compression by % of -10µm particles.

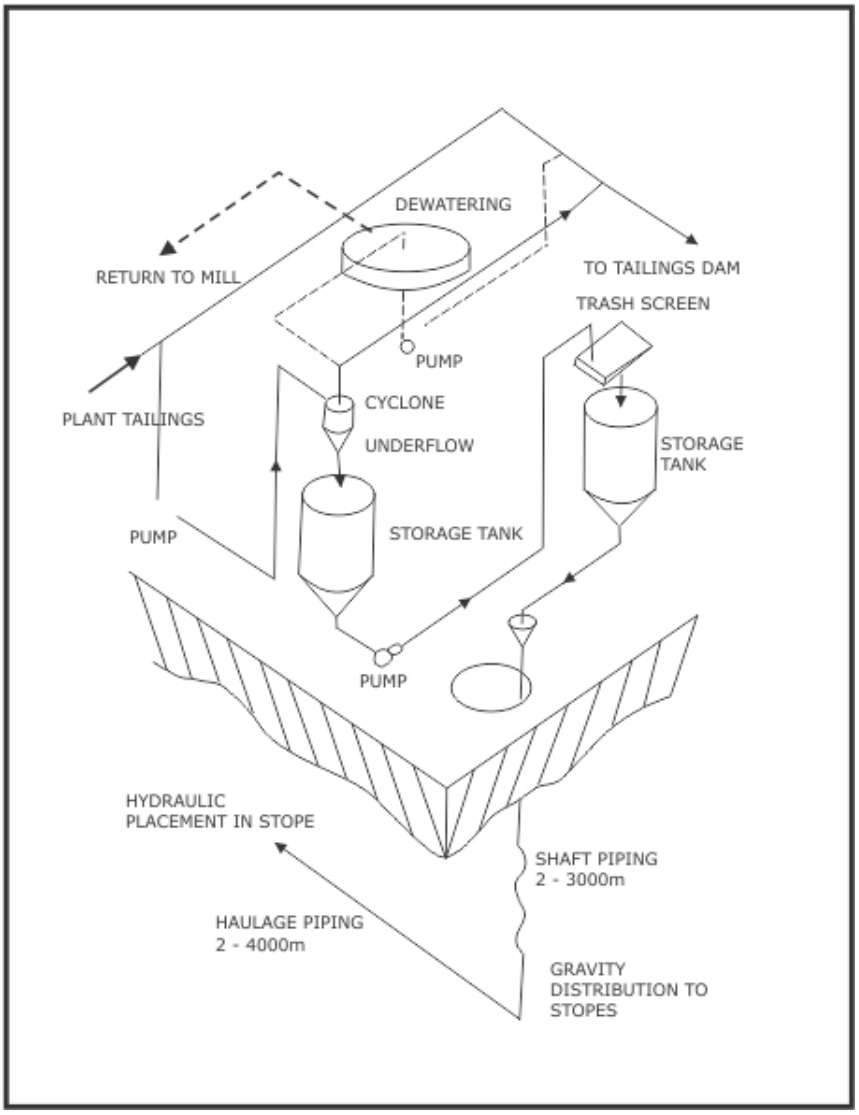


Figure 43: Typical CCT system

Paste fill

Paste backfill is a high density backfill that uses full plant tailings and means that the water content of the material is very low compared to slurry backfill. Paste fill systems are generally regarded as the "Rolls Royce" of backfill systems and their popularity is increasing. They are high-capital and operating cost systems that require high-tech paste pumps, disc or belt filters and other capital intensive equipment. Paste fill is mostly utilised in shallower operations where the fill transport distances are limited and where pipe friction levels can be managed.

INTERESTING INFO

Moisture content influences not only the slump of the product, but also the pressure gradient, and must be well controlled. For example, a 2% reduction in moisture content from 24% to 22% by mass can result in an increase in friction loss in the pipeline from 10kPa to 20kPa per metre. Control of the moisture content is therefore the most critical aspect of the backfilling operation.

Additives are similar to those discussed for the other fill types above and can be applied in a similar manner, even though paste fill systems generally have limited transport distances and cement addition on surface is a general practice.

Systems to manage the manufacturing and provision of this fill include:

Plant:

- Creates the tailings after grinding and extraction of the mineral.

Tailings dam:

- The first storing facility for the fill.
- Equipment for monitoring is required and may include water jets, piping and pumps to transport the FPT and water mix to the backfill plant.

Backfill plant:

- This plant may exist of simple storage bins with large 'agitators' maintaining a good mix and preventing settling out of the solid material in the bins.
- Filters (disc or belt) to remove water and allow water addition controls to be put in place to create the required paste fill design.
- Piping to receive and transport tailings down the mine, with pumps to transport the higher density material to the placement sites.
- Bins to store cement and other additives and pumps to control addition rates.

Underground:

- Piping and pumps to transport the material to the site.
- Storage bins for the fill at the filling site if cement is added at the delivery point.
- Storage bins and pumps for any additives that are added at the delivery point.

NOTE

The filling system for the paste fill is very similar to the cycloned system, but has the following differences:

- *filters replace the cyclones;*
- *high pressure pumps are utilised underground (similar to the FPT system).*

REFERENCE

- Cooke R, Patterson & Cooke (2007) Backfill Pipeline Distribution Systems – Design Methodology Review Mine fill 2007 Symposium
- Cooke R (2001) Design Procedure for Hydraulic Backfill Distribution Systems SAIMM
- Malan DF, Piper PS, Clements T & Hayes SJ (2005)Development of Practical Guideline for Support Quality Assurance for Different Systems on Tunnels and Stopes
- Ryder JA &Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines
- Stacey TR (2001) Best Practice Rock Engineering Handbook for "Other" Mines

LEARNING OUTCOME 5.3.4

DESCRIBE AND EXPLAIN THE USES OF CEMENTITIOUS BINDERS IN BACKFILL

Cementitious binders are used to bind or bond the fill particles together thereby providing a significant cohesive component of strength to the medium. It includes, but is not restricted to cement. A wide variety of man-made and natural bonding agents have been used. The natural ones, mainly pyrite and pyrrhotite, are usually sulphide minerals. Upon oxidation, these minerals have a strong bonding action, but the reaction is exothermic and emits heat and noxious gases.

Cement is used in two ways in fill. The first application is at an ultra-low cement content (20:1 to 40:1 sand: cement ratio) in the bulk of the fill material. The second is at higher ratios, 10:1 to 5:1, for use in slushing and mucking floors in cut and fill operations. At the higher ratios, conventional cement and concrete technology generally applies, although the composition of the sand and water has some effect on the strength.

back

The addition of cement, due to costs and practical difficulties, must have some benefits to the user to justify its use. Figure 47 shows one of the benefits as the increase of initial stiffness of the backfill when cement is added. When stress is applied to the cemented backfill, the material deforms elastically until yielding (onset of failure) occurs. At this point, the fill starts to behave in a more plastic manner and reverts towards the stress-strain relationships that would have been provided by an un-cemented fill of the same type. However, cemented fill is still slightly stiffer at large strains.

NOTE

Increases in the width:height ratios of the backfill ribs or paddocks will maintain high stress generation at similar strains (Figure 47).

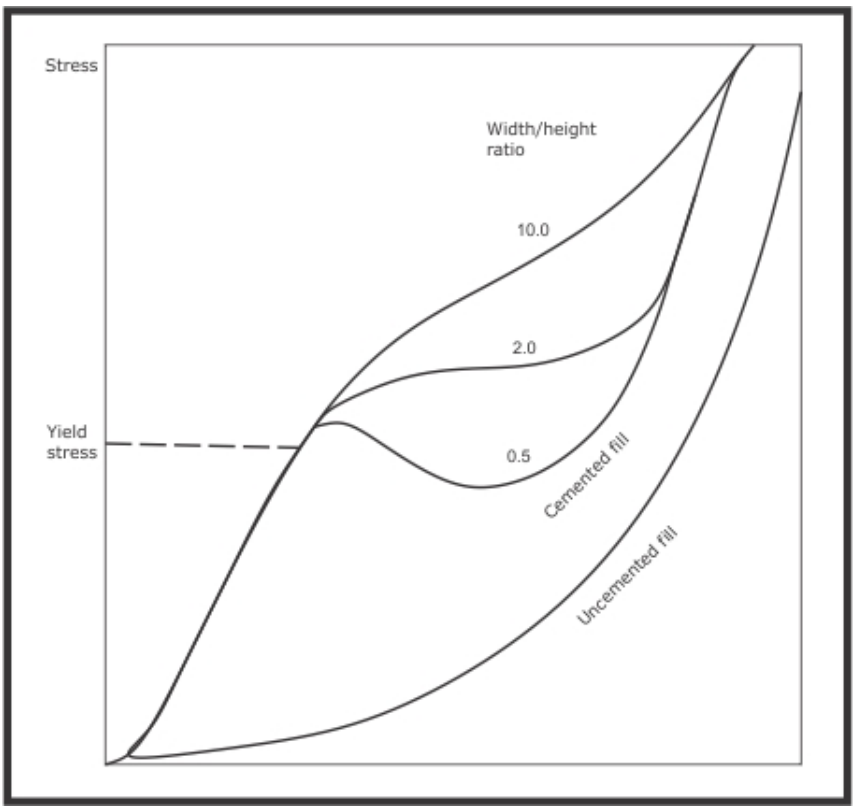


Figure 47: Impact of cement addition on stress-strain behaviour

back

back

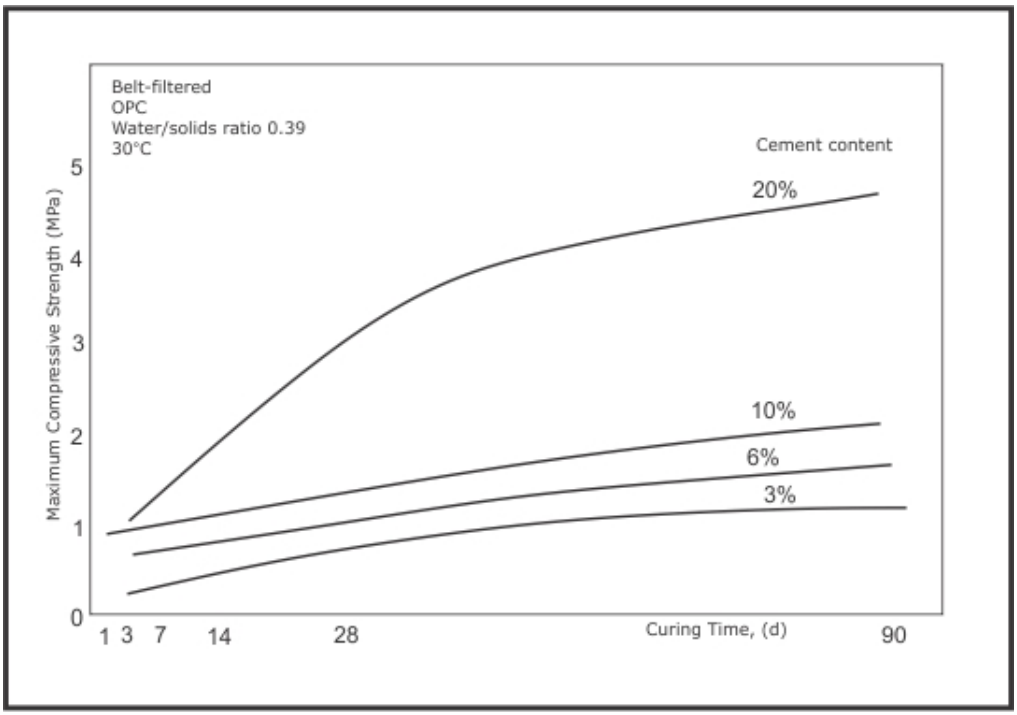


Figure 48: Cement addition % impact on strength development with time

Figure 48 shows that increased cement content (% addition) increases the strength of backfill products, with the largest impact on later strength development (long-term strengths).

IMPORTANT

Recent work has indicated that some cement types may cause deterioration of long-term strengths (several months). The user should ensure that tests on cemented fills are conducted if the long-term strength is critical to the mining method or regional stability.

Cement addition to different fill types also affects the strength of the resulting backfill, as shown in Figure 49, with increased strengths when waste and paste fills are used compared to that attained by the cyclone tailings.

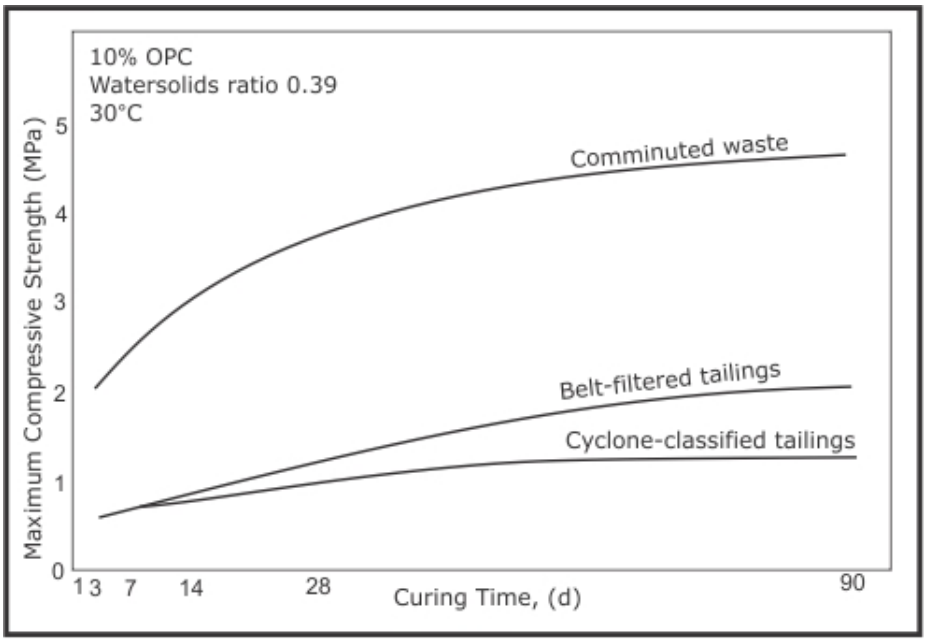


Figure 49: Strength development in different cemented backfill types with time

INTERESTING INFO

Pulverised fuel ash (PFA) can be added to cemented fill to increase its strength. In other words, methods of attaining additional strengths are possible without necessarily increasing the % cement content. Figure 50 shows the strengths to be gained by adding different ratios of PFA to the cemented fill product. PBFC (Portland blast furnace cement) outperforms the other additives in Figure 50, due to its

back

back

DEFINITION

OPC is ordinary Portland cement.

Pulverised fuel ash (PFA) is a by-product of pulverised fuel (typically coal) fired power stations. The fuel is pulverised into a fine powder, mixed with heated air and burned. Approximately 18% of the fuel forms fine glass spheres. They are extracted from the flue gasses by cyclones and electrostatic precipitation. The resultant material is used as engineering fill and as a component for concrete. PFA can undergo a pozzolanic reaction and become brittle over time. PFA has a fine dust texture and is grey in colour.

PBFC is Ground-granulated blast-furnace slag (GGBS or GGBFS) and is obtained by quenching molten iron slag (a by-product of iron and steel-making) from a blast furnace in water or steam, to produce a glassy, granular product that is then dried and ground into a fine powder. GGBS is used to make durable concrete structures in combination with ordinary Portland cement and/or other pozzolanic materials. GGBS has been widely used in Europe, and increasingly in the United States and in Asia for its superiority in concrete durability, extending the lifespan of buildings from fifty to a hundred years. Two major uses of GGBS are in the production of quality-improved slag cement, namely Portland Blast-furnace cement (PBFC) and high-slag blast-furnace cement (HSBFC), with GGBS content ranging typically from 30 to 70%; and in the production of ready-mixed or site-batched durable concrete. Concrete made with GGBS cement sets more slowly than concrete made with ordinary Portland cement, depending on the amount of GGBS in the cementitious material, but also continues to gain strength over a longer period in production conditions.

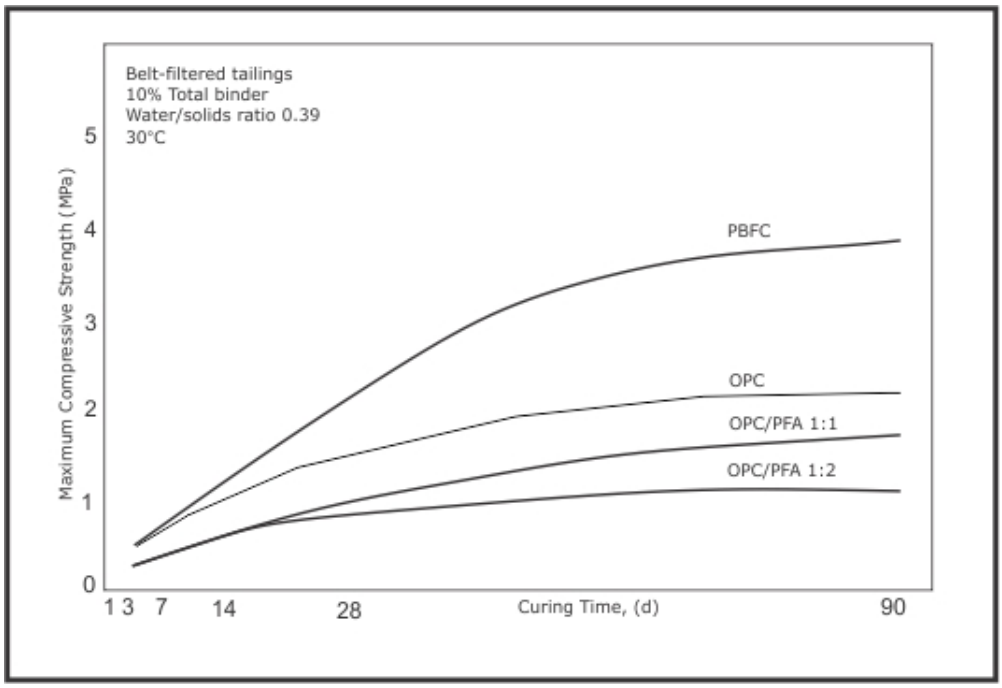


Figure 50: Impact of OPC and PFA ratio on backfill strength

INTERESTING INFO

Pozzolanic is a term often used when discussing cement / concrete technology. Basically, it means that the material being discussed exhibits cementitious properties.

CONNECTION 44

Refer to the discussion on backfill types, methods and applications in "LEARNING OUTCOME 4.2.2.1" on page 243

REFERENCE

- Godbout & Bussier, Cemented backfill
- Benzaazoua et al, Hardening process of cemented backfill
- Farzam et al, Unique admixture technologies in mining backfill
- Budavari S (ed) (1983) Rock Mechanics in Mining Practice

back

LEARNING OUTCOME 5.3.5

SKETCH, DESCRIBE AND EXPLAIN IN PRINCIPLE HOW BACKFILL IS TRANSPORTED FROM PREPARATION PLANT TO UNDERGROUND STOPE

Before addressing backfill flow behaviour and pipeline hydraulics, it is worth reviewing the requirements and expectations of a backfill distribution system:

- The system must operate in a reliable and safe manner and not expose mine personnel to operational hazards.
- The system must deliver the flow rate or tonnage to a number of different locations and levels within the mine. Typically, the distribution piping is extended as mining operations proceed and the delivery flow rate should remain within fairly narrow limits.
- The system should operate without any blockages.
- The piping system should not fail due to excessive pressures or worn piping. Importantly, the piping should wear at a predictable rate to enable the effective planning and execution of pipe replacement schedules.

A well-designed system will meet these requirements, with the provision that the backfill quality is within the defined limits.

Hydraulic design of backfill distribution systems – Design process:
The basic process of designing a backfill system entails the following:

- Establish the design duty specification for the system. This must take into account any planned variations in production during the envisaged system life.
- Determine the pipe routing. Generally, the route is largely dictated by the mine layout, although there may be scope to optimise the routing.
- Establish the backfill flow behaviour characteristics.
- Perform the hydraulic analysis of the system considering all envisaged operating conditions. The output of this analysis will be the preliminary piping specification. Other hydraulic components such as pumps, chokes, and valves will also be specified at this stage.
- Undertake the mechanical design of the piping and piping support system. Depending on the output of this activity, changes to the piping specification may be required.
- Specify operating, monitoring, and maintenance procedures for the backfill distribution system.

Pipeline routing:
The distribution piping routing is largely defined by the mine layout. However, there are usually some opportunities for optimising the piping layout and the following issues should be considered:

- Ideally, the slope of the piping should reduce along the distribution system, i.e. vertical down sections should be at the start of the distribution system, with horizontal piping towards the end of the system.
- It is preferable to have a series of vertical piping sections instead of a single vertical section. This reduces the pipeline operating pressures.
- Pipes may be installed in shafts to reduce installation costs, although this is generally not preferred due to safety considerations.
- Inter-level boreholes can be used to reduce the total pipeline length and to optimise the pipeline profile.

back

Evolution of backfill distribution systems:
Backfill is transported by pipeline from the surface to stopes underground, where the backfill is placed in paddocks. There are two modes by which backfill is transported in a shaft column, i.e. free-fall or full-flow.

Free-fall systems
Figure 51a shows an idealised layout for a free-fall backfill hydraulic-transport system. The backfill slurry is supplied to the shaft column in which it falls freely under gravity until it reaches the air-slurry interface. The height of this interface is established so that the static head available (due to height Hi in Figure 51a) balances the frictional losses in the pressurised section of the pipeline for a particular flow-rate. Therefore, if the backfill flow-rate is increased, the level of the air-slurry interface will rise owing to the increased frictional losses.

The disadvantages of free-fall systems are as follows.

- High velocities are attained in the free-fall zone, resulting in extreme rates of pipeline wear.
- High impact pressures generated at the air-slurry interface may lead to pipeline 'bursting' failure.

Full-flow systems
A full-flow, or balanced backfill hydraulic-transport system is shown in Figure 51b. The air-slurry interface is maintained at surface level by ensuring that the pressure head available is matched by the frictional losses in the system. The advantage of the full-flow system is that pipeline wear rates, and hence failures, are minimised. The difficulty of implementing such a system is that for slurry velocities below 3 m/s the static head available is far greater than the frictional losses. The following measures have been taken to dissipate the excess energy.

- Ceramic orifices have been installed at pipeline flanges. However, these orifices have led to localised wear downstream of the choke and, consequently, pipeline failure.
- Lengths of small-bore pipe have been installed and successfully used to dissipate excess energy. However, this is regarded as only a temporary measure since wear rates in small-bore pipelines are excessive owing to the high slurry velocities, and long lengths of pipe are required to dissipate the excess energy (typically over 500m).
- Various inline energy dissipaters have been developed to dissipate large amounts of energy in short lengths of pipelines. These backfill systems are referred to as dedicated-range systems since the pipeline is able to supply backfill to a maximum of only three stope panels.

Pressure-break systems
The layout of a typical pressure-break (gravity-dam) backfill-distribution system is shown in Figure 51c. The backfill slurry is supplied to an underground dam by pipes of large diameter (100 to 200mm nominal bore). The slurry is gravity-fed from the underground dam to the slopes in pipes of smaller diameter (50 to 80mm nominal bore).

The advantages of underground storage dams are as follows.

- Large-diameter pipes are used in a vertical column, considerably reducing the number of pipelines in a shaft. Consequently, the installation of the column requires minimal shaft time.
- The maximum static head that can occur in the pipeline in the event of a blockage is less than that in a dedicated-range system.

back

back

The following operational problems are associated with the pressure-break system:

- The shaft column supplying the gravity dam generally has a relatively short horizontal distance. In order to establish a full-flow system, very high mean mixture velocities are required to balance the frictional resistance with the static head available. This leads to high rates of pipeline wear. It is preferable to operate at lower mean mixture velocities by dissipating the excess energy to ensure that full-flow conditions are maintained in the shaft column.
- For a particular backfill-supply pipeline from an underground dam, the static head remains constant while the horizontal distance increases as the mining and backfilling operations progress. Therefore, the quantity of backfill supplied to the stopes decreases as the length of the horizontal pipeline increases.

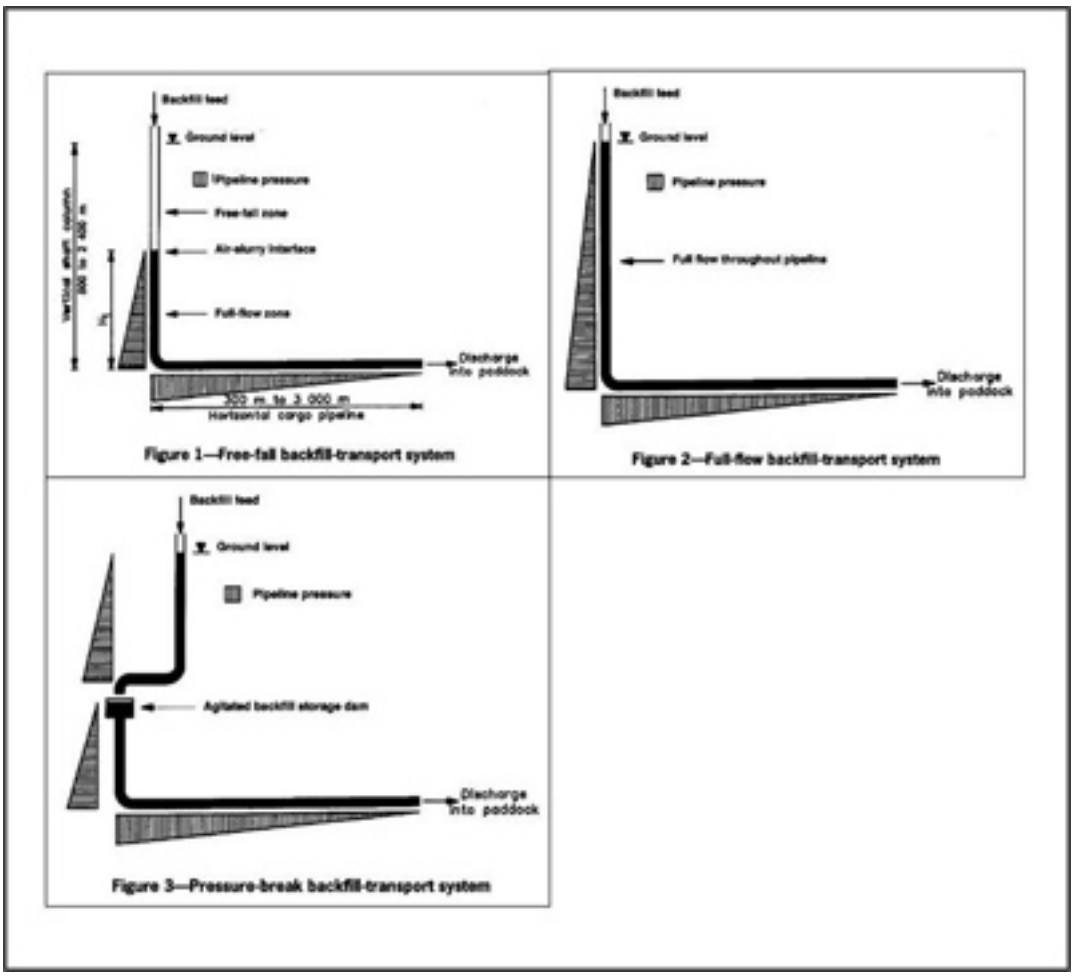


Figure 51: Backfill transportation systems (a) free-fall, (b) full flow and (c) pressure break

REFERENCE

- Cooke & Spearing, Impact of cement on Transportation of backfill paper
- Cooke R, Patterson & Cooke (2007) Backfill Pipeline Distribution Systems – Design Methodology ReviewMine fill 2007 Symposium
- Cooke R & Lazarus JH (1993) Hydraulic Transport Systems for the Backfilling of Deep Mines SAIMM Vol. 93 No. 2

back

LEARNING OUTCOME 5.3.6

SKETCH AND DESCRIBE THE METHODS OF CONTAINING BACKFILL IN STOPES

Backfill paddocks are constructed using bags (also called socks) or 'mesh-and-fabric' and fastened to mine poles or elongates. In both cases, it is important to confine the backfill since bulging edges generate very little load against the hanging wall.

Backfill bags/socks
Typically, the backfill is pumped tightly into porous 'backfill bags' that retain the +45 micron sediments, yet release the water, to leave a restricted compression structure to resist and contain closure. Backfill bags shall be manufactured from a synthetic polymer woven geo-textile material that complies with the controlled quality specifications as per SABS 0221-1988 code of practice for the testing of geo-textiles.

The backfill bags are constructed of fabric that is self-supporting and allow for rapid and continuous drainage. The fabric must have excellent retention qualities allowing for the rapid settlement of sediments thereby providing initial support.
The backfill bags often have ties on the outside of the bag approximately every meter, longitudinally along the upper corners of the bag and at the closed end.

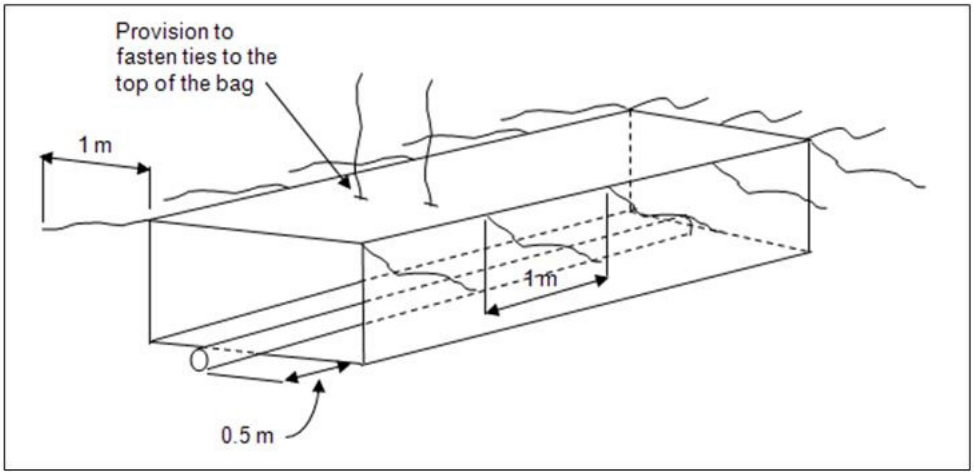


Figure 52: Schematic of a typical backfill bag with ties

Backfill paddocks
A generalised installation procedure of a backfill paddock follows:

- The area where the paddock is to be built must be barred down and all bad hanging supported before any work commences. Start construction from the bottom of the panel to the top. The most critical point on the bag is the bottom of the bag. If needed, double mesh and additional elongates must be installed to strengthen the bulkhead as this area of the bag must bear most of the weight.
- Install mining poles at a maximum spacing as per mine standard on dip and on line where the new paddock is to be constructed. Install elongate support in such a manner that there are no sharp protrusions facing the bag to be filled.
- Roll mesh out from top to bottom inside the paddock with roll tension towards the footwall. Install mesh from top working downward and tension by hand as stiff as possible and fasten onto the mining poles with wire staple or nails. The whole area

from hanging- to footwall must be covered with mesh. Use additional mesh if required and ensure mesh is overlapping old fill at top and bottom for at least 1m on dip. In high stoping widths, use double mesh to ensure the full width/height is covered, from footwall to hangingwall.

- Install cloth from the top of the panel downward to the gully fastening it right in the corners against the hanging wall onto every stick and secure with wire staples or nails. A 0.5m overlap must be left against the hanging wall by folding the cloth up twice and nail it against the sticks. Install a second line of slack half way between the hanging wall and footwall. Leave an overlap of not less than 0.5m on the footwall. The cloth must extend at least 3m on to the old fill at the top and bottom of the fill. Nail or staple the 3m extension onto the sticks of the old fill with nails or staples. When installing the bag, the ties must be supported as close to the hangingwall as possible. Allow sufficient slack at corners and at pack and brows.
- Bulging backfill often removes other support types installed too close to the paddocks, and can cause the bag to tear. Ensure that you adhere to the mine-specific standards with regard to support installation distance from backfill paddocks.

Figure 53: Backfill paddock construction with mesh and fabric

LEARNING OUTCOME 5.3.7

DESCRIBE AND EXPLAIN THE METHODS OF CONTROLLING THE QUALITY OF BACKFILL

The variables that need to be monitored as part of the quality control process to ensure that the backfill placed meets the design criteria and load bearing requirements include:

- Particle size,
- Water run-off,
- SG or RD (specific or relative densities),
- Binder or cement addition,
- In-situ backfill stress/load,

- Stope closure rate,
- Backfill strength, and the
- In-situ strain rate.

The proper performance of a backfill distribution system is dependent on the backfill properties being within the specified range. To minimise the risk of operational problems associated with "out of specification" backfill, consideration should be given to the installation of a pressure gradient loop on surface. By measuring the flow rate, density and pressure loss across a straight horizontal length of pipe, an indication of the backfill properties can be gathered. This allows corrective measures to be implemented at the backfill plant or, if necessary, to take pre-emptive action to avoid a blockage of the distribution system.

In order to ensure the required backfill properties and flow behaviour are met, the following tests can be conducted:

Slump test
The standard 300mm concrete slump cone has become the industry standard for paste plant quality control and communicating paste properties (e.g. 150mm slump paste). High slump values indicate low rheology or "viscosity," while low values indicate a "stiff" paste. The typical range of slump values used in paste fill applications is 150 to 250 mm. The slump test can only be meaningfully applied to paste backfill, i.e. it is not applicable for hydraulic backfill. Any test work for the design of paste distribution systems should be conducted using fully-sheared material.

Friction loss versus flow rate
There is a variation of pressure gradient with flow rate for typical hydraulic fill and slump paste fill in a pipeline. The friction loss for a typical paste is much greater than hydraulic fill for an equivalent pipe size and velocity. It is also important to notice that the hydraulic fill pressure gradient is much more sensitive to changes in flow rate than the paste fill pressure gradient.

Laminar flow settling
Some mines have experienced operational problems related to the settlement of coarse particles in cemented high-slump paste fill lines. Generally, laminar flow settling should not be a problem for pastes with a slump of less than 200mm.

Generally, sufficient knowledge is available on the flow behaviour of slurry backfills, allowing design and implementation to proceed based on laboratory scale test work.

Test work is required to establish the flow behaviour of paste mixtures. For preliminary investigations, this work can be conducted in laboratories using slump tests, viscometers, and small-scale pipe loops. However, for final engineering and specification of the system, pipe loop tests should be conducted on site. The following points should be considered:

- The pipe loop tests should be conducted in at least two pipe diameters, although three are preferable, so that possible slip effects can be quantified. One of the pipe diameters should be the size anticipated for the distribution system.
- The paste should be as representative as possible. This includes tailings sampling, production method (sedimentation dewatering or filtration), flocculent type and dosage, cement/binder type and dosage, and shear state.
- Ideally, to minimise particle degradation effects, once through pipe loop tests should be conducted instead of using a recirculation system. This is more important for coarse particle paste and tailings aggregate mixtures.

Once the backfill has been placed, the rockmass response to the backfill can be measured be means of closure metres (closure and strain rate) and flat jacks (loads/stress generated by backfill):

- Stress generated in the backfill and closure measurements can be used to draft an in-situ stress-strain relationship for the placed fill and can be compared to the designed relationship;
- Strain rate can assist in determining the stage of mining when significant stress generation occurs.

The quality of the placed backfill material can be monitored as follows:

Strength (or cement addition):

- Representative backfill samples are taken, kept underground for two days to allow the backfill to settle and placed in a curing room where it is kept at a constant temperature of 30° to simulate the conditions underground.
- Seven-day, 14-day and 28-day strengths are gathered when cores drilled from the samples are tested to obtain the uni-axial compressive strength.

Particle size distributions:

- An appropriate sample of the backfill material (uncemented or prior to the addition of the cement) is sent for particle size distribution tests.
- Results are received in the form of Figure 45 and can be compared with the designed material distribution.

Filling quality:

- This refers to the % of mining height or paddock height filled (i.e. indicates the degree of non-hangingwall contact) and should be monitored rigorously as poor filling of the mining height will change the backfill behaviour. Poor filling will move the stress-strain graph to the right, along the bottom axis (see Figure 54) and will mean that any benefits to be obtained by the placement of backfill can either be postponed, reduced or completely negated.
- Monitoring of the fill vs mining height will provide sufficient data.

Density:

- Samples are taken from the backfill line and are weighed. The density is calculated by dividing the mass (kg) of the sample by the volume of the sample and is compared to the designed density.
- Density tests are conducted continuously and at least at hourly intervals. The variations registered will indicate whether shorter sampling periods are required.

Porosity:

- Porosity is critical for the stress generation of backfill and tests should be executed at least once per shift.
- The volume of the sample is taken directly after sampling. The volume is again determined after the sample has been left to dry out and the ratio between the (total volume - dry volume) and total volume will provide the porosity.

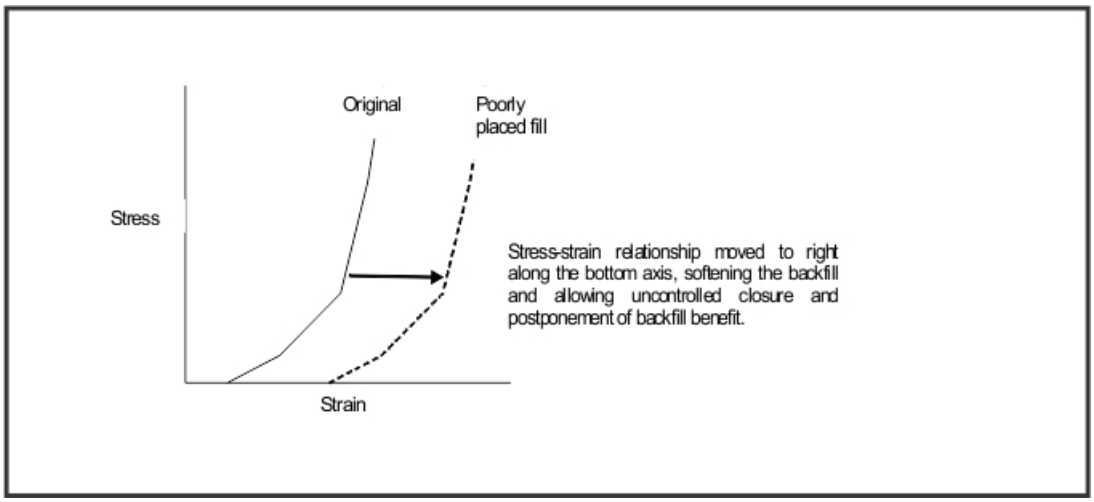


Figure 54: Impact of poor fill on stress-strain relationship

REFERENCE

- Cooke R, Patterson & Cooke (2007) Backfill Pipeline Distribution Systems – Design Methodology Review Mine fill 2007 Symposium
- Harmony-Target Mine Backfill Strategy Ref. Target RE 68-05

LEARNING OUTCOME 5.3.8

SKETCH, DESCRIBE AND EXPLAIN THE FEATURES OF BACKFILL SYSTEMS TO FILL PARTICULAR BLOCKS OF GROUND

Design of backfill for stoping operations must consider two sets of requirements:

- the in-situ backfill must meet the various criteria for 'self-support' to permit the excavation of the adjacent rockmass;
- its role as a functional structural element (support the rock mass) in the mine must be considered.

NOTE

It is possible to design backfill to perform specific functions, such as provision of support of stope boundaries or lateral confinement of pillars to improve their post-peak deformation characteristics. Modern use of backfill is often as a structural component in pillar recovery. Its application in underground mining also evolved from a need for achieving regional ground control above a mining area, i.e. controlling surface displacements above a mining area. Mining under sensitive surface features is frequently permitted by close integration of mining and backfill.

Below, mining methods (usually applied to specific blocks of ground) where backfill systems are used are described and explained:

Tabular underground stopes

The selection methodology for the installation of backfill in tabular underground stoping horizons is as follows:

- The expected in-situ stope width and closure rate should be determined, from which the applicable strain rate can be calculated.
- Uncemented backfill with the lowest achievable porosity is appropriate for all high strain rate conditions, particularly with respect to narrow stope widths at depths where acceptable initial stiffness can be obtained regardless of the backfill used, but through high strain (closure) rates. At the same time, the damage to the cementitious bonds in cemented fill could be damaged, reducing the advantage of cemented fill in these conditions.

back

- In the case of moderate to low strain-rate conditions, it is possible to use either cemented or uncemented backfill depending on whether or not damage occurs to the cementitious bonds. Cemented backfill should not be selected if cementitious bonds would be damaged, except as a means of reducing drainage water in stopes where this is considered to be a problem.
- For narrow stopes at shallow mining depths (i.e. low strain rates) when a high early stiffness is required, e.g. in multi-reef mining situations, cemented backfill should be considered. However, if backfill is required only for ventilation control or for 'dead-weight' support, an uncemented backfill may be sufficient.

Below is the schematic representation of the methodology that can be followed for selecting the appropriate backfill type for use in tabular stoping horizons (after Squelch 1993).

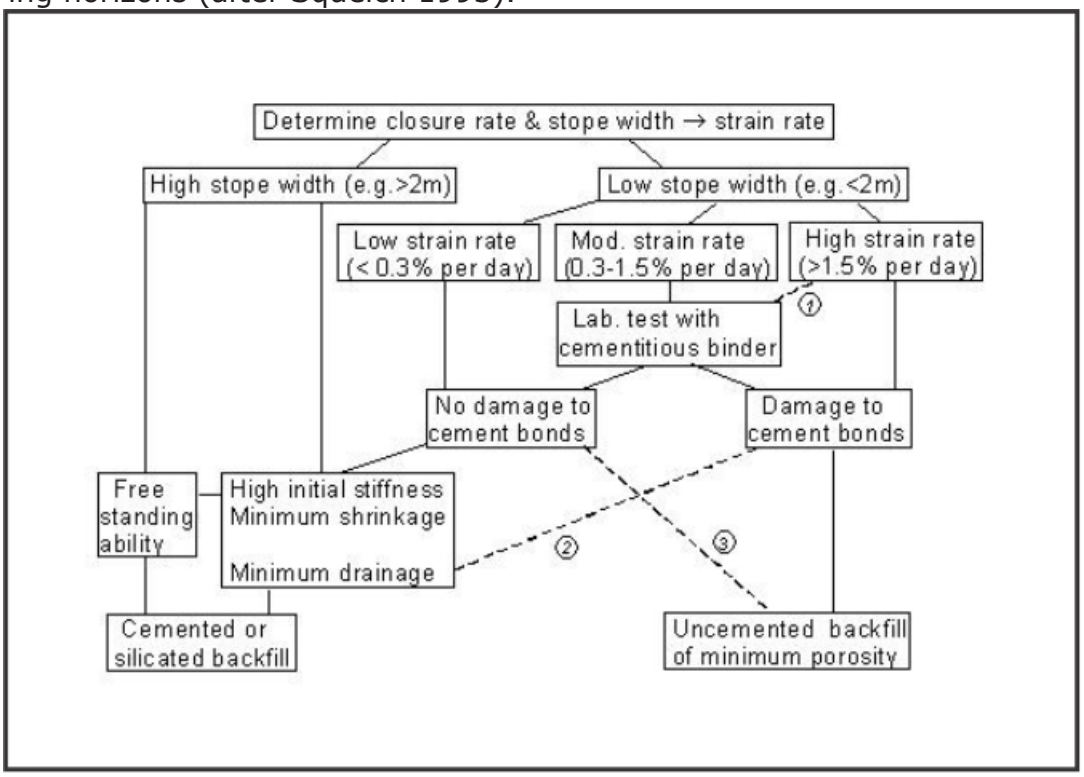


Figure 55: Schematic representation of the methodology to be followed for selecting the appropriate backfill type

In wide stopes (e.g. wider than 2m), in deep or shallow mining situations, cemented backfill should be selected because strain rates are likely to be low with no damage to cementitious bonds, while free-standing and low shrinkage characteristics are required.

The mining cycle should be designed to ensure that the backfill is kept close to the stope face and that appropriate rockfall or rockburst props are installed as face-area support, in order to fully realise the effective support system provided by the use of backfill.

Cut-and-fill stoping

Cut-and-fill stoping is performed mainly in narrow steeply dipping ore bodies, where the stope peripheral rock cannot sustain stable, free-standing spans suitable for open stoping. This mining method involves the incremental development of a slot-shaped excavation. Fill is placed in the mined excavation after each increment of sub-vertical stope advance. Because miners work beneath rock surfaces (in overhand stoping) or cemented fill (in underhand stoping), which are regenerated with each mining cycle, the success of the method is dependent on effective ground control. The function of the placed fill is to prevent disintegration of the stope rock wall.

back

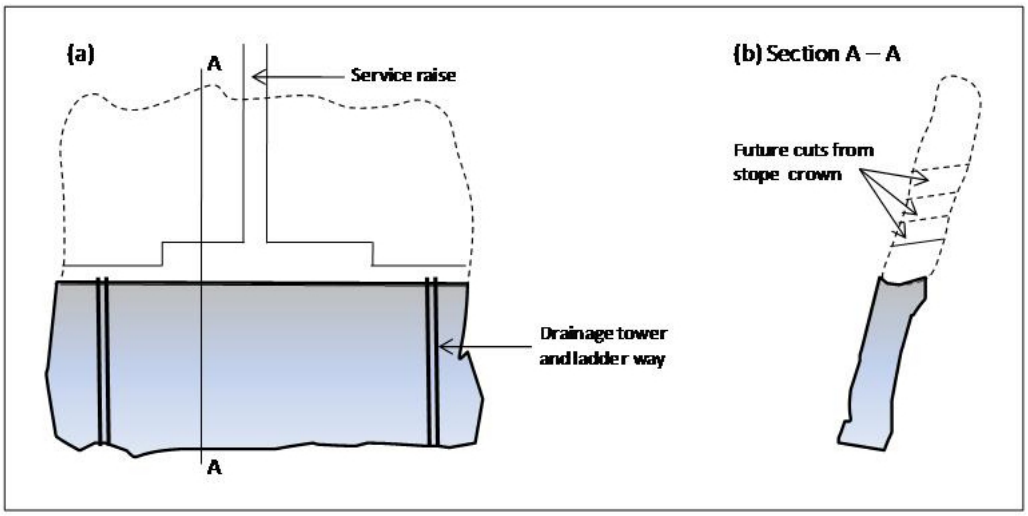


Figure 56: Typical cut-and-fill stoping

Backfill applications in open stoping

Open stoping is a naturally supported mining method, in which control of rockmass displacements is achieved by the generation of ore remnants to form support elements in the ore body. In general, the stope filling operation in this method of mining is not as closely integrated in the mine production as it is in cut-and-fill stoping. However, in both cut-and-fill stoping and open stoping with filling and pillar recovery, the support potential of the fill is exploited to achieve a high proportion extraction of the ore reserve.

In many mining applications, an essential property of the fill mass is a capacity to sustain the development of a large, unsupported fill surface. The figure below illustrates a pillar that is to be recovered between fill masses emplaced in the adjacent stope voids. Extraction of the ore from the stope without dilution requires maintenance of the integrity of the complete fill mass. The fill must have sufficient strength to sustain gravity loads, and also any stresses imposed by the displacement of the adjacent stope walls.

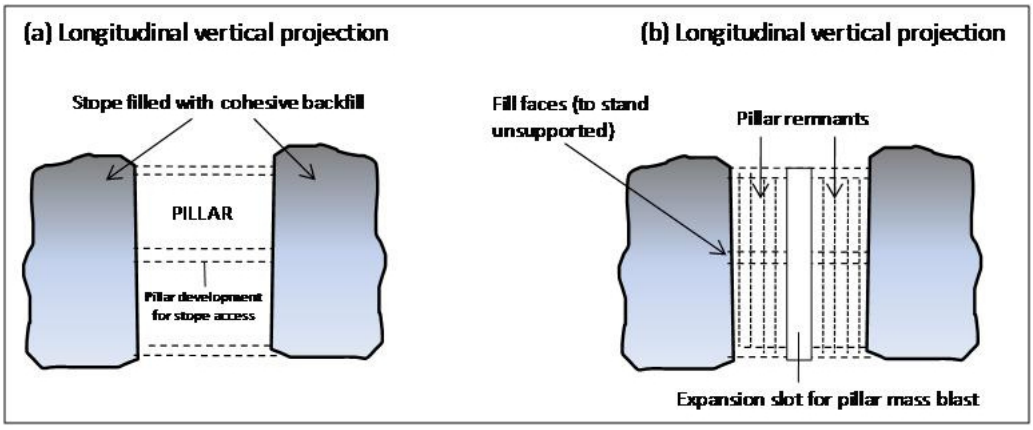


Figure 57: Typical open stoping

- Brady BHG & Brown ET (1993) Rock Mechanics for Underground Mining
- Chapman & Hall Chapter 11
- Ryder JA & Jager AJ (1999) A Handbook on Rock Engineering Practise for Tabular Hard Rock Mines
- A handbook on mine backfill that can be purchased from the site www.acg.uwa.edu.au/page/2146

It is strongly suggested that the user should get a copy of the proceedings of the Minefill 2011 conference for the latest developments in mining backfill technology from www.saimm.co.za .

CHAPTER

6

INVESTIGATION AND EVALUATION

INTRODUCTION

Investigation and evaluation of a rock mass and its behaviour can be done in many ways, from evaluating the behaviour of small rock samples in tests to how this material, or the combination of different materials, behaves when excavations are created and support is installed.
Mine layout and support design is based on a number of empirical and theoretical methods and the need to evaluate expected and actual rock and rock mass behaviour is critical to build confidence in implemented designs.

This chapter therefore deals with investigation and evaluation methods by looking into behaviours created with:

- Rock property determination
- Rock mass monitoring
- Seismic monitoring systems
- Numerical modelling

LEARNING OUTCOMES

6.1. ROCKMASS CHARACTERISATION
6.1.1 FUNDAMENTALS OF ROCK PROPERTY DETERMINATION

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

6.1.1.1 Sketch, describe and explain how uni-axial compressive strength tests are carried out

6.1.1.2 Sketch, describe and explain the test equipment required for uni-axial compressive strength tests

6.1.1.3 Describe and explain the test procedure for uni-axial compressive strength tests

6.1.1.4 Sketch, describe and explain how tri-axial compressive strength tests are carried out

6.1.1.5 Sketch, describe and explain the test equipment required for tri-axial compressive strength tests

6.1.1.6 Describe and explain the test procedure for tri-axial compressive strength tests

6.1.1.7 Describe and explain how samples may be collected for compressive strength tests

6.1.1.8 Describe and explain how samples are prepared for compressive strength tests

6.1.1.9 Describe and explain how load measurements are made during compressive strength tests

back

6.1.1.10 Describe and explain how strain measurements are made during compressive strength tests

6.1.1.11 Sketch, describe and explain how discontinuity shear strength tests are carried out

6.1.1.12 Sketch, describe and explain the test equipment required for discontinuity shear strength tests

6.1.1.13 Describe and explain the test procedure for discontinuity shear strength tests

6.1.1.14 Describe and explain how samples may be collected for shear strength determinations

6.1.1.15 Describe and explain how samples are prepared for shear strength determinations

6.1.1.16 Sketch, describe and explain how Brazilian indirect tensile strength tests are carried out

6.1.1.17 Sketch, describe and explain the test equipment required for Brazilian indirect tensile strength tests

6.1.1.18 Describe and explain the test procedure for Brazilian indirect shear strength tests

6.1.1.19 Describe and explain how samples may be collected for Brazilian indirect shear strength determinations

6.1.1.20 Describe and explain how samples are prepared for Brazilian indirect shear strength determinations

6.1.1.21 Describe the method of calculating strength for Brazilian indirect tensile strength tests

6.1.1.22 Sketch, describe and explain how point load index tests are carried out

6.1.1.23 Sketch, describe and explain the test equipment required for point load index tests

6.1.1.24 Describe and explain the test procedure for point load index tests

6.1.1.25 Describe and explain how samples may be collected for point load index determinations

6.1.1.26 Describe and explain how samples are prepared for point load index determinations

6.1.1.27 Describe the method of calculating strength for point load index tests

6.1.1.28 Evaluate the shortcomings of each of the above tests as indicators of the large-scale strength of rock

6.1.1.29 Describe and explain how the following variables may affect the indicated strength of rock samples tested in uni-axial compression:

- Specimen end-effects
- Volume of rock tested
- Test machine stiffness
- Loading rate
- Sketch and describe the complete stress-strain graph for rock in uni-axial compression, indicating:
- Hysteresis in the post-peak portion of the curve
- The associated radial strain
- The associated volumetric strain
- The brittle-ductile transition
- The residual strength

back

back

6.1.1.30 Sketch and describe the complete stress-strain graph for rock in tri-axial compression, indicating:

- Hysteresis in the post-peak portion of the curve
- The associated radial strain
- The associated volumetric strain
- The brittle-ductile transition
- The residual strength

6.2. FUNDAMENTALS OF MONITORING ROCKMASS BEHAVIOUR

6.2.1 OBJECTIVES OF MONITORING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

6.2.1.1 Describe the general objectives of a monitoring system

6.2.1.2 Describe and explain the observational method of design in rock engineering

6.2.1.3 Describe and explain what may be learned from monitoring

6.2.1.4 Describe and explain how the results learned from monitoring may be used in design

6.2.1.5 Describe the rock parameters that may be measured in a monitoring programme

6.2.1.6 Describe the rock response that may be measured in a monitoring programme.

6.2.2 PRINCIPLES OF MONITORING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

6.2.2.1 Describe and explain the general features of a monitoring system

6.2.2.2 Describe and explain measuring instruments in terms of:

- Accuracy
- Error
- Sensitivity
- Precision

6.2.2.3 Describe and explain how one would measure:

- The stress tensor in a rock mass
- The depth of fracturing in tunnel walls
- Convergence
- Tunnel wall dilation
- Stress changes in rock
- Pressure changes in backfill
- Height of caving of hangingwall.

6.2.3 MONITORING INSTRUMENTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

6.2.3.1 Describe the following instruments and explain their purpose, installation and operation:

- Hydraulic pressure cells
- Tapes and tape extensometers

back

- Doorstopper stress cell
- Tri-axial stress cell
- Closure meter
- Peizometer
- Inclinometer
- Multiple Point Borehole Extensometers
- Hydraulic Pressure Cells
- Vibrating Wire extensometers and stress meters
- Petroscopes
- Brunton Compass

6.2.3.2 Interpret results from the above instruments

6.2.3.3 Interpret rock mass response indicated by results from the above instruments

6.2.3.4 Describe and explain the principles of operation of strain gauges.

6.3. FUNDAMENTALS OF SEISMIC MONITORING SYSTEMS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

6.3.1 Identify, describe and explain the objectives of seismic monitoring in mines

6.3.2 Identify, describe and explain the features of a seismic event that are used to locate the source of the event

6.3.3 Describe and explain the differences between geophones and accelerometers

6.3.4 Identify, describe and discuss typical applications of geophones

6.3.5 Identify, describe and discuss typical applications of accelerometers

6.3.6 Describe, explain and discuss the functions and physical attributes of the following components of a seismic system:

- Seismometer, Amplifier
- Analogue to Digital Converter
- Trigger Unit, Storage Unit
- Data transmission to surface
- Data processing, Database

6.3.7 Describe and explain the factors that need to be considered in the placement of seismometers

6.3.8 Describe and explain how placement could affect event location accuracy

6.3.9 Describe and explain how placement could affect the sensitivity of a seismic system

6.3.10 Describe and explain possible reasons for differences between ideal seismic network layouts and actual installed seismic network layouts

6.3.11 Describe and explain the physical limitations of seismic network performance in terms of the accuracy of source parameters

6.3.12 Identify, describe and explain the various factors that may influence the quality of seismic data

6.3.13 Describe the factors affecting hardware performance

back

6.3.14 Describe the factors affecting the quality of seismic data processing

6.3.15 Describe the factors affecting data storage and data analysis

6.3.16 Describe the controls that may be instituted to minimise the above effects

6.3.17 Estimate approximate installation and operating costs associated with various size seismic networks.

6.4. FUNDAMENTALS OF NUMERICAL MODELLING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

6.4.1 Describe and explain the principles of the following numerical modelling techniques:

- Finite Element Methods
- Finite Difference Methods
- Boundary Element Methods
- Distinct Element Methods
- Key block Methods

6.4.2 Describe and explain for which rock engineering applications the above numerical modelling techniques are suitable

6.4.3 Describe and explain what the limitations of the above numerical modelling techniques are in terms of the following aspects:

- When that method should be used
- Why that particular method should be used
- How the results should be interpreted
- How the results should be correlated with actual conditions.

back

LEARNING OUTCOME 6.1.1.1

SKETCH, DESCRIBE AND EXPLAIN HOW UNI-AXIAL COMPRESSIVE STRENGTH TESTS ARE CARRIED OUT.

Uni means one and axial refers to the axis along which forces are applied. Uni-axial test therefore means that rock samples are compressed along one axis or in one direction only. In uni-axial tests, the rock is loaded from the top and bottom of the sample only. The rock is allowed to change in size and bulge until it fails. When the rock is completely fractured, the maximum force applied to the sample is read off the testing machine and the stress at that point is calculated using the applied force and sample contact area. This stress level is called the uni-axial compressive strength (UCS).

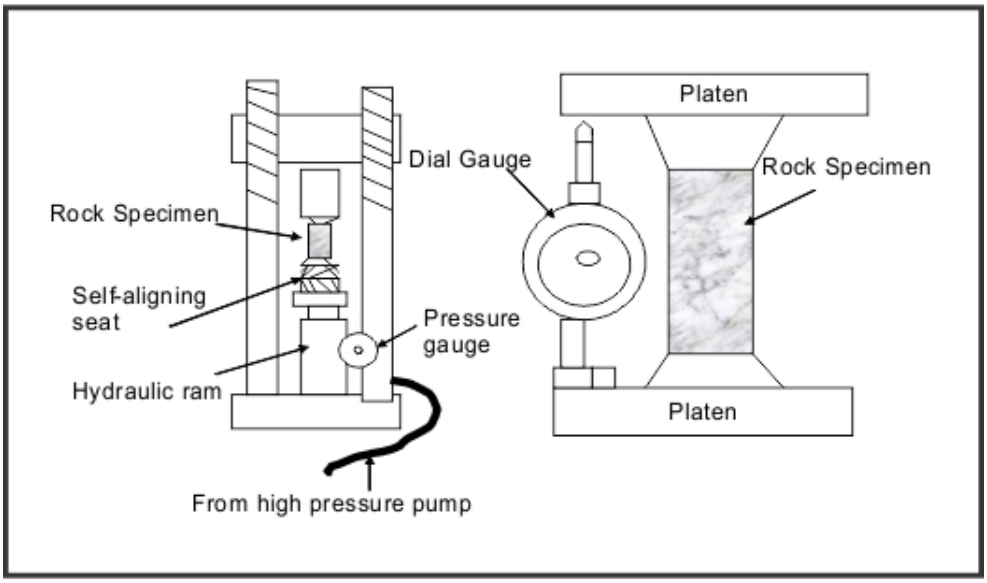


Figure 1: Sketch of a typical uni-axial compressive test machine

This machine typically consists of a framework supporting an upper cylindrical platen, connected to a high pressure pump, which applies the force through the platens onto the sample. Dial gauges and/or resistance strain gauges can be glued to the sample to monitor deformation during loading, as shown in Figure 2 below.

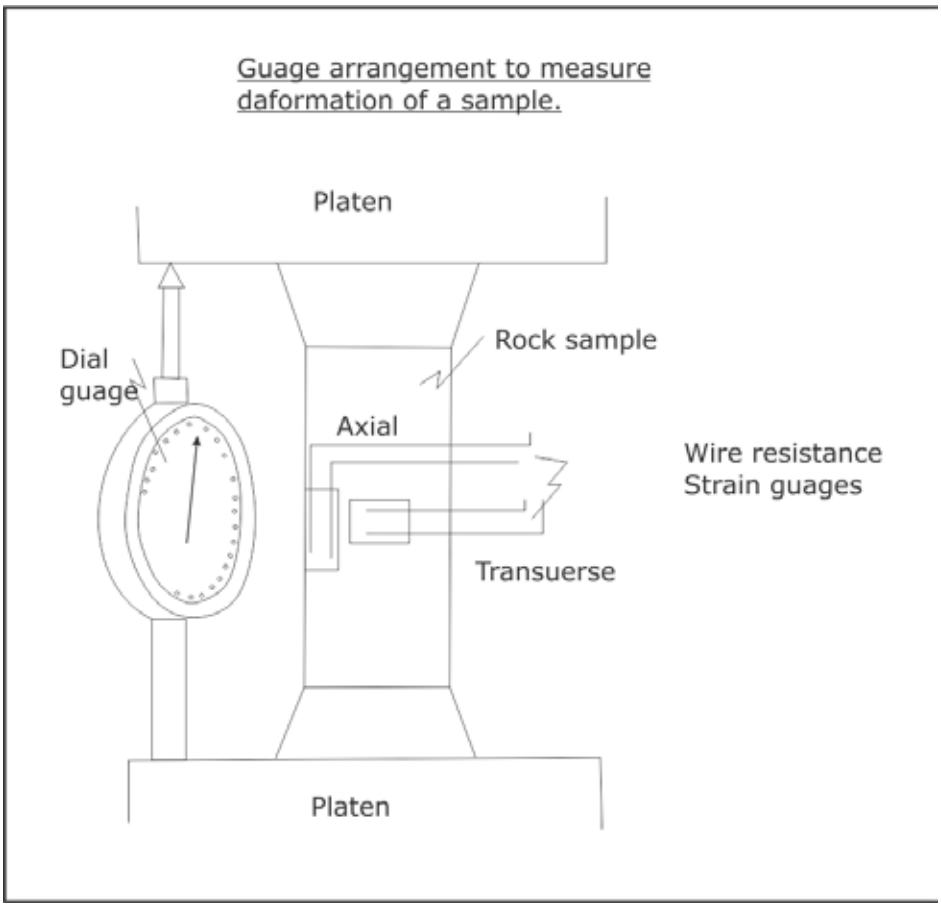


Figure 2: Dial gauge and wire resistance strain gauges glued to sample

Basic description of UCS tests:

- **Equipment**

  Uni-axial press and strain cells.

- **Sample preparation**

  Normally, ten air-dried samples are tested to have a representative test. The length of the samples is twice the diameter (length / diameter = 2). The ends of the samples are cut parallel and ground smooth.

- **Test apparatus preparation**

  Test apparatus must be calibrated, clean and checked for leaks of hydraulic fluid prior to usage. Persons using the machine must do it from a safe position and a safety guard must be used to prevent broken sample pieces from injuring any personnel during the test.

- **Data recording**

  The machine is also connected to a computer that records the test and draws a graph from which the rock strength can be read. During such a test, axial and radial (transverse) strain experienced during loading is plotted against applied stress.

ANIMATION

Click on the URL below **(You must have internet access to view the video.**
http://www.youtube.com/watch?v=qLQcSTSMHO0&feature=player_detailpage

NOTE

The following videos were provided for your interest by the Rock Testing Laboratory at the University of the Witwatersrand, Courtesy of Dr Halil Yilmaz

To watch this video, you need Quick Time. If you do not have Quick Time installed on your computer, it can be downloaded for free from http://www.apple.com/quicktime/

ANIMATION

CONNECTION 45

Laboratory testing provides the starting point for inferring the physical behaviour of rocks and rock masses, and for estimating the numerical parameters involved. Uni-axial compressive strength test results have been discussed in full in Paper 1 – Learning Outcome 4.1.1. on page 96

REFERENCE

- ISRM procedure for Uni-axial testing

LEARNING OUTCOME 6.1.1.2

SKETCH, DESCRIBE AND EXPLAIN THE TEST EQUIPMENT REQUIRED FOR UNI-AXIAL COMPRESSIVE STRENGTH TESTS

Equipment required to perform UCS testing include the following (Figure 3):

- Saw to cut the test samples from core samples delivered;
- Grinders to grind the core contact surfaces;
- Measuring equipment to measure the angle between the flat contact surfaces and the core sample axis (must be 90°);
- Vernier scale to measure the core sample diameter;
- Flasks to determine the core sample density (weigh sample and determine its volume to allow calculation of density as kg/m3);
- Test machine to perform the uni-axial test;
- Strain gauges if information on strain and Moduli is required; and
- Computers and software to manage the test machine, record and present results.

Figure 3: (a) Saw, (b) computers and software, (c) stain gauges, (d) testing machine

REFERENCE

- ISRM procedure for Uni-axial testing

LEARNING OUTCOME 6.1.1.3

DESCRIBE AND EXPLAIN THE TEST PROCEDURE FOR UNI-AXIAL COMPRESSIVE STRENGTH TESTS.

The ISRM has developed a standard procedure to be followed when conducting UCS tests. Critical issues have been highlighted below, but it is advised that the full procedure is read.

Procedure:

- A suitable machine shall be used to apply and measure axial load to the specimen. It shall be verified at suitable time intervals and shall comply with accepted national requirements such as prescribed in either ASTM Methods E4: Verification of Testing Machines or British Standard 1610, Grade A or Deutsche Normen DIN 51 220, DIN 51 223, Klasse 1 and DIN 51 300.
- Steel platens in the form of discs and having a Rockwell hardness of not less than HRC58 shall be placed at the specimen ends. Surfaces of the discs should be ground and their flatness should be better than 0.005mm.
- Test specimens shall be right circular cylinders having a height to diameter ratio of 2.5-3.0 and a diameter preferably of not less than NX core size, approximately 54mm. The diameter of the specimen should be related to the size of the largest grain in the rock by the ratio of at least 10:1.

back

- The ends of the specimen shall be fiat to 0.02mm and shall not depart from perpendicularity to the axis of the specimen by more than 0.001 radian (about 3.5 min) or 0.05mm in 50mm.
- The diameter of the test specimen shall be measured to the nearest 0.1mm by averaging two diameters measured at right angles to each other at about the upper-height, the mid-height and the lower height of the specimen. The average diameter shall be used to calculate the cross-sectional area.
- The height of the specimen shall be determined to the nearest 1.0mm.
- Load on the specimen shall be applied continuously at a constant stress rate such that failure will occur within 5-10min of loading, alternatively the stress rate shall be within the limits of 0.5-1.0MPa/s.
- The maximum load on the specimen shall be recorded in newtons (or kilonewtons and meganewtons where appropriate) to within l°~o.
- The uni-axial compressive strength of the specimen shall be calculated by dividing the maximum load carried by the specimen during the test, by the original cross-sectional area.

REFERENCE

- ISRM procedure for Uni-axial testing

LEARNING OUTCOME 6.1.1.4

SKETCH, DESCRIBE AND EXPLAIN HOW TRI-AXIAL COMPRESSIVE STRENGTH TESTS ARE CARRIED OUT

Tri means 'three'. A tri-axial test means that forces are applied to all three sides (or from three mutually perpendicular directions) of a sample that is being tested. In this type of test, a rock sample is prepared and placed in the machine and compressed axially (vertically) and transversely (horizontally). The forces applied to the specimen from the top and the sides are recorded and the stress (force / contact area) is called the tri-axial compressive strength.

CONNECTION 46

See also Outcome 4.1.15 in Paper 1 on page 123

In essence, the horizontally applied forces confine the rock sample that is being subjected to the axial force. In order to determine the effect of this confinement on the strength of the rock, tri-axial compressive tests are carried out under various confining pressures. Generally, as the confinement increases, so does the strength and behaviour of the sample.

LEARNING OUTCOME 6.1.1.5

SKETCH, DESCRIBE AND EXPLAIN THE TEST EQUIPMENT REQUIRED FOR TRI-AXIAL COMPRESSIVE STRENGTH TESTS

Equipment required to perform TCS testing include the following (Figure 4):

- Saw to cut the test samples from core samples delivered;
- Grinders to grind the core contact surfaces;
- Measuring equipment to measure the angle between the flat contact surfaces and the core sample axis (must be 90°);

back

back

- Vernier scale to measure the core sample diameter;
- Flasks to determine the core sample density (weigh sample and determine its volume to allow calculation of density as kg/m3);
- Test machine to perform the tri-axial test;
- Strain cells to record deformation during compression (record impact of confinement on sample behaviour);
- Computers and software to manage the test machine, record and present results.

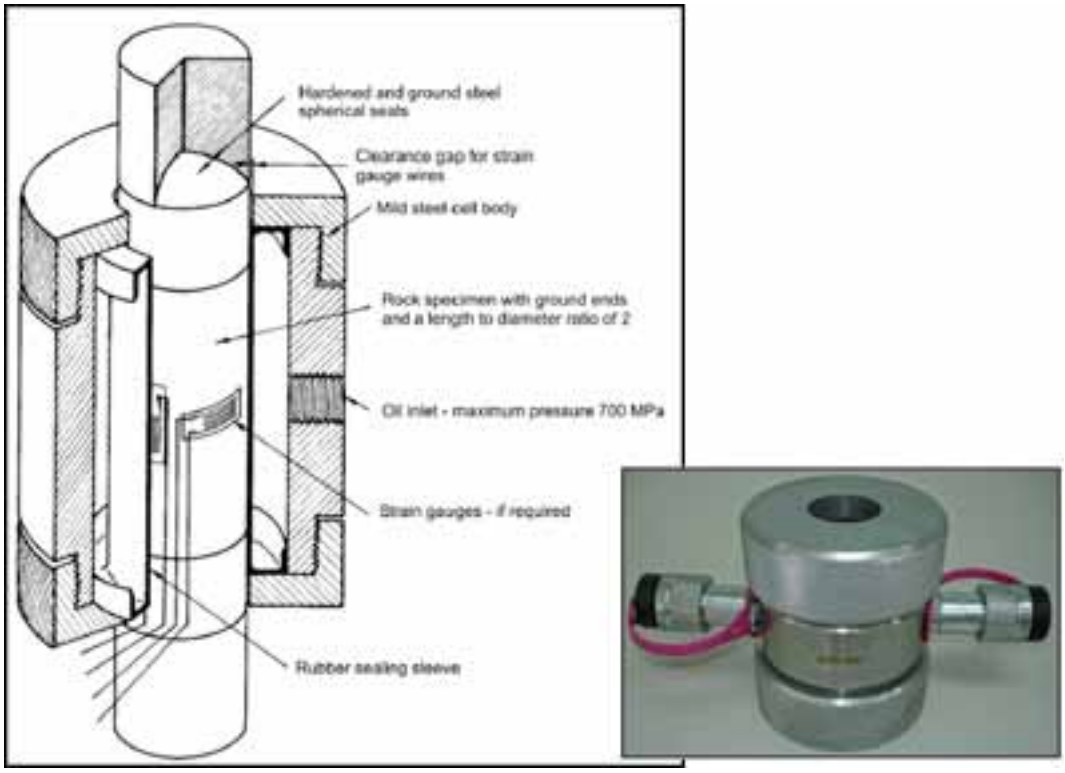


Figure 4: A tri-axial cell for testing rock specimens (Hoek's Notes)

Figure 5: (a) Testing machine and (b) strain gauges

back

LEARNING OUTCOME 6.1.1.6

DESCRIBE AND EXPLAIN THE TEST PROCEDURE FOR TRI-AXIAL COMPRESSIVE STRENGTH TESTS

The ISRM has developed a standard procedure to be followed when conducting UCS tests. Critical issues have been highlighted below, but it is advised that the full procedure is read.

Procedure:

- A stiff loading machine should preferably be used to apply and measure the axial load in the rock specimen. It should be of sufficient capacity to fail the specimen at the selected confining pressure of pressure range and capable of applying the strains at a selected rate. It shall be calibrated at suitable time intervals.
- A flexible membrane of suitable material shall be used to prevent the confining fluid from entering the specimen; the membrane shall not penetrate significantly into the surface pores and it should be sufficiently long to extend well on to the platens. When slightly stretched it should be the same diameter as the specimen.
- A hydraulic pump or some other system of sufficient capacity and capable of fine regulation of the pressure to within + 1% shall be used.
- Assemble specimen, platens, membrane, tri-axial cell and load pressure and displacement measurement devices. The exact procedure will depend on tri-axial cell design and on type and location of the measurement devices.
- Connect the hydraulic line and fill the tri-axial cell with oil, allowing the air to escape through an air bleeder hole.
- Place the tri-axial cell in the axial loading device.
- Select the initial confining pressure.
- Individual test
  - The axial load and the confining pressure should normally be increased simultaneously until the predetermined level for the confining pressure is reached;
  - The axial load on the specimen shall correspond to a constant strain rate such that failure will occur within 5-15 rain of loading. Alternatively, the stress rate shall be within the limits of 0.5-1.0MPa/sec. The maximum axial load and the corresponding confining pressure on the specimen shall be recorded.
  - However, it is advisable to record axial load and displacement continuously.
- Multiple failure state test
  - Apply the initial confining pressure P0. The axial load and the confining pressure should normally be increased simultaneously until the hydrostatic pressure reaches the value of the initial confining pressure.
  - The axial load is then increased keeping confining pressure P0 constant until the corresponding peak strength is observed in the axial stress-axial strain curve. The axial load shall be increased continuously at a constant strain rate within the limits set.
  - The confining pressure is then increased manually in one step.
  - This is followed by an axial load increase using the same procedure described above.

- The stepwise procedure described above is continued.
- The confining pressure will then be kept constant while the axial loading is continued. This will cause failure, and the axial stress will drop to its residual value.
- The confining pressure is continuously reduced until the specimen is completely unloaded.

- Continuous failure state test
  - Apply the initial confining pressure P0. The axial load and the confining pressure should normally be increased simultaneously until the hydrostatic pressure reaches the value of the initial confining pressure.
  - The actual load is then increased keeping confining pressure P0 constant until the corresponding peak strength is observed.
  - The axial load on the specimen shall be increased continuously at a constant strain rate within the limits set.
  - While the axial load increases at the selected rate above, the confining pressure is simultaneously increased.
  - The variable axial load and confining pressure are continuously recorded and plotted.
  - At a chosen point, the confining pressure will be kept constant while the axial loading is continued.
  - When further compressing the specimen, the axial stress will drop to a residual value.
  - The confining pressure is continuously reduced until the specimen is completely unloaded.

CONNECTION 47

A stiff loading machine is discussed in "LEARNING OUTCOME 6.1.1.29" on page 422

REFERENCE

- ISRM procedure for tri-axial testing

LEARNING OUTCOME 6.1.1.7

DESCRIBE AND EXPLAIN HOW SAMPLES MAY BE COLLECTED FOR COMPRESSIVE STRENGTH TESTS

Geotechnical samples are tested for a variety of parameters, the most important being the strength of the rock (compressive or tensile) or the shear strength of rock or natural discontinuities.

Number of samples
Before any geotechnical sampling commences, the information, data and results required from the testing need to be delineated. For example: The correct number of samples must be collected to provide a representative strength value for the rock type tested.

Sample position
The geotechnical engineer should specify the position of the samples to be tested to ensure that appropriate samples are selected. For instance, if the strength of the sandstone roof material in a Coal Mine is required, sample selection of sandstone layers remote from the Coal Seam mined, is inappropriate and should not be selected. However, if the data is required for the sinking of a shaft, the selection of samples all along the planned shaft position is required to allow an indication of strong and weak layers that will be intersected. In this case, the selection of sandstone samples remote from the Coal Seam is appropriate.

Shape and size of the samples
Rock samples can be selected as bulk samples (solid piece of rock of any size and shape) or drilled core samples. In the former case, the

laboratory will drill core from the rock sample to prepare samples for testing.

When bulk samples are selected, it is critical that the original orientation of the rock sample is indicated so that the core can be drilled in the correct direction to appropriately determine the rock strength. If this is not done, the core may be drilled parallel to natural laminations in the rock material and so report much weaker strengths when placed in the test machine and tested parallel to the laminations, instead of perpendicular to them. In the case of using existing core samples, the orientation of the boreholes from which the samples originate is important for the same reasons.

For most rock strength tests, it is best practice to select a complete core sample that is at least 30cm in length to allow the cutting of an acceptable length sample. However, a minimum length of more than 2 x core diameter is required, together with a 10:1 diameter to largest grain size in the rock material.

Sample records

Samples should, when taken be recorded as follow:

1. Sample marking:
   - When selecting a sample from a core tray, it is good practice to insert a marker in the tray core where the sample was removed. This allows any future users of the core trays to be aware of core that has been removed from the tray;
   - The selected sample is immediately bagged (individually, i.e. each sample in its own bag) and at least the following information written on the bag:
     - A sample number should be assigned to each sample taken and could be anything that allows good control of samples. Good practice is to use the borehole code (e.g. AGT01), followed by GS (Geotechnical Sample) and then a number. For example, this will then read: AGT01 GS 01.
     - Borehole number from which sample has been taken (if a different sample numbering system is used);
     - Position of the sample (depth) in the core;
     - Rock type sampled;
2. Sample records:
   - Records should be maintained for all samples taken as they can be sent with the sample to the laboratory to ensure good sample control
     - Sample number;
     - The correct depth of the sample is recorded (if from core) and should include the 'depth from' and the 'depth to';
     - Rock type;
     - Sample length and diameter.

Table 1 and Table 2 show examples of sample sheets or logs used for UCS and Tri-axial tests.

UCM tests							
No	BH ID	Sam-ple ID	Rock type	From	To	Core length	Core diameter
1	NDC4236	ND09	Dolerite	17.45	17.70	0.25	4.50
2	NDC4236	ND10	Dolerite	27.00	27.20	0.20	4.50
3	NDC4236	ND11	Siltstone	131.10	131.30	0.20	4.50
4	NDC4236	ND12	Siltstone	151.16	151.35	0.19	4.50
5	NDC4236	ND13	Shale	171.46	171.70	0.24	4.50
6	NDC4236	ND14	Shale	170.87	171.04	0.17	4.50
7	NDC4236	ND15	Sandstone	190.13	190.42	0.29	4.50
8	NDC4236	ND16	Sandstone	189.76	189.98	0.22	4.50
9	Nd4242	ND27	Coal(S4)	243.36	243.56	0.20	6.00
10	NDC4241	ND39	Coal(S4)	226.50	226.67	0.17	6.00

Table 1: UCS sample sheet

TRI-AXIAL								
No core pieces taken	BH ID	Sample ID	Rock type		From	To	Core length	Core diameter (Cm)
1	Nd4241	ND31	Coal(S4)	A	225.10	225.30	0.20	6.00
2				B	225.30	225.50	0.20	6.00
3				C	225.50	225.70	0.20	6.00
4	Nd4243	ND34	Siltstone	A	183.65	183.81	0.16	4.50
5				B	183.91	184.13	0.22	4.50
6				C	184.13	184.31	0.18	4.50
7	Nd4246	ND37	Dolerite	A	10.35	10.53	0.18	4.50
				B	10.53	10.72	0.19	4.50
				C	10.72	10.92	0.20	4.50
10	NDC4240	ND61	Shale	A	92.85	93.06	0.21	4.50
				B	93.06	93.20	0.14	4.50
				C	93.32	93.47	0.15	4.50

Table 2: Tri-axial sample sheet

Sample handling

Mishandled samples can have major implications on the strength results and the integrity of the decisions based on the tests. Sample break during transportation, contact with water or even excessive drying out of samples must be prevented to ensure that the tests reflect appropriate values.

LEARNING OUTCOME 6.1.1.8

DESCRIBE AND EXPLAIN HOW SAMPLES ARE PREPARED FOR COMPRESSIVE STRENGTH TESTS

CONNECTION 48

Refer to "LEARNING OUTCOME 6.1.1.3" on page 394 and "LEARNING OUTCOME 6.1.1.6" on page 397 as well as the ISRM procedures linked in those sections.

Rock specimens are typically selected from standard core samples and:

- are made relatively slender, i.e. with a width:height ratio of 0.5 or 0.33, in order to minimise the effect of friction on the specimen's ends (excessive friction would tend to clamp the specimen and strengthen it) – Samples are cut to the correct lengths.

- the ends of the specimen need to be prepared to a high degree of parallelism (within 0.0005mm per mm of length of the specimen), otherwise bedding-in and bending effects can become excessive and could distort the measured UCS and other values – Samples are grinded and polished.

REFERENCE

- Rock Engineering Textbook - Jager and Ryder Chapter 2.
- ISRM standard procedures (UCS tests)

LEARNING OUTCOME 6.1.1.9

DESCRIBE AND EXPLAIN HOW LOAD MEASUREMENTS ARE MADE DURING COMPRESSIVE STRENGTH TESTS

Load measurements are recorded on gauges such as those shown Figure 6 below. The load measurements are recorded during the test on a computer connected to the test machine and the load measurements are then used to draw the load-deformation or stress-strain graphs that represent the sample behaviour.

Figure 6: Gauges to record loads on samples

LEARNING OUTCOME 6.1.1.10

DESCRIBE AND EXPLAIN HOW STRAIN MEASUREMENTS ARE MADE DURING COMPRESSIVE STRENGTH TESTS

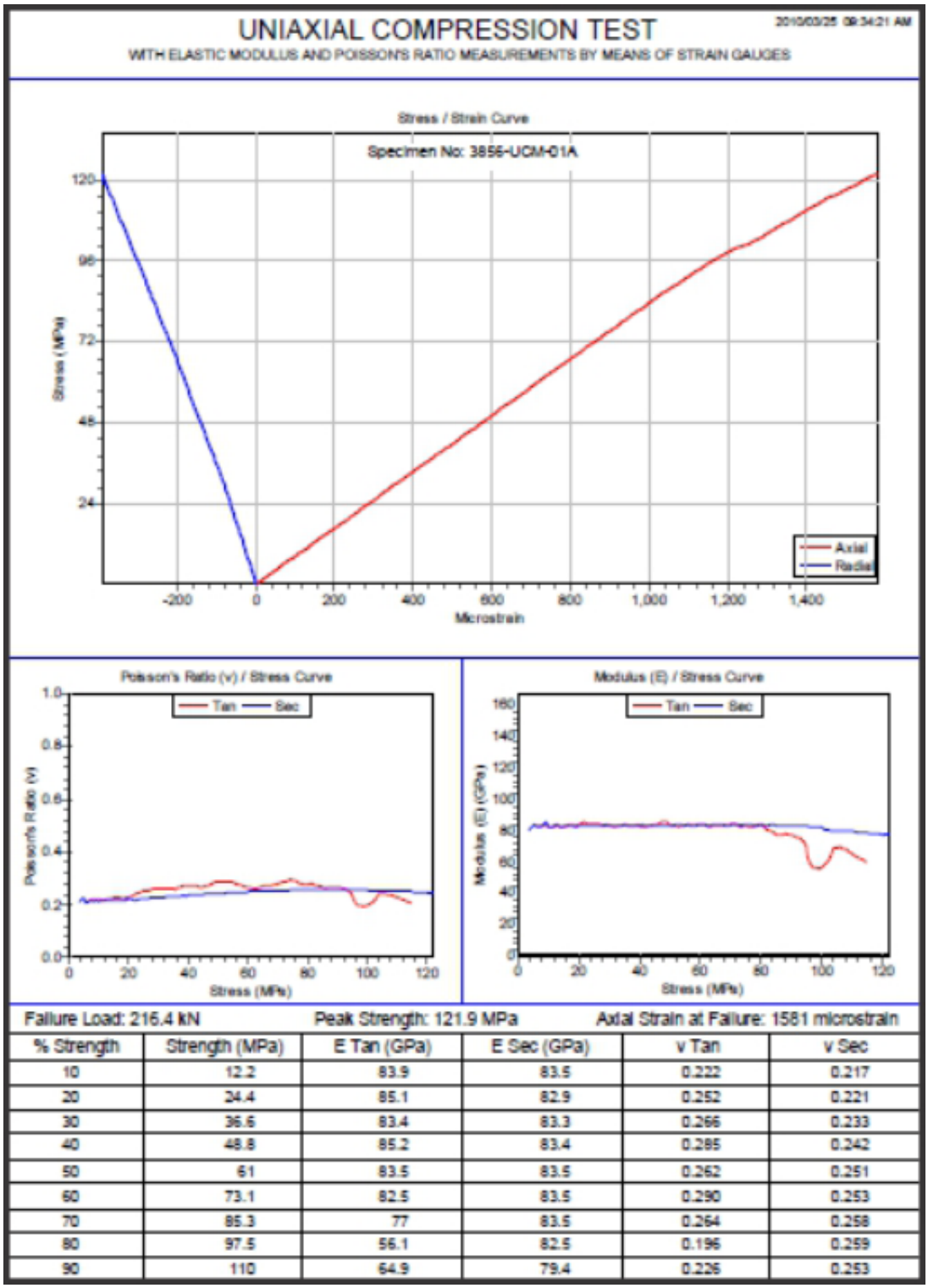
CONNECTION 49

See also Paper 1 outcomes 3.1 on page 61 and 4.1.3 on page 99

The strain gauges measures the axial and diametric displacements that the sample undergoes during the test. The original dimensions (length and diameter) of the test sample are known and allow, by applying the basic strain formulae, the calculation of the axial and diametric strains for complete stress-strain graphs of the rock sample behaviour. The same information can be used (together with the stresses calculated from the loads in outcome 6.1.1.9) to calculate the elastic properties of the sample, namely Poisson's Ratio and Young's Modulus.

In Figure 7 below, stress-strain graphs resulting from a uni-axial compressive strength test are shown.

back



% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	v Tan	v Sec
10	12.2	83.9	83.5	0.222	0.217
20	24.4	85.1	82.9	0.252	0.221
30	36.6	83.4	83.3	0.266	0.233
40	48.8	85.2	83.4	0.285	0.242
50	61	83.5	83.5	0.262	0.251
60	73.1	82.5	83.5	0.290	0.253
70	85.3	77	83.5	0.264	0.258
80	97.5	56.1	82.5	0.196	0.259
90	110	64.9	79.4	0.226	0.253

Figure 7: Example of stress/strain graphs drawn from a uni-axial compressive strength test

LEARNING OUTCOME 6.1.1.11

SKETCH, DESCRIBE AND EXPLAIN HOW DISCONTINUITY SHEAR STRENGTH TESTS ARE CARRIED OUT

These tests are usually carried out for cases where discontinuity surfaces may play an important role of the stability of a mining area, such as in open pit mines (slope stability) or massive mines where the stability of large open stopes is a concern.

back

back

CONNECTION 50

See outcomes 4.1.8 on page 108 and 4.2 on page 127 in Paper 1

The shear strength of a discontinuity surface can be determined by the Mohr-Coulomb strength criterion represented by the equation below:

$$|\tau| = So + \mu\sigma n.$$

The constant 'μ' is called the coefficient of internal friction of the material, which can be calculated by the Tan(Friction angle) and So is the cohesion of the material.

Cohesion and friction angles along discontinuity surfaces can be determined from rock testing by means of direct shear tests.

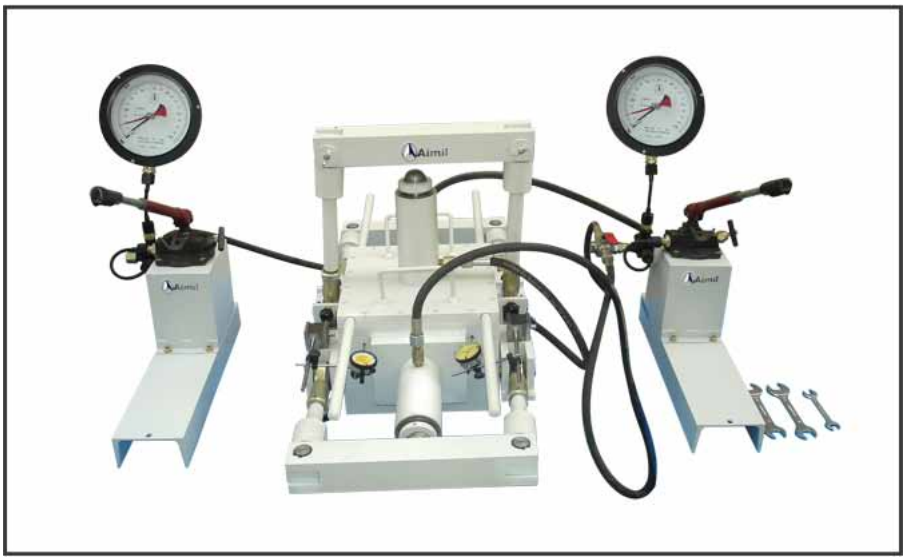


Figure 8: Direct shear testing equipment

During such shear testing of discontinuity surfaces, horizontal load-to-horizontal displacement (Figure 9) as well as shear stress-to-normal stress graphs can be drafted (Figure 10).

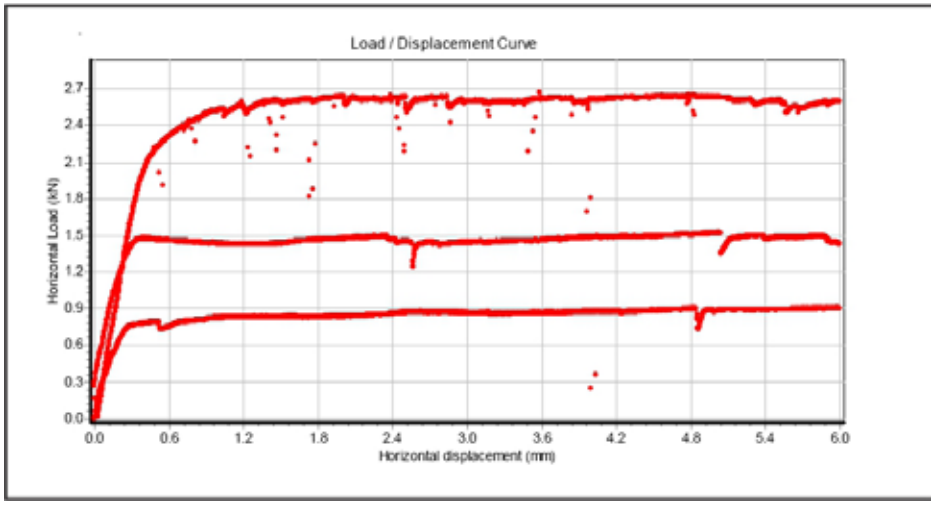


Figure 9: A horizontal load-horizontal displacement graph

back

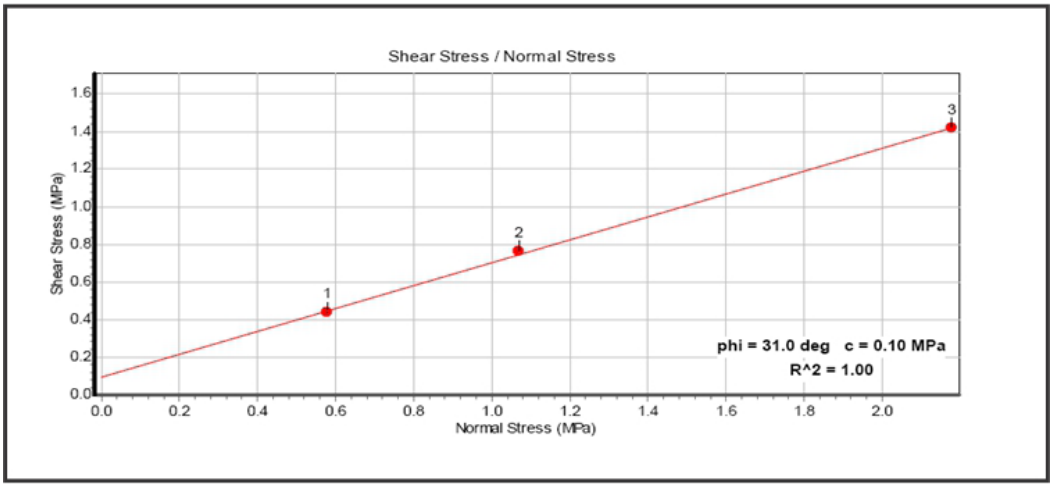


Figure 10: A shear stress-normal stress graph

REFERENCE

- ISRM Shear test procedure.

LEARNING OUTCOME 6.1.1.12

CONNECTION 51

"LEARNING OUTCOME 6.1.1.11" on page 402

SKETCH, DESCRIBE AND EXPLAIN THE TEST EQUIPMENT REQUIRED FOR DISCONTINUITY SHEAR STRENGTH TESTS

LEARNING OUTCOME 6.1.1.13

DESCRIBE AND EXPLAIN THE TEST PROCEDURE FOR DISCONTINUITY SHEAR STRENGTH TESTS

From the shear test equipment shown in Outcome 6.1.1.11, it is clear that the normal force to the discontinuity is applied by a hydraulic jack clamping the discontinuity surface, while another hydraulic jack also provides the shear force along the discontinuity.

The shear force is gradually increased until slippage occurs. The shear strength of the discontinuity surface is then determined from the shear force applied immediately before slippage occurs and the contact area of the discontinuity sample.

From the ISRM procedures, the following important issues are shown:

- equipment to cut the sample must be prepared, including large diameter core drill, saws, tools to measure dip, dip direction, roughness etc;
- equipment to hold and mount the sample must be prepared, including wire or metal bands, cement, plaster or resin;
- equipment for the test must be prepared (shear box), including means of applying a normal load and shear force on and along the discontinuity, equipment to measure the applied forces and displacements;
- the test horizon is selected and the dip and dip directions of the discontinuity plane are recorded for a number of samples;
- remove any protective packaging (e.g. protect against water), excluding any wires used to stabilise the sample without

mechanically damaging the sample and placed in the sample carriers so that the horizon to be tested is secured and in the correct position and orientation;

- the encapsulation material is poured around the sample, 1st the one half and then the other;
- check all gauges and record preliminary load and displacement readings;
- raise the normal load to the specified value (for the particular test);
- apply the shear force in increments / continuously;
- approximately 10 readings are taken before reaching the peak shear strength of the discontinuity and afterwards the peak strength reading should be taken at increments of between 0.5 to 5mm shear displacement;
- after the test, the shear plane must be exposed and described, the area measured (to calculate stresses) and photographs taken for reference.

REFERENCE

- refer to the complete ISRM procedure.
- Shear strength tests ISRM

INTERESTING INFO

A laboratory tilt test can also be used to determine friction angles of surfaces (Figure 11). In this test, the angle of the surface is increased gradually until sliding starts to occur. The resulting angle is then the base friction angle used in the equation below to determine the shear strength of the discontinuity surface (cohesion is assumed to be zero).

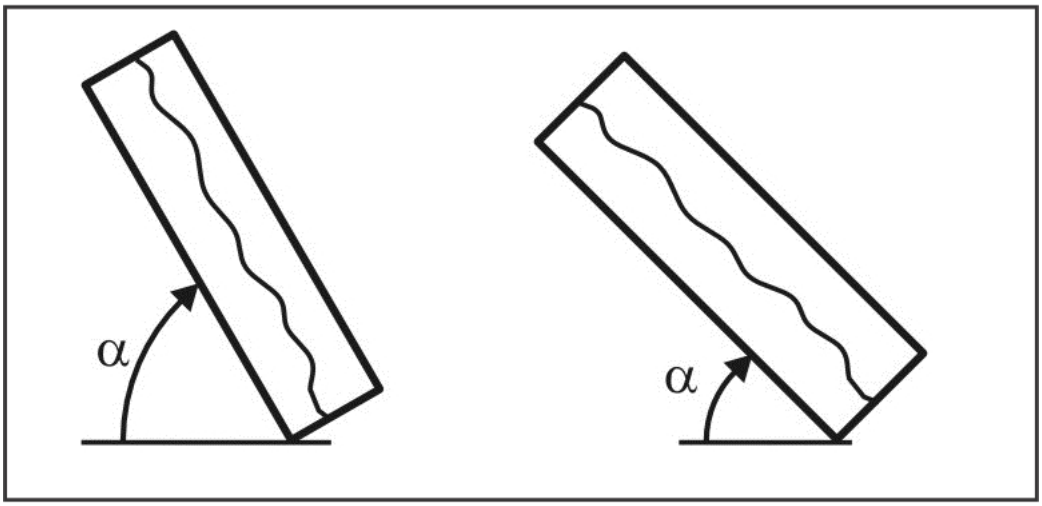


Figure 11: The 'tilt test' for estimating frictional properties (after Barton et al. 1985)

REFERENCE

- Practical Rock Engineering Hoek and Brown.
- Barton et al 1985
- ISRM test procedures

LEARNING OUTCOME 6.1.1.14

CONNECTION 52

"LEARNING OUTCOME 6.1.1.7" on page 398 for general sample selection protocols that can be followed in a similar manner.

DESCRIBE AND EXPLAIN HOW SAMPLES MAY BE COLLECTED FOR SHEAR STRENGTH DETERMINATIONS.

For shear strength tests, block or core samples can be selected, as long as the target discontinuity is present in the sample and the test plane must preferably have a minimum area of 2500mm2.

back

NOTE

This minimum area of 2500mm2 equates to a sample dimension of:

- *circular: 28mm radius and 56mm diameter;*
- *square: 50mm.*

LEARNING OUTCOME 6.1.1.15

CONNECTION 53

Refer to "LEARNING OUTCOME 6.1.1.7" on page 398, "LEARNINGOUTCOME 6.1.1.14" on page 405 and the ISRM shear test procedure.

DESCRIBE AND EXPLAIN HOW SAMPLES ARE PREPARED FOR SHEAR STRENGTH DETERMINATIONS

Test specimens are generally prepared by sawing rock cores into 7.5 to 10cm lengths or by sawing square rock samples to fit into the two shear boxes. The shear surface must be aligned to fall between the two shear boxes in order to determine the shear strength of the discontinuity surface. Care is taken not to damage the discontinuity surface during transportation and preparation.

LEARNING OUTCOME 6.1.1.16

SKETCH, DESCRIBE AND EXPLAIN HOW BRAZILIAN INDIRECT TENSILE STRENGTH TESTS ARE CARRIED OUT

This test is intended to measure the uni-axial tensile strength of prepared rock specimens indirectly by the Brazil test. The justification for the test is based on the experimental fact that most rocks in biaxial stress fields fail in tension at their uni-axial tensile strength when one principal stress is tensile and the other finite principal stress is compressive with a magnitude not exceeding three times that of the tensile principal stress.

Figure 12 shows the setup for the Brazilian disk test used to determine the tensile strength of the specimen in an indirect test, where compressive stresses are used to induce tensile failure of the sample. The thin disc is placed in a test machine, which applies a load (Figure 13) in a single direction allowing sample expansion in the other direction and therefore tensile failure. Figure 13 also shows the existence of the induced tensile stress in the sample when subjected to this test, as estimated with numerical models.

Various samples are prepared to have a representative result and tested after being installed in the test machine by applying a load using a pressure pump. The load indicator (gauge) will show an increase in load up to the point where the test specimen fails, after which the load will drop.

back

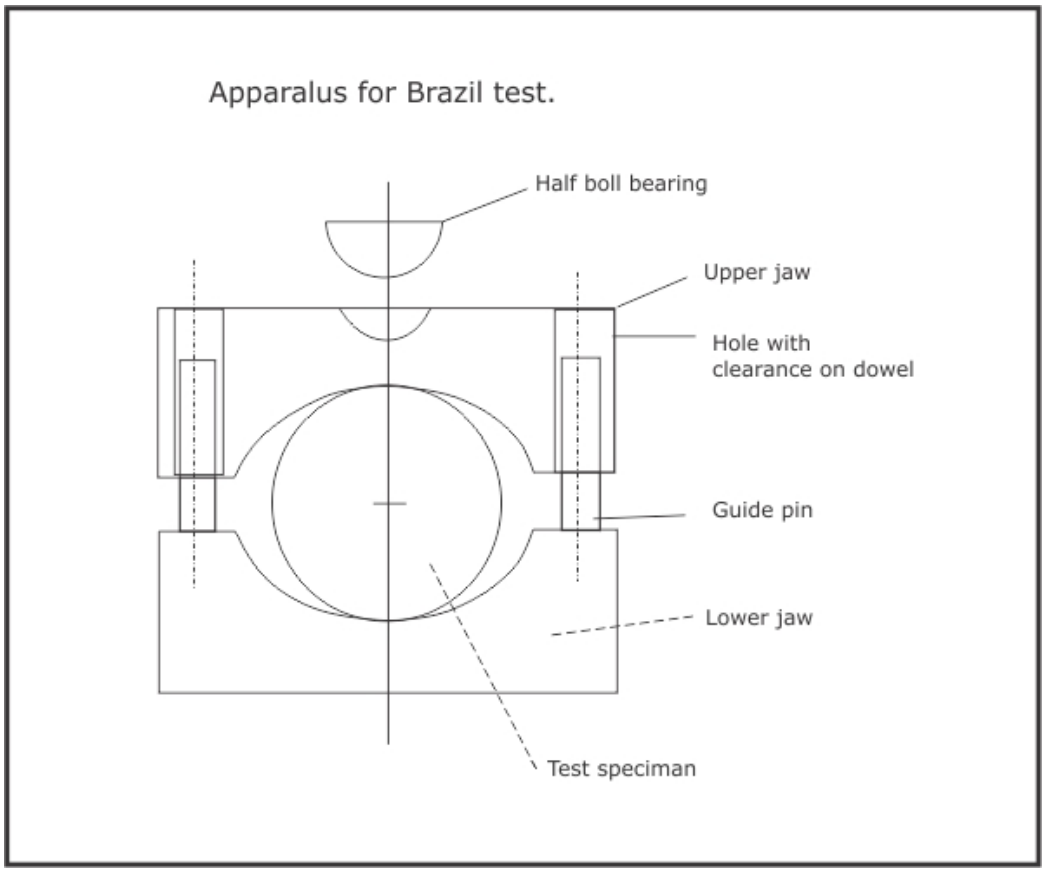


Figure 12: Brazilian disk test

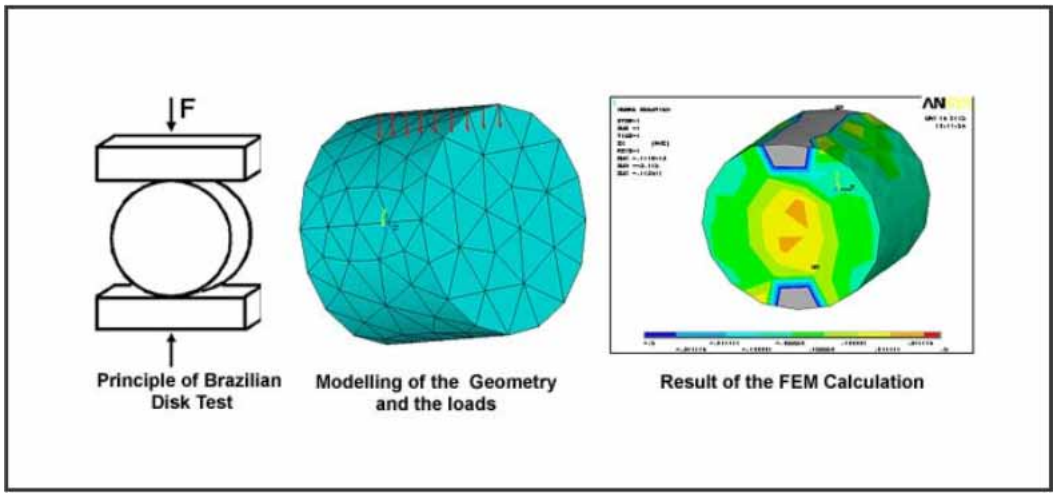


Figure 13: (a) test principle, (b) and (c) numerical modelling of the sample during testing

INTERESTING INFO

Figure 14 shows the typical failure mode for a rock sample in a Brazilian disc test, while the potential to track deformation in the samples using strain gauges, in these tests, is also shown.

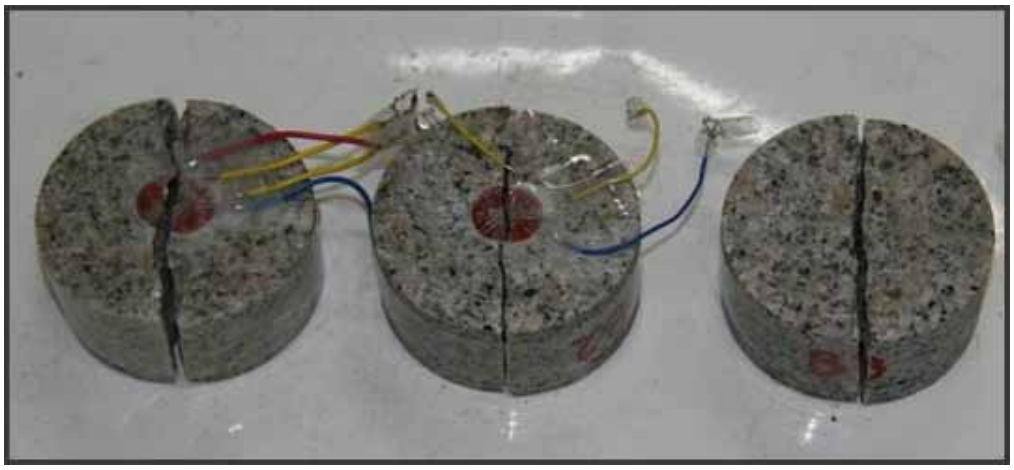


Figure 14: A typical Brazilian disc specimen failure

REFERENCE

- RTH 203-80 Method of tests for direct shear strength of rock core specimens.)
- RocG 143 Learning Guide
- A modified "Brazilian" disk test – an indirect method to determine the tensile strength of ceramics.-(Christene Rash & Dr. Wolfgang Kollenberg.)

LEARNING OUTCOME 6.1.1.17

SKETCH, DESCRIBE AND EXPLAIN THE TEST EQUIPMENT REQUIRED FOR BRAZILIAN INDIRECT TENSILE STRENGTH TESTS

Figure 15 shows a Brazilian disc test machine consisting of a loading frame, a base and a cross head joined with solid steel pillars and nuts. The loading frame normally has a capacity of 100KN. Most Brazilian disc testing machines are designed to test NX core samples. A hydraulic pump provides the load and the pressure gauge has a red needle that remains at the peak load during the test.

Figure 15: A Brazilian disc testing machine (Heimco rock testing machine) A computer may be connected to the testing machine that will graphically plot the results and do the calculations.

REFERENCE

- Heimco rock testing machines

LEARNING OUTCOME 6.1.1.18

DESCRIBE AND EXPLAIN THE TEST PROCEDURE FOR BRAZILIAN INDIRECT TENSILE STRENGTH TESTS.

From the ISRM procedures, the following important issues are shown:

- Two steel loading jaws designed so as to contact a disc-shaped rock sample at diametrically-opposed surfaces over an arc of contact of approximately 10 ° at failure.
- The critical dimensions of the apparatus are the radius of curvature of the jaws, the clearance and length of the guide pins coupling the two curved jaws and the width of the jaws.
- The upper jaw contains a spherical seating conveniently formed by a 25-mm diameter half-ball bearing.
- A suitable machine for applying and measuring compressive loads to the specimen. It shall be of sufficient capacity and be capable of applying load at a rate conforming to the requirements.

- It is preferable but not obligatory that the testing machine be tted with a chart recorder to record load against displacement to aid in the measurement of the failure load.
- The test specimens should be cut and prepared using clean water. The cylindrical surfaces should be free from obvious tool marks and any irregularities across the thickness of the specimen should not exceed 0.025mm. End faces shall be fiat to within 0.25mm and square and parallel to within 0.25°.
- Specimen orientation shall be known.
- The specimen diameter shall not be less than NX core size, approximately 54mm, and the thickness should be approximately equal to the specimen radius.
- The test specimen shall be wrapped around its periphery with one layer of the masking tape and mounted squarely in the test apparatus such that the curved platens load the specimen diametrically.
- Load on the specimen shall be applied continuously at a constant rate such that failure in the weakest rocks occurs within 15-30 s. A loading rate of 200 N/s is recommended.
- Where the testing machine is fitted with a force/displacement recorder, a record should be taken during testing so that the load for primary fracture can be precisely determined (in some cases load continues to increase after primary failure as the split specimen is still bearing load).
- If a load/displacement recorder is not available on the testing machine, then care must be taken by the operator to detect the load at primary failure. At primary failure, there will be a brief pause in the motion of the indicator needle. However, the difference between the load at primary failure and ultimate load bearing capacity is at most only about 5%.
- The number of specimens per sample tested should be determined from practical considerations, but normally ten are sufficient.

REFERENCE

- Refer to the complete ISRM procedure: Tensile strength tests ISRM

LEARNING OUTCOME 6.1.1.19

DESCRIBE AND EXPLAIN HOW SAMPLES MAY BE COLLECTED FOR BRAZILIAN INDIRECT TENSILE STRENGTH DETERMINATIONS

CONNECTION 54

Refer to "LEARNING OUTCOME 6.1.1.7" on page 398 , for general sample selection protocols that can be followed in a similar manner.

Advantages of this method include the unproblematic producible specimen preparation and an easy testing set-up. Because of the small sample size it is possible to prepare a larger number of samples from of the same rock volume (if core is drilled from rock sample) or core length (usually from NX core samples).

LEARNING OUTCOME 6.1.1.20

DESCRIBE AND EXPLAIN HOW SAMPLES ARE PREPARED FOR THE BRAZILIAN INDIRECT TENSILE TESTS

The discs are cut from core to the correct thickness required for the Brazilian disc test. Discs are normally cut to a width equal to half the diameter of the core.

back

From the ISRM tensile test procedure (link to tensile strength test) and Outcome 6.1.1.18, the following is noted on sample preparation:

- The test specimens should be cut and prepared using clean water.
- The cylindrical surfaces should be free from obvious tool marks and any irregularities across the thickness of the specimen should not exceed 0.025 mm. End faces shall be flat to within 0.25mm and square and parallel to within 0.25°.
- Specimen orientation shall be known and the water content controlled or measured and reported in accordance with the "Suggested method for determination of water content of a rock sample", Method 1, ISRM Committee on Laboratory Tests, Document No. 2, November 1972.
- The specimen diameter shall not be less than NX core size, approximately 54mm, and the thickness should be approximately equal to the specimen radius.
- The test specimen shall be wrapped around its periphery with one layer of the masking tape and mounted squarely in the test apparatus such that the curved platens load the specimen diametrically with the axes of rotation for specimen and apparatus coincident.

LEARNING OUTCOME 6.1.1.21

DESCRIBE THE METHOD OF CALCULATING STRENGTH FOR BRAZILIAN INDIRECT TENSILE STRENGTH TESTS

The tensile strength of the specimen (σt), can be calculated by the following formulae:

$$\sigma t = 0.636\ P/(D.t)\ \text{(MPa)}$$

or

$$\sigma t = 2F/(\pi.d.D)\ \text{(MPa)}$$

where
P and F represent the load at failure (N),
D is the diameter of the test specimen (mm),
t and d represent the thickness of the test specimen measured at the centre (mm).

REFERENCE

- ISRM tensile strength tests

LEARNING OUTCOME 6.1.1.22

SKETCH, DESCRIBE AND EXPLAIN HOW POINT LOAD INDEX TESTS ARE CARRIED OUT

The point load strength tests is intended to be an index test for the strength classification of rock materials and can be used to predict uniaxial tensile and compressive strengths of the materials tested. The tests determine the Point Load Strength Index (Is(50)) of rock samples and can be determined for core samples (axial or diametrical tests), cut blocks (block test) or irregular lumps (lump test). This test has the benefit that it requires simple and transportable equipment and can be performed on site during the logging of core runs.

Figure 16 shows a point load test apparatus to determine the point load index of the rock sample. A force is applied on a rock sample by hydraulics

back

back

by moving the pump handle up and down. The pressure gauge records the applied force at failure from which the rock strength is calculated. Most point load testing apparatuses have steel balls on the heads that are replaceable. Borehole core or lump samples can be used for point load index tests. The core samples may be Normal NX core or others, but when tested, results are calibrated and compared to a standard 50mm core diameter called the IS 50 curves

There are many advantages to carrying out point load tests, and these include:

- The tests can be carried out on site;
- The tests are cheap and are far less costly than sending specimens to a laboratory for testing;
- Servicing the point load test apparatus is cheap and easy;
- The tests are fast and reliable;
- The test results can be used immediately for core logging parameters.

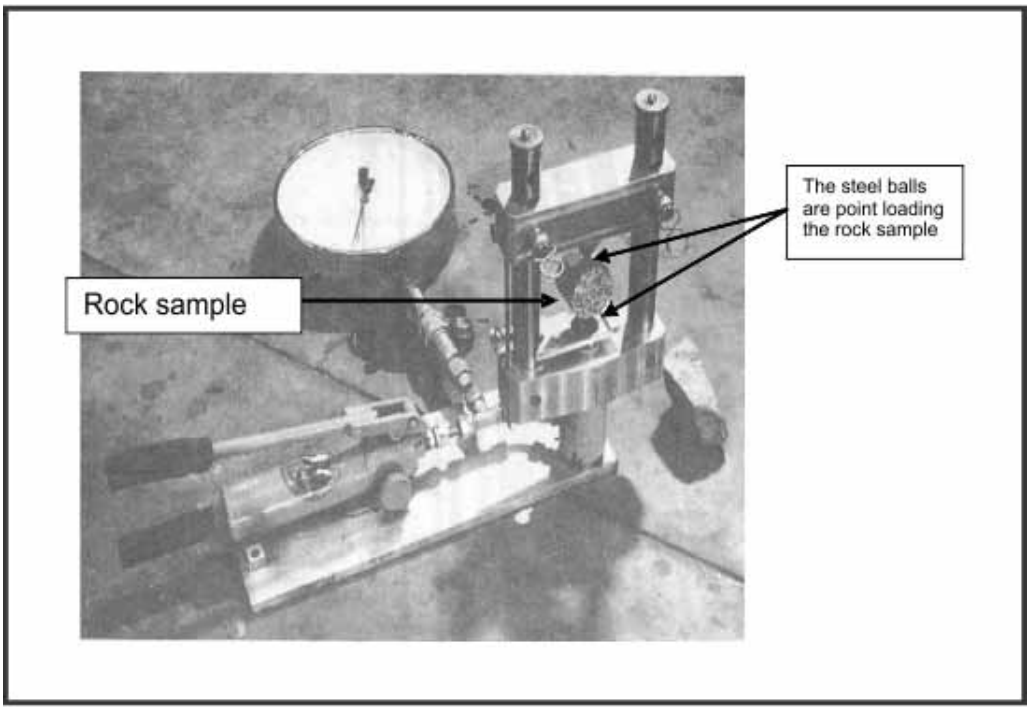


Figure 16: Photo indicating a point load testing apparatus

The small steel balls used as platens in the point load test that make contact with the rock sample may wear resulting in a larger contact area. For this reason, the point load tester apparatus must be serviced and calibrated on a regular basis.

Correlation of the Point Load Index to the compressive and tensile strengths of the material is normally done by testing samples of the same rock type in a laboratory. The results are graphed and calibration factors derived that can then be applied to the point load index test results for this rock type, allowing predictions on rock strengths for this rock material, based on point load strength tests only.

back

LEARNING OUTCOME 6.1.1.23

SKETCH, DESCRIBE AND EXPLAIN THE TEST EQUIPMENT REQUIRED FOR POINT LOAD INDEX TESTS

CONNECTION 55

For more detail on the apparatus specifications, refer to the ISRM Point Load Index test procedure.

The testing machine consists of a loading system (loading frame, pump, ram and platens) and a pressure gauge for measuring the load that P that is required to break the specimen. The samples are broken by a concentrated load applied through a pair of spherically truncated, conical platens (Figure 17).

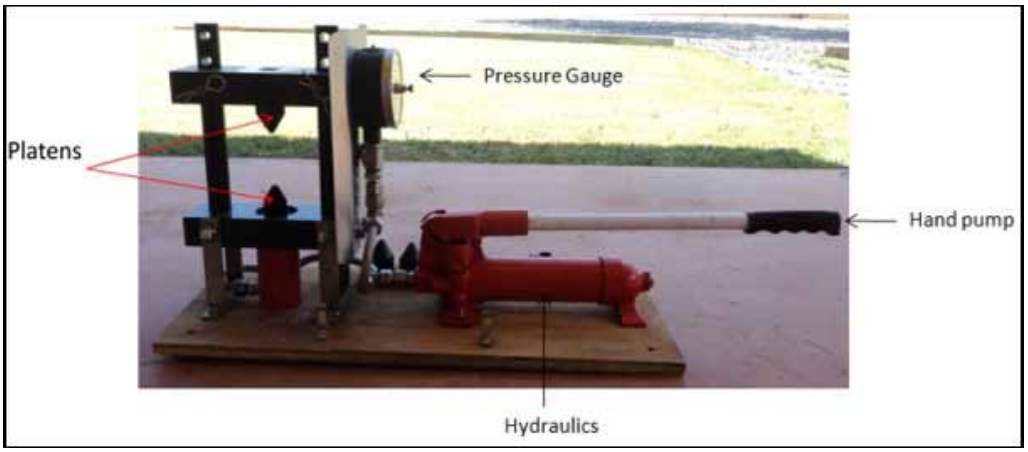


Figure 17: Point Load Test Apparatus

LEARNING OUTCOME 6.1.1.24

DESCRIBE AND EXPLAIN THE TEST PROCEDURE FOR POINT LOAD INDEX TESTS

From the ISRM point load strength test procedure, the following is noted on the test procedure:

- A test sample consists of many specimens with similar strengths that can be used to determine a single Point Load Strength value for the specimens.

NOTE

This means that different Point Load Strength values will be determined for significantly different materials.

- The test equipment should be calibrated using independent load cells and displacement blocks so that the force and displacement readings can be verified;
- At least 10 tests per sample are required (more if material is anisotropic or heterogeneous);

1. Diametrical test:
   - Sample is placed diametrically in the apparatus so that the platens can be closed onto the round core surfaces, ensuring that the distance L (Figure 18a) between the contact points and the end of the sample is at least 0.5 x sample core diameter;
   - The distance D is recorded;
   - The load on the sample is increased until failure of the sample occurs at a rate that would see failure occur within 10-60s;
   - The force at failure P is recorded;
   - Repeat this process for all the samples;

back

2. Axial test:
   - The specimen is placed in the apparatus and the platens closed to make contact along a line perpendicular to the core ends (Figure 18b);
   - The distance D and the specimen width W perpendicular to the direction of loading are recorded (Figure 18b);
   - The load on the sample is increased until failure of the sample occurs at a rate that would see failure occur within 10-60s;
   - The force at failure P is recorded;
   - Repeat this process for all the samples;
3. Block and irregular lump test:
   - The specimen is placed in the apparatus and the platens closed to make contact with the smallest dimension of the lump or block, away from corners and edges (Figure 18c and Figure 18d);
   - The distance D and the smallest specimen width W perpendicular to the direction of loading is recorded (Figure 18b);
   - If the sides are not parallel, W is calculated from (W1+W2)/2 (Figure 18d);
   - The load on the sample is increased until failure of the sample occurs at a rate that would see failure occur within 10-60s;
   - The force at failure P is recorded;
   - Repeat this process for all the samples.

DEFINITION

Isotropy: The rock material has the same properties in all directions.
Anisotropic: The rock material has different properties in all directions.
Homogeneity: The rock material has the same properties at all points.
Heterogeneity: The rock material has different properties at all points.

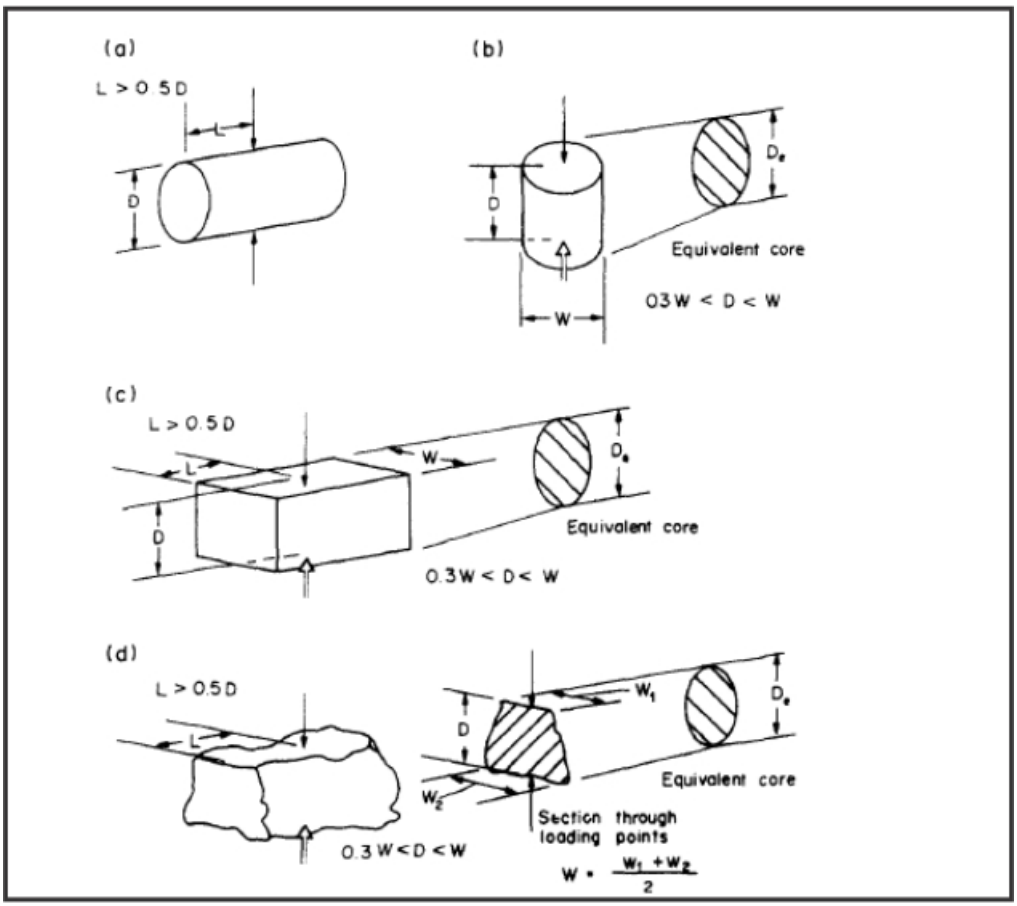


Figure 18: Point Load Index tests

back

back

IMPORTANT

It is important that invalid test results are excluded from the recordings used to assess the rock material strengths. Figure 19 shows failed samples that represent valid and invalid tests based on the mode of sample failure experienced:

Valid tests:

- Diametrical (Figure 19a)
- Axial (Figure 19b)
- Block (Figure 19c)

Invalid tests

- Diametrical (Figure 19d)
- Axial (Figure 19e).

More detail and descriptions are given in the ISRM Point Load Strength test procedures.

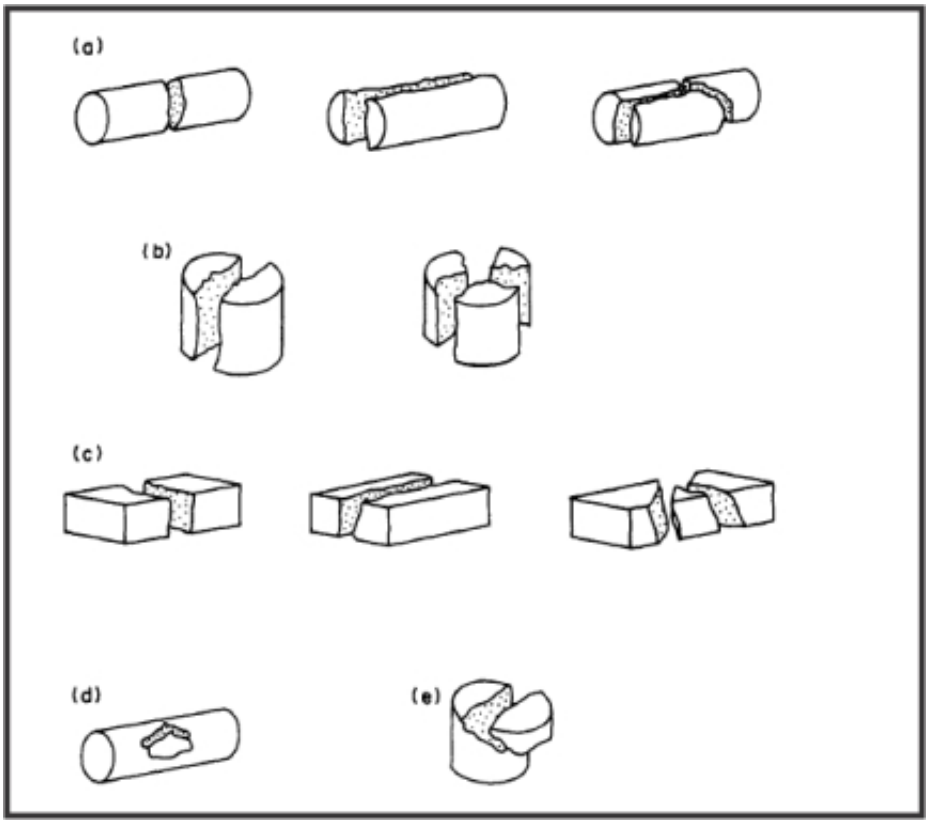


Figure 19: (a) – (c) represent valid and (d) and (e) invalid test results

back

INTERESTING INFO

Fracturing of the sample is caused by induced tension; therefore, it is essential that a consistent mode of failure be produced if the results obtained from different specimens are comparable. Marked planes of weakness (bedding, foliation, cleavage, sealed joints etc.) will have a strong influence on the test results. If rock types to be tested are anisotropic, specific test procedures are discussed in the ISRM procedure.

Point load tests are generally not applicable for rocks with strengths below 25MPa, as the points indent the rock. It is therefore very probable that point load index tests are not appropriate for material such as coal. A strength testing method was developed to allow field strength tests on coal samples, called the Impact Splitting Test. In general, the impact splitting test was developed whereby a constant impact is applied to every 20mm of drill core by dropping a 1.5kg chisel from a predetermined height according to core diameter. The resulting fracture frequency is then used to determine a roof rating. The instrument shown in Figure 3 consists of an angle iron base that holds the core. Mounted on this, is a tube containing a chisel with a mass of 1.5kg and a blade width of 25mm. The chisel is dropped onto the core from a constant height according to core size – 100mm for a 60mm diameter core and 64mm for 48mm diameter core. The impact splitter causes weak or poorly cemented bedding planes and laminations to open, thereby giving an indication of the likely in-situ behaviour when subjected to bending stresses.

Figure 20: Impact splitting test equipment

REFERENCE

- COL812 Roof ratings systems for Coal Mines
- Impact splitting paper

IMPORTANT

Extreme caution must be exercised when carrying out tests on very strong rock (above 200 MPa) as the specimens may shatter or explode when they fail.

LEARNING OUTCOME 6.1.1.25

DESCRIBE AND EXPLAIN HOW SAMPLES MAY BE COLLECTED FOR POINT LOAD INDEX DETERMINATIONS

CONNECTION 56

Refer to "LEARNING OUTCOME 6.1.1.7" on page 398 for general sample selection protocols that can be followed in a similar manner

Point load test specimens can be collected from borehole cores or lump samples from the area of interest. Most point load tests are carried out using core samples. Point load testers may have a limit on the size of samples to be tested and samples must be able to fit in the test machine. Lump samples and core samples may be broken into smaller pieces to fit into the point load apparatus, providing the limits as set out in the ISRM procedure are adhered to.

LEARNING OUTCOME 6.1.1.26

DESCRIBE AND EXPLAIN HOW SAMPLES ARE PREPARED FOR POINT LOAD INDEX DETERMINATIONS

From the ISRM point load strength test procedure (link to point load strength test), the following is noted on sample preparation:

- Samples could be gathered from core, regular or irregular shaped rocks;
- For routing testing, samples must be tested either fully water saturated or at the natural water content;
- Diametrical tests:
  - Length: diameter orientations > 1.0;
- Axial tests:
  - Length / diameter ratios of 0.3 to 1.0;
- Regular block or irregular lump tests:
  - Blocks or lumps of size 50 ± 35mm;
  - Shapes shown in Figure 18 (c and d);
  - A ratio D/W of between 0.3 and 1.0, but preferably closer to 1.0 (Figure 18).

Core used in point load tests does not need to be prepared except for breaking it up into manageable sizes. However, the core must be correctly marked to avoid confusing different test specimens. Lump samples are broken into pieces that fit into the test machine.

NOTE

Avoid specimens with obvious flaws such as sealed or partially open joints, obvious bedding and foliation planes etc. Choose the most homogenous specimen possible so that the results will not be biased.

LEARNING OUTCOME 6.1.1.27

DESCRIBE THE METHOD OF CALCULATING STRENGTH FOR POINT LOAD INDEX TESTS

The calculation of an appropriate point load strength index follows the following steps:

- Perform the point load strength tests and record the necessary information;

- Calculate an uncorrected PLS Index (Is);
- Correct the results for the actual size (diameter) of the core to the 50mm core diameter standard.

Uncorrected PLS Index

Is is determined for each of the tests conducted, using the relationship

Is = P / De2,

where P is the load at failure and De is the 'equivalent core diameter' given by

- Diametrical tests: De2 = D2;
- Axial, lump and block tests: De2 = 4A/π where A = WD = Minimum cross sectional area of a plane through the platen contact points.

Correction for core diameter size

Is varies with varying D (diametrical tests) and De (axial, block and lump tests), such that a size correction must be done. The size corrected Point Load Strength Index Is(50) suggests the value of Is that would have measured for a specific sample if the diameter of the sample had been 50mm.

The size correction can be done with one of the following methods:

1. Tests conducted at a variety of dimensions (Figure 21):
   - Test the samples over a range of D or De values;
   - Plot the relationship between P (failure load) and De2 (log-log plot should provide a straight line);
   - Points that plot far away of the straight line that result from the previous step could be discarded, but should not be deleted from the records;
   - Obtain the value of P50, corresponding to a De2 of 2500mm2 (De = 50mm) from the abovementioned line and
   - Calculate the material's representative PLS Index IS(50) by dividing the P50 by 2500mm2.
2. Tests conducted on similar sized samples:
   - Test the samples;
   - Determine the size correction factor F for each test from F=(De/50)0.45 ;
   - Obtain the value of Is(50), corresponding to a De2 of 2500mm2 (De = 50mm) for each of the tests by using Is(50) = F x Is, where Is is the Point Load Strength Index calculated using the recorded failure load (P) and dimension (D or De) values for each sample test;
   - The mean Is(50) value for the material tested can be derived as follows:
     - For 10 or more tests: Remove the lowest and highest two results and calculate the mean for the remainder of the test results;
     - For less than 10 tests: Remove the single lowest and highest results and calculate the mean for the remainder of the test results.

back

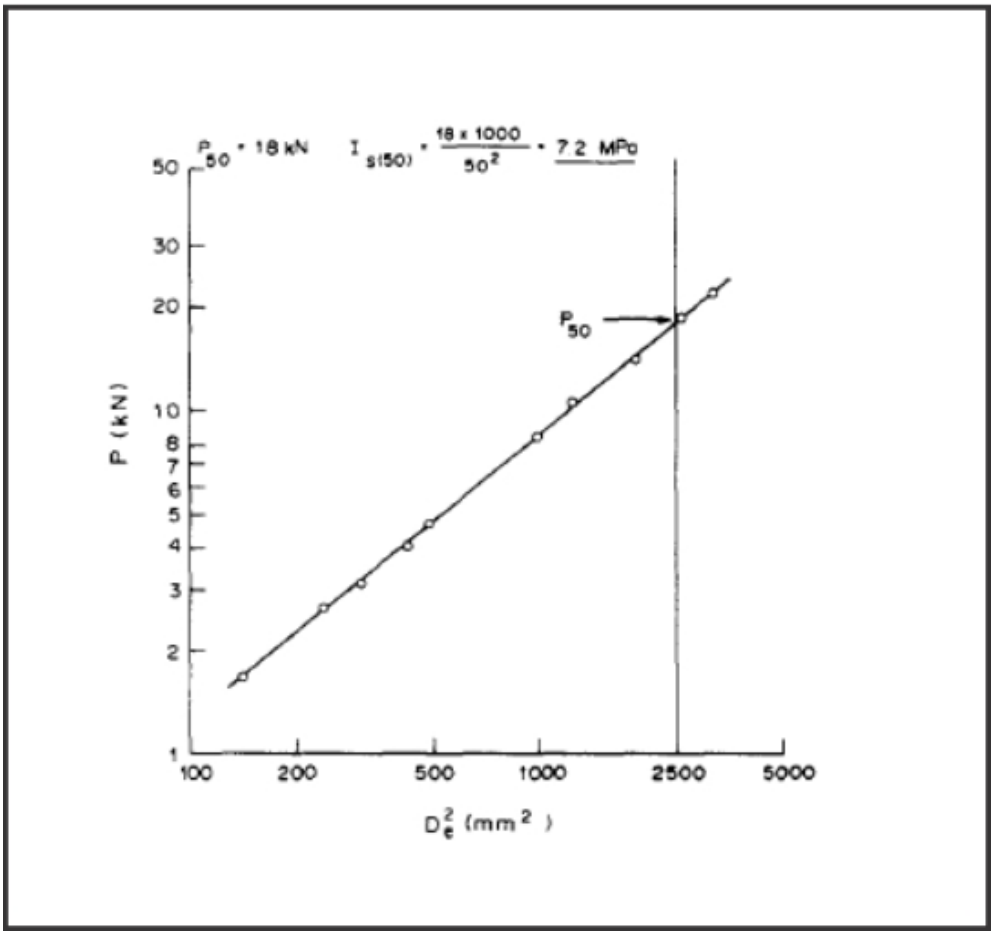


Figure 21: Size correction

EXAMPLE

Examples of the steps discussed in the process above are shown below:

- Table 3: Sample sheet;
- Table 4: Results of Is determination, including size correction;
- Figure 22: The relationship between the calculated PLS Index Is and laboratory UCS test results.

Sample ID	Borehole ID	Diameter	Depth (from)	Depth (to)	Lithology
SNL1 UCS1	SNL1	46	1182.13	1182.33	Anorthosite
SNL1 UCS2	SNL1	46	1189.81	1190.05	Pyroxenite above triplets
SNL1 UCS3	SNL1	46	1196.38	1196.55	Pyroxenite below triplets
SNL1 UCS4	SNL1	46	1201.06	1201.21	FW Norite
SNL1 UCS5	SNL1	46	1223.04	1223.21	Deep FW Norite

Table 3: Example of a point load input data sheet

Sample ID	Bore-hole ID	Depth (from)	Depth (to)	Diam-eter (mm)	Size Correction	Read-ing (KN)	Reading (MN)	Point Load Index (I_8)
SNL1PLT2	SNL1	1182.33	1182. 45	46	0.963173493	23	0.023	10.47
SNL1PLT4	SNL1	1190.17	1190. 30	46	0.963173493	15.5	0.0155	7.06
SNL1PLT5	SNL1	1196.38	1196. 75	46	0.963173493	9.6	0.0096	4.37
SNL1PLT6	SNL1	1201.06	1201. 36	46	0.963173493	10	0.0100	4.55
SNL1PLT7	SNL1	1223.04	1223. 29	46	0.963173493	11.9	0.0119	5.42
SNL1PLT1	SNL2	1198.00	1198. 09	46	0.963173493	27	0.0270	12.29
SNL1PLT2	SNL2	1194.73	1194. 88	46	0.963173493	18	0.0180	8.19
SNL1PLT3	SNL2	1200.78	1200.89	46	0.963173493	13	0.0130	5.92

Table 4: Point Load Strength Index test results

back

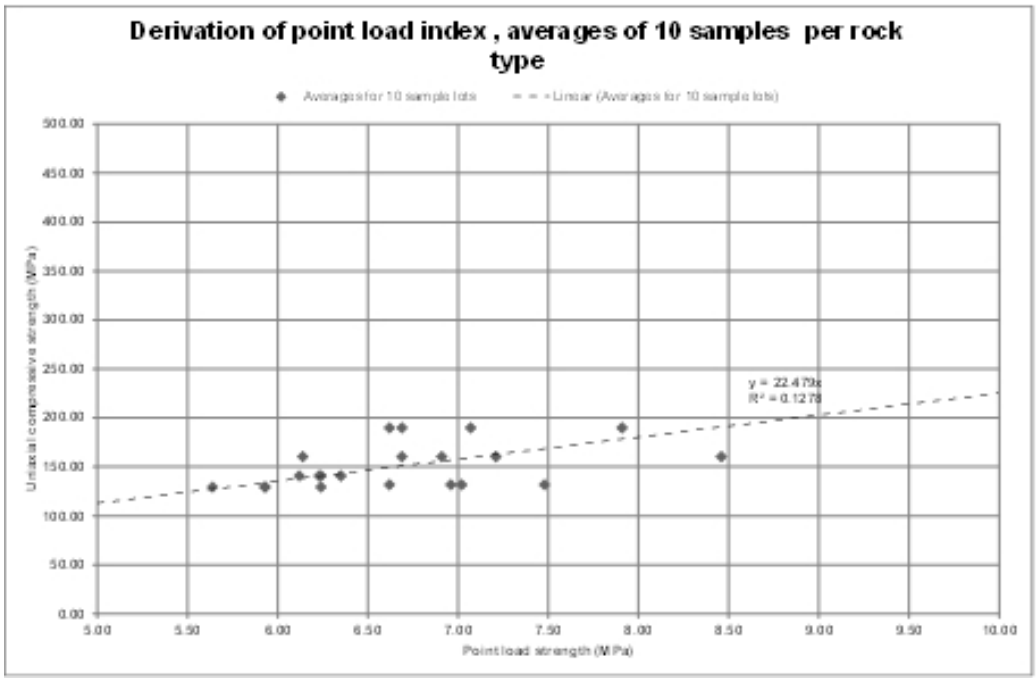


Figure 22: Relationship between UCS and Is

LEARNING OUTCOME 6.1.1.28

EVALUATE THE SHORTCOMINGS OF EACH OF THE ABOVE TESTS AS INDICATORS OF THE LARGE-SCALE STRENGTH OF ROCK

Inherently, a rock mass is traversed by discontinuities of various properties (i.e. sizes, strengths, orientations, shapes, roughness, frequency, infilling and moisture content), intact rock portions of various properties (i.e. strength, grain constitution, grain size, inherent hairline cracks and anisotropy) and several intrusions of different rock types, while it may also contain water or fluids in various amounts.

The rock mass is the total in-situ described above, but a rock sample subjected to any of the abovementioned tests should at worst have hairline cracks and is considered 'an intact rock sample'.

The diagram below (Figure 23) depicts the relationship between a rock sample and rock mass characteristics as applied to both underground and surface mining environments. The difference between the strength of a rock sample and a rock mass is that the strength of the small-scale rock sample is invariably stronger than the rock mass that may contain several weaknesses and discontinuities.

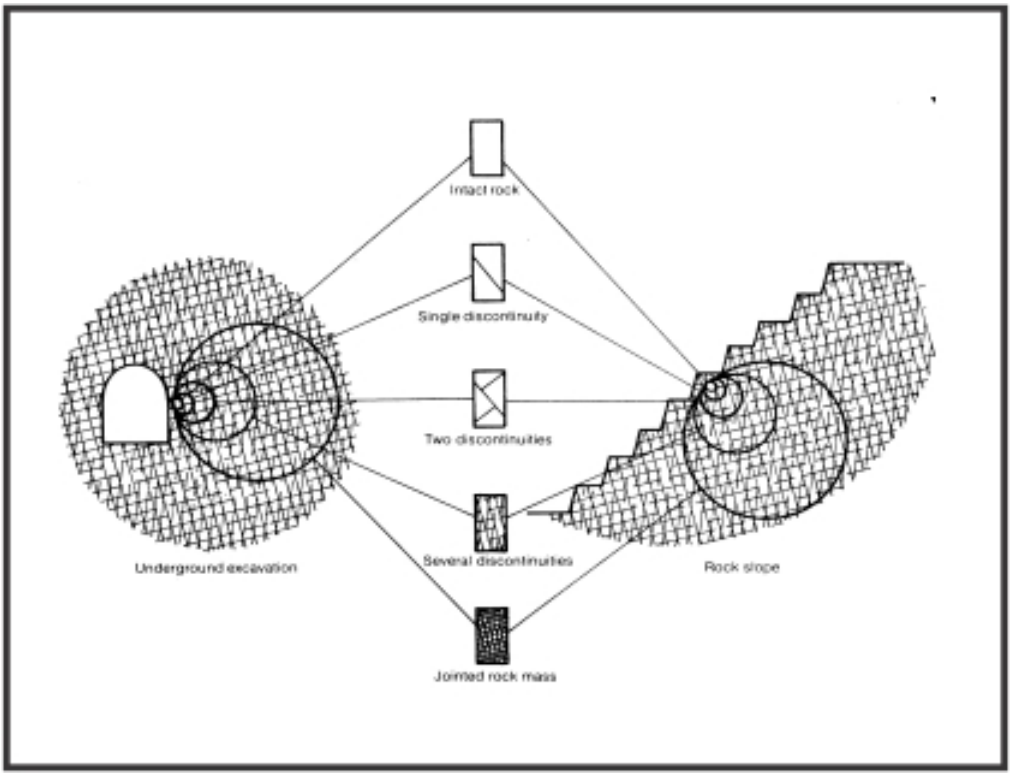


Figure 23: Scale effects on rock mass characterisation. (Brady & Brown, 2004)

The following comments regarding the applicability of test results to the rock mass are also justified:

- In a uni-axial compressive strength test, the a rock sample is tested under unconfined conditions that may only be true in the unconfined skin of underground excavations;
- In a tri-axial compressive strength test, the small rock sample is tested under tri-axial loading conditions that are representative of the confined rock mass, but the levels of actual confinement are unknown;
- Shear strength tests estimate the shear strength of a rock sample or discontinuity surface and might have been selected from a 'weaker' or 'stronger' section within the rock mass or along a discontinuity.

NOTE

The fact that the testing of small intact samples does not appear to represent the rock mass strength, suggests that rock tests may be unnecessary.

However, the tests also provide essential understanding of the behaviour of the rock material and rock mass. Properties such as the Young's Modulus, Poisson's ratio, Friction angle, Cohesion, Confinement parameters etc can be derived.

It is also essential that the rock engineer obtains these properties and then embarks on other processes to understand and predict the rock mass behaviour, using the test results as base and input into the processes. These processes can include the combination of rock mass quality indices and test results to determine rock mass strengths or input of test results into computer-based numerical modelling.

REFERENCE

- Brady & Brown, 2004

LEARNING OUTCOME 6.1.1.29

DESCRIBE AND EXPLAIN HOW THE FOLLOWING VARIABLES MAY AFFECT THE INDICATED STRENGTH OF ROCK SAMPLES TESTED IN UNI-AXIAL COMPRESSION:

Specimen end-effects

The end effects along the platens of the test machine include:

- Non-uniform distribution of the load across the test sample contact area if contact areas (ends) of the specimen are not prepared to a high degree of parallelism, otherwise bedding-in and bending effects in the test can become excessive and could distort the results;
- Deformation of platens due to the use of incorrect platen material. Hardened steel end pieces are usually used to better distribute the load onto the specimen (Figure 24a).
- Non-axial loading: The ISRM standards on rock testing prescribe the use of spherical seats on the end pieces to reduce non-axial-loading;
- High levels of friction on sample contact with platens. If high levels of friction are created between the platen and sample, the top and bottom parts of the sample are confined by frictional forces and the behaviour of the material altered (Figure 24b). ISRM procedures suggest that the use of capping materials or end surface treatments other than machining is not permitted.

CONNECTION 57

Also refer to ISRM Uni-axial test procedure and Paper 1, Outcome 4.1.1 on page 96

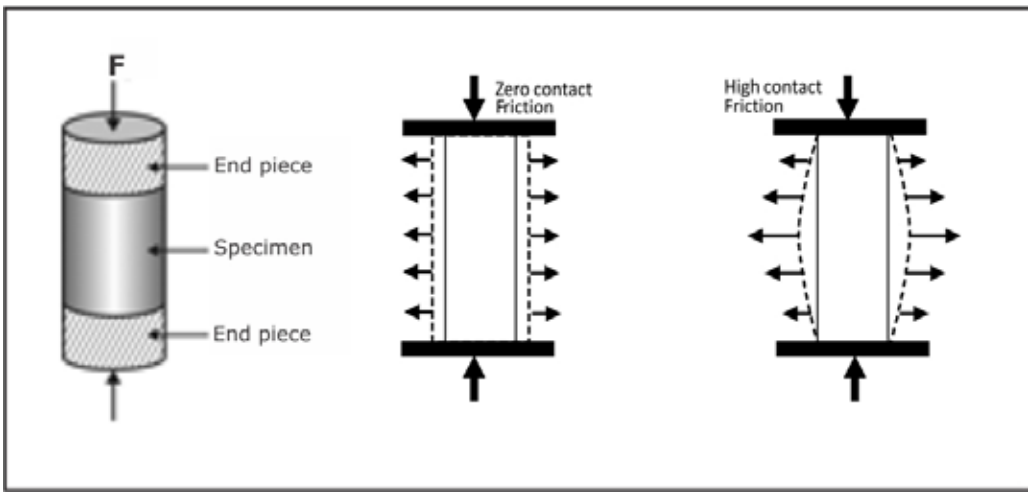


Figure 24: (a) Hardened end pieces used in rock testing (b) impact of contact friction on sample behaviour

CONNECTION 58

Also refer to Paper 1 Outcome 4.1.16 on page 124

Volume of rock tested

Test machine stiffness

When a rock sample fails, the post-failure behaviour of the sample will be different when tested in a 'stiff' loading machine compared to a 'soft' loading machine. Consider the relationships graphed in Figure 25 representing a specimen failing under soft (a) and stiff (b) loading conditions:

- When loaded in a soft system, the energy that can be absord by the specimen during displacement (post peak) under applied loads is given by the area ABED. The energy provided by the loading system is represented by area AFED. The surplus energy available in the machine (AFED - ABED) results in the transfer of energy from the machine to the sample, resulting in violent failure of the sample.

- When loaded in a stiff system, the energy that can be absorbed by the specimen during displacement (post-peak) under applied loads is given by the area ABED. The energy provided by the loading system is represented by area AGED. The surplus energy absorption capacity in the sample (ABED - AGED) results in 'stable' failure of the sample due to no energy transfer to the sample. Post-peak behaviour of the sample can now be properly described by the equipment used in the tests.

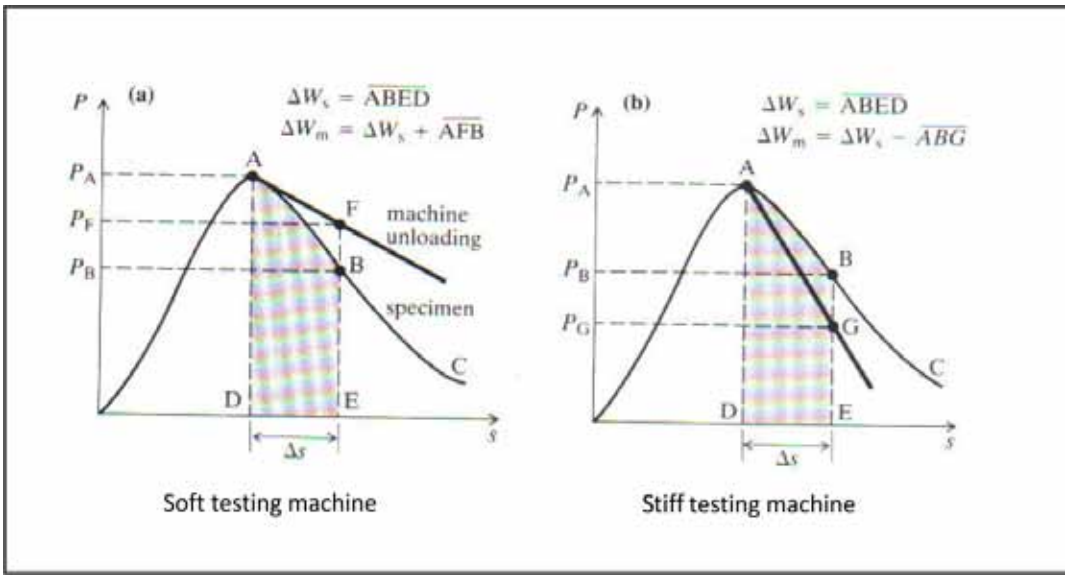


Figure 25: Sample behaviours when loaded in stiff and soft systems (Brady and Brown)

REFERENCE

- Page 92, Brady H.G.B. and Brown E.T., Rock mechanics for underground mining, 2006

Loading rate

The rate at which any material is loaded is known to affect the forces that work on the material. A simple example is the fact that a motor vehicle crash causes more damage and injury when the vehicle travels at high speeds, due to the large forces that are applied to counteract the movement of the vehicle.

The loading rate in a rock test must be controlled so that the following will not occur:

- Variation in strength test results due to variations in loading rates. This will not allow any comparison of results conducted at different times or laboratories;
- Over-stating of rock material strengths due to too rapid loading rates leads to increases in measured strength, even though it has been reported to be quite small for rock material. For example: An increase of five orders of magnitude in the rate of loading increases the strength of rock by around 20-30%.
- Recording and reporting of creep behaviour due to too low loading rates. At elevated stress levels, very low loading rates may involve creep behaviour that can reduce effective strengths. Example: reduction of up to 20% for typical quartzite material.

For this purpose the industry has accepted standards for loading rates to be used during rock testing (see ISRM testing procedures in previous outcomes).

back

LEARNING OUTCOME 6.1.1.30

SKETCH AND DESCRIBE THE COMPLETE STRESS-STRAIN GRAPH FOR ROCK IN UNI-AXIAL COMPRESSION, INDICATING:

CONNECTION 59

Refer to Paper 1 Outcome 4.1.5 on page 104

- Hysteresis in the post-peak portion of the curve
- The associated radial strain
- The associated volumetric strain
- The brittle-ductile transition
- The residual strength

LEARNING OUTCOME 6.1.1.31

SKETCH AND DESCRIBE THE COMPLETE STRESS-STRAIN GRAPH FOR ROCK IN TRI-AXIAL COMPRESSION, INDICATING:

CONNECTION 60

Refer to Paper 1 Outcome 4.1.4 on page 101 and to Paper 1 Outcome 4.1.5 on page 104

- Hysteresis in the post-peak portion of the curve
- The associated radial strain
- The associated volumetric strain
- The brittle-ductile transition
- The residual strength

NOTE

The following Rock testing presentation was provided courtesy of Dr Halil Yilmaz of the University of the Witwatersrand.

back

back

FUNDAMENTALS OF MONITORING ROCK-MASS BEHAVIOUR

OBJECTIVES OF MONITORING

LEARNING OUTCOME 6.2.1.1

DESCRIBE THE GENERAL OBJECTIVES OF A MONITORING SYSTEM

The following important definitions will assist you in understanding some of the issues mentioned in this outcome.

DEFINITION

Monitoring	Monitor or monitoring generally means to be aware of the state of a system, such as to observe a situation for any changes that may occur over time, using a monitor or measuring device of some sort.
Instrumentation	Instrumentation is the branch of engineering that deals with measurement and control. An instrument is a device that measures or manipulates variables such as mass, acceleration, displacement, stress etc.
Collect	To gather or to assemble; to receive or compel information or something with a physical value.
Integrity	Integrity is consistency of actions, values, methods, measures, principles, expectations and outcome.
Data	Data are pieces of information that represent the qualitative or quantitative attributes of a variable or set of variables.
Archiving	An archive is a collection of historical records, and the location in which the collection is kept.
Back-up	Back-up refers to making copies of data so that these additional copies may be used to restore the original after a data loss event.

Table 5: Definitions of terms used in monitoring

Monitoring is the surveillance of engineering structures, either visually or with the help of instruments. Within a general geomechanics context, monitoring may be carried out for four main reasons:

1. to record the natural values of, and variations in, geotechnical parameters such as:
   - water table level,
   - stress levels and fracturing,
   - deformations,
   - load in support elements,
   - condition and spacing of joints and
   - seismic events;
2. to ensure safety during construction and operation by giving warning of the development of excessive:
   - ground deformations,
   - groundwater pressures;
   - loads in support and reinforcement elements;
3. to check the validity of the assumptions, conceptual models and values of soil or rock mass properties used in design calculations;
4. to confirm layout and support performance.

back

NOTE

'every instrument on a project should be selected and placed to assist in answering a specific question; if there is no question, there should be no instrumentation' (taken from Ryder and Jager).

IMPORTANT

It is a responsibility of a Rock Engineer to monitor the safety of mine excavations, the rock mass response, check it against designs and when necessary, adjust the design or recommend remedial actions to ensure the long-term stability of the excavation and more importantly to ensure the safety of workers.

LEARNING OUTCOME 6.2.1.2

DESCRIBE AND EXPLAIN THE OBSERVATIONAL METHOD OF DESIGN IN ROCK ENGINEERING

Visual observations form a major part of the monitoring processes implemented by Rock Engineers. It is for that reason that rock engineering personnel should regularly visit and audit working places. Observations give the Rock Engineer information on the actual behaviour of the rock mass as well as the performance of the support designed for the area. In many cases, designs and support strategies had to be changed following observations made during visits to the working places.

Observations should be made in a planned and controlled manner to be effective. If not, non-essential issues may be noted and the most important observations not made. To assist the user, the following suggestions are made:

- Before embarking on an observational monitoring visit (underground visit), the aim of the visit should be clear as this aim will determine the type of observations to be made. For example: If the aim is to determine the behaviour of support, everything that may affect their behaviour should be observed, while if the aim is to determine the causes of a fall of ground, many more issues, including but not limited to all issues related to support should be noted;

During routine observations, many routine issues must be noted and the following process is suggested:

- Enter the area of interest and linger in a particular position. At this position, note the following:
  i. Rock mass:
     - Rock types exposed;
     - Rock quality;
     - Jointing (dip and dip direction, spacing, conditions);
     - Geological structures other than jointing faults, dykes, rolls, etc);
     - Stress fracturing (shape, dip, density, condition, etc);
     - Whether the area is stable (safe), unstable or potentially unstable (conditions appear stable but may become unstable soon);

NOTE

If conditions are unstable, ensure no personnel enter the working place and determine the contributing factors to the instability. If they are potentially unstable, determine those factors that can be addressed to remain stable or withdraw personnel.

     - Unusual conditions not normally associated with this area of the mine such as ground water, additional joint sets, shear

zones, etc

ii. Support:
- Support type, spacing;
- Installation quality (preloading, orientation, etc);
- Behaviour (yielding, failing, stable, etc);
- Behaviour (is it ensuring stable conditions?);

iii. Worker practices:
- General adherence to standards;
- Adherence to safety precautions w.r.t. support installation, working in supported areas, etc;
- Execution of drilling, charging up, cleaning and the impact of those on the conditions (if any);

- Move to the next position in the working place and repeat the observations;
  - It is critical to make accurate notes while making the observations so that the precise position of observations can later be available for discussion and further analysis, while the comparison of issues with those taken earlier in the same working place serves as good data to monitoring these issues.
  - Take photographs if possible for reference, inclusion in reports and explanation of conditions to others that have not visited the site;
  - Perform measurements to confirm observations using tapes, distometers, clinorules, etc. Actual measurements allow good data that can be used for monitoring of behaviour of the rock mass (closure rates, etc), support (deformation, etc) and worker practices (socket lengths, etc);

When problematic areas are visited and observations are used as monitoring method, the following is suggested:

- Prior to the visit ensure total familiarity of the working place in terms of mining layouts, production rates, production standards applicable, geology in the area, seismic history of the area, previous reports on the area by rock engineers and safety personnel;
- Develop a 'mind picture' of the conditions to be expected and the 'possible' reasons for, or at least the factors that could contribute to the problems experienced, based on the knowledge gathered above;
- Perform observations looking for the factors devised previously, constantly testing the 'mind picture' made previously and asking the following questions:

  i. If a factor expected to be present is:
  - present, did it contribute?
  - absent, why is it absent?

  ii. If conditions are different to those expected, why are they different?

  iii. If factors or conditions that were not expected are present, what impact did they have?
  - Make clear and precise notes for later analysis;
  - Perform measurements to augment the observations and assist with analysis such as deformation on support, yielding of support and spacings to allow calculation of estimated support resistance etc.

NOTE

REFERENCE

Observations may not be limited to the above, but must at least include the issues mentioned.

- Site investigation paper

CONNECTION 61

Refer to "LEARNING OUTCOME 3.1.2.12" on page 163 and

"LEARNING OUTCOME 3.1.2.13" on page 167 and

"LEARNING OUTCOME 3.1.2.14" on page 169

Design based on observations is called empirical designs (i.e. is based on experience) and include methods such as rock mass quality support and span design methods.

NOTE

Design just using observation experience on a particular mine or area can be valid, but must be used very carefully as it assumes that the rock mass and support unit behaviour would be similar as that experienced in the past and that nothing exists in the area that could result in any other outcome as the one derived from experience.

In applying this design method, the observations made, noted and recorded through the process above are of critical importance as they form the base of the design. The analysis of this data, conclusions and design should be carefully thought through and recorded with all assumptions clearly stated, so that any future evaluation of the design is fully aware of input data, analysis methods and final conclusions.

Due to this requirement, empirical systems as that mentioned above were found valid as the determination of the rock quality, ratings given, conclusions drawn etc have all been well recorded and tested over many years by using the system correctly and similarly every time.

LEARNING OUTCOME 6.2.1.3

DESCRIBE AND EXPLAIN WHAT MAY BE LEARNED FROM MONITORING

CONNECTION 62

Refer to "LEARNING OUTCOME 6.2.1.1" on page 425

If rock engineering has four main reasons for conducting instrumentation or monitoring programmes, each of these reasons will be able to provide specific information:

1. To record the natural values of, and variations in, geotechnical parameters before the start of a project:
   - Rock mass properties, strengths, quality;
   - Joint sets, spacings, orientations;
   - Support loads, yielding, failure;etc
2. To ensure safety during mining by giving warning of unexpected or excessive deformations:
   - Closure rate;
   - Seismic activity;
   - Slope stability;
   - Wedge stability in large excavations;

- Support capability / capacity to handle conditions;etc

3. To confirm the validity of assumptions and the modelling used in design calculations (i.e. calibration of models and back analysis):
   - Support loads,
   - Closure rates;
   - Slope geometry and stability;
   - Rock mass conditions;
   - Seismic activity; etc
4. To confirm support and layout performance in particular environments:
   - Support loads;
   - Closure / wall deformation;
   - Seismic activity;
   - Slope geometry and stability;etc.

Knowledge of the information listed above will assist the rock engineer to understand rock and rock mass behaviour, and its interaction with support in different mining excavations. It is critical for the rock engineer to admit that monitoring and instrumentation is a critical part of rock mechanics science and can not be ignored.

NOTE

'you do not really KNOW, until you have measured'

LEARNING OUTCOME 6.2.1.4

DESCRIBE AND EXPLAIN HOW THE RESULTS LEARNED FROM MONITORING MAY BE USED IN DESIGN

CONNECTION 63

The uses of monitoring results are well described in examples such as the Outcome 6.3 (seismic monitoring systems) and "LEARNING OUTCOMES 3.2." on page 190 (principles of seismicity) and the GAP034 report on 'Mining layouts at depth'.

The relevant parameters relating to the situation that must be monitored need to be identified and measured. Based on these measurements, conditions, assumptions, etc included in the designs can then be evaluated, conclusions drawn, models calibrated, and designs improved.

EXAMPLE

Using a scanline data gathering method to assist in deriving optimum lead / lag distances between panels in deep level mines.

Information retrieved during the scanline mapping include:

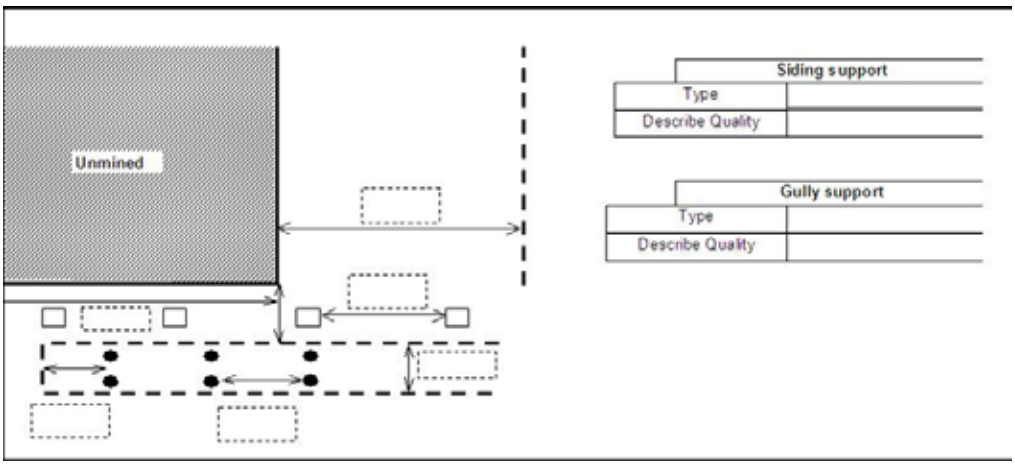
- The type of fracture (joint, stress, bedding, flow structure, fault, dyke) intersected by the scanline
- The orientation (dip angle and dip direction) of the fracture
- The visible length of the fracture
- Aperture and roughness
- Infilling
- Rock type
- Water infiltration
- Rock Quality Designation (RQD)

Basic support information was captured using a diagrammatical data capture sheet.

FRACTURE MAPPING OF THE FACE LEADS OR LAGS

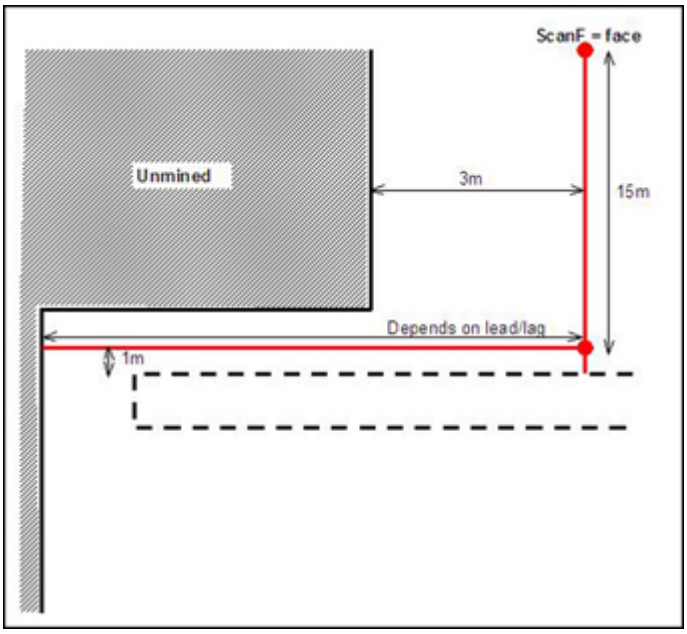
MINE: H/W ROCK TYPE: FACE OR LEAD/LAG SCANLINE
SHAFT: F/WALL ROCKTYPE:
DATE: REEF TYPE:
PANEL NO: APPROX STRENGTH OF H/WALL
DEPTH BELOW SURFACE: APPROX STRENGTH OF F/WALL

DISCONTINUITIES								INFILL		ROCK			
Distance along scanline (m)	Description	Type	Dip angle	Dip Direction	Length of discont.	Aperture	Rough-ness	Type	Thick-ness (mm)	Type	Water	Weath-ering	RQD



Diagrammatical support data capture sheet

Fractures were mapped onto a scanline that was placed on the hanging-wall of the face area at approximately 3 m back from the face and along the updip side of the gully. Although every effort was made to map at least two-thirds of the face scanline and the complete gully scanline, ambient environmental and safety related issues prevented this from being achieved in all scanlines.



Typical scanline layout adopted

The rock mass rating was originally developed for application to logged core. For this study, the rock exposed adjacent to a scanline (tape measure) stretched along the stope hangingwall was viewed as similar to drill core. For each parameter, the numeric value was selected based on observations made at the fracture mapping sites. The strength of the rock was estimated from rock strength databases. For this project, it was assumed that the 'hard lava' hangingwall of the VCR has an unconfined strength of 300 MPa, the quartzites of the Vaal reef and Carbon Leader reef are about 200 MPa, the quartzites of the Basal and B-reef are about 180 MPa, and the 'soft lava' hangingwall of the VCR has a strength of about 70MPa.

A calculation was adopted for RQD where the distance between the joints was calculated and then the RQD was determined as:

RQD=[(Total length of sections between fractures>10cm)/Total length of scanline]%

This measure produced a consistent set of RQD values that never exceeded 100%. The measure was used to compare the rock masses from the different sites and so is valid as long as a consistent measure of RQD is used. The method provides and approximate comparison with RMR values measured using other techniques, but direct comparisons should be performed with caution.

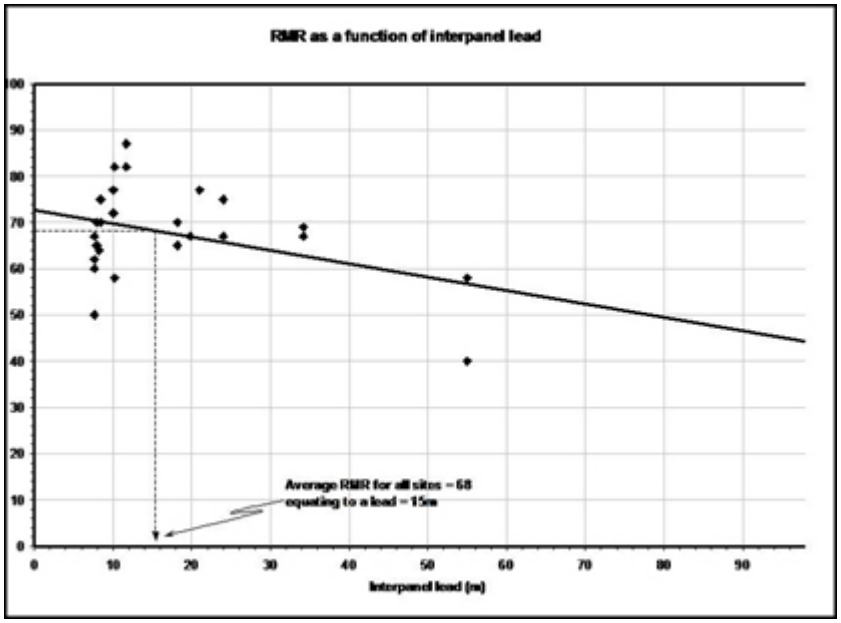
The spacing of the discontinuities was determined as the number of discontinuities divided by the total scanline length. The ground water condition for all fractures was dry and so the same rating was applied to all ratings.

The orientation of fractures parameter becomes ambiguous in a stope, as there are a wide range of fractures at different angles to the strike and dip of the reef. Thus, this parameter was excluded in order to provide a consistent estimate of the rock mass rating as a comparison between the various lead/lag conditions.

The results of the RMR classification are plotted against lead distance below. A best-fit linear extrapolation indicates that the rock mass rating degrades with increasing lead distance. The data for leads greater than 25 m is sparse which, suggests caution in using these graphs. Nevertheless,

back

the spread of data between lead distances of 5 m and 25 m suggests that a relationship does exist between RMR and interpanel lead distance.



The data from the face scan lines tend to show higher RMR values than the gullies. The trend of RMR with lead length is evaluated for the gully scan lines, regardless of rock type. The plot shows that the RMR does decrease with increasing lead length when considering the longer leads. However, the majority of lead lengths and RMR ratings are clustered within a narrow band that does not allow for a good correlation.

NOTE

Much has been written and presented at Sanire, SAIMM and ISRM conferences that shows the use of monitoring results. The user is advised to read as many papers as possible to build an understanding of the use of monitoring results in different environments and for different aims.

INTERESTING INFO

In terms of Chapter 14 of the Mines Health and Safety Act of 1996, the manager must ensure that a competent person approves planning, operational issues, support monitoring, and designs with an auditable paper trail of at least three months.

To allow a competent person to sign off on the requirements of the Act, the following should be ensured:

- That designs are based on reliable instrumentation and monitoring.
- That the necessary instrumentation and monitoring are identified and carried out to mine-specific requirements.

NOTE

The following negative spin-offs could result if inadequate monitoring data exists to confirm or improve designs:

- *Loss of in mineable ore reserves due to over designed pillar sizes;*
- *Increase in support cost, due to areas being over supported;*
- *Loss of revenue;*
- *Incorrect data leading to a lower support resistance and energy absorption requirement that will lead to the under supporting of working areas that may lead to severe falls of ground;*
- *Falls of ground;*
- *Rock bursts;*
- *Fatalities;*
- *Serious injuries;*
- *Damage to equipment.*

back

LEARNING OUTCOME 6.2.1.5

DESCRIBE THE ROCK PARAMETERS THAT MAY BE MEASURED IN A MONITORING PROGRAMME

The following categories of monitoring were identified and described by Jager and Ryder (1999):

1. Rock deformation;
   - Pillar deformations
   - Ground motion
   - Tilt and strain in shaft barrels
2. Rock stress;
   - Stress fracturing density and orientations
3. Support performance;
   - Backfill performance
   - Pillar stresses
4. Joints and rock conditions;
   - Joint sets, spacings, orientations, conditions
5. - Mining layouts;
   - Face shape index
   - Remnant size
6. Falls of ground;
7. Rock related accidents.

CONNECTION 64

Refer to "LEARNING OUTCOME 6.2.1.3" on page 428

Within these categories, it is possible to monitor the following list of parameters:

- Fracture or slip of the rock on the excavation boundary (observed visually);
- Movement along or across a single joint or fracture (either monitored by a simple mechanical 'tell-tale' or measured more accurately);
- The relative displacement or convergence of two points on the boundary of an excavation;
- Displacements occurring within the rock mass away from the excavation periphery;
- Surface displacements or subsidence;
- Changes in the inclination of a borehole along its length;
- Groundwater levels, pressures and flows;
- Changes in the normal stress at a point in the rock mass;
- Changes in loads in support elements such as steel sets, props, rock bolts, cables and concrete;
- Normal stresses and water pressures generated in fill;
- Settlements in fill;
- Seismic and micro-seismic emissions; and
- Wave propagation velocities.

Other variables of interest are determined indirectly, for example via the use of a constitutive model to derive stress from strain. The following are indirect parameters that can be monitored in underground conditions.

- Absolute stress, or changes in stress, at a point in the rock mass;
- Seismic emissions (ground velocity or acceleration);
- Wave propagation velocities.

REFERENCE

- Roc-G 034 Learning Guide.
- Mine Health and Safety Act, Act 29 of 1996
- SIMRAC Report; GAP 852
- Rock Engineering Handbook Jager and Ryder Chapter 9
- Brady and Brown 3 rd Edition Chapter 18

LEARNING OUTCOME 6.2.1.6

DESCRIBE THE ROCK RESPONSE THAT MAY BE MEASURED IN A MONITORING PROGRAMME

In most geotechnical instrumentation cases, there are only three basic responses that can be measured using existing technology. These are:

- Displacement or deformation;
- Pressure;
- Time.

PRINCIPLES OF MONITORING

LEARNING OUTCOME 6.2.2.1

DESCRIBE AND EXPLAIN THE GENERAL FEATURES OF A MONITORING SYSTEM

Monitoring programmes are necessary to provide the rock engineer with data for a variety of purposes. There is a wide range of instrumentation available to fulfil this function. The instrumentation used normally has three different components. (After Brady and Brown Chapter 18):

- A sensor or detector responds to changes in the variable being monitored.
- A transmitting system that may use rods, electrical cables, hydraulic lines or radiotelemetry devices, transmits the sensor output to the read-out location.
- A read-out and/or recording unit such as a dial gauge, pressu gauge, digital display, magnetic tape recorder or computer, which converts the data into a usable form.

Four broad categories of instrumentation can be identified, and these are listed in order of decreasing simplicity and reliability

- optical

- mechanical
- hydraulic and pneumatic
- electrical

Optical systems are often the simplest methods and can provide a rapid assessment of prevailing conditions, sometimes at relatively low cost. These systems include borehole cameras and petroscopes, and conventional and photogrammetric surveying methods for establishing excavation profiles and boundary movements and for recording natural and mining-induced fractures.

Mechanical systems are used for displacement measurements and involve the use of rods, wires, cables, and tapes to achieve this. They are often the simplest, cheapest and most reliable methods of performing the required detection, transmission and read-out of data, e.g. wire or rod type extensometers, and various types of closure meters. However, they do not lend themselves to remote reading or continuous recording, and for that reason often have to be overlooked in favour of less reliable electrical systems.

Hydraulic and pneumatic devices are diaphragm-based transducers (fluid pressure acts on the side of a flexible diaphragm made of metal, rubber or plastic), which are used for measuring support loads and normal components of stress. Read-out is usually achieved via a standard pressure gauge, but for remote or continuous reading an electrical pressure transducer can be used.

Also falling into this category are hydro-fracturing in boreholes as a particular means of carrying out stress measurement, and hydro-packers to determine the location of and magnitude of displacement in a borehole, extent and orientation of borehole breakout or spalling, and location of fractures.

Electrical devices lend themselves to remote and continuous readout, and this often is a deciding factor in their selection. However, experience has shown that instruments in this category are extremely prone to failure in the harsh underground environment. The provision of a suitable power supply underground is also sometimes problematic, though in some cases the use of battery power can alleviate this problem. Unless the remote readout feature is of paramount importance or unless no alternative form of instrument is available, it is strongly recommended that electrical-based devices are not selected without careful consideration and use of appropriately stringent preparation and protection procedures.

Some typical electrical devices are:

- Electrical resistance strain gauges (determine stress)
- Vibrating wire gauges (measure strain and determine stress changes)
- Ultrasonic reflections (measure dilation at a number of points)
- Potentiometers (measure convergence)
- Linear variable differential transformers – LVDTs (measure deformation between two points)
- Seismometers and accelerometers (detect seismic ground motions)
- Ground penetrating radar (GPR), and seismic tomography equipment.

back

In order for the monitoring system to fulfil its intended function economically and reliably, it should satisfy a number of requirements:

- Easy installation, if necessary under adverse conditions;
- Adequate sensitivity, accuracy and reproducibility of measurements;
- Robustness and suitable protection to ensure durability for the required period of operation;
- Ease of reading and immediate availability of the data to the engineer;
- Negligible mutual interference with mining operations.

REFERENCE

- Chapter 9, Ryder J.A. and Jager A.J., A textbook on rock mechanics for tabular hard rock mines, 2002
- Chapter 10, Jager A.J. and Ryder J.A., Handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 2002
- Chapter 19, Brady H.G.B. and Brown E.T., Rock mechanics for underground mining, 2006

LEARNING OUTCOME 6.2.2.2

DESCRIBE AND EXPLAIN MEASURING INSTRUMENTS IN TERMS OF:

Accuracy

The accuracy of an instrument indicates the deviation of the output, or reading from a known input. It is expressed at a percentage of the full scale reading. For instance, a pressure gauge reading up to 20MPa having an accuracy of 2% will therefore be accurate to within 400kPa on every reading given. However, a 100MPa gauge that also has a 2% accuracy will be accurate to within 2MPa on readings. Accuracy levels should always be evaluated keeping the full scale of the instrument in mind.

Error

The error is the difference between an observed or calculated value and the true value. Types of errors, causes and remedies are provided below:

back

Type of error	Causes	Remedies
Gross error	Inexperience Misreading Misrecording Computational error	Care Training Duplicate readings Dual observers Checking against previous readings
Systematic error	Improper calibration Loss of calibration Hysteresis Non-linearity	Use of correct calibration Recalibration Use of standards Use of consistent readings procedures
Conformance error	Inappropriate installation details Instrument design limitations	Select appropriate instrument Modify installation procedure Improve instrument design
Environmental error	Weather Temperature Vibration Corrosion	Record environmental changes and apply corrections Make correct choice of instrument materials Temperature and humidity control in enclosed boxes Suitable warming procedure before taking readings
Observation error	Variation between observers	Training Use of automatic data acquisition systems
Sampling error	Variability in the measured parameter Incorrect sampling techniques	Install a sufficient number of instruments at representative locations
Random error	Noise Friction Environmental effects	Correct choice of instrument Temporary elimination of noise Multiple readings Statistical analysis

Table 6: Errors, their causes and remedies (Ryder and Jager)

Sensitivity

The sensitivity of a monitoring instrument can be defined the smallest unit of measurement on the instrument.

NOTE

High sensitivity does not necessarily indicate high accuracy.

Precision

Precision of monitoring instruments is a measure of the ability of the instrument to reproduce a certain reading when a similar reading is taken. This would mean that a very small standard deviation is reported from a large number of readings, effectively indicating that all readings taken are very close to the mean value of all the readings.

REFERENCE

- Chapter 9, Ryder J.A. and Jager A.J., A textbook on rock mechanics for tabular hard rock mines, 2002
- Chapter 10, Jager A.J. and Ryder J.A., Handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 2002

LEARNING OUTCOME 6.2.2.3

DESCRIBE AND EXPLAIN HOW ONE WOULD MEASURE:

- The stress tensor in a rock mass
- The depth of fracturing in tunnel walls

- Convergence
- Tunnel wall dilation
- Stress changes in rock
- Pressure changes in backfill
- Height of caving of hangingwall

The following flow diagram indicates the steps to be taken to measure and process monitoring data (Figure 26) for any of the measurements discussed in this outcome. It forms a general approach and should be kept in mind when performing any measurements.

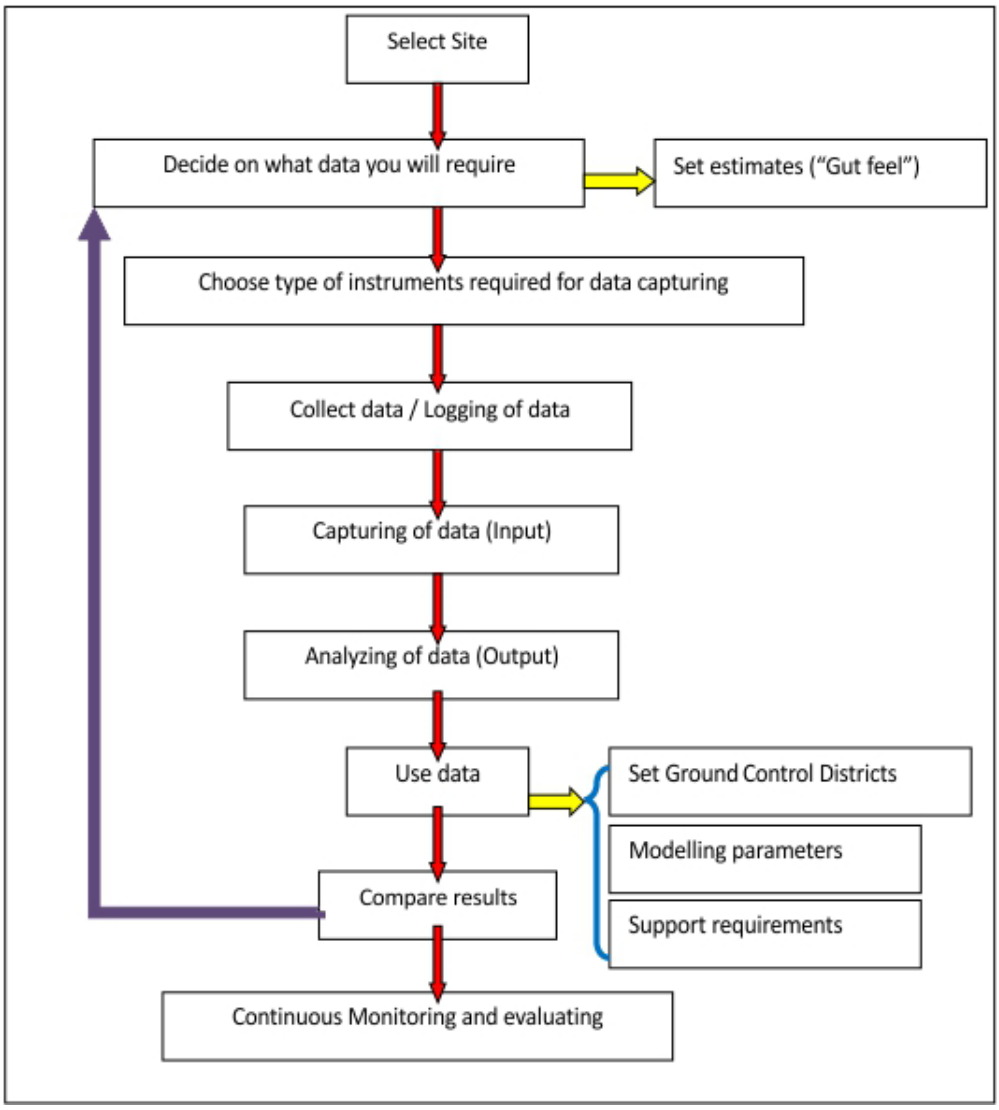


Figure 26: Flow chart: Monitoring and processing of data

In any instrumentation programme, the planned position of the instrument must be carefully considered to obtain the intended results. Persons installing the instruments must be able to read mine plans to ensure the correct location and position of the instrument. Placing an instrument in the incorrect position will defeat the objective of the instrumentation programme.

back

CONNECTION 65

Refer to PAPER 1 Learning Outcome 5.5 on page 155

The stress tensor in a rock mass

The depth of fracturing in tunnel walls

Since stress fracturing forms in the rock around a tunnel, one of the following situations must exist to allow measurement of the fracture planes:

- Blasting a cubby into the tunnel sidewall and through the fracturing to visually observe and measure the stress fractures. This could, however, cause some blast damage to the fractures and affect the measurements, but will allow accurate determination of the fracture orientations;
- Drilling a borehole into the sidewall and through the stress fractures and using a borehole camera measure the stress fracturing density visually. However, the orientations are more difficult to estimate (smaller exposure area) and will be less accurate than the previous method.

Please download and install the following Open Code video software package (free download) on your computer to be able to view this videos of a Borehole Camera.

VLC Media Player: http://vlc-media-player.en.softonic.com/

Convergence

Convergence and closure can be measured in several ways, from crude to very accurate and sensitive methods:

- **Tape or clinorule**. This is probably the simplest but most inaccurate method purely due to the inaccurate application of the method and not because of the instruments. However, in high closure / convergence areas, the inaccuracy is often of less concern and can be used successfully. In low convergence areas, this method is inadequate due to its poor accuracy in taking small measurements;
- **Closure metre**. This instrument is installed semi-permanently and can even measure the closure on a continuous basis. They are accurate and often very sensitive;
- **Four-peg closure / ride stations**
- **Extensometers.** These units can consist of a large number of different instruments, but all work similarly in that they measure rock displacement. By installing those into the hangingwall / roof (assuming that the floor does not heave), the closure / convergence can be measured.

CONNECTION 66

Refer to "LEARNING OUTCOME 6.2.3.1" on page 441

Tunnel wall dilation

Extensometers.

These units can consist of a large number of different instruments, but all work similarly in that they measure rock displacement. By installing those into the hangingwall / roof (assuming that the floor does not heave), the closure / convergence can be measured.

CONNECTION 67

Refer to PAPER 1 Learning Outcome 5.5 on page 155

Stress changes in rock

By repeating stress measurements at a particular site, the changes can be measured.

Pressure changes in backfill

Pressure changes in backfill are measured with load cells (often called flatjacks) that are installed within the backfill paddock, the paddock filled

back

back

and the pressure changes read on an instrument situated alongside the paddock (Figure 27).

Figure 27: Backfill stress metres (Ryder and Jager)

Height of caving of hangingwall

Since the caving of hangingwall suggests a collapse into the working area, access to the area is normally prohibited. Methods used to assess this must then be able to operate at a distance. It has been known that borehole ground penetrating radar (GPR) equipment has been used successfully in such conditions. In areas where the hangingwall has disintegrated but is kept reasonably in place by areal coverage support systems, installed extensometers could indicate the maximum extent of the collapse. Remote reading on electronic extensometers is available. However, this would have required the installation of the extensometers prior to the collapse.

back

back

6.2.3 MONITORING INSTRUMENTS

LEARNING OUTCOME 6.2.3.1

DESCRIBE THE FOLLOWING INSTRUMENTS AND EXPLAIN THEIR PURPOSE, INSTALLATION AND OPERATION

Hydraulic pressure cells:
Hydraulic load cells are load cells filled with hydraulic fluid mainly used to determine the in-situ load generated by support units.

Load cells:
(Figure 28). These are used to determine the in-situ load generated by support units. The support units that can be monitored include elongates, packs, tendons, cable anchors and backfill.

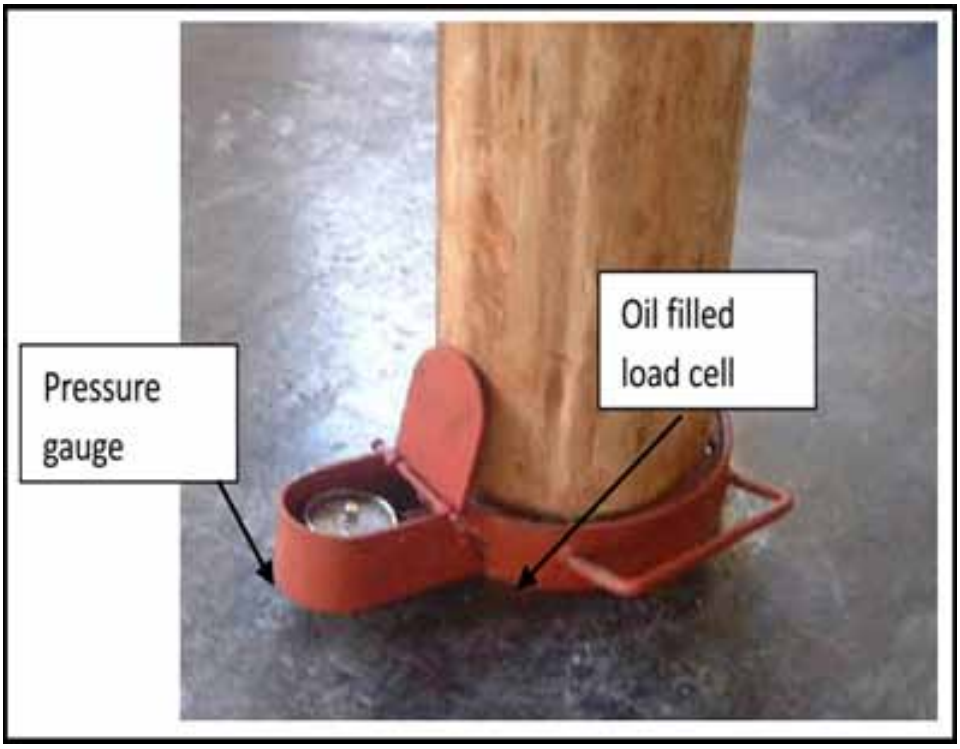


Figure 28: Typical load cell used on elongates

Elongate load cells:(Figure 29)
A variety of load cell configurations are available, which vary in method of readout (strain gauge or pressure gauge) and shape or size, depending on the type of stick or elongate to be monitored and the readout requirements. In all cases, the load cells have to be calibrated in a laboratory press before use, and the resulting calibration curve is used to convert readings into true loads.

Use of these load cells is quite straightforward. The cell is placed on a level area of footwall and an elongate unit, cut a little shorter in length than would normally be required, and is installed on top of the cell. It is common practice to also install some form of closure measurement device close to the support unit being instrumented to determine the amount of displacement corresponding to the measured load. In this way, it is possible to obtain the in-situ load-deformation behaviour of support units. It is important to select gauges with sufficient load ranges to comfortably accommodate the anticipated loads.

back

Typical load ranges should be in the region of the following (keep in mind that some exceptions may exist):

Elongate	Packs	Cable anchors
0-500kN	0-3MN	0-400kN
Backfill	Tendons	
0-3MPa	0-250kN	

Figure 29: Elongate load cell in operation (note the closure meter to the right of the elongate support unit)

Pack load cells:(Figure 30 and Figure 31)

Packs of all designs can be instrumented by means of appropriate combinations and sizes of fluid-filled load cells. Readout is usually via a pressure gauge. Load cells are built into the pack as a single layer during its normal construction procedure, except that it is usually advisable to sandwich the cells between two thick steel plates for protection from punching and other damage. Closure is again usually measured in order for the in-situ load-deformation behaviour to be obtained.

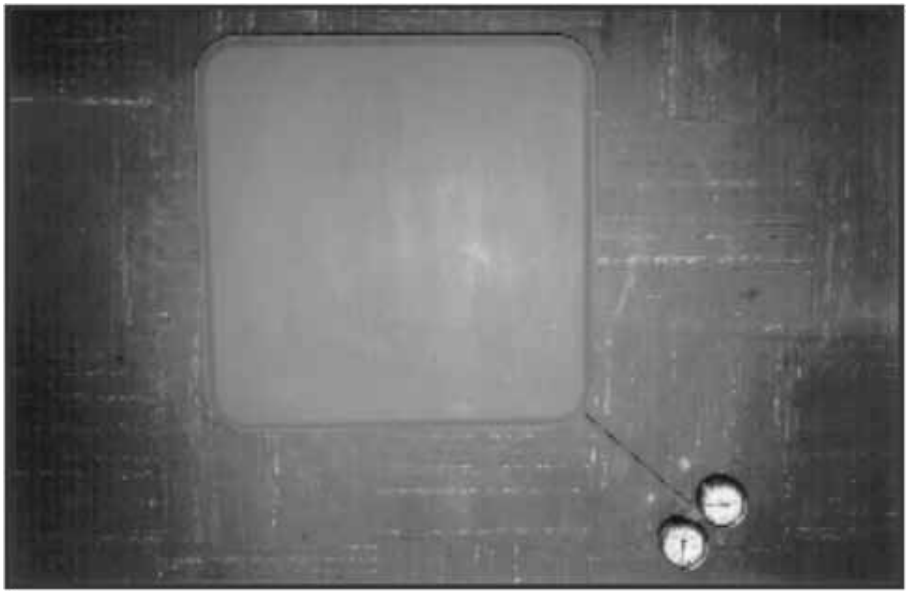


Figure 30: Pack load cell, square outline (0.5 m x 0.5 m), plan view

The attached readout gauges need to be carefully positioned so as to be afforded the best possible level of protection from blasting and other mining activities.

Figure 31: Pack instrumented with load cells (note the protective steel plates)

Tendon load cell: (Figure 32).

A small annular load cell, usually using strain gauges as the sensor, can be slipped over the end of a roof bolt between the collar plate and tensioning nut. As load is transferred from the bolt to the rock surface via the plate, it is recorded in the load cell.

Figure 32: Tendon load cell (Strain gauges)

Tapes:

A measurement tape is a basic tool that is usually used to measure distances, such as spacing of support units, where accuracy to 1mm is more than sufficient. Tapes are available in many lengths from approximately 5m to 100m plus.

Tape extensometer: (Figure 33).

Tape extensometers are used to monitor displacements in the rock mass. Despite its name, this steel tape and vernier-based device is mainly used for measuring the convergence of tunnel walls. It is particularly suitable for making measurements of small movements between reference points on the boundary of excavations with larger dimensions or over longer distances (0.5 m to 30 m) at an accuracy of up to 0.1mm.

Figure 33: Tape extensometer

back

Doorstopper stress cell: (CSIR doorstopper strain cell).
This device consists of a set of electrical strain gauges (often 'four-rosette' with 0°, 45°, 90° and 135° orientations) mounted on the end of a 'doorstopper' shaped plastic holder. This cell is glued to the end of a specially prepared BX borehole and enables over-coring.

Readings are obtained via a special down-the-hole installation tool coupled, by means of a robust cable, to an external strain readout unit. The installation tool also includes a dummy strain gauge glued to a representative disc of rock, where this is required to balance the strain measuring system in the readout unit to compensate for temperature changes.

The 2-D nature of the measurements requires that readings must be taken in three approximately orthogonal holes to obtain a complete stress tensor. Moreover, in order to address the intrinsic relative unreliability of stress determinations, it is common practice to take more than one set of readings from a given borehole by means of successive steps of re-installation and further over-coring.

The installation procedure involves:

- Drilling a clean BX-size hole, preferably slightly up-dip to facilitate water drainage, to the desired measurement depth;
- The end of the hole is then flattened and polished with special drilling crowns, to create a smooth, clean, perpendicular surface;
- The strain cell can subsequently be glued to this surface.
  - The cell is inserted into the front end of the installation/reading tool and a layer of special two-component resin glue applied to the strain rosette surface;
  - The tool, with attached cable, is then inserted and pushed down the hole using a series of inter-linking rods;
  - Just prior to reaching the end of the hole, the tool is carefully orientated, with the assistance of a built-in sensor, and eventually the strain cell is pushed and glued onto the end of the hole;
  - By wedging the rods in the hole, the assembly is kept perfectly still and pressure maintained on the doorstopper until the glue has set;
- After giving the glue time to set, a series of base readings is taken on each of the strain gauges;
- Once a stable set of base readings has been obtained, the installation tool is removed from the hole, leaving the strain cell attached to the end of the hole;
- Figure 34 shows the steps to be taken when over coring the doorstopper;
  - By means of very careful drilling to extend the BX hole, the doorstopper is over-cored to a depth of at least 300mm;
  - and removed while still attached to the deliberately snapped off length of core;
- The doorstopper, with attached piece of core, is again plugged into the installation tool and a further set of readings is taken after the strains have stabilised;
- The difference in micro-strains between the two sets of readings provides the strain relief values required for the numerical conversion into 2-D stress values.

A number of additional options are available using the doorstopper strain cell. For instance, by obtaining three sets of strain relief readings, each obtained from one of three orthogonal (X, Y and Z directions) boreholes,

back

it is possible to calculate a full 3-D stress tensor. Also, 2-D stress changes can be monitored using a doorstopper equipped with a permanent cable connection, but the reliability of these can be compromised by creep effects.

Laboratory tests of the host rock are required to obtain the elastic constants used in the stress calculations.

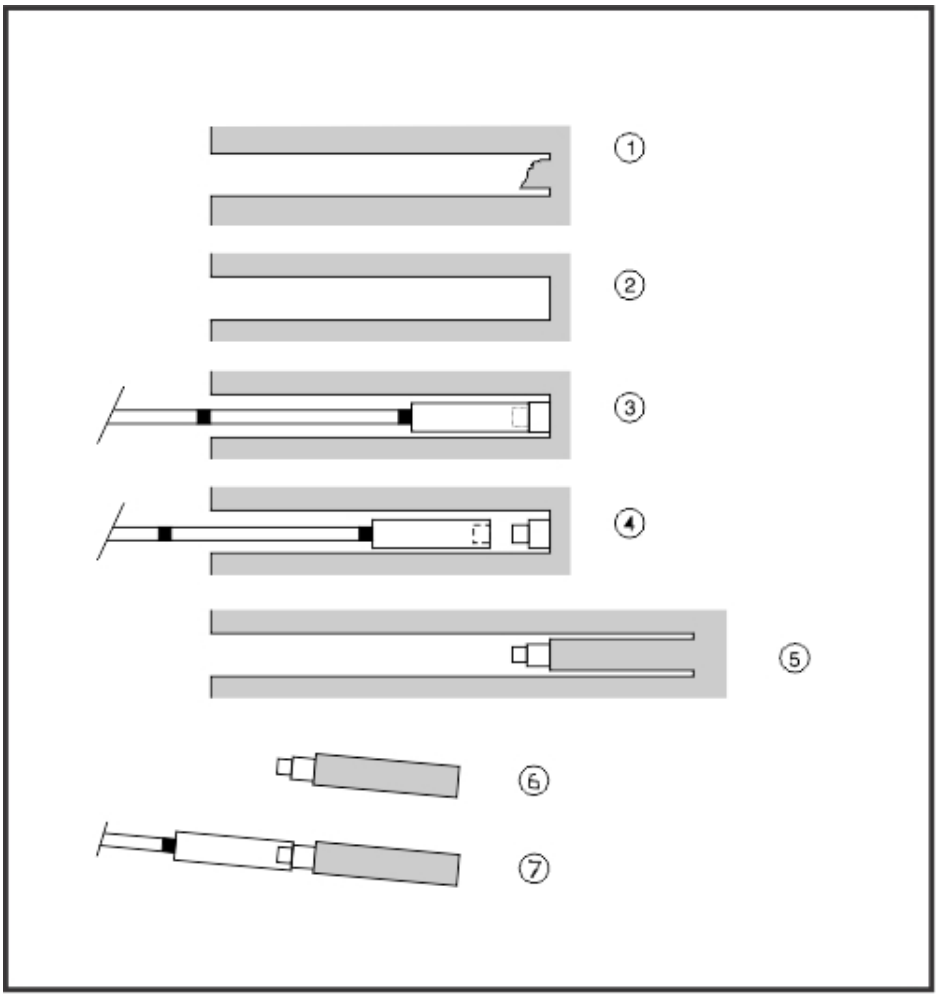


Figure 34: Over-coring of a doorstopper

CSIR tri-axial strain cell: (Figure 35)

This strain cell is effectively the 3-D equivalent of the doorstopper strain cell and requires a similar installation and calculation process, albeit somewhat more complicated. The cell has three four-rosette (0°, 45°, 90° and 135°) strain gauges arranged equally on its circumference.

The tri-axial nature of the cell means that, in principle, measurements in one hole only are required to obtain the full 3-D stress tensor. A dummy gauge is built into the cell and requires a piece of representative rock to be glued to it. A more complicated installation tool and a supply of compressed air are also required. With modification, this cell can be used for long-term monitoring of stress changes.

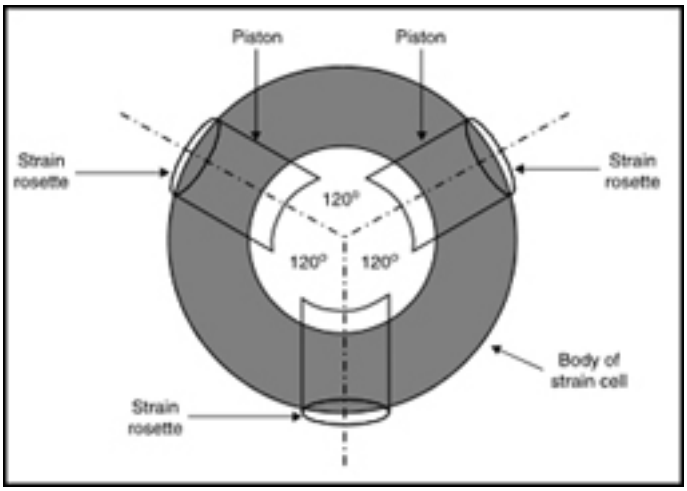


Figure 35: CSIR tri-axial strain cell

The installation procedure (Figure 36) involves:

- Drilling of an NXCU size hole to the desired measurement depth;
- The end of this hole is then flattened and extended with an EX size bit for a further 450mm;
- The cell is plugged into the front of the special installation tool and glue is applied to the three strain gauge rosettes;
- The whole assembly is pushed down the hole using connecting rods until, after careful orientation, the cell is positioned inside the EX stub hole;
- The pistons supporting the strain gauge rosettes are activated using a supply of compressed air, and in this way the gauges are pushed and glued to the inner surface of the EX borehole;
- Initial readings are taken after allowing the glue to harden;
- The installation tool is then removed and the strain cell over-cored for about 300mm with an NXCU sized bit;
- On removal of the hollow core with the strain cell still glued inside, a second set of strain readings is taken to provide the strain relief values needed for the numerical determination of the stress tensor.

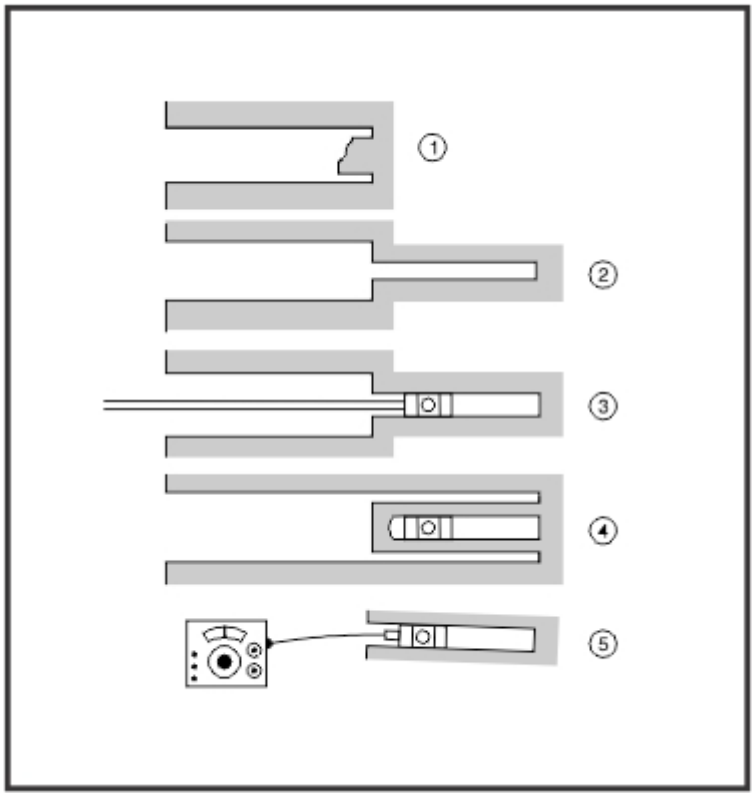


Figure 36: Installation procedure for the CSIR tri-axial strain cell

CSIR solid inclusion strain cell:

This instrument works on the same principle as the CSIR doorstopper 2-D strain cell, except that the strain gauge rosettes are contained within a machined aluminium cylinder and the whole device is grouted into a borehole at the desired position. These devices are generally used for the monitoring of stress change, such as in or under support pillars. Under high stress conditions, the solidity of the cell also better maintains the integrity, and linear elasticity, of the borehole sidewalls.

CSIRO tri-axial hollow inclusion stress cell:

This device is very similar in installation and function to the CSIR tri-axial cell and also provides means for calculating a full 3-D stress tensor from a single borehole. The three strain gauge rosettes (0°, 45° and 90°) are arranged in the outside skin of a hollow epoxy cell that is bonded to the inner surface of the borehole with resin. The drilling requirements and sequence are the same as for the CSIR tri-axial strain cell except that special provision must be made for the permanently-attached data cable to pass through the drill machine mechanism during over-coring. This permits valuable diagnostic readings to be taken during the over-coring process. This cell is also suitable, without modification, for the long-term monitoring of stress changes.

Installation (Figure 37) involves:

- Diamond drilling of an NXCU size hole to the required depth;
- Flattening of the hole end, and extending an EX size hole to accommodate the required measuring depth;
- The depth of the EX portion of the hole must be accurately known and a wooden plunger stick cut to length accordingly. The assembled unit is moved down the borehole and inserted into the EX portion of the hole by means of a trolley device;
- As the cell is pushed into the EX hole, the wooden stick is forced against the end of the hole causing the attached plunger inside the cell to push out the bonding resin;

back

- After the resin has had time to set, the strain readings are taken;
- With the appropriate modification of the drilling machine to accommodate the data cable, readings can continue to be taken during over-coring;
- Final readings are taken on removal of the over-cored cell for numerical determination of the 3-D stress tensor.

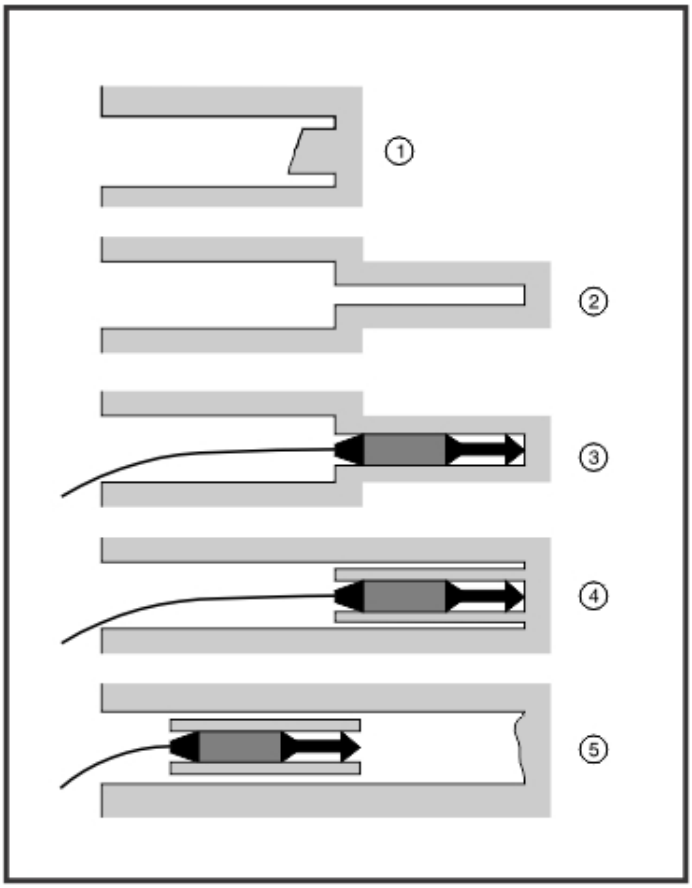


Figure 37: CSIRO tri-axial hollow inclusion stress cell installation

Closure meters

Closure meters, sometimes also called convergence meters, are probably the most frequently installed rock mechanics monitoring devices in the South African hard rock mining industry. This is due both to the importance of convergence in mining conditions and the relative simplicity of the devices. Closure meters are used to determine the convergence of the boundaries of underground excavations. This is usually achieved by determining the amount by which two opposite walls of an excavation (i.e. hangingwall vs. footwall or left sidewall vs. right sidewall) move towards each other.

Spring vernier closure meter: (Figure 38)

This device consists of a spring vernier and a suitable measured length of wire trace, and is used to measure relative displacements between bolts installed in the skin of excavations. It is used to measure convergence over short distances, with its maximum operational length being limited to 4m due to the soft nature of the spring in the meter. The accuracy of this device is not particularly high at 1.0mm, but it is adequate for most tasks. Frequency of measurement depends on closure rate, but typically readings are taken daily or at least every second day.

This is a low cost device, is reasonably robust, and is easy to use and carry. Only one instrument is required per monitoring site, together

back

back

with appropriate lengths of wire trace or chain, as it is carried from one measuring location to the next and is hooked onto the reference points (bolts or pegs) only for the duration of each measurement. There are therefore only the reference bolts left in place to survive the rigors of the underground environment.

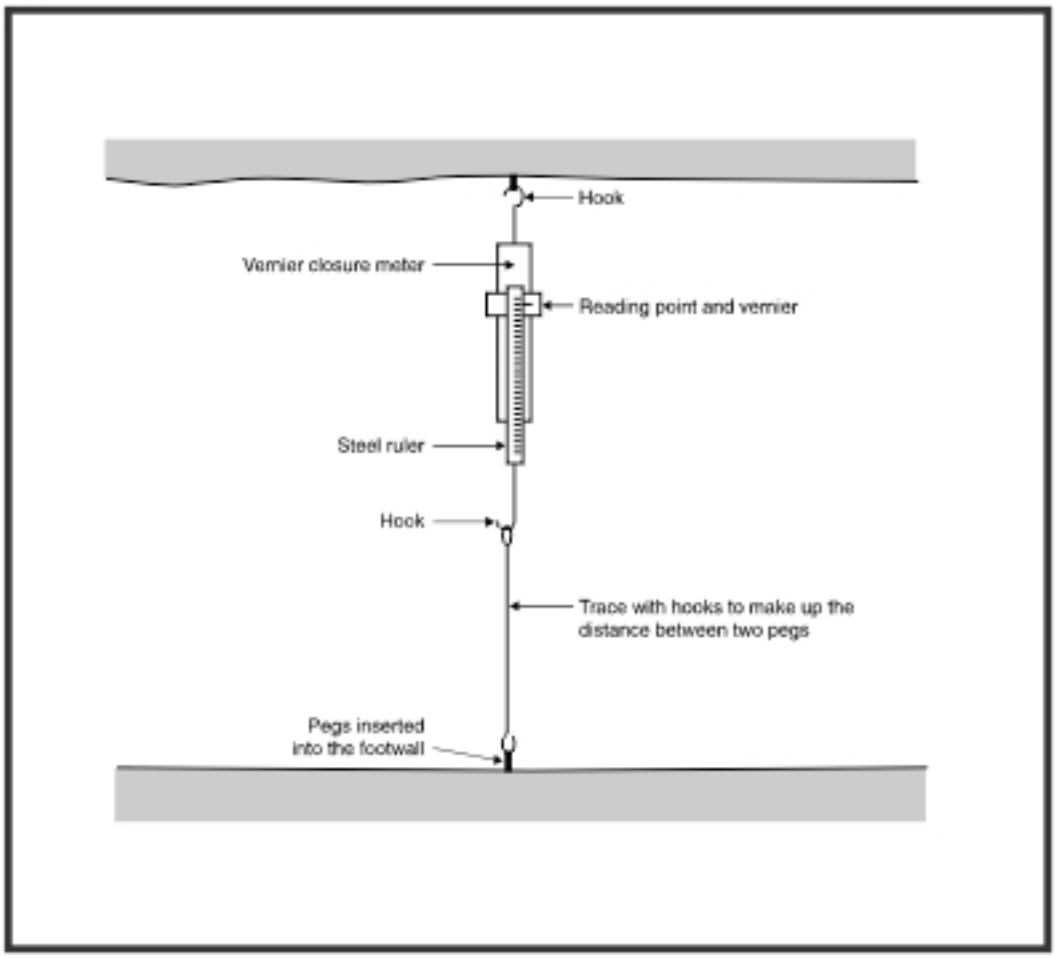


Figure 38: Vernier closure meter

The main application for a four-peg closure/ride station (Figure 39) is in the narrow tabular stopes of the gold and platinum mines. This form of monitoring involves the installation and use of three hangingwall pegs and one footwall peg. Measurements are taken with a spring vernier between each of the hangingwall pegs and the one footwall peg can be converted, through an easily computerised calculation process, into closure and ride components. This calculation process also requires initial measurements to be taken of the distances between hangingwall pegs and a base set of hangingwall-footwall spring vernier readings.

back

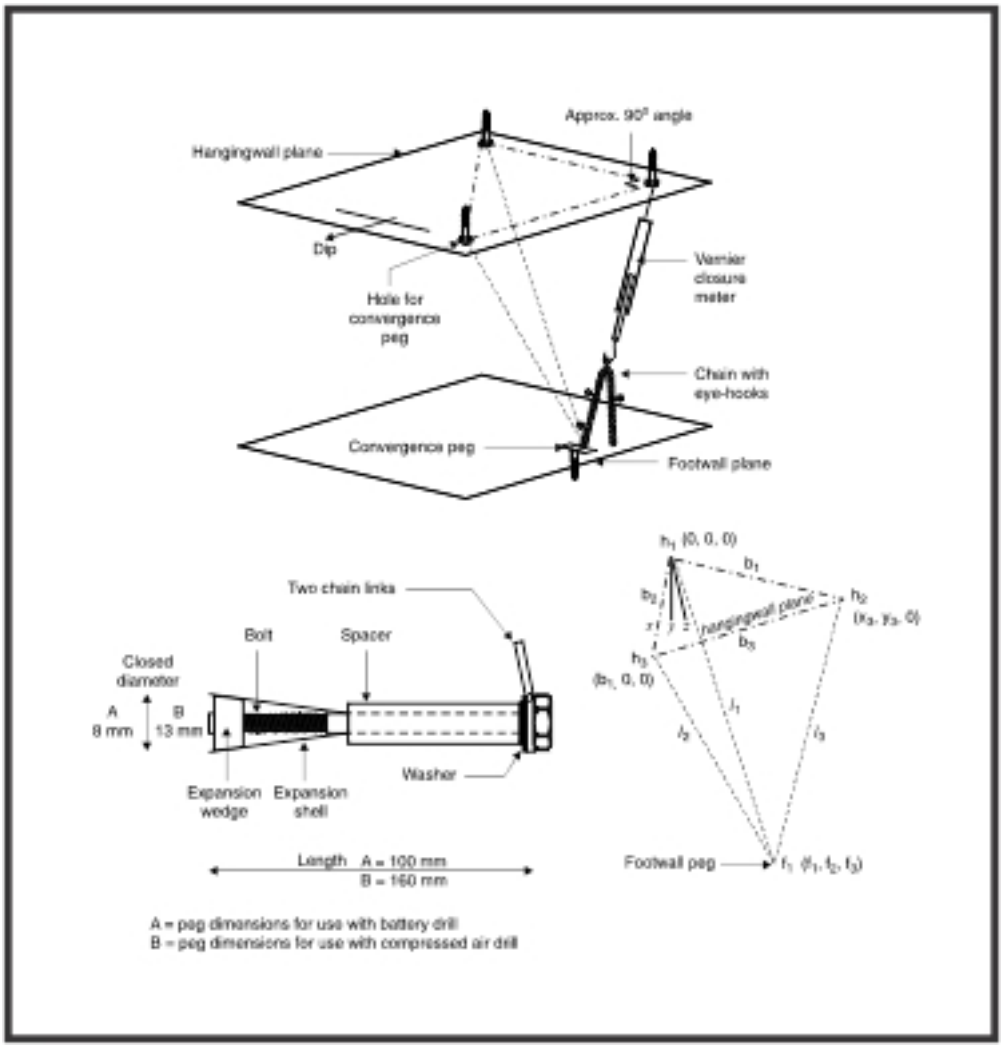


Figure 39: Schematic of four –peg closure ride station using a spring vernier closure meter (after Piper & Gürtunca 1987)

Figure 40 shows the typical set-up and Figure 41 shows the components used with this instrument.

Installation involves:

- To ensure a successful monitoring session, the selection of a suitable installation site is very important. The site must be well protected so that pegs are not lost due to normal work occurring in the stope;
- Mark out the position of the hanging wall pegs. The side A of the resultant triangle should be parallel with the face (Figure 40). The larger the triangle the more accurate the results will be (at least half of the stoping width);
- Mark the footwall hole using the "drop method" (i.e. a stone should be dropped from the hangingwall peg and the point of impact noted for the footwall peg);
- Drill holes for the pegs;
- Securely fit pegs;
- Take base readings. Measure sides 1-2, 1-3, 2-3, i.e. distances A, B and C;
- Use a spring vernier or tape to take the first set of readings from the footwall peg to each of the hanging wall pegs.

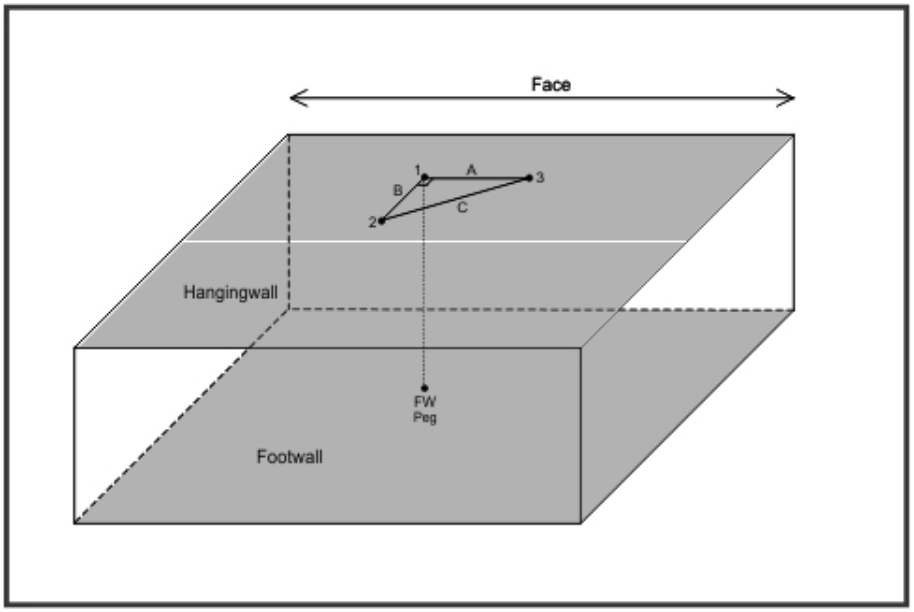


Figure 40: Diagrammatic representation of 4 peg closure ride station installation

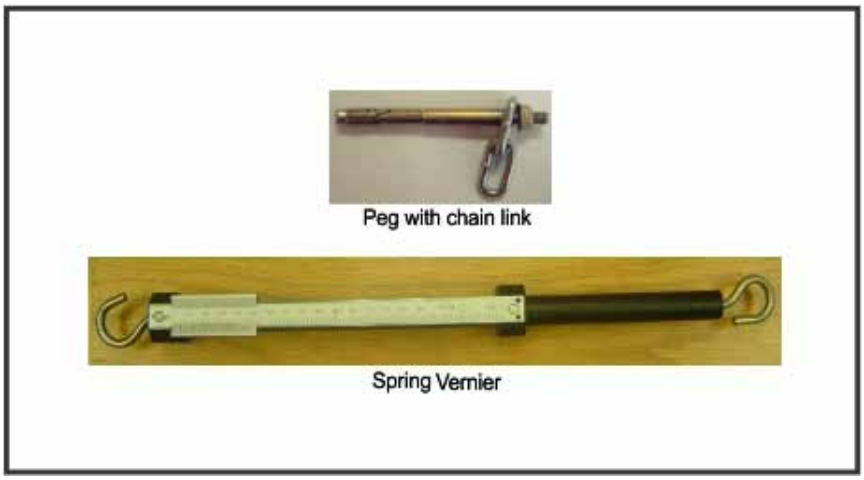


Figure 41: Component parts of the four-peg closure station

Potentiometer closure meter: (Figure 42).

Movement between two reference points, usually on the stope hangingwall and footwall, is indicated by the change in electrical output (resistance) of a spring-loaded potentiometer. This device is effectively the continuous or remote readout equivalent of the spring vernier closure meter. Direct readings of resistance are best taken using a digital multi-meter and are converted into displacements using appropriate calibration curves or equations. Alternatively, a data logger with suitable interface card can take continuous readings.

NOTE

By installing these devices between the footwall peg and each of the hangingwall pegs of a four-peg closure/ride station, it is possible to create a continuous or remote readout closure/ride facility. An alternative implementation of this concept, which provides for greater robustness and ease of installation, incorporates a potentiometer with a spring loaded telescopic steel tube. Although capable of giving continuous readout, this device is only suitable for single instrument installations, and therefore provides only a single closure component.

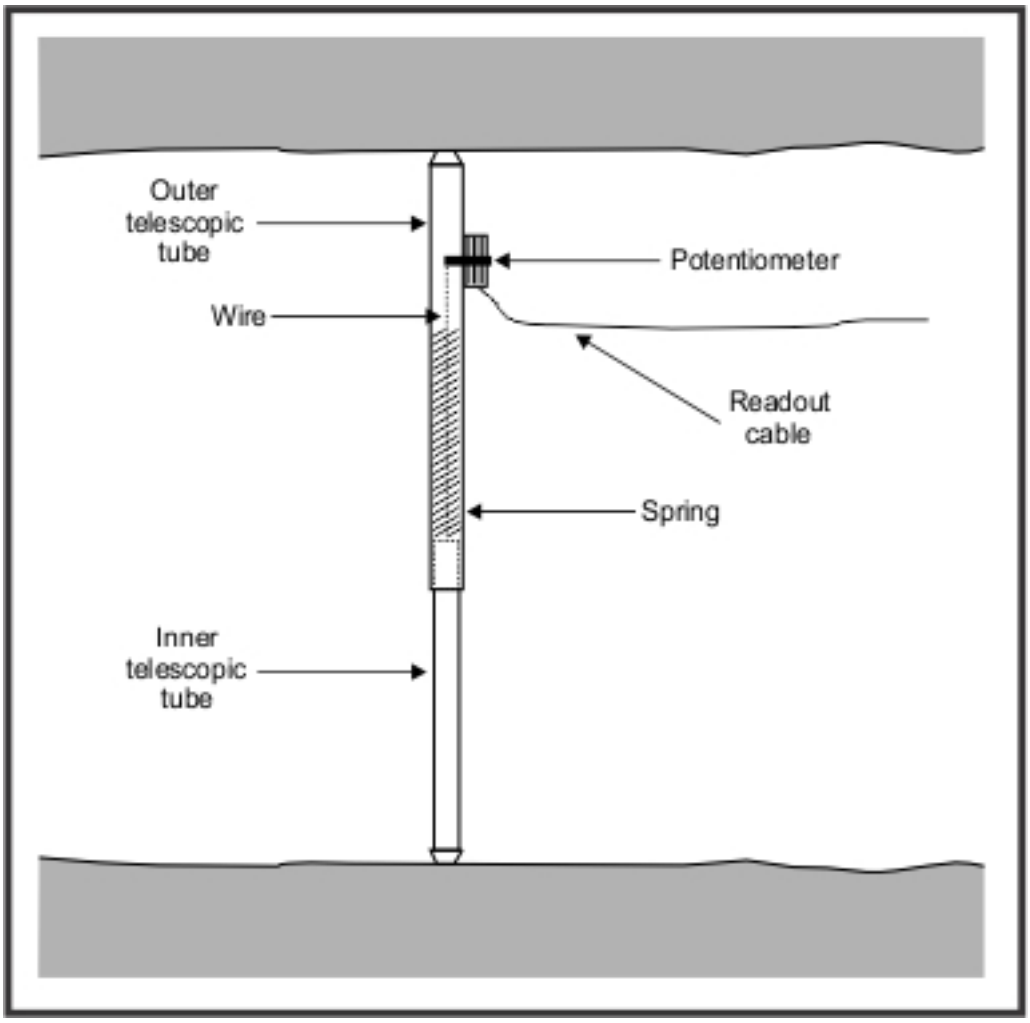


Figure 42: Potentiometer closure meter

Clockwork closure meter: (Figure 43 and Figure 44)

This device provides another way of undertaking continuous closure measurements between hangingwall and footwall in narrow stopes. A seven-day clockwork mechanism rotates a drum that holds a recording paper against an inscribing pointer or needle. The pointer is itself moved vertically about a pivot point, by the relative sliding movement of two telescopic tubes pushed together as a result of the convergence of stope hangingwall and footwall. The rotational movement of the pointer produces an arc-shaped response that must be compensated for when interpreting the results. A variation in the design is to replace the clockwork mechanism with a battery powered slow speed electric motor. However, this has the usual disadvantages of electrical devices.

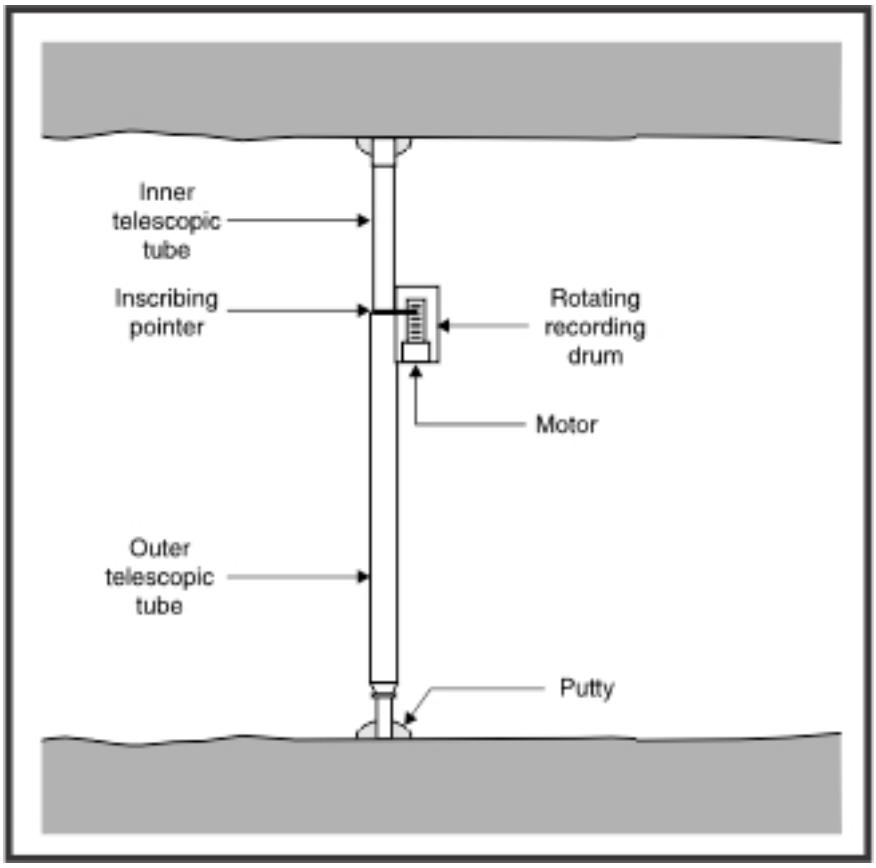


Figure 43: Clockwork closure meter

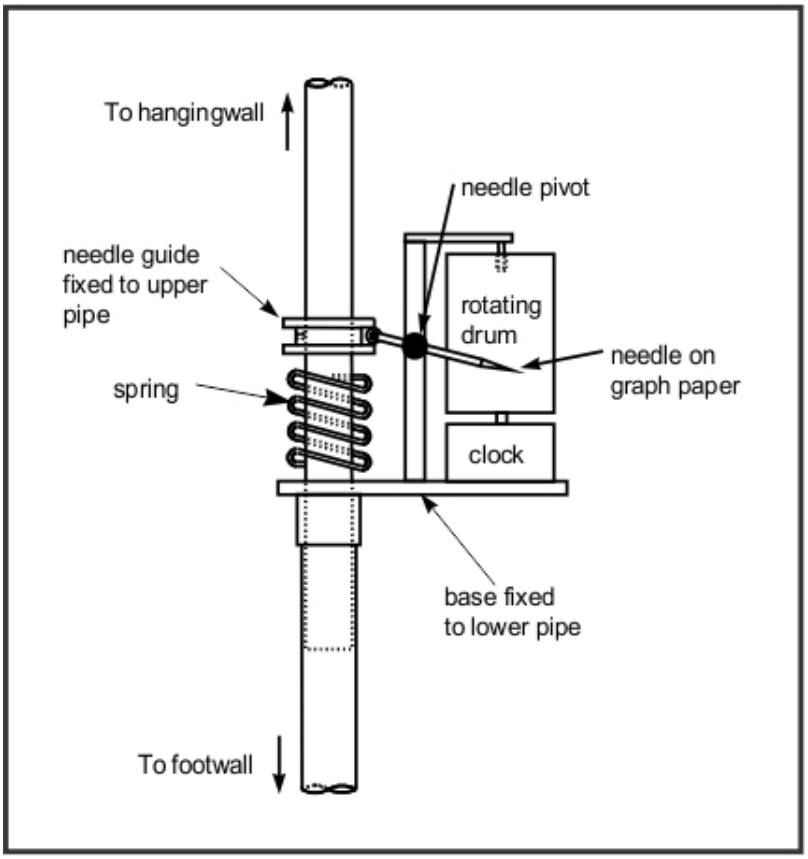


Figure 44: Detailed schematic of clockwork recording drum and inscribing pointer (Malan 1998)

back

Backfill closure meter: (Figure 45).

This is a specialised telescopic tubular device designed to withstand being embedded in a deforming, abrasive, wet medium. Its purpose is to record stope closure inside backfill, more particularly in conjunction with backfill tri-axial stress stations. In backfill monitoring, there is the need to make measurements at an inaccessible position inside the backfill, and yet permit read-out of the instrument from an outside position, e.g. in an adjacent gully. In a later design of this device, a wire attached to the base of the smaller inner tube is pulled into the larger outer tube as the inner tube itself is pushed into the outer tube by closure of the stope. This wire is guided and protected through the deforming backfill by a double-block cable (similar to a bicycle brake cable) to reach an accessible point outside the backfill where the protruding length of wire is measured. A reduction in stoping width is denoted by a shortening in the length of the exposed wire.

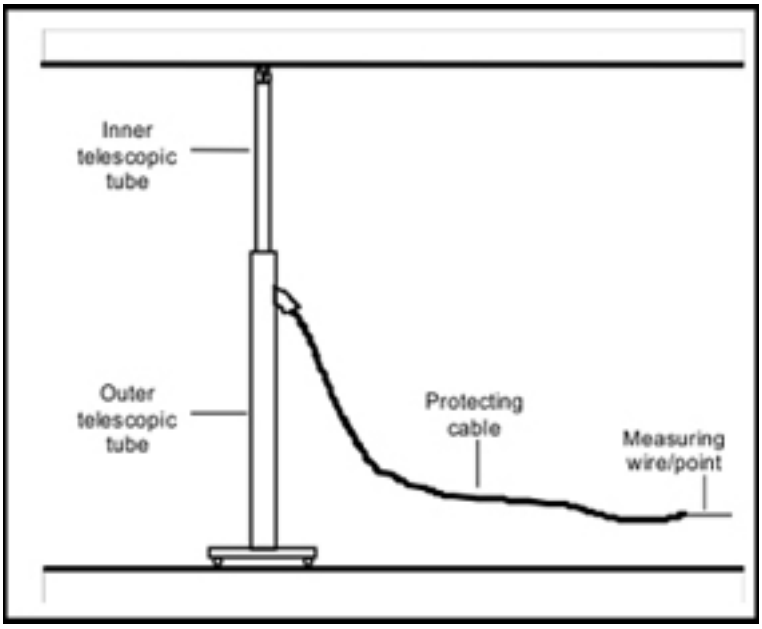


Figure 45: Backfill closure meter

back

back

Piezometer:

A piezometer is an electronic instrument used to measure the depth of water in boreholes to determine ground water levels. It is extensively used for water table measurements in boreholes around open pits. In Figure 48 below, the installation of such a piezometer is shown. Sometimes a portable piezometer is used to monitor water levels in various boreholes. In this case, the piezometer is lowered with a tape fastened to it into a borehole and an audible sound is transmitted when the piezometer touches water. The tape reading is then taken and recorded.

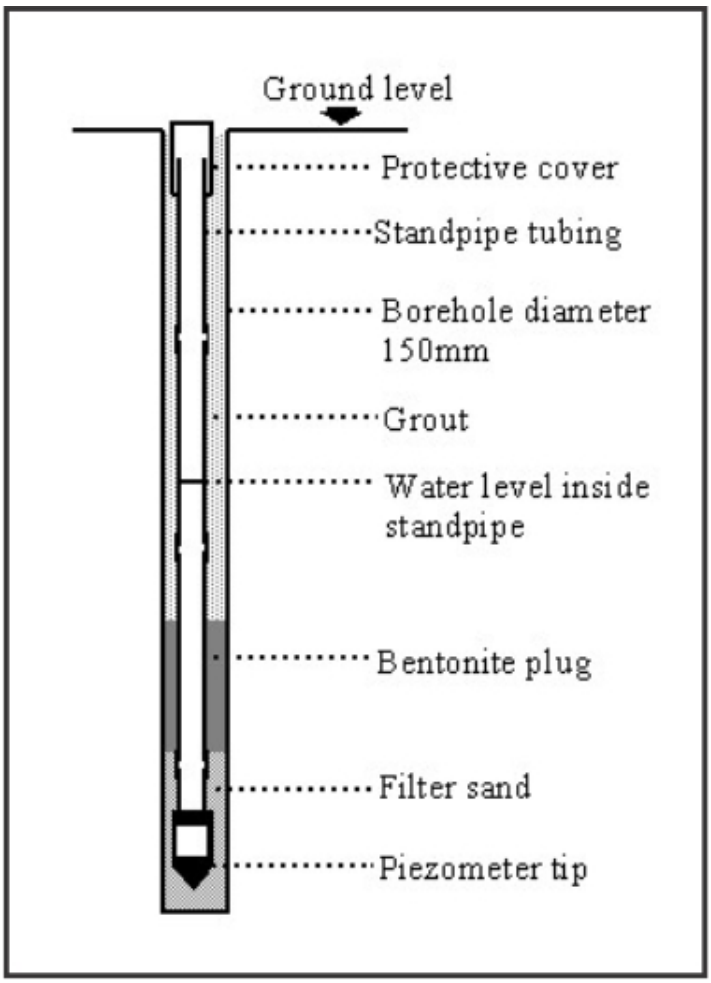


Figure 46: Piezometer installed into a surface borehole

back

back

Inclinometer:

An inclinometer is an instrument used to measure angles of slopes and the inclination of an object with respect of gravity by creating an artificial horizontal plane. In rock engineering, tilt meters, which are a type of inclinometer, are sometimes used. A tilt meter (Figure 49) is an instrument designed to measure very small changes from the horizontal level, either on ground or in a structure. Monitoring applications include the response of dams to filling, the small ground movements before landslides and the response to various influences of loading and settlement.

Figure 47: Tilt meter

Modern electronic tilt meters use a simple bubble-level principle: An arrangement of electrodes senses the exact position of the bubble in the electrolytic solution. Any small changes in the level are recorded using a standard data logger.

Rock Engineers also use clinorules (Figure 48) to determine the dip and strike of surfaces as well as to measure angles. A clinorule is a simple device consisting of two arms connected at one end by a pivot hinge. There is an angle measure ranging from 0 to 180 degrees to record the angle formed by the two hinged arms. One of the arms is fitted with a bubble level along its edge. The arms can also be used as a ruler since it has ruler markings. In order to measure the dip of a discontinuity surface, hold one arm in a horizontal position such that the other arm moves in a vertical plane about the hinge. Move the second arm until it is parallel with the dip of the discontinuity surface. Read off the angle as the dip.

back

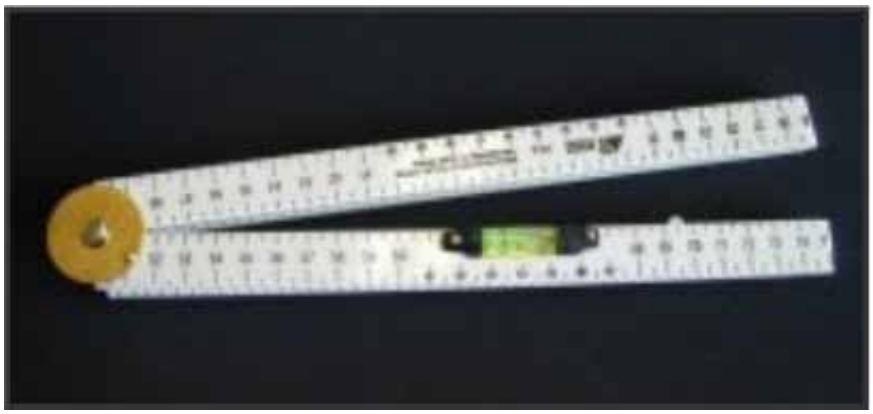


Figure 48: Clinorule

Figure 49: Inclinometers from a product catalogue (Ricker instruments)

back

Wire extensometer:

Figure 50 below shows an example of a simple wire extensometer, which consists of a steel clip used to anchor the extensometer into a hole drilled for monitoring purposes. The steel cable is attached to the anchor and a short piece of tape is inserted into the hole up to a point where just the tape is sticking out of the hole. After installation, mark a reference point on the edge of the hole and take an initial reading on the tape. Subsequent measurements, using the same marking on the edge of the hole, are compared to this initial measurement, in order to determine the amount of displacement that has occurred.

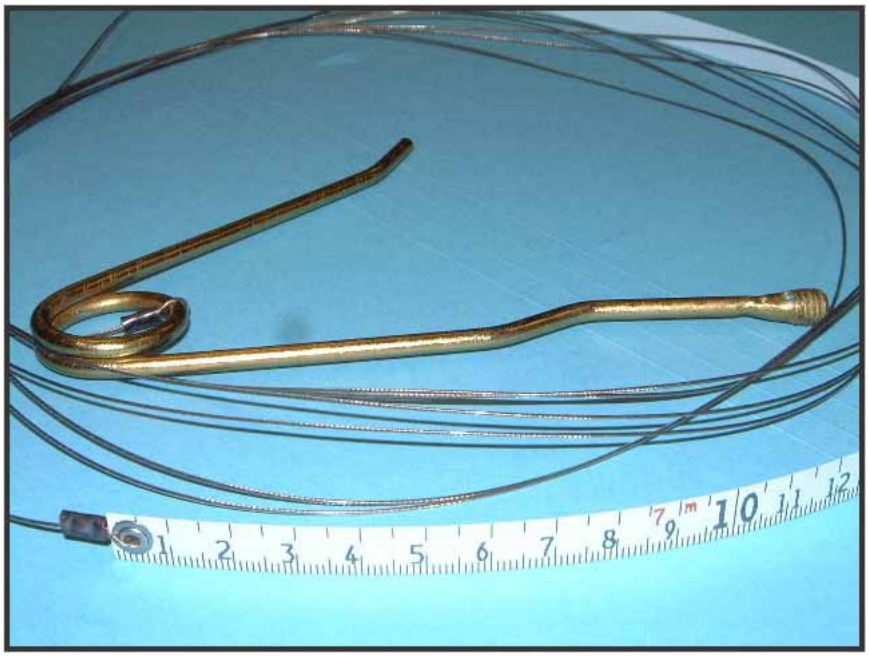


Figure 50: Wire extensometer

back

back

Multiple Point Borehole Extensometers:

In some situations, extensometers are installed in different locations in the same hole to determine differential movements in layered rocks (Figure 34). There are different multiple point borehole extensometers used in the industry, as discussed in the sections below.

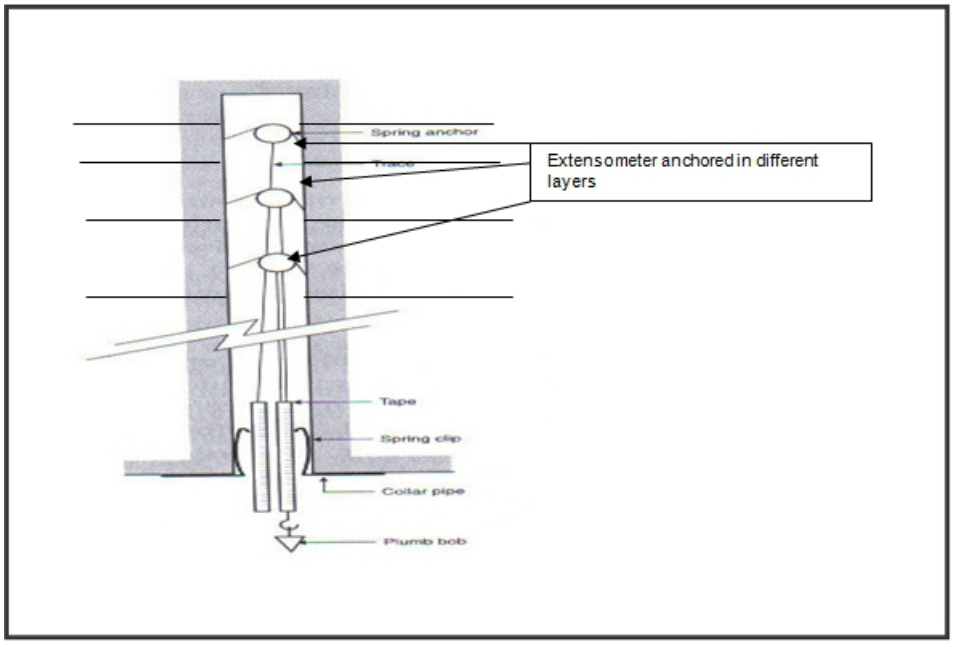


Figure 51: Multiple extensometers installed in the same hole

Wire extensometer: (Figure 52)

The simplest and cheapest of the borehole extensometers, this device consists of small spring anchors that, with a suitable length of wire trace attached, are pushed into a borehole to predetermined depths. The spring anchors have expanded diameters that are greater than the diameter of the borehole and in this way can be jammed in the borehole, thereby anchoring the attached trace wire at the desired position. Each wire trace is cut to a length such that a short (e.g. 300 mm) length of measuring tape attached to the free end will just protrude from the collar of the borehole. Readings made on these tapes against a reference point at the collar of the borehole (usually on a collar pipe) are used to determine the axial deformation along the borehole relative to the deepest anchor position. A tensioning system is required at reading time to ensure consistency of readings; this is usually some form of plumb bob that is hooked onto the end of each tape in turn. Remote and continuous readout can be achieved by attaching a potentiometer to each wire and using a data logger.

The number of measuring points depends on the size of hole. Typically, a BX size hole can accommodate up to six anchors with accompanying wire trace, and an NX size hole up to ten. Although the accuracy of this system is limited to only ±1mm, this is sufficient in many cases and its simplicity and cheapness makes it a popular choice

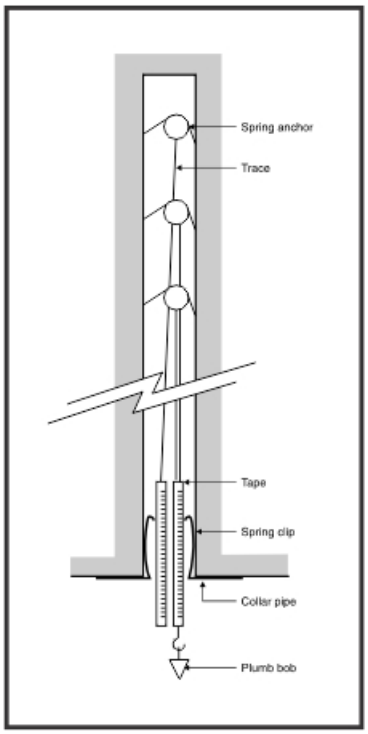


Figure 52: Wire extensometer installed in a borehole

back

Rod extensometer: (Figure 53).

A rod extensometer is a system of grouted reference points in a borehole that are connected to a measuring head at the collar of the borehole by means of rods encased in PVC tubes. A dial gauge or an electronic movement detector detects movement of the rods at the borehole collar. This system is more robust and more amenable to remote and/or continuous readout than the wire extensometer. This, together with its higher cost and somewhat more involved installation procedures, advocates its use for long-term monitoring projects, generally for critical excavations where these features are warranted. The interpretation of results from this device follows along the same lines as for the wire extensometer. Accuracy is, however, much improved over the wire extensometer at a possible ±0.1mm.

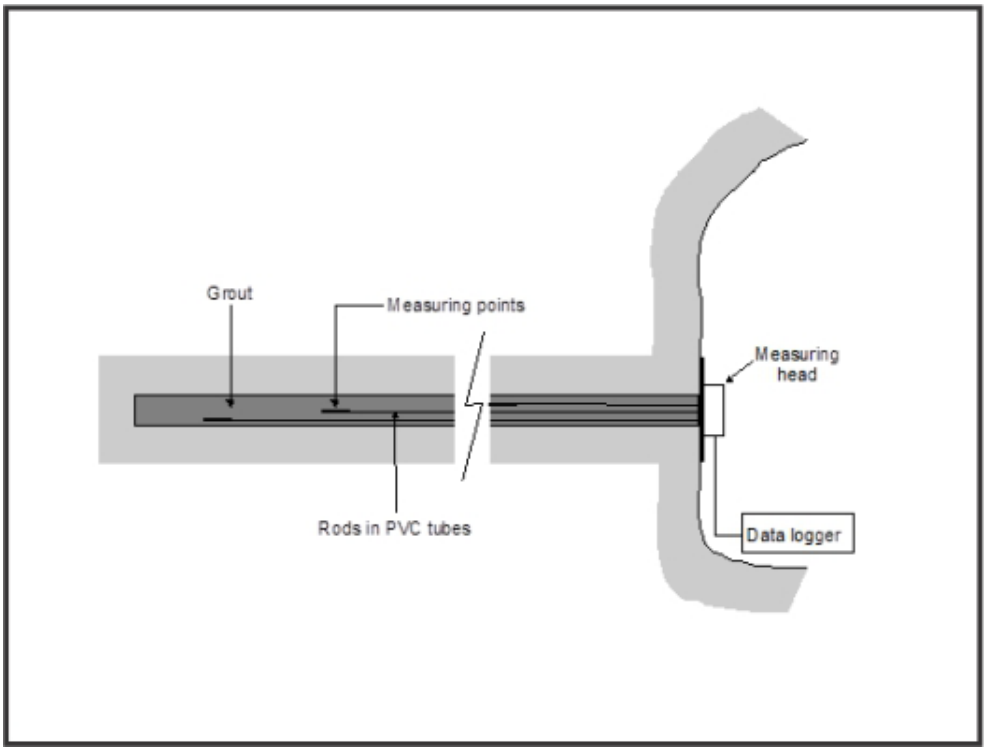


Figure 53: Rod Extensometer

back

Sonic probe extensometer: (Figure 54).

Sonic probe extensometers are another variation on the concept of measuring points positioned at predetermined depths along a borehole. In this case, the spring clip anchored reference points contain magnets that 'reflect' ultrasonic pulses back down a flexible magneto-strictive probe to a portable readout unit. The distances between reference points are determined by measuring the relative differences in arrival times of the reflected pulses.

While detailed and accurate (±0.2mm) measurements are possible with up to 21 reference points accommodated in one hole, the depth of readings is limited to 7m due to the length of the probe. Other than the expendable magnetic reference points and central access tube, no other items remain permanently installed. Therefore, since the probe only needs to be inserted in the hole at the time of reading, this system is suitable for situations where the hole and contents could be damaged between readings, e.g. into and ahead of advancing faces. The probe and the readout unit are relatively expensive items, but since they are not tied to any one place, they can be used to take readings at a number of installations. A skilled electrical technician is, however, required for regular maintenance of the equipment.

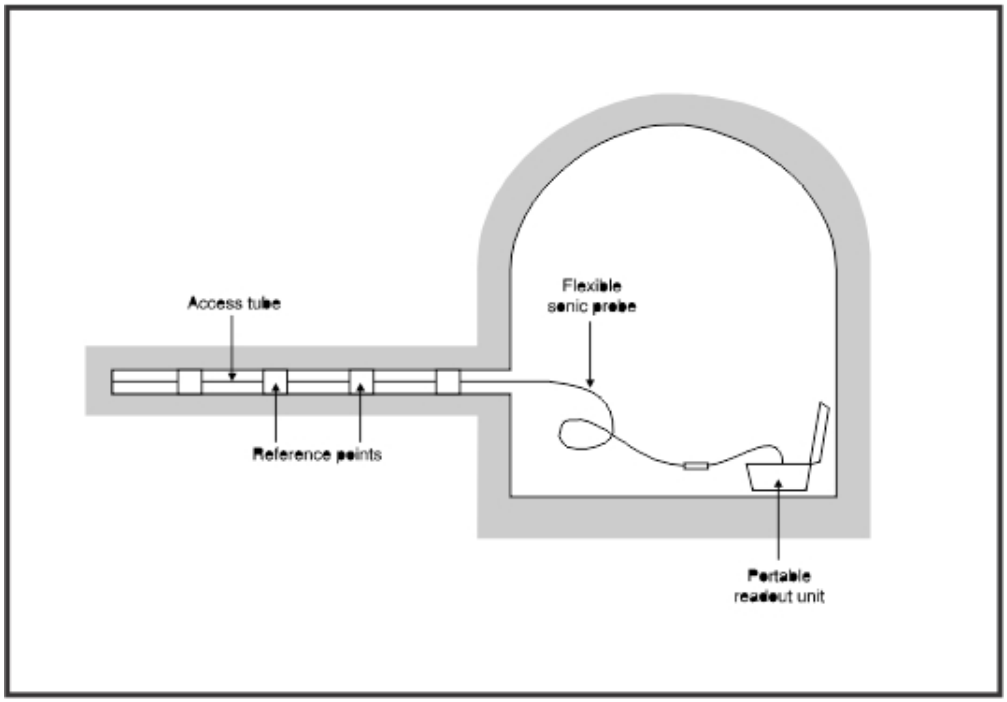


Figure 54: Sonic probe extensometer (RMT Rock Mechanics technology)

back

back

Vibrating wire stress meters:

A major advantage is that numerical conversion of vibration readings into uni-axial stress-change values requires only an approximate estimate of the rock modulus. This is because the vibrating wire is housed in a high-modulus steel cylinder, whose changes in diameter are sensitive mainly to applied stress and not to strain in the rock mass, and the term 'stress meter' is therefore justified. A vibrating wire stress meter provides a uni-axial stress determination, and multiple installations in orthogonal boreholes are required if the full 3-D stress tensor is to be estimated (Figure 55). Based around a vibrating wire gauge, this device can be used for reliable long-term stress-change monitoring. The procedures for its use are similar to those of the strain gauge-based instruments described above. A vibrating wire stress meter, available in either NXCU or BX sizes, is wedged-in at the required depth in an appropriately sized borehole using special installation equipment (this equipment is expensive and so rental from an existing owner is recommended). Readout units, specific to the maker of the stress meter, are also required.

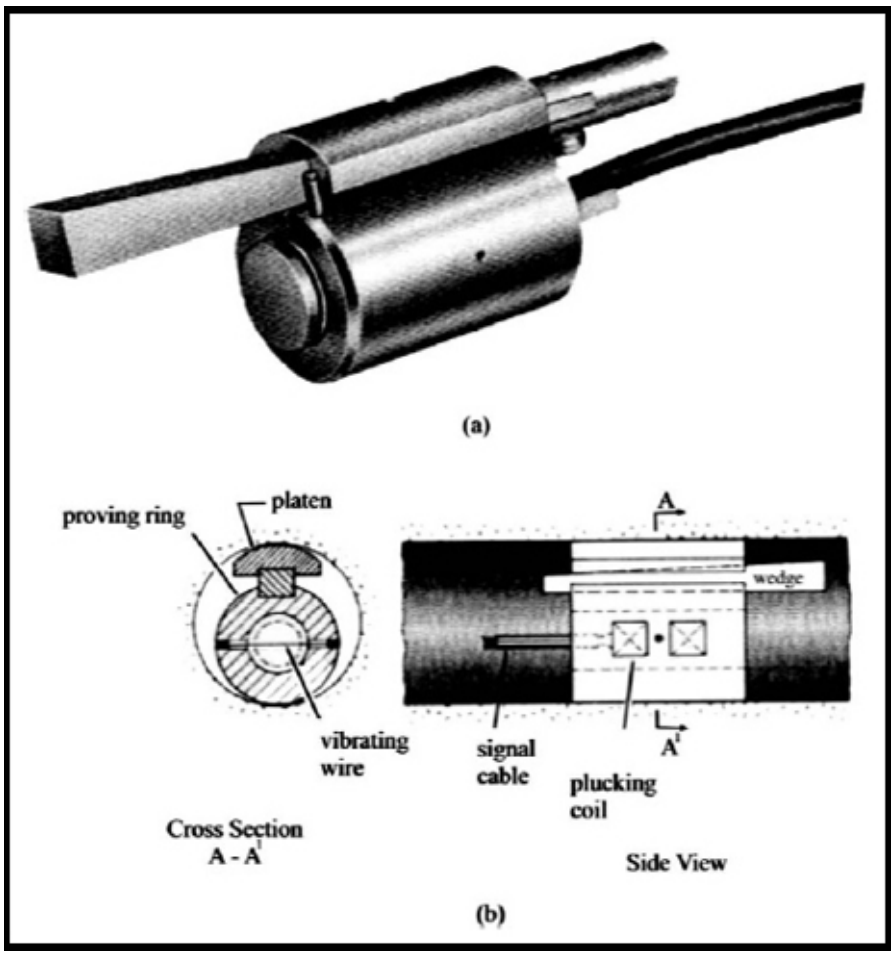


Figure 55: Vibrating wire stress meter (Brady and Brown)

back

back

Petroscope:

A petroscope (Figure 56) is an instrument used in rock engineering to look inside boreholes. This instrument enables a person to see fractures and bed separation inside the hole. It consists of a round brass casing that has a mirror installed at an angle fitted to it. The mirror has millimetre markings on it and, once inserted into the borehole, the thickness of separations can be read off the mirror. Petroscopes used in longer holes also have a light fitted that assists in making the mirror more visible. The petroscope is pushed into the hole with the help of extension rods, which screw into the petroscope and into each other.

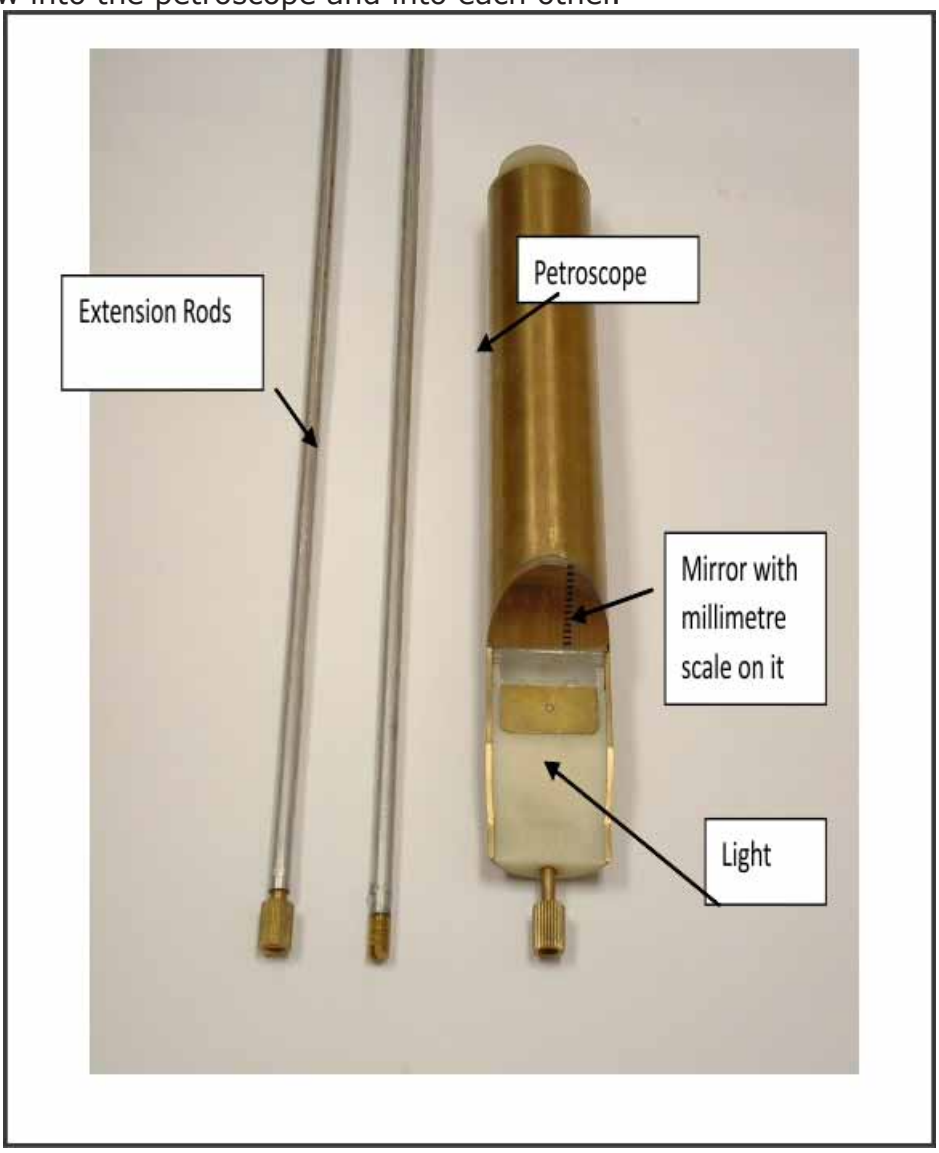


Figure 56: Petroscope and extension rods

back

back

Brunton compass:

A Brunton (Figure 57), or other types of compasses such as the Cocla stratum compass, enables both the dip and the dip direction of strata to be read off directly from the compass, in one operation. With these types of compasses, the lid is placed along the discontinuity surface to be measured, the levelling bubble levelled and the dip and dip direction read off.

The Brunton compass is a small handheld compass. It utilises a mirror with a sighting line in the centre through which a distant target is sighted and aligned. The mirror allows viewing the bearing through the mirror. There is also a set button at the centre of the needle, which when pressed, holds the needle in a fixed position for viewing.

Figure 57: Brunton compass

Figure 58 and Figure 59 below give the instructions to the use of a brunton compass to get a bearing reading:

- Hold the compass level, or place it on a flat surface. It is vital that the compass is held as level as possible. Use the bubble to centre and level the compass;
- Assume a comfortable position for looking through the sight. Adjust the mirror so that it reflects the compass needle and markings;
- Open the small rear sight. Align the compass so that the distant object, front sight and small rear sight are all in alignment. The compass must remain level during use;
- View the bearing on the compass, after sighting the distant object by viewing through the mirror or by viewing the bearing directly;
- Double check the level of the compass, take a second sighting and compare the results to ensure a proper reading.

Also find more on:
http://www.ehow.com/how_4912560_use-brunton-compass.html)

back

back

Taking the dip measurement of a discontinuity plane: Hold the compass level (using the bubble) and align the top cover of the compass to the plane. Look at the dial and read off the angle (in degrees) that intersects the reference line. In Figure 58 below, the dip angle is 70°.

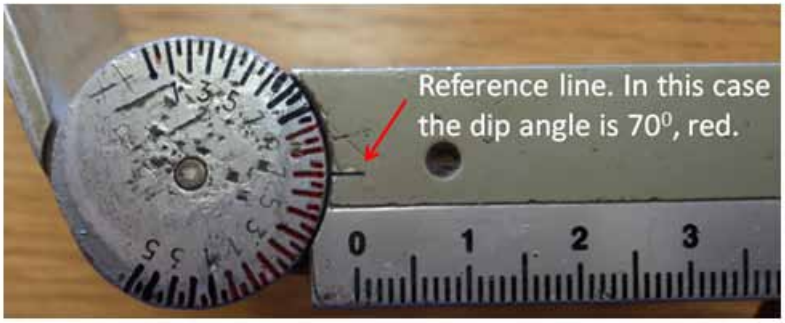


Figure 58: Reading of dip

Taking the dip direction measurement of a discontinuity plane: To take the dip direction measurement, hold the compass as for the dip measurement above and look at the corresponding colour compass needle on the face of the compass. Read the dip direction from the dial. In Figure 59 below, the dip direction is 169° on the red compass needle.

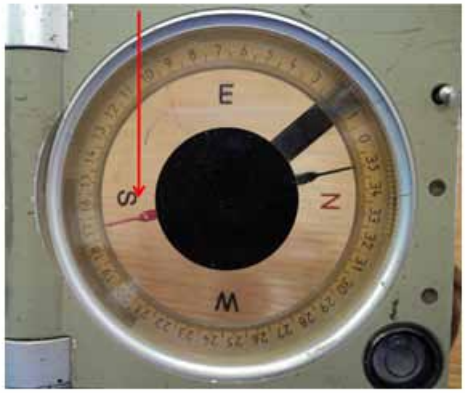


Figure 59: Reading of dip direction

In cases where the direct magnetic measurement of orientation is not possible (e.g. mapping of an adit in a magnetic deposit or orientating fractures with respect to a borehole axis), the azimuth must be measured with respect to a reference line that can in turn be orientated geographically or by other surveying techniques.

- Chapter 11, Hoek E, and Brown E.T., Underground excavations in rock, 1980
- Chapter 19, Brady H.G.B. and Brown E.T., Rock mechanics for underground mining, 2006
- Chapter 9, Ryder J.A. and Jager A.J., A textbook on rock mechanics for tabular hard rock mines, 2002
- Chapter 10, Jager A.J. and Ryder J.A., Handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 2002
- http://www.ehow.com/how_4912560_use-brunton-compass.html

back

back

LEARNING OUTCOME 6.2.3.2

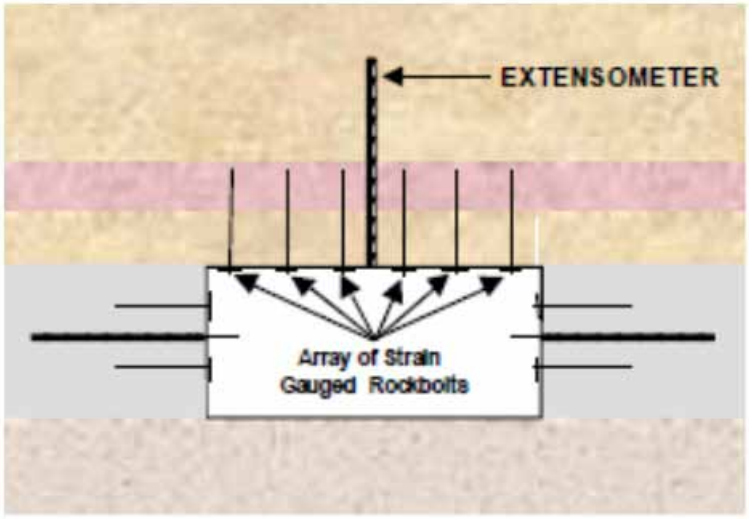
INTERPRET RESULTS FROM THE ABOVE INSTRUMENTS

Results from measuring are normally plotted in graphical format to allow interpretation and analysis. Changes in the behaviour of the rock mass or support units can then be detected and interpreted.

Interpretation of results is explained using the examples below:

EXAMPLE

In Figure 60, an example of a sonic probe measuring station provided the results shown in Figure 61. Note that the results graphed indicate the displacement that had taken place since the last reading and not the total displacement to date.



A TYPICAL ROCKBOLTED TUNNEL MONITORING STATION

Figure 60: Example of a sonic extensometer monitoring station

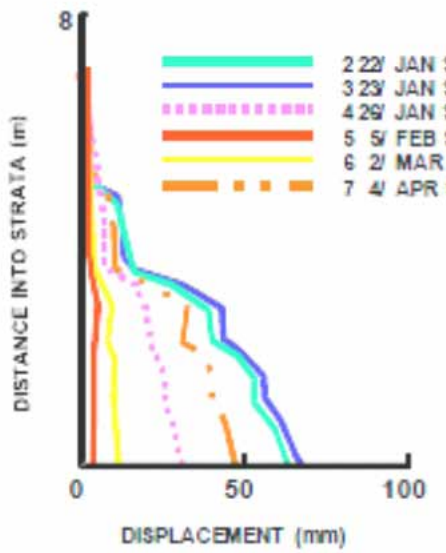


Figure 61: Example of a sonic extensometer monitoring station results

back

back

NOTE

Changes in readings are always indicative of 'something happening'. This 'something' is either known (the reason for the instrumentation) and the trends in the readings are important or the 'something' is unknown and the reason for the changes should be looked for.

The results in Figure 61 could be interpreted as follows:

- The furthest points into the rock instrumented appear not to be affected, i.e. are not moving at all;
- The surface of the excavation (zero metres into rock) displaces the most;
- Significant displacement occurs only from the surface to approximately 5m into the rock;
- Large increases in displacement had been measured on at least four dates, namely 22, 23 and 26 January and 4 April. These "jumps" in displacement could be correlated with recorded seismic events (dates of event occurrences coincide with dates between measurement of the 'jumps');
- Seismic events appear to increase the displacement of the excavation surface into the excavation;
- Normal closure due to stress-related issues appears to be small compared to that induced by seismic activity.

EXAMPLE

In the case of open pit mining, ground water levels may seriously affect the stability of slopes. Boreholes are therefore dewatered and piezometer readings are taken at regular intervals to evaluate the effectiveness of the de-watering programme.

Example results from such a monitoring programme are shown in Table 7, while the results were graphed in Figure 62.

Bore-hole	1 March	1 April	1 May	1 June	1 July	1 August	1 September	1 October	1 November	1 December
WB153	24.9	28.12	27.12	28.0	30.35	30.35	30.6	30.24	30.29	30.3
WB154	21.36	25.0	25.0	30.0	29.95	29.95	31.73	31.23	31.52	31.86
WB155	36.52	39.9	40.0	40.0	31.5	31.5	36.6	38.55	39.23	41.96

Table 7: Example of piezometer readings

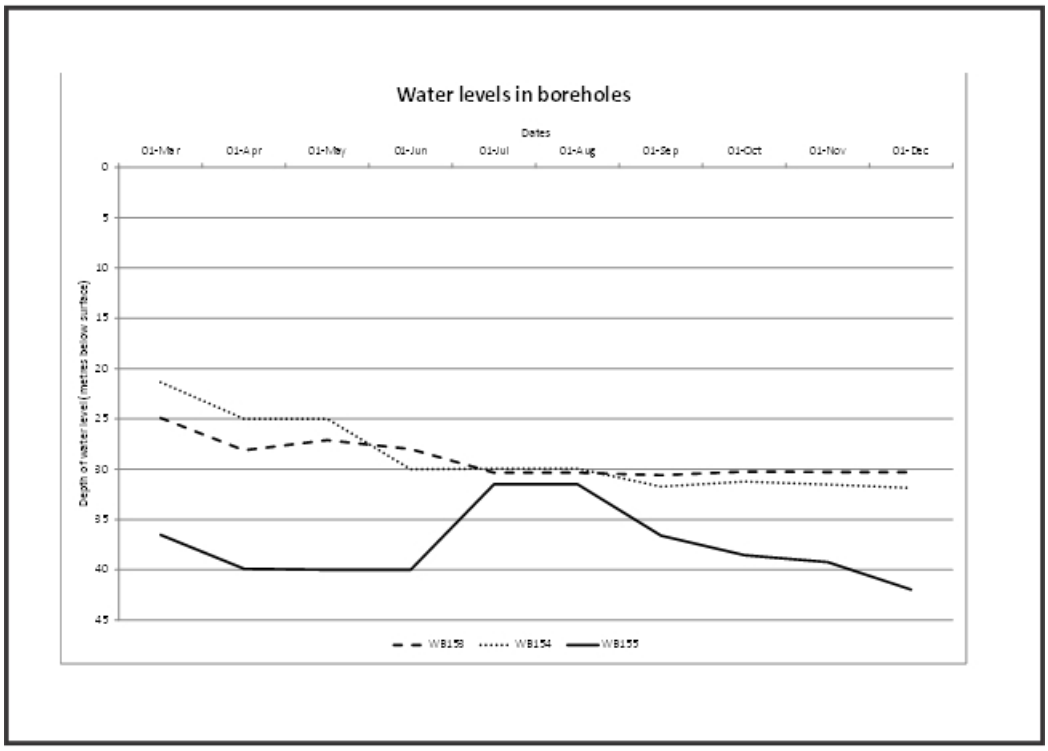


Figure 62: Results from piezometer readings

back

back

The results in Figure 62 could be interpreted as follows:

- Dewatering in boreholes W153 and W154 appears to be working with water levels that are falling constantly;
- Whether the final water levels reached in those borehole are sufficient is unknown, but it appears as if the levels have stabilised over the last few month;
- Borehole W155's dewatering appears to be initially unsuccessful as the water level significantly increased between June and August, but later on succeeded to reduce the water levels significantly;
- There is a need to indicate rainfall on this graph as well as to include the replenishing of ground water due to rainfall in this interpretation;
- Either rainfall affected only W155, or W155 is being replenished via other water conduits.

EXAMPLE

In the case of ground penetration radar, results were drawn to represent complex results usually provided by the radar equipment. Since these result graphs are usually complex, they should be interpreted by an experienced practitioner. Figure 63 shows the results (bottom) indicating some disturbances in the reflected waves, while the top picture shows the interpretation.

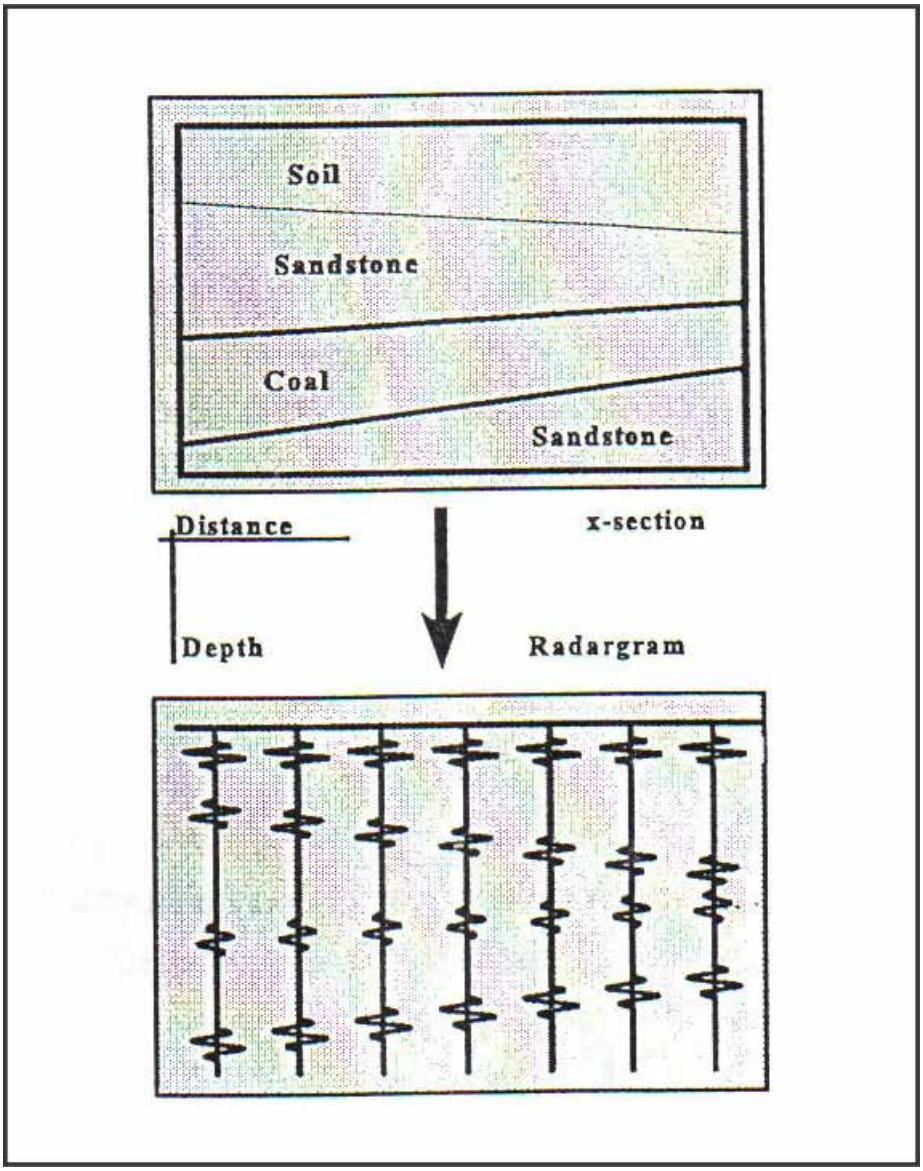


Figure 63: Typical results plot from a GPR study (bottom) and its interpretation (top)

back

LEARNING OUTCOME 6.2.3.3

INTERPRET ROCK MASS RESPONSE INDICATED BY RESULTS FROM THE ABOVE INSTRUMENTS

EXAMPLE

Rock mass response interpretations depend on the instrumentation programme and results obtained. In an attempt to indicate how this could be done, possible interpretations of the rock mass responses for each of the examples shown in Outcome 6.2.3.2 are shown below:

Rock mass response in the sonic probe measurement example:

- Rock mass deeper than 5m into the excavation walls does not displace towards the excavation, even in cases where significant seismic events occurred;
- Excavation walls displace at low rates under normal conditions, but much faster rates when seismic events occur;
- Support that will be anchored beyond 8m into the excavation walls appears to have no impact on wall displacements;
- Support that will be anchored within 5m of the excavation surface will displace with the rock mass and will not prevent or reduce the displacements.

EXAMPLE

Rock mass response in the water level measurement example:

- The rock mass around the W155 hole may be more porous (higher permeability) and allow larger volumes to seep towards the borehole in a certain period compared to the other boreholes where the rock mass may be less permeable;
- Other water conduits may be intersected by W155 that allow the water level to be replenished.

EXAMPLE

Rock mass response in the GPR example:

The soil material thickness is increasing towards the north (assume north is right in Figure 63) increasing potential concerns for access development (if deep enough) or increases the need to do pre-stripping of the soil (open cast);

- The thickness of the coal seam remains reasonably constant, but slightly reduces towards the north and may not have a significant impact on the design of the pillar systems;
- The coal roof material, sandstone, reduces significantly in thickness towards the north, indicating potential roof concerns in areas where the roof is less massive.

LEARNING OUTCOME 6.2.3.4

DESCRIBE AND EXPLAIN THE PRINCIPLES OF OPERATION OF STRAIN GAUGES

Strain gauges are widely used in rock engineering practices (as shown in the previous outcomes on instrumentation) and their basic operation is discussed below.

REFERENCE

http://www.scribd.com/doc/26839029/Strain-Gauge-Working-Principle

- The majority of strain gauges are foil types, available in many shapes and sizes;
- They consist of a pattern of resistive foil that is mounted on a baking material;
- They operate on the principle that 'as the foil is subjected to stress, the resistance of the foil changes in a defined manner';
- The strain gauge is connected into a Wheatstone bridge circuit with a combination of four active gauges (full bridge), two gauges

(half bridge) and sometimes, a single bridge (quarter bridge). The Wheatstone bridge is excited by a small DC supply;

- As stress is applied to the bonded strain gauge, a resistive change takes place and unbalances the Wheatstone bridge;
- This unbalance results in a signal output (few millivolts);
- This is enhanced using electronics and collected into data recorders and analysis systems that provide the required output to the user.

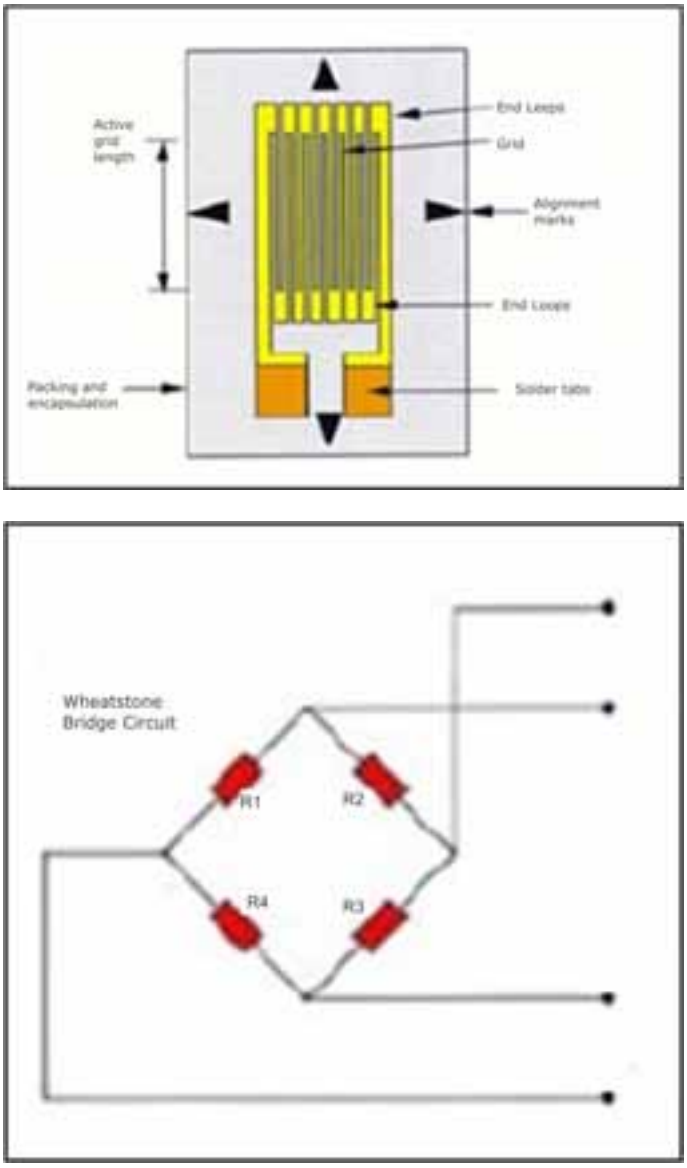


Figure 64: (a) Strain gauge and (b) Wheatstone Bridge Circuit

FUNDAMENTALS OF SEISMIC MONITORING SYSTEMS

LEARNING OUTCOME 6.3 PART 1

NOTE

This section on seismic monitoring systems does not necessarily follow the outcomes as stipulated in chapter 6.3, but is compiled in a logical and practical order to simplify the concepts and their applications.

Conceptual approach to SO 1-7

- Introduce basic seismic monitoring tasks, data recording and storage.
- Distinguish between deep tabular, shallow massive and surface mining.
- Explain sensor types and their capabilities and main network and system components incl. their functionality.
- Point out similarities and differences between ISSI, PRISM and other systems.
- Explain basic network configurations and design rules.

OBJECTIVES OF MONITORING

Prior to the installation of seismic equipment the objectives of monitoring should be clearly defined. Depending on the history of seismicity, the size of the area affected, the severity of associated losses and the projected time frame of monitoring a strategy is needed that will answer the following questions:

- where does seismic failure occur?
- what are the failure mechanisms?
- what are the spatial and temporal trends in seismicity?
- Are there any uncertainties in the mine design or methods?
- Is the seismic potential of geological discontinuities (faults & dykes) understood?

IMPORTANT

Apart from being a legal obligation to monitor seismic hazard, seismic monitoring can and should be used to assist with the optimisation of mining methods and with the control of mine design parameters, such as the optimum width of bracket pillars on geological structures.

In general, the approach should be from coarse to fine: A mine that was never covered by a seismic system should begin with a small set of sensors that provide basic information first, rather than starting the monitoring with a dense array of highly sensitive equipment. As an initial step, a spatial resolution of 1/20 to 1/50 of the size of the object is recommended, e.g. a mine measuring 3km across and experiencing effects of seismicity in several areas should design for location accuracy in plan of 60-150m. Once seismogenic regions and mechanisms of seismic failure are better known sub-networks can be installed that specifically monitor selected areas.

The Guide to routine seismic monitoring in mines (Mendecki et al, 1999) in the Handbook for Rock Engineering Practice (Jager and Ryder, 1999) lists a number of seismic monitoring objectives for deep tabular mines and defines the required network sensitivities to achieve each objective.

DEFINITION

The following terms are commonly used:

- "Location": Hypocentral co-ordinates and magnitude to pinpoint the affected area and assist with rescue operations, if necessary. This often refers to a location obtained through automatic processing of a large event.
- "Prevention": Study of the relationship between (seismic) failure and design; feed back into the mine's planning process to optimise sequence and layouts
- "Control": Search for medium-term changes in seismicity, e.g. clustering in space or increases in activity rate over days; check for compliance and sub-standard conditions; possible reaction by reducing mining rates or temporarily suspending shifts.
- "Warning": Detection of short-term changes in seismic source parameters that constitute precursory behaviour; analysis of seismic data in terms of rock mass instability and imminent failure
- "Back-analysis": Detailed analysis of single, important events in terms of mining, geology and damage caused.

The recommendations regarding system performance listed in Mendecki et al (1999) are shown in
Table 8.

Monitoring Objective	Sensitivity (Mmin)	Location Accuracy (in Plan)	Time Delay
Detect potential rockbursts	0.5	200m	2 minutes
Long term hazard assesment	1.0	200m	Days
Medium term hazard assesment	0	30m	Days
Short term hazard assesment	-1.0	30m	10 minutes
Emergency alarm	-3.0 (using crack counting)	30m	Seconds
Back analysis of large events	0.0 (for aftershocks)	30m	Hours
Regional monitoring	1.0	200m	10 minutes

Table 8: Guidelines for system performance (Durrheim et al, 2007)

Even though the recommended sensitivities and accuracies are somewhat arbitrary, they provide basic guidelines for system design: The smaller the seismic source, the finer the recommended location detail. The benefit of accuracy better than the source extent of larger events is of limited value as the failure region may extend for distances larger than the location accuracy. If seismic hazard assessments are to be carried out for single panels, then seismic data should be located to within 20-30m accuracy. A shortcoming of Table 8 is that the impression might be created that a certain location accuracy and sensitivity would enable a specific objective to be fulfilled, which is not the case. To implement the 'control' and 'warning', in-time detection of unfavourable seismicity trends, requires more than a sensitive and accurate network.

IMPORTANT

Warning procedures are followed on a large number of mines daily for a fixed number of active working places. Although some studies have shown that warnings are somewhat better than random (e.g. Stewart and Spottiswoode. 1993 & 1996, and van Aswegen, 2005) mines have been loathed to take warnings particularly seriously. The value of warnings has been questioned by Spottiswoode (2010) who found that seismic patterns before large events are not distinctly different from the patterns

back

before small events. As small events are very common and regular on all highly-stressed faces, warnings that rely on day-to-day changes are therefore unlikely to be reliable.

Research for earthquake prediction by Japanese and South African workers is continuing encouraged by evidence of strain changes prior to mine events (Naoi et al., 2006, also see the current JST-JICA project).

A different approach to network performance would be to orient the desired sensitivity on the threshold of relevance, i.e. the estimated magnitude of seismic failures that either cause damage or carry information meaningful to seismic data interpretation. Unless there is evidence that small events contain valuable information (approximately M<-0.5), sensitivity of networks could be limited to relevant magnitude ranges, possibly only events resulting in rockbursts.

Setting the required sensitivity below M=0.0 also introduces the problem that development blasts are recorded and are often included in the database (e.g. Spottiswoode and Linzer, 2005 and Spottiswoode, 2010).

The standard set of objectives listed above applies to hard-rock tabular mines with an appreciable seismic risk. Shallow mines and mines with differently shaped ore bodies that implement massive mining methods may have different reasons to operate a seismic network. For open-cast operations the monitoring of slope stability may be of importance. For shallow underground mines the stability of the crown pillar may be of concern or to monitor the controlled caving process.

EXAMPLE

Underground operation of Palabora Mining Company Ltd.

In all cases and irrespective of the ideal network layout, the monitoring system has to be planned within certain constraints which usually relate to resources and physical access:

- financial planning
- technical capacity in the engineering department
- access limited by tunnels to host rock volume and ore body
- existing infrastructure for data transmission.

It is advisable to plan not only for the system installation but also for system maintenance, personnel to manage, operate and maintain the system, to process the data and to evaluate the seismic information regularly and on a standard that is commensurate with the complexity of dynamic rock mass failure in a mining environment. Seismic monitoring systems require continuous technical support and periodic reviews. The following section describes seismic monitoring equipment used on mines and how the various components are integrated into a functional seismic system.

SEISMIC SYSTEM COMPONENTS

Mine seismic systems comprise sensors, units to filter and record seismic signals, data transmission infrastructure and a central computer for processing and storage with an event data base. South African mines use the store-and-forward system (e.g. Figure s2-11): Seismic signals from underground sensors are temporarily stored close to the recording site and only high-quality, valid seismograms associated with seismic events are transmitted to surface. Noise and blast recordings are discarded to reduce the amount of data sent to surface.

Other network types include stand-alone networks, which gather and pre-process data near the area of interest and require manual data collection and final processing in a separate location. This network type is

back

deployed where access is difficult or unsafe, data transmission is unreliable or data quantities are small, for instance in the case of remnant extractions of for small-scale research applications.

- www.imseismology.org/
- www.esg.ca

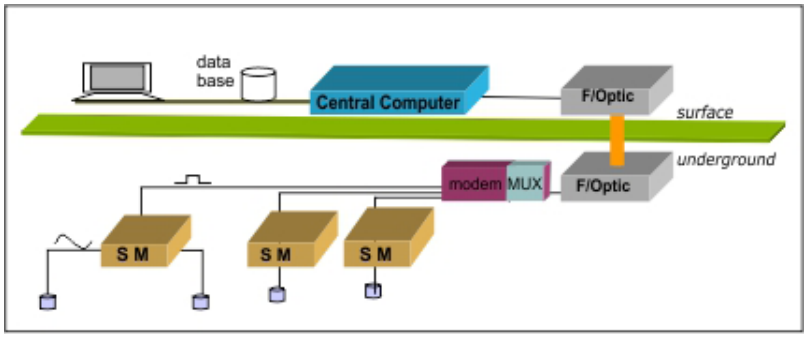


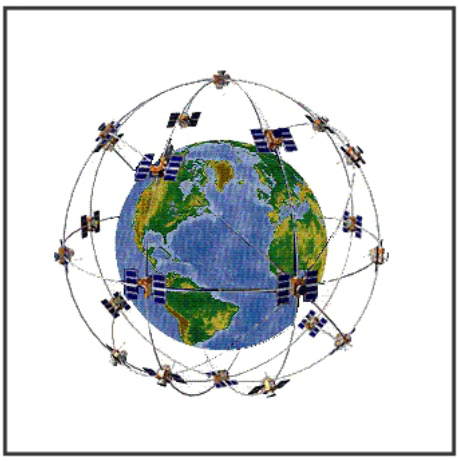
Figure 65: Example of a digital store and forward system

Sensor signal is digitised and stored in seismometer box (SM), digital communication via FSK standard along tunnels, conversion to RS232 in modem, bundling via multiplexer (MUX), vertical communication in the shaft via optic fibre, processing and storage in central computer on surface (courtesy IMS).

A third network type, analogue stream, is used in shallow mines with short distances between observation area and central computer. These networks transmit the sensor signal directly to the processing unit without making use of seismometers for temporary signal storage. A trigger algorithm continuously screens all incoming analogue signals for high ground motion amplitudes that may originate from a valid seismic event.

A seismic system is made up of hardware units such as sensors and seismometer boxes, fibre-optic communication devises, computers and hard drives for data storage, and seismic system software that allows data processing, communication between seismometers and central computer and the dissemination of seismic information to users. Software licenses are required for each sensor channel, the system software and specialised processing and viewing software.

Common on today's networks are GPS units that supply global satellite time to the network to ensure that recordings are properly referenced in a global time frame. The small time delay between arrival times of waves at different seismic stations requires a high degree of synchronisation between each of the seismometers and the central computer, but global timing assists with synchronising data from different mines close to one another.



Global Positioning System (http://www.pdhonline.org/courses/l116/l116content.htm**)**

SENSORS

Sensors vary in size and are based mainly on two different physical principals: Induction and piezo-electric effect. Their output signals differ accordingly. Depending on the application the suitable sensor type needs to be chosen according to the frequency range to be recorded by the sensor and the dynamic range (lowest and highest recoverable ground motion). At the lower end of the ground motion range background noise levels are determined by the sensor and random vibrations of the rock, tramming and mechanical vibrations of machinery, but also electric fields induced by power lines, electric motors such as fans and pumps. The upper end of the usable range is determined by sensor clipping due to strong ground motion that exceeds the ability of the sensor mechanics to accommodate movement. To record the strong ground motion associated with large events requires more robust sensors than the recording of micro-seismicity.

REFERENCE

- A good reference for seismic sensors is: http://www.geophys.uni-stuttgart.de/oldwww/seismometry/man_html/node1.html

Different types of seismic sensors are most useful within ranges of frequencies and ground velocities (Figure 66).

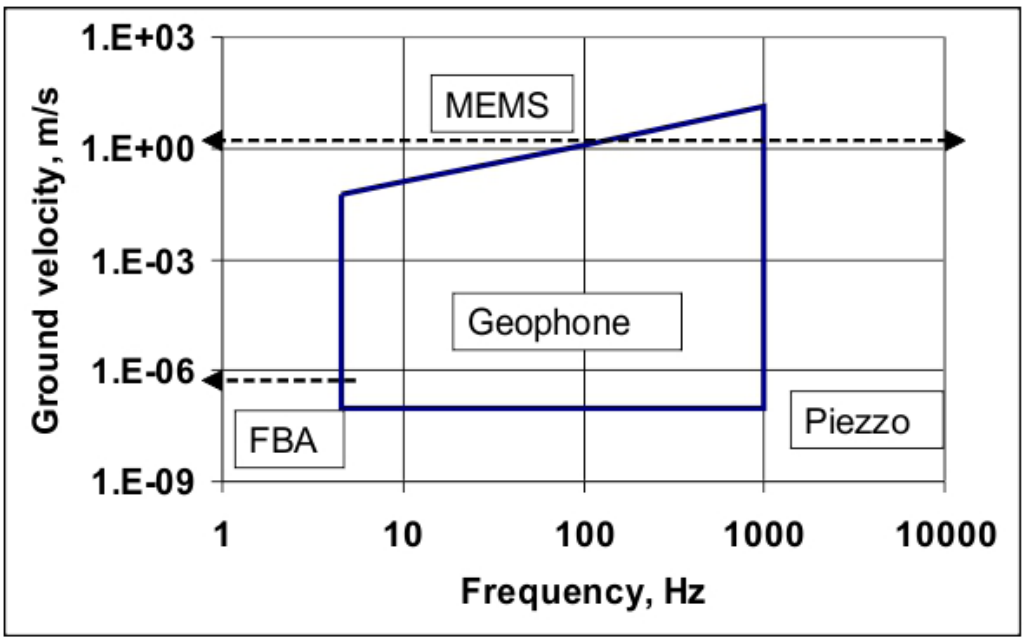


Figure 66: Performance of different seismic senor types in a range of frequencies and ground velocities

back

Geophones are most commonly used in mine seismic networks. They are very sensitive devices and cover the range of frequencies of greatest interest in mining seismology.

Figure 67: Seismic sensors (a) 30V/g uni-axial accelerometer, (b) 0.5 V/g tri-axial accelerometer, (c) 0.7 V/in/s uni-directional geophone and (d) 1.3 V/in/s omni-directional geophone (courtesy: ESG)

Limitations of geophones are overcome by a range of accelerometers, namely Force-balance accelerometers (FBA) record small signals from distant earthquakes at frequencies down to zero. The classic accelerometer type for mining and industrial applications is the Piezo electric device. Micro Electro Mechanical Systems (MEMS) are being used in increasing numbers and applications. A mine might use a few tens of geophones. In comparison, seismic exploration for oil and minerals, including South African Platinum and Gold, use a few tens of thousands of geophones.

Micro Electro Mechanical System (http://www.memx.com)

back

Geophones function according to the principle of electromagnetic induction (see Figure 18): The electric field $\bar{E}$ induced in a conductor is proportional to the rate of change of the magnetic field inducing $\bar{E}$. If a coil moves inside a magnet attached to a vibrating rock mass the voltage induced in the coil is proportional to the velocity of vibration. Thus, geophones record ground velocities and the usable frequency range depends on the construction of the geophone.

DEFINITION

Usable range refers to the band of frequencies that is recorded with a flat transfer function, i.e. identical ground motion amplitude induces identical voltage amplitudes, irrespective of the frequency of the motion.

The mass is suspended by a flat spring resulting in the spring stiffness setting the lower and upper useful frequency of the sensor.

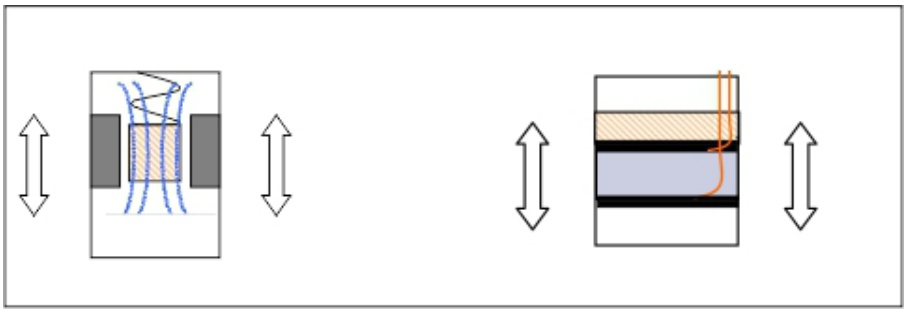


Figure 68: Principle of a geophone (left) and an piezoelectric accelerometer embedded in a vibrating rock mass.

Low frequency seismometers for natural earthquake monitoring – unless electronically enhanced – are bulky and can weigh several kilograms. Geophones for higher frequency ranges are small and can be grouted into drill holes with diameters in the range of 50-75mm. In mining applications, natural frequencies ranging from 4.5 to 30Hz are most common, up to several hundred Hertz. These same sensors are then unsuitable when it comes to large tremors with dominant frequencies in the range 0.1 to 3 Hz.

NOTE

(USGS): A seismometer is the internal part of the seismograph, which may be a pendulum or a mass mounted on a spring; however, it is often used synonymously used with "seismograph".

In order to reliably record small, high frequency events geophones would have to be extremely light and would be susceptible to background noise. The natural limit to sensitivity when using a sensor containing moving parts, is the background vibration level due to tramming, drilling and similar activities. Thus, a different principal is used to record small, low amplitude vibrations: Certain materials generate electric fields on a molecular scale when they experience mechanical pressure (piezo-electric effect). The field is proportional to the force applied, which in turn is proportional to acceleration (Newton II): Transducers built according to this principle measure ground acceleration. The advantage of an accelerometer is that the rise in signal associated with a wave arrival is steeper than that from a geophone making it easier to determine arrival times. On the other hand an accelerometer's output signal is weak and requires amplification at the sensor (accelerometers need a power supply to function, geophones do not).

back

BRIDGING GAP 6

"Newton's laws of motion" on page 19

EXAMPLE

It is likely that the source parameters derived from the accelerograms produced by the accelerometers are in error for several reasons:

1. Seismic parameters are derived from ground velocity (seismic energy) or from ground displacement (seismic moment) through two stages of integration.
2. Accelerograms require an additional stage of amplification and therefore are particularly prone to noise at low frequencies.

EXAMPLE

There are two crucial parameters that determine the quality of digitisation: Resolution and dynamic range. Typically, a geophone trace varies between +10 and -10V, and the noise level is around 10-6m/s equivalent to 4*10-5V. In order to truly represent a 20V range in steps of 10-5V (resolution), one needs 2 million different numbers. In binary code that would require 21 digits with the smallest being 1=20 and the largest 221=2 097 152 (dynamic range). Typical recordings last about 1s and signals are sampled approximately 10 000 times per second yielding 10 000 integer numbers for every second of recording, each integer being 32 digits long (internally, computers handle numbers with 16 or 32 digits).

To estimate the data flow, consider a network with 25 tri-axial seismic stations, each recording an M=2.0 event on the mine. The system has to handle 3x25x10 000 integers, each being 4 bytes (32 digits) long: 3Mb of data that needs to be transmitted to the surface without delay, excluding messages between the central computer and stations regarding trigger times, maximum amplitudes, dominant frequencies, etc.

	Frequency range	Sensitivity	Maximum accel./velocity	Resonance frequency
Geophone	1-500Hz	20-200 V/m/s	40-100mm/s	1-40Hz
Accelerometer	0.1 - 20 000Hz	10-1 000mV/g	1-100g	5-50KHz

Table 9: Range of specifications for geophones and piezoelectric accelerometers (after Mendecki (Editor), 1997)

Together, the two sensor types can complement each other and are able to cover the requirements of most deep-level mining applications (Table 4). For small scale micro-seismic monitoring accelerometers are more suitable, but large events with low dominant frequencies are better recorded with geophones with resonance frequencies of a few Hz. The problem when using mixed sensor types in a seismic system is one of calibration of source parameters: Currently, the calculation of seismic energy and moment from these two sensor types is not entirely compatible.

SIGNAL PROCESSING

Electric signals from sensors are rapidly attenuated in conventional copper cable and when the noise levels comes close to the signal level the latter becomes irrecoverable. This is the reason for signals being amplified and digitised within 200-400m cable length from the sensor before being stored in a seismometer. Digitising entails conversion from a smooth, uninterrupted AC voltage to a binary code that represents the electric signal. This is done in analogue/digital or A/D converters that measure the signal strength at regular time intervals (several thousand times per second), encode the amplitude and store these values in memory. The better the signal is represented the more data points or samples are required per second.

Other than ease of storage and transmission, digitised signals can be amplified, filtered, compressed and later analysed through automated processes. For an accurate analysis the signal must be well represented, i.e. nothing relevant must be removed and nothing added. The amount of data awaiting transmission after a large event (or multiple events) justifies the use of fibre optic cable in the shaft. To save on bandwidth sampling frequencies are reduced and seismograms shortened whenever possible which explains a network design with 'intelligent' components underground that discard un-associated triggers and analyse recordings for optimum sampling frequency and signal length. The process of reducing seismograms to the minimum sample density required to retain all relevant information in a signal is called 'decimation'.

Once seismograms from different sensor sites have been associated (marked as belonging to the same seismic event), they are sent to surface. Data transmission consists of horizontal and vertical transmission lines to the central computer on surface. Data communication within modern mine networks is similar to an intranet where each seismometer has a unique identifier (IP address) and can be directly communicated with.

While the larger amount of data flows from underground to surface, there is also a need for downward communication. Time markers are sent regularly to seismometers to synchronise their internal clocks, test messages to measure data speed, requests for internal status reports, rebooting instructions and similar messages.

back

back

OTHER NETWORK COMPONENTS

Apart from sensors and signal cables, seismic networks consist of the following components with their respective functions:

- The trigger unit is located inside a seismometer. Its role is to decide when an incoming signal is sufficiently strong to be considered a valid trigger; once a trigger is declared the seismogram is stored in memory and basic trigger information such as the trigger time and the maximum signal amplitude are sent to surface.
- The storage unit (seismometer) has several hundred buffers to temporarily store signals that are amplified, digitised in hard memory prior to transmission to surface. Invalid triggers that cannot be associated with triggers from other sensors are overwritten.
- The associator algorithm continuously searches for triggers from the sensor sites that may belong to a seismic event. In most networks association takes places once at least 5 stations have reported a trigger within a pre-defined time window, roughly the time it would take an S-wave to travel from one end of the monitoring area to the other. Once the associator has declared an event, the respective seismograms are sent to surface.
- Data processing takes place in seismometers and in the central site on surface: Pre-processing in the seismometer includes DC-offset and noise removal, maximum amplitude and trigger time determination and digitising. Manual processing on surface includes picking wave arrivals, determining the hypocentre and determining source parameters.
- The database stores files containing source parameters and seismograms, usually on a hard-disk drive at the central network computer on surface or backed-up to CD or DVD. Early versions of analogue networks, in use until the late 1980s, stored seismograms on magnetic tape. These records have largely been lost due to the limited life span of the magnetisation and the tape material itself.

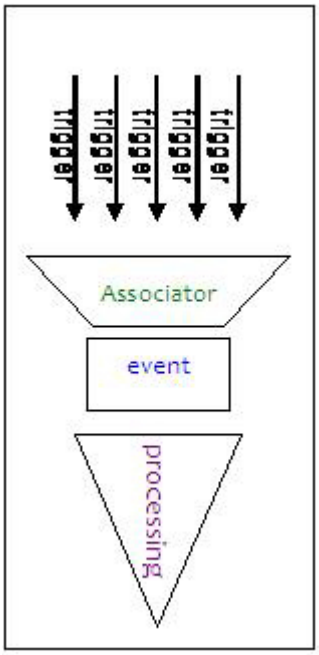


Table 10: Other network components

To install a sensor, a hole is drilled approximately 8-10m into the foot- or hangingwall and the sensor grouted into the hole. In a nearby cubby that is well ventilated, dry and well-lit, a stand is erected or a steel container is mounted on the side-wall that contains a transformer, the seismometer and the cable connection boxes (Figure 19). The choice of a sensor site should be such that it is safe to access for maintenance and repair work and at least 100m away from sources of electric noise such as substation, fans and large pumps. A location on a major tramming route should be avoided.

These requirements place constraints on the placement of seismic

back

back

stations. The final configuration is likely to be a compromise between the desired layout and the one that can be practically implemented. Planning a network is an ongoing process, usual done annually as active mining areas move, sections are closed down and new sources of seismic hazard emerge. It is important to periodically review the purpose of seismic monitoring and modify the system accordingly.

Figure 69: Seismic station

COSTS

Network installation costs in the case of surface operations and shallow mines consist mainly of network hard-ware, licenses, firm- and software (examples in Figure 20 through Figure 22). The costs of retrieving the data from sensor sites do not normally play a dominant role for these operations. This is different in the case of intermediate to deep mines where horizontal cabling and the fibre-optic links can contribute up to half of the installation costs. It is not uncommon for a deep gold mine to install 10-20km of multi-pair copper cabling along tunnels in order to link up seismometers to the fibre optic converters at the shaft.

back

back

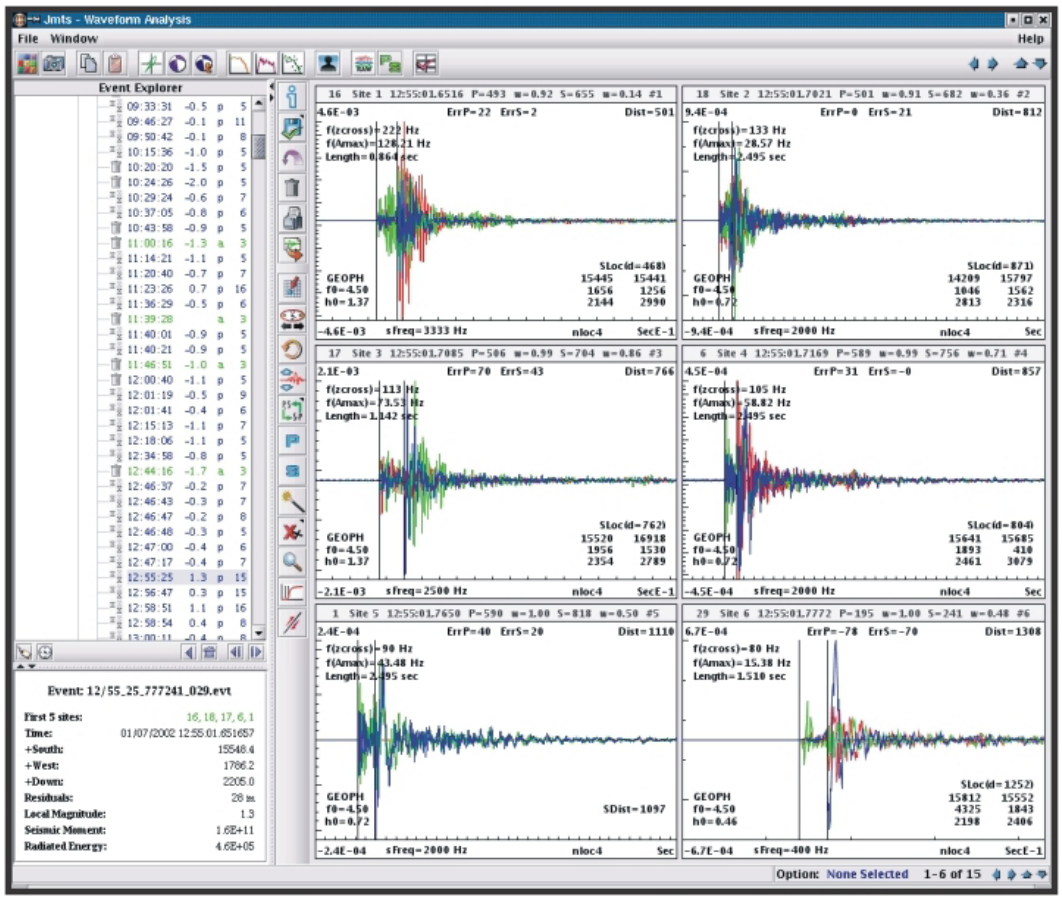


Figure 70: JMTS processing software (courtesy: ISSI)

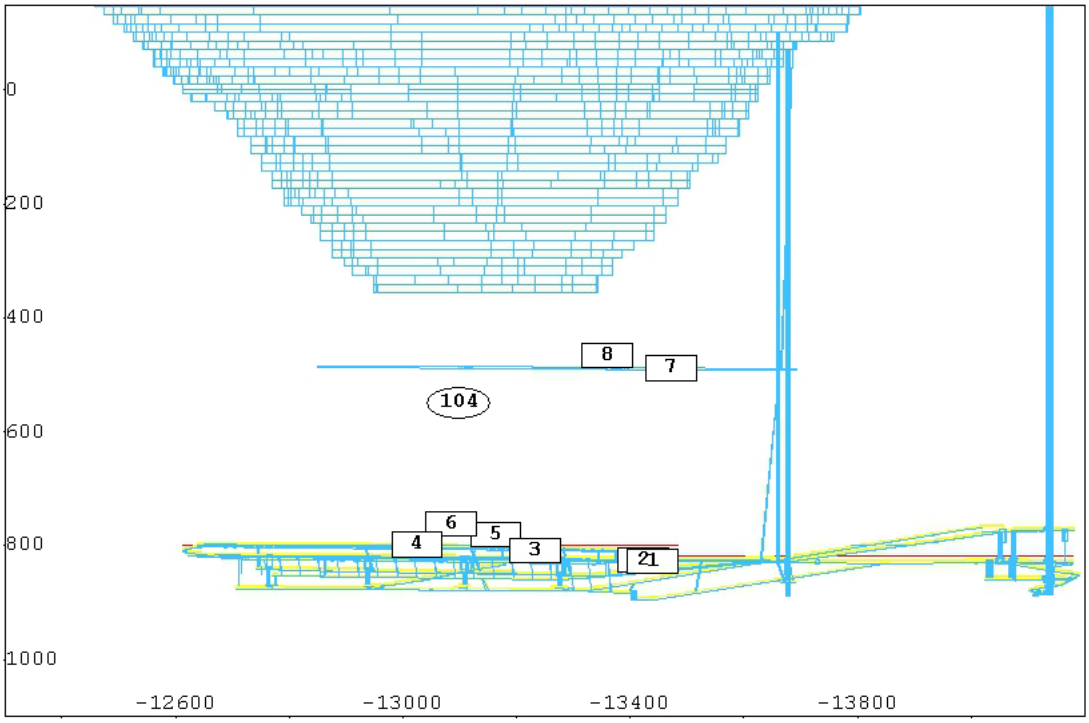


Figure 71: Seismic sensor layout (courtesy: HAMERKOP Scientific Services)

back

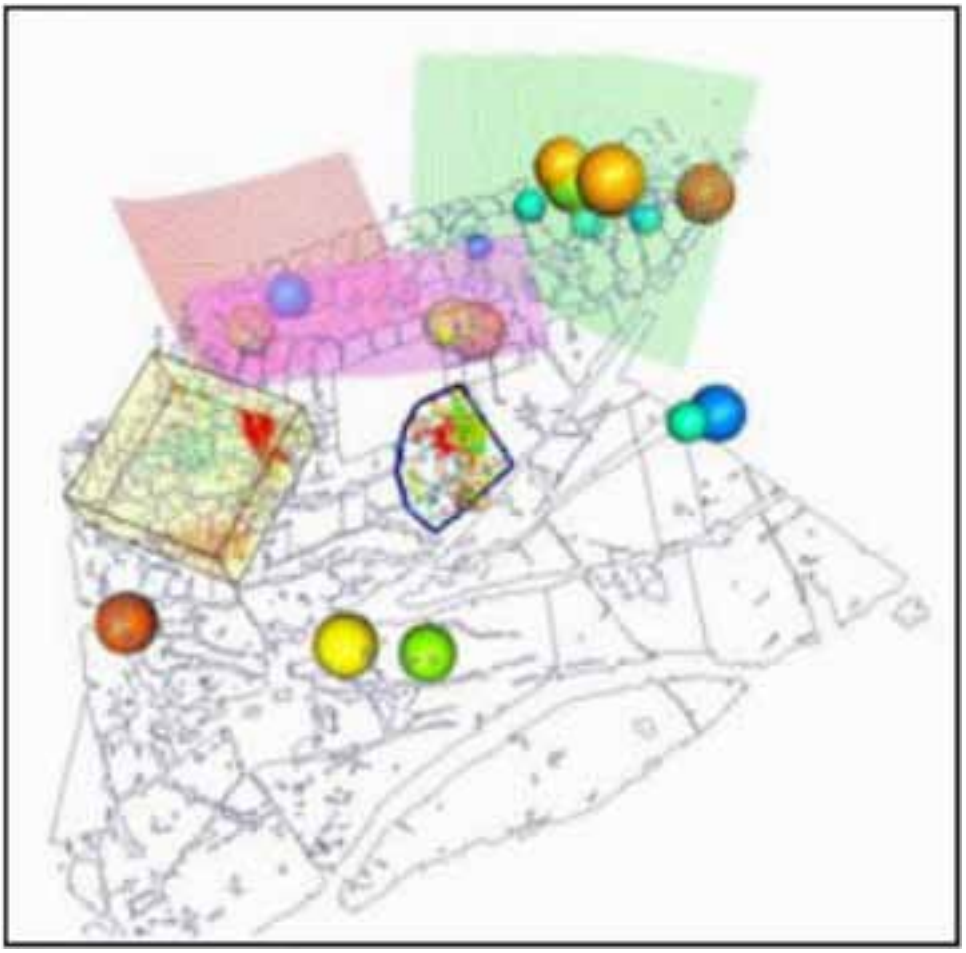


Figure 72: Seismic data viewing software (courtesy: ISSI)

In the simplest of cases, a network consists of a single seismometer box and 1-6 sensors, which may be a combination of uni-axial and tri-axial sensors. Such a standalone system is suitable for temporary monitoring of a small area of interest and, where possible, rental agreements can be entered into with the network supplier. The following example assumes purchase of the sensors, a single seismometer, manual data retrieval and 3-months' rental. When installation and commissioning are included, this basic network from a South African supplier requires an initial budget of R128 000 for the first quarter with a recurring quarterly cost of approximately R33 500, allowing the operator to not only process seismic data but also make use of data viewing and analysis software. On completion of the monitoring project, all hard- and software are returned to the supplier except for the sensors, which are usually grouted into sensor holes and are not recoverable. The expenditure profile of a small network is relatively even over time, with an initial amount that is similar to the regular operating costs.

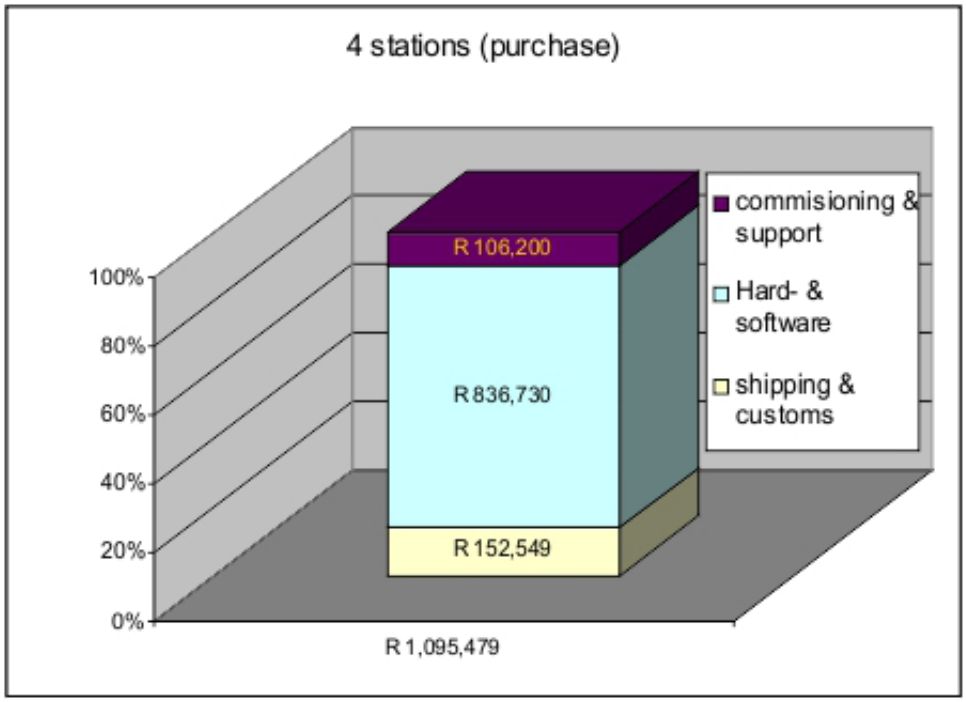


Figure 73: Typical network costs

The expenditure in case of a larger network is less even over time since the initial investment is for hard- and software, licenses and installation and continuous costs only relate to technical support and maintenance. A 4-station network quoted for by a Canadian supplier at the end of 2008 totals just over R1m (at R9/US$) including shipping and handling, customs and installation. This network was designed to cover a shallow tabular gold mine with 4 remote sensor sites, 13 geophones and accelerometers, solar power supply and micro-wave communication between stations and central site (Figure s2-16). The operating and maintenance costs amount to roughly 10% of the initial system price per year of operation.

The approximate cost of a 10-station network from a South African supplier is currently closer to R2.5m when excluding copper cabling, labour and commissioning costs. Added to this would have to be processing and viewing and analysis software. Technical support and maintenance amount to around 10-15% of initial equipment costs annually. These costs are reasonable where the seismic network can assist in better managing seismic risk, but if the network is ill maintained and data quality is poor, the return on investing in a seismic system may not be high enough to warrant the expense.

LEARNING OUTCOME 6.3 PART 2

The following chapter looks into factors determining location accuracy and sensitivity and a range of issues that could reduce seismic data quality.

REFERENCE

- RJ Durrheim, A Cichowicz, R Ebrahim-Trollope, F Essrich, O Goldbach, L Linzer, S Spottiswoode, T Stankiewicz, G van Aswegen [2006]: Minimising the Rockburst Risk: Guidelines, Standards and Best Practice for Seismic Hazard Assessment and Rockburst Risk Management, Mine Health & Safety Council, SIM050302 (Phase 2, Output 3)

back

- Glazer 20XX or
- Hudyma, et al 20XX
- Naoi, M, Ogasawara,H., Takeuchi,J., Yamamoto, A., Shimoda,N., Morishita, K., Ishii, I., Nakao, S., van Aswegen, G., Mendecki, A., Lenegan, P., Ebrahim-Trollope, R. and Iio, Y. (2006) Small slow-strain steps and their forerunners observed in gold mine in South Africa, Geophysical Research Letters, Vol. 33, L12304, doi:10.1029/2006GL026507, 2006
- Spottiswoode, S.M & Linzer, L.M. (2005) A Hybrid Location Methodology, J. S. A. Inst. Min. Metall., Vol. 105, pp 417-426.
- Spottiswoode, S.M. (2010) Mine seismicity: prediction or forecasting?, Journal of the Institute of Mining and Metallurgy, Paper number P110-01-011.
- Stewart, R.D. and Spottiswoode, S.M. (1993) A technique for determining seismic risk in deep-level mining, 3rd Intl Symposium on Rockbursts and Seismicity in mines, Balkema, pp123-128.
- Stewart, R.D. and Spottiswoode, S.M. (1996) Multiparameter seismic risk assessment for deep-level mining EUROCK '96, Barla (ed), Balkema, pp1163-1170.
- http://earthquake.usgs.gov/learn/faq/
- Van Aswegen, G. (2005) Routine seismic hazard assessment in South African mines. 6th Intl Symposium on Rockbursts and Seismicity in mines, Perth, Australia, pp. 437-444.

back

back

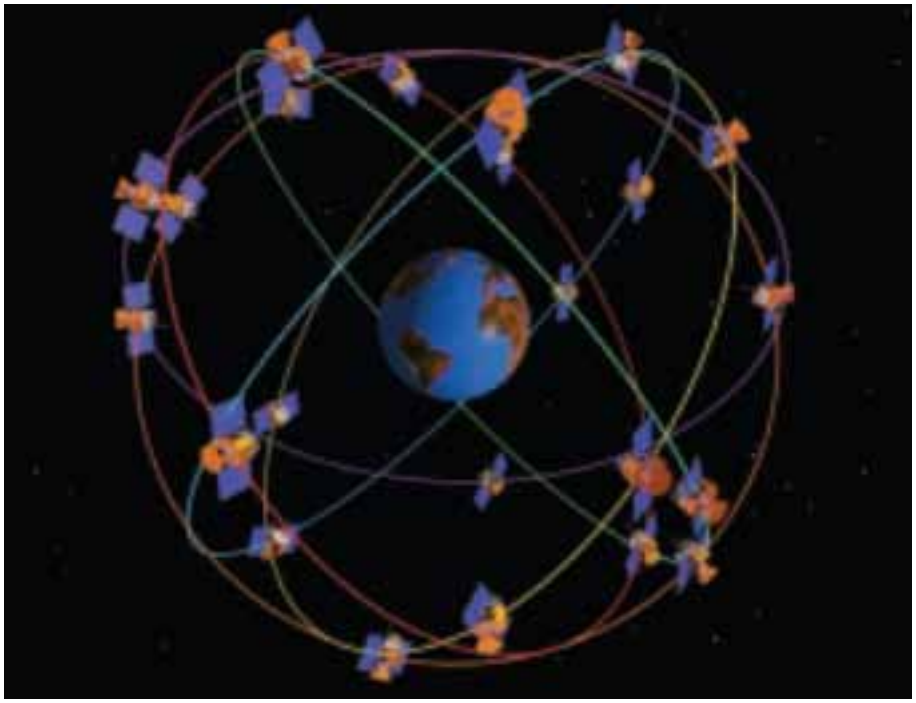
CONCEPTUAL APPROACH TO SO 8-16:

- Introduce the concept of limitations relating to physics of wave recordings, electronic instrumentation, data transmission, automatic and manual processing and storage.
- Point to limitations ito. access, sensor configuration, budget constraints. Provide brief guidelines for seismic monitoring practice.
- Estimate installation and operation costs.

The configuration of sensors is of fundamental importance to the performance of a network. Too few sensors will generally result in poor locations and inaccurate source parameters. Sensors placed only on one side of the rock volume of interest only capture a limited amount of detail of the seismic source. In this regard, it is advisable to think of seismic sources being 3-dimensional objects that need to be viewed from all sides to be fully analysed and understood.

INTERESTING INFO

Satellite navigation uses methods similar to seismic event location.



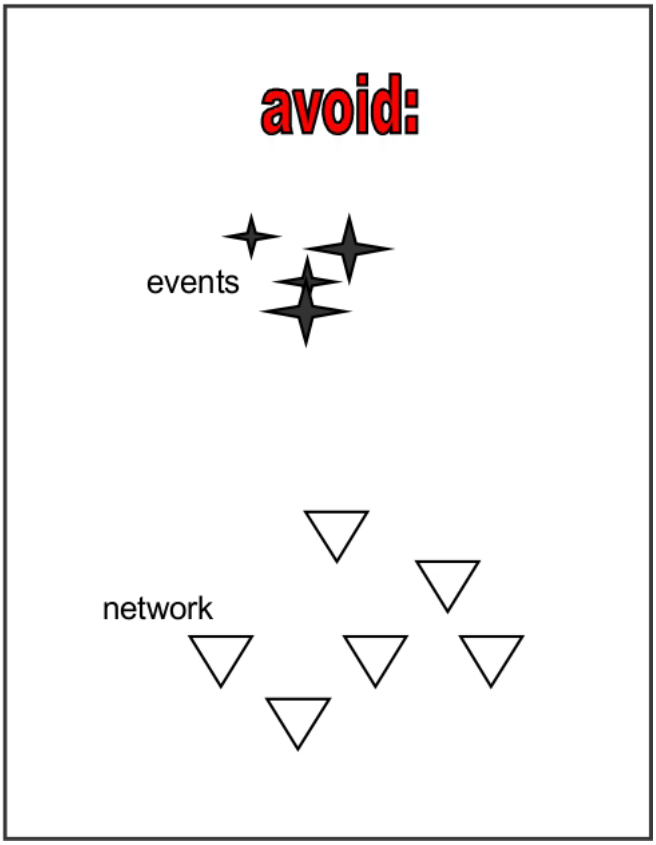
www.ams.org/mathmoments

Sensors arranged in a straight line cannot distinguish events on one side of the network from those on the opposite side. Similarly unfavourable is a configuration where the average distance between sites is small compared to the average distance between sites and events. In such a case the source-receiver travel time differences for the various sites are small and the location algorithm has difficulty deciding which of a set of location options is the correct one.

NOTE

Avoid a network where the sensors are placed outside the area of active seismic activity.

back



The sensitivity of a sensor set is determined by the number of sensors, their density and their position relative to the source volume. A higher density generally results in greater sensitivity, i.e. an increased ability of the network to detect small sources. Small events are more frequent than large ones, their numbers increasing exponentially with magnitude.

DEFINITION

Network sensitivity Mmin is defined as the magnitude above which all events are reliably recorded.

M_{min} is also called minimum magnitude or threshold of completion due to it marking the magnitude level above which the network will reliably record all events and deliver a complete data catalogue. As a general guideline, a sensor density of 0.5 per km^2 results in a sensitivity around M_{min}=0.7, with 2 sensors per km^2 M_{min}=0.0 and to reach M_{min}<-1 approximately 8 sensors are required per km^2.

Specialised software is available to model the expected performance of a planned network design prior to its deployment. This allows modifications in the planning stage where necessary. Location error and sensitivity models produce contour plots superimposed on mine plans (Figure 24) providing an estimate for each working place or any special area of interest. Some of the software tools allow 3D viewing of the models others are for a fixed depth or for the reef horizon. In all cases the following input parameter are required:

- minimum number of sensors triggered
- background noise-level
- pick error and velocity error
- sensor co-ordinates
- ground-motion relation PPV(M,d)

Figure 74: Sensitivity model for a 4-sensor layout

Figure 75 shows the stations of a mine network with varying sensor density in a plan view: fewer sensors in the shallow back area (top), more sensors in the deep areas currently being mined (bottom). Stations 1 and 42 are placed at shallow and deep levels in the shaft to increase the vertical extent of an otherwise flat network where most sensors are installed close to reef.

Where required, several sensors can be connected to one seismometer; several seismometers could be placed on one level, e.g. sites 43-45 and 37-40 above in Figure 25 below. This results in local high-sensitivity zones which may result in many blasting related events being recorded or the actual blasts themselves.

Figure 75: Mine seismic network with higher sensor density in the lower part of the mine (active mining).

The consequence of a flat network configuration is poor location accuracy in the direction normal to the network plane, i.e. vertical for most hard-rock tabular mines. With locations being based on travel times, events with the same source-receiver distance end up being placed the same distance into the hanging- and footwall.

back

In the case of sensors being close to reef, the location algorithm would arbitrarily throw events above and below reef (Figure 26 and Figure 27), hence the decision to create deep and shallow stations 1 and 42 in the case of the mine depicted in Figure 25.

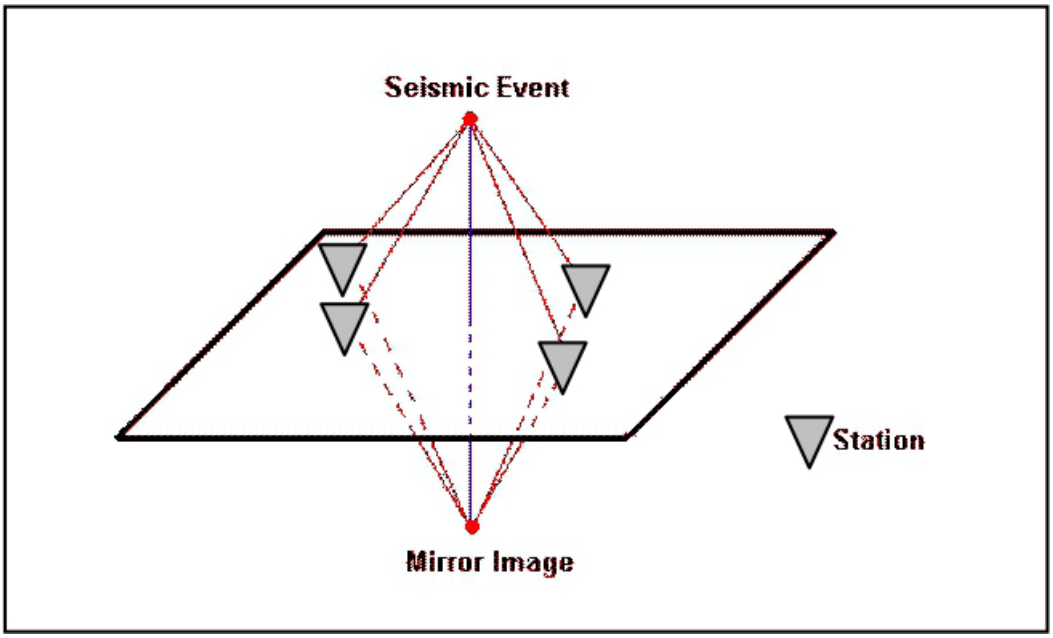
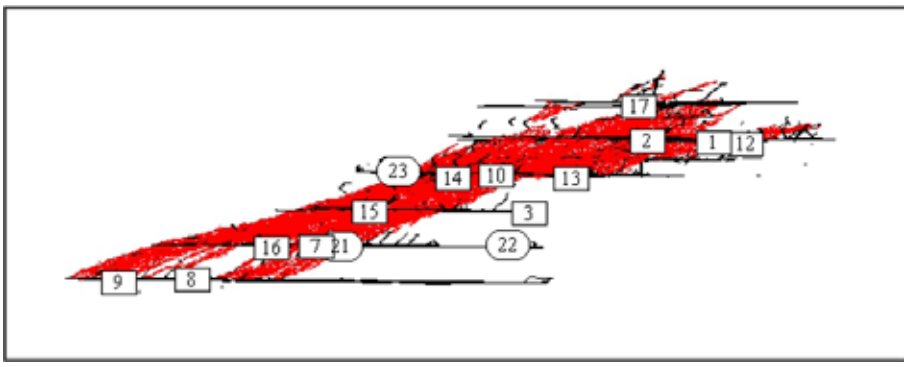


Figure 76: Hanging-wall event recorded by seismic stations near reef may be misplaced below reef.



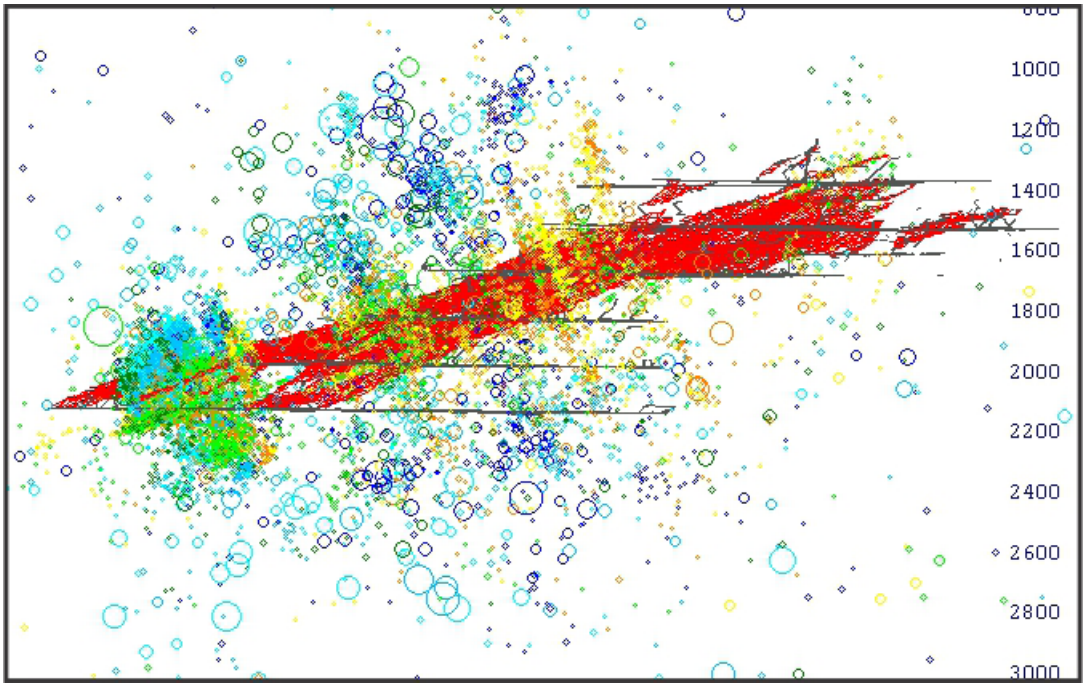


Figure 77: Flat sensor layout (a) and resulting vertical spread of events locations (b)

back

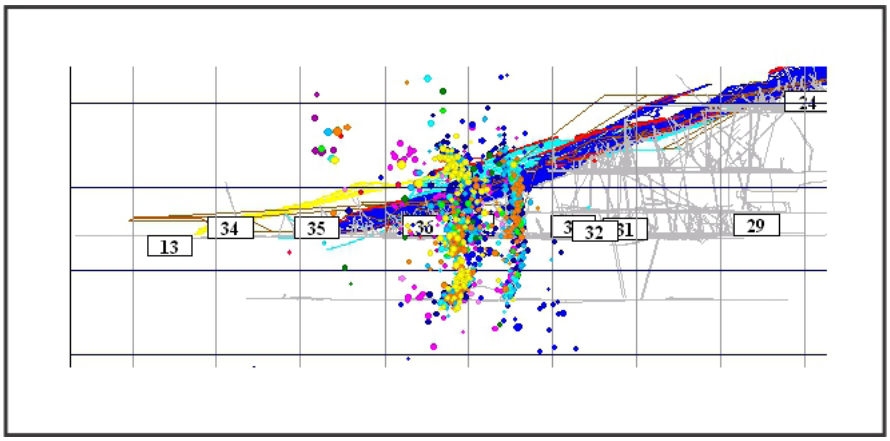


Figure 78: Artificial event locations cause by unfavourable sensor configuration

Small, blasting related events may contribute little useful information to assist seismic hazard estimation and the network design process should attempt to balance the need for detailed information with the danger of collecting (and later deleting) data that is irrelevant.

LIMITATIONS TO ACCURACY AND SENSITIVITY

Seismic networks have an upper and a lower sensitivity limit, their coverage is finite and they transmit only a certain band of frequencies and amplitudes. That means that out of all non-elastic deformation processes in the rock volume a system will only record those that happen fast enough to emit waves, have amplitudes that are not too strong and not too weak and are in the right frequency band. These are serious limitations especially since, in practice a certain percentage of stations is also faulty (roughly 10-30% at any given time) further limiting the system performance.

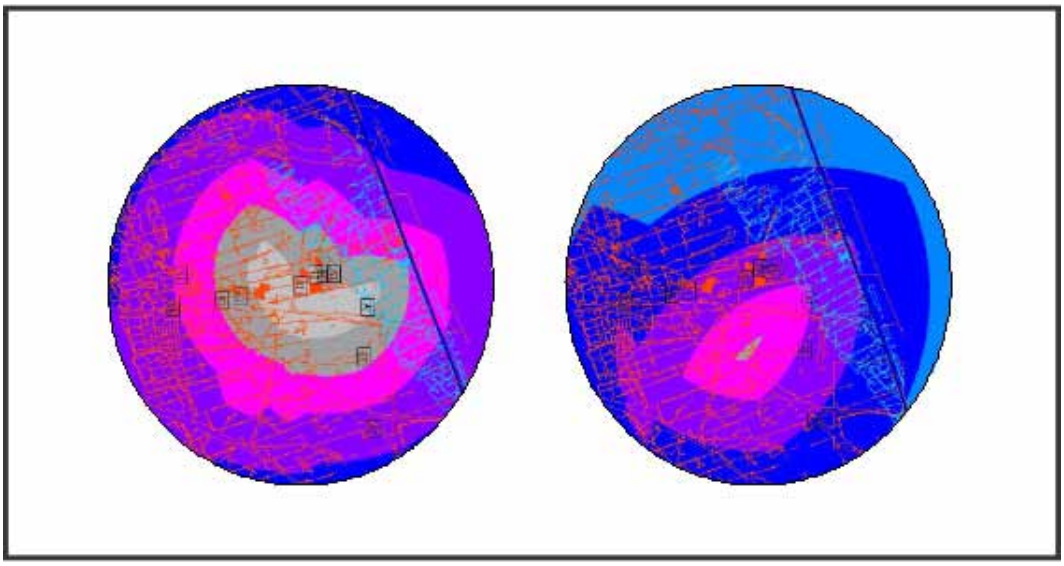


Figure 79: Nominal sensitivity, all sites (a) and healthy sites only (b)

An important physical limitation of location accuracy is the physical source size, which may range from tens to hundreds of metres for medium and large events. The accuracy of the hypocentral location can never be better than the physical source size of the event. The best possible location accuracy to be achieved with a seismic network is also limited by a number of random and systematic errors such as:

- wave velocities are known within approximately 5% (5m location error over a 100m source-receiver distance);
- deviation from the straight ray path estimated at <5% of hypocentral distance;
- P-wave arrival time picking is accurate within 2-3 samples (equivalent to 6–9m at 2kHz sampling rate);
- S-wave picking only half as accurate as P-picking;
- physical source size in the range of 5m to several 100m;
- unfavourable station configuration (lack of deep and shallow sites).

Most of these errors will occur in combination depending on the local network setup, geotechnical conditions and the quality of seismic data processing. Expressed as a percentage, under unfavourable conditions they may add up to 10-15% of hypocentral distance. In addition, the source parameters and locations discussed in preceding chapters are based on idealised conditions that are never truly met in a natural rock-mass (homogenous, elastic and isotropic medium; point source). Even basic variables such as seismic energy and moment are un-calibrated quantities calculated to represent a physical process, but the end user of seismic information should make allowances for inaccuracies.

In addition to the physical limits of recording, network performance is commonly affected by poor sensor installation, the deterioration of sensors over time, loss of power at a seismometer and interruptions in data communication between underground and surface. Sensor deterioration frequently manifests itself in the form of noise, random or periodic, and in the inability of the sensor to provide a linear signal response to an external ground vibration. On local mines, the replacement time for a failed sensor is in the range of 3-9 months, which impacts negatively on network efficiency.

As a result network audits show that only a percentage of licensed stations are actually on-line and also operational. The example in Figure 30 demonstrates the gap between planned capacity (licensed), implementation (installed) and return on investment, benefit to the mine (operational). Continued effort is required and significant resources are needed to keep this gap small.

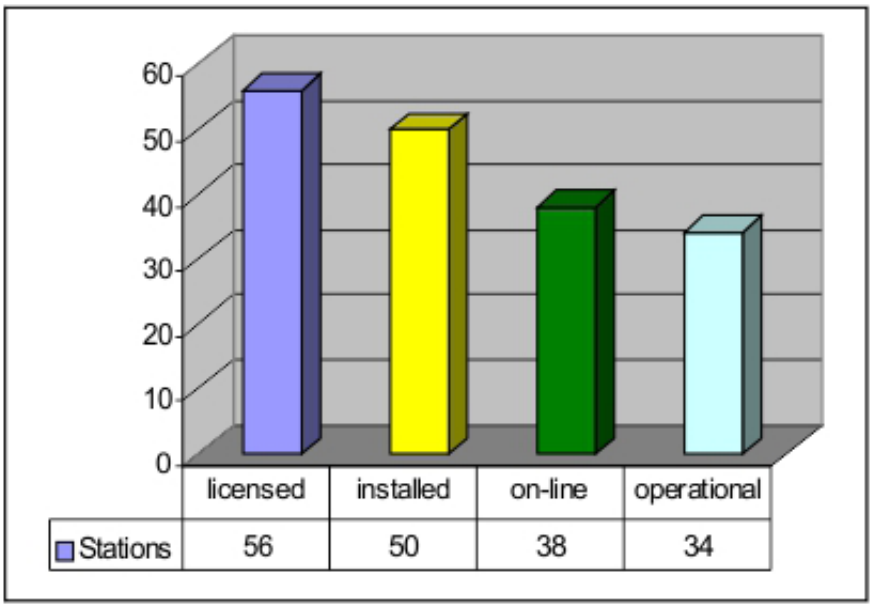


Figure 80: Licensed vs. installed, on-line and operational stations in a network

back

To be able to diagnose hard- and software problems in a network, a standard feature used by technicians is to test the functionality of a sensor independent from seismic ground motion. A test pulse is sent into the geophone that should produce a standard response by displacing the coil in the magnetic field and allowing it to swing freely until it returns to its resting position. If the geophone functions normally, it should display a curve similar to the one in Figure 31a where the signal shape indicates that the coil, after being released from its displaced position, drops through the zero-line, swings back and comes to rest (zero velocity). This is the response function of a healthy geophone set. In the case of a faulty sensor, Figure 81b, the response shows the lack of free swinging motion of the coil.

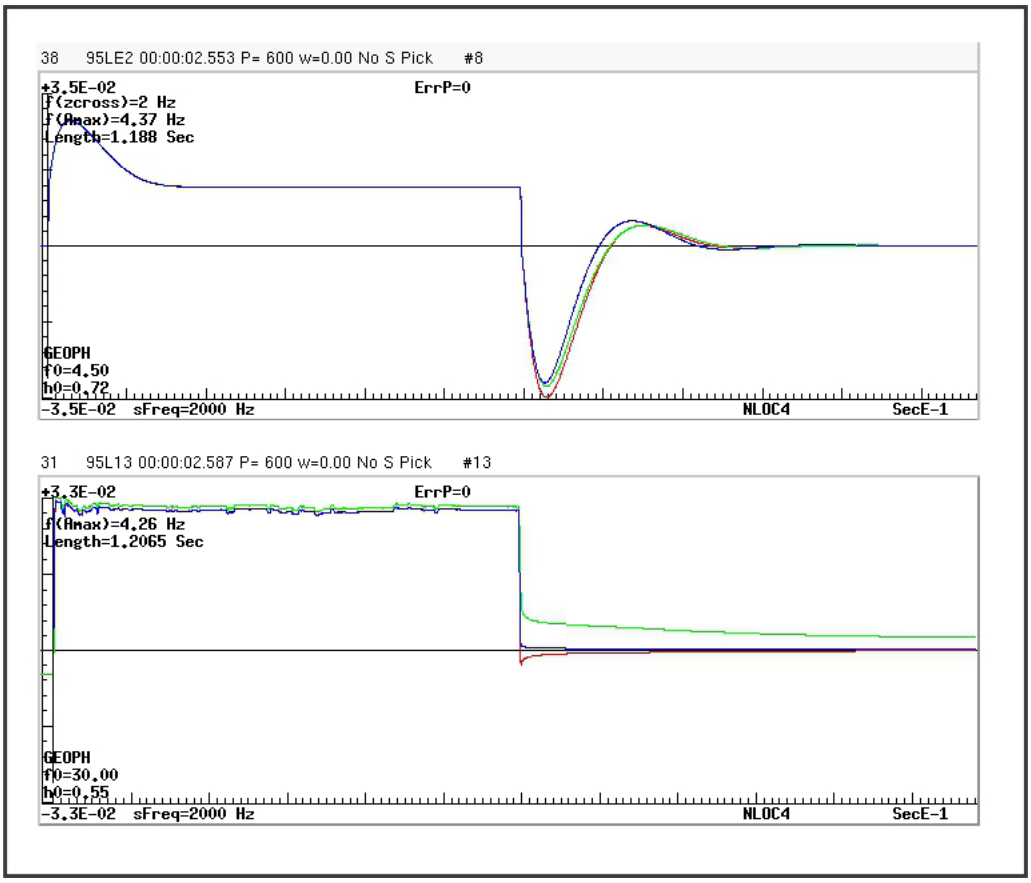


Figure 81: Test signal: a) healthy geophone, b) impaired geophone (no free swinging motion)

back

back

For individual components, the results of a test pulse / trigger is displayed

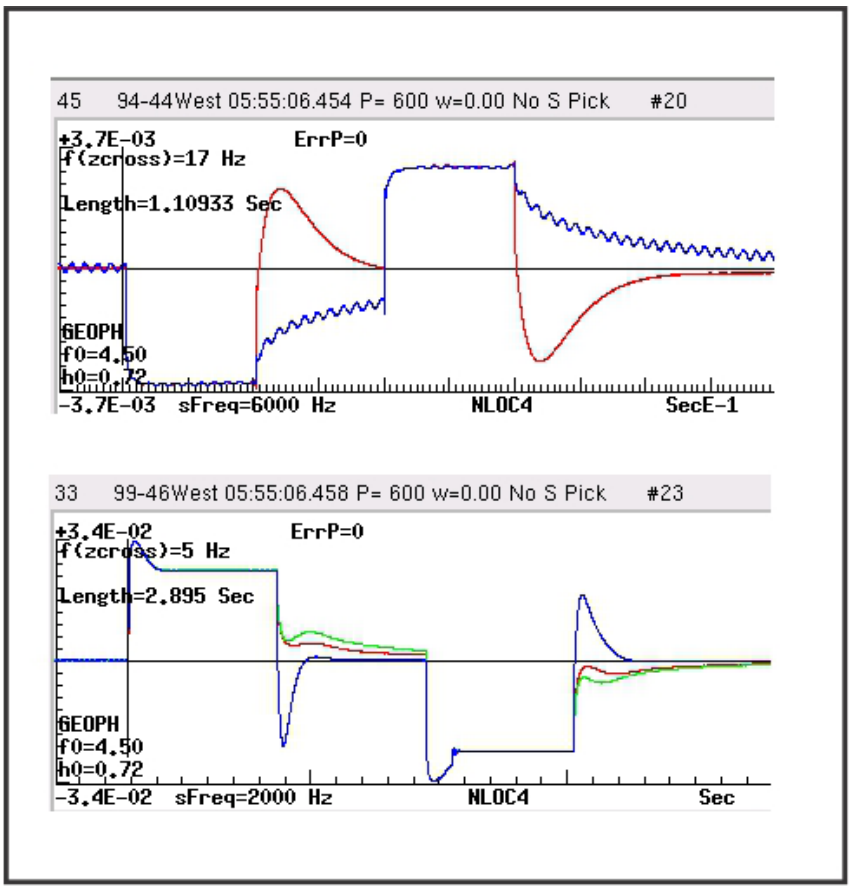


Figure 82:Test trigger (a) red=good, blue component=noisy and (b) blue=good, red and green faulty

Easily affected by poor sensor health is the accuracy of source parameters which rely on spectral analysis of wave recordings. Failure of sensors and recording equipment and the data processing steps following on the recording to accurately represent ground motion results in spurious seismic information: Considering the physical relation between seismic energy release and source size these should be positively correlated. In some cases they can be seen to be very poorly correlated especially towards the lower end of the energy scale (Figure 83a). The data seems to indicate a wide range of source volumes (Apparent Volume) for a given energy release. This type of scatter is usually caused by partly faulty sensors, too few seismograms and lenient data processing standards. Data analyses based on such a data set delivers results that should not necessarily be trusted.

back

back

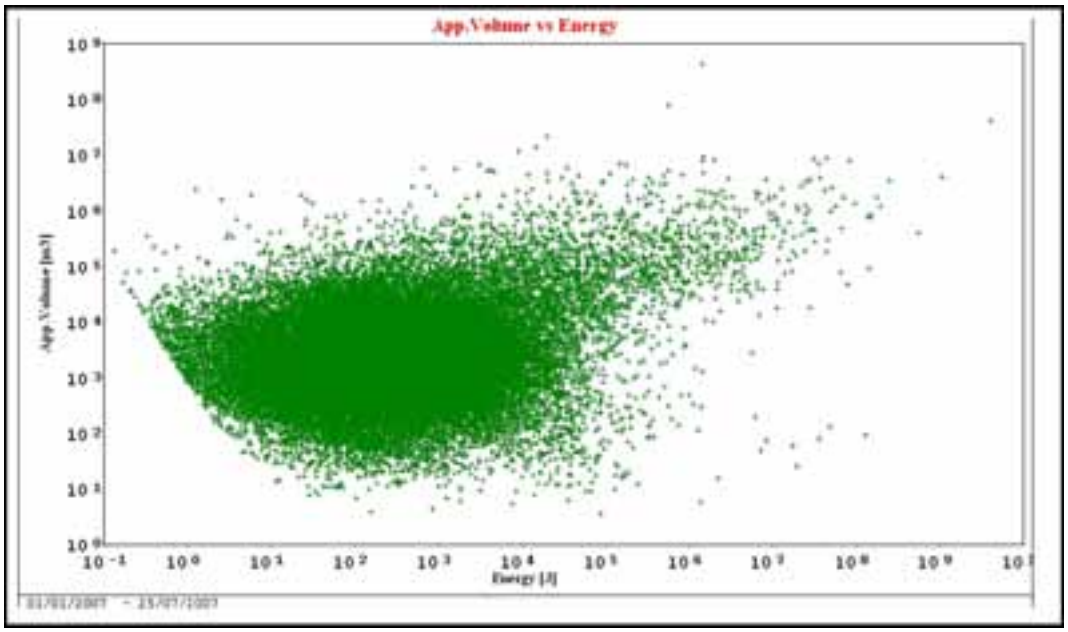


Figure 83: Small events (ellipse) showing poor correlation between seismic energy release and source volume

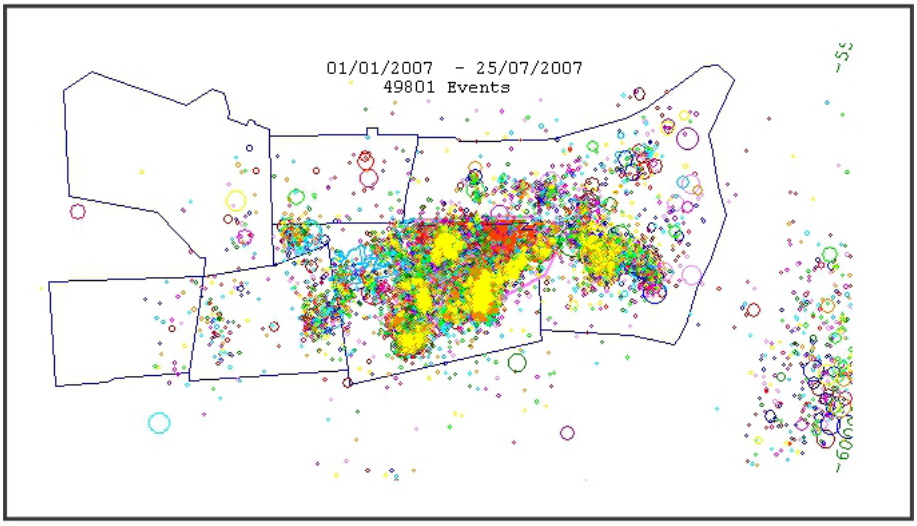


Figure 84: (a) Nearly 30% of recorded events are located outside this mine and

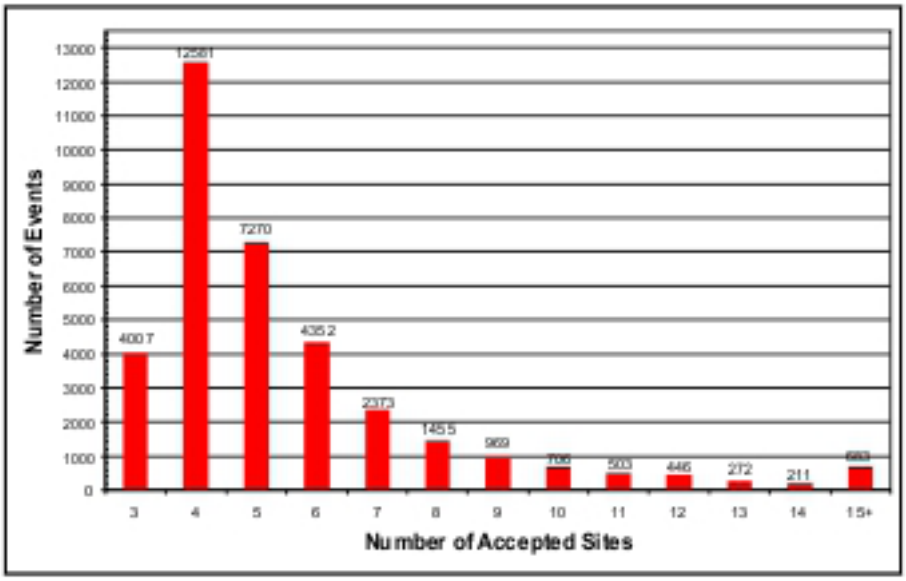


Figure 84: (b) the number of triggers in accepted events: approximately 2/3 of recorded events have fewer than 5 triggers

back

back

The quality of seismic data is usually enhanced where these guidelines are adhered to:

- conversion rate analogue-to-digital of the recorded signal is appropriate for the frequency content of the signal;
- reliable data transmission from underground;
- optimal setting of seismic system parameters such as trigger levels (Figure 34b), sampling frequency, filter settings and velocity model parameters;
- effective hard- and software maintenance;
- high quality manual data processing.

The final item refers to a lack of understanding of basic theoretical concepts of seismology on the side of data processors, the shortage of time available to process each event and the lack of quality assurance procedures implemented to ensure data quality. Common mistakes observed in manually-processed data are the wrong placement of arrival time picks, altogether missing picks and noisy or incomplete waveforms being accepted for source parameter calculation (see seismograms). The combined result of impaired sensors, unreliable data communication, sub-optimal settings of system parameters is illustrated by seismograms that show a range of problems which, in combination, prevent these from being used for data analysis (Figure s2-20).

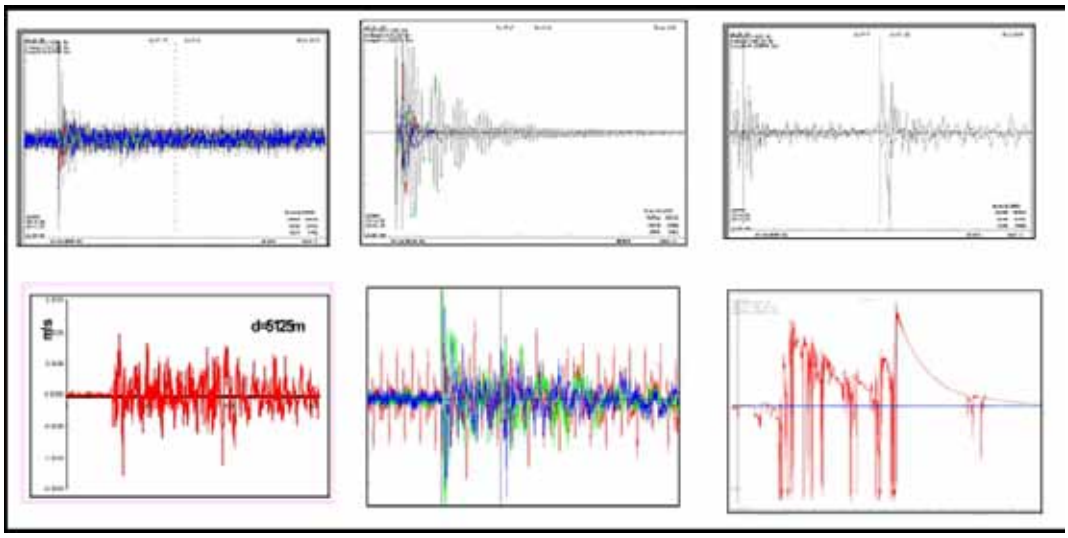


Figure 85: Sub-standard seismograms: (a) No clear S-arrival, b) ringing in green component, (c) incomplete (d) blast (e) periodic noise in red component (f) non-seismic signal (electric noise).

Once seismograms are processed, events are stored in data bases ready for data analysis methods to be applied. Large, seismically active gold mines record several hundred events daily, each with five or more valid seismograms. At monthly intervals these data are cleaned of rejected events and seismograms and backup copies are created on CD or DVD with at least one copy kept off site.

When seismic system software versions change, historic data should be recalculated to the new standard to ensure a consistent database. If this is not done, the source parameters of recent events are not easily and directly comparable to historic seismic events (Figure 36). Even basic parameters such as energy and magnitude may be affected by software changes in which case long-term trend analysis spanning across software upgrades may lead to wrong conclusions.

back

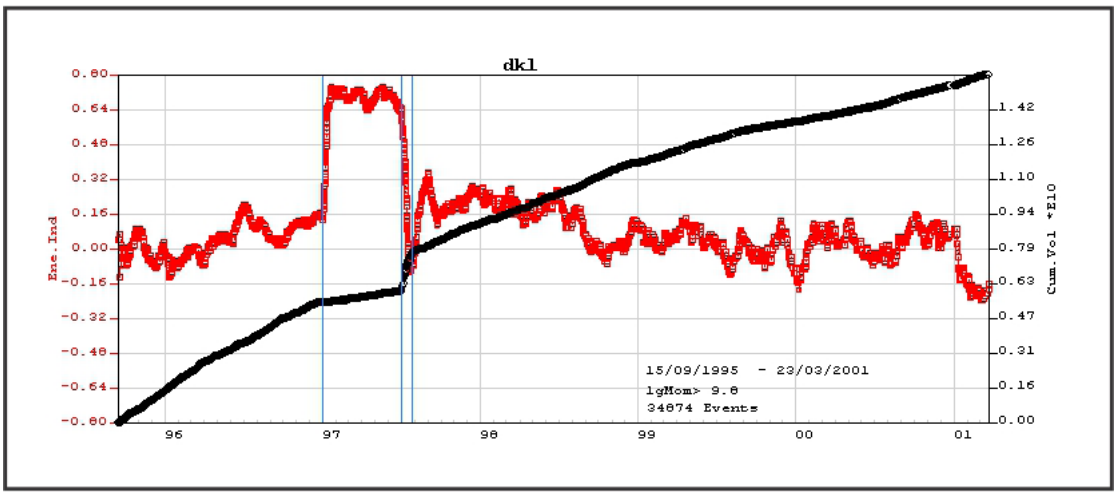


Figure 86:Changes to source parameters introduced by new system software in January and in June 1997

With respect to the quality of data analysis, the central question is one of expertise, i.e. of skills and knowledge to carry out suitable analysis methods that deliver relevant information to stakeholders. This must be done at intervals that are adjusted to the needs of those who receive seismic information, mainly rock engineering, production personnel and management. The preferred way to ensure the quality of seismic raw data and the suitable and speedy analysis of event parameters is to draw up a set of standard procedures for these functions and to regularly evaluate and review actual performance against the set standards. The "Guidelines, Standards and Best Practice for Seismic Hazard Assessment and Rockburst Risk Management" compiled and published by SIMRAC as one of the outputs of SIM050302 (Phase 2 Output 3, Durrheim et al, 2007) provide a detailed overview not only over the current practice but also recommended procedures and benchmarks for sound seismic monitoring practice.

NOTE

The question could be asked: What is good quality seismic data?

There is a perception that quality is quantifiable and that it can be measured according to benchmarks. This is correct in the sense that benchmarks can be defined for products and service to comply with. But these benchmarks depend on the purpose for which a product was designed and the expectations of those who will use it. In other words, quality is measured and expressed relative to expectations, not in absolute terms. The ISO 9000:200 standard on quality management uses the following definition (ISO (EN) 9000:2000 Quality Management Systems - Fundamentals and vocabulary quoted in Oakland JS, 2003):

DEFINITION

Quality:= Degree to which a set of inherent characteristics fulfills requirements.

Applied to seismic data, one first needs to decide on the purpose of seismic monitoring, the limits of accuracy required from the data, the frequency and format of reporting and other details before the data quality can be judged. To illustrate the principle, for a mine without appreciable seismic hazard and zero rockburst risk, a location error of 200m in plan for the monthly reporting of larger events may be acceptable. A network delivering these data would be considered sufficient and adequate for the purpose and the seismic data would be considered 'good'.

In the case of a high seismic hazard mine attempting interventions on a daily basis on the scale of panels or panel sets, as it is common in deep level gold mines, 200m location error would not be acceptable and the

back

data would be considered too inaccurate to be of use. This principle illustrates the importance of deciding on the objectives of monitoring before planning and installing monitoring equipment and of instituting periodic practice reviews to reassess the aims of seismic monitoring on a mine and to decide whether the network in operation is suitable.

REFERENCE

- R.J. Durrheim, A. Cichowicz, R. Ebrahim-Trollope, F. Essrich, O. Goldbach, L. Linzer, S.M. Spottiswoode, T. Stankiewicz and G. van Aswegen (2007): Guidelines, Standards and Best Practice for Seismic Hazard Assessment and Rockburst Risk Management, SIMRAC SIM050302
- JS Oakland (2003): Total Quality Management, 3rd ed, Butterworth & Heinemann

FUNDAMENTALS OF NUMERICAL MODELLING

LEARNING OUTCOME 6.4

THE CANDIDATE MUST BE ABLE TO DEMONSTRATE KNOWLEDGE AND UNDERSTANDING OF THE ABOVESUBJECT AREA BY BEING ABLE TO:

- Describe and explain the principles of the following numerical modelling techniques:
  - Finite Element Methods
  - Finite Difference Methods
  - Boundary Element Methods
  - Distinct Element Methods
  - Keyblock Methods
- Describe and explain for which rock engineering applications the above numerical modelling techniques are suitable
- Describe and explain what the limitations of the above numerical modelling techniques are in terms of the following aspects:
  - When that method should be used
  - Why that particular method should be used
  - How the results should be interpreted

LEARNING OUTCOME 6.4.1

INTRODUCTION

Numerical modelling is an analysis and prediction method often used to evaluate rock mass or support response prior to, during or after mining has been conducted. In all different methods included under the term 'numerical modelling', the modeller constructs or builds a model of the 'problem' or 'situation' he is faced with, allocates appropriate parameters to the model and evaluates the outcome of the model. All of this in an attempt to better understand or predict behaviour. Starfield (1986) actually suggests five benefits to be gained from model building in an attempt to solve a 'problem':

- Improved problem definition: Model building helps the modeller to define the 'problem' simply because it is difficult to build a

back

model that describes a problem that is not well defined. During the model building, the builder is therefore forced to define fully the problem to be modelled. For example: if a model is to be built that will describe the behaviour of rock around an ore pass, all the aspects that would affect this behaviour must be identified and defined. Should the model exclude for instance the presence of weak material layers, the predicted behaviour could be remote from the actual behaviour and the model therefore not appropriate to gain understanding or predicting behaviour.

- Organisation of thoughts: The modeller is forced to concentrate on important issues that play a role in the behaviour presented by the 'problem', assisting with an understanding of the 'problem' or issues that play a role in the 'problem' or 'situation'. It also assists the modeller in anticipating possible causes of or solutions to the 'problem' based on the logical thinking process that the modeller follows during the building of the model.
- Understanding of data: Model behaviour can be tested against real situations and data. A model that agrees with the real data in the area of concern is an appropriate model for the intended purpose. Back analysis is a process whereby models can be tested against known and measured data in an attempt to show appropriateness of the model.
- Communication and testing of the current state of understanding: The behaviour of the constructed model to different conditions and the monitoring of the model's response is a good method to establish the appropriateness of the model. Should the model response indicate 'impossible' behaviour, it indicates an error within the model, which can be the result of the model constructed or inadequate understanding of the 'problem' as it was included in the model.
- Prediction from the resulting model: Appropriate models can be used to make predictions about rock mass behaviour when subjected to conditions that have not yet been monitored in real situations. These prediction results can be classified as qualitative or quantitative. Qualitative predictions are provided when the models appropriately predict the impact of various conditions on behaviour. Quantitative predictions require the models to provide near precise and measurable behaviour. Reliable quantitative predictions are only possible if sufficient data exists and an understanding of the mechanics of the system is accurate. This is more often the case in engineering fields where there is a great deal of accurate and precise data relating to materials for which clear rules of behaviour exist (such as aeronautical engineering).

Holling (1978) classifies modelling in terms of 'level of understanding' and 'availability of trustworthy data'. The classification indicates four quadrants within which quadrant 3 is the ideal and quadrant 4 the worst-case scenarios. Models that fall within quadrant 4 represent disciplines in which both the understanding of the problems and the available data are limited, such as in rock engineering (where data is often limited and the mechanics of rock are not well understood), and where it is impossible to use modelling to make quantitative predictions.

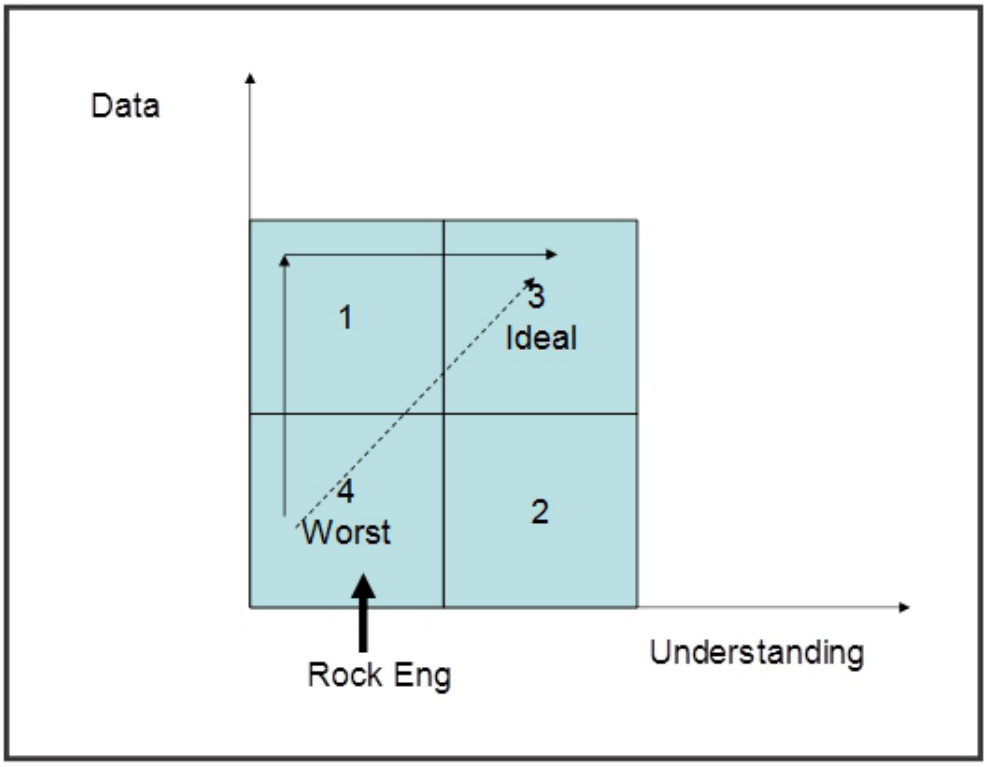


Figure 87:Holling's (1978) classification of modelling

The usual approach to solving rock engineering problems involves the collection of more data in an attempt to move from quadrant 4 to quadrant 3 (Figure 87, Holling's diagram – go from A to B via C). Other approaches suggest that it might be better to move directly from A to B by improving the understanding of the mechanics involved, while increasing the data collection. This can be achieved by building models that clearly describe the system behaviour and using the input required for these models to drive the increased data gathering process. This would suggest a move diagonally across the Hollings diagram from A to B.
Where good data already exists, it can be used to develop conceptual models. The conceptual models should drive the collection of further relevant data, while eliminating the collection of irrelevant data. The new data assists improvement of the conceptual models and the process is repeated.

In terms of practical modelling, it is important to ask a number of questions continually during the modelling process. For example:

1. Purpose:
   - Why am I building this model?
   - What do I hope to achieve?
2. Reference:
   - What am I modelling?
   - Is it what I intended to model?
3. Cost-effectiveness: :
   - Am I wasting time?
   - Is there a cheaper, faster way?

Occam's razor (Figure 88) is a conceptual model of the modelling process. William of Occam was a 14th century philosopher who suggested: "eliminate all unnecessary information relating to the problem that is being analysed" (Starfield et al.,1990).
It is important to eliminate only unnecessary data and not data required

back

for an appropriate analysis of the 'problem'. It should be applied at all stages of modelling, but especially during the planning stages.
The modeller often tends to include too much data, hoping that the inclusion of the data will improve the understanding of the system's behaviour and assuming that more data means increased model accuracy. In fact, the inclusion of too much data will increase the model complexity, time to run and cost, and will most often decrease the confidence in the results and increase confusion due to the large number of variables affecting the model's behaviour.
Exclusion of too much data may again lead to an oversimplified model that will not be able to assist in solving or understanding the 'problem', as the model does not represent the conditions present in the situation being modelled.

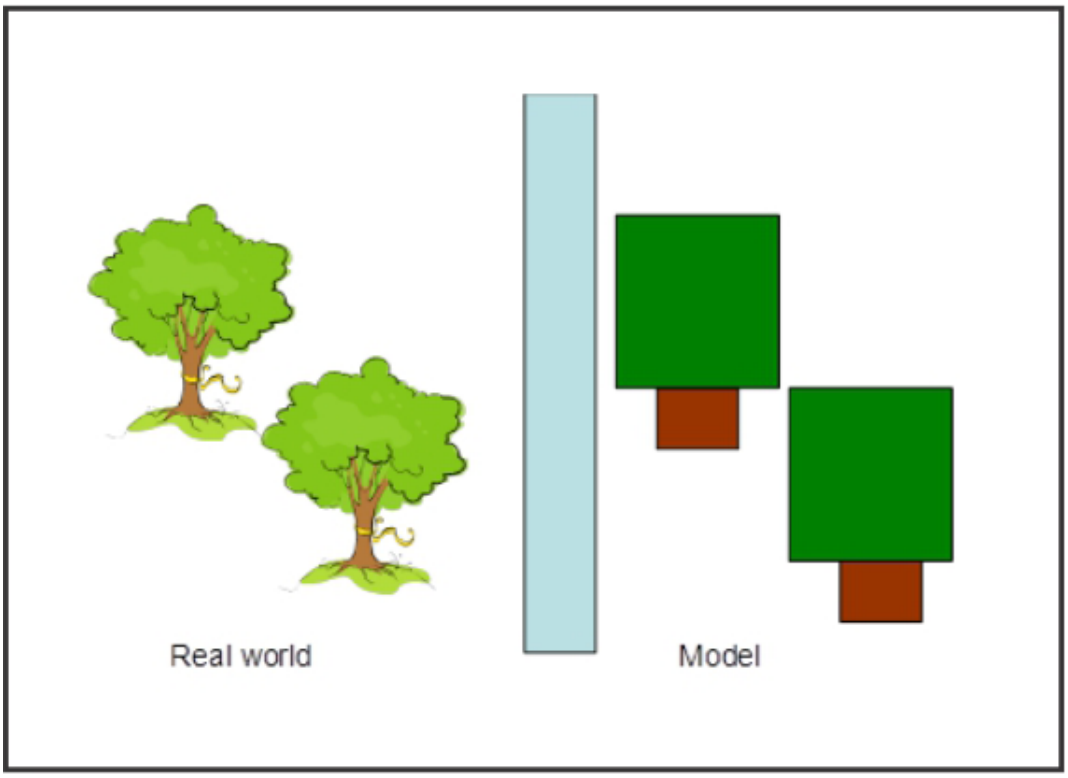


Figure 88: Occam's razor (Starfield, Smith & Bleloch, 1990)

6.4.2 Modelling Tools in Rock Engineering
The rock engineer has three principal types of models at his disposal. These are:

- Physical models;
- Analytical models;
- Numerical models.

LEARNING OUTCOME 6.4.2.1

PHYSICAL MODELS

Physical models come in many forms, but are currently rarely used in rock engineering, even though many basic concepts were developed making use of physical models in the past. Examples of these include the analysis of stress distributions around a hole in a stressed field using Perspex plates and the behaviour of rock pillars using rock samples in a press. Physical models are generally expensive to produce and require considerable time and effort in analysis.

back

LEARNING OUTCOME 6.4.2.2

ANALYTICAL OR CLOSED-FORM SOLUTIONS

There are many mathematical models in rock engineering that have exact solutions. Unlike the numerical solutions, the governing equations are solved exactly as they are prescribed and no numerical approximations are required. These solutions, however, apply only to very simple geometries that are rarely achieved in practice (such as a circular tunnel profile in an intact rock mass). Well-known examples of a very simple analytical solution are the Kirsch equations. These are useful in designing tunnel dimensions, spacings and tunnel support requirements underground, but they consider the tunnel to be a perfectly circular cylinder that continues along its axis in a straight line to infinity – an approximation that may or may not be valid or acceptable in different circumstances. Many underground problems can be adequately described using simple two-dimensional analytical solutions. Examples of these include:

- Beam deflection and beam stresses can be estimated in cantilever, built-in and freely supported beams (Ryder and Jager, Page 174);
- Backfill stress, ERR (energy release rate) and APS (average pillar stress) levels in deep level mines (Ryder and Jager, Page 173);
- Stress levels and ERR levels for different stope conditions (Salamon's 2-dimensional solutions – Ryder and Jager, Page 161).

LEARNING OUTCOME 6.4.2.3

NUMERICAL SIMULATION MODELS

A large number of computer codes are currently available to analyse the distribution of stress and rock movements around underground openings. These tools vary in complexity depending on the degree of material failure that is to be described:

- At one end, no failure is assumed to occur and modelling tasks are reduced to the solution of elasto-static or elasto-dynamic boundary value problems.
- Deviations of observed movements, i.e. deviations from those predicted by the boundary element codes, signal primarily that inelastic processes dominate the physical reality. Once inelastic failure is to be admitted, numerous additional considerations must be assessed before any model analysis is done. These considerations include the following:
  - The effect of so-called "path-dependency" where the sequence of executing mining / simulation steps changes the outcome of the model. Path dependency means that different simulation sequences will results in different results, even though the input parameters and the original model are exactly the same;
  - Failure mechanisms and the sequence of failure around the mining openings;
  - Constitutive descriptions of the rock mass;
  - Interaction of geological features such as faults and dykes.

LEARNING OUTCOME 6.4.3

A MODELLING METHODOLOGY

The following steps are suggested as an outline for a disciplined modelling methodology that can be applied to a wide range of problem-solving situations:

- Identify problem areas and the potential problems.
- Design a number of mining strategies to alleviate the potential problems.
- Define relevant design criteria.
- Develop a model that incorporates the necessary detail to assess the mining strategies in terms of the design criteria.
- Choose a relevant tool or tools to test the proposed mining strategies based on the model.
- Perform simulations on each strategy using the model.
- Specify the design criteria.
- Analyse the results of the simulations in terms of the specified design criteria.
- Select a mining strategy based on the analyses or propose new/modified mining strategies and go back to 4.

The methodology of using modelling for problem-solving should be appreciated and well understood.

- It is important to select the most appropriate tool for the job, i.e. to use a computer code that will be of the most help in solving the problem.
- Time and economic constraints should always be considered. A quick solution using a code that is available but not ideally suited may be a better course of action than waiting for the ideal programme. In general, it is best to use the simplest tool for the job. Computer codes that require few parameters are generally easier to control from an intellectual point of view than ones that require multiple (and often interdependent) parameters.
- If the problem concerns a local area of the mine, then it is unnecessary to digitise distant stope outlines accurately; it is only necessary to get a reasonable approximation of the contribution of the distant mining to the local loading conditions.
- The question of model resolution also occurs when assigning values to model parameters. Here the resolution is controlled by the underlying assumptions of the numerical formulation used in the code. Giving extremely accurate values for the elastic constants in a linear elastic boundary element is futile, as the approximations made in the representation of the rock mass are enormous by comparison. It is a question of balancing the input with the tool being used. An educated guess at the stiffness of the flawed rock mass as a whole is generally more appropriate than an accurate determination of the elastic modulus based on countless tests of unflawed rock specimens.
- It is important to recognise the limitations of the tool or tools being used. If the tool cannot address the problem at hand, then avoid using it. For example, continuum codes may be well suited to analyse hangingwall damage in an advancing stope, but they are of very limited value if the problem is one of hangingwall stability. Hangingwalls that are more damaged (i.e. more fractured) may be more stable than hangingwalls that are less damaged. This is observed in practice, as shallow-level mine stopes are often more prone to large falls of ground than deep-level stopes.

Modelling is not a skill that human beings acquire naturally and, generally, good conscious modelling is something that must be taught. Human beings are good at modelling unconsciously (they have to be to survive), but they are particularly poor at conscious model building.
Models should be built for specific purposes and used as intellectual tools to solve well-defined engineering and scientific problems. Modelling is about problem-solving. Each problem is specific and should be treated as such, while relying heavily on past experience and current knowledge.

LEARNING OUTCOME 6.4.4

NUMERICAL MODELLING APPROACHES

1. **Elastic vs nonlinear:** An elastic model will often allow you to determine the boundaries within which the real results lie, before inelastic analysis is attempted. Since non-linear models are often difficult to calibrate and interpret, it is important to question the model accuracy, its ability to reproduce known outcomes and its deformation pattern stability at various grid sizes before applying the model in the analysis. Depending on the 'problem' to be analysed and the outcome expected, a decision on the appropriate approach is made. If a comparison of the depth of fracturing into the pillar walls caused by different mining layouts is required, an elastic model could be appropriate and the results could be analysed to indicate the probability of fracturing, the extent of stress increase, and even the application of potential for failure using the Hoek-Brown failure criterion. However, if an accurate estimation of the depth of fracturing into the pillar walls is required, the use of a non-linear or inelastic model is required.
2. **Two dimensions vs three dimensions:** The underground environment is a three-dimensional environment. In some cases, a two-dimensional analysis is physically incorrect (wave propagation), but may still be useful. Three-dimensional analyses may never just be discarded in favour of less time-consuming and easier two-dimensional analyses and when a two-dimensional programme is to be used, it must be justified. In many cases, the application of plane-strain and plane-stress concepts can be used to justify the use of two-dimensional models.
3. **Static vs Dynamic:** Most commonly used codes are static in that they allow stress changes and the determination of equilibrium in steps. Executing dynamic analyses, the effects of damping and wave dispersion must be evaluated while grid sizes must allow the suitable presentation of waveforms.
4. **Deterministic vs Probabilistic:** Since most models are based on deterministic mechanics (where results will be unique), it is important to test models for sensitivity to input parameters or to present results in a probabilistic form.
5. **Small strain vs large strain:** Most codes assume that the final distorted shape of the excavation does not affect the calculated stress distribution. However, very close to the failed boundaries around underground openings, this does not hold true, but in cases where fault slip or stope closures are investigated, this assumption is not a major concern.

NUMERICAL MODELLING TECHNIQUES

The different modelling techniques are briefly described in the paragraphs below. Although some appreciation of the techniques is provided in this section, it is advised to study the references given for more information and detail.

LEARNING OUTCOME 6.4.5.1

BOUNDARY ELEMENT TECHNIQUES

The Boundary element technique creates elements along the boundary of the excavations included into the model only, instead of including elements within the complete model. This allows for fast solution / calculation times due to a smaller number of elements and one-dimensional linear elements. BE codes utilise either the Flamant or Kelvin solutions to solve models. The Flamant solution suggests that stress is not dependent on elastic constants E and v, while deformation is. The Kelvin solution accepts that stress and deformation (strain) are both dependent on the elastic constants E and v.

BE codes solve stresses and deformation in the boundary elements and then use those results to calculate the stresses and deformations at other points in the model in areas where grids / windows were placed by the user. Only results on these grids / windows are calculated reducing calculation times significantly.

The accuracy of the results depends on the size of the boundary elements created. Large elements will reduce accuracy, while too small elements may not provide any additional accuracy and only increase calculation time. The appropriate selection of the boundary element sizes for each specific model is critical to ensure adequate accuracy. If you are interested in the average pillar stress in a 3m x 3m pillar, an element size of 10m will obviously provide inaccurate results since the element will include the pillar and some mined-out area. With element sizes of approximately 1.0m, nine elements will fit onto the pillar and will be able to provide good average values.

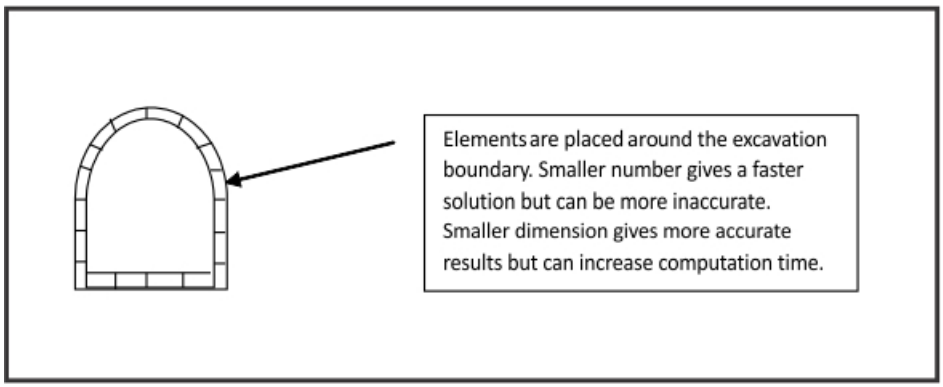


Figure 89: Boundary elements

The displacement discontinuity is a special type of boundary element. It indicates that the displacement is discontinuous across the element, such as displacement on a fault plane where the BE model does not allow the physical displacement along the fault plane, but the DD elements can allow different displacements on both sides of the fault. In a two-dimensional model, the DD are one-dimensional and in a 3D model, they are two-dimensional, simplifying the calculations associated with these elements.

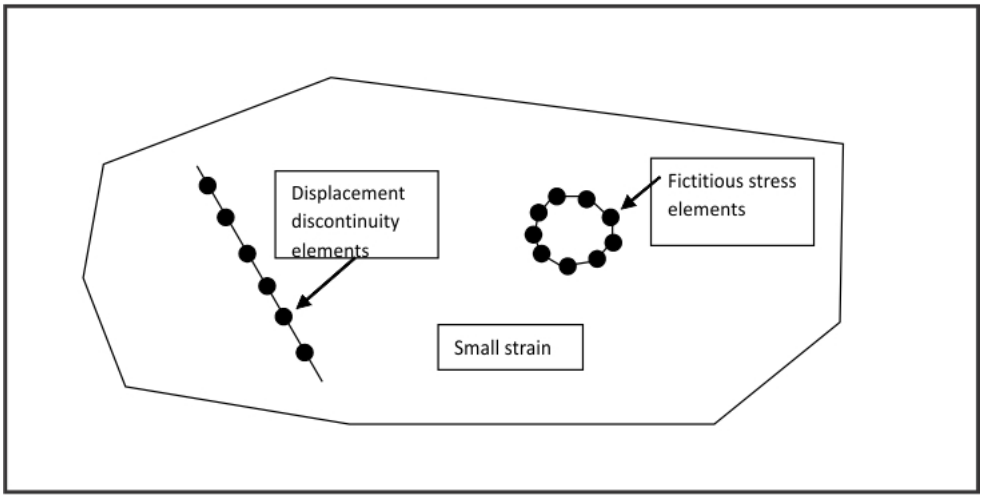


Figure 90: DD and FF elements

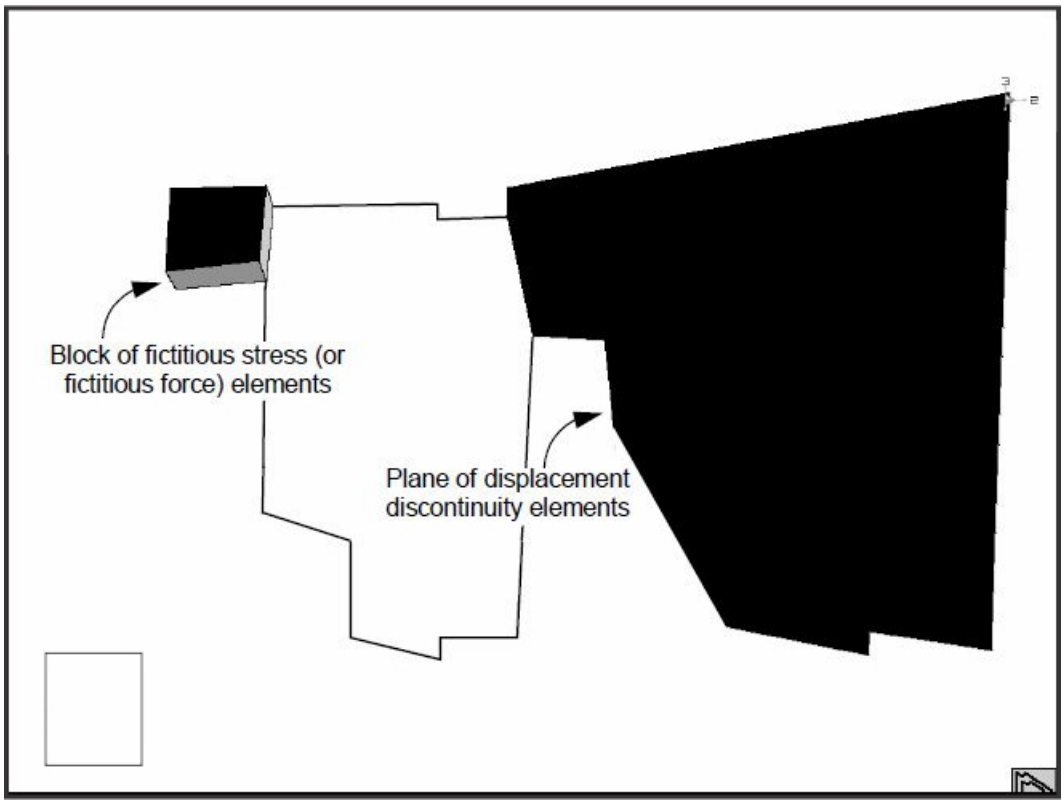


Figure 91: Fictitious Force and Displacement Discontinuity elements

Fictitious force / stress elements allow the existence of un-equal stresses to either side of the elements, such as on the boundary of an excavation. Boundary element codes are used extensively in the mining industry due to the relative simplicity of the codes, low costs and fast computing times of these codes. A large number of these codes exist and include three-dimensional codes such as Minsim, Besol/MS and Map3D and two-dimensional codes such as MINAP and DIGS.

back

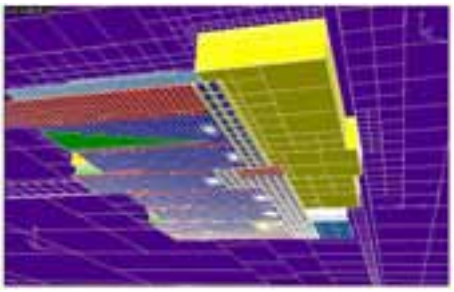
Dimension	Code	Results possible	Environment it can be applied to	Analyses of results (note examples at end of the outcome)
Semi and fully three-dimensional codes	Minsim	Analyse stresses, displacements and strains at any position, on and off-reef, within the model. Various stresses, strains and displacements can be requested from the model and used within design criteria such as ESS, APS, and RCF.	Mining layouts associated with tabular excavations in a homogenous, linear elastic rock mass. They can simulate several mining steps and can be used to back analyse previous mining conducted or predict changes to be expected with planned mining. Tunnels and shafts cannot be constructed within the model but results can be determined at the planned tunnel positions.	The assumed linear elastic rock mass must be kept in mind when results are analysed. Results can be graphed or dumped into excel for manipulation into average, minimum and maximum values, standard deviations etc. It is suggested that changes in values should be used to indicate risk rather than absolute values, even though absolute values can be used if the assumptions are kept in mind.
	Besol/ MS			
	Map3D		Different rock materials can be included in the model allowing a form of in-homogeneity but linear elastic properties remain a basic assumption. Mining layouts that include all of the above (Minsim) as well as massive mining layouts where ore bodies are, are not tabular, but have massive irregular shapes. Fully three-dimensional layouts can be constructed and analysed with all excavations (tunnels and shafts) in full three dimensions.	
2-Dimensional codes	MINAP	Analyse stresses, strains and displacements associated with excavations under plane strain.	Mining layouts associated with tabular excavations in a homogeneous, linear elastic rock mass. If the impact of other mining in the area on the excavation is not critical, or is known, the model can be used to provide a quick indication of possible behaviour. Can also be used to test variables before more complex simulations start.	Results should be used comparatively or as an indication of possible behaviour. If used with a three-dimensional code where the stress levels due to all mining can be included, results can be more accurate.
	DIGS	All of the MINAP results plus the development of fracturing under high stress levels.	Fracture growth and patterns can be studied ahead of highly stressed faces or stressed pillars.	Results are viewed graphically showing fracture patterns and growth in two dimensions.

Table 11: The application of boundary element codes

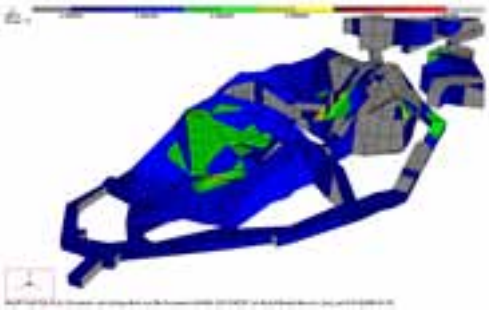
REFERENCE

- JPage 341, Jager A.J. and Ryder J.A., Handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 2002

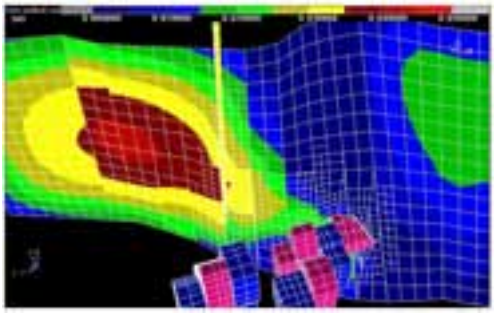
back



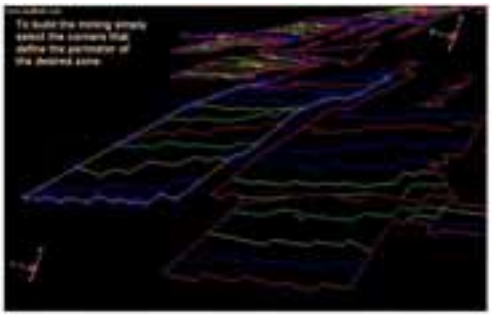
Coal mine longwall face



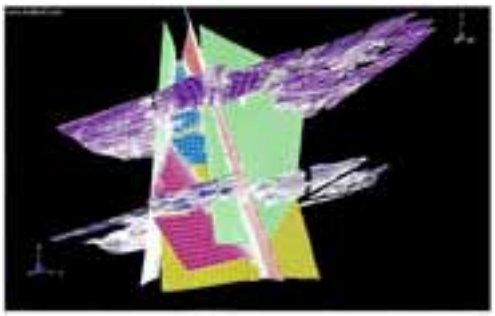
Massive ore body



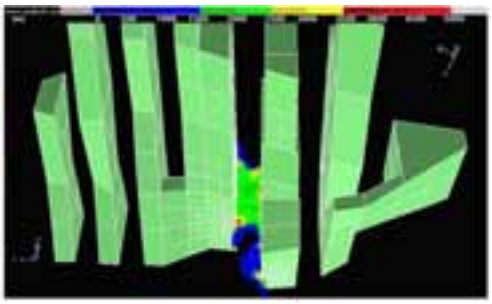
Massive mine with faulted rock mass



Longwall tabular gold mine layout



Multi-reef conditions on tabular ore bodies with substantial faulting



Massive ore body with pillars

Examples of simulations using Map3D

LEARNING OUTCOME 6.4.5.2

FINITE ELEMENT TECHNIQUES

Finite element solutions are codes where all results at all points in the model are calculated. The model creates a mesh (grid) throughout the whole modelled area. These grids or mesh can have various shapes and orientations and impact on the accuracy of the model. Too large mesh sizes will reduce accuracy as will poorly-shaped mesh. The aim is to select appropriate size and shape mesh in these solutions (i.e. do not have small and large triangular mesh next to each other in areas where results are important, but ensure relatively similar sizes).

For good solution accuracy, it is critical to have a fine (small size) mesh in the area where stress gradients are possible. The codes solve the elements, but communicate to the next element via the nodes.

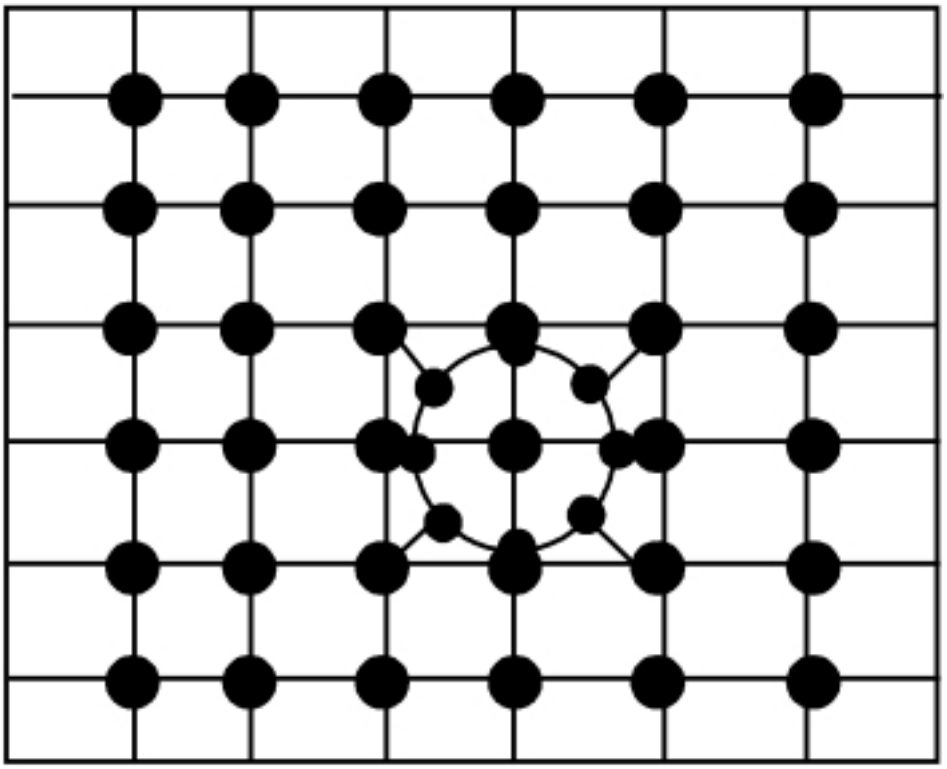


Figure 92: Finite elements

Finite element codes apply the relationship between force, stiffness and displacement (F=kd) in its calculations, where k = stiffness and d = displacement. The stiffness is a function of the mining geometry, stress / strain relationship and the strain / displacement relationship. Determining the stiffness and using applied forces, the displacement can be calculated, allowing the determination of strain and then stress. These calculations are relatively simple and therefore fast to execute, especially since matrix calculations are utilised.

Since calculations are done for each element, elements can have different properties, failure criteria, strengths etc, allowing non-homogeneity into the model.

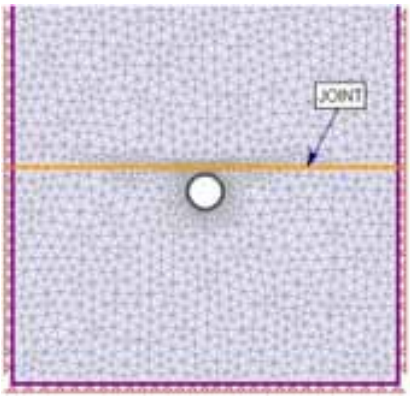
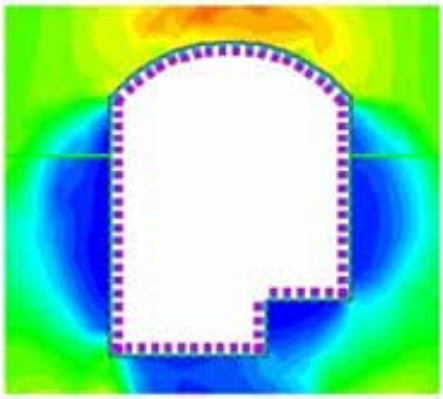
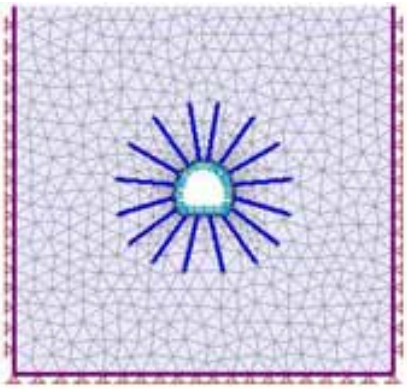
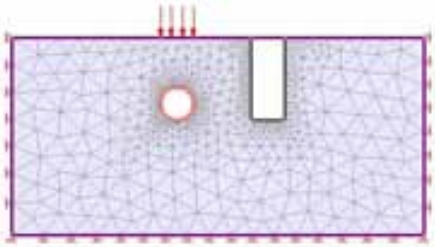
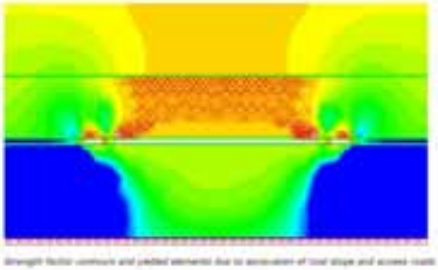
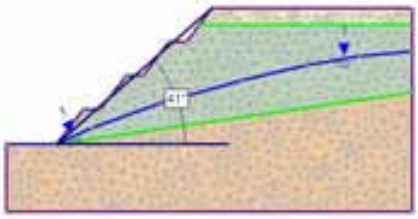
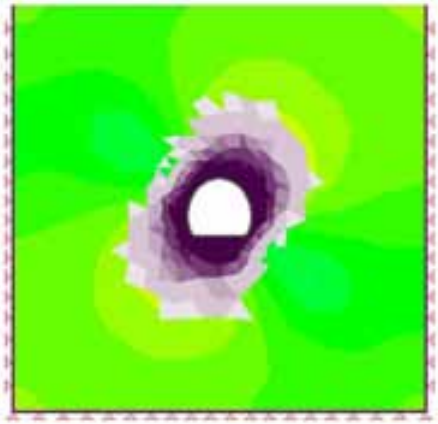
back

Dimension	Code	Results possible	Environment it can be applied to	Analyses of results (note examples at end of the outcome)
Two-dimen-sional codes	Phase 2	Analyse stresses, strains and displace-ments associ-ated with excavations under plane strain. Can also evalu-ate plastic and tensile failure.	Mining tunnels and excavations associated with underground rock mass that can consist of various material types and can be elastic or assume plas-tic failure. Mining steps can also be simulated. Surface layouts such as trenches, slots or slopes can also be evalu-ated for stability. Support units and areal cov-erage support such as shotcrete can be applied and evaluated.	Results are evaluated using graphs (such as stress along a line or load within a tendon) or contour prints from the screen. Measure-ments can be made on-screen to estimate tensile zone heights etc.

Table 12: The application of finite element codes

back

back

 Excavations in rock mass with prominent joints	 Excavations with non-standard dimensions and shapes, also allowing for the placement of composite support such as shotcrete
 Tendon support installed into the excavation walls can be included.	 Trenches from surface and the presence of non-standard loading conditions on underground tunnels can be catered for.
 Plastic failure of the rock mass can be indicated by referring to 'yield' or 'tension failure' points.	 Slope stability can be analysed.
 Failure, using the yield zone probability can be estimated.	

Example of simulations using Phase 2

back

LEARNING OUTCOME 6.4.5.3

FINITE DIFFERENCE TECHNIQUES

In the finite difference model (such as the FLAC code), grids are created throughout the model similar to the mesh generated in finite element codes. These codes are based on a cycle of calculations, performed for each grid point in the model.

Before an excavation is created and the model is in equilibrium, the acceleration experienced at the nodes is zero and therefore the sum of all the forces in the model is zero. The counter situation suggests that if the sum of the forces is not zero, there would be a non-zero acceleration in some direction.

When an excavation is created, the equilibrium is disturbed and calculations are required to establish the new state of stress equilibrium. These calculations are based on Newton's Law F = mass x acceleration (or rather a = F/m) and is done by integrating acceleration to velocity and velocity to displacement, allowing the calculation of strain, then stress and the calculation of the force = stress x area. Once this new force has been determined, the calculation process is repeated until equilibrium is reached. The calculation can be stopped at any point during these cycles and restarted again since the calculation is in 'time' steps (cycle of calculation). This allows the code to model non-linear behaviour.

DEFINITION

FLAC – fast le grangian analysis of continua

The accuracy of the model depends on the distances between node points, which are a function of the mesh / grid sizes. Calculation times increase with smaller grids and therefore more nodes.
Small and large strains can be modelled. The nodes will physically move in the model. Grid points / nodes are entered into the model and the solutions are determined for the nodes, rather than for the elements between nodes indicating that the node layout will determine the quality of the results.

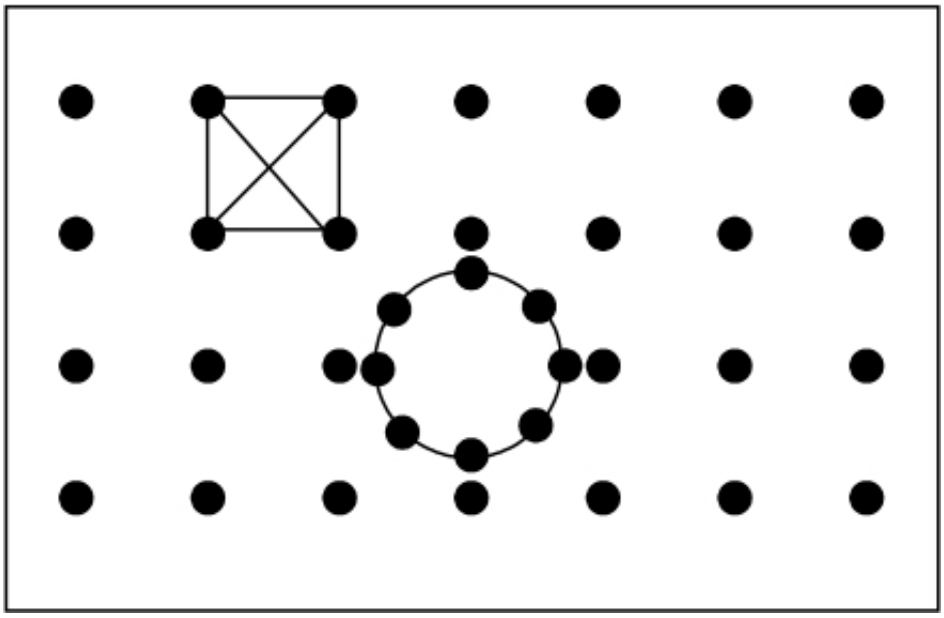


Figure 93: Finite difference

back

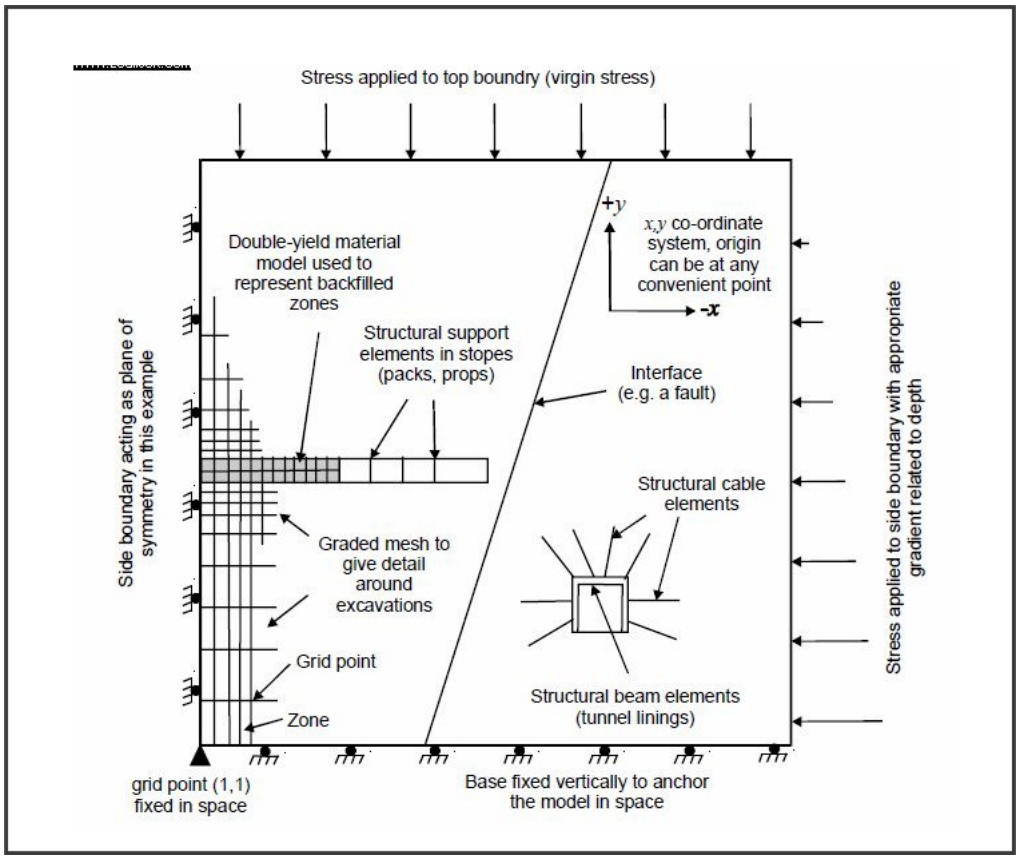


Figure 94: FLAC models

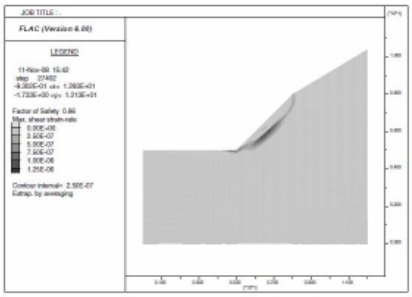
back

back

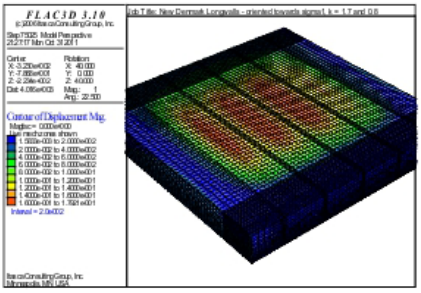
Dimen-sion	Code	Results possible	Environment it can be applied to	Analyses of results (note examples at end of the outcome)
Three-dimen-sional codes	Flac 3D	Analyse stresses, displace-ments and strains at any position, on and off-reef, within the model. Various stresses, strains and displace-ments can be request-ed from the model and used within design crite-ria such as ESS, APS, and RCF.	Mining layouts associated with tabular and mas-sive excavations in rock mass that can consist of vari-ous materials. It can simulate sev-eral mining steps and can be used to back analyse previous mining conducted or pre-dict changes to be expected with planned mining. Tunnels and shafts can be constructed within the model.	Results can be drafted along certain lines or areas and can be averaged, graphed etc to indicate trends and values.
Two-dimen-sional codes	Flac	Analyse stresses, strains and displace-ments associated with excava-tions under plane strain.	Mining tunnels and excavations associated with underground rock mass that can consist of various materials and that can be presented in a simple layout.	

Table 13: The application of finite difference codes

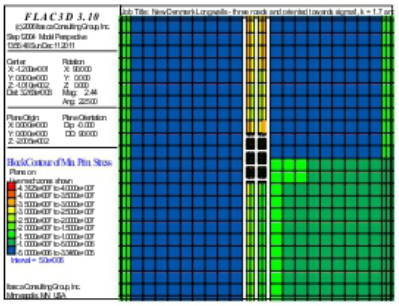
back



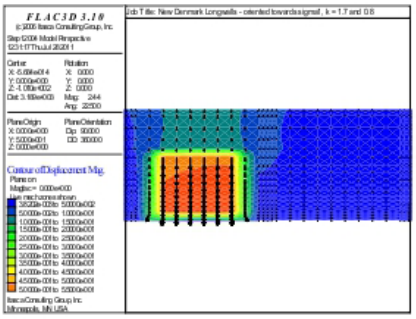
Slope stability using Flacslope or Flac 2D



Surface subsidence using FLAC3D



Pillar stress levels between Coal Longwalls.



Roof / hangingwall behaviour above a series of longwalls.

Example of simulations using FLAC

LEARNING OUTCOME 6.4.5.4

DISTINCT ELEMENT

DEFINITION

UDEC – Universal distinct element code

These codes deal with blocks created by joints and not with the joints. Each block formed is an element that is in contact with the next block / element where the contact is expressed in terms of stiffness values. Blocks do not have to deform, but can remain rigid and behave according to the contact rules or laws set out in the code. Laws of motion are applied to the blocks that have the freedom to move in x or y directions, including rotation of the block. Hard (stiff) contact rules allow the transfer of energy between blocks, while soft rules do not transfer energy, but result in overlapping of the blocks. Rounding of the block corners prevents errors due to undefined force and infinite contact stress at sharp corners. Several constitutive laws can be applied, such as Mohr-Coulomb or Linear Elasticity.

back

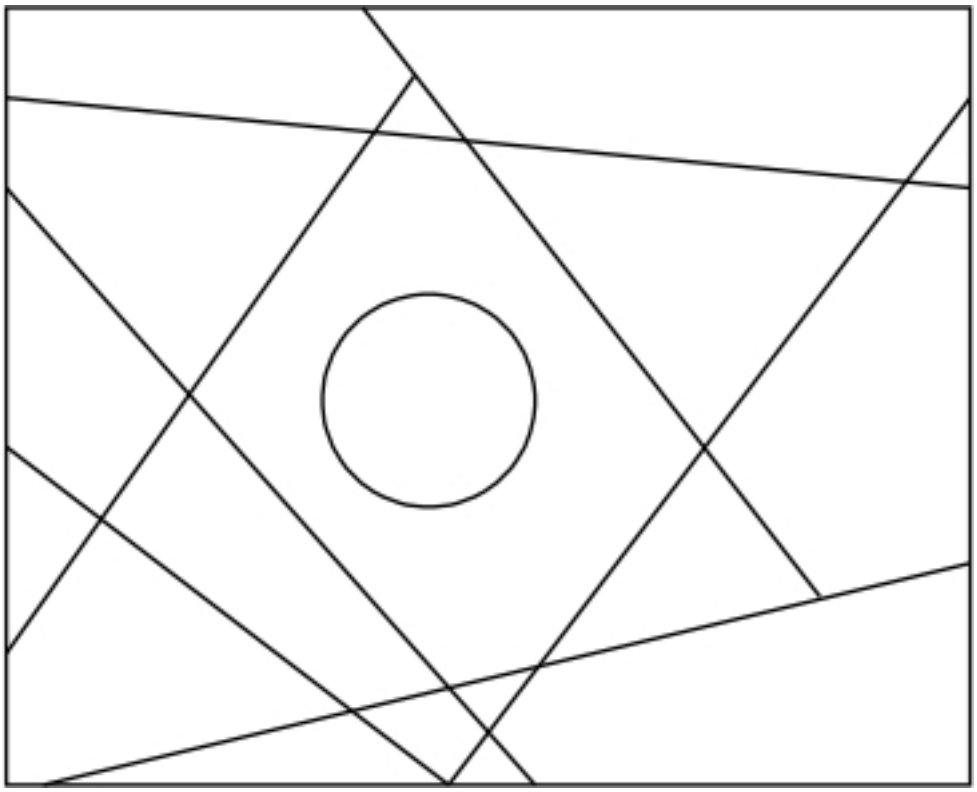
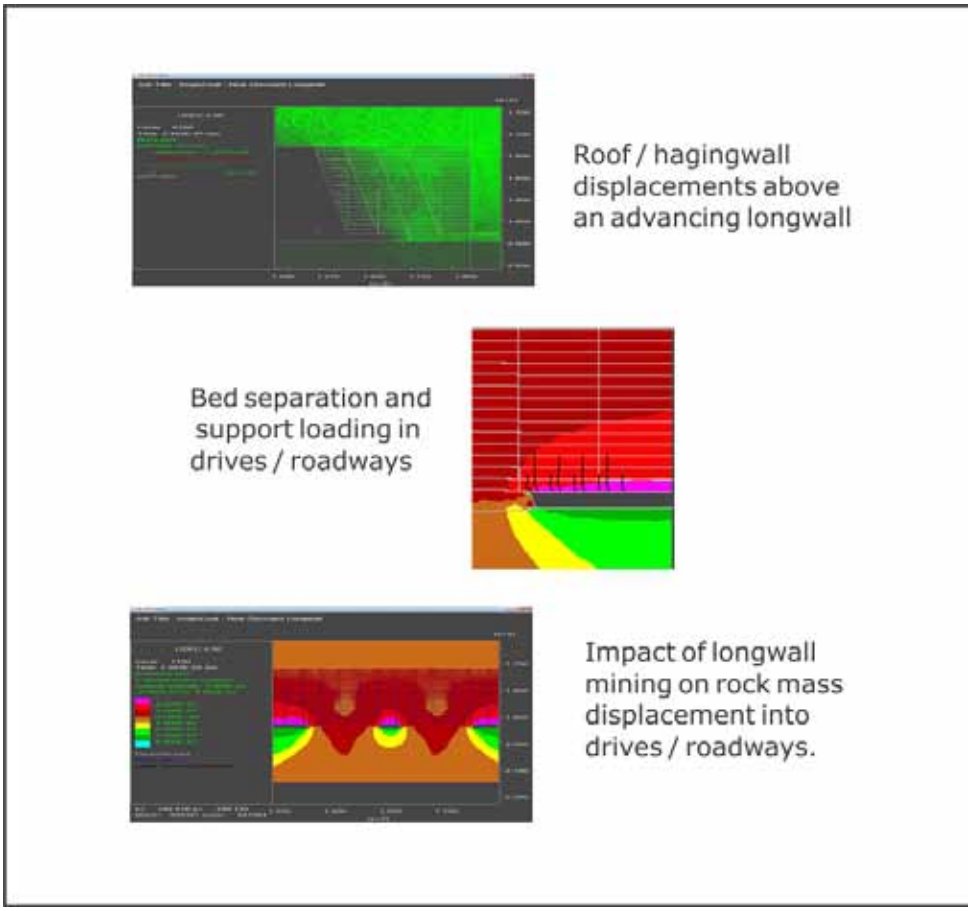


Figure 95: Blocks formed in UDEC

Dimension	Code	Results possible	Environment it can be applied to	Analyses of results (note examples at end of the outcome)
Three-dimensional codes	3Dec	Analyse stresses, displacements and strains at any position, on and off-reef, within the model. Various stresses, strains and displacements can be requested from the model and used within design criteria such as ESS, APS, and RCF.	Mining layouts associated with tabular and massive excavations in rock mass that can consist of various materials. It can simulate several mining steps and can be used to back analyse previous mining conducted or predict changes to be expected with planned mining. Tunnels and shafts can be constructed within the model.	Results can be graphed or dumped into excel for manipulation into average, minimum and maximum values, standard deviations etc.
Two-dimensional codes	UDEC	Analyse stresses, strains and displacements associated with excavations under plane strain.	Mining tunnels and excavations associated with underground rock mass that can consist of various materials and that can be presented in a simple layout. When rock block movement is important and development of separation on contact planes, displacement of rocks or beams is critical to the analysis. Support can be simulated and the load / deformation behaviour tracked.	

Table 14: The application of universal distinct element codes

back



Example of simulations using UDEC

LEARNING OUTCOME 6.4.5.5

KEYBLOCK METHODS

Keyblock analysis methods (Goodman and Shi, 1985) are used to evaluate whether blocks are removable and whether the chosen support will be sufficient to ensure stability.

BlockEval, Unwedge and JBlock are but a few of the software packages available to assess keyblock failure in excavation walls. JBlock follows a probabilistic approach where areas of high probability are indicated, while BlockEval and Unwedge use a deterministic approach, where no randomness is assumed to occur in the formation of the wedge and a single outcome is suggested.

BlocEval was developed for SIMRAC as part of OTH602 (Stacey – Best practice rock engineering handbook for "other" mines) and is available for download below. Blocks can be created within an excavation by providing joint orientation data and the stability of the block is evaluated by applying the provided support data.

The BlockEval software can be installed on the user's computer.

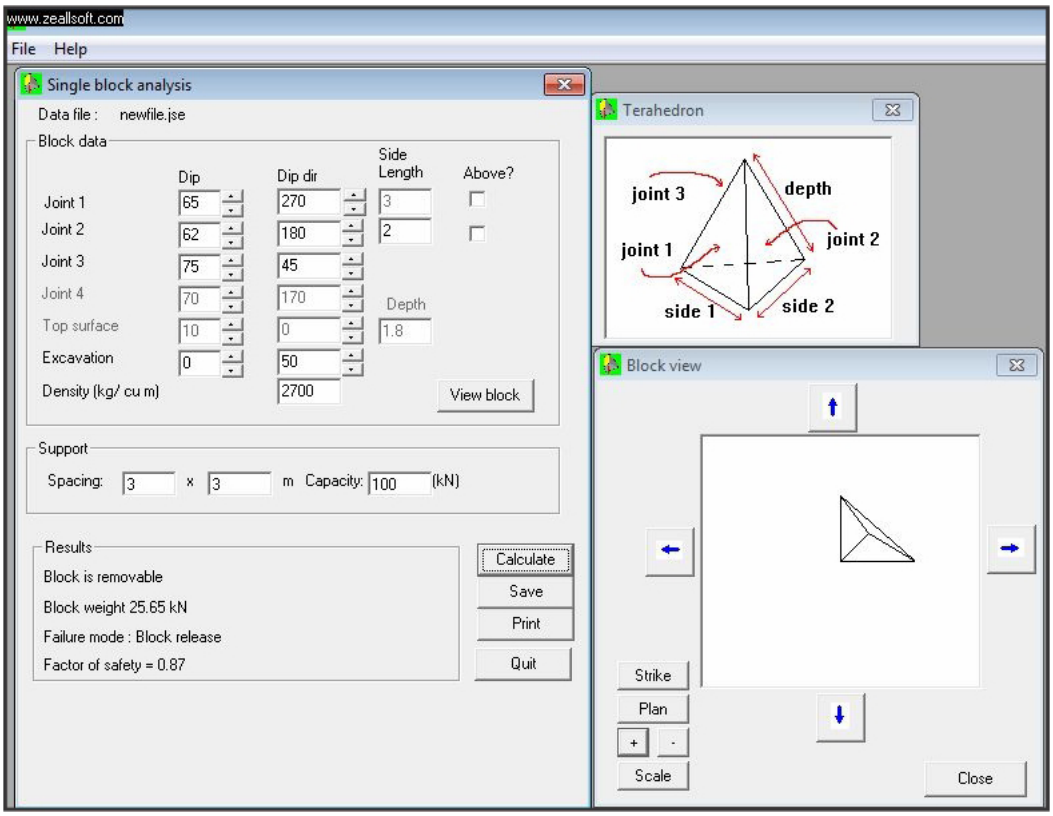


Figure 96: BlockEval example

The stability of wedges can also be analysed using the Unwedge programme (Rocscience, 2005) to analyse wedge failure around excavations in hard rock, where discontinuities are persistent, and where stress-induced failure does not occur. The analysis is restricted to three discontinuity planes at one time and it is necessary to conduct multiple analyses on combinations of planes if more than three discontinuity planes are present. Unwedge calculates the maximum sized wedges that can form around an excavation. The user can scale the size of the wedges based on experience and field observations. This is a deterministic approach and does not consider the range of wedge sizes that could occur or the likelihood of a particular wedge size. Support units can be installed and the effectiveness of the support to ensure wedge stability is evaluated.

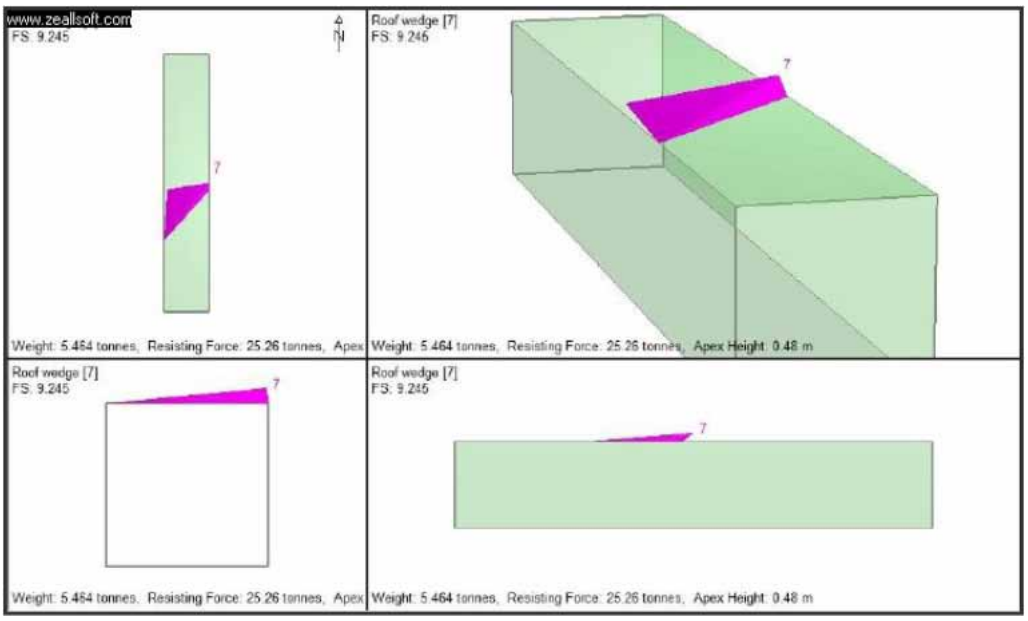


Figure 97: Unwedge example

JBlock generates removable keyblocks and places them at random locations in a pre-defined excavation and evaluates their stability. JBlock was developed to evaluate the potential for gravity-driven rock falls and a probabilistic approach is used to determine potential keyblock dimensions and their interaction with support (Esterhuizen and Streuders 1998). Using information such as the spacing, orientation and length of discontinuities, it is possible to simulate blocks in the walls of an excavation (Esterhuizen and Streuders, 1998). JBlock has some limitations in that it can only consider one surface at a time and therefore blocks in corners are not considered and random joints are not included.

REFERENCE

- JBlock V33 Manual
- Dunn et al : Support design using probabilistic keyblock methods
- Esterhuysen and Streuders: Rockfall hazard evaluation using probabilistic keyblock analysis,

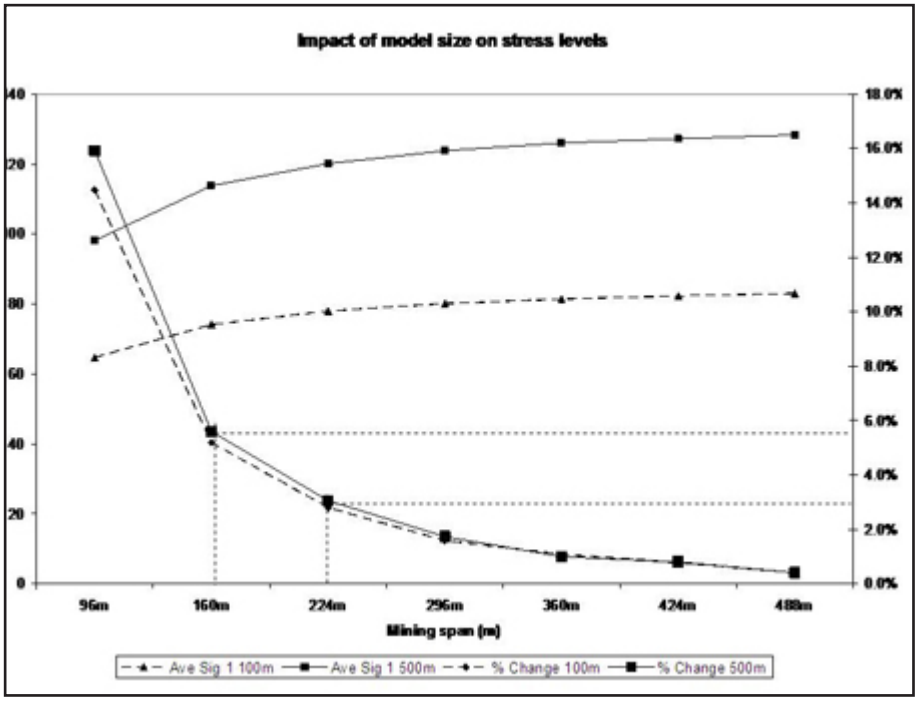


Figure 98: J Block example

Research conducted by SIMRAC into Numerical modelling has resulted in the creation of a large volume of information provided in three volumes that are attached to this material. The user is advised to open the 'Contents' chapters to find the correct chapter for the information sought, and then open the relevant chapter in the relevant volume, using the links in Table 15.

Volume	Topics covered	Chapters (linked)
1	Numerical modelling basics such as which has been covered above. Theory on the following: Solution methods, Material models, Creep, Elasticity, Solid mechanics, Waves, Modelling criteria	Index Chapter 1 Chapter 2 Chapter 3 Chapter 4 Chapter 5 Chapter 6 Chapter 7 Chapter 8 References
2	Different modelling codes and how to use them: Minsim, Minap, Besol, Map3D, Flac, Udec.	Index Chapter 1 Chapter 2 Chapter 3 Chapter 4 Chapter 5 Chapter 6 Chapter 7 Chapter 8 Chapter 9 References
3	Examples of simulations performed	Index Chapter 1 Chapter 2 Chapter 3 Chapter 4 Chapter 5 Chapter 6 Chapter 7 Chapter 8 Chapter 9

Table 15: SIMRAC research into numerical modelling

LEARNING OUTCOME 6.4.5.6

ANALYSES OF DATA FROM NUMERICAL MODELS

The selection of data to analyse and method of analysis of the results are subjected to the numerical simulation code used and the aims of the simulations. This means that 'recipe' type analyses guidelines cannot be provided, but that the user of numerical codes should consider the following:

- Assumptions made within the code (e.g. elastic medium) and its limitations (e.g. two dimensions) should be kept in mind when applying actual numerical simulation result values;
- The comparison of results from different options modelled remains a good method to indicate effectiveness, risk levels and to indicate the best approach to be followed by mining;
- Graphical screen prints of contoured results (e.g. 'hot' vs 'cold' plots etc) are good when the parameter evaluated is clearly indicated by the plots and changes can be accurately evaluated;
- However, analyses of actual values remain the best approach. Example: showing 10 plots of ESS contours as mining progresses towards a fault can show changes, but a line graph showing how

'maximum' or 'average' ESS or even 'maximum expected seismic event magnitude' for each mining step changes, will allow the identification of significant changes and clearly indicate the step at which risk levels increase substantially;

- The aim of the simulations should lead the user into the method of evaluation.
  - Example: Designing pillar dimensions will require the user to estimate average pillar stress levels from the models. Plotting the stress values in each pillar in MS Excel, allows the user to utilise all the functions in this programme to calculate maximum values, average values, draw graphs, etc. In the case of multiple options being modelled, it also allows the user to set up 'master' files in MS Excel that would allow successive result files to be copied into already set-up spreadsheets for immediate calculation of maximum and average values, as well as the drafting of, for example, line graphs of the average results;
  - If the aim is to indicate a position where a longwall should stop when approaching a dyke, results analysed should be such that a parameter, or parameters (e.g. maximum stress), should be graphed against distance to dyke. This will allow the user to read the optimum stopping distance from the graph when applying a limit to the 'parameter'.
- The examples below are included to indicate some forms of analyses that can be applied.

EXAMPLE 1 Analysis of pillar stress distributions for multi-seam mining conditions

A bord-and-pillar mine is considering the mining of two seams situated some distance apart and requested an investigation into the possibility of mining both seams and a recommendation with regard to a specific middling (distance) between the seams at which only one of the seams would be mined.

The investigation utilised the Map3D, Phase 2 and UDEC programmes to evaluate the mining environment as it was suggested that the following would be critical to this evaluation:

- Stress levels within pillars – inducing failure
- Stress distribution within the middling – inducing middling failure or bord hangingwall failure.

Since it was suggested that stress interaction between the pillars and the bord hangingwalls would play a large role in the stability of the middling, accurate stress levels on pillars and within the middlings were sought using the three-dimensional Map3D programme. The seams are extensive in their tabular dimensions and including all current and future mining in Map3D to get accurate stress levels would therefore create a large model that would increase computer running times significantly. To solve this problem, models at various mining spans were created and stress levels on pillars in the centre of the model evaluated.

Figure 99 indicates that the stress levels on the pillar are not significantly (less than 3%) affected by any mining outside a 224m x 224m mining block.

This sensitivity study allowed an optimum model to be created for the rest of the analysis.

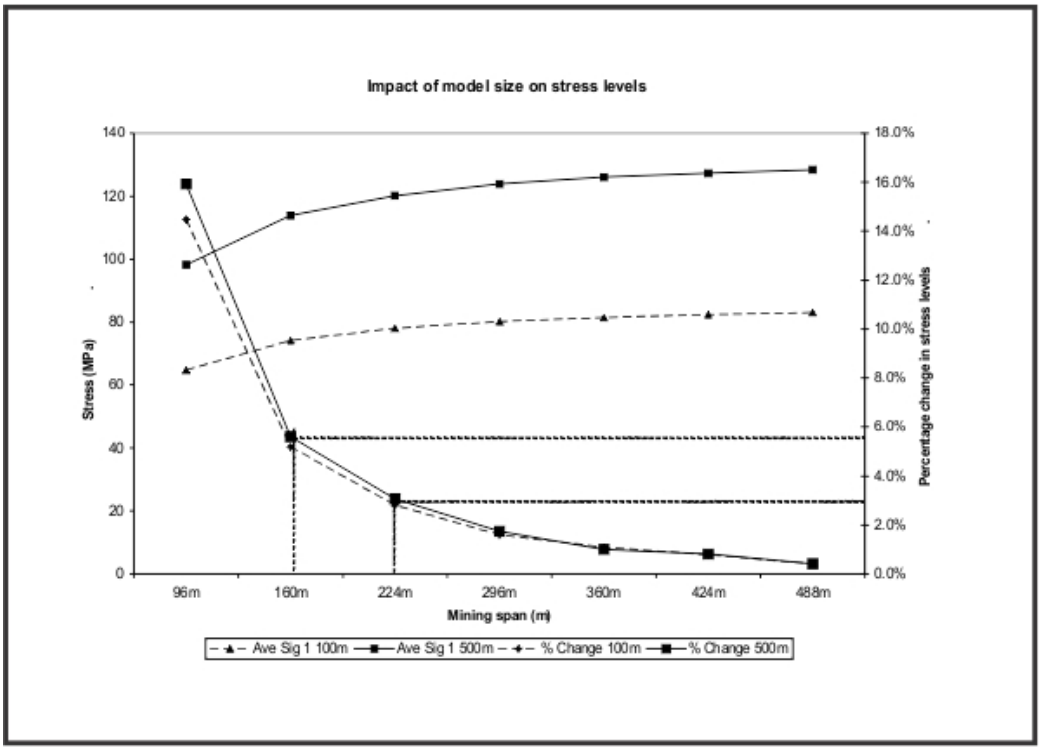


Figure 99: Sensitivity study for optimum model size

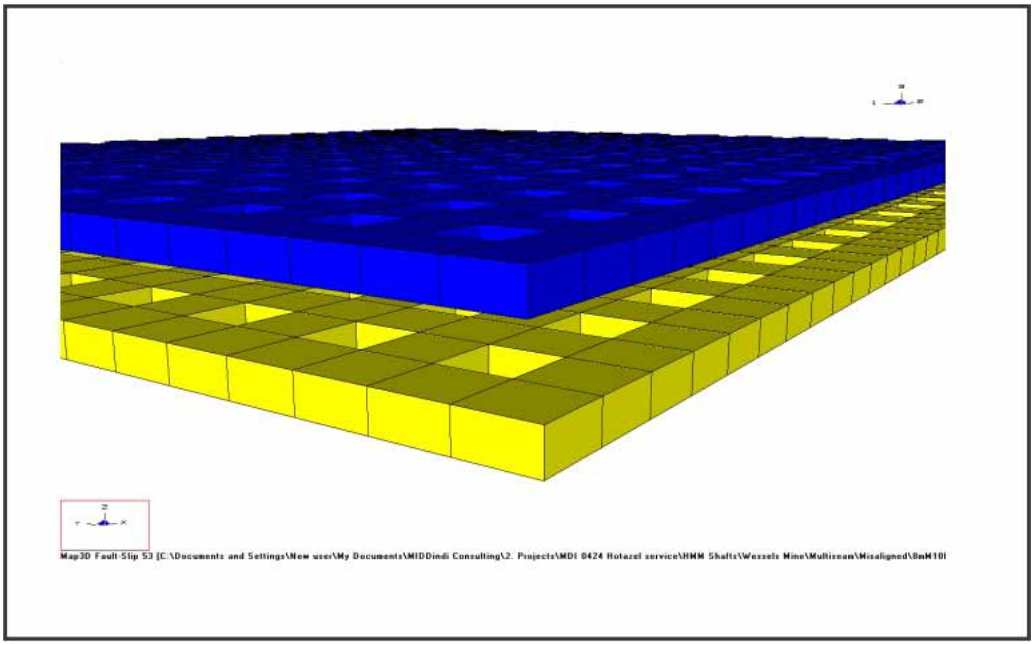


Figure 100: Map3D model showing multi-seam conditions

From the models created and the subsequent analysis of the pillar stress levels at various middlings between the seams, it was possible to draw the graph in Figure 100. This relationship indicates that at shallow mining depths (100 m), the mining of both seams does not seem to have a significant impact on pillar stresses, even at middlings as little as 5m. However, when the mining depth increases to 500m, a 5m middling between seams creates a sharp increase in pillar stresses.

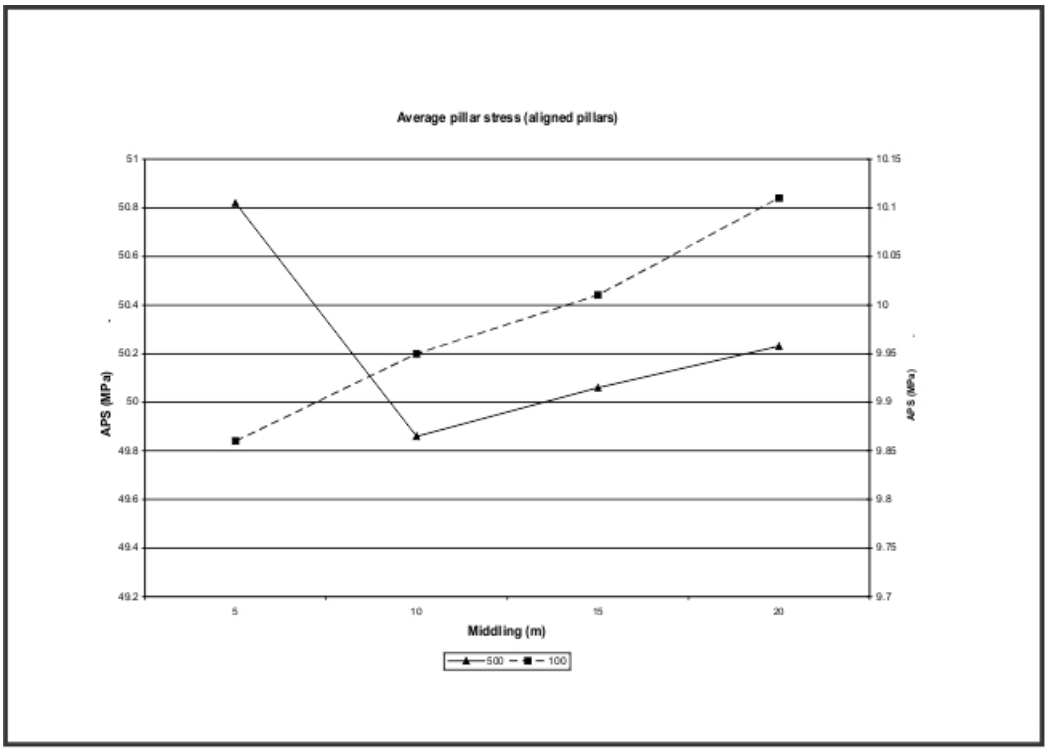


Figure 101: Results of sensitivity study for model size

Since pillar stress is critical for pillar stability, such an increase cannot be tolerated and one is left with two options:

- Do not mine the second seam at these middlings – loss of ore can be tolerated in a less economical seam and this is the safe option as pillars in a single seam will remain stable.
- Re-run models with larger pillar dimensions to cater for the expected increase in stress levels – this will mean a significant reduction in the overall extraction ratio and the risk of underestimating final pillar stress levels with the models, designing inadequate pillar dimensions and experiencing pillar failures.

By utilising the two-dimensional stress analysis programmes, Phase 2 and UDEC, the stress distribution within the middling between the seams could be analysed using the pillar stress levels deduced from the three-dimensional Map3D programme.

Figure 101 and Figure 102 show the models set up in both these programmes, while Figure 103 and Figure 104 show examples of the results and the similarity in the results. This similarity allowed the more complex and time-consuming UDEC (distinct element) model to be used only to rectify the results, while the large number of models (at varying depths, middlings, position of pillars, etc.) were run on the more time-effective Phase 2 (finite element) programme.

Once the results from all the models run had been analysed, the relationship in Figure 105 could be drafted. This relationship indicates that there seems to be a middling at which the risk of experiencing some form of failure (shear) in the middling between the seams increases significantly. This and other similar relationships could now be used to justify a middling of 10m as that distance between the seams when mining of both seams would not be allowed.

back

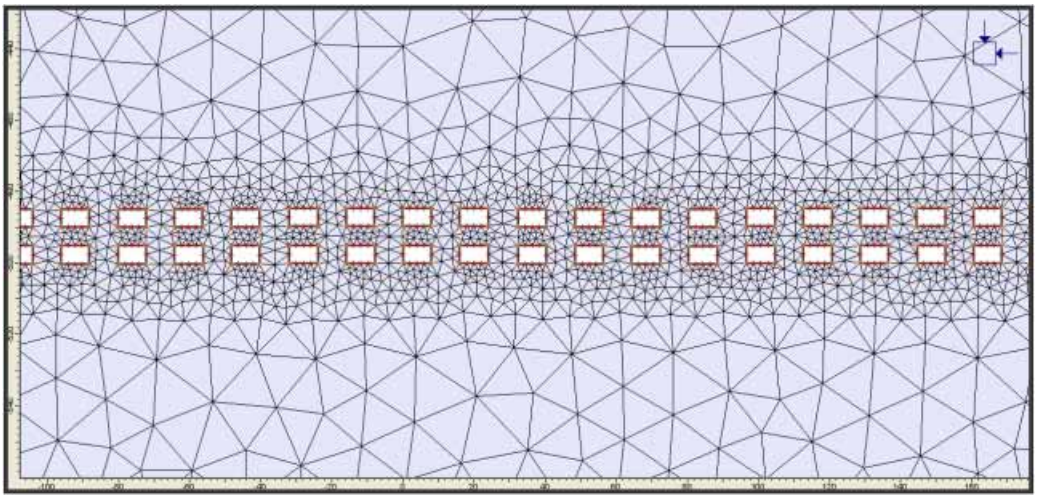


Figure 102: Phase 2 model

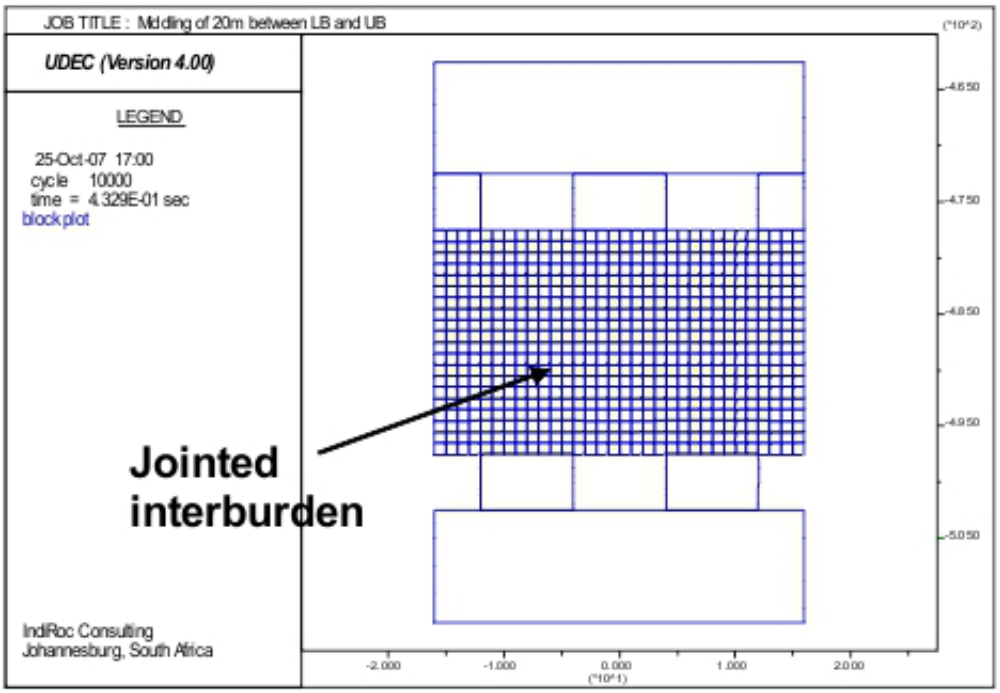


Figure 103: UDEC model

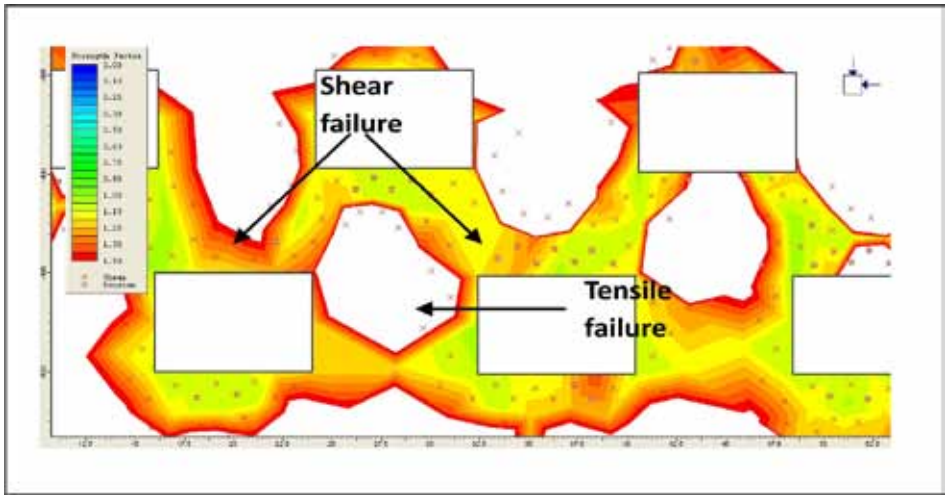


Figure 104: Phase 2 results

back

back

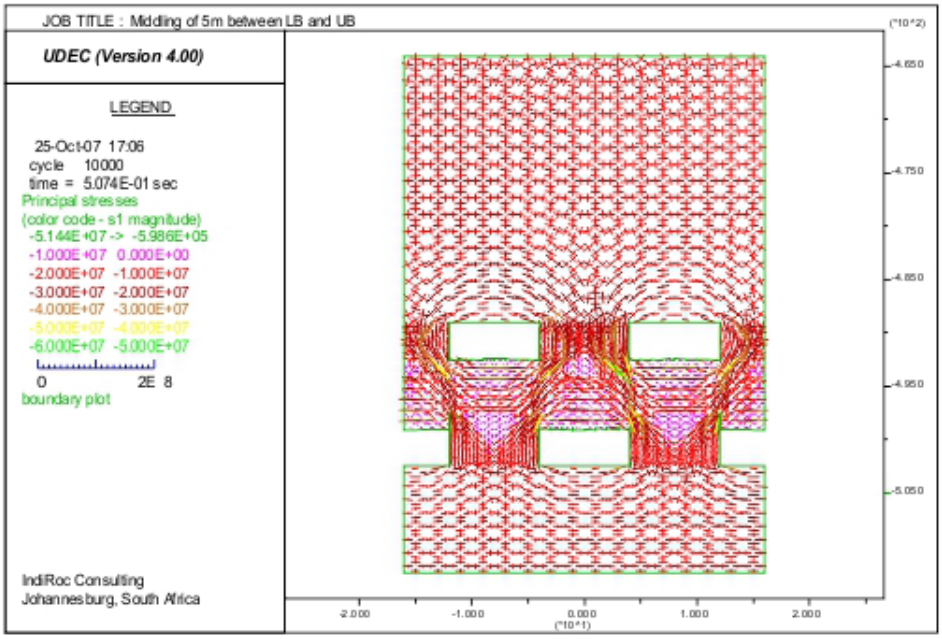


Figure 105: UDEC results

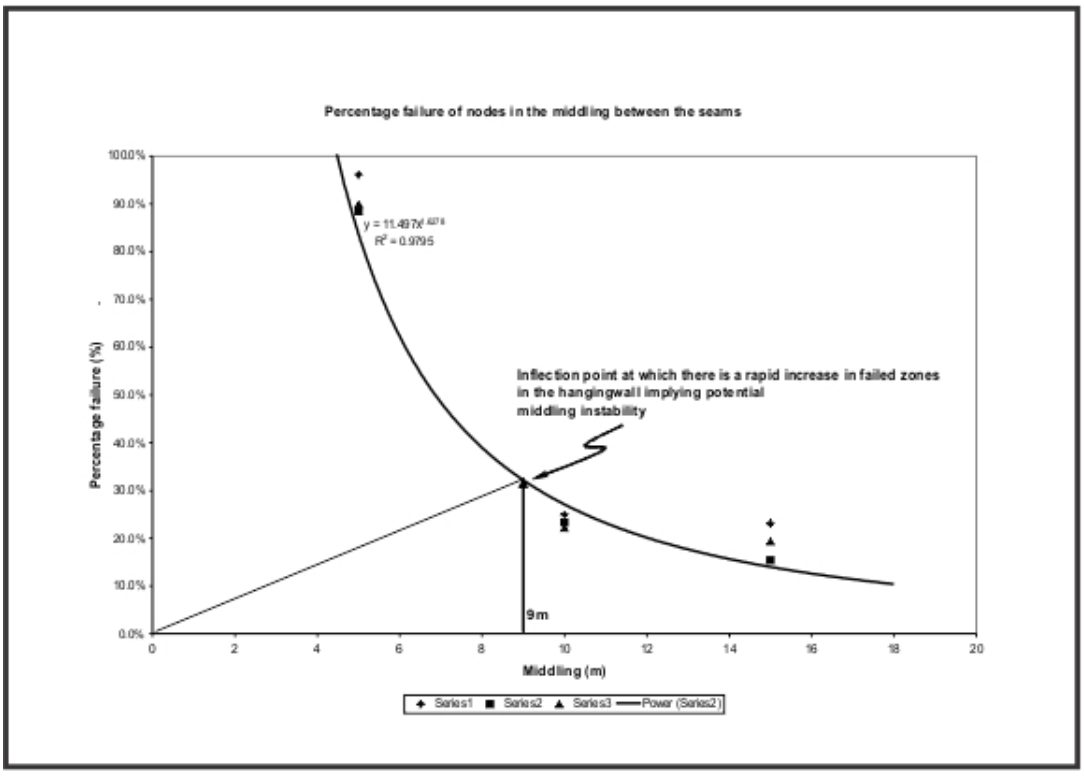


Figure 106: Final results of investigation indicating minimum middling at which mining of both seams would be allowed

EXAMPLE 2

Analysis of placement of footwall tunnels

The placement of footwall tunnels is affected by stress levels transferred through pillars left intact around mining stopes and stresses created around mining abutments. Placement of tunnels in the lowest possible stress field is the best approach, but this usually increases the distance between the ore body and the tunnels, increasing development metres and therefore also development costs. When placed closer to the ore body, substantial support may be required to maintain stability at higher stress levels and also add to the development costs. The analysis conducted on footwall tunnels was based on the Rockwall Condition Factor (RCF) methodology (the criterion is valid only for the Witwatersrand).

back

The RCF methodology is based on the assumption that the two-dimensional stress field and the rock strength determine the impact of stress on a tunnel. The RCF methodology states that, where σ_1 and σ_3 are the major and minor principal stresses acting perpendicular to the tunnel, σc is the rock material UCS and F is a downgrading factor to cater for excavation sizes and rock mass quality (a highly discontinuous rock mass would be downgraded). The RCF values are normally categorised as 'less than 0.7', 'between 0.7 and 1.4' and 'greater than 1.4', where RCF values exceeding 1.4 indicate very poor rock mass conditions that require substantial support installation to remain stable.

Map3D provided the stress levels acting at specific points within the footwall of the ore body (Figure 106) and allowed exportation of the results to MS Excel for the calculation of the RCF values. In this calculation, the UCS for was taken to be 164MPa and the F factor was estimated at 0.65. It was assumed that an RCF value of 2.1 would form the upper boundary of the 'RCF greater than 1.4' category, and that stability could not be ensured, even with substantial support installation, when the RCF value exceeded this 2.1 level. At the same time, it was assumed that the footwall drive would be a temporary excavation that could be abandoned if conditions deteriorate, while the haulage, which is placed deeper into the footwall, must remain operational.

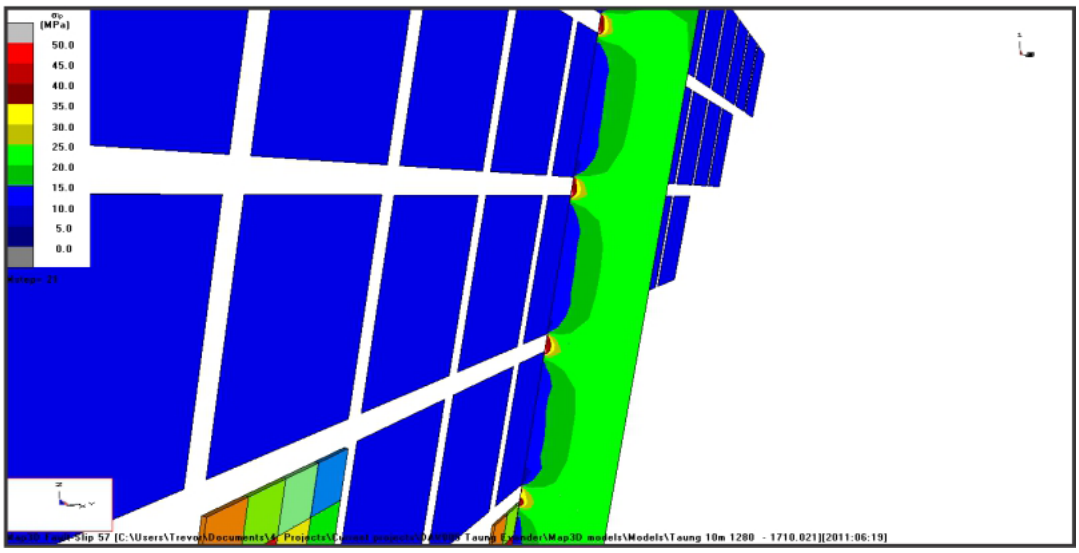


Figure 107: Stress levels in the ore body footwall

For all mining depths and pillar dimensions, the RCF values at the approximate tunnel elevations and at increasing positions away from the ore body were determined and are presented in Figure 107. Using the RCF = 2.1 limit, the minimum distance that the haulage must be placed from the ore body to ensure that it can be supported to remain operational, could be determined for each mining step in an active mining stope (Figure 107), as well at the stress levels present in back areas.

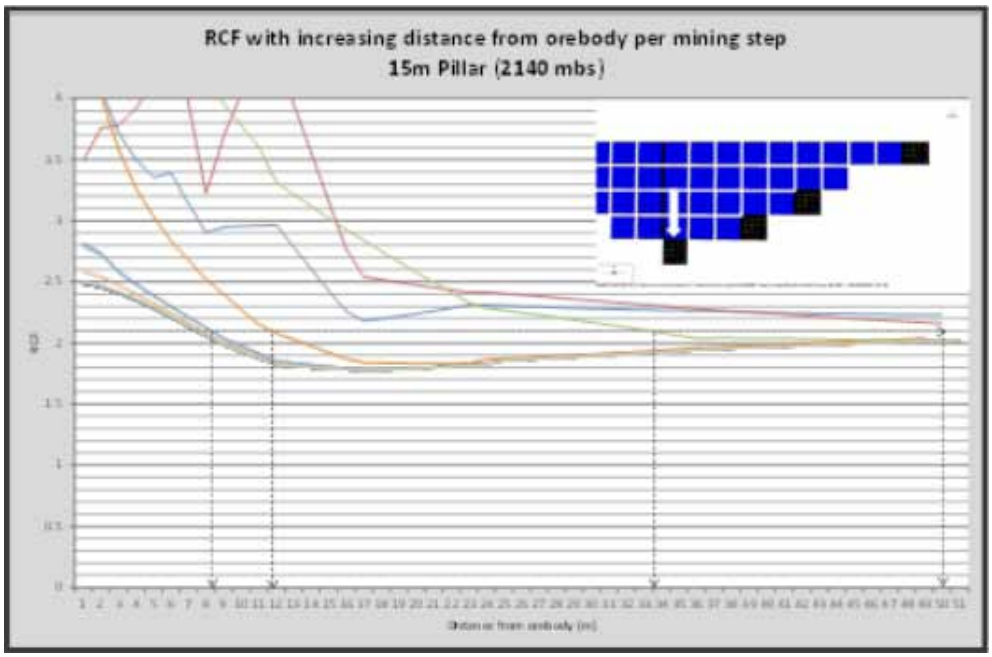


Figure 108: RCF values with increasing distance from ore body

Presentation of the distances from the ore body at which the RCF < 2.1, in Figure 108 indicates the suggested position of haulages with increasing depth in relation to the ore body. A minimum distance of 28m from the ore body was taken assuming that the haulage cannot be placed closer than this distance (drive 7m from ore body, 3m wide drive, haulage 18m from drive – 3*(combined widths)). It is clear that this distance increases to 50m in the deepest sections, while between 1400mbs and 2300mbs pillar dimensions affect the haulage position by as much as 7m (2000mbs). Table 16 gives the suggested haulage distances from the ore body based on the results.

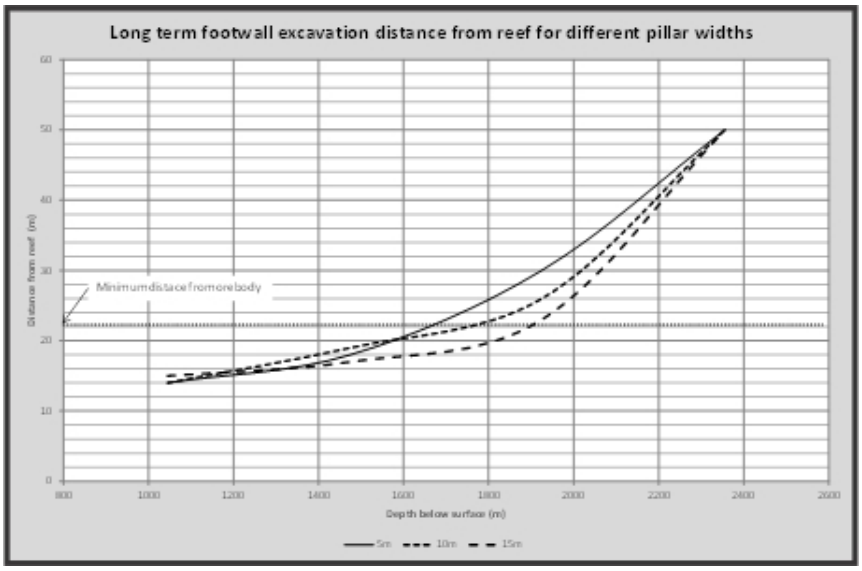


Figure 109: Haulage positions near active stopes for different pillar widths

back

	Distance from ore body (m)		
Depth (mbs)	5m Pillar	10m Pillar	15m Pillar
800	28	28	28
1000			
1200			
1400			
1600			
1800			
2000	33	29	
2200	42	40	39

Table 16: Suggested distances for placement of the haulage

Stress levels in sill pillars that were used to place the footwall excavations will, due to the elastic analysis performed with Map3D, be overestimated as any rock or rock mass yielding / failure is ignored by the model. In an attempt to calibrate the results given above, the sill pillar stress levels suggested by UDEC are shown below in Figure 109. The onset of significant stress increases due to the presence of the stope as mining and drawing occurred, indicated that long-term footwall excavations should be placed at least 23m from the stope. This result confirms that the distances shown in Table 16 are not excessive and that it can be used to determine the minimum distance between permanent long-term excavations and the ore body.

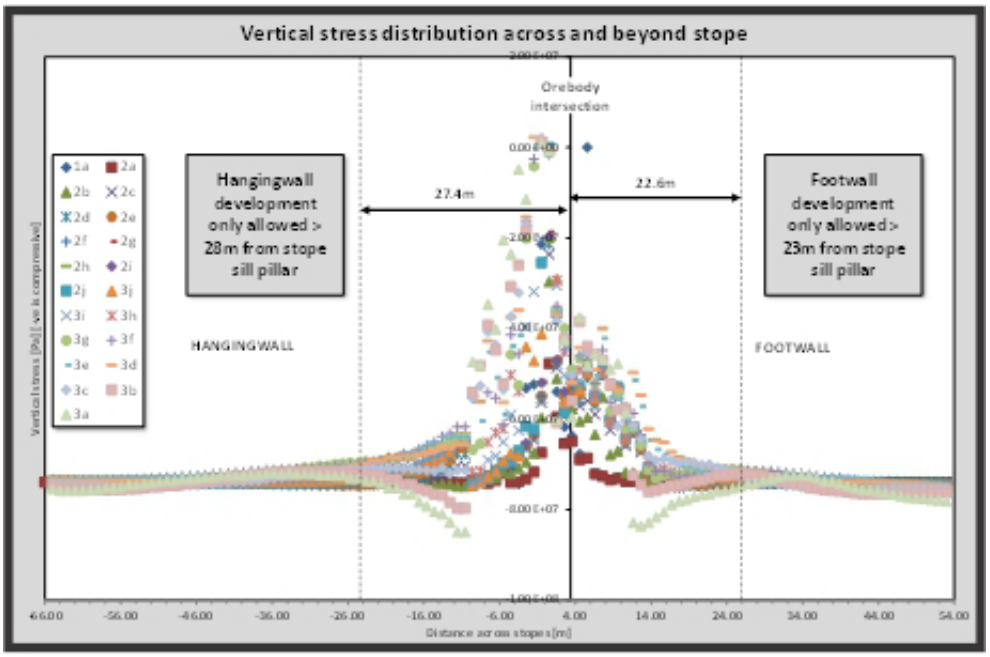


Figure 110: Footwall excavation placement (UDEC)

EXAMPLE

Statistical methods of analysing numerical modelling results have been utilised by many practitioners. The link below provides access to one such example.

Statistical analysis example

INTERESTING INFO

Gap603 Seismics and numerical modeling

Gap629 Updated numerical modelling

back

back

- SIMRAC reports on Numerical modelling
- Chapter 11, Jager A.J. and Ryder J.A., Handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 2002
- Chapter 8, Jager A.J. and Ryder J.A., A textbook on rock mechanics for tabular hard rock mines, 2002
- Chapter 6, Brady H.G.B. and Brown E.T., Rock mechanics for underground mining, 2006
- Chapter 10, Hoek E., Practical rock engineering, 2006.
- Chapter 8, Van der Merwe J.N, Madden B.J., Rock engineering for coal mining, 2002.
- Page 78, Stacey T.R., Best practice rock engineering for Other Mines, 2001.

back

CHAPTER 7

DRILLING AND BLASTING

INTRODUCTION

The behaviour of rock will be affected (except for the properties of the rock mass and forces imparted on the rock mass) by blasting practices. Blasting, or the detonation of explosives, creates blasting fractures (which is why the rock is broken into smaller pieces) while it also creates a shock wave that moves through the rock mass and results in severe 'shaking', exerting additional forces on the rock mass and inducing displacements on rock and installed support units.

The impact of blasting on the rock mass and support units is affected by the explosive type, initiation systems applied and by the holes drilled (diameter, length, orientation) as implemented during drilling practices. It is therefore important to understand the effects of drilling and blasting on the surrounding rock mass, to be able to minimise blast damage as much as possible and also to evaluate the impact of blasting practices on observed conditions and rock mass behaviour.

Optimised blasting designs have the potential to achieve consistently good face advances with reasonable control over fragmentation and over break. A combination of blast design, drilling accuracy and accurate and reliable initiation can produce dramatic improvements to mining costs and stability of workings.

This chapter therefore deals with:

- Explosives
- Drilling
- Blasting

LEARNING OUTCOMES

7. DRILLING AND BLASTING

7.1. EXPLOSIVES

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

7.1.1. Describe and explain the main types of explosives used in mining

7.1.2. Describe the ingredients of the main types of explosives used in mining

7.1.3. Describe and explain the types of initiating explosives used in mining

7.1.4. Describe and explain the mechanism of rock breaking by blasting

7.1.5. Describe and explain the effect of shock energy on rock fragmentation

back

7.1.6. Describe and explain the effect of gas pressure on rock fragmentation

7.1.7. Describe and explain the function of stemming

7.1.8. Apply knowledge of explosives and their characteristics to select appropriate explosives for a given task.

7.2. DRILLING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

7.2.1. Describe and explain the mechanism of rock breakage by chisel bit

7.2.2. Describe and explain the mechanism of rock breakage by button bit

7.2.3. Describe the types of equipment used by the following drilling methods:

7.2.4. Describe, explain and discuss the use of the following drilling methods:

- Percussion drilling
- Rotary drilling
- Diamond drilling
- Raise / Tunnel boring

7.2.5. Describe and explain how cuttings are removed from the hole for each of the above drilling methods.

7.3. BLASTING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

7.3.1. Describe, explain and discuss the effect of the following parameters on blast damage :

- Explosive type
- Initiation method
- Initiation sequence
- Hole orientation

7.3.2. Describe, explain and discuss the following cushion blasting techniques:

- Smooth blasting
- Post-splitting
- Pre-splitting

7.3.3. Describe typical applications of each

7.3.4. Describe, explain and discuss the objectives of decoupling explosives

7.3.5. Describe, explain and discuss the effects of decoupling explosives

7.3.6. Describe, explain and discuss the methods by which decoupling of explosives is achieved.

back

LEARNING OUTCOME 7.1

INTERESTING INFO

The information below is given to allow appreciation of the history and uses of explosives, since its inception in 50B.C. Table 1 shows some interesting uses of explosives, while Table 2 shows a few historical moments in the history of explosives.

Aerospace As in ejector seats and separation devices for rocket boosters.	**Aggregate blasting art** As in metal-clad art and large projects such as mountain carving.
Automotive restraint systems To inflate airbags	**Welding** Explosives jigs are often used to weld large diameter pipelines.
Emergencies at sea Project lifelines to ships in distress off storm-beaten shores.	**Metal-cladding** Explosives are sometimes used to bond various metals to each other.
Food preparation Meat tenderising	**Law enforcement and security** Used in exploding dye capsules and other security systems.
Metalworking Hardening of metals, including railway frogs and hardening of crusher wear parts, shovel teeth, ripper shanks, and speciality wear components such as striker bars and striker plates for mills. The explosive depth hardening process will take manganese steel from 350 brinell up to 490 brinell after two shots. The EDH process increases wear life substantially.	**Demolition** The quantities of explosive used on this type of operation vary, depending on the size, construction and location of the structure to be demolished. Whether it be bridges, buildings, chimneys, or towers.
Pyrotechnics Display fireworks / Theatrical special effects	**Riveting** Blind rivets are needed when space limitations make conventional rivets impractical. Explosive riveting is an engineering practice.

Table 1: Interesting uses of explosives other than mining

back

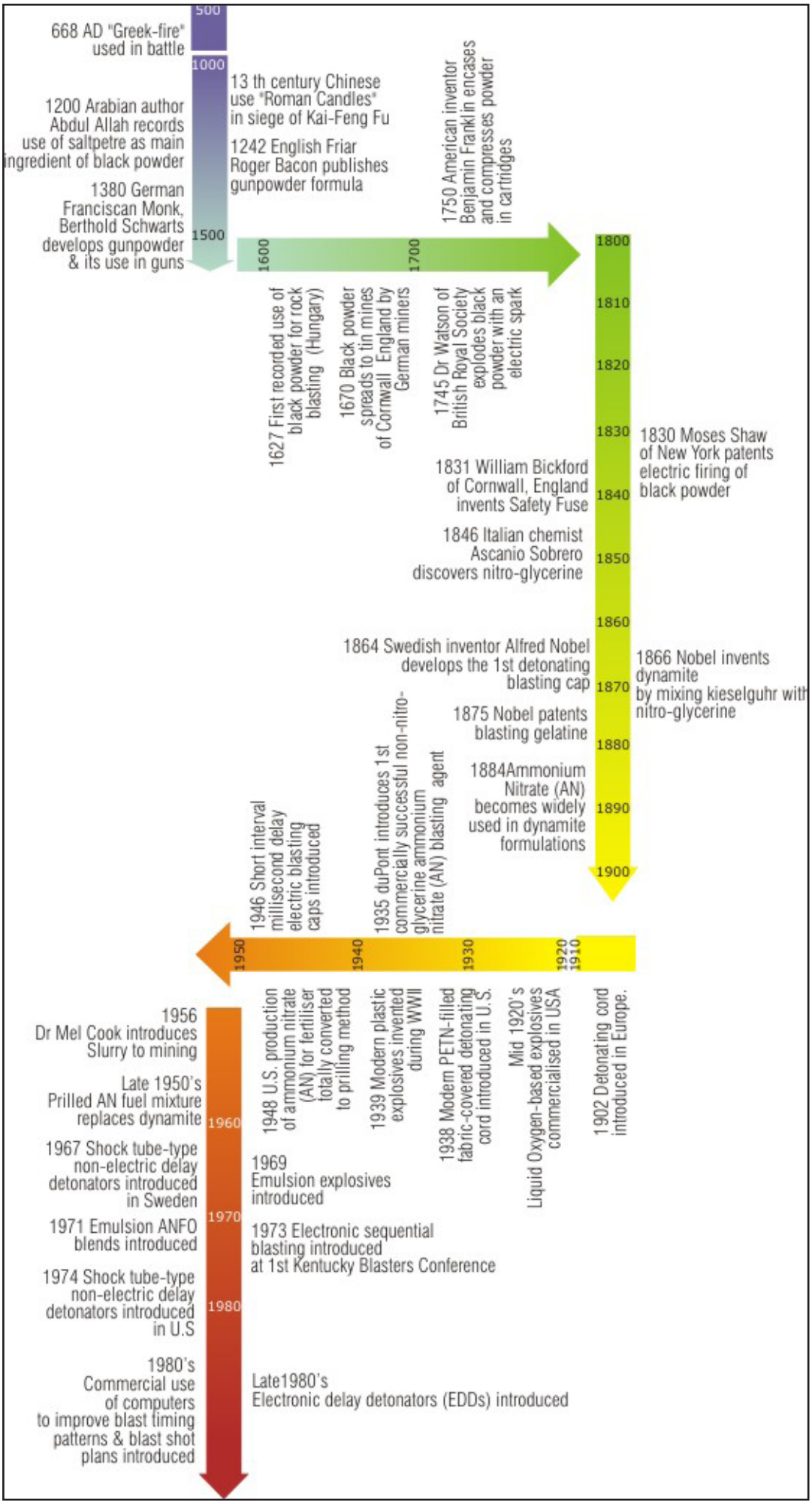


Table 2: A few historical moments in the history of explosives

back

LEARNING OUTCOME 7.1.1

DESCRIBE AND EXPLAIN THE MAIN TYPES OF EXPLOSIVES USED IN MINING

In an attempt to describe the general sequence in the use of explosives, Figure 1 shows that several routes can be followed to detonation, while Figure 2 shows that several routes are available from detonation to explosion. It is therefore possible to design a blast system for any particular application, using any of a variety of products, as long as an appropriate sequence of blasting includes the necessary (not necessarily all) components to:

- ignite,
- initiate,
- detonate,
- propagate,
- boost, and
- explode

Ignition	Fuse igniter or Igniter cord		Shot exploder
Initiation	Safety fuse	Initiator Nonel tubing	Electric fuse head
Detonation		Detonator	

Figure 1: Chain reaction sequences: ignition to detonation

Detonation	Detonator			
Propagation		Booster (ANSTART)	Detonating fuse (Cordtex)	Detonating fuse (Cordtex)
Boosting				Booster (Pentolite)
Explosion	Explosives	Explosives	Explosives	Explosives

Figure 2: Chain reaction sequences: detonation to explosion

NOTE

Where empty cells exist in Figure 1 and Figure 2, it is suggested that no component is required when using that sequence, e.g. a design could include igniter cord, safety fuse, detonator, cordtex, NO BOOSTER COMPONENT and explosives OR igniter cord, safety fuse, detonator, NO PROPAGATOR COMPONENT, booster (ANSTART) and explosives.

Chemical explosives are the most commonly used in mines and consist of deflagrating (low) explosives and detonating (high) explosives.

DEFINITION

Deflagrating explosives are explosives such as black powder that burn at a slow rate and create less pressure than high explosives (detonating). Velocity of detonation is approximately 90m.s-1.
Detonating explosives detonate at high velocities and create large pressures. Velocity of detonation is approximately 2000 - 9000m.s^{-1}.
Note: Speed of sound is approximately 330m.s^{-1}.

Low explosives

For many years, black powder was the most common low explosive used throughout the world. However, black powder, or gun powder, produced

back

a large amount of smoke and was dangerous to use. Today, black powder is still used for pyrotechnics (fireworks), special effects, and other specialised work, but it has been replaced in commercial blasting by safer, more productive explosive materials.

High explosives

High explosives create more pressure and burn faster, detonating almost instantaneously. The proper use of high explosives by today's explosives engineer produces minimal ground vibrations and air overpressure.
The first high explosive used in commercial blasting was nitro-glycerine, also called "blasting oil." Nitro-glycerine was dangerous to use because it is an unstable chemical. In the late 1800s, a Swedish chemist, named Alfred Nobel, invented dynamite by mixing nitro-glycerine with a special clay, called kieselghur, and packed it into sticks.

Detonating explosives are divided into three groups:

- Primary explosives – Examples are, lead styphnate and lead azide. This explosive type needs a spark to initiate, and is therefore used in blasting caps.
- Secondary explosives – Examples are nitro-glycerine (NG), ethyleneglycoldinitrate (EGDN) and pentaerythorotetranitrite (PETN). This explosive type needs the use of a blasting cap as a top charge.
- Tertiary explosives – These explosive types are the dry blasting agents or slurry explosives and blasting agents. An example is ammonium nitrates (Anfo and Anfex).

Table 3 shows a short list of the most common types of explosives used on underground mines.

Explosive type	Where used	Characteristics
Anfex (low density)	**Development** **Stoping**	**Low VOD (2900 m.s^{-1} – 3600 m.s^{-1})** **High gas energy (excellent heave)** **Low shock energy (good fragmentation)** **Must be primed (insensitive)** **Not water resistant**
Energex Barrels	**Development perimeter holes**	**Medium VOD (3700m.s^{-1})** **Water resistant (watergel)** **Must be primed (insensitive)** **Developed for perimeter blasting (de-coupled)**
Powergel	**Stoping (downdip)**	**Must be primed (insensitive)** **Water resistant** **VOD (4000 m.s^{-1})** **Medium energy (favorable energy partitioning for undercutting)**

Table 3: Typical explosive types used on RSA mines

back

Figure 3: Examples of some of the most common mining explosives (AEL Products)

The major suppliers of explosives in South Africa have a large number of explosive types that can be viewed on the following websites:
http://www.explosives.co.za/home.html
http://www.bme.co.za

In the case of surface and some massive mining methods, pumpable emulsions are often used. These explosives are easy to pump into long and large diameter blast holes. An emulsion is a mixture of two liquids with one liquid phase dispersed uniformly throughout the second phase. Emulsion explosives are dispersions of water solutions of oxidisers in an oil medium or "water-in-oil" emulsions.

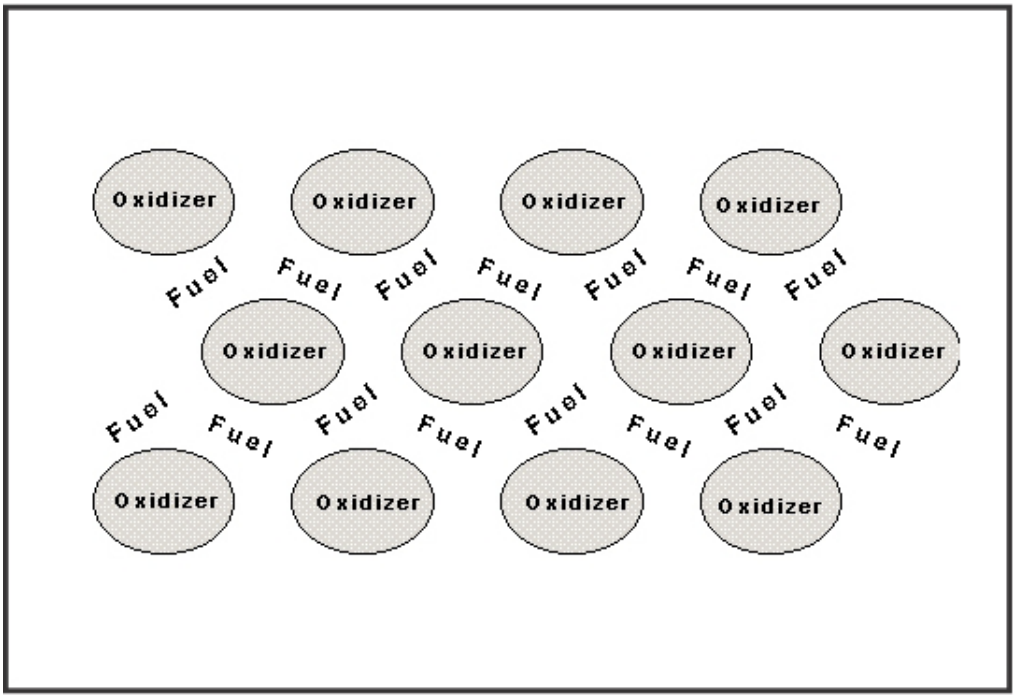


Figure 4: Oxidiser surrounded by fuel

The oil phase is known as the continuous or external phase because it surrounds and coats all of the oxidiser droplets. The fuel phase is generally oil or wax or a combination of the two. The water or oxidiser solution phase is called the discontinuous or internal phase because the microscopically fine droplets are kept apart and surrounded by the continuous fuel phase. The oxidiser phase always contains ammonium nitrate. Other salts such as sodium nitrate, calcium nitrate and ammonium may also be included. The oxidiser remains dispersed in the fuel to form a stable emulsion through the action of an emulsifier.

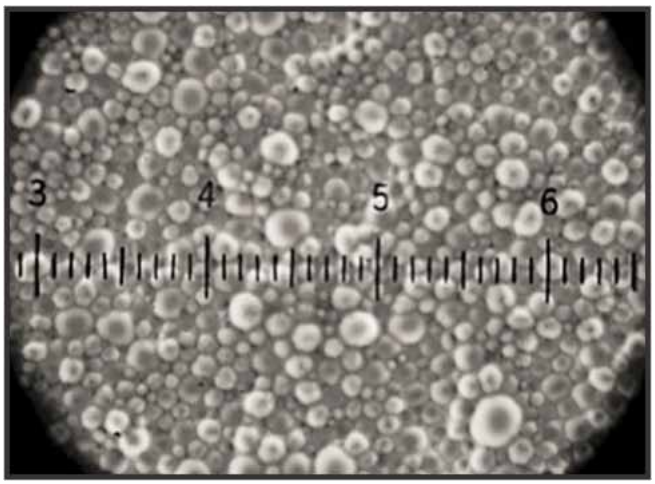


Figure 5: Typical water-in-oil emulsion

Emulsions fail to detonate in impact and friction tests. When placed against a metal plate, the emulsions fail to detonate under the impact of a projectile. Other high velocity impact tests with larger calibre projectiles show emulsions to have a greater resistance to initiation by impact than either water gels or dynamites. Normally, emulsion explosives will not detonate during burning, but there is no guarantee of this, particularly if the material is contaminated with foreign materials such as rust, detonators, dynamites or aluminium powders.

back

NOTE

When pumping emulsions, care must be taken so that the pump does not run dry or against a closed system. In either case, friction can raise the temperature of the emulsion in the pump beyond the decomposition point of ammonium nitrate or other ingredients. If this happens, a detonation can occur.

It can be just as hazardous to pump unsensitised emulsion oxidisers as it is to pump sensitised ones. Although tests have demonstrated that emulsions offer a great degree of safety, they will detonate if subjected to severe conditions. They are explosives, and regardless of their degree of safety, should never be abused.

REFERENCE

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining
- Chapman & Hall New York,Chapter 17,
- AEL products and training material.
- http://www.exploenergy.com/understanding-emulsion-explosives.html

LEARNING OUTCOME 7.1.2

DESCRIBE THE INGREDIENTS OF THE MAIN TYPES OF EXPLOSIVES USED IN MINING

Binary explosives are cap-sensitive (detonatable with a standard #8 blasting cap) two-part explosives mixtures, shipped separately and combined at the use site. Many of these mixtures are based on ammonium nitrate as an oxidiser plus a volatile fuel, but unlike ANFO (ammonium nitrate fuel oil explosive), these binaries can be detonated by blasting caps. ANFO requires high explosive boosters to detonate it.

Most binary explosives are a slurry after mixing, but some form a fluid with solid components dissolved into liquid ones.

Blasting agents are explosive materials or mixtures that are not detonatable by standard #8 blasting caps. The best-known blasting agent is ANFO explosive, a mixture containing primarily ammonium nitrate with a small quantity (typically around 6%) of fuel oil, most commonly diesel fuel. Other fuels and additives are used as well. While ANFO is often made on-site using fertiliser grade ammonium nitrate, blasting agents can also be purchased in pre-packaged form, usually in metal or cardboard cylinders.

INTERESTING INFO

Castings, or castable explosives, are explosive materials or mixtures in which at least one component can be safely melted at a temperature that is safe to handle the other components, and which are normally produced by casting or pouring the molten mixture or material into a form or use container. In modern usage, Trinitrotoluene or TNT is the basic meltable explosive used in essentially all castable explosives. Other ingredients found in modern castable explosives include:

- Active, energetic or explosive ingredients:
  - Aluminium powder
  - Ammonium nitrate
  - Ammonium picrate
  - Lead nitrate
  - PETN
  - letryl

back

- Inert ingredients
  - Boric acid
  - Calcium chloride
  - Wax
  - Silica, often in the form of silica fume (Cab-o-sil for example)

Polymer-bonded explosives, also known as Plastic-bonded explosives or simply PBX, are a relatively solid and inflexible explosive form containing a powdered explosive material and a polymer (plastic) binder. Technically known as putties, but more commonly plastic explosives, these mixtures are a thick, flexible, mouldable solid material that can be shaped and will retain that shape after forming, much like clay. Putties normally contain mostly RDX explosive, but may include some PETN (Semtex, for example). Some common putties are:

- • C4
- • Semtex

Rubberised explosives are flat sheets of solid but flexible material, a mixture of a powdered explosive (commonly RDX or PETN) and a synthetic or natural rubber compound. Rubberised sheet explosives are commonly used for explosive welding and for various other industrial and military applications. Rubberised explosives can be cut to specific shape, bent around solid surfaces, glued or taped in place, or simply laid on relatively flat surfaces.

Extrudable explosives are an extremely viscous liquid, similar in properties to silicone-based caulking materials used in construction. It is used in similar ways – stored in a container, then extruded out a nozzle into thin cracks, holes, or along surfaces. Some extrudable explosives can then be hardened using a heat-curing process. Others will remain a viscous fluid permanently.

REFERENCE

- http://en.wikipedia.org/wiki/Use_forms_of_explosives
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining
- Chapman & Hall, New York, Chapter 17
- AEL products and training material.

LEARNING OUTCOME 7.1.3

DESCRIBE AND EXPLAIN THE TYPES OF INITIATING EXPLOSIVES USED IN MINING

The following are common initiation systems used on the mines:

- Shock tube
- Capped fuse systems
- Electronic delay detonators
- Electric Initiators

NOTE

Since all of these systems have specific applications, features, safety and production advantages etc, it is advised that the explosive manufacturers' websites are visited for more information on these products:

AEL: http://www.aelminingservices.com/products/initiating-systems.html
BME: http://www.bme.co.za

The following short references to these systems will suffice:

- **Shock tube:** A non-electric initiation system that is designed for use in underground and surface mines. The unit consists of an out-hole detonator as well as a length of green shock tubing, which includes a short period delay detonator and a rubber sleeve that has been crimped on one end. The free end of the shock tube is sealed.

Figure 6: Shock tube (AEL)

- **Electronic delay detonators:** Electronic detonators are user-friendly, fully programmable detonators that can accommodate complex blast designs where flexible milli-second timing is required. Some units have test-on-connection features that ensure successful deployment of the unit. A detonator is crimped onto a round, twin-core steel downline on one end and terminated with a connector on the other end.

Figure 7: Smartdet (AEL)

- **Capped fuse systems:** Igniter cord is a flexible, incendiary cord comprising of a copper wire core covered with a plasticised, incendiary composition and waterproofed with a thin layer of polyethylene. The cord burns progressively with a constant, vigorous and robust external flame.

Figure 8: Igniter cord fastened to blasting fuses along a stope face (AEL)

- **Electric Initiators:** The instantaneous electric detonator is used to initiate blasts by firing trunk lines of detonating cords or shock tube, secondary blasts and blasting gunpowder. A copper detonator with PVC leg wires wound in a figure of eight coil.

Figure 9: Electric detonator

REFERENCE

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining
- Chapman & Hall, New York, Chapter 17
- AEL products and training material
- http://www.aelminingservices.com/products/initiating-systems.html
- http://www.bme.co.za

LEARNING OUTCOME 7.1.4

DESCRIBE AND EXPLAIN THE MECHANISM OF ROCK BREAKING BY BLASTING

Blast holes are drilled in a specific pattern and spacing to suit the host rock conditions. Explosives are placed in the blast holes and detonated, resulting in shock and gas energy that fractures the host rock. In mining it is important that the correct fragmentation of the host rock be achieved to reduce mineral losses or loading / cleaning problems. For this reason, blasts must be properly designed to be effective.

When an explosive is detonated, compressive waves (shock waves) radiate outwards from the hole. As the stress wave is propagating outwards, a chemical reaction occurs resulting in the creation of expanding gases. The expanding gases create pressure that acts on the rock for a longer period of time than the stress wave.

The expanding gases, under pressure, penetrate the cracks caused by the stress wave. The cracks then extend until the pressure drops below a critical level. Certain cracks will extend further than others, as the explosive gases will take the path of least resistance.

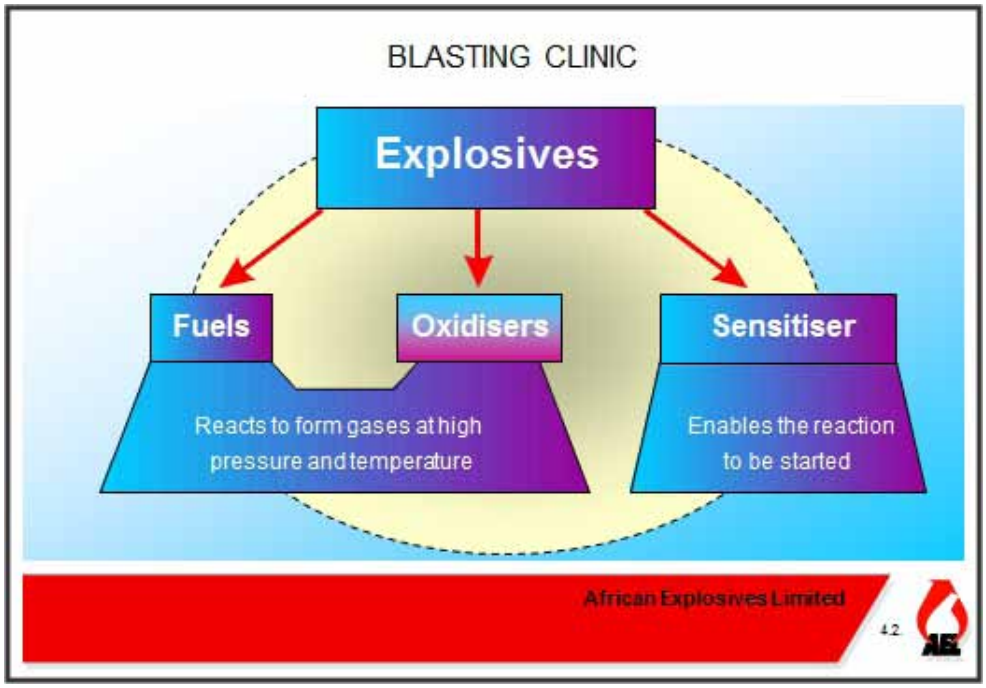


Figure 10: Composition of explosives (Courtesy AEL)

Damage Zones

Three distinct zones of damage can be identified around such a blast hole, as shown in Figure 11 below.

Figure 11: Zones of failure around a blast hole

Zone 1	Represents a zone of complete shattering of the rock. In this zone, the compressive stresses created by the shock-wave exceed the compressive strength of the rock.
Zone 2	Begins where zone 1 ends and is characterised by radial cracks. The cracks are produced by the tangential component of the stress wave. Under the influence of the wave, the rock particles are moved radially outwards, resulting in a tensile stress in the tangential direction. As the tensile strength of the rock is considerably less than its compressive strength, this tangential stress is large enough to cause radial fractures.
Zone 3	The stress wave entering this zone is too low in magnitude to cause further fracturing of the rock and there-fore merely dissipates into the rock mass.

Effect of stress on blast holes

Blast holes are blasted in a stressed rock mass. In deep mining situations where high vertical stress conditions exist, the major blast fractures are formed in the direction of this vertical stress.

As the blast holes are normally in rock that is subjected to varying levels of stress (where the major component is usually vertical in deep mines, but can be at an angle to vertical or horizontal in some mines), the fractures extend parallel to the major stress component more readily.

The ratio of the length of maximum to minimum blast fractures depends on the following:

- Stress condition of the surrounding rock
- Burden
- Type of rock
- Proximity of parting planes, etc.

In a case where the stress field is equal in all directions (hydrostatic stress field), the blast fractures around the blast hole will be approximately the same length around the blast hole.

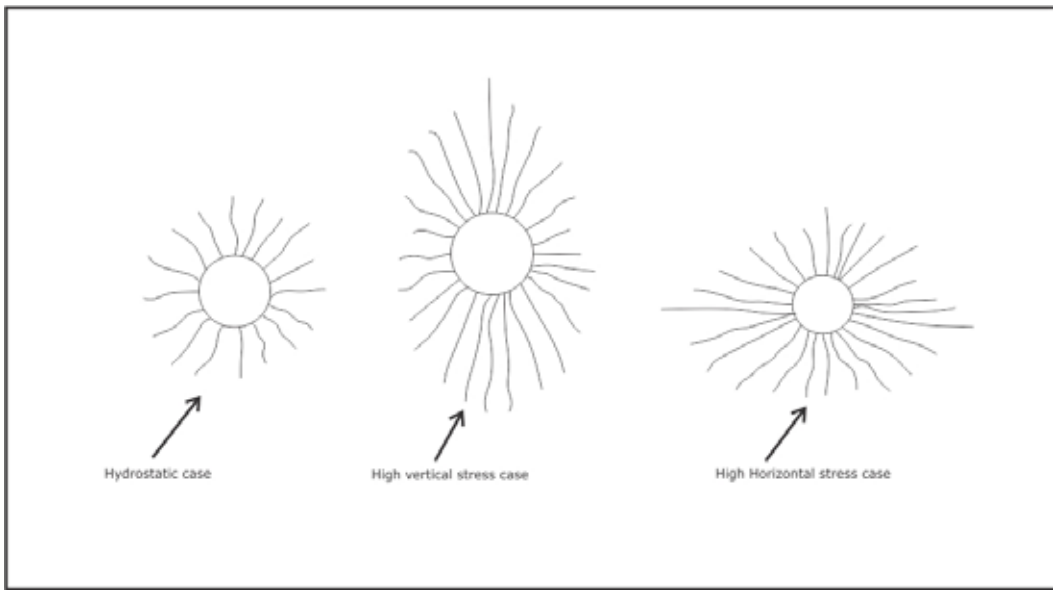


Figure 12: Blast fracture lengths and orientations under different stress conditions

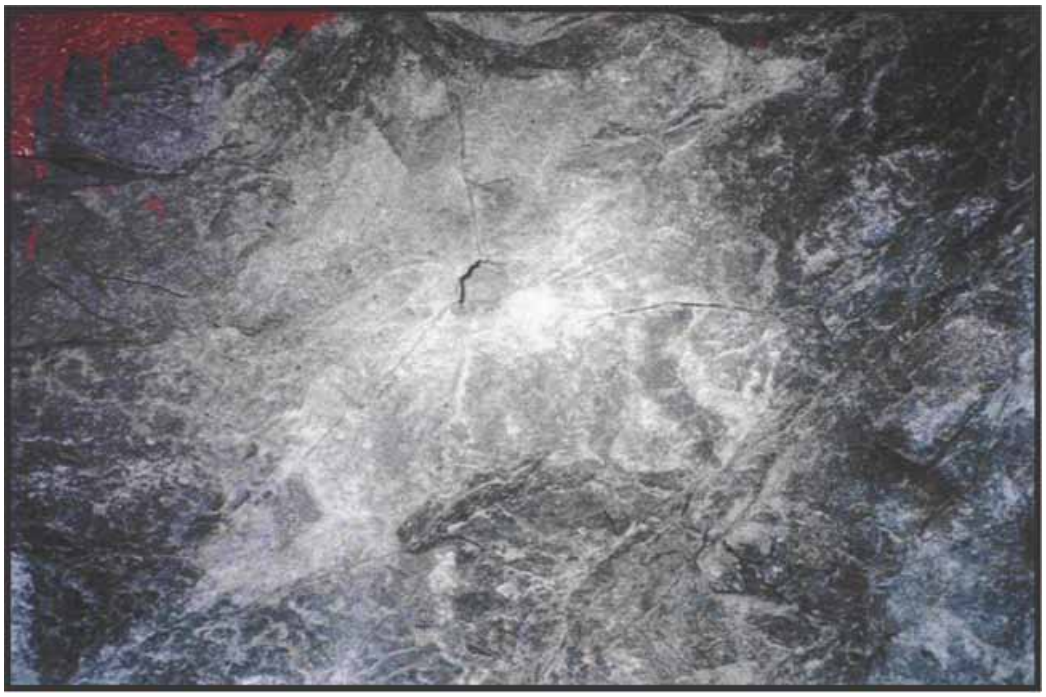


Figure 13: Blast fractures around a blast hole

NOTE

It is advised that Outcome 7.1.6 is also studied for a complete understanding of the above.

REFERENCE

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining
- Chapman & Hall, New York, Chapter 17
- AEL - Products and training material.

LEARNING OUTCOME 7.1.5

DESCRIBE AND EXPLAIN THE EFFECT OF SHOCK ENERGY ON ROCK FRAGMENTATION

The detonation of explosives has two resulting impacts on the rock:

- A shock wave (strain wave) that compresses and deforms the rock and causes formation of fractures; and
- An expansion of gas (created by the explosion) in the hole, and increase in gas pressure and an extension of fractures.

The shock wave (also called the detonation front) is in essence a compressive stress wave and fractures the rock based on the compressive strength of the rock material.

Fragmentation is a factor of the fractures created by the shock wave. It is therefore logical to conclude that fragmentation will be smaller with explosives with lower shock energy and better with high shock energy content. Since the shock energy is a function of the VOD of the explosive:

- High VOD indicates higher shock energy and therefore smaller fragmentation;
- Low VOD indicates lower shock energy and therefore larger fragmentation.

DEFINITION

VOD Velocity of detonation *– The velocity at which the detonation front (shock wave) moves through the explosive.*

NOTE

Different explosives have different VODs, for example

- *Primary explosives (lead styphnate, lead azide and mercury fulminate) – Typical VODs of 5200 m.s-1, 4630 m.s-1 , 4250 m.s-1 respectively.*
- *Secondary explosives (nitro-glycerine (NG), ethyleneglycoldinitrate (EGDN) and pentaerythorotetranitrite (PETN)) – Typical VODs of 7700m.s-1 , 7570m.s-1 , 8400m.s-1 respectively.*
- *Tertiary explosives (ammonium nitrates) – Typical VOD of 5270 m.s-1 .*

IMPORTANT

Extreme levels of shock energy could pulverise rock and, in many cases, this is also an unwanted result of the blast. Extreme care in the selection and application of explosives is required to attain the wanted result.

NOTE

Explosive studies have become a significant science and what happens during an explosion is not 'as simple' as described in this outcome. If more detail information on how explosions work is required, the user is advised to attend higher level explosive courses.

REFERENCE

- AEL – Products and training material.

LEARNING OUTCOME 7.1.6

DESCRIBE AND EXPLAIN THE EFFECT OF GAS PRESSURE ON ROCK FRAGMENTATION

As the strain (shock) wave propagates outwards, the gas behind the detonation front expands and since it is still confined in the blast hole, results in increased gas pressure within the hole. Gas in the blast hole can only reduce its pressure by expanding into fractures created by the detonation front (shock wave) and extending them.

This extension is occurring in tension (gas pushing the tips of the fracture apart) and the extension of the fracture is therefore a function of the tensile strength of the material.

INTERESTING INFO

The in-situ stress field present before the explosion also affects the extension of the blast fractures (as discussed in Outcome 7.1.4). After the fracture is created by the shock wave, the in-situ stress field either assists in the development or prevents opening up of the fracture. The major stress component has two functions:

- It clamps the rock ahead of the fracture that develops in a direction perpendicular to the σ1 direction and
- It creates tension around the tip of the fracture that develops in the same direction as the σ1 direction.

If the in-situ σ1 component assists the extension of the fracture by creating tension around its tip, the fracture is more easily extended and will develop faster than when the in-situ σ1 prevents opening of the fracture (Figure 11). The result is longer blast fractures in a direction parallel to the major principle stress and shorter blast factures in the direction of the minor principle stress.

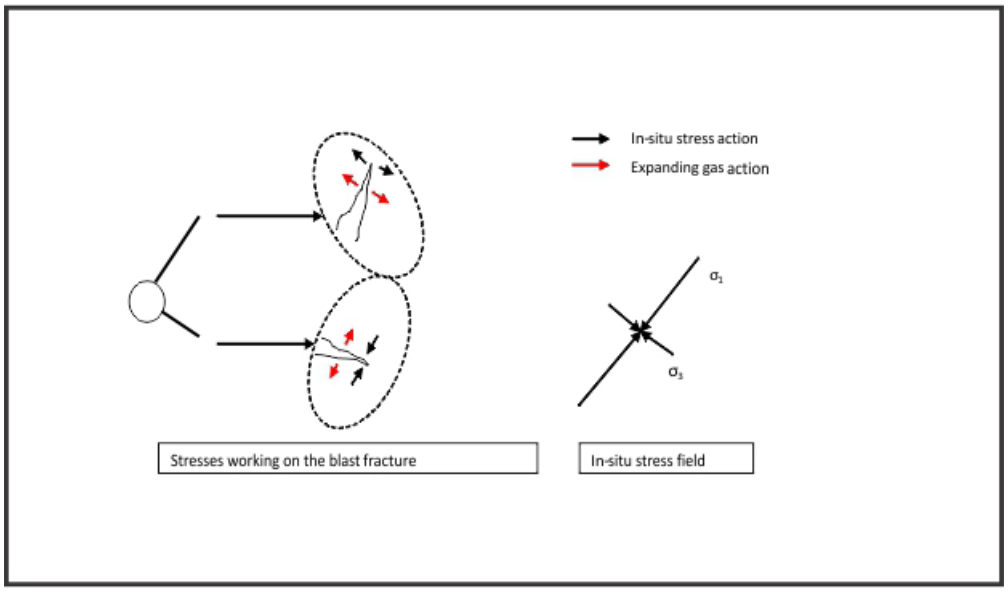


Figure 14: Stresses working on the blast fractures

NOTE

It is therefore possible to estimate the direction of the major principle stress purely from a study of blast fractures around a hole in which explosives had been detonated (socket).

Gas pressure (also referred to as heave energy as it throws the blasted material) extends the blast fractures that intersect other weakness planes and fractures, resulting in fragmentation of the rock.

When blast fractures intersect geological structures, the expanding gas could take 'the path of least resistance' and may therefore easily penetrate the planes, open them up and causing separation and possibly instability.

Figure 15 shows the effect of increasing the distance of the blast hole to the free face, i.e. increasing the burden (B). Fractures will only propagate (or extend) as long as the gas pressure is maintained in the blast hole and blast fractures. This pressure is released when any of the propagating fractures reach a free surface and the gas can 'escape' to the atmosphere. As soon as this happens, the gas pressure drops and the rest of the fractures cease to extend. If burdens are too large, the gas pressure may result in fractures extending into the hangingwall of the excavation, assisting in causing ground instability, or 'over break' (breaking the excavation to larger diameters than planned).

The burdens of blast holes are therefore a very important factor to consider in blasting practice. For a given blasting situation, there is a fixed relationship between burden, diameter of the hole, type and quantity of explosives used.

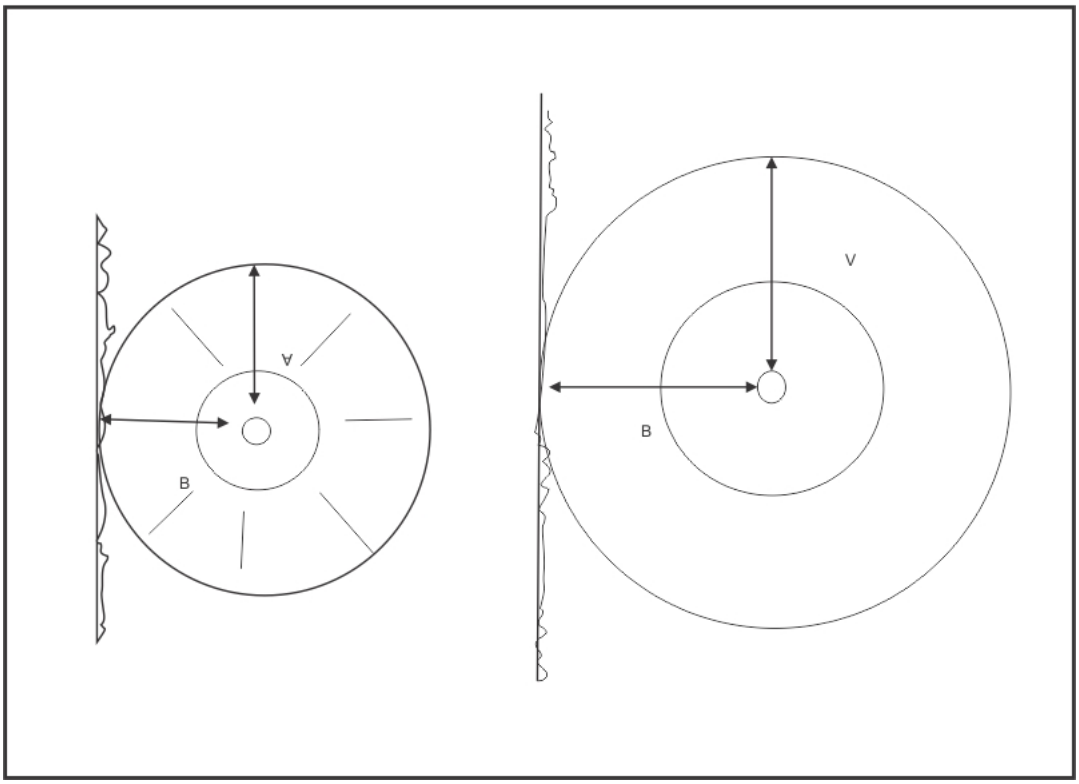


Figure 15: The effect of increasing the distance of the blast hole to the free face, i.e. burden (B)

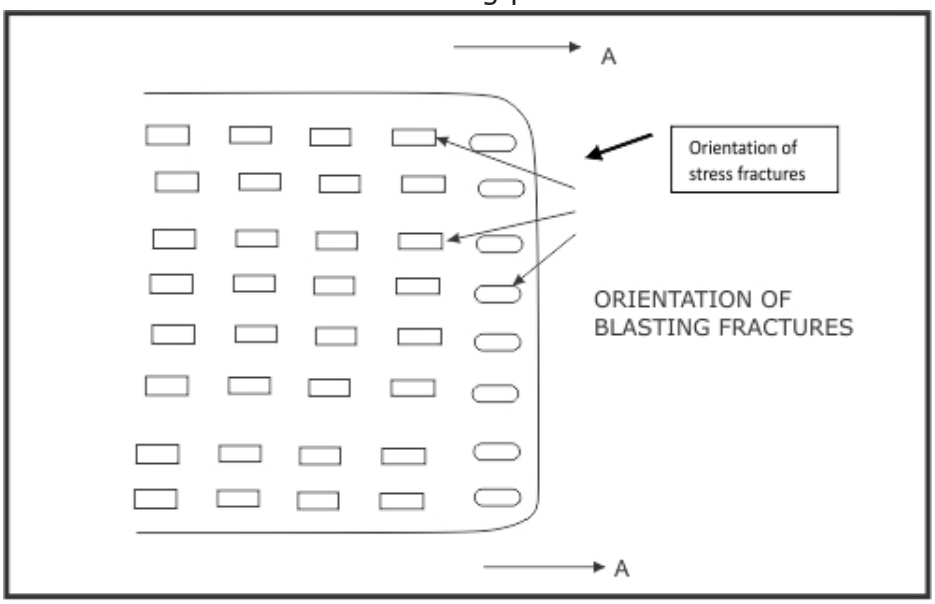
Signs of too large burdens include:

- *extensive damage to the perimeter of the excavation,*
- *falls of ground,*
- *blocky hangingwall conditions,*
- *over break,*

back

- *large fragmentation,*
- *poor advance and*
- *long sockets.*

Blocky hangingwall conditions in hangingwalls of excavations are the result of the intersection of blast-induced fractures and jointing or stress fracturing. For a standard mining panel face area (same principles applies to other excavations), Figure 16 shows the orientation of blast fractures along the hangingwall (Figure 15a) and into the hangingwall (Figure 15b), where the latter also shows the creation of small blocks when blast fractures intersect bedding planes.



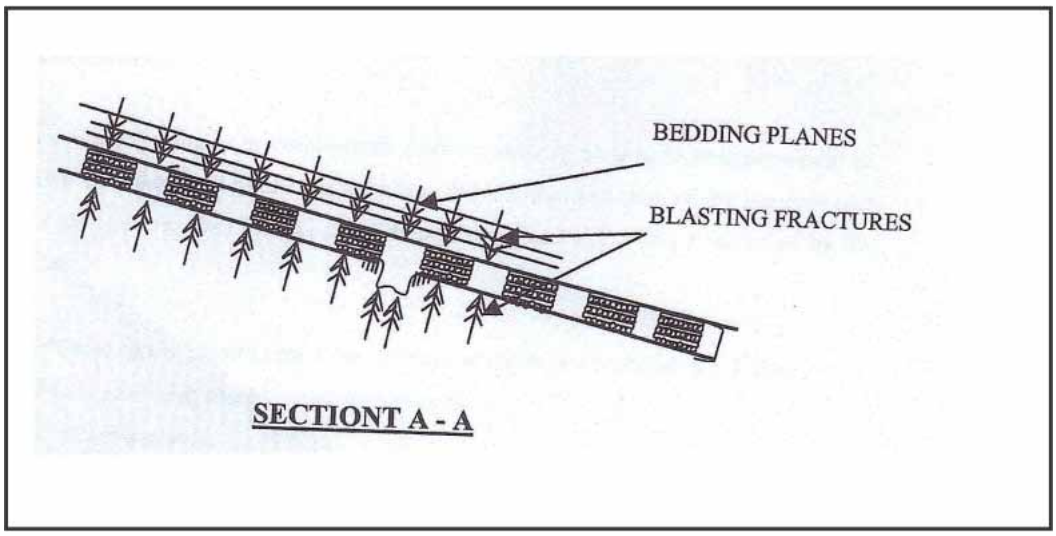


Figure 16: Orientation of blast fractures (a) along the and (b) into the excavation hanging wall

The orientation of stress fractures ahead of the mining face and detrimental joint orientations is superimposed on the original Figure 16a and is shown below, suggesting the formation of blocky (small blocks) hangingwall conditions.

back

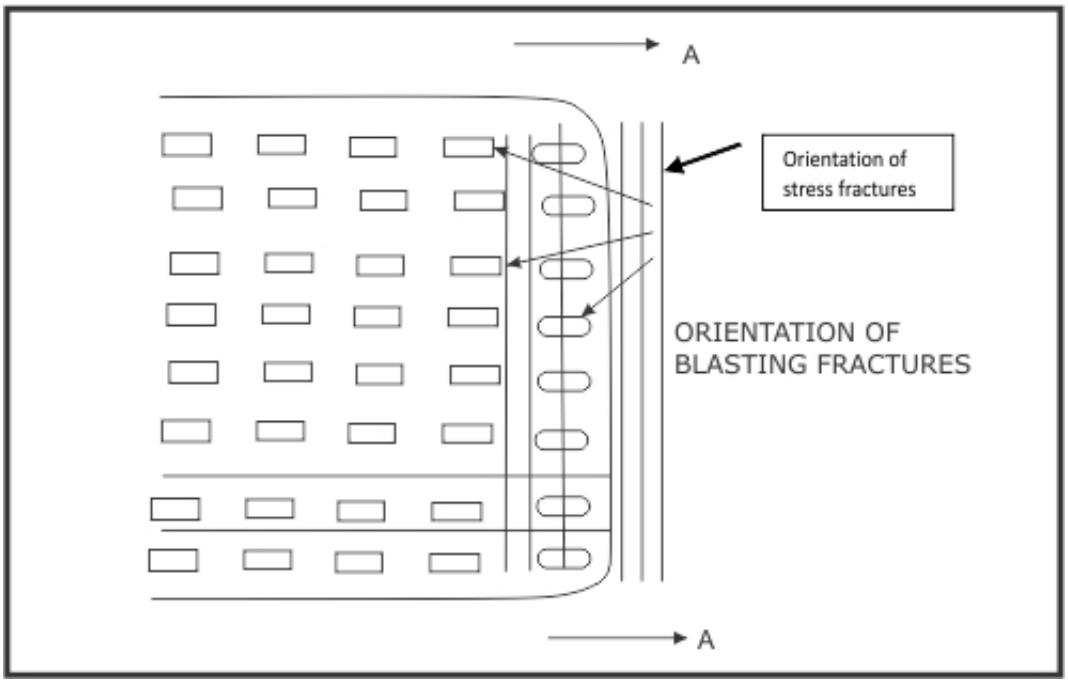


Figure 17: Blocky hangingwall conditions

REFERENCE

- AEL – Products and training material.

LEARNING OUTCOME 7.1.7

APPLY KNOWLEDGE OF EXPLOSIVES AND THEIR CHARACTERISTICS TO SELECT APPROPRIATE EXPLOSIVES FOR A GIVEN TASK

NOTE

Explosive type selection is a not a menial task and should be left to specialists in this field, such as explosive engineers. Most of the explosive supply companies employ engineers that could assist with blast and explosive design.

Some basic principles should, however, be known so that the impact of those types on rock mass conditions can be evaluated:

- The 'strength' of explosives is indicated by the velocity of detonation or VOD (See Outcome 7.1.5). The higher the VOD, the greater the shock energy and the more fractures are created by the shock wave;
- The throw capacity of the explosive is a function of the gas pressure created by the explosive;
- The selection or evaluation of an explosive for a given conditions is then a function of the selection of the most appropriate 'shock' and 'gas' characteristics of the explosive.

NOTE

It is advised that these characteristics of the explosive type used on the operation are known and that a comparison is made to other available explosive types.

- In poor ground condition areas, stability is a large concern and the loosening of rock blocks by excessive gas content of an explosive can be detrimental. This confirms why so many mines, in these conditions, move away from Anfex (high gas content) and towards low density Anfex (reduced gas), gels (cartridge) or emulsions, known to create lower gas pressures;

- In very competent and massive rock material, the creation of blast fractures is essential to fragmentation, as few inherent planes are present to assist. A high VOD explosive is therefore preferred.

REFERENCE

- AEL – Products and training material.

LEARNING OUTCOME 7.1.8

DESCRIBE AND EXPLAIN THE FUNCTION OF STEMMING

DEFINITION

Stemming / tamping is the process of filling the empty part of a blast hole with an inert stemming material (sand, gravel, or clay) to retain the products of the detonation of an explosive charge in an enclosed space.

In underground as well as with surface blast holes, the blast holes are stemmed after the charging operation has been completed. Stemming increases the resistance to the escape of gases from the blast hole. It also lengthens the duration of application of the explosion energy to effectively break and throw the rock to a particular distance from the working face.

The length of the stemming in the blast hole is approximately 8 to 10 times the diameter of the blast hole, if the most effective materials and the most efficient explosive charge arrangements are used.

Stemming materials are normally friable materials that are:

- inert to the action of an explosion,
- are sufficiently hard, and
- have a high coefficient of internal friction.

The best stemming materials are reportedly:

- crushed stone chips (with water filling the spaces between the grains),
- coarse sand,
- clays,
- water-filled plastic ampoules and
- fast-hardening concretes.

EXAMPLE

Stemming of blast holes in the case of pre-conditioning is very important as an effective stemming will maximise the stemming retention time, which will contain the explosive energy in the hole for as long as possible. Poor or no stemming can result in misfires of the production and pre-conditioning holes. As can be seen from Figure 18, the front 1/3 of a preconditioning blast hole must be tamped with competent stemming material.

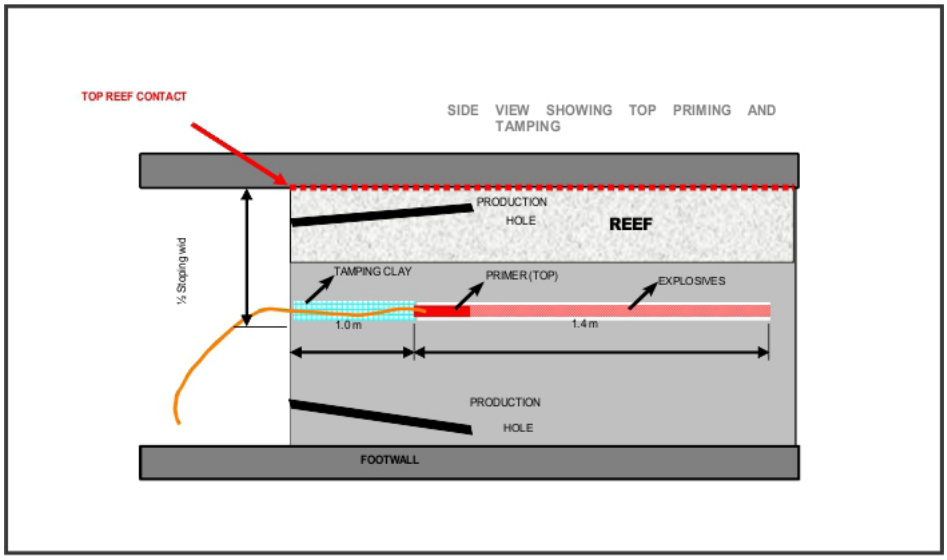


Figure 18: Stemming (tamping) of the first 1/3 of a pre-conditioning blast hole

NOTE

Production personnel do not always use tamping in the blast holes, as stipulated in the mine's specific standards, as it is considered extra work. From tests done, it was evident that escaping high speed gases during the explosion carry away more than half the energy provided by the explosives when tamping is not used or holes are incorrectly tamped. This escaping energy damages installed support and results in an increase in explosive costs and falls of ground that may injure workers.

IMPORTANT

Tamping using explosives is not acceptable. There is a tendency to use Anfex explosives to 'tamp' the hole by filling the complete hole. Testing of this practice against all the information given in outcomes in this chapter concludes that such a practice is technically flawed and practically damaging.

7.2 DRILLING

LEARNING OUTCOME 7.2.1

DESCRIBE AND EXPLAIN THE MECHANISM OF ROCK BREAKAGE BY CHISEL BIT

Historically, blast holes were made using a hammer and a steel chisel. The chisel head was rotated continually with each hammer blow to be effective. The chisel bit generally contains tungsten carbide type as the chisel bit. In modern mining, jackhammers are used, which automatically hammer and rotate a drill steel having a chisel head (Figure 19a). These jackhammers are normally air, hydro- of electrically driven. The drill hole is flushed with water from the drill machine, through the drill steel and is discharged at the drill bit to flow around the drill steel out of the hole, removing particles from the hole.

NOTE

Solid drill steels with a chisel head were used in mining, but due to high costs, steel 'knock-off bits' were developed ('knock-off' refers to bits that could be removed from the steel stem or 'jumper').

Figure 19: Drill bits (a) chisel head and (b) button bits

- Underground mining equipment – Atlas Copco at http://www.atlascopco.co.za/zaus/products/navigationbyproduct/index.aspx#1460287

LEARNING OUTCOME 7.2.2

DESCRIBE AND EXPLAIN THE MECHANISM OF ROCK BREAKAGE BY BUTTON BIT

Current drilling methods also make use of button bits (Figure 18b), where commonly tungsten carbide type bits are set within the drill bit. This drill bit arrangement has been found to be very effective in larger boreholes in hard rock mining environments. The mechanism of rock breakage is similar to the chisel bit where the jackhammer provides the hammering and rotation action. Water is pumped through the small hole in the bit to clear the sludge.

- Rock Tool Webpage
- Underground mining equipment – Atlas Copco at http://www.atlascopco.co.za/zaus/products/navigationbyproduct/index.aspx#1460287

LEARNING OUTCOME 7.2.3

DESCRIBE THE TYPES OF EQUIPMENT USED BY THE FOLLOWING DRILLING METHODS:

- Percussion drilling
- Rotary drilling
- Diamond drilling
- Raise / Tunnel boring

Percussion drilling

With percussion drilling methods used in mining drilling, machines may vary from small handheld drilling machines to large hydraulic mechanised drill rigs as used on surface mines. Small handheld percussion machines are powered by either compressed air or hydropower. Larger machines are mechanised and utilise electrical or hydraulic motors to power the drilling motors.

back

Figure 20: Commonly used jackhammer (Photo from Rock Tools webpage)

Figure 21: ROC F7 Drill Rig working on a surface mine (Atlas Copco)

back

back

Rotary drilling

Rotary cutting is normally carried out using claw or drag bits, which works well in soft ground conditions. When the rotary crushing method is used, tricone bits are used and a down thrust is applied to crush the rock.

Figure 22: Bit for rotary drilling

Figure 23: A T4BH rotary drill rig used in surface mining (Atlas Copco)

back

back

Diamond drilling

In diamond drilling, rotary machines are also used, but the drill bit has a hollow inside section (barrel) and the diamond bit has industrial diamonds set into the bit to actually cut the rock as it rotates. This type of drilling provides core samples of the rock drilled for geological and geotechnical scrutiny.

Craelius diamond drill bit.

Figure 24: Typical diamond drill bit (Atlas Copco)

back

Figure 25: Computerised diamond drilling machine (Atlas Copco)

Raise / Tunnel boring

Raise boring machines are normally large rotary machines that drill large diameter holes for ventilation raises, ore passes and shafts in underground mining operations. Large excavations are blasted and equipped to house the drilling machine that is removed on completing of the excavation. Pilot holes are first drilled and then reamed to the required dimension of the excavation.

Figure 26: Pilot drill bit used in raise boring (Atlas Copco)

Figure 27: Reamer bits typically used in raise boring (Atlas Copco)

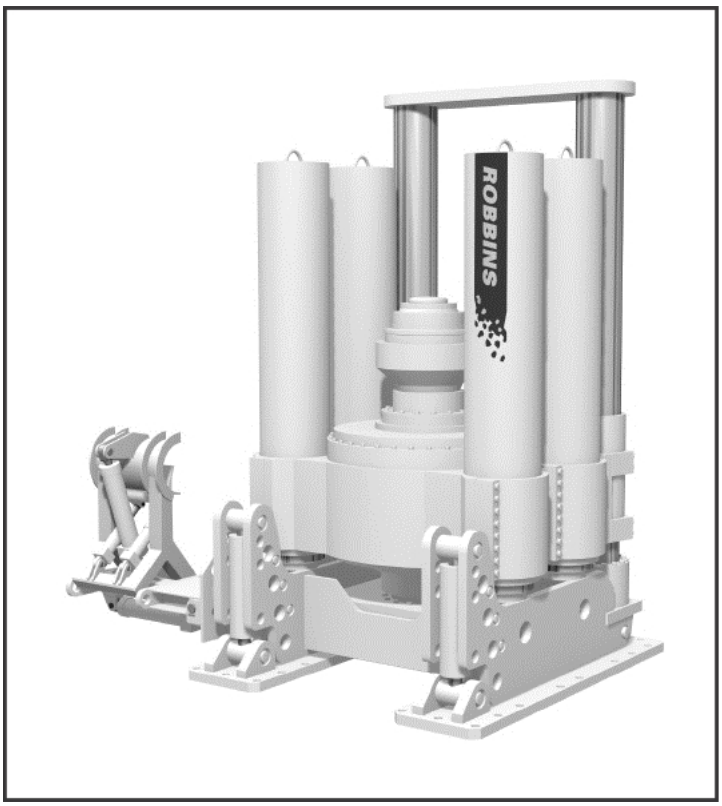


Figure 28: Robbins Raise boring machine (Atlas Copco)

REFERENCE

- Underground mining equipment – Atlas Copco at:
- http://www.atlascopco.co.za/zaus/products/navigationbyproduct/index.aspx#1460287

LEARNING OUTCOME 7.2.4

DESCRIBE, EXPLAIN AND DISCUSS THE USE OF THE FOLLOWING DRILLING METHODS:

- Percussion drilling
- Rotary drilling
- Diamond drilling
- Raise / Tunnel boring

Percussion drilling

Percussion drilling is used in normal stoping and development operations to create blast holes.

In percussion drilling methods, energy is transmitted from the rock drill to the drill steel and drill bit to the rock, where it crushes the rock as a result of the hammer action of the rock drill. In hard rock environments, the percussion pressure results in maximum achievable energy per blow. In softer ground, the percussion pressure per blow needs to be reduced to reduce recoils. The drill feed supplied from an air leg in hand-held percussion drill machines should be enough to keep contact with the rock, but still allow rotation. The purpose of rotation is to rotate the bit to a new position for the next blow, thereby effectively chipping the rock in contact.

Rotary drilling

Rotary drilling is extensively used in diamond bit drilling methods where core is recovered for sampling and geological and geotechnical scrutiny.

Rotary drilling is also used in developing tunnels in soft rock such as coal and in long hole drilling as well as raise boring methods.

In rotary drilling, the drill bit is kept in contact with the rock by applying a pressure and the rock is chipped or grinded away due to the rotary action. The main difference of rotary drilling is the absence of percussion.

Rotation is provided by a hydraulic or electrical driven gearbox called a rotary head.

Diamond drilling

Diamond drilling is extensively used in mining environments where drill core is recovered for sampling, as well as geological and geotechnical scrutiny.

Rotary, diamond drilling machines are used in diamond drilling where machines used underground are normally light machines to facilitate easy transporting and setting up along underground excavations.

Rotation is provided by a hydraulic or electrical driven gearbox called a rotary head.

Raise / Tunnel boring

Raise boring and tunnel boring methods are used underground to excavate raises, ore passes, travelling ways and even vertical shafts.

The method involves the use of a large raise bore drilling machines. These machines can drill vertically to inclined holes ranging from 2- 65m diameters. Many systems work on:

- drilling of a pilot hole and
- then to ream the hole to the size required for the excavation.

back

As the resulting circular hole is a competent shape for excavations, it can be used in high stress situations rather than conventionally developing the excavation. Tunnel boring machines (Figure 29) have been developed to provide tunnels at low risk to personnel safety (and high production levels) and are used extensively in other countries.

Figure 29: Tunnel boring machine

NOTE

A large number of rock-related casualties have occurred while developing steeply inclined excavations (box holes, ore passes) conventionally. With the raise boring method, no persons work inside the excavation and this provides a significant safety advantage.

REFERENCE

Underground mining equipment – Atlas Copco at http://www.atlascopco.co.za/zaus/products/navigationbyproduct/index.aspx#1460287

LEARNING OUTCOME 7.2.5

DESCRIBE AND EXPLAIN HOW CUTTINGS ARE REMOVED FROM THE HOLE FOR EACH OF THE ABOVE DRILLING METHODS

Percussion drilling

Flushing is provided from a water feed and is necessary to remove drill cutting and chips from the drill hole in percussion drilling methods. Larger percussion drilling machines use compressed air to remove cuttings from the drill hole.

Rotary drilling

Flushing of rotary drill rigs to remove cuttings is normally done by means of compressed air.

back

back

Diamond drilling

Water flushing is used in diamond drilling methods for:

- Removal of cuttings;
- Cooling of bit;
- Lubrication of drill string;
- Reduction in vibration; and
- Stabilisation of hole walls.

NOTE

Bit life can be increased with proper flushing.
The actual core drilled is done by pulling the core barrel from the hole and removing the core.

Raise / Tunnel boring

In the case of raise boring methods, the drill cuttings need to be removed only when drilling the pilot hole. Most raise boring machines use a drive train featuring a hollow centre, enabling the efficient transmission of any flushing medium to clear the pilot hole from cuttings and sludge. In this case, the flushing medium can consist of water under pressure or compressed air.

7.3 BLASTING

LEARNING OUTCOME 7.3.1

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF THE FOLLOWING PARAMETERS ON BLAST DAMAGE:

- Explosive type
- Initiation method
- Initiation sequence
- Hole orientation

CONNECTION 69

Refer to "LEARNING OUTCOME 7.1.5" on page 544 to "LEARNING OUTCOME 7.1.7" on page 548

Explosive type

In selecting the explosive type to use, several rock properties determine the performance of a particular explosive in the medium. These include:

- The capacity of the rock to transmit energy is related to the Young's Modulus of the rock.
- The ease of generating new fractures is a function of the uni-axial compressive strength of the rock.
- The unit weight of the rock to be displaced.

NOTE

Brady and Brown suggested an empirical matching of explosive and rock mass properties as shown in Figure 30 below.

back

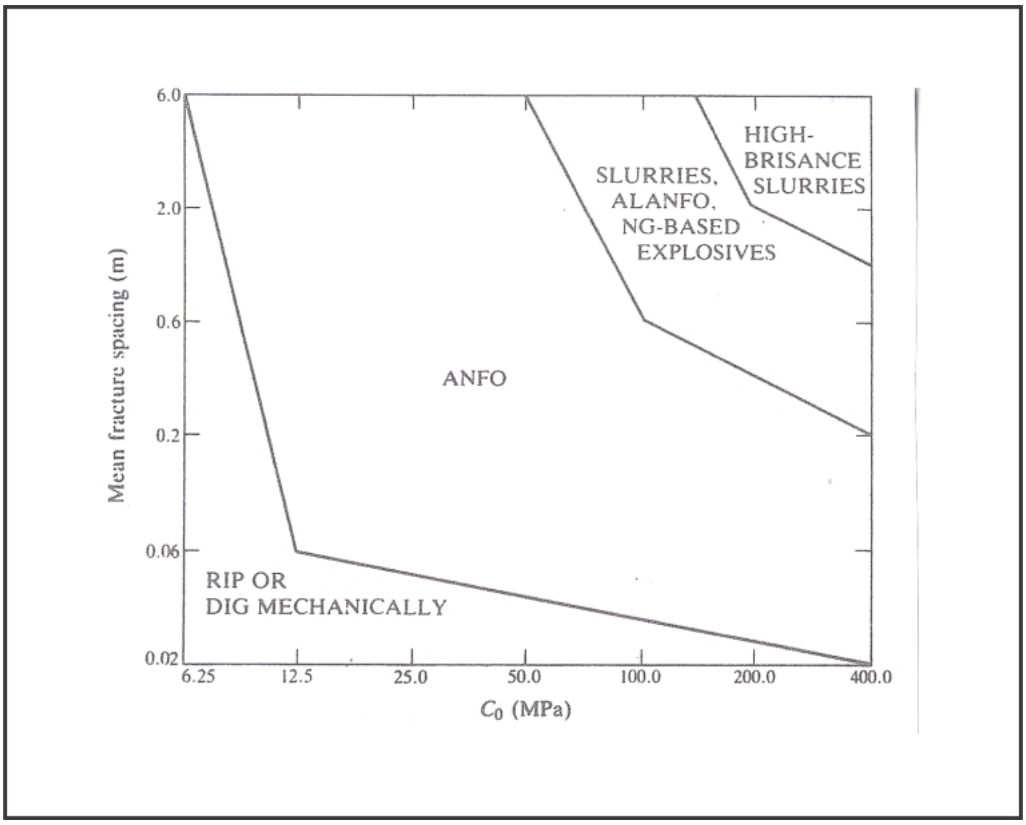


Figure 30: Empirical matching of explosive and rock properties (Brady and Brown)

Over and above using the correct explosive, the explosive used must also be used correctly to prevent damage to the rock mass. Some suggestions are given below:

- Some mines' specific standards stipulate that only a portion of the blast hole should be filled with explosives and that tamping should be used to optimise the energy from the explosion.
- In some cases, where tamping is not used, the production crew fills the complete hole with explosives, which means that much more than the required energy is released that may seriously damage the rock mass and support units in close proximity.
- Poor drilling and blasting practices may result in excessive burdens that cause unfavourably orientated fractures and poor conditions that may seriously affect the long-term stability of an excavation.
- Overcharging, excessive burdens and poorly directed blast holes may cause excessive hangingwall vibrations that may loosen rocks and cause falls of ground.
- The worst condition for over break occurs when the gases are confined for unusually long periods in the blast hole as a result of an overburdened charge (wide spacing of holes). During long periods of confinement, the gases penetrate any pre-existing fracture or plane near the blast holes
- Poorly directed, overcharged and out of sequence blast holes may fracture and damage the surrounding rock walls and must be avoided at all times

Initiation method

Most initiation methods used on mines consist of electric blasting using electro-detonators, where a detonator sets off the blast round electronically.

- The detonator is coupled to the blast round with stope igniter cord in the case of stoping and slow igniter cord in the case of development ends.
- The igniter cord is coupled to the blast rounds to ensure a specific blast hole sequence where it is coupled to the blast hole with detonating fuses.

Any malfunctioning of the initiation system used may result into out-of-sequence firing of the blast hole and damage to the rock mass.

Mines must use the best initiation system proven for their mining conditions and should test other options as the technology around initiation systems improve. Options available currently include:

- capped fuse and igniter cord;
- shock-tube systems,
- electronic systems such as electrodets / smartdets.

Initiation sequence

Blast holes must explode in a certain sequence to effectively break the rock. This sequence of firing is also called the 'timing' of the blast round. In cases where the igniter cord is incorrectly connected, the timing of the round will be incorrect. Such poor timing may result in 'out-of-sequence' or 'mis'-firing and cause damage to the surrounding rock mass.

When a poorly-timed blast is detonated, the holes in the blast explode 'out of sequence', resulting in a burden to the free face that is too big (Figure 30), causing damage to the surrounding rock mass and long sockets in the face.

In Figure 31, the designed sequence is '1,2,3,4' at burdens denoted by BD. If poor timing results in 'out-of-sequence' firing '1,3,2,4' (hole 3 explodes before hole 2), will result in an increased burden denoted by BPT.

back

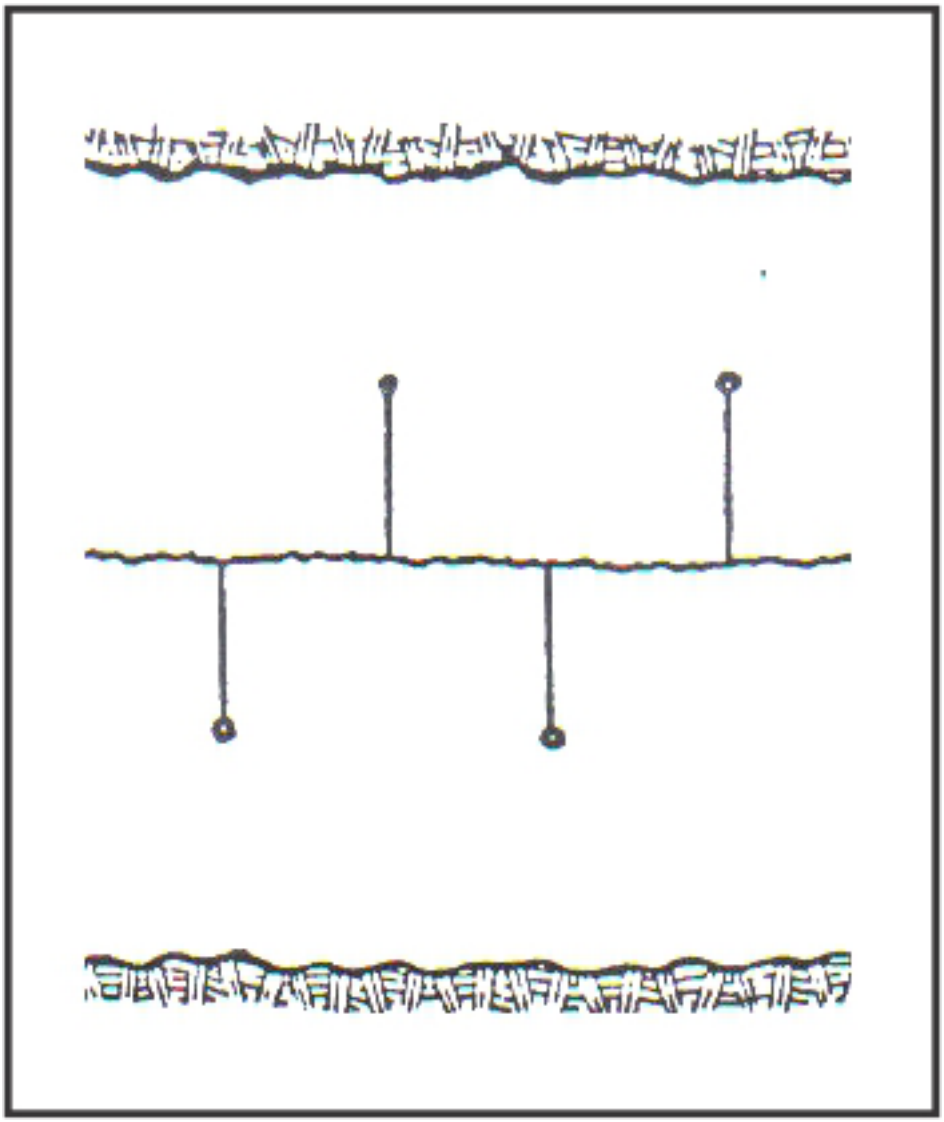


Figure 31: Poor timing resulting in excessive burdens

DEFINITION

Socket is that part of the blast hole that is still present in the rock face after the explosion has occurred and the area cleaned out.

back

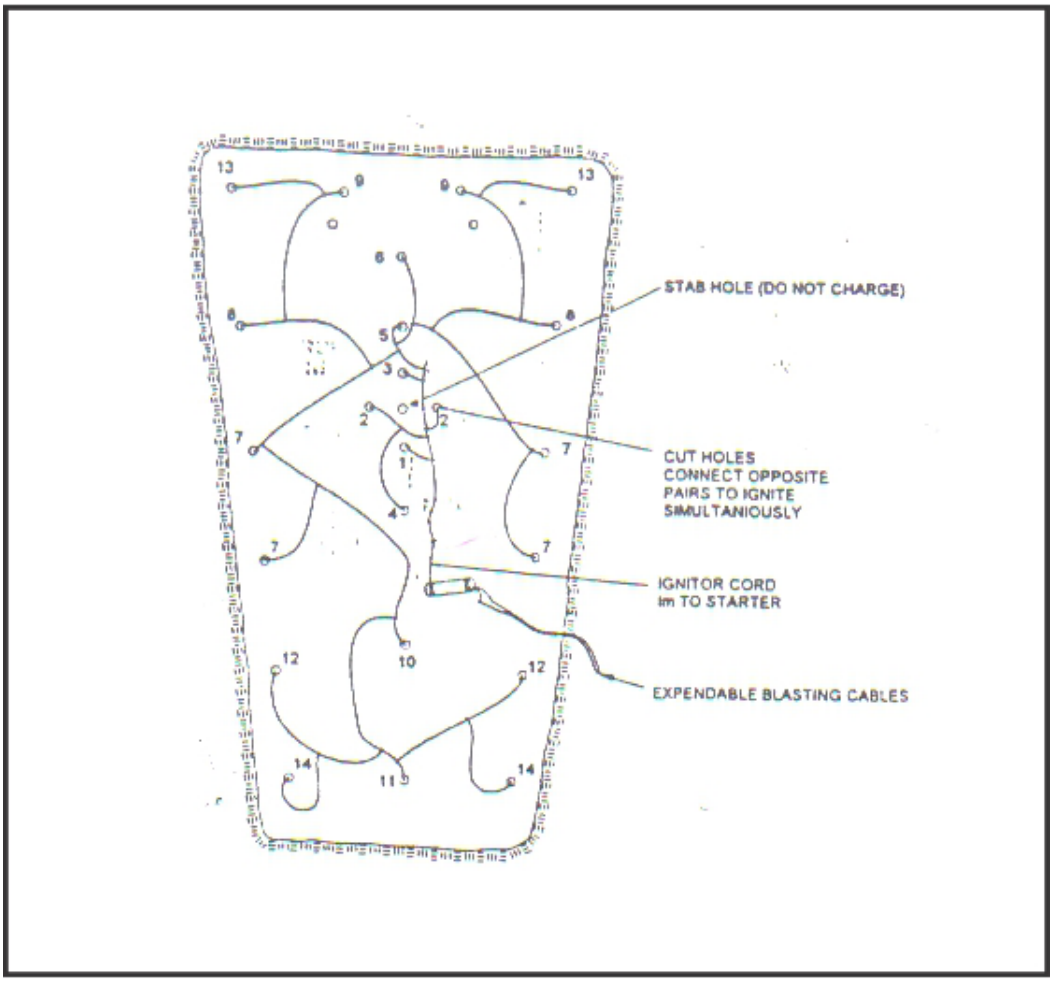


Figure 32: Typical initiation sequence for a blast round in a development end

In the case of pre-conditioning blast holes, the timing of the pre-conditioning holes relative to the production holes is very important. Poor timing may lead to ineffective pre-conditioning and / or misfires, which in turn may result in blast damage of the rock mass and the loss of any advantages that can normally be gained from pre-conditioning. The pre-conditioning hole should be timed such that detonation of the hole will take place approximately 1.0m ahead of the production holes.

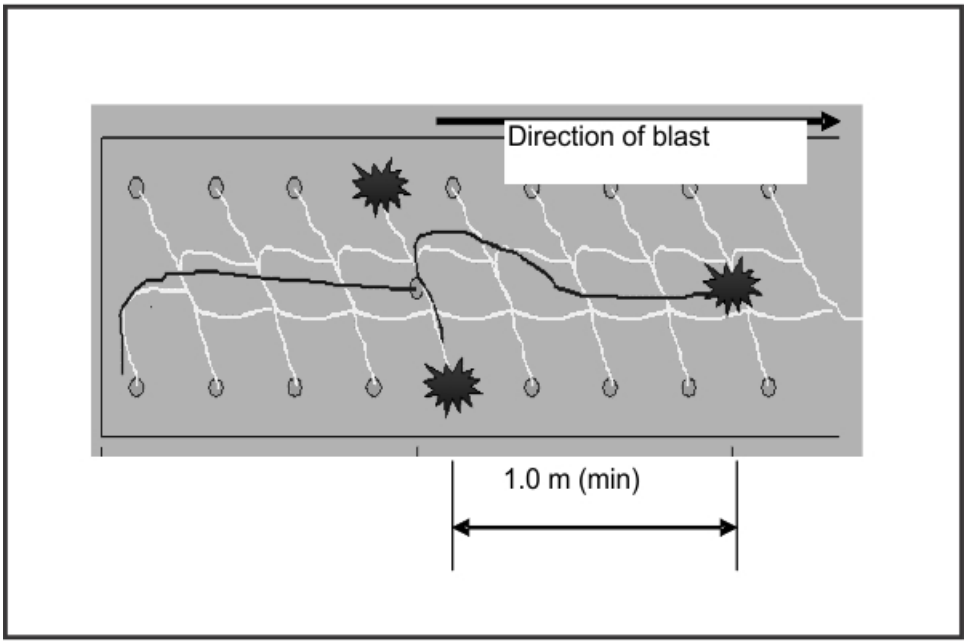


Figure 33: An initiation sequence for a pre-conditioning blast

Hole orientation

Blast holes must be drilled correctly to be effective. Incorrectly orientated blast holes increase hole burdens (Figure 34), result in blast damage and subsequent falls of ground and must be minimised.

The miner draws in the direction lines along the hangingwall / roof of an excavation and rock drill operators must drill on these direction lines to ensure 'back-of-the-hole' burdens equal to those marked off on the face.

NOTE

When drilling the bottom holes, the jumper can be left inside the top hole to assist the rock drill operator to judge the direction of the bottom holes.

Figure 34: Typical blast hole drilling directions in a stope

REFERENCE

- Rock Mechanics for underground mining – BHG Brady and ET Brown – Second edition

INTERESTING INFO

Blast damage can be assessed in many ways, including observations, but two methods exist that allow the auditor to compare different areas, developers, machinery, explosives etc in a less subjective manner: Percentage Over break and Half Cast Factor.

Percentage over break can simply be calculated using the following relationship:

% Over break
= [(excavation dimension – original dimension)/Original dimension] x 100.

Cutting excavations smaller than the intended dimensions will provide a negative %, while cutting excavations larger than intended will result in a positive %. This method can be applied to the width, height or circumference of tunnels, as well as the mining height or width in a stope.

Half-cast factor is applied in tunnels where smooth wall blasting techniques result in the visible signs of 'barrels' in the excavation walls. The HCF can be calculated from:

HCF = (total length of barrels measured) / (total length of holes drilled)

and is expressed as a factor, i.e. HCF < 1.

For example, an HCF = 0.85 means that 85% of the length of holes drilled was visible in barrels against the excavation walls. It is obvious that if HCF = 1, the blasting damage is zero and blasting practices are perfect.

LEARNING OUTCOME 7.3.2

DESCRIBE, EXPLAIN AND DISCUSS THE FOLLOWING CUSHION BLASTING TECHNIQUES:

- Smooth wall blasting
- Post-splitting
- Pre-splitting

NOTE

Cushion blasting is a term very familiar to open-cast miners and refers to the dressing of the highwall when workers trim excess material from the final high wall.

Cushion blasting is a term that suggests 'improving the finishing of the blasted excavation' is very applicable to both:

- surface mining, where the objective is to trim the final high wall to improve stability; and
- underground excavations, where the surrounding rock mass must be protected to improve conditions and stability.

Smooth wall blasting

It is sometimes necessary to cut an excavation to a specific shape such as a rounded or arched shaped roof. Smooth wall blasting techniques are used to achieve these shapes. In excavations that applied a smooth wall technique, the half barrels of the blast holes are visible around the perimeter of the excavation.

DEFINITION

Barrels – a part of the blast hole is visible in the excavation wall (Figure 35).

Figure 35: Blast barrels in the walls of an underground tunnel

In smooth wall blasting, the perimeter holes are closely spaced, charged with a decoupled explosive and are blasted in a separate blast before (pre-split) or after (post-split) the main blast.

DEFINITION

Decoupled – see "LEARNING OUTCOME 7.3.4" on page 569, "LEARNING OUTCOME 7.3.5" on page 569 and "LEARNING OUTCOME 7.3.6" on page 569

Post-splitting

Post-splitting is a perimeter blasting technique used in mining to produce a geometrically precise and relatively undisturbed ultimate surface. In post-splitting techniques, the perimeter holes are blasted in a separate blast after the main blast removed the centre portion of the excavation. The closely spaced post-split blast (perimeter) holes are charged up with decoupled explosives and are detonated simultaneously in a second blast (after the blast removing the centre of the excavation). The stress fields around each hole interact causing a tensile force along the plane of the holes, which then splits the rock along the line between the holes and provides a smooth surface.

As the removal of the inside portion of the excavation in the first blast actually creates a large free surface for the post-split blast holes to break into, the resulting surface after the blast should reveal the half barrels of the post-blast holes.

Pre-splitting

Pre-splitting isolates the blasting area from the rest of the rock formation by creating an artificial crack along the theoretical excavation plane. This is achieved by drilling a single row of relatively closely-spaced holes along the perimeter of the blast. The pre-splitting holes are fired before the rest of the round.

REFERENCE

- Rock Mechanics for underground mining – BHG Brady and ET Brown - Second edition.)
- Underground mining equipment – Atlas Copco.

LEARNING OUTCOME 7.3.3

DESCRIBE TYPICAL APPLICATIONS OF EACH

Smooth blasting

Smooth blasting is normally used in:

- Surface applications like road cuts, cuts for foundations
- Underground applications like tunnelling, underground chambers etc

Post-splitting

Post splitting is normally used in:

- smooth blasted excavations in deep mining environments
- trim blasts in surface mines to trim high walls to the correct positions.

Pre-splitting

Pre-splitting is extensively used in:

- surface mining environments where it is used as controlled blasting method that creates a plane on the desired line of break
- surface blasting like road cuts, foundations where concrete lining or structural concrete is poured directly against the rock wall, etc

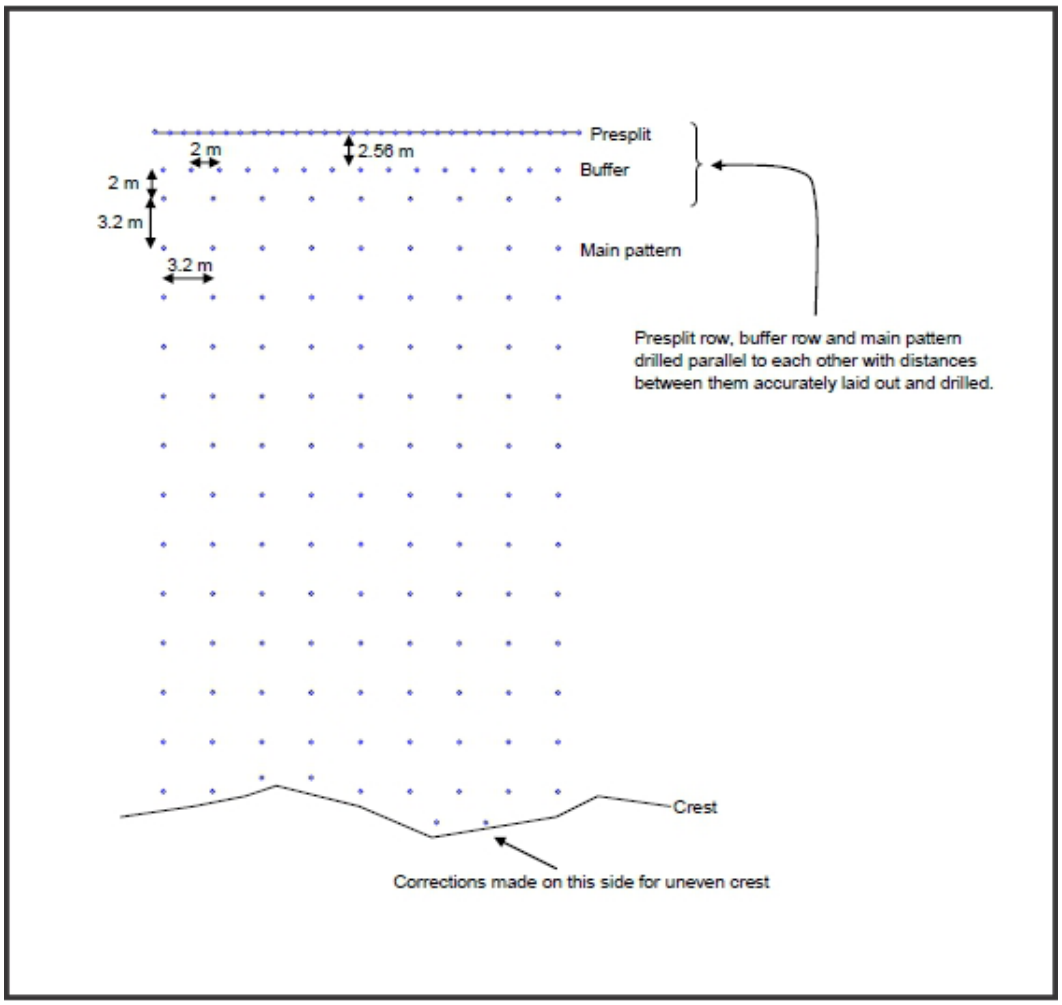


Figure 36: Pre-split in a drilling pattern used on an open pit mine

LEARNING OUTCOME 7.3.4

DESCRIBE, EXPLAIN AND DISCUSS THE OBJECTIVES OF DECOUPLING EXPLOSIVES

The objective of decoupling of explosives is to reduce the explosive energy by:

- reducing the volume of explosives in a blast hole and
- reducing the gas pressure by providing additional space for gas to expand into before it comes into contact with the rock wall.

By reducing the explosive energy, less blast damage to the immediate rock mass will result. Decoupling is therefore extensively used in post- and pre-split blasting, where the explosive interaction with the rock walls is reduced and controlled.

Decoupling is a ratio, expressed as a percentage, of the volume occupied by the explosive to the volume of the blast hole:

Decoupling % =(Volume of explsive / Volume of blast hole) x 100

LEARNING OUTCOME 7.3.5

CONNECTION 70

Refer to "LEARNING OUTCOME 7.3.4" on page 569

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF DECOUPLING EXPLOSIVES

LEARNING OUTCOME 7.3.6

DESCRIBE, EXPLAIN AND DISCUSS THE METHODS BY WHICH DECOUPLING OF EXPLOSIVES IS ACHIEVED

Decoupling is usually done by:

- separating the explosive charges with the use of stemming (tamping) material to fill the voids between the explosives in the hole, as can be seen in Figure 38;
- separating the explosives (Figure 37).

back

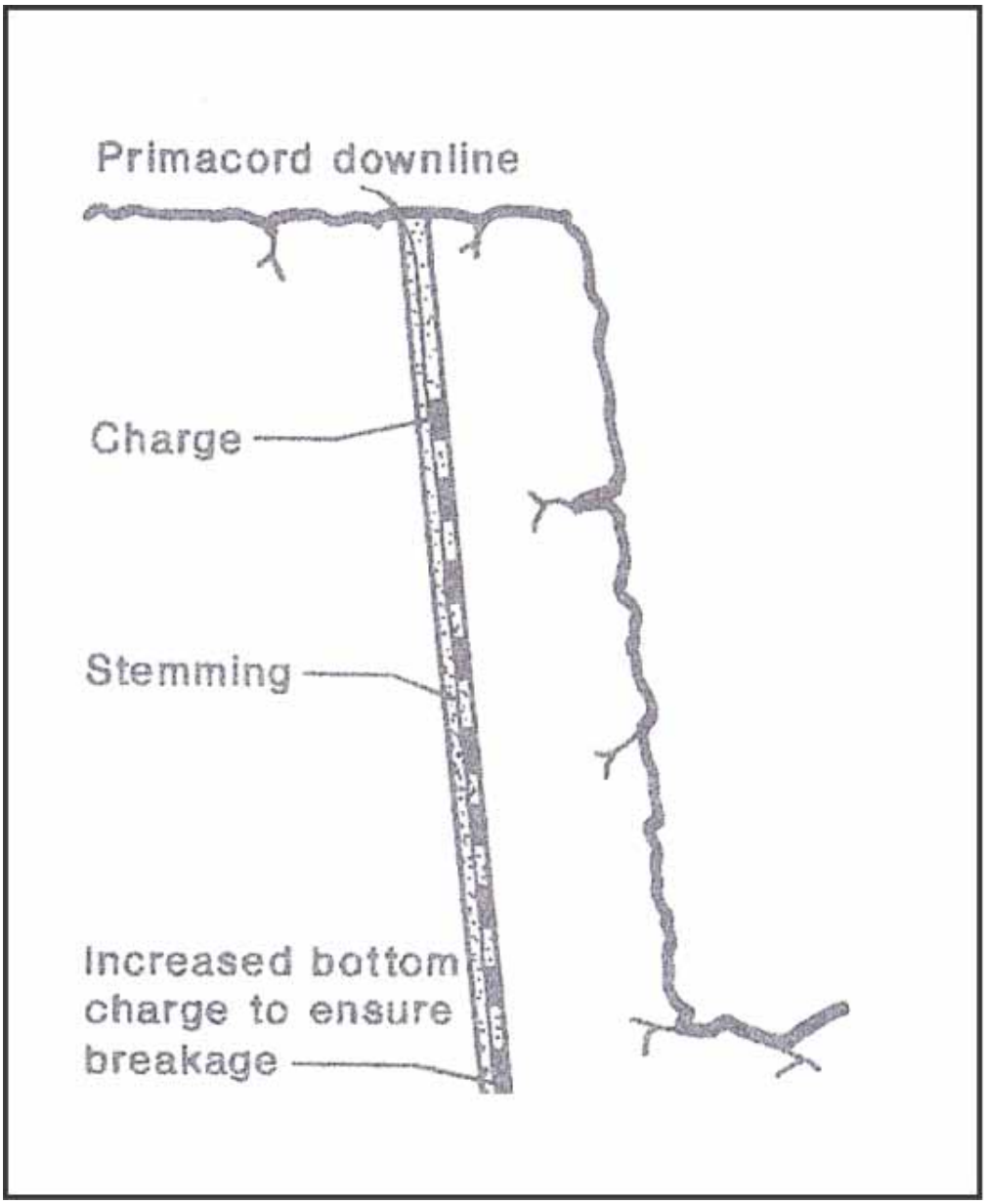
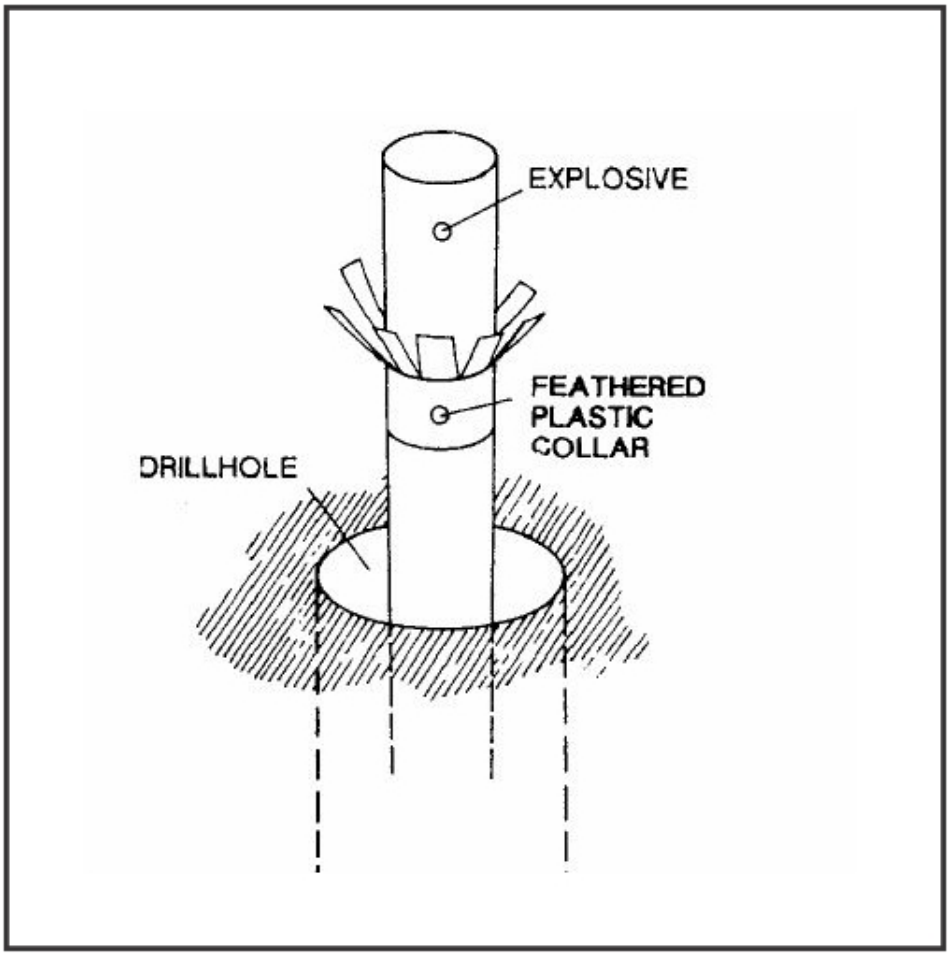


Figure 37: Decoupled blast hole

back



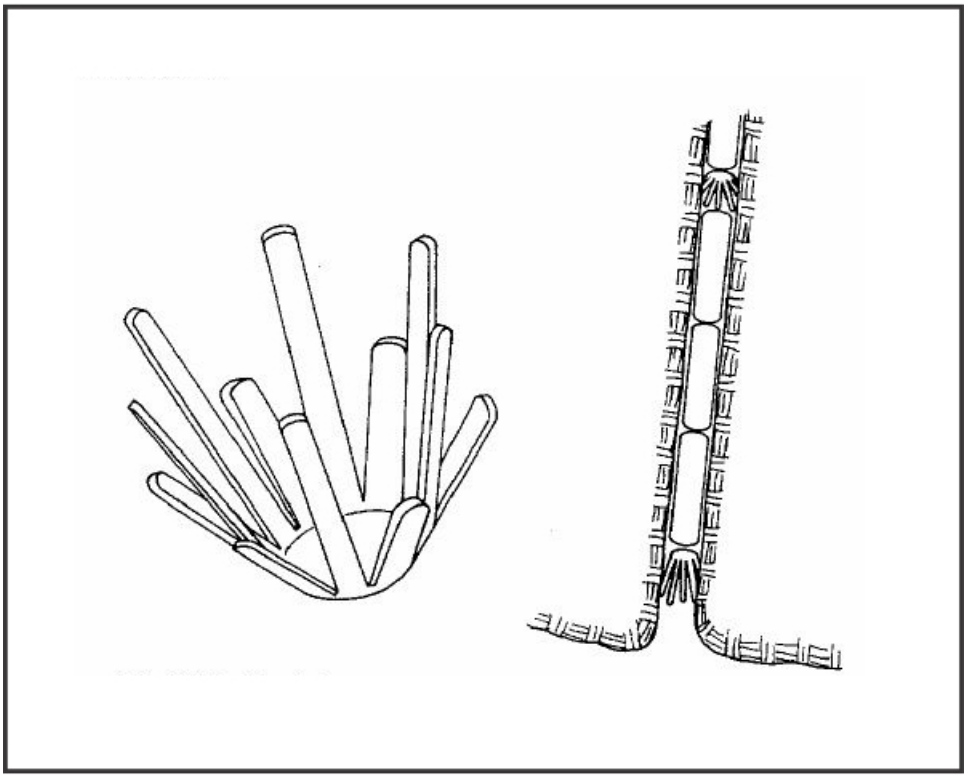


Figure 38: Methods to ensure decoupling in charged holes

CONNECTION 71

Refer to Glossary of explosive and blasting terms http://www.austinpowder.com/blastersguide/resources/Glossary.htm

CHAPTER

8

LEGAL COMPLIANCE AND GENERAL MANAGEMENT

INTRODUCTION

Chapter 8 deals with legal compliance and general management issues as captured in the Mine Health & Safety Act, the Regulations and Guidelines for Mandatory Codes of Practice. This legal compliance regime came about as a recommendation of the 1994 Leon Commission, which was set up to address some of the major mining-related hazards such as falls of ground. Legislation is therefore an integral part of mining and it is imperative that you view legislation as a tool to assist safe mining practices rather than a system that hinders mining.

Mining legislation has a hierarchy in the following order:

- 1st the Mine Health & Safety Act (passed by Parliament);
- 2nd the Regulations (passed by the Minister of Minerals & Resources); and
- 3rd the Guidelines of Mandatory Code of Practice (passed by the Chief Inspector of Mines).

In the general management of mining legal compliance, the employer is called to:

- Eliminate the hazard (e.g. stop using that energy source, etc).
- Minimise or replace the hazard (e.g. reduce the amount of energy or find an alternative, etc).
- Control the hazard (e.g. roof support, etc).
- Control the hazard (e.g. codes of practice, standard operating procedures, etc).
- Control the hazard (e.g. PPE, etc).

To assist the rock engineer in knowing and understanding his responsibility towards the Mine, Health and Safety Act and other forms of legislation, which all aim to achieve safe mining practices, this chapter deals with:

- The Mine, Health and Safety Act, Act 29 of 1996
- Regulations
- Code of Practices
- General management issues

back

8. LEGAL COMPLIANCE AND GENERAL MANAGEMENT

LEARNING OUTCOMES

8.1. MINE HEALTH AND SAFETY ACT AND REGULATIONS

8.1.1. THE MINE HEALTH AND SAFETY ACT, ACT 29 OF 1996

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to describe and explain the requirements of the Act in terms of:

8.1.1.1. Owner's responsibilities and duties
8.1.1.2. Manager's responsibilities and duties
8.1.1.3. Employee's rights and duties
8.1.1.4. Suspension of working due to inadequate safety
8.1.1.5. Manufacturer and Supplier's duties
8.1.1.6. Manager's duty to consult a Rock Engineer
8.1.1.7. Rock Engineer's responsibilities and duties
8.1.1.8. Rock-related codes of practice requirements
8.1.1.9. Hazard identification and risk assessment
8.1.1.10. Negotiation and consultation forum's function, duties and powers
8.1.1.11. Tripartite institution's functions and duties
8.1.1.12. Inspector' functions and powers
8.1.1.13. Requirements for enquiries into accidents
8.1.1.14. Minister's abilities and powers
8.1.1.15. Scope of safety-related regulations.

8.1.2. REGULATIONS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to describe and explain the requirements of regulations formulated in terms of the Act under the following headings:

8.1.2.1. Appointments and Administration
8.1.2.2. Duties and Responsibilities
8.1.2.3. Health and Safety Representatives and Committees
8.1.2.4. Inspectorate of Mine Health and Safety
8.1.2.5. Miscellaneous and General Provisions
8.1.2.6. Outlets, Ladderways and Travellingways
8.1.2.7. Protection of the Surface and in the Workings
8.1.2.8. Qualifications and Certificates of Competency
8.1.2.9. Surveying and Mine Plans
8.1.2.10. Tripartite Institutions
8.1.2.11. Definitions.

8.2. CODES OF PRACTICE

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to describe and explain the fundamental requirements of codes of practice to combat rock-related accidents as contemplated in terms of the Act and the DME Guideline for the Compilation of a Mandatory Code of Practice for:

back

8.2.1. Tabular Metalliferous Mines

8.2.2. Underground Coal Mines

8.2.3. Massive Mining Operations

8.2.4. Surface Mines.

8.3. BASIC RISK ASSESSMENT

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

8.3.1. Describe and explain the process and procedure of geotechnical hazard and risk evaluation and assessment.

8.4. GENERAL MANAGEMENT

8.4.1. REPORTING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

8.4.1.1. Describe the requirements and format of a formal technical report

8.4.1.2. Compile clear technical reports to senior mine officials on subjects covered in this syllabus

8.4.1.3. Compile clear technical reports to head office personnel on subjects covered in this syllabus.

8.4.2. MINE ACCIDENTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

8.4.2.1. State, describe and explain the requirements of investigations into rockfall and rockburst accidents

8.4.2.2. State, describe and explain the main causes of rock-related accidents in various mining operations in various mining districts.

8.4.3. ROCK ENGINEERING FUNCTIONS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

8.4.3.1. Describe, explain and discuss the functions of the rock engineering discipline on various types of mines

8.4.3.2. Describe, explain and discuss the functions of various staff categories encountered in a rock mechanics department.

8. LEGAL COMPLIANCE AND GENERAL MANAGEMENT

IMPORTANT

Very useful information is available from the websites below and should be visited to gather more information.

- Downloads of interesting documents from Department of Mineral and Energy website: http://www.dmr.gov.za/publications/guidelines/viewcategory/19-guidelines.html
- Safety statistics from the Department of Mineral and Energy website: http://www.dmr.gov.za/mine-health-a-safety.html

- Accident statistics and levies from the Department of Mineral and Energy website: http://www.dmr.gov.za/mine-health-a-safety/occupational-safety.html
- Mining Qualifications Authority: http://www.mqa.org.za
- South African Qualifications Authority: http://www.saqa.org.za/index.asp
- Mine, Health and Safety Council: http://www.mhsc.org.za

NOTE

A copy of the MHSA No 29 of 1996 can be perused here.
mhs_act_29_of_1996

DEFINITION

The following abbreviations are used in this outcome:

- PPE - Personal Protective Equipment
- MHSA - Mine, Health and Safety Act
- COP - Code Of Practice
- SIMRAC - Safety in Mines Research Advisory Committee
- MHSC - Mine, Health and Safety Council
- MRAC - Mine Research Advisory Committee
- MOHAC - Mining Occupational Health Advisory Committee
- MQA - Mine Qualifications Authority
- SAQA - South African Qualifications Authority
- SETA - Sector Education and Training Authority
- SDAct - Skills Development Act
- SABS - South African Bureau of Standards
- MMCC - Mine Managers Competency Certificate
- DME - Department of Minerals and Energy
- MR - Mining Regulations

8.1. MINE HEALTH AND SAFETY ACT AND REGULATIONS

8.1.1. THE MINE HEALTH AND SAFETY ACT, ACT 29 of 1996

IMPORTANT

Mine Health and Safety Act, Act 29 of 1996.
Section 14.1 (8) of the Regulations under the MHSA (Mine Health and Safety Act, Act 29 of 1996) state that:

"At every underground mine where a risk of rock bursts, rock falls or roof falls exists, and at every other mine where a significant risk of rock bursts, rock falls or roof falls exists, the employer must ensure that the input of a competent person is properly and timeously considered and integrated into mine design, planning and operations."
A competent person is defined in Section 22.14.1(8) of the Regulations under the MHSA as "a person who is at least in possession of either the Chamber of Mines Certificate in Rock Mechanics [Metalliferous Mines], or the Chamber of Mines Certificate in Rock Mechanics [Coal Mines], whichever is appropriate for the type of mine concerned."

LEARNING OUTCOME 8.1.1.1

OWNER'S RESPONSIBILITIES AND DUTIES

- Appoint Chief Executive Officer to ensure health and safety at mines.
- To comply with workers' right that there is controlled risks at their workplace.
- Carry out hazard identification and risk assessment so as to come up with a risk management strategy.
- To ensure proper supervision and instruction, through the delegation of responsibilities.
- Consult health and safety representatives or health and safety committees in matters of health and safety management on the mine.
- Provide health and safety training to employees.
- Provide adequate supply of health and safety equipment and PPE.
- Sanitation facilities and portable water to be made available at the workplace.
- First aid facilities should be made available for emergencies at the workplace.
- Report incidents/accidents timeously to the Inspectorate.
- Have insurance cover for employees against getting hurt at work or getting ill at work.
- Treat contractors as his own employees.
- Appoint a manager for mine that is being worked, for the day-to-day running of mine or a mine that has not received a closure certificate.
- Appoint Health and Safety Representatives
- Engage occupational hygienist for occupational hygiene measurements on the mine.
- Engage an occupational medical practitioner for the establishment of a medical surveillance system.
- Must appoint the employer representative on the health and safety committee.
- Appoint any person except manager to perform any functions (under sect 2 & 3) of the employer.
- Appoint any person with qualifications to perform any function (under sect 7(2)) of the employer.
- Entrusting responsibilities to other persons.
- Maintain health and safety at mines.
- Staff the mine for health and safety management.
- Provide a policy on health and safety.
- Draw codes of practice (mandatory and optional).

- Chapter 2 of the MHSA

LEARNING OUTCOME 8.1.1.2

MANAGER'S RESPONSIBILITIES AND DUTIES

- Appoint any competent person to perform any function of the manager as per section 7(2) of the MHSA, i.e. excluding responsibilities and duties under the MHSA Regulations.

- Chapter 2 of the MHSA

LEARNING OUTCOME 8.1.1.3

EMPLOYEE'S RIGHTS AND DUTIES

1. Duties
   - Protect their own health and safety.
   - Protect health and safety of other persons from any acts or omission by self.
   - Take proper care of PPE and other facilities for the protection of health and safety.
   - Report promptly any situation that presents risk to health and safety to anyone, if not capable of rectifying the situation.
   - Cooperate with others for compliance with the MHS Act.
   - Comply with given health and safety measures.
   - Get trained and follow what has been learnt.

2. Rights
   - Request occupational hygiene measurements that relate to their own wellbeing.
   - Request any information relating to their own medical surveillance.
   - To have own copy of their exit certificate.
   - Appeal to Medical Inspector on any findings by the Occupational Medical Practitioner, on unfitness to perform work.
   - Not to pay for PPE or health safety measures.
   - To leave dangerous working areas.

- Chapter 2 of the MHSA

LEARNING OUTCOME 8.1.1.4

SUSPENSION OF WORKING DUE TO INADEQUATE SAFETY

An Inspector of Mines gives an instruction of suspension to the employer or a person designated by the employer in terms of section 7(2) of the MHSA, in the absence of the employer or the designated person; the instruction can be given to the most senior person available.

- The instruction from the Inspector of Mines may be issued either orally or in writing.

- Any instruction to halt work operations at the mine or a part thereof must either be confirmed, varied or set aside by the Chief Inspector of Mines as soon as practicable.
- Any instruction issued to halt the operations at the mine or a part thereof is effective from the date fixed by the Inspector of Mines and remains in force until set aside by the Chief Inspector of Mines or until the Inspector's instructions have been complied with.

- Chapter 5 of the MHSA

LEARNING OUTCOME 8.1.1.5

INSPECTOR'S POWERS TO DEAL WITH DANGEROUS CONDITIONS:

1. Inspector's power in terms of Section 54(1) of the MHSA

   - Give an instruction to stop operations at the mine.
   - Suspend or halt the performance of any act/practice.
   - Instruction to be given to the employer, a person designated by him, or any senior person.
   - Instruction may be oral or written. Oral instruction must be confirmed in writing.
   - If the instruction was not issued to the employer, the inspector must give a copy to the employer as soon as possible
   - Instruction to be confirmed, varied or set aside by Principal Inspector of Mines.
   - Instruction is effective from the time fixed by the Inspector and remains in force until set aside/ complied with.

This is also known as getting a 'Section 54' when all work on the mine or section is stopped by the Inspector.

2. Inspector's powers to order compliance:

   - In terms of Section 55 of MHSA.
   - Circumstances under which such order can be given:
     - When inspector has reason to believe.
     - That the employer has failed to comply with provisions of MHSA.
   - Inspector may instruct employer in writing to take:
     - Necessary steps to comply with provision.
     - Specified steps to comply with instruction.
   - Time period for taking necessary steps:
     - Must be specified.
     - May be extended in writing.

- Chapter 5 of the MHSA

LEARNING OUTCOME 8.1.1.6

MANUFACTURER AND SUPPLIER'S DUTIES

- Supply articles that are safe and pose no risk when used properly.
- Articles supplied should comply with the MHSA.
- Erect or install articles that should pose no risk to health and safety during installation and use.
- Design, manufacture and install while observing ergonomic principles on usage of articles.

REFERENCE

- Chapter 2 of the MHSA
-

LEARNING OUTCOME 8.1.1.7

MANAGER'S DUTY TO CONSULT A ROCK ENGINEER

CONNECTION 71

Refer to "8 .2 CODES OF PRACTICE" on page 597

Covered in the codes of practice as enumerated below:

- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Rockburst Accidents in Tabular Metalliferous Mines
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Roof Fall Accidents in Underground Coal Mines
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockmass Failure Accidents in Massive Mining Operations.
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Slope Instability Related Accidents in Surface Mines

REFERENCE

- COP Guidelines for Tabular Metalliferous Mines
- COP Guidelines for Underground Coal Mines
- COP Guidelines for Surface Mines
- COP Guidelines for Massive Mining Operations

LEARNING OUTCOME 8.1.1.8

CONNECTION 72

The responsibilities of the Rock Engineer are described in detail in "LEARNING OUTCOME 8.4.3.1" on page 635

ROCK ENGINEER'S RESPONSIBILITIES AND DUTIES

- Provide rock engineering support service, ensuring that a competent person referred to in regulation 14.1(8) provides the relevant rock engineering inputs.

Also covered in the codes of practice, as enumerated below:

- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Rockburst Accidents in Tabular Metalliferous Mines

- Guideline for the Compilation of a Mandatory Code of Practice to Combat Roof Fall Accidents in Underground Coal Mines
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockmass Failure Accidents in Massive Mining Operations.
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Slope Instability Related Accidents in Surface Mines

REFERENCE

- COP Guidelines for Tabular Metalliferous Mines
- COP Guidelines for Underground Coal Mines
- COP Guidelines for Surface Mines
- COP Guidelines for Massive Mining Operations

LEARNING OUTCOME 8.1.1.9

ROCK-RELATED CODES OF PRACTICE REQUIREMENTS

Covered in the codes of practice as enumerated below:

CONNECTION 73

Refer to "8 .2 CODES OF PRACTICE" on page 597

- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Rockburst Accidents in Tabular Metalliferous Mines
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Roof Fall Accidents in Underground Coal Mines
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockmass Failure Accidents in Massive Mining Operations.
- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Slope Instability Related Accidents in Surface Mines

REFERENCE

- COP Guidelines for Tabular Metalliferous Mines
- COP Guidelines for Underground Coal Mines
- COP Guidelines for Surface Mines
- COP Guidelines for Massive Mining Operations

LEARNING OUTCOME 8.1.1.10

HAZARD IDENTIFICATION AND RISK ASSESSMENT

In terms of section 11 of the Act, an employer must assess and respond to risk. More specifically, the employer must:

- Identify hazards,
- assess risks,
- record significant hazards and risks and
- make records available for inspection by employees.

REFERENCE

- SIMRAC Practical Guide to Risk Assessment.

back

After consulting with the health and safety committee of the mine, the employer must determine all measures, including changing the organisation of work and the design of safe systems of work, necessary to:

- eliminate any recorded risk,
- control the risk at source,
- minimise the risk and
- insofar as the risk that remains:
  - provide for personal protective equipment and
  - institute a programme to monitor the risk to which employees may be exposed.

DEFINITION

Hazard: Any source of potential damage, harm or adverse health effects on something or someone under certain conditions at work.

Risk: Risk is the chance or probability that a person will be harmed or experience an adverse health effect if exposed to a hazard. It may also apply to situations with property or equipment loss.

INTERESTING INFO

What is the difference between 'hazard' and 'risk'?

- A hazard is something that can cause harm, e.g. electricity, chemicals, working up a ladder, noise, a bully at work, stress and doing a re-entry after a blast can be a hazard. A hazard does not necessarily put you at risk.
- A risk is the chance, high or low, that any hazard will actually cause somebody harm.

Some examples of hazards by category:

- **Biological:** Bacteria, viruses, insects, plants, birds, animals, and humans.
- **Chemical:** Toxic properties of the chemical.
- **Ergonomic:** Repetitive movements, improper set up of workstation.
- **Physical:** Radiation, magnetic fields, high pressure or vacuum, noise.
- **Psychosocial:** Stress, violence.
- **Safety:** Slipping/tripping, machine guarding, equipment failure.

DEFINITION

Risk assessment: Risk assessment is the process where you: identify hazards, analyse or evaluate the risk associated with that hazard, and determine appropriate ways to eliminate or control the hazard.

Types of risk assessment:

1. Baseline risk assessment:
   - Review all potential hazards in a particular area, caused by the process or activities
   - Identify hazards and risk in a business process (Table 1).

back

Area	Description	Examples
Geographical	List of all areas of business Identify risk in each area	Plant (mill section). u/g (stope face, gulley area, cross cuts). Shaft (station, equipment, brow, shaft bottom).
Activities	List of activities. Identify significant risk for each activity	Loading, drilling, and blasting.
Occupations	List of occupations responsible for each activity Identify significant risk for that occupation	Loader driver, hoist driver, miner, etc
Tasks	List task done by each occupation Identify significant riskfor each task	Charging draw point, re-entry, etc

Table 1: Baseline risk assessment (example of selected areas for hazard and risk assessment)

.

2. Issue-based risk assessment:
   - New machinery or equipment,
   - Changes in work processes,
   - Incident/Accident, and
   - New information.

3. Continuous risk assessment:
   - Day-to-day management by supervisor ,
   - Audits ,
   - Checklist before work, and
   - Waiting place talk suggestions.

CONNECTION 74

Risk assessment methods are described in "8.3 BASIC RISK ASSESSMENT" on page 607

REFERENCE

- Chapter 2 of the MHSA
- SIMRAC Practical Guide to Risk Assessment

LEARNING OUTCOME 8.1.1.11

NEGOTIATION AND CONSULTATION FORUM'S FUNCTION, DUTIES AND POWERS

Where there is a requirement to have a health and safety representative in terms of section 25 of the MHSA, an employer must meet with a representative trade union to negotiate on a collective agreement to:

- Designate working places on the mine.
- Health and safety representatives:
  - Discuss the number of full-time health and safety representatives.
  - Elect or appoint health and safety representative.
  - Come with terms of office of health and safety representatives.
  - Filling of vacancies.
  - How health and safety representatives can perform their functions.
- Discuss procedures on the right to refuse to work in dangerous places by employees.

- Discuss how meetings on health and safety should be held.
- Discuss training for the health and safety representatives.
- Discuss procedures on handling disputes resulting from interpretation of the MHSA.
- Discuss any relevant matter to promote health and safety.
- Employer duties in negotiation forums:
  - Employer to consult other unions with members at the mine before concluding a collective agreement with the representative trade union.
  - In the absence of representative trade union, the employer must consult employees instead, i.e. for the same reason he/she was going to consult a representative trade union.
  - A health and safety representative to be provided with an advisor when necessary by an employer.
- Duties and Powers of Health and Safety representative:
  - Health and safety representative may represent employees on aspects of health and safety.
  - Direct and assist employees to leave a dangerous working area.
  - Identify hazards and risks to health and safety.
  - Forward concerns on health and safety to employer or health and safety committee.
  - Inspect any relevant documents that relate to the MHSA.
  - Request information or reports from an Inspector of Mines.
  - Request an Inspector of Mines to carry an investigation.
  - Request the Chief Inspector of Mines for an inquiry.
  - Participate in health and safety consultation with employer or an Inspector of Mines.
  - Inspect workings on health and safety.
  - Participate in health and safety audits.
  - Respond to complaints by employees on health and safety by investigating such complaints.
  - Examine causes of accidents.
  - Visit sites of accidents.
  - Attend post-accident inspections.
  - Participate in inquiries.
  - Perform agreed functions or prescribed functions.

- Chapter 3 of the MHSA

LEARNING OUTCOME 8.1.1.12

TRIPARTITE INSTITUTION'S FUNCTIONS AND DUTIES

Tripartite institutions are the MHSC (including its permanent committees comprising of MRAC, SIMRAC & MOHAC) and MQA.

1. Functions of the Council (MHSC)
   - Advise the Minister on health and safety at the mines

back

- Co-ordinate the activities of the committees
- Consider the reports of the committees
- Liaise with the MQA on matters relating to health and safety training
- Liaise with any other statutory bodies concerned with matters relating to health and safety
- Promote a culture of health and safety in the mining industry
- Arrange and co-ordinate a tripartite summit at least once every two years to review the state of health and safety in the mining industry
- Consider an annual overall programme for relevant health and safety for consideration
- Perform every duty imposed upon the council in terms of the MHSA: http://www.mhsc.org.za

2. Functions of the MQA:
   - Seek registration in terms of SAQA, as a body responsible for generating training and qualifications standards
   - Seek accreditation in terms of SAQA as a body responsible for monitoring and auditing achievement as contemplated in section 5(1) of SAQA.
   - Propose education and training standards and qualifications to bodies registered with MQA and responsible for developing education and training standards
   - Generate education and training standards and qualifications
   - Perform functions of SETA in terms of the Skills Development Act
   - Monitor and audit achievements in terms of set standards and qualifications
   - Accredit education and training providers in the mining industry
   - Analyse, prioritise and recommend training needs in the industry
   - Promote a culture of learning in the mining industry
   - Do education and training quality assurance, without being the provider
   - Facilitate the industry's strategic human resource development planning
   - Liaise with SAQA and other bodies concerned with training
   - Record keeping of learning in the industry
   - Perform any other functions as per SAQA's requirement
   - Perform any other function that must be performed by MQA in terms of the MHSA & SDAct

- Mining Qualifications Authority: http://www.mqa.org.za
- South African Qualifications Authority: http://www.saqa.org.za/index.asp

3. Mine Health and Safety Council – Permanent Committees

   1. Duties of MRAC

      The MRAC must advise the Council on:
      - proposed changes to legislation to improve health and safety at mines.
      - Proposal for legislative changes made by other MHSC committees.

back

back

- Guidelines for COP and Guidance notes.
- Standards approved by the SABS.

2. Duties of MOHAC

The MOHAC must advise the Council on:

- Policy relating to health.
- Standards, systems, procedures, for assessing, avoiding, controlling, minimising and eliminating health risk.
- Regulations on any health aspect.
- Health research. Collecting, processing and distributing health data in the industry.

3. Duties of SIMRAC

SIMRAC must advise the Council on:

- Criteria for determining funding on health research.
- The need for research into health and safety at the mines.
- Research projects, including prioritising, costs, assessments and implementation. Communication and publication of research results.
- The cost of the overall programme.

4. Minister's duty with respect to Tripartite Institutions

- The minister appoints members of the tripartite institutions, as per regulations.
- Provides sufficient funds annually, for use by the Council.
- The Minister may, after consulting with the Council, by notice in the Government Gazette, add to this Act a further Schedule containing the constitution of the Council and its committees.
- The Minister may, after consulting with the Council and the MQA, by notice in the Government Gazette, add to this Act a further Schedule containing the constitution of the MQA and its committees.
- Appoints the executive officer of the MQA.

- Chapter 4 of the MHSA

LEARNING OUTCOME 8.1.1.13

INSPECTOR' FUNCTIONS AND POWERS

1. Functions:
   - Lead agent in:
     - Ensuring the health and safety of workers and the public affected by mining;
     - Develop policies and legislation;
     - Promote awareness of health and safety among all stakeholders;
     - Encourage co-operation and continuous improvement in health and safety;
     - Promote monitoring, enforcing legislation and initiating prosecution in terms of the Mine Health and Safety Act of 1996 (Act No. 29 of 1996).

back

- Achieve:
  - Consistency in the enforcement of the Act by regional offices of the Inspectorate;
  - A structured framework for decisions concerning the levels of enforcement;
  - Appropriate and timely enforcement interventions (Figure 1).

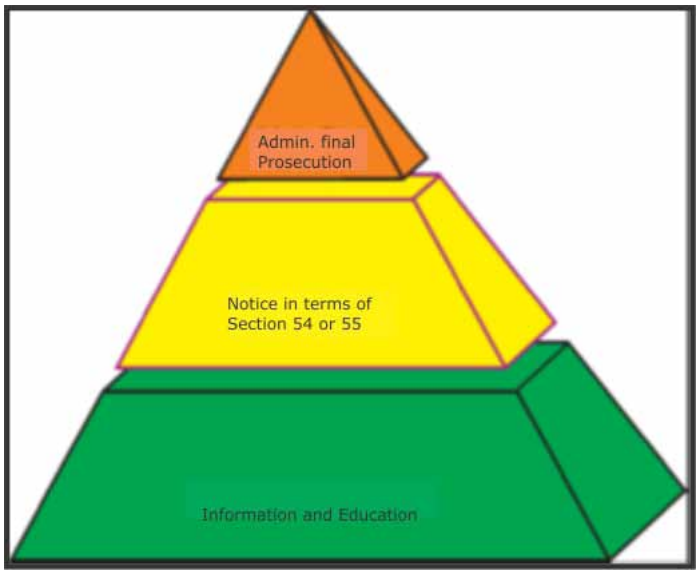


Figure 1: Enforcement pyramid

The seriousness of an offence must be judged against the following criteria:

- Magnitude of a contravention;
- Failure to comply with an instruction;
- Ineffective preventative health and safety management systems to prevent harm;
- Inadequate measures required to achieve compliance; and
- Previous history of non-compliance.

Information and education. This tier of enforcement takes the form of:

- Letters to mine employers and mine equipment suppliers and/or manufacturers, advising of statutory obligation and recourse issues;
- Education programmes or counselling sessions in the form of formal/informal meetings with stakeholders;
- Discussions within the tripartite structures;
- Reviewing, modifying or upholding a previous instruction.
- In cases where the following circumstances exist, an Inspector must recommend a fine; if he/she does not, s/he must give reasons for not making a recommendation in writing to the Principal Inspector.
- Contraventions or failures that pose a high risk of harm to persons directly affected by the activities at a mine;
- A category of non-compliance contemplated in the guidelines issued in terms of Section 55G;
- Knowingly exposing persons to serious hazards;
- Repetitive failure to comply with instructions issued in the interest of health and safety.

back

The following factors should be used to help decide whether to recommend prosecution:

- How much the outcome could have been anticipated;
- The extent of the offender's intentional negligence;
- The offender's attitude; and
- The recommendation's deterrent effect.

Powers

1. Inspector's powers to monitor & enforce compliance:
   - May enter mine at any time without warrant or notice.
   - May enter any other place with warrant.
   - Bring into and use at a mine / any other place any equipment / materials necessary to perform any function in terms of the MHSAct.
   - Question any person on ANY matter to which this Act relates.
   - Require any person who has control over ANY document to produce such to the Inspector immediately / as per request.
   - Require any person who has control over ANY document to explain any entry or non-entry in such a document.

2. Inspector's powers to deal with dangerous conditions:
   - Give an instruction to stop operations at the mine.
   - Suspend or halt the performance of any act/practice.
   - Instruction to be given to the employer, a person designated by him, or any senior person.
   - Instruction may be oral or written. Oral instruction must be confirmed in writing.
   - If the instruction was not issued to the employer, the inspector must give a copy to the employer as soon as possible
   - Instruction to be confirmed, varied or set aside by Principal Inspector of Mines.
   - Instruction is effective from the time fixed by the Inspector and remains in force until set aside/ complied with.

3. Inspector's powers to order compliance or recommend a fine:
   - Give any written instruction to employer that the Inspector considers necessary to comply with provisions.
   - Employer to take steps specified in an instruction.
   - Define the period within which the instruction is to be carried.
   - May recommend to the Principal Inspector of Mines that an administration fine be imposed on the employer.
   - Provide reasons as to why he is not recommending an admin fine.
   - Provide a copy of the admin fine to the employer, Health and Safety Committee, any Health and Safety Representative responsible for the area, representative trade union or any registered trade union with members.

- Chapter 5 of the MHSA
- Mine Health and Safety Inspectorate Guideline for Enforcement of the Mine Health and Safety Act

back

back

LEARNING OUTCOME 8.1.1.14

REQUIREMENTS FOR ENQUIRIES INTO ACCIDENTS

- Chief Inspector of Mines may appoint one or more persons to carry out an inquiry.
- Persons presiding at an inquiry may decide who may attend.
- Persons who can participate at an inquiry should: have material interest in the matter, represent a trade union with members at that particular mine, and a health and safety representative designated to the accident area.

Powers of presiding person:

- Summon persons.
- Question persons summoned.
- Demand any records or demonstration of an act relevant to the inquiry.

Contents of an inquiry report:

- Findings
- Recommendations
- Remedial steps

Inquiry and inquest:

- An inquiry and inquest can be conducted jointly.
- An inquest is presided over by a judiciary officer.

- Chapter 5 of the MHSA

LEARNING OUTCOME 8.1.1.15

MINISTER'S ABILITIES AND POWERS

1. Delegate power to the Chief Inspector to administer the Mine Health and Safety Act.
2. After consulting with the Mine Health Safety Council, the Minister may:
   - Replace or change certain schedules of the MHSAct.
   - Add a schedule into the MHSA, on matters that health and safety committee may consult on.
   - May add another schedule to the representation of the Council, MQA and committees.
   - May add items to schedule 4.
   - May also consult the Minister of Health to add a further schedule to the required application of occupational diseases in mines and works Act

- Chapter 6 of the MHSA

back

LEARNING OUTCOME 8.1.1.16

SCOPE OF SAFETY-RELATED REGULATIONS

The Minister, after consulting the Mine Health Safety Act, may make regulations on:

- •Health and safety at mines.
- Health and Safety standards at mines.
- Code of Practice at mines.
- Personal Protective Equipment for employees.
- Hazard identification, risk assessment and risk management at mines.
- Health and safety management systems at mines.
- General administration of health and safety on mines

REFERENCE

- Chapter2 and 6 of the MHSA

8.1.2 REGULATIONS

NOTE

The Regulations can be scrutinised on the DMR website: http://www.dmr.gov.za/regulations.html

LEARNING OUTCOME 8.1.2.1

APPOINTMENTS AND ADMINISTRATION

An Inspector of Mines may issue an administrative fine to an employer.

Before the Principal Inspector of Mines considers an administrative fine (imposed by an Inspector of Mines), he/she will call for representation from the employer on a mutually agreed date with an employer or within 30 days of his/her receipt of the fine.

An administrative fine can be paid by an employer to the Principal Inspector of Mines within 60 days from the date in which it was imposed or on any other date as per appeal process.

REFERENCE

- Chapter 1 of the MHSA

LEARNING OUTCOME 8.1.2.2

DUTIES AND RESPONSIBILITIES

- When the appointed CEO representing the employer is out of SA, he/she must delegate his/her duties to another employer representative who would be in SA.
- Where a mine has more than 50 persons employed underground, the mine manager should be a holder of a relevant MMCC; if not,

back

he/she should appoint an MMCC holder who would shoulder the responsibilities of health and safety as per the act/regulations.

- No mine should operate for more than 60 days, within a six-month period, without an appointed MMCC holder in management.
- A mine manager can only be responsible for one mine unless permission was given by the Principal Inspector of Mines.
- The employer representative can only manage a mine with the permission of the Principal Inspector of Mines.
- Copies of appointment of managers should be sent to the Principal Inspector of Mines within three days of signing.

Responsibilities of mine manager:

- Safeguard incompetent persons' health and safety
- Have a system of supervision of employees.
- Report and investigate accidents.
- Blasting operations for mining.
- Take measures to prevent flooding of mine working.
- Control of disused workings and surfaces, e.g. slimes.
- Appointment of assistants:
  - sub-managers,
  - Surveyors,
  - Engineers,
  - Mine overseer,
  - Shift boss,
  - Environmental officer,
  - Safety officer,
  - Rock engineer

- Chapter 2 of the MHSA Regulations

LEARNING OUTCOME 8.1.2.3

HEALTH AND SAFETY REPRESENTATIVES AND COMMITTEES

- Requirements of an employer to enter into negotiations with the representative trade union, at a mine where there must be a health and safety representative.
- Disputes between the parties should be referred to the commission for conciliation, mediation and arbitration of the Labour Relations Act.
- The mine manager should establish an election committee to elect health and safety representatives.
- Duties of the election committee and mine manager, laid out in the establishment of election committee.
- Candidates for health and safety representatives, to be nominated by the committee.
- Drawn out procedures for the election of health and safety representatives and term in office.
- Manager is responsible for the appointment of health and safety representatives who have been elected.

back

- Chapter 6 of the MHSA Regulations

LEARNING OUTCOME 8.1.2.4

INSPECTORATE OF MINE HEALTH AND SAFETY

- Appointed Inspector of Mines should meet the qualifications prescribed in the Public Service Staff Code.
- Appointed Inspectors of Mines should be issued with certificates DMR 34 and DMR 35 by the Chief Inspector of Mines.
- Any other person authorised by the Chief Inspector of Mines to carry out inspectorate duties must be issued with certificate DMR 35
- These certificates should be carried at all times when visiting a mine.
- In the absence of certificates DMR 34 and DMR 35, the Principal Inspector of Mines may issue the appointed Inspector of Mines or person a letter of authorisation.

- Chapter 7 of the MHSA Regulations

LEARNING OUTCOME 8.1.2.5

MISCELLANEOUS AND GENERAL PROVISIONS

The integrity of water storage and pumping systems underground:

- Dam wall stability
- Water plugs' reliability
- Barricades to keep water back
- Design and location plans of water dams to be kept safely during the life of such dams.
- Procedures should be in place to prevent employees getting injured

The rest of this sub-topic is essentially definitions of the following:

- Certified
- Double protected
- Explosion protected apparatus
- Hazardous location
- Light metal
- Box front
- Draw point
- Rock
- Rock pass
- Tipping point

- Chapter 10 of the MHSA Regulations

LEARNING OUTCOME 8.1.2.6

OUTLETS, LADDERWAYS AND TRAVELLINGWAYS

- Whenever it is practicable, there must be two outlets at every underground working place.
- Where two outlets are not possible, a risk assessment should determine what other measures can be taken to prevent employees being trapped in any underground excavation.
- Only in an emergency or with authorisation may any person enter or leave underground workings through other means that are not the designated ingress or egress.

REFERENCE

- Chapter 13 of the MHSA Regulations

LEARNING OUTCOME 8.1.2.7

PROTECTION OF THE SURFACE AND IN THE WORKINGS

Fall of ground regulations:

- No person permitted to enter workings that pose a risk of rockburst or ground fall, except a competent person making safe.
- Risk areas would be areas between face and the nearest permanent line of support, access ways, travelling ways and work areas
- The hazardous areas should be examined as often as the risk assessment prescribes
- The records of such examinations should be kept for at least three months
- Measures should be taken to prevent inadvertent entry into unsafe areas
- A quality assurance system should be used to assess if support units employed meet ground loading characteristics
- Any support unit must be installed, maintained and removed by a competent person.
- Professional input from a competent person must be integrated into mine design, planning and mining operations.
- Measures should always be taken to prevent any inrushes into workings, such as ground, mud, water, gases, slimes or any other fluid.
- Measures should be taken to eliminate any significant risk associated with subsidence or opening caused by mining operations.
- Precautions should be taken to prevent injuries of persons due to collapse of surface structures and buildings that could result from the removal of underground support.
- Definition of a competent person

REFERENCE

- Chapter 14 of the MHSA Regulations

LEARNING OUTCOME 8.1.2.8

QUALIFICATIONS AND CERTIFICATES OF COMPETENCY

- Laid out appeal process by candidates to the examining authority.
- Invalidation of unauthorised altered certificates.
- Chief Inspector of Mines may appoint and discharge the commission of examiners.
- Rules and dates of examinations.
- How to re-apply for a lost certificate.
- Chief Inspector of Mines may suspend or cancel certificates if circumstances so warrant.
- There is laid out criteria to qualify for the Certificates of Competency, on the following certificates:
  - Mine Manager's Certificate of Competence
  - Mine Overseer's Certificate
  - Mine Surveyor's Certificate
  - Mechanical Engineer's-Mines & Works Certificate
  - Electrical Engineer's- Mines & Works Certificate
  - Mine Assayer's Certificate
  - Winding Engine Driver's Certificate
  - Locomotive Driver's Certificate
  - Stationary Engine Driver's Certificate
  - Boiler Attendant's Certificate
  - Ground Breaking Certificate
  - Lampman's Certificate
  - Onsetter's Certificate

REFERENCE

- Chapter 28 and 29 of the MHSA

LEARNING OUTCOME 8.1.2.9

SURVEYING AND MINE PLANS

Applicable definitions in survey:

- Bedded mineral deposit
- Chart datum
- Competent person
- Fixed position
- Fluid material
- Hazardous service
- Plan(s)
- Reserve
- Restricted area
- Risk assessment

- Safety pillar
- Sea
- Survey point
- Survey station
- Workings

Responsibility for surveying, mapping and mine plans

- Units of measure-units to conform to metric system, except angular measurements, which must conform to the sexagesimal system.
- Survey system – all mine surveys to conform to the National Control Survey System.
- Datum plane – elevations above or below ground on a mine refer to mean sea level and elevations with mining at sea refer to chart datum.
- Chart datum – elevations determined at sea must refer to the chart datum unless specified otherwise by the Chief Inspector of Mines
- Colours and sign conventions – signs and colours should conform to those provided by the Director of Mine Surveying
- Back up of electronic information – survey records should be adequately backed up electronically
- Responsibilities regarding safety precautions and competent person to be aware of:
  - Workings being advanced
  - Surface structures or objects that could be affected by mining
  - Workings being abandoned or closed down
  - Workings within 50 metres from other workings or a lesser distance determined by risk assessment.
  - Safety pillars that are being mined or have been mined
- Restrictions of mining under or within a horizontal distance of 100 metres or a distance determined by risk assessment from any structure or surface that has to be protected.
- Survey plans to be continuously updated by a competent person.
- Requirements of boundary pillars for every mine boundary.
  - These boundary pillars can be removed or cut only with permission of adjacent mine owners and the Principal Inspector of Mines.
- In the absence of adjacent owners, only the permission of the Principal Inspector of Mines would suffice.
- The Director Mine Surveying may direct that a check survey be done if he/she suspects an error on the plans.

Standards of accuracy, surface and underground:

- Survey stations
- Representation of plans
- Standards of accuracy at sea

Details required on plans:

- Date
- Surface contours
- Planes of sections/plans
- Name of mine & plan
- Name of competent person and signature against date of updating
- Allotted identification number
- Survey system and co-ordinates
- North point
- Scale of plan
- Legend on colours and signs
- Co-ordinates allowing scale of plan to be verified
- Mining at sea, geographic co-ordinates should be used
- Scale and material of plans on land and at sea are specified
- Plans to be kept up to date

Mine plans

- Plans should be indexed to show the areas covered
- Surface plan should show boundary mining areas and all surface detail
- Surface contour plan should show relevant farm and mine boundaries
- Mine ventilation and rescue plan giving ventilation detail and location of necessities in emergency
- Rehabilitation plan
- Mine residue deposit plans
- Geological plans
- Plans of the workings
- Level plans and vertical sections
- General plan – mining on land
- General plan – mining at sea
- Departmental copies of plans
- Unsatisfactory plans – where the Director Mine Survey has asked the mine to be surveyed again
- Information on plans to be kept confidential

Mine closure

- Competent person to bring plans up to date before closure, suspension or abandonment of a mine
- Employer to hand in plans to the Director Mine Survey at closure
- Certificate of compliance to be issued to the employer within 60 days, if mine closure requirements have been met.

REFERENCE

- Chapter 17 of the MHSA Regulations

back

LEARNING OUTCOME 8.1.2.10

TRIPARTITE INSTITUTIONS

- Minister is empowered to make regulation on appointment of members of the MHSC and MQA
- Minister must appoint nominated members representing the employers and employees
- Minister appoints members representing the state after consulting the Chief Inspector of Mines
- Members who vacate office should be replaced following the same process that was used in the appointment of the member that has vacated
- Appointed member names are published in the gazette

- Chapter 18-MHSA Regulations

LEARNING OUTCOME 8.1.2.11

DEFINITIONS (APPLICABLE)
The following definitions are given and can be studied in the MHSA

- Audiogram
- Braking system
- Cardio respiratory examination
- Dynamic type test
- Full size chest X-ray
- Locomotive
- Lung function test
- Principal Inspector of Mines
- Rolling stock
- Static test
- Train
- Tripartite institutions
- Competent Person
- Controlled Mine
- Emergency
- Explosive
- Flammable gas
- Ganger or Miner
- Forms
- Hazardous Area
- Misfired Hole and Shot Hole
- Non Fiery Mine
- Shaft
- Steeply inclined
- Stope
- Ventilation district

back

- Winding plant
- Winze / Raise
- Explosion Protected Equipment (Flameproof)
- Intrinsically safe
- Workings

- Chapter 20 of the MHSA Regulations

THE APPLICATION OF THE LAW IS A COMPLEX ISSUE AND SPECIALISTS SHOULD BE APPROACHED FOR IN-DEPTH UNDERSTANDING AND APPLICATION OF THE LAW AND ITS REGULATIONS.

For an overview of some implications, the user is referred to the following paper:
'Brief exposition of the SA law pertaining to criminal and civil liability of the employer and rock engineer and related matters' by W Le Roux, P Colyn (Brink, Cohen and Le Roux Inc)
as published in the SAIMM proceedings for the School 2011: Rock Engineering for Managers in hard rock tabular mines (1 June 2011).

8 .2 CODES OF PRACTICE

The following are common to all Codes of Practice and should be read with any of the sections in Outcome 8.2:

- The guideline is a generic document.
- Guideline is issued in terms section 9(2) and 9(3) of the MHSAct.
- The DMR does not approve COPs.
- Failure to comply with the COP is not a breach of the MHSAct.
- But failure to comply with COP:

i. Constitutes a breach of employer's obligations under the MHSAct.
   - The employer could be liable to an administrative fine.
   - The employer could be liable to section 55 of the MHSAct instruction.
   - The employer could be liable to section 54 of the MHSAct instruction.

ii. For an employee, failure to comply with COP could be a breach of duties as laid out in section 22 of the MHSAct.

iii. Employer can apply employer's disciplinary standards to an employee who is in breach of the COPs.

Objective of the guideline:

To enable the employer to compile with a COP to reduce rockfall and rockburst accidents.

Only some of the most critical definitions and included in this list

- COP – Code of Practice
- DMR – Department of Minerals and Energy;
- Geology – scientific study of Earth, the rocks that compose it and changes that it has undergone or is undergoing;
- Ground control – ability to predict and influence the behaviour of

rock in a mining environment

- Ground Control Districts – portion of mine where similar geological conditions exist, which give rise to a unique set of identifiable rock-related hazards for which a common set of strategies can be employed to minimise the risk resulting from mining;
- Ground Control District Plan – a plan on good quality transparent drafting material of thickness not less than 0,08mm indicating to scale of 1 in 2500 all applicable ground control districts of the mine;
- Hazard – source of, or exposure to, danger;
- MHSA – Mine Health and Safety Act, 1996 (Act No. 29 of 1996);
- MRAC – Mining Regulation Advisory Committee;
- Primitive (virgin) stress – the state of stress in a geological formation before the stress field is altered by mining operations;
- Risk – likelihood that occupational injury or harm to persons will occur;
- Rockburst – a seismic event that causes damage to underground workings;
- Rock Engineering – the engineering application of rock mechanics;
- Rockfall (fall of ground) – a fall of rock fragment or a portion of fractured rockmass without the simultaneous occurrence of a seismic event;
- Rockmass – the sum total of the rock as it exists in place, taking into account the intact rock material, groundwater, as well as joints, faults and other natural planes of weakness that can divide the rock into interlocking blocks of varying sizes and shapes;
- Rock mechanics – the scientific study of the mechanical behaviour of rock and rockmasses under the influence of stress;
- Seismically Active Mine – a mine that sustains losses to persons and/or property, underground or on surface, caused by the dynamic response to a seismic event induced when creating or enlarging an excavation;
- Seismic event – the transient earth motion caused by a sudden release of the strain energy stored in the rock;
- Seismicity – the geographic and historical distribution of earthquakes;
- Seismology – the scientific study of seismic events by the analysis of vibrations transmitted through rock and soil materials;
- Significant Rock-related Risk – the likelihood that the harm from a particular hazard will result in the death or permanent disability of a person;
- Special areas – areas requiring additional attention and precautions where an increased risk of rockfalls or rockbursts may develop;
- Strain burst – rockburst at the surface of an excavation;
- Support – a structure or a structural feature built into or around an underground excavation to maintain its stability.

General information to be provided:

1. Locality

Locality map to indicate the location of the mine in relation to other relevant features; for example, other mines sharing a common boundary, dams, rivers.

back

2. Geological setting

Faults, dykes and stratigraphy, around individual coal seams that can present hazardous conditions to mining.

3. Mining environment

Mining environment describes major subdivisions of coal seams.
Additional information to be provided: general description of coal seams being mined.

4. Regional hydrology

Rainfall pattern, rain quantities, surface water and groundwater aquifers.

5. Ground control districts must be described in the COP:

The nature of the virgin stress field in which mining is to take place, significant pore water pressures and geology to be considered (inter alia);
Location and extent of ground control districts must clearly be described in the COP

6. Mine rockfall and/or rockburst incident analysis must also be described in the COP.

A tabulation of the mine's five-year history of rock-related casualties, i.e. fatalities, reportable, disabling incidents and non-casualty incidents, categorised according to rockfalls per 1000 employees for surface and underground operations.
Information for analysis must be graphically represented.
COP must reflect the incident trends associated with the identified hazards.

7. Risk management:

Section 11 of the MHSAct requires that the employer should identify hazards, assess the health and safety risks and record the significant hazards and risk assessed.
The COP must address how the significant risks identified in the risk assessment process are dealt with.

LEARNING OUTCOME 8.2.1

TABULAR METALLIFEROUS MINES

NOTE

The COP Guidelines can be accessed here

Aspects to be covered:

- Significant risks
- Risk management strategies.
- Risk management strategies should cover mining layouts, mining sequence, support and monitoring procedures.
- Hazards should be identified as per ground control districts.
- Subsections should describe how to deal with each hazard.
- Strategies should cover even obvious significant risk, e.g. ground conditions after blasting.
- Records on any experimentation that deviates from the COP should be recorded.
- Strategies should be defended by a competent person.

back

back

Implementation plan:

- Must be prepared
- Should cover issues such as organisational structures and responsibilities.
- Measures should be in place on the enforcement of the COP.
- A copy of a COP should be made available to persons with material interest.

Support strategy:

- Support strategy should be in line with the work conditions and the environment.
- Good quality grouting should be ensured at all times.
- Materials used in support must be subjected to quality control testing.
- Specific procedures of re-supporting of tunnels should be in place.
- Design of large excavations must take into account the following:
  - stress fields,
  - rockmass classification,
  - numerical modelling,
  - seismic data analysis,
  - support design criteria,
  - size of excavation.

Design guidelines:

- Strategy must define the functions of each support system.
- Volume of rock to be supported should be determined.
- Structure of the rockmass should be determined.
- Establish a support system criterion.

Control of rockfalls:

- Extent of areal coverage should be stipulated.
- Determine the resistance required to control rockfall.

Reduction of rockburst damage:

- Ability of support system to absorb kinetic energy should be determined.
- Yielding support system required in high kinetic energy environments.

Designation of special areas:

- Existing or proposed workplaces that appear to have increased risk of rockfalls or rockburst should be designated as special areas.
- A competent person should monitor special areas.
- Special areas should be classified as restricted or precautionary.
- A procedure must be established for declaring a special area.

Rock Engineering Support Service (RESS):

- Assist the employer in proactively identifying significant rock-related hazards.
- Assist the employer in safe and economic design of mine workings.
- RESS must participate in planning.

back

- RESS must review, identify and make recommendations on management systems.
- RESS, assisted by a competent person, should analyse and interpret seismic incidents.
- RESS should also play an important role in monitoring, investigating and assuring quality of support systems.

REFERENCE

- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Rockburst Accidents in Tabular Metalliferous Mines

LEARNING OUTCOME 8.2.2

NOTE

UNDERGROUND COAL MINES

The COP Guidelines can be accessed here

Aspects to be addressed:

- Significant risks must be identified and assessed according to the risk assessment process.
- Mine stability strategies.
- Strategies for protection of mine accesses/exits.
- Strategies for ensuring stability of current underground workings.
- Strategies for mining the coal.
- Strategies for designating hazardous locations.
- Strategies for ground development e.g. blasting, drilling patterns.
- Impact of mining activities on neighbouring mines.
- Monitoring of ground stability in relation to risk management.
- Integrated mine process design and planning, i.e. integrating the management of falls of ground to risks into mine planning processes.
- Rock Engineering Support Service;
  - ensuring that a competent person referred to in regulation 14.1(8) provides the relevant rock engineering inputs.
  - Rock engineering competency records should be kept at the mine.

Implementation plan
Employer must prepare an implementation plan for the COP.

Compliance with the COP
Measures for monitoring and ensuring compliance with the COP must be in place.

Access to the COP and related documents
Complete COP and related documents must be kept for examination by any persons with material interest.

Bord and Pillar Mining Design Strategy / Roadway Support Strategy / High / Total Extraction Panel Design Strategy / Subsidence Control

Strategies for each area below must be recorded

- Pillar design and mining layouts
- Strategies for roadway and roof support development

- High/total extraction and design strategy
- Subsidence control measures
- Communication of changes in ground conditions

NOTE

- *Design Methodology as in SIMCOL 814 should be consulted when performing any mine design.*
- *Bord and pillar mining pillar design should follow the safety factors using the Salamon and Oravecz (1976) formula or any other formula with proven applicability.*

Safety factors for the following should be specified:

- Primary development
- Secondary development

Roadway roof support design methodology by classification:

- No systematic support required
- Suspension type support
- Beam creation type support
- Roof that is affected by high or low horizontal stresses

Generic roof support requirements:

- Geological discontinuities
- Road width
- Burnt coal
- Ribside support
- Water and gas pressure
- A specified procedure to determine changes in roof condition should be determined.

High/total extraction panel and pillar design strategy: to allow the overburden to fail or not, the criteria should be based on the significant risk with the following considerations:

- Underground stress conditions
- Surface control
- Groundwater control, i.e. water influx into the mine
- Environmental considerations

Subsidence control:

There should be mitigation of the impacts on surface subsidence

Strata management in hazardous locations

- The description of hazardous locations.
- Mine Planning and Mine Design should incorporate strata management strategies.
- Routine Monitoring and Special Investigations should be in place.
- Research and Technology Transfer is encouraged.
- Quality Control should be verified by audits.
- Learning/Assessment in personnel training programmes.

Rock Engineering Support Service (RESS):

- Assist the employer in proactively identifying significant rock-related hazards.
- Assist the employer in safe and economic design of mine workings.
- RESS must participate in planning.
- RESS must review, identify and make recommendations on management systems.
- RESS, assisted by a competent person, should analyse and interpret seismic incidents.
- RESS should also play an important role in monitoring, investigating and assuring quality of support systems.

REFERENCE

- Guideline for the Compilation of a Mandatory Code of Practice to Combat Roof Fall Accidents in Underground Coal Mines

LEARNING OUTCOME 8.2.3

MASSIVE MINING OPERATIONS

NOTE

The COP guidelines can be accessed here

Aspects to be addressed:

- Overall mine stability:
  - The strategies to ensure overall stability of the mine.
  - The strategies to reduce the risks of rockmass failures.
- Risk of rockfalls caused by the interaction of underground operations overlain by an open pit must be covered in the COP.
- The COP should prevent incidents of mud-rush.
- The COP should cover sequencing and mining methods, where more ore bodies are in proximity of each other.
- Measures should be in place in the COP to prevent support failure incidents. COP should cover methodologies of protecting surface structures and topography.

Protection of mine accesses/exits:
The COP must cover strategies for the protection of the access/exit ways of the mine, due to ground movement.

Stability of tunnel and service excavations:
The COP must have strategies to protect tunnels and service excavations from failure resulting from damaged excavations, resulting from:

- explosives,
- poor rock conditions,
- high or changing stress conditions and
- proximity to other workings.

Ore body extraction:
Relevant risks (obvious and not obvious) associated with the extraction processes should be covered in the COP.

Special areas:
Method of identifying and designating risk areas that could result from routine mining operations should be drawn in the code of practice.

Rock-breaking:
The COP should cover rock-breaking strategies that protect the integrity and stability of the excavation walls by looking at:

- type of explosives used,
- blasting accessories used,
- drilling patterns ,
- accuracy of drill holes,
- initiation methods and
- prevailing ground conditions.

The impact of mining activities on neighbouring mines:

- The COP must allow setting out of strategies in massive mining operations that do not present rockmass instability to adjacent mines.
- Means of exchange of information with adjacent mines should also be included.

Integrated mine process design and planning:
The COP covers the management of rock-related risks in the overall mine planning process, including human roles.

Support design methodology:
Support design methodologies to be well motivated in the COP.

Rock Engineering Support Service:
The need for appropriate rock engineering input to overall mine stability must be captured in the COP.

Other aspects to be addressed in the COP
Risk assessments and motivation records must be kept for any research methods that deviate from the COP.

Implementation plan
An implementation plan for issues like organisational structures, responsibilities and schedules must be covered in the COP.

Compliance with COP
Monitoring and compliance with the COP must be maintained.

Access to the COP and related documents
COP-related documents must be made available to persons with material interest

REFERENCE

- Guideline for the Compilation of a Mandatory CODE of Practice to Combat Rockmass Failure Accidents in Massive Mining Operations.

LEARNING OUTCOME 8.2.4

SURFACE MINES

NOTE

The COP guidelines can be accessed here

Aspects to be addressed:
Overall mine stability

- Strategies to ensure overall stability of the mine must be well described.
- Strategies should be in place to prevent uncontrolled surface structure collapses of the mine.
- Strategies should be in place to prevent rockmass failures during mining operations.
- COP must have a comprehensive slope management programme.
- Procedures of making safe should be outlined to protect persons.
- Procedures of installing, maintaining and removing support should be in place.

Protection of mine accesses/exits
Access/exit ways should have rock-engineering appraisals that support their integrity.

Mining operations
Hazards from daily mining operations should be identified in the COP.

Rock-breaking
Rock-breaking strategies that minimise jeopardising the integrity of ground should be adopted.

The impact of mining operations on neighbouring mines
Mining operations should not cause slope instability on adjacent mines.

Slope instability
Major geological structures, discontinuities and intrusions should be continuously monitored to prevent slope instability.

Other aspects to be addressed:
- Significant rock-related risks and slope instability hazards must be recorded in the COP.
- Risk relevant to a certain ground control district must also consider mining layouts, mining sequence, support and monitoring procedure.
- Where hazards/risks are related to specific ground control districts, the relationship must be clearly indicated.
- Relevant risks (obvious and not obvious) associated with the extraction processes should be covered in the COP.
- Documentation regarding experimentation that differs from the COP, including related risk assessment records.
- Management processes on rock-related risks should inform overall mine planning process.
- Significant rock-related risks appraised by a rock engineer.

Implementation plan
Implementation plan should cover appointments and responsibilities in the COP.

Compliance with the COP
There should be systems in place for monitoring and ensuring compliance with the COP.

Access to the COP and related documents

- The COP documents should be accessible to people with material interest.
- Employees should fully understand sections of the COP that are relevant to their areas of responsibility.

Geotechnical considerations

i. Planning for total mine life: Design approach

Geotechnical considerations, from pre-feasibility studies, to mining operations and mine closure should be in place.

ii. Geotechnical design considerations

Geotechnical data for design should be obtained from a number of varied sources because of limitations of information at the time due to early phases of mining.

iii. Operational geotechnical considerations

During pit operations, ground control management plans should be used to improve the geotechnical database.

iv. At mine closure:

There should be adequate data to address the long-term geotechnical concerns associated with the abandonment of a mine.

v. Geological structure and rockmass strength

There should a balance on rock inherent stresses and mining-induced stresses, to maintain a stable rockmass.

vi. Hydro-geological considerations

The groundwater, surface water and rainfall influence, should be given importance in designing a mine.

Rock support and reinforcement

The design of ground support and reinforcement should be based on an understanding of the following:

- geological structure of rockmass,
- stress levels around excavations,
- rockmass strength,
- behaviour of the rock support or reinforcement system under load,
- groundwater patterns and
- potential for seismic events.

Pit wall design

Pit design geometry should be based on the overall mining plan.

Blasting considerations

Drilling and blasting strategies should take into account:

- rockmass properties,
- blasting patterns (i.e. burden, depth of charged holes, spacing, etc),
- efficiency of rock removal,
- degree of rock fragmentation required, etc.

Monitoring
Strategies should include regularly monitoring the performance of the pit walls to verify the stability.

Mining through underground workings
Hazards associated with such work should be well defined.

Ground slope management plan
Professional expertise (geotechnical engineers, surveyors, geologists, miners, and mining engineers) should be solicited for coming up with a ground slope risk management plan.

Hazard recognition
A strategy of identifying ground control hazards should be in place.

Mine design criteria
Pit wall design governed by these factors: consequences of failure and the degree of inherent uncertainty.

Rock Engineering Support Service (RESS)

- Assist the employer on proactively identifying significant rock-related hazards.
- Assist the employer in safe and economic design of mine workings.
- RESS must participate in planning.
- RESS must review, identify and make recommendations on management systems.
- RESS, assisted by a competent person, should analyse and interpret seismic incidents.
- RESS should also play an important role in monitoring, investigating and assuring quality of support systems.

REFERENCE

- Guideline for the Compilation of a Mandatory Code of Practice to Combat Rockfall and Slope Instability Related Accidents in Surface Mines

8.3 BASIC RISK ASSESSMENT

This section deals with basic hazard identification and risk assessment processes as applied to geotechnical mining hazards and risks such as falls of ground and rockbursts. The diligent application of these processes is required on every mine in terms of the Mine Health and Safety Act.

REFERENCE

- Mine Health and Safety Act, Act 29 of 1996
- Simrac Booklet, 1996, Practical Guide to Risk Assessment
- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process
- General knowledge of standards, practices and procedures concerning the identification of geotechnical hazards and the assessment of geotechnical risks on mines.

LEARNING OUTCOME 8.3.1

DESCRIBE AND EXPLAIN THE PROCESS AND PROCEDURE OF GEOTECHNICAL HAZARD AND RISK EVALUATION AND ASSESSMENT

The following will be discussed in this outcome:

1. Define and explain common terms and terminology used in connection with the process of hazard identification and risk assessment.
2. Describe, explain and evaluate the underlying purpose of the process of identifying geotechnical hazards and assessing geotechnical risks on mines as prescribed in terms of the Mine Health and Safety Act
3. Describe and explain the legal requirements prescribing the identification of geotechnical hazards and the assessment of geotechnical risks at mines.
4. Describe and explain the legal requirements prescribing the response required of managers to deal with identified geotechnical risks at mines.
5. Describe and explain the legal requirements prescribing the review of geotechnical hazards and geotechnical risks at mines
6. Describe the required risk management process arising from the preceding legal requirements and pictorially represent this as a flow chart.
7. List and describe the three types of hazard identification and risk assessment approaches that are required to be used on mines in terms of the Tripartite Working Group practical guide to the risk assessment process.
8. Describe the purpose, method and output of a baseline hazard identification and risk assessment process.
9. Describe the purpose, method and output of an issue-based hazard identification and risk assessment process
10. Describe the purpose, method and output of a continuous hazard identification and risk assessment process.
11. Describe and explain the relationship between the three types of hazard identification and risk assessment approaches.
12. Describe and explain the properties, purpose and expected outcomes of a suitable and sufficient geotechnical hazard identification and risk assessment exercise.
13. Identify, describe and explain by whom a geotechnical hazard identification and risk assessment exercise should be conducted.
14. Describe and explain the role of safety professionals in the geotechnical hazard identification and risk assessment process.
15. Describe and explain the general principles of how geotechnical hazard identification and risk assessment exercises are conducted.
16. Describe and explain the principles of determining the scope of geotechnical hazard identification and risk assessment exercises.
17. Describe and explain the principles of assembling teams and gathering data for geotechnical hazard identification and risk assessment exercises.
18. Describe and explain the principles of selecting and empowering geotechnical hazard identification and risk assessment teams.

back

19. Describe and explain the principles and practice of identifying geotechnical hazards.
20. Describe and explain the principles and practice of measuring and assessing geotechnical risk.
21. Describe and explain the principles and practice of prioritising geotechnical risk.
22. Describe and explain the principles and practice of reporting and recording geotechnical risks.
23. Describe and explain the principles and practice of preventing or controlling geotechnical risks.
24. Describe and explain the principles and practice of reviewing and revising geotechnical hazard identification and risk assessment initiatives.
25. Summarise the steps to effective geotechnical risk management.

LEARNING OUTCOME 8.3.1.1

DEFINE AND EXPLAIN COMMON TERMS AND TERMINOLOGY USED IN CONNECTION WITH THE PROCESS OF HAZARD IDENTIFICATION AND RISK ASSESSMENT.

DEFINITION

- HIRA – An acronym referring to the Hazard Identification and Risk Assessment process.
- HAZOP – A term referring to a study of the hazards of an operation, in which a structured approach, using guidewords, is applied to each activity in a process element to identify deviations from standard practice, their possible causes and probable consequences.
- Hazard – A source of potential harm capable of causing death or injury or disease to a person or persons, and/or of causing damage or loss or wastage of resources.
- Risk – The likelihood and consequence of a hazard actually causing death or injury or disease or damage or loss or wastage. (Risk is measured as the product of the frequency of occurrence of an event and its consequence)
- Frequency – A measure of the likelihood of recurrence of a harmful event (usually expressed in terms of a time period or probability).
- Consequence – The outcome of the occurrence of a harmful event (usually expressed in terms of severity). (i.e. The potential degree of harm, the potential severity of injuries and/or ill health, and/or the number of people potentially affected).
- Exposure – A measure of for how long and how often a person or persons are exposed to a hazard.
- Probability – The chance that a person or persons will be harmed during exposure to a hazard.
- Risk Ranking Matrix – A table that numerically ranks hazards and/or risks in terms of their likelihood and consequence.
- Protective Measures – The combination of risk reduction strategies taken to achieve at least tolerable risk. (Protective measures include risk reduction by inherent safety, protective devices, personal protective equipment, information for use and/or installation, and training).
- Reasonably Practicable – A subjective assessment of countermeasures, having regard to the severity and scope of the hazard or risk, the state of reasonably available knowledge concerning

back

the hazard or risk and the means of removing or mitigating the hazard or risk, the availability and suitability of means to remove or mitigate the hazard or risk, and the cost of removing or mitigating the hazard or risk in relation to the benefits derived there from.

- Residual Risk – Risk remaining after protective measures have been taken.
- Tolerable Risk – Risk that is accepted within a given context based on the current values of society.
- Safe – Free from any hazard and/or unacceptable risk.

- Simrac Booklet 1996 Practical Guide to Risk Assessment
- Tripartite Working Group 1996 Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.2

DESCRIBE, EXPLAIN AND EVALUATE THE UNDERLYING PURPOSE OF THE PROCESS OF IDENTIFYING GEOTECHNICAL HAZARDS AND ASSESSING GEOTECHNICAL RISKS ON MINES AS PRESCRIBED IN TERMS OF THE MINE HEALTH AND SAFETY ACT

The primary purpose of identifying geotechnical hazards and assessing geotechnical risks is to eliminate and/or reduce these hazards and risks, and thereby to prevent accidents and therefore ensure the safety of employees while they are at work.

In the past, risk management in the mining industry was mainly focused on reducing financial risk. The Leon Commission recommended that the mining industry should also provide for hazard identification and risk assessment processes in the management of occupational health and safety. This recommendation was included as one of the corner-stones of the Mine Health and Safety Act No.29 of 1996.

As a general rule, employers and employees carry out risk assessments on a day-to-day basis during the course of their work. They note changes in working practices, they recognise unsafe working conditions and practices as these develop, and they usually take the necessary corrective action. The Act requires that this process becomes more formalised, methodical and systematic.

In particular, employers must undertake thorough HIRA examinations of all work activities, and then record the findings of the hazard identification and risk assessments conducted. These analyses must be comprehensive, complete and recorded so that the results can be reliably used to eliminate hazards and/or reduce risks.

The risk assessment process at a mine must be continuous and should not be regarded as a one-off exercise. While it is necessary to establish a base-line hazard identification and risk assessment, the requirements of the Act will not be satisfied by the creation of a single risk assessment report that stands for all time.

- Mine Health and Safety Act, Act 29 of 1996
- Simrac Booklet, 1996, Practical Guide to Risk Assessment
- Tripartite Working Group 1996 Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.3

DESCRIBE AND EXPLAIN THE LEGAL REQUIREMENTS PRESCRIBING THE IDENTIFICATION OF GEOTECHNICAL HAZARDS AND THE ASSESSMENT OF GEOTECHNICAL RISKS AT MINES.

- Every manager must identify the hazards to health and safety to which employees may be exposed while they are at work.
- Every manager must assess the risks to health and safety to which employees may be exposed while they are at work.
- Every manager must record the significant hazards identified and risks assessed.
- Every manager must make those records available for inspection by employees.

REFERENCE

- Mine Health and Safety Act, Act 29 of 1996

LEARNING OUTCOME 8.3.1.4

DESCRIBE AND EXPLAIN THE LEGAL REQUIREMENTS PRESCRIBING THE RESPONSE REQUIRED OF MANAGERS TO DEAL WITH IDENTIFIED GEOTECHNICAL RISKS AT MINES.

Every manager, after consulting the health and safety committee at the mine, must determine all measures, including changing the organisation of work and the design of safety systems of work, necessary to:

a. eliminate any recorded risk,
b. control the risk at source,
c. minimise the risk, and,
d. in so far as the risk remains,
i. provide for personal protective equipment, and
ii. institute a programme to monitor the risk to which employees may be exposed.

Every manager must, as far as reasonably practical, implement the measures determined necessary in the order in which the measures are listed above.

REFERENCE

- Mine Health and Safety Act, Act 29 of 1996

LEARNING OUTCOME 8.3.1.5

DESCRIBE AND EXPLAIN THE LEGAL REQUIREMENTS PRESCRIBING THE REVIEW OF GEOTECHNICAL HAZARDS AND GEOTECHNICAL RISKS AT MINES

- Every manager must periodically review the hazards identified and risks assessed to determine whether further elimination, control and minimisation of risk is possible.

- Every manager must consult with the health and safety committee on the review.
- Every manager must conduct an investigation into every accident that must be reported in terms of the Act.

REFERENCE

- Mine Health and Safety Act, Act 29 of 1996

LEARNING OUTCOME 8.3.1.6

DESCRIBE THE REQUIRED RISK MANAGEMENT PROCESS ARISING OUT OF THE PRECEDING LEGAL REQUIREMENTS AND PICTORIALLY REPRESENT THIS AS A FLOW CHART.

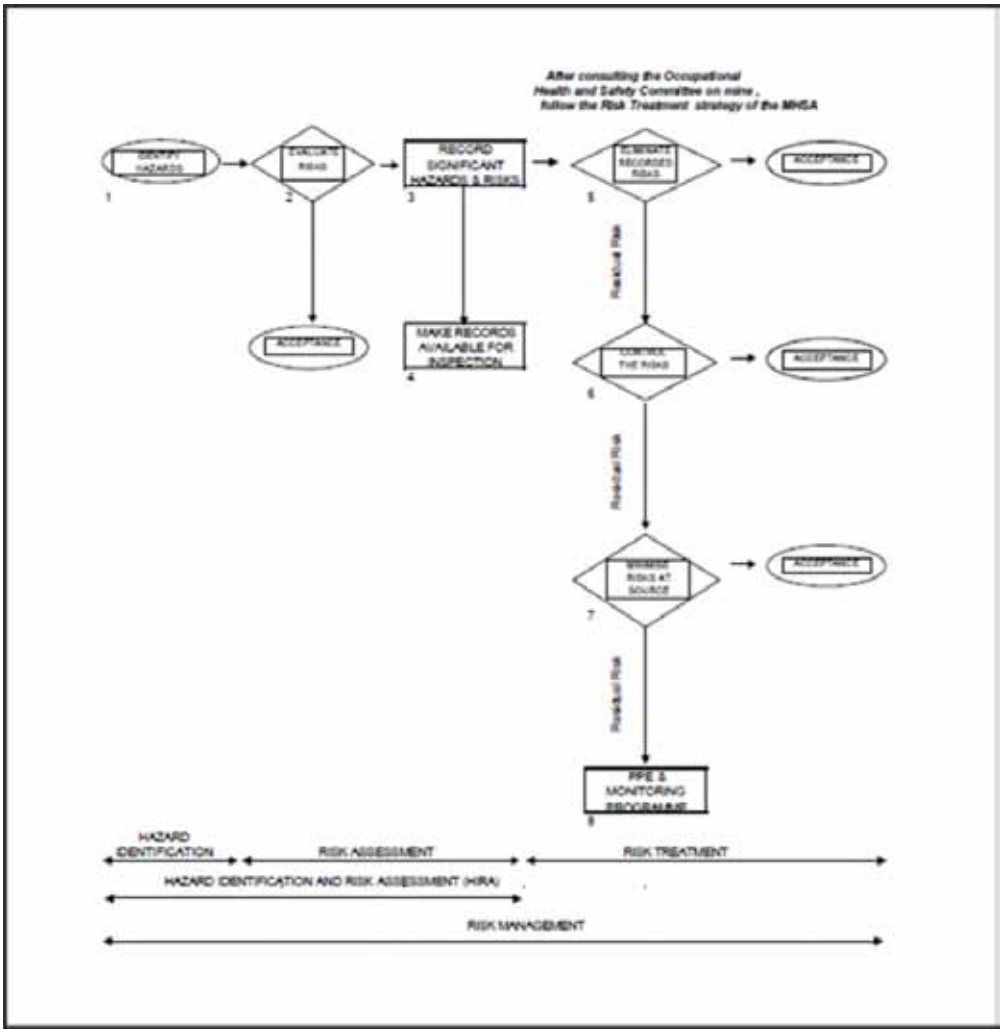


Figure 2: Risk management process

REFERENCE

- Mine Health and Safety Act, Act 29 of 1996
- Simrac Booklet, 1996, Practical Guide to Risk Assessment

LEARNING OUTCOME 8.3.1.7

LIST AND DESCRIBE THE THREE TYPES OF HAZARD IDENTIFICATION AND RISK ASSESSMENT APPROACHES THAT ARE REQUIRED TO BE USED ON MINES IN TERMS OF THE TRIPARTITE WORKING GROUP PRACTICAL GUIDE TO THE RISK ASSESSMENT PROCESS.

The three individual types of hazard identification and risk assessment approaches to be used on mines in terms of the Tripartite Working Group practical guide to the risk assessment process complement each other and are inter-related. Together they form an integral part of a management system.

Baseline HIRA Process
Comprises a process based upon a scan of the complete business and/or operational process to identify the hazards and determine the respective risks under the general headings of geographical areas, activities, occupations, and tasks.

Issue-based HIRA Process
Comprises a process based upon the evaluation of business and/or operational hazards and risks (issues) suggested by proactive information (inspection and observation reports) and reactive information (accident and incident investigations).

Continuous HIRA Process
Comprises a process based upon regular and continual inspections, measurements and observations of all working areas associated with a business or operation to constantly identify the hazards and determine the respective risks.

REFERENCE

- Simrac Booklet, 1996, Practical Guide to Risk Assessment
- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.8

DESCRIBE THE PURPOSE, METHOD AND OUTPUT OF A BASELINE HAZARD IDENTIFICATION AND RISK ASSESSMENT PROCESS.

Baseline HIRA Process
The purpose of a baseline hazard identification and risk assessment is to establish a risk profile in which all risks associated with the operation are identified, rated and ranked in order of significance within selected categories such as: geographical areas, operational activities, employee occupations, and individual tasks.

A baseline HIRA comprises a process based upon a comprehensive scan of the complete business process to identify the hazards and determine the respective risks within the categories referred to above (geographical areas, activities, occupations, and tasks).

Baseline risk assessments need to be comprehensive, and invariably lead to further, separate, more in-depth risk assessment studies. They are normally reviewed on an annual basis (usually to support planning and budgeting), or if and when business and/or operational circumstances could or do significantly change, in order to ensure that they remain relevant and accurate.

The outcome of the baseline process is used to prioritise action programmes for issue-based and continuous risk assessments. Other assessment studies will need to be incorporated to achieve the complete picture.

- Simrac Booklet, 1996, Practical Guide to Risk Assessment
- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process
- General knowledge of standards, practices and procedures concerning the identification of geotechnical hazards and the assessment of geotechnical risks on mines.

LEARNING OUTCOME 8.3.1.9

DESCRIBE THE PURPOSE, METHOD AND OUTPUT OF AN ISSUE-BASED HAZARD IDENTIFICATION AND RISK ASSESSMENT PROCESS

Issue-based HIRA Process

As circumstances change and needs arise, further separate risk assessment studies need to be conducted that go beyond baseline assessments. These will normally be associated with systems for the management of change. The purpose of an issue-based HIRA is to conduct a detailed assessment study that will result in the development of action plans for the treatment of significant, specific risks.

Issue-based risk assessments need to be carried out, for example, when a new product is introduced into the mine, or when a system of work is changed or operations are altered, or after an accident or 'near-miss' has occurred, or when new knowledge or information comes to light that may influence the level of risk to employees at the mine making previous risk assessments invalid.

An issue-based HIRA comprises a process based upon the evaluation of hazards and risks (issues) suggested by proactive information (knowledge, inspections, observations, reports) and reactive information (accident and incident investigations).

The output of issue-based HIRAs is recommendations to management regarding the implementation of all measures necessary to eliminate, or control and minimise, and to monitor the risk to which employees may be exposed.

- Simrac Booklet, 1996, Practical Guide to Risk Assessment
- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.10

DESCRIBE THE PURPOSE, METHOD AND OUTPUT OF A CONTINUOUS HAZARD IDENTIFICATION AND RISK ASSESSMENT PROCESS.

Continuous HIRA Process

This is the most important form of risk assessment. It may not use the more sophisticated hazard identification and risk assessment tools employed for baseline and issue-based assessments, but in terms of eliminating risks and ensuring safety in the workplace, this form of risk assessment is the most powerful and most important.

A continuous HIRA comprises a process based upon regular and continual (daily, or more often) inspections, measurements and observations of all working areas to identify the hazards and determine the respective risks. This should take place constantly and frequently, as an integral part of day-to-day management. As it is conducted mainly by frontline supervisors, it is essential that formal training be provided to enable this process to be effective and efficient.

Examples of continuous risk assessments include regular audits, general hazard awareness linked to a suggestion scheme, and pre-work assessments using checklists and other memory joggers (developed during audit and general hazard awareness exercises).

In a continuous risk assessment, the emphasis is on hazard awareness through hazard identification, rather than on the prioritisation of hazards, as this will already have been accomplished by the other risk assessment processes. In general, a continuous risk assessment involves identifying the hazards present in a work activity or working area, followed by an evaluation of the extent of the risk involved, followed by immediate risk treatment, taking into account those precautions already in place.

An outcome of this risk assessment process could be the development of a checklist that deals with critical activities and operations, using techniques such as critical task analyses, planned task observations, and key performance indicators to show the effectiveness of the underlying systems of control on an ongoing basis.

- Simrac Booklet, 1996, Practical Guide to Risk Assessment
- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.11

DESCRIBE AND EXPLAIN THE RELATIONSHIP BETWEEN THE THREE TYPES OF HAZARD IDENTIFICATION AND RISK ASSESSMENT APPROACHES.

Risk profiles generated by baseline HIRA are used for planning the other issue-based and continuous hazard identification and risk assessment programmes, and provide clear guiding principles so that issue-based HIRA and continuous HIRA are more effective, enabling continuous improvement.

In turn, new information from the issue-based HIRA can be used to improve on the continuous HIRA, which, in turn again, can update the baseline HIRA so that it remains comprehensive. The issue-based HIRA and the baseline HIRA draw from data captured by the continuous HIRA process to be effective. Therefore, there is constant interaction between the various types of HIRA, and a continuous closed loop of improvement is maintained.

Work practices and their associated defining documents such as codes of practice, standard procedures and management instructions are all strongly informed and influenced by the results of baseline, issue-based and continuous HIRA.

These risk management processes serve to keep management informed, and therefore provide a solid foundation and platform from which management is enabled to make sound, informed decisions for the protection of the workforce and to ensure their safety.

REFERENCE

- Simrac Booklet, 1996, Practical Guide to Risk Assessment

LEARNING OUTCOME 8.3.1.12

DESCRIBE AND EXPLAIN THE PROPERTIES, PURPOSE AND EXPECTED OUTCOMES OF A SUITABLE AND SUFFICIENT GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT EXERCISE.

A suitable and sufficient hazard identification and risk assessment process must consider all of the geotechnical risks to which the workforce may be exposed and that are likely to arise because of work activities. It must enable the development and implementation of actions and systems to manage the risks. These actions and systems must be appropriate to the nature of the work and should be such that they remain valid for a reasonable period of time.

A suitable and sufficient hazard identification and risk assessment process will result in the production of significant findings that may be used positively to, for example, introduce short-, medium-, and long-term controls, or to change working procedures if necessary. A sound risk assessment process can be the most important tool for changing the culture of a company by providing a basis for education and training.

For operations that do not change, the risk assessment should be such that it is not necessary to repeat it every time someone is exposed to a hazard in comparable circumstances. On the other hand, for operations and circumstances that do change often, for example, where the work activity may change fairly frequently, or the workplace itself changes and develops, or where the work involves workers moving from site to site, the risk assessment will have to concentrate more on the broad range of risks. Operational planning and employee training should take account of these types of risks and enable them to be managed as and when they arise.

REFERENCE

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.13

IDENTIFY, DESCRIBE AND EXPLAIN BY WHOM A GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT EXERCISE SHOULD BE CONDUCTED.

The HIRA process must to be transparent, impartial and focused. Most of all, it must deliver practical accident prevention and safety results. The process must, therefore, involve management and all employees, regardless of whether or not advisors or consultants assist with the process. Those involved in the risk assessment team have a duty of care to make sure that they and their colleagues are aware of the gravity of the exercise, and that they do not make critical errors of judgement.

For straightforward situations, where only a few hazards exist, or where the hazards are simple and well known, suitable and sufficient risk assessment can be a very clear-cut and uncomplicated process based upon judgements that require no specialist skills or complicated techniques.

back

On the other hand, when dealing with complex, technical safety systems, more detailed and specialised techniques will need to be applied, which actually quantify the levels of risk. In these cases, specialised advice may be necessary so that the team carrying out the risk assessment thinks as widely as possible in terms of the potential hazards, some of which they may be unfamiliar with. Preparation of the team carrying out the risk assessment is very important to ensure they do not ignore events because of their lack of personal experience.

In most cases, several risk assessments will need to be carried out for particular work activities and/or for particular working places. This approach will need to be carefully structured to ensure that all potential hazards are considered. Different teams, using different techniques, will approach the assessment of a work activity of workplace from different points of view. It is necessary to use a systematic approach to risk assessment to ensure, not only that an accurate and complete assessment is obtained, but also to ensure that all similar risk assessments produce similar results.

REFERENCE

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.14

DESCRIBE AND EXPLAIN THE ROLE OF SAFETY PROFESSIONALS IN THE GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT PROCESS.

The role of safety professionals in risk assessment processes is important, particularly in the case of complex situations. However, they should under no circumstances carry out risk assessments themselves, but must act primarily as facilitators. Their role is to guide and advise.
They must advise management on aspects such as the gathering of data, the selection of the team, and the scope and development of the risk assessment process.

After the risk assessment has been completed, safety professionals may assist in the development of measures, such as the production of codes of practice, which follow.

REFERENCE

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.15

DESCRIBE AND EXPLAIN THE GENERAL PRINCIPLES OF HOW GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT EXERCISES ARE CONDUCTED.

The character of a hazard identification and risk assessment process will depend very much upon the nature of the work, and the type and the extent of the hazards and risks that are involved. Although there are no fixed rules about how a risk assessment process should take place, there are some general principles that should be followed when conducting a risk assessment to ensure that it is suitable and sufficient.

All relevant hazards and/or risks must be systematically and thoroughly addressed and examined. However, remembering that risk assessment

back

back

is a process in which the aim is to identify the major risks involved in a work activity and/or workplace, these must not be obscured by concentrating too heavily on minor risks or those in which an excess of information exists. The level of detail in a risk assessment should closely match the actual level of risk.

Whatever safety controls and other measures already exist (such as, for example, codes of practice, standard procedures and special instructions) must be acknowledged and taken into account. However, the effectiveness of these controls must be carefully and regularly reviewed as they may not be working properly. It is particularly important that this is objectively assessed.

All relevant aspects of the work space that have the potential to cause harm must be considered. These include aspects such as the work environment, the materials and equipment used, and the work processes and/or work organisation.

All relevant aspects of the work activity must be reviewed. Risk assessments should address what actually happens in the workplace during the work activity. Actual practice may differ from what is supposed to happen in written instructions and written procedures. This is frequently the way that risks creep into an operation unnoticed.

All changes to the working environment (that may be potentially hazardous) need to be carefully managed. The effect on all employees must be considered, as well as those who may be only peripherally affected (including maintenance staff, security guards, visitors and contractors). Non-routine activities, such as maintenance operations, work interruptions, or changes in working methods or the workplace (equipment, materials and/or people) should especially receive consideration as these are frequent causes of accidents.
Those groups and individuals who may particularly be at risk, such as the young and inexperienced, and those who are required to work alone or who have disabilities, should also receive special consideration.

In summary, the purpose of a risk assessment is not to deal with every minor hazard. A suitable and sufficient risk assessment reflects what employers might reasonably foresee in terms of hazards and risks in workplaces. In most cases it is better to first make a rough assessment to prioritise the risks, and then, in the second assessment, more sophisticated techniques can be used to deal with the major risks.

REFERENCE

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.16

DESCRIBE AND EXPLAIN THE PRINCIPLES OF DETERMINING THE SCOPE OF GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT EXERCISES.

The first task when embarking upon a risk assessment is to define the scope of the process as a whole as well as the scope of the separate risk assessment exercises. The intention must be that all geotechnical mine activities are comprehensively reviewed and assessed. Some care is therefore needed to ensure that no areas or activities are missed.

Risk assessments may be scoped in different ways. They may be geographically based, such as shafts, tunnels and stopes. They may be

back

back

functionally based, such as travelling, cleaning, supporting and blasting. They may be hazard or issue based, such as rockfalls and rockbursts. They may be discipline based, such as drilling crew, cleaning crew and support crew. They may also be some combination of these approaches. Care must be taken if the approach to risk assessment is restricted to a single base, because of the danger that preconceptions of the areas of greatest risk may lead to a failure to identify a particular hazard. It is normally found that a selection of two bases is the most effective in ensuring a comprehensive and complete risk assessment of an operation or activity.

REFERENCE

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.17

DESCRIBE AND EXPLAIN THE PRINCIPLES OF ASSEMBLING TEAMS AND GATHERING DATA FOR GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT EXERCISES.

Before any risk assessment can proceed, a team has to be selected, background information has to gathered and processed, and the team has to be prepared for the task ahead. It is vitally important that all of these preliminary processes, and the subsequent hazard identification and risk assessments initiatives, are carried out within a participative framework by involving all stakeholders and their representatives.

The gathering and analysis of information from on-mine sources and from external sources is an essential task before the risk assessment can start. This is normally undertaken by the safety professional on the mine, and it is one of his more important duties.

He must access and interrogate the databases on the mine to assess the types and underlying causes of past accidents and incidents. He must also review accident reports and investigations, together with any other relevant records or sources of information generated, kept and maintained by production and rock staff, such as log books and audit reports. He may be able to gather external information from government departments and industry organisations, and/or from industry publications and databases. Increasingly, the Internet is a valuable means of gathering such local and international data.

All of this data needs to be gathered, assimilated and converted into a useful format to prepare the team who are to undertake the risk assessment.

The creation of a committed assessment team is essential to ensure ownership of the work and to help bring about culture change. The importance of this is shown below, where each of the elements that help to build up culture change is an output from the risk assessment process. The three steps to cultural change that support positive behaviour are awareness, motivation and systems.

back

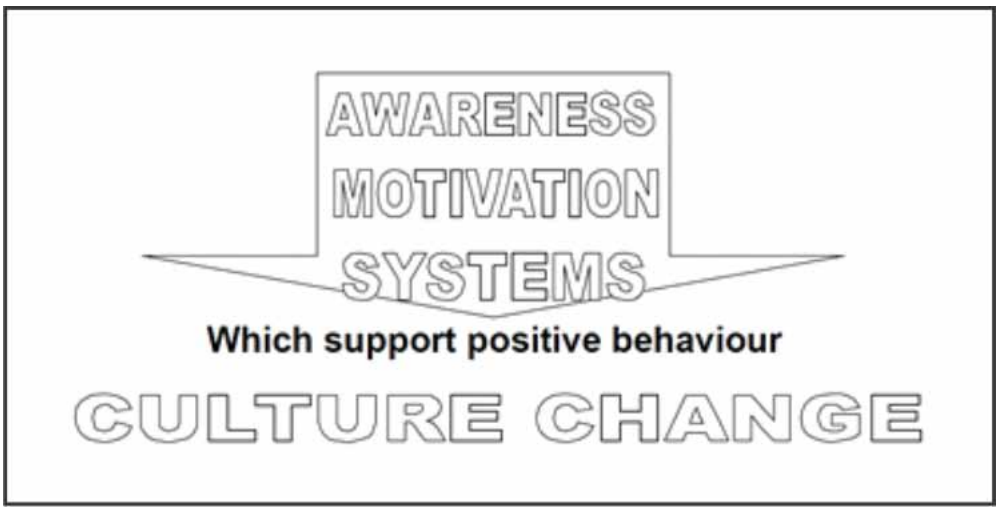


Figure 3: Culture change

- Tripartite Working Group 1996 Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.18

DESCRIBE AND EXPLAIN THE PRINCIPLES OF SELECTING AND EMPOWERING GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT TEAMS.

The assembly of the risk assessment team is another important role of the safety professional. It is normal to select a team from the workplace being reviewed by taking a vertical slice through the management structure to the lowest levels in the organisation. In practical terms, this will mean a selection from the front line supervisor and the team from the workplace being assessed. Where necessary, specialist expertise should also be brought in, for example, with regard to psychological factors, ergonomics and/or rock mechanics.

Once the team has been assembled, they have to be trained, conditioned and empowered for the work in hand. This will involve the safety professional presenting the analysed data, describing the scope of work, discussing the potential hazards the team might encounter, and encouraging them to consider not just the superficial causes, but also the underlying causes, thereby increasing their awareness. The purpose of this preparatory phase is to ensure that the team are able to visualise the hazards clearly when they go to the workplace.

Despite the fact that inspecting the workplace and visualising things more clearly are valuable steps in the process of hazard identification, it is also necessary to use a systematic approach to ensure a comprehensive and complete identification of hazards. There are a variety of tools and techniques available to assist the team in identifying the hazards, from simple checklists to the most sophisticated quantitative approaches.

The team should agree upon the appropriate tools, techniques and approaches that they will use for hazard identification. These must, however, be in keeping with the scope of the exercise. Where necessary, the safety professional will be able to advise and guide the team on the selection of the correct tools.

He should also be able to advise them on where additional training in the use of the tools or techniques is required, before the team proceeds with the hazard identification exercise. It is important that those conducting

back

risk assessments should be competent and should receive formal training in order to be so. For example, it is essential that all frontline supervisors are trained in critical task analysis and planned task observation methods, and in workplace inspections.

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.19

DESCRIBE AND EXPLAIN THE PRINCIPLES AND PRACTICE OF IDENTIFYING GEOTECHNICAL HAZARDS.

The first and most important stage in the risk assessment process is the systematic identification of hazards. If this is not carried out conscientiously and carefully, the subsequent analysis of risk and the development of risk control measures become pointless.

The identification of hazards is not only an essential part of the risk assessment process, but also acts very effectively to change the way people think and so becomes more proactive in safety awareness.

There are many techniques and tools that are available to be used as part of the hazard identification process, but in general they may be separated into two groups, namely top-down techniques and bottom-up techniques.

The top-down technique involves analysing a particular event, working downwards from the top, to arrive at the underlying causes. This technique is then applied to arrive at the potential underlying causes of potential other events. Examples of the techniques applied in this top-down approach include checklists, accident analyses, fault tree analyses and brain-storming. Task analysis, in its many forms, can also be classified as a top-down approach.

The bottom-up approach is one of breaking the work system or work practice into small components, and then seeing whether and how hazard or failure may occur in any of these, possibly building up to a major event. Examples of the techniques applied in this bottom-up approach include hazard and operability studies (HAZOP), failure modes effects analyses (FMEA) and its extension of criticality analysis (FME(C)A). Human reliability analysis, in its many forms, can also be classified as a bottom-up approach.

As there are so many such techniques available, the advice of knowledgeable safety professionals should be sought to guide the team on the best approach.

For the most complete and comprehensive identification of hazards, it is necessary to combine a top-down approach with a bottom-up approach. For example, systematic causal analyses of accidents are complementary to structured, predictive approaches such as HAZOP or What-If studies.

It can be seen from the definition of 'hazard' as the potential for harm, that 'near miss' investigations are a potentially powerful form of hazard identification. A near-miss incident is, in fact, a revealed hazard. In any workplace, there are more near-miss incidents than actual accidents. There are, therefore, great benefits to be had from extending causal analyses of accidents to near-miss incidents. Thorough analysis of these can be very informative. However, this does require a reliable near-miss

back

reporting system, and this is only possible in those companies where 'blame-culture' has been removed.

Whichever method is adopted, the approach to hazard identification must be holistic. It should not concentrate on one type of hazard, but should rather be broadly based and seek to identify all possible hazards to those at work and those who may be affected by work activities.

REFERENCE

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.20

DESCRIBE AND EXPLAIN THE PRINCIPLES AND PRACTICE OF MEASURING AND ASSESSING GEOTECHNICAL RISK.

Once hazards have been identified, it is necessary to prioritise them in terms of the risks that they pose so that an action programme can be devised and implemented to deal with them in a manner that will satisfy the 'reasonably practicable' requirement in the Act.

The aim of risk assessment processes is to enable management to make better decisions relating to the prevention of accidents and incidents. Risk assessments themselves do not make the decisions, they only provide the basis for decision-making and set the best direction for management to follow. The setting of priorities is also an important way to change employee understanding, and to build hazard awareness.

The approach to risk measurement and assessment does not have to be complex or sophisticated. It must, however, be sufficiently adequate to allow good decisions to be made with confidence. These must be presented in a form that can be communicated to employees, inspectors, and other stakeholders, and must demonstrate a sense of competent and responsible management.

There are several approaches that can be used for the measurement and assessment of risk. They all have to be supported by clear explanations and descriptions of the process so that the risk assessment team can all agree and can use a particular tool consistently.

Perhaps the most popular of these approaches include risk matrix type techniques, whereby the consequences (severity) and likelihood (frequency/probability) of particular hazards are decided upon and established separately by the team, and these values are then combined in a matrix to produce a risk-rating value, which in turn determines the priority of each risk element. A simple example of a fall of ground risk matrix is presented below to show basically what it looks like. It is necessary that individual mines generate their own customised matrices.

Alternative approaches include nomograms or risk-ranking tables and equations. Many of the structured hazard identification methods such as the FMECA and What-If techniques already include their own built-in risk assessment approaches.

All of these approaches are designed to achieve the same objective. They all provide the risk assessment team with a means of deciding on hazard consequence and hazard frequency, and then help the team decide on and prioritise the levels of risks.

Prioritising Risks	IMPORTANCE
High Consequences, High Frequency	
High Consequences, Low Frequency	
Low Consequences, High Frequency	
Low Consequences, Low Frequency	

Table 2: Risk level assessment

REFERENCE

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.21

DESCRIBE AND EXPLAIN THE PRINCIPLES AND PRACTICE OF PRIORITISING GEOTECHNICAL RISK.

Whatever system is used to rate and prioritise risks, it must have the following basic components:

A consequence assessment in which the degree of harm from the identified hazard is measured in terms of the potential severity of the injuries and the potential number of people affected.

A frequency or likelihood assessment in which the two separate components of exposure (an analysis of how often and for how long employees involved are exposed to the hazard) and probability (an analysis of the chance that employees involved will be harmed during the exposure period) are measured.

Even if no formal system for prioritisation of risks is used, all of the above three components (consequence, exposure, probability) of the risk should be considered qualitatively during any risk assessment.

Whichever approach is taken to prioritise risks, it must ensured that high frequency and high consequence risks are allocated the highest priority, that low frequencies and low consequence risks are given low priority, and that those risks with either high frequencies and low consequences or low frequencies and high consequences are considered medium priority. This principle is illustrated below (Table 3). It is also particularly important that high consequence events with low frequency are not ignored by the analysis.

The outcome of an analysis of these three components of the risk not only enables risks to be prioritised, but also helps in the development of appropriate risk control measures.

SEVERITY	Once a month	Once a year	Once every 10 years	Once in a lifetime
Multiple fatalities	1	2	3	4
Fatality	2	3	4	5
Reportable accident	3	4	5	6
Loss time accident	4	5	6	7

FREQUENCY

Table 3: Principle of priority

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.22

DESCRIBE AND EXPLAIN THE PRINCIPLES AND PRACTICE OF REPORTING AND RECORDING GEOTECHNICAL RISKS.

It is important that risk assessment processes and results are reported upon and recorded so that there is a reliable statement and evidence of what appropriate actions management is taking to protect the safety of employees.

Risk assessments must be an integral part of an employer's overall approach to safety, and the results need to be linked to other safety records and documents, such as rock-related codes of practice. Ultimately, they should become an integral part of a company's overall management system.

Whatever the format of a risk assessment report, it should be easily accessible by employees, their representatives and the inspectorate. Records of a risk assessment may consist of several documents that describe procedures and safeguards, the approach used and the results of the exercise.

Although the recording of the risk assessment process is important, the generation of a system on paper alone is not sufficient to ensure compliance with Section 11 of the Mine Health and Safety Act. Producing documentation should not detract from the major purpose of risk assessment, which is to improve the management of risks and thereby to ensure the health and safety of employees.

It is vitally important that the recorder of the risk assessment reports findings that are judged to be significant. These include the major

back

hazards and risks identified in the assessment (those risks that, if not properly controlled, pose serious consequences to employees or others who may be affected); a review of existing safety measures and the extent to which these are effective in controlling the existing risks; and a list of those who may be affected by major hazards and significant risks, including those employees and groups who are deemed especially at risk.

- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.23

DESCRIBE AND EXPLAIN THE PRINCIPLES AND PRACTICE OF PREVENTING OR CONTROLLING GEOTECHNICAL RISKS.

An analysis of the three components of risk not only enables risks to be prioritised, but also helps in the development of appropriate risk prevention, control and protection measures required in terms of Section 11(2) of the Mine Health Safety Act, which states that, if possible, risks should be eliminated, or, if this is not possible, that risks should either be mitigated, controlled or minimised or, if all else fails, that personal protection should be provided.

The types of preventative and protection measures that need to be provided for a particular risk will be one of the most important outcomes of the risk assessment process. In deciding upon those measures, the following principles provide guidance.

If possible, it is always best to avoid a risk altogether, by using a different approach or method of work.

Risks should be combated at the source rather than by adopting secondary measures. For example, if ground is dangerous, treating it appropriately is always better than providing a warning sign.

Wherever possible, work should be adapted to the individual rather than expecting the individual to adapt to the work. This means that the designs of the job and the workplace, including the choice of equipment and working methods, are important considerations. Particular care should be taken to alleviate monotonous work and to establish a work rhythm.

When treating risks, advantage should be taken of technological and technical progress. Just because solutions attempted in the past have failed, does not mean that new opportunities for improving working methods and making them safer should not be look for.

The adoption of risk prevention measures must be part of a coherent policy and approach to safety management, which act to progressively reduce those risks that cannot be prevented or avoided altogether. This must take into account the way the work is organised, the working conditions, the working environment and relevant social factors. Risk reduction measures taken in isolation are likely to fail. They must be part of an ongoing cycle of risk management that involves performance measurement, goal setting, feedback and analysis.

Those measures that protect the whole of the workforce should be given a priority to yield the greatest benefit. In other words, collective protective measures should be given priority in the interests of net benefit.

Whatever the protection provided, employees and those affected need to understand what they must do to make sure that the protection measure

back

back

works. All controls will fail unless they are backed up with adequate training and supervision.

Of great importance is that measures to avoid, prevent and reduce risks need to be an accepted part of the approach and attitudes applied to all activities at all levels of the organisation. Without an active health and safety culture supported by all levels of the organisation, isolated risk reduction initiatives will inevitably fail.

- Mine Health and Safety Act, Act 29 of 1996
- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

LEARNING OUTCOME 8.3.1.24

DESCRIBE AND EXPLAIN THE PRINCIPLES AND PRACTICE OF REVIEWING AND REVISING GEOTECHNICAL HAZARD IDENTIFICATION AND RISK ASSESSMENT INITIATIVES.

Section 11 (3)(a) of the Mine Health and Safety Act requires managers to review and, if necessary, to modify their risk assessments. Risk assessment is a continuous process and not a once-off activity.

As aspects associated with work or working environment change, so the associated hazards and risks may change and so also must the risk assessment be modified accordingly. Risk assessments may need to be reviewed or modified if an accident occurs, or if more is learnt about hazards in the workplace. Hazards may exist that were not fully anticipated in the existing risk assessment and now require further, appropriate preventative and protective measures.

After an accident has occurred, it is important to go back to the risk assessment to see whether the possibility of the accident was anticipated, whether it was decided to prevent that accident, and, if so, why the preventative measures did not work. If the accident was not anticipated by the risk assessment, does the risk assessment process need to be revised to ensure that it is suitable and sufficient. If the accident was anticipated by the risk assessment, but it was decided to tolerate the risk, the validity of that decision must be questioned. The question of why the accident occurred and what should be done to prevent similar accidents as far as is reasonably practicable, must be addressed.

The manager must cause the risk assessment to be reviewed if there are any changes or developments that suggest it may no longer be valid, or that it can be improved. It is good practice to plan to review risk assessment at regular intervals; the time between reviews being dependant upon the nature of the risks and the degree and rate of change that are likely to occur in the work activity. Such reviews form part of sound management practice.

- Mine Health and Safety Act, Act 29 of 1996
- Tripartite Working Group, 1996, Practical Guide to the Risk Assessment Process

back

LEARNING OUTCOME 8.3.1.25

SUMMARISE THE STEPS TO EFFECTIVE GEOTECHNICAL RISK MANAGEMENT.

- Make sure the risk assessment process is practical and realistic.
- Involve as many people as possible in the process, especially those at risk and their representatives.
- Use a systematic approach to ensure that all risks and hazards are adequately addressed.
- Aim to identify the major risks; do not waste time on the minor risks, do not obscure the process in too much detail.
- Gather all the information you can and analyse it as well as possible before starting the risk assessment.
- Start by identifying the hazards.
- Assess the risks arising from those hazards, taking into account the effectiveness of the existing controls
- Look at what actually occurs and exists in the workplace and, in particular, include non-routine operations.
- Consider the risks to all employees, visitors and contractors.
- Always keep a written record of the assessment, including all assumptions made, and the reasons for those assumptions.

Baseline Risk Assesment

NOTE

The plots on the graph follows the same sequence as the ratings in the table below.

EXAMPLE

Baseline assessments evaluate all hazards on a mine and indicate the relevant differences in risk levels between hazards, jobs or areas. The graph below indicates the fact that certain work to be performed on the operation has higher risk levels associated with it than others.

It is clear that early examination (5th on the list) and making safe practices (10th on the list) have the highest inherent risk levels associated with them, compared to other work being performed.

EXAMPLE

Issue based risk assessment

Every rock engineering visit conducted underground is an issue based risk assessment.

- If support is audited: Assess condition of support units and assess risk that it will not be able to perform its required function;
- FOG investigation: Evaluate all factors that played a role in the fall, rate their contributions and derive the most critical items that must be addressed to remedy the area (if risk is not too high) or abandon the area;
- Pillar behaviour: List and evaluate all factors that contribute to pillar behaviour, assess and indicate potential (risk) for pillars not being able to sustain the loads and maintain stability;
- etc

EXAMPLE

Continuous risk assessment

The panel risk ratings performed on many operations are viewed as continuous risk assessment systems as the working areas are continuously rated / assessed by evaluating critical factors, giving it a rating and determining overall risk levels to be addressed by remedial action such as increased support density, reduced advance rates etc.
The system shown below evaluates specific factors in two categories:

issues that cannot be controlled such as geology
issues that can be controlled, such as support installation.
By determining the risk levels, based on allocated ratings, for these two categories, remedial action can be drafted that are specific and appropriate. It can also be used to indicate that the risk is inherent to the area being mined (uncontrollable) or man-made (controllables).

From the ratings for the two categories, a final rating is provided for the working place, in this case described as a percentage where higher percentage indicates lower risk.

8.4. GENERAL MANAGEMENT

8.4.1 REPORTING

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

LEARNING OUTCOME 8.4.1.1

DESCRIBE THE REQUIREMENTS AND FORMAT OF A FORMAL TECHNICAL REPORT

In brief, a technical report is:

- a document that describes the process, progress, or results of a technical issue or problem that was researched or investigated;
- normally includes recommendations and conclusions;
- where there is a review process, it is often limited to within the originating organisation;
- a major source of technical information.

back

The following report guidelines are provided to assist in creating good technical reports, but may not be completely similar to specifications set by many companies that have internal requirements and which should also be adhered to.

Document structure

One major difficulty with a large document is the task of starting the report. The task of achieving good structure from the start is compounded by the myriad of thoughts in an attempt to sort out the logical progression of the document. The key to achieving both well lies in the contents list.

The contents list

For most people, the contents list is a summary of the chapter and section headings, together with a page index, and is normally written when the document is already complete. However, the contents list is the one place in the document where overall structure can be examined, so derive the contents list during the planning phase of the report. Early organisation of the contents list is certainly not necessarily easy, but will allow you to give structure to your thoughts.

Logical structure

This is the natural unfolding of a story as the reader progresses through the document. This is achieved by going from the general to the specific, with the background material preceding the technical data and evaluations, which should lead logically to the conclusions. If some of the detail is standard, but possibly difficult to obtain, it can be included as an appendix.

Bite-sized chunks

From the structure in the contents list, it should now be possible to write each of the paragraphs as a more-or-less independent entity, observing, however, the relationship between different sections. Assign section numbers to the different document sections using, for example, 'Headings' in MSWORD, making cross-referencing relatively simple.

Chapter order

With a technical document, it is often beneficial to write the technical chapters first, i.e. the core material, leaving the executive summary, introduction, discussion and conclusions until the end. This is especially important when some results are still not available and the time has come to begin writing your document. Even in cases where all results are available, leaving the introduction until the end allows a better perspective to be had on the document as a whole.

Appendix material

Many authors are uncertain as to what to include in the appendix section. Generally, appendices should contain relatively standard derivations and perhaps lists of parameter values, which would interfere with the continuity of the main body of the document. In particular, the appendix section should not contain:

- All the figures corresponding to the document. Ideally, these should appear alongside the appropriate text, or else after the references in a separate section.
- Photocopies of data sheets, or other easily-accessible material.
- Any material that is crucial to the continuity or flow of the 'story' in the main technical sections.

All appendices should be numbered consecutively, for example Appendix A1, Appendix A2, etc., in order to allow cross-referencing from the text.

back

back

Typical report sections

Depending on the nature of your document, or the requirements of your company, it may have the following sections:

- Title page: Title, Authors name, qualifications and affiliations, date, reference number, recipient, etc.
- (Optional) Dedication to a friend, family member, or loved one
- (Optional) Declaration that the material in the report is the author's own work
- (Optional) Acknowledgement to those who have helped or influenced your work
- Contents list that lists items from here on with appropriate page references, including a list of Tables and Figures;
- Abstract or Executive Summary, which summarises the report contents
- Introduction, which introduces the work, provides the motivation and context and outlines other related work
- Main technical chapters, which document the core technical work and that could include:
- Observations made during site visits
- Technical literature studies
- Data gathered
- Calculations performed
- Numerical modelling performed
- Designs executed
- etc
- Conclusions, which may also identify appropriate future work,
- Recommendations that might flow from the conclusions
- References and
- Appendices

Writing style is probably the most individual aspect of a report, but useful guidelines include:

The writer

All reports should be written in the third person, i.e. as an objective observer. Avoid using terms such as "I did this experiment". Instead, substitute terms, such as "The experiment was performed"'.

Who is the recipient

Decide, in advance of writing, who the likely reader of the document is. The document must be pitched at an appropriate level with sufficient background to allow understanding by the target audience. Failure to pitch the level correctly will also inevitably result in failure to communicate your ideas effectively.

Spelling

Poor spelling makes a document seem sloppy and may convey an impression that the engineering content is as loose as the general appearance. There are spelling checkers in virtually every word processor that could be used. Be careful, as a spelling checker will not flag an error if the 'incorrect' word is spelled 'correctly', e.g. If 'snow the results' was typed instead of 'show the results', the mistake will not be flagged. The document must therefore be checked rigorously for spelling mistakes and incorrect word usage.

back

Grammar

Poor grammar also makes a document seem sloppy and may convey a poor impression of the work conducted. Most word processors have grammar checkers, but the usefulness of these is debatable, so do not rely on them. If in doubt, keep your sentences short and ask someone to read through the document, especially if the language being communicated in the document is not your 'first' language.

Capital letters

Avoid excessive use of capital letters. Only use capitals for proper nouns (such as place names, company names, etc) and acronyms. Acronyms should be defined at the first point of usage and the acronym can then be used freely. Avoid the use of capitals for emphasis, use boldfacing or italics instead. Capitals can be used effectively to differentiate between different section heading levels. Make sure you are consistent in applying the same formats to headings throughout the document.

Justification

Engineers and scientists are constant sceptics and need to be constantly re-assured that your work is pragmatic. For each idea presented, you should establish some motivation for its undertaking and any assumptions made must be justified. Similarly, critical assessment should be made of your results.

Plagiarism

Plagiarism is an unacceptable breach of copyright, where an author presents methods, text or results as his/her own, without reference to the original source. Ignorance is no excuse and the consequences for plagiarism are serious where it appears in examinable documents.

Referencing

Make sure that your referencing method is acceptable to the company or the body to which the report is submitted.

Conclusions

Conclusions must give some overall insight into the value of your work in general and inform the reader of what the major impact is, together with any concerns / problems the reader should be aware of. To fill the conclusions section with a summary of what was discussed in the technical chapters is not a conclusion but a summary.

Visual appearance

A technical report can contain information in text, figures, tables and equations. The appropriate use of each can, for example, be used to break long sections of text.

Pictures

'A picture tells a thousand words'. Use figures liberally to communicate specific results (graphs) and show an overview of the system being described through block diagrams, etc. Where possible, put multiple plots on the same axes, so that comparative conclusions can be drawn. Ensure that each figure has a number and a title, so that it can be referenced from the text. Use automatic numbering for figures that is available in most word processing programmes, such as 'References – insert caption – figure' that is available in MSWORD, as it ensures easy cross referencing and the creation of lists of figures in the contents.

Tables

Tables are an excellent way of giving an overview of numerical results or providing information in a form that lends itself to comparison. Use automatic numbering for tables that is available in most word processing

back

programmes, such as 'References – insert caption – table' that is available in MSWORD, as it ensures easy cross referencing and creation of lists of tables in the contents.

Equations
Some authors shun the formality of equations, preferring to describe the required relationships in textual form. Give equations in a technical report where possible. Number all equations by using automatic numbering for equations that is available in most word processing programmes, such as 'References – insert caption – figure' that is available in MSWORD, as it ensures easy cross referencing and creation of lists of figures in the contents. If available in tour word processing software, type equations using the 'Insert Equations' option, as equations can then be viewed in a mathematically correct format. For example, typing an equation could read Stress = Force / Area = Force / (Length x Width) when typed, but will read Stress=Force/Area=Force/(Length x Width) when using the 'Insert Equations' option in the software.

Quality control
Having completed the document, you should submit the report to a supervisor for his input and quality control. However, it is not their job to correct spelling, grammar, etc. but to check on content, technical correctness, conclusions and recommendations. If a supervisor has to correct typos, grammar, etc., it is unlikely that he/she is going to have time to focus on the more important technical points.

The primary responsibility for the quality of your document therefore lies with yourself. It is worth taking that extra small amount of time to ensure that your document is professional and is free from grammatical and spelling mistakes before submitting it to your supervisor. This is done by Proofreading the document yourself and in this process you should attempt to look at the document in a fresh light as a reader completely new to the material would.

REFERENCE

- Hints on Technical Report Writing, John Ringwood, School of Electronic Engineering, Dublin City University .
- http://odtl.dcu.ie/wp/1999/odtl-1999-03.html

LEARNING OUTCOME 8.4.1.2

COMPILE CLEAR TECHNICAL REPORTS TO SENIOR MINE OFFICIALS ON SUBJECTS COVERED IN THIS SYLLABUS

This outcome must be an output from the user, using the information in the learning material guidelines, with special reference to "LEARNING OUTCOME 8.4.1.1"
However, the following is suggested in reports addressed to Mine Officials:

CONNECTION 74

Refer to "LEARNING OUTCOME 8.4.1.1" on page 628

- Maintain good quality report writing, but:
- Keep the technical parts of the reports short, though complete, and if possible, place the technical parts in appendices for completeness, reference and for the use of technically inclined personnel;
- Ensure the Executive summary is complete and easy to access by the recipient;
- Ensure the Recommendations are complete and easy to access by the recipient.

back

LEARNING OUTCOME 8.4.1.3

COMPILE CLEAR TECHNICAL REPORTS TO HEAD OFFICE PERSONNEL ON SUBJECTS COVERED IN THIS SYLLABUS

This outcome must be an output from the user, using the information in the learning material guidelines, with special reference to all the requirements described in "LEARNING OUTCOME 8.4.1.1", but ensure that the standards w.r.t. report writing in your Company are adhered to, especially if the reports are at the level of feasibility studies.

8.4.2 MINE ACCIDENTS

The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

LEARNING OUTCOME 8.4.2.1

CONNECTION 75

Refer to "LEARNING OUTCOME 6.2.1.2" on page 426

STATE, DESCRIBE AND EXPLAIN THE REQUIREMENTS OF INVESTIGATIONS INTO ROCKFALL AND ROCKBURST ACCIDENTS

LEARNING OUTCOME 8.4.2.2

STATE, DESCRIBE AND EXPLAIN THE MAIN CAUSES OF ROCK-RELATED ACCIDENTS IN VARIOUS MINING OPERATIONS IN VARIOUS MINING DISTRICTS

Statistics are available from the following website and include a large number of statistics kept by the Department of Mineral and Energy.

http://www.dmr.gov.za/publications/viewcategory/168-accident-statistics.html

NOTE

Statistics are regularly updated and to prevent this learning material from being outdated, the user is simply referred to the website above to download the latest statistics.

However, it is interesting to note a steady decline in the number of fall of ground accidents over the last approximately eight years in South African Mines, the result of huge efforts by all in the Industry, including rock engineers.

Figure 4: Decline in fall of ground accident numbers (Dated November 2011)

CONNECTION 76

SIMRAC (MHSC) reports, GAP055 and OTH003 - Accident causes in mines OTHER than Gold and Platinum, discuss some of the most pertinent causes of falls of ground accidents.

From these reports, the following summary on fall of ground accident causes:

- Systems and facilities available to prevent accidents may be inadequate to cope with the risk;
- Inadequate examination (making safe);
- Poor hazard identification;
- Failure of personnel to act responsibly;
- Difficulties in communication;
- Lack of adequate or suitable training.

In addition to those mentioned above, it is also suggested from experience that the following conditions play a large role in fall of ground accidents:

- Ground conditions (quality, stress fracture density and orientation);
- Geological structures (presence, density of jointing, orientations);
- Seismic activity (affecting ground conditions and support stability);
- Support installation (absent support, excessive spacings, poor timing of support installation);
- Human factors (health, training); and
- Environmental factors (temperature, air flow, mining height, etc).

NOTE

These causes are listed as a current view and should not be maintained indefinitely, but it is suggested of the Industry stands and it is suggested that these reports are studied, but that the data is updated by the user, using recent sources such as:

- *DMR statistics and reports (DMR website)*
- *Codes of Practice*
- *Seminar proceedings.*

It is suggested that more MHSC research reports are studied to improve knowledge and understanding of the causes of fall of ground accidents.
http://www.mhsc.org.za

8.4.3 ROCK ENGINEERING FUNCTIONS

This section deals with the functions of the rock engineering discipline on various types of mine, and the functions of various staff categories encountered in a rock mechanics department.
The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Describe, explain and discuss the functions of the rock engineering discipline on various types of mines.
- Describe, explain and discuss the functions of various staff categories encountered in a rock engineering department.

REFERENCE

- Mine Health and Safety Act, Act 29 of 1996
- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.
- A Handbook on Rock Engineering Practice, AJ Jager and JA Ryder.

LEARNING OUTCOME 8.4.3.1

DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF THE ROCK ENGINEERING DISCIPLINE ON VARIOUS TYPES OF MINES.

The following will be discussed in this outcome:

- Describe, explain and discuss the purpose of an effective rock engineering support service in mining operations.
- Describe, explain and discuss the basic duties of a rock engineering support service in order to achieve the above purpose.
- Describe, explain and discuss the duties of a rock engineering support service in respect of the mine planning and design function.
- Describe, explain and discuss the duties of a rock engineering support service in respect of seismic emission monitoring and analysis, and in respect of seismic damage control.
- Describe, explain and discuss the duties of a rock engineering support service in respect of routine monitoring and special investigations.
- Describe, explain and discuss the duties of a rock engineering support service in respect of quality assurance of support elements and support systems.
- Describe, explain and discuss the duties of a rock engineering support service in respect of research and technology transfer.
- Describe, explain and discuss the duties of a rock engineering support service in respect of generating and updating rock-related codes of practice and mine standards.
- Describe, explain and discuss the duties of a rock engineering support service in respect of training and instruction.

back

1. Describe, explain and discuss the purpose of an effective rock engineering support service in mining operations.

The purpose of a rock engineering support service (RESS) is to assist the employer in ensuring that rock mechanics and strata control principles are applied for the safe and economic design of mine workings.

The rock engineering support service must assist the employer with the proactive identification of significant rock-related hazards and risks, and advise on appropriate measures to deal with these before persons are injured or workings are damaged.

REFERENCE

- Mine Health and Safety Act, Act 29 of 1996
- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.

2. **Describe, explain and discuss the basic duties of a rock engineering support service in order to achieve the above purpose.**

A rock engineering support service (RESS) must provide the following basic support services in order to achieve the purpose outlined above.

- The RESS must participate in planning activities in order to identify and evaluate all layouts and face positions, and to determine any potentially dangerous or damaging situations created by, or likely to be created by, mining operations.
- The RESS must identify, recommend and review systems, procedures and techniques employed by the mine to reduce or eliminate rockfall and rockburst hazards.
- The RESS must establish monitoring, recording and reporting systems that ensure that relevant information is timeously provided to the appropriate people in planning and operating functions.

REFERENCE

- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.

3. Describe, explain and discuss the duties of a rock engineering support service in respect of the mine planning and design function.

A rock engineering support services (RESS) must make the following contributions to the mine planning and design function.

- The RESS must assist with the design of rock excavation layouts that conform with the requirements of relevant authorities in respect of surface structures.
- The RESS must propose and assist with the design of mine support systems.
- The RESS must assist in the selection of the most appropriate rock excavation processes and techniques commensurate with prevailing conditions to ensure that the desired level of excavation stability is maintained for the operational life by applying best practice rock-engineering analyses.
- The RESS must assist in the selection of the most appropriate regional support systems commensurate with prevailing conditions to ensure that the desired level of excavation stability is maintained for the operational life by applying best practice rock-engineering analyses.

back

back

- The RESS must advise on risk management strategies to treat significant rock-related hazards and risks.
- The RESS must advise on the use of protection, control and support pillars at the mine.
- The RESS must advise on the location, juxtaposition, shape, support and damage prevention measures for all tunnels and other service excavations to ensure stability throughout the active life of these excavations.
- The RESS must evaluate and advise in respect of rock excavation processes, mining sequences and mine plans to ensure, as far as reasonably practicable, that the probability of seismic events and rockbursts is eliminated, minimised or controlled, that the factors affecting the stability of off-reef excavations are taken into account; that systems perform to design criteria under current and anticipated rock-related conditions and that precautions necessary for the excavation of remnant blocks are specified.
- In bord and pillar workings, the RESS must advise on the appropriate bord width in accordance with sound rock engineering principles and best practice, as well as on the appropriate accompanying bord support systems; it must advise on the appropriateness and correct application of standardised rock excavation process planning and design techniques; and must evaluate and advise on the planning and design of pillar extraction processes in accordance with sound rock-engineering principles and best practice.

REFERENCE

- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.

4. Describe, explain and discuss the duties of a rock engineering support service in respect of seismic emission monitoring and analysis, and in respect of seismic damage control.

On mines where seismicity and the occurrence of rockbursts is, or is potentially, an issue, the rock engineering support service (RESS), assisted where necessary by competent persons using appropriate equipment, must monitor, analyse and interpret the incidence of seismic emissions and advise on damage control measures.

In seismically active mines, the RESS must advise on strategies to deal with the incidence and impacts of rockbursts, for inclusion in the mine's mandatory code of practice.

On mines or sections of mines where the bulk of the ore reserves are located in remnants and/or pillars, the RESS must perform a periodic, detailed hazard identification and risk assessment of the whole mine, focusing on the sequence, phasing and manner of pillar and/or remnant extraction so as to ensure that these can be mined without significant risk to the people involved with such extraction.

REFERENCE

- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.

5. **Describe, explain and discuss the duties of a rock engineering support service in respect of routine monitoring and special investigations.**

A rock engineering support services (RESS) must make the following contributions to the mining operation in respect of routine monitoring and special investigations.

back

- The RESS must assist with the design of rock excavation layouts that conform with the requirements of relevant authorities in respect of surface structures.
- The RESS must regularly monitor pillar performance to ensure that they conform to design requirements.
- The RESS must conduct regular inspections of production and service workings to detect abnormal conditions and/or departures from planned layouts.
- The RESS must conduct regular inspections of important chambers during their excavation to ensure adherence to the designed excavation sequence and installation of the designed permanent support.
- The RESS must conduct regular monitoring of the performance of support systems in important and working excavations.
- The RESS must conduct regular investigations of unusual ground conditions, and report findings and make recommendations regarding remedial action.
- The RESS must appropriately participate in and assist with the investigation of all rock-related fatal accidents, including the compilation and completion of the rock engineering aspects of the official accident report.
- The RESS must conduct inspections of all incidents of reportable rockfalls and rockbursts, and must submit reports thereon.
- Where a significant risk of instability exists in shafts, the RESS must conduct regular monitoring of displacements, particularly on fault plane intersections.

REFERENCE

- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.

6. Describe, explain and discuss the duties of a rock engineering support service in respect of quality assurance of support elements and support systems.

The rock engineering support service (RESS) must advise management in respect of quality assurance systems for the support elements and support systems used at the mine

REFERENCE

- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.

7. **Describe, explain and discuss the duties of a rock engineering support service in respect of research and technology transfer.**

The rock engineering support service (RESS) must remain aware of new technological developments and must actively pursue the introduction of those that can be gainfully employed on the mine.

The rock engineering support service (RESS) must investigate and review, on an on-going basis, the possibility of improving existing support and other rock engineering systems on the mine.

REFERENCE

- A Handbook on Rock Engineering Practice, AJ Jager and JA Ryder.

back

8. **Describe, explain and discuss the duties of a rock engineering support service in respect of generating and updating rock-related codes of practice and mine standards.**

The rock engineering support service (RESS) must assist in the compilation and updating of the mine's rock-related codes of practice and related mine standards, in consultation with the mine's Health and Safety Committee.

- Mine Health and Safety Act, Act 29 of 1996

9. **Describe, explain and discuss the duties of a rock engineering support service in respect of training and instruction.**

The rock engineering support service (RESS) must assist with the training of departmental rock mechanics staff in all aspects of rock mechanics and in strata control applicable to their mines.

The rock engineering support service (RESS) must assist with the training of underground production and other relevant personnel in strata control applicable to their mines, with particular emphasis on the identification of dangerous ground conditions.

The rock engineering support service (RESS) must assist with the training of mine officials in relevant aspects of rock mechanics and strata control applicable to their mines.

- A Handbook on Rock Engineering Practice, AJ Jager and JA Ryder.

LEARNING OUTCOME 8.4.3.2

DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF VARIOUS STAFF CATEGORIES ENCOUNTERED IN A ROCK MECHANICS DEPARTMENT.

All of the forgoing functions, duties and responsibilities of a rock engineering support service outlined in the previous section may be, and are, allocated appropriately to various staff members occupying the various responsibility levels within the organisation.

Therefore, overall responsibility for the functions, duties, responsibilities and running of a rock engineering support function will rest with the rock mechanics engineer that heads the department.

Rock engineering support service responsibilities will be divided up, apportioned appropriately and delegated downwards to various levels of technical staff, rock mechanics officer, strata control officer, rock mechanics observer, trainee, and so on, that exist within the particular department. Reporting channels normally follow these delegation structures in reverse.

The responsibilities, and, in fact, the various responsibility levels within an organisation, are allocated in accordance with their import in terms of qualifications required to meet the responsibility and, to a lesser extent, job experience. Also, areas of individual responsibility are normally allocated in terms of workload, geographical considerations, and fairness. While not absolutely inflexible and rigid, the allocation of responsibilities outlined in the previous section generally conforms to the principles discussed above.

back

- Mine Health and Safety Act, Act 29 of 1996
- DME Guidelines for the Compilation of Rockfall and Rockburst Codes of Practice.
- A Handbook on Rock Engineering Practice, AJ Jager and JA Ryder.

BRIDGING MATERIAL

CONTENTS

- Greek alphabet
- Constants
- Units
- Signs and symbols
- Metric pre-fixes
- Glossary
- Terminology
- Maths library
  - Co-ordinate systems
  - Quadrants and trigonometry signs in quadrants
  - Cubic equations
  - Absolute values
- Physics library
  - Energy
  - Stress & strain

GREEK ALPHABET

Name	Lower case	Upper case	Name	Lower case	Upper case
alpha	α	A	nu	ν	N
beta	β	B	xi	ξ	Ξ
gamma	γ	Γ	omicron	ο	O
delta	δ	Δ	pi	π	Π
epsilon	ε	E	rho	ρ	P
zeta	ζ	Z	sigma	σ	Σ
eta	η	H	tau	τ	T
theta	θ	Θ	upsilon	υ	Y
iota	ι	I	phi	φ	Φ
kappa	κ	K	chi	χ	X
lambda	λ	Λ	psi	ψ	Ψ
mu	μ	M	omega	ω	Ω

CONSTANTS

G	Gravitational constant	6.6742 *10-11 Nm2/kg2
RE	Earth radius (average)	6.376 * 106 m
g	gravitational acceleration (sea level)	9.806 m/s2
k	Coulomb-force constant	8.988 * 109 Nm2/C2

back

c	Speed of light in vacuum	299 792 458m/s
e	Euler's number	2.71828
π	Pi	3.14159
π2	Pi squared	9.86960...
√2		1.41421...
√3		1.73205...
1°		0.01745... rad
1 rad		57.29577... °

METRIC PREFIXES

tera	T	10^{12}
giga	G	10^{9}
mega	M	10^{6}
kilo	k	10^{3}
hector	h	10^{2}
deka	da	10^{1}
-	-	10^{0}
deci	d	10^{-1}
centi	c	10^{-2}
milli	m	10^{-3}
micro	µ	10^{-6}
nano	n	10^{-9}
pico	p	10^{-12}
femto	f	10^{-15}

MEASURING UNITS

Length	metre [m]
Mass	kilogram [kg]
Time	second [s]
Angle	radian [rad], degree [°]
Force	Newton [N] = {kgm/s2}
Energy	Joule [J] = [Nm]
Pressure	Pascal [Pa] = [N/m2]
Frequency	Hertz [Hz] = [1/s]
Current	Ampere [A] = [C/s]
Electric potential	Volt [V]
Stress	Pascal [Pa] = [N/m2]
Strain	- [m/m]

SIGNS & SYMBOLS

= equals
≈ equals approximately

back

back

~ is in the order of
≠ is not equal to
is defined as
> is greater than
< is less than
≥ is greater than or equal to
≤ is less than or equal to
± plus or minus
is proportional to
∑ is the sum of

GLOSSARY

Accelerometer	A seismic transducer that produces an electric signal proportional to ground acceleration. Accelerometers are suitable for the detection of very small seismic events (micro-seismicity).
Aftershock	A seismic event that follows soon after a larger seismic event and originates at or near the focus of the first event. Aftershocks are generally at least 1,5 magnitude units smaller than the main shock.
Amplifier	An electronic device to increase the level of an analogue signal.
Amplitude (wave)	The maximum height of a wave, or depth of a trough, as measured from the equilibrium particle position.
Analogue to digital converter (a / d)	An electronic device that converts an electrical signal to numbers suitable for use in a digital computer.
Apparent stress	A model independent measure of the stress change at the seismic source. It is defined as: σ A = GE / Mo where G = modulus of rigidity, E = seismic energy, and Mo = moment
Apparent volume	Measures co-seismic inelastic deformation.
Attenuation	Reduction in the amplitude of a wave as the distance to the source increases.
Bedding plane	Visible planar discontinuity in sedimentary or metamorphic rock, which may be sufficiently weak to form a separation.
Blocky	Refers to roughly equi-dimensional fragments of rock formed between steeply intersecting joints, induced fractures, bedding planes or other discontinuities.
Body waves	P-waves and S-waves as opposed to surface waves. S-waves cannot propagate in liquids.
Brune's model	One of the most widely used models of the seismic source. It allows the displacement (D) and average static stress drop(□ S) over the area of rupture to be calculated according to: D = Mo / (6.28GRs2) where Rs = source radius
Burst fracture	Steep-dipping shear fracture sub-parallel to the line of the face, invariably comminuted or shattered within a zone of up to several centimetres wide. Small secondary or "feather" fractures usually form at an acute angle pointing in the direction of relative displacement. Primary displacement can have magnitudes of up to a few centimetres.
Choked	Descriptive of a working place that is almost completely filled with fragmented rock that has fallen or been thrust into the opening.
Cleavage surface	Can split through grains or crystals. Sometimes it has curved or fan-like traces usually starting at a single point, which may indicate rapid exclusion of a slab by spilling. This surface results from indirect tensile failure or extension fracture.
Closure	The amount by which the cross-sectional dimension is actually reduced by the combined effect of convergence and additional inelastic movements, such as bed-separation, dilation, bulking, footwall heave etc.
Clustering	Expresses the proximity in time and space between events in a group. Clustering proportional to the product of mean time difference and average distance between events.

back

back

Coefficient of friction	Constant of proportionality relating normal stress and corresponding critical shear stress at which sliding starts between surfaces.
Coda	That part of a seismogram that follows the source pulse, either P-waves or S-waves. Coda waves originate from scattering of body waves by inhomogeneities.
Cohesion	Shear resistance at zero normal stress.
Comminution	White, finely-ground rock material (rock-flour) characteristic of burst fractures. May be indicative of secondary movement on induced fractures.
Compressive stress	Normal stress tending to shorten a body in the direction in which it acts.
Conglomerate	Sedimentary rock containing (semi-) rounded pebbles
Convergence	The amount by which the cross-sectional dimension of an excavation (usually the working height of a stope) is reduced as a result of elastic relaxation in the surrounding rock continuum.
Corner frequency	The frequency at which the spectrum of ground velocity of P- or S-waves reaches a maximum. The corner frequency is generally assumed inversely proportional to the source dimension.
Creep	Time-dependant deformation without seismic energy release.
Cross bedding	Internal bedding oblique to main bedding planes.
Crushed	Descriptive of rock that has been reduced to small whitened fragments (sometimes elongated and "splintery"), where cohesion has been reduced to the extent that the rock can be removed by hand.
Dip	Angle at which a stratum or planar feature is inclined from the horizontal (positive down).
Dispersion	The distortion of a wave form due to a change of velocity with frequency.
Dodlomite	Limestone containing magnesium carbonate and calcium carbonate.
Dome	Approximately circular, convex geological feature.
Dyke	Tabular, near-vertical intrusion.
Dynamic stress drop	Difference between initial stress and the kinetic friction level on a fault.
Earthquake	A sudden motion in the earth caused by the abrupt release of accumulated strain of tectonic origin. When associated with mining, earthquakes are labelled "seismic events".
Energy index	Ratio of seismic energy of a given event to the average energy of events of the same seismic moment; indicator of the stress level in the rock in which the event took place.
Epicentre	The point on the earth's surface directly above the focus of a seismic event.
Fast fourier transform (fft)	A rapid numerical technique used for determining the frequency content of a wave.
Fault	Discontinuity where one block of the rockmass has moved relatively to the adjacent block.
First motion	The initial swing in a wave radiated from a seismic event as recorded at a given station.
Fluctuation	Change over time, variation.
Focus	The initial rupture point within a seismic source at which strain energy is first converted to elastic energy.
Foreshock	A seismic event that precedes a larger seismic event and that originates at or near the hypocentre of the main shock.
Fracture	Break in the continuity of a body of rock due to mechanical failure as a result of excessive stress. Fractures may be open or closed, but not "bonded" together.
Geophone	A sensor converting ground velocity into an electric signal. Geophones record seismic events in a wide range of frequencies.
Graben	Elongated block of the rock mass that has dropped down between the two adjacent blocks.
Hazmag	Estimate of the largest magnitude event to be expected in an area. Usually within 0.2 to 0.3 of the largest event recorded in the past. Based on the cum. moment of all events in a group converted to an equivalent magnitude.
Head waves	Waves critically refracted along a lithological boundary with a velocity contrast.

back

back

Hypocentre	Idealised point source of a seismic event, defined in 3D-space.
Joint	A usually smooth-walled discontinuity with no displacement. Often shows slight alteration, sometimes bonded. It is occasionally difficult to distinguish from an induced fracture.
Joint-laminations	Closely-spaced jointing that cuts rock into slices.
Lamination	Fine layering of rock due to closely-spaced bedding.
Limestone	Layered sedimentary rock consisting mainly of calcium carbonate ($CaCO3$).
Local magnitude	Measure of the strength of a seismic event; based on energy release, source size, and forces acting at the source.
Magnitude	An arbitrarily defined quantity characteristic of the total energy released by a seismic event. The Richter magnitude scale is related to the logarithm of an observed maximum displacement on a calibrated instrument and to its distance from the epicentre. The Gutenberg-Richter formula is an empirically determined relationship between energy and magnitude: E (In MJ) = 101.5M-1.2 Local magnitude as it is used in mine seismology is calculated from seismic energy and seismic moment.
Mechanism (Source mechanism)	The physical process of ground disturbance at the seismic source.
Micro-seismic event	A very small seismic event that is normally detectable only by means of sensitive instruments and that is unlikely to cause damage, but whose effects may extend into the audible range of 'rock noise' underground.
Mine seismology	Application and extension of seismological techniques to seismic events associated with mining. It includes the study of the relationship between seismicity and mining-related parameters.
Moment tensor (seismic)	3D distribution of forces inside the seismic source volume that is associated with inelastic deformation
Origin time	The initiation time of a seismic event. The origin time is usually determined simultaneously with the focus when an event is located.
Particle velocity (ground velocity)	The velocity with which a particle is displaced by a seismic wave.
Pegmatite	Coarse-grained, vein-like intrusion consisting of quartz, feldspar and mica.
Precursor	A phenomenon that precedes a seismic event.
Prediction	Anticipation of the time or magnitude or position (or all three) of a seismic event.
P-wave	The first seismic body wave emanating from a seismic source. Particle displacement is in the direction of wave propagation.
Pyroxene	Group of silicates found in igneous rocks, rich in Mg, Fe, Ca;
Pyroxenite	Coarse-grained igneous rock consisting mainly of pyroxene.
Quartzite	Common metamorphic rock consisting of silica ($SiO2$) grains.
Quick magnitude	Quick estimate of the local magnitude of a seismic event; available from the seismic system within seconds after the event.
Rayleigh wave	A type of surface wave propagated along a free surface of a semi-infinite medium. The particle motion takes place in a vertical plane parallel to the direction of propagation.
Reflection	A type of ray-bending in which part of the energy is reflected from the velocity contrast.
Refraction	Change in direction of wave propagation when waves pass obliquely from one region of velocity to another.
Rock type	Igneous = originates from deep parts of the earth's crust and cools slowly at shallow depth (plutonic, coarse grained); or erupts onto surface and cools rapidly (volcanic, fine grained); Sedimentary = result of sedimentation process of rock fragments, of organic or of chemical materials; compacted over time forming hard rock; Metamorphic = Igneous or sedimentary rocks transformed by high heat and/or pressure into rocks of different mineral content and texture.

back

back

Rockburst	A sudden and violent disruption of rock or disturbance of excavation walls in mines that is caused by, or accompanied by, a 'shock' or tremor. Injury to persons (or damage to equipment) resulting from an isolated fall of rock or from flying fragments does not necessarily indicate the occurrence of a rockburst, although a tremor from a distant event can cause the fall of an unstable piece of rock. Such an occurrence might be denoted as "fall of ground associated with a seismic event".
Sampling rate	The rate at which an analogue-to-digital converter samples the analogue signal. In seismology a constant sampling rate is used.
Sandstone	Sedimentary rock made up of visible sand grains.
Scaling	The non-violent formation of slabs that have become completely separated from the excavation surface.
Seismic deborah number (S.Relaxation time / flow time)	The ratio of elastic to viscous processes in seismic deformation expresses stability of a system; low values correspond to low stability.
Seismic energy	That part of total strain energy that is converted to elastic waves and radiated away from a seismic source. The energy released by a tremor can be calculated directly from a seismogram. It is usually assumed to have spread out in a spherical wave front from the source. The energy of the passing seismic waves is calculated at each seismometer site and is used to estimate the total radiated seismic energy.
Seismic event	Sudden failure due to stresses exceeding the strength of the rock mass or a discontinuity. The resulting emission and radiation of kinetic energy in the form of ground vibrations cause a noticeable 'shock' or tremor.
Seismic hazard	Hazard (probability of loss) associated with seismicity.
Seismic hazard assessment	Seismic hazard, in its basic form, is expressed as the probability and magnitude of the largest event expected. More complex assessments can include event frequency, expected PPV, probability of occurrence of large events, etc.
Seismic moment	Seismic moment is a measure of the earthquake size and is related to the leverage of the forces (couples) across the area of slip. Unlike magnitude, the seismic moment is not a scalar but a tensor (i.e. its value is different depending on the direction). This characteristic can be used to determine other source parameters, such as slip direction. Scalar seismic moment is the product of the modulus of rigidity (G), the rupture area (A), and the displacement or "ride" (D): Mo = GAD (in Nm).
Seismic network	System for data collection and analysis of seismic data utilising a number of sensors, data transmission and storage facilities.
Seismic potency	A measure of earthquake size or strength. P is equal to seismic moment divided by shear modulus G.
Seismic parameters	A set of variables that are used to describe seismic events or the seismicity of a confined area.
Seismic response to production=srp (Σva / σvm)	Ratio of cumulative Apparent Volume VA in a given time and space window over volume mined VM
Seismic relaxation time \ (S.Viscosity / rigidity)	Measures predictability of the seismic process: short relaxation time indicates a quick system response, i.e. low predictability;
Seismic schmidt number (S.Viscosity / diffusivity)	Expresses the degree of complexity of seismic processes in time and space; unpredictable systems with quick response time have low Schmidt number values
Seismic source	A volume of rock that fails suddenly, deforms, and releases energy of various types.
Seismic strain (Σmo / gδv)	Measures seismic moment density in rock volume ΔV: Proportional to ratio of cum. seismic moment / ΔV
Seismic stress (Cum. Energy / cum. Moment)	Indicates areas of high stress by measuring relative seismic energy release per unit of seismic moment
Seismology	A geophysical science, which is concerned with the study of earthquakes and measurements of the elastic properties of rock.
Seismometer	Contains A/D converter, data storage, and communication devices. Forms part of a seismic network.
Shale	Fine-grained, layered sedimentary rock.

back

back

Shattered	a) Of rock – disruption into fragments so small that it is not easy to determine whether these are basically slabs, slices or blocks. b) Of support packs – timber component severely compressed and splintered and concrete bricks fragmented.
Shear-wave splitting	Result of differing horizontal and vertical S-wave velocities in a layered medium; visible in seismograms.
Sill	Shallow-dipping tabular intrusion, parallel to the bedding of the intruded strata.
Slabbing	The non-violent formation of slabs that are not detached from the free surface.
Slicken-siding	Polished, linear features or grooves on discontinuities of geological age, such as faults.
Slip	Used to denote a minor fault or joint with very slight displacement.
Source dimension	Measure of the linear dimension of seismic source, inversely proportional to the corner frequency.
Source mechanism	The 3-dimensional distribution of forces and resulting movements within the seismic source.
Source pulse	The part of a seismogram resulting from waves that radiate directly from the source.
Source volume	Volume of inelastic deformation of a seismic source.
Spalling	The violent formation of slabs that separate from a strained surface. If the force is sufficient for the slab to be ejected from the surface, this would constitute one form of strain burst.
Spitting	Small strain burst that causes splinters of rock to "explode" from a strained surface.
Strain burst	A commonly used term denoting sudden rock failure at the lower end of the magnitude spectrum of violent events. The failure is associated with the surface of the excavation and involves only a few hundred kilograms of rock.
Stress drop	The decrease in shear stress acting on that part of a fault that slips during a seismic event.
Stress-induced fracture	Generally clean, "cleavage type" discontinuity parallel to stope face with no primary displacement.
Striations	Linear markings in a comminuted surface or recent source region indicating direction or movement.
Surface wave	Wave that propagates along a free surface (e.g. the Earth's surface or perhaps the stope hanging wall or footwall). Types of surface waves are Rayleigh and Love waves.
S-wave	Distortional, equi-voluminar, secondary or shear wave. Particle motion in a plane normal to direction of wave propagation.
Talking	The noise associated with small failures of rock occurring beyond the exposed surface and not causing any fall of rock fragments.
Time history	The plotting of seismic parameters vs. time; used to detect trends in seismic activity.
Wave velocity (phase velocity)	The speed with which an individual wave of a certain frequency advances.

MATHS LIBRARY

NUMBERS

- Real numbers are the whole set of numbers one requires to describe the real world (complex numbers are ignored here). They include natural numbers, integers, fractions, square roots and zero. Their geometric representation is a 1-dimenisonal co-ordinate axis stretching from negative infinity (-∞) through 0 to positive infinity (+∞) that is fully occupied by numbers without any gaps.
- Natural numbers are 0,1,2,3,4.....
- Integers are ...-4,-3,-2,-1,0,1 2 3 4...
- Rational numbers are the result of dividing integers: -5/3, 0/12, 3/4 etc.

back

back

- Irrational numbers are those that are not included in the above and cannot be created by dividing integers, such as certain square roots ($\sqrt{2}$), Euler's number e, π etc.
- Negative numbers are those <0, positive numbers are >0.

EXPONENTS AND ROOTS

- 5^3 means 5x5x5=125 (5 multiplied 3 times with itself).
- The superscript 3 is called exponent, 5 is called the base. In general, a^n is read: 'a to the power of n'.
- There are two special cases to keep in mind: $a^0 = 1$ and $a^1 = a$. Example: $100^0=1$, $(-15)^1=-15$
- Multiplying two numbers with the same base: $(a^n)(a^m)= a^{n+m}$ Example: $(5^3)(5^5) = (5x5x5)(5x5x5x5x5)=5^8$
- Multiplying two numbers with the same exponent: $(a^n)(b^n)= (ab)^n$ Example: $(2^2)(4^2) = (2x2)(4x4)=(2x4)(2x4)=8^2$
- Dividing two numbers with the same base: $a^n/a^m=a^{n-m}$
- Dividing two numbers with the same exponent: $a^n/b^n = (a/b)^n$
- A root is a fractional exponent, for example $5^{1/2}$ is the square root of 5, so that multiplying $5^{1/2}$ with itself yields 5. The square root of any number b must be squared (multiplied with itself) to yield the number: $(b^{1/2})^2 = b$. The cubic root of b, multiplied 3 times with itself, yields b. Note the alternative notations: $b^{1/2}=\sqrt{b}$.

SCIENTIFIC NOTATION

- In scientific notation a quantity is written as a number between 1 and 10 multiplied by the appropriate power of 10, e.g. $1.5x10^8$km meaning 150 000 000km, which happens to be close to the average distance between Earth and the sun. The number is followed by the appropriate unit.
- A change in unit affects the power of 10, e.g. $1.5x10^8$km = $1.5x10^{11}$m.
- To write in scientific notation, the number of places the decimal has to be moved to get a number between 1 and 10 is equal to the power of ten: The decimal following 4 200 must be moved 3 places to the left to get 4.2, thus 4 200 = $4.2x10^3$.
- If the decimal is moved to the right, the exponent becomes negative. Example: 0,000 000 5m/s. To express this number in standard notation,
- the decimal is moved seven times to the right: $5x10^{-7}$m/s. This, by the way, is roughly the background noise levels on most geophone sites in mine seismic network.
- For the correct metric pre-fixes associated with various exponents of 10 see prefixes above.

ABSOLUTE VALUES

- In mathematics, the absolute value |a| of a number 'a' is the numerical value of 'a' without regard to its sign. So, for example, the absolute value of 3 is 3, and the absolute value of -3 is also 3. The absolute value of a number may be thought of as its distance from zero.

QUADRATIC FORMULA

- Many equations in physics contain quantities that are squared, e.g. E=mc2 , E=½mv2, or y=x2+7x-2. Solving such equations for the case of y=0 is not possible simply by taking the square root and solving for x (try it!).

back

back

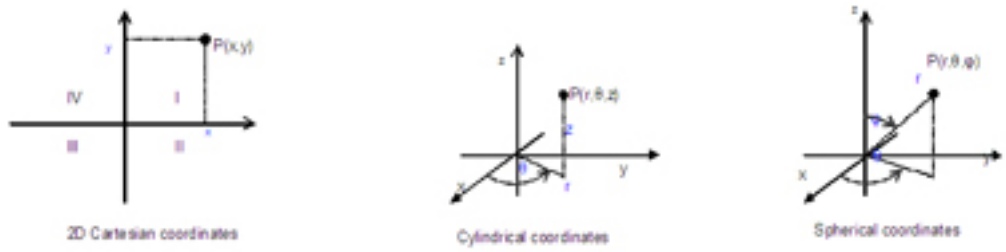
- The quadratic formula offers the general solution for problems of the form $ax^2+bx+c=0$ where a, b and c are numbers and x is the unknown: $x_{1,2}=(-b\pm(\sqrt{b^2-4ac}))/2a$. The two solutions x_1 and x_2 are obtained by adding and subtraction the square root term, respectively.
- In the above example, a=1, b=7 and c=-2. The two solutions to the problem $x^2+7x-2=0$ are: $x_{1,2}=(-7\pm(\sqrt{49+8}))/2 =-7\pm(7.55/2)$; thus, $x_1=0.275$ and $x_2=-7.275$

GEOMETRIC FORMULAS

- Circle of radius r: Circumference=$2\pi r$; area= πr^2.
- Sphere of radius r: Surface area=$4\pi r^2$; volume=$\frac{3}{4}\pi r^3$.
- Cylinder of radius r and heighth: Surface area = $2\pi r^2+2\pi h$; volume = $\pi r^2 h$.
- Triangle with base b and altitude h:area=$\frac{1}{2}bh$.
- Pythagorean theorem (right angled triangle with sides a, b and c): $a^2+ b^2= c^2$.
- In any triangle with angles α, β and γ: $\alpha+\beta+\gamma=180°$.

Co-ordinate Systems

The commonly used **Cartesian co-ordinate** system is formed by two perpendicular axes in a plane, one horizontal and one vertical, which intersect at their origins. Each axis carries a scale of real numbers so that any point in the plane can be uniquely identified by a pair of numbers, the point's coordinates (P(x,y)). The coordinate axes divide the plane into four quadrants I to IV. The arrows on the coordinate axes show the direction in which the numbers increase.



Cartesian coordinate systems are suitable for planar or, if a third orthogonal axis is added, 3-dimensional mathematical problems that involve rectangular shaped objects and straight line trajectories. In 3D, the position of point P would be fully described by P(x,y,z).

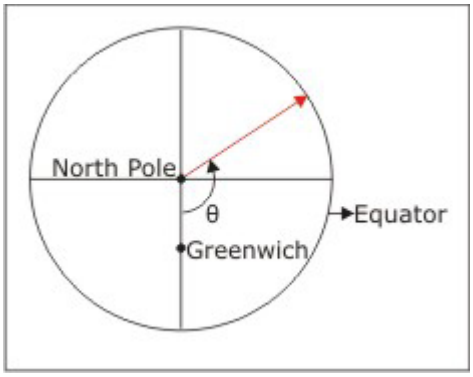
Polar co-ordinates or cylindrical co-ordinate systems are more suitable. for problems involving spherical or cylinder shaped objects and curved paths. The geographical coordinates describing locations on the Earth's surface are an example of such spherical coordinates:
Longitude is measured in degrees east or west from the Prime Meridian defined by the location of the Royal Observatory in Greenwich, East of London, UK. In terms of the sketch above θ ranges between -180 and +180 degrees. Each of the 360 degrees is divided into 60 minutes, each minute into 60 seconds.

Latitude is measured in degrees North or South of the equator, the plane that stands normal on the Earth's axis of rotation and intersects Earth's centre of gravity.

back

back

The Greek letter Φ is used to denote latitude, so that Φ=(90°- φ) in the above sketch of spherical co-ordinates, which is the angle between the x-y plane and the line with length r. Thus, latitude Φ is in the range of -90° to +90°.

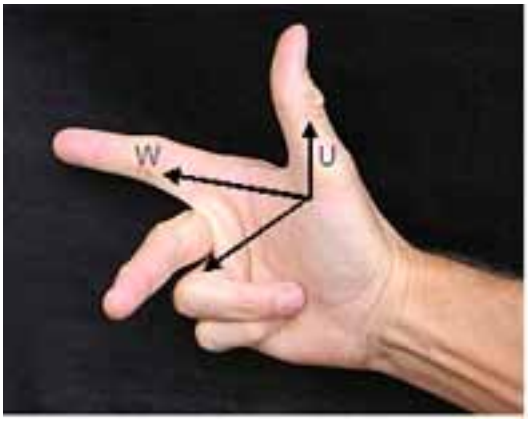


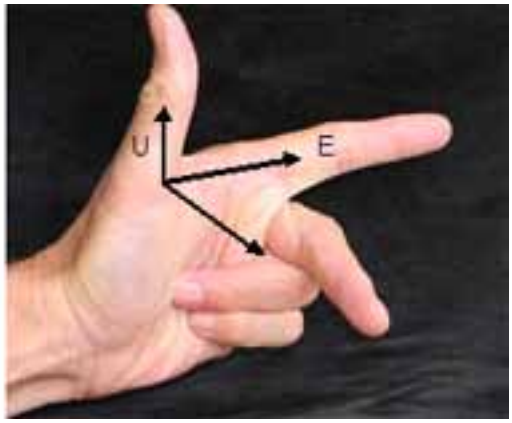
Looking down onto Earth from above the North pole: The definition of longitude θ.

In many cases, mine coordinate systems are a special case of a 3D Cartesian coordinate system whose origin is defined by a geographical location. Their axes refer to geographical directions such as South/West/Up (SWU), but the definition of South=0, West=0 and Up=0 is often specific for a single mine or group of neighbouring mines.

SWU refers to x increasing South, y increasing West and z increasing Up. Some mines choose mean sea level for zero elevation, others a historic datum line 1818.297 above mean sea level.

South/West/Up is a left-handed system while Up/West/South is a right-handed system, depending on whether the first three fingers of the right or left hand can open up the coordinate space: If the thumb, first digit and middle finger of the right hand – in that specific order - are able to align with the coordinate axes the system is called right-handed, as is the case with Up/West/South. Else, it is called left-handed.





Right-handed system,
(for example Up/West/South)

Left-handed system,
(for example Up/East/South)

Examples: Are NWU, ESD and SED left- or right-handed?

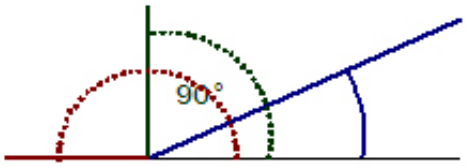
Many South African mines use survey co-ordinates based on the LO27 system, but with certain modifications. The origin of the LO27 system is at the intersection of 27° longitude with the equator; x increases to the South and y increases to the West. The problem of very large x-values – Johannesburg is approximately 2 900km below the equator – is resolved by dropping the first two digits from the x-coordinates bringing x-values for West Wits mines to around 20 000m and y-values to around -50 000m.

back

back

ANGLE MEASUREMENT

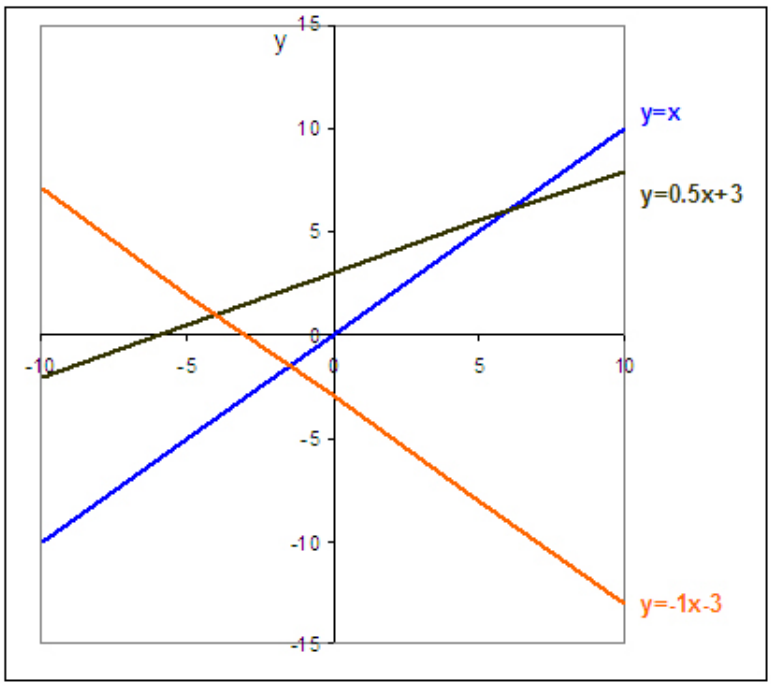
An angle Φ as shown in the sketch below is a measure of how much two lines diverge (how the distance between them grows). The greater the angle, the greater the divergence of the lines. Angles are measured in degrees (°) or in radians (rad). There are 360° or 2π rad in a full circle.
Two straight lines are defined to be parallel if the angle between them is 0°, i.e. they have the same direction. Two lines are defined to be perpendicular if the angle between them is 90°. Another term for perpendicular is 'normal'.



360° degrees is equivalent to 2π rad (full circle), 180° to π rad and 90° to π/2 rad.
The equivalent numerical values using π=3.142 are 360°=6.283rad, 180°=3.142rad and 90°=1.571rad.

LINEAR FUNCTIONS

- Linear functions are of the general form f(x)=y=mx+b with constants m and b. The graph of a linear function is a straight line with slope m and offset b.
- F(x) means any function that has x as the independent and f(x) as the dependant variable. The common placeholder for the dependant variable is y.
- The sketch below shows the graphs of three straight line functions, each having a different slope and offset.



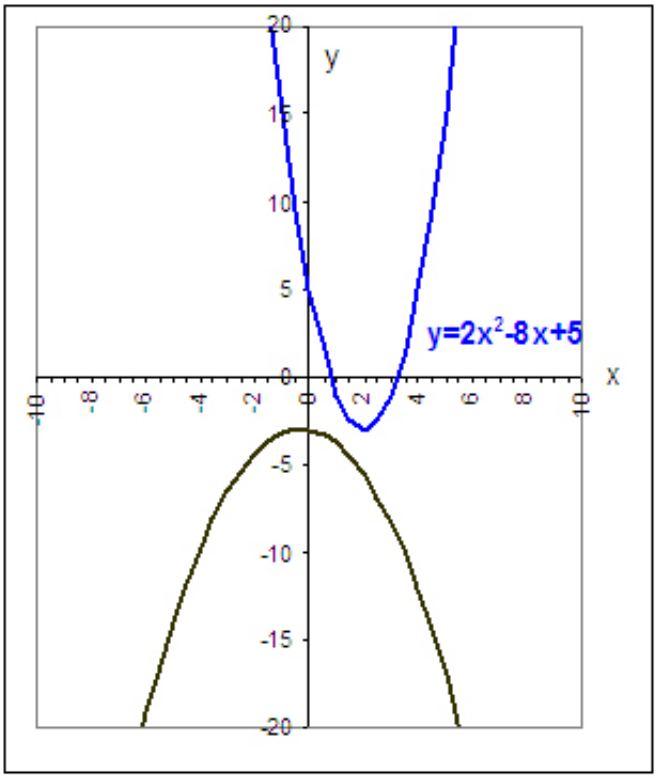
y=x: slope 1, offset 0
y=0.5x+3: slope 0.5, offset 3
y=-1x-3: slope -1, offset -3.

back

Example:
Assume monthly telephone costs on a landline to consist of line rental and the costs of calls. If calls are charged at R2.50 each and the monthly line rental is R220,
the phone bill Bph amounts to Bph=2.5c+220 where c is the number of calls. Here, the dependant variable (costs) is Bph and c (number of calls) is the independent variable.

Exercise: Try to mentally picture the graphs of x=4, y=-5 and y=-x.

QUADRATIC FUNCTION
Quadratic functions are of the general form $f(x)=ax^2+bx+c$ where a, b and c are constants and x is the independent variable. The graph of a quadratic function is a parabola that is symmetrical to the vertical axis. The parabola opens upward if a>0 and downward if a<0. If a=0 the quadratic function is not quadratic but linear (see above).



y=2x2-8x+5:
narrow, open upwards, offset is 5

y=-0.5x2-3x-3:
less narrow and steep, open downwards,
offset is -3.

Exponential Functions

First to be noted is the difference between $f(x)=x^2$ and $g(x)=2^x$. The former was introduced above as a quadratic function. The latter, with base 2 and the independent variable x as the exponent, is called an exponential function. Their general form is b^x .

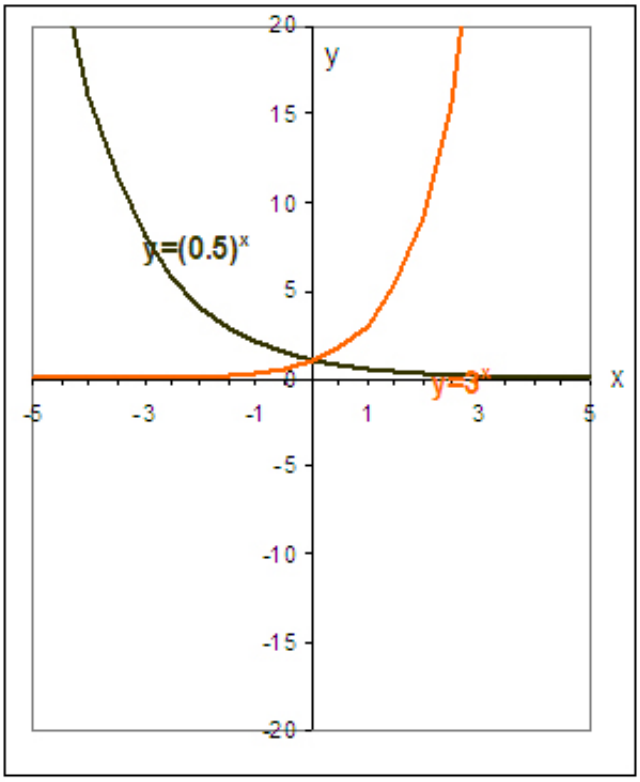
Exponential functions do not behave as easily predictable as linear and quadratic functions.

For once, there is a significant difference between b<0, 0<b<1 and b>1 due to the way exponents are defined. Let b=-2: $-2^{-x}=1/(2^{-x})=-1/2$ which is defined for x=-2, but $x=\sqrt{2}$ is an irrational number that can be approximated at best. As a result, $y=b\sqrt{2}$ and similar exponents are problematic. Exponential functions based on b<0 are not continuous and their graphs

back

are not smooth lines. The case of b=1 is trivial as $1^{-5}=1^{0}=1^{407}=1^{r}=1$ where r can be any real number.

For most practical applications, exponential functions can be treated like others provided b>0 and b≠1:



$y=3^x$ approaches the x-axis for x<0, but never reaches zero;
$y=(0.5)^x$ approaches the x-axis for x>0 but never reaches zero.

All graphs of exponential functions...

- pass through the point (0,1),
- are continuous (no holes or jumps) and
- have the x-axis as a horizontal asymptote (it is approached but not touched by the graph).

Examples of natural processes that can be approximated with exponential functions are population growth, radioactive decay and compound interest.

A special base: Euler's number e.
Likely the most used mathematical constant is π, the proportionality constant relating the radius of a circle to its circumference. For exponential and logarithmic functions, e (also an irrational number) plays an important role. It is derived from the expression $(1+1/m)^m$, which approaches the value 2.718281... for increasing values of m:

m	1	10	100	1 000	10 000	100 000
$(1+1/m)^m$	2	2.59374..	2.70481..	2.71692..	2.71814..	2.71827..

Thus, e is the limit of the expression $(1+1/m)^m$ for m growing to infinity.

One can now define the exponential function with base e: $f(x)=e^x$ whose graph is similar to that of $y=3^x$ in the sketch above (orange line).

The following simplifications apply to all exponential function, including those with base e:
$b^x b^y=b^{x+y}$; $(b^x)^y=b^{xy}$; $b^x/b^y=b^{x-y}$

back

back

LOGARITHMIC FUNCTION

Logarithmic functions are the inverse to exponential functions. Considering the two expressions $y=2^x$ and $x=2^y$ the symmetry is obvious. For example, the x-y value pair (3,8) satisfies the first equation while (8,3) satisfies the second. The problem is with the second expression, which is not in the familiar form of y=... . For a solution, function $y=\log_2 x$ is introduced with argument x and base 2. This function is valid for the case that $x=2^y$.

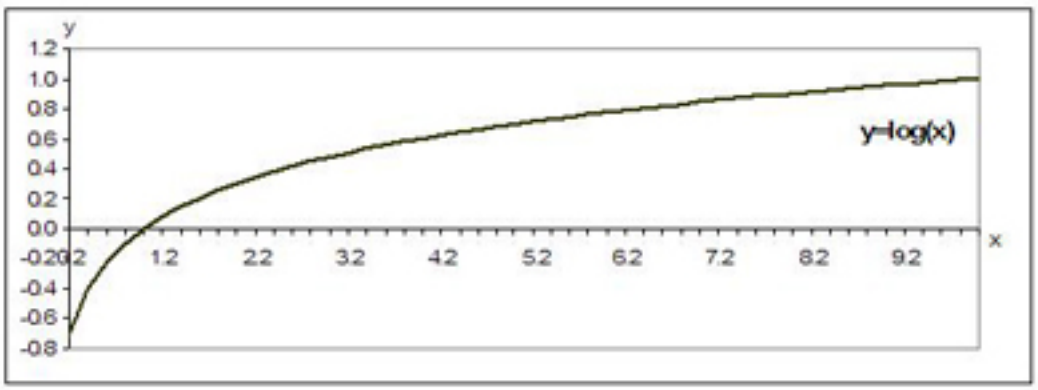
The definition of logarithmic functions is valid for all $b>0$ and $b\neq 1$ and is linked to their respective bases:

- $y=\log_b x$ is equivalent to $x=b^y$
- $y=\log_{10} x$ is equivalent to $x=10^y$
- $y=\log_e x$ is equivalent to $x=e^y$

In words, the logarithm to the base b is the exponent to which b must be raised to obtain x. In other words, a logarithm is an exponent. Following examples should illustrate the principle:

- $y=\log_{10}(100)=2$ since $10^2=100$,
- $y=\log_5(25)=2$ since $5^2=25$,
- $y=\log_4(2)=½$ since $4^{½}=\sqrt{4}=2$
- $y=\log_e(1)=0$ since $e^0=1$.

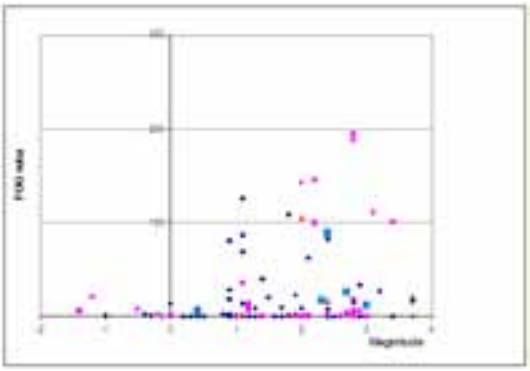
The most common base is 10 so that, as a general convention, $y=\log_{(x)}$ automatically assumes base 10. In cases where the base is different from 10, it is specifically noted as in $\log_5(x)$.
The graph of $f(x)=\log_{(x)}$ is shown below for $0<x<10$.



From the definition of logarithmic functions it is clear that $y=\log_{(-5)}$ has no solution. There exists no exponent of 10 so that $10^y=-5$: $\log_{(x)}$ is not defined for $x\leq 0$.

Because of the property of $\log_{(x)}$ to associate small y with large x as in $y=\log_{(1000)}=3$ and $y=\log_{(1000000)}=6$, graphing software such as EXCEL offer a logarithmic scale to plot large numbers, in effect compressing the scale. While on a linear scale the distance from the origin doubles for twice as large a value (for example from 4 to 8), on a logarithmic scale the distance doubles if the exponent doubles (for example from 100 to 10000). The effect is a shrinking of the scale that allows very large values to be graphed together with small values.

back



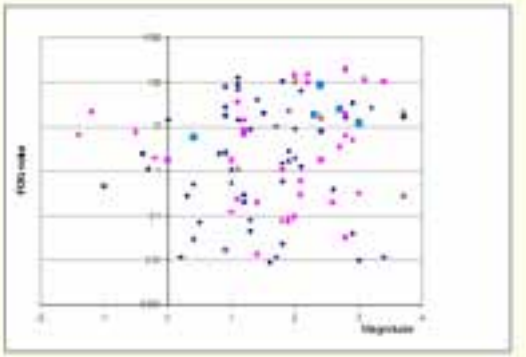


Figure 1: Rockburst magnitude plotted against FOG volume: linear scale
Figure 2: Rockburst magnitude plotted against FOG volume: logarithmic scale

When analysing the graphs, the detail that can be obtained from each is different: On the linear scale graph the largest FOGs seem associated with the largest magnitude in the pink data set, while the logarithmic scale allows a distinction between blue and pink data set in the magnitude range 0-2. Thus, one method of scaling is not superior to the other, rather they offer different benefits.

The following properties of logarithmic functions can sometimes be useful when dealing with combined functions (see exponents above):

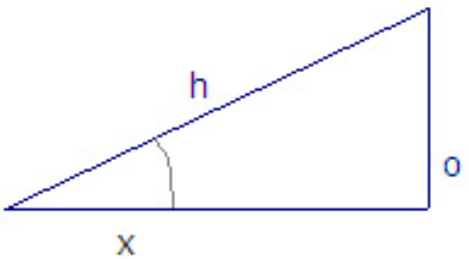
- $\log_{(mn)}=\log_{(m)}+\log_{(n)}$
- $\log_{(m/n)}=\log_{(m)}-\log_{(n)}$
- $\log_{(m)}p=p*\log_{(m)}$
- $\log_{(1)}=0$

While, by convention, $y=\log_{(x)}$ assumes a base of 10, $y=_{\ln}(x)$ assumes a base of e. ln(x) is also called the natural logarithm.

Quadrants and trigonometry signs in quadrants

The most commonly used trigonometric functions are sine and cosine: sin(x) and cos(x) respectively. Both of these are periodic meaning that they are fully described for values of x between 0° and 360° (or 0 and 2π rad), and repeat themselves for x values outside of this range. For the simplified case of amplitude and angular velocity being equal to 1, and the initial phase being zero, the two functions take on the form f(x)=sin(x) and f(x)=cos(x).

The properties of sin(x) and cos(x) originate from their geometric definitions, which are based on a triangle that includes a right angle of 90°. This definition sets sin(x) as the length of the opposite side o divided by the hypotenuse h of the triangle. Cos(x) is defined as the side adjacent side a divided by h.

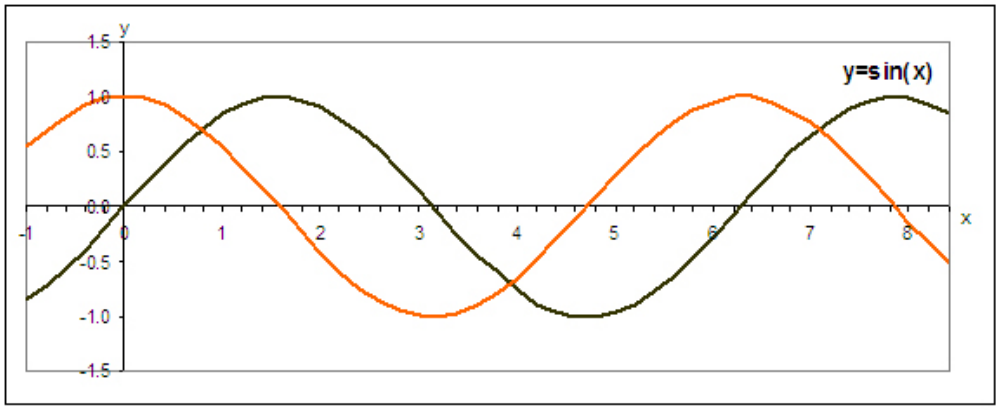


sin x = o/h

cos x = a/h

Example: Consider a car travelling up an inclined road (h) for 2km. The angle to the horizontal x of the incline is 16°. By how much has the car's elevation o increased? Correct answer: 551m

For growing angles x, sin(x) will increase up to 90°, then decline to zero when x reaches 180°, while cos(x) is initially equal to 1, then declines for growing x until reaching zero for x=90°. This is
the shape and behaviour of the two functions in the graph below. The graphs of sine and cosine are periodic:



y=sin(x) is periodic, equals 0 when x=0 and maximum/minimum at odd multiples of π/2;
y=cos(x) is periodic, equals 1 when x=0 and maximum/minimum at even multiples of π/2.

The properties of sine and cosine give rise to the following relationships:

$$\sin 2x + \cos 2x = 1$$
$$\sin(\alpha \pm \beta) = \sin(\alpha)\cos(\beta) \pm \cos(\alpha)\sin(\beta)$$
$$\cos(\alpha \pm \beta) = \cos(\alpha)\cos(\beta) \pm \sin(\alpha)\sin(\beta)$$

Cubic Equation

No info available as yet.

Absolute Values

In mathematics, the absolute value |a| of a number 'a' is the numerical value of 'a' without regard to its sign. So, for example, the absolute value of 3 is 3, and the absolute value of -3 is also 3. The absolute value of a number may be thought of as its distance from zero.

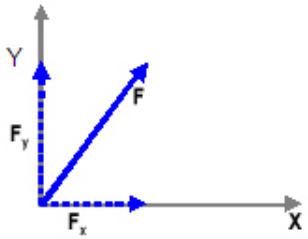
PHYSICS LIBRARY

SCALAR

A physical quantity that is fully described by a single value, such as the temperature in °C, the energy content of a candy bar in J,
the height of a person or the width of this page in metres.

VECTOR

A physical quantity that requires both a magnitude and a direction to be described fully such as displacement, velocity and acceleration. Vectors require a reference frame in form of a co-ordinate system to be properly defined. Vectors can be added, multiplied and manipulated in a variety of ways. In rock engineering, vectors are commonly used to describe forces acting on a support unit or one-directional stresses at a point in the rock mass.



$F=[F_x, F_y]$
Length (magnitude) of vector F:
$|F|=\sqrt{F_x^2+F_y^2}$

Vectors are used to describe physical quantities where the problem that needs to be solved requires this or where the answer to a question is obtained easier using vector algebra. In order to calculate the acceleration of an object due to a force F the direction of the force may be irrelevant. In this case, force and acceleration can be described by scalar numbers.

Two vectors $a=[a_x,ay]$ and $b=[b_x,b_y]$ can be multiplied in two ways:

Scalar product

$a \bullet b=|a||b|\cos \varphi$

where φ is the angle between a and b.

Vector product

$a \times b = c$

where c is a vector of length $|a||b|\sin\varphi$

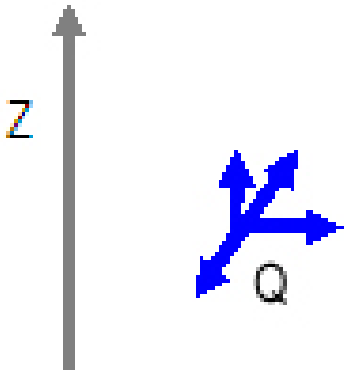
with an orientation normal to the plane defined by a and b.

TENSOR

A physical quantity that has more than one directed component at a given point requires a higher-order tensor for its full description.

To fully describe the stress field in a point in the rock mass requires quantification of all shear and compression/tension components.
In a 2-dimensional co-ordinate space the stress tensor has 4 components, in a 3-dimensional space 9 components:

$$\sigma = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{yx} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{zx} & \sigma_{zy} & \sigma_{zz} \end{bmatrix}$$



Stress and Strain

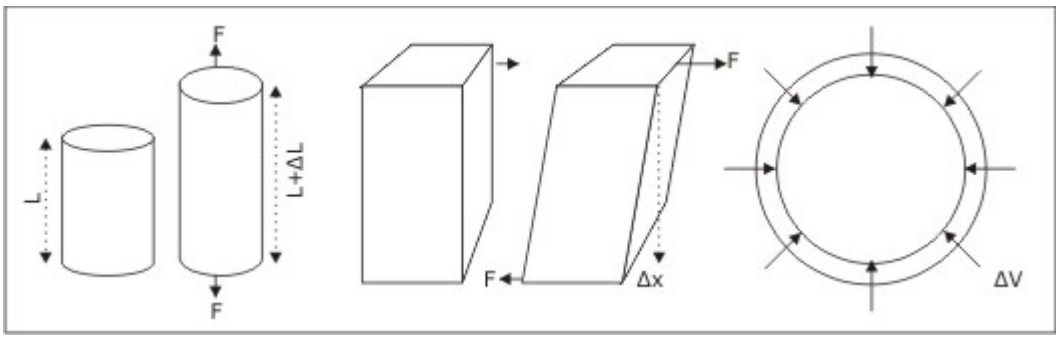


Figure A Figure B Figure C

Three simplified ways in which forces act on areas, each resulting in stress. In all cases is the body deformed as a result of the stress.

Figure A: A cylinder subjected to tensile stress stretches by an amount ΔL, its diameter is reduced; Figure B: A brick-shaped body subjected to shearing stress is deformed by an amount Δx; Figure C: A sphere is subjected to hydraulic stress, shrinking in volume as a result. For demonstration purposes
the deformation in the sketches is greatly exaggerated.

Stress is defined as force per unit area, the resulting strain is defined as deformation per unit length. For many solid materials stress is proportional to
strain, the proportionality factor is called the modulus of elasticity (Hooke's law):

stress = modulus * strain

Tension and compression:
The modulus for compressive and tensile stresses is the Young's Modulus E. The stress is determined by the force F acting
perpendicular to the area A (end area of the cylinder in Figure A):

$$F/A = E*\Delta L/L$$

The strain ΔL/L is a dimensionless quantity and is often in the range of <<1%. E has the same unit as stress: [N/m^2].

Shear:
In the case of shearing the force vector acts parallel to the surface of the object. The corresponding modulus is the shear modulus G, also called or rigidity:

$$F/A = G*\Delta x/L$$

The strain $\Delta x/L$ is dimensionless. G has the same unit as stress: [N/m^2].

Hydraulic stress:
The object in Figure C is subject to hydraulic compression. Its volume decreases by ΔV due to the pressure P, a force per unit area.
The strain is defined as the relative volume change $\Delta V/V$. The relevant modulus is the bulk modulus B of the material:

$$P = B*\Delta V/V$$

In deep level rock engineering applications the volume V consists of rock and the pressure P is due to the weight of the overburden. P is determined by the depth and the density of the overlying rock mass, usually between 2 000 and 3 000kg/m3 for most rock types in South African mines.

Newton's laws of motion

The first of Newton's laws of motion states that a body will remain at rest if at rest or continue at constant velocity if already in motion unless it is acted on by an external force. This law states that a force (the cause) will lead to a change in a body's state of motion (result).
Example: *A donkey cart only starts to move once the donkeys start pulling, otherwise it stays at rest.*

Newton's second law of motion quantifies this relationship between cause and result: A body of mass m experiences the acceleration a (change in velocity) when experiencing a force F. The acceleration experienced by the body is proportional to the force F and inversely proportional to its mass m:
Example: *A small force F acting on a heavy object (read: large mass m) results in minimal acceleration.*

Newton's third law of motion states that whenever one body exerts a force on a second body, it experiences a force that is equal in magnitude and opposite in direction to the one it exerts.
Example: *The direction of a gun recoil when firing a bullet is opposite to the direction of the bullet. The same principle is used to propel jet planes and rockets, which travel in a direction opposite to the burnt fuel expelled by their engines.*

Energy

kinetic energy:
Energy is a fundamental concept in physics. Energy takes on many different forms and is part of almost all physical process. On a basic level, it can be defined as 'stored work', meaning energy can be converted to work: The kinetic (motion related) energy in flowing water can drive a saw mill; on impact, a moving hammer can drive a nail into wood.

In a closed process, energy can only be converted from one form into another: Energy is conserved and can neither be gained nor lost. One of the most common forms of energy that is frequently converted into other forms is kinetic energy, E_{kin}, which is proportional to the mass m of the moving body:

$$E_{kin}=1/2mv^2$$

where v is the velocity of the body.
The unit of energy is Joule [J=Nm]

back

Because $E_{kin} \infty v^2$, a car travelling at 100km/h has four times as much kinetic energy as a car travelling at 50km/h. This means, it will cause four times as much damage on impact than the slower car!

In rock mechanics, E_{kin} is of interest when choosing suitable support types for areas which experience falls of ground. The support unit must be able to absorb the kinetic energy of ejected rock and convert this energy into other harmless forms of energy, such as deformation and heat.

Potential (gravitational) energy:
In falling objects, potential energy is converted to kinetic energy. The potential energy E_{pot} is due to gravity: An object at higher altitude (further away from the Earth's centre) has a greater potential energy due to gravity than an object of equal mass at lower altitude. Potential energy is proportional to mass, height and gravitational acceleration:

$$E_{pot}=mgh$$

where h is the difference in height.
The unit of energy is Joule [J=Nm].

Strictly speaking and in most cases, E_{pot} means a change in potential energy due to a change in height rather than an absolute energy value. For a pumped storage scheme to generate electricity, it is only relevant how far up the water is pumped between a lower and a higher altitude dam, than at which altitude the scheme operates. The amount of electricity generated by the scheme depends on the height difference between the two storage dams.

Conservation of energy
The conservation of energy law states that the total energy in any process is constant:

Energy may change in form or be transferred from one part of a process to another, but the sum of all energy forms stays the same.

Frequently converted forms of energy common to processes in a mining environment are chemical (explosives), thermal (ventilation), strain (deformation in rocks and support units), electric (pumps, drills, fans) and gravitational (hoist).

back