

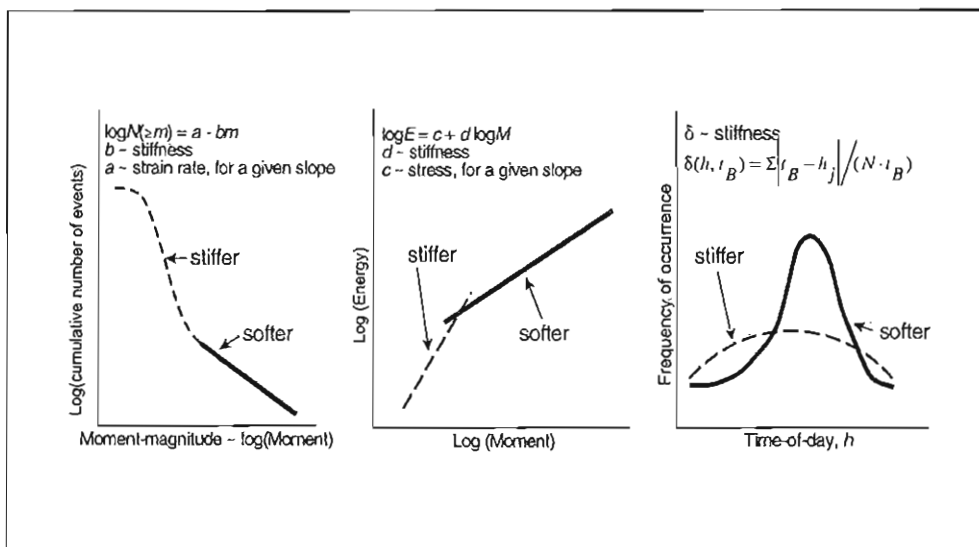


Figure 9.5.3 An illustration of a cumulative frequency-magnitude plot of a mixed data set over a limited period of time (*left*). Events associated with a stiffer system (broken line) and with a softer system (solid line) frequently have different slopes. On a log Energy vs log Moment plot (centre) the same applies. Intermediate and larger events associated with softer systems tend to occur within a few hours of blasting time t_B , while a stiffer system responds moderately but with larger statistical dispersion (right), as measured by the normalised standard deviation δ .

Seismic hazard derived from the size distribution of seismic events may not be adequate to quantify and to manage the exposure to seismicity, due to the differences in time-of-day distribution of intermediate and large events associated with different mining scenarios, see Figure 9.5.2 as an example. In general, stiffer systems/layouts are characterised by a lower m_{max} but with a time-of-day distribution having larger statistical dispersion in relation to the time of blasting, thus they are less time-predictable. Softer systems, on the other hand, have larger m_{max} but they trigger or induce most events during the few hours after blasting.

If $\bar{t}(\geq m, t - t_B) = (t - t_B) / N(\geq m, t - t_B)$, is the average time between events with magnitude not smaller than m at the time t after the time of blasting t_B , then the probability of having an event at that size in a small time interval $t + \Delta t$ is approximately $P(\geq m, t + \Delta t) = \Delta t / \bar{t}(\geq m, t - t_B)$.

Seismic exposure at a given hour of day, $SE(h)$, averaged over Δt can be estimated by the product of the average frequency of potentially damaging events $N(\geq m, h) / \Delta t = 1 / \bar{t}(\geq m, h)$, and the average number of people exposed at the time, $npe(h)$

$$SE(h) = npe(h) / \bar{t}(\geq m, h).$$

The daily SE can then be taken as a sum

$$SE = \sum_{h=1}^{24} npe(h) / \bar{t}(\geq m, h).$$

9.6 GENERAL GUIDELINES FOR INTERPRETATION

Mining excavations induce considerable gradients in strains and stresses and in their rates, which the rock mass continuously reduces, mainly by inelastic deformation. This process is strongly time-dependent, e.g. the relaxation time changes over a few orders of magnitude within a short distance from the excavation faces. Such strong spatial and temporal gradients are conducive to the development of excess stresses which, if not diffused, result in large and/or strong seismic events.

Analysis and interpretation of seismic events:

The magnitude and the strength of the event: The occurrence of a larger (in terms of its seismic moment) and stronger (in terms of its stress release) seismic event increases the potential for rockbursting in the vicinity of the event. The higher the stress release, for a given magnitude, the higher the hazard.

The occurrence of an unexpected event: The occurrence of a larger seismic event not predicted spatially and/or temporally by numerical modelling should raise concern.

Event mechanism: The seismic moment tensor solution, if available, may assist in identifying the specific structure involved in the event, and in confirming the results of numerical modelling by correlating the expected and the observed directions of principal stresses in the area and the expected and the observed isotropic and deviatoric components of the coseismic strain change.

Analysis of seismicity in space - contouring. The main objective of spatial analysis of seismicity is to delineate the areas/volumes of concern from a stability point of view. Frequently, larger seismic events occur or are initiated at 'spots' of significant spatial stress and/or strain-related gradients.

Spatial or 2D contours of stress-related parameters is the simplest, and in many cases, a very effective, way of delineating sites of increased rockburst hazard. Contours of Energy Index (*EI*) are sensitive indicators of stress variation.

Routine spatial analysis can be improved if seismic stress and strain indicators are viewed simultaneously. Frequently, larger events occur at sites of high strain gradients, characterised by high stress. The situation can be recognised where contours of, say, seismic strain, seismic viscosity or seismic Deborah number are closely spaced and, at the same time, superimposed contours of *EI* show high values.

The time span over which such analyses are done depends on the nature of the rock mass behaviour and the scale of the analysis. It is advisable to limit the time period over which the strain-related parameter is contoured to not more than the seismic relaxation time. For the stress-related parameter a shorter period is required, since the cumulative strain over a period of time is to be compared with the present state of stress.

Analysis of seismicity in time. The main objective of temporal analysis of seismicity is to establish and, if possible, to quantify the imminence of potential instability. Frequently, larger seismic events are preceded by an increase in stress followed by its decrease and accelerated coseismic deformation - see Figure 9.6.1. Consequently, one may observe a decrease in seismic viscosity and an increase in diffusivity d_s ,

resulting in a sharp drop in seismic Schmidt number before instability. Since the dynamics of the preparation and nucleation of rock mass instabilities is not well understood, unexpected strong changes in seismic parameters in any direction should raise immediate concerns - see Figure 9.6.2 as an example.

Cumulative apparent volume (ΣV_A): The slope of the ΣV_A curve sensitively reflects changes in strain rate. Accelerating deformation over a period of time is an indication of unstable rock mass deformation. Larger events stand out as jumps in the cumulative curve without distorting the scale, as is the case with cumulative seismic energies or moments. In addition, its time history does not have to be treated with statistical smoothing. It is therefore a useful reference curve on a time history plot when comparing the behaviour of other parameters requiring filtering procedures. The time history of seismic viscosity similarly measures the nature of the seismic rock deformation, and a decrease in the value of this parameter reflects softening. It is sensitive to the size of the moving time window used.

It is recommended that seismic strain-related parameters in spatial and/or temporal analysis be compared with volume mined V_m .

Energy Index: A moving median EI has been proven a reliable and sensitive indicator of stress variation both in space and time. Using a linear scale, a time plot of EI does, by definition, emphasise above-average values. If the purpose is more general, i.e. to monitor variation above and below average, $\log(EI)$ is the recommended parameter.

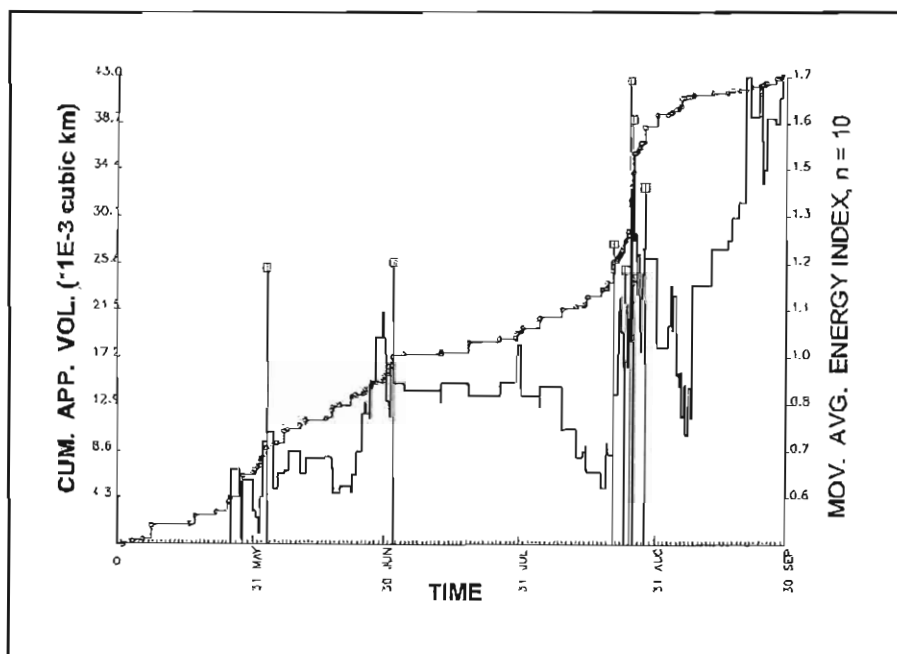


Figure 9.6.1 Cumulative apparent volume (thick line) and moving average energy index (thin line) versus time for mining an area surrounding a dyke in Welkom, South Africa. The largest seismic events with $m \geq 1.7$ are marked by square capped vertical lines. (Reproduced with kind permission from Kluwer Academic Publishers)

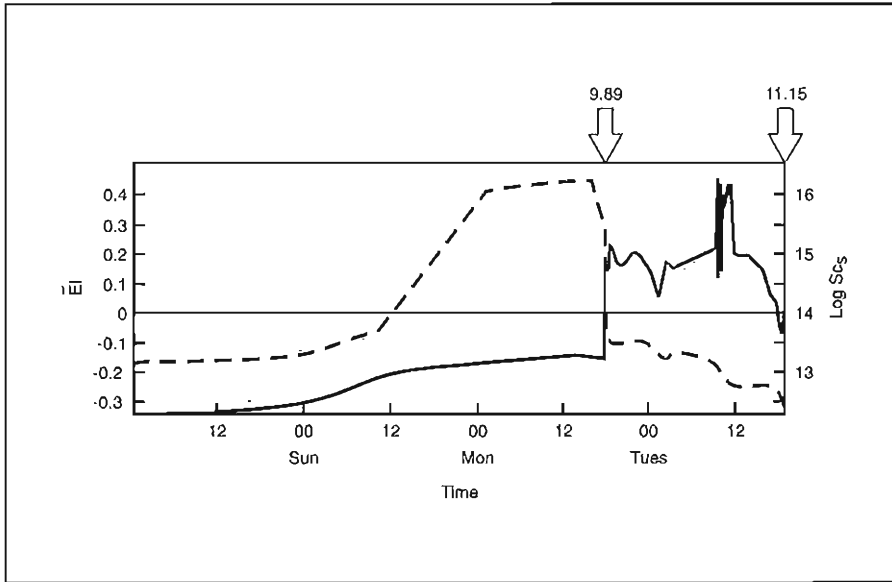


Figure 9.6.2 Time history of a moving median energy index, $\bar{E}$, for the 90 hours preceding a magnitude $m = 4$ event ($\log E = 11.15$) in a South African gold mine. An increase in $\bar{E}$ associated with the $\log E = 9.89$ event on Monday late afternoon had been maintained for approximately 18 hours when another sudden increase occurred (late Tuesday morning). At that point the decision to evacuate the area was taken by the management on the advice of the mine seismologist. The dotted line shows the behaviour of the seismic Schmidt number.

Activity rate: Since very small events do not contribute significantly to seismic strain and cannot significantly change the state of stress, their occurrence may be missed by analyses which concentrate on overall strain rate and average stress level. Specific attention to activity rate could help to recognise anomalous patterns in rock mass behaviour. Any sudden unexpected increases in activity rate should raise concern. For meaningful interpretation of activity rate, the seismic network sensitivity must remain constant.

Spatio-temporal parameters:

Seismic Schmidt number measures the spatio-temporal complexity of seismicity - the lower the seismic Schmidt number the less stable the seismic deformation. By encompassing the four independent seismicity parameters of $(\Sigma E, \Sigma M_0, X, t)$, seismic Schmidt number is the single most useful indicator of potential instability. It is found practical to use the log of the parameter in time-history analyses - see Fig. 9.6.2.

Local clustering of seismic activity in time and/or in space can relatively easily be accounted for when computing average values of different parameters quantifying seismicity. Let Ω represent the parameter of interest, say:

$\Omega = \varepsilon_s, \sigma_s, \dot{K}_s, \eta_s, D_s(\Delta V, \Delta t)$, then the local space and/or time clustering of Ω can be expressed as $C(\Omega; \bar{X}, \bar{t}) = \Omega / (\bar{X} \cdot \bar{t})$, where $\bar{X}$ and/or $\bar{t}$ would cater for the local spatial and/or temporal clustering of seismic activity - see Figure 9.6.3 as an example.

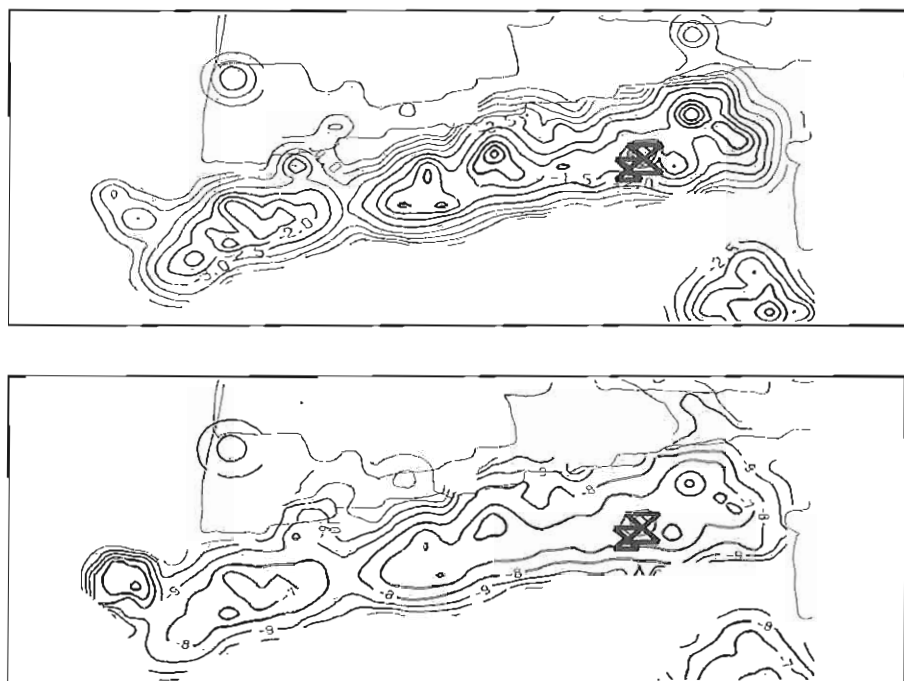


Figure 9.6.3 In the *top* figure contours of $\log D_s$ of one month's data of seismic response to longwall mining in a South African gold mine are shown. One can distinguish at least four areas of high gradient with D_s as high as 0.5 to 1 m^2/s . One of these areas attracted two larger seismic events, marked by the hourglass symbols, a day later. The *bottom* figure shows contours of $C(D_s; i) = D_s / i$ in $[\text{m}^2/\text{s}/\text{s}]$, where seismic diffusivity is divided by the local average time (over grid size) between seismic events i . The two larger events which subsequently occurred located in the area of the maximum D_s / i . (Reproduced with permission from AA Balkema, Rotterdam.)

Differential maps. The objective of differential maps is to delineate the areas or volumes of significant changes in the parameter of interest over time. It may help to identify 'hot spots', i.e. areas/volumes of undesirable rock mass response to mining, for more detailed time domain stability analysis.

Differential maps are presented as contour lines in 2D or surfaces in 3D of equal difference in a given parameter between two periods of time, say Δt_2 and Δt_1 , over the same area or volume of interest. For routine analysis, it is advisable that Δt_2 and Δt_1 be equal and consecutive. The grid spacing, hence the spatial resolution of the map, is a function of location accuracy and the network's sensitivity - the lower $m_{\min}$ the better the spatial coverage by small events.

Differential maps may be used to reconcile the results of numerical modelling with those of quantitative seismic monitoring, e.g. the expected percentage changes in stress and/or strain-related parameters predicted by time step modelling can be compared qualitatively and quantitatively with percentage change in the relevant observed seismic parameters. Back-analysis of stable and unstable cases would generally assist in validating the procedure before routine use.

9.7 CHARACTERISTICS OF SEISMIC MONITORING SYSTEMS

The transducers, data acquisition hardware and processing software which comprise seismic monitoring systems can best be characterised in terms of the amplitudes and frequencies of the ground motion which they can faithfully represent, and the rate at which events may be recorded and processed.

Frequency bandwidth and dynamic range. The magnitudes which may practically be monitored in mines range from -4, which are the smallest from which one can routinely extract useful information about the state of the rock mass, to 5, which are the largest events experienced. The predominant frequency f_0 radiated from the source, called the *corner frequency* of the seismic event, is related to the seismic moment and stress drop of the event by $f_0 = 1815 \cdot (\Delta\sigma / M_0)^{1/3}$ for the S-wave in hard rock. Figure 9.7.1 illustrates this relation, where the width of the lines is due to variation with rock type. The most information from seismic radiation on coseismic strain change is contained in the lower frequencies, and on coseismic stress change in the higher frequencies. Thus, a frequency bandwidth of at least $f_0/2$ to $5f_0$ is recommended to measure these changes with useful accuracy.

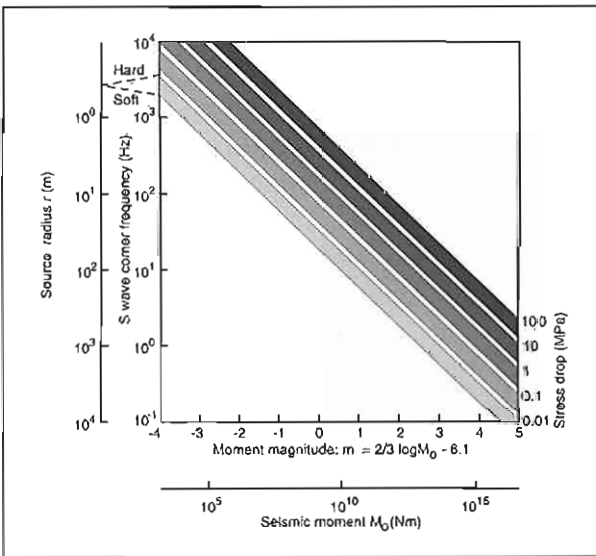


Figure 9.7.1

Expected source radius r and S-wave corner frequency f_0 as a function of seismic moment for a range of stress drops. (Reproduced with kind permission from Kluwer Academic Publishers)

Since the amount of energy radiated by events within the magnitude range -4 to +5 varies by at least 10 orders of magnitude, and the energy received by each sensor varies with the inverse square of distance from the source over a wide range of distances, it can be seen that a very wide range of ground motion amplitude must be accommodated. Allowing only for geometrical spreading to account for reduction of ground motion with distance, then for each event, the product of distance from the source R and peak ground velocity v_{max} is a constant. In hard rock, it is related to the source parameters by: $Rv_{max} = 7 \times 10^{-9} \cdot (\Delta\sigma^2 M_0)^{1/3}$, see Figure 9.7.2.

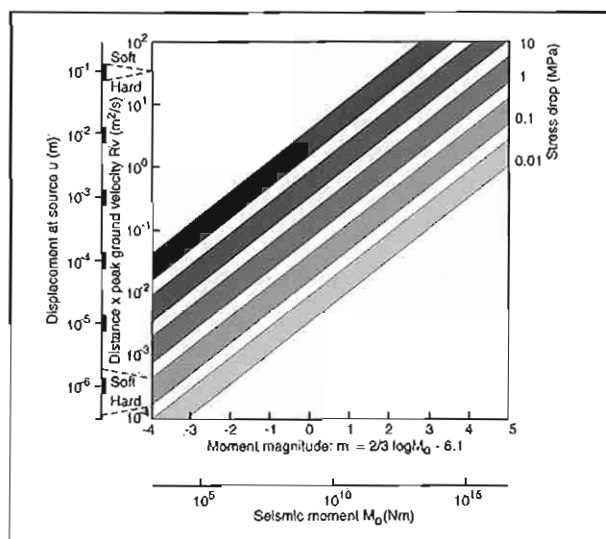


Figure 9.7.2

The product of distance from the source and the far field peak ground velocity, Rv_{max} , and displacement at the source, u , plotted as a function of moment for a range of stress drops. The width of the lines represents the variations due to a range of rock types. (Reproduced with kind permission from Kluwer Academic Publishers)

The sensitivity of a network may be gauged from peak amplitudes on recorded data, the distance from the source at which they were measured and the energy radiated from the source. A cumulative frequency-amplitude plot, analogous to the frequency-magnitude plot in Figure 9.5.1, allows the determination of the smallest amplitude which is reliably recorded on a particular network, with given noise levels and trigger settings. Similarly, the decay of amplitude with distance, which is always greater than the $1/\text{distance}$ from geometrical spreading, may be determined from the recorded data, as may dependence on seismic moment. Combining these quantities yields a maximum distance at which a given magnitude event may be detected. Such a relation, from data recorded on a South African gold mine, is shown in Figure 9.7.3, for ground velocity and acceleration.

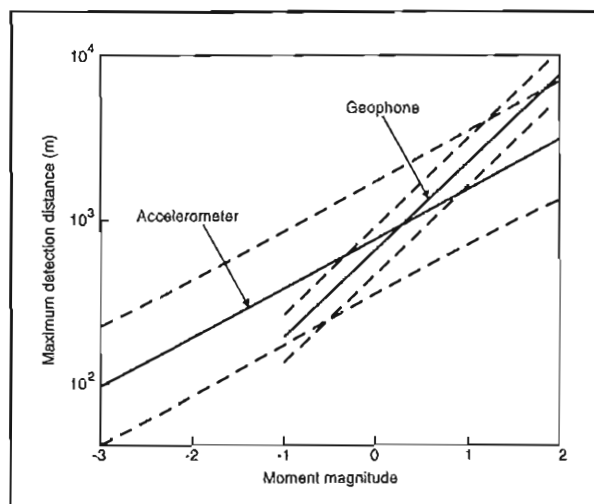


Figure 9.7.3

Minimum detectable distance as a function of magnitude, measured in a South African gold mine. The accelerometer shows much less variation with magnitude than the geophone, as expected. These values depend on environmental noise, trigger levels set by the operator, wave attenuation through the rock, and other factors which vary from network to network.

The dynamic range of a system is defined as the ratio of the maximum signal level that may be recorded to the noise level when no signal is present. This ratio is usually expressed in decibels: $R = 20 \log (S/N)$ [dB]

The dynamic range of commonly used sensors is illustrated in Figure 9.7.4. The system should have approximately 120 dB dynamic range or more if it is to take full advantage of sensor capability. This range may also be expressed as 1 000 000 : 1 or 20 bits, since this is the number of binary digits necessary to specify integer values in this range.

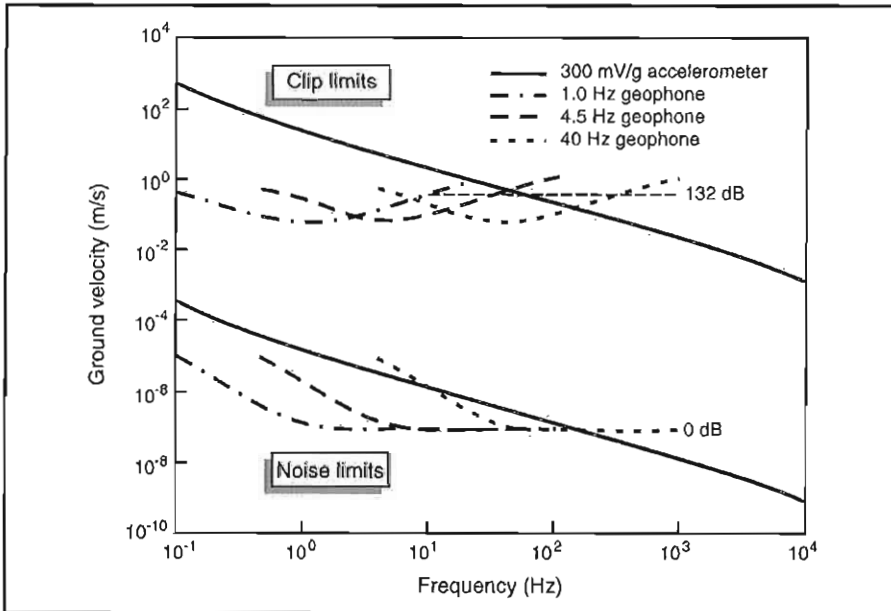


Figure 9.7.4 Sensitivity and dynamic range of sensors commonly used in mine seismic systems. The region between the limits represents the usable range for each instrument. The geophone's greater sensitivity up to several hundred hertz is clearly shown, as is the loss of dynamic range due to displacement clipping below these frequencies. A data acquisition system dynamic range of 132 dB with its quantization noise matching the expected ground noise of 10^{-7} m/s is illustrated. (Reproduced with kind permission from Kluwer Academic Publishers)

The resolution, or number of significant bits, on the other hand, depends on the ratio of signal to noise plus distortion in the presence of signal. The sensors commonly used in mines generate at least 0.1% distortion at large amplitudes, which is 1000 : 1 or 60 dB or 10 bits signal to distortion ratio. Thus the system should have at least 10 bits resolution at all points within its 20 bit dynamic range.

Since noise power is usually evenly distributed over the whole recorded frequency band, the noise level may be reduced, and hence the dynamic range increased, by reducing the bandwidth of the recording to match that of the signal. Decimation is the process of filtering a recording to remove unwanted frequencies and resampling at a lower rate to match the reduced bandwidth. For the case of evenly distributed noise power, 3 dB dynamic range is gained for each halving of the bandwidth.

The latest technology, known as sigma-delta, delta-sigma, oversampling or noise-shaping, for producing digitisers with high dynamic range and high resolution makes extreme use of this concept. The signal is digitised at a high sampling rate with low resolution, typically 1 bit, in such a way that the majority of the noise induced by the quantisation is at very high frequencies. These data are then decimated to the required sampling rate,

typically by a factor of 256, by which stage the dynamic range is very good. The simplicity of the 1-bit modulator front end usually results in excellent resolution as well.

Sensors. Two types of sensor which provide coverage within the frequency range of Figure 9.7.1 are miniature geophones and piezoelectric accelerometers. The amplitudes of ground velocity which they can measure are shown between the noise limits and clip limits in Figure 9.7.4 as a function of frequency. Choice of sensor type and other logistical and physical network parameters are summarised in Table 9.7.1, for different application domains, based on the following considerations:

- * Geophones are ideal for sparse networks, where sensors are separated by distances of the order of 1 km on average. Their sensitivity allows them to record relatively distant events, and there is little chance of a large event being near enough to more than one sensor to cause clipping. Geophones do not exhibit particularly high levels of linearity, 0.2% distortion at 18 mm/s being the typical specification.

- * Piezoelectric accelerometers are ideal for dense networks where sensors are separated by of the order of 100 m on average. The high frequencies to which they are sensitive are attenuated by propagation through the rock, so many sensors must be close to the active source volumes. Their comparative insensitivity to low frequencies ensures that very large events occurring within the same volume will not cause clipping. Accelerometers exhibit fairly large nonlinearities, distortion generally being specified at 1% or 2% near full amplitude. Transverse sensitivity can be as high as 5%.

- * By comparing Figures 9.7.1 and 9.7.4, it may be seen that neither miniature geophones nor piezoelectric accelerometers cover the low frequencies generated by the larger magnitude lower stress drop events. For these cases it is useful to add at least one short period earthquake monitoring instrument to a mine network in the form of a 1 Hz geophone or strong motion force balance accelerometer which provides for the recovery of moment for the largest events.

Sensor installation. Sensors should be installed in boreholes which extend beyond the heavily fractured rock surrounding the excavation. The sensor should be installed in grout that provides good coupling to the rock, and the borehole filled to avoid the trapping of acoustic energy. For this purpose grout should have similar acoustic impedance, i.e. the product of density and propagation velocity, to the rock. These precautions are necessary for accurate measurement of higher frequency signal components.

The sensors should be installed with a known orientation. For lower frequency geophones this is important as proper operation depends on being closely aligned, within 5 degrees, of the vertical or horizontal. Moreover, knowledge of the orientation of the sensors provides additional information which is useful for location and vital for the calculation of the moment tensor.

Where a sensor is placed on the surface, on soil rather than on bedrock, a concrete mount may be used, provided the radius of the mount is less than $1/6$ of the wavelength at the highest frequency to be recorded.

Table 9.7.1 System characteristics required for different scales of application

Application	Recommended Minimum System Characteristics
Regional mining, monitoring of several operations on a regional basis, larger seismic events, $m_{min} \geq 0$ over longer distances, 1 - 30 km	Low Frequency: 1 Hz - 500 Hz Sensor density: 5 sites within 5 km of the source Sensors: Geophones/Force Balance Accelerometers Dynamic range: 120 dB Resolution at all signal levels: 12 bits Event rate: 1 - 100 events per day Sustained event rate: 25 per hour Communication rate: 1.2 kb/s Communication method: single twisted pair and/or radio
Mine or shaft-wide monitoring $m_{min} \geq -7$ and at 300 m to 5 km	Medium frequency: 1 Hz - 2 kHz Sensor density: 5 sites within 1 km of the source Sensors: Geophones/Piezoelectric Accelerometers Dynamic range: 120 dB Resolution at all signal levels: 10 bits Event rate: 100 - 1000 events per day Sustained event rate: 250 per hour Communication rate: 9.6 kb/s Communication method: dual twisted pair and/or optical fibre
Microseismic monitoring, $m_{min} \geq -3$ and at 100 m to 1 km	Wide frequency band: 1 Hz - 10 kHz Sensor density: 5 sites within 300 m from the source Sensors: Piezoelectric Accelerometers Dynamic range: 110 dB Resolution at all signal levels: 10 bits Event rate: 1000 - 10 000 events per day Sustained event rate: 2500 per hour Communication rate: 115 kb/s Communication method: copper cable and/or optical fibre

System performance. The throughput of a system in terms of the rate at which events can be recorded and processed must generally be considered on three different time scales. The shortest time scale defines the *burst* rate, and is a reflection of how much storage is available for events immediately after triggering - this is temporary storage only and should accommodate at least 10 events to allow for fluctuation in time between events and seismogram duration. Second is the *sustained* rate, determined by the speed with which data may be transferred to permanent storage and automatically processed - see Table 9.7.1. Third is the *daily* rate, which is largely governed by the pattern of mining activity. For example, if blasting occurs once a day and generates 90% of the seismic activity within 2 hours, the daily event tally will be only those events which could be transferred at the sustained rate during those two hours. Additional local storage, up to hundreds of events, can usefully extend this time.

If a frequency-magnitude relation for the volume of interest is known or inferred and a desired network sensitivity in terms of m_{min} has been decided, then an estimate may be made of the required communication rate R [bits/s]. Given $N(\geq m_{min})$ for $\Delta t = 1$ day, and assuming:

- * 10 000 seconds per day in which to transmit the events, following the 90/10 time distribution above,
- * 2000 samples per seismogram and 6 bytes per triaxial sample, and

* 10 bits per byte including protocol overheads, yields

$$R = [N(\geq m_{min}) + n_r] \cdot 12 \text{ [bits/s]}$$

per triaxial sensor site, where n_r is the expected number of rejected waveforms per site per day due to environmental noise and events of less than m_{min} which cause the network to trigger.

9.8 LIMITATIONS OF SEISMIC MONITORING AND FUTURE DEVELOPMENT

Waveforms of seismic events contain newly-created information about the state of the rock mass at and in the vicinity of their sources at the time of their occurrence. The information becomes useful only once extracted and translated into parameters relevant to associated changes in strain and stress and/or into characteristics of the complex dynamics of seismic rock mass response to mining.

The fundamental limitation of seismic monitoring is our limited understanding of the radiation processes at seismic sources and inability to recover all useful information from waveforms. The same applies to seismic processes leading to rockbursts, where certain recognisable spatio-temporal patterns exist only over limited time periods, after which a dynamic reorganisation occurs that leads to the appearance of a new but still temporary pattern of events. The nature of the processes responsible for these complex dynamics is not as yet understood, and it severely limits the predictability of rock mass response to mining by either the numerical modelling or seismic monitoring techniques. In addition the sparseness of seismic sites, the use of geophones insensitive to high frequency (as opposed to accelerometers), and the low bandwidth of the communication network, limit the sensitivity of the system and thus the information about the rock mass behaviour in space and in time.

In today's practice of quantitative seismology in mines, only waveforms with well-developed P and S wave signatures, sufficient signal to noise ratio, low complexity, negligible near and intermediate field terms of source radiation and weak site effects are being processed routinely. This exacerbates the sparseness of spatial and temporal information about the state of the rock mass.

The following are the main directions in future research and development thought to be necessary to realize the stated objectives of monitoring seismic rock mass response to mining:

- One needs to increase the sensitivity, i.e. the frequency range, amplitude range and the throughput of seismic monitoring systems, to account for a greater portion of stress and strain changes due to mining and to gain information from the parts of the rock mass otherwise considered inactive and from the periods of time otherwise considered quiet. In addition, use of non-seismic sensors, e.g. strain-gauges, creep-meters; convergence measurements, etc, would assist in monitoring changes in the relatively slowly varying so-called static stress field which are thought to be an important triggering mechanism, implying the existence of longer range correlations before failure. This is specifically important when monitoring the stability of geological structures which are subjected not only to slowly-changing field stresses, but also to transient loads associated with seismic waves from nearby seismic events.

- There is a need to reliably describe the complexity of the source of a seismic event not only by its time, location, radiated seismic energy, seismic moment, and size but in addition by its shape, orientation, rupture directivity and duration, and to quantify seismicity by parameters pertaining to changes in stress, strain and rheology of seismic deformation processes. Such a quantification would allow the integration of results of seismic monitoring with numerical rheological models that capture the underlying dynamics. Developed models should be capable of reconstructing the main patterns of seismicity, i.e. size distribution, time distribution, clustering and migration, facilitating the design-as-you-mine process whereby the difference in modelled and observed rock mass behaviour could be explained and reconciled while mining.
- Quantitative stability analyses are not yet routine practice at all mines. They are currently done on the basis of qualitative interpretation of quantitative data provided by modern seismic systems, e.g. by analysing changes in parameters describing the size and time distribution of seismic events and their relation to stress and stiffness, and/or by monitoring the rate of seismic deformation and of seismically inferred stress change. Procedures need to be developed to formalize this qualitative process, to quantify the potential for rock mass instability that may result in rockbursts and to correlate it with the specifics of mine layouts, rates of mining, ways of excavating etc., for a given quantified geological setting.
- The recent development in 'time-to-failure', that takes into account rates of change in strain, stress and the complexity of seismic rock mass behaviour, may provide a tool not only to quantify the imminence of potential instability in the short term and to make the process more objective, but also to correlate with and eventually to integrate this type of development into numerical modelling.
- By analysing only 'good quality' waveforms of 'well behaved' seismic events one is utilising only a fraction, less than 1%, of the time the rock mass responds seismically to mining. There is a considerable information loss by not recording and analysing waveforms associated with fracturing, complex seismic events and non-stochastic rock mass noise, that constitutes a legitimate seismic rock mass response to mining. Such noise, or rather fluctuations, play a creative role in processes of self-organisation, pattern formation and coherency and can alter the system in fundamental ways. Consequently the models of rock mass behaviour close to excavations should allow for the influence of such fluctuations. It is therefore important to develop methodologies, technologies and logistics for *continuous*, in addition to intermittent, monitoring of seismic rock mass response close to the excavation faces, where sensors are positioned in the near-field at the centre of the volume of interest *where the action is*, as opposed to being outside in the far field. The processing and analyses could utilise the methods of nonlinear complex dynamics where characteristics of the motion in space and time, indicating the stability of the system behaviour, are computed in the reconstructed phase space from recorded ground motion. These also facilitate the development of data driven models of rock mass dynamics close to the excavation faces.

10.1 INTRODUCTION

Monitoring and instrumentation are an integral part of good rock engineering both from the point of view of understanding and quantifying local rock mass behaviour, and for monitoring the effects of the implementation of new designs or procedures. **Auditing** is concerned with checking that specified rock engineering standards are being adhered to and are having the desired effect, and that the whole rock engineering strategy for a mine is appropriate and is being implemented effectively.

10.2 WHY CONDUCT UNDERGROUND MONITORING?

Underground monitoring programmes are valuable for providing realistic input data for purposes ranging from support design to numerical modelling. They also enable particular real-life situations to be evaluated and compared on a quantitative basis. There should always be a clearly defined and valid purpose for undertaking any monitoring programme. If there are no *specific* questions that need to be answered, then in fact no instrumentation should be undertaken. In rock engineering there are four main reasons for conducting monitoring programmes and these are covered in the following sub-sections.

10.2.1 Record Initial Geotechnical Parameters

It is often necessary and prudent to determine the inherent geotechnical condition of an area, before the start of a project or before embarking on some rock engineering design. This not only enables a valid assessment of the prevailing conditions to be made but also provides a reference point for comparison with subsequent changes that take place during the life of the project. Typical geotechnical parameters measured are:

- rock deformation and stress fracturing in existing excavations or structures
- rock field stresses
- load in support elements
- condition, spacing and orientation of rock joints, bedding planes and other discontinuities; for estimating Q, RMR and other descriptors of rock quality.

10.2.2 Warn Of Unfavourable Deformations

During the life of a mine or individual excavations, it is advisable to obtain early

warning of undesirable deformations or deterioration in rock conditions. This particularly applies to excavations which are large and/or are required to remain safe and operational for the life of the mine or at least for a long period of time.

10.2.3 Validate Design Assumptions

Underground excavations and support systems are designed using the knowledge available at the time and based on certain assumptions. It is necessary to confirm that these assumptions are valid and to become aware if modifications are required. This is helpful not only for current operations but also for future designs.

10.2.4 Confirm Layout And Support Performance

Once a support system or excavation design has been implemented, it is important for current and future undertakings that confirmation is obtained that the design is fulfilling its purpose. Any deviation from the expected performance needs to be known as soon as possible to motivate timeous corrective action.

10.3 WHAT CAN BE MONITORED?

Parameters to be monitored in underground mining operations are usually those relating to the movement and behaviour of the surrounding rock and the behaviour of support elements. These parameters can be divided into those that are measured directly, e.g. rock displacement, and those that are derived indirectly, e.g. rock stress.

10.3.1 Rock Deformation

In most instances **rock deformation** is the most important rock engineering parameter that needs to be quantified. It is the factor that plays a dominant role in determining the useful life and integrity of underground excavations and determines load generation in most support systems. Rock deformation is manifest in the closing up or misalignment of excavations, e.g. stope closure and shaft tilt, and the movement of discontinuities, e.g. slipping of fault and bedding planes. The process can take place either gradually or rapidly. Typically, rock engineers are concerned with measuring the relative movement of opposing surfaces of an excavation, e.g. stope closure and ride; and the movement of individual layers of rock around an excavation, e.g. opening and closing of fractures in the sidewall of a large excavation, or the movement of an exposed fault plane.

Rock deformation is a directly measured parameter that can be monitored either as and when required to confirm the behaviour of a particular feature, or continuously to warn of any undesirable movement or trends in deformation that would be detrimental to worker safety or mining operations.

10.3.2 Rock Stress

Knowledge of the prevailing or anticipated **rock stress** is crucial to the design, siting and stability of underground excavations; and determining its value is an important aspect of rock engineering instrumentation. The level of stress in the rock, whether

inherent or mining-induced, will determine the amount of deformation and damage that an excavation will undergo. Particular situations will dictate whether it is required to determine the absolute stress at a point in the rock and/or the stress change at that point over time. Stress determination can be undertaken to obtain prior knowledge of the prevailing stress regime before the design and carrying out of mining operations in any particular area and/or for the study of any stress changes occurring due to mining operations. The magnitude of the virgin stress tensor, particularly the vertical stress and the k -ratio, are important data required for realistic numerical modelling studies necessary for the design of mine layouts and rock structures including tunnels, service excavations, orepasses and all forms of pillars.

Rock stress is not directly measurable: its determination is based on measuring small strains (i.e. deformations) and converting these values by means of a mathematical procedure into component stress values. This conversion process also requires the values for certain **rock properties** to be known; these values are obtained from laboratory tests conducted on representative samples of the rock at the strain-measuring site.

10.3.3 Support Performance

A considerable amount of effort and expense goes into designing support systems and choosing appropriate support units to address the various support requirements of underground mining operations. It is therefore important to selectively monitor installed support units and thereby determine that their performance is as expected, and to be warned of any irregularities which might indicate problems with the design, the support units and/or rock behaviour. **Support performance underground** can be very different from that measured in the laboratory, and there is no substitute for knowing how the support really performs under any particular set of conditions.

All types of support units can be monitored, and this usually takes the form of measuring the load generated in the support elements and the amount of deformation to achieve this load. Load cells of various designs, shapes and sizes are available to cater for the wide variety of support types in use, and all must be calibrated prior to use in order that readings can be accurately converted to the desired engineering behavioural units.

10.3.4 Backfill Performance

The monitoring of backfill is more specialised than that required for other support types and so warrants its own discussion. **Backfill performance** is assessed in terms of its stress-strain behaviour and this is determined by remote monitoring of special load cells and closure meters which are buried inside the backfill rib. Shrinkage of the backfill, which affects load generation and therefore the provision of support, is reflected in the stress-strain behaviour by an initial increase in strain without a commensurate increase in stress.

The performance of backfill is dictated by parameters such as placement height, shrinkage, porosity, relative density and, where applicable, percentage of cementitious binder, which influence the drainage, shrinkage and stiffness of the placed backfill. These parameters should, therefore, be regularly measured in conjunction with stress-strain monitoring for meaningful analysis of backfill behaviour. The peri-

odic monitoring of these parameters fulfils a quality control role and provides a measure of confidence that the backfill is delivering the expected support.

For comparison purposes and a more complete understanding of the benefits of backfill, it is common to measure closure outside the backfill rib and in equivalent unfilled panels.

10.3.5 Rock Joints/Fractures And Rock Condition

Knowledge about the depth, orientation and intensity of **rock jointing and fracturing** around an excavation is required for support design and evaluations. Although linked to rock deformation, the **rock condition** has additional implications for the selection of support. For example, the depth of fracturing will often influence the support resistance required and, in the case of tendon support, the length required to reach a stable anchoring point. The intensity of fracturing may dictate the need for headboards or support linking systems to achieve the necessary levels of areal coverage required in a stope. The degree and condition of jointing will affect the design of layout and support of service excavations.

The determination of rock jointing/fracturing and rock condition is mainly a visual process combined with measurements of fracture spacing, extent and orientation. This may be done by examining exposed jointing/ fracturing in cubby or stope rockwalls, or in borehole cores. To obtain further information about fracturing into the rock, a borehole petroscope or camera can be used, or a ground penetrating radar or radio tomography system. By following a recognised **rock mass classification** system, the assessment process is formalised and meaningful comparisons can be made. The NGI Q-system, widely used in civil engineering designs in Europe and elsewhere, is little used in South African mining. Two somewhat simpler systems have found application however, and are briefly described below.

The CSIR rock mass rating (**RMR**) can be estimated by simply adding together ratings based on five separate parameters: the UCS σ_c of the rock, the rock quality designation (RQD, based usually on core recovery percentages), the spacing of discontinuities (corrected for sampling geometric biases), the condition of the discontinuities, and groundwater conditions. Having estimated the RMR, standardized methods allow estimation of in situ Hoek-Brown strength parameters (Chapter 1.3.2), large-scale deformation moduli, etc.

An obvious defect of the standard RMR approach is that it ignores the influence of high field stresses, which in fact dominate the stability of deep hard rock excavations. The COMRO rockwall condition factor (**RCF**, Chapters 3.2.9, 6.4.2) addresses this by directly estimating the ratio of maximum tunnel sidewall stress to the estimated rockwall strength: $RCF = (3\sigma_1 - \sigma_3)/F\sigma_c$. The condition factor F takes on a value of 1 for typical good quality quartzites (whose RMR is about 85), but in poor rockmass conditions, the estimate $F \approx RMR / 85$ may be used.

10.3.6 Mining Layouts

Regular visual monitoring of **mining layouts** and **face shapes** by rock engineering practitioners will help to identify possible problem situations at an early stage and to

ensure that sound rock engineering principles are being followed. Aspects to be assessed include the angle of approach of two converging stopes, the angle at which a stope approaches a fault or dyke, the proximity of stoping to footwall development, the size of stabilising or bracket pillars, the volume of backfill being placed and the size of leads and lags.

To assist in the assessment process, a **face shape index** can be calculated, based on the deviation from a specified 'ideal' mining geometry (c.f. Chapter 3.2.6). This should be regularly reviewed at stope planning meetings, for example.

10.3.7 Fall Of Ground And Rockburst Accidents And Incidents

A study and analysis of fall of ground and rockburst accidents and incidents can be carried out to quantify the situation regarding these hazards, and to direct attention to the relevant problem areas. This will also form part of any **risk assessment** process.

Further, a more detailed study will help lead to the cause of accidents being better identified. To achieve this, a comprehensive database of all accidents and incidents needs to be set up which includes values for all the pertinent and controlling parameters. In this way, the necessary counter-measures to eliminate or ameliorate the risk can also be identified.

A study of this nature will aid in assessing whether a particular change in mining strategy or support system is having the desired effect in creating a safer working environment, or in motivating such a change.

10.3.8 Routine And Special Monitoring

Depending on its purpose, monitoring can be classified as routine or special. The choice will be determined by the nature of the study and this in turn may influence the type of instruments chosen for the task. **Routine monitoring** covers those rock engineering aspects that should be monitored on a regular, if not continuous, basis and forms a daily, weekly or monthly component of a mine's rock engineering function. **Special monitoring** is applicable to the start-up of a new project or system, unexpected rock behaviour or any other once-off monitoring situation, and may require the assistance of external specialists in some situations.

Routine monitoring includes the reviewing of mining layouts, periodic evaluation of support unit performance, and recording of rock displacement within critical excavations, e.g. underground hoist or pump chambers. Special monitoring would apply to once-off cases such as comparing a range of new support types, investigating the behaviour of a particular pillar for a short duration, determining the state of rock stress at a particular location, or the instrumentation of a shaft during shaft pillar extraction.

10.3.9 Long-term And Short-term Monitoring

An alternative classification is that of **long-term** and **short-term** monitoring. These categories are similar to but not necessarily identical to routine and special. For

example, the special case of monitoring a shaft during shaft pillar extraction would most likely be of relatively long duration. Support performance monitoring, while perhaps routine, would usually be of short duration for any given study. The frequency at which readings are taken, i.e. on a continuous basis, or at regular or ad hoc intervals, is not dependant so much on the duration of the monitoring programme but rather on the rate of change in the monitored parameter. Thus, a long-term programme may not require continuous measurements but instead regularly-spaced monthly readings if a slow rate of deformation is anticipated. The duration of the monitoring programme does however have an influence on the choice of instrumentation and monitoring procedure.

10.4 INSTRUMENTATION ACCURACY AND ERRORS

Of concern in any monitoring programme is the level of confidence that can be placed on the measurements and on the outcome of the measurements. In any instrument-based system there are inherent inaccuracies and errors, and it is important to know of their existence so that strategies can be implemented to counter and reduce their effect as much as possible. Reading inaccuracies arise from inherent inadequacies in the construction of instruments, e.g. accuracy and precision, and from external factors which can also result in significant errors.

10.4.1 Accuracy, Precision, Resolution And Sensitivity

The terms accuracy and precision relate to inherent aspects of instruments that affect confidence in their readings. **Accuracy** refers to the difference between an instrument's reading for a parameter and a *known* value of the same parameter. **Precision** refers to the extent by which an instrument can *reproduce* a certain reading for the *same* input. Instruments must exhibit an acceptable degree of both accuracy and precision to generate reliable results. High precision by itself does not, for example, imply high accuracy.

The ability to obtain a useful reading is also dependent on resolution and sensitivity. **Resolution** refers to the smallest division on an instrument readout scale (i.e. number of *significant digits* of output), whereas **sensitivity** refers to the *smallest unit* of measurement detectable by the instrument. These two features must be suitably matched to provide meaningful readings, and high sensitivity does not necessarily imply high resolution or accuracy.

10.4.2 Errors

Errors in measurement can arise from additional and wider-ranging sources than those inherent in instrument construction. Such **errors** can arise from temperature variations, electrical noise, inaccurate calibration, inappropriate installation procedure, incorrect choice of instrument, different people taking readings, and misreadings, misrecording and computational errors. Steps can however be taken to counter these sources of error; these include taking duplicate readings, care with selection and installation of instruments, use of automatic data capture, and installing sufficient instruments at representative locations.

10.5 WHAT TO MONITOR, WHERE AND WHY

Monitoring activities vary according to the location and parameter involved. Typical monitoring activities for the main categories of mining excavations, i.e. stopes, tunnels, shafts and large service excavations, are covered in the following sub-sections.

10.5.1 Stopes

Stopes, by their very nature, cause significant changes to occur in the surrounding rock mass, and the consequences of these for the safety of workers and ongoing stability of the workings is of great concern. Several monitoring activities are therefore applicable.

10.5.1.1 Closure

Closure is the most commonly measured rock engineering parameter, especially in stopes where it plays a vital role for the design and performance of support systems and stability of the excavations. In stopes at shallow depths, the monitoring of closure can also warn of impending backbreaks or collapses.

Closure is measured perpendicular to the dip of the hangingwall/footwall via one of a number of instruments depending on requirements of future site accessibility and frequency of reading. Instrumentation is available to measure closure on a daily or a continuous basis. Continuous readout measurements offer the opportunity to identify the major components of closure, such that components due to blasts and seismic events can be isolated from the general closure activity (see Figure 10.5.1). Time and face advance are usually also recorded for analysis purposes.

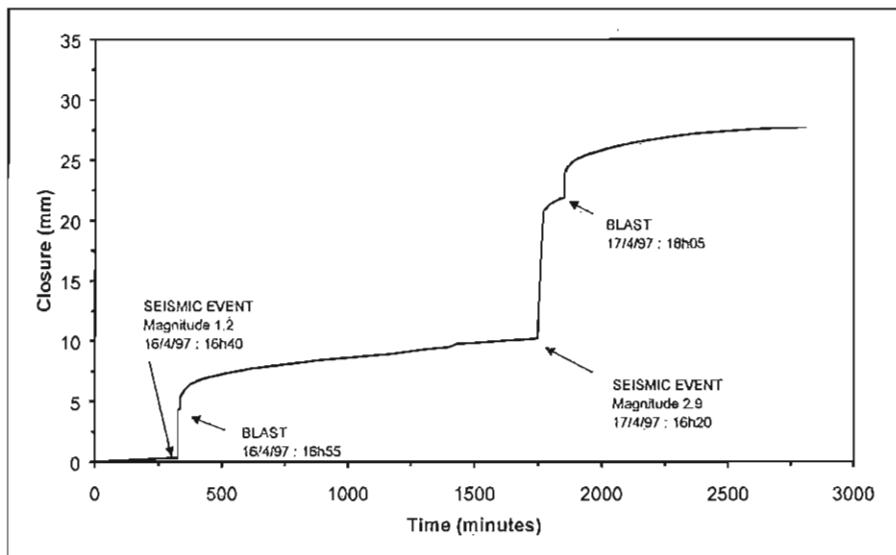


Figure 10.5.1 Stope closure as recorded by a continuous readout closure meter

10.5.1.2 Deformation of hangingwall and footwall

Gross deformation of the stope hangingwall and footwall manifests as stope closure; however more detailed monitoring also covers the more complex behaviours of ride,

heave, opening/closing of fractures in the surrounding rock (Figure 10.5.2) and the displacement of gully sidewalls. For example, footwall heave in some stopes leads to the unexpected failure of timber props. Time and face advance are usually also recorded for analysis purposes.

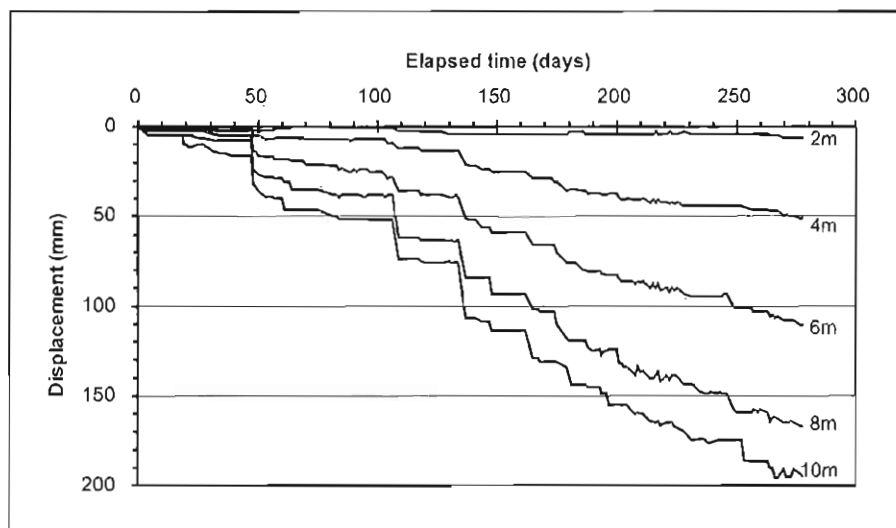


Figure 10.5.2 Example of h/w deformation from extensometer readings, as a function of distance into borehole from collar

10.5.1.3 Mining layouts

Routine monitoring of mining layouts and survey plans can timeously identify potential rock engineering problems. These include: the inappropriate proximity or angle of approach of excavations to one another or to geological features, the incorrect sequencing and advancing of panels, incorrect panel leads and lags, inadequate gully sidings, the unplanned leaving or creation of pillars and the incorrect size of planned pillars. The **face shape index** procedure (Chapter 3.2.6) can aid aspects of this process by assessing the deviation of the mining geometry from a pre-defined 'ideal' shape. The identification of a problem situation should then result in appropriate action being taken to rectify the situation.

10.5.1.4 Support performance

With the large variety of available stope support options, the varying underground conditions and the requirement to design stope support systems that meet certain criteria, the need exists for reliable knowledge about support performance under the prevailing conditions. Monitored behaviour includes load in support element as a function of deformation of the element (i.e. closure), yieldability of support element, buckling potential of support element as a function of element length, creep effects in timber and the influence of pre-stressing. Time and face advance are also recorded for analysis purposes.

10.5.1.5 Backfill performance

The stress-strain behaviour of a backfill material should be determined during the ini-

tial phase of backfill implementation. Comparing these results to standard laboratory test results for the same material will identify any deviation from this “theoretical” behaviour and may indicate shrinkage or loss of horizontal confinement. Values for the relative density, porosity and cementitious binder content of the backfill should also be measured in conjunction with the stress-strain monitoring to enable meaningful analysis and comparison with laboratory behaviour.

Although it is not normally necessary to prolong the stress-strain monitoring (unless significant changes have taken place in mining conditions or backfill placement), the values of relative density, porosity and cementitious binder content should be routinely checked to ensure continued predictability and quality-control of backfill behaviour.

10.5.1.6 Pillar stress and deformation

Pillars are designed and expected to sustain certain stress levels while maintaining stability of the stope. Therefore, monitoring the stress and deformation in a pillar will indicate the validity of the design assumptions and whether or not the desired pillar performance is being achieved, and warn of unexpected stress levels. Parameters of interest include initial stress level, subsequent change in stress over time, vertical and horizontal deformation, and fracturing.

The condition of pillars can be monitored on a semi-routine basis by observing open boreholes for fracturing and spalling. The early stages of fracturing of a pillar can give an indication of the ultimate pillar strength.

10.5.1.7 Fracturing and jointing

The fracturing and jointing of the rock surrounding a stope has an influence on the choice of systems to be implemented to maintain the stope’s stability. The orientation and frequency of fracturing and jointing will dictate the rock fallout potential, and the relative need for areal support systems. The depth and opening up of fractures, joints and parting planes in the hangingwall will help determine the support resistance and energy absorption required from the support system. Monitoring consists of mapping the orientation and distribution of discontinuities on the skin of the excavation and in the surrounding rock (Figure 10.5.3) and measuring the deformation of the surrounding rock with particular reference to the opening and closing of discontinuities.

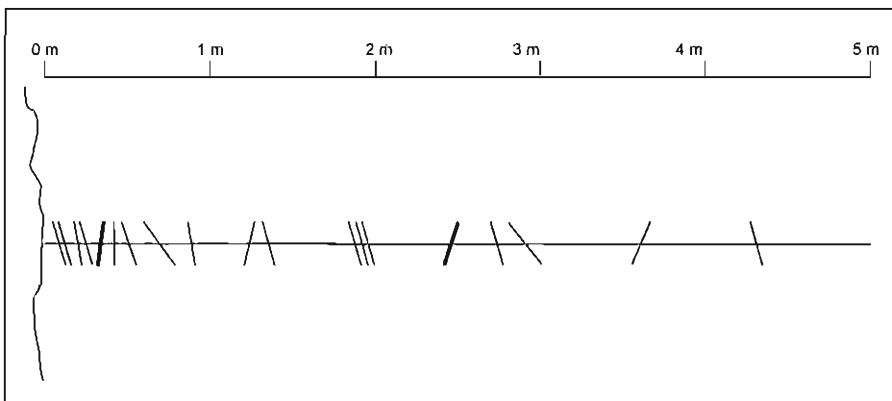


Figure 10.5.3 Example of fracture mapping by petroscope

10.5.1.8 Ground motion

Monitoring of **ground motion** caused by seismicity is desirable for the design of the stope support system, as this will influence the energy absorption requirement of the system. The relevant parameters are measured using accelerometer or geophone based equipment installed in the stope and attached to the rock surface.

10.5.1.9 FOGs and rockburst damage

The analysis of **accident statistics** and other data relating to FOGs and rockburst damage can lead to the identification of problem areas, an understanding of the causes, and guide the formulation of corrective measures and/or design changes in the support and mining system. This is achieved through the recording and collation of all relevant information, i.e. location, size, shape, bounding discontinuities, and installed support, in a comprehensive database. If this process is to be effective, regular analysis and communication of findings is required.

10.5.2 Tunnels

Tunnels are generally required for long-term use, and are subjected to displacements and field stress changes due to stoping operations. Various monitoring activities apply to tunnels.

10.5.2.1 Levelling

An accurate survey **levelling** exercise will determine to what extent a tunnel is being displaced by rock movement (e.g. Figure 1.3.6). These movements are caused by stoping, e.g. from the creation of a nearby abutment, and by dislocation of major geological features. A stable reference point is required for the measurements to be meaningful. Levelling surveys give useful estimates of effective rock mass Young's moduli, as well as of the effectiveness of regional support (stabilising pillars or back-fill) in resisting overall rock mass closure movements.

10.5.2.2 Deformation of excavation boundaries and surrounding rock

Deformation of tunnel rockwalls has implications for support effectiveness and accessibility of men and equipment. These deformations will usually be the result of unusual stress conditions or changes, or reflect creep over long periods of time. Measurements are taken between opposing rockwalls, and (using wire extensometers or borehole petroscopes) in holes drilled into the surrounding rock. It is also possible to conduct surveys of tunnel profiles to provide more detailed measurements of the changes over time.

10.5.2.3 Mining layouts

Routine checking of mining layouts and survey plans will enable timeous identification of situations which can adversely affect a tunnel's viability. Situations to be aware of include: follow-behind haulages being advanced too close to stope abutments, development taking place through faults and dykes, and the presence of nearby pillars and other stress concentrators.

10.5.2.4 Rock stress

The changing stress regime in which a tunnel is developed and to which it is subsequently subjected, plays a significant role in the tunnel's long-term stability. Knowledge of the initial stress field and any subsequent changes in stress is required for the confident design of the tunnel, its placement and support system. By drilling to a depth greater than three times a tunnel's largest dimension it is possible to con-

duct strain-relief measurements and thereby to determine the prevailing field stresses excluding the tunnel's own effect. A minimum of one borehole is required for 3D strain cell measurements, or three orthogonal holes in the case of 2D strain cell measurements. Multiple determinations are usually necessary, because of the inherent variability and relative lack of repeatability involved in field stress measurements.

10.5.2.5 Support performance

Monitoring tunnel support performance aids in the validation of tunnel and support system design assumptions and confirms the support elements' behaviour. Typically, the loads generated in tendon support can be monitored, in conjunction with the deformation of the relevant rock surfaces.

10.5.2.6 Fracturing and rockmass classification

Determining the fracturing in a tunnel provides information necessary for the design of suitable support strategies and systems. The depth, frequency and orientation of the fracturing is required. Use of a rock mass classification system helps to put into perspective the fracturing data and provides an indication of the stability and true strength of the rock mass. Rock mass classification also provides a basis on which to design and select support systems and to make comparisons with other working areas.

10.5.2.7 Ground motion

Where it occurs, seismically induced ground motion is a factor that has to be considered in the design of tunnel support systems. Incorrect assumptions as to the magnitude of ground motion affecting a tunnel can lead to either inadequate or unnecessarily expensive support being installed.

10.5.2.8 FOGs and rockburst damage

A routine analysis of all FOGs and rockburst damage, including accident statistics, not only aids in the identification of problem areas but also provides motivation for necessary changes in mining strategy and/or support system design. The maintaining of a database with all relevant information should be an on-going and regular process.

10.5.3 Shafts

Shafts are the longest serving and most vital of mine excavations and, although generally positioned in stable ground, there are occasions in which special monitoring is required.

10.5.3.1 Deformation of shaft barrel

Deformation of the shaft can occur due to the slow build-up of field stress in a shaft pillar, during shaft-reef extraction, or due to the movement of a geological feature. As such, monitoring of deformation parameters will usually be of a special nature and may be of relatively short duration. If it is known that mining operations are likely to cause deformation of the shaft, then instrumentation should be planned for and installed prior to this occurring.

10.5.3.2 Tilt and shaft misalignment

Shaft tilt and misalignment is usually associated with extraction of the shaft-reef or movement on a geological feature. In the first case, a shaft instrumentation programme should be a part of the whole process and commenced before the associated mining starts. In the second case, it may not be possible to predict this occurrence, however once identified a monitoring programme should be immediately instigated.

10.5.4 Large Service Excavations

Large excavations invariably contain vital equipment and often require special monitoring of long-term duration in order to provide warning of any detrimental rock engineering problems.

10.5.4.1 Deformation of excavation boundary

Monitoring of the deformation of the excavation boundary is important, not only as an indication of the stability of the actual excavation and for checking validity of design assumptions, but also to warn of detrimental movement that may affect the alignment and integrity of any installed equipment. The relative movements of all opposing rock walls are monitored. The vital nature of most large excavations and the long-term monitoring duration usually warrants the use of more expensive and sophisticated instrumentation.

10.5.4.2 Support performance

As with deformation monitoring, the determination of support performance provides validation of design assumptions, and advance warning of any detrimental activity.

10.5.4.3 Fracturing and jointing

Knowledge of fracturing and jointing affecting a large excavation is required for the design stage, and for the correct interpretation of the data obtained above.

10.6 INSTRUMENTATION AND ANALYSIS

After identifying and deciding on which parameters to monitor, the relevant instrumentation can be selected. The correct choice of instrumentation is an important step and this needs to bear in mind certain aspects of the parameters themselves, the data gathering and analysis facilities, and the overall monitoring strategy. The analysis process will also be determined by aspects of the monitoring programme, and the final audience or format for the results.

10.6.1 Instrument Selection

For each rock engineering parameter to be monitored there is usually a number of **instrumentation options**. The choice of instrument may, for example, depend on where the instrument is to be located, the required accuracy/precision, the expected range of change in parameter, who is going to install it/ take the readings/ analyse the data, what other instruments are being used, duration of operation, and budget.

Instruments fall into four broad categories: **optical, mechanical, hydraulic/pneumatic, and electrical**. In most cases instruments from more than one category are available to measure a given parameter. There are situations for which selection of an instrument from one particular category is indicated over the others, e.g. if remote or continuous readout are required, electrical devices are indicated. Table 10.6.1 lists the recommended instruments for common monitoring purposes.

The overriding desirable feature of any instrument should be its **reliability**, i.e. its ability to withstand the rigors of the underground environment and continue to provide usable data. In its broader sense, reliability can also include the ability to consistently

provide readings with minimal error. In general, the less complicated an instrument, the greater is its reliability. Consequently, mechanical instruments are preferred to electrical, and those with no moving parts are preferred to those with moving parts.

If an instrument, chosen for other overriding reasons, is not likely to survive for the full duration of the monitoring programme, a replacement instrument should be installed in good time prior to the first instrument's failure.

Table 10.6.1 Recommended instruments for common monitoring purposes

Parameter to be monitored	Recommended instrument	Operation and complexity
Stope closure (daily or manual reading)	Four-peg closure-ride station	Mechanical / simple
Stope closure (24 hr or remote reading)	Potentiometer closure meter Clockwork closure meter	Electronic / sophisticated Mechanical / simple
Closure in backfill rib	Backfill closure meter	Mechanical / specialized
Opening/closing up of discontinuities in rock	Wire extensometer	Mechanical or electrical / simple to sophisticated
Load in elongate support	Stick load cell	Mechanical / simple
Load in pack support	Pack load cells	Mechanical / simple
Load in backfill	Backfill load cells	Mechanical / specialized
Load in roofbolt	Roofbolt load cell	Mechanical or electrical / simple
Fracturing in rock close to an excavation <10m	Borehole petroscope	Mechanical & electrical / simple
Fracturing in rock close to an excavation <50m	Borehole camera	Electronic / sophisticated
Fracturing around an excavation	Ground penetrating radar or Radio tomography	Electronic / sophisticated
Detection of hazardous geological structures	Various geophysical including Radio and Seismic tomography	Electronic / sophisticated
Stress field (2D)	CSIR doorstopper strain cell in one hole	Electronic / sophisticated
Stress field (3D)	CSIR triaxial strain cell in one hole or doorstopper in three holes	Electronic / sophisticated
Stress field (3D)	CSIRO triaxial strain cell in one hole	Electronic / sophisticated
Ground motion	Accelerometers or geophones (e.g. CSIR Ground Motion Monitor)	Electronic / sophisticated but simple operation

10.6.2 Analysis And Interpretation

The **analysis** and **interpretation** of data will vary depending on requirements of the individual monitoring programme. Data from underground measuring stations will rarely be of any use in raw form, and will require some processing to be of use. For many measured parameters, readings will first have simply to be referenced to an initial or **base reading**, e.g. closure and fracture movements. For others, a more lengthy mathematical conversion process is required, e.g. load and stress determinations. **Laboratory testing** may also be required to provide additional parameters such as the strength and elastic modulus of the rock.

The most common interpretation process takes the form of plotting graphs of two recorded parameters against each other, e.g. closure vs time or load vs closure (deformation). In addition, information such as **closure rate** or **support stiffness** can be determined from these graphs. For other parameters, e.g. rock stresses, more complex plots are made.

In many cases, the successful interpretation of monitoring results can only be achieved by analysing the data in a variety of formats and combinations. For example, the analysis of stope closure in terms of time alone will ignore the additional influence of face advance (or lack thereof), since closure usually continues to occur even when a face is standing still (see Figure 10.6.1). This interplay could be an important factor in the design or assessment of a particular stope support system. Similarly, the example of extensometer results previously shown in Figure 10.5.2 depicts the absolute displacements of each anchor relative to the collar of the

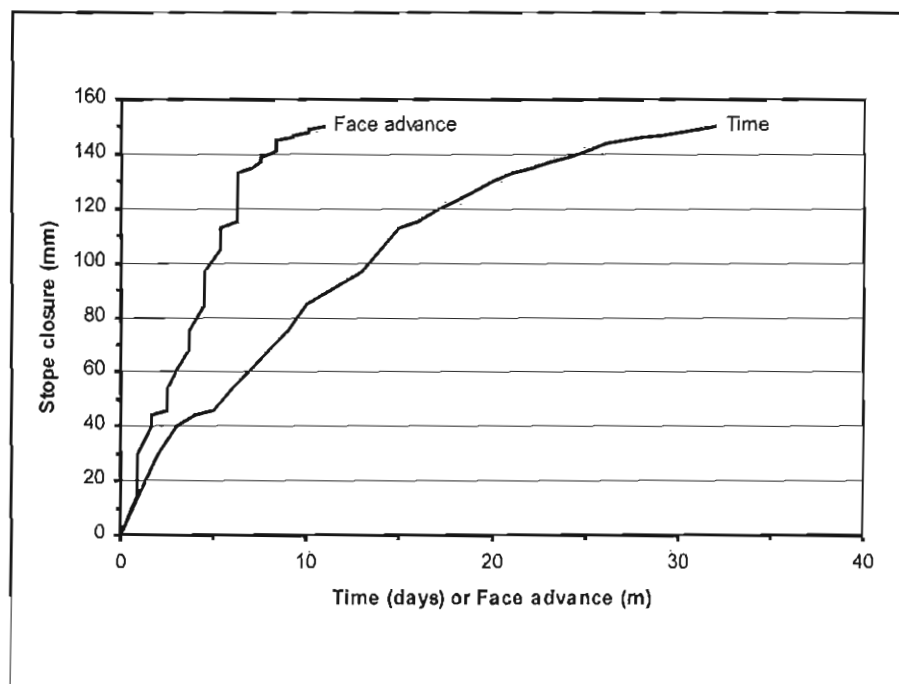


Figure 10.6.1 Example of stope closure with respect to time and face advance

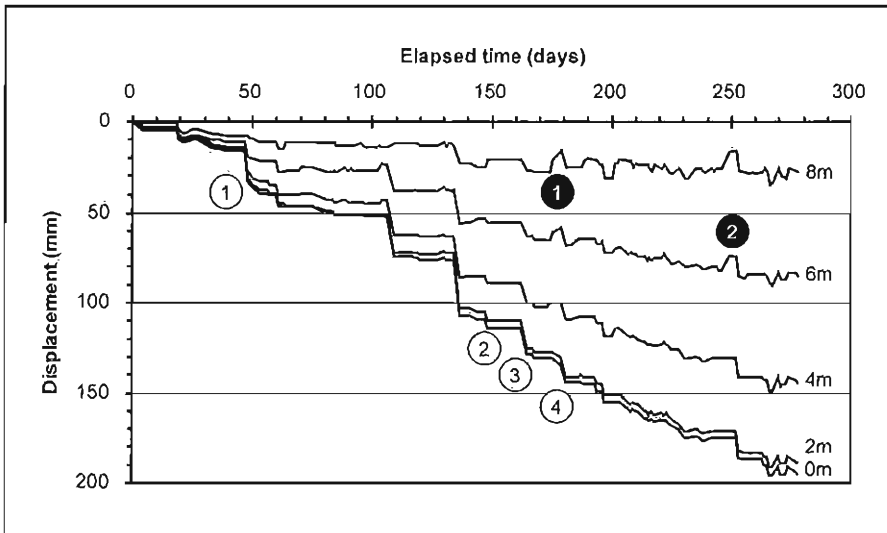


Figure 10.6.2 Example of extensometer data analysed relative to end anchor

borehole, as a direct representation of the field measurements, but this does not as clearly show the movement of the rock surface and rock layers with respect to each other and the general rock mass. Therefore, an alternative format is that shown in Figure 10.6.2 wherein the deepest anchor in this example (i.e. 10 m) is assumed to be stationary, and the displacements of all other anchors are calculated in relation to it. The displacement of the hangingwall skin is now explicitly represented (at 0 m), as is the movement of anchor points with respect to each other and the 'stable' rock mass. Thus, the interpretation of Figure 10.6.2 is that no significant separation has occurred in the first two metres of hangingwall rock, but that separation has occurred between each of the other horizons. These separations have culminated in the hangingwall rock opening 45 mm, 64 mm, 59 mm, and 27 mm in the zones 2-4 m, 4-6 m, 6-8 m, and 8-10 m, respectively. Closer examination of Figure 10.6.2 shows that a number of large, rapid separations have affected all anchor horizons. A cross-check with the mine's seismic data for the dates of these occurrences, shows that some of these movements (marked ①-④) coincided with recorded seismic events. However, not all movement is one of opening-up, for example, the plots for the 6 m and 8 m anchors show two instances (marked ❶ and ❷) when a closing-up of the strata occurred between the 6 m and 10 m horizons.

A further example of underground data interpretation is that of backfill stress and strain measurements. Figure 10.6.3 shows a typical plot of in situ backfill stress-strain behaviour from a measuring station located 3 m from the bottom edge of a backfill rib. At this depth inside a backfill rib, it is expected that the backfill will be loaded under confined-compression conditions and its behaviour should reflect this situation. A comparison of the general shape of the in situ vertical stress curve and that for a laboratory confined-compression test showed good correlation, confirming that confined-compression conditions existed at the underground measuring station. The 43% porosity of the laboratory sample compared well with the 44% average porosity measured at the site. The significant factor that is evident in Figure 10.6.3 is that the in situ stress-strain curves are offset to the right of the laboratory curve.

The value of this strain offset, which is between 3 and 4 %, is interpreted as being equivalent to the amount of backfill shrinkage that had occurred since placement. This shrinkage arises from the settling of the backfill material after placement and poor placement technique, whereby gaps are formed between the backfill and the hangingwall. The result of shrinkage is therefore a slower generation of load in the backfill and a reduced support benefit.

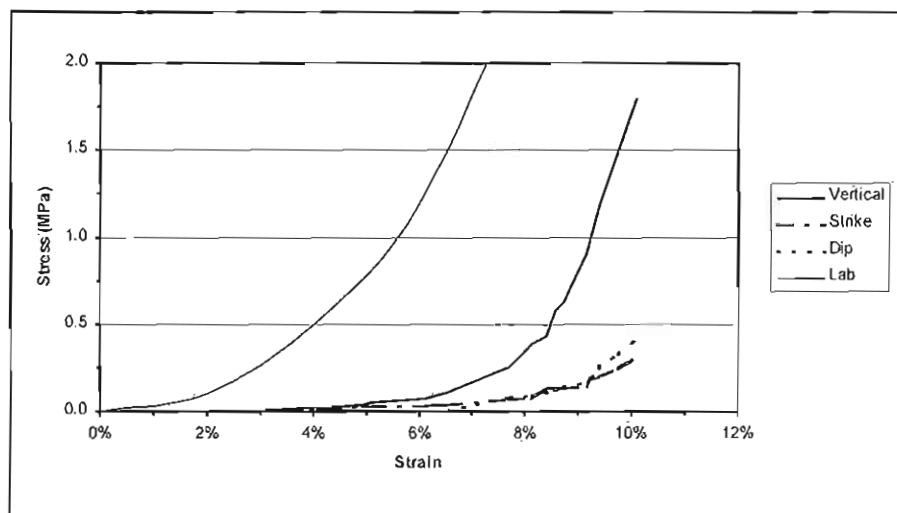


Figure 10.6.3 Example of backfill in situ stress-strain behaviour

A review of the results by an appropriately experienced person, at least in the early stages of monitoring, will help to identify any irregularities or unexpected values for rechecking before it is too late to correct the situation.

10.7 ROCK ENGINEERING AUDITING

In general, **rock engineering auditing** sets out to assess a mine's overall rock engineering strategy and its implementation. Two areas are involved: an assessment of the effectiveness of the *technologies* being used on a mine, and an assessment of the *skills* of the personnel. In this way, auditing helps to identify any shortcomings in support or mining practices, and any deficiencies in rock engineering knowledge or application. The auditing procedure should, moreover, aim to provide solutions to any deficiencies detected. Auditing should be carried out by teams of impartial rock engineering and other specialist personnel, including personnel having a sound understanding of production needs.

10.7.1 Technology Auditing

Technology auditing is concerned with assessing the implementation of rock engineering knowledge on a mine, in the context of the prevailing mining conditions. This includes reviewing the standard support measures used to protect workers and the mining layouts being used. [The standard **code of practice** in force on a mine,

itself needs to be pre-audited by qualified mining and rock engineering personnel.]

The auditing process involves *inspections* of underground areas and mine plans. Priority areas for assessment are generally those where most workers are concentrated; these are usually in-stope areas, development ends and travelling routes to and from the working areas.

During the underground inspections, the utilisation and installation of support are assessed in terms of adherence to the relevant **code of practice**. In this regard, factors such as: support spacings, support to face distances, missing or incorrectly installed support units, presence and orientation of headboards, and placement parameters of backfill are recorded. The use of additional support to stabilise geological features and potentially loose hangingwall is also noted.

Hangingwall conditions, including the intensity and orientation of stress-induced fracturing and the presence of known weak strata, are recorded for their potential to produce falls of ground. The installed support is assessed, in terms of these observed ground conditions, for its quality and suitability to provide the necessary support requirements (rockfall and/or rockburst) for maintaining the stability of the hangingwall.

Mining layouts are scrutinised for conformance to accepted rock engineering principles, which includes assessing face shapes, leads and lags, middlings between various excavations and mining horizons, design and position of pillars, creation of remnants, approach of stoping to geological hazards and proximity of development to abutments. In rockburst-prone mines, this process is combined with a review of seismic records to identify problematic areas and appraise control of the seismic hazards.

Support quality control procedures are also audited. For example, the prop shop or alternative prop maintenance system is audited, since the reliable functioning of hydraulic props is vital to the integrity of the support system in which they are being used. Procedures under review would include those relating to the monitoring of each prop's whereabouts on the mine, service history of props and setting pumps, and the use of manufacturers' approved spares.

10.7.2 Skills Auditing

The skills auditing process assesses the level and quality of rock engineering knowledge available on a mine. The understanding of mine personnel, as appropriate to their position and work requirements, is assessed by means of interviews with regard to rock engineering principles and the implementation of support. In particular, the ability and knowledge of the rock engineering staff is assessed, as well as their access to necessary training courses and input from the rock engineering consultant of the mining group. Also reviewed are the level and quality of the rock engineering department's technical communications with management, production and other service department personnel.

In addition, the rock engineering content in mine training programmes is assessed for its relevance and promotion of sound rock engineering principles, that will further enhance the implementation of rock engineering knowledge and foster the improvement of safety and productivity on the mine.

11.1 INTRODUCTION

In recent decades many scientific and engineering disciplines (e.g. architecture, ecology, medicine) have been forced to consider and develop a rigorous approach to problem solving through conscious (or explicit) model building. Conscious model building is the process of developing a model with an explicit and well-defined purpose that is consciously recognised by the model builder. However, there is another type of modelling: all human beings build models without consciously thinking about it. For example, it would be difficult to drive a car at speed along a busy freeway without the use of unconscious models that predict the likely, and possible, behaviour of other vehicles and indeed the one being driven. This type of model is used, unconsciously, and on a continual basis, by human beings in order for them to function in the real world. Unconscious model building is a natural human attribute and human beings are very good at it. However, human beings are not automatically good conscious model builders. Conscious model building is not a natural human attribute; it requires the implementation of a disciplined methodology that must be studied, practised and understood.

In order to develop a practical rationale for model building, to solve problems, it is necessary to understand the conscious model building process. Once model building is understood it is possible to propose a disciplined methodology of modelling. The implementation of such a modelling methodology within the rock engineering fraternity will lead to modifications and improvements to the initial methodology.

A model can be considered as any representation or abstraction of a system or process. It is an intellectual abstraction that includes purpose, reference and cost-effectiveness. Scientists and engineers build models for specific, well-defined purposes. Such models must always have a referent, that is, it must be a model of something specific. The model must be cost-effective: it must be less costly to build the model than to use the referent itself to achieve the required intellectual goal. In terms of practical modelling, it is important to continually ask a number of questions during the modelling process based on these three concepts. For example

Purpose:	Why am I building this model? What do I hope to achieve?
Reference:	What am I modelling? Is it what I intended to model?
Cost effectiveness:	Am I wasting time? Is there a cheaper faster way?

Five primary benefits have been said to be obtainable from conscious and well disciplined model building. These are:

- (1) improved problem definition, (2) organisation of thoughts,
- (3) understanding of data, (4) communication and testing of the current state of understanding, and (5) prediction from the resulting model.

Model building helps in the process of problem definition, because it is difficult to build conscious models to describe ill-defined problems. Hence, during the course of model building, the modeller is forced to fully define the problem being addressed. The same process helps the model builder organise the thinking process. This can have the added benefit of eliminating spurious areas of investigation and force the modeller to concentrate on the important issues that could lead to a solution to the problem.

Model behaviour can be tested against real situations and corresponding data. A model that agrees with the real data in the area of concern is a good model in that scope. One that does not must be modified or discarded. Simple models, physical or conceptual, provide excellent tools for communicating to others the current state of understanding of complex systems.

Furthermore, it is possible to 'experiment' with a model and subject it to various conditions and monitor the response. If the response is non-physical or in some way absurd then it may be that there is an error or omission in the current understanding of the mechanics of the system as implemented in the model.

Well-developed models can be used to make predictions about the behaviour of systems when they are subjected to conditions that have not yet been monitored in real situations. However, it is only possible to make reliable quantitative predictions if there exist sufficient descriptive data and a very well developed understanding of the mechanics of the system in question. This is often the case in mechanical and aerospace engineering (for example) where there is a great deal of very precise data relating to the construction materials being used and where the designs are accurately known.

Occam's Razor is a conceptual model of the modelling process frequently used by modellers. William of Occam was a fourteenth century philosopher who is remembered in the modelling fraternity for having written 'Entities should not be multiplied unnecessarily'. In modelling parlance this is interpreted as 'eliminate all unnecessary information relating to the problem that is being analysed'. However, it is important to eliminate only *unnecessary* data and not that which is required for the correct analysis. Occam's razor should be applied at all stages of modelling but particularly at the very early stages. A model that is developed that includes extraneous detail, results in excessive processing time and over complex input and output. On the other hand, of course, a model that excludes any of the real governing factors pertaining to the problem at hand leads to incorrect, and probably misleading, conclusions.

The inclusion of unnecessary detail often results from the assumption that in order to understand the behaviour of a given system it is necessary to include *all* the available data relating to that system: 'the more data the more accurate the model'. Taken to the limit, this assumption results in including all aspects of the system being modelled down to the micro-mechanical level. The model becomes the reality being modelled,

and results in the same degree of difficulty in understanding the model as in understanding the reality itself. Before this limiting case is reached the modeller will have lost all intellectual control and the model will simply have become a tool to confuse.

Models are used in engineering for two different but interconnected purposes, these are:

- Design - optimisation of an engineering system based on redefined criteria;
- Mechanics - development or improved understanding of an engineering or natural system.

Most practical engineers are concerned with the first of these, where they must optimise the design of some construction based on criteria such as economics and stability. Research engineers and scientists are primarily concerned with the second aspect. However, there remains a strong interdependency between the two camps as new understanding leads to better design methods and tools, and the need for better or more complex designs drive research.

Rock engineers have three principal types of model at their disposal. These are:

- Physical models
- Analytic models
- Numerical models

These three types of model comprise a set of toolboxes available to the engineer. Each contains a plethora of tools. An awareness of these and the ability to make a well-educated choice of the right tool or tools for the job are an important aspect in achieving the best solution to a given problem.

Numerical modelling, the subject of this chapter, provides the rock engineer with many powerful tools for mine layout and support design.

11.2 ASPECTS OF NUMERICAL MODELS

11.2.1 Eulerian and Lagrangian Analyses

There are three types of continuum material, these being gases, liquids and solids. Although all three can be analysed using the same general principles of continuum mechanics, solids tend to be observed in a very different way to that pertaining to liquids and gases.

Liquids and gases are both fluids. When measuring the properties of a fluid, one tends to immerse a stationary measuring point in a material that is free to move in respect to that point. An example of this is placing a thermometer in water to measure the temperature of the water. This observation is of a type, based on a fixed point in a moving continuum, that in classical mechanics is termed an *Eulerian* analysis. Eulerian analyses are well suited to the observation and understanding of the behaviour of fluids, whether they be liquids or gases.

Placing a fixed thermometer in a moving solid is not a good idea. In the case of solids, one tends to apply an alternative method of observation whereby the observa-

tion is fixed to a point in the solid and is allowed to move in sympathy with the motion of that point in the solid. In classical mechanics, this type of observation is called a *Lagrangian* analysis. In most fields of rock mechanics, one is almost exclusively concerned with the behaviour of a solid (albeit, not necessarily continuous) rock mass. Hence, Lagrangian analyses tends to form the root of most analytical methods in rock mechanics, whether these be experimental or theoretical. When it comes to the application of numerical modelling in rock mechanics the type of analysis is almost always Lagrangian.

11.2.2 Dimensionality

In general, numerical modelling is able to consider problems in up to four dimensions, these being the three geometrical dimensions plus time (see next section). In terms of numerical analysis, the lower the order of the dimensionality that has to be considered, the easier and faster it is to solve the problem. In rock mechanics, 2D and 3D geometries tend to be most frequently modelled, while time-dependent or transient studies are less common.

In terms of geometry, all real world problems are three dimensional in nature; however, it is often convenient from a modelling point of view to eliminate one dimension and reduce the problem to a 2D analysis. In continuum mechanics, two potential forms of 2D approximation exist, these being 'plane strain' and 'plain stress' analyses. It is not important to consider the details of these two approaches here, other than to understand that both are special cases of true 3D analysis. As special cases, they make certain assumptions that may or may not be valid for any given situation.

It is important when modelling to attempt to reduce the dimensionality of the problem as far as possible in order to improve efficiency and discard unnecessary detail. However, it is also important to understand the implications of eliminating higher dimensions. In some circumstances, it is impossible to eliminate the third geometrical dimension and achieve meaningful results. In other cases, the fourth-dimension transient effects are critical (for example in analysis of seismic events, or rock mass creep).

Reducing geometrical dimensionality allows for consideration of other more complex effects such as inelastic failure. The trend here is that efficient 3D numerical analysis programs tend to consider the rock mass as a linear elastic medium. In order to incorporate non-linear, inelastic and even discontinuum behaviour it is generally necessary to resort to 2D programs. As a result of this, the trend is to use 3D linear elastic programs to model large-scale scenarios such as the entire mining geometry, and to use 2D non-linear inelastic programs to analyse more complex local processes such as failure in a single pillar.

11.2.3 Disturbing The Equilibrium - Static and Dynamic Analyses

In almost all cases observed in real engineering circumstances, the rock mass, is in a state of equilibrium. That is to say, all forces at any point in the rock mass act in such a way as to cancel each other out resulting in a net force of zero at that point. As a result of the zero net force, the rock remains in a stationary state: it neither translates nor rotates.

In many cases, in order to simplify the problem and get a solution more quickly, it is possible to consider the rock mass *only* when it is in equilibrium. In such an analysis, one generally starts with the rock mass in an initial equilibrium state. It is then disturbed in some manner, such as by creating or enlarging an 'excavation', and it is then allowed to revert to a new equilibrium state. This type of analysis ignores time-dependent behaviour of the rock mass, and is referred to as a *Static Analysis*. Static analyses are very useful for looking at the state of the rock mass when it is in equilibrium after being disturbed in some way. In such circumstances, only the instantaneous reaction of the rock mass to disturbances of the equilibrium are computed: behaviour such as creep is not accounted for. If one is only interested in the new stresses and displacements around an advancing stope face at any given mining step, this is generally acceptable.

If one is interested in the way a rock mass reacts to the propagation of a seismic wave resulting from a seismic event, then it may be necessary to perform a *Dynamic Analysis*. In the case of a dynamic analysis, the equations of motion are solved at a series of time steps. In this way one can look at the state of the rock mass some given time after the event occurred, even if it has not reached equilibrium.

There exists a third class of analysis that is neither static nor truly dynamic, which is referred to as a *Quasi-Static Analysis*. Although quasi-static analyses solve the equations of motion for the rock mass at a series of time steps, the concept of time is not truly valid. The solution follows a physical solution from an initial state of equilibrium to a new state of equilibrium, but the time computed for the model to move from the first to the second equilibrium state bears no resemblance to the time it would take the physical reality to do the same thing. However, because the path is physically meaningful one can stop the model at any point between the initial and final equilibrium states and be confident that what is seen is meaningful. For example, plots of velocity vectors will indicate the correct direction of motion and the correct relative magnitude of velocity. However, 'snapshots' of a model that has not reached equilibrium should be treated with caution, as the real observed circumstances almost always concern a state of true equilibrium. An exception to this could be the case of steady state flow in say a failing slope or a mud-rush.

11.2.4 Analytical and Numerical Methods

Analytical methods, or closed form solutions, solve mathematical equations exactly. In other words they provide the exact solutions to prescribed mathematical problems. Unfortunately, it is rare that mathematicians are able to explicitly solve the equations governing complex physical environments or irregular geometries. In general, analytic solutions are limited to simple well-defined problems. Their main utility is in giving clearer insight into certain real-world situations since the sensitivity to key parameters is made explicitly clear, and in providing confirmation of the integrity of numerical modelling code formulations.

In rock mechanics, analytic solutions exist for such problems as the stress and displacement distributions around circular tunnels in a homogeneous elastic medium (the Kirsch equations). Further examples of such analytic solutions include elliptical holes, slit-like discontinuities ('stopes' or cracks), and approximations to triangular, square and rectangular openings, all in an infinite elastic region. In three dimensions,

or for more complex material constitutive laws, few analytic solutions are available.

Although these analytic solutions all evaluate the mathematical equations exactly, and hence give the exact solution to the mathematical problem, they are clearly limited in scope. In rock mechanics this means that analytic solutions are restricted to very simple mining geometries, such as circular tunnels and infinite elastic rock masses. In other words, although the equations can be solved exactly, the actual mining geometry has to be represented by coarse approximations. In order to provide better approximations to the actual mining geometry it is necessary to resort to *numerical* methods that can incorporate good representations of the mining geometry but rely on approximate solutions to the mathematical equations.

In the case of numerical modelling, the complex geometry is represented more accurately than with analytic solutions and the governing equations are represented by an approximation. Of course, in the case of numerical modelling, both the geometry and the governing equations are only approximations of reality; nevertheless, even approximations can be of extreme value. When one wishes to cross a busy road it is not necessary to have an exact value of the speed of an approaching vehicle to judge whether or not it is safe to cross, a good approximation suffices. Again, a fielding cricket player does not need to know the exact velocity or direction of the approaching ball to enable him to catch it. In reality, well-understood approximations are often perfectly adequate to get the job done. Numerical modelling methods are discussed in more detail below.

11.3 NUMERICAL MODELLING METHODS

Most rock engineering numerical modelling techniques are Lagrangian and can be categorised into either differential or integral methods. This categorisation is important to understanding how any given technique can best be applied and how efficient it may be in solving any given problem.

Differential methods rely upon modelling a specified volume of the rock mass in its entirety. As the rock mass is generally infinite in lateral extent, it is necessary to curtail the model at some artificial boundaries in order to render the problem tractable. Hence in differential methods, the rock mass is modelled as a volume (in 3D) or an area (in 2D) that is large enough to ensure that the boundary conditions do not significantly effect the region in which one is interested. In reality, of course, the boundary influence can only be negated if the boundaries are at infinity. Again, approximation enters the arena. If the influence of the boundaries is sufficiently small (i.e. they are relatively far enough away) then they may be considered to be at infinity. In addition, as a result of the volume-filling methods required in the differential methods, the stresses and strains computed in the solid away from any excavation are not continuous but are discrete. There are three types of differential methods in common use; these are the Finite Difference Method (FDM), the Finite Element Method (FEM) and the Distinct Element Method (DEM).

Boundary Element (BE) or Integral methods consider only the boundary of the excavations to be modelled. As a result, the stresses and displacements in the continuum outside the boundary are continuous. In addition the far-field, or infinite, boundary

conditions are solved exactly and there is no need to apply artificial boundary conditions. The need to represent only excavation boundaries can, in certain implementations of the boundary element method, result in a whole unit reduction in the dimension of the problem to be solved; this can result in a considerable increase in computational efficiency. On the downside, the cost of these benefits is that the boundary element method tends to be well suited to solving problems only in a homogeneous isotropic linear elastic material. This is because there is no explicit representation of the rock mass away from the boundary, it is simply considered to be in the far field, whether it is close or very far away. Because the integral method deals only with the excavation boundaries, it is often referred to as the Boundary Element (BE) method. There are three types of integral method, these being the Fictitious Stress (FS) method, the Displacement Discontinuity (DD) method and the Direct Boundary Integral method. Confusion can arise as the Fictitious Stress method is often referred to simply as the Boundary Element method.

11.3.1 The Finite Difference Method

The Finite Difference Method (FDM) is a differential method that fills the solid with a series of discrete grid points and then approximates the true differential equations as difference equations sited at these grid points. The FDM code FLAC uses a quasi-static solution scheme that involves time stepping from an initial equilibrium condition to a final equilibrium state. Because the scheme is quasi-static, the solution process can be interrupted at any time between the initial and final equilibrium states to view a 'snapshot' of the path to the solution. It is important to remember that the time taken to arrive at any given physical state does not relate to real physical time (this statement refers to the numerical time, and not the amount of time taken to compute the solution). However, certain modifications to the general scheme can ensure a true dynamic FDM solution, although this will always result in a slower solution than can be obtained with the quasi-static method, sometimes orders of magnitude slower.

Because the FDM fills the whole space of interest with a mesh of explicit grid-points, it is easy to apply any material properties at any point in the mesh. As a result of this, the FDM is not limited to homogeneous materials, but can represent inhomogeneous materials as well as layering due to bedding or intrusions such as dykes.

In general, FDM schemes do not form matrices that require implicit solution methods, but rather use explicit solution schemes that solve the difference equations as explicit equations. Because of this, it is easy to apply non-linear, inelastic constitutive laws into the solution scheme. This makes the FDM a very powerful tool for solving non-linear, inelastic, heterogeneous problems.

On the downside, because the whole region of interest must be filled with grid-points, it is necessary to provide artificial boundaries in the case of a semi-infinite rock mass. This requires the application of boundary conditions that can never truly satisfy the 'real' boundary conditions. However, providing the boundaries are located sufficiently distant from the 'area of interest', then the real boundary conditions can be approximated with a sufficient degree of accuracy.

In addition to filling the complete region of interest with grid-points and the need to provide artificial boundaries, it is also necessary to iterate in small time steps to reach

the equilibrium solution. This time stepping means that the FDM tends to be slow, and hence tends to be best suited to small-scale problems.

11.3.2 The Finite Element Method

The Finite Element Method (FEM) is a differential method that breaks the solid into a series of interconnected elements of a finite length (1D), area (2D), or volume (3D). The governing differential equations are then solved exactly for each element. If the elements were infinitely small then the behaviour of the continuum would be represented exactly. Hence, it is true that the smaller the elements the better the numerical solution, but the longer the solution time required.

As the FEM fills the whole space of interest with a mesh of elements, it is again easy to apply any material properties to any element in the mesh. As a result of this, the FEM is not limited to homogeneous materials, but can represent inhomogeneous materials as well as layering due to bedding or intrusions such as dykes.

The FEM has been used extensively in the mechanical and civil engineering fields, but has not been popular in rock engineering. Traditionally the input to FEM programs has been difficult and time consuming to set up. In addition, computer memory requirements have always been high due to the implicit solution scheme used that requires the construction and manipulation of large, sparsely populated matrices, in computer memory. With modern codes and computing resources, these disadvantages have tended to fall away; but it is probably still true to say that explicit FDM methods are more robust than FEM methods (which are otherwise very similar in capabilities) when it comes to modelling highly non-linear constitutive behaviour, such as tracking the crushing and dilation of rock in a high-stress environment.

11.3.3 The Distinct Element Method

The Distinct Element Method is used to represent a fractured rock mass as a collection of discrete rock blocks that are detached from each other but interact through their boundaries. This technique is very powerful for modelling discontinuous rock masses, such as the highly fractured skin of a deep level tunnel or stope. However, it is mostly limited to two dimensions, is fairly complicated and requires many long computer runs to provide meaningful results. Use of the DEM should be limited to experts with both the knowledge and time to achieve useful results.

The distinct element method is a subset of a more general method known as the Discrete Element Method. The main distinction between the Distinct Element Method and its more general super-class is that Distinct Element Methods must be able to recognise when new contacts are formed during a computational simulation without any external user intervention of any kind.

11.3.4 The Fictitious Stress Method

Despite its odd name, the Fictitious Stress method is fairly simple in concept. The edges of an excavation are represented as a series of lines (in 2D) or planes (in 3D) across which the force (or stress) is discontinuous. In practice, this means that an unsupported excavation can be modelled as a closed contour of elements that have

zero normal and shear forces acting across them.

Excavation support can be represented as a set of specific non-zero normal forces acting on the 'inside' of appropriate elements. This technique can also be used to model pressure tunnels where an equal force is applied normal to all the elements forming the excavation.

The fictitious stress method is also known as the 'force-discontinuity' method, and it is best suited for efficiently modelling excavations such as tunnels or low-span stopes having slot-like cross sections, unlike the idealised slit-like cross sections assumed in the displacement discontinuity method described below.

11.3.5 The Displacement Discontinuity Method

The Displacement Discontinuity, or DD, method is an indirect integral method for modelling areally extensive but narrow, slit-like excavations in either two or three dimensions. Closed contours of DDs can also be used to represent true slot-like excavations in a similar fashion as with Fictitious Stress elements; but it is generally more efficient to use Fictitious Stress elements in these circumstances.

Mathematically, the excavation is considered as a zero width crack or slit element across which the rock mass displacements are non-continuous (hence the name). Conceptually the crack can open or its opposing edges (in 2D) or faces (in 3D) can be forced into each other when loads are applied to the element. In this way the adjacent rock mass can move apart under tensile loads, or together under compressive loads. Displacements can also occur parallel to the crack surfaces to represent stope ride. All components of displacement (normal and ride) are equal on either side of the element. The amount by which the element's faces can move relative to one another is not limited in any way by the method itself. However, it is often useful when modelling a stope to apply a 'stopping width' limit to the element. This is actually a fictitious construct as mathematically the element has no width. The prescribed stopping width is used to control the amount by which the element faces can overlap in order to model total closure, or the more general behaviour of emplaced support such as backfill. Once total closure of the element has occurred it can either be allowed to slide freely, be clamped rigidly, or it can be given some inelastic slip behaviour based on a Mohr-Coulomb failure model controlled by a cohesive strength and friction angle specified by the user.

In general, support such as backfill is represented in the DD method as an internal linear or non-linear spring system that is compressed under compressive stress but has no effect under tensile stress. Under compression, the spring exerts a reaction force opposing the compressive displacement, thus reducing the amount of convergence that can occur across the element.

DD elements can also be used to efficiently model fault surfaces. This is achieved by specifying a line (in 2D) or sheet (in 3D) of DD elements with a negligible stopping width, and specified friction and cohesion properties. Very large and geometrically complex layouts, including tabular multi-reef mining, can be addressed using DD methods.

11.3.6 The Direct Boundary Integral Method

The Direct Boundary Integral Method is another form of Boundary Element Method, but it is not used in any of the general rock mechanics numerical modelling programs, and is not discussed further here.

11.3.7 Hybrids

Hybrid programs use a combination of two or more of the numerical methods already described in order to expand their capabilities beyond what could be achieved using only one method. Common combinations are the incorporation of a differential method into boundary element codes in order to provide localised plasticity modelling facilities. Often discrete element codes will use an additional differential method to allow for deformation of individual elements. In some cases, programs based on a differential method will incorporate a coupled boundary element scheme in order to solve for far-field conditions without having to apply tractions or displacements to artificial boundaries.

11.3.8 Keyblock Theory

Although not strictly a numerical modelling method, keyblock theory can be helpful in analysing potential hangingwall stability in underground excavations. The 'keyblock' concept states that potentially loose blocks of rock can only slide or drop out of the rock mass if they can be removed without breaking intact rock. A jointed or fractured rock mass may contain a few keyblocks, and many other blocks that are locked in place and cannot fail until the keyblocks have been removed. When designing support it is important to stabilize the keyblocks, since these are the blocks which determine the stability of an excavation. Techniques have been developed to identify keyblocks and quantify the stability in supported excavations if the jointing geometry is known; the Jblock program is a currently popular code that expedites this process.

11.4 PRACTICAL ASPECTS

11.4.1 Discretisation, Iteration and Convergence

All numerical modelling methods rely on a technique known as **discretisation**. Discretisation in space is the process of subdividing the continuous material body into a set of discrete volumes, areas, or lines, depending on how many dimensions are being modelled. The relevant governing equations are then solved, not for the body as a whole, but individually for each of these discrete entities. By combining the effects of each discrete solution, the behaviour of the body can be approximated and analysed. In this way a good approximation of behaviour associated with a complicated geometry can be achieved if a sufficiently large number of such entities are used.

Not only do numerical models rely upon discretisation in space, but also discretisation in time. This relies upon redefining the real continuous progression of time into a series of discrete time-steps of a finite duration. Often, the necessary time-step required for numerical stability can be measured in less than thousandths of a second, consequently a large number of time-steps may be required to solve a particular

rock mass problem that might require a few tenths of a second to equilibrate fully.

Often, numerical models rely upon solving equations in a way that is not precise but brings them closer to the actual solution. After repeated applications of this 'iteration' process, a measure of error falls to an acceptable level and the solution is said to have 'converged'.

A second definition of convergence in numerical modelling concerns a concept that is similar to that of 'diminishing returns' in economics. Repeatedly doubling the number of elements or number of iterations in a solution process, for example, will yield lower and lower gains in terms of reaching the 'true' solution to some problem. Ultimately the improved accuracy with the additional work required for these additional elements or iterations becomes so small that it is negligible: at this point the solution, although not exact, is said to have converged *acceptably* close to the exact solution. Judging the point of acceptable convergence is important in ensuring efficient simulations. In aerospace engineering, for example, it may be necessary to converge to within very small tolerances of the exact solution; however, in rock engineering it is often sufficient to converge to within 10 % or so of the 'exact' solution to provide acceptable design guidelines.

11.4.2 Design Criteria

Numerical modelling in rock engineering, as with almost all computational analyses in traditional engineering disciplines, is based upon the assumption that the problem is well defined. In reality this means that numerical modelling must be used to test proposed design specifications against well-established predefined design criteria. A number of such criteria (Chapter 3.2) include

- Average Pillar Stress (APS)
- Energy Release Rate (ERR)
- Excess Shear Stress (ESS)
- Rockwall Condition Factor (RCF)

In general, quantitative values should be assigned as limits to these criteria, based upon other analyses, before any mine design numerical modelling simulations are undertaken.

11.4.3 Application of Numerical Models

A host of computer codes are currently available to analyse the distribution of stress and deformations around underground openings, some of which are described in more detail below. These tools vary in complexity, depending on the degree of material failure that is to be described. At one extreme no failure is assumed to occur, and the problem is considered to be an essentially elastic boundary value problem.

Sophisticated boundary element computer codes, such as MINSIM and MSBESOL, which are used to analyse irregular tabular layout problems, provide an accurate determination of how stresses are *globally* redistributed on the mining horizon when the geometry is too complex to be dealt with analytically. These codes are mainly useful in assessing the large-scale effects of mining on more localised issues such as pillar loading (APS levels), stoping conditions (ERR levels), relative stability of geological weaknesses (ESS levels), deformations and stresses in shafts and shaft pillars, and the siting and support of tunnels and service excavations (RCF levels).

Observed movements within the *fractured ground* that envelops deep mining excavations deviate significantly from those predicted by simple elastic boundary element codes, indicating that local inelastic processes dominate the physical reality here. For example, extensive back area bulking and closure or face fracturing result in stress and closure redistributions that cannot directly be accounted for in the elastic models. Such phenomena need to be taken into account, approximately, by reducing the nominal stoping width or the rock mass modulus, and by increasing the effective span mined by a few metres. However, these types of adjustment can be somewhat arbitrary and needing of calibration, and once inelastic failure is to be explicitly considered, numerous additional considerations need to be assessed prior to any model analysis. These include the following items.

- The effect of so called path dependency. This requires that the actual extraction sequence be modelled with some accuracy.
- Failure mechanisms and the sequence of failure around the mining openings.
- Constitutive descriptions of the rock mass.
- Interaction of geological features such as faults and dykes.

The incorporation of these considerations increases the complexity of the modelling problem enormously. Additional physical parameters have to be specified, and the formulation of the models as boundary value problems becomes more difficult. In the mining context, it is likely that only minimal physical data are available to support proposed constitutive behaviour. It has to be accepted that to make further progress, the modelling methodology must now become 'self-adaptive'. The modelling activity has to be viewed as an organic process that is continually changing and the ultimate outcome of this process is initially unknown! In this respect, a variety of model types and formulations can be proposed and evaluated against known physical observations. The meaning associated with model 'calibration' has to be assessed carefully.

Computer codes developed for the analysis of inelastic phenomena (for example FLAC and UDEC) depend strongly on the constitutive assumptions used to define the models. General failure mechanisms (such as pillar 'punching') should be robustly predicted by different numerical approaches if the same physical attributes are correctly incorporated in each model. The modelling should indicate not only the detailed failure mechanisms, but also the ranges of constitutive properties that support the physical observations. In extremely complex cases, it must be accepted that the models will be used not only to provide a predictive or design capability, but also to motivate or regenerate the formulation of the models themselves.

11.4.4 Specific Numerical Modelling Packages and Programs

A number of numerical modelling packages and programs are used routinely for mine design purposes. The most widely used are the 3D linear elastic analysis programs, because they tend to be relatively quick and easy to use, and allow for the modelling of medium to large scale mine layouts that are complex in geometry.

The two programs that find most routine use are The CSIR Division of Mining Technology's MINSIM, and Mining Stress Systems' and Geologic Research's BESOL/MS. Both of these are 3D Displacement Discontinuity codes designed for

modelling of medium to large-scale tabular stoping layouts. In principle both of these codes are similar, differing mainly in detail. MINAP and DIGS (2D boundary element codes) are also used fairly extensively for very rapid analyses where the assumptions of 2D plane-strain are not significantly violated. MAP3D is a 3D Fictitious Stress and Displacement Discontinuity program that allows for the modelling of large-scale massive excavations in 3 dimensions. However, MAP3D is somewhat more difficult to use than the four previous programs and tends to be used only by specialists.

The 2D plasticity and discontinuum codes FLAC and UDEC respectively were developed by Itasca Consulting Group in the USA. These programs allow for complex nonlinear, inelastic, discontinuum behaviour to be incorporated in 2D modelling of excavations. Both programs are relatively difficult to use, require powerful computers, and are somewhat slow. As with MAP3D, the use of FLAC and UDEC tends to be limited to specialists and research work. However FLAC has, for example, been used for the design of roof-bolt support for tunnels and large excavations.

ELFEN is a fully 3D finite element program for modelling non-linear inelastic behaviour in both continua and discontinua. The program is marketed by the CSIR Division of Mining Technology (Miningtek). Although in its original form ELFEN is difficult to use, Miningtek have been involved in developing a special graphical user interface to simplify the use of ELFEN for routine mining 2D analyses.

There are many other numerical modelling programs available for stress-strain analysis. Many are used in the rock engineering and geotechnical fields in other countries. Only the packages and programs currently routinely used in South African mines are covered in any detail here.

MINSIM

MINSIM is a 3D program designed for analyzing stresses and displacements associated with tabular excavations in linear elastic ground. It is well suited to the analysis of narrow tabular stopes such as those found in the Witwatersrand gold fields and the platinum and chrome mines of the Bushveld Complex. MINSIM is optimised for deep level mining, but can be used for modelling shallow mines if the 'finite depth' option is selected.

In its basic form, MINSIM comprises two separate programs. The data input for these programs is in the form of pure ASCII text files, which can be created or edited with any ASCII text editor, such as Microsoft's Notepad. However, the input data is fairly complicated, and even slight errors can lead to major problems at a later stage.

The first program of the package is the *Solution Program*, which solves the DD equations for an arbitrary number of 'windows', each consisting of a planar array of 64 by 64 DD elements. The windows represent the tabular mining geometry by allowing individual elements to behave as if they are mined or partially-mined stopes, or unmined rock. As many windows as is necessary can be spread out across the mining geometry. They can be placed side by side, or lie at arbitrary positions or orientations (MINSIM is not limited to strike-parallel multi-reef windows), but they may not intersect. All windows must be of the same size. Windows are used for calculating the stresses and displacements on the plane of the excavations (on-reef) in the unmined and mined portions of ground respectively.

The second program of the basic MINSIM package, the *Benchmark Program*, is used for converting the on-reef calculations to a crude but readable format, and for calculating additional field variables including values at positions not lying on the plane of the excavation (off-reef). Although this program has a fairly extensive set list of variables that can be calculated, it does not accommodate all possible design criteria that may be required. For example, it does not provide computations of Rock Condition Factor, although it does provide all the necessary values for subsequent calculation of RCF. It would be difficult to use MINSIM with just these two basic programs. Additional programs are required for viewing results, and indeed, it is preferable to have a program to graphically create the input files for both the Solution Program and the Benchmark Program rather than having to create them in a text editor.

Miningtek provide a number of programs running under Microsoft Windows for graphically producing the input data files for the MINSIM programs. These programs allow for the capture of complicated mine plans using a digitizer. Windows and Benchmark sheets can then be placed graphically on the digitized mine plane by moving them around and sizing them by dragging a computer mouse. The Graphical User Interface (GUI) programs then write the complete set of files for input to the MINSIM Solution and Benchmark programs, which are then called and executed from within the GUI. Finally, analysis and plotting of the results can also be performed graphically using a GUI program.

MINSIM allows for face elements to be represented as 'partially mined' based upon the percentage of mined and unmined ground occupied by the element. These partially mined face elements allow for extremely accurate computations of displacements in the mined-out areas. However, the use of partially mined elements can result in poor estimations of average pillar stresses if they are not used carefully.

MINSIM is best suited for evaluating medium to large stope layouts in terms of some pre-selected and well-defined design criteria such as Energy Release Rate (ERR), Excess Shear Stress (ESS), Average Pillar Stress (APS), or Rockwall Condition Factor (RCF). It is also useful for ascertaining a better estimation of *local field stresses* for use in additional modelling with a more specialised program. MINSIM is not well suited to investigations of local inelastic behaviour, due to the way in which the rock mass is everywhere represented as a linear elastic, homogeneous and isotropic continuum.

BESOL/MS

BESOL/MS is a 3D DD program designed for analyzing stresses and displacements associated with tabular excavations. Like MINSIM, it is also well suited to the analysis of narrow tabular stopes such as those found in the Witwatersrand gold fields and the mines of the Bushveld Complex. BESOL/MS can be used for efficiently solving deep or shallow level mining problems without having to make any explicit selection of which type of problem is being solved; though in so doing, it sacrifices the option of being able to model non-strike-parallel multiple orebodies or fault planes.

BESOL/MS is best suited for comparing medium to large stope layouts with some preselected and well-defined design criteria such as Energy Release Rate (ERR), Excess Shear Stress (ESS), Average Pillar Stress (APS) or Rockwall Condition

Factor (RCF). It is also useful for ascertaining a better estimation of local field stresses for use in additional modelling with different programs, and is not well suited to fundamental investigations of inelastic rock mass behaviour.

MINAP

MINAP is a 2D program designed for the analysis of stresses and displacements associated with arbitrary shaped excavations under plane strain. It uses both the Fictitious Stress method and the Displacement Discontinuity method for representing excavations of different types. Generally speaking, it is limited to analysis of excavations in homogeneous, linear-elastic rock masses. A recent version of MINAP incorporates a Windows Graphical User Interface. The program is well suited to providing 'quick-and-dirty' solutions to large-scale mining problems, and should often be used as a precursor to more complex or comprehensive modelling exercises.

MAP3D

MAP3D is a 3D boundary element code developed by Mine Modelling Ltd of Canada. It incorporates both Displacement Discontinuity and Force Discontinuity elements, allowing it to model both tabular and massive excavations. Although this combination of excavation types can be useful, it comes at the cost of simplicity to use. Definition of a tabular orebody in MAP3D is somewhat more complex than with the special purpose tabular deposit programs. However, MAP3D provides attractive capabilities for the analysis of massive excavations in a rock mass that can be considered to behave in a linear elastic manner.

FLAC

FLAC is a Finite Difference program developed and marketed by Itasca Consulting Group of Minneapolis USA. Although FLAC is capable of 2D modelling of complex behaviour of any continuum material, it was originally designed for soil mechanics problems. As a consequence of this, it is particularly well suited to modelling the internal behaviour of backfill, as backfill is fundamentally a highly engineered soil-type material. In terms of rock mass analysis, FLAC is generally used for modelling local material failure effects, such as pillar punching, or failure around tunnels. Although FLAC incorporates logic for handling simple discontinuities as interfaces, it is not well suited to anything other than very simple discontinuity analyses (involving a limited number of fractures, not exceeding around 10).

FLAC incorporates a powerful scripting language called FISH (from FLACish). This language allows the user to customise FLAC's behaviour in an extremely flexible manner. Although FISH was designed for use by non-programmers, some degree of programming knowledge is useful in order to be able to use FISH effectively.

FLAC is a powerful non-linear inelastic modelling program; however, this power comes at a price. FLAC uses a text-based interactive command line interface that is highly flexible, but is not easy to use. In general, FLAC analyses are best limited to *experienced* users.

UDEC

UDEC is a Distinct Element program developed and marketed by Itasca Consulting Group of Minneapolis USA. It is well suited to 2D analyses of highly jointed rock

masses, as it can model many discrete fractures with ease. Although the fractures must be defined at the start of an analysis and no new fractures are allowed to form (intact rock can fail, but not fracture), this is not a major limitation for the analysis of rock stability in regions where the fractures already exist, e.g. the skin of a deep level excavation or a rock slope.

UDEC is a powerful analysis program and now also contains the scripting language FISH to allow for maximum flexibility of the program. However, UDEC is not particularly easy to use, as many mathematical parameters need to be prescribed. The text-based command line interface is also far from user-friendly. UDEC is also a program best used by experts only.

ELFEN

ELFEN is a 3D FEM and DEM program developed for the civil and mechanical engineering fields by RockField Software, UK. ELFEN is now being applied to mining situations in South Africa. It incorporates the combined capabilities of a number of other programs in a unique way, and also allows for explicit block fracturing during a run. In addition, ELFEN allows for fully 3D dynamic analyses.

Jblock

In mining applications large rock surfaces are exposed daily and it is not practical to attempt to map each joint or stress fracture to determine whether it defines a key-block. A probabilistic approach has been developed in the form of a computer program Jblock which uses a statistical description of jointing and stress fractures in underground excavations to generate potential keyblocks in the surface of predefined excavations. The program identifies keyblocks and tests their stability against defined support systems. The output comprises histograms showing the probability of failure of keyblocks of different sizes, and the areal distribution of potential rock falls. The results may be used to optimize support layouts and face orientations relative to natural discontinuities and stress-induced fractures.

11.4.5 Program Overview

The following table provides a summary of the main advantages and disadvantages of using the programs described in the previous section.

	Minap	Minsim	MSBesol	Map3D	Flac	Udec	Elfen
User Interface	A	S	S	S	W	W	A
Ease of use	A	S	S	W	W	W	A
Global	S	S	S	S	W	W	W
Local	W	N	N	S	S	S	S
Rock Failure	N	N	N	N	S	S	S
Geology	N	N	N	N	S	S	S

S: Strong A: Average W: Weak N: Not applicable

11.4.6 A Modelling Methodology

The following steps are suggested as an outline for a disciplined modelling methodology that can be applied to a wide range of problem-solving situations encountered in rock engineering.

1. Identify problem areas and the potential problem.
2. Design a number of potential mining strategies to alleviate the problems.
3. Define relevant design criteria.
4. Develop a model that incorporates necessary detail to assess the mining strategies in terms of the design criteria.
5. Choose a relevant tool or tools to test the proposed mining strategies based on the model.
6. Perform simulations on each strategy using the model and the specified design criteria.
7. Analyse the results of the simulations in terms of the specified design criteria.
8. Select a mining strategy based on the analyses, or propose new/modified mining strategies and go back to 4.

The modelling process itself should not be governed by any hard and fast rules, but the methodology of using modelling for problem solving should be appreciated and well understood. It is important to select the most appropriate tool for the job, taking due account of time and economic constraints. A 'quick and dirty' solution using a code that is readily available may be a better course of action than the exigencies of running the ideal program. In general, it is best to use the simplest tool that can get the job done. Computer codes that require few parameters are generally easier to control from an intellectual point of view than ones that require multiple (and often inter-dependent) parameters. Elasto-static boundary element codes offer a vast potential for solving many mine design problems, if appropriate design criteria are selected and the analyses are performed comparatively and/or qualitatively. Invariably, it is better to perform many varied small and simple simulations than a single massive simulation in the process of optimising a design.

It is essential to tailor the resolution of the model to the problem being addressed. If the problem is one of global stope design, then it is foolish to include such aspects as prop support. If the problem concerns a local area of the mine, it is unnecessary to accurately digitise distant stope outlines; it is only necessary to capture an approximate contribution of the distant mining on the local loading conditions.

The question of model resolution also occurs when assigning values to model parameters. Here, the resolution is controlled by the underlying assumptions of the numerical formulation used in the code. Giving extremely accurate values for the elastic constants in a linear elastic boundary element is futile, for example, as the approximations made in the representation of the rock mass are enormous by comparison. An educated guess at the stiffness of the flawed rock mass as a whole is generally more appropriate than an accurate determination of the elastic modulus based on countless tests of unflawed rock specimens.

It is important to recognise the limitations of the tool, or tools, being used. For example, continuum codes may be well suited to analysing hangingwall damage in an advancing stope, but they are of very limited value if the problem is one of hanging stability. More damaged (i.e. more fractured) hangingwalls may be more stable than less damaged hangingwalls, as is observed in practice in shallow stopes which are often more prone to large falls of ground than deep level stopes.

ROCK ENGINEERING TRAINING REQUIREMENTS

12.1 INTRODUCTION

Training recommendations set out in this chapter should be seen in the context of the South African Mines Qualification Authority (MQA), and need to conform to the requirements of this body. The South African Qualification Authority (SAQA) act of 1995 and the Mine Health and Safety Act (MHSA) of 1996 led to the establishment of the MQA. This tripartite body (with representation from government, labour and employers) currently advises the Minister of Minerals and Energy on education and training standards and qualifications in the South African mining industry. In the Department of Mineral and Energy's 'Guideline for the compilation of a mandatory code of practice to combat rockfall and rockburst accidents in metalliferous mines and mines other than coal', Appendix IV deals specifically with the function of the rock engineering service on a mine. One of the functions of the rock engineering department is to assist with training at all levels of underground mine personnel in rock engineering appropriate to their occupations and mines. Emphasis should be placed on strata control and the identification of dangerous ground conditions. Their task should also include instruction of training officials on their mine in aspects of rock engineering. Reference is made to the need for the rock engineering service to be proactive in general, and training provides an ideal opportunity to do this.

From the points of view of both improved safety and improved productivity, all persons who work for any time underground need to have a basic knowledge of the nature and potential hazards of the rock mass surrounding them and how this responds to changes, such as fracturing under stress and the influence of support. As the level of responsibility of a job increases, so the depth and breadth of understanding of rock engineering principles needs to grow. With many staff having little formal education in mathematics or science, the communication of the necessary knowledge poses a significant challenge. It becomes important to demonstrate the principles practically, and to devise methods of testing candidates' rock engineering knowledge and understanding in an appropriate way.

The 'syllabus' suggested in the following sections has been set out on a cascading basis to cover different job functions and levels. The content expands with each successive level: starting with symptoms of problems and a focus on 'what' needs to be done; followed by an increased emphasis on 'why' things happen; and, at the most senior level, a quantification of these issues. Some examples illustrating how to effect this training have been included in the following.

The method used to train people who work on the mine will vary depending on the level of the staff being trained. However, the material should be practical, with important concepts and theory communicated as visually as possible. A workshop format with opportunities for discussion is a method which encourages effective transfer of knowledge at all levels. Use should be made of case studies, pitched to the level of the learners. If at all possible, underground trips in the company of a rock engineering instructor/practitioner should be arranged to identify the problems and situations in reality. There should be some assessment of the effectiveness of the training. This may take one of the following forms: a questionnaire which probes the candidates' understanding of rock engineering issues; discussions with candidates (although this is extremely time consuming); people being examined in the workplace underground regarding their understanding of rock mechanics in their current environment (also time consuming).

It is recommended that all underground workers be given a rock engineering training course appropriate to their level of work. Should their job change, the additional subjects and/or detail should be taught to them. The level of their rock engineering training should feature in their employment record. All mine employees should undergo a short refresher course in rock engineering on returning from their statutory leave period. Outside contractors should show proof of suitable training appropriate to their job. It is suggested that, in the event of their having inappropriate rock engineering training, they should participate in a course offered by the mine. Similarly, if their training is more than 12 months previous, they should undergo the mine's short refresher course. It is envisaged that the rock engineering staff should play a pivotal role in the training on their mine. They should expand the syllabi provided below to suit local requirements and work with the training staff on the mines to ensure that practical, applicable, yet accurate rock engineering knowledge is communicated. The trainers should be briefed by the rock engineering personnel at least once a year to refresh their knowledge and keep it up to date.

12.2 ALL PERSONS WHO GO UNDERGROUND

(including service personnel and all sub-contractors)

- The importance of remaining vigilant and examining the rock, support and general conditions routinely and regularly to identify discontinuities and other hazards; and the appropriate action to take or person to report to when an unsafe condition is identified. [See the DME's document 'Guideline for the compilation of a mandatory code of practice to combat rockfall and rockburst accidents in metalliferous mines and mines other than coal'].

12.2.1 Basic Geology

- Understanding that rock is not continuous (e.g. Figures 4.2.2/5/8):
 - Sedimentary layers, joints, faults and dykes, and mining-induced fractures.
 - Discontinuity planes where the rock breaks easily, and may be filled with weak material or water.
 - The dangers of discontinuities intersecting to produce key blocks around excavations.
 - Identification of situations which may look stable but which are hazardous and

pose a threat to safety.

[Training should include practical underground instruction on how to identify weak and problematic rock types, as well as unstable or dangerous blocks.]

12.2.2 The Force of Gravity

- The likely consequences of being hit by a falling object, i.e. injury or death. [For example, a physical illustration of the destructive potential of a 50 kg slab dropped from heights of 0.5 or 2 m on to a yielding object (impact velocities 3+ and 6+ m/s, respectively).]

12.2.3 Special Mining Areas

- Rock engineering risks; need for multiple accessways (Chapters 3.5, 4.6).
- The common basic requirements when entering and working in a declared special area.

[The incidence of special mining situations such as multi-reef mining, steep dip mining, remnant mining and mining in the vicinity of geological disturbance varies from mine to mine. Training should focus on addressing situations likely to be encountered.]

12.2.4 Support

(i) *Stopes* (Chapter 4, e.g. Figure 4.4.2)

- The function of local support: not able to reduce closure significantly, but simply to hold key blocks in position and prevent unravelling of the hangingwall rocks.
- The influence of support spacing on stability, and hence the importance of installing support close to the face.
- Actual stope support units used on the mine.
- The dangers of removing support.

(ii) *Tunnels* (Chapter 6)

- The purpose of tunnel support.
- Types of tunnel support: e.g. Split Sets, grouted tendons and the importance of good quality grouting, pre-tensioned tendons, meshing, lacing, shotcrete.

[In all the above, it is important that workers be shown effective support in an underground environment. Likewise, common types of support failure and defective performance should be pointed out.]

12.2.5 Seismicity

- Rockbursts and the associated dangers (e.g. Figures 8.3.3/4/6/7/8).
 - Rockburst-prone areas underground.
 - Limit time exposure in known seismically active areas
 - Confirmation of adequate support

12.3 STAFF WHOSE PRIMARY FUNCTION IS STOPING OR TUNNELLING

These workers need to understand all of the above plus the following:

12.3.1 Examination and Making-Safe Procedures

- Reasons for and particular hazards associated with re-entry time. [Blast damage to h/w and support, increased unsupported face span, time-dependent h/w deterioration.]
 - Recognition of hazardous conditions. When to bar and when to support. Misfires.
 - Position from where barring should be undertaken, and appropriate tools.
- [A significant number of accidents are related to barring-down operations, either due to lack of barring-down or poor barring-down practice. It is important to demonstrate proper barring-down procedures as well as potential hazards associated with this operation.]
- The potential for ground water intersection and hazardous gases.

12.3.2 Support

(i) *Stopes* (Chapter 4)

- Support appropriate for different work areas and rock conditions.
- Importance of overlap of timber units in packs.
- Perpendicularity of support units to stope floor and roof.
- Good support installation practice including reasons for and procedures to attain effective prestressing.
- Support spacing.
- The dangers of removing support, particularly temporary support, and the proper procedures for carrying out this operation.
- Support for special areas.

(ii) *Tunnels* (Chapter 6)

- Support appropriate for different work areas and rock conditions (e.g. Tables 6.4.1/2/3).
- The influence of discontinuities on instability and the need to orient tendons across these.
- The importance of properly installed temporary support:
 - the importance of quality of grouting,
 - the potential consequences of poorly installed units,
 - pinning mesh into hollows in a tunnel profile,
 - tensioning of lacing.
- Good support installation practice.
- Support spacing.

[Effective stope and tunnel support is dependent on location, orientation and spacing in the excavation. An understanding of support standards and practices on the mine is necessary.]

12.3.3 Drilling of Blast and Support Holes

- Direction and position. [The effectiveness of tendon support is highly dependent on the correct position and orientation of drill holes in relation to joints, bedding and fractures].
- Spacing.
- The consequences and hazards caused by drilling shot holes off-line into the hangingwall of stopes.
- Proper handling of explosives.

12.4 MINERS AND SHIFT BOSSES

These personnel need to understand all of the above plus the following:

12.4.1 Support

(i) *Stope and gully support* (Chapter 4)

- Load-deformation curves of different pack and elongate supports (e.g. Figures 4.2.12/13) and where and why different support types should be used.
- Support resistance of different support patterns (principles) (Chapter 4.3.3).
- Hydraulic prop performance and routine maintenance and setting requirements of the systems.
- Matching support to conditions. [The different requirements of support for rockfall and rockburst conditions].
- The fact that support standards are the minimum requirements and that additional support should be installed where conditions deteriorate.
- Gully support: packs/gully sidewall/hangingwall (Chapter 4.4.9). [The importance of installing tendons at a high angle to bedding, fractures and joints].

(ii) *Tunnel support* (Chapter 6)

- Full column grouting.
- Grout types: OPC, OPC with accelerators and other additives, resins and shelf life of different products. The importance of spin and hold times.
- End anchored tendons – pretensioning and retensioning.
- Pre-stressed tendons.
- Mesh and lacing. [The importance of good installation and the importance of lacing on performance of mesh and lacing systems].
- Shotcrete functions, properties and quality control.
- Hazards associated with support removal in tunnel rehabilitation operations.

(iii) *Installing support.*

- Prestressing (packs and tendons).
- The effect of height to width ratio on support performance (Figure 4.2.13).
- The importance of installing all support as close to the face as practical.
- The consequence of blasting if face support is not up to standard.

[An understanding of the 'zone of influence' is required for all support types used on the mine. The impact on excavation stability of too large a support spacing or missing support units needs to be understood. It should be emphasised that 'good looking conditions' do not necessarily mean safe conditions if support standards are compromised. This can be true in both rockburst and rockfall conditions.]

12.4.2 Drilling and blasting

(i) *Blasting*

- Properties of different explosive types used on the mine.
- The importance and advantages of consistent hole burden.
- Effects of over charging.

(ii) *Drilling blast holes*

- Direction of drilling to minimise host rock damage.
- Fracturing around blast holes (e.g. Figure 5.5.1a).

- Drilling patterns: Stopping, Development, Special excavations.

[The effect of poor drilling and blasting practice on underground conditions, and the degree of rock fracturing and fragmentation, needs to be highlighted. These concepts should be illustrated underground. The impact of this on support effectiveness, cost and safety should be understood.]

12.4.3 Barring

- Ability of rocks to rotate when peeling away from the rock surface.
- Dangers of unravelling of the rockmass during barring.
- Barring vs supporting.

12.4.4 Layouts – Headings and Gullies

- Fracturing around tunnels and stopes (e.g. Figures 1.3.7/9, 4.2.5/8).
- High risk areas and suitable support (Chapters 3.5, 4.6).
- Holing/trenching/approaching geological features.
- Importance of up to date sidings of correct depth (Chapter 3.4.7).

[Particular emphasis should be placed on hazardous layouts commonly occurring at the mine. The difficulties and dangers associated with poor layouts, such as the formation of dome-like fractures over wide headings or low-angled fractures should be stressed. The use of suitably designed visual aids can assist in developing this understanding.]

12.4.5 Geology

- Strong and weak rocks and failure (Chapter 1.3).
- Slip on discontinuities (Chapter 1.4.3).
- Importance of recognising and reporting changing ground conditions.

12.4.6 Pillars

- Safety factors (Figure 3.4.2).
- Influence of width:height ratio on strength and behaviour of pillars (Figure 3.2.11a), and therefore the need they be accurately cut.
- Cutting of pillars; Fracturing around pillars.
- Pillar behaviour – Non-yield, crush, and yield pillars (Chapter 3.2.8).
- The circumstances where additional pillars may be necessary.

[The importance of appropriate pillar dimensions should be stressed. For example, a crush pillar should not be too large, to ensure the pillar will crush in a stable manner and not fail violently. The stability of mining in a pillar mining situation may be compromised if pillars are of the wrong dimension, if spans are too great or if planned pillar positions are changed.]

12.4.7 Stresses

- Increase with depth (Chapter 1.3.1).
- Horizontal stresses (Chapter 1.3.1).
- Fracturing, deformation and failure (Chapter 1.3.5).

12.5 SENIOR OFFICIALS

These personnel need to be familiar with all of the above plus the following:

12.5.1 Overview

- Differentiate between layouts for shallow and deep mining (Chapters 2.2 - 2.4).
- Rock engineering advantages and disadvantages of different layouts emphasising potential hazards
- Rock mass characterisation, mining environments and ground control districts specific to the mine (Chapter 4.2.4).
- Methodology and requirements for introduction of new technology, products or procedures.
- Accident investigation - determine root cause.

12.5.2 Support Systems and Strategies for Shallow and Deep Mining

(i) *Stope Support Requirements* (Chapter 4)

- Stope support design function.
- Different stiffnesses and deformation characteristics of packs and elongates.
- Backfill and pillars as local and regional support which do prevent closure if sufficiently stiff/strong. Design of these systems.
- A basic understanding of Support Design Analysis (SDA) methodologies.
- Code of practice requirements.
- Quality assessment and control

(ii) *Tunnel Support Requirements* (Chapter 6)

- Tunnel support design
- Support resistance
- Code of practice requirement
- Quality assessment and control
- Corrosion control in tunnel support

12.5.3 Mine Layouts and Strategy

- Stresses are primarily due to the deadweight of rock up to surface.
- Understanding the *effective* depth at which mining is occurring, and its impact on mining conditions.
- Influence of the tensile zone on stability (shallow mining).
- Stresses previously supported by the removed rock are redistributed around any mined excavation.
- This leads to elevated stresses around the opening which extend some distance into the rock, depending on the opening size and shape.
- Need for regional support.
- Seismicity and rockburst control.
- Stresses sometimes exceed the strength of the rock and result in fracturing. Mining-induced fracturing leads to h/v dilation and clamping.

- Fracturing around tunnels and stopes.
- (i) *Layout principles* (Chapters 3, 5, 7)
 - On reef
 - Different depths
 - Influence of face shapes on mining conditions and ERR
 - Special mining situations (remnants & geology)
 - Special orebody geometries (wide reef, multi-reef, steep reef)
 - Off reef
 - Cross-cuts
 - Haulages
 - Infrastructure
 - Material and rock handling.
 - Boxholes
 - Gullies
 - Orepasses
- (ii) *Regional support: Stabilising pillars and bracket pillars*
 - Average pillar stress.
 - Stress to strength criterion for dykes.
 - Importance of maintaining design dimensions.
- (iii) *Pillars and remnants*
 - Safety factors.
 - Stress to strength criterion.
 - Methods for mining remnants
- (iv) *Tunnels and large excavations*
 - Effect of size.
 - Effect of excavation sequence.
 - Orientation.
 - Support.
 - Interaction between excavations.
- (v) *Mining through, or in the vicinity of geological structures*
 - Potential seismicity.
 - Choice of initial mining adjacent to or distant from a structure.
 - Face orientation of face approaching a structure.
 - Need for overstepping development or locating development deep in the footwall to avoid high abutment stresses.
- (vi) *Shafts*
 - Design and protection
 - Sinking
 - Support
 - Shaft pillar extraction
 - Design of ore-pass systems.

12.5.4 Backfill

- (i) *Regional effects* (Chapter 3.3.5).
 - The importance of filling at least 60 % of the stope.
 - Fill to face distances kept to a minimum.

- Appropriate working area support.
- Continuous filling at the face.
- (ii) *Local effects* (Chapter 4.4.8).
 - Reduction in vibration magnitude and duration during a seismic event.
 - Energy absorption compared to conventional stope support.
 - Importance of keeping fill close to face.
- (iii) *Quality assessment and control* (Chapters 3.3.5, 10.3.4).
 - Importance of porosity, specific gravity, cement and water content.
 - Method for measurement of specific gravity and porosity.
 - Monitoring programs.

12.5.5 Instrumentation (Chapter 10).

- (i) *Closure and extensometer measurements*
 - Methods of measuring closure and dilation.
 - Benefits of measuring rock deformation.
 - Assistance required to ensure reliable monitoring.
- (ii) *Stress measurements*
 - Instruments for measuring stress
 - Principles of stress measurement.
 - Benefits of stress measurement.

12.5.6 Seismicity and Seismic Systems (Chapter 1.4, Chapter 9)

- Hardware required to operate a basic system.
- Basic layout of a system:
- Event location & magnitude.
- What can be learnt from seismic data.

12.5.7 Rockburst control.

- Preconditioning methodology, productivity and rock engineering advantages (Chapter 8.4).

12.5.8 Numerical Modelling (Chapter 11)

- Available rock engineering software and their capabilities in term of time and solutions.
- The limitations of modelling.

12.5.9 Geology (Chapters 1.3, 4.2, 5.2)

- Formation of different rock types viz. sedimentary, igneous and metamorphic.
- Understand that different materials have different strengths etc.
- Definition and description of mining environments and ground control districts on the mine.
- The causes of seismicity.
- Measures to prevent damage, injuries and fatalities

[In all training categories mentioned above, discussion of accidents from the mine's accident data-base which involve topics being taught will improve the understanding of their importance and relevance.]

[Members of the Safety Committee, from whatever job category, should be given the opportunity to progress through all the rock engineering training and education modules in order to fulfil their function more effectively and efficiently.]

12.6 ROCK ENGINEERING PRACTITIONERS

In the DME's document 'Guideline for the compilation of a mandatory code of practice to combat rockfall and rockburst accidents in metalliferous mines and mines other than coal', reference is made to properly or suitably qualified personnel in a mine's rock engineering department. In their definition, a suitably qualified rock engineering practitioner is a person who is in possession of at least the Chamber of Mines Certificate in Rock Mechanics, now the 'Certificate in Rock Mechanics' offered by Teknikon SA.

There are currently no undergraduate degrees offered in rock mechanics at South African Universities. However, short rock mechanics courses are offered in mining, engineering geology and civil engineering degrees. Post graduate degrees in rock engineering may be obtained through course work and/or a dissertation through the mining departments at the Universities of the Witwatersrand and Pretoria.

There are three locally recognised non-degree qualifications in rock mechanics: the Certificate in Strata Control, the Certificate in Rock Mechanics, and the Advanced Certificate in Rock Engineering. The former two courses are offered at Teknikon SA and the third is offered at the Universities of the Witwatersrand and Pretoria through their Graduate Diploma Courses. A prerequisite for the Advanced Certificate in Rock Engineering at both universities is a Certificate in Rock Mechanics.

It is suggested that those in possession of the Certificate in Strata Control operate up to the level of Rock Mechanics officer on a mine. A Certificate in Rock Mechanics should be a prerequisite for Section Head or Rock Engineering Manager on a mine. This position also requires considerable experience, and it is recommended that such a person demonstrate sufficient experience in rock mechanics and rock engineering and knowledge of the mine before being appointed to the position. The suitability of a qualification will change from time to time in line with recommendations made by the South African and the Mines Qualification Authorities.

12.7 IMPLEMENTATION OF NEW TECHNOLOGIES

12.7.1 Introduction

There has been a constant stream of new developments in the rock engineering arena over the past years. Such developments are generally aimed at improved safety on the mines, and the challenge is to see these new technologies implemented in the industry and the safety record improved. The introduction of codes of practice has increased the profile and responsibility of rock engineering in the mining industry,

which in turn presents an opportunity and an obligation to show the benefits of new technology.

The introduction of new rock engineering technology will never be easy. Many new technologies, for example, are based on an understanding of rock behaviour that may not immediately be apparent to some levels of production personnel. The technology may then seem inappropriate or irrelevant, and may not be given the initial encouragement to prove itself. The need therefore exists to devise effective ways of conveying new ideas and technologies in a simple way, perhaps visually, stressing the benefit to the mining operation. This will require a sound understanding of the new technology and of the mine's problems, and how the technology relates to these. Where appropriate, the underlying principles of new technologies should be communicated to production personnel. This will require good communication skills and is related to the training function that the rock engineering department of the mine should provide.

12.7.2 Implementation Strategies

- *Understand the new technology.*

The implementation methodology will depend on the nature of the new technology, which may be a new concept of mine design or strata control, or may be a new piece of equipment or hardware.

In the former case (a new concept of some type), the implementation process will probably require the decisions of senior management to proceed, and these same decision makers are likely to monitor progress. Only a limited number of people need to be convinced and motivated as to the benefits of the technology. Therefore, these types of new technologies are most likely to succeed on the mines.

When introducing new equipment and hardware to a mine, a much wider group will be involved. This complicates the technology transfer, as more people have to be motivated to accept the change, large changes in organisation and/or mining system are more likely to be required, and considerable additional human resource time may be required.

- *Show the benefits.*

In transferring new rock engineering technology into the mines, it is important to demonstrate the benefit to the production personnel from a safety and production point of view. Clear technical reasons should be demonstrated and the underlying theory should be translated into layman terms. A simple but accurate financial justification for the introduction should also be presented, including a consideration of the initial costs and the expected delay in return on the investment in the new technology. Awareness should be raised as to the possibility or probability of early teething problems.

- *Select Champions.*

For any technology transfer to succeed it is essential to have champions on the mine who will drive the process of implementation. It is wise to have the strong support of senior management who will both drive and monitor the progress of implementation. It is important that such champions are fully committed and convinced of the benefits of the new technology at the start of the process of implementation. As the process progresses, they should be kept fully briefed at all times regarding progress, successes and failures. An accurate log of all events should be maintained.

- *Involvement of workforce from the bottom up.*

Depending on the type of technology to be transferred to the mine, all personnel who are effected in any way by the technology should be involved from the beginning of the implementation process. This may mean selecting a sample group that represents a larger group, or an entire group, depending on the technology. There is great danger of rejection of new technology if it is thrust on personnel without their being involved from an early stage in making the choice to embark on its use. Involving a wide spectrum of the workforce will require a varied approach to communicating the benefits and correct use of new technology, and the rock engineer will need to develop skills in this area.

- *Summary*

1. Be familiar with new technology, and understand the mine's problems.
2. Select a technology that should make a noticeable contribution.
 - Study it intensively and understand the benefits (cost and safety)
 - Determine whose consent is required for successful implementation
 - Determine on whom it will make an impact
 - 'Sell' the technology
3. Recognise that the problems of the introduction of a new process or design are different from those of a new piece of equipment.
4. Plan or assist in the implementation
5. Train at all levels
 - Monitor and record the problems of implementation
 - Trouble shoot
6. Evaluate the new technology and report the results, including successes and failures.

12.8 RISK ASSESSMENTS

The Mine Health and Safety Act requires the manager to identify the health and safety hazards on a mine, assess the risks of such hazards and implement reasonable measures to control the risk.

As a result, over the past few years, the use of risk assessment techniques in mining and rock engineering has become more widespread and more formally applied. The participation of all levels of staff in the risk assessment process is seen as essential. These are people who are daily at risk and so are well placed to contribute to this process.

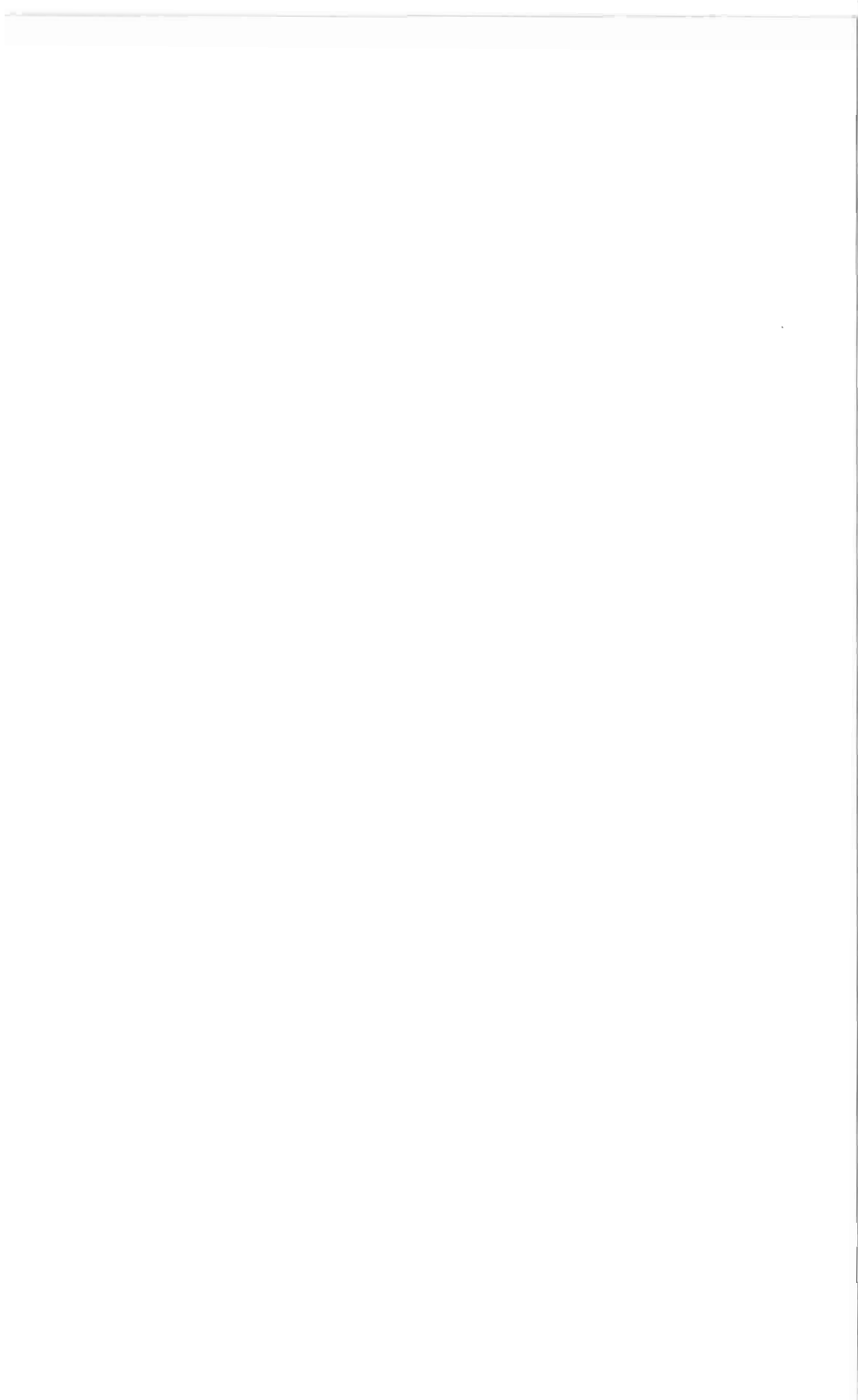
Risk assessments relating to rock engineering and safety need to be considered for a wide range of issues as diverse as the overall mining method or regional support strategy, through design processes, staffing of the rock engineering department, to the more day to day issues such as the risks involved in cutting of pillars, barring and making safe or the introduction of a new support unit. In addition to all the operations and procedures carried out on a mine that need to be assessed, it is also incumbent on suppliers of equipment to mines that they carry out risk assessments on their products. This could involve the identification and control of not only the potential hazards in the installation or use of the product, but also in the supply of raw material or component parts, the manufacturing process and on the instructions supplied for the storage and application of the product. Because of the many procedures that

potentially would require risk assessment, priorities need to be set. Here, analysis of accident data required to be kept by each mine, would provide useful information on high-risk areas or procedures.

Formal risk assessments are best carried out according to the Tripartite Risk Assessment Guidelines provided by SIMRAC. These involve thorough preparation for the assessment by identification of the most important issues to be assessed (analysis of the accident database will highlight high-risk areas or procedures and indicate possible root causes); collation of relevant data; and assemblage of the appropriate spectrum of participants (e.g. for a stoping-related issue, this could include production personnel from operator to mine overseer or section manager, safety officials, union representatives, rock engineer and geologist).

The assessment itself should be a systematic process led by a trained facilitator. It would start by identifying all the hazards related to the operation and the control measures currently in place to reduce the hazard or risk. The remaining risks, taking into account the control measures, are then approximately quantified and the major issues identified. The main purpose of the risk assessment is then addressed: that of devising additional control measures that would minimize or ideally eliminate the risk. Based on these findings, practicable measures to control risks would be implemented, and at the same time responsibilities in this regard would be set.

The benefits of regular risk assessments lie not only in the improved procedures and systems that are put in place, but also in their contribution to the training process by developing improved employee knowledge and skills; and, importantly, in the provision of excellent, open communication channels between employees, supervisors and management. A good example of this is regular informal risk assessments carried out in stopes, to assess current hangingwall conditions and the need for any change in support standards.



Glossary of Terms and Index

The following Glossaries cover two specialised areas: terminology used in the *support of tunnels* and other service excavations, and definitions and brief discussions of parameters used in *seismic monitoring* and seismic hazard assessment. Definitions of the many other specialized terms used in rock engineering can be found by referring to the Index at the end of this book, in which the page numbers point to sections of the main text where the topic is **defined** (page number highlighted in **bold**) or otherwise discussed in context.

Glossary 1. Tunnel support terminology

The nomenclature used for tunnel support units and elements has never been standardized: synonyms abound and incorrect terminology is often used. This is a worldwide phenomenon, which the ISRM has recognized and is addressing by setting up a commission to provide a recommended terminology to improve communication and understanding in this area. The following list of definitions or descriptions are the preferred meanings that have been given to the terms used in this book; acceptable [synonyms] are also indicated.

Support units, elements	A generic term, including all types of mechanical devices used for rock reinforcement and containment; e.g. end-anchored rockbolts, grouted bars and cables, steel arches, mesh and lacing (the latter often referred to as <i>fabric support</i>). Reinforcing units can be classified as: (i) Discretely-coupled (mechanical or frictional), using expandable wedges or resin, together with external bearing plates which are pretensioned to provide active support. Examples: bolts, studs, cables, yielding or non-yielding cone-bolts. (ii) Continuously-coupled (mechanical). Examples: all fully grouted bars and cables. (iii) Continuously-coupled (frictional). Examples: Split Sets, Swellex.
Bearing plates [Face plates, Washers]	These plates provide the external fixture necessary for end-anchored bars and cables and for anchoring mesh, but can also be used with grouted tendons to improve their areal support effectiveness in a highly discontinuous wall rock. They comprise flat or usually domed plates to accommodate spherical seats to facilitate installation on an irregular surface. They can be supplied with upturned corners with holes or with welded steel bar loops for threading and anchoring lacing. The eye or loop of a <i>shepherd's crook</i> bar plays a similar role.
Linings [Coatings]	Materials usually sprayed onto rockwalls to provide passive support and bonding of fragmented rock material; e.g. cementitious shotcretes, polymer membranes, formed concrete linings (for shafts, stations, orepasses etc).
Support system	A combination of one or more rock reinforcement unit types, with or without fabric support or lining.
Support system performance:	the overall combined force-deformation behaviour of the component elements in terms of stiffness, peak load, yieldability and energy absorption capacity. These values are normally controlled by the weakest element; thus a design which is balanced with regard to the properties of all the elements is the most cost-effective.

Rockbolt [Roofbolt]	Used as a generic term for all types of <i>inflexible</i> rock reinforcement units, as well as to the process of rock reinforcement (e.g. "roofbolting"). Often used specifically for end-anchored bars with bearing plates, spherical seats and tensioning units. (The correct term for such a unit is <i>rockstud</i> , but this is seldom currently used; while a true <i>roofbolt</i> was originally a long bolt with forged head used with a mechanical end anchor.)
Tendon [Support tendon, Reinf. tendon]	Includes the generic "rockbolt", plus <i>flexible</i> forms such as "cable anchors".
Grout	(i) Cementitious or resinous material used to bond reinforcing bars or cables to the rockwalls of installation boreholes. Plasticizers and thixotropic agents are usually added to enhance installation. The grout can be either pumped into the borehole to fill the annulus between bar and rock, or preloaded into the boreholes in capsule form and through which the bar must be forced. (ii) Cementitious or resinous material used to bond highly fragmented rock together, providing "internal" support for "running" dykes etc., or to prevent <i>ground water</i> seepage and seal off water fissures.
Fully grouted tendons	Bars or cables grouted along the full length of the borehole. They have a stiff response at the point of loading, such as at a dilatant fracture located anywhere along its length, if the grouted length beyond the fracture is greater than the <i>critical bond length</i> . Grouting also provides some protection against corrosion.
Critical bond length:	that minimum bonded length of a particular tendon and grout combination that develops a pull-out resistance equal to that of the tensile strength of the tendon.
Fabric support: - Mesh	Steel mesh is the most commonly applied fabric support in South African mines. Galvanised or other corrosion-protected material is available.
- Weld mesh	This comprises a square pattern (50 mm - 100 mm) of steel wires welded at their crossing points. This mesh has the support advantage of being relatively stiff compared to other types, and localized failure does not spread to the rest of the installation. The stiffness makes for greater difficulties in installation, and the material is weak in bending.
- Diamond mesh	This comprises interlaced steel wires (3 mm - 4 mm) with common aperture sizes of 50 mm to 75 mm. The mesh is flexible in one direction and is thus relatively easy to install. It has a low stiffness, and local failure of a few wires weakens a significant portion of the installation.
- Woven mesh	This comprises a square pattern of interlaced "crinkle" wire. It is fairly stiff but can be moulded reasonably well over humps and into hollows in the rock surface. It is thus well suited for use with shotcrete when material with apertures of about 70 mm is selected.
- Expanded metal	This very rigid form of support is occasionally used in critical situations, such as in shaft sinking through poor ground.

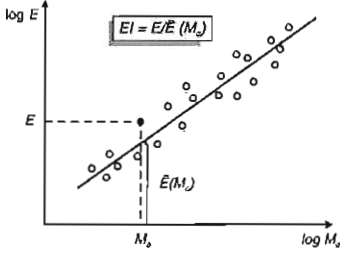
Lacing	Steel rope lacing between tendons is often sufficient fabric support for moderately jointed rockwalls. It is an essential component of fabric support where flexible mesh is used in situations where large wall rock dilation or dynamic loading are expected, providing extra strength and stiffness. Various patterns of lacing can be used depending on stiffness requirements and tendon spacing. De-stranded hoist rope is economic and usually suitable for lacing, but more flexible rope types can be more readily installed and pretensioned.
Shotcrete [Sprayed concrete, Gunite]	This is a mixture of cement, aggregate and water which is pumped pneumatically through a nozzle onto the walls of an excavation to form a bonded coherent layer. It may contain admixtures, additives and fibres or a combination of these to improve tensile, flexural and shear strength, resistance to cracking and shock resistance.
Wet shotcreting	This is a process whereby cement, aggregate and water, together with any prescribed additives, are batched and mixed prior to being fed into a pump and conveyed through a pipeline to a nozzle, where the mixture is pneumatically and continuously sprayed into place.
Dry shotcreting	This is a process whereby dry cement and aggregate together with any prescribed additives are batched, mixed and fed into a special machine where the mixture is pressurized, metered into a compressed air stream and conveyed through hoses or pipes to a nozzle where water is introduced as a spray to wet the mixture, which is then projected continuously into place.
Rebound losses:	wasted material which, having been sprayed, does not adhere to the required surface.

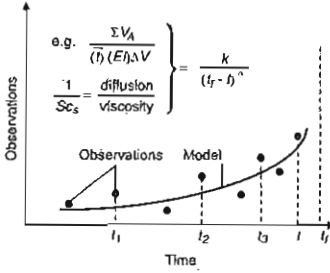
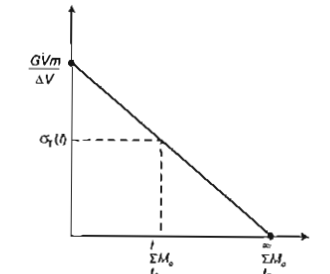
Glossary 2. Parameters used in seismic monitoring and hazard assessment.

In the following table, the symbol E refers to radiated energy from a seismic event (not Young's modulus), G is the bulk modulus of the rock mass [$G = \text{Young's mod.}/2(1+\nu)$], and m (not M) denotes event magnitude.

Parameter, relevant formula	Description
Location (Hypocentre)	The 3-D location of a seismic event, as determined by best analysis of seismogram data.
Epicentre	The same location, as plotted on a plan.
Magnitude m $m = \log(A/T) + C$ where A/T = max. displacement over associated period for P or S waves, C = correction for path effects, site response and source region	Magnitude is a relative measure of the strength of a seismic event based on measurements of maximum ground displacement at a given frequency at multiple seismic sites. A unit increase in magnitude corresponds to a 10-fold increase in amplitude of ground displacement. Gutenberg and Richter related seismic energy and magnitude derived from P-waves recorded at large distances from the source at 1sec period as $\log E(\text{ergs}) = 2.4m + 5.8$.

Parameter, relevant formula	Description
Seismic moment M_0 [Nm] and Moment-magnitude m $m = 2/3 \log M_0 - 6.1$	A scalar that measures the coseismic inelastic deformation at the source. Since seismic moment is proportional to the integral of the far field displacement pulse, it can easily be derived from recorded waveforms. A relation that scales seismic moment into magnitude of a seismic event is called moment-magnitude.
Seismic moment tensor $\Delta f_{ij} = \int_V c_{ijkl} \Delta \epsilon_{kl} dV = \int_V \Delta \sigma_{ij} dV$ where c_{ijkl} = elastic constants $\Delta \epsilon_{kl}$ = strain change at the source $\Delta \sigma_{ij}$ = stress change or change in moment per unit volume $\Delta \theta = (M_{11} + M_{22} + M_{33}) / E'$ where $E' = (\text{Young's Mod.}) / (1-2\nu)$	The most general description of the processes at the seismic source V , is by the distribution of forces or moments equivalent to the inelastic deformation. For long waves compared to the source size, the whole source volume V can be considered to be a system of couples located at, say, the centre of V , and the moment tensor components can be defined by the equation at left. The moment tensor measures the inelastic deformation at the source during the seismic event, and its value at the end of the source process measures the permanent inelastic strain produced by the event. The seismic moment tensor can be decomposed into isotropic (or volume change) and deviatoric (i.e. shear) components, providing an additional insight into the nature of the coseismic strain changes. For a homogeneous body, the coseismic volumetric change $\Delta \theta$ can be calculated from the second equation at left. The eigenvalues and corresponding eigenvectors of the deviatoric component of the seismic moment tensor describe the magnitude and orientation, respectively, of the principal moment axes (neglecting gravity) acting at the source. These principal moment axes are uniquely determined by moment tensor inversion. Principal moment orientation data can provide sufficient information to find the best stress tensor.
Radiated seismic energy E [J]	The portion of the energy released or work done at the source that is radiated as seismic waves. Seismic energy is proportional to the integral of the squared velocity spectrum in the far field and can be derived from recorded waveforms. Radiated seismic energy increases with stress drop, seismic moment and with the traction rate.
Corner frequency f_0 [Hz] and Source size l [m] $l = c_1 / f_0$ $c_1 = 2500$ for S-wave in hard rock	The frequency at which a source radiates the most seismic energy observed as the maximum on the source velocity spectrum, or as the point at which a constant low frequency trend and a high frequency asymptote on the recorded source displacement spectrum intersect. The corner frequency is inversely proportional to the characteristic size of the source.
Stress drop $\Delta \sigma$ [Pa] $\Delta \sigma = c_2 M_0 f_0^3$ $c_2 \approx 1.8 \times 10^{-10}$ for S-wave in hard rock. The 'strain drop' $\Delta \epsilon = \Delta \sigma / G$.	Stress drop estimates the stress release at the seismic source. Although it is model-dependent, it provides reasonable estimates and a fair comparison amongst different sources from the same region recorded by the same seismic system.
Source area [m ²] $A = M_0 / (Gu)$ where u = av. source displ.	The area of coseismic inelastic deformation over the planar source.
Source volume [m ³] $V = M_0 / \Delta \sigma$	The 'volume' of coseismic inelastic deformation, scaled by $\Delta \sigma / G$ (e.g. a scaled 'volume of ride' for a pure shear-type event).

Parameter, relevant formula	Description
Apparent stress [Pa] $\sigma_A = GE / M_0 = E / (\Delta \epsilon V)$ or $\sigma_A = E / (uA)$.	Apparent stress is recognised as a model-independent measure of the stress change at the seismic source.
Apparent volume [m ³] $V_A = M_0 / (c_3 \sigma_A) = M_0^2 / (c_3 G E)$ where c_3 = scaling factor ≈ 2 .	The 'apparent volume' also scales the volume of coseismic inelastic strain in terms of stress drop over rigidity. The apparent volume V_A is less model-dependent than the source volume V .
Energy index EI 	The notion of comparing the radiated energies of seismic events of similar moments can be translated into a practical tool called Energy Index (EI) – the ratio of the radiated energy of a given event (E) of moment M_0 to the energy $\bar{E}$ derived from the regional $\log E$ vs $\log M_0$ relation. Since $\log \bar{E} = c + d \log M_0$, then $\bar{E} = 10^{c+d \log M_0}$ where c and d are constant for a given ΔV and Δt. In general, the d-value increases with the system's stiffness and c increases with stress. A small or moderate event with $EI > 1$ indicates a relatively high stress drop and suggests a higher than average shear stress at its location. The opposite applies to the $EI < 1$ case.
Seismic strain , $\epsilon_s = \Sigma M_0 / (2G \Delta V)$ and Seismic strain rate $\dot{\epsilon}_s = \epsilon_s / \Delta t$	Seismic strain measures strain due to cumulative coseismic deformations within the volume ΔV over the period Δt. Its rate is measured by $\dot{\epsilon}_s$.
Seismic stress [Pa] $\sigma_s(\Delta V, \Delta t) = 2G \Sigma E / \Sigma M_0$	Seismic stress measures overall stress changes due to seismicity.
Seismic stiffness K_s [Pa] $K_s(\Delta V, \Delta t) = \sigma_s / \epsilon_s$ $= 4 G^2 \Delta V \Sigma E / (\Sigma M_0)^2$	Seismic stiffness measures the ability of the system to resist seismic deformation with increasing stress. The stiffer systems limit both the frequency and the magnitude of intermediate and large events, but have a time-of-day distribution with larger statistical dispersion and thus are less time-predictable.
Seismic viscosity [Pa.s] $\eta_s(\Delta V, \Delta t) = \sigma_s / \dot{\epsilon}_s$	Seismic viscosity characterises the statistical properties of the seismic deformation process. Lower seismic viscosity implies easier flow of seismic inelastic deformation or greater stress transfer due to seismicity.
Seismic relaxation time [s] $\tau_s(\Delta V, \Delta t) = \eta_s / G$	Seismic relaxation time quantifies the rate of change of seismic stress during seismic deformation processes, and it separates the low frequency response from the high frequency response of the system under consideration. It also defines the usefulness of past data and the predictability of the flow of rock. The lower the relaxation time, the shorter the time span of useful past data and the less predictable the process of seismic deformation.
Seismic Deborah number $De_s(\Delta V, \Delta t) = \tau_s / \text{flowtime}$ where <i>flowtime</i> is a design parameter not necessarily equal to Δt .	Seismic Deborah number measures the ratio of elastic to viscous forces in the process of seismic deformation, and has successfully been used as a criterion to delineate volumes of rockmass softened by seismic activity ('soft clusters'). The lower the Deborah number, the less stable is the process or the structure over the design <i>flowtime</i> – what may be stable over a short period of time (large De_s) may not be stable over a longer time (lower De_s).

Parameter, relevant formula	Description
Seismic diffusivity [m^2/s] $D_s(\Delta V, \Delta t) = (\Delta V)^{2/3} / \tau_s$ Or, in a statistical sense, $d_s = \bar{X}^2 / \bar{t}$	Seismic diffusivity can be used to quantify the magnitude, direction, velocity and acceleration of the migration of seismic activity and associated transfer of stresses in space and time. There is an inverse relationship between the diffusivity D_s and the friction parameters.
Seismic Schmidt number $Sc_{sd}(\Delta V, \Delta t) = \eta_s / (\rho D_s)$ or $Sc_{sd} = \eta_s / (\rho d_s)$ where ρ is rock density.	Seismic Schmidt number measures the degree of complexity in space and time (the degree of turbulence) of the seismic flow of rock. Note that seismic Schmidt number Sc_{sd} , encompasses all four independent parameters describing seismicity: $\bar{t}$, $\bar{X}$, ΣM_0 , ΣE .
Time to failure ($t_f - t$) [sec] $d\Omega / dt = k / (t_f - t)^\alpha$ where Ω = measurable quantity t = current time t_f = time of failure k, α = constants 	This concept describes the behaviour of materials in the terminal stages of failure. It views instability as a critical point. Precursors would then follow characteristic power laws in which the rate of strain or other observable quantity Ω is proportional to the inverse power of remaining time to failure. Observed oscillations in Ω of an increasing frequency as the failure approaches are part of the solution to the time-to-failure equation with a complex exponent, where the imaginary part relates to discrete scale transformation and introduces log-periodic oscillations decorating the asymptotic power law. The observations Ω can be a combination of different seismic parameters that would exhibit a power law type increase before failure. For well behaved data sets, the time at failure t_f can be estimated from the times of three successive maxima (t_1, t_2, t_3) of the observed process $t_f = (t_2^2 - t_1 t_3) / (2t_2 - t_1 - t_3)$. Note that, in theory, $t_3 - t_2 < t_2 - t_1$.
Seismic moments, volume mined and relative stress 	If a volume of rock V_m is mined out at time t_0 , and if the altered stress and strain field can readjust to an equilibrium state through seismic movements only, the sum of seismic moments released within a given period of time should be proportional to the excavation closure and in the long term at $t = \infty$ $\Sigma M_0 \approx G V_m$ where M_0 is the scalar seismic moment. The relative stress level at the time t , in a given volume of rock ΔV surrounding the excavation, can be calculated from the difference between $G V_m$ and the cumulative moments released to date: $\sigma_r(t) = (G V_m - \Sigma M_0) / \Delta V$.
Seismic moments and volume of elastic convergence $\Sigma M_0 = \gamma G V_e$	The amount of strain energy stored when mining in elastic rock is directly proportional to the volume of elastic convergence V_e . It has been found that the total amount of seismic moment resulting from mining within a large area and time period is related to the change in elastic convergence V_e . The proportional constant gamma, γ , has been found to vary between about 0.03 and 1.0. There is some evidence that γ is a function of the geotechnical area being mined.

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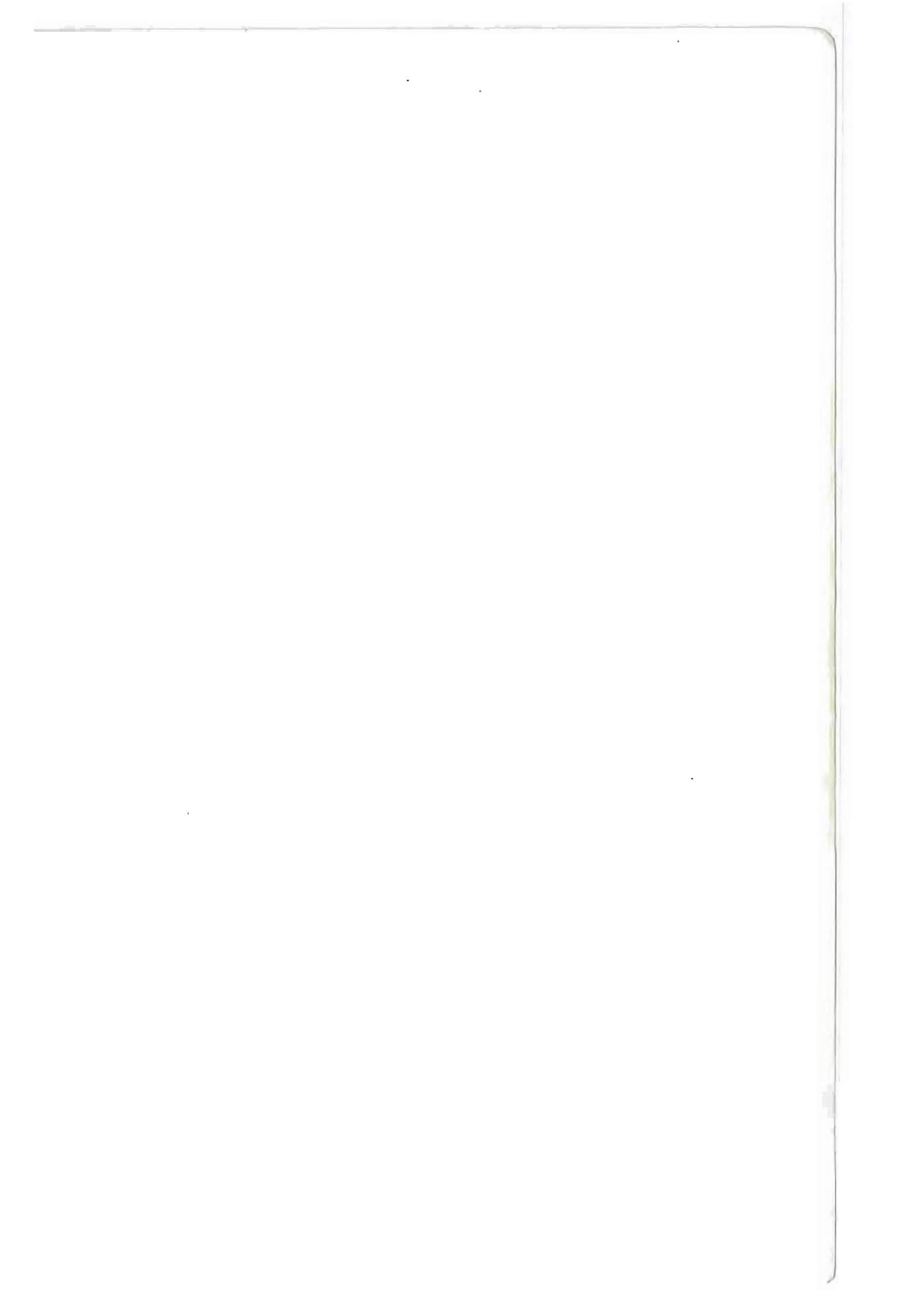
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