CERTIFICATE IN ROCK MECHANICS

LEARNING GUIDE FOR:
PART 3 : BASIC ROCK MECHANICS PRACTICE



SOUTH AFRICAN
NATIONAL INSTITUTE OF
ROCK ENGINEERING



CHAMBER OF MINES
OF SOUTH AFRICA



MINING
QUALIFICATIONS
AUTHORITY

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CHAPTER 1

PAPER 3.1 HARD ROCK TABULAR MINING

PAPER 3 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 specific mining type paper covering rock mechanics practice applicable intabular, hard rock mining environments at all depths. The rock engineering knowledge required here is thus of a specific nature, relating to the mining of tabular orebodies in hard rock at shallow, moderate and great depth.

2. CRITICAL OUTCOMES

The examination is aimed at testing the candidate's abilities in the six cognitive levels of 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.



This syllabus is available in PDF format on the SANIRE webpage

1. GEOTECHNICAL CHARACTERISTICS

1.1. GEOLOGY

1.1.1. GEOLOGICAL SEQUENCES

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

- Identify and describe the rock types associated with the main South African tabular, hard rock orebodies
- Describe, explain and discuss how the rock types associated with tabular, hard rock orebodies were formed
- Sketch, describe and discuss the geological sequences associated with tabular, hard rock orebodies.

1.1.2. GEOLOGICAL STRUCTURES

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

- Sketch, describe and discuss major geological structures associated with the main South African tabular, hard rock orebodies
- Describe and discuss the following aspects of major geological structures associated with the main South African tabular, hard rock orebodies :
  - Origin, Genesis, Mechanism of formation
  - Residual effects of formation on geotechnical behaviour
  - Subsequent effects of formation on geotechnical behaviour.

1.1.3. ROCK STRENGTH

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

- Discuss the relative strengths of rock types associated with tabular, hard rock orebodies
- Describe and discuss the geotechnical characteristics of rock types associated with tabular, hard rock orebodies
- Apply the above knowledge to the design of total extraction workings
- Apply the above knowledge to the design of partial extraction workings.

1.2. ROCKMASS CHARACTERISTICS

1.2.1. GEOTECHNICAL ROCKMASS CLASSIFICATION

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

- Describe, discuss and apply rockmass classification techniques for the seclection of hard rock tabular mining methods

- Describe, discuss and apply standard rockmass classification and assessment systems to predict excavation stability and support requirements
- Describe, discuss and apply the rockwall condition factor (RCF) to predict tunnel stability and support requirements.
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2. ROCK AND ROCKMASS BEHAVIOUR

2.1. ROCKMASS BEHAVIOUR AROUND TABULAR EXCAVATIONS

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

- Sketch, describe, explain and discuss the behaviour of the rockmass around tabular excavations at shallow, intermediate and great depth
- Describe, explain and discuss the changing amount and nature of rock that needs to be supported at shallow, intermediate and great depth
- Describe, explain and discuss the different rates of deformation of the rockmass at shallow, intermediate and great depth
- Sketch, describe, explain and discuss the types of fractures found around stope faces at shallow, intermediate and great depth
- Describe, explain and discuss the effects of stope face fracturing on the stability of the stope hangingwall, stope footwall and stope face
- Sketch, describe, explain and discuss the types of fractures found around stope gullies at shallow, intermediate and great depth
- Describe the effects of stope gully fracturing on the stability of the gully hangingwall and sidewalls
- Sketch, describe, explain and discuss the behaviour of rock surrounding tabular excavations during seismic events
- Describe, explain and discuss typical heights of rockfalls in tabular underground gold and platinum mines in South Africa
- Describe, explain and discuss the significance of rockfall height in terms of support requirements in gravity loading conditions
- Describe, explain and discuss the significance of rockfall height in terms of support requirements in seismic loading conditions
- Evaluate given rockmass conditions and excavation dimensions to determine or predict stope stability
- Evaluate given rockmass conditions and excavation dimensions to determine remedial measures to improve stope stability
- Describe, explain, discuss and apply the following techniques to the above evaluation :
  - Rockmass class, Stress analysis, Continuum mechanics,
  - Discrete block kinematics, Beam theory, Failure criteria
- Sketch, describe, explain and discuss rockmass behaviour and conditions associated with stoping abutments at depth
- Sketch, describe, explain and discuss the on-reef effects associated with stoping abutments at depth
- Sketch, describe, explain and discuss the off-reef effects associated with stoping abutments at depth
- Sketch, describe, explain and discuss rockmass behaviour and conditions associated with stope pillars at shallow, intermediate and great depth
- Sketch, describe, explain and discuss the on-reef effects associated with stope pillars and/or remnants at shallow, intermediate and great depth

- Sketch, describe, explain and discuss the off-reef effects associated with stope pillars and/or remnants at shallow, intermediate or great depth
- Sketch, describe, explain and discuss the purpose, procedure and effects of rockmass pre-conditioning ahead of stopes at depth
- Sketch, describe, explain and discuss the various methods of pre-conditioning
- Describe, explain and discuss the critical features of pre-conditioning.
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2.2. ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS

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

- Describe, explain and discuss how rocks and rockmasses may be classified
- Describe, explain and discuss how rockmass classification systems may be used to predict tunnel stability at shallow, intermediate and great depths
- Describe, explain and discuss the nature of fracturing around tunnels at depth
- Describe, explain and discuss the effects and consequences of fracturing on tunnel size
- Describe, explain and discuss the effects and consequences of fracturing on tunnel shape
- Describe, explain and discuss the effects and consequences of fracturing on tunnel stability
- Describe, explain and discuss the phenomenon of time-dependant fracturing and deterioration
- Describe, explain and discuss the effects and consequences of time-dependant fracturing and deterioration on tunnel stability
- Describe, explain and discuss the effects of stress regime and geological structure on rock behaviour and tunnel stability
- Describe, explain and discuss the effect of tunnel size, tunnel shape and tunnel excavation technique on rock behaviour and tunnel stability
- Describe, explain and discuss the optimisation of tunnel size, shape and orientation to suit geological considerations
- Describe, explain and discuss the optimisation of tunnel size, shape and orientation to suit field stress considerations
- Describe, explain and discuss the behaviour of rock surrounding tunnels at depth during seismic events
- Describe, explain and discuss the siting of tunnels with respect to existing, current and future stoping operations
- Describe, explain and discuss the siting of tunnels with respect to geological stratigraphy and structures
- Describe how rockwall condition factor (RCF) is determined
- Describe and discuss how the rockwall condition factor may be applied to predict tunnel stability and support requirements
- Apply the rockwall condition factor to predict tunnel stability and support requirements at depth
- Evaluate given rock conditions and field stresses to predict excavation stability

- Evaluate given rock conditions and field stresses to recommend remedial measures to improve excavation stability.

2.3. ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS

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

- Describe, explain and discuss how rocks and rockmasses may be classified
- Describe, explain and discuss how rockmass classification systems may be used to predict service excavation stability at shallow, intermediate and great depths
- Describe, explain and discuss the nature of fracturing around vertical sinking shafts at depth
- Describe, explain and discuss the nature of fracturing around inclined sinking shafts at depth
- Describe, explain and discuss the nature of fracturing around rock passes at depth
- Describe, explain and discuss the effect of stress fracturing and geological structure on the final shape of vertical and inclined shafts
- Describe, explain and discuss the effect of stress fracturing and geological structure on the final shape of rock passes
- Describe, explain and discuss the conditions that may lead to the formation of large unstable rock wedges in vertical shaft sidewalls
- Describe, explain and discuss the conditions that may lead to the formation of instabilities in rock passes
- Describe, explain and discuss the nature of fracturing around service excavations at depth
- Describe, explain and discuss the effects and consequences of fracturing on service excavation size
- Describe, explain and discuss the effects and consequences of fracturing on service excavation shape
- Describe, explain and discuss the effects and consequences of fracturing on service excavation stability
- Describe, explain and discuss the phenomenon of time-dependant fracturing and deterioration
- Describe, explain and discuss the effects and consequences of time-dependant fracturing and deterioration on service excavation stability
- •Describe, explain and discuss the effects of stress regime and geological structure on excavation stability
- Describe, explain and discuss the effects of excavation size, shape and orientation on excavation stability
- Describe, explain and discuss the effects of excavation technique on fracturing and the stability of service excavations
- Describe, explain and discuss the effects of excavation sequence on stress fracturing and the stability of service excavations at depth
- Describe, explain and discuss the effects of support sequence on stress fracturing and the stability of service excavations at depth
- Describe, explain and discuss the optimisation of service excavation size, shape and orientation to suit geological considerations
- Describe, explain and discuss the optimisation of service excavation size, shape and orientation to suit field stress considerations

- Describe, explain and discuss the behaviour of rock surrounding service excavations at depth during seismic events
- Describe, explain and discuss the siting of service excavations with respect to existing, current and future stoping operations
- Describe, explain and discuss the siting of service excavations with respect to geological stratigraphy and structures
- Describe, discuss and explain how rockmass characterisation may be applied to predict service excavation stability and support requirements
- Describe, discuss and explain how rockmass characterisation may be applied to predict service excavation stability and support requirements
- Apply rockmass characterisation techniques to predict service excavation stability and support requirements
- Evaluate given rock conditions, field stresses, geological conditions, excavation sizes, shapes and orientations to predict excavation stability
- Evaluate given rock conditions, field stresses, geological conditions, excavation sizes, shapes and orientations to recommend remedial measures to improve excavation stability
- Evaluate given rock conditions, field stresses and geological conditions to predict shaft stability
- Evaluate given rock conditions, field stresses and geological conditions to recommend remedial measures to improve stability.
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3. MINE SEISMICITY

3.1. PRINCIPLES OF SEISMICITY

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

- Define, describe, explain and discuss the term seismic event
- Describe, explain and discuss the phenomenon of foreshocks and aftershocks
- Describe, explain and discuss the processes that give rise to violent failure and rupture in rock
- Describe, explain and discuss the rupture process in solid rock
- Describe, explain and discuss the rupture process on a pre-existing fault plane
- Differentiate between the terms seismic event and rockburst
- Define, describe, explain and discuss the term rockburst
- Identify, describe, explain and discuss the conditions required for a rockburst to occur
- Describe, explain and discuss the main mechanisms responsible for seismic events in South African mines
- Describe, explain and discuss the main mechanisms responsible for rockbursts in South African mines
- Define, describe and explain the phenomenon of seismic body waves
- Describe, explain and distinguish between different types of seismic body waves
- Define, 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

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- Describe, explain and discuss the differences between seismic body waves and seismic surface waves
- Describe, explain and discuss the factors that affect the velocity of propagation of seismic waves in rock
- Describe and explain how the rockmass environment affects the velocity of Pwaves and S-waves
- State typical velocities of P-waves and S-waves in hard rock and soft rock environments
- Describe, explain and discuss how this information may be used to locate the source of seismic events
- Describe, explain and discuss how the energy of a seismic body wave is attenuated as it propagates through elastic rock
- Describe, explain and discuss 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, explain and discuss the phenomena of reflection and refraction of seismic body waves
- Describe, explain and discuss how seismic wave reflection and refraction gives rise to and affects rockburst damage in excavations
- Describe, explain and discuss the objectives of seismic monitoring in mines
- Identify, describe, explain and discuss the features of seismic events that are used to locate the source of seismic events
- 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
- Distinguish between seismic source mechanism and seismic damage mechanism
- Define, describe, explain, discuss and contrast the following primary source parameters in terms of their physical meaning and the data required to determine them :
  - Richter magnitude, Local magnitude, Moment magnitude
  - Seismic energy, Seismic moment, Seismic displacement
- Define, describe, explain, discuss and contrast the following secondary source parameters, in terms of their physical meaning and the data required to determine them :
  - Corner frequency, Source radius, Apparent volume,
  - Stress drop, Energy index
- Describe and explain the common methods of seismic data analysis.

3.2. MINE SEISMOLOGY AND ROCKMASS STABILITY

3.2.1. MINE SEISMOLOGY

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

- Describe, explain and discuss the characteristics of mine seismicity in terms of :
  - Causes
  - Location, Incidence, Magnitude, Time
  - Effects
- Define, describe, explain and discuss the following terms :
  - Rockburst, Strong ground motion, Peak particle velocity

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- List, describe, explain and discuss the factors that determine the nature of seismicity in tabular hard-rock mining
- Describe, explain and discuss the relationship between mine lay-out and seismicity
- Describe, explain and discuss the relationship between depth and seismicity
- Describe, explain and discuss the relationship between rock types and seismicity
- Describe, explain and discuss the relationship between local geology and seismicity
- Describe, explain and discuss the relationship between regional geology and seismicity
- List, describe, explain and discuss the measures that may be applied to control seismicity
- Describe, explain and discuss measures to reduce the maximum magnitude of seismic events
- Describe, explain and discuss measures to reduce the frequency of seismic events
- Define, describe, explain, discuss and contrast the following secondary source parameters in terms of :
  - their physical meaning :
  - the data required to determine them :
    i. Source dimension, Apparent volume
    ii. Static seismic stress drop, Dynamic seismic stress drop
    iii. Energy index, Apparent stress
    iv. Dominant frequency, Corner frequency
- Explain the difference between the following source parameters :
- Richter magnitude, Local magnitude, Moment magnitude
- State, describe and explain the main characteristics of the Brune Model
- Describe and explain how to obtain a fault plane solution from moment tensor analysis
- Describe and explain the following common methods of seismic data analysis :
  - Gutenberg-Richter analysis
  - Magnitude-Frequency relationship
  - Energy-Moment relationship
  - Trend analysis
  - Time-of-day distribution
  - Instability analysis
- Describe typical outputs of the above analyses
- Describe, explain and discuss the relationships between :
  - Extent of stressed rock and Maximum potential seismic event magnitude
  - Volumetric closure and Seismic moment
  - Event magnitude and Damage caused by seismic events
  - Peak particle velocity and Damage caused by seismic events
- Describe, explain and discuss how peak particle velocity decreases with distance from a seismic event source
- Identify, describe and explain the basic steps that are involved in

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carrying out a rockburst site investigation

- Describe and explain the relevant factors to inspect and investigate at a rockburst site
- Describe and explain the relevant seismic source parameters to investigate
- Describe and explain the relevant aspects of seismic history to analyse
- Describe, explain, discuss and contrast the relationship between:
  - Rockburst damage and Seismic source characteristics
  - Rockburst damage and Excavation support characteristics
- Describe, explain and discuss the different types of seismic emission sources in tabular hard-rock mining
- Demonstrate familiarity with the guideline for the compilation of a code of practice to combat rock related accidents by being able to :
  - Define, describe and explain the term 'seismically active' as defined in the guideline
  - Describe and explain the methods of estimating seismic hazard in tabular hard-rock mines
  - Describe and explain the steps necessary to define emission sources in various ground control districts
  - Describe and explain the term 'reasonably practicable' in the context of seismic risk management
  - Describe and explain the various strategies to combat seismicity related risks
  - Describe and explain the various seismic energy emission control measures
  - Describe and explain the various rockburst damage control measures.

3.2.2. ROCKMASS STABILITY

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

- Describe, explain and discuss how the following strategies assist in reducing rockburst hazards:
- Pre-conditioning, Backfilling, Stabilising pillars.

4. MINING LAYOUT STRATEGIES

4.1. HARD ROCK TABULAR MINING METHODS

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

- Sketch, describe, explain and discuss the fundamental mining and rock engineering principles associated with the following tabular, hard rock mining methods :
  - Room and Pillar mining
  - Scattered mining
  - Longwall mining
  - Sequential Grid mining
  - Closely Spaced Dip Pillar mining

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4.2. REGIONAL STABILITY STRATEGIES

4.2.1. PILLARS AS REGIONAL SUPPORT

4.2.1.1 REGIONAL PILLARS

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

- Sketch, describe, explain and discuss the functions of regional stability pillars at shallow to intermediate depth where in-stope pillars are not used as local support
- Sketch, describe, explain and discuss the functions of regional stability pillars at shallow to intermediate depth where in-stope pillars are used as local support
- Sketch, describe, explain and discuss the functions of regional stability pillars at great depth
- List, describe, explain and discuss the various design criteria that may be applied to the design of regional pillars
- Describe, explain and discuss the assumptions associated with these pillar design criteria
- Describe, explain and discuss the limitations associated with these pillar design criteria
- Design regional stability pillars for workings at shallow depths
- Design regional stability pillars for workings at intermediate depths
- Design regional stability pillars for workings at great depths
- Sketch, describe, explain and discuss the functions of stabilising pillars at depth
- Describe, explain and discuss the effect of stabilising pillars on energy release rate at depth
- Describe, explain and discuss the effect of stabilising pillars on excess shear stress at depth
- Apply empirical design criteria to design stabilising pillars for workings at depth
- Calculate the effect of regular stabilising pillars on energy release rates assuming elastic conditions.

4.2.1.2 STABILISING PILLARS

See Regional Pillars

4.2.1.3 BRACKET PILLARS

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

- Sketch, describe, explain and discuss the functions of bracket pillars at depth
- Describe, explain and discuss the effect of bracket pillars on excess shear stress at depth
- List, describe, explain and discuss the various design criteria that may be applied to the design of bracket pillars
- Describe, explain and discuss the assumptions associated with these pillar design criteria
- Describe, explain and discuss the limitations associated with these pillar design criteria
- Design bracket pillars for workings at depth
- Explain the problems associated with designing adequate bracket pillars.

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4.2.2. BACKFILL AS REGIONAL SUPPORT

4.2.2.1 BACKFILL PERFORMANCE

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

- Sketch, describe, explain and discuss typical purposes of :
- Sketch, describe, explain and discuss typical applications of :
- Sketch, describe, explain and discuss typical selection criteria for :
- Sketch, describe, explain and discuss typical partical size distribution, for :
- Sketch, describe, explain and discuss typical stress-strain curves for :
- Sketch, describe, explain and discuss typical SG s for :
- Sketch, describe, explain and discuss typical drainage characteristics of :
- Sketch, describe, explain and discuss typical pumping characteristics of :
- Sketch, describe, explain and discuss typical stowing characteristics of :
  - Classified tailings backfill
  - Full plant tailings
  - Paste fill
  - Sand fill
  - Aggregate fill
- Sketch, describe, explain and discuss the use and effects of cementitious binders with the above types of fill
- Specify performance criteria, backfill type and placement method to ensure successful backfilling on a mine.

4.2.2.2 BACKFILL FOR ENERGY RELEASE RATE CONTROL

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

- Describe, explain and discuss the ability of backfill to reduce energy release rates
- Explain and discuss the effect of placement lag on the effectiveness of backfill as a means of reducing energy release rates
- Explain and discuss why it is not necessary to completely backfill the mined area to ensure a significant reduction in energy release rate
- Estimate and discuss the effect of backfill on ultimate energy release rates given backfill performance parameters and stope closure rates.

4.2.2.3 BACKFILL FOR EXCESS SHEAR STRESS CONTROL

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

- Describe, explain and discuss how backfill affects the excess shear stress acting on geological features.

4.2.3. REGIONAL MINING LAYOUTS

The candidate must be able to demonstrate knowledge and understanding

of the above subject area by being able to :

- Sketch, describe, explain and discuss mining layout strategies to ensure regional stability of workings at shallow, intermediate and great depth
- Sketch, describe, explain and discuss stoping layouts to ensure regional stability of workings at shallow, intermediate and great depth
- Sketch, describe, explain and discuss stoping sequences to ensure regional stability of workings at shallow, intermediate and great depth
- Design stoping layouts and sequences to ensure the regional stability of workings at shallow, intermediate and great depth.

4.2.4. LOCAL STABILITY STRATEGIES

4.2.4.1 STOPING LAYOUTS AND SEQUENCES

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

- Sketch, describe, explain and discuss the layout of the following stoping methods:
  - Bord and Pillar mining
  - Scattered mining
  - Longwall mining
  - Sequential Grid mining
  - Indicate the location of access tunnels, service excavations and rockpasses
- Sketch, describe, explain and discuss how the following orebody geometries and/or combinations of orebody geometries may be mined :
  - Wide reefs, Multiple reefs
  - Steep reefs, Flat reefs
  - Shallow reefs, Deep reefs
  - Erratically payable reefs
- Sketch, describe, explain and discuss how a longwall may be re-established beyond a large geological displacement
- Indicate appropriate development and stoping sequences for re-establishing a longwall beyond a large geological displacement
- Estimate relative mining rates for the various re-establishment methods

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4.2.5. SERVICE EXCAVATION LAYOUTS

4.2.5.1 TUNNEL LAYOUTS

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

- Describe, explain and discuss the rock engineering criteria used to determine the optimum size, shape and orientation of service tunnels
- Describe, explain and discuss the rock engineering criteria used to determine the optimum layout, positioning, orientation and spacing of service tunnels relative to :
  - Static stope abutments
  - Moving stope abutments
  - Pillars of various shape, size and orientation

- Geological features of various types, orientation and history
- Other excavations

- Apply these criteria to design service tunnel layouts for :
  - Tunnels of various purpose and life expectancy
  - Various combinations of timing.

4.2.5.2 LARGE CHAMBER LAYOUTS

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

- Describe, explain and discuss the rock engineering criteria used to determine the optimum size, shape and orientation of large service chambers
- Describe, explain and discuss the rock engineering criteria used to determine the optimum excavation methodology and sequence for large service chambers
- Describe, explain and discuss the rock engineering criteria used to determine the optimum support methodology and sequence for large service chambers
- Describe, explain and discuss the rock engineering criteria used to determine the optimum layout, positioning, orientation and spacing of large chambers relative to :
  - Static stope abutments
  - Moving stope abutments
  - Pillars of various shape, size and orientation
  - Geological features of various types, orientation and history
  - Other excavations
- Apply these criteria to design large service chamber layouts for :
  - Chambers of various purpose and life expectancy
  - Various combinations of timing.

4.2.5.3 ROCKPASS LAYOUTS

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

- Describe, explain and discuss the causes of instability in rockpasses
- Describe, explain and discuss the effects of instability in rockpasses
- Describe, explain and discuss the options available to optimise rockpass layouts
- Describe, explain and discuss the options available for stabilising rockpasses
- Describe, explain and discuss the options available for supporting rockpasses
- Determine appropriate rockpass layout, orientation and support measures using rockmass and stress characterisation techniques
- Assess the suitability of rockpass layout, orientation and support measures for given sets of rockmass and structural conditions
- Assess the suitability of rockpass layout, orientation and support measures for given sets of field stress and induced stress conditions
- Describe, explain and discuss the rock engineering criteria used to determine the optimum size, shape and orientation of rockpasses
- Describe, explain and discuss the rock engineering criteria used to determine the optimum layout, positioning and spacing of rockpasses relative to :

- Static stope abutments
- Moving stope abutments
- Pillars of various shape, size and orientation
- Geological features of various types, orientation and history
- Other excavations
- Apply these criteria to design rockpass layouts for :
- Passes of various purpose and life expectancy
- Various combinations of timing

- Describe, explain and discuss the rock engineering criteria used to rehabilitate damaged rockpasses
- Estimate the rockpass life expectancy as a result of various support and rehabilitation measures.

4.2.6. SHAFT LAYOUTS

4.2.6.1 SHAFT PROTECTION PILLARS

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

- Sketch, describe, explain and discuss the rock engineering criteria used to design incline shaft protection pillars at shallow, intermediate and great depth
- Sketch, describe, explain and discuss the rock engineering criteria used to design vertical shaft protection pillars at shallow, intermediate and great depth
- Calculate stress, strain and tilt in vertical shafts using equations for circular shaft pillars in an elastic medium
- Apply the results from the above calculations to design appropriate shaft pillar dimensions
- Design shaft layouts for given depth, geology and rockmass conditions
- Explain the limitations that concrete linings and steelwork place on allowable movements in shafts.

4.2.6.2 SHAFT PILLAR EXTRACTION

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

- Sketch, describe, explain and discuss the preparatory work required prior to shaft pillar extraction at shallow, intermediate and great depth
- Describe, explain and discuss the extent and magnitude of stress changes to be expected around a shaft before, during and after shaft pillar extraction
- Describe, explain and discuss the extent and magnitude of movements to be expected in and around a shaft before, during and after shaft pillar extraction
- Describe, explain and discuss the extent and magnitude of seismicity to be expected around a shaft before, during and after shaft pillar extraction
- Sketch, describe, explain and discuss appropriate mining sequences for extracting a shaft pillar
- Sketch, describe, explain and discuss appropriate methods of support when extracting a shaft pillar
- Design appropriate shaft pillar extraction sequences for given shaft pillar layouts and rockmass data

- Design appropriate support for shaft pillar extraction workings for given shaft pillar layouts and rockmass data
- Design appropriate support and protection measures for associated shaft pillar excavations for given shaft pillar layouts and rockmass data.

4.2.6.3 ALTERNATIVE SHAFT PROTECTION METHODS

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

- Sketch, describe, explain and discuss the geotechnical issues associated with the following alternative shaft protection methods at shallow, intermediate and great depth :
  - Satellite Pillars
  - Bracket Pillars
  - Partial Extraction
  - Shaft Reef Extraction
  - Non-Intersection
- Sketch, describe, explain and discuss the functions of satellite shaft protection pillars
- Sketch, describe, explain and discuss the design of satellite shaft protection pillars
- Describe, explain and discuss the advantages and disadvantages of this approach
- Sketch, describe, explain and discuss the functions of shaft reef bracket pillars
- Sketch, describe, explain and discuss the design of shaft reef bracket pillars
- Describe, explain and discuss the advantages and disadvantages of this approach
- Sketch, describe, explain and discuss the purpose of partial shaft reef extraction
- Sketch, describe, explain and discuss the design of partial shaft reef extraction
- Describe, explain and discuss the advantages and disadvantages of this approach
- Sketch, describe, explain and discuss the effects of extracting the shaft reef at an early stage to avoid leaving a shaft protection pillar
- Describe, explain and discuss the advantages and disadvantages of this approach
- Sketch, describe, explain and discuss the precautions required to allow early extraction of the reef around shafts
- Sketch, describe, explain and discuss the adjustments required to the shaft to allow early extraction of the reef around a shaft
- Evaluate mining layouts, sequences, support and precautions necessary to allow early extraction of the reef around a shaft for given circumstances
- Sketch, describe, explain and discuss the precautions required to mine within shaft pillars transgressed by seismically active geological structures
- Sketch, describe, explain and discuss the siting of shafts entirely in the hangingwall or footwall so as not to intersect reef
- Describe, explain and discuss the advantages and disadvantages of this approach.

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4.2.7. LAYOUT DESIGN CRITERIA

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

- Describe, explain and discuss the derivation and application of the following mine layout and support design criteria for shallow, intermediate and great depths :
  - Average Pillar Stress
  - Energy Release Rate
  - Excess Shear Stress
  - Rockwall Condition Factor.

5. MINING SUPPORT STRATEGIES

5.1. STOPING EXCAVATION SUPPORT STRATEGIES

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

- Sketch, describe, explain and discuss the various behaviour of rock around excavations at shallow, intermediate and great depth
- Describe, explain and discuss how this affects stope support requirements at shallow, intermediate and great depth
- Describe, explain and discuss the concept of geotechnical areas / ground control districts
- Describe, explain and discuss rockmass classification schemes for the determination of geotechnical areas / ground control districts
- Describe, explain and discuss the determination of geotechnical areas / ground control districts
- Describe, explain and discuss the application of the concept of geotechnical areas / ground control districts
- Sketch, describe, explain and discuss appropriate support strategies to control the hangingwall in narrow stopes under static conditions at shallow, intermediate and great depth
- Sketch, describe, explain and discuss appropriate support strategies to control the hangingwall in narrow stopes under seismic conditions at depth
- Sketch, describe, explain and discuss the special problems of supporting wide reef stopes at shallow, intermediate and great depths
- Describe, explain and discuss appropriate strategies to ensure stability under wide reef conditions
- Sketch, describe, explain and discuss the special problems of supporting multiple, closely spaced reef stopes at shallow, intermediate and great depths
- Describe, explain and discuss appropriate strategies to ensure stability under multiple, closely spaced reef conditions
- Sketch, describe, explain and discuss how gullies should be supported at shallow, intermediate and great depths
- Describe, explain and discuss appropriate support strategies to ensure gully stability in various layouts at shallow, intermediate and great depths
- Describe, explain and discuss the special problems of supporting gullies relative to various stope layout and fracture configurations at shallow, intermediate and great depths
- Determine the required support reaction for static conditions given typical fall of ground dimensions and loading conditions
- Determine the required support reaction for dynamic (rockburst)

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conditions given typical fall of ground dimensions and loading conditions

- Determine appropriate stope support strategies for given rockmass conditions, field stress regimes and geological circumstances.

5.2. SERVICE EXCAVATION SUPPORT STRATEGIES

5.2.1. TUNNEL SUPPORT STRATEGIES

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

- Sketch, describe, explain and discuss the various behaviour of rock around excavations at shallow, intermediate and great depth
- Describe, explain and discuss how this affects tunnel support requirements at shallow, intermediate and great depth
- Include in design deliberations the influences occasioned by the following considerations :
  - Gravity loading environment, Seismic loading environment
  - Rock characteristics, Rockmass characteristics, Geological structure
  - Stress mechanisms, Stress cycle, Rate of stress change, Initial stress
  - conditions
  - Mining configuration, Mining cycle, Overstoping techniques,
  - Destressing techniques
  - Tunnelling configuration, Tunnelling cycle, Pre-development, Followbehind
  - development
  - Support components, Support systems, Support appropriateness
  - Implementation quality, Management effectiveness, Quality control
- Describe, explain and discuss the objectives of primary support in tunnels
- Describe, explain and discuss the objectives of secondary support in tunnels
- Describe, explain and discuss the objectives of tertiary support in tunnels
- Describe, explain and discuss the concept of integrated support in tunnels
- Describe, explain and discuss various support strategies for :
  - Return Airways
  - Haulages, Crosscuts
  - Travellingways
  - Winzes, Raises
  - Tunnel intersections
  - Tunnels traversing faults, Tunnels traversing dykes
  - Trackless mining declines in stopes
- Describe, explain and discuss the strategies to support tunnels under the following conditions :
  - Low stress with joint controlled behaviour
  - High stress where stress fractures dominate stability

- Changing stress cycles and environments over time
- Changing spatial stress environments
- Seismic and rockburst conditions

- Determine appropriate support strategies for tunnels for given layouts, rockmass conditions, stress regimes and geological circumstances.

5.2.2. SUPPORT STRATEGIES FOR SHAFTS AND LARGE EXCAVATIONS

5.2.2.1 SUPPORT STRATEGIES FOR VERTICAL SHAFTS

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

- Determine appropriate support strategies for vertical shafts for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.2.2.2 SUPPORT STRATEGIES FOR INCLINED SHAFTS

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

- Determine appropriate support strategies for inclined shafts for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.2.2.3 SUPPORT STRATEGIES FOR ROCKPASSES

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

- Determine appropriate support strategies for rockpasses for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.2.2.4 SUPPORT STRATEGIES FOR LARGE CHAMBERS

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

- Sketch, describe, explain and discuss the various behaviour of rock around large excavations at shallow, intermediate and great depth
- Describe, explain and discuss how this affects large excavation support requirements at shallow, intermediate and great depth
- Describe, explain and discuss rockmass classification schemes for the determination of support strategies for large chambers
- Describe, explain and discuss the application of the rockwall condition factor methodology to design support for large excavations at depth
- Apply the rockwall condition factor (RCF) methodology to design support for large excavations at depth
- Apply the Wiseman methodology to design support for large excavations at depth
- Describe, explain and discuss the effects of excavation sequence on support installation
- Determine appropriate excavation sequences to facilitate support installation
- Describe, explain and discuss the effects of excavation sequence on support effectiveness
- Determine appropriate excavation sequences to facilitate support effectiveness

- Describe, explain and discuss the rules of thumb for determining the length and spacing of tendons in large excavations
- Apply these rules of thumb to the design of support in large excavations
- Describe, explain and discuss the effects of overstoping and understoping on the stability and support requirements of off-reef excavations
- Determine appropriate support strategies for large chambers for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.3. SUPPORT DESIGN CRITERIA

5.3.1. STOPE SUPPORT DESIGN CRITERIA

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

- Describe, explain and discuss stope support requirements in terms of :
  - Initial stiffness, Yieldability
  - Areal coverage, Area of influence, Beam reinforcement, Suspension
  - Support resistance, Energy absorption
- Describe, explain and discuss how support resistance varies for static and dynamic conditions
- Calculate required support resistances for static conditions
- Calculate required support resistances for dynamic (rockburst) conditions
- Calculate support loads for support systems made up of various units
- Calculate energy absorption for support systems made up of various units
- Calculate required support resistances and energy absorption for complete mining cycles to ensure minima are always achieved throughout the cycle
- Consider the effects of variability of support unit quality
- Consider the effects of variability of support unit installation.

5.3.2. TUNNEL, CHAMBER AND SHAFT SUPPORT DESIGN CRITERIA

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

- Describe, explain and discuss excavation support and rockwall reinforcement requirements in terms of :
  - Initial stiffness, Yieldability
  - Unit length, Areal coverage, Area of influence, Beam reinforcement,
  - Suspension
  - Support resistance, Energy absorption
- Describe, explain and discuss how support resistance varies for static and dynamic conditions
- Calculate required support resistances for static conditions
- Calculate required support resistances for dynamic (rockburst) conditions

- Calculate support loads for support systems made up of various units
- Calculate energy absorption for support systems made up of various units
- Consider the effects of variability of support unit quality
- Consider the effects of variability of support unit installation.

5.4. SUPPORT UNIT AND SUPPORT SYSTEM TYPES AND CHARACTERISTICS

5.4.1. PILLARS AS STOPE SUPPORT

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

- Sketch, describe, explain and discuss the meaning, application and behavioural characteristics of :
  - Rigid pillars, Yield pillars, Crush pillars
- Describe, explain and discuss the factors that should be taken into consideration when designing a pillar support system
- Describe, explain, discuss and account for the limitations in determining the strength of hard rock pillars
- Describe, explain and apply empirical formulae to derive the strength of hard rock pillars
- Sketch, describe, explain and discuss the role of loading system stiffness in the stability of pillar systems once pillars have exceeded their peak strength
- Describe, explain and discuss the mechanism of foundation failure of pillars
- Describe, explain and discuss the factors that affect foundation failure of pillars
- Design in-stope pillar support layouts for :
  - Breast mining, Up-dip mining.

5.4.2. BACKFILL AS STOPE SUPPORT

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

- Sketch, describe, explain and discuss the main types of backfill used in tabular, hard rock mining support
- Sketch, describe, explain and discuss the preparation, reticulation and placement of the main types of backfill used in mining support
- Sketch, describe, explain and discuss the applications of the main types of backfill used in mining support
- Sketch, describe, explain and discuss the important backfill material characteristics that affect performance
- Sketch, describe, explain and discuss the important backfill placement characteristics that affect performance
- Describe, explain and discuss the backfill characteristics required for filling flat-dipping narrow tabular stopes at depth
- Describe which backfill types are suitable for filling flat-dipping narrow tabular stopes at depth
- Describe, explain and discuss the backfill characteristics required for filling shallow wide-reef tabular stopes
- Describe which backfill types are suitable for filling shallow wide-reef tabular stopes

- Describe, explain and discuss how backfill assists in improving the stability of rockwalls at the face
- Describe, explain and discuss how backfilling fits into the mining cycle
- Calculate the support resistance of backfill given the backfill performance characteristics and the stope closure rate
- Evaluate potential benefits of backfilling for given rockmass conditions and mining situations
- Determine appropriate backfill types for given rockmass conditions and mining situations
- Determine appropriate placement procedures for given rockmass conditions and mining situations.

5.4.3. PACK AND ELONGATE SUPPORT IN STOPES

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

- Describe, explain and discuss the stress-strain behaviour of timber :
  - When compressed parallel to the grain
  - When compressed perpendicular to the grain
- Describe, explain and discuss the effect of loading rate on the stress-strain behaviour of timber
- Describe, explain and discuss the effect of moisture content on the stress-strain behaviour of timber
- Describe, explain and discuss the effect of construction and installation on the stress-strain behaviour of timber
- Sketch, describe, explain and discuss typical load-deformation characteristics of the following support units :
  - Mine poles, Profile props, Pipe-sticks
  - Mechanical face props, Rapid-yielding hydraulic props
  - Cluster sticks
  - Other commonly available elongate support units
  - Timber packs, Timber-Cement (Composite) packs
  - Cement-based packs
- Describe, explain and discuss prestressing devices and systems in terms of :
  - Their effects on the above support types
  - Their limitations
- Sketch, describe, explain and discuss the principles of operation of rapid yielding hydraulic props
- Evaluate the various types of support units in terms of given mining environments, rockmass conditions and stress regimes
- Determine the support resistance of support units and support systems at various distances from the face for various support layouts
- Determine appropriate support designs for given mining environments, rockmass conditions and stress regimes.

5.4.4. SUPPORT AND REINFORCEMENT OF TUNNELS

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

- Apply the various rockmass classification systems to select appropriate support for tunnels in shallow, intermediate and deep mines
- Apply the rockwall condition factor (RCF) to select appropriate support for tunnels at depth
- Apply other appropriate criteria to select appropriate support for tunnels at depth
- Describe and discuss the following rockwall support types :
  - Mechanically anchored bolts, Cable bolts, Friction 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 the above units :
  - 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
- Design and evaluate the use of appropriate support units, support systems, support patterns and installation procedures for given rockmass conditions, stress regimes and mining layouts.

6. INVESTIGATION TECHNIQUES

6.1. ROCK TESTING

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

- Describe, explain and discuss various rock testing procedures in terms of :
  - Type of test procedure relevant to data requirements (UCS, TCS, UTS, etc)
  - Sample gathering, Sample description, Sample labeling, Sample preparation, Sample quality
  - Test methods and procedures to obtain required data
  - Data collection, Data classification, Failure modes
  - Interpret and incorporate test results in analysis and design
  - Account for the limitations of laboratory tests results relative to mine design requirements.

6.2. MONITORING

6.2.1. IN-SITU STRESS MEASUREMENT AND MONITORING

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

- • Sketch, describe, explain and discuss the techniques used to measure in-situ stress in the underground rockmass

- • Describe, explain and discuss the equipment used to measure in-situ stress in the rockmass
- • Interpret, explain and discuss given stress measurement data in terms of likely rockmass, pillar or excavation behaviour.

6.2.2. SEISMIC MONITORING SYSTEMS

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

- Describe, explain and discuss the differences between geophones and accelerometers
- Describe, explain and discuss typical applications of geophones
- Describe, explain 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
- Describe, discuss and explain how to determine objectives for a seismic monitoring system in tabular hard-rock mines
- Explain and discuss key performance criteria for seismic networks
- Explain and discuss the evaluation of historic seismicity for network design
- Explain and discuss the requirements of spatial seismic coverage
- Explain and discuss methods to quantify required seismic sensitivity and location accuracy
- Explain and discuss the needs of various stakeholders for seismic information with respect to monitoring objectives
- Describe, discuss and explain the role of seismic sensitivity and location accuracy modelling in network design
- Describe, discuss and explain how to derive a compromise between ideal network layout and the following constraints :
- Accessibility constraints
- Budgetary constraints
- Describe, discuss and explain the factors that need to be considered in placing seismometers
- Explain and discuss how seismometer placement could affect the event location accuracy of a seismic system
- Explain and discuss how seismometer placement could affect the sensitivity of a seismic system
- Describe, discuss and explain relevant procedures to ensure that seismic monitoring information is distributed effectively to stakeholders
- Describe, discuss and explain methods of quality assurance in seismic system management
- Describe and discuss relevant aspects of seismic system audits
- Describe and discuss methods to assess stakeholder needs
- Describe and discuss methods to identify possible shortfalls between seismic monitoring objectives and seismic system performance
- Describe, discuss and explain the predictability of seismicity
- Describe, discuss and explain the predictability of rockbursts

- Describe, discuss and contrast the various seismic magnitude scales
- Describe and discuss the use of seismic data in pro-active and re-active mine design
- Describe and discuss the main seismic parameters used in analysis and interpretation
- Describe and discuss how seismic data and parameter relations are interpreted.

6.2.3. SUBSIDENCE MONITORING

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

- Sketch, describe, explain and discuss various subsidence models
- Sketch, describe, explain and discuss various subsidence modelling procedures
- Sketch, describe, explain and discuss various subsidence monitoring procedures.

6.2.4. MODELLING

6.2.4.1 NUMERICAL MODELLING

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

- Describe, explain and discuss the differences between physical, empirical and numerical models
- Describe, explain and discuss the limitations of physical, empirical and numerical models
- Describe, explain and discuss the selection of appropriate models and codes to tackle various problems
- Describe, explain and discuss the input of appropriate parameters to investigate various problems
- Describe, explain and discuss the interpretation of output in the investigation of various problems
- Describe, explain and discuss the difference between relative and absolute results from numerical models
- Describe, explain and discuss the appropriate use of relative and absolute results from numerical models
- Describe and explain the relevance of calibration, correlation and back analysis in interpreting absolute results from numerical models
- Describe and explain the relevance of calibration, correlation and back analysis in interpreting relative results from numerical models.

6.2.5. AUDITING

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

- Describe, explain and discuss the concept of monitoring for understanding, prediction and design.

7. ROCKBREAKING IN HARD ROCK

7.1. CUTTING TECHNIQUES

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

- Describe, explain and discuss geotechnical aspects associated with various
- Non-explosive rock breaking procedures that include :
  - Expanding grout techniques
  - Block sawing
  - Reef cutting, Reef boring
  - Tunnel boring, Raise boring
- Describe, explain and discuss the methodologies and applications of these techniques
- List and discuss the advantages and disadvantages of these techniques.

7.2. DRILLING TECHNIQUES

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

- Sketch, describe, explain and discuss the different rounds used in shaft sinking
- Describe, explain and discuss the different cuts used in shaft sinking
- Describe, explain and discuss the types of initiation used in the above rounds
- Describe, explain and discuss the sequence of initiation of blast holes used in the above rounds
- Describe, explain and discuss the optimum explosive types and their properties for use in the above rounds
- Sketch, describe, explain and discuss the different rounds used in tunnel development
- Describe, explain and discuss the different cuts used in tunnel development
- Describe, explain and discuss the types of initiation used in the above rounds
- Describe, explain and discuss the sequence of initiation of blast holes used in the above rounds
- Describe, explain and discuss the optimum explosive types and their properties for use in the above rounds
- Sketch, describe, explain and discuss blast hole layouts in stopes
- Describe, explain and discuss the direction of drilling of blast holes in stopes
- Describe, explain and discuss the sequence of initiation of blast holes in stopes
- Describe, explain and discuss the optimum explosive types and their properties for use in the above rounds
- Describe, explain and discuss the importance of blast-hole drilling accuracy in the following applications:
- Shaft sinking, Chamber excavation, Orepass excavation, Tunnel development,
- Stope advance
- Cushion blasting, Smooth blasting.

7.3. BLASTING PRACTICES

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

- Sketch, describe, explain and discuss the chemical reactions associated with various explosives used in mining
- Sketch, describe, explain and discuss the physical manifestations associated with various explosives used in mining
- Sketch, describe, explain and discuss the basic differences between explosives used in mining
- 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 objectives and effects of decoupling explosives
- Describe, explain and discuss the methods by which de-coupling of explosives is achieved
- Describe, explain and discuss the following excavation cushion blasting and smooth blasting techniques :
  - Pre-splitting
  - Concurrent smooth blasting
  - Post-splitting
- Describe, explain and discuss the methodologies and typical applications of each technique
- List and discuss the advantages and disadvantages of these techniques
- Evaluate and determine blasting requirements for tunnels making use of knowledge of explosives
- Evaluate and determine appropriate blasting rounds to suit given conditions in tunnels
- Evaluate and determine appropriate explosive types to suit given conditions in tunnels
- Evaluate and determine blasting requirements for stopes making use of knowledge of explosives
- Evaluate and determine appropriate blasting rounds to suit given conditions in stopes
- Evaluate and determine appropriate explosive types to suit given conditions in stopes.

8. SURFACE EFFECTS

8.1. SUBSIDENCE ENGINEERING

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

- Sketch, describe, explain and discuss the geotechnical aspects associated with subsidence of rock to surface above shallow tabular hard rock workings.

SURFACE PROTECTION

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

- Sketch, describe, explain and discuss the protection of the surface subject to subsidence above shallow tabular hard rock workings
- Design various appropriate pillars for the protection of the surface subject to subsidence above shallow tabular hard rock workings.

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9. MINING STRATEGIES IN DIFFICULT CIRCUMSTANCES

9.1. SPECIAL AREAS

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

- Describe, explain and discuss the methods of determining hazards and the level of risk associated with mining excavations
- Describe, explain and discuss the methods of determining hazards and the level of risk associated with mining layouts
- Describe, explain and discuss the reasons and motivation for implementing special area procedures on mines
- Describe, explain and discuss the procedures associated with assessing and declaring special areas on mines
- Describe, explain and discuss the procedures associated with implementing and managing special areas on mines
- Describe, explain and discuss the measures and precautions to be applied in special areas in terms of :
  - Continuous risk assessment, Operational planning
  - Access control, Mining methods, Support methods
- Describe, explain and discuss the additional mining and support precautions to be taken in the following situations :
  - Mining in remnant areas
  - Mining adjacent to geological features
  - Mining through geological features
  - Workings that are about to be stopped
  - Workings that are to be re-opened after standing for some time
  - Workings that are to be re-opened after a collapse.

9.2. PRE-CONDITIONING

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

- Sketch, describe, explain and discuss how pre-conditioning is applied to relieve high-stress conditions in stopes at depth
- Sketch, describe, explain and discuss how pre-conditioning is applied to relieve high-stress conditions in tunnels at depth
- Describe, explain and discuss the theoretical basis for pre-conditioning with reference to :
  - Rock strength, Rockmass strength, Stress regime, Rock
  - fracturing, Seismic environment
- Typical applications where pre-conditioning is required or indicated.

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1. GEOTECHNICAL CHARACTERISTICS

1.1. GEOLOGY

1.1.1. GEOLOGICAL SEQUENCES

- Lurie J 1987 South African Geology for Mining, Metallurgical, Hydrological and Civil Engineering
- Lexicon Publishers Jhb Chapter 9
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines
- SIMRAC Jhb Chapter 1

1.1.2. GEOLOGICAL STRUCTURES

- Lurie J 1987 South African Geology for Mining, Metallurgical, Hydrological and Civil Engineering Lexicon Publishers Jhb Chapter 6
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 1

1.2. ROCK STRENGTH

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 2
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 4
- Obert L & Duvall WI 1967 Rock Mechanics and the Design of Structures in Rock John Wiley & Sons New York Chapters 10, 11
- Jaeger JC & Cook NGW 1969 Fundamentals of Rock Mechanics Chapman & Hall London Chapter 4, 6
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 2

1.3. ROCKMASS CHARACTERISTICS

1.3.1. GEOTECHNICAL ROCKMASS CLASSIFICATION

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 2
- Hoek E & Brown ET 1980 Underground Excavations in Rock IMM London Chapter 2
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 3
- Stacey TR 2001 Best Practice Rock Engineering Handbook for 'Other' Mines SIMRAC Jhb Chapter 2

2. ROCK AND ROCKMASS BEHAVIOUR

2.1. ROCKMASS BEHAVIOUR AROUND TABULAR EXCAVATIONS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 8
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 3, 4, 5
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 7, 8, 9

2.2. ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 8

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- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 3, 4, 5
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 7, 8, 9

2.3. ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS

- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 3, 4, 5
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 8
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 7, 8, 9

3. MINE SEISMICITY

3.1. PRINCIPLES OF SEISMICITY

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

3.2. MINE SEISMOLOGY AND ROCKMASS STABILITY

3.2.1. MINE SEISMOLOGY

- Chief Inspector of Mines Latest Guideline for the Compilation of a Madatory
- Code of Practice to Combat Rock Fall and Rock Burst Accidents in Tabular Metalliferous Mines - Ref.No. DME 16/3/2/1-A3 DME
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 9
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
- Mendecki AJ (ed) 1997 Seismic Monitoring in Mines Chapman & Hall London Chapters 1, 2, 5, 7, 8

3.2.2. ROCKMASS STABILITY

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 8
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
- Mendecki AJ (ed) 1997 Seismic Monitoring in Mines Chapman & Hall London Chapters 10, 11

4. MINING LAYOUT STRATEGIES

4.1. HARDROCK TABULAR MINING METHODS

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 12, 13, 15

4.2. REGIONAL STABILITY STRATEGIES

4.2.1. PILLARS AS REGIONAL SUPPORT

4.2.1.1 REGIONAL PILLARS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 13

4.2.1.2 STABILISING PILLARS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6

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4.2.1.3 BRACKET PILLARS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6

4.2.2. BACKFILL AS REGIONAL SUPPORT

4.2.2.1 BACKFILL PERFORMANCE

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapters 6, 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

4.2.2.2 BACKFILL FOR ENERGY RELEASE RATE CONTROL

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapters 6, 7

4.2.2.3 BACKFILL FOR EXCESS SHEAR STRESS CONTROL

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapters 6, 7

4.2.3. REGIONAL MINING LAYOUTS

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 2
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 7, 8, 9

4.2.4. LOCAL STABILITY STRATEGIES

4.2.4.1 STOPING LAYOUTS AND SEQUENCES

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 3
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
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- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapters 5, 6

4.2.5. SERVICE EXCAVATION LAYOUTS

4.2.5.1 TUNNEL LAYOUTS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 5
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 7, 8, 9
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapters 4, 9

4.2.5.2 LARGE CHAMBER LAYOUTS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 7, 8, 9
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4.2.5.3 ROCKPASS LAYOUTS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapters 7, 8, 9

4.2.6. SHAFT LAYOUTS

4.2.6.1 SHAFT PROTECTION PILLARS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 10

4.2.6.2 SHAFT PILLAR EXTRACTION

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 10

4.2.6.3 ALTERNATIVE SHAFT PROTECTION METHODS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
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4.3. LAYOUT DESIGN CRITERIA

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5. MINING SUPPORT STRATEGIES

5.1. STOPING EXCAVATION SUPPORT STRATEGIES

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4
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- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
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5.2.2. SUPPORT STRATEGIES FOR SHAFTS & LARGE EXCAVATIONS

5.2.2.1 SUPPORT STRATEGIES FOR VERTICAL SHAFTS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7

5.2.2.2 SUPPORT STRATEGIES FOR INCLINED SHAFTS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7

5.2.3. SUPPORT STRATEGIES FOR ROCKPASSES

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5.2.4. SUPPORT STRATEGIES FOR LARGE CHAMBERS

5.3. SUPPORT DESIGN CRITERIA

5.3.1. STOPE SUPPORT DESIGN CRITERIA

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
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5.3.3. SUPPORT UNIT AND SUPPORT SYSTEM TYPES AND CHARACTERISTICS

5.3.3.1 PILLARS AS STOPE SUPPORT

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 8
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 13

5.3.3.2 BACKFILL AS STOPE SUPPORT

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 11
- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 14

5.3.3.3 PACK AND ELONGATE SUPPORT IN STOPES

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4
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- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 7

5.3.4. SUPPORT AND REINFORCEMENT OF TUNNELS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6
- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 7
- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter 4

6. INVESTIGATION TECHNIQUES

6.1. R OCK TESTING

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 2

6.2. MONITORING

6.2.1. IN-SITU STRESS MEASUREMENT AND MONITORING

6.2.1.1 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
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6.2.1.2 SUBSIDENCE MONITORING

- van der Merwe JN 1995 Practical Coal Mining Strata Control Sasol Coal Division Jhb Chapter 5

6.3. MODELLING

6.3.1. 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

6.4. AUDITING

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

7. ROCKBREAKING IN HARD ROCK

7.1. CUTTING TECHNIQUES

7.2. DRILLING TECHNIQUES

7.3. BLASTING PRACTICES

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 17
- Local References ??
- AEL Blasting News ??

8. SURFACE EFFECTS

8.1. SUBSIDENCE ENGINEERING

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining Chapman & Hall New York Chapter 16
- Obert L & Duvall WI 1967 Rock Mechanics and the Design of Structures in Rock John Wiley & Sons New York Chapter 18

- van der Merwe JN 1995 Practical Coal Mining Strata Control Sasol Coal Division Jhb Chapter 3, 5
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- van der Merwe JN 1995 Practical Coal Mining Strata Control Sasol Coal Division Jhb Chapter 3, 5
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- Stacey TR & Swart AH 2001 Practical Rock Engineering Practice for Shallow and Opencast Mines SIMRAC Jhb Chapter 4

9. MINING STRATEGIES IN DIFFICULT CIRCUMSTANCES

9.1. SPECIAL AREAS

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 3
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9.2. PRE-CONDITIONING

- Jager AJ & Ryder JA 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 8

CHAPTER 2

PAPER 3.1 HARD ROCK TABULAR MINING

INTRODUCTION

Geological and geotechnical characteristics form the basis of any rock engineering design that takes place in the tabular, hard rock environments. These characteristic's include knowledge of the different rock types present, the presence and properties of geological structures, the strength of the rock out of which the rock mass is composed and the quality of the rock mass.

This chapter will discuss all of the above but it is critical that the user also reads and study similar chapters on geology. Geological structures, geological sequences and rock mass ratings systems discussed in the COM Certificate in Rock Engineering Paper 2 material package.

LEARNING OUTCOMES

2. GEOTECHNICAL CHARACTERISTICS
2.1. GEOLOGY
2.1.1. GEOLOGICAL SEQUENCES
The candidate must be able to demonstrate knowledge and understanding of the above subject area by being able to:

- Identify and describe the rock types associated with the main South African tabular, hard rock orebodies
- Describe, explain and discuss how the rock types associated with tabular, hard rock orebodies were formed
- Sketch, describe and discuss the geological sequences associated with tabular, hard rock orebodies.

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

- Sketch, describe and discuss major geological structures associated with the main South African tabular, hard rock orebodies
- Describe and discuss the following aspects of major geological structures associated with the main South African tabular, hard rock orebodies :
  - Origin, Genesis, Mechanism of formation
  - Residual effects of formation on geotechnical behaviour
  - Subsequent effects of formation on geotechnical behaviour.

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

- Discuss the relative strengths of rock types associated with tabular, hard rock orebodies

- Describe and discuss the geotechnical characteristics of rock types associated with tabular, hard rock orebodies
- Apply the above knowledge to the design of total extraction workings
- Apply the above knowledge to the design of partial extraction workings.

2.3. ROCKMASS CHARACTERISTICS
2.3.1. GEOTECHNICAL ROCKMASS CLASSIFICATION

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

- Describe, discuss and apply rockmass classification techniques for the seclection of hard rock tabular mining methods
- Describe, discuss and apply standard rockmass classification and assessment systems to predict excavation stability and support requirements
- Describe, discuss and apply the rockwall condition factor (RCF) to predict tunnel stability and support requirements.

2.1 GEOLOGY

2.1.1 GEOLOGICAL SEQUENCES

LEARNING OUTCOME 2.1.1.1

IDENTIFY AND DESCRIBE THE ROCK TYPES ASSOCIATED WITH THE MAIN SOUTH AFRICAN TABULAR, HARD ROCK OREBODIES

Three basic rock types exist:

- igneous,
- sedimentary and
- metamorphic rocks.

Within each of these basic categories, a large number of different rock types exist. Typical rock types that can be found within South Africa have been summarised into Table 1.

Igneous rocks	Sedimentary rocks	Metamorphic rocks
Andesite - an Intermediate volcanic rock Anorthosite - an igneous ultramafic rock Diabase or dolerite - an intrusive mafic rock forming dykes or sills Diorite - a coarse grained intermediate plutonic rock Dunite - an ultramafic cumulate rock Harzburgite - a variety of peridotite; an ultramafic cumulate rock Kimberlite - a rare ultramafic, ultrapotassic volcanic rock Lamprophyre - an ultramafic, intrusive rock Norite - a hypersthene bearing gabbro Pegmatite - an igneous rock (or metamorphic rock) with giant sized crystals Pyroxenite - a coarse grained plutonic rock composed of >90% pyroxene Syenite - a plutonic rock dominated by orthoclase feldspar Tuff - a fine grained volcanic rock formed from volcanic ash Granite –a common type of intrusive, igneous rock which is granular and consists mainly of quartz, mica, and feldspar. Dolerite –is a mafic, subvolcanic rock equivalent to volcanic basalt and are typically shallow intrusive bodies	Argillite - a sedimentary rock composed primarily of clay-sized particles Banded iron formation - a fine grained chemical sedimentary rock Chert - a fine grained chemical sedimentary rock composed of silica Coal - a sedimentary rock formed from organic matter Conglomerate - a sedimentary rock composed of large rounded fragments of other rocks Diamictite - a poorly sorted conglomerate Dolomite - a carbonate rock composed of the mineral dolomite Greywacke - an immature sandstone with quartz, feldspar and rock fragments within a clay matrix Jaspillite - an iron-rich chemical sedimentary rock similar to chert or banded iron formation Lignite - a sedimentary rock composed of organic material; otherwise known as Brown Coal Limestone - a sedimentary rock composed primarily of carbonate minerals Mudstone - a sedimentary rock composed of clay and muds Sandstone - a clastic sedimentary rock Shale - a clastic sedimentary rock Siltstone - a clastic sedimentary	Anthracite - a type of coal Mylonite - a metamorphic rock formed by shearing Pseudotachylite - a glass formed by melting within a fault via friction Quartzite - a metamorphosed sandstone Serpentinite - a metamorphosed ultramafic rock dominated by serpentine minerals Slate - a low grade metamorphic rock formed from shale or silts Schist –constitute a group of medium-grade metamorphic rocks, and by definition, contains more than 50% platy and elongated minerals Gneiss – is a common and widely distributed type of rock formed by high-grade regional metamorphic processes from pre-existing formations that were originally either igneous or sedimentary rocks.

Table 1: Typical rock types in South African hard rock, tabular environments



Figure 1: Andesite (igneous rock)



Figure 2: Limey shale overlain by limestone (sedimentary rocks)



Figure 3: Banded gneiss (metamorphic rock)

LEARNING OUTCOME 2.1.1.2

DESCRIBE, EXPLAIN AND DISCUSS HOW THE ROCK TYPES ASSOCIATED WITH TABULAR, HARD ROCK OREBODIES WERE FORMED

Igneous Rocks

Igneous rocks are crystalline solids that form from the cooling of magma, i.e. are formed from melted rock that has cooled down and solidified. This is an 'exothermic' process (i.e. it loses heat) and involves a phase change from a liquid to a solid state.

The melted lava (magma) is made up of different chemical elements and different types of minerals and thus forms different rocks when is solidifies:

- If the magma crystallizes below the earth surface, it forms igneous rocks such as granite, pyroxenite with large grain sizes.
- If the magma crystallize above the earth surface, it forms igneous rocks such as basalt with fine grain sizes

Igneous rocks are given names based on:

- composition (chemicals and minerals they are made of, especially the silica content which relates to brightness)
- colour;
- mode of occurrence
- grain size
- hardness etc.

INTERESTING INFO

Rocks deep within the Earth melt because of the existing high pressure and temperature levels. The molten rock (magma) flow upward or erupts from a volcano onto the surface. When magma cools slowly (well below surface) crystals grow slowly and a coarse-grained rock forms. When the magma cools rapidly on the surface, the crystals are extremely small, and a fine-grained rock results. (Courtesy of Wikipedia)

Sedimentary Rocks

In most places on the earth's surface (either in water or on land), the igneous rocks (which make up the majority of the crust) are covered by a thin layers of loose sediment formed through either fluvial, Aeolian, ice or solution processes.

These sediments are layered accumulations of fragments formed by the breakaway or weathering of other rocks, minerals, animal or plant material. The layers are compacted and cemented together, forming hard rocks (clastic sedimentary rocks) but can also be formed through chemical accumulation (non-clastic sedimentary rocks).

INTERESTING INFO

The sedimentary layers are normally parallel or nearly parallel to the earth's surface. If they are at high angles to the surface, twisted or broken, it is the result of earth movements. Sedimentary rocks are forming around us all the time. Sand and gravel on beaches or in river beds look like sandstone and conglomerate, which they will become with time. Compacted and dried mud hardens into shale with time.

Sedimentary rocks are called secondary, because they are the result of the accumulation of small pieces broken off of pre-existing rocks. There are three main types of sedimentary rocks:

- Clastic: The basic sedimentary rock consist of accumulations of little pieces of broken up rock which have piled up, compacted and cementated.
- Chemical: Many rock types form when standing water evaporates, leaving dissolved minerals behind.
- Organic: The accumulation of sedimentary debris caused by organic processes. Animals consist of calcium-rich shells, bones and teeth. The calcium can pile up on the seafloor and accumulate into a thick enough layer to form an "organic" sedimentary rock.

INTERESTING INFO

Rock-forming and rock-destroying processes have been active for billions of years.

- Today, in the Guadalupe Mountains of western Texas, you can stand on limestone, a sedimentary rock, that used to be a coral reef in a tropical sea about 250 million years ago.
- In Vermont's Green Mountains you can see schist, a metamorphic rock, that was once mud in a shallow sea.
- Half Dome in Yosemite Valley, California, which now stands nearly 8,800 feet above sea level, is composed of quartz monzonite, an igneous rock that solidified several thousands of feet below the earth's surface. (Courtesy of Wikipedia)

Metamorphic Rocks

Metamorphic comes from the words "meta" (change) and "morph" (form). Any rock (sedimentary or igneous rocks) can be metamorphosed into a new rock type. All that is required is for the rock to be transformed, either physically or chemically, under the influence of temperature or pressure. The metamorphic changes in the minerals is governed by the 'parent' rock and thus determines the type of metamorphic rock that will be formed. E.g. limestone is metamorphosed to marble and granite is metamorphosed to a gneiss.

The process of metamorphism does not melt the rocks, but instead transform them into denser, more compact rocks. New minerals are created either by rearrangement of mineral components or by reactions with fluids that enter the rocks.

NOTE

Some kinds of metamorphic rocks (such as granite gneiss and biotite schist) are strongly banded or foliated (the parallel arrangement of certain mineral grains that gives the rock a striped appearance) (Figure 3).

Pressure or temperature can even change previously metamorphosed rocks into new types.

INTERESTING INFO

The formation of the earth that finally consist of the three basic rock types described above, is found to be based on a number of theories, including, although may not be limited to the following:

- Creation: The earth was created by God in a process described in the Holy Bible over a period of 7 days, according to His divine plan and through His power, as part of the total creation, consisting of a great number of solar systems;
- Nebular hypothesis: The solar system originally consisted of spinning nebula of gaseous material that, as it cooled down, ejected 'outer rings' that formed the planets, planets ejected rings to form moons;
- Meteoric theory: The sun and other stars were formed by compaction of a number of meteorites;
- Planetismal hypothesis: The earth from a vast number of smaller meteorites revolving around the sun;
- Tidal theory: A star passing too close by the sun, drew (through gravitational forces) a sigar-shaped mass that broke up into several lumps, each solidifying into a planet;
- Impact theory: The start mentioned above, collided with the sun;
- Von Weizsacker's theory: The stars and planets originated as 'centres of condensation' and the sun attracted material to itself, becoming hotter;
- Big bang theory: The universe is the remainder of a large fireball that exploded. The debris of the explosion are constantly moving away from the position of the explosion.

CONNECTION 1

Also refer to Paper 2, Outcomes 2.1.1.1 and 2.1.1.2

REFERENCE

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

LEARNING OUTCOME 2.1.1.3

CONNECTION 2

Also refer to Paper 2, Outcome 2.1.2.1

SKETCH, DESCRIBE AND DISCUSS THE GEOLOGICAL SEQUENCES ASSOCIATED WITH TABULAR, HARD ROCK OREBODIES

REFERENCE

Page 44, Lurie, J. 1987. Geology.

2.1.2 GEOLOGICAL STRUCTURES

LEARNING OUTCOME 2.1.2.1

CONNECTION 3

Also refer to Paper 2, Outcome 2.1.3.3 to 2.1.3.6

SKETCH, DESCRIBE AND DISCUSS MAJOR GEOLOGICAL STRUCTURES ASSOCIATED WITH THE MAIN SOUTH AFRICAN TABULAR, HARD ROCK OREBODIES

REFERENCE

- Page 5 – 50, Ryder J.A. and Jager A.J, A textbook on rock mechanics for tabular , hard rock mines, 2002.
- Page 28, Lurie, J. 1987. Geology.

LEARNING OUTCOME 2.1.2.2

DESCRIBE AND DISCUSS THE FOLLOWING ASPECTS OF MAJOR GEOLOGICAL STRUCTURES ASSOCIATED WITH THE MAIN SOUTH AFRICAN TABULAR, HARD ROCK OREBODIES:

- Origin, Genesis, Mechanism of formation
- Residual effects of formation on geotechnical behaviour
- Subsequent effects of formation on geotechnical behaviour.

The origin and mechanism of formation has been described in Paper 2, Outcomes 2.1.3.3 to 2.1.3.6.

Table 2 below summarises the impact of each of the common structures on geotechnical behavior of the rock mass in and around the structures.



The impacts may not be limited to those given below and it is advised that specific impacts that realized in the environment within which the user works, must be identified and this list be augmented.

Structure	Residual impact	Subsequent impact
Bedding planes / laminations / weak planes	Assists water flow through the rock mass by providing alternative flow paths.	Creates beams of variable thickness that must remain stable over the excavation span or require support installation. Extreme variability in thickness over short distances, such as with crossbedding, create significant control concerns. Allows easy separation under gravity and require small spans or support installation to maintain stability.
Faults	Forms sympathetic faults with smaller throws parallel to the main structure, reducing rock mass strength and quality. Affects rock mass around it due to metamorphism of the rock mass and may improve or reduce rock mass strengths and quality. Forms 'infilling' on the plane that affects the occurrence of natural earthquakes. Could affect principal stress components (magnitude and orientation) due to 'stress lock-up' close to the structure.	Assists water flow through the rock mass by providing alternative flow paths. Create wedges with excavation walls that must be supported to maintain stability. Assists the occurrence of mining induced seismicity. Reduces rock mass quality and thus increases support requirements. Increases potential for instability if poor frictional strength due to infilling material.
Dykes / sills	Affects rock mass around it due to metamorphism of the rock mass and may improve or reduce rock mass strengths and quality. Could affect principal stress components (magnitude and orientation) due to 'stress lock-up' close to the structure. Assists water flow through the rock mass by providing alternative flow paths.	Contacts can create wedges with excavation walls that must be supported to maintain stability. Assists the occurrence of mining induced seismicity. Reduces rock mass quality and thus increases support requirements.
Joints	Creates poorer rock mass quality and strength. Creates blocky ground. Allows water flowing through the rock mass. Reduce rock mass quality and thus increases support requirements.	Create 'blocky' ground conditions and increases support requirements. Create wedges with excavation walls that must be supported to maintain stability.

Structure	Residual impact	Subsequent impact
Domes	Could affect principal stress components (magnitude and orientation) due to 'stress lock-up' close to the structure, if the result of shear.	Create large blocks of rock that collapses under gravity and that must be supported to maintain stability.
Folds	Could affect principal stress components (magnitude and orientation) due to 'stress lock-up' after folding.	Create difficult mining and support installation practices. Causes mining into the hangingwall / roof and potential instability of the workings.
Potholes	Creates poorer rock mass quality and strength due to increased jointing.	Allows exposure of the flat orientated and densely spaced stress fractured anorthositic material in the hangingwall of the workings, increasing potential for instability.
Ramps and Rip outs	Creates unstable wedges and domes in lava.	Create large blocks of lava that collapses under gravity from beneath the fault plane (dome-like shape) and the ramps (wedge shape), that must be supported to maintain stability.
Pilloid structures	No impact.	Effects haningwall behavior due to the shear occurring along the sides of the structures, often requiring denser support spacings to address.
Iron Rich Ultramafic Permatoids	Creates dangerous ground conditions	Normaly avoided due to poor ground conditions.

Table 2: Geological structures and their impact on geotechnical behaviour



For the purpose of this outcome it is understood that 'residual impacts' refer to the impact that remained in the rock mass after the formation of the structure and 'subsequent impact' refer to those impacts that occurs specifically due to mining practices.

2.2 ROCK STRENGTH

LEARNING OUTCOME 2.2.1

DISCUSS THE RELATIVE STRENGTHS OF ROCK TYPES ASSOCIATED WITH TABULAR, HARD ROCK OREBODIES

Typical rock strengths of some of the most common rock types are shown in Table 3 below, showing significant variability in the results quoted from various sources.

Rock type	UCS (MPa)	Rock Type	UCS
Quartzite	150 – 475	Lava (hard)	200 - 500
Shale	80 - 427	Anorthosite	120 - 210
Lava (soft)	80 - 160	Norite	100 - 250
Dolomite	300	Pyroxenite	100 - 180
Manganese	130 - 250	Banded Ironstone	350 - 660

Table 3: Typical rock type strengths



Appreciation of typical rock strengths is important, but it must be noted that values quoted in literature is subject to the following that must be kept in mind when quoting and using strength values:

- *Number of sample tested: If sampling is not adequate, analysis of the results could be incorrect;*

- *Impact of transverse isotropy: If a rock type has substantially different strengths when tested in different directions, such as testing a schist parelel to or perpendicular to foliation, large scale variations will result;*
- *Position of the sample in the 'stratigraphy': In materials that are extremely non-homogeneous (different properties at different positions), strength results will be dependent on the position of the borehole and position of the sample within the core;*
- *Averaging results: The tendency to average strength test results can be a dangerous practice if standard variations are large and where knowledge of the actual material strengths (such as with pillar designs) are crucial to the stability of the workings. Statistical analysis should be done carefully and with complete understanding of the abovementioned concerns.*
- *Outliers: Outliers should be removed from any analysis and can be done on the following basis:*
  - *Remove results where the samples failed on existing discontinuities since the test results report the strength of the discontinuity and not the intact sample;*
  - *Remove those results that affect the standard deviation most, i.e. the results that appear too high;*
  - *Keep in mind that low results could be real and could affect pillar behavior if tests were conducted for this purpose. In this case, careful consideration to the removal of low test results values is required.*

LEARNING OUTCOME 2.2.2

DESCRIBE AND DISCUSS THE GEOTECHNICAL CHARACTERISTICS OF ROCK TYPES ASSOCIATED WITH TABULAR, HARD ROCK OREBODIES

Table 4 to Table 7 show summaries of some of the geotechnical characteristics of the most common rock types associated with the mining of different orebodies in the hard rock, tabular environments.



The list of rock types and common geotechnical characteristics is not suggested to be complete for all environments and the user is urged to gather information from the specific mining environment in the same manner as shown below.

Orebody	Rock types	Geotechnical characteristics	Impact on designs for total or partial extraction mining methods
Platinum reefs (Ryder and Jager, Page 42-48)	Pyroxenite, Norite, Anorthosite	Rock material strengths are mostly relatively high, except for the pyroxenite in some areas, where lower strengths have been reported. Rock mass quality is mostly a function of the joint sets found in these rock types, but is generally high (eg. 90). The quality varies across the material types and across the mining areas and does not allow simple listing of characteristics. Joint spacings vary but mostly occur in at least three sets and at variable dips (although mostly steep) and dip directions. Joint infilling is often serpentinsed increased potential for instability on joints, whilst other filling such calcite and pegmatoid result in high strength joints. Structures present in this environment include weakness planes (eg. Bastard reef), faults (mostly low density with small throws), dykes (dolerite, diabase, syenite and lamprophyre), domes (cooling / ramp fault structures) and potholes (replacing or thinning of seams, exposing anorthositic material, increased joint density).	Pillar design on Merensky smaller dimensions due to material strength. Rock mass strength high due to good material quality, affecting span and support designs. Jointing requires adequate support spacing and length (tendons) design. The presence of ground water and serpentised filled joints increases the need for support installation and result in reduced spacings. The impact of jointing on rock mass strength and thus pillar design to be considered. Structures must be taken into consideration when determining the required support resistance. Support and mining practices must be adapted through the Lamprophyre dykes. Mining and support practices must be adapted as potholes are approached.

Table 4: Geotechnical characteristics of most common rock types in Platinum environment

Orebody	Rock types	Geotechnical characteristics	Impact on designs for total or partial extraction mining methods
Chrome reefs (Ryder and Jager, Page 40-42, 49)	Pyroxenite, Norite	Rock material strengths are mostly relatively high, except for the chromotite, where lower strengths have been reported. Joint spacings are mostly wide occur but blocky conditions due to small spacings have been observed. Structures present in this environment include weakness planes (eg. undulating calcite break and triplets above the UG2 and clay-like features within the chromotite seam), faults (UCB is often viewed as a bedding plane fault traversing from the footwall through the seam and into the hangingwall), dykes (dolerite, diabase, syenite and lamprophyre) and domes (cooling / ramp fault structures).	Lower rock material strengths mean lower rock mass strength and thus larger pillar dimensions compared to other hard rock environments. Larger fall-outs are possible due to larger joint spacings and needs to be considered in terms of support resistance requirements. Areal coverage support systems to be considered in blocky conditions. Notice of parting planes in the determination of support resistance requirements is critical. Support and mining practices must be adapted through the Lamprophyre dykes.

Table 5: Geotechnical characteristics of most common rock types in Chrome environment

Orebody	Rock types	Geotechnical characteristics	Impact on designs for total or partial extraction mining methods
Gold reefs (Ryder and Jager, Page 15 – 38)	Quartzite, shale, lava	The environments (and thus the geotechnical characters) are very different between the different reefs that include (but is not limited to) Basal, Leader, B-reef and A-reef in the Free State, the Vaal, C and VCR reefs in the Klerksdorp area, the Carbon Leader, VCR in the Carletonville area and the Kimberley and Elsburg reefs in the West Rand and Evander areas. Since each of these reefs (and other reefs not listed here) is different, only major differences are listed here for some of the mining areas:	
		Free State: Shale in the Basal reef hangingwall that makes support installation and span limitations critical. The shale affects stress fracture formation, their dip and dip directions, as well as closure rates. The B-reef has shale footwall that affects the installation of support units as punching of units into the footwall can reduce the effectiveness of the support. Low middling between the Basal and Leader reefs in some areas affect sequencing. Thrusting and folding occurred along the western limits of the goldfields, creating multi-reef areas and increasing reef dips to vertical. Ball-and-pillow formations (boulders of various dimensions and round shapes) are sometimes exposed in the argillaceous Basal reef hangingwall.	Presence of shale and competent quartzite beam thickness in determining beam stability / support spacings and unsupported spans is important. Presence of weak footwall material considered when selecting optimum support types and / or load spreaders on the footwall. Selecting and / or implementation of mining methods in steep and multi-reef areas need to consider reef interaction. Ball-and-pillow formations, where exposed, to be considered when designing support stiffness requirements and spacings.
		Klerksdorp: Crossbedding on the Vaal Reef cause severe ground control problems as support spacings and unsupported spans have to be limited, of which the Zandpan Marker that forms a distinct weakness plane in the immediate hangingwall is the best known.. The conglomerate bands present in the hangingwall can affect stability. Significant faulting of all possible throws and dips are present forcing the implementation of a scattered mining method. C-reef environment is known to create very dense stress fracturing in the siliceous hangingwall.	Variable quartzite beam thickness to be considered in designing support spacings and unsupported mining spans. Position of bands in hangingwall to be evaluated in terms of support resistance requirements. Impact of faulting on mining sequencing, the need for bracket pillars to counter seismic activity and the need for high yield or energy absorption capacity becomes important in this environment. Fracture patterns to be considered in designing support spacings and unsupported mining spans.

Orebody	Rock types	Geotechnical characteristics	Impact on designs for total or partial extraction mining methods
Gold reefs (Ryder and Jager, Page 15 – 38)	Quartzite, shale, lava	Carletonville: The Carbon Leader is often overlain by the 'greenbar', an argillite, causes ground control problems, especially during seismic activity. The quartzite is extremely siliceous and brittle, resulting in the occurrence of strain bursting and 'face burst' events, caused by inter alia high shear stress levels ahead of the deep working faces. The VCR is overlain by well jointed lava material showing very different characteristics in different areas, with the 'tuff' (also known as the western area formation) being incompetent and affecting stress fracturing and closure. The stronger lavas have shown a variety of responses to mining, resulting in the identification of different types of lava namely aphanitic, porphyritic, amygdaloidal and pillow lavas (Figure 4). The presence of bedding plane faults and domes or ramp structures create areas of concern with regards to ground control.	Presence and position of the greenbar to be considered in support resistance and support spacing designs. Energy absorption capacity is required for support units in this environment, whilst mining practices to reduce the occurrence/ impact of face bursting is required. Impact of stress fracturing on pillar width:height ratios to be considered in pillar design. Evaluation ot the lava type in the VCR hangingwall is critical to establish lava behavior and thus support requirements. Presence of domes to be considered in determining the support resistance requirements.
		West rand: Multi-reef mining is common due to the large number of payable reefs and is found in both flat and steep dipping areas. Good ground conditions in areas with no stress fracturing (shallow depth), widely spaced joints and strong quartzite on the Elsburg reefs. The VCR, with poor strength WAF as hangingwall experience ground control problems.	Impact of various reefs on each other to be considered in determining mining sequencing, pillar design, generation of seismic activity, etc. Impact of poor quality lava on support requirements to be considered.
		Evander: Gold reefs are often overlain by several Coal Seams. A high density of small throw faults is present.	Impact of possible interaction between these mine workings must be considered.

Table 6: Geotechnical characteristics of most common rock types in Gold environment



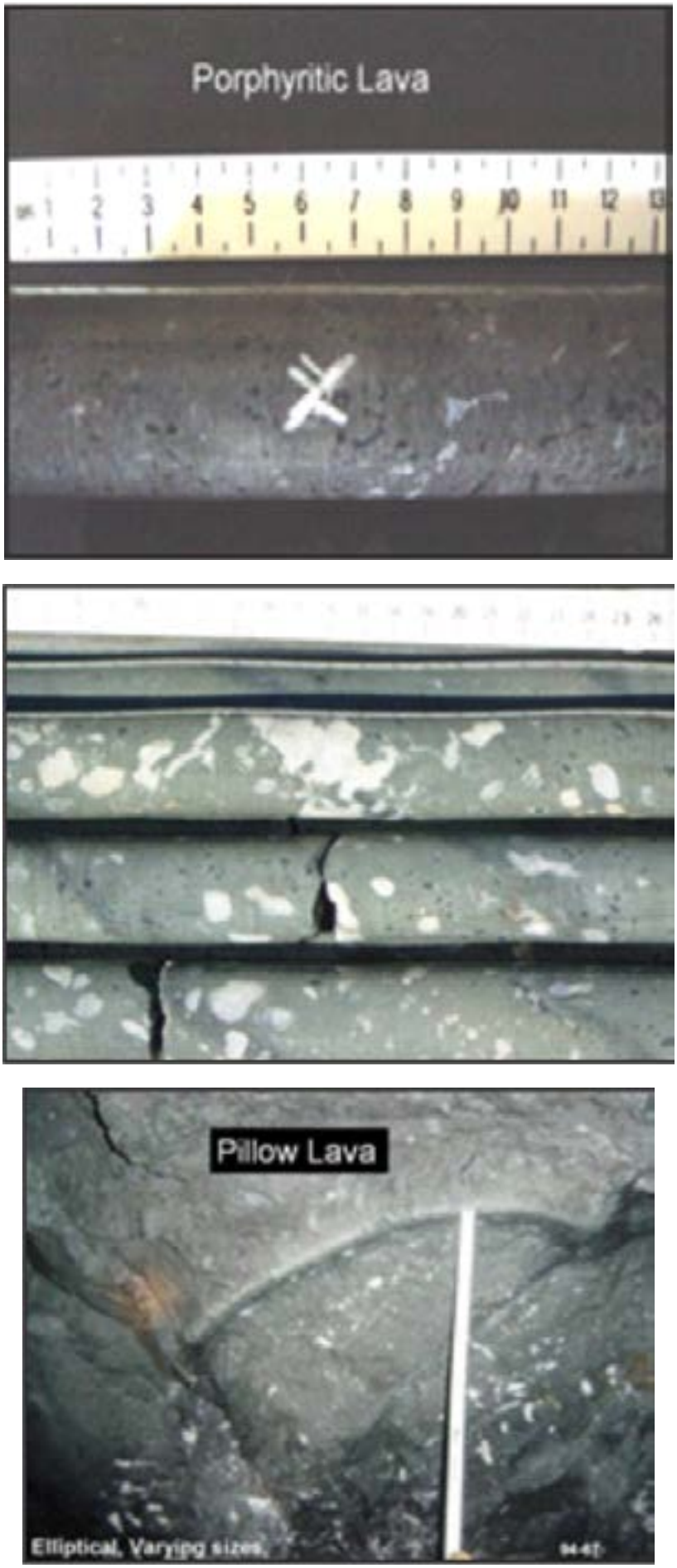


Figure 4: Lava types: (a) aphanitic, (b) porphyritic, (c) amygdaloidal and (d) pillow lavas (Courtesy of Anglogold Ashanti)

Ore-body	Rock types	Geotechnical characteristics	Impact on designs for total or partial extraction mining methods
Manganese seams	Manganese, banded ironstone, shale, quartzite, clay, calcrete	The manganese seams are surrounded by Banded Ironstone formations which are very strong, but laminated and well jointed. Laminated nature requires reduced mining spans and sufficient support installation. Joint sets are present and consist of several sets, mostly vertical and if infilling is present, it is normally a calcite infilling (although quartz filled joints can also be found). Joint spacings vary quite significantly. Faulting is less significant in that large throw faults are present but small throw faults (although present) are much less of a concern than on the other areas discussed in this section. Folding of the seams is quite common and is the result of the thrusting that had occurred in this area, increasing the presence of joint calcite filling in some areas. Dyke intrusions are normally of significant thickness and reduce the quality of the rock mass around the structures due to increased joint density and calcite infilling. Other rock types such as shale, quartzite, tillite, clay and calcrete are only exposed in shafts and declines and can cause some ground control problems.	Strength of material to be considered in pillar design. Laminated nature of banded ironstone to be evaluated when designing unsupported and supported mining spans. Impact of increased joint density on rock mass strength and thus pillar and span design. Impact on thrusting and dyke intrusions on rock mass composition and quality. Different mining practices close to dyke structures due to decreased rock mass quality to be considered. Traversing weaker material types during access development to be considered in support and excavation width design.

Table 7: Geotechnical characteristics of most common rock types in Manganese environment

REFERENCE

- Page 5 – 50, Ryder J.A. and Jager A.J, A textbook on rock mechanics for tabular , hard rock mines, 2002.

LEARNING OUTCOME 2.2.3

APPLY THE ABOVE KNOWLEDGE TO THE DESIGN OF TOTAL EXTRACTION WORKINGS

Total extraction workings are defined to include environments where the following mining methods are practiced:

1. Scattered mining

CONNECTION 4

Refer to Table 4 in "LEARNING OUTCOME 2.2.2" on page 44

The focus of designs in these areas is on maintaining stability in spite of the geologically disturbed rock mass with artificial support types.

LEARNING OUTCOME 2.2.4

CONNECTION 5

Refer to Table 4 in "LEARNING OUTCOME 2.2.2" on page 44

APPLY THE ABOVE KNOWLEDGE TO THE DESIGN OF PARTIAL EXTRACTION WORKINGS.

Partial extraction workings are defined to include environments where the following mining methods are practiced:

- Scattered mining with in-stope pillars
- Longwall and sequential grid mines with stabilizing pillars
- Bord-and-pillar operations.

The focus of designs in these areas is on maintaining stability with pillars (various types and functions) and artificial support types.

2.3 ROCK MASS CHARACTERISTICS
2.3.1 GEOTECHNICAL ROCKMASS CLASSIFICATION

LEARNING OUTCOME 2.3.1.1

CONNECTION 6

Refer to Paper 2, Outcomes 3.1.1 and 3.1.2 for detail on rock mass classification systems and their applications

Refer to Paper 2, Outcome 3.1.2.11 and 3.1.2.12 for the use of rock mass ratings in mining method selection.

DESCRIBE, DISCUSS AND APPLY ROCK MASS CLASSIFICATION TECHNIQUES FOR THE SELECTION OF HARD ROCK TABULAR MINING METHODS

In the selection of the most appropriate mining method from a geotechnical perspective, the following options are available:

- Naturally supported stopes: Stability is maintained with pillars. These methods ensure stability as small spans are maintained but normally result in low extraction ratios.
- Artificially supported stopes: Stability is maintained using timber based, cement based or backfill support types only.
- Stopes supported by a combination of natural and artificial material: Pillars are left to ensure separation of the mining area into compartments or to limit mining spans with selective pillar positioning. Artificial support is utilised to maintain stability between pillars.
- Unsupported, caving method: No support is utilised in back areas and the mined out area is allowed to collapse after a period of time that is sufficient to allow the extraction of the ore.

In determining the best solution in terms of mining method, the state of the rock mass and its ability (or the absence thereof) to remain stable when unsupported over certain mining spans is essential.

Rock mass classification systems can be used to establish the quality of the rock mass, its deformation characteristics and ability to remain stable over certain mining spans until supported. At the same time, stabs at indications toward a possible support requirements for development, can also be derived from rock mass classification systems.



Most of the rock mass rating systems were based on civil engineering tunnelling projects and not for tabular orebodies. Even though some systems made alterations to improve its applicability to other environments, results must be used with caution.

EXAMPLE

Usually some form of rating system is utilised in the selection of the most appropriate mining method. Table 8 shows issues to be considered for each of the common mining methods under the stated conditions and end in a rating that accumulates statements that suggest appropriateness of the method to the condition. In this system 'Yes' means 'appropriate', 'No' means 'not appropriate' and 'maybe' (M) suggests that it 'can be applied with some difficulty':

Mining method	Spans to be limited to 8m	Laminated hanging-wall can be supported	Jointed reef does not affect on-reef stability	Jointed hangingwall can be supported	Poor footwall material strength and quality can be avoided	No seismic activity	Shallow mining depth	High reef width	Rating for the stated conditions
Bord and pillar	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	7
Scattered	No	Yes	Yes	Yes	No	Yes	No	M	4.5
Scattered with in-stope pillars	Yes	Yes	No	Yes	Yes	Yes	Yes	M	6.5
Longwall mining	No	Yes	Yes	Yes	No	Yes	No	M	4.5

Mining method	Spans to be limited to 8m	Laminated hanging-wall can be supported	Jointed reef does not affect on-reef stability	Jointed hangingwall can be supported	Poor footwall material strength and quality can be avoided	No seismic activity	Shallow mining depth	High reef width	Rating for the stated conditions
Sequential Grid	No	Yes	Yes	Yes	No	Yes	No	M	4.5

Table 8: Critical geotechnical issues for the different mining methods

From this rating, it is suggested that the bord-and-pillar mining method should be most appropriate for the stated conditions.

In the final selection of a mining method, many other factors such as percentage extraction requirements, grade variability, service excavations required, and employee availability or skill level, etc are also considered and can be included in a similar rating system to indicate the ultimately most appropriate method.

Although not critical for selection of mining methods, the execution of geotechnical data collection and rock mass classification together with geological investigations, assist in identifying sources of potential mining problems or safety related hazards in the mining area.

LEARNING OUTCOME 2.3.1.2

CONNECTION 7

Refer to Paper 2, Outcomes 3.1.1 and 3.1.2

DESCRIBE, DISCUSS AND APPLY STANDARD ROCK MASS CLASSIFICATION AND ASSESSMENT SYSTEMS TO PREDICT EXCAVATION STABILITY AND SUPPORT REQUIREMENTS

LEARNING OUTCOME 2.3.1.3

DESCRIBE, DISCUSS AND APPLY THE ROCKWALL CONDITION FACTOR (RCF) TO PREDICT TUNNEL STABILITY AND SUPPORT REQUIREMENTS.

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.

In evaluating the proposed development, the σ_1 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).

$$RCF = \sigma_{\theta max} / F\sigma_c$$

Where $\sigma_{\theta max}$ 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 $\sigma_{\theta max}$ is calculated for an excavation with a circular cross-sectional shape as follows:

$$\sigma_{\theta max} = 3\sigma_{1i} - \sigma_{3i}$$

Where σ_{1i} and is σ_{3i} 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 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 are 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 9);
- 0.7 > RCF < 1.4 average conditions prevail with typical support systems (Table 10) and for
- RCF > 1.4 poor ground conditions prevail with special support requirements (Table 11).

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 >1.5 m in length installed on a basic 2 m pattern. Support resistance: 30 – 50 kN/m2. Welded mesh (sidewall only).
Seismic activity anticipated	Friction bolts or tendons > 1.2 m in length, installed as close to the face as possible, on a basic 2 m or 1.5 m pattern. Support resistance: 30 – 50 kN/m2.	Fully grouted (pref. yield) tendons/friction bolts >1.5 m in length installed on a basic 2 m pattern. Support resistance: 50 kN/m2. Welded mesh (hanging-wall and sidewalls).

Table 9: 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 > 1.5 m in length, installed as close to the face as possible, on a basic 2 m or 1.5 m pattern. Support resistance: 30 – 50 kN/m2.	Welded mesh integrated with primary support tendons/friction bolts.
Stress changes anticipated	Fully grouted tendons or steel ropes >1.8 m in length installed as close to the face as possible, on a basic 2 m or 1.5 m pattern. Support resistance: 40 – 60 kN/m2.	Welded mesh integrated with primary support tendons/friction bolts.
Seismic activity anticipated	Fully grouted (pref. yield) tendons or steel ropes >1.8 m in length installed as close to the face as possible, on a 1.5 m or double 2 m pattern. Support resistance: 80 – 100 kN/m2.	Welded mesh integrated with primary support tendons/friction bolts, plus optional shotcrete.

Table 10: Support recommendations for average ground conditions (0.7<RCF<1.4) (by Ryder JA &Jager AJ)

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 > 1.8 m on a basic 1.5 m pattern, as close to the face as possible. Support resistance: 80 – 110 kN/m2.	Welded mesh integrated with primary support tendons/friction bolts; optional shotcrete.
Stress changes anticipated	(If necessary) shotcrete to face, then fully grouted steel ropes or yielding tendons / friction bolts, length >1.8 m on a basic 1 m or double 2 m pattern, as close to the face as possible.Support resistance: 120 – 230 kN/m2.	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 >2.3 m on a basic 1 m pattern, as close to the face as possible.Support resistance: 80 – 100 kN/m2.	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 11: Support recommendations for poor ground conditions (RCF>1.4) (by Ryder JA &Jager AJ)

EXAMPLE

The following principal stress components that act perpendicular to the axis of a 3m x 3m tunnel developed in 80MPa (UCS) shale, were derived from numerical modeling: σ_1= 60MPa and σ_3=20MPa. The tunnel is planned for an intermediate depth mine where seismic activity occurs. The shale is stable when exposed, but when subjected to seismic activity disintegrates. Recommend a suitable support system using the RCF factor and list any specifications wrt to the suggested support deemed appropriate.

$$RCF = \sigma_{\theta max} / F\sigma_c$$

Where F is an empirical factor representing the condition of the rock mass and σ_c is the intact rock strength (UCS) of the rock material, while

$$\sigma_{\theta max} = 3\sigma_{1i} - \sigma_{3i}$$

The UCS and the principal stress components were given, but selection of the factor F is required. Since the material is shale and it is know that shale is normally laminated and jointed, i.e. discontinuous, it is assumed that an appropriate F value would be 0.75.

Therefore:

RCF = (3 x 60MPa – 20MPa)/(0.75 x 80MPa)
= 2.67

From Table 11 (RCF > 1.4) it follows that the following support is suggested:

Primary support (typical requirements)	Secondary support (typical requirements)
(If necessary) shotcrete to face, then fully grouted steel ropes or yielding tendons /friction bolts, length >2.3 m on a basic 1 m pattern, as close to the face as possible. Support resistance: 80 – 100 kN/m2.	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.

For the conditions specified and from the suggestions above, the following is recommended:

- Support systems

Primary support	Secondary support
Shotcrete to the face prior to support installation. Fully grouted yielding tendons with length 2.3 m to provide a support resistance of 100 kN/m2.	Mesh integrated with yielding tendons.

- Shotcrete applied to a 50mm thickness. If fibre reinforced, the mesh as secondary support can be disposed of.
- Yielding tendon type to be selected to allow sufficient energy absorption capacity (based on the mine's criterion).
- Support spacings to be determined by support resistance and energy absorption capacity criteria.



- Page 358, Ryder J.A. and Jager A.J., A textbook on rock mechanics for tabular hard rock mines, 2002
- Page 207, Jager A.J. and Ryder J.A., Handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 2002

CHAPTER

3

PAPER 3.1 HARD ROCK TABULAR MINING

INTRODUCTION

The knowledge of the geological and geotechnical characteristic of the material within which mining will not occur, allows us to determine the behaviour of the rock mass that consist of the materials discussed in the previous chapter. The knowledge and understanding of quasi-static rock mass behaviour around the stopes (tabular excavations), tunnels, service excavations and shafts, leads into the section on dynamic (seismic) rock mass behaviour in the latter sections. In operations where seismic activity is not presently occurring, it must be realised that in general, increases in stress levels due to increasing mined out areas, may at some stage start to manifest into seismic activity. The principles of seismicity must therefore be known to the rock engineering practitioner.



Quasi-static is defined at those conditions where it is not implied that no displacements are occurring (completely static) but where displacements occur at such a slow rate that conditions can be viewed as 'nearly-static'.

LEARNING OUTCOMES

3. ROCK AND ROCKMASS BEHAVIOUR

3.1. Rockmass behaviour around tabular excavations

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

3.1.1. Sketch, describe, explain and discuss the behaviour of the rockmass around tabular excavations at shallow, intermediate and great depth

3.1.2. Describe, explain and discuss the changing amount and nature of rock that needs to be supported at shallow, intermediate and great depth

3.1.3. Describe, explain and discuss the different rates of deformation of the rockmass at shallow, intermediate and great depth

3.1.4. Sketch, describe, explain and discuss the types of fractures found around stope faces at shallow, intermediate and great depth

3.1.5. Describe, explain and discuss the effects of stope face fracturing on the stability of the stope hangingwall, stope footwall and stope face

3.1.6. Sketch, describe, explain and discuss the types of fractures found around stope gullies at shallow, intermediate and great depth

3.1.7. Describe the effects of stope gully fracturing on the stability of the gully hangingwall and sidewalls

3.1.8. Sketch, describe, explain and discuss the behaviour of rock surrounding tabular excavations during seismic events

3.1.9. Describe, explain and discuss typical heights of rockfalls in tabular underground gold and platinum mines in South Africa

back

3.1.10. Describe, explain and discuss the significance of rockfall height in terms of support requirements in gravity loading conditions

3.1.11. Describe, explain and discuss the significance of rockfall height in terms of support requirements in seismic loading conditions

3.1.12. Evaluate given rockmass conditions and excavation dimensions to determine or predict stope stability

3.1.13. Evaluate given rockmass conditions and excavation dimensions to determine remedial measures to improve stope stability

3.1.14. Describe, explain, discuss and apply the following techniques to the above evaluation :

- Rockmass class, Stress analysis, Continuum mechanics,
- Discrete block kinematics, Beam theory, Failure criteria

3.1.15. Sketch, describe, explain and discuss rockmass behaviour and conditions associated with stoping abutments at depth

3.1.16. Sketch, describe, explain and discuss the on-reef effects associated with stoping abutments at depth

3.1.17. Sketch, describe, explain and discuss the off-reef effects associated with stoping abutments at depth

3.1.18. Sketch, describe, explain and discuss rockmass behaviour and conditions associated with stope pillars at shallow, intermediate and great depth

3.1.19. Sketch, describe, explain and discuss the on-reef effects associated with stope pillars and/or remnants at shallow, intermediate and great depth

3.1.20. Sketch, describe, explain and discuss the off-reef effects associated with stope pillars and/or remnants at shallow, intermediate or great depth

3.1.21. Sketch, describe, explain and discuss the purpose, procedure and effects of rockmass pre-conditioning ahead of stopes at depth

3.1.22. Sketch, describe, explain and discuss the various methods of pre-conditioning

3.1.23. Describe, explain and discuss the critical features of pre-conditioning.

3.2. ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS

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

3.2.1. Describe, explain and discuss how rocks and rockmasses may be classified

3.2.2. Describe, explain and discuss how rockmass classification systems may be used to predict tunnel stability at shallow, intermediate and great depths

3.2.3. Describe, explain and discuss the nature of fracturing around tunnels at depth

3.2.4. Describe, explain and discuss the effects and consequences of fracturing on tunnel size

3.2.5. Describe, explain and discuss the effects and consequences of fracturing on tunnel shape

3.2.6. Describe, explain and discuss the effects and consequences of fracturing on tunnel stability

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3.2.7. Describe, explain and discuss the phenomenon of time-dependant fracturing and deterioration

3.2.8. Describe, explain and discuss the effects and consequences of time-dependant fracturing and deterioration on tunnel stability

3.2.9. Describe, explain and discuss the effects of stress regime and geological structure on rock behaviour and tunnel stability

3.2.10. Describe, explain and discuss the effect of tunnel size, tunnel shape and tunnel excavation technique on rock behaviour and tunnel stability

3.2.11. Describe, explain and discuss the optimisation of tunnel size, shape and orientation to suit geological considerations

3.2.12. Describe, explain and discuss the optimisation of tunnel size, shape and orientation to suit field stress considerations

3.2.13. Describe, explain and discuss the behaviour of rock surrounding tunnels at depth during seismic events

3.2.14. Describe, explain and discuss the siting of tunnels with respect to existing, current and future stoping operations

3.2.15. Describe, explain and discuss the siting of tunnels with respect to geological stratigraphy and structures

3.2.16. Describe how rockwall condition factor (RCF) is determined

3.2.17. Describe and discuss how the rockwall condition factor may be applied to predict tunnel stability and support requirements

3.2.18. Apply the rockwall condition factor to predict tunnel stability and support requirements at depth

3.2.19. Evaluate given rock conditions and field stresses to predict excavation stability

3.2.20. Evaluate given rock conditions and field stresses to recommend remedial measures to improve excavation stability.

3.3. ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS

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

3.3.1. Describe, explain and discuss how rocks and rockmasses may be classified

3.3.2. Describe, explain and discuss how rockmass classification systems may be used to predict service excavation stability at shallow, intermediate and great depths

3.3.3. Describe, explain and discuss the nature of fracturing around vertical sinking shafts at depth

3.3.4. Describe, explain and discuss the nature of fracturing around inclined sinking shafts at depth

3.3.5. Describe, explain and discuss the nature of fracturing around rock passes at depth

3.3.6. Describe, explain and discuss the effect of stress fracturing and geological structure on the final shape of vertical and inclined shafts

3.3.7. Describe, explain and discuss the effect of stress fracturing and geological structure on the final shape of rock passes

3.3.8. Describe, explain and discuss the conditions that may lead to the formation of large unstable rock wedges in vertical shaft sidewalls

back

3.3.9. Describe, explain and discuss the conditions that may lead to the formation of instabilities in rock passes

3.3.10. Describe, explain and discuss the nature of fracturing around service excavations at depth

3.3.11. Describe, explain and discuss the effects and consequences of fracturing on service excavation size

3.3.12. Describe, explain and discuss the effects and consequences of fracturing on service excavation shape

3.3.13. Describe, explain and discuss the effects and consequences of fracturing on service excavation stability

3.3.14. Describe, explain and discuss the phenomenon of time-dependant fracturing and deterioration

3.3.15. Describe, explain and discuss the effects and consequences of time-dependant fracturing and deterioration on service excavation stability

3.3.16. Describe, explain and discuss the effects of stress regime and geological structure on excavation stability

3.3.17. Describe, explain and discuss the effects of excavation size, shape and orientation on excavation stability

3.3.18. Describe, explain and discuss the effects of excavation technique on fracturing and the stability of service excavations

3.3.19. Describe, explain and discuss the effects of excavation sequence on stress fracturing and the stability of service excavations at depth

3.3.20. Describe, explain and discuss the effects of support sequence on stress fracturing and the stability of service excavations at depth

3.3.21. Describe, explain and discuss the optimisation of service excavation size, shape and orientation to suit geological considerations

3.3.22. Describe, explain and discuss the optimisation of service excavation size, shape and orientation to suit field stress considerations

3.3.23. Describe, explain and discuss the behaviour of rock surrounding service excavations at depth during seismic events

3.3.24. Describe, explain and discuss the siting of service excavations with respect to existing, current and future stoping operations

3.3.25. Describe, explain and discuss the siting of service excavations with respect to geological stratigraphy and structures

3.3.26. Describe, discuss and explain how rockmass characterisation may be applied to predict service excavation stability and support requirements

3.3.27. Apply rockmass characterisation techniques to predict service excavation stability and support requirements

3.3.28. Evaluate given rock conditions, field stresses, geological conditions, excavation sizes, shapes and orientations to predict excavation stability

3.3.29. Evaluate given rock conditions, field stresses, geological conditions, excavation sizes, shapes and orientations to recommend remedial measures to improve excavation stability

3.3.30. Evaluate given rock conditions, field stresses and geological conditions to predict shaft stability

3.3.31. Evaluate given rock conditions, field stresses and geological conditions to recommend remedial measures to improve stability.

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3.4. MINE SEISMICITY

3.4.1. Principles of seismicity

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

- Define, describe, explain and discuss the term seismic event
- Describe, explain and discuss the phenomenon of foreshocks and aftershocks
- Describe, explain and discuss the processes that give rise to violent failure and rupture in rock
- Describe, explain and discuss the rupture process in solid rock
- Describe, explain and discuss the rupture process on a pre-existing fault plane
- Differentiate between the terms seismic event and rockburst
- Define, describe, explain and discuss the term rockburst
- Identify, describe, explain and discuss the conditions required for a rockburst to occur
- Describe, explain and discuss the main mechanisms responsible for seismic events in South African mines
- Describe, explain and discuss the main mechanisms responsible for rockbursts in South African mines
- Define, describe and explain the phenomenon of seismic body waves
- Describe, explain and distinguish between different types of seismic body waves
- Define, 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, explain and discuss the differences between seismic body waves and seismic surface waves
- Describe, explain and discuss the factors that affect the velocity of propagation of seismic waves in rock
- Describe and explain how the rockmass environment affects the velocity of Pwaves and S-waves
- State typical velocities of P-waves and S-waves in hard rock and soft rock environments
- Describe, explain and discuss how this information may be used to locate the source of seismic events
- Describe, explain and discuss how the energy of a seismic body wave is attenuated as it propagates through elastic rock
- Describe, explain and discuss 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, explain and discuss the phenomena of reflection and refraction of seismic body waves
- Describe, explain and discuss how seismic wave reflection and refraction gives rise to and affects rockburst damage in excavations
- Describe, explain and discuss the objectives of seismic monitoring in mines

- Identify, describe, explain and discuss the features of seismic events that are used to locate the source of seismic events
- 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 epi-centre used in describing the location of seismic events
- Distinguish between seismic source mechanism and seismic damage mechanism
- Define, describe, explain, discuss and contrast the following primary source parameters in terms of their physical meaning and the data required to determine them :Richter magnitude, Local magnitude, Moment magnitude, Seismic energy, Seismic moment, Seismic displacement
- Define, describe, explain, discuss and contrast the following secondary source parameters, in terms of their physical meaning and the data required to determine them :Corner frequency, Source radius, Apparent volume, Stress drop, Energy index
- Describe and explain the common methods of seismic data analysis.

3.5. MINE SEISMOLOGY AND ROCKMASS STABILITY

3.5.1. Mine seismology

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

- Describe, explain and discuss the characteristics of mine seismicity in terms of :Causes, Location, Incidence, Magnitude, Time, Effects
- Define, describe, explain and discuss the following terms: Rockburst, Strong ground motion, Peak particle velocity
- List, describe, explain and discuss the factors that determine the nature of seismicity in tabular hard-rock mining
- Describe, explain and discuss the relationship between mine layout and seismicity
- Describe, explain and discuss the relationship between depth and seismicity
- Describe, explain and discuss the relationship between rock types and seismicity
- Describe, explain and discuss the relationship between local geology and seismicity
- Describe, explain and discuss the relationship between regional geology and seismicity
- List, describe, explain and discuss the measures that may be applied to control seismicity
- Describe, explain and discuss measures to reduce the maximum magnitude of seismic events
- Describe, explain and discuss measures to reduce the frequency of seismic events
- Define, describe, explain, discuss and contrast the following secondary source parameters in terms of :
  - their physical meaning
  - the data required to determine them

    - Source dimension, Apparent volume, Static seismic stress drop, Dynamic seismic stress drop, Energy index, Apparent stress, Dominant frequency, Corner frequency
- Explain the difference between the following source parameters :
    - Richter magnitude, Local magnitude, Moment magnitude
- State, describe and explain the main characteristics of the Brune Model
- Describe and explain how to obtain a fault plane solution from moment tensor analysis
- Describe and explain the following common methods of seismic data analysis : Gutenberg-Richter analysis, Magnitude-Frequency relationship, Energy-Moment relationship, Trend analysis, Time-of-day distribution, Instability analysis
- Describe typical outputs of the above analyses
- Describe, explain and discuss the relationships between : Extent of stressed rock and Maximum potential seismic event magnitude, Volumetric closure and Seismic moment, Event magnitude and Damage caused by seismic events, Peak particle velocity and Damage caused by seismic events
- Describe, explain and discuss how peak particle velocity decreases with distance from a seismic event source
- Identify, describe and explain the basic steps that are involved in carrying out a rockburst site investigation
- Describe and explain the relevant factors to inspect and investigate at a rockburst site
- Describe and explain the relevant seismic source parameters to investigate
- Describe and explain the relevant aspects of seismic history to analyse
- Describe, explain, discuss and contrast the relationship between : Rockburst damage and Seismic source characteristics, Rockburst damage and Excavation support characteristics
- Describe, explain and discuss the different types of seismic emission sources in tabular hard-rock mining
- Demonstrate familiarity with the guideline for the compilation of a code of practice to combat rock related accidents by being able to :
    - Define, describe and explain the term ‘seismically active’ as defined in the guideline,
    - Describe and explain the methods of estimating seismic hazard in tabular hard-rock mines,
    - Describe and explain the steps necessary to define emission sources in various ground control districts,
    - Describe and explain the term ‘reasonably practicable’ in the context of seismic risk management,
    - Describe and explain the various strategies to combat seismicity related risks,
    - Describe and explain the various seismic energy emission control measures,
    - Describe and explain the various rockburst damage control measures.

3.5.2. Rockmass stability

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

- Describe, explain and discuss how the following strategies assist in reducing rockburst hazards:
  - Pre-conditioning,
  - Backfilling,
  - Stabilising pillars.

LEARNING OUTCOME 3.1

ROCKMASS BEHAVIOUR AROUND TABULAR EXCAVATIONS

Depth related conditions are estimated for the South African context using Figure 1, showing increased occurrence of stress induced fracturing and closure as mining depth increases. The depth categories are based on depth below surface only and suggest that:

- Shallow mining conditions exist at depth < 1000m
- Intermediate (medium) mining conditions exist at 1000m < depth < 2250m
- Deep mining conditions exist at depth > 2250m while
- Depths > 3500m suggest ultra-deep mining conditions.

INTERESTING INFO

Another method of estimating 'depth related conditions' can be found in the depth (H) to span (l) ratio.

Consider a scenario where H = depth below surface and l = mining span.

The categories between infinite and shallow depth are identified in terms of H/l where,

- 8≤H/l≤∞ suggests deep mining conditions
- 1≤H/l≤8 is a transitional range from infinite to shallow depths and where
- 0≤H/l≤1 suggests shallow mining conditions

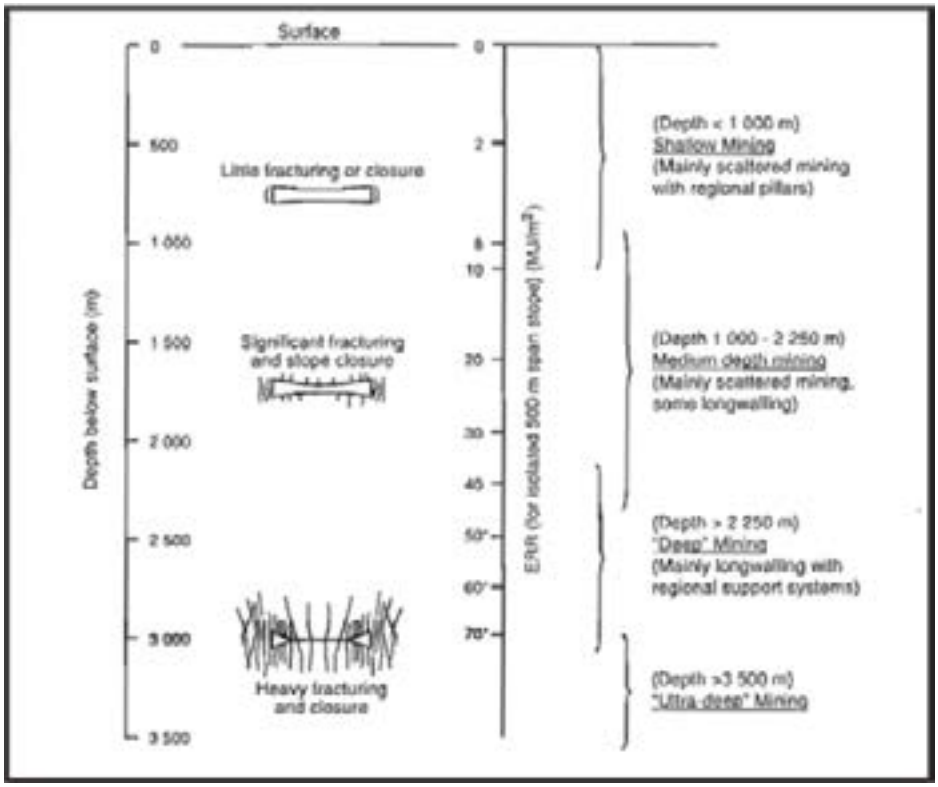


Figure 1: Depth related conditions (Jager and Ryder, 1999)

LEARNING OUTCOME 3.1.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE BEHAVIOUR OF THE ROCKMASS AROUND TABULAR EXCAVATIONS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Shallow mining conditions

Shallow depth environments are characterized by the existence of large tensile stress zones (areas where bed separation occur) in excavation hangingwalls (Figure 2).

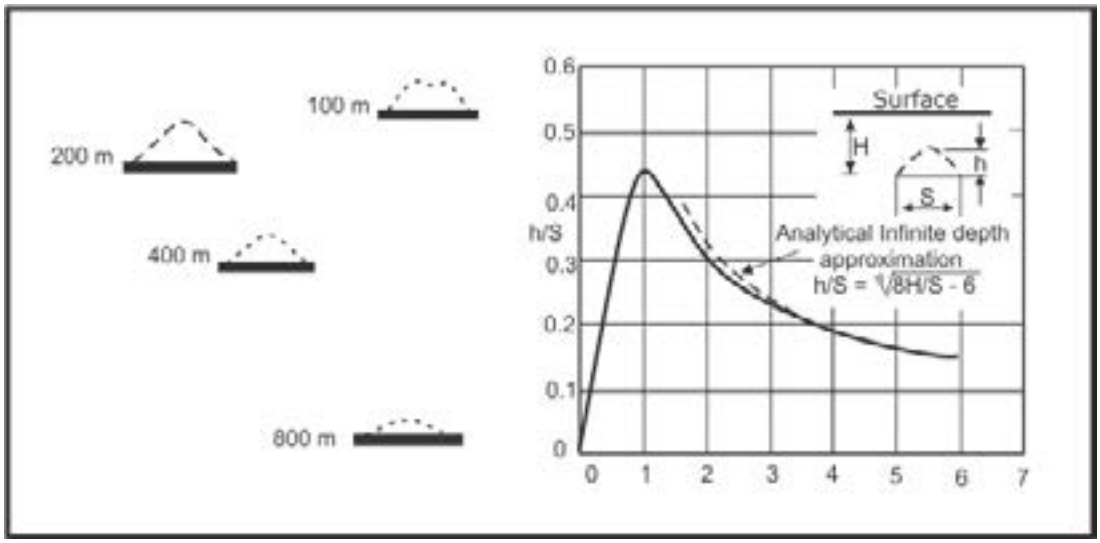


Figure 2: Tensile zone above shallow stopes with little or no in-panel pillar protection. In (a), 200 m span stopes and (b) tensile zone heights h are plotted normalized with respect to the mining span S (Jager and Ryder, 1999)

Virtual absence of face fracturing and relatively high and sometimes highly variable virgin horizontal stresses (represented by the k-ratio) is also common in shallow depth excavations.

NOTE

The k-ratio is the ratio between the horizontal and vertical stress components, i.e. k=σ_h/σ_v .

REFERENCE

- Pages 42- 45 TR Stacey Best practice rock engineering handbook for "other" mines SIMRAC Jhb chapter 7

As a result, potential failures are defined by relatively widely spaced tensile fractures or opened-up geological discontinuities (often joints or weak semi-horizontal planes), and can take the form of hangingwall beam failure or keyblock dropout, followed by large scale hangingwall unravelling or even massive stope backbreaks.

CONNECTION 8

Refer to Paper 1, Chapter 5 on page 148

EXAMPLE

Highly variable k-ratios are reported in the western limb of the Bushveld Complex where ratios can change over short distances, where:

- The on-reef k-ratio in one area is given as 0.65 while the off-reef k-ratio is 1.53;
- Neighbouring mines reported k-ratios of between 0.8 and 2.4 (North-South) and 2.1 and 3.4 (East-West);

INTERESTING INFO

The concept of 'clamping' of hangingwall blocks are explained by referring to figure 3:

When a rock fractures, is has undergone a certain amount of deformation. At the formation of the crack (or fracture), the integrity of the material is broken and 'separation' (also called dilation), even though small, is caused along the crack / fracture. This 'separation' induces a horizontal force in the beam that actually results in the 'locking' of the blocks into place. It is similar to the well-known timber block game children play where their ability to pick up a large number of blocks is tested. The larger force exerted on the blocks, the greater the stability that can be maintained.

Deep mining conditions

At deep levels, tabular excavations are subjected to high compressive stresses. These compressive stresses deform the rock mass around the excavations. Deep level tabular excavations experience the following physical phenomena: dense rock fracturing, strain bursts / seismic activity and high levels of stope closure.

The highly fractured rock forming the hangingwall to the tabular excavation experience 'clamping' of the blocks formed by the stress fractures (caused by dilation) and are virtually self-supporting under static conditions.

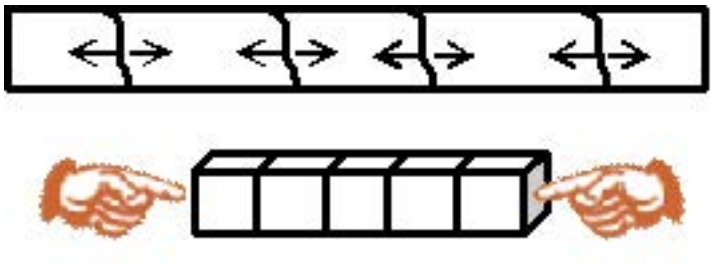


Figure 3: (a) 'Clamping' forces caused by stress fracturing (b) children's block game

Seismicity associated with ruptures on geological structures as well as face-bursts in some environments, is common.

Intermediate mining conditions

Intermediate depth excavations experience similar rock mass behaviour to deep level excavations, but to a lesser degree. Stress conditions range from low to moderately high, with k ratios generally ≤ 1 and the excavations are normally surrounded by an envelope of fractured rock. As a result, compressive horizontal stresses are induced in the immediate stope hangingwall, dilating and assisting in stabilisation of the strata. The friction on the block contacts and the orientation of the block surfaces all affect the level of stabilization induced.

Parameter	Shallow	Intermediate	Deep	Ultra-deep
Typical depth(m)	< 1000	1000 – 2250	2250 – 3500	> 3500
Typical ERR(MJ/m2)	< 8	8 – 40	40 – 80	> 80
Vertical virgin stress(MPa)	< 25	25 – 60	60 – 95	> 95
Stress fracturing	Little/None	Moderate	Deep	Very deep
Stope closure(mm/m advance)	Low (<10)	Moderate (10-30)	High (30-60)	Very High (>60)
Infl. Of geology on h/w stability	Strong	Moderate	Moderate	Moderate
Possible extent of FOG's	Can be large	Often small	Usually small	Small
Rockburst hazard	Minimal	Moderately Severe	Severe	Very severe

Table 1: Values of parameters associated with different mining depths (Jager and Ryder, 1999)

REFERENCE

- Pages 10, 16 to 20, 29 to 31 Jager A.J. and Ryder J.A., A handbook on rock engineering practice for tabular hard rock mines
- Pages 42-45 Jager A.J. and Ryder J.A., A handbook on rock engineering practice for tabular hard rock mines

LEARNING OUTCOME 3.1.2

DESCRIBE, EXPLAIN AND DISCUSS THE CHANGING AMOUNT AND NATURE OF ROCK THAT NEEDS TO BE SUPPORTED AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

CONNECTION 9

Also refer to Figure 2: on page 63 in Outcome 3.1.1

The rock mass around shallow depth excavations is not highly stressed, in fact, the hangingwall is subjected to tensile vertical and low level compressive horizontal stress levels. Although the horizontal stress levels are much larger than the vertical, i.e. high k-ratio, the actual stress levels are still low. In this rock mass, fewer stress induced fractures compared to deep rock masses are present resulting in the absence of dilation or horizontal clamping (due to the forming of stress fractures). Due to this stress environment, the dead weight of the rock mass above a stope must be supported in shallow areas. This requires substantial forces to be generated by support units tasked with maintaining stability, resulting in the use of support types such as pillars, backfill, cement packs, etc.
As the mining depth increases, the rock mass becomes more and more self-supporting to a certain extent due to the build-up of clamping forces, caused by the formation of stress fractures. This self-supporting nature of the rock mass allows a reduction in the artificial support units' contribution to maintaining stability. The responsibility of support units in deeper environments change towards a 'control of the rock mass immediately around the excavation' rather than supporting the complete rock mass, as in shallow areas. It is often referred to as control of the 'skin' of the excavation. Support units utilised in this environment can thus include lower force generating units such as elongates.

REFERENCE

- Pages 31 to 38, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.1.3

DESCRIBE, EXPLAIN AND DISCUSS THE DIFFERENT RATES OF DEFORMATION OF THE ROCKMASS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

CONNECTION 10

Refer to LEARNING OUTCOME 3.1.1 on page 62

Prior to the creation of excavations, virgin or primitive stresses exist in the rockmass due to the weight of the overburden and often also due to various events in the geological history of the rockmass. This stress field sets the scene for any closure rates that may occur when excavations are eventually created, as it drives the rock mass behaviour.
At shallow depths, rock masses are subjected to very low stress levels, where the horizontal stress is often larger than the vertical stress (k-ratio > 1). Large zones of tensile stress are created around excavations due to the absence of fracturing caused by high compressive stress levels. Elastic rock mass behaviour will see very little bending of the rock mass into the mined out excavations (low closure rates), since the stresses driving the elastic behaviour is low. However, inelastic behaviour in these tensile zones are characterized by bed separation on geological structures such as joints or bedding planes and, due to the absence of large stress levels,

can result in sudden and massive 'sit-downs' in tabular excavations. Deep level excavations should experience high levels of stope closure, both elastic and inelastic, as stress levels that drive both behaviours, are often very high. Due to normally high vertical stress levels, the rock is strained and reduces in dimension in the vertical direction, but is able to extend (become larger) in the horizontal direction, as the excavation provides space for the rock to deform into. This increases sidewall and face closure rates, but also contributes to the high hangingwall closure rates in tabular stopes. Due to the high stress levels ahead of the mining face, the rock is fractured and results in dilation. The dilationary stresses are created in a horizontal direction in the beams above the mined out tabular stope and forces the beam to bend downward into the mined out stope (Figure 4). The higher the face stress, the more fractures are created, increasing the dilation and also increasing the closure rate of the hangingwall. Deep level excavations undergo stope closure that often results in complete stope closure (hangingwall touching the footwall).

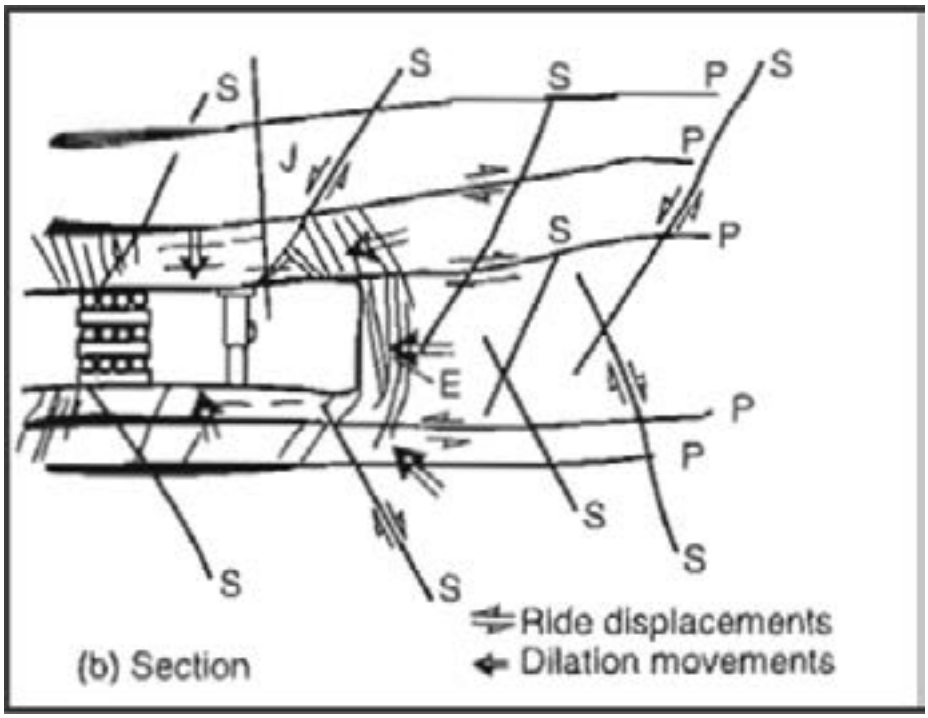


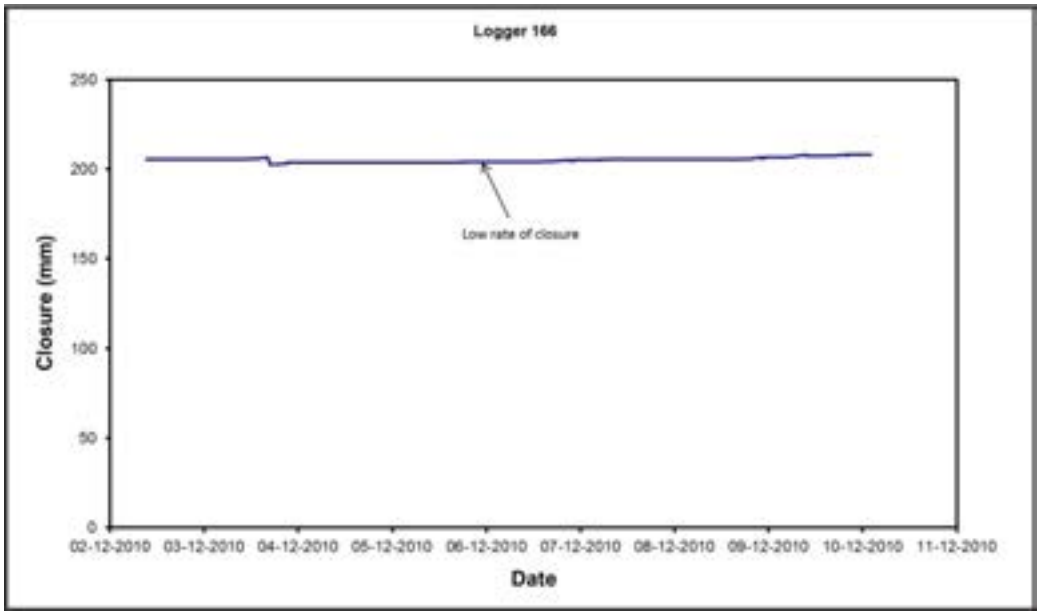
Figure 4: Dilation occurring into the mined out area

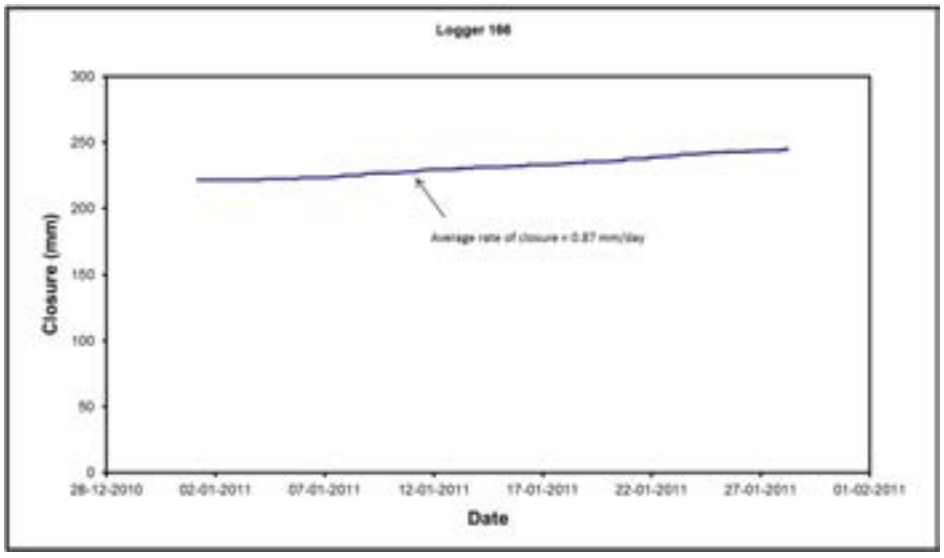
Intermediate depth excavations imitates the closure behaviour seen in deep excavations, but reports lower values closure rate due to lower stress levels. Intermediate level excavations undergo stope closure, but complete stope closure is normally absent.

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EXAMPLE

The measurement of total excavation closures, which include elastic and inelastic deformations, for excavations at different mining depths, are shown in Figure 5 (a) to (c).





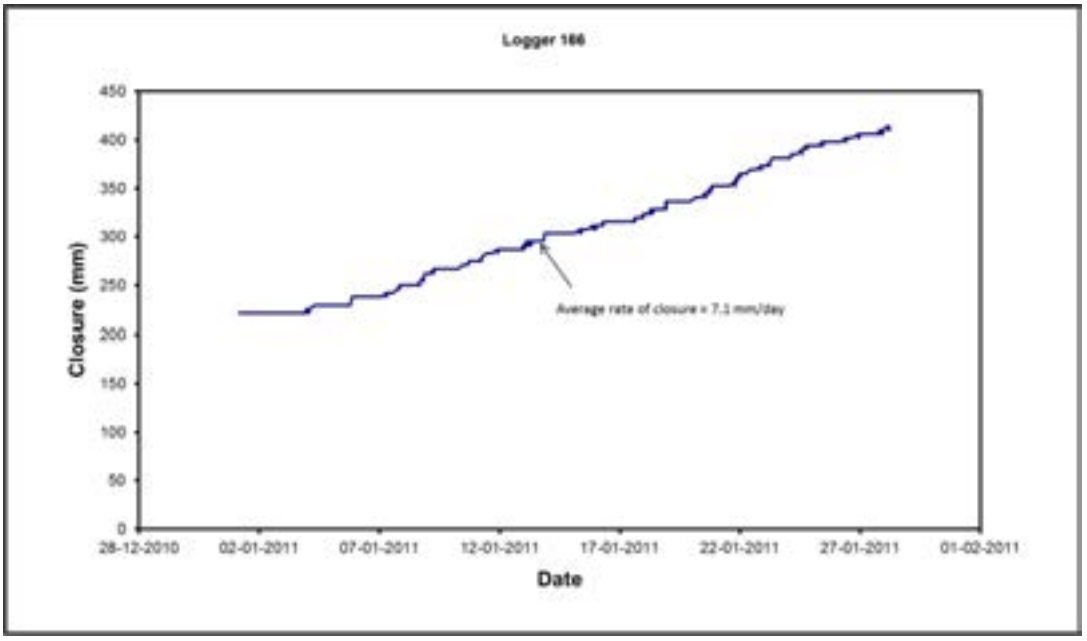


Figure 5: Typical rate of closures for different mining environments; (a) Indicate shallow workings with little to no closure, (b) indicate moderate closure and (c) indicating high closure rates in deep environment.

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It is important for future discussions that the differences between actual closure and rate of closure is clearly understood.

- *At any point is time, hangingwall closure that had occurred in two similar sized excavations can be similar, but the closure level has been reached in a shorter time period in excavation A, suggesting that the closure rate is higher in the excavation A environment.*
- *When closure is measured close to the advancing mining face in a tabular excavation, the actual closure would be small in relation to that seen in the back area. However, the rate at which closure occurs close to the mining face is higher than the rate in the back area. This suggests that an increase of 10% in closure would occur faster (shorter time period) on the face than in the back area (refer to the slope of the closure graph in Figure 6)*

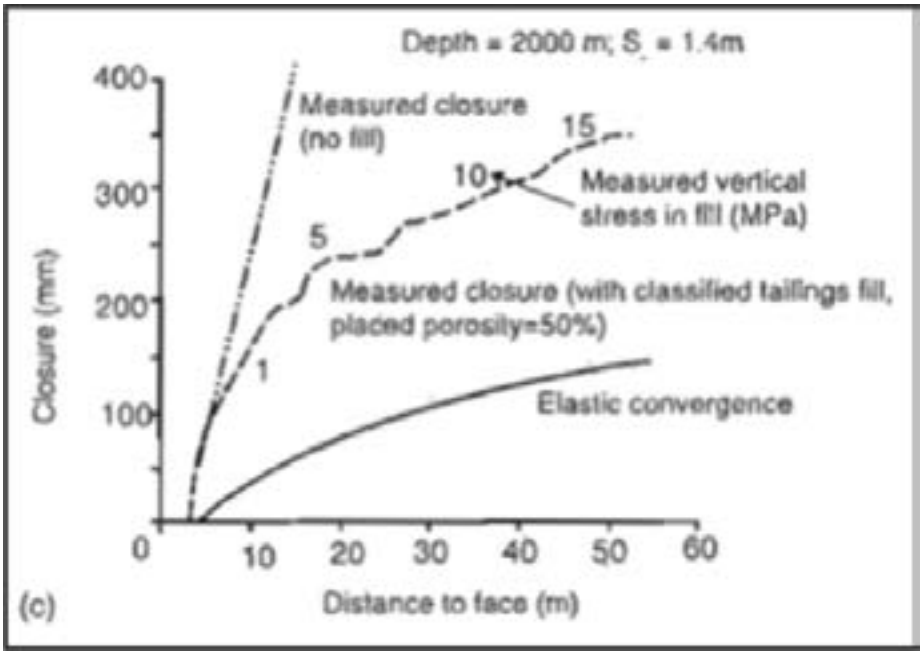


Figure 6: Closure and closure rates at different distances from the face

- Pages 18 to 20, Jager J.A. and Ryder A.J. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 5, Budavari S, Rock mechanics in practice, SAIMM, 1983.

LEARNING OUTCOME 3.1.4

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE TYPES OF FRACTURES FOUND AROUND STOPE FACES AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Types of fractures as well as the frequency of the fractures are greatly affected by the depth at which the excavation is mined. At shallow depths fracturing is not as predominant as that seen at deeper depths since stress levels are lower and often tensile. However, it is the stress-to-stress ratio that affects the occurrence of stress fracturing since a very weak material could fracture at very low stress levels.

Mining Depth Classification	Types of Fractures	Reason for Fracturing	Effects
Shallow	Widely spaced tensile fractures are more common place at shallower depths Very little or no face fractures are often visible in the stopes Fracturing in hang-ingwalls of tunnels is possible in some materials (Norite)	Extended tensile zone due to deadweight from surface High levels of virgin horizontal stress	Hangingwall beam failure Keyblock failures as a result of opened up fractures Large scale stope back breaks
Intermediate	Excavations are surrounded by an envelope of fractured rock Some stress induced fracturing ahead of the face of the excavation, mainly extension fractures	Moderately high stress conditions with a k-ratio ≤ 1 Compressive hori-zontal stresses in the immediate hangingwall and footwall due to stress conditions	Significant closure is observed.
Deep and Ultra Deep	Excavations that are not overstoped are often surrounded by extensive frac-ture zones. Stress fractures exist of extension fractures as well as shear frac-tures cause by high levels of shear along the mining faces.	Highly stressed environment Clamping forces in the hang-ingwall make the rock basically self-support-ing in static conditions	High levels of clo-sure. Complete closure of back areas is common place. High levels of seismic-ity leading to rockburst problems especially around geological intrusions as well as remnant areas

Table 2: Classification of mining depths and fracture associated.

CONNECTION 11

Refer to LEARNING OUT-COME 3.1.1 on page 62

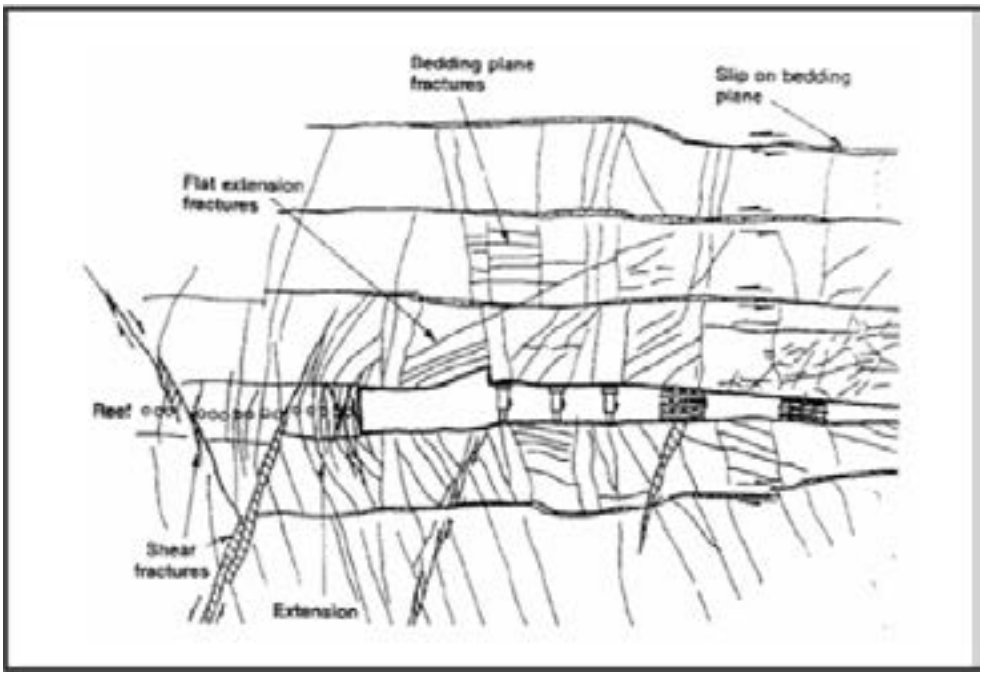


Figure 7: Typical fracture patterns around a deep level gold mine stope (COMRO, 1988)

DEFINITION

- Extension fractures: A fracture that develops perpendicular to the direction of greatest stress and parallel to the direction of compression
- Shear fractures: A fracture resulting from shear stress
- Tensile fractures: A minor rock fracture developed at right angles to the direction of maximum tension. Also known as subsidiary fracture

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- Chapter 2, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Page 144, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.1.5

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF STOPE FACE FRACTURING ON THE STABILITY OF THE STOPE HANGINGWALL, STOPE FOOTWALL AND STOPE FACE

CONNECTION 12

Refer to LEARNING OUTCOME 3.1.1 on page 62

Hangingwall and footwall

Face fracturing occurs mostly in intermediate to deep level environments but may be experienced under certain conditions around shallow depth excavations. Horizontal dilational stresses are created by the occurrence of face parallel fracturing and stabilises the hangingwall by forming a coherent self-supporting beam and locking rock blocks into place.
Without the presence of this self-supporting beam, hangingwall failure may become a common occurrence in underground excavations. Since inelastic behaviour in the stope hangingwall is exacerbated by gravitational acceleration, the hangingwall sag is more pronounced, creating the impression that the footwall is static. This, however, is not correct as compressive, horizontal stresses are induced in both the stope hangingwall and footwall and result in high levels of closure due to footwall heave as well as hangingwall sag.
Hangingwall and footwall stability only poses significant problems in situations where geologicaly disturbed ground/incompetent ground is found and where flat dipping mining induced fracturing has occurred.

Stope face

The stope face at intermediate depth, which is surrounded by a fracture envelope, may initiate rock bursts within the fractured zone. These rockbursts (or face bursts) can be created by any or combinations of the following:

- Due to 'interlocking' or prevention of shear displacement on stress fractures. This 'inter locking' occurs due to shear displacements that occur on stress fractures ahead of the face, were the shear displacement is driven by the high face stress levels. While shear displacement occurs on a number of fractures, some blocks are prevented from displacing and a stress build-up occurs very close to the mining face. When the stress exceeds the strength of the 'interlocked' fractured mass, violent failure occurs in the form of a face burst.
- The existence of the fractured material immediately ahead of the face, confines the rock further into the rock mass, increasing its strength and thus preventing it from fracturing. This causes the 'stress peak' (Figure 8) to move closer to the mined out face area and when the fractured rock is removed by blasting, the confinement is released, the rock strength reduces and the fracturing is created immediately. Often this fracturing occurs violently, resulting in a face burst.

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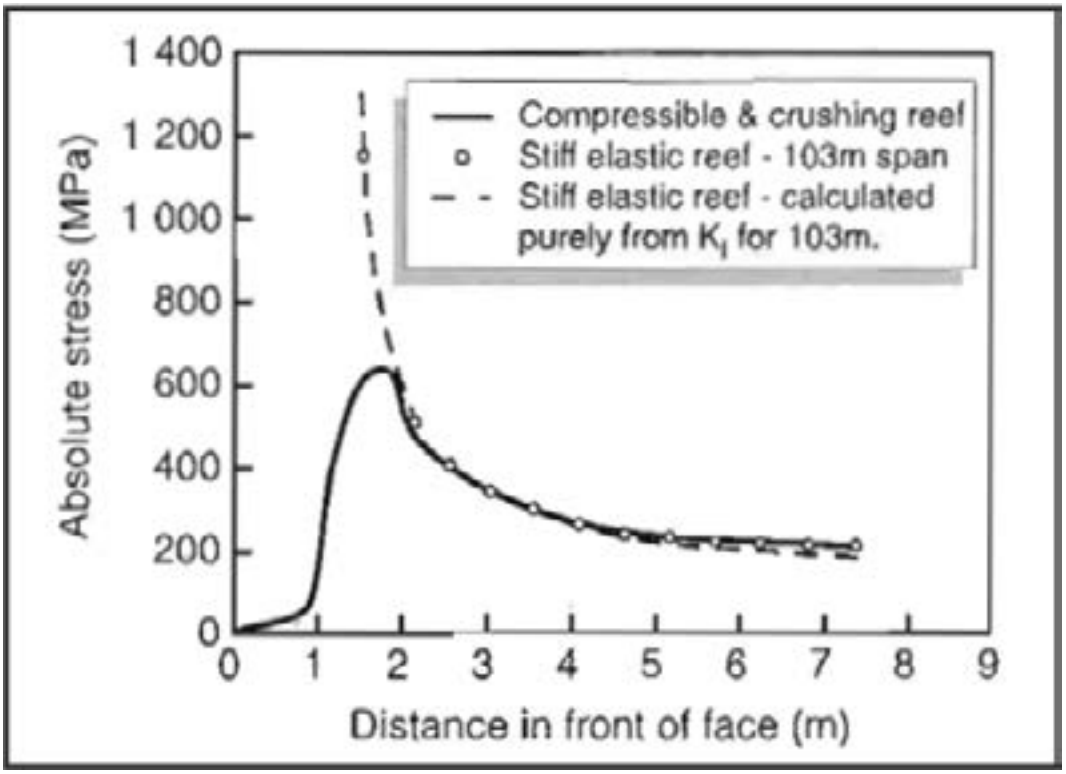


Figure 8: Stress peak ahead of the mining face (Jager and Ryder, 1999)

The stress distribution ahead of the mining face shown in Figure 8 suggests:

- *Low stress immediately in front of the face due to the severely fractured rock's inability to sustain high stress levels;*
- *Stress peak ahead of the face due to stress concentrations around mined out areas and*
- *Drop in stress levels towards virgin vertical stress levels far ahead of the mining face.*

- Page 48, Jager J.A. and Ryder A.J. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.1.6

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE TYPES OF FRACTURES FOUND AROUND STOPE GULLIES AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Stope gullies form vital access routes in 'scattered and longwall mining methods' to mining faces and are generally expected to have long lives (1-2 years) with high levels of through-put in terms of rock/ore, men and material to and from the working face. Understanding the effects of mining on these ends is crucial to ensure correct placement and support installation.

Fracturing around the gully is largely affected by the shape, size and position of the gully and the siding layout (depth, position, shape), but also by blasting practices.

Stability of the hangingwall is greatly affected by discipline during the blasting cycle (drilling operations, charging of explosive and timing of blast holes). A poor blast would create more than desired fractures and with these fractures propagating into the hangingwall, poor conditions

are created. When these blast fractures then intersects the already present stress induced fractures, blocky ground are created, which are very difficult to effectively support. The blocky conditions also reduced the hangingwall's natural ability to support it-self as unravelling often overmine support units.

At shallow depth, stress induced fracturing may be noticeably absent due to the relatively low levels of stress. In these cases, simple gully layouts can be used without affecting the stability of the gully. In shallow depth designs where in-stope pillars are used, low angle stress fracturing may, however, occur due to increased stress levels on the pillars. These fractures may align with parting of stratification planes and may extend over the gully. Fracturing at this depth does not often extend deep into the hangingwall and relatively standard forms of support such as roof bolting can be used to stabilise the gully.

At intermediate and large depths, stress induced fracturing around the gully is significant and a bit more complex in that many factors affect the fracture orientations. Stress levels are high enough to create fracturing which is a natural result of mining at these depths, but it is the orientation of the fractures that cause instability and that can be controlled.

CONNECTION 13

Refer to LEARNING OUTCOME 3.1.7 on page 75

In plan view, Figure 9 shows that stress fractures follow the mining abutment and are steep dipping when the abutment is straight (as along a face), but reduce in dip angle as soon as the abutment turns (as around the edge of a panel).

NOTE

Stability of flat dipping fractures is difficult to maintain and should be avoided as far possible.

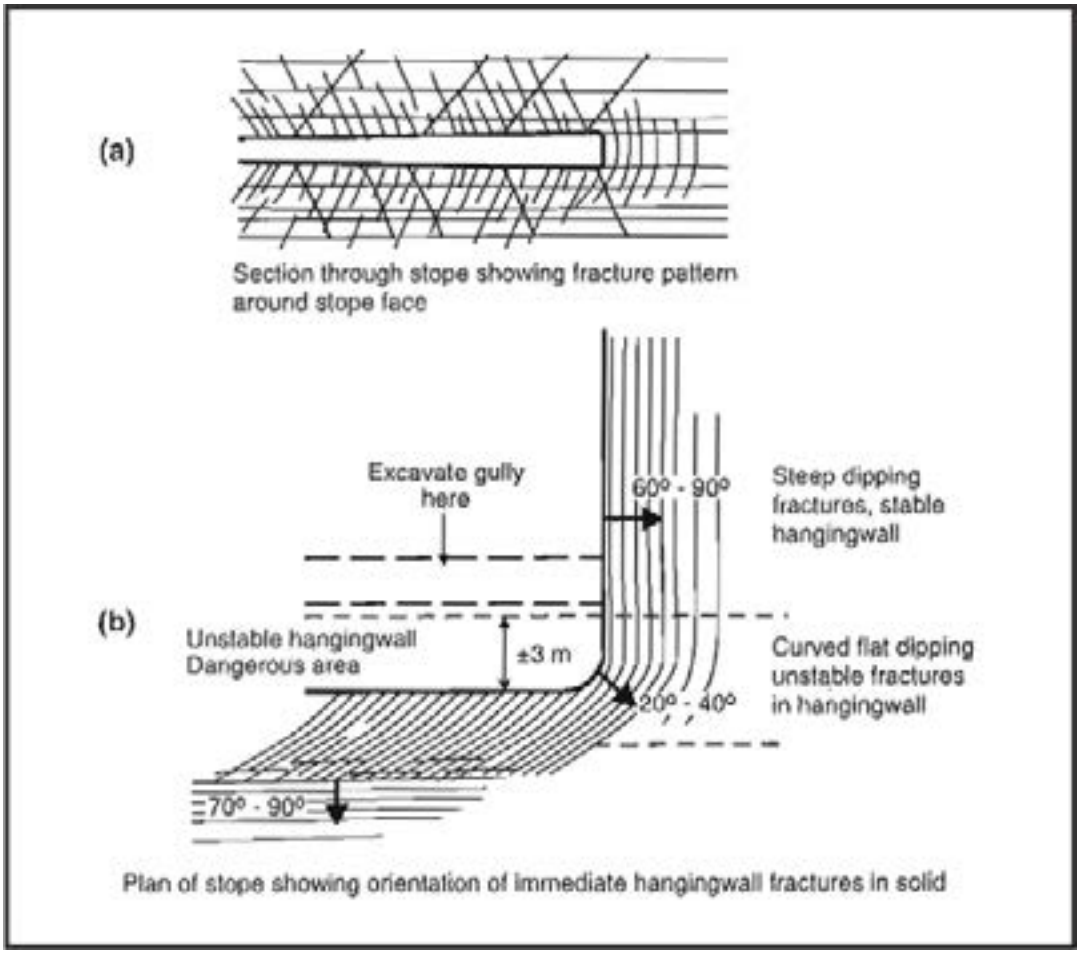


Figure 9: Fracturing around a gully (Jager and Ryder, 1999)

In section view, fracturing around a gully that leads the mining face takes the form of 'onion rings' (Figure 10) and create stress fractures in the hangingwall and sidewalls of the gully that improves potential for scaling (wide gullies result from sidewall fracture scaling) and instability (hangingwall falls of ground due to flat dipping fractures).

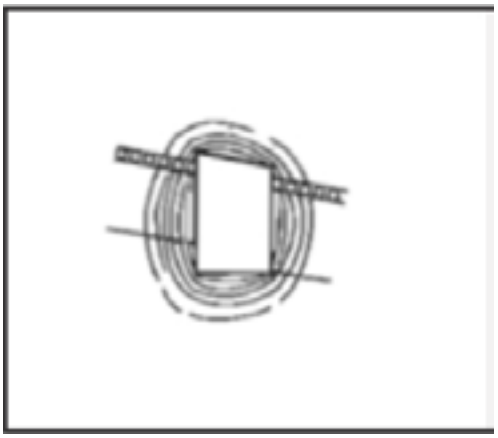


Figure 10: "onion ring' stress fractures around leading gullies (Jager and Ryder, 1999)

CONNECTION 14

Refer to Gap 602 Gully and sidings

NOTE

The following presentation was drafted on an intermediate depth mine, where different gully layouts (as those depicted in Figure 11) resulted in different stress fracture patterns and levels of instability. The presentation should be viewed to complete the understanding of the stress fracture creation around gullies.(Courtesy of Tony Steyn)

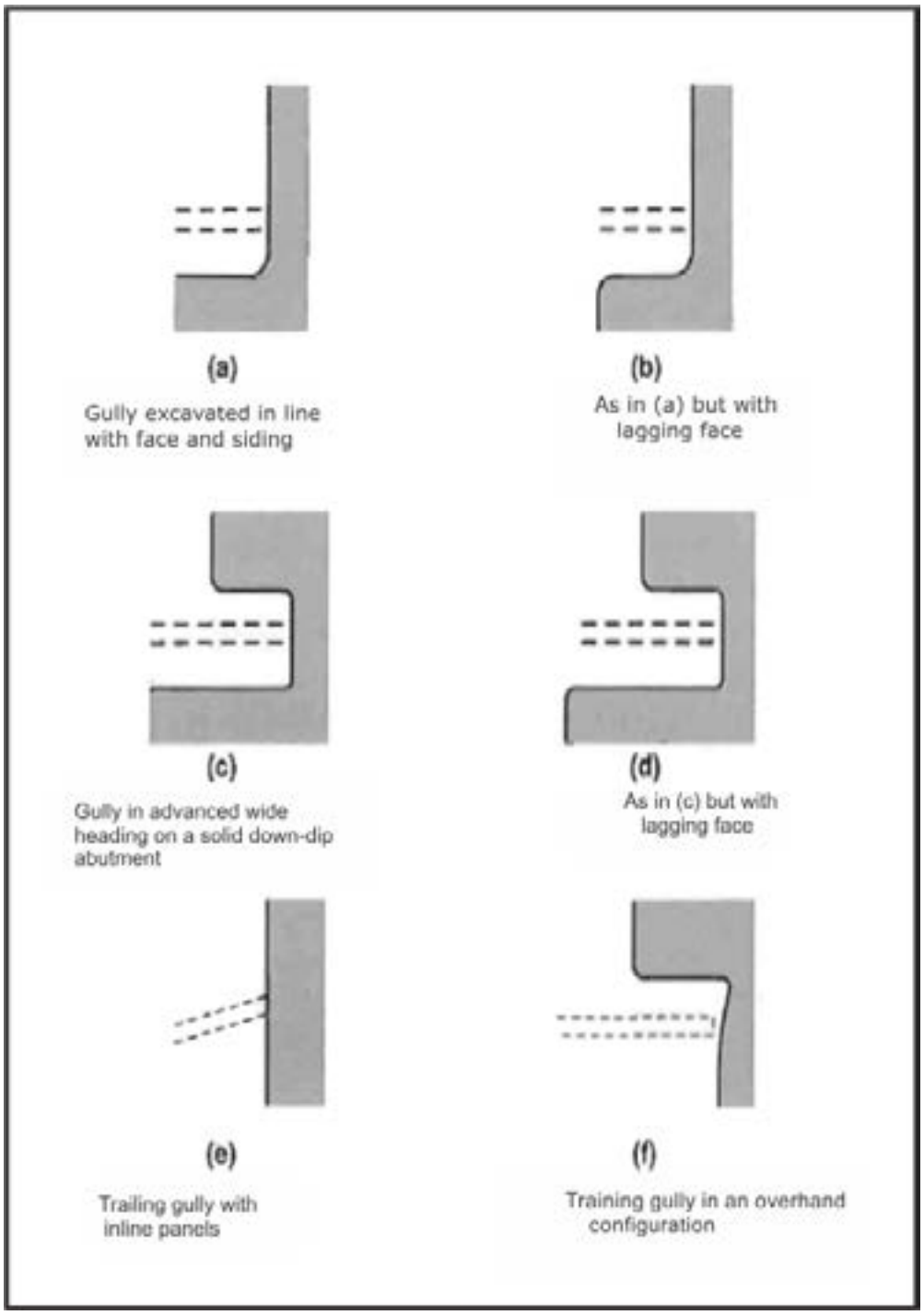


Figure 11: Different gully layouts (Jager and Ryder, 1999)



- Pages 94 - 101, Jager A.J. and Ryder J.A. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Steyn, A, Presentation created for Great No-ligwa, AngloGold-Ashanti.
- Page 144, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.1.7

DESCRIBE THE EFFECTS OF STOPE GULLY FRACTURING ON THE STABILITY OF THE GULLY HANGINGWALL AND SIDEWALLS

CONNECTION 15

Refer to LEARNING OUTCOME 3.1.6 on page 71

As stress levels into which the gully is mined increases, fracturing will become more dense (i.e. smaller spacing) and result in potentially unstable ground leading to falls of ground.

As the stresses need to pass around corners of the mining excavation, the dip orientation of the principal stress component tends to flatten, whereas the orientation of the same stress component along a face is steeply dipping.

EXAMPLE

The angle of stress fractures can be viewed as a person riding a motorcycle. If he is on a straight, he will sit upright, whereas if he would like to negotiate a turn, he will lean into the curve.



Figure 12: Position of motorcycle driver when driving straight and negotiating a turn

These stress orientations yield stress fractures that are formed parallel to the major principal stress with (obviously) similar dips to that of the stress components.

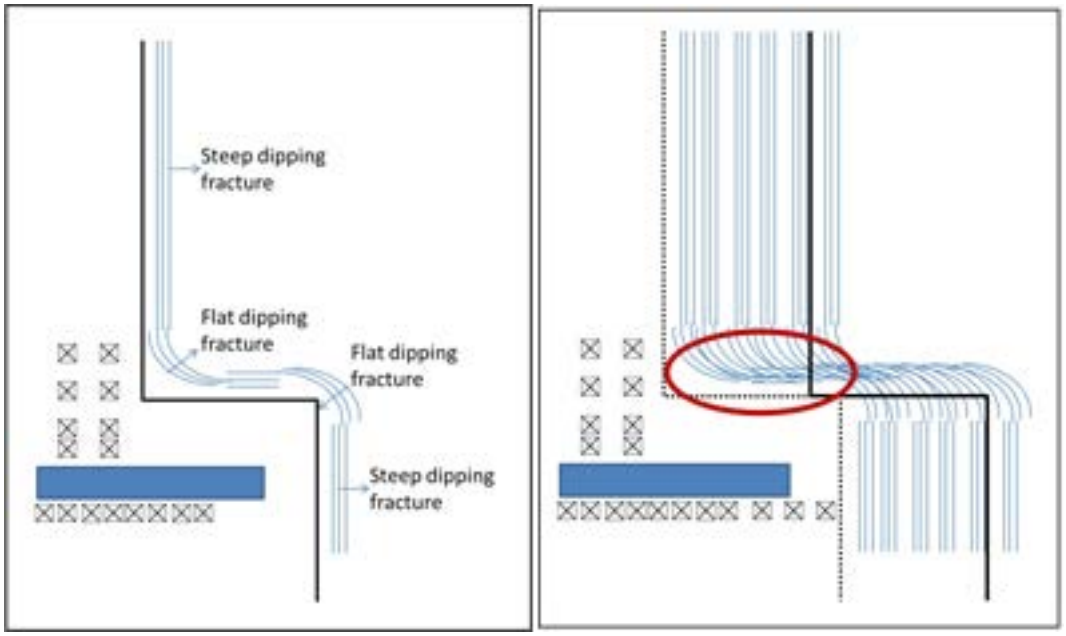


Figure 13: Fracturing ahead of the face (a) and interaction on advance (b)

back

The interaction of these fractures could result in unstable and blocky ground conditions as steep and flat dipping fractures overlap (Figure 13).

- Pages 94 - 101, Jager A.J. and Ryder J.A. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Page 144, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.1.8

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE BEHAVIOUR OF ROCK SURROUNDING TABULAR EXCAVATIONS DURING SEISMIC EVENTS

Whereas rock mass behaviour until now has been viewed as quasi-static, resulting in the conditions discussed in the outcomes above, it is clear that when dynamic displacements are induced, the rock mass behaviour should change in certain ways. It is known that the seismic waves are transmitted from the seismic source in a radial fashion (i.e. radially outward in all directions) and that these waves are changed due to refraction (change when entering another medium, i.e. different rock type) and reflection (bounces off prominent structures and excavations). This understanding of waves being reflected and refracted and intersecting mining excavations from different directions, different velocities and intensity, would suggest that the interaction of these waves with excavations must be a difficult issue to fully understand.

It is for this reason that the complete interaction has been researched and simulated using experimental blasts, but is still not completely understood.

This outcome will suggest some basic effects of the impact of the dynamic rock mass movement on the rock mass. Table 3 suggests some effects and behaviour of rockmasses during different seismic events.

Seismic Event	Postulated damage mechanism	Richter magnitude ML
Strain-burst	Superficial spalling with violent ejection of fragments	-0,2 to 0,0
Buckling	Outward expulsion of large slabs pre-existing parallel to surface of opening	0,0 to 1,5
Free crush / pillar burst	Violent expulsion of rock from stope face or pillar sides	1,0 to 2,5
Shear rupture	Violent propagation of shear fracture through intact rock mass	2,0 to 3,5
Fault-slip	Violent renewed movement on existing fault or dyke contact	2,5 to 5,0

Table 3: Description of different seismic events and their source mechanisms (Jager and Ryder, 1999)

CONNECTION 16

Refer to "3.4 Mine seismicity" on page 128

In essence, the rock mass is subjected to 'shaking' when a seismic wave interacts with a stope, of which the intensity of the shaking varies quite significantly. This shaking causes all 'loose' material to dislodge immediately, but also induces shear displacements on many of the existing fracture and geological planes. This again, may result in the loss of keyblocks or disintegration of rock or may even eject rocks from their original position in the hangingwall or sidewalls (faces) of tabular excavations.

These results are better understood when it is accepted that a seismic wave is also a 'strain wave' in that the wave deforms and displaces the

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rock mass particles it intersects. Any weakness plane within the affected rock mass will allow increased displacement while competent material may be strained to a point of failure, creating new fractures and decreasing ground conditions.



The presence of 'rock flour' (white powder) on structures (fractures, joints, faults, etc) indicates the occurrence of substantial displacements on the structures. Ejections are visible through rocks (usually small pieces) lying on the excavation footwall, often some distance away from the affected area.

Support installation may have some effect on parts of the resulting behavior, especially where support is utilised to 'control' the excavation perimeter and where the energy absorption capacity of the support (ability to generate load whilst undergoing large deformations) is sufficient. The single support medium that seems to affect the behavior of the whole rock mass when subjected to seismic activity is backfill.

The placement of backfill in tabular stopes appears to have the following impacts on rock mass behaviour when subjected to seismic waves:

- Reduces the length of the beam between support and face that can vibrate (shorter beams vibrate at higher frequency but smaller displacements, reducing potential for large displacements and disintegration of the hangingwall beam);
- Increases the resonant frequency of the hangingwall beam (as mentioned above);
- reduce the differential movements between hangingwall and footwall (which is normally seen as 'ride' and makes impact on other support units less severe);
- damps ground movement (reduces the vibration time by damping the viibrations) and
- reduces the presence of tensile stresses in the hangingwall (reduces the availability of rocks that are in a favourable position to be destabilised).



- Hangingwall behavior in tabular stopes, Kirsten H.A.D et al
- The effect of backfill on ground motion in stopes during seismic events, Hemp D et al.
- Page 241, Budavari S, Rock mechanics in practice, SAIMM, 1983
- Pages 108, 163, 186, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 178, 267, 346, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.

LEARNING OUTCOME 3.1.9

DESCRIBE, EXPLAIN AND DISCUSS TYPICAL HEIGHTS OF ROCKFALLS IN TABULAR UNDERGROUND GOLD AND PLATINUM MINES IN SOUTH AFRICA



- Rockfall height can be defined as the thickness of the rock that dislodged and collapsed into the mining excavation. In most cases, this height should conform to the distance between the excavation wall and the furthest known unstable parting plane in the hangingwall.
- Rockfalls occur when gravitational forces (due to the weight of the rock) overcome the friction and cohesion on the surrounding rock surfaces.

- Rockfall height should not be confused with rockburst height or ejection height defined as the thickness of rock that is / or can be ejected from the excavation walls under seismic conditions.



The Rockfall height or ejection height is used (depending on which is more appropriate for the mining environment) to determine support requirements in terms of Support Resistance of Energy Absorption Capacity.

Rockfall heights in South African platinum and gold mines can vary from 0.2m to as large as 3.0 – 4.0m in some areas where domes are present (platinum mines). Statistics vary so vastly that data from a particular mine or GCD (Ground Control District) / Geotechnical area would prove more valuable than that obtained from an entire industry. In new areas where this kind of data is not available, statistics from surrounding mines at similar depth may be used. Some typical values are shown below for interest.

Reef type	Fall-out height (m)	Ejection height (m)
Carbon Leader	1.0	2.2
VCR	1.2	1.8
Vaal Reef	1.2	1.2
Basal Reef	1.8	2.6

Table 4: Typical fall-out heights at 95% frequency level (Ryder and Jager, 2002)



The 95% fallout criterion (for rockfalls) and the 95% ejection thickness criterion (for rockbursts) was developed by MKC Roberts after the analysis of a rock related fatal accident database developed within the SIMRAC project GAP032. In this study Roberts found that the inclusion of 95% of all fatal accident data, actually just excluded those fatal accidents where the mining layout had a direct impact on the fall of ground or rockburst ejection thickness and appeared to be the main driver. As would be expected, the largest falls were often affected / created by the mining layout and would thus be seen as controllable by layout rather than by support. A 95% fallout thickness criterion based on a fatal accident database will not necessary be the same as a 95% fallout thickness criterion based simply on fallouts that occurred on the mine.

The use and analysis of the data in a database must therefore be done carefully and it is suggested that the practitioner first ensure that all data that represents layout driven falls are removed from the database. Then a decision must be made on whether designs would be done to a frequency of > 95% and at what level it should be done. In determining this final frequency level, it is suggested that the geotechnical environment is also analysed and that both the frequency level and the geotechnical environment study is used to make an informed and technically sound decision.

For example: If 98% of the fallouts suggest a fallout height of 1.5m but at 1.8m a very prominent plane exists above the reef, the practitioner should increase the fallout height to 1.8m, independent of the frequency level.



The setting of criteria for support design is one of the most important functions of a rock engineer on a mine. Determining the criteria must be a technically sound and technically defensible process.

EXAMPLE

By recording and graphically displaying all dimensions related to falls of ground that has been reported, it becomes easier to evaluate the data and decide on an acceptable fall-out height to be implemented for a particular area. The graph below indicates that a 95% criterion (suggesting that 95% of all falls have dimension less than those quoted) was deemed appropriate for this area and that it yields a fall-out height of 1.0m, with length and width dimensions of between 4m and 5.2m.

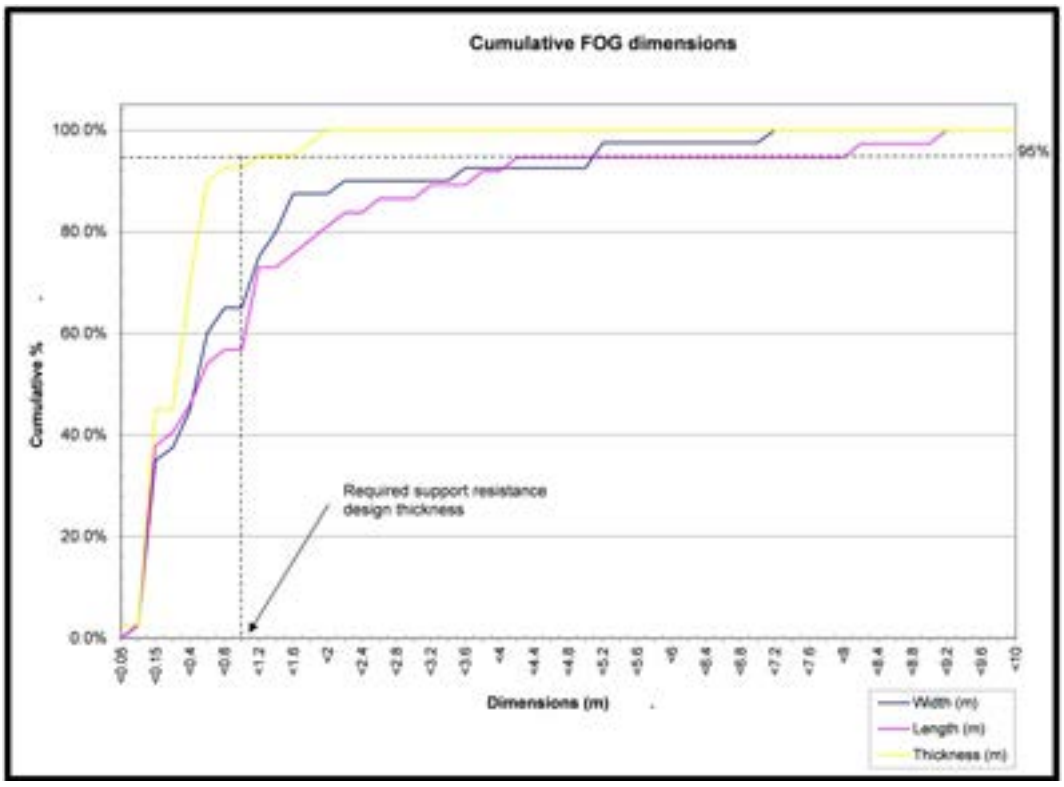


Figure 14: FOG dimension results for a specific area / mine

REFERENCE

- SIMRAC report Gap 416 on geotechnical areas.
- Pages 148, Jager A.J. and Ryder J.A. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 307, Ryder JA &Jager AJ 2000 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.

L EARNING OUTCOME 3.1.10

DESCRIBE, EXPLAIN AND DISCUSS THE SIGNIFICANCE OF ROCKFALL HEIGHT IN TERMS OF SUPPORT REQUIREMENTS IN GRAVITY LOADING CONDITIONS

In quasi-static conditions, shallower mines generally have a higher fall out height compared to deeper mines, mainly due to the clamping (horizontal clamping) offered by dilation at depth. In most cases, the occurrence of seismic activity also increases the expected fallout heights.
As the height of the falls are recorded, a database is established which will indicate the distribution of the fallout heights (example in Outcome 3.1.9) and could allow the selection of an appropriate value.
If one would consider 1m2 area, the weight of a block of rock that would need to be supported will be determined by its thickness. If the fallout height increases, a higher loadbearing support unit will be required to support the larger weight of the rock block. On the other hand, given the same support unit, the area of the block must be reduced (i.e. < 1m2), suggesting a reduction in support spacings with increasing fallouts thickness.

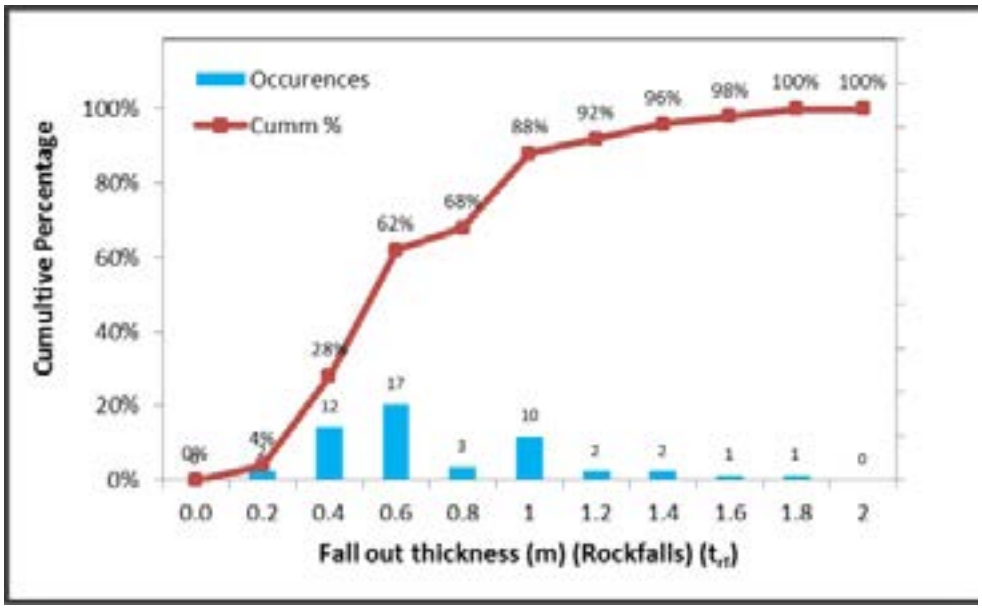


Figure 15: Cumulative rockfall thickness for a particular ground control district (95% at 1.4m height).

EXAMPLE

The rockfall-out height is the thickness of strata that will potentially fall out, if not supported and is depicted as trf. The standard support system (after analyses of all related data) should prevent 95% of all fallouts trf (refer to figure above) and data show that 95% of the fallout thickness occurs at 1.4m or less.
The thickness of strata to be supported would therefore be trf = 1.4m.
The density of the rock is assumed to be 2750 kg/m3 and
the gravitational acceleration is taken as 9.8 m/s2.
The support resistance criterion required to support the above mentioned would be:

Support resistance criterion = (ρgh)
= 1.4m x 9.8 m/s2 x 2750 kg/m3
= 38kN/m2.

Under gravitational loading conditions, the support installed therefore has to generate 38kN per m2 supported, to be able to keep a rock with height 1.4m in unstable equilibrium (Unstable equilibrium means that the support resistance is just enough to maintain equilibrium but that any small change will result in instability).

NOTE

To counter this concern of 'unstable equilibrium', safety factors are usually applied to increase the suggested support resistance criterion and ensure stability.

Support resistance criterion = (ρgh) x FOS
= 1.4m x 9.8 m/s^2 x 2750 kg/m^3 x 1.5
= 57kN/m^2.

the selection of the FOS is subject to various considerations. Commonly

factors of safety of between 1.2 and 1.5 are applied to support design.



- Pages 148, Jager A.J. and Ryder J.A. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 307, Ryder JA &Jager AJ 2000 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.

LEARNING OUTCOME 3.1.11

DESCRIBE, EXPLAIN AND DISCUSS THE SIGNIFICANCE OF ROCKFALL HEIGHT IN TERMS OF SUPPORT REQUIREMENTS IN SEISMIC LOADING CONDITIONS

A similar design methodology as mentioned above in Outcome 3.1.10 can be used to design support for rockfall heights under seismic loading conditions. However, rockfall height under seismic conditions is often referred to as 'ejection height' and in this case, a support unit will need to be able to rapidly deform and absorb the kinetic energy of the expected ejected rock.



The fallout height and ejection height should not necessarily be equal (Table 4).

The yieldability (yield range) and load bearing capacity of the support unit now become critical and can be used to determine a support units' ability to sustain seismic loading conditions, called 'energy absorption capacity'. The energy absorption capacity of the support unit can be determined using the simple formula:

$$EA_{capacity} = 0.5(F.d)$$

where F is the load generated by the support unit and
d is the deformation undergone by the support unit at the point where load F is generated.



In essence this capacity is equal to the area under the support unit's Load-Deformation graph.

The seismic loading conditions that would be exerted on a support unit is indicated by a term called the Energy Absorption Criterion, which gives the amount of energy that must be absorbed by the support unit when a rock of a certain dimension is ejected and stopped by the installed support within a certain distance. The Energy Absorption Criterion for the specific environment can be determined using the simple formuli:

E_k (kinetic energy due to movement) = $0.5mv^2$
where m is mass of the rock with area of $1m^2$ and height equal to that of the ejection height selected,
v is the peak particle velocity (generally accepted at 3m/s) and

E_p (potential energy due to gravitation) = mgh
where m is mass of the rock with area of 1m and height equal to that of the ejection height selected,
g is the gravitational acceleration ($9.81m/s^2$) and
h is the distance within which the support is expected to arrest the ejection of the rock and

Energy absorption capacity = $E_k + E_p$

EXAMPLE

If the ejection height was determined to be 2m,
the rock has a density of 3000kg/m^3,
the stopping distance h is 0.3m
and the ejection velocity is 3m/s,
the Energy Absorption Criterion can be determined as follows:

Mass of block = base area x ejection height x density
= 1m^2 x 2m x 3000kg/m^3
= 6000 kg

$EA_{Criterion}$ = 0.5mv2 + mgh
= (0.5)(6000kg)(3m/s)^2 + (6000kg)(9.81m/s^2)(0.3m)
= (3000kg)(9m2/s^2) + (17658 kgm^2/s^2)
= 27000kgm2/s^2 + 17658 kgm^2/s^2
= 44658 kgm^2/s^2
= 44658 Joule
= 44.658 kJ

Since the $EA_{criterion}$ is calculated for a rock block with base dimensions 1m2, the $EA_{criterion}$ calculated above is 44.658 kJ/m^2.

NOTE

1 Joule (J) = 1 kgm^2/s^2

The higher the $EA_{criterion}$, the more stringent the support requirements become in that either more support units must be installed (smaller spacings) or higher loadbearing and larger yield range support units must be used.

REFERENCE

- Pages 309, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Pages 137, 147, 174, 210, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.1.12

CONNECTION 17

This outcome is addressed at the hand of an example but also refer to Paper 2, Outcome 3.2.1.14 on page 200

EVALUATE GIVEN ROCKMASS CONDITIONS AND EXCAVATION DIMENSIONS TO DETERMINE OR PREDICT STOPE STABILITY

EXAMPLE

A quartzite rock mass at a depth of 200m contains three joint sets:

- One set comprises bedding planes; these are highly weathered, slightly rough surfaces, and are continuous with an orientation of 180/10.
- Another set is jointing; these joints are slightly weathered, slightly rough, and have an orientation of 185/75.
- The third set is also jointing that are slightly weathered and slightly rough, and have an orientation of 090/80.

The strength of the intact rock has been assessed as 155 MPa, and values for the RQD and mean fracture spacing are reported as 60% and 0.4m, respectively.

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Use the RMR system to classify this rock mass, and assess the stability of a 10m wide bord being driven from east to west.

Answer

In order to apply the RMR system, five principal parameters are assessed:

- intact rock strength,
- groundwater conditions,
- RQD,
- fracture spacing, and
- fracture condition.

CONNECTION 18

Refer to Paper 1, Outcome 6 on page 186

This provides the basic RMR value for the rock mass. The orientation of the joints is then accounted for through the use of rating adjustment factors to determine the final RMR value.

There are three sets of joints, and so we will need to apply the RMR system to each set in turn, and so identify the set that is most critical for this particular excavation. The classification parameters of intact rock strength, groundwater, RQD and mean joint spacings are reported generally and thus relate to the rock mass as a whole, they are also applied identically to all sets.

Overall rock mass assessment

The strength of the intact rock has been assessed as 155 MPa, and so a conservative value for the strength rating is 13. The excavation is situated at a depth of 200m, and the rock mass is a quartzite. At a depth of 200m, the vertical stress component wilI be in the region of 5 MPa (assuming a unit weight of rock of 27 kN/m3), and this stress will probably be sufficient to keep the fractures tightly closed.

The groundwater conditions will probably be in the range of damp to wet, with rating values ranging from 7 to 10.

For an average RQD value of 60%, the rating value is about 12, and for a mean joint spacing of 0.4 m the rating value is about 10.

Adding these four ratings together gives a total rating value of

$$13 + (7 \text{ to } 10) + 12 + 10 = 42 \text{ to } 45.$$

Classification using Set 1

These bedding planes are highly weathered, with slightly rough surfaces, and are continuous. The rating values for these specific attributes are 1, 3 and 0, respectively.

An assessment of aperture can be made on the basis of the in-situ stress state at the location of the excavation where a stress of 5 MPa will mean that the aperture is low, and so a reasonable rating value is 5.

We have no information regarding infilling but, as there is no evidence to suggest the presence of any, we can assume a rating value of 6.

The total rating value for these fractures is then

$$1 + 3 + 0 + 5 + 6 = 15.$$

The dip direction of the bedding is 180°, and so the excavation is being driven along the strike. This is considered to result in 'fair' conditions, with a corresponding rating adjustment value of -5.

Thus, the overall RMR value based on Set 1 is

$$(42 \text{ to } 45) + 15 = 57 \text{ to } 60,$$

which is classified as 'fair to good rock'.

Taking into account the orientation rating adjustment of -5, the RMR value is reduced to 52 to 55, but this does not alter the classification significantly.

Classification using Set 2

We know that this jointing is slightly weathered, slightly rough, and has

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an orientation of 185/75. We can therefore assign rating values of 5, 3 and -12 to these attributes.

Their persistence will probably be in the range of 1 m to 2 m, and an appropriate rating value for this is 2.

As with Set 1, we can assign rating values for the aperture and infilling of 5 and 6, respectively.

The total rating value for these fractures is then

5+3+2+5+6=21,

giving an overall RMR value of

(42 to 45) + 21 = 63 to 66.

This is classified as 'good rock'. Taking the orientation rating adjustment of -12 into account reduces the RMR value to 51 to 54, which reduces to 'fair rock'.

Classification using Set 3

The jointing representing Set 3 has identical mechanical characteristics to Set 2, and so has a rating value of 21, giving an overall RMR value of 63 to 66 and a classification of 'good rock'.

The orientation rating adjustment is now -5 (the strike is perpendicular to the excavation axis, and we are driving against the dip which is classed as 'fair'), which reduces the RMR value to 58 to 61, on the border between 'fair' and 'good' rock types.

Overall assessment

We can see that Set 2 leads to the most critical classification, with a range of probable RMR values of 51 to 54.

Using a chart linking RMR and excavation span to stand-up time shows that an excavation 10 m wide in such a rock mass would suffer from immediate collapse between 3 days and 1 week and require rock stabilisation measures.



- Hudson J.A. and Harrison J.P. Engineering Rock mechanics. Illustrated worked examples, 2000.

LEARNING OUTCOME 3.1.13

EVALUATE GIVEN ROCKMASS CONDITIONS AND EXCAVATION DIMENSIONS TO DETERMINE REMEDIAL MEASURES TO IMPROVE STOPE STABILITY

CONNECTION 19

Refer to LEARNING OUTCOME 3.1.12 on page 82.
Refer to Paper 2, Outcome 3.2.1.14

The results of the processes discussed in each of the abovementioned outcomes must be applied into the implementation of one of the following:

- reduction in excavation dimensions to allow acceptable unsupported span periods;
- installation of artificial support;
- decrease in maximum support to face spans;
- increase in support unit density;
- increase in support unit lengths and
- increase in support system capacity.

LEARNING OUTCOME 3.1.14

DESCRIBE, EXPLAIN, DISCUSS AND APPLY THE FOLLOWING TECHNIQUES TO THE ABOVE EVALUATION:

- Rock mass class, Stress analysis, Continuum mechanics,
- Discrete block kinematics, Beam theory, Failure criteria

Once an evaluation based on the quality of the rock mass has been done, the need and possible available options for remedial action / measures will be known ("LEARNING OUTCOME 3.1.13").

A variety of methods to assist the engineer in determining the best options / solutions are available and include the methods listed in this outcome. The rock engineer should select and apply the most appropriate method to the conditions designing for.

CONNECTION 20

Rock mass classification
Refer to Paper 1 Outcome 6 on p186, Paper 2 Outcome 3.2.1.14 on p 190 and Outcomes 3.1.12 and 3.1.13 above.

Stress analysis
Refer Paper 2, Outcome 6.4 on p 497

Continuum mechanics
Refer Paper 2, Outcome 6.4 on p 497

Discrete block kinematics
Refer Paper 2, Outcome 6.4 on p 497

Beam theory
Refer Paper 1, Outcome 2.11 on p 57

Failure criteria
Refer Paper 1, Outcome 4.1.6 on p 106

NOTE

Using more than one of the methods is also acceptable.

LEARNING OUTCOME 3.1.15

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS ROCKMASS BEHAVIOUR AND CONDITIONS ASSOCIATED WITH STOPING ABUTMENTS AT DEPTH

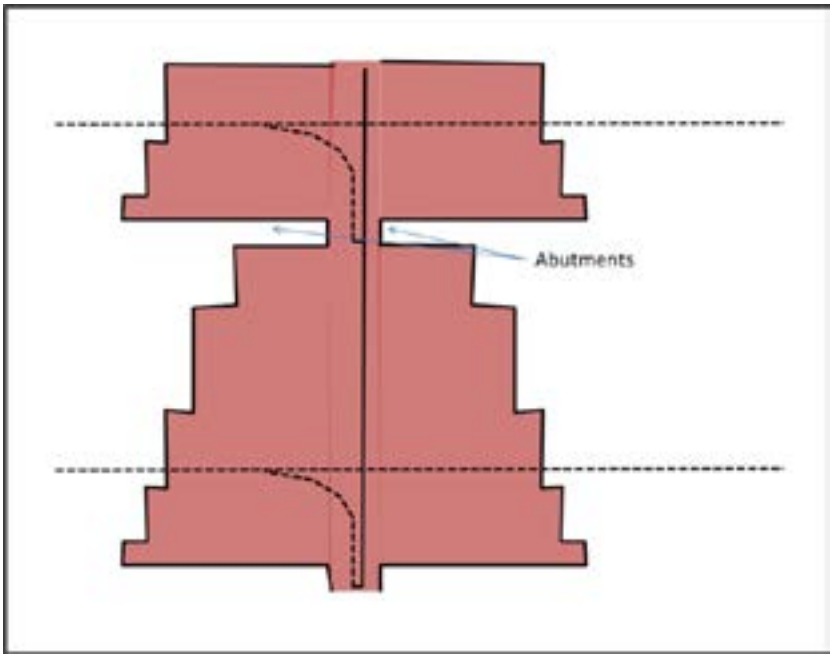


Figure 16: Typical schematic of scattered layout indicating the position of abutments.

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DEFINITION

Abutment: When used on the mining operations ('slang' – common meaning) 'abutment' simply means the 'edge' of the unmined rock around a mined out area.

CONNECTION 21

Refer to LEARNING OUTCOME 3.1.3 on page 65

As mining advances on the reef elevation, stress levels ahead along the 'edge' of the mined out area increases due to the increased overburden weight that must be 'carried' by the remaining solid rock.
High stress levels along abutments would suggest intense fracturing of the first few metres of the abutment, causing a stress drop along the edge of the abutment and a displacement of the stress peak away from the immediate edge of the abutment (Figure 8). This zone of fracturing extends into the reef footwall and hangingwall and induces dilation, which manifests as closure of the rock mass into the mined out area.

CONNECTION 22

Refer to LEARNING OUTCOME 3.1.5 on page 70

Usually when reference is made to abutments, the intension is to indicate the presence of high stress levels which may indicate higher risk mining conditions, especially at depth where rock or face bursts are common when the stress increase along the face abutment results in violent energy release, even though the position of the stress peak is not along the edge of the abutment (Figure 8).

Conditions and behaviour that can usually be expected along and close to abutments can be summarised into (may not be limited to):

Conditions	Behaviour
Dense stress fracturing of the face	Face scales into stope Shearing on stress fractures and other planes (see closing of drilled holes) Could assist in creation of conditions prone to provide face bursts Moves peak stress point into solid rock and increases potential for shear type seismic events ahead of face Increases potential for 'locking-up' of fractures and therefore face bursts
Dense fracturing of the hangingwall and footwall	Closure into mined out area Create small rocks in hangingwall Can create blocky ground conditions in hangingwall Reduces the actual pillar width : height ratio and thus its performance and function in supporting the rock mass
High stress levels ahead of the face	Seismic activity may occur and be concentrated along the abutment Stress damage occurs to footwall excavations developed into high stress zones
Fracturing of pillars	Increases fracture depth into pillars reduce the actual pillar width and therefore the pillar's behaviour in supporting the rock mass If extreme, pillars could fail and may not be able to perform its intended function in supporting the rock mass
Face stress levels	Create potential for blasting fractures to penetrate the hangingwall and footwall, increasing creation of blocky conditions

Table 5: Rock mass conditions and behavior associated with abutments

REFERENCE

- Pages 33, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 3, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 3, 5, Budavari S, Rock mechanics in practice, SAIMM, 1983

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LEARNING OUTCOME 3.1.16

CONNECTION 23

Refer to LEARNING OUTCOME 3.1.5 on page 70

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE ON-REEF EFFECTS ASSOCIATED WITH STOPING ABUTMENTS AT DEPTH

LEARNING OUTCOME 3.1.17

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE OFF-REEF EFFECTS ASSOCIATED WITH STOPING ABUTMENTS AT DEPTH

CONNECTION 24

Refer to LEARNING OUTCOME 3.1.15 on page 85

The extremely high stress peaks created ahead of the mining face and along the stoping abutment, extends into the hangingwall and footwall of a stope. This means that off-reef excavations situated in this high stressed zone, may intersect these high stress zones and may then be subjected to increased stress fracturing, closure, support failure and even seismic damage.

Figure 17 shows stress concentrations along abutments using a 2-dimensional simulation package Phase2 (Rocscience) and indicates the following important issues:

- Very high stresses exist on, above and below the reef elevation;
- These high stress zones take on the form of lobes;
- The footwall tunnel is subjected to higher stress levels due to its position in one of the high stress lobes.

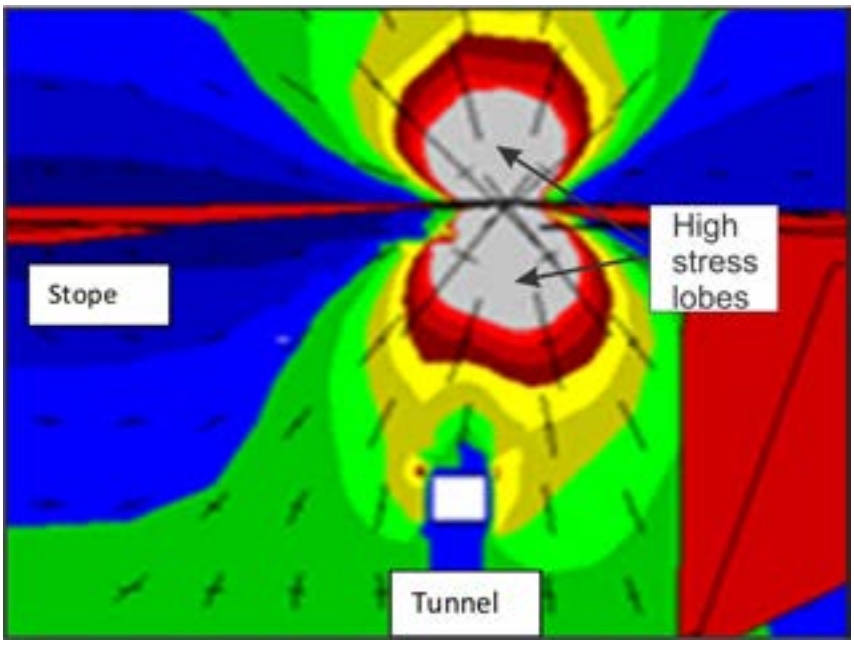


Figure 17: High stress lobes / zones along abutments

NOTE

As the remnant between the two mined out stopes decreases in size (or if it is a pillar), the sizes of the high stress lobes will also decrease, i.e. the distance below and above the reef elevation will decrease (see interesting information below). This means that the on-reef layouts will determine the off-reef stress levels within certain distances of the reef. It is critical that abutment stress levels are evaluated when placing footwall tunnels so that these high stress lobes can either be avoided (place

back

tunnel on another elevation or position) or support requirements can be increased.

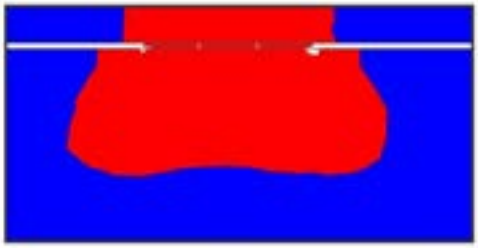
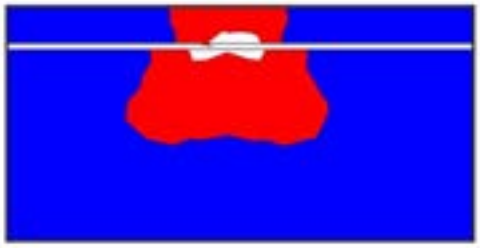
	Small zones of high stress along mining abutment (white) and a large area subjected to increased stress levels in remnant (red). Note: Contour ranges have been kept constant to indicate changes in stress lobes.
	As remnant becomes smaller, the high stress zones (white) along the mining abutment changes slightly. The higher stress lobe (red) reduces in size as distressed (blue) area increases below mined out areas. NOTE: The high stress zone (red) also reduces in vertical extent.
	Only a small remnant remains and it is completely highly stressed (white) as this zone increases substantially. However, the high stress zone (red) decreases further in horizontal and vertical extent.

Table 6: Stress lobe changes between mined out areas

INTERESTING INFO

The high stress lobe dimensions in the remnant between two approaching stopes will change as the stopes approach each other. Initially it will be small due to the relatively low stress levels, then it will increase in stress level and lobe size up to a certain point where the stress levels will still increase, but the area it affects (lobe) will reduce in size.

When footwall tunnels follow behind the mining face (surprisingly called 'follow-behind' tunnels), it is critical that the tunnel is positioned in the 'de-stressed' zone, i.e. where the stress levels are lower than normal, so that stress fracturing, support requirements and seismic damage can be limited.

A '45° rule' is normally referred to, to ensure that the tunnel will not be placed in the highly stressed abutment stress lobes (cold colour areas rather than warm colour areas in Figure 18). The 45° angle is normally measured from the reef elevation at the edge of the mined out area (face). However, care must be taken in applying this rule and numerical modelling is suggested to confirm stress abutment lobes positions, while the rule is only used as a first stab at a possible position. As long as the tunnel face remains behind the higher stressed zones, the whole tunnel will be subjected to lower stress levels and stability will be easier to maintain.

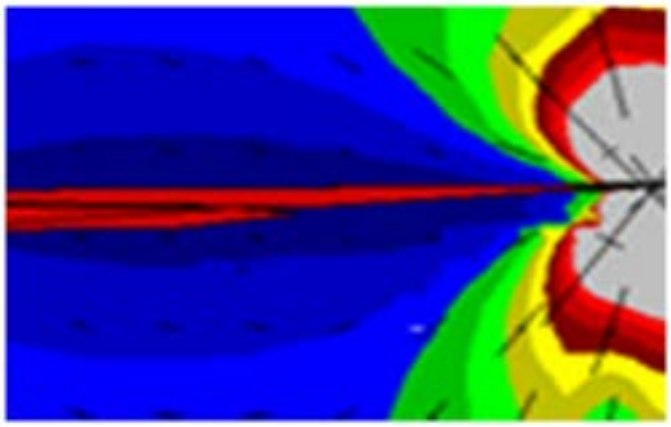


Figure 18: 'Follow behind' tunnels and the 45° rule

The 45° rule is also used in scattered mining environments to place foot-wall tunnels. However, it is critical to know that this rule (as measured above) only applies when the section drawn (as in Figure 18), show a mined out area (stope) that is completely horizontal (dip is 0°). As soon as a reef dip > 0° is shown on the section used to analyse the position of the tunnel, the angle between the reef and the line that separates the 'highly stressed' and 'de-stressed' zones changes. In these cases, the use of numerical modelling to determine the best position for the tunnel is a requirement (even though findings may suggest that these lines still remain at an angle of 45° to the horizontal, measured on the edges of the mined out stope).

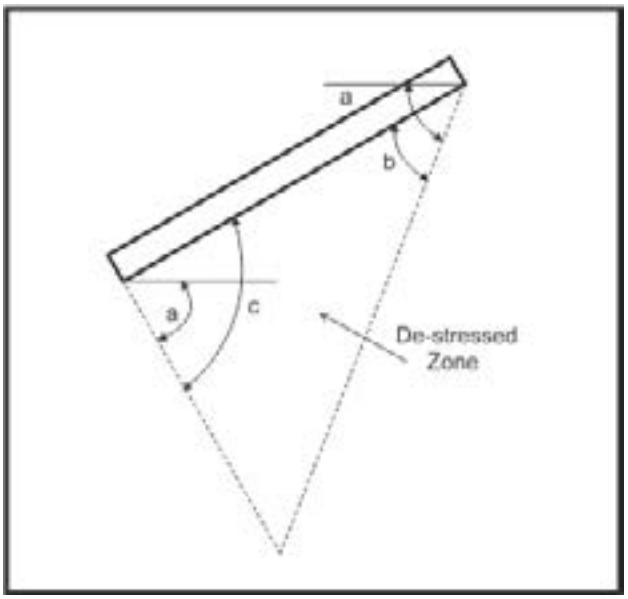


Figure 19: The application of the 45° rule in dipping reefs (a = 45°, c > 45° and b < 45°)

- Pages 35, 86, 194, 200, 277, Jager J.A. and Ryder A.J.. A hand-book on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 3, 5, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.1.18

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS ROCKMASS BEHAVIOUR AND CONDITIONS ASSOCIATED WITH STOPE PILLARS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

CONNECTION 25

Refer to LEARNING OUTCOME 3.1.1 on page 62 to LEARNING OUTCOME 3.1.5 on page 70

It has been indicated earlier in this material that the rock mass, de-formations occurring, stress fracturing and support requirements in the different depth categories could be quite different.

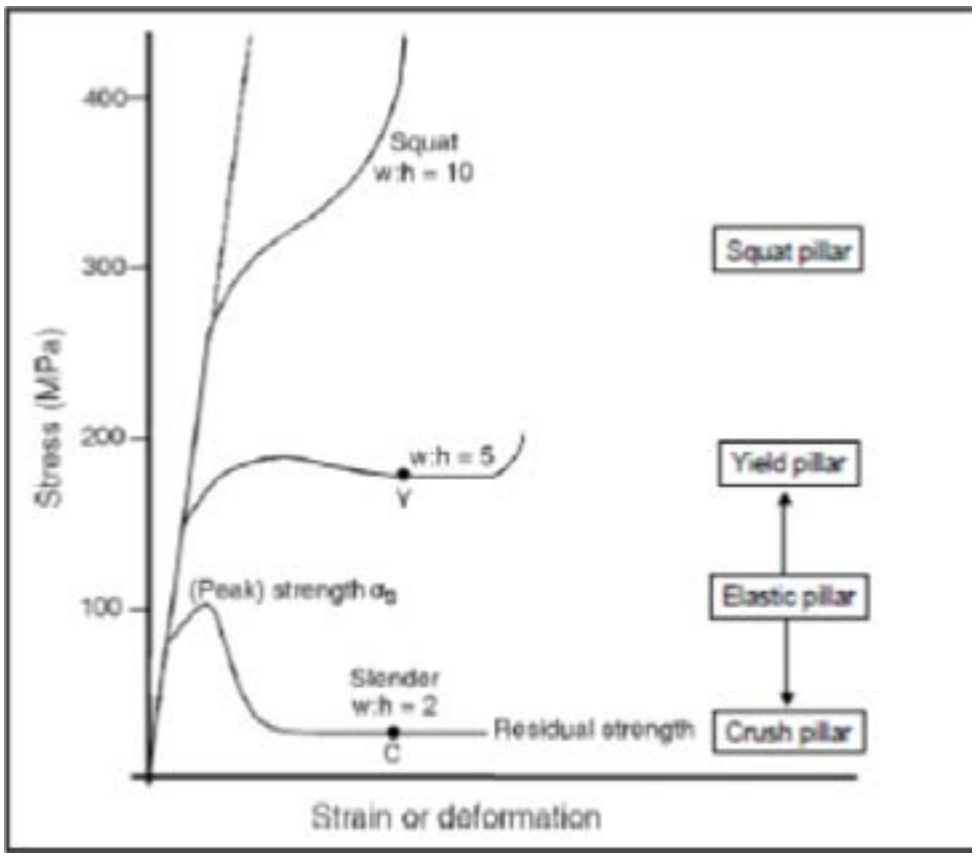


Figure 20: Pillar types (Ryder and Jager, 2002)

INTERESTING INFO

Before the impact of pillars is discussed, some basic definitions with regards to pillar types are shown below. Non-yield, yield and crush pillars are terms often used when classifying in-stope pillars. The naming is born from the stress-strain relationships of various pillars as shown in Figure 20 and implies the following reasoning:

- Non-yield or elastic pillars : Pillars are designed using the peak (pre-failure point) strength (σc) and indicates that the average stress will not cause failure of the pillar;
- Crush-pillar: The pillar crushes and has failed (post failure point) and is designed using the residual strength,
- Yield pillar: Pillar starts to deform without losing its ability to sustain load,
- Squat pillar : Large width:height ratio suggests high loadbearing whilst deforming.

The rock mass behaviour discussed in the following outcomes assumes that the utilisation of pillars in the different depth categories will have an impact on the overall rock mass behaviour and it is this impact that is described below.

In very shallow stopes (depths < ± 400mbs), in-stope pillars are required to carry the full weight of overburden up to surface due to the absence of stress levels sufficient of assisting stability. At the same time, it is known that this environment is dispersed with geological weakness and that large dimension wedges or domes formed by these structures, may require additional (more than planned for overburden weight) pillars to be left to ensure stability of the stopes (again due to absence of stabilising stress levels).

Stope pillars utilised at these depths are:

- High loadbearing and therefore usually large to ensure that adequate strength is generated to maintain stability of the overburden and thus of surface structures and
- 'non-yield' to ensure that pillar failures (yield) does not affect surface stability.

In shallow mining depth stopes between approximately 400mbs and 1000mbs, the use of non-yield in-stope pillars is still possible, but due to the extreme increase in pillar dimensions to sustain the higher stress levels and still remain 'non-yield', the reduction in the percentage extraction is significant. If pillar dimensions cannot increase, the higher stress levels will cause fracturing of the pillars and that means that the pillars will 'yield' and generate lower loads.

Even though this seems like a disaster, it is also clear that the function of pillars change with depth as stress levels increase and stress components start to assist overall stability. At these depths, pillars are not required

to support the whole overburden but only to assist in maintaining stope stability by controlling the tensile zone above the stope. Loads that they need to generate are thus lower (smaller weight of tensile zone) and controlled displacement of the hangingwall (due to yielding pillars) is not a major concern as artificial support units are used to control the stability of the areas where people work.



The presence of regional pillars in this environment will determine the dimensions of the tensile zone to be supported as they are used to control the mining span.

CONNECTION 26

Refer to LEARNING OUTCOME 3.1.1 on page 62

Stope pillars utilised at these depths are:

- Sufficiently loadbearing to maintain stability of the tensile zone;
- 'yielding' pillars going over into 'crush' pillars with increasing depth, when local stability needs to be maintained and hangingwall displacements is not a concern ;
- Regional pillars to control the size of the tensile zone and maintain surface stability.



Care should be taken in the design of these pillars as yielding in shallow areas, as well as non-yield in deeper areas both create high risk conditions. The first can acuse the loss of the mining excavations and affect surface structures while the second can results in pillar bursts.

At mining depths > 1000m, in-stope yield pillars are still utilised in some areas, but with increasing depths eventually become unnecessary as high stress levels will cause crushing and scaling of pillars so quickly and close to the mining face, that they will perform no real function in terms of stabilising the stope. In the meantime, increased stress levels increases the contribution of stress components to stope stability and reduces the need for extensive high loadbearing, pillar type support to main stability. The function of support in intermediate and deep level mines changes from a supporting role to a protection role, where pillars are only used to ensure workable, safe conditions but where stope stability is maintained using artificial support such as backfill, packs and elongates.

Pillars utilised in this environment are thus used as:

- regional pillars are utilised to control mining spans and therefore also face stress, closure, energy release rate and seismic activity levels;
- bracket pillars to limited shear displacement on large geological structures and therefore also seismic activity;
- crush pillars to protect against wedges formed by geological structures or to protect re-raises from old collapsed workings.



Figure 21 graphically shows the change in pillar functions as depth increases within the 'shallow' environment. It must be noted that no exact depth relations are shown since specific investigations must be executed to determine the optimum depths at which pillar types should change for any given environment.

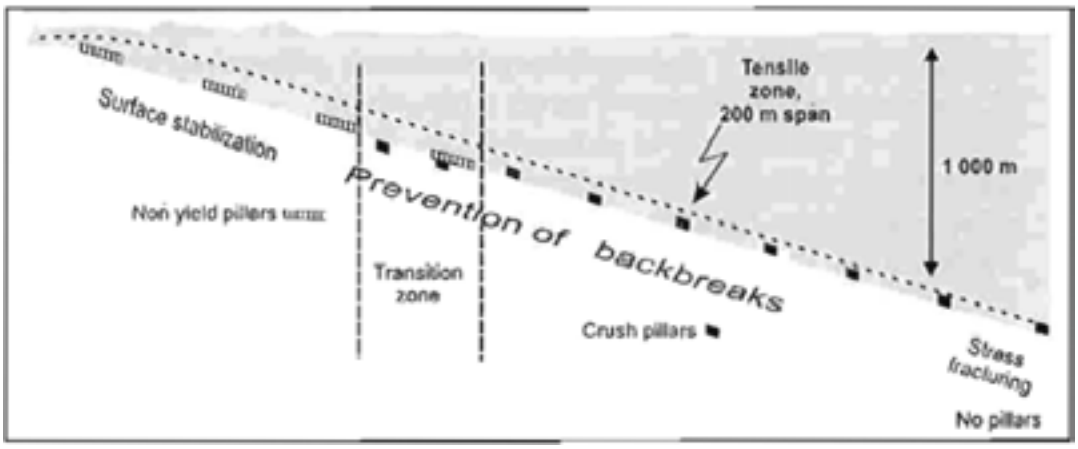


Figure 21: In-stope pillar types appropriate to different (shallow) mining depths

REFERENCE

- Pages 250, 319, 56, 276, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 178, 233, 244, 284, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Chapter 8, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.1.19

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE ON-REEF EFFECTS ASSOCIATED WITH STOPE PILLARS AND/OR REMNANTS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

CONNECTION 27

Refer to LEARNING OUTCOME 3.1.18 on page 89

The following on-reef effects are suggested:

	Shallow mining	Intermediate depth mining	Deep level mining
Pillars	Hangingwall beam failure on pillar edges Pillar punching into footwall or hangingwall Premature pillar failure due to weak zones in pillars Source of potential FOG if pillars are jointed and mining height is large	Scaling of stress fractured pillar edges Punching of unexpected strong pillars into hangingwall or footwall Pillar bursts if yielding does not occur Hangingwall beam failure on pillar edges Varying stress fracture orientations around the pillar	Complete scaling of stress fractured pillars with timeFoundation failures if width:height ratio on regional pillars is insufficientHangingwall beam failure on pillar edges
Remnants	Increased stress levels could result in stress fracturing	Poor ground conditions due to excessive stress levels and thus stress fracturing High closure rates Seismic activity on geological structures within remnant	Seismic activity on remnants when stress levels cause failure of the remnant Poor ground conditions due to excessive stress levels and thus stress fracturingHigh closure rates Seismic activity on geological structures within remnant

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- Pages 250, 319, 56, 276, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 178, 233, 244, 284, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Page 143, Budavari S, Rock mechanics in practice, SAIMM, 1983
- Chapter 8, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.1.20

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE OFF-REEF EFFECTS ASSOCIATED WITH STOPE PILLARS AND/OR REMNANTS AT SHALLOW, INTERMEDIATE OR GREAT DEPTH

Pillars or remnants may create highly stressed pieces of reef that affects off-reef excavations as explained in Outcome 3.1.17.



Since remnants and pillars have limited dimensions, the discussion in Outcome 3.1.17 on the distance above and below the reef where stress levels are increased (stress lobes) is important and should be carefully studied.



- Pages 250, 319, 56, 276, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 178, 233, 244, 284, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Chapter 5, Budavari S, Rock mechanics in practice, SAIMM, 1983
- Page 143, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.1.21

CONNECTION 28

Refer to LEARNING OUTCOME 3.4.2 on page 142 and 9.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PURPOSE, PROCEDURE AND EFFECTS OF ROCKMASS PRE-CONDITIONING AHEAD OF STOPES AT DEPTH

LEARNING OUTCOME 3.1.22

CONNECTION 29

Refer to LEARNING OUTCOME 3.4.2 on page 142 and 9.2 and 9.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE VARIOUS METHODS OF PRE-CONDITIONING

LEARNING OUTCOME 3.1.23

CONNECTION 30

Refer to LEARNING OUTCOME 3.4.2 on page 142 and 9.2 and 9.2

DESCRIBE, EXPLAIN AND DISCUSS THE CRITICAL FEATURES OF PRE-CONDITIONING.

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3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS

LEARNING OUTCOME 3.2.1

CONNECTION 31

Refer to Paper 1 Outcome 6.2 on p200
Refer to Paper 2 Outcome 3 on page 93
Refer to Paper 2 Outcome 5.1.9 on page 331

DESCRIBE, EXPLAIN AND DISCUSS HOW ROCKS AND ROCKMASSES MAY BE CLASSIFIED

LEARNING OUTCOME 3.2.2

CONNECTION 32

Refer to Paper 1 Outcome 6.2 on p200
Refer to Paper 2 Outcome 3 on page 93

DESCRIBE, EXPLAIN AND DISCUSS HOW ROCKMASS CLASSIFICATION SYSTEMS MAY BE USED TO PREDICT TUNNEL STABILITY AT SHALLOW, INTERMEDIATE AND GREAT DEPTHS

LEARNING OUTCOME 3.2.3

DESCRIBE, EXPLAIN AND DISCUSS THE NATURE OF FRACTURING AROUND TUNNELS AT DEPTH



Stress fracturing is the result of stress levels that are sufficient to cause failure of the rock it acts on. Tunnels at depth are subjected to high stress levels purely due to the virgin stress components at depth, but may also be subjected to further stress level increases due to mining conducted in the area. At the same time, the creation of an excavation causes stress changes around the tunnel, also contributing to the final level of stress around the perimeter of the tunnel. At depth, rock failure around a tunnel is therefore definite and a normal occurrence.

At depth, the maximum principal stress component is usually vertical (unless changed by mining in the near vicinity or where significant tectonic stress is present). The position of maximum stress levels around a tunnel is thus determined by the orientation of the stress field (and also the tunnel shape – to be discussed later). Under normal conditions, a vertical major principal stress field will induce maximum stress levels in the sidewalls of a tunnel and will induce fracturing that are orientated similar to the major principal stress orientation. In practice, the stress fracturing around a tunnel will have the same 'look' as the rings of an onion (also Refer to "LEARNING OUTCOME 3.1.6"), with the most dense fracture development positioned along the tunnel borders that are 90° to the major principal stress (σ_1) direction (Figure 22).

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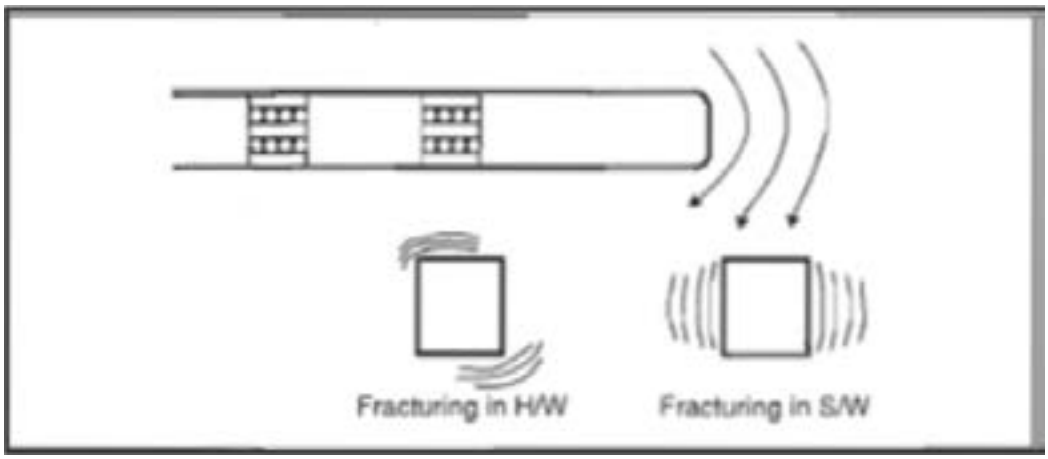


Figure 22: Fracturing around tunnels (Jager and Ryder)

CONNECTION 33

Refer to Paper 1 Outcome 4.2.15 on page 143

On plan, fracturing around a tunnel follows the same pattern as 'water lines' around the bow of a boat (Figure 23a). These fractures would be shaped such that they would be visible in the tunnel sidewall (Figure 23b), always orientated such that the fractures strike towards the 'back' of the tunnel (direction from which the tunnel was developed).

INTERESTING INFO

Determining the direction in which a tunnel was developed is easy if stress fracturing formed around the tunnel. By a fast study of the tunnel sidewalls, the fractures will always point) from the edge of the excavation into the rock) towards the direction from where a tunnel was developed). This could also help you to find the way to a station if we assume that all development was initiated from the shaft station.

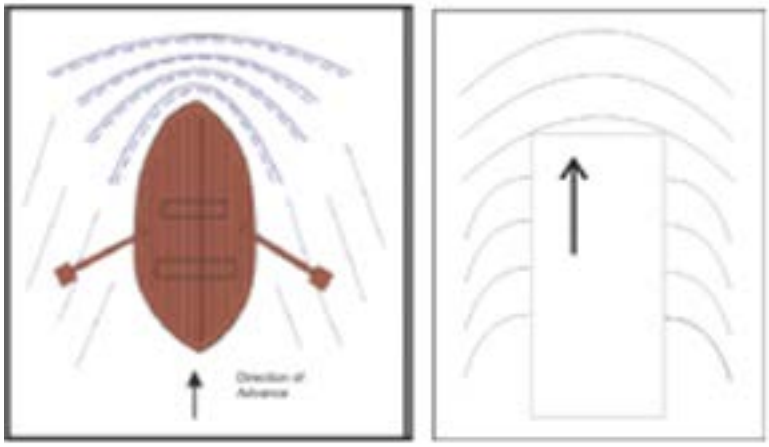


Figure 23: (a)Water-lines around a boat and (b) Fracturing around a tunnel

REFERENCE

- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 3, 4, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.2.4

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF FRACTURING ON TUNNEL SIZE

Fracturing (Outcome 3.2.3) of the rockmass around a tunnel results in dilation (displacement into the excavations due to fracture formation) of the rockmass and closure of the tunnel walls. If this fractured zone is supported and controlled, the size of tunnels subjected to high stress will reduce after formation of the fracturing. If the fractures zone is allowed to scale and is removed by workers, the tunnel will continuously grow in size.

INTERESTING INFO

When stress fractures form, closure of the walls is initiated. This closure will occur over time, but, if stress levels remain the same and the rock material does not deteriorate, the amount of closure will decrease with time. Tunnels that are very old should close to a point and then remain practically static for many years. This allows us to go back into tunnels and reuse them. In other cases, renewed closure could result in collapses and may require rehabilitation.
However, if:

- stress levels acting on the tunnel changes, new fractures will form and closure will continue to occur, sometimes even completely closing up a tunnel;
- support installed corrodes and fails, renewed closure is allowed and the tunnel will experience closure until the rock movement is stopped with new support;
- support fails due to long term displacement overcoming the yield range of the support, renewed closure is allowed and the tunnel will experience closure until the rock movement is stopped with new support;
- seismic activity occurs and subject the tunnel to larger stress levels, new fractures can occur and closure is allowed again.



- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 4, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.2.5

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF FRACTURING ON TUNNEL SHAPE

Fracturing (Outcome 3.2.3) that occurs around a tunnel is determined (inter alia) by the principal stress field orientations. If fracturing occurs and dilation or closure remains uncontrolled, i.e. fractured rock is allowed to scale, the excavation shape will change drastically.



The same shape changes will occur if support is installed that controls but not prevent dilation / closure.

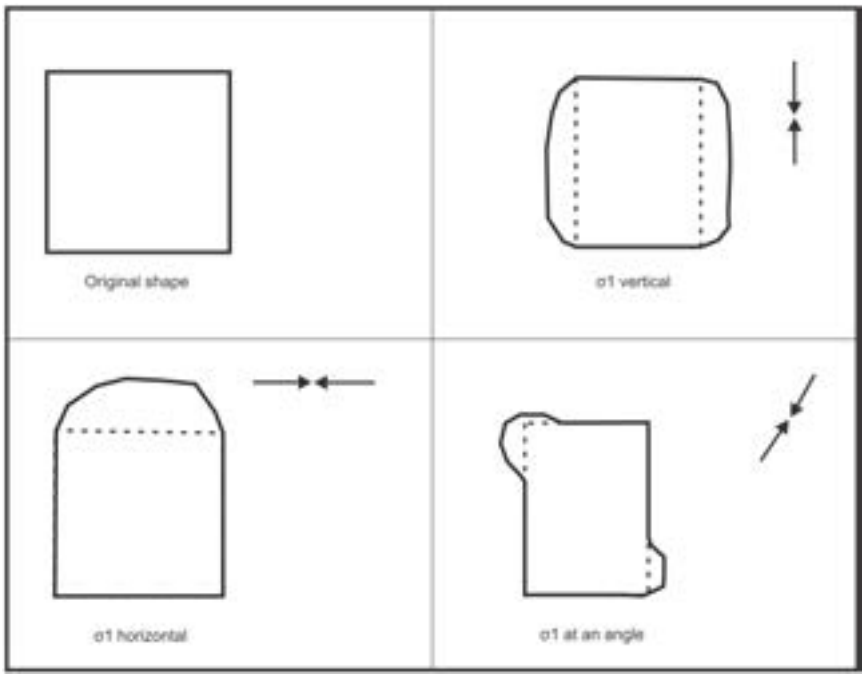


Table 7: Impact of fracturing on tunnel shape

REFERENCE

- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 4, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.2.6

CONNECTION 34

Refer to LEARNING OUTCOME 3.2.1 on page 94 to LEARNING OUTCOME 3.2.5 on page 96

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF FRACTURING ON TUNNEL STABILITY

LEARNING OUTCOME 3.2.7

CONNECTION 35

Refer to Paper 1 Outcome 3.11 on page 86

DESCRIBE, EXPLAIN AND DISCUSS THE PHENOMENON OF TIME-DEPENDENT FRACTURING AND DETERIORATION

Significant rock mass deformations and displacements occur in deep level mines and generally follow certain trends in fracturing and closures. However, time dependent displacements also occur within the fractured zone around the excavation.

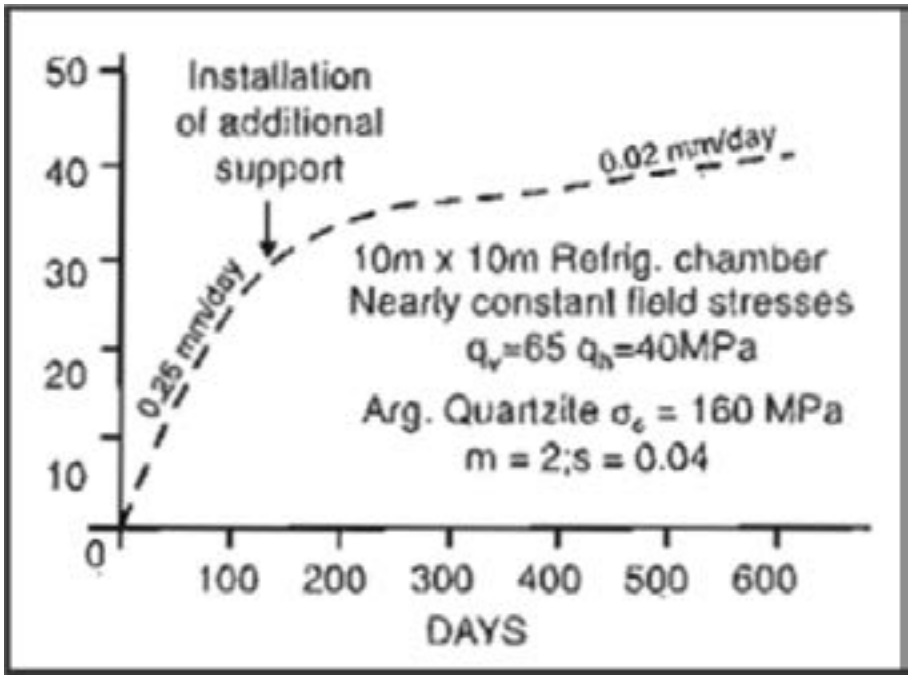
Primary creep usually occurs over a number of days or weeks and is considered complete once fracturing and sliding on weakness planes has ceased and the stress redistributed following the last stress change has been completed.

When Primary creep halts, Secondary creep commences and takes place over a much longer period of time. Secondary creep is more pronounced in weaker materials such as argillaceous quartzite and at higher stress levels.

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Figure 24 shows sidewall movement (in mm) in an excavation as a function of time (days). The trend suggested with these measurements is that:

- significant displacements (steep slope of the graph) occurred soon after the blast (primary creep) while
- slow (shallow slope of graph), continued displacements occurred after 100 days (secondary creep).



CONNECTION 36

Refer to LEARNING OUTCOME 3.2.4 on page 95

Figure 24: Time dependent fracturing (Jager and Ryder, 1999)

REFERENCE

- Pages 20, 94, 131, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 106,182, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Page 46, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.2.8

CONNECTION 37

Refer to LEARNING OUTCOME 3.2.4 on page 95

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF TIME-DEPENDANT FRACTURING AND DETERIORATION ON TUNNEL STABILITY

Fracturing processes in overstressed zones around mining excavations are subject to primary creep and secondary creep. As stated earlier, primary creep consist mainly of fracturing and sliding processes to reach equilibrium through stress adjustments. This sliding and fracturing induces displacements into the excavation which if left unsupported could result in falls of ground but, if supported, would strain the support units. In cases where insufficient yield on the support is available, support units could fail and falls of ground can occur. Where adequate support yieldability and load generation is available, stability is maintained.

Secondary creep is also of importance and consist of slow, non-terminating (takes place over long periods) displacements take place. These displacements could, in the long term and following the primary creep displacements, be just sufficient to take up all the strength or yieldability

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of the support units and cause its failure, resulting in falls of ground many years after the creation of the tunnel.

The impact of time on support unit conditions should also be kept in mind. Eg. Corroding support subjected to secondary creep may reach its failure point sooner and result in falls of ground.

This time dependant fracturing that occurs around tunnels emphasizes the displacements that can occur over long periods of time and suggest that long term tunnel designs take cognisance of this concern and that 'old' tunnels should be approached carefully when opening up occurs. Risk assessments conducted prior to opening-up operations should take cognisance of support deterioration and rock mass movements that have occurred in assessing the excavation's stability.

- Pages 20, 94, 131, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 106,182, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Page 46, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.2.9

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF STRESS REGIME AND GEOLOGICAL STRUCTURE ON ROCK BEHAVIOUR AND TUNNEL STABILITY

CONNECTION 38

Refer to LEARNING OUTCOME 3.2.1 on page 94 to LEARNING OUTCOME 3.2.8 on page 98.

Stress regime

Stress regimes can also impact on rock mass behaviour and tunnel stability in the following manners:

Stress regime	Rock mass behaviour	Tunnel stability
High stress	Fracturing occurs Dilation / closure into the tunnel Increased potential for creep	Blocky ground is created and require support installation Depending on stress orientation hangingwall or sidewall problems can be encountered Tunnel dimension reduces and additional support may be required Footwall heave is possible, increasing transport risks Long term stability may be suspect
Tensile stress	Loosening of the rock mass Opening up of weakness planes Relaxation of previously stressed and fractured rock material	Falls of ground when contact planes allow separation and loose blocks / wedges are formed that must be individually supported to maintain stability Reduction in excavation dimensions when fractured rock is suddenly de-stressed Opening-up operations and re-supporting may be required.

Stress regime	Rock mass behaviour	Tunnel stability
Quasi-static (slow) stress changes	Stress changes causes new fracturing Dilation / closure occurs into tunnel	Increased deformation on support and potential for support failure Support unable to control closure, resulting in loss of tunnel when dimensions become too small
Dynamic (seismic) stress changes	Stress change occur fast and are often associated with new, large displacement fractures Fractured rock is accelerated into the excavation	Excessive displacements allow closure to increase, closing the excavation, straining support units and may result in complete failure of the support system and collapse of the excavation Closure significantly reduces tunnel dimensions and often cause the loss of the tunnel

Table 8: Impact of stress regimes on rock mass behaviour and tunnel stability

Geological structure

Geological structures can impact on rock mass behaviour and tunnel stability in the following manners:

Structure	Rock mass behaviour	Tunnel stability
Bedding planes	Beam creation in hangingwall Laminated (thin beams) rock mass allows easy separation	Beam stability is affected by direction of development and exposure of beam (along strike of tunnel as in Figure 25a or 'factory roof' shapes) Additional planes to allow creation of blocky conditions with stress fractures Laminated material collapse to stable shape (arch) if left unsupported
Joints	Create wedges in excavation perimeter Reduce rock mass quality and increase deformation under stress as rock mass becomes softer	Single blocks may have to be supported individually or a system of support units are required to ensure stability Increased deformation due to poorer and thus softer rock mass requires appropriate support systems to maintain stability, as small yield range units may fail prematurely (while excavation is still needed)
Faults	Form source of seismic activity, resulting in increase closure and newly created fractures Create wedges that might allow separation Changing joint properties (orientation, spacing, infilling) reduces rock mass quality Could cause local field stress changes due to tectonic stress	Large displacements and large energy release is associated with seismic events, damaging support and rock material around an excavation Potential FOG from fault planes require support installationSmaller blocks require increased support density Poorer rock mass quality creates softer rock mass, increased deformation and straining of support units. FOG due to support failure may result. Stress changes (refer to "LEARNING OUTCOME 3.2.9")

Structure	Rock mass behaviour	Tunnel stability
Dykes	Form source of seismic activity, resulting in increase closure and newly created fractures Create wedges that might allow separation Changing joint properties (orientation, spacing, infilling) reduces rock mass quality Changing rock material properties (weaker of more competent) Could cause local field stress changes due to tectonic stress	Large displacements and large energy release is associated with seismic events, damaging support and rock material around an excavation Potential FOG from fault planes require support installation Smaller blocks require increased support density Poorer rock mass quality creates softer rock mass, increased deformation and straining of support units. FOG due to support failure may result. Changing rock properties require additional / other support to maintain stability and remain optimum Stress changes (refer to "LEARNING OUTCOME 3.2.9")

Table 9: Impact of geological structures on rock mass behaviour and tunnel stability

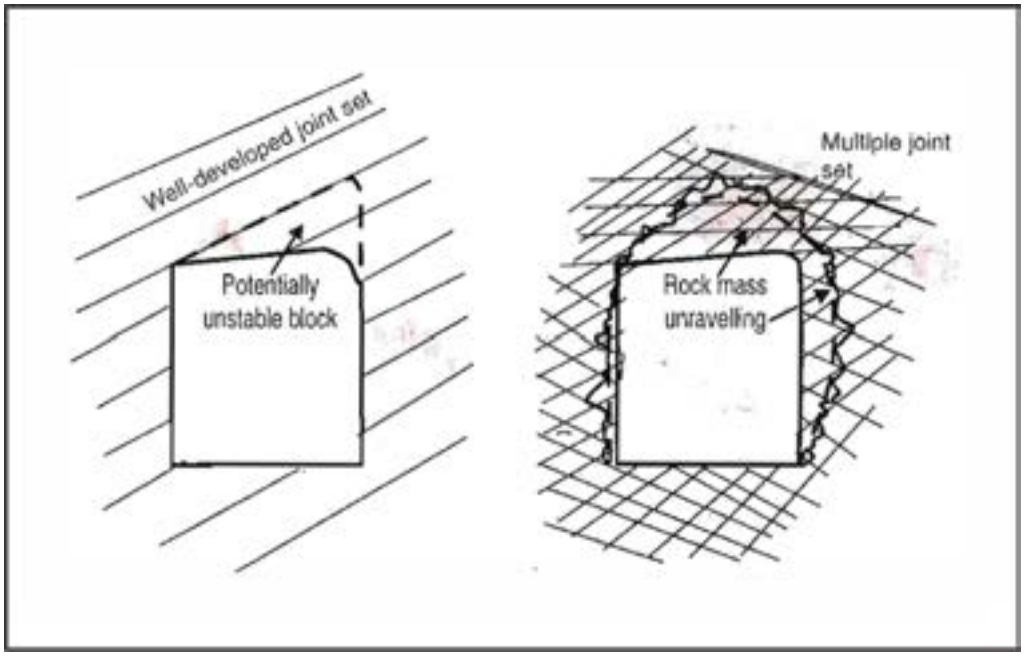


Figure 25: Impact of geological structures on tunnel stability

REFERENCE

- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.2.10

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF TUNNEL SIZE, TUNNEL SHAPE AND TUNNEL EXCAVATION TECHNIQUE ON ROCK BEHAVIOUR AND TUNNEL STABILITY

Tunnel size

Excavation size does not directly influence the levels of induced stresses, as they are a function of the excavation shape, but increases the amount

of rock exposed and, due to increased exposure of discontinuities, reduces the rock mass strength around the excavation. This means that fracture depth (how deep into walls of the excavation stress fractures will form) is increased when excavations dimensions increase.

In shallow mining environments, the increase in discontinuity exposure with increased excavation dimensions reduces the exposed rock mass quality and increases support requirements to maintain stability. At the same time, since stability in these environments is largely driven by the presence of geological structures (and not stress), increased dimensions can increase potentially unstable wedge dimensions.



Normally tendon lengths are increased as the excavation size increases. It should be clear that this is to cater for the increased fracture depth in deeper mines, and increased unstable block dimensions in shallow mines.

Tunnel shape

Tunnel shape influences the stress distribution and stress levels around the tunnel, and therefore also the stress fracturing, significantly. By changing the tunnel shape, improved stress distributions and levels (caused by stress concentrations) can be ensured. Due to this, circular or ellipsoid tunnel shapes provide better stress conditions around tunnels than the standard square or rectangular tunnel shapes.

It has been shown that the theoretically optimum shape from a stress distribution point of view is an elliptical shape with the longest axis parallel to the maximum principal stress. This allows the excavation to have minimal impact on stress distributions. However, as soon as underground excavation stability is considered, the best shape is an elliptical shape with the long axis perpendicular to the major principal stress.

Figure 26 (for conditions where k-ratio is 0.5) shows that stress concentrations and fracture depth are both affected by the excavation shape. When the long axis of the excavation is parallel to σ_1 (Figure 26a), the stress concentrations in the sidewall is low (1.5) but the depth of fracturing due to the large vertical dimension significant. However, with the long axist perpendicular to σ_1 (Figure 26c), even though the sidewall stress concentrations are high (4.5), the depth of fracturing is small and can easily be controlled with support.

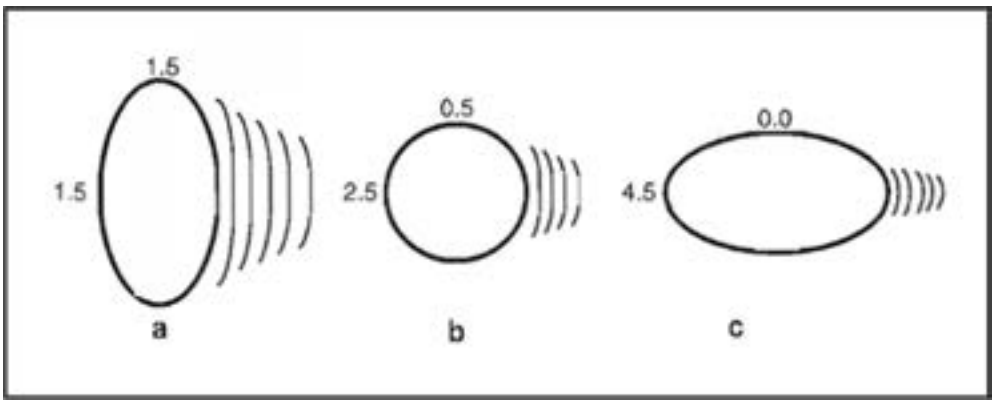


Figure 26: Influence of tunnel shape on fracture development and stress concentrations (Jager and Ryder, 1999)



An ellipse is not always practical under mining conditions and it is therefore suggested that excavations dimensions should always be made as small as possible, especially in the direction parallel to the maximum principal shape.

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CONNECTION 39

Refer to Paper 1 Outcome 5.8 on page 177

The stress fracturing is driven by the stress around the tunnel, i.e. tangential stress.

CONNECTION 40

Refer to Paper 2 Outcome 7 on page 529

Excavation technique

Excavation technique is very important in ensuring a stable tunnel. Blasting creates and propagates blast induced fracturing. These fractures can easily extend into the excavation walls and case 'overbreak', resulting in larger excavation dimensions or poor excavation shapes, realising all the concerns discussed for excavation sizes and shapes above.

Specialized blasting techniques (smoothwall, post split etc) need to be employed so as to ensure maximum control of the excavation perimeter and therefore also its stability.



- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.2.11

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIMISATION OF TUNNEL SIZE, SHAPE AND ORIENTATION TO SUIT GEOLOGICAL CONSIDERATIONS

The recognition of geological structures in all mining environments is a critical part of the excavation design in terms of its position, orientation, size and shape.

Geological condition	Shallow operations	Deep operations
Rock type, strength and quality	Place tunnels in best rock mass and reduce excavation dimensions when in poorer rock strength and quality material. Attempt stable arch shapes in really poor material to make maximum use of self-supporting capacity of the material.	Place tunnels in best rock mass and reduce excavation dimensions (fracture depth) and shape (reduce fracturing) when in poorer rock strength and quality material due to expected stress fracturing.
Bedding planes	Bedding planes provide weakness planes to which rock is inclined to collapse. Dense bedding plane spacings suggest potentially unstable hangingwall and excavation width must be controlled. Thick beddings (large bedding plane spacings) suggest possible scaling (with stress fractures) or wedge creation (with joints) concerns in sidewalls. Limit excavation height as far as possible. Orientate excavation to best possible stability in relation to bedding plane intersection ('lean-to' or 'factory roof' shape).	

back

Geological condition	Shallow operations	Deep operations
Faults	Wedge creation in tunnel that must be stabilised with support. Reduce exposure distance and thus and wedge size by implementing the smallest possible excavation size. Orientate tunnel to ensure smallest possible fault exposure (i.e. intersect at 90° if possible). Often rock mass has poorer quality around structures (see rock quality comments above).	Wedge creation in tunnels that must be stabilised with support. Reduce exposure distance and thus and wedge size by implementing the smallest possible excavation size. Orientate tunnel to ensure smallest possible fault exposure (i.e. intersect at 90° if possible).Increased potential for seismic activity on structure requires careful look at intersection and final excavation size. Smallest exposure distance limits extent of damage while potential sidewall closure must be accommodated (i.e. sufficient excavation size is required to prevent the excavation from being too small to allow travelling).
Dykes	Wedge creation in tunnels that must be stabilised with support. Reduce exposure distance and thus and wedge size by implementing the smallest possible excavation size. Orientate tunnel to ensure smallest possible fault exposure (i.e. intersect at 90° if possible). Often rock mass has poorer quality around structures (see rock quality comments above).	Wedge creation in tunnel hangingwalls that must be stabilised with support. Reduce exposure distance and thus and wedge size by implementing the smallest possible excavation size. Orientate tunnel to ensure smallest possible fault exposure (i.e. intersect at 90° if possible). Increased potential for seismic activity on dyke requires careful look at intersection and final excavation size. Smallest exposure distance limits extent of damage while potential sidewall closure must be accommodated (i.e. sufficient excavation size is required to prevent the excavation from being too small to allow travelling). Consider T-shaped excavations or ledging of tunnels in dyke intersection areas to cater for potential seismic damage. Refer to Outcome 3.2.12
Jointing	Affects rock mass quality (see rock quality comments above). Creates wedges in tunnels that must be stabilised with support. Reduce exposure distance or prominent joints and thus and wedge size by implementing the smallest possible excavation size. Orientate and change shape of excavation to ensure smallest possible wedge dimension (i.e. evaluate using stereonet or computer simulation packages to determine optimum orientation). Refer to Paper 2 Outcomes 3.1 on page 93 and 6.4 on page 497	Affects rock mass quality (see rock quality comments above). Creates wedges in tunnels that must be stabilised with support. Reduce exposure distance or prominent joints and thus and wedge size by implementing the smallest possible excavation size. Orientate and change shape of excavation to ensure smallest possible wedge dimension (i.e. evaluate using stereonet or computer simulation packages to determine optimum orientation). Refer to Paper 2 Outcomes 3.1 on page 93 and 6.4 on page 497

Table 10: Impact of geology on tunnel layouts

CONNECTION 41

Refer to Paper 2 Outcome 5.1.9 on page 331



- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.2.12

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIMISATION OF TUNNEL SIZE, SHAPE AND ORIENTATION TO SUIT FIELD STRESS CONSIDERATIONS

Where an excavation is located in a high stress environment, tunnel shape changes becomes a critical solution. Since the occurrence (intensity) and depth of fracturing should be minimised to assist in the stabilisation of the excavation, different tunnel shapes have been proved to be successful. Since excavation dimension does not affect the stress levels around the excavation significantly, it must only be reduced after careful consideration of the expected closure following the occurrence of stress fracturing. Smaller excavations subjected to high stress levels may find closure resulting in tunnels too small to be utilised for the required mining activities.



While most tunnels in deep level mines that are square and 3m and 5m in dimension have shown that stability can be maintained under most stress conditions, extreme high stress levels have resulted in the loss of these excavations. A method to counter the impact of these extreme stress levels, an excavation shape called the T-shape was adopted (Figure 27).

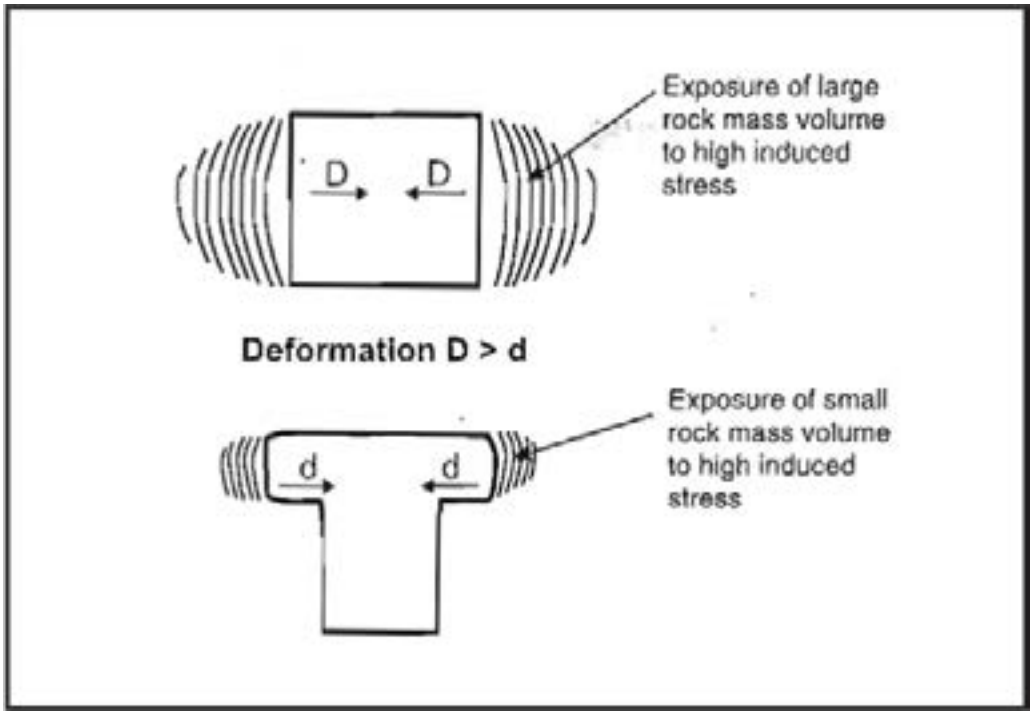


Figure 27: Deformation excavations undergo, under different cutting methods.

This T shape :

- reduces the impact of stress on tunnel sidewall fracturing as the lower sidewalls are actually distressed by the 'T' ledge at the top of the tunnel;

- closure will be limited as freedom to move into the excavation is limited by the small vertical dimension of the 'T' ledges;
- additional (even timber based) support can be installed on the 'T' ledges to inhibit scaling of the 'T' ledges sidewalls without affecting the use of the tunnel.



- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.2.13

DESCRIBE, EXPLAIN AND DISCUSS THE BEHAVIOUR OF ROCK SURROUNDING TUNNELS AT DEPTH DURING SEISMIC EVENTS

A seismic event emits seismic waves that travel through the rock mass and therefore also interact with tunnels. When the strain wave and tunnel interact, the following occurs:

- the rock around the tunnel is subjected to compression and tension due to the seismic wave;
- the compression can cause the creation of additional fracturing while the tension loosens the damaged rock material;
- rock under compression may burst (stiff material types) and eject into the tunnel;
- rock under tension loses its ability to maintain stability, negates most of the strengthening of the rock mass provided by support and rock is often deposited into the tunnel.
- support units are stressed (tension and compression) and may fail, removing or reducing its assistance in maintaining stability.



Seismic wave interaction with rock material already under high levels of stress, such as under a stoping abutment, will easily fail, increasing fracture intensity and often also ejecting rocks into the tunnel.

Placement of tunnels as remote as possible from seismic sources (dykes / faults, etc) and outside higher stress abutments (distress tunnels) reduce the potential of significant seismic damage to the tunnel. This is the result of attenuation of the seismic wave with distance travelled by the wave (amplitude of the wave, i.e. displacements induced, reduces as the wave travels) and lower stress levels on the rock suggests that deformation of the rock must be significant before additional failure will occur.



Refer to the stress / strain relationships for rocks as described in Paper 1 material and the fact that if a brittle rock is subjected to deformation, it will fail after reaching some limit. Since rock under stress has already taken up some of the deformation that the rock can sustain, the seismic wave could induce just enough deformation to cause substantial rock failure. The same happens with the installed support when high stress levels caused displacement of the rock, straining the tendon units to some level. If the seismic wave adds to this strain, the unit can fail after reaching its tensile strength limit.

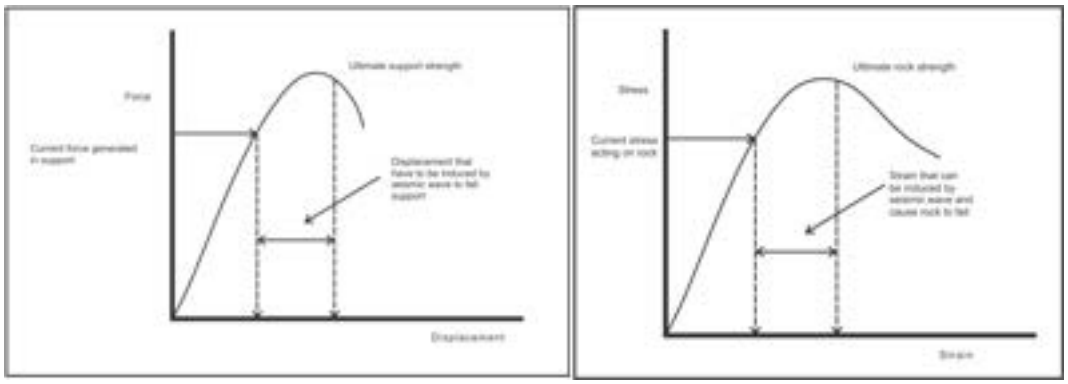


Figure 28: Impact of additional strain / displacement on (a) rock and (b) support

REFERENCE

- GAP530 Rockburst site response
- Page 97, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.2.14

DESCRIBE, EXPLAIN AND DISCUSS THE SITING OF TUNNELS WITH RESPECT TO EXISTING, CURRENT AND FUTURE STOPING OPERATIONS

CONNECTION 42

Refer to "LEARNING OUTCOME 3.1.17", and "LEARNING OUTCOME 3.2.11" to 3.2.13

When siting tunnels, good practice suggests the following:

- A good understanding of the current and future stress levels:
  - Siting of tunnels such that current and future stress levels will be as low as possible, but also that stress changes are limited. Stress changes refer to situations where an excavation can be stressed and then de-stressed. This has been shown to be quite detrimental to the stability of the tunnel as rock crushed by the high stress levels displaces into the tunnels when stress levels are removed. This is usually associated with extreme closing of the tunnel as crushed rock sags into the excavation.
- Knowledge on the possibility for and level of seismic activity:
  - Placement remote from seismic sources reduces the potential for damage. If an excavation must traverse a potential seismically active structure, precautions in terms of limiting exposure distance, changing tunnel shapes and support upgrading is required.
- Agreement on the expected life of the tunnel:
  - The effort put into the siting of the tunnel is determined by the tunnel's expected life. Placement of a short life excavation is less critical while a life of mine tunnel will require substantial investigation before placing the tunnel.

REFERENCE

- Chapter 5, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.2.15

CONNECTION 43

Refer to LEARNING OUTCOME 3.2.11 on page 103

DESCRIBE, EXPLAIN AND DISCUSS THE SITING OF TUNNELS WITH RESPECT TO GEOLOGICAL STRATIGRAPHY AND STRUCTURES

LEARNING OUTCOME 3.2.16

CONNECTION 44

Refer to Paper 2 Outcome 5.1.9 on page 331

DESCRIBE HOW ROCKWALL CONDITION FACTOR (RCF) IS DETERMINED

LEARNING OUTCOME 3.2.17

CONNECTION 45

Refer to Paper 2 Outcome 5.1.9 on page 331

DESCRIBE AND DISCUSS HOW THE ROCKWALL CONDITION FACTOR MAY BE APPLIED TO PREDICT TUNNEL STABILITY AND SUPPORT REQUIREMENTS

LEARNING OUTCOME 3.2.18

CONNECTION 46

Refer to Paper 2 Outcome 5.1.9 on page 331

APPLY THE ROCKWALL CONDITION FACTOR TO PREDICT TUNNEL STABILITY AND SUPPORT REQUIREMENTS AT DEPTH

The Rockwall Condition Factor is used to estimate stress induced damage which will be experienced in developments at certain depths. To predict the σ_1 and σ_3 values for a proposed layout, numerical modelling techniques are used.

EXAMPLE

For applying the RCF methodology, the following excavation was modelled. The size of the stoped out area is 824m by 288m, with in-stope pillar dimensions 8m x 8m, with 8m bords and splits and regional pillars 20m x 24m. Other input parameters include 70 GPa Young's modulus, 0.2 Poisson's ratio, 1.75 k-ratio, 3100kg/m3 density, 0° dip reef and mining at a depth of 800mbs.

Placement of footwall tunnels were evaluated along a two possible orientations and positions (BB' and DD' in Figure 29) using the RCF concept. The concern was the impact of stress levels generated on the regional pillars along BB' on footwall tunnels after mining (in-stope pillars still stable) and in the long term (assumed in-stope pillars could fail).

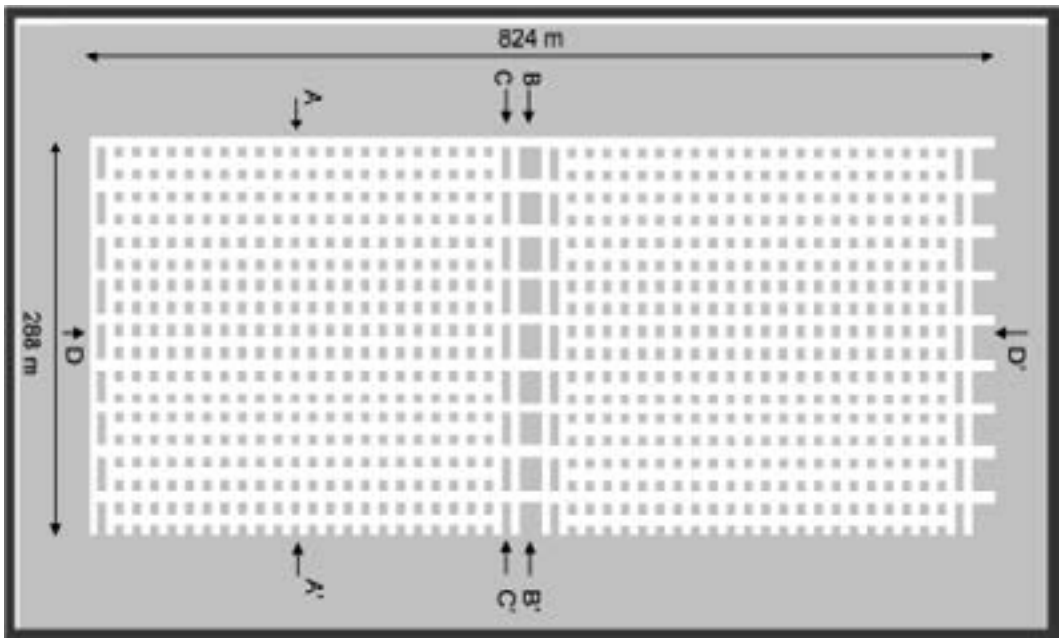


Figure 29: Geometry of the example layout

The graphs depicted below (Figure 30 through Figure 33) show the RCF values along different sections of the stope for each of the potential positions below the reef (10m to 35m) with the stable in-stope pillar results shown on Figure 30 and Figure 31 and the failed in-stope pillar results on Figure 32 and Figure 33.

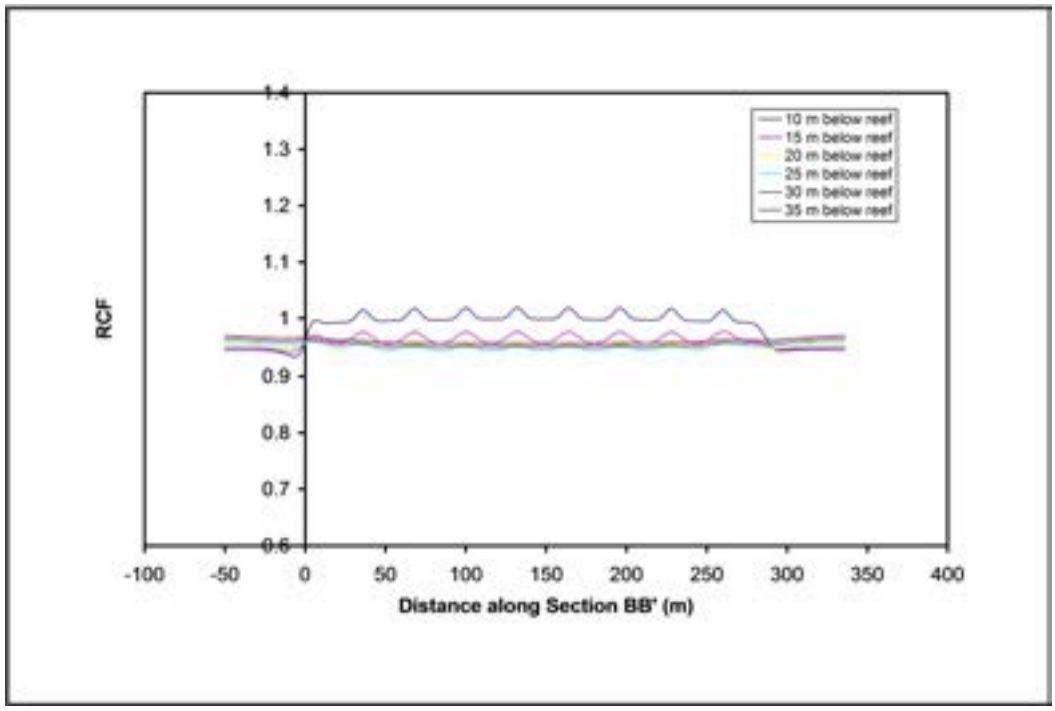


Figure 30: RCF values along Section BB' (stable in-stope pillars)

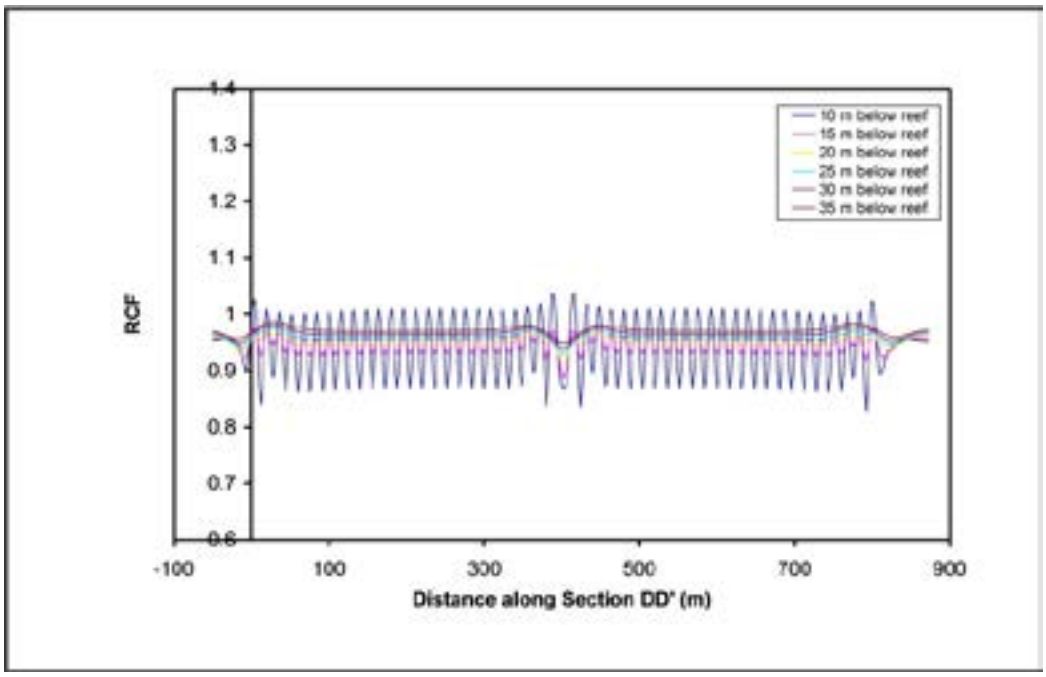


Figure 31: RCF values along Section DD' (stable in-stope pillars)

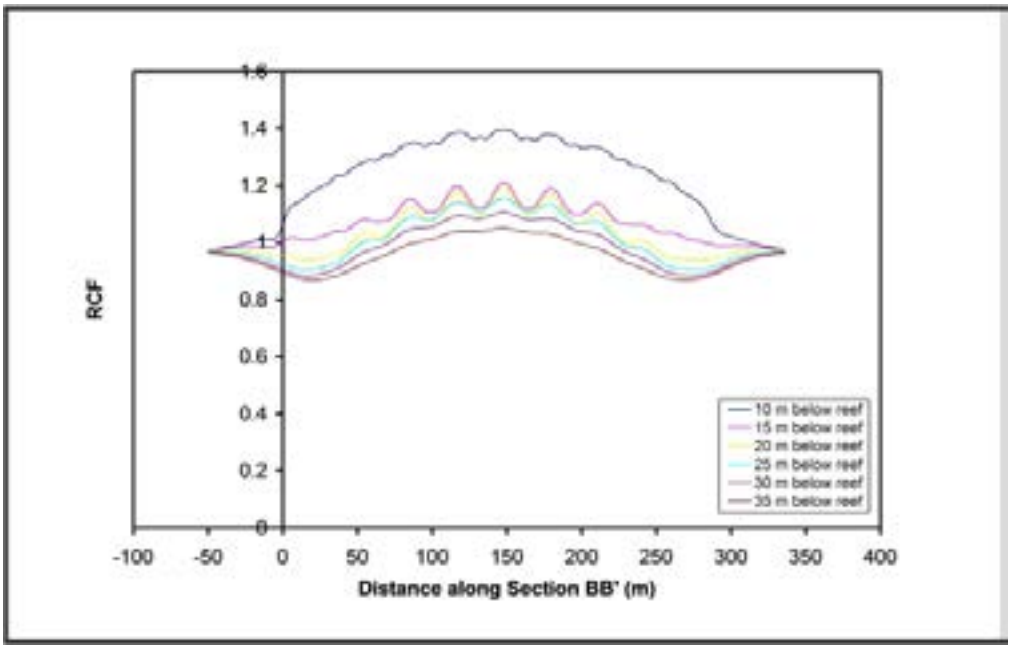


Figure 32: RCF values along Section BB' (failed in-stope pillars)

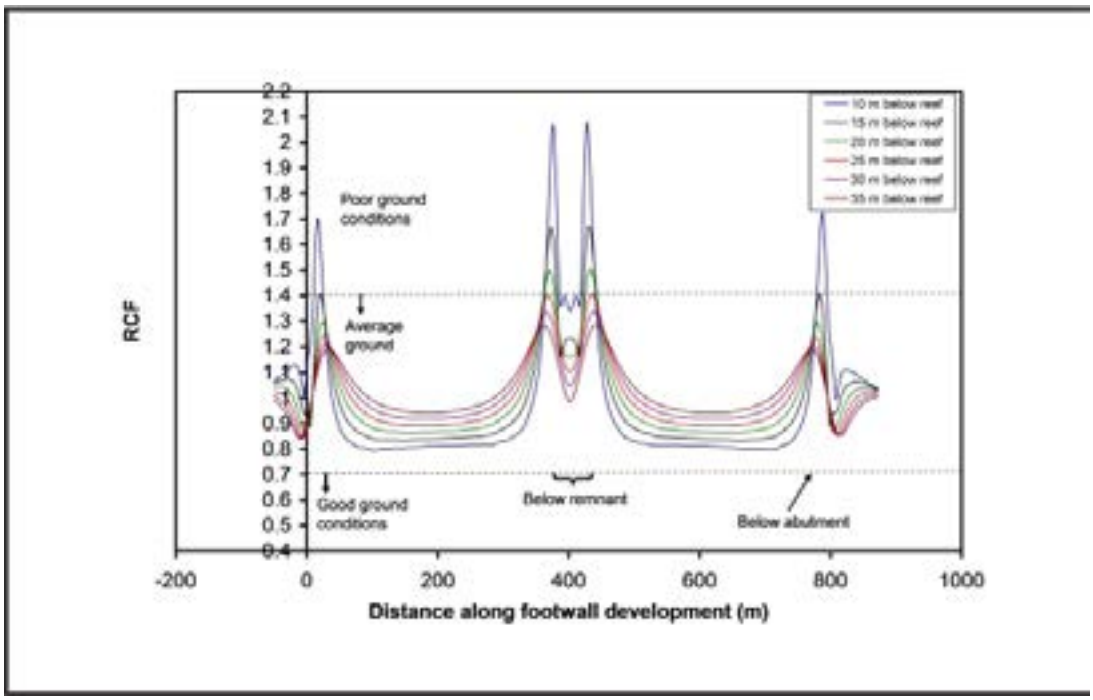


Figure 33: RCF values along Section DD' (failed in-stope pillars)

For a life of mine placement of the tunnels, it was decided that the RCF values obtained should be below 1.4. It is clear that as long as the in-stope pillars remain stable, RCF values remained at around 1, but with failed in-stope pillars the 1.4 RCF criterion was exceeded as soon as the tunnel was placed closer than 25m from the reef. It is therefore recommended that the middling between the reef and haulage be a minimum of 25m.

CONNECTION 47

Refer to Paper 2 Outcome 5.1.9 on page 331

REFERENCE

- Pages 58, 82, 195, 203, 218, 314, 339, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 157, 377, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.

LEARNING OUTCOME 3.2.19

CONNECTION 48

Refer to LEARNING OUTCOME 3.2.1 on page 94 to 3.2.18

Refer to Paper 2 Outcome 5.1.9 on page 331

EVALUATE GIVEN ROCK CONDITIONS AND FIELD STRESSES TO PREDICT EXCAVATION STABILITY

LEARNING OUTCOME 3.2.20

CONNECTION 49

Refer to LEARNING OUTCOME 3.2.17 on page 108
Refer to Paper 2 Outcome 5.1.9 on page 331

EVALUATE GIVEN ROCK CONDITIONS AND FIELD STRESSES TO RECOMMEND REMEDIAL MEASURES TO IMPROVE EXCAVATION STABILITY.

3.3 ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS

LEARNING OUTCOME 3.3.1

CONNECTION 50

Refer to Paper 1 Outcome 6.2 on page 200

Refer to Paper 2 Outcome 3 on page 93

Refer to Paper 2 Outcome 5.1.9 on page 331

DESCRIBE, EXPLAIN AND DISCUSS HOW ROCKS AND ROCKMASSES MAY BE CLASSIFIED

LEARNING OUTCOME 3.3.2

CONNECTION 51

Refer to Paper 1 Outcome 6.2 on page 200

Refer to Paper 2 Outcome 3 on page 93

Refer to Paper 2 Outcome 5.1.9 on page 331

DESCRIBE, EXPLAIN AND DISCUSS HOW ROCKMASS CLASSIFICATION SYSTEMS MAY BE USED TO PREDICT SERVICE EXCAVATION STABILITY AT SHALLOW, INTERMEDIATE AND GREAT DEPTHS

LEARNING OUTCOME 3.3.3

DESCRIBE, EXPLAIN AND DISCUSS THE NATURE OF FRACTURING AROUND VERTICAL SINKING SHAFTS AT DEPTH

CONNECTION 52

Refer to LEARNING OUTCOME 3.2.1 on page 94

Stress fracturing is the result of the stress field and the rock / rock mass strength. Once the stress levels exceed the strength of the material, stress fracturing occurs. When fracturing occurs around a horizontal tunnel, it has been shown in Outcome 3.2 that the tangential stress (i.e. stress around the tunnel) drives the fracturing that occurs and that the stress component parallel to the tunnel axis does not affect the fracturing around the tunnel.

In the same way, stress fracturing around a vertical shaft occurs due to the stress field components perpendicular to the shaft axis, i.e. the two horizontal components. Stress fracture patterns around the shaft would then follow exactly the same patterns, depending on the shaft barrel shape (circular, square or rectangular), as around horizontal tunnels if a section view through the barrel is taken (as in).



Plane stress conditions can be assumed along the shaft axis, as only the x-y plane (horizontal plane) stress components affect fracturing while the z component (depth related) only contributes to the horizontal stress components (k ratio) but not to the fracture orientations.

INTERESTING INFO

Kirsch equations can be used to determine the level of tangential stress around a shaft barrel (Paper 1 Outcome 5.1.8) and could give, depending on the input parameters a stress distribution as shown in Figure 34. This distribution shows an increase in tangential stress in some portion of the shaft walls to 2.75q (that is 2.75 times the maximum virgin horizontal stress). When these stress levels exceed the strength of the rock, stress fracturing would occur and if scaling is allowed, 'dog- earing' will result (Figure 35) with scaling at the ends of the excavation that are perpendicular to the largest stress.

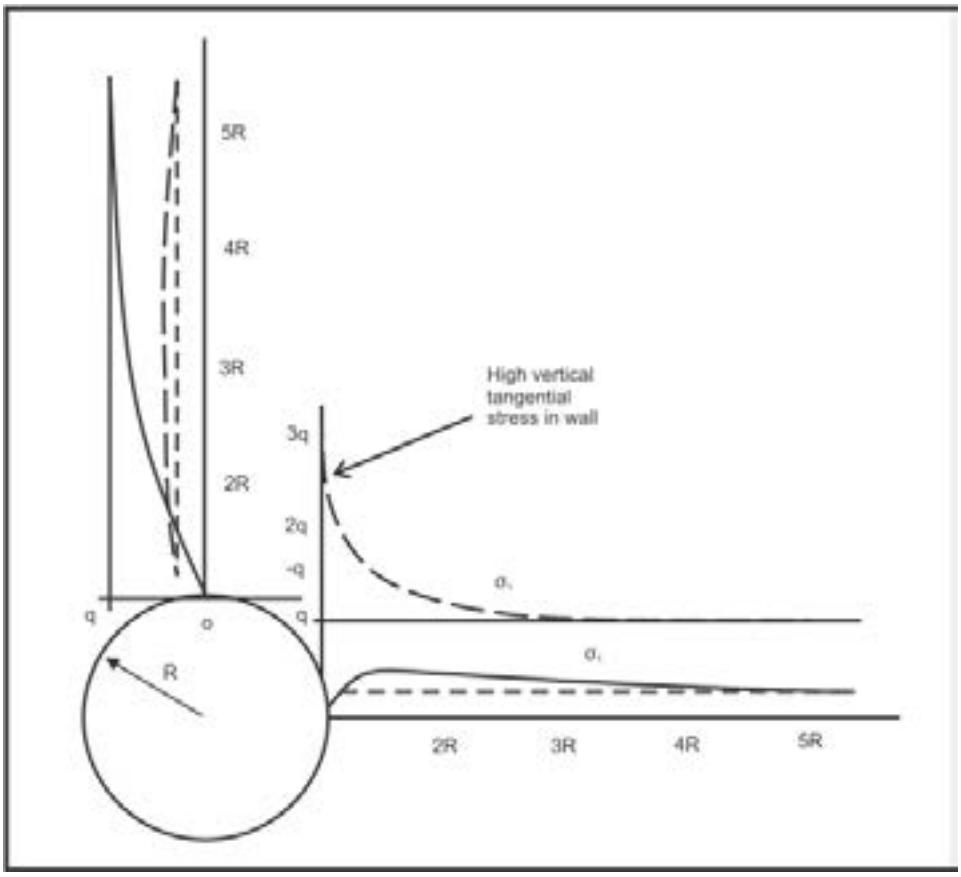


Figure 34: Distribution of stress around a circular hole (Budavari S 1986)

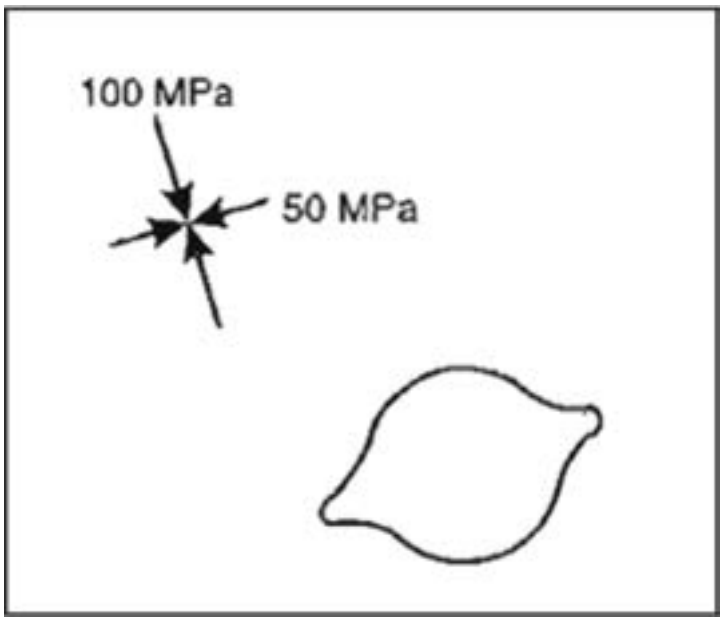


Figure 35: Dog earing

REFERENCE

- Chapter 4, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Pages 36, 236, 321, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 10, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.3.4

DESCRIBE, EXPLAIN AND DISCUSS THE NATURE OF FRACTURING AROUND INCLINED SINKING SHAFTS AT DEPTH

Incline shafts are generally square to rectangular in shape. Unlike a circular or elliptical excavation, stress concentrations are more complicated and higher than around a square excavation, especially around the corners. Fracture patterns will emulate that around square and rectangular tunnels.

LEARNING OUTCOME 3.3.5

DESCRIBE, EXPLAIN AND DISCUSS THE NATURE OF FRACTURING AROUND ROCK PASSES AT DEPTH

Rock passes (orepasses / boxholes) are circular in shape when bored (raise bore / blind bore) but are often square to rectangular, depending on the layout and development method preferred at a specific mine. Fracturing around the rock passes would be similar to those around vertical shafts, except that rock passes are often not vertical, suggesting that all three stress components could fracturing around the rock pass.

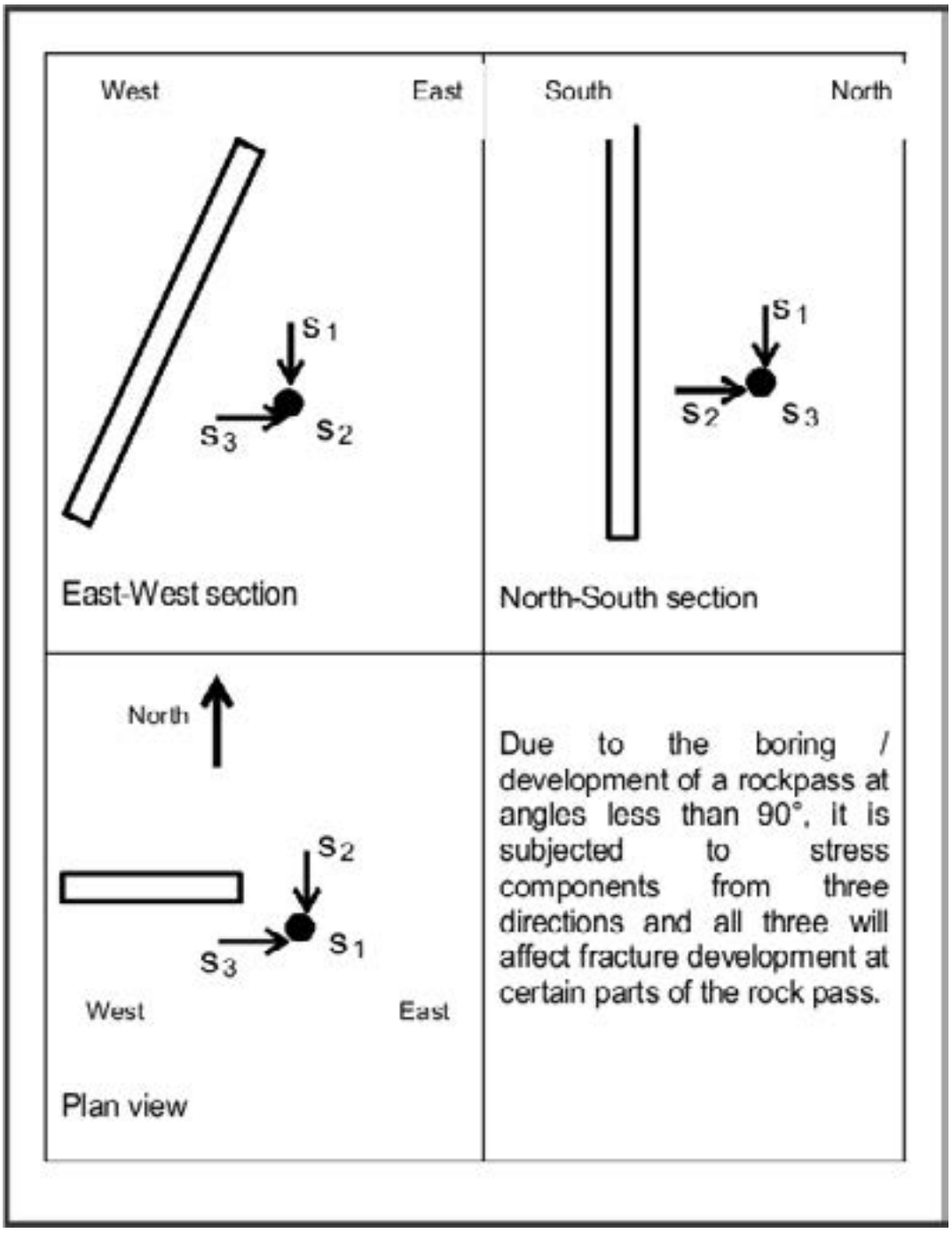


Table 11: Example of stress components affecting rock pass fracturing

Stress fracturing and especially 'dog-earing' is very common as rock passes are often not supported and scaling is assisted by the flow of rock down the rock pass. The result is normally extensive rehabilitation after a period of years which include the installation of tendons and shotcrete to reduce wear.



Figure 36: Orepass 'dog-earing' (Brenchley, 2006)

REFERENCE

Orepass scaling (Brenchley)

INTERESTING INFO

Due to rock flow in a rock pass, the walls are usually subjected to friction (sliding of rock against the rock pass walls) and impact (rocks bouncing off the walls) related scaling.

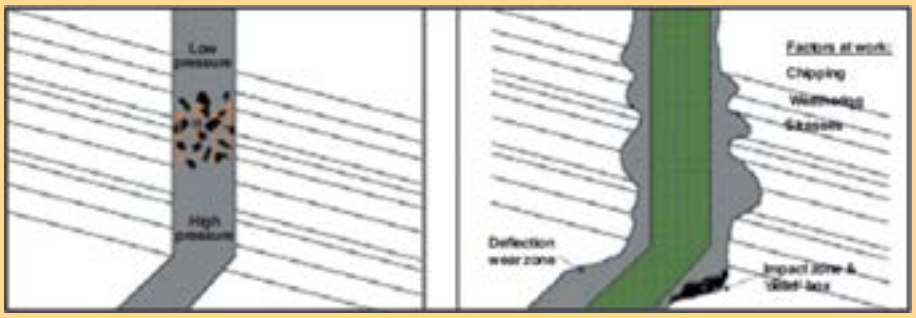


Figure 37: Normal abrasive driven scaling in orepasses (Brenchley, 2006)

When scaling occurs as described earlier and the rock pass is rehabilitated, high abrasive resistant shotcrete is often applied. The 'abrasiveness' of any shotcrete material is affected by the compressive strength of the shotcrete, the cement type used, the presence or not of fibres and the aggregate used to improve abrasiveness.
High abrasive materials are divided into 'frictional abrasive' or 'impact abrasive' types. The former type suggests that it is resistant to abrasion that might occur when materials slide past each other and where friction is the main driver of wear. The latter type represents those materials that appear abrasive when impacts between material particles occur. It has been suggested that the type of abrasion that occurs in rock passes include both 'impact' and 'friction'.
Increased strength of the shotcrete is usually related to increased abrasive resistance. However, at high strengths other factors also start to affect abrasiveness, such as cement type, fibres and aggregates.
The inclusion of fibres in shotcrete has two results, namely increased energy absorption capacity and increased impact resistance. Since impact resistance is related to the ability of the material to sustain impact with

other materials and not to fracture in the process, the addition of fibres assist absorption of energy during impact and prevent failure, improving the impact resistance of the material. The higher the impact forces, which is also a function of the ore density, the more important this abrasion resistance becomes.
High 'hardness' of material (as measured on Moh's scale) is related to frictional abrasiveness while the materials' ability 'not to fracture' when impacted on, affects its ability to be impact resistant. This 'ability not to fracture' is not easily measured, but can be estimated by using particles of specific maximum dimension (Dmax). When Dmax is smaller than the 'critical dimension' (smallest dimension such that the particle will not have an inherent weakness plane within the particle), it is stated to be very resistant against fracturing when impacted on. Since the 'critical dimension' is a function of the geological nature of material, it can be suggested that igneous rock types, such as andesite, would be more 'resistant to fracture' than schist (which is layered) material.

REFERENCE

- Pages 241, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

LEARNING OUTCOME 3.3.6

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF STRESS FRACTURING AND GEOLOGICAL STRUCTURE ON THE FINAL SHAPE OF VERTICAL AND INCLINED SHAFTS

CONNECTION 53

Refer to LEARNING OUTCOME 3.3.3 on page 112 and LEARNING OUTCOME 3.3.7 on page 117

Stress fracturing and geological structures will, if left uncontrolled, create similar excavation shapes in shafts and orepasses. The large difference is normally that shafts are well supported during sinking with tendons, mesh, shotcrete and the concrete lining, all of which inhibits the scaling of the shaft walls. However, the intersection of very poor rock types will create a similar shape to that shown for orepasses intersecting weak material types in Table 12.

LEARNING OUTCOME 3.3.7

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF STRESS FRACTURING AND GEOLOGICAL STRUCTURE ON THE FINAL SHAPE OF ROCK PASSES

Geological structures affect scaling of rock pass walls due to:

- Poor rock mass quality close to / in the structure due to increased jointing or metamorphism of the host rock;
- Significant planes of weakness along dyke contact / fault plane due to infilling and continuous nature of large structures;
- Small block dimensions increase possibility of loosening and scaling when impacted on by flowing / bouncing rock material;
- Tectonic stress levels around large structures assist fracturing;
- Contact planes provide a point of freedom that can allow rock movement and scaling.

When significant geological structures are present, scaling around the structure is accentuated due to the reasons given above. This accentuation means that scaling is more pronounced close to these structures than in areas of the rock pass remote from the structure.

- Orepass scaling (Brenchley)

The scaling caused by geological structures in orepasses is depicted in the diagrams below:

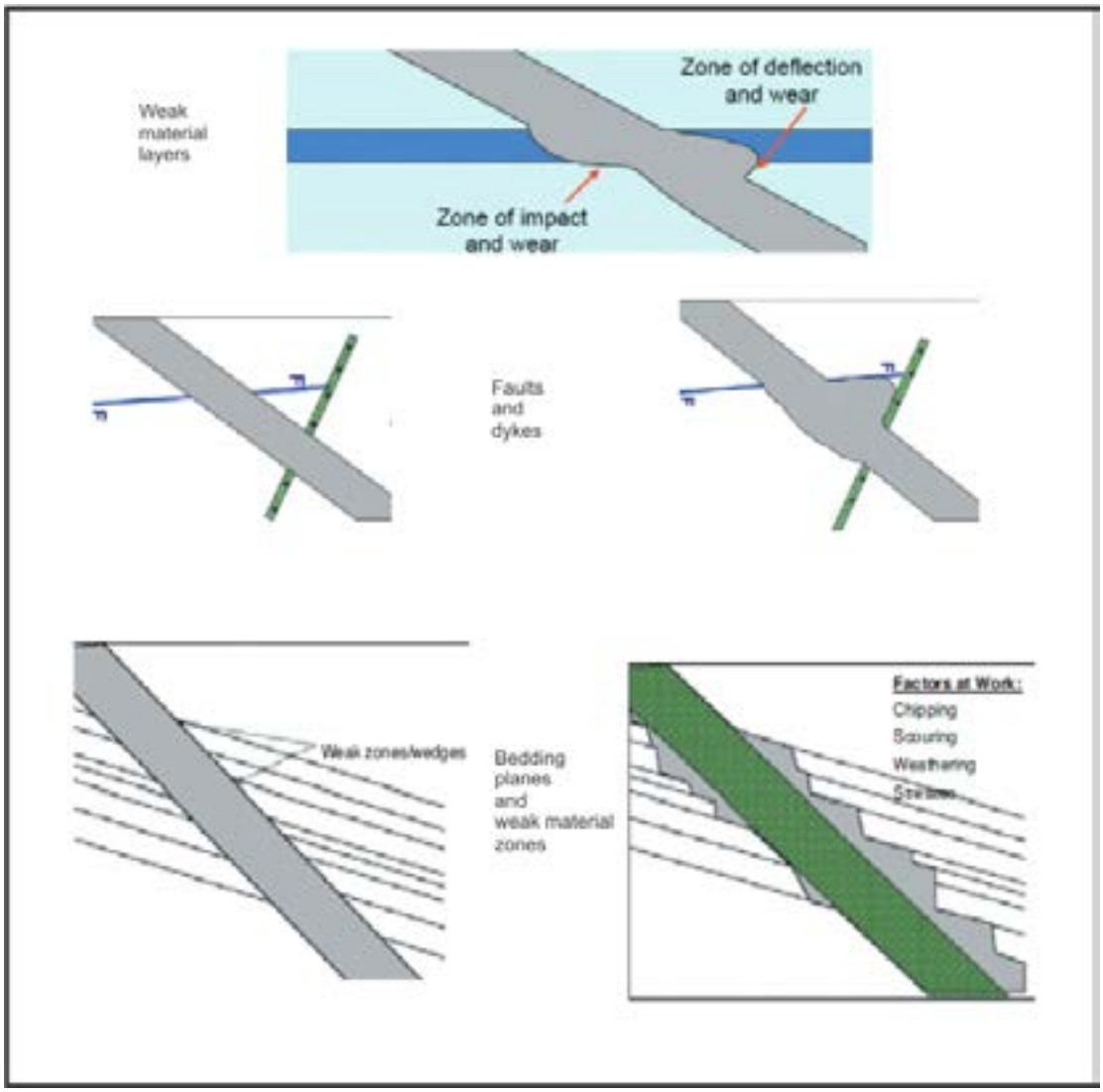


Table 12: Geology driven scaling

Orepass scaling is a major issue on underground mines and the causes for scaling and rehabilitation measures have been documented on many mines. It is suggested that the user search for reports on their mine or nearby mines that describes the scaling and rehabilitation measures for a specific environment.

LEARNING OUTCOME 3.3.8

DESCRIBE, EXPLAIN AND DISCUSS THE CONDITIONS THAT MAY LEAD TO THE FORMATION OF LARGE UNSTABLE ROCK WEDGES IN VERTICAL SHAFT SIDEWALLS

CONNECTION 54

Refer to LEARNING OUTCOME 3.3.3 on page 112 LEARNING OUTCOME 3.3.5 on page 114 and LEARNING OUTCOME 3.3.7 on page 117.

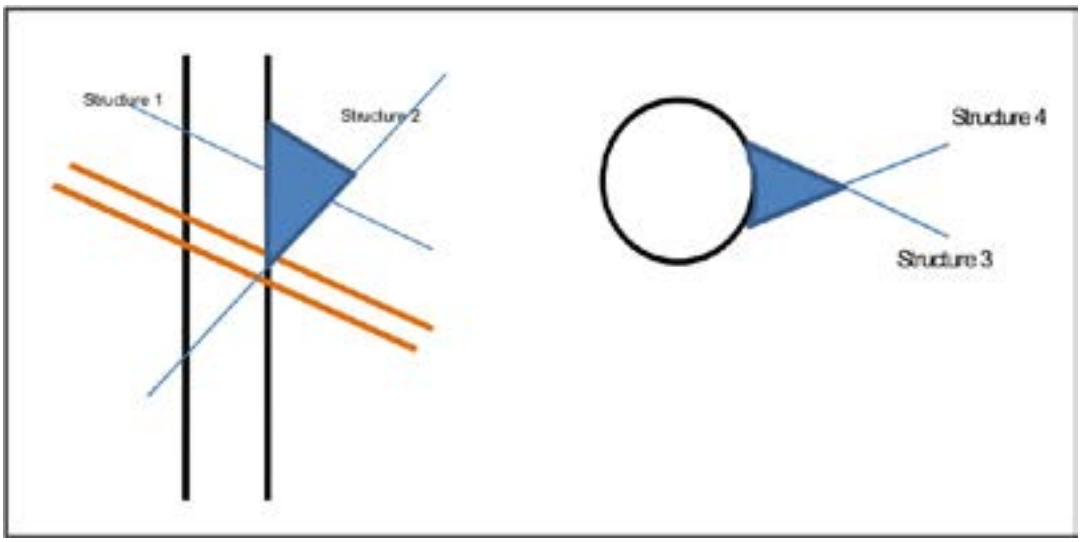


Figure 38: Formation of wedges in a shaft barrel by a number of structures

Wedges are mostly the result of intersection between the shaft walls and geological structures such as bedding planes, dykes, faults or very prominent and continuous joints. These wedges will remain stable if they can not move mechanistically, i.e. if they are formed in such a way that displacement on the contacts is impossible. However, if enough degrees of freedom exist (4 structures), the wedge will be free to move into the barrel and can cause lining failure in extreme cases.

LEARNING OUTCOME 3.3.9

DESCRIBE, EXPLAIN AND DISCUSS THE CONDITIONS THAT MAY LEAD TO THE FORMATION OF INSTABILITIES IN ROCK PASSES

CONNECTION 55

Refer to LEARNING OUTCOME 3.3.5 on page 114 and LEARNING OUTCOME 3.3.7 on page 117.

Except for conditions discussed in previous outcomes (stress and geology), the following also contribute to rock pass instability:

- When rock passes exceed lengths of 70m, impact abrasion due to falling rocks and friction abrasion due to large weight of the material flowing through the rock pass become severe and may result in the fracturing and hence fall-out of rock fragments from the sidewall.
- Another factor that is commonly forgotten is the placement of rock passes in relation to other infrastructure (excavations). Stress interaction between the orepass and other excavations increase potential for stress fracturing and scaling.
- Water flowing through then rock pass reduces the strength of geological discontinuities exposed in the rock pass walls. This reduction in strength can easily result in scaling when displacement is induced on the discontinuities by falling and sliding rock material.
- It is good practice to keep a rock pass as full as possible, as that reduces the impact abrasion and therefore also the potential for scaling. By emptying a rock pass, conditions are created where the next material tipped into the rock pass fall through large distances and exert large forces on the walls at the point of impact.
- Blasting within rock passes when material 'hangs up' is not a great practice to ensure stable rock pass walls. The blasting causes significant air pressure (shock wave) and vibration within the rock pass, both of which increase potential for scaling. At

the same time, the material loosened by the blast falls collapses down the rock pass and impacts on the rock pass walls, increasing potential for scaling.



- McCracken and Stacey Geotechnical risk assessment for shafts
- Improving raisebore risk assessments, Coombs et al.
- Fine tuning risk assessments for shafts, Coombs et al.
- Rock pass design, Stacey
- pp. 69–79. HAGAN, T.O. and ARCHEAMPONG, E. Current design, support and maintenance of rockpasses and assessment of practices applicable at depth, Proc. SARES 99, 2nd Southern African Rock Engineering Symp., S. Afr. Nat. Inst. Rock Eng, 1999.

LEARNING OUTCOME 3.3.10

DESCRIBE, EXPLAIN AND DISCUSS THE NATURE OF FRACTURING AROUND SERVICE EXCAVATIONS AT DEPTH

Service excavations normally have much larger dimensions than the standard tunnels on an operation. As discussed earlier, the size of an excavation does not affect the stress distribution around the excavation. However, service excavations at depth experience the maximum principal stress in the vertical direction and stress fracturing in the sidewalls of the excavation are created to a depth related to the height of the excavation. The conditions (fracture orientation) are similar to that formed around tunnels (Outcomes 3.2.3 to 3.2.7), except for the depth to which the rock is fractured into the excavation walls.



- Pages 225, 361, 322, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 9, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.3.11

CONNECTION 56

Refer to LEARNING OUTCOME 3.2.3 on page 94 to 3.2.7.

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF FRACTURING ON SERVICE EXCAVATION SIZE

If uncontrolled, scaling due to the fracturing in service excavations walls can be severe, increasing the dimensions of the excavation dramatically. However, service excavations are normally well supported to protect equipment, personnel and processes usually for long periods of time. This means that scaling is usually prevented and support maintains the stability of the fracturing, such that the excavation size will not be affected by the stress fracturing.



Support units are usually much longer in service excavations due to the greater depth of expected stress fracturing in the sidewalls and larger tensile zone in the excavation hangingwall.



- Pages 225, 361, 322, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 9, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.3.12

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF FRACTURING ON SERVICE EXCAVATION SHAPE

As mentioned in Outcome 3.2.4, fracturing induces dilation and deformation in the surrounding rock mass, reducing the excavation dimensions. However, due to the additional support usually installed in these excavations (often at large factors of safety), closure is limited and large scale closing of service excavations due to stress fracturing is not common.

LEARNING OUTCOME 3.3.13

CONNECTION 57

Refer to LEARNING OUTCOME 3.2.3 on page 94

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF FRACTURING ON SERVICE EXCAVATION STABILITY

The impacts of stress fracturing will be similar to that for tunnels.

LEARNING OUTCOME 3.3.14

CONNECTION 58

Refer to LEARNING OUTCOME 3.2.7 on page 97 and LEARNING OUTCOME 3.2.8 on page 98

DESCRIBE, EXPLAIN AND DISCUSS THE PHENOMENON OF TIME-DEPENDANT FRACTURING AND DETERIORATION

LEARNING OUTCOME 3.3.15

CONNECTION 59

Refer to LEARNING OUTCOME 3.2.7 on page 97 and LEARNING OUTCOME 3.2.8 on page 98

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS AND CONSEQUENCES OF TIME-DEPENDANT FRACTURING AND DETERIORATION ON SERVICE EXCAVATION STABILITY

LEARNING OUTCOME 3.3.16

CONNECTION 60

Refer to LEARNING OUTCOME 3.2.9 on page 99

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF STRESS REGIME AND GEOLOGICAL STRUCTURE ON EXCAVATION STABILITY

LEARNING OUTCOME 3.3.17

CONNECTION 61

Refer to LEARNING OUTCOME 3.2.10 on page 101

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF EXCAVATION SIZE, SHAPE AND ORIENTATION ON EXCAVATION STABILITY

LEARNING OUTCOME 3.3.18

CONNECTION 62

Refer to LEARNING OUTCOME 3.2.10 on page 101

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF EXCAVATION TECHNIQUE ON FRACTURING AND THE STABILITY OF SERVICE EXCAVATIONS

LEARNING OUTCOME 3.3.19

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF EXCAVATION SEQUENCE ON STRESS FRACTURING AND THE STABILITY OF SERVICE EXCAVATIONS AT DEPTH

The excavation sequence of a large chamber and choice of its associated support are closely inter-related with the size, shape and geotechnical setting of the excavation.

NOTE

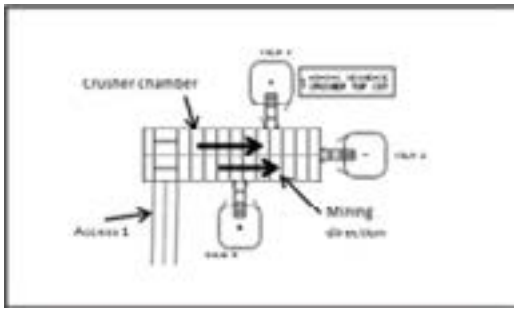
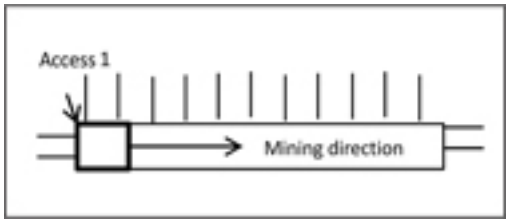
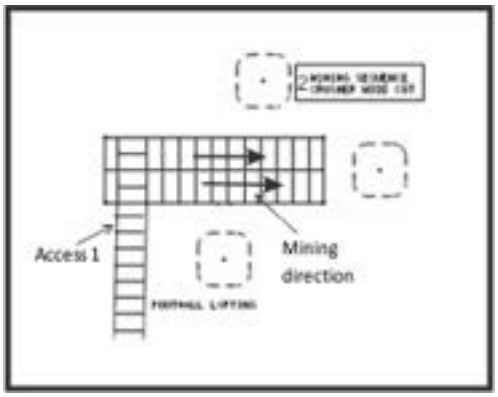
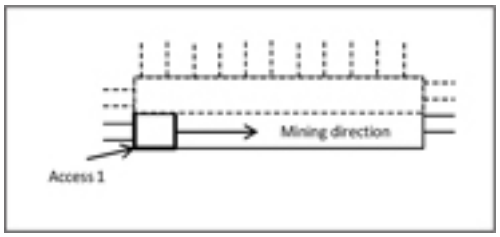
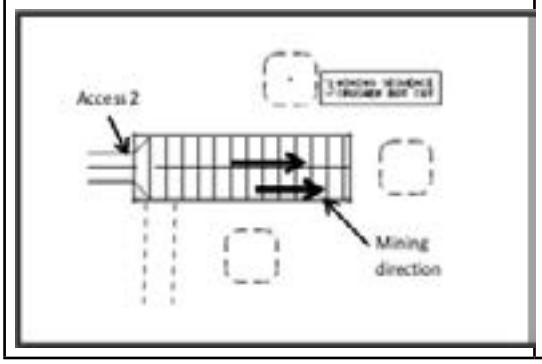
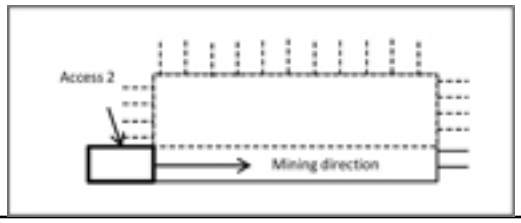
In large chambers where rock fracturing is slight and failure is controlled by geological structures (shallow environment), excavation sequencing may seem less important than in deep excavations where stress fracturing becomes a major concern. However, if the rock mass is extremely jointed, blocky conditions can result in very similar concerns such as those expected in deep excavations. Precautions in terms of the excavation sequence is thus discussed for deep excavations but can be just as appropriate for shallow mine service excavations.

The need to ensure that the excavated portion of an excavation remains stable is critical to the stability of the completed excavation. The sequence of excavating becomes an important factor in ensuring stability as it may or may not allow sufficient and appropriate support installation to maintain stability.

The rock mass around the excavation is naturally fragmented (jointed) and consists of rock blocks of different sizes. The occurrence of stress fracturing induced by elevated stress levels, around the excavation or portions of excavations, changes this 'natural' block size distribution and affect stability as well as required support practices. It is therefore beneficial to ensure that stress fracturing is limited or at least created in such a way that it assists stability.

EXAMPLE

The development of a large crusher chamber (shallow mining) is shown in Table 13 and is risk assessed in Table 14. This sequence should be viewed as a description of a possible sequence of development. However, the practices suggested are appropriate to all environments, even though the exact layout may not be the same for all large chambers.

Drafted plan view	Section views
	The first cut is taken at the top of the planned final excavation and also with two ends, the second end lagging the first by at least 5m. Mining occurs from access 1 and the final width and length dimensions of the crusher chamber are established on this elevation. Under high stress conditions, fracturing will occur around the two ends, but standard support practices will be able to ensure stability. The installation of secondary support (shotcrete, cable anchors, mesh, etc) into hangingwall and sidewalls is done basd on final dimensions of the excavation, as the ends advances and is not left until the final stage. 
	After completion of the support on the first cut, a 2nd cut is taken below the first cut. This level is accessed from same excavation as 1st cut but has been footwall lifted to allow access at this elevation. The cut is taken by footwall lifting / benching towards the final end of the excavation. Since these ends will be situated below the first cut (i.e. de-stressed), stress fracturing is no longer a concern. Support is installed in the exposed sidewalls as determined by the final excavation dimensions. 
	The final cut (depends on the final excavation height required) is taken similar to the second but along the bottom of the planned excavation. It is accessed from Access 2, and the final support is installed into the sidewalls. 

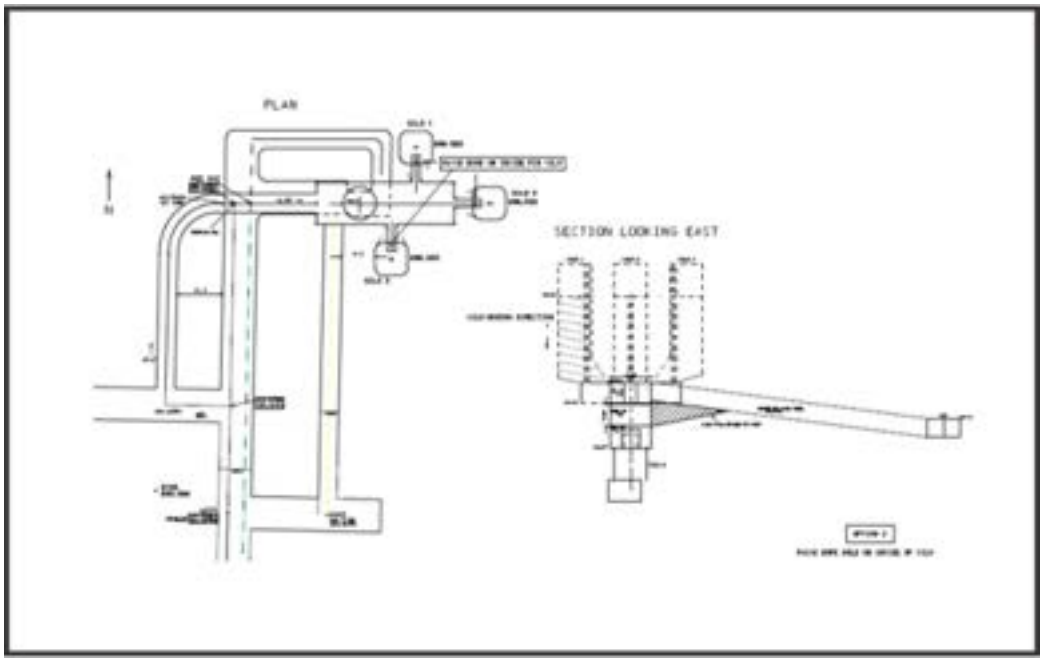
Drafted plan view	Section views
	

Table 13: Mining sequence of large chambers

Risk	Assessment	Remedial measures
Falls of ground during development in primary support area (face)	The occurrence of falls of ground on the face during development of the cuts will depend on the actual rock mass quality (geological structures and stress fracturing), rock material strengths and the installed support measures. Sufficient stand-up time is critical as secondary support will follow. Stress fracturing at depth will be limited to the hangingwall during cut 1 and sidewalls during cuts 2 and 3. Normal extension fractures such as found around development ends will occur and standard primary support measures will be sufficient to ensure stability, provided that secondary support follows immediately.	Primary support methods for the 1st cut should include bolting to protect against wedge and fractured block failures and areal coverage support to adequately support the smaller block dimensions if densely jointed or fractured. The proposed extraction method will mine the top cut first, allowing the installation of hangingwall support within a normal excavation height, using and applying standard excavation and supporting techniques. The completion of the 1st cut at the top of the planned excavation will also ensure that the following cuts are protected against the risk of small scale wedge collapses from the hangingwall and, since the same support will be installed into the exposed sidewalls, will limit the overall fall of ground risk whilst cutting the middle and bottom cuts.
Creation of large spans during development exceeding primary support capability	The proposed extraction method allows the extraction of the top cu) with two drives, each approximately 5m wide, lagging each other by at least 5. This ensures that large spans (10m) are not created with a single blast and allows stabilisation of the hangingwall in each face using primary support. As the large spans are created behind the lagging face, the installed primary support capacity will be exceeded and additional support installation is warranted.	Secondary support installation should follow as close to the advancing faces as possible. The proposed method thus allows the installation of long term support whilst the excavation height is limited to that of the first cut and whilst standard support installation and quality control measures can easily be performed. The application of areal coverage support during this 1st cut is also possible and easy to perform due to the standard excavation height. Sidewall support during the cuts will be easy to install in the dimensions created with each cut whilst also under the protection of the already supported hangingwall and sidewalls of the previous cuts.
Difficulty with support installation	Poor support installation due to excessive excavation dimensions could rick the long term stability of the chamber.	The proposed method will ensure that support can be installed within practical distances from the advancing faces and will prevent exposure of unsupported rock walls since the next cut cannot be taken if all support, at the required density, thickness and quality, had not been installed in the cut taken.

Table 14: Risk assessment based on mining sequence of large chambers

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REFERENCE

- Pages 225, 361, 322, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Chapter 9, Budavari S, Rock mechanics in practice, SAIMM, 1983

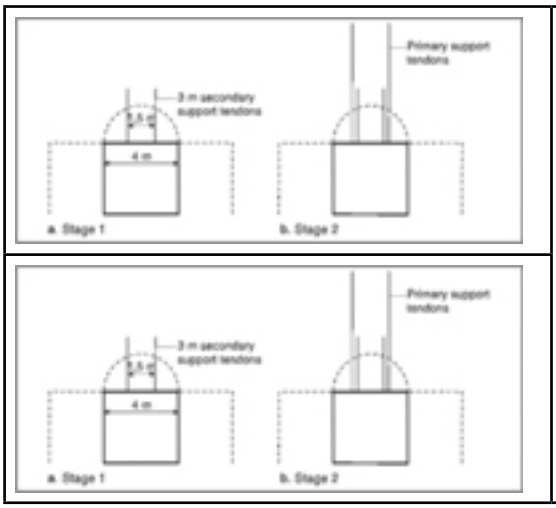
LEARNING OUTCOME 3.3.20

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF SUPPORT SEQUENCE ON STRESS FRACTURING AND THE STABILITY OF SERVICE EXCAVATIONS AT DEPTH

The support sequence has no impact on the creation or formation of stress fractures, but is critical in controlling the fractured rock mass.

EXAMPLE

Figure 39 shows an example of a support installation sequence whilst creating a standard height, but large width excavation. Even though is an example only, the principles remain the same independent of the excavation dimensions or purpose.



Principles:
- Install primary support as determined for a standard dimension excavation.
- Install 'final' excavation dimension support (assuming dotted line dimensions indicate final dimensions) prior to enlarging the excavation.
- Enlarge the excavation in small steps so that the support is not overloaded.
- Install 'final' excavation dimension support (cables) after each excavation enlargement cut.

Figure 39: Preferred sequence for supporting and excavating a wide excavation

LEARNING OUTCOME 3.3.21

CONNECTION 63

Refer to LEARNING OUTCOME 3.2.11 on page 103, LEARNING OUTCOME 3.3.8 on page 118, LEARNING OUTCOME 3.3.9 on page 119 and LEARNING OUTCOME 3.3.16 on page 121

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIMISATION OF SERVICE EXCAVATION SIZE, SHAPE AND ORIENTATION TO SUIT GEOLOGICAL CONSIDERATIONS

LEARNING OUTCOME 3.3.22

CONNECTION 64

Refer to LEARNING OUTCOME 3.2.12 on page 105

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIMISATION OF SERVICE EXCAVATION SIZE, SHAPE AND ORIENTATION TO SUIT FIELD STRESS CONSIDERATIONS

The most important consideration when the selecting dimension, shape and orientation of an excavation is the nature of the rock mass. In strongly bedded sedimentary rocks, where the orientation and shape

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of the excavation will be affected by the rock mass, the long axis of the excavation should preferably be orientated to the strata dip direction or perpendicular to significant jointing.

Complex excavation shapes give rise to high stress concentrations and often destabilising stress fracture orientations and density.
Excavations should also be kept to a minimum dimensions at all times as smaller excavations suggest improved stability, whilst support installation is simple and easy to control.



Computer assisted evaluation of conditions will assist by indicating wedge sizes formed by joints / faults / bedding, while stress concentrations can be avoided by clever use of the stress field and excavation shapes.



- Chapter 9, Budavari S, Rock mechanics in practice, SAIMM, 1983

LEARNING OUTCOME 3.3.23

CONNECTION 65

Refer to LEARNING OUTCOME 3.2.13 on page 106

DESCRIBE, EXPLAIN AND DISCUSS THE BEHAVIOUR OF ROCK SURROUNDING SERVICE EXCAVATIONS AT DEPTH DURING SEISMIC EVENTS

LEARNING OUTCOME 3.3.24

CONNECTION 66

Refer to LEARNING OUTCOME 3.1.17 on page 87, LEARNING OUTCOME 3.2.3 on page 94and LEARNING OUTCOME 3.2.5 on page 96

DESCRIBE, EXPLAIN AND DISCUSS THE SITING OF SERVICE EXCAVATIONS WITH RESPECT TO EXISTING, CURRENT AND FUTURE STOPING OPERATIONS



Service excavations are usually long term excavations and all efoort should be made to prevent placement in high stress abutments. Numerical modelling should be used to avoid such situations for future and current stoping layouts with conjunction to excavation positioning to ensure the lowest possible induced stress levels over the life of the excavation.

LEARNING OUTCOME 3.3.25

CONNECTION 67

Refer to LEARNING OUTCOME 3.3.16 on page 121 ; LEARNING OUTCOME 3.3.17 on page 121 and LEARNING OUTCOME 3.3.6 on page 117

DESCRIBE, EXPLAIN AND DISCUSS THE SITING OF SERVICE EXCAVATIONS WITH RESPECT TO GEOLOGICAL STRATIGRAPHY AND STRUCTURES

LEARNING OUTCOME 3.3.26

CONNECTION 68

Refer to LEARNING OUTCOME 3.1.12 on page 82

DESCRIBE, DISCUSS AND EXPLAIN HOW ROCKMASS CHARACTERISATION MAY BE APPLIED TO PREDICT SERVICE EXCAVATION STABILITY AND SUPPORT REQUIREMENTS

Rockmass characterisation must be carried out to determine the geological and geotechnical conditions that are present. This characterisation could include the use of rock mass quality rating systems as in Outcome 3.1.12, or simply just and description of the rock mass:

- Massive rock mass: Excavation stability is suggested since massive rock types are usually also competent. However, stress fracturing of massive layers can be detrimental to the excavation stability as slabbing and scaling occurs easily and is also high risk to safety due to the large slabs formed;
- Laminated rock mass: Laminated material separate easily across large dimensions such as those in a service excavation and reinforcing of the beams forms by support becomes essential;
- Major structures present: Features such as faults, dyke contacts and major joints provide surfaces on which displacement can occur. At the same time, these structures could also be conduits for water and channels seismic energy, increasing potential for instability and / damage. The presence of tectonic stresses could also negatively affect stress fracturing around the excavation and the closure of walls due to dilation.
- Jointed rock mass: Blocky conditions are usually created, leading to potential wedge failures (of varying dimension) which require dens support patterns.



- TR Stacey Best practice rock engineering handbook for “other” mines SIMRAC Jhb chapter 7

LEARNING OUTCOME 3.3.27

CONNECTION 69

Refer to Paper 1 Outcome 6.2
Refer to Paper 2 Outcome 3 on page 93
Refer to Paper 2 Outcome 5.1.9 on page 331

APPLY ROCKMASS CHARACTERISATION TECHNIQUES TO PREDICT SERVICE EXCAVATION STABILITY AND SUPPORT REQUIREMENTS

LEARNING OUTCOME 3.3.28

CONNECTION 70

Refer to LEARNING OUTCOME 2.3.1.1 on page 50
Refer to all Outcomes in 3.1, 3.2 and 3.3

EVALUATE GIVEN ROCK CONDITIONS, FIELD STRESSES, GEOLOGICAL CONDITIONS, EXCAVATION SIZES, SHAPES AND ORIENTATIONS TO PREDICT EXCAVATION STABILITY

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LEARNING OUTCOME 3.3.29

CONNECTION 71

Refer to LEARNING OUTCOME 2.3.1.1 on page 50
Refer to all Outcomes in 3.1, 3.2 and 3.3

EVALUATE GIVEN ROCK CONDITIONS, FIELD STRESSES, GEOLOGICAL CONDITIONS, EXCAVATION SIZES, SHAPES AND ORIENTATIONS TO RECOMMEND REMEDIAL MEASURES TO IMPROVE EXCAVATION STABILITY

LEARNING OUTCOME 3.3.30

CONNECTION 72

Refer to LEARNING OUTCOME 2.3.1.1 on page 50
Refer to all Outcomes in 3.1, 3.2 and 3.3

EVALUATE GIVEN ROCK CONDITIONS, FIELD STRESSES AND GEOLOGICAL CONDITIONS TO PREDICT SHAFT STABILITY

LEARNING OUTCOME 3.3.31

CONNECTION 73

Refer to LEARNING OUTCOME 2.3.1.1 on page 50
Refer to all Outcomes in 3.1, 3.2 and 3.3

EVALUATE GIVEN ROCK CONDITIONS, FIELD STRESSES AND GEOLOGICAL CONDITIONS TO RECOMMEND REMEDIAL MEASURES TO IMPROVE SHAFT STABILITY.

3.4 MINE SEISMICITY

LEARNING OUTCOME 3.4.1

PRINCIPLES OF SEISMICITY

- Define, describe, explain and discuss the term seismic event
- Describe, explain and discuss the phenomenon of foreshocks and aftershocks
- Describe, explain and discuss the processes that give rise to violent failure and rupture in rock
- Describe, explain and discuss the rupture process in solid rock
- Describe, explain and discuss the rupture process on a pre-existing fault plane
- Differentiate between the terms seismic event and rockburst
- Define, describe, explain and discuss the term rockburst
- Identify, describe, explain and discuss the conditions required for a rockburst to occur
- Describe, explain and discuss the main mechanisms responsible for seismic events in South African mines
- Describe, explain and discuss the main mechanisms responsible for rockbursts in South African mines
- Define, describe and explain the phenomenon of seismic body waves
- Describe, explain and distinguish between different types of seismic body waves
- Define, describe and explain the phenomenon of seismic surface waves
- Describe and explain how seismic surface waves are generated

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- Describe and explain how seismic surface waves are related to seismic body waves
- Describe, explain and discuss the differences between seismic body waves and seismic surface waves
- Describe, explain and discuss the factors that affect the velocity of propagation of seismic waves in rock
- Describe and explain how the rockmass environment affects the velocity of Pwaves and S-waves
- State typical velocities of P-waves and S-waves in hard rock and soft rock environments
- Describe, explain and discuss how this information may be used to locate the source of seismic events
- Describe, explain and discuss how the energy of a seismic body wave is attenuated as it propagates through elastic rock
- Describe, explain and discuss 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, explain and discuss the phenomena of reflection and refraction of seismic body waves
- Describe, explain and discuss how seismic wave reflection and refraction gives rise to and affects rockburst damage in excavations
- Describe, explain and discuss the objectives of seismic monitoring in mines
- Identify, describe, explain and discuss the features of seismic events that are used to locate the source of seismic events
- 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
- Distinguish between seismic source mechanism and seismic damage mechanism
- Define, describe, explain, discuss and contrast the following primary source parameters in terms of their physical meaning and the data required to determine them :
  - Richter magnitude, Local magnitude, Moment magnitude
  - Seismic energy, Seismic moment, Seismic displacement
- Define, describe, explain, discuss and contrast the following secondary source parameters, in terms of their physical meaning and the data required to determine them :
  - Corner frequency, Source radius, Apparent volume,
  - Stress drop, Energy index
- Describe and explain the common methods of seismic data analysis.

Introduction

In a mining environment, instead of tectonic processes driving stress accumulation and strain release, the creation of excavations, their position and their shapes are seen as critical factors with respect to causes of seismic failure. In addition, there is a range of geological conditions known to be associated with seismic events in mines. These conditions occur in different geotechnical areas and under varying stress regimes. It is not surprising therefore that seismic response to mining activity varies greatly and that seismic events bear characteristics of and reflect the conditions under which they occur.

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Tabular mines in hard rock environments such as the Witwatersrand gold mines and the Western Bushveld Platinum mines are marked by mining excavations whose strike and dip dimensions are much larger than their vertical height (stoping height); by medium to ultra-deep mining depth and their corresponding stress levels; and by mostly strong and brittle rock types whose UCS is in the range of 50- 100MPa (a typical tabular mine is shown in Figure 40). In combination, the resultant mining environment is marked by high field stresses and high rock strength, which both contribute to the potential for sudden, violent release of strain energy.

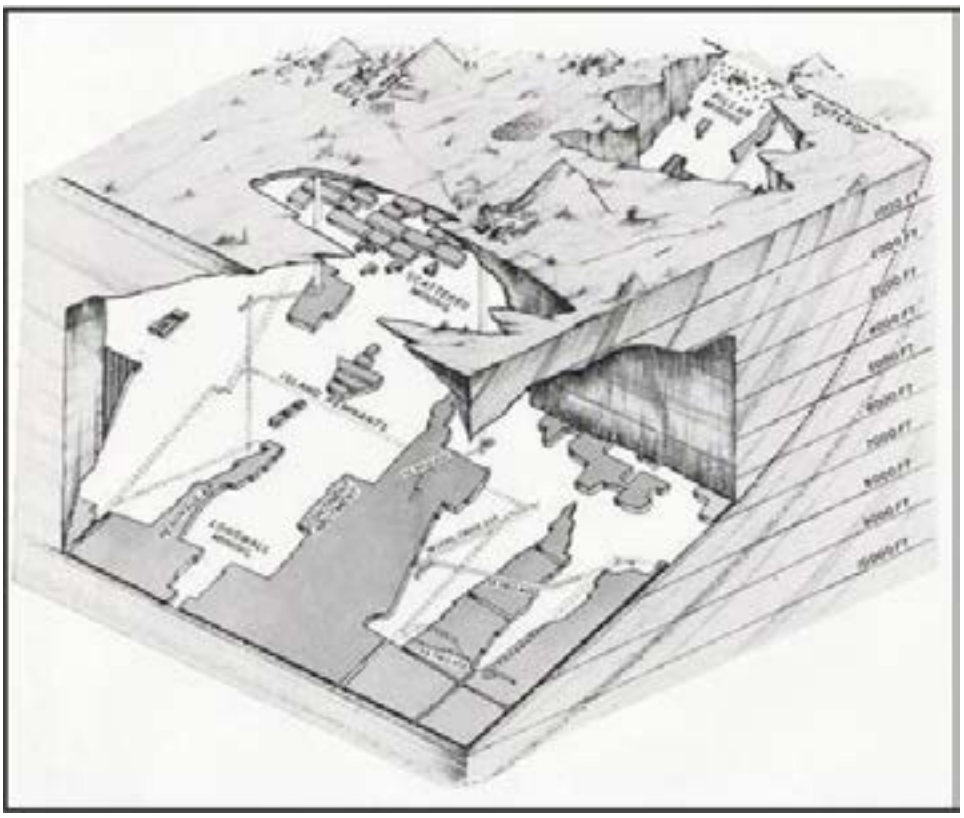


Figure 40: Layout of a tabular mine (Cook et al, 1966, from Durrheim, 2010)

In general, seismicity is induced and triggered by either high stress levels in excess of the rock mass strength, or where high rates of stress change increase the probability of failure (for example directly after blasting). The purpose of this section is to describe in detail the specific circumstances of mining related seismicity in South African tabular hard rock mines in order to derive an understanding of the underlying causes and the contributing factors, all of which are essential to the management of seismic risk. There is a great likelihood that the continued efforts of the mining industry to develop sophisticated seismic monitoring technology, paired with successful research into the quantification and analysis of mining induced seismic events, formed the basis for the reduction in rockburst fatalities achieved over the past decade.

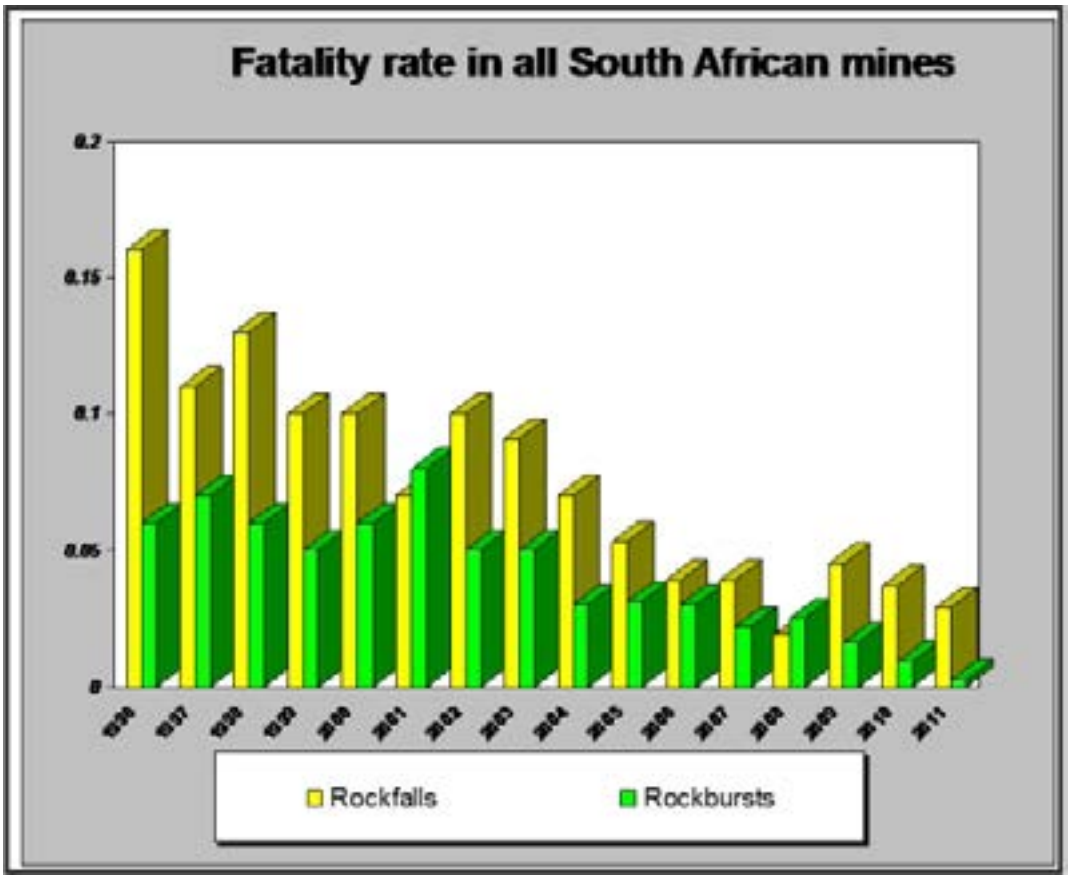


Figure 41: Rockburst fatality rates in SA mines (courtesy: D Adams, DARE)

Seismic events

A seismic event (called earthquake when caused by tectonic forces in the earth's crust) is the result of strain accumulation in rock due to stress. When stresses exceed the rock strength (or the cohesion and friction on a plane of discontinuity), sudden failure occurs accompanied by the release of strain energy. By definition, the energy release has to happen rapidly, in fractions of a second, in order to be called 'seismic'. A rock-mass failure incident which happens slowly or in distinct stages and does not emit seismic waves is by definition not a seismic failure.

Shown below are typical seismic failure mechanisms as they are observed in and around mining excavations:

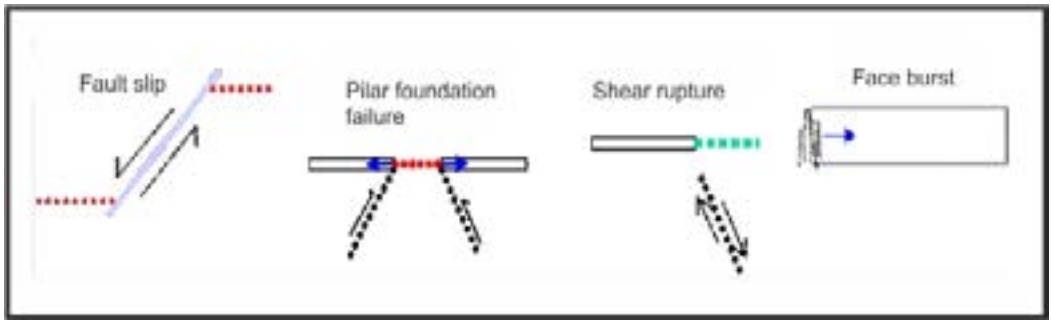


Figure 42: Seismic failure types typical for intermediate to deep tabular mines.



"seismic" is any physical process generating seismic waves

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NOTE

The time a seismic event takes to occur is equal to the size, or physical extent, of the event divided by the "rupture velocity". The rupture velocity is of the order of half the shear-wave velocity.

CONNECTION 74

Refer to Paper 2 Outcome 2.2 on page 35

Face bursts are observed where high face stresses are combined with brittle rock types and high stoping heights. Where the failure occurs close to the skin of a void, it may result in the ejection of face rock into the stope. Where the failure locates deeper in the rock mass it may constitute an on-reef pillar failure where the source of strain energy release is within the pillar volume. In both cases sidewall rock is ejected into the excavation.

DEFINITION

Seismic event:= Sudden release of strain energy due to stresses exceeding the rock mass strength.
Rockburst:= A seismic event which results in damage to excavations

Shake-down, crushing, footwall heave and side wall ejections are examples of damage mechanisms. When investigating rockbursts the distinction between failure and damage mechanism is essential as the same failure mechanism can lead to a range of different damage types. The photos in Figure 43 illustrate two of these damage forms.



Figure 43: Sidewall ejection after pillar failure (left) and complete collapse of a tunnel as a result of seismic energy release.

CONNECTION 75

Refer to Paper 2 Outcome 2.2 on page 35 where primary source parameters were introduced

Seismic events are described and their properties quantified by seismic event or source parameters, an essential set of which was introduced in the materials for Part II 'Rock engineering practice'. Below follows an introduction to secondary source parameters in common use where mine seismic data are analysed and interpreted.

Secondary source parameters

Conceptual approach to SO1-6:
General comments on virgin and field stresses, stress accumulation, strong & brittle rock types, local zones of weakness (refer to issues in SO8-12 below); conditions for RBs to occur; reference to mine design elements; definition and explanation of sec. source parameters.

In order to formulate a basic physical model of seismic sources, Brune (1970) made certain simplifying assumptions about seismic events that would allow a mathematical description of the source. The key assumptions are:

- the medium in which the source is located is elastic and homogenous, which simplifies wave generation and propagation;
- the slip area is circular and the stress drop is constant and instantaneous across the slip area zone and the failure zone itself is 2-dimenaional and rounded;

back

back

- the waves radiated from such simplified source have a simple spectrum consisting of a superposition of basic harmonic vibrations;
- where waves are recorded for the purpose of source quantification, the measurements are taken in the far-field (>3 times source radius); waves far from the source are simpler and easier to describe mathematically than near-field waves.

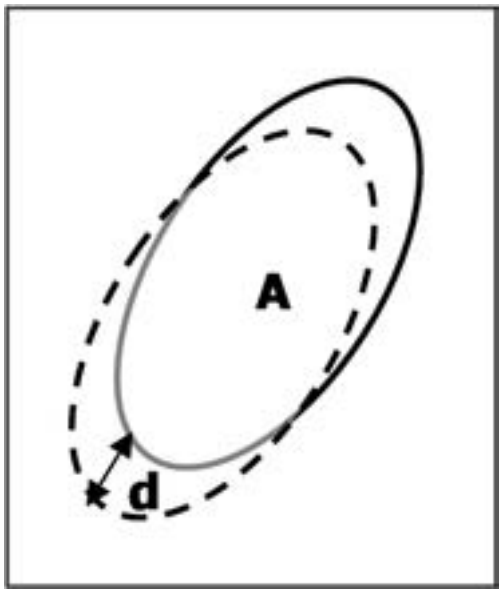


Figure 44: Constant slip across the slip area



The near-field peak particle velocity is proportional to the static stress drop in the same way that the particle velocity of a plane shear wave is related to the shear stress of the wave.

Brune's paper also introduced two concepts that have been widely accepted over the last 40 years and are used routinely in seismic analysis.

- The high-frequency fall-off of the spectrum of ground displacement of the shear wave is proportional to f-2 (the omega squared model.);
- the frequency at which the displacement spectrum of the shear wave changes from being flat (and related to seismic moment) to falling off at f-2 is inversely proportional to the source dimension. This frequency is called the corner frequency.

Using these assumptions, seismologists devised ways to estimate seismic energy and seismic moment, corner frequency and stress drop from seismograms, which are all commonly used seismic parameters.

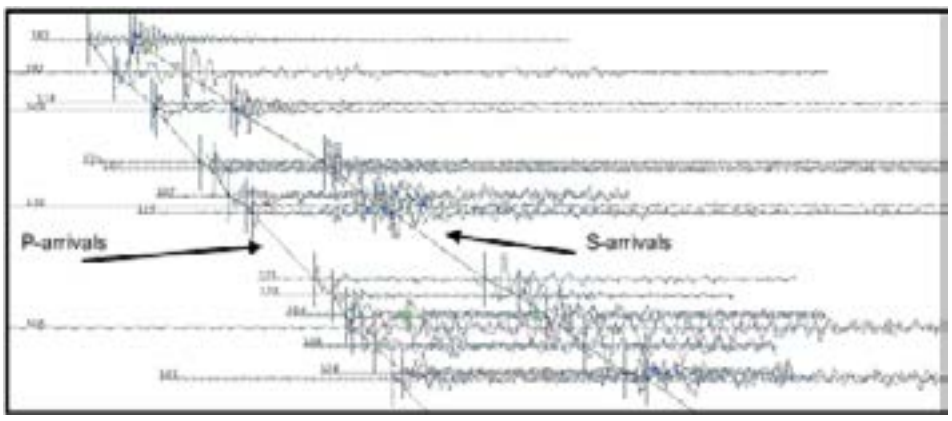
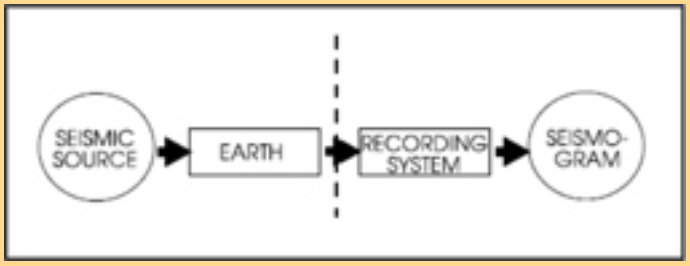


Figure 45: Set of seismograms recorded at increasing distance from the source (top to bottom, note growing distance between P-wave arrivals and S-wave arrivals).

back

INTERESTING INFO

How to connect seismograms back to the seismic source (Lenhardt)



http://www.oge.or.at/Lecture_Introduction.pdf

In order to describe seismic events a set of primary source parameters was introduced. Primary source parameters are those derived directly from the wave forms of recorded ground motion associated with an event. These include: Time and date, seismic energy (a measure of strain energy release in excess of that required to overcome frictional resistance) and seismic moment (a measure of source dimension and slip distance). A slightly expanded definition would also include: Magnitude, source dimension, corner frequency and Energy Index which are directly derived from energy, moment and wave velocity.

Irrespective of the 'primary' or 'secondary' label being applied to a parameter, it is essential to have an understanding of the physical interpretation of each of these variables since they form the basis of the data interpretation and contain important clues with respect to the rockmass properties and behaviour and to the static and dynamic stress conditions where seismicity is observed. The following are important source parameters frequently used in data analysis:

Moment magnitude m: Translates the seismic moment Mo of an event into a magnitude according to m=2/3*log(Mo)–6.1 according to Hanks & Kanamori (1979). Moment magnitude refers to the amount of deformation in the source area and, where the seismic source is accessible to direct measurements, can also be estimated from 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 [Nm]:

CONNECTION 75

GAP612 Relationship between ERR, seismic energy release and stiffness"

$$Mo = G\bar{u}\bar{A}$$

The permanent displacement ū is also known as the seismic displacement.

- *The dominant frequency* fd is the frequency at which a seismic wave carries most of its energy. fd is low for large sources and high for small sources, which confirms our general perception that large objects emit lower pitched sounds than small objects (e.g. a double bass versus a violin).
- *The corner frequency* is an estimate of the signal's dominant frequency, found from the low-frequency end of the displacement spectrum of a recorded wave (see sketch below). fC marks the threshold above which the displacement spectrum begins to decline towards higher frequencies:

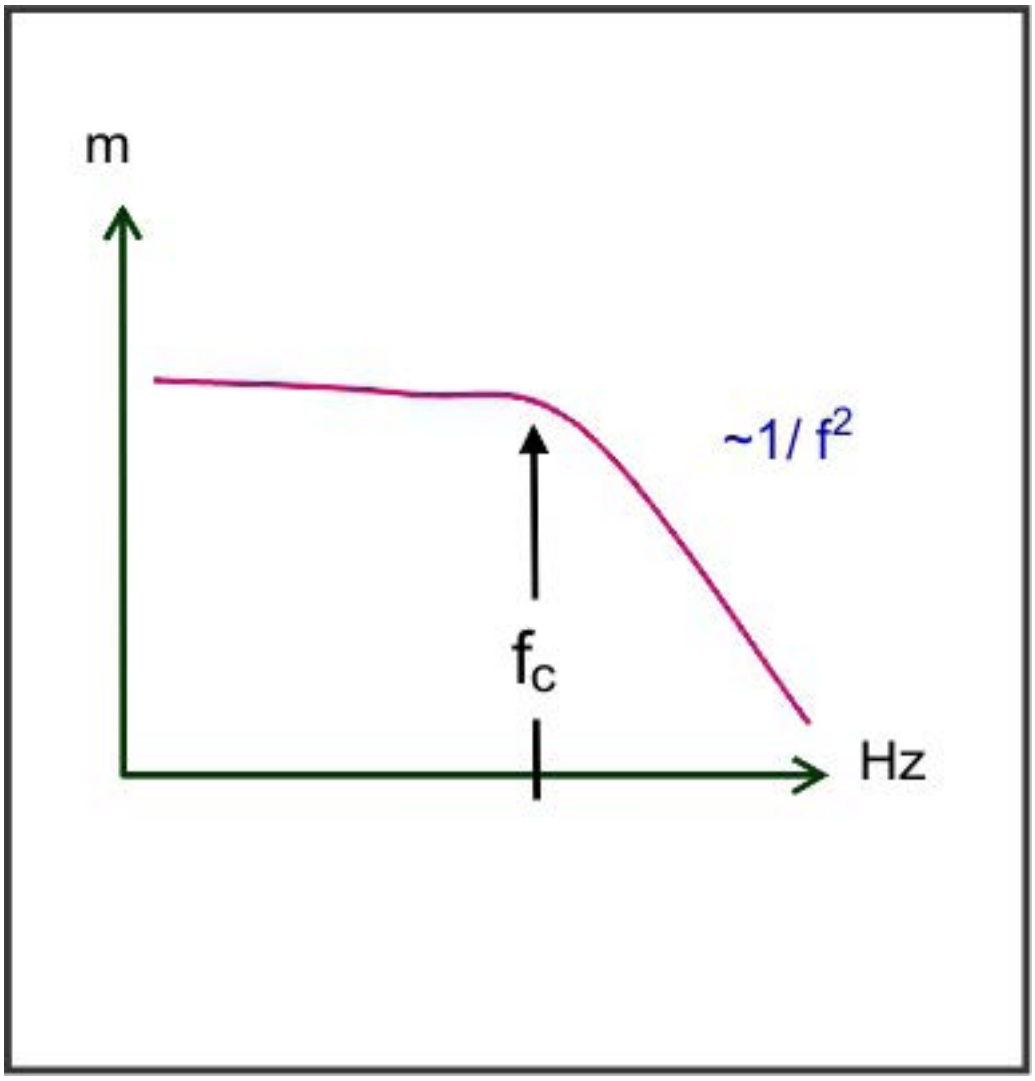


Figure 46: Seismic wave spectrum and corner frequency

- *The source radius* R is equal to half the source diameter of an idealised circular source as described by the Brune model. R is inversely proportional to fC:

$$R = V_s / f_c$$

where Vs is the S-wave velocity. A scatter plot of mine magnitude against source radius (Figure 47) usually displays a positive correlation between the two variables:

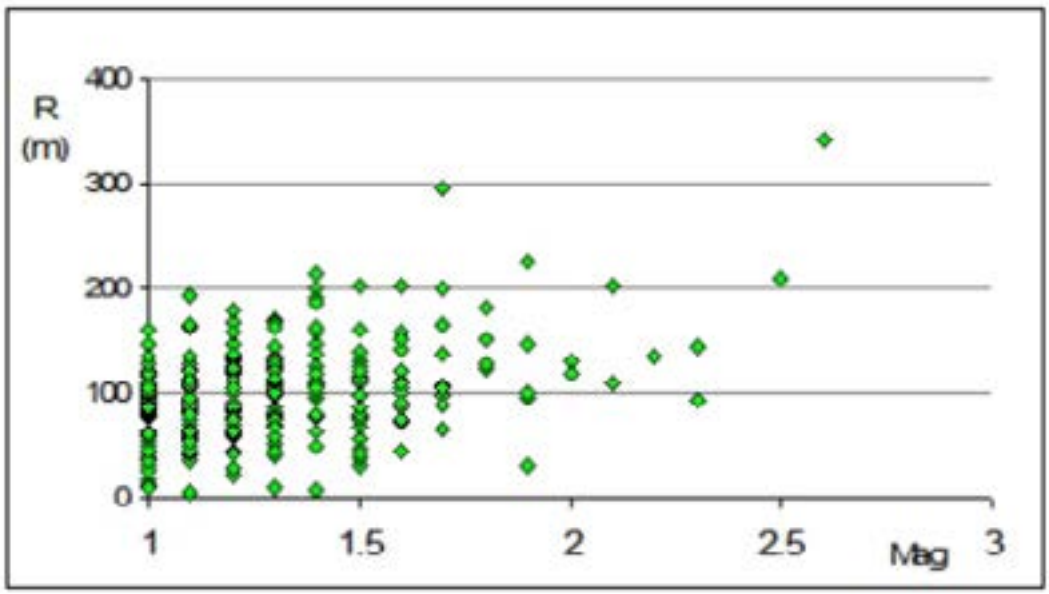


Figure 47: Mine magnitude vs. source radius R in metres

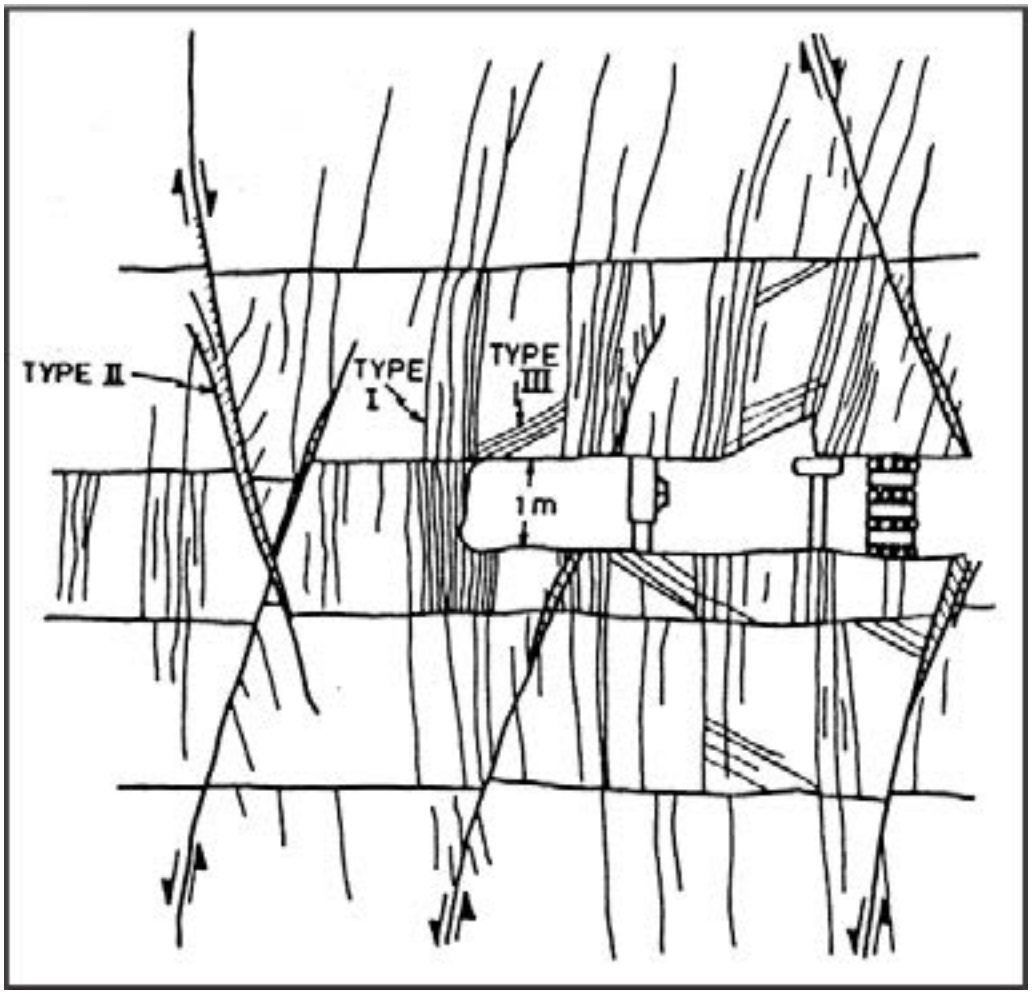


Figure 48: Type I, II and III fractures all constitute potential seismic sources (Malan, 1999)

Remember this:

lower stress drop
↓
larger fault
↓

back

longer source time
↓
smaller corner frequency f_c

Note the range of radii encountered for the same magnitude: Events can be high-stress failures with relatively small sources or low-stress, high-volume failure, both resulting in the same magnitude. Also note outliers with over- or underestimated diameter possibly due to a lack of valid triggers.

In practice, source radius as calculated by many ISSI systems is often overestimated by a factor of 2-3 when compared to the physical source dimensions found in rockburst investigations on deep mines.

Apparent volume V_{app} was introduced by network supplier ISSI and is not generally used in earthquake science. It is proportional to Moment2/ Energy and measures co-seismic, inelastic deformation or "source volume" of a simplified spherical source:

$$V_{app}=Mo/2\sigma_A=Mo^2/c_1GEn$$

with apparent stress σA, rigidity G and scaling factor c1.

V_{app} is a seismic trend analysis parameter as it measures the volume of rock with co-seismic inelastic strain, or resultant permanent rockmass deformation (Mendecki ed, 1997). Seismic hazard is considered high when increases in seismic stress are not accompanied by corresponding increases in seismic deformation.

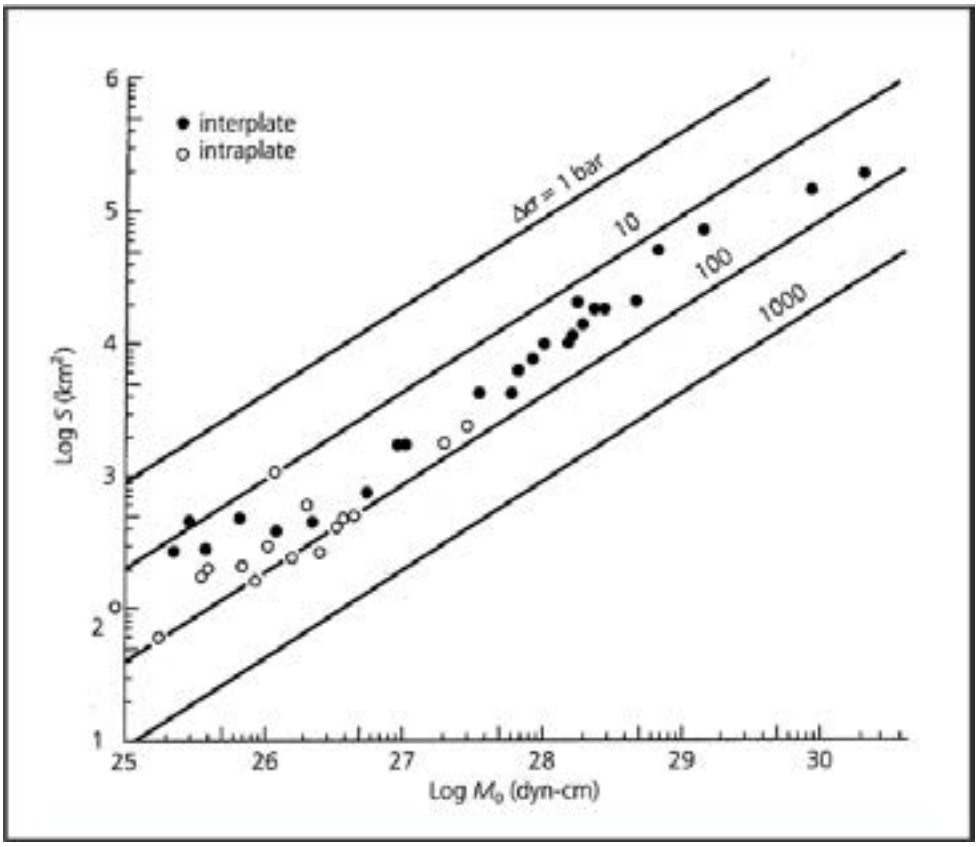


Figure 49: Slip area vs. Moment for range of stress drops associated with different earthquake types (Käser, 2010)

- A quantification of the stress change in the source region before and after the event is provided by stress drop Δσ: proportional to Mo / f_C^3:

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$\Delta\sigma = c_2 Mo / f_c^3$ with factor c2.

Figure 50 illustrates the different types of stress drop commonly used in mine seismology. In combination, they describe the process of stress change during failure.

Static stress drop is the difference Δσ between principle stresses prior to stress build-up and the state after all excess shear stresses have been released and the fault has stabilised (σ1- σ2 in the sketch; after Lenhardt, 2001).

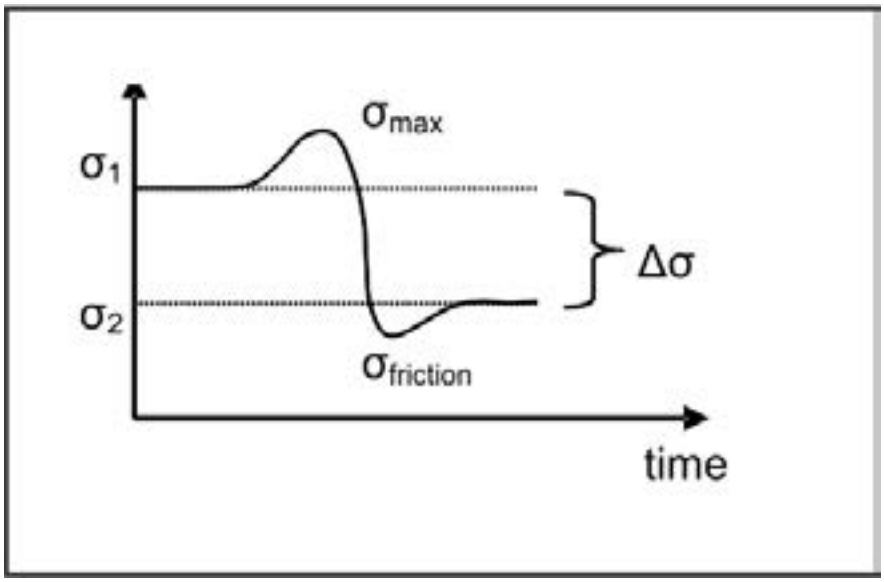


Figure 50: Stress drop

Dynamic seismic stress drop is the difference between the maximum stress level on the locked fault immediately prior to slip and the dynamic friction level of the fault (σ_{max}- $\sigma_{friction}$ in Figure 50, after Lenhardt).

Apparent stress σapp is the average between shear stress before and after the failure occurred; Apparent Stress is proportional to the ratio Energy / Moment of the event and is not related to the Brune model ((σ_1+ σ_2)/2 in the sketch).

BRIDGING GAP 1

Explanation of Tensor on page 17 are provided in the bridging materials

These source parameters or seismic variables should be seen as a toolbox enabling those in charge of seismic data interpretation to make sense of the dynamic failure processes recorded in tabular mines. There are many more variables in use on our mines, foremost a set known as 'seismicity parameters' which describe the behaviour over time and in space of entire event groups rather than single events. These will be dealt with in a later chapter on trend analysis and rock mass instability.

As a final parameter to describe single seismic sources the tensor of moments is introduced: Where a seismic event was recorded by a sufficient number of surrounding stations, an analysis of the P-eave arrival, the initial section of a seismogram, allows a determination of the 3D-strain distribution at the source (Moment Tensor).

INTERESTING INFO

SIMRAC project GAP604 developed a MT inversion toolkit (Andersen & Spottiswoode)

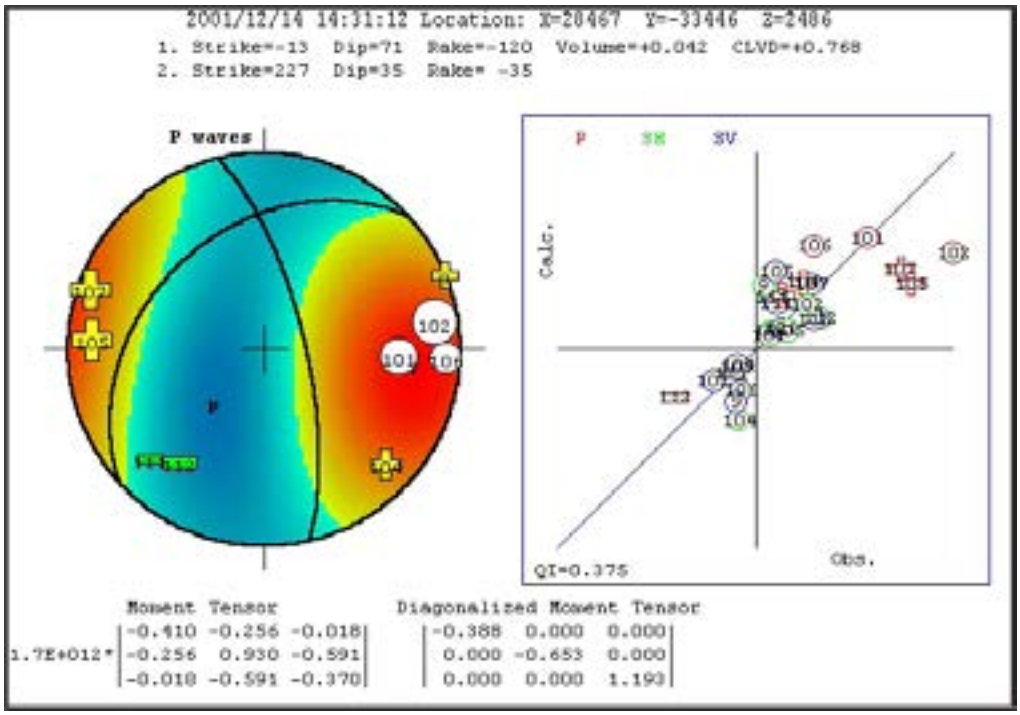


Figure 51: Moment tensor (Courtesy of Hamerkop)

The moment tensor not only provides the orientation of the main failure plane, it also quantifies the proportion of volumetric and shape change in the source volume, important clues as to which object failed in which mode. Moment tensors can have several principal components and the fault plane solution is often graphed as a lower hemispheric projection (Figure 51).

The MT delivers two mutually perpendicular fault planes as possible planes of failure. Where there is other evidence of a geological structure or of abutments or pillars suggesting possible failure a matching fault plane solution can increase the probability to have identified the correct source. This means in practice that the two possible failure planes offer by the MT inversion need to be compared with the mining configuration and geological detail of the failure area. The question to be answered is: Are there any structures or excavation outlines that would match one of the suggested fault planes?

The above example yields the strike, dip and rake of two failure planes and the likely volumetric and double-couple portion: Reverse slip type failure on a plane with strike -13° and dip 71°, or strike 227° and dip 35°. The first arrivals at the seismic stations are indicated by a "+" when away from the source, and "-" when towards the source. Note the lack of seismic stations above and below the source, which reduces the reliability of the result. Successful interpretation of seismograms for fault plane solutions requires good 3D coverage of the source with sensors above and below, and a low noise environment so that P-arrivals are clearly visible and can be evaluated for the direction of first motion.

Characteristics of mine seismicity

The group of primary and secondary source parameters introduced above, together with seismicity parameters such as activity rate and clustering, serve to describe mine seismicity or better: mining induced seismicity. The following aspects of mining induced seismicity are commonly quantified in order to determine trends over time, typical seismic response patterns associated with certain mining strategies, depth ranges and rock types and their recognisable causes. The overall purpose is

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to mitigate seismic hazard through an improved understanding of the underlying principles:

Causes
Causes of mining-related seismicity can be summarised as the interaction of field stresses with the rock mass surrounding mining excavations. Where and when stresses exceed the rock strength failure occurs resulting in the release of strain energy. Where brittle rock types are affected, energy release is sudden and quick resulting in the generation of elastic waves.

Mining-related seismic events can be separated into mining-triggered (slip along geological faults set off by mining) and mining-induced when most of the released energy is accounted for by mining (Durrheim et al.)

Location
Seismicity is encountered where stress maxima exist or where the rate of change of field stresses is high such as after blasting when the local stress field adjusts and new rock volumes are exposed to higher stresses. Alternatively, seismic activity may indicate zones of weakness in the rock mass, such as geological discontinuities.

Incidence
The incidence of seismicity correlates with the mine design elements that attract high stresses, such as pillars and abutments, where stresses exceed the rock mass strength, and with geological features that become destabilised by shear forces (faults) or localised strain energy storage (dykes). For more information on what triggers seismicity refer to the chapter “Drivers of seismicity” below.

Magnitude
Local magnitude is a function of source size and energy release, thus the largest magnitude events occur where objects experiencing failure are of large physical dimensions, for instance regional fault systems, stabilising pillars and remnants.

Time
The time-of-day and day-of-week distribution of seismicity is commonly a direct reflection of the mine’s blasting activity. Generally, smaller events are more closely linked to blasting than large events (see Figure 52 below, a sample histogram of daily activity during weekdays).

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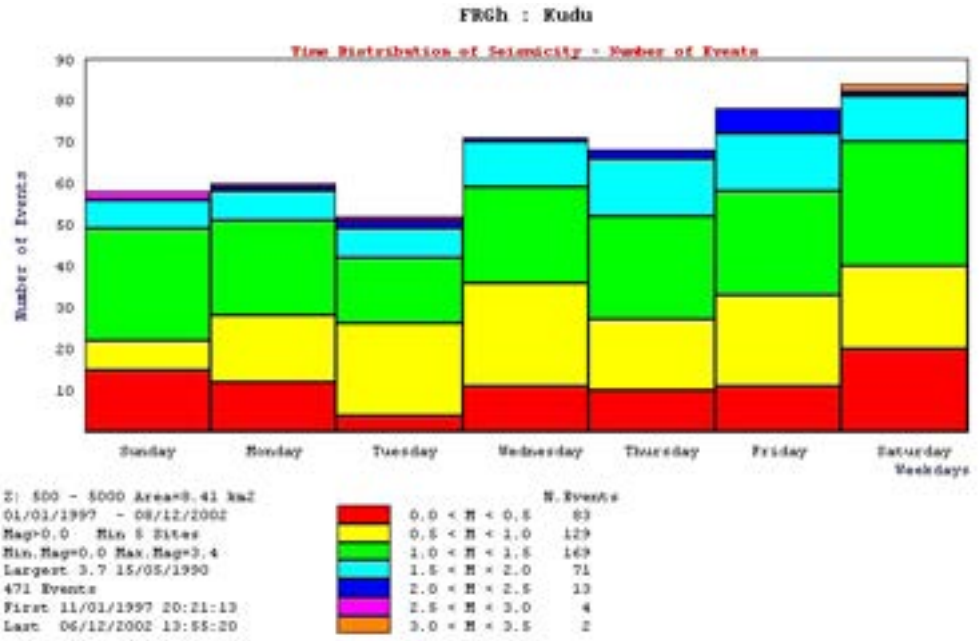


Figure 52: Histogram of activity rate per week day during a 6-months period (colour according to magnitude range)

A time-of-day graph is sometimes called "diurnal distribution", that is: hourly event frequency during a typical 24hrs period

However, distributions can also be influenced by short bursts of activity such as fore shocks, main and aftershock sequences.

Effects

Because seismic event catalogues recorded on mines contain mostly small events below the rockburst threshold, the majority of events have no visible effect on the operation. They are merely the result of small-scale fracturing of the rock mass. Yet, events sufficiently large to result in damage can have severe consequences ranging from injury to workers, damaged equipment and machinery, temporary production loss or even permanent closure of a shaft.

The next chapter identifies drivers of seismicity in tabular mining environments followed by a section on means to control seismic rockmass response in such an environment. The principles of seismic data analysis, common methodologies and outputs are the subject of a specialised chapter further below.

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LEARNING OUTCOME 3.4.2

DRIVERS OF SEISMICITY

Conceptual approach to SO7-14:
Relevance of layout, stress, rock type and geology; hard-rock tab. mine design elements with seismic hazard; measures to control seismicity in hard rock tab. mines: stress and stress change; link to improved mining methods; reduction in severity and frequency of seismic response.

The causes of seismicity in tabular hard-rock mining can be roughly divided into three groups of factors: Depth with its implications for field stress levels; rock mass properties which are generally hard and brittle; and geology, which can range from minor faulting and horizontal intrusions (sills) to major regional fault systems. These three categories of contributing factors determine the environment in which mining takes place, i.e. the forming of excavations for short- or long-term use with the ultimate purpose of extracting portions of the ore body. As will become apparent in this chapter, the detail of the mining layout, excavation design and placement and support strategies determine the rockmass response to mining to a significant degree:

Mining Layout:
High stresses and high rates of stress change over time are widely accepted as determining the seismic rockmass response to deep-level mining activities. Therefore, all design elements attracting high stress levels are prone to seismic failure provided stresses also exceed the in loco rock strength. It is seen as good practice to avoid large leads on panels, mining configurations that create or result in final remnants and pillar designs that do not ensure stability. In a longwall environment, for instance, seismic activity is concentrated along the actively mined panel faces, along strike pillars and near geological features (see seismic deformation contours in Figure 53).

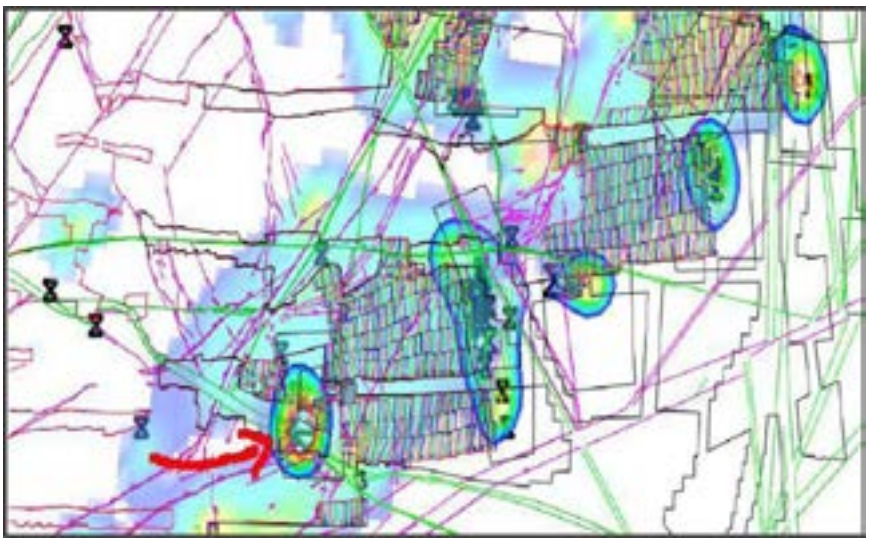


Figure 53: Event locations of large events (X) and contours of co-seismic deformation on a long-wall mine.

back

In scattered mining, holings, remnants and abutments are layout elements most prone to failure. Where a choice exists between over- and underhand configuration, the latter is preferable where the final remnant is down-dip from the active panels and the overall mining directions is towards solid ground, in compliance with general rock engineering rules (Figure 54).

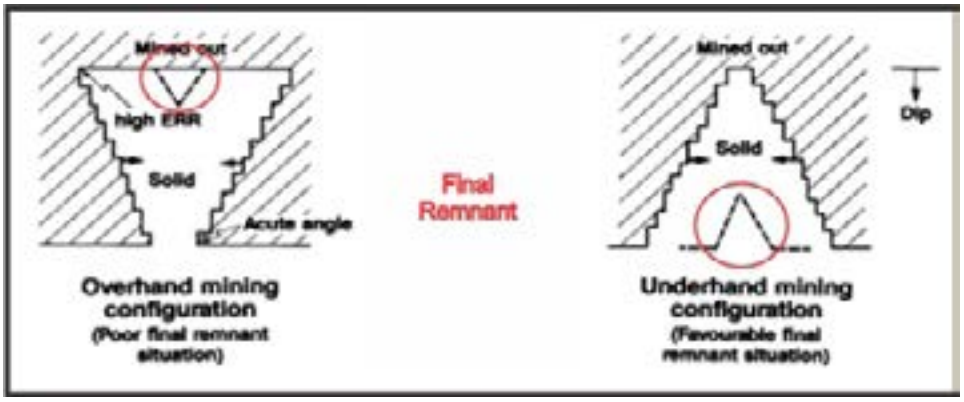


Figure 54: Position of final remnant in over- and underhand configuration (CoMRO, 1988).

In multi-reef situations such as PGM mining whe horizons is important in order to avoid failure of MER pillars when being undermined. This is particularly important where intrusions are involved that may act as local stress concentrators (Figure 55).

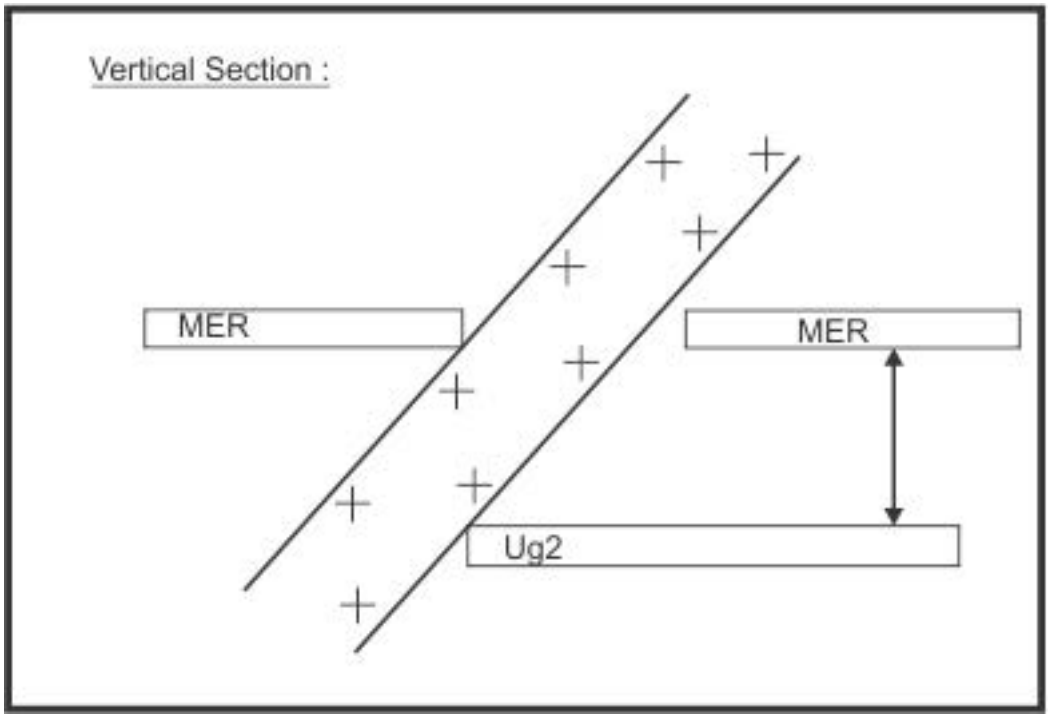


Figure 55: UG2 extraction undermining a MER pillar in the presence of a dyke.

In sequential grid layouts, regional support measures include dip pillars of 20-40m width spaced 140-200m on strike. Dip pillars are less prone to shear failure under vertical stress than strike pillars and have to a large extent replaced strike pillars In ultra-deep mining. Dip pillar layouts are successfully implemented at Harmony, Gold Fields and AngloGold Ashanti mines in the West Rand region. They are likely to be systematically introduced in the platinum mines of the next generation for reef extraction below 1200m.

back

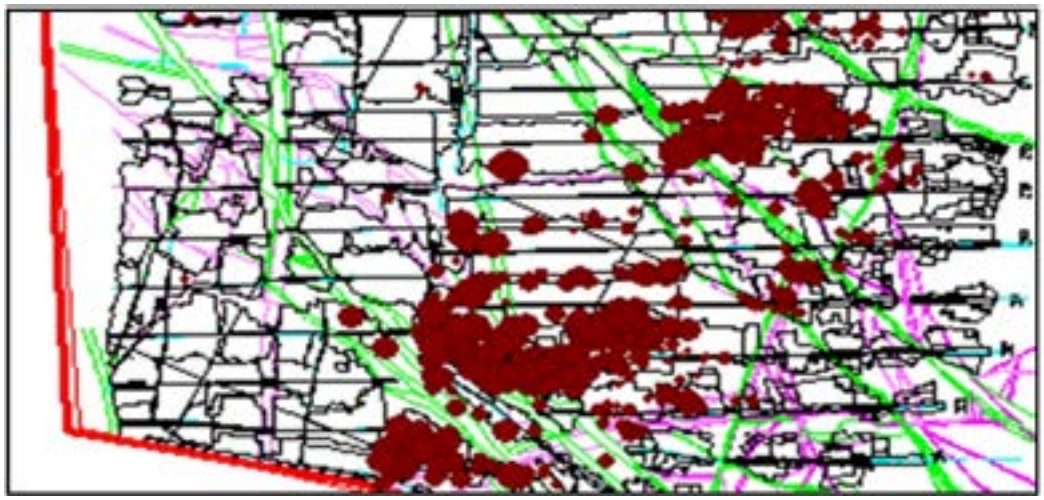


Figure 56: One month of activity in a sequential grid mine (note activity on dip pillars in the back area)

The main layout elements prone to seismic failure in sequential grid mining are mining faces (mostly small events), geological features and their intersections, dip pillars and abutments. Taking all events recorded during one month of operation (side bar), activity is clustered around active mining, whereas single events are found along dip pillars 2-3 raise lines behind mining, and, to a lesser extent, on structures. Measures to reduce seismic hazard in sequential grid mining are: Strict control of optimal sequence, regulated overall panel configuration and panel leads, the 70m rule and one-sided mining. To control geology related failure, bracket pillars are introduced where necessary.

CONNECTION 76

GAP033 Part 1 Develop improved strategies for mining highly stressed areas

GAP033 Part 2 Develop improved strategies for mining highly stressed areas

Depth and rock type:
As mines approach greater depth, they tend to implement different regional support systems, reduce the extraction ratio and increase the size of bracket pillars on structures. These measures effectively counteract increased stress conditions with the result that seismic events are not necessarily more frequent or more severe at greater depth. With depth being the prevailing factor for increases in the virgin major principal stress in the Witwatersrand gold mines, it is also one of the dominant factors with respect to seismicity levels on these mines. All other factors being equal, one would expect an increase in seismicity with growing depth due to higher virgin stresses and their impact on field stresses.

CONNECTION 77

GAP 334 Design guidelines for pillars and inter-pillar HW stability in shallow mines

In practice, a mine's increase in mean rock breaking depth as mining advances to greater depths does not necessarily result in an increase in seismic hazard or even a generally higher level of seismic response to mining. This is often due to alleviating factors coming into play such as improved mine designs, better knowledge of local seismicity and better management of seismic risk associated with geological structures. It may also be the result of changes in rockmass from a brittle to a more ductile rock type, e.g. increasing shale content in quartzites. Brittle rock types store higher amounts of strain energy than ductile rocks, therefore releasing relatively more seismic energy per unit rock volume. Softer rock types on the other hand tend to produce seismic events with larger source volumes and greater amounts of deformation.

Geology:
The presence of faults and dykes with traceable lengths of tens to hundreds of metres is likely to contribute to seismicity in the medium magnitude range, equivalent to source dimensions up to 100-150m. Steeper dipping structures are more likely to slip than flatter dipping ones, where the

major principal stress is vertical. The opposite is true where significant horizontal forces exist such as in the Western Bushveld platinum mines.

Sills and other intrusions are known to attract stresses because of their material strength often being higher than the host rock resulting in seismic failure with significant energy release. But must also be said that among a group of discontinuities with similar dimensions and orientation there may be some that are seismically active and others that are not. This is often recognised as a structure's sensitivity to approaching panels in that they are divisible into those that are not active at all, active when approached and active when negotiated (mined through and extracted).

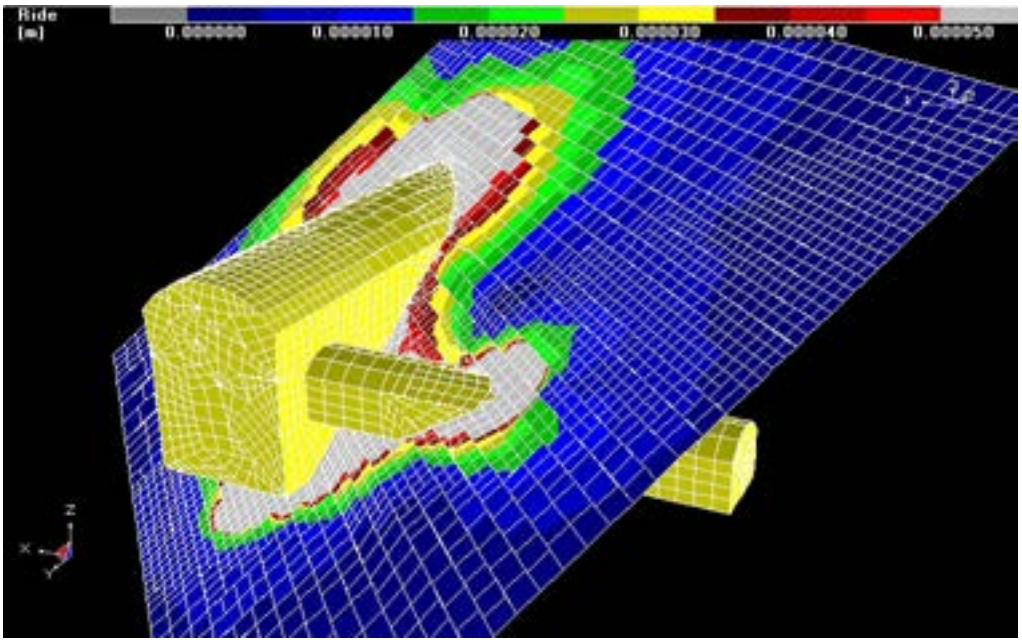


Figure 57: Estimated ride on a fault plane intersected by a tunnel.

Jointing, where it exists with a common orientation over larger areas, can be a determining factor with regard to the predominant direction of the slip of geology-related events, especially where joints are oriented parallel to the face. It is common practice in some of the West Rand gold mines to rather mine West than East from a raise line due to the unfavourable orientation of joint sets in the hangingwall above the VCR.

EXAMPLE

Divide the major geological features on your mine into these categories:
1. Safe (no seismicity, low hazard)
2. Active when mined
3. Active when approached (high hazard)

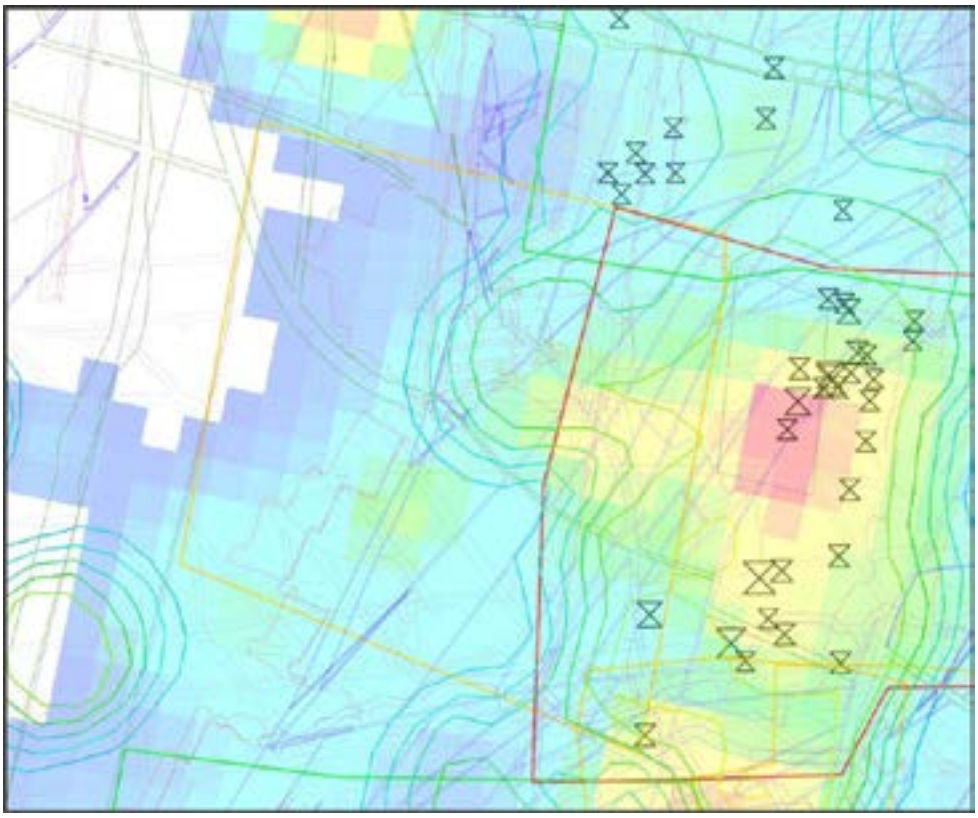


Figure 58: Slip along a fault could be the reason for a local high (red) in a contour plot of seismic displacement

Fault systems and dykes of regional importance are known sources of the very large events that are occasionally recorded in mining districts. These regional failures make up the top end of the seismic data catalogue reaching magnitudes above ML=5. The following examples are quoted from Durrheim et al. (2006):

Date	Magnitude	Region
8-Dec-1976	5.1	Welkom: Dagbreek Fault
28-Oct-1986	4.8	Klerksdorp: No.2 Shaft fault
7-Mar-1992	4.7	Carletonville: Possibly due to large sinkhole
23-Apr-1999	5.1	Welkom-Mathjabeng Mine: Dagbreek Fault
9-Mar-2005	5.3	Klerksdorp – Harties & Stilfontein Town: 5 Shaft Fault
13-Apr-1982	4.6	Welkom: Wesselia/Erfdeel Faults

Table 15: Large magnitude seismic events

The theory of "seismic gaps" attempts to explain the observation that very large regional events in mining districts seem not to occur twice in the same spot, but rather to affect different sections of fault systems. Thus, a seismic gap is a potential site for catastrophic failure in the future, whereas previously failed fault sections are less likely to be hit again.

Rock mass models can, with some certainty, predict the amount of ride expected on a feature where it is exposed to a given stress environment, provided the properties of the feature can be accurately estimated (most importantly friction angle and cohesion, see Figure 57).

Means to control seismicity

A mine experiencing significant seismic hazard levels should aim to reduce both the severity and the frequency of events that have the potential to

result in damage. Each of these two aspects deserves special and separate attention as the methods applied to limit maximum failure size and those to reduce the rate of occurrence rate differ.



INTERESTING INFO

Figure 59: Building of flats collapsed as a result of the 8/12/1976 tremor in Welkom (source: The Star)

Because of its link volume mined, reducing the overall mining rate also reduces the seismicity levels, as does the spreading of active stopes across the mine instead of concentrating mining in one spot. Removing stressed rock from the face by means of blasting initiates a process of rapid stress transfer and slow stress migration that exposes fresh, previously intact rock to high face stresses and leads to fracturing on various scales, from micro-fracturing to shear fracturing extending several metres into the hanging- and footwall around excavations. This is observed as seismic response to blasting and causes well-known daily activity distributions that tend to peak around blasting time.

A disciplined blasting practice – all panels being blasted within a short period, or even centralised blasting in the early afternoon - shortens the response time to stress changes and concentrates seismicity in time. In this way, activity is transferred from night and morning shifts when workers are exposed to the hazard. It is evident from daily distributions recorded under different conditions that seismic activity throughout a typical 24-hrs period can pose varying degrees of seismic risk as seen in Figure 60. Workplace A experiences seismicity early in the morning and late at night, in addition to the main afternoon peak, features not seen in the daily distribution of workplace B. The former clearly possesses a higher hazard level than the latter.

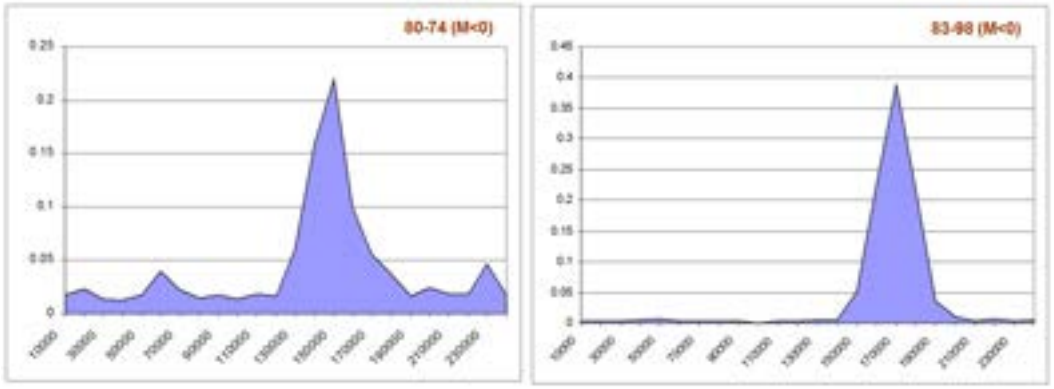


Figure 60: Event frequency during average 24-hrs periods in workplace A and B. A (left) poses the higher risk in terms of workforce exposure to seismic hazard.

But event frequency is only one of two aspects describing seismic hazard, severity being the other aspect. Magnitude is closely related to seismic source size and the elimination of large potential seismic sources is likely to limit maximum magnitude. Among the common interventions to reduce magnitude are:

- bracketing of unstable geological structures;
- reduction of span and limiting the length of abutments;
- limiting total face length mined by sets of conjugate panels;
- ensuring the stability of regional and stabilising pillars;
- optimising sequence and angle of approach when negotiating structures;
- backfilling where it is known to eliminate large seismic events.

There are various documented cases in which a review of regional support strategy resulted in a lower severity of seismic hazard: Raising the percentage of backfilled stopes and reducing the mining spans in longwall mining can lead to a significantly lower ratio cumulative seismic source volume / volume mined (Murphy, 2001). The introduction of 30m strike pillars, a decrease in dip span from 350 to 180m in combination with a lower rate of production resulted in drastically lower rates of events with ML>1.5 (Scheepers, 2004). The most relevant of the large events are those close to mining areas and to access way. The primary focus should therefore be on removing potential sources form these types of excavations by applying the strategies listed above.

The monthly seismic energy release can vary significantly between raise line sections of equal length where they were mined with or without seismicity limiting measures in place (Figure 61). When 32 line was extracted special attention was given to control panel leads, underhand face configuration and limited abutment lengths as these had been previously identified as contributing to seismic energy release.

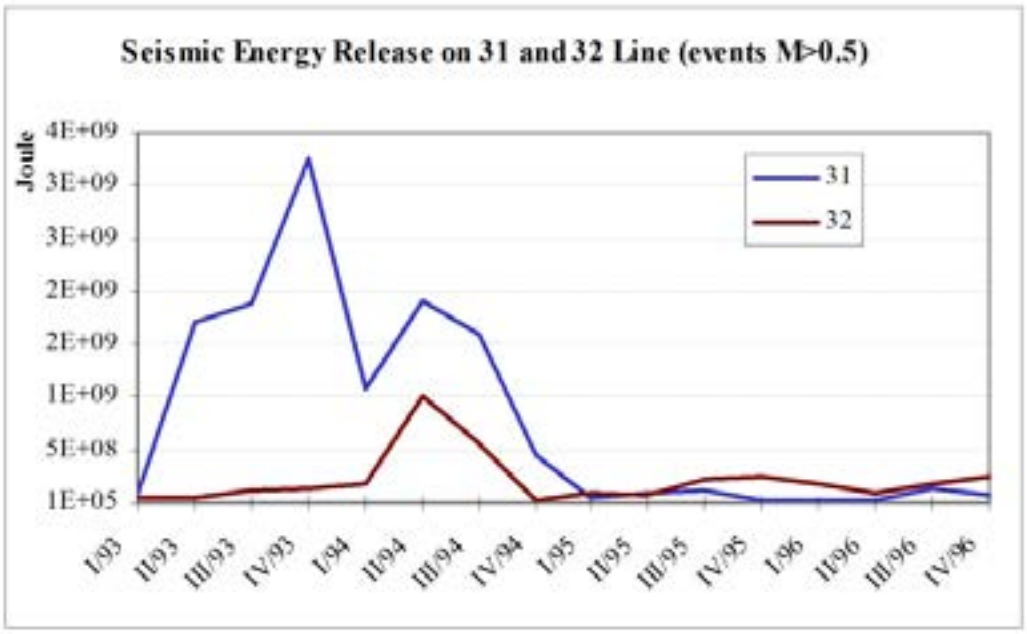


Figure 61:Comparison of 2 raise line sections mined in at equal depth , but with different seismicity levels.



- SK Murphy [2001]: An evaluation of the effect of extensive back-filling on seismicity in longwall mining, RaSiM5, Johannesburg, S. Africa
- LJ Scheepers [2004]: Strategy change in the 31 line west on the VCR horizon at Savuka Mine, 2nd Intern. Seminar on Deep and High Stress Mining
- Durrheim et al (2006) SIM050302 report

LEARNING OUTCOME 3.4.3

DATA ANALYSIS

Conceptual approach to SO16-24:
Definition and explanation of terms; refer back to Part II for magnitudes; Brune's assumptions; basic data analysis methods incl. MT, their outputs and purpose; physical interpretations of magnitude, moment and PPV and their relevance to damage potential (refer to rockburst chapter below).

There is a range of data analysis tools which are based on the common set of seismic parameters describing source location in time and space, aspects of energy and moment release, stress drop and source dimension. These tools have their roots either in global tectonic seismology or were developed specifically for mining applications in South Africa.

Gutenberg-Richter analysis
The Gutenberg-Richter analysis belongs to the first group. It describes the cumulative frequency–magnitude distribution of a seismic data catalogue. It shows, in the form of a histogram, the number of events having occurred with magnitude equal to or larger than a given magnitude threshold (blue bars in Figure 62).

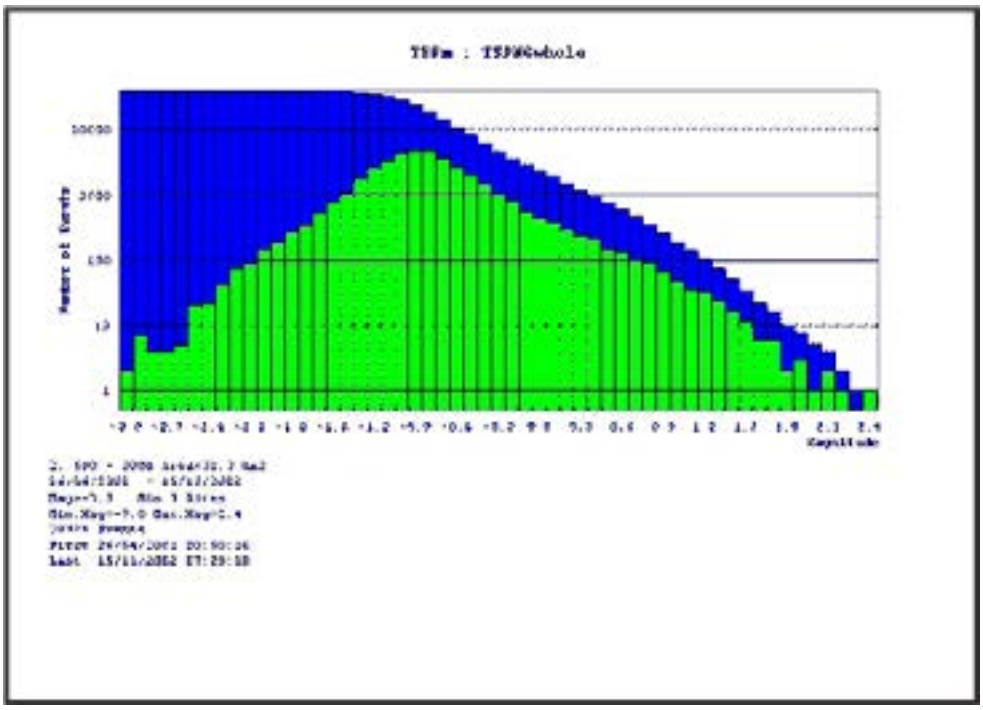


Figure 62: Gutenberg-Richter graph characterising properties of a seismic event group and the network that recorded the events.

BRIDGING GAP 2

A Gutenberg-Richter graph is a cumulative histogram of event magnitude in which the event numbers per bin are displayed on a log scale. It is important to beware of the difference between a normal histogram and a cumulative histogram.

In Figure 62, green bars reflect the frequency of events in each magnitude bin, i.e. a standard histogram of magnitude, while blue bars provide cumulative values. Axes are generally in a logarithmic scale to accommodate the large number of events in most data sets (the data set in the illustration below contains approximately 39 000 events of which roughly 2 000 are ML≥0.0 and approximately 200 events ML≥1.0.

The Gutenberg-Richter analysis yields a measure of the ratio of small to large events, which is known as the "b-slope". The effect of different b values on the distribution is such that for an increase in magnitude by 1, the number of events decreases by the following factor:

for b = 1 10 times
for b = 1.5 30 times
for b = 0.5 3 times.

The Gutenberg-Richter graph also allows an estimate of the largest potential magnitude Mmax and of the sensitivity of the network, i.e. the magnitude above which all events are reliably recorded. This threshold is called the minimum magnitude or Mmin.

The steeper the slope of the graph, the fewer large events occur for a given number of small events which equates lower hazard. The shape of the graph is also of significance as it may indicate either a single or a range of different failure processes. In mine seismic data, GR graphs often contain more than one event population which results in two or more different b-slopes visible in different parts of the graphs. The example in Figure 63 exhibits a bi-modal shape: A shallow slope in the lower magnitude range approximately up to M=-1.1 and a steeper slope above this magnitude.

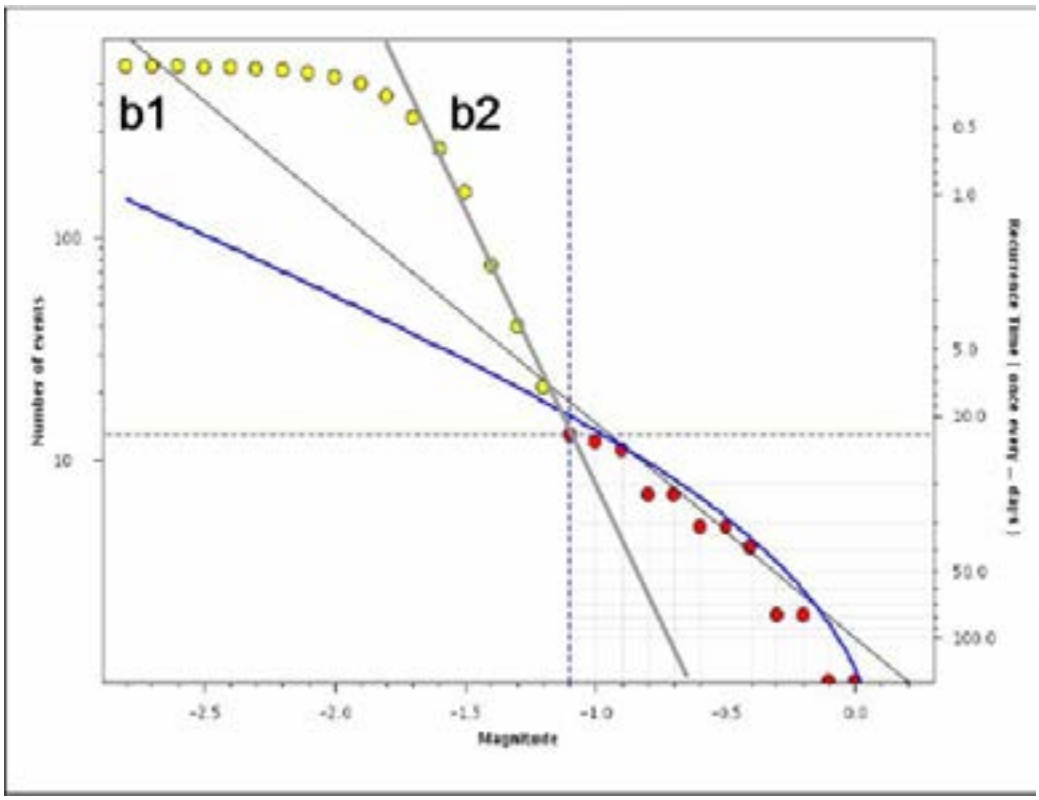


Figure 63: GR graph with two distinct event populations.

Magnitude-frequency relationship
The Gutenberg-Richter relation mathematically describes the distribution shown graphically in the example above. Theoretical considerations suggest that for each increase in magnitude there should be a decrease in event numbers by approximately 90%: For each N events between M_L=1 and M_L=2, there should be roughly 10N events between M_L=0 and M_L=1. This is expressed by the Gutenberg-Richter relation:

Error! Objects cannot be created from editing field codes with magnitude M_L, a and b constants

N is the number of events with size equal to or larger than M_L, where M_L could also be log of moment, energy, or a magnitude based on these two parameters. Note that 'a' merely expresses the overall level of activity, while 'b' refers to the steepness of the graph, which is a measure of the ratio of small to large events. By convention, the value of b is always taken as a positive number even though the slope of the linear fit is negative (e.g. where the slope is -1.3 for example, the relevant b value to be inserted into above equation would be 1.3).



Gutenberg-Richter graphs can take on a variety of different shapes. What may be the reasons?

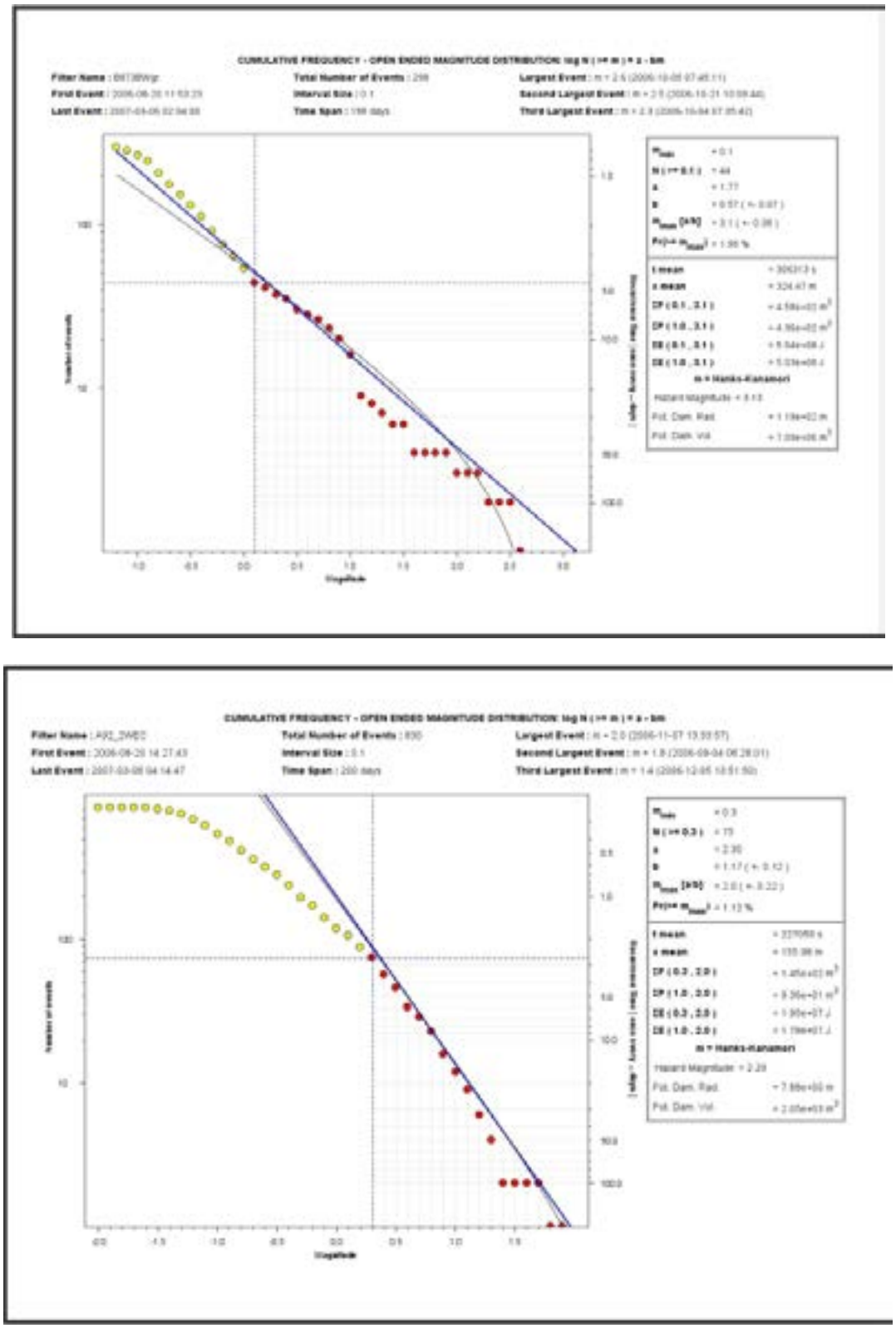


Figure 64: Gutenberg-Richter graphs

1. Trends in b-value over time

Where the b-value is determined periodically, for instance for monthly reporting, there may be benefit in tracing b-value over time. In this way, changes in a certain aspect of seismic hazard – the ratio of large to small events – will become apparent and unfavourable trends can be detected (Figure 65).

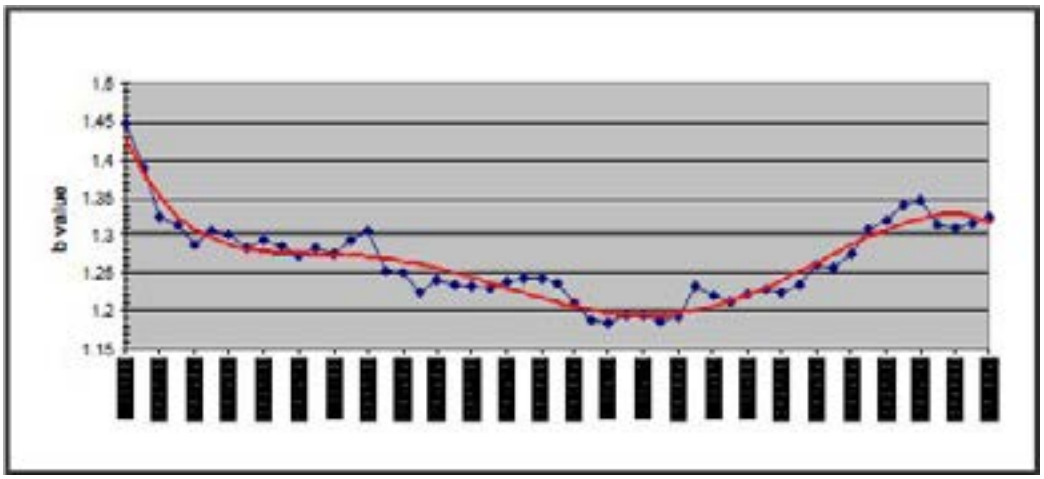


Figure 65: b-value of a platinum shaft over a 4-year period.

Favourable in terms of hazard levels are periods of higher b parameters as at the beginning and towards the end of the observation period.

2. Energy-moment relationship

The energy-moment graph and its associated EM-relation can reveal unique features and qualities of a cluster of events. It is based on a scatter plot of $\log_{10}$ (seismic moment) versus $\log_{10}$ (seismic energy) with energy being the dependant variable (Figure 66, courtesy HAMERKOP Scientific Services).

In a routine analysis, the data is represented by a linear fit which describes the positive correlation between the two parameters. In this case, the linear regression suggests a best fit with slope 1.146 and offset -7.073. The corresponding EM-relation is log(En)=1.146+log(Mo)-7.073.

Obviously, the steeper the slope of the line the higher is the energy released by large events for an event of the same moment. It may be surprising that the slope not only differs between mining areas but varies over time when analysing data from the same area. In this context it should be noted that apparent stress does NOT vary with magnitude over larger ranges of magnitudes.

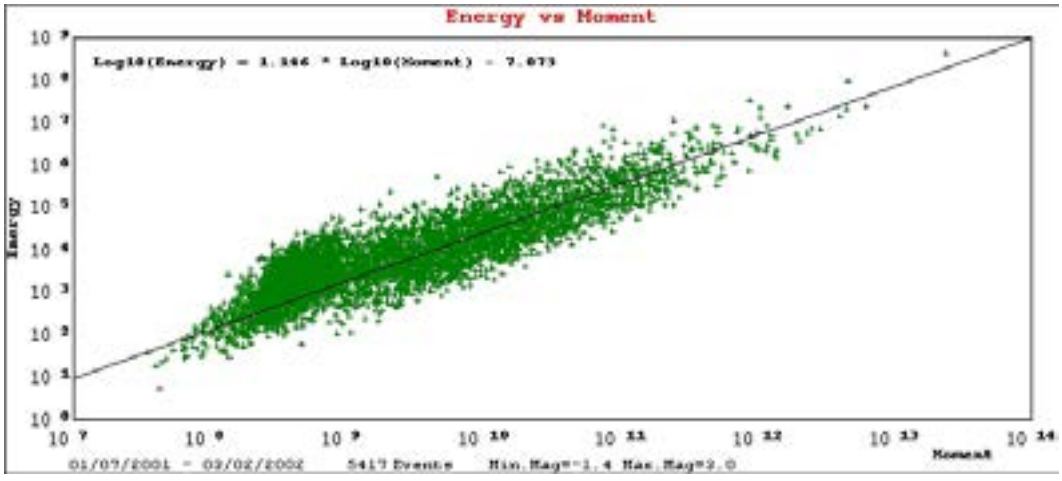


Figure 66: Moment-Energy relationship (Courtesy Hamerkop)

The most pertinent use of the EM-relation is for the calculation of Energy Index (EI), which was explained in detail in an earlier chapter. The slope of the EM-relation has been shown to correlate with loading system stiffness, which relates to mining layout and rock mass properties. Large

INTERESTING INFO

In some incidents of earthquake sequences, the b-value decreased prior to the main shock, which means that temporarily more medium size events occurred (Mendecki, 1997)

Read GAP517 on the quantification of seismic hazard from seismic events in mines

span excavations in a soft, ductile rock mass make for a soft loading system, while the opposite is true for a mining layout incorporating limited spans in a hard-rock environment.

3. Trend analysis

Trend analysis is a widely-used tool in qualitative seismic hazard analysis, both in short- and medium-term intervals. The underlying idea is that seismicity parameters, such as activity rate and cumulative seismic deformation correlate with seismic hazard in the way that upward trends in recorded seismicity indicate a higher likelihood of seismic damage in future.

This may be useful where the inputs into the 'seismic response equation' remain constant, for instance when production rate and mining configuration do not change significantly. Then, a steepening of the slope would be an expression of a more intense and potentially more hazardous rock mass response to the mining process.

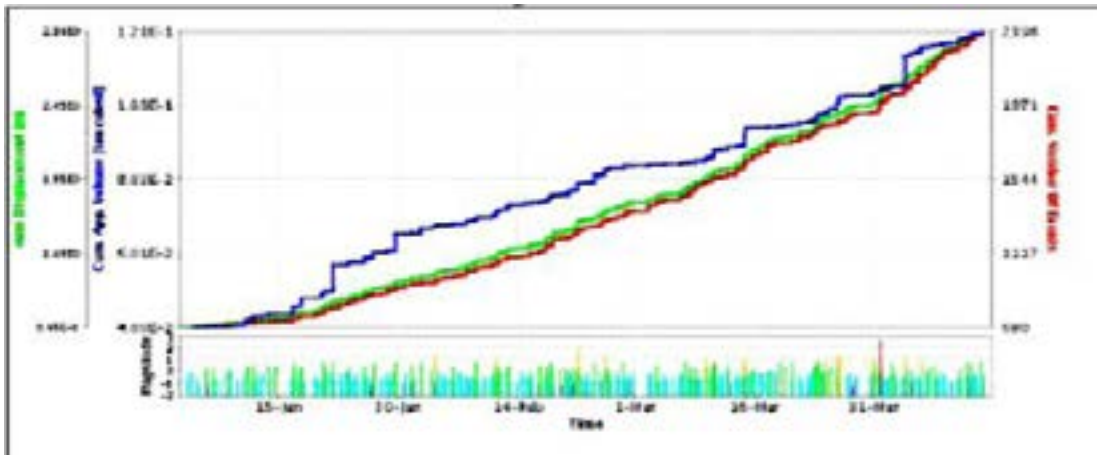


Figure 67: Seismic history plot of cumulative Vapp (blue), co-seismic displacement (green) and activity rate (red)

DEFINITION

Energy Index – Refer to Paper 2, Outcome 2.2 on page 35

BRIDGING GAP 3

For an explanation of linear regression see Linear Functions on page 11 of the bridging material

Graphs of cumulative parameters are interpreted by evaluating the slope of the graph rather than the actual parameter values. Periods of steepening in cumulative Vapp for instance indicate faster deformation compared to flatter periods. This could be caused by an average increase in source size or more frequent large events. Unusually large events are recognisable by the steps they cause in the cumulative deformation graph, as seen during mid January in Figure 67.

4. Instability analysis

Instability analysis is a type of short-term seismic hazard assessment that takes into account changes in seismicity trends over the period of hours or days. A number of seismic and non-seismic parameters are evaluated to determine a score for a working place or set of working places covered by an event selection polygon.

Depending on the scale applied the score is translated into a colour code representing low, medium or high probability of imminent seismic instability in the near future, hence the name "robot system" used on many gold and platinum mines. In its original form, the highest rated event groups received a red colour code indicating imminent danger and suggesting an urgent need for intervention. In the mid 2000s the colour coding was dropped and replaced with an integer number, in part because there was insufficient evidence that red could reliably be interpreted as being a precursor of a large event.

back

BRIDGING GAP 4

Note the difference between time series parameters being displayed as actual values or as cumulative values

The current form of periodic trend analysis is carried out each day for hundreds of event polygons covering an even greater number of working areas in the gold and platinum mines. They take the general form shown in Figure 68, consisting of four panels each evaluating a different aspect of recent seismicity.

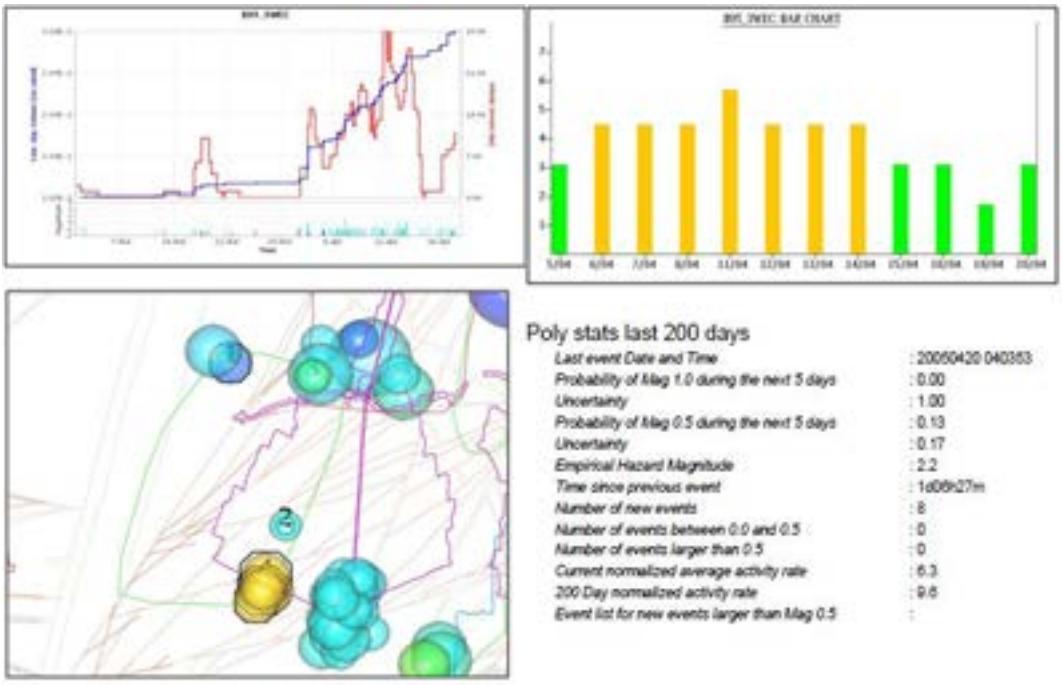


Figure 68: Instability analysis tools for daily hazard ratings in event polygons: recent trends in seismic parameters (top L), past 12 days' ratings (top R), activity since the last report (bottom L) and statistical parameters.

INTERESTING INFO

The use of averaging time windows is common in trend analysis.
'Smoothing' of time series can significantly change the shape of a graph!

5. Time-of-day (diurnal) distribution

Hourly activity rate during the course of an average working day can be expected to peak just after blasting time, especially in terms of small events which originate close to the face. Larger events are known to occur more randomly distributed over time. Time-of-day distributions are studied to judge the effect that blasting practice has on seismic response and to quantify the exposure of employees to seismicity levels. Certain working places have been shown to experience higher percentages of large events during the night shift, possibly due to the delayed effect of the afternoon blast on the triggering of large events.

back

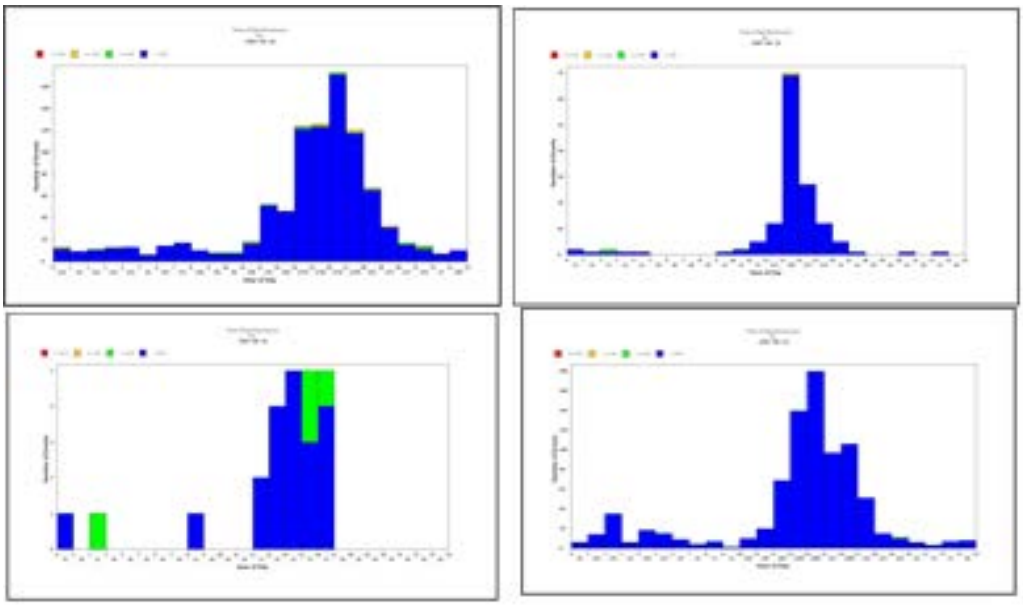


Figure 69: Diurnal distributions of four event polygons from the same gold mine (colours for magnitude).

The daily activity profile is also shaped by manual blasting at the end of the morning shift and the beginning of night shift, and by development blasting, which can occur at any time of the day. Some mines evaluate daily seismicity distributions in order to track compliance with the recommended blasting practice.

6. Day-of-week distribution

The typical day-of-week activity distribution would show a low on a Sundays and an activity peak at some other day during the week. The distribution shape is mainly influenced by blasting and by occasional event swarms that influence the activity rate. In the example in Figure 70, Wednesday was unusually quiet which is inexplicable without knowledge of the local conditions of the mine and the specific time window used for event selection.

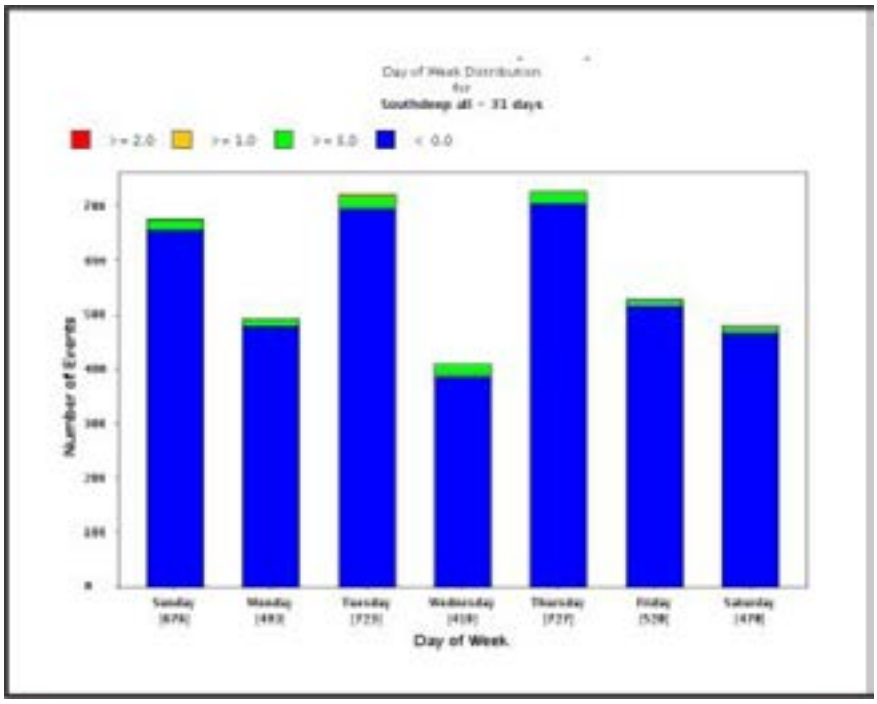


Figure 70: Event frequency in magnitude ranges for the day of the week (31-day time window).

7. Mean return period

Also based on event frequency is the probability of large events to occur within a given time period, for instance the probability P(M) of events M>1.5 within the subsequent 30 days. An increase in P obviously means higher likelihood of occurrence and therefore a higher hazard level.

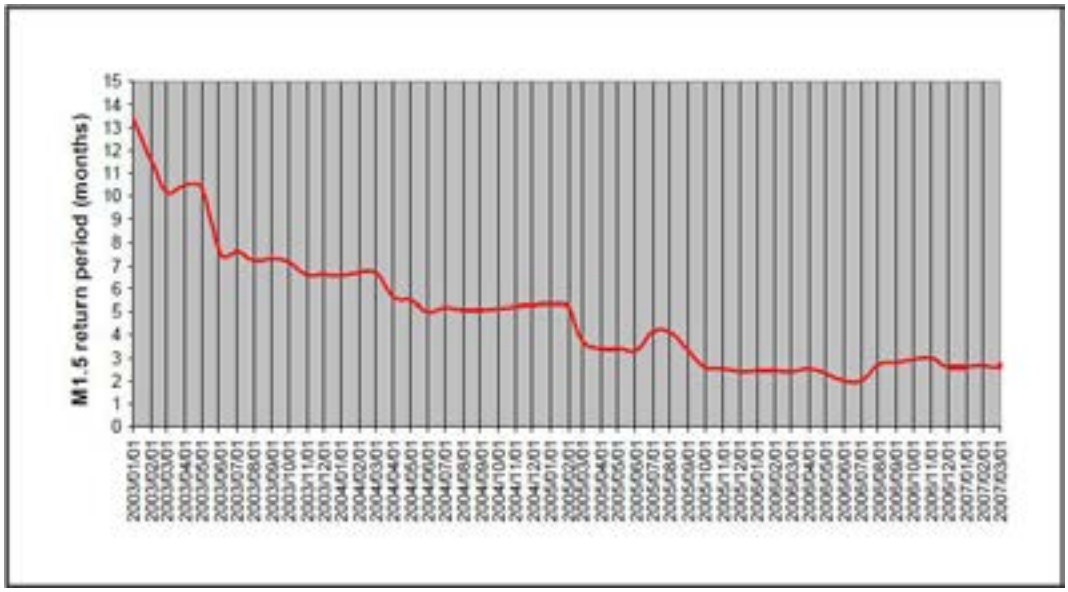


Figure 71: Trend in mean return period of M>1.5 events during 4 years of monitoring.

The inverse of probability is recurrence time or return period, i.e. the average time ΔT_a between events. A rise in P equates a drop in ΔT_a and vice versa. A seismically active gold mine may record 5 events M>1.5 every month on average, in which case P(M>1.5)=1 and the mean return period ΔT_a=6 days. Figure 71 shows an example of a platinum mine whose return period decreased significantly over a period of four years of observation.

The following chapter investigates ground motion as a result of seismic energy release and its potential to cause damage. It leads onto a chapter advising on common practice when investigating rockburst incidents, how to maintain a systematic approach and what to look out for.

ROCKBURSTS

Damage caused by seismic waves interacting with excavations is primarily the result of two mechanisms: Forced oscillation cause 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. In 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.

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CONNECTION 78

For further detail on waves interacting with excavations see Paper 2 Outcome 2.2 on page 35

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). An alternative term also in use is PGV for Peak Ground Velocity.

The following mechanisms are recognised as the main mechanisms underlying rockbursts in local mines (Jager & Ryder, 1999): With increasing intensity and severity of damage, the shakedown burst (exposure to transient seismic waves), the violent shakedown burst (severe and extensive shaking), and the near-field rockburst (exposure to dynamic deformation, part of the seismic source volume).

A shakedown burst is the result of wave interaction with the skin of excavations, where loose blocks and slabs of rock are destabilised, especially in poorly supported and insufficiently barred areas. The effects and symptoms are similar to gravity-driven collapses. Where peak particle velocities and accelerations are very high, entire sections of tunnels and stopes can collapse (violent shakedown).

Fundamentally different from these are near-field rockbursts, which include a component of direct, near-field seismic energy release where the rockburst site is part of the seismic source volume. Footwall heave, crushing, bulking and dynamic closure are commonly observed under these conditions.

The probability of experiencing serious damage generally increases with event magnitude and decreases with hypocentral distance. Small magnitude events, although frequent, hardly result in significant damage. Large magnitude events are infrequent, but can occasionally cause significant damage. Zones of low rock mass quality reduce the likelihood of seismic failure, but increase the likelihood of damage.



For guidelines on how to conduct a rockburst investigation see sections below.

Damage is near certain above 0.7m/s, which can be expected at distances of less than approximately 15m from a M=2 event and less than 40m from a M=3 event. There is a line of thought which argues that if peak particle velocities and peak particle accelerations are very low (say less than 1 mm/s and less than 0.1 g), then any Fall of Ground (FOG) should be attributed to poor quality of support and/or poor barring and not to the seismic event.

According to Butler and van Aswegen (1993), ground velocities can vary significantly between mining regions such as the Welkom and the Carletonville area, and the PPV values estimated by ground motion relations in the far field grossly overestimate the actual ground motion recorded in the near field. Milev and Spottiswoode (2005) documented damaging ground motion levels in an underground experiment at a Vaal River gold mine and produced Figure 72 : 800mm/s appears to be the threshold below which damage was unlikely.

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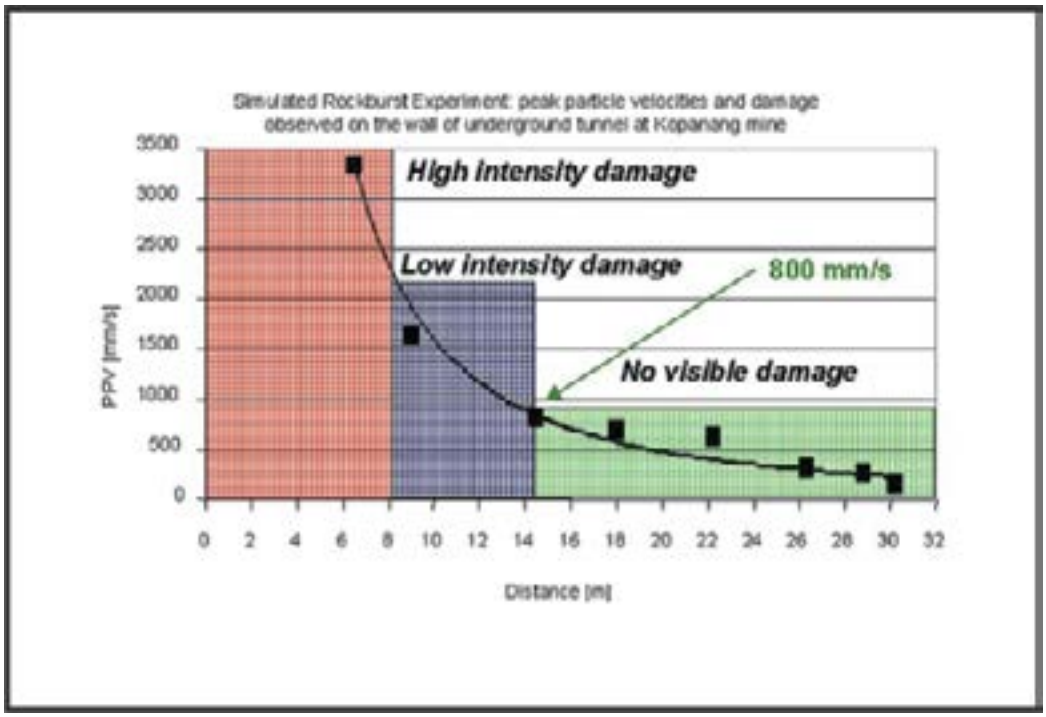


Figure 72: Damage intensity as a function PPV (Milev & Spottiswoode, 2005)



Figure 73: Failed crush pillar and support in a platinum mine

INTERESTING INFO

The Carbon Leader Reef is embedded in quartzites and considered part of the surrounding rock mass, i.e. a relatively soft rock environment. The Ventersdorp Contact Reef is overlain by strong & brittle lavas, i.e. a hard rock environment.

Rockburst investigation

Rockburst investigations are an essential part of seismic risk management. Only after the hazard and its potential consequences are established and well understood can successful mitigating measures be devised.

Investigations should be carried out by competent personnel wherever safe access exists to a rockburst site. The results of the observations, their analysis and interpretation should be documented in a detailed report in which a range of aspects are contained. These relevant aspects are motivated and described in detail in this chapter.

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CONNECTION 79

Gap 201 Investigation of the site response to rockbursts

Amongst the factors central to a rockburst incident are those contributing to the underlying seismic event, and those contributing to the resulting damage. Both need to be covered adequately in the investigation. Rockbursts can, to some extent be analysed from surface based on seismic data and descriptions of the damage by third parties. But the preferred approach should be for a seismologist and a rock engineer together to visit the affected area and to combine the understanding of both technical experts in order to derive an integrated understanding of the process.

The purpose of the rockburst investigation is to gather all evidence that could be relevant in the process of determining the source mechanism and in understanding the damage mechanism with the ultimate goal to prevent a recurrence. A rockburst investigation should consists at least of the following:

- A site visit to the affected area to record the extent, position and intensity of damage;
- Geological detail that may explain the seismic source and its effects;
- Support details (design, installation and performance);
- Evidence of high stress (fracturing) and high strain, such as dynamic closure;
- Recovery of rock-samples and support elements from the site, where appropriate;
- Seismic event details and source parameters of the event considered responsible for the damage;
- Documentation of the seismic history of the area, including previous large events, rockbursts and short-term seismic trend analysis;
- Analysis of aftershock distributions;
- Review of relevant rock engineering reports and the mining strategy.

All of the above should be analysed to derive a coherent and integrated understanding of the origin of seismic energy release and the effect it had on the damaged excavations. Seismic event detail obtained from the seismic system such as location, failure type (shear vs. burst), source type, source dimensions and magnitude should be in agreement with underground observations. Source dimension in metres and magnitude may confirm the potential emission source. Apparent stress and Energy Index could assist in estimating the state of stress at the source. The ratio ES/EP, or where available the fault plane solution from Moment Tensor inversion, could enhance the understanding of the failure mechanism.

One important aspect is to establish whether the seismic energy release was close by or far away, i.e. can the location provided by the seismic system be confirmed? Simple shakedown would indicate a distant source, while crushing, bulging, rock powder and rapid closure would indicate a near source.

The purpose of analysing the seismic history of the affected area is to determine whether the underlying event could have been foreseen in any way, i.e. whether the historic seismic responses reported from the area and recent trends should have raised attention and triggered precautions. The history also describes the way in which the rock mass has previously reacted to mining and allows the rock engineer to see whether the response had changed or not.

A further aspect is the possible change in seismic character that the rockburst represents. The question is whether this damaging event is

back

back

within the seismic hazard level previously considered for the area or not. If this is not the case seismic hazard ratings would need to be adjusted and necessary precautions implemented as far as they are prescribed for areas with increased rating.

The overall objective of the analysis is to derive recommendations for improved hazard assessment procedures, mining practice and support standards. A successful rockburst evaluation produces concrete recommendations on how to eliminate seismic sources of the type considered responsible for the rockburst and on how to increase the level of protection of excavations. For the former, the source mechanism needs to be established from source parameters and underground observations. For the latter, inspection of the support performance is indicated, especially yield and any observable damage to units.

In addition, rock mass modelling is recommended in cases where the results can shed light on the source mechanism.

EXAMPLE

Learn more about the relationship between damage and seismic source and the excavation support characteristics:

While location, damage mechanism and intensity of damage are functions of seismic source characteristics, two other factors also play an important role: size and shape of the excavation and the ground support characteristics.

Obviously, larger seismic sources directly affect larger rock volumes and can therefore result in extensive crushing, large-scale falls of ground, and significant stope closure. For a given magnitude, high moment - low energy sources have comparatively larger source dimensions. Similarly, high energy – low moment sources radiate waves with greater ground motion amplitudes and acceleration, thereby increasing the likelihood of violent shake down.

Events whose main failure mechanism is slip typically have ES/EP ratios well above 10, with an average for mixed mechanism events being in the region of 3-10. Such slip events radiate strong S-waves, which, under certain conditions, are considered more damaging than P-waves.

Relevant support performance characteristics reflect the ability to withstand dynamic loading at high ground velocities. Most relevant in this context are areal coverage, yieldability, stiffness, and energy absorption capability.

CONNECTION 80

Refer to Paper 2 Outcome 8.2 on page 597

Code of Practice (seismic section)

The current regulatory environment under which South African mines operate requires each mine to identify and manage the various risks to which mining employees are exposed. In the context of seismicity, mines considered 'seismically active' are called upon to monitor this hazard, identify the sources of seismic energy emission and devise strategies to mitigate any risk exposure as far as it is reasonably practicable. 'Reasonable' refers to a balance between the effort to avoid risks and the achieved risk reduction.

It would be considered unreasonable to close a mine in order to eliminate all seismic and other forms of risk. On the other hand, where means can be devised to reduce risk they are expected to be taken. Measures must be implemented which are affordable and effective to reduce the level of seismic hazard and the exposure to loss. In order to be reasonably practicable an intervention should be cost-benefit balanced.

back

The central document required by mines to be created and maintained regularly while dealing with rock related risk is a Code of Practice (CoP) to combat rock fall and rockburst accidents in tabular metaliferous mines. In 2001 the then Department of Minerals and Energy (now the Department of Mineral Resources) published guidelines on the specific contents of such CoP with respect to seismically active mines. These guidelines and the methods employed to achieve compliance are relevant for all mines considered seismically active.



Seismically Active Mine means 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. (from DME 7/4/118 AB1, AUGUST 2001).

This definition differs from a more intuitive definition in that it couples losses and seismic activity. The point could be made that seismicity and seismicity-related losses are two different issues that deserve to be separated. Many mines experience seismic responses to mining but do not experience rockbursts. According to the above definition, such mines would not be termed 'seismically active'.



The CoP is drafted by a mine and submitted to the DMR. The DMR does not approve the content of the submitted CoP, nor the information or the strategies described in it. It merely judges the degree to which the CoP complies with the guidelines. Once the CoP is found to be in compliance, it becomes a binding standard of practice for the mine, i.e. it serves as a vehicle for self commitment. The mine must act in accordance with its CoP. A breach or non-compliance is unlawful and may be prosecuted where the mine acts contrary to the provisions of the MHSA.

INTERESTING INFO

The "Leon Commission" of inquiry into Safety and Health in the mining industry recommended (May 1994) ...

Employees have the right to remove themselves from any place where they have reasonable cause to think that their health or safety are endangered.

- A methodology to be developed, e.g. risk management, to target the most pressing hazards.
- Mine Manager is responsible for suitable and sufficient assessments of the risks to which persons are exposed.

A workmen's representative has the right to relevant training, be informed of accidents, have access to all information in health and safety matters, and to be consulted

In terms of the CoP to combat rock related injuries the following is required from seismically-active mines:

- Procedures for seismic monitoring and seismic data analysis
- Identification of seismic hazards
- Assessments of seismic risk considering all relevant information
- Detail of strategies to combat rock-related hazards and risks (principles, techniques and methodologies) and proof of appraisals by competent persons
- Description of emission sources in different ground control districts
- Strategies to control seismic energy emission
- Explanation of how processed seismic data is utilised in the mine design process
- Description of rockburst damage control measures

The term 'seismic emission source' frequently appears in the guidelines. These need to be described in detail as their recognition is fundamental to the development of mitigating measures. The specific combination of depth, mining strategy and geotechnical setting together determine the seismic emission sources relevant to a mining operation. One or more of the following emission sources may be found in intermediate to deep tabular hard-rock mines:

- small-scale fracturing directly on or ahead of the mining face and ahead of development ends;
- failure of abutments and along excessive leads between panels;
- structural failure in the form of slip on faults and bursting of dykes;

- foundation failure beneath pillars and remnants;
- pillar bursting;
- combination of geology related failure with pillars and abutments.

An additional potential source of seismic emissions recognised on tabular mines is a fault intersecting the shaft pillar, possibly even the shaft barrel.

The identification of relevant emission sources in a mining area requires careful analysis of seismic information with regard to location, time of occurrence and failure mechanism. Data analysis with regard to emission sources requires the seismic network location accuracy to be in the same range as the seismic source dimensions, i.e. a few metres for face-related events, a few ten metres around pillars and abutments, and approximately 100-150m along geological structures and in the back area.

BRIDGING GAP 5

Glossary on page 3

A second requirement is good spatial configuration to obtain reliable estimates of stress drop, ES/EP and where possible, moment tensor inversions. With reliable hypocentral location being of highest importance, an attempt can then be made to associate each recorded seismic event with a likely object of failure, progressing from large to small events. Where there is doubt with regard to pillar versus structure related failure, the decision could be based on relative S-wave energy release or, where available, fault plane solutions from moment tensors.

CONNECTION 81

Refer to Paper 2, Outcome 2.2 on page 35

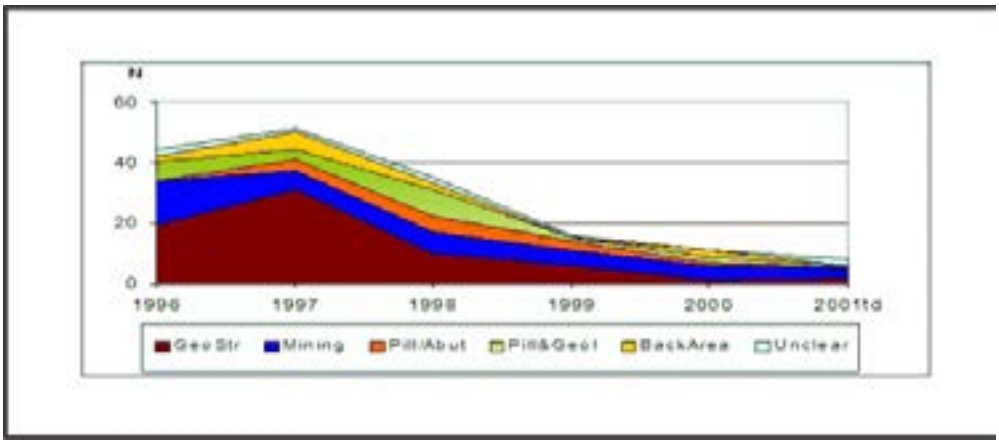


Figure 74: Proportion of annual event frequency M>2 per emission sources (five years).

For detailed instructions on how to derive and formulate monitoring objectives for a mine refer to Outcome 6.2.2

Figure 74 demonstrates the principle of emission source classification when carried out for a deep-level gold mine over a 5-year period (events ML≥2.0). Note the variation in relative importance of the different source classes over time, and the successful partial elimination of structure related events.

The five standard objectives of seismic monitoring on SA mines:

1. Location: Fast location and magnitude for large events;
2. Prevention: Input into planning and design;
3. Control: Track medium-term changes in trends;
4. Warning: Detect short-term pre-cursors of large events;
5. Back analysis: Detailed analysis of single, significant events

back

Once emission sources are recognised and catalogued their relative importance need to be quantified. The same applies to the geographical distribution of these sources as they are likely to play more or less significant roles in different parts of a mine. Many mines are likely to include areas of active, concentrated mining; back areas which are no longer in production; and areas such as the shaft pillar which contain access routes but no blasting. The emission sources typical for each of these are bound to differ.

A description of seismicity in different parts of a mining operation leads to a quantification of seismic hazard. Hazard is defined as the source of or exposure to danger. Statistically, hazard can be expressed as the probability of a loss-incurring incident. In the context of mine seismicity, hazard is usually described in the form of long-, medium- and short-term seismic hazard assessments (SHAs). Each one has specific objectives and a distinct methodology.

1. Long-term SHA is often based on a statistical analysis of recorded event catalogues, such as a mathematical model of the Gutenberg-Richter distribution. This yields recurrence times, probabilities of occurrence for different magnitude ranges, and estimated Mmax. If applied to geological structures, it allows a quantification of geo-seismic hazard.

A similar type of analysis may forecast the seismic hazard to be expected when mining a new area, provided its characteristics are similar to those encountered previously. This allows parameters such as expected seismic energy release per square metre mined and expected percentage of events during night shift and during day shift to be determined. A further long-term SHA method consists of calibrated numerical rock mass models that estimate the seismic response when implementing a planned mining step. Common parameters to forecast are ESS, cumulative seismic moment and M_{max}.



Figure 75: Trends over time in Hazard magnitude for a number of mining areas.

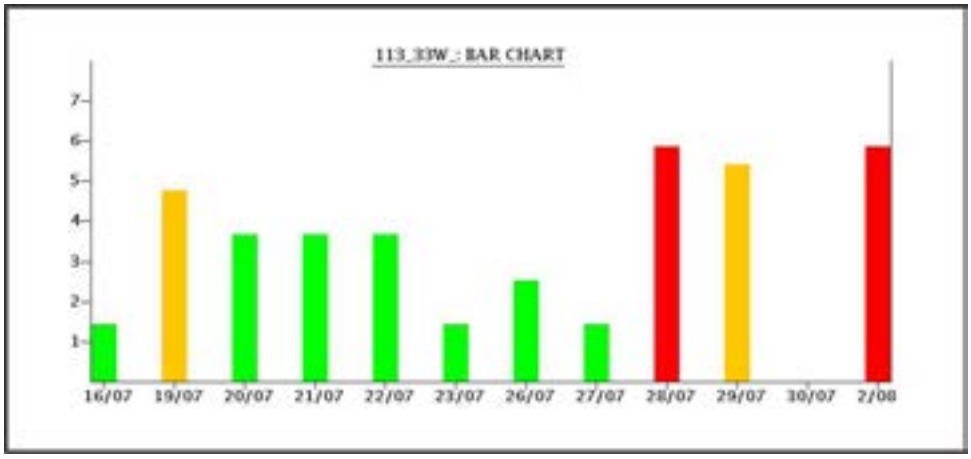
2. Medium-term SHA is intended mainly for monthly planning purposes and relies on spatial and temporal event analysis. Methods make use of event selection by means of polygons and one or more of the following parameters, depending on the methodology (Figure 75):

Trends in activity rate, Hazard Magnitude, cum. Apparent Volume, cum.

back

back

Seismic Energy (ΣEn), cum. Seismic Moment (ΣMo); Contours of Seismic Displacement, Apparent Stress and clustering; stiffness plots and plots of ΣEn/ΣMo per working place or polygon. Polygon ratings can also be based on Potential Damage Volume and M_{max}. Also common are combinations of seismic parameters with mining- and geology-related parameters, such as face shape index, proximity to structures, mining rate etc.



EXAMPLE

Figure 76: A 12-day history of seismic hazard ratings of a working area

Polygon	Medium Term Hazmag	Long Term Hazmag	M_{max}	Apparent Stress Level	Comments & Observations
A	3.3	3.2	3.5	8.6	Decrease in activity from last month.
B	3.1	3.0	2.9	15.4	Backarea events spatially scattered, the largest event being a m1.9 recorded on 3 May @ 06:43.
C	3.0	3.0	3.2	11.9	Increase in activity from last month. Cluster of events m>-0.5 recorded at anomalous TOD near active panels.
D	2.9	2.8	2.5	4.7	Significant increase in the number of events m>0 - accelerated seismic strain rate. Cluster of anomalous TOD events observed.
E	2.8	2.7	2.7	13.1	Minimal seismic activity recorded this month.
F	2.5	2.4	2.4	17.7	Activity decreased from last month.
G	2.1	2.1	1.8	19.0	Cluster of events m>-0.5 recorded at anomalous TOD locating near active panels east of the polygon.

Table 16: An example of monthly hazard level ratings is shown in the table below:

3. Short-term SHA intends to detect changes in seismic hazard that occurs over the period of days and that may indicate instability. It relies on daily seismic activity plots and time history analyses of some or all of the following: Cumulative Vapp, activity rate, Energy Index, Seismic Softening, Seismic Schmidt Number and production related information, for instance volume blasted.

Each polygon rated as such receives a cumulative score that is translated into a symbol (e.g. the bells-and-whistle system) or a colour code (robot system). The symbol/colour is thought to reflect the current level of seismic hazard with regard to rockburst potential. An example of daily hazard level rating is shown Table 17 below:

back

Polygon	Last Event Date/Time	New events	Va	log10 El	Schmidt Number	Activity	Spatial	Large events	Rating
A	20110328 045049	1	0	0	1	1	0	-	0
B	20110319 031520	0	0	0	0	2	0	-	0
C	20110325 102015	0	0	2	0	3	0	-	1
D	20110321 181619	1	0	0	2	3	4	-	1
E	20110322 161238	0	0	1	3	1	2	-	1
F	20110326 170046	0	0	0	0	3	2	-	0
G	20110324 064107	7	1	0	0	1	1	M0.0@25/03,11h05 M1.4@25/03,11h19 M0.1@27/03,05h16	4

Table 17: Daily hazard level rating

Seismic hazard assessments are tools routinely and periodically applied to detect unfavourable trends in seismic rockmass response. For the rock engineer in charge, the question arises how specific emission sources in hard-rock tabular mines can be controlled. As a set of general recommendations for this purpose, the following is suggested:

- leave sufficiently-sized bracket or safety pillars on seismically-active structures; design for ESS values below the critical level at which they show seismic failure;
- protect travelling ways intersecting structures by wide headings;
- optimise sequence and angle of approach when mining near structures;
- avoid the formation of abutments beyond the critical length;
- maintain face lead/lags at moderate levels; design for ERR values below the critical level;
- consider face pre-conditioning and backfilling where it was shown to reduce seismic response levels;
- mine high-hazard areas first, and avoid the formation of remnants.

Contrary to seismic emission control which aims to reduce the frequency and severity of seismicity, rockburst damage control aims to alleviate the negative effects that seismicity has on the mining operation. For rockburst prone mines, the following control measures may prove successful where implemented:

- installation of tunnel and stope support types that are designed for dynamic loading conditions at high ground velocities;
- effective rockburst hazard assessment methods and relevant procedures to communicate and act on the results of such assessments;
- identification of a mine's typical rockburst scenario (where it exists) and systematic elimination of the defining elements of this scenario;
- PPE issued to and worn by the work force at all times.

The combination of hazard assessment, short-term trend analysis and rockburst damage control is meant to give a mine a range of options to optimise seismic risk management with the aim to reduce the recurrence of potentially damaging incidents and the negative effects these could have for the operation. Thus, the ultimate test for successful management of seismic risk is the reduction of seismicity related losses achieved over time.

back

The subsequent chapters introduce principles of seismic system functionality and network layout. This is followed by a discussion of seismic data quality management and methods used to audit seismic services provided to mines.



- Spottiswoode SM, Scheepers J, Ledwaba L (2006): Pillar seismicity in the Bushveld Complex, in FACING THE CHALLENGES, South African National Institute of Rock Engineering, pp140-158.
- Spottiswoode, S.M., Milev, A., Linzer, L.M. and Majiet, S. (2009) Evaluation of the design criteria of Regularly Spaced Dip Pillars (RSDP) based on their in-situ performance, Final project report SIM 04 03 01, MHSC

3.5 MINE SEISMOLOGY AND ROCKMASS STABILITY

LEARNING OUTCOME 3.5.1

3.5.1. MINE SEISMOLOGY

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

3.5.1.1 Describe, explain and discuss the characteristics of mine seismicity in terms of :
3.5.1.1.1 Causes
3.5.1.1.2 Location, Incidence, Magnitude, Time
3.5.1.1.3 Effects

3.5.1.2 Define, describe, explain and discuss the following terms :
3.5.1.2.1 Rockburst, Strong ground motion, Peak particle velocity

3.5.1.3 List, describe, explain and discuss the factors that determine the nature of seismicity in tabular hard-rock mining
3.5.1.4 Describe, explain and discuss the relationship between mine layout and seismicity
3.5.1.5 Describe, explain and discuss the relationship between depth and seismicity
3.5.1.6 Describe, explain and discuss the relationship between rock types and seismicity
3.5.1.7 Describe, explain and discuss the relationship between local geology and seismicity
3.5.1.8 Describe, explain and discuss the relationship between regional geology and seismicity
3.5.1.9 List, describe, explain and discuss the measures that may be applied to control seismicity
3.5.1.10 Describe, explain and discuss measures to reduce the maximum magnitude of seismic events
3.5.1.11 Describe, explain and discuss measures to reduce the frequency of seismic events
3.5.1.12 Define, describe, explain, discuss and contrast the following secondary source parameters in terms of :

3.5.1.12.1 their physical meaning :
3.5.1.12.2 the data required to determine them :
- Source dimension, Apparent volume
- Static seismic stress drop, Dynamic seismic stress drop
- Energy index, Apparent stress
- Dominant frequency, Corner frequency

3.5.1.13 Explain the difference between the following source parameters :
3.5.1.14 Richter magnitude, Local magnitude, Moment magnitude
3.5.1.15 State, describe and explain the main characteristics of the Brune Model
3.5.1.16 Describe and explain how to obtain a fault plane solution from moment tensor analysis

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3.5.1.17 Describe and explain the following common methods of seismic data analysis :
3.5.1.17.1 Gutenberg-Richter analysis
3.5.1.17.2 Magnitude-Frequency relationship
3.5.1.17.3 Energy-Moment relationship
3.5.1.17.4 Trend analysis
3.5.1.17.5 Time-of-day distribution
3.5.1.17.6 Instability analysis

3.5.1.18 Describe typical outputs of the above analyses

3.5.1.19 Describe, explain and discuss the relationships between :
3.5.1.19.1 Extent of stressed rock and Maximum potential seismic event magnitude
3.5.1.19.2 Volumetric closure and Seismic moment
3.5.1.19.3 Event magnitude and Damage caused by seismic events
3.5.1.19.4 Peak particle velocity and Damage caused by seismic events

3.5.1.20 Describe, explain and discuss how peak particle velocity decreases with distance from a seismic event source
3.5.1.21 Identify, describe and explain the basic steps that are involved in carrying out a rockburst site investigation
3.5.1.22 Describe and explain the relevant factors to inspect and investigate at a rockburst site
3.5.1.23 Describe and explain the relevant seismic source parameters to investigate
3.5.1.24 Describe and explain the relevant aspects of seismic history to analyse

3.5.1.25 Describe, explain, discuss and contrast the relationship between
3.5.1.25.1 Rockburst damage and Seismic source characteristics
3.5.1.25.2 Rockburst damage and Excavation support characteristics

3.5.1.26 Describe, explain and discuss the different types of seismic emission sources in tabular hard-rock mining

3.5.1.27 Demonstrate familiarity with the guideline for the compilation of a code of practice to combat rock related accidents by being able to :
3.5.1.27.1 Define, describe and explain the term 'seismically active' as defined in the guideline
3.5.1.27.2 Describe and explain the methods of estimating seismic hazard in tabular hard-rock mines
3.5.1.27.3 Describe and explain the steps necessary to define emission sources in various ground control districts
3.5.1.27.4 Describe and explain the term 'reasonably practicable' in the context of seismic risk management
3.5.1.27.5 Describe and explain the various strategies to combat seismicity related risks
3.5.1.27.6 Describe and explain the various seismic energy emission control measures
3.5.1.27.7 Describe and explain the various rockburst damage control measures.

LEARNING OUTCOME 3.5.2

ROCKMASS STABILITY

DESCRIBE, EXPLAIN AND DISCUSS HOW THE FOLLOWING STRATEGIES ASSIST IN REDUCING ROCKBURST HAZARDS:

- Pre-conditioning,
- Backfilling,
- Stabilising pillars

back

back

By definition, a rockburst suggest that damage or injury followed the occurrence of a seismic event. The impact of each of the above stated practices on reducing the potential for the events to occur, as well as for damage to be reduced is discussed in this Outcome.

Pre-conditioning

Pre-conditioning is a blasting process applied in deep level South African mines in an attempt to mitigate rockburst (face bursts) risk. Stress fracturing occurring in the immediate face rock material, results in the loss of the rock's ability to sustain stress and yield a stress profile ahead of the mining face where the peak stress is situated ahead of the face. Under some conditions, this stress peak as allowed to be formed closer to the immediate face and therefore increase the risk to experience face bursts.

Two possible mechanisms are:

- Due to confinement of rock just ahead of the face, it becomes stronger and does not fail / fracture. This results in the peak stress position moving closer to the mining face position. When the stress increases to a point where the rock fails, the small distance between the overstressed rock and the mining face, allows rock material to be ejected into the face;
- A 'lock-up' of displacement within the fractured rock in the immediate face, allows the stress in this area to increase up to a point where shear failure on the fractures is induced. Since this occurs within the first few metres ahead of the face, rock is violently ejected into the face area.

The impact of pre-conditioning on these two mechanisms is that the detonation of explosives ahead of the mining face and beyond the normal 1.2m production blast:

- fractures the rock so that it loses its ability to sustain stress which moves the stress peak further away from the face area and reduces the risk of a face burst;
- releases the 'locked-up' fractures ahead of the mining face, thereby removing the stresses that built up in this area and reducing the risk of a faceburst.

CONNECTION 82

Refer to "LEARNING OUTCOME 3.1.5" on page 70

Other benefits of pre-conditioning include improved face advances and hangingwall conditions. The improvement in hangingwall conditions (Figure 77) directly affect the potential for falls of ground during the occurrence of a seismic event, even if the event source is quite remote from the working area.

*A possible explanation for this improvement in conditions when applying pre-conditioning is that the precondition blast reduces immediate face stress (releasing locked-up fractures) and moves the stress peak further ahead of the face (fracturing rock ahead of the face) resulting in an area ahead of the production face that is subjected top very low stress l*evels. Since blast damage is assisted by stress levels, the potential for blast damage reduces within this low stressed area. At the same time, the production blast has only to remove rock material that is already completely broken up and uses the energy to throw already broken rock instead of having to fragment the rock before being able to move it.

back

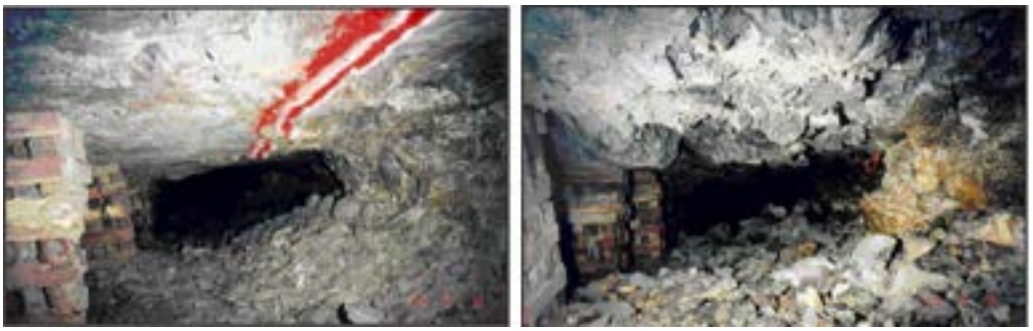


Figure 77: Comparison of ground conditions in mining panels without and with pre-conditioning

CONNECTION 83

Gap336 Preconditioning
Refer to Outcome 9.2

- Pages 251, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.
- Pages 36, 102, 279, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999

CONNECTION 84

Refer to "LEARNING OUTCOME 3.1.7" on page 75

Backfill for regional support

Hangingwall behavior in stopes when subjected to a seismic event

Backfilling

Backfill has a regional as well as local effect on rockbursts:

- Regional: Reduces overall stress levels, thereby reducing shear stress along structures and also face stress levels, which reduces the potential for seismic activity;
- Local: Affects the way in which the hangingwall responds to seismic vibrations and improves ability of hangingwall to remain stable.

The following are also some of the advantages of the use of backfill in rock burst prone mining conditions:

- *it reduces the exposed rock surface area in excavations;*
- *it prevents or reduces rock mass deformations.*

- Chapter 11, Budavari S, Rock mechanics in practice, SAIMM, 1983

CONNECTION 85

Refer to "LEARNING OUTCOME 4.2.1.1" on page 168 and "LEARNING OUTCOME 4.2.1.2" on page 168

Stabilising pillars

Stabilising pillars are classed as protection pillars and are designed to affect the occurrence of seismic activity due to high stress levels. These pillars are utilised to reduce face stress levels (by sustaining large stress levels on these pillars) and also closure into the back areas (by reducing mining spans), both of which are related to the energy available in the rock mass to be released as seismic events.

The Energy Release Rate (ERR) concept has been qualitatively correlated with the overall occurrence of seismic events and is determined from the product of the face stress levels and the closure into the mined out area.

- Chapter 8, Budavari S, Rock mechanics in practice, SAIMM, 1983
- Pages 36, 61, 277, Jager J.A. and Ryder A.J.. A handbook on Rock Engineering Practice for Tabular Hard Rock Mines, 1999
- Pages 80, Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 4.

CHAPTER 4

PAPER 3.1 HARD ROCK TABULAR MINING

INTRODUCTION

Mining layouts are a contributor to the rock mass response to mining, as discussed in the previous chapter. However, this implies that the implementation of an optimum or 'good' mining layout can also assist in changing the rock mass behaviour to be more favourable, more stable, require less support etc. This chapter will look at mining layouts and the possibility of using these layouts to define the rock mass behaviour required by the rock engineer.

4. MINING LAYOUT STRATEGIES

LEARNING OUTCOMES

4.1. HARD ROCK TABULAR MINING METHODS

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

4.1.1. Sketch, describe, explain and discuss the fundamental mining and rock engineering principles associated with the following tabular, hard rock mining methods : Room and Pillar mining, Scattered mining, Longwall mining, Sequential Grid mining, Closely Spaced Dip Pillar mining

4.2. REGIONAL STABILITY STRATEGIES

4.2.1. REGIONAL PILLARS

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

4.2.1.1 Sketch, describe, explain and discuss the functions of regional stability pillars at shallow to intermediate depth where in-stope pillars are not used as local support

4.2.1.2 Sketch, describe, explain and discuss the functions of regional stability pillars at shallow to intermediate depth where in-stope pillars are used as local support

4.2.1.3 Sketch, describe, explain and discuss the functions of regional stability pillars at great depth

4.2.1.4 List, describe, explain and discuss the various design criteria that may be applied to the design of regional pillars

4.2.1.5 Describe, explain and discuss the assumptions associated with these pillar design criteria

4.2.1.6 Describe, explain and discuss the limitations associated with these pillar design criteria

4.2.1.7 Design regional stability pillars for workings at shallow depths

4.2.1.8 Design regional stability pillars for workings at intermediate depths

4.2.1.9 Design regional stability pillars for workings at great depths

4.2.1.10 Sketch, describe, explain and discuss the functions of stabilising pillars at depth

4.2.1.11 Describe, explain and discuss the effect of stabilising pillars on energy release rate at depth

back

4.2.1.12 Describe, explain and discuss the effect of stabilising pillars on excess shear stress at depth

4.2.1.13 Calculate the effect of regular stabilising pillars on energy release rates assuming elastic conditions.

4.2.2. STABILISING PILLARS

See Regional Pillars

4.2.3. BRACKET PILLARS

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

4.2.3.1 Sketch, describe, explain and discuss the functions of bracket pillars at depth

4.2.3.2 Describe, explain and discuss the effect of bracket pillars on excess shear stress at depth

4.2.3.3 List, describe, explain and discuss the various design criteria that may be applied to the design of bracket pillars

4.2.3.4 Describe, explain and discuss the assumptions associated with these pillar design criteria

4.2.3.5 Describe, explain and discuss the limitations associated with these pillar design criteria

4.2.3.6 Design bracket pillars for workings at depth

4.2.3.7 Explain the problems associated with designing adequate bracket pillars.

4.2.4. BACKFILL PERFORMANCE

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

4.2.4.1 Sketch, describe, explain and discuss typical purposes of :

4.2.4.2 Sketch, describe, explain and discuss typical applications of :

4.2.4.3 Sketch, describe, explain and discuss typical selection criteria for :

4.2.4.4 Sketch, describe, explain and discuss typical partical size distribution, for :

4.2.4.5 Sketch, describe, explain and discuss typical stress-strain curves for :

4.2.4.6 Sketch, describe, explain and discuss typical SG s for :

4.2.4.7 Sketch, describe, explain and discuss typical drainage characteristics of :

4.2.4.8 Sketch, describe, explain and discuss typical pumping characteristics of :

4.2.4.9 Sketch, describe, explain and discuss typical stowing characteristics of : Classified tailings backfill, Full plant tailings, Paste fill, Sand fill, Aggregate fill

4.2.4.10 Sketch, describe, explain and discuss the use and effects of cementitious binders with the above types of fill

4.2.4.11 Specify performance criteria, backfill type and placement method to ensure successful backfilling on a mine.

4.2.5. BACKFILL FOR ENERGY RELEASE RATE CONTROL

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

4.2.5.1 Describe, explain and discuss the ability of backfill to reduce energy release rates

4.2.5.2 Explain and discuss the effect of placement lag on the effectiveness of backfill as a means of reducing energy release rates

back

4.2.5.3 Explain and discuss why it is not necessary to completely backfill the mined area to ensure a significant reduction in energy release rate

4.2.5.4 Estimate and discuss the effect of backfill on ultimate energy release rates given backfill performance parameters and stope closure rates.

4.2.6. BACKFILL FOR EXCESS SHEAR STRESS CONTROL

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

4.2.6.1 Describe, explain and discuss how backfill affects the excess shear stress acting on geological features.

4.2.7. REGIONAL MINING LAYOUTS

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

4.2.7.1 Sketch, describe, explain and discuss mining layout strategies to ensure regional stability of workings at shallow, intermediate and great depth

4.2.7.2 Sketch, describe, explain and discuss stoping layouts to ensure regional stability of workings at shallow, intermediate and great depth

4.2.7.3 Sketch, describe, explain and discuss stoping sequences to ensure regional stability of workings at shallow, intermediate and great depth

4.2.7.4 Design stoping layouts and sequences to ensure the regional stability of workings at shallow, intermediate and great depth.

4.3. LOCAL STABILITY STRATEGIES

4.3.1. STOPING LAYOUTS AND SEQUENCES

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

4.3.1.1 Sketch, describe, explain and discuss the layout of the following stoping methods: Bord and Pillar mining, Scattered mining, Longwall mining, Sequential Grid mining, Indicate the location of access tunnels, service excavations and rockpasses

4.3.1.2 Sketch, describe, explain and discuss how the following orebody geometries and/or combinations of orebody geometries may be mined : Wide reefs, Multiple reefs, Steep reefs, Flat reefs, Shallow reefs, Deep reefs, Erratically payable reefs

4.3.1.3 Sketch, describe, explain and discuss how a longwall may be re-established beyond a large geological displacement

4.3.1.4 Indicate appropriate development and stoping sequences for re-establishing a longwall beyond a large geological displacement

4.3.1.5 Estimate relative mining rates for the various re-establishment methods.

4.4. SERVICE EXCAVATION LAYOUTS

4.4.1. TUNNEL LAYOUTS

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

4.4.1.1 Describe, explain and discuss the rock engineering criteria used to determine the optimum size, shape and orientation of service tunnels

4.4.1.2 Describe, explain and discuss the rock engineering criteria used to determine the optimum layout, positioning, orientation and spacing of service tunnels relative to : Static stope abutments, Moving stope abutments, Pillars of various shape, size and

orientation, Geological features of various types, orientation and history, Other excavations

4.4.1.3 Apply these criteria to design service tunnel layouts for : Tunnels of various purpose and life expectancy, Various combinations of timing.

4.4.2. LARGE CHAMBER LAYOUTS

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

4.4.2.1 Describe, explain and discuss the rock engineering criteria used to determine the optimum size, shape and orientation of large service chambers

4.4.2.2 Describe, explain and discuss the rock engineering criteria used to determine the optimum excavation methodology and sequence for large service chambers

4.4.2.3 Describe, explain and discuss the rock engineering criteria used to determine the optimum support methodology and sequence for large service chambers

4.4.2.4 Describe, explain and discuss the rock engineering criteria used to determine the optimum layout, positioning, orientation and spacing of large chambers relative to :Static stope abutments, Moving stope abutments, Pillars of various shape, size and orientation, Geological features of various types, orientation and history, Other excavations

4.4.2.5 Apply these criteria to design large service chamber layouts for : Chambers of various purpose and life expectancy, Various combinations of timing.

4.4.3. ROCKPASS LAYOUTS

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

4.4.3.1 Describe, explain and discuss the causes of instability in rockpasses

4.4.3.2 Describe, explain and discuss the effects of instability in rockpasses

4.4.3.3 Describe, explain and discuss the options available to optimise rockpass layouts

4.4.3.4 Describe, explain and discuss the options available for stabilising rockpasses

4.4.3.5 Describe, explain and discuss the options available for supporting rockpasses

4.4.3.6 Determine appropriate rockpass layout, orientation and support measures using rockmass and stress characterisation techniques

4.4.3.7 Assess the suitability of rockpass layout, orientation and support measures for given sets of rockmass and structural conditions

4.4.3.8 Assess the suitability of rockpass layout, orientation and support measures for given sets of field stress and induced stress conditions

4.4.3.9 Describe, explain and discuss the rock engineering criteria used to determine the optimum size, shape and orientation of rockpasses

4.4.3.10 Describe, explain and discuss the rock engineering criteria used to determine the optimum layout, positioning and spacing of rockpasses relative to : Static stope abutments, Moving stope abutments, Pillars of various shape, size and orientation, Geological features of various types, orientation and history, Other excavations

4.4.3.11 Apply these criteria to design rockpass layouts for : Passes of various purpose and life expectancy, Various combinations of timing

4.4.3.12 Describe, explain and discuss the rock engineering criteria used to rehabilitate damaged rockpasses

4.4.3.13 Estimate the rockpass life expectancy as a result of various support and rehabilitation measures.

4.5. SHAFT LAYOUTS

4.5.1. SHAFT PROTECTION PILLARS

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

4.5.1.1 Sketch, describe, explain and discuss the rock engineering criteria used to design incline shaft protection pillars at shallow, intermediate and great depth

4.5.1.2 Sketch, describe, explain and discuss the rock engineering criteria used to design vertical shaft protection pillars at shallow, intermediate and great depth

4.5.1.3 Calculate stress, strain and tilt in vertical shafts using equations for circular shaft pillars in an elastic medium

4.5.1.4 Apply the results from the above calculations to design appropriate shaft pillar dimensions

4.5.1.5 Design shaft layouts for given depth, geology and rockmass conditions

4.5.1.6 Explain the limitations that concrete linings and steelwork place on allowable movements in shafts.

4.5.2. SHAFT PILLAR EXTRACTION

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

4.5.2.1 Sketch, describe, explain and discuss the preparatory work required prior to shaft pillar extraction at shallow, intermediate and great depth

4.5.2.2 Describe, explain and discuss the extent and magnitude of stress changes to be expected around a shaft before, during and after shaft pillar extraction

4.5.2.3 Describe, explain and discuss the extent and magnitude of movements to be expected in and around a shaft before, during and after shaft pillar extraction

4.5.2.4 Describe, explain and discuss the extent and magnitude of seismicity to be expected around a shaft before, during and after shaft pillar extraction

4.5.2.5 Sketch, describe, explain and discuss appropriate mining sequences for extracting a shaft pillar

4.5.2.6 Sketch, describe, explain and discuss appropriate methods of support when extracting a shaft pillar

4.5.2.7 Design appropriate shaft pillar extraction sequences for given shaft pillar layouts and rockmass data

4.5.2.8 Design appropriate support for shaft pillar extraction workings for given shaft pillar layouts and rockmass data

4.5.2.9 Design appropriate support and protection measures for associated shaft pillar excavations for given shaft pillar layouts and rockmass data.

4.5.3. ALTERNATIVE SHAFT PROTECTION METHODS

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

4.5.3.1 Sketch, describe, explain and discuss the geotechnical issues associated with the following alternative shaft protection

methods at shallow, intermediate and great depth : Satellite Pillars, Bracket Pillars, Partial Extraction, Shaft Reef Extraction, Non-Intersection

4.5.3.2 Sketch, describe, explain and discuss the functions of satellite shaft protection pillars

4.5.3.3 Sketch, describe, explain and discuss the design of satellite shaft protection pillars

4.5.3.4 Describe, explain and discuss the advantages and disadvantages of this approach

4.5.3.5 Sketch, describe, explain and discuss the functions of shaft reef bracket pillars

4.5.3.6 Sketch, describe, explain and discuss the design of shaft reef bracket pillars

4.5.3.7 Describe, explain and discuss the advantages and disadvantages of this approach

4.5.3.8 Sketch, describe, explain and discuss the purpose of partial shaft reef extraction

4.5.3.9 Sketch, describe, explain and discuss the design of partial shaft reef extraction

4.5.3.10 Describe, explain and discuss the advantages and disadvantages of this approach

4.5.3.11 Sketch, describe, explain and discuss the effects of extracting the shaft reef at an early stage to avoid leaving a shaft protection pillar

4.5.3.12 Describe, explain and discuss the advantages and disadvantages of this approach

4.5.3.13 Sketch, describe, explain and discuss the precautions required to allow early extraction of the reef around shafts

4.5.3.14 Sketch, describe, explain and discuss the adjustments required to the shaft to allow early extraction of the reef around a shaft

4.5.3.15 Evaluate mining layouts, sequences, support and precautions necessary to allow early extraction of the reef around a shaft for given circumstances

4.5.3.16 Sketch, describe, explain and discuss the precautions required to mine within shaft pillars transgressed by seismically active geological structures

4.5.3.17 Sketch, describe, explain and discuss the siting of shafts entirely in the hangingwall or footwall so as not to intersect reef

4.5.3.18 Describe, explain and discuss the advantages and disadvantages of this approach.

4.6. LAYOUT DESIGN CRITERIA

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

4.6.1. Describe, explain and discuss the derivation and application of the following mine layout and support design criteria for shallow, intermediate and great depths : Average Pillar Stress, Energy Release Rate, Excess Shear Stress, Rockwall Condition Factor.

MINING LAYOUT STRATEGIES

4.1 HARD ROCK TABULAR MINING METHODS

LEARNING OUTCOME 4.1.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNDAMENTAL MINING AND ROCK ENGINEERING PRINCIPLES ASSOCIATED WITH THE FOLLOWING TABULAR, HARD ROCK MINING METHODS :

- Room and Pillar mining,
- Scattered mining,
- Longwall mining,
- Sequential Grid mining,
- Closely Spaced Dip Pillar mining

Room and pillar mining



Also called a 'bord and pillar' layout

Room and pillar layouts are defined by the creation of a number of 'rooms' or 'bords' (also called drives) between a set spacing of pillars. In a standard layout the rooms intersect each other at 90° and the pillars are usually of a square (or rectangular) shape. Room-and-pillar mining is usually applied in a flat-lying orebodies, although variations of the method can accommodate an orebody dip up to about 30°. Close control of product ore grade is possible since the method allows highly selective extraction of ore. Variability of grade distribution can thus be accepted, with low-grade ore being left as pillars while waste rock produced during mining can be easily be stowed in mined out voids.



Figure 1: Standard 'room and pillar' layout

Basic outlines of room and pillar mining is also given in Figure 2, Figure 3 and Figure 4 below. It is clear that a number of variations to the standard room and pillar mining method are possible, depending on the orebody dip and thickness or mining method (mechanised or conventional):

- Thick orebody: Orebody heights greater than about 6 m are generally worked by multiple passes. Benching is done when two cuts are taken where the top cut is mined by driving 'rooms' along the top of the orebody. The bottom part of the orebody is then extracted by vertical or horizontal benching. 'Vertical' and 'horizontal' refers to the drilling direction of the bench face where 'vertical' suggests the drilling of vertical holes from the top cut elevation and 'horizontal', the drilling of horizontal holes from the bottom cut elevation (Figure 2).
- Inclined orebody: "step' room and pillar can be applied in a mechanised method, where the access for the vehicles need to

be provided at an 'apparent dip', resulting sometimes in the creation of pillars with acute corners (Figure 3).

- Mechanised mining: Require a fairly uniform orebody thickness and dip to accommodate the equipment, but the method is sufficiently flexible to accommodate some local variations in thickness of the mineralised zone.
- Conventional mining: When the orebody is too thin to allow mechanised equipment entry, the conventional 'drilling and scraping' method can be applied to create the rooms and pillars (Figure 4).

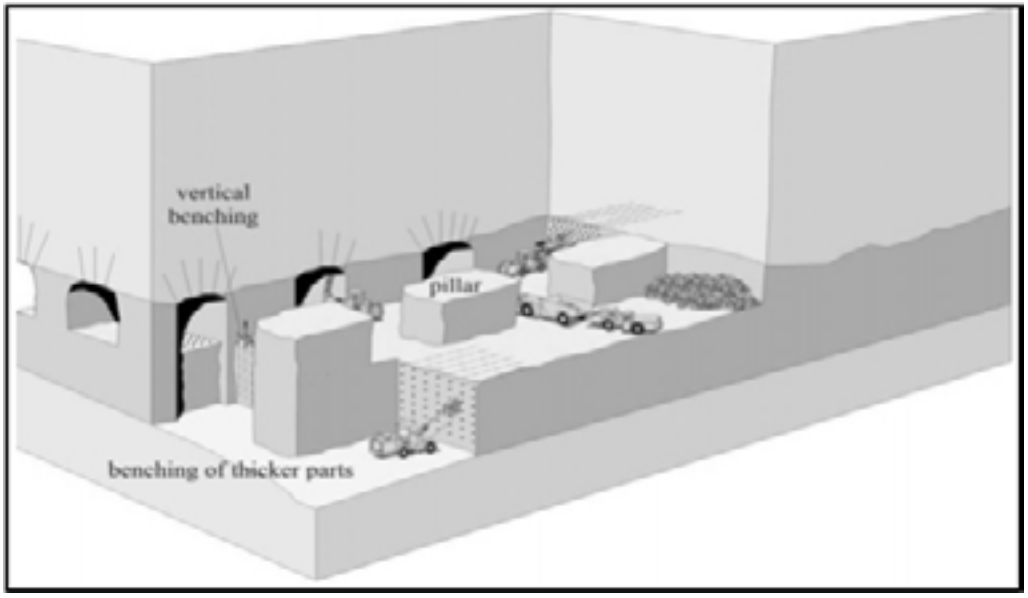


Figure 2: Room and pillar mining configuration for a flat dipping reef or orebody

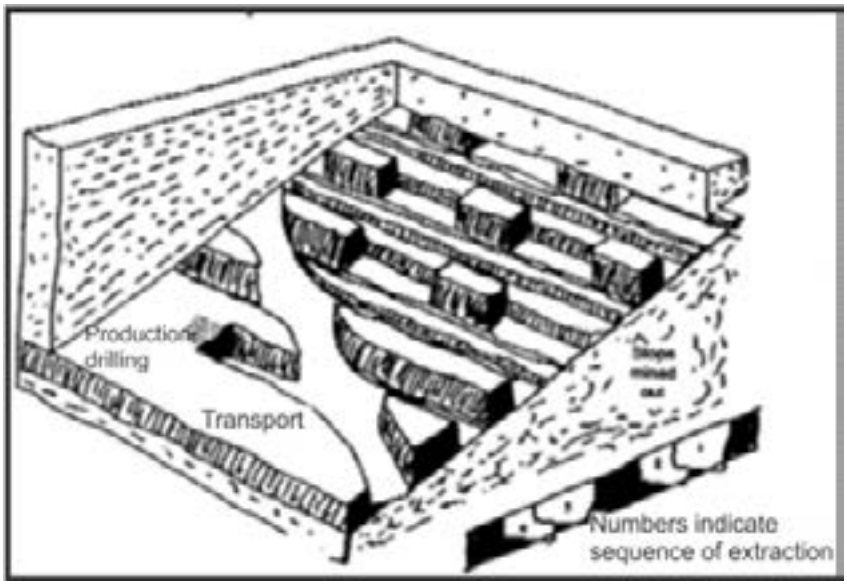


Figure 3: Step room and pillar applied to an inclined reef

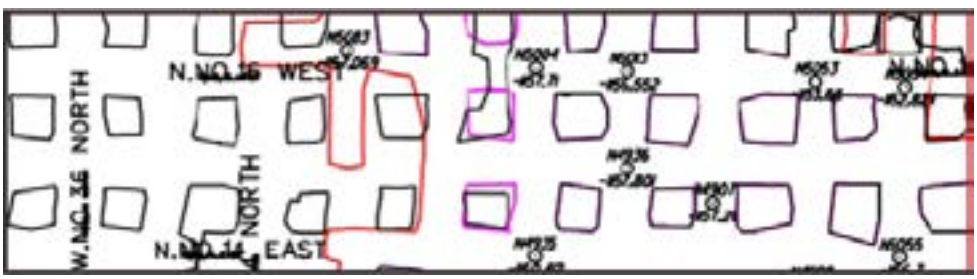


Figure 4: Conventional mining applied to an inclined room and pillar configuration

Ore is produced from rooms (also called bords or entries), each of which serves as a source of ore, but are also access and transport routes as well as airways. Pillars are generated as ore 'remnants' between rooms.

The pillars may be left unmined permanently or alternatively, pillar ore may be recovered in an orderly retreat within a dedicated mine panel or district, inducing collapse of the immediate hangingwall of the mined void and caving of the super incumbent strata.



This is very common on the Coal Mines.

The ore pillars are left intact to control both the local performance of immediate hangingwall rock and the regional response of the host rock mass. This is achieved by the pillar spacings designed to ensure stable spans between the pillars while pillar dimensions are derived by ensuring that regional support (pillars must be able to support the overburden weight). Due to this requirement, the orebody must be relatively shallow to prevent excessive ore pillar dimensions. It is preferable to arrange pillars in a regular grid array, to simplify planning, design and operation and estimation of its support function using for instance the Tributary Area Theory.



TAT (Tributary Area Theory) suggests that pillars support an tributary area around each pillar where the area is common to all pillars with similar size, shape and spacing.

Since personnel operate continuously under exposed hangingwall spans, close observation of the performance of the rooms' hangingwall and pillars is required. Room hangingwalls may be unsupported (this is not a common practice) but is mostly supported / reinforced artificially.



Artificial support refers to support units that are constructed out of steel, concrete, cement, timber, etc. Rock 'pillars' are viewed as 'natural' and not 'artificial'.

The geomechanical setting suitable for implementation of room-and-pillar mining is:

- strong, competent orebody (stable pillars)
- strong overburden (pillar loading occurs regionally)
- relatively low frequency of cross jointing in the immediate hangingwall (stable spans).

Rock transportation is usually via a series of conveyor belts from the workings to either a vertical shaft or continuous incline shaft fitted with an inclined conveyer. Dependant on the depth below surface, the ore is accessed either by vertical shaft (deeper orebodies) or decline shafts.

Scattered mining

Scattered mining is merely a method employed to mine selectively. This entails that the layout is based on a method were development will lead the stoping faces to "unlock" mining ground. The stoping is then done by either a breast, up or down dip face configuration.

The layout allows the mining of 'scattered' pieces of ore and was thus specifically derived for orebodies that are geologically disturbed (eg. high frequency of faulting) and variable grade distributions. These orebodies are usually thin at less than 2.0m thickness, but has been implemented to orebodies of up to 5.0m.

Due to the 'scattered' nature of the mineable parts of the orebody, mining within a specific area may occur at various elevations and orebody dips. This require a lot of pre-development of haulages, crosscuts, travellingways and raises to access the orebody between raise lines over a large number of levels (Figure 5).

This mining method is suited to shallow to intermediate depth mines and can cater for various commodities (platinum, gold, etc) as well as seismically active environments because it is so flexible.

INTERESTING INFO

Haulage: Footwall tunnel developed from the shaft along reef strike, usually at a constant depth below the reef to allow reef accessing excavations to be developed from them.
Crosscut: Tunnels developed from the haulages towards the reef, i.e. along reef dip direction, and is thus 'cross-cutting' through the strata.
Travellingway: Access tunnel with steps developed from the crosscut to the reef elevation.
Raise: Small dimension tunnel developed on-reef and along the reef dip direction, from where stoping panels will be mined.
(See Figure 5 and Figure 7)

The off reef development is initially developed in virgin stress conditions, with the field stress levels increasing due to stresses induced by the on-reef mining that follows the development. Once mining in an area is completed large portions of footwall off-reef development may be destressed (where they have been sufficiently overstoped) but other parts of the footwall development may remain highly stressed where situated under unmined remnants.

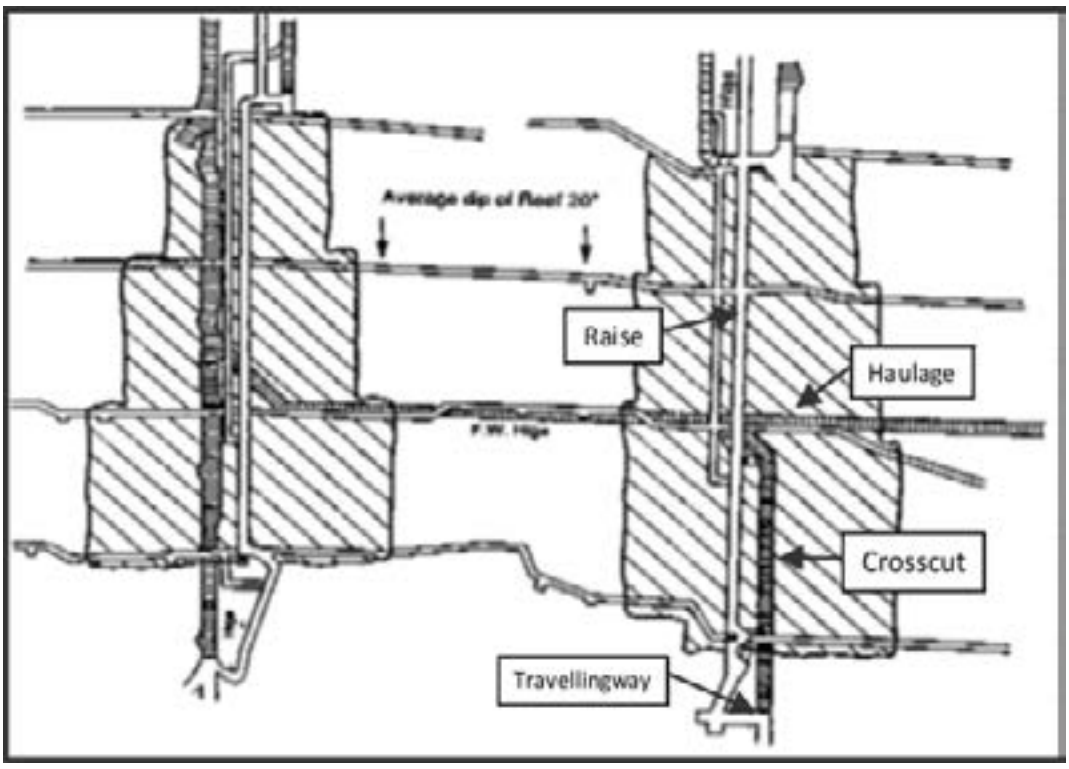


Figure 5: Scattered mining layout

Non-yield, yield or crush pillars, as well as artificial support types, can be used as local support in these scattered mine layouts.

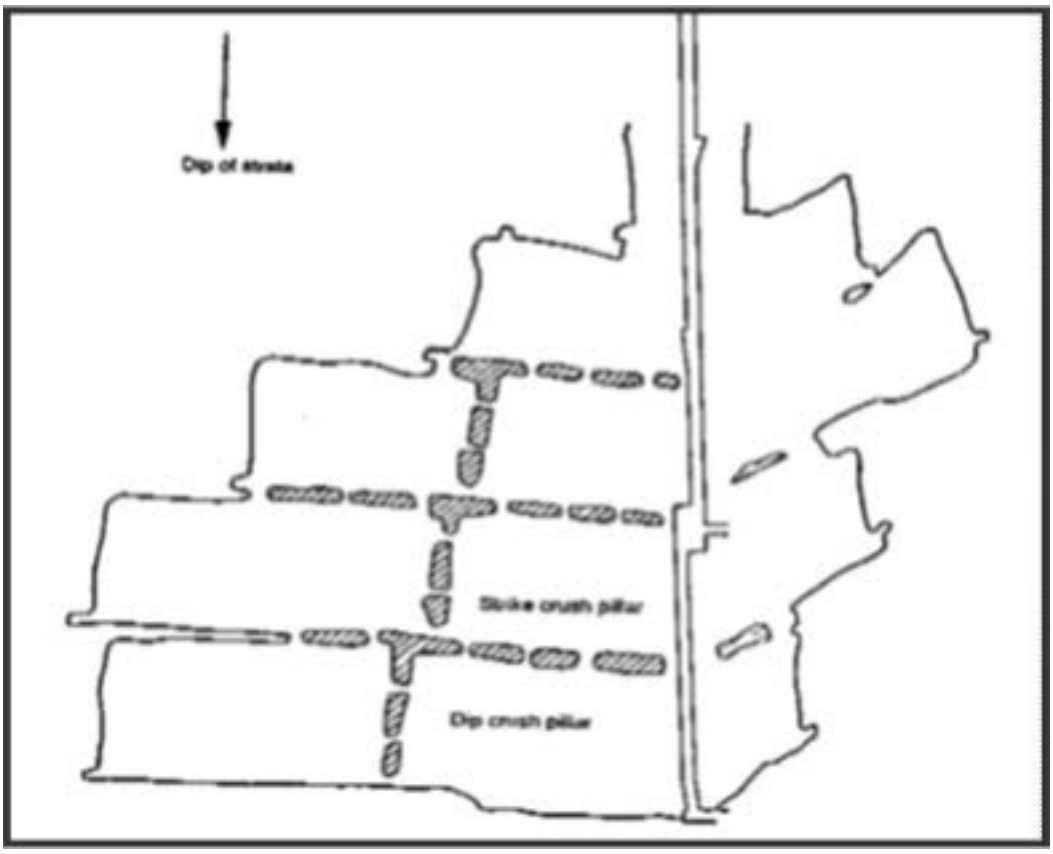


Figure 6: Scattered mining using pillars as in-stope support

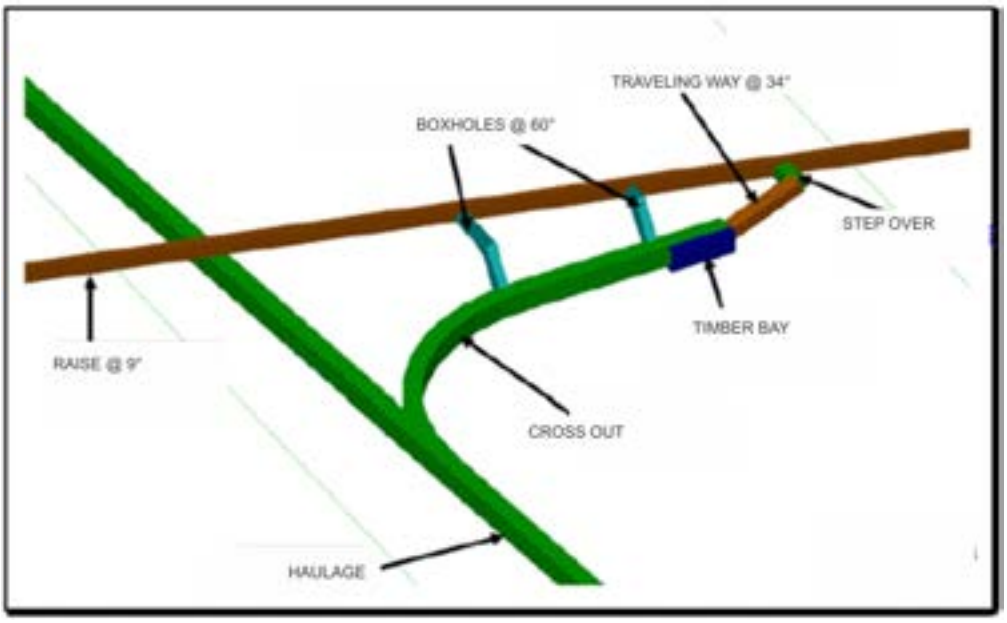


Figure 7: 3D visualisation of scattered mining access

Long wall mining

Longwall mining is used in relatively flat-lying ore bodies where the use of in-stope pillars is not required to maintain stope stability. However, an average ore body dip of 20° is beneficial to assist cleaning methods, while a uniform grade distribution in the orebody and limited presence of geological disturbances (dykes and faults) is critical due to the relatively inflexibility of the method (Error! Reference source not found. and Figure 11). The orebody is accessed via follow-behind footwall haulages (ensure low stress levels), crosscuts and travellingways.

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Longwall mining is applicable only to tabular thin-bedded deposits and a reef of large horizontal extent. Longwall mining developments mainly consist of providing a system of 'strike gullies' for the handling the blasted rock, mainly by using scrapers and for access to the production face areas. The mining (drilling, blasting and broken ore removal) is performed along a face that connects the strike gullies.

Twin haulages, one for intake air (IAW) and the other for return air (RAW), are driven from the shaft (or subshaft) to the block of ground to be mined (Figure 10). To establish a longwall, a single on-reef raise is developed between strike pillars, providing the platform for ledging and stoping. The panels are mined in the strike direction following an overhand or underhand configuration.



Basically two longwall configurations are used in the South African mines: underhand and overhand (Figure 8). The underhand method is usually preferable at depth due to the fact that the faces are always mined towards the solid downdip abutment, reducing face stresses and reducing potential for face bursts.

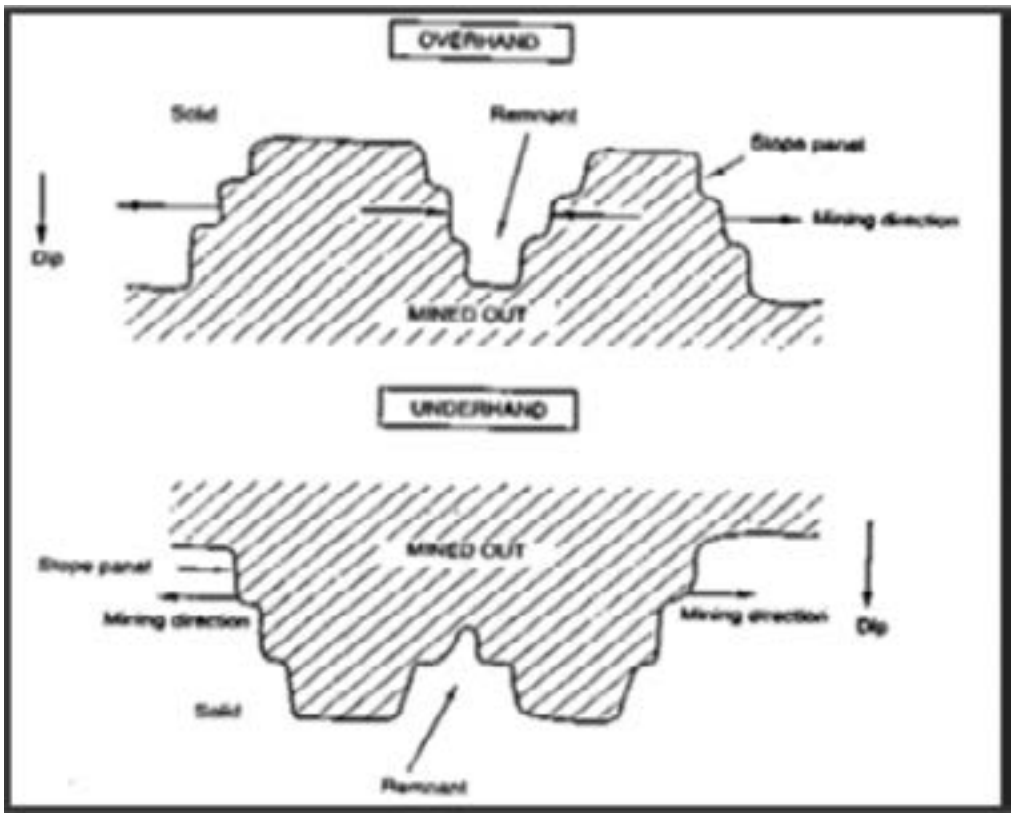


Figure 8: Overhand and underhand configurations for longwall mining

Strike haulages are developed along strike beneath and to within 40 m of the mined-out reef plane. These are known as follow-behind (FB) tunnels (Figure 9) because they lag sufficiently far behind the advancing faces to avoid the extreme abutment stresses. Crosscuts are established at regular intervals on either side of the FB tunnels. Travelling-ways are established at the end of the reef crosscut to provide access to the stope. Upon intersecting the reef with the travel way, a 1.8m high and 1m wide raise is developed on-reef. This is then followed up by ledging and equipping of the raise line.

From the box holes, the rock is transported by train to the transfer orepass or main transfer orepass. The boxhole is inclined approximately at 55°. It is 1x1 m in dimension and connects the crosscut with the strike gullies.

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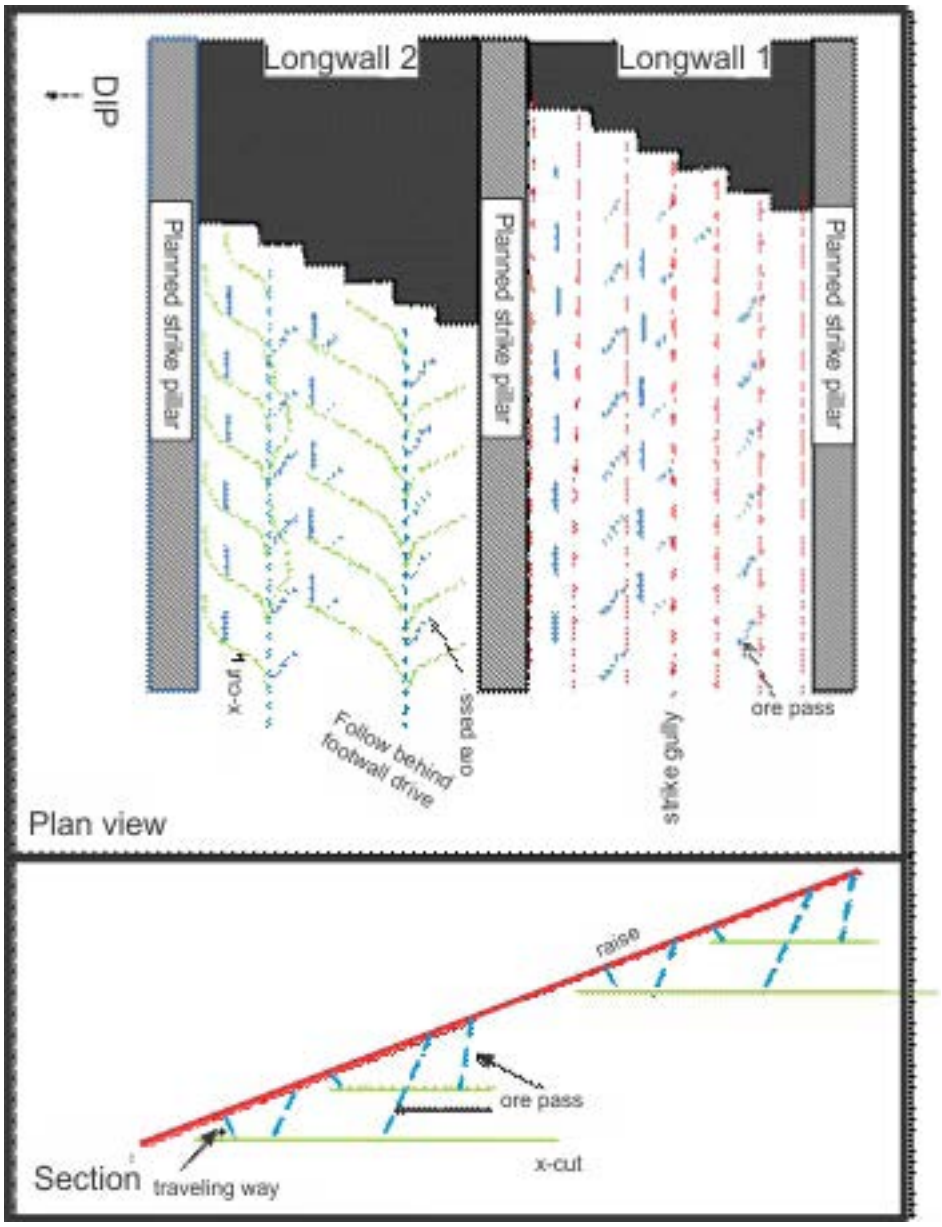


Figure 9: Plan and cross section view of longwall mining. On plan view – Longwall 1 indicate the on reef excavations with longwall 2 depicting the off-reef development

The lower and upper follow-behinds are used as intake and return airways to the stopes. Careful attention must be given to the airflow at the top of a longwall to ensure that no methane accumulates. The top of the longwall is connected to the upper crosscut by a box hole or vent hole. The high virgin rock temperatures at ultra-depth will probably make instope cooling necessary.

The upper longwalls advance ahead of the lower longwalls, yielding an overall underhand mining configuration.

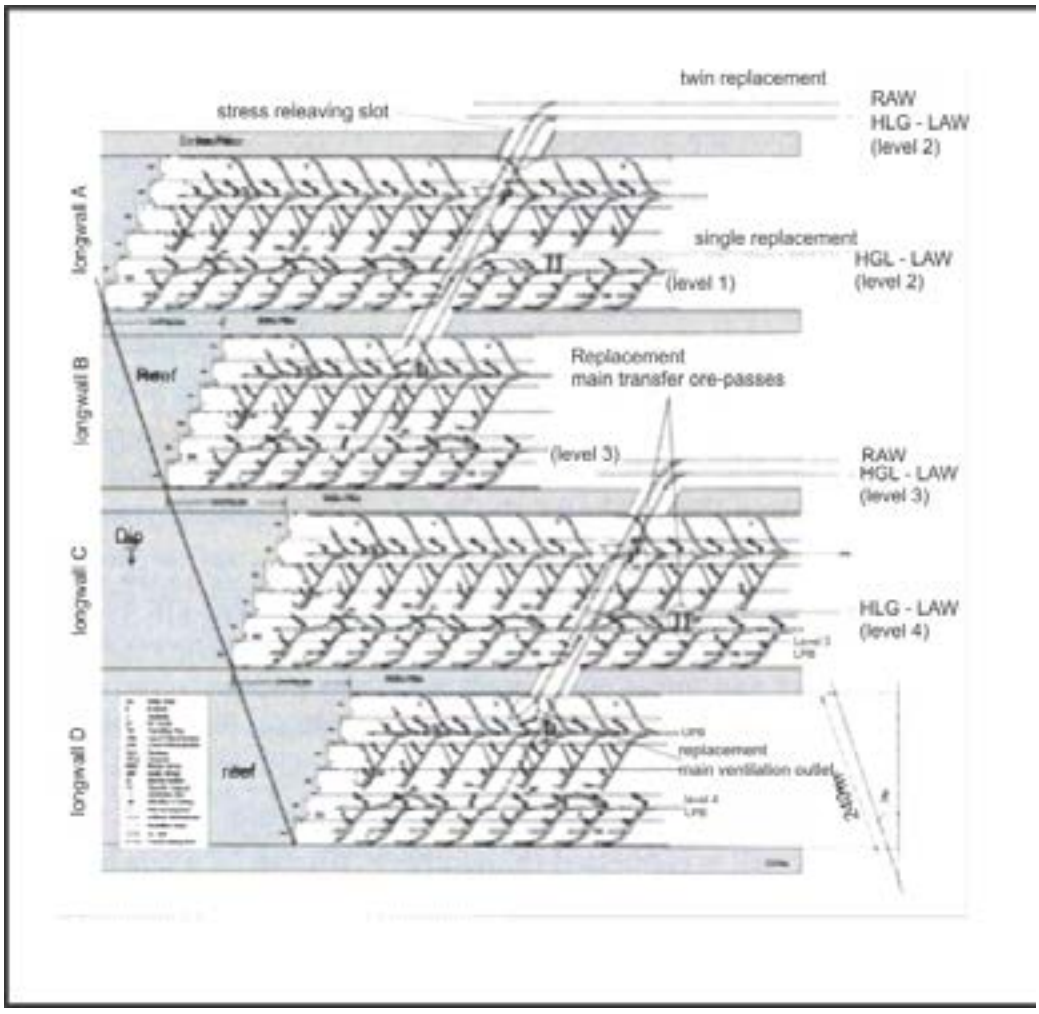


Figure 10: Macro layout of longwall mining

The method allows measures to address high stress levels to be implemented in that footwall tunnels can be de-stressed while regional pillars can be left intact to lower regional stress, closure and available seismic energy levels

Some of the advantages of using the system are:

- Only a single raise needs to be developed for a longwall or mini-longwall which can be mined on strike for long distance (geology permitting).
- Off reef excavations follow behind the advancing panel or stope faces in basically de-stressed ground in the footwall.
- Few highly stressed remnants are left in this mining method.

Some of the shortcomings of this mining method include:

- Lack of flexibility leading to grade control problems.
- Little forward knowledge of geological structures is provided as the primary development lags the stope faces.
- High stresses associated with the stoping abutment that can lead to high levels of seismicity.

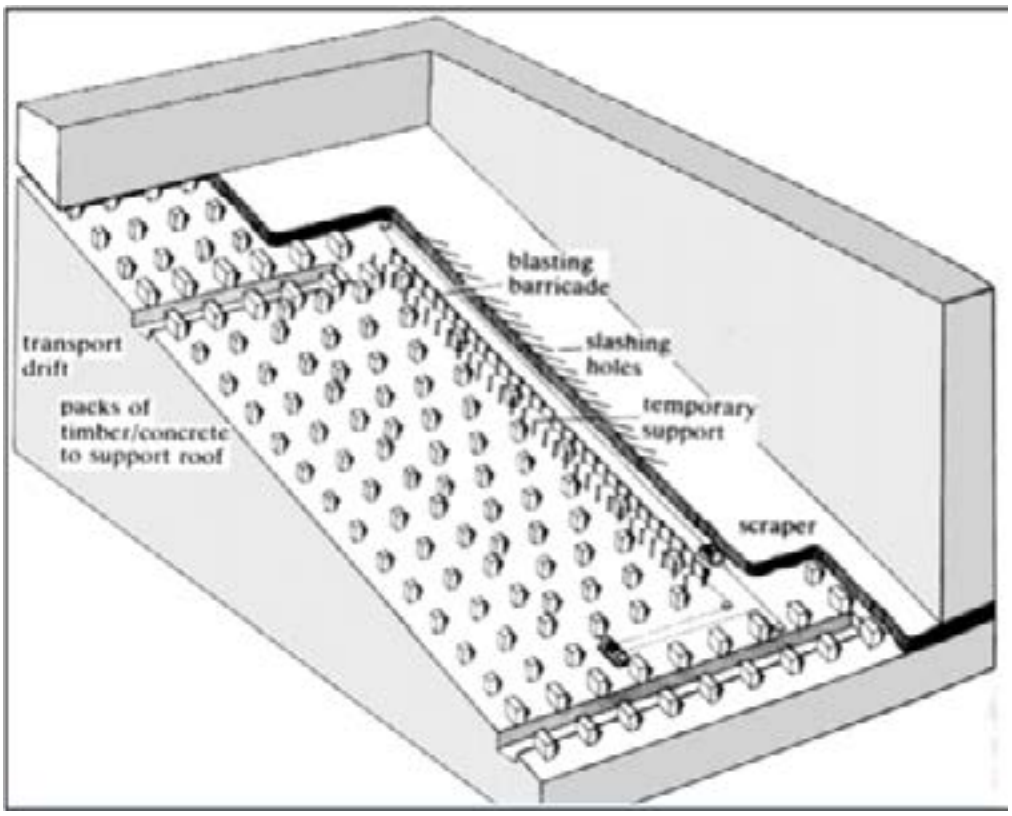


Figure 11: A 3-D representation of longwall mining

Sequential grid mining

This mining method was developed to reduce the seismic activity experienced with longwall and scattered mining layouts on final remnants and strike stabilizing pillars, (Handley et al 2000):

- Sequential places emphasis on the sequence of mining in a mining method very similar to scattered mining methods.
- Grid refers to a grid of regularly spaced crosscuts and raises, borrowed from the scattered mining system.
- Hence the name sequential grid mining.



Figure 12: Sequential grid mining strategy (Handley, 2002)

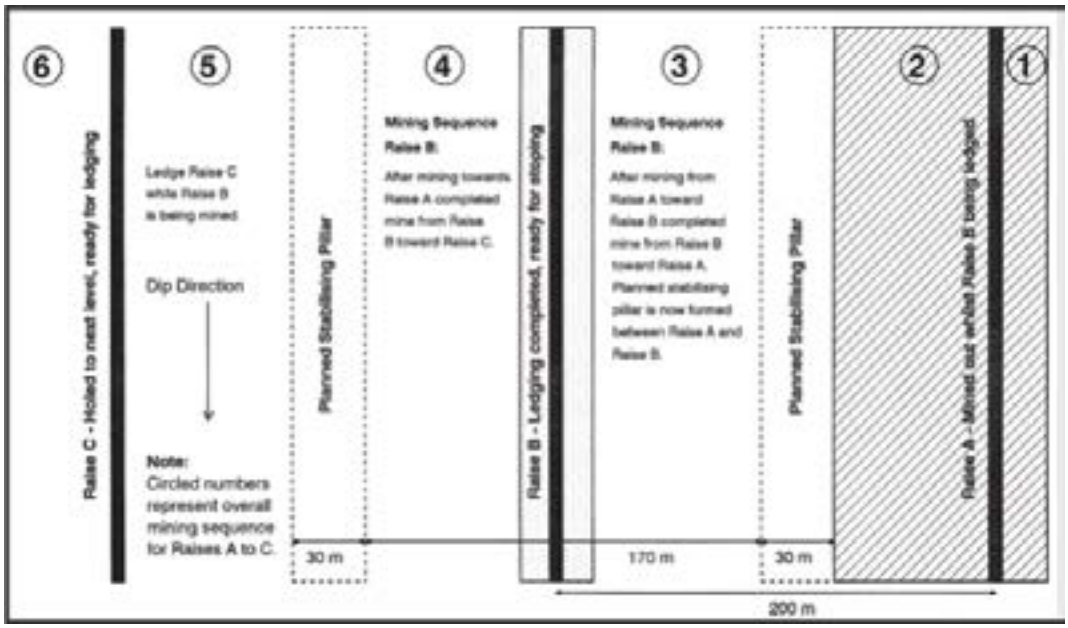


Figure 13: Typical sequential grid mining sequence (Handley, 2002)

As in the longwall method, twin haulages are driven from the shaft to access the block of ground to be mined. These haulages, as well as the reef-access crosscuts, are developed prior to stoping (as practiced in the scattered mining layout), allowing faults and dykes to be located and pillars to be designed to bracket any hazardous structures.

Haulages are situated deep in the footwall (about 120 m) to reduce their vulnerability to stress changes associated with stoping. Crosscuts are developed at regular intervals to give access to the orebody, and the raises are developed, ledged, and equipped.

Mining is carried out according to a strict sequence to ensure low ERRs and pillar stresses. Panels are first mined in an underhand configuration, from the raise line towards the planned pillar position closest to the shaft, followed by mining away from the shaft pillar, called 'single side mining'. Stoping may take place concurrently on two adjacent raise lines, while the next raise is being ledged, and a fourth raise is being developed or equipped. If stoping is occurring in two adjacent raise lines, it is important that the two faces are never closer than 70 m to avoid placing an abnormal load on the pillars.

The broken rock is moved from the face area to the strike gully at the foot of the panel either by scraping or water jetting, and then scraped along the gully to the nearest box hole. One of the most demanding aspects of the SGM is the development of the orepasses servicing the uppermost part of the stope, as the vertical distance from crosscut to reef may be as great as 100 m, depending on the dip of the reef and the back length.

The microventilation system is quite simple, with a dedicated intake at the bottom of the stope, a maximum of seven panels, and a return airflow through a ventilation box hole at the top of the stope to the lower-level return airway. The SGM layout requires ventilation air to be returned from the top of a stope to the lower-level return airway via a disused box hole, which may be a restriction in the ventilation system. Care must be taken to avoid the accumulation of methane in the top section of the stope.

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Fundamentals of closely spaced dip pillar mining
The closely spaced dip pillar mining method is a rarely used mining method for deep level mines and was derived for a particular mining area where neither the sequential grid nor the longwall methods initially appeared to be successful.

In this layout, dip pillars are left in place at close intervals to significantly reduce closure and face stress levels. Mining was conducted using conventional drilling and blasting techniques.

4.2 REGIONAL STABILITY STRATEGIES
4.2.1 REGIONAL PILLARS

LEARNING OUTCOME 4.2.1.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF REGIONAL STABILITY PILLARS AT SHALLOW TO INTERMEDIATE DEPTH WHERE IN-STOPE PILLARS ARE NOT USED AS LOCAL SUPPORT

In a shallow mining environment, stress levels are low and stability of mine workings is maintained by support. The support at very shallow depth includes the use of non-yield pillars that are designed to carry the full overburden (such as in room and pillar layouts) and, as the depth increases, the need for pillars to support the overburden weight slowly decreases until at depth, pillars are no longer required.

During this depth increase, pillar types utilised in-stope changes from solid, non-yield pillars to yield and to crush pillars. It is clear that the ability to generate load reduces as the pillar changes from a solid to crush pillar, while rock mass displacement is allowed when the pillars yield / crush. In an attempt to reduce the in-stope pillar stress levels and still maintain regional stability, regional stabilising pillars are used.

In these environments, the regional stabilising pillars thus have the following functions:

- Support the overburden weight;
- Reduce the size and weight of the rock mass between regional pillars that are subjected to tensile stress fields;
- Compartmentalise the mine so that a collapse in one area do not affect mining in another.

Figure 14 shows that for a certain depth and span, an area is created between the regional pillars that is subjected to a vertical tensile stress field. If this vertical tensile stress field is allowed to intersect surface, a mined out reef or consist of a very poor rock material, a ‘sit-down’ / regional collapse between the pillars results. Regional pillar spacings should thus ensure that this vertical tensile stress zone is as small as possible and remote from any of the situations mentioned above. However, some area will still exist between regional pillar that will be subjected to a vertical tensile stress field and it is this zone that must be supported by the in-stope pillars.

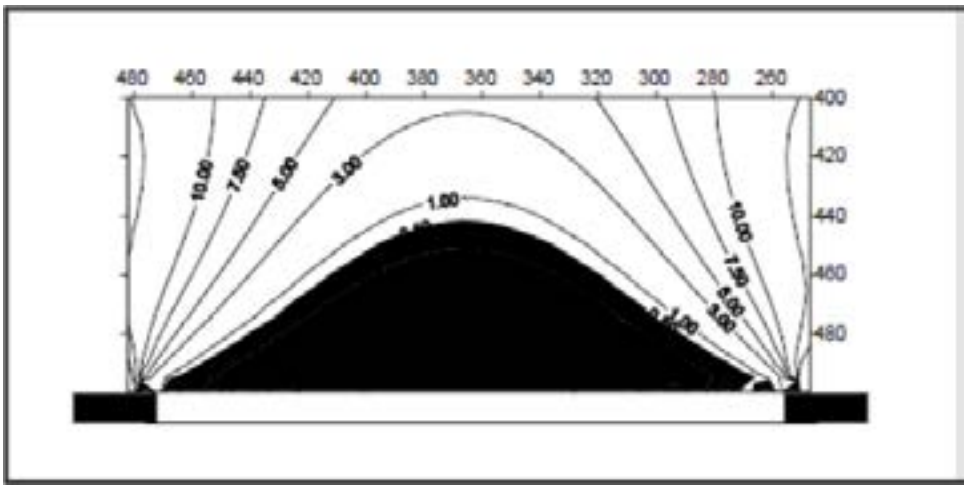


Figure 14: Stress distribution between regional pillars with no in-stope pillars (Ryder and Jager, 2002)

Figure 15 shows that the vertical tensile zone is nearly eliminated by the use of in-stope pillars, as they generate enough load to be able to support the weight of the tensile zone created between regional pillars.



The small tensile stress zones still present between in-stope pillars are then addressed using artificial support such as packs, elongates or tendons.

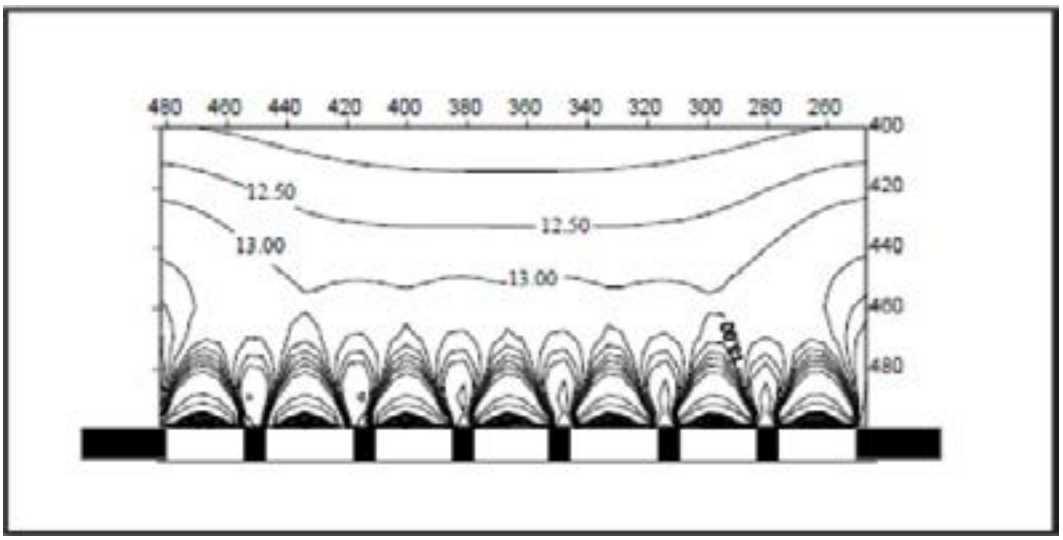


Figure 15: Stress distribution between regional pillars with in-stope pillars (Ryder and Jager, 2002)

By leaving regional pillars in the shallow to intermediate depth environment, stress build-up on the regional pillar reduces the stress levels that will be induced in the in-stope pillars and allows the cutting of smaller dimension in-stope pillars, whilst maintaining overall rock mass stability. At the same time, the use of in-stope pillars also reduce the stress levels generated on regional pillars and allows the optimisation of regional pillar dimensions. Figure 16 and Figure 17 show this pillar stress impact on regional pillars if in-stope pillars are used. There thus exists an interaction between regional and in-stope pillars in this environment.

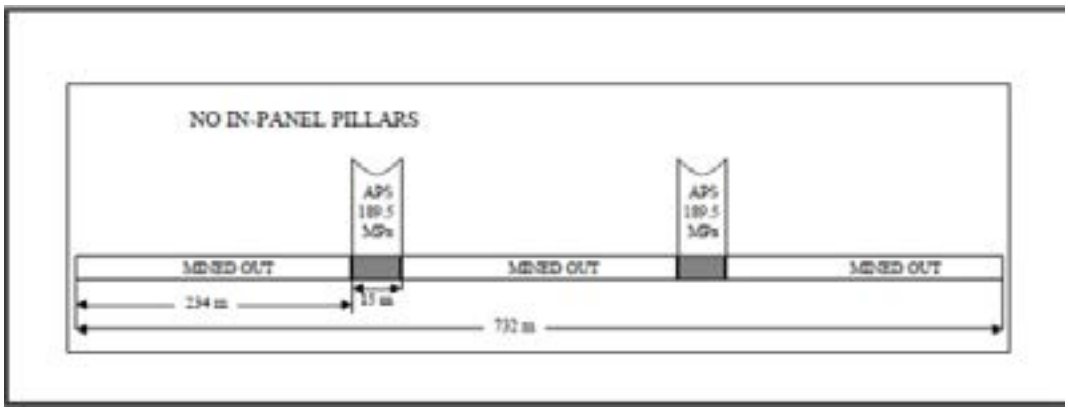


Figure 16: Stress levels in regional pillars with no in-stope pillars (Ryder and Jager, 2002)

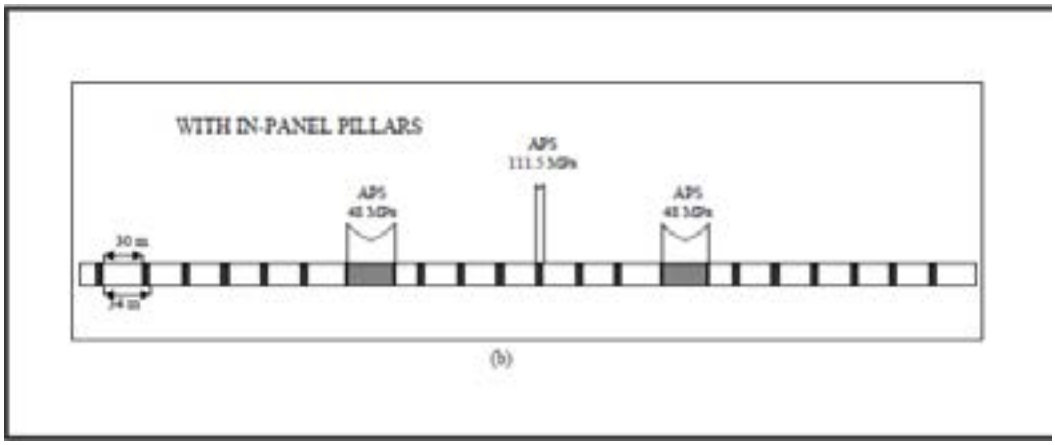


Figure 17: Stress levels in regional pillars with in-stope pillars (Ryder and Jager, 2002)

LEARNING OUTCOME 4.2.1.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF REGIONAL STABILITY PILLARS AT SHALLOW TO INTERMEDIATE DEPTH WHERE IN-STOPE PILLARS ARE USED AS LOCAL SUPPORT

CONNECTION 86

Refer to LEARNING OUTCOME 4.2.1.1 on page 187

LEARNING OUTCOME 4.2.1.3

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF REGIONAL STABILITY PILLARS AT GREAT DEPTH

Regional stability pillars at great depth are designed to ensure that ERR's at the stope face, are substantially reduced as they limit the span, and thus elastic closure and face stress levels. Deep level mines prefer to use regional pillars without in-stope pillars (as discussed in "LEARNING OUTCOME 4.2.1.1").

In general, regional support at depth includes the use of stability pillars or backfill (or both) and has as main aim to provide regional support to limit mining induced seismicity. Regional stability pillars refer to the use of strategically placed pillars (orientated on reef strike or dip) or 'unmined ground' to limit face stress levels and closure. This reduction in face stress levels and closure both improves ground conditions and reduce potential for seismic activity.

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INTERESTING INFO

A water barrier pillar does perform some support function but its design is not based on support requirements. It can however, be incorporated into a stabilising pillar design and allow the pillar to perform a duel function. These pillars are briefly discussed below for information purposes.
All mines produce water in way or another, it is therefore important that this water is separated from the active mining areas. Normally disused water is pumped to old closed off sections of the mine and water barrier pillars need to be established to separate old disused mine water from rushing into the active mining areas.
In designing water barrier pillars certain criteria has to be met such as:

- That the pillars are indestructible in terms of pure crushing (normal width to height ratio of at least 15:1)
- That along the vertical centreline of the pillar, the minor stress σ_3 does not fall below the water pressure involved(to prevent water from opening up sub vertical fissures
- And that rock doesn't fail in triaxial compression according to standard rock failure criteria (to prevent linking up of the foundation failure lobes).
- Cases of multiple reef mining needs to be addressed with particular care. For example a flooded excavation closer than a few hundred meters could pose a significant hazard of water inrush.

Refer to Col 702

The concept of ERR (Energy Release Rate) has been developed to indicate the presence and impact of high stress levels and high stope closure. It has historically been shown that some correlation exists between ERR levels and seismic event occurrences, where high ERR levels not only relate to higher seismic activity rates and magnitudes, but also to poorer ground conditions and reduces ground control by support in cases where seismic activity occurs.

Figure 18 shows that as 100% extraction is approached (i.e. the point where no regional stabilising pillars are left), ERR levels increases substantially with the major increase in ERR commencing at approximately 75% extraction ratio. At the same time Figure 18 shows that for the same percentage extraction, larger pillar widths will require larger mining spans and will thus results in larger ERR levels. In inter-relationship exists between the size of the pillars left and span at which it is left. For instance, larger pillars and larger spans could provide larger ERR levels than smaller pillars and reduced spans.

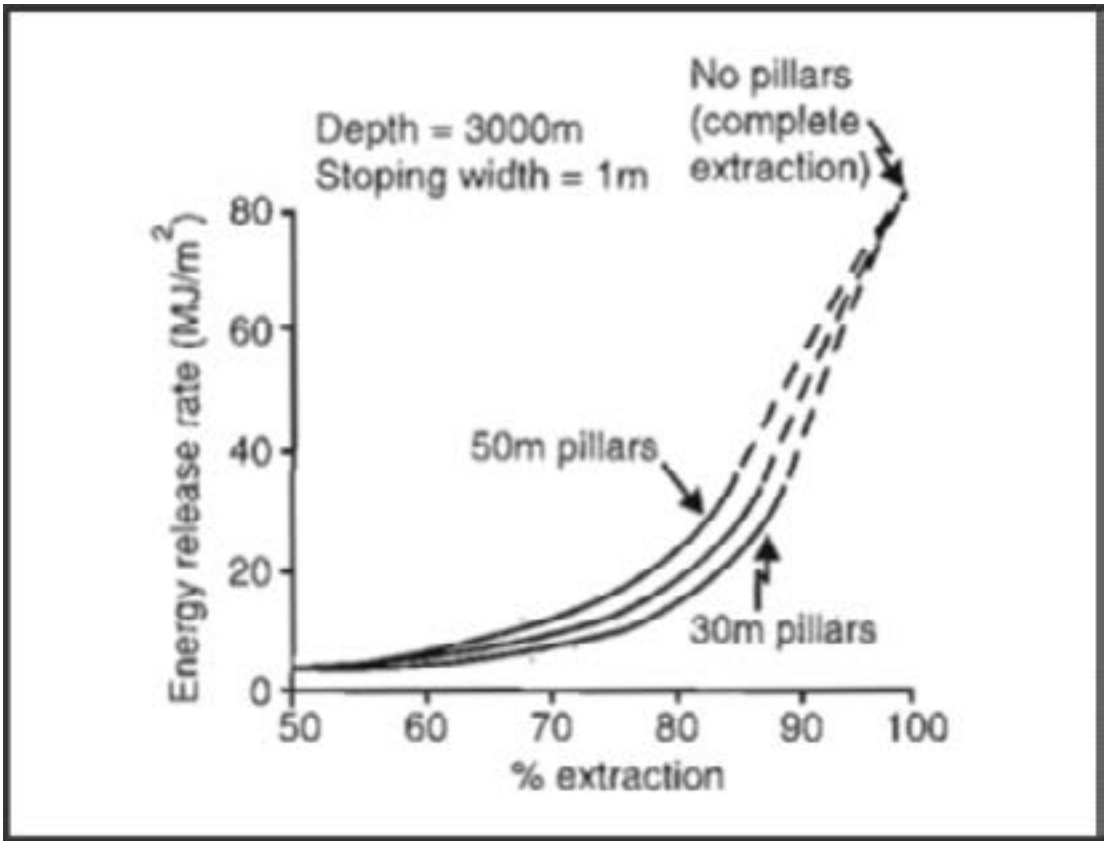


Figure 18: Impact of stabilising pillars have on ERR's at depth (Jager and Ryder, 1988)

The design of stabilising pillars is governed by the following factors:

- ERR control: A target average face ERR needs to be established. Typical values used are 20 MJ/m2 to 40 MJ/m2 and are shaft / site specific.

Refer to the Mine's Code of Practice to address rockfall and rockburst accidents to determine the appropriate ERR criterion (if applicable).

- Pillar width: height ratio: Originally, the acceptable pillar width of 20m for a regional stabilising pillar was viewed to be sufficient. However, the presence of the footwall gullies and significant pillar failures were reported and investigation showed that the fracture patterns within and around the pillar increased the 'real pillar

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height' substantially above the 1m mining height to an 'effective pillar height' of up to 6m in highly stressed pillars (Figure 19). This reduced the original 20:1 width to height ratio to only approximately 4:1 (in the yield pillar range). This resulted in the practice to ensure a wider pillar at between 30m – 40m for the normal mining heights.

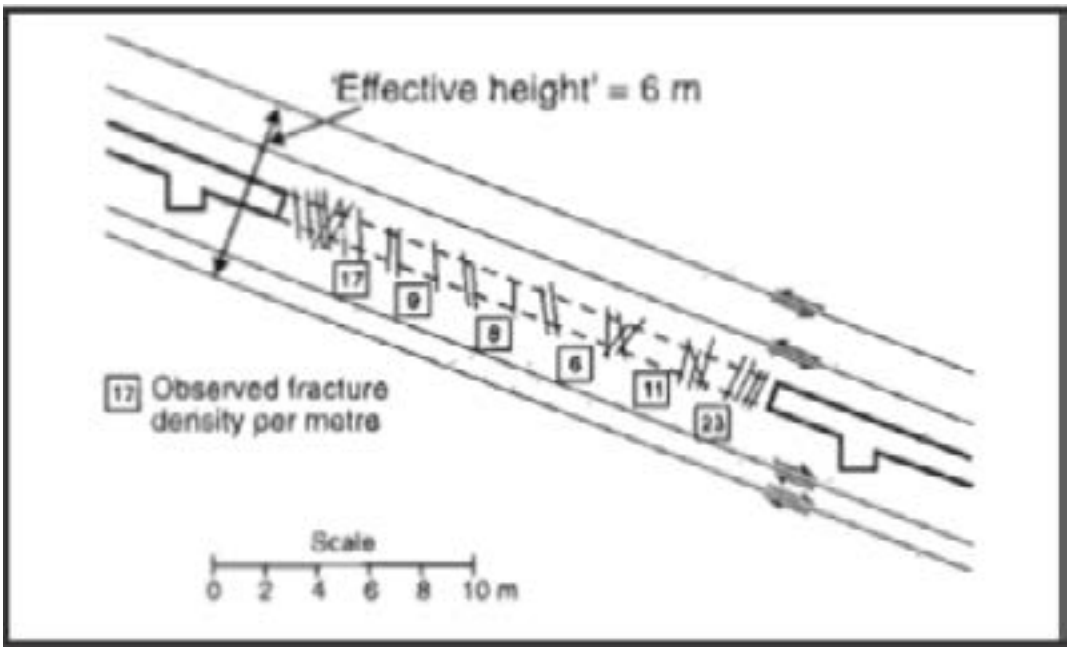


Figure 19: Observed fracturing through a 20m wide stabilising pillar at depth.

- Extraction ratio e: For a given pillar width, the wider the pillar spacing the higher the extraction ratio e, but also the higher the ultimate pillar stress levels (APS or Average Pillar Stress). Higher APS leads to increased potential for violent foundation failures taking place.

NOTE

In designing regional pillars APS levels of less than 2.5 times the UCS of the footwall rock material is suggested.

LEARNING OUTCOME 4.2.1.4

LIST, DESCRIBE, EXPLAIN AND DISCUSS THE VARIOUS DESIGN CRITERIA THAT MAY BE APPLIED TO THE DESIGN OF REGIONAL PILLARS

CONNECTION 87

Refer to LEARNING OUTCOME 4.2.1.3 on page 189

The following are criteria that could be used when designing regional pillars:

NOTE

The user should ensure that the most appropriate and applicable methods are used for the environment designed for.

- Foundation failure: Pillars should be designed to an APS (average pillar stress) less than 2.5σc (where σc is the UCS of the material in the footwall of the pillar). This criterion ensures that the pillar will not punch into the footwall material when highly stressed. Although the 2.5σc criterion is widely used, it has been shown that a ratio at low as 1.2σc has yielded seismic activity. Most applicable environment : Intermediate to deep (or stressed).
- ERR: Acceptable Energy Release Rate levels are available for each operation where this criterion is applicable (mostly highly

stressed, deep level operations). The design include the analysis of ERR levels at any given pillar width and spacing. Most applicable environment : Deep.

- ESS: Large Excess Shear Stress (ESS) lobes are created along a longwall mining face. By limiting the regional pillar spacings and dimensions, these lobes can be limited in size, thereby reducing the rock of large seismic events. Most applicable environment : Intermediate to deep.
- Spans (or regional pillar spacing): Spans are commonly kept to less than ¼H (where H = depth) but spans as large as ½H have been mined successfully in the past. Most applicable environment : Shallow.
- Spacing (or span): Spans are limited in an iterative manner to limit the ERR, ESS and APS levels. Most applicable environment : Intermediate to deep.
- Width:height ratio: Pillar behaviour is affected by the width:height ratio of the pillar. By ensuring that a stable pillar ratio is created, stable pillar behaviour can be ensured. Stable, non-yield pillars usually have width:height ratios in excess of 5:1. Most applicable environment : All environments.
- Pillar strength: If the pillar strength is exceeded by the pillar stress, fracturing, yielding and failure is probable. Pillar strength is a factor of the rock material and pillar dimensions suggesting that this criterion is related to the width:height ratio and APS criteria. Most applicable environment : All environments.

LEARNING OUTCOME 4.2.1.5

DESCRIBE, EXPLAIN AND DISCUSS THE ASSUMPTIONS ASSOCIATED WITH THESE PILLAR DESIGN CRITERIA

CONNECTION 88

Refer to LEARNING OUTCOME 4.2.1.4 on page 191

Design criterion / Environment	Assumptions	Limitations
Foundation failure	Footwall material has a constant strength Pillar material is stronger than footwall material UCS gives an indication of the shear strength of material APS sufficiently indicates pillar stress levels	Intermediate to deep environments Limited to longwall and Sequential grid mining methods Limited to strike stabilising pillars
ERR	ERR is related to seismic activity levels Seismic events do not occur in structures but is generated ahead of the mining face only	Deep environment Limited to longwall and Sequential grid mining methods Insensitive to geological conditions
ESS	Rock material cohesion and friction angle (calculate shear strength) Stress distribution around mined out workings is known Stress field components is known	Intermediate to deep environments Highly stressed environments Mohr-Coulomb shear strength criterion is applied and applicable
Width: height ratio	Stoping width is known, accurate and constant Loading is normal to the pillar axis	Stress fracturing affects the pillar height and is in most cases unknown
Spans	Pillar supports overburden	Shallow environmentSpans become excessive as depth increase and criterion becomes inappropriate, i.e. depth is a limitation.
Spacing	Pillar spacing is constant and known	Intermediate to deep environments Limited to conditions where face stress, closure and thus ERR is appropriate can be accurately determined

Design criterion / Environment	Assumptions	Limitations
Pillar strengths	Tributary area theory is appropriate Modelled stress levels are appropriate Pillar strength is acceptably described by the squat pillar strength formula	Constant pillar dimensions and spacings Pillar is made up of one rock material only (eg. not pyroxenite and chromotite)

Table 1: Regional pillar design criteria, assumptions and limitations

LEARNING OUTCOME 4.2.1.6

CONNECTION 89

Refer to "LEARNING OUTCOME 4.2.1.5" on page 192

DESCRIBE, EXPLAIN AND DISCUSS THE LIMITATIONS ASSOCIATED WITH THESE PILLAR DESIGN CRITERIA

LEARNING OUTCOME 4.2.1.7

DESIGN REGIONAL STABILITY PILLARS FOR WORKINGS AT SHALLOW DEPTHS

Apply the appropriate concepts listed in "LEARNING OUTCOME 4.2.1.5" and "LEARNING OUTCOME 4.2.1.6" to design regional pillars for the specific environment.

EXAMPLE

A mine is contemplating mining a 2m thick orebody at a depth of 500mbs. Suggest a possible regional pillar size and spacing for this environment. The material that will form the pillars have a k-value of 72MPa and the overburden material have an average density of 3000kg.m3. Standard panel lengths has been indicated to be 25m.

From Table 1 follows that the following criteria are appropriate to this environment:

a. Span: Spans between ¼H and ½H where H = 500m, suggests a span of between 125m and 250m;
b. Pillar strength: See Table 2 below
c. Width: height ratio: See below

A possible layout of regional pillars and the squat pillar formula appropriate to squat pillars are given in Figure 20.

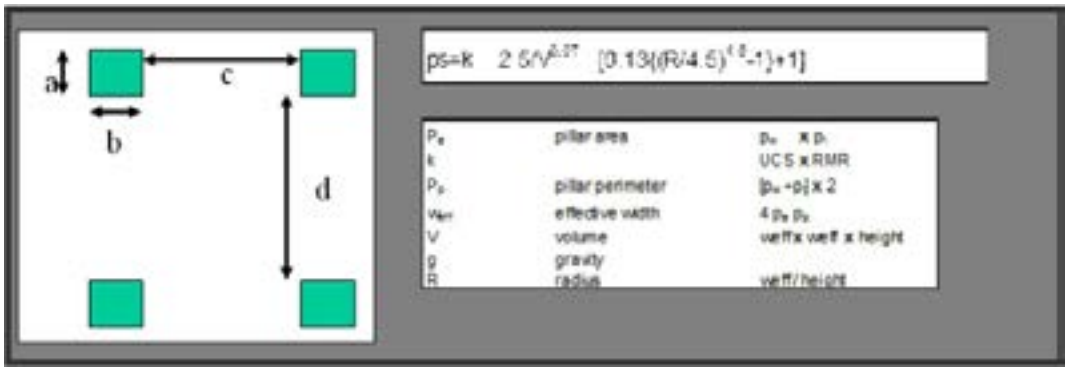


Figure 20: Shallow mine regional pillar layout (example)

It is assumed that:

a. the regional pillars will be large enough to be squat (width : height ratios > 5) in regional pillar so that the squat pillar formula is acceptable
b. tributary area will be sufficient to estimate pillar stresses
c. an acceptable Factor of Safety is 1.6.

Fixed parameters include:

a. depth 500mbs
b. density 3000kg/m3
c. k-value 72 MPa
d. stoping width 2m
e. acceptable FOS of 1.6
f. strike pillar spacing is 90m (places regional pillars halfway between raise and panel limits)
g. dip pillar spacings between 125m and 250m (span criterion).

The variable parameters (i.e. that can be iterated until the best solution is found) include:

a. pillar width and length (a,b)
b. pillar spacing (d)

By creating a spreadsheet with the required formuli embedded into the correct cells, the dimensions and dip spacing are changed according to the following until a FOS > 1.6 results (Table 2). The result indicates that pillars of between 21m (100m dip span) and 25m (250m dip span) provide stable pillars i.t.o FOS.

Depth	Pillar Width	Pillar Length	Span 1	Span 2	Stoping width	k	Overburden Density	Pillar Perimeter	Wur	V	R	Pillar Strength	Extraction Ratio	Pillar Stress	Factor of safety
	a	b	c	d											
[m]	[m]	[m]	[m]	[m]	[m]	[MPa]	[kg/m³]	[m]	[m]	[m²]		[MPa]		[MPa]	
500	21	21	90	100	2	72	3000	84	21.00	882.00	10.50	756	0.9677	448	1.69
500	22	22	90	125	2	72	3000	88	22.00	968.00	11.00	904	0.9706	501	1.81
500	23	22	90	150	2	72	3000	90	22.49	1011.50	11.24	985	0.9740	565	1.74
500	23	23	90	175	2	72	3000	92	23.00	1058.00	11.50	1076	0.9764	622	1.73
500	24	23	90	200	2	72	3000	94	23.49	1103.50	11.74	1170	0.9783	678	1.73
500	24	25	90	250	2	72	3000	98	24.49	1199.50	12.24	1364	0.9809	795	1.80

Table 2: Regional pillar design

The width:height ratios that result from the pillar dimensions in Table 2 are shown in Table 3, indicating that all pillars have w:h ratios > 10, suggesting stable squat pillars.

Pillar Width	Pillar Length	Height	Width : Height ratio
a	b		
[m]	[m]	[m]	
21	21	2	10.5
22	22	2	11
23	22	2	11
23	23	2	11.5
24	23	2	11.5
25	24	2	12

Table 3: Width : height ratio

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The optimum selection will now be affected by practical mining, panel stability and percentage extraction considerations:

a. a 21m wide pillar means that only 4m of a panel will be mined, which is completely impractical. From this viewpoint, it makes sense to rather leave a full panel as a regional pillar every 90m(or investigate rectangular pillar dimensions);
b. a 25m pillar suggests that the dip span can be increased to 250m while the regional pillar will still be stable i.t.o. FOS and w:h ratio;
c. at the same time a 25m pillar at 250m dip spacings provide the optimum extraction ratio of 98%;

However, the 250m dip spacing suggests that the in-stope pillars required to maintain stability of the panels must be able to support a tensile zone that is significantly larger than at smaller regional pillar layouts. These pillars may have to be bigger than for other regional pillar layouts and will have to be designed to determine the final configuration and extraction ratio before a final decision can be made. For the purpose of this example, the regional pillar design is accepted at 25m x 25m pillars, spaced 90m apart on strike and 250m on dip.



The tensile zone height h can be estimated using the formula

$h \approx S.1/((\sqrt{((8.H)/S-6)}))$

where S is the span,

H the depth and

h the height of the tensile zone.

For S= 100, h = 17m and for S=250m, h = 80m indicating the need for substantial in-stope pillars in the final layout.

LEARNING OUTCOME 4.2.1.8

DESIGN REGIONAL STABILITY PILLARS FOR WORKINGS AT INTERMEDIATE DEPTHS

Apply the appropriate concepts listed in "LEARNING OUTCOME 4.2.1.5" and "LEARNING OUTCOME 4.2.1.6" to design regional pillars for the specific environment.

EXAMPLE

At a mining depth of 3000mbs, a longwall mine is being planned. From neighbouring mines it is known that 40 MJ/m^2 is an appropriate maximum ERR level for this environment. Use calculations to determine an initial acceptable mining span for this mine if the following data is also available from test work and planning: Elastic rock properties v = 0.21, E = 60 GPa, the average overburden rock density is $2800kg/m^3$ and the planned mining height is 1.2.

Answer:
Determine the critical half span from $l_c=(s_m\ G)/(2\sigma_v\ (1-v))$



The critical half span is the half of the total span when the hangingwall and footwall comes into contact with each other.

In this formula Sm is the stoping width, G the Modulus of Rigidity (or

Shear Modulus), σv is the vertical virgin stress and v is the Poisson's ratio.

From known elastic relationships G=E/(2(1+v)) where E and v are elastic rock properties and σ_v=ρgh where g is 9.81m/s^2, h is the mining depth and ρ is the overburden material density.

From the above:

$G = 60GPa/(2(1+0.21))$
$= 60GPa/2.42$
$= 24.79GPa$

$\sigma_v = (2800kg/m^3)(3000m)(9.81m/s^2)$
$= 82.4MPa$

Thus:

$L_c = (1.2 \times 24.79\ GPa) / (82.4\ MPa \times 2\ (\ 1 - 0.21))$
$= 189.6m.$

Using the formuli given below, calculate ERR for each of the following halfspan conditions:

1. For halfspans < 189.6m use equation ($l < l_c$)
   $ERR=(\pi(1-v).l.\sigma_v^2)/2G$
2. For halfspans = 189.6m use equation (l =lc)
   $ERR=(\pi s_m\ \sigma_v)/4$
3. For halfspans > 189.6m use equation (l > lc)
   $ERR=(\pi s_m\ \sigma_v)/4((1-(1-l_c/l)^{1.27})/(l_c/l))$
4. For halfspans >>>> 189.6m use equation (l = infinity)
   $ERR=s_m\ \sigma_v$

Using the results from these calculations, plot the ERR values against half-span distances:

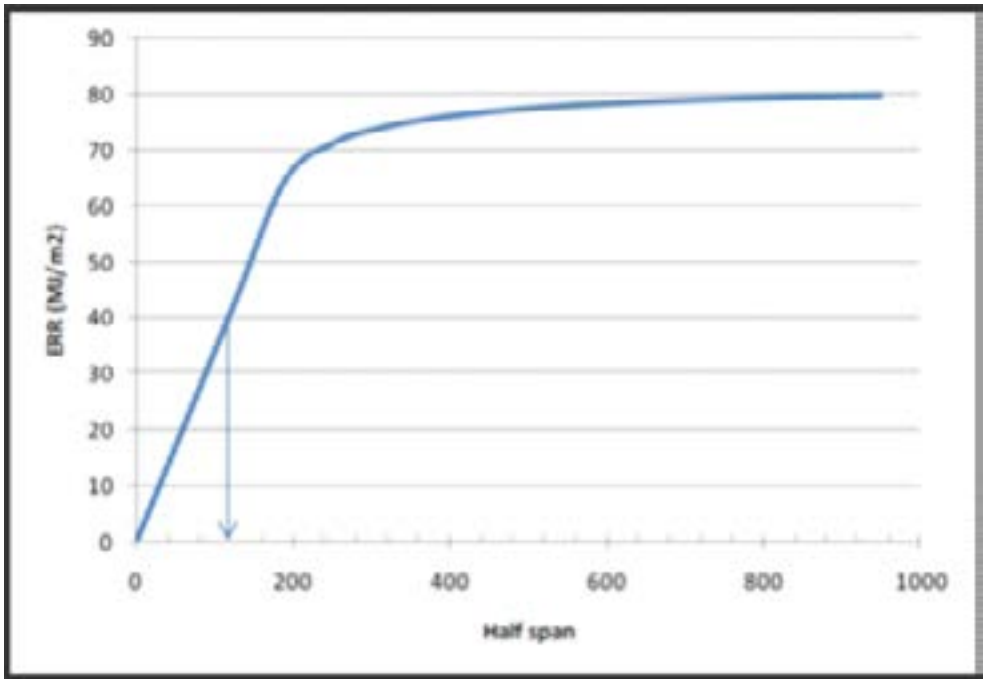


Figure 21: ERR and halfspan relationship

Using the graph above, read off the halfspan at the given ERR limit of 40 MJ/m^2 as 120m, which suggests the full span to be 240m.To ensure that the ERR levels remain below the ERR limit, the span between regional stabilising pillars should remain below 240m.

LEARNING OUTCOME 4.2.1.9

CONNECTION 90

Refer to LEARNING OUTCOME 4.2.1.8 on page 195

DESIGN REGIONAL STABILITY PILLARS FOR WORKINGS AT GREAT DEPTHS

LEARNING OUTCOME 4.2.1.10

CONNECTION 91

Refer to LEARNING OUTCOME 4.2.1.3 on page 189

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF STABILISING PILLARS AT DEPTH

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LEARNING OUTCOME 4.2.1.11

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF STABILISING PILLARS ON ENERGY RELEASE RATE AT DEPTH

Research has shown correlation between ERR levels and seismicity as well as incidence of rockbursts. Energy release rate is thus an important indication of potential seismic / rockburst incidents.

NOTE

Rockbursts are seismic events that cause damage to workings or injury to personnel.

Figure 22 relates actual seismic event (event magnitudes > 1) and rockburst rates (per 1000m^2 mined) for different orebodies and mining areas. It is clear that a relationship between high ERR and higher levels of seismicity is evident.

It is also clear that all orebodies and mining areas do not provide the same level of relationship between seismic rock mass response and ERR levels:

- The carbon leader and VCR reef environments (West rand) have a much larger seismic response (1 to 5 rockbursts / 1000m^2 at 50MJ/m2) to mining than the mining on the reefs in the Free State and Central Rand areas (0.5 to 0.75 seismic events / 1000m^2 at 50MJ/m2);

NOTE

Number of rockbursts is less than number of seismic events (see definition of rockburst above)

- VCR is more hazardous than the Carbon Leader (CL) reef with 3 rockbursts per 1000m2 on the VCR compared to the 1 on the CL.

INTERESTING INFO

Recently, a parameter called Elastic Energy Release (similar to Elastic Strain Energy) is being considered as another good measure of seismic potential. The user is advised to read up on this parameter to test its applicability to specific environments.

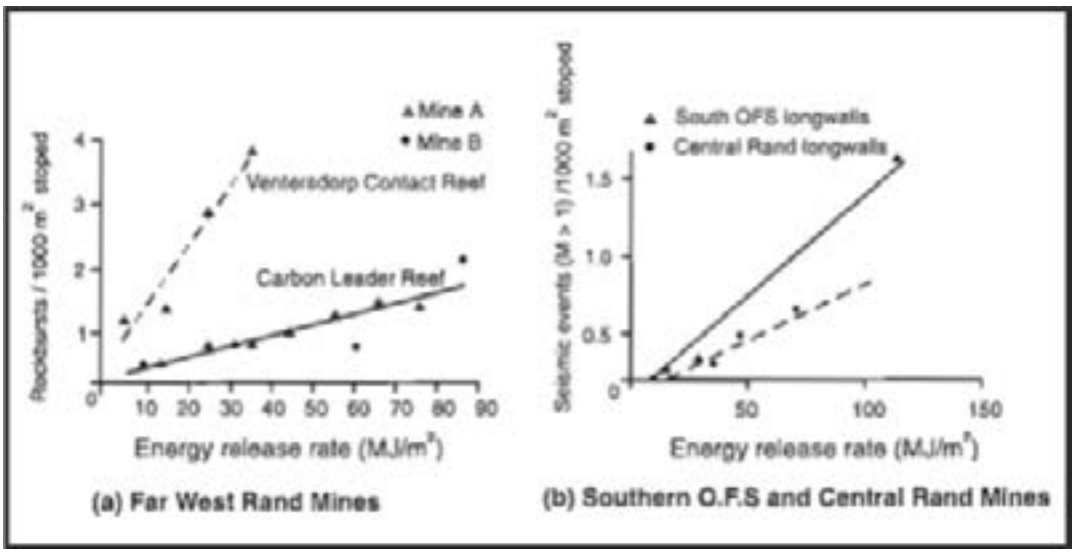


Figure 22: Rockburst and seismicity vs. ERR (Jager and Ryder, 1988)

Since ERR has been shown to be a function of mining span (Outcome 4.2.1.8), it is clear that ERR is affected by the spacing between stabilising pillars. However, the impact of the pillars on ERR is also substantial. Since ERR is a parameter based on the energy balance in an assumed elastic rock mass environment, it is affected by the volumetric convergence taking place around and into a mined out excavation. The stiffness (ease with which it deforms) of the rock mass (provided by the Young's Modulus, spans and pillar dimensions) has a direct influence on the resulting convergence and thus on ERR levels.

CONNECTION 92

Refer to LEARNING OUTCOME 4.2.1.3 on page 189

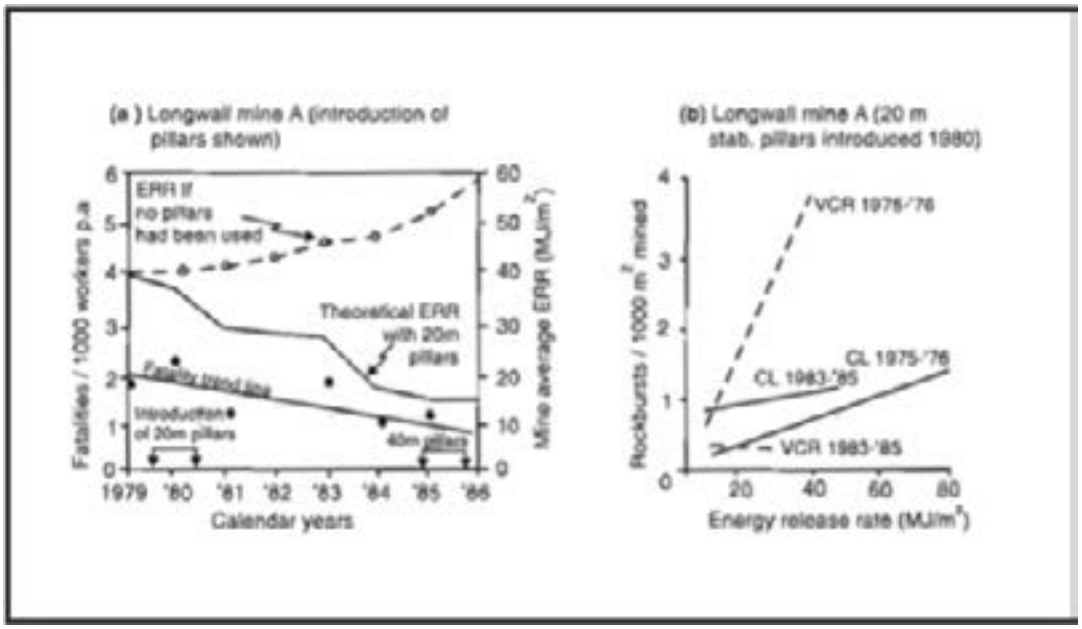


Figure 23: Impact of stabilising pillars on ERR and accidents (Jager and Ryder, 1988)

Figure 23a indicates the impact of the introduction of stabilising pillars in the early 1980's on ERR and rock and seismic related accidents and suggests a reduction in both accidents and ERR levels due to its implementation. The pillars had a clear impact on the slope of the ERR and Rockburst relationships in Figure 23b, indicating that seismic response of the rock mass at high ERR levels was changed by its implementation.

LEARNING OUTCOME 4.2.1.12

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF STABILISING PILLARS ON EXCESS SHEAR STRESS AT DEPTH

The use of stabilising pillars not only helps reduce face ERR levels with corresponding improvements in mining conditions, but also reduces seismicity-inducing ESS) excess shear stress) lobes which exists within the rock in front of the mining faces as seen in Figure 24.

ESS lobes suggest some assumption wrt shear strength of the rock material within the results shown in Figure 23.

An unprotected longwall sees an indefinitely long lobe of ESS just in front of its advancing face and can be subject to face-parallel bursts of large magnitude. A stope protected by stabilising pillars, can advance into areas where the ESS is broken into smaller isolated lobes, where seismic events will be more localised and will have relatively small expected magnitude.

Larger ESS lobe suggests a larger seismic source and thus larger potential seismic event magnitude. Refer to sections on seismicity and the relationship between source size and expected seismic event magnitude.

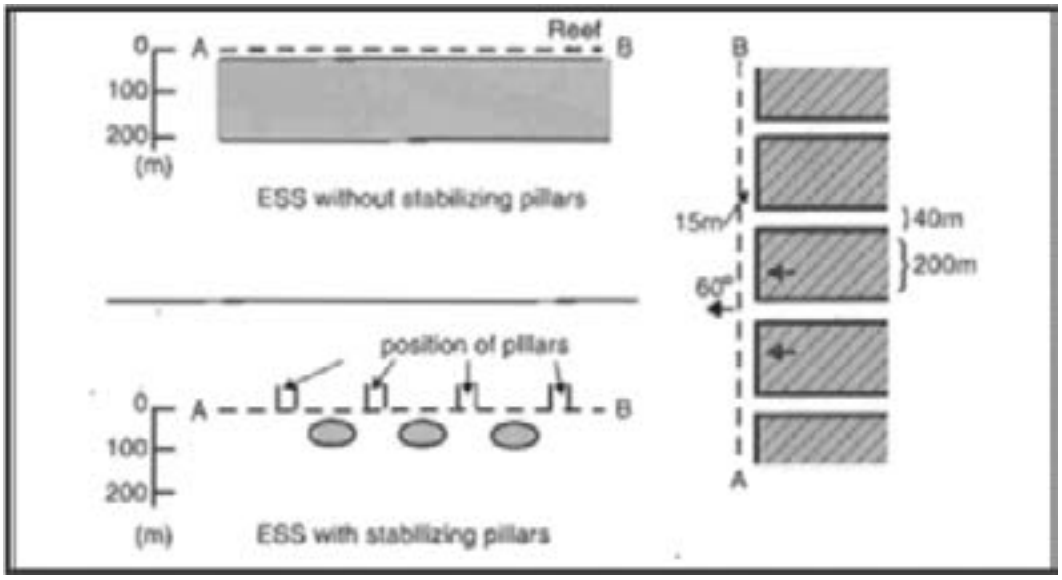


Figure 24: Effect of presence of stabilising pillars on ESS in-front of stope faces (Jager and Ryder, 1988)

LEARNING OUTCOME 4.2.1.13

CALCULATE THE EFFECT OF REGULAR STABILISING PILLARS ON ENERGY RELEASE RATES ASSUMING ELASTIC CONDITIONS.

CONNECTION 93

Refer to LEARNING OUTCOME 4.2.1.8 on page 195

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CONNECTION 94
See REGIONAL PILLARS on page 187

4.2.2 STABILISING PILLARS

4.2.3 BRACKET PILLARS

INTRODUCTION

The need to control stress in the deep level mining environment is aggravated by the existence of problematic geological discontinuities. Essentially, these features can result in the mining operations becoming so hazardous due to the occurrence of seismicity, that mining operations may be abandoned. Bracket pillars are strips of un-mined ground left behind, normally on both sides of a geological discontinuity, in order to:

- 'clamp' the fault / dyke contact planes and inhibit shear failure or
- reduce stress levels on the dyke sufficiently to prevent burst failure.

The bracket pillar may form part of the regional stability system of the area, which will also assist in the controlling of ERR and ESS ahead of the mining face.

The design of bracket pillars is based on the results obtained from numerical modeling, historical data on seismic activity and practical layout considerations.



Especially in highly disturbed geological regions, the identification of which structures to protect with bracket pillars is based on local experience (seismic activity on the structure previously in adjacent sections or mines) supplemented with well analysed quantitative seismic information (estimation of sources of seismic events occurring).

The following basic concepts are applied in the design of bracket pillars:

- Bracket pillars that enclose dykes should have a stress to strength ratio of less than 1.
- Bracket pillars that enclose faults should be designed to cope with excess shear stress, and the reduction of all excess shear stress.
- Bracket pillars should also be designed to limit magnitude of seismic events.

LEARNING OUTCOME 4.2.3.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF BRACKET PILLARS AT DEPTH

The major function of bracket pillars is to control stress levels (compressive and / or shear) on major geological discontinuities, such as faults or dykes.

Shear strength on a plane in the rock mass is determined through the following:

$$\tau_s = c_0 + \mu\sigma_n$$

where

c_o = cohesion between the planes,
μ = coefficient of friction and
σ_n = normal stress acting on across the surface.

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Where cohesion and friction are properties of the rock and/or the surface planes and thus a geological given, the normal stress is a function of the induced stress field component in the rock mass around the structure. The latter (induced stress field) can thus be controlled and manipulated in order to affect the shear strength of the discontinuity as well as the shear stress acting on the structure plane. It is this ability to control the shear stress on and shear strength of structures that forms the basis of the design of bracket pillars in the rock mass together with the effect on regional stability.

On the other hand, bracket pillars may be used to reduce the compressive stress levels on dyke structures that are prone to burst under high stress conditions. The function of the bracket pillars under these conditions is twofold:

- Reduce stress levels on the dyke by leaving unmined rock around the dyke
- Confine the dyke and increase the dyke material's strength, increasing the stress levels required to result in failure.

Both these functions are based on the stress / strength relationship of the dyke material and assume that failure of the material is probable if the stress levels exceed the material strength.



Refer to stress / strain relationships of rock samples or pillars and the point of failure, as well as the impact of confinement on sample strength and the increase in pillar width (suggesting confinement of the pillar centre) on pillar strength.



A Handbook on Rock Mechanics Practice for Hard Rock Tabular Mines (Jager and Ryder) Ch. 3.3.3.

LEARNING OUTCOME 4.2.3.2

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF BRACKET PILLARS ON EXCESS SHEAR STRESS AT DEPTH

When mining occurs, stress distributions ahead of the mining face create zones of high compressive stress and lobes of high shear stress (Figure 25). If the lobes of high shear stress intersect 'unfavourably' with a geological discontinuity, high levels of shear impact on the structure and can induce shear failure if the induced shear stress exceeds the shear strength of the structure (this means that excess shear stress is available).

Bracketing the feature will reduce the influence of the shear stress on the planes of weakness, thereby limit the potential for slip to occur and thus the possibility of seismic events. This impact is due to:

- The bracket pillar increases the distance between the mining stope and the structure, thereby limiting or preventing intersection of the shear stress lobes with the structure
- The bracket pillar increases the stress component normal to the structure, increasing its shear strength ("4.2.2 STABILISING PILLARS").

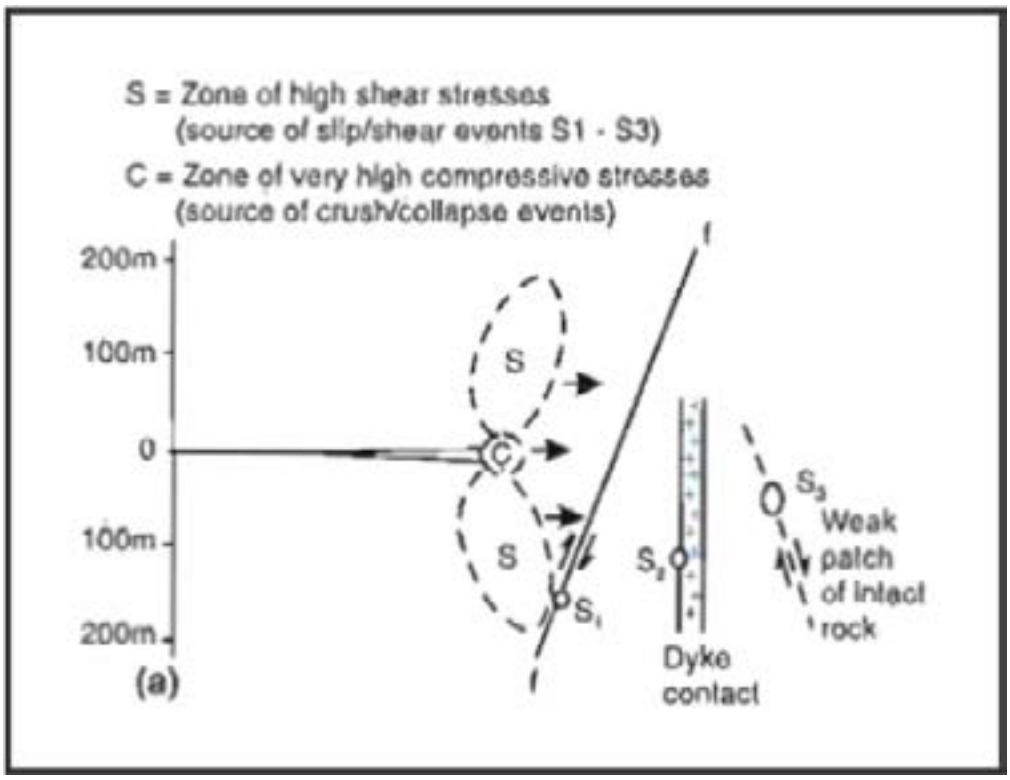


Figure 25: ESS lobes ahead of mining faces (Jager and Ryder, 1988)

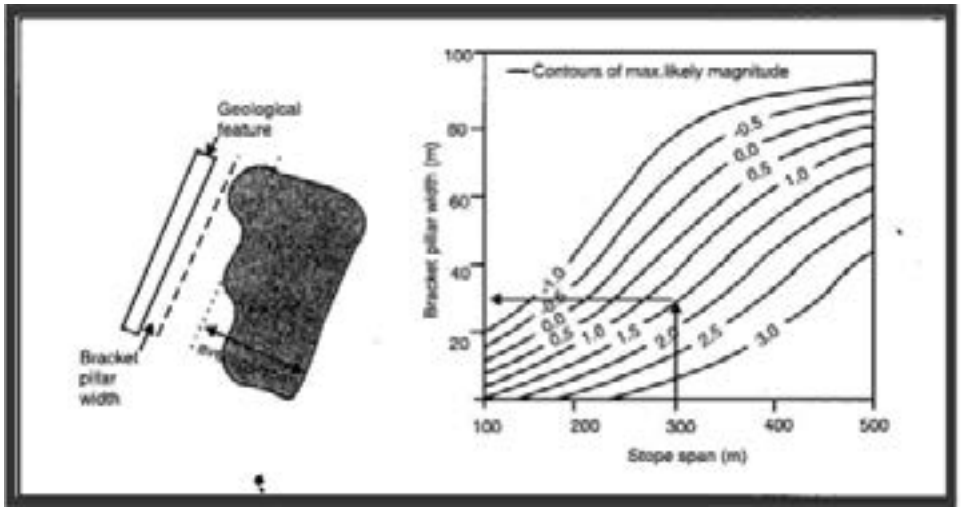


Figure 26: Chart assisting in design of bracket pillar width (Jager and Ryder, 1988)

Figure 26 shows a rough estimate of seismic event magnitudes possible with certain bracket pillar dimensions and mining spans ion that, at a mining span of 300m, a 30m wide bracket pillar could allow seismic events of approximate 1.5 magnitude.



This relationship is based on a number of assumptions, such as mining depth, cohesion and friction on the structure, etc and should not be used for bracket pillar design. It is merely an example of the impact of bracket pillar widths on seismic activity.

In some cases buttress pillars have been used, leaving a thin strip of un-mined ground perpendicular to the geological feature at set distances apart. This fragments and reduces the dimensions of the ESS lobes, limiting the magnitude of possible seismic events by limiting the potential source dimension.

Buttress pillars are pillars not left all along the exposure of the structure, but consist of a number of separate pillars spaced some distance apart along the structure.
Refer to Figure 24 for a similar impact of stabilising pillars on shear stress lobe dimensions.

LEARNING OUTCOME 4.2.3.3

LIST, DESCRIBE, EXPLAIN AND DISCUSS THE VARIOUS DESIGN CRITERIA THAT MAY BE APPLIED TO THE DESIGN OF BRACKET PILLARS

The criteria for the design of bracket pillars discussed below exclude the use of bracket pillars as an integral part of the regional stability design in a specific situation.

- For dykes which are known to be active ('bursting' dykes), the design of bracket pillars are based on the strength versus stress criteria where stress:strength is kept below unity:
- $\sigma_1/\sigma_c<1$: where σ_1 is the stress at the edge of the dyke where the bracket pillar is started, and σ_c is the uni-axial compressive strength (UCS) of the dyke material.

The impact of confinement on the dyke by the bracket pillar is excluded from this criterion while the stress utilised is not the average, but the peak stress on the dyke, making results conservative, and the ratio of 1 acceptable.

- The excess shear strength (ESS) criterion is often used where faults and/or dykes are known to produce shear-type events. The design criterion is based on reducing the occurrence of lobes of positive ESS on or around such features by leaving adequate bracket pillars which result in clamping and strengthening the feature. ESS modeling is used to determine the seismic moment and thus the magnitude associated with slip on a given plane of weakness which is appropriately quantified.

Positive ESS suggests that the shear stress is greater than the shear strength of the structure while negative ESS suggests that the shear stress is lower than the shear strength of the structure, i.e. ESS = shear stress – shear strength or $ESS=\tau_{stress}-\tau_{strength}$ *where the* $\tau_{strength}=C_0+\mu\sigma_n$.

- A number of design charts are available for guidance in the determining the size of bracket pillars but must be used with caution due to the assumptions that are included in each.

- ESS impacts on Layouts Spottiswoode
- Handbook on Rock Engineering Practice section 3.3.3 p65 to 67.

LEARNING OUTCOME 4.2.3.4

DESCRIBE, EXPLAIN AND DISCUSS THE ASSUMPTIONS ASSOCIATED WITH THESE PILLAR DESIGN CRITERIA

Some of the assumptions associated with bracket pillar design criteria are:

- Strength of the rock mass: It is normally assumed in elastic analyses that the rock has infinite strength and can cause serious over-estimation of the performance of a bracket pillar. A stress ceiling is suggested (Spottiswood, 1997) to create a more realistic strength assumption and Vieira et.al. suggested a 250MPa ceiling strength, which correlated well with seismic behaviour at a West Rand test site.
- Worst-case conditions are assumed: For numerical modeling purposes, worst case conditions are assumed and may result in over-estimation, but conservative bracket pillar sizes.
- Strength of the feature: Properties of the geological feature are estimated unless exposure of the feature allows for test measurements of the different strength properties to be done. Cohesion can vary significantly where the feature is not closed. Properties of the contact plane(s) are accepted as continuous throughout the feature. The rock mass is mostly assumed to be homogeneous and linear elastic for modeling purposes.
- K-ratio of the area: Although the k-ratio is accepted as 0.5 for deep level mining scenarios, variations often occur where adverse geological features or abnormal tectonic situations are encountered. This will obviously affect the shear stress distribution ahead of the mining and thus potentially the ESS on the structure.
- Dynamic friction angle: The dynamic friction angle φ are normally accepted as 30°, but on slippery planes of weakness, seem to take on values of as low as 15° to 20°. The use of the plane's base friction angle in the determination of the shear strength may also be an overestimation if previous slip has occurred on the structure.
- The planarity and continuity of the plane of weakness: The extent and planarity of the plane of weakness will have an influence on the strength of the plane and thus on the potential movement along the plane (if it occurs), finally affecting the potential magnitude of seismic event to be expected. Exposure of a feature over a large area will influence the stability and thus increase the possibility of major seismic events while bracket pillars will reduce the area of exposure and hence reduce the size of the area subjected to potential movement.
- The integrity of remnants or pillars in the area: If remnants and/or other pillars in the area have been damaged by excessive stresses or repeated seismic activity, they should be treated as non-existent for the purpose of numerical modeling of the area.
- Maximum acceptable event magnitude: During the design phase of bracket pillars, and especially using the ESS criteria, it is assumed that the pillars should not be designed to totally prevent the occurrence of positive ESS lobes, but rather to minimize it. Therefore, an assumption is made on what the acceptable level of seismic event magnitude will be, and the modeling is then designed to determine this through bracket pillar performance.

LEARNING OUTCOME 4.2.3.5

DESCRIBE, EXPLAIN AND DISCUSS THE LIMITATIONS ASSOCIATED WITH THESE PILLAR DESIGN CRITERIA

The bracket pillar design criteria are not exact science due to the many assumptions included into the design (Outcome 4.2.3.4) as well as:

- Various parameters are based on historical and experiential data
- The knowledge of the properties of the surface planes are often limited
- Strength of the dyke material is not a constant and the stress / strength ratio thus an estimate only
- The information of structure properties can differ significantly in different locations
- The analyses of the stress filed is based on numerical modeling and the use of the results affected by the limitations and assumptions within the models.

LEARNING OUTCOME 4.2.3.6

DESIGN BRACKET PILLARS FOR WORKINGS AT DEPTH

Bracket pillar design does not aim to remove seismicity totally, but the current approach is to design pillars so that seismicity is brought down to a tolerable level in the vicinity of geological discontinuities.

Pillar design charts are utilized in order for rock mechanics engineers to determine the preliminary pillar dimensions for a given structure. The following factors need to be considered:

- The seismic hazard (represented by the seismic moment or event magnitude) along a bracketed geological structure is assumed to be dependent on the amount of arbitrary slip displacement on the discontinuity.
- The largest tolerable magnitude for the region is set. If a tolerable magnitude of 2.0 is accepted, the local stope support would also remain effective for events below the tolerable magnitude. The tolerable magnitude will be determined statistically for a geotechnical area. In the case where such events do occur, the assumption is that no significant influence will be noticeable on the overall ground conditions in the nearby stopes.
- A chart similar to that shown in Figure 26 is created for the prevailing conditions using numerical modeling. In this modeling, mining span and pillar widths are varied whilst the structure strength parameters are maintained as constants. The results can either be expressed as positive ESS lobe dimensions (m2) or be transferred into possible event magnitudes.



The following method of transferring positive ESS to seismic event magnitudes is a possible process.

Ryder suggested (Webber, 1990) that the Seismic Moment can be determined from positive ESS values and the size of the positive ESS lobes, using the following relationship: $M_o = 2\tau_e a^2 l$, *where* τ_e *is the maximum ESS value, and a and l are the half-width and strike length of the ESS lobe, respectively.*

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The seismic moment can then be used in the Hanks-Kanamori moment–magnitude relationship $\log M_o = 1.5M + 9.1$, to determine a probable seismic event magnitude.

- Once the planned mining span has been established, and having determined the tolerable seismic magnitude, the pillar width is read off from the chart that characterizes the prevailing mining conditions (i.e. stope dip, stope span, discontinuity dip, depth, k-ratio, backfill).

Figure 27 below indicates a flow chart of the process.

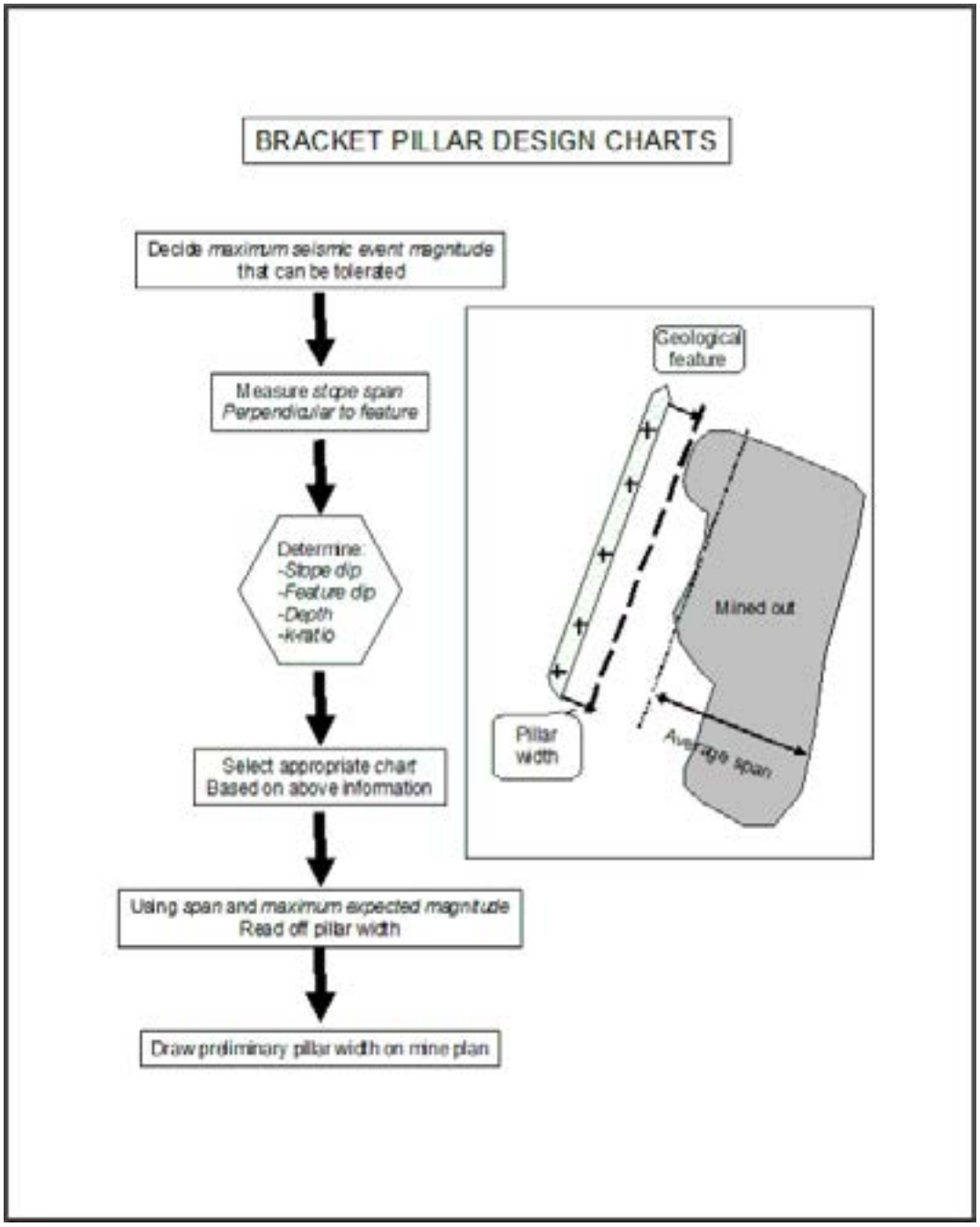


Figure 27: Methodology of obtaining bracket pillar widths from bracket pillar design charts

REFERENCE

- Stabilizing and bracket pillar design (F.M.C.C. Vieira, T. Dede, M.J. Macelari, and S.M. Spottiswood 1997)

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LEARNING OUTCOME 4.2.3.7

EXPLAIN THE PROBLEMS ASSOCIATED WITH DESIGNING ADEQUATE BRACKET PILLARS.

- The design of bracket pillars is based on empirical data, and often relies on the quality of observations and availability of data on the properties of features.
- Numerical modeling programs assume homogeneous, linearly elastic medium and results needs to be used carefully.
- Worst case scenarios assumed can result in overdesign and practical considerations needs to be incorporated in the design phase.
- The properties of geological discontinuities are not always well-documented, and are in some cases virtually impossible to obtain, unless the feature is exposed over large areas.
- Discontinuities are often irregular, and, especially dykes, tend to 'wander around' and 'bulge' around the reef horizon resulting in bracket pillars being ineffective or not large enough.
- Mining discipline in mining around geological structure needs to be maintained to ensure that pillars are cut to the required size and shape.



- A handbook on Rock Engineering Practice for tabular hard rock mines, AJ Jager, JA Ryder
- Stabilizing and bracket pillar design to reduce seismicity by F.M.C.C. Vieira, T. Dede, M.J. Maccelari and S.M. Spottiswood.
- GAP 516 report: Further validation of bracket pillar design methodology by F. Vieira, T. Dede, R. Stewart and A. Ruskovich.

4.2.4 BACKFILL PERFORMANCE

LEARNING OUTCOME 4.2.4.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL PURPOSES OF CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

CONNECTION 95

Refer to Paper 2 Outcome 5.3 on page 361

Backfill is a very important component of underground mining and is increasingly used all around the world. Carefully engineered and efficiently run backfill systems can significantly enhance a mining operation. By contrast, badly engineered and poorly run backfill systems can decrease the productivity of a mine and be a risk to safety.

Backfill is a form of artificial support and is widely used in underground mining environments. Backfill is placed in stope voids and to backfill an area, the stoping geometry and work (production, cleaning, filling, etc) sequence needs to be established and controlled to allow ore removal to proceed unrestricted whilst the placement of the fill should be efficient and to acceptable quality. If this is not achieved, either the production cycle will suffer or the backfill medium will not fulfil its support potential. Three mechanisms may be proposed to demonstrate the support potential of mine backfill:

- by imposing a kinematic constraint on the displacement of key rocks in the stope boundary, backfill can prevent the disintegration of the rockmass in low stress settings

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- both pseudo-continuous and rigid body displacements of stope wall rock, induced by adjacent mining, activate the passive resistance of the backfill to continued displacements
- if backfill is confined, it will generate large stresses levels and may contribute to the regional support function.



All forms of backfill take on one major purpose, and that is to accommodate deformation in the fill mass in order to reduce stress levels throughout the mine whilst maintaining excavation stability. This makes backfill a local and regional support type.

CONNECTION 96

Refer to Paper 2 Outcome 5.3.1 on page 361 (local support) and Outcome 5.3.2 (regional support) on page 363

Backfill has a typical lower modulus of elasticity at typically one to two orders of magnitude lower than the surrounding rock. The purpose of the backfill is not to transmit stresses generated in the rock mass, but to reduce the displacement of the rock mass so that the rock itself will retain a load carrying capacity reducing load shedding to crown pillars and abutments.

Backfill also has the following purposes:

a. Reduces damages due to rockbursts
b. Reduces effective stoping width improving overall stope stability
c. Allows for greater levels of extraction
d. Reduces fire hazard by eliminating timber support
e. Allows for a ventilation barrier

INTERESTING INFO

Originally, backfill consisted of waste rock from development processes or was handpicked ('waste sorting') from broken ore. Some mines quarried rock and dropped it down fill raises to the mine workings. Filling with rock alone is seldom practiced today except for filling or tertiary stopes in some massive mining environments. Recently, paste fill (which contains full plant tailings) has become desirable for many mining methods all over the world. In North America, paste fill is often the default selection when planning for new mining projects and, in a number of instances, was installed at older mines to replace or supplement an existing backfill system.

The natural progression from the original waste backfill was to add fines to the waste at increasing percentage fines until no waste particles remain as part of the final backfill product and it consists of tailings (material dumped after the mineral extraction process has been completed) only. As the material particle size distribution changed from coarse to fine, so did the name given to the backfill type:

- The inclusion of waste particles and fines is referred to as aggregate fill (or comminuted waste),
- large fill material particles is referred to as sand fill
- 'selected' (sizes fall within a certain range) particle sizes are called cycloned (after the process to remove certain particle sizes) or classified tailings and
- the full inclusion of all tailings in the backfill (including all particle sizes), is called full plant tailings.

Rockfill or Aggregate fill
Waste rock from surface or underground is crushed to a typical top size of around 40 mm and placed as is or with cemented hydraulic backfill slurry.

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Sand Fill
Also called hydraulic fill due to the method of transportation. Hydraulic sand fill is used when the tailings are cycloned to separate slimes and sand fractions. The removal of the slimes or clay sized particles provides a material with on average only 4% of -10µm (smaller than 10µm) particles present and where most of the sand particles present are in the 150 - 600µm size range. The slimes are extracted from the fill material due to their poor permeability and are generally stored in a surface impoundment. The sands are hydraulically pumped underground into the voids and can be mixed with binders if need be. As the sands settle and consolidate the excess water is bled off.



Micrometers (µm): 1 x 10^{-6}m or 1 x 10^{-3}mm or 0.001mm, thus 600m = 0.6mm

Classified (also called hydraulic fill)
Hydraulic backfill is produced by cycloning of milled tailings into slurries to produce a material that can be transported hydraulically (with water).

Typical specifications of hydraulic backfill include:

- Not more than 10% by weight of size fraction smaller than 10µm (micrometres). This ensures that acceptable permeability of the placed fill is achieved.



Permeability: The ability of material to allow the flow of water through the material.
Porosity: The percentage void volume to total volume of a material.

- Slurry density (tonnes/m^3) is greater than 70% Cw (solids by weight) which minimises the drainage of transport water. The target density is a function of particle 'specific gravity' and the controlling parameter is % Cv (solids by volume) which balances the requirement to transport the slurry without plugging the line with limiting excess drainage water. The practical range is between 45% Cv and 50% Cv.

Hydraulic backfill slurries are transported by gravity through boreholes and pipelines to the stopes. The slurry enters the stope and the solid particles consolidate rapidly (called flocculants and can be added to the fill to speed up the consolidation and water drainage). The rate of water drainage is a function of the permeability of the backfill. The quantity of water that will drain is a function of the initial slurry density and the residual moisture content of the backfill.

Free draining hydraulic backfill has a typical porosity of around 50%, can be walked on in a few hours and is trafficable within 24 hours. Where the hydraulic fill is to be exposed, cement addition rates of around 6% are typically required and this will result in unconfined compressive strengths of greater than 750 kPa within 28 days. Higher strengths and faster curing can be achieved by adding extra cement.

Paste Fill
Full plant tailings are filtered or thickened and water is then added to achieve the required rheological characteristics. Paste backfill consists of the full size fraction of the tailings stream prepared at a high slurry density. Due to this high density, the slurry requires an applied force to commence flowing (non-Newtonian fluid).



An example of a Non-Newtonian fluid is toothpaste and the applied force (yield stress) explains why you have to squeeze the toothpaste out of the tube.

The paste has a higher viscosity than hydraulic backfill and exhibits plug flow when transported in a pipe. The outer portions of the slurry shear against the sidewall of the pipe and the central core travels as a plug. The friction factors generated require that high pressure pipelines are used to transport paste-fill and typically exceed 5 MPa

The higher slurry densities improve the water to cement ratios and for a given tonnage of paste-fill, lesser quantities of cement are required to achieve comparable strengths to hydraulic backfill, which has obvious operating cost benefits. The higher density and reduced permeability due to the inclusion of fines also results in negligible water drainage from the paste fill. The need for engineered barricades is removed and the nuisance of drainage water and impact on ventilation is eliminated.

CONNECTION 97

Refer to Paper 2 Outcome 5.3.3 on page 363

There are a number of disadvantages to paste backfill systems and these relate to the higher capital cost primarily attributable to the expensive filter dewatering systems and the complexity of operations and supervision required to ensure that line plugging does not occur. Because of the inclusion of the slimes fraction and the non draining nature of the backfill, cement must be added to all paste-fill placed underground including the fill that will not be exposed by subsequent mining to eliminate the risk of liquefaction. This aspect places a significant operating cost penalty to the use of paste fill compared to hydraulic fill which due to its free draining nature can be readily be placed in uncemented forms.

REFERENCE

- Brady BHG & Brown ET 1993 Rock Mechanics for Underground Mining
- Chapman & Hall New York Chapters 14

LEARNING OUTCOME 4.2.4.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL APPLICATIONS OF CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

Although applications are as many as there are mining layouts, orebodies and goals for the use of backfill, Table 4 gives the most common applications, including applications outside of the hard rock, tabular environment.

Application goal	Environment of application	Potential use of backfill types to reach application goal				
		Classified tailings	Full plant tailings	Paste fill	Sand fill	Aggregate fill
		Commonly used in hard rock tabular mines		Not commonly used in hard rock tabular mines		
Reduce ERR, seismicity (regional)	Hard rock tabular	Yes	Yes	Yes	No	No
Improve ground conditions (local)	Hard rock tabular	Yes	Yes	Yes	No	No
Pillar confinement	Any	Yes	Yes	Yes	Yes	Yes
Working platform	Cut and fill	Yes (with cement)	Yes (with cement)	Yes (with cement)	No	Yes (with cement)
Working place roof	Undercut and fill	No	No	Yes (with cement)	No	No
Surface protection	Any	Yes	Yes	Yes	Yes	Yes

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Prevent backbreaks	Shallow tabular	Yes (with cement)	Yes (with cement)	Yes (with cement)	No	No
Ventilation control	Any	Yes	Yes	Yes	Yes	Yes
Improving extraction ratio	Any	Yes	Yes	Yes	Yes	Yes
Maintaining small open mining spans	Massive mining	Yes (with cement)	Yes (with cement)	Yes (with cement)	Yes (with cement)	Yes (with cement)
Stowage of waste	Any	No	No	No	No	Yes
Protection of middling during multi-reef extraction	Any	Yes (with cement)	Yes (with cement)	Yes (with cement)	No	Yes (with cement)

Table 4: Backfill applications

The figures below (Figure 28 and Figure 29) shows graphically some of the impacts backfill has on ERR and pillar stress levels in confirmation to the information given in the table above.

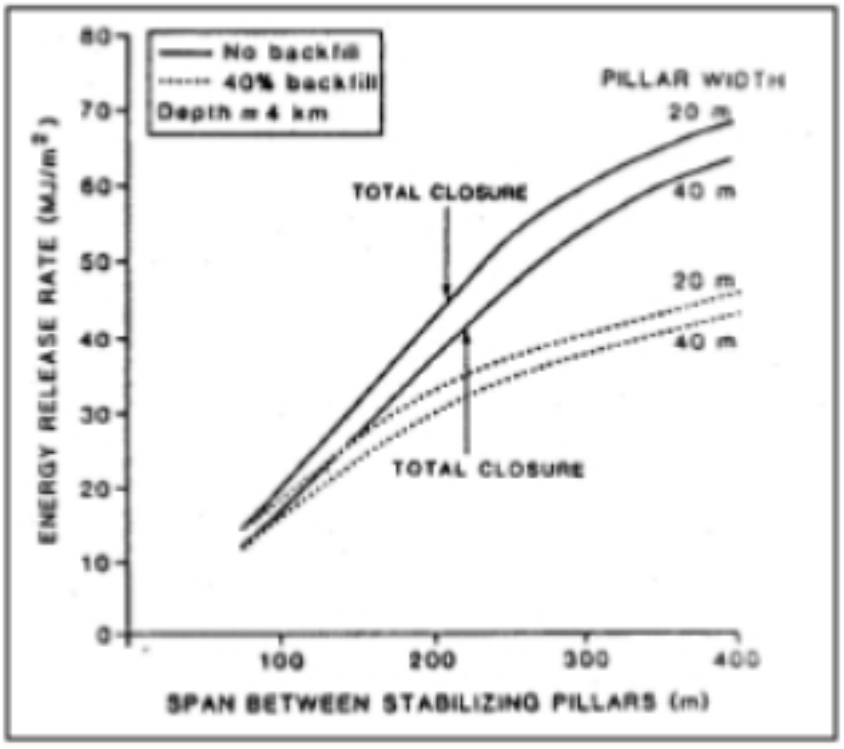


Figure 28: Impact of backfill on ERR and stabilizing pillar spans (Ryder and Jager, 2002)

CONNECTION 98

Refer to Paper 2 Outcome 5.3.1 on page 361, Outcome 5.3.2, Outcome 5.3.3, on page 363, Outcome 5.3.4, on page 371 and Outcome 5.3.8 on page 382.

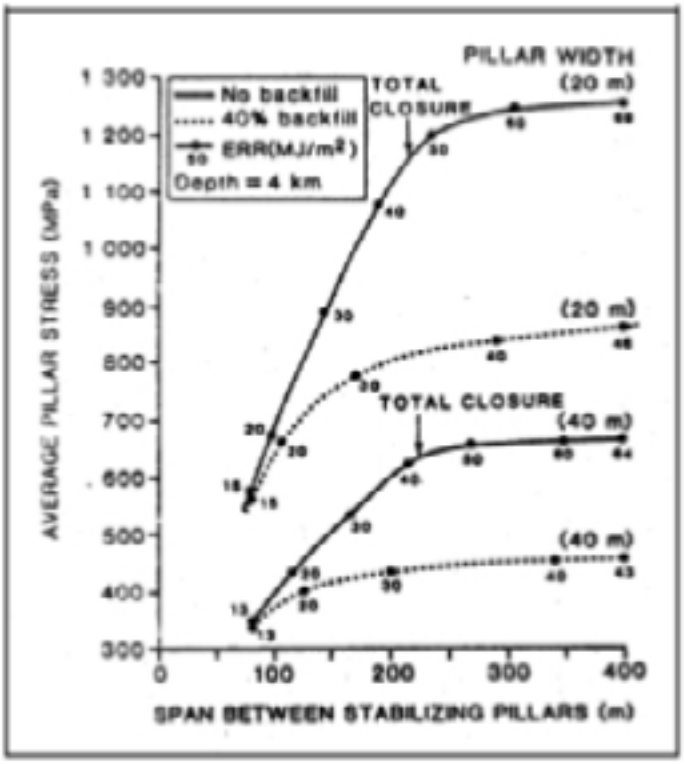


Figure 29: Impact of backfill on average pillar stress (Ryder and Jager, 2002)

LEARNING OUTCOME 4.2.4.3

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL SELECTION CRITERIA FOR CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

The correct selection of backfill type is important if the benefits (goals) are to be realised effectively. The goal for the application of backfill determines the backfill characteristics required and thus also the type of backfill selected.

CONNECTION 99

Refer to LEARNING OUTCOME 4.2.4.2 on page 210

Backfill placement parameters influence backfill effectiveness and include and must be evaluated to determine the required backfill type:

1. Stope width: Higher stoping widths increases the closure that must occur for the backfill to undergo the strain to generate the required stress within the fill. This means that the mining spans must become much larger before the closure occurs and the stress is generated, that the distance from the face where the closure occurs and thus the backfill generates the stress requited increases, negating its local support function.

NOTE

If a strain of 0.4 is required to generate 100MPa in fill, a 1.0m stoping width will require less closure than a stope at a 2.0m stoping width for the strain to induce the 100MPa stress. Remember that strain=(change in dimension)/(original dimension) where the required strain of 0.4 is constant. Therefore:
0.4 = (Closure/ 1.0m stoping width) yield a closure of 0.4m or 40cm, while
0.4 = (Closure / 2.0m stoping width) yield a closure of 0.8m or 80cm.

2. Distance to face of the placed backfill: The further fill is placed from the face, the longer it will take to generate stress as the closure rate decreases with increasing distance from the face.

NOTE

The zone of influence of backfill can be estimated using the following concept (Ryder and Jager, 2002): Zone of influence from backfill edge Zb = b.TanΦ – c, where b is the thickness of the bedding, Φ is the bedding plane contact friction angle and c is the average distance from the edge of the fill paddock to the point of backfill contact with the hangingwall (Figure 30).

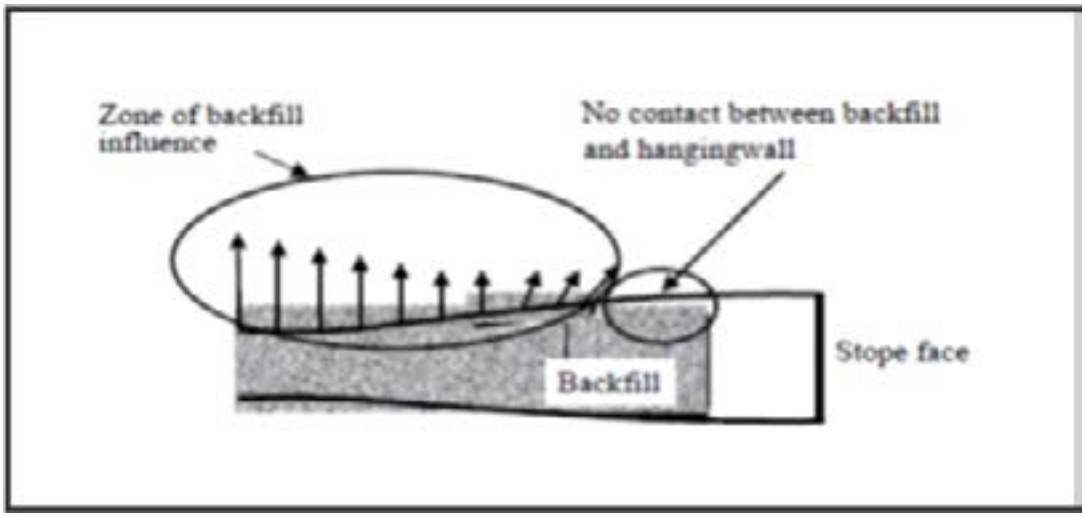


Figure 30: Backfill zone of influence (Ryder and Jager, 2002)

EXAMPLE

If a beddingplane exist 0.8m above the stope hangingwall surface and has a friction angle of 45°, while the fill contact is 40cm behind the paddock edge, the backfill 'zone of influence' is

Zb = 0.8m.Tan45° - 0.4m = 0.4m.

This means that the backfill will only have a support influence 0.4m past the backfill paddock edge.

NOTE

Underground observations have indicated that it is essential to keep the backfill-to-face distance less than 6m if local support benefits is to be realised.

3. Percentage area filled: Research has shown that increases in percentage areas backfill in stopes improves the impact the backfill has on the rock mass. Figure 31 shows that the ERR levels are significantly reduced if backfill is used.

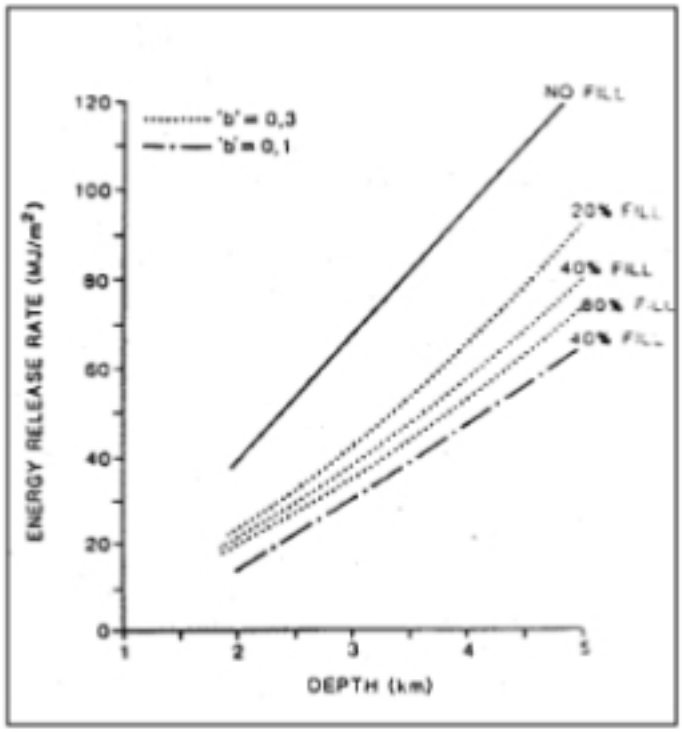


Figure 31: Impact of percentage fill on ERR (Ryder and Jager, 2002)

Backfill material have certain properties that describes different backfill behaviors which should be evaluated to select the required backfill type:

4. Particle size: Particle size distributions provide a description of the percentage of fine and coarse material within the backfill product. This distribution affect backfill stiffness, porosity, permeability (allow water flow), water drainage, etc and has to be optimised. ("LEARNING OUTCOME 4.2.4.4" and Paper 2, Outcome 5.3.3 on page 363)
5. Stiffness: This parameter is a function of the stress / strain behaviour of the fill type and simply suggests the stress generated at a certain strain. The parameters used to explain this relationship are called the 'a' and 'b' values of the fill and were discussed in Paper 2, Outcome 5.3.1. Stiffness (initial) can be increase by the addition of cement (see Paper 2 Outcome 5.3.4 on page 371).
6. Porosity: This a measurement of the voids within a material where higher porosity values indicate more voids within a material and thus a potentially 'softer' material in that more strain is required before the required stress is generated. Lower porosity backfill types are more suited to low closure areas. Thus, a material of low porosity (27 %) is stiffer than a material of high porosity (49 %) at the same strain.

The initial porosity of a backfill material also determines its stiffness and thus the stress or support resistance generated at a given strain level.

INTERESTING INFO

However, from a study of the in situ stress-strain curves for backfill materials of different porosities (Figure 32) it can be seen that over the low range of strains (that usually occur during the first few metres of face advance and the period when local support is required) there is relatively little difference in backfill type performances. In this regard it should be borne in mind that support resistances of about 0.2 MPa (200kN/m2) are commonly considered adequate, even in rockburst-prone stopes. Backfill porosity is, therefore, not normally a critical factor in terms of local support in narrow stope widths, unless significant shrinkage occurs and is not compensated for by the topping up of the backfill paddocks. Shrinkage delays the onset of load generation and therefore affects the backfill support capability and early stabilisation of the hangingwall. It must be noted, however, that porosity influences the effectiveness of backfill as regional support, and this aspect must be considered if the backfill is intended to provide regional support as well as local support.

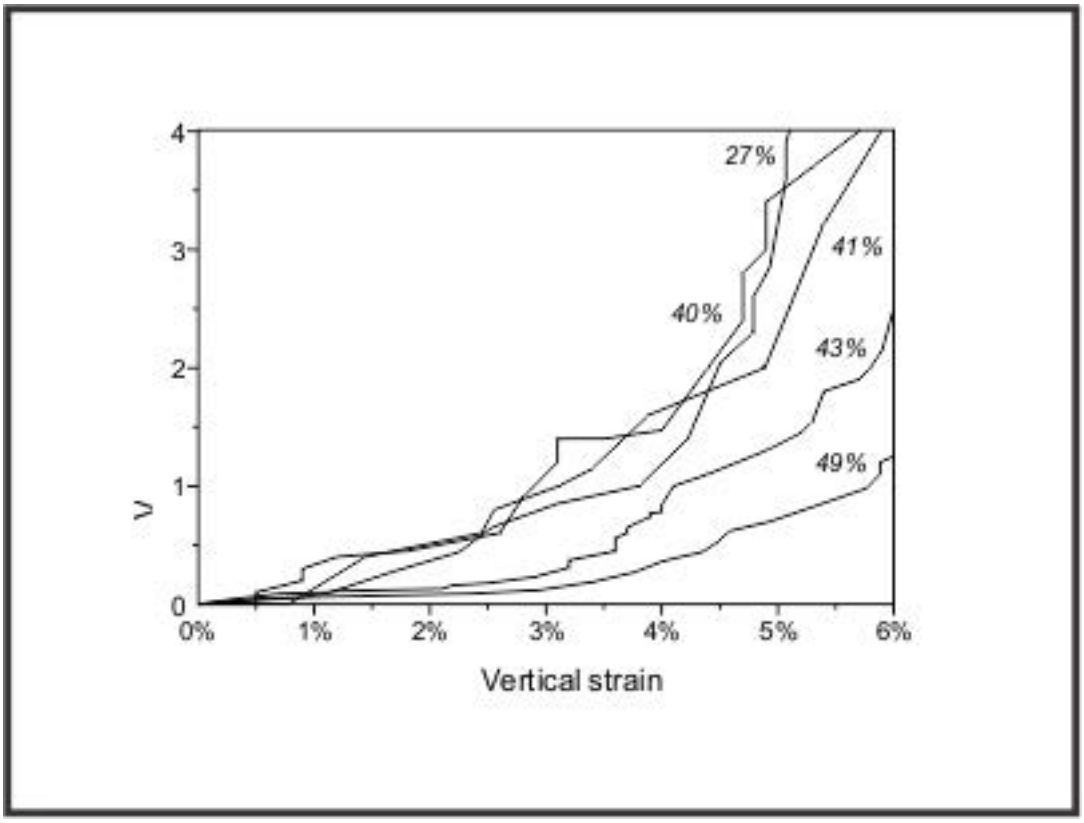


Figure 32: In situ stress-strain curves for backfills of different porosity during the first 2 m face advance after placement in narrow stopes under moderate to high closure rates (Ryder and Jager, 2002)

Shrinkage suggest the process whereby the backfill 'shrinks' away from the hangingwall due to water drainage from the material over time.
In the case of wide stopes (e.g. wider than 2 m) the backfill needs to exhibit minimal shrinkage levels and this can be achieved by the addition of a cementitious binder or the use of paste fill types.

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INTERESTING INFO

The reductions in accident rates and rockburst damage in backfilled panels reported int he industry was achieved when uncemented backfill was placed correctly. This indicates that there is the potential to affect he accident rate in many hard rock tabular mines by the use of uncemented backfill rather than the more expensive cemented backfills.

A flowchart to assist the user in deciding on a potential backfill type is shown in Figure 33 below.

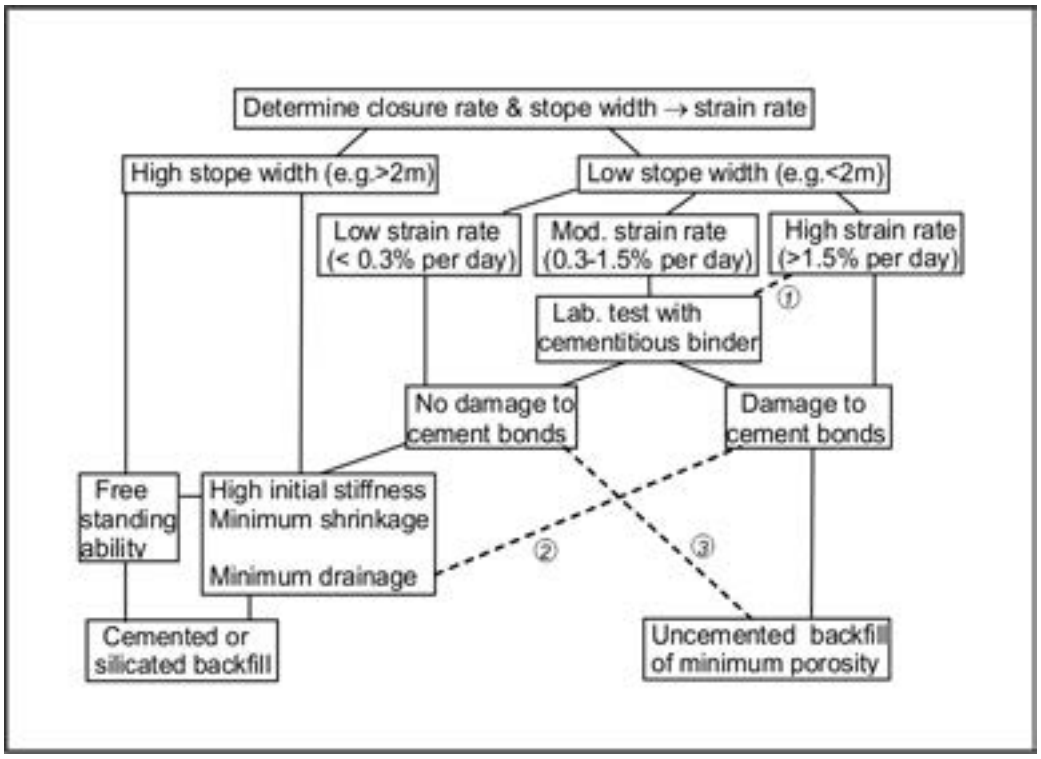


Figure 33: Methodology for the selection of backfill as local support

Although the flow chart is self-explanatory, the following is given to assist:

- 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 should be selected for all high, and whenever possible moderate, strain rate situations, particularly with respect to narrow stope widths at depth where acceptable initial stiffness is obtained regardless of backfill used and where cementitious bonds would be damaged.
- In the case of moderate strain-rate conditions, it is possible to use either cemented or uncemented backfill depending on whether or not damage occurs to the cementitious bonds, which should be established by means of a laboratory test. 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 if the extra expense is justified. However if backfill is required only for ventilation control or for 'dead-weight' support, an uncemented backfill may be sufficient.
- In wide stopes (e.g. wider than 2 m), 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, and free-standing and low shrinkage characteristics are required.

EXAMPLE

A closure rate of 50 mm per metre of face advance, which is approximately 25 mm per day in a mining cycle of 2 days per blast, and a stope width of 1.2 m, would produce a strain rate of 2 % per day. This is categorised as a situation of low stope width and high strain rate, and

back

back

working through the chart in the figure above leads to uncemented backfill as the appropriate choice.

By contrast a closure rate of 20 mm per metre of face advance, which is approximately 10 mm per day in a mining cycle of 2 days per blast, and a stope width of 1.2 m produce a strain rate of 0.8 % per day. This is categorised as a situation of low stope width and moderate strain rate. Depending on the outcome of laboratory tests on the backfill material with a cementitious binder, either a cemented backfill (no damage to cementitious bonds) or an uncemented backfill (damage to cementitious bonds) would be appropriate.

CONNECTION 100

Refer to Paper 2 Outcome 5.3.3 on page 363, and Outcome 5.3.4 on page 371

In a situation of low strain rate (0.2 % per day) resulting from a closure rate of 5 mm per metre in a stope width of 1.2 m, high initial stiffness and minimum shrinkage characteristics are required in order to ensure early load generation in the backfill. From the figure above it can be seen that, under these conditions, cemented backfill is the appropriate choice. In the case of a high stope width of 3 m, the strain rate is not the overriding factor. What is important is that the backfill has free-standing ability so that a collapse of the placed backfill does not occur. Also of possible importance is a high initial stiffness and minimum shrinkage characteristic.

REFERENCE

- Ryder JA & Jager AJ 2002 Rock Mechanics for Tabular Hard Rock Mines SIMRAC Jhb Chapter 6

LEARNING OUTCOME 4.2.4.4

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL PARTICLE SIZE DISTRIBUTION FOR CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

CONNECTION 101

Refer to Paper 2 Outcome 5.3.3 on page 363

The size of particles forming the backfill and their distribution within the total fill mass, has proven to be of importance in design of good fill materials. The particles' sizes have a major influence on certain properties of the fill, notably the percolation rate (permeability), on the size of equipment to transport the fill and in achieving desired densities. Even though many references to fill materials with no -74µm particle sizes (Hypothetical fill material, Figure 34), the following size distribution curve (Figure 34), derived by the USBM (United States Bureau of Mines), shows that typical fill types have approximately 15% of -74 µm material present.

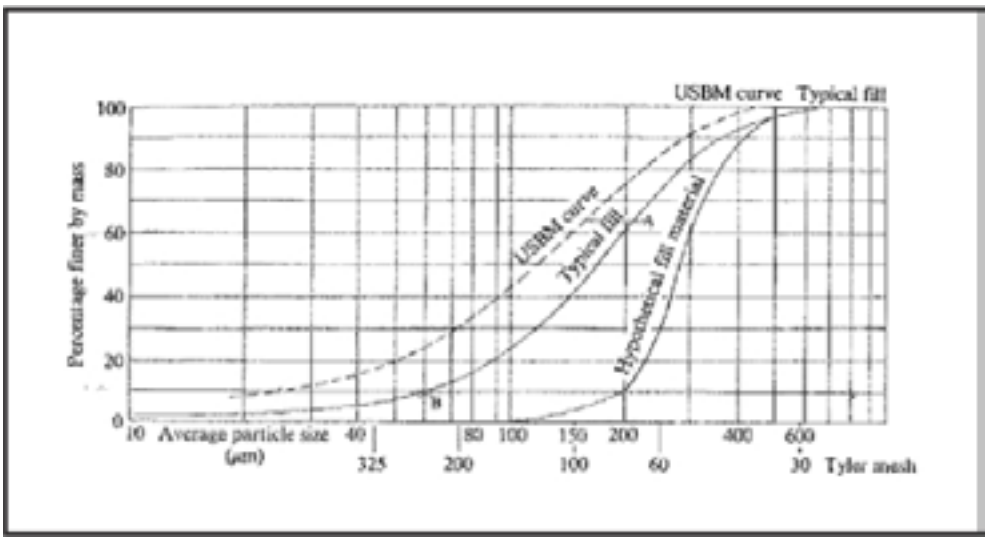


Figure 34: Particle size distribution curves (Corson, 1970)

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back

Size distribution is also important in terms of the equipment that must facilitate the movement of the fill. Generally +20 mm fill particles are difficult to pump conventionally, but some backfill systems can handle fill particle sizes as big as +100mm.

If the material size distribution graph has a very steep slope (Figure 35), the material is said to be 'uniformly graded' (all materials have very similar dimensions). If the graph has a shape as for typical fills (Figure 34), it called 'well graded' (Figure 35).

- *Sand fill consist of 'uniformly graded' materials, where it is often a courser material size rather than a fine size that is selected*
- *Classified or cyclone fill materials are most often created as 'well graded' materials*
- *Paste fill consist of full plant tailings where the particle size distribution is whatever the material consist of that exists the plant. Most often this material, will have a large amount of fines present and graphs similar to the USBM line in Figure 34 (higher % of fine materials) are attained.*
- *Aggregate fill suggests much larger particles and could be represented by a 'uniformly' graded material around a larger particle dimension.*

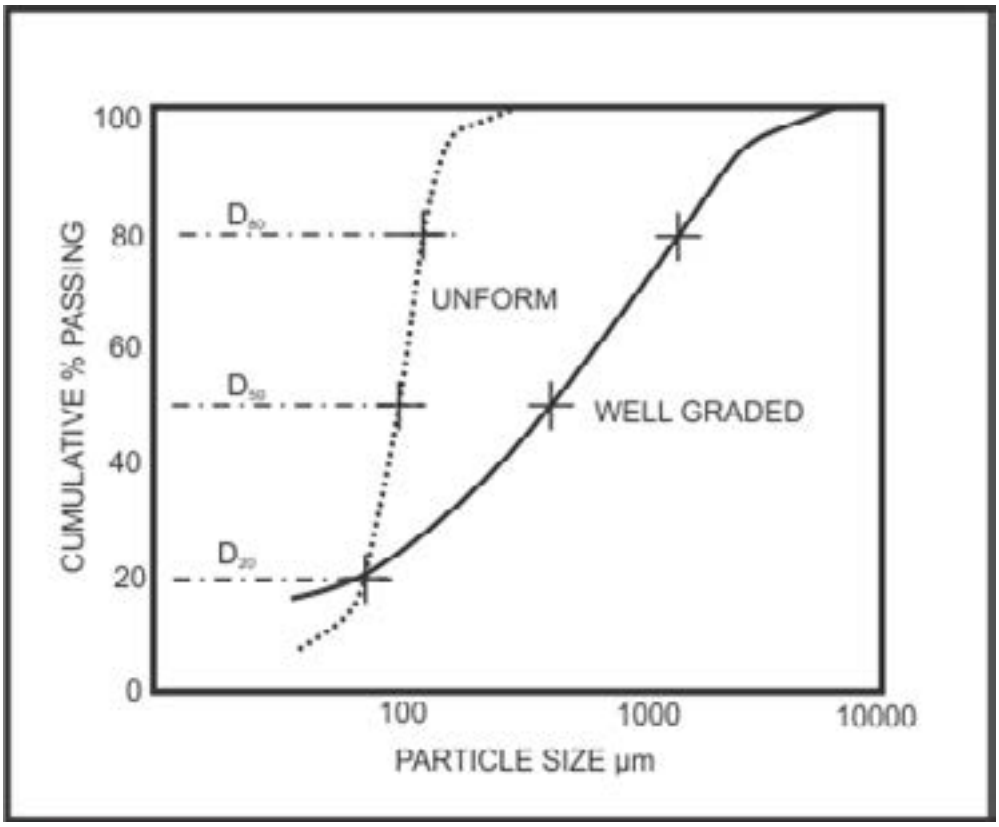


Figure 35:Typical particle size grades

Sand fill:
The highest proportion of the fill product lies in the 40–150 µm range of particle sizes. In soil mechanics terms, this corresponds to a fairly narrow grading of coarse silt and fine sand. Natural sands which have been used or considered for use as mine backfill are generally coarser than artificial sands.

Paste backfill:
Paste backfill is a high density backfill using full plant tailings and has a high fines content, with approximately 15% -20µm particles. Full plant tailings are preferred since well graded tailings are generally easier to pump in paste form than uniform-graded tailings.

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Full plant tailings:
See paste fill.

Classified tailings:
Classified cycloned tailings (CCT) is prepared from concentrator tailings by hydro-cyclone treatment to remove the slimes or clay-sized fraction. In most cases, the highest proportion of the fill product lies in the 40 – 150 µm range of particle size and the proportion of -10 µm material is less than 15%.

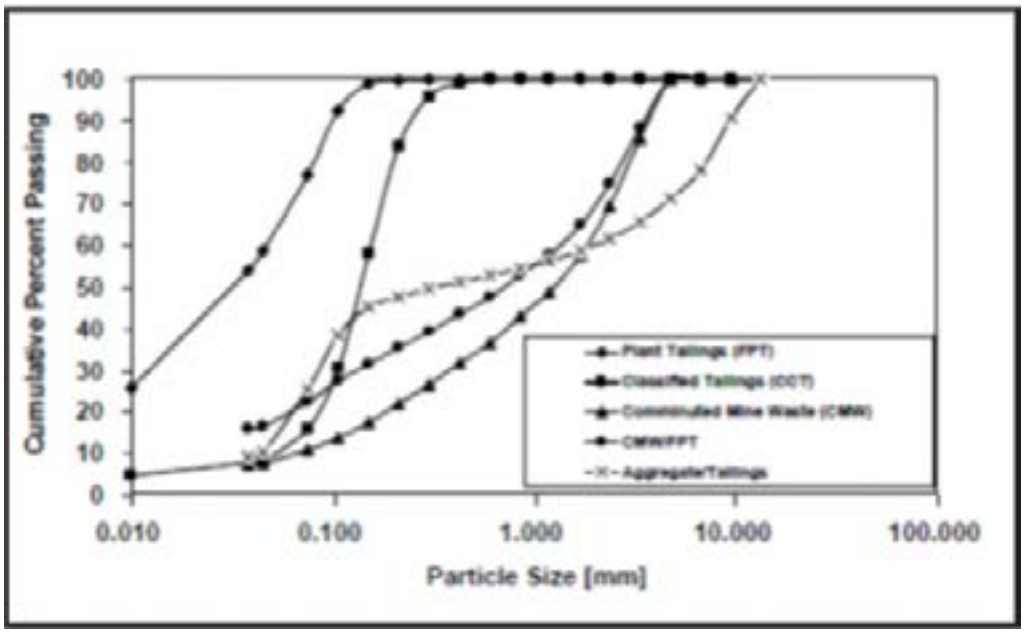


Figure 36: Particle size distributions for various backfill types (Ryder and Jager, 2002)

REFERENCE

- Budavari S (ed) 1986 Rock Mechanics in Mining Practice SAIMM Jhb Chapter11

LEARNING OUTCOME 4.2.4.5

CONNECTION 102

Refer to Paper 2 Outcome 5.3 on page 361

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL STRESS-STRAIN CURVES FOR CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

LEARNING OUTCOME 4.2.4.6

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL SG S FOR CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

Backfill type	Density (tonnes/m3)
Classified	1.75
Full plant	1.85
Cemented full plant	1.85
Aggregate / waste	2.1
Paste	2.0

Table 5: Typical backfill type densities (Ryder and Jager, 2002)

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LEARNING OUTCOME 4.2.4.7

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL DRAINAGE CHARACTERISTICS OF CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

INTERESTING INFO

Backfill is usually placed (gravity being used as the transportation mechanism) as a viscous slurry having a relative density (RD) of 1.7 to 1.8 tonnes/m3. Excess water has to drain from the slurry for it to become a frictional solid, with the solid particles in contact and interlocked. If a binder or cement is used, it hydrates and develops chemical bonds, cementing the interlocked particles and increasing the strength.
The resistance to compression of a fill arises largely from its shear strength as given by
$s = c + (\sigma - u).\tan\Phi$, where s is the shear strength, σ is the normal stress on the plane of shearing and u is the pore-water pressure. The cohesion (c), is the strength that arises from bonds at inter-particle contacts. Except at very low stresses, the term $[(\sigma - u).\tan\Phi]$ is greater than the cohesion (c). The effective stress, (σ-u), takes account of the strength-reducing effect of the pore-water pressure (u). Immediately after placing a fill, u will be at its highest, reducing as the water drains from the fill.



It is important that drainage be completed as soon as possible since it is only in a drained fill that shear resistance will develop.

- Blight cemented backfill

The drainage process can be divided into two phases:

- Phase 1: Excess water leaves the fill by draining from free surfaces, and the particles move closer together and into contact. However, the pore spaces in the fill remain fully saturated with water at this stage
- Phase 2: Once a threshold value of negative pore pressure has been reached, air starts to enter the pores of the fill, displacing more water.

back

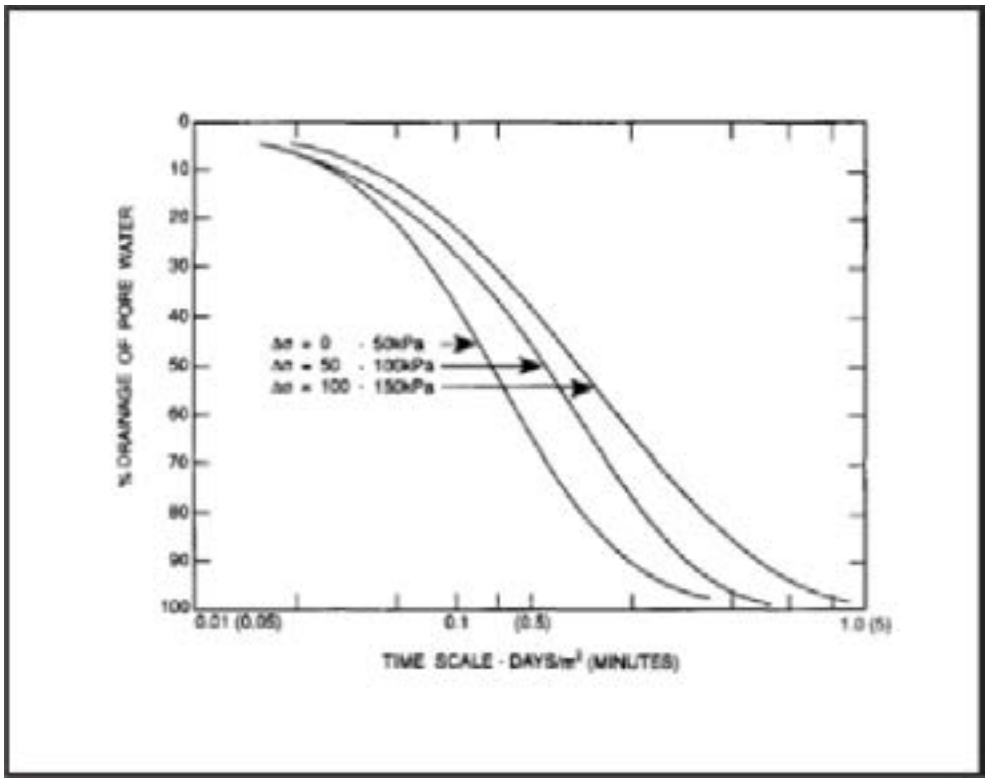


Figure 37: Laboratory drainage curves for a cyclone-underflow backfill

Figure 37 shows drainage-time curves measured in the laboratory for a certain specimen of cyclone backfill, which indicates that drainage occurs slower if the applied stress is higher. Increased stress on the backfill forces the solid particles closer together, reducing the void space in the fill and thus the permeability (ability to allow water flow through the material).

Drainage is thus a function of the porosity and permeability of the fill material where:

- porosity indicates the amount of voids in the fill (potentially filled with water which must be drained) and
- permeability indicates the ability of the material to allow water flow

Backfill type	Porosity at placement (%)	Permeability
Classified	56	Medium / Low
Full plant	51	Low
Aggregate / waste	36	High
Paste fill	< 50%	Very low

Table 6: Typical fill porosities and comparative permeability

- Backfill drainage

REFERENCE

INTERESTING INFO

Some advantages and disadvantages of the different backfill types discussed in this section are listed below in Table 7.

System	Advantages	Disadvantages
Crushed waste fill	Available waste rock on surface can be used, reducing the amount of waste to be handled / hoisted. Immediate equipment transport is possible on the fill	High porosity Placement difficulty Waste source must be significant

System	Advantages	Disadvantages
Classified cycloned tailings (CCT)	Simple (piping) method of transporting large volumes of fill from surface. Good drainage characteristics	Large volumes of excess water have to be handled- Slow initial load generation Plant material that is too fine means that sufficient material must be available as the discard underflow is large
Cemented full plant tailings	Simple (piping) method of transporting large volumes of fill from surface.Increase in stiffness is achieved	High moisture content requires greater quantities of binder.Costs increase Addition o f cement requires a good backfill system and quality control
Paste fill	Small % cement addition can provide a high strength backfill (1.5 to 3.5MPa)No water run-offHigh strength development	High cost system which requires expensive pumps, pipes, filtering and dewatering equipment.Good quality control is necessary.

Table 7: Advantages and disadvantages of fill systems



- G.E Blight and A.J.S Spearing The properties of cemented silicated backfill for use in narrow, hard-rock tabular mines. The Journal of South African Institute of Mining and Metallurgy.

LEARNING OUTCOME 4.2.4.8

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL PUMPING CHARACTERISTICS OF CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

CONNECTION 103

Refer to Outcome LEARNING OUTCOME 4.2.4.1 on page 207

There is a conflict between properties of backfill that allow practical and cost effective transportation of backfill, and those properties that best satisfy the properties of placement and load generation. Backfill displaying slow settlement behaviour, or in other words backfill material that has a lot of fines is normally cheaper to pump, but has significant effects on drainage and also load generation during drainage.

In general, full plant tailings exhibit pressure losses in the pipe during pumping far greater than classified tailings. The main characteristic when pumping backfill is the pressure loss exhibited, as a function of relative density of the backfill. In general, greater relative densities result in greater pressure losses and slower pump velocities (Figure 38).

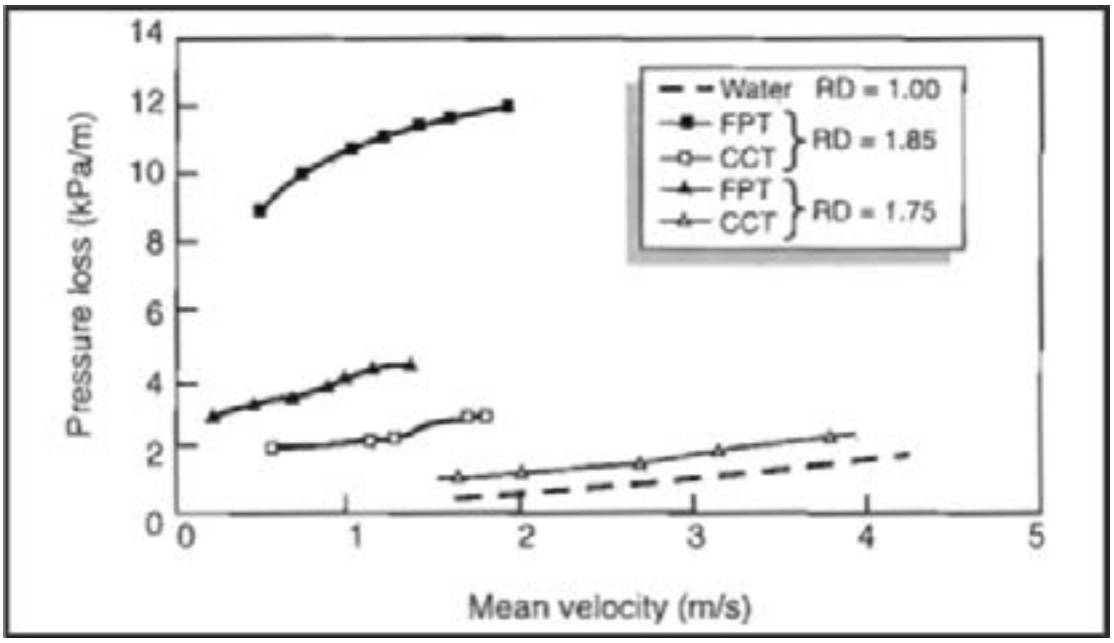


Figure 38: Pressure loss and mean velocity as a function of relative density of, full plant tailings (FPT) and classified tailings (CCT) at different relative densities (Jager and Ryder, 1999)

REFERENCE

- Ryder JA & Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 3

LEARNING OUTCOME 4.2.4.9

CONNECTION 104

Refer to Outcome LEARNING OUTCOME 4.2.4.1 on page 207

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL STOWING CHARACTERISTICS OF CLASSIFIED TAILINGS BACKFILL, FULL PLANT TAILINGS, PASTE FILL, SAND FILL, AGGREGATE FILL

NOTE

Stowing refers to the placement of the backfill.

LEARNING OUTCOME 4.2.4.10

CONNECTION 105

Refer to Paper 2 Outcome 5.3.4 on page 371

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE USE AND EFFECTS OF CEMENTITIOUS BINDERS WITH THE ABOVE TYPES OF FILL

LEARNING OUTCOME 4.2.4.11

CONNECTION 106

Refer to Paper 2 Outcome 5.3.6 on page 378

SPECIFY PERFORMANCE CRITERIA, BACKFILL TYPE AND PLACEMENT METHOD TO ENSURE SUCCESSFUL BACKFILLING ON A MINE.

4.2.5 BACKFILL FOR ENERGY RELEASE RATE CONTROL

LEARNING OUTCOME 4.2.5.1

DESCRIBE, EXPLAIN AND DISCUSS THE ABILITY OF BACKFILL TO REDUCE ENERGY RELEASE RATES

CONNECTION 107

Refer to LEARNING OUTCOME 4.2.4.3 on page 212

Backfill, when strained, generates high levels of stress within the fill, while closure is continuously inhibited. This means that backfill increasingly affects the parameters that determine the ERR levels, namely stress levels ahead of the mining face and closure into the mined out voids. Figure 39 shows that good quality fill manages to keep ERR levels lower than 'standard' quality fill or unfilled conditions.

NOTE

Mf is called the 'figure of merit' and is a backfill quality measurement parameter where a lower value indicates better quality. Mf expresses the ratio of ERR with fill to ERR with no fill and thus is a measure of the impact the fill has on ERR levels.

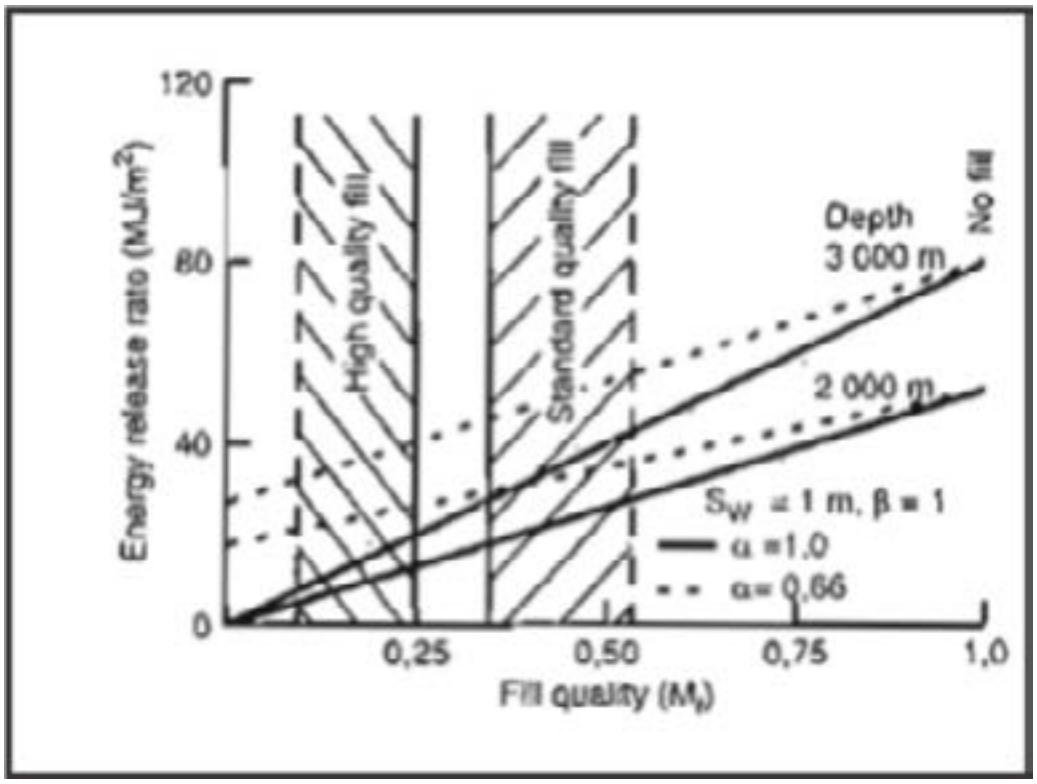


Figure 39: Backfill ability to reduce ERR (Jager and Ryder, 1999)

NOTE

The α and β parameters shown in Figure 39 are explained in Outcome 4.2.5.3.

LEARNING OUTCOME 4.2.5.2

EXPLAIN AND DISCUSS THE EFFECT OF PLACEMENT LAG ON THE EFFECTIVENESS OF BACKFILL AS A MEANS OF REDUCING ENERGY RELEASE RATES

NOTE

Placement lag is defined as the lagging in placement behind the mining face in terms of distance from face.

The greater the stoping width at which the fill is placed, the less effective backfill becomes at initially controlling ERR due to lower load generation

in the fill ("LEARNING OUTCOME 4.2.4.3"). When placing the fill further away from the face, closure had already taken place and stoping widths are lower, suggesting that placement far from the face would be beneficial to load generation and thus ERR reduction.

However, it is critical to note that closure rates decrease further away from the mining face, decreasing the rate at which the backfill is compressed and thus ultimately reduces load generation in the fill. Placement lag behind the mining face will thus provide slower load generation and reduced impact on ERR levels. Backfill should be placed as close to the face as possible to benefit from high closure rates in this area, which leads to high compression rates and thus also high load generation.



Placement of backfill in back areas (far from the mining face) will expose the fill to very low closure rates, minimal compression and low load generation. Since high loads will not be generated, backfill will not affect face ERR levels.



- Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 3

LEARNING OUTCOME 4.2.5.3

EXPLAIN AND DISCUSS WHY IT IS NOT NECESSARY TO COMPLETELY BACKFILL THE MINED AREA TO ENSURE A SIGNIFICANT REDUCTION IN ENERGY RELEASE RATE

Backfill's ERR effectiveness is shown in Figure 40 to be a function of 3 parameters:

- The fraction of stoping width into which the backfill is placed, known as α (which is the ratio of the Filled width to the Stoping width);
- The percentage of area filled within the total stope, known as β and
- Stoping width or Sw.

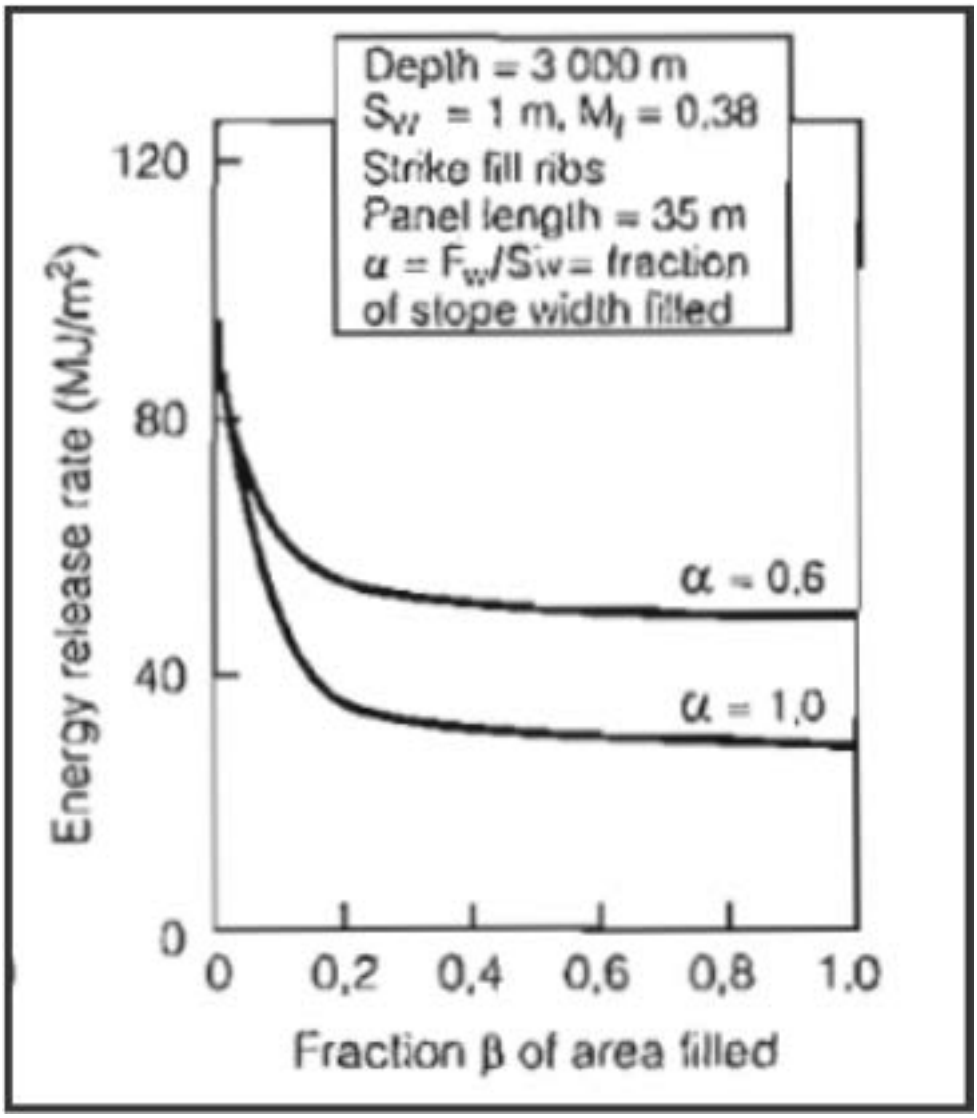


Figure 40: Effects of different backfill parameters on reducing ERR (Jager and Ryder, 1999)

Figure 40 suggests that:

- That maximum impact on ERR is achieved in placing the first 20% area fill, but that as β increases, the impact on ERR diminishes. This suggests that 100% filling (all mining excavations) is not optimum when the aim is only to reduce the ERR. Smaller filled areas may provide similar ERR levels and allow practical filling of panels, leaving gullies and face areas open for access and egress;
- ERR reduction is possible by increasing the backfill placement quality, i.e. increasing α. Based on previous outcomes discussed, a low α value would suggest that the fill is not touching the hangingwall which means that closure must occur before the hangingwall touches the backfill and then only start to compress the fill material. For this reason, compression of the fill and load generation in the fill will be lower than if the fill was placed against the hangingwall (pressure fill) and the impact on ERR will be lower.



- Ryder JA &Jager AJ 1999 Rock Engineering Practice for Tabular Hard Rock Mines SIMRAC Jhb Chapter 3

LEARNING OUTCOME 4.2.5.4

ESTIMATE AND DISCUSS THE EFFECT OF BACKFILL ON ULTIMATE ENERGY RELEASE RATES GIVEN BACKFILL PERFORMANCE PARAMETERS AND STOPE CLOSURE RATES.

ERR is a function of load generation and closure inhibition. The impact of closure rate and fill performance parameters on load generation and closure inhibition can thus be shown to be:

- Stope closure rate: Closure rate determined the rate at which the fill is compressed. Low rates suggest long periods before the fill is compressed to a point where load generation is significant, while high closure rates suggest short periods to reach this load generation point. Since closure rates are either a result of the stress and depth environment or the distance from the face that the fill is placed, it follows that:
- shallow, low stressed environments will have low closure rates and thus slow low generation on fill with small impact on ERR;
- deep, highly stressed environments will have high closure rates, fast compression and load generation and larger impact on ERR;
- installation far from the working face will experience lower closure rates, lower load generation and thus small impact on ERR, whilst
- placement close to the working face will ensure higher closure rates, faster compression and load generation and larger impact on ERR.
- Backfill performance parameters: These parameters include those discussed in all previous outcomes within this section:
- Placement quality: Includes factors such as α , β , distance to face
- Material quality: Includes porosity, permeability, stiffness, fill type, etc.

4.2.6 BACKFILL FOR EXCESS SHEAR STRESS CONTROL

LEARNING OUTCOME 4.2.6.1

DESCRIBE, EXPLAIN AND DISCUSS HOW BACKFILL AFFECTS THE EXCESS SHEAR STRESS ACTING ON GEOLOGICAL FEATURES

Excess shear stress (ESS) relates to the excess amount of shear stress (where shear stress is the driver of shear displacement also referred to as 'slip') present on any particular geological structure. ESS is determined by (Shear stress – Shear strengthstructure), where the shear strength of the structure is a function of the cohesive and frictional properties of the structure, as well as the stress normal to the structure (clamping). The amount of shear stress and normal stress on a structure is a function of the field stress distribution around the structure.



Mohr-Coulomb suggests that the shear strength of a structure can be given by τ = Co + (σn-u).TanΦ, where τ is the shear strength, σn is the normal stress on the plane, Co is the cohesion on the plane, Φ is the friction angle of the plane and u is the pore-water pressure (if present). For backfill to impact on ESS on a structure, it has to affect (reduce) the shear stress on the structure and / or increase the normal stress on the structure, i.e. affect the stress field around the structure.

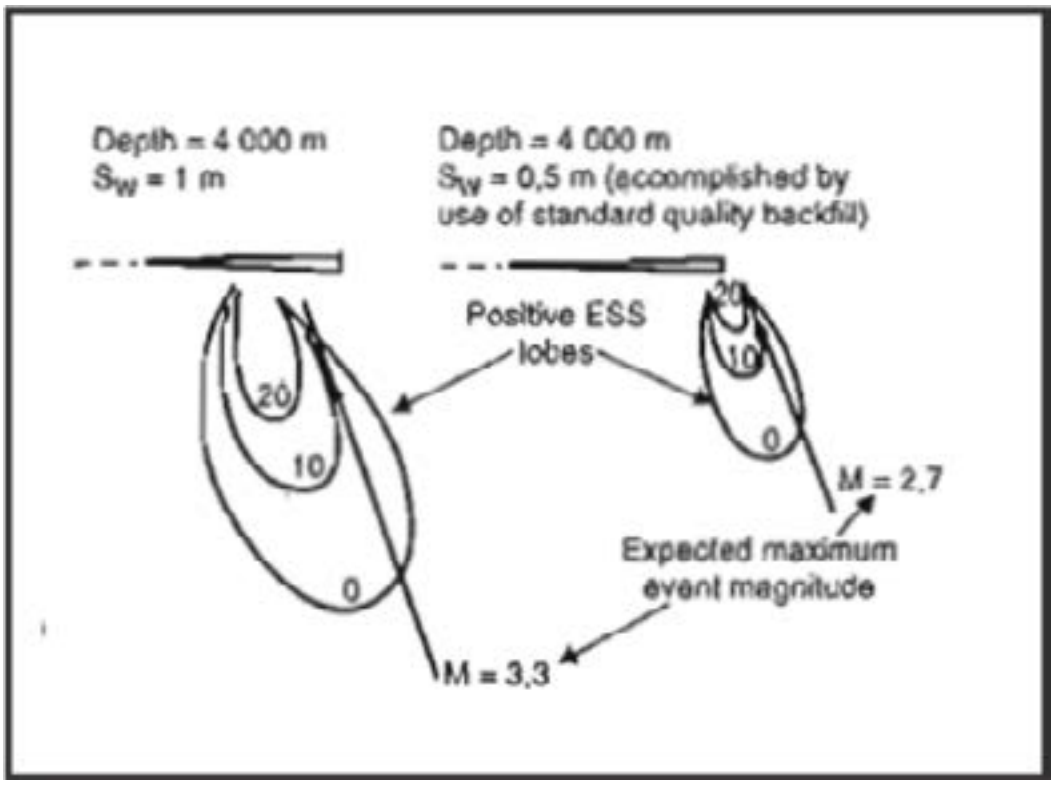


Figure 41: Impact of fill on ESS on geological structures (Jager and Ryder, 1999)

CONNECTION 108

Refer to LEARNING OUTCOME 4.2.3.1 on page 200 to 4.2.3.6

The impact of backfill on ESS lobes ahead of the mining face (and thus potentially along geological structures) when no backfill is placed (Figure 41a) and when backfill placement yielded a final 0.5m stoping width (Figure 41b), reduces the lobe sizes and positive ESS levels and thus also the probable seismic event magnitude.

4.2.7 REGIONAL MINING LAYOUTS

LEARNING OUTCOME 4.2.7.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS MINING LAYOUT STRATEGIES TO ENSURE REGIONAL STABILITY OF WORKINGS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Regional stability is achieved by maintaining control of the overburden (rock between the reef and surface). In this process, the use of pillars to support the overburden and span control to limit overburden displacement, is critical.

Shallow:
The most common mining layouts used in shallow mines are:

- Bord and pillar (see detail in Outcome 4.1.1) where the bords are limited (typically < 10m) and the pillars are non-yield. Due to non-yield pillars, regional pillars are often not included in this design.
- Scattered mining with non-yield pillars and small span (typically < 20m) stopes (See Outcome 4.1.1 and Figure 6). Due to non-yield pillars, regional pillars are often not included in this design.
- Scattered mining with yield pillars and small span (typically < 30m) stopes towards the deeper end of the 'shallow' category

(See Outcome 4.1.1 and Figure 6). Regional pillars are included to limit the size of the tensile zone above the mining area and maintain overburden stability.

Intermediate:
At intermediate depths, mining methods can allow some pillar yield to occur as the rock mass and stress state has changed to an environment where the rock mass is partly supporting itself and non-yield pillars become less critical. Methods include:

- Scattered mining with yield pillars and small span (typically < 30m) stopes (See Outcome "LEARNING OUTCOME 4.1.1" and Figure 6). Regional pillars are included to limit the size of the tensile zone above the mining area and maintain overburden stability.
- Scattered mining with crush pillars and small span (typically < 30m) stopes (See "LEARNING OUTCOME 4.1.1" and Figure 6). Regional pillars are included to limit the size of the tensile zone above the mining area and maintain overburden stability.
- Scattered mining with no pillars and spans typically < 150m and where spans controlled with fault and dyke losses, but also regional pillars (See "LEARNING OUTCOME 4.1.1").
- Sequential grid (scattered mining with dip regional pillars) is also used towards the deeper parts on the 'intermediate' category (See "LEARNING OUTCOME 4.1.1").

Deep:
Deep mining layouts can experience substantial rock mass movement and high stress levels. To limit both these parameters, spans are controlled with wide regional pillars in:

- Sequential grid which is scattered mining with dip regional pillars (See "LEARNING OUTCOME 4.1.1").
- Longwall layouts where strike stabilising pillars are used for regional support (See "LEARNING OUTCOME 4.1.1").

LEARNING OUTCOME 4.2.7.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS STOPING LAYOUTS TO ENSURE REGIONAL STABILITY OF WORKINGS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

CONNECTION 109

See LEARNING OUTCOME 4.1.1 on page 177

LEARNING OUTCOME 4.2.7.3

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS STOPING SEQUENCES TO ENSURE REGIONAL STABILITY OF WORKINGS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Stoping sequences are critical to assist in limiting the risk of mining by controlling local rock mass reactions:

- Shallow layouts:
  - Creation of areas where stress levels can become high enough to fracture the rock, must be prevented;
  - Approach to geological structures such as domes, faults, joints, dykes etc should be optimised so that the creation of potentially unstable wedges / domes are limited;

- Limit potential for seismic activity on geological structures towards the deeper parts of the 'shallow' category;
- Prevent different mining directions in one area (eg updip, downdip, breast, diagonal mining) as higher stressed areas are created, whilst the stable exposure of geological structures becomes difficult to control.

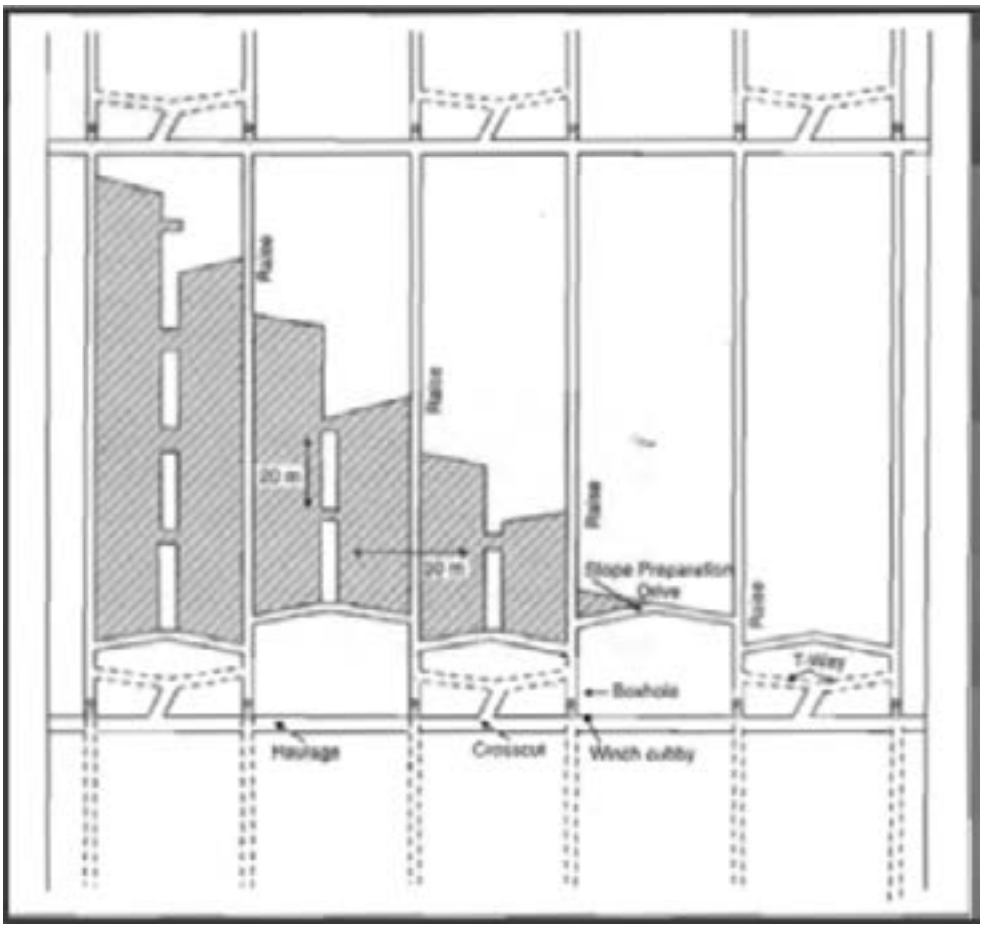


Figure 42:Mining sequence in shallow updip layout (Jager and Ryder, 1999)

- Intermediate layouts:
  - Prevent creation of highly stressed mining faces to limit potential for face bursts and dense fracture patterns;
  - Prevent high stress concentrations on geological structures so that seismic activity can be reduced;
  - Prevent sudden increases in field stress on geological structures by optimised approach angles, to limit the occurrence of seismic activity;
  - Create the most favourable possible stress fracture patterns (density and orientation) to allow support systems to maintain stability;
  - Prevent final extraction at mining face stress levels that are significantly higher than on other faces, as seismic activity and dense (also often poorly orientated) stress fracturing are normally associated with such situations.
- Deep layouts:
  - Prevent creation of highly stressed mining faces to limit potential for face bursts and dense fracture patterns;
  - Prevent high stress concentrations on geological structures so that seismic activity can be reduced;

- Prevent sudden increases in field stress on geological structures by optimised approach angles, to limit the occurrence of seismic activity;
- Create the most favourable possible stress fracture patterns (density and orientation) to allow support systems to maintain stability;
- Prevent final extraction at mining face stress levels that are significantly higher than on other faces, as seismic activity and dense (also often poorly orientated) stress fracturing are normally associated with such situations;
- Prevent mining against highly stressed strike stabilising pillars so that the distance between mining faces and seismic event sources on stabilising pillars are maximised;
- Prevent mining faces to progress into highly stressed dip regional pillars to limit potential for face bursts and dense fracture patterns.

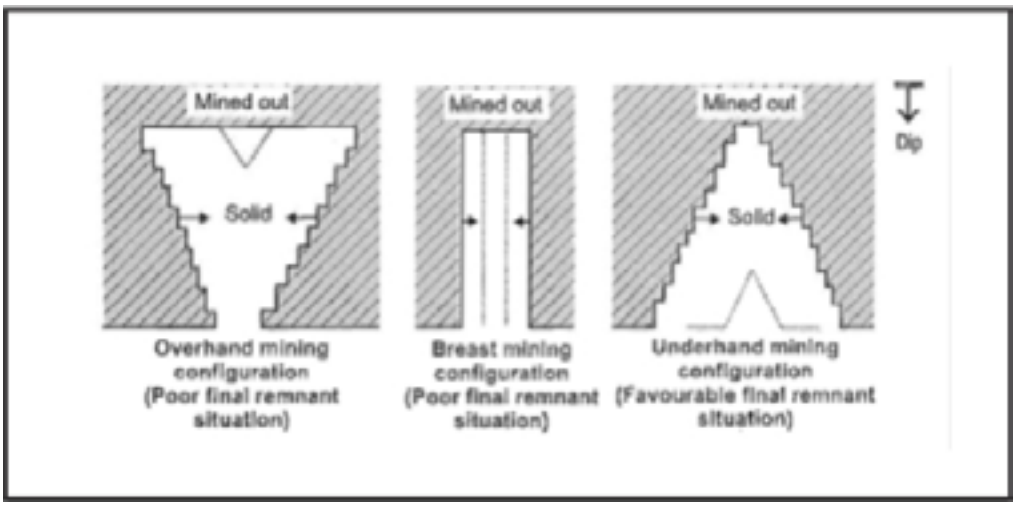


Figure 43: Mining sequences in intermediate scattered to deep sequential grid mining layouts (Jager and Ryder, 1999)

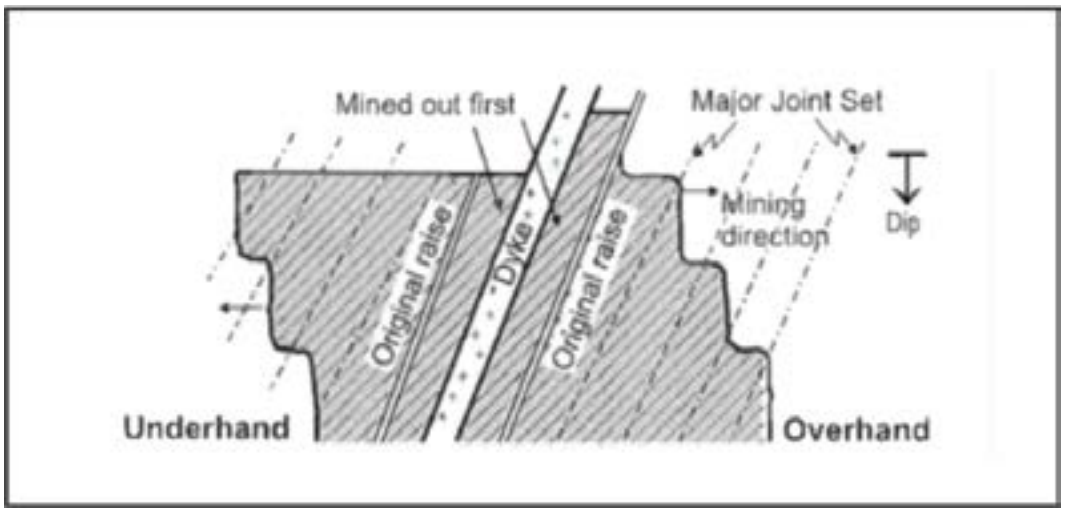


Figure 44: Approach to geological structures (Jager and Ryder, 1999)

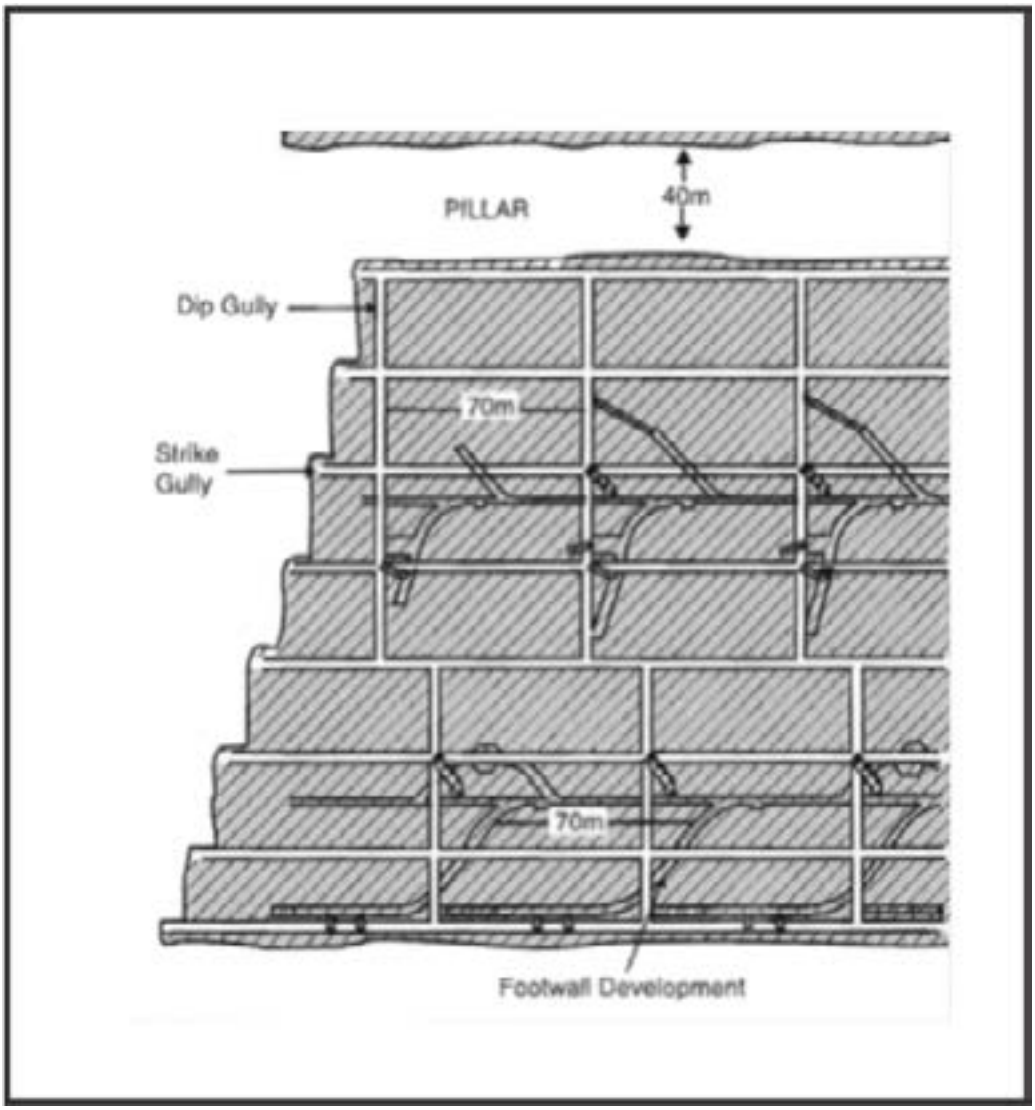


Figure 45: Longwall sequencing (Jager and Ryder, 1999)

For regional stability, it is critical to avoid:

a. The exposure of a geological structure over a large area that could allow significant closure to occur (back break);
b. Highly stressed areas that might generate seismic activity;
c. Pillar failures:
d. Overstressed non-yield or yield pillars in a shallow environment that could lead to :
e. pillar failure and large scale rock mass closure into mined out voids;
f. punching of pillars into footwall or hangingwall resulting in large collapses;
g. Overstressed regional pillars that fail or result in foundation failure, increasing mining spans and face stress levels and give rise to more seismic activity in intermediate and deep environments.



Those practices that are optimum for the control of local rock mass reactions are all also beneficial for regional rock mass stabilisation.

LEARNING OUTCOME 4.2.7.4

DESIGN STOPING LAYOUTS AND SEQUENCES TO ENSURE THE REGIONAL STABILITY OF WORKINGS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH.

CONNECTION 110

The design of stoping lay-outs and sequences merely require the application of "LEARNING OUTCOME 4.1.1" and "4.2.7", while the design of regional support was discussed in "LEARNING OUTCOME 4.2.1.1"



The user is advised to study all appropriate outcomes and combine them into a final design, taking cognisance of all the issues listed and discussed in the outcomes, as well as referenced literature.

4.3 LOCAL STABILITY STRATEGIES

4.3.1 STOPING LAYOUTS AND SEQUENCES

LEARNING OUTCOME 4.3.1.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE LAYOUT OF THE FOLLOWING STOPING METHODS:

- Bord and Pillar mining,
- Scattered mining,
- Longwall mining,
- Sequential Grid mining,
- Indicate the location of access tunnels, service excavations and rockpasses

CONNECTION 111

Refer to LEARNING OUTCOME 4.1.1 on page 177

LEARNING OUTCOME 4.3.1.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS HOW THE FOLLOWING OREBODY GEOMETRIES AND OR COMBINATIONS OF OREBODY GEOMETRIES MAY BE MINED:

- Wide reefs,
- Multiple reefs,
- Steep reefs,
- Flat reefs,
- Shallow reefs,
- Deep reefs,
- Erratically payable reefs

Wide Reefs:
Wide reefs commonly refer to reefs or seams in excess of 2.5m in thickness. Although wide reefs are not common for the platinum reefs, chrome, gold and manganese reefs / seams are often in excess of this thickness. Mining methods that can be utilised to extract wide reefs include bord and pillar, in-stope pillars with small span panels and practices such as multi- or double-cutting.

Bord and pillar operations have been discussed in detail in previous sections (Outcome 4.1.1) and will not be repeated here. However, the practice of using pillars as support medium in wide reef areas as in

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bord-and-pillar is sound and often used. The form of pillars may vary between the normal square or rectangular in-stope pillars with larger regional pillars (to allow smaller in-stope pillars), rib pillars (rectangular but of large length) or 'drift' mining methods.

The design of pillars in large width seams become difficult as the strength of pillars are more difficult to estimate (large number of weaknesses included) and their width:height ratio starts to impact significantly on pillar behaviour. In such cases, the use of a backfill type (Figure 46) to perform the support and also confine the pillars still left intact is a practice often considered.

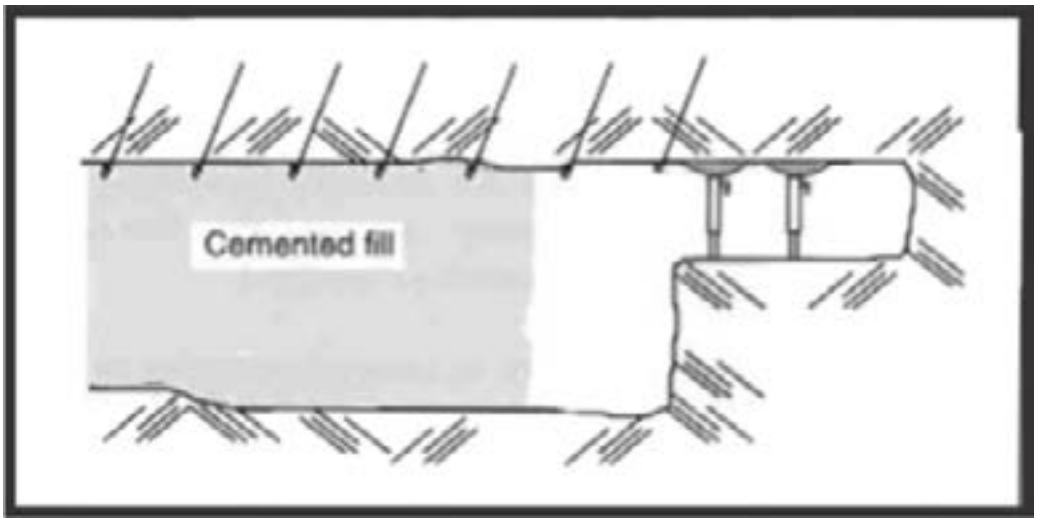


Figure 46: Backfill and double-cut mining in wide reefs

Rib pillars and 'drift' mining suggests that stopes of limited dimension is driven (drift) in a specific direction, leaving a rib pillar along the drift before cutting another drift on the other side of the rib pillar. Figure 47 shows a similar layout for a massive orebody where 'drift-and-fill' mining method was implemented – for a single, wide reef, just of the mining 'layers' shown would be implemented.

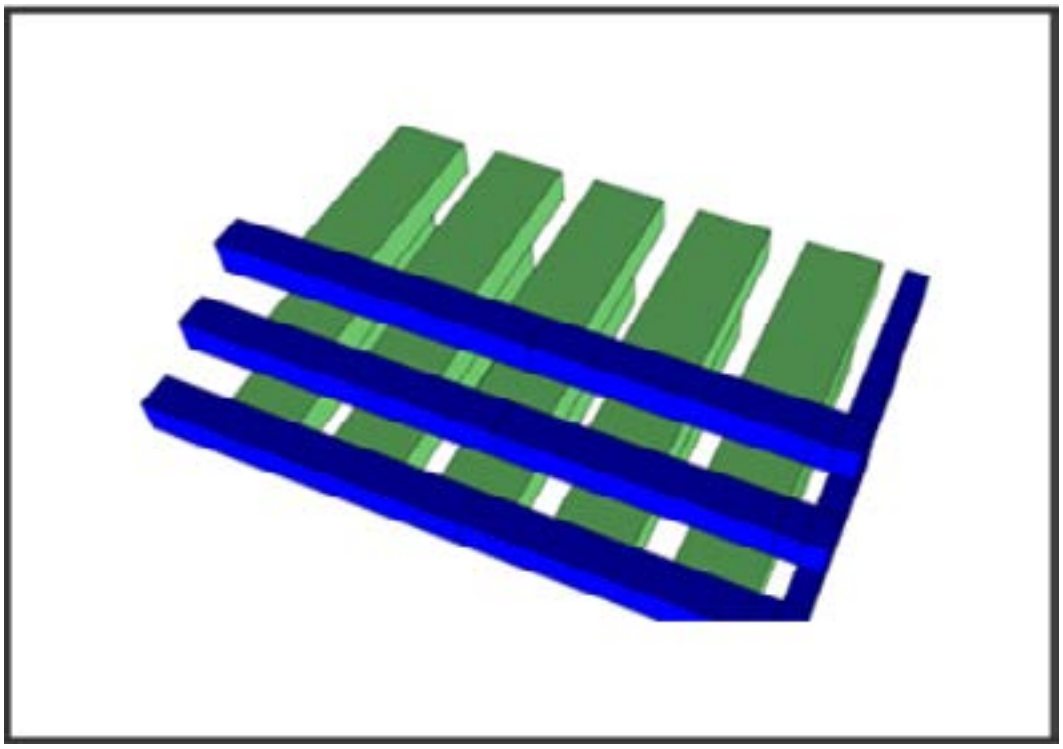


Figure 47: Drift and fill method with rib pillars

back

Double or multi-cut mining has often applied in the gold mines where wide reefs were encountered. It included the mining of a single cut against the reef hangingwall and at a height that suited the standard support systems used on the mine (eg. 1.4m). The rest of the wide reef was extracted some distance from the first cut advancing face. This practice required the removal of support from the ledge either before blasting (when mechanical or hydraulic props were used on the ledge) or during the blast (when timber poles were used). Support behind the ledge included the use of cluster sticks, steel props or backfill (Figure 46).



Cluster sticks are timber mine poles that have been tied to each other and included at least two to three poles within a cluster. The aim is to improve the width:height behaviour of the individual units when combined into a single unit, resulting ig good load bearing characteristics. Take note that the type of poles and the manner of tying the cluster together all affect the behaviour of the cluster as a unit.

Multiple Reefs:

INTERESTING INFO

Multiple reefs are quite common in most of the hard rock, tabular mining environments with the most common being:

Gold:
- VCR and Carbon Leader in Carletonville area
- Main reef and South reef in Central Wits area
- Kimberley reefs (the Kimberley reef consist of several units)
- Basal reef and Leader Reef in Free State
- Vaal Reef and C Reef in Klerksdorp

Platinum:
Merensky reef and UG2 reef

Chrome:
MG reef units (MG reef consist of several units)
LG reef units (LG reef consist of several units)

Manganese:
Upper and Lower seams

The mining of multiple reefs is not an uncommon practice in South Africa. The stress and displacement impact between reefs is what affects the type of mining to be practiced on either or both the reefs and this impact is dependent on mining depth and the middling (distance) between the reefs / seams. Mining methods on the two reefs are totally independent of each other at large middlings, but very dependent on each other in small middlings. At large middlings, any mining method can be implemented on the reefs and they are analysed separately to ensure stability of the individual mining environments. However, at small middlings, they need to be analysed together and the impacts of the one on the other should be quantified.

At small middling between reefs, the impacts could materialise in the following manners:

- Total extraction on one of the reefs could create a tensile vertical (and sometimes even horizontal) stress field around the second reef, affecting the hangingwall behaviour in this reef. This could place the second reef in a low stress, similar to very shallow mining, environment. If the second reef is below the first, the small middling affect the design of spans, pillars and support significantly. Normally, the mining method in the second reef is very similar to shallow mining methods as the first stope is seen as the 'surface' and pillars and small spans must protect the middling to 'surface'. If the second reef is above the first, pillars will have no stable footwall at low middlings and the mining of the top reef becomes improbable, unless it is close enough to allow hangingwall stripping from the lower reef or if the bottom reef can be backfilled.
- If pillars were left intact on the first reef, the second should place pillars directly below the top pillars (superimpose) at small middlings. If this is not done, loading of the top pillars could shear through the middling and cause failure of the middling. If the first reef was mined on the scattered mining method, remnants were probably left in poor condition or low grade areas. These remnants could generate high stresses even in shallow mining environments and create 'deep' level conditions for the mining of the second reef in the vicinity of the remnants.



The decision on the mining method for a particular multi-reef environment thus depends on the middling, depth, mining method implemented on the first reef, etc.

Mid-stope pillars refer to pillars left in the centre of the panel and not along the gully as is the norm.

back

INTERESTING INFO

The following cases (from the Industry) serve as examples of how to mine multiple reefs:

- Multiple reefs (several closely spaced reefs) in a deep level mine are extracted by performing a 'de-stress' cut on the upper of the reefs and the implementation of a massive mining method, drift-and-fill (usually used in shallow environments) in the now 'low field stress' environment to extract the rest of the reefs, spaced very closely, as one unit.
- Multiple reefs in a shallow depth mining environment utilised scattered mining with in-stope pillars on both reefs, but increased the support resistance on in-stope support, or in some cases left 'mid-stope' pillars, to protect the middling between the reefs.
- Multiple reefs in an intermediate depth environment both implemented scattered mining layouts and only took cognisance of each other when remnants had been created on the reef mined first.

Steep Reefs:

The methods for the extraction of steeply dipping orebodies include:

- Scatterred mining: This method allows the pre-development of reefs independent of the reef dip and applies conventional methods to drill and blast, but also to support the stopes. It can be done using an 'underhand' face configuration (see Figure 43) to reduce the dip along the face and allow work on an 'apparent' dip. Support is installed using pre-stressed units to prevent sliding of support due to gravity on installation.

Scattered mining of steep dipping reefs could present significant problems, some of which are:

1. *At certain reef dips, the closure ride component (differential movement between the hangingwall and footwall) could become significant and could render normal scattered mining support systems useless. When mining on elongates, they could fall out or refuse to yield as designed (closure direction / loading is not along the axis of the elongate) and packs could experience shear within the units (closure direction / loading is not perpendicular to the pack axis), both resulting in poor behaviour of the support units.*
2. *At very steep (near vertical) reef dips, closure occurs in the horizontal or semi-horizontal directions and is normally much lower than what would be expected on flat dipping reefs at the same depth. Since support will not be compressed significantly, the use of pack support systems should be considered very carefully.*
3. *Safe personnel access and travelling is a concern and must be considered carefully.*
4. *'Massive' mining methods: Open stoping, cut and fill, drift and fill (and shrinkage) are all mining methods that can be implemented when orebodies are tabular and steeply dipping and are more suitable to very steep dipping reefs, than other methods. These methods are briefly touched on below but will be discussed in more detail in the material for the Paper 3.4 COM Certificate.*

INTERESTING INFO

Massive mining methods are often very suitable to extract steeply dipping, wide and even multiple orebodies. The information below is provided purely for interest sake and includes shrinkage only due to its known application within South African hard rock tabular mines.

Shrinkage: In this method, the ore is excavated in horizontal slices, starting at the bottom of the stope and advancing upward. Part of the broken ore is left in the mined-out stope, serving as a working platform, while mining the ore above and supporting the stope walls (Figure 48). The blasted ore increases in volume by about 70%, allowing 30% to 40% of the ore to be drawn from the stope to keep a suitable working distance between the back (roof) and the top of the broken ore. When the stoping has advanced to the upper limit of the planned stope, drilling and blasting practices are discontinued and the remaining broken ore can be drawn from the stope, leaving the final stope opening open. Shrinkage stoping can be used in ore bodies with the following characteristics:

- Orebody with a steep dip that exceeds the angle of repose of the blasted material substantially;
- The orebody must be firm and relatively competent as it forms the roof to the working area;
- The hanging wall and footwall must be stable as it should remain in place when drawing off the final ore;

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- The orebody must have regular boundaries to simplify ore flow and
- The ore must not be affected by storage in the stope (i.e. must not compact as it will not flow when drawn).

The drawbacks of the system are that it is a labour intensive method with difficult and sometimes dangerous working conditions with limited production capacity. The bulk of the ore remains tied up in the mine for a long period of time. Under similar conditions, sub-level stoping, sub-level caving and cut-and-fill mining are usually more productive, safer and result in considerable economic advantages. The system remains however, suitable for small-scale, steeply dipping mining undertaken with minimum capital investment.

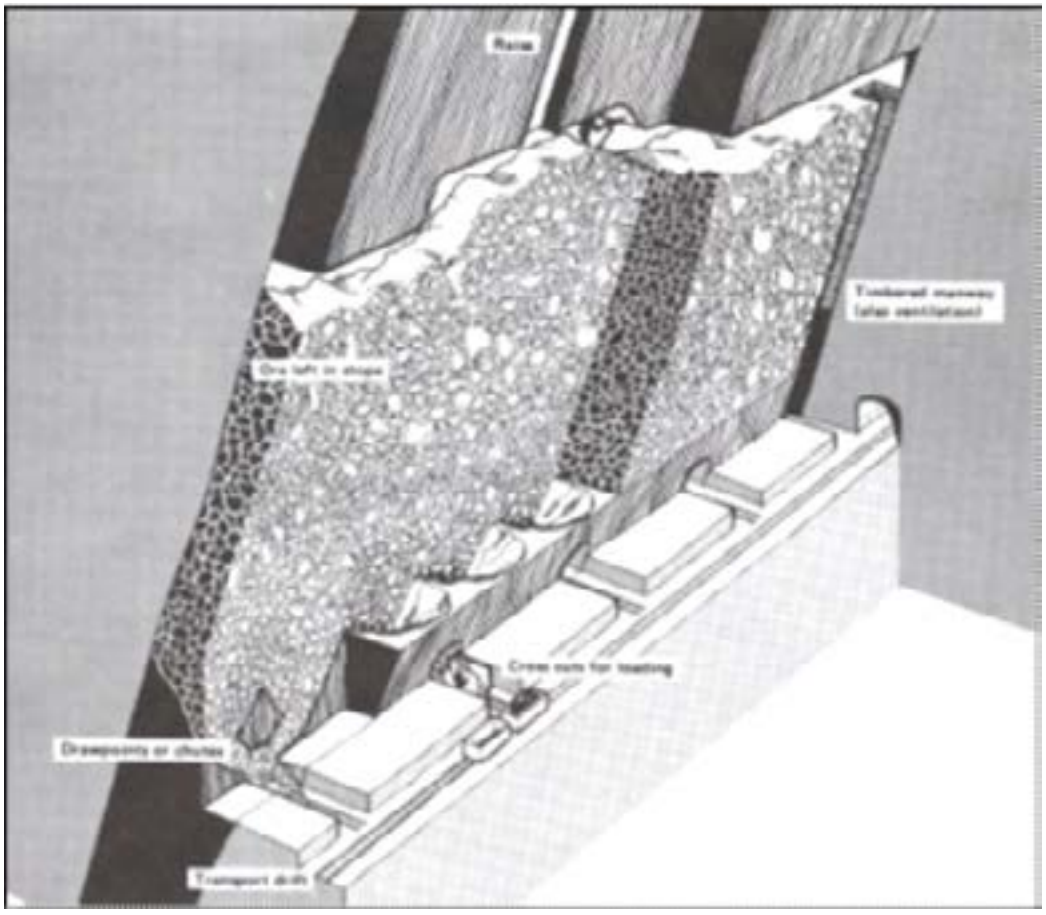


Figure 48: Shrinkage stoping in a large vertical body (Brady and Brown, 2004)

Flat dipping reefs:
This is the norm in most cases in the hard rock, tabular orebodies.
Shallow Reefs:

Deep Reefs:

Erratically Payable Reefs:
These reefs must be mined using mining methods where mining can be very selective without compromising the stability of access or stopes and may include:

- Very selective:
- Scattered (includes all variations of this method): The method allows the mining of any pay zone simply by developing it from the access tunnels and mining it in the best possible sequence.
- Bord and pillar: By developing drives past unpay areas, only pay

CONNECTION 112

For flat dipping reefs, shallow reefs and deep reefs, refer to LEARNING OUTCOME 4.1.1 on page 177 and LEARNING OUTCOME 4.2.1.1 on page 187

areas could be extracted while unpay areas are left solid.

- Selective under certain conditions:
- Sequential Grid: This method is selective if unpay areas can be left intact without increasing the risk to experience seismic activity on the 'pillar' left intact or stress damage to access tunnels.

LEARNING OUTCOME 4.3.1.3

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS HOW A LONGWALL MAY BE RE-ESTABLISHED BEYOND A LARGE GEOLOGICAL DISPLACEMENT

There are some common methods for re-establishing a long wall section beyond a fault / dyke that has displaced the reef or prevented the continuous mining of the longwall:

1. Up-throw fault (throws up on side of fault that must still be mined):

Select the area from where the reef on the other side will be accessed (normally it is the bottom panels within the longwall). Determine the required 'waste cut' dimensions to allow establishment in a lower stress environment and mine the panels the required distance through the structure to ensure de-stressing of on-reef development on the other side of the fault. Develop gullies on the lower side of the fault, through the fault and establish access to and from the up-thrown reef using travellingways and boxholes. Re-establish the longwall by developing a raise (wide raise) along the upthrow side of the fault from the access created above the waste cut (Figure 49).



- *Create other accesses from the upper panels of the longwall to ensure second escape routes.*
- *Do not allow the wide raise to develop along the fault plane until the bottom panels have stripped out against the fault, to reduce the risk of causing stress or seismic related damage to the wide raise whilst it is the only access to the upper reef.*

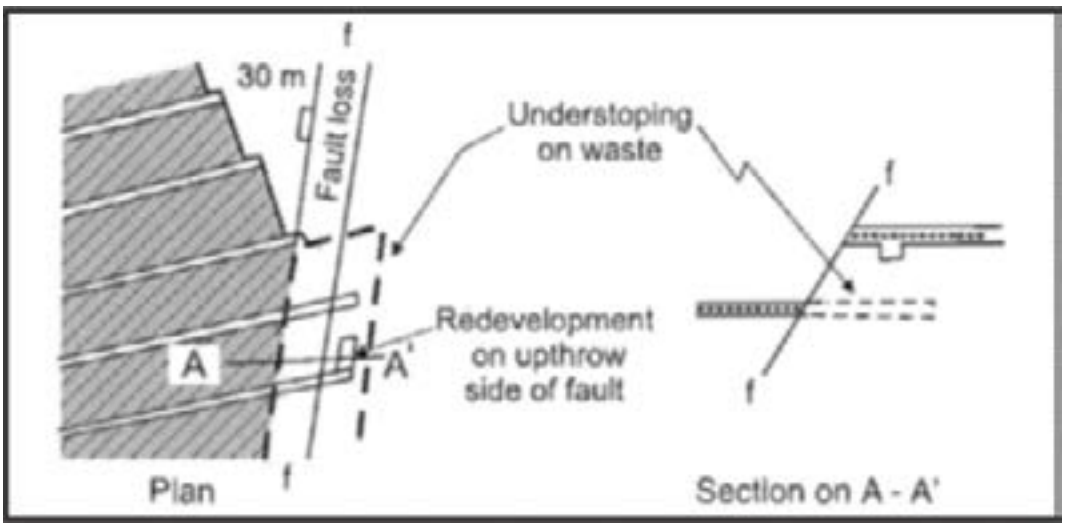


Figure 49: Re-establishment beyond an upthrow fault (Jager and Ryder, 1999)

2. Down-throw fault (throws down on side of fault that must still be mined): Access must be established from the footwall tunnels but can also utilise the de-stressing practice of waste stoping to ensure travellingways are developed in low stress environments.

3. Footwall crosscuts can also be developed from haulages deep in the reef footwall so that the high abutment field stress will not be detrimental to the stability of the access. From these tunnels, the reef is then re-established using travellingways, boxholes and on-reef raises placed well ahead of the structure (Figure 50). Mining is first conducted from the new raise back towards the previous mining (depends on stress levels and seismic risk) and then only towards the solid ground.

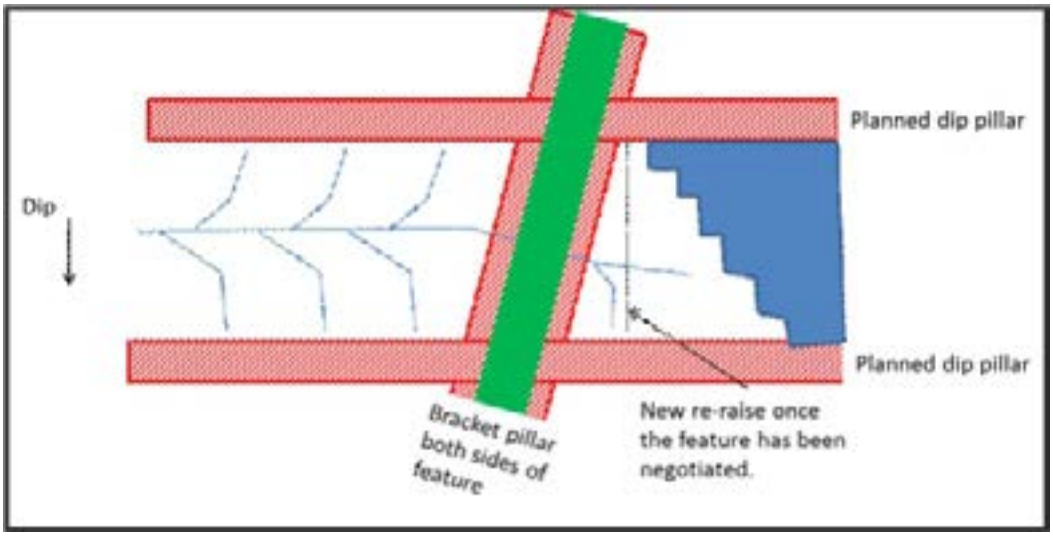


Figure 50:Re-establishing beyond a dyke in a Longwall section



If a collapse in a single panel occurs and re-establishment is required, the gully (using wide end gullies) may be driven ahead of the face and a new dip gully (wide end) may be established for the panel to remain in sequence with the rest of the panels in the longwall (Figure 51). This method could be implemented 'open' (leaving no pillar between raise and old panel) or 're-raising' (leaving a pillar between the re-raise and old panel)

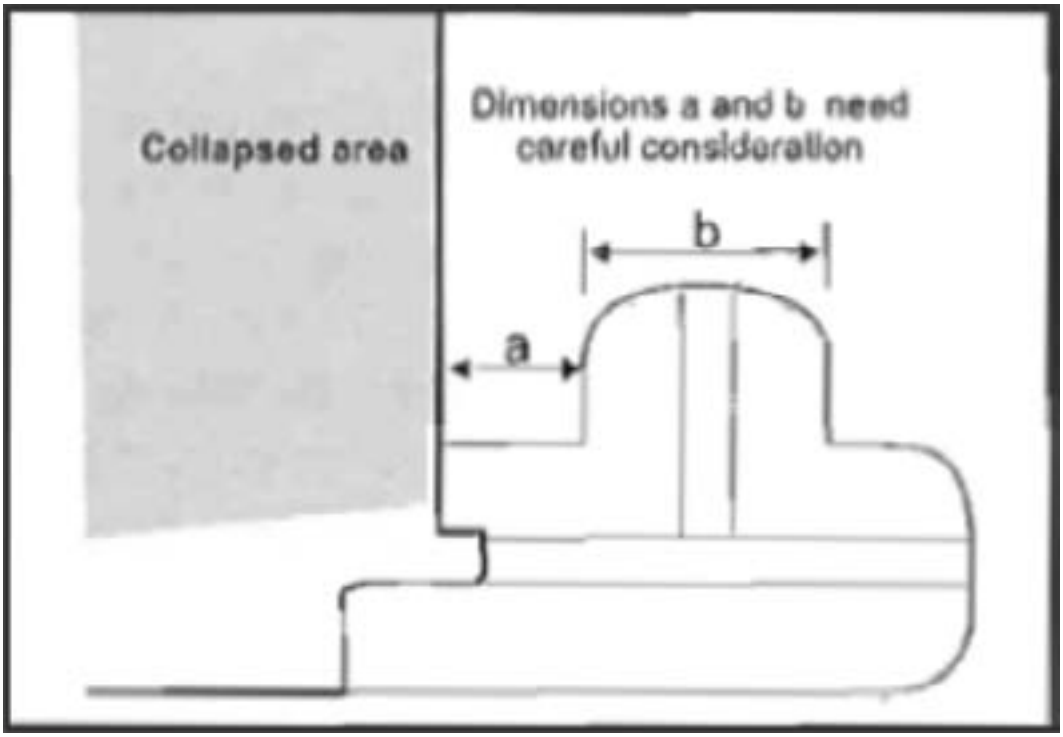


Figure 51:Re-establishment after an FOG in a panel (Jager and Ryder, 1999)

LEARNING OUTCOME 4.3.1.4

INDICATE APPROPRIATE DEVELOPMENT AND STOPING SEQUENCES FOR RE-ESTABLISHING A LONGWALL BEYOND A LARGE GEOLOGICAL DISPLACEMENT

CONNECTION 113

Refer to LEARNING OUTCOME 4.3.1.3 on page 238

LEARNING OUTCOME 4.3.1.5

ESTIMATE RELATIVE MINING RATES FOR THE VARIOUS RE-ESTABLISHMENT METHODS

Actual mining rates is dependent on several factors which may include ventilation, labour compliments, distance from shaft, shaft time available to provide equipment and personnel, etc. However, all of these factors being equal, the different re-establishment methods are comparatively rated below in :

Size of area to be re-established	Method	Condition	Reason	Comparative rate of establishment rating (fastest = 1)
Whole longwall	Waste cut from reef	Upthrow fault	Waste cut and re-development of gullies and wide end	3
	Waste cut and footwall tunnels	Downthrow fault	Waste cut, re-development of footwall tunnels and wide end	4
	Deep footwall tunnels only		Re-development of deep footwall tunnels, access to reef and raise, ledging (or wide raise), mining back towards structure.	5
Panel only	Re-raise		Develop wide end forward to allow the correct pillar size on panel, wide end to hole in top panel.	2
	Open raise		Immediate raising ahead of the panel face using wide end.	1

Table 8: Comparative re-establishing rates

4.4 SERVICE EXCAVATION LAYOUTS

4.4.1 TUNNEL LAYOUTS

INTRODUCTION

The stability of tunnels (and other excavations) is controlled by rock mass condition, field stress levels and changes, and the support system installed. Field stress levels are kept at tolerable levels through good regional support design and the positioning of the excavations in relation to high stress abutments (including overstoping or de-stressing of the excavations). A number of methodologies are applied during the design phase to ensure appropriate layout, size, shape and placement of tunnels.

The design of excavations in rock, especially service excavations such as access tunnels, is a standard engineering function, and requires a systematic approach similar to any engineering design project. The process commences with an initial tunnel configuration which satisfies its operational or service duty requirements, such as minimum dimensions required for operating equipment, or airway capacity required as well as initial positioning and orientation.

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The life expectancy of such service excavations will play an important part in the design, excavation and support of the excavation.
The framework below is a typical design process flow chart which will assist in following a structured method of analyzing the design of an excavation:

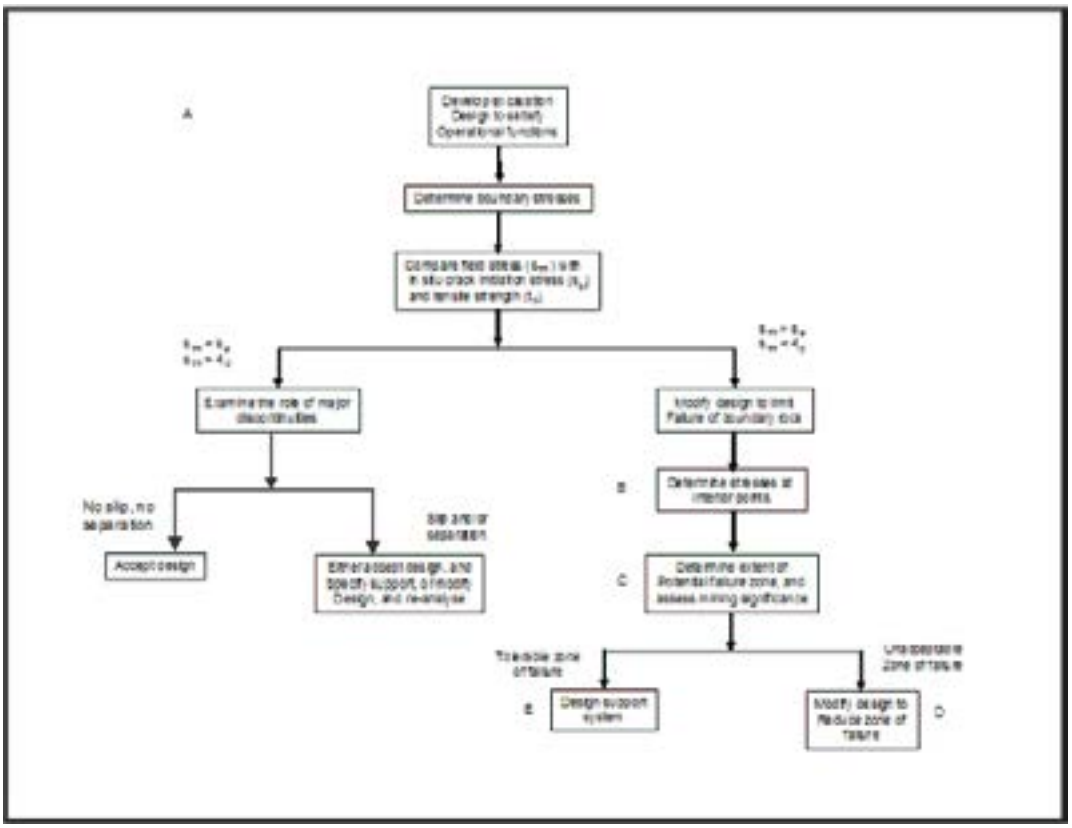


Figure 52: A logical framework for mine excavation design in massive rock (Brady and Brown, 2005) after Brady & Brown) p200

LEARNING OUTCOME 4.4.1.1

CONNECTION 114

Refer to Outcomes LEARNING OUTCOME 3.1.15 on page 85 ; LEARNING OUTCOME 3.1.17 on page 87 and LEARNING OUTCOME 3.1.20 on page 93
Refer to LEARNING OUTCOME 3.2.3 on page 94 through 3.2.5
Refer toLEARNING OUTCOME 3.2.9 on page 99 through 3.2.14

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM SIZE, SHAPE AND ORIENTATION OF SERVICE TUNNELS

Rock wall condition factor (RCF)
One of the criteria used in tunnel design, is rockwall condition factor (RCF). This is calculated using the values of principle stresses which are working on the rock mass at the point where the tunnel is to be placed:

$$RFC = (3\sigma_1-\sigma_3)/F\sigma_c$$

where σ_1 and σ_2 are the major and minor field stress components acting on the tunnel, σ_c is the uniaxial compressive strength of the host rock, and F is an empirical rock mass condition factor.

- *In normal competent rock, the factor F is assumed to be unity.*
- *In highly incompetent rock (in which there are both joints and weak bedding planes spaced less than 0,1m apart), an appropriate value of F is approximately 0,5.*
- *Excavation size (6m x 6m as against a normal access tunnel of 3m x 3m, the value of F is further downgraded by approximately 20% to compensate for the effect of the increase in span on the strength of the rock.*

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- *Tables and graphs of a range of values for RCF are available which can be used to determine stability of an excavation relative to RCF. These values have been correlated to ground conditions experienced in different situations underground. It must be emphasized that the RCF criteria assumes circular tunnel outline and caters for weak or incompetent host rock conditions.*

The application of the RCF criterion allows the analyses of stress fields and rock conditions to determine the best possible size (through the use of the factor F) and orientation (using the stress fields) of tunnels. Relationships such as those shown in Figure 53 and Figure 54 can also assist in making decisions on tunnel positioning.

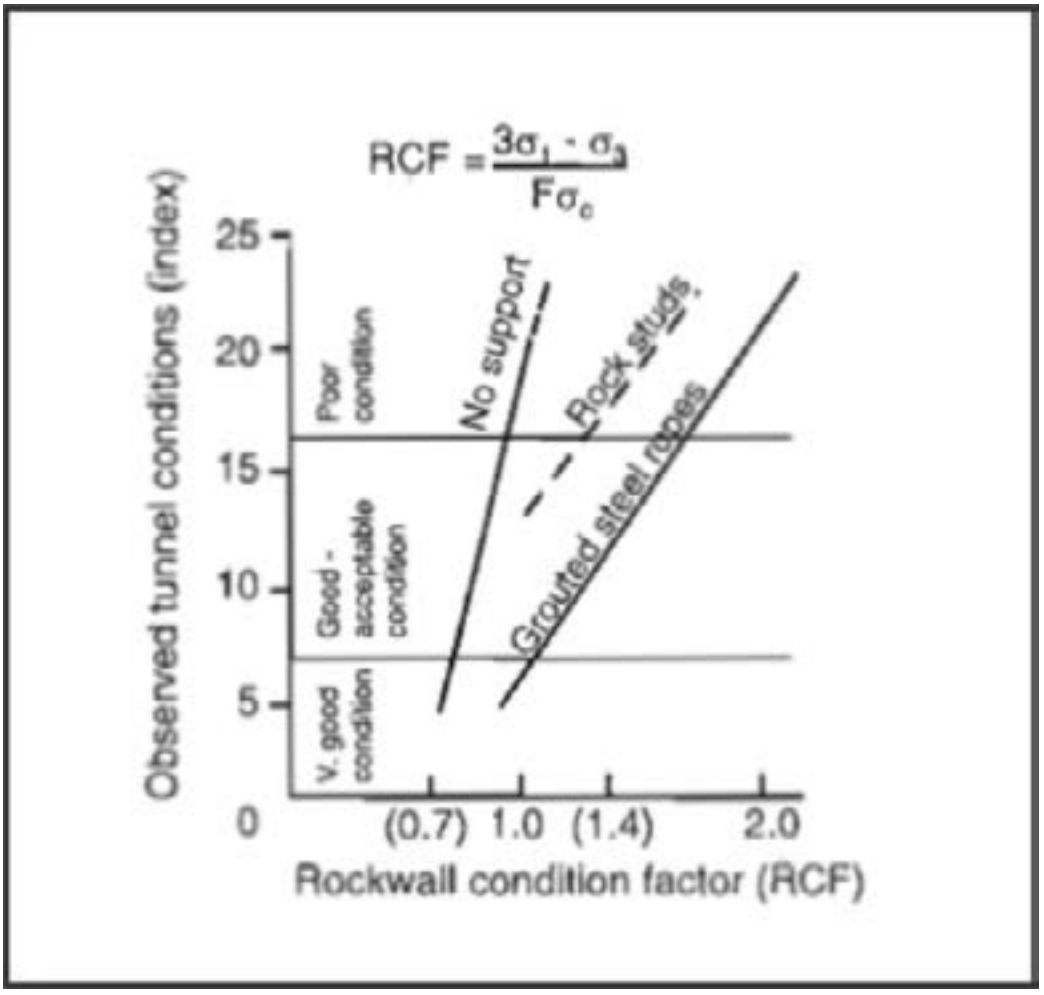


Figure 53: RCF related to expected ground conditions and support (Jager and Ryder, 1999)

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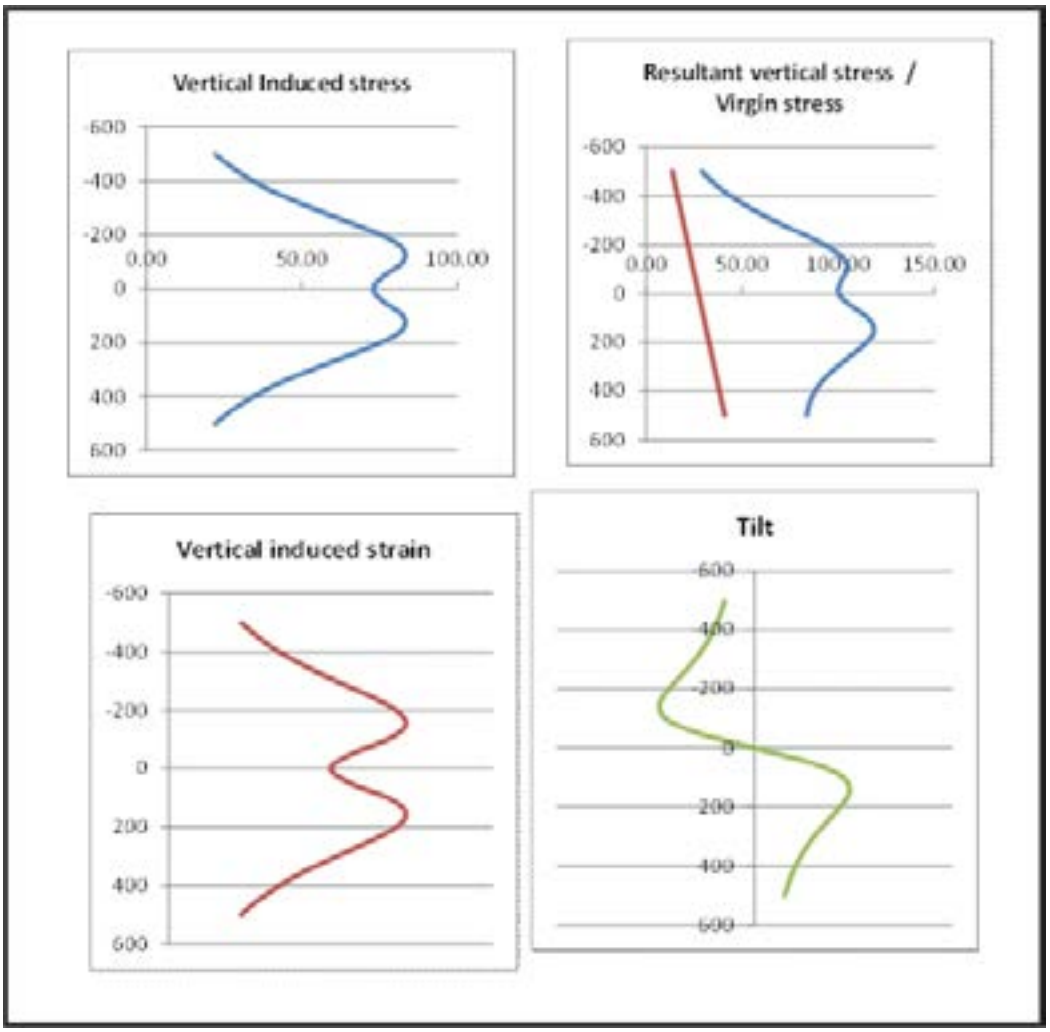


Figure 54: RCF related to position from longwall face (Jager and Ryder, 1999)

Rock mass classification systems

Rock mass classification systems (or commonly known as rock mass rating systems) have been designed to give a rating of the quality of the rock mass in which a tunnel is developed. Numerous systems are available, of which the Q-system, Rock Mass Rating (RMR) of Bieniawski, and the Mining Rock Mass Rating (MRMR) of Laubscher are the most commonly used systems. The latter have been adapted for use in the deep level underground application, where the first two systems are more applicable in shallow environments. The rock mass rating systems employed give a rating of the quality of the rock mass in which the tunnel will be (or are) situated. The main components of a rating system are: Strength of the material, rock quality designation (RQD), joint condition and orientation, and influence of water.

In all the systems, ground condition is related to excavation stability, stand-up time, and support requirements. Once the ground condition has been determined through the selected rating system, positioning (in best possible ground conditions) and orientation (with regards to joint dips) of the tunnels can be determined..



- Handbook on Rock Engineering Practice for Hard Rock Tabular Mines p58 – 59
- Brady & Brown p78 to 80.

LEARNING OUTCOME 4.4.1.2

CONNECTION 115

Refer to Outcomes LEARNING OUTCOME 3.1.15 on page 85 ; LEARNING OUTCOME 3.1.17 on page 87 and LEARNING OUTCOME 3.1.20 on page 93

Refer to LEARNING OUTCOME 3.2.3 on page 94 through 3.2.5

Refer toLEARNING OUTCOME 3.2.9 on page 99 through 3.2.14

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM LAYOUT, POSITIONING, ORIENTATION AND SPACING OF SERVICE TUNNELS RELATIVE TO :

- Static stope abutments,
- Moving stope abutments,
- Pillars of various shape, size and orientation,
- Geological features of various types, orientation and history,
- Other excavations

Static and moving stope abutments

The stability of tunnels is controlled by the effectiveness of the installed support system, changes in stress during the life of the tunnel, the rock mass environment in which the tunnel is developed, the influence of other excavations in close proximity to the tunnel and the field stress in the rock mass. In order to address the influences of the above-mentioned situations, various empirical and analytical methods are employed by the rock engineer to establish the ultimate layout for tunnels that will also suit the practical needs of the mining environment. General rules include:

- Tunnels should be placed in the most suitable rock mass environment and the layout and orientation should be such that the influence of mining operations and other excavations are minimized.
- The orientation of major geological features and jointing should be taken into account so that the tunnel(s) traverses the features over the shortest possible distance.

Numerical modeling of a preliminary layout will give an estimation of expected field stress levels around the mining layout and can be used to determine the optimum vertical distances between the tunnel position and the stoping horizon as well as expected changes in stress as mining progresses.

EXAMPLE

The rock mass will fail once its strength has been exceeded by the field stress. When fracturing has occurred and the stress acting on the rock is reduced by over-stoping, the fractured rock can lose all integrity and collapse into the tunnel. Numerical modeling will assist in understanding the level, orientation and changes in field stress levels as mining progresses.

EXAMPLE

On one of the gold mines in the Far West Rand, follow-on development below a longwall was placed less than 20m below the reef horizon. Extreme ground control problems caused by excessive relaxation of fractured rock around the tunnels, resulted in significant damage and all access development had to be redone.

Perimeter stresses can also be calculated using the Kirsch equations as developed for circular excavations and elastic rock mass properties. The stress levels around tunnel calculated using this method can assist in determining the estimated distances between tunnels and other excavations. At the same time, potential for fracturing (using appropriate failure criteria) and depth of fracturing can be estimated, assisting in determining:

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PAPER 3.1: CHAPTER 4

CONNECTION 116

Refer to Paper 1 material for more detail on Kirsch equations

- optimum excavation dimensions (by changing the radius of the tunnel in the calculations),
- best orientation (by calculating stress levels at different field stress orientations) and
- positioning of the tunnel (by calculating stress levels at different field stress orientations and measuring the impact of other excavations on stress levels around the tunnel).

NOTE

Determination of the k-ratio between horizontal and vertical stress components is a very simple method to assist with tunnel orientation, size and shape selection. This ratio will influence the fracture pattern (position of fractures, depth of fractures, etc) around the tunnel. .

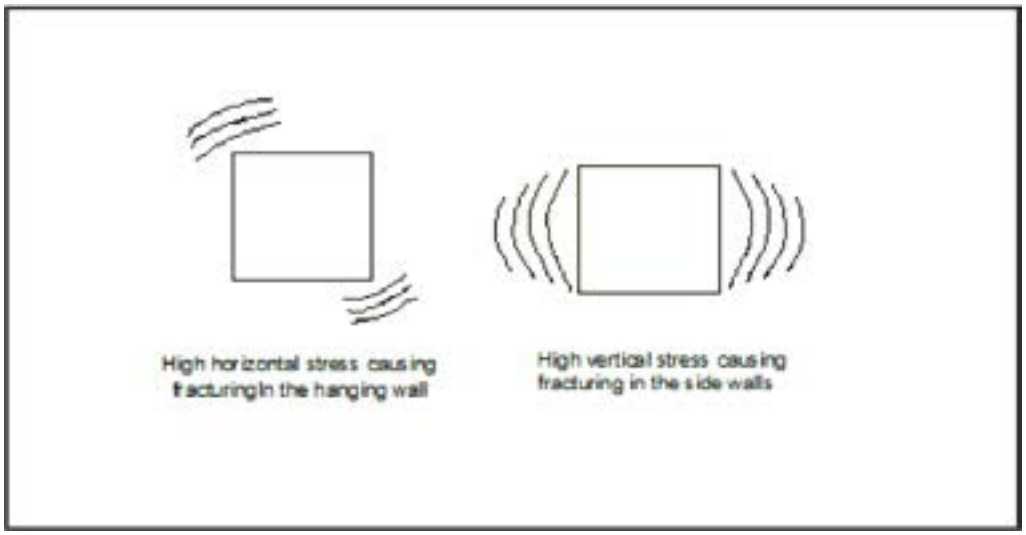


Figure 55: Influence of k-ratio on stress fracture positions

Influence of Abutments on Tunnel Stability

Any tunnel should be placed outside of the influence of unacceptable stress levels. Large abutment stress levels can extend far into the hangingwall and footwall of a mining stope. If the stress exceeds the strength of the rock mass at the point where the tunnel is located, the rock will fracture and dilate and if the support systems are insufficient, the tunnel can collapse.

The optimum orientation and shape of the tunnel is influenced by the stress distribution in the rock mass around the tunnel. Stress fracturing takes place parallel to the direction of the major principle stress component. The tunnel shape should be adapted so that minimum exposure of the sides of the tunnel (smallest dimension) is parallel to the direction of the major principle stress component.

Influence of Pillars on Tunnel Layout and Stability

In a mining layout where pillars form part of the regional and/or stope support system, the location of tunnels relative to the pillars become very important. Historically, large strike stability pillars in the long wall mining layouts on the deep level gold mines have resulted in pillar-foundation failure events. These seismic events caused significant damage to the infrastructure, as it occurred in the footwall of the pillars close to the tunnel position. Any tunnel located below these pillars is bound to be affected by the high stress levels generated on the pillars.

EXAMPLE

In the sequential grid mining layout, the tunnels are placed deep in the footwall of the stopes (as much as 80m) to prevent stress damage to the tunnels under dip regional pillars, as mining progresses. In longwall mining layouts, where the tunnels are developed in de-stressed rock

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parallel to the pillars, tunnels can be placed much closer to the reef horizon (about 20 to 30m).

Influence of Geological features on the layout, positioning and spacing of Tunnels

It is wise to orientate tunnels perpendicular to the direction of major geological features such as joints or faults. If a flat-dipping joint runs parallel to the direction of the tunnels, exposure can cause instability over a large distance along the tunnel. Where practical considerations prevent the tunnel direction to be altered, it may be necessary to adapt the shape of the tunnel to incorporate the weak plane of the joint or fault as one of the walls (see Figure 56).

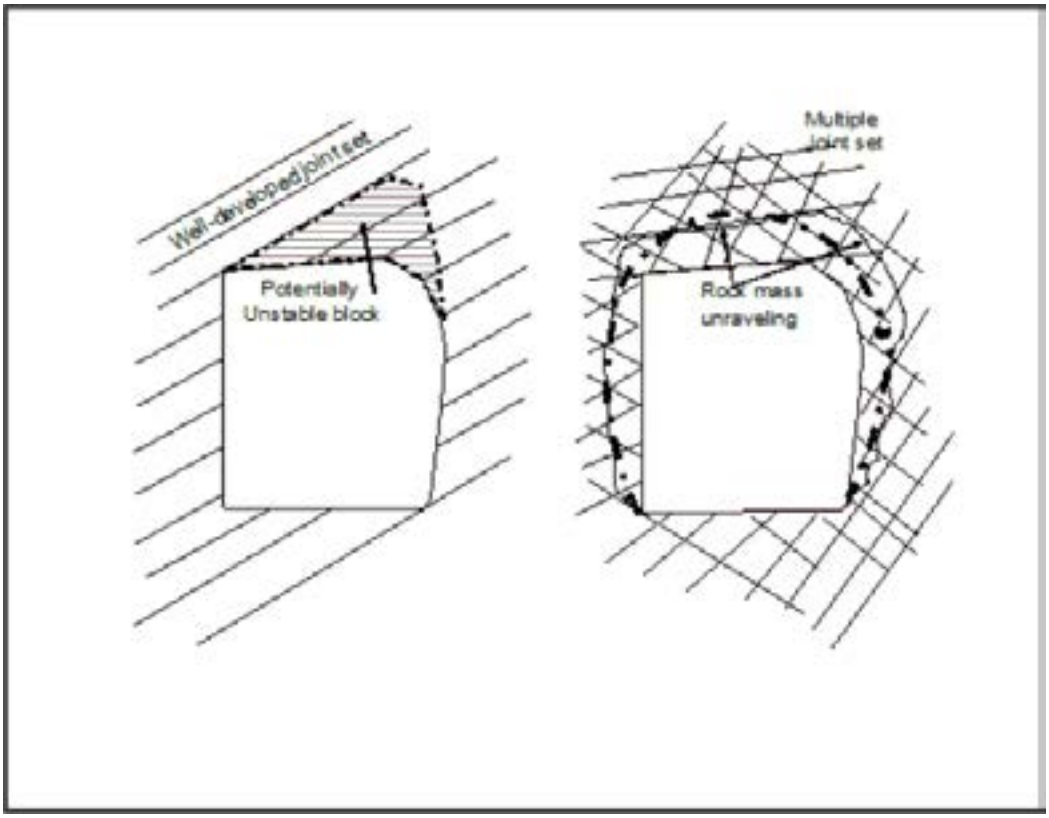


Figure 56: Effect of adverse jointing on tunnel shape

Different geological environments often result in different stress distributions as a result of the tectonic stress influence of nearby major geological features or the rock mass formation processes. In some areas of the Bushveldt Complex, very high horizontal stresses exist in one horizontal direction. Development of tunnels within this stress field results in the formation of a 'Gothic arch' caused by horizontal fracturing in the hangingwall of the tunnel.



One of the solutions is to cut the tunnel shape in the form of a Gothic arch to expose only a small area to the high stress direction. However, if practical, the tunnel direction can be altered to run parallel with the direction of the high stress in order to prevent the formation of fracturing and arching.

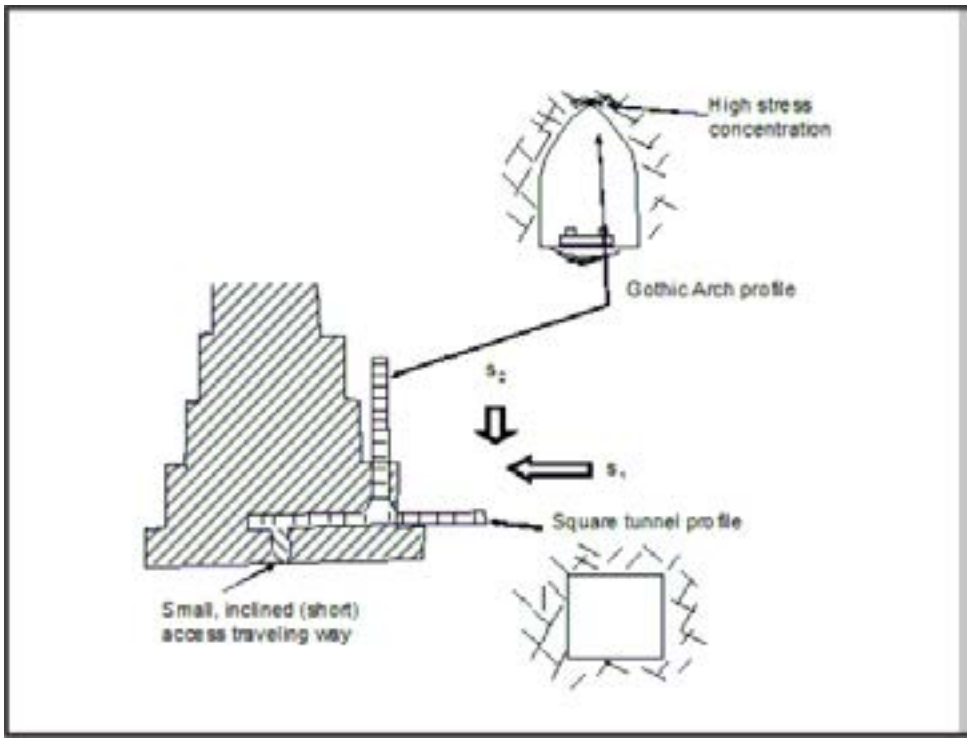


Figure 57: Tunnel layout in a high horizontal stress environment (Jager and Ryder, 1999)

- Jager and Ryder p201

Other Excavations

Interaction of multiple excavations should be prevented by placing such excavations away from each other. A guideline is to develop twin tunnels at least 3 times the combined width of the tunnels away from each other, to reduce the effect of interacting induced stresses.
Successive breakaways should be at least 6 times the width of the tunnel away from each other to ensure that bull-noses are not over- stressed.

Bullnose is the acute angle shaped pillar formed when two excavation intersect.

Figure 58: a. Induced stress profiles b: positioning of tunnels to avoid stress interaction (high stress conditions).

- Jager and Ryder p201

LEARNING OUTCOME 4.4.1.3

APPLY THESE CRITERIA TO DESIGN SERVICE TUNNEL LAYOUTS FOR : TUNNELS OF VARIOUS PURPOSE AND LIFE EXPECTANCY, VARIOUS COMBINATIONS OF TIMING.

CONNECTION 117

Refer to all Outcomes in Chapter 3 and 4 of this material.

- Refer to A handbook on Rock Engineering Practice for Tabular Hard rock Mines by Jager and Ryder Chapter 5

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EXAMPLE

QUESTIONS TO STUDY:

1. RMC P3.1 May 2010 Question 1
2. RMC P3.1 May2008 Question 1.6 (together with the whole Question 1)

4.4.2 LARGE CHAMBER LAYOUTS INTRODUCTION

CONNECTION 118

Refer to 4.4.1 Tunnel layouts on page 240

Refer to"3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS"and "3.3 ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS"

The stability of large service chambers is controlled by rock mass condition, field stress levels and changes, and the support system installed. Field stress levels are kept at tolerable levels by good positioning of such excavations in relation to all current and future mining abutments. A number of methodologies are applied during the design phase to ensure appropriate layout, size, shape and placement of large service chambers such as pump chambers, settler dams and hoist chambers.

The life of operation of these chambers are often several decades and therefore the design and support of such excavations should take into account the changes in mining operations and field stress levels and orientations over the expected life span.

The design of excavations in rock, especially service excavations, is a standard engineering function, and requires a systematic approach similar to any engineering design project (Figure 52).

LEARNING OUTCOME 4.4.2.1

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM SIZE, SHAPE AND ORIENTATION OF LARGE SERVICE CHAMBERS

CONNECTION 119

Refer to 4.4.1 Tunnel layouts on page 240 on tunnel layout and design.

The rock engineering criteria used for determining optimum size, shape and orientation of large service chambers are the same as those used for tunnels. The difference in application lies with the size and life expectancy of the excavations, as large chamber dimensions are significantly bigger than those of normal tunnels, and such chambers are normally expected to remain serviceable for the total life of the mine.

NOTE

Shaft protection pillars are often left intact around the shaft barrel and large service chambers are then placed within the shaft pillar area. It is important to place these excavations in areas within the shaft pillar where field stress levels will be as small as possible and stress changes limited over the life of the excavation. These conditions can be found as close to the centre of the shaft pillar as possible (Figure 58), where the stress levels are relatively low and constant, due to its placement far enough away from the pillar edge or high stress abutment.

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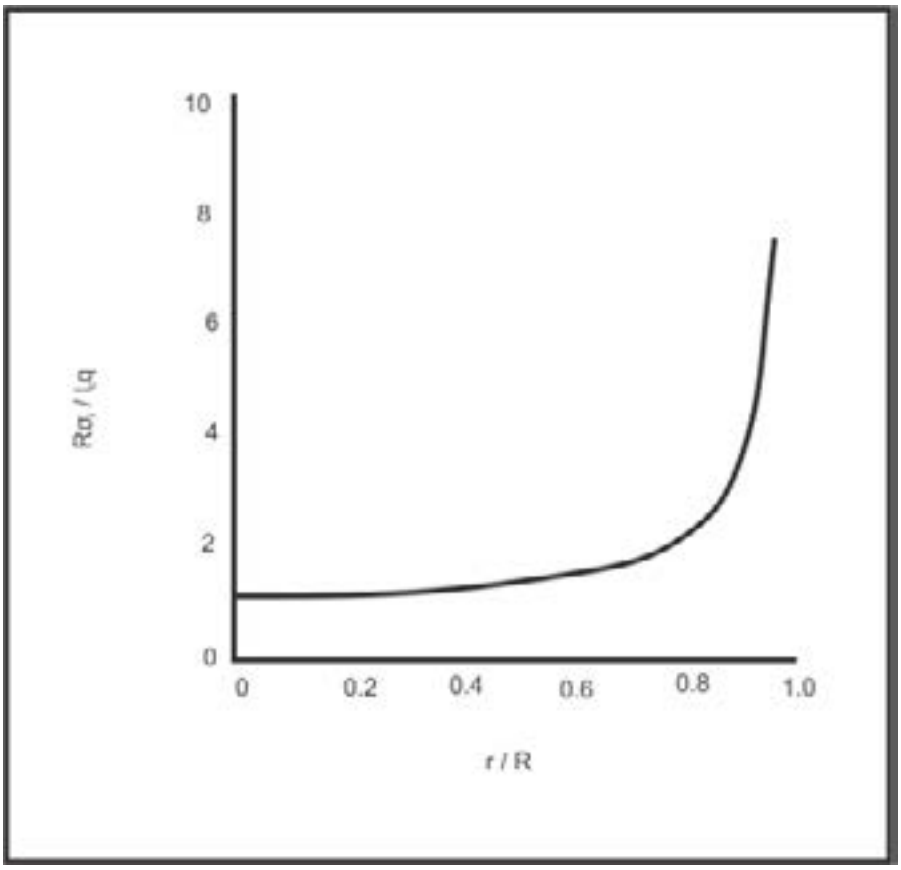


Figure 58: Vertical stress distribution across a shaft pillar (Budavari, 1983)

LEARNING OUTCOME 4.4.2.2

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM EXCAVATION METHODOLOGY AND SEQUENCE FOR LARGE SERVICE CHAMBERS

CONNECTION 120

Refer to LEARNING OUTCOME 3.3.19 on page 122 and LEARNING OUTCOME 3.3.20 on page 125

Apart from the positioning of the large excavations relative to other excavations and geological discontinuities, the excavation sequence and support of these excavations will have a significant influence on the long-term stability. The excavation sequence and technique is particularly important where the excavation is situated in high stress or incompetent rock mass conditions. The cut sequence chosen should aim to minimize the extent of unsupported rock wall and exposure of adverse fracturing during each stage of the development process. The development process should follow a downward pattern, starting with a top cut to enable the installation of support immediately in the hanging wall before excavation of the next cut. In extreme conditions, pre-supporting the excavation walls may become essential for long-term stability.

The development technique used should minimize blast damage, which otherwise could fragment the rock walls and produce uneven surfaces that are often difficult to support, and can cause deterioration of the rock. Perimeter control such as smooth-blasting will improve conditions and prevent damage to the rock walls. Drilling accuracy, spacing of blast holes, sequencing of the blast holes and correct application of explosives must be maintained throughout the development phase.

REFERENCE

- Refer to Jager & Ryder: The Handbook on Rock Engineering Practice for Hard Rock Tabular Mining chapter 7.2.4 p 228

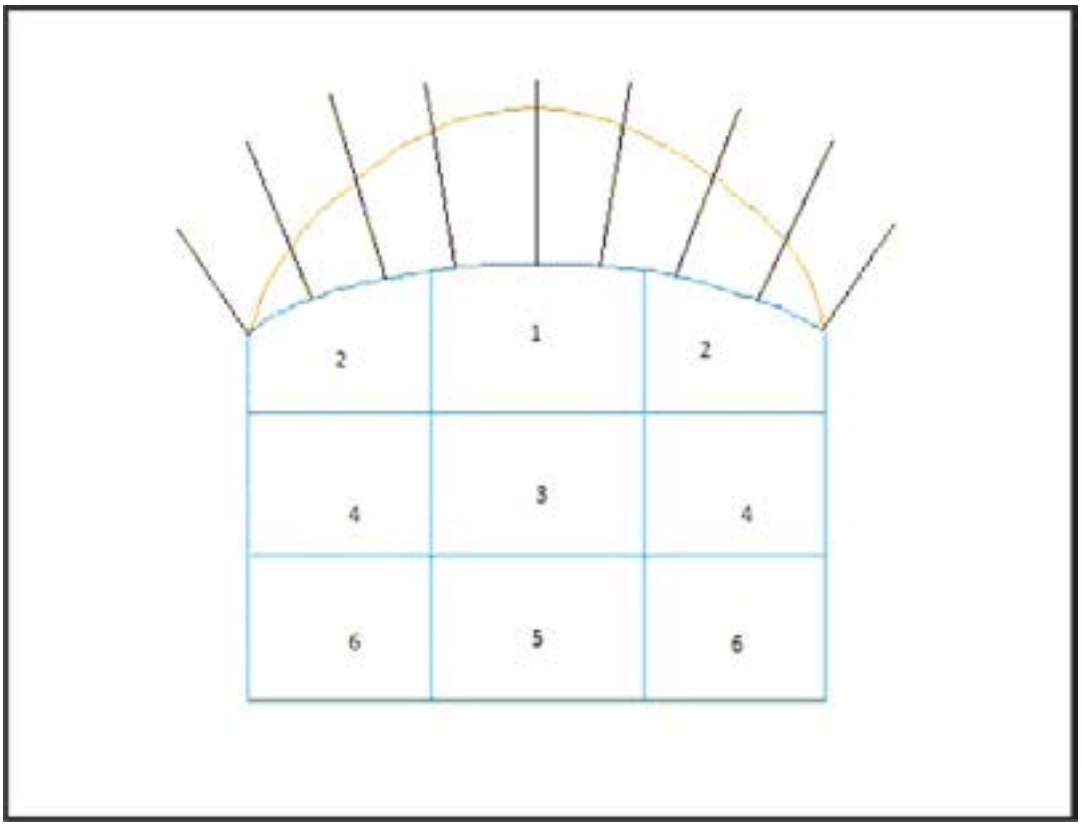


Figure 59: Excavation sequence for a large underground chamber

LEARNING OUTCOME 4.4.2.3

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM SUPPORT METHODOLOGY AND SEQUENCE FOR LARGE SERVICE CHAMBERS

The support system for large service chambers are governed by the following objectives:

- Ease of installation and timeous installation of support during the excavation phase in order to control the fractured zone around the excavation and create a safe working environment.
- Support installed must be capable of controlling any long-term stress changes expected in and around the excavation including possible seismicity.
- Consolidation of the rock surfaces of the chamber to avoid long-term unraveling so as to prevent damage to machinery and injury to people working in the area.

In order to address the above issues, the whole essence of support design is to create a stable beam in the hangingwall that will stand up for the life of the excavation. It is essential to establish the strength and competency of the rock mass by doing detail rock mass rating assessments. A well-established 'rule of thumb' is that a semi-circle (radius 0.5 x excavation width) volume of rock above the excavation with diameter equal to the width of the excavation will be subjected to a vertical tensile stress environment and will require supporting. This results in the implementation of support elements of length 'half the excavation width' to support the possible failure zone.

Some rock engineering textbooks refer to primary support units (Figure 60) as those units installed to support the semi-circle mentioned above

(long support units) while secondary support is used to stabilise the rock between the primary units (shorter and denser spaced support units). Although the reason for this classification is understood as ensuring that the large potential collapse is stabilized immediately on creation, the mining sequence utilised to develop such excavations and which limits initial excavation spans, makes it unnecessary. The normal practice is to install the shorter and denser spaced units as primary support (first) to protect the development crew. Then the longer support units are installed as secondary support to protect the excavation against large collapses. This installation occurs as part of the development sequence of large excavations as discussed in Refer to Outcomes 3.3.19 and 3.3.20.

Figure 60: Support design for a flat roof chamber (Jager and Ryder, 1999)

The strength of the roof of a chamber can also be enhanced by arching the shape. The failure zone is considerably smaller, thus the amount of rock to be supported is also reduced. Shorter support tendons than those used in flat roof excavations can thus be used to create a stable beam.



Figure 61: Reduced length of tendons above arched roof (Jager and Ryder, 1999)

For large chambers, side wall and foot wall support may be necessary. Side wall support length usually is half of the height of the chamber wall, and the same spacing is applied as for the hanging wall support. Mesh and lacing and thin layered spraying is applied where deterioration of the rock is expected in the long term. Various linings are available and the selection of the lining will depend on the function and finish required.
Foot wall support is normally restricted to chambers housing critical machinery, and the foot wall is therefore reinforced in order to remain solid for the machinery.



- Refer to the Handbook on Rock Engineering (Jager et al) chapter 7.2

LEARNING OUTCOME 4.4.2.4

CONNECTION 121

Refer toandLEARNING OUTCOME 4.4.2.1 on page 248

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM LAYOUT, POSITIONING, ORIENTATION AND SPACING OF LARGE CHAMBERS RELATIVE TO :

- Static stope abutments,
- Moving stope abutments,
- Pillars of various shape, size and orientation,
- Geological features of various types, orientation and history,
- Other excavations

LEARNING OUTCOME 4.4.2.5

APPLY THESE CRITERIA TO DESIGN LARGE SERVICE CHAMBER LAYOUTS FOR : CHAMBERS OF VARIOUS PURPOSE AND LIFE EXPECTANCY, VARIOUS COMBINATIONS OF TIMING.

CONNECTION 122

Refer to all Outcomes 4.4.1 Tunnel layouts on page 240 and 4.4.2 Large chamber layouts on page 248

In the design of large excavations, the following process is suggested:

1. Establish the requirements of the tunnels with regards to equipment size, volume of air, shape and orientation.
2. Determine the initial stress environment (prior to any other mining) in which the excavation will be situated.
3. Apply a rock mass rating system to determine expected ground conditions in the excavation using data acquired from similar conditions and available geotechnical data about the area.
4. Apply geological knowledge of the rock mass regarding geological disturbances such as joints, rock type, strength, etc. to determine the best orientation of the excavation relative to the major features.
5. Set up and run a numerical model of the design and analyse the results of the simulation to determine the stress environment in which the excavation will be situated. The numerical analysis will be a guide to what stress changes to expect, and where maximum stress buildup will result.
6. Apply the criteria for design of large service excavation layout and determine the most suitable size, layout, orientation and spacing of the excavation according to the information gathered through applying the criteria.
7. Apply the available tables and graphs to determine stand-up times which can be used to design the excavation shape and size for applicable ground conditions.
8. Compare the design to existing excavations in similar conditions to verify that it is acceptable.



- A handbook on Rock Engineering Practice for Tabular Hard rock Mines by Jager and Ryder Chapter 7

EXAMPLE

EXAMPLE QUESTIONS:

RMC p3.1 May 2011 Question 1
An existing pump chamber on 32 level in a working gold mine is shown in Figure 62. The figure shows the development and stoping layout around the shaft pillar. Mining is taking place at an average depth of 2000 metres below surface and the mine is seismically active. The reef dips

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at 20° to the South and the footwall strata is known to contain closely spaced bedding planes. 31 Level has been marked with a dark pen in the figure. The middling between 31 Level and the stoping above it (at "Pt A") is 40m. 32 Level is situated 30m below 31 level. A near vertical dolerite dyke traverses the area from the south west to the north east.

1. The existing pump chamber arrangement is clearly not well located and has suffered significant damage over the last few years. Discuss the factors that make the existing layout less than ideal given a typical deep level gold mining environment. (6)
2. It has been decided that a new chamber needs to be developed on 31 level. The chamber is to be accessed from the 31 Level vertical shaft station. Based on good rock engineering practice, recommend a suitable location for the new pump chamber. Motivate your recommendation. (A rough sketch can be made on the figure). Remember to hand figure 1 in with your exam paper and include your exam number on the sketch. (4)
3. The largest of the planned chambers measures 10m wide by 10m high by 60m long.
   a. 1.3.1 Use sketches to show the preferred excavation sequence of such a chamber and describe the reasons behind the chosen sequence. (5)
   b. 1.3.2 Give recommendations regarding temporary and permanent support requirements. In particular, recommend when permanent support should be installed giving due regard to practicality. (5)

[20]



Figure 62: Example Question 1

MODEL ANSWER:
Question 1



Important to note from the Question

- Deep mining - thus failure is stress driven
- Closely spaced bedding planes (dipping to the South) – makes N-S long axis orientation preferable.
- 40m middling has been given – this information should be used in some way (for the 45° rule)
- The dyke (both strata control and seismic risk)

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PAPER 3.1: CHAPTER 4

1. The chamber developed in elevated stress (2 marks)
   - Chamber is below the shaft pillar (elevated stress)
   - Chamber is close to the abutment (very high abutment stress)
   - The result is increased stress driven failure and vulnerability to ground motions from seismic events.
   - Always preferable to develop important excavations in a de-stressed environment.
- The chamber is located in the path of the dyke (2 marks)
   - Chamber intersected by dyke. This increases the chance of nearby seismicity and also the vulnerability of the chamber. Students could also refer to localized weakness due to sympathetic jointing and localized weaknesses.
- The long axis of the chamber is not ideal (2 marks)
   - The long axis is roughly orientated E-W. This is parallel to the strike of the bedding planes (which are reported as being closely spaced). From a stability point of view it is better to orientate the chamber N-S reducing the span over which individual joint planes traverse.

2. Refer to drawing

   Main points are (3 marks)
   - move the chamber outside of the stressed area (below the mined out area).
   - Orientate the long axis N-S
   - Apply the 45 degree rule to ensure you are out of the abutment stress
   - Note that the smaller pillars are not a problem at this middling!
   - Students may mention risk due to the access passing through the dyke and that it should be supported.
   - Second access way could also be suggested.
   - Additional 1 mark for accompanying and sensible comments (1 mark)

3. a. Any sensible approach will be accepted. Main points are (5 marks)
   - Break larger excavation into smaller sections i.e. 5m x 5m
   - Start with a top 5m cut and install support as you advance (1)
   - BEST to install long anchor support (6m) now else later at *
   - Develop the second top cut opening the span to 10m.
   - (*) Install remaining 6m long anchor support in the hanging-wall and tendon support in the sidewalls
   - Develop the lower cuts and install sidewall support as you progress. (3 and 4)

   b. **Temporary support** (1 mark)
   - Top cuts – Typical camlok jacks on a suitable 1.5m x 1.5m spacing. Nets should be used for friable ground.
   - Bottom cuts – Not necessary – permanent support in place. Barring of sidewalls is to be done and/or sidewall nets used.

   c. **Permanent support**
   - Will depend on local ground conditions but should include a combination of
   - Shorter tendons (H wall) can be designed for the 5m span (cut 1). (1 mark)

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SR = p g h (where h = ½ span = 2.5m)
= 2.7*10*2.5 = 67.5 kN/m^2

Assume 1 tendon gives 10 tons
= 100 kN and 2m advance

Spacing = 100/67.5 = 1.48m^2 per bolt.

Spacing = 1.0m x 1.5m (1/2 mark for some form of calculation)

- In poor ground conditions, areal coverage should be provided. Wire mesh and lace using installed tendons or fibre reinforced shot crete. WM&L preferable for highly seismic areas. (1 mark)
- Longer tendons (H/Wall) designed for the final 10m span (after cut 2). Length should be > ½ span i.e. 6m in length minimum. (1 mark)

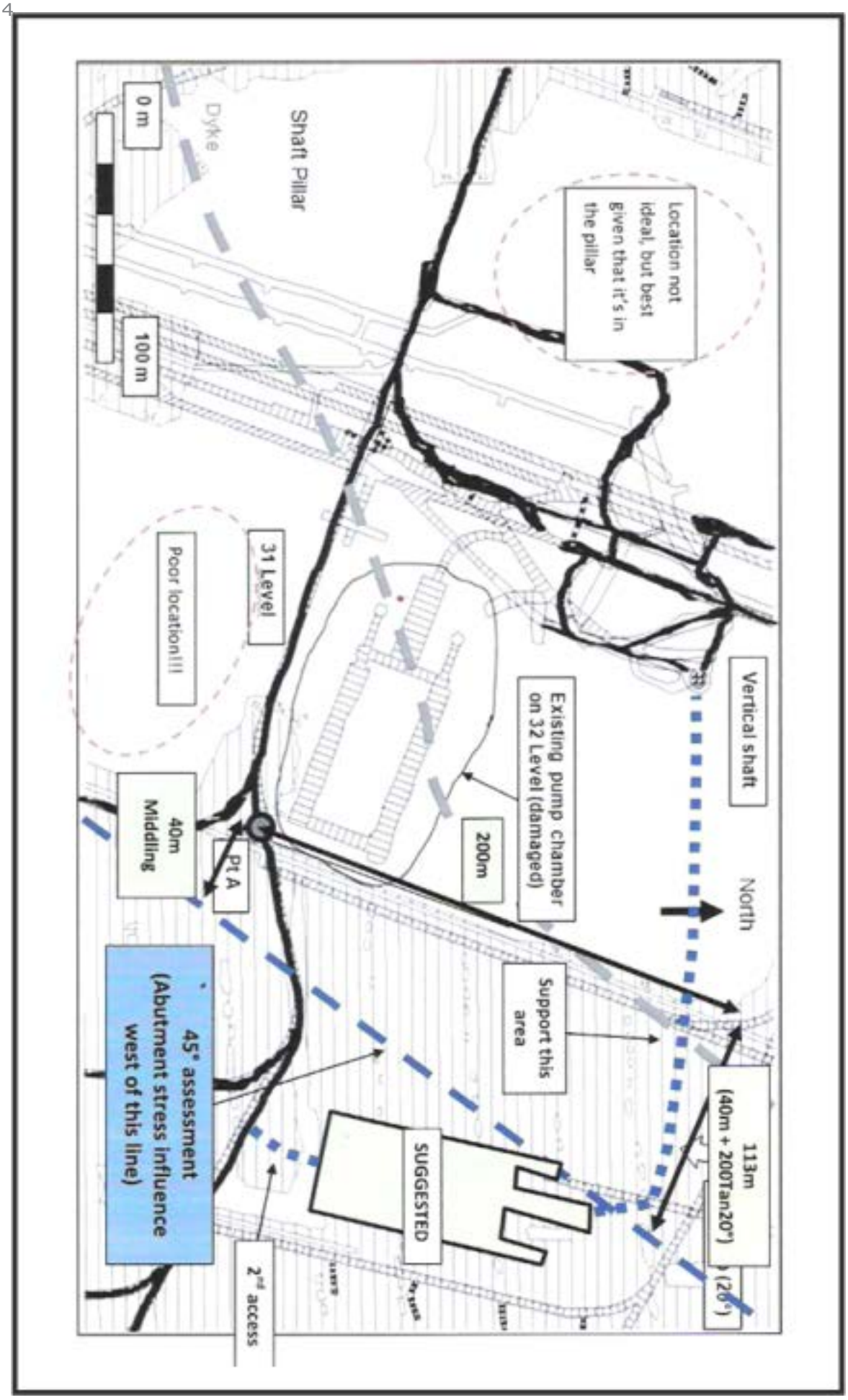
SR = pgh (where h = ½ span = 5m)
= 2.7*10*5 = 135 kN/m^2

Assume 1 Anchor gives 38 tons
= 380 kN and 2m advance

Spacing = 380/135 = 2.8m^2 per bolt.

Spacing = 2.0m x 1.4m

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Location not ideal, but best given that it's in the pillar
Shaft Pillar
Dyke
0 m
100 m
Vertical shaft
North
113m
(40m + 200Tan20°)
(20°)
Existing pump chamber on 32 Level (damaged)
200m
Support this area
SUGGESTED
31 Level
Poor location!!!
40m Middling
Pt A
45° assessment
(Abutment stress influence west of this line)
2nd access

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4.4.3 ROCKPASS LAYOUTS

INTRODUCTION

Even though main rock pass systems should last the life of a mine, too often expensive and hazardous remedial measures are required to repair damaged rockpasses in an attempt to extend their lives. Failure of a rockpass (or system) can have serious financial implications:

- indirectly because the rockpass cannot deliver the broken ore either from level to level or directly in the shaft rockpass system to the required level of production
- directly due to the cost of the repair work that is required.

Several factors need to be taken into consideration so that production interruptions are eliminated or minimized. In order for the rockpass system on a mine to have the necessary life expectancy, and to remain operational for that period of time, the layout, excavation and support of the rock passes become extremely important.

This section will deal with the design methodology implemented by rock engineers to accomplish this, as well as the rehabilitation processes when rockpass failure does occur.

LEARNING OUTCOME 4.4.3.1

DESCRIBE, EXPLAIN AND DISCUSS THE CAUSES OF INSTABILITY IN ROCKPASSES

A rockpass system normally forms part of the shaft layout and structure, and is placed close to the shaft in order to reduce tramming distances. Several factors will influence the stability of rockpasses in this area:

- Stress levels and stress level changes that takes place at the location of the rockpass: Scaling of rockpasses can occur when one of the horizontal field stresses exceeds the strength of the rock mass and the original shape of the rockpass, which is typically cylindrical, changes to an oval shape after scaling. The extent of the scaling depends on the strength of the rock mass and the ratio of the horizontal stresses. The closer the ratio of the horizontal stresses is to unity, the more even the stress damage and the greater the extent of the scaling that takes place perpendicular to the major horizontal stress direction.
- Orientation of the rockpass in relation to the direction of the major principle stress: The direction of the rockpass should be aligned with the direction of the major principle stress ($\sigma 1$) to minimize the effect of high stress on the rock. Furthermore, the magnitude and direction of the horizontal stress components will also influence the stability of the rockpasses. If the stress levels are higher than the strength of the rock in which the rockpasses are situated, it may be necessary to look at the design and shape of the rockpass as a method of protecting it from deterioration. An oval shape with the long axis of the oval perpendicular to the direction of the major horizontal stress may be advisable.
- Distance between the rockpass and other excavation(s) such as the shaft barrel, pump and hoist chambers, i.e. the location of the rockpasses: Often the locality of the rockpasses is dictated by other factors such the proximity to the shaft system, or proximity to the orebody, and there is very little scope for choice of location.
- Rock mass characteristics such as rock strength, layering, strata direction, etc: The quality of the rock mass in which the pass is

located may have the most significant effect on the performance of the pass. Due to the major influence of rock mass quality on damage, the location of the rockpasses should be optimized as far as possible.

- Influence of geological structures such as orientation, direction, strength and extent of the features, i.e. the orientation of rockpasses with respect to geological structures: It has been established that failure in rockpasses in the same rock mass quality occurs more readily for some orientations with respect to the geological structures than others. For example, in gold mines in which the rock mass is stratified, it has been found that passes intersecting the strata at acute angles are less stable than passes orientated perpendicular to the plane of the strata.
- Wear, i.e. the influence of the falling or flowing rock in the rockpass causing fracturing and scaling of the rockpass walls: Due to the passage of rock down the rockpass, wear (abrasion) may lead to enlargement of the pass. Wear will be enhanced in weak rock material and in the presence of stress fracturing. Wear is usually greatest on the footwall surface of the rockpass, but may not be when interaction takes place with weak rock materials and stress scaling.

All these factors will be discussed in the chapter on the layout and design of the rockpass system.

LEARNING OUTCOME 4.4.3.2

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF INSTABILITY IN ROCKPASSES

Instability of rockpasses can have a serious influence on the successful mining operation on a mine:

- Ore transportation through the rockpass system is essential for the production on the mine. When the quality of the rockpass deteriorates, it can result in ore being locked up in the rockpass when it hangs up or prevent transportation from the whole mining area as ore cannot be removed.
- In extreme cases, rockpasses have deteriorated to such an extent that the waste rock pass and the ore pass have scaled to such an extent that they holed into each other, resulting in a mixing of ore and waste rock, reducing the grade delivered to the plants and financial loss.
- When the rockpass deterioration is not controlled, the possibility of interaction between the pass and neighboring excavations increases. Known cases exist where the rockpass has caused failure in the shaft barrel with major implications to the stability of the shaft.
- The flow of rock through the rockpass can be blocked, called 'hanging up'. To release material from such a blocked orepass, explosives are often used which either required personnel to enter the orepass underneath the blocked zone to place the explosives (unsafe and not the suggested option) or remotely placed explosives are used. In both cases, scaling is enhanced when blasting occurs and often results in continued scaling and orepass problems.



Removal or prevention of water flowing into orepasses reduce the potential for hangups to occur.

LEARNING OUTCOME 4.4.3.3

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIONS AVAILABLE TO OPTIMISE ROCKPASS LAYOUTS

The options available to optimize rockpass layouts are summarized as follows:

- Determine the stress field in the rock mass where the rockpass will be situated;
- Use the results of laboratory, analytical and numerical modeling studies available to determine the stress field changes expected during the life of the rockpass;
- Determine the optimum spacing between the rockpass and neighboring excavations to ensure minimum interaction of excavations;
- Geotechnical and geological information about the host rock and structures in the proposed position of the rockpass must be scrutinized to analyse the possible influence on the stability of the rockpass.

LEARNING OUTCOME 4.4.3.4

Describe, explain and discuss the options available for stabilising rockpasses

The options for stabilising rockpasses are:

- Optimisation of layout with respect to stress, strata layers, geological discontinuities and other excavations to prevent deterioration of orepasses.
- Where stress levels are high or are expected to increase to high levels, the shape of the rockpass may be adapted to form a stable excavation (oval instead of circular) against stress induced scaling.
- Orepasses can be lined with a high abrasive concrete linings.
- Orepasses can be developed oval, fitted with circular high abrasive concrete lining and filled with cement fill around the lining.
- Installation of support to prevent unraveling of the rockpass can include mesh and lacing onto tendon support and spraying of high abrasive shotcrete over the mesh.

LEARNING OUTCOME 4.4.3.5

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIONS AVAILABLE FOR SUPPORTING ROCKPASSES

Support installation of rockpasses will depend on quality of the rock, stress configuration, expected stress changes, proximity of other excavations, geological discontinuities and extent of scaling that has already occurred. Accepted support methodologies currently include:

1. **Tendon Support**

In rockpasses, tendon support can form the basis of the support system and will perform adequately depending on the environment around the rockpass. Slabbing and spalling failures are typical in highly competent rock under high stress, but can be resisted by a dense pattern of tendon or rope anchor support. In moderately competent and jointed rock, block and wedge failures are of greater concern and tendon installation will assist in preventing the scaling.

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Tendons should be supplemented by wire mesh and lacing and a strong lining (shotcrete) as normal rockpass wear will result in the loss of rock pieces between the tendons, reducing the embedded length of the tendons as portions of it will now protrude and become less effective.

2. Lining of rockpasses

Lining of the rockpass can be done by applying mesh and lacing onto tendons as well as the application of a sprayed concrete lining. Apart from protecting the rockpass walls from the effect of the displacing rock, it will also protect the installed tendons and mesh and lacing from damage and perform a support function through confinement.
As ore and waste rock in South African mines are often highly abrasive, special types of concrete / shotcrete (e.g. corundum or andesite lava constituents) are often required.

INTERESTING INFO

The 'abrasiveness' of any shotcrete or lining material is affected by the following:

- Compressive strength of the material
- Cement type
- Fibres and
- Aggregate.

High abrasive materials are divided into 'frictional abrasive' or 'impact abrasive' types. The former type suggests that it is resistant to abrasion that might occur when materials slide past each other and where friction is the main driver of wear. The latter type represents those materials that appear abrasive when impacts between material particles occur.

It has been suggested (Van Heerden et al, 2004) that the type of abrasion that occurs in silos is more 'impact' than 'friction' related. Since normal forces between the loose material and the walls of the silo are small, the frictional resistance would be small and thus result in limited frictional abrasion. Shotcrete for use in silos would thus require to be abrasive against impact between the particles and the silo walls.

Compressive strength: Increased strength is usually related to increased abrasive resistance. However, at high strengths other factors also start to affect abrasiveness, such as cement type, fibres and aggregates.

Cement type: Strength reduction between in-situ application and laboratory results is a known concern in 'dry shotcrete' applications. Van Heerden recently noted that better applied strength results can be achieved when OPC rather than High Alumina Cement types are used. Although data to substantiate this claim has not been provided at this stage, the experience of the author is accepted.

Fibre: The inclusion of fibres in shotcrete has two results, namely increased energy absorption capacity and increased impact resistance. Since impact resistance is related to the ability of the material to sustain impact with other materials and not to fracture in the process, the addition of fibres assist absorption of energy during impact and prevent failure, improving the impact resistance of the material. The higher the impact forces, which is also a function of the ore density (with manganese at ± 4 tonnes/m3), the more important this abrasion resistance becomes. It appears from tests reported by Van Heerden (2004), that polypropylene fibres (MBT products) have a greater effect on impact abrasiveness than standard (Dramix) steel fibres.

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Aggregate: High 'hardness' of material (as measured on Moh's scale) is related to frictional abrasiveness while the materials' ability 'not to fracture' when impacted on, affects its ability to be impact resistant. This 'ability not to fracture' is not easily measured, but can be estimated by using particles of specific maximum dimension (Dmax). When Dmax is smaller than the 'critical dimension' (smallest dimension such that the particle will not have an inherent weakness plane within the particle), it is stated to be very resistant against fracturing when impacted on. Since the 'critical dimension' is a function of the geological nature of material, it can be suggested that igneous rock types, such as andesite, would be more 'resistant to fracture' than schist (which is layered) material. A Dmax of 20mm for Andesite was suggested by Van Heerden (2004).



Corundum is not readily available in South Africa anymore, so most high abrasive shotcrete available will contain andesite lava particles.

LEARNING OUTCOME 4.4.3.6

DETERMINE APPROPRIATE ROCKPASS LAYOUT, ORIENTATION AND SUPPORT MEASURES USING ROCKMASS AND STRESS CHARACTERISATION TECHNIQUES

Poor rock strength or poor rock mass quality

In the mine design process, locations of rock passes should be chosen to avoid poor strength or poor quality rock material (Figure 63). Evaluating rock material strengths (UCS) and rock mass ratings conducted along the planned rockpass location is critical in estimating the quality of the material to be exposed. Comparison of rock strengths or roc mass quality along the planned rockpass will indicated possible areas of concern while a comparison between different positions will allow selection of best possible rockpass location.

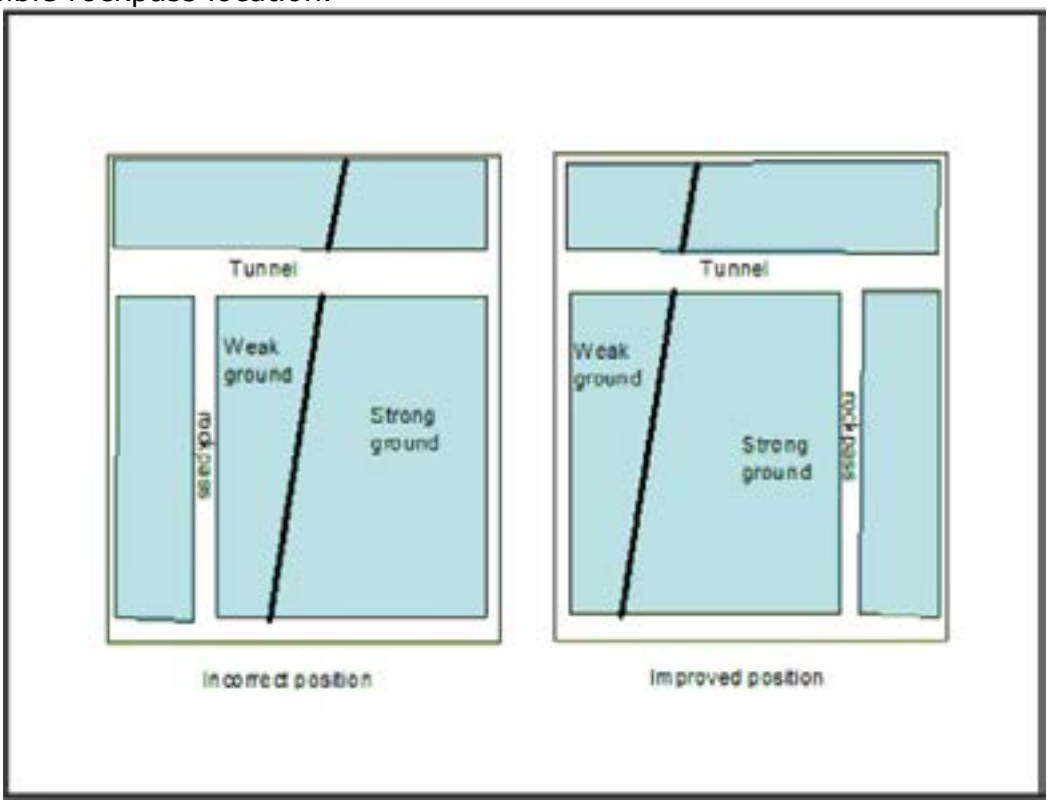


Figure 63: Positioning of rockpass in weak as against strong ground

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Stacey and McCracken developed a rock mass rating system that is based on Barton's Q Index, specifically for the design of rockpasses. This system adjusts the normal Q Index by factors for weathering, excavation wall evaluated (wall or roof) and orientation to obtain a Q for raisebores parameter Q_R. The raisebore diameter suitable to the resulting rock mass quality is then related to the Q_R Index via $Span_{max}=2RSR.Q_R^{0.4}$ where RSR is similar to the ESR used by Barton in his Q Index system.



The typical range for RSR is 0.8 for long term and public excavations and 5 for temporary excavations. Stacey suggested that a RSR of 1.3 is acceptable for shafts and 1.6 for rockpasses.



- McCracken and Stacey 1989 geotechnical risk assessment for shafts

Orientation of rockpasses with respect to geological structures

CONNECTION 123

Refer to Paper 1 and Paper 2 material for core logging detail

Failure occurs more readily for some orientations with respect to the geological structure than others. It has been found that orientation of significant structures 90° to the largest field stress component, result in the most significant scaling as stress fracturing occurs around the structure (as with dog earing) which then allows easier scaling due to the creation of smaller slabs (stress fractures and geological structure plane). A study of the geology around the planned position through the logging of core (orientated, as it will allow derivation of structure dip and dip directions) can provide enough information to allow the best decision.



- Rockpass stability prediction geology Stacey et al

Orientation of passes with respect to stress

CONNECTION 124

Refer to Paper 1 material for more detail on Kirsch equations

Stress analysis using Kirsch equations can provide an initial indication of stress concentrations and possible stress impact on the rockpass, but numerical modelling remains the most appropriate method to assess the impact of field stresses on and around the planned rockpass, especially if extensive mining is planned close to the rockpass.



- Rockpass risk associated deep T.R. Stacey.

LEARNING OUTCOME 4.4.3.7

CONNECTION 125

Refer to LEARNING OUTCOME 4.4.3.6 on page 261

ASSESS THE SUITABILITY OF ROCKPASS LAYOUT, ORIENTATION AND SUPPORT MEASURES FOR GIVEN SETS OF ROCKMASS AND STRUCTURAL CONDITIONS



- Rockpass risk associated deep T.R. Stacey.

LEARNING OUTCOME 4.4.3.8

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM SIZE, SHAPE AND ORIENTATION OF ROCKPASSES

The criteria used to determine the optimum size, shape and orientation of rockpasses does not deviate from the criteria used in tunnel and

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service excavation layouts as discussed in "4.4.1 Tunnel layouts".

LEARNING OUTCOME 4.4.3.9

CONNECTION 126

Refer to 4.4.1 Tunnel layouts on page 240

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DETERMINE THE OPTIMUM LAYOUT, POSITIONING AND SPACING OF ROCKPASSES RELATIVE TO :

- Static stope abutments,
- Moving stope abutments,
- Pillars of various shape, size and orientation,
- Geological features of various types, orientation and history,
- Other excavations

LEARNING OUTCOME 4.4.3.10

APPLY THESE CRITERIA TO DESIGN ROCKPASS LAYOUTS FOR : PASSES OF VARIOUS PURPOSE AND LIFE EXPECTANCY, VARIOUS COMBINATIONS OF TIMING

INTERESTING INFO

Stacey (2004) presented a series of six design principles (derived by Bieniawski) that encompass a design methodology. The design principles are summarized below:

Design Principle 1:
Clarity of design objective and functional requirements – A statement of the 'problem' and a statement of the design objectives, taking account of any constraints that are present, to satisfy this problem, is essential to any design process. These statements clarify the design thinking outset.

Design Principle 2:
Minimum uncertainty of geological conditions – The rock masses in which mining takes place are variable, which is true of any natural material, and design therefore takes place in an environment of considerable uncertainty. In mining, which is almost always tightly cost-controlled, there is usually an aversion to incurring costs on geotechnical investigations, with the result that geological conditions are often unknown or, at best, little known. Designs are often carried out with inadequate knowledge of in situ tresses, the rock material strengths and deformation properties, and the rock mass behavioural conditions. The minimization of uncertainty will provide an environment in which more confident design can be carried out, and hence will reduce the risk.

Design Principle 3:
Simplicity of design components – Bieniawski (1991, 1992) indicates that, in terms of the simplicity principle, a design should be broken down into a series of simpler components. It is suggested here that the principle should be viewed, in addition, in its broadest context – simpler designs, design methods and design analysis are easier to understand and therefore likely to be more robust.

An important step in the design of a rockpass is to develop a geotechnical model. This may be conceptual, but it is important to be able to describe the likely behaviour of the rock mass in which the pass is located and the possible mechanisms of instability. Only once this has been done, can appropriate design (failure) criteria be decided on, design limits are defined, required factors of safety or probabilities of failure

be defined, a design model (or models) be developed, and appropriate design analysis methods be decided upon. This will ensure that the design is appropriate, and as simple as possible.

Design Principle 4:
State of the art practice – The implication of this principle is that up to date concepts, analysis and methods must be used whenever they are appropriate.

Design Principle 5:
Optimization – Risk integrally involves numerous factors including safety, cost, productivity, seismicity, water, manpower, etc. Therefore, to optimize risk, designs must be optimized. An optimized design will result from the evaluation of the output from alternative designs.

Design Principle 6:
Constructability – If the design cannot be implemented safely and efficiently it does not satisfy this principle and therefore is also not optimized.

The design methodology corresponding with the above six design principles is summarized in the ten steps given in Table 1 below. The link is given between the steps in the methodology and the corresponding principle. This methodology represents a thorough design process and can be used as a checklist to ensure that a defensible design has been carried out.



- Design process Stacey

Application of the information shared to date in designing rockpass can be adequately described by the following papers:



- Rockpass design and support Stacey
- Rockpass design and support at deep Vieira

LEARNING OUTCOME 4.4.3.11

DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO REHABILITATE DAMAGED ROCKPASSES



To describe the criteria used to rehabilitate rock passes, two papers on case studies are included:

- Rockpass system at Kloof No. 3 Shaft (R. Hart)
- Rockpass stability and rehabilitation at Anglogold Ashanti's Lekoa Mine (G.B. Dukes, J. van Oort, and F.D. van Heerden)

LEARNING OUTCOME 4.4.3.12

ESTIMATE THE ROCKPASS LIFE EXPECTANCY AS A RESULT OF VARIOUS SUPPORT AND REHABILITATION MEASURES.



Estimation of the life of rock passes is discussed in the paper on the case study at Anglogold's Moab Khotsong Mine.

- Optimizing orepass life deep Brenchley Spies

4.5 SHAFT LAYOUTS

4.5.1 SHAFT PROTECTION PILLARS

LEARNING OUTCOME 4.5.1.1

CONNECTION 127

Refer to Outcomes 4.2.2.2 on page 244 and 4.2.2.4 on page 249 in Paper 2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DESIGN INCLINE SHAFT PROTECTION PILLARS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

REFERENCE

- Jager pg 244

Shallow depth:
Since stress levels are low and non-yield pillars are utilised to ensure local and regional stope stability, declines are usually developed on-reef and are protected using rib pillars all along the decline. These pillars are designed to be non-yield as well to ensure that the decline is not subjected to stress fracturing and closure over its life. Substantial factors of safety (often > 2 and even as high as 2.5) are used in the design of the decline protection pillars.

Intermediate depth:
Declines are placed below the reef or orebody at an distance that would (1) prevent stress interaction with the stoping above and (2) allow over stoping of the declines early in their lives to ensure that stress levels do not increase significantly enough to induce stress fracturing and tunnel closure. The overstoping distances and middling analyses are executed as discussed for tunnels in Outcome 4.1.1.

Deep level:
Same as for intermediate, even though declines are most often placed well below the reef so that no stress interaction can occur with the mining of the orebody.

LEARNING OUTCOME 4.5.1.2

CONNECTION 128

Refer to Paper 2 Outcomes 4.2.2.1 on page 232, and 4.2.2.3 on page 245

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE ROCK ENGINEERING CRITERIA USED TO DESIGN VERTICAL SHAFT PROTECTION PILLARS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

The aim of leaving shaft pillars around vertical shafts is to protect the shaft barrel against mining induced stress increases and displacements. The size and shape of the pillars are thus mainly driven by the assurance that low stress levels / changes and displacements are induced at the shaft barrel position. Figure 58 shows that stress levels towards the edges of a shaft pillar is expected to be very high while that towards the centre is lower and more constant. In deriving the optimum shaft pillar size, the selection of a critical stress level (σcr) beyond which the stress levels throughout the shaft pillar should not rise.

EXAMPLE

(Jager and Ryder, 1999):

For good quality quartzite with UCS > 200MPa, σcr ≈ 100MPa.

Once this critical stress value has been selected, a shaft pillar size can be estimated using Figure 64 below for the appropriate mining depth.

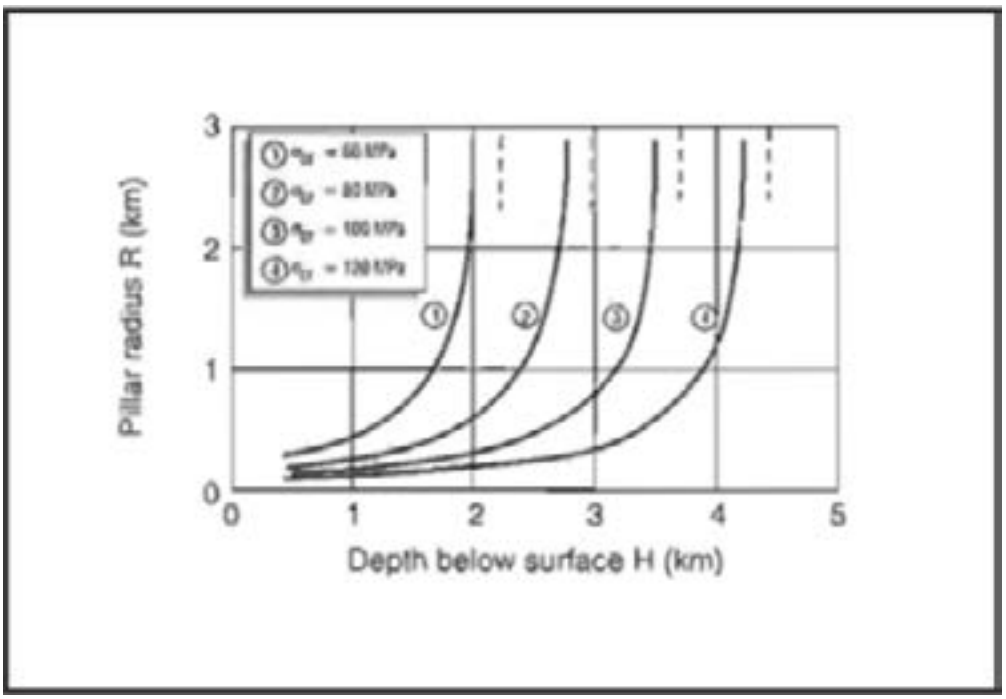


Figure 64: Critical stress levels related to depth below surface and shaft pillar radii

- Jager page 238

A simple rule of thumb that is often used as an first estimate of shaft pillar size is the relationship between radius and mining depth, given by Radius= 1/5 x mining depth. However, this was taken from the coal mining and has resulted in under designed shaft pillars in some older gold mines. Due to this, the critical stress level method described earlier and numerical modelling has become preferred design methods.

- Jager page 236
- Spearing page 111

Strain to which the shaft barrel will be subjected due to the increase in stress levels around the barrel, should be limited to approximately 0.4mm/m for steelwork and 0.7mm/m for concrete linings. Since the determination of these strain values is complex, numerical modelling techniques are utilised.

The criteria discussed above are independent of the mining depth but are dependent on rock strength and stiffness only. They are thus appropriate for all mining depths where shaft pillars are deemed important.

LEARNING OUTCOME 4.5.1.3

CALCULATE STRESS, STRAIN AND TILT IN VERTICAL SHAFTS USING EQUATIONS FOR CIRCULAR SHAFT PILLARS IN AN ELASTIC MEDIUM

The distribution of the vertical induced stress along a shaft excludes the effect of the shaft barrel on the stress levels. Since the impact of a barrel on vertical stress at depth is negligible, it is accepted that this practice is sound.

The distribution of the vertical induced stress along the shaft barrel, is given by

$$\sigma_z^i=(s_m.E)/(4R(1-v^2)).((1+4\beta^2))/(1+\beta^2)^{2.5}$$

Where s_m is the convergence that equals the width of the extracted reef, R is the shaft pillar radius, E and v are the elastic constants of the rock mass and β =z/R, where z is the depth at which the induced stress will be calculated.

The resultant stress along the axis of the shaft pillar can be calculated by adding the induced and virgin stresses, and can be calculated using:

$$\sigma_z=(s_m.E)/(4R(1-v^2)).((1+4\beta^2))/(1+\beta^2)^{2.5} +q.(1+z/H)$$

where H is the depth of the origin of the co-ordinate system below surface and q is the vertical virgin stress component at reef elevation, z=0.

NOTE

The co-ordinate system used places the origin of the system on reef elevation and in the centre of the shaft pillar, where the positive z-axis is towards the footwall.

EXAMPLE

The resultant vertical stress on the reef elevation in the centre of the shaft pillar (z=0) will be given by

$$\sigma_z^i=(s_m.E)/4R(1-v^2) +q$$

The distribution of induced vertical strain along the shaft barrel, the result of the induced stress, is given by

$$\varepsilon_z^i=s_m/R.([2(2-v)\ \beta^2+(1-2v)])/(4(1-v)(1+\beta^2)^{2.5})$$
where β = z/R.

And where the maximum vertical induced strain is given by

$$\varepsilon_{zmax}^i=0.051.s_m/R.(2-v)^{2.5}/((1-v))$$

Tilt along a shaft barrel can be determined form

$$t_x^i=1/\pi.s_m/z\{((1-2v))/(4(1-v))+[R/|z|\ ]^3.[(2(1+v)-(1-2v).(R/|z|\ )^2)/(4(1-v).[1+(R/|z|\ )^2]^{2.5}\ )]\}$$

NOTE

The maximum values for Induced vertical Strain and Tilt along the shaft barrel, have been shown to be given by the equations below:

$$t_{max}^i=0.13(s_m/R) \quad \text{and} \quad \varepsilon_{max}^i=0.28(s_m/R)$$

EXAMPLE

At a reef depth of 2000m in a rock mass with elastic constants E=70GPa and v=0.2 and density 2.78t/m3, max convergence outside the shaft pillar of sm=1.0m and a shaft radius of 250m, calculate and sketch the induced and resultant vertical stress, induced vertical strain and tilt for 500m above and below the reef elevation.

Using the equations given in this Outcome, the required parameters are substituted and the values calculated as required.

The user is advised to calculate the results and confirm answers using the spreadsheet below.

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REFERENCE

- Excell spreadsheet Stress strain tilt along shaft barrel

LEARNING OUTCOME 4.5.1.4

CONNECTION 129

Utilise all aspects covered in Outcomes LEARNING OUTCOME 4.5.1.1 on page 265 to 4.5.1.3 to design shaft pillars such as the method shown in the example:

APPLY THE RESULTS FROM THE ABOVE CALCULATIONS TO DESIGN APPROPRIATE SHAFT PILLAR DIMENSIONS

EXAMPLE

(Courtesy of course notes by Budavari)

The radius of a shaft pillar and distribution of vertical stress along the barrel are to be determined for the conditions given: E = 70GPa, v=0.2, sm=1.0, σcr=100MPa, H=2000m and q=53MPa.

Using

$$\sigma_z^i = (s_m.E)/4R(1-v^2)+q$$

the radius of a shaft pillar can be calculated by re-arranging the parameters in the equation to get

$$R = E/(4(1-v^2)).s_m/((\sigma_{cr}-q)).$$

A radius of 387.9m is calculated using the input parameters provided.

Using the equation given above for maximum induced strain values along the shaft barrel, it can be shown that:

$$\varepsilon_{imax} = 0.28(1.0m / 387.9m) = 0.72x10\text{-}3$$

and if the maximum strain for rock material is 1x10-3, it follows that a shaft pillar woth radius 387.9m will adhere to the critical stress and maximum induced strain criteria for shaft pillar design.

NOTE

Recently the use of numerical models to calculate the stresses and strains, as well as tilt, along the shaft barrel has gained wide acceptance and is the preferred method of designing suitable shaft pillars. The same criteria of critical stress, induced strain and tilt levels are still used as indicators of possible concerns.

Keep the strain levels given in Outcome 4.5.1.6 for shaft concrete linings and steelwork in mind when analysing the results shown above.

LEARNING OUTCOME 4.5.1.5

CONNECTION 130

Refer to Outcomes LEARNING OUTCOME 4.5.1.1 on page 265 to LEARNING OUTCOME 4.5.1.4 on page 268

DESIGN SHAFT LAYOUTS FOR GIVEN DEPTH, GEOLOGY AND ROCKMASS CONDITIONS

In designing shaft pillars for the conditions mentioned in this Outcome, it is suggested that numerical models be used, but that the following should be kept in mind:

- The stress, strain and tilt criteria described above as results from the models;
- The impact of regional or un-mined pillars in the vicinity of the shaft pillar on stress levels;
- The creation of potentially unstable wedges between the shaft and geological structures;

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- The correct estimation of elastic constants for the rock mass using rock mass quality parameters;
- Possible seismic activity on structures within the shaft pillar;
- The final extraction of the pillar (square pillars are preferred to circular pillars as it makes later extraction easier).



The application of numerical modelling techniques allows the opportunity to evaluate all the concerns stated above, while the elastic analyses shown in previous outcomes do not.

LEARNING OUTCOME 4.5.1.6

EXPLAIN THE LIMITATIONS THAT CONCRETE LININGS AND STEELWORK PLACE ON ALLOWABLE MOVEMENTS IN SHAFTS.

Both concrete linings and shaft steelwork is not able to sustain infinite strain levels but either fail in compression (bending caused by compression as stress increases) or tension (pulled apart when the shaft pillar is extracted).

Generally accepted maximum strain levels include:

a. approximately 0.4mm/m (0.0004 or 0.4 x 10^{-3} m/m) for steelwork and

b. approximately 0.7mm/m (0.0007 or 0.7 x 10^{-3} m/m) for concrete linings.

CONNECTION 131

More criteria are available from Paper 2, Outcome 4.2.2.3 on page 245



These values are lower than the normally accepted maximum value of strain for rock, which is 1 x10-3 m/m. It suggests that concrete and steel are more sensitive to strain levels than rock material.

4.5.2 SHAFT PILLAR EXTRACTION

LEARNING OUTCOME 4.5.2.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PREPARATORY WORK REQUIRED PRIOR TO SHAFT PILLAR EXTRACTION AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Planning of the shaft pillar extraction is critical and unique as conditions and layouts vary from site to site. However, the following are issues that should be addressed:

1. Extraction sequence: The sequence can be affected by many issues, but the first is to determine whether the shaft barrel stability is critical or not. If the mining will be conducted through another shaft barrel, the shaft pillar can be extracted as a normal remnant. However, if the barrel must remain stable during the extraction, care must be taken to implement the best possible sequence. At the same time, other critical excavations such as hoist chambers, refrigeration chambers, orepasses etc should be protected by the sequence. These decisions must be made up front as they determine the rest of the preparations.
2. Estimation of displacements and strains: The likely levels of convergence and ride must be determined based on the planned extraction sequence. With good support systems (such as backfill) convergence

should be less than half the stoping width in the mining workings. By minimising the stoping width, the displacements. Strains and tilt can be minimised. However, it is good practice to plan slightly larger stoping widths on the edge of the shaft barrel to prevent the build-up of high stresses in this area. The strain limits calculated should be evaluated for rock, concrete lining and steelwork as each of these materials will be affected by the displacements. Commonly the steelwork will be affected over the largest distance along the barrel, followed by the concrete lining and then the rock material itself. If strain limits are exceeded, the extraction sequences or support methods must be re-evaluated.

3. Monitoring: Displacements must be monitored along the shaft barrel to measure the actual against the planned or calculated displacements or strains. This monitoring can consist of various methods and instruments but must be able to record the displacements as accurately as possible for some distance up and down the shaft barrel around the reef intersection.
4. Ventilation: Ventilation requirements must be kept in mind and short-circuiting of fresh air is possible when mining the shaft pillar. At the same time, failure of the brattice wall in the shaft (that is the wall that separates the air intake from the return air flow in a shaft barrel with dual flow) could also affect mining in other areas on the mine if air flow is disrupted.
5. Shaft lining alterations: Since the expected displacements are the greatest around the reef intersection points, the shaft lining in these areas will be subjected to tensile strain as the mined out area experiences closure. Removal of the lining and supporting of the rock mass behind the lining must be planned for approximately 15m up and down the shaft barrel. Support could consist of systems that would sustain the expected displacements and could include mesh and tendons.
6. Shaft steelwork alterations: Since the steelwork will be subjected to compressive strains as closure occurs, the steelwork is normally altered by suspending the steelwork from above the expected area of concern or by installing telescopic guides that can accommodate the expected displacements.
7. Shaft services: Shaft water and air pipe columns and electric cables as well as backfill pipes (and any other services within the shaft barrel) must be protected by replacing some portions with flexible pipe materials, bracketing the pipes and cables above and below the affected area, etc.



- Budavari pag 229

LEARNING OUTCOME 4.5.2.2

DESCRIBE, EXPLAIN AND DISCUSS THE EXTENT AND MAGNITUDE OF STRESS CHANGES TO BE EXPECTED AROUND A SHAFT BEFORE, DURING AND AFTER SHAFT PILLAR EXTRACTION

It is anticipated that numerical modelling will be utilised when analysing the stress changes along the shaft barrel and displacements around the reef intersection areas. The method of determining these values are not discussed here, but merely explanation of the rock mass behaviour to understand the risks.

When a shaft pillar is extracted, the following rock mass behaviour results:

- Stress levels along the barrel reduce as mining continues, as the barrel is de-stressed by the mining conducted around it. This means that the points of maximum stress, which was originally was situated just above and below the reef elevation, moves further up and down the barrel as mining continues;
- If the reef dips at an angle, mining may create high stress lobes ahead of mining faces that intersect the shaft barrel higher up or lower down the barrel. These stress concentrations should be prevented by changing the mining sequence;
- As the inner portion of the shaft pillar is extracted, the stress around the reef intersection reduces drastically (Stage 1, Figure 65), but the changes are limited to the area in the barrel close to the reef intersection;
- As mining then continues, the stress levels around the reef intersection reduce even further, but the effect also moves higher up and down the shaft barrel (Stages 2 and 3, Figure 65).

Figure 65: Stress changes during shaft pillar extraction (Budavari, 1983)



Please refer to this graph on Page 227 of Budavari (1983) as a clear copy could not be created.



- Budavari page 226

LEARNING OUTCOME 4.5.2.3

DESCRIBE, EXPLAIN AND DISCUSS THE EXTENT AND MAGNITUDE OF MOVEMENTS TO BE EXPECTED IN AND AROUND A SHAFT BEFORE, DURING AND AFTER SHAFT PILLAR EXTRACTION

It is anticipated that numerical modelling will be utilised when analysing the stress changes along the shaft barrel and displacements around the reef intersection areas. The method of determining these values are not discussed here, but merely explanation of the rock mass behaviour to understand the risks.

When a shaft pillar is extracted, the following rock mass behaviour results:

- The hangingwall and footwall of the stoped out area closes towards each other. That means that the rock mass above the reef intersection moves down and that below the intersection moves upwards;
- The maximum displacement is realised around the reef intersection and decreases as the distance up and down the barrel increase;
- As mining around the barrel increases, the displacements increase since maximum closure is affected by the mining span around the barrel;
- Around the reef intersection, closure occur perpendicular to the bedding. In dipping reef, the hangingwall and footwall closes towards each other and creates tilt along the barrel, where the barrel in the hangingwall of the reef intersection displaces towards the updip and the barrel in the reef intersection footwall, displaces in the downdip direction. This creates a dislocation in the barrel at the reef intersection and could be detrimental for shaft conveyances.

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- The amount of ride or tilt is a function of the vertical displacement and by limiting this component of the closure (by limiting stoping width and using support systems such as backfill), the tilt can be minimised.

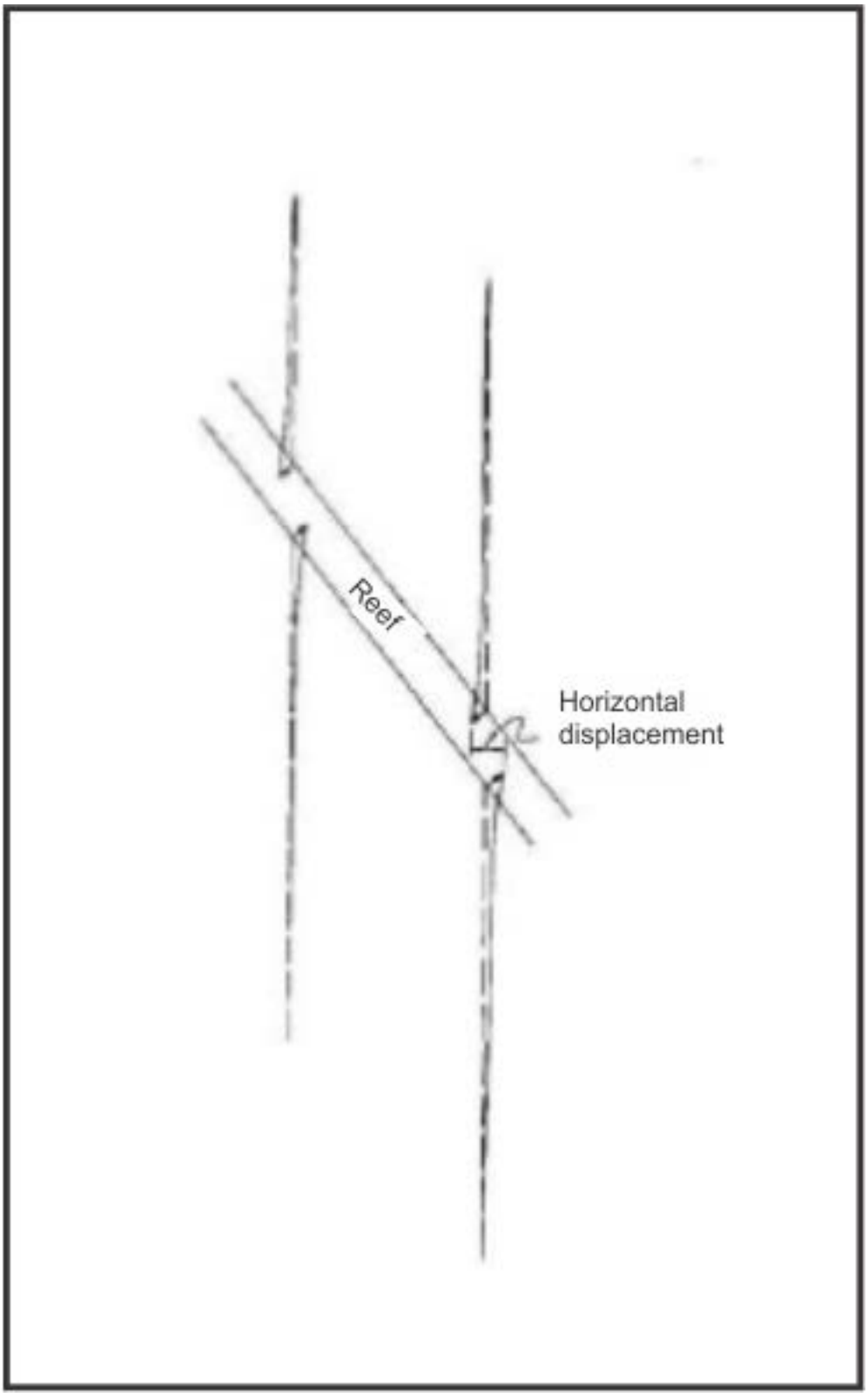


Figure 66: Tilt along the barrel caused by horizontal and vertical displacement (Budavari, 1983)

- Budavari page 227

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LEARNING OUTCOME 4.5.2.4

DESCRIBE, EXPLAIN AND DISCUSS THE EXTENT AND MAGNITUDE OF SEISMICITY TO BE EXPECTED AROUND A SHAFT BEFORE, DURING AND AFTER SHAFT PILLAR EXTRACTION

Depending on the depth of the shaft pillar, extent of mining around the shaft pillar and presence of geological structures within the pillar, seismic activity of various magnitude of frequency can be expected. High stress conditions will improve the potential for the generation of seismic activity and will determine the level of seismicity experienced.

As mining around the barrel increases, the tendency is to experience an increase in seismic activity as stress levels increase on the reef elevation but also on geological structures within the shaft pillar. As the final extraction phase of the shaft pillar is approached, experience in some environments have been that the level of seismic activity reduce as stress is transferred from the shaft pillar to surrounding unmined areas.

Depending on the extraction method, seismic activity could reduce after shaft pillar extraction (rock mined out) or continue to occur on satellite or regional pillars or on geological structures.

LEARNING OUTCOME 4.5.2.5

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS APPROPRIATE MINING SEQUENCES FOR EXTRACTING A SHAFT PILLAR

CONNECTION 132
Refer to LEARNING OUTCOME 4.5.2.1 on page 269

Mining sequences will be controlled by (inter alia) the following:

- The decision on the need to maintain shaft stability or not;
- The presence of seismically active structures that must be bracketed, mined out or de-stressed;
- The presence of shaft service excavations that must be protected;
- The need to protect shaft orepass systems;
- The minimising of stress, displacement and strain changes along the barrel;
- Protection of mining on secondary reefs, etc.

However, in most cases, the extraction is planned to start from the shaft barrel with:

- The extraction of an inner pillar: This is a small area (typically some 30m x 30m) immediately around the shaft barrel that is extracted first, at low stress levels and painstakingly accurate with regards to breaking into the barrel, support installation and controlling of stoping widths.



This mining is critical to the longer term stability of the barrel and should be done slowly and to design, not driven by production, but by the quality of the mining and remedial measures.

- Followed by the extraction of the rest of the shaft pillar: Follow an extraction sequence that will address all the issues listed above.



- Shaft pillar extraction Noble

LEARNING OUTCOME 4.5.2.6

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS APPROPRIATE METHODS OF SUPPORT WHEN EXTRACTING A SHAFT PILLAR

During the extraction of the inner pillar, the main aims of support systems utilised are:

- Ensure local stope stability;
- Limit closure around the shaft barrel and
- Protect the shaft barrel edge against high induced stress levels on support.

To achieve this, the following support has been used:

- Soft support types around the barrel edge such as yielding packs;
- Stiff, high loadbearing support units such as concrete or end-grain timber packs further away from the barrel to limit closure;
- Small crush pillars to limit closure;
- Backfill in inner pillar but not on the barrel edge to generate low loads during initial stoping, but regionally limit closure as the mining in the shaft pillar progresses.

CONNECTION 133

Apply LEARNING OUTCOME 4.5.2.1 on page 269 to Outcome 4.5.2.6

For the extraction of the outer portion of the shaft pillar, support systems that will ensure local stability of the stopes is critical and should be determined as for normal stoping on the operation. However, if it is required to substantially limit the vertical closure and thus displacements, strains and tilt, the use of backfill as local and regional support is beneficial.



- Budavari page 234

LEARNING OUTCOME 4.5.2.7

CONNECTION 134

Apply LEARNING OUTCOME 4.5.2.1 on page 269 to Outcome 4.5.2.6

DESIGN APPROPRIATE SHAFT PILLAR EXTRACTION SEQUENCES FOR GIVEN SHAFT PILLAR LAYOUTS AND ROCKMASS DATA



- Shaft pillar extraction Goldfields
- Shaft pillar extraction Noble

LEARNING OUTCOME 4.5.2.8

DESIGN APPROPRIATE SUPPORT FOR SHAFT PILLAR EXTRACTION WORKINGS FOR GIVEN SHAFT PILLAR LAYOUTS AND ROCKMASS DATA

CONNECTION 135

Apply LEARNING OUTCOME 4.5.2.1 on page 269 to Outcome 4.5.2.6



- Shaft pillar extraction Goldfields
- Shaft pillar extraction Noble

LEARNING OUTCOME 4.5.2.9

DESIGN APPROPRIATE SUPPORT AND PROTECTION MEASURES FOR ASSOCIATED SHAFT PILLAR EXCAVATIONS FOR GIVEN SHAFT PILLAR LAYOUTS AND ROCKMASS DATA.

CONNECTION 136

Apply LEARNING OUTCOME 4.5.2.1 on page 269 to Outcome 4.5.2.6



- Shaft pillar extraction Goldfields
- Shaft pillar extraction Noble

4.5.3 ALTERNATIVE SHAFT PROTECTION METHODS

LEARNING OUTCOME 4.5.3.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE GEOTECHNICAL ISSUES ASSOCIATED WITH THE FOLLOWING ALTERNATIVE SHAFT PROTECTION METHODS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH :

CONNECTION 137

Refer to Paper 2 Outcome 4.2.2 on page 243

- Satellite Pillars,
- Bracket Pillars,
- Partial Extraction,
- Shaft Reef Extraction,
- Non-Intersection

Protection method	Geotechnical issues	Design	Advantages and disadvantages
Satellite pillars	High stress levels on pillar can cause seismic activity and stress fracturing. Ground conditions can be poor if later extraction is contemplated.	Satellite pillars are left intact at specific areas and spacings and have as aim to reduce stress levels around the barrel if a pillar still exists around the barrel or to limit closure if the area around the barrel has been extracted.	Since stress levels become high on these pillars, extraction is often not contemplated at a later stage. However, if left large enough, extraction when the barrel stability is no longer required (mined out through another shaft barrel), is probable. In most cases, service excavations are situated below the mined out areas between satellite pillars, whilst the area around the barrel is also extracted. This protects the service excavations and also de-stresses the shaft barrel. Care should be taken with satellite pillars not to overstress access tunnels, orepasses and other service excavations while any stress raising impact on the shaft barrel must be prevented (i.e. barrel must be far enough away from the pillars). Stress levels on satellite pillars must also remain low enough not to cause pillar failure of pillar foundation failure.
Bracket pillars	Bracket pillars that become overstressed can result in large seismic events and their dimensions must take account of all mining that will be conducted in the shaft pillar.	Bracket pillars are utilised to bracket seismically active geological structures in an attempt to limit seismic activity. Use of the ESS parameter is made in designing bracket pillars. Refer to Outcome 4.2.1.3	Whilst this is their first role, they also act as pillars that reduce stress levels around the barrel if a pillar still exists around the barrel or to limit closure if the area around the barrel has been extracted. Care should be taken with bracketed geological structures not to overstress access tunnels, orepasses and other service excavations that might be situated close to the structures or bracket pillars. Bracket pillars cannot be extracted at a later stage.
Partial extraction	Low grade areas will become highly stressed and ground conditions will deteriorate.	Unpay pillars that remain intact act as protection pillars to reduce stress levels around the barrel if a pillar still exists around the barrel or to limit closure if the area around the barrel has been extracted. Design is similar to satellite pillars.	In conditions where grade distribution is highly variable, partial extraction can be utilised to mine high grade portions of a shaft pillar only. In this case, the low grade areas remain intact. Care should be taken with partial extraction not to overstress access tunnels, orepasses and other service excavations that might be situated close to the remaining, higher stressed unpay areas
Shaft reef extraction	Outcome 4.5.2		
Non-intersection	Paper 2 Outcome 4.2.2		

Table 9: Other shaft protection methods



- Jager pg 245
- Rock engineering considerations for the protection of vertical shafts Budavari and Ozbay

LEARNING OUTCOME 4.5.3.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF SATELLITE SHAFT PROTECTION PILLARS

CONNECTION 138

Refer to LEARNING OUTCOME 4.5.3.1 on page 275

LEARNING OUTCOME 4.5.3.3

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE DESIGN OF SATELLITE SHAFT PROTECTION PILLARS

The design of satellite pillars includes the following numerical modelling to establish:

- Non-critical stress levels on the satellite pillars;
- Limiting closure between pillars and around the barrel;
- Sufficient de-stressing of the shaft barrel, tunnels and service excavations;

The design requires the simulation of a number of satellite pillar layouts (dimensions and positions) and the comparison of the parameters given above.



- Rock engineering considerations for the protection of vertical shafts Budavari and Ozbay

LEARNING OUTCOME 4.5.3.4

DESCRIBE, EXPLAIN AND DISCUSS THE ADVANTAGES AND DISADVANTAGES OF THIS APPROACH

CONNECTION 139

Refer to LEARNING OUTCOME 4.5.3.1 on page 275 to Outcome 4.5.3.3

LEARNING OUTCOME 4.5.3.5

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE FUNCTIONS OF SHAFT REEF BRACKET PILLARS

CONNECTION 140

Refer to Table 9, LEARNING OUTCOME 4.5.3.1 on page 275

LEARNING OUTCOME 4.5.3.6

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE DESIGN OF SHAFT REEF BRACKET PILLARS

CONNECTION 141

Refer to Table 9, LEARNING OUTCOME 4.5.3.1 on page 275

LEARNING OUTCOME 4.5.3.7

DESCRIBE, EXPLAIN AND DISCUSS THE ADVANTAGES AND DISADVANTAGES OF THIS APPROACH

CONNECTION 142

Refer to Table 9, LEARNING OUTCOME 4.5.3.1 on page 275

LEARNING OUTCOME 4.5.3.8

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PURPOSE OF PARTIAL SHAFT REEF EXTRACTION

CONNECTION 143

Refer to Table 9, LEARNING OUTCOME 4.5.3.1 on page 275

LEARNING OUTCOME 4.5.3.9

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE DESIGN OF PARTIAL SHAFT REEF EXTRACTION

CONNECTION 144

Refer to Table 9, LEARNING OUTCOME 4.5.3.1 on page 275

LEARNING OUTCOME 4.5.3.10

DESCRIBE, EXPLAIN AND DISCUSS THE ADVANTAGES AND DISADVANTAGES OF THIS APPROACH

CONNECTION 145

Refer to Table 9, LEARNING OUTCOME 4.5.3.1 on page 275

LEARNING OUTCOME 4.5.3.11

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF EXTRACTING THE SHAFT REEF AT AN EARLY STAGE TO AVOID LEAVING A SHAFT PROTECTION PILLAR

CONNECTION 146

Refer to Outcome 4.2.2 in paper 2 on page 245

LEARNING OUTCOME 4.5.3.12

DESCRIBE, EXPLAIN AND DISCUSS THE ADVANTAGES AND DISADVANTAGES OF THIS APPROACH

CONNECTION 147

Refer to Outcome 4.2.2 in paper 2 on page 245

LEARNING OUTCOME 4.5.3.13

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PRECAUTIONS REQUIRED TO ALLOW EARLY EXTRACTION OF THE REEF AROUND SHAFTS

CONNECTION 148

Refer to Outcome 4.2.2 in paper 2 on page 245

LEARNING OUTCOME 4.5.3.14

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE ADJUSTMENTS REQUIRED TO THE SHAFT TO ALLOW EARLY EXTRACTION OF THE REEF AROUND A SHAFT

CONNECTION 149

Refer to 4.5.2 Shaft pillar extraction on page 269

LEARNING OUTCOME 4.5.3.15

CONNECTION 150

Refer to 4.5.2 Shaft pillar extraction on page 269

EVALUATE MINING LAYOUTS, SEQUENCES, SUPPORT AND PRECAUTIONS NECESSARY TO ALLOW EARLY EXTRACTION OF THE REEF AROUND A SHAFT FOR GIVEN CIRCUMSTANCES

LEARNING OUTCOME 4.5.3.16

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PRECAUTIONS REQUIRED TO MINE WITHIN SHAFT PILLARS TRANSGRESSED BY SEISMICALLY ACTIVE GEOLOGICAL STRUCTURES

Normal precautions taken when mining seismically active areas are also appropriate in shaft pillar mining and will include (inter alia):

- Acceptable mining sequences:
  - Lead / lags minimised but panels not inline;
  - Mine highly stressed areas first;
  - Do not mine panels out of sequence:
  - Approach geological structures favourably;
  - Limit production levels;
- Appropriate support systems:
  - Support units with sufficient energy absorption capacity
  - Limiting spans between support units;
  - Good areal coverage;
- Sufficient seismic monitoring practices:
  - Appropriate seismic network;
  - Sufficient use of the seismic data to indicate risk levels.

LEARNING OUTCOME 4.5.3.17

CONNECTION 151

Refer to Outcome 4.2.2 in paper 2 on page 245

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE SITING OF SHAFTS ENTIRELY IN THE HANGINGWALL OR FOOTWALL SO AS NOT TO INTERSECT REEF

LEARNING OUTCOME 4.5.3.18

DESCRIBE, EXPLAIN AND DISCUSS THE ADVANTAGES AND DISADVANTAGES OF THIS APPROACH.

CONNECTION 152

Refer to Outcome 4.2.2 in paper 2 on page 245

4.6 LAYOUT DESIGN CRITERIA

LEARNING OUTCOME 4.6.1

CONNECTION 153

Refer to LEARNING OUTCOME 4.2.1.1 on page 187

DESCRIBE, EXPLAIN AND DISCUSS THE DERIVATION AND APPLICATION OF THE FOLLOWING MINE LAYOUT AND SUPPORT DESIGN CRITERIA FOR SHALLOW, INTERMEDIATE AND GREAT DEPTHS :

- Average Pillar Stress,
- Energy Release Rate,
- Excess Shear Stress,
- Rockwall Condition Factor.

CHAPTER 5

PAPER 3.1 HARD ROCK TABULAR MINING

5. MINING SUPPORT STRATEGIES

LEARNING OUTCOMES

5.1. STOPING EXCAVATION SUPPORT STRATEGIES

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

5.1.1. Sketch, describe, explain and discuss the various behaviour of rock around excavations at shallow, intermediate and great depth

5.1.2. Describe, explain and discuss how this affects stope support requirements at shallow, intermediate and great depth

5.1.3. Describe, explain and discuss the concept of geotechnical areas / ground control districts

5.1.4. Describe, explain and discuss rockmass classification schemes for the determination of geotechnical areas / ground control districts

5.1.5. Describe, explain and discuss the determination of geotechnical areas / ground control districts

5.1.6. Describe, explain and discuss the application of the concept of geotechnical areas / ground control districts

5.1.7. Sketch, describe, explain and discuss appropriate support strategies to control the hangingwall in narrow stopes under static conditions at shallow, intermediate and great depth

5.1.8. Sketch, describe, explain and discuss appropriate support strategies to control the hangingwall in narrow stopes under seismic conditions at depth

5.1.9. Sketch, describe, explain and discuss the special problems of supporting wide reef stopes at shallow, intermediate and great depths

5.1.10. Describe, explain and discuss appropriate strategies to ensure stability under wide reef conditions

5.1.11. Sketch, describe, explain and discuss the special problems of supporting multiple, closely spaced reef stopes at shallow, intermediate and great depths

5.1.12. Describe, explain and discuss appropriate strategies to ensure stability under multiple, closely spaced reef conditions

5.1.13. Sketch, describe, explain and discuss how gullies should be supported at shallow, intermediate and great depths

5.1.14. Describe, explain and discuss appropriate support strategies to ensure gully stability in various layouts at shallow, intermediate and great depths

back

5.1.15. Describe, explain and discuss the special problems of supporting gullies relative to various stope layout and fracture configurations at shallow, intermediate and great depths

5.1.16. Determine the required support reaction for static conditions given typical fall of ground dimensions and loading conditions

5.1.17. Determine the required support reaction for dynamic (rockburst) conditions given typical fall of ground dimensions and loading conditions

5.1.18. Determine appropriate stope support strategies for given rockmass conditions, field stress regimes and geological circumstances.

5.2. SERVICE EXCAVATION SUPPORT STRATEGIES

5.2.1. TUNNEL SUPPORT STRATEGIES

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

5.2.1.1 Sketch, describe, explain and discuss the various behaviour of rock around excavations at shallow, intermediate and great depth

5.2.1.2 Describe, explain and discuss how this affects tunnel support requirements at shallow, intermediate and great depth

5.2.1.3 Include in design deliberations the influences occasioned by the following considerations :

- Gravity loading environment, Seismic loading environment
- Rock characteristics, Rockmass characteristics, Geological structure
- Stress mechanisms, Stress cycle, Rate of stress change, Initial stress
- conditions
- Mining configuration, Mining cycle, Overstoping techniques,
- Destressing techniques
- Tunnelling configuration, Tunnelling cycle, Pre-development, Followbehind
- development
- Support components, Support systems, Support appropriateness
- Implementation quality, Management effectiveness, Quality control

5.2.1.4 Describe, explain and discuss the objectives of primary support in tunnels

5.2.1.5 Describe, explain and discuss the objectives of secondary support in tunnels

5.2.1.6 Describe, explain and discuss the objectives of tertiary support in tunnels

5.2.1.7 Describe, explain and discuss the concept of integrated support in tunnels

5.2.1.8 Describe, explain and discuss various support strategies for :

- Return Airways
- Haulages, Crosscuts
- Travellingways
- Winzes, Raises
- Tunnel intersections
- Tunnels traversing faults, Tunnels traversing dykes

back

- Trackless mining declines in stopes

5.2.1.9 Describe, explain and discuss the strategies to support tunnels under the following conditions :

- Low stress with joint controlled behaviour
- High stress where stress fractures dominate stability
- Changing stress cycles and environments over time
- Changing spatial stress environments
- Seismic and rockburst conditions

5.2.1.10 Determine appropriate support strategies for tunnels for given layouts, rockmass conditions, stress regimes and geological circumstances.

5.3. SUPPORT STRATEGIES FOR SHAFTS AND LARGE EXCAVATIONS

5.3.1. SUPPORT STRATEGIES FOR VERTICAL SHAFTS

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

5.3.1.1 Determine appropriate support strategies for vertical shafts for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.3.2. SUPPORT STRATEGIES FOR INCLINED SHAFTS

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

5.3.2.1 Determine appropriate support strategies for inclined shafts for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.3.3. SUPPORT STRATEGIES FOR ROCKPASSES

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

5.3.3.1 Determine appropriate support strategies for rockpasses for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.3.4. SUPPORT STRATEGIES FOR LARGE CHAMBERS

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

5.3.4.1 Sketch, describe, explain and discuss the various behaviour of rock around large excavations at shallow, intermediate and great depth

5.3.4.2 Describe, explain and discuss how this affects large excavation support requirements at shallow, intermediate and great depth

5.3.4.3 Describe, explain and discuss rockmass classification schemes for the determination of support strategies for large chambers

5.3.4.4 Describe, explain and discuss the application of the rockwall condition factor methodology to design support for large excavations at depth

5.3.4.5 Apply the rockwall condition factor (RCF) methodology to design support for large excavations at depth

5.3.4.6 Apply the Wiseman methodology to design support for large excavations at depth

5.3.4.7 Describe, explain and discuss the effects of excavation sequence on support installation

back

5.3.4.8 Determine appropriate excavation sequences to facilitate support installation

5.3.4.9 Describe, explain and discuss the effects of excavation sequence on support effectiveness

5.3.4.10 Determine appropriate excavation sequences to facilitate support effectiveness

5.3.4.11 Describe, explain and discuss the rules of thumb for determining the length and spacing of tendons in large excavations

5.3.4.12 Apply these rules of thumb to the design of support in large excavations

5.3.4.13 Describe, explain and discuss the effects of overstoping and understoping on the stability and support requirements of off-reef excavations

5.3.4.14 Determine appropriate support strategies for large chambers for given mining and shaft layouts, rockmass conditions, stress regimes and geological circumstances.

5.4. SUPPORT DESIGN CRITERIA

5.4.1. STOPE SUPPORT DESIGN CRITERIA

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

5.4.1.1 Describe, explain and discuss stope support requirements in terms of :

- Initial stiffness, Yieldability
- Areal coverage, Area of influence, Beam reinforcement, Suspension
- Support resistance, Energy absorption

5.4.1.2 Describe, explain and discuss how support resistance varies for static and dynamic conditions

5.4.1.3 Calculate required support resistances for static conditions

5.4.1.4 Calculate required support resistances for dynamic (rockburst) conditions

5.4.1.5 Calculate support loads for support systems made up of various units

5.4.1.6 Calculate energy absorption for support systems made up of various units

5.4.1.7 Calculate required support resistances and energy absorption for complete mining cycles to ensure minima are always achieved throughout the cycle

5.4.1.8 Consider the effects of variability of support unit quality

5.4.1.9 Consider the effects of variability of support unit installation.

5.4.2. TUNNEL, CHAMBER AND SHAFT SUPPORT DESIGN CRITERIA

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

5.4.2.1 Describe, explain and discuss excavation support and rockwall reinforcement requirements in terms of :

- Initial stiffness, Yieldability
- Unit length, Areal coverage, Area of influence, Beam reinforcement,
- Suspension
- Support resistance, Energy absorption

5.4.2.2 Describe, explain and discuss how support resistance varies for static and dynamic conditions

back

back

5.4.2.3 Calculate required support resistances for static conditions
5.4.2.4 Calculate required support resistances for dynamic (rockburst) conditions
5.4.2.5 Calculate support loads for support systems made up of various units
5.4.2.6 Calculate energy absorption for support systems made up of various units
5.4.2.7 Consider the effects of variability of support unit quality
5.4.2.8 Consider the effects of variability of support unit installation.

5.5. SUPPORT UNIT AND SUPPORT SYSTEM TYPES AND CHARACTERISTICS

5.5.1. PILLARS AS STOPE SUPPORT

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

5.5.1.1 Sketch, describe, explain and discuss the meaning, application and behavioural characteristics of :
- Rigid pillars, Yield pillars, Crush pillars

5.5.1.2 Describe, explain and discuss the factors that should be taken into consideration when designing a pillar support system
5.5.1.3 Describe, explain, discuss and account for the limitations in determining the strength of hard rock pillars
5.5.1.4 Describe, explain and apply empirical formulae to derive the strength of hard rock pillars
5.5.1.5 Sketch, describe, explain and discuss the role of loading system stiffness in the stability of pillar systems once pillars have exceeded their peak strength
5.5.1.6 Describe, explain and discuss the mechanism of foundation failure of pillars
5.5.1.7 Describe, explain and discuss the factors that affect foundation failure of pillars
5.5.1.8 Design in-stope pillar support layouts for :
- Breast mining, Up-dip mining.

5.5.2. BACKFILL AS STOPE SUPPORT

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

5.5.2.1 Sketch, describe, explain and discuss the main types of backfill used in tabular, hard rock mining support
5.5.2.2 Sketch, describe, explain and discuss the preparation, reticulation and placement of the main types of backfill used in mining support
5.5.2.3 Sketch, describe, explain and discuss the applications of the main types of backfill used in mining support
5.5.2.4 Sketch, describe, explain and discuss the important backfill material characteristics that affect performance
5.5.2.5 Sketch, describe, explain and discuss the important backfill placement characteristics that affect performance
5.5.2.6 Describe, explain and discuss the backfill characteristics required for filling flat-dipping narrow tabular stopes at depth
5.5.2.7 Describe which backfill types are suitable for filling flat-dipping narrow tabular stopes at depth
5.5.2.8 Describe, explain and discuss the backfill characteristics required for filling shallow wide-reef tabular stopes
5.5.2.9 Describe which backfill types are suitable for filling shallow wide-reef tabular stopes
5.5.2.10 Describe, explain and discuss how backfill assists in improving the stability of rockwalls at the face

back

5.5.2.11 Describe, explain and discuss how backfilling fits into the mining cycle

5.5.2.12 Calculate the support resistance of backfill given the backfill performance characteristics and the stope closure rate

5.5.2.13 Evaluate potential benefits of backfilling for given rockmass conditions and mining situations

5.5.2.14 Determine appropriate backfill types for given rockmass conditions and mining situations

5.5.2.15 Determine appropriate placement procedures for given rockmass conditions and mining situations.

5.5.3. PACK AND ELONGATE SUPPORT IN STOPES

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

5.5.3.1 Describe, explain and discuss the stress-strain behaviour of timber :
- When compressed parallel to the grain
- When compressed perpendicular to the grain

5.5.3.2 Describe, explain and discuss the effect of loading rate on the stress-strain behaviour of timber

5.5.3.3 Describe, explain and discuss the effect of moisture content on the stress-strain behaviour of timber

5.5.3.4 Describe, explain and discuss the effect of construction and installation on the stress-strain behaviour of timber

5.5.3.5 Sketch, describe, explain and discuss typical load-deformation characteristics of the following support units :
- Mine poles, Profile props, Pipe-sticks
- Mechanical face props, Rapid-yielding hydraulic props
- Cluster sticks
- Other commonly available elongate support units
- Timber packs, Timber-Cement (Composite) packs
- Cement-based packs

5.5.3.6 Describe, explain and discuss prestressing devices and systems in terms of :
- Their effects on the above support types
- Their limitations

5.5.3.7 Sketch, describe, explain and discuss the principles of operation of rapid yielding hydraulic props

5.5.3.8 Evaluate the various types of support units in terms of given mining environments, rockmass conditions and stress regimes

5.5.3.9 Determine the support resistance of support units and support systems at various distances from the face for various support layouts

5.5.3.10 Determine appropriate support designs for given mining environments, rockmass conditions and stress regimes.

5.5.4. SUPPORT AND REINFORCEMENT OF TUNNELS

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

5.5.4.1 Apply the various rockmass classification systems to select appropriate support for tunnels in shallow, intermediate and deep mines

5.5.4.2 Apply the rockwall condition factor (RCF) to select appropriate support for tunnels at depth

5.5.4.3 Apply other appropriate criteria to select appropriate support for tunnels at depth

5.5.4.4 Describe and discuss the following rockwall support types :

- Mechanically anchored bolts, Cable bolts, Friction 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

5.5.4.5 Characterise the following aspects of the above units :

- 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

5.5.4.6 Design and evaluate the use of appropriate support units, support systems, support patterns and installation procedures for given rockmass conditions, stress regimes and mining layouts.

5.1. STOPING EXCAVATION SUPPORT STRATEGIES

Supporting of stope excavations in the underground mining environment has become one of the biggest challenges facing the extraction of ore in the mining industry. Rock behavior changes as mining progresses from surface to depths of 3 to 4km below surface due to ore extraction in varying rock types with different properties in different loading environments, and these challenges need to be addressed to create a safe and profitable working environment.

Following on the design of good mine layouts and regional support systems, and the implementation of sound strata control practices, stope support is the final strategy to combat the hazards of rock falls and rock bursts. The primary function of stope support is to stabilise the rock mass immediately surrounding stoping excavations, that is, the zone of solid (except for geological breaks) rock around shallow workings or the zone of fractured rock around intermediate and deep stopes.

LEARNING OUTCOME 5.1.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE VARIOUS BEHAVIOUR OF ROCK AROUND EXCAVATIONS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

CONNECTION 154

Also read LEARNING OUTCOME 3.1 on page 62

The behavior of the rock mass changes significantly as mining progresses from shallow to deep level conditions. At shallow depth, the rock material tends to behave within its elastic boundaries (i.e. no fracturing), but the rock mass is usually affected by the presence of geological structures and the inelastic behaviour that take place along those structures are

back

often critical to excavation stability. The presence of joints, low strength infilling, domes, bedding etc all directly influence the stability of the rock in and around excavations.

Although stress fracturing is usually absent in these environments, since stress levels are usually well below the strength of the rock, some shallow depth environments are prone to stress induced fracturing, especially if horizontal stresses are high enough to induce fracturing of anisotropic material with lower strengths when stressed horizontally compared to its strength when stressed vertically.

EXAMPLE

Fracturing of Norite in the Bushveld Complex by horizontal stress levels that are not high, but are high enough to induce horizontal stress fracturing in excavation hangingwalls, resulting in 'arch' shaped excavation hangingwalls



Table 1: Horizontal stress fracturing in shallow mining environments

As mining progresses deeper, stress levels increase to above the strength of the rock and stress fracturing commences around mining excavations. This fracturing is most pronounced in the excavation sidewalls, as the major principal stress at depth is usually vertical. Although still present, the impact of geological structures tends to become less important (not unimportant though) to excavation stability in this environment, as rock mass stability is affected by high stress levels.

back



Figure 1: Typical sidewall stress fracturing as it occurs at depth

Changes in stress conditions to which mining excavations are subjected, occur as the depth of mining increases from shallow to ultra depths. These stress fields, as well as the excavation shape and dimension result in specific stress distributions around the excavations. For example:

- principal stresses in the immediate hanging wall may be completely tensile in shallow stopes and in a discontinuous rock mass this can lead to major instabilities
- while in deeper stopes, principal stresses in the hangingwall may all become compressive.

Each of these stress conditions significantly influences the behavior and stability of the usually discontinuous rock mass and would thus also influence the support requirements to maintain stability.

As mining proceeds to greater depths, the stress concentrations and strain energy stored in the rock mass increase, leading to situations where rapid and violent instabilities could occur.

The impact of mining depth on rock mass behavior has significant influence on mining strategies selected to maintain excavation stability. Figure 2 and Figure 3 explains the general mining depth environments and appropriate mining strategies applicable to the indicated mining conditions.

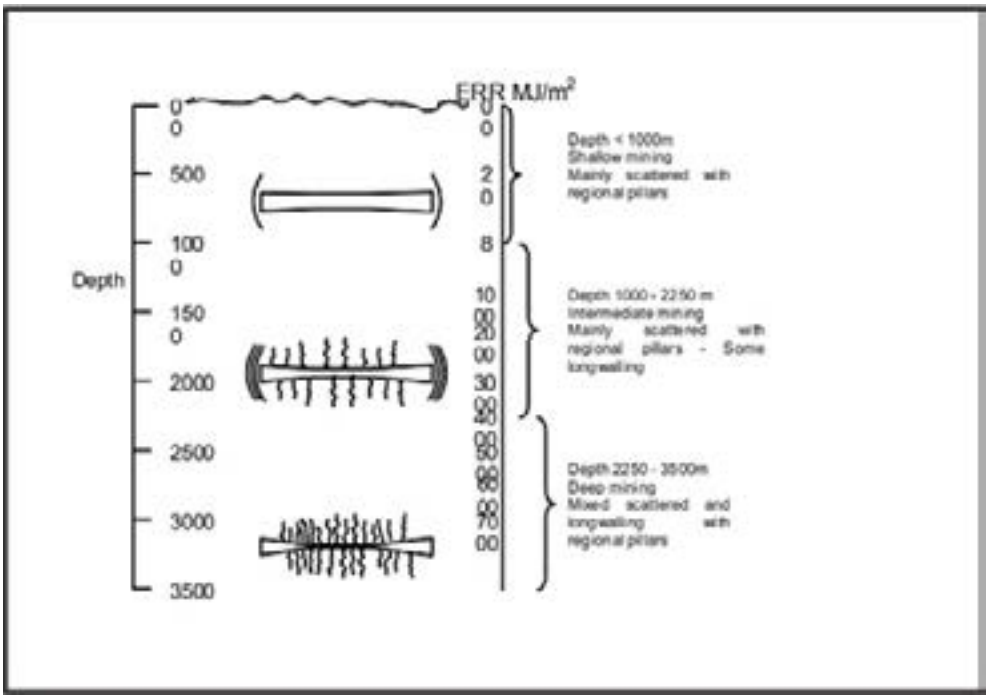


Figure 2: The influence of depth on the fracture patterns expected around a stope (Jager and Ryder, 1999)

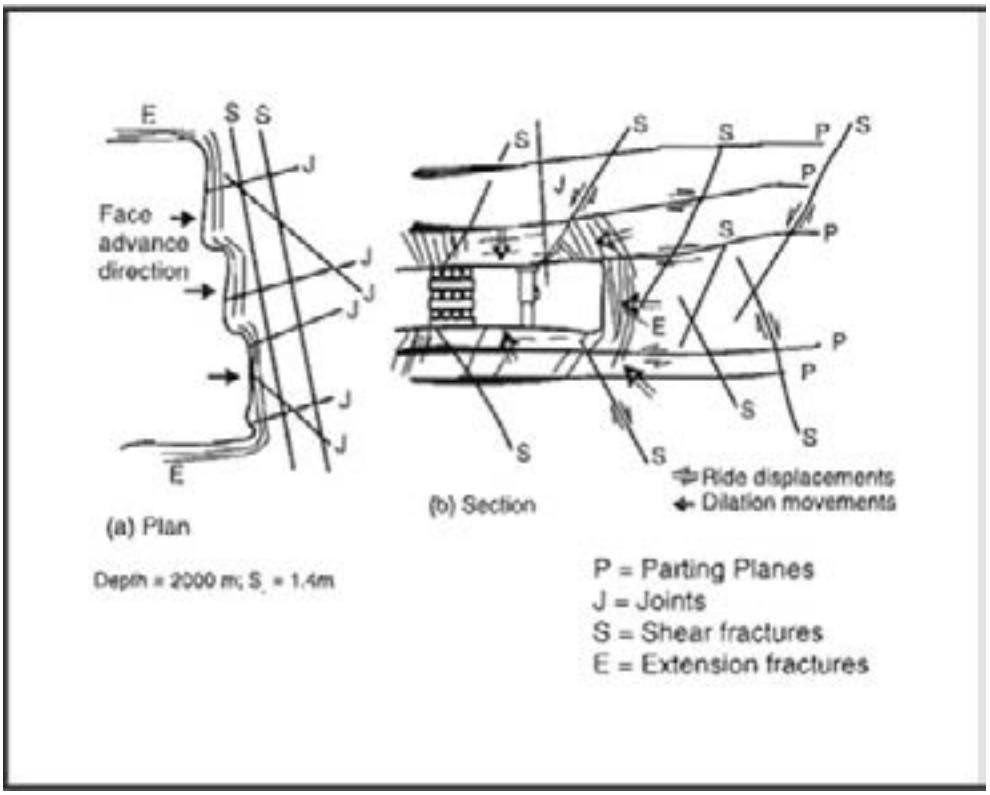


Figure 3: "Shallow", "Deep" and "Ultra-Deep" mining environments (Jager and Ryder, 1999)

REFERENCE

- Jager and Ryder page 115

LEARNING OUTCOME 5.1.2

DESCRIBE, EXPLAIN AND DISCUSS HOW THIS (5.1.1) AFFECTS STOPE SUPPORT REQUIREMENTS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Shallow Depth Environment:
Since stress levels around excavations are usually low, the stress field contributes little to rock mass stability and failure if often governed by the presence of geological structures that break the rock mass into blocks that, if allowed to displace, could lead to instability. Support design is thus aimed at preventing / limiting movement of the rock mass overburden and support design is based on the strength and load-bearing capabilities of the support units, such as pillars, elongates, grout packs and other support types which generates high loads under low levels of closure.



Overburden : The rock mass situated between the underground workings and surface.

CONNECTION 155

Refer to Outcome 2.11 on page 56 in Paper 1 to understand impact of beam thickness on beam stability.

In this usually tensile stress environment, the presence of jointing and joint infilling has a major influence on stability, and support design and layouts must be able to cater for these structures. These geological discontinuities breaks the hangingwall beam into smaller blocks that can displace along and fall out to the discontinuities if not well supported. In layered rock types (such as in sedimentary rocks), bending of the layers under gravitation leads to bed separation along the layers (could be bedding planes) and opening up of joints (as the beam bends downwards) and a reduction in stability of the blocky (due to joints), thin (due to bedding thickness), loose (due to opening-up of all contacts) layers that has to remain stable over the required mining spans. To facilitate stability by limiting the bending, bed separation on bedding planes and opening up of joints, support installation is critical.



In shallow mining situations, fall of ground incidents can involve huge dimension blocks that become unstable (as the sizes of the blocks are determined by the presence of geological structures) when allowed to displace into workings. It is therefore essential that the support strategy prevents initial rock mass movements (stiff and high loadbearing) to maintain the integrity of the hangingwall.

EXAMPLE

- The support system consisted of narrow in-stope pillars combined with elongates installed between the pillars. The pillars started failing, causing additional loading of the elongates but also succeeded in taking up all the yield on the elongates. Subsequently the whole support system failed, causing a massive collapse to occur where the total mining area closed by the fall of ground measured approximately 750m long, 300m wide and 10m high.
- Pillar robbing in a bord-and-pillar layout, where the pillar sizes were reduced by 1m on two sides, was allowed. Eventually, after the pillar size reduction took place over a large enough area, pillar failure occurred, which allowed the whole hangingwall to collapse.

It is thus essential that support requirements for this type of environment are met and that a sufficient factor of safety is built in to ensure that support failure does not occur. Stiff support units with high load bearing capability and large areal coverage in blocky conditions are required for this environment.

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EXAMPLE

- Pillars only;
- Pillars with elongates;
- Pillars with cement packs (grout packs);
- With some form of netting on the mining face.

Intermediate Depth Environment:
At intermediate depths, stress levels start to increase and could reach the strength of the rock mass, allowing stress fracturing to become more prominent. Rock mass displacement (under the higher stress levels) and the observed closure due to stress fracturing and inelastic displacements on fractures and geological structures, increases. Due to the stress fracturing, dilation of the rock within the immediate hangingwall of the stope increases horizontal clamping in the hangingwall beam, assisting in the stabilisation of the beam. At the same time, the presence of tensile stresses in the hangingwall becomes smaller as a result of this higher horizontal stress and clamping.

Since stress distributions in the immediate hangingwall beam assist in stabilisation of the hangingwall, the fall out thickness is usually related to the thickness of the layers in the hanging wall or the distance to the first weakness plane in the hangingwall, and is usually less than approximately 1.5m thick.



Large fall-out heights are not common and are mostly limited to the immediate hangingwall. However, under different conditions, this could change, such as when an area is de-stressed by another mining cut, creating 'shallow mining conditions' around this stope, even though it may be at depth.

The support system applied must now combine load bearing capacity and active loading to reinforce the hangingwall beam, with long term stability and yieldability to cater for the expected closure levels. Areal coverage and spacing of support elements becomes important if blocky ground conditions exist, usually due to stress fracturing and jointing.



Support requirements for seismically active areas are discussed under 'deep mining environments', but can also be important in some intermediate depth mines. If present, these support requirements should also be included within the section above.

Deep Mining Environment
This environment is a continuation of the intermediate depth environment, except that stress levels are usually much higher and easily exceed the strength of the rock, resulting in significant stress fracturing around all excavations. This stress fracturing then increase clamping in the immediate hangingwall beam, but also increase closure and closure rates into the excavation. The closure rates increase to such an extent that total closure occurs relatively close to the mining face position. High closure rates means that yield available in support units are taken up very quickly (close to the face) and that support units with high yield ranges are required in this environment.



Total closure: hangingwall and footwall of the stope is in contact.

High face stress levels and high levels of excess shear stress on geological features result in the occurrence of seismicity on a regular basis.

Fall out thickness is, as for intermediate depth mines, mostly dependent on the thickness of layers and intersection of stress induced fractures with joints and other planes of weakness.

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Support design is now dominated by the need to provide support in a dynamic environment, but strategies are based on static (when seismic events due to occur) and dynamic (during the occurrence of a seismic event) conditions.

Since the dynamic support design criterion is 'energy absorption', high load bearing and yielding capabilities of the support units are critical to ensure that the beam stability is maintained (active, stiff, high load bearing units) during high amounts and rates of closure (yieldability, behaviour during rapid compression).



- Jager and ryder page 115-132

LEARNING OUTCOME 5.1.3

DESCRIBE, EXPLAIN AND DISCUSS THE CONCEPT OF GEOTECHNICAL AREAS / GROUND CONTROL DISTRICTS

CONNECTION 156

Outcome 8.2 on page 588in Paper 2 material on Codes of Practice

Geotechnical areas or ground control districts are defined as areas (or portions) of the mine where similar geological conditions exists which gives a unique set of identifiable rock-related hazards for which a common set of strategies can be employed to minimize the risk resulting from mining.



In other words, if there are areas on your mine where the rock mass behaves so differently that you require different support strategies to address the associated hazards, you have different geotechnical areas that must be defined and described.

EXAMPLE

A gold mine mines on both the Carbon leader and Ventersdorp Contact reefs. The two reefs are vastly different in composition, as well in the host rock that exists around the reef horizon. It is known that the seismic behaviour of these two horizons differ with face bursts being more prominent on the VCR reef. Typically, the two reef horizons will be classified as separate geotechnical areas requiring different strategies to maintain stability and safety of personnel.

However, the intension is not only to classify different reefs as geotechnical areas but also to refine the definition and identify areas within a particular reef environment where special circumstances exist.

EXAMPLE

On a deep level gold mine, which extracts the VCR reef, the footwall rock mostly consist of competent quartzite, but a portion of the reef is underlain by less competent shale. Since 'punching' of support into the shale is possible, support elements used must utilise loadspreaders, while this is not required for the areas with quartzite footwalls.

At the same time, it is essential not to limit the concept of geotechnical areas to stoping environments, but to also select these areas within the development excavations. It is possible to separate areas where poorer footwall material is expected from competent material areas and classify these as different geotechnical areas, also applying different support strategies.

EXAMPLE

Development within shale requires shotcrete onto the face after each blast, prior to cleaning, whilst development in quartzite can utilise mesh and lacing for areal coverage purposes.

- Jager and ryder page 133

LEARNING OUTCOME 5.1.4

DESCRIBE, EXPLAIN AND DISCUSS ROCKMASS CLASSIFICATION SCHEMES FOR THE DETERMINATION OF GEOTECHNICAL AREAS / GROUND CONTROL DISTRICTS

CONNECTION 157

LEARNING OUTCOME 2.3.1.1 on page 50

Rock mass classification systems were originally designed for use in tunnel and large service excavation layout and design. However, the importance of these systems to indicate different geotechnical areas has been realized as the system allows areas of significantly different rock mass quality to be identified. From experience it is clear that poorer conditions may warrant closer support spacings and even significantly different support strategies such as areal coverage in very poor conditions.

EXAMPLE

It is known that pillar strengths are affected by the quality of the rock mass that forms the pillar material. By determining the RMR of a reef that is to be mined across the mining area (either from borehole core logging or in-situ mapping of conditions), areas of significantly different quality can be identified (Figure 4) and the pillar strengths adapted to ensure adequate pillar dimensions in poor quality areas and optimum dimensions in good quality areas.

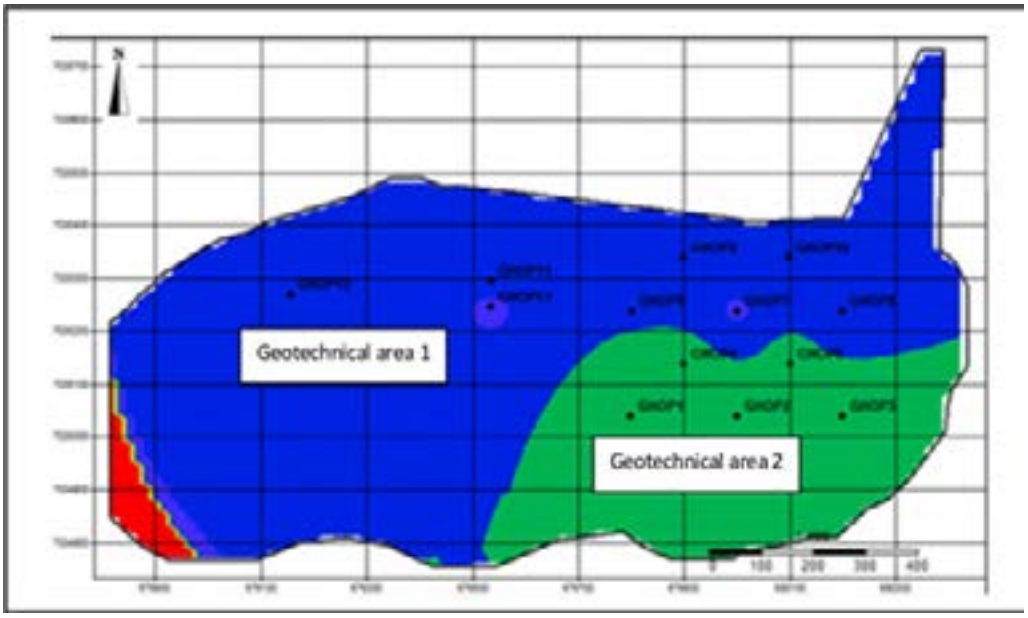


Figure 4: Geotechnical areas based on quality of the reef material

CONNECTION 158

GAP 416 Definition of Geotechnical areas

More information on the use of rock mass quality systems can be found in the research report given below.

If information contained in the noted appendices to this repott is required, download the required files from http://www.mhsc.org.za

LEARNING OUTCOME 5.1.5

DESCRIBE, EXPLAIN AND DISCUSS THE DETERMINATION OF GEOTECHNICAL AREAS / GROUND CONTROL DISTRICTS

To identify the different geotechnical areas, a detailed description of the whole mining environment is required. Typical factors that are usually addressed when in the process of identifying the Geotechnical Areas include (inter alia):

- Seismic activity: Seismic activity will indicate whether the situation is static or dynamic. With static conditions support design is based on support resistance (load bearing) alone. Under dynamic conditions, energy absorption is also taken into account, and support must have yielding capabilities according to amount of energy released during a certain magnitude of event.
- Geological Environment: Detail knowledge of the geological environment is required to classify a mine into geotechnical areas. Rock mass properties, geological discontinuities, layering and any other relevant information needs to be taken into account for this.
- Geometry of the Orebody: Factors such as mining width and dip of the orebody are some of the factors to consider in this case.
- Virgin Stress: The virgin stress is used mainly to determine the k-ratio. Any significant variations could indicate a different approach in the support loading and also mining strategy.
- Closure Rate: Closure rate have a significant influence on support performance. Backfill, for instance, is not suitable for low closure rates, as insufficient closure will not generate the required loads as part of the support strategy. Support yielding capabilities are based on the amount of closure expected to ensure that sufficient load is generated for the expected life of the support system.



When declaring geotechnical areas on a mine, it is imperative to use criteria which are measurable and controllable. This will give a definition of geotechnical areas which is quantifiable, and will address the uniqueness of the response of the area to mining.

Detailed guidelines to determining geotechnical areas can be found in the research report shown below.

CONNECTION 159

GAP 416 Definition of Geotechnical areas

If information contained in the noted appendices to this repott is required, download the required files from www.mhsc.org.za

LEARNING OUTCOME 5.1.6

CONNECTION 160

Refer to LEARNING OUTCOME 5.1.3 on page 293 through LEARNING OUTCOME 5.1.5 on page 295

DESCRIBE, EXPLAIN AND DISCUSS THE APPLICATION OF THE CONCEPT OF GEOTECHNICAL AREAS / GROUND CONTROL DISTRICTS

LEARNING OUTCOME 5.1.7

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS APPROPRIATE SUPPORT STRATEGIES TO CONTROL THE HANGINGWALL IN NARROW STOPES UNDER STATIC CONDITIONS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

Static conditions imply that small amounts of closure and a low closure rate (quasi-static) is expected during the excavation process. The support system must be able to maintain stability by exerting sufficient forces on the slow moving / static rock mass.

Support strategies are driven by the conditions within which they must maintain stability. Since the conditions have been sketched in previous outcomes, they will not be repeated in this section. However, typical stope support strategies discussed below will refer to:

- Temporary support strategies;
- Permanent stope support strategies;
- Working area;
- Back area;
- Access route support.

CONNECTION 161

Refer to LEARNING OUTCOME 5.1.1 on page 287 and LEARNING OUTCOME 5.1.2 on page 291

CONNECTION 162

GAP024 SIMRAC report: Develop guidelines for the design of pillar support for shallow to intermediate depth, tabular, hard rock mines and provide a methodology for assessing hanging wall stability and support requirements for the panels between the pillars, by A.T. Haile and A.J Jager (1995) GAP 334

	Shallow	Intermediate	Deep
Temporary face support	Active, stiff support units to ensure loadbearing in low closure face area, such as mechanical props. Areal coverage in blocky conditions by using netting hooked onto temporary support units or stope tendons.	Active, stiff support units to ensure loadbearing in low closure face area, such as mechanical props. Limited yieldability is required in higher closure areas within this category. In poor conditions netting or loadspreaders on props.	Active, stiff support units to ensure loadbearing in face area, such as mechanical props. Medium yieldability is suggested in high closure rate and seismically active areas. In poor conditions netting or loadspreaders on props.
Permanent face area support	Short tendons in hangingwall to prevent falls from joints or weak planes. Pre-stressed to prevent displacement on joints / planes.	Short tendons in hangingwall to prevent falls from joints or weak planes. Pre-stressed to prevent displacement on joints / planes.	Short tendons in hangingwall to prevent falls from joints or weak planes. Pre-stressed to prevent displacement on joints / planes.
Working area support	Active, stiff support units such as elongates to ensure loadbearing at low closure. Limited yieldability in solid pillar layouts and medium yieldability in yield and crush pillar layouts.	Active, stiff support units such as elongates to ensure early loadbearing. Medium yieldability in higher closure areas and areas where seismic activity occurs.	Active, stiff support units such as elongates to ensure early loadbearing. High yieldability is essential.
Back area support	Solid, yield or crush pillars, depending on the design of the mine layout, geology and mining depth.	Limited back area support with rows of packs at large spacings where some back area support is required. Backfill in areas where benefit of fill is required.	Backfill as back area support is beneficial to reduce overall stress levels, closure and strain energy available for seismic release.
Access routes to stope working area	Solid, yield or crush pillars, depending on the design of the mine layout, geology and mining depth. Rows of grout packs or cluster sticks.	Rows of packs or backfill along major access routes to ensure access and egress from the panel after failure of elongates.	Rows of packs or backfill along major access routes to ensure access and egress from the panel.

Table 2: Typical support strategies for shallow, intermediate and deep level mining environments



- Jager and Ryder page 172-182

LEARNING OUTCOME 5.1.8

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS APPROPRIATE SUPPORT STRATEGIES TO CONTROL THE HANGINGWALL IN NARROW STOPES UNDER SEISMIC CONDITIONS AT DEPTH

In geotechnical areas where seismicity is one of the factors delineating the area, support requirements are adapted to enable the support system to absorb energy under dynamic conditions. The support unit must be able to yield under normal closure rate conditions, but then also absorb energy under dynamic closure conditions. The support resistance offered by the unit must be sufficient to control the dynamic movement of the hanging- and foot wall under seismic conditions, while the velocity of the closure during the seismic event becomes critical in the design of the support system.

A number of timber and steel props have been manufactured which performs adequately under the dynamic conditions when a seismic event occurs. In conjunction with the timber props, backfill installed close to the face has proven to generate sufficient support resistance as well as energy absorption to prevent large scale damage as a result of seismic activity.

CONNECTION 163

Refer to Outcome 5.3 Paper 2.
Refer 4.2.2 STABILISING PILLARS on page 200

At the same time, the backfill has a substantial impact on the vibrations of the hangingwall and so affect stope stability.



Although the support system installed is very important, it must be combined with good mining practices, face layout controls, leads and lags between panels and control of mining around geological structures. Good support practices will not rectify undisciplined mining processes.



- Jager and Ryder page 172-182

LEARNING OUTCOME 5.1.9

CONNECTION 164

Refer to LEARNING OUTCOME 4.3.1.2 on page 233

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE SPECIAL PROBLEMS OF SUPPORTING WIDE REEF STOPES AT SHALLOW, INTERMEDIATE AND GREAT DEPTHS

Wide reef mining (usually viewed as wide reef when the mining height is in excess of 2.5m) poses special challenges to supporting the hangingwall of a stope when standard (conventional) mining methods in tabular orebodies are applied. Many of these issues are practical, but it also brings some design concerns:

1. Practical issues:
   - Installation of bolting at excessive heights becomes a problem due to access and requires special equipment;
   - Support units can become unstable and pose a hazard themselves;

- Drilling and blasting becomes more cumbersome as more rows of holes are required;
- Large amounts of lasted ore must be removed, increasing the time required to clean and access the face, allowing time dependent loosening to occur before making safe or support installation can occur;
- Making safe becomes difficult especially due to potential for person making safe to stand below the rock to be barred;

2. Design issues:
   - Width to height ratios of support units becomes a problem, as loadbearing and stability of a unit decreases with increasing height;
   - Stiffness of support units reduce as their height increases;
   - Loadbearing of support units must be downgraded to ensure that loads generated are not overestimated;
   - Stability of tall support units (buckling) must be considered if closure rates increase;

To counter the concerns with the conventional mining, other mining layouts have been developed and include adapted conventional, pillar mining and massive mining methods.



Conventional mining: Creation of a stope face that is advanced through the use of jackhammer drilling and blasting and (usually) scraper cleaning methods and where support utilised include pack, elongate and tendon type units.

Pillar mining: Where pillars form the main support method of stopes, even though tendon type support can be utilised to support the (usually) small span openings created between pillars. These include bord-and-pillar (square or rectangular pillars surrounded by nmined out bords) and rib pillar (long continuous pillars between drives) layouts.

Massive mining methods: Include methods such as drift-and-fill, where small span drives are mined and filled with free-standing backfill that will allow the mining of the next drive against the fill. During mining of the drives, stability is maintained by using tendon support types.

Shallow mining conditions:
At depths down to 1200m, pillar mining is the most commonly used method of extracting wide reefs (normally less than 6m heights). Different methods of extraction are available, such as bord-and-pillar, rib pillar and drift-and-fill mining. The use of barrier pillars between mining areas is often used to prevent large scale instability and collapse, but they also assist in the reduction of in-stope pillar sizes. Support strategies include tendon type support units in the bord or drive hangingwalls while free-standing backfill placed in drives ensure overall stope stability and may even allow for the extraction of the rib pillars at a later stage.

Intermediate depth mining conditions:
Partial extraction using pillars as support is not generally practiced at depths greater than 1500m below surface. Conventional, multi-cut mining is an option at these depths and can be achieved by taking a first narrow cut (similar to standard narrow reef mining) which is then followed by a second cut to the full reef width (Figure 5).

Support strategies for the bench can include standard tendon type (roofbolt in Figure 5) as well as standard narrow reef support units (mechanical props, RYHPs, elongates). However, support types for the full width second cut is more difficult due to all the concerns mentioned at

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the start of this outcome. The use of conventional support types is not impossible, especially when the final height is till manageable to approximately 3.5m. However, if packs are used, their widths should increase to maintain acceptable width:height ratios, while cluster sticks are essential if elongates are installed. However, the use of cemented backfill is probably the ultimate, providing good hangingwall contact is achieved on placement and shrinkage is minimized or prevented.

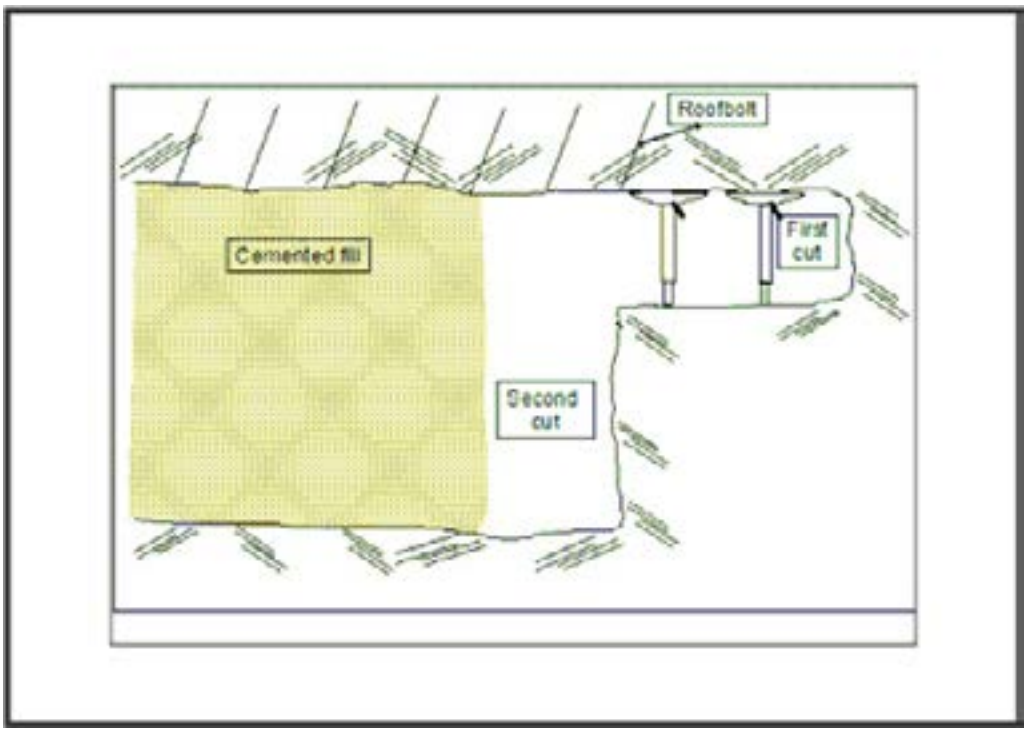


Figure 5: Section through a double-cut mining panel showing layout and support

The lead / lag between the 1st and 2nd cuts are critical, as a too short lead will create an unstable bench wall (stress fracturing will create unstable bench), and a too large lead will increase the face to permanent support / fill distance and reduce the impact these support units might have on stability. At the same time support units installed on the bench might start to yield and fail before the permanent support / fill can be installed / placed, increasing the possibility of hangingwall failure between the cuts.

Deep mining environment:
Wide reef mining in the deep level environment increases the challenges to acceptable support design as:

- The use of multi-cuts is possible, but the 'large' span between the stope face and the permanent support (at full width) increases potential for failures during seismic events.
- Elongates can become unstable at high closure rates and during seismic events if installed at these heights;
- Packs do not generate sufficient support resistance because of the increased "softness" of the unit (due to height) and the increased potential to buckle under higher closure rates or during seismic events;
- Backfill has been used successfully when cemented.

REFERENCE

- Jager and Ryder page 185
- Budavari page 142

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EXAMPLE

The mining of a very wide orebody is being successfully done at depth by implementing a mixture of the methods defined earlier in this outcome:

- A de-stressing cut is mined on narrow reef support systems, following conventional mining methods;
- In the de-stressed 'shadow' of the abovementioned cut, a massive mining methods is applied (drift and fill) with cemented fill placed in the mined out drifts.

LEARNING OUTCOME 5.1.10

CONNECTION 165

Refer to LEARNING OUTCOME 4.3.1.2 on page 233Refer to LEARNING OUTCOME 5.1.9 on page 297

DESCRIBE, EXPLAIN AND DISCUSS APPROPRIATE STRATEGIES TO ENSURE STABILITY UNDER WIDE REEF CONDITIONS

LEARNING OUTCOME 5.1.11

CONNECTION 166

Refer to LEARNING OUTCOME 4.3.1.2 on page 233

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE SPECIAL PROBLEMS OF SUPPORTING MULTIPLE, CLOSELY SPACED REEF STOPES AT SHALLOW, INTERMEDIATE AND GREAT DEPTHS

Multi-reef mining is not uncommon on the South African context of gold, platinum, chrome or manganese reefs / seams. History shows that mining of a large number of multi-seam environments has been done successfully at a large variation of middlings, some even at middlings as low as a few metres.



The middling between reefs / seams is normally viewed as the perpendicular distance between two seams / reefs, even though the vertical distance is often used for design purposes.

EXAMPLE

Examples of multi-reefs historically mined:

- VCR and Carbon Leader reef
- Vaal and C-Reefs
- Basal and Leader reefs
- Elsburg reefs
- Kimberley reefs
- Main, Main Reef leader and south reefs
- Merensky and UG2 reefs
- MG Chrome reefs
- LG Chrome reefs
- Manganese lower and upper seams

The critical issues with regards to multi-reef mining concerns are briefly listed in Table 3

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Issue	Concern
Middling	Two possible concerns exist during multi-reef mining, that of middling instability (i.e. the middling fails) and that of stress interaction between reefs that impact on face or pillar stress levels (seismicity or pillar failure). The most probable concern for any specific application depends on some of the other issues discussed in this table.
Rock material within middling	Middling instability indicates the loss of integrity if the rock between the reefs and may occur in tension (bending) or shear (high vertical or horizontal stress levels) and result in the collapse of the middling. This failure is a function of the stress field (tension, horizontal compression, etc), rock material (jointing and their orientation, shear strength of joints, bedding thickness and tensile strength or cohesion, etc) and the middling. Massive, competent rock types forming the middling will allow mining at lower middlings than broken up, bedded materials (given the same stress fields).
Mining depth	Mining depth directly influences the stress field around the reefs being mined with known higher k-ratios in shallow depth and higher k-ratios at greater depth. This stress field is altered by the mining of the reefs orientations of the principal stresses can change dramatically. As mentioned above this stress field has a role to play in the possible failure of the middling while stress increases on face and pillar stresses will probably manifest in stress fracturing and orientation of fractures, possibly face bursts and seismic events on intersected geological structures (reactivated) and pillar failure (loaded beyond the design).
Order of extraction Jager and Ryder page 186	Order of extraction plays a significant role in the stability of the mining conducted on the individual reefs. Subsequent extraction (one after the other) could affect additional loading on existing pillars on the first reef which has to be included in the original design, while remnants left on the first could affect mining of the second close to this remnant. At the same time, depending on mining spans, the second reef could mine in de-stressed (possible tensile) stress field, affecting hangingwall behaviour. Stress fractures created around the first reef may also be intersected by the second reef mining at low middlings. Simultaneous extraction could see the same interactions as those above, but where the stress field is constantly changed on both reef elevations. The lead / lag between faces on the individual reefs is critical as it affects stress levels and closure profiles on the reefs.
Methods of determining stress interaction between reefs Coal Mining (Salamon, page ???)	Although a basic method to determine the middling at which stress interaction is possible has been derived for the Coal Mining environment, back analyses and local knowledge is mostly used in the hard rock environment. Numerical modelling is viewed as absolutely essential in determine the possible levels of interaction on stress levels, middling failure and closure profiles.

Table 3: Multi-reef concerns



- Jager and Ryder page 186
- Budavari page 143

LEARNING OUTCOME 5.1.12

CONNECTION 167

Refer to LEARNING OUTCOME 4.3.1.2 on page 233

DESCRIBE, EXPLAIN AND DISCUSS APPROPRIATE STRATEGIES TO ENSURE STABILITY UNDER MULTIPLE, CLOSELY SPACED REEF CONDITIONS

Based on the issues discussed in LEARNING OUTCOME 5.1.11 on page 300, the support strategies for the different conditions (when interaction is expected) are summarised in Table 4.

Extraction sequence	Depth environment	Support strategies
Subsequent extraction	Shallow	Support when mining the lower reef second should be able to protect the middling by being able to carry the deadweight of the middling material. For this purpose pillars at small spans or stiff, high loadbearing support such as backfill or grout packs are required. When mining the upper reef second, re-support or backfilling of the lower reef is required to maintain the stability of the middling and avoid sagging of the middling and falling out of support units on the upper reef. Tendon support in the upper reef hangingwall can assist if middling stability is uncertain
	Intermediate	Mining the lower reef may require remnant precautions when remnants or pillars left on the first reef are approached. The density and orientation of stress fractures intersected may require areal coverage, reduction in support spacings and the use of in-stope tendons. Possible regeneration of seismicity on the second reef will require designs for seismic conditions. Mining on the upper reef will encounter the same conditions as in shallow mining environments, requiring tendon support and possible high areal coverage support systems.
	Deep	In deep level mines, remnants are not that common, but regional pillars are left intact to limit closure and stress levels and must be catered for when mining the subsequent reef as stress levels will be high and seismicity probable. Stringent support systems such as those derived for highly stressed areas and seismically active areas should be used in both reefs. Backfill is extremely beneficial for middling stability and the reduction of face stresses and stope stability during seismic activity.
Simultaneous extraction	Shallow	Support on lower reef must be able to support the deadweight of the full middling, including pillars and backfill as appropriate support systems. When panels on different reefs are carried in line with each other, support must be stiff to cater for low closure as with all shallow mining environments. When the upper reef panels lead the lower, support on the upper will be as for normal shallow mining but that on the lower must be able to maintain a stable middling and include systems such as backfill and pillars. When the lower leads the upper, backfill and pillars to protect the middling and allow extraction of the upper, is critical.
	Intermediate	The trailing reef, independent of which reef leads, will be destressed by the mining of the first reef, but support strategies should cater for intersection with stress fractures and be installed as reduced spacings and include stiff support such as tendons or elongates with headboards so that areal coverage is increased. Support in the lower reef should be able to stabilise the middling using elongates and backfill. If the lower reef leads and backfill is not available to stabilise the middling, pillars and high loadbearing support such as grout pillars are required. All support systems should be designed keeping seismic activity in mind.
	Deep	

Table 4: Support strategies for multi-reef environments



- Jager and Ryder page 186

LEARNING OUTCOME 5.1.13

CONNECTION 168

Refer to "LEARNING OUTCOME 3.1.6" on page 301 and "LEARNING OUTCOME 3.1.7" on page 301 for conditions around gullies.

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS HOW GULLIES SHOULD BE SUPPORTED AT SHALLOW, INTERMEDIATE AND GREAT DEPTHS

Support installation in and around gullies, at all mining depths, have a single aim only and that is to maintain the gully, as the major access to the producing face area, stability. The support installation, although strategies may vary (Outcome 5.1.14), is based on the expected conditions (fracturing, jointing, etc) and environment (low stress, high stress, seismic).

In spite of the variable conditions and environments some general gully layout and support practices include:

a. Prepare the gully layouts on a plan, taking cognisance of the following:

- Siding depths and position behind the face (in-line with face is optimum);
- Gully depth (too deep create unstable sidewalls, too shallow result in breaking of the gully hangingwaal during mining to create depth);
- Line up the gully in the correct direction (get survey lines installed);

b. Development method:

- Footwall lifting is preferred above advance development of the gully as it ensures more stable fracture orientations and reduces gully sidewall blast damage;

c. Support:

- Support installed on the gully edge should be stiff and high load-bearing but not such that the loads generated damage the gully sidewalls, so yielding at acceptable loads is preferred in deeper environments;
- Consider the conditions of the gully hangingwall and sidewalls for support requirements, both in shallow (but jointed) and deeper (fractured) environments;
- Support installation must be as close as possible to the face as closure is low in this area and it is beneficial to stability if support units get the benefit of all closure that occurs to generate load quickly;
- Support quality must be high to ensure that full benefit of loads generated by the gully support is achieved;
- Footing on which support is constructed must be sound and stable to generate the required loads.



- Jager and Ryder page 167
- Budavari page 146

LEARNING OUTCOME 5.1.14

CONNECTION 169

Refer to LEARNING OUTCOME 3.1.6 on page 71 and LEARNING OUTCOME 3.1.7 on page 75 for conditions around gullies.

DESCRIBE, EXPLAIN AND DISCUSS APPROPRIATE SUPPORT STRATEGIES TO ENSURE GULLY STABILITY IN VARIOUS LAYOUTS AT SHALLOW, INTERMEDIATE AND GREAT DEPTHS

Following knowledge and understanding of the possible conditions around gullies, support systems in and around gullies are aimed at addressing those conditions. The general support precautions utilised in the industry are given in Table 5.

CONNECTION 170

GAP602 on gully design and stability by A R Leach, K Naidoo and D Spencer, gives a design and support methodology for gully stability.

Support position	Need for support installation	Support strategy
Gully hangingwall	Support must stabilise the area above the gully to protect access into the stope against falls of ground from joints, stress fractures, faults and bedding planes.	Poor conditions due to poor gully layouts and practices will require increased tendon density and the application of areal coverage. Tendons are installed into the gully hangingwall and can consist of resin or cement grouted bars, pre-stressed end-anchor tendons, splitsets and areal coverage systems such as sets, shotcrete, thin skin liners or mesh and lacing.

Support position	Need for support installation	Support strategy
Gully edges	Maintain small spans across the gully to ensure stability of the hangingwall over the access to the stope.	Shallow: Cluster sticks or grout packs. Support must be stiff and generate high loads at small closure. Yieldability is not a great concern until depths approaching intermediate where crush pillars are used. Seismic related designs are usually not required.
		Intermediate and deep: Timber and/or cement Packs of larger dip (long axis) than strike dimension to ensure that gully sidewall scaling do not render whole pack useless. Stiff, high loadbearing support units with substantial yield ranges are required. Seismic conditions require the designs to include energy absorption criterion in design.

Table 5: General support strategies in and around gullies



Attempts at using backfill only as support on gully edges have not been successful to date.

- Jager and Ryder page 167 and 172-182

LEARNING OUTCOME 5.1.15

CONNECTION 171

Stope layouts and gully fracturing are discussed in LEARNING OUTCOME 3.1.6 on page 71, LEARNING OUTCOME 3.1.7 on page 75 and LEARNING OUTCOME 4.3.1.2 on page 233

DESCRIBE, EXPLAIN AND DISCUSS THE SPECIAL PROBLEMS OF SUPPORTING GULLIES RELATIVE TO VARIOUS STOPE LAYOUT AND FRACTURE CONFIGURATIONS AT SHALLOW, INTERMEDIATE AND GREAT DEPTHS

Depth	Gully support problems
Shallow	Spans across gullies are often stretched by poor gully development, causing gully support to be installed too far apart and reducing the impact of the support in controlling the stability of the gully hangingwall. Even in shallow workings, fracturing may occur in pillars that are often orientated into the gully hangingwall at low angles. These hazards should be realised and tendon support installed wider than the planned gully width should be considered. Spans in the gully / face intersection area must be controlled and limited to assist stability in this vulnerable area. The creation of large blocks by geological structures and the possibility of these blocks sitting down on support is enhanced by the fact that stress levels are mostly low and support is fully responsible for maintaining stability. Support units must therefore be high loadbearing and / or the use of long anchors in large span areas (such as around tips) should be considered.
Intermediate	If gully layout is correct, stable conditions are found and normal gully support approaches are followed. However, if the layout is not correct stress fracturing at shallow dips are created in the gully hangingwall and sidewalls which will require substantial support installation. Sidewall fracturing can cause gully packs to be rendered useless if allowed to scale. The installation of long axis packs can limit this scaling unless the packs are too stiff and they assist in pushing out the sidewall. Spans in the gully / face intersection area must be controlled and limited to assist stability in this vulnerable area.
Deep	

Table 6: Special problems with supporting of gullies



- Jager and Ryder page 167

LEARNING OUTCOME 5.1.16

CONNECTION 172

See LEARNING OUTCOME 3.1.9 on page 77, L EARNING OUTCOME 3.1.10 on page 79

DETERMINE THE REQUIRED SUPPORT REACTION FOR STATIC CONDITIONS GIVEN TYPICAL FALL OF GROUND DIMENSIONS AND LOADING CONDITIONS

For static conditions (pure rock fall control), the support resistance of the support system has to be sufficient to carry the deadweight of the strata in the potential fall. This can be calculated by using the following formula (after Jager, et al.):

$$SR \geq (\rho.trf).g \ (N/m^2)$$

Where ρ is the density of the hanging wall rock (e.g. 2750 kg/m3 for quartzite), g = 9.8 m/s2, and trf is the thickness (m) of the potential fall. This approach is somewhat conservative, in the sense that cohesion and frictional forces on the boundaries of the potential fall are deemed negligible and ignored.

The thickness parameter trf needs to be determined for each ground control district, by taking the average thickness of fall outs which has occurred in the area as norm. Historical incident investigation results are analyzed to determine this, and the support resistance is designed to prevent the fall of ground which are likely to occur. Any available geological data are also analyzed to determine whether any persistent parting planes exist in the hanging wall of the stope.

NOTE

Refer to LEARNING OUTCOME 3.1.9 on page 77 *for an explanation of the 95% value that is often incorrectly used for the design.*

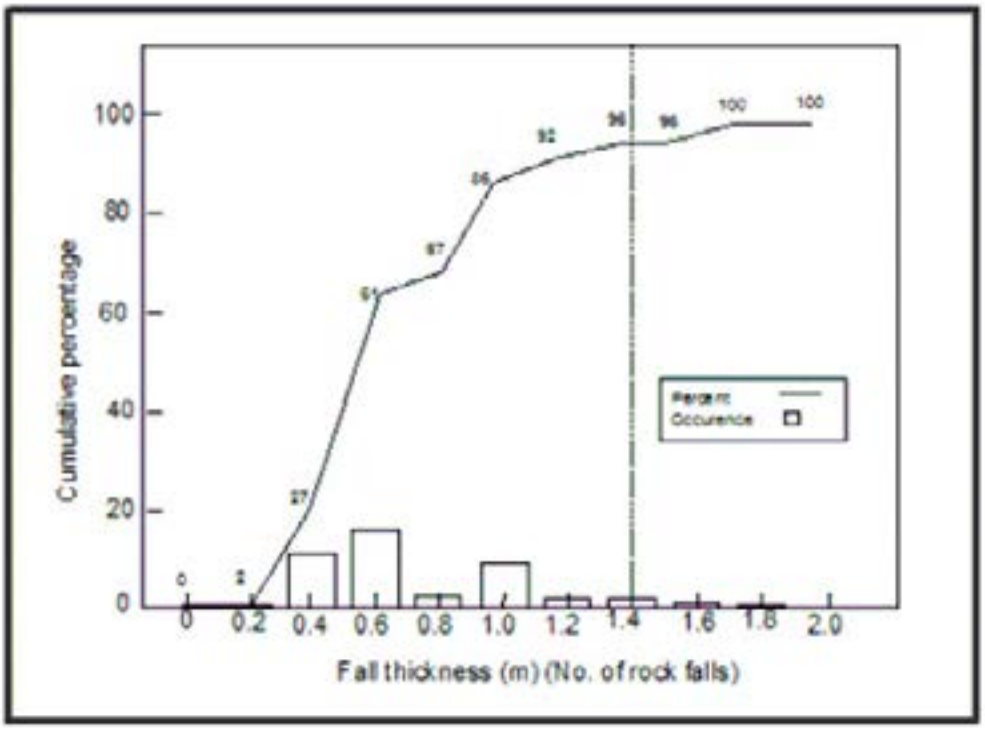


Figure 6: Cumulative rockfall thickness for a particular ground

control district (Jager and Ryder, 1999)



- Jager and ryder page 148,
- Ryder and Jager, page 307

LEARNING OUTCOME 5.1.17

CONNECTION 173

Refer to LEARNING OUTCOME 3.1.11 on page 81

DETERMINE THE REQUIRED SUPPORT REACTION FOR DYNAMIC (ROCKBURST) CONDITIONS GIVEN TYPICAL FALL OF GROUND DIMENSIONS AND LOADING CONDITIONS



Based on the information given and explained in Outcomes 3.1.9, 3.1.10 and 5.1.15 the following application is deemed appropriate. It is clear that for dynamic conditions (rock bursts and seismic activity), the support resistance theorem is enhanced by adding an energy absorption component. This means that the support element generates a certain amount of support resistance, whilst also yielding and maintaining a certain load when high ground velocities are experienced during seismic events. This support resistance behavior of a support unit subjected to a seismic event is given by SR=(1+v^2/(2gy_min))(pgh/sd) where v=peak particle velocity (m/s), ymin is the support yield range (typically less than 0.5m), g=gravitational acceleration (m/s2), h=maximum closure allowed to result in the minimum stoping width of 0.6m, s and d are support spacings (m) and ρ=density (tonnes/m3).



Although there is a limit to the amount of energy which a support system will be able to absorb, historical data has shown that ground velocities of up to 3 m/s has been controlled successfully.

Figure 7 attempts to explain this in the following manner:

- The static support resistance requirement is given by $SR=\rho.g.t_{rf}=mg$ where t_{rf} is the thickness of the rock, which remains constant;
- The dynamic support resistance is NOT constant and is a function of the loading rate or peak particle velocity (v). As the velocity increases, the SR required increases but also diminishes as the velocity decreases;
- After some closure d has occurred in the stope at low closure rates, the seismic event occurs and additional closure dmax (or h) is induced on the support;
- The potential energy absorption capacity of the support is given by the area between the static SR and the d and (d+h) lines, i.e. Area = length x width = mg x h = mgh;
- The kinetic energy absorption capacity is given by the region where the dynamic SR exceeds the static SR (coloured region) between the d and (d+h) lines;
- As the closure d that occurs prior to the occurrence of the event increases (move the d line to the right), both the potential energy and kinetic energy absorption capacities of the support reduces. This is a critical concept as it indicates that the potential for support to sustain seismic events without failing, reduces if the quasi-static closure that had occurred prior to the event increased, i.e. towards the back area of the stope, centre of the panel, old areas, etc.



See page 157 Jager and Ryder *for correct sketch.*

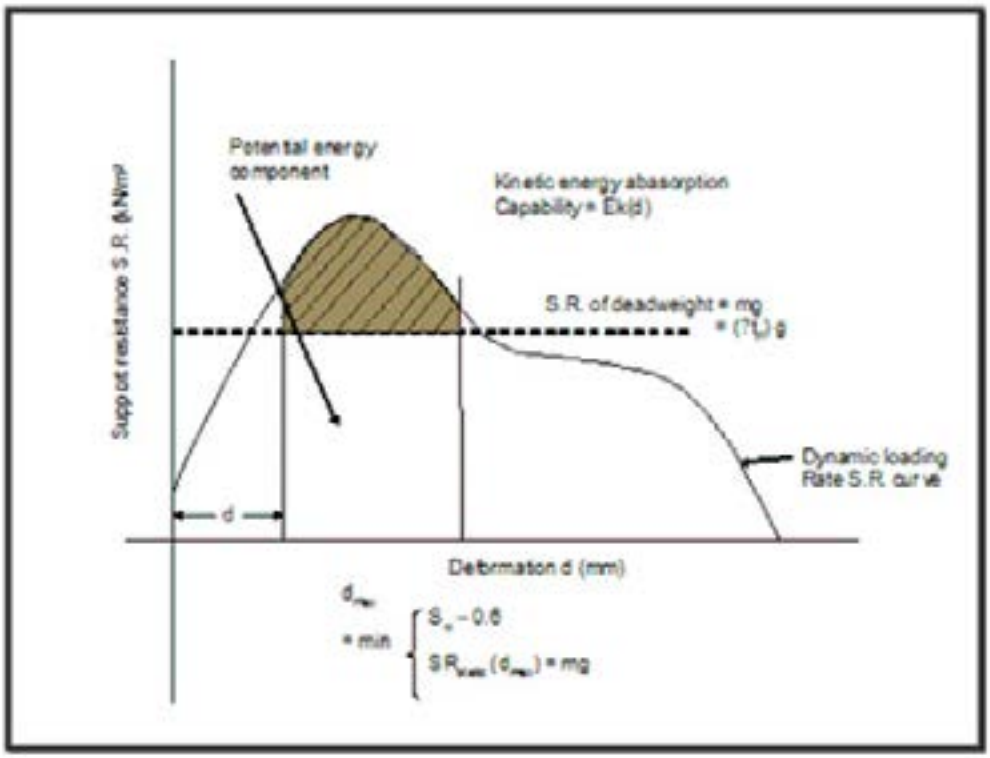


Figure 7: Derivation of energy absorption capacity of a given support system (Jager and Ryder, 1999)

- Jager and ryder page 156

LEARNING OUTCOME 5.1.18

DETERMINE APPROPRIATE STOPE SUPPORT STRATEGIES FOR GIVEN ROCKMASS CONDITIONS, FIELD STRESS REGIMES AND GEOLOGICAL CIRCUMSTANCES.

- Jager and Ryder Chapter 4 and 5

In this chapter some examples of support design are discussed, for different stress and rock mass conditions. Some examples are given of appropriate support systems for given situations. It is important to note that although general rules exist for the design of a support system, local conditions which exist in a particular geotechnical area needs to be carefully analyzed to ensure that the design criteria used is appropriate for the given conditions. As an example, if the hanging- or footwall rock is much softer than the other rock above or below the reef horizon, special adaptations to the support needs to be done to prevent excessive yielding when loads become large on the support elements. Punching into the host rock can result in excessive yielding, and the hanging wall may collapse when closure allows fractures or joints in the hanging wall to open up.

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PAPER 3.1: CHAPTER 5

EXAMPLE

Answer the questions that follow with reference to the summarised results of geotechnical logging of 42mm diameter drill core provided in the table below.

Hole ID	Q Ratings			Distance to slickensided hangingwall horizontal parting (m)	Orebody Point load strength (kN)
	Hangingwall	Orebody	Footwall		
A1	2.86	11.32	39.41	2.54	6.2
A2	2.54	10.86	38.42	3.23	6.4
A3	4.10	12.64	40.02	3.63	6.7
A4	4.32	12.32	37.64	2.67	6.3
A5	4.54	10.96	41.01	3.24	6.6
A6	2.56	11.43	40.23	3.54	6.4

1.1 Convert and tabulate the orebody Q-ratings into adjusted MRMR values using the following information: (5)

MRMR = 9 ln(Q) + 44

Adjustments of 82% for weathering, 85% for joint orientation, and 80% for blasting effects

1.2 Using a point load factor of 22 and the equations that follow, calculate the average UCS (MPa) of the orebody assuming that no size correction is required for the core. (5)

$I_S = P/(D_e^2)$

$\sigma_c = K.I_{50}$

1.3 Determine the mean design rock mass strength (DRMS) for the orebody using the result(s) obtained from (1.1) and (1.2) and the following formula and accompanying UCS rating table: (3) (3)

DRMS = UCS x (MRMR – RUCS)/100

UCS (MPa)	164-145	144-125	124-105	104-85	84-65	44-25	24-5	4-0
Rating	14	12	10	8	6	4	2	0

1.4 Using the approach adopted by Hedley and Grant and assuming Tributary Area Loading and a mining height of 1.5m,

a. Calculate the strength of a 5m wide square on-reef pillar (5)

b. Calculate the FOS of the above pillar at a depth of 500m assuming bord and pillar workings with bord widths of 6m and a overburden density of 3 tons/m^3. (2)

1.5 The underground manager has suggested using timber elongates between pillars with a yield strength of 20 tons spaced 2.0m x 2.0m apart. Is his suggestion valid? Motivate. (5)

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Model Answer provided:

1.1

Hole ID	Orebody Q	MRMR	Weath-ering	Joint orientation	Blasting	MRMRadj
A1	11.32	65.84	0.82	0.85	0.8	36.71
A2	10.86	65.47	0.82	0.85	0.8	36.51
A3	12.64	66.83	0.82	0.85	0.8	37.26
A4	12.32	66.6	0.82	0.85	0.8	37.14
A5	10.96	65.55	0.82	0.85	0.8	36.55
A6	11.43	65.93	0.82	0.85	0.8	36.76
Average	11.59	66.04	0.82	0.85	0.8	36.82

1.2 Average Point Load Strength

= (6.2 + 6.4 + 6.7 + 6.3 + 6.6 + 6.4) kN / 6 = 6.43 kN

I_s = (6.43 x 103) / (42 x 10-3)^2
= 3 645 124 Pa
= 3.645 MPa

σ_c = (22 x 3.645 MPa)
= 80.19 MPa

1.3 DRMS = 80.19 x (36.82 – 6)/100

= 24.72 MPa

1.4 a $\sigma_s = k\, w^{0.5}/h^{0.75}$ where k = DRMS = 24.72 MPa.

σ_s = 24.72 MPa x ($5^{0.5}$ / $1.5^{0.75}$)
= 24.72 x (2.236/1.355) MPa
= 40.78 MPa

1.4 b APS = q / (1 - e) where

q = ρgh = 3000kg/m^3 x 9.817 m.s^{-2} x 500m = 14.93 MPa

and

e = (A_{total} – A_{pillar}) / (A_{total})
= (112 – 52) / 112
= 0.793 or 79.3%

APS = 14.93 MPa / (1 - 0.793) = 71.16 MPa

And Factor of Safety
= FOS = Pillar strength / pillar stress
= 40.78 MPa / 71.16MPa
= 0.57

1.5 Average parting distance

= (2.54 + 3.23 + 3.63 + 2.67 + 3.24 + 3.54) / 6
= 3.14m

Weight of parting =ρ. Volume.g
= 3000kg/m^3 x 3.14m^3 x 9.817$m.s^{-2}$
= 92 476.14 N/m^2
= 92.48 kN.m^2



Assume that volume is (thickness of parting x 1 x 1) m^3, giving 3.14m^3 on a 1m^2 base area, so that the final results is in N per m^2.

Support resistance of support = 200kN / (2m x 2m) = 50 kN/m^2

FOS = Resistance provided / resistance required
= 50kN/m^2 / 92.48kN/m^2
= 0.54

The suggestion is not valid and closer spacings and / or higher support resistance is required.

Resistance required to meet FOS of 1.5
= 1.5 x 92.48kN/m^2 = 138.72kN/m^2

Sticks at 1.0 spacing = 200 kN/m^2
Sticks at 1.5m spacing = 88.90 kN/m^2

Suggestion: Stiff breaker lines of packs to improve support resistance whilst maintaining 2.0m x 2.0m spacings.

EXAMPLE

Draw detailed load deformation curves for any two support units of your choice

1. Describe a stope support system using these units (using sketches),
2. Describe the geotechnical environment that will use the support system
3. Calculate the support resistance and energy absorption in the face area for the system (show calculations). The stope closure should be assumed to be 10 mm per day with a face advance of 15 m per month.

Model Answer provided:



1. Candidate to refer to Pages 322-325 (Ryder & Jager, 2002)
2. Candidate to refer to Pages 151-161 (Jager & Ryder, 1999)
3. Process and examples are given in both textbooks

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5.2. SERVICE EXCAVATION SUPPORT STRATEGIES

5.2.1. TUNNEL SUPPORT STRATEGIES

LEARNING OUTCOME 5.2.1.1

CONNECTION 174

Refer to 3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS on page 94 and 3.3 ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS on page 112

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE VARIOUS BEHAVIOUR OF ROCK AROUND EXCAVATIONS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

LEARNING OUTCOME 5.2.1.2

DESCRIBE, EXPLAIN AND DISCUSS HOW THIS AFFECTS TUNNEL SUPPORT REQUIREMENTS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

The stability of tunnels are governed by several factors such as stress environment, levels and changes, rock mass quality and properties, size, orientation and middling between excavations, life span, seismicity, geological structures, etc. Each of these factors, as well as combinations of them has some impact on the support requirements for a specific tunnel.

Although common impacts of these environments (shallow, intermediate, deep) on support requirements can be drafted, individual tunnels within each of these environments can be subjected to conditions usually associated with other environments and thus also support requirements. Table 7 attempts to indicate that tunnel support requirements are not necessary controlled by the depth environment, but rather the actual environment around the tunnel or parts of the tunnel.

Depth environment	Actual environment	Support requirements
Shallow	Tunnel passing underneath oversized pillar left on-reef. High stress levels, fracturing around the tunnel (especially sidewalls due to higher vertical stress component) and possibly seismic activity must be catered for.	High initial stiffness, yieldability and areal coverage. High support resistance and some level of energy absorption capacity required.
Deep	Tunnel is developed follow-behind in longwall layout. Low stress levels, even tensile and possible higher horizontal stress component causing hangingwall concerns.	Pre-stressed, high tensile strength support units to maintain integrity in tensile stress environment. Increased hangingwall support density due to possible fracturing. Set support if middling to stope is very small and all stress components are tensile.
Intermediate	Tunnel is develop through incompetent dyke material (kimberlite). Stress levels are average, vertical and significantly larger than the rock material strength. Material, due to poor quality and induced stress levels, collapses into small particles, unravelling around the tunnel.	Injection of resin to kimberlite to consolidate material and allow development to continue. Keep all spans between support and face small. Set type support to cater for deadweight of potential collapsed material.

Table 7: Tunnel support requirements for different rock mass behaviour



The field stress around the tunnels plays a major role in the instability expected and therefore, it is advised that numerical modeling is done to get an estimate of expected stress fields (levels, orientations, changes).

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Various tables exist in the literature which can assist in determining support requirements for different environments, layouts and orientations (eg. Laubscher, Jager, Stacey) and can be used to assist.
Refer to paper 2 Outcomes 3.1.2.8 onwards



- Jager and Ryder page 208

LEARNING OUTCOME 5.2.1.3

INCLUDE IN DESIGN DELIBERATIONS THE INFLUENCES OCCASIONED BY THE FOLLOWING CONSIDERATIONS:

- Gravity loading environment
- Seismic loading environment
- Rock characteristics
- Rockmass characteristics
- Geological structure, Stress mechanisms
- Stress cycle
- Rate of stress change
- Initial stress conditions
- Mining configuration
- Mining cycle, Overstoping techniques,
- Destressing techniques
- Tunnelling configuration
- Tunnelling cycle, Pre-development
- Follow behind development
- Support components
- Support systems
- Support appropriateness
- Implementation quality
- Management effectiveness
- Quality control

CONNECTION 175

Refer to LEARNING OUTCOME 5.1.16 on page 305

Gravity loading environment:
Support systems consists of different elements, each performing a specific task in stabilising the rock mass. 'Suspension' of rock on support elements refers to support units 'carrying' the weight of the rock (due to gravitational forces) it has to support. This is calculated by determining the thickness and the weight of the rock beam that has to be supported. The design value calculated through this process is referred to as the 'support resistance requirement' that has to be at least equaled by the 'support resistance offered' by the support element to maintain stability. To cater for risk, FOS of 1.2 to 1.5 (FOS= (SR offered by support)/(SR requirement)) are often used in this design process.

CONNECTION 176

Refer to LEARNING OUTCOME 5.1.17 on page 306

Seismic loading environment:
Seismicity may cause significant damage to tunnels, even though the event(s) may be at a distance from the tunnel location. Most often seismic events are caused by the stress changes inflicted by stoping

operations, either on pillars, stope faces or on major geological structures. The tunnel may have intersected or be situated close to structures / pillars / stopes, causing extensive damage to the tunnel when a seismic event occurs. Sometimes large sections of the tunnel may be damaged to such an extent that the tunnel becomes unserviceable. To be able to maintain tunnel stability during the seismic event, support has to be able to dissipate / absorb the dynamic behavior of the rock mass. This characteristic of the support is called the 'energy absorption capacity' of the support elements.



When the seismic strain wave intersects the tunnel, displacement of the rock on existing fractures or planes occur. In many cases, new fractures are also created when the strain becomes excessive. Although the 'site response' during a seismic event is not fully understood yet, it has been successfully shown that an increase in energy absorption capacity of support systems, reduces damage to the tunnel and ensure that support units are still able to perform static support functions after the event.

CONNECTION 176

Refer to3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS on page 94 and 2.1 GEOLOGY on page 37 and 2.2 ROCK STRENGTH on page 43

Rock characteristics

The characteristics of the rock in which the tunnel is placed, plays a major role in the stability of the tunnel and thus the support requirements. The rock characteristics within which the tunnel is placed and that play a role include:

- Type of rock (igneous, metamorphous, sedimentary) : origin of rock indicated probable characteristics of the rock that will lead to the next properties, but also indicate presence of bedding and / or jointing for which support must cater to ensure stability;
- Strength (compressive, tensile and shear): failure under different stress fields that support has to cater for to ensure stability;
- Behaviour under stress (elastic, plastic, ductile): amount and type of deformation expected that support will have to cater for or arrest.

CONNECTION 178

Refer to3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS on page 94 and 2.3 ROCK MASS CHARACTERISTICS on page 50

Rock mass characteristics

Characteristics to evaluate will include layering, layer thickness, state of weathering, presence and orientation of geological structures (faults, dykes, joints, bedding), infilling and ground water, all of which will influence the rock mass quality and thus the stability of an excavation. Rock mass characteristics are measured using any of the available rock mass classification systems through which the rock mass quality may be determined. The outcome of such a classification can then be used to determine stand-up time and an appropriate support system (empirical). It follows logically from the rock mass quality parameters that:

- poorer rock mas quality areas will require more substantial support installation than good quality areas,
- areas with significant groundwater will require resin grouted support units as water will wash out the cement from cement based grouts and result in poor anchorage;
- poorly orientated joints will require support to carry the deadweight of the wedged formed;
- poor strength infilling may require pre-stressed support units to maximize friction along the joints / bedding by increasing the clamping forces on the planes;
- etc.

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CONNECTION 179

Refer to "3.2 Rock-mass behaviour around access and service tunnels" on page 94 and 2.1.2 Geological structures on page 41

Geological structures

Any geological structure implies a weakness in the rock mass, and needs to be assessed and treated in order to prevent instability around the structure. The extent, size, properties and history of the structure must be taken into account in the design of a support system for the tunnel in the vicinity of the structure in order to prevent the tunnel from collapsing or conditions deteriorating. The following issues regarding structures should be evaluated in terms of their impact of support requirements:

- Occurrence of seismic events on geological structures to allow designs for seismic resistant support with high energy absorption capacity;
- Reduction in rock mass quality around structures to ensure suitable support (refer to section 4 above);
- Exposure distance along the tunnel and reduced distance from the structure increases support requirements;
- The creation of large wedges in the tunnel by the intersection of structures must be evaluated and addressed;

EXAMPLE

a. A dyke, which was formed by the intrusion of molten lava, would have caused, due to very high temperatures, burning and changing of the rock at the contact zone and ground conditions around the dyke zone will probably be completely different from that in the undisturbed rock mass;
b. The contact planes between the dyke material and the host rock could be filled with gouge and other filling material;
c. Sympathetic fracturing, jointing and faulting could have resulted from fault creation events.

CONNECTION 174

Refer to LEARNING OUTCOME 3.2.16 on page 108

Stress mechanisms

The stress field in which the tunnel is developed will have a large influence on the behavior of the rock mass around the tunnel as well as the performance of the support system. It is important to take cognizance of the dimensions and orientations of the stress field components near the position of the tunnel. The virgin stress k-ratio can assist in estimating the behavior of the rock mass, such as in the Bushveld Complex, high horizontal k-ratios of as large as 5:1 can result in stability problems in tunnels developed perpendicular to the high horizontal stress direction.

The impact of the stress field around tunnels can also be analysed using numerical modeling and analytical systems such as the Rock Wall Condition factor (RCF) to estimate tunnel support requirements.

Tectonic stress fields exist as a result of the formation processes of the rock mass. Extreme stress variations can occur over short distances due to tectonic stress impacts. Stress measurements and observations can assist in estimating areas subjected to higher or different tectonic stress fields. The impact on support would be similar to normal overburden stress, but the orientations may vary significantly, requiring a change in support unit positions in the tunnel.



- Refer to Rock Mechanics for Underground Mining by Brady and Brown Chapter 11.

CONNECTION 180

Refer to Paper 1 Outcome 3.11 on page 86

Stress cycle and Rate of Stress Change

The stress cycle around a tunnel refers to the stress build-up and release during mining operations in the vicinity of the tunnel. An excavation can be developed in a stress field which is a result of primitive / virgin stress, induced stress from other mining and / or tectonic stress. During the life of the tunnel, the stress fields may change considerably in level

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or orientation and it is these changes that have large influence on the stability of the tunnel.

This stress change or cycle of stress change can follow many routes, but as an example the following cycle is described:

- Pre-development of a tunnel in unmined areas to access the ore-body occur at moderate stress levels. Stress fracturing occurs but is easily stabilized by the installed support;
- As mining commences and progresses, the stress field in the area around the tunnel changes by dramatic stress level increases and stress orientation changes, increasing fracturing around tunnels and thus displacement into the tunnels, which is also controlled by the ability of the support to yield and absorb the energy;
- When mining finally relieves the stress around the tunnel by mining over is (de-stressing the tunnel), the stress drop may result in a loss of the clamping and even become tensile, fractures will open up, and rock which is not contained by support, will unravel and fall out causing loss of excavation integrity.

The rate of the change in the stresses is also important, as rapid stress changes causes rapid rock mass deformations which change the behavior of the rock mass and the support units. The ultimate stress change rate occurs during the interaction of a seismic strain wave with a tunnel, resulting is significant support and design changes. At the same time, slow stress changes could result in creep rock mass behavior with long term implication for support units.



- Jager and Ryder page 196, 215 – 226

Initial stress conditions

The initial stress conditions will have an effect on the stability of the tunnel during the development phase. Designing support based on the initial stress field only, could be disastrous as designing for a low stress environment may result in insufficient support at a later stage when stresses reach high levels and seismic activity occurs.



It is critical that support is not design for the 'current' stress state but that the whole stress cycle of an excavation is evaluated when performing support designs for tunnels, as failure to do so will result in damage and could require substantial rehabilitation.

Mining configuration and mining cycle

The effect of mining configuration is largely the ability of the mining to control stress levels induced on tunnels during the life of the mining plan. Abutment stresses can become extreme, and experience and modeling has shown that the influence can be as much as 60 to 80m above and below the reef horizon. Configurations must be used to minimise stress levels (not place abutments near tunnels) and changes (overstope tunnels as early in the mining cycle as possible) and it is only in situations where it cannot be done (practical reasons), support measures must be in place to ensure excavation stability.

CONNECTION 181

Refer to LEARNING OUTCOME 3.1.16 on page 87 and LEARNING OUTCOME 3.1.17 on page 87

Overstoping and de-stressing techniques

Overstoping or de-stressing an area before tunnels are developed, or early in the mining cycle (stress levels till low) are one of the methods of

protecting tunnels from adverse stress conditions.



The longwall mining layout, which has been successfully applied in deep level mining situations, was designed with follow-behind tunnels which are placed in de-stressed ground.

Overstoping can, however, result in other problems with regards to stability, such as severely fractured ground and the presence of a tensile stress field that causes unraveling of the fractures. Placement of the tunnel outside of the highly fractured ground, and maintaining the 450 follow on position must be combined with appropriate support measures to contain the fractured rock in the tunnel walls.

Tunnelling configuration and cycle

CONNECTION 182

Refer to4.4.1 Tunnel layouts on page 240 and 4.4.2 Large chamber layouts on page 248

During the design phase, size, shape and orientation of tunnels will be decided on (tunnel configuration). The size and shape is usually dependent on the equipment and airflow required in the tunnel, but it is known that these factors will impact on conditions and thus support requirements. Special support measures are required where the tunnel width changes, for example, when a cross-cut is broken away, or where a cubby is mined into the sidewall of the tunnel.

The development cycle refers to the cycle of drilling, blasting, cleaning and support installation that is repeated as the tunnel advances. However, especially the installation of support within the development cycle will depend on the conditions experienced in the tunnel. This cycle has been adapted under some conditions by allowing some support installation earlier in the cycle, as shown in Table 8 below:

Standard cycle	Adapted cycle for poor rock mass conditions
Drilling of the face Charging up and blasting of the face Cleaning the face areaInstalling primary tendon support to protect the drilling crew Drilling of the face (cycle starts again)	Drilling of the face Charging up and blasting of the face Placement of areal coverage support such as shotcrete or thin skin liners on the exposed rock walls Cleaning the face areaInstalling primary tendon support to protect the drilling crew Drilling of the face(cycle starts again)

Table 8: Support in the development cycle

Pre-development

Certain mining layouts such as scattered mining and sequential grid mining requires that tunnels are pre-developed into virgin ground to gain access to the reef horizon. This method has certain advantages with regards to exploring the area, and important information about the geology expected in the mining phase are made available. The stability of such tunnels comes under threat with regards to stress changes during the life of the tunnel, as mining of the reef over the tunnels causes abutment stresses to be induced over the whole length of the tunnel. This can reach critical levels and cause fracturing, and, if not protected sufficiently can result in collapse of the rock mass into the tunnel when the mining has advanced over the tunnel, reducing stress levels.



In order to prevent this from happening, such tunnels are often placed far below the reef horizon.

CONNECTION 183

Also see Chapter 3, LEARNING OUTCOME 3.1.17 on page 87

Follow behind development

When the mining layout allows for follow behind development, for example, long wall mining, support is adapted according to the positioning of the tunnels relative to the reef horizon. When the tunnels are close to

the reef, fracturing caused by the mining may result in instability, and ground control becomes more difficult.



An accepted standard is placing the tunnel approximately 30m below the reef. Fracturing may still occur but the following distance of the tunnel must be equal to or more than the middling (45° rule).

CONNECTION 184

Refer to Paper 2 Outcome 5.1 and 5.2 from page 270

Support components

Support components have different meanings, it can refer to the components within individual support units or to the different components of support within the total support system.

Support components for tunnel support units vary significantly and it is advised that the components given in the outcomes 5.1 and 5.2 in the paper 2 material are studied.

Support components within tunnel support design refer to the presence of primary, secondary and tertiary support units within a total support system. By definition, these components are normally used to refer to the following:

- Primary support: First tendons installed in exposed rock and / or areal coverage support, typically such as shotcrete and thin skin liners with the aim of protecting the development crew only;
- Secondary support: Support installed to protect exaction against current and medium term stress levels and typically include mesh and lacing on tendon support units to some acceptable pattern or shotcrete;
- Tertiary support: Support required for the long term protection of tunnels or to address specific issues such as the size of the tunnel in a specific area and typically includes long cable anchors, arches and sets.



- Brady and Brown Page 339
- Hoek on support systems

Support systems

A support system is constructed using a combination of different support types into a structured design to accomplish stability of an excavation. The support system comprises of the components discussed above (primary, secondary and tertiary) but may also have a combination of support types within a specific component, such as tendons and thin skin liners as primary support.

Depending on the factors that impact on the stability of the tunnel over its life span, support systems could vary significantly:

- Short term tunnels in low stress and aseismic areas may only be pattern bolted with primary tendons;
- Areas where seismic activity is anticipated, primary support could consist of thin skin liners and tendons, with mesh and lace as secondary support and cable anchors in areas.



- Jager and Ryder Page 208

Support appropriateness and Implementation quality
Appropriate support design is achieved by applying the correct design criteria and is also dependent on the characteristics of the support units installed such as initial stiffness, yieldability (yield range), loadbearing (tensile strength) and shear capacity (resistance against shear failure). Under specific conditions, the rock mass around a tunnel will require specific support characteristics to ensure that stability can be maintained.

EXAMPLE

In high levels of deformation, support units should have high yield ranges.

In a tensile environment, support should be pre-stressed so that the integrity of the rock mass is maintained.

EXAMPLE

When a traveler intends to visit the Sahara Desert, conditions in the desert would suggest that an elevated vehicle with 4-wheel drive capability is required to ensure that he passes the desert successfully. If a vehicle on low profile tyres, limited clearance to the ground, small engine capacity and 2-wheel drive capability is issued to him, he would be expected to get stuck in the desert. This does not mean that the second vehicle is a poor vehicle, is just simply not appropriate for the conditions.

Table 9 lists typical characteristics for a number of support units, but it is critical that each support units utilised within a support system should be classified in terms of these characteristics to ensure that the support is appropriate to the conditions.

Support element		Initial stiffness	Yield capacity	Load capacity	Shear capacity	Comments (applicability)
Friction tendon:	Swellex/ split set	Fair	Fair	Low	Fair/poor	Simple installation, corrosion a problem, primary support only
End anchored Pre-tensioned:	Rock stud	Very good	Poor	Medium	Fair	Used under low stress conditions.
	Cable anchor	Very good	Fair	High	Good	Large excavations/wide areas
Fully grouted:	Smooth bar	Good	Fair	Medium	Fair	Easily debonded, requires good grouting
	Rebar	Very good	Poor	Medium	Poor	High initial stiffness, requires good grouting
	Drill steel	Very good	Poor	High	Poor	High shear resistance
	Yielding tendon	Fair	Very good	Medium	Fair/good	Good yieldability
	Cable tendon>4m	Fair	Fair/ poor	High	Good	Yieldability + flexibility, may require good grouting
	Wire loops,3m	Fair	Fair/ poor	Medium	Good	Yieldability + flexibility, may require good grouting
Sets:	Arches and Cribbing	Poor	Fair	High		Areal coverage in poor areas

Support element		Initial stiffness	Yield capacity	Load capacity	Shear capacity	Comments (applicability)
Fabric:	Mesh and lacing	Poor	Good	Low		Areal coverage + flexibility, labour intensive
	Reinforced shotcrete (50mm)	Good	Fair/ poor	Medium		Areal coverage + limited deformability
	Reinforced shotcrete (50mm) & lace	Good	Good	Good		Areal coverage + limited deformability
	Un-reinforced Shotcrete (50mm)	Good	Poor	Medium		Areal coverage in areas of low deformation

Table 9: Support element design characteristics (Jager and Ryder, 1999)



Appropriate support will thus be the support that maintains the integrity of the tunnel by maintaining the stability of the rock mass around the tunnel without failing.



Appropriate support should also be cost-effective without jeopardizing stability or safety.

It can never be more expensive to properly develop and support a tunnel initially than having to rehabilitate (or even replace) a tunnel at a later stage.

Management effectiveness and quality control

CONNECTION 185

Refer to Paper 2 Outcome 4.4 on page 265 and 5.2.4 on page 359

Timeous installation of support in a tunnel, and the quality of such installation, is a critical aspect in successfully creating a stable tunnel and must be managed well. This management includes:

- ensuring adequate quality support materials are available – support units delivered and stored must be quality controlled to ensure that it adheres to the specifications set out and that it is of acceptable quality (corrosion, all parts delivered, cement storage, resin storage, etc);
- ensuring the support standards arising from support design are implemented – the installation of units as per the design or recommendation must be followed and quality control of the installed support will indicate compliance;
- ensuring that complete instructions as to what, where, when and how to install the support exist and are followed – ensure recommendations are complete and clear and that it is followed. This includes the quality of installation, i.e. poorly grouted support units will not perform as designed and is thus not acceptable.

The management of compliance to these standards, and a system whereby quality control is done, is essentially part of the duties of the rock engineering department.



Support not installed according to standard spacings, installation procedures and quality will not perform as required, and may lead to instability. A system whereby support quality audits can be performed regularly must be in place to ensure that support installation is done properly and according to specifications and standards.



There is no sense in using the most advanced support system, but the installation quality results in support elements (and maybe the whole support system) failing and rock mass instability. Examples of rock falls in tunnels where support elements have failed can be found throughout

the industry. Once an element has failed, load is redistributed to neighboring elements, and the change become bigger that those elements may fail as well.

LEARNING OUTCOME 5.2.1.4

DESCRIBE, EXPLAIN AND DISCUSS THE OBJECTIVES OF PRIMARY SUPPORT IN TUNNELS

Primary support is installed to secure the immediate hanging wall rock as soon as possible after the blast, where installation takes place as part of the development cycle (drill, blast, clean , support, drill, blast, clean, support, ….) and is designed to cater for the current, short term rock mass conditions.

Typically, some type of rock tendon (bolt) will be used eg. split sets, rockstuds or grouted (cement or resin) tendons. In cases where ground is severely broken up, a fabric may be used to cover the rock surface such as welded mesh or mesh and lacing.

Primary support is normally installed before drilling of the face commences. Primary support may form part of the total support system designed for the tunnel, but is normally not and the support system is normally upgraded with the installation of additional support (secondary support, LEARNING OUTCOME 5.2.1.5 on page 320).



- Jager and Ryder Page 207
- Brady and Brown Page 312

LEARNING OUTCOME 5.2.1.5

DESCRIBE, EXPLAIN AND DISCUSS THE OBJECTIVES OF SECONDARY SUPPORT IN TUNNELS

Secondary support in tunnels is installed with the aim of maintaining stability in the tunnel over the long term and should cater for stress changes, seismicity, time dependent deformation, etc.



- Jager and Ryder page 208
- Brady and Brown Page 312

This support forms the backbone of the final support system designed to maintain stability of the tunnel and is usually designed based on the specific conditions that exist, or is expected to exist at some time in the future, in and around the tunnel, or even portions of a tunnel.



- Jager and Ryder page 219

Jager and Ryder (1999) shows a series of tables (Table 10, Table 11 and Table 12) that can used as guidelines to select support systems for different stress conditions (static or stress change), seismic conditions (none or anticipated) and rock quality (RCF).



It is critical for the rock engineer understands that these tables are not intended to be the answer to all possible scenarios and should be used with good engineering judgement, rather than blind application.

Case	Primary support (Typical requirements)	Secondary support (Typical requirements)
Static stress conditions	'Spot' support where necessary (rock bolts, etc)	'spot' support where necessary (rock bolts, etc.) Light mesh where ground conditions are inherently poor.
Stress changes anticipated	'Spot' support where necessary (rock bolts, etc.)	Fully grouted tendons > 1.5m in length, installed on a basic 2m pattern. Support resistance: 30 – 50 kN/m2. Rope lacing (sidewalls only) plus light mesh where ground conditions are inherently poor.
Seismic activity anticipated	Tendons > 1.2m in length, installed as close to the face as possible, on basic 2m or 1,5m pattern. Support resistance: 30 – 50 kN/m2.	Fully grouted (pref. yielding) tendons > 1.5m in length, installed on a basic 2m pattern. Support resistance: 50 kN/m2 . Light mesh and lace (hanging wall and side walls)

Table 10: Support recommendations for good ground conditions (RCF<0.7) (Jager and Ryder, 1999)

Case	Primary support(Typical requirements)	Secondary support(Typical requirements)
Static stress conditions	Rock bolts or tendons > 1,5m in length, installed as close to the face as possible on basic 2m pattern. Support resistance: 30 – 50 kN/m2	Steel strapping or rope lacing integrated with primary support tendons, or Shotcrete
Stress changes anticipated	Fully grouted tendons or steel ropes > 1,8m in length, installed as close to the face as possible, on basic 2m or 1,5m pattern. Support resistance: 40 – 60 kN/m2.	Medium strength rope lacing and moderate strength wire mesh integrated with primary support tendons.
Seismic activity anticipated	Fully grouted tendons (pref. yielding) or steel ropes > 1,8m in length, installed as close to the face as possible, on 1,5m or 2m double pattern. Support resistance: 80 – 110 kN/m2.	Strong rope lacing and strong, stiff (welded) wire mesh integrated with primary support tendons, plus optional shotcrete.

Table 11: Support recommendations for average ground conditions (0,7 <RCF<1,4) (Jager and Ryder, 1999)

Case	Primary support(Typical requirements)	Secondary support(Typical requirements)
Static stress conditions	(If necessary) Shotcrete to face, then fully grouted tendons or steel ropes, length>1,8m on basic 1,5m pattern, as close to the face as possible. Support resistance: 80 – 110 kN/m2	Moderate strength steel wire mesh usually with additional tendons but may be integrated with primary support, optional Shotcrete.
Stress changes anticipated	(if necessary) reinforced Shotcrete to face, then fully grouted steel ropes or yielding tendons, length>1,8m on a basic 1m or double 2m pattern, as close to the face as possible. Support resistance: 120 – 230 kN/m2.	Medium strength rope lacing and moderate strength wire mesh with additional tendons but may be integrated with primary support. Add integral Shotcrete in long life tunnels. If necessary, additional hanging wall support comprising grouted steel ropes.

Case	Primary support(Typical requirements)	Secondary support(Typical requirements)
Seismic activity anticipated	(if necessary) reinforced Shotcrete to face, then fully grouted steel ropes or yielding tendons, length>2,3m on a basic 1m or double 2m pattern, as close to the face as possible. Support resistance: 220 – 290 kN/m2.	Strong rope lacing and strong, stiff (welded) wire mesh with additional tendons but may be integrated with primary support tendons. Add integral Shotcrete in long life tunnels. If necessary, additional hanging wall support comprising grouted steel ropes.

Table 12: Support recommendations for poor ground conditions (RCF>1,4) (Jager and Ryder, 1999)

LEARNING OUTCOME 5.2.1.6

DESCRIBE, EXPLAIN AND DISCUSS THE OBJECTIVES OF TERTIARY SUPPORT IN TUNNELS

Although secondary support installed to standard is designed to ensure stability in a tunnel, circumstances may require additional support for specific situations. Tertiary support is referred to when additional support to the normal secondary, long term support is required to maintain stability.

EXAMPLE

A tunnel that has been developed under an abutment of a stope may suffer damage as a result of seismic events. Additional support may then be required apart from the secondary support system to ensure that the tunnel does not collapse. This may be in the form of long anchors, shotcrete and wire mesh and lacing in an area where the secondary support only comprised of rock bolts.

The complete support system may thus consist of primary, secondary and tertiary support units, depending on the conditions around the tunnel.

EXAMPLE

A tunnel is developed along reef strike but faulting created lower middlings between the tunnel and the reef in one area and resulted in a highly stressed area (fault loss) over another part of the tunnel. The secondary support designed for the tunnel where it is at a large middling to the reef must be different to that designed for the lower middling parts, as well as for the highly stressed fault loss area. For example, the support systems could be something like this:

- large middling: 1.5m long resin grouted tendons on a 1.5m pattern as primary support with meshing and lacing as secondary support, utilizing the primary tendons
  - this may be the standard support system for the mine in all their long term tunnels and is sufficient to ensure stability of a tunnel that will not be subjected to high stress or seismic levels.
  - It requires active primary support and some areal coverage to prevent small rock that dislodge from falling into the tunnel.
- lower middling: 1.5m long resin grouted tendons on a 1.5m pattern as primary support. Secondary support to be meshing and lacing on 2.2m long cement grouted tendons installed to a 1.0m pattern
  - the slight increase in stress levels around the tunnel as the mining, at lower middling, progresses over the tunnel increases the requirement for support.

- It requires the same primary support during development as that above as the mining above is not occurring yet.
- However, it also requires additional strengthening of the rock mass around the tunnel by installing longer tendons to a denser pattern. This increases the support resistance when the rock mass starts to deform as mining occurs over the tunnel, confining the rock (not allowing deformation) and thus increases the rock mass strength around the tunnel, allowing it to 'handle' the increased stress levels and maintain stability.

- fault loss: 1.5m long resin grouted tendons on a 1.5m pattern as primary support. Secondary support to be meshing and lacing on 2.2m long cement grouted tendons installed to a 1.0m pattern plus 4.5m long 25ton yielding anchors installed to a 2.0m pattern.
  - High stress level increases are expected and even seismic activity in the future when this fault loss will be highly stressed due to the large amount of mining that had occurred in the area. These stress levels will induce rock deformation and even failure around the tunnel.
  - This requires even more support and a higher support resistance to maintain stability, while an increase in the ability of the support system to allow deformation without failure is critical. The installation of longer, stronger and yielding units increases the support resistance and thus also the strength of the rock mass around the tunnel, but maintains stability when stress or seismically induced deformations cause rock failure.

LEARNING OUTCOME 5.2.1.7

DESCRIBE, EXPLAIN AND DISCUSS THE CONCEPT OF INTEGRATED SUPPORT IN TUNNELS



Integrated support is defined as the use of a number of support types, often as primary, secondary or tertiary support, in such a way that the support units combine their functions in impact on the rock mass around the tunnel.

Acceptance of this definition means that integrated support is much more than just a support system with different support types installed. Integrated support means that the support units from different types combine to provide specific support system reactions.

EXAMPLE

- Mesh and lacing is installed using the tendons installed as primary support and the additional tendons installed to connect all the tendons with the lacing and combining the different units into the system;
- Shotcrete is applied to the freshly exposed rock surfaces and primary support is installed through the shotcrete using large face plates and these are covered by another layer of shotcrete;
- W-straps or OSRO straps are fixed to the rock wall using primary or secondary support tendons.

LEARNING OUTCOME 5.2.1.8

Describe, explain and discuss various support strategies for :

- Return Airways
- Haulages, Crosscuts
- Travellingways

back

- Winzes
- Raises
- Tunnel intersections
- Tunnels traversing faults
- Tunnels traversing dykes
- Trackless mining declines in stopes



The support strategies discussed in this section will note the special considerations only.

Return airways
Return airways conduct the air flowing out of the working places to cooling centres or to surface and is humid while it often also contains corrosive materials. This may cause time related deterioration of the support element condition used in the tunnel as the humidity or corrosive material attach the integrity of the support unit.



The loss of unit integrity can become so severe that meshing disintegrates when touched and tendons can be bent by hand.

Support units must therefore be protected against this attack:

- cover the support with Shotcrete, Gunite or thin sprayed layer to prevent corrosion of the support
- use anti-corrosion steel material such as galvanized bolts and mesh.



- Jager and Ryder Page 222

Haulages and cross-cuts
Support strategies for haulages and cross-cuts are governed, as discussed in outcomes above by orientation, location, depth, stress environment, changes in stress, size of the excavation, and ground condition expected in the area.

CONNECTION 186
except for those given previously in LEARNING OUTCOME 5.2.1.5 on page 320 to LEARNING OUTCOME 5.2.1.7 on page 323

Support strategies should, :
- Take into account the life of the excavation in the design of the support
- Keep in mind that conditions may change as mining progresses
- Acknowledge that these excavations are the ingress and egress routes for all mining personnel.

Travelling ways
Travelling ways are inclined excavations and will usually intersect reef parallel bedding or layering at very acute angles, creating ground control problems.

CONNECTION 187
except for those given previously in LEARNING OUTCOME 5.2.1.5 on page 320 to LEARNING OUTCOME 5.2.1.7 on page 323

Support strategies should:
- Be installed closer to the face to 'catch' the thin wedge failures caused by the intersection of the strata layers;
- Be active at installation to ensure separation on layers do not occur;
- Be combined with good temporary support practices (Camlock prop types) to ensure safety of personnel during tendon support drilling.

back

Winzes and raises

Strategies will concur with that for normal development ends during the development of the largest portions of raises / winzes but strategies will change when the following conditions occur:

- Intersection of faults: This is usually accompanied by an increase in excavations height and deterioration of ground conditions. Except for the support of the 'weak' side of the fault, an increase in areal coverage and support unit lengths is normally required. If seismic activity is expected on this fault, include yielding support units, ledging of the intersection area in the strategies.
- Intersection of dykes: Blocky conditions within and poor ground conditions around the dyke requires an increase in areal coverage and the support of the dyke contact areas. Take into account the possibility of seismic activity and include yielding support (inclusion of mesh rather than just shotcrete) and ledging of the dyke area as strategies.
- Off-reef development: off-reef development either exposes the hangingwall material or results in reef material as hangingwall. Depending on the composition and conditions within these materials, support strategies may have to include longer tendons and brow support and a decrease in support to face distances, while areal coverage could also be beneficial in poor conditions.

Tunnel intersections

Where tunnels intersect, or where breakaways are developed, the width of the tunnel increases. This increased width has the following implications:

- It increases the potential size of the tensile stress zone above the intersection area: As the width of the tunnel increase, so does the height and size of the 'tensile' zone that must theoretically be supported. This increase in height that must be supported can be estimated in the following ways:
  - Half the width of the area (Jager and Ryder page 221)
  - Calculate the height to be supported using Stimpsons method (see below)
- It increases the potential for deformation of the hangingwall into the tunnel: Increased deformation means larger potential for instability and therefore places a priority on the installation of the additional support as soon as possible.
- It increases the potential to intersect geological structures that can influence the hangingwall behavior: Increased structure intersections increase potential for instability, especially in larger span areas, confirming the need to install additional support.

Strategies include:

- Where ground conditions permit, the larger portion of the tunnel is supported with long anchors designed according to the maximum dimensions.
- Where high stress or changes in stress is expected, fabric support such as wire mesh and lacing, Shotcrete or similar areal coverage is added.
- In severe cases it may be necessary to install sets.
- Bullnoses may require strapping with steel ropes to prevent unraveling.

EXAMPLE

Stimpson developed a semi-empirical method for estimating the depth of the zone around a tunnel of certain dimensions and ground conditions, so that adequate support can be designed.

- Ryder and Jager Page 368

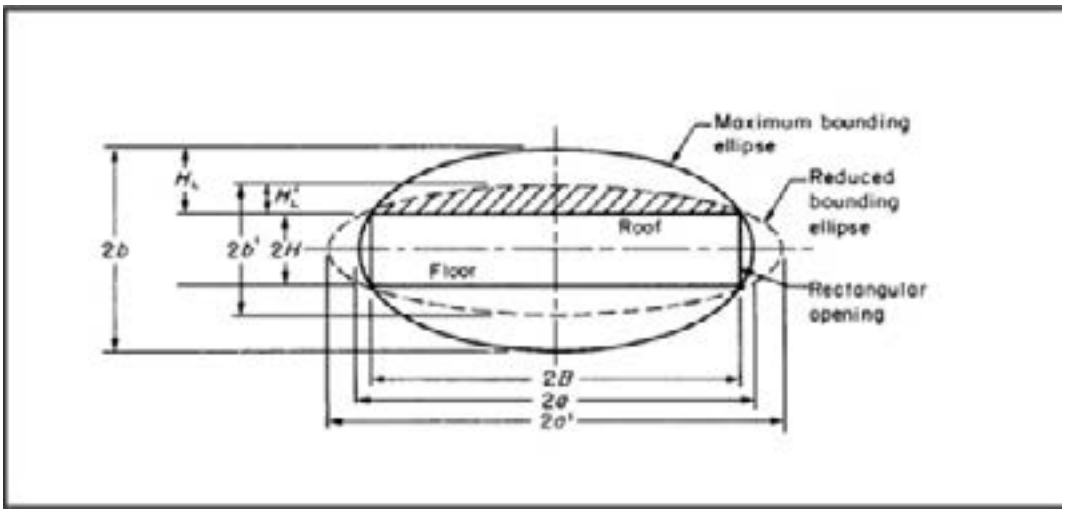


Figure 8: Stimpsons method to estimate support height

Stimpson suggests that the supported height H_L can be determined using the equations in below where k = ratio of maximum to minimum field stress, 2B = opening width, 2H = opening height, RMR = rock mass rating, HL = maximum height requiring support, H_L = reduced height requiring support, a and a' = semi major axis of the ellipse, b and b' = semi minor axis of the ellipse.

Table 13: Stimpsons method

An old saying goes:
It is better to prevent ground control problems from arising than causing them with poor mining practices and then having to stabilise the rock mass around the tunnel.

This means that intersections must either be prevented and where not possible it must be done with care and according to good practice. Examples of good mining and support practice include:

- Breakaways: Develop the tunnel past the planned break away position, upgrade the support in the planned breakaway area to that required by the final large span and then only blast the tunnel;
- Ledging (timber bays etc) : Install additional support, based on the final width, in the existing tunnel and then only commence with ledging. Install additional support as the width is increased, i.e. blast, clean, install support, blast, etc…
- Do not create acute angles in the breakaway area (i.e. turn away as close to 90° as practically possible) as it increases the final
- excavation width when the bullnoses fail.

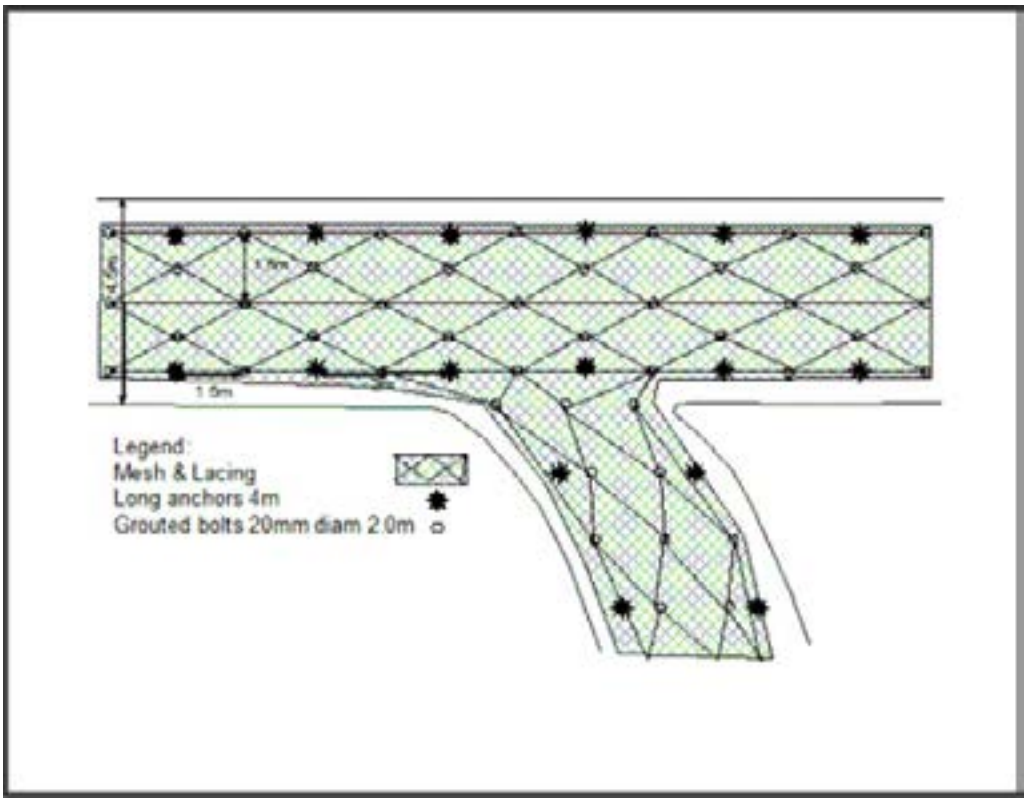


Figure 9: Example of a support standard for a breakaway at depth

CONNECTION 188

T-shape tunnels : See LEARNING OUTCOME 3.2.12 on page 105

Tunnels traversing faults and dykes

Ground conditions tend to deteriorate in the vicinity of dykes and faults, and support systems need to be adapted to cater for this. Furthermore, the chance of seismic activity also exists.

Support needs to have large yield ranges and high energy absorption capabilities in order to maintain support loads at large deformation as experienced under dynamic conditions.

Support strategies for situations where tunnels have to be developed through major geological features include sets with void filling :

- when the dyke scales significantly such as experienced with many lamprophyre dykes in the platinum mines;
- long anchors, wire mesh and lacing, and a pattern of tendons : depending on the ground conditions, excavation size and future conditions expected (stress , seismicity);
- In some cases the tunnels is mined as a T-shapes heading allowing for pack support to be installed on the shoulders.
- Ledging of tunnels when dykes are expected to provide a high risk of bursting had been shown to be very successful. The tunnels are ledged at full width (height of the tunnel) by at least 1.5m to both sides and long anchors with mesh and lace are added to the primary tendon support in the whole ledge area (hanging-wall and sidewalls). Timber packs / elongates are then installed within the ledges and the distances between them closed off using gumplanks. When the dyke bursts, the tendons and mesh and lace will control the deformation, but the timber support will prevent displacement into the working area of the tunnel. The broken material confined by the mesh and lace and timebr support now acts as a 'cushion' to absorb later bursts.

Trackless mining declines in stopes
Trackless mining operations usually require larger access ways because of the size of the equipment. Support requirements will therefore be adapted for the larger excavations while they must also be protected against damage caused by the vehicles. At the same these tunnels are created on-reef and will thus be exposed to the type and composition of material that will be found on-reef and support strategies should be appropriate.

EXAMPLE

- A decline on the UG2 horizon must take cognizance of the position of the triplets (chrome bands) above the reef and would probably limit the decline width and install active tendons of appropriate length and strength to carry the weight of the exposed rock beam between the decline hangingwall and the triplets.
- A decline developed on the manganese seam would take notice of the manganese / banded ironstone contact and ensure that support is adequate to reinforce the BIS layering (full cut) or carry the deadweight manganese beam (undercut).

Trackless mining declines are important life-of-mine excavations and critical to the operation. Support strategies should include:

- Appropriate support to ensure stability of the tunnel at the designed width for the life of the mine;
- Include areal coverage of steel units as the equipment releases residues which may corrode the steel with time;
- Use of sprayed-on cover such as shotcrete together with the tendons / anchors / mesh and lace.

LEARNING OUTCOME 5.2.1.9

DESCRIBE, EXPLAIN & DISCUSS THE STRATEGIES TO SUPPORT TUNNELS UNDER THE FOLLOWING CONDITIONS :

- Low stress with joint controlled behavior
- High stress where stress fractures dominate stability
- Changing stress cycles and environments over time
- Changing spatial stress environments
- Seismic and rockburst conditions

Low stress with joint controlled behavior
Joint behavior is controlled by the properties of the joints within the sets. The smoothness of the joint planes, infilling and properties of the infilling, dip and orientation of the joints, spacing and interaction of the joints with other features and fracturing all play a role in the stability of the rock surrounding the tunnels.

Support design and installation must cater for these abnormalities and prevent movement and unraveling.

- Clamping of the joint planes to prevent separation and movement on the joint plane is essential for stability
- tendons must be installed across joint planes as close to 900 as is practically possible
- At least one third of the tendon must protrude through the joint plane in order to create a strong anchor beyond the plane

- The tunnel must be developed with the long axis perpendicular to the major joints set strike (if practical)
- The use of strapping or wire mesh and lacing to secure the blocky ground conditions should be considered.



When analysing the stability of the different joint sets, the following is usually considered as important in deciding which joint set will most probably control the conditions / stability:

- *Density of joints (spacing): the joints with smallest spacing are usually called joint set 1 because they would be the most common joints, but it may not be the most critical set;*
- *Joint orientation: the orientation of the joint wrt the direction of development could create ground control problems and should be considered;*
- *Joint condition: Infilling, surface roughness etc control the strength of the joints.*

CONNECTION 189

See Paper 2 Outcome 3.1.2 on page 142 and Paper 1 Outcome 6.2 from page 200

Most of the rock mass rating systems are applied to this type of environment where structures under low field stress levels affect stability due to the fact that joint spacings, orientations and surface conditions plays the largest role in stability.

In poor rock mass conditions, it seems matural that tendon spacings must be reduced, lengths increased and areal coverage considered. These precautions are standard outcomes of the rock mass raying systems discussed in the outcomes mentioned above.

CONNECTION 190

See Numerical software that can assist in this wedge identification in Paper 2 Outcome 6.1.1.4 on page 395

Any wedge and key block forming must be pre-empted by doing mapping of the jointing and carefully analysing the data (also using numerical modeling) to ensure that the support will perform effectively.

When analysing joint controlled stability, solutions as to the required support are possible.

In these cases, it is assumed that the dimension of the wedges created by the joints and the excavation surface is known. If the dimension is known, it is possible to calculate the weight of the wedge so formed, allowing the estimation of support strength required to ensure that the wedge will remain in place.

The scenarios shown in Figure 10 indicate wedged formed in the tunnel sidewall and hangingwall. From the sketches and the understanding that the support must be able to prevent the wedge from moving and collapsing into the tunnel, the following are very important to note:

- To be able to calculate the wedge weight, it is necessary to have the wedge volume and material density (weight = volume x density x gravitational acceleration). To be able to get the volume it is assumed that the depth of the wedge will be equal to the strike spacing of the support units to be installed. This assumes that this wedge will be present 'all along' the tunnel and that support will be installed 'along' the tunnel to ensure stability. Usually it is assumed to be 1.0m.
- The analysis compares the 'stabilising forces' to the 'destabilising forces' to determine the stability. Within the 'stabilising forces' are the cohesion on the joints, friction along the joints and the support being installed, while the 'destabilising forces' is the weight of the wedge only (seismic impacts are excluded). In each case, the forces acting on the wedge (both stabilizing and destabilizing forces) must be calculated and the support added to ensure that it will remain stable (adding some factor of safety).
- Hangingwall wedge:

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- It is assumed that the joints have zero cohesion and zero friction as the wedge would simply fall from those surfaces and no movement along them would occur;
- This means that the forces acting on the wedge is limited to its weight and the support;
- The number of support units required can thus easily be calculated from N=(W.f)/C where w is the weight of the wedge (in Newton), f is the factor of safety and C is the maximum load bearing capability of the support to be installed (in Newton). (Ryder and Jager page 359).
- This equation simply multiplies the weight of the wedge with the safety factor to get a destabilizing force that must be equaled by the installed of N number of tendons, each having a strength of C Newton (stabilizing force).

- Sidewall wedge:
  - This wedge will slide along the steep dipping joint but will simply separate from the top joint when moving downwards due to gravity.
  - This movement will generate friction on the steep dipping joint determined by the weight of the wedge and the friction angle (Φ) of the joint (frictional force = normal force x tanΦ). The strength of the joint will thus be given by the cohesion PLUS the frictional force (Mohr-Coulomb failure criterion – See Paper 1 Outcome 4.1.8)
  - This joint strength and the intended support, installed at some angle through the wedge, will attempt to stabilize the wedge while its weight will attempt destabilisation.
  - The number of support units to stabilize the wedge can be determined from

    $N=(W.(fSin\beta-Cos\beta.Tan\Phi)-c.A)/(C.(Cos\alpha.Tan\Phi+fSin\alpha))$
    (Ryder and Jager page 360).

  - In this equation W = weight of the wedge (N), f = safety factor, β = dip of the joint on which sliding will occur, Φ = friction angle of the joint on which sliding will occur, c = cohesion (Pa) of joint on which sliding will occur, A is the area (m^2) of the joint on which sliding will occur, C = strength of the tendons (N) and α = the angle between the installed tendons and the NORMAL to the sliding joint surface.
  - This equation simply determines the number of tendons required to stabilize the wedge by dividing the 'destabilising forces' (top factor of the equation) by the 'stablising forces' (bottom factor in the equation) using the strength of the tendons C (Newtons) and the impact of the tendons on the joints when installed at angle α to the joint surface.
  - This equation will also allow you to determine the optimum installation angle for a certain tendon strength and controlling joint orientation.

back

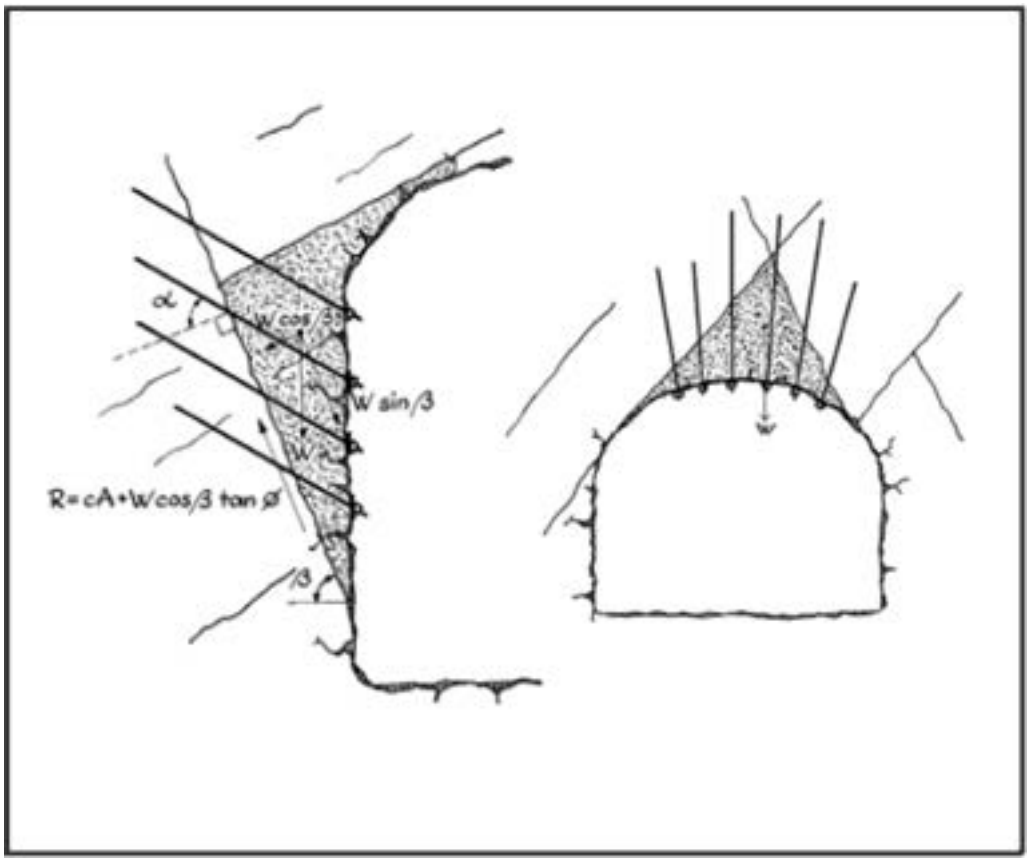


Figure 10: Structurally controlled support design (Ryder and Jager, 2002)

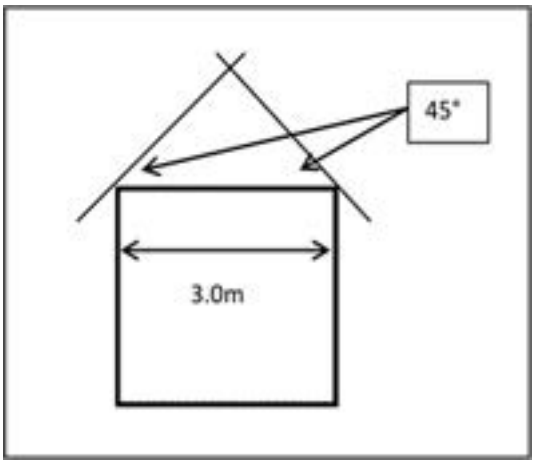
- Ryder and Jager page 359

REFERENCE

EXAMPLE

Hangingwall wedge:

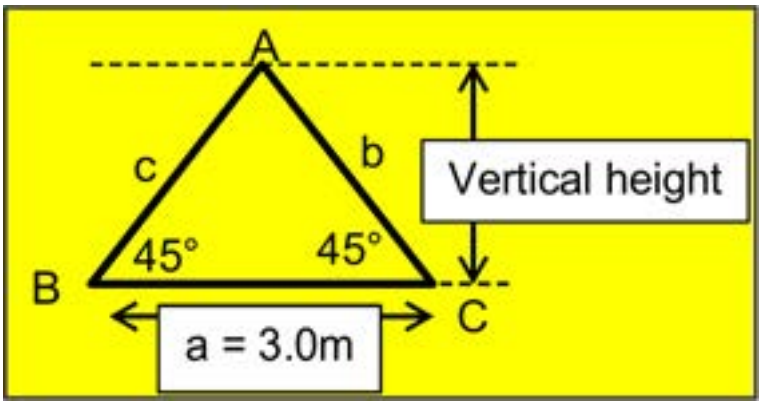
The width of the tunnel is 3.0m and the joints that form the wedge intersect the corners of the tunnel while both dip at 45° to the flat tunnel hangingwall. It is assumed that tendons with a tensile strength of 100kN will be used. Gravitational acceleration is 9.81 $m.s^{-2}$ and the rock has a density of 2700kg/m^3. Determine the required support to stabilise the wedge.



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Answer:
The number of tendons can be calculated from : N=(W.f)/C. For this calculation it is thus required to calculate the weight of the wedge (W), assume a factor of safety (f) and utilise the strength of the tendon given (C = 100kN).

The weight of the wedge is determined by its volume, gravitation acceleration (given 9.81 $m.s^{-2}$) and density (given 2700kg/m^3)



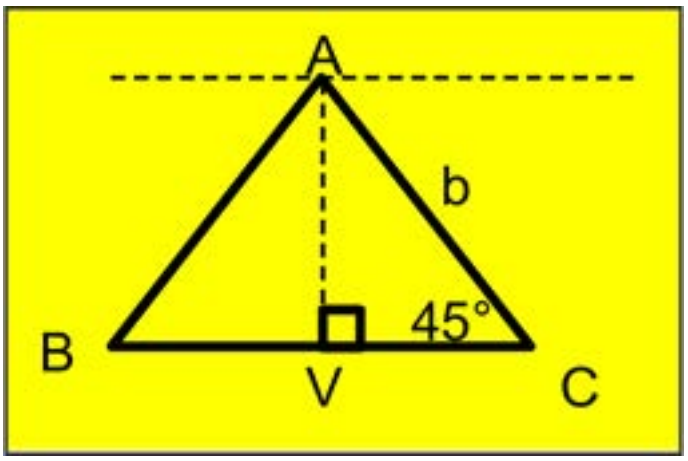
The volume is calculated from the dimensions of the wedge, i.e. area x thickness. Lets assume the thickness will be 1.0m (into page), then the area of the triangular wedge can be calculated using the well known equation Area = ½ x base x vertical height. The base is given as 3.0m, but the vertical height is unknown.

Using geometry and trigonometric functions, the information given indicate that the vertical height can be calculated. To solve for all the dimensions of the ABC triangle we apply the Sin Rule which states that: SinA/a=SinB/b=SinC/c

Applying this rule and the fact that it is given that
B =45°, C=45°, a = 3.0m and
A = 180° - (B+C) which comes from the 'sum of all angles in a triangle is 180°'
= 180° - (45°+45°)
= 180° - 90°
= 90°

SinA/a=SinB/b now becomes Sin90/3=Sin45/b allowing you to solve for b and gives that
b = 2.12m.

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Performing the same calculation using the C angle, in this case also gives
c = 2.12m.

Draw a line from the top corner of the wedge (A) to the base line BC so that it is perpendicular to the base line. This represents the vertical height of the wedge that is required to calculate the area. Applying standard trigonometry we know that within the triangle ACV

Sin C = Vertical height / b and thus that
Vertical height = b.Sin C = 2.12m x Sin (45°) = 1.5m.
The wedge triangle area is now determined from
Area = ½ x base x vertical height = ½ x 3.0m x 1.5m = 2.25m2

Volume = Area x thickness = 2.25m2 x 1.0m
(as assumed earlier) = 2.25m3

Weight = Volume x density x gravitational acceleration
= 2.25m3 x 2700kg/m3 x 9.81 m.s-2
= 59 595.75 N
= 59.6 kN

Now, from N=(W.f)/C the support can be derived, where

N = (W.f)/C
= (59.6 kN x 1.5) / (100kN) (for a factor of safety of 1.5)
= 0.89 tendons
= 1 tendon (we cannot install 0.89 of a tendon)

This means that 1 tendon of 100kN strength can be installed every 1.0m along the tunnel to keep the wedge in place.



The length of the tendon is critical and can be determined as follows:

- *End-anchored tendon: Anchor to be beyond the joint plane, giving at least a 1.6m long tendon;*
- *Grouted tendon: Use the critical bond length concept to determine the length of the tendon. (see Paper 2 Outcome 5.1.7)*

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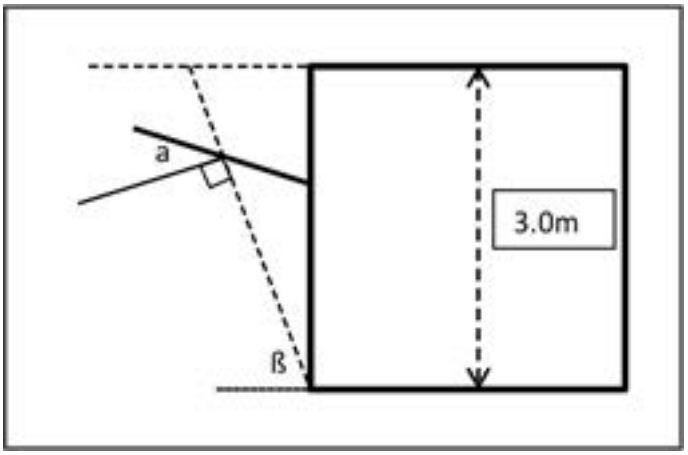
PAPER 3.1: CHAPTER 5

EXAMPLE

Sidewall wedge:

The height of the tunnel is 3.0m and the joints that form the wedge intersect the corners of the tunnel where the top joint is parallel to the hangingwall plane and the sliding joint forms an angle of 65° with the horizontal. It is assumed that tendons with a tensile strength of 100kN will be used and a practical angle of installation results in an angle of 35° to the normal on the sliding joint surface (as shown). Gravitational acceleration is 9.81 $m.s^{-2}$ and the rock has a density of 2700kg/m^3 but assume that cohesion on the plane is 1kPa and its friction angle is 15°.

Determine the required support to stabilise the wedge.



The number of tendons to support this wedge can be calculated from

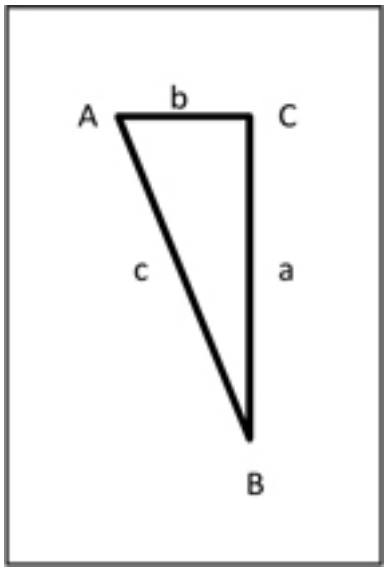
$$N=(W.(fSin\beta-Cos\beta.Tan\Phi)-c.A)/(C.(Cos\alpha.Tan\Phi+fSin\alpha))$$

but for this calculation it is necessary to determine the wedge weight (W), assume a factor of safety (f) and determine the sliding surface area (A).

The weight of the wedge is determined by its volume, gravitation acceleration (given 9.81 $m.s^{-2}$) and density (given 2700kg/m^3)

The volume is calculated from the dimensions of the wedge, i.e. area x thickness. Lets assume the thickness will be 1.0m (into page), then the area of the triangular wedge can be calculated using the well known equation Area = ½ x base x vertical height. The base is given as 3.0m (height of the excavation), and the vertical height of the triangle is unknown.

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In the wedge, represented by the triangle ABC above,

B = 90° - β = 90° - 65° = 25°,

while for this specific wedge

a = 3.0m (height of the excavation in this case) and

C = 90°, leading to the fact that

A = 180° - (90° + 25°) = 180° - 115° = 65°

as all angles in a triangle sums to 180°.

Using the Sin RULE discussed in the previous example, the solutions

b = 1.4m

c = 3.3m

In this example it is clear that the triangle ABC has base = 3.0m and vertical height 1.4m.

Area = ½ x base x vertical height = ½ x 3.0m x 1.4m = 2.1m2

Volume = Area x thickness = 2.1m2 x 1.0m

(as assumed earlier) = 2.1m3

Weight = Volume x density x gravitational acceleration

= $2.1m^3$ x $2700kg/m^3$ x $9.81\ m.s^{-2}$

= 55 622.70 N

= 55.6 kN

The area of the sliding surface A can be given by :

A = triangle side length c x 1.0m wedge thickness = 3.3m2 .

Assuming a factor or safety of 1.5, the number of tendons can now be calculated to be

N = (W.(fSinβ-Cosβ.TanΦ)-c.A)/(C.(Cosα.TanΦ+fSinα))

N = ((55.6kN x(1.5Sin65°–Cos65°.Tan15°)–(1kPa x $3.3m^2$)) / (100kN x(Cos35°.Tan15°+1.5Sin35°))

= ((55600 x 1.246) – 3300) / (107985.57)

= (69289.91 – 3300) / (107985.57)

= (65989.91) / (129591.55)

= 0.61

= 1 tendon

(as we cannot install 0.61 of a tendon)



Often, when the in-situ value of the cohesion is unknown it is assumed to be zero and the part of the equation (c x A) is zero and is removed from the equation.

High stress where stress fractures dominate stability

When a tunnel is subjected to high stress and fracturing result, deformation into the tunnel is caused by dilation of the stress fractures. This dilation into the tunnel releases confinement on rock deeper into the tunnel walls, allowing the creation of new stress fractures deeper into the walls. The installation of support under these conditions aims to create a reinforced arch around the excavation so that the rock mass is confined (support limits dilation into the tunnel) and strengthened, enabling it to withstand the stress levels.

The support must thus be capable of generating high loads (or support resistance) and have adequate yielding capacity at high yield loads to absorb and limit rock mass dilation.

Areal coverage by the support becomes important to prevent unraveling of the rock mass in broken up material.

When analysing high stress or seismically active environment stability, solutions as to the required support are possible. These solutions include:

Environment	Methods	Reference
Structural analysis. Where the solution can be found by analysing disturbing and stabilising forces, so deriving the required support force.	1.Creating a stable arch in fractured rock: support must be able to generate sufficient forces to keep the failed zone around the excavation in place and pin the unstable rock to the stable material above. The method includes the use of support resistance and energy absorption criteria.	1.Ryder and Jager Page 362
Empirical methods. Experience gained in high stress environments.	2.Wojno, Jager and Roberts: Support guidelines ito support resistance and energy absorption, yield range and tendon strengths. 3.Wiseman: Rockwall condition factor (RCF).	2.Ryder and Jager Page 366 3.Ryder and Jager Page 377
Mechanistic methods. Methods that are based on the damage mechanisms observed or experienced around tunnels.	4.Haile: Tunnel support for deep gold mines in South Africa. He investigated the interaction between bolts and a highly discontinuous (fractured, blocky, jointed) rock mass. The block aspect ratio, block volume and RQD is used to determine rock mass classes, leading to suggested stable spans and fall-out thicknesses. This allows the user to define the depth of unstable rock so that one of the high stress methods can be applied to design the support	5Ryder and Jager Page 380

Table 15: High stress and seismically active tunnel support design processes



The mechanistic method also includes any other methods where the damage mechanism is utlised to design the final support system, such as failure zone, height of instability etc derived from numerical models.

EXAMPLE

Mechanistic method (Haile)

Haile conducted numerical modeling (2 dimensional) and derived the following application of the results.

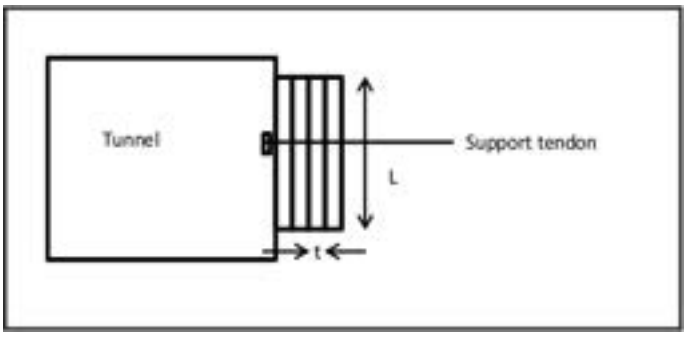
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In this analyses, the defining block parameters are:

- the volume and
- shape of the blocks relative to the tendon axis, expressed as the block aspect ratio (BAR) in the plane of the analyses.



This means that a high aspect ratio (BAR = L / t) suggests that the block shapes are long (L), thin (t) slabs, formed perpendicular to the tendon in the sidewall of a tunnel, i.e. where L >> t.



For blocks formed in a tunnel sidewall with dimensions L=0.5m and t=20cm, determine the stable span (height) and potential fall-out height that must be stabilised with support under rockfall and rock burst conditions if tendons are installed 2.0m apart.

Answer:
Utilise the Block Volume with the calculated BAR to indicate the rock mass class (A to G).

Block volume
= L x t x 1
= 0.2m x 0.08m x 1m
= 0.016 m^3.

Block aspect ratio
= L / t
= 0.2m / 0.08m
= 2.5.

Plot these results on Figure 11, giving rock mass class D.

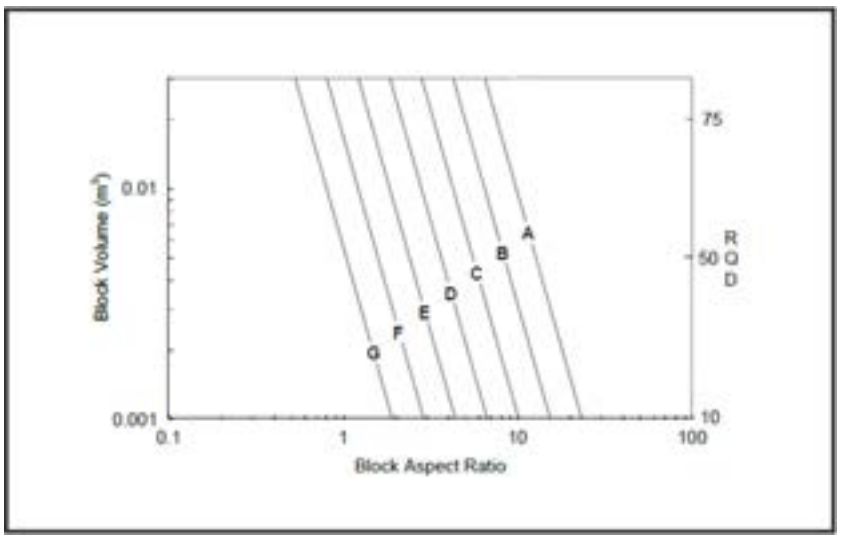


Figure 11: Relative stability of rock mass classes (Ryder and Jager, 2002)

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On Figure 12 or Figure 13, plot the spacing of tendons against the rock class D and read of the depth of instability:

- Rockfall: 0.4m
- Rockburst: 1.1m

Now, the required Support Resistance or Energy Absorption can be calculated, using the depth of failure suggested above as fall-out of ejection thickness.



In this work, Haile made a number of assumptions that must be kept in mind when utilized (Ryder and Jager Page 383).

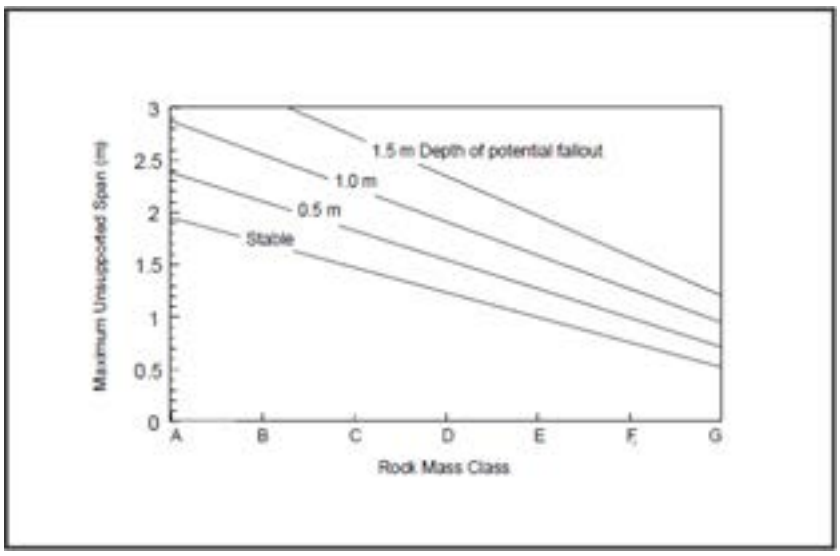


Figure 12: Tendon spacing and rockfall height for rockfall conditions (Ryder and Jager, 2002)

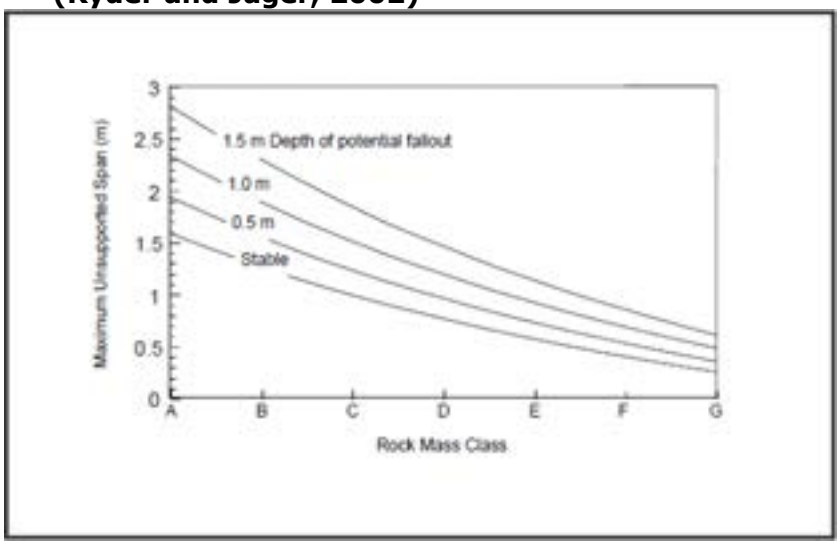


Figure 13: Tendon spacing and rockfall height for rockburst conditions (Ryder and Jager, 2002)

Changing stress cycles and environments over time

When changes in stress are anticipated, especially when the stress increases to levels above the strength of the rock and then is lowered through distressing, the support system must be designed to absorb the changes.

Increasing the strength of the rock through confinement (such as Shotcrete, over wire mesh and lace) will assist to prevent extensive fracturing when the stresses reach very high levels. If fracturing of the rock can be

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minimized the effect of the lowering of the stress after a period of time will also be minimized. Secondary and tertiary support must be installed before the stress reaches the extreme levels.

Changing spatial stress environments

Understanding of changing stress levels is critical to support design for tunnels. Within a specific static stress field, the rock mass and installed support around the tunnel eventually reaches a stage of stable equilibrium. This means that the strength of the rock mass and the strengthening impact of support on the rock mass reach a balance with the deformations caused by dilation of fractures created by the stress field.

At this stage of a tunnel's life, the stress field around the tunnel has to change to affect conditions:

- If the stress field increases, new fractures are created and dilation occurs into the tunnel deforming the support. If the support remains intact, loads generated by the support increases or it yields until balance (equilibrium) is reached again. If the support is strained beyond its yield range, it will fail, resulting in the collapse of the tunnel.
- If the stress fields decreases, stresses normal to fractures, joints and bedding planes reduce. This could possible result in movement on these fractures and often end up in disintegration fo the rock mass. If areal support had not been installed, the rock mass would fall apart around the tendons. In the case when areal coverage support was installed, the broken material could be maintained.
- If the field stress maintain its value but changes direction, new failures are now induced at other locations around the tunnel. For instance, if the major principal stress in vertical, failure would be mainly in the sidewalls of the tunnel and support precautions are focused here. If the tunnel is overstoped and the orientation of the major principal stress now changes to semi-vertical, the corners of the tunnel now become the area of increased fracturing. Further changes to a horizontal major principal stress direction will see increased fracturing in the hangingwall of the tunnel. In each case, new fracturing causes dilation that has to be addressed with support.



Installing support just in the hangingwall when fracturing will be induced in the sidewalls means the support would be inappropriate and will not necessarily maintain stability.

Depending on the stress change expected, support strategies would then vary to address the conditions expected during and after the changes:

- Increased stress levels: More support at smaller spacings and higher loads but also higher yield range is required to be able to attempt to balance the dilation caused by increase fracturing as a result of the higher stress levels.
- Reduced stress levels / de-stress: Increased areal coverage is required to maintain balance of the relaxed rock mass.
- Changing stress orientations: This does not necessarily affect the support selection strategy, but will affect the area of support installation.

Seismic and rock burst conditions

Since the waves created by the seismic event are in fact 'deformation' or 'strain' waves, the wave will, when it intersects the tunnel, compress and relax the rock mass. This additional compression when the wave passes through / over the tunnel is realised as a momentary increase in stress

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around the tunnel. This increase in stress causes additional fracturing and obviously dilation into the tunnel. The concern is that the dilation caused by these new fractures occurs at very fast velocities, often even resulting in the ejection of broken material from the tunnel walls.

The support strategy for a tunnel subjected to these conditions are very similar to that of high stress levels, except for the fact that the dilation occurs quickly (portions of a second) rather than over a longer period of hours / days as experienced under mining induced stress levels. Under high stress levels, yield rang was shown to be critical to ensure that the dilation do not cause failure of the tendon, but in fact the tendon was dissipating (absorbing in common terms) the energy offered by the slow moving rock mass. In seismic conditions, the support has to dissipate the energy of a fast moving rock mass, indicating that the loads generated by the support and the yield range is critical to maximize the energy that can be dissipated.

There is a number of yielding tendons available that can be used while wire mesh and lacing also have good yielding capacity (Table 9).

LEARNING OUTCOME 5.2.1.10

DETERMINE APPROPRIATE SUPPORT STRATEGIES FOR TUNNELS FOR GIVEN LAYOUTS, ROCKMASS CONDITIONS, STRESS REGIMES AND GEOLOGICAL CIRCUMSTANCES.

This outcome is achieved by application of the issue studied to date for a specific situation. It is best explain at the hand of an example.



- Jager and Ryder Chapter 6
- Budavari Chapter 4

EXAMPLE

CONNECTION 191

Paper 3.1 Oct 2009 Question 4 sub sect. 4.3

Mine ABC has conducted a concept level study to extend the VCR stoping from 2500 mbs to 3500 mbs. The study has indicated the following:

- An 8.7 m diameter sub-vertical shaft will need to be sunk to intersect the reef horizon
- Stope back lengths of 250 m will be used for half-level planning and scheduling
- A system of strike stabilising pillars will be required to control convergence and seismicity
- Haulages will be positioned 60 m below the reef horizon with a system of crosscuts accessing reef that will be spaced 240 m apart
- Stoping will be conducted using hand-held drills and scraper cleaning to box holes
- Ore tramming will be via rail bound locomotives

With reference to Mine ABC, answer the following questions:

1. Suggest and motivate the typical support systems that one would consider for the following excavations:
   - Haulages (3)

Answer:



The answer is given in broad terms to explain the thinking behind the suggested support and even though you should follow this thinking process in the examination, all of this information for 3 marks may not be required.

Critical information given:

- VCR
- 2500-3500 mbs
- Stope back lengths 250m
- Stabilising pillars
- 60m below reef
- Sounds like longwall mining with strike pillars and follow behind haulages with access to reef every 240m.

Based on this information, the conditions to be encountered and the most important support characteristics for the haulage are:

a. High vertical field stress levels due to depth and mining spans : yield range, load bearing
b. High pillar stresses : yield range, load bearing
c. Middling to reef reducing stress impact on haulage, even partially de-stressed if follow behind
d. Seismic activity :energy absorption, yield range, load bearing, areal coverage
e. Stress fracturing exist :yield range
f. Rock material probably quartzite (VCR footwall), bedded and strong but will fracture under high enough stress levels : yield range
g. Faulting and dykes will exist and poor conditions may exist :areal coverage
h. Support for the haulages must now cater for the conditions specified above:
i. Primary support: Active, grouted tendons to a 1.0m pattern in hangingwall and sidewalls
j. Secondary support: Yielding tendons with meshing and lacing to a 2.0m diamond pattern, except if / where exposed to high pillar stresses, where the pattern should be 1.0m diamond, and where 4.5m long yielding anchors should be included to a 2.0m pattern in all walls.
k. Additional: areal coverage such as shotcrete in areas close to structures where ground conditions may be poor. Long cable anchors around intersections with faults / dykes.

5.3. SUPPORT STRATEGIES FOR SHAFTS AND LARGE EXCAVATIONS

5.3.1. SUPPORT STRATEGIES FOR VERTICAL SHAFTS

LEARNING OUTCOME 5.3.1.1

DETERMINE APPROPRIATE SUPPORT STRATEGIES FOR VERTICAL SHAFTS FOR GIVEN MINING AND SHAFT LAYOUTS, ROCKMASS CONDITIONS, STRESS REGIMES AND GEOLOGICAL CIRCUMSTANCES.

CONNECTION 192

4.5 Shaft layouts on page 265

Shafts together with ancillary service excavations like tunnels, haulages and ore passes form the basic ingress and egress ways in mines. It is therefore important to maintain the stability of these excavations for the life of the mine.

CONNECTION 193

Refer to LEARNING OUTCOME 4.5.1.1 on page 265 and LEARNING OUTCOME 4.5.1.2 on page 265

Support, in terms of protection of the shaft against stress levels and rock mass movement commences with leaving adequate shaft pillars. The traditional and most widely used method of shaft protection is to leave unmined ground around the shaft in a square or circular shape.

CONNECTION 194

See Paper 2 Outcome 3.1 on page 95 and Paper 1 Outcome 6.2 on page 200

Rockmass conditions

It is inevitable for the shaft barrel to traverse portions of poorer ground conditions, which will require support to stabilise the conditions. Depending on the conditions encountered, support requirements may range from spot bolting to pattern bolting with sheet metal or welded mesh or shotcrete in heavily fractured ground.

Rock masses with a high closely spaced joint frequency require area cover support in the form of shaft lining to curb the dislodgement of the blocks of rock the shaft walls.

Vertical shafts mined in wet rock mass conditions may require support that is not affected by water while the water pressure build-up behind the lining should be considered.

As far as possible shafts should placed in areas where good ground conditions exist to avoid excessive ground control problems. To be able to ensure this, borehole cores should be evaluated to identify the areas where the rock mass consist of good material or poor material. Figure 11 shows an example of one borehole where the quality of the different materials was indicated by the RMR value. It is clear that the poorest conditions are expected in the shale intersections and support can now be suggested following any of the systems available.

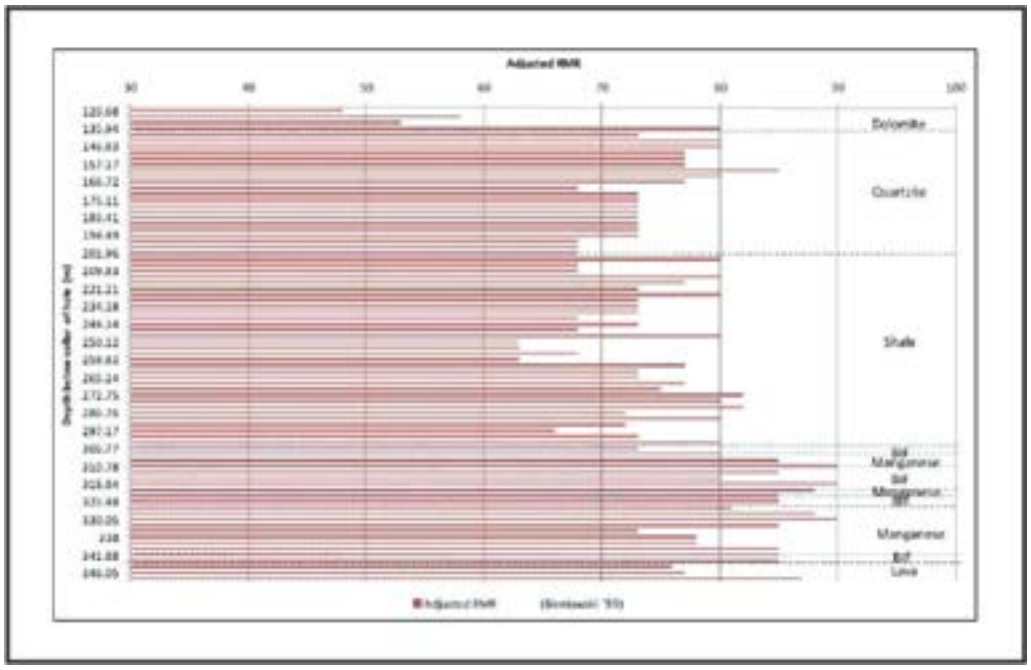


Figure 14: Rock Mass rating (RMR) values along a vertical borehole

By executing this process for a number of boreholes, the best possible site for the shaft can be ensured. This is done by identifying areas where the general rock conditions are better than most other areas or by finding the areas where the exposure to poor quality materials is limited. If it is accepted that the poorest rock mass quality is in the shale exposures (Figure 11), then Figure 12 indicates that exposure to the shale will be limited in the areas where boreholes had no / limited shale intersection.

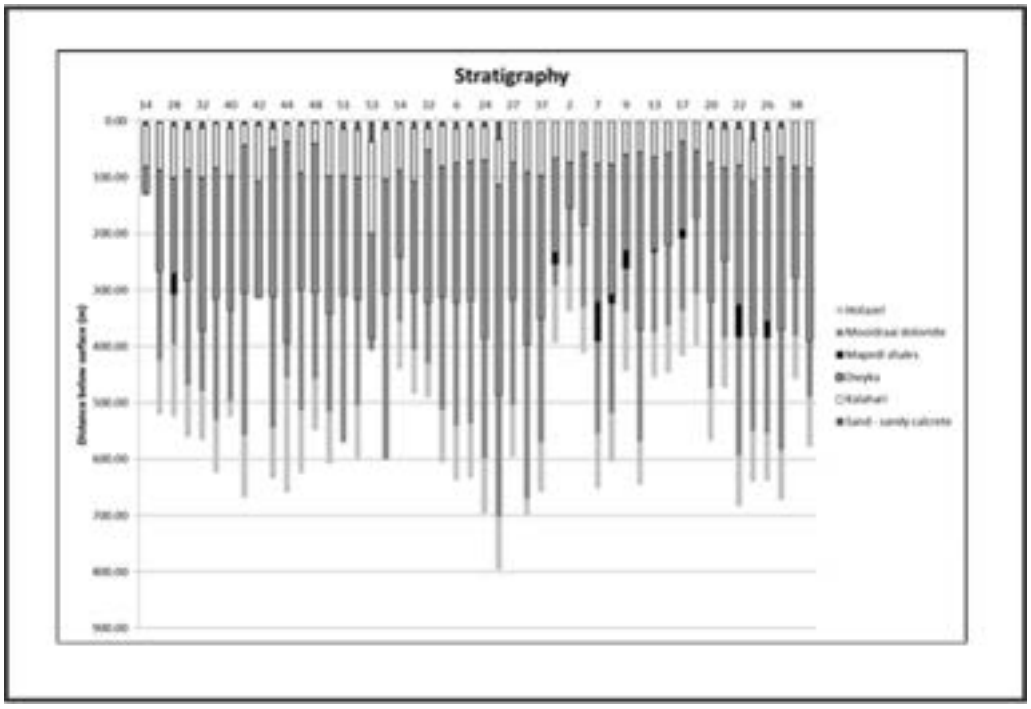


Figure 15: Several boreholes indicating exposure to various materials

CONNECTION 195

See LEARNING OUTCOME 3.3.3 on page 112

Stress regimes

With increased mining depth, the field stress levels increases. As stress impacts on vertical shafts are driven by the horizontal stress components, it is those components that need investigation. In general it is accepted that in shallower areas, the highest stress component will be horizontal (at ± 2 to 3 times the vertical component), even though the

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actual stress levels will be low. At depth, the highest stress component will be vertical, and horizontal components should be anything between 0.5 and 1.0 of the vertical component.

It is not often that these stress levels are large enough to cause failure of the rock material around the shaft barrel. However, if material such as clay in encountered, the stress field may be sufficient to cause flow of the clay into the barrel and this has a substantial impact on barrel support. The pressures caused by clay (in this example) on the planned lining, have been shown to be as high as 8 MPa (upper limit) and should be taken into account when designing the concrete lining.

Geological circumstances

It is highly recommended to site vertical shafts away from geological structures in the form of faults or dykes as they not only create ground control problems during sinking but also increase the risk of wedge failure and seismic damage in the barrel later in the life of the barrel.

Sometimes however it is impossible to avoid siting shafts in geologically disturbed ground and in these instances special support strategies are required to mitigate the effects on stability. These strategies are similar to those applied for tunnels in similar conditions as discussed in outcomes above.



Some geological situations exist that does not affect shaft barrel support during its development, but that can affect barrel stability later on in the life of the barrel. These include:

- *Existence of multiple reefs (two reefs or more) and wide reefs may result in larger convergence and vertical displacement in the shaft barrel during shaft pillar extraction.*
- *Where large throw faults are intersected or reverse faults exist, high stress impacts on the shaft barrel is possible during extraction*

EXAMPLE

Courtesy of T Leach, Latona Consulting.

The method is outlined by Hoek and Marinos (2000) and has been used to determine the potential for stress failure and squeezing as well as support requirements for different sections along a shaft. The method analysis the failed zone around a circular excavation and the elastic and plastic deformation that occurs in response to the failure.

The method estimates the rock mass strength (σ_{cm}) based on the UCS (σ_{ci}) and rock mass rating GSI (often the RMR is also used), plus an estimate of the Hoek-Brown parameter mi.

$$\sigma_{cm}=(0.0034m_i^{0.8}).\sigma_{ci}.\{1.029+0.025e^{(-0.1mi)}\}^{GSI}$$

Using this value, it is possible to calculate the plastic (failed) zone around the circular excavation as well as the sidewall deformation for a range of possible internal support pressures.

$$d_p/d_o=(1.25-0.625\ p_i/p_o).(\sigma_{cm}/p_o)^{(pi/po\ -0.57)}$$
$$\delta_i/d_o=(0.002-0.0025\ p_i/p_o).(\sigma_{cm}/p_o)^{(2.4\ pi/po\ -2)}$$

Where

d_p = plastic zone diameter (m),
d_o = original excavation diameter (m),
δ_i = sidewall deformation,

back

p_i = internal support pressure,
p_o = in-situ stress = depth x unit weight and
σ_{cm} = rock mass strength.

From these calculations, a Ground Reaction Curve can be constructed in which support pressure is plotted against excavation wall deformation (assumed to be equal at all points around the excavation).

If the shaft lining is treated as a structural shell that is evenly loaded, the strength and support capacity can be presented by the method suggested by Hoek and Brown in 1980 where

$$\text{Support pressure} = k_c.(\text{Sidewall deformation})/(\text{Excavation radius})$$

Where k_c = is the stiffness of the concrete shell, given by

$$k_c=(E_c\ (r_i^2-(r_i-t_c)^2))/((1+v_c\ )\{(1-2v_c\ )\ r_i^2+(r_i-t_c)^2\})$$

Where
E_c = elastic modulus of the concrete,
v_c = Poisson's ratio of concrete,
r_i = excavation radius and
t_c = concrete or shotcrete thickness.



Typically
E_c = *15 GPa and*
v_c = *0.3, whilst*
$\sigma_{c.conc}$ = *20 to 30 MPa.*

The maximum support pressure that the lining can provide is a function of the concrete strength σc.conc and the shell geometry, as given by

$$p_{scmax}=1/2\ \sigma_{c.conc}\ [1-((r_i-t_c)^2)/r_i^2\ )]$$

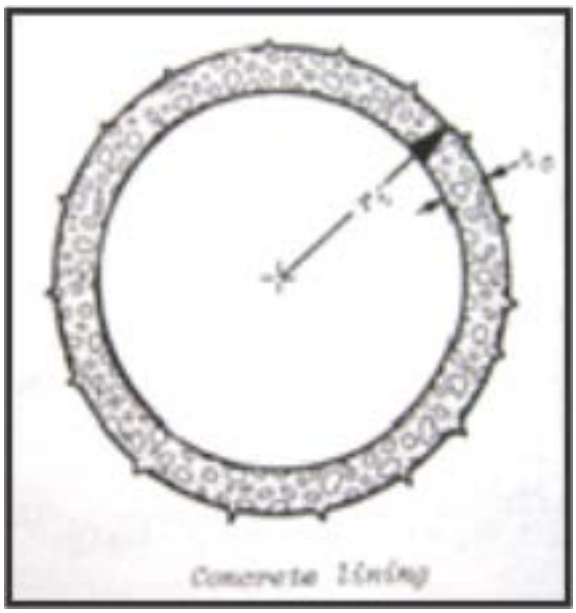


Figure 16: Excavation for lining design

Using this method, an assessment of the support requirements for clay was made based on a UCS value of between 1.7 MPa and 4 MPa, RMR

(GSI) of 75 and m_i value of 5. The stress regime was estimated at approximately 97mbs.

The resultant Ground Reaction Curves, relating support pressure to deformation is shown below and excludes any swelling pressures.

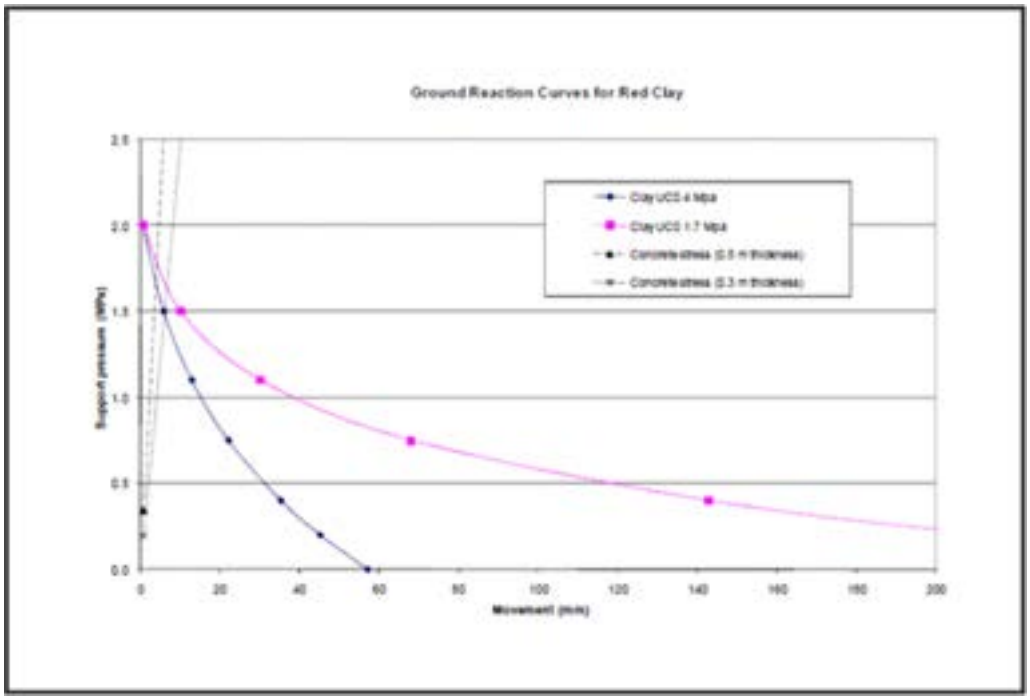


Figure 17: Ground reaction curves for lining

On the relationship above (Figure 17), the support pressures due to a 0.3m thick as well as 0.5m thick lining are shown.

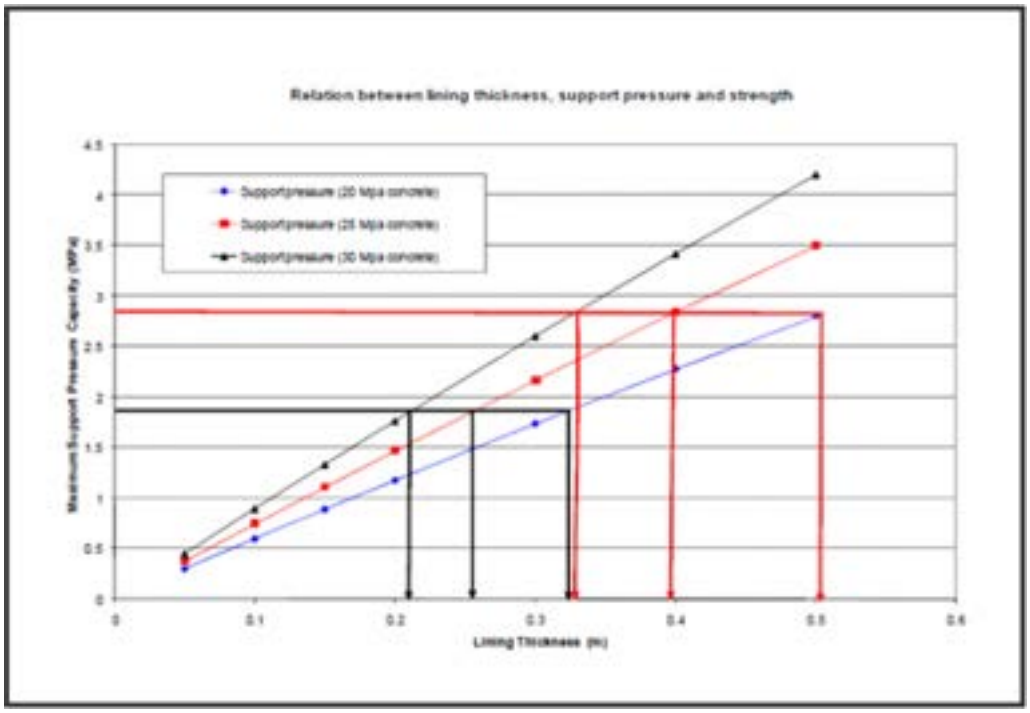


Figure 18: Lining support pressure

The relationship between lining thickness and maximum support capacity is shown in Figure 18 for concrete strengths 20, 25 and 30 MPa.

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In Figure 17, the load lines for the lining was also overlain. Depending on the clay strength and lining thickness, the maximum support pressure requirement varies between 1.5 Mpa and 1.8MPa.

Taking this 1.8 MPa to Figure 15, it is shown that a:
0.22m thick lining of 30 MPa strength
0.26m thick lining of 25 MPa or
0.33, thick lining of 20 MPa
concrete is required for adequate support.



If swelling pressures of the clay is included, the required pressure can be increased by 1 MPa (reasonable swelling pressure), giving a 2.8 MPa support pressure requirement. On Figure 15, this I sshwon in the red lines, indicating lining thockness of between 0.33m and 0.5m, depending on the concrete strength.

This lining thickness can refer to concrete, shortcrete or combinations thereof.



- Squeezing tunnel Hoek

5.3.2. SUPPORT STRATEGIES FOR INCLINED SHAFTS

LEARNING OUTCOME 5.3.2.1

DETERMINE APPROPRIATE SUPPORT STRATEGIES FOR INCLINED SHAFTS FOR GIVEN MINING AND SHAFT LAYOUTS, ROCKMASS CONDITIONS, STRESS REGIMES AND GEOLOGICAL CIRCUMSTANCES.

Inclined shafts are mostly used in shallow and intermediate mining conditions. When the decline shafts are on-reef, they are protected by shaft pillars that prevent significant stress increases around the declines whilst they also prevent hangingwall deformation in the shaft. Support strategies take cognisance of the long term stability requirement of the decline shafts and often consist of support systems that include tendons, cable anchors and areal coverage support such as welded mesh or shotcrete.

When they are off-reef, decline support strategies are similar to that of tunnels as discussed in previous outcomes.

Rockmass conditions

CONNECTION 196

Refer to LEARNING OUTCOME 5.3.1.1 on page 342

See section on support strategies in a vertical shaft where similar issues apply.

CONNECTION 197

LEARNING OUTCOME 3.3.4 on page 114

Stress regimes

Since decline shafts are normally situated parallel or close to parallel to the orebody (similar dip angle), they are initially subjected to the same stress field that is present around the orebody. The impact of different k-ratios on the decline varies the areas of concern within the decline shaft and thus also the support to be designed during its development. As mining progresses around a decline shaft (when on-reef) and above the shaft (when off-reef), stress levels around the decline could change, affecting the support strategies.

The existence of fault losses create higher stress levels in some areas

around the decline as the decline is forced to continue through the fault loss towards the reef on the other side of the fault. These stress level could become very high towards the latter part of the shaft's life and require high stress support strategies, even in relatively shallow environments.

Geological circumstances
The most common geological related issues that impact on support strategies for declines are:

- Cross cutting bedding in cases where the dip of the decline is not equal to that of the orebody:
  - This creates thin wedges on the face of the developing decline shaft and will require support installation closer to the face and probably areal coverage support such as shotcrete if the wedge cannot be controlled.
- Exposure to similar weakness planes as the stoping when on-reef:
  - When specific ground control affecting structures exist, such as the triplets above the UG2 seam, the decline will be subjected to the potential instability if not adequately supported or if the decline is too wide. This will require the correct length and type of tendon installation to ensure stability.
- Fault losses and dyke intersections:
  - These losses expose the decline to high abutment stress levels as discussed above.
- Poor rock strength / condition horizons:
  - When declines are placed in a rock mass that has alternating good and poor strength / condition material layers, it is exposed to changes in support strategies in these different areas. Although these excavations should be placed well away from poor materials, it cannot always be ensured. Support strategies could vary from pattern bolting only to complete areal coverage after the blast, tendons and cable anchors, with sets utilized in extreme conditions.

5.3.3. SUPPORT STRATEGIES FOR ROCKPASSES

LEARNING OUTCOME 5.3.3.1

DETERMINE APPROPRIATE SUPPORT STRATEGIES FOR ROCKPASSES FOR GIVEN MINING AND SHAFT LAYOUTS, ROCKMASS CONDITIONS, STRESS REGIMES AND GEOLOGICAL CIRCUMSTANCES.

CONNECTION 198

Refer to 4.4.3 Rockpass layouts on page 257 for discussions on orepass rehabilitation and support and links to practical examples.

Rock passes should be developed at steep angles (> 60°) to the horizontal. To minimise damage from the impact of the rock during transportation rock passes should be protected by mining legs less than 70m long. If these lengths are not practical for the mining layout under consideration then silos to break the length of drop of rock down a rock pass may be excavated. Elbow legs may also be used to break the momentum in the fall of rock being transported down the rockpass.

Rockmass conditions
Rock passes will inevitably mine through different rock mass conditions, while it will also intersect them at different orientations, depending on the rock pass layout. As with shafts, intersection with poor and heavily jointed rockmasses must be avoided as far as possible to avoid scaling of the rock pass walls during material transportation.

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PAPER 3.1: CHAPTER 5

CONNECTION 199

See LEARNING OUTCOME 3.3.5 on page 114

Stress regimes
It is important to know the stress regime in a rock mass hosting a rock-pass as it determines the position of possible stress related scaling and also the extent of the scaling. If stress related scaling is anticipated, support installation in a rock pass should be considered prior to

As a general recommendation to mitigate the amount of damage expected in a rockpass, the orientation of a rockpass should be sub-parallel to the major principal stress. In stress anisotropy conditions, the mining of circular shaped rockpasses is not recommended. Instead an oval shaped excavation is preferred as it caters for the major and minor horizontal stress orientations. The longer axis of the oval shaped rockpass is best oriented in the minor principal direction and the short axis in the major horizontal stress direction.

Geological circumstances
Rock passes should be mined in such a way to intersect weak strata as near perpendicular as possible (Budavari 1983 & Jager and Ryder 2002: 241). Faults and intrusive dykes must be intersected as perpendicularly as possible and rock passes must not be mined parallel to the or on strike of geological structures (faults/dykes).

5.3.4. SUPPORT STRATEGIES FOR LARGE CHAMBERS

See 3.3 ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS on page 112 and 4.4.2 Large chamber layouts on page 248

LEARNING OUTCOME 5.3.4.1

CONNECTION 200

See 3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS on page 94

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE VARIOUS BEHAVIOUR OF ROCK AROUND LARGE EXCAVATIONS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

LEARNING OUTCOME 5.3.4.2

CONNECTION 201

See LEARNING OUTCOME 3.3.2 on page 112, LEARNING OUTCOME 3.3.20 on page 125 and LEARNING OUTCOME 3.3.27 on page 127

DESCRIBE, EXPLAIN AND DISCUSS HOW THIS AFFECTS LARGE EXCAVATION SUPPORT REQUIREMENTS AT SHALLOW, INTERMEDIATE AND GREAT DEPTH

LEARNING OUTCOME 5.3.4.3

CONNECTION 202

See LEARNING OUTCOME 3.3.2 on page 112

DESCRIBE, EXPLAIN AND DISCUSS ROCKMASS CLASSIFICATION SCHEMES FOR THE DETERMINATION OF SUPPORT STRATEGIES FOR LARGE CHAMBERS

LEARNING OUTCOME 5.3.4.4

CONNECTION 203

Refer to LEARNING OUTCOME 2.3.1.3 on page 51 and LEARNING OUTCOME 3.2.16 on page 108

DESCRIBE, EXPLAIN AND DISCUSS THE APPLICATION OF THE ROCKWALL CONDITION FACTOR METHODOLOGY TO DESIGN SUPPORT FOR LARGE EXCAVATIONS AT DEPTH

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LEARNING OUTCOME 5.3.4.5

APPLY THE ROCKWALL CONDITION FACTOR (RCF) METHODOLOGY TO DESIGN SUPPORT FOR LARGE EXCAVATIONS AT DEPTH

CONNECTION 204

Refer to LEARNING OUTCOME 2.3.1.3 on page 51 and LEARNING OUTCOME 3.2.16 on page 108

LEARNING OUTCOME 5.3.4.6

APPLY THE WISEMAN METHODOLOGY TO DESIGN SUPPORT FOR LARGE EXCAVATIONS AT DEPTH

CONNECTION 205

Refer to LEARNING OUTCOME 2.3.1.3 on page 51 and LEARNING OUTCOME 3.2.16 on page 108

From Wiseman's extensive application of the RCF concept to the South African Witwatersrand gold mine tunnels, an empirical relationship between support systems and typical (3m x 3m) mine tunnels in this environment was created. It was generally found that for RCF < 0.7 good conditions prevailed with minimum support required. For 0.7 < RCF < 1.4 average conditions prevailed with typical support systems. For RCF > 1.4 poor ground conditions prevailed with special support requirements.



This empirical approach may well fit deep level hard rock mining conditions and may not necessarily be applicable to other mining conditions. When attempting application of this concept in other environments, it is required that its appropriateness must be confirmed before using is extensively in that environment.

LEARNING OUTCOME 5.3.4.7

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF EXCAVATION SEQUENCE ON SUPPORT INSTALLATION

CONNECTION 206

Refer to LEARNING OUTCOME 3.3.19 on page 122, and LEARNING OUTCOME 3.3.20 on page 125

LEARNING OUTCOME 5.3.4.8

DETERMINE APPROPRIATE EXCAVATION SEQUENCES TO FACILITATE SUPPORT INSTALLATION

CONNECTION 207

Refer to LEARNING OUTCOME 3.3.19 on page 122, and LEARNING OUTCOME 3.3.20 on page 125

LEARNING OUTCOME 5.3.4.9

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF EXCAVATION SEQUENCE ON SUPPORT EFFECTIVENESS

CONNECTION 208

Refer to LEARNING OUTCOME 3.3.19 on page 122, and LEARNING OUTCOME 3.3.20 on page 125

LEARNING OUTCOME 5.3.4.10

DETERMINE APPROPRIATE EXCAVATION SEQUENCES TO FACILITATE SUPPORT EFFECTIVENESS

CONNECTION 209

Refer to LEARNING OUTCOME 3.3.19 on page 122, and LEARNING OUTCOME 3.3.20 on page 125

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LEARNING OUTCOME 5.3.4.11

DESCRIBE, EXPLAIN AND DISCUSS THE RULES OF THUMB FOR DETERMINING THE LENGTH AND SPACING OF TENDONS IN LARGE EXCAVATIONS

Rules of thumb for support length and spacing

In flat roofs where the principal stress direction is near vertical, it is general practice to assume that the volume of rock in the hangingwall requiring support is enclosed by a semi-circle of diameter equal to the width of the excavation. For tendon support to adequately support the hangingwall, the length of support tendons should at least be longer than half the chamber span or width.

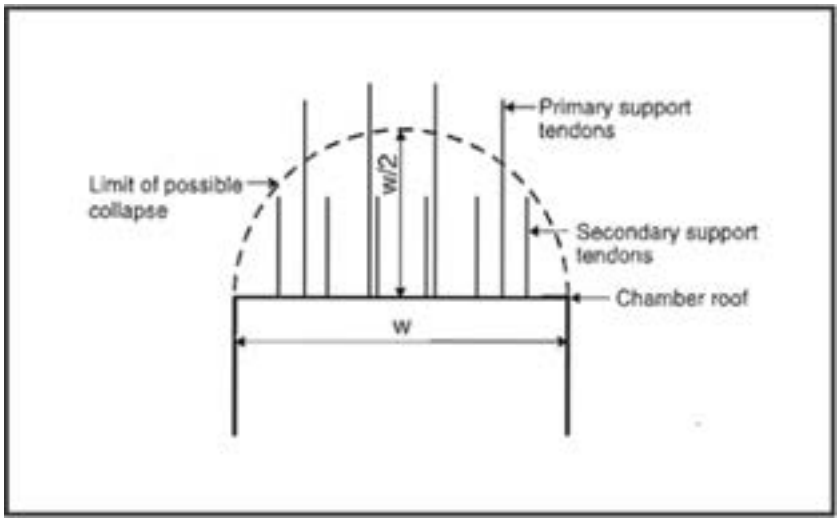


Figure 19: Rule of thumb for hangingwall support

Length of tendon = 0.5 x span or height of ecavation in which they are to be installed

Spacing of tendon = 0.5 x length to tendon

- Jager and Ryder Page 221

These recommendations must be used as guidelines and design of tendons must be site specific and should include some form of factor of safety.

LEARNING OUTCOME 5.3.4.12

APPLY THESE RULES OF THUMB TO THE DESIGN OF SUPPORT IN LARGE EXCAVATIONS

EXAMPLE

For a 5m wide excavation, estimate the tendon lengths and spacings using the rules of thumb.

Length of tendon
= 0.5 x 5m excavation width
= 2.5m

Spacing of tendon
= 0.5 x 2.5m tendon length
= 1.25m (or 1.2m).

LEARNING OUTCOME 5.3.4.13

CONNECTION 210

Refer to LEARNING OUTCOME 3.1.17 on page 87

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECTS OF OVERSTOPING AND UNDERSTOPING ON THE STABILITY AND SUPPORT REQUIREMENTS OF OFF-REEF EXCAVATIONS

LEARNING OUTCOME 5.3.4.14

CONNECTION 211

Apply Outcomes LEARNING OUTCOME 5.3.4.1 on page 349 to LEARNING OUTCOME 5.3.4.13 on page 352
Refer to 3.3 ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS on page 112 and refer to Paper 2

DETERMINE APPROPRIATE SUPPORT STRATEGIES FOR LARGE CHAMBERS FOR GIVEN MINING AND SHAFT LAYOUTS, ROCKMASS CONDITIONS, STRESS REGIMES AND GEOLOGICAL CIRCUMSTANCES.

5.4. SUPPORT DESIGN CRITERIA

5.4.1. STOPE SUPPORT DESIGN CRITERIA

LEARNING OUTCOME 5.4.1.1

DESCRIBE, EXPLAIN AND DISCUSS STOPE SUPPORT REQUIREMENTS IN TERMS OF : INITIAL STIFFNESS, YIELDABILITY, AREAL COVERAGE, AREA OF INFLUENCE, BEAM REINFORCEMENT, SUSPENSION, SUPPORT RESISTANCE, ENERGY ABSORPTION



Item	Definition	Stope support requirements
Initial stiffness	Load generated by support unit at very low deformation / closure / compression.	Support installed close to the mining face will experience low closure and will require high initial stiffness to assist with stability.
Yieldability	Ability of support to deform whilst maintaining load generation,	Where closure is expected support units should be able to yield to prevent them from failure and total loss of support.
Areal coverage	Contact area between the support unit and rock walls.	Increased areal coverage in stopes reduce exposure to rock walls and thus the risk of injury. Smaller spacings or larger unit contact area is possible.
Area of influence	Distance beyond the edge of the support unit to which the unit still affects the stability of the rock wall	All support units have some influence on the rock mass around the unit. Larger areas of influence suggest that larger spacings are possible between units.
Beam reinforcement	Binding beams together with support forming a thicker unit that is stronger and more resistant against bending.	In laminated or severely bedded rock mass, separation on bedding planes is possible as mining spans increase. By reinforcing beams, thicker beams are created and stability is maintained easier. This requires support to be active / pre-stressed between the end of the unit and its face plate.

Item	Definition	Stope support requirements
Suspension	'Hanging' a beam from rock above it using tendons.	If the eight of a beam is 'carried' by support units anchored above the beam, it is suspended. The correct length, strength and spacing of units is required to ensure that the tendons are able to carry the weight of the beam.
Support resistance	Load generated by the support unit as a function of supported area.	Support resistance criteria are developed for each mine. The support resistance generated support must exceed the required resistance by the FOS. Spacings must be reduced or higher loadbearing support units utilised if the support resistance required is not initially met.
Energy absorption	Ability of support unit to absorb / dissipate energy by deforming whilst generating load.	In dynamic situations, support must not deteriorate / fail in such a manner that no load is generated against the rock walls. This will lead to complete rock mass failure into excavations. By ensuring sufficient yield range and load generated in the utilised support units, the unit is able to deform, generate load and remain operational to support the rock mass.

LEARNING OUTCOME 5.4.1.2

CONNECTION 212

Refer to LEARNING OUTCOME 3.1.11 on page 81, LEARNING OUTCOME 3.1.12 on page 82, LEARNING OUTCOME 5.1.16 on page 305 and LEARNING OUTCOME 5.1.17 on page 306

DESCRIBE, EXPLAIN AND DISCUSS HOW SUPPORT RESISTANCE VARIES FOR STATIC AND DYNAMIC CONDITIONS

In static or quasi-static conditions, the rock mass moves at a very slow rate and failures are depicted as 'falls of ground'.

In dynamic conditions, the rock is ejected from the rock wall and does not move under gravity only. In this case, the energy balance must be maintained. The calculation of the energy available in the process of ejecting the rock must be compared to the ability of the support to absorb / dissipate this energy. This ability of the support unit is determined from the area under the load - deformation curve and the ratio of the two calculations provides the factor of safety.

However, during the dynamic movement of the rock from the rock walls, the support is also required to generate a support resistance that would be able to carry the deadweight of the rock being ejected. If the support units has been deformed and the loads generated have been reduced, the rock's weight may exceed the ability of the support to keep it in place, the support would fail and the rock would collapse into the stope.



Graphs have been developed to assist in the derivation of the loads OR spacings required to ensure that adequate support resistance or energy absorption is provided by the intended support system. These graphs are available in Ryder and Jager pages 321 and 322, but should be used with care as the graphs were created using specific input parameters that could be different in your area of work.

It is relatively easy to create these graphs and it is advised that you create these graphs for conditions and rock mass properties at your operation.



- Ryder and Jager page 308 and 309
- Ryder and Jager page 320 - 323

LEARNING OUTCOME 5.4.1.3

CONNECTION 213

Refer to LEARNING OUTCOME 3.1.11 on page 81 , LEARNING OUTCOME 3.1.12 on page 82, LEARNING OUTCOME 5.1.16 on page 305 and LEARNING OUTCOME 5.1.17 on page 306

CALCULATE REQUIRED SUPPORT RESISTANCES FOR STATIC CONDITIONS

- Ryder and Jager page 320 - 323

LEARNING OUTCOME 5.4.1.4



CALCULATE REQUIRED SUPPORT RESISTANCES FOR DYNAMIC (ROCKBURST) CONDITIONS

- Ryder and Jager page 320 - 323

CONNECTION 214

Refer to LEARNING OUTCOME 3.1.11 on page 81, LEARNING OUTCOME 3.1.12 on page 82, LEARNING OUTCOME 5.1.16 on page 305 and LEARNING OUTCOME 5.1.17 on page 306

LEARNING OUTCOME 5.4.1.5

CALCULATE SUPPORT LOADS FOR SUPPORT SYSTEMS MADE UP OF VARIOUS UNITS



This outcome is assumed to mean one of the following two things:

- *The support unit consist of more than one component , such as timber packs with a number of layers of timber mats or*
- *The support system consists of a number of different support units, such a packs and elongates.*
- The stiffness of a load bearing unit kc is inversely proportional to the sum of the reciprocal stiffness of all the individual members of the unit:

  $k_c = 1/(\Sigma_{(i=1)}^{n} 1/k_i)$

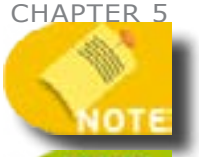
Where k_i is the stiffness of the individual layers and there are 'n' number of layers in the unit.



Inversely proportional: As the input parameter increases, the resultant value decreases and vice versa.
Reciprocal: The reciprocal of a value B is equal to (1/B).

This means that the stiffness of a support unit that consist of many individual layers (such as a timber pack or composite pack) is a function of the stiffness of each of the layers in the unit. As the individual layer stiffness increase, so does the stiffness of the total unit. Since load generation is a function of stiffness (stiffness = load generated / deformation undergone), increased stiffness suggests increased load generation.

However, it also means that the reduction in stiffness of some layers in the unit will lead to the reduction of the overall unit stiffness and thus load generating ability.



Poor construction and poor quality material will reduce layer stiffness and thus also support unit stiffness.



- Budavari page157

EXAMPLE

How will the following information gathered from installed packs affect the resulting pack stiffness if the installation height was 1.0m and the stiffness values were all calculated at 20mm compression?

Layers	Designed stiffness	Actual stiffness due to poor installation
1	80kN/mm	70kN/mm
2	80kN/mm	50kN/mm
3	80kN/mm	80kN/mm
4	80kN/mm	60kN/mm

The total unit stiffness values are shown below, calculated as follows:

$k_c = 1/(\Sigma_{(i=1)}^{n} 1/k_i)$
$= 1/(1/80+1/80+1/80+1/80)$
$= 1/0.05$
$= 20$kN/mm for the design and

$k_c = 1/(\Sigma_{(i=1)}^{n} 1/k_i)$
$= 1/(1/70+1/50+1/80+1/60)$
$= 1/\ 0.06345$
$= 15.7$kN/mm actual,

Which means the actual pack generates 22% less load for the same deformation as the designed packs.

To derive representative support resistances (derived from support spacings and loads generated) from a number of support units, the following must be acknowledged:

a. The units are not all installed at the same time or at the same distance from the face: This means that the closures (and even closure rates) to which each unit has been exposed are not similar, and thus the loads generated will also not be similar.
b. Installation heights, quality of footing and general installation may be different: Loads generated will not be the same.

A method to derive the loads represented by each individual unit was proposed in Ryder and Jager (page 324) and includes the following:

a. A graph that shows the Support resistance to Deformation relationship for each of the support types within the system (Figure 20 top);
b. Lines that represent the closure rates in terms of Closure (deformation) as a function of the distance from the face (Figure 20 bottom).

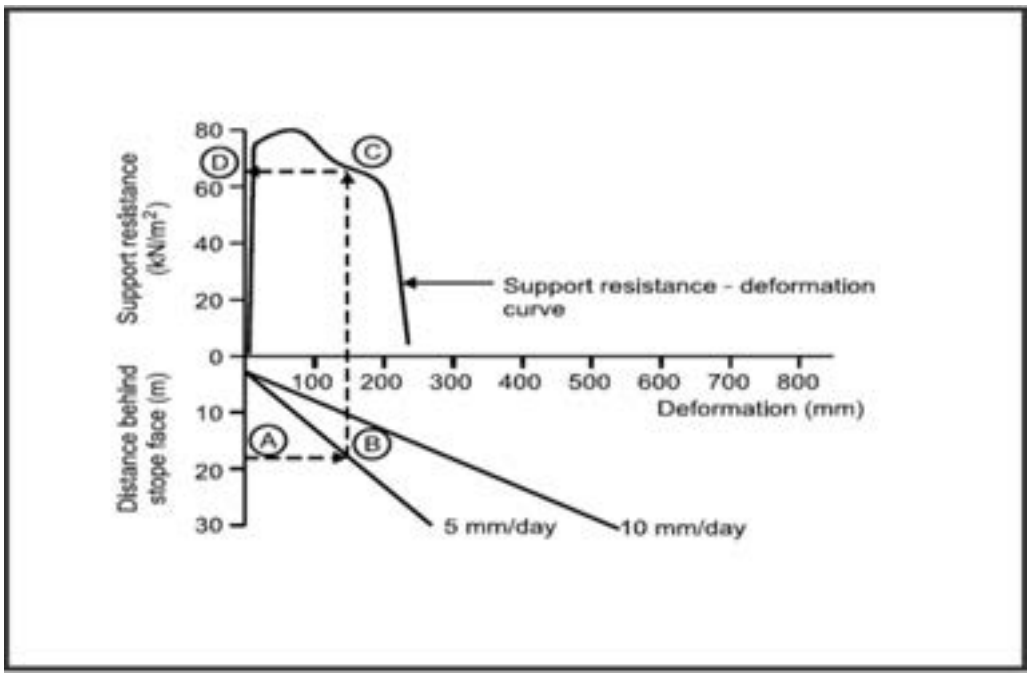


Figure 20: Support resistance as a function of closure (Ryder and Jager, 2002)

EXAMPLE

This graph can be used as follows:

a. Mark the position from the face where the support is currently situated (Example: 18m);
b. Draw a horizontal line to the appropriate closure rate line (Example: 5mm/day);
c. Draw a vertical line to the support resistance graph of the support unit;
d. Draw a horizontal line to intersect the Support Resistance axis and read off the value (Example: 65 kN/m2).

NOTE

Figure 20 was drafted for a specific support type (load – deformation graph) installed at a specific spacing to give the Support Resistance graph. At the same time, the appropriate closure rate lines for the specific area (5mm/day and 10mm/day) was included and the closure rate lines start at the distance at which is support is installed from the face in this specific area (± 2m).

IMPORTANT

You are required to draft your own versions of Figure 20 to cater for your specific applications.

LEARNING OUTCOME 5.4.1.6

CALCULATE ENERGY ABSORPTION FOR SUPPORT SYSTEMS MADE UP OF VARIOUS UNITS

CONNECTION 215

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352 for explanation for the use of the types of graphs also to be used in determining the energy absorption of a number of support units.

REFERENCE

- Ryder and Jager page 325.

LEARNING OUTCOME 5.4.1.7

CALCULATE REQUIRED SUPPORT RESISTANCES AND ENERGY ABSORPTION FOR COMPLETE MINING CYCLES TO ENSURE MINIMA ARE ALWAYS ACHIEVED THROUGHOUT THE CYCLE

The mining cycle refer to the process where support is installed and the face advanced until the next line of support can be installed. This means that the support resistance generated within a certain distance from the face constantly changes as the mining continues through this cycle. Within a selected tributary area, the appropriate support resistances through the mining cycle can be calculated after each face advance.

EXAMPLE

It is required that the support resistance criterion is met at all times in a working place when personnel is allowed to enter the working place.

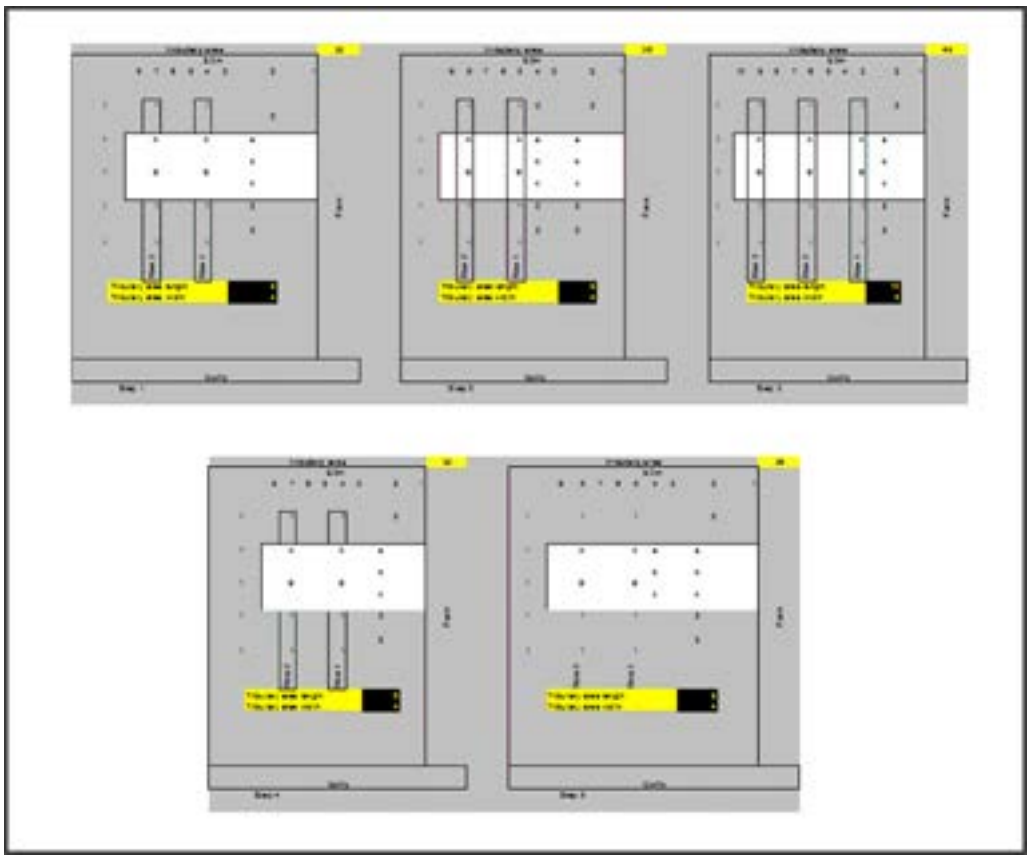


Figure 21: Mining cycle impact on tributary area

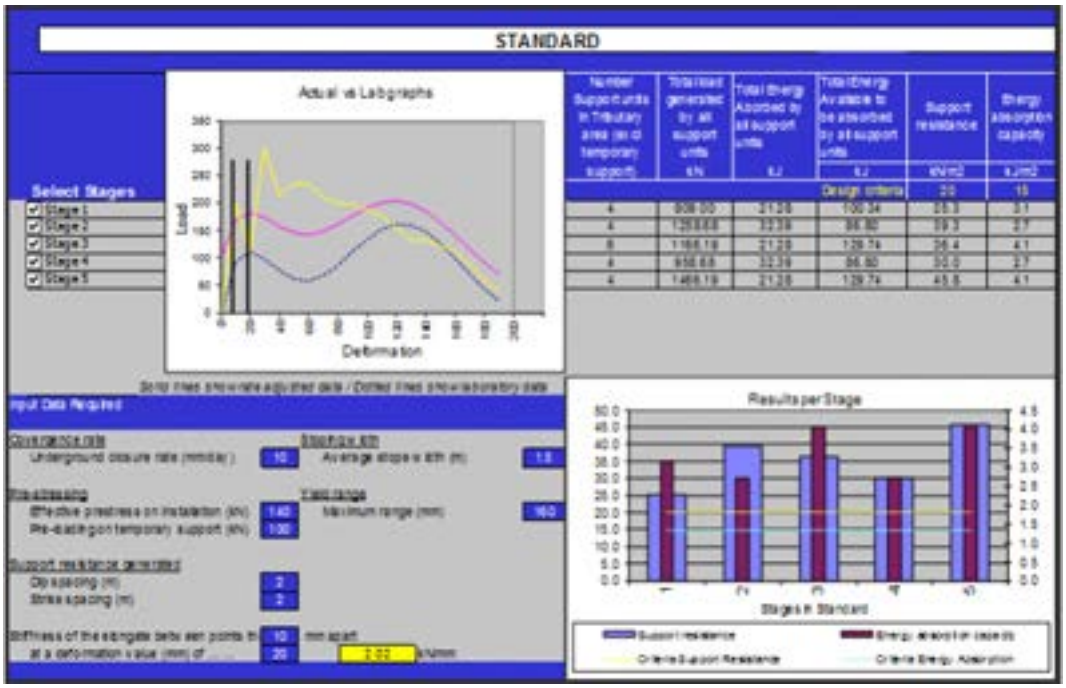


Figure 22: Support resistance for the full mining cycle

- Spreadsheet Mining cycle support resistance

REFERENCE

LEARNING OUTCOME 5.4.1.8

CONSIDER THE EFFECTS OF VARIABILITY OF SUPPORT UNIT QUALITY

CONNECTION 216

Refer to LEARNING OUTCOME 5.4.1.5 on page 354 and LEARNING OUTCOME 5.4.1.6 on page 356

LEARNING OUTCOME 5.4.1.9

CONSIDER THE EFFECTS OF VARIABILITY OF SUPPORT UNIT INSTALLATION.

CONNECTION 217

Refer to LEARNING OUTCOME 5.4.1.5 on page 354 and LEARNING OUTCOME 5.4.1.6 on page 356

5.4.2. TUNNEL, CHAMBER AND SHAFT SUPPORT DESIGN CRITERIA

LEARNING OUTCOME 5.4.2.1

CONNECTION 218

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352

DESCRIBE, EXPLAIN AND DISCUSS EXCAVATION SUPPORT AND ROCKWALL REINFORCEMENT REQUIREMENTS IN TERMS OF :

- Initial stiffness,
- Yieldability,
- Unit length,
- Areal coverage,

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- Area of influence,
- Beam reinforcement,
- Suspension,
- Support resistance,
- Energy absorption

Support element		Initial stiffness	Yield capacity	Load capacity	Shear capacity	Comments (applicability)
Friction tendon:	Swellex/ split set	Fair	Fair	Low	Fair/poor	Simple installation, corrosion a problem, primary support only
End anchored Pre-ten-sioned:	Rock stud	Very good	Poor	Medium	Fair	Used under low stress conditions.
	Cable anchor	Very good	Fair	High	Good	Large excava-tions/wide areas
Fully grouted:	Smooth bar	Good	Fair	Medium	Fair	Easily debonded, re-quires good grouting
	Rebar	Very good	Poor	Medium	Poor	High initial stiffness, requires good grouting
	Drill steel	Very good	Poor	High	Poor	High shear resistance
	Yielding tendon	Fair	Very good	Medium	Fair/good	Good yieldability
	Cable tendon>4m	Fair	Fair/ poor	High	Good	Yieldability + flex-ibility, may require good grouting
	Wire loops,3m	Fair	Fair/ poor	Medium	Good	Yieldability + flex-ibility, may require good grouting
Sets:	Arches and Cribbing	Poor	Fair	High		Areal coverage in poor areas
Fabric:	Mesh and lacing	Poor	Good	Low		Areal coverage + flex-ibility, labour intensive
	Reinforced shotcrete (50mm)	Good	Fair/ poor	Medium		Areal coverage + limited deformability
	Reinforced shotcrete (50mm) & lace	Good	Good	Good		Areal coverage + limited deformability
	Un-reinforced Shotcrete (50mm)	Good	Poor	Medium		Areal coverage in areas of low deformation

Table 15: Support element design characteristics (Jager, et al)

LEARNING OUTCOME 5.4.2.2

CONNECTION 219

Refer toSTOPE SUP-PORT DESIGN CRITERIA on page 352

DESCRIBE, EXPLAIN AND DISCUSS HOW SUPPORT RESISTANCE VARIES FOR STATIC AND DYNAMIC CONDITIONS

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LEARNING OUTCOME 5.4.2.3

CONNECTION 220

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352

CALCULATE REQUIRED SUPPORT RESISTANCES FOR STATIC CONDITIONS

LEARNING OUTCOME 5.4.2.4

CONNECTION 221

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352

CALCULATE REQUIRED SUPPORT RESISTANCES FOR DYNAMIC (ROCKBURST) CONDITIONS

LEARNING OUTCOME 5.4.2.5

CONNECTION 222

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352

CALCULATE SUPPORT LOADS FOR SUPPORT SYSTEMS MADE UP OF VARIOUS UNITS

LEARNING OUTCOME 5.4.2.6

CONNECTION 223

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352

CALCULATE ENERGY ABSORPTION FOR SUPPORT SYSTEMS MADE UP OF VARIOUS UNITS

LEARNING OUTCOME 5.4.2.7

CONNECTION 224

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352

CONSIDER THE EFFECTS OF VARIABILITY OF SUPPORT UNIT QUALITY

LEARNING OUTCOME 5.4.2.8

CONNECTION 225

Refer to STOPE SUPPORT DESIGN CRITERIA on page 352

CONSIDER THE EFFECTS OF VARIABILITY OF SUPPORT UNIT INSTALLATION.

5.5. SUPPORT UNIT AND SUPPORT SYSTEM TYPES AND CHARACTERISTICS

5.5.1. PILLARS AS STOPE SUPPORT

LEARNING OUTCOME 5.5.1.1

CONNECTION 226

Refer to LEARNING OUTCOME 3.1.18 on page 89

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE MEANING, APPLICATION AND BEHAVIOURAL CHARACTERISTICS OF: RIGID PILLARS, YIELD PILLARS, CRUSH PILLARS

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LEARNING OUTCOME 5.5.1.2

DESCRIBE, EXPLAIN AND DISCUSS THE FACTORS THAT SHOULD BE TAKEN INTO CONSIDERATION WHEN DESIGNING A PILLAR SUPPORT SYSTEM

The main issues to be evaluated when designing pillar support systems are:

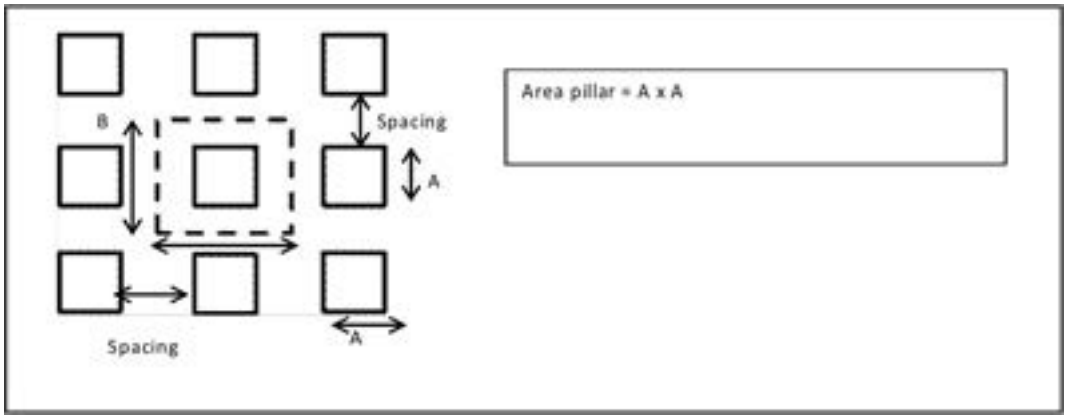
Stress acting on the pillars:
Due to mining, stress levels are induced on the remaining solid areas, which include the support pillars. This stress acts on the pillars and induces strain. As the pillar moves along its stress-strain behavioural curve, it approaches the yield and / or failure points where the stress exceeds the strength of the pillar. Pillar failure then commences and its post failure behaviour depends on its dimensions. The pillar stress can be estimated using the Tributary Area Theory or numerical modelling, each with its own assumptions and limitations.



The Tributary Area Theory (TAT) suggests that the stress on a pillar in a uniform layout of pillars can be determined as follows:

Stress = Virgin stress at that elevation / (1 – extraction ratio)
Or
$Pillar_{stress}=q/(1-e)$

The extraction ratio e is given by:
$e=(Area_{total}-Area_{pillar})/Area_{total}$ based on the following layout:



Strength of the pillars:
The strength of the pillars is determined by three different parameters, namely the pillar width, pillar height and rock mass strength where the dimensions are mostly given (unless potential exist for high-grading an orebody and extraction of only a portion of the orebody width). The rock mass strength is suggested to be the strength of a $1m^3$ sample of rock, including the discontinuities that are found in such a sample.

CONNECTION 226

Refer to Paper 2 Outcome 3.1.2.5 on page 150



The strengths of hard rock pillars are currently determined using the equation derived by Hedley and Grant, stating that $\sigma_s=k\ (w^{0.5}/h^{0.75})$ *where k is the rock mass strength.*

Behaviour of the pillars:
The behaviour of pillars are largely affected by the pillar stress levels and strengths (pre-failure) and their width : height ratios in the post-failure behaviour. The larger the width : height ration, the more ductile the post-failure behaviour becomes, while small width : height ratios give rise to sudden brittle failures (Figure 22).

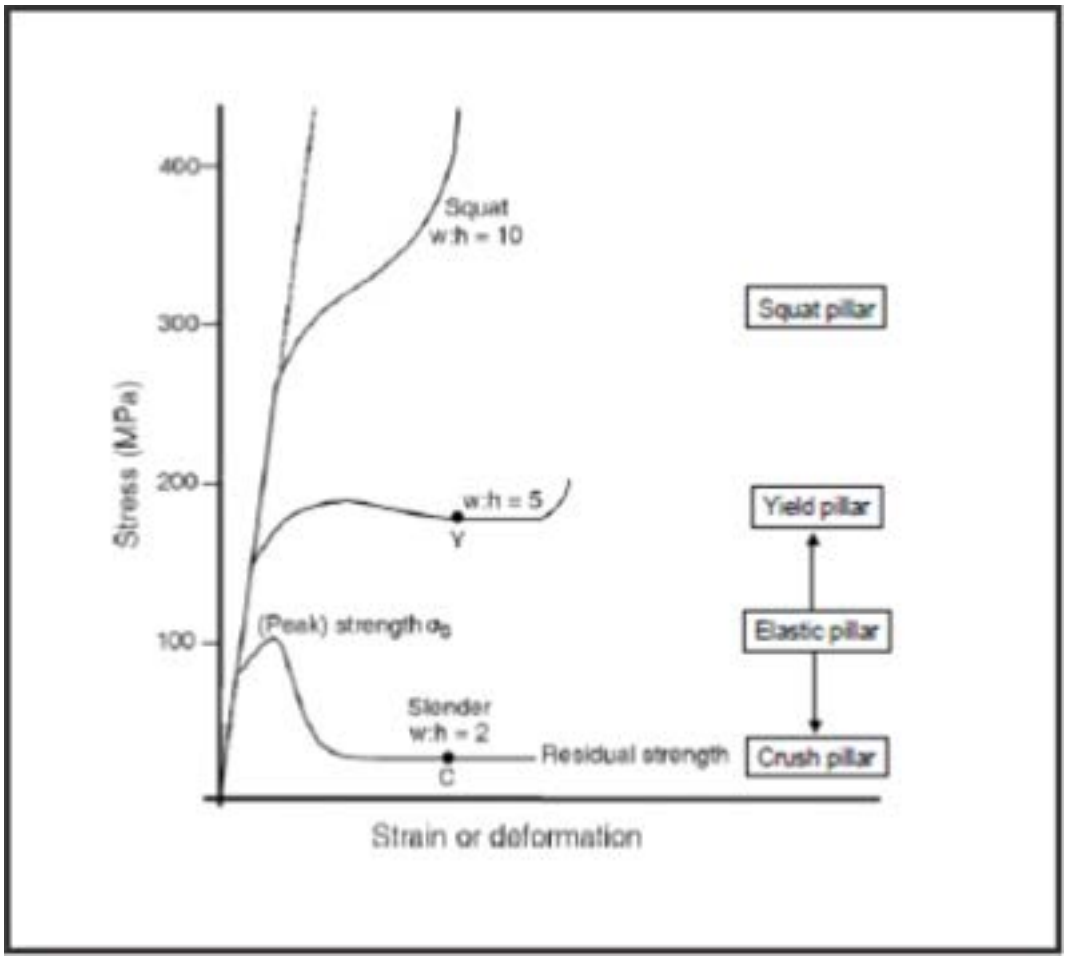


Figure 22: Pillar types (Ryder and Jager, 2002)

LEARNING OUTCOME 5.5.1.3

DESCRIBE, EXPLAIN, DISCUSS AND ACCOUNT FOR THE LIMITATIONS IN DETERMINING THE STRENGTH OF HARD ROCK PILLARS

Pillar strength (according to Hedley and Grantt) is given by

$\sigma_s = k\ (w^{0.5}/h^{0.75})$ where

k = rock mass strength.

CONNECTION 227

Refer to Paper 2 Outcome 3.1.2.5 on page 150

As discussed in LEARNING OUTCOME 5.5.1.2 on page 361, the strength of the pillar is determined by its dimensions and the rock mass strength. Due to the impossibility of testing a 1m3 sample, the value of k must be determined in some other way. Since it suggests rock mass and not intact rock sample strength, it follows that k < UCS. This resulted in a number of methods to determine k, which include (from work executed in the Industry):

- RMR x UCS: k is a fraction of the UCS, where the fraction is determined be the Rock mass quality parameter RMR (Bienawski).
- 2/3 x UCS: This 2/3 fraction of UCS can be found in some literature but is assumed to be founded on the execution of a large number of RMR ratings for a specific environment.
- DRMS: Determination of the Design Rock Mass Strength (Laubscher)
- Back analyses of pillar failures: A back analysis is suggested when sufficient failed and stable pillars are available for analyses.

- York and Canbulat suggested a method whereby the k value can be determined from laboratory data where samples of various dimensions were tested.



- Ryder and Jager Page 294
- Gap334

Other limitations and issues regarding this method are explained in the research report Gap334 on Pillar designs for shallow, intermediate and deep mining environments.

LEARNING OUTCOME 5.5.1.4

CONNECTION 228

Refer to Example in LEARNING OUTCOME 5.1.17 on page 306

DESCRIBE, EXPLAIN AND APPLY EMPIRICAL FORMULAE TO DERIVE THE STRENGTH OF HARD ROCK PILLARS

LEARNING OUTCOME 5.5.1.5

CONNECTION 229

Refer to Paper 2 Outcome 6.1.1.29

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE ROLE OF LOADING SYSTEM STIFFNESS IN THE STABILITY OF PILLAR SYSTEMS ONCE PILLARS HAVE EXCEEDED THEIR PEAK STRENGTH



Similar to the process described in the abovementioned outcome, the pillar behaves similar to the sample in the different loading systems.

LEARNING OUTCOME 5.5.1.6

CONNECTION 230

Refer to LEARNING OUTCOME 4.2.1.4 on page 191

DESCRIBE, EXPLAIN AND DISCUSS THE MECHANISM OF FOUNDATION FAILURE OF PILLARS

Foundation failure is stated to be the shear failure of the foundation material underneath highly stressed pillars (eg. stabilising pillars in a longwall deep level mine), but where large scale closure also occurs around the pillars. The mechanism is suggested to be:

- Highly stressed pillar punches into weaker footwall;
- Shear failure occurs along the edge of the pillar where
- The pillar moves downward into the footwall material and the surrounding rock 'heaves' around the pillar.

INTERESTING INFO

Foundation failure of pillars is best described by referring to laboratory testing of the foundation strength of samples during 'punch' testing.

In a punch test, a hardened steel punch is pressed against a large specimen of a foundation rock type (Figure 23). Horizontal confining pressure is applied to the specimen to avoid splitting caused by induced horizontal tensile stresses. Loads and deformations can be monitored with standard triaxial test transducers.



A convenient way of providing confinement is a steel annulus (Figure 23).



A suitable specimen shape is a cylinder of diameter at least five times the diameter of the die, with a width:height ratio of 1.

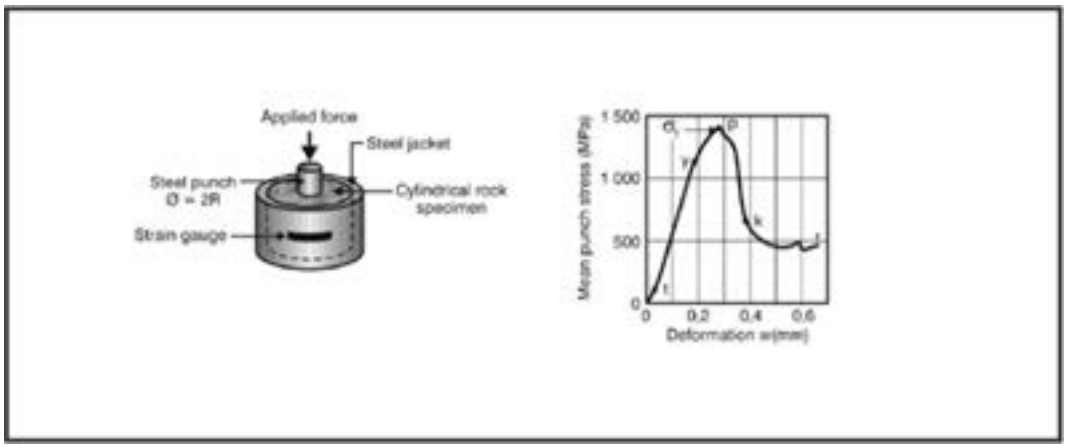


Figure 23: Punch test (Ryder and Jager, 2002)

Figure 23 illustrates the graphed results of a typical circular punch test (stress vs deformation).

At the 'yield' point y, tensile surface cracks and minor crushing round the rim of the punch occur. When the punch stress reaches its peak, the limiting 'foundation strength' is attained. Post-peak, there is rapid load-shedding and 'punching' into the specimen with a damaged zone under the rim of the punch and uplift of rock on the free surface near the punch ('footwall heave'). The stress then reaches quasi-constant yielding at a 'residual strength'.

REFERENCE

Ryder and Jager page 92

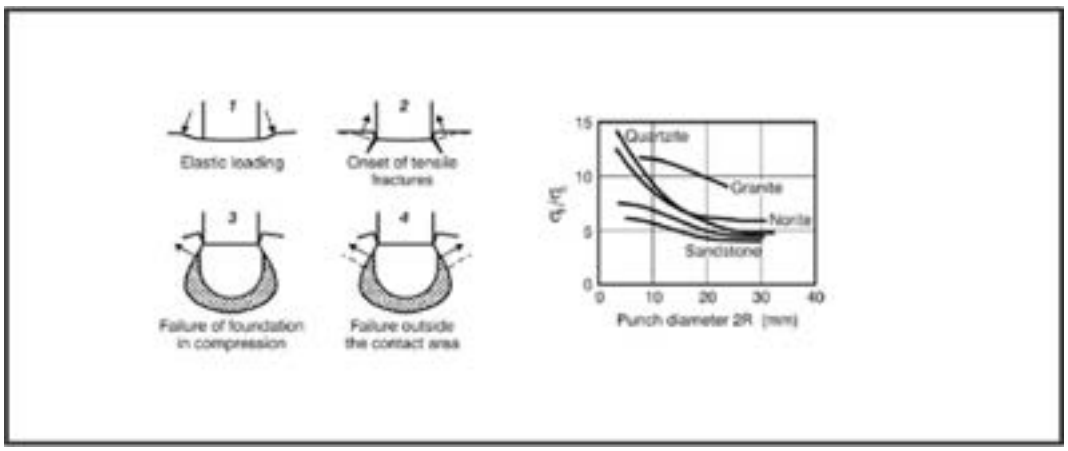


Figure 24: Foundation failure (Ryder and Jager, 2002)

Figure 24 demonstrates the damage in brittle foundation rock:

INTERESTING INFO

See the similarity between the crushed zones in Figure 21 and the location of seismic events beneath a stabilising pillar as shown in Figure 22.

- Initial tensile fracturing on the edge of the punch;
- crushing under the rim of the punch until the crushed zone extends to below the centre of the punch to form a complete failed zone;
- the growth of horizontal compressive strains on the free surface caused by the dilation of the crushed zone underneath the punch;
- finally, the dilation results in upward movement of the free surface followed by downward movement of the punch into the material.

NOTE

The magnitude of the peak punch stress ('foundation strength') is expressed as a multiple of the foundation rock's UCS (σ_c) and may fall in the range $3\sigma_c$ - $8\sigma_c$.

INTERESTING INFO

- Terzaghi's classical soil bearing-strength analyses: (based on Mohr-Coulomb-plastic material behaviour) over-predicts hard-rock foundation strengths [e.g. $9\sigma_c$ to $17\sigma_c$]. Most rocks possess significant cohesive strength but undergo rapid post-failure strain-softening, and the original Terzaghi formulae are thus inappropriate for these materials;
- Wagner & Schumann and Cook suggested that under purely elastic conditions, the stresses under the axis of the punch are sufficient to cause shear failure there. These calculations invariably under-predict the foundation strength to about 2σc.
- Pells & Turner focus on the mechanism of brittle foundation failure and recognises that abrupt load-shedding seems to be associated with failure and uplift of the free surface surrounding the punch, called 'footwall heave' in an underground environment. The rock here is viewed as being subjected to 'nearly' uniaxial loading and suggests that at failure, the horizontal stress must be approximately equal to the UCS. The vertical stress σv in the crushed envelope of rock under the rim of the punch must then be given by σv = βσc, where β is the strengthening coefficient of the foundation rock. The ultimate foundation strength under a punch is expected to be equal to the triaxial residual strength of crushed rock under the (high) confining pressure σc. However, this suggests that foundation strength should increase with rock internal friction angle while it is independent of other factors. Careful application of this method to underground pillars is suggested while if used, the downgrading of the in situ UCS is critical.

REFERENCE

- Ryder and Jager Page 92

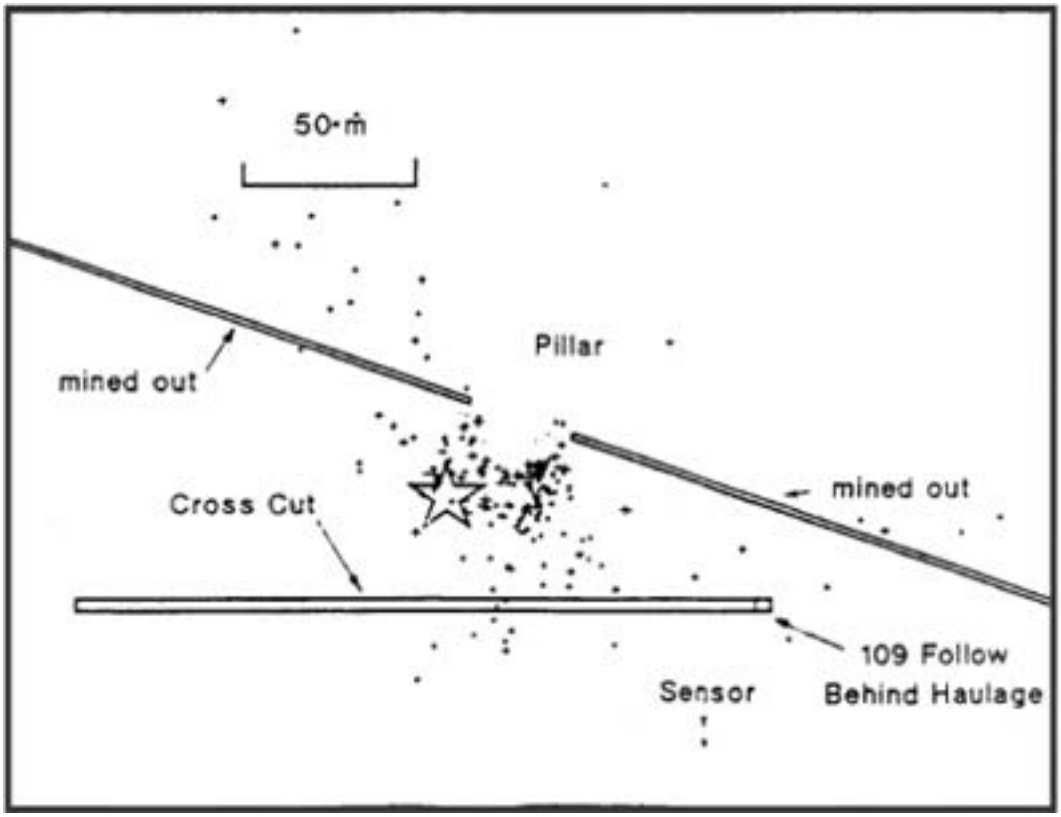


Figure 25: Foundation failure related seismic activity

Designs to avoid foundation failure have traditionally been based on the following rule of thumb:

$$\text{Design APS} \leq f_a.\sigma_c$$

where σ_c is the UCS of the foundation strata (hangingwall or footwall, whichever is the weakest), and fa is an empirical factor. This factor is suggested to be:

- 2.5 - COMRO 1988;
- 1.2 - Handley et al (1997)

However, York (1998) indicated that foundation strength is not a simple multiplier of UCS but that cohesion, friction angle and stress field (k-ratio) all have an impact on this foundation failure APS and indicated the following relationship:

$$\sigma_y \approx 0.8\ \beta_o\ [C_o\ (\sigma_h + 30)]^{(1/2)}$$

Where the strengthening coefficient β is given by

$$\beta_o = (1+\sin\Phi_i)/(1-\sin\Phi_i)$$

and the cohesion is given by

$$C_o = \sigma_c/(2\sqrt{(\beta_0)})$$

EXAMPLE

If the friction angle is 45°, the UCS is 200 MPa and σ_h = 40 MPa:

The strengthening factor β_o is

$$\begin{aligned}\beta_o &= (1+\sin\Phi_i)/(1-\sin\Phi_i)\\ &= (1+\sin 45°)/(1-\sin 45°)\\ &= 1.707 / 0.293\\ &= 5.826\end{aligned}$$

And cohesion is

$$\begin{aligned}C_o &= \sigma_c/(2\sqrt{(\beta_0)})\\ &= (200\text{MPa}) / (2(\sqrt{5.826}))\\ &= (200\text{MPa}) / 4.827\\ &= 41.433\ \text{MPa}\end{aligned}$$

And foundation strength σ_y is

$$\begin{aligned}\sigma_y &\approx 0.8\ \beta_o\ [C_o\ (\sigma_h + 30)]^{(1/2)}\\ &= 0.8\ (5.826)\ [41.33\ \text{MPa}(40\ \text{MPa} + 30\ \text{MPa})]^{1/2}\\ &= 4.661\ [2.8931\text{E}15\ \text{Pa}]^{1/2}\\ &= 4.661\ [53.787\text{MPa}]\\ &= 250.704\text{MPa}\end{aligned}$$

This means that the f_a factor in Foundation strength = f_a.UCS is given by

$$\begin{aligned}fa &= (\sigma_y / \text{UCS})\\ &= (250.707\ \text{MPa} / 200\ \text{MPa})\\ &= 1.25\end{aligned}$$

And not the 2.5 suggested by some of the other rules.

REFERENCE

- Foundation Failure York

LEARNING OUTCOME 5.5.1.7

CONNECTION 231

Refer to LEARNING OUTCOME 4.2.1.4 on page 191and LEARNING OUTCOME 5.5.1.6 on page 363

DESCRIBE, EXPLAIN AND DISCUSS THE FACTORS THAT AFFECT FOUNDATION FAILURE OF PILLARS

LEARNING OUTCOME 5.5.1.8

CONNECTION 232

Refer to LEARNING OUTCOME 4.2.1.7 on page 193

DESIGN IN-STOPE PILLAR SUPPORT LAYOUTS FOR : BREAST MINING, UP-DIP MINING.

5.5.2. BACKFILL AS STOPE SUPPORT

LEARNING OUTCOME 5.5.2.1

CONNECTION 233

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207 , LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE MAIN TYPES OF BACKFILL USED IN TABULAR, HARD ROCK MINING SUPPORT

LEARNING OUTCOME 5.5.2.2

CONNECTION 234

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207 , LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PREPARATION, RETICULATION AND PLACEMENT OF THE MAIN TYPES OF BACKFILL USED IN MINING SUPPORT

LEARNING OUTCOME 5.5.2.3

CONNECTION 235

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207 , LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE APPLICATIONS OF THE MAIN TYPES OF BACKFILL USED IN MINING SUPPORT

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LEARNING OUTCOME 5.5.2.4

CONNECTION 236

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE IMPORTANT BACKFILL MATERIAL CHARACTERISTICS THAT AFFECT PERFORMANCE

LEARNING OUTCOME 5.5.2.5

CONNECTION 237

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE IMPORTANT BACKFILL PLACEMENT CHARACTERISTICS THAT AFFECT PERFORMANCE

LEARNING OUTCOME 5.5.2.6

CONNECTION 238

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

DESCRIBE, EXPLAIN AND DISCUSS THE BACKFILL CHARACTERISTICS REQUIRED FOR FILLING FLAT-DIPPING NARROW TABULAR STOPES AT DEPTH

LEARNING OUTCOME 5.5.2.7

CONNECTION 239

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

DESCRIBE WHICH BACKFILL TYPES ARE SUITABLE FOR FILLING FLAT-DIPPING NARROW TABULAR STOPES AT DEPTH

LEARNING OUTCOME 5.5.2.8

CONNECTION 240

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

DESCRIBE, EXPLAIN AND DISCUSS THE BACKFILL CHARACTERISTICS REQUIRED FOR FILLING SHALLOW WIDE-REEF TABULAR STOPES

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LEARNING OUTCOME 5.5.2.9

CONNECTION 241

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

DESCRIBE WHICH BACKFILL TYPES ARE SUITABLE FOR FILLING SHALLOW WIDE-REEF TABULAR STOPES

LEARNING OUTCOME 5.5.2.10

CONNECTION 242

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

DESCRIBE, EXPLAIN AND DISCUSS HOW BACKFILL ASSISTS IN IMPROVING THE STABILITY OF ROCKWALLS AT THE FACE

LEARNING OUTCOME 5.5.2.11

CONNECTION 243

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

DESCRIBE, EXPLAIN AND DISCUSS HOW BACKFILLING FITS INTO THE MINING CYCLE

LEARNING OUTCOME 5.5.2.12

CONNECTION 244

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

CALCULATE THE SUPPORT RESISTANCE OF BACKFILL GIVEN THE BACKFILL PERFORMANCE CHARACTERISTICS AND THE STOPE CLOSURE RATE

LEARNING OUTCOME 5.5.2.13

CONNECTION 245

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

EVALUATE POTENTIAL BENEFITS OF BACKFILLING FOR GIVEN ROCKMASS CONDITIONS AND MINING SITUATIONS

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CONNECTION 246

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

LEARNING OUTCOME 5.5.2.14

DETERMINE APPROPRIATE BACKFILL TYPES FOR GIVEN ROCKMASS CONDITIONS AND MINING SITUATIONS

CONNECTION 247

Refer to Paper 2 Outcome 5.3

LEARNING OUTCOME 4.2.4.1 on page 207, LEARNING OUTCOME 4.2.4.2 on page 210 and LEARNING OUTCOME 4.2.4.3 on page 212

LEARNING OUTCOME 5.5.2.15

DETERMINE APPROPRIATE PLACEMENT PROCEDURES FOR GIVEN ROCKMASS CONDITIONS AND MINING SITUATIONS.

5.5.3. PACK AND ELONGATE SUPPORT IN STOPES

LEARNING OUTCOME 5.5.3.1

DESCRIBE, EXPLAIN AND DISCUSS THE STRESS-STRAIN BEHAVIOUR OF TIMBER:

- When compressed parallel to the grain
- When compressed perpendicular to the grain

The stress-strain behaviour of a support unit described how the support unit reacts to deformation (i.e. it is being compressed) by generating load (force) in response to the deformation.



Normally this behaviour is reported as a force-displacement graph, and not as a stress – strain graph. However, it is easy to see convert force and displacement to stress and strain by dividing the force by the contact area of the support (stress = force / area) and taking the fraction of displacement to the height of the support unit (strain = change in height / original height).

This stress-strain or force-displacement behaviour of support is expressed in a graph such as that shown below in Figure 26. In this example, the behaviour of typical grout packs (A), pre-stressed timber packs (B), Elongates (C) and Hydraulic props (D) is shown.

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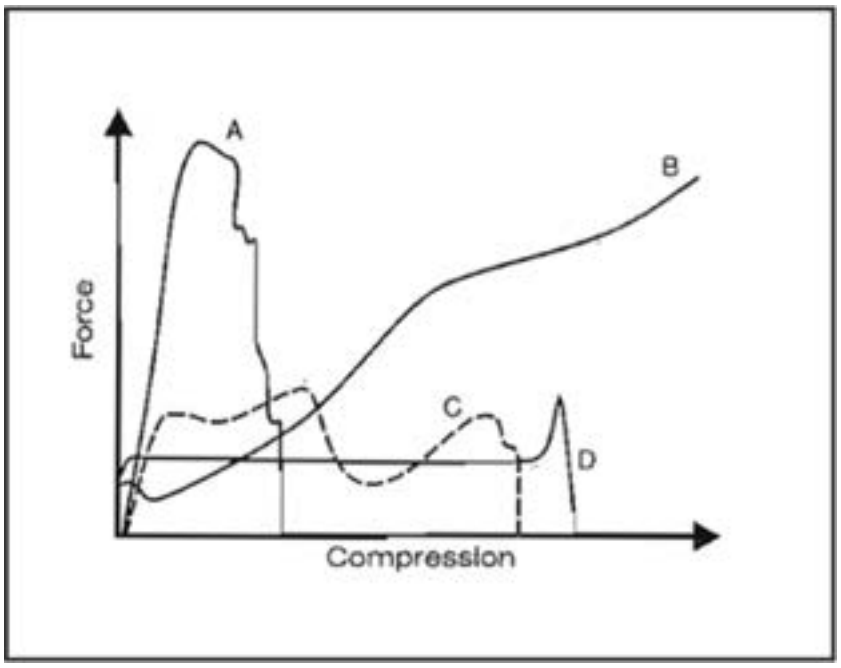


Figure 26: Support force-displacement graph (Jager and Ryder, 1999)

These behaviours have the following characteristics:

- Strength: Peak strength is the maximum force generated by the specific support unit;
- Stiffness: Is the slope of the force-displacement graph. Normally reference is made to the 'initial' stiffness only, which is the 1st part of the graph where the force increases with increasing displacement. High stiffness is a very steep (near vertical) line while low stiffness / soft behaviour is a very flat (near horizontal) line.
- Yieldability: Consist of two concepts, namely the yield load and the yield range. The yield load is the load at which the support unit is able to sustain the displacement without losing its ability to generate the load. This is usually a near horizontal line on the graph. The yield range is the displacement (mm) that the unit can be deformed by while generating the yield load.
- Energy absorption capacity: Is presented by the area under the graph on a force-displacement graph. The larger the area, the larger the units capacity to absorb energy. This means that high yield loads and high yield range support types will have high energy absorption capacity.
- Active: When the support unit generates load / force at ZERO (0mm) displacement. In this case the graph will start somewhere on the Force (y) axis and not at the origin (origin = 0kN,0mm)
- Passive: The support unit requires displacement before a force can be generated. This means the graph will start at the origin.

REFERENCE

- Page 137, Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999

EXAMPLE

Using Figure 26:

Support type	A	B	C	D
Strength	High strength at small displacements	High strength after significant displacement	Low strength	Low strength

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Support type	A	B	C	D
Stiffness	Very steep line – high stiffness	Flat dipping line – soft unit	Stiffness less than A but more than B	High stiffness
Yieldability	High yield load. Low yield range.	Increasing yield load, high yield range	Lower yield load, medium yield range	Constant, lower yield load, medium yield range
Energy absorption	Low – small area	High – large area	Medium	Medium
Active	No – line starts on x-axis	Yes – line starts on y-axis	No – line starts on x-axis	Yes – Line starts on y-axis
Passive	Yes – require displacement to gener-ate load	No – active	Yes – require displacement to generate load	No – active

When compressed parallel to the grain

A typical compressive loading of timber parallel to its grain (which is the common loading orientation for elongates in mining stopes) is shown in Figure 27 below.

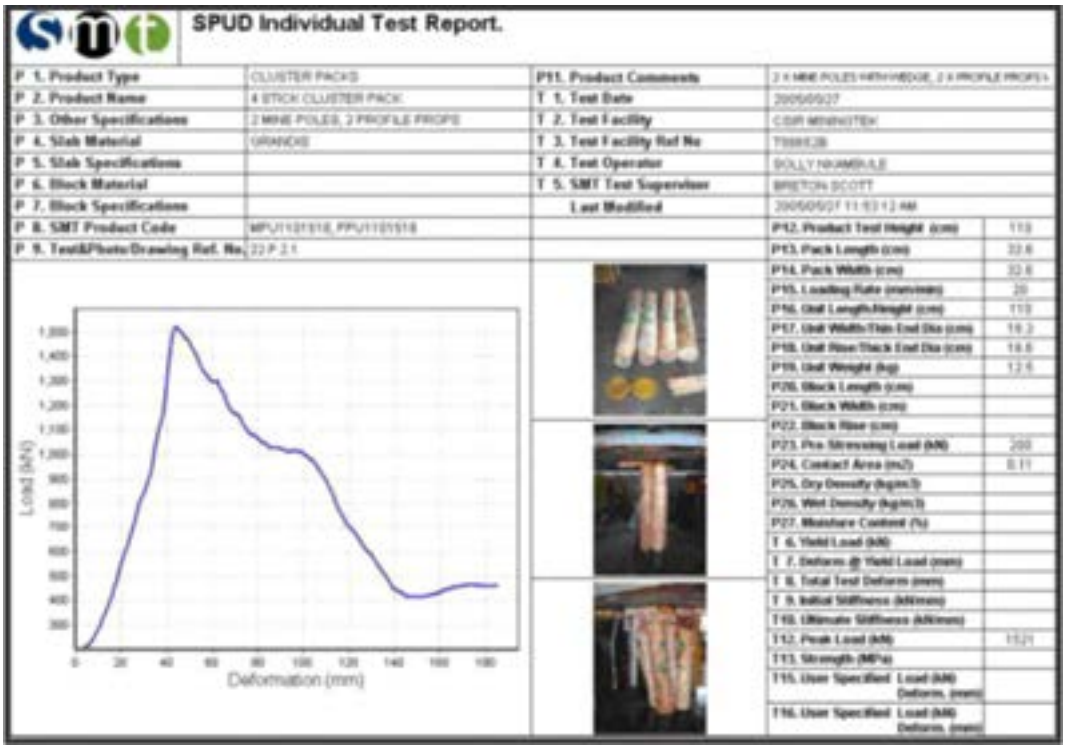


Figure 27: ompressive loading of timber parallel to the grain (Courtesy SMT)

Timber loaded parallel to its grain reaches a peak load at a particular deformation and from there on continue to deform through mushrooming, buckling or grain splitting gradually loosing load carrying capacity with increased deformation sustained. Eventually total collapse or failure of the timber occurs at various deformation values depending on the loading regime, timber strength and diameter.

When compressed perpendicular to the grain

Compressing a timber perpendicular to its grains has a similar effect to diametrical loading and the timber is expected to split along its diameter like in the Brazilian Indirect Tensile strength test.

This behaviour is typically observed in timber packs as loaded from typical roof closure in deep high stress mines. Due to a reduction in stiffness

CONNECTION 248

Refer to Paper 2 Outcome 6.1

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compared to the 'perpendicular loading regime', the load deformation graph shows initial 'soft' or 'yielding' characteristics with increased loading as compaction occurs and the response of the pack increases, Figure 28.

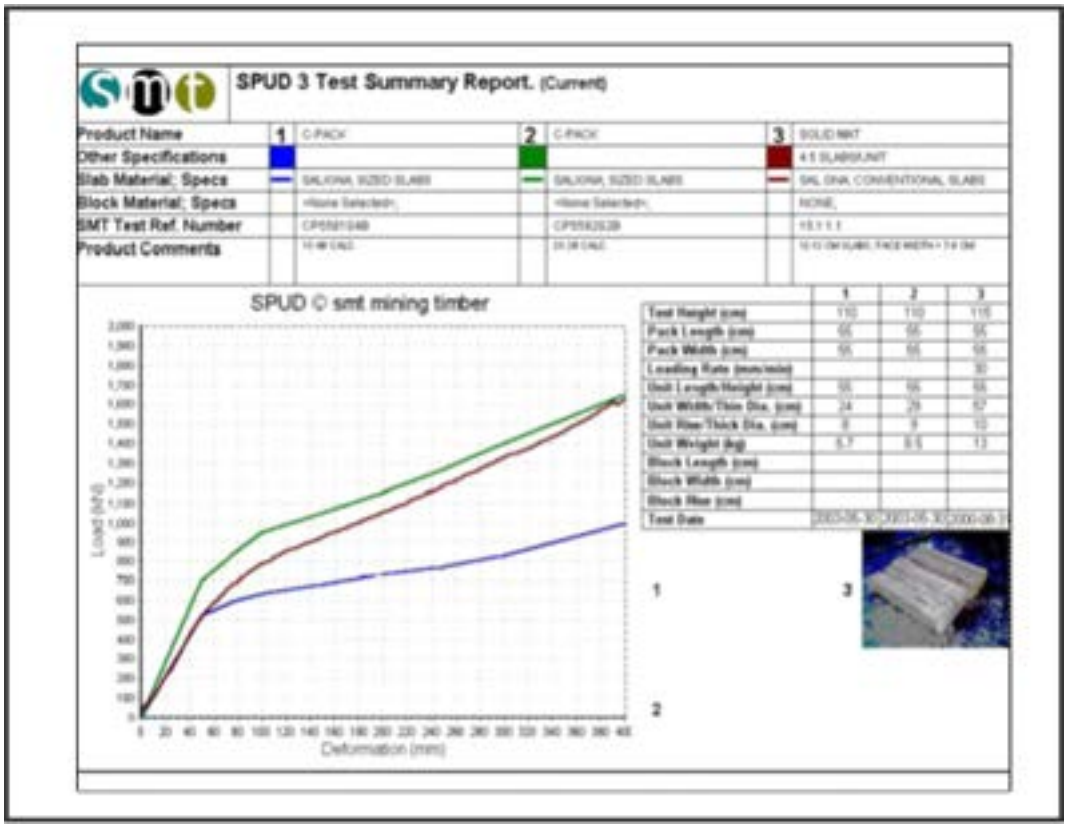


Figure 28: Load deformation graph for a timber pack with loading perpendicular to the grains.

EXAMPLE

Using the tests conducted on cluster sticks (Figure 27) and timber packs (Figure 28), the following results are clear.

- After 40mm closure loading parallel to the grain of the timber (elongate) gave a response from the support of 1300 kN.
- At the same time, 40mm closure when compressed perpendicular to the grain (packs), yielded only a response of 600kN.
- At a closure of 170mm, the elongate system has all but failed and generated a low 400 kN, whilst the pack was now at a level of 1100kN and increasing.

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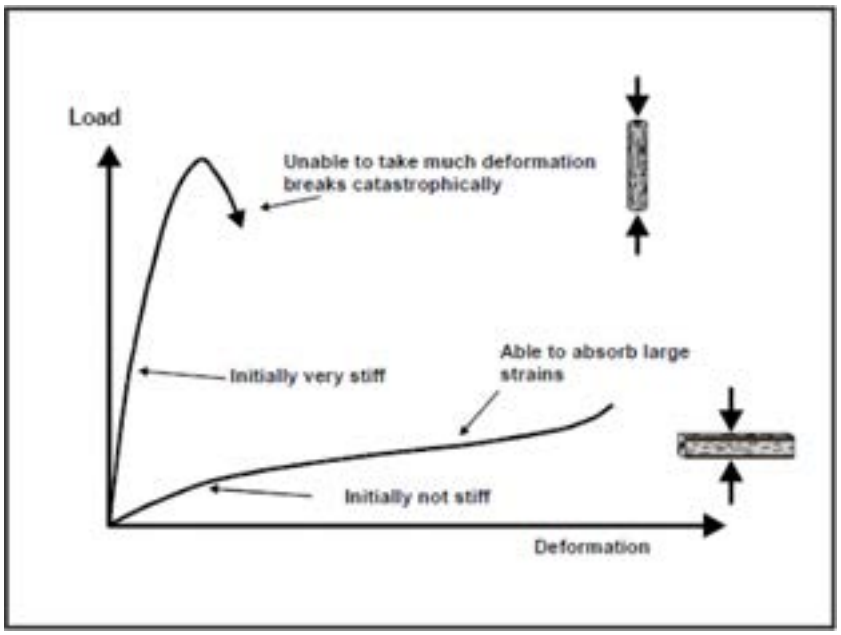


Figure 30: Anisotropic performance of timber (SIMRAC, Gap 508)

LEARNING OUTCOME 5.5.3.2

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF LOADING RATE ON THE STRESS-STRAIN BEHAVIOUR OF TIMBER

Loading rate refers to the fact that the rate at which the support unit is compressed. It is clear that this compression rate is important as it is known that the underground closure rate varies within a stope and across the mine or mines, and is therefore not a constant parameter.

IMPORTANT

To be able to compare the different characteristics of the support units (LEARNING OUTCOME 5.5.3.1 on page 370) with each other, a constant method of support testing is required. This means that all support tests should thus be conducted at the same compression rate. However, if the underground closure rate is different from the testing compression rate, then the characteristics from the test is not appropriate to underground and we cannot use those results to design support.

INTERESTING INFO

To solve this concern, a large number of support tests were conducted in RSA (elongates) and Germany (packs) at various loading rates. These test results allowed the development of an appropriate system to adapt the test results to the actual expected closure rates in any specific area of application.

REFERENCE

Page 315, Ryder JA and Jager AJ, A textbook on rock mechanics for tabular hard rock mines, 2002

The following relationships were derived:

Timber elongates:

$F_{ug}=F_{Lab}\ [m\log(v_{ug}/v_{lab})+1]$

Timber packs:
$F_{ug}=F_{Lab}\ (1+16/100)^{(\log vug/vlab\ )}$

Cementitious packs:
$F_{ug}=F_{Lab}(1+10/100)^{(\log vug/vlab\ )}$

Where F_{ug}=force generated underground ,
F_{lab} = force generated in laboratory test,
m = 0.123 for rockburst conditions and
m=0.084 for rockfall conditions ,
v_{ug}=underground closure rate (mm/minute) ,
v_{lab}= laboratory compression rate (mm/minute).

EXAMPLE

For rockfall conditions, determine the peak strength of the following support types when placed underground, applying the following input parameters:

Test compression rate = 30 mm/min, underground closure rate 10mm/day and peak strengths as shown in below.

Support type	Laboratory (given)	Underground (calculated below)
Elongate	200 kN	138 kN
Timber pack	1.5 MN	875 kN
Cementitious pack	500 kN	353.5 kN

Example calculations:

Convert the underground closure rate to mm/minute:

10mm/day means that 10mm closure occurs in 24 hours or 1440 minutes. This calculates to (10mm / 1440 minutes) = 0.007 mm/minute.

Elongate:

F_{ug} $=F_{Lab}\ [m\log(v_{ug}/v_{lab}\ )+1]$
$= 200kN\ [0.084.\text{Log}\ (0.007 / 30) + 1]$
$= 200kN\ [0.084.(- 3.632)+1]$
$= 200kN\ [\ - 0.305 + 1]$
$= 200\ kN\ [0.695]$
$= 138\ kN$

Timber pack:

F_{ug} $F_{Lab}\ (1+16/100)^{(\log vug/vlab\ )}$
$= 1.5MN\ (1.16)^{\text{Log}(0.007/30)}$
$= 1.5MN\ (1.16)^{-3.632}$
$= 1.5MN\ (0.583)$
$= 0.875\ MN$
$= 875\ kN$

Cementitious pack:

F_{ug} $=F_{Lab}(1+10/100)^{(\log vug/vlab\ )}$
$= 500kN\ (1.1)^{\text{Log}(0.007/30)}$
$= 500kN\ (1.1)^{-3.632}$
$= 500kN\ (0.707)$
$= 353.5\ kN$

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These loading rate adaptation formulae are not necessarily appropriate for new support units that are manufactured from materials that may behave differently under loading than those tested and utilised to provide these formulae. They should be used with care.

INTERESTING INFO

Gap508 summarised the downgrading of pack support into the following formula:

$F_{ug}=F_{Lab}\ (1+x)^{(\log vug/vlab\ )}$

Where x is now the rate adjustment factor and is expressed as a percentage but included into the equation as a fraction. Eg, 12% is used as (12/100) = 0.12.

Based on a number of tests the following rate ajustment factors should be used:

Pack type	Rapid loading rate effect (%)	Slow loading rate effect (%)
Solid mat	16	12
Composite timbers	12	12
Brick composite	16	16
Cementitious	10 - 12	10 - 12

LEARNING OUTCOME 5.5.3.3

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF MOISTURE CONTENT ON THE STRESS-STRAIN BEHAVIOUR OF TIMBER

Courtesy of Gap 508

"Probably the most critical factor that is within the control of the people who handle timber is the moisture content.

When the tree is felled, depending on the time of year and rainfall patterns, etc, the moisture content can be well in excess of 100 per cent (defined as the weight of water relative to the weight of dry wood).

The tree is left in the plantation for some period of time, to allow it to lose some of its excess water. It is then taken to the mill and processed and delivered to the mine.

From felling to delivery may take up to six weeks, again depending on the time of year, etc. In this time, the moisture content will have dropped to between 50 and 80 per cent.

The timber will then stand in the delivery or mine yard for another period of time during which the moisture content will drop still further, again depending on the conditions of storage. Dry, warm, windy conditions will obviously drop the moisture content more rapidly than cold wet weather.

When the timber is sent underground, the rate of moisture content reduction tends to be much lower than on surface as the ambient conditions will generally have high humidity and low air flow.

The lowering in moisture content, however, only becomes a problem when it drops below the fibre saturation point (around 28 per cent in Eucalyptus). In fact, the lowering of moisture content makes the timber lighter to handle.

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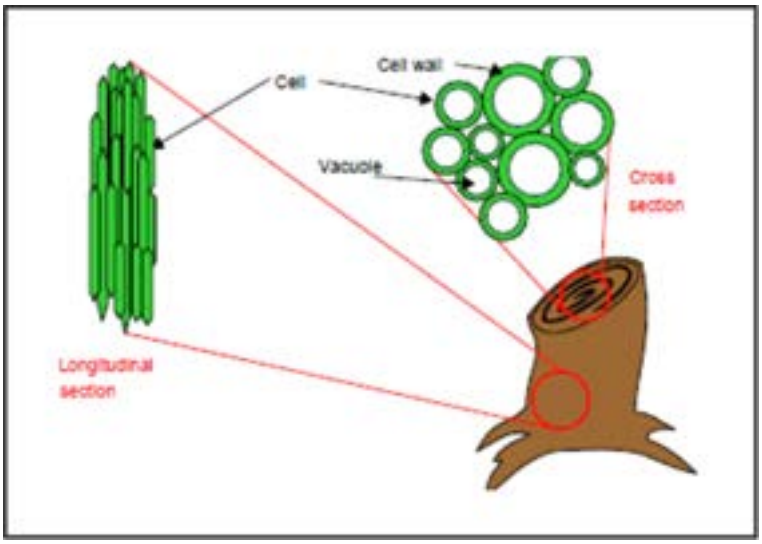


Figure 30: Sections showing the structure of timber (SIMRAC, Gap 508)

As the moisture content drops to the fibre saturation point, all that is removed is free water in the cell vacuoles. When the moisture content drops below this point, however, the moisture is then removed from the structure of the cell wall itself, and the wood becomes stronger. This may sound like an advantage, but the wood also becomes more brittle.

For elongates, this is highly disadvantageous, as they will snap and collapse rather than yield. For packs which use mostly the compressive strength of timber, such as mat and end grain type packs, this is not such a serious problem, as it just makes them stronger, though less able to withstand very large deformations. For packs such as composites, however, which use the tensile and shear strength properties of timber to a much greater extent, it can also be a major problem. The pack members will tend to fail sooner in compression than the equivalent fresh timber.

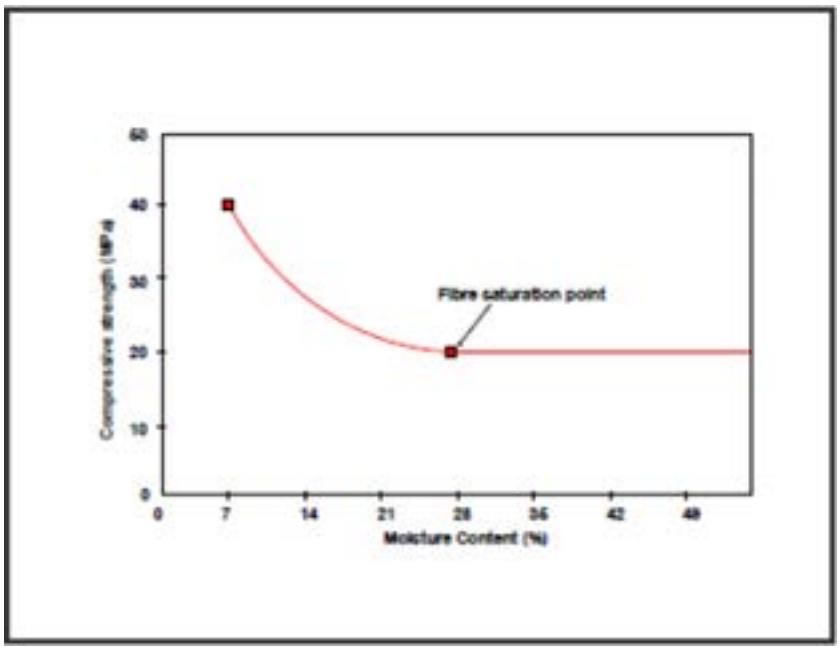


Figure 31: Relationship between moisture content and strength (SIMRAC, Gap 508)

LEARNING OUTCOME 5.5.3.4

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF CONSTRUCTION AND INSTALLATION ON THE STRESS-STRAIN BEHAVIOUR OF TIMBER

Construction and installation factors that affect its behaviour include:

- General effects: Width-to-height ratio, type, perpendicular to dip, etc
- Internal effects: Miss-alignment within pack between units, too many gaps in the pack, etc
- End effects: Smoothness of the hangingwall or footwall, point loading, etc
- Pre-stressing: Method used, position of the pre-stressing unit, etc
- Manufacturing quality: Constant dimensions, correct binding, etc

REFERENCE

- Gap 508 Support performance

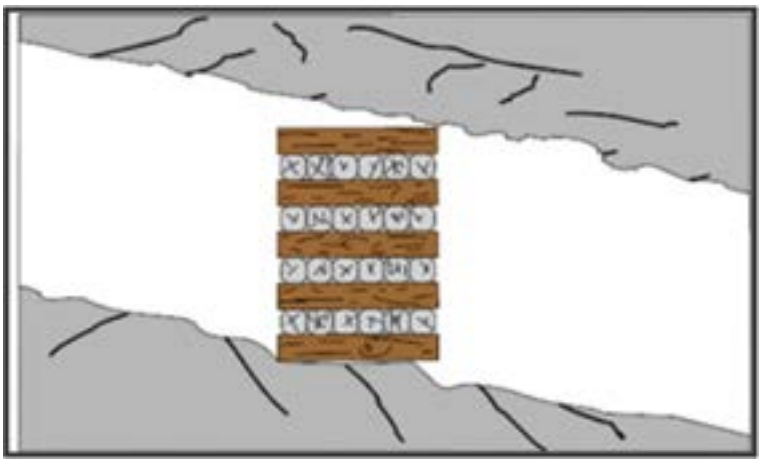


Figure 32: General effect

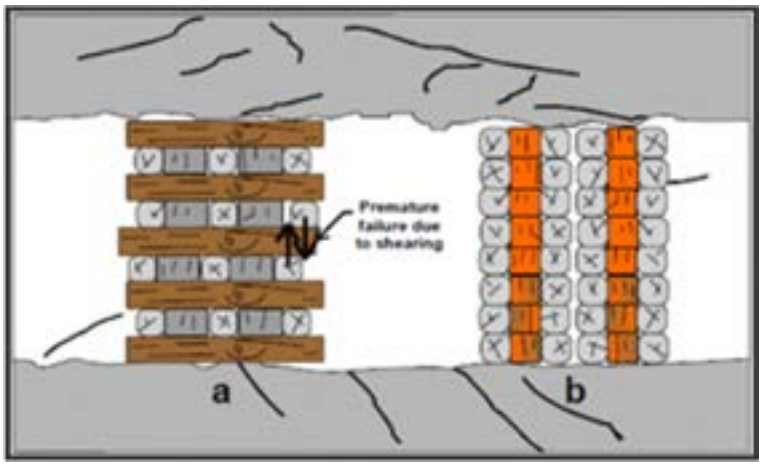


Figure 33: Internal effects

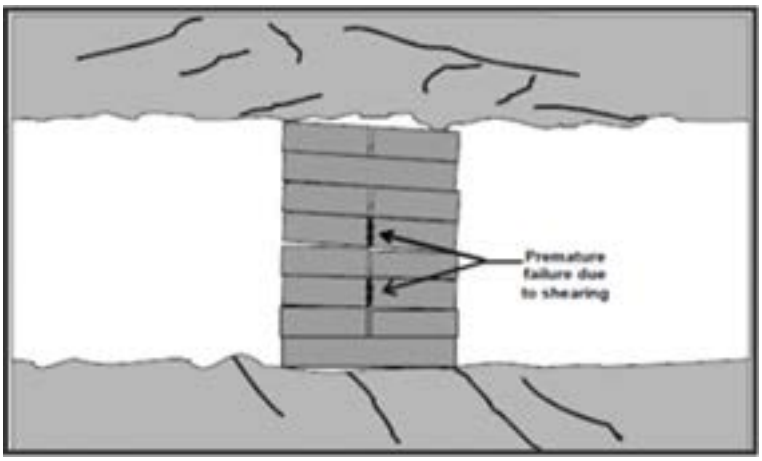


Figure 34: Manufacturing effects

LEARNING OUTCOME 5.5.3.5

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS TYPICAL LOAD-DEFORMATION CHARACTERISTICS OF THE FOLLOWING SUPPORT UNITS :

CONNECTION 249

Refer to Paper 2 Outcome 5.1 for general support related terms and definitions.

- Mine poles,
- Profile props,
- Pipe-sticks,
- Mechanical face props,
- Rapid-yielding hydraulic props,
- Cluster sticks,
- Other commonly available elongate support units,
- Timber packs,
- Timber-Cement (Composite) packs,
- Cement-based packs

REFERENCE

- Gap330 Stope face support systems

Mine poles

Minepoles fall in a category of elongate support that is simply a cut timber pole with no modifications to allow yielding or any changed behaviour. They are normally used in shallow stopes with low stope closure rates (1 to 5mm/day) and where minimal deformation of the hanging wall is allowed.

NOTE

A rule of thumb for the maximum deformation that a minepole can sustain to failure, is roughly 10% of the minepole length.
Their main purpose is to offer adequate support resistance to the hanging wall. Figure 35 below shows the load deformation curves of the mine pole in the lab and in an underground environment.

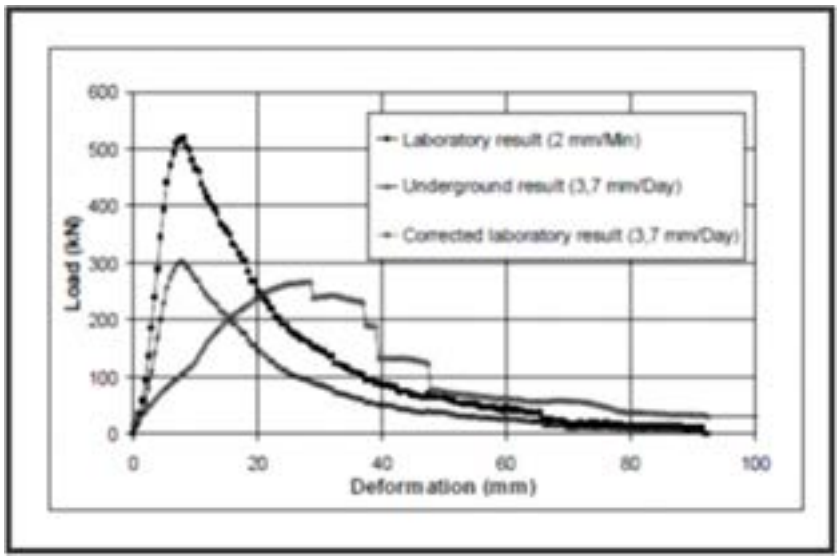


Figure 35: Load – Deformation behaviour of a minepole (unknown source)



The load-deformation graphs depicted in this material are examples and the user is advised to make sure that recent load-deformation curves appropriate to the length, diameter and product type considered is gathered and utilised.

Profile props / Pencil sticks

Profile props are timber props that have been sharpened at one end of the pole in a circular fashion to produce and 'pencil' shape. The weakness introduced in the sharpened end allows the prop to yield, typically up to about 150mm of deformation. This sharpening has a pronounced impact on the behaviour of the elongate, such as:

- The longer the distance over which the pole is sharpened, the larger the yield range of the unit (Figure 36a);
- The shallower the dip of the sharpened shape, the easier it yields and results at lower yield loads (Figure 36b);
- If the sharpened shape is not in the centre of the elongate, it will induce ride of the elongate (Figure 36c).

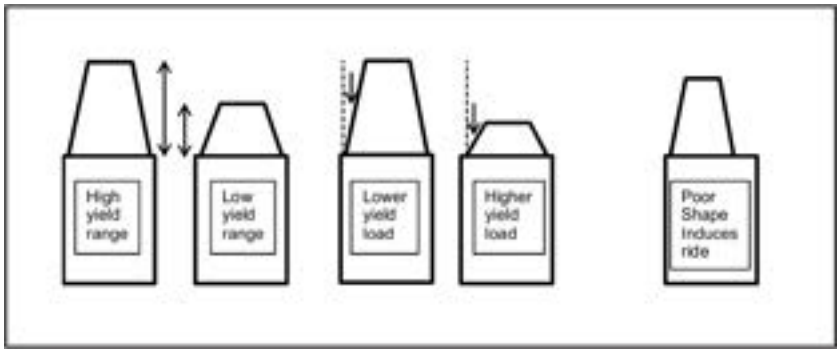


Figure 36: Profile prop shapes

When pre-stressed, these profile props offer enough support resistance and energy absorption capabilities to allow them to be used as face area support in seismically active stopes.

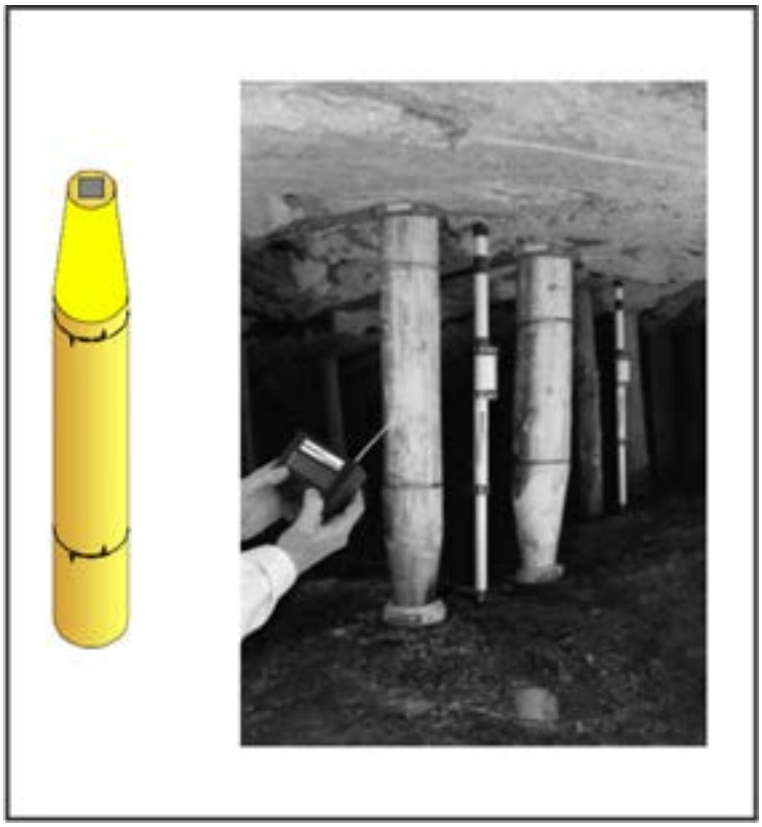


Figure 37: Profile / pencil sticks

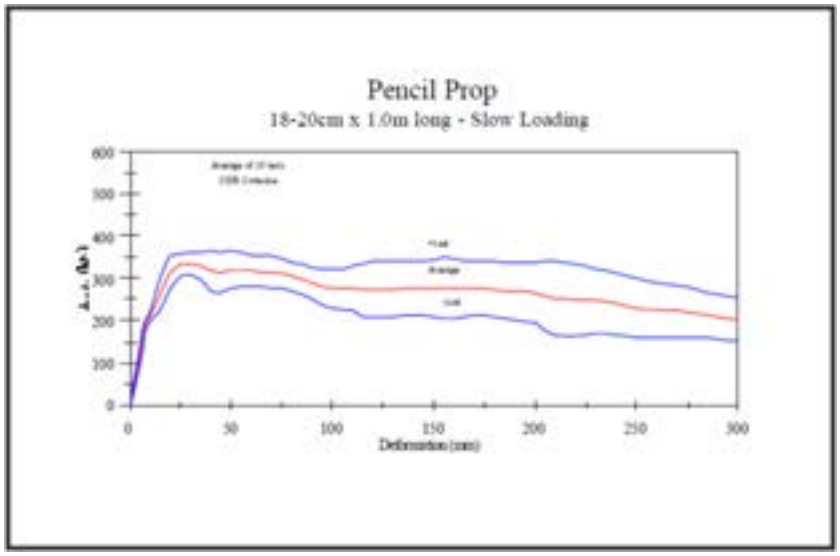


Figure 38: Load deformation curve for profile / pencil prop (unknown source)



The load-deformation graphs depicted in this material are examples and the user is advised to make sure that recent load-deformation curves appropriate to the length, diameter and product type considered is gathered and utilised.

Pipe-sticks

Pipe sticks were initially developed to provide stiff support close to the face with large yield ranges at high yield loads. They consist of a turned mine pole that is press-fitted into a steel pipe (Figure 39). When a pie-stick is installed with its long axis parallel to the direction of the loading force, it rapidly reaches its maximum load-bearing capacity and failure

back

of the pole begins. Enclosing a stick with a steel pipe provides lateral restraint to the failing timber and therefore enables the support system to carry considerable loads long after the timber stick has started to fail.

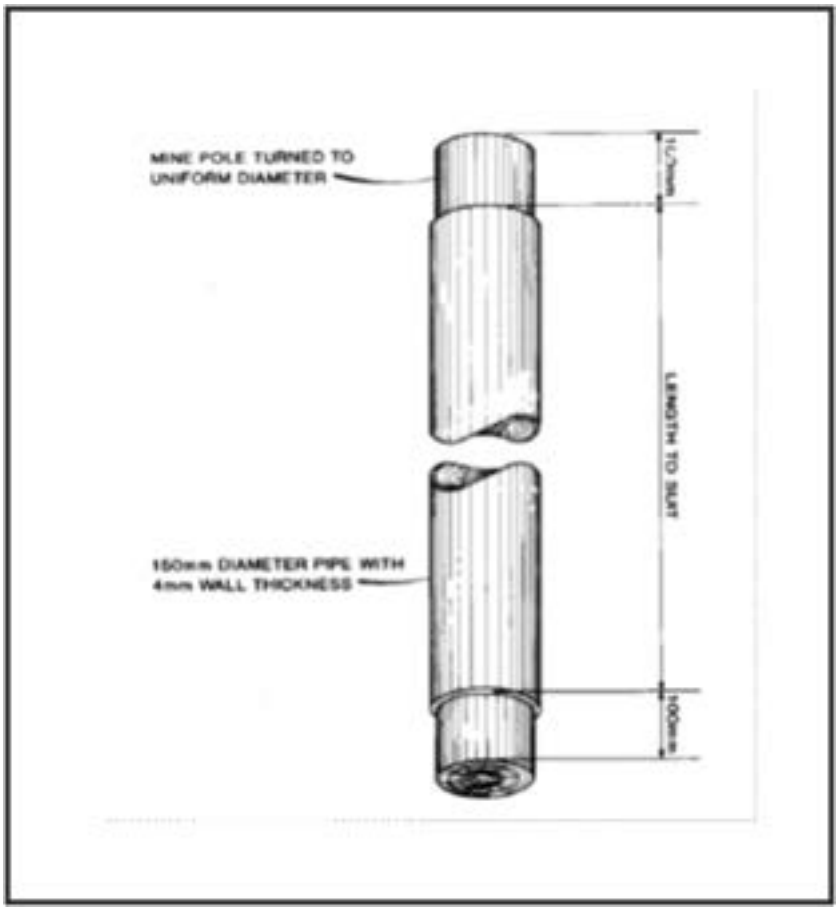


Figure 39: Pipestick (unknown source)

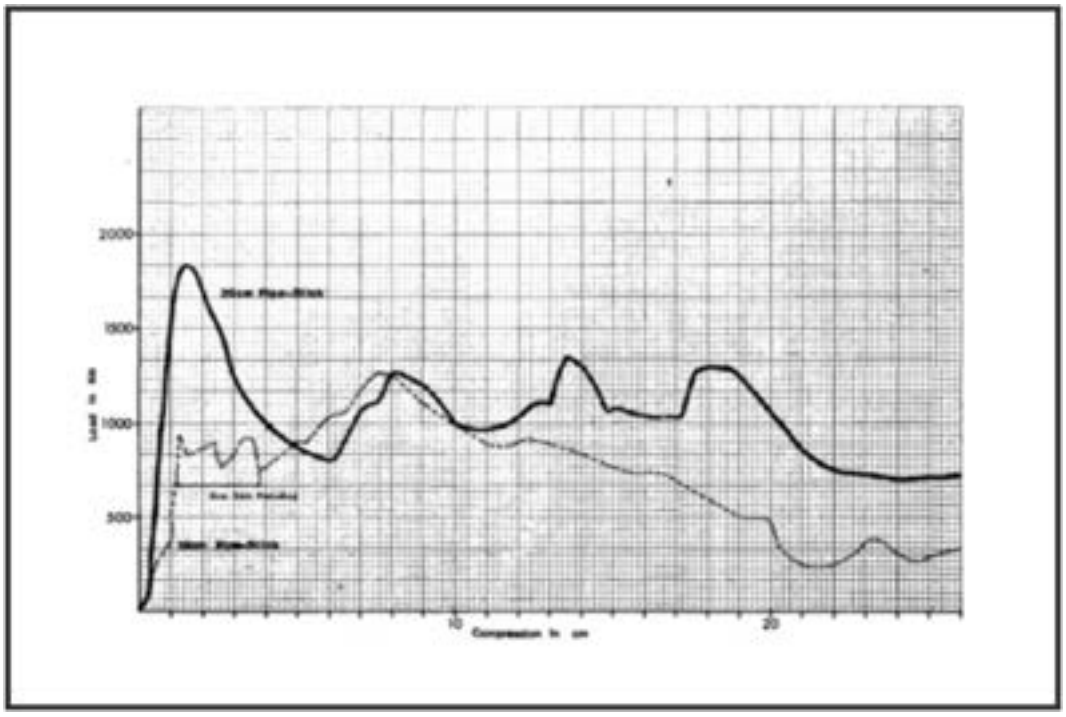


Figure 40: Typical load deformation curves of different diameter pipe sticks (unknown source)

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The load-deformation graphs depicted in this material are examples and the user is advised to make sure that recent load-deformation curves appropriate to the length, diameter and product type considered is gathered and utilised.

Timber packs

Timber based pack support units are used as permanent support of stopes in areas where high closure or long excavation life are critical. Timber packs also form an integral part of gully support units. Several designs are available depending upon the conditions found underground. Unlike in elongates, loading in timber packs is perpendicular to the grains. This allows for larger amounts of closure before the pack fails and disintegrates. Timber packs are very popular as panel support in varying rockmass conditions because of their high strength, their ability to maintain structural integrity and increased contact areal coverage compared to elongated support units.

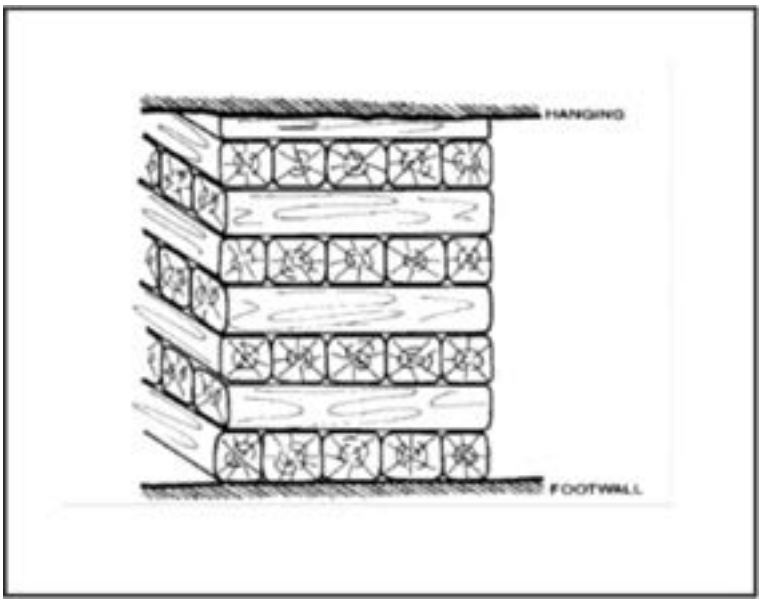


Figure 41: Typical timber pack construction





Due to the large number of variations in timber packs available on the market, the user is referred to the manufacturers for detailed information on timber pack compositions, sizes, etc.

- http://www.bedrockms.co.za/products/products-services.php

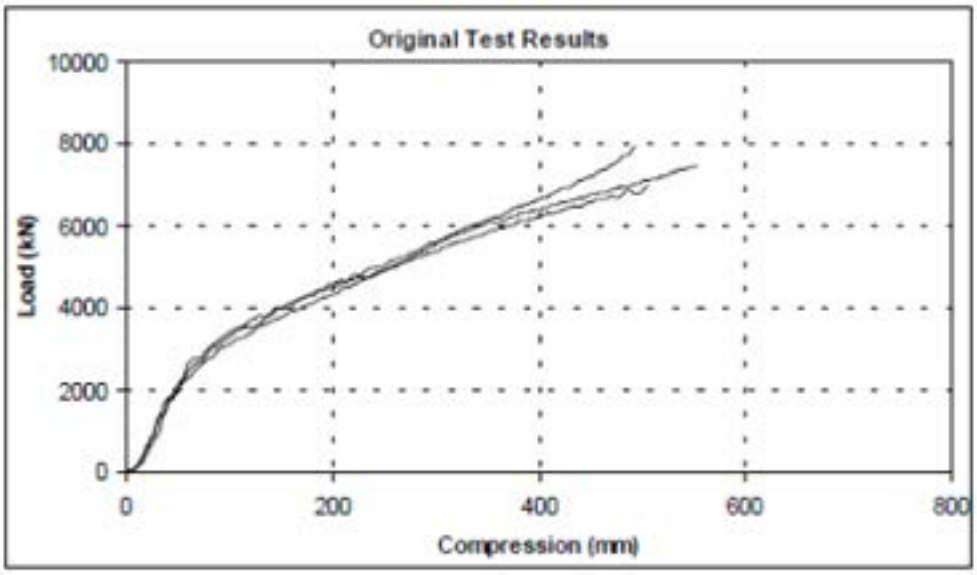


Figure 42: Load-deformation graphs for a timber pack (Gap508)

IMPORTANT

The load-deformation graphs depicted in this material are examples and the user is advised to make sure that recent load-deformation curves appropriate to the length, diameter and product type considered is gathered and utilised.

REFERENCE

- Gap508

Composite packs

Composite packs consist of alternating layers of chocks and concrete blocks OR alternating units of soft and hard timber units. These packs are often preferred above the conventional timber packs, because of:

1. The lack of compressibility in the concrete / harder timber units resulted in a stiffer unit;
2. Reduced fire hazard when concrete is used.

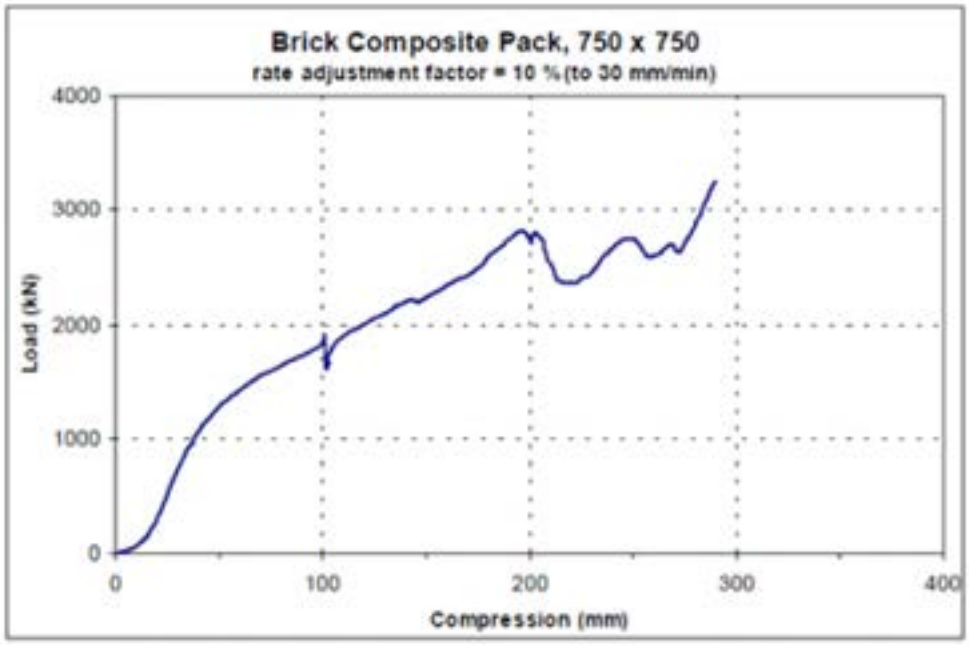


Figure 43: Brick composite pack load-deformation graph (Gap 508)

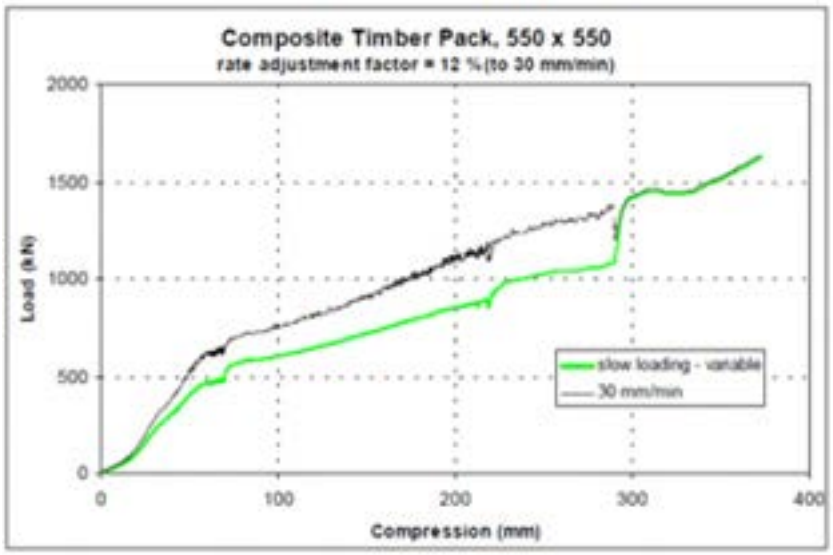


Figure 44: Timber composite pack load-deformation graph (Gap 508)

- Gap508

The load-deformation graphs depicted in this material are examples and the user is advised to make sure that recent load-deformation curves appropriate to the length, diameter and product type considered is gathered and utilised.

Cement based packs

Composite packs consist of alternating layers of chocks and concrete blocks. These packs are in many respects better than the conventional timber packs, because:

1. The lack of compressibility in the concrete resulted in a stiffer and more stable pack. .
2. Reduced fire hazard.
3. They behave in a less passive manner.
4. The packs are slightly cheaper than timber packs.

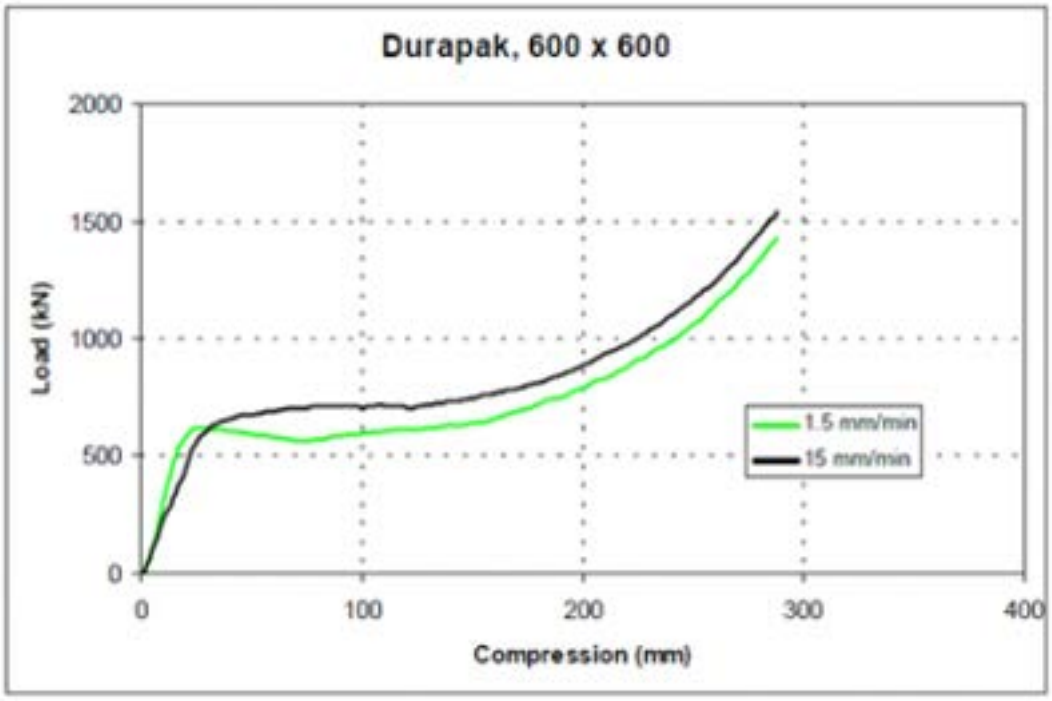


Figure 45: Cement based pack load-deformation graph (Gap 508)

INTERESTING INFO

(Courtesy of M Grave MSc)
'Grout packs were first introduced to the South African mining industry in 1975. The concept was motivated by a need to reduce labour-intensive and costly material handling in the stopes and bottlenecks in the shafts. A tailings-based support system transported in pipe ranges and pumped over long distances was, therefore, attractive to the industry. Using tailings pumped from surface was a logical extension to the cement / water batching plant technology.'

'Following tests the grout pack support system was conceived. First used at Randfontein and Elsburg Gold Mines, grout packs proved to be a very effective, stiff and strong support, particularly for high stoping width and multi-reef excavations (Heinz 2001). The first grout based packs were designed with the lower portion consisting of a 1200 mm square outer skeleton for containing the bag, and three or four layers of solid slabs for blocking it against the hangingwall. A bag capable of containing cemented grout was suspended inside the skeleton portion. Different types of bag material were tried; for example, hessian and various polypropylenes. An open-weave polypropylene became the preferred grout bag fabric because it retains almost all the solids while allowing the water to permeate out (Coetzee 1975).'

NOTE

For more information and important notes on factors affecting the performance of RSS packs see the report on PSS Packs by M Grave MSC

Mechanical face props

CONNECTION 250

See LEARNING OUTCOME 5.5.3.7 on page 388

Rapid-yielding hydraulic props

REFERENCE

Cluster sticks

- Gap 613 The impact of soft loading conditions on the performance of elongate support elements.
- Gap330 Stope face support systems
- Elbroc website: http://www.elbroc.co.za
- The design and testing of pipe-sticks for underground stope support. P. J. J. VAN VUUREN, SAIMM 1978
- Stoping in Witwatersrand gold mines during the past forty years. H G Mosenthal, J. S. Atr. Inst. Min. Metal., vol. 90, no. 9. Sep. 1990. pp. 243-255.

LEARNING OUTCOME 5.5.3.6

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PRINCIPLES OF OPERATION OF RAPID YIELDING HYDRAULIC PROPS

CONNECTION 251

See Paper 2 Outcome 5.1.1 , 5.1.2 and 5.1.3 from page 270

Pre-stressing devices are normally used with support units at the time of installation to generate a support force at installation, i.e. provide an 'active' support unit.

INTERESTING INFO

The value of pre-stressing support units were realised during the early years of mining already and initially includes a method called 'timber wedging' where timber wedges were driven into packs or above elongates using hammers. 'Wedging' was later replaced with hydraulic, mechanical or cement bag (weeping and non-weeping) pre-stressing methods due to labour intensive as well as 'effectiveness' concerns.

In essence, all these methods had it in common to ensure maximum possible load generation at installation of the support unit.

Currently used devices are usually hydraulic jacks that are placed on support units and inflated with high pressure water through a non-return valve.





Figure 46: Two common types of pre-stressing devices used (a) the "jackpot", used with elongate support and, (b) the "jack-pack", used to pre-stress timber, cementitous packs or elongate clusters.

- New Concept Mining website http// www.ncm.co.za
- Gap508

LEARNING OUTCOME 5.5.3.7

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PRINCIPLES OF OPERATION OF RAPID YIELDING HYDRAULIC PROPS

Although this support unit is referred to in the current syllabus, its use had diminished significantly over the last few years. This was mainly the result of combinations of the following:

- *Unit costs: This is a highly machined support unit and expensive to manufacture;*
- *Maintenance costs: Valve failures, blast protection covers etc, all became very expensive to maintain;*
- *Maintenance concerns (difficulty to maintain performance): Several methods to maintain these units were tried within the industry but all reported high losses in units and difficulty in ensuring optimum operational status of the unit;*
- *The need for 'support removal' to move the unit forward was being frowned upon: After several accidents during the removal of the units to move it forward, many mines converted to permanently installed support units.*

INTERESTING INFO

RYHPs absorb energy and yield at different rates. The first props used in South African gold mines could yield at a rate of 1m/s at 40 tons while the later props had a rapid yield capability of up to 3m/s with variable loads.

RYHP were designed to offer support in stopes subjected to rockburst conditions. They are active support units, i.e. they exert a positive force to the hanging wall on installation. A key feature RYHP is their energy absorption capabilities. The RYHP comprises:

- the prop with its footpiece,
- a headboard,
- a pneumatic pump that pumps fluid via high pressure hoses to a setting pistol which then pressurises the cylinder/ram steel props.
- a control valve opens at a predetermined pressure, allowing a rapid ejection of fluid at closure rates greater than 1m/s. After the rapid closure has stopped, the control valve closes whilst maintaining the original pressure.

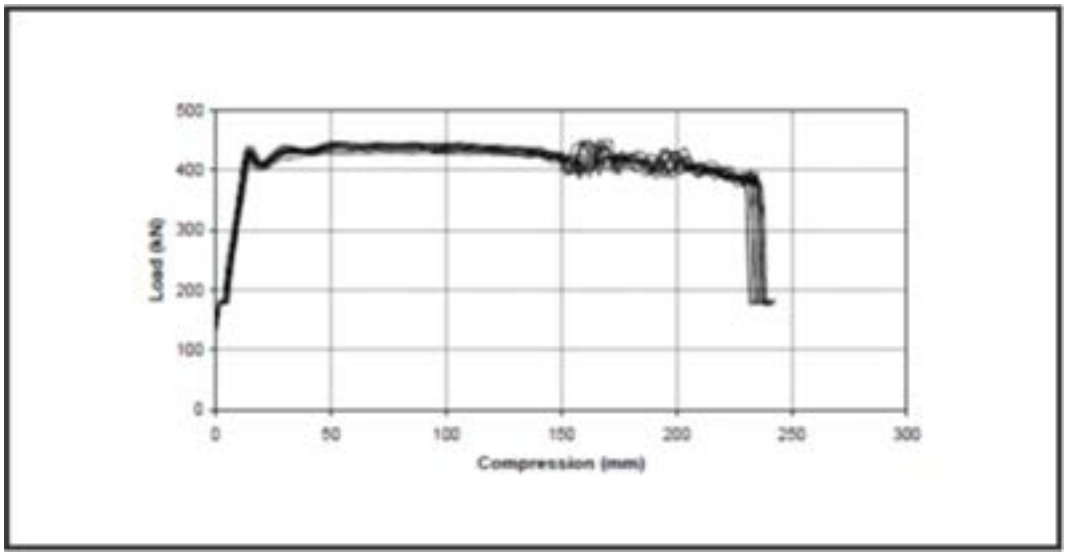


Figure 47: Typical load deformation curve of RYHPs tested at a yield rate of 3m/s (unknown source)



Figure 48: The ONMI 150 hydraulic prop in a stope (unknown source)

RYHPs are made to very specific performance requirements, with very complex components and at high engineering precision. As a result, effects such as blasting, mine water, grit and rough handling can compromise the performance of these units resulting in high maintenance costs.

- Gap330 Stope face support systems
- Page 161, 178, 267, Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999
- Page 52, Spearing AJS, Handbook on hard rock strata control, 1995
- Page 161, Budavari , Rock mechanics in mining practice, 1983

LEARNING OUTCOME 5.5.3.8

EVALUATE THE VARIOUS TYPES OF SUPPORT UNITS IN TERMS OF GIVEN MINING ENVIRONMENTS, ROCKMASS CONDITIONS AND STRESS REGIMES

Support Unit	Mining Environment	Rockmass Conditions	Stress Regime
Mine Poles	Shallow to Medium Low closure Stiff support required to control the immediate hanging-wall.	Widely spaced tensile fractures Weaknesses along geological discontinuities Low stress	Low stress fracturing- Horizontal clamping stresses absent
Profile Prop	Medium to Deep environment Moderate to high closure Rockburst hazard moderate to severe	Rock fractured around excavations Hanging-wall generally stable due to compressive horizontal stresses	Medium to high stresses High stress fracturing Compressive horizontal stresses in the immediate hanging-wall and footwal
Pipe Sticks	Designed to limit closure Preferred where a heavy type of support is required, e.g. remnants Good back area support	Weakness largely driven by geological weakness	Low stress Low stress fracturin Horizontal clamping stresses absent
Timber Packs	Ideally to be used in cases where hanging-wall is generally stable	Both low and high stress environments	Passive, unless pre-stressed Effective back area and gully support in both shallow (when pre-stressed) and deep environments
RYHP	Good face area support in high closure and rockburst prone areas	Rock highly fractured around excavations	High stress environment High stress fracturing

Table 16: Support types for given environments



- Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999 Chapters 2&3.
- Stoping in Witwatersrand gold mines during the past forty years. H G Mosenthal, J. S. Atr. Inst. Min. Metal., vol. 90, no. 9. Sep. 1990. pp. 243-255.

LEARNING OUTCOME 5.5.3.9

CONNECTION 252

Refer to LEARNING OUTCOME 5.3.1.1 on page 342 , LEARNING OUTCOME 5.1.16 on page 305 and LEARNING OUTCOME 5.1.17 on page 306

DETERMINE THE SUPPORT RESISTANCE OF SUPPORT UNITS AND SUPPORT SYSTEMS AT VARIOUS DISTANCES FROM THE FACE FOR VARIOUS SUPPORT LAYOUTS

Support resistance refers to the load generated by a support unit or a support system over a tributary area of the hanging-wall supported by that support unit or system. The load generated by a support unit a certain distance from the face is obtained from the force:deformation curve of that support unit, taking into account the rate of closure and the actual closure experienced by that unit.

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PAPER 3.1: CHAPTER 5

EXAMPLE

Determine the support resistance for the support system shown in the diagram below, given the force deformation curves for each unit.

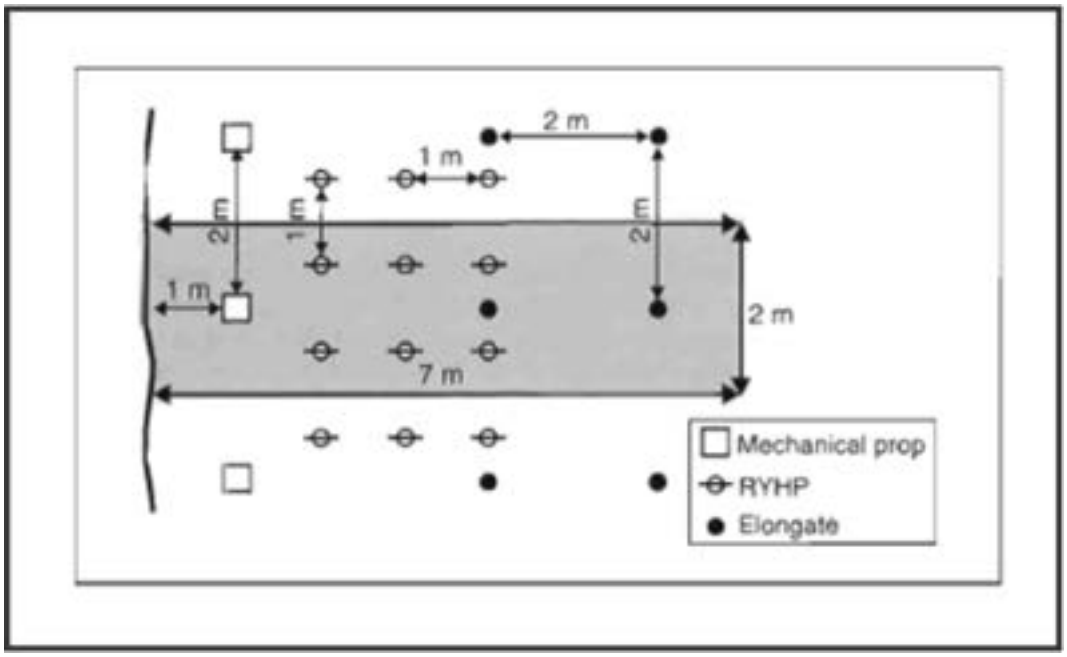


Figure 49: Tributay area of the support system (Jager and Ryder, 1999)

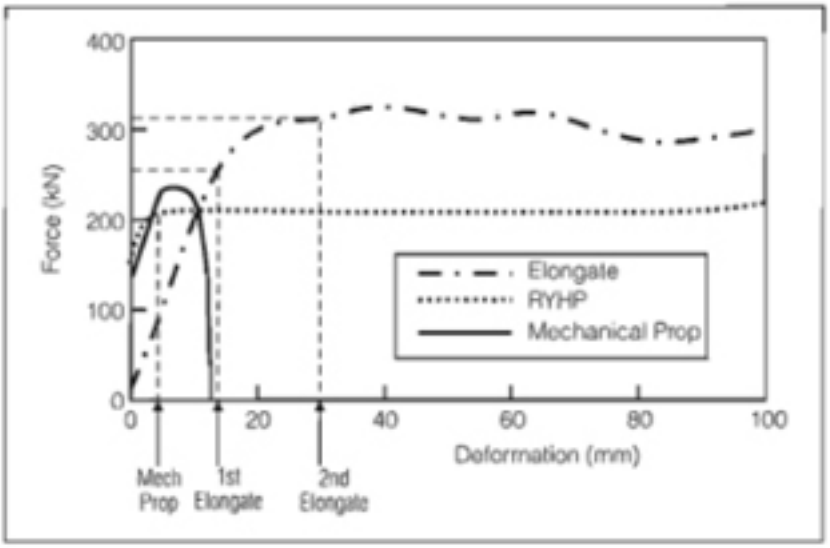


Figure 50: Force deformation curves of individual support units (Jager and Ryder, 1999)

SUPPORT RESISTANCE CALCULATION	FORCE
Mechanical Prop, installed that day and pre-stressed, 3mm closure	200kN
6 x RYHP at a slow yield force of 200kN	1200kN
First row elongate- installed previous day, 13mm closure	250kN
Second row elongate- installed for 3 days, 30mm closure	310kN
TOTAL	1960kN

Tributary area (shaded)2m x 7m=14m2

Support resistance= 1960/14 = 140kN/m2

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PAPER 3.1: CHAPTER 5



- Page 207, Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999
- Page 358, Ryder JA and Jager AJ, A textbook on rock mechanics for tabular hard rock mines, 2002

LEARNING OUTCOME 5.5.3.10

CONNECTION 253

Refer to LEARNING OUTCOME 5.3.1.1 on page 342 , LEARNING OUTCOME 5.1.16 on page 305 and LEARNING OUTCOME 5.1.17 on page 306

DETERMINE APPROPRIATE SUPPORT DESIGNS FOR GIVEN MINING ENVIRONMENTS, ROCKMASS CONDITIONS AND STRESS REGIMES.

- Page 244, Hoek E and Brown ET, Underground excavations in rock, 1980
- Page 216, Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999
- Page 358, Ryder JA and Jager AJ, A textbook on rock mechanics for tabular hard rock mines, 2002
- Page 312, Brady BHG and Brown ET, Rock mechanics for underground

5.5.4. SUPPORT AND REINFORCEMENT OF TUNNELS

LEARNING OUTCOME 5.5.4.1

APPLY THE VARIOUS ROCKMASS CLASSIFICATION SYSTEMS TO SELECT APPROPRIATE SUPPORT FOR TUNNELS IN SHALLOW, INTERMEDIATE AND DEEP MINES

Table 17 below gives some areas of application of rock mass classification systems to determine support for mining excavations.

Rock mass rating system	Area of application in mining industry
RMR	Underground and surface applications. Underground applications in tunneling and stoping environments. Surface application in strip mining and open pit mining including quarrying.
MRMR	Underground and surface applications. Underground applications in tunneling and stoping environments. Surface application in strip mining and open pit mining including quarrying. Particular interest is in weathering prone environments and blasting affected mining environments where the stability of the walls of the excavation is important to the operation and hence adjustment of the RMR to cater for weathering and blasting damage
Q-system	Tunneling in underground workings and used sparingly in conventional stoping operations. Applied in room and pillar operations where the mining environment can be likened to tunneling in civil Engineering operations.
RCF	Deep level gold mining where the principal stresses (sigma 1 & sigma 2) are obtained from numerical modeling results and doorstopper in-situ stress measurements.
RQD	Applies to open pit operations and exploration stage drilling where the drill core is readily available. The results of exercise RQD are used in numerical modeling to give an indication of the expected integrity of the rock mass in question.

Table 17: Application of rock mass classification systems to the selection of excavation support

Rock mass classification systems help determine how blocky the ground to be supported is and consequently the size of potential rock blocks expected to fall out of the tunnel sidewalls.

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CONNECTION 254

See Paper 1 Outcome 6.2.14 and Paper 2 Outcome 3.1.2 Also refer to LEARNING OUTCOME 2.3.1.1 on page 50, LEARNING OUTCOME 3.2.16 on page 108, LEARNING OUTCOME 3.2.17 on page 108, LEARNING OUTCOME 3.2.18 on page 108 , LEARNING OUTCOME 5.1.1 on page 287

This will in turn determine the type of support that needs to be installed in the tunnel to cater for the dead weight of loose dislodged bolts. In most rock mass classification systems the stress conditions are included which gives a direct indication of the stress conditions in which installed tunnel support will be exposed to and hence the support capabilities expected from the support type chosen.

At shallow depth geotechnical structures in the rock mass determine the stability of a rock mass and the size of the potential unstable blocks.

In deep level tunnels fracturing plays a huge role in block formation and these considerations through rock mass classification methods can help in determining the type of tunnel support required.

LEARNING OUTCOME 5.5.4.2

CONNECTION 255

Refer to LEARNING OUTCOME 2.3.1.1 on page 50, LEARNING OUTCOME 3.2.16 on page 108, LEARNING OUTCOME 3.2.17 on page 108, LEARNING OUTCOME 3.2.18 on page 108, LEARNING OUTCOME 5.1.1 on page 287

APPLY THE ROCKWALL CONDITION FACTOR (RCF) TO SELECT APPROPRIATE SUPPORT FOR TUNNELS AT DEPTH

LEARNING OUTCOME 5.5.4.3

CONNECTION 256

See Paper 1 Outcome 6.2.14 and Paper 2 Outcome 3.1.2 Also refer to LEARNING OUTCOME 2.3.1.1 on page 50, LEARNING OUTCOME 3.2.16 on page 108, LEARNING OUTCOME 3.2.17 on page 108, LEARNING OUTCOME 3.2.18 on page 108, LEARNING OUTCOME 5.1.1 on page 287

APPLY OTHER APPROPRIATE CRITERIA TO SELECT APPROPRIATE SUPPORT FOR TUNNELS AT DEPTH

LEARNING OUTCOME 5.5.4.4

CONNECTION 257

See Paper 2 Outcome 5.1.7 on page 279

DESCRIBE AND DISCUSS THE FOLLOWING ROCKWALL SUPPORT TYPES :

- Mechanically anchored bolts,
- Cable bolts, Friction 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,

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INTERESTING INFO

Future in-stope support options

- Shotcrete,
- Gunite,
- Thin sprayed linings,
- Wire mesh,
- Rope lacing,
- Tendon straps

Mechanically anchored bolts

The mechanically end anchored bolts and cable use a three leaf end anchor rage system at the toe of the hole, Figure 51.

NOTE

Reference is made here to the Duraset website for further description of the product specification. Videx mechanical roof bolts also fall into this category of bolts with similar anchorage as described herein.



Figure 51: Three leaf expansion shell end anchorage bolts. (After Duraset product specifications)

The expansion unit, which is available as a 'bail type' unit or a 'regular type' (loose leaf) unit, is a 3-leafed mechanical anchor which conforms to the hole diameter. This feature ensures a more even load distribution between the anchor and the rock at the anchor point. The bail type unit incorporates a pop-out top. This feature will not allow the bolt to slip past the bail as often happens with a narrow strap bail. The pop-out holds until initial anchorage is achieved, then pops away to let the bolt pass freely and secure the bearing plate firmly against the rock face. The serrations on the shell are rounded off. This is done to reduce point loading at the contact point between the shell and the surface of the borehole. This reduces crushing of the rock and breaching of the hole, thus taking maximum advantage of the strength of the undisturbed rock at the anchor point.

Some popular types of mechanical anchor bolt 3-leaf expansion units are shown in Figure 52.



Figure 52: Some popular 3-leaf expansion units

Typical drill hole dimensions and expansion diameters are given in Table 19 below.

Description	Min. hole size	Max. hole size
25mm	25mm	29mm
28mm	28mm	32mm
30mm	31mm	35mm
32mm	32mm	37mm
35 TT	35mm	38mm
35 RB	36mm	42mm
38mm	38.5mm	45mm
40mm	40mm	47mm
43mm	43mm	50mm
60mm	60mm	68mm

Table 19: Hole sizes required to accommodate various expansion unit sizes

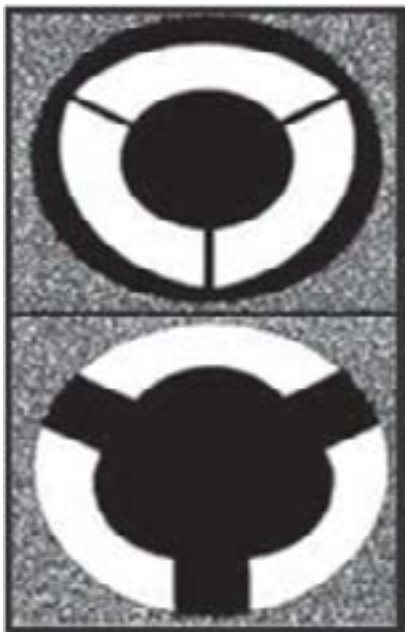


Figure 53: shows the features of the expansion shell units in a drill hole cross section.

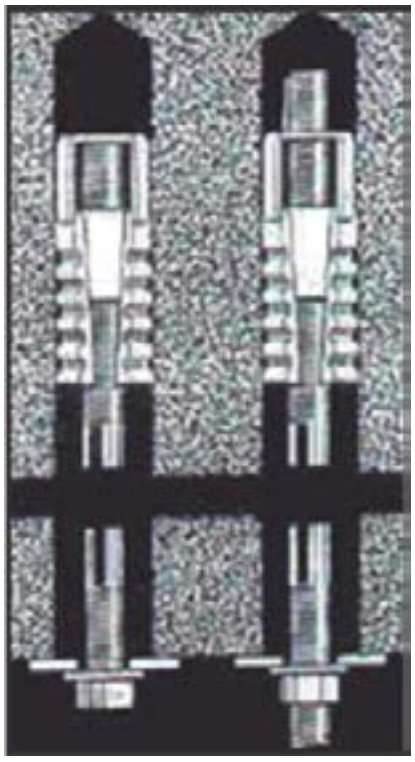


Figure 54: Features of the expansion shell units

Material specifications for the different components of the expansion shell units are given by Duraset as stated below:

Material

1. Leaves: to SAE 1008 cold forged
2. Expander plugs: SABS 936/937 – 1969 SG iron castings grade 42 or 50
3. Bail: commercial-quality mild steel.

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1. *Expansion units are normally supplied with ISO metric threads or DIN 405 threads. Units for UNC or BSW threaded bolts can be supplied on special order*
2. *Left-hand expansion units are available for left-hand threaded rock studs*
3. *Expansion units can be supplied as regular or bail type. (Duraset)*

Cable bolts

In many narrow reef applications there is need for a reliable flexible support system to be used in the stopes and gullies. This problem has been solved by the use of cable bolts or cable anchors. The cable bolt has an ultimate tensile strength of 25 tons. The anchor can be made to any length according to your specific requirements.

The anchor consists of a threaded steel bar swaged onto the bottom of a strand cable (15.2mm). The locking device consists of a distribution plate (120 x 6mm) and nut which can be tensioned by impact wrench or by hand, Figure 55.

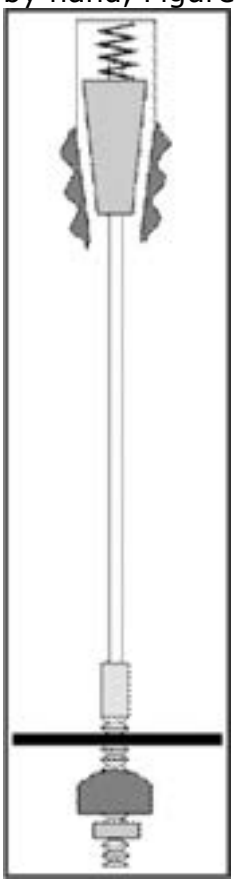




Figure 55: Face plate, nut and cable bolt

A mechanical 3 part shell provides the fixed end support inside a standard 36mm to 41mm hole, Figure 52 and Figure 53. A schematic illustration of the end anchorage set up for a cable bolt is shown on the left. Once tensioned the cable bolt may be full column grouted to prevent rusting and corrosion from contact water and moisture.

The cable bolt has the following advantage:

- being flexible, the anchor can be installed vertically regardless of height available thus ensuring all of the anchor length is active.
- The cable bolt provides instantaneous pre-stressed stope support.

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Further examples of cable bolts are illustrated here in the form of mechanical bolts. Duraset supplies 'Surelock' mechanical anchors which are flexible high-tensile steel strands of specific lengths, anchored inside a drilled hole with an innovative locking-off unit called the 'Surelock swivel barrel' at the collar. The advantage of using this locking-off device is that once the required prestressed load is achieved, the shear ring shears on the barrel into a positive visual indicator. Surelock mechanical 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 56: Cable anchors

The advantages of using cable anchors include the following:

- Easy installation using trained support installation crews.
- The cable anchors can be manufactured to the required specifications of the customer e.g. the cut length of the strand.
- All mechanical anchors are tensioned to the required loads as specified by the customer, with hydraulic equipment approved by the supplier.
- The mechanical anchors require reasonably competent rock in order for the expansion shell to imbed itself inside the drilled hole.
- All mechanical anchors may be grouted either before or after pre-stressing the anchors introducing operational or installation flexibility.

The following specifications provided by Duraset for their product can be used to provide guidelines for the choice of bolt strength as per application requirements of factor of safety and support resistance, Table 19.

Strand diameter (mm)	Ultimate tensile strength (tons)	Working load (tons)	Hole diameter (mm)
12.5	18.0	10.0	38.0 to 42.0
15.24	25.0	15.0	38.0 to 42.0
18.0	35.0	20.0	48.0 to 51.0
22.0	50.0	20.0	48.0 to 51.0

Table 20: Cable anchor specifications

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Friction bolts

Friction rock anchors are simply a long length of open seam, high strength steel tube. One end is equipped with a welded steel ring which retains a bearing plate. The opposite end is tapered to facilitate easy insertion into the drilled hole in the rock Figure 56.

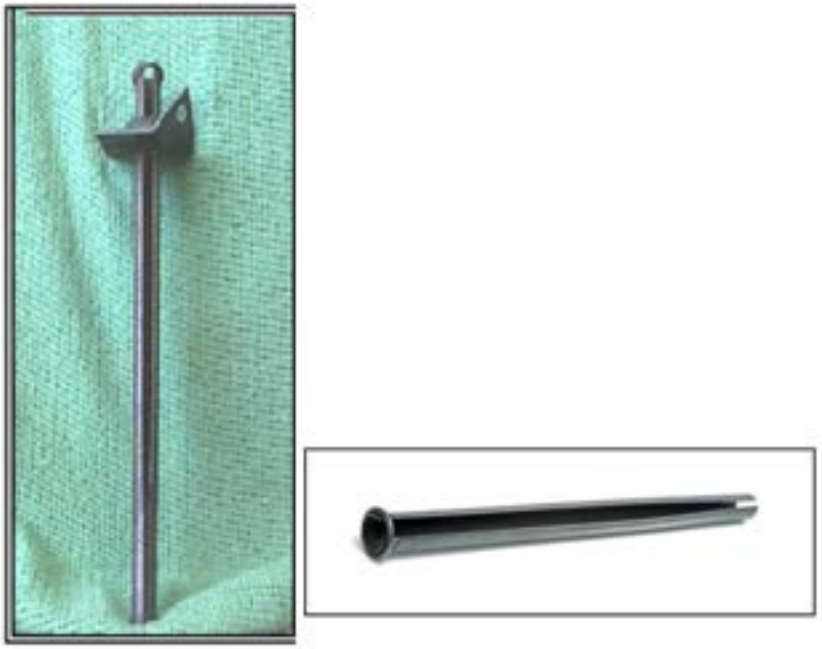


Figure 57: Friction bolt also known as the 'split set' bolt.

This unit is also known as a split set and is used as temporary support usually with area cover support like wire mesh in quick advance development ends awaiting the installation of permanent support. The friction rock anchor is driven into a drilled hole slightly longer and slightly smaller in diameter than the unit. The slot along the length of the tube is compressed as it is driven into the drill hole. The outward radial force generated by the compressed tube on the surrounding rock, anchors it in the hole. Other applications include open pit slope support and as anchorage for strainaing wire used to hang pipes and cables. Duraset suggested hole diameters for use with friction bolts given in Table 21. The hole diameter should be 1 to 4mm smaller in diameter than the Friction bolt to be installed, this is important for good performance. The friction bolt is very useful close to the stoping face as it withstands blast damage owing to its full column anchorage along its whole length.

FRA diameter	33mm	39mm	46mm
Reccommended hole diameter	30 -32mm	35 -38mm	41 - 45mm

Table 21: Suggested hole diameters for friction bolt installation

Some examples of friction bolt length and hole diameters are given in Table 22.

Length (mm)	Width (mm)	Thickness (mm)	Hole size
125	125	5 or 6	30, 40 or 47
150	150	5 or 6	30, 40 or 47

Table 22: Friction bolt specifications

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The disadvantages of the use of friction bolts come when used in wet conditions where the bolt is prone to corrosion and rusting and loss of friction anchorage. Also weak friable ground will lead to slipping of the bolt in the the hole. Although not so common some friction bolt collar have been known to collapse or break when loaded leading to compromised strength of bolt.

The other type of friction bolts is swellex bolts, Figure 58.



Figure 58: Swellex bolts

These bolts are applied in mining, tunnelling and civil engineering work. The swellex bolt incorporate the following physical attributes for its function as reinforcement:

- Physical characteristics.
  - Ground movement result in immediate bolt load bearing response.
  - Deforms to accommodate ground movement while maintaining full load carrying capacity.
- Functional characteristics
  - Instant full load bearing capacity
  - Every single bolt displays identical behaviour when subjected to rocl load.
  - Insensitive to blasting vibrations
- Environmental characteristics
  - Introduces no harmful chemicals to the environment.
- Installation characteristics
  - Safe and easy to install
  - High tolerance to variations in borehole diameter
  - Quality assured with each installation
- Economical characteristics
  - Application adaptable
  - Fast installation
  - Unique, quick and low cost non-destructive in-situ strength testing.

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Some product specifications for the swellex bolts are given in Table 23.

Specifications	Metric
Minimum breaking load*	100 kN
Minimum elongation A_5 *	15 %
Standard bolt lengths	1.2 - 3.6 m
Bolt lengths available, in 0.3 m (1 ft) intervals	0.6 - 12.0 m
Recommended hole diameters	32 - 39 mm
Optimal hole diameters	35 - 38 mm
Original tube dimension	41.0 x 2.0 mm
Bushing head dimension (waist/collar)	30/36 mm
Upper bushing diameter	28 mm
Inflation water pressure	30MPa

*Relates to profile only according to quality standards: AC-STD 1295 K, 4559 K and 4560 K.
1) When pump reaches pre-set pressure (30 MPa) the quality of bolt and installation is verified
2) Re-pressurization of the bolt in situ, confirms support capability.

Table 23: Some product specifications for swellex bolts.

Cement grouted bolts

There are quite a number of rock support bolts that use cement grouting for anchorage. These include shepherd crooks, full column grouted cable anchors and z-bars. These are often used in soft rock material, bad ground conditions and wet ground conditions where bolt support is prone to corrosion. More detail on these bolts is given in the section on full column grouted bolts.

Resin bonded bolts

There are quite a number of rock support bolts that use cement grouting for anchorage. These include videx bolts (Figure 59), shepherd crooks, These are often used in soft rock material, bad ground conditions and wet ground conditions where bolt support is prone to corrosion.

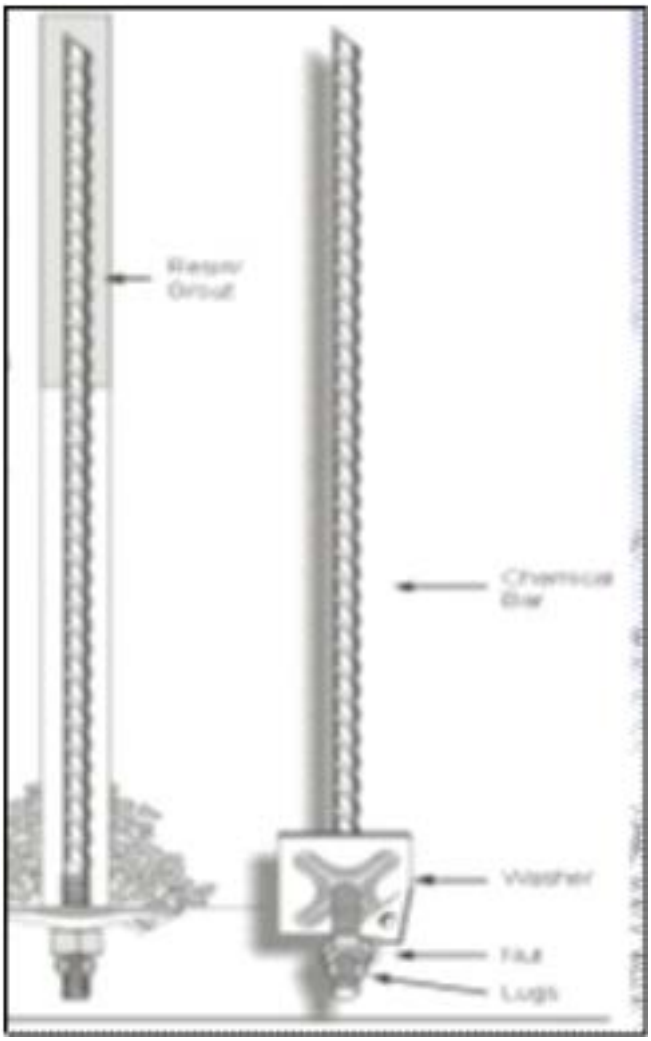


Figure 59: Videx resin bolt

The Videx bolt uses the mechanical spinning action of the bolt's ribbed structure during installation to mix resin and ensure full column contact of the bolt length. Some advantages of using resin bolts include:

- Mixing of resin during installation using the unique rib pattern on the bolt.
- Provides high strength tendons
- Good adhesion properties in rocks.

Some product options and product specifications follow, (Table 24 and Table 25):

Features	Options
Typical Length of Bolt	Increments of 300mm From 600mm to 3000mm Lengths
Size & Type of Thread	M20 R/H, M24 R/H, DIN 18 R/H, DIN 22 R/H
Bearing Plates (Washers)	Rib Dome Washer or Rib Dome Dog Ear Washer
Load Indicators	20mm (Optional)
Spherical Seat	Standard (On Hard Rock Mines)
Tightening Mechanisms	Nut Fitted, Reverse Spin, Pop-Out Nut or Forged Head Nut

Table 24: Some product options for Videx bolts (Videx brochures)

Thread Size	Nominal Bar Dia. (mm)	Profile of Bar	Yield Load (kN)	Ultimate Breaking Load (kN)
M20 x 2.5	17.5	VCB	130	160
M24 x 3.0	20.5	VCB	185	240
DIN187 x 3,185	16	Y16	100,5	140
M20 x 2,5	20	Y20	157	219
DIN22 x 3,185	20	Y20	157	219

Table 25: Product specifications for the videx bolt

Full-column grouted/bonded bolts

The Videx condom bolt is an example of a full column grouted bolt in use in industry. Some pictures of its application are given below (Figure 60, Figure 61 and Figure 62).

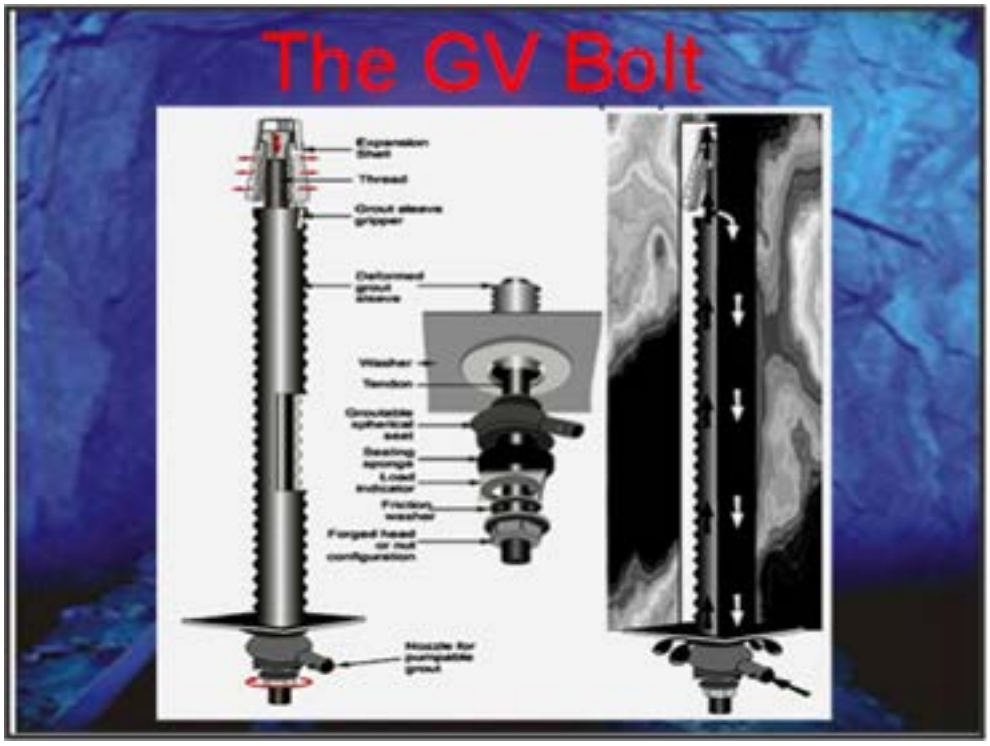


Figure 60: The full column grouted videx bolt



Figure 61: Illustration of the pumping of a videx bolt using a conduit pipe to represent the drill hole (After Videx product promotions)



Figure 62: Resin pump nozzle (after Videx product promotion slides)

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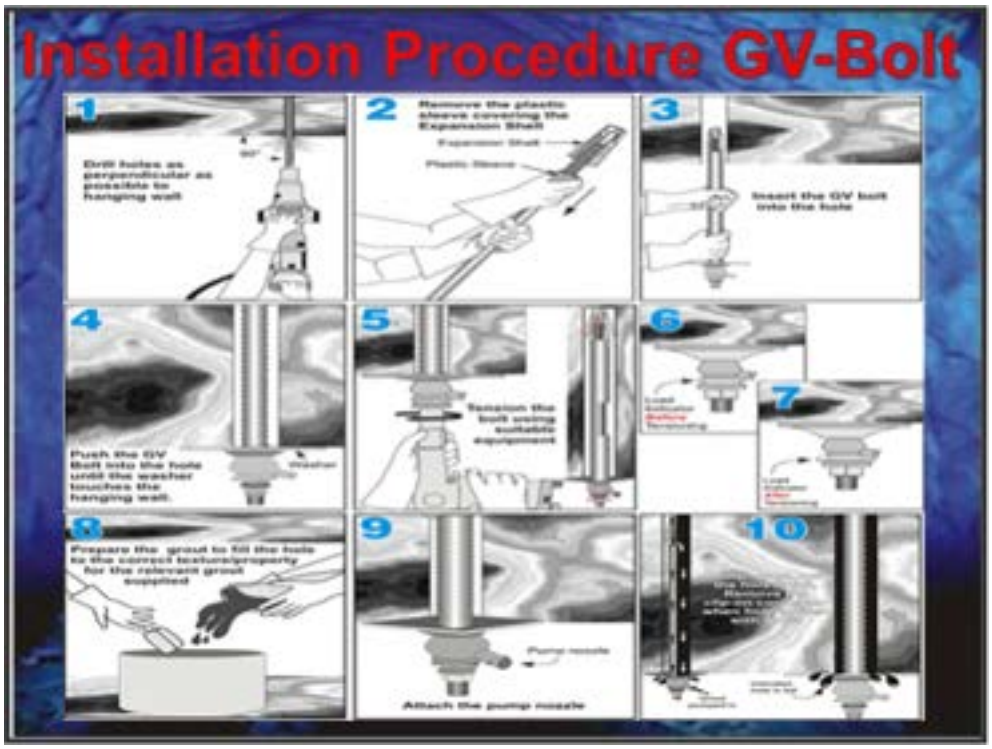


Figure 63: Installation procedure for full column grouted bolts (after Videx product promotion pamphlet)



Figure 64: Videx bolts used with area cover support in the form of Osro straps (Videx product promotion pamphlet)

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Yielding bolts (cone bolts)

Yielding bolts are designed to cater for rock bursts and dynamic loading experienced in the mines especially hard rock deep mines. The bolt is designed to absorb the energy generated by the rock burst and minimise the damage caused to the excavation walls. Ortlepp (2008) designed a yielding bolt meant to contain rock mass material, provide absorption and controlled deformation to a rock mass under dynamic loading, Figure 65.

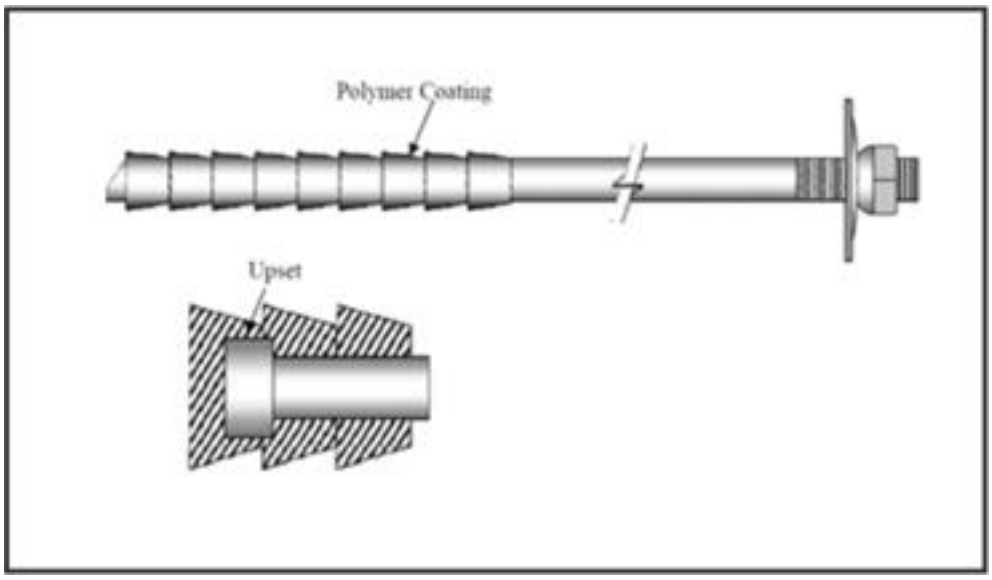


Figure 65: Cross section through the yielding bolt designed by Ortlepp

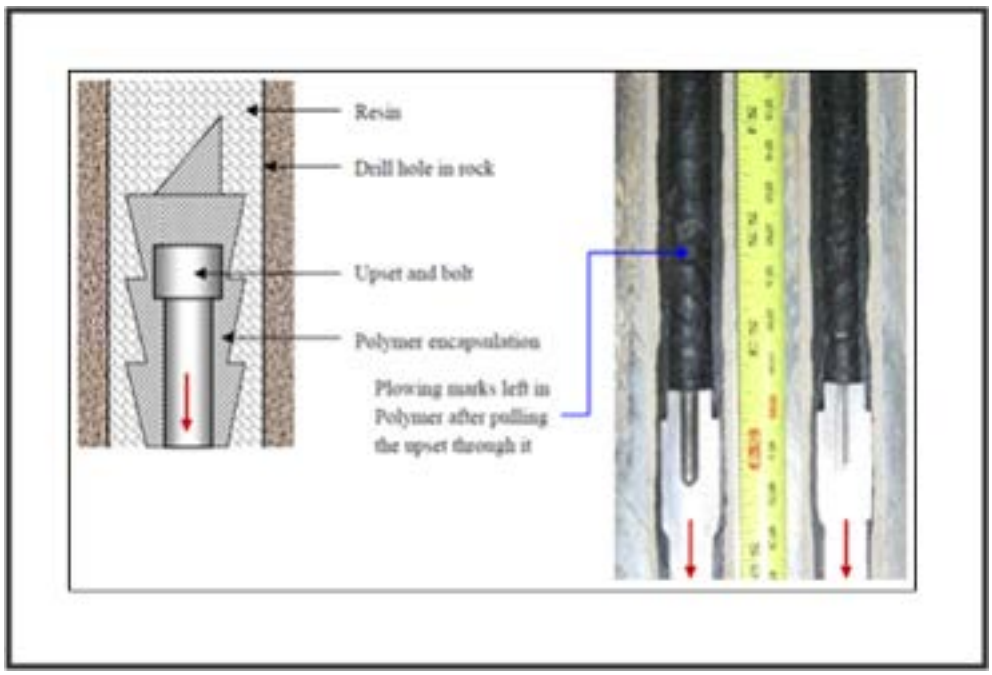


Figure 66: illustration of the interaction of resin and yielding bolt in a hole

The cone bolt is a yielding tendon capable of providing effective support in areas prone to seismic events and high stress changes. Damage to tunnels normally resulting from, or associated with, rockbursts and large quasi-static rock deformations are consequently reduced.

Construction of the yielding "cone" bolt
Typical cone bolt studs are shown in Figure 67 below illustrating the yielding conical head and its construction details are given below.



Figure 67: Cone bolt studs (Aveng manufacturing pamphlets)
Various versions of the cone bolt exist like the modified cone bolt shown below:

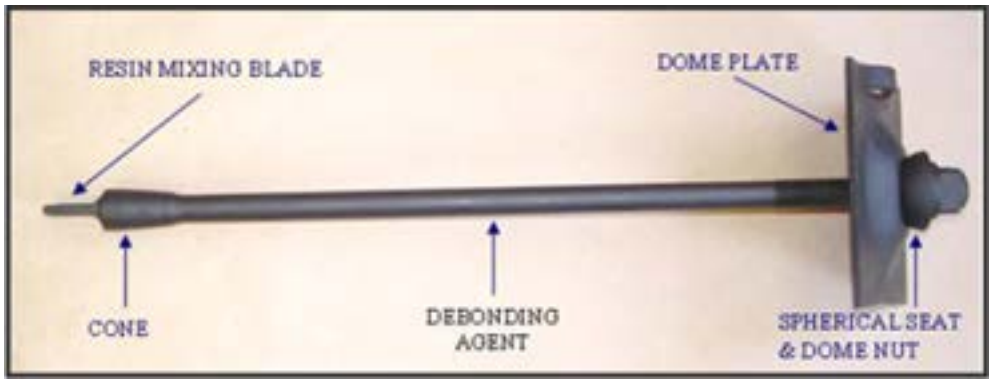


Figure 68: The cone bolt from Mansour Mining Technology (After Mansour Mining technology pamphlet)



- http://www.mansourgroupinc.com/MansourMining/gsMCBconebolt.html

Description of the mechanism of the cone bolt form Mansour Mining Technology follows:

- The end of the tendon remote from the collar of the hole, has a conical enlargement forged onto it. The opposite end is either threaded like a conventional end anchored tendon, or may have a 'Shepherd's crook' eye to facilitate cable lacing. The entire length of the tendon is coated with a debonding agent. The short leg of the 'Shepherd's crook' is left uncoated to prevent unravelling of the eye under load.
- Both types of tendons are used in conjunction with a suitable bearing plate and in the case of the threaded tendon, a nut and spherical seat is also used.

Modified cone bolt
The Modified Cone Bolt is a smooth bar threaded at its outer end, with a forged cone and mixing blade at the other end. The specially formulated Dupont Coneloc Resin is effectively mixed by this patented cone bolt design. The smooth bar has a special coating to reduce the friction and bonding effect of the resin. Under Dynamic loading conditions, the MCB will yield or plough through the Dupont Coneloc resin, thus absorbing the energy through controlled deformation.

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Mechanism of operation

A description of the mechanism of operation of the modified cone bolt follows:

- The forces generated by the dilating rock are transferred to the tendon via the bearing plate. As this force reaches the yielding point of the tendon, the cone is drawn through the surrounding grout. In so doing, work is done and energy absorbed from the surrounding rock.
- Once all of the energy has been absorbed, a state of equilibrium is achieved and the tendon stops yielding.
- Any subsequent build up of force in the tendon, will result in the yielding process being reinitiated.

Types and sizes available

Typical cone bolt types and sizes available in industry are given in Table 25 below. Depending on the application's yielding or energy containment requirements or drill diameter availability the operation may order from the bolt types and diameters given below.

Type	Shank dia. (mm)	Thread	Length (m)	Ave. yield load (kN)
Stud	16	M18	1.5 - 2.4	100
Stud	22	M24	1.8 - 3.0	200
Shepherd's Crook	16	-	1.5 - 3.0	100

Table 25: Cone bolt types and sizes available to industry

Two types of cone bolts exist:

- Cone bolt studs are used in conjunction with 125 x 125 x 6 x 40 domed washers and spherical seats.
- Cone bolt 'Shepherd's crooks' are used in conjunction with 125 x 125 x 6 domed slotted washers.

Performance

Typical performance parameters and load deformation graphs (Figure 69) for the cone bolt are given below.

- Yields for 0.5m or more at displacement rates ranging from very low to > 7m per second whilst still maintaining support loads of up to 200kN
- Readily integrates with other elements of the support system thus requiring minimum training of personnel
- As easy to install as conventional end anchored or grouted tendons
- Easily meeting the criteria or being able to absorb 25kJ of energy, the 16mm cone bolt will absorb 40kJ of energy over a yield length of 500mm and the 22mm unit, up to 100kJ without failing.

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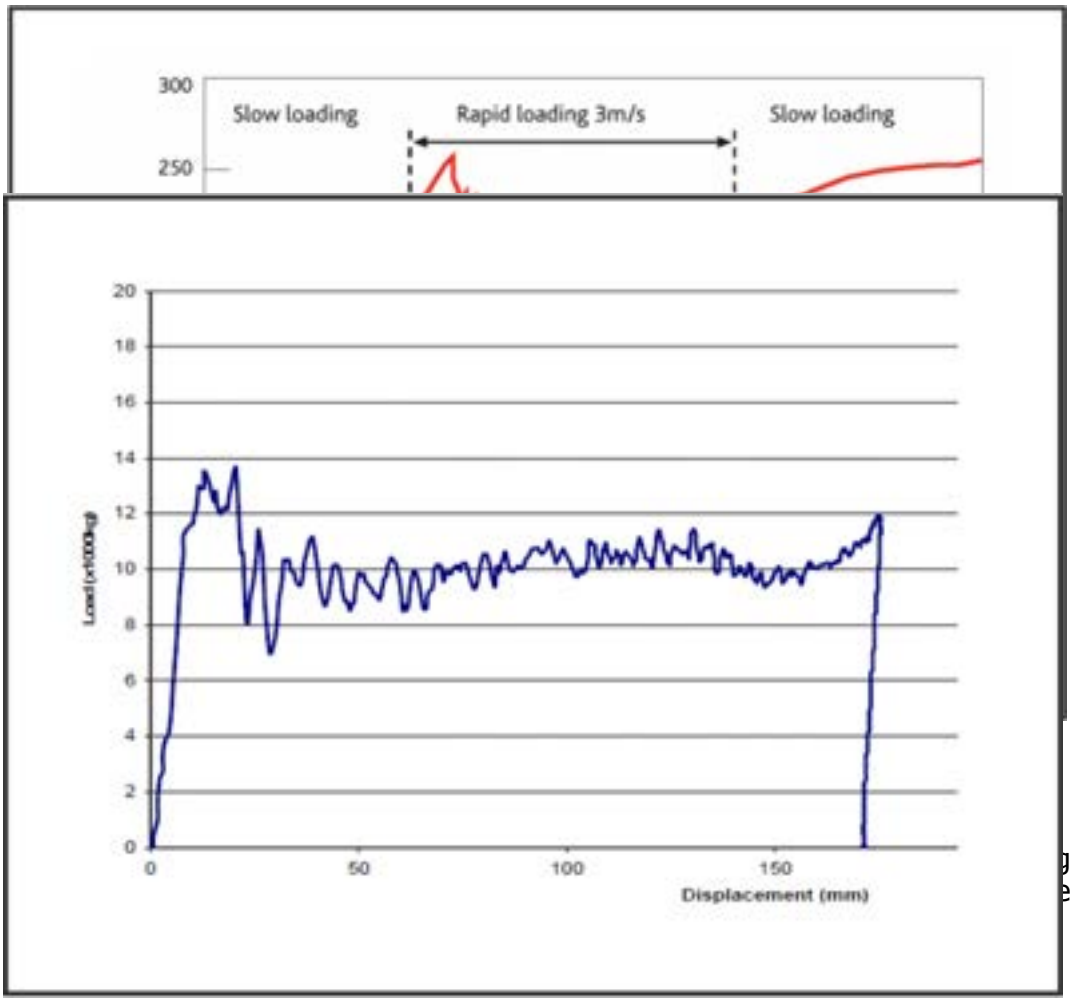


Figure 70: Load deformation plot of a drop test on the cone bolt done by Wu et al (2008)

Notice the extended deformation endured with almost constant load carrying capacity maintained with eventual failure after protracted deformation.

An example of the application of cone bolts is given here from an Australian mine (Mckenzie 2002). The cone bolt was used in rock burst conditions with good rock mass containment results. The mechanism of the functionality of the cone bolt can be described as follows:

The forces generated in a rockburst are transferred to the bolt via the bearing plate. Depending on the grout strength and quality of installation, the cone is drawn through the surrounding grout. The resistance to displacement is a combination of the friction between the grout and the cone and the force required to compress or crush the grout. In so doing, work is done and energy absorbed from the surrounding rock. Once all of the energy has been absorbed, a state of equilibrium is achieved and the bolt stops yielding and contains the surrounding rock. Any subsequent build up of force in the bolt will result in the yielding process being re-initiated. The broken rock between the bolts is supported by mesh and/or lacing. (Mckenzie 2002).

- Mckenzie R., (2002). Use of cone bolts in ground prone to rockbursts. In Aziz, N (ed), Coal 2002: Coal Operators' Conference, University of Wollongong & the Australasian Institute of Mining and Metallurgy, 2002, 169-175. (http://ro.uow.edu.au/cgi/viewcontent.cgi?article=1206&context=coal).
- Wu R. Y., Oldsen J. And Gagnon R., (2008). Development of the Yield-Lok yielding Rock Bolts. (http://www.miningcongress.com/pdf/presentations-downloads/Jennmar-John-Oldsen-1.pdf).

- http://www.avengman.com/duraset/products/cone-bolts-yielding-roof-bolt
- http://www.mansourgroupinc.com/MansourMining/gsMCB-conebolt.html

Multiple rod bolts
Coupled rods may be used to extend to the full length of long holes that require bolt support.

Multiple strand cable anchors
The 35 ton multi-bulb passive anchor is a flexible, high tensile steel strand of a specific length, anchored inside a drilled hole, with an innovative multi-coloured breathing tube system. The advantages of using this breathing tube system in a vertical crater retreat mining operation is, that once a round has been blasted, the remaining coloured breather tube indicates the actual depth of the blast. Multi-bulb passive anchors are made to requirements as specified by the end user. These anchors were designed as a support for vertical crater retreat mining operations and are normally grouted, but not pre-stressed, thus
aptly named as passive anchors. An example of a full column grouted multiple bulb anchor is shown in Figure 71.



Figure 71: Multiple strand/bulb cable anchor

CONNECTION 258

See section on Cable bolts on page 397 and mechanically anchored bolts.

Pre-stressed tendons

Steel arches
Steel arches are also commonly called RSJ s ets in industry. They can be made of rail steel, I-beams and shaped circular (for circular excavations, Figure 72) or rectangular with void fill to cover the eminent voids between the walls of the excavation and the steel arches.



Figure 72: Steel arches installation at Mponeng Mine (after Mponeng Mine support catalogue – unpublished)

Depending on the stress conditions in the particular haulage steel sets can provideactive and passive support to squeezing ground and highly damaged rock masses. Steel sets can be used in conjunction with areas cover; wire mesh, TSL, gunite/ shotcrete, linking straps and massive concrete linings to contain the rock mass between the steel arches.

Massive concrete linings

Massive conctrete linings are usually used in long term civil engineering structures due to the expensive process involved and the high cost of material used. Tunnel linings are usually designed to fulfil the following: functional requirements, geology and hydrology, constructability, economy. There are several types of concrete linings used for specific quality or supportrequirements:

a. Unreinforced concrete lining

An unreinforced concrete lining is used to protect the rock from exposure and to provide a smooth hydraulic surface. Most shafts that are not subject to internal pressure are lined with unreinforced concrete. This type of lining is acceptable if the rock is in equilibrium prior to the concrete placement, and loads on the lining are expected to be uniform and radial.

b. Shotcrete lining

A shotcrete lining will provide ground supporl and may improve leakage and hydraulic characteristics of the tunnel. It also protects the rock against erosion and deleterious action of the water. To protect water-sensitive ground, the shotcrete should be continuous and crack-free and reinforced with wire mesh or fibers.

c. Reinforced concrete linings

The reinforcement layer in linings with a single layer should be placed close to the inside face of the lining to resist temperature stresses and shrinkage. This lining will remain basically undamaged for distortions up to 0.5 percent, measured as ‘diameter change / diameter’, and can remain functional for greater distortions. Multiple layers of reinforcement may be required due to large internal pressures or in a squeezing or swelling ground to resist potential non-uniform ground displacements with a minimum of distortion. It is also used where other circumstances

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would produce non-uniform loads, in rocks with cavities. For example, non-uniform loads also occur due to construction loads and other loads on the ground surface adjacent to shafts; hence, the upper part of a shaft lining would often require two reinforcement layers. Various types of reinforcement steel may be used in tunnel reinforcement.

The following pictures give examples of concrete linings used in Germany civil works and tunnels.



Figure 73: Vault formwork car for massive tunnel concrete lining in German



Figure 74: Reinforcement steel work for tunnel lining (Germany)

back



Figure 75: Shotcrete preparatory coat



Figure 76: Tunnel lining using concrete blocks



- DAUB, (2000). Concrete linings for tunnel built by underground construction.
- http://www.daub-ita.de/fileadmin/daub/gtcrec08e.pdf

Shotcrete, Gunite, Thin sprayed linings
Shotcrete and gunite are names that can be or are sometimes used interchangeably in industry. A more flexible area cover option is given by

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"Thin Sprayed Liners" TSLs or "superskin". A "Superskin" can be considered as a thin lining (3mm to 5mm) that can be sprayed onto a surface in a liquid form and that within a short (preferable) period of time sets to provide a high degree of tensile strength and flexibility. Thin sprayed liners are usually applied to 4 – 6 mm average thickness much thinner compared to usual shotcrete thicknesses of 50mm. Thin sprayed liners (TSLs) have been gaining popularity in the mining industry due to their lightness (less bulky than shotcrete products) and quick and easy application procedures. A typical application of TSL is shown in Figure 76.



Figure 77: Application of thin sprayed liner on excavation walls. After Falconbridge Mine documents.

Tests results from CSIR have shown TSL strength to improve with increased curing time. The strength of a TSL application is also largely dependent on the uniformity of the application thickness achieved. A typical load deformation graph is shown in Figure 78 from tensile strength tests conducted by CSIR on TSL products.

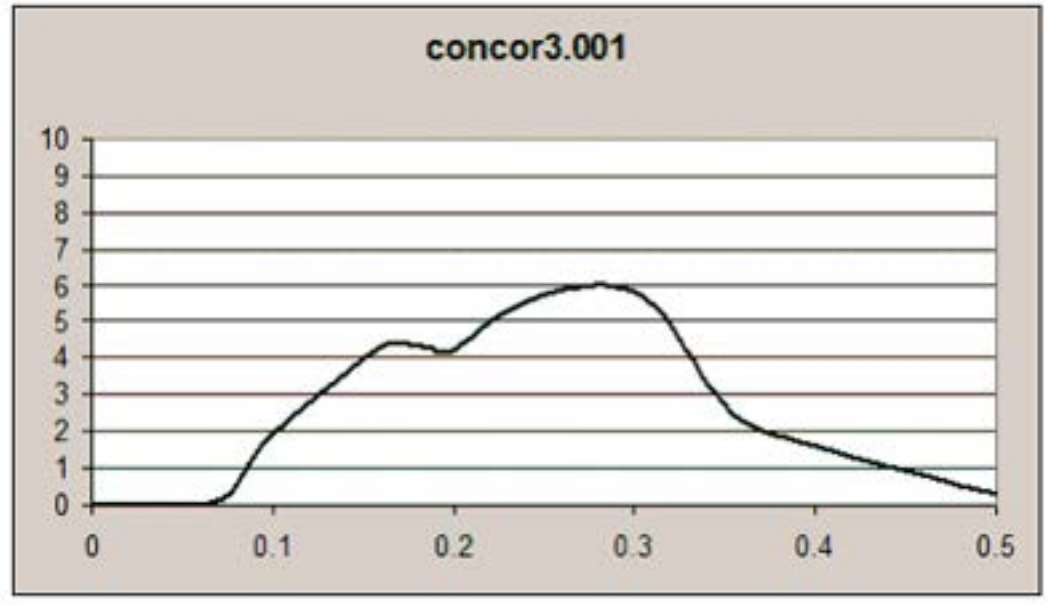


Figure 78: Tensile strength test result (after Kuijpers 2005)

Pre-requisite physical properties of TSL products, besides its tensile strength and flexibilty include but is not limited to the following:

- A good bond with the surface it is sprayed onto.
- Penetration of fissures and cracks in the surface to be sprayed.
- Toughness, to handle the general wear and tear of mine operations in terms of blasted rock and mishandled tools.
- Resistance to the mine environment, humidity, temperature.

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- That the product is self-extinguishing or does not support combustion. The latter is ideal but unlikely as most products are polyurethane, polyurea, methacrylate or latex based. These products can however be self-extinguishing based on flame-retardants.
- The ease with which the product can be applied and the simplicity and robustness of the required equipment.



- Kuijpers Thin Skin Liners
- NS Consultancy (2004). Report for AngloGold Corporate Rock Engineering. In-Situ Testing of a Thin Skin Liner.
- Kuijpers J.S., (2005). Testing of concor liner. CSIR report.

Wire mesh, Rope lacing, Tendon straps

Wire mesh, shotcrete, wire mesh and shotcrete in combination, and shotcrete reinforced with various types of fibres together form a collective group of rock excavation support called "membrane" support. Additional types of support to these area cover support types include wire rope lacing and various types of metal or synthetic straps to cater for bad ground conditions. Area cover support sysstems have to have qualities that allow them to yield material in rockbursts and blocky ground conditions. They therefore are required not to have stiff loading characteristics to achieve this. Various membrane systems can be combined to form one support system or used individually to achieve area cover support for rock blocks. An example of welded wire mesh and metal straps is shown in Figure 79.



Figure 79: Welded mesh and tendon straps, (Ortlepp and Erasmus)

An example of woven diamond wire mesh together with wire rope lacing in rock burst conditions emphasises the containment capacity of membrane support.

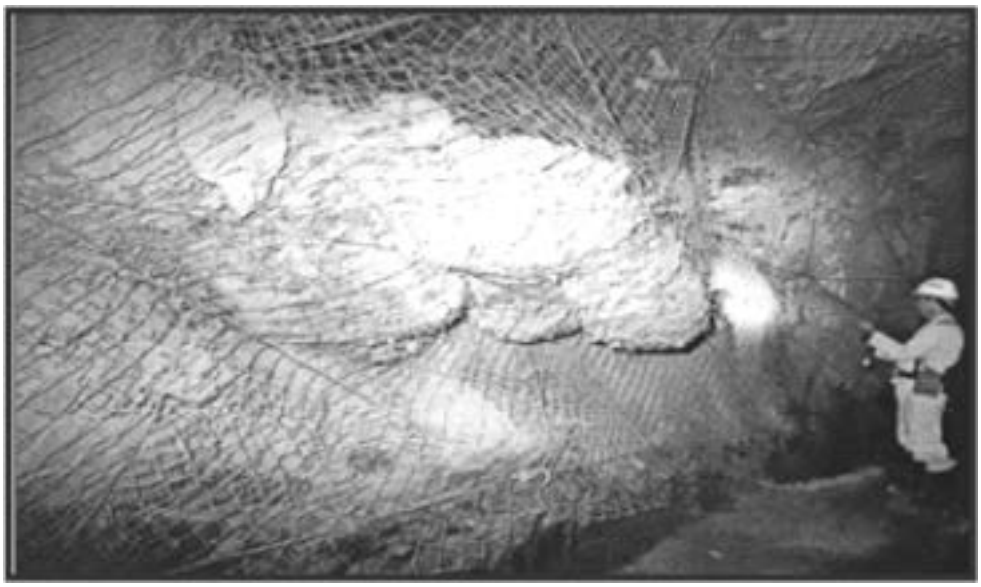


Figure 80: Wire mesh and wire rope lacing combination (Stacey 2001)

- Ortlepp W. D. and Erasmus P.N.. Capacities of Wire Mesh Under Dynamic Loading. In: The performance of yielding straps under dynamic loading. South African Institute of Mining and Metallurgy International Seminar on Surface Support Liners. Duraset SCS, Westonaria, South Africa.
- Stacey T.R., (2001). Review of membrane support mechanisms, loading mechanisms, desired membrane performance, and appropriate test methods. SAIMM, v101n07p343.
- Page 329, Hoek E and Brown ET, Underground excavations in rock, 1980
- Page 207, Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999
- Page 312, Brady BHG and Brown ET, Rock mechanics for underground mining, 2006
- Page 45, Spearing AJS, Handbook on hard rock strata control, 1995
- Page 87, Budavari , Rock mechanics in mining practice, 1983
- Page 133, Stacey TR, Best practice rock engineering for other mines, 2001
- Page 256, Hoek E, Practical rock engineering, 1993

LEARNING OUTCOME 5.5.4.5

CONNECTION 259

Refer to Paper 2 Outcome 5.1.8 on page 293

CHARACTERISE THE FOLLOWING ASPECTS OF THE ABOVE UNITS :
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

LEARNING OUTCOME 5.5.4.6

CONNECTION 260

Refer to Paper 2 Outcome 5.1.9 on page 331

DESIGN AND EVALUATE THE USE OF APPROPRIATE SUPPORT UNITS, SUPPORT SYSTEMS, SUPPORT PATTERNS AND INSTALLATION PROCEDURES FOR GIVEN ROCKMASS CONDITIONS, STRESS REGIMES AND MINING LAYOUTS.

CHAPTER 6

PAPER 3.1 HARD ROCK TABULAR MINING

LEARNING OUTCOMES

6. INVESTIGATION TECHNIQUES

6.1. R OCK TESTING

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

6.1.1. Describe, explain and discuss various rock testing procedures in terms of :

- Type of test procedure relevant to data requirements (UCS, TCS, UTS, etc)
- Sample gathering, Sample description, Sample labeling, Sample preparation, Sample quality
- Test methods and procedures to obtain required data
- Data collection, Data classification, Failure modes
- Interpret and incorporate test results in analysis and design
- Account for the limitations of laboratory tests results relative to mine design requirements.

6.2. MONITORING

6.2.1. IN-SITU STRESS MEASUREMENT AND MONITORING

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

6.2.1.1 Sketch, describe, explain and discuss the techniques used to measure in-situ stress in the underground rockmass

6.2.1.2 Describe, explain and discuss the equipment used to measure in-situ stress in the rockmass

6.2.1.3 Interpret, explain and discuss given stress measurement data in terms of likely rockmass, pillar or excavation behaviour.

6.2.2. SEISMIC MONITORING SYSTEMS

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

- Describe, explain and discuss the differences between geophones and accelerometers
- Describe, explain and discuss typical applications of geophones
- Describe, explain 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

- Describe, discuss and explain how to determine objectives for a seismic monitoring system in tabular hard-rock mines
- Explain and discuss key performance criteria for seismic networks
- Explain and discuss the evaluation of historic seismicity for network design
- Explain and discuss the requirements of spatial seismic coverage
- Explain and discuss methods to quantify required seismic sensitivity and location accuracy
- Explain and discuss the needs of various stakeholders for seismic information with respect to monitoring objectives
- Describe, discuss and explain the role of seismic sensitivity and location accuracy modelling in network design
- Describe, discuss and explain how to derive a compromise between ideal network layout and the following constraints :
- Accessibility constraints
- Budgetary constraints
- Describe, discuss and explain the factors that need to be considered in placing seismometers
- Explain and discuss how seismometer placement could affect the event location accuracy of a seismic system
- Explain and discuss how seismometer placement could affect the sensitivity of a seismic system
- Describe, discuss and explain relevant procedures to ensure that seismic monitoring information is distributed effectively to stakeholders
- Describe, discuss and explain methods of quality assurance in seismic system management
- Describe and discuss relevant aspects of seismic system audits
- Describe and discuss methods to assess stakeholder needs
- Describe and discuss methods to identify possible shortfalls between seismic monitoring objectives and seismic system performance
- Describe, discuss and explain the predictability of seismicity
- Describe, discuss and explain the predictability of rockbursts
- Describe, discuss and contrast the various seismic magnitude scales
- Describe and discuss the use of seismic data in pro-active and re-active mine design
- Describe and discuss the main seismic parameters used in analysis and interpretation
- Describe and discuss how seismic data and parameter relations are interpreted.

6.2.3. SUBSIDENCE MONITORING

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

6.2.3.1 Sketch, describe, explain and discuss various subsidence models

6.2.3.2 Sketch, describe, explain and discuss various subsidence modelling procedures

6.2.3.3 Sketch, describe, explain and discuss various subsidence monitoring procedures.

6.3. MODELLING

6.3.1. NUMERICAL MODELLING

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

6.3.1.1 Describe, explain and discuss the differences between physical, empirical and numerical models

6.3.1.2 Describe, explain and discuss the limitations of physical, empirical and numerical models

6.3.1.3 Describe, explain and discuss the selection of appropriate models and codes to tackle various problems

6.3.1.4 Describe, explain and discuss the input of appropriate parameters to investigate various problems

6.3.1.5 Describe, explain and discuss the interpretation of output in the investigation of various problems

6.3.1.6 Describe, explain and discuss the difference between relative and absolute results from numerical models

6.3.1.7 Describe, explain and discuss the appropriate use of relative and absolute results from numerical models

6.3.1.8 Describe and explain the relevance of calibration, correlation and back analysis in interpreting absolute results from numerical models

6.3.1.9 Describe and explain the relevance of calibration, correlation and back analysis in interpreting relative results from numerical models.

6.4. AUDITING

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

6.4.1. Describe, explain and discuss the concept of monitoring for understanding, prediction and design.

6.1. ROCK TESTING

LEARNING OUTCOME 6.1.1

CONNECTION 260

Refer to Paper 2 Outcome 6.1.1 on page 391

DESCRIBE, EXPLAIN AND DISCUSS VARIOUS ROCK TESTING PROCEDURES IN TERMS OF :

- Type of test procedure relevant to data requirements (UCS, TCS, UTS, etc),
- Sample gathering,
- Sample description,
- Sample labeling,
- Sample preparation,
- Sample quality,
- Test methods and procedures to obtain required data,
- Data collection,
- Data classification,
- Failure modes,
- Interpret and incorporate test results in analysis and design,
- Account for the limitations of laboratory tests results relative to mine design requirements.

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6.2. MONITORING
6.2.1. IN-SITU STRESS MEASUREMENT AND MONITORING

LEARNING OUTCOME 6.2.1.1

CONNECTION 262

Refer to Paper 1 Outcomes 5.5 on page 155 and 5.7 on page 175

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE TECHNIQUES USED TO MEASURE IN-SITU STRESS IN THE UNDERGROUND ROCKMASS

LEARNING OUTCOME 6.2.1.2

CONNECTION 262

Refer to Paper 1 Outcomes 5.5 on page 155 and 5.7 on page 175

DESCRIBE, EXPLAIN AND DISCUSS THE EQUIPMENT USED TO MEASURE IN-SITU STRESS IN THE ROCKMASS

LEARNING OUTCOME 6.2.1.3

CONNECTION 263

Refer to Paper 1 Outcomes 5.5 on page 155 and 5.7 on page 175

INTERPRET, EXPLAIN AND DISCUSS GIVEN STRESS MEASUREMENT DATA IN TERMS OF LIKELY ROCK MASS, PILLAR OR EXCAVATION BEHAVIOUR.

Relate the stress levels to rock strength and estimate failure using failure criteria or pillar behaviour as in "STOPE SUPPORT DESIGN CRITERIA" on page 352

LEARNING OUTCOME 6.2.2

CONNECTION 264

Refer to Paper 2 Outcome 6.3 from page 472

6.2.2. SEISMIC MONITORING SYSTEMS

- Describe, explain and discuss the differences between geophones and accelerometers
- Describe, explain and discuss typical applications of geophones
- Describe, explain 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
- Describe, discuss and explain how to determine objectives for a seismic monitoring system in tabular hard-rock mines
- Explain and discuss key performance criteria for seismic networks
- Explain and discuss the evaluation of historic seismicity for network design
- Explain and discuss the requirements of spatial seismic coverage
- Explain and discuss methods to quantify required seismic sensitivity and location accuracy
- Explain and discuss the needs of various stakeholders for seismic information with respect to monitoring objectives
- Describe, discuss and explain the role of seismic sensitivity and location accuracy modelling in network design
- Describe, discuss and explain how to derive a compromise between ideal network layout and the following constraints :

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  - Accessibility constraints
  - Budgetary constraints
- Describe, discuss and explain the factors that need to be considered in placing seismometers
- Explain and discuss how seismometer placement could affect the event location accuracy of a seismic system
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- Describe, discuss and explain the predictability of rockbursts
- Describe, discuss and contrast the various seismic magnitude scales
- Describe and discuss the use of seismic data in pro-active and re-active mine design
- Describe and discuss the main seismic parameters used in analysis and interpretation
- Describe and discuss how seismic data and parameter relations are interpreted.

INTRODUCTION

The following is quoted from RJ Durrheim's summary of methods to mitigate rockburst risks in South Africa's deep hard-rock mines after 100 years of rockburst research (Durrheim, 2010):

South Africa hosts two of the world's richest ore bodies, the gold-bearing conglomerates of the Witwatersrand Basin and the platinum-bearing pyroxenites of the Bushveld Complex. Both ore bodies extend to depths of several kilometers.

Mining-induced seismicity and its hazardous manifestation, rockbursting, were first encountered in the early 1900s when extensive stopes reached depths of several hundred meters. A committee, appointed in 1908 to investigate the cause of the tremors, recommended that seismographs be installed, and scientific observations began in 1910. Research was carried out in an ad hoc way until a mining research laboratory was established in 1964. Since then, research addressing the risks of deep hard rock mining has mostly been conducted under the auspices of the Chamber of Mines Research Organization, the Mine Health and Safety Council, and several collaborative research programs.

Research work to mitigate the rockburst risk has focused on three main areas:

1. development of macro-layouts (e.g. sequential grid) and regional support (e.g. backfilling) to control the release of seismic energy through the geometry and sequence of mining;

2. development of support units and systems (e.g. rapid-yielding hydraulic props, pre-stressed elongates) to limit rockburst damage; and
3. mine seismology, which seeks to develop techniques to continually assess the seismic hazard and control seismic activity (e.g. through the rate of mining).

Implementation of these technologies, together with improvements in training, work organization and regulation, have reduced fatality rates and made it possible to mine successfully at depths exceeding 3.5 kilometers.

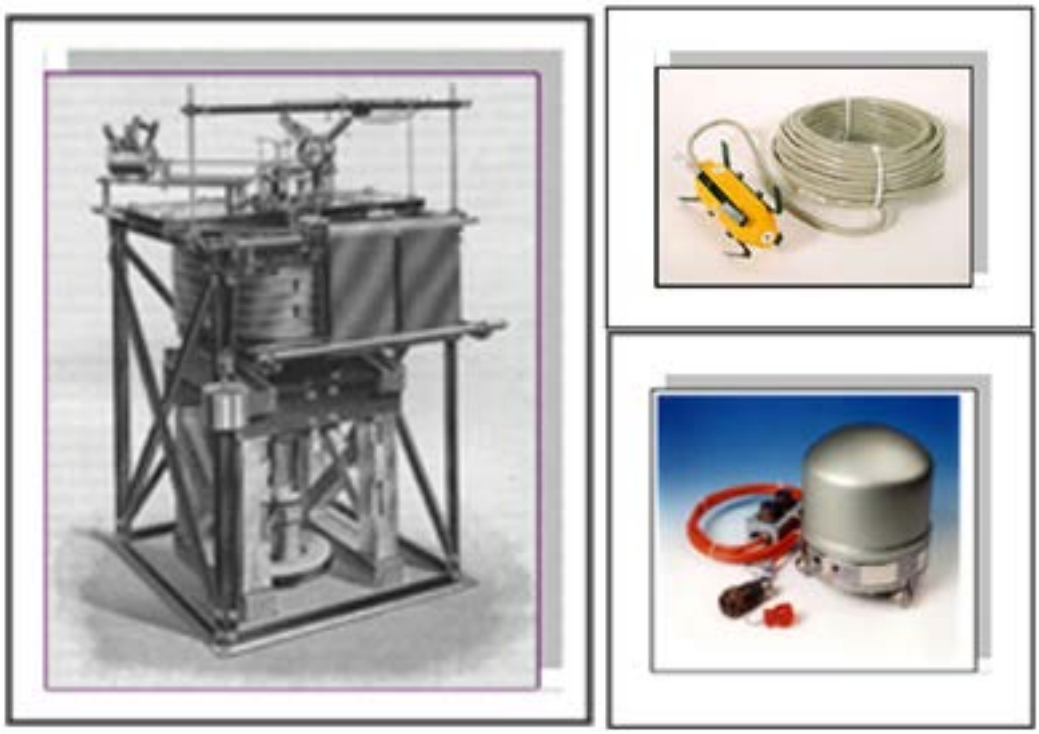


Figure 1: (a) Seismograph installed at the Union Observatory in Johannesburg (1910, courtesy Africana Museum; Durrheim, 2010), (b) Tri-axial sensor set used in mines (grouted into bore hole), (c) SANSN broad-band seismometer (surface mounted)

All of the deep- and ultra-deep gold mines along the edge of the Witwatersrand basin now operate seismic systems, collect data and analysis these regularly. The same holds for many of the intermediate depth platinum mines in the Western Bushveld, the deepest of which is Northam Platinum Mine near Thabazimbi. The standard for seismic system technology is a combination of tri-axial geophone sets delivering seismograms to near-by seismographs (or data acquisition units) where signals are digitised, stored and subsequently transmitted to surface, provided they pass validity tests.

In addition to mine networks, the Council for Geosciences in Pretoria operates seismometers near or directly within mining districts, as in the West Rand and Vaal River regions. Over 95% of seismicity recorded by the South African National Seismic Network originates in mining areas and the cluster of recorded events geographically linked to the deep mines follows the arch-shaped outline of the Wits basin, from Virginia in the South West to Boksberg in the North East (Figure 2.

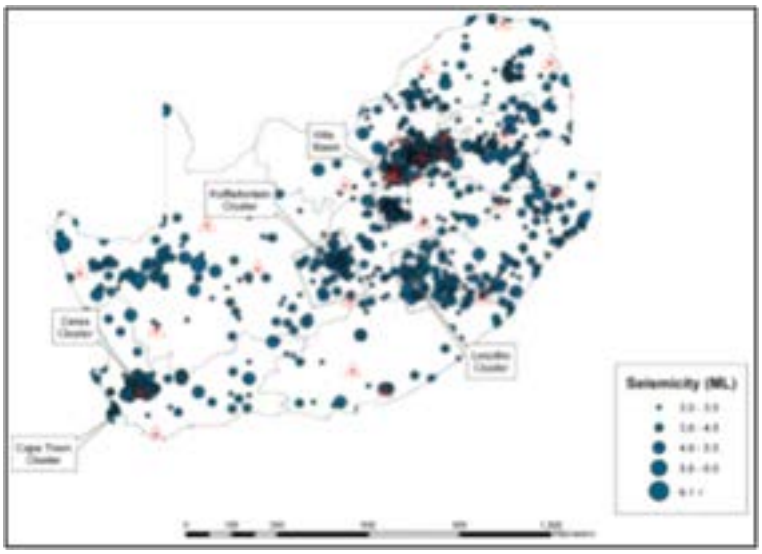


Figure 2: Witwatersrand Basin Cluster as shown in Map of Earthquakes (1620 – 2008) contained in the South African National Seismological Database (SANSD). Known clusters relating to natural and mining-induced seismicity are highlighted.

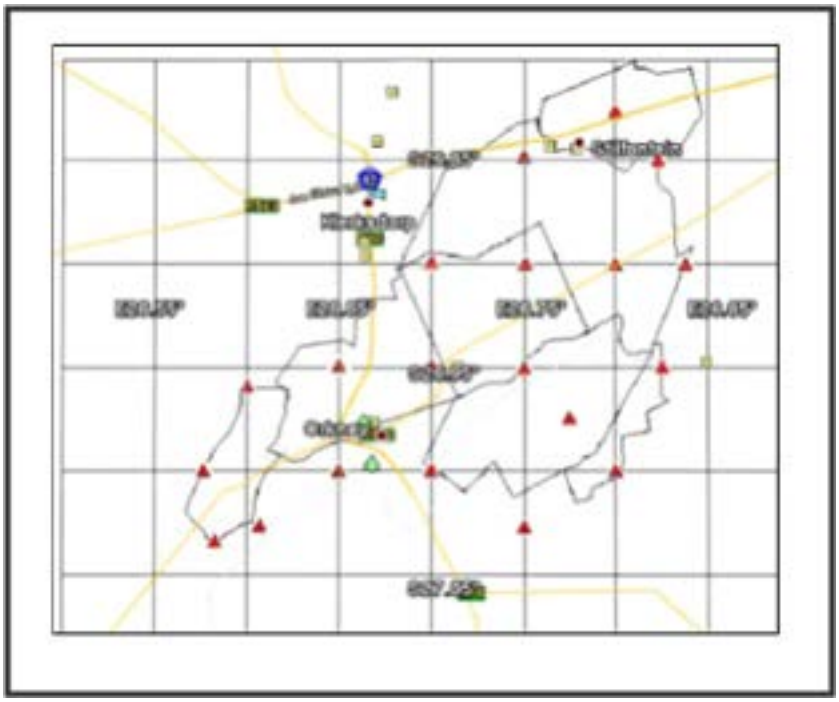


Figure 3: Planned network layout in the KOSH region (with permission from CGS). The seismic recording stations are represented by triangles (from Singh et al., 2009)

If current plans are implemented, the SANSN will include 12 stations in the Johannesburg area, 10 in the Far West Rand and a further 25 in the Klerksdorp region, between Stilfontein and Orkney (Figure 3). The main purpose of the SANSN is to fill a "monitoring gap" that exists in the large-event range due to the sensors deployed in mines, which are aimed at recording small magnitude sources down to M=-1 or even lower. These sensors are unable to record large magnitude events equally well as sensors are built for a specific, limited frequency range. The SANSN's broad-band technology is suitable for seismic events of roughly M>2.0, a range not well covered by mine sensors. The other objective is to bring information pertaining to mining induced seismicity into the public domain. The public, as the Stilfontein quake and several tremors in the

Free State have shown, may be negatively affected by this phenomenon.

To illustrate the effect between a standard mine network (sensitivity M=0) and the national network (sensitivity M=2) see Figure x: 29 events with the national network versus 4291 events recorded by the mine network, which is equivalent to a 99% reduction in information obtained by the SANSN when compared to the mine system. In practice, seismic systems are not entirely deaf to seismic events outside their designed frequency range. Events are recorded beyond the optimum range, but the data are likely to be both inaccurate and incomplete.

INTERESTING INFO

Magnitude relation used by the national seismic network:

$M_L = \log_{10}(A) + 1.11 * \log_{10}(D) + 0.00189 * D - 2.09$

with factors A (wave amplitude in nanometers) and D (hypo-central distance in kilometers).



Figure 4: (a) Events recorded by a network with Mmin=0 (left) as opposed to a network with Mmin=2 (b); colour coding is according to time from cold to warm (warm = recent).

The following section explains principles of seismic network design in tabular mining environments, how to audit seismic systems and how to optimise the value derived from a mine system.

SEISMIC SYSTEM DESIGN

Quote from "Minimising the rockburst risk (SIM050302 Phase 2, output 3), Durrheim et al [2007]:

There is often a fundamental difference between civil and mining engineering projects (Stacey et al., 2006). The design requirements for a civil engineering project (e.g. a road tunnel or dam) are usually clearly defined, and usually require a conservative design as the projects are usually of a long-term nature and open to the public. In contrast, many mining excavations are temporary or short term, not open to the public, and the viability of a project is usually critically sensitive to costs.

The legal requirements formulated in the MHSA stipulate the obligation by mines to monitor and mitigate seismic risk, but there are also operational needs which should be considered when planning the installation of a seismic system. A stakeholder survey should be conducted initially when setting up seismic information distribution procedures. Stakeholder requirements are central to the setting up of a seismology service as they determine the information type and reporting procedures seen to add value to the operation. A seismic system design and operation should be oriented on these needs.



Figure 5: (a) Acceptable seismic station setup and (b) Not an adequate seismic station setup.

The process of installing a seismic system on a mining operation with seismic hazard potential begins with the formulation of seismic monitoring objectives: What is the purpose of seismic system operation and what kind of seismic network would be adequate for a given mine? In answering these questions, one will come across the notion that not all parts of a mine require equal coverage, usually because the mine consists of active production areas with high worker exposure, back areas with little or no exposure, and areas where vertical, inclined or horizontal access ways are concentrated. As a result, the required network coverage varies spatially.

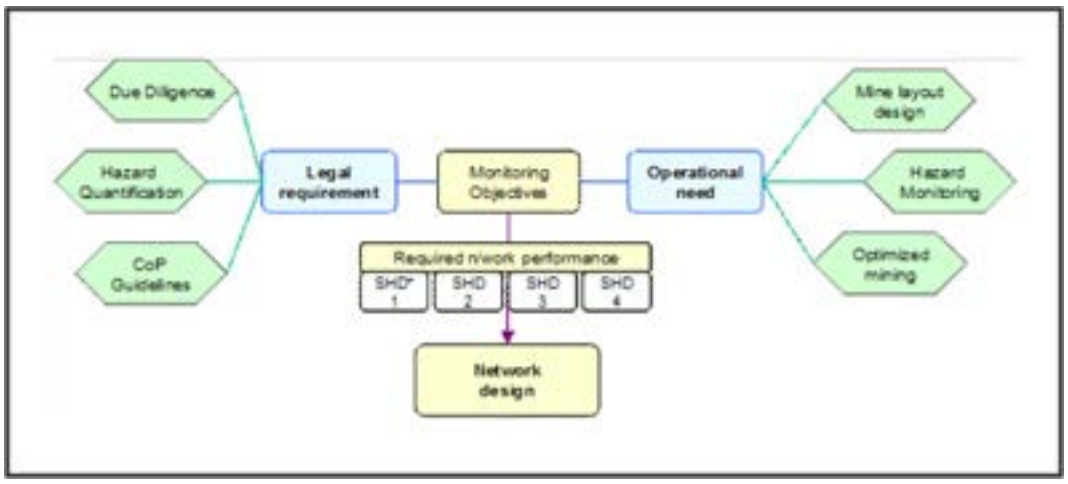


Figure 6: Process of defining monitoring objectives for each of the SHDs, based on legal requirements and stakeholder needs

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PAPER 3: CHAPTER 6

CONNECTION 265

See Paper 2 Outcome 6.3 FROM PAGE 472

The study material on mine seismicity in Paper 2 introduced a set of standard monitoring objectives widely accepted for hard-rock tabular mines, in line with The Guide to routine seismic monitoring in mines (Mendecki et al, 1999) in the Handbook for Rock Engineering Practice (Jager and Ryder, 1999):

- "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.

These are the basic objectives and others may be added, such as the monitoring of known seismically active faults and dykes, for instance in the shaft pillar area; the monitoring of specific high-grade remnants earmarked for extraction; or the detection of discontinuities ahead of the face in long-wall mining and similar applications.

Further guidance on the purpose of network operation is given by Durrheim et al. (2007), who summarised the results of a research programme into monitoring needs by stating the following "primary objectives":

a. Rapid response to rockbursts to limit the loss of life and infrastructure.
b. Seismic hazard assessments (SHA) to guide efforts to mitigate the risk associated with rockbursts, routinely described as long, medium and short term.
c. Back analysis of large and/or damaging events to determine the cause of the event(s).
d. Research and experimental development to improve knowledge of the rockbursting phenomena and technologies to mitigate the hazard.

The list reflects current practice on nine participating seismically active mines whose procedures were assessed in 2006 during the course of the SIM050302 research programme. As such, A to D summarise the state of seismic data analysis on SA gold and platinum mines at present.

The seismic network design has to make provision for these monitoring objectives in the designated parts of the operation in which they have to be met (see flow chart in Figure 6 above). A suitable methodology to accommodate such spatial variations is that of seismic hazard districts (SHDs)), similar to ground control districts. For each SHD the following three steps are performed, each needing separate considerations:

1. Quantification of the seismic risk experienced in an area;
2. Formulation of the objectives of seismic monitoring appropriate for a given risk level;
3. Translation of seismic monitoring objectives into the required seismic system performance.

BRIDGING GAP

Learn more about:

- sensitivity (...is the magnitude threshold above which all events are reliably recorded)
- location error (... measure of the network's ability to locate a seismic source accurately)
- source parameter quantification (is reliable and accurate where events are recorded by a large array of fully functional tri-axial sensors)
- station down-time (... reflects on the overall state of health of a network and the quality of upgrades and upkeep)
- % rejected events (...is an indicator of network health and data processing standards)

INTERESTING INFO

How to evaluate historic seismicity?

- rock engineering reports of FOG incidents;
- rock-related injuries;
- eye-witness accounts of shaking, rock-talk, side-wall ejection, footwall heave, crushing, and bursting;
- seismic information gathered by neighbouring mines;
- seismic information available from the SANSN.

back

back

The quantification of seismic risk levels and the delineation of hazard districts, which is a recommended but not formally required practice, results in the subdivision of a mine into zones each being exposed to one of three possible risk level.

The following classifications could be used:

a. Low = Occasional large events in the potentially damaging range, but no exposure of personnel or mineable ore reserves to rockburst damage (typical: back area). Alternatively, development and low-production zones with small events well below the damage threshold.
b. Intermediate = Seismic events in the potentially damaging range combined with limited exposure of personnel and ore reserves (typical: remnant mining). Alternatively, rare events in the damaging range locating in regular production area (high exposure).
c. High = Full range of events, including those in the damaging range (typical: regular production area with rockburst history). Alternatively, potential for very large events combined with moderate exposure (typical: shaft pillar extraction).

Where required, risk levels may be quantified numerically for these three cases to determine an area score and to classify mining sections into their respective Seismic Risk Zones (SHD) which are then converted into monitoring classes: Monitoring Class I is adequate for high risk levels, Class II for intermediate, and Class III for low risk areas. The result of this exercise is shown in Figure 7, in which a mine in the Vaal River region was subjected to the classification exercise.

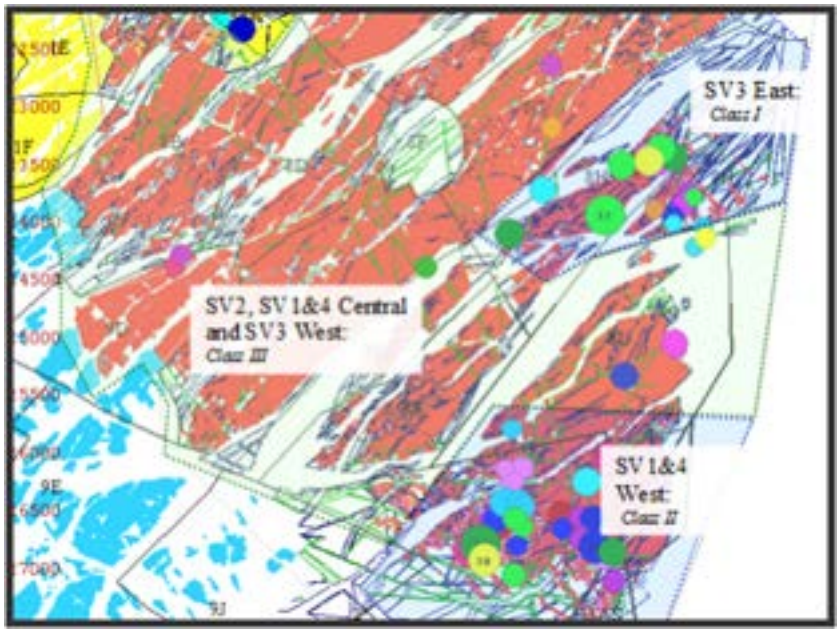


Figure 7: Subdivisions in a tabular gold mine into Class I, II and III seismic monitoring areas.

NOTE

In general, it is not useful to locate events to an accuracy much smaller than their source size, e.g. to 20m accuracy when sources measure 50-100m across.

Once seismic risk zones are established and classifications carried out, the final step towards network design is taken by converting the monitoring requirements for each class into seismic system performance benchmarks. To this effect, the following conversions are suggested on the basis of past design experience:

back

Class I – High production area (high requirement):
High system reliability and quality of raw data: Minimum of 15 waveforms per event with not less than 10 picks per event (mixed P&S), at least 4 P and 4 S-spectra (note 1) complete with high-frequency drop-off; locations with 2 directions and additional 2 azimuths; location accuracy in plan equal to 1-2 panel lengths; quick location and quick magnitude enabled (note 2); data throughput sufficient for twice daily instability estimation;

Class II – Widely scattered or remnant mining (intermediate requirement):
Minimum 12 waveforms and 8 picks per event (mixed P&S); locations with 2 directions; location accuracy in plan equal to 3-4 panel lengths; quick location & quick magnitude enabled;

Class III – Back area (low requirement):
Mainly large events: Minimum 6 tri-axial stations; location accuracy in plan equal to source size of potentially damaging events; may include stand-alone units for remnant extraction;

Common operational requirement for all areas: Sensitivity better than the minimum magnitude of damaging events; MT enabled (note 3)



Note 1: Quality spectra: Q-values fixed per station or area, corner frequency well defined

Note 2: Ground-motion relation regularly updated; quick magnitude & location in under 2 minutes;

Note 3: Quality index of configuration >0.5, min. 10 stations, at least 2 stations near pole

APPLICATION

How to...

quantify required sensitivity and
max. allowed location error?

Suggestion: Start with a size estimate of potentially damaging events to ensure these are accurately recorded
and located in 3D.

Apart from the number of seismograms and detected wave arrivals (picks) prescribed to ensure accurate and reliable source parameters in line with monitoring requirements, the assignments formulate specific sensitivity and locations accuracy targets for each monitoring class. These are two fundamental performance criteria for seismic networks, but they are not the only ones: Data speed, recorded frequency range, background noise levels and dynamic range are set by the system manufacturer and are important for system design and data interpretation as they feed into the operational requirements listed above.

In order to ensure adequate network location accuracy and sensitivity, an initial sensor layout and data communication plan is drafted which is then tested against the performance benchmarks derived from the above process. Location error in the reef plane and in the vertical direction are modelled using suitable software (Figure 8). Where necessary, seismic sensors can be added or re-located until the design matches the requirements.

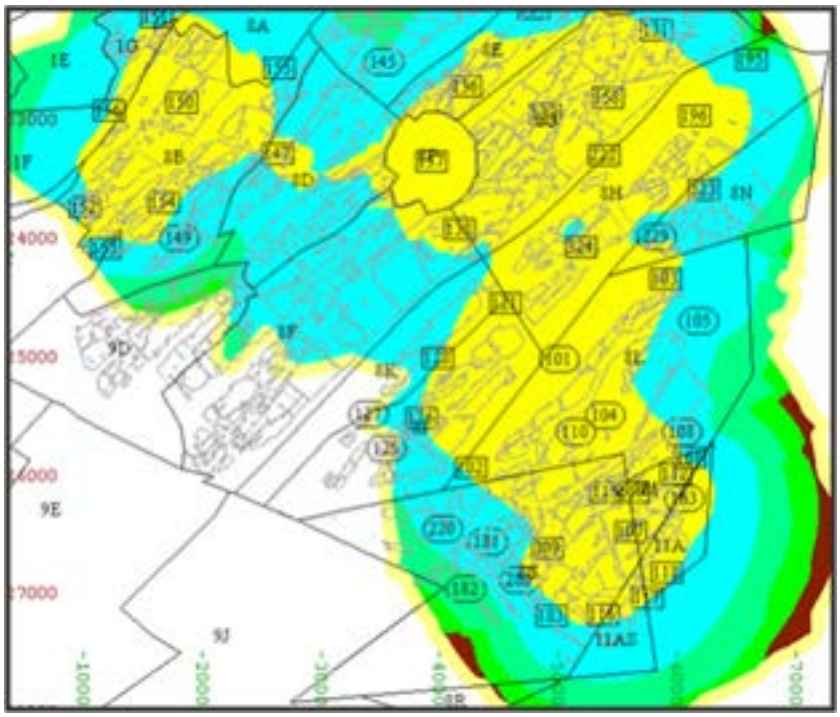


Figure 8: Model of XY location error in the reef plane based on a given sensor layout (numbered □'s and o's).

At this stage of the network planning, two possible constraints need to be considered: Access to the planned sensor sites, and budgetary constraints, should they exist. Both of these may affect the design and both can change with time. Seismic networks do not only need to be purchased and installed, but also maintained for the duration of the monitoring period. And shafts and tunnels currently maintained and safe may not be safe or accessible in a few years time. It is essential to consider both these aspects during the planning phase.

Of similar importance during the planning is the identification of all relevant stakeholders and their needs. Stakeholders are all parties who benefit from the information generated by the seismic monitoring service, who require information in certain time intervals (daily, week, monthly etc.) and in certain formats (verbal communications, by e-mail or cell phone etc.). The seismic system design should to the greatest possible extent, provide for the interests of all stakeholders.

The following chapter focuses on ways and means to audit seismic systems and to review seismic service performance. A number of measures are suggested to ensure that optimal value is derived from an operating seismic system.

SYSTEM AUDITING

Quote:
"Quality has to be managed – it does not just happen."
Johan S. Oakland, 2003



Quality: Totality of features and characteristics of a service or product that bear on its ability to satisfy stated or implied needs [ISO 8402, 1986]

Principles of seismic system audits are based on quality management procedures developed in the 1980s and now well established in manufacturing and service industries worldwide. Some of the first comprehensive texts were authored by WE Deming (1982), PB Crosby (1979) and JM

Juram (1988). Over the past twenty years, comprehensive Total Quality Management (TQM) systems were devised which integrate stakeholders, customers, suppliers and processes into fully managed systems with the aim to achieve optimal results for all parties involved.

The generation of seismic data and the flow of analysis results on mines usually affect three parties: The system manufacturer, the rock engineering department, and the main customer groups on the mine and in head-office. The system supplier usually designs, installs and commissions the seismic system and often takes responsibility for maintenance and data analysis as well; the rock engineering department manages the process of seismic monitoring and reporting; and the customers receive reports and other updates on various aspects of seismicity within their respective areas of responsibility.

A review of seismic systems should therefore be widened to include all product and service providers, management processes and all stakeholders. The central process under scrutiny is that of data interpretation (Figure 9). This process receives amongst other equipment, raw data and methodologies on the input side and produces information, documents, knowledge and compliance on the output side. Two important feedback functions connect customers to the core process (quality of service received) and the interpretation process to the input side (optimisation of inputs).

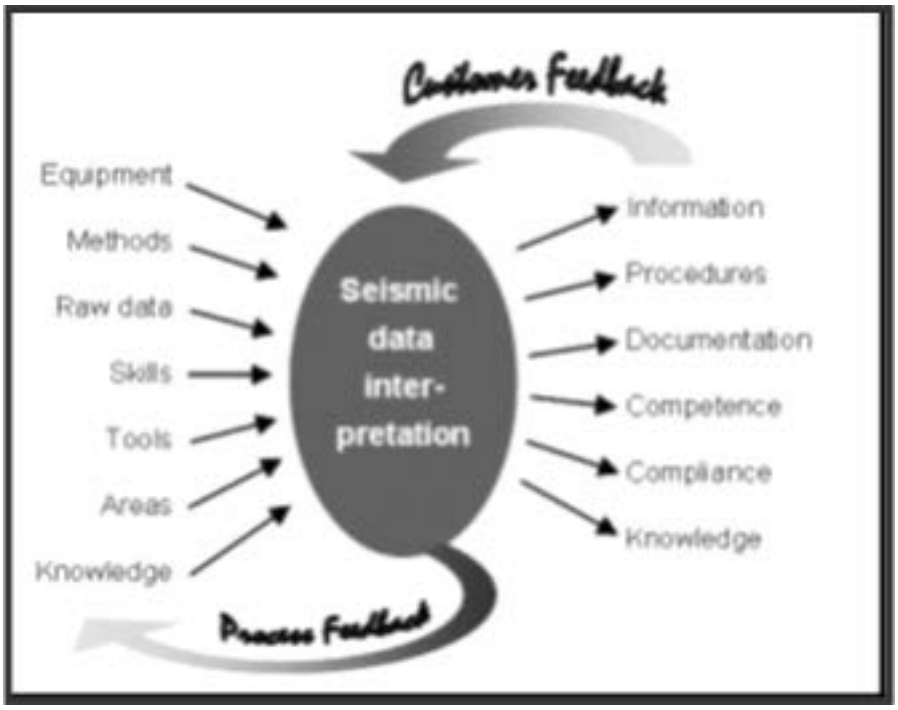


Figure 9: Seismic data interpretation process (Essrich 2006): Inputs (left), outputs (right) and relevant feedback functions from the customer group and from the process back to the inputs.

Initially, when setting up the seismology service, stakeholders should have been informed of standard seismic data analysis techniques and they should be enabled to interpret the results of such analyses. The main stakeholder groups are mine management, service departments such as rock engineering, geology and mineral resource management, and production personnel. Amongst the service departments, the primary customers of a seismology service are rock engineering personnel who should be trained in matters related to mining induced seismicity. Common and most suitable means of seismic information distribution to various interested groups include:

- Monthly and quarterly planning/review meetings; safety and other meetings;
- Placing of seismicity reports on intranet websites or servers accessible to all employees;
- Notification via phone, e-mail and SMS;
- Displays on notice boards, e.g. at the u/g working place;
- Printed and bound report collections on a specific subject, and circulars;
- Informal verbal communications.

To ensure effective information distribution, stakeholders should be awarded an opportunity to provide feedback on the perceived service level. Corrections can then be made to procedures where necessary.

A generally accepted principle in achieving optimum quality targets it that of "never ending improvement". In other words, no process is expected to perform optimally right from the start. Instead, a continuous cycle of improvements is the way to achieve highest performance, and the steps of each cycle are: evaluate – plan –do- check – amend

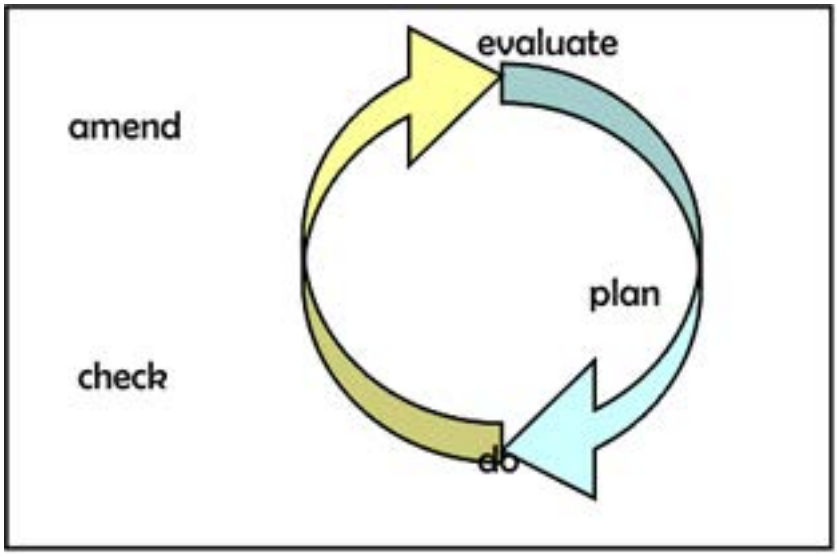


Figure 10: Cycle of improvements

APPLICATION

How to:

- identify external stakeholders (look at corporate, regional office, neighbouring mines, DMR)
- identify internal stakeholders (look at mine management, production management, underground workforce, rock engineers, geology and other service departments
- derive a compromise system layout (find a compromise between optimal layout under existing limitations)

In other words, analyse the current performance (evaluate), consider appropriate improvements (plan), implement improvements (do), check the effect the changes had and finally amend the process. After some time, re-evaluate the system and start over (sketch). This is seen as the proven way to achieve an efficient and effective process flow that satisfies the customer needs.

In the case of a mine seismic service review, it is suggested to consider and evaluate three main parts:

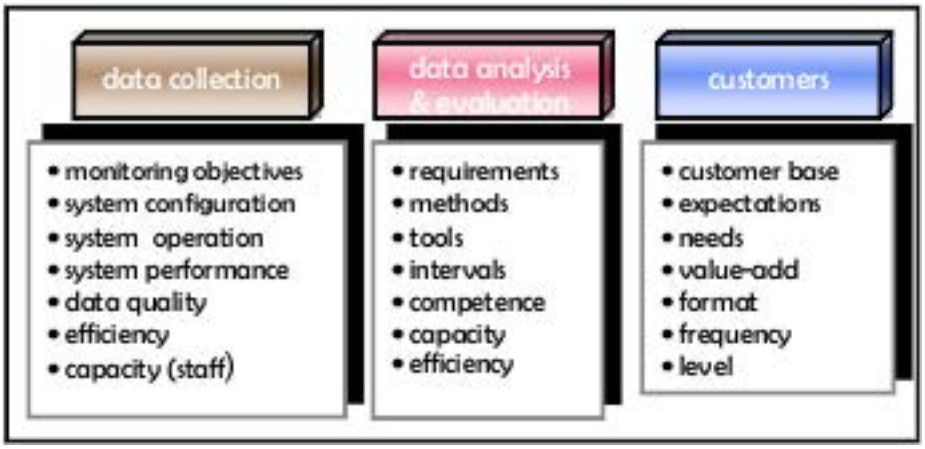


Figure 11: Main aspects of a seismic service review.

It is evident that all three parts are interdependent in that customer needs determine the type and format of information that should be produced by the seismic department, which in turn determines the type of raw data needed from the network. Similarly, should the network or its components be impaired this would negatively impact on the reports delivered to customers.



- no process without data collection
- no data collection without data analysis
- no analysis without decisions
- no decisions without action

A customer questionnaire is a good way to quantify the satisfaction levels achieved by the seismology service. Questions could be formulated such that customers agree/disagree whether seismic information is useful and whether it is provided in the appropriate format. Figure 12 shows the results of such an exercise.

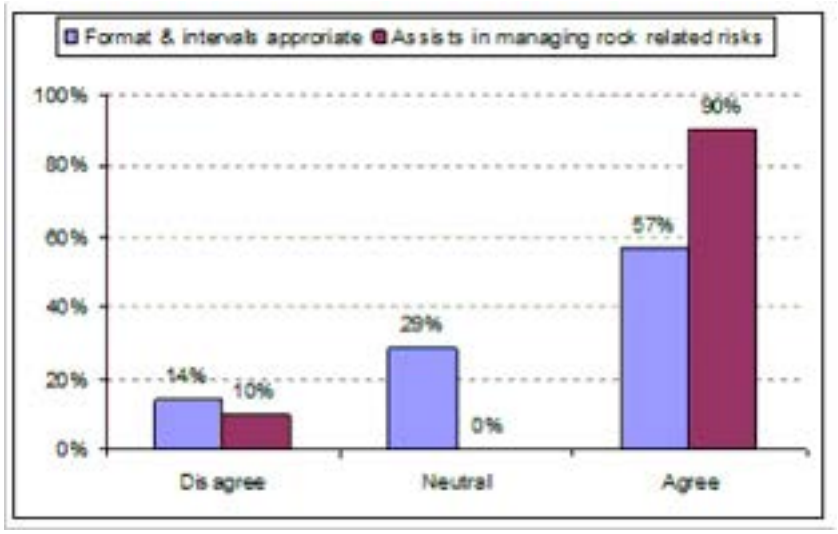


Figure 12: Customer response to two questions related to the format and usefulness of seismic information

For the data analysis and interpretation part of the audit, reviewers need to evaluate the appropriateness of analysis methods, the tools used to conduct analyses, the complement and competence of staff and other capacity related issues. As audit findings show, routine data analysis is not always performed by adequately qualified staff and the results are not always understood by the recipients (training needs!).

The same might be true for the technical maintenance of deep-level mines, which can negatively affect data quality. Mines often do not have trained seismic network technicians and rather outsource this function to the network supplier or an associated agency. As a result, the maintenance function is not adequately managed and is subject to budget constraints; and one chosen way to achieve savings is to reduce technician time on the shaft.

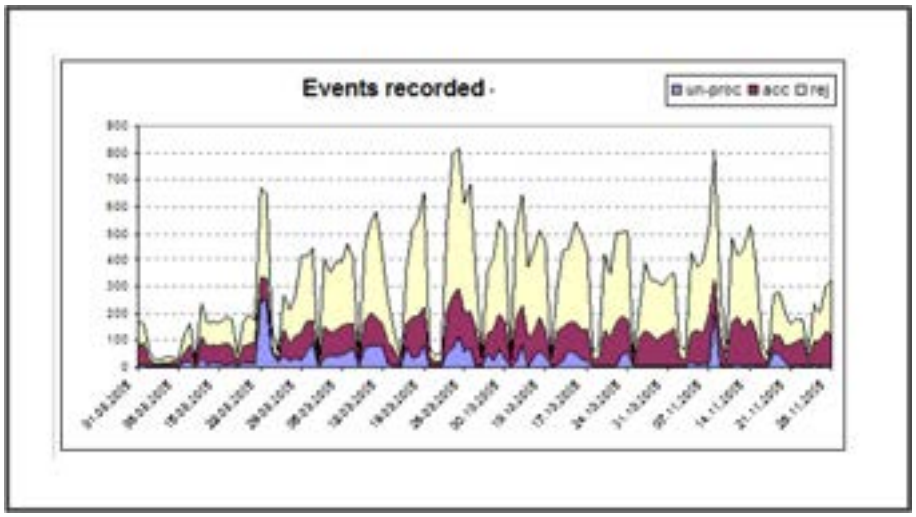


Figure 13: Daily percentage of unprocessed, accepted and rejected events (minima correspond to weekends).

APPLICATION

Question:
Think of reasons why events may be rejected or never processed in the first place?

Unprocessed = associated, but below quality criteria and rejected by the data processor.

Common symptoms of poor hard- and software maintenance are faulty sensors, poor data communication between seismometer stations and surface, sub-optimal settings of system parameters and an overall inadequate level of data quality. In some cases, the percentage of recorded but rejected events exceeds the percentage of accepted events in the data base, an issue that may have a number of interlinked causes. In the example in Figure 12, 50-70% of all recorded events are rejected.

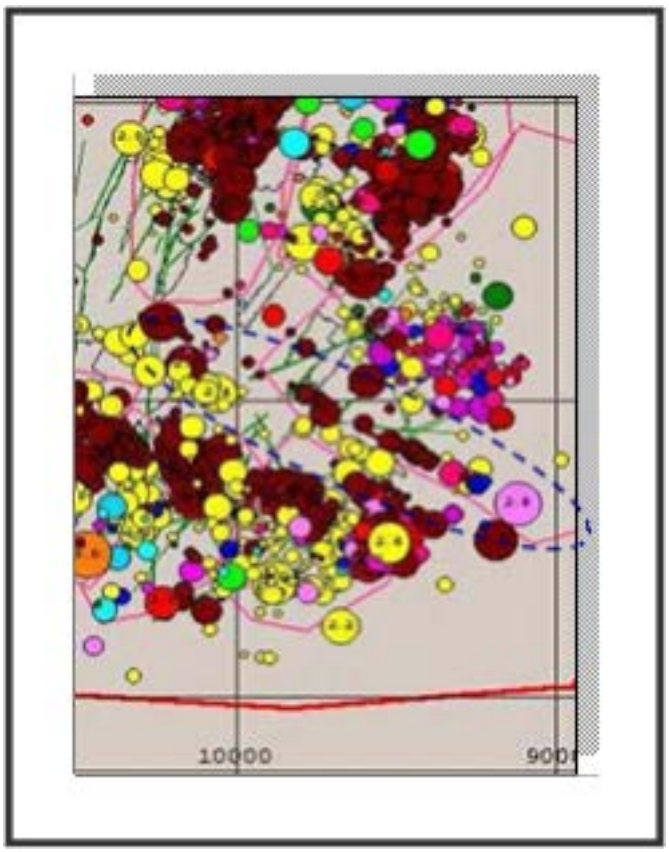


Figure 14: Location concerns

Certain sensor configurations can cause artificial event locations such as the 'spine' in this plan view of a longwall mine's seismic activity.

As an example of possible causes underlying levels of poor seismic data, the functionality of seismic sensors installed underground needs to be considered. In an example from seven deep gold mines in the various operations of a large gold producer, 20-50% of sensors may be impaired or faulty, which negatively impacts on the value the mines receive from their respective seismic systems. Sensor health is a crucial element of quality data collection, on par with optimal system settings and sound data processing procedures.

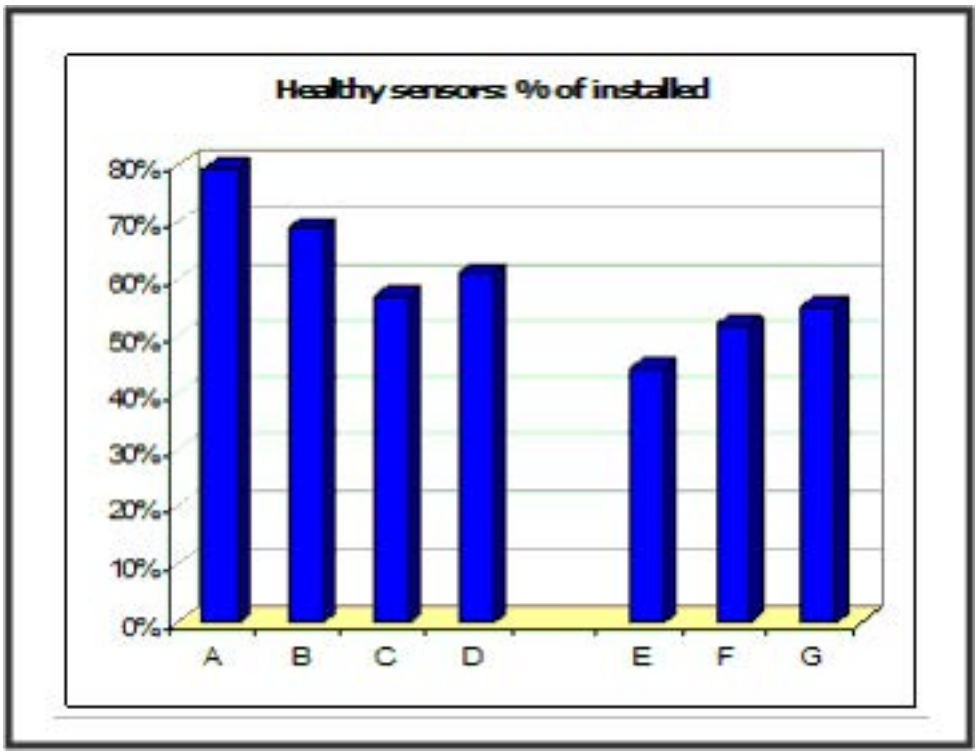


Figure 15: Percentage of installed sensors considered fully functional (7 mines)

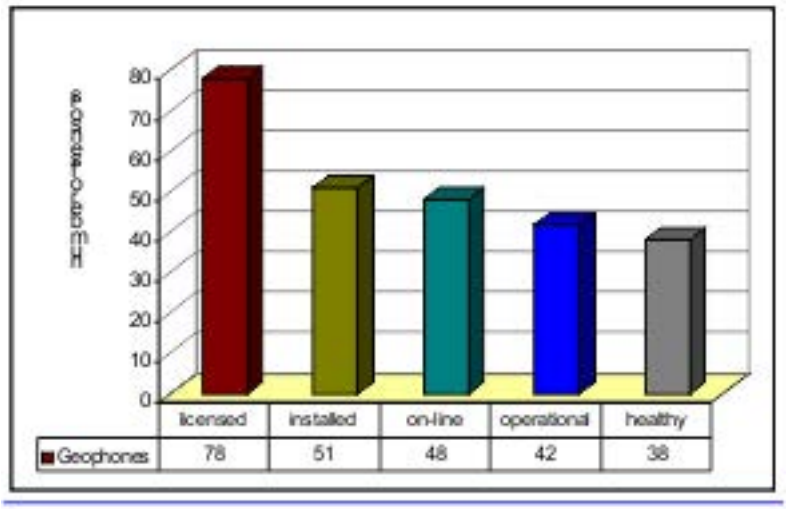


Figure 16: Number of geophones licensed, installed, on-line, operational and healthy

On some mines, the number of healthy sensors in a network can be less than half the number of licensed sensors (Figure 16). Licensed sensors are those for which the mine purchased operating licenses and pays technical support for. Of course, once a sensor site was approved and hard- and software to operate it was purchased, it should also be installed and commissioned. Only a functional healthy sensor adds value to the operation and generates a return on investment.

In the context of value creation it should be noted that, according to surveys conducted over the past ten years, the average cost per recorded, processed and accepted seismic event lies in the range of R0.10 to R40 for some of the deep level gold mines. It is therefore important that seismic system operations are optimised to the full extent of their technical possibilities and that all parts of a mine seismology service, from

data collection through analysis to reporting, are effective, efficient and relevant.



- Durrheim, RJ, 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.
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VALUE-ADD PROCESSES

It must count amongst the most frequently quoted objectives of seismic monitoring that it should allow the forecasting of unfavourable trends in seismic rock mass response to mining. Thus, it is central to the expectations of customers irrespective of whether it is technically feasible and practically possible or not. Any value-add consideration should take into account to which extent seismic data analysis allow the prediction of seismic hazard, or even rockburst risk, which is the most relevant to the mining operation.

Predictability of seismicity is matter of scale, both in time and in space. If the factors determining the nature of seismic response remain constant over extended periods, the seismic response is likely to be predictable. Among the determining factors, we count stress levels and rate of stress change, rate of production, the presence of geological structures with seismic hazard potential, and the existence of mine design elements prone to seismic failure, such as under-designed pillars, excessive panel leads and abutments and high-stress mining areas.

Under these circumstances, the rate of small and large events can be predicted with some accuracy, even the approximate location of large events, as long as the forecast period is in the range of weeks or months. More difficult and usually not successful is the prediction of the exact locations of large events on specific structures and over a period of a few days, i.e. on small time scales and within limited spatial extent.

The reason for the poor predictability on small scales lies with the mechanical process of seismic failure and the conversion of elastic strain energy into wave motion, permanent deformation and heat. When seismic failure is initiated in the rock mass, strain energy is released through the destruction of the molecular bond between particles. Some of this energy appears as dynamic movement, which may in turn break more bonds leading to an escalation and a growth in fracture length resulting in a large-scale failure. Alternatively, energy may simply be radiated and the heat dissipated, in which case the fracturing process stops.

The end result is that at failure initiation the final extent of the failure has not been decided upon.

These characteristics of seismic failure processes may partly explain the low overall success rate of event prediction on small time scales and for specific work places. Since hazard is fundamental to risk, the same principle unpredictability applies to rockburst risk. However, the way in which

seismic sources and seismic waves interact with excavations can be specific to a certain mining operation, depending on local conditions. The use of in-stope pillars on some tabular hard-rock mines, for instance leads to a specific rockburst scenario that is to a certain extent predictable.

Similarly, poor hanging wall conditions caused by poor blasting practice or dense jointing (or stress fracturing) make shake-down damage from seismic wave interactions more likely, especially where support installation does not provide sufficient coverage. It is advisable to assess rockburst damage systematically and to collect the relevant records that could be of assistance when identifying vulnerable parts of a mining operation. Where sufficient records have been kept, the following should be established (see rockburst investigations above):

- minimum magnitude that causes rockbursts;
- typical mission sources associated with rockbursts;
- rock conditions at rockburst sites;
- support systems and quality of installation at rockburst sites;
- range of FOG area and fallout thickness;
- mining method, layout and production levels;
- presence of geological features.

BRIDGING GAP

Think of slip or fracture initiation as being a veld fire that started with a single match thrown into dry grass. The local conditions will decide wither it will grow into a major fire of will consume a few bushels of grass d then burn itself out.

In combination, the above factors can assist in describing a local rockburst scenario where it exists thereby making rockbursts increasingly predictable, but also enabling the operation to systematically eradicate scenarios and contributing factors involved in rockbursts. The true value of seismic data interpretation lies in its contribution to understanding rockmass response, not in the short-term predictability of massive, potentially damaging failures.

Where seismic data is evaluated to optimise mine design and where the recommended design strategies result in markedly lower seismic response with regard to either frequency or severity of seismic response compared to the initial study area, we call the use of seismic data pro-active.

Where severe seismic failure has already occurred and the workforce is temporarily withdrawn, blasting is suspended or the area is abandoned, we call the use of seismic data analysis re-active.

LEARNING OUTCOME 6.2.3

SUBSIDENCE MONITORING

The effect of mining on surface subsidence does not necessarily affect the extraction of minerals itself but in terms of surface damage may affect surface structures significantly.

Given sufficient time, any act of removing material from underneath the surface of the earth will result in some form of and some level of subsidence on surface. For example, the extraction of oil from underground reservoirs in California has resulted in subsidence of more than 6 meters in the Long Beach area.

Whether or not the subsidence becomes noticeable depends on factors such as the amount of material removed, the depth at which it takes place, the overburden material and the time after which the subsidence occurs.

back

PAPER 3: CHAPTER 6

In South African mining environment, surface subsidence is a large factor in shallow coal mining, especially where total extraction of the coal seams are practiced, and not so evident in the deeper level Gold and Platinum environments where the overburden is more than 500m and overburden consists of strong, non-yielding rock layers. However, discontinuous subsidence such as 'sinkholes' has been experienced in this environment as a result of the lowering of the water table and cave formation in the Dolomite rocks close to surface.

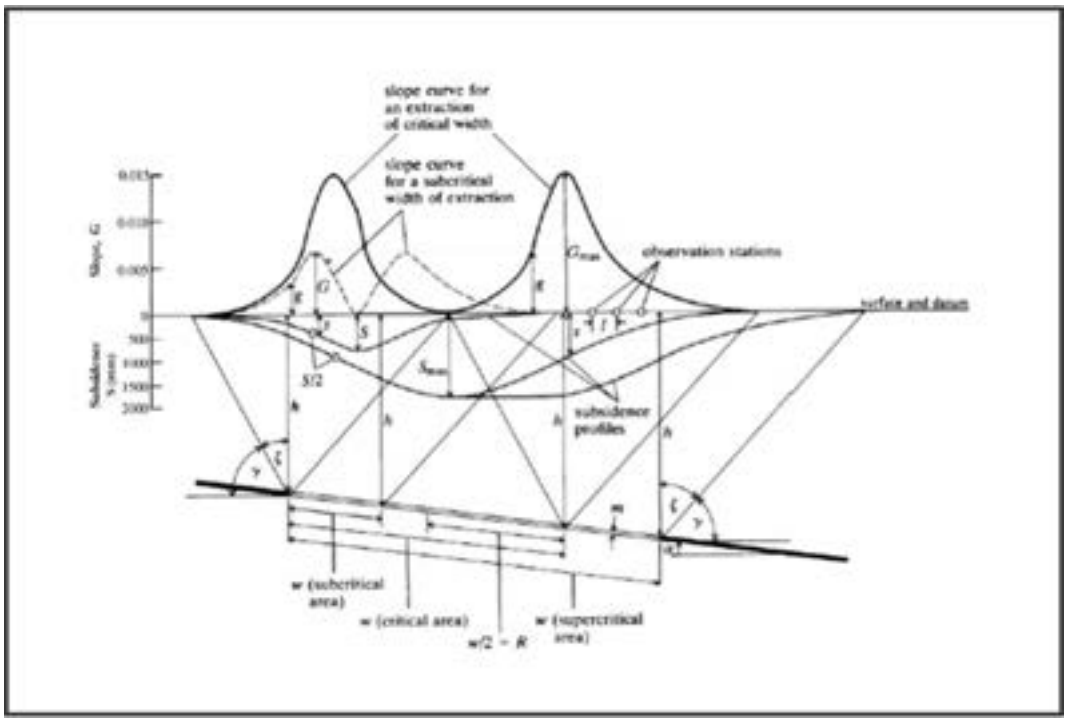


Figure 17: Subsidence nomenclature and symbols (Brady and Brown, 2006)

In Figure 17 above the following is presented:

- subsidence profiles for three panel widths namely wsubcritical ($S < S_{max}$), wcritical ($S = S_{max}$ at a point) and wsuper critical ($S = S_{max}$ over a finite distance);
- s is the vertical displacement or surface subsidence;
- S is the maximum subsidence in a given profile and equals the mining height S_{max};
- Angle of draw (ζ) : angle between the vertical at the edge of the mining and the point on surface where subsidence is zero;
- Depth if seam is h;
- G is the slope of the subsidence profile (Note: this commonly referred to as the 'tilt' of the surface due to subsidence).

REFERENCE

- Page 484, Brady BHG and Brown ET, Rock mechanics for underground mining, 2006
- Page 57, Stacey TR, Best practice rock engineering for other mines, 2001
- Page 113, Vd Merwe JN and Madden BN, Rock engineering for underground coal mining, 2002

LEARNING OUTCOME 6.2.3.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS VARIOUS SUBSIDENCE MODELS

Subsidence types are often divided into two basic types:

Continuous subsidence :
Continuous or trough subsidence is associated with the subsidence of the surface above the mining extraction of a thin, horizontal or flat-dipping orebody such as a coal seam, over a wide area whereby the overlaying rock subsides into the opening created by the mining. The overlaying rock is normally weak, non-brittle sedimentary strata. The resulting surface displacements of surface points may be only of elastic order of magnitude when compared to the dimensions of the subsiding area or the mining depth.

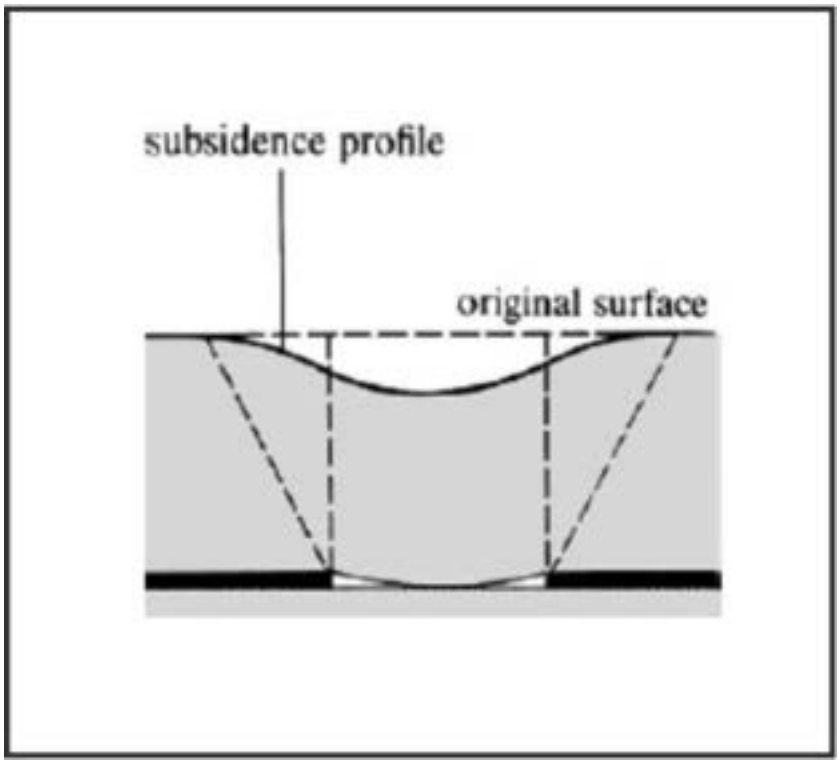


Figure 18: Trough subsidence (Brady and Brown, 2006)

When mining takes place close to surface, subsidence of the surface can cause severe damage to surface structures such as buildings, roads and railway lines. Van der Merwe (1991) classified surface subsidence into classes of severity (Figure 17 and Table 1) using the ratio between the mining height sm and depth H as range indicator.

Class	Sm/H ratio	Description
A	<0.001	Barely noticeable, smooth, continuous profile, hair-line cracks
B	0.00-0.005	Difficult to notice, smooth profile, cracks 1-2 cm wide
C	0.005-0.02	Noticeable in flat terrain, smooth, cracks 2 – 10 cm wide, compression ridges 1 to 5 cm high
D	0.02-0.05	Noticeable in most terrains, visible vertical displacements across cracks, cracks 10 to 50 cm wide, compression ridges 5 to 50 cm high

Class	Sm/H ratio	Description
E	>0.05	Severe profile, almost vertical sides, cracks wider than 50 cm, compression ridges higher than 50 cm

Table 1: Classification of subsidence profiles (vd Merwe, 1991)

Class B 0.001<Sm/H<0.005	
Class C 0.005<Sm/H<0.02	
Class D 0.02<Sm/H<0.05	
Class E Sm/H>0.05	

Figure 19: Examples of different subsidence classes (van der Merwe, 1991)

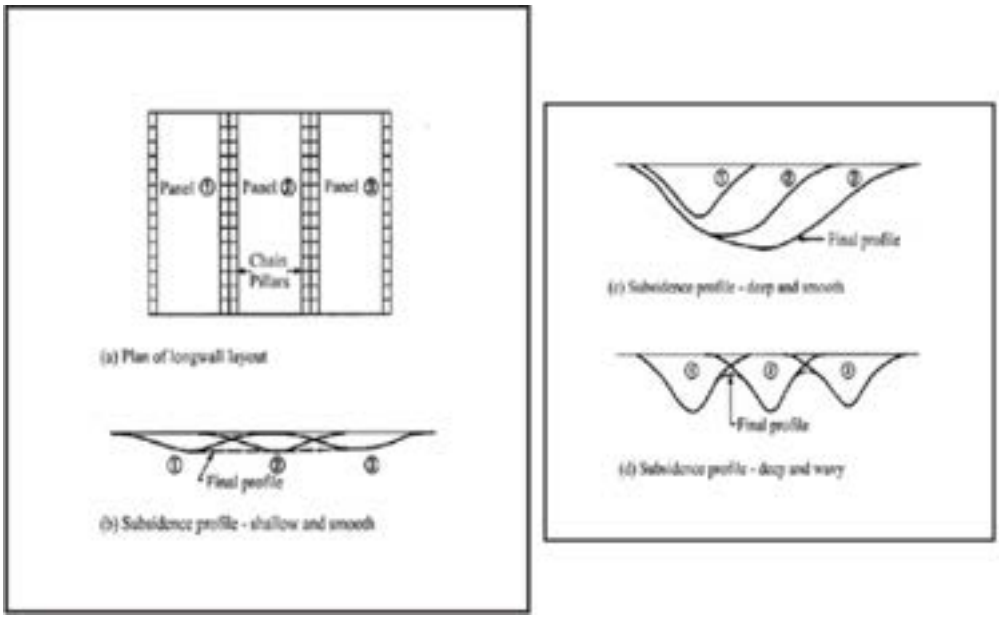
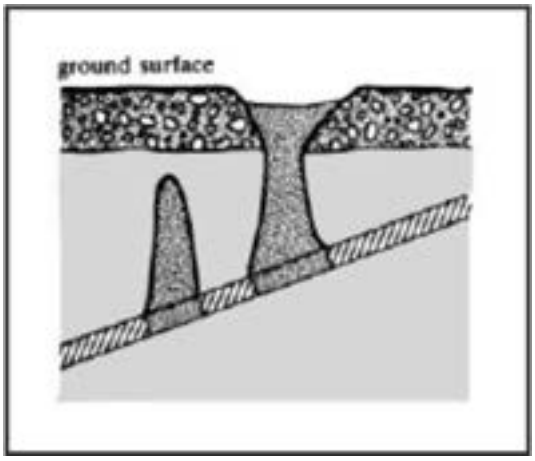


Figure 20: Subsidence profiles over longwalls (Brady and Brown, 2006)

Discontinuous subsidence:

Discontinuous subsidence is characterized by large surface displacements over limited areas and the formation of steps or discontinuities in the surface profile. This subsidence may be associated with a wide variety of mining methods, and are often secondary occurrences such as sinkholes, crown holes or pillar collapse. Chimney caving, piping or funneling occurring where the unsupported mining cavity migrates through the overlying material to surface. The shape of the surface subsidence may assume the original shape of the mined area. In areas where dolomitic rocks have been de-watered through mining activity, large cave formations may take place and is referred to as sinkholes (Carletonville area).

Crown hole	Solution cavities
	

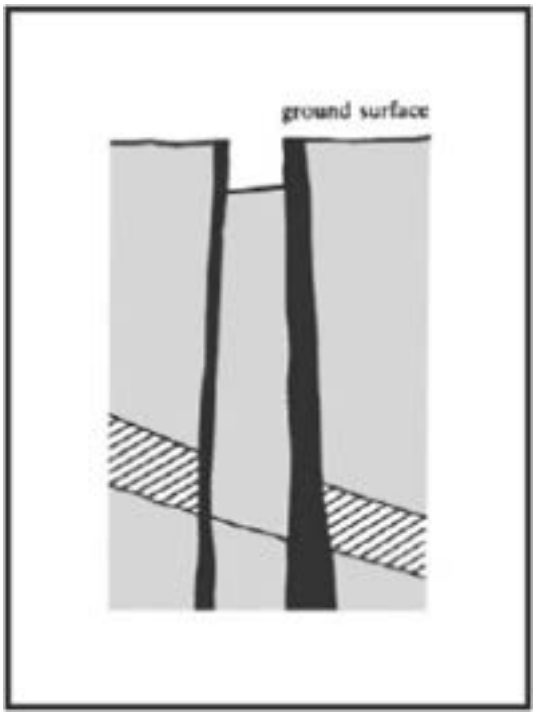
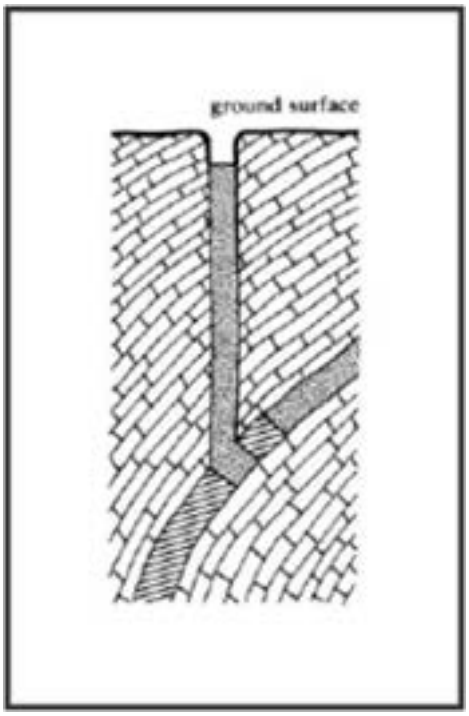
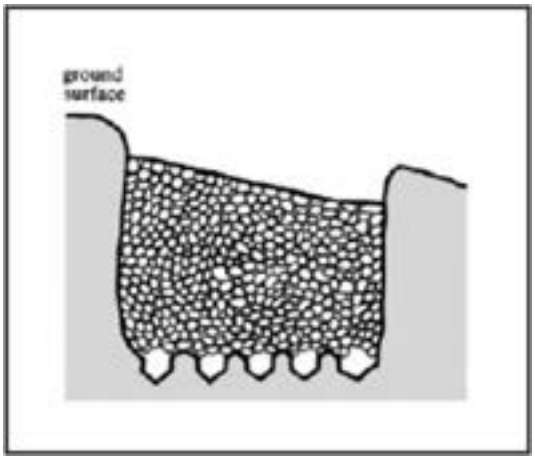
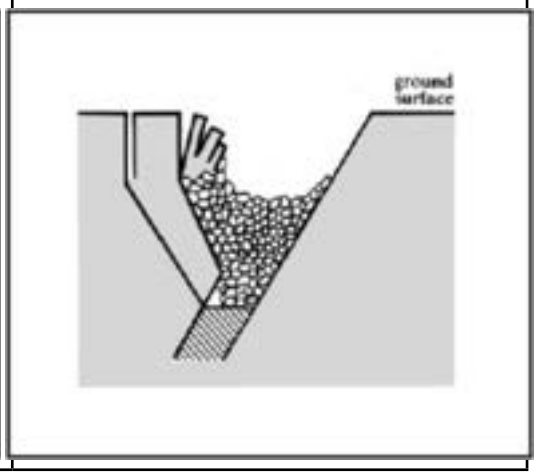
Plug	Chimney
	
Block caving	**Progressive hangingwall caving**
	

Table 2: Discontinuous subsidence types (Brady and Brown, 2006)



- Page 484, Chapter 16, Brady and Brown, Rock Mechanics for Underground Mining, 2006
- Page 113, Vd Merwe JN and Madden BN, Rock engineering for underground coal mining, 2002

LEARNING OUTCOME 6.2.3.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS VARIOUS SUBSIDENCE MODELLING PROCEDURES

Subsidence modeling procedures can be done using:

1. Empirical methods:
   - Van der Merwe (vd Merwe, 2002) provides an empirical based method to estimate the subsidence in areas where a high extraction mining method is used. The maximum vertical subsidence Sm is approximated as a function of the mining depth H, mining height h and panel width W in applying

     $S_m = 0.39h(W/H)^{0.32}$

- For pillar layouts, vd Merwe (2002) suggests the subsidence can be estimated from

  $S_m=0.8h_e$ for depths < 80mbs and
  $S_m=0.5h_e$ to $S_m=0.1h_e$ for depths > 80mbs
  Where h_e = h.e and e = extraction ratio

- Other empirical methods are suggested by Brady and Brown (2006) and although derived for coal mines in the UK, was found to be appropriate in many other cases.

REFERENCE

- Page 508, Chapter 16, Brady and Brown, Rock Mechanics for Underground Mining, 2006
- Page 114, Vd Merwe JN and Madden BN, Rock engineering for underground coal mining, 2002

EXAMPLE

CONNECTION 266

Asadi subsidence model

Asadi's Mathematical subsidence model

This paper was written by Asadi et al and was presented by the SAIMM in November 2004. The aim of the paper was to derive a simple but accurate mathematical model to predict surface subsidence above a coal mine. Even though several prediction methods exist (profile function, influence function, numerical modelling, etc), they are mostly complex and require extensive work and data to allow accurate predictions of surface subsidence. The model derived by the authors assumes that subsidence would occur across the whole tabular orebody and provide the normal subsidence profile while they were able to measure the impact of orebody dip on the subsidence profile.

From Asadi (2004), the following equations were utilised:

Nr	Equation	Comments
1		f,g,p,q are empirical constants, x is measured from point where subsidence is a maximum with updip negative and downdip positive
2		m is the seam thickness, a the subsidence factor and α the orebody dip angle
3		h is the depth on the updip side of the mining, l the mining span, βu the updip angle of draw and βl the downdip angle of draw, while θ is the trough angle
4		
5		c and d are constants with values as follows: Sign(x)=-1 if x<0 Sign(x)= 1 if x>0 Sign(x)= 0 if x=0
6	d=0.5(sign(x)+1)	

Table 3: Equations utilised by Asadi at al (2004)

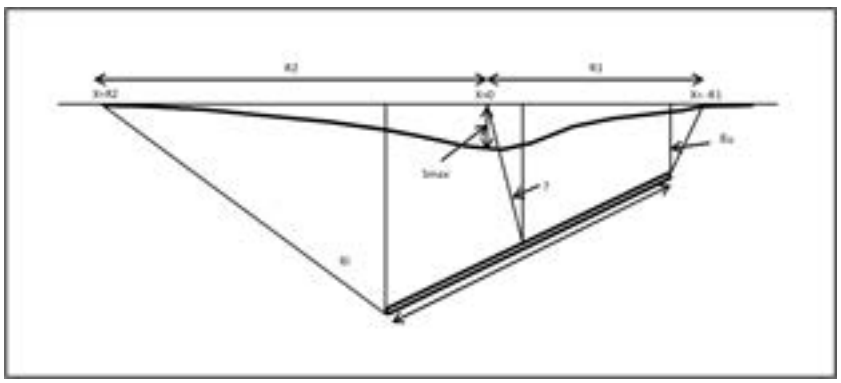


Figure 21: Parameters utilised in the equations

back

EXAMPLE

CONNECTION 267

Subsidence calcs

Also refer to Paper 2 Outcome 6.4

Numerical models: Numerical models of various types can be utilised to estimate the expected subsidence and can vary from elastic programs to approaches where inelastic deformations can be estimated (eg Phase 2, Udec, etc). The accuracy of these models are dependent on the appropriateness of the models to the simulated environment and the input parameters.

LEARNING OUTCOME 6.2.3.3

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS VARIOUS SUBSIDENCE MONITORING PROCEDURES.

It is virtually impossible to detect subsidence of the order of one to two meters over widths of 150m to 200m without the help of measurements. The natural inclination of even a level surface is in the order of 6° to 10° which is more than the inclinations induced by subsidence which are in the order of 2° to 4° for mining depths of 100m or more.

Surface structures can assist in the identification and crude monitoring of progress of or increase in subsidence:

- Where fences (or electric or telephone lines) were erected prior to mining, it is possible to observe the inclination of subsidence when looking along a familiar type of structure.
- Roads, houses and other engineering structures can indicate tilt or misalignment;
- Water accumulation in the veld can indicate areas of subsidence;
- Cracks are easy to detect if they are fresh and the grass has not grown high. Older cracks normally fill up after one or two rain seasons and only the scars are left.

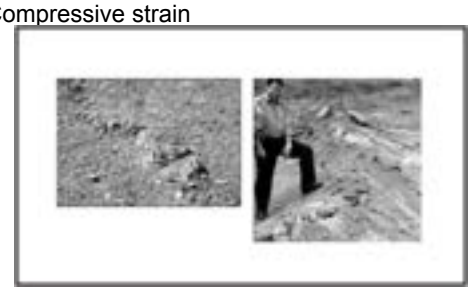
Strain and tilt



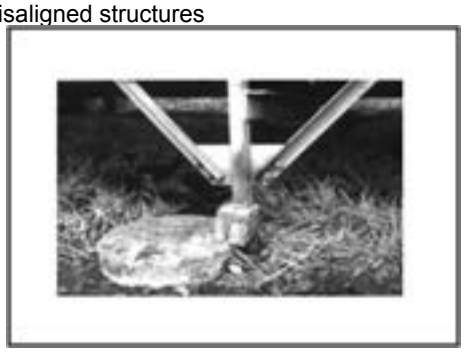
Tension



Compressive strain



Misaligned structures



back

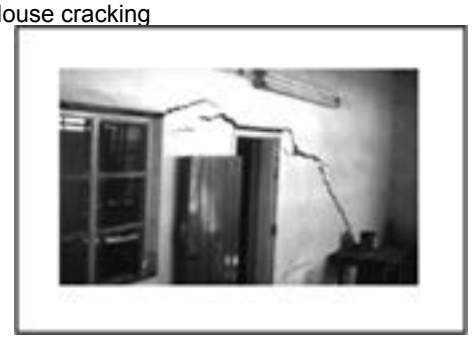
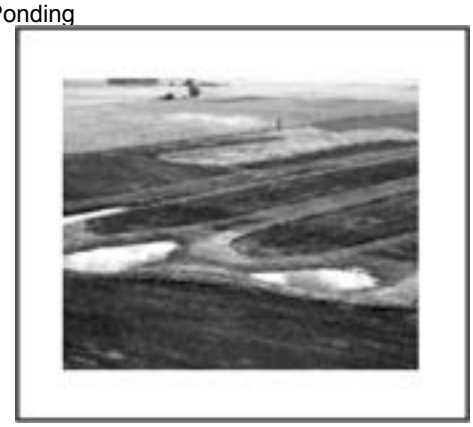
House cracking	Ponding
	

Table 4: Signs of subsidence

Subsidence monitoring may be done for three purposes, namely to check whether goafing is progressing satisfactory, to satisfy statutory requirements or for research. In the latter two cases, proper survey measurements are taken on stations installed prior to mining:

a. For statutory requirements, it is sufficient to determine the amount of subsidence to centimeter accuracies and a small number of measurement stations are normally sufficient.

b. For research purposes, a higher level of precision is required, as it is important to determine both the vertical and horizontal displacements at an accuracy of millimeters or better. A single line of stations is sufficient for statutory requirements, for research at least a double line is necessary. Sometimes no prior knowledge or measurements of subsidence is available as in the case of unexpected pillar failure. It is then necessary to rely on indirect indicators of subsidence such as fences, conveyor belts, electricity and telephone lines.

Monitoring methods include:

- Visual observations (such as surface structures discussed above);
- Surveying of pegs / beacons (using survey theodolites);
- Satellite systems (GPS –global positioning systems)

EXAMPLE

Surface subsidence has been monitored closely and are shown in Figure 22. Measurements received indicated that all measurement stations (eg. CMR 2/2) were static when compared to remote site positions and that differences measured (and showed below) are thus either measurement errors or actual subsidence at the site. From the results shown it is clear that some areas are still subsiding slowly into the mined out workings at a rate of roughly 0.67mm/month (Station CMR 2/2), while other sites appear static (eg. CMR 4/2). These sites will continue to be monitored as mining commences and progresses. The aim of the subsidence monitoring would be to confirm, through measurement, that the impact of mining on surface infrastructure occurs within the acceptable limits.

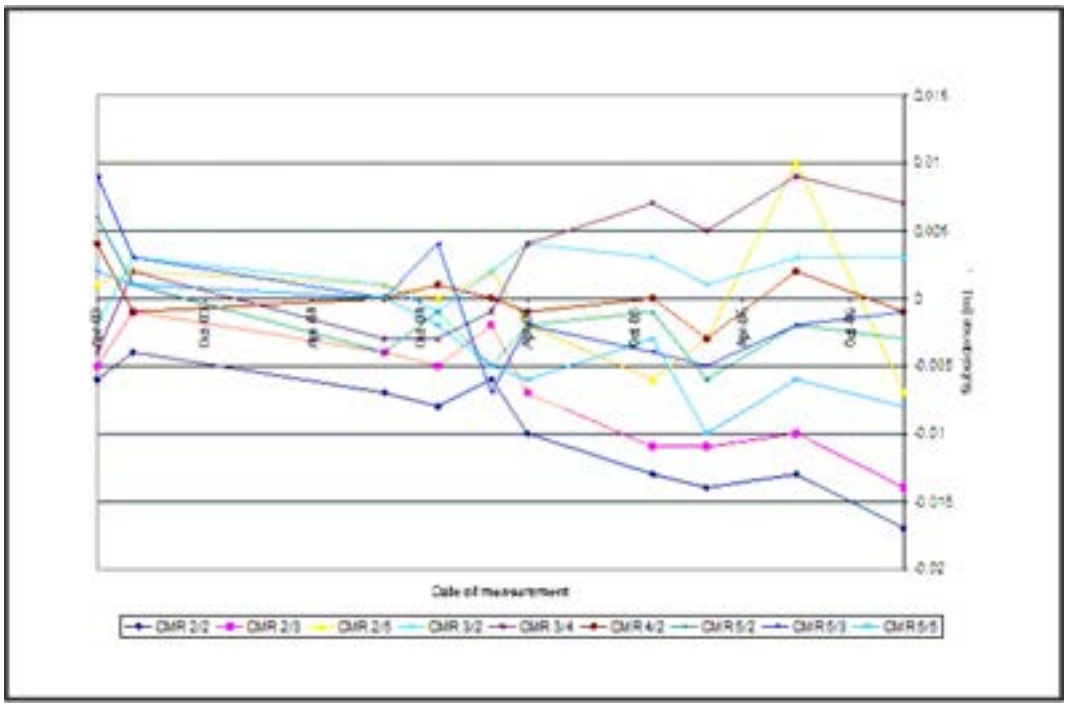


Figure 22: Surface Subsidence

EXAMPLE

'Surface subsidence associated with coal mining activities in the Mpumalanga Province of South Africa changes the natural environment in several ways, and current challenges for mining companies include rehabilitation of the natural environment and the prevention of further degradation.'

CONNECTION 268

Surface subsidence monitoring

'To monitor the spatial and temporal evolution of surface subsidence, traditional field based monitoring approaches such as GPS and spirit leveling are employed at a number of locations. However, the resulting measurements are point-based, and frequent re-visitations are necessary to map the evolution of surface subsidence basins over time.'

'To address these limitations, differential interferograms derived from repeat-pass satellite-borne synthetic aperture radar (SAR) systems were tested for their ability to measure and monitor surface deformation.'

'The SAR data was captured by the European ERS-1 and ERS-2 satellite and covered a timeframe between 2008- 09-15 and 2008-09-15. The resulting interferograms revealed several features indicative of surface subsidence. Ground truth data confirmed the presence of a subsidence basin, detected using differential interferometry techniques during the 35-day period between April 12, 2008 and May 17, 2008; a maximum vertical deformation of 3.2 cm (1.3 in) was recorded.'

'Interferometric monitoring revealed an eastward migration of the subsidence basin between June 2, 2008 and September 15, 2008 with an additional 4.7 cm (1.9 in) of subsidence observed. This migration coincides with the advance of the working face of the mine during this period.'

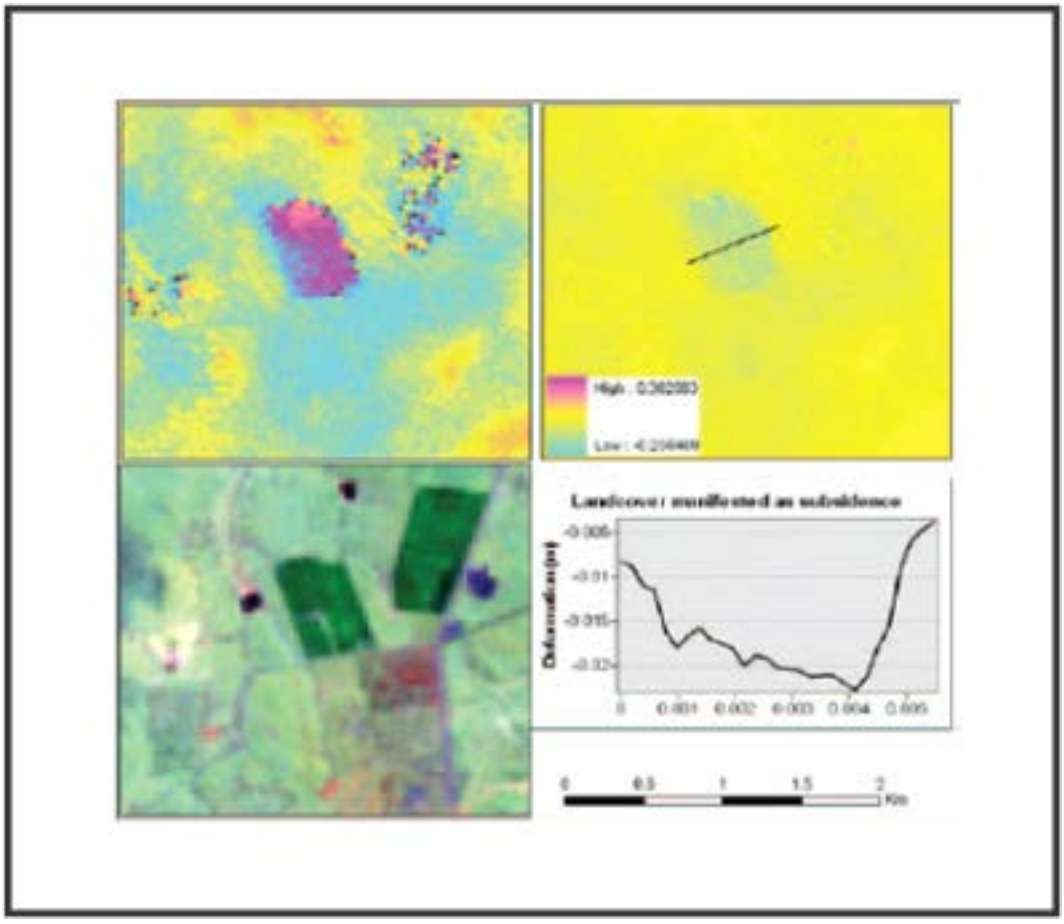


Figure 23: (a) Interferogram, (b) Deformation map in metres, (c) Landsat image, (d) Profile graph showing deformation (Surface subsidence, Makusha)

EXAMPLE

'Mining subsidence is so common that in order to ensure the safety of surface transportation and local residents, current monitoring programs in many mines have included GPS surveys, conventional precise levelling and theodolite surveys, EDM surveys, and remote electronic monitoring (Hebblewhite et al, 2000). These techniques are costly and inefficient compared with remote sensing techniques such as that afforded by Synthetic Aperture Radar Interferometry (InSAR).'

'However, InSAR results may not be accurate enough for this application due to the uncertainties in the SAR satellite orbit and the signal disturbances due to the atmosphere. GPS surveys can be used to account for these biases.'

'The Satellite Navigation And Positioning (SNAP) group at The University of New South Wales, with its strength in GPS deformation monitoring, and the Radar Interferometry Group at Stanford University, with its strength in InSAR data processing, have developed a joint project to monitor ground subsidence due to underground mining, by combining the techniques of InSAR and GPS. In order to test the combined technique, both radar corner reflectors and GPS receivers are collocated in the mining region whenever the SAR satellite (ERS-2) passes over.'
'The pattern of ground subsidence due to mining, determined from the analysis of such measures across a GPS network, is an important input to models that seek to explain the mechanisms for such deformation, and hopefully to mitigate the damage to society caused by such (slow or fast) ground movements. However, continuous GPS monitoring has generally been considered relatively expensive for many localised ground

subsidence monitoring applications.'

'InSAR is a technique first suggested in 1974 and, after more than two decades of development, is now well developed with many applications in mapping topography, and topographic change determination following earthquakes (Massonnet et al, 1993). Although InSAR for ground subsidence monitoring has been a research topic in recent years, it has not become an operational technique because of the presence of biases that seriously degrade the accuracy of InSAR-only results (Goldstein, 1995; Zebker et al, 1997).'

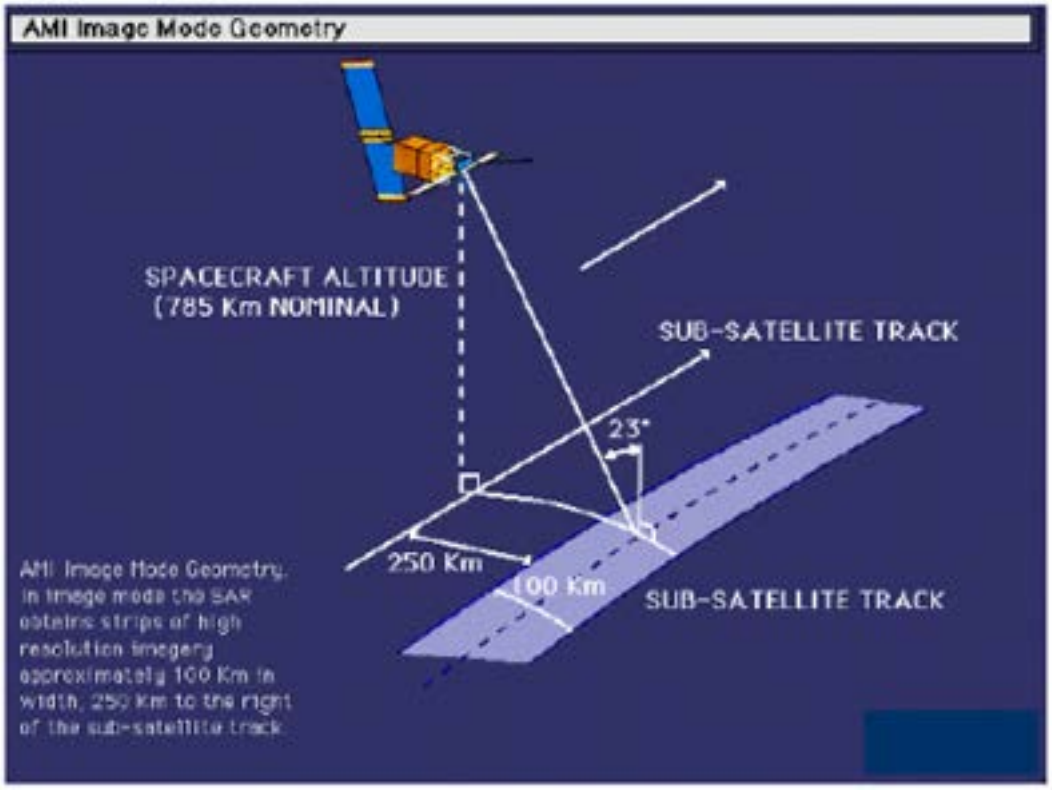


Figure 24: maging geometry of ERS satellites (Surface subsidence monitoring, Ge)

Stacey and other derived subsidence limits for surface structures after doing significant investigations into subsidence in the central Johannesburg area. Table 5 through Table 8 5 gves the categories and limits derived through that process.

Surface structure	Category 1	Category 2	Category 3
Buildings	Brick and masonry structures with brick load bearing walls	Steel and reinforced concrete frame structures	Massive structures of considerable rigidity, including those of central core design
Roads	Low volume gravel roads and paved roads	High volume paved roads with kerbs and substantial storm water drainage facilities	
Railway lines	Goods lines and low volume passenger lines on which speed limits can easily be imposed	Main lines and high volume passenger lines where it would be impractical to impose speed limits	
Pipes	Secondary sewers, water supply pipelines and stormwater drains and all steel pipelines carrying non-hazardous materials	Main sewers, water supply pipelines, stormwater drains and pipes carrying hazardous materials	

Table 5: Surface infrastructure subsidence limits

The following suggested limits of ground movements in the South African context is given by Stacey and Bakker (1992) and is shown below in Tables 6-2 through 6-5.

Building category	Damage level	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	Superficial	0.5	1.0
1	Hazardous	2.0	3.0
2	Superficial	2.0	1.5
2	Hazardous	4.0	3.5
3	Superficial	1.0	3.5
3	Hazardous	3.0	1.5

Table 6: Building categories

Road category	Damage level	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	Minor	2.0	10.0
1	Severe	5.0	15.0
2	Minor	1.0	5.0
2	Severe	2.0	10.0

Table 7: Road categories

Railway line category	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	3.0	10.0
2	2.0	5.0

Table 8: Railway line categories

Pipeline category	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	2.0	4.0
2	1.0	2.0

Table 9: Pipeline categories

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- Stacey, TR, Stabilising undermined sites: Case studies illustrate the sophistication of modern techniques, Journal. Geotechnical Engineering, SA Construction World – March 1983.
- Stacey TR, Interaction of underground mining and surface development in a central city environment.
- Stacey TR and Bakker D, The erection or construction of buildings and other structures on undermined ground, 1992
- Page 114, Vd Merwe JN and Madden BN, Rock engineering for underground coal mining, 2002

6.3. MODELLING
6.3.1. NUMERICAL MODELLING

LEARNING OUTCOME 6.3.1.1

DESCRIBE, EXPLAIN AND DISCUSS THE DIFFERENCES BETWEEN PHYSICAL, EMPIRICAL AND NUMERICAL MODELS

CONNECTION 269
Refer to Paper 2 Outcome 6.4 from page 497

LEARNING OUTCOME 6.3.1.2

DESCRIBE, EXPLAIN AND DISCUSS THE LIMITATIONS OF PHYSICAL, EMPIRICAL AND NUMERICAL MODELS

CONNECTION 270
Refer to Paper 2 Outcome 6.4 from page 497

LEARNING OUTCOME 6.3.1.3

DESCRIBE, EXPLAIN AND DISCUSS THE SELECTION OF APPROPRIATE MODELS AND CODES TO TACKLE VARIOUS PROBLEMS

CONNECTION 271
Refer to Paper 2 Outcome 6.4 from page 497

LEARNING OUTCOME 6.3.1.4

DESCRIBE, EXPLAIN AND DISCUSS THE INPUT OF APPROPRIATE PARAMETERS TO INVESTIGATE VARIOUS PROBLEMS

CONNECTION 272
Refer to Paper 2 Outcome 6.4 from page 497

LEARNING OUTCOME 6.3.1.5

DESCRIBE, EXPLAIN AND DISCUSS THE INTERPRETATION OF OUTPUT IN THE INVESTIGATION OF VARIOUS PROBLEMS

CONNECTION 273
Refer to Paper 2 Outcome 6.4 from page 497

LEARNING OUTCOME 6.3.1.6

DESCRIBE, EXPLAIN AND DISCUSS THE DIFFERENCE BETWEEN RELATIVE AND ABSOLUTE RESULTS FROM NUMERICAL MODELS

CONNECTION 274
Refer to Paper 2 Outcome 6.4 from page 497

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CONNECTION 275

Refer to Paper 2 Outcome 6.4 from page 497

LEARNING OUTCOME 6.3.1.7

DESCRIBE, EXPLAIN AND DISCUSS THE APPROPRIATE USE OF RELATIVE AND ABSOLUTE RESULTS FROM NUMERICAL MODELS

LEARNING OUTCOME 6.3.1.8

CONNECTION 276

Refer to Paper 2 Outcome 6.4 from page 497

DESCRIBE AND EXPLAIN THE RELEVANCE OF CALIBRATION, CORRELATION AND BACK ANALYSIS IN INTERPRETING ABSOLUTE RESULTS FROM NUMERICAL MODELS

LEARNING OUTCOME 6.3.1.9

CONNECTION 277

Refer to Paper 2 Outcome 6.4 from page 497

DESCRIBE AND EXPLAIN THE RELEVANCE OF CALIBRATION, CORRELATION AND BACK ANALYSIS IN INTERPRETING RELATIVE RESULTS FROM NUMERICAL MODELS.

6.4. AUDITING

LEARNING OUTCOME 6.4.1

CONNECTION 278

Refer to Refer to Paper 2 Outcome 6.2.3

DESCRIBE, EXPLAIN AND DISCUSS THE CONCEPT OF MONITORING FOR UNDERSTANDING, PREDICTION AND DESIGN.

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CHAPTER 7

PAPER 3.1 HARD ROCK TABULAR MINING

7. ROCKBREAKING IN HARD ROCK

LEARNING OUTCOMES

7.1. CUTTING TECHNIQUES

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

- Describe, explain and discuss geotechnical aspects associated with various
- Non-explosive rock breaking procedures that include :
  - Expanding grout techniques
  - Block sawing
  - Reef cutting, Reef boring
  - Tunnel boring, Raise boring
- Describe, explain and discuss the methodologies and applications of these techniques
- List and discuss the advantages and disadvantages of these techniques.

7.2. DRILLING TECHNIQUES

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

- Sketch, describe, explain and discuss the different rounds used in shaft sinking
- Describe, explain and discuss the different cuts used in shaft sinking
- Describe, explain and discuss the types of initiation used in the above rounds
- Describe, explain and discuss the sequence of initiation of blast holes used in the above rounds
- Describe, explain and discuss the optimum explosive types and their properties for use in the above rounds
- Sketch, describe, explain and discuss the different rounds used in tunnel development
- Describe, explain and discuss the different cuts used in tunnel development
- Describe, explain and discuss the types of initiation used in the above rounds
- Describe, explain and discuss the sequence of initiation of blast holes used in the above rounds
- Describe, explain and discuss the optimum explosive types and their properties for use in the above rounds
- Sketch, describe, explain and discuss blast hole layouts in stopes
- Describe, explain and discuss the direction of drilling of blast holes in stopes

- Describe, explain and discuss the sequence of initiation of blast holes in stopes
- Describe, explain and discuss the optimum explosive types and their properties for use in the above rounds
- Describe, explain and discuss the importance of blast-hole drilling accuracy in the following applications:
- Shaft sinking, Chamber excavation, Orepass excavation, Tunnel development,
- Stope advance
- Cushion blasting, Smooth blasting.

7.3. BLASTING PRACTICES

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

- Sketch, describe, explain and discuss the chemical reactions associated with various explosives used in mining
- Sketch, describe, explain and discuss the physical manifestations associated with various explosives used in mining
- Sketch, describe, explain and discuss the basic differences between explosives used in mining
- 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 objectives and effects of de-coupling explosives
- Describe, explain and discuss the methods by which de-coupling of explosives is achieved
- Describe, explain and discuss the following excavation cushion blasting and smooth blasting techniques :
  - Pre-splitting
  - Concurrent smooth blasting
  - Post-splitting
- Describe, explain and discuss the methodologies and typical applications of each technique
- List and discuss the advantages and disadvantages of these techniques
- Evaluate and determine blasting requirements for tunnels making use of knowledge of explosives
- Evaluate and determine appropriate blasting rounds to suit given conditions in tunnels
- Evaluate and determine appropriate explosive types to suit given conditions in tunnels
- Evaluate and determine blasting requirements for stopes making use of knowledge of explosives
- Evaluate and determine appropriate blasting rounds to suit given conditions in stopes
- Evaluate and determine appropriate explosive types to suit given conditions in stopes.

7.1. CUTTING TECHNIQUES

LEARNING OUTCOME 7.1.1

DESCRIBE, EXPLAIN AND DISCUSS GEOTECHNICAL ASPECTS ASSOCIATED WITH VARIOUS NON-EXPLOSIVE ROCK BREAKING PROCEDURES THAT INCLUDE :

- Expanding grout techniques,
- Block sawing,
- Reef cutting,
- Reef boring,
- Tunnel boring,
- Raise boring

Non-explosive Breaking Techniques

The breaking of rock without using explosives makes use of either increasing the stress on the rock and thus fracturing the rock through exceeding the strength, or by cutting the rock mechanically. Non-explosive mining means that the mining operation can be conducted on a continuous basis, with no delays for the removal of blasting fumes.

Expanding Grout Techniques

Although this technique has not been utilized in the underground environment on a large scale, it can be used for breaking up of large blocks of rock where the use of normal explosives is not viable.

The procedure implies the drilling of holes in the rock (similar to normal blasting), and filling the holes with a specialized expanding grout mixture. This grout expands when drying through a chemical reaction and the expansion creates stress on the rock, thus breaking the rock up into fragments (Figure 1 and Figure 2)



Figure 1: Large rock showing drill pattern for applying non-explosive agent in breaking the rock (Bustar technologies)

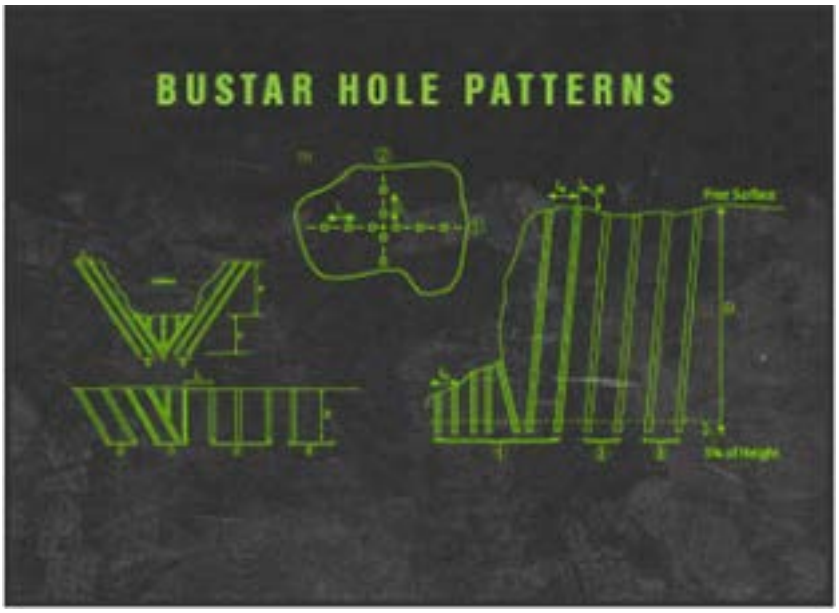


Figure 2: Drilling pattern for non-explosive agent (Bustar Technologies)



- Frag ag brochure Courtesy of CEP, cep@netactive.co.za
- http://www.chimicaedile.com
- http://crackag.en.alibaba.com/product/457461204-212251142/Fract_AG_Expansive_Mortar.html

Block Sawing

This application is mainly used in the granite production industry, where large sawing machines are used to cut blocks of particular size and shape. The granite is then processed into smaller forms for special applications. Figure 3 is an example of a block sawing machine.

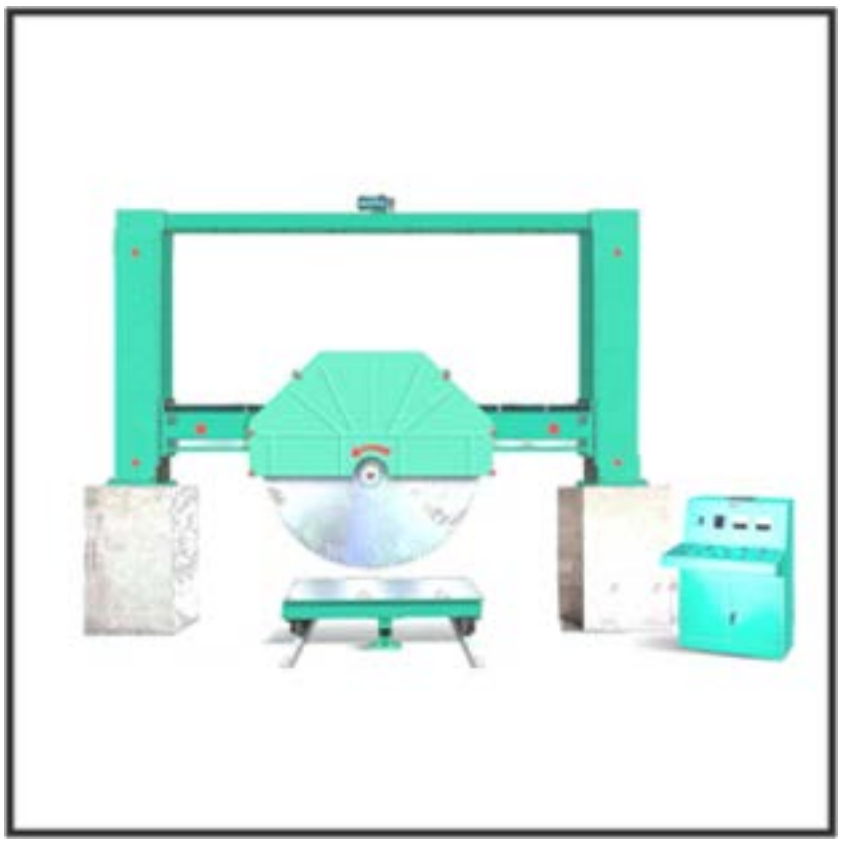


Figure 3: Photograph of a block sawing machine

Reef cutting and reef boring methods

Rock cutting has been used successfully to extract ore in the soft rock environment for a long period of time. In the hard rock environment, however, the success of the cutting of the reef with machines has been limited to a few cases. The principle is close to that of tunnel boring, but the design is for a narrow, nearly horizontal reef band (Merensky and UG2 in the above example).

REFERENCE

- Pickering rock cutting narrow reefs

Tunnel boring and raise boring

CONNECTION 279

Refer to Paper 2 Outcome 7.2.4 on page 558

A tunnel boring machine (TBM) also known as a "mole", is a machine used to excavate tunnels with a circular cross section through a variety of soil and rock strata. They can bore through anything from hard rock to sand. Tunnel diameters can range from a metre (done with micro-TBMs) to 19.25 m to date. Tunnels of less than a metre in diameter are typically done using trenchless construction methods or horizontal directional drilling rather than TBMs.

Tunnel boring machines are used as an alternative to drilling and blasting (D&B) methods in rock and conventional "hand mining" in soil. TBMs have the advantages of limiting the disturbance to the surrounding ground and producing a smooth tunnel wall. This significantly reduces the cost of lining the tunnel, and makes them suitable to use in heavily urbanized areas. The major disadvantage is the upfront cost. TBMs are expensive to construct, and can be difficult to transport. However, as modern tunnels become longer, the cost of tunnel boring machines versus drill and blast is actually less—this is because tunneling with TBMs is much more efficient and results in a shorter project.

INTERESTING INFO

The largest diameter TBM, at 19.25 m, was built by Herrenknecht AG for a recent project in Orlovski Tunnel, St.Petersburg. The machine was built to bore through soft ground including sand and clay. The largest diameter hard rock TBM, at 14.4 m, was manufactured by The Robbins Company for Canada's Niagara Tunnel Project. The machine was used to bore a hydroelectric tunnel beneath Niagara Falls, the machine has been named "Big Becky" in reference to the Sir Adam Beck hydroelectric dams to which it is tunnelling to provide an additional hydroelectric tunnel.



Figure 4: Cutting shield used for the New Elbe Tunnel.



Figure 5: Top view of a model of the TBM used on the Gotthard Base Tunnel.

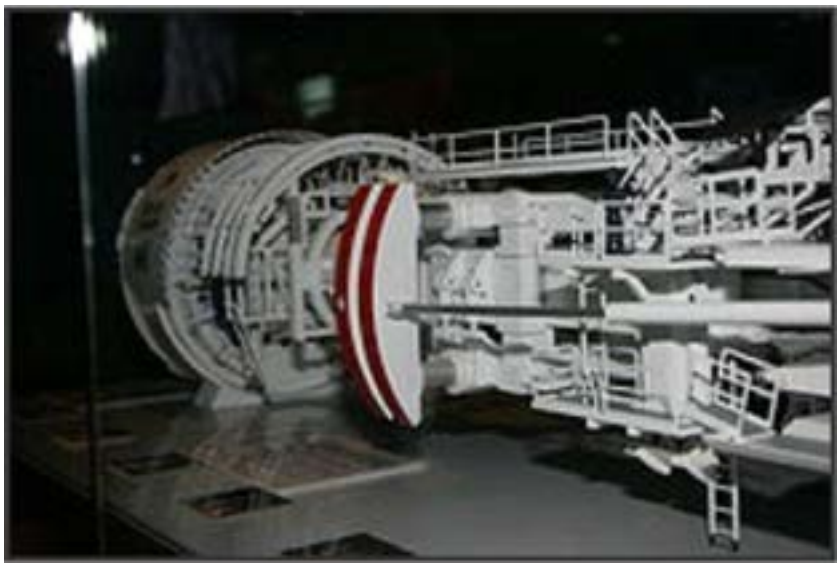


Figure 6: Looking towards the cutting shield at the hydraulic jacks.

back



Figure 7: Hydraulic jacks holding a TBM in place.



Figure 8: The support structures at the rear of a TBM. This machine was used to excavate the main tunnel of the Yucca Mountain nuclear waste repository in Nevada.

back



Figure 9: Tunnel boring machine at the site of Weinberg tunnell Altstetten-Zürich-Oerlikon near Zürich Oerlikon train station.

INTERESTING INFO

The first successful tunnelling shield was developed by Sir Marc Isambard Brunel to excavate the Thames Tunnel in 1825. However, this was only the invention of the shield concept and did not involve the construction of a complete tunnel boring machine, the digging still having to be accomplished by the then standard excavation methods.

The first boring machine reported to have been built was Henri-Joseph Maus' Mountain Slicer. Commissioned by the King of Sardinia in 1845 to dig the Fréjus Rail Tunnel between France and Italy through the Alps, Maus had it built in 1846 in an arms factory near Turin. It consisted of more than 100 percussion drills mounted in the front of a locomotive-sized machine, mechanically power-driven from the entrance of the tunnel. The Revolutions of 1848 affected the funding, and the tunnel was not completed until 10 years later, by using innovative but less expensive methods such as pneumatic drills.

In the United States, the first boring machine to have been built was used in 1853 during the construction of the Hoosac Tunnel. Made of cast iron, it was known as Wilson's Patented Stone-Cutting Machine, after inventor Charles Wilson It drilled 10 feet into the rock before breaking down. The tunnel was eventually completed more than 20 years later, and as with the Fréjus Rail Tunnel, by using less ambitious methods.

Description

Modern TBMs typically consist of the rotating cutting wheel, called a cutter head, followed by a main bearing, a thrust system and trailing support mechanisms. The type of machine used depends on the particular geology of the project, the amount of ground water present and other factors.

Hard rock TBMs

In hard rock, either shielded or open-type TBMs can be used. All types of hard rock TBMs excavate rock using disc cutters mounted in the cutter head. The disc cutters create compressive stress fractures in the rock, causing it to chip away from the rock in front of the machine, called the tunnel face. The excavated rock, known as muck, is transferred through openings in the cutter head to a belt conveyor, where it runs through the machine to a system of conveyors or muck cars for removal from the tunnel.

Open-type TBMs have no shield, leaving the area behind the cutter head open for rock support. To advance, the machine uses a gripper system that pushes against the side walls of the tunnel. Not all machines can be continuously steered while gripper shoes push on the side-walls, as in the case of a Wirth machine which will only steer while ungripped. The machine will then push forward off the grippers gaining thrust, At the end of a stroke, the rear legs of the machine are lowered, the grippers and propel cylinders are retracted. The retraction of the propel cylinders repositions the gripper assembly for the next boring cycle. The grippers

back

are extended, the rear legs lifted, and boring begins again. The open-type, or Main Beam, TBM does not install concrete segments behind it as other machines do. Instead, the rock is held up using ground support methods such as ring beams, rock bolts, shotcrete, steel straps, Ring steel and wire mesh.

In fractured rock, shielded hard rock TBMs can be used, which erect concrete segments to support unstable tunnel walls behind the machine. Double Shield TBMs have two modes; in stable ground they can grip against the tunnel walls to advance. In unstable, fractured ground, the thrust is shifted to thrust cylinders that push off against the tunnel segments behind the machine. This keeps the significant thrust forces from impacting fragile tunnel walls. Single Shield TBMs operate in the same way, but are used only in fractured ground, as they can only push off against the concrete segments.

Raise Boring

CONNECTION 280
Refer to Paper 2 Outcome 7.2.4 on page 558

Modes of Operation

Raise borers can be used in various modes of operation, the modes most often used are:

- Conventional pilot drilling
- Conventional up reaming of vertical and inclined holes
- Down boring with a pre-drilled pilot hole
- Blind up boring
- Directional pilot and raise boring used in conjunction with a shaft boring machine (V-mole), for the drilling and supporting of a large diameter shaft.

Conventional Pilot Drilling

A tri-cone pilot bit is normally used varying from 229 mm to 381 mm. The 381 mm bit is normally used on long holes with a 327 mm integral drillsteel string with 253 mm DI 42 tool joints. During drilling fluid is pumped through the center of the drillstring to the cutting face, where the rock cuttings are flushed and raised from the bottom of the hole through the annulus around the drillstring to the collar of the hole. The drilling fluid is settled in a closed loop via a series of settling dams so that the drilling water can be re-used. (See Figure 10)

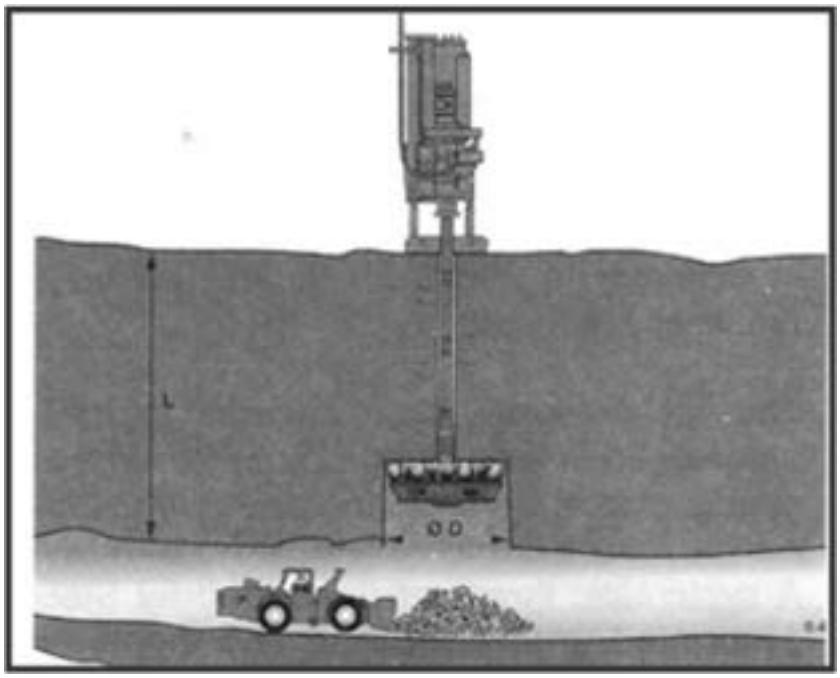


Figure 10: Sketch showing conventional up-reaming of pilot hole

Conventional up-reaming of Pilot Holes

On completion of pilot hole drilling and at such time that the pilot hole breaks through into the lower excavation, a reaming head is fitted to the end of the drillstring. The size of the reaming heads ranges between 1,2 m and 6,1 m in diameter. The head is rotated by the machine and is pulled back against the rock face at the same time. Tungsten Carbide insert cutters are fitted to the head and these cut grooves in the rock in a rotary crushing mode. The 'kerfs' of rock between the grooves 'spall' out and rock failure occurs in tensile mode. The rock cuttings fall to the bottom of the hole where it is mucked out by a mechanical loader.

It is a safe, efficient and cost-effective method of making holes through different geological formations with the use of powerful machines, high strength drillstring and reliable heads. The maximum loading capacity of the drillstring limits the diameter as well as length of the shaft. The loading is dynamic and only approximately calculable because tensile, torsional and bending stresses are overlapping. (Figure 10).

Down Boring with a Pre-drilled Pilot Hole

In this case an oversize pilot hole is drilled. The cutting head is installed at the top of the pilot hole and drilling takes place in the downward mode. Rock cuttings are flushed down the oversized pilot hole to the bottom of the hole where it is removed. In the case of smaller holes, the machine provides cutter thrust and in the case of large diameter shafts the cutter head is weighted through the addition of steel collars. The down boring method is not used often as the risk of blocking the pilot hole and creating mud rushes at the bottom of the hole is too high. (Figure 11).

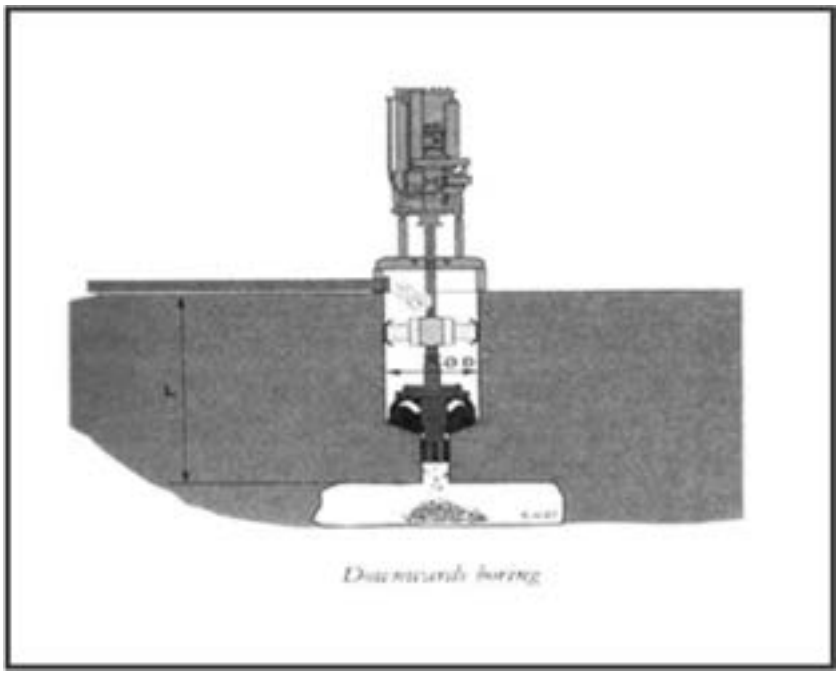


Figure 11: Sketch showing the down-reaming of a pilot hole

LEARNING OUTCOME 7.1.2

CONNECTION 281

"LEARNING OUTCOME 7.1.1" on page 455

DESCRIBE, EXPLAIN AND DISCUSS THE METHODOLOGIES AND APPLICATIONS OF THESE TECHNIQUES

LEARNING OUTCOME 7.1.3

LIST AND DISCUSS THE ADVANTAGES AND DISADVANTAGES OF THESE TECHNIQUES.

Technique	Advantages	Disadvantages
Expanding grout technique	No blasting fumes – work can continue directly after breaking Non-violent Accurate breaking No damage to surrounding rock	Limited application in very hard rock Expensive
Block sawing	Accurate Fast and effective	Limited application for cutting of granite and marble blocks
Reef cutting/reef boring	Limited damage to surrounding rock mass Accurate Narrow band cutting Fast and safe	Limited success in hard rock Not flexible in uneven reef horizon Expensive
Tunnel boring/raise boring	Limited damage to surrounding rock mass Accurate and fast in softer rock Can be done continuously – no blasting	Expensive in very hard rock Time consuming installation

Table 1: Advantages and disadvantages

7.2. DRILLING TECHNIQUES

LEARNING OUTCOME 7.2.1

CONNECTION 282

Refer to Paper 2, Outcomes 7.1 and 7.2

INTERESTING INFO

Shaft sinking history paper

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE DIFFERENT ROUNDS USED IN SHAFT SINKING

In mining, shafts form a system of vertically or inclined passageways, which are used for transportation of ore, refill, personnel, equipment, air, electricity, ventilation etc.

An important requirement in shaft sinking is to provide optimum fragmentation of the rock so that it can be cleared quickly from the congested shaft-face area.

Blasting operation is carried out against gravity, and the scatter of the broken rock is confined in the shaft. It is common to use generous distribution of explosives throughout the rock using a large number of small diameter (35 – 42 mm) shotholes.

The number of holes N required for sinking a shaft of cross sectional area A in m^2 is given by:

$$N = 2.5A + 22$$

The drilling patterns for shaft sinking are basically the same as those used in tunneling but generally the cone cut is favoured (Figure 12 and Figure 13). Another commonly used pattern, unique to shaft sinking, is the bench cut (Figure 14 and Figure 15)

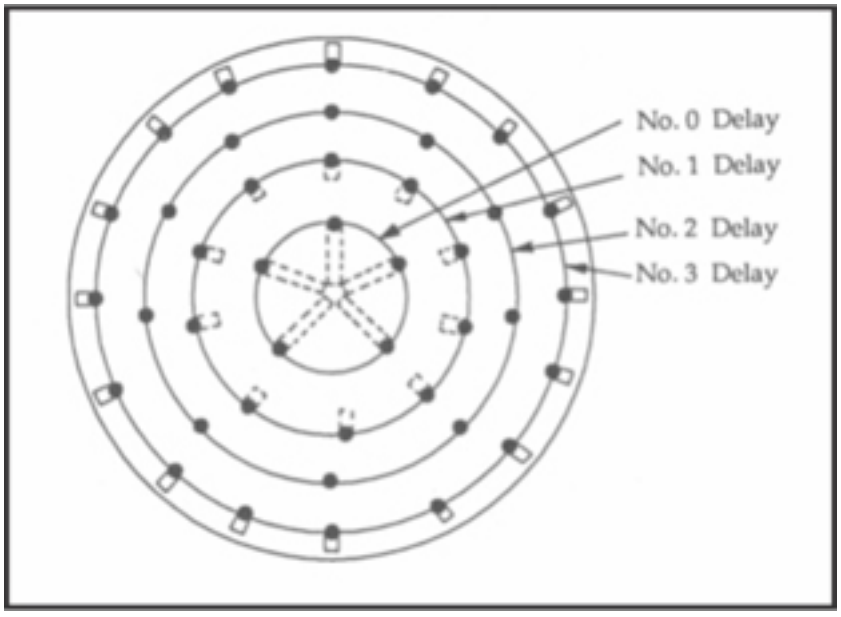


Figure 12: Full-face blasthole pattern for shaft sinking.

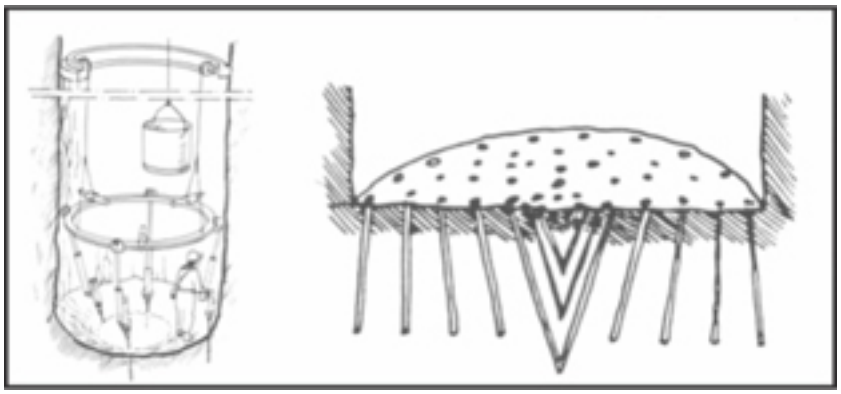


Figure 13: Shaft sinking with cone or pyramid cut.

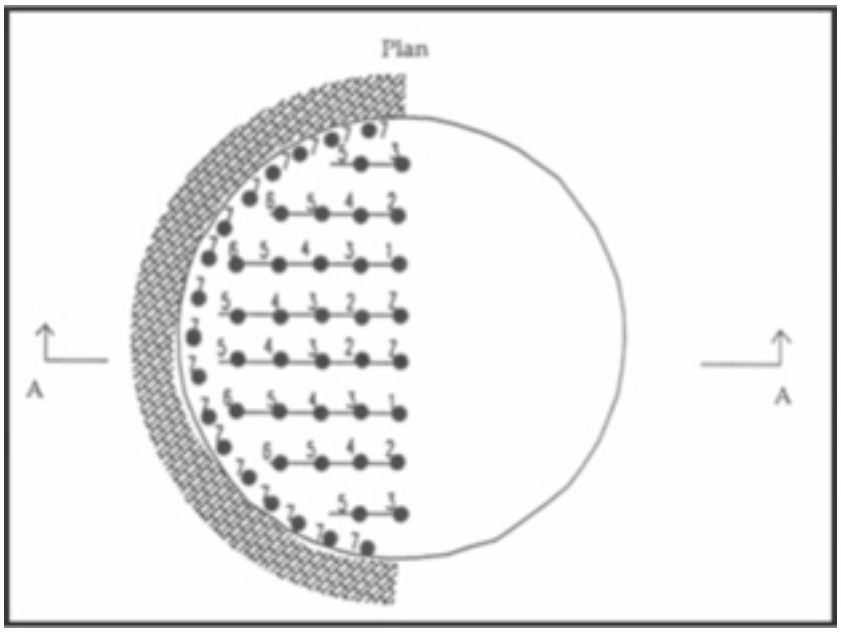


Figure 14: A bench-cut blasthole for shaft sinking.

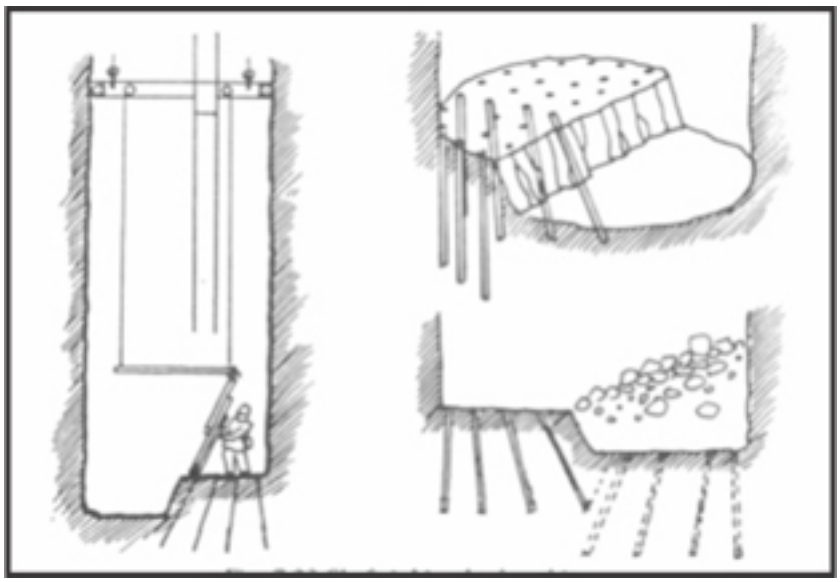


Figure 15: Shaft sinking by benching.

The explosives used in shaft sinking must always be water resistant. Even if the ground is dry, the flushing water from the drilling machines will always stay in the blastholes. Suitable explosives such as Emulex 150 or any suitable NG based explosives are easily tamped to utilize the

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hole volume well. The powder factor in shaft sinking is rather high, ranging from 2.0 kg/m^3 to 4.0 kg/m^3.

If charges are fired electrically, great care must be taken in wiring the circuit. Since more than 100 detonators can be involved in each blast, they are connected either in parallel or in series-parallel. It is therefore important to ensure that the resistance of the circuit is properly balanced and that no charged hole is omitted from the circuit. Due to higher risk of such errors in the generally unfavourable shaft sinking environment, Nonel type detonators are increasingly preferred for initiation.

LEARNING OUTCOME 7.2.2

CONNECTION 283

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE DIFFERENT CUTS USED IN SHAFT SINKING

Cut design

As when driving drifts in development headings, proper design of the cut is important in order to ensure a successful pull of full face circular shaft rounds. The holes in the cut area must be perfectly parallel when using only one reamer hole; if the first two cut holes to be blasted do not destroy the circular geometry of the single reamer hole along its entire length, the round will begin to crater out. It should be noted, however, that there is slightly more tolerance in drill hole deviation when using four 3 or 4 inch diameter reamer holes in the shaft cut designs. Again, experience has taught the shaft sinking industry that when using multiple small diameter reamer holes, it is more efficient to position the cut's first blast hole directly in their center, thus increasing the chances of pulling a perfect cut.

As shown in Figure 12 and Figure 13 above, the v-shape cut is the most popular when blasting a full cut of the shaft diameter. The alternative is the bench cut, as shown in Figure 14 and Figure 15 above.

LEARNING OUTCOME 7.2.3

CONNECTION 284

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE TYPES OF INITIATION USED IN THE ABOVE ROUNDS

A number of initiating systems are currently available on the market. AEL, a supplier of explosives and initiating systems in South Africa, has the following systems available:

a. Shock Tube initiating

   A thin tube with a thin layer of explosives on the inner wall which is ignited and sends a shock wave through the tube initiating the explosives. This is a very fast initiating method, and needs to be timed carefully.

b. Capped fuse systems

   The traditional fuse delay system which has been in use for a long time. A fuse is placed in the explosives, and lit with some form of initiation such as igniter cord.

c. Electronic delay detonators

   A small electronic fuse that can be set for a certain delay in order to time the shots to go off in the correct sequence. The detonator is initiated to set off the explosives.

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d. Electric initiators

An electrical impulse is sent through a cable to initiate the detonator and thus set off the explosives. This method s applied where centralized blasting is used.

e. Igniter cord

A high intensity explosives based wire cord which burns intensely, and with which a fuse can be initiated to ignite explosives. It is also a method of timing the round sequence.

LEARNING OUTCOME 7.2.4

CONNECTION 285

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE SEQUENCE OF INITIATION OF BLAST HOLES USED IN THE ABOVE ROUNDS

Initiations systems
Initiation is the process of detonating explosives. So far as initiation is concerned, careful selection of an initiation system and good training are important to eliminate the possibility of perimeter holes being fired before inner holes. In particular, capped fuse/igniter cord system should be avoided for long rounds (>2,4m), as it is difficult to obtain sufficient assurance of sequential firing under these conditions. Nonel Tunnelmasters, Magnades or electric detonators are much better suited, as timing intervals are independent of hole length.

The most important requirement of an initiating system in blasting excavations are that holes fire in sequence and that no cut-offs of the system occur. Three main systems, each with specific advantages and disadvantages exist:

Fuse and igniter cord
In this system, the oldest and cheapest, a flame front, started with a flame starter "cheesa" stick. Flame travels via the igniter cord to each fuse where it in turn initiates the detonation of the explosives. The greatest disadvantage is the lack of reliable sequential detonation.

Electric detonators
In this arrangement electric current initiates the detonator. The system is relatively expensive and requires a slightly higher level of training. Sequential firing is easily achieved.

Nonel systems
These techniques use a system called "shock tubing", where a narrow diameter hollow plastic tubing is lined with a dusting of high explosives. This explosive dust, when initiated, propagates a high speed shock pulse through the trunk line to linked detonators. Relatively good sequential firing can be achieved, in the millisecond range, but the cost is relatively high.

Figure 12 and Figure 15 above shows the sequencing of the different rounds used in shaft sinking.



In the pyramid round, the cut is timed to go off first in order to create a free breaking surface for the remainder of the blast to break into. With the bench cut, there is a free surface, and the round is sequenced to break into the bench.

LEARNING OUTCOME 7.2.5

CONNECTION 286

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIMUM EXPLOSIVE TYPES AND THEIR PROPERTIES FOR USE IN THE ABOVE ROUNDS

Basic types of explosives

Explosives can be divided broadly into low and high explosives. The classic example of a low explosive being gunpowder. This discussion will however be confined to high explosives as these are commonly used in hard rock applications.

Three basic types of explosives can be identified all of which are characterized by differing qualities in terms of shock/gas energy, safety and cost:

1. Nitro-glycerin based explosives

These are highly adaptable cartridge explosives, which have wide use in the industry. In general these explosives have shock energy lower than water gels or emulsions but higher than ANFO. These explosives have high water resistance. Characteristic problems are 'Powder headaches" and excessive blasting fumes.

2. ANFO (Low VOD)

A low cost, high power, relatively safe pourable explosive made from porous prilled ammonium nitrate and fuel oil. It has no water resistance and produces large amounts of gas energy. Due to different loading densities, consistent loading of holes is often difficult and overcharging may occur. ANFO is a suitable explosive for use in a wide range of rock mass conditions.

3. Water–based ammonium nitrates (water gels and emulsions)

These are essentially ANFO's made water resistant by adding water and forming either gel or stable oil/water emulsion. Emulsions tend to have the highest shock energies and velocities of detonation but produce little gas energy. Powergel, a high VOD explosive, delivers more shock wave energy and the remaining gasses are at lower pressure.

Surface Bulk Products, Surface Bulk Emulsions, Bulk Anfex® Eco Bulk Anfex Eco Porous Prill Ammonium Nitrate and recycled fuel oil blasting agent for use in commercial blasting operations.  Bulk Anfex for surface applications	The prill characteristics are optimized to provide excellent resistance to friability. The prills are mixed in situ with recycled fuel oil in Mobile Manufacturing Units (MMUs) or Anfo Loaders. Appearance: Bulk Anfex Eco comprises prills that are grey in colour. Application: Open pit, quarrying
Underground Bulk Emulsions, UG100 Range Underground bulk emulsion explosives consist of a base emulsion and sensitizer, which are classified as non-detonable and non-explosive, respectively. UG100 is used in development and stoping whereas UG100S is used primarily in up-hole/ring blasting but can also be used in development. The suffix 'S' in UG100S infers the product is 'sticky', which means it can effectively stick to the lining of blast holes, making it suitable for up-holes. The suffix 'C' in UG100 C infers the product is customized as per specific site/customer requirement/s.	These products are characterized as high strength denoted by the 100 in the name. AEL manufactures all the necessary mobile charging equipment for these products. Appearance: The base emulsion is a honey coloured mixture of two immiscible liquids and an emulsifier. The sensitizer is an aqueous or emulsion product, which, when added to the base emulsion and mixed, creates an explosive in the blast-hole. Application: Narrow reef and massive orebody tunnelling, stoping, ring blasting

UG200 Range	Underground bulk emulsion explosives consist of a base emulsion and sensitizer, which are classified as non-detonable and non-explosive, respectively. UG200 is used in development whereas UG200S is used primarily in up-hole/ring blasting but can also be used in development. The suffix 'S' in UG200S infers the product is 'sticky', which means it can effectively stick to the lining of blast holes, making it suitable for up-holes. The suffix 'C' infers the product is customized on site to customer requirements. These products are characterized as high stability and suitable for use in remote areas and/or where large temperature gradients are anticipated denoted by the 200 in the name. AEL manufactures all the necessary mobile charging equipment for these products. Appearance: The base emulsion is a honey coloured mixture of two immiscible liquids and an emulsifier. The sensitiser is an aqueous product, which, when added to the base emulsion and mixed, creates an explosive in the blast-hole Application: Narrow reef and massive orebody tunnelling, stoping, ring blasting, Coal mining
Packaged Explosives ANfex Prill Based Anfex®  Anfex packaged in bag	Anfex® is a free flowing Ammonium Nitrate and fuel oil blasting agent for use in commercial blasting operations. The prill characteristics are optimised to provide excellent resistance to friability yet loads exceptionally well in pneumatic loaders. Anfex® fuel ratios are carefully controlled and provide the optimum chemical balance for good post detonation fume characteristics. The explosive is conveniently packed into 6.25kg inner plastic bags. Appearance: The product is packaged in a 25kg plastic bag, which contains four 6.25kg "multibags". The prills are white to off-white in colour and may appear grey if recycled oil is used for fuelling. Application: Underground and surface mines. Tunneling, traneching, open pit, civils.
Magnum Buster Gel 	Magnum™ Buster Gel PE Buster gel is a medium energy jelly-like, water gel explosive that is used in commercial blasting operations. The product is used extensively in quarrying and trenching where the energy must be tailored to achieve optimum blasting results. The explosive is packed into plastic sleeves of variable dimensions. It is an extremely popular water gel explosive and used in a wide variety of applications Appearance: The product is packaged in printed sausage-shaped cartridges with a jelly-like consistency. The explosive appears grey to silver in color Application: Narrow reef, open pit. Tunnelling, stoping, open pit, quarying.

Table 2: Commercial explosives available on the market

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INTERESTING INFO

BME is a supplier of explosives to the South African mining industry. The

information given below was borrowed courtesy of BME (http://www.bme.co.za)

HEFTM
(High Energy Fuel) emulsion is classified as a non-explosive oxidizer in its un-gassed state and can be stored for up to three months in field tanks. This means that you can safely store HEF until you need it and you can have immediate access to adequate quantities at all times.

ANFO
is an oxygen-balanced mixture of porous prill ammonium nitrate (PPAN) and fuel oil. It is soluble in water and insensitive to normal handling or an open flame. PANFOTM has excellent heave and good fragmentation characteristics..

Underground Pumpable Emulsion MegapumpTM
is a gas-sensitized waterproof explosive that is carried underground in bulk containers and pumped into blastholes in a similar fashion to the charging of holes in opencast mines but with specially designed underground ECU's (emulsion charging units),. The non-explosive emulsion is transported to the mine in standard chemical tankers, stored in silos or tanks at the shaft head and underground and is sensitized and pumped into the blastholes by the ECU's.

MegapumpTM is a light gold in color and has a high viscosity that allows it to be pumped into up-holes.The density of MegapumpTM can be changed on site to increase or decrease the relative bulk strength.

MegapumpTM is stored in surface silos and transported underground in cassettes that vary in size depending on the operational needs.

Once it has been sensitized during the charging process, MegapumpTM is cap sensitive and can be initiated using an 8D strength detonator. It is advisable to use a 15 g booster for efficient initiation.

MegapumpTM is classified as an oxidising agent and transportation, storage and handling must be in accordance with the appropriate regulations. MegapumpTM can be transported in tankers to site and stored in silos on surface or underground. Magazine facilities are not necessary.

Bagged ANFO
PanexTM is an oxygen-balanced mixture of porous prill ammonium nitrate (PPAN) and fuel oil. It is soluble in water and insensitive to normal handling or an open flame. PanexTM has excellent heave and good fragmentation characteristics. It is supplied in bulk or 25 kg bags. In bags, you have the choice of receiving it with or without four smaller internal bags of 6.25 kg each.

Cartridged Explosives
MegamiteTM is a high-energy cap sensitive emulsion explosive suitable for use in tough breaking conditions in hard rock mining environments. MegamiteTM is water resistant, safe to handle and has excellent performance characteristics.

Electronic Delay Detonators
AXXISTM is a fully programmable, accurate and easy-to-use digital initiation system. It is one of the safest initiation systems available. Using the AXXISTM System, you can program AXXISTM detonators to fire accurately at any time between 0 and 10000 ms at one millisecond intervals. You can fire up to 600 detonators from one AXXISTM Blasting Box. Five AXXISTM Blasting Boxes can be linked together or remotely fired a total of 3600 AXXISTMdetonators.

The AXXISTM System comprises:

- AXXISTM digital initiation system with Intelligent Connectors (IC's)
- AXXISTM passive logger to establish Connector ID and firing time for each detonator
- AXXISTM blasting box to programme and fire the detonators. Blasting can be done with a blasting wire or by wireless link. Long lead-in lines are thus eliminated.
- AXXISTM low-energy line leakage tester is used for safely and quickly finding leakage points in a network of detonators
- AxiSoftTM PC-based software for high-level blast and timing design

There are other commercial products on the market, and can be researched by referencing the following websites:
AEL explosives http://www.explosives.co.za.
OMNIA explosives http:// www.omnia.co.za
Example of explosive types: Attached file on explosives from AEL.

Explosive characteristics

Different explosives partition their available energy into shock and gas energies in different ways. For example, an explosive with a low-velocity of detonation (VOD) tends to deliver more 'gas energy' and less 'shock wave energy'. This can be detrimental in rocks which contain open joints enabling the gases to penetrate far beyond the intended perimeter causing blocks of rock to be loosened.

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Conversely, a high VOD explosive, such as Powergel, delivers a larger portion of its energy as 'shock energy' and the remaining gases are at a lower pressure. The result is less damage caused by gas penetrating into cracks, and more fragmentation of the intended body of rock.

High shock energy explosive	Low shock energy explosive
Short cracks	Long cracks
Softening of ground near blast hole walls	Little softening of blast hole wall
Less penetration by gas	Gas penetration of joints and extending cracks

Table 3: Comparison between high and low shock energy explosives

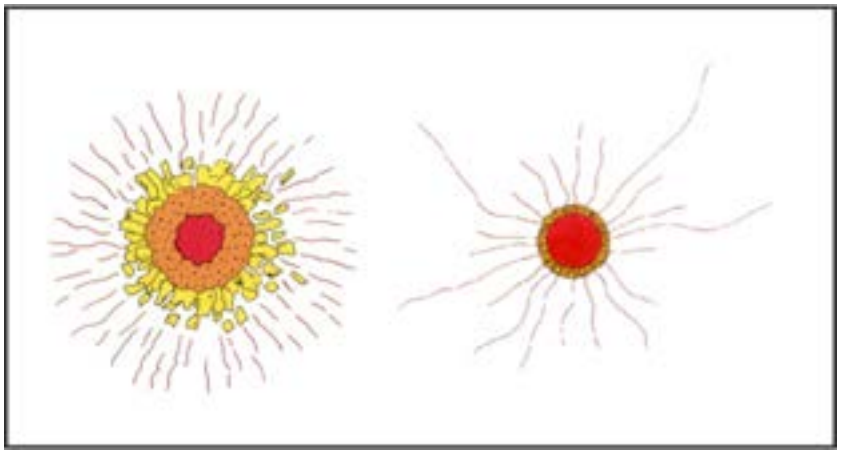


Figure 16: Effect of energy partitioning on fracture development

This difference is illustrated in Figure 16, which shows a cross-section of two blast holes. The hole (a) that contained a high VOD explosive, caused crushing around the blasthole but shorter cracks. The hole (b) that contained the low VOD explosive, has less crushing near the blast-hole wall and long cracks.

The effect of shock energy and gas pressure on rock fragmentation
The high pressure gasses, having initiated the initial stress wave in the rock, continue to expand the blast hole until equilibrium exists between the gasses and the containing rock.

The energy released up to this point, known as the Borehole Equilibrium State, is defined as SHOCK ENERGY, while the subsequent energy release is defined as HEAVE or GAS ENERGY.

Following the EQUILIBRIUM STATE, a relatively slow but major fragmentation process occurs as the high-pressure gases penetrate every available opening, forcing movement towards the point of least resistance. Cracks are lengthened and joints opened until pressure is released at a free face; if the free face is too distant then the gases are violently expelled from the collar of the hole.

It can thus be concluded that the greater the burden on a hole:

- The longer will be the back–cracking away from the free face,
- The further the gases will penetrate any jointing in the rock mass, and
- The greater will be concussion in the excavation, blasting out support damaging ventilation columns and shaking down relaxed rock from the side and hanging wall.

SHOCK ENERGY pulverizes the blasthole wall and weakens the nearby rock mass by introducing fine cracks;

GAS ENERGY is largely responsible for ripping out loosely jointed blocks, extending cracks into solid rock, and producing concussive effects in the existing underground space.

LEARNING OUTCOME 7.2.6

CONNECTION 287

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE DIFFERENT ROUNDS USED IN TUNNEL DEVELOPMENT

Various drilling patterns have been developed for blasting solid rock faces, such as:

- wedge cut or V cut
- pyramid or diamond cut
- drag cut
- fan cut
- burn cut

Wedge cut
Blast holes are drilled at an angle to the face in a uniform wedge formation so that the axis of symmetry is at the centre line of the face. The cut displaces a wedge of rock out of the face in the initial blast and this wedge is widened to the full width of the drift in subsequent blasts, each blast being fired with detonators of suitable delay time.

The apex angle is as near as possible to 60° (Figure 17). This type of cut is particularly suited to large size drifts, which have well laminated or fissured rocks. Hole placement should be carefully preplanned and the alignment of each hole should be accurately drilled.

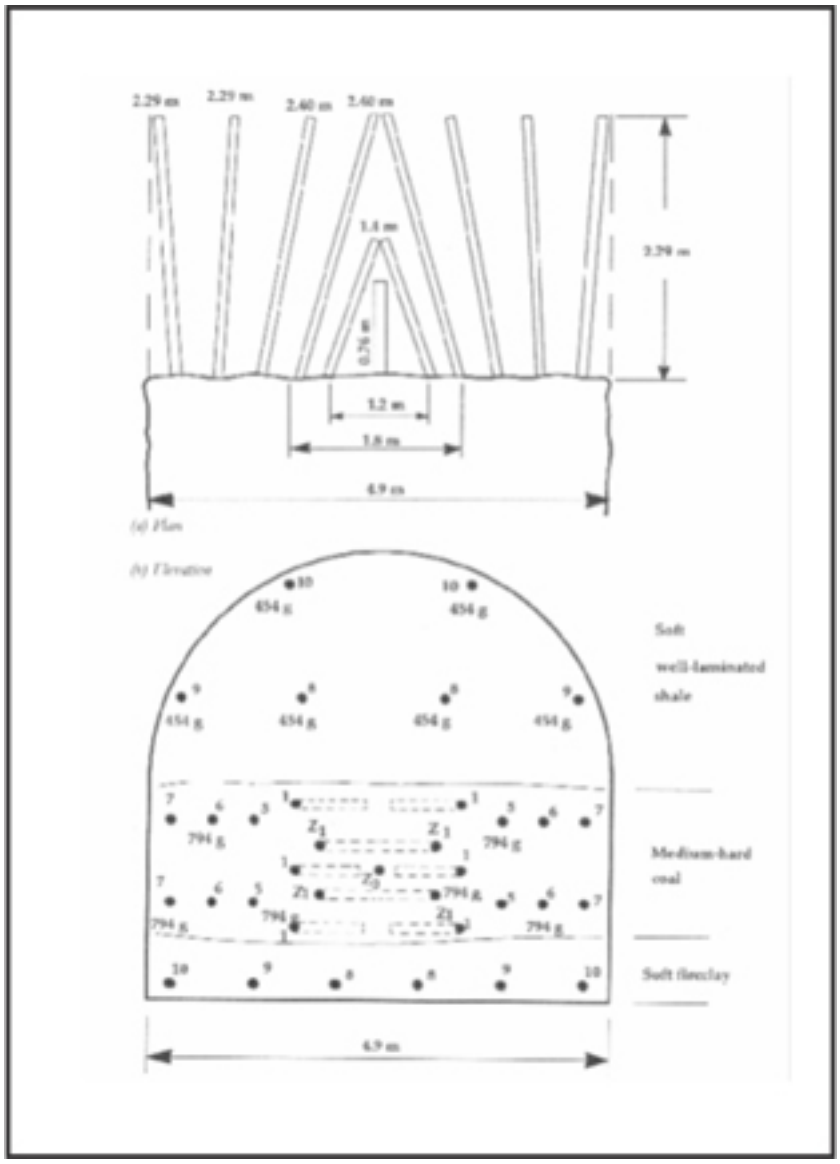


Figure 17: Wedge cut (after ICI)

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Pyramid or diamond cut

The pyramid or diamond cut is a variation of the wedge cut where the blastholes for the initial cavity may have a line of symmetry along horizontal axis as well as the vertical axis (Figure 18).

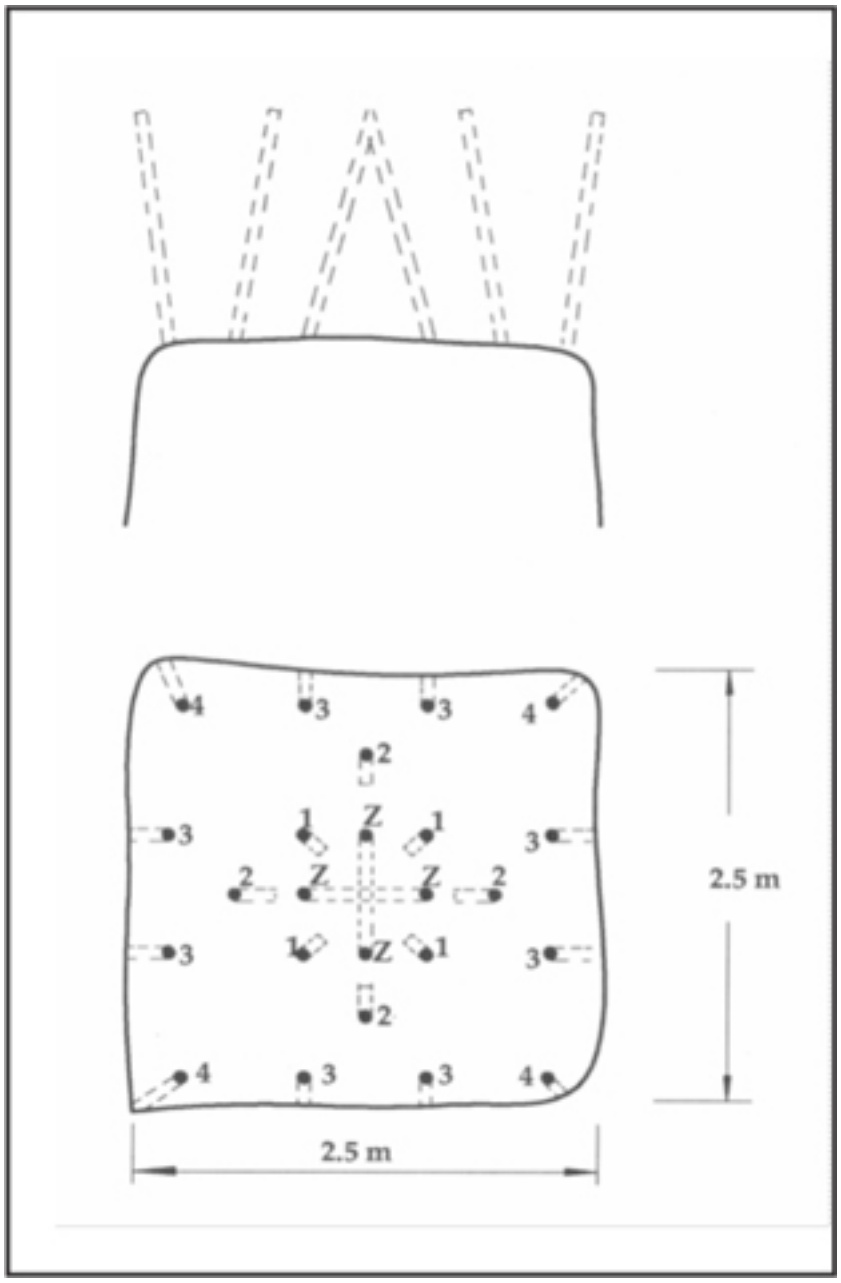


Figure 18: Pyramid or diamond cut (after ICI)

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Drag cut

The drag cut is particularly suitable in small sectional drifts where a pull of up to 1 m is very useful (Figure 19).

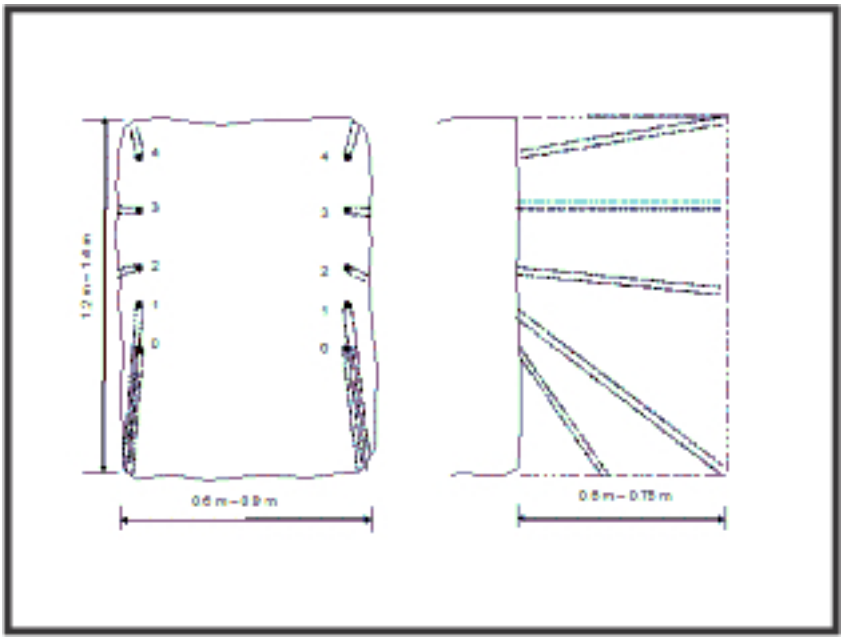


Figure 19: Drag cut

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Fan cut

The fan cut is one-half of a wedge cut and is applicable mainly where only one machine is employed in a narrow drive. Generally the depth of pull obtainable is limited to 1.5 m (Figure 20).

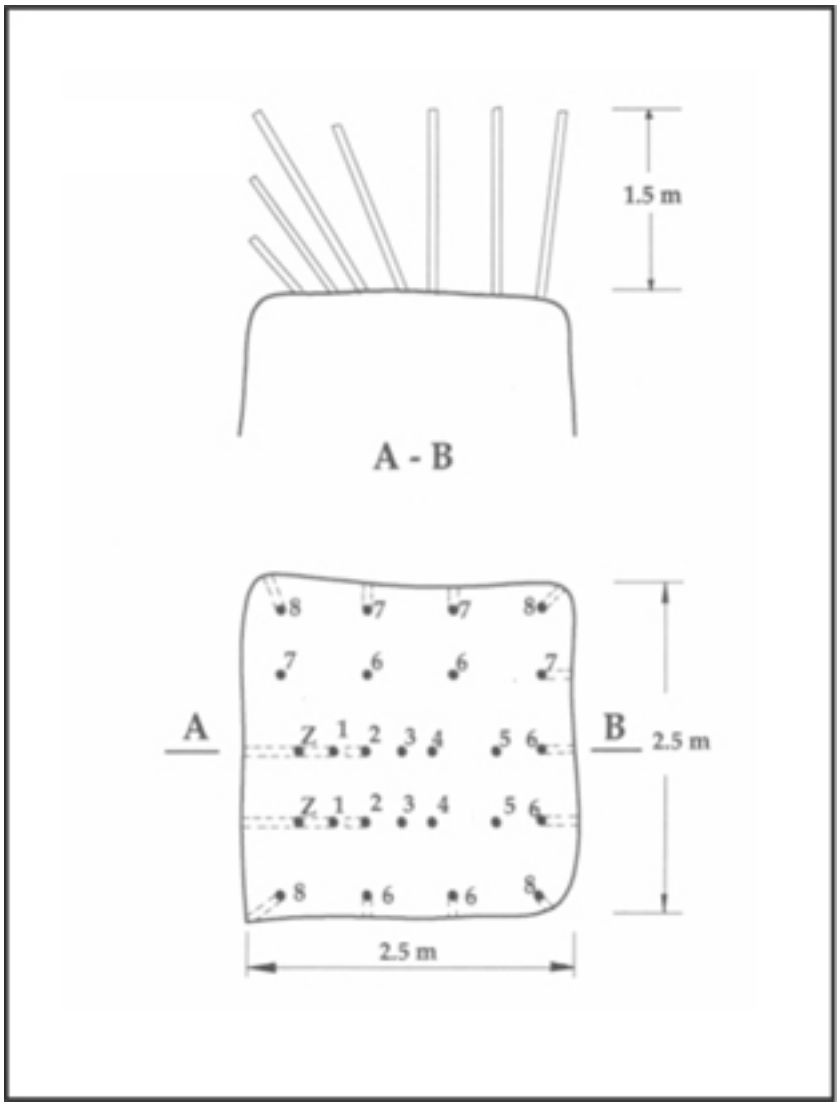


Figure 20: Fan cut

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Burn cut

A series of parallel holes are drilled closely spaced at right angles to the face. One hole or more at the centre of the face are uncharged. This is called the burn cut (Figure 21).

The uncharged holes are often of larger diameter than the charged holes and form zones of weakness that assist the adjacent charged holes in breaking out the ground. Since all holes are at right angles to the face, hole placement and alignment are easier than in other types of cuts. The burn cut is particularly suitable for use in massive rock such as granite, basalt etc.

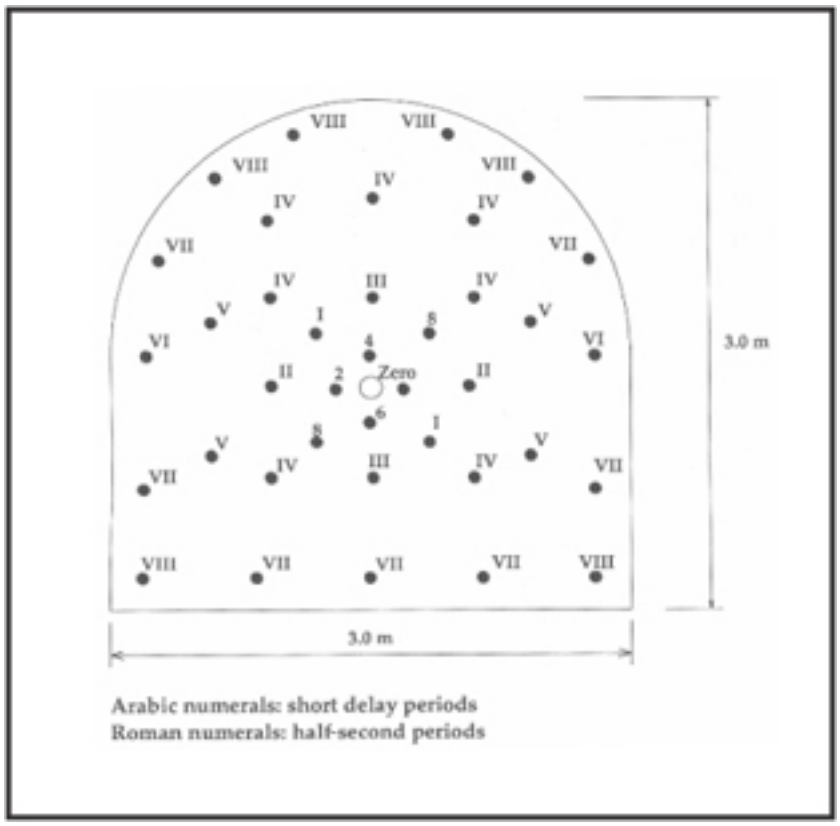


Figure 21: Burn cut

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Sequence of detonation

For both fragmentation and throw, blasting efficiency depends on the delay sequence of blasthole detonation. Delayed detonation improves loadability of the entire cut, contributes to a better strata control and reduction of blast-induced vibrations.

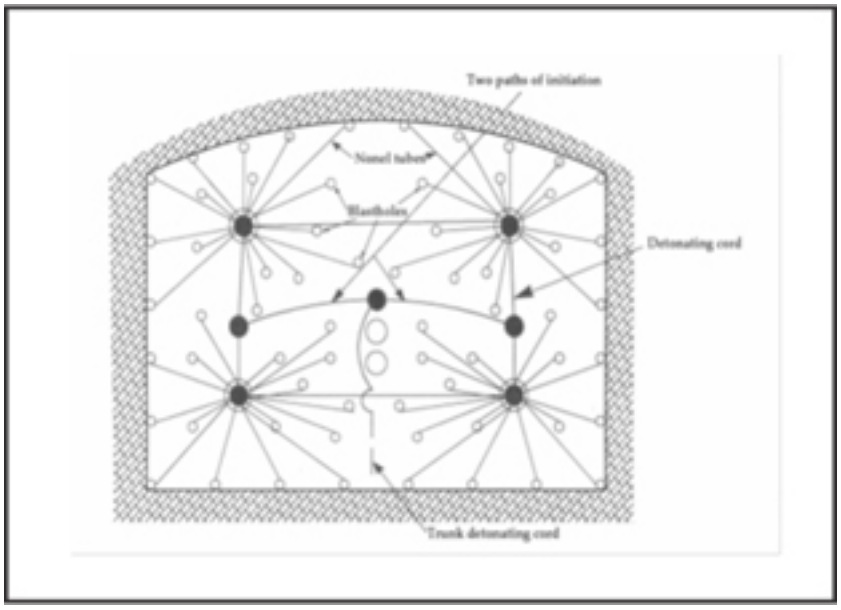


Figure 22: Bunch blasting with Nonel tubes and detonating cord.

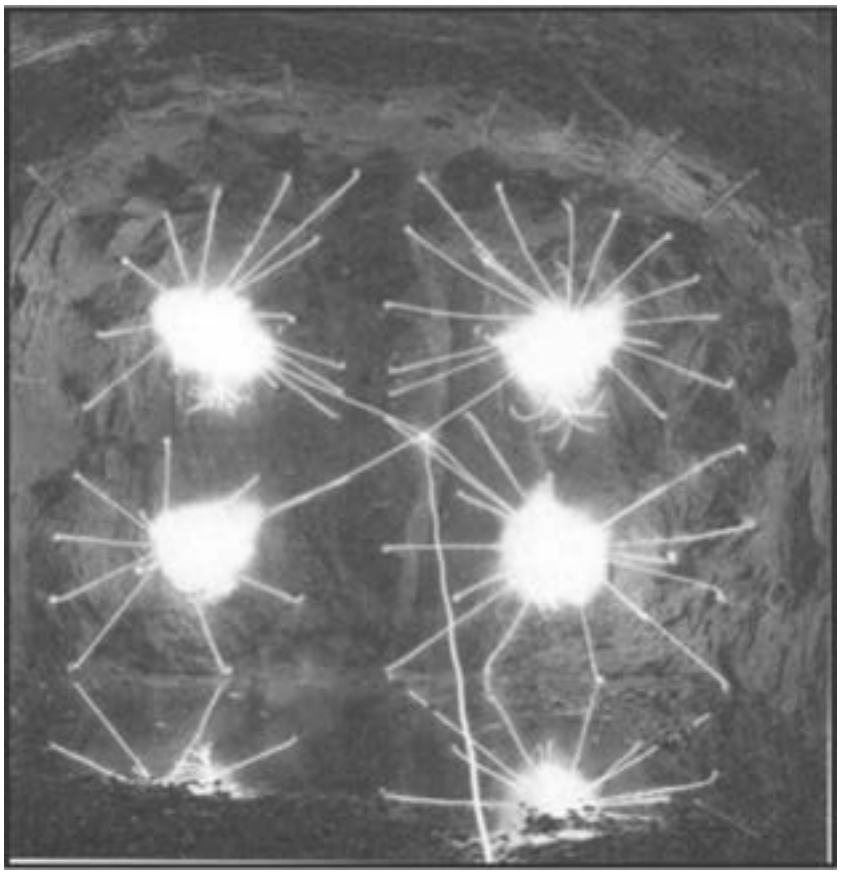


Figure 23: Firing of tunnel round with Nonel (after Olofsson)

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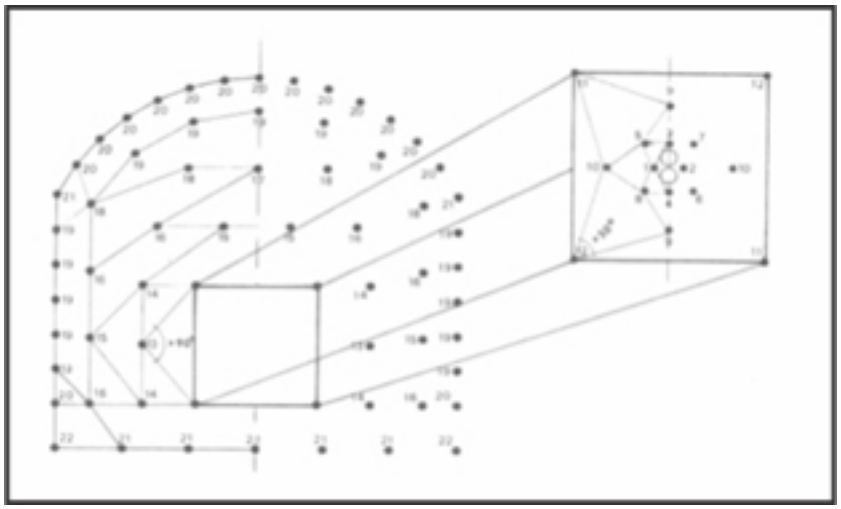


Figure 24: Firing sequence for tunnel numerical order.

LEARNING OUTCOME 7.2.7

CONNECTION 288

Refer to "LEARNING OUTCOME7.2.6" on page 472
Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE DIFFERENT CUTS USED IN TUNNEL DEVELOPMENT

LEARNING OUTCOME 7.2.8

CONNECTION 289

Refer to "LEARNING OUTCOME7.2.3" on page 466
Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE TYPES OF INITIATION USED IN THE ABOVE ROUNDS

LEARNING OUTCOME 7.2.9

CONNECTION 290

Refer to "LEARNING OUTCOME7.2.3" on page 466
Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE SEQUENCE OF INITIATION OF BLAST HOLES USED IN THE ABOVE ROUNDS

LEARNING OUTCOME 7.2.10

CONNECTION 291

Refer to "LEARNING OUTCOME7.2.3" on page 466
Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIMUM EXPLOSIVE TYPES AND THEIR PROPERTIES FOR USE IN THE ABOVE ROUNDS

LEARNING OUTCOME 7.2.11

CONNECTION 292

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS BLAST HOLE LAYOUTS IN STOPES

The mechanism of rock breakage by blasting

Blasting is the fundamental rock breaking process used in mining and as such it has a tremendous impact on all aspects of stoping operations.

When explosives confined in a blasthole are detonated, a high-speed compressive radial strain wave is generated which travels outwards from the hole in the rock. Under the influence of the wave, rock particles are moved radially outwards resulting in a tensile stress in the tangential direction. Where this tensile stress is greater than the tensile strength of the rock, radial stress fractures develop.

As the stress wave propagates outwards through the rock, it is followed by high pressure gas associated with the reacting explosive. The gas expands and penetrates the cracks caused by the stress wave. The cracks then extend until the gas pressure drops below the critical level required propagating the fractures. This gas pressure is what produces the heave that ejects the broken rock from the face. Explosive selection and use must, therefore, be characterized by selection of shock and gas energy combinations that achieve the desired blast characteristics.

Blasting techniques
It is vitally important to ensure the accuracy of marking, spacing, drilling and charging up of all blast holes in both stoping and development. The consequences of not maintaining the accuracies mentioned above will result in the creation of friable hanging wall conditions in the working area affecting both safety and production.

Marking
Marking of the holes is important and should be done as accurately as possible, since the other three parameters are based on it.



If you do not mark accurately, how can you drill accurately?

All holes should be marked according to the mine standard.

The positioning of top holes may be altered to suite local hanging wall conditions, when there is not a well defined parting plane existing above the reef. Parting planes in the immediate hanging wall do contribute to less vertical fracturing into the hanging wall.



Marking includes direction lines.

Spacing (Burden)
Burden is the distance between top holes measured along the face, and also the vertical distance between top and bottom holes, sometimes called the hole spacing.

The correct burden will ensure proper breaking of the rock, along the entire length of the panel, making it easier to clean and support the panel.

The burdens should be such that holes:

- Break to full depth,
- Do not blast out excessive hanging wall and footwall waste,
- Do not crater (blow-out) causing excessive fly-rock and damage to support.

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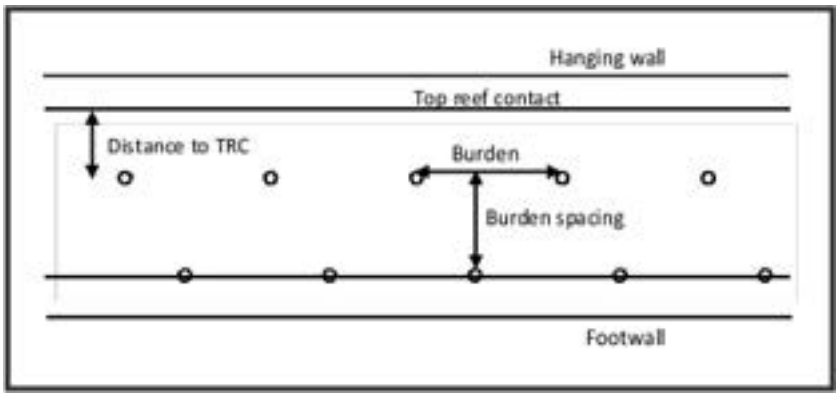


Figure 25: A typical drilling layout using v-shape in narrow tabular stope.

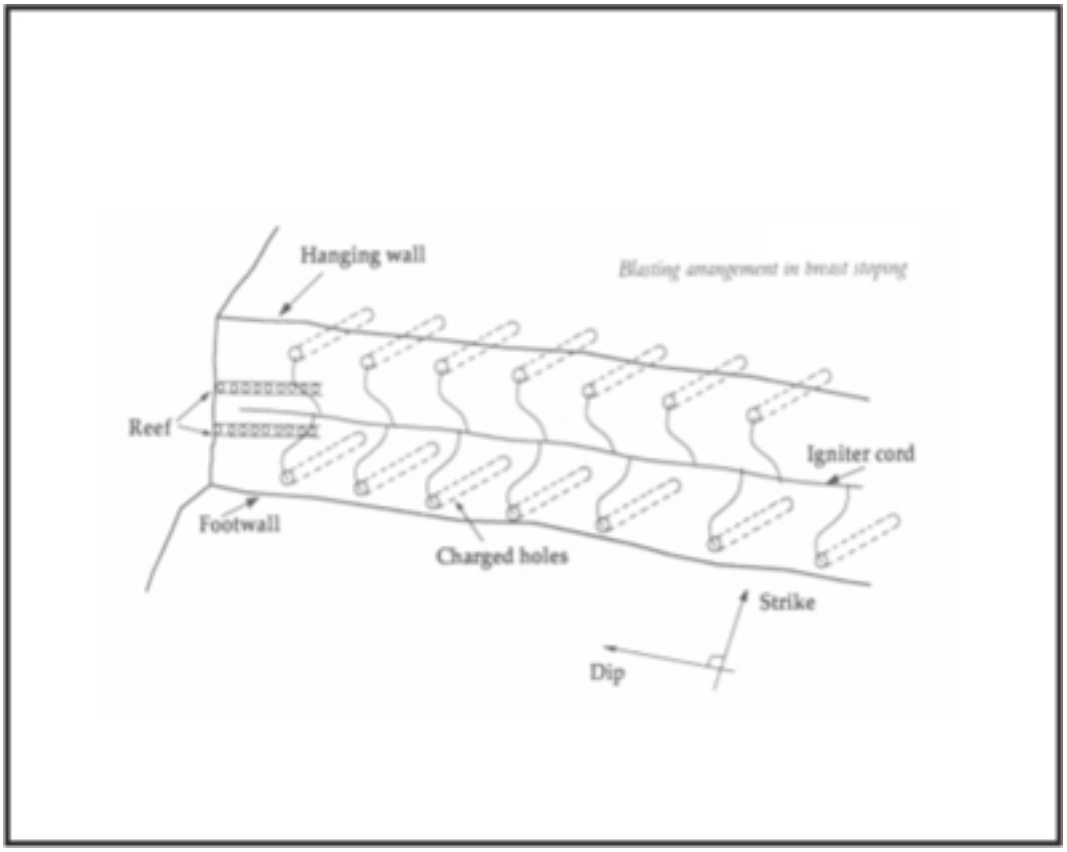


Figure 26: Blasting arrangement using square layout in narrow stope.

NOTE

Overburdened means holes are too far apart.
Under burden means holes are too close to each other.

LEARNING OUTCOME 7.2.12

CONNECTION 293

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE DIRECTION OF DRILLING OF BLAST HOLES IN STOPES

Drilling
When drilling blast holes there are three factors of critical importance.

- Drilling angle.
- Length of drill hole.
- Drilling direction.

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Drilling Angles

A shot hole drilled at a horizontal angle of 70º to the face, minimizes the length of sockets, because this angle helps the hole to break, without significantly reducing the advance, or throw of the rock. It is also complimentary to the natural breaking angle of rock which is 110º.

Breaking effect of angled holes

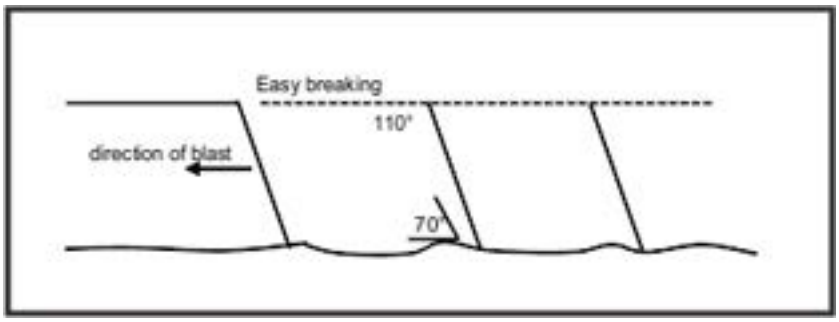


Figure 27: Drilling holes at 70° to the face line

In ground that is difficult to break (hard rock), drilling at 90° to the stope face is more likely to result in long sockets and an uneven face advance, unless the rock had been pre-fractured by a rock conditioning method such as pre-conditioning.

Other aspects of drilling at 90º which may change are:

- Length of hole could be shorter
- Type of explosive used
- Burden could be reduced

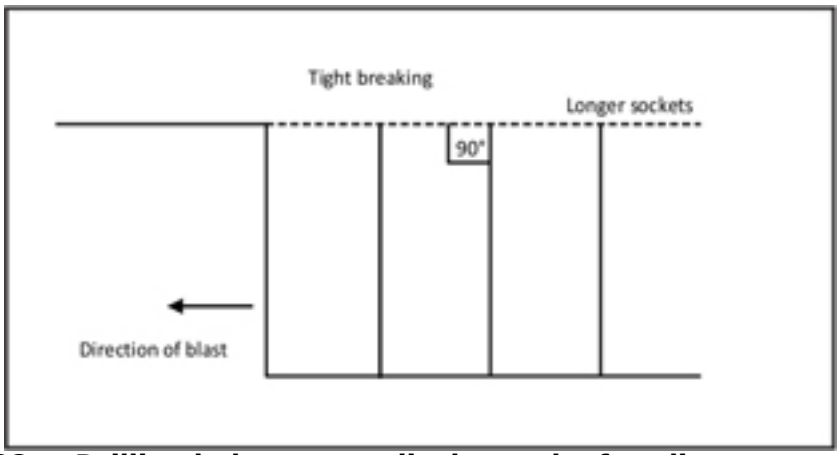


Figure 28: Drilling holes perpendicular to the face line

Drilling all holes in the same direction is important this will help with the breaking of the rock and ensures a straight panel, it will also prevent damage to the hanging wall and footwall.

Length of the drill hole

The length of the hole drilled should be approximately the same as the stoping width (on narrow reef). It is extremely important that all holes be drilled to the same depth so as to ensure an even break of the rock, and leaving a straight panel, which is easy to clean and support.

Drilling all the holes to the correct depth will also minimize the damage to the hanging wall and to the installed support.

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Short holes increase the actual "toe end" burden of the adjacent or neighboring holes. This results in long sockets of these holes, which causes an uneven face shape.

Direction of drill holes
When marking the face it is important for the miner to paint in direction lines for the machine operator so that he knows in which direction to drill. Failure to do so will result in holes drilled in different directions leading to over/under burdened holes.

When all the holes are drilled in the same direction the broken rock from the blast will be evenly spread out assisting in the cleaning time of the panel.

Avoid drilling holes directed up into the hanging wall since this will result in mining unnecessary waste and damage the hanging wall beam.

Avoid drilling holes directed down into the footwall, since this will also result in mining unnecessary waste

LEARNING OUTCOME 7.2.13

CONNECTION 294

Refer to "LEARNING OUTCOME 7.2.12" on page 481
Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE SEQUENCE OF INITIATION OF BLAST HOLES IN STOPES

LEARNING OUTCOME 7.2.14

CONNECTION 295

Refer to "LEARNING OUTCOME 7.2.3" on page 466, "LEARNING OUTCOME 7.2.4" on page 467, "LEARNING OUTCOME 7.2.5" on page 468

Refer to Paper 2, Outcomes 7.1, 7.2 and 7.3 from page 533

DESCRIBE, EXPLAIN AND DISCUSS THE OPTIMUM EXPLOSIVE TYPES AND THEIR PROPERTIES FOR USE IN THE ABOVE ROUNDS

Charging Up
All charging up should be done according to the standard determined by explosives experts for the particular application.

The use of the correct amount and type of explosives in the holes combined with a correct burden should cause the rock to be fragmented to the correct size for easy handling and cleaning.

Overcharging of holes will cause damage to the surrounding rock mass, and damage to face support elements, leading to strata control problems. This may leave the face area unsupported, and accidents can occur when people go back to work in the face area after the blast.

There are no advantages of overcharging. It will not help you achieve extra face advance, the only extra rock to be broken will be the surrounding country rock leaving you with more cleaning and more support to install.

LEARNING OUTCOME 7.2.15

DESCRIBE, EXPLAIN AND DISCUSS THE IMPORTANCE OF BLAST-HOLE DRILLING ACCURACY IN THE FOLLOWING APPLICATIONS:

- Shaft sinking,
- Chamber excavation,
- Orepass excavation,
- Tunnel development,
- Stope advance,
- Cushion blasting,
- Smooth blasting.

Blasting practices in shaft sinking, chamber excavation, orepass excavation and development ends are as important as in the stoping environment. In most instances the service excavations have to stay open for long periods of time, probably for the life of the mine. It is therefore important that damage to the hanging walls and/or sidewalls of the excavation be kept to a minimum. This will improve the longevity of the excavation, and reduce support costs.

Most blasting methods used in developing a tunnel are designed to create a free breaking point, which again will prevent damage to the surrounding hanging- and footwall. Good blasting practices can have a major influence on the support costs of development ends.

Blast consultants take into account rock conditions versus required excavation shape and size to design blast rounds. In order to achieve the required shape and size with minimal damage, it is essential for operators to drill exactly to specification.

Good blasting practice:

- starts with correct marking of the drill holes.
- This is followed up by correct drilling direction.
- The drilling direction will have an influence on the burden of the individual holes,
- which in turn will influence the effectiveness of the blast.

If overbreaking and severe fracturing of the rock surrounding the excavation result, ground control problems and even severe damage to the excavation is caused, leading to large rehabilitation costs. .

As discussed in the individual sections above, it is therefore essential that drilling and blasting control is carried out and the correct blasting and drilling procedures for the different excavations are followed at all times.

7.3. BLASTING PRACTICES

LEARNING OUTCOME 7.3.1

CONNECTION 296

Refer to Paper 2, Outcome 7.3 from page 560

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE CHEMICAL REACTIONS ASSOCIATED WITH VARIOUS EXPLOSIVES USED IN MINING

An explosive is any material or device which can produce a sudden outburst of gas, applying a high impulsive loading to the surrounding medium.

Chemical explosives are the most widely used in mining practice. Industrial chemical explosives are of two main types:

- deflagrating explosives, such as black powder, which burn relatively slowly, and produce relatively low blasthole pressure;
- detonating (high) explosives, which are characterized by super-acoustic reaction rate and high blasthole pressure.

The detonating explosives are themselves considered in three categories, reflecting their respective sensitivities to ease of initiation of detonation.

- Primary explosives, such as lead azide, lead styphnate, or mercury fulminate, are initiated by spark or impact. They are highly unstable compounds, and used industrially only in initiating devices such as blasting caps, as the top charge.
- Secondary explosives require the use of a blasting cap for practical initiation, and in some cases may require an ancillary booster charge. Explosives in this category are formulated from chemicals such as nitroglycerin (NG), ethylene-glycoldinitrate (EGDN), or pentaerythrotetranitrate (PETN), mixed with other explosive materials and stabilizing agents.
- Tertiary explosives are insensitive to initiation by a standard (No. 6) strength blasting cap. Most explosives in this category are the dry blasting agents (DBAs) or slurry explosives and blasting agents.

Historically, most high explosives were manufactured from the organic nitrates, mixed with organic and inorganic chemicals to produce mechanically and chemically stable materials known as gelignites and dynamites. The proportion of organic nitrates in these types of explosives has been reduced by their partial replacement with such chemicals as ammonium nitrate (AN).



Industrial consumption of these explosives has declined, due to the formulation of more convenient explosive mixtures.

A blasting agent such as ANFO (94% AN–6% fuel oil) is an oxygen-balanced mixture of an oxidiser and a fuel. This means that by achieving close admixture of these components, the material can be detonated, to yield H_2O, NO_2, and CO_2.

Slurry and emulsion blasting agents and explosives are also formulated as an oxygen-balanced mixture, but instead of a simple physical admixture, the oxidizer and fuel constituents of the explosive are suspended in a stable water–gel matrix. They are typically more stable chemically than dry blasting agents, and may be water resistant.

Detonation in an explosive column (contained, for example, in a blast hole) involves the passage through the column of a chemical reaction front. The front is driven through the column, by the products of the reaction, at super-acoustic velocity, called the detonation velocity. Passage

of the front causes a step rise in pressure in the explosive, achieving a detonation pressure which may exceed many GPa. The blasthole wall is subjected to a pressure approaching that in the detonation front. Pressure decay in the blast hole is relatively slow, due to the persistent action of the detonation product gases.

The response of rock to explosive attack is determined by both the explosive properties and the rock properties. The main explosive property is its strength, which is a standardized measure of the explosive's capacity to do useful work on its surroundings.

It may be measured in standard tests, such as an underwater test, in which pressure pulses transmitted in the water after detonation of a charge are used to estimate the energy release from the charge. These tests indicate that the release of the explosive's chemical potential energy does not occur in a single episode. The initial energy release associated with detonation (the shock energy) is followed by oscillation of the product gas bubble.

Successive phases of energy output are identified with the bubble energy of the explosive, which represents energy possessed by the detonation product gas due to its high pressure.

INTERESTING INFO

Another current method of assessing the strength of an explosive is by estimating its free-energy output, from the thermodynamics of its detonation reaction. The Absolute Strength Value (ASV) of an explosive is expressed in terms of the free-energy output (in joules) per 0.1 kg of explosive. The Relative Strength Value of an explosive is the ratio of its calculated useful work output, relative to that of ANFO, taken as a basis of 100 units.

Several rock properties determine the performance of a particular explosive in the medium. The capacity of the rock mass to transmit energy is related in part to the Young's modulus of the rock medium. The ease of generating new fractures in the medium is a function of the strength properties of the rock material, which may be represented for convenience by the uniaxial compressive strength. The unit weight of the rock mass affects both the energy required to displace the fragmented rock, and the energy transmissive properties of the intact medium.

Efficient and successful performance of an explosive in a rock mass requires that its properties be compatible with those of the subject rock mass. An empirical correlation of the preferred explosive type, for a range of rock material and rock mass properties, is proposed in Figure 29.

For very low average fracture spacing, or very low material compressive strength, the rock mass may be ripped or dug mechanically, rather than blasted. A significant observation from Figure 29 is that ANFO is a suitable explosive for use in a wide range of rock mass conditions. The application of high-brisance explosives is justified only in the strongest, more massive rock formations.

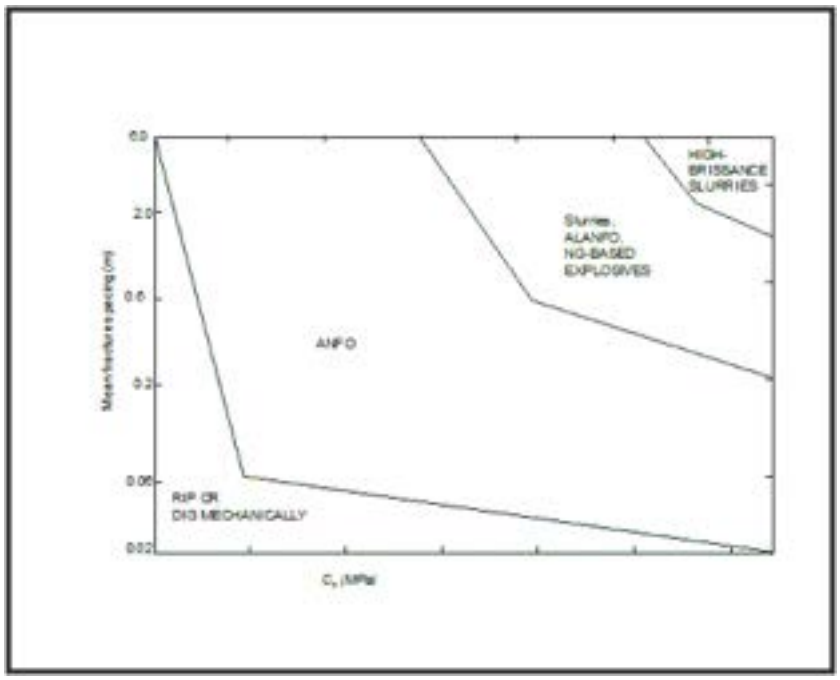


Figure 29: An empirical matching of explosive and rock mass properties (Brady & Brown)

Other explosive properties in addition to strength need to be considered in explosive selection for a particular blasting operation. In underground applications, the fume properties of the explosive, i.e. the chemicals present in the detonation product gases, determine the air quality in the working area after blasting.

The chemical properties of the explosive itself and of the lithology in which it is to be used, can present safety problems. For example, some sulphide ores are subject to rapid, exothermic oxidation on contact with ammonium nitrate. The nature of the flame or flash produced by the explosive on detonation determines its potential for initiating detonation in the mine atmosphere. Such unfavorable atmospheric conditions can arise in gassy coal seams, or in highly pyritic metalliferous orebodies. Finally, explosive properties such as the air gap sensitivity and impact sensitivity have direct operational consequences related to their use in fractured ground and in development blasting.



The explosive types and application are discussed in detail in Outcome 7.2

LEARNING OUTCOME 7.3.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PHYSICAL MANIFESTATIONS ASSOCIATED WITH VARIOUS EXPLOSIVES USED IN MINING

Phenomenology of rock breakage by explosives

Explosive attack on rock is an extremely violent process. In the period during and following the passage of a detonation wave along an explosive charge, the rock around the blast hole is subjected to the following phases of loading:

- dynamic loading, during detonation of the explosive charge;
- quasi-static loading, under the residual blasthole pressure applied by the detonation product gases;
- release of loading, during the period of rock displacement.

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The evolution of fracture patterns associated with these intervals of loading is illustrated in Figure 30.

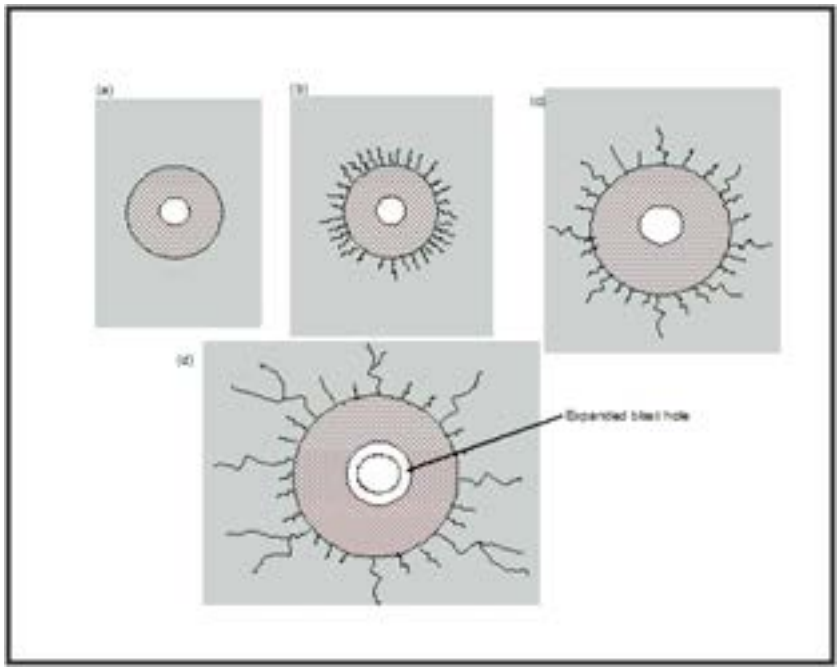


Figure 30: Successive stages of destruction of rock by an explosive charge under dynamic load (a,b,c) and quasi-static load (d) (after Kutter and Fairhurst, 1971).

Three zones of material response to the impulsive loading and high-intensity stress wave are recognized.

- In the immediate vicinity of the blast hole, passage of the stress wave causes the rock to be crushed or extensively cracked. For a blast hole of radius (rh), the radius (rs) of the shock zone equals approximately 2 x rh. In some cases, superficial observation may not, in fact, reveal a crushed zone around a blast hole.
- The domain immediately outside the shock zone is called the transition zone. New fractures are initiated and propagated in the radially compressive stress field, resulting in a severely cracked annulus. Generation of the radial cracks extracts energy from the radial P wave, resulting in reduction in the stress intensity. The radius (rt) of the transition zone is approximately 4 to 6 x rh.
- Crack propagation occurs exclusively by extension of the longest cracks of the transition zone. The P wave rapidly outruns the crack tips, and propagation ceases. It appears that at a radius of about 9 x rh, macroscopic crack generation by the primary radial wave ceases.
- When the radial compressive wave is reflected at a free face, a tensile wave, whose apparent source is the mirror image of the blast hole reflected in the free face, is generated (Figure 31). In the medium, crack extension may be promoted by the tensile stress field, resulting in further accumulation of damage.

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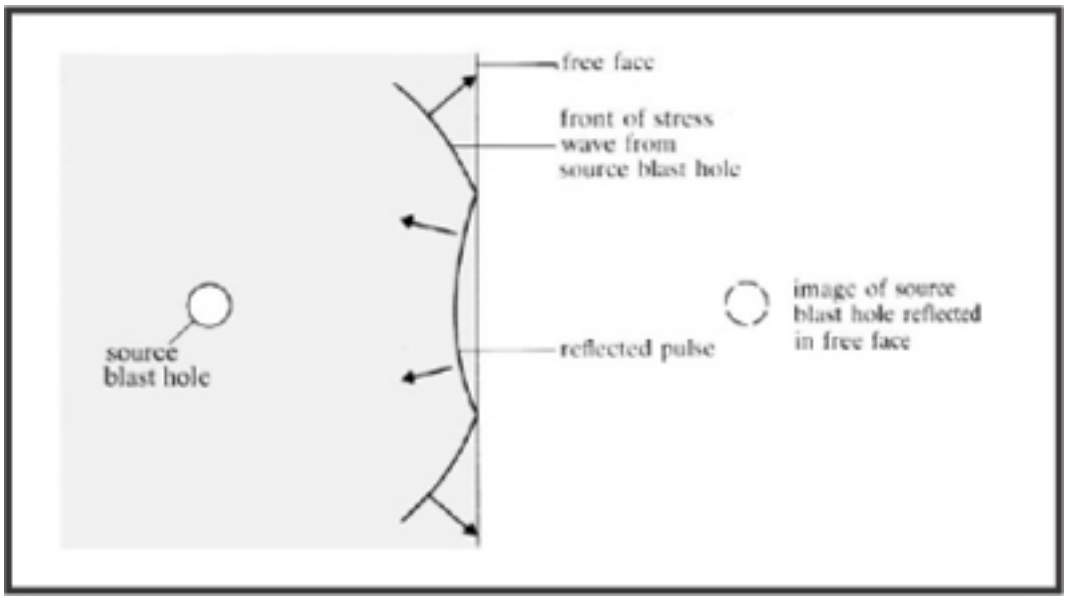


Figure 31: Tensile reflecting wave (Brady and Brown, 2006)

REFERENCE

- Page 518, Chapter 17, Brady BHG and Brown ET, Rock mechanics for underground mining, 2006

LEARNING OUTCOME 7.3.3

CONNECTION 297

See "DRILLING TECHNIQUES" on page 464 above for details of different explosives and their application.

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE BASIC DIFFERENCES BETWEEN EXPLOSIVES USED IN MINING

LEARNING OUTCOME 7.3.4

DESCRIBE, EXPLAIN AND DISCUSS THE EFFECT OF THE FOLLOWING PARAMETERS ON BLAST DAMAGE :

- Explosive type,
- Initiation method,
- Initiation sequence,
- Hole orientation

NOTE

The effect of the above parameters on blast damage have been discussed at length in Outcome 7.2

Explosive Type

Explosives are designed to perform to certain benchmarks with regards to energy release and gas release. Rock properties play an important role in the decision of what type and amount of explosives is needed for a successful blast. Very competent rock will require a large amount of energy to fracture the rock, whereas rock which is broken up by stress fracturing will require less energy to be broken up. Excess energy which is not used in breaking up of the rock will be available to extend fractures into the surrounding rock mass. Using the wrong type of explosive will result in overbreak in development tunnels and large excavations. In stope faces, this will result in blast fractures extending into an already fractured hanging wall and possible ground control problems. Excessive

back

fracturing of the rock will also result in stoping width exceeding the standard, and diluting the ore because of waste extraction.

Initiation sequence
In tunnels and service excavations, the round is made up of a cut, normal blast holes and perimeter holes. The sequencing is of utmost importance as the cut needs to be initiated first in order to create a free breaking surface into which the remainder of the holes will break. Depending on the method of smoothblasting (pre- or post-splitting), the perimeter holes will either go off first or last.

In stope blasting, a free breaking surface exists next to the gully. Sequencing of the blast is done by starting to break into the free surface, and following on in steps up to the solid end of the panel (Figure 27 and Figure 28).

Hole orientation
The round is marked off in a set pattern to ensure the correct burden is achieved for the individual holes. Orientation of the holes is very important to maintain the burden in the blast sequence. In tunnel blasting, the holes in the cut may be directed at an angle, and needs to be drilled accurately. Drilling off-line will result in unsuccessfull breaking of the rock, and damage to the surrounding rock mass because of excess energy released into the rock.

In stope face blasting, the most effective drilling direction is 70° (Figure 27). However, because of ease of control, the tendency is to drill holes at 90° in stope blasting. It is more important to ensure that all the holes are drilled parallel to one another. This will ensure constant burden and more effective breaking of the rock.

Controlling all the above parameters is of utmost importance to control damage to the surrounding rock mass as well as to support installed in the excavation.

LEARNING OUTCOME 7.3.5

CONNECTION 298

Refer to Paper 2 Outcome 7.3.4 and 7.3.5 on page 569

DESCRIBE, EXPLAIN AND DISCUSS THE OBJECTIVES AND EFFECTS OF DE-COUPLING EXPLOSIVES

LEARNING OUTCOME 7.3.6

CONNECTION 299

Refer to Paper 2 Outcome 7.3.4 and 7.3.5 on page 569

DESCRIBE, EXPLAIN AND DISCUSS THE METHODS BY WHICH DE-COUPLING OF EXPLOSIVES IS ACHIEVED

Decoupling
The use of cartridged explosive products significantly smaller in diameter than the diameter of the blasthole. Decoupling or the use of decoupling charges is designed to reduce the charge concentration in the blasthole and minimize stresses exerted on the walls of the blasthole.

De-coupling of explosives results in lower energy levels during the blast. This is required when blasting under special conditions such as perimeter blasting. The energy released by decoupled explosives are design to create a fracture only between two existing points, such as two holes drilled close together.

There are commercial explosive products available that can be used to

achieve this. Alternatively, the amount of explosives used in the blast hole can be reduced. This can be achieved by either using smaller cartridges, or to use a de-coupling spacing insert between cartridges.

LEARNING OUTCOME 7.3.7

CONNECTION 300

Refer to Paper 2 Outcome 7.3.2 on page 560

DESCRIBE, EXPLAIN AND DISCUSS THE FOLLOWING EXCAVATION CUSHION BLASTING AND SMOOTH BLASTING TECHNIQUES :

- Pre-splitting,
- Concurrent smooth blasting,
- Post-splitting

Pre-splitting
With this method, a continuous fracture which will form the final surface of the excavation is generated in the absence of a local free face. In Figure 32 , the pre-splitting holes are shown on the actual perimeter of the tunnel/excavation. These holes will be sequenced to explode before the main cut is blasted in order to create a fracture around the perimeter of the tunnel. The production blast will then be initiated, and break out the rock inside the perimeter, leaving a smooth excavation wall. The pre-splitting thus creates a cushion around the tunnel to prevent blast damage from extending into the host rock surrounding the tunnel.

Concurrent smooth blasting
Concurrent smoothblasting takes place when the perimeter holes are sequenced as part of the total blast. The perimeter holes are spaced closer together than the standard blast holes, and de-coupled explosives are used to charge them up. This assists in breaking a smooth perimeter for the excavation, and is used where stresses are moderate.

Post splitting
Post splitting entails the blasting of a perimeter on the excavation surface, and is blasted after the main blast has been completed. Often, the two blasts are done separately, with the post splitting only set off after the blast has been completed. The procedure is the same as with pre-splitting except for the timing of the blast.

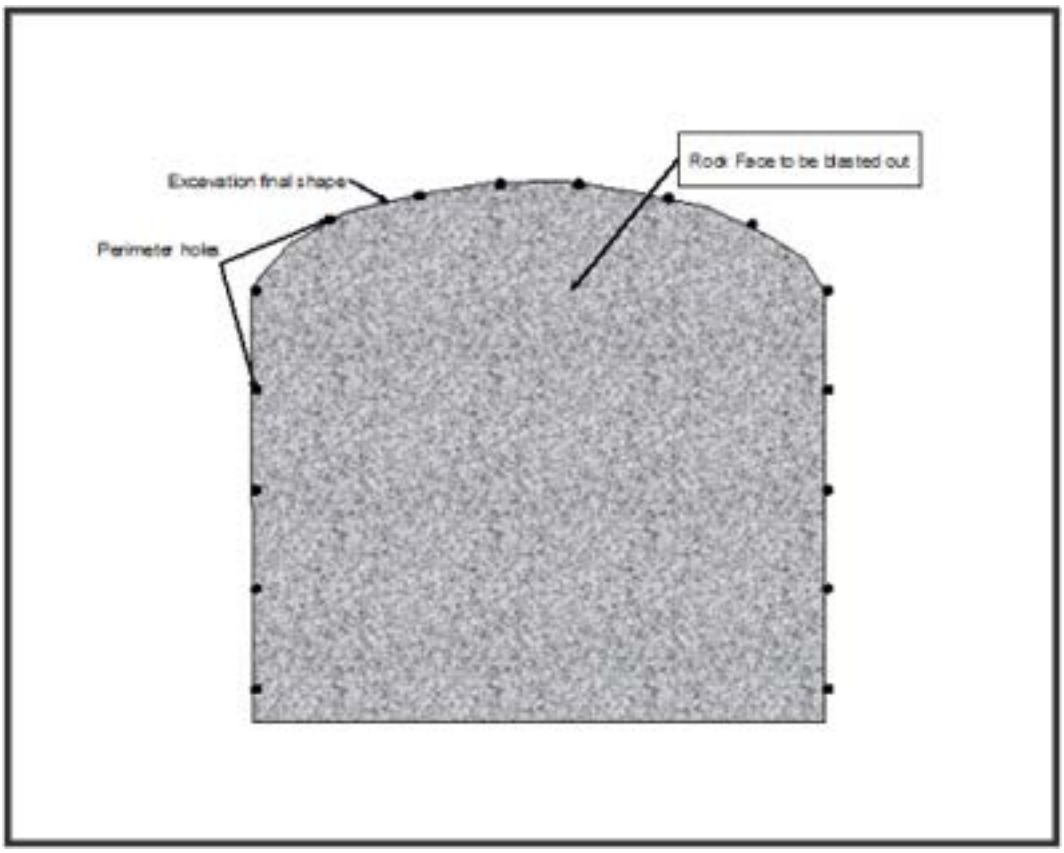


Figure 32: Smooth blasting or cushion blasting layout for a tunnel

REFERENCE

- Page 528, 531, Chapter 17.6.1 & 17.6.2, Brady BHG and Brown ET, Rock mechanics for underground mining, 2006

LEARNING OUTCOME 7.3.8

CONNECTION 301

Refer to "LEARNING OUTCOME 7.3.7" on page 491 and Paper 2 Outcome 7.3.2

DESCRIBE, EXPLAIN AND DISCUSS THE METHODOLOGIES AND TYPICAL APPLICATIONS OF EACH TECHNIQUE

LEARNING OUTCOME 7.3.9

LIST AND DISCUSS THE ADVANTAGES AND DISADVANTAGES OF THESE TECHNIQUES

Pre-splitting techniques are less effective in high stress environments, and therefore is usually used in surface mining and soft rock environments. With surface mining, pre-splitting is used to split the bench, for example, and then blast the main production blast to remove the ore from the bench.

Contour blasting



Figure 33: Barrels through pre-splitting

In civil construction, it has become more and more important that the rock wall remaining after the blast is of good quality to avoid or minimize rockfall, rockslides and excessive stabilization work. Two methods are used to produce stable and smooth rock contours.

1. smooth blasting
2. pre splitting

1. Smooth blasting

Closely spaced holes are drilled in the final required excavation wall position (contour or perimeter) which are charged with small diameter (de-coupled) light charges producing relatively low detonation velocity and low gas content. The blasting of the contour holes is carried out with the rest of the round but the contour (perimeter) holes are given the highest period number, i.e. fired last.

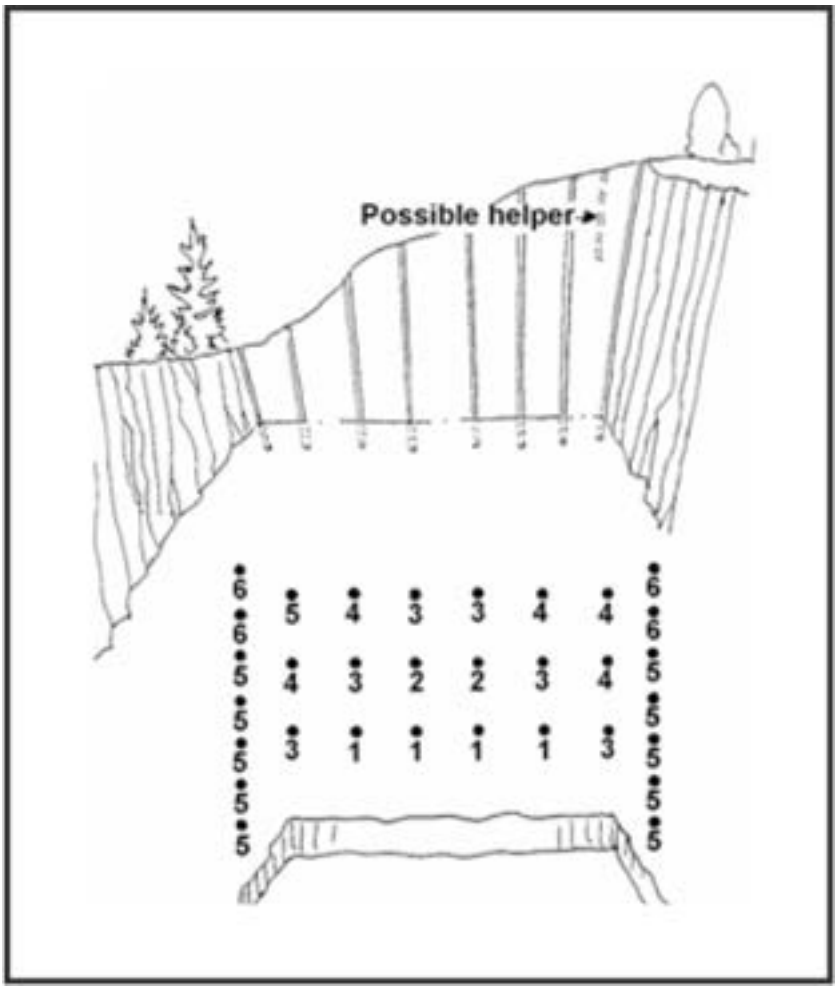


Figure 34: Layout of pre-splitting holes with normal production holes

The holes closest to the contour / perimeter must also have well balanced charges. If charge concentration is too high there is a risk that those holes will break the rock beyond the theoretical excavation contour line.

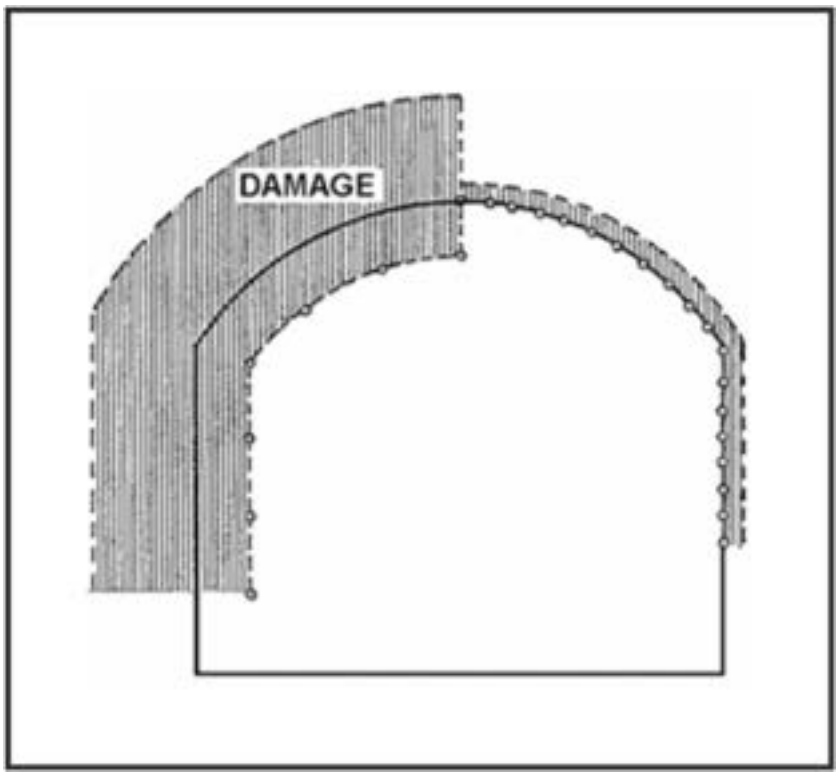


Figure 35: Area of damaged rock where smoothblasting is not effective / applied

back

2. Pre-splitting

The idea of pre-splitting is to isolate 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. It may be done with firing of the round, but using the lowest number of the detonator series, i.e. fired first. It can also be done before the drilling of the main round.

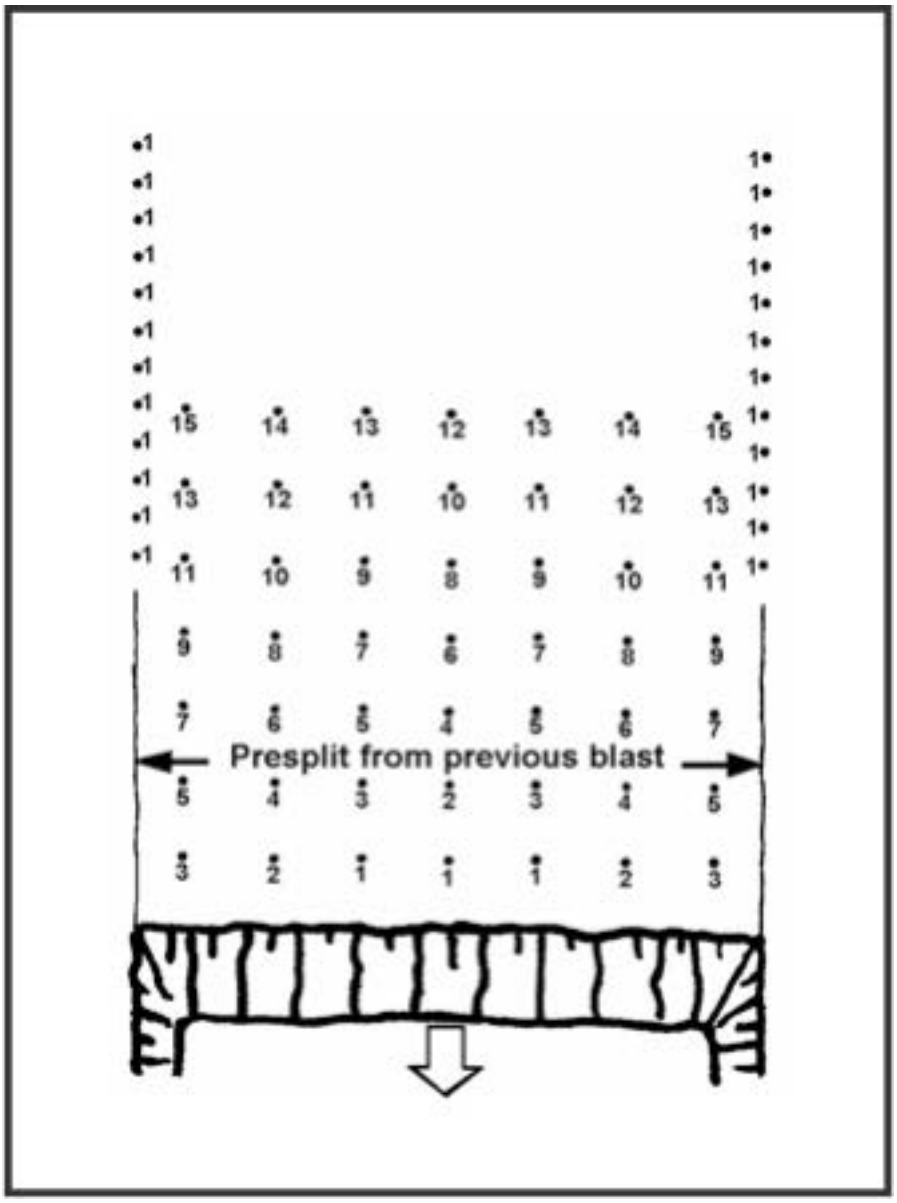


Figure 36: Pre-splitting layout

During pre-splitting, shock waves from simultaneously detonating charges in a closely spaced row of holes collide and a tensile stress occurs in a direction away from the line of the fired holes, forming a crack in the rock. It is thus important that all holes are fired simultaneously to allow the creation of this tensile stress during wave interactions.

For best results, detonating cords or instantaneous detonators should be used for initiation. Noise may be a problem as pre-split holes should not be stemmed. Another problem can be excessive ground vibrations.

back

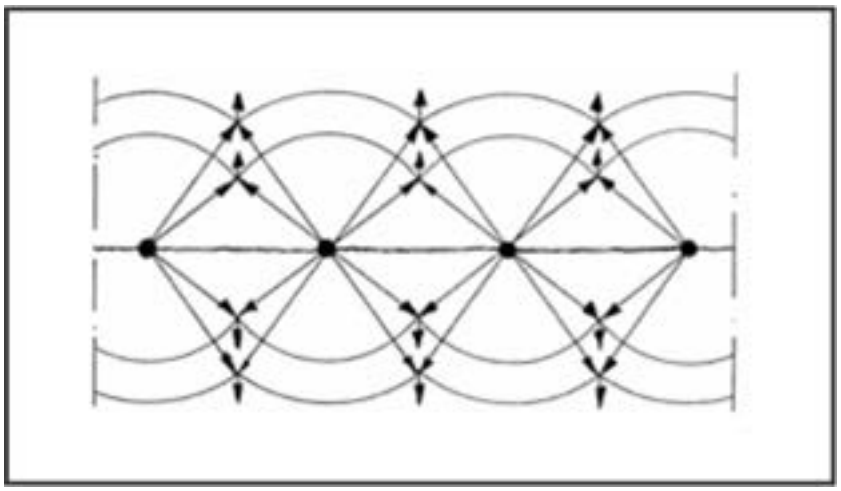


Figure 37: Tensile stress contours around pre-splitting blasting holes interacting to form a crack

LEARNING OUTCOME 7.3.10

EVALUATE AND DETERMINE BLASTING REQUIREMENTS FOR TUNNELS MAKING USE OF KNOWLEDGE OF EXPLOSIVES



Apply all information and knowledge in Outcomes 7.1 to 7.3.9, as well as Paper 2 Outcomes 7.1, 7.2 and 7.3 to select appropriate solutions for specific situations.

In addressing Outcomes 7.3.10 to 7.3.15, the following must be taken into account when deciding on the optimum drilling and blasting practice:

a. Rock medium to be blasted:

- Strength (uniaxial strength)
- Behaviour (failure mode)
- Quality (blocky, solid and massive, etc)
- Presence of structures (joints, fractures, faults, etc)

b. Other conditions

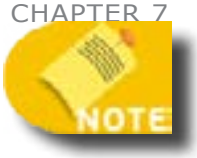
- Presence of water (resistance to water)
- Stress levels (impact on drilling, hole diameters, fractures, energy flow)
- Ventilation (fumes)

c. Expected results:

- Excavation perimeter shape (de-coupling, pre- or post-splitting, smoothwall)
- Fragmentation (fine, small, large)
- Excavation (tunnel, stope, cutting, pit wall, etc)
- Throw required (position of blasted material)

d. Available services:

- Drilling methods (handheld, rigs)
- Explosive types (types)
- Initiation systems (electronic, igniter cord, detonators, etc)



Although it is prudent to know and understand issues around drilling and blasting, the services of an explosives engineer cannot be over-estimated when designing final blast rounds.

A quick reference guide is available from the website below, courtesy of Dyno Nobel.

- http://apac.dynonobel.com/files/2010/02/Blasting-and-Explosives-Quick-Reference-Guide-2011.pdf

LEARNING OUTCOME 7.3.11

EVALUATE AND DETERMINE APPROPRIATE BLASTING ROUNDS TO SUIT GIVEN CONDITIONS IN TUNNELS



Apply all information and knowledge in Outcomes 7.1 to 7.3.9, as well as Paper 2 Outcomes 7.1, 7.2 and 7.3 to select appropriate solutions for specific situations.

LEARNING OUTCOME 7.3.12

EVALUATE AND DETERMINE APPROPRIATE EXPLOSIVE TYPES TO SUIT GIVEN CONDITIONS IN TUNNELS



Apply all information and knowledge in Outcomes 7.1 to 7.3.9, as well as Paper 2 Outcomes 7.1, 7.2 and 7.3 to select appropriate solutions for specific situations.

LEARNING OUTCOME 7.3.13

EVALUATE AND DETERMINE BLASTING REQUIREMENTS FOR STOPES MAKING USE OF KNOWLEDGE OF EXPLOSIVES



Apply all information and knowledge in Outcomes 7.1 to 7.3.9, as well as Paper 2 Outcomes 7.1, 7.2 and 7.3 to select appropriate solutions for specific situations.

LEARNING OUTCOME 7.3.14

EVALUATE AND DETERMINE APPROPRIATE BLASTING ROUNDS TO SUIT GIVEN CONDITIONS IN STOPES



Apply all information and knowledge in Outcomes 7.1 to 7.3.9, as well as Paper 2 Outcomes 7.1, 7.2 and 7.3 to select appropriate solutions for specific situations.

LEARNING OUTCOME 7.3.15

EVALUATE AND DETERMINE APPROPRIATE EXPLOSIVE TYPES TO SUIT GIVEN CONDITIONS IN STOPES.



Apply all information and knowledge in Outcomes 7.1 to 7.3.9, as well as Paper 2 Outcomes 7.1, 7.2 and 7.3 to select appropriate solutions for specific situations.

CHAPTER 8

PAPER 3.1 HARD ROCK TABULAR MINING

LEARNING OUTCOMES

8. SURFACE EFFECTS
8.1. SUBSIDENCE ENGINEERING

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

- Sketch, describe, explain and discuss the geotechnical aspects associated with subsidence of rock to surface above shallow tabular hard rock workings.

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

- Sketch, describe, explain and discuss the protection of the surface subject to subsidence above shallow tabular hard rock workings
- Design various appropriate pillars for the protection of the surface subject to subsidence above shallow tabular hard rock workings.

8.1. SUBSIDENCE ENGINEERING

LEARNING OUTCOME 8.1.1

CONNECTION 302
Refer to "LEARNING OUTCOME6.2.3"onpage438

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE GEOTECHNICAL ASPECTS ASSOCIATED WITH SUBSIDENCE OF ROCK TO SURFACE ABOVE SHALLOW TABULAR HARD ROCK WORKINGS.

Subsidence caused by high extraction mining has a different mechanism than that caused by pillar system failure and causes a different magnitude and rate of subsidence.

High extraction methods leaves back areas unsupported and allows the hangingwall to collapse. The collapsed area extends vertically and increases in volume due to the presence of voids in the collapsed material. This increase in volume eventually reaches a stage where the collapsed material makes contact with the overlying stable rock (Figure 1). At this stage, the uncontrolled collapse ceases.

If the remaining overburden rock material is a continuous, massive rock mass, surface subsidence consists of bending of the overlying plate until it touches the collapsed material. The amount of bending is determined by the rock mass properties, the mining span and the compressibility of the collapsed material.

back

However, if the overlying rock mass is jointed, gravity will cause large blocks to slide along pre-existing joint planes, compressing the collapsed material below (Figure 2). This process continues until the resistance offered by the failed material equals the weight of the overlying material.

This process explains two very important characteristics of high extraction subsidence.

- The total subsidence is less than the original mining height and often even less than half of the mining height since the weight of the overlying material is insufficient to recompress the failed material to the solid state (fill all voids).
- The rate of subsidence is relatively slow and takes weeks to manifest 90% of the full subsidence and the final 10% may take several years as the rate of subsidence is determined by the rate at which the failed material is compressed.

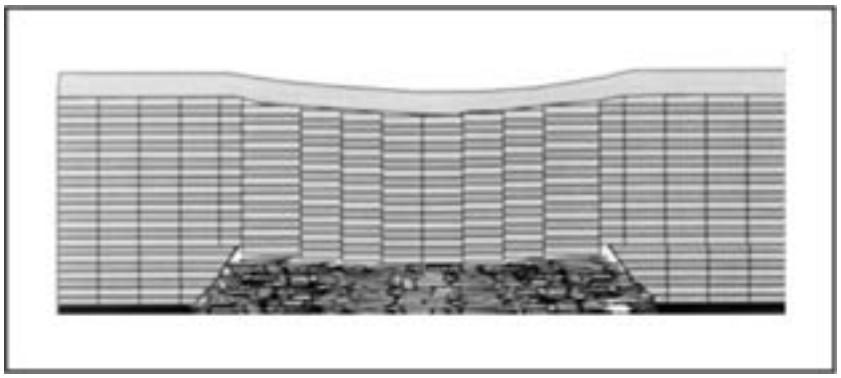


Figure 1: Simplified illustration of the subsidence process in high extraction methods (vd Merwe, 2002)

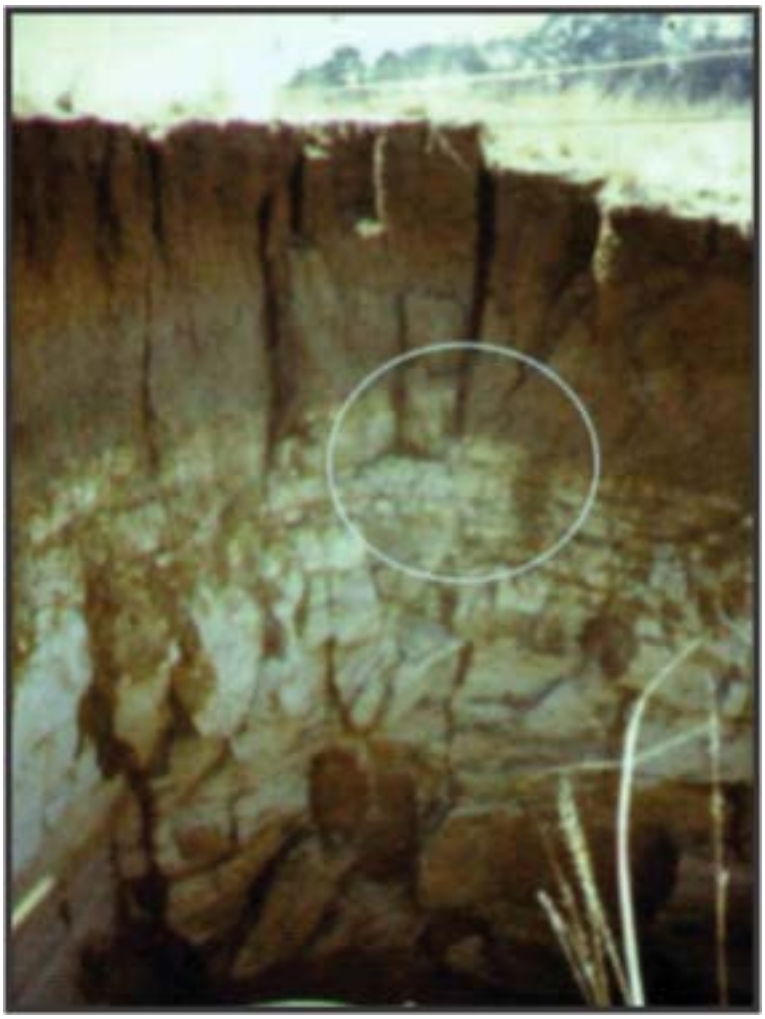


Figure 2: Block movement in the overburden

The circled area in Figure 2 clearly shows discrete blocks of rock that have displaced relative to one another.

In contrast to high extraction mining, failure of pillars suggests that the area may not be totally devoid of support (some pillars are still generating some load) and that there is some resistance to the uncontrolled collapse of the overlying rock mass. This allows the overburden rock mass to 'sit down' on the 'failing' pillars (Figure 3).

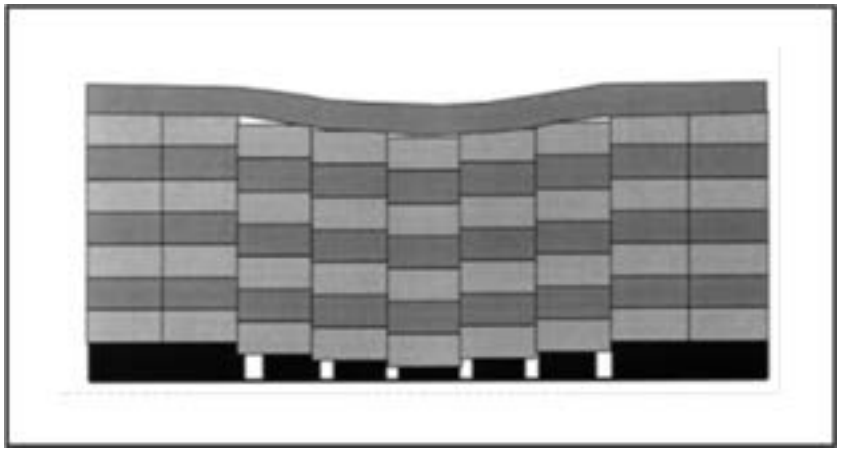


Figure 3: Subsidence in case of pillar failure (vd Merwe, 2002)

There is no collapse above the mined out area in contrast to that caused by high extraction mining.

- Since there is no failed material to compress, the rate of subsidence is governed by the rate at which the pillars fail and is normally greater than the rate of failed material compression, often almost immediately.
- Due to the absence of failed material and the subsequent increase in material volume, the total subsidence is close to the underground mining height in shallow mining areas (within 80% of the mining height multiplied by the extraction ratio) and less (20%) at larger depths (Figure 5). Eg. if the mining height is 3m and the extraction ratio is 70%, the effective mining height is 2.1 m and the expected subsidence in shallow areas is 1.68m and 0.42m at larger depths.

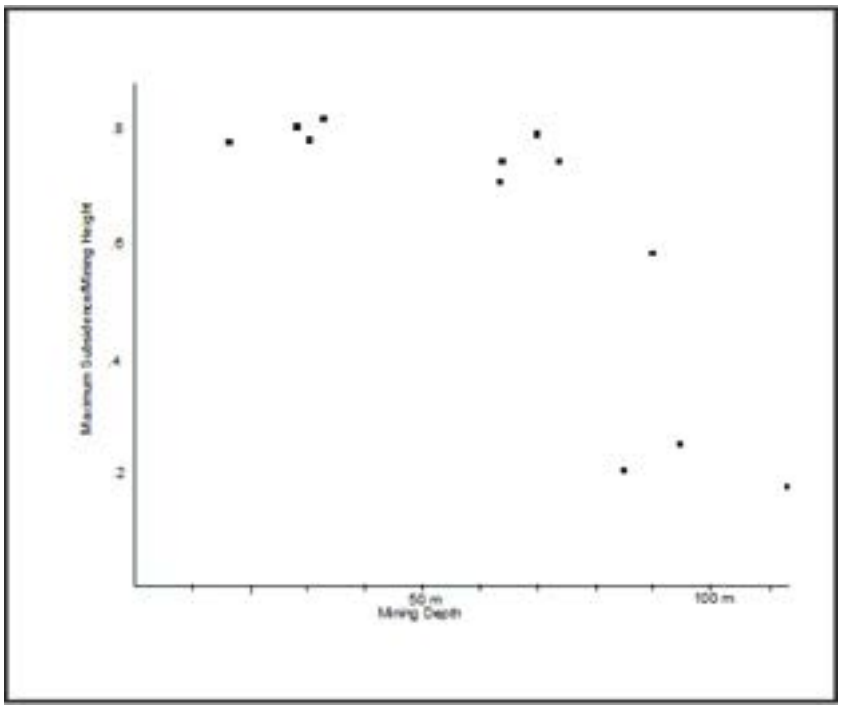


Figure 4: Ratio of maximum subsidence to mining height and the mining depth (vd Merwe, 2002)

Based on the information provided above, the following geotechnical issues that affect the type of subsidence include:

1. Overburden
   - composition / type of material (rock types, weak or strong layers)
   - rock mass properties (thickness, stiffness, etc)
   - quality (jointing, orientation, infilling, weathering, water, etc)
   - depth (pillar loading)
2. Orebody mined:
   - Thickness (mining method)
   - Strength (pillar strength / failure)
   - Properties (pillar failure mode, brittle, strain softening, etc)
   - Continuity (spans, pillar stress levels, pillar runs)
3. Mining method
   - is the result of the geotechnical environment (longwall, bord and pillar, panel and pillar, etc)

- regional pillars (restrict spans).



Where competent layers are present in the overburden, the subsidence may be partially delayed in that the total amount of subsidence is significantly less than expected. Care should be taken to understand mining underneath a competent layer may initially resulted in limited subsidence but can fail completely after a period of time causing substantial subsidence.



- Subsidence and Hydrologic Environmental Impacts

INTERESTING INFO

Subsidence oil fields

8.2. SURFACE PROTECTION

LEARNING OUTCOME 8.2.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS THE PROTECTION OF THE SURFACE SUBJECT TO SUBSIDENCE ABOVE SHALLOW TABULAR HARD ROCK WORKINGS

When subsidence is expected and design for, previous outcomes indicated that the major issues to be dealt with include:

- Vertical subsidence;
- Horizontal movement (strain);
- Tilt.

Often it is not the amount of vertical subsidence as the effects accompanying the subsidence (strain and tilt) that is the cause of damage but the amount of subsidence can serve to indicate the severity of the other concerns.

Vertical subsidence is accompanied by horizontal displacement of a magnitude of up to one-third of the vertical subsidence. Subsidence will also induce tilt of the surface, which is evident as a subsidence trough on surface if the magnitude of tilts is larger than the naturally occurring surface contours. The magnitude of the tilt is a function of the panel width, mining depth and the amount of vertical subsidence experienced.

Stacey et al derived subsidence limits for surface structures after doing significant investigations into subsidence in the central Johannesburg area. Table 1 through Table 5 gives the categories and limits derived through that process.

Surface structure	Category 1	Category 2	Category 3
Buildings	Brick and masonry structures with brick load bearing walls	Steel and reinforced concrete frame structures	Massive structures of considerable rigidity, including those of central core design

Surface structure	Category 1	Category 2	Category 3
Roads	Low volume gravel roads and paved roads	High volume paved roads with kerbs and substantial storm water drainage facilities	
Railway lines	Goods lines and low volume passenger lines on which speed limits can easily be imposed	Main lines and high volume passenger lines where it would be impractical to impose speed limits	
Pipes	Secondary sewers, water supply pipelines and stormwater drains and all steel pipelines carrying non-hazardous materials	Main sewers, water supply pipelines, stormwater drains and pipes carrying hazardous materials	

Table 1: Surface infrastructure subsidence limits

The following suggested limits of ground movements in the South African context is given by Stacey and Bakker (1992) and is shown below in Tables 6-2 through 6-5.

Building category	Damage level	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	Superficial	0.5	1.0
1	Hazardous	2.0	3.0
2	Superficial	2.0	1.5
2	Hazardous	4.0	3.5
3	Superficial	1.0	3.5
3	Hazardous	3.0	1.5

Table 2: Building categories

Road category	Damage level	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	Minor	2.0	10.0
1	Severe	5.0	15.0
2	Minor	1.0	5.0
2	Severe	2.0	10.0

Table 3: Road categories

Railway line category	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	3.0	10.0
2	2.0	5.0

Table 4: Railway line categories

Pipeline category	Horizontal strain (x 10-3)	Tilt (x 10-3)
1	2.0	4.0
2	1.0	2.0

Table 5: Pipeline categories

CONNECTION 303

Refer to "LEARNING OUTCOME6.2.3" on page 438

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REFERENCE

- Page 114, Chapter 9, Vd Merwe JN and Madden BJ, Rock engineering for underground coal mining, 2002
- Stacey, TR, Stabilising undermined sites: Case studies illustrate the sophistication of modern techniques, Journal. Geotechnical Engineering, SA Construction World – March 1983.
- Stacey TR, Interaction of underground mining and surface development in a central city environment.
- Stacey TR and Bakker D, The erection or construction of buildings and other structures on undermined ground, 1992

LEARNING OUTCOME 8.2.2

DESIGN VARIOUS APPROPRIATE PILLARS FOR THE PROTECTION OF THE SURFACE SUBJECT TO SUBSIDENCE ABOVE SHALLOW TABULAR HARD ROCK WORKINGS.

Subsidence can only prevented / limited in shallow workings by designing:

- stable pillar layouts: Bord-and-pillar or panel-and-pillar layouts where the pillars are of sufficient dimension and strength to support the overburden weight without yielding or failing.
- limited mining spans: Utilising regional pillar to limit spans and thus also the risk or levels of subsidence should pillar failure occur.

Both these issues have been addressed in previous outcomes.

CONNECTION 304

Refer to "LEARNING OUTCOME 4.2.1.1" on page 187, "LEARNING OUTCOME 4.2.3.1" on page 200, "4.3.1 STOPING LAYOUTS AND SEQUENCES" on page 233, and "STOPING EXCAVATION SUPPORT STRATEGIES" on page 287

CHAPTER 9

PAPER 3.1 HARD ROCK TABULAR MINING

LEARNING OUTCOMES

9. MINING STRATEGIES IN DIFFICULT CIRCUMSTANCES

9.1. SPECIAL AREAS

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

- Describe, explain and discuss the methods of determining hazards and the level of risk associated with mining excavations
- Describe, explain and discuss the methods of determining hazards and the level of risk associated with mining layouts
- Describe, explain and discuss the reasons and motivation for implementing special area procedures on mines
- Describe, explain and discuss the procedures associated with assessing and declaring special areas on mines
- Describe, explain and discuss the procedures associated with implementing and managing special areas on mines
- Describe, explain and discuss the measures and precautions to be applied in special areas in terms of :
  - Continuous risk assessment, Operational planning
  - Access control, Mining methods, Support methods
- Describe, explain and discuss the additional mining and support precautions to be taken in the following situations :
  - Mining in remnant areas
  - Mining adjacent to geological features
  - Mining through geological features
  - Workings that are about to be stopped
  - Workings that are to be re-opened after standing for some time
  - Workings that are to be re-opened after a collapse.

9.2. PRE-CONDITIONING

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

- Sketch, describe, explain and discuss how pre-conditioning is applied to relieve high-stress conditions in stopes at depth
- Sketch, describe, explain and discuss how pre-conditioning is applied to relieve high-stress conditions in tunnels at depth
- Describe, explain and discuss the theoretical basis for pre-conditioning with reference to :
  - Rock strength, Rockmass strength, Stress regime, Rock fracturing, Seismic environment

- Typical applications where pre-conditioning is required or indicated.

9.1. SPECIAL AREAS

INTRODUCTION

RISK AND HAZARD IDENTIFICATION AND TREATMENT

For the safe operation of any mining project a Code of Practice to Combat rockfalls and Rockbursts is required and needs to be administered according to the Mine Health and Safety Act guidelines. This entails undertaking the following processes:

- Conducting Risk Assessments in line with the Mine Health and Safety Act requirements and stipulations.
- Back analyses from accidents and incidents form part of the risk assessment undertaken by mines to designate areas as potentially high risk or low risk areas. Details of records of rock falls and injury causing rock related incidents need to be recorded and analysed to determine the root causes of the incident.

Data gathering in excavations is achieved by conducting mapping or observational exercises, logging of drill core from exploration drilling exercises and laboratory strength tests of rock samples. These methods can be used to designate ground control districts that represent areas of similar rock mass characteristics to be plotted on plans for easy identification by all mine employees. These are usually and should be based on known geological and geotechnical descriptions of the mining environment and noting how these affect local and regional support, stoping layouts, mining methods and mining direction and any other rock engineering strategies important for the mine's operation.

Having completed the Code of Practice to combat rockfalls and rockbursts by describing the full mining environment, identifying the different mining hazards and their potential mitigating measures, overall rock engineering strategies are called into action. These strategies may include:

- The overall stability of the mine and shaft systems. These involve regional support systems and overall mining layouts and sequences.
- Local support systems for stopes, tunnels and other large excavations, gullies, lead/lags and excavation sequence for large chambers.
- Rockburst control. Here seismic monitoring requirements, data analysis and incorporation of data into the mine planning process are considered.
- Continuous monitoring and reassessment of the mining methods is required to keep abreast with changes in mining conditions.
- Monitoring can vary from simple daily reporting of visual assessments of rock conditions, through routine closure and backfill porosity measurements to sophisticated measuring instruments. Rock Engineering practitioners need also to regularly confirm adherence to the stipulated mining layouts by analysing stope plans and carry out quality assessments on support installation.

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LEARNING OUTCOME 9.1.1

CONNECTION 305

Refer to Paper 2 Outcome 8.3 from page 607

DESCRIBE, EXPLAIN AND DISCUSS THE METHODS OF DETERMINING HAZARDS AND THE LEVEL OF RISK ASSOCIATED WITH MINING EXCAVATIONS

LEARNING OUTCOME 9.1.2

CONNECTION 306

Refer to Paper 2 Outcome 8.3 from page 607

DESCRIBE, EXPLAIN AND DISCUSS THE METHODS OF DETERMINING HAZARDS AND THE LEVEL OF RISK ASSOCIATED WITH MINING LAYOUTS

LEARNING OUTCOME 9.1.3

DESCRIBE, EXPLAIN AND DISCUSS THE REASONS AND MOTIVATION FOR IMPLEMENTING SPECIAL AREA PROCEDURES ON MINES

Conditions vary significantly on any mine, and it is affected by:

- Stress levels and changes
- Rock type exposed
- Seismic activity
- Mining depth
- Mining layouts
- Mining sequences
- Excavation dimensions
- Local geology, etc.

The development of standards or standard operating procedures assumes that conditions are standard or at most could be one a number of 'standard conditions'. This simplifies the mining environment to a 'predictive' environment, something it is known not always to be.

- Special area systems allows the rock engineer to set a method or process of identifying non-standard conditions so that additional precautions could be set to ensure stability and safety.
- The system suggests a method of investigation and evaluation or assessment of risk level, so that the precautions implemented are appropriate and meaningful.
- At the same time, special area systems provide a clear process whereby non-standards conditions could be reported, recorded, evaluated and addressed.
- The system documentation maintains good records of risk identified, precautions set and the success of the set precautions, so that future, similar hazards could be addressed with more confidence.

REFERENCE

- Page 101, Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999

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LEARNING OUTCOME 9.1.4

DESCRIBE, EXPLAIN AND DISCUSS THE PROCEDURES ASSOCIATED WITH ASSESSING AND DECLARING SPECIAL AREAS ON MINES

Procedures are dependent on the system implemented on any specific mine and should be known to all rock engineering personnel on the operation. However, some standards are provided to the Industry such as the DMR guidelines as set out in the Code of Practice guidelines shown below:

"The COP must set out a description of the strategies capable of identifying and treating an
increased risk of rock falls or rock bursts, which may develop during the course of routine rock excavation operations covering at least the following:

- conditions leading to the designation and classification of such areas as "Special Areas";
- description of additional/special attention or precautions measures;
- measures to effect rapid modifications to processes and/or procedures should such action
- require urgent review;
- description of the role and responsibility of the competent person in the identification of such special areas."

As long as the system adheres to these guidelines, the detail within the system could take on many forms.

EXAMPLE

An example is shown below where ratings are allocated to specific issues of concern which allows the calculation of risk levels and the implementation of additional precautions when risk ratings exceed some limit.

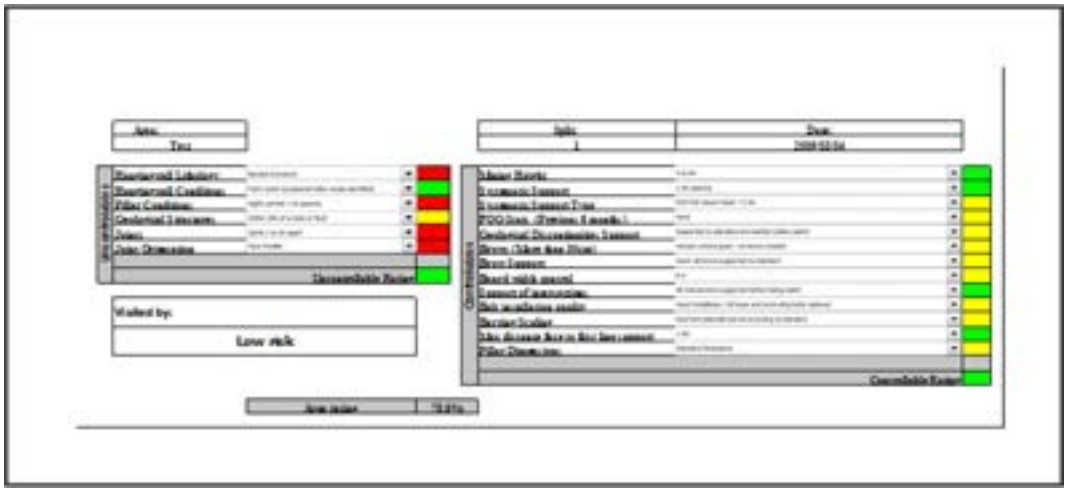


Figure 1: Risk rating system

This system allows the identification of high risk conditions or practices and allows:

- the derivation of appropriate remedial action for each working place, depending on the reasons for the high risk (Figure 1).
- tracking of risk levels over time (Figure 2) : Example of a mining area showing that controllable issues (support, making safe etc) is decreasing in quality i.e. increasing risk while conditions are improving as seen in the higher ratings for uncontrollable

(geology, rock types, etc)

- per issue (Figure 3): Example of tracking support installation quality showing a period in which the general quality decreased, but that it improved again at a later stage.

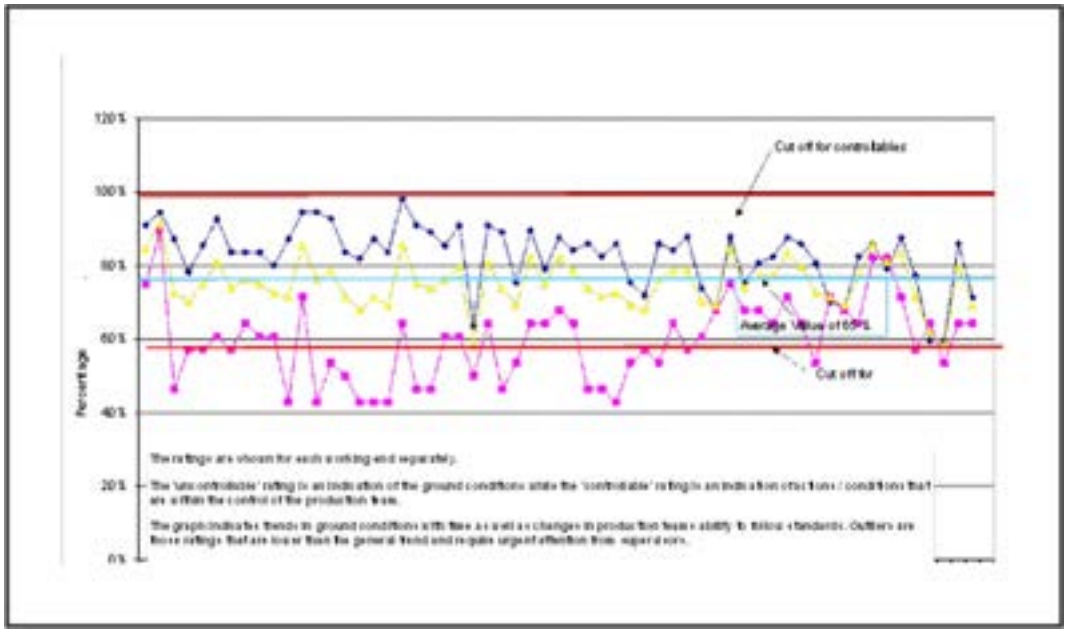


Figure 2: Tracking risk levels over time

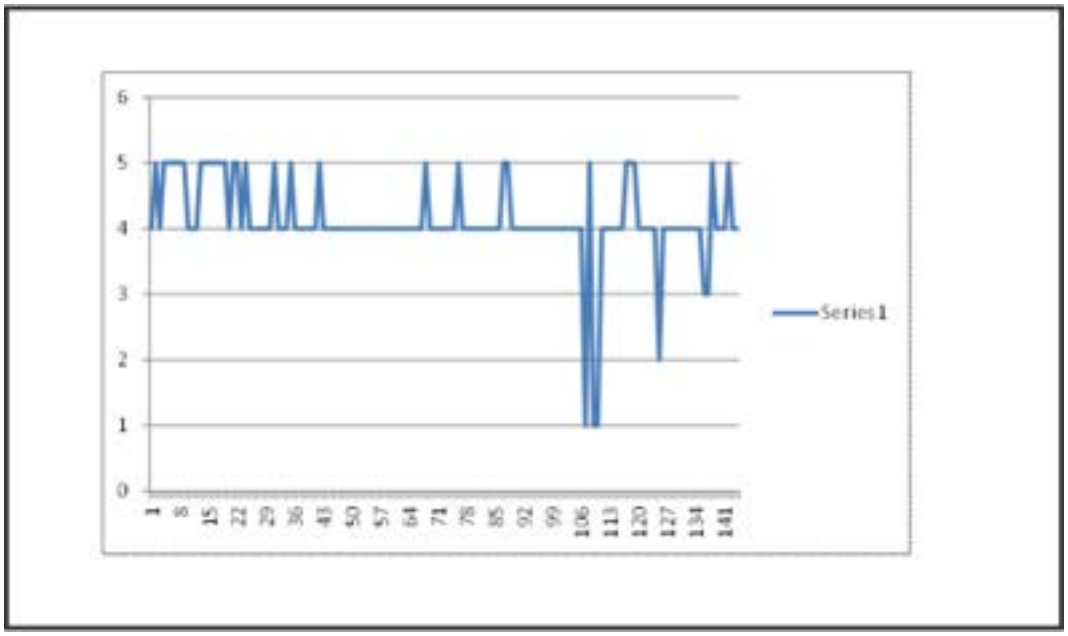


Figure 3: Tracking support installation quality

LEARNING OUTCOME 9.1.5

DESCRIBE, EXPLAIN AND DISCUSS THE PROCEDURES ASSOCIATED WITH IMPLEMENTING AND MANAGING SPECIAL AREAS ON MINES

Several types of special areas arise during the mining process and these include remnant extraction, the negotiation of geological structures, anomalous stress conditions, the re-opening of collapsed or rockburst damaged panels and environments requiring the use of caving procedures. These result in an increased risk for rockfalls and rockbursts and thus require special area treatment.

When special areas on a mine have been identified following the process described above there is a need to:

- Designate responsible persons to oversee the implementations of the strategies discussed above.
- Train all personnel of the pertinent rock mechanics issues that allow them to undertake their duties safely and effectively. These include explaining to them the reasons for undertaking particular procedures and the implications of not carrying out the stipulated procedures.
- Determine and communicate the role of the Rock Engineering department
- As well as the mine's Rock Engineering requirements.

The following example was borrowed and represents a possible system through which Special areas are declared and managed.



The system required is not prescribed, but the existence of an effective system to identify, report and remedy special conditions is. Many examples of different systems may exist, as long as they are effective in dealing with non-standard conditions.

EXAMPLE

The part below was taken from the COP while the Special Areas Procedure is attached.

Designation And Classification Of "Special Areas"
The risk rating form (RRF) indicates conditions leading to the designation of "Special Areas". Special Areas will be designated when the "uncontrollable factor" rating falls below 50%. This 'value' is based on empirical observations and will be adapted as the database and trends within the results increase (such adaptation will be documented in the Special Area SOP). This does not preclude the declaration of any area as a "Special Area" even though the RRF may not indicate it to be so.

A Special Areas Standard Operating Procedure in which special precautions and measures to be taken in areas deemed to be Special Areas should exist. A Rock Engineering Consultant must approve the Special Areas SOP.

Measures To Effect Rapid Modifications
It is important that rapid changes to processes and or procedures can be made in "Special Areas" and also that such actions be reviewed as a matter of urgency. This can include, but may not be limited, to the following:

- The Mine Overseer will be empowered to change, in consultation with the Production Superintendent and the Rock Engineering department, the mining sequence and support type and/ or pattern as dictated by conditions.
- The Mine Overseer will ensure that any changes to the precautions are communicated to the Rock Engineering Consultant immediately.
- The Mine Overseer will ensure that the provisions of the Special Areas Standard Operating Procedure as well as site specific instructions are understood and followed by all personnel in any section deemed to be a Special Area.
- The Mine Overseer will ensure that all necessary support units are available in good order and sufficient quantity.



- Special Areas Procedure

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CONNECTION 307

Refer to Paper 2 Outcome 8.3 from page 607

LEARNING OUTCOME 9.1.6

DESCRIBE, EXPLAIN AND DISCUSS THE MEASURES AND PRECAUTIONS TO BE APPLIED IN SPECIAL AREAS IN TERMS OF :

- Continuous risk assessment,
- Operational planning,
- Access control,
- Mining methods,
- Support methods

Continuous risk assessment
Panels are rated according to the rock mass in the area and risk assessments are repeatedly carried out in the mining area. Instrumentation installed in the area assists in quantifying the amount of damage or movement as a result of the new mining conditions. "Policemen" sticks, "tell tales", closure meters and load cells are some of the instruments that can be included in the stope to help continuously assess the rock mass.

Operational planning
This may entail reducing the production rate or output to cater for a slower mining approach that will:

- allow the workmen to concentrate on making safe and rehabilitating any damaged or emergent poor ground conditions or
- to affect the seismic response of the rock mass.

Mining ground may be sacrificed to remain as in-situ pillars to stabilise the rock mass and form pillars around the special area.

Different mining procedures and strategies will have to be set out and the crews working in the area have to be trained on the specific operational requirements of the special mining area and how they are different to the mine wide standards.

Economic losses due to changes in mining and operational procedures have to be analysed to be able to assess the viability of the process or where the system can borrow resources to cater for the planned loos in production.

Access control
Access records should be kept at the primary and secondary access to the special area working place, thus allowing control of personnel working in the area at any one time. The number of personnel working in special areas should be limited as far as possible to the maximum necessary required for the work planned.

Mining Methods
Change in mining method is usually necessary in the sense of changing the mining direction or approach to geological structures, reduction in production output, relocation and redirection of primary development, change of support requirements (spacing and pattern and type of support used) and focussing on re-establishing ground beyond the current bad ground areas.

Support methods
The support types used should be redesigned to suit the prevailing ground conditions observed in the area. This may entail the use of more

robust or stiffer support, a reduction of support spacing, combining different support systems not usually installed together to give the required resultant support resistance properties.

Ore or reef may have to be left in-situ to provide support and to separate the poor ground conditions from the new re-established ground. Often, support units providing area cover in the form of wire mesh, thin sprayed liners (TSLs), steel straps and in extreme cases shotcrete or gunite may be required to rehabilitate the special area.

LEARNING OUTCOME 9.1.7

DESCRIBE, EXPLAIN AND DISCUSS THE ADDITIONAL MINING AND SUPPORT PRECAUTIONS TO BE TAKEN IN THE FOLLOWING SITUATIONS :

- Mining in remnant areas,
- Mining adjacent to geological features,
- Mining through geological features,
- Workings that are about to be stopped,
- Workings that are to be re-opened after standing for some time,
- Workings that are to be re-opened after a collapse.

Mining in remnant areas

Remnants are blocks of ground surrounded by extensively mined out areas created in the final stages of mining out an area. As the remnant dimensions become smaller and critical sizes are achieved the ERR and ESS levels increase. The following additional precautions need to be carried out with regards to the mining of remnants:

- The stoping layout should be such that mining is directed towards the largest closest solid area.
- Mining away from geological structures and mining obliquely to geological features (>35 degrees).
- Avoiding splitting remnants and mining them from two different directions.
- Stop one approaching panel advance on the remnant avoiding panels approaching or mining towards each other.
- Avoiding large leads and lags between panels.
- Final change in mining direction for example from a down-dip to final up-dip mining.
- Limit the advances on the panels and blast the faces regularly to avoid time-dependent deterioration.
- Primary and secondary escape or access routes should be established for each remnant face.
- Use pre-conditioning to prevent high stress built up leading to rockbursts.



- Remnant Mining

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CONNECTION 308
Refer to Outcomes "LEARNING OUTCOME 3.1.19" on page 92, "LEARNING OUTCOME 3.1.20" on page 93, "3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS" on page 94, "3.3 ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS" on page 112, "4.4.1 Tunnel layouts" on page240,"TUNNELSUPPORT STRATEGIES" on page 311, "SUPPORT STRATEGIES FOR INCLINED SHAFTS" on page 347

CONNECTION 309
Refer to Outcomes "LEARNING OUTCOME 4.3.1.3" on page 238, "LEARNING OUTCOME 4.3.1.4" on page 240

Remnant mining approach

Mining adjacent or through geological structures

Geological structures are high risk from a rockfall but also rockburst point of view. The expected risk needs to be clarified prior to mining through or adjacent to such a structure. Experience has shown that faults with small throws (typically < 10m) and thin dykes (typically < 5m) are not seismically active but pose a rockfall risk only.

Note:
This does not indicate that such structures can not be seismically active, but indicate a general trend only.

For a rockfall risk:

- the concern is the fault plane orientation and infilling type as well as the presence of water;
- these issues affect the fractional strength of the plane and thus potential for a collapse under gravity;
- support installation to increase frictional resistance or ensure deadweight support of the rock is essential;
- continuous monitoring is important to note the effectiveness of the support.

For rockburst risk:

- avoid mining through the structure and if it must be done, do so at small mining spans and lowest possible stress levels;
- leave bracket pillars along structures to limit potential for seismicity and avoid exposure to the structure in the case of an event occurring;
- control rate of mining as well as direction of mining around the structure;
- mine areas around the structure first or at the lowest possible stress levels;
- write off ore adjacent to the structure if risk is perceived to be too high.

CONNECTION 310
Refer to Outcomes "3.2 ROCKMASS BEHAVIOUR AROUND ACCESS AND SERVICE TUNNELS" on page 94, "3.3 ROCKMASS BEHAVIOUR AROUND SERVICE EXCAVATIONS AND SHAFTS" on page 112, "4.4.1 Tunnel layouts" on page240,"TUNNELSUPPORT STRATEGIES" on page 311, "SUPPORT STRATEGIES FOR INCLINED SHAFTS" on page 347

Workings that are to be stopped or re-opened after standing for some time or after a large collapse

Re-raising around the extreme ground conditions area is the procedure usually employed to reopen previously standing or collapsed ground. The actual positioning of the on-reef development is determined largely by the stress conditions and magnitudes ahead of the stope, the fracture zone and general ground conditions. The pillar left between the re-raise and the stope face should be small enough to be able to yield slowly in a stable manner or be large enough not to become burst-prone. The re-raise position is best positioned in a position ahead of the stope face in virgin stress conditions.



- Page 101, Jager AJ and Ryder JA, Handbook on rock engineering practice for tabular hard rock mines, 1999
- Chapter 2, Ryder JA and Jager AJ, A textbook on rock mechanics for tabular hard rock mines, 2002
- Chapter 2, Hoek E and Brown ET, Underground excavations in rock, 1980

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- Chapter 3, Brady BHG and Brown ET, Rock mechanics for underground mining, 2006
- Chapter 2, Stacey TR, Best practice rock engineering for other mines, 2001

9.2. PRE-CONDITIONING

INTRODUCTION

CONNECTION 311

Refer to Outcome "LEARNING OUTCOME 3.4.2" on page 142
Refer to Paper 2 Outcome 7.1.8 on page 549, 7.3.1 on page 560

Pre-conditioning is a mining process used in deep level mines in South Africa to mitigate the build-up of high stress at the mining panel faces, thus reducing the rockburst hazard.

Preconditioning extends the position of high stress build-up and stress induced fracturing further ahead of the mining face and away from the immediate mining face (Figure 4, Courtesy Snowden).

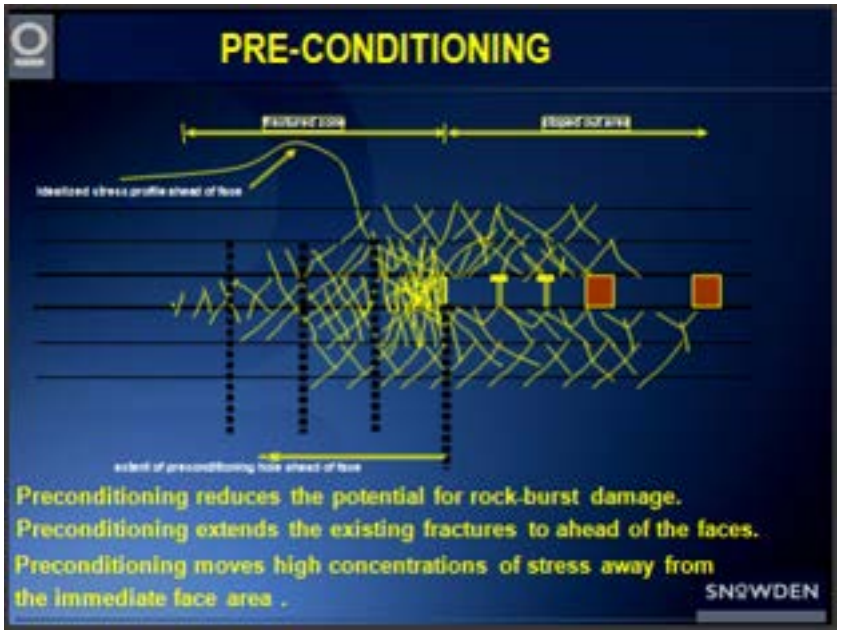


Figure 4: Use of pre-conditioning in high stress mining environments (Courtesy Snowden)

Depending on the daily primary blast advance and mining set-up, pre-conditioning holes are usually drilled one-and-half (1.5 x primary blast length) times longer than the primary blast hole length. This entails at least two blasts in de-stressed ground with renewal of the preconditioning blast on the third production blast. A pre-conditioning charge is stemmed and bottom charged with back priming. The rest of the primary blast and preconditioning holes to the collar are stemmed or tamped.

Charging up
The following charging process is recommended for an effective pre-conditioning blast:

- Preconditioning holes must be charged up with the same explosives as the normal production round.
- The holes are to be charged up from inside to shothole limit.
- The hole must be primed from the top.
- The remainder of the hole must be tamped.

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Timing

It is critical for the effectiveness of pre-conditioning and to achieve its full benefits to blast the panel face daily avoiding strain and stress build-up and time-dependent deterioration of the hanging and panel walls. The following timing requirements are given:

- The same fuses must be used for preconditioning holes as for the normal production holes.
- The precondition holes must be detonated 4 holes prior to the normal production holes.
- All preconditioning hole positions must be noted and recorded.

Direct benefits of the use of pre-conditioning include reduced face bursts, improved face blast advances and improved hanging profile rock mass conditions, Figure 5.



Figure 5: Comparison of ground conditions in mining panels without (a) and with (b) pre-conditioning

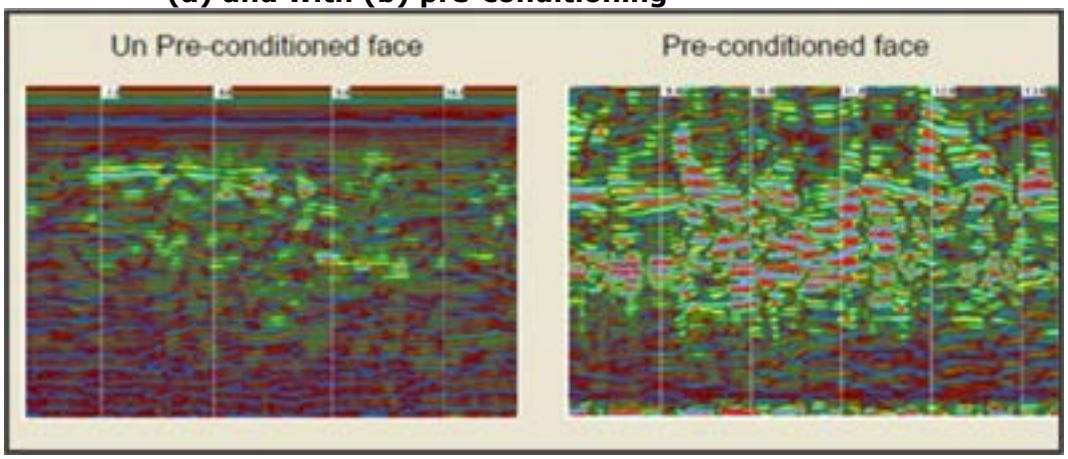


Figure 6: Radar representation of fracture density without and with preconditioning (Courtesy Anglogold Ashanti)



- Gap336 Preconditioning
- Gap811 Preconditioning booklet

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LEARNING OUTCOME 9.2.1

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS HOW PRE-CONDITIONING IS APPLIED TO RELIEVE HIGH-STRESS CONDITIONS IN STOPES AT DEPTH

Two types of preconditioning can be distinguished, namely Face Parallel preconditioning and Face Perpendicular preconditioning

Face parallel preconditioning involves the setting off of a designed blast ahead of the stope face via blast holes, longer than the panel length, that are drilled parallel to and at a certain distance ahead of the from the gully of the leading panel.

Common practice includes the excavation of a cubby in the gully, before the hole is drilled. The hole can lead the face by a minimum of 3.5m, and a maximum of 5.5m ahead of the face (this is for an 89mm hole). The pre-conditioning hole can be drilled to extend the entire length of the panel and even propagate into the lagging panel.

- Drilling
- The holes must be drilled parallel to the face and inclined slightly for draining the sludge.
- The preconditioning holes must be longer than the face length to ensure preconditioning along the entire face
- The holes should be drilled between 3.5m and 5.5m ahead of the panel face for optimal blasting
- The holes must be drilled as close to the reef plane as possible
- The holes must be more than 50cm from the hanging wall contact to prevent blasting damage to the hanging wall.
- Charging up
- The holes must be stemmed properly

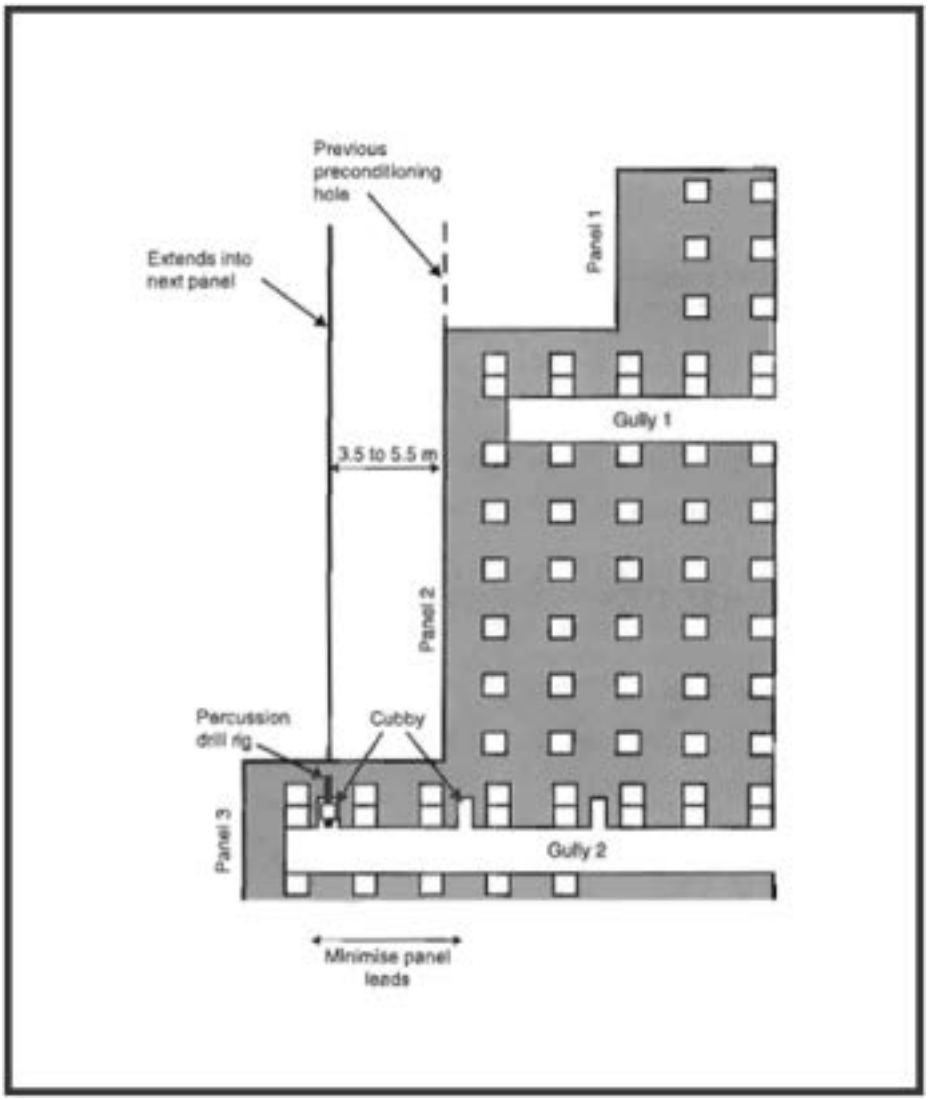


Figure 7: Face-parallel preconditioning method (Jager and Ryder, 1999)

Face perpendicular pre-conditioning involves the setting off of a designed blast ahead of the stope face via blast holes longer than the normal production holes that are drilled into the face at a certain burden.

- Drilling
- The preconditioning holes should be twice as long as the normal shot holes
- The preconditioning holes should be the same diameter as the shot holes.
- The preconditioning holes should be drilled in the plane of the reef
- The maximum burden should be 3,0m as the effective preconditioning zone is about 1.5m in diameter.
- Charging up
- These holes are charged up and blasted with the production rounds.

This method is well suited to the mining of long and narrow strike pillars as well as remnants. This method consists of drilling 3m long holes, spaced 3m apart perpendicularly into the face. The timing of these holes is set-out so that these holes will detonate, slightly before the production round. Due to the length of these pre-conditioning holes, they create a pre-conditioned zone ahead of the face.

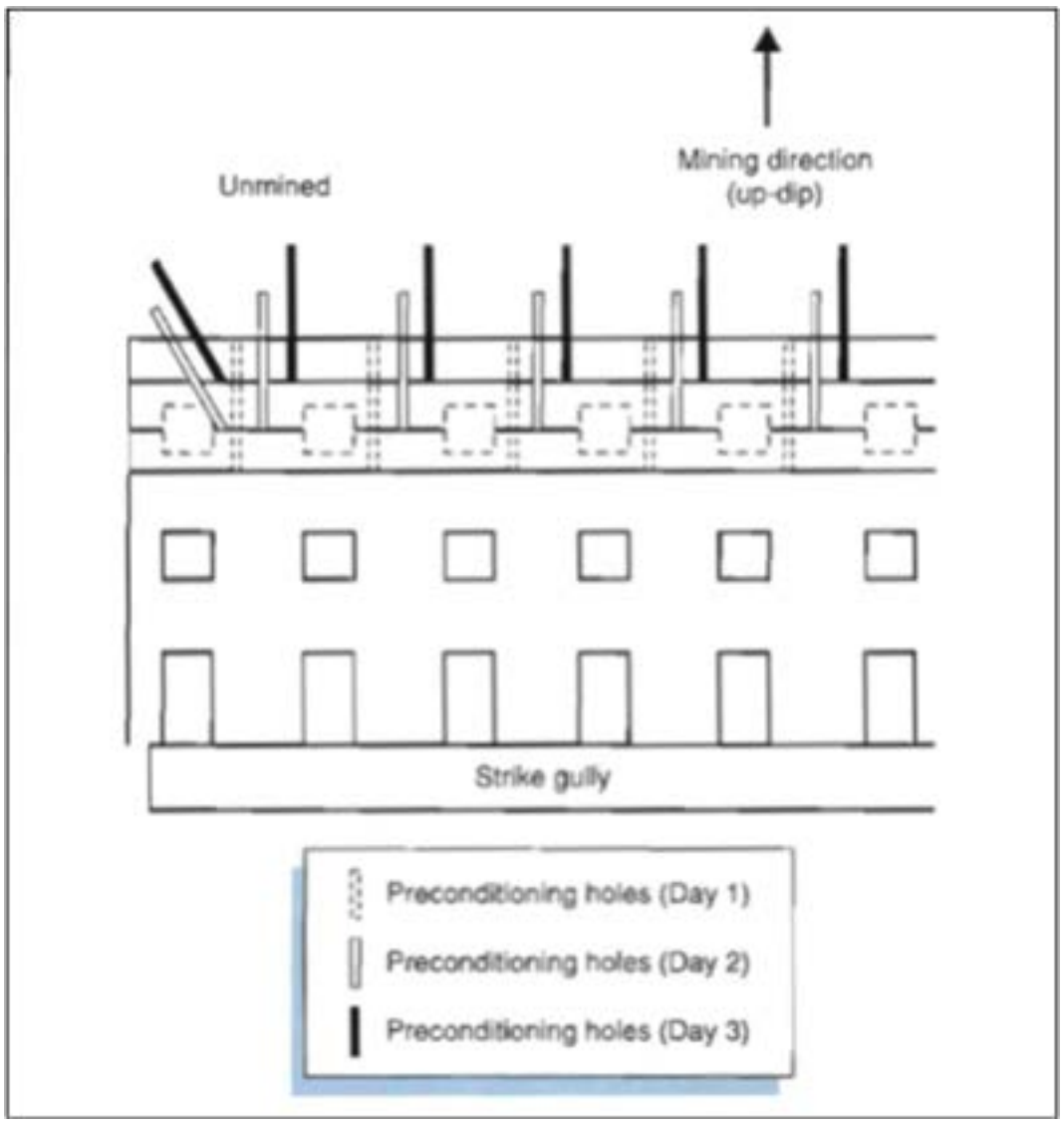


Figure 8: Face perpendicular preconditioning (Jager and Ryder, 1999)

The critical features of pre-conditioning are the dimensions of pre-conditioning holes, as well as the manner in which the blast is carried out. If these two aforementioned features are not correctly monitored and implemented the pre-conditioning may be unsuccessful.
It must also be noted that pre-conditioning may affect the production cycle and require additional resources. If pre-conditioning isn't carried out correctly, it will be to the detriment of safety as well as production.
Various factors could influence the effectiveness of pre-condition and include;

- Drilling discipline
- Blasting practices (explosives)
- Timing of blast holes
- Spacing of pre-conditioning holes along the face
- Position of pre-conditioning holes in relation to production holes.

REFERENCE

- Stope preconditioning
- Chapter 8 , Ryder JA and Jager AJ, A textbook on rock mechanics for tabular hard rock mines, 2002

LEARNING OUTCOME 9.2.2

SKETCH, DESCRIBE, EXPLAIN AND DISCUSS HOW PRE-CONDITIONING IS APPLIED TO RELIEVE HIGH-STRESS CONDITIONS IN TUNNELS AT DEPTH

Preconditioning was developed for deep level stoping to relieve face bursting conditions. The idea of preconditioning is to use explosives to damage the rock mass and reduce its load carrying capacity. The high stress region ahead of the face migrates into the deeper solid rock and the damaged, preconditioned rock provides a buffer of crushed rock that is less likely to burst at the face. By all accounts this works and provides better drilling conditions if there is minimum delay between the preconditioning and the advance.

There is however, very little information on preconditioning in development ends. The preconditioning around a tunnel through a dyke at Kloof mine:

- Drilling four additional 4 m holes into the tunnel face.
- The holes were tamped with 2.5 m of clay and charged with 1.5 m of ANFO type explosives.
- Following some minor footwall strain bursting after the first preconditioning blast an additional two preconditioning holes, angled towards the footwall from the centre area of the tunnel face were included in each preconditioning round.



- Tunnel preconditioning

LEARNING OUTCOME 9.2.3

DESCRIBE, EXPLAIN AND DISCUSS THE THEORETICAL BASIS FOR PRE-CONDITIONING WITH REFERENCE TO :

CONNECTION 312

Refer to "LEARNING OUTCOME 9.2.1" on page 517 "LEARNING OUTCOME 3.4.2" on page 142
Refer to Paper 2 Outcome 7.1.8 on page 549, 7.3.1 on page 560

- Rock strength,
- Rockmass strength,
- Stress regime,
- Rock fracturing,
- Seismic environment

Typical applications where pre-conditioning is required or indicated

The mechanism of preconditioning is aimed at preventing the accumulation of stress ahead of the stope face. As mining continues, fractures that exist in the face may be prevented from sliding past one another due to asperities. This lock-up allows for strain energy to be stored in the rock, which when released can result in an ejection of rock into the working place, creating the potential for a fatal accident.

- Preconditioning is thus a technique to reduce the probability of face burst by negating excessive accumulations of strain energy in the rock ahead of the stope face. Although the use of preconditioning is not precluded in any other areas, it should be employed where the ratio of strength to stress is expected to be in the proximity of 1 (range of 0.9 to 1.5).
- Face-parallel preconditioning is suggested for special areas, such as remnant or pillar extraction, as it can be more effective than

the face-perpendicular method for dealing with the exceptionally high stresses encountered.

- The face-perpendicular preconditioning is an integral part of the actual production blast cycle. Face perpendicular preconditioning is suited to normal production faces as it integrates well into the mining cycle.

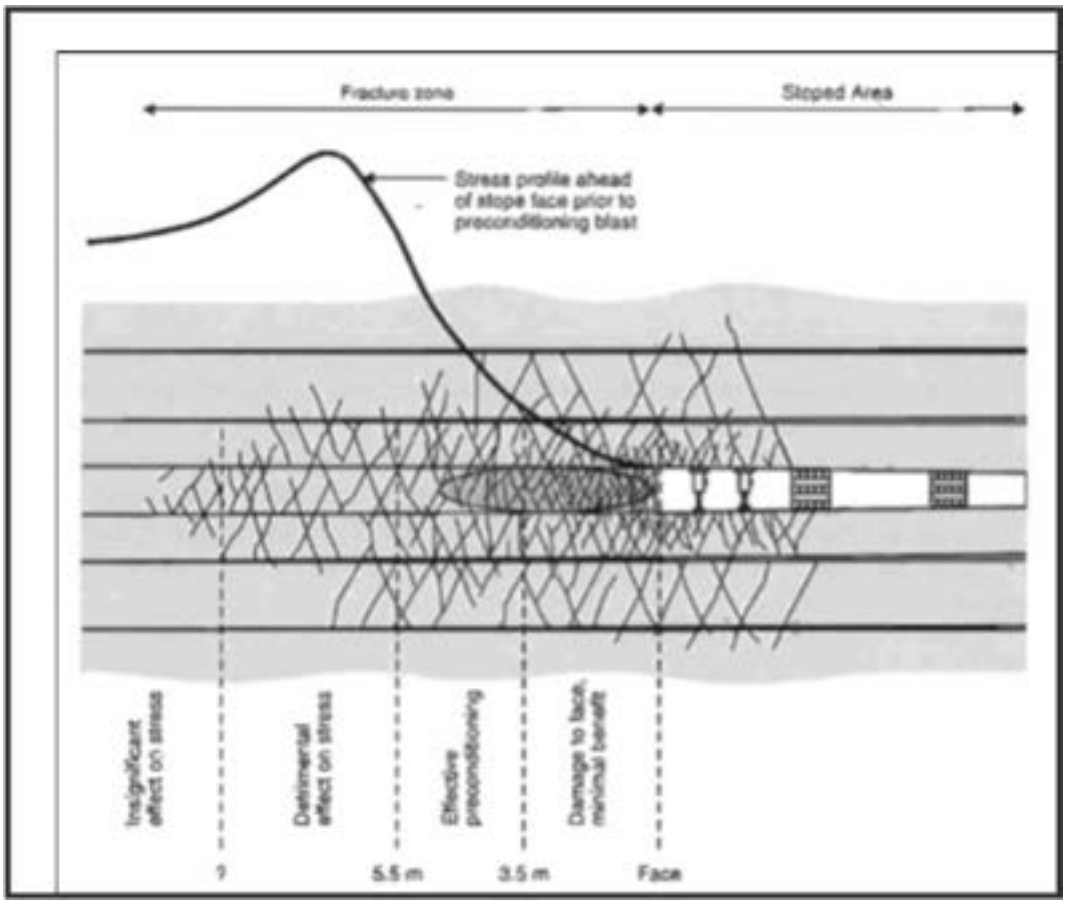


Figure 9: Stress profile ahead of a pre-conditioned face (Jager and Ryder, 1999)



- Chapter 8., Ryder JA and Jager AJ, A textbook on rock mechanics for tabular hard rock mines, 2002

BRIDGING MATERIAL

CONTENTS

- Absolute values
- Constants
- Units
- Signs and symbols
- Metric pre-fixes
- Glossary
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  - 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

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BRIDGING MATERIAL

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

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~ 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.

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BRIDGING MATERIAL

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.

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BRIDGING MATERIAL

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 ($CaCO_3$).
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 (SiO_2) 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.

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BRIDGING MATERIAL

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.

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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.

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- 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!).

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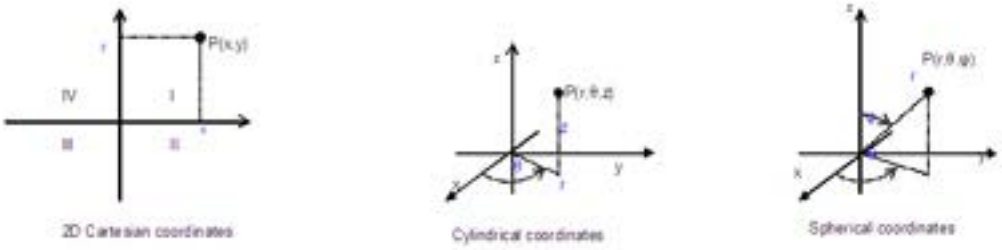
- The quadratic formula offers the general solution for problems of the form $ax^2+bx+c=0$ where a, b and c are numbers and x is the unknown: $x_{1,2}=(-b\pm(\sqrt{b^2-4ac}))/2a$. The two solutions x_1 and x_2 are obtained by adding and subtraction the square root term, respectively.
- In the above example, 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).

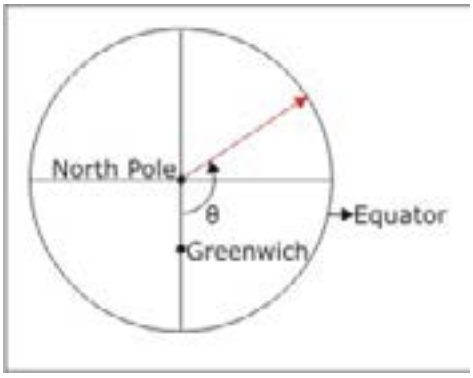
Polar co-ordinates or cylindrical co-ordinate systems are more suitable. for problems involving spherical or cylinder shaped objects and curved paths. The geographical coordinates describing locations on the Earth's surface are an example of such spherical coordinates:
Longitude is measured in degrees 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.

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BRIDGING MATERIAL

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°.

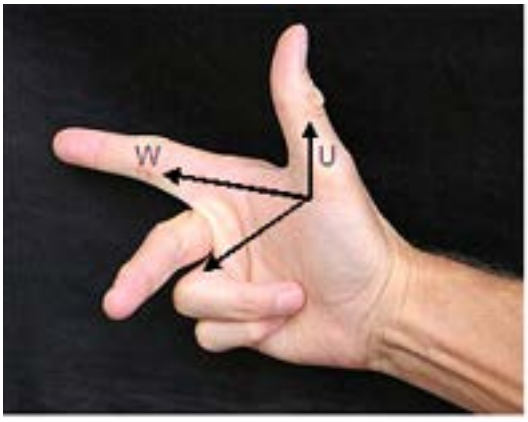


Looking down onto Earth from above the North pole: The definition of longitude θ.

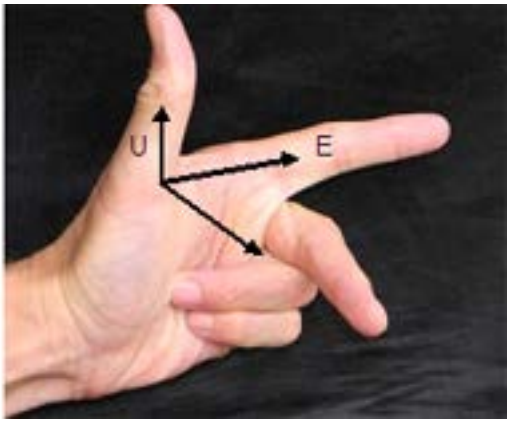
In many cases, mine coordinate systems are a special case of a 3D Cartesian coordinate system whose origin is defined by a geographical location. Their axes refer to geographical directions such as South/West/Up (SWU), but the definition of South=0, West=0 and Up=0 is often specific for a single mine 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)

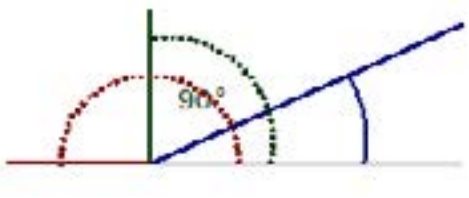
Examples: Are NWU, ESD and SED left- or right-handed?

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.

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ANGLE MEASUREMENT

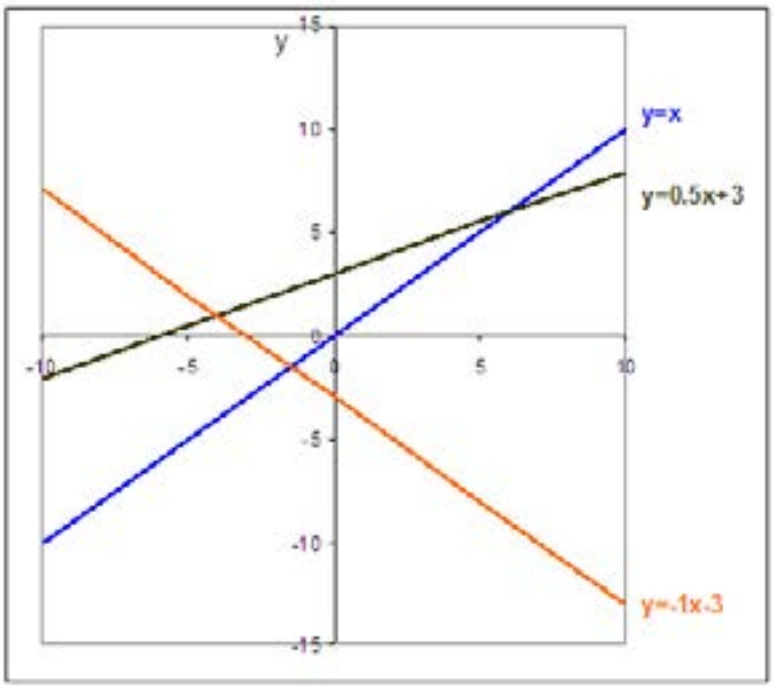
An angle Φ as shown in the sketch below is a measure of how much two lines diverge (how the distance between them grows). The greater the angle, the greater the divergence of the lines. Angles are measured in degrees (°) or in radians (rad). There are 360° or 2π rad in a full circle.
Two straight lines 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.

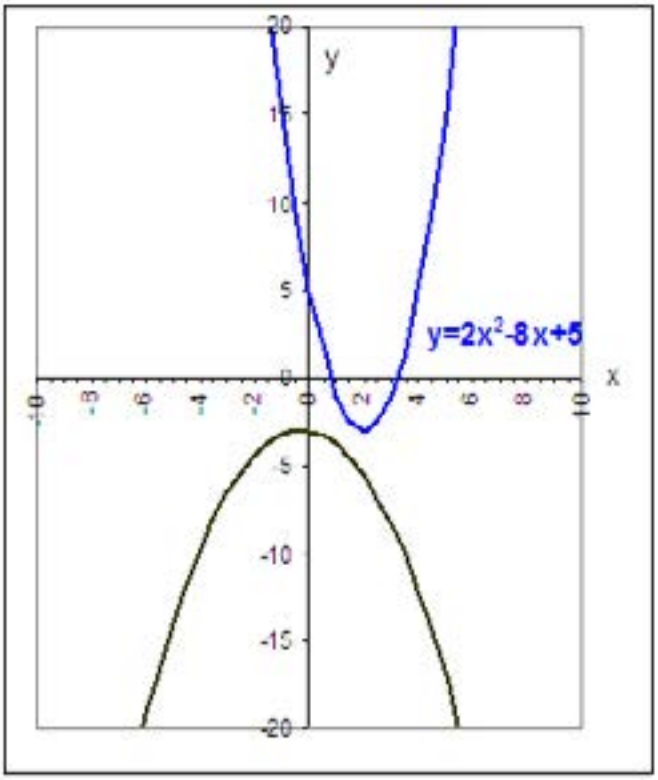


y=x: slope 1, offset 0
y=0.5x+3: slope 0.5, offset 3
y=-1x-3: slope -1, offset -3.

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

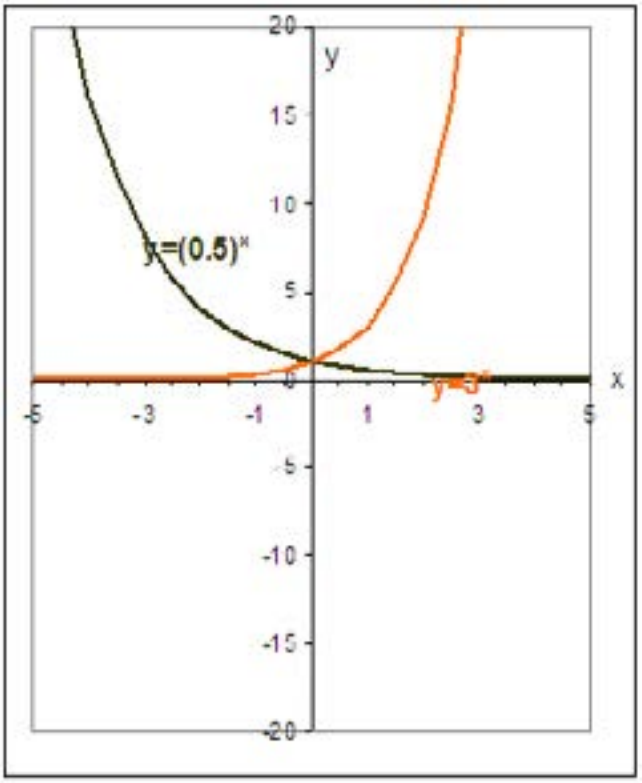
First to be noted is the difference between $f(x)=x^2$ and $g(x)=2^x$. The former was introduced above as a quadratic function. The latter, with base 2 and the independent variable x as the exponent, is called an exponential function. Their general form is b^x .

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

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:
$b^x b^y=b^{x+y}$; $(b^x)^y=b^{xy}$; $b^x/b^y=b^{x-y}$

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BRIDGING MATERIAL

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 $x=2^y$.

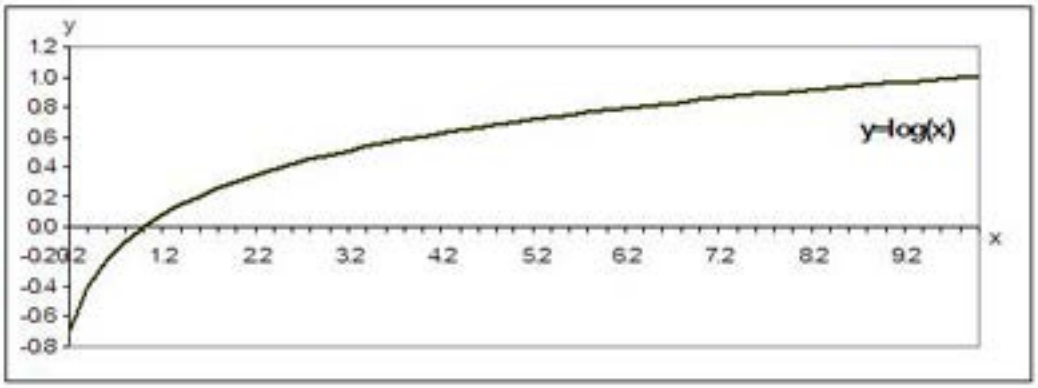
The definition of logarithmic functions is valid for all $b>0$ and $b\neq 1$ and is linked to their respective bases:

- $y=\log_b x$ 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.

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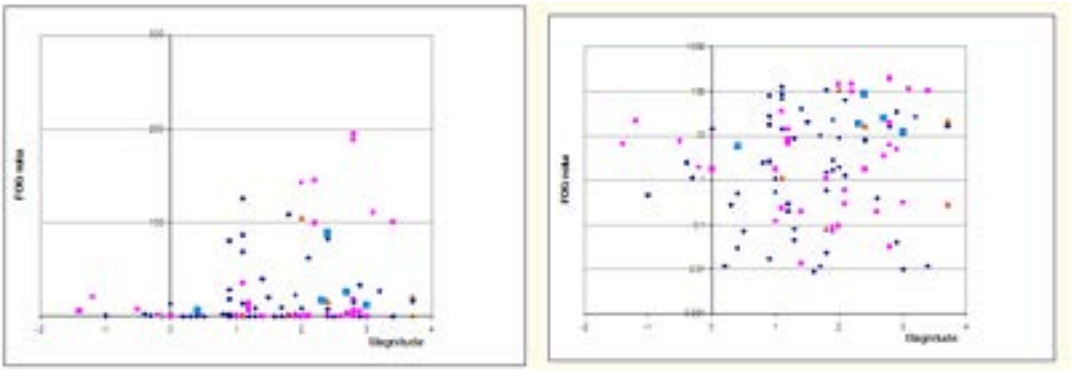


Figure 1: Rockburst magnitude plotted against FOG volume: linear scale
Figure 2: Rockburst magnitude plotted against FOG volume: logarithmic scale

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):

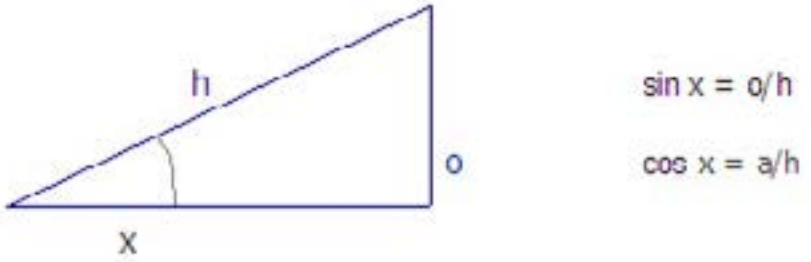
- $\log_{(mn)}=\log_{(m)}+\log_{(n)}$
- $\log_{(m/n)}=\log_{(m)}-\log_{(n)}$
- $\log_{(m)}p=p*\log_{(m)}$
- $\log_{(1)}=0$

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

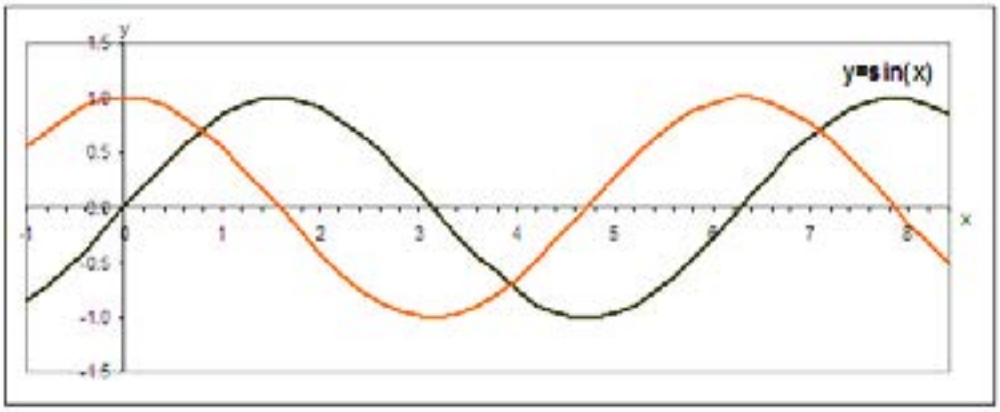
The most commonly used trigonometric functions are sine and cosine: sin(x) and cos(x) respectively. Both of these are periodic meaning that they are fully described for values of x between 0° and 360° (or 0 and 2π rad), and repeat themselves for x values outside of this range. For the simplified case 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.



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:

$$\sin 2x + \cos 2x = 1$$
$$\sin(\alpha \pm \beta) = \sin(\alpha)\cos(\beta) \pm \cos(\alpha)\sin(\beta)$$
$$\cos(\alpha \pm \beta) = \cos(\alpha)\cos(\beta) \mp \sin(\alpha)\sin(\beta)$$

CUBIC EQUATION
No info available as yet.

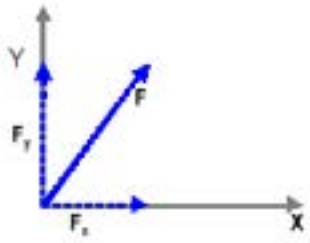
ABSOLUTE VALUES

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.

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

$a \bullet b=|a||b|\text{Cos } \varphi$

where φ is the angle between a and b.

Vector product

$a \times b = c$

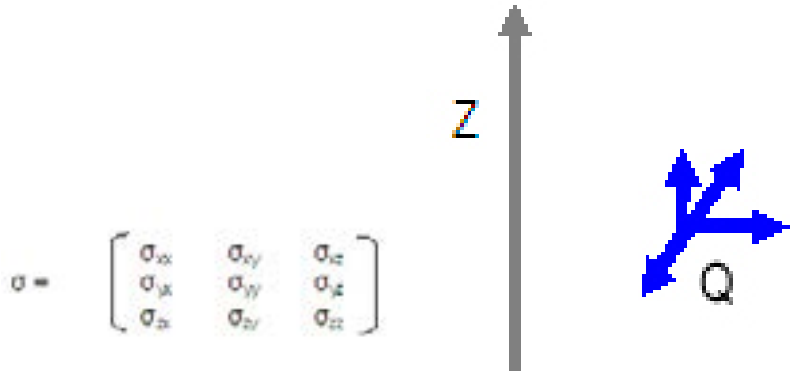
where c is a vector of length $|a||b|\sin\varphi$

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:



Stress and Strain

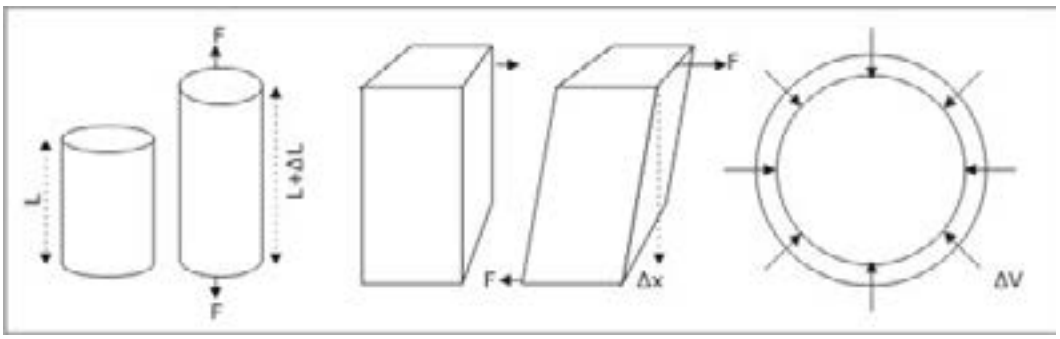


Figure A Figure B Figure C

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 ΔV/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).