

# **SAFETY IN MINES RESEARCH ADVISORY COMMITTEE**

**DRAFT. FINAL PROJECT REPORT**

## **Strata control in tunnels and an evaluation of support units and systems currently used with a view to improving the effectiveness of support, stability and safety of tunnels**

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# Executive Summary

The design of support systems, comprising rock bolt reinforcement and fabric containment components for tunnels in deep level mining environments, does not currently cater well for adverse rock mass conditions. This often results in periodic failure of the support system, particularly under dynamic (rockburst) conditions, with the potential for total collapse of the excavation. The design of support systems is currently based on either empirical design guidelines often not applicable to this environment or simple mechanistic models.

This report details a methodology for the rational design of tunnel support systems based on a mechanistic evaluation of the interaction between the components of a tunnel support system and a highly discontinuous rock mass structure. This analysis is conducted under both static and dynamic loading conditions. Due to the highly complex and variable nature of the rock mass structure and the dynamic loading environment, a large component of the practical work on the evaluation of the mechanisms of rock mass deformation and support interaction is based on rockburst case studies. The understanding gained from these investigations is further evaluated by means of laboratory testing of the performance of the components of the support systems, numerical modelling of the interaction of the components of the support system with the rock mass and in situ monitoring. This report is compiled to satisfy the requirements of output 10 of GAP335 for 'recommendations on design of support systems and design methodology'.

Due to the complex nature of this design environment, the methodology developed in this report is but a step towards our greater understanding of the behaviour of the rock mass, and the interaction of support systems in the stabilisation of tunnel excavations. However, in comparison to empirical design, this methodology now allows the design engineer to make better estimations of the anticipated demand on the different components of the support systems, under a defined rock mass environment, based on engineering principles. This understanding will give the design engineer greater flexibility and confidence to design an appropriate tunnel support system for a specific rock mass and loading condition. This often has to be conducted with the support units at the operator's disposal, unlike empirical support classifications where significant differences in support components may be stipulated for slightly different rock mass classes.

# Preface

The work detailed in this document addresses the identified need for an improved understanding of the stability and support requirements of tunnel excavations in adverse rock mass environments in the South African mining industry and the development of a rational design methodology for tunnel support systems.

In order to address the defined limitations in the current support system design, and, thus, the needs of the South African mining industry, it was considered appropriate to examine the interaction of the support systems with the rock mass on a mechanistic basis. This would enable the development of a rational design methodology based on an understanding of the interaction between the components of the support system and the rock mass environment.

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# Table of Contents

1	Introduction.....	18
1.1	Nature of problem and previous work .....	18
1.2	Scope of research .....	19
1.3	Objectives .....	20
2.	Literature survey .....	21
2.1	Introduction.....	21
2.2	Rock mass behaviour around highly stressed tunnel excavations.....	21
2.3	Design of tunnel support systems .....	23
2.3.1	Structural Analysis .....	24
2.3.2	Empirical design methodologies .....	32
2.4	Conclusions .....	46
3	Research Methodology .....	47
3.1	Introduction.....	47
3.2	Observations of rock mass behaviour and support performance ....	48
3.3	Definition of the depth of rock mass instability (stress induced failure) .....	51
3.3.1	Accident data base evaluation of depth of instability .....	51
3.3.2	Case study evaluation of depth of instability .....	53
3.4	Definition of stress driven rock mass behaviour .....	57
3.4.1	Rock mass deformation mechanisms .....	57
3.4.2	Influence of rock bolt reinforcement on magnitude of deformation. ....	61
3.5	Analysis of support systems and rock mass interaction .....	71
3.5.1	Numerical modelling analysis – rock mass retainment / containment. ....	73
3.5.2	Numerical modelling analysis – reinforced rock mass structure. ....	79
3.5.3	Numerical analysis – excavation shape .....	82
3.5.4	In situ monitoring .....	83
3.5.5	Conclusions – support system interaction. ....	90
3.6	Performance of support system components .....	90
3.6.1	Shear capacity of rock bolt systems .....	91
3.6.2	Load-deformation characteristics of mesh and lace support .....	93
3.6.3	Conclusions of support system component performance.....	97
4	Discussion of results .....	97
4.1	Support design philosophy.....	97
4.2	Application of design methodology .....	103
4.2.1	Evaluation of rock mass environment .....	103
4.2.2	Design based on rock mass reinforcement .....	110
4.2.2.1	Application of design procedure for rock mass containment / retainment.....	110
4.2.2.2	Application of design procedure for rock mass structural reinforcement .....	115
4.2.3	Fabric demand and support system interaction .....	117
5	Conclusion and recommendations .....	125
6	References .....	131

# Figures

Figure 1-1.	Aspects of investigation for the proposed tunnel support design methodology. ....	20
Figure 2-1:	Support of key block structure (after Stillborg 1986) .....	25
Figure 2-2:	Support of sliding block (after Stillborg 1986). ....	26
Figure 2-3:	Suspension of laminated rock mass based on tributary area loading (after Stillborg 1986) .....	27
Figure 2-4:	Formation of unstable zone and natural stable arch within moderately jointed rock mass structures (after Stillborg 1986). ...	28
Figure 2-5:	Creation of reinforced beam within stratified rock mass (after Stillborg 1986). ....	30
Figure 2-6:	Creation of reinforced rock mass arch within highly discontinuous rock mass structures ( after Stillborg 1986 ) .....	31
Figure 2-7.	Semi-empirical design of the extent of instability around an excavation based on RMR (after Stimpson (1989)) .....	36
Figure 2-8.	Support recommendations based on the Q system (Barton, 1997). ....	41
Figure 2-9.	Determination of support pressure from Q value and joint roughness condition (Barton 1997). ....	42
Figure 2-10.	Approach for support design under rockburst conditions after Kaiser et al. (1996) .....	45
Figure 2-11.	Rockburst damage mechanisms as defined by Kaiser et al. (1996) .....	45
Figure 2-12.	Primary functions of support elements. ....	46
Figure 3-1.	Analysis of tunnel fall of ground data for the South African gold mining industry .....	52
Figure 3-2.	Analysis of tunnel rockburst ejection thickness data for the South African gold mining industry. ....	52
Figure 3-3.	Definition of equivalent excavation dimension and depth of instability (Kaiser et al. 1996) .....	54
Figure 3-4.	Relationship between induced stress and depth of instability after Martin et al (1997). ....	54
Figure 3-5.	Ground velocity with distance from seismic event (Anon., 1988) ...	56
Figure 3-6.	Determination of anticipated maximum induced dynamic stress due to a defined seismic event. ....	56
Figure 3-7.	Revised relationship of depth of instability for additional South African case studies. ....	57
Figure 3-8.	History of deformation with vertical stress change from Buffelsfontein case study (Haile 1998) .....	58
Figure 3-9.	Analysis of deformation mechanisms with vertical stress increase. ....	58
Figure 3-10.	History of hangingwall deformation with vertical stress reduction from Hartebeestfontein case study. ....	59
Figure 3-11.	Proposed mechanism of hangingwall and sidewall deformation due to vertical stress reduction. ....	60
Figure 3-12.	UDEC model of rock mass deformation around a square tunnel under vertical stress reduction. ....	61
Figure 3-13.	Influence of rock bolt density of hangingwall deformation from Buffelsfontein case study (Haile 1998) .....	62
Figure 3-14.	Dilation of hangingwall rock mass with depth into the hangingwall and the influence of rock bolt reinforcement density. ....	63
Figure 3-15.	The influence of support density on sidewall deformation for updip (U)and downdip (D) sidewalls under vertical stress increase and decrease. ....	64
Figure 3-16.	The influence of support density on sidewall dilation with depth into rockwall. ....	64

Figure 3-17.	Measurements of total closure of the sidewalls at the Kloof site. ....	65
Figure 3-18.	Closure rate (mm/MPa) with time at Kloof experimental site. ....	66
Figure 3-19.	Extensometer results for 'up-dip' sidewall at Kloof site. ....	66
Figure 3-20.	Rock mass dilation with depth for Kloof instrumentation site. ....	67
Figure 3-21.	Development of deformation for an increase in the vertical stress component between 89MPa and 90 MPa .....	68
Figure 3-22.	Development of deformation for an increase in the vertical stress component between 89 MPa and 108 MPa .....	69
Figure 3-23.	Mapping of the development of fracturing with vertical stress increase. ....	70
Figure 3-24.	Maximum development of fracturing at peak vertical stress level of 108 MPa. ....	70
Figure 3-25.	Width of fracture with depth into sidewall of the tunnel at the Kloof site. ....	71
Figure 3-26.	Tributary area loading and stable anchorage of rock bolt reinforcement within a support system based on current design practice (Anon. 1996a). ....	72
Figure 3-27.	Example of numerical modelling of rock mass stability between rock bolt reinforcement. ....	74
Figure 3-28.	Chart of relative stability of rock mass structure between rock bolt reinforcement based on numerical modelling. ....	74
Figure 3-29.	Chart of the influence of rock bolt spacing, based on rock mass classification as derived from Figure 3-28, on depth of instability between the rock bolts. ....	75
Figure 3-30.	Chart illustrating the increased potential for rock mass unravelling within a rock mass class per unit increase in dynamic ground motion. ....	76
Figure 3-31.	Schematic representation of the interaction of the components of a support system within a discontinuous rock mass structure. ....	77
Figure 3-32.	"Ground reaction curve" for class C rock mass and 3 m reinforcement spacing. ....	78
Figure 3-33.	Analysis of minimum fabric stiffness to maintain the inherent rock mass strength for the given rock mass class and rock bolt reinforcement spacing. ....	79
Figure 3-34.	Numerical analysis of reinforced rock mass structure. ....	80
Figure 3-35.	Relationships between beam geometry, rock mass class and maximum energy absorption capacity for rock bolt density of 1 unit/m <sup>2</sup> . ....	81
Figure 3-36.	Influence of rock bolt reinforcement density on reinforced rock mass beam capacity. ....	82
Figure 3-37.	Structural deformation per unit of sidewall dilatatory stress (MPa) of reinforced shell for a base case of 3.5 m square excavation with 1.25m structural reinforcement. ....	83
Figure 3-38.	Rock mass dilation with distance from rock bolt reinforcement. ....	84
Figure 3-39.	Load and deformation characteristic for 0.5 m anchor in shale rock type. ....	85
Figure 3-40.	Summaries of peak pull out loads for rock mass type and anchor depth. ....	85
Figure 3-41.	Relationship between the peak ground velocity and distance from rock bolt reinforcement unit. ....	86
Figure 3-42.	Relationship between rock mass discontinuity and the relative amplification of peak ground velocity at a distance of 40% of the rock bolt spacing compared to at the rock bolt location. ....	87
Figure 3-43.	Distribution of significant damage at the simulated rock burst site (Le Bron, 1998) .....	88
Figure 3-44.	Attenuation of peak ground velocity along tunnel axis and estimated limit of rock burst damage. ....	88

Figure 3-45.	Extent of dynamic damage relative to rock bolt reinforcement with intensity of dynamic loading. ....	89
Figure 3-46.	Amplification of peak ground velocity with distance from rock bolt unit. ....	90
Figure 3-47.	Shear characteristics of typical simulated rock bolt installations under static laboratory shear loading. ....	91
Figure 3-48.	Relationship between rock bolt diameter, borehole diameter and relative system shear deformation capacity. ....	92
Figure 3-49.	Stiffness characteristic at defined loading points for mesh and lacing configurations based on diagonal lacing length. ....	94
Figure 3-50.	Stiffness characteristic of the mesh panel for typical mesh types. ....	95
Figure 3-51.	Comparison of in situ mesh and lace fabric energy absorption capacity and contained simulated rock mass energy absorption capacity. ....	96
Figure 3-52.	Influence of fabric stiffness of contained simulated rock mass energy absorption characteristic per unit volume of rock mass. ....	96
Figure 4-1.	Definition of principle methodologies of excavation stabilisation ....	98
Figure 4-2.	Generalised deformation characteristics for principal support methodologies under high loading conditions. ....	98
Figure 4-3.	A conceptual model of design methodologies and potential rock mass deformations in the presence of weak bedding planes. ....	101
Figure 4-4.	Selection of support design philosophy based on rock mass characteristics and rock bolt reinforcement length. ....	103
Figure 4-5.	Chart to determine depth of instability due to stress induced fracturing. ....	104
Figure 4-6.	Estimation of maximum dynamic induced stress. ....	105
Figure 4-7.	Estimation of sidewall dilation rates per metre per unit increase in vertical stress field. ....	106
Figure 4-8.	Estimation of relative sidewall dilation rates for vertical stress increase and decrease. ....	107
Figure 4-9.	Estimation of relative hangingwall dilation rates for vertical stress increase and decrease. ....	108
Figure 4-10.	Estimation of hangingwall dilation rates per metre per unit decrease in vertical stress field (MPa). ....	108
Figure 4-11.	Conceptual illustration of reinforcement and rock mass confinement characteristics ....	109
Figure 4-12.	Estimation of remaining deformation and energy absorption capacity of rock bolt reinforcement system due to stress change. ....	110
Figure 4-13.	Classification of rock mass based on block volume and aspect ratio relative to the rock bolt reinforcement with equivalent RQD values after Palmstrøm (1996). ....	111
Figure 4-14.	Determination of depth of instability due to unravelling of the rock mass under gravitational loading. ....	113
Figure 4-15.	Rate of increased unravelling instability due to dynamic loading. ....	114
Figure 4-16.	Reinforced rock mass structure for defined rock mass class and energy demand. ....	116
Figure 4-17.	Analysis of stability of reinforced 3:1 aspect ratio beam for defined rock mass class and energy demand. ....	117
Figure 4-18.	Estimation of energy absorption capacity of contained discontinuous rock mass based on a typical mesh and lace fabric support stiffness of 200 kN/m. ....	119
Figure 4-19.	Estimation of weld mesh and lace fabric system stiffness for rock bolt spacing of 1.5 m. (Haile, 1998) ....	120
Figure 4-20.	Example rock bolt load – deformation characteristic ....	121

Figure 4-21.	Distribution of deformation and dynamic energy demand / absorption. ....	122
Figure 4-22.	Typical lacing configuration for square rock bolt reinforcement pattern. ....	122
Figure 4-23.	Distribution of deformation and dynamic energy demand / absorption. ....	124
Figure 4-24.	Example of ground reaction curve for estimation of stiffness of fabric support to maintain the inherent rock mass strength. ....	125
Figure 5-1.	Conceptual representation of the influence of relative reinforcement and fabric support stiffness on rock mass unravelling. ....	128

## Tables

Table 2-1:	Tunnel support guidelines after US Corps of Engineers. ....	32
Table 2-2.	Empirical guidelines for design of rock bolt system in excavations <15 m span (after Farmer and Shelton 1980) ....	33
Table 2-3.	Geomechanics Classification system (Bieniawski 1973) .....	34
Table 2-4.	Adjustment to RMR for joint orientation .....	34
Table 2-5.	Adjustment to RMR for joint orientations applicable to tunnels ....	34
Table 2-6.	Rock mass class and description as determined from RMR ....	34
Table 2-7.	Rock mass parameters based on RMR .....	35
Table 2-8.	Excavation technique and support selection based on RMR. ....	35
Table 2-9.	Percentage adjustment to joint spacing rating for joint number and number of non-vertical excavation rockwalls. ....	37
Table 2-10.	Support system guide based on Modified Geomechanics Classification after Laubscher and Taylor (1976). ....	37
Table 2-11.	Tables for classification of the rock mass based on the Q system. ....	38
Table 2-12.	Recommended ESR values for excavation safety level. ....	41
Table 2-13.	Estimations of support pressure, length and spacing of primary support based on the Q system. ....	42
Table 2-14.	Support recommendation for good ground conditions (RCF < 0.7). ....	43
Table 2-15.	Support recommendation for average ground conditions (0.7 < RCF < 1.4). ....	43
Table 2-16.	Support recommendation for poor ground conditions (RCF > 1.4). ....	44

# Photographs

Photograph 3-1. Failure of mesh panel. ....	48
Photograph 3-2. Rock mass containment by mesh panel. ....	49
Photograph 3-3. Large scale sidewall deformation. ....	49
Photograph 3-4. Guillotining of hangingwall rock bolt. ....	50
Photograph 3-5. Tensile failure of hangingwall rock bolt. ....	50
Photograph 3-6. Rock mass failure and support system performance at rockburst site illustrating the interaction between the rock bolt reinforcement and the rock mass. ....	73
Photographs 3-7. Shear deformation of rebar rock bolt in highly bedded rock mass structure. ....	93
Photographs 3-8. Shear deformation of rebar rock bolt in highly bedded rock mass structure. ....	93
Photograph 4-1. Shear deformation of smooth bar rock bolt. ....	102
Photograph 4-2. Example of typical fracturing and resultant blocky nature of rock mass within a deep level gold mine in quartzite rock mass. ....	112

## List of Symbols

R	Rand
km	kilometre
m	metre
cm	centimetre
mm	millimetre
MPa	Mega Pascal
Pa	Pascal
$\sigma_c$	Uniaxial compressive strength
$\sigma_1$	Maximum principal stress
$\sigma_2$	Intermediate principle stress
$\sigma_3$	Minor principal stress
szz	vertical stress component
syy	dip parallel horizontal stress component
sxx	strike parallel horizontal stress component
$\rho$	density of rock mass
g	gravitational acceleration constant (10 m/s <sup>2</sup> )
E	energy
m	mass
v	velocity
M	magnitude of seismic event (M=)



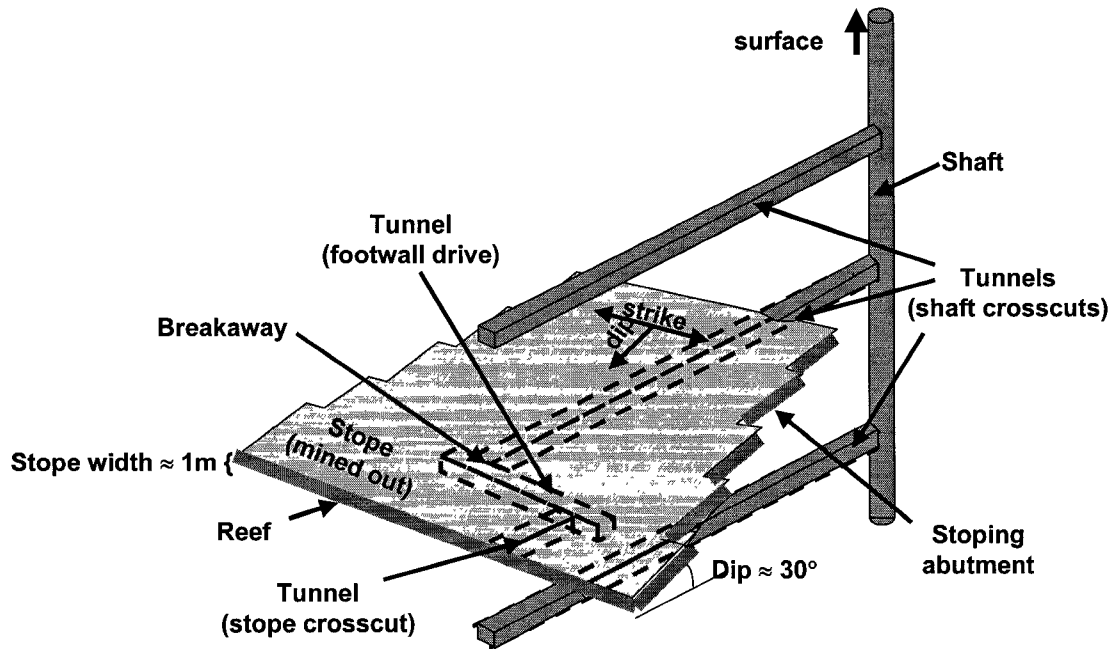
## List of Abbreviations

RCF	Rockwall Condition Factor (Anon., 1988)
UCS	Uniaxial Compressive Strength
RMR	Rock Mass Rating (Bieniawski 1973, 1979)
Q	Rock Quality - rock mass classification (Barton, 1974)
UDEC	Universal Distinct Element Code
FLAC	Fast Lagrangian Analysis of Continua

# Glossary of Terms

Because of the specialised nature of the mining industry and the local terms often associated with a certain sector of the mining industry, it is considered useful to review some of the common terminology associated with the deep level South African mines which may be encountered within this document.

Figure I illustrates a typical South African deep level mining layout.



**Figure I. Schematic of principal features of a typical deep level South African gold mine (not to scale).**

The following gives a brief description of the terminology shown in Figure I and additional terminology applied to the large scale mine layout and rock mass behaviour typical of the South African deep level mines.

**Rock mass** - This refers to the large scale rock volume that encompasses the intact rock material and the natural, or mining induced, ubiquitous discontinuities such as joints and fractures. The presence of large scale geological structures, such as faults and dykes, are generally referred to specifically but may cause a change in the local rock mass characteristics. The structure within the rock mass is orientated with regard to its dip direction (steepest inclination of the plane) and strike direction, 90° to dip, and defines a horizontal line at any point on the plane surface. The rock mass structure within the South African gold mines generally conforms to the structure of the reef horizon with the principal natural structure within the rock mass being the reef parallel bedding planes. The spacing of the bedding planes may vary from 10 cm to 1 m.

**Reef** - (orebody, reef horizon) The mineralised rock from which the mining product is derived. In the South African context this is usually a very thin structure (1 cm – 1 m) with a large lateral extent and relatively low dip, typically referred to as a tabular deposit.

**Shaft** - The excavation giving the primary means of access to the reef from surface. Typically these are vertical with diameters of the order of 10 m, in which conveyances run to transport the men, material and the extracted reef to and from surface. These excavations may also be inclined (>30°). For deep level mining a shaft system will consist of a series of shafts to access depths between 2000 m and 4000 m below surface.

**Tunnel** - (haulage, crosscut, footwall drive) A horizontal (or very low inclination) excavation, typically 3.5 m x 3.5 m, which provides the access infrastructure for men and material from the main shaft access to the working places on the reef horizon. The tunnels usually contain pipe work for the supply of water and compressed air to the workings and rail tracks for locomotives and rail cars for the transport of men, material and broken rock. The local terms of crosscut and footwall drive refer to the orientation of the tunnel relative to the reef horizon and local rock mass structure. Thus a crosscut will cut across the general rock mass structure and is perpendicular to the strike of the reef horizon, whereas a footwall drive is generally orientated parallel to the strike of the reef and rock mass structure.

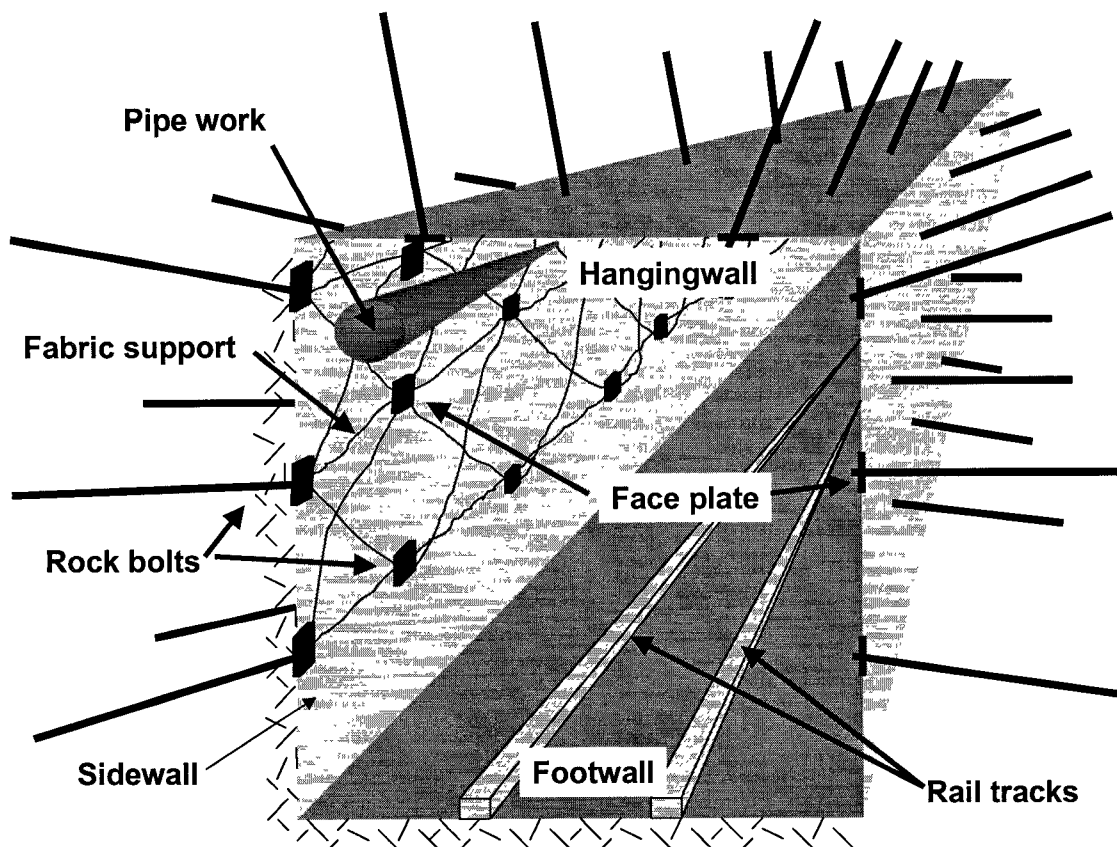
**Breakaway** - The junction between two tunnels is often referred to as the breakaway position. This area of the excavation results in an increased excavation dimension and thus reduced stability.

**Stope** - The stope excavation is the mined out area of the orebody (reef) with dimensions defined by the strike and dip spans and the **stopping width** (typically slightly greater than the reef width). The large lateral extent of this excavation results in large stress concentrations in the rock mass at the boundary of the excavation (stope face). These large stress concentrations are often referred to as **stopping abutments**.

**Seismic event** - (seismicity) The large stress changes and concentrations in the rock mass due to extensive mining operations (stope excavations) result in large strain energy within the rock mass, the violent release of which is termed a seismic event (singular) or seismicity (plural). The mechanism of energy release is due to either the creation of a fracture within previously intact rock or slip on an existing discontinuity within the rock mass.

**Rockburst** - A rockburst is the manifestation of damage within an excavation due to the radiation of energy from a seismic event. The radiation of energy from a seismic event is in the form of transient stress waves in the rock mass which may result in violent failure of the rock mass around an excavation and/or expulsion of a discontinuous rock mass in the boundary of an excavation. The intensity of damage is a function of the proximity of the excavation to the source of the seismic event, the magnitude (M) of the event and the attenuation of the energy within the rock mass.

Terminology specific to tunnel excavations is indicated below in Figure II.



**Figure II. Illustration of typical components of a tunnel and its reinforcement / support system.**

The following gives a brief description of the terminology shown in Figure II and terminology applied to tunnel excavations and local rock mass behaviour.

**Hangingwall, Footwall and Sidewall** - (roof, floor and side respectively) These terms describe the relative position of the exposed rockwall on the boundary of the typically square tunnel profile. These terms are often used to also reference sections of the tunnel support system. Within the tunnel excavation, service infrastructure such as **pipe work** and **rail tracks** are often installed.

**Rock bolts** - (rock anchors, tendons, cables, reinforcement unit) These are the primary rock mass reinforcement units used in the stabilisation of the rock mass around tunnel excavations. A rock bolt is installed in a hole drilled into the rock mass generally perpendicular to the excavation boundary. The rock bolt is anchored within the rock mass either by means of a mechanical end anchorage and / or bonding over the length of the bolt by cementitious or resin based grouts. The use of end anchorage may also allow pre-tensioning of the rock bolt between the end anchorage and a **face plate** on the boundary of the tunnel. The characteristic of the rock bolt unit, surface profile or shape, may also be used to describe this component of the support system, such as smooth bar, rebar, or shepherd crook, Split Set etc. The arrangement of a series of rock bolts within a unit length of the excavation is referred to as the rock bolt pattern. This pattern is then repeated on a regular basis along the length of the excavation. The term cables refers to reinforcement elements constructed of wire rope.

**Fabric support** - The fabric component of the support system spans between the rock bolt reinforcement on the surface of the excavation. It is generally a high areal coverage support system but may vary from single lacing (cable rope) strands between rock bolts, through the incorporation of wire mesh, to 100 % coverage membrane or shotcrete (gunite) support. The role of the fabric is to provide support, or even reinforcement, to the potentially unstable rock mass between the rock bolt units.

**Sets** - Sets are the construction of an internal framework type support within the tunnel excavation. It will generally consist of either simple vertical concrete filled pipes, embedding in the footwall with rail beams across the hangingwall, or more sophisticated use of specialised 'I' beam units to construct the framework. The framework is not in direct contact with the rock mass surrounding the tunnel and thus use is made of 'cribbing' to fill the space between the set framework and the rock mass surface. This cribbing can again vary from simple timber packing to pumped foam grout systems.

# 1 Introduction

## 1.1 Nature of problem and previous work

Tunnels at great depths, which are typical of the South African gold mines, are subjected to stresses (loading) which are sufficient to cause fracturing of the rock around the tunnel. The rock breaks up into discontinuous blocks and the rock mass dilates causing deformation of the rock wall into the tunnel. A seismic event may occur on a fault due to changes in the stress environment in close proximity to a stope. Tunnels that are accessing this section of the mine are subjected to violent shaking, which may be sufficient to result in the rapid deformation and potential expulsion of this fractured rock into the tunnel. If this movement cannot be controlled by the tunnel support system, then total failure of the tunnel may result. This can have serious implications with regard to safety of personnel and the operational status of the tunnel. This damage is known as rockburst damage. It is in this environment that the design of tunnel support systems is evaluated in this investigation.

Approximately 10000 km of tunnels are in use in South African gold and platinum mines and constitute the essential link between shafts and production stopes for men, materials, services and ventilation. The environments in which these tunnels are located vary from depths close to surface to approximately 3500 m below surface. They may be sited in rock types varying from sedimentary rock masses to igneous. This results in a significant variation in the rock mass condition of tunnels. In a low stress environment, control of potentially unstable key blocks defined by the geotechnical structure of the rock mass is of primary concern. In a high stress environment, stress fracturing of the rock mass for distances of one to three times the tunnel height into the sidewall may occur. As mining progresses to depths of 4 to 5 km, the ability to ensure the stability of tunnels becomes an even more critical issue. At current mining depths the support systems in use, and the way in which they are applied, makes it impossible to ensure the integrity of tunnels under adverse rock mass or loading conditions. This is demonstrated by the statistic that greater than 85 km of tunnel have to be rehabilitated to varying degrees annually (Jager and Wojno 1991). In addition, some tunnels have to be abandoned due to severe rockburst damage or because impossible conditions are encountered. The estimated cost of these losses, rehabilitation and associated disruption to mining operations is estimated at R 400 million per annum (Jager and Wojno, 1991).

Analysis of accident data shows that 12 % of rock related fatalities within the South African mining industry occur in tunnels and other service excavations. The majority of these fatalities occur within the first 10 m back from the development face where, usually, only primary support is installed. However, almost 30 % occur outside of this region where excavation stability should have been secured by the more robust secondary support. In the face area, 77 % of the fatalities are caused by rockfalls and 23 % by rockbursts. Significantly, this proportion of rockfall to rockburst accidents is reversed away from the tunnel face where 64 % are the result of rockbursts. This distribution of the cause of accidents suggests that the primary support systems and procedures are deficient in their ability to prevent simple rockfalls, while secondary support is successful in preventing rockfall accidents but is likely to fail under the severe loading conditions associated with rockbursts.

There is thus a strong incentive from both the safety and economic points of view to improve the design and support of tunnels subjected to these severe conditions.

Current support design philosophy as applied within the South African mining industry is generally empirically or semi-empirically based. Within the shallow mining environments, reference may be made to the rock mass classification systems of Bieniawski (1979) or Barton, Lien and Lunde (1974), based on the geotechnical structure of the rock mass. Within the deeper, high stress, rock mass environments, reference is generally made to the empirical tables derived from the Chamber of Mines Research Organization (Anon 1988) and applied via the Rockwall Condition Factor (RCF) criterion. Alternatively, the design may be based on

consideration of energy absorption criteria and support resistance determined from simple mechanistic analysis. The application of numerical models to excavation design has also increased in popularity. However, the complexity of establishing the models means that, as a daily design tool, these are not used on a routine basis within the mining industry.

These design methods, especially the ones based on in situ experience, have been developed over many years and have proved more than adequate within their general areas of application. The basis of their design methodology is either to create an integrated reinforced shell of rock around the excavation by interaction of the support units or the retainment / containment of the unstable rock mass by suitable anchorage. However, where rock mass conditions are abnormal to those under which the support system was generally developed, or there is a significant change in the loading conditions due to stress-induced failure or dynamic loading, then failure of the support system may occur. This is generally observed as large deformations and unravelling of the rock mass between the rock bolts, or as a total failure of the support system and subsequent collapse of the excavation. Under these conditions, it is presumed that the support interaction and load distribution were insufficient to control the associated dilation of the rock mass due to the imposed loading condition.

The mines generally have a limited variety of support units at their disposal, and, thus, "standard" support systems are designed to cater for the majority of rock mass conditions. These may well have to cater for the support of approximately 1500 - 2000 m of new development per month on a large gold mine. Unless the excavation is of a special nature or problematic ground conditions are foreseen well ahead, then the operator only has access to these standard support types to ensure excavation stability. The semi empirical basis of the current design methodologies makes it difficult for the support design engineer to utilise a limited number of support unit types to cater for variations in the rock mass environment.

A survey of tunnel conditions and support practices (Haile, Jager and Wojno, 1995) for the South African mining industry indicated that a significant cause of problems, particularly in abnormal ground conditions, was the lack of a rational design methodology. This was found to be particularly true in the high stress, highly fractured environments of the gold mining industry. In addition, significant excavation damage was associated with dynamic loading due to seismicity.

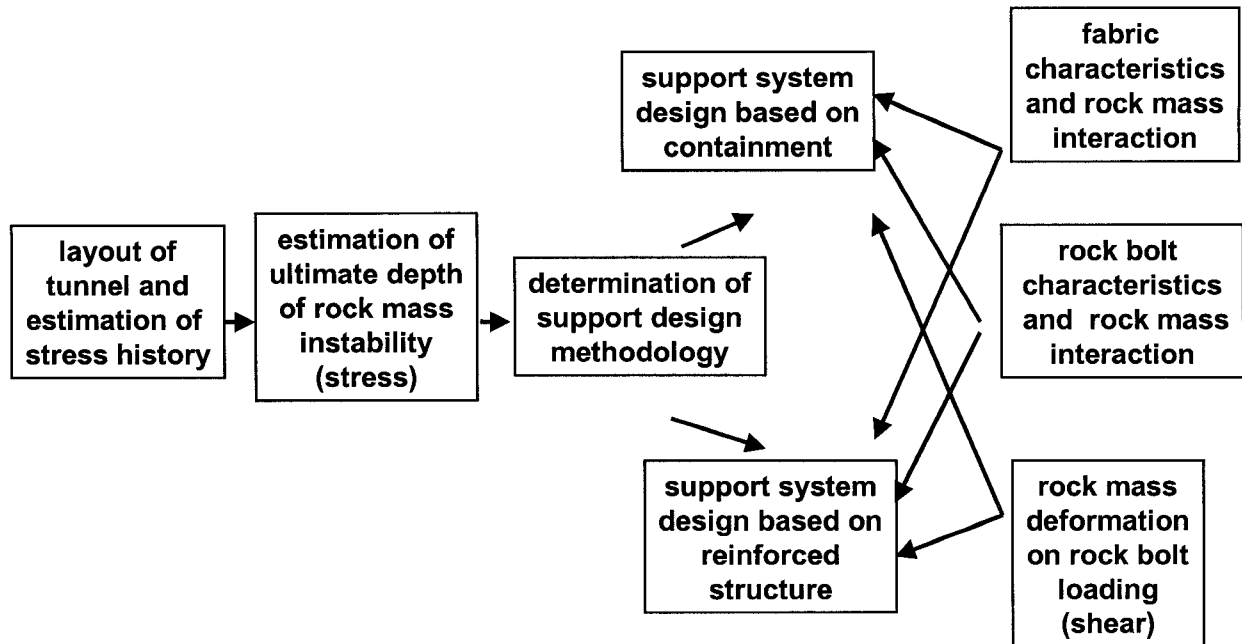
## **1.2 Scope of research**

The scope of this research is to improve the design considerations for tunnel support systems for excavations sited in a high stress environment and under the influence of high quasi static and dynamic deformations, based on a mechanistic analysis. Of fundamental importance to the investigation is an understanding of the interaction between a tunnel support system and the highly discontinuous (fractured) rock mass structure. The in situ interaction of the components of the support systems with the rock mass is extremely complex, as are the loading conditions, particularly those associated with seismic events.

Components of the support systems, such as the rock bolts and fabric, may be tested in the laboratory under conditions considered representative of the in situ environment. The interaction of the support system with the rock mass, and thus an understanding of the applicable loading conditions, is however a far more complex problem. Numerical models may provide a suitable means of evaluating the performance of the support system. These may give insight into the mechanistic interaction between the support components and the rock mass. It is however important to calibrate these models with in situ evaluation. Due to the highly variable nature of seismicity and associated rockburst damage, it is extremely difficult to implement detailed instrumentation at an underground site and to be guaranteed of capturing this behaviour. Detailed observations of numerous rockburst incidents could form a database of the in situ performance of different support systems within different rock mass environments, and, thus, the evaluation of the interaction between the support system and the rock mass.

Due to the inherent complexities of design within a highly variable medium such as the rock mass, this observational approach has formed the basis of the majority of practical support design guidelines within the civil tunnelling and mining industries.

Due to the breadth of the investigation, the interconnectivity between the different aspects of the proposed design methodology, and the areas of research and respective chapters within this document, are shown in Figure 1-1.



**Figure 1-1. Aspects of investigation for the proposed tunnel support design methodology.**

### 1.3 Objectives

The objective of this research is to provide a better mechanistic understanding of the interaction between the components of a support system and the rock mass surrounding an excavation in a deep level mining environment. It is envisaged that this will allow:

- Site specific understanding of the influence of the rock mass structure on support system performance.
- An understanding of the relationship between the excavation geometry, rock mass environment and the mechanism of support system interaction
- Estimation of the mechanism of loading and demand on the individual components of the support system
- An ability to design the capacity of the components of the support system based on mechanistic analysis
- Flexibility of design, with a limited availability of support component types, with changing rock mass environment
- Practical guidelines for the design of support systems in a deep level rock mass environment



It is envisaged that this understanding will allow improved design of support systems, for tunnel type excavations, within this adverse underground environment, with consequent improvement in the safety of personnel as well as the operational viability of these excavations.

## **2. Literature survey**

### **2.1 Introduction**

This section reviews the rock mass characteristics which may be anticipated around tunnel excavations sited in high stress environments and the design of support systems under these conditions. The first aspect deals with the former area and thus sets the scene with regard to the rock mass environment in which the support design engineer must conduct the analysis. The latter aspect of the review examines support design concepts and methodologies that are currently used in mining. The inclusion of the detailed literature review in this document is considered to be of value for the general design of tunnel support systems in different rock mass environments.

### **2.2 Rock mass behaviour around highly stressed tunnel excavations**

Due to the abnormally deep nature of gold mining in South Africa, the characteristic of the rock mass behaviour around tunnel excavations is fairly unique to this environment. It is in this environment that the majority of the fundamental work on in situ fracturing around tunnel excavations has been conducted.

The early work conducted into the analysis of fracturing around a tunnel examined the phenomenon of vertical slabbing of the sidewall of the excavation in the direction parallel to the maximum principle stress (Fairhurst and Cook, 1965). This evaluation indicated that the intensity of fracturing was a function of a constant slenderness ratio, which led to the formation of more finely fractured rock at depth. This contrasted with subsequent observations by Ortlepp (1984) that indicated that fracturing was closely spaced at the surface of the excavation and that the spacing increased with depth into the rock wall.

Barrow (1966) investigated the development of failure around an unsupported square tunnel (3 m x 3 m), which is typical of the current excavation dimensions, that was sited in quartzite at a depth of approximately 1600 m below surface. The tunnel was originally in near virgin stress conditions and was subsequently destressed by the adjacent stoping operations (overstopping). The displacement history of the sidewall of the excavation indicated an initial linear increase, with respect to vertical stress increase, up to a point where rapid displacement occurred prior to a subsequent lower rate of deformation during significant overtopping. Almost all the sidewall movement was associated with the first 0.75 m during the period of stress increase, and deeper movement was only noted during overtopping. This displacement was directly associated with the development of the fracture zone. However, within the upper corners of the tunnel, although intense fracturing was observed, displacement was only associated with overtopping. No movement was detected in the hangingwall or abutments of the tunnel until overtopping was in progress. Hangingwall displacements were measured to depths in excess of 7 m, and again these were associated primarily with overtopping. Barrow noted that the application of support is not practical to prevent failure, but must be designed to control the failed rock mass by the use of a yielding bolt.

Observations of the development of the fracture zone around a 5 m x 5.5 m excavation sited in quartzite indicated the presence of "bow wave" fractures up to 2 m in advance of the face in a stress regime of approximately 100 MPa (More O'Ferrall and Brinch, 1983). During this trial it was observed that fracturing of the sidewall had occurred to a depth of 3 m at a point 2 m back from the face, and this increased to 4.3 m at a distance of 20 m. This compared favourably

with the work of Hepworth (1985) who noted that up to 60% of sidewall convergence occurred in the first 3 m to 4.5 m of the sidewall. Jager, Wojno and Henderson (1990) estimated that deformation of the sidewall of a tunnel may occur up to three times the diameter behind the development face. It was noted that, in the hangingwall, fracturing was to a depth of approximately half of that in the sidewall. It was observed that bedding planes had a limited influence on the development of the fracture zone. This was attributed to the initial development of fractures in the confined environment in advance of the face. It was further noted that, where irregularities in the profile of the excavation occurred, this was associated with excessive scaling. With regard to the support system (9 mm x 2.4 m rock studs), it was noted that there was little failure of the support elements in the sidewall of the excavation but noticeable failure of the support elements in the hangingwall. This was related to the rapid development of load within the hangingwall support units subsequent to installation, due to the development of the hangingwall fracture zone, whereas the sidewall support units were placed in pre-fractured ground.

Analysis of the development of fracturing around a large excavation was conducted by Kersten, Piper and Greef (1983). The excavation had a cross-sectional dimension of 10 m x 10 m and was sited in argillaceous quartzite. The approximate depth of fracturing was indicated to be in the region of 7 m, with a fracture intensity of approximately 3 fractures per metre. This resulted in an approximate sidewall deformation of 90 mm. This deformation led to the failure of the initial support system of 3 m rock studs integrated with 5 m cable anchors that were designed on the basis of standard support practices. The installation of the secondary, or remedial, support system resulted in the stabilisation of the excavation. It was proposed that the presence of bedding had allowed for increased movement of the rock mass, in excess of that anticipated, resulting in failure of the initial support system. The increase in the support resistance of the remedial support was considered to have stabilised this deformation.

A study was conducted on the performance of an experimental tunnel subjected to stresses ranging from 50 MPa to 230 MPa (Ortlepp and Gay, 1984). The tunnel was situated in massive siliceous quartzite of an approximate uniaxial compressive strength (UCS) of 350 MPa, at a depth of 3075 m below surface. It was noted that very limited fracturing occurred in this competent ground in the absence of significant geological features. In subsequently developed cobbles, the stress fracturing around the excavation was seen to be confined to approximately 200 mm from the side of the tunnel. This observation contrasts markedly with those described above (Barrow, 1966, More O'Ferrall and Brinch 1983, Hepworth 1985). Where a geologically weak zone was present in the sidewall of the tunnel, this area created a stress raising notch which initiated fracturing. The fracturing rapidly stabilised with the formation of an elliptical profile. This was considered to be due to the indicated increase in stress confinement deeper into the rock mass. In general it was noted that abrupt changes in fracturing occurred and this was caused by geological inhomogeneities. In such an environment there was limited influence on the stability of the tunnel by the support installation, and thus it was proposed that even the weakest support system was adequate. However, the application of a yielding type support system was proposed as essential to ensure long term stability.

Wagner (1983) indicated that the intensity of fracturing was a function of the brittleness of the rock type and the magnitude of the local stress regime. This was illustrated with reference to glassy quartzite having a fracture separation of approximately 10 mm, and argillaceous quartzite with a fracture separation of 50 mm; this was also noted by Wojno, Jager and Roberts (1987) and Jager, Wojno and Henderson (1990). Fractures were initiated at a compressive field stress level of approximately 40 % of the laboratory uniaxial compressive strength of the rock. It was also noted that the presence of bedding planes, although not considered to influence the development of the fracture zone, assisted in the slabbing of the sidewall of the excavation. However, Wagner (1979) concluded that there is still very little understanding of the mechanism of rock fracturing in the design of excavations.

Empirically determined levels of excavation deterioration, due to the maximum principal stress, are indicated to be as follows (Wagner 1979):  $> 0.2 \sigma_c$  start of scaling,  $> 0.3 \sigma_c$  severe scaling,

$> 0.4 \sigma_c$  heavy scaling. Thus it was advised that, where the boundary stress exceeds the rock mass strength, excavation design should consider minimisation of the fracture zone. This will be a function of the distribution of the maximum principal stress ( $\sigma_1$ ) and the minimum principal stress ( $\sigma_3$ ), the stress gradient at the periphery of the excavation, and the radii of curvature of the sidewall (in a vertical stress regime). Support recommendations made by Wagner concern the control of deformation to improve the self supporting role of the rock mass, and, within a fracture zone, to provide confinement to further deformation. Support characteristics should include active loading and yield potential.

The influence of seismicity on the stability of the excavation must also be considered. The incidence of seismicity will result in a dynamic stress increase on the excavation periphery, which would be a function of the size and proximity of the seismic event to the excavation (Kaiser, McCreath and Tannant, 1996).

Wiseman (1979) examined the significance of rock fracturing in the design and support of mine tunnels. He estimated that a tunnel will be influenced by an induced stress at a level  $> 0.5 \sigma_c$ . He produced a relationship between the stress concentration on the sidewall of an excavation and tunnel deterioration, known as the Stress Concentration Factor:

$$SCF = (3\sigma_1 - \sigma_3) / \sigma_c .$$

Where SCF = Stress concentration factor

$\sigma_1$  = Major stress component

$\sigma_3$  = Minor stress component

$\sigma_c$  = Uniaxial compressive strength

This was related to support recommendations based on the value of the stress concentration factor. It was estimated that if this were greater than 0.8, then deterioration of the excavation could be expected. It was indicated that the function of the support was to stabilise the fracture zone and provide confinement, rather than the prevention of failure.

Hoek and Brown (1982) developed the Rock Support Interaction Analysis to investigate the support pressure in relation to a ground reaction curve. However, this is based on circular tunnels in a hydrostatic stress field in a homogeneous rock mass with uniform support pressure and thus is generally more applicable to shallow, civil engineering type excavations.

The applicability of rock mass classification methods for the design of underground excavations at depth is generally limited as the majority of systems are based on structurally controlled environments, and thus are considered not applicable. Piper (1984) conducted an investigation into the effect of rock mass characteristics on fracturing around a large excavation at depth. The analysis was based on a comparison of the elastic stress field around an excavation based on the Mohr Coulomb strength criteria. The rock mass strength was derived by application of the RMR (Bieniawski 1973) and Q (Barton *et al*, 1974) rock mass classification systems. This still resulted in an under estimation of the extent of fracturing by approximately 50 %.

## 2.3 Design of tunnel support systems

The reinforcement of the rock mass in the immediate periphery of an excavation is in general still incompletely understood. This may be reflected by the lack of rational design methodologies for rock mass reinforcement, and the widespread use of empirical design methods and the experience of the design engineer (Stillborg 1986). The mechanical behaviour and properties of the support units may be analysed in detail from a mechanical engineering perspective, but the rock mass - rock bolt interaction is very poorly understood. The interaction of the rock bolt system with the rock mass is a function of the qualitative differences in the immediate interaction of the rock bolt unit with the rock mass. Fully grouted or frictional support units interact with the rock mass along the full barrel of the borehole in

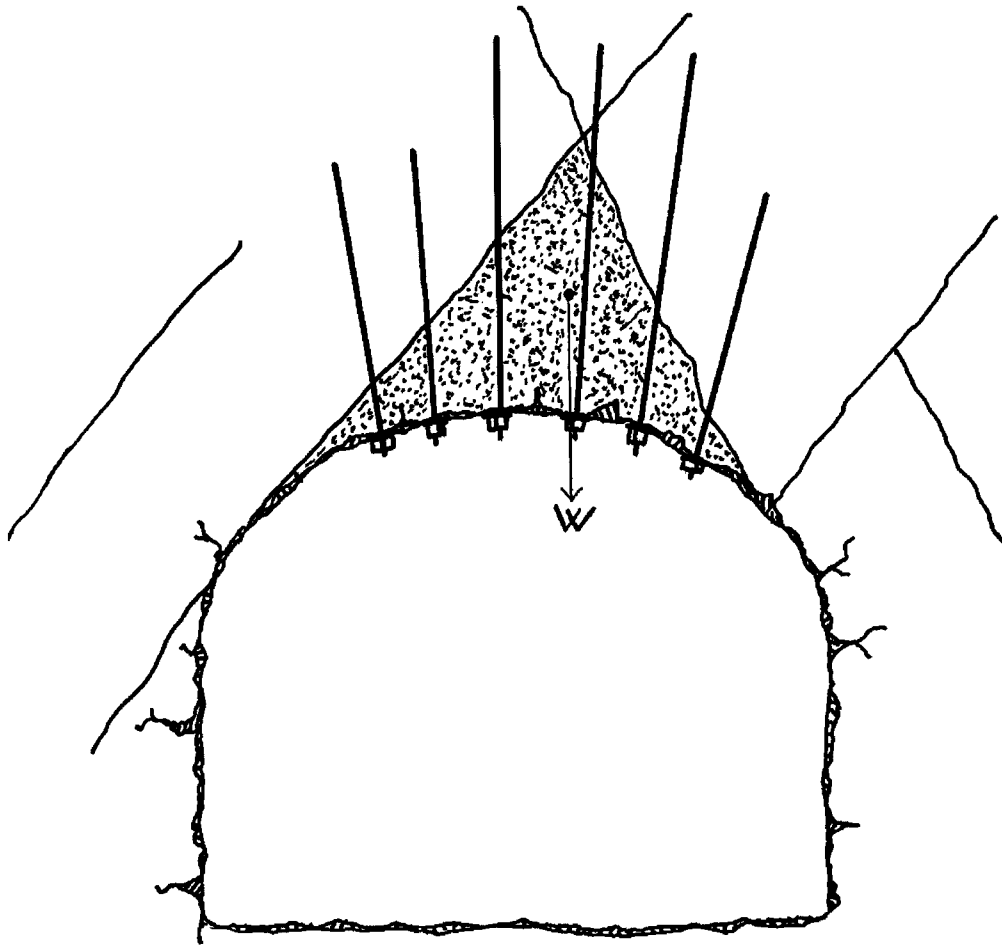
which they are installed. These units primarily influence the blocks with which they make contact, and the stiff nature of the support unit interaction makes it generally applicable to the control of discrete structural blocks which require retainment to maintain excavation stability. The capacity requirement of the support unit is thus directly a function of the unstable volume to be supported. This forms the basis of the mechanistic design of support systems within the South African mining environment (Wojno and Jager 1987). The use of point anchored support systems, with subsequent tensioning, results in the development of confinement within the rock mass. This may result in the interaction of individual rock blocks to create a reinforced rock arch structure. The capacity and loading of the support units in this structure is small compared to the loads acting within the rock mass. The basic design principle is to create the most effective structure to allow the rock mass to be self-supporting. A review of methodologies for the design of tunnel support systems (Choquet and Hadjigeorgiou, 1993, Stillborg, 1986) characterises excavation design principles into three systems: analysis of structural stability, empirical design and numerical modelling.

Within a low stress environment the principle design methodologies are based on either the analysis of the structural stability of discrete blocks or the application of empirical design criteria to more structurally complex rock masses. It is within this environment that most support design methodologies have been developed and refined. As the depth at which excavation takes place increases, so the understanding of the rock bolt - rock mass interaction and applicable design methodologies decreases. It is within the highly discontinuous and fractured rock mass structures of the South African gold and platinum mines that the lack of rational design criteria results in potentially unstable ground conditions (Haile *et al.*, 1995).

The design of tunnel support systems will be dependent primarily on the rock mass environment in which these excavations are sited. The design engineer must thus presently use either experience or a rock mass classification system in order to assess the potential excavation stability and determine the support system requirements. The rock bolt support system should be designed to minimise the instability of the excavation peripheral rock mass, and thus excessive deformations of blocks within the rock mass. The philosophy of the design process is currently based on structural analysis, empirical design guidelines or support rules. It is useful to review all the current support design methodologies applicable to discontinuous rock mass structures. This will allow an evaluation of their applicability to the rock mass environments typical of the South African deep level gold mines.

### **2.3.1 Structural Analysis**

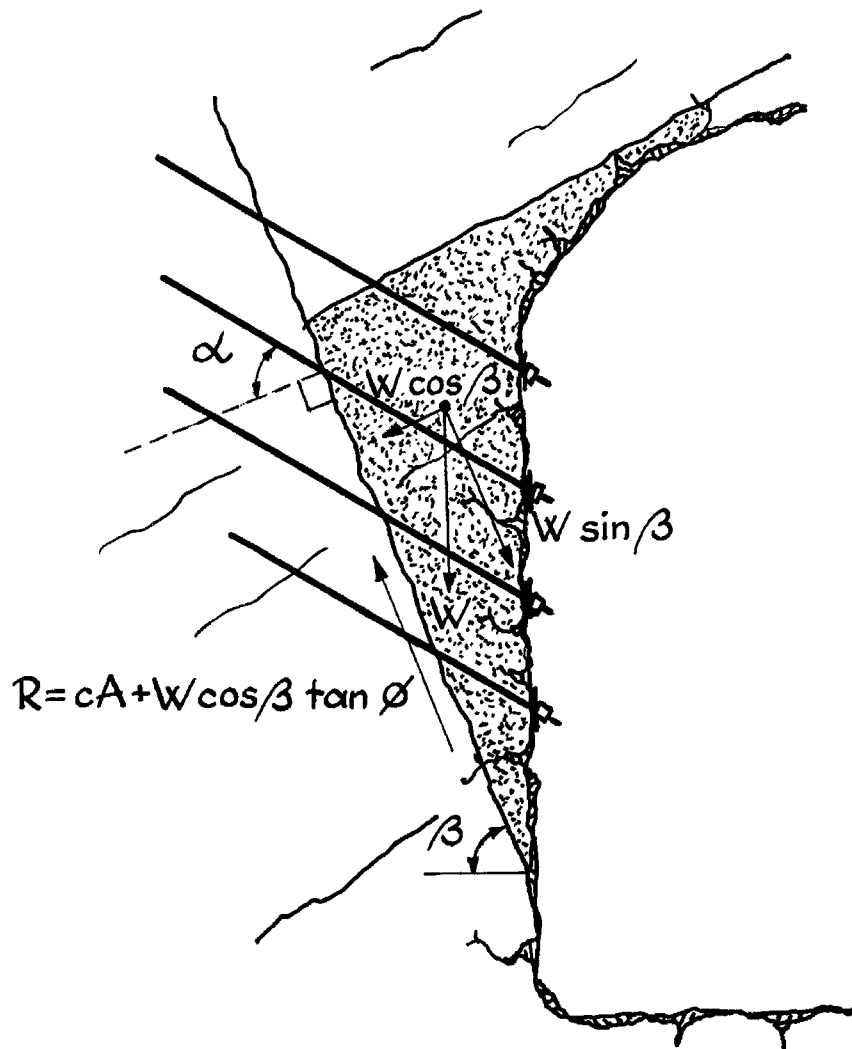
Within low stress environments, and fairly competent rock mass structures, the principal excavation instability is due to the formation of blocks that are free to move, either by falling or sliding, into the excavation. These blocks may be defined by direct discontinuity mapping or probabilistic analysis of the potential for block formation, based on known discontinuity sets and excavation orientation. The support requirement is usually specific to individual blocks and the optimum support system is based on spot bolting as opposed to systematic bolting patterns. The simplest analysis considers the ability of a block to fall from the rock mass (Figure 2-1). The characteristics of the discontinuities, which define the block, are generally neglected in this analysis.



**Figure 2-1: Support of key block structure (after Stillborg 1986)**

Under these conditions, static design considerations are based on the load capacity and length of the rock bolt system in relation to the size and mass of the defined unstable block. This will allow the number of rock bolt units that will provide a suitable factor of safety to be determined. However, complex failure modes may still result in sequential loading of the rock bolt units, and failure of the support system in excess of its static capacity. Thus, even with this simplest of support design analyses, a suitable factor of safety is necessary.

Where sliding of the block may occur, then the influence of the discontinuity characteristics must be considered in addition to the factors discussed above (Figure 2-2). The development of tension within the rock bolt units will increase the normal stress on the sliding plane and thus increase its frictional resistance. Resolving of the forces within this system allows the determination of the minimum rock bolt capacity to stabilise the block of weight  $W$  over area  $A$ . If the rock bolt reinforcement is of sufficient stiffness to limit any shear deformation on the discontinuity plane, then the cohesive strength ( $c$ ) of the discontinuity may be considered in the analysis of stability.

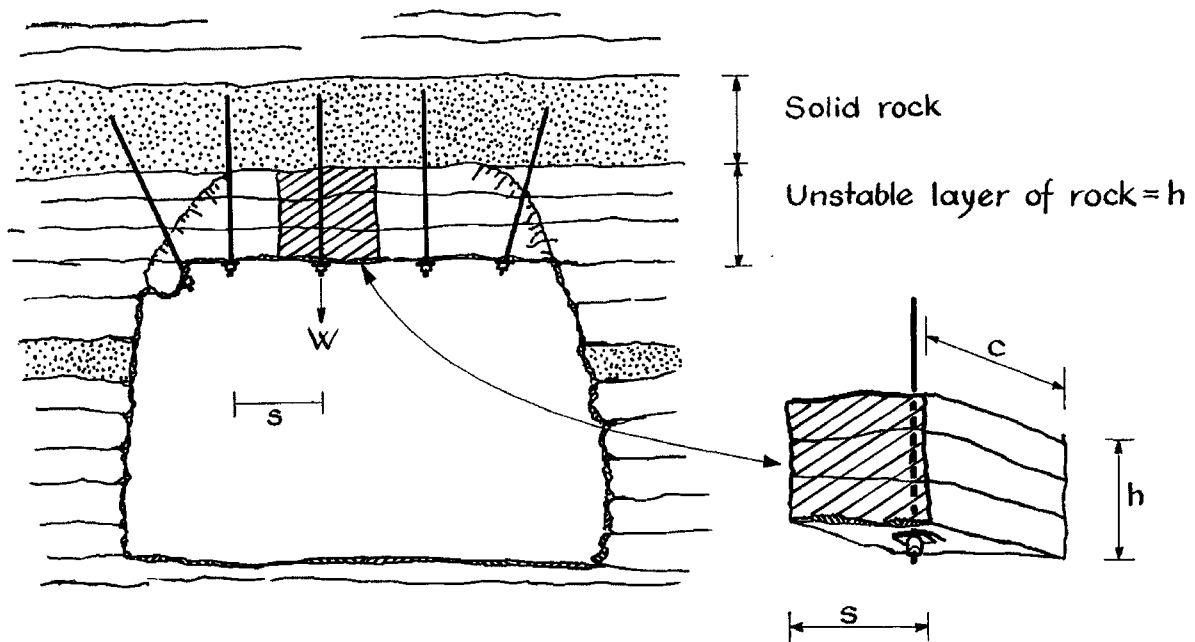


**Figure 2-2: Support of sliding block (after Stillborg 1986).**

However, generally the cohesive strength is ignored from the analysis as an additional factor of safety, but also due to the lack of confidence with regard to its in situ value. If significant deformation on the discontinuity plane is allowed to occur prior to the development of the support forces, then the influence of the cohesion will be lost due to a breakdown of the discontinuity infilling.

As the frequency of discontinuities within the rock mass increases, so the rock bolting requirement changes from the stabilisation of individual, well defined blocks to more systematic rock bolt patterns in order to reinforce and/or support the potentially unstable rock mass volume.

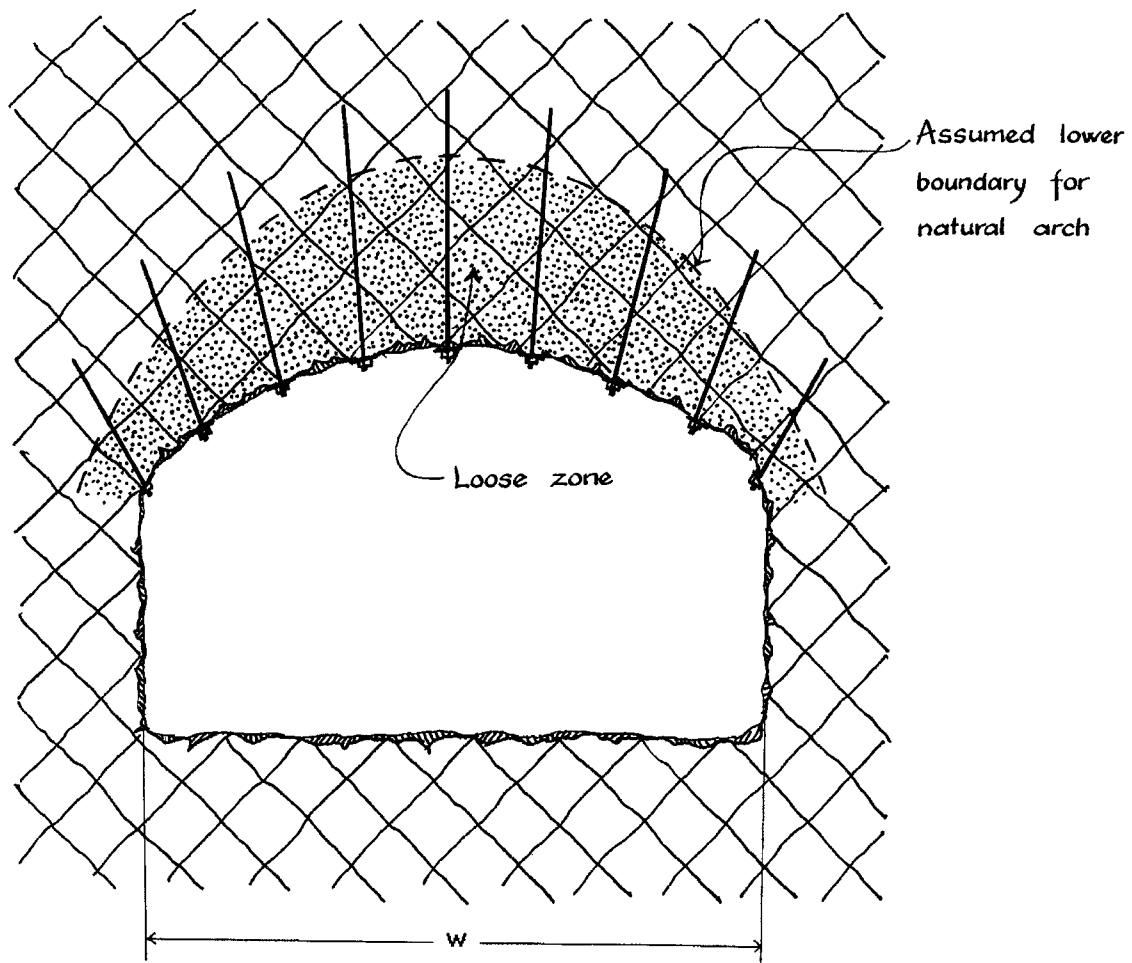
Where the limits of rock mass instability can be defined, and are within a practical depth from the excavation boundary, then the rock bolt system is designed on the basis of suspension or containment. This will be possible where a more competent horizon occurs immediately above the unstable strata (Figure 2-3).



**Figure 2-3: Suspension of laminated rock mass based on tributary area loading (after Stillborg 1986)**

Under these conditions the length of the rock bolt unit must be sufficient to anchor beyond the limit of rock mass instability. The spacing of the rock bolts is based on the load capacity of the rock bolt unit in relation to the tributary area loading of the rock mass. In Figure 2-3 the tributary area loading is given by the density of the rock mass and the tributary area unstable volume ( $s \times c \times h$ ).

Within a moderately jointed, discontinuous rock mass structure, the stabilisation of the excavation may be based on a similar analysis. The creation of an excavation results in the formation of an unstable tensile zone in the immediate roof, above which a natural arch is formed where the rock mass loading condition is primarily compressive (Figure 2-4). The extent of the unstable zone will be a function of the rock mass structure, loading environment, span of the excavation and stability of the sidewalls. The application of untensioned, grouted support units within this environment is to pin the unstable rock mass to the competent natural arch.



**Figure 2-4: Formation of unstable zone and natural stable arch within moderately jointed rock mass structures (after Stillborg 1986).**

The length of the rock bolts must be sufficient to anchor into this competent rock mass. An estimation of the required rock bolt length is given by:

$$L = 1.40 + 0.184w \quad (2-1)$$

where  $w$  is the span of the excavation (metres) and  $L$  the required rock bolt length (Norwegian Institute for Rock Blasting Technique, after Stillborg, 1986). This technique, and the use of untensioned grouted rock bolts, is generally more applicable to competent, moderately jointed rock mass structures, which result in the lower boundary of the natural arch being closer to the boundary of the excavation.

This design philosophy currently forms the basis of the mechanistic methodology for the design of rock bolt support systems within the South African mining industry (Anon, 1996). It is recommended that the determination of the depth of instability is based on analysis of accident data within an applicable geotechnical environment, and may incorporate both gravity induced falls of ground and seismically induced rockbursts. It must be recognised that the use of such a database implicitly incorporates all factors that may influence the stability of the rock mass. This would include the influence of the current support systems and excavation dimensions, in addition to the critical rock mass characteristics. As such this may not necessarily define the natural depth of instability of the rock mass. The spacing of the rock bolts is principally based on tributary area loading in relation to the load capacity of the rock bolt under the defined loading conditions. Thus, under gravitational loading, the minimum required capacity of the rock bolts, or maximum spacing of defined support units, will be given by:



$$W = \rho \times g \times (s \times c \times h) \quad (2-2)$$

where W is the load on the bolt and thus the minimum capacity,  $\rho$  is the rock mass density, g is gravitational acceleration, and s, c and h (see Figure 2-3) are the tributary area dimensions of the rock mass volume to be supported by a rock bolt unit. The demand on the support system is often expressed per unit area of excavation rockwall and is termed the required minimum support resistance of the rock bolt system, expressed as kN/m<sup>2</sup>. If highly friable ground conditions are anticipated, then additional use of areal coverage support systems is made. These are utilised to try to maintain the rock mass integrity between the rock bolt units. The demand on the fabric support system is currently not considered in the support system design analysis.

Under dynamic loading conditions the same design philosophy is adopted. However, consideration is now given to the energy demand on, and absorption capability of, the rock bolt unit. Loading is again assumed to be under tributary area conditions, even within highly discontinuous rock mass environments. The depth of instability is again based on historical accident data for rockburst conditions within the typical geotechnical environment. The minimum energy absorption requirement, per unit area of rockwall, will be a function of the kinetic energy of the unstable rock mass volume and, in the hangingwall, the constant gravitational energy of the rock mass. This is defined for the hangingwall and sidewall of the excavation by:

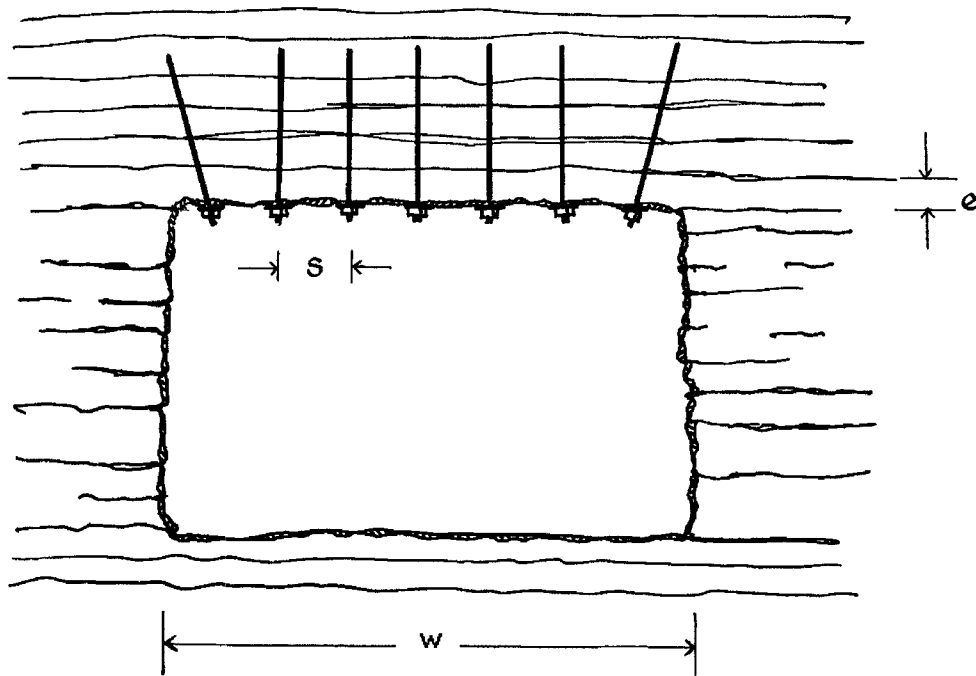
$$\text{Hangingwall} \quad E = \frac{1}{2} m v^2 + m g h \quad (2-3)$$

$$\text{Sidewall} \quad E = \frac{1}{2} m v^2 \quad (2-4)$$

where: E = energy absorption requirement of the rock bolt system per unit area,  
m = mass of ejected rock based on the defined unstable rock mass depth and density,  
v = anticipated peak ground velocity,  
h = yield capacity of the support system at a given point in time,  
g = gravitational acceleration.

Within high stress environments, and particularly mining environments, where the stress state may change over the life of the excavation, consideration must also be given to the progressive deformation of the rock mass. This will determine the remaining yield capacity of the rock bolt units at a given time in the life of the excavation, and thus the support system energy absorption capacity.

In high stress environments, where significant fracturing of the rock mass may occur, or under low stress conditions but within more highly discontinuous rock mass structures, the depth of instability may exceed the practical depth of rock bolt anchorage. Under these conditions excavation stability may be achieved by the creation of reinforced beam (Figure 2-5) or arch (Figure 2-6) structures within the discontinuous rock mass.



**Figure 2-5: Creation of reinforced beam within stratified rock mass (after Stillborg 1986).**

The basis of the design of the rock bolt system is the use of tensioned rock bolts to increase the friction between slabs or blocks within the defined arch. This will result in an enhancement of the resistance of the rock mass to shear and deformation. The design of a rock bolt system under these conditions is principally based on empirical rules and guidelines, which may be adapted into design charts or nomograms for specific geotechnical conditions.

Guidelines for the design of a rock bolt system to create a reinforced structure within a stratified rock mass (Figure 2-5) (Lang and Bischoff, 1982), based on the increase in shear resistance between layers within the rock mass, are:

$$\text{Bolt length} = w^{2/3} \quad (2-5)$$

$$\text{Bolt tension} = \left( \frac{a \cdot \gamma \cdot A \cdot R}{(\tan \phi) k} \right) \cdot \left( 1 - \frac{c}{\gamma \cdot R} \right) \cdot \left( \frac{1 - \exp^{-(\tan \phi) \cdot kD/r}}{1 - \exp^{-(\tan \phi) \cdot kL/R}} \right) \quad (2-6)$$

- where: P = shear perimeter of reinforced rock unit (4 x s)  
 $\phi$  = angle of internal friction for the rock mass  
k = ratio of average horizontal to average vertical stress  
R = shear radius of the reinforced rock unit ( $A/P = s/4$ )  
w = excavation span  
A = area of roof carried by one bolt (s x s)  
L = bolt length  
s = bolt spacing  
c = apparent cohesion of the rock mass  
D = height of distressed zone  
a = factor depending on time of installation of bolts  
 $\gamma$  = unit weight of the rock

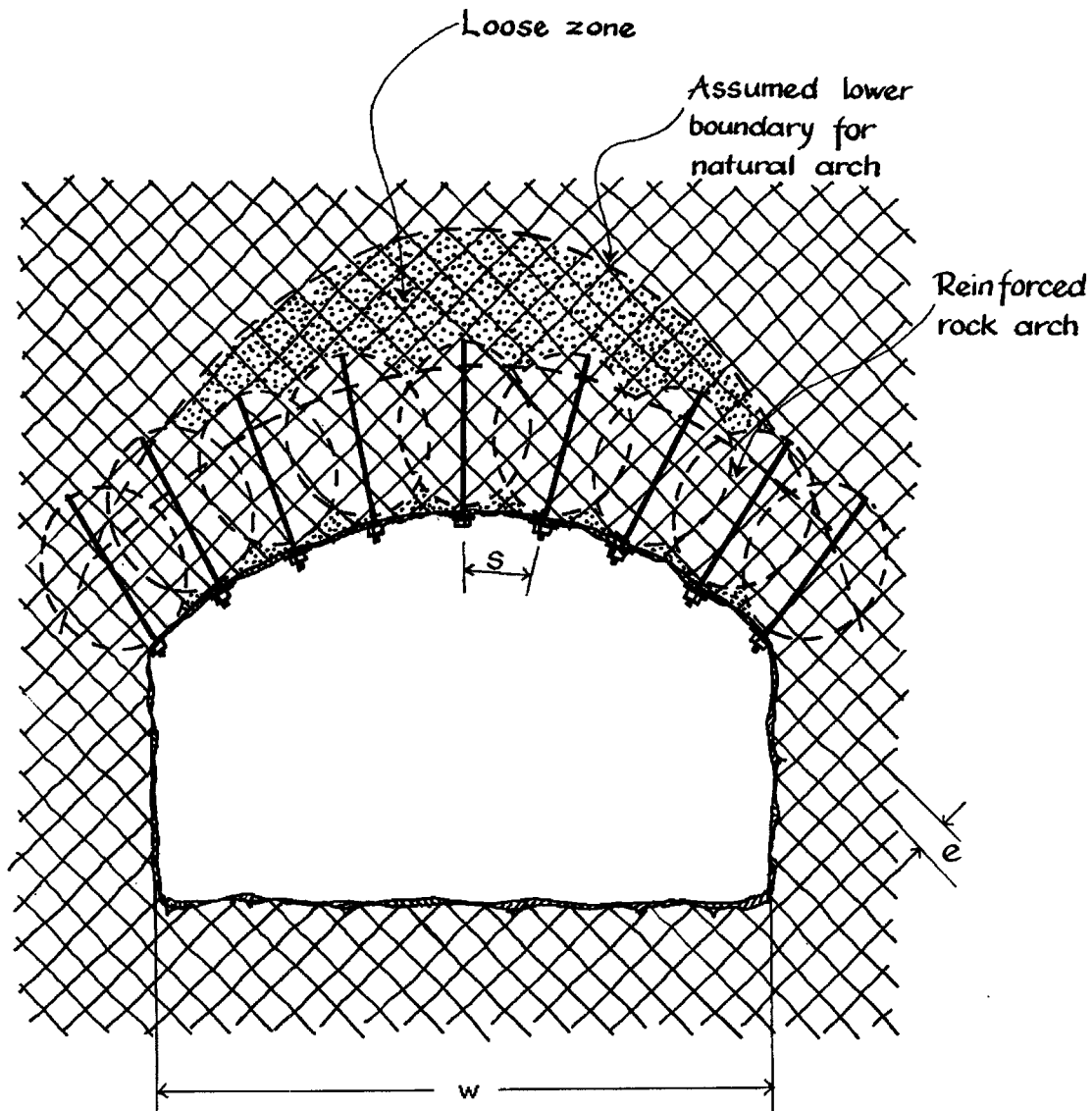
- Notes: 1) If the rock reinforcement is installed prior to the occurrence of significant deformation, then it is considered to have an active contribution to the excavation stability ( $a=0.5$ )  
2) If passive (more conservative) reinforcement is assumed,  $a=1.0$   
3) Cohesion should be taken as zero for initial design.

Guidelines for support system design in order to create a reinforced rock mass arch within a highly jointed rock mass (Figure 2-6) are:

$$\text{Bolt length} = 1.60 + \sqrt{(1.0 + 0.012 \cdot w^2)} \quad (\text{Schach et al. 1979}) \quad (2-8)$$

$$\text{Bolt spacing} = 3 \times \text{joint spacing (e)} \quad (\text{Stillborg 1986}) \quad (2-9)$$

$$\text{Bolt tension} = 0.5 - 0.8 \times \text{capacity of bolt} \quad (\text{Stillborg 1986}) \quad (2-10)$$



**Figure 2-6: Creation of reinforced rock mass arch within highly discontinuous rock mass structures ( after Stillborg 1986 )**

In addition, the requirement for areal coverage support systems such as mesh or shotcrete must be considered in order to maintain the integrity of the rock mass between the rockbolt reinforcement. These guidelines may find application within the South African mining environment although they are not commonly used. Consideration is also not given to specific characteristics of the rock mass and thus the design will tend to be conservative in nature.

It is considered (Stillborg 1986) that the concept of the creation of a structurally competent reinforced rock mass arch is the basis of most of the empirical rock bolt system design methods.

### 2.3.2 Empirical design methodologies

Due to the complexities within the rock mass structure, or the lack of detailed design parameters, the design of the rock bolt system based on specific structural analysis may not be possible. Under these conditions it is convenient to utilise empirical design methodologies based on analysis of the geotechnical environment. The analysis of the geotechnical environment, and thus design confidence, will be a function of the maturity of the design process and the availability of suitable geotechnical data.

The majority of empirical design methodologies are based on experience gained within shallow, low stress, civil engineering applications. They thus take limited consideration of an environment of stress change due to mining and the potential for rockbursting. As a result they find limited application within the general South African mining industry (Piper 1985). Exceptions can be found, particularly in the platinum and diamond mining industries, that operate at shallower depths.

Table 2-1: Tunnel support guidelines after US Corps of Engineers.

Parameter	Empirical Rule
Minimum length	Greatest of:
	a) 2 x bolt spacing
	b) 3 x thickness of critical and potentially unstable rock blocks
	c) For elements above the springline*
	spans < 6 m : 0.5 x span
	spans between 6 m and 18 m : interpolate 3 - 4.5 m
Minimum spacing	d) For elements below the springline*
	height < 18 m : as c) above
	height > 18 m : 0.2 x height
Minimum spacing	Least of:
	a) 0.5 x bolt length
	b) 1.5 x width of critical and potentially unstable blocks
Minimum spacing	c) 2.0 m (greater spacing difficult to attach fabric support)
	0.9 to 1.2 m
Minimum average confining pressure	Greatest of
	a) Above springline
	either pressure = vertical rock load of 0.2 x opening width
	or 40 kN/m <sup>2</sup>
	b) Below springline
either pressure = vertical rock load of 0.1 x opening height	
or 40 kN/m <sup>2</sup>	
	c) Intersections: 2 x confining pressure determined above

\* this is typically interpreted in South Africa as the point of deviation from vertical in the upper sidewall.

Simple design rules based on an evaluation of case studies were developed by the US Corps of Engineers. These rules have found widespread application, particularly as an initial estimate of support requirements, and are applied as the basis for rock bolt spacing in the design of rock bolt support systems within many South African mines. A summary of the recommendations is indicated in Table 2-1.

Other design rules for rock bolting within jointed rock mass structures, with tight joint conditions, were put forward by Farmer and Shelton (1980). These are similar to those given by the US Corps of Engineers, but consideration is also given to the number of joint sets and their orientation relative to the excavation and rock bolt installation (Table 2-2).

Table 2-2. Empirical guidelines for design of rock bolt system in excavations <15 m span (after Farmer and Shelton 1980)

Number of discontinuity sets	Rock bolt design	Comments
≤ 2 inclined at 0 - 45° to horizontal	$L = 0.3 B$ $s = 0.5.L$ (depending on thickness and strength of strata). Install bolts perpendicular to lamination where possible with wire mesh to prevent flaking	The purpose of bolting is to create a load carrying beam over the span. Fully bonded bolts create greater discontinuity shear stiffness. Tensioned bolts should be used in weak rock, sub-horizontal tensioned bolts where vertical discontinuities occur.
≤ 2 inclined at 45 - 90° to horizontal	For side bolts: $L > h.\sin\psi$ (if installed perpendicular to discontinuity): $L > h.\tan\psi$ (if installed horizontally). Where $h$ is the distance of installation from the point of daylight of laminations in the sidewall that defines the largest unstable wedge, and $\psi$ the inclination of laminations from vertical.	Roof bolting as above. Side bolts designed to prevent sliding along planar discontinuities. Spacing should be such that anchorage capacity is greater than sliding or toppling weight. Bolts should be tensioned to prevent sliding.
≤ 3 with clean, tight interfaces	$L = 2s$ $s = 3-4 \times$ block dimension. Install bolts perpendicular to excavation periphery with wire mesh to prevent flaking.	Bolts should be installed quickly after excavation to prevent loosening and retain tangential stresses. Pre-stresses should be applied to create a zone of radial confinement. Sidewall bolting where toe of wedge daylights in sidewall.

Wojno, Jager and Roberts (1987) and Jager, Wojno and Henderson (1990) have conducted investigations into the design and support of tunnels under high stress conditions and the definition of support performance requirements. The support guidelines applicable to this rock mass environment indicate the need for yielding tendons. The proposed specification of the support system, based on the previously discussed mechanistic evaluation (section 2.3.1), is as follows:

Work Done > 25 kJ  
 Static Yield > 100 KN  
 Dynamic Yield > 50 KN  
 Maximum Yield < 500 mm  
 Tendon Strength > 25 % static yield

These simple design rules allow initial estimations of support requirements but give limited or no consideration of the rock mass characteristics and as such will be very conservative in nature. A more detailed empirical design methodology can be based on a rock mass classification system to indicate the selection of a support system. The most widely used classification systems are the Q system (Barton, Lien and Lunde 1974) and the Geomechanics Classification, or Rock Mass Rating (RMR) system (Bieniawski 1973, 1979), which also forms the basis of the Modified (or Mining) Rock Mass Rating (MRMR) system (Laubscher, Taylor 1976).

The Geomechanics Classification was based on case studies of civil constructions within South Africa. The basic RMR value is derived from a rating based on the consideration of critical rock mass parameters. These include the intact rock strength, discontinuity frequency, discontinuity condition and the presence of water within the rock mass (Table 2-3)

Table 2-3. Geomechanics Classification system (Bieniawski 1973)

Parameter			Range of Values						
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial compressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25	1 - 5	< 1
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90 % - 100 %	75 % - 90 %	50 % - 75 %	25 % - 50 %	< 25 %		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surface. Not continuous. No separation. Un-weathered wall rock	Slightly rough surfaces. Separation < 1 mm. Slightly weathered walls	Slightly rough surfaces. Separation < 1 mm. Highly weathered walls	Slickensided surfaces. OR. Gouge < 5 mm thick. OR. Separation 1 - 5 mm continuous	Soft gouge > 5 mm thick. OR. Separation > 5 mm continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel	None	< 10 litres / min	10 - 25 litres / min	25 - 125 litres / min	> 125 litres / min		
		Ratio (joint water press. / major principle stress)	OR	OR	OR	OR	OR		
		General conditions	0	0.0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
	Rating		15	10	7	4	0		

Further adjustments may also be made in relation to the orientation of the joints relative to the excavation orientation and type to obtain an overall rock mass description (Tables 2-4, 2-5 and 2-6).

Table 2-4. Adjustment to RMR for joint orientation

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 2-5. Adjustment to RMR for joint orientations applicable to tunnels

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

Table 2-6. Rock mass class and description as determined from RMR

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

The description of the rock mass may be utilised to define overall rock mass parameters which may be used to analyse the large scale response of the rock mass (Table 2-7)

Table 2-7. Rock mass parameters based on RMR

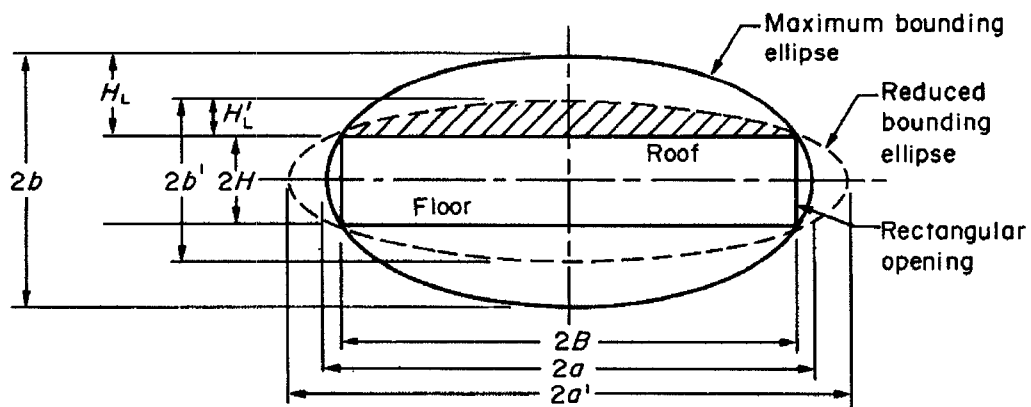
Class No.	I	II	III	IV	V
Average stand up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	35 - 45°	25 - 35°	15 - 25°	< 15°

Based on the Geomechanics Classification system as proposed by Bieniawski, support recommendations were made for a specific civil construction as detailed in Table 2-8. Due to the empirical nature of the design methodology, the recommendations are based on the experience and technology of the specific sites incorporated in the analysis.

Table 2-8. Excavation technique and support selection based on RMR.

<b>Shape: horseshoe; width: 10 m; vertical stress: below 25 MPa; construction: drill and blast</b>				
Rock mass class	Excavation	Support		
		Rock bolts (20 mm dia. fully bonded)	Shotcrete	Steel sets
Very good rock I RMR: 81 - 100	Full face 3 m advance	Generally no support required except for occasional spot bolting		
Good rock II RMR: 61 - 80	Full face 1.0 - 1.5 m advance Complete support 20 m from face	Locally bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
Fair rock III RMR: 41 - 60	Top heading and bench 1.5 - 3 m advance in top heading. Commence support after each blast. Complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5 m - 2 m in crown and walls with wire mesh in crown.	50 - 100 mm in crown and 30 mm in sides	None
Poor rock IV RMR: 21 - 40	Top heading and bench 1 - 1.5 m advance in top heading. Install support concurrently with excavation 10 m from face	Systematic bolts 4 - 5 m long, spaced 1 - 1.5 m in crown and walls with wire mesh	100 - 150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required.
Very poor rock V RMR: < 20	Multiple drifts. 0.5 - 1.5 m advance in top heading. Install support concurrently with excavation, Shotcrete as soon as possible after blasting	Systematic bolts 5 - 6 m long, spaced 1 - 1.5 m in crown and walls with wire mesh. Bolt invert	150 - 200 mm in crown, 150 mm in sides and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and fore poling if required. Close invert.

Applications of the rock mass rating system have included a semi-empirical method for the estimation of the depth of instability around an excavation (Stimpson 1989). This system defines an elliptical boundary around the excavation within which the rock mass is considered to require reinforcement (Figure 2-7).



Maximum principal stress is horizontal	Maximum principal stress is vertical
$a = kb$	$a = (H^2 + k^2 \cdot B^2)^{0.5}$
$b = (B^2 + k^2 \cdot H^2)^{0.5} / k$	$b = a/k$
$a' = Bb' / (b^2 - H^2)^{0.5}$	$a' = (1 - 0.01 \cdot \text{RMR})a + H(0.01 \cdot \text{RMR})$
$b' = (1 - 0.01 \cdot \text{RMR})b + H(0.01 \cdot \text{RMR})$	$b' = Ba' / (a'^2 - H^2)^{0.5}$
$H_L = b - H$	$H_L = a - H$
$H'_L = (1 - 0.01 \cdot \text{RMR}) \cdot H_L$	$H'_L = (1 - 0.01 \cdot \text{RMR}) \cdot H_L$

**Figure 2-7. Semi-empirical design of the extent of instability around an excavation based on RMR (after Stimpson (1989))**

where  $k$  = ratio of maximum to minimum stresses  
 $2B$  = opening width  
 $2H$  = height of opening  
RMR = Rock Mass Rating  
 $H_L$  = maximum height requiring support  
 $H'_L$  = reduced height requiring support  
 $a, a'$  = semi major axis of ellipse  
 $b, b'$  = semi minor axis of ellipse

Laubscher and Taylor (1976) suggested modifications to the Geomechanics Classification system, and its associated support recommendations. These modifications are to cater for the differences in support requirements between the original civil environment and a mining environment. The modifications represent an alteration (percentage) of the individual parameters within the RMR as a function of weathering, stress environment, excavation orientation relative to the rock mass structure, and the excavation method.

The influence of weathering is applied to the intact rock strength (decrease down to 96 %), RQD (decrease down to 95 %) and condition of joints (decrease down to 82 %), dependent on the degree of weathering. The influence of the stress environment is a function of the initial field stress level and any potential stress changes, and is considered to influence the relative joint condition. Where the stress environment is, or becomes, compressive, relative to the rock mass structure and excavation periphery, this will be favourable to joint conditions (increase up to 120 %). Where shear deformation may be anticipated on the joints in the periphery of the excavation, or a low stress results in the opening of joints, this will be unfavourable to rock mass stability (decrease down to 90 % or 76 % respectively). Where subsequent stress changes occur, this may also cause adverse shear movement or joint opening within the excavation peripheral rock mass (decrease down to 60 %).



Laubscher and Taylor considered the influence of the excavation orientation relative to the rock mass structure as accounted for by Bieniawski to be insufficient, and thus further adjustments may be required. These adjustments are made to the joint spacing rating and are based on the number of non-vertical rockwalls of the excavation and the number of joints (Table 2-9).

Table 2-9. Percentage adjustment to joint spacing rating for joint number and number of non-vertical excavation rockwalls.

Number of joints	70 %	75 %	80 %	85 %	90 %
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6		4	3	2 - 1

A further adjustment to the joint spacing rating is also considered necessary for the presence and orientation of shear zones (decrease dependent on angle of incidence down to 0 - 15° = 76%, 15 - 45° = 84 % and 45 - 75° = 92 %). Finally an adjustment may be made to the RQD and condition of joints rating for the influence of the method of excavation. The degree of reduction of the parameters is based on the consideration of the adverse effects of blasting (boring – 100 %, smooth wall blasting – 97 %, good conventional blasting – 94 %, poor conventional blasting – 80 %) on the stability of the immediate peripheral rock mass of excavations.

It was considered by Laubscher and Taylor that the application of these adjustments to the original parameters of the Geomechanics Classification, and in some cases several parameters for a specific site, should not reduce the modified RMR by more than 50 % of the original value. These modified Geomechanics Classifications were compared to typical mining support practices in order to define a relationship between the MRMR and support systems as indicated in Table 2-10.

Table 2-10. Support system guide based on Modified Geomechanics Classification after Laubscher and Taylor (1976).

Adjusted ratings	Original rock mass ratings, RMR									
	90 - 100	80 - 90	70 - 80	60 - 70	50 - 60	40 - 50	30 - 40	20 - 30	10 - 20	0 - 10
60 - 100										
50 - 60		a	a	a	a					
40 - 50			b	b	b	b				
30 - 40				c,d	c,d	c,d,e	d,e			
20 - 30					g	f,g	f,g,j	f,h,j		
10 - 20						i	i	h,i,j	h,j	
0 - 10							k	k	l	l

- a) Generally no support but locally joint intersections might require bolting.
- b) Patterned grouted bolts at 1 m collar spacing.
- c) Patterned grouted bolts at 0.75 m collar spacing.
- d) Patterned grouted bolts at 1 m collar spacing and shotcrete 100 mm thick.
- e) Patterned grouted bolts at 1 m collar spacing and massive concrete 300 mm thick and only used if stress change is not excessive.
- f) Patterned grouted bolts at 0.75 m collar spacing and shotcrete 100 mm thick.
- g) Patterned grouted bolts at 0.75 m collar spacing with mesh reinforced shotcrete 100 mm thick.
- h) Massive concrete 450 mm thick with patterned grouted bolts at 1 m spacing if stress changes are not excessive.
- i) Grouted bolts at 0.75 m collar spacing if reinforcing potential is present, and 100 mm reinforced shotcrete, and then yielding steel arches as a repair technique if stress changes are excessive.
- j) Stabilise with rope cover support and massive concrete 450 mm thick if stress changes not excessive.
- k) Stabilise with rope cover support followed by shotcrete to and including face if necessary, and then closely spaced yielding arches as a repair technique where stress changes are excessive.
- l) Avoid development in this ground otherwise use support systems j or k.

Supplementary notes:

- I The original Geomechanics Classification as well as the adjusted ratings must be taken into account in assessing the support requirements.
- II Bolts serve little purpose in highly jointed ground and should not be used as the sole support where the joint spacing rating is less than 6.
- III The recommendations contained in the above table are applicable to mining operations with stress levels less than 30 MPa.
- IV Large chambers should only be excavated in rock with adjusted total classification ratings of 50 or better.

These support design guidelines have found some application within the shallow mining environments, such as platinum and diamond, within the South African mining industry. Although comparable support systems are utilised at depth, the design is generally not based on these guidelines.

Applications of the MRMR have included the determination of the minimum rock bolt density derived from case studies of rock bolting in hard rock mines of Quebec (Choquet and Charette 1988). The minimum required support density ( $D$  = No. of bolts/m<sup>2</sup>) is given by:

$$D = - 0.0214.MRMR + 1.68 \quad (\text{or } D = - 0.227.\ln Q + 0.839) \quad (2-11)$$

thus

$$s = 1/D^{0.5} \quad (2-12)$$

where:  $Q$  = rating based on the NGI  $Q$  system classification and  $s$  = bolt spacing on a square pattern.

The other widely utilised and internationally recognised rock mass classification system is the  $Q$  system (Barton, Lien and Lunde, 1974). This again is generally applicable to shallow tunnel excavations, as these form the majority of the empirical database. However, recent experience with dynamic failure of the immediate peripheral rock mass of excavations, known as “strain bursting”, has been incorporated within this system (Barton 1997). The system utilises rock mass parameters to determine a  $Q$  value for which there is a recommended support system based on excavation dimensions and utilisation. The tables of the rock mass parameters, which are used to define the rock mass, and their relative weighting, are shown under Table 2-11.

Table 2-11. Tables for classification of the rock mass based on the  $Q$  system.

1. Rock Quality Designation		RQD
A	Very poor	0 - 25
B	Poor	25 - 50
C	Fair	50 - 75
D	Good	75 - 90
E	Excellent	90 - 100
Note i) Where RQD is reported or measured as $\leq 10$ (including 0), a nominal value of 10 is used to evaluate $Q$ .		
ii) RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate		

2. Joint Set Number		Jn
A	Massive, no or few joints	0.5 - 1.0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, “sugar cube”, etc.	15
J	Crushed rock, earth like.	20
Note i) For intersections use, $3 \times J_n$ .		
ii) For portals, use $2 \times J_n$ .		

3. Joint Roughness Number		Jr
a) Rock wall contact, and b) rock wall contact before 10 cm shear		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
Note: i) Descriptions refer to small scale features and intermediate scale features in that order		
c) No rock wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J	Sandy, gravely or crushed zone thick enough to prevent rock wall contact.	1.0
Note: i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m. ii) Jr = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are orientated for minimum strength.		

4. Joint Alteration Number		$\phi_r$ approx.	Ja
a) Rock wall contact (no mineral filling, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling i.e. quartz or epidote.	-	0.75
B	Unaltered joint walls, surface staining only	25 - 35°	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay free disintegrated rock, etc.	25 - 30°	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20 - 25°	3.0
E	Softening or low friction clay mineral coatings i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc. and small quantities of swelling clays	8 - 16°	4.0
b) Rock wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay free disintegrated rock, etc.	25 - 30°	4.0
G	Strongly over consolidated non-softening clay mineral fillings (continuous, but < 5 mm thickness)	16 - 24°	6.0
H	Medium or low over consolidation, softening, clay mineral fillings (continuous, but < 5 mm thickness)	12 - 16°	8.0
J	Swelling clay fillings, i.e. montmorillonite (continuous, but < 5 mm thickness). Value of Ja depends on percent of swelling clay-size particles, and access to water, etc.	6 - 12°	8 - 12
c) No rock wall contact when sheared (thick mineral fillings)			
K,L,M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6 - 24°	6, 8, or 8 - 12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	-	5.0
O,P,R	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6 - 24°	10, 13, or 13 - 20

5. Joint Water Reduction Factor		approx. water pres. (kg/cm <sup>2</sup> )	Jw
A	Dry excavations or minor inflow, i.e. < 5 l/min locally	<1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1 - 2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5 - 10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5 - 10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2 - 0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1 - 0.05
Notes: i) Factors C to F are crude estimates, Increase Jw if drainage measures are installed. ii) Special problems caused by ice formation are not considered.			

6. Stress Reduction Factor		SRF
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated		
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leq 50$ m)	5
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $> 50$ m)	2.5
D	Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5
E	Single shear zones in competent rock (clay free) (depth of excavation $\leq 50$ m)	5.0
F	Single shear zones in competent rock (clay free) (depth of excavation $> 50$ m)	2.5
G	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)	5.0
Note: i) Reduce these values of SRF by 25 - 50 % if the relevant shear zones only influence but do not intersect the excavation.		

b) Competent rock, rock stress problems		$\sigma_c / \sigma_1$	$\sigma_\theta / \sigma_c$	SRF
H	Low stress, near surface, open joints	$> 200$	$< 0.01$	2.5
J	Medium stress, favourable stress condition	200 - 10	0.01 - 0.3	1
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability	10 - 5	0.3 - 0.4	0.5 - 2
L	Moderate slabbing after $> 1$ hour in massive rock	5 - 3	0.5 - 0.65	5 - 50
M	Slabbing and rock burst after a few minutes in massive rock	3 - 2	0.65 - 1	50 - 200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	$< 2$	$> 1$	200 - 400
Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$ , reduce $\sigma_c$ to $0.75 \sigma_c$ . When $\sigma_1 / \sigma_3 > 10$ , reduce $\sigma_c$ to $0.5 \sigma_c$ , where $\sigma_c$ = unconfined compression strength, $\sigma_1$ and $\sigma_3$ are the major and minor principle stresses, and $\sigma_\theta$ = maximum tangential stress (estimated from elastic theory).				
iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H)				

c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure.		$\sigma_\theta / \sigma_c$	SRF
O	Mild squeezing rock pressure	1.5	5 - 10
P	Heavy squeezing rock pressure	$> 5$	10 - 20
Note: iv) Cases of squeezing rock may occur for depth $H > 350 Q^{1/3}$ (Singh et al., 1992). Rock mass compression strength can be estimated from $q = 7 \gamma Q^{1/3}$ (MPa) where $\gamma$ = rock density in gm/cc (Singh 1993).			
d) Swelling rock: chemical swelling activity depending on pressure of water			
R	Mild swelling rock pressure		5 - 10
S	Heavy swelling rock pressure		10 - 15

Note: Jr and Ja classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance (where $\tau = \sigma_n \tan (Jr/Ja)$ )	
$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (2-13)$	

Thus, by the determination of the applicable rock mass characteristic, within the defined parameter fields, and their associated rating, an overall classification of the rock mass is obtained. This may now be utilised to assess the stability of the proposed excavation and recommend suitable support systems based on case studies.

The assessment of the relative stability of an excavation from the classification of the rock mass, based on the Q system, will also be dependent on the size and utilisation of the excavation. An assessment of the relative safety of an excavation defines the Excavation Support Ratio (ESR) as shown in Table 2-12.

Table 2-12. Recommended ESR values for excavation safety level.

Type of Excavation	ESR	
A	Temporary mine openings, etc.	2 - 5
B	Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers.	1.6 - 2.0
C	Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels.	1.2 - 1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections.	0.9 - 1.1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels.	0.5 - 0.8

For unsupported stability the maximum span of an opening is given by:

$$\text{Span of opening} = 2 \times \text{ESR} \times Q^{0.4} \quad (2-14)$$

The Q classification caters for rock mass structures that necessitate systematic support systems. For excavation spans that are indicated to be self supporting, consideration must still be given to the support of potentially unstable blocks defined by jointing.

At spans in excess of those indicated to be self supporting, support recommendations are given as shown in Figure 2-8.

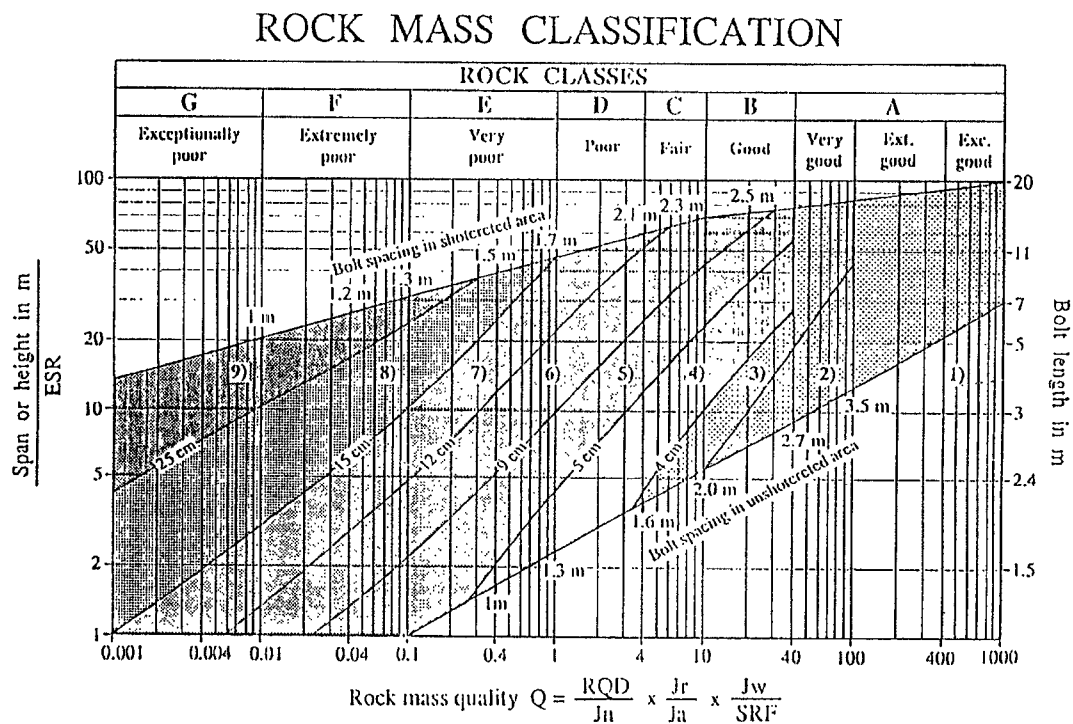
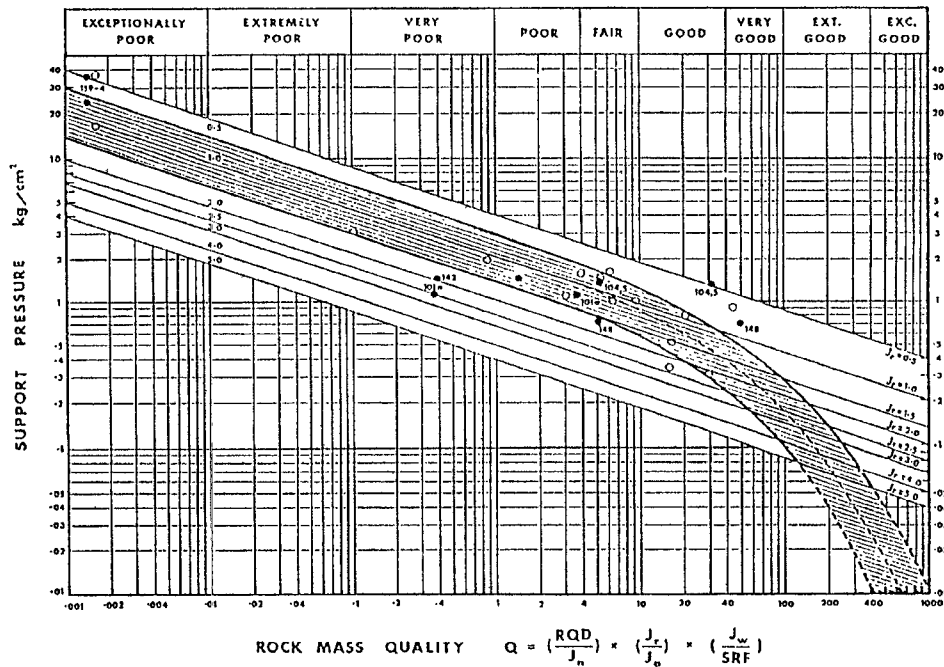


Figure 2-8. Support recommendations based on the Q system (Barton, 1997).

The Q system has found application in tunnel excavations of shallow mining orebodies within South Africa, but has found very limited application in deep level mining environments and usually then with local adjustments.

Within the Q system, the length of the rock bolts is purely a function of the relative size of the excavation, although the spacing of the rock bolts and the necessity for, and type of, fabric support is a function of the rock mass classification. The support pressure, or distributed rock bolt load, may also be defined as a function of the Q value and the roughness of the discontinuity which is likely to result in the potential instability (Figure 2-9).



**Figure 2-9. Determination of support pressure from Q value and joint roughness condition (Barton 1997).**

Guidelines for the design of primary support systems were proposed by Barton, Lien and Lunde (1974) based on the Q system as shown in Table 2-13.

Table 2-13. Estimations of support pressure, length and spacing of primary support based on the Q system.

Rock mass jointing	Support pressure	Length and spacing of reinforcement
If the number of discontinuity sets > 2	$P_{\text{roof}} = (0.2Q^{-1/3}) / J_r$	Bolts $L_{\text{roof}} = (2 + 0.15B)/\text{ESR}$ Anchors $L_{\text{roof}} = (0.4B)/\text{ESR}$ $s = ((Cx10^{-3})/P_{\text{roof}})^{1/2}$
If the number of discontinuity sets $\leq 2$	$P_{\text{roof}} = (0.2 \cdot J_n^{-1/2} Q^{-1/3}) / 3J_r$	
	$P_{\text{wall}}$ : calculated with the same formulae as $P_{\text{roof}}$ , by replacing Q with $Q'$ : $Q' = 5Q$ if $Q > 10$ $Q' = 2.5Q$ if $0.1 \leq Q \leq 10$ $Q' = Q$ if $Q < 0.1$	Bolts $L_{\text{wall}} = (2 + 0.15H)/\text{ESR}$ Anchors $L_{\text{wall}} = (0.35H)/\text{ESR}$ $s = ((Cx10^{-3})/P_{\text{roof}})^{1/2}$

Where  $J_r$  = joint roughness number;  $J_n$  = joint set number;  $P$  = support pressure (MPa);  $B$  = span of excavation (m);  $H$  = height of excavation (m);  $\text{ESR}$  = excavation support ratio;  $L$  = length of reinforcement (m);  $s$  = spacing of reinforcement (m);  $C$  = load exceeding yield strength of bolt (kN).

Tunnel support design guidelines applicable to tunnels developed under high, and often variable, stress environments, and with the potential for rockbursts, are limited in number, due to the unique nature of this environment. The recommended, and widely utilised, design guidelines within the South African mining industry are those based on the work of Wiseman (1979), and given in the Chamber of Mines Research Organization industry design guidelines (Anon., 1988). The design criterion for the control of tunnel condition and the recommendation of suitable support systems is the Rockwall Condition Factor (RCF).

$$\text{RCF} = (3\sigma_1 - \sigma_3) / F \cdot \sigma_c \quad (2-15)$$

where  $\sigma_1$  and  $\sigma_3$  are the maximum and minimum principle stresses within the plane of the excavation cross section and F is a factor to represent the downgrading of  $\sigma_c$ , the uniaxial compressive strength, for the rock mass condition and excavation size. In a highly discontinuous rock mass it is recommended that F is approximately 0.5, and in large excavations (> 6 m x 6 m) F may be further downgraded by 10 %.

The formulation of the RCF represents a comparison of the maximum induced tangential stress of an assumed circular excavation to the estimated rock mass strength. Wiseman (1979) used this criterion to examine the implementation of support systems within South African Witwatersrand gold mine tunnels. This allowed the development of an empirical relationship between the support systems and typical (3 m x 3 m ) mine tunnels within this specific geotechnical environment.

In general it was found that for  $RCF < 0.7$  good conditions prevailed with minimum support (Table 2-14); for  $0.7 < RCF < 1.4$  average conditions prevailed with typical support systems (Table 2-15); and for  $RCF > 1.4$  poor ground conditions prevailed with specialist support requirements (Table 2-16).

Table 2-14. Support recommendation for good ground conditions ( $RCF < 0.7$ ).

Case	Primary support (Typical requirements)	Secondary support (Typical requirements)
Static stress conditions	"Spot" support where necessary (Split Sets, rock studs, etc.)	"Spot" support where necessary (rock studs, etc.)
Stress changes anticipated	"Spot" support where necessary (Split Sets, rock studs, etc.)	Fully grouted tendons > 1.5 m in length, installed on basic 2 m pattern. Support resistance: 30 - 50 kN/m <sup>2</sup> . Rope lacing (sidewalls only)
Seismic activity anticipated	Split Sets or tendons > 1.2 m in length, installed as close to face as possible, on basic 2 m or 1.5 m pattern. Support resistance: 30 - 50 kN/m <sup>2</sup> .	Fully grouted (pref. yield) tendons > 1.5 m in length, installed on basic 2 m pattern. Support resistance: 50 kN/m <sup>2</sup> . Mesh and lace (hangingwall and sidewalls)

Table 2-15. Support recommendation for average ground conditions ( $0.7 < RCF < 1.4$ ).

Case	Primary support (Typical requirements)	Secondary support (Typical requirements)
Static stress conditions	Rock studs or tendons > 1.5 m in length, installed as close to face as possible on basic 2 m pattern. Support resistance: 30 - 50 kN/m <sup>2</sup> .	Steel strapping or rope lacing integrated with primary support tendons; or gunite.
Stress changes anticipated	Fully grouted tendons or steel ropes > 1.8 m in length, installed as close to face as possible, on basic 2 m or 1.5 m pattern. Support resistance: 40 - 60 kN/m <sup>2</sup> .	Rope lacing and wire mesh integrated with primary support tendons.
Seismic activity anticipated	Fully grouted (pref. yield) tendons or steel ropes > 1.8 m in length, installed as close to face as possible, on 1.5 m or double 2 m pattern. Support resistance: 80 - 110 kN/m <sup>2</sup> .	Rope lacing and wire mesh integrated with primary support tendons, plus optional gunite.

Table 2-16. Support recommendation for poor ground conditions (RCF > 1.4).

<b>Case</b>	<b>Primary support (Typical requirements)</b>	<b>Secondary support (Typical requirements)</b>
<b>Static stress conditions</b>	(If necessary) gunite to face, then fully grouted tendons or steel ropes, length > 1.8 m on basic 1.5 m pattern, as close to face as possible. Support resistance: 80 - 110 kN/m <sup>2</sup> .	Steel wire mesh integrated with primary support; optional gunite / shotcrete.
<b>Stress changes anticipated</b>	(If necessary) gunite to face, then fully grouted steel ropes or yielding tendons, length > 1.8 m on basic 1 m or double 2 m pattern, as close to face as possible. Support resistance: 120 - 230 kN/m <sup>2</sup> .	Rope lacing and wire mesh integrated with primary support. Add integral gunite in long life tunnels. If necessary, additional hangingwall support comprising grouted steel ropes.
<b>Seismic activity anticipated</b>	(If necessary) gunite to face, then fully grouted steel ropes or yielding tendons, length > 2.3 m on basic 1 m pattern, as close to face as possible. Support resistance: 220 - 290 kN/m <sup>2</sup> .	Rope lacing and wire mesh integrated with primary support. Add integral gunite in long life tunnels. If necessary, additional hangingwall support comprising grouted steel ropes.

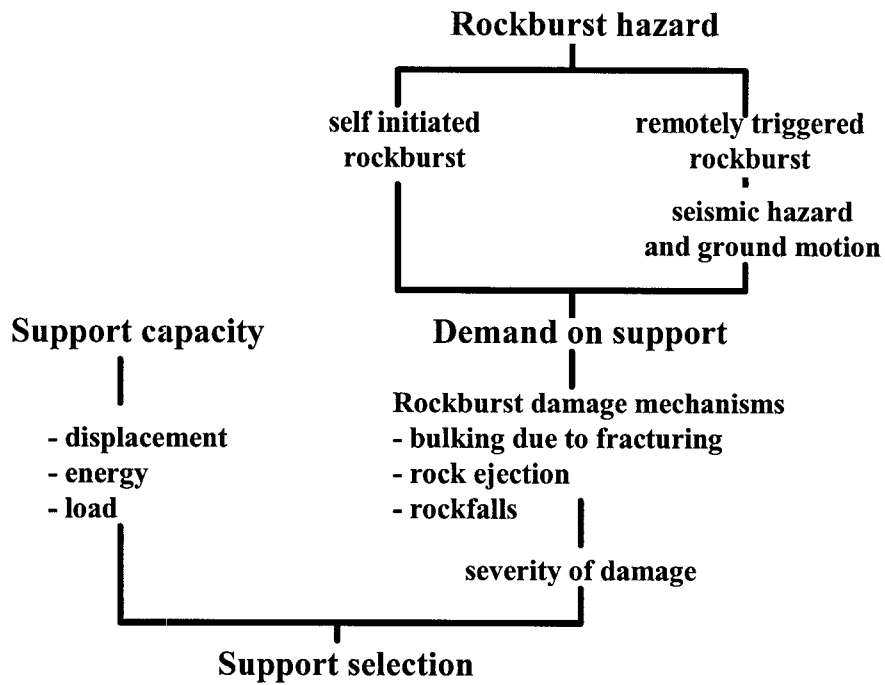
The empirical relationship, as evaluated by Wiseman, thus primarily considered the influence of the stress environment on the excavation rock mass condition, in relation to the typical support systems utilised. In addition (Anon., 1988) consideration was also taken of the potential for seismicity and thus the rockburst hazard, and also for an environment of stress change. The empirical nature of the design is considered to cater well for bedded deep level rock mass environments, but may be less effective in environments outside of this empirical range.

The classification systems, as reviewed in this section, tend to result in a specific support category for a given rock mass condition. Thus, as the rock mass changes, due to a change in stratigraphy or stress environment, then the support recommendations may dramatically alter due to the transition to a different rock mass class. The coarser the classification system support categories, the greater this change is likely to be. Within a mining environment, with numerous areas of simultaneous tunnel excavation, the logistics of supply and control of the support system components make this potential scenario problematic. Thus there is a tendency towards standard support systems which will generally cater for the more adverse ground conditions. This results in inefficient, over supporting of the more favourable ground conditions.

Recent work based on experience in Canadian mines at depth (Kaiser, McCreath and Tannant, 1996) also examines the influence of seismicity on the design of tunnel support. The basis of the design recommendations is an evaluation of the potential for rockburst damage, and the mechanism of damage. This may vary from rockfall to violent ejection of the rock mass. An evaluation of this failure process allows the determination of the demand on a support unit, which thus allows an evaluation of the capacity of the support system.

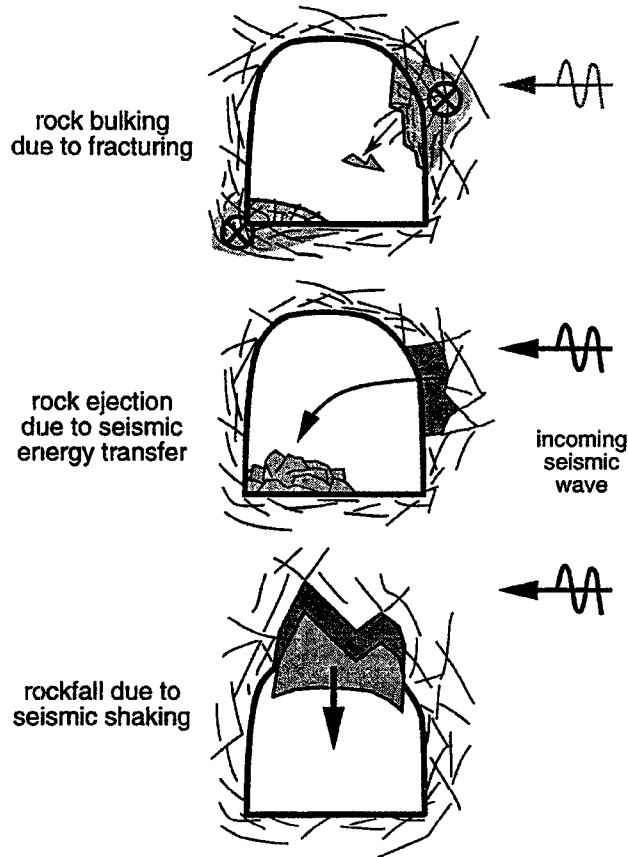
The design methodology for support selection as detailed by Kaiser *et al.* (1996) is shown in Figure 2-10.





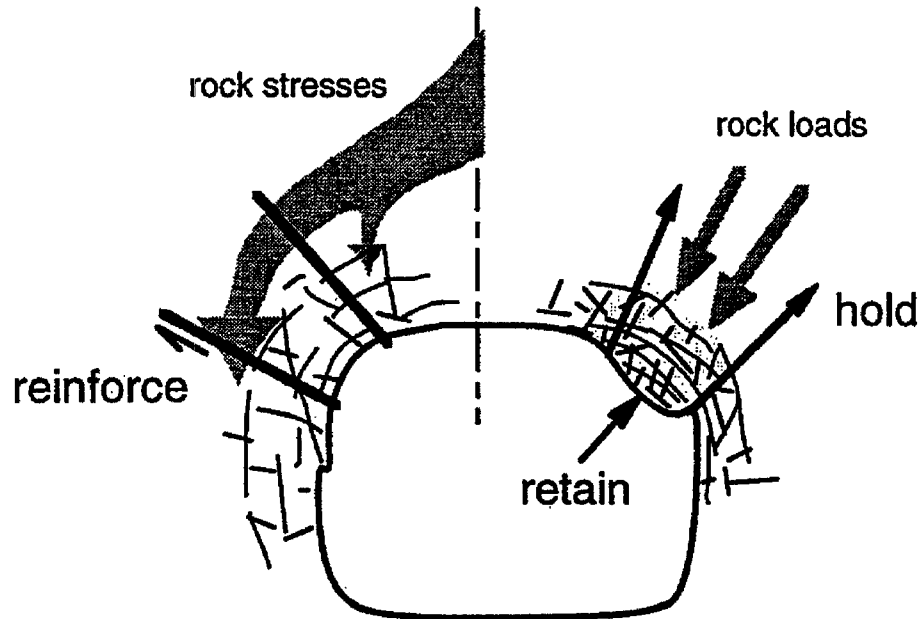
**Figure 2-10. Approach for support design under rockburst conditions after Kaiser et al. (1996)**

The demand on the support will be a function of the rock mass environment and the potential for seismic events, which will determine the mechanisms of rockburst damage as shown in Figure 2-11.



**Figure 2-11. Rockburst damage mechanisms as defined by Kaiser et al. (1996)**

The design requirements for the support system are indicated to be based on the envisaged damage mechanisms. From this, the support capacity and rock mass interaction can be defined as indicated in Figure 2-12.



**Figure 2-12. Primary functions of support elements.**

The capacity of typical support systems (rock bolts, mesh and shotcrete) was evaluated by Kaiser *et al.* (1996) to determine the energy absorption capability of the systems compared to the envisaged demand. This knowledge can then be utilised to enable the design engineer to select suitable support systems based on the proposed methodology.

Aspects of this recent work are compatible with the study being conducted by the author, and these relevant sections are discussed in more detail under the appropriate aspects of research within this report. The work of Kaiser *et al.* (1996) gives good insight into the demands on the support systems, and consideration was also given to the loading of the support system. However, the assumption of this analysis is that the support is loaded as a system and thus tributary area loading and average support resistance of the support system are considered. This assumption is generally valid under the more massive rock mass conditions of the excavation periphery in the North American mining environment. However, this differs significantly from the current investigation where the interaction of the components of the support system within a highly discontinuous rock mass structure may result in significant differential loading.

## 2.4 Conclusions

The design of tunnel support systems is principally based on either mechanistic evaluation or empirical design rules. Mechanistic evaluation allows the estimation of the demand and characteristics of the support system based on an understanding of the interaction between the support and the rock mass. This allows an estimation of the load and deformation requirements of the specific components of the support system. However, these design considerations are currently based on relatively simple mechanisms and are generally only applicable to simple rock mass structures. As the complexity of the rock mass environment increases, design procedures tend to be based more on empirical design rules derived from experience in similar rock mass environments. This design procedure requires some form of assessment, or classification, of the rock mass in order to define the comparable rock mass environment. Most classification systems currently in use have been derived from civil

engineering experience. They thus reflect a shallow, generally low, static stress environment and support systems that will ensure levels of excavation stability suitable for public use over long time frames. The applicability of these design procedures to the mining environment, and particularly deep level mining, is limited due to the significant differences in the dynamic rock mass environment and the required level of support. Empirical design procedures that have been developed specifically for the South African mining environment have proved successful in the majority of rock mass conditions encountered. However, in more adverse rock mass conditions, where significant failure of the rock mass around a tunnel excavation has occurred, and where the tunnel is subjected to dynamic loading, then failure of the support systems are encountered. The large variability in the rock mass environments within mining operations also leads to complex rock mass behaviour that is often not captured within the empirical design rules. Many of the failures of the support systems can be linked to the lack of a rational design procedure which encompasses an understanding of the mechanistic interaction between the rock mass and the support system. Recently legislation by the Department of Minerals and Energy (Anon., 1996) has recommended that support design should be based on mechanistic evaluation. The basis of this current procedure gives consideration to the support resistance or energy absorption capability of the rock bolt reinforcement system based on tributary area loading. This assumes that the rock mass between the rock bolts is inherently stable and there is thus no mechanistic consideration of the requirement for, or demand on, a fabric support system. However, these are often considered a necessity in many tunnels within the South African deep level mining environment, and, thus, the selection of these systems is still based on empirical guidelines.

There is thus an immediate need for a more rigorous method to determine, on an engineering basis, the interaction between the rock mass and all components of the support system to provide a practical and flexible support design alternative.

The focus of this research work is to improve the understanding of the mechanistic interaction of the components of the support system, both the rock bolt reinforcement and the fabric support systems, in an environment typical of that encountered in deep level mining. This understanding should allow improved estimation of the demand on the components of the support systems and thus enhanced design procedures for these mining environments.

### **3. Research Methodology**

#### **3.1 Introduction**

Off reef excavations are often subjected to ground motions radiated from seismic events, which may result in severe, and sometimes quite erratic damage. This damage will be a function of the site response, which will be controlled by the current state of the support system and rock mass, in relation to the proximity to the seismic event.

In order to evaluate the current performance of support systems within this environment, a series of detailed case studies were conducted over a period of approximately three years (Haile, 1998). These visits concentrated on events that caused significant failure within the rock mass, and of the support system. In this way a detailed picture of the mechanisms of support interaction, and the relative performance of the support systems, could be constructed.

These investigations reviewed the mechanisms of deformation of the rock mass in the vicinity of these tunnels. In this way it is considered that insight may be gained into the interaction between the rock mass and the support system.

In addition to the observational evaluation of deformation mechanisms under dynamic loading, in situ investigations based on instrumentation were conducted. These included the evaluation of the deformation of the rock mass around a tunnel, an estimation of the inherent integrity of the fractured rock mass by physical pull tests, estimation of rock mass dilation and

measurement of ground velocities of the tunnel skin in relation to the rock bolt reinforcement. These in situ evaluations were further evaluated by detailed numerical modelling with particular emphasis on discrete element modelling. These evaluations were carried out to enable a mechanistic understanding of the interaction between the rock mass and the rock bolt reinforcement.

The investigation of the rock mass environment in the vicinity of a tunnel will allow an estimation of the quasi static and dynamic demand on the support system. The other major component of the project was thus a mechanistic evaluation of the capacity of the typical elements of a support system under comparable loading conditions. An improved understanding of the demand and capacity of a support system will enable improved support selection and system design.

### **3.2 Observations of rock mass behaviour and support performance**

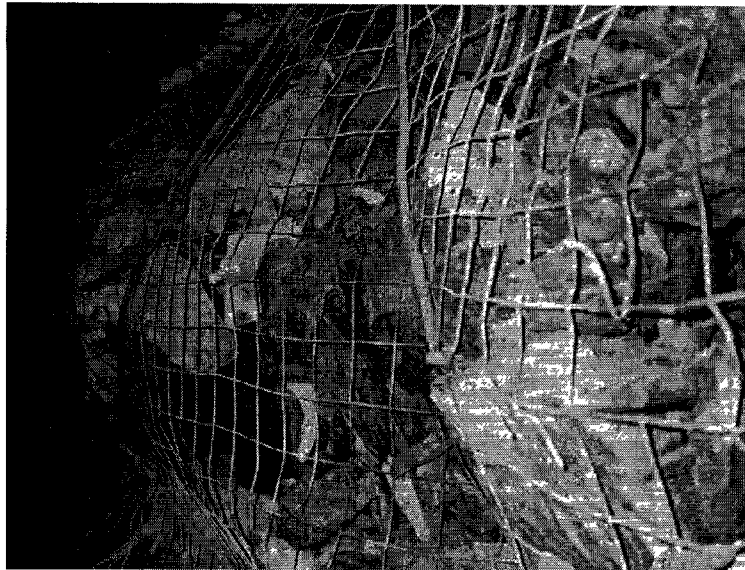
The work in this section of the investigation addresses aspects of enabling output 4 (Classification, behaviour and limits of stability of a fractured rock mass around tunnel excavations), output 5 (Determination of mechanisms of excavation stabilisation) and output 7 (Determination of interaction between support and the rock mass).

From the detailed rockburst case studies, the following points are highlighted with regard to the performance of the support systems and the mechanisms of failure of the rock mass.

- Bulking of the rock mass between the rock bolt reinforcement was observed at all the rockburst sites. At 70 % of the sites this was observed to have resulted in failure of the fabric support system over a significant area of investigation (Photograph 3-1). At 30 % of the sites the integrity of the fabric support system was maintained and the unstable rock mass between the rock bolt reinforcement was contained (Photograph 3-2).

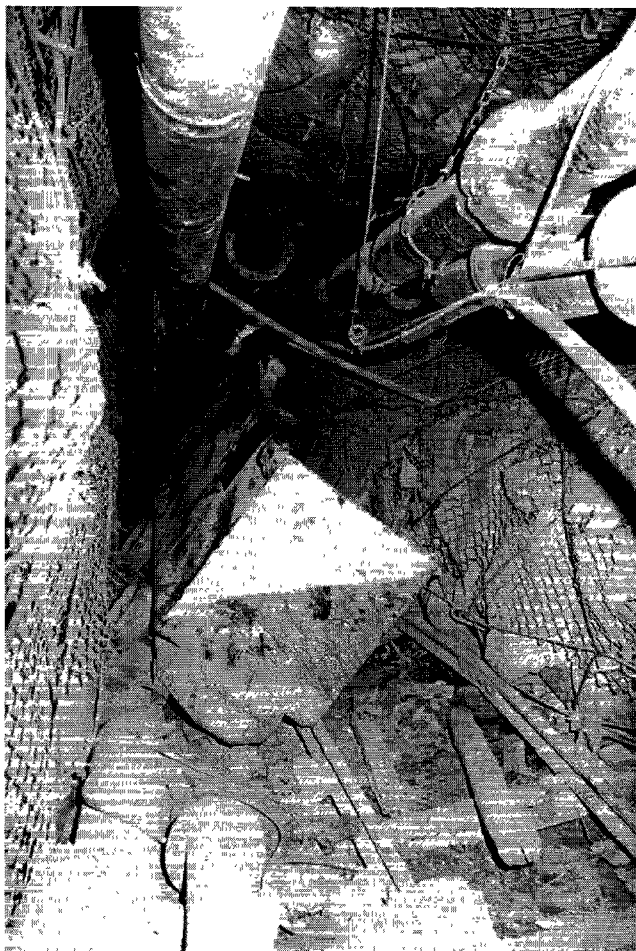


***Photograph 3-1. Failure of mesh panel.***



***Photograph 3-2. Rock mass containment by mesh panel.***

- Large scale deformation over the whole height of the sidewall was observed at 60 % of the investigations (Photograph 3-3). This was especially prevalent at investigations conducted in the Klerksdorp gold field. This may be a function of the different geotechnical environment and / or the more prolific use of smooth bar rock bolt reinforcement compared to the use of ripple bar on the West Rand gold field. Only isolated beam type failure was associated with the hangingwall of the tunnel.



***Photograph 3-3. Large scale sidewall deformation.***

- Shear failure of rock bolt reinforcement was observed at 70 % of the rockburst sites (Photograph 3-4). In all cases this was in the hangingwall of the tunnel excavation.



***Photograph 3-4. Guillotining of hangingwall rock bolt.***

- Tensile failure of rock bolt reinforcement was directly observed at 60 % of the rockburst sites (Photograph 3-5) but of limited occurrence at each site. This was associated with both the hangingwall and sidewalls of the tunnels.



***Photograph 3-5. Tensile failure of hangingwall rock bolt.***

Of significance from this investigation is the large proportion of damage associated with the bulking and unravelling of the sidewall rock mass between the rock bolt reinforcement and the prevalence of shear failure of hangingwall rock bolt units. These mechanisms are not considered in the current design procedures. The purpose of this study was to improve the mechanistic understanding of this interaction between the rock mass and the support system to improve the design considerations in these environments.

The erratic nature of the damage to tunnel excavations with respect to the source of the causative seismic event is indicative of the role that site response has in understanding the failure mechanisms. The main factors that influence the type and severity of damage are:

- the structure and competence of the rock mass prior to the seismic event.
- the previous history of deformation of the support system and thus its remaining capacity immediately prior to the seismic event.
- the orientation and proximity of the excavation relative to the source of the seismic event.

The containment of potential rockburst damage will involve the ability to define high risk excavations with a reasonable degree of certainty and also the implementation of effective excavation and support design strategies. The design of these support systems will require the estimation of the volume of unstable rock mass and the mechanisms of rock mass deformation. In addition these aspects of design will require due consideration of the relative cost of these strategies compared to the identified level of risk and the limits of these design considerations.

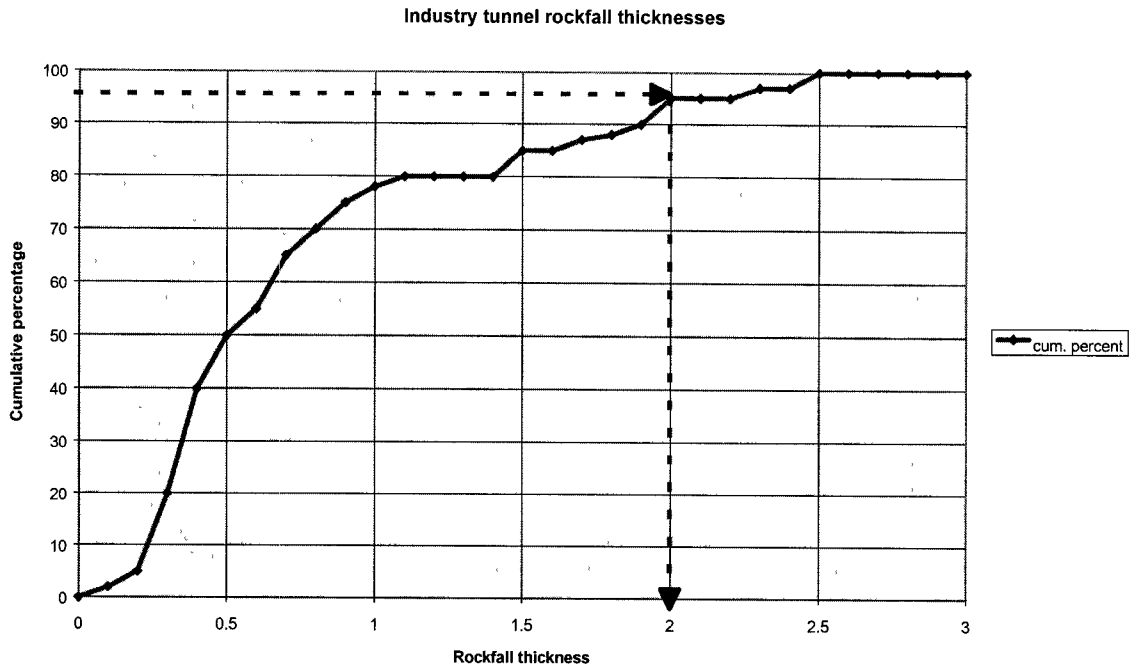
### **3.3 Definition of the depth of rock mass instability (stress induced failure)**

The work in this section of the investigation addresses aspects of enabling output 3 (Determination and classification of initiation of stress induced rock mass failure) and output 6 (Influence of excavation shape and size on stabilisation mechanisms).

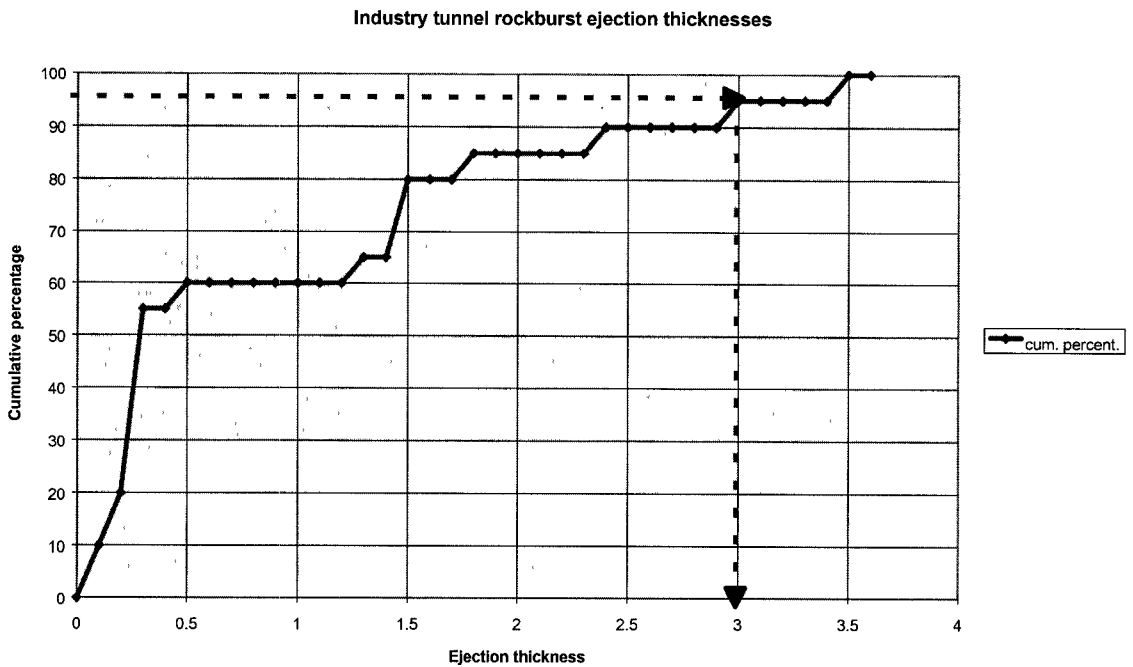
In situ monitoring and historical data analysis of the behaviour of the rock mass around tunnel excavations was conducted to quantify the rock mass characteristics. In addition, previous case studies within the South African mining environment were re-evaluated to investigate the mechanisms of deformation and the influence of support systems.

#### **3.3.1 Accident data base evaluation of depth of instability**

A primary consideration of support design is the volume of unstable rock mass that must be reinforced and supported. In the current design guidelines (Anon. 1996), the depth of instability of the rock mass is determined from analysis of an appropriate accident database. Examples of the data sets, for rockfalls and rockbursts for the South African gold mining industry, are shown in Figures 3-1 and 3-2. This data has been restricted to off reef excavations of dimensions of less than 3.5 m. The analysis on the graphs indicates the 95 % confidence limit of maximum unstable height. This is generally accepted as the minimum limit for support design purposes in South Africa.



**Figure 3-1. Analysis of tunnel fall of ground data for the South African gold mining industry.**



**Figure 3-2. Analysis of tunnel rockburst ejection thickness data for the South African gold mining industry.**

The design procedure utilising this data is to ensure that the length of the rock bolt, excluding the required bond / anchorage length, is greater than the height of instability.

However, it must be remembered that the use of the data set implicitly includes all other factors that may influence the depth of failure. These will include the implemented support system, rock type, stress environment, magnitude of the seismic event, etc. If the data set is limited to the mine of application then some factors may be considered constant, such as rock mass type,



and thus are directly applicable. However, the data will still be influenced by the presence of the support systems, unless the data is restricted to where the depth of failure is substantially greater than the reinforcement length. Thus, for the 95 % confidence limit indicated in Figures 3-1 and 3-2, for rockfall and rockburst accident data respectively, the average unstable depths for the South African gold mining industry are 1.8 m and 3 m respectively.

The length of tunnel reinforcement units, as typically used in the South African mining industry, is between 1.8 m and 3 m. Thus, most of the unstable height collapses in the data sets are within the length of the typical reinforcement units. The assumption must therefore be that a substantial number of these may have fallen between the tendons, where the peripheral rock mass is more discontinuous. In these cases the reinforcement may still act to stabilise the deeper rock mass and thus, in the absence of any reinforcement, the depth of instability would have been substantially greater. Where collapse in excess of typical tendon lengths has occurred, then it may be considered that this would represent the natural depth of instability for the given rock mass environment.

### 3.3.2 Case study evaluation of depth of instability

Work conducted by Martin *et al* (1997) investigated the depth of breakout for tunnels where the mode of failure of the rock mass was stress induced fracturing, and the failure process was not constrained by rock mass reinforcement or support. A large proportion of the case studies utilised by Martin were based on South African mine tunnels.

Under high stress environments, typical of South African gold mining conditions, the mechanism of rock mass failure around an excavation is principally stress induced fracturing. In this environment it is considered that the original rock mass structure is of lesser importance in determining the ultimate extent of instability (Jager, Wojno and Henderson, 1990). The initiation and development of the fracture zone is thus a function of the ratio between the induced tangential stress at the excavation boundary and the rock strength. This formulation is consistent with that proposed by Wiseman (1979), and incorporated in the Rockwall Condition Factor criterion (Anon., 1988).

$$RCF = \frac{3 \cdot \sigma_1 - \sigma_3}{F \cdot \sigma_c} \quad (3-1)$$

where:  $\sigma_1$  is the maximum principle stress in the plane of analysis

$\sigma_3$  is the minimum principle stress in the plane of analysis

F is a factor between 0.1 and 1, dependent on the rock mass characteristics and excavation size used to downgrade the intact rock strength

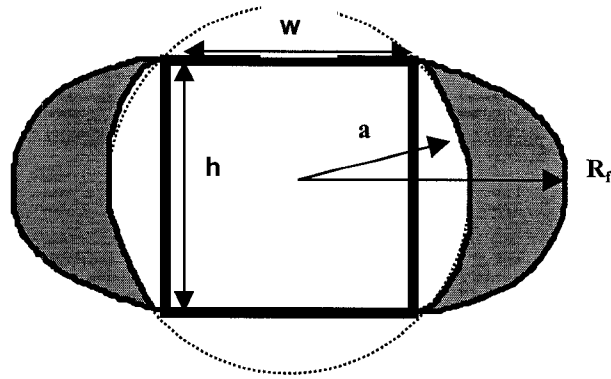
$\sigma_c$  is the uniaxial compressive strength of the intact rock from laboratory testing.

The numerator of equation 3-1 represents an approximation of the stress concentration on the rockwall of the excavation and the denominator an estimation of the in situ rock mass strength.

The relationship as utilised by Kaiser *et al* (1996) also evaluates the influence of the induced excavation boundary stress in relation to the rock strength on the depth of failure as a function of the excavation size. For a comparison of excavations of different shapes, but of similar size, the dimension of the excavation is simplified to an equivalent excavation dimension. This is represented by the radius of an encompassing circle, and is considered valid for excavations of  $w/h < 2$ . The equivalent excavation dimension (a) is given as

$$a = \frac{h \text{ (or } w)}{\sqrt{2}} \quad (3-2)$$

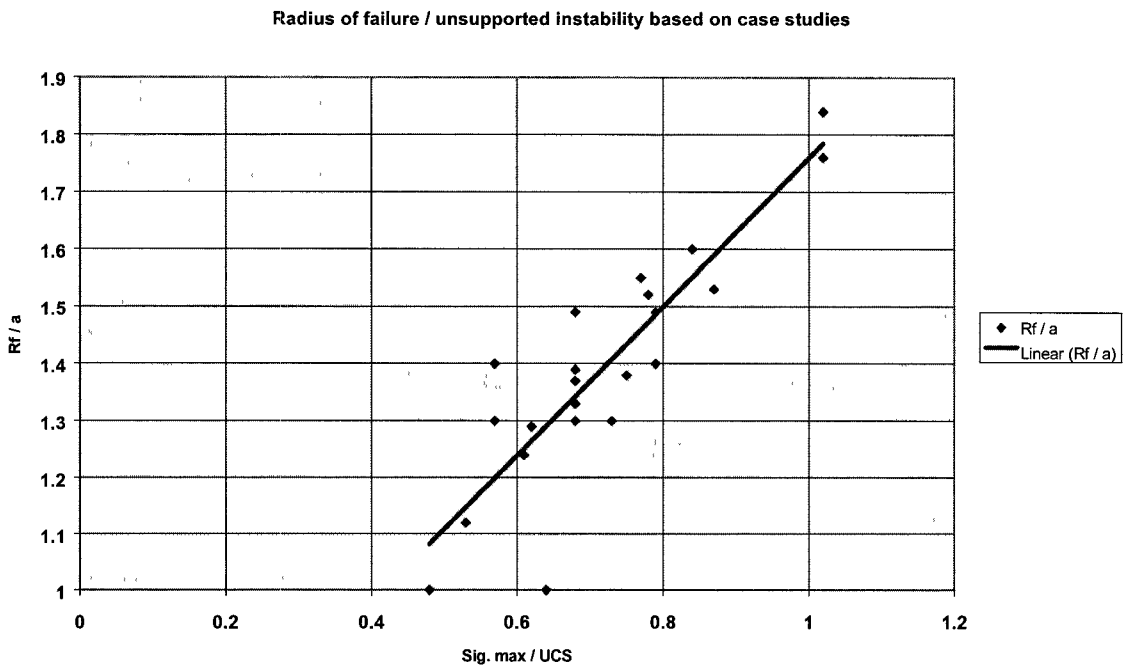
This is represented in Figure 3-3.



**Figure 3-3. Definition of equivalent excavation dimension and depth of instability (Kaiser et al. 1996)**

Figure 3-3 indicates the depth of instability  $R_f$  due to stress induced fracturing around an excavation.

The relationship between the stress level and depth of instability as derived by Martin is shown in Figure 3-4. This shows the data of case studies used to derive the relationship between the ratios of induced tangential stress to rock strength, and depth of instability to equivalent excavation dimension.



**Figure 3-4. Relationship between induced stress and depth of instability after Martin et al (1997).**

This relationship may be expressed as:

$$R_f/a = 1.34 \sigma_{max} / \sigma_c + 0.43 \quad (3-3)$$

where  $\sigma_{max} = 3\sigma_1 - \sigma_3$  (3-4)

Analysis of Martin's data indicates the onset of significant fracturing to commence at a stress level of 43 % of the rock strength. This is consistent with other authors (Wagner, 1983).

Kaiser *et al* (1996) have also examined the theoretical influence of dynamic loading, as a function of seismic activity in the vicinity of the excavation, on the extent of rock mass damage and instability of the excavation. The incidence of a shear wave associated with a seismic event will result in a peak dynamic stress of:

$$\sigma^d = +/- c_s \rho ppv_s \quad (3-5)$$

where:  $\sigma^d$  = the induced dynamic stress in the rock mass  
 $c_s$  = the propagation speed of the shear wave  
 $\rho$  = the rock mass density  
 $ppv_s$  = the peak particle velocity of the shear wave at the point of analysis.

The waveform of a seismic event may be divided into a primary compressional wave (p-wave) and a secondary shear wave (s-wave) if the rock mass is assumed to be an isotropic elastic medium. Most of the seismic energy is associated with the s-wave component of an event. At the point of analysis the ground motion velocity of the shear wave is utilised as this is generally substantially greater than that of the p-wave. This dynamic stress is additive to the in situ stress state and thus alters the in situ stresses according to its magnitude and direction relative to the principle stress directions. This dynamic stress is also concentrated in the vicinity of an excavation as are the static field stresses. For an excavation of circular geometry, this stress concentration factor is given by:

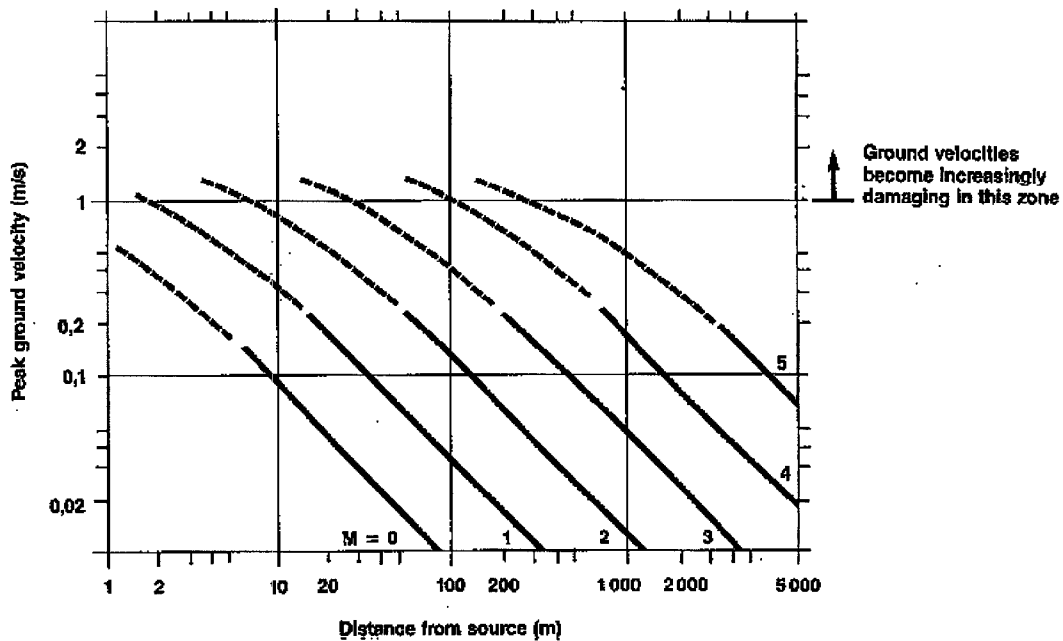
$$n = 4\cos^2\theta \quad (\text{Kaiser } et \text{ al, } 1996) \quad (3-6)$$

Where:  $n$  = dynamic stress concentration factor  
 $\theta$  = angle of incidence between the seismic wave and the maximum principal stress direction.

For design purposes it is considered that a value of  $n=4$  should be taken. This would represent the worst case scenario when the angle of incidence between the dynamic shear wave and the maximum principal stress direction is  $0^\circ$ . Under these conditions the maximum dynamic stress concentration in the periphery of the excavation is given by:

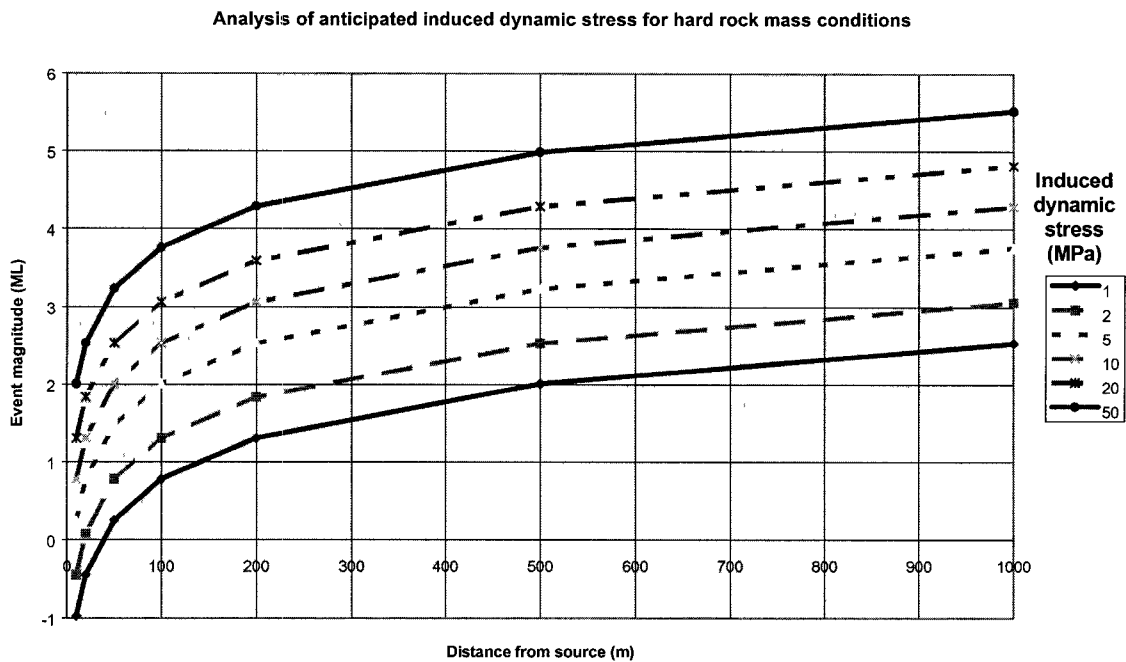
$$\sigma_{max} = 3.\sigma_1 - \sigma_3 + 4 c_s \rho ppv_s \quad (3-7)$$

The value for  $c_s$  is a function of the rock mass density and elastic constants and may be taken as approximately 3650 m/s for intact hard rock mining environments. The shear wave peak particle velocity ( $ppv_s$ ) will be a function of the magnitude of the seismic event and the distance from source. This may be estimated from charts such as that shown in Figure 3-5.



**Figure 3-5. Ground velocity with distance from seismic event (Anon., 1988).**

Substitution of data derived from Figure 3-5 into equation 3-7 yields a relationship between the parameters of a seismic event and the anticipated maximum induced dynamic stress on the boundary of an equivalent circular excavation (Figure 3-6).

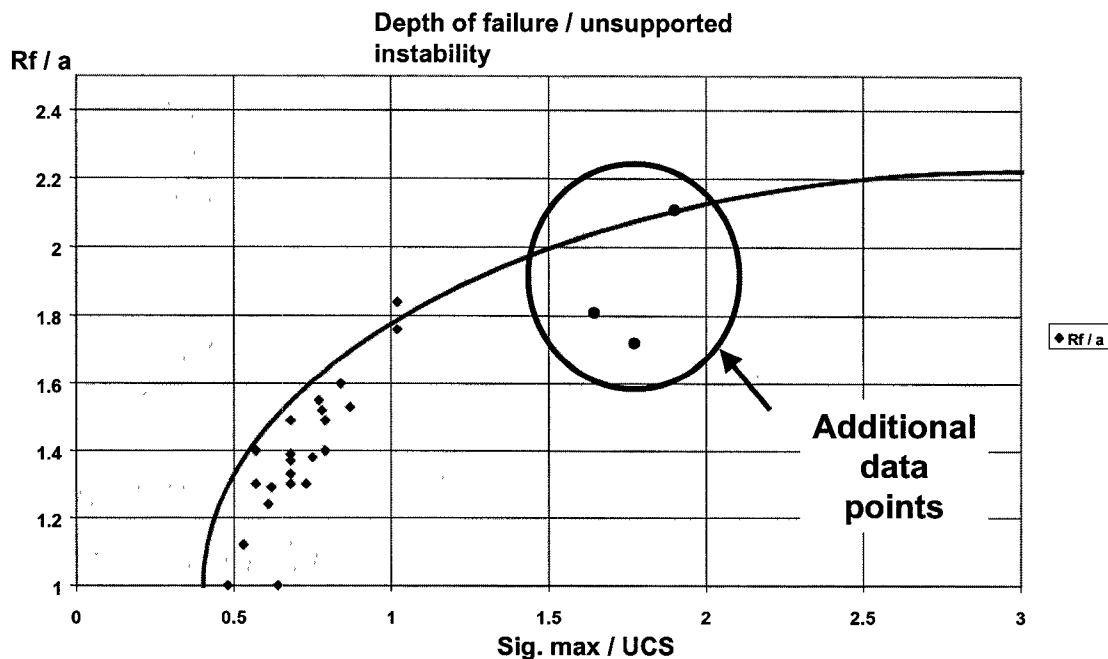


**Figure 3-6. Determination of anticipated maximum induced dynamic stress due to a defined seismic event.**

Data from Figure 3-6 is added to the static stress concentration to estimate the additional depth of instability as a function of the induced dynamic stress due to a seismic event

Analysis of the empirical range of Martin's data indicated that a large number of tunnels in the South African mining industry would be sited in environments with substantially higher induced stress levels. An investigation was conducted to increase this data set to capture the

unsupported depth of instability at these higher stress levels (Haile, 1998). The availability of applicable data was limited, and, thus, suitable caution should be applied to the extrapolation of the relationship as indicated in Figure 3-7.



**Figure 3-7. Revised relationship of depth of instability for additional South African case studies.**

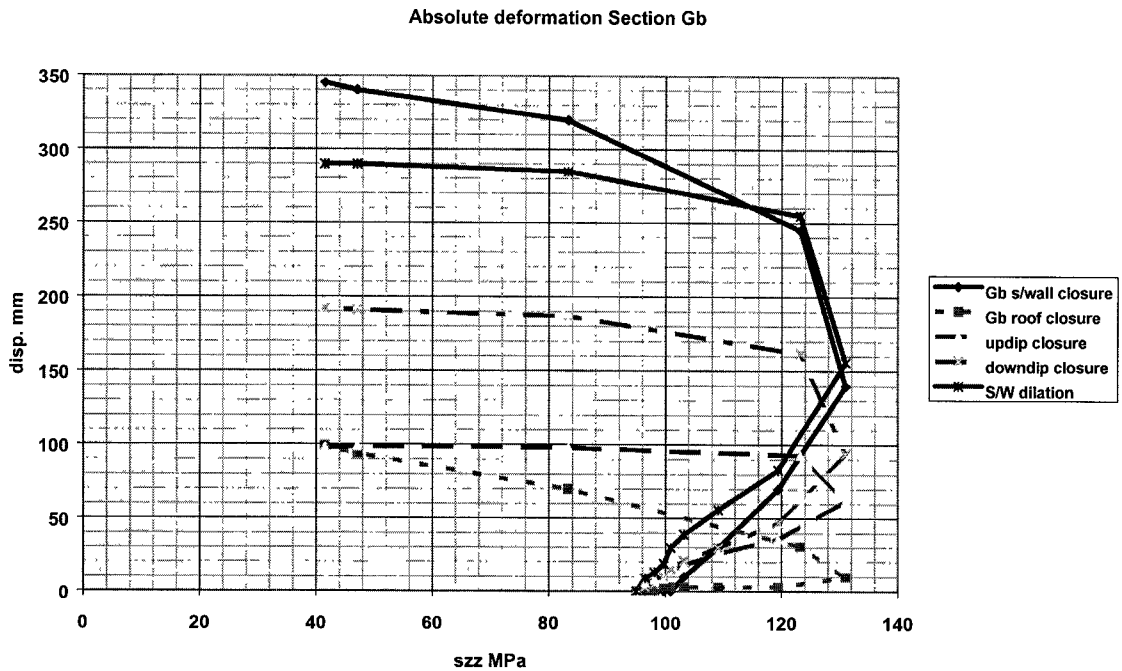
The tendency for a maximum upper limit of depth of instability is indicated in Figure 3-7 by the “design curve” which encompasses the majority of data points. This illustrates the observed tendency for an excavation to attain a stable profile. This may be as a function of the rock mass structure, or the gradual restriction in the area of maximum tangential stress and active rock mass fracturing. Data on excavation stability at these high stress levels is very limited and thus suitable regard must be given to the degree of confidence in the design relationship. However, it is considered that this relationship is more appropriate for the design of rock mass reinforcement systems in the high stress environments as experienced in many of the deep level South African gold mines.

### 3.4 Definition of stress driven rock mass behaviour

The work in this section of the investigation addresses aspects of enabling output 4 (Classification, behaviour and limits of stability of a fractured rock mass around tunnel excavations), output 7 (Determination of interaction between support and the rock mass) and output 9 (Influence of stress change on excavation stability).

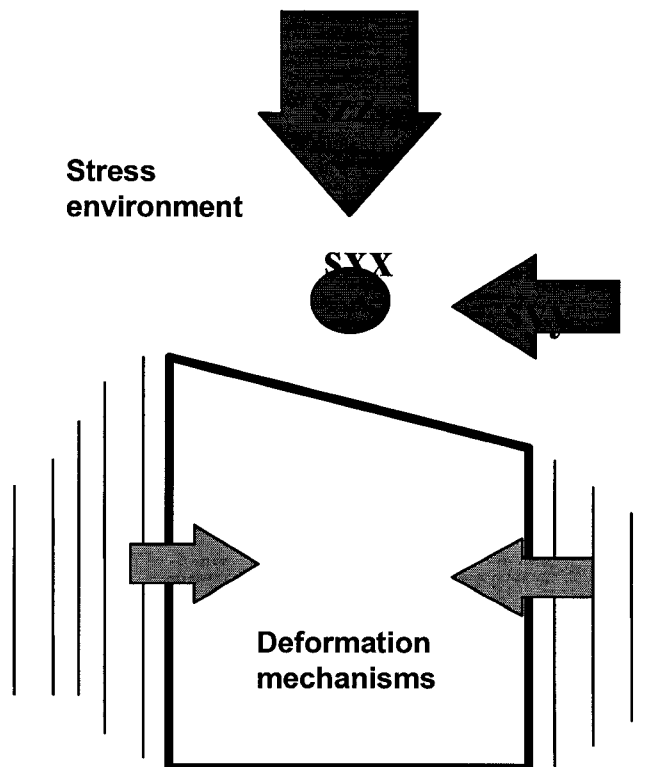
#### 3.4.1 Rock mass deformation mechanisms

The stress changes over the life of the excavation will influence the deformation of the rock mass in the vicinity of the tunnel. The support system implemented must be able to control this deformation to maintain the tunnel in an operational condition. In situ instrumentation and the evaluation of previous case studies were initiated under this investigation to determine the mechanisms of rock mass deformation due to stress change and the influence of support systems on the deformation characteristics. Case studies were examined from Buffelsfontein (Haile, 1998) and Hartebeestfontein (Sevume, 1998) gold mines. An example of the deformation history for a tunnel section from the Buffelsfontein case study is shown in Figure 3-8. This figure indicates the deformation of the sidewall and hangingwall with respect to vertical stress change.



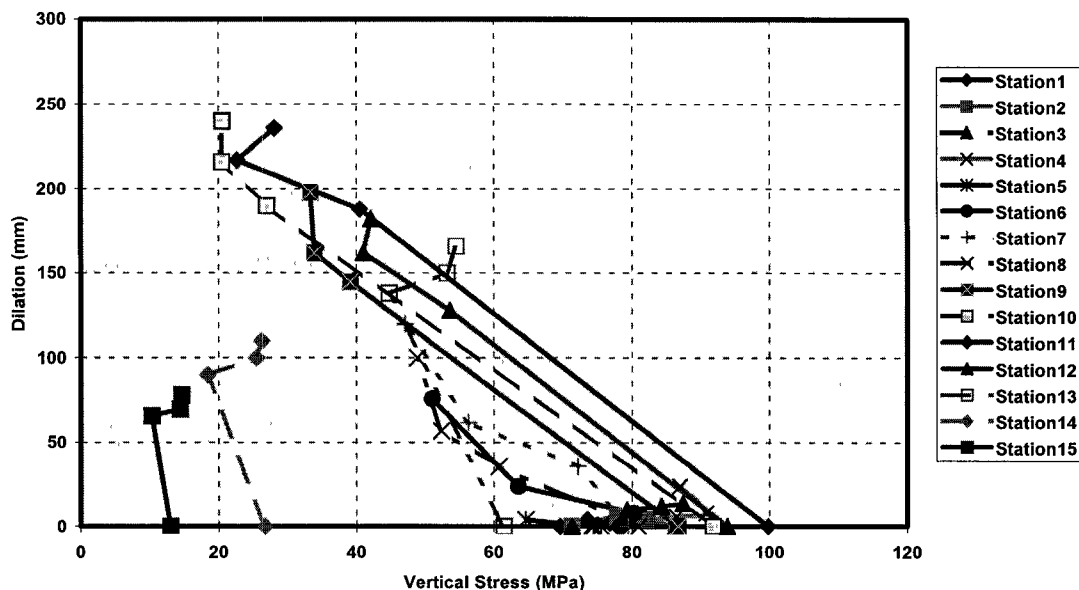
**Figure 3-8. History of deformation with vertical stress change from Buffelsfontein case study (Haile 1998)**

Within the sidewall of the tunnel most of the deformation is a result of vertical stress increase although continued deformation of the sidewall occurs under vertical stress reduction. The mechanism of sidewall deformation under vertical stress increase may be attributed to the generation of new fracturing with increased load and continued deformation in the fracture zone due to shearing and dilation of blocks (Figure 3-9).



**Figure 3-9. Analysis of deformation mechanisms with vertical stress increase.**

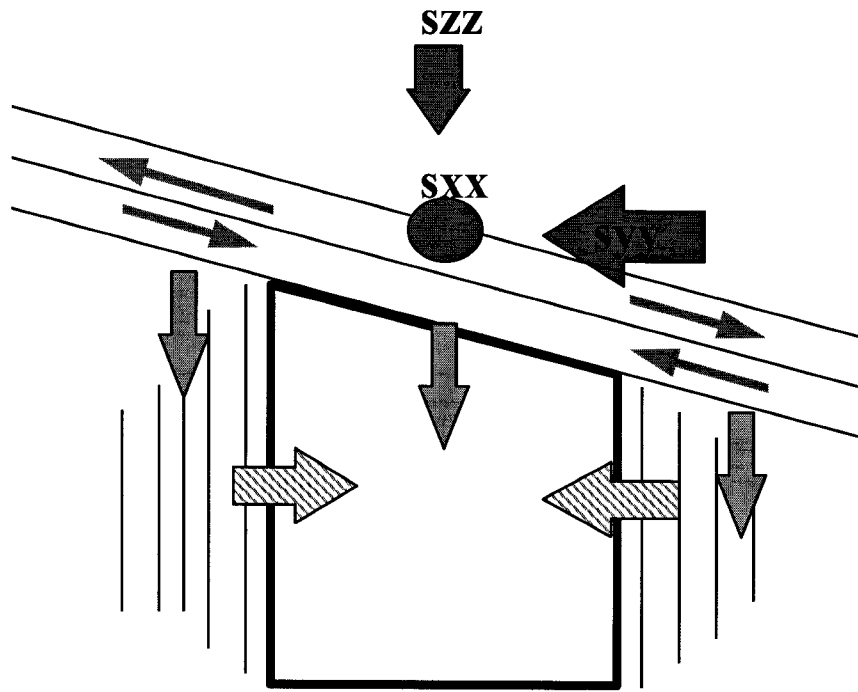
Of interest from the case study is the significant proportion of deformation of the hangingwall under vertical stress reduction. This was also reflected in the Hartebeestfontein case study (Figure 3-10) where the majority of instrumentation stations analysed were only subject to vertical stress reduction.



**Figure 3-10. History of hangingwall deformation with vertical stress reduction from Hartebeestfontein case study.**

It is proposed that the mechanism of hangingwall deformation is a result of the increased freedom for shear deformation on sub-horizontal planes within the hangingwall of the excavation due to a relative reduction in vertical clamping (Figure 3-11). The relative reduction in vertical clamping may be a function of either vertical stress reduction or an increase in the horizontal stress component. The propensity for shear deformation in the hangingwall of a tunnel has been identified by Hepworth (1985), Laas (1995) and by observation under this investigation (section 3.2). This shear deformation will result in dilation of the shear planes and cause deformation of the hangingwall rock mass into the tunnel excavation.

It is also proposed that the dilation of the hangingwall rock mass will result in additional loading of the fractured sidewall rock mass, which, in the immediate skin of the tunnel, will be under relatively low confinement. This vertical closure will cause shearing between the blocks within the sidewall fracture zone with associated dilation of the rock mass. Due to the already disturbed nature of the sidewall rock mass from the initial fracturing process, this further deformation will result in relatively lower sidewall dilation. It has also been noted from the case studies that deformation of the 'down-dip' sidewall is generally greater than that of the 'up-dip' sidewall, although usually considered to be within experimental error and thus not significant. A mechanistic evaluation based on the above proposed deformation mechanisms may indicate that the typically lower 'down-dip' sidewall height may experience increased vertical strain under a given hangingwall dilation and thus increased sidewall dilation.



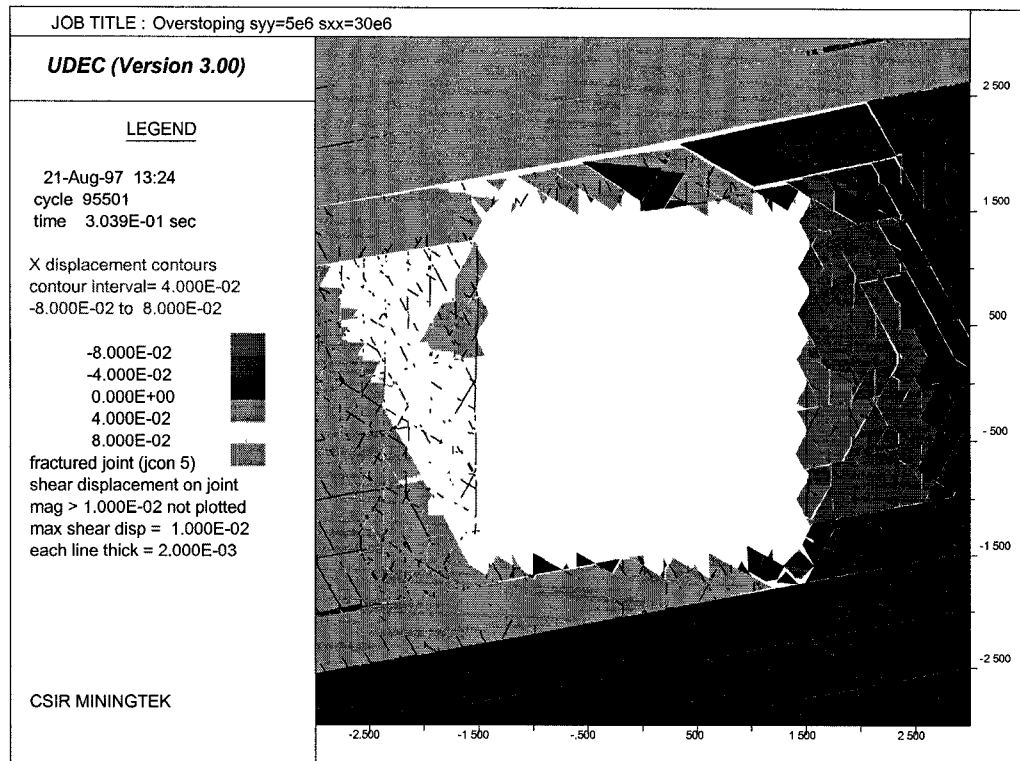
**Figure 3-11. Proposed mechanism of hangingwall and sidewall deformation due to vertical stress reduction.**

The deformation mechanisms as described above may also be associated with dynamic loading due to seismic events, where the transient stress waves may result in reduced effective vertical stress or increased horizontal stress components.

In order to verify the proposed deformation mechanisms, numerical modelling was conducted (Haile, 1998) utilising the distinct element code UDEC (Itasca 1997). This modelling allowed the creation of distinct blocks due to failure of contact conditions between blocks and the evaluation of subsequent discrete block deformation mechanisms. The model was subjected to a simulated vertical stress increase and a subsequent vertical stress decrease. The magnitudes of the stress changes and characteristics of the rock mass were based on the conditions from the Buffelsfontein case study. No support elements were modelled as the purpose of the modelling was to observe the mechanisms of deformation and not evaluate the potential influence of the support in limiting the deformation.

The modelling, an example of which is shown in Figure 3-12, clearly indicated the large scale deformation of the sidewall under vertical stress increase, and the initiation of significant hangingwall deformation to be associated with vertical stress reduction. Figure 3-12 illustrates the large scale block movement in the hangingwall where the x-direction displacement contours illustrate the relative shear deformation in the hangingwall. The indicated sidewall deformation is a function of the 'failure' of the sidewall rock mass under the initial high vertical stress field.





**Figure 3-12. UDEC model of rock mass deformation around a square tunnel under vertical stress reduction.**

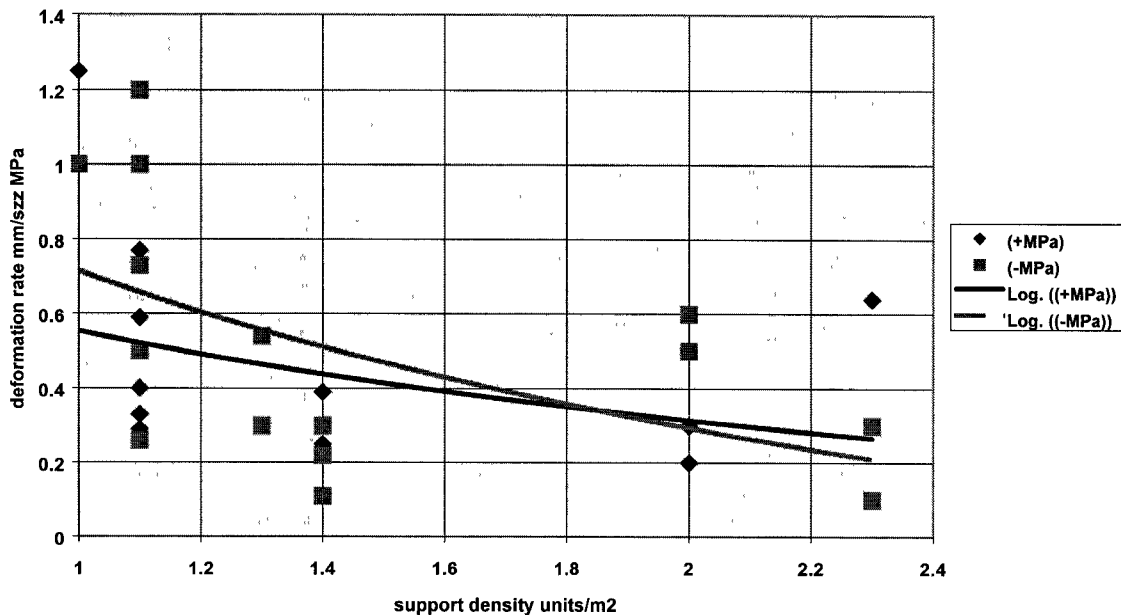
It is considered that an understanding of the mechanisms of rock mass deformation around a tunnel is fundamental to the selection of a rock bolt reinforcement system with suitable characteristics. Of significance from this investigation is the understanding of the deformation of the hangingwall under vertical stress reduction. In addition, the proposed mechanism of shear displacement will have important implications on the characteristics of the rock bolt reinforcement under these loading conditions. The importance of the consideration of shear deformation on the performance of the rock bolt reinforcement was also shown under investigations of rockburst damage and it is proposed that the same deformation mechanisms are applicable.

### **3.4.2 Influence of rock bolt reinforcement on magnitude of deformation.**

The influence of rock mass reinforcement on the deformation of the rock mass was also evaluated. This work was based on the further evaluation of the Buffelsfontein case study (Haile 1998), where the tunnel was supported by different support systems (Hepworth 1985). The evaluation as conducted by Haile (1998) considered the influence of the support resistance, shear resistance and support density on the control of the rock mass deformation. As may be anticipated from an in situ evaluation, a high degree of variability in the data was observed and due regard to this variability should be given in analysis of the data. However, for ease of illustration, general trend lines are shown in the following analyses based on best fit relationships.

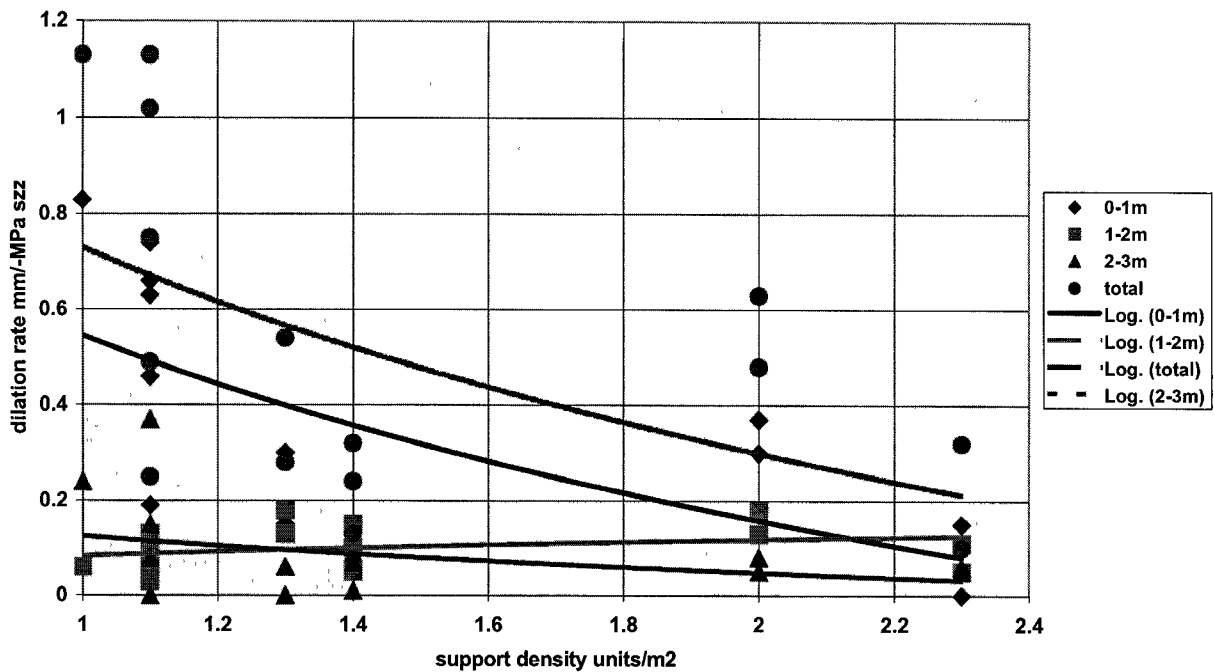
The analysis of the influence of the rock bolt reinforcement on rock mass deformation is based on the density of the rock bolt installation. This parameter was found to best correlate to the control of rock mass deformation, encompassed in both the support resistance and shear resistance parameters of the rock bolt reinforcement system. The response of the rock mass is measured by the anticipated deformation of the tunnel boundary per unit change in vertical

stress component. For simplicity, and based on the relative influence from the case studies, the vertical stress component is taken as the primary parameter. The relationship between the vertical and horizontal stress components is considered to adequately capture the magnitude and inclination of the maximum principal stress in the plane of analysis. The relationship between hangingwall rock bolt density and vertical stress change is shown in Figure 3-13 for vertical stress increase and decrease. The rate of deformation under relative stress increase or decrease is comparable. The previous analysis of deformation of the hangingwall under vertical stress reduction may be attributed to the substantially greater stress change due to overstepping. Figure 3-13 may be used to estimate the overall closure of the tunnel hangingwall over the life of the excavation based on an estimation of its stress path.



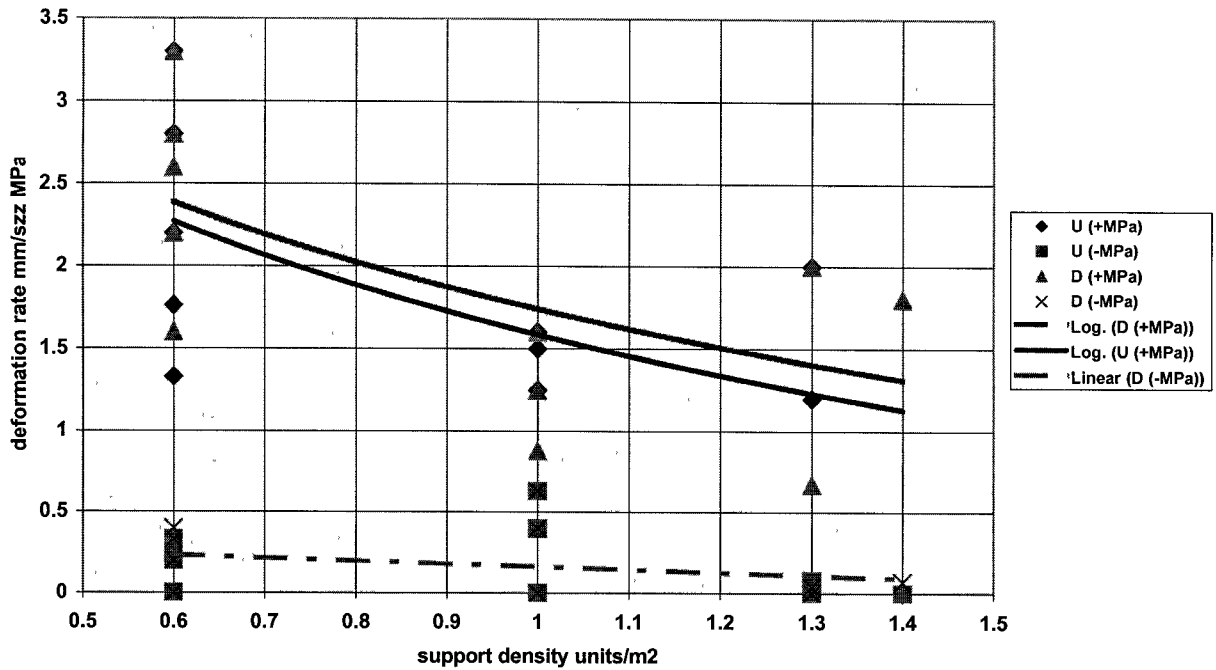
**Figure 3-13. Influence of rock bolt density of hangingwall deformation from Buffelsfontein case study (Haile 1998)**

For the design of the yield capacity of the rock bolt reinforcement system, the support design engineer will require an estimation of the dilation of the rock mass in the skin of the excavation. Figure 3-14 examines the dilation of the rock mass in the hangingwall of the tunnel with distance into the rock mass from the skin of the tunnel. It is evident from this case study that most of the significant deformation is restricted to the first metre of hangingwall rock mass. The density of the rock bolt reinforcement also influences the deformation of this rock mass volume. In excess of 1 m depth there is no discernible influence of the rock bolt reinforcement on the rock mass behaviour, although the typical rock bolt length was approximately 2 m. Deformation of the rock mass in excess of a depth of 1 m is thus considered to be solely a function of the rock mass characteristic.



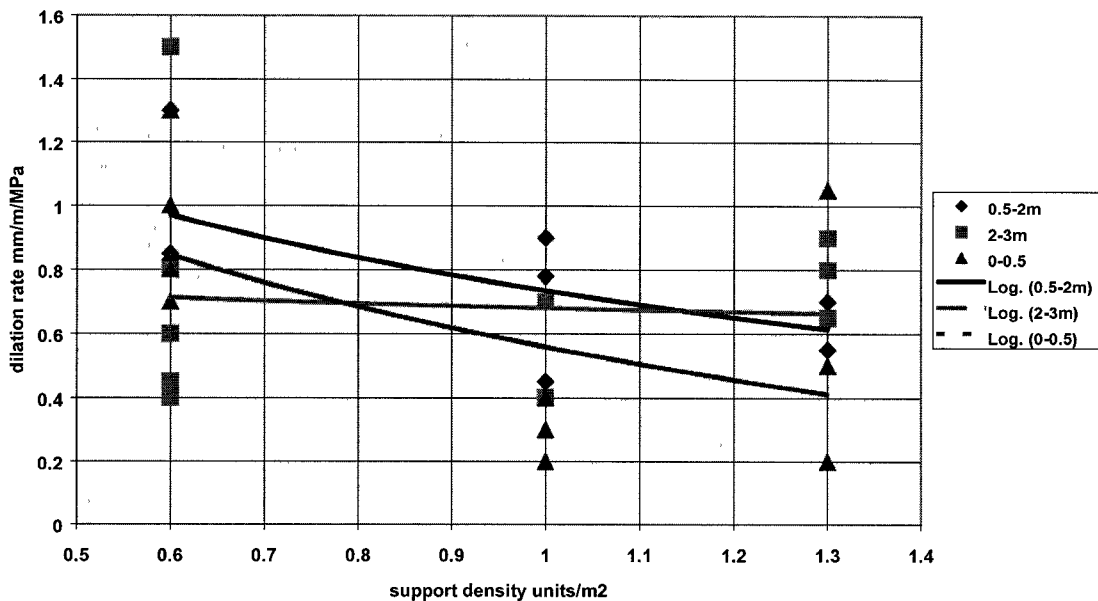
**Figure 3-14. Dilation of hangingwall rock mass with depth into the hangingwall and the influence of rock bolt reinforcement density.**

Analysis of the influence of the rock bolt reinforcement on the deformation of the sidewall of the case study tunnel is shown in Figure 3-15. A similar relationship of decreasing deformation with increasing support density is evident, particularly for an environment of vertical stress increase. However, under vertical stress reduction there is a less well defined influence. The magnitude of deformation with unit increase in vertical stress, of the order of 2 mm/MPa, is much larger than that for the hangingwall rock mass as shown in Figure 3-13. The deformation of the sidewall rock mass under vertical stress reduction is only of the order of 0.2 mm/MPa. This deformation is an order of magnitude decrease in deformation compared to that under vertical stress increase, and also substantially lower than the comparable hangingwall deformation. Analyses of the 'up-dip' and 'down-dip' sidewall deformations show the same deformation characteristics but a slightly greater 'down-dip' deformation magnitude. This observation has been mechanistically described in section 3.4.1.



**Figure 3-15. The influence of support density on sidewall deformation for updip (U) and downdip (D) sidewalls under vertical stress increase and decrease.**

Analysis of the distribution of deformation within the sidewall rock mass, as shown in Figure 3-16, shows the reinforcing action of the rock bolt units in the immediate sidewall, to a depth of approximately 2 m (for vertical stress increase). In excess of this depth, which is approximately the rock bolt length, there appears to be no influence of the increasing support density on the deformation of the rock mass.

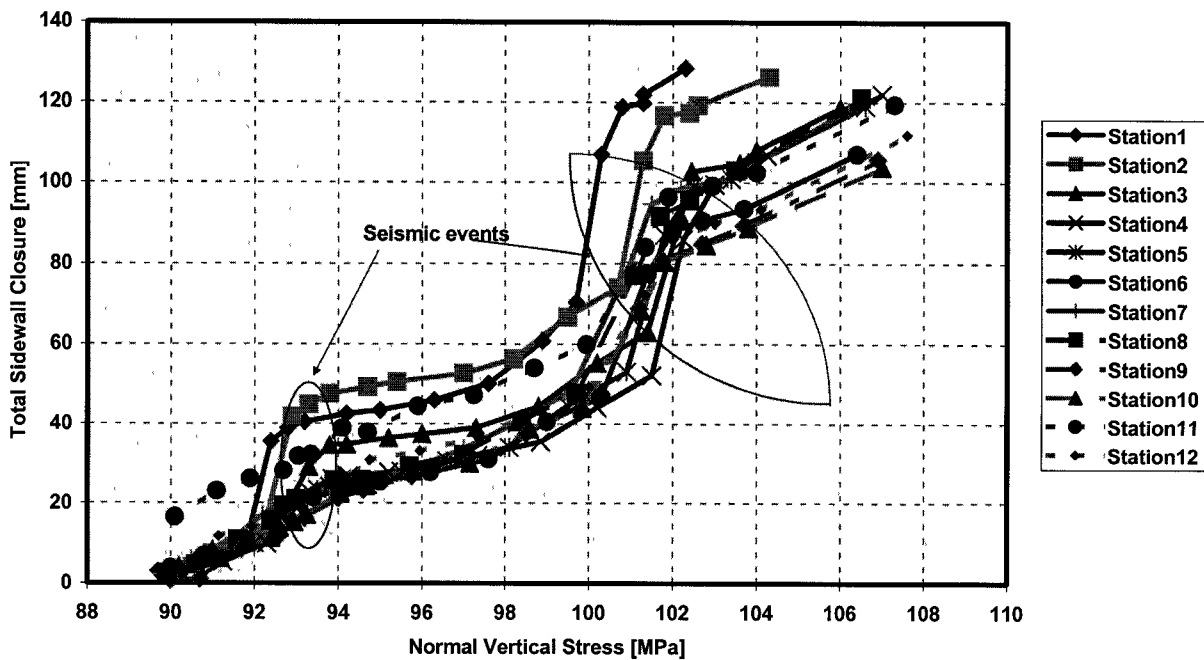


**Figure 3-16. The influence of support density on sidewall dilation with depth into rockwall.**

Figures 3-15 and 3-16 can again be used to estimate the total deformation of the sidewall of a tunnel and the distribution of dilation within the sidewall over the life of the tunnel. This will

allow the quasi static yield capability of the rock bolt reinforcement to be estimated to ensure the operational safety of the tunnel.

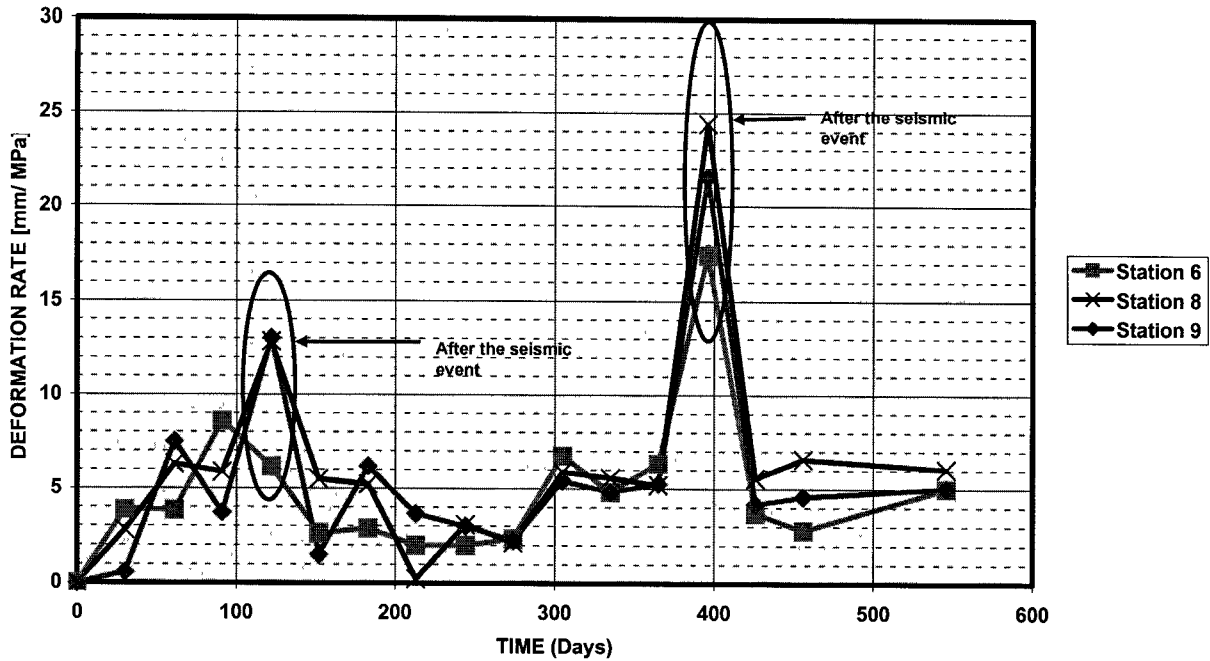
As part of this investigation a comprehensively instrumented site was established in a tunnel at a depth of approximately 2300 m below surface in Elsburg quartzites at Kloof gold mine (Sevume, 1998). This site was subjected to a gradual stress increase due to stoping activities in the vicinity, and periodic dynamic loading from distant seismic events. The site was planned to be overstoped but the stoping was terminated short of the instrumented section. Support in this area consisted of 2.2 m, 16 mm rebar rock bolts on a 1.5 m diamond pattern. The motivation for this additional monitoring programme was based on the requirement to develop a detailed understanding of the distribution of deformation around the tunnel. Historically, instrumentation of tunnels consisted of extensometers located at the mid-point of each sidewall and the centreline of the hangingwall. This configuration measured what was considered to be the maximum anticipated deformation and extent of damage to the rock mass. However, the position of the extensometers with respect to support elements was not recorded. Work carried out in this project has shown this to be an important influence on total deformation. The distribution of deformation was thus roughly extrapolated between these measuring points. This investigation implemented an intensive array of extensometers to develop a 'full' picture of the deformation around the tunnel. An array of closure pegs was also implemented over a distance of approximately 30 m along the length of the tunnel to measure the total closure of the sidewall as shown in Figure 3-17.



**Figure 3-17. Measurements of total closure of the sidewalls at the Kloof site.**

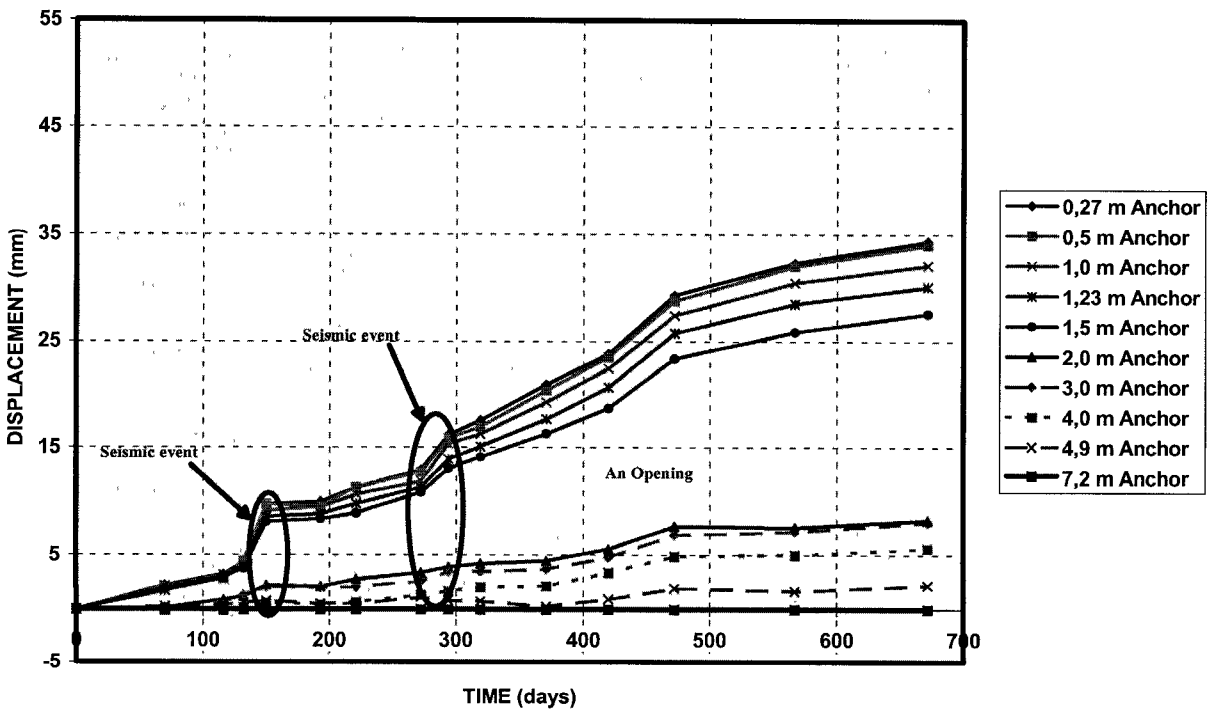
From Figure 3-17 it is noted that, as stoping progresses, the rate of deformation of the sidewall is approximately 5 mm/MPa. This is comparable to the deformation rate from the Hartebeestfontein case study (Sevume, 1998) and is approximately double that as determined from the Buffelsfontein case study (Haile, 1998). Of note in Figure 3-17 are the 'jumps' in deformation associated with seismic events, the first of which was a magnitude M=1 at a distance of approximately 130 m from the site and the second of which was a magnitude M=2.5 at a distance of approximately 200 m. These seismic events would result in theoretical ground velocities (no amplification) of approximately 0.03 m/s and 0.13 m/s respectively. The second seismic event was associated with physical damage of a shakedown nature at the experimental site.

Figure 3-18 shows the relatively consistent deformation rate with stress increase, apart from the periods of seismic activity.



**Figure 3-18. Closure rate (mm/MPa) with time at Kloof experimental site.**

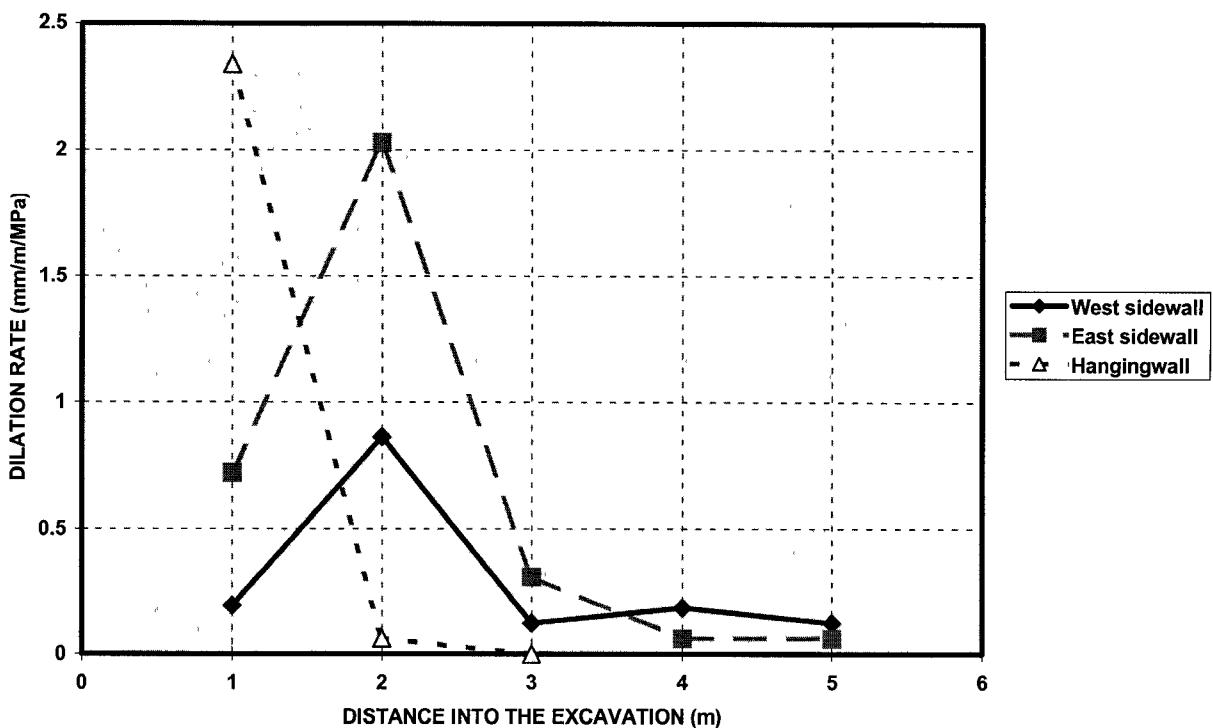
More detailed information on the distribution of deformation within the rock mass around the tunnel was determined from the extensometer arrays. The sonic probe extensometer was utilised at this site to obtain an increased density of measuring points within a measuring hole. Figure 3-19 illustrates the dilation profile of the rock mass in the 'up-dip' sidewall of the tunnel.



**Figure 3-19. Extensometer results for 'up-dip' sidewall at Kloof site.**

Of significance at this site is the 'opening' within the rock mass between 1.5 m and 2 m. This is comparable to the length of the rock bolt reinforcement. At depths less than this, the rock bolt reinforcement is seen to reinforce and confine the rock mass as indicated by the limited dilation over this range. This reinforced shell therefore acts as a structural element as shown from the limited dilation, relative to the deeper rock mass during the incidence of seismic events. If the integrity of the structure had been compromised it would be anticipated that differential deformations within the reinforced rock mass would occur. The response of the reinforced rock mass under the dynamic loading associated with the seismic events is also greater than that of the deeper, un-reinforced rock mass. This again illustrates the structural characteristic of the reinforced rock mass shell. It was also observed that the deformation response to the seismic events was that the sidewall closest to the source of the seismic event is approximately three times that of the sidewall in the 'shadow' (Sevume, 1998). This explains the relatively smaller response of the 'up-dip' west sidewall to the second M=2.5 magnitude event, which occurred to the east of the tunnel, compared to the first, M=1 magnitude event which occurred approximately to the north east of the site (Figure 3-19).

Analysis of the average dilation rates within the sidewalls and hangingwall of the tunnel is shown in Figure 3-20. The very high deformation associated with the hangingwall at this site was due to block instability within the first 0.5 m, and is considered anomalous to the generally anticipated dilation of the rock mass. The reinforcing action of the rock bolts on the sidewall is illustrated by the relatively lower dilation in the immediate tunnel skin. The higher dilation rates associated with the 'down-dip' eastern sidewall is considered to be a function of the relative effectiveness of the rock bolts due to the installation orientation relative to the rock mass bedding. In the up-dip sidewall the typical orientation of rock bolts in the lower portion of the sidewall will be perpendicular to the bedding, and thus be more effective in containing deformation in this section of the tunnel profile (Figure 3-21).

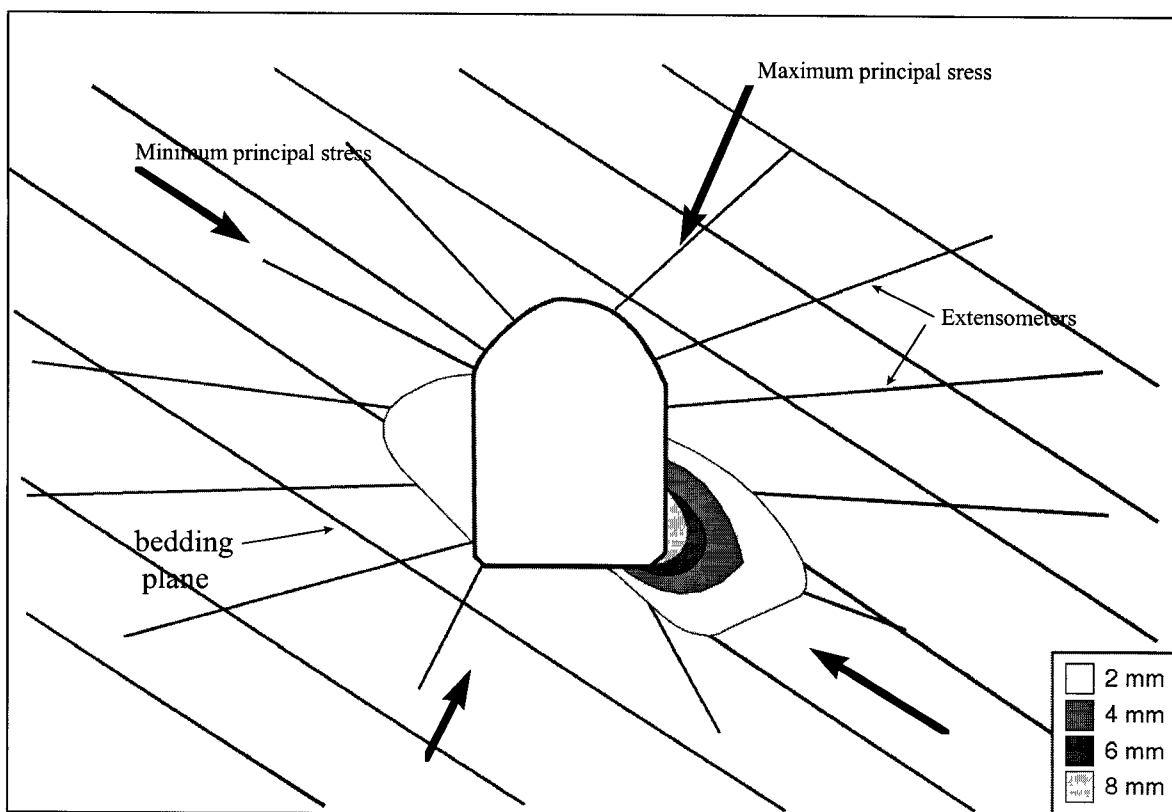


**Figure 3-20. Rock mass dilation with depth for Kloof instrumentation site.**

The compilation of the extensometer data was used to formulate a detailed picture of the deformation around the tunnel at the Kloof site. The development of deformation over time, and with increasing stress, is shown in Figure 3-21 and 3-22. These figures illustrate the general orientation of the rock mass structure and the direction of the principal stresses. These figures illustrate as expected that, under this environment, the development of deformation

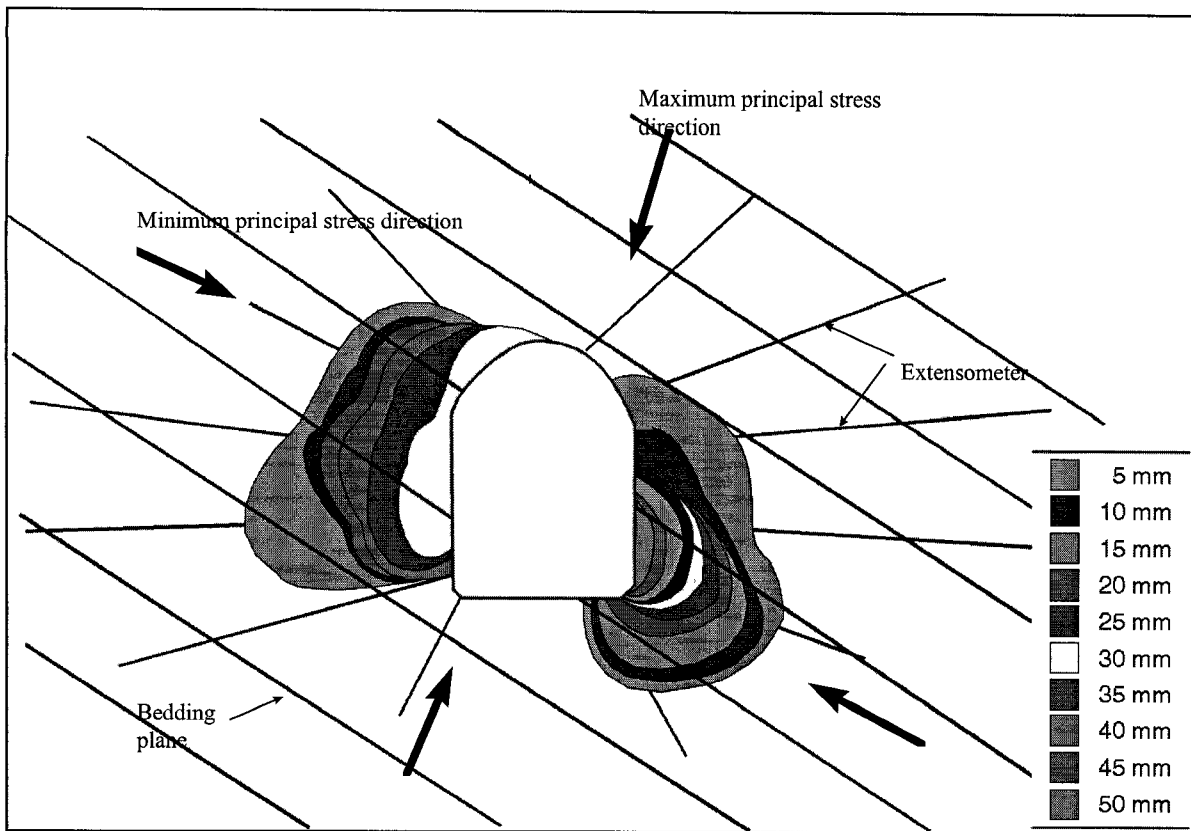
around the tunnel is orientated perpendicular to the maximum principal stress. The extent of the zone of deformation is also controlled by the bedding structure, being greater parallel to the bedding. It is considered that the bedding planes enhance the freedom for deformation of the fractured rock mass and will thus contribute to the localisation of deformation.

The localised nature of the deformation associated with the eastern, down-dip sidewall is believed to be due also to the effectiveness of the rock bolt reinforcement installation. Deformation associated with the up-dip sidewall is far more uniform over the whole sidewall area (Figure 3-22). Very limited deformation occurred in the hangingwall and footwall of the tunnel.



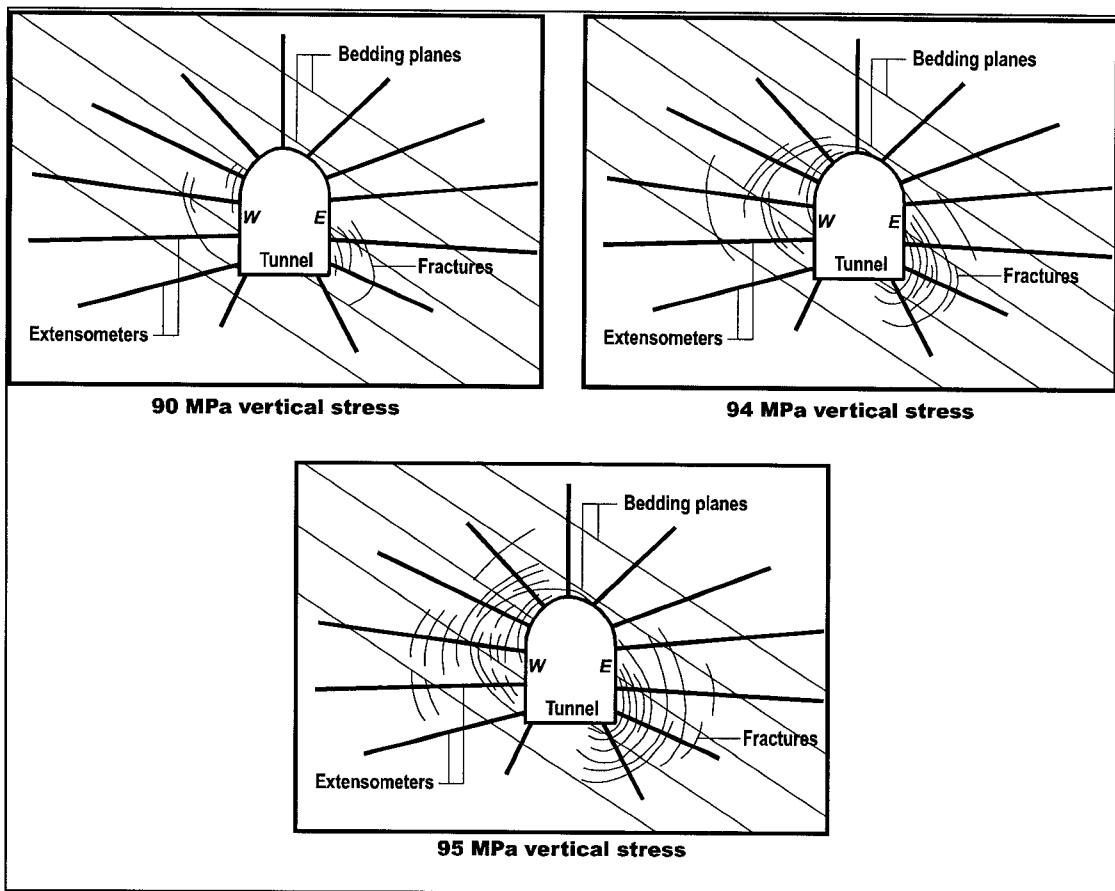
**Figure 3-21. Development of deformation for an increase in the vertical stress component between 89 MPa and 90 MPa.**



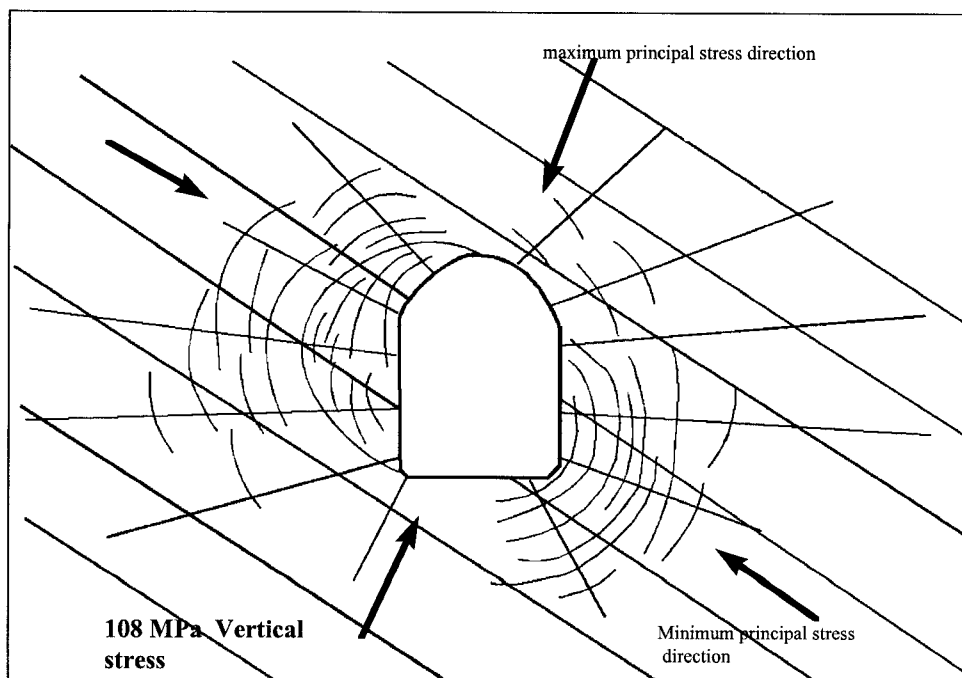


**Figure 3-22. Development of deformation for an increase in the vertical stress component between 89 MPa and 108 MPa**

The damage to the rock mass, as defined by the development of fracturing associated with the increasing stress environment and the measured deformation, was examined by petroscope holes. These were located adjacent to the extensometer holes and thus should correlate with the deformation measurements. The development of fracturing is illustrated in Figures 3-23 and 3-24. It is considered that the observed physical damage to the rock mass corresponds well with the areas of significant deformation, as may be anticipated. The location and intensity of fracturing, and its interaction with the pre-existing rock mass structure, have important implications in the design and effectiveness of the rock bolt reinforcement system as is discussed latter.



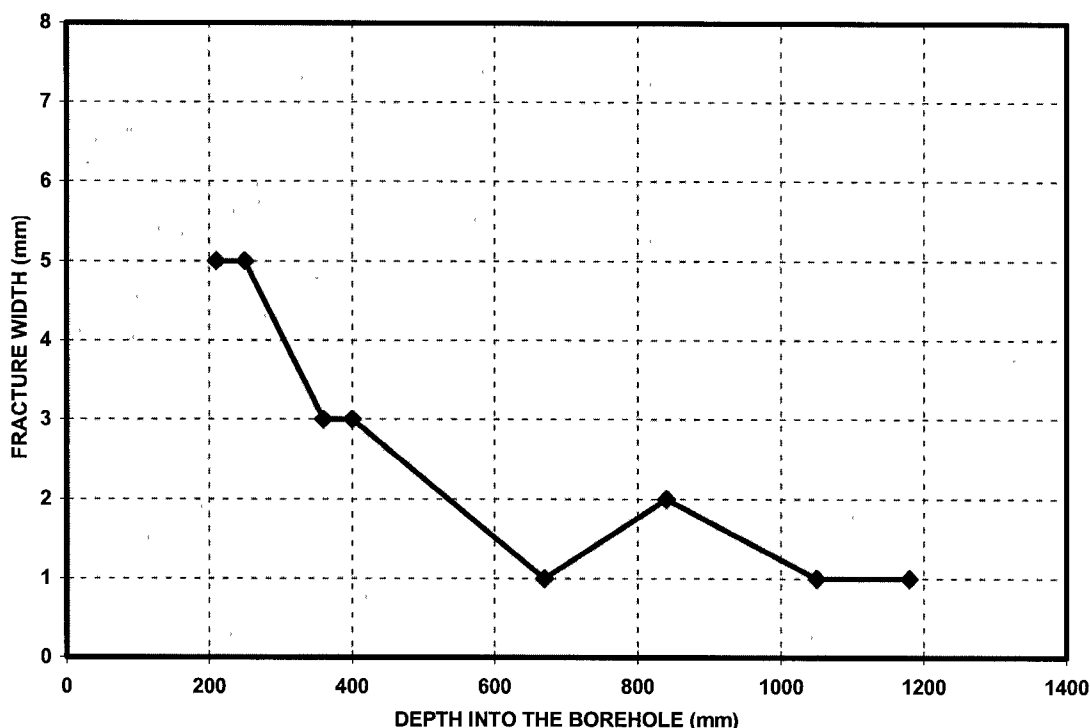
**Figure 3-23. Mapping of the development of fracturing with vertical stress increase.**



**Figure 3-24. Maximum development of fracturing at peak vertical stress level of 108 MPa.**

The observation of the limited depth of fracturing at an estimated vertical stress level of approximately 90 MPa is slightly lower than the predicted anticipated depth of instability (section 3.3). However, the depth of observed fracturing associated with the maximum stress

level of 108 MPa (3.5 m) corresponds well with the analysis in section 3.3. The characteristic of the fracturing, expressed as the width of the fracture opening, is shown in Figure 3-25. It may be that the width of the fracture is a reflection of the amount of deformation on the discontinuity and thus a measure of the increased damage and reduced inherent strength of the rock mass. As may be anticipated, the closer to the skin of the excavation and thus the more reduced the confinement to the rock mass, the greater the opening of the fractures. As the periscope holes were approximately mid-way between the rock bolt reinforcements (Sevume, 1998), this may also be a reflection of the limited interaction and influence of the rock bolts at this point. The significant opening of fractures is limited to approximately 600 mm depth, where it is considered that the increased confinement of the rock mass due to the spacing of the rock bolt reinforcement becomes effective.



**Figure 3-25. Width of fracture with depth into sidewall of the tunnel at the Kloof site.**

The depth of detailed examination of the fractures was limited by the ability to make clear observations with the available equipment.

The investigations described in this section of the report have greatly improved the understanding of the fractured rock mass around a tunnel excavation, the mechanisms of deformation and the influence of the rock bolt reinforcement as a system. Parameters determined in this section now allow the design engineer to estimate the overall deformation of a rockwall, and the dilation within the rock mass, based on an anticipated stress path and the rock bolt density. Of significance is the mechanism of hangingwall deformation under vertical stress reduction and the implications on the shear characteristics of the rock bolt reinforcement.

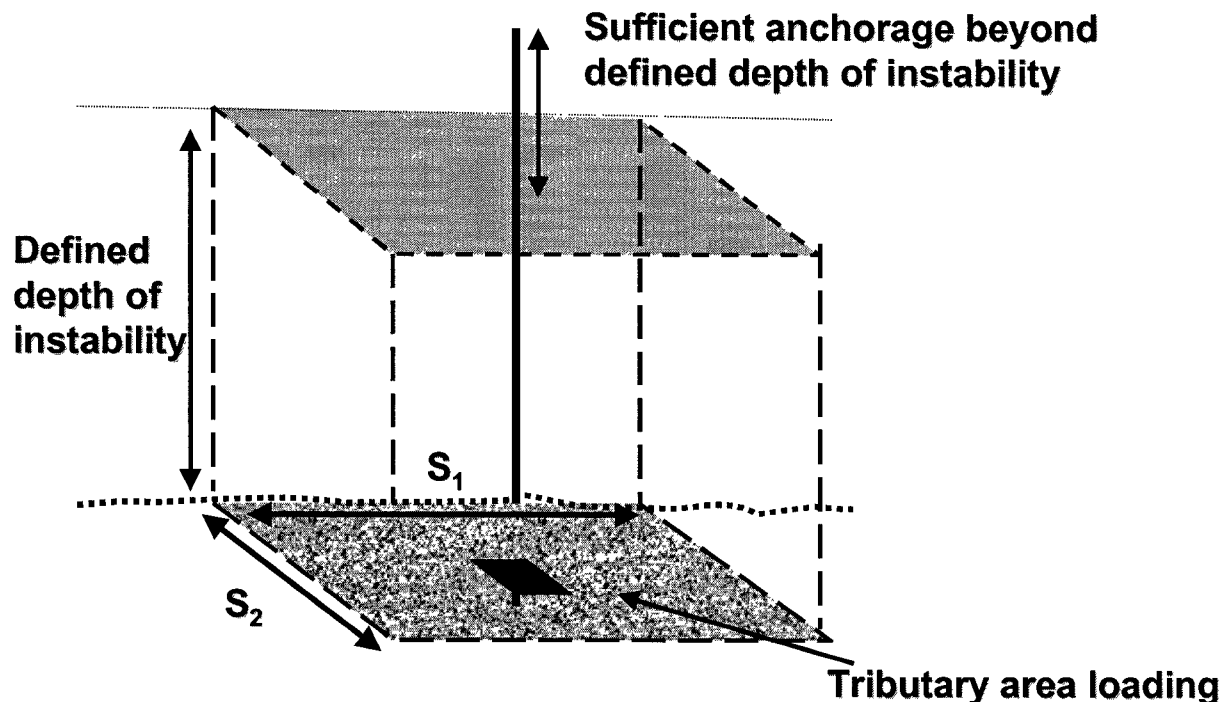
### **3.5 Analysis of support systems and rock mass interaction**

The work in this section of the investigation fulfils aspects of output 5 (Determination of mechanisms of excavation stabilisation), output 7 (Determination of interaction between support and the rock mass) and output 8 (Influence of seismicity on excavation stability).

A major emphasis of this research project has been to examine the effectiveness of the design of the current support systems, particularly under poor rock mass and dynamic loading

conditions. It is under these conditions that current support systems have been observed to fail (Haile *et al*, 1995). In order to develop our understanding of the behaviour and design of support systems, a mechanistic evaluation of the interaction between the rock mass and the components of a support system was considered appropriate. This should not only capture the success of current empirical design systems based on classifications of the rock mass environment, but also allow greater flexibility in design across different rock mass environments and an ability to make engineering judgements on support design in new environments.

The current design method used in the South African mining industry (Anon 1996a) is based on tributary area loading of rock bolt units for an assumed depth of rock mass instability, with anchorage of the rock bolt in excess of this depth (Figure 3-26).



**Figure 3-26. Tributary area loading and stable anchorage of rock bolt reinforcement within a support system based on current design practice (Anon. 1996a).**

In situ observations, as discussed in section 3.2, indicate that in the highly fractured rock mass environment of the deep level gold mines there is limited interaction between the rock bolt reinforcement and the rock mass. This is illustrated by the often observed bulking of the mesh and lace by the fragmented rock mass between the points of rock bolt anchorage. It is also illustrated by the survival of fully grouted rebar rock bolts, of limited yield capability, subsequent to a major seismic event (background Photograph 3-6). This is explained by either the total unravelling of the rock mass between the rock bolt units or the excessive deformation of the mesh and lace fabric. Where the rock mass is more massive, then the extent of interaction between the rock bolt reinforcement and the rock mass is greater. Under dynamic loading conditions, this will result in more direct loading of the rock bolts and failure if they are of insufficient energy absorption capability (foreground Photograph 3-6). The interaction and effectiveness of the rock bolt reinforcement within the rock mass has thus been shown to be a function of the rock mass structure with respect to the bolt spacing. The consideration of the structure of the rock mass in relation to the rock bolt reinforcement system is thus a critical parameter in the design of support systems in these environments.

The relationship between the rock mass structure and the relative stability of the rock mass between the rock bolt reinforcement is shown in Figure 3-28, based on the numerical modelling analysis.

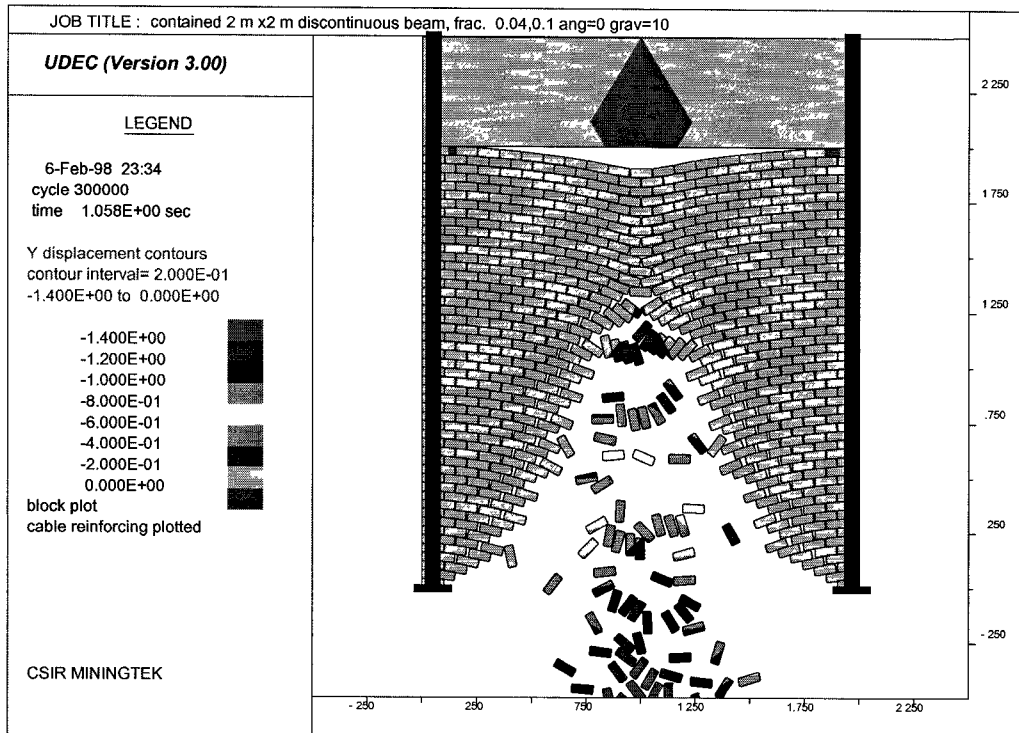


Figure 3-27. Example of numerical modelling of rock mass stability between rock bolt reinforcement.

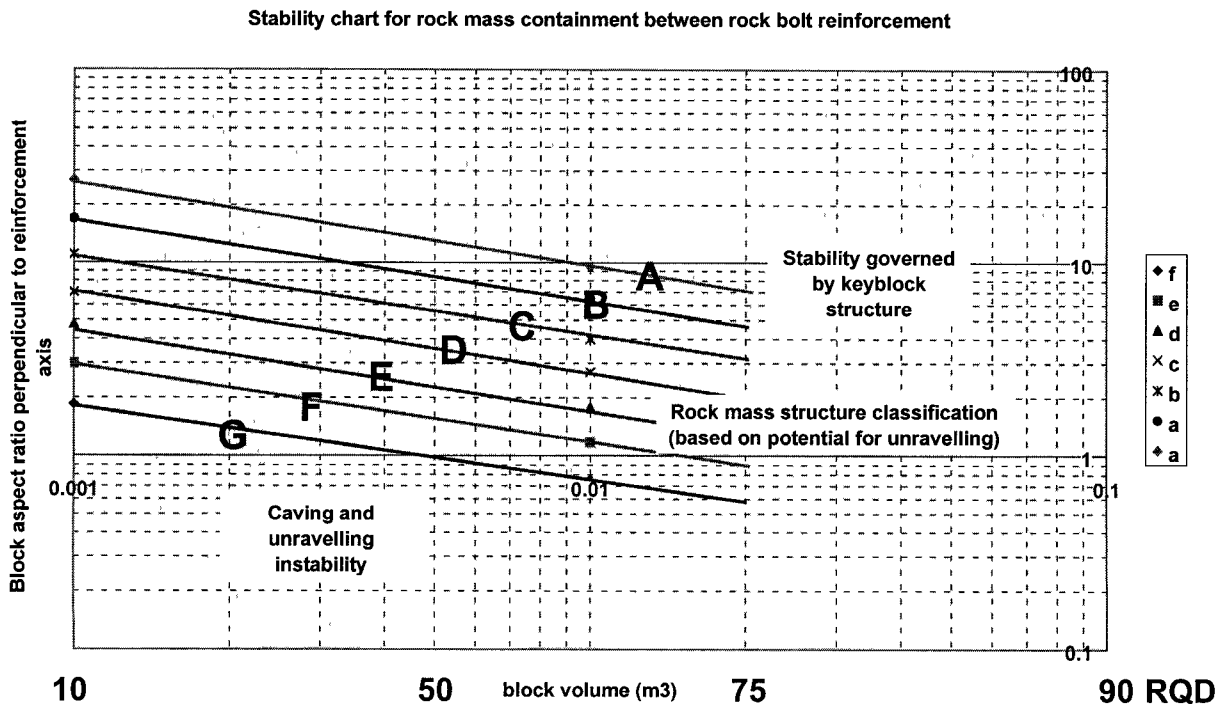
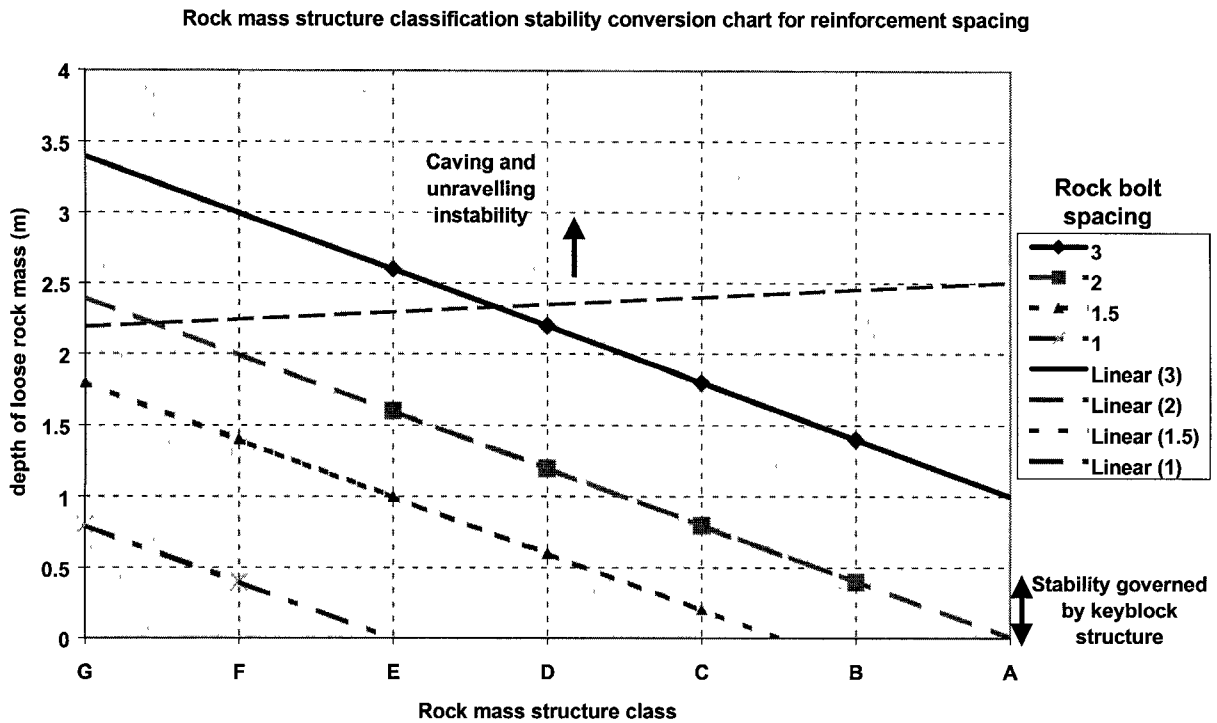


Figure 3-28. Chart of relative stability of rock mass structure between rock bolt reinforcement based on numerical modelling.

In Figure 3-28 the classifications A to G represent the relative stability of the rock mass from relatively stable to unstable respectively. The classification of a given rock mass structure is based on the intercept between the block volume (or RQD) on the x-axis and the block aspect ratio on the y-axis within Figure 3-28. A given level of stability, as reflected by a rock mass class, may be attained by either increased block volume for a lower aspect ratio or by a high aspect ratio at a reduced volume. This high aspect ratio is comparable to large thin slabs perpendicular to the rock bolt in the sidewall of a tunnel.

The stability of the rock mass between the rock bolt reinforcement will also be a function of the spacing, and, thus, the interaction of the rock bolts within the support system. It would be anticipated that the closer the rock bolt spacing, and, thus, the higher the reinforcement density, the more stable the rock mass between the rock bolts would be. This has been shown from in situ observations (section 3.2) and from the in situ instrumentation of the performance of the overall support system (section 3.4). The numerical modelling analysis, as illustrated in Figure 3-27, was also used to evaluate the influence of the rock bolt reinforcement spacing on the extent of instability between the rock bolts under gravitational loading conditions (Haile, 1998). The results of this analysis are shown in the form of a chart in Figure 3-29.



**Figure 3-29. Chart of the influence of rock bolt spacing, based on rock mass classification as derived from Figure 3-28, on depth of instability between the rock bolts.**

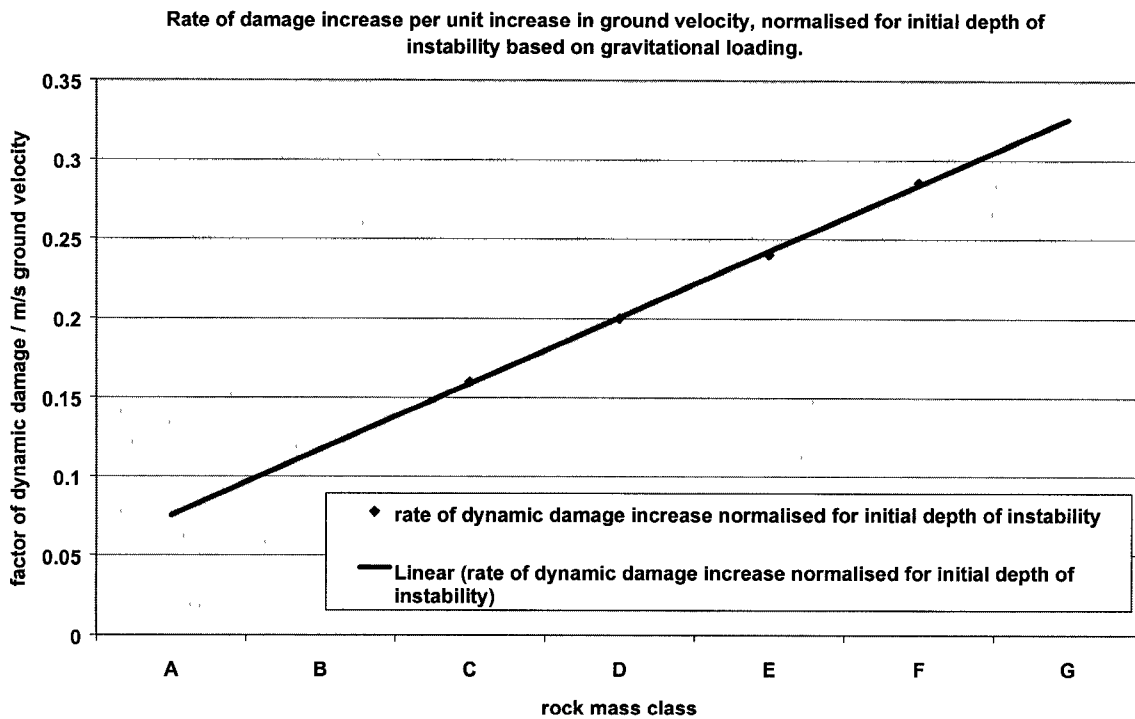
The y-axis of the chart in Figure 3-29 represents the depth to which anticipated unravelling of the rock mass may occur between the rock bolt reinforcement. Figure 3-29 illustrates the tendency for reduced instability of the rock mass with reduced rock bolt spacing and also the increased stability of the rock mass from G to A, ie increasing block volume and reducing block aspect ratio.

At high values of unravelling of the rock mass, interaction between the rock bolt units may be completely lost (in the absence of fabric support) and result in total unravelling of the rock mass from around the rock bolts. Where unravelling of the rock mass structure is indicated to be restricted due to a good rock mass class or low support spacing, consideration must still be

given to the potential for isolated structural instability of blocks not directly retained by the rock bolt reinforcement.

The chart shown in Figure 3-29 illustrates the relationship between the rock mass classification and the rock bolt spacing on the estimated depth of instability based on gravitational loading. In many cases tunnels within the deep level South African mines will also be subjected to loading due to seismic events. This dynamic loading of the rock mass may result in an increased extent of instability around the tunnel as discussed in section 3.3, but also in the increased instability of the rock mass between the rock bolt reinforcement. This mechanism has again often been observed in situ as described in section 3.2. The numerical modelling programme, as illustrated in Figure 3-27, was used to examine the potential for increased unravelling of the rock mass between the rock bolt reinforcement under simulated dynamic loading (Haile, 1998). The dynamic loading within the model was simulated by artificially increasing the gravitational loading of the model and equating this to an additional kinetic energy.

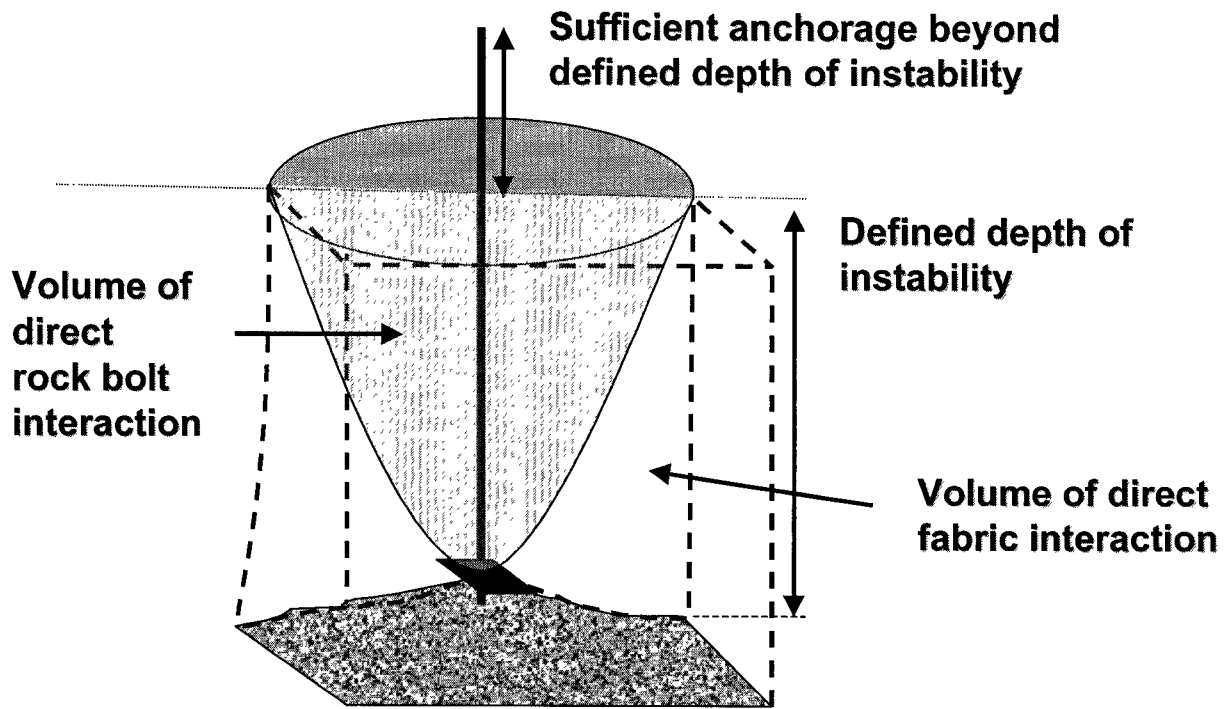
The potential of a given rock mass class for unravelling is shown in Figure 3-30. The y-axis of this chart gives a factor, per unit increase in dynamic ground motion, which represents the increased potential for rock mass unravelling. This value is in excess of that defined by the rock bolt spacing under gravitational loading for the equivalent rock mass class.



**Figure 3-30. Chart illustrating the increased potential for rock mass unravelling within a rock mass class per unit increase in dynamic ground motion.**

Within the sidewall of a tunnel the potential for unravelling, apart from inherent structural instability (toppling, sliding), will be purely due to the effects of additional dynamic loading. The influence of gravitational loading shown in Figure 3-29 must therefore be discounted for sidewall analysis. Based on an analysis of the rock mass classes, gravitational loading has been equated to a ground velocity of approximately 0.7 m/s (Haile 1998) for application within this analysis.

By simple geometrical analysis, the volume of the potentially unstable rock mass between the rock bolt reinforcement can now be estimated for a defined rock bolt pattern and loading condition (Haile, 1998) (Figure 3-31).



**Figure 3-31. Schematic representation of the interaction of the components of a support system within a discontinuous rock mass structure.**

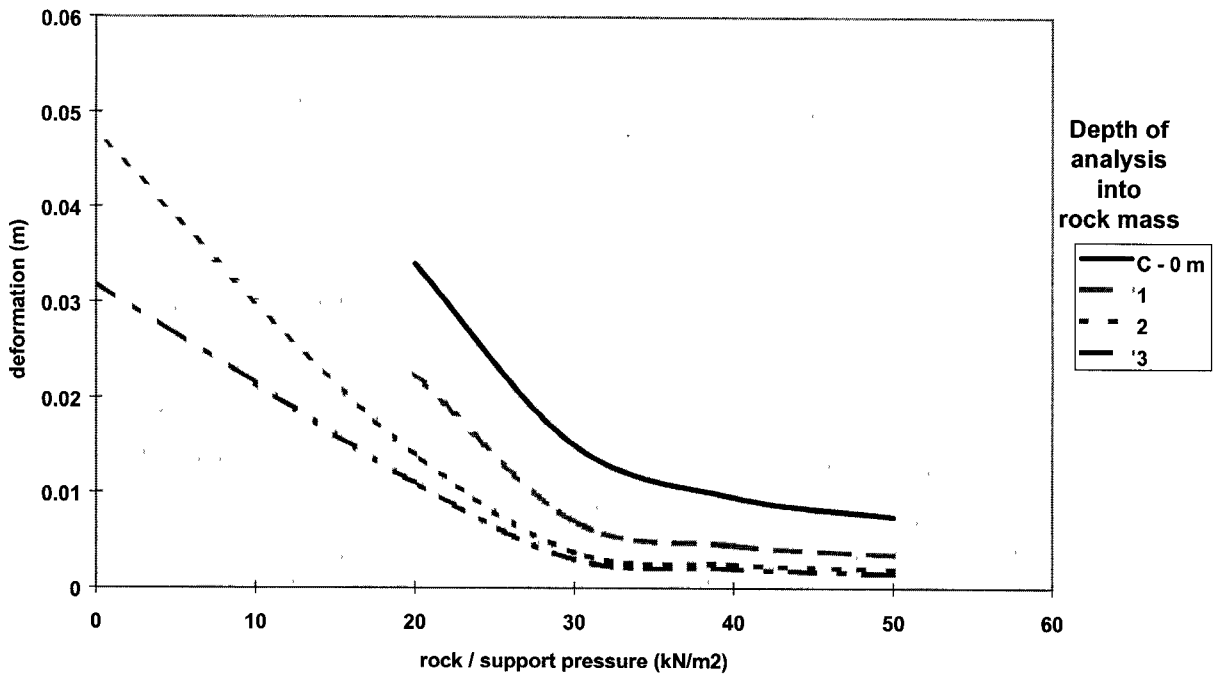
This defined potentially unstable volume can be used to estimate the requirement for, or loading of, a fabric support system to enable the design of the fabric based on a mechanistic evaluation. In addition, the required capacity of the attachment between the fabric support and the rock bolt reinforcement anchorage can also be estimated. Similarly, the volume of rock mass that will directly interact with the rock bolt reinforcement can be estimated. The understanding that the tributary area rock mass volume does not directly load the rock bolt reinforcement in these rock mass environments explains the often observed survival of these components of the support system under conditions when the current design criterion would indicate failure. The role of the fabric in absorbing kinetic energy and the transfer of loads to the rock bolt reinforcement will be discussed under section 3.6.

The above analysis considers that the rock mass between the rock bolt reinforcement is free to deform and unravel until a natural state of stability is attained dependent on the rock mass class and rock bolt spacing. However it is considered that the characteristics of the fabric support system will influence the extent of unravelling, or loss of inherent rock mass strength, and, thus, the distribution of loading between the fabric and the rock bolt components.

Further numerical analysis was conducted to evaluate the concept and mechanisms of interaction between a fabric support system and the extent of instability between the rock bolt reinforcement (Haile, 1998). This analysis considered the concept of a 'ground reaction curve' of the rock mass between the rock bolts. The evaluation is based on an applied surface load and resultant limitation of displacement of a given rock mass to maintain the inherent rock mass strength. An example is shown in Figure 3-32 for a class C rock mass at a 3 m reinforcement spacing under gravitational loading.



Ground Reaction Curve for class C rock mass at rock mass depth at 3 m span



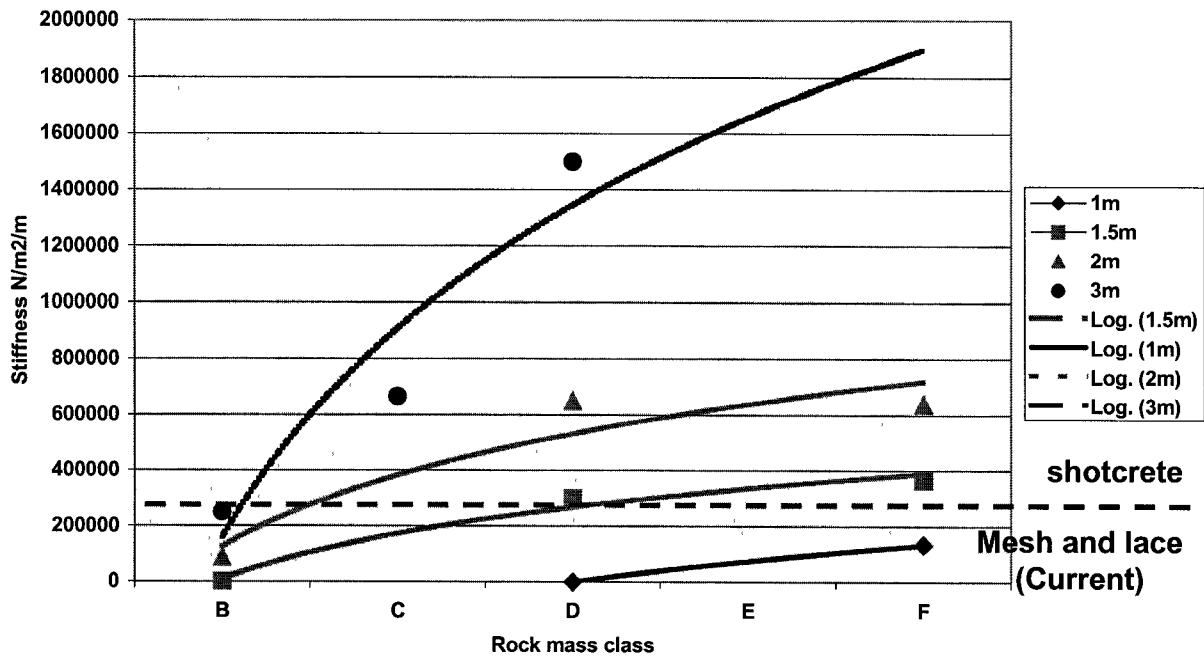
**Figure 3-32. “Ground reaction curve” for class C rock mass and 3 m reinforcement spacing.**

The evaluation of the characteristic of the fabric support is based on the ability of the system to generate the required support pressure within the limit of stable rock mass deformation. In the above example, the fabric support must generate a minimum of 20 kN/m<sup>2</sup> at approximately 0.035 m deflection (Figure 3-32) to maintain the interlocking rock mass structure and thus the inherent rock mass strength.

The implication of the above analysis is that the use of relatively stiff fabric support systems, with sufficient capacity to maintain the rock mass structure under a defined loading condition, will result in increased direct loading of the rock bolt reinforcement. Thus, a change in one component of the system will influence the demand on the other components and must be considered in the holistic design process.

A summary of the relationships between the rock mass class, rock bolt reinforcement spacing and the minimum fabric stiffness to maintain the inherent rock mass strength under gravitational loading is given in Figure 3-33.

### Analysis of fabric stiffness



**Figure 3-33. Analysis of minimum fabric stiffness to maintain the inherent rock mass strength for the given rock mass class and rock bolt reinforcement spacing.**

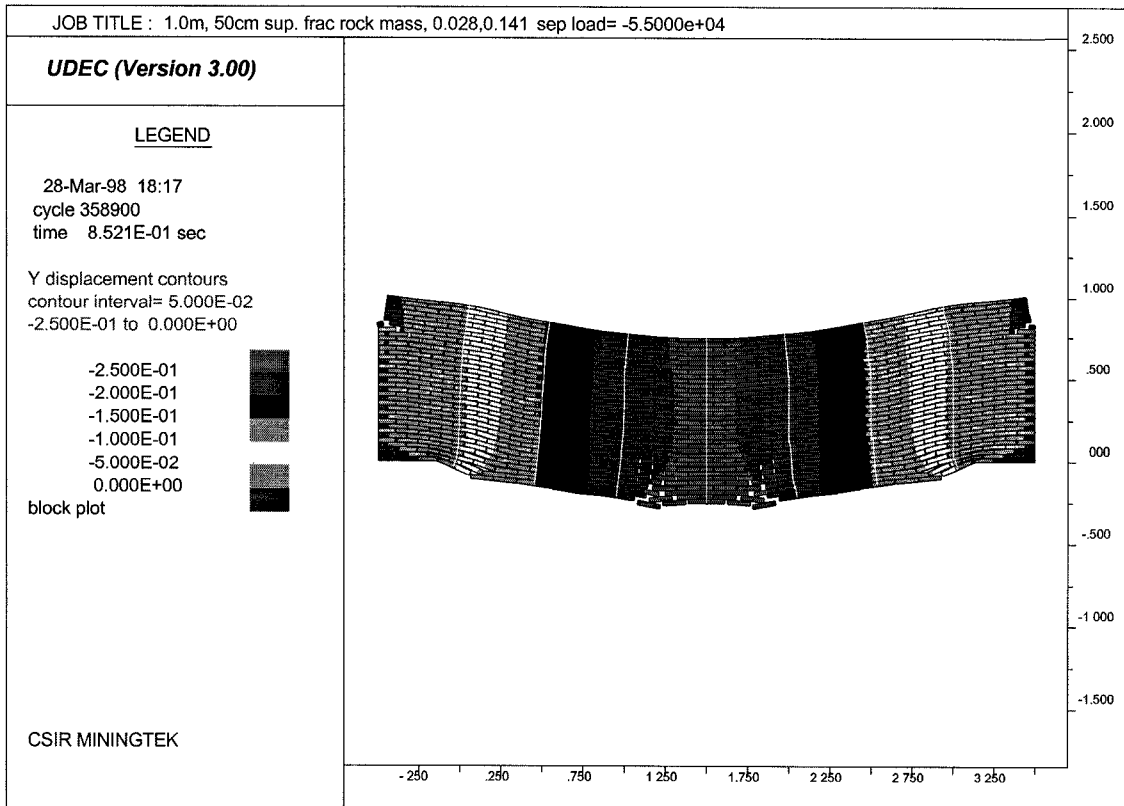
Figure 3-33 indicates that the poorer rock mass classes and larger rock bolt spacing require increased fabric stiffness to maintain the inherent rock mass strength. For illustrative purposes, an estimated upper limit of effective, good quality mesh and lacing stiffness is shown in Figure 3-33. This indicates that, for a typical rock bolt reinforcement spacing of approximately 1.5 m, shotcrete is necessary to maintain the inherent rock mass strength in excess of a class C rock mass. It would be anticipated that, under dynamic loading, the necessity for stiff fabric support systems (such as shotcrete) would become more critical to maintain the rock mass strength. However, it must be remembered that this will result in an increased direct loading on the rock bolt reinforcement and thus suitable design considerations must be applied to this component of the support system.

The above analyses have been primarily based on the design concept of rock mass retainment and containment, where the rock bolt reinforcement is of a length sufficient to ensure anchorage in excess of the anticipated depth of rock mass instability due to the tunnel. An alternative design approach is the creation of a reinforced rock mass shell at the boundary of the excavation sufficient to contain the deeper unstable rock mass volume. In this case the length of the rock bolt reinforcement is less than the anticipated depth of instability, but the spacing of the reinforcement is critical to ensure a competent reinforced structure under the anticipated loading conditions. In situ observations, as discussed in section 3.2, have illustrated this mechanism of support system performance, although it is not specifically considered in the current design processes (Anon. 1996b).

### 3.5.2 Numerical modelling analysis – reinforced rock mass structure

An evaluation of the critical parameters that influence the stability of excavations based on this design philosophy has been conducted using numerical modelling (Haile, 1998). The critical rock mass parameters that were evaluated corresponded to those used in the analysis of rock bolt containment (section 3.5.1). The classification of the rock mass, as defined in Figure 3-28, is applied. This therefore allows a comparative evaluation of the two support methodologies

based on the numerical modelling. With this design philosophy the critical rock bolt reinforcement system parameters are the length of the rock bolts defining the reinforced structure thickness and the spacing of the rock bolts within the reinforced structure. The rock bolt reinforcements were modelled with the characteristics of fully grouted rebars. The performance of the reinforced rock mass structure was evaluated on the basis of its energy absorption capacity either up to the point of failure, or to a design operational deformation limit of 0.25 m. An example of the numerical analysis of a reinforced structure, illustrating its deformation capability, is shown in Figure 3-34.

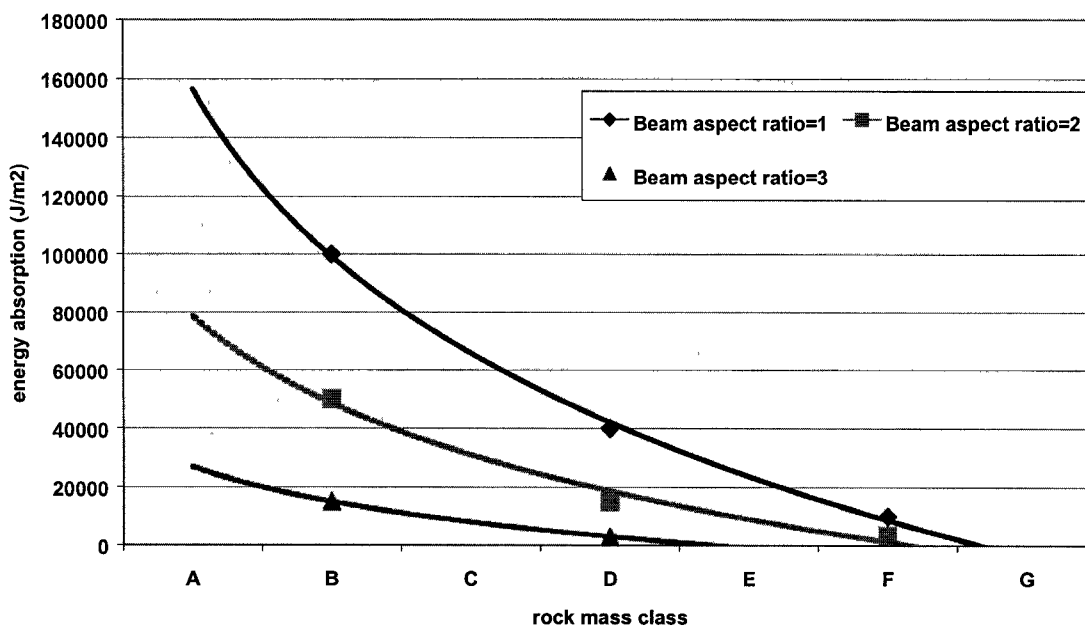


**Figure 3-34. Numerical analysis of reinforced rock mass structure.**

The numerical analysis was controlled to derive the load and deformation characteristic of the structure, which allows the derivation of its energy absorption capacity from the area under this curve. It is of interest to note in Figure 3-34 the degree of shear deformation to which the rock bolt reinforcement units are subjected. The localisation of shear will be dependent on the structure of the rock mass and the position of the rock bolt reinforcement unit within the structure.

An analysis of the numerical modelling programme, indicating the relationship between the beam geometry and its energy absorption capacity for different rock mass classes, is shown in Figure 3-35. The energy absorption capacity of the reinforced rock mass system as evaluated in Figure 3-35 is its total capability and must consider both its self weight loading in addition to the surcharge loading of the deeper unstable rock mass.

General relationship between rock mass class and reinforced (1 unit/m<sup>2</sup>) beam aspect ratio based on relative energy absorption capacity

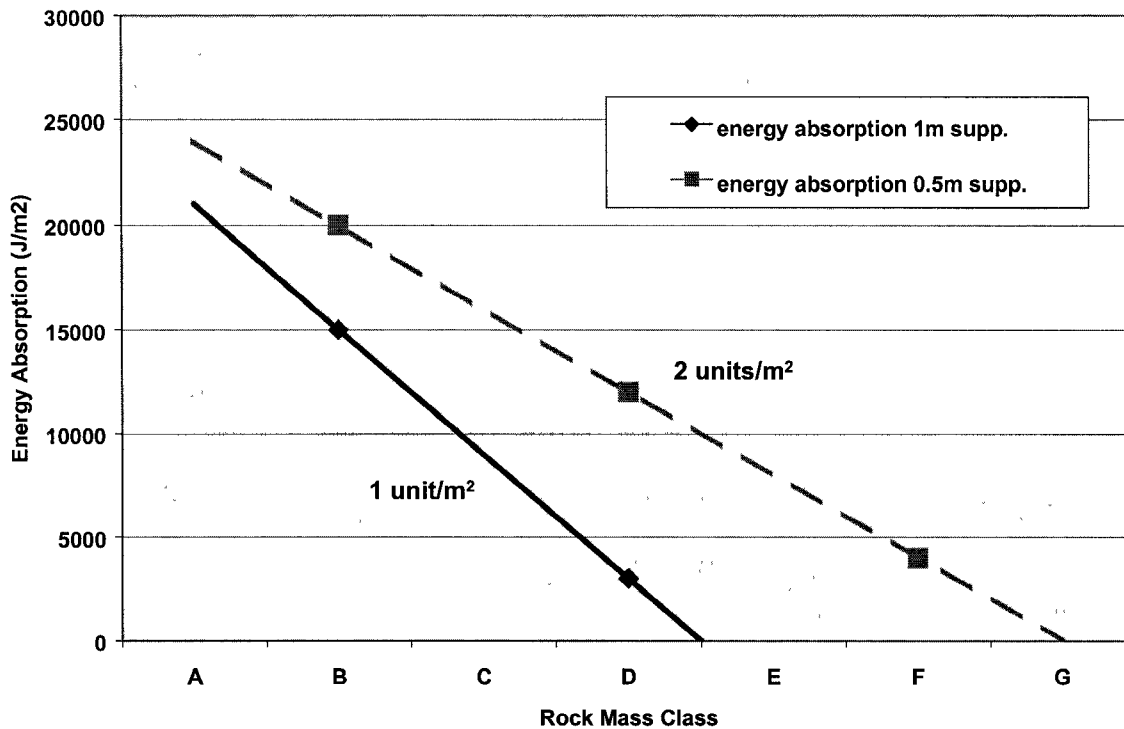


**Figure 3-35. Relationships between beam geometry, rock mass class and maximum energy absorption capacity for rock bolt density of 1 unit/m<sup>2</sup>.**

In Figure 3-35 the geometry of the beam is expressed as the aspect ratio; this is taken as the free length of the exposed beam in relation to its thickness. Thus a beam aspect ratio of two would indicate a beam thickness of half the length between major abutments. In a typical tunnel layout this would represent 1.8 m rock bolts in a 3.6 m high sidewall and would be inclusive of the potentially unstable zones between the rock bolt reinforcement. The numerical modelling analysis, as summarised in Figure 3-35, indicates the anticipated relationships that a better rock mass class and a thicker reinforced structure results in a more competent reinforced rock mass structure. This illustrates the importance of the proposed concept to ensure adequate interaction between the rock bolt reinforcement for the defined rock mass characteristics. At the defined rock bolt density, the ability to contain sufficient energy in excess of a class E rock mass is limited. In this case the density of the rock bolt reinforcement within the structure must be increased to ensure sufficient interaction and thus beam competency.

The influence of the density of the rock bolt reinforcement within the rock mass structure is shown in Figure 3-36. This illustrates the significant impact that the density of the rock bolt reinforcement within the rock mass can have on the competency of the reinforced rock mass structure, particularly in poor rock mass conditions. The reduced influence of the rock bolt spacing in good rock mass conditions is due to the increased inherent rock mass strength and thus a lower proportional effect of the rock bolt reinforcing action.

Analysis of Rock Mass Class on Energy Absorption for beam aspect ratio 3:1

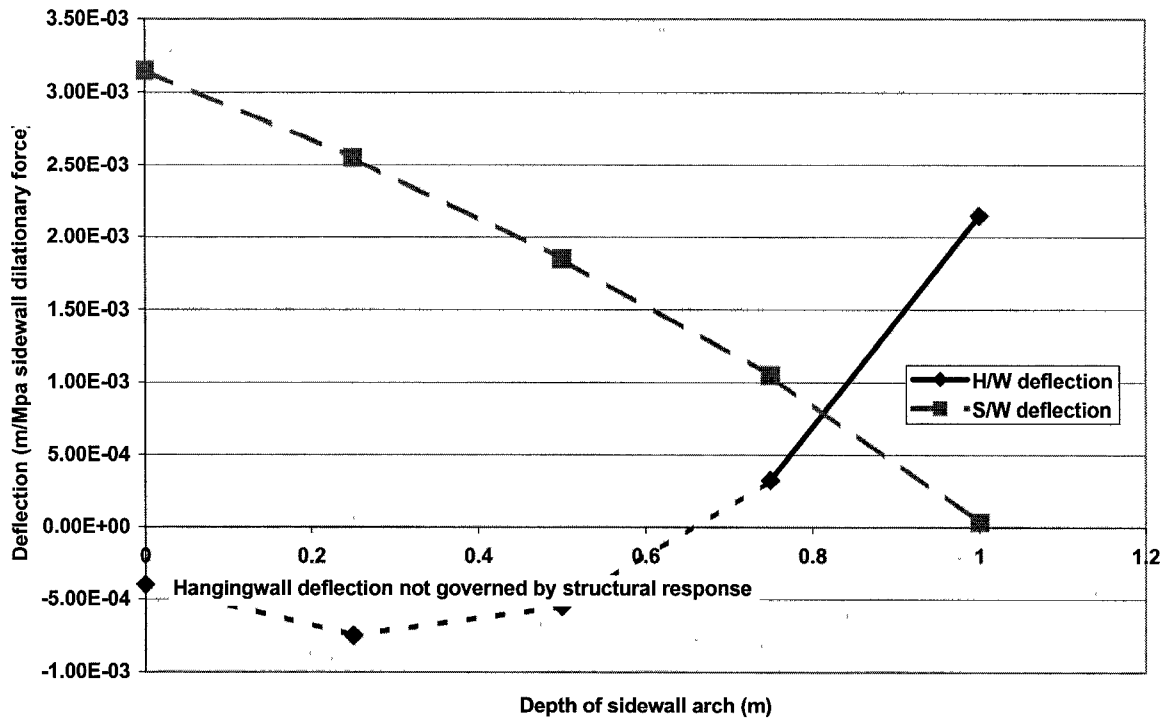


**Figure 3-36. Influence of rock bolt reinforcement density on reinforced rock mass beam capacity.**

The above analyses have indicated, on a mechanistic basis, the viability of rock bolt reinforcement design based on the creation of a reinforced rock mass structure. This investigation has also shown that, on the basis of this design principal, relatively stiff, limited yield rock bolts can operate in a system with the rock mass to absorb large amounts of energy under potential seismic loading. This analysis has focused on the performance of reinforced beams typical of the square profile tunnels within the South African mining industry.

### 3.5.3 Numerical analysis – excavation shape

Further analysis of the relative stability of a reinforced rock mass structure examined the influence of the shape of the structure on the deformation characteristics per unit of dilational or dynamic force on the reinforced structure (Figure 3-37) (Applied Mechanics Group, 1998). This analysis is based on the depth of sidewall curvature from the vertical sidewall for a constant reinforced thickness and height of excavation.



**Figure 3-37. Structural deformation per unit of sidewall dilatatory stress (MPa) of reinforced shell for a base case of 3.5 m square excavation with 1.25 m structural reinforcement.**

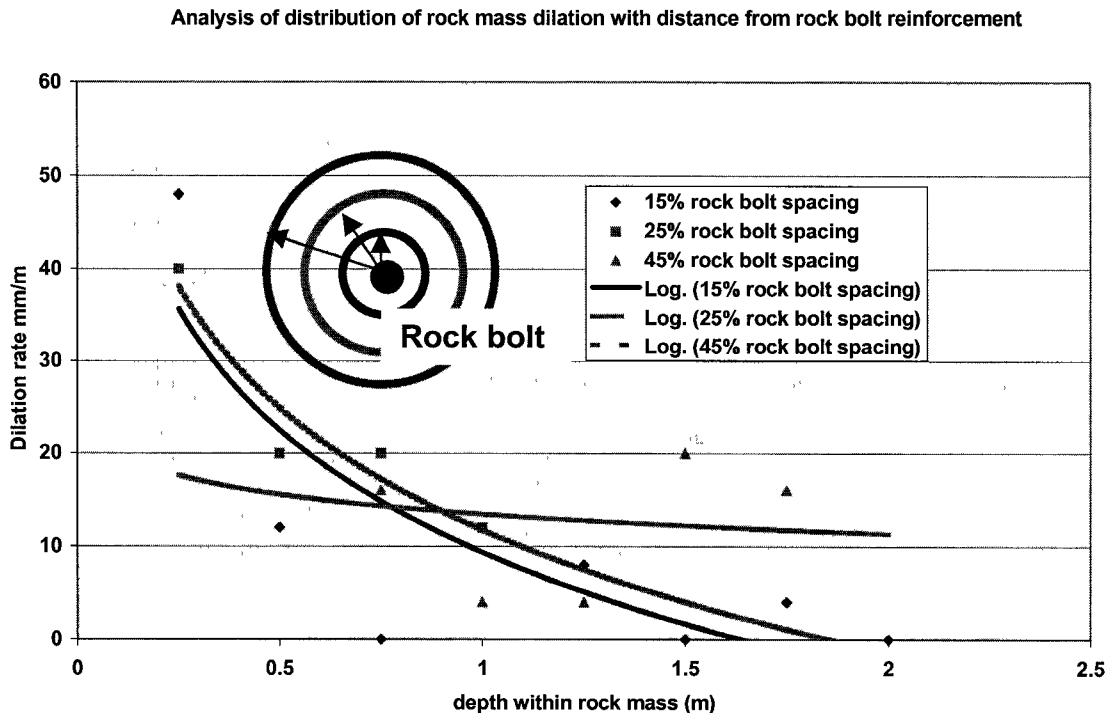
This analysis quantifies the relative reduction in sidewall deformation with increased curvature of the rock wall, however, it also indicates that, although the structure is more able to resist sidewall deformation, this results in increased lateral loading of the hangingwall rock mass and thus potential buckling. This is illustrated in Figure 3-37 by the rapid increase in hangingwall deflection at higher depth of the sidewall arch profile. This is aggravated by the increased curvature of the sidewall which results in an increased effective hangingwall span and thus reduced hangingwall beam aspect ratio. This evaluation indicates that the mechanism of excavation stabilisation based on the creation of a reinforced rock mass structure is viable and that the excavation profile can be engineered to control the relative degree of deformation.

The previous analyses have examined the mechanisms of interaction between the rock bolt reinforcement and the unstable rock mass volume to ensure excavation stability. This has been based on the ability of the numerical model to capture the observed mechanisms of rock bolt and rock mass interaction as determined from in situ observations. A far better understanding of the critical parameters that must be considered in the design of these reinforcement systems and their relative capabilities has been derived.

### 3.5.4 In situ monitoring

To further verify these new design concepts of the interaction between the rock bolt reinforcement and the rock mass, detailed in situ instrumentation based on deformation and seismic measurements was implemented. The deformation monitoring focused on the measurement of the relative dilation of the rock mass with distance from the rock bolt reinforcement (Grave, 1998). It may be anticipated, based on the numerical analysis, that the degree of confinement provided to the rock mass by the rock bolt reinforcement would decrease with distance from the rock bolt. Therefore, for a given loading of the rock mass, the rock on the boundary of the tunnel furthest from the rock bolt reinforcement will dilate the most and that this dilation will be evident to the greatest depth.

A detailed array of short extensometer holes between the grouted rock bolt reinforcement units within a support pattern was used to evaluate the differential deformation within this reinforced rock mass. Three sites were established at mines in the Klerksdorp and West Rand gold fields. A summary of the results obtained is shown in Figure 3-38.



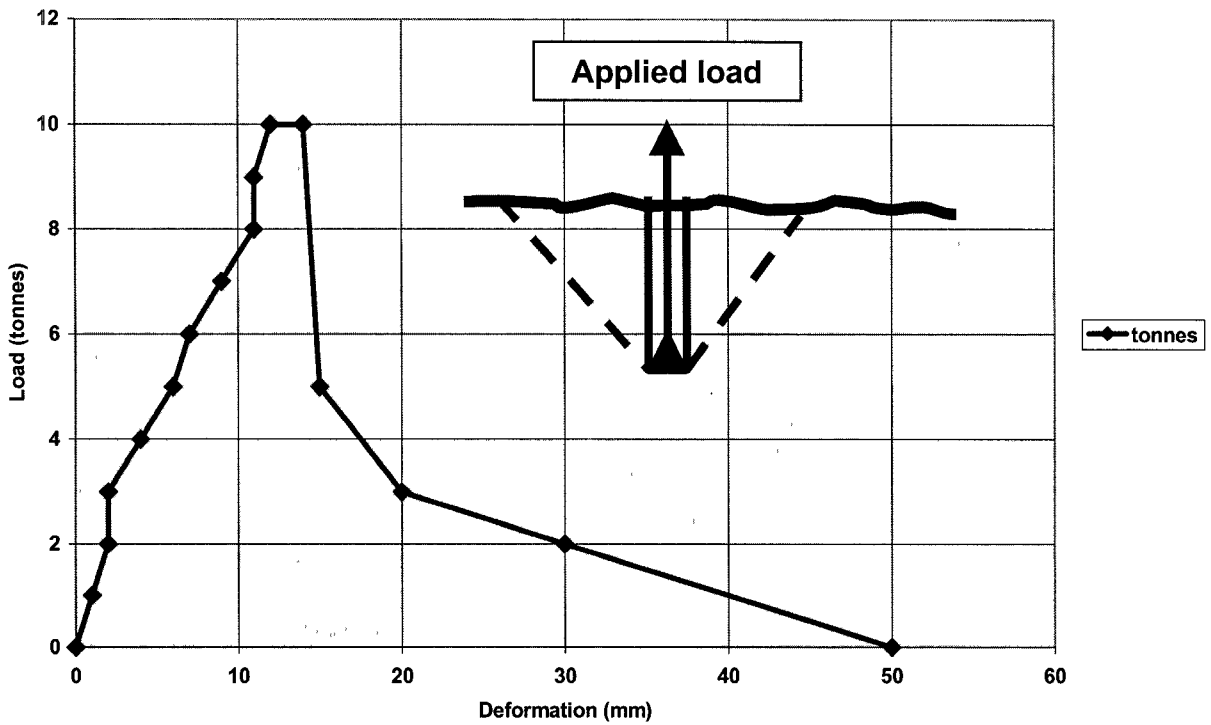
**Figure 3-38. Rock mass dilation with distance from rock bolt reinforcement.**

Although the data in Figure 3-38 is very scattered, as may be anticipated for a highly complex in situ environment, there is an indicated tendency for the dilation of the rock mass closer to the rock bolt to be more effectively contained than that further from the rock bolt. There appears to be an anomaly with regard to the less confined rock mass in the immediate skin of the excavation, but this may be a function of the extrapolated trends. In addition, the rock bolt units may not effectively influence the probable high dilation forces in the immediate skin of the tunnel. Dilation of the rock mass close to the rock bolt unit is indicated to be more rapidly restricted with depth compared to that more distant where limited interaction of the rock bolt within the fractured rock mass is indicated even at a depth of 2 m.

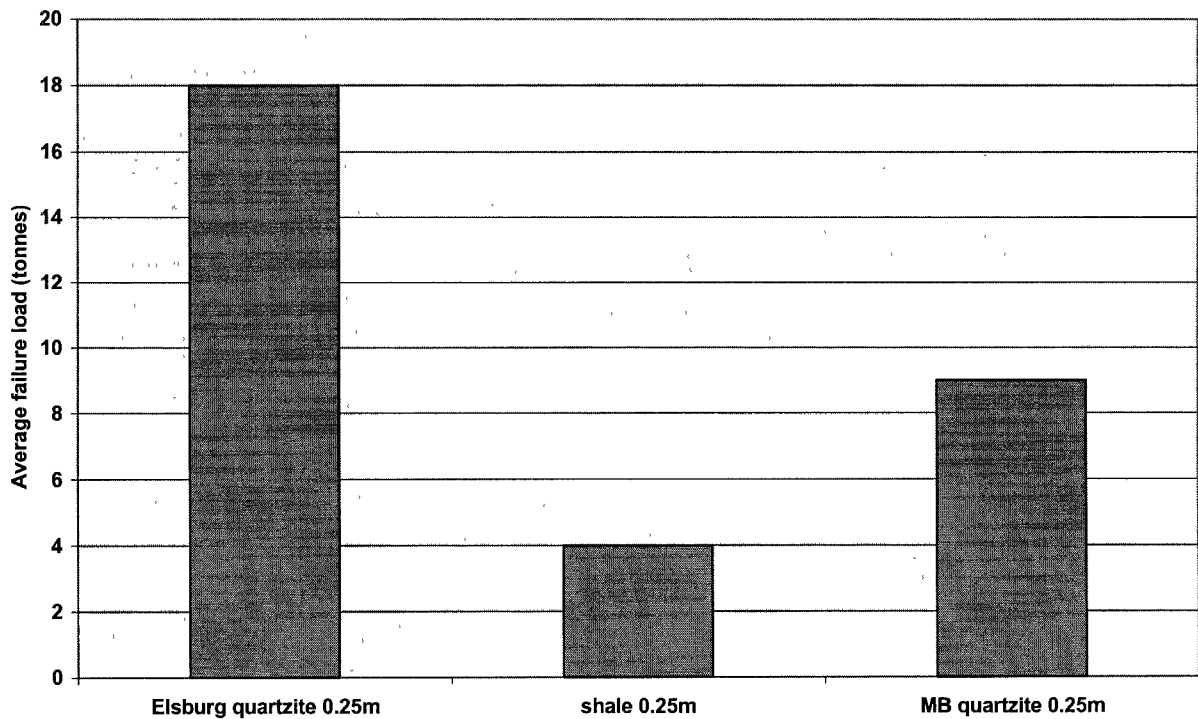
A further investigation of the competency of the reinforced rock mass and of the stability of the rock mass between the rock bolt reinforcement was based on an evaluation of the inherent strength of the skin rock mass (Grave, 1998). This was assessed by evaluation of the pull out strength of the rock mass by attachment of a point anchor at a specified depth. An example of the load and deformation characteristic of the un-reinforced skin rock mass and a diagrammatic representation of the pull test set up are shown in Figure 3-39.

Based on the limited number of tests conducted it was found that the more discontinuous the rock mass structure is, the lower the inherent strength of the sidewall rock mass. This is derived from the direct value of the load at sidewall failure for a specific rock mass type. A summary of the average pull out loads for the rock mass types as tested is shown in Figure 3-40. Although the results of this evaluation may not be directly applicable to the design of the tunnel support system, they reinforce the concept that the characteristics of the rock mass between the rock bolt reinforcement influence the relative stability of the tunnel rock wall. This

has implications on the potential capacity of a fabric support system under a given loading condition.



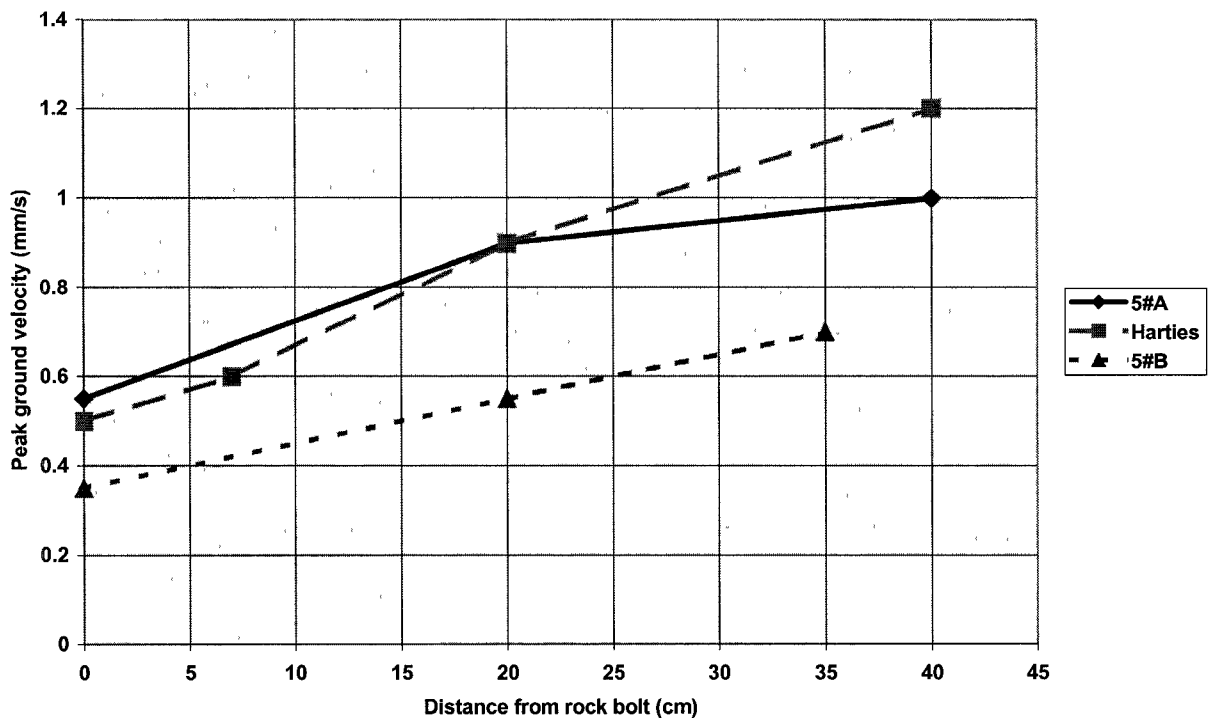
**Figure 3-39. Load and deformation characteristic for 0.5 m anchor in shale rock type.**



**Figure 3-40. Summaries of peak pull out loads for rock mass type and anchor depth.**



The influence of the rock bolt reinforcement system on the response of the rock mass under seismic loading conditions was also examined (Le Bron, 1998). This evaluation utilised a detailed array of geophones between rock bolt units to measure the peak ground velocity perpendicular to the rock wall relative to the distance from the rock bolt reinforcement. The geophones were attached to the tunnel rock wall in a regular radial pattern away from a selected rock bolt unit. Several sites of different rock mass characteristics were selected for the evaluation. Based on the average response of numerous natural seismic events, the trends, as shown in Figure 3-41, have been derived for a comparable velocity at the rock bolt location of approximately 0.5 mm/s. The maximum peak ground velocity measured over the period of the experiment was of the order of 1 m/s, but the majority of results were of the order of 0.1 m/s.

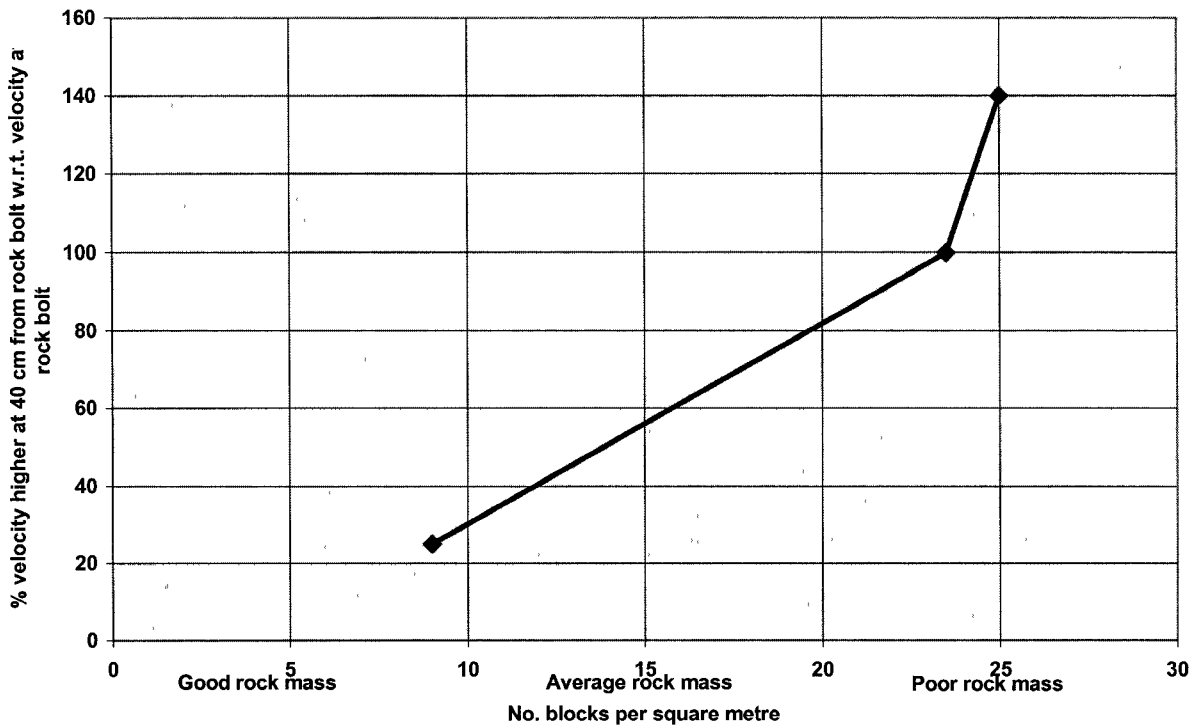


**Figure 3-41. Relationship between the peak ground velocity and distance from rock bolt reinforcement unit.**

It is clear from Figure 3-41 that there is an increased degree of freedom of the rock mass with distance from the rock bolt reinforcement, as reflected by the increased peak ground velocity for a given seismic source. This will also be indicative of the increased potential for rock mass ejection with distance from the rock bolt unit for a given inherent rock mass strength. The increased peak ground velocity may be attributed to the reduced confinement of the rock mass resulting in the increased trapping of dynamic energy, and, thus, associated increased excitation of the blocks with distance from the rock bolt unit. The increased differential velocity may result in increased potential for differential deformation within the rock mass, although this was not measured at these sites. This analysis clearly indicates the design concepts that in a highly discontinuous rock mass structure there is a reducing degree of influence on rock mass stability with distance from a rock bolt unit. Therefore the necessity for, and design of a fabric support system will be dependent on the spacing of the rock bolt reinforcement within a support pattern. The increased peak ground velocity and potential deformation away from the rock bolt unit will result in increased loading of the fabric support system in relation to the rock bolt.

An analysis of the rock mass structure at these instrumentation sites allowed the derivation of a relationship between the rock mass structure, based on the rock block geometry, and the relative amplification of the peak ground velocity (Figure 3-42). Although there is a limited

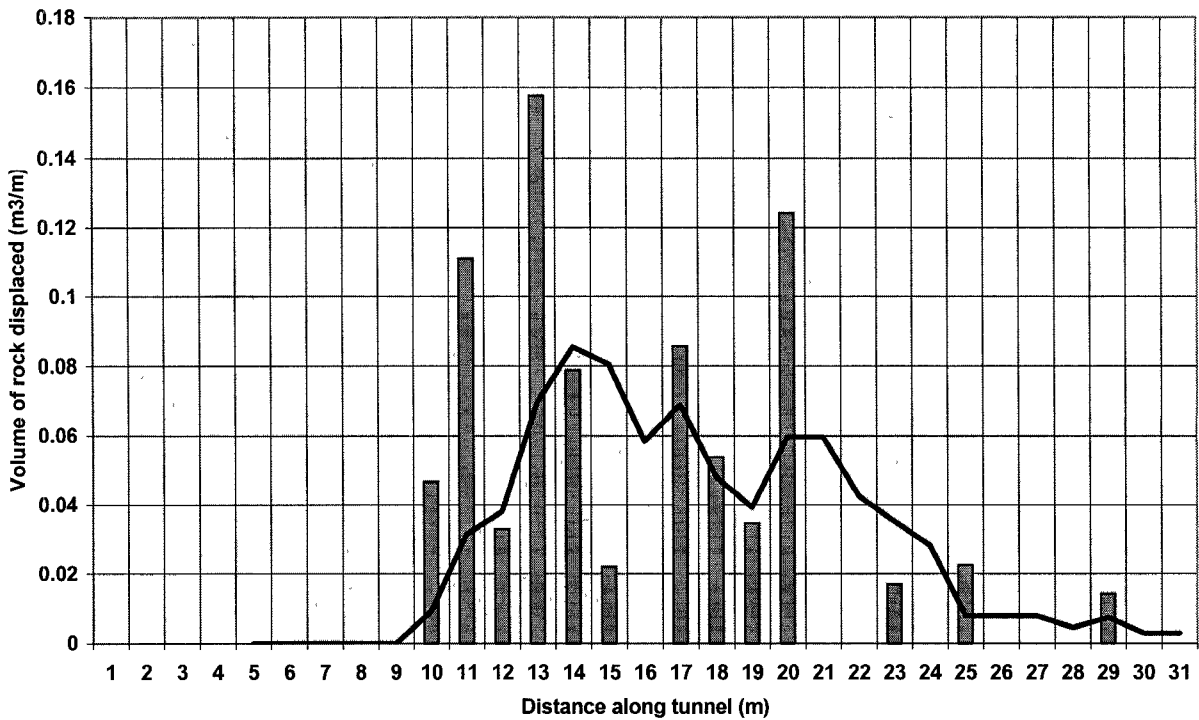
number of data points, there is an indicated trend of increased amplification of the peak ground velocity with increased discontinuity of the rock mass.



**Figure 3-42. Relationship between rock mass discontinuity and the relative amplification of peak ground velocity at a distance of 40 % of the rock bolt spacing compared to at the rock bolt location.**

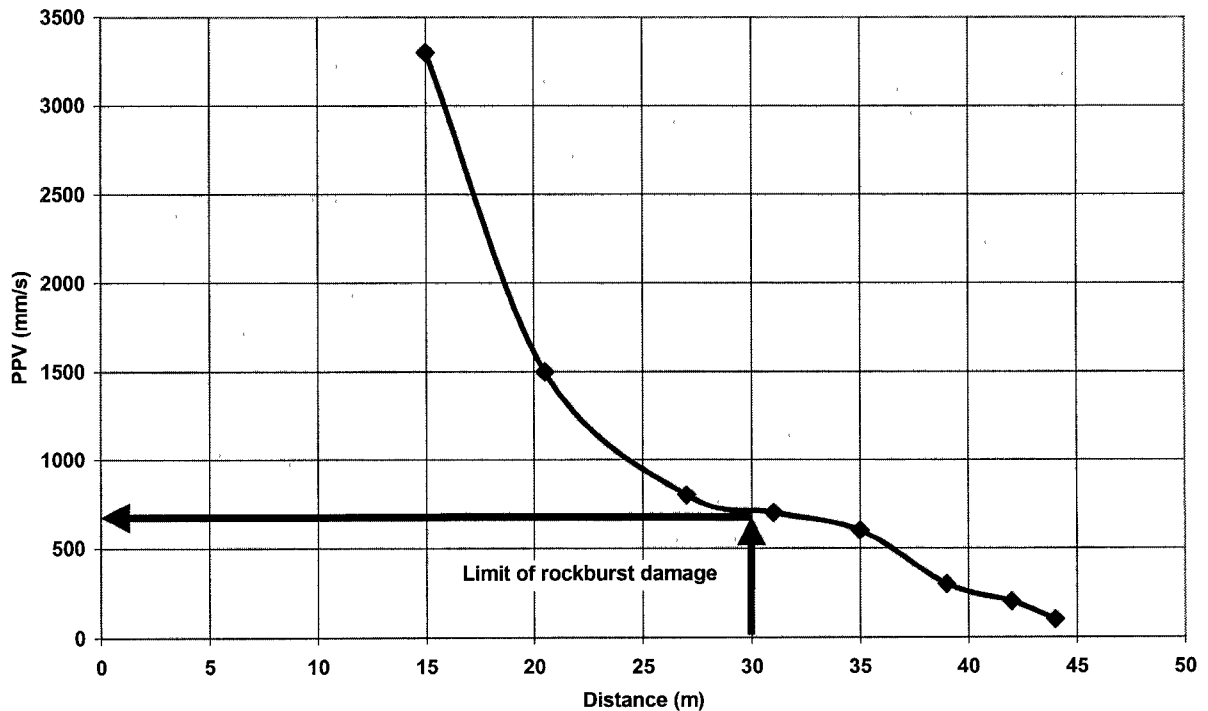
The implication of the results, as shown in Figure 3-42, is that in good rock mass conditions there is a more uniform behaviour of the rock mass and thus it may be envisaged that loading of the rock bolt will tend towards tributary area loading. In the more discontinuous, poor rock mass conditions there is increased differential behaviour and therefore it would again be envisaged that increased loading of a fabric support may occur with resultant reduced direct loading of the rock bolt units.

In addition to the above experimental sites, which were excited by natural seismicity, an investigation was conducted at a simulated rockburst site (Le Bron, 1998). This site utilised blast holes drilled parallel to and approximately 5 m from the sidewall of a tunnel to excite the tunnel excavation by dynamic loading from the blast. Hagan *et al* (1998) discusses the details of this experiment. The extent of damage to the tunnel excavation from the simulated rock burst is shown in Figure 3-43. From a local reference point on the tunnel axis, the charged length of the blast holes was between 9 m and 19 m (Le Bron, 1998).



**Figure 3-43. Distribution of significant damage at the simulated rockburst site (Le Bron, 1998).**

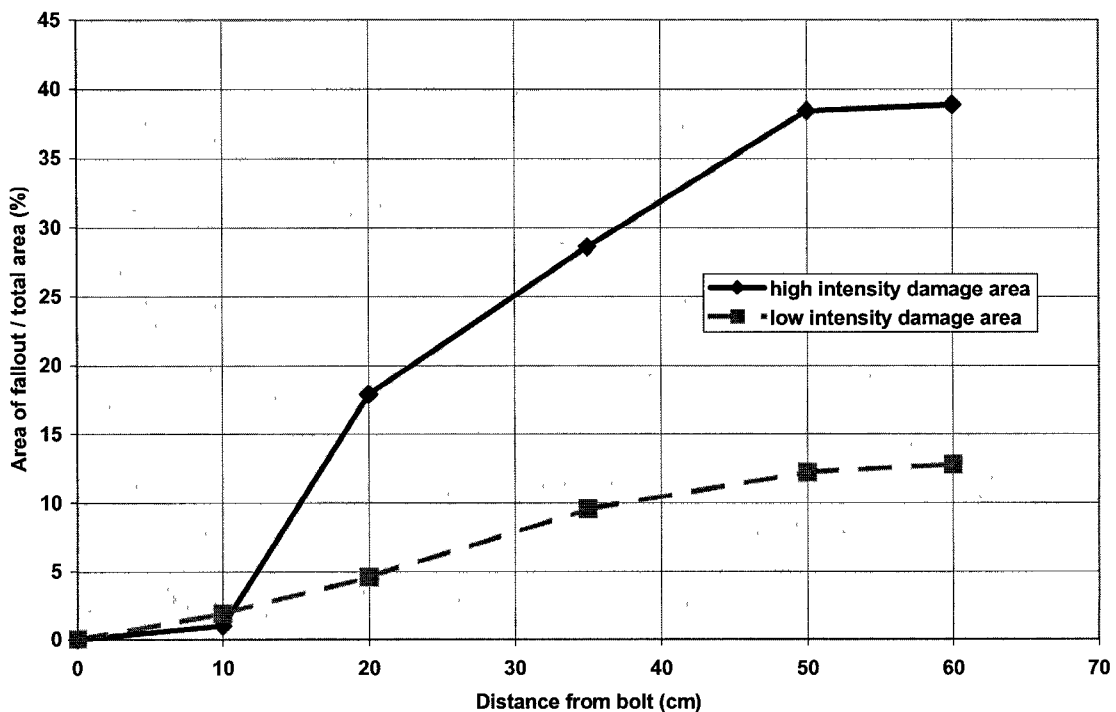
From Figure 3-43 the limit of damage is approximately 30 m from the reference point. The measured peak ground velocities on the skin of the tunnel were as shown in Figure 3-44. The minimum peak velocity associated with dynamic damage was of the order of 0.7 m/s measured at the centre of the rock bolt spacing. Analysis of the rock mass structure at this site indicated an average of 15 blocks per square metre thus representing an average rock mass class in Figure 3-42.



**Figure 3-44. Attenuation of peak ground velocity along tunnel axis and estimated limit of rockburst damage.**

If it is accepted that in these 'average' rock mass conditions, damage associated with a seismic event may be anticipated for a peak ground velocity of 0.7 m/s on the skin of the excavation, then the spacing of the rock bolts may be designed to limit rock mass unravelling. This analysis would be based on the application of the relationships as shown in Figures 3-41 and 3-42 to determine the maximum rock bolt spacing. However it would also be anticipated that the peak ground velocity limit for rock mass stability would also be dependent on the degree of rock mass discontinuity.

Analysis of the extent of damage around the rock bolt units along the tunnel has also been conducted based on the percentage of rock mass areal damage compared to the tributary area of the rock bolt. The results of this evaluation are shown in Figure 3-45. Two distinct areas of damage are again defined, that of the high intensity damage which may be equated to dynamic expulsion, and an area of low intensity damage that may be attributed to vibration induced fallout of local unstable blocks. Both trends again indicate the increased instability of the rock mass with distance from the rock bolt reinforcement unit. As illustrated in the numerical modelling analysis, and again shown by this in situ evaluation, the degree of interaction between the rock bolt and the rock mass rapidly decreases under increased dynamic loading as reflected by the gradient of the trends in Figure 3-45.

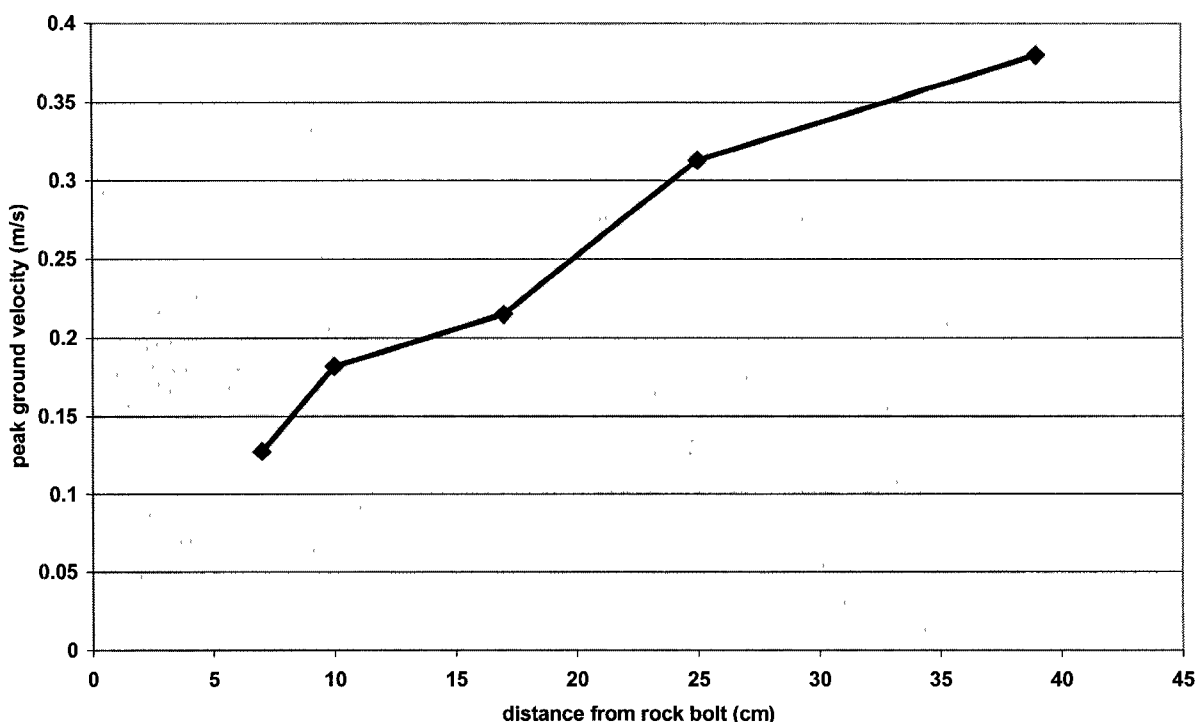


**Figure 3-45. Extent of dynamic damage relative to rock bolt reinforcement with intensity of dynamic loading.**

These observations reinforce the proposed design concepts which requires an understanding of the interaction of the components of a support system within a discontinuous rock mass structure and an understanding of the influence of the loading condition on the relative extent of interaction.

Analysis at the simulated rockburst site also examined the amplification of the peak ground velocity with distance from a rock bolt unit (Figure 3-46). Again Figure 3-46 indicates the relationship of increased excitation of the rock mass with distance from the rock bolt reinforcement. However, in this case the level of amplification is far higher than that previously

derived for the natural seismic events (Figure 3-42). Detailed analysis of the energy associated with the simulated seismic source (Hagan *et al* 1998) indicates increased energy within the compressive wave component of the dynamic energy over natural seismic events. The distribution of energy within the seismic event and its orientation relative to the excavation may explain these differences.



**Figure 3-46. Amplification of peak ground velocity with distance from rock bolt unit.**

The trend as shown in Figure 3-46, however, reinforces the general concept that the rock bolt reinforcement has a defined influence on the stability of a discontinuous rock mass structure.

### 3.5.5 Conclusions – support system interaction

Analyses conducted within this section of the investigation have clearly illustrated the importance of consideration of the interaction of the rock bolt reinforcement within a discontinuous rock mass environment. This understanding has enabled explanation of the observed deformations of the rock mass in situ and the performance of the support systems, which are not captured in the current design processes. Application of these design concepts will allow the support design engineer to estimate the interaction of the components of the support system with the rock mass and thus design their capacity for the anticipated rock mass structure and loading conditions. This work has also implied the importance of consideration of the support as a system with the rock mass and that a change in one component of the system will influence the distribution of loading, and, thus, the required characteristics of the other components of the system.

## 3.6 Performance of support system components

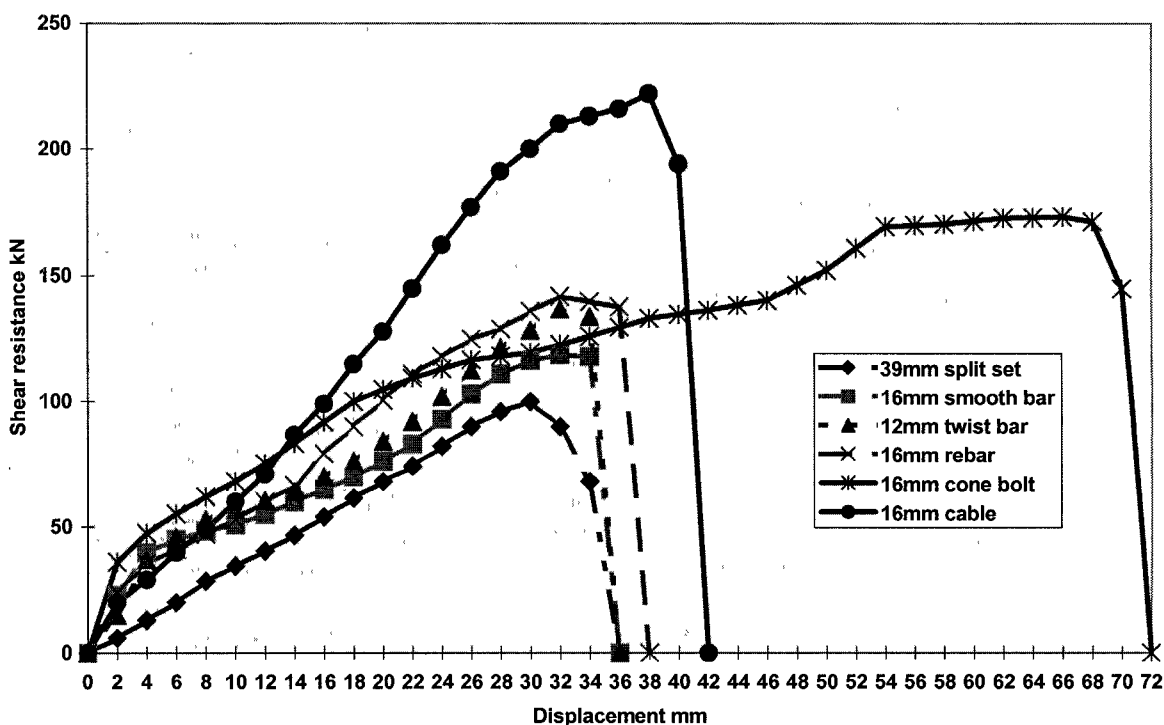
The analysis in section 3.5 has enabled an estimation of the degree of interaction between the rock bolt reinforcement and the rock mass and the unstable rock mass volume between the rock bolt reinforcement which will load the fabric component of the support system. In situ evaluations have also indicated the failure of rock bolt reinforcement and fabric support due to

the structure of the rock mass and the relative interaction between the support and the rock mass.

The work in this section of the investigation fulfils aspects of output 7 (Determination of interaction between support and the rock mass).

### 3.6.1 Shear capacity of rock bolt systems

The rockburst investigations as discussed in Section 3.2 identified the significant number of shear failures of rock bolts under dynamic loading, particularly associated with the hangingwall of the excavation (Haile, 1998). This characteristic of the rock bolt reinforcement system is not considered in the current design procedures and has thus historically not been tested in this mode of loading. In order to gain an understanding of the behaviour of a rock bolt system under shear loading, and to quantify the shear capacity of typical rock bolts, a laboratory test programme was established (Haile, 1998, Roberts, 1995). This laboratory test programme consisted of a simulated rock bolt installation incorporating the rock bolt unit, applicable grout encapsulation and a quartzite rock annulus within a single shear plane box. Both static loading and dynamic loading were applied to the apparatus and the load and deformation response of the system monitored. Typical results of this testing programme are shown in Figure 3-47.



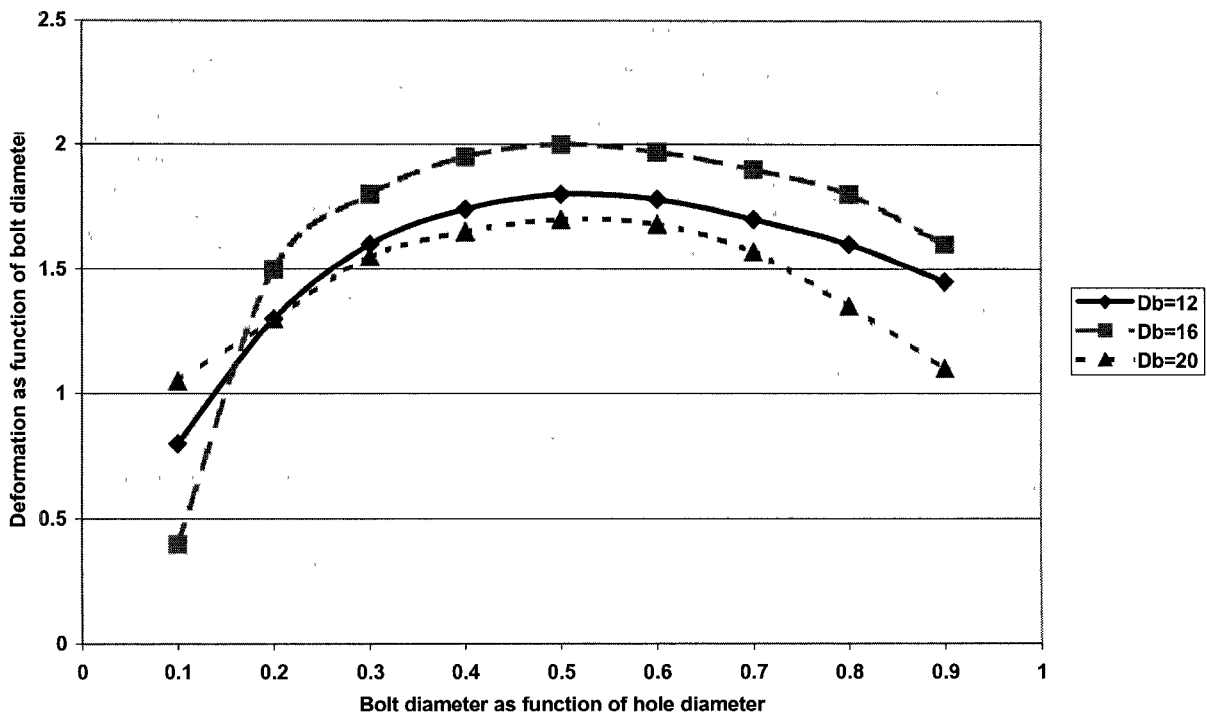
**Figure 3-47. Shear characteristics of typical simulated rock bolt installations under static laboratory shear loading.**

The theoretical maximum shear load of the rock bolt is half its tensile peak capacity. In all cases the peak shear load exceeded this theoretical level. Haile (1998) gives the specific modes of failure of the different rock bolt systems. The general mode of system failure was for debonding and crushing of the grout annulus adjacent to the shear plane and the development of plastic bending within the rock bolt unit. The amount of shear deformation and peak shear load was then a function of the ability of the rock bolt system to yield and develop a 'crank handle' profile across the shear plane. The increased ability of a rock bolt system to deform along the shear plane changed the anticipated mode of failure from guillotining due to direct shear, to tensile failure of the portion of the rock bolt bar sub-parallel to the shear plane. This mechanism of deformation would account for the measured peak 'shear' loads in excess of the theoretical shear capacity.

From Figure 3-47 it is indicated that the cone bolt and the grouted rope systems have generally higher shear capacity (energy absorption). This may be explained by the ability of these systems to yield and thus allow greater deformation along the shear plane. In the case of the cone bolt this is due to the debonded nature of the rock bolt system; and with the grouted cable it was observed that the inner layers of the cable were able to yield relative to the outer layers where failure was initiated. The limit of shear deformation of the other systems was defined by the diameter of the borehole (40 mm). At a shear deformation approaching that of the diameter of the borehole, the surfaces of the rock annulus were found to result in pinching of the rock bolt bar and rapid localisation of failure at this point.

Tests conducted under dynamic loading conditions (3 m/s) were generally found to have similar deformation characteristics although the peak load and deformation limit were slightly reduced due to the stiffer nature of the system under the rapid loading conditions. The exception to this was the cone bolt system where the deformation characteristic under dynamic loading is dependent on the position of the yielding cone relative to the shear plane. In two test configurations it was found that, with the yielding cone at a distance of 600 mm from the shear plane, failure comparable to that of the smooth bar was observed, but at 300 mm distance from the shear plane the cone bolt system yielded comparable to the static tests. The performance of the system is thus dependent on the ability of the dynamic shear energy to be transmitted to the yielding cone under the rapid loading conditions. With the in situ uncertainty of the position of the yielding cone relative to a potential shear plane, the shear performance of the cone bolt should be downgraded for dynamic loading.

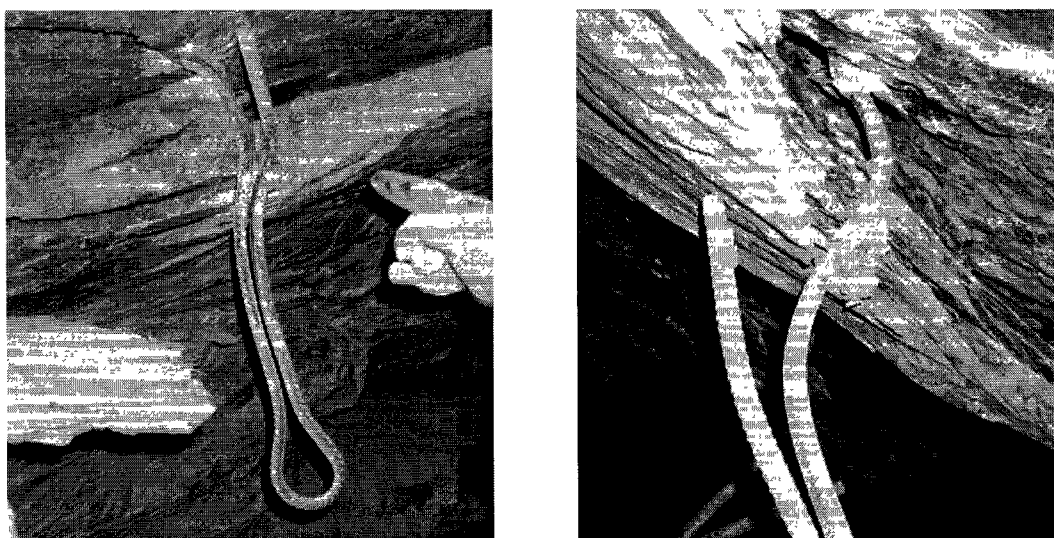
To evaluate the critical geometric parameters influencing the performance of a rock bolt system under shear loading, finite element numerical modelling was also conducted (Haile, 1998). Several factors were examined in this investigation including the influence of the type of steel, the orientation of the shear plane relative to the rock bolt axis and the relationship between the rock bolt diameter and the borehole diameter. It is considered that, due to the limitations of the rock bolt steel type and the flexibility to design the orientation of installation, the most critical design parameter is that of the diameter of the rock bolt relative to the borehole diameter. The results of this analysis are shown in Figure 3-48.



**Figure 3-48. Relationship between rock bolt diameter, borehole diameter and relative system shear deformation capacity.**

The indicated optimum rock bolt diameter for shear deformation capacity is approximately half the borehole diameter. At a lower ratio of rock bolt to borehole diameter, premature failure is due to the generation of high tensile stresses within the rock bolt for limited shear deformation, while at high ratios the mechanism of failure is premature guillotining of the rock bolt unit under more direct shear loading.

What is not captured by the numerical modelling evaluation, but was often observed underground, is the ability of the rock bolt to undergo significant shear deformation within a highly fractured rock mass (Photographs 3-7 and 3-8).



**Photographs 3-7 and 3-8. Shear deformation of rebar rock bolts in highly bedded rock mass structure.**

In this environment it is considered that the rock bolt reinforcement can accommodate significant shear deformation by small incremental shear on the numerous potential shear planes. In a more massive rock mass structure the shear energy for a given rock mass volume must be accommodated by only a few potential shear planes.

Although the shear demand on the rock bolt system has not been quantified in this investigation, the above analysis will allow the support design engineer to implement the most appropriate rock bolt reinforcement system where shear deformations are anticipated.

### **3.6.2 Load-deformation characteristics of mesh and lace support**

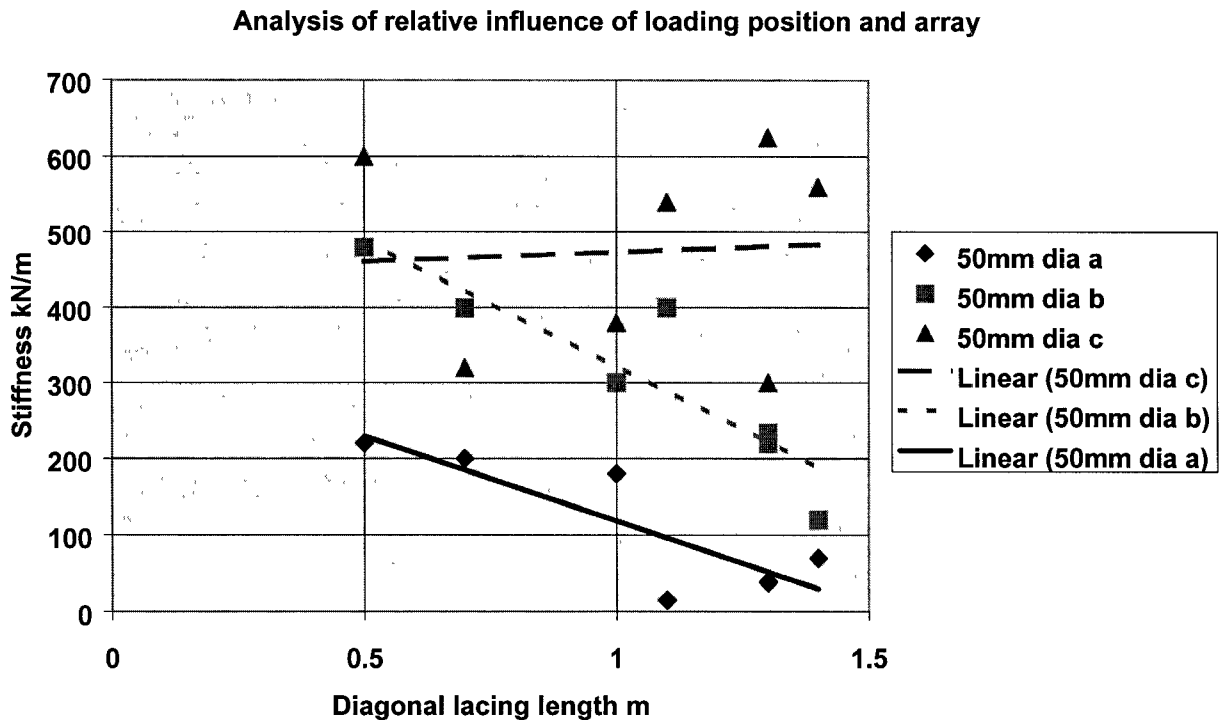
The evaluations conducted in section 3.5 have examined the concept of the extent of interaction of a rock bolt with a discontinuous rock mass structure and thus a potentially unstable rock mass volume between the rock bolt reinforcement pattern. This concept has been verified, particularly by the use of ground motion monitoring, and numerical models have been used to define the critical rock mass parameters and develop relationships to estimate the potentially unstable volume. This potentially unstable volume between the rock bolt reinforcement will define the demand on a fabric support system under a given loading condition. The characteristics of the typical mesh and lace fabric support systems were evaluated to determine their interaction with the rock mass and the rock bolt reinforcement anchorage to estimate the capacity and load transfer within the support system.

Laboratory, analytical and in situ evaluations were conducted (Roberts 1995, Haile 1998) to determine the characteristics of the typical mesh and lace support systems as utilised in the South African mining industry. The evaluation of the mesh and lacing systems considered the load-deformation characteristic, expressed as the stiffness of the system at the rock bolt



anchorage (point c), below the midpoint of the lacing strand (point b) and at the midpoint of the mesh panel (point a). The stiffness of the mesh and lace system was based on its performance perpendicular to the rockwall surface once the initial slackness in the system was taken up. In all cases this represented a deformation of approximately 7 cm, with very little load generation (Haile, 1998). This characteristic of mesh and lace systems implies that they have limited capability in maintaining the inherent strength of the rock mass and will generally act only to contain an already established unstable rock mass volume between the rock bolt reinforcement.

The typical characteristics of the mesh and lace configurations as utilised in the mining industry, based on direct, in situ pull tests of the diagonal lacing length, is shown in Figure 3-49.

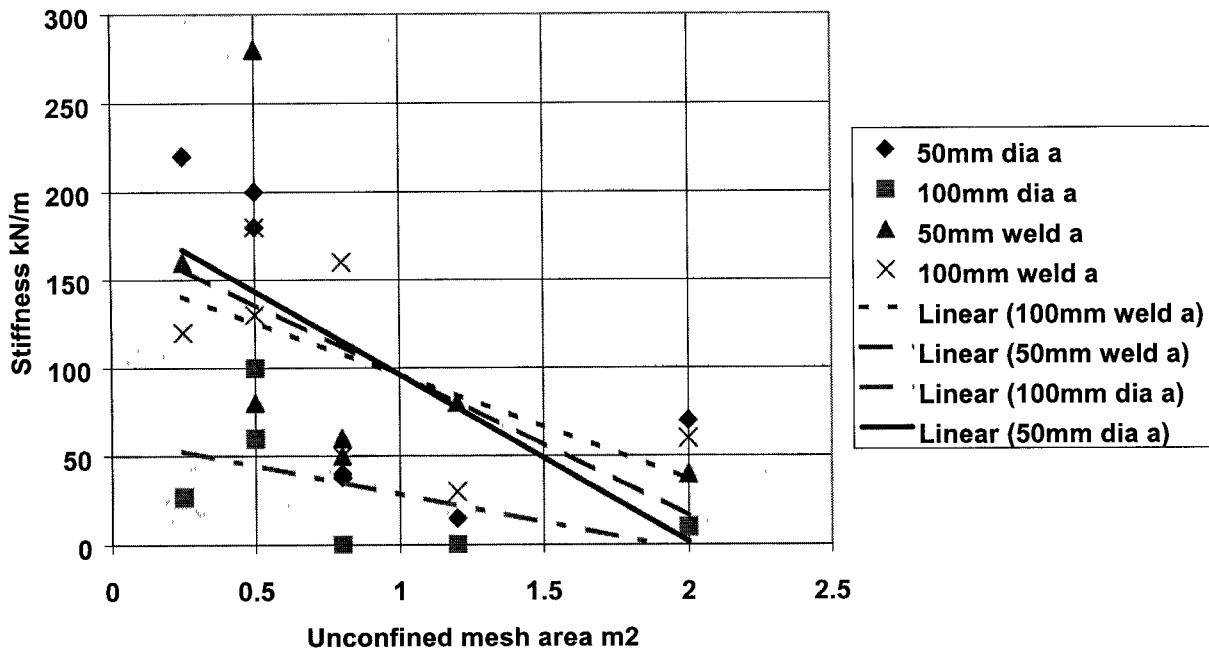


**Figure 3-49. Stiffness characteristic at defined loading points for mesh and lacing configurations based on diagonal lacing length.**

Figure 3-49 quantifies the characteristic of the mesh and lacing configurations. It also indicates the anticipated characteristic that the system has highly variable load-deformation response over its surface area. In the vicinity of the rock bolt anchorage the performance of the system is dominated by the proximity of the rock bolt constraint and the performance is fairly consistent irrespective of the overall rock bolt and lacing pattern. The stiffness characteristic of the lacing strand and mesh panel within the mesh and lacing system is influenced by the rock bolt anchorage pattern and thus lacing configuration. The stiffness response of the exposed mesh panel is substantially lower than that of the lacing contained loading point. It is typically the area of the mesh panel that will be subjected to the greatest loading due to the unstable rock mass volume between the rock bolt reinforcement. It would thus be the characteristic of this component of the system that will often determine the capacity of the system.

More detailed analysis of the performance of the mesh panel for typical weld mesh and diamond mesh fabrics was conducted (Haile, 1998). The summary results of this analysis, for different lacing configurations that define the unconstrained mesh panel area, are shown in Figure 3-50.

**Analysis of relative stiffness of mesh panel (A)**



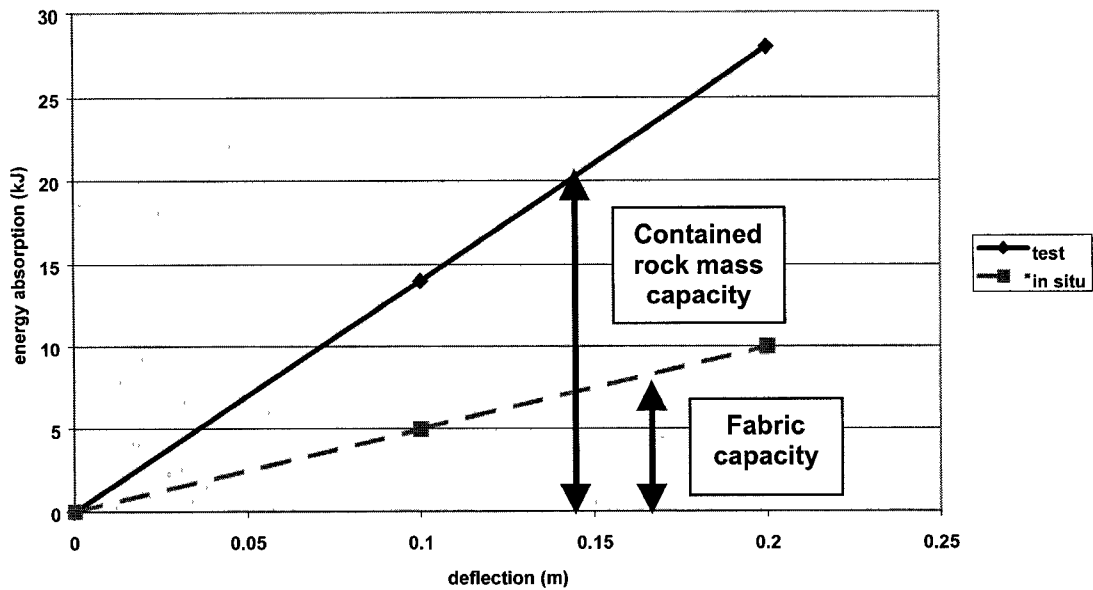
**Figure 3-50. Stiffness characteristic of the mesh panel for typical mesh types.**

From Figure 3-50 the load-deformation characteristic of the mesh panel can be estimated based on a lacing configuration which defines the mesh panel area. Analysis of this figure would indicate that there is little difference in the performance of the two weld meshes and the 50 mm diamond mesh. However, an important consideration may be the generally higher initial stiffness of the weld mesh panel in controlling the initial rock mass unravelling. The 100 mm diamond has very low stiffness. However, a more comprehensive test programme is required before quantitative performance characteristics can be defined

From the above analysis the load generated in the mesh panel for a given rock mass deformation may be determined and compared to the design capacity of the mesh fabric in order to determine the potential for failure of the fabric system. This load will also act as a surcharge load to the rock bolt reinforcement in excess of the loading, due to the direct interaction of the rock bolt with the rock mass.

In addition to the above analysis, the in situ static load-deformation data collected under this investigation was compared to dynamic tests incorporating a simulated rock mass conducted by Stacey and Ortlepp (1997). The direct energy absorption capability of the mesh and lacing system was determined from the area under the load-deformation curve. This energy was compared to that of the contained rock mass system with increasing deformation (Figure 3-51) (Haile, 1998).

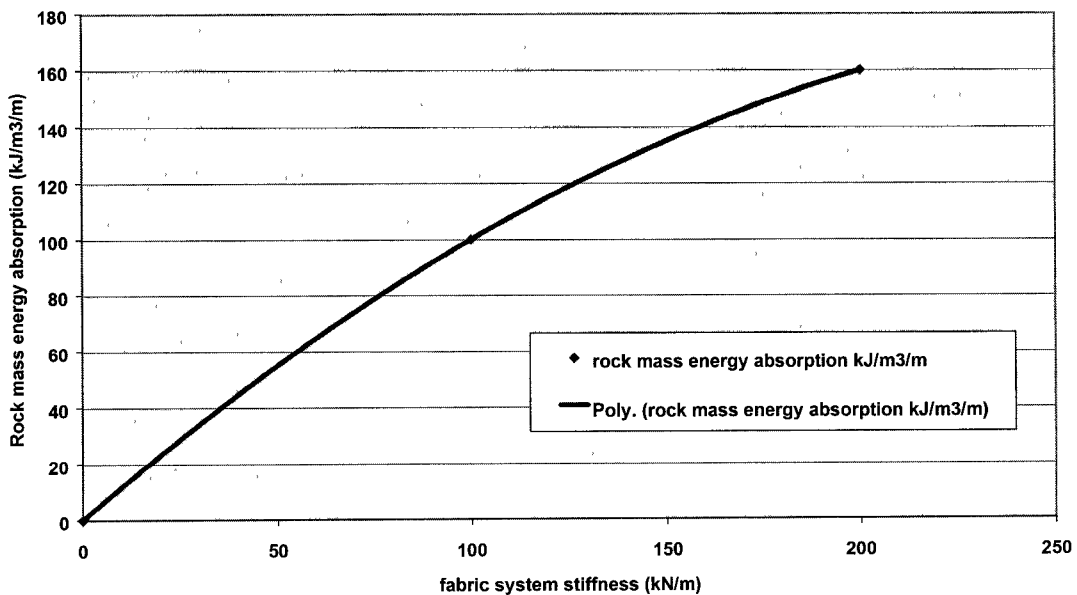
Comparison of in situ and simulated rock mass test of weld mesh and lacing



**Figure 3-51. Comparison of in situ mesh and lace fabric energy absorption capacity and contained simulated rock mass energy absorption capacity.**

The greatly increased energy absorption capacity of the mesh and lace contained simulated rock mass, for a given deflection, is considered to indicate the large energy absorption capacity of the discontinuous rock mass. Analysis of different mesh and lace configurations indicated that the energy absorption capability of this system was also a function of the stiffness of the fabric (Figure 3-52).

Analysis of discontinuous rock mass energy absorption under fabric containment



**Figure 3-52. Influence of fabric stiffness of contained simulated rock mass energy absorption characteristic per unit volume of rock mass.**

Although very preliminary in nature, due to the limited evaluation and available data, the understanding of the ability of the discontinuous rock mass volume to absorb relatively large

amounts of kinetic energy will have important implications on the design of support systems in discontinuous rock mass environments. The energy absorption within this simulated rock mass structure is considered to be due to the further breakage of blocks and the absorption of energy in the inter-block deformations and frictional heating. The fabric, however, continues to have an important role in maintaining the confinement to the discontinuous rock mass structure and thus the increased energy absorption to overcome inter-block friction under deformation. This mechanism is illustrated in Figure 3-52 for the mesh and lace systems tested and the simulated rock mass structure.

Additional support design considerations based on this analysis would imply that a discontinuous rock mass structure contained by a stiff fabric support system such as shotcrete would have a far greater energy absorption capability per unit deformation. However consideration must also be given to the stable deformation limit of the fabric structure to determine the total energy absorption capacity of the system.

### **3.6.3 Conclusions of support system component performance**

Analysis of the performance characteristics of the components of the support system and their interaction with the rock mass structure has enabled quantification of the capacity of the rock bolt system under shear deformation and the load-deformation characteristic of typical mesh and lace systems. These preliminary values may be used by the design engineer to define the most suitable support system for the envisaged rock mass environment.

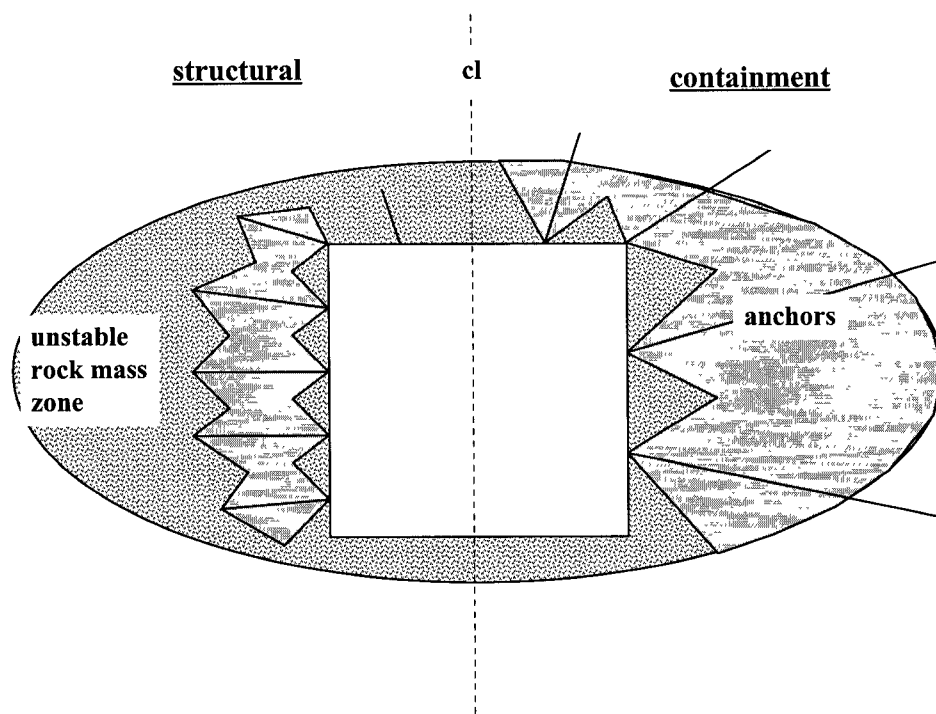
## **4 Discussion of results**

### **4.1 Support design philosophy**

For an anticipated level of rock mass instability around an excavation, the design engineer must be able to select a suitable support system for the given conditions. This will be based on the current and anticipated rock mass condition, anticipated loading conditions and allowable deformation. Also of consideration are the availability of support elements and practicalities of installation, and the cost effectiveness of the support system on the basis of material and installation costs, and support performance.

Within relatively competent rock mass structures, the role of the rock bolts is to pin potential key block structures by anchorage to stable ground in excess of the defined unstable block geometry. The purpose of this investigation is to develop support design methodologies for highly discontinuous rock mass structures where specific unstable block geometries cannot be identified and the dominant mode of rock mass instability is an unravelling of the rock mass structure.

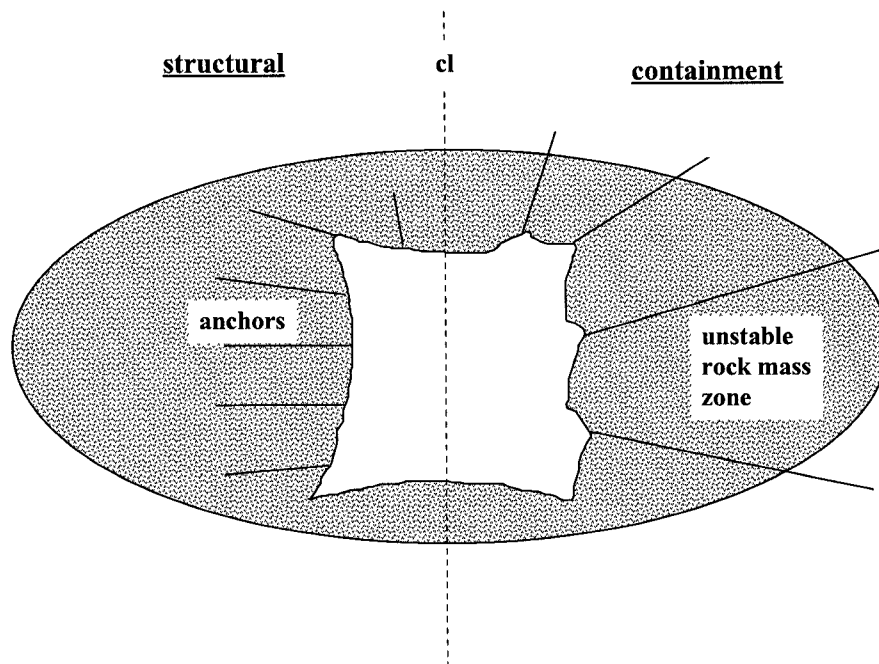
The principal design methods for support systems are either by containment or structural reinforcement. Containment of the rock mass is achieved by ensuring anchorage beyond the limit of rock mass instability, with sufficient capacity for the support system to accommodate the full rock mass loading conditions. Alternatively, the support system may act to reinforce the unstable rock mass and thus create a reinforced rock mass structure, again capable of withstanding the envisaged loading conditions. Within most support systems these support mechanisms may be combined to derive an optimum rock mass support system. Such a system may involve relatively short anchors and fabric support combined with long anchors. This would result in reinforcement of the immediate skin of the excavation with suitable anchorage to the deeper rock mass. The principal design considerations as discussed above are shown in Figure 4-1.



**Figure 4-1. Definition of principle methodologies of excavation stabilisation**

Of importance is an estimation of the depth of rock mass instability around the excavation, to enable the determination of a suitable anchor length. This may be based on data of anticipated depths of instability, empirical relationships, or numerical modelling analysis.

The importance of the influence of the mechanism of the support system interaction with the rock mass, on the deformation characteristics of the excavation, is illustrated in Figure 4-2.



**Figure 4-2. Generalised deformation characteristics for principal support methodologies under high loading conditions.**

These deformation mechanisms are clearly illustrated by the rockburst case studies, where deformations often occurred to the extent that failure of components of the support system resulted. A review of the basis of these design methodologies is given below.

The methodology for the creation of a reinforced rock mass structure will be dependent on the interaction of the support system with the rock mass to create a reinforced structure of sufficient capacity to maintain the rock wall stability under the defined rock mass environment. The behaviour of the reinforced structure under loading will result in a more uniform sidewall deformation characteristic. If, however, the reinforcement is under designed, differential deformation within the structure will occur, causing a loss of capacity of the structure to withstand increased loading. In this methodology the influence of the fabric support is to maintain the integrity of the near surface discontinuous rock mass between the rock bolt reinforcement. The required load capacity of the fabric will thus be relatively low. However, it is important that relatively stiff fabric systems be employed, such as shotcrete, in order to limit differential deformations within the structure, and thus maintain its overall integrity.

The methodology of excavation stabilisation based on retainment /containment is considered to be a more robust support design methodology, but it is critical that anchorage of the system within stable ground is achieved. Although the engineer should strive to design the support system in order to maximise the inherent strength of the rock mass structure, and thus minimise the support requirements, this methodology will allow consideration of loss of the inherent rock mass strength. This methodology therefore must carefully consider the interaction of the individual components of the support system with the rock mass, and the anticipated demand on these units. To maximise the inherent rock mass strength, for optimum design considerations, the yield capacity of the anchors should be compatible with the envisaged rock mass deformation characteristics. That is, yield of the anchors must be compatible with the dilation of the rock mass between anchor points in order to minimise differential deformations. Incompatibility will result in differential deformation, and thus loosening within the rock mass structure with resultant loss of rock mass strength. The incorporation of high quality, relatively stiff fabric support systems will result in a more even load distribution between the rock mass directly confined by the rock bolts and the potentially unstable rock mass between the rock bolt reinforcement. If the inherent strength of the rock mass is lost due to the degree of rock mass discontinuity, or deformation, then increased demand on the fabric component of the support system must be considered.

Analysis of the case studies clearly indicates the development of characteristic deformation mechanisms and also indicates the shortcomings of the current support systems. Major support system failure was due to loss of anchorage of rock bolts at approximately 60 % of the rockburst sites. This may have resulted from the limited yield and debonding, or snapping of bolts and thus significant reduction in support resistance and rock mass reinforcement; or due to inadequate anchorage depth in relation to depth of rock mass instability, and limited containment of the unstable rock mass volume. Inadequate fabric support (mesh and lacing) capacity and stiffness also resulted in failures of the support system and reduced capacity of a potentially reinforced rock mass structure at all rockburst sites.

The design of the current support system interaction with the rock mass is based on anchorage into stable ground (Anon., 1996). Where tunnels negotiate elevated stress fields due to major stoping abutments, the depth of rock mass fracturing may increase the extent of instability in excess of the length of these anchors. Under these conditions, the basis of support design should be the formation of a competent rock mass structure (beam, arch or shell). The large closures, which occur in tunnels at these sites, may be indicative of low levels of rock mass reinforcement and thus weak structural competency. This may often be a result of the current design procedures not considering this mechanism of support interaction. Under these conditions it is important to ensure sufficient interaction between the reinforcement units as opposed to relying on the fabric support to provide sufficient structural strength to the rock mass. Therefore, rock bolt reinforcement spacing should be reduced to improve rock mass interaction.

The current design recommendations for tunnels in deep level mining indicate the necessity for yielding tendons, particularly under dynamic loading conditions associated with major seismic events, to ensure energy absorption capacity. However, throughout numerous case studies, it

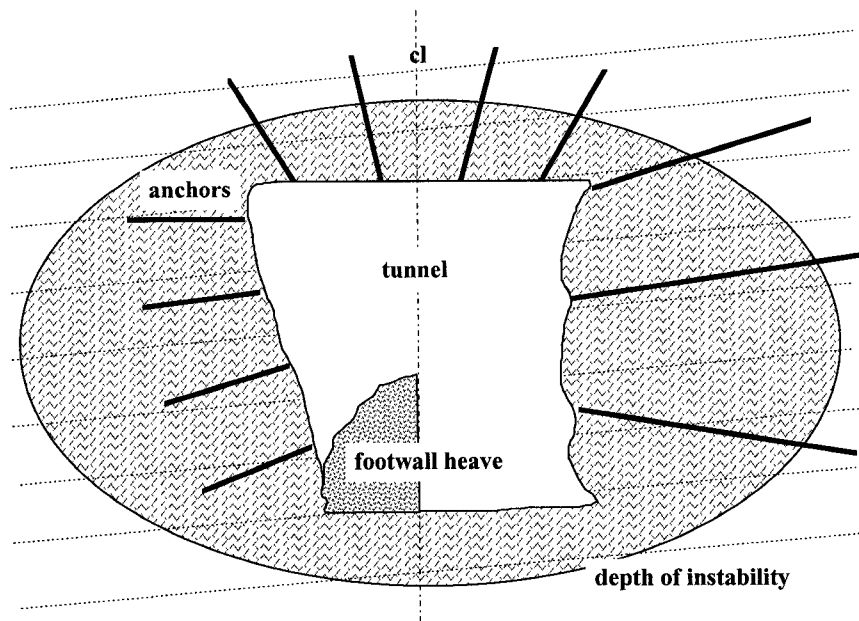
was observed that relatively stiff rebar rock bolts, with very limited yield capability and thus energy absorption, would survive the major dynamic deformations and damage. This is due to the poor interaction between the rock bolts and the rock mass in these environments. This causes the loading of the rock bolts at levels far lower than that anticipated by the design process. Under these conditions dynamic energy associated with the unstable rock mass is dissipated through the deformation of the rock mass contained by the relatively soft mesh and lace fabric support systems, resulting in the often observed bulking profile of the tunnel. This large bulking process between the rock bolt reinforcement, and thus large differential deformations, results in a further reduction of the rock bolt interaction with the rock mass. Thus this mechanism, although not catered for in the design process, tends to result in the stability of the tunnel under these loading conditions.

It must thus be appreciated that, if the stiffness of the fabric support is increased, to limit the deformations of the rock mass and thus maintain the excavation in an operational condition subsequent to such events, then the distribution of loading between the fabric and the rock bolts will change. This will result in increased direct loading of the rock bolt reinforcement to the extent that failure may occur and the total support system unravels. The importance of the yield capacity of the rock bolt system to absorb the dynamic energy is thus heightened.

The design of the support system must therefore carefully evaluate the relative demand on the reinforcing rock bolts and the fabric support. In some areas, particularly where anchorage of the tendons was maintained or had relatively higher resistance than the general peripheral rock mass, excessive bulking of the rock mass between the reinforcing rock bolts was observed. This often resulted in failure of the fabric support. It is thus important to ensure compatibility between load deformation characteristics of the tendons and the fabric support based on an estimation of the relative demand due to rock mass loading. Incompatibility, such as the use of very stiff tendons and soft fabric, will result in large relative deformations within the tunnel peripheral rock mass and associated reduced structural competency. This will lead to subsequent higher loading of the fabric support, due to a further reduction in the interaction of the rock bolt reinforcement. Under dynamic loading conditions, it was observed that in 70 % of the rockburst sites the kinetic energy associated with this unstable rock mass volume was in excess of the capacity of the typical mesh and lacing fabric support panels.

Within certain geotechnical areas, the occurrence of highly persistent, and mobile, bedding planes may result in a reduction in confinement provided by a reinforced rock mass structure. This is particularly evident if these bedding planes coincide with the hangingwall or footwall of an excavation and are of low inclination. This was often observed in the case studies conducted in the Klerksdorp gold field, where tunnels are sited in the argillaceous MB Formation quartzites. These planes of weakness are orientated sub parallel to the typical orientation of the rock bolt reinforcement. This appears to result in low interaction between the rock bolt units and the bulk of the rock mass, and also poor interaction between the rock bolts within the reinforcing system.

The observed increased deformation of the lower sidewall in comparison to the upper sidewall is considered to be a function of the presence of these dominant sub-horizontal bedding planes, in addition to a lack of continuity of the lower sidewall support into the footwall. The presence of hangingwall support, and the general upward inclination of the upper sidewall rock bolt reinforcement, can create a more competent reinforced rock mass structure and thus greater end constraint to the sidewall reinforced rock mass structure (Figure 4-3).



**Figure 4-3. A conceptual model of design methodologies and potential rock mass deformations in the presence of weak bedding planes.**

Analysis of hangingwall stability often indicated shear failure of tendons in the hangingwall of the tunnel profile. Consideration of the shear capacity of tendons would not be catered for in standard support design practice, but this has been indicated to contribute to hangingwall failures in 70 % of the rockburst case studies. Consideration of the shear capacity of tendons and cables is suggested and also that these reinforcing systems be utilised in areas where hangingwall shear, due to existence of prominent bedding and dynamic loading, is anticipated.

The relative intensity of the discontinuities within the hangingwall of an excavation had an influence on the potential of the support system to either accommodate or fail under shear loading. In a highly discontinuous rock mass, rock bolts were able to accommodate relatively large shear deformations (Photograph 4-2). It is thought that under these conditions numerous incremental shear dislocations take place along the rock bolt length, each within the shear capacity of the rock bolt. This results in dispersed direct shear interaction between the rock mass and the rock bolt and thus reduced dynamic shear loading per shear plane.





**Photograph 4-2. Shear deformation of smooth bar rock bolt.**

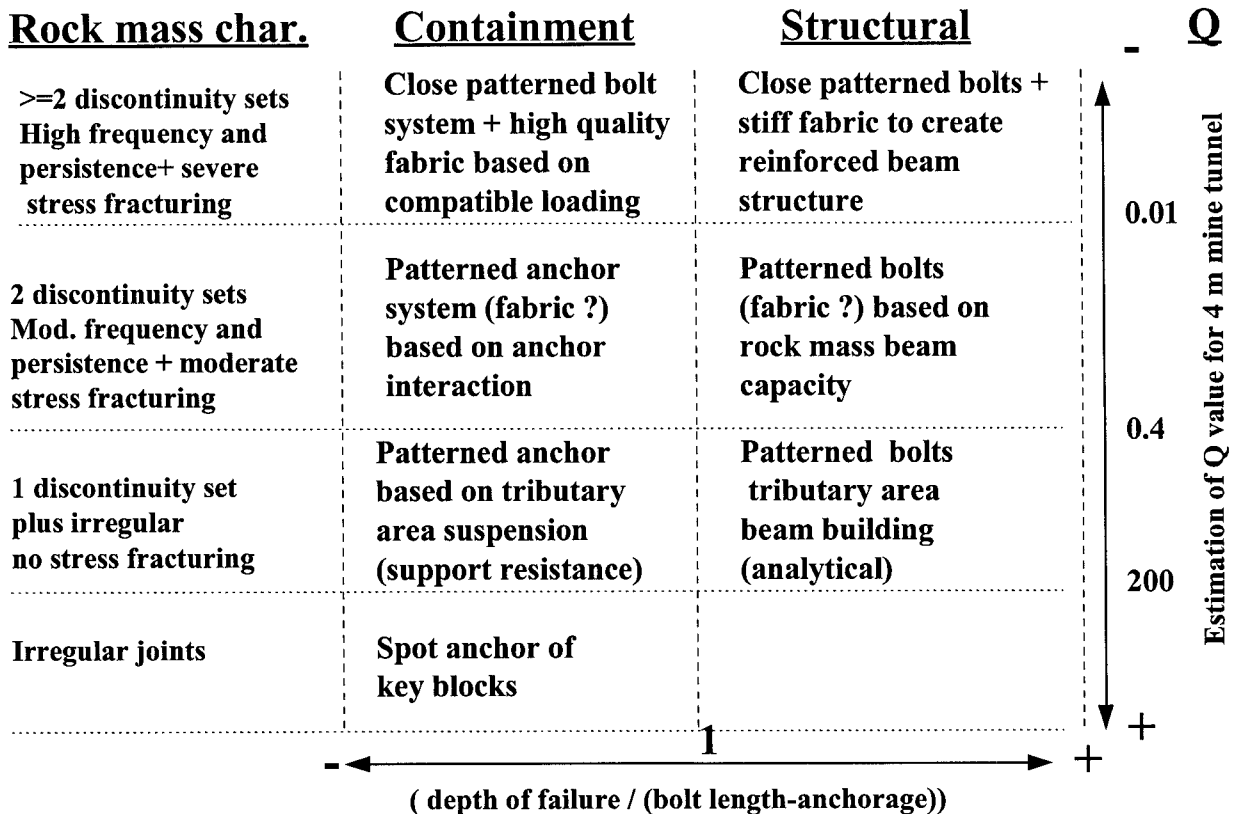
The depth of instability within the hangingwall of the tunnel excavations (3.0 m x 3.0 m) has been estimated at 1.5 m to 2.0 m, based on these case studies. With typical support systems as used in the South African mining industry, this implies that hangingwall stabilisation is primarily based on anchorage of the rock bolt reinforcement and containment of the potentially unstable rock mass, rather than structural reinforcement.

Observations of the performance of the tunnel support systems have allowed a mechanistic evaluation of the current design procedures. Of significance for the improved design of the support systems under high stress, rockburst conditions are the following:

- Consideration of the natural depth of instability of the rock mass around the excavation under dynamic loading conditions.
- Consideration of mechanisms of interaction of the rock bolt reinforcement with the unstable rock mass volume.
- Consideration of the loading of the fabric support due to the unstable rock mass volume between the rock bolt reinforcement and the influence of the characteristics of the fabric support.
- Consideration of the shear demand and capacity of the rock bolt reinforcement, particularly in the hangingwall of the tunnel.

To assist the design engineer in the selection of an appropriate design philosophy, Figure 4-4 gives a broad classification of design considerations for rock mass conditions and rock bolt reinforcement length (Haile 1998).

## Selection of support design methodology



**Figure 4-4. Selection of support design philosophy based on rock mass characteristics and rock bolt reinforcement length.**

The influence of rock mass structure on the extent of interaction of rock bolt reinforcement with anchorage within, or external to, an unstable rock mass has been examined in section 3. Design tools have been derived to enable an assessment of the degree and extent of interaction between a reinforcement unit and the adjacent rock mass for specific geotechnical environments and support characteristics. This previous analysis may be used to design a support system based on a mechanistic understanding of the interaction of the rock bolt units within a highly discontinuous rock mass structure

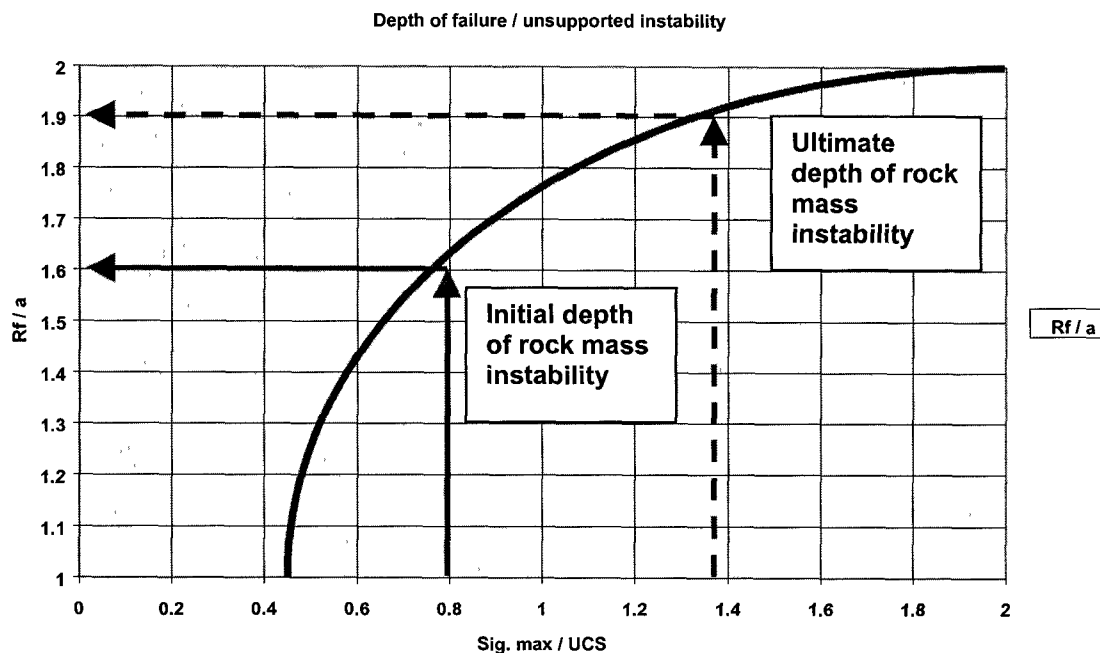
## 4.2 Application of design methodology

### 4.2.1 Evaluation of rock mass environment

The mechanism of rock mass stabilisation is dependent on the relationship between the depth of instability of the rock mass in the vicinity of an excavation and the length of the rock bolt reinforcement. Thus, for the application of the design methodologies, the length of the rock bolt reinforcement unit must be established in relation to the extent of the unstable rock mass, under the anticipated geotechnical environment and loading conditions.

The thickness of a potentially unstable rock mass may be determined from failure criteria, instrumentation, or empirical design guidelines. It is important that these methods fully reflect the design geotechnical environment and extent of rock mass instability. It is suggested that in the environment typical of deep level mines that the natural depth of instability around an excavation be determined from the application of the chart as shown in Figure 4-5. An example of the application of the methodology follows.

Let it be assumed that the excavation under consideration is a tunnel with a typical square profile of dimensions 3.5 m x 3.5 m (h), and sited in a quartzite rock mass with a uniaxial compressive strength of 200 MPa. Virgin stress conditions are estimated at approximately 60 MPa vertical and 30 MPa horizontal and the tunnel is initially isolated from the influence of stoping operations. Over the life of the excavation the mining of a stoping abutment, which results in a maximum vertical stress level of 90 MPa and a subsequent reduction to 20 MPa, will influence the tunnel. In addition, it is anticipated that the tunnel will subsequently be subject to dynamic loading due to seismicity associated with the stoping abutment. The influence of seismicity may be determined from either the maximum anticipated magnitude of seismic events and proximity to the tunnel, or directly from the maximum anticipated dynamic ground velocity. At this stage let it be assumed that the maximum anticipated seismic event is magnitude M=3.0 at a distance of approximately 50 m from the tunnel.



**Figure 4-5. Chart to determine depth of instability due to stress induced fracturing.**

The following methodology indicates the application of the charts in the determination of the rock mass characteristics in the vicinity of the tunnel.

The equivalent dimension (a) for the square tunnel is given by equation 4-1

$$a = h / \sqrt{2} = 3.5 / \sqrt{2} = 2.5 \text{ m} \quad (4-1)$$

The initial induced stress on the sidewall of the tunnel (parallel to the maximum stress vector) is given by equation 4-2.

$$\sigma_{\max} = 3\sigma_1 - \sigma_3 = 3 \times 60 - 30 = 150 \text{ MPa} \quad (4-2)$$

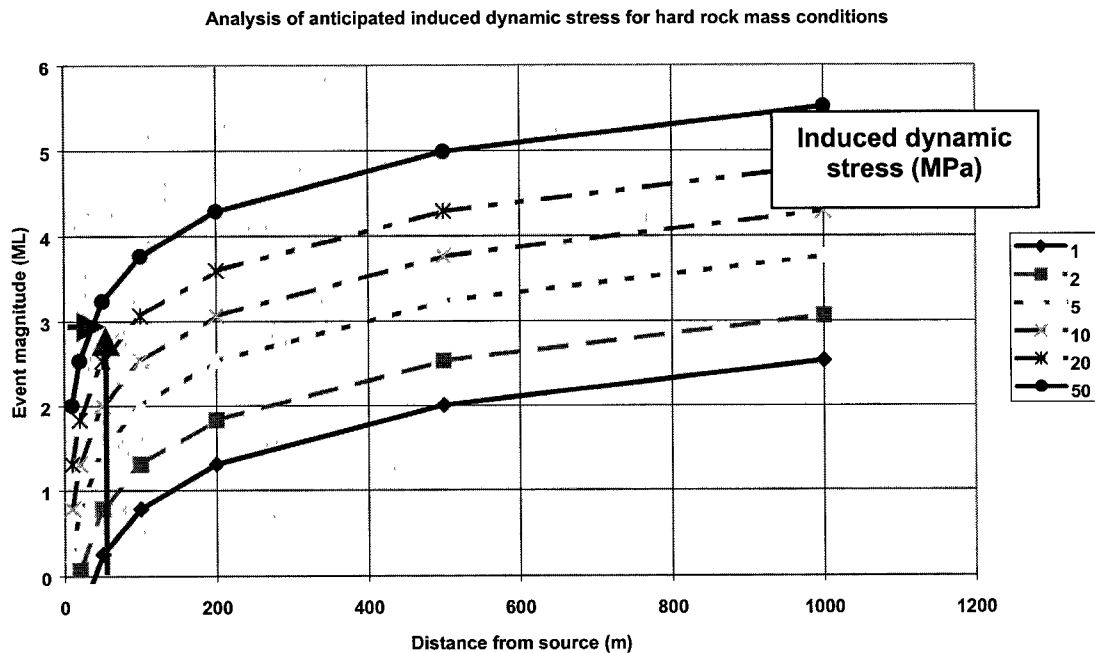
Thus sig.max ( $\sigma_{\max}$ ) / UCS in figure 4-2 is given by equation 4-3.

$$\text{sig. max/UCS} = 150 / 200 = 0.75 \quad (4-3)$$

From Figure 4-5 the initial depth of instability relative to the excavation size is given by  $R_f / a = 1.6$ . The natural depth of instability of the rock mass in the sidewall of the tunnel is given by equation 4-4 for (a) as derived from equation 4-1.

$$R_f - h/2 = 1.6 \times a - 3.5 / 2 = 1.6 \times 2.5 - 1.75 = 2.25 \text{ m} \quad (4-4)$$

It is now necessary to evaluate the increased depth of instability due to the influence of the higher induced stress levels and seismicity associated with the stoping abutment. From Figure 4-6 an estimate of the dynamic stress associated with seismicity can be made. This is a function of the maximum anticipated seismic event magnitude and the proximity of the seismic event to the excavation. From the description of the excavation environment as defined previously, the maximum induced dynamic stress, as estimated from Figure 4-6, is approximately 30 MPa.



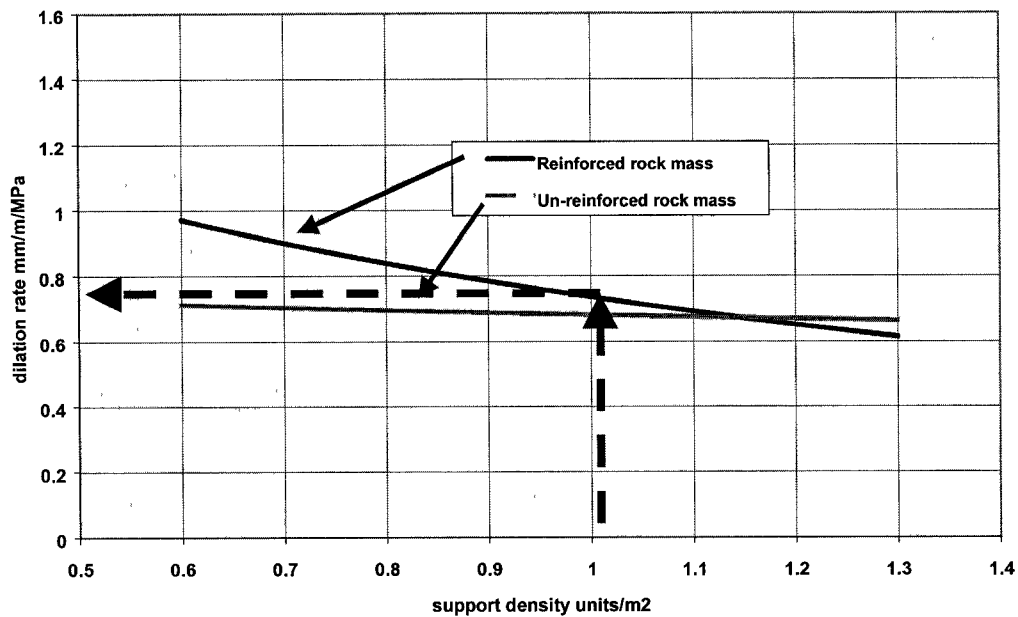
**Figure 4-6. Estimation of maximum dynamic induced stress.**

This dynamic stress is additive to the higher induced stress level due to the proximity of the stoping abutment. The maximum induced stress is thus given by equation 4-5.

$$\sigma_{\max} = 3\sigma_1 - \sigma_3 + \sigma_d = 3 \times 90 - 30 + 30 = 270 \text{ MPa} \quad (4-5)$$

Following the procedure as indicated above in equations 4-1 to 4-4, the ratio of  $\sigma_{\max} / \text{UCS}$  is 1.35, thus  $R_f / a$  is approximately 1.9 as derived from Figure 4-5. The natural depth of instability due to stress induced fracturing under the future static and dynamic stresses will be approximately 3.0 m.

The tunnel is subsequently subjected to a significant stress reduction, and, although this is currently considered not to result in a further extent of natural instability, it does result in significant further dilation of the rock mass around the tunnel excavation. An estimation of the dilation history of the rock mass in relation to the rock bolt characteristics is important in order to determine the capacity of the rock bolt system at any point during the life of the excavation. Dilation of the fracture zone around the excavation, as a function of stress change, both positive and negative, may be estimated from the charts indicated in Figures 4-7 to 4-10 as derived from section 3. Let it be estimated that the initial support density is 1 unit/m<sup>2</sup>. From the example case study, the tunnel is subjected to a vertical stress increase of 30 MPa, from its initial state of 60 MPa, to 90 MPa, and a subsequent stress reduction of 70 MPa to an overstoped stress state of 20 MPa.



**Figure 4-7. Estimation of sidewall dilation rates per metre per unit increase in vertical stress field.**

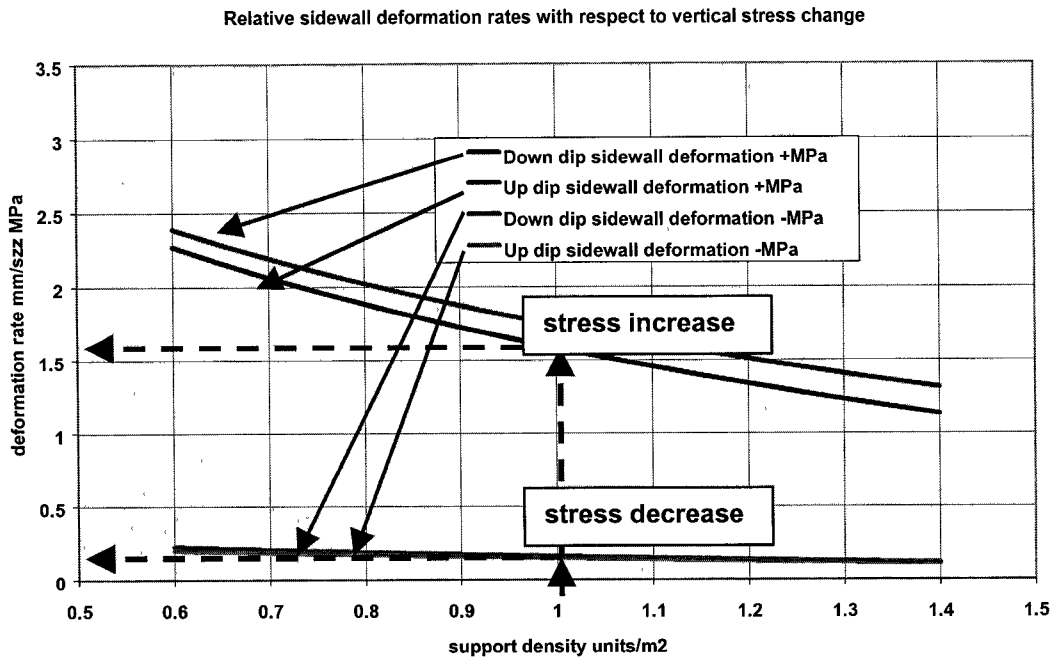
Analysis for the sidewall of the excavation indicates that the estimated dilation rate per unit increase (MPa) in vertical stress is approximately 0.75 mm per metre depth of fractured rock mass. Thus, over the vertical stress increase of 30 MPa, it is anticipated that the rock bolt reinforcement would be subjected to a strain of:

$$0.00075 \times 30 = 0.023 \text{ or } 2.3\% \text{ elongation.}$$

Over the maximum depth of natural instability (3 m) this would equate to a yield demand of approximately 69 mm.

Using Figure 4-8, the rate of dilation of the sidewall of the excavation during the period of vertical stress reduction is estimated. This is based on a factor of the rate of deformation due to vertical stress increase to that of vertical stress decrease. It is assumed that the deformation mechanism is mechanistically similar and thus dilation per unit volume of rock mass will be comparable, however, the rate is substantially lower by a factor of:

$$0.2 / 1.7 = 0.12$$



**Figure 8-5. Estimation of relative sidewall dilation rates for vertical stress increase and decrease.**

Therefore during the period of vertical stress decrease (70 MPa), it is estimated that the rock bolt reinforcement system will be subjected to a further elongation during this period of stress change of:

$$0.00075 \times 0.12 \times 70 = 0.0063 \text{ or } 0.63\% \text{ elongation}$$

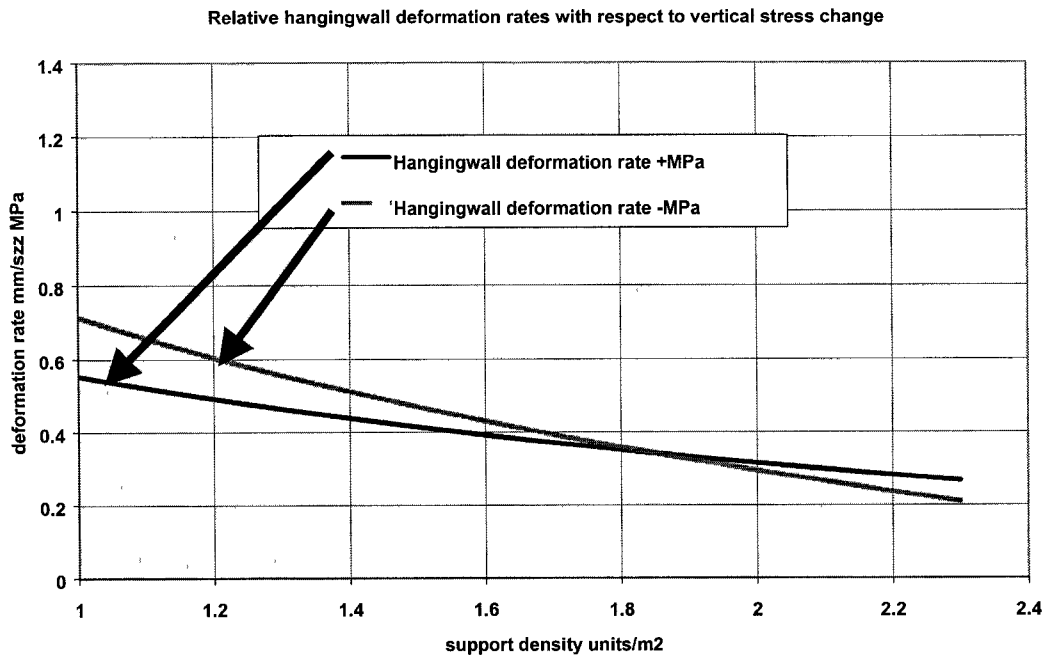
Thus over the maximum depth of natural instability (3 m), this would equate to a further yield demand of approximately 20 mm.

Analysis of the hangingwall of the excavation in this stress environment indicates limited potential for stress induced fracturing, although structural instability, or damage due to blasting practice, may be envisaged. An estimation of the extent of potential natural instability may be made from the difference between the actual excavation size and the effective excavation dimension (Kaiser *et al* 1996) as given by equation 4-6.

$$a - h/2 = 2.5 - 3.5 / 2 = 0.75\text{m} \quad (4-6)$$

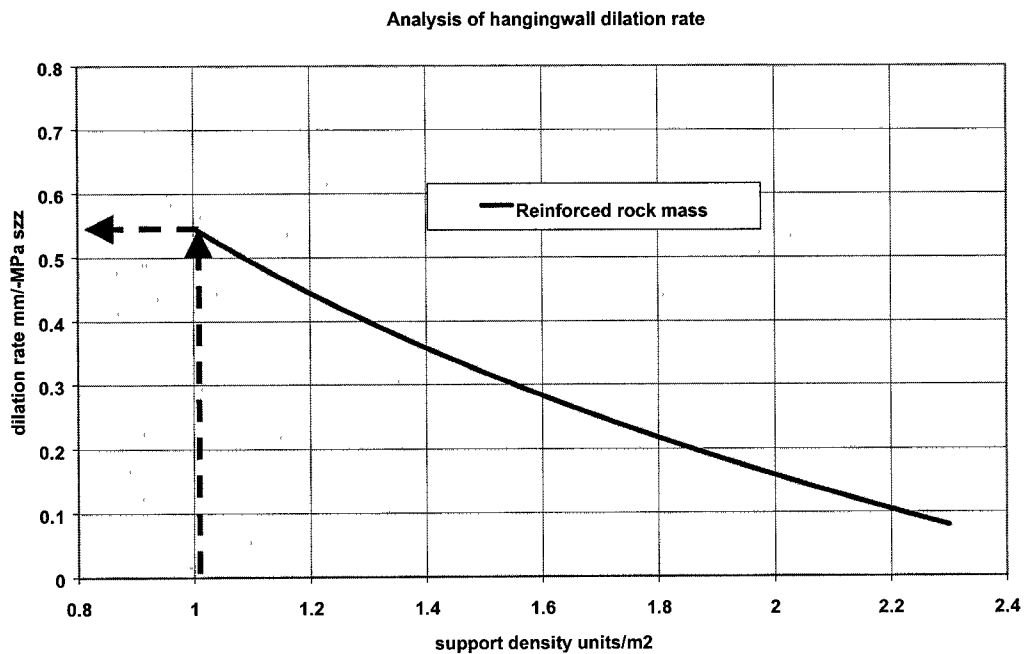
The depth of natural instability as defined by equation 4-6 is similar to that defined in the in situ analysis conducted in section 3.4.2.

An estimation of the dilation associated with the unstable hangingwall rock mass may be made from Figures 4-9 and 4-10.



**Figure 4-9. Estimation of relative hangingwall dilation rates for vertical stress increase and decrease.**

The overall deformation rates as shown in Figure 4-9 indicate a limited difference between vertical stress increase and vertical stress reduction. The influence of the reinforcement density on the dilation rate within the reinforced rock mass is shown in Figure 4-10.

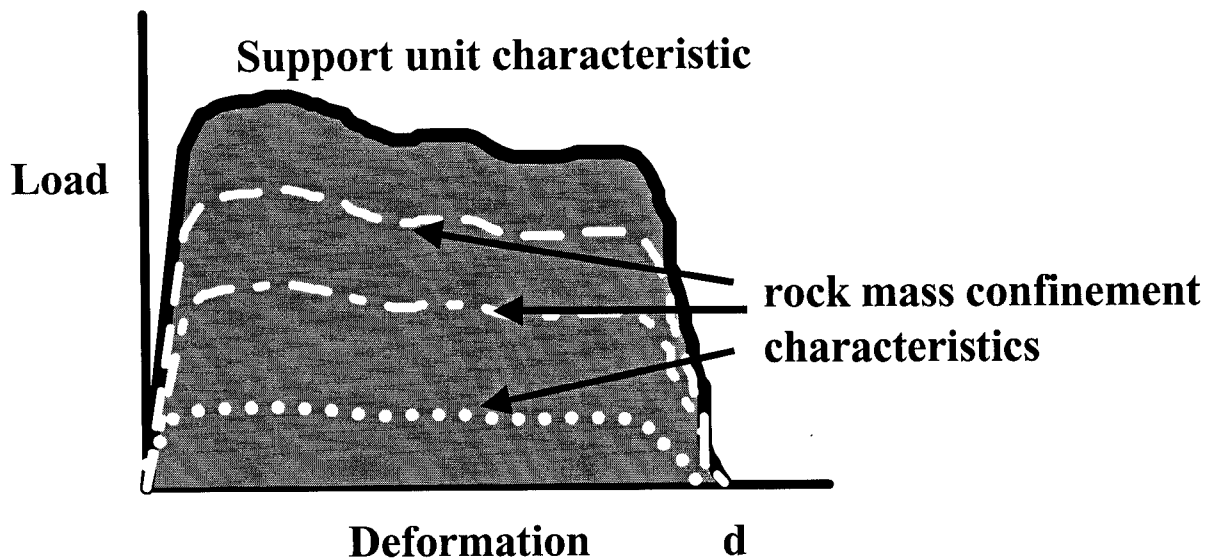


**Figure 4-10. Estimation of hangingwall dilation rates per metre per unit decrease in vertical stress field (MPa).**

Thus, in this analysis, with a vertical stress increase of 30 MPa, the anticipated dilation of the hangingwall rock mass is estimated to be 17 mm per metre reinforcement, and, for the period of vertical stress reduction of 70 MPa, it is estimated to be 39 mm per metre reinforcement. Elongation over the unstable hangingwall height of 0.75 m is thus estimated at 5.6 %.

The above procedure may alternatively be expressed graphically by representation of the deformation of the rock mass in relation to the rock bolt reinforcement load-deformation

characteristics. The loading, or confinement, of the rock mass, is currently assumed to be a function of the rock bolt reinforcement characteristics, and, thus, it is assumed that the deformation characteristics of the directly reinforced or contained rock mass are similar to that of the reinforcement unit (Figure 4-11).



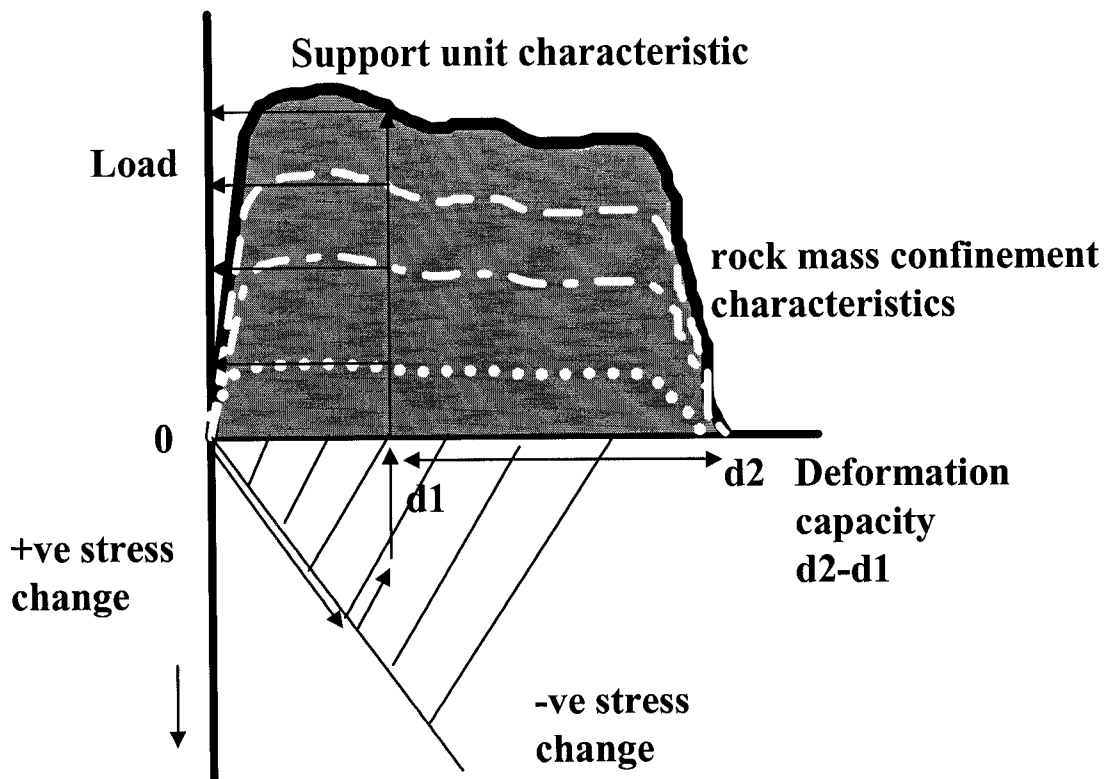
**Figure 4-11. Conceptual illustration of reinforcement and rock mass confinement characteristics.**

The energy absorption capability of the rock bolt unit and the associated relative levels of rock mass confinement are given by the area under the indicated load-deformation curves. Analysis in section 3.5 has indicated that, under external loading conditions, the volume of rock mass that is directly reinforced by the rock bolt unit is reduced. In Figure 4-11 the potential for increased instability in the rock mass is illustrated by the rock mass confinement characteristics. The lower this confinement characteristic with distance from the rock bolt axis, the greater the potential for rock mass unravelling.

The rate of progressive rock mass dilation with stress change is estimated from Figures 4-7 to 4-10 for the typical geotechnical environment of a deep level gold mine. It is assumed that the closure of the excavation may be attributed to deformation between the point of natural instability and the skin of the excavation. The deformation will thus limit the degree of remaining energy absorption capability within the reinforcement support system, principally as a function of the reduced deformation capability.

The incorporation of the defined rock mass deformation characteristics for the excavation sidewall or hangingwall, on the negative Y axis of the rock bolt load-deformation characteristic curve, will allow a graphical estimation of the remaining deformation capability, and thus energy absorption capability, of the rock bolt reinforcement system at any time in the life of the excavation (Figure 4-12).





**Figure 4-12. Estimation of remaining deformation and energy absorption capacity of rock bolt reinforcement system due to stress change.**

For the above case, the energy absorption for the zone of rock bolt confinement may be expressed as:

$$E = F.d \quad (4-7)$$

where E = energy absorption capability  
 F = average ultimate force over the deformation range  
 d = deformation capacity (remaining deformation capacity)

Once the overall characteristic of the rock mass behaviour has been defined for the excavation, over its anticipated life, the next stage of the design methodology is to determine the interaction of the components of the support system within the unstable rock mass volume.

## 4.2.2 Design based on rock mass reinforcement

Application of the design concepts and charts as derived in section 3.5 are applicable to a blocky (fractured), interlocking rock mass where the major discontinuity within the rock mass is sub-parallel to the boundary of the excavation. In this analysis both rock mass containment / retainment and rock mass structural reinforcement are evaluated. The initial steps of the analysis are based on the determination of the natural extent of instability around the excavation and the anticipated quasi static deformation due to the stress history over the life of the excavation as derived in section 4.2.1.

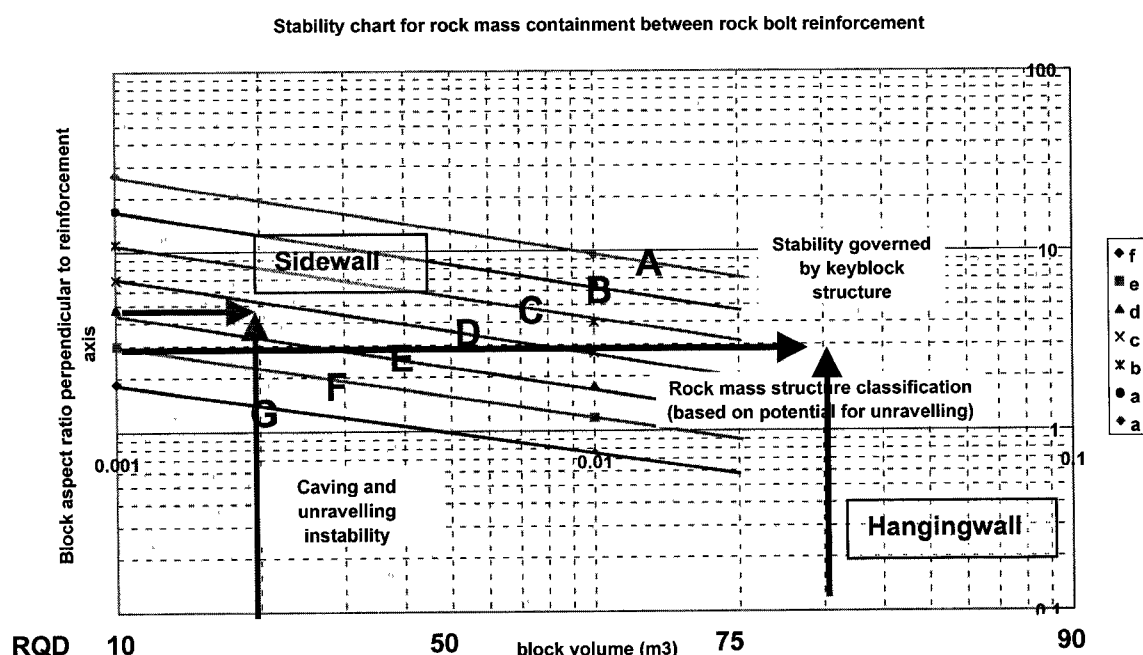
### 4.2.2.1 Application of design procedure for rock mass containment / retainment

Using this design consideration, the length of the rock bolt reinforcement is sufficient to ensure adequate anchorage in excess of the depth of natural rock mass instability around the excavation. This is not necessarily the total thickness of stress fractured rock. Classification of

the rock mass is based on the geometry of the blocks within the rock mass structure relative to the rock bolt installation. Classification is based on the chart as shown in Figure 4-13.

Also indicated on this chart are equivalent RQD (Rock Quality Designation) values as derived from the work of Palmstrøm (1996). He developed a rock mass classification system (Rock Mass Index -RMI) which gives consideration to the block volume and a correlation to RQD.

An example of a typical blocky rock mass structure due to stress induced fracturing and dynamic loading is shown in Photograph 4-3. The dimensions of the blocks that comprise the rock mass structure are approximately 30 cm in length (L) (horizontal) and 15 cm width (W) (vertical) with a thickness (T) of approximately 5 cm (fracture intensity within the rockwall). This would give an average estimated block volume of 0.002 m<sup>3</sup> and an average aspect ratio in the horizontal direction of six and in the vertical direction of three. The difference in the aspect ratio of the blocks relative to the rock bolt reinforcement will result in different levels of rock mass instability in the orthogonal directions. If a regular rock bolt pattern is to be analysed then an average aspect ratio (4.5) may be used for simplified analysis.

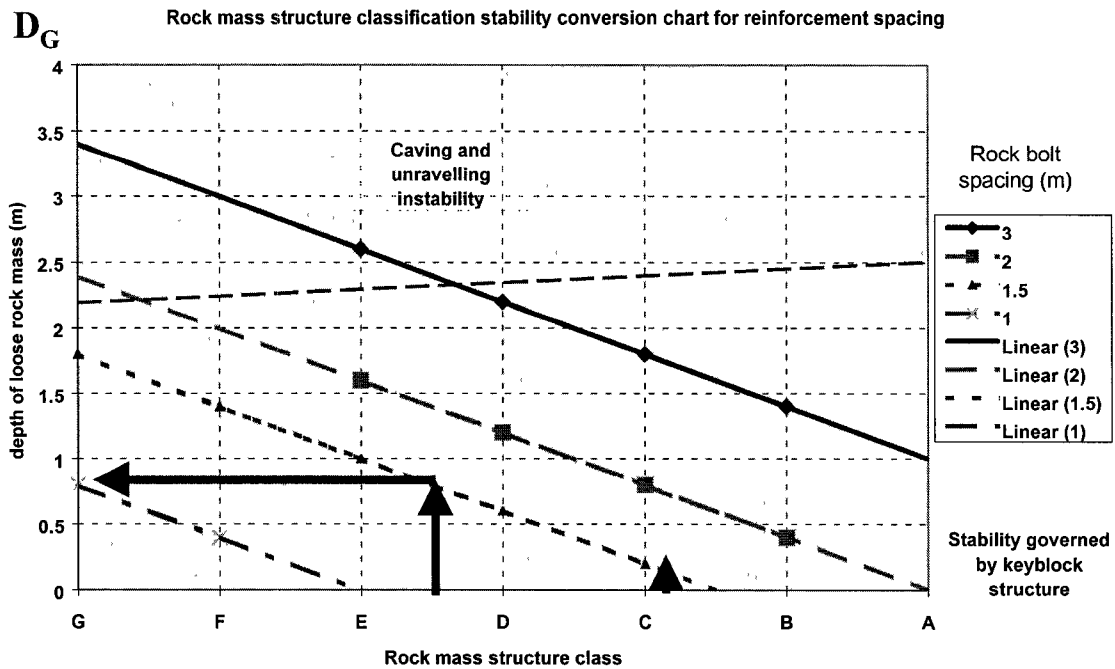


**Figure 4-13. Classification of rock mass based on block volume and aspect ratio relative to the rock bolt reinforcement with equivalent RQD values after Palmstrøm (1996).**

Based on a required assessment of only the average depth of instability between the rock bolt reinforcement, a rock mass class of D/E would be used for further analysis, based on evaluation of the rock mass structure in Photograph 4-3.

Estimation of the rock mass structure in the hangingwall of the excavation is based on an assumed block dimension of approximately 0.1 m thickness and 0.3 m width. This would give an average volume of 0.03 m<sup>3</sup> and a block aspect ratio of three. From Figure 4-13 this would define approximately a class B/C rock mass.

Classification of the rock mass now allows an estimation of the volume of rock mass instability between the rock bolt reinforcement for the specified, or initial design estimation, rock bolt reinforcement spacing (Figure 4-14).



**Figure 4-14. Determination of depth of instability due to unravelling of the rock mass under gravitational loading.**

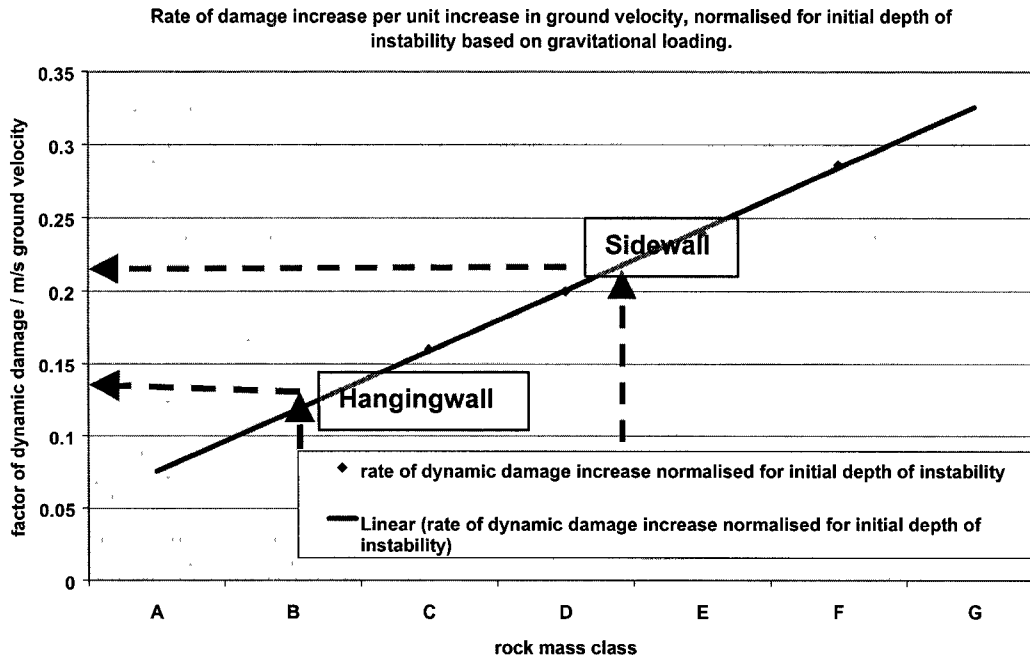
In the hangingwall of the excavation, only structural instability would be anticipated (estimate 0.25 m).

For the example rock mass environment, it is also anticipated that the excavation will be subjected to dynamic loading due to the close proximity of a seismic event. An estimation of the increased potential for unravelling of the rock mass under these conditions is thus necessary. This is based on the anticipated ground velocity that the excavation peripheral rock mass will be subjected to under the conditions of dynamic loading (Figure 4-15). This chart is based on the relative increase in the depth of unravelling as a function of that under gravitational loading. For analysis of the sidewall of the excavation, the equivalent ground velocity that will result in instability equivalent to that due to gravitational loading should be considered first. This must then be subtracted from that of the anticipated maximum ground velocity in order to estimate the ultimate depth of rock mass instability between the rock bolt reinforcement.

In this example (Figure 4-15), for a rock mass class D/E, the factor of increased unravelling due to dynamic loading in excess of gravitational instability is approximately 0.22 times the initial depth of instability per unit increase in dynamic ground velocity (m/s) ( $F_D$ ). The maximum depth of unravelling between the defined rock bolt reinforcement spacing is estimated from equation 4-8.

$$D_{\max} = D_G (1 + F_D \cdot V_{\max}) \quad (4-8)$$

where  $D_{\max}$  is the maximum anticipated depth of instability and  $V_{\max}$  is the maximum anticipated dynamic ground velocity (in excess of gravitational loading). For the sidewall it is assumed that the maximum dynamic ground velocity (estimated to be 2 m/s), in excess of the equivalent gravitational loading of 0.7 m/s, is 1.3 m/s. The maximum depth of unravelling between the rock bolt reinforcement is thus estimated to be approximately 1.0 m (equation 4-8).



**Figure 4-15. Rate of increased unravelling instability due to dynamic loading.**

A similar evaluation for the hangingwall rock mass would consider the full dynamic loading of 2 m/s, in addition to gravitational loading. This gives an anticipated maximum depth of unravelling between the hangingwall reinforcement of 0.31 m.

The above analysis can be conducted for orthogonal planes within the support system, and, thus, enable determination of the potential for unravelling of the rock mass in three dimensions. Based on a simplified geometry (Haile, 1998), the area of potential unravelling between the rock bolt reinforcement may be estimated. The generalised solution for this is given in equation 4-9:

$$Vol_l = \frac{1}{2} \cdot D_{max} \cdot S_{max} \cdot S_{min} + \frac{2}{6} \cdot S_{min} \cdot (D_{min})^2 \cdot \tan(90 - \tan^{-1}(D_{max}/(\frac{1}{2} \cdot S_{max}))) \quad (4-9)$$

where  $Vol_l$  is the volume of rock mass unravelling per unit spacing of the rock bolt reinforcement, and  $D_{max}$  and  $D_{min}$  are the depth of unravelling in the two orthogonal directions,  $D_{max}$  being the larger of the two.  $S_{max}$  and  $S_{min}$  are the respective spacing of the rock bolt reinforcements.

If however  $D_{max}$  is indicated to be in excess of the natural depth of instability, and if  $D_{min}$  is in excess of the natural depth of instability, then include the function contained in brackets [], and let

$$\alpha = D_{min} \cdot \tan(90 - \tan^{-1}(D_{max}/(\frac{1}{2} \cdot S_{max}))) \quad (4-10)$$

Then

$$Vol_l' = \frac{1}{2} \cdot ((D_{max} - D_N)^2 \cdot S_{max}/D_{max}) \cdot S_{min} + [2 \cdot \frac{1}{2} \cdot ((D_{min} - D_N)^3 \cdot S_{min}/D_{min}^2) \cdot S_{min} \cdot \alpha + \frac{1}{3} \cdot (\frac{1}{2} \cdot ((D_{min} - D_N)^2 \cdot S_{min}/D_{min}) \cdot (1 - (D_{min} - D_N)/D_{min}))] \quad (4-11)$$

where  $D_N$  is the natural depth of instability in the absence of rock bolt reinforcement, as derived in section 4.2.1. The volume of rock mass instability between the rock bolt reinforcement is then given as

$$Vol_U = Vol_l - Vol_l' \quad (4-12)$$

In the example application, the depth of unravelling in the sidewall of the excavation, based on a regular rock bolt pattern, is given by (4-8)

$$\begin{aligned}\text{Vol}_U &= 0.5 \times 1 \times 1.5 \times 1.5 + \frac{2}{6} \times 1.5 \times 1^2 \times \tan(90 - \tan^{-1}(1 / (0.5 \times 1.5))) \\ &= 1.5 \text{ m}^3\end{aligned}$$

For the hangingwall of the excavation the anticipated volume of unravelling would be estimated at 0.47 m<sup>3</sup>.

Alternatively, this analysis may be utilised to define a spacing of the rock bolt reinforcement that will prevent complete unravelling of the rock mass between the rock bolts by comparison of the depth of unravelling instability ( $D_{\text{max}}$ ) with the natural depth of instability ( $D_N$ ).

The volume of the rock mass that is directly reinforced by the rock bolt units ( $\text{Vol}_R$ ) is calculated as the difference between the tributary area volume ( $\text{Vol}_T$ ) and the volume of unravelling ( $\text{Vol}_U$ ):

$$\text{Vol}_R = \text{Vol}_T - \text{Vol}_U \quad (4-13)$$

For the example application for the sidewall and hangingwall of the excavation, the volume of direct loading on the rock bolt unit, for the maximum developed depth of instability, is estimated to be 5.25 m<sup>3</sup> and 1.78 m<sup>3</sup> respectively.

The direct loading on the rock bolt reinforcement and the fabric support may now be estimated based on the analysis as conducted in section 4.2.3.

#### **4.2.2.2 Application of design procedure for rock mass structural reinforcement**

The relative effectiveness of excavation stabilisation based purely on rock mass structural reinforcement is also examined. With this design, the length of the rock bolt reinforcement is less than the depth of natural rock mass instability around the excavation.

Classification of the rock mass is based on the same system as discussed in section 4.2.2.1 and shown in Figure 4-13.

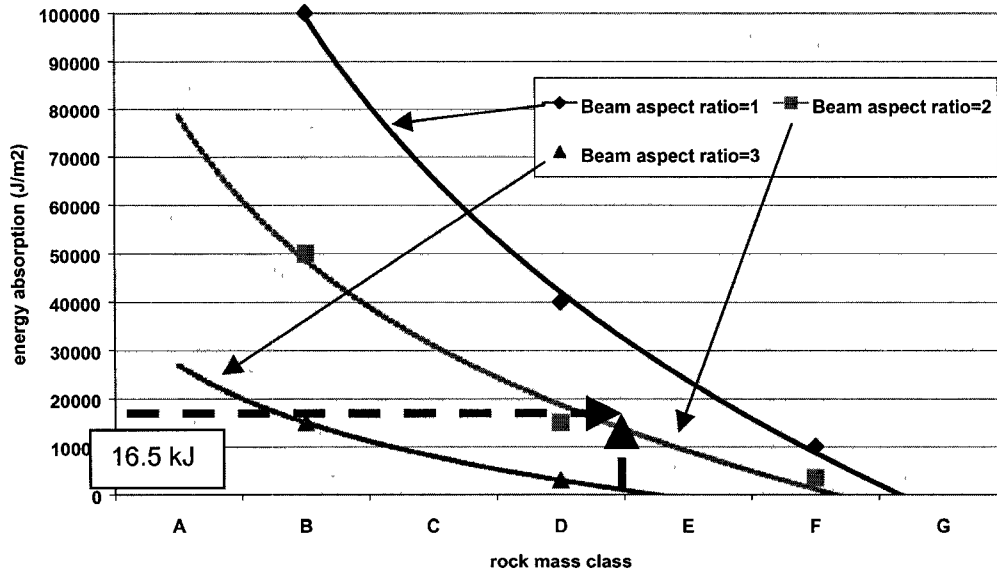
The analysis of the effectiveness of the rock bolt reinforcement within the defined rock mass structure is based on the performance of the overall system as evaluated by its energy absorption capability at the mid-point of the structure, express as kJ/m<sup>2</sup>. Additional analysis can however also be conducted to determine the requirement for, or capacity of, fabric support within the support system.

The total unstable volume and the defined loading condition (static or dynamic) define the loading of the structure. Generally, in the environment of the deep level gold mines, the mechanism of structural reinforcement is only applicable to the sidewalls of the excavation, due to the increased depth of instability in this area in comparison to the hangingwall. Thus loading of the structure is a function of either progressive dilation of the rock mass (section 4.2.1) or dynamic loading. Under conditions of dynamic loading, the energy demand on the structure is assumed to be given by the kinetic energy of the unstable rock mass volume for the maximum anticipated ground velocity:

$$\text{Kinetic energy} = \frac{1}{2} \cdot \rho \cdot D_N \cdot v^2 \quad (4-14)$$

If the depth of instability in the sidewall of a typical 3.5 m x 3.5 m excavation is 3 m ( $D_N$ ), under dynamic loading, with a maximum anticipated ground velocity of 2 m/s ( $v$ ) and a rock mass density of 2750 kg/m<sup>3</sup> ( $\rho$ ), then the energy demand will be approximately 16.5 kJ/m<sup>2</sup>. If we assume a comparable rock mass structure as shown in Photograph 4-3, then a support system can be defined from Figure 4-16 for the defined energy absorption requirement.

General relationship between rock mass class and reinforced (1 unit/m<sup>2</sup>) beam aspect ratio based on relative energy absorption capacity

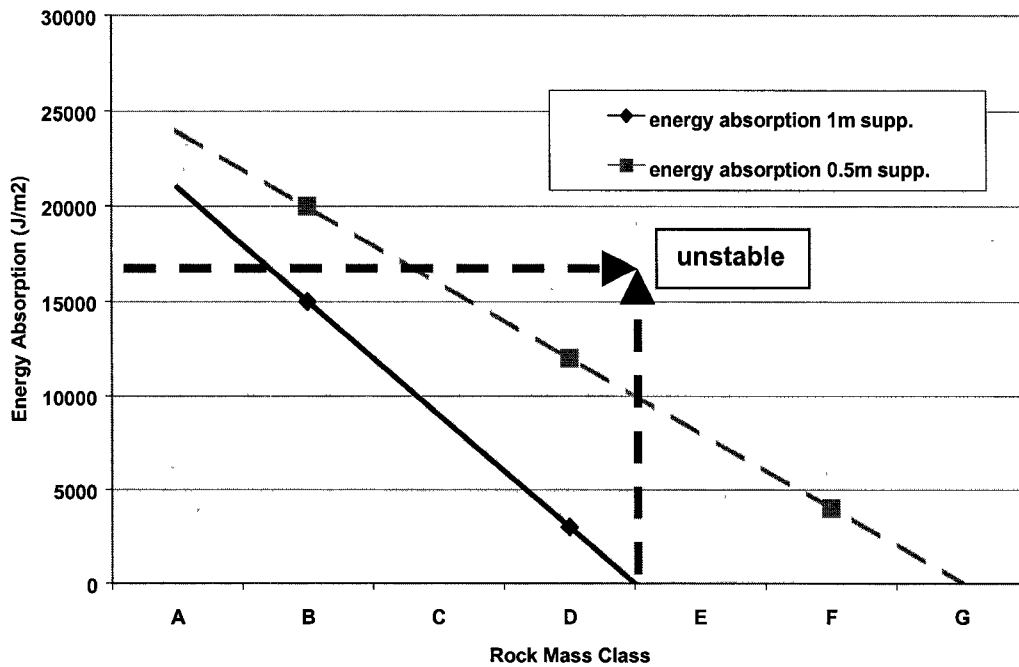


**Figure 4-16. Reinforced rock mass structure for defined rock mass class and energy demand.**

Thus, at a reinforcement density of 1 unit/m<sup>2</sup>, a beam aspect ratio of approximately 2:1 is required. For a typical 3.5 m x 3.5 m tunnel this would indicate a rock bolt reinforcement length of 1.75 m. At a rock bolt reinforcement spacing of 1 m, the extent of instability between the rock bolts would only be due to structural instability (Figure 4-14). This therefore will only necessitate the use of light mesh and a low density lacing pattern to maintain the stability of the surface rock mass.

At a beam aspect ratio of 3:1, an increased reinforcement density to the level of 2 units/m<sup>2</sup> is still indicated to be insufficient to maintain structural integrity of the reinforced rock mass (Figure 4-17).

Analysis of Rock Mass Class on Energy Absorption for beam aspect ratio 3:1



**Figure 4-17. Analysis of stability of reinforced 3:1 aspect ratio beam for defined rock mass class and energy demand.**

This simple analysis thus allows a comparison of the relative merits of the support design philosophy based on the mechanistic interaction of the rock bolt reinforcement units within the rock mass structure.

### 4.2.3 Fabric demand and support system interaction

This section examines the envisaged loading of the components of the support system in relation to their load deformation characteristics, and, thus, the ability to evaluate, or design, the support system based on the proposed mechanistic approach. This process is examined again by means of an example.

The quasi static loading environment due to the progressive stress changes over the life of the excavation has been considered in section 4.2.1. In this evaluation it is assumed that the dilation of the rock mass acts across the whole support system and does not result in differential deformation across the support system. Consideration could be given to this differential deformation by evaluation of the extent of interaction of the components of the support system at the initial stress state and at any point during the stress history. This analysis, and a detailed understanding of the influence of the support resistance on the degree of dilation of the rock mass, may allow an estimation of differential bulking factors which will result in differential dilation over the rockwall of the excavation. However, for the purposes of this analysis, let it be assumed that the rockwall undergoes uniform deformation as a function of the quasi static stress history. This therefore results in limited initial loading of the fabric of the support system but may result in significant deformation of the rock bolt reinforcement. The ability of the rock bolt reinforcement to absorb additional dynamic energy, at any time in its history, will be a function of its initial load and deformation capacity minus the deformation utilised in controlling the quasi static dilation of the rock mass. More detailed analysis may also consider the influence of prior seismic loading, during the excavation history, on the current status of the support system. This could be captured by site specific analysis that encompasses the average dynamic dilation rates within the general dilation characteristic. The specific consideration of the rate and influence of prior seismicity is not considered in this analysis.

Based on the example rock mass environment, previous evaluations have considered a tunnel in an initial vertical quasi static stress environment which increases from 60 MPa to 90 MPa due to the advance of a mining abutment. This is anticipated to result in seismic events of maximum magnitude  $M=3.0$  at a source distance of approximately 50 m, which is anticipated to result in a maximum ground velocity on the skin of the excavation of approximately 2 m/s. The excavation will then be subjected to a vertical stress reduction of approximately 70 MPa due to the over mining of the stoping abutment. For purposes of illustration of the design methodology, consider the extent of interaction of the components of the support system as derived under section 4.2.2.1 for untensioned rock bolt reinforcement.

From the characterisation of the unstable rock mass around the tunnel excavation based on the above environment, and a 1.5 m x 1.5 m rock bolt pattern, it was defined that the rock bolt reinforcement, under dynamic loading of the sidewall of the excavation, would contain approximately 5.25 m<sup>3</sup>. This would thus result in approximately 1.5 m<sup>3</sup> being unstable between the rock bolt reinforcement, which would cause loading of the fabric support. The ultimate depth of natural instability in the sidewall of the excavation is estimated to ultimately be approximately 3 m. A similar analysis of the hangingwall of the excavation indicated the unstable volume between the rock bolt reinforcement to be approximately 0.47 m<sup>3</sup> and the volume directly reinforced by the rock bolt units to be 1.78 m<sup>3</sup> over an unstable depth of approximately 1 m.

In a dynamic environment the criterion for evaluating, and designing, tunnel support systems is based on the energy absorption capacity of the components of the support system. This is evaluated in relation to the anticipated energy demand due to the kinetic energy associated with the unstable rock mass volume, under the dynamic loading conditions.

The initial step in the methodology for evaluation of the loading of the components of the support system is to estimate the anticipated demand on the fabric support system between the defined rock bolt reinforcement pattern. Consider first the anticipated demand on the hangingwall area of the tunnel. The energy demand due to the unstable rock mass volume between the rock bolt reinforcement is given by:

$$E = m g d + \frac{1}{2} m v^2 \quad (4-15)$$

Where: E = energy demand on components of the support system (J)

g = constant gravitational demand on rock mass

m = mass of unstable rock loading the components of the support system ( $\rho \times \text{vol.}$ ) (kg)

d = deformation capability of support component (m)

v = anticipated peak ground velocity perpendicular to excavation boundary (m/s)

$$E = 2750 \times 0.47 \times 9.81 \times d + 0.5 \times 2750 \times 0.47 \times 2^2$$

$$E = 13000d + 2600 \text{ (J)}$$

The energy demand on the fabric support is evaluated against the capacity of the fabric contained rock mass. The energy absorption capacity of a typical weld mesh and lace fabric containment system has been examined in Chapter 6 of this document. Traditionally, the consideration for support system design has been that of tributary area loading of the support component and is therefore a function of its load – deformation characteristic. However, the analysis in section 3.6.2 (Figure 3-52) examined the concept that a large proportion of the energy absorption associated with the deformation and containment of an unstable rock mass volume may be associated with the deformation processes which occur in a discontinuous rock mass medium. This was estimated to be of the order of 160 kJ/m<sup>3</sup>/m for the specific rock mass structure contained by a typical weld mesh and lacing support (Figure 4-18). The energy absorption capacity of the system under evaluation, based on typical load deformation characteristics of fabric support systems, may be expressed as:



$$E = \frac{1}{2} \cdot F \cdot d + E_{RM} \cdot \text{vol}_U \cdot d \quad (4-16)$$

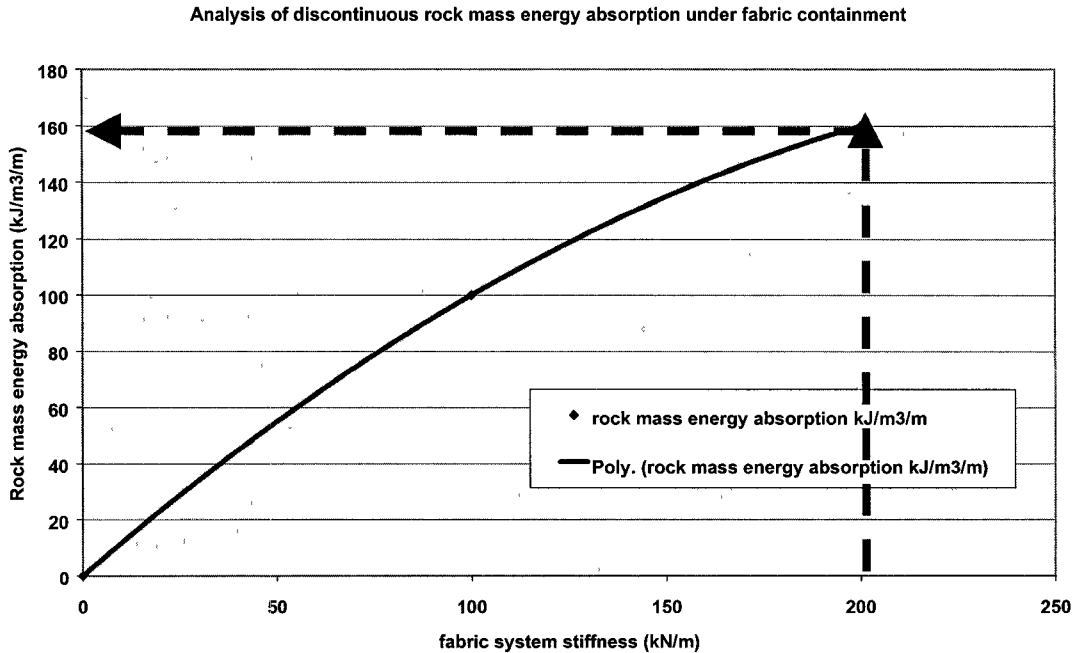
Where: E = energy absorption capacity of contained rock mass system (kJ)

F = normal force on fabric support system (kN)

d = deformation of fabric support system (m)

$E_{RM}$  = estimated energy absorption capacity of the specific discontinuous rock mass (160 kJ/m<sup>3</sup>/m)

$\text{Vol}_U$  = volume of unstable rock mass under analysis



**Figure 4-18. Estimation of energy absorption capacity of contained discontinuous rock mass based on a typical mesh and lace fabric support stiffness of 200 kN/m.**

It is estimated that the stiffness of a typical weld mesh and lacing system at the defined rock bolt reinforcement spacing of 1.5 m is approximately 200 kN/m, based on analysis of the lacing strand (Figure 4-19). It is assumed that this approximately represents the overall stiffness of the analysed mesh and lacing system. Thus, by substitution in equation 4-16, the energy capacity of the fabric support system is expressed as:

$$E = 0.5 \times 200 \times d^2 + 160 \times 0.47 \times d \\ = 100d^2 + 75d \text{ (kJ)}$$

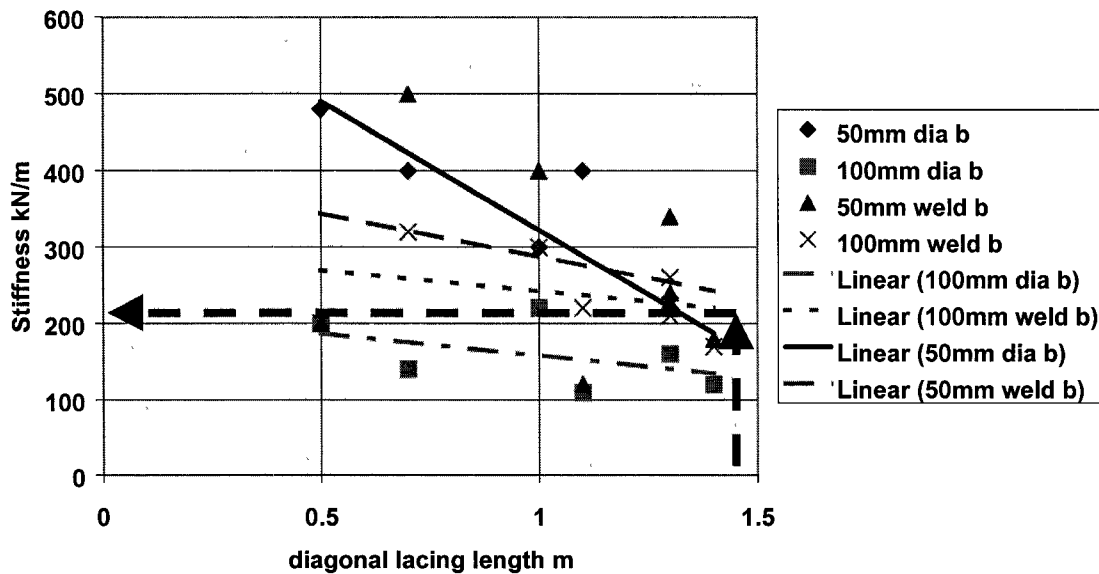
Balancing of the energy demand and energy capacity from equations 4-15 and 4-16 gives:

$$13d + 2.6 = 100d^2 + 75d$$

therefore

$$d \cong 0.04 \text{ m}$$

**Analysis of relative stiffness of diagonal lacing length (B)**



**Figure 4-19. Estimation of weld mesh and lace fabric system stiffness for rock bolt spacing of 1.5 m. (Haile, 1998)**

It is therefore anticipated that the fabric support will deflect approximately 40 mm under the defined loading condition. Substitution back into the normal stiffness, representative of the mesh and lacing system, would indicate an additional normal force of approximately 8 kN due to dynamic loading. This dynamic loading will be additional to any gravitational loading of the fabric and also result in additional loading on the rock bolt reinforcement. The original gravitational loading of the fabric for the example rock mass structure of the hangingwall is estimated to be approximately 9 kN due to structural instability ( $Vol_U = 0.32 \text{ m}^3$ ). The peak normal loading in the fabric under conditions of dynamic loading is thus approximately 17 kN. The additional dynamic deformation of the fabric may be partially recovered after the seismic event, but this will be a function of the fabric and rock mass characteristics and is not considered in this investigation.

A similar process is applied to the rock mass directly reinforced by the rock bolt units. It is assumed that the rock bolts have a yield characteristic similar to that illustrated in Figure 4-20. Thus, over the majority of the deformation range, the energy absorption capacity can be expressed as the average yield force, say 140 kN, multiplied by the deformation.

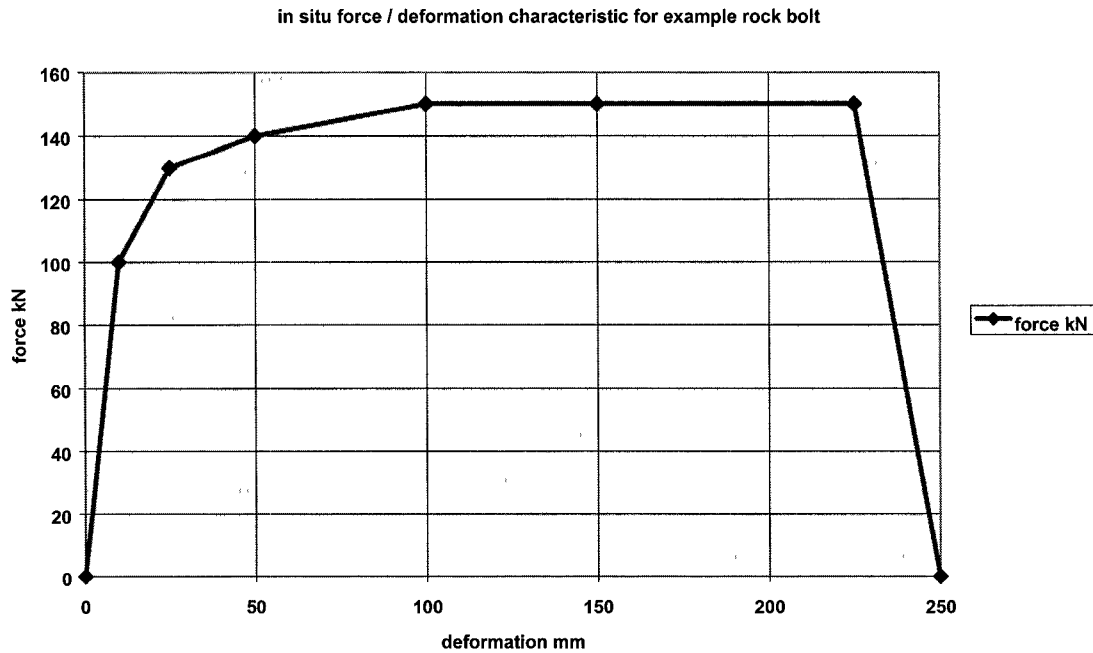
$$E_R = F d \tag{4-17}$$

Where:  $E_R$  = energy absorption of the rock bolt over the defined deformation (kJ)

$F$  = average yield force (140 kN)

$d$  = deformation, or elongation, of the rock bolt (m)

Due to the reinforcing mechanism of the rock bolt on the rock mass, the energy absorption capability of the discontinuous rock mass volume is not considered in this analysis. The reinforcement of the rock mass by the rock bolt unit is considered to limit the normal and shear deformation within the rock mass and thus the energy absorption capacity of this component of the system. The laboratory test work conducted by Ortlepp and Stacey (1997) only incorporated the energy absorption of a fabric contained rock mass structure. With regard to the rock bolt reinforced rock mass system, the omission of this capability may result in a conservative design consideration.



**Figure 4-20. Example rock bolt load – deformation characteristic**

The energy demand on the rock bolt unit due to the 1.78 m<sup>3</sup> of unstable rock mass volume is given by equation 8-18:

$$E = \frac{1}{2} (F_1 + F_E) d + m g d + \frac{1}{2} m v^2 \quad (4-18)$$

where  $F_1$  and  $F_E$  are the initial and final additional forces (kJ) transferred to the rock bolt due to the fabric support system during the period of dynamic loading. For the defined loading environment and support system the energy demand on the rock bolt is:

$$E = 0.5 \times (9 + 17) d + 2.75 \times 1.78 \times 9.81 d + 0.5 \times 2.75 \times 1.78 \times 2^2$$

$$E = 61 d + 10$$

Therefore balancing the energy demand and the capacity from equations 4-17 and 4-18, the deformation of the rock bolt system is:

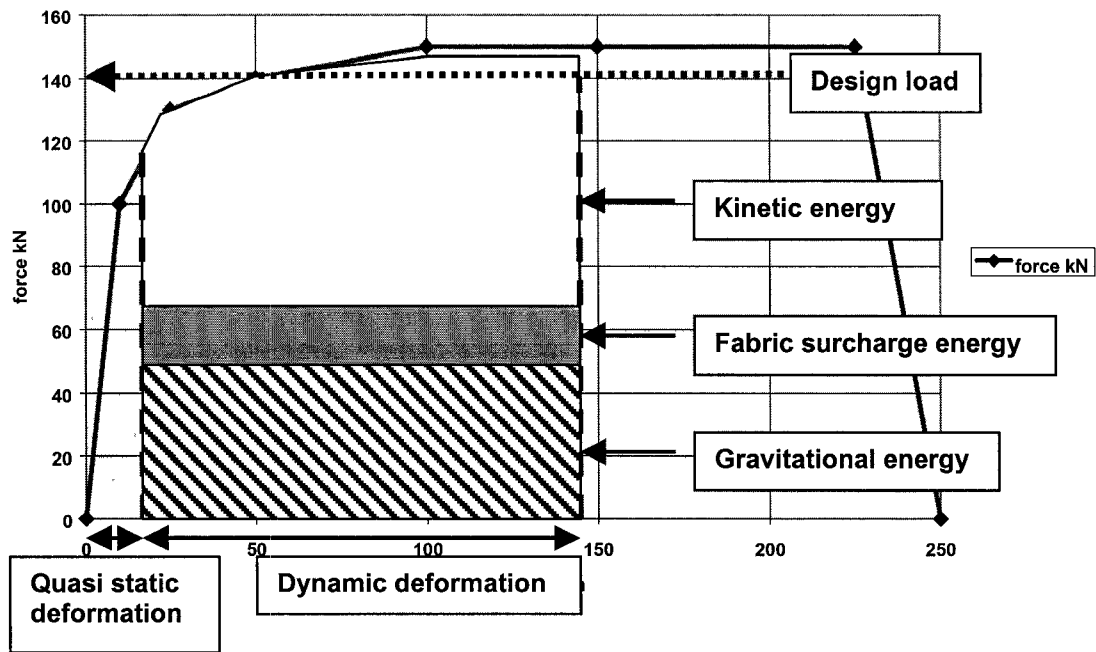
$$140 d = 61 d + 10$$

$$d = 0.13 \text{ m}$$

As discussed above, it is considered that this deformation may be conservative (over estimation) due to the omission of the consideration of any energy absorbing potential of the discontinuous rock mass volume directly reinforced by the rock bolt unit.

From the analysis in section 4.2.1, it is estimated that the hangingwall of the excavation will undergo progressive deformation of approximately 17 mm due to the vertical stress increase from 60 MPa to 90 MPa. With consideration purely of this quasi static deformation and the deformation associated with the analysed seismic event, the total deformation of the rock bolt system will be 147 mm. The deformation and energy absorption distribution due to the analysed seismic event is illustrated in Figure 4-21.

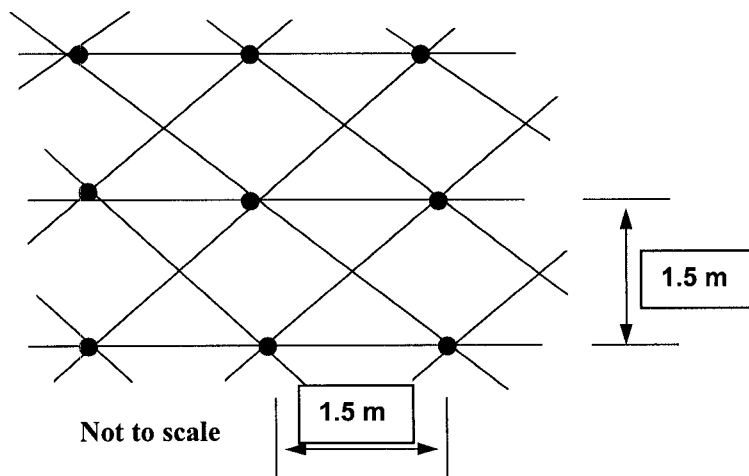
in situ force / deformation characteristic for example rock bolt



**Figure 4-21. Distribution of deformation and dynamic energy demand / absorption.**

For the hangingwall of the excavation it is thus estimated that, for the defined rock mass environment and support system, the rock bolt reinforcement will deform to approximately 130 mm, and the mesh and lace fabric will deform a further 40 mm relative to the rock bolt anchorage.

The deformation of the mesh and lace fabric is well within its capacity. However, it may be anticipated that the deformation of the mesh panels (1.1 m<sup>2</sup>) defined by the lacing pattern will deform more due to lower stiffness (Figure 3-49). For the defined rock bolt pattern and typical lacing configuration as illustrated in Figure 4-22, the relative stiffness of the mesh panel is approximately 40 % that of the diagonal lacing strand. For purposes of determining the load transfer to the rock bolt reinforcement, it is considered more suitable to base the fabric deformation analysis on the mesh and lace system derived from the lacing stiffness.



**Figure 4-22. Typical lacing configuration for square rock bolt reinforcement pattern.**

Based on the reduced volume acting on an individual mesh panel and the reduced confined rock mass energy absorption capability as estimated from Figure 4-18, for a mesh panel stiffness of 80 kN/m, it is estimated that the deformation of an isolated mesh panel would be approximately 85 mm. This deformation is within the capacity of a mesh panel; thus failure would not be anticipated.

This analysis has indicated that, for the hangingwall of the excavation, the rock bolt reinforcement at a spacing of 1.5 m, and of a length to ensure stable anchorage in excess of an unstable depth of 1 m, must have a yield capability of at least 0.15 m.

The stability of the sidewall of the excavation is now evaluated. In this evaluation gravitational loading of the rock mass is not considered as the analysis of dynamic expulsion is based on expulsion perpendicular to the vertical rockwall. Thus the loading consideration, in excess of the quasi static deformation of the rock mass, is purely due to the dynamic loading.

In the sidewall of the excavation, the volume of unstable rock mass between the rock bolt reinforcement has been estimated to be approximately 1.5 m<sup>3</sup>. The energy absorption demand on the fabric, for the defined dynamic loading environment, is given as:

$$E = \frac{1}{2} m v^2 \quad (4-19)$$

Thus

$$E = 0.5 \times 2.75 \times 1.5 \times 2^2 = 8 \text{ kJ}$$

The energy absorption capacity of the mesh and lace contained rock mass system, as determined from equation 4-16 is:

$$E = 100 d^2 + 240 d$$

Balancing of the energy demand and energy capacity indicates an anticipated deformation of the mesh and lace system, relative to the rock bolt anchorage, of approximately 35 mm. Based on a mesh and lace system stiffness of 200 kN/m, this would give a peak dynamic loading force on the rock bolt attachment of approximately 7 kN.

The demand on the rock bolt reinforcement in the sidewall of the excavation is given by:

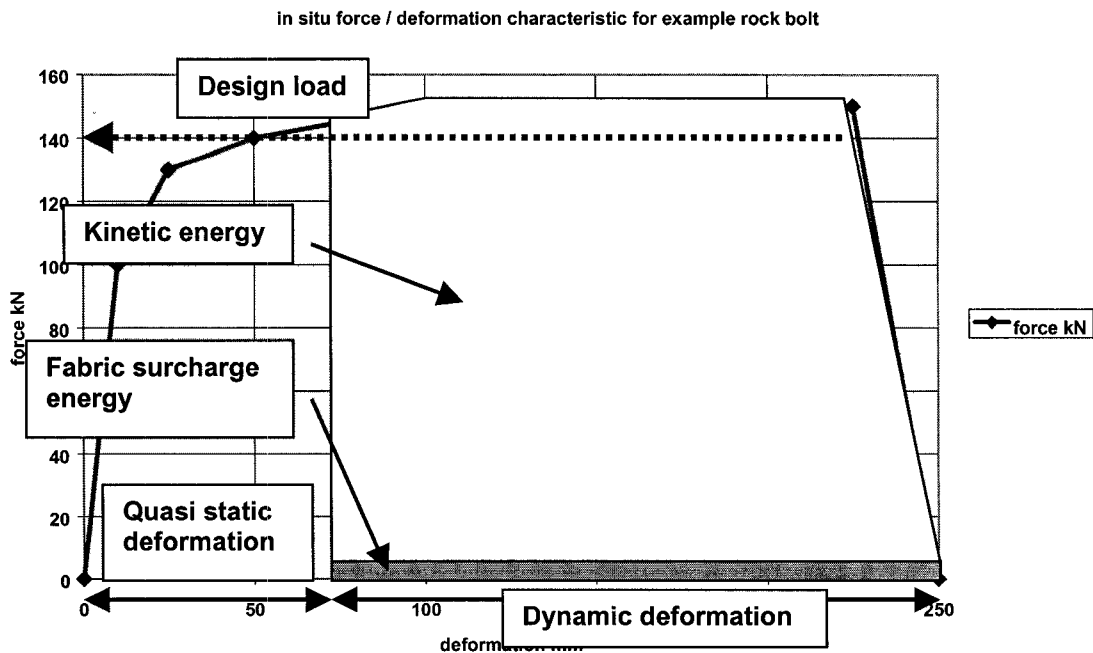
$$E = \frac{1}{2} (F_i + F_E) d + \frac{1}{2} m v^2 \quad (4-20)$$

Thus

$$E = 0.5 \times (0 + 7) d + 0.5 \times 2.75 \times 5.25 \times 2^2$$

$$E = 3.5 d + 29 \text{ (kJ)}$$

The load deformation characteristics of the rock bolts in the sidewall of the excavation are assumed to be similar to that represented in Figure 4-20. Balancing the energy demand and capacity of the rock bolt system based on equations 4-19 and 4-20, the anticipated deformation of the rock bolt system is estimated to be approximately 210 mm. The quasi static deformation associated with the sidewall of the tunnel, as derived in section 4.2.1, is estimated at 70 mm. This would indicate a total deformation of 280 mm, which is in excess of the deformation capacity of the defined rock bolt system, Figure 4-23.



**Figure 4-23. Distribution of deformation and dynamic energy demand / absorption.**

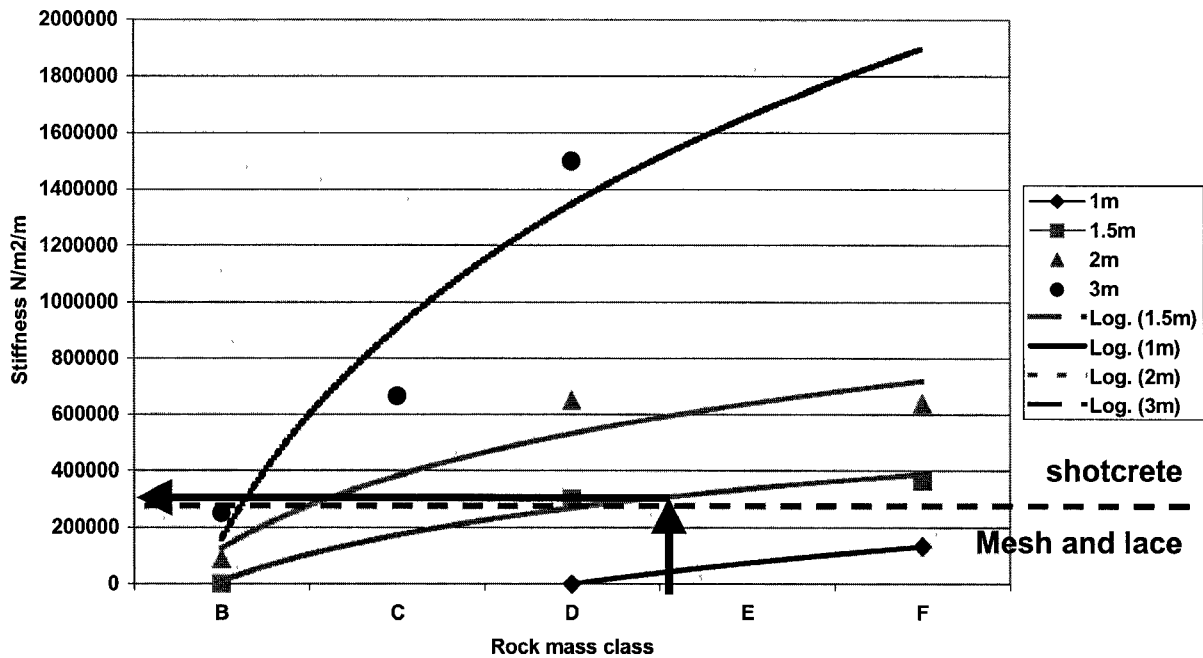
It would thus be necessary to re-evaluate the sidewall rock bolt reinforcement by increasing the density of similar rock bolt reinforcement, or increasing the capacity of the rock bolt reinforcement for the defined support pattern.

For a sidewall rock bolt reinforcement pattern based on a spacing of 1.5 m and a length to ensure stable anchorage in excess of the unstable rock mass depth of 3 m, a yield capability of greater than 0.28 m, or a yield load capacity greater than approximately 150 kN, is required. Alternatively, a more closely spaced rock bolt pattern should be evaluated.

Due to the increased depth of instability associated with the sidewall of the excavation, a comparison can be made between the above design analysis based on unstable rock mass containment and that based on the creation of a reinforced rock mass structure, as conducted in section 4.2.2.2. This analysis indicates that, for the defined rock mass environment, and a comparable deformation, a rock bolt reinforcement system based on 1.75 m long, 120 kN grouted rock bolts, at a spacing of 1 unit/m<sup>2</sup>, would result in a similar energy absorption capability. In this instance only mesh with a low density lacing pattern would be required as a fabric support. The cost effectiveness of the support systems may be evaluated to determine the optimum support design philosophy.

In the above analysis it has been assumed that the defined volume of unstable rock mass between the rock bolt reinforcement is able to detach fully from the bulk rock mass and load the fabric in isolation. Evaluation may also be conducted as to the effectiveness of the fabric support system to maintain the inherent strength of the rock mass. This is based on the ability of the fabric support system to prevent differential deformation of the rock mass between the rock bolt reinforcement, to the extent that detachment and unravelling of the rock mass between the rock bolts does not occur. This may be evaluated for a specific rock mass classification based on its ground reaction curve, for the defined reinforcement spacing as discussed in section 3.5.1. An estimation of the ground reaction curve allows an evaluation of the minimum fabric stiffness to maintain rock mass integrity (Figure 4-24).

### Analysis of fabric stiffness



**Figure 4-23. Example of ground reaction curve for estimation of stiffness of fabric support to maintain the inherent rock mass strength.**

For the example illustrated in Figure 4-23, the minimum fabric stiffness to prevent unravelling of the surface rock mass, for a class D/E rock mass and a reinforcement spacing of 1.5 m, is estimated at 300 kN/m<sup>2</sup>/m. This is based on the assumption that, at the point of installation of the fabric support system, unravelling of the rock mass had not initiated.

If this requirement is compared with typical mesh and lacing systems, even ignoring the initial slackness of the order of 70 mm, the required stiffness is in excess of that generated by mesh and lacing systems. However, comparison against typical shotcrete panel tests (Ortlepp and Stacey 1998) indicates stiffnesses of an order of magnitude greater (20 MN/m, over 1 m<sup>2</sup> panels). This is an important design consideration, as it implies that, if the inherent rock mass strength is maintained between the rock bolt reinforcement by the use of stiff fabric, then it may be envisaged that the rock bolts will be subjected to increased direct loading under dynamic conditions. Where historically mesh and lacing systems were attached to rock bolt units with limited yield capability, and these systems have been observed to survive dynamic loading, it may now be anticipated that a change to shotcrete systems will result in the failure of these rock bolts. This illustrates the importance of the consideration of the interaction between the rock mass and all components of the support system.

## 5 Conclusion and recommendations

The work contained in this report and formulated into a design procedure has attempted to elucidate our understanding of the complex interaction between rock bolt reinforcement and a highly discontinuous rock mass structure. This understanding is based on a mechanistic evaluation of in situ observations and measurements, and detailed numerical analysis.

Due to the highly complex, and erratic nature of the highly fractured and discontinuous rock mass around deep level tunnel excavations, particularly under dynamic loading due to seismic events, much of the mechanistic understanding has been based on observations made at underground sites of severe rockburst damage. These conditions would be extremely difficult to simulate physically, but it is considered that a mechanistic understanding was gained from suitable numerical modelling. There are many components to the analysis and design of tunnel

support systems, and, where applicable, the work of other authors and researchers has been incorporated into the design process to try to formulate a coherent design methodology.

Empirical design guidelines for the selection of support systems cater well in the environments for which they have been calibrated and for the technology on which they are based. However, as the rock mass environment changes, these systems generally indicate a step wise change in the support system. That is, within a rock mass classification system a slight change in the rock mass condition may result in the definition of a different rock mass class and thus a significantly different support recommendation. A more fundamental understanding of the interaction of rock bolts and fabric support systems with the rock mass allows a continual adjustment and optimisation of a support system based on available support units. This leads to greater flexibility in the design process, particularly in highly variable rock mass environments, and thus improved utilisation of available resources. Particularly in the deep level South African gold mining environment, a more fundamental, mechanistic, understanding of the design of a support system should lead to improved design considerations with regard to ensuring the stability of excavations and optimum economic support installation. The significant aspects of the proposed design process and areas of investigation are reviewed in the following section.

Within the South African deep level gold mining environment, the principal design mechanism is rock mass retainment based on tributary area loading of the rock bolts, and the depth of instability derived from historical data. No consideration is given to the demand on the often required use of mesh and lace fabric support. Often large differential deformations occur within this discontinuous rock mass structure, which result in dilation and significant bagging, or ultimately failure, of the typical mesh and lace fabric between the rock bolt reinforcement. In this environment the survival of rock bolt units, with very limited yield capability, is often observed under severe dynamic loading conditions. Again, on the basis of the current design considerations, this would not be anticipated. This phenomenon has been shown to be a reflection of the very limited direct interaction between the rock bolt reinforcement and the unstable rock mass volume. In areas of abnormally high stresses, due to major mining abutments, or in areas of generally weaker rock, the mechanism of structural reinforcement may be observed, although this is often not considered in the design process. The stability of this reinforced rock mass structure is also very dependent on the interaction of the rock bolt reinforcement units with the rock mass and thus an understanding of the critical rock mass parameters is required. Additional observations of support system behaviour under conditions of dynamic loading indicated the susceptibility of the rock bolt reinforcement, particularly in the hangingwall of the excavation, to shear failure. This is not directly addressed in the methodology but is an important design consideration in these rock mass environments.

The need for a more mechanistic understanding of the interaction of the components of a support system with the unstable rock mass volume, and thus the derivation of a rational design methodology, was established. An investigation of the behaviour of the components of this complex system was thus conducted in the project.

In order to establish support design methodologies, an understanding of the anticipated extent and volume of instability around an excavation is required. This determines whether a rock bolt reinforcement system should retain the unstable rock mass volume by anchorage beyond this limit of natural instability, or reinforce the boundary of the excavation to form a stable structure which also provides sufficient constraint to the deeper unstable rock mass. A simple design philosophy and support system selection table to this effect has been proposed. The work of Martin (1997) and Kaiser *et al.* (1996) in defining the depth of instability due to stress fracturing is considered very suitable for this purpose in the deep level mining environment. However, it was found that the linear empirical data range established by these authors did not cater for the higher stress conditions which are sometimes experienced in the South African mining environment. On a very limited number of available case studies, it was attempted to extrapolate this data for higher field stress levels. This work indicated that there is a tendency for the increasing extent of natural instability, with increasing stress level, to flatten off.



Mechanistically, for a given size of excavation and typical stress fractured rock mass characteristics, this would indicate a finite depth of instability beyond which it is increasingly improbable for instability to occur. The observation of the tendency for deep level excavations to break out to an elliptical profile, with the short axis parallel to the maximum stress trajectory, supports this hypothesis. The use of the derived criterion to estimate the natural depth of instability (in the absence of reinforcement and support), in this particular environment, is considered to be superior to the current use of historical databases on fallout thickness. The use of these databases will inherently encompass many other factors of excavation instability, including the performance of the support systems and the relative stability of past rock mass environments. In rock mass environments where instability is due to the natural discontinuity within the rock mass, then this criterion is not applicable and it is recommended that the effect of local geological features be incorporated into the evaluation of the depth of instability.

In a mining environment the tunnel excavations are often subjected to large stress changes over their operational life. These stress changes cause large progressive deformations of the rock mass around the tunnel. Of significance from this investigation is the large component of hangingwall deformation associated with a reduction in the vertical stress environment. Mechanistically it is proposed that this is due to shear deformation on the sub-horizontal bedding within the typical rock mass of the South African gold mining environment, caused by the vertical stress relaxation. An understanding of the deformation associated with quasi static stress changes is important for the determination of the required yield capacity and characteristics of the rock bolt reinforcement to ensure excavation stability, and to design the support system to limit deformation so as to maintain the excavation in operational condition.

Analyses carried out as part of this investigation have shown that there is a better correlation, although poorly defined, between the rate of dilation within the reinforced rock mass and the density of the rock bolt reinforcement. In the past, support resistance has been used as the primary means of evaluation. Although support resistance does encompass the rock bolt density, it does not differentiate between the actual in situ support resistance or loading and the design value. Also, the support resistance does not give consideration to the effective zone of interaction of a rock bolt unit. For example, a single 200 kN rock bolt unit would be given the same weighting as two 100 kN units over the same area, whereas the increased density of the reinforcement within the rock mass volume has been shown to have a significant beneficial effect on the overall rock mass stability.

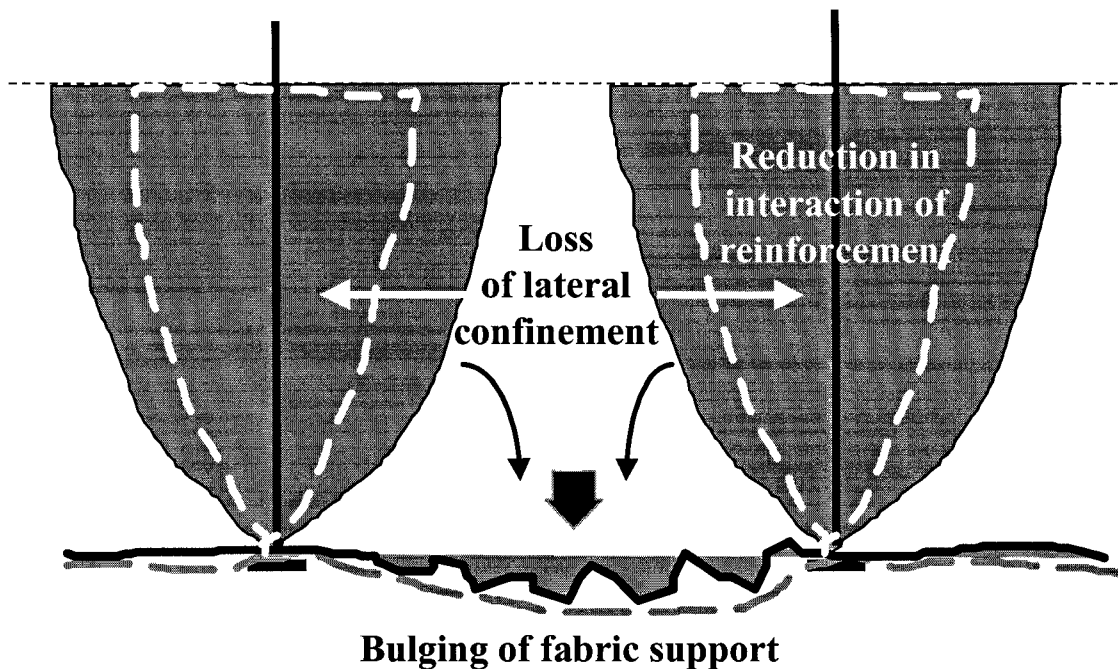
The current work on tunnel support design has also evaluated the analysis technique to derive dilation rates which may be used in applicable geotechnical environments, and evaluated the mechanisms of rock mass deformation due to changes in the stress environment. This is applicable to a typical South African gold mine rock mass environment, which consists mainly of bedded quartzites subjected to stress induced fracturing, and caution should be observed in the application of these deformation rates and mechanisms in other rock mass environments.

The installation of rock bolt reinforcement is generally the primary method of maintaining the stability of the excavation. It has been shown that within these highly discontinuous rock mass environments the interaction between the rock bolt reinforcement and the rock mass is critical to the mechanisms of quasi static and dynamic rock mass deformation. In an attempt to understand this interaction, in situ evaluations and relatively simple numerical models were established. The construction of these models gave consideration to the practical limitations of the ability to classify a highly complex rock mass structure in a relatively simple manner. Thus, simple, planar, and regular discontinuities, with discontinuity properties generally applicable to the South African gold mining environment, were used to represent the rock mass structure.

The analysis scheme used evaluates the interaction between untensioned rock bolt reinforcement and a stress fractured interlocking blocky rock mass structure. The rock mass structure in the analysis is dominated by discontinuities perpendicular to the rock bolt axis and is broken into interlocking blocks by intermittent discontinuities. The stability of the system is assessed directly by the extent of rock mass unravelling under the defined loading conditions

and reinforcement spacing. The mechanism of stabilisation between the rock bolt reinforcement and the rock mass is based on the restriction of block deformation and rotation within the unstable rock mass volume. This analysis is applicable to the South African gold mines where untensioned, fully grouted rock bolts are used for excavation stabilisation. The purpose of the analysis is to estimate the extent of direct reinforcement of the rock bolt units and thus the volume of unstable rock mass that will load the fabric support system. Further evaluation examined the interaction of rock bolt reinforcement to create a reinforced rock mass structure that is loaded by the deeper unstable rock mass volume. This analysis is used to compare the relative merits of the two support design philosophies.

An important design parameter, which can now be derived from this analysis, is the stiffness requirement of the fabric support under both static and dynamic loading conditions. If the fabric support has a relatively soft load deformation characteristic, such as the analysed mesh and lacing systems, then significant loading of the fabric will result as illustrated in Figure 4-25.



**Figure 5-1. Conceptual representation of the influence of relative reinforcement and fabric support stiffness on rock mass unravelling.**

The mechanism illustrated in Figure 5-1 has been observed to result in either significant bagging or failure of the fabric (mesh and lacing), with the consequence that the relatively stiff rebar rock bolt reinforcement, which is considered to be unsuitable under dynamic loading conditions, has survived. It is now clear that this is due to the loss of interaction of the intervening rock causing limited load being transferred directly onto the rock bolt reinforcement units. The load no longer carried by the rock bolts must therefore result in higher loading of the fabric.

Of significance to the recent trend in the use of shotcrete in the deep level mining environment is that the limitation of the deformation of the rock mass between the rock bolt reinforcement is likely to result in increased direct loading of the rock bolt units. This must be considered in relation to the previous observation of the survival of stiff rock bolt units under severe dynamic loading due to low direct rock mass interaction. Thus, a change in the fabric component of a support system must be evaluated in relation to the impact on the loading distribution within the system and the capacity of the other components, particularly their yieldability.

The assessment of the influence of seismicity on excavation stability is based on the energy absorption capability of the confined rock mass normal to the excavation boundary and parallel

to the reinforcement confinement. Dynamic ground motions may also result in shear movement within the rock mass and/or transient induced tensile tangential stresses. These may result in a further reduction in the extent of reinforcement confinement and thus an increase in fabric support demand.

Finally, the demand on the components of the support system may be estimated based on the defined rock mass and loading environment. For the design of a suitable support system this must be compared with the capacity of the components of the system. The axial capacity of rock bolt units has historically been investigated and is relatively well understood, and thus these performance characteristics have been accepted for this analysis. However, the shear capacity of the rock bolt system has previously received limited consideration, particularly within the South African mining environment. This has been shown to be an important consideration of rock mass deformation under quasi static stress changes, and particularly under dynamic loading. Of significance is the importance of yield capability and the relationship between the diameter of the rock bolt and the grout annulus in the estimation of the shear capacity of the system. The anticipated shear demand on the system is still not understood, but in situ observations have indicated the necessity for improved shear capacity under the specified rock mass environments.

The other major component of tunnel support systems in the South African mining environment is the mesh and lacing fabric support. Analysis of this component of the support system under quasi static and dynamic loading conditions indicates the high initial slackness in the system, which is detrimental to maintaining the inherent strength of the potentially unstable rock mass volume between the rock bolt reinforcement. However, its load deformation characteristic allows for large deformations under dynamic loading, and, thus, the associated capability to absorb kinetic energy of the unstable rock mass. Of significance to the evaluation of these fabric support systems is the proposed consideration of the energy absorbing capability of the contained discontinuous rock mass system, which may be significantly greater than that implied by analysis of the load-deformation characteristic of the fabric in isolation. This important aspect of the interaction between the support system and the rock mass requires far greater investigation.

It is considered that the analyses as reviewed above enable a mechanistic approach to support system design based on an understanding of the interaction between the components of the support system and the rock mass. Estimations of the loading of the components of the support system under rock mass environments as defined by the design engineer can be made. However, consideration must be given to the inherent assumptions of this analysis in relation to the extremely complex rock mass environment. The design process does, however, capture the behavioural characteristics of the rock mass and the support systems as observed in situ, and thus, compared to the current design process, gives greatly improved understanding into the design of support systems in this environment.

The following is a summary of the major conclusions of the project with respect to the enabling outputs defined in the project tender document.

- E.O. 1) **Preliminary tunnel design considerations** indicated the mechanisms of support interaction with the rock mass and were communicated to the industry to improve design understanding.
- E.O. 2) **Review of GAP026 project**, which formed the basis for the development of the proposed new design procedures.
- E.O. 3) **Determination and classification of initiation of stress induced rock mass failure** considered the influence of both the static and dynamic stress environment on the potential and extent of rock mass failure. Empirical design relationships were adapted for the high stress conditions prevalent in the South African mines to derive

tools for the estimation of the depth of stress induced instability. The influence of the transient stresses associated with seismic events was also examined.

- E.O. 4) **Classification, behaviour and limits of stability of a fractured rock mass around tunnel excavations.** The development of a simple classification system based on the structure of the rock mass allows the design engineer to estimate the extent of interaction between the components of a support system and the unstable rock mass volume. The performance of the support system may also be evaluated with regard to the 'ground reaction curve' characteristic of the rock mass to ensure compatibility between the rock mass characteristics and the support system characteristics. The mechanisms of rock mass deformation around a tunnel have also be elucidated upon and to some extent quantified. This again will enable the design engineer to select the most suitable rock bolt reinforcement system for the anticipated deformation. Of significance is the significant amount of hangingwall shear deformation that occurs under conditions of vertical stress reduction and dynamic loading.
- E.O. 5) **Determination of mechanisms of excavation stabilisation** was based on detailed in situ observations and numerical modelling. This work indicated the importance of the understanding of the interaction of the support system with the rock mass. This will be the basis of the design philosophies of retainment and containment of the rock mass by rock bolt anchorage beyond the depth of instability or by the creation of a reinforced rock mass shell within the extent of instability. The optimisation of the support system design can be conducted by the comparison of the relative energy absorption capability of the respective support systems.
- E.O. 6) **Influence of excavation shape and size on stabilisation mechanisms** has highlighted the influence of these parameters on the depth of instability around the excavation, which will impact on the practical design philosophy, and the relative stability of the reinforced rock mass shell. Design tools have been developed to estimate the depth of instability around a tunnel as a function of the size of the excavation. The increased curvature of the sidewall of the tunnel has been shown to reduce the deflection of the sidewall rock mass. However, this increased load bearing capacity of the reinforced sidewall rock mass may ultimately result in the increased loading and deflection of the hangingwall.
- E.O. 7) **Determination of interaction between support and the rock mass.** This part of the investigation has formed the basis of the new understanding of, and design considerations for tunnel support systems. This mechanistic evaluation and associated design procedure has indicated the relative loading of the components of a support system. This has enabled explanation of the survival of relatively stiff grouted rock bolt systems under severe dynamic loading where the current design would indicate failure of the system. A preliminary understanding was also developed of the ability of the contained, fractured rock mass volume to absorb a relatively large proportion of the dynamic energy imparted to the fabric system. With regard to the increased use of shotcrete as a fabric system, this analysis has indicated that the increased stiffness of the fabric will change the distribution of loading within the system and thus may necessitate an associated re-evaluation of the rock bolt energy absorption characteristics.
- E.O. 8) **Influence of seismicity on excavation stability** has been evaluated both from the point of view of the overall extent of instability around a tunnel excavation and the influence of seismicity on the interaction of the components of the support system with the rock mass. Charts based on the transient dynamic stress due to seismicity are used to estimate the total induced stress on the tunnel boundary and thus derive the anticipated depth of instability. The influence of seismicity on the interaction between the rock bolt reinforcement and the rock mass has been shown by in situ

observation and instrumentation, and by numerical modelling. This clearly illustrates that there is reduced interaction between the rock bolt reinforcement and the rock mass under dynamic loading resulting in the observed unravelling of the rock mass between the rock bolts and, where applicable, associated bulking of the typical mesh and lace fabric support. Relationships have been developed between the rock mass characteristics and the potential for unravelling under dynamic loading. In situ data has indicated a minimum ground velocity of 0.7 m/s as the initiation of rockburst damage in an 'average' rock mass class. In situ instrumentation also allowed a preliminary derivation of a relationship between the rock mass characteristics and the amplification of ground velocity from the rock bolt reinforcement. This relationship clearly illustrated the concept of increased freedom and potential instability with increased rock mass discontinuity.

E.O. 9) **Influence of stress change on excavation stability** was based on the re-evaluation of previous case studies and the detailed instrumentation of a tunnel subjected to gradual stress increase within the project. These evaluations have enabled the derivation of estimates of the anticipated dilation rates within the fractured rock mass due to stress change. Of significance from this investigation is the understanding and quantification of the difference in location and magnitude of deformation under relative vertical stress increase and decrease. This understanding will allow the support design engineer to select rock bolt reinforcement elements of suitable characteristics for the anticipated deformation of the excavation over its operational life.

E.O. 10) **Recommendations on the design of support systems and design methodology.** The recommendations for support and the development of the design methodology are encompassed in this document with a design application example. A significant development for the design of tunnel support systems, particularly in the deep level mining environment, from this project is the mechanistic understanding of the interaction of a support system with a highly discontinuous rock mass structure. Although fairly simple in nature and based on idealised rock mass structures, these design concepts and the developed tools capture the mechanisms of support performance as observed underground and now allow the design engineer to conduct relatively simple evaluations of the design of the support system. This methodology encompasses the understanding and estimation of a volume of unstable rock mass between the rock bolt reinforcement that will determine the necessity for, or design of a fabric support system based on the rock mass characteristics and rock bolt reinforcement pattern. The methodology also allows a mechanistic evaluation of the different design philosophies for a defined rock mass classification and loading condition. These mechanistic evaluations give greater flexibility in the utilisation of available support components in the design of a suitable support system to ensure operational safety of the excavation compared to the empirically based support classifications. Although not encompassed in the design methodology, this work has also indicated the importance of shear capacity, particularly in the hangingwall of tunnel excavations of the rock bolt reinforcement system.

E.O. 11) **Development of software for implementation of project outputs to industry.** This design aide is aimed to improve implementation of the design methodology and a mechanistic understanding of the interaction of support with the rock mass within the South African mining industry. This is based on a process of taking the design engineer through the design process encompassed within this document with an understanding of its applicability and assumptions.

In summary, the following recommendations are made with regard to the design of tunnel support systems in a highly discontinuous rock mass environment:

- 1) The extent of natural rock mass stability, under conditions of quasi static and dynamic stress fracturing, can be estimated from the proposed design analysis. This will form the basis of the selection of a suitable tunnel support design philosophy for the prevailing rock mass environment. This analysis captures the increased extent of instability due to the dynamic loading of the rock mass as a result of seismic events.
- 2) The dilation rates for the reinforced and unreinforced rock mass, as a function of the anticipated stress change over the life of the excavation, can be estimated. These will define the minimum deformation requirements of the rock bolt reinforcement in order to ensure excavation stability due to stress induced rock mass deformation. It has been shown that the density of the rock bolt reinforcement may better predict the influence on dilation of the rock mass than the in situ support resistance. Analysis of the deformation mechanisms has indicated the importance of the consideration of shear deformation within the rock mass, particularly under conditions of a vertical stress reduction.
- 3) The extent of interaction of the rock bolts within the defined rock mass structure and loading environment can be estimated from the application of charts based on rock mass reinforcement. The charts indicate the importance of variations in the structure of the rock mass, which defines the geometry of the rock mass blocks relative to the axis of the rock bolt installation. The dominance of discontinuities orientated sub-parallel to the rock bolt reinforcement result in a significant reduction in the interaction of the rock bolt reinforcement. This will limit the extent of direct loading on the rock bolt unit and thus define the extent of potential instability between the rock bolt reinforcement dependent on the designed reinforcement spacing.
- 4) The extent of the potentially unstable rock mass volume between the rock bolt reinforcement can be estimated. This will allow the determination of either the necessity for, or the demand on, the fabric support system. The design process captures the increased unravelling of the rock mass between the rock bolt reinforcement under conditions of dynamic loading, and thus the increased loading and deformation of the fabric support system under these conditions.
- 5) The load-deformation characteristic of the fabric support system also influences the extent of interaction of the rock mass with the rock bolt reinforcement units. Mesh and lacing fabric support systems as used in South African gold mines allow relatively large deformations of the rock mass between the rock bolt reinforcement. Analysis of these mesh and lacing configurations has shown that all the systems had an average initial slackness of approximately 70 mm prior to any appreciable load generation. This may increase the potential for rock mass unravelling, and, thus, reduced direct loading on the rock bolt units. However, the containment of this unstable rock mass volume by the fabric support systems results in significantly greater energy absorption than analysis of the fabric in isolation would indicate. The implementation of relatively stiff fabric support systems, such as shotcrete, is likely to result in a significant reduction in initial rock mass unravelling, and, thus, significantly increased direct loading on the rock bolt reinforcement. Consideration of this interaction between the fabric support system and the rock bolt units is therefore an important aspect of the support system design that ensures compatibility of the characteristics of the components of the support system.
- 6) Under conditions of dynamic loading and large vertical stress reductions, rock bolt reinforcement with high shear capacity is indicated, particularly in the hangingwall of the excavation. Where grouted rock bolt systems are used, the optimum hole diameter is approximately twice the bolt diameter.

Many of the concepts explored in this analysis are simple estimations of an extremely complex environment and system behaviour, and as such there is much scope for further work. But this should not detract from the insight gained, and the design procedures developed, in relation to the current understanding of tunnel support system design in the high stress, highly

discontinuous, dynamic rock mass environment of deep level mines. Important areas of further work are highlighted below.

- 1) Additional case studies of the extent of natural instability of the rock mass should be obtained and analysed to verify the method used to determine the extent of instability under highly stressed conditions.
- 2) Deformation mechanisms and dilation rates should be evaluated for different rock mass structural and loading environments.
- 3) The in situ shear demand on rock bolt reinforcement should be investigated to quantify the required shear capacity of a rock bolt system.
- 4) The energy absorption capability of a reinforced and contained rock mass system should be further investigated. This is the implied ability of a contained rock mass system to absorb significantly greater energy than would be indicated by analysis of the fabric, or rock bolt reinforcement in isolation. This will have important implications for the understanding of the energy absorption capacity of the stabilised rock mass system.
- 5) The three dimensional interaction of the support system within a defined rock mass structure should be evaluated.
- 6) An empirical evaluation of the applicability of the rock mass classification systems and support design process, as proposed in this analysis, with regard to the successful and failed performance of current support systems should be carried out.
- 7) This investigation has assumed that rock mass instability under conditions of dynamic loading is due to expulsion normal to the excavation boundary, or simple increased acceleration loading within the rock mass. There is evidence of diagonal and longitudinal dynamic loading and deformation of tunnels. It is therefore necessary that the mechanism of interaction of dynamic waves impinging on tunnel excavations from different directions, and, thus, the deformation and stability of the rock mass under these conditions would require far more detailed analysis.
- 8) An evaluation of the interaction of shotcrete with a potentially unstable rock mass volume has only been implied in this analysis and thus would require further investigation. It is considered that the interaction of shotcrete, and other membrane type support systems, will be fundamentally different due to their direct adhesion to the rock mass and potential, significantly greater system stiffness. However, their limited ability to tolerate large deformations, or their ability combined with the implied greater effectiveness of the reinforcing bolts to control the large potential deformations, needs to be thoroughly investigated before application of these systems can be fully endorsed.

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