SAFETY IN MINES RESEARCH ADVISORY COMMITTEE

SIMRAC

Final Project Report

Title: 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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Research Agency: CSIR: Division of Mining Technology

Project No: GAP 026

Date: December 1995

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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 GAP026 project addresses the issue of strata control in tunnel excavations with the aim of improving the stability of the excavation through improved design methodologies and support systems. Over the 3 year period of the project several changes in the scope of the required research were made by the SIMRAC committees. Initially (1993) the focus of the project was on the stability of tunnels in the Bushveld Complex. Subsequently (1994) two significant changes in the scope of the project were made. These changes were; to address the stability of tunnels in all gold and platinum mines with particular emphasis on poor / high stress / rockburst conditions; and laboratory investigation of support units under simulated underground loading conditions. These changes in scope were made with no adjustment to the current financial position of the project but with an allowance for the extension of the project over the initial 3 year research period. Due to these changes in research emphasis within the confines of the original resource allocation this report describes the work carried out to date, and emphasises conclusions which may be currently implemented within the industry, and gives more detailed, up to date recommendations on the future research strategy than provided in the accepted proposal for the continuation of this project

Initial work concentrated on industry surveys to determine the major deficiencies of the current support systems and practices within the mining industry and to identify the main problems particularly with respect to mines in the Bushveld Complex. This work highlighted several factors which would form the basis of the future research strategy. Of prime importance was the establishment of a rational design methodology for tunnels within the South African mining environment, to address problematic ground conditions. It was found that the current empirically derived design methodologies sufficed in most areas, however they were found to be deficient in problematic ground (low cohesion, high stress and rockburst conditions). In addition it indicated the need for improved primary support systems, an understanding of the interaction of support elements and the influence of excavation shape on tunnel stability.

Due to a need expressed by the SIMRAC committees the laboratory evaluation of support elements under simulated underground loading conditions was introduced within the project. It was considered of prime importance that an evaluation of rock bolts under shear loading conditions be carried out. Tests were conducted on all the major units used within the South African mining industry under static and dynamic, tensile and shear loading. The tensile tests were conducted to allow comparison of the shear properties with this standard measure of bolt performance. Theoretical studies indicated that the shear resistance of the tendon should be approximately half the ultimate tensile strength. In the majority of the shear tests this anticipated load was exceeded. Examination of the mechanism of failure indicated that under shear loading plastic bending of the tendon would occur with associated crushing of the grout annulus surrounding the tendon. The degree of plastic bending that the tendon can accommodate results in a change in the failure mode from shear to tensile. Thus it was found that the more brittle / rigid tendons accommodated less shear than did the more ductile tendons or yielding tendons. Under static and dynamic loading the same characteristics were observed, but dynamic loading generally resulted in lower shear resistance due to the inability of the tendon to undergo plastic deformation at high rates. Recommendations for the industry are that if shear deformation within the rock mass is anticipated the use of yielding or ductile (grouted hoist rope) support tendons be utilised.

In situ evaluations of long cable anchors were conducted in collaboration with Vaal Reefs Gold Mining Co. Ltd.. This work indicated that in general the stability, or limiting of deformation, was controlled more by the number of support units installed per unit area than by the length of the cable anchors. As the length of the anchor was increased so the distribution of sidewall dilation within the rock mass changed, but the overall dilation was not significantly influenced. The support resistance of the support system had a limited influence. It was also noted that as the support resistance of the sidewall support was increased the deformation of the hangingwall increased, but this was limited to the immediate 2 metres of tunnel hangingwall. The hangingwall of the tunnel also indicated high shear deformation associated with bedding planes and this was considered to be a significant parameter in support selection.

Due to the fragmented nature of the project subsequent to the changes as discussed above it was considered necessary to first develop a conceptual design methodology which would form the basis for continued work to provide a quantitative design method. This was formulated by the project team in order to focus the research and develop a strategy to address the design problems as experienced by the mining industry for problematic ground conditions. The conceptual design methodology that was developed is based on the supposition that properly designed support should provide sufficient interaction of support elements to create a reinforced rock mass shell within the discontinuous rock mass around an excavation. This shell is envisaged to be analysed as a series of beams or plates. The degree of support interaction required would be determined by the rock and loading conditions (gravity, dilation, rockburst) anticipated for the rock mass shell over the operational life of the excavation and be a function of the spacing, length and type of support including the fabric type elements. Initial validation work on this concept has been conducted by numerical modelling techniques and the results are most encouraging.

In conclusion the worked conducted under the GAP026 project has allowed a clear picture of the deficiencies of the current tunnel support practices to be developed. This has enabled the formulation of a conceptual design methodology to address these problems that will form the basis of the research strategy. In addition initial evaluation work has been conducted on the performance of support units under simulated and in situ loading conditions to derive recommendations for support selection within the South African mining industry. Of significance to the mining industry is the recommended use of yielding tendons or grouted hoist rope for support in areas of anticipated shear deformation within the rock mass. Shear failure of rock bolts has been noted at several rockburst sites.

1 Introduction

The GAP026 project was initially formulated in 1993 to address problems associated with tunnel excavations in the Bushveld Complex. In 1994 two significant changes in the scope of the project were implemented at the request of the SIMRAC committees. The basis of these changes were to address problems associated with tunnels in all gold and platinum mining environments with particular attention to high stress and rockburst conditions, and in addition to implement a programme to test support units in the laboratory under simulated underground loading conditions. These changes were implemented with no change in the current financial resources of the project but with an extension in the term of the project. Due to these changes of scope many of the enabling outputs have been initially examined but their completion dates have been pushed out.

There are five major areas of research activities that have been addressed within the project to date and are covered in this report:

1. Evaluation of tunnel stability and support problems in South African mines.

2. Strata control in tunnels in the Bushveld Complex.

3. Laboratory and field evaluation of the currently used tunnel support units and systems.

4. Evaluation of the influence of long anchors on tunnel stability

5. Development of practical concepts for the design of support systems.

The most important findings, conclusions and recommendations with regard to the above research activities are included in this report, and in some cases the relevant details are appended as separate reports while in others reference is made to the previously published SIMRAC interim reports.

2 The main components of investigations on tunnel stability and support problems in South African mines were as follows:

This aspect of the project addresses enabling output 1.1.

2.1. The scope and nature of tunnel damage

The main objective for this milestone was to identify and evaluate conditions in tunnels where the conventional support systems are not adequate. As a result of the investigations a foundation has been laid for the formulation of tunnel support requirements.

The investigations included the influence of geological factors; effects of stress; shape and orientation of tunnels; damage to tunnels under quasi-static conditions with types, causes and extent of damage; mechanical properties of rocks compared to observed stability and rockburst damage.

Damage to tunnels is observed for a wide range of mining depths. This is an indication that factors which adversely affect excavations are not only related to high vertical stresses. Other important factors that influence the stability of tunnels are mechanical and rheological properties of the surrounding rock, geological discontinuities which intersect the excavations, k-ratio as well as support practice including quality of installation.

The evaluation of the tunnel damage problem was based on an analysis of data that have been gathered during underground observations and field investigations, discussions with mine personnel, a survey of research reports and other relevant publications and through a questionnaire. To gain first hand experience of the strata control problems where no written information was available, additional underground visits were arranged and problems of tunnel stability investigated.

Table 1 summarises the relative importance of causes of poor conditions in tunnels from the various gold fields.

Table 1 THE RELATIVE IMPORTANCE, IN PER CENT, OF CAUSES OF POOR CONDITIONS IN GOLD MINE TUNNELS

Type of conditions	East Rand	West Rand	Far West Rand	Klerksdorp	Free State
High initial field stress	9	10	14	7	6
High anticipated stress changes	73	15	34	33	40
Fault and dyke intersections	5	19	14	14	11
Poor rock quality	11	51	16	8	38
Anticipated rockburst conditions	1	3	22	38	3
Other	1	2	-	-	2

In gold mines the main factors contributing to poor conditions, both under quasi-static and rockbursts loading, are stress related. However, in many instances problems with stability of tunnels are further complicated by the presence of geological discontinuities such as faults, dykes and bedding planes and weak, usually altered rock.

To learn more about types, causes and extent of damage a survey was carried out to determine the relative proportions of the different types of damage which occur in the gold fields. The results are summarised in Table 2.

Table 2 TYPES OF DAMAGE OBSERVED IN TUNNELS

Type of damage	East Rand	West Rand	Far West Rand	Klerksdorp	Free State
H/Wall falls	30	39	36	19	29
Sidewall falls	60	30	44	26	24
Sidewall closure	9	21	11	15	34
Footwall heave	1	4	1	38	9
Other	-	6	8	2	4

The most common type of damage varies form gold field to gold field, but it is clear that damage to the sidewalls of tunnels is a much more common problem than falls of ground from the hangingwall. From observations it appears that sidewall falls and excessive closure tend to be associated with large stress changes in the West Rand and Free State gold fields. The East Rand and to a lesser extent, Far West Rand have a relatively high proportion of their tunnels in shale. This also tends to aggravate hangingwall falls.

Support problems due to inherently poor rock mass conditions are not widely experienced in deep gold mines and are confined to certain areas and specific formations which have to be traversed. Large numbers of faults and dykes intersections in West Rand and Free State mines are the main cause of poor ground conditions leading to a relatively high proportion of hangingwall falls.

In the Klerksdorp district, together with sidewall and hangingwall falls, footwall heave related problems are important, probably because a high proportion of the tunnels are located in well bedded, argillaceous quartzwackes. Another factor that increases footwall heave appears to be abnormally high k-ratios or an orientation of $\sigma 1$, substantially off vertical on mines such as Hartebeestfontein.

Photographs in Figure 1 not only illustrate examples of the very serious types of damage that can occur and the intensity of rock fracturing around highly stressed tunnels but they also introduce a measure of appreciation for the very high support requirements needed in these demanding conditions.

To effectively control such a rock mass environment under both slow and rockburst loading it is necessary that more efficient and yet cost effective support systems are developed and tested, to complement already available support systems which are usually adequate for all but the most severe conditions. A survey has indicated that about 20 % of all gold mine tunnels are subject to such conditions.

2.2 Cost of Inadequate Tunnel Support

Two components of the cost of non-effective tunnel support systems need to be considered. These are:

a) Human costs, and
b) Financial losses.

A survey of data on rockburst / rockfall related fatalities on South African mines, for the period 1991-94, showed that a total of 1076 fatalities occurred

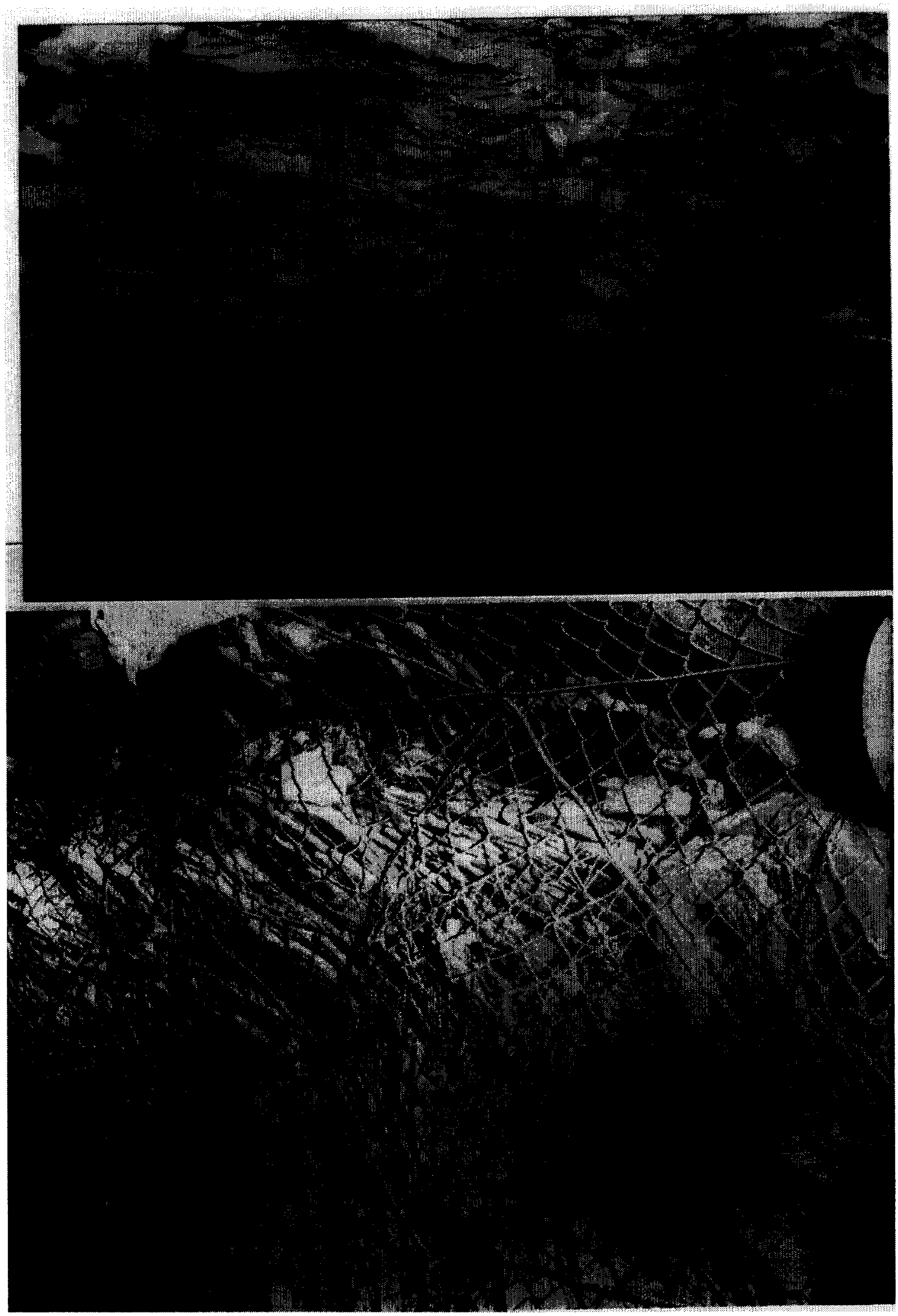


Figure 1. Example of damage due to high stress environments

of which 198 (18,4%) have taken place in tunnels. Of these, 64 % were caused by the falls of ground while the remaining 36 % were caused by the rockbursts in tunnels. A more detailed breakdown of the accident data is given in the table 3 below.

Table 3. Analysis of rock related accident data for tunnels

Mining District	Tunnel Fatalities %	Rockfalls %	Rockburst %	Average Rockfall height (m)	Average Rockburst height (m)
OFS	18,6	88,2	11,8	1.1	0.7
E. Rand	13,6	63	37	0.2	1.0
Klerksdorp	18,1	67,4	32,6	0.74	1.6
W. Rand	19,2	50	50	0.85	1.9
Overall	18,4	64,1	35,9	0.9	1.7

Fatalities associated with tunnel excavations vary from approximately 14% in the East Rand to almost 20% in the West Rand of all rock related fatalities. In general most tunnel fatalities are due to rock falls, apart from the West Rand district were 50% of all fatalities are associated with rockbursts. This is probably a reflection of the general seismic hazard of the relevant mining districts but is indicative of the importance with regard to tunnel support design. In most cases the height of the fall of ground associated with rockburst fatalities is substantially greater than those for rockfalls within the same mining district.

A comparison was conducted of the relative risk of fatalities associated with fatal rock related incidents for tunnels and stopes under rockfall and rockburst conditions. This indicated a relative risk between tunnels and stopes of 1,82 to 1,62 for rockbursts and 1,15 to 1,06 for rockfalls respectively. This is considered to be due to the concentration of people in the face area of a development tunnel where the majority of fatal incidents are indicated to be located.

It is considered that many of the incidents occurred because area support had not been installed in time or had deteriorated.

Data on financial losses that relate directly or indirectly to inadequate support while also has a bearing on safety, was obtained from another survey. Three major components of these losses in gold mine tunnels are: the rehabilitation of tunnels; non-integration of primary and secondary support; and poor installation of support.

The results of the survey carried out several years ago indicated that approximately 85 km of tunnels have to be comprehensively re-supported and re-graded annually, or in some cases after severe damage, re-developed. This is a surprisingly high figure, particularly when considering that an additional 110 km or so require minor maintenance such as bleeding of broken rock from behind the mesh, the replacement of some of the support or re-grading of tracks.

Rehabilitation of damaged tunnels and non-integration of primary and secondary support comprise R81 million or 36 per cent of the total annual financial losses associated with tunnel damage.

Available statistics for the gold mines indicate that seventy five per cent of the horizontal tunnels have support installed in two stages. Primary support is that which is installed by the developer close to the face of the development end mainly to protect the development crew, while secondary support is that installed some distance behind the face, usually by specialist support crews or contractors. About 80 per cent of the support units used for primary support are not incorporated into the secondary support pattern. The three main reason for this are:

a) holes for primary support tendons are drilled out of position so that they do not fit into the secondary support pattern - in particular tendons are not installed into the corners of tunnels which is necessary for good mesh and lacing installation;

b) the quality of installation, especially of grouting, is often unacceptable to the specialist secondary support crews;

c) frittering of rock slabs from beneath the bearing plates of end-anchored tendons or from around shepherd's crooks makes the integration of such tendons into the secondary support impossible.

The poor quality of support installation is one of the biggest contributors to financial losses and safety associated with tunnels. Due to the highly fractured nature of the rock to be supported, the drilling of holes for support tendons and the grouting of tendons are often difficult. Also rock slabs tend to fall from beneath bearing plates on end-anchored units and from around shepherd's crooks. In the former case, the support unit loses all its support capability, while in the latter the quality of mesh and lacing attached to the tendon is diminished, since the lacing can not be tensioned against the rock face. It is difficult to produce a reliable figure for the industry on how much and to what degree the support installed is not servicing its expected function. Limited data from spot checks, however, indicate that the figure may be higher than 20 per cent of the gold mine tunnel development.

In the investigations to define the scope and nature of damage to tunnels, the main emphasis was given to gold and platinum mining, because of the magnitude of the problem on those mines. However, the more serious problems concerning stability of tunnels observed in other mining areas are also included in a reference report written by Wojno and Jager (1994).

The review of tunnel stability and support problems identified eight major groups of problems that are mainly responsible for the more serious damage to tunnels and hence reduced safety. The most important problems are as follows:

I) Lack of a comprehensive, rational tunnel design methodology.

ii) Uncertainties about rockburst mechanisms in tunnels and how these influence support requirements.

iii) Inadequate support for extremely high stress and severe rockburst conditions.

iv) Inadequate support for low cohesion rock environments.

v) A need for improved primary support for poor ground conditions.

vi) Insufficient understanding of the interaction between support elements and the rock and between adjacent tendons.

vii) Insufficient data and experience of the influence of tunnel shape on stress fracturing, deformation and overall tunnel stability under high stress and rockburst conditions.

viii) Variability in the quality of support installation, particularly tendon grouting.

Other issues that need to be addressed are:

a. protection against strain bursting,

b. a means to prevent frittering of rock from beneath bearing plates,

c. a need for improved corrosion protection of tunnel support for specific areas and environments,

d. tunnel support systems and methods of installation that will integrate into the mining cycle for ultra high speed development,

e. the need and potentials of supporting the footwall in deep or rockburst prone tunnels, and

f. a need for resilient support that would withstand shock loading and blast resistant area coverage support for the face area.

The following briefly outlines the envisaged research requirements that are necessary to achieve solutions to the major problem areas as identified in the review.

Ad. i. Define and quantify support requirements by determining the influence of rock properties, stress field and stress changes and the effect of geological discontinuities on the mode and extent of tunnel failure. This will include the amount of fracturing and resulting tunnel deformation that in effect is responsible for the support needed. It will also involve the re-evaluation of the RCF criterion especially with reference to mines not in Witwatersrand rock types.

State of the Art non-linear numerical modelling codes, should be used to quantify the effects of the above parameters, analysis of which should be incorporated in the formulation of a comprehensive rational tunnel support design methodology. The numerical codes need to be calibrated with field data on the rockmass and support behaviour from test sites, case studies in the literature, and additional monitoring of the relevant parameters if required.

Ad. ii. Define and explain mechanisms involved in rockbursts and how they influence tunnel support requirements using non-linear dynamic modelling, and analysing all the available field data on rockburst damage. Recently developed dynamic numerical codes will require calibration against peak ground velocities in tunnels, and analysis of rockbursts in tunnels.

Ad. iii. Identify and quantify the weak links in the conventional support systems by analysing the available data on damage to tunnels for extremely high stressed and rockburst environments, and by designing and carrying out tests of support elements and rockmass-support systems under quantifiable and realistic loading conditions. By using the data obtained from these tests, develop performance requirements and design criteria for support systems that would eliminate the deficiencies of the existing support. Use static and dynamic numerical codes to quantify the effects of pre-determined loading conditions and support characteristics on the overall stability of tunnels and their modes of failure.

Ad. iv/v. Define and quantify deficiencies of the existing tunnel support systems with respect to their performance characteristics, difficulties with tendon installation in low cohesion rock, retaining the integrity of the newly exposed rock in the face area, etc. by carrying out tests on surface or underground. Specify performance requirements and design criteria for low cohesion rock environments, by numerical modelling of the reinforcing effect of tunnel support including improved area coverage characteristics as compared with conventional support systems, to allow the development of more appropriate support. Calibrate numerical codes with data on support performance from tests carried out on surface or underground.

Ad.vi. Determine and quantify the support mechanisms of the various support elements and their performance in fractured rock by numerical modelling to simulate their reinforcing effect. Determine and quantify the interaction between adjacent support units and the interaction between support and the rock mass: how are the support forces distributed through the rockmass, and how an annulus of reinforced yet fractured rock of various thickness supports the deeper rock, by numerical modelling and calibration of the codes with data from laboratory experiments.

Ad.vii. Determine most favourable tunnel shapes for various stress and geological conditions by carrying out static and dynamic numerical modelling of various tunnel shapes to determine the inherently most stable shape by estimating the relative extent of failure and amount of tunnel deformation from numerical modelling.

Ad.viii. Estimate the effect of poor quality of tendon support installation on the overall stability of tunnel by numerical modelling of the reinforcing effect of tendon support with various performance characteristics. Develop support installation techniques and recommendations that could ameliorate the installation problems.

A research report on conditions and problems in mine tunnels was written and distributed to SIMRAC members. It has summarised and discussed the main issues and indicated the important areas and methodology for research to minimise these problems.

3.1 Strata control in tunnels in the Bushveld Complex

This aspect of the project is based on the original proposal to address only problems associated with tunnel excavations in the Bushveld Complex and may be considered as a sub-section of enabling output 1.1 of the current project proposal.

As part of the initial GAP026 project, a major effort was put into investigating strata control problems in platinum mines of the Bushveld

Complex. Discussions with the mine personnel and underground visits to most of the platinum mines were carried out with a view to define the extent and character of damage to tunnels in this region. For practical reasons, most of the underground visits were arranged in areas where problems with strata control in tunnels occur.

It should be noted that much of the major damage to tunnels in the platinum mines of the Bushveld Complex is not stress or rockburst driven as is the case in gold mine tunnels but it is controlled by the complex geology of the region.

In platinum mines of the BC there are two main types of problems pertaining to the stability of tunnels which are related to:

a) Inherent ground conditions:

   1) serpentinization, that greatly reduces the self supporting capacity of the rockmass, that results in falls of ground

   2) high sub-horizontal stresses resulting in "gothic arch" type damage to the hangingwall

   3) intensive and multi-oriented jointing

   4) combination of the above mentioned conditions.

b) Tunnel support practice:

   1) falls of ground between the individual rock tendons

   2) rock frittering around the bearing plates, when such plates are installed

3) damage to tunnels caused by poor quality of tendon grouting, where the load carrying capacity of affected tendons is much lower than their failure load. This results in tendons pulling out which adversely affects the overall stability of tunnels.

Gothic arch type damage is confined to tunnels in certain areas and of specific orientation but has serious implications to their overall stability. This is because such a major change in the shape of the damaged zone around the tunnel requires a marked modification to standard support practice involving not only a hazard from falls of ground but also difficulty in installing the support. Increased support costs also have to be allowed for in these conditions. The hazard is however to a large extent reduced, by the mines affected, by blasting the tunnels to conform to the stress induced gothic arch shape.

"Gothic arch" type of damage to hangingwall of excavations in high k-ratio areas is illustrated in Figure 2.

One of the major problems that causes serious instabilities in tunnels is due to serpentinization creating virtually "cohesionless" ground conditions , especially if it follows the joints. In some of the problem areas a joint density as high as 20-25 joints per metre occurs. The problem becomes even more severe in areas where an almost random joint orientation occurs, as in some areas of Union Section. There are areas where approximately 50 % of cross cuts have problems in developing through such ground.

In other sections of Amandelbult (UG2, depth 300-600 m) where the ground is totally overstoped there is no problem with the stability of tunnels. However, under partially stoped Merensky Reef, the resulting changing stress regimes, from overstoped areas with high sub-horizontal stresses to sections

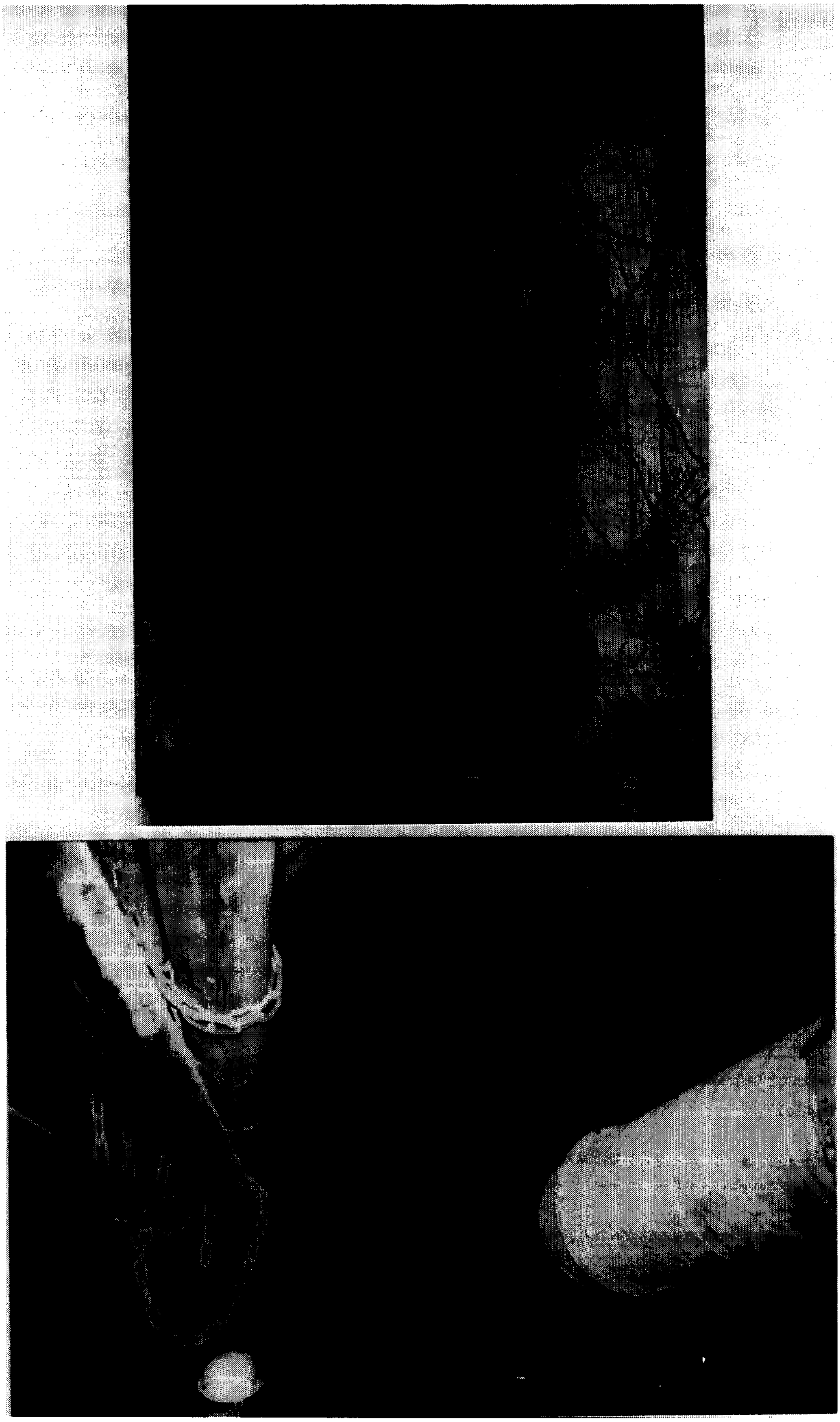


Figure 2. Examples of gothic arch tunnel profiles in Bushveld Complex

with remnants where high sub-vertical stresses occur, falls of ground become a problem.

On the same mine, in Merensky Reef cross-cuts, below 1 000 m, it is estimated that up to 30 % of development ends have problems with their structural stability.

In Union Section, on average, problems with the stability of excavations are encountered in 6 to 10 development ends at a time. It becomes particularly difficult in tunnels where falls of intensely jointed and almost cohesionless ground, from the hangingwall and sidewalls, are not effectively controlled on exposure i.e. rock retained in place, by the temporary support. To illustrate the problems in the more difficult cases, support systems comprising timber and steel sets (full cribbed) plus sliding rails supported by Camlock props, together with conventional support have to be resorted to in order to prevent falls of ground and are not always successful. On occasions, falls of ground of 2 m or more, both from hangingwall and sidewalls occur, leading to significant deformations of the fully cribbed, steel sets.

In other cases, special construction straps are used in conjunction with tendon support as illustrated in Figure 3.

Often problems with tunnel stability are observed in the vicinity of pothole structures which are generally circular depressions where the reef transgresses its footwall stratigraphy. The origin of potholes is thought to be due to the slumping of a heavy layer into a partly-consolidated footwall material with subsequent shaping of the pothole by vortex currents generated as a result of this slumping.

Reasons given for problems in the vicinity of potholes are:

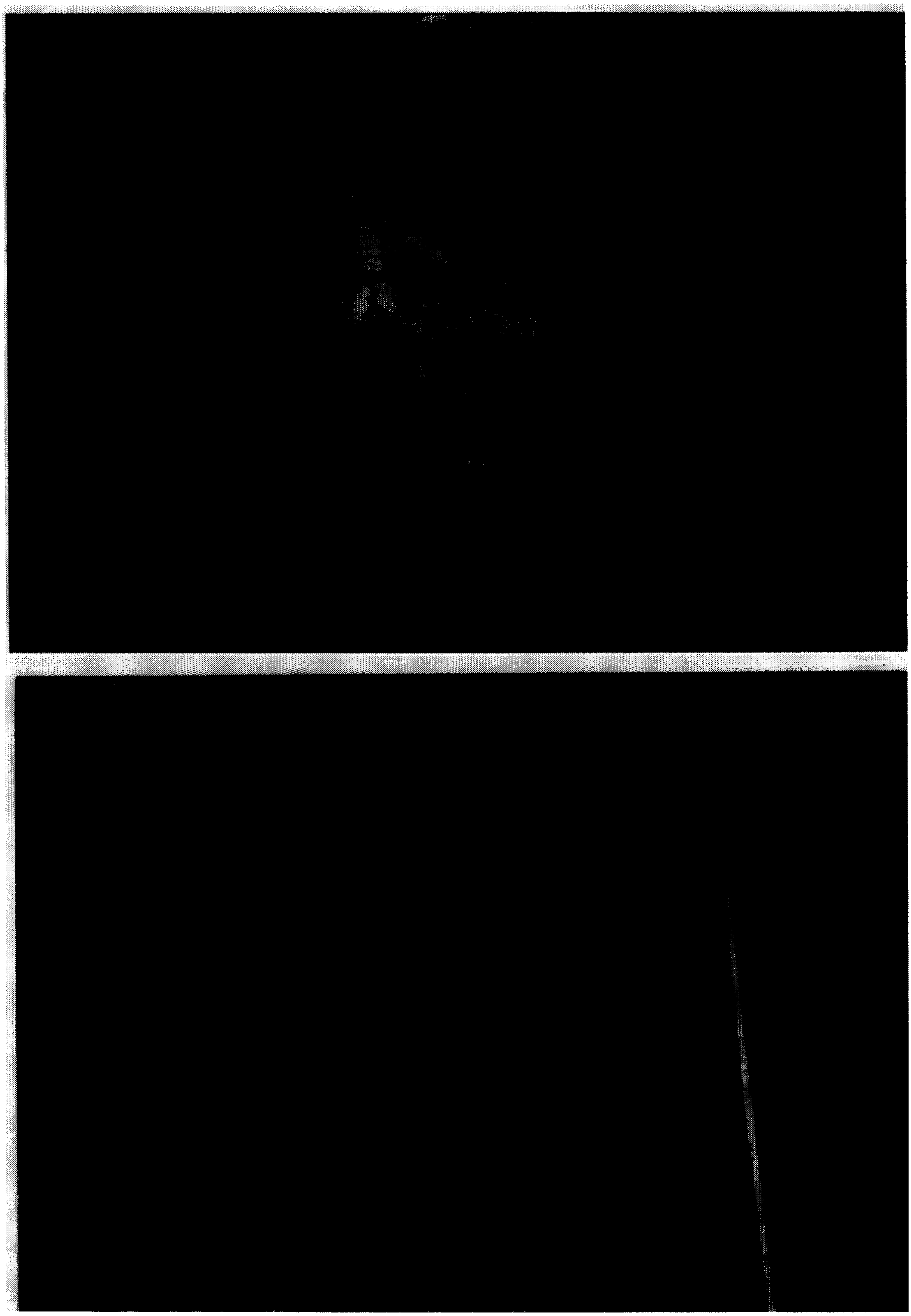


Figure 3. Examples of the use of straps for tunnel support

a) that the jointing in the vicinity of potholes is often more intense, orientations of joints are deflected around potholes, additional joint directions are present and their dip is commonly of lower angle.

b) the field stress can be distorted in the vicinity of potholes such that significant changes in the magnitude and direction of the major principal stress (as later discussed in the report) occur.

c) potholes left as remnants on the stoping horizon act as a stress raisers which can adversely effect off-reef excavations below them.

Examples of serious damage to tunnels caused by the poor rock quality in platinum mines are given in Figures 4a, 4b.

One of the important implications of tunnel damage are falls of ground causing fatalities, injuries and production losses. To quantify and qualify the problem an analysis of the accidents statistics available in the GME files was performed.

Useful data for the rock mechanics practitioners were extracted. Some of the statistical data gave information on the position of falls of ground in the excavations (pin pointing critical areas from the overall stability point of view and which require attention), and the thickness and mass of the unstable blocks. This type of data can be used in a preliminary assessment of support requirements for excavations where overall stability is controlled by the tunnel geology rather than a field stress i.e. mostly at shallow depths.

Examples of data on accidents statistics obtained from 16 shafts of the Bushveld Complex are given in Figures 5a., 5b and 5c. The data clearly indicate that the majority of falls of ground occur within the first 10 metres

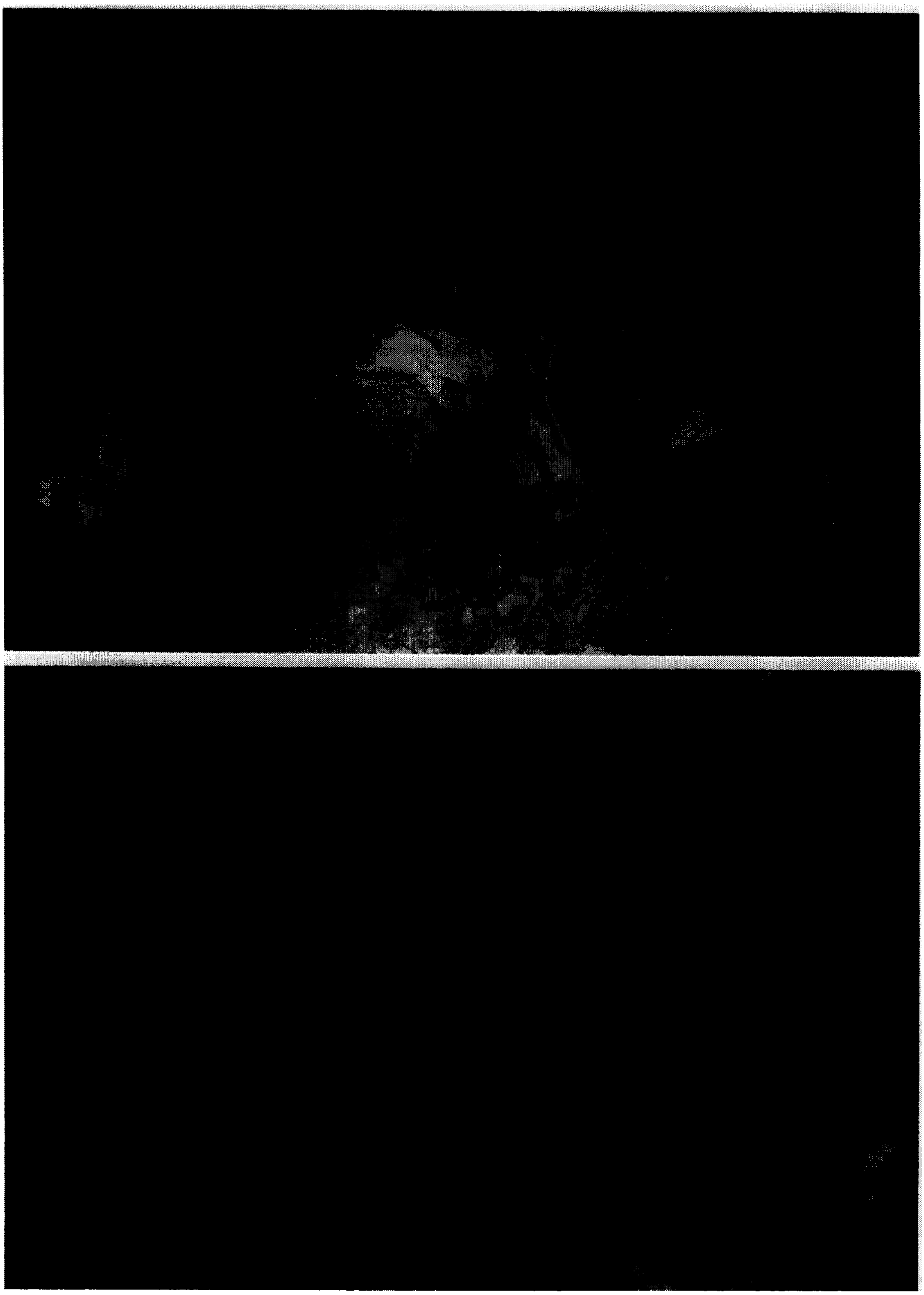


Figure 4a. Examples of tunnel damage in poor rock mass conditions

Figure 4b. Examples of tunnel damage in poor rock mass conditions

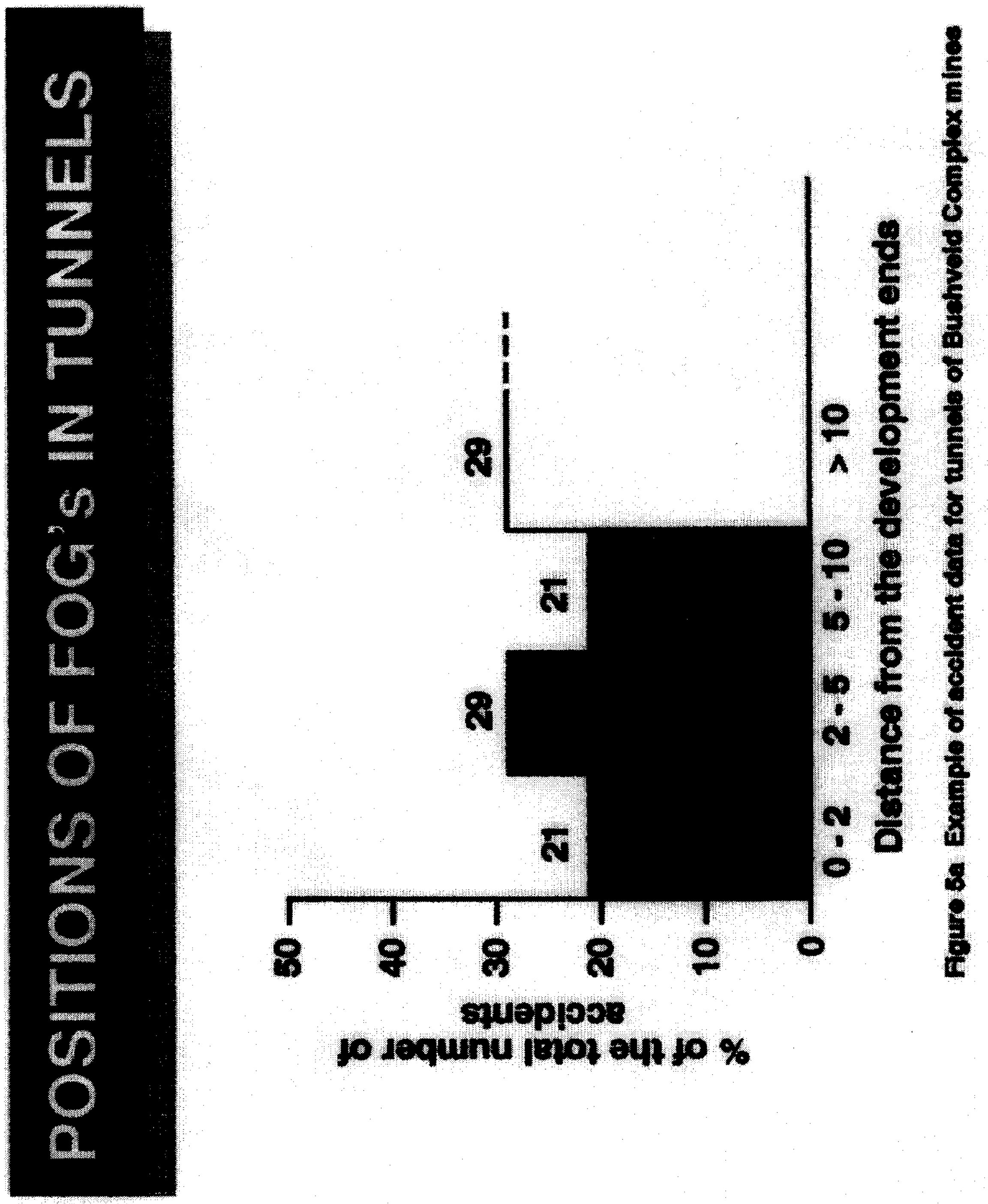
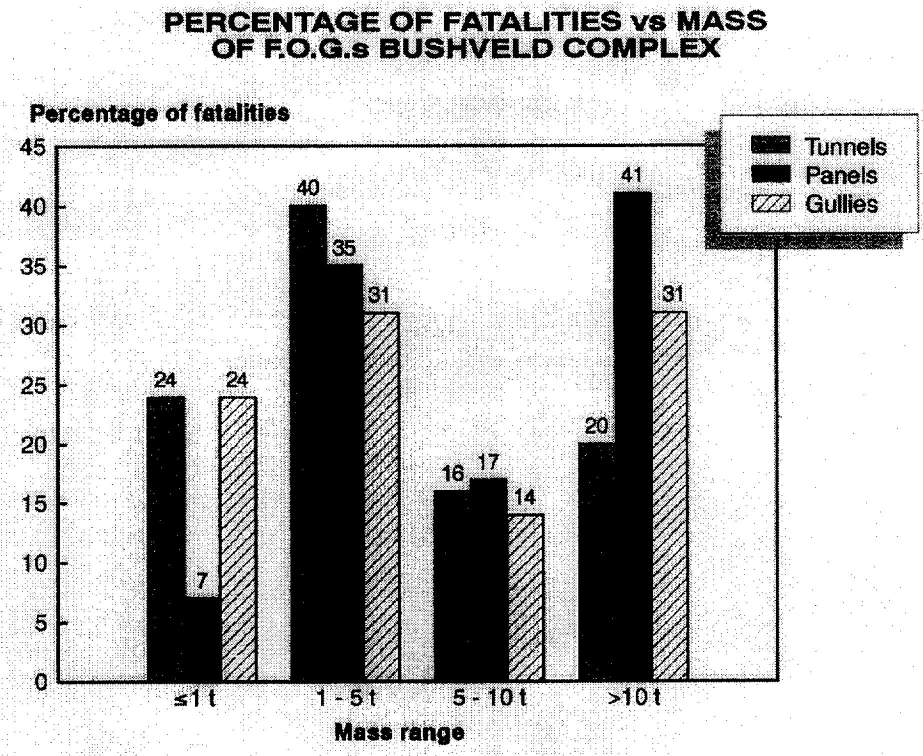


Figure 5a. Example of accident data for tunnels of Bushveld Complex mines



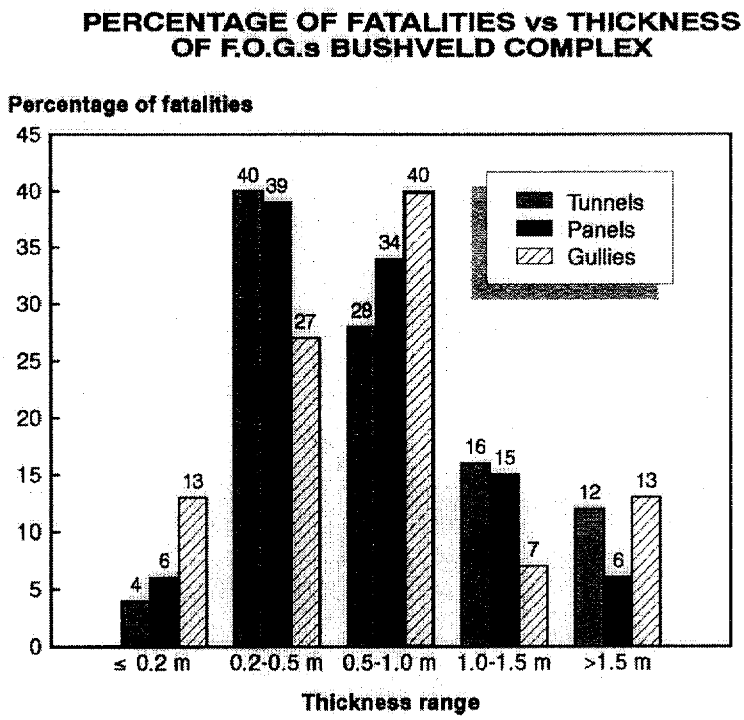


Figure 5b,c. Examples of accident statistics for BC tunnels from the development end. These sections of tunnels usually do not have sufficiently large areal coverage of support installed. Therefore, the main implication of the data obtained should be an improved areal coverage of the temporary / primary support systems used in the vicinity of a tunnel development end. Data on the mass range and the thickness range indicate that given a sufficient areal coverage of the existing support systems, the length of the most of the tendon support as well as their load carrying capacity would be sufficient to prevent a majority of the falls.

3.1 Identification of probable keyblock shapes and sizes for various joint set and bedding combinations for drives and cross-cuts in Bushveld Complex mines

This aspect of the project addresses the work of enabling output 1.2.

On average, variations in mining conditions are much more pronounced in tunnels of the BC when compared with conditions prevailing in the gold fields. It is therefore important to take into account the differences by developing solutions that in many instances would have to be more site specific.

Generally it also appears, that under severe conditions of the Bushveld Complex, the installation of conventional tunnel support, both temporary and permanent, often does not ensure stability of excavations. It is therefore necessary to investigate and develop other, non-conventional supports.

The problem of the fall out of discrete joint bounded blocks in Bushveld Complex tunnels was investigated. For this purpose field data on geological discontinuities that are responsible for potential instabilities in the excavations, were collected. This data was then applied in the SAFEX computer code in order to evaluate the code and it's potential applicability in shallow South

African mines. This code was developed by the CSIRO Australia to analyse the stability of excavations developed in a structurally controlled rock mass environment.

An interim SIMRAC report on this subject matter was written by Wojno and Jager (1994).

A user friendly computer code, that could quickly evaluate the potential for falls of ground and the size and position of potentially unstable key blocks, would be very useful in a preliminary assessment of tunnel stability and potential for gravitational falls. Such a tool would be applicable in tunnels surrounded by intensely jointed and/or stratified rock, as are commonly found in Bushveld Complex mines.

Following field surveys of geological discontinuities in various tunnels on nine BC mines (Amandelbult, Atok, Impala, Northam, RPM: Brakspruit, Frank and Townlands shafts, Union) stability analyses were carried out using SAFEX modules. A summary of results obtained that gives the number of blocks generated and analysed, number of potentially unstable blocks and the mass of the biggest blocks generated around the tunnels are given in Figure 6.

A quick analysis can be performed on such data allowing for a preliminary assessment of the possible effect of change in tunnel direction (haulage vs. cross-cut) with respect of a suite of geological discontinuities on the number and size of blocks generated. The results of such an analysis are also summarised in Figure 6.

Data obtained from such analyses have a number of advantageous and practical implications for mining personnel responsible for the design of tunnel support. Among the most important conclusions that could be drawn is the type, position and required load carrying capacity of the support that is needed for the area under consideration. Another useful aspect, related to data on the

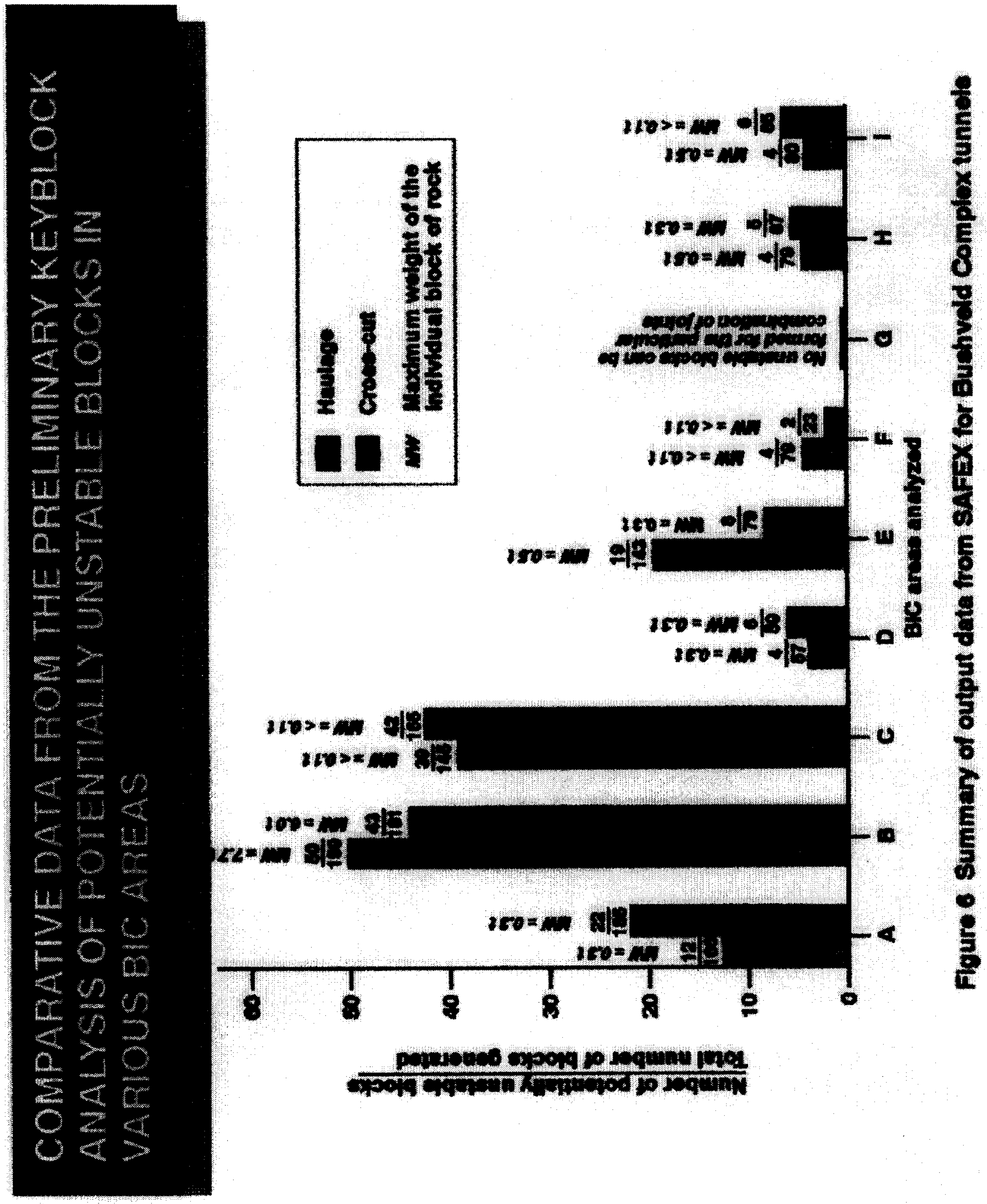


Figure 6. Summary of output data from SAFEX for Bushveld Complex tunnels

size and number of potentially unstable blocks, is in the decision that could be made as to whether or not improved areal coverage support is required.

In the process of investigating any practical potentials of the available techniques to analyse geological discontinuities in a rock mass, with a view to performing a relatively quick stability assessment for support requirements of tunnels in jointed rock, the following conclusions were drawn:

* the SAFEX package was found to be a quick program to perform preliminary stability assessment and analysis of support requirements for tunnels in geologically structured rock.

* the program allows for identification of the potentially unstable blocks, their sizes and the resultant dead-weight or sliding forces. From this the length and the required load carrying capacity of the tendon support can be decided upon, and whether or not fabric support is required.

* the data generated by the various program modules need to be used in conjunction with other considerations that influence rock reinforcement design such as general stress state; ground water conditions; possible failure modes, previous experience, cost; purpose of excavation, etc.

* the influence of stress changes is not taken into account, therefore the program is applicable mainly in lower stress environments where stability of tunnels is controlled by the geological discontinuities in the surrounding rock under the action of gravity loading and not by the stress field.

* the program provides a relatively quick analysis to establish whether the support requirements for different orientations of tunnels, for a particular geology, are different in the same rock mass i.e. haulage vs. x/cut.

* The code can also be used in analysing beforehand the suites of 'typical patterns of discontinuities' in tunnels for specific areas and then be used in a preliminary analysis of their stability and support requirements. This requires a fairly detailed quantitative knowledge of the discontinuities, which in turn is necessary for the design of support for shallow mine tunnels.

* The mines should apply SAFEX in the preliminary stability assessment and analysis of support requirements for tunnels in lower stress environments and geologically structured rock.

3.2 Mechanical Properties of Rocks vs. Observed Stability of the Bushveld Complex Tunnels

This aspect of the project was based on the initial examination of stability of tunnels in the Bushveld Complex and was motivated by an observed discrepancy between the application of the RCF criterion and observed tunnel conditions.

The rockwall condition factor (RCF) that correlates tunnel condition with the major (SIG1) and minor field stress (SIG3) components, uniaxial compressive strength (UCS) of the host rock and an empirical rockmass condition factor (F), has been found to be a useful design criterion for expressing tunnel condition in gold mines and indicating support requirements.

Despite the usefulness of the RCF for gold mine tunnels, its application for platinum mine tunnels conditions had not been evaluated. Much less data on the stress environment and its effect on stability of tunnels are available for the platinum mines of the Bushveld Complex. However, underground observations of tunnel damage and other evidence indicate, that high horizontal virgin stresses are present in parts of the BC. Mean k-ratios as high as 2,6 were calculated from stress measurements carried out at RPM

(Rustenburg Section). One of the important consequences of the relatively high horizontal virgin stresses is the formation of a "gothic arch" type damage to the hangingwall of tunnels. Thus there is a significant difference in tunnel behaviour in these conditions compared to tunnels at similar depths in gold mines. Recommendations made for the strata control of gold mine tunnels can be therefore totally inappropriate for tunnels in situations where the k-ratio is high. For example overstoping of tunnels is carried out to stress relieve tunnels in gold mines and improve conditions. The effect of overstoping in areas with high k-ratios is to remove the confining stress and hence cause deterioration of the tunnel.

In an attempt to answer the question on applicability of the RCF criterion for the Bushveld Complex tunnels, an extensive investigation into the overall stability of over 45 km of tunnels has been carried out. To this effect, the overall condition of these tunnels was evaluated, a photographic record of the representative tunnel conditions prepared, MINSIM runs performed and rock samples representative of each type of tunnel condition were collected and tested in the laboratory. In total over 210 tests have been performed.

An example of a geological column together with the UCS for various rocks through which the evaluated tunnels were developed is shown in Figure 7.

Field data were collected as per a check list as shown in Figure 8.

Data on stress components obtained from the MINSIM runs for a k-ratio of 2,6 (average k-ratio calculated from field stress measurements in BC) was used to calculate the RCF values. The RCF values varied between 0,7 to 13,3. Figure 9 illustrates a frequency distribution of the RCF values for the BC tunnels that were investigated.

BUSHVELD IGNEOUS COMPLEX - MINE "A"

- **Most problems with stability of x/cuts in pyroxenites**
- **Haulages - generally stable (in norites) - except for serpentinized fault zones**
- **Delineating fault zones by diamond drilling from haulage development ends**
- **Distribution of serpentinization in pyroxenites - random**

No.	Average UCS of tested rock (MPa)	Range of UCS (MPa)
①	104	78 - 127
②	72	62 - 82
③	263	241 - 276
④	247	234 - 260
⑤	126	117 - 132
⑥	164	197 - 132
⑦	92	90 - 96
⑧	176	160 - 193
⑨	151	128 - 192

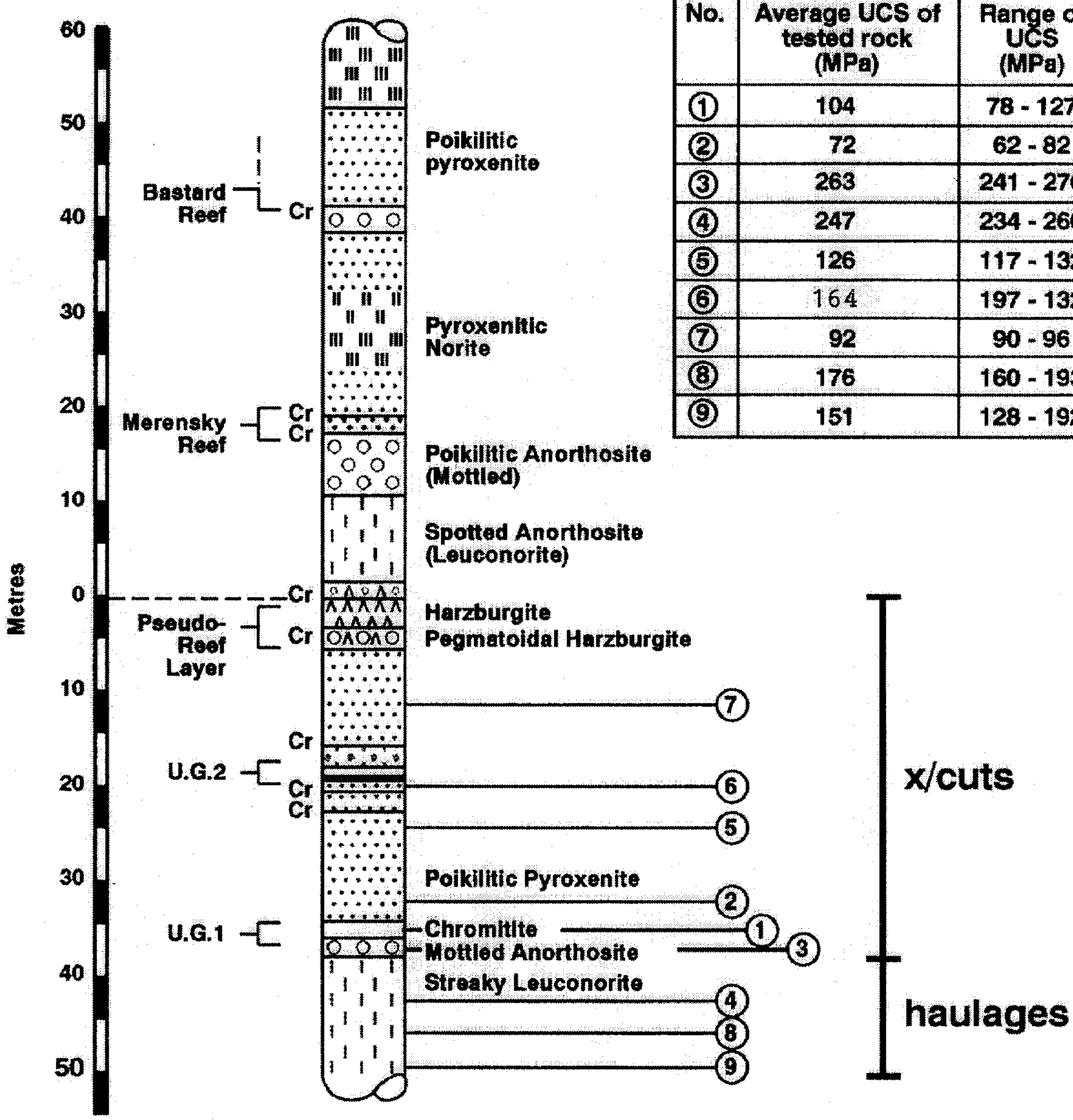


Figure 7. Geological column for typical Bushveld Complex mine

Date:

1. Mine / area / level / dev. end or existing tunnel

2. Tunnel size, m Tunnel shape: Depth Below Surface

3. Tunnel orientation

4. Type of rock (available rock testing data)

UCS, MPa

5. **Major geological discontinuities (max. 3)** Brief Description
....
....

a. Dip .... Dip Direction .... Spacing
b. Dip .... Dip Direction .... Spacing
c. Dip .... Dip Direction .... Spacing

7. **Data on stress/stress changes available:(Y/N;** k-ratio ?)

Area(s) for which MINSIM runs are required Maps to collect (1:2500; 1: 1000; 1:200)

8. Extent of fracture zone, m (from mini petroscope observations)

Face Sidewalls H/Wall

9. Fracture intensity and orientation:

10. Typical photographic record (min 4 to 6 photos):

Overall situation S/Walls H/Wall

11. Support (type, length, spacing, shoterete thickness, etc)

a. primary
....I
b. secondary
....
c. other
....

12. Comments (adjacent excavations, seismic activity, blasting practiceldamage, etc)
....I
....
....

Figure 8. Check list for tunnel survey's

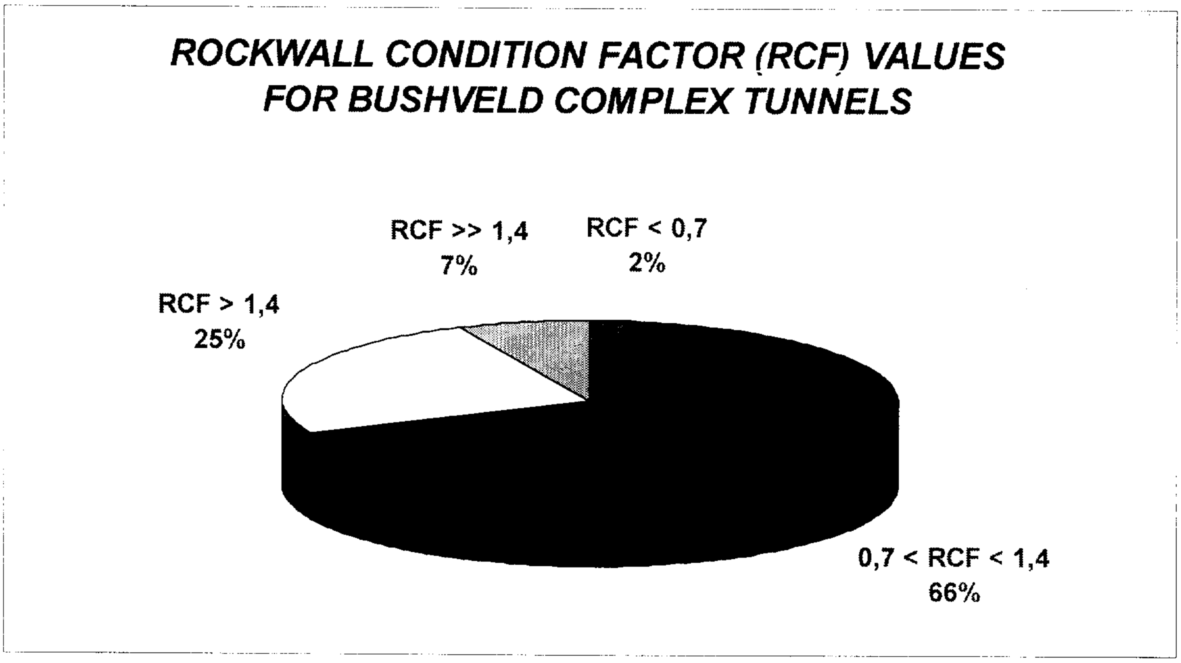


Figure 9. Frequency distribution of RCF values for Bushveld Complex tunnels

Subsequent analysis of the RCF values vs. observed condition of tunnels indicated that in many cases the calculated values were not related to the overall condition of those tunnels or the support systems implemented.

Preliminary analysis of the data obtained so far indicate that the RCF criterion in it's present form, is not readily applicable to the Bushveld Complex rockmass. The main reason being that most of the observed damage to the BC tunnels was found to be controlled by the complex geology of the rockmass rather than stress concentrations around excavations

4 Laboratory and field evaluation of currently used tunnel support units and systems

This aspect of the project addresses enabling outputs 2.4, 2.5, 2.7 and 3.2.

This area of research activities was subdivided into three sections:

4.1 Current tunnel support practice in South African gold and platinum mines

Investigations into current tunnel support practices in South African gold and platinum mines, involving a literature and mine survey encompassing all aspects of the subject was undertaken and results have been presented in a form of a report by Roberts (1995). It contains information on various support elements and systems including their construction, mechanical behaviour and interaction with the surrounding rock. A discussion of the most important support characteristics of various systems in use is also included in this report.

In discussion of the investigations the following objectives have been achieved:

- The literature relevant to support types, systems, loading and operation has been reviewed. In the report the mechanics of operation of the support elements have been discussed. Data was obtained from most of the support manufacturers about their products.

- In terms of the objectives of SIMRAC project GAP026, the detailed report on the subject matter that was written forms a basis upon which further work may be built. The analytical solutions presented may prove useful in evaluating the results from tests that have been completed and those that are contemplated for the future. A number of areas that require further research have been identified and they are given in the Chapter: Conclusions and Recommendations from the Project.

4.2 Laboratory investigations into the shear strength of the most commonly used support tendons.

Since only limited data on the failure of rock tendons under transverse shearing conditions was available, and in response to the SIMRAC committee request to quantify the problem - a series of comprehensive investigations into the matter were carried out.

Shear damage is particularly severe in a slabbed or fractured rockmass subjected to stress changes. Given the fractured nature of the rockmass surrounding most deep level tunnels, it is to be expected that even small rock movements will impart shear deformation to any type of tendon, particularly those that are in full contact with the borehole. These deformations may be large enough to significantly weaken or even break the tendon. Failure of such tendons involves a number of mechanisms, including combinations of bending, shear and tensile loading.

A literature review was carried out to gain information on the complexity of the problem and determine the effect of such parameters as bolt diameter, friction angle, rock type, bolt inclination and joint dilatancy on shear damage to tendons. Moreover, examples of analytical models were investigated, that relate shear load or stress to shear displacement, quantitatively evaluate the influences of the various parameters involved, enable prediction of ultimate loads and displacements, and qualitatively at least, determine the stress distribution in a grouted bar subjected to shear loading.

To be able to perform shear tests in the laboratory, two types of shear apparatus were used:

- double shear apparatus that had to be modified to ensure accurate representation of loading conditions as encountered underground, and

-single shear testing rig which was developed to enable easier interpretation of the test results than those obtained from double shear tests, where difficulties occurred in ensuring perfect symmetry of loading forces.

Schematics of the double and single shear rigs are shown in the Figures 10a and 10b. respectively.

The tests were carried out under both static and dynamic loading. Table 4. summarises and outlines the results obtained from the shear tests.

Table 4. Static and Dynamic Shear Strengths and Displacements of Tested Tendons

Tendon	Tensile Strength [kN]	Static Shear Strength [kN]	Static Shear Strength [% of UTS]	Dynamic Shear Strength [kN]	Dynamic Shear Strength [% of UTS]	Dynamic Shear Strength [% of static]	Loss in Strength Stat. to Dyn [%]	Static Disp. at Failure [mm]	Dynamic Disp. at Failure [mm]
Smooth bar (16mm)	116	120	103.4	122	105.2	101.7	0.0	34	33
Rebar (16mm)	156	140	89.7	124	79.5	88.6	11.4	33	32
Twist bar (12mm sq.)	139	137	98.6	63	45.3	46.0	54.0	33	32
V-bar (16mm) *	120	85	70.8	38	31.7	44.7	55.3	11-20	10-15
Cone bolt (16mm)	116	190	163.8	120	103.4	63.2	36.8	68	33-65
Rope #1 (12 mm)	117	116	99.1	76	65.0	65.5	34.5	40	-
Rope #2 (12 mm)	135	142	105.2	73	54.1	51.4	48.6	42	23-30
Rope #3 * (14 mm)	243	242	99.6	179	73.7	74.0	26.0	37	33
Rope #4 (16 mm)	221	227	102.7	118	53.4	52.0	48.0	39	-
Split Set (SS39)	110	97.7	88.8	60	54.5	61.4	38.6	31	26-33

* Results are for double shear tests

The following conclusions were reached based on the results of the tests performed in this investigation:

- Tests were performed successfully in the double shear apparatus, however, due to difficulties in interpretation of the curves produced, especially under dynamic conditions, further experiments to obtain shear strength of steel

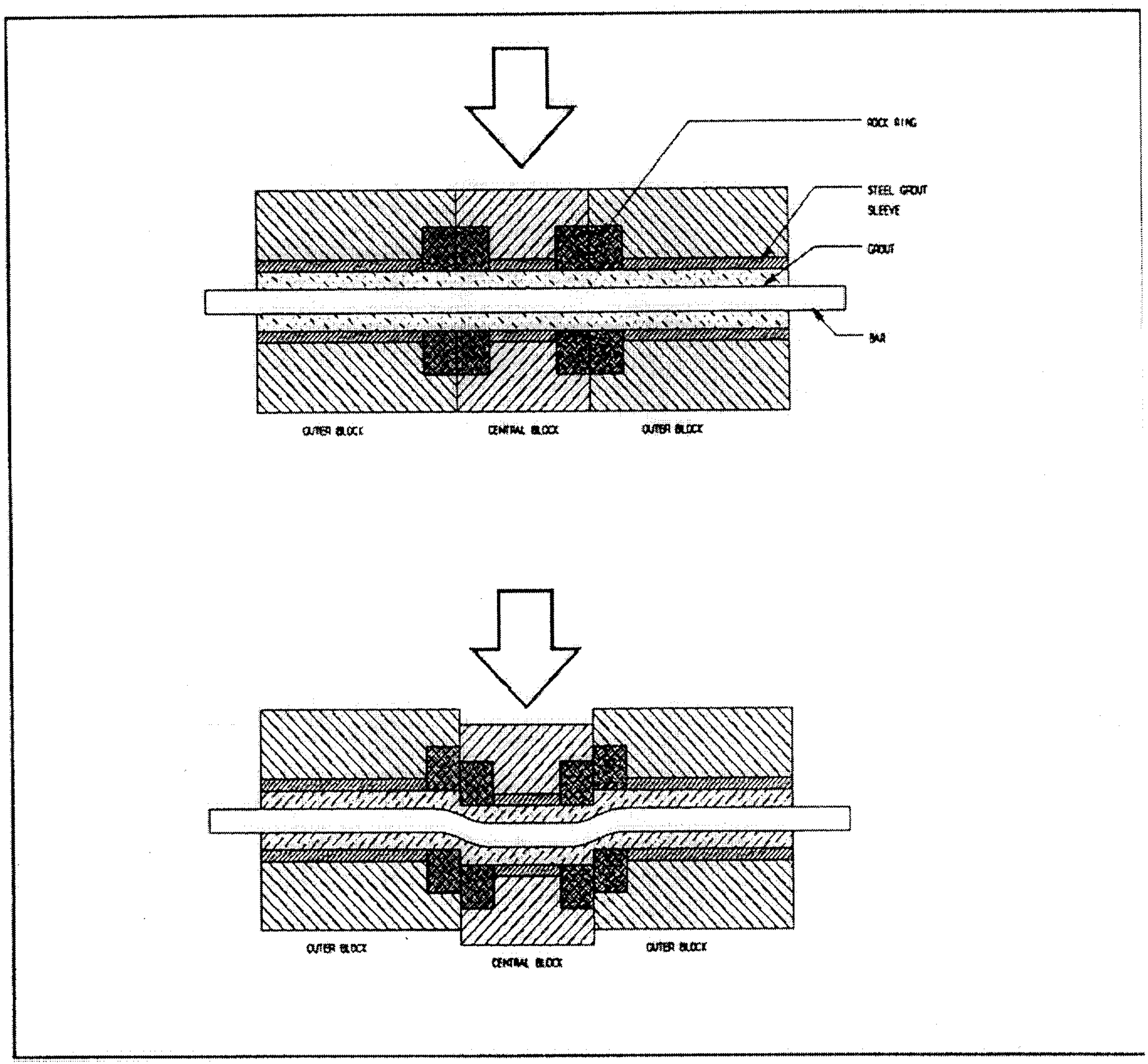


Figure 10a. Schematic of double shear test rig

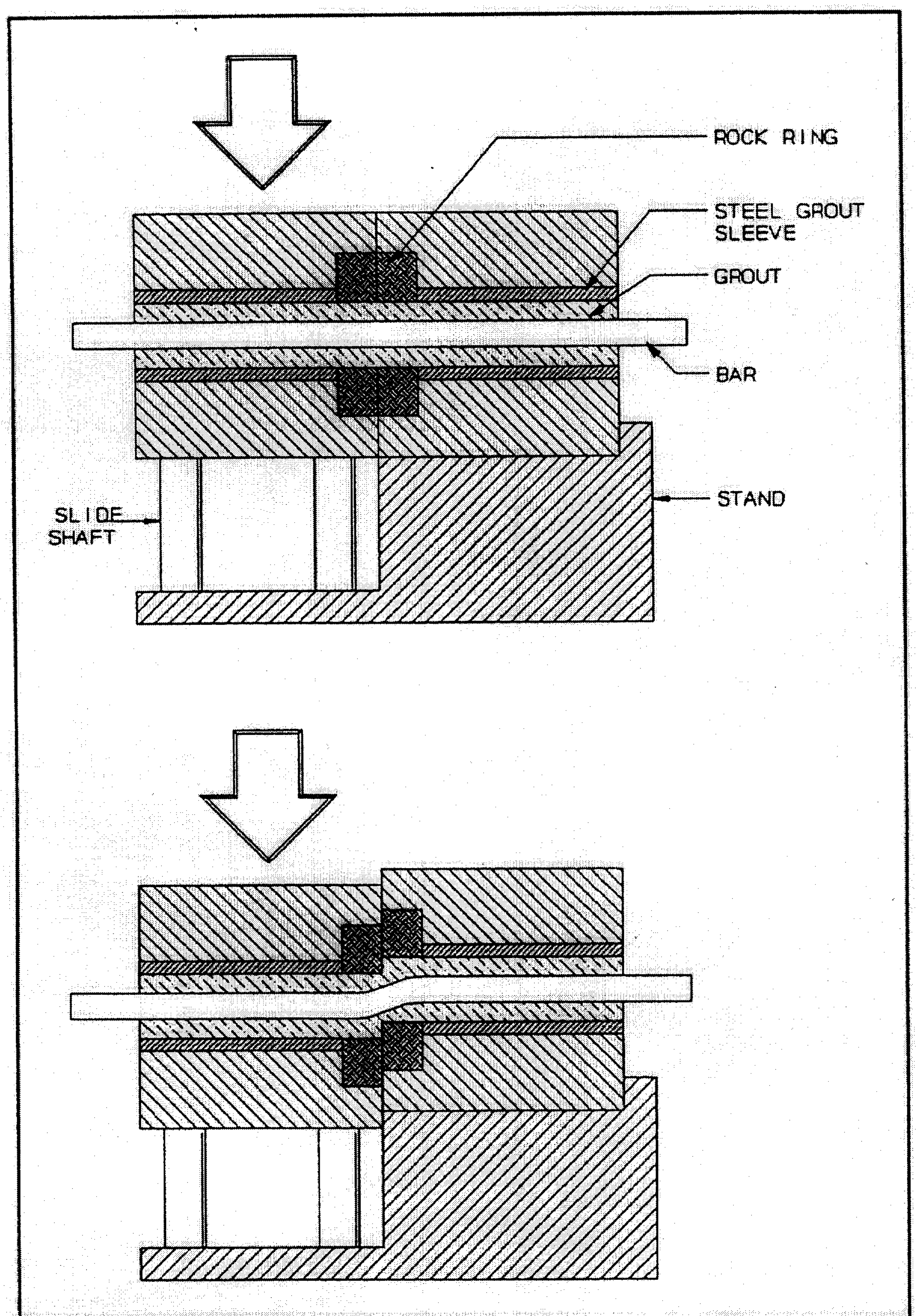


Figure 10b. Schematic of single shear test rig

tendons should not be carried out in the double shear rig. The difficulties stem mainly from the fact that two shear surfaces were loaded but did not always fail simultaneously.

-The single shear test rig is adequate to provide meaningful shear \strength data.

- Grouted smooth bar had a shear strength of 120 kN under both static and dynamic conditions. This figure was equal to the tensile strength of the bar.

- Rebar had a higher tensile strength and consequently a higher grouted shear strength.

- The static shear strength of the twist bar was 137 kN, approximately equal to the tensile strength of the bar. It was found that the twist bar lost over 50 per cent of its static shear strength under dynamic conditions. This was attributed to the reduced toughness of the bar introduced by cold-working of the steel.

- The cone bolt presented 170 kN of shear resistance, nearly 64 per cent greater than the tensile strength of the bar, and similarly greater than the grouted shear strength of the smooth bar. This increase was considered to be a function of interference of the bar along the shear plane and to the fact that the deformation of the cone bolt had allowed the bar to fail in the most favourable orientation possible, i.e. completely in tension. The effective shear strength of the cone bolt dropped to that of a smooth bar of the same diameter under dynamic loading conditions. It was noted that the position of the cone relative to the shear plane may have affected the dynamic shear strength. Given the high resistance presented by the cone bolt, it is recommended over the smooth bar, rebar, Split Set and twist bar in applications where shear movements in the rock are predicted.

Destranded hoist cable ropes of diameters greater than 14 mm had the greatest grouted shear strength of all the tendons tested. The shear strengths were all within 6 per cent of the tensile strengths of the ropes, although it must be pointed out that a comparatively small number of tests were performed. Displacements at failure were larger than those noted for solid bars apart from cone bolts. Under dynamic loading the ropes lost at least 26 per cent, and in some cases up to 50 per cent, of their shear strengths. In terms of shear resistance, shear stiffness and displacement at failure, the ropes are recommended over all the other tendons tested, with the exception of the cone bolt.

The results of the test programme raised some questions and indicated where further work would be of practical benefit. The more important of these are:
- the effect of using larger diameter bars in the same diameter boreholes; - it is expected that the shear resistance of certain bars would increase linearly with increasing cross sectional area. However, in the case of the cone bolt, where the strength of the tendon lies with the fact that the failure mode is altered, an increase in bar diameter may reduce the capability of the bar to bend in such a way that tensile failure occurs. Hence it may be that the shear strength of the 22 mm cone bolt is not significantly greater than that of the 16 mm cone bolt.

Another area, in which questions were raised, was that of the clamping of the ends of the bars or ropes. In this investigation the ends of the tendons were clamped to stimulate a laterally moving crack positioned such that the tendon was embedded beyond its critical bonded length on both sides of the crack. Further work could investigate the effect of having the end of the tendon free, on one or both sides of the shear plane. This would simulate the situation where a crack begins to move at a position close to the end of a tendon.

One of the most important questions this investigation has raised is what the effect of shear deformation is on the pull-out strength of the different tendons.

To clarify this matter it would be necessary to construct equipment capable of imposing shear deformation and, at the same time, pulling the free end of the tendon. It would be feasible to construct a shear rig which could easily be manoeuvred such that the tendon end could be gripped for pull testing. Such a study would quantify the effect of shear deformations on the most important functions of tendon-rock reinforcement and tensile performance.

4.3 Laboratory and in-situ testing of mesh and lacing support systems

Despite the importance of wire mesh and rope lacing support systems for the overall stability of tunnels, and the fact that wire mesh makes up the largest proportion of all material costs for mine tunnel development, a number of very important quantities related to these systems are not known. For example, the strength of the various types and sizes of mesh (under loading perpendicular to the strand direction) is not known, even to the manufacturers of these mesh products. It seems that wire rope for lacing is also somewhat arbitrarily selected. Without such information, the rock mechanics practitioners cannot accurately estimate the strength of proposed support systems and are expected to rely solely on empirical techniques.

The aim of this investigation was to quantify the contributions of the mesh and lacing towards the overall strength of a support system, and to integrate these quantities in a more rigorous design rationale. The parameters which most influence the performance of these units and systems were also identified. Subsequently, recommendations regarding mesh type, mesh strength, lacing strength and lacing pattern can be made.

The means by which these quantities and parameters were to be determined was by experiment. A laboratory panel type test was used for comparative tests, while underground tests were conducted to quantify the performance of different support systems in-situ.

Laboratory Testing

A frame and loading device for the testing of fabric type support was designed and manufactured. An isometric view of the mesh panel testing apparatus used in the laboratory is shown in Figure 11. The apparatus is positioned below beams of the TerraTek testing machine frame. The TerraTek has the capacity to impart both quasi-static and dynamic loads. The mesh types selected for preliminary testing represented the most common types encountered as support underground. These were as follows:

Weldmesh	- 75 mm aperture, 3,15 mm wire diameter
	- 100 mm aperture, 3,15 mm wire diameter
Diamond mesh	- 50 mm aperture, 3,15 mm wire diameter
	- 75 mm aperture, 3,15 mm wire diameter
	- 100 mm aperture, 3,15 mm wire diameter

The testing schedule was similar to that applied to the shear tests. Two static and two dynamic tests were performed on each type and size of mesh.

It was found that the selected method of testing mesh panels in the laboratory did not accurately represent the situation encountered underground, for example: clamping the panel edges introduced boundary conditions that did not simulate properly the tensioning effect of long mesh runs of installed support. The clamping of the mesh panels by the edge plates moreover caused failure mechanisms in some of the mesh types, that were not typical of underground environments and hence, it is considered, led to premature failure that could have distorted the results.

It was therefore decided, that a final analysis of the laboratory data should be carried out once the underground tests, discussed in the following chapter

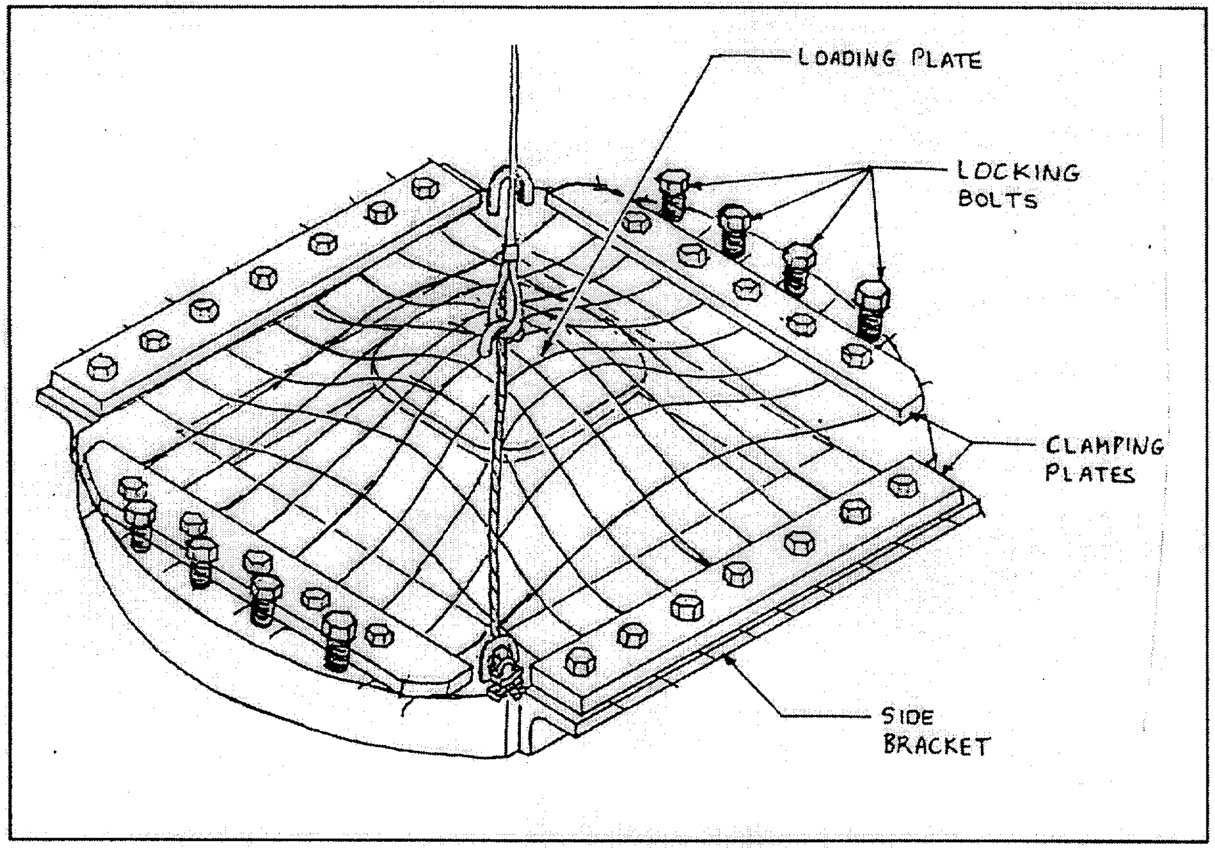


Figure 11. Isometric view of mesh panel testing apparatus

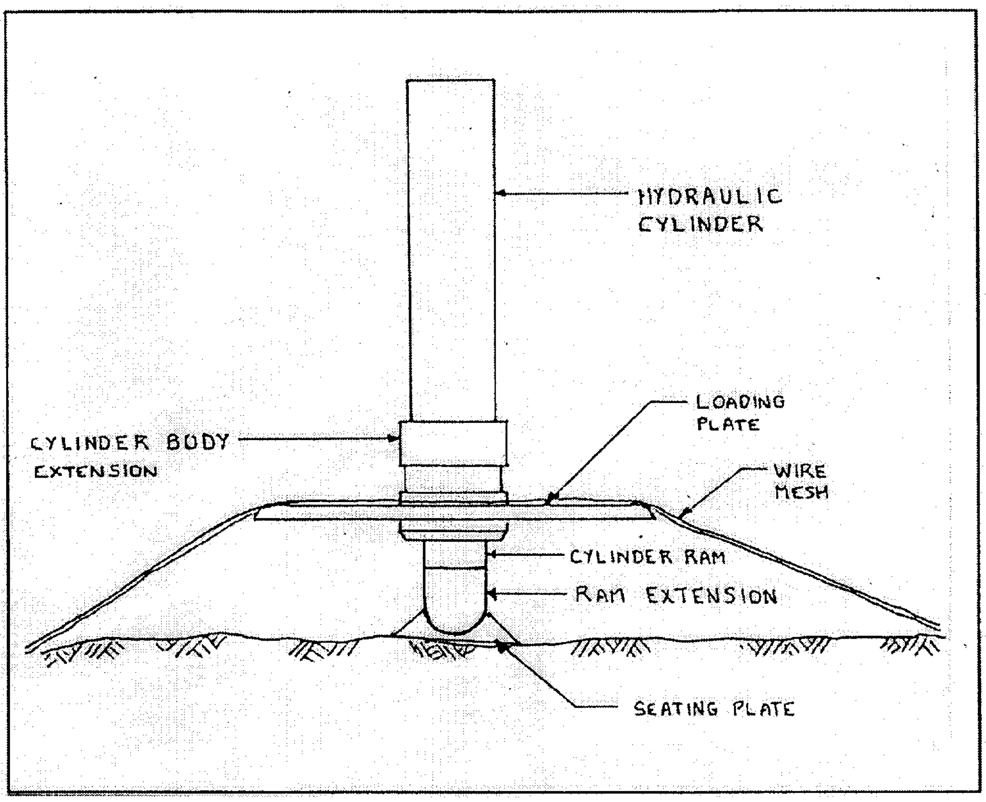


Figure 12. Diagram of in situ mesh and lacing testing rig

have been completed. By doing so it will be possible to "calibrate" laboratory data against field results for the same mesh as well as avoid any potential problems with interpretation of the laboratory test results under different loading and boundary conditions to those encountered underground.

Field investigations on conventional mesh and lacing systems are expected to be completed by the end of January 1996.

To optimise the crucial relationship between support performance and cost, the possible use of alternative materials for mesh were also investigated. Plastic mesh and expanded metal were identified as possible alternatives to wire mesh. Analysis of these areal coverage support systems is presented below.

Expanded metal panels

Panels of four types of expanded metal were tested. These types were specified as VEM330A, VEM420E, VEM6355H and VEM6360F. The tests were conducted with the panels clamped over a span of 800 mm, loaded by a 250 mm square plate. Two tests were conducted on each type of mesh.

The tests on VEM330A mesh indicated an ultimate load carrying capacity of 13 kN, with failure occurring at one "strand" intersection adjacent to the loading plate

On VEM429E, panels multiple failures occurred at "strand" intersections adjacent to the loading plate. The maximum loads recorded were 18 kN and 14 kN, giving an average strength of 16 kN.

VEM6355H had flattened "strand" intersections, as opposed to the "standing" intersections of the previous mesh types. Failures were observed between and adjacent to intersections, and occurred adjacent to the plate and the edges of the panels. The maximum loads recorded were 16 kN and 13 kN, giving an average of 14,5 kN. Note that the panels took load up to 200 mm displacement.

VEM6360F also had flattened intersections. Slipping from under the clamping plates was observed in one of these tests. The other test indicated a strength of 29 kN. Failures were once again observed between and adjacent to intersections, and occurred throughout the panel.

Polypropylene mesh

At the request of one of the GAPREAG committee members panels of two types of polypropylene mesh were also tested. These two types were specified by Trempak Trading as LBO 201 and LBO 301. The tests were conducted with the same set-up as for the expanded metal panels. Two tests were conducted on each type of mesh, and an additional test, conducted at a higher temperature, was performed on the LBO 301 mesh.

In all tests the following pattern was observed. Once sufficient tension was mobilised in the panels, the individual fibres making up the mesh began to fail. The structure was therefore weakened progressively without immediately losing it's load carrying capacity. failure of the mesh strands occurred nearly simultaneously, with a large number of strands failing, resulting in a very limited post-failure load carrying capacity, The results for the LBO 201 panel show that the mesh absorbed load with increasing stiffness until an abrupt, "violent" failure occurred. Ultimate loads were 6,6 kN and 8,1 kN, yielding at an average strength of 7,4 kN.

The results for tests performed on LBO 301 panels indicated similar load-displacement trends to those observed for LBO 201. There was, however, a lack of consistency in the results obtained. The ultimate loads recorded for the tests performed at room temperature were 9,4 kN and 5,4 kN. The results for the tests performed at 50 Deg C indicated an ultimate load of 6,4 kN. However, it is expected that for very much higher temperatures that are generated during underground fires, the load carrying capacity of any polymer material would become negligible, or if exposed to an open flame drop to zero. Another major disadvantage of the polymer mesh is that it melts when exposed to an open flame. The toxicity of the fumes was not tested as it appears that the meshes tested have little if any support application underground.

Field Testing

In order to quantify the field performance of different mesh and lacing support systems a special test apparatus was developed, and manufactured. The in-situ mesh and lacing testing apparatus is shown schematically in Figure 12. The loading plate has a diameter of 250 mm and the load capacity of the hydraulic ram is 150 kN. A photograph illustrating a field test performed on a grouted tendon, mesh and lacing system on one of the gold mines is shown in Figure 13.

Preliminary field tests were conducted at Kloof Gold Mine on
100 mm x 3,15 mm weld mesh. The shepherd crooks anchoring the mesh were positioned on a 1,5 m staggered pattern (Figure 16d) with straight and diagonal 16 diameter lacing ropes as illustrated in Figure 16c. Tests were

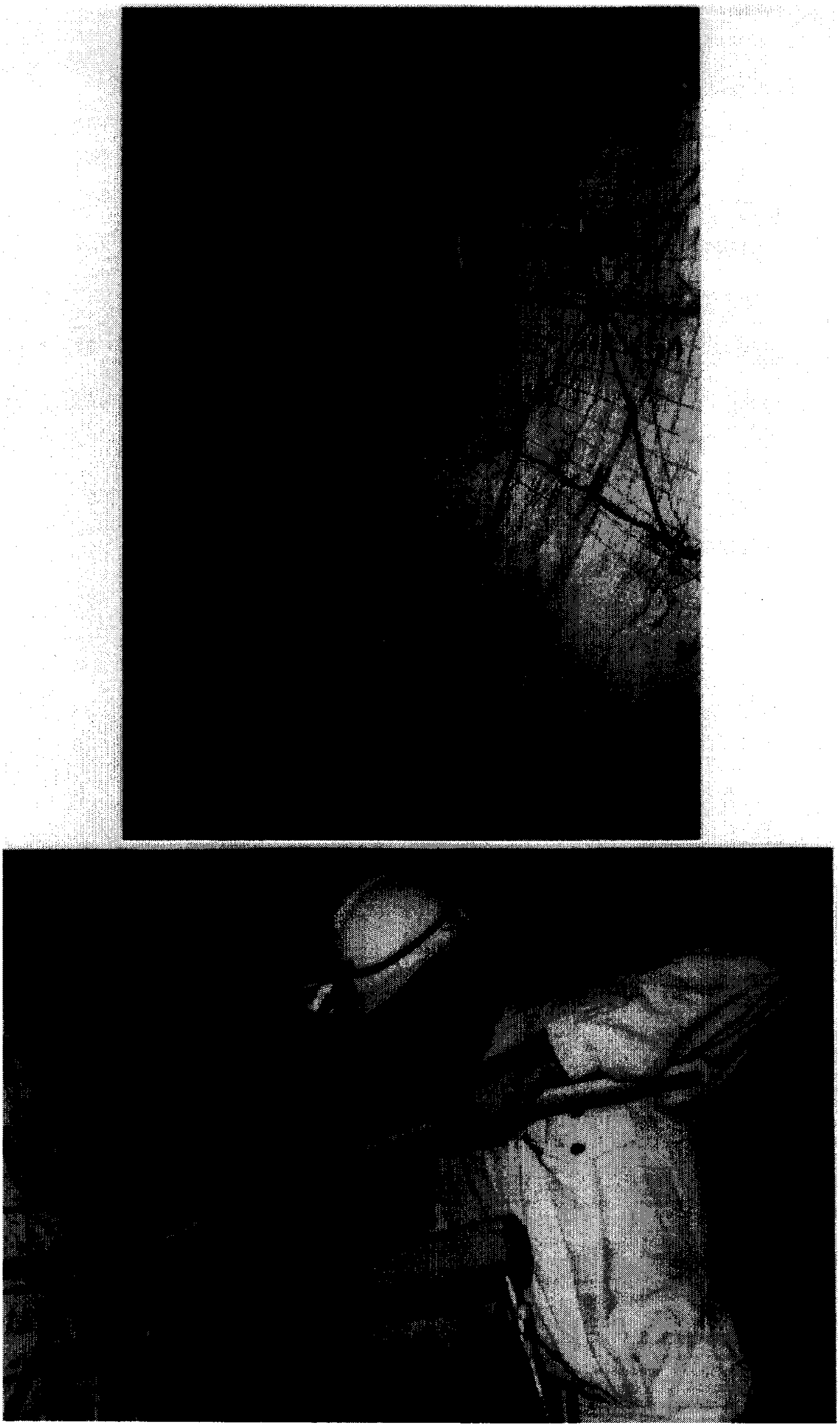


Figure 13. Photographs of in situ mesh and lacing tests

performed at three points in the mesh and lacing system, as illustrated in Figure 14 to obtain data on the variation in stiffness and deformability across the lacing pattern.

Position A represents the weakest point in the system, where initially only the mesh is loaded.

Position B is under one of the lacing ropes midway between tendons and at position C, close to the tendon, two or more ropes are immediately loaded

The Load-Displacement curves obtained are presented in Figure 15.

In positions A and B the 340 mm stroke of the cylinder was exhausted. At point C it was found that further displacement could only be achieved through the application of a load in excess of the tensile strength of the tendon as Figure 15 indicates. No mesh or lacing failures were induced by the test.

The stiffness of the responses at each point up to 20 cm displacement were as follows:

Point A	-	0,22 kN/cm
Point B	-	1,59 kN/cm
Point C	-	2,5 kN/cm

A number of questions were raised in this investigation, as outlined above. It is clear that the best way to try and answer these questions is to perform more tests on mesh and lacing systems in-situ. At present, various types of mesh and lacing in 7 different configurations have been installed at an underground site at 1# East Driefontein Gold Mine. The equipment described above is being used to test these systems.

The following support patterns were installed and are being tested:

•

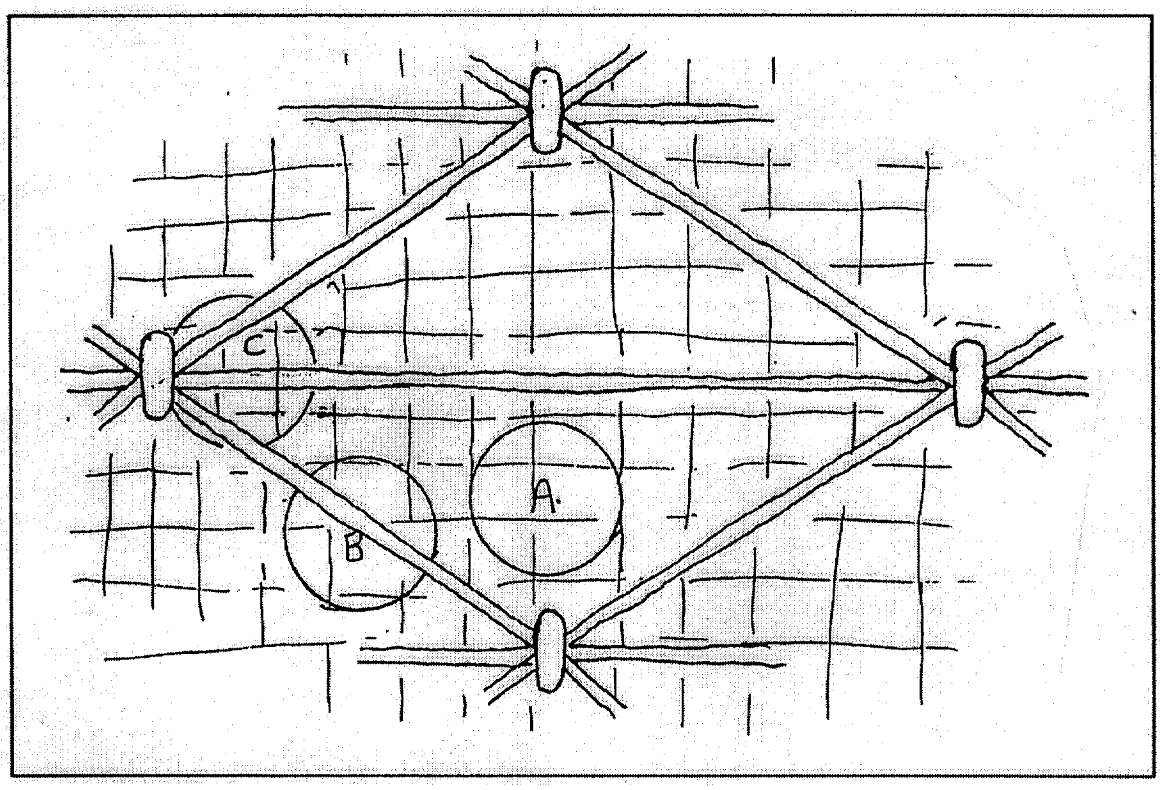


Figure 14. In situ load testing points for mesh and lacing systems

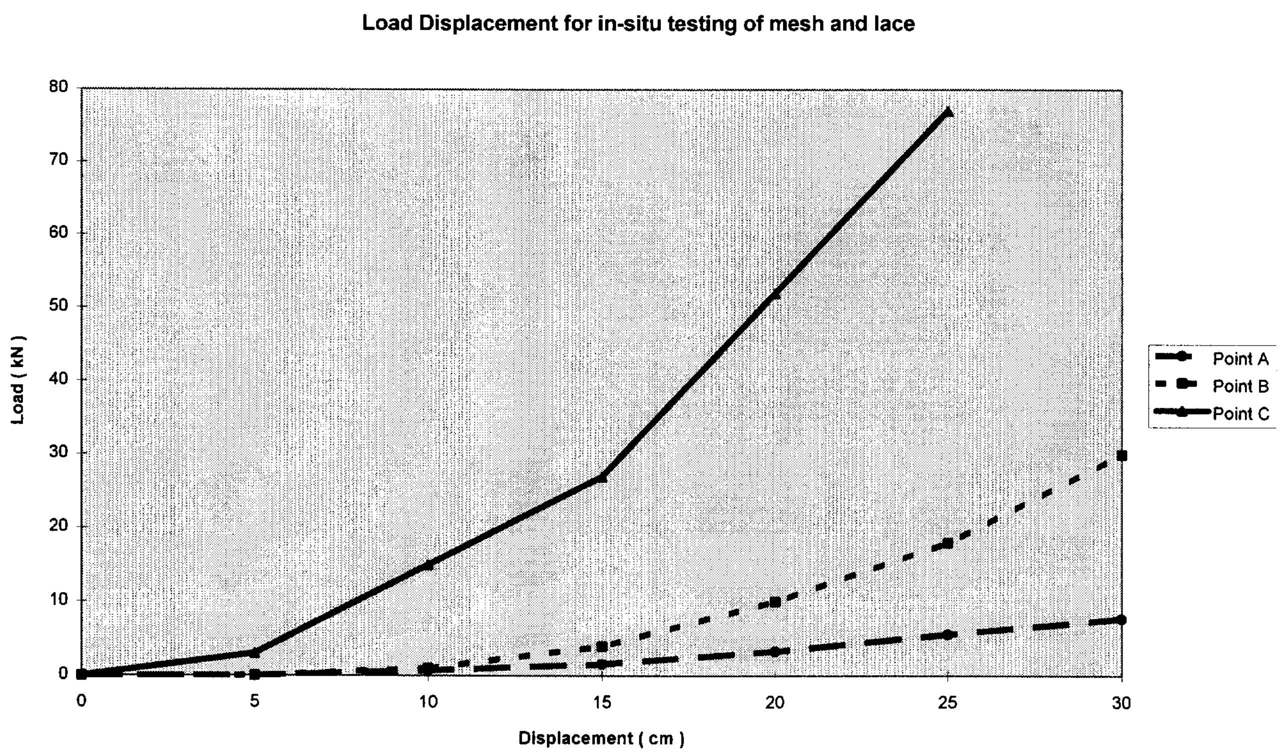


Figure 15. Load deformation graphs for mesh and lacing system tested in-situ

* 2m staggered pattern with diagonal lacing
* 2m staggered pattern with diagonal and horizontal lacing
* Alternate 2m staggered pattern with diagonal and horizontal lacing (East Driefontein configuration)
* 1,5m staggered pattern with diagonal lacing
* Double 1,5m pattern (0,75m square pattern) with diagonal lacing
* 1m square pattern with diagonal and horizontal lacing
* 1m staggered pattern with diagonal lacing

The above systems are schematically shown in Figures 16a to 16 g. To obtain data on the influence of the wire mesh on the system performance four different types of wire mesh will be tested with each support pattern. These are 50 mm and 100 mm diamond mesh as well as 50 mm and 100 mm weld mesh as schematically illustrated in Figure 17.

5. <u>Evaluation of the in situ performance of long cable tendons</u>

This aspect of the project addresses the enabling outputs 2.1 and 3.1.

As part of the field evaluation of currently used tunnel support units an in-situ experiment was carried out to establish the performance and applicability of cable tendons as support for tunnels at depth, where field stress changes and seismic activity occur. This work was carried out by Mr. J Laas, a collaborator from Vaal Reefs Exploration and Mining Co. Ltd., as the topic for an MSc thesis. The work was partly funded by SIMRAC. Appendix A of this report contains the raw data tables extracted from the MSc thesis for reference purposes, more detailed reference to this work should be made from the original thesis.

A site was selected where the various support types being investigated could be installed into similar rock strata, and which would subsequently be subjected to similar induced stress changes during the experiment. Different

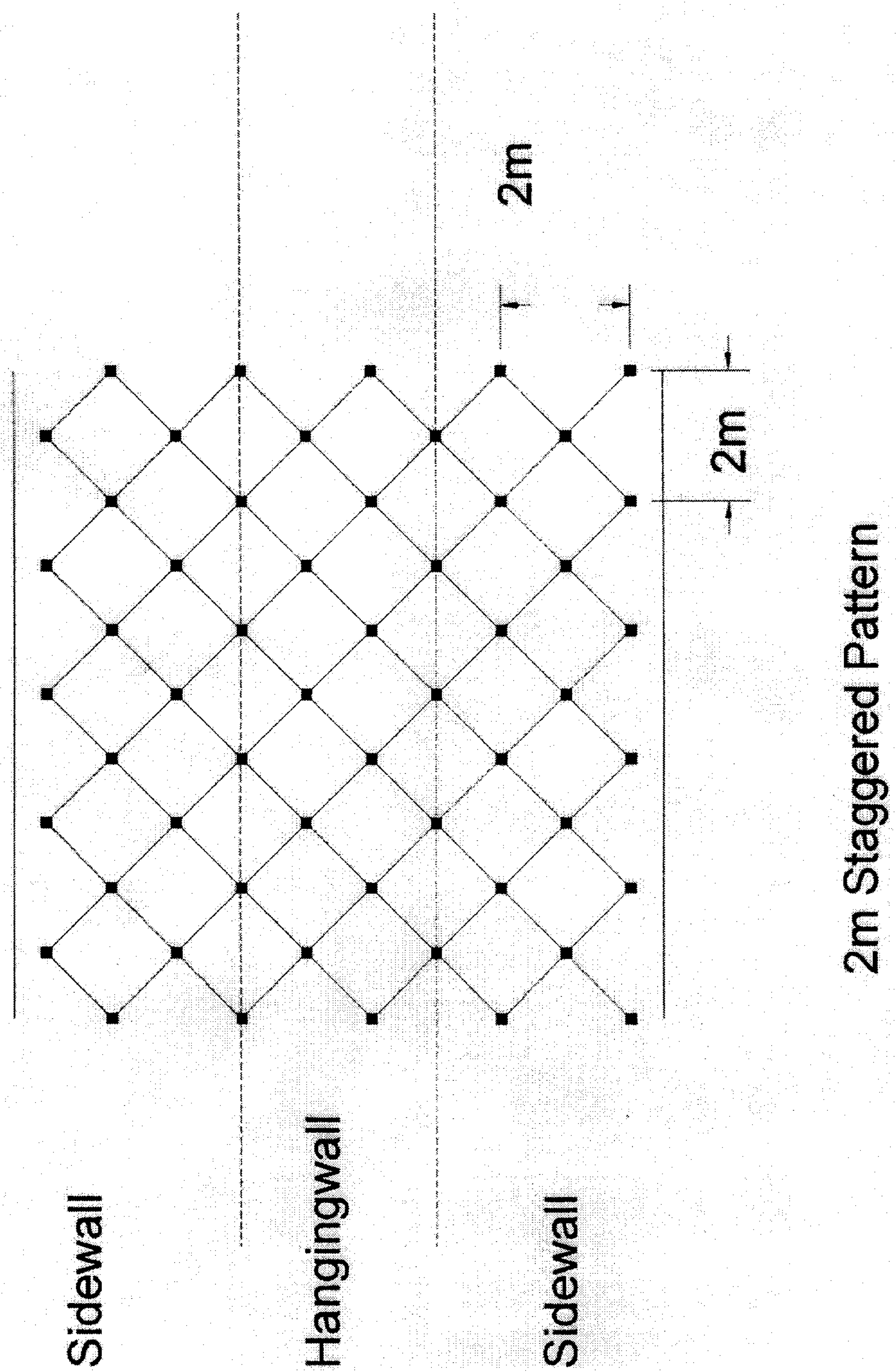


Figure 16a. Example of 2m staggered mesh and lace pattern

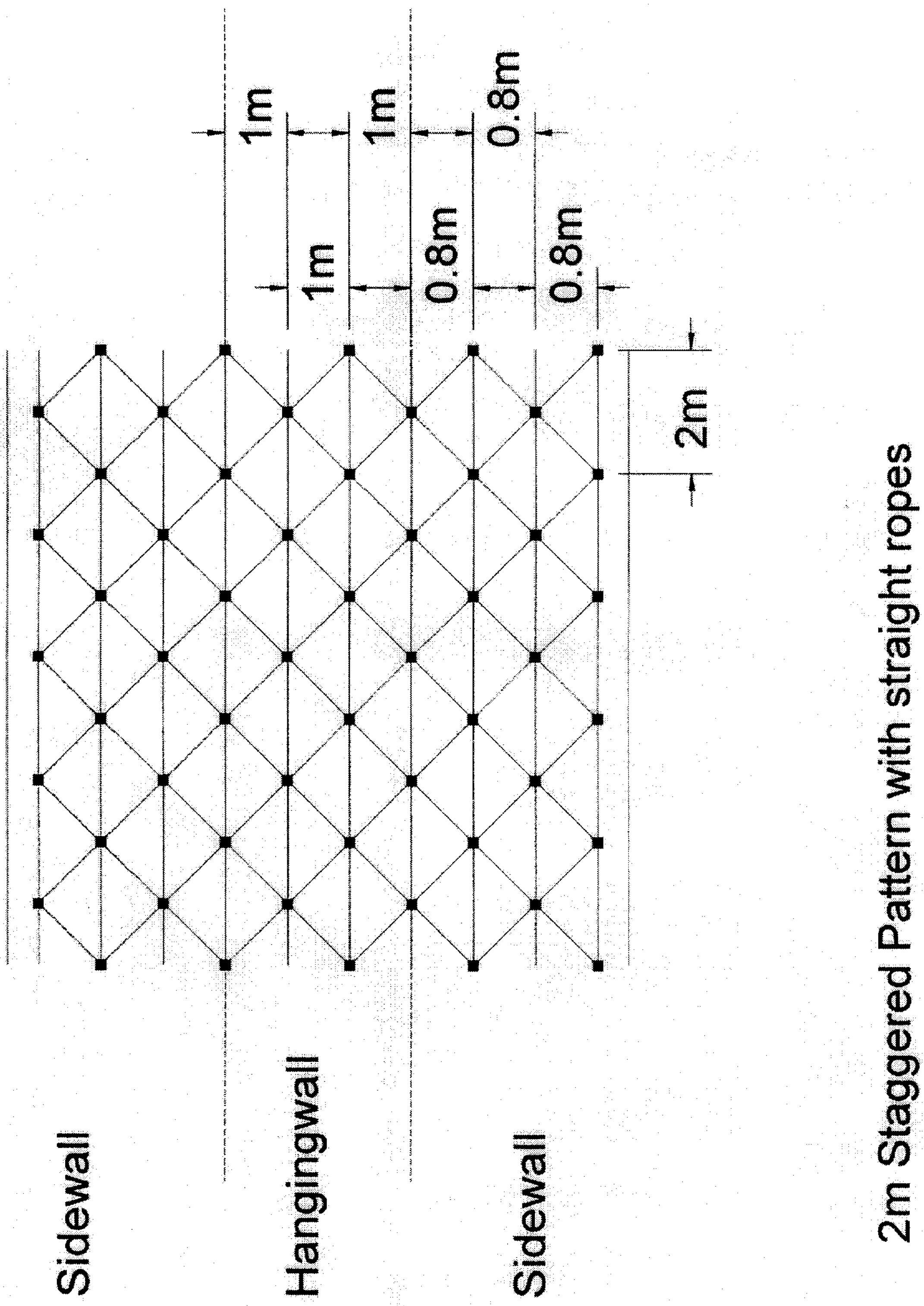


Figure 16b. Example of 2m staggered mesh and lace pattern with straight ropes

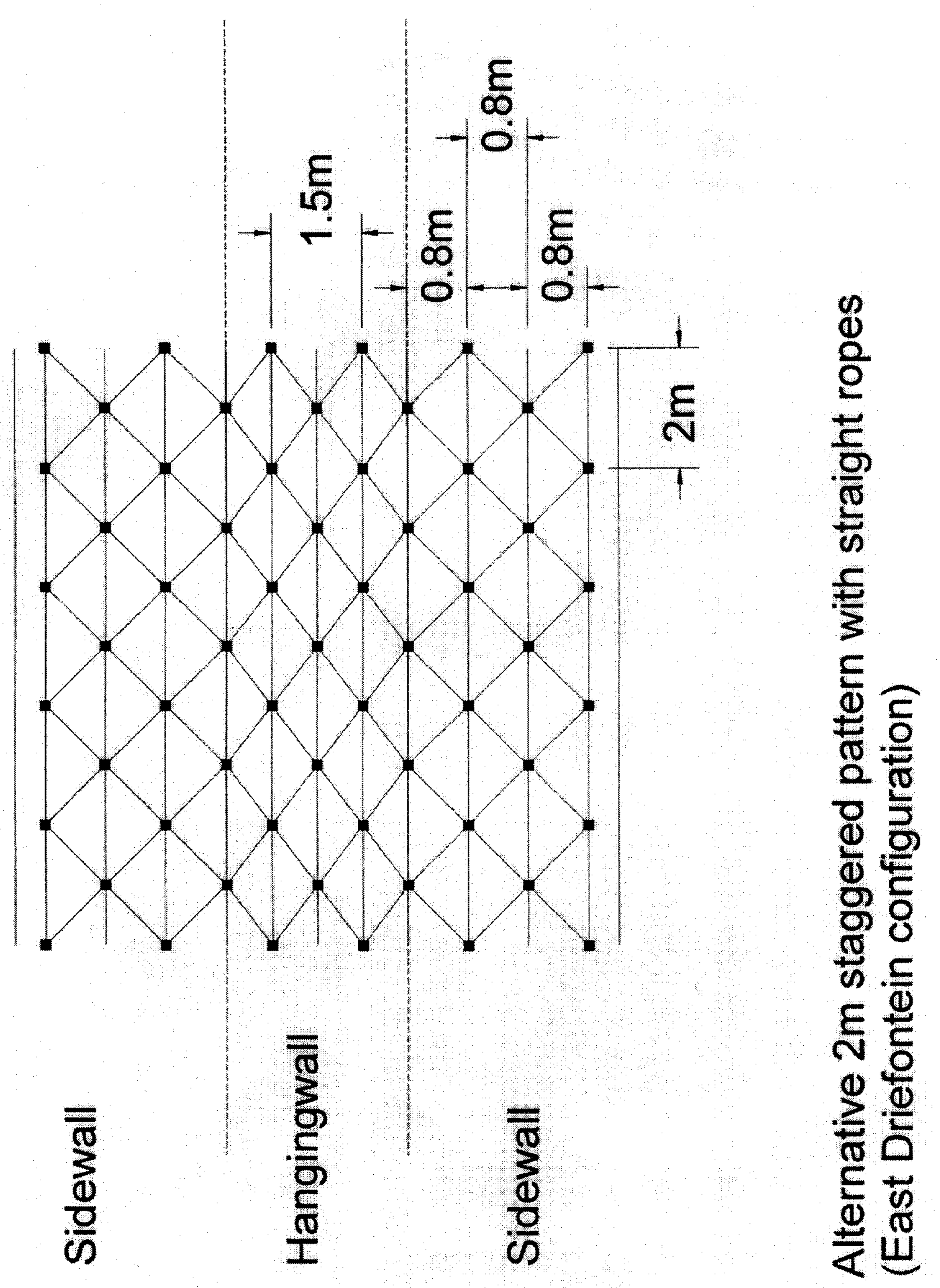


Figure 16c. Example of alternative 2m staggered mesh and lace pattern

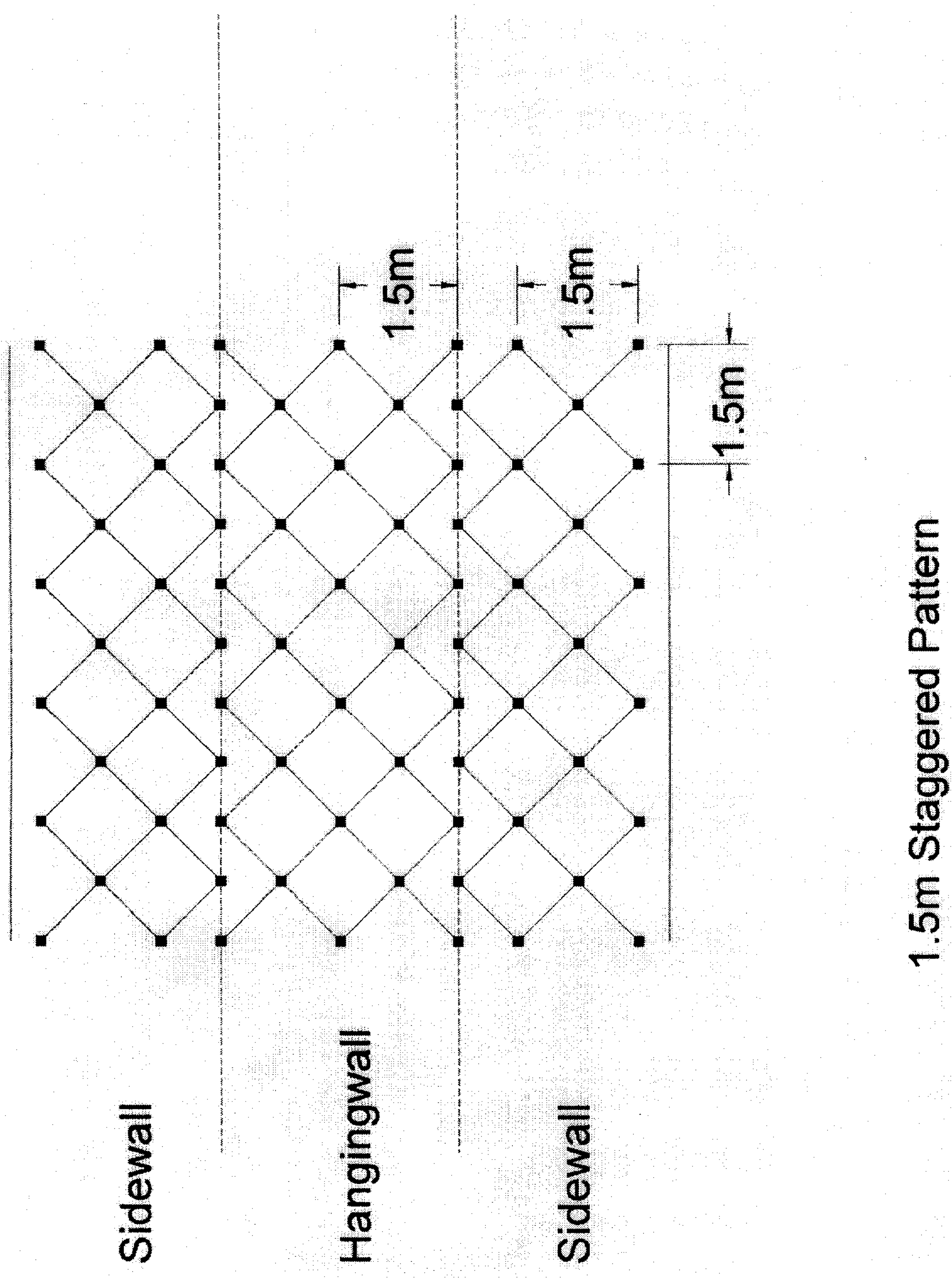


Figure 16d. Example of 1.5m staggered mesh and lace pattern

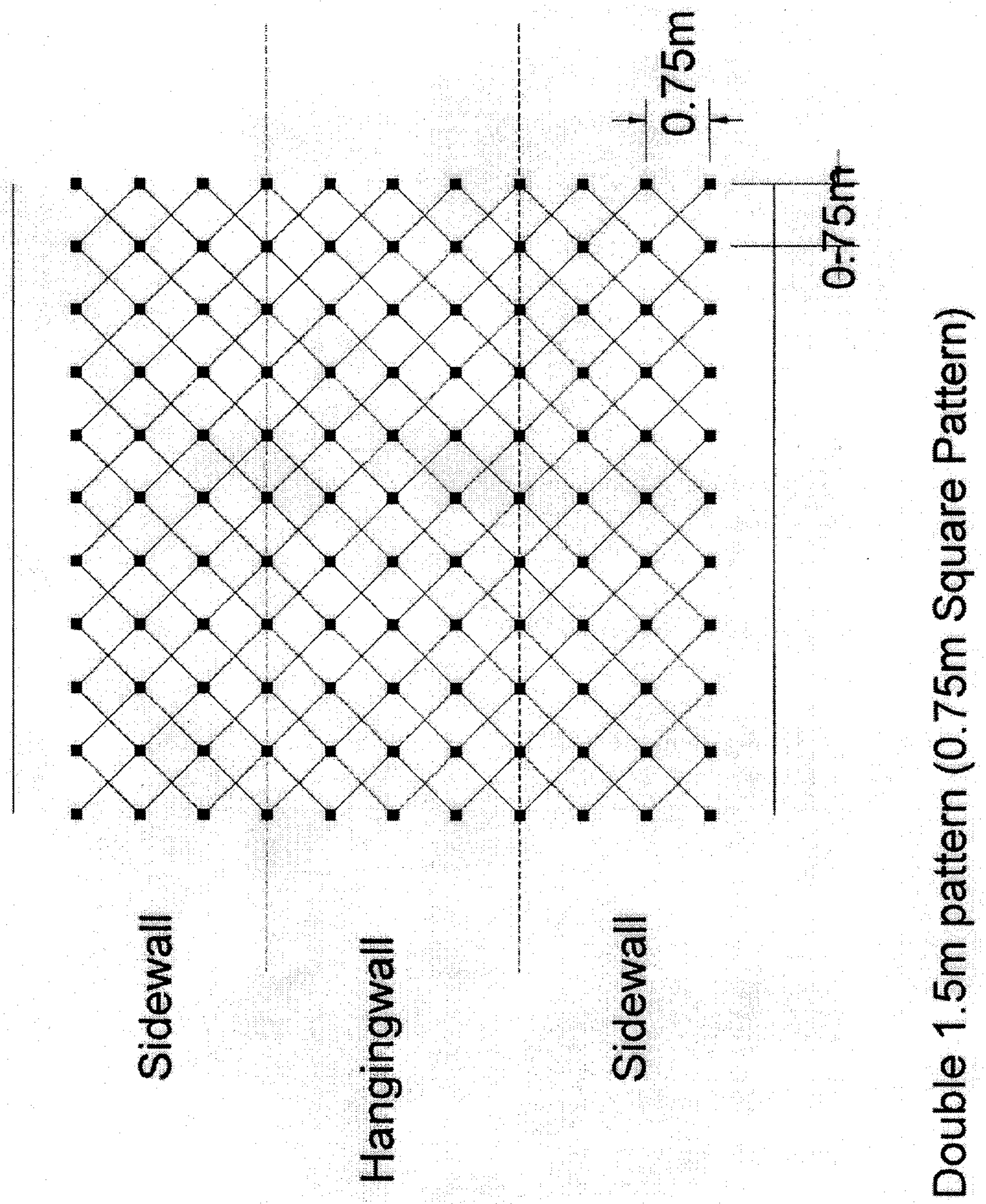


Figure 16e. Example of double 1.5m mesh and lace pattern

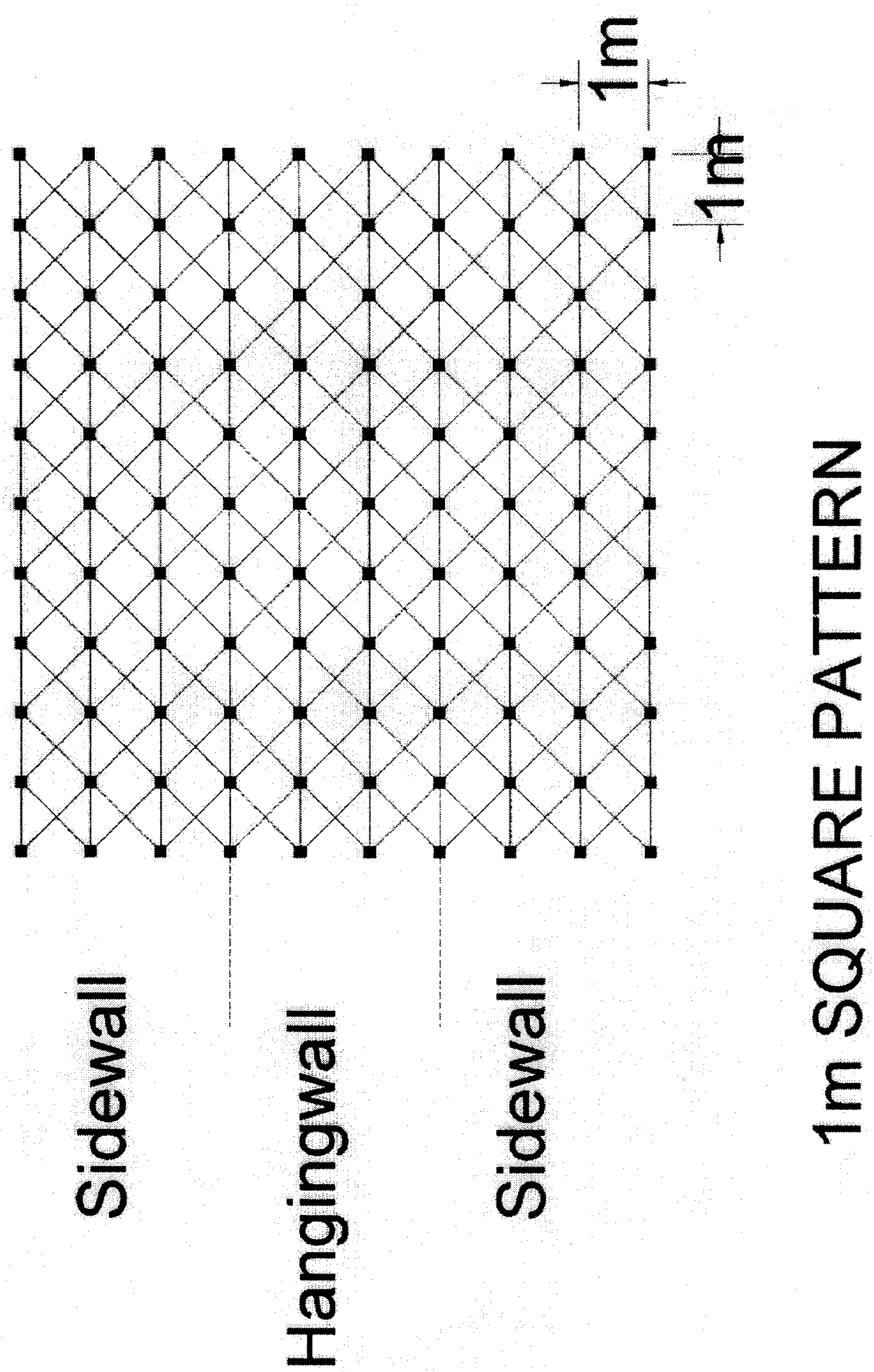


Figure 16f. Example of 1 m mesh and lace square pattern

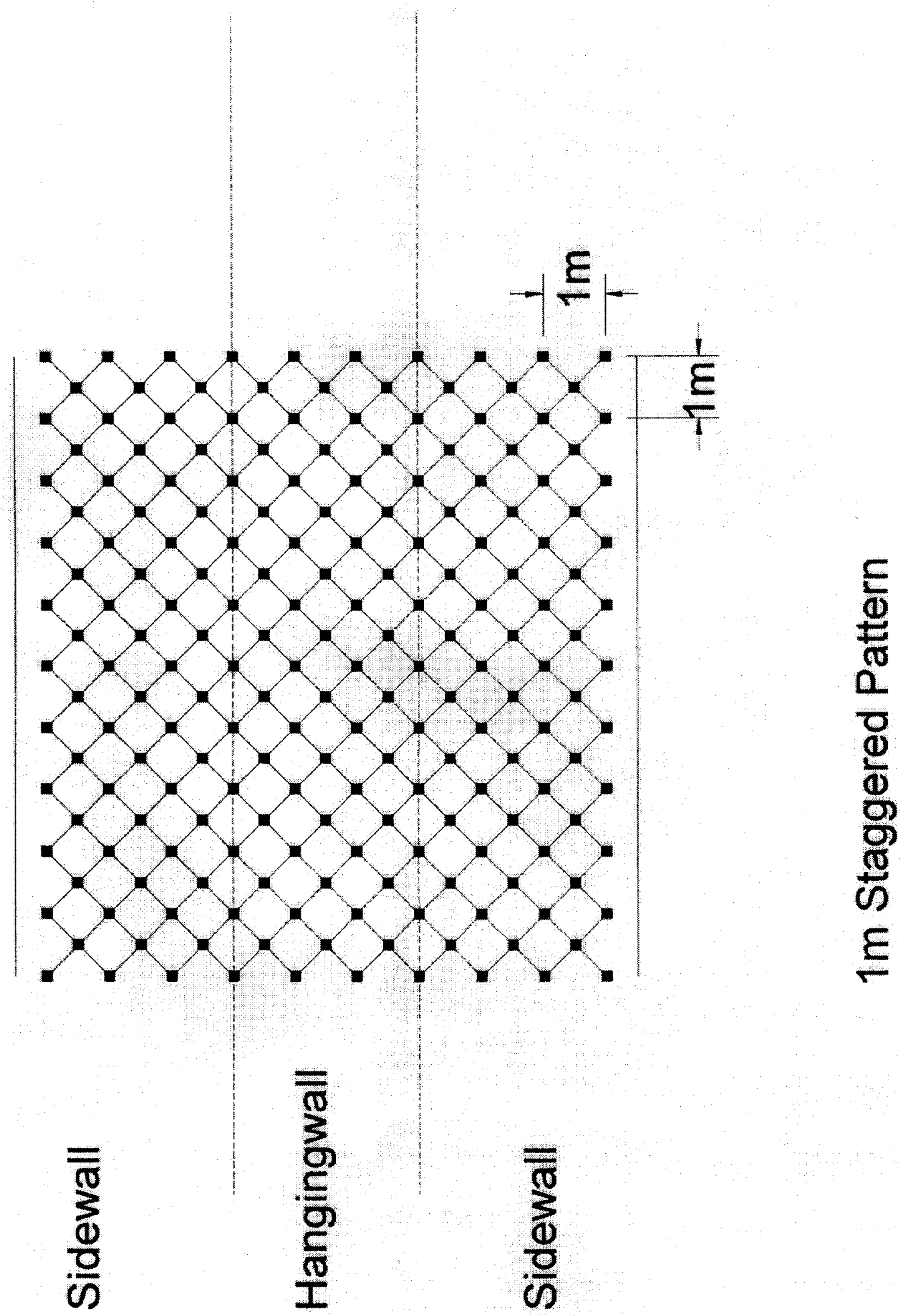


Figure 16g. Example of 1 m mesh and lace staggered pattern

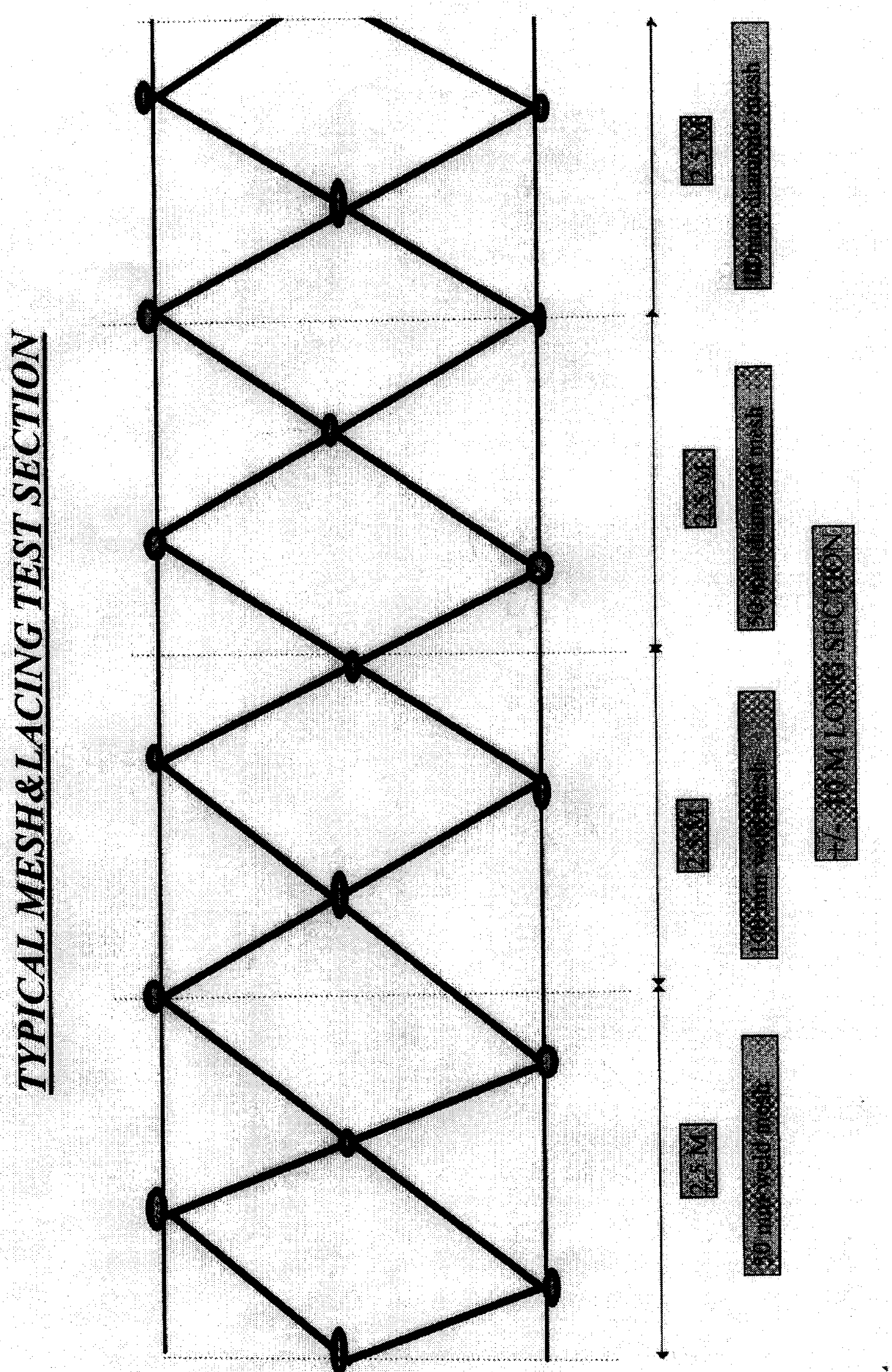


Figure 17. Mesh and lacing test sections

lengths and types of tendons were installed to form specific sections within a basic pattern of conventional length tendons. These sections were separated by sections supported by the standard support pattern.

Site Location

A site was selected in the 70 North Haulage 2, at 2100m below surface, in the 2K-area of number 2 Shaft, Vaal Reefs Exploration and Mining Company Limited. The site was at sufficient depth below surface to qualify as being under relatively high stress. It was underneath a block of ground which was to be stoped out in the course of the experiment. Seismic events occurred frequently in the area. The tunnel was aligned along the strike of the strata so that all the test sections were expected to experience stress changes of a similar magnitude and be located in the same geological formation. It was found after the support was installed, that a strong sill of UCS 340 MPa, was 1 m - 2 m in the hangingwall of the tunnel over the whole length of the section of tunnel where the experiment was carried out. The presence of the sill increased the potential for rockburst damage. The tunnel was located for most of its length in a strong glassy quartzite, UCS 260 MPa.

No other significant geological disturbances were present in the tunnel. Apart from one minor fault of near vertical inclination in Section 4 which was a control section. A basic 2m diamond pattern with 2,2 m long tendons plus mesh and lacing had been installed at the time the site was selected.

In summary, the site had most of the required characteristics to test different lengths and types of cable tendon support in comparable conditions. An element of potential excessive damage was present as in most deep, hard rock mining tunnels.

Support Installed

Secondary support ~ control sections in the test tunnel

The experimental site was supported by a 2m pattern of 2,2m long, 100 kN, wire rope tendons which have a loop folded back at one end and fixed with a ferrule. The looped ends were left protruding 0,10m from the borehole. These tendons were fully grouted with a sand and cement mix designed to achieve a uniaxial compressive strength (UCS) of at least 13 MPa. Woven, galvanised 4,5mm wire mesh with lOO mm apertures was placed over the protruding loops all along the sidewalls and hangingwall. Then wire rope lacing of minimum 6OkN tensile strength (UTS) was threaded horizontally and diagonally through all the loops and subsequently tensioned to 3OkN and clamped.

Tertiary support - experimental support sections

Tertiary support along the experimental sections of tunnel consisted of five different types and lengths of cable tendons installed in addition to the primary and secondary support as previously described. It was installed on a diamond pattern similar to the secondary support pattern and positioned midway between the secondary support units.

The tertiary support units were purposely <u>not</u> incorporated into the mesh and lacing so as to assess their independent contribution to the deformation behaviour in the tunnel.

The experimental tunnel was divided into ten sections. The five tendon types which were installed in alternative sections separated by unmodified secondary support sections were as follows:

- 2,2m 100 kN looped wire ropes (clean)
- 4,0m 160 kN greased, looped wire ropes
- 4,0m 380 kN straight cables (clean)
- 4,0m 380 kN mechanically anchored cables (clean), and
- 6,0m 160 kN greased, looped wire ropes.

Details of the tendons are given in Table 5 and of the support systems installed in each section in Table 6. The support density varied from 0,53 tendons / m^2 to 1,51, and the support resistance from 53 kN/m^2 to 246 kN/m^2.

Table 5 Description of support types in the experimental site

Type	Description (length; diameter; UTS; construction; material type)
Rockstud	1,5m; 14,5 mm; 100kN; round rod, rolled thread, three leaf bail type mechanical anchor, 100 mm square 4,5 mm thick bearing plate, domed washer and hexagonal nut; high tensile steel
Short, looped wire rope	2,2 m; 10 mm; 100 kN; round strand consisting of ten round 2 mm external, and nine 1 mm internal wires; high tensile steel
Long, looped wire rope	4 and 6 m; 16 mm; 160 kN; triangular strand consisting of twelve round 3 mm external and six 2,5 mm internal wires; high tensile steel
Long, straight wire rope	4 m; 18,5 mm; 380 kN; round strand consisting of seven 6 mm round wires; high tensile steel
Long mechanically anchored wire rope	4 m; 18,5 mm; 380 kN; round strand consisting of seven 6 mm round wires, three leaf remote release spring type anchor, 150 mm square 12, 5 mm thick bearing plate, domed washer and retaining wedge tensioning assembly, high tensile steel.

Table 6 Support type and density in the experimental site

Section; length; width; height; area	Support types	No of units per type	Tendons/ m² per type : support resistance (kN/m²)	Tendon density per square metre: total support resistance (kN/m²)
1; 7,9; 3,2; 3,1; 74,2	1,5 m rockstuds 2 m looped wire ropes double 2 m pattern	21 91	0,28 : 28,3 1,22 : 122,6	1,51 : 150,9
2; 7,6; 4,5; 3,6; 88,9	1,5 m rockstuds 2m looped wire ropes on a 2 m pattern	15 43	0,17 : 16,9 0,48 : 48,4	0,65 : 65,3
3; 7,5; 3,6; 3,6; 81,0	1,5 m rockstuds 2 m looped wire ropes on a 2 m pattern 4m greased, looped wire ropes on a 2 m pattern	16 46 45	0,20 : 19,7 0,57 : 56,7 0,56 : 111,1	1,32 : 187,5
4; 19,5; 3, 8; 195, 6	1,5 m rockstuds 2 m looped wire ropes on a 2 m pattern	58 93	0,30 : 29,7 0,48 : 47,5	0,77 : 77,2
5; 6,1; 3, 4; 4,0;69,5	1,5 m rockstuds 2m looped wire ropes on a 2m pattern 4m straight wire ropes on a 2 m pattern	24 36 35	0,34 : 34,5 0,52 : 51,8 0,50 : 191,4	1,36 : 277,7
6; 8,1; 3,2; 4,8; 103,6	1,5 m rockstuds 2 m looped wire ropes on a 2 m pattern	19 36	0,18 : 18,3 0,35 : 34,7	0,53 : 53,1
7; 6,1; 3,4; 4,4; 74,4	1,5 m rockstuds 2 m looped wire ropes on a 2 m pattern 4m mechanically anchored ropes on a 2m pattern	15 35 35	0,20 : 20:1 0,47 : 47,0 0,47 : 178,8	1,14 : 245,9
8; 9,0; 3,3; 4,3; 107,1	1,5 m rockstuds 2 m looped wire ropes on a 2 m pattern	16 48	0,15 : 14,9 0,45 : 44,8	0,60 : 59,7
9; 6,4; 3,5; 4,8; 83,8	1,5 m rockstuds 2m looped wire ropes on a 2m pattern 6m greased, looped wire ropes on 2m pattern	10 36 37	0,12 : 11,9 0,43 : 42,9 0,44 : 88,3	0,99 : 143,1
10; 7,0; 3,7; 4,5; 88,9	1,5 m rockstuds 2m looped wire ropes	15 37	0,17 : 16,9 0,42 : 41,6	0,58 : 58,5

Summary of research methodology used

Measurements of seismic induced ground velocities

The experimental site was situated within the coverage of the Klerksdorp Mine Managers Association Regional Seismic Network. The influence of seismic activity in the vicinity of the tunnel site was estimated from both calculated ground velocities (Peak Particle Velocities PPV) and from the results of a geophone at the site installed in the solid rock out of the influence of tunnel induced fracturing. The PPV data were used in conjunction with the extensometer and closure measurements. The maximum measured PPV was only 0,16 m/s. For the same event the McGarr equation relating PPV to magnitude and distance predicted a velocity of 0,53 m/s. This inconsistency between measured and predicted velocities occurred for all events and varied by a factor of 9 for very low velocities to a minimum of 3,4 for the above event.

Tunnel condition assessment

The Wiseman tunnel rating system was used to describe the geological and rock mechanics aspects and to evaluate the condition of the experimental tunnel. Ratings were carried out for each section both before and after mining induced stress changes affected the tunnel, to assess the influence of the various installed support systems on limiting the change in condition.

Deformation measurements

Tunnel deformation was measured by two complementary methods:- extensometers and closure meters. The maximum sidewall to sidewall closure measured was only 78mm and the maximum dilation in any one metre interval into the sidewall was only 45mm. Of the total 174 extensometer intervals monitored only 13 experienced dilation of more than 10mm and these usually in the first metre, the majority dilated by less than 3mm. With these limited deformations interpretation of results was difficult and the conclusions drawn tentative.

Computer modelling

The MINSIM-D program was used to model stresses and stress orientations for quarterly mining steps for each tunnel section monitored. The results obtained were subsequently used to compare with the measured tunnel deformation data. Increases in the maximum principle stress due to stope abutment stress concentrations ranged from 4 MPa to 15 MPa across the various test sections. This was followed by stress relaxation's of up to 24 MPa accompanied by a stress rotation of approximately 90° following overstoping.

The FLAC and UDEC codes were used to calculate the effect of mining induced stresses on rock deformation and support reaction for each experimental section. It was concluded that the UDEC code was the more appropriate for the type of modelling carried out and that both codes were sensitive to the value of input parameters used.

Conclusions and Recommendations from the Project on In-Situ Performance of Long Cable Tendons

The influence of five support systems, incorporating long cable tendons, on rock deformation and overall tunnel conditions has been monitored and the data analysed. From this analysis the following conclusions are drawn.

1) Initially intense fracturing in the sidewalls of the tunnel extended to 0,5m and thereafter with increased spacing to a maximum of 2,5m. Fracture apertures of 1mm or more occurred to a depth of 0,7m.

2) After overstoping, during which stress increases of 4 MPa to 15 MPa followed by stress relaxation's of 2 MPa to 24 MPa occurred accompanied by a stress rotation of 90°, the intense fracturing had increased in extent to 1,1m and fractures had opened to apertures of 2mm - 4mm.

3) The maximum sidewall to sidewall closure measured was only 78mm, hence the amount of dilation measured in each 1m interval to a depth of 6m was very small, particularly as most of the dilation occurred in the first metre.

4) The relatively small dilation's measured result in very small differences when comparing the influence of different support systems on deformation. This makes interpretation of the data and discrimination between systems rather tentative.

5) With few exceptions the amount of dilation per MPa stress change and rate of dilation with respect to the rate of stress change was greater during stress relaxation than when field stresses were increasing. Associated with the stress relaxation dilation could be significant debonding of cables (Kaiser 1992). This has not as yet been corroborated at the site. With different absolute stress values and magnitude of stress increase this observation may however be reversed. However regrading of the tracks became necessary because of footwall movement.

6) Shearing across fractures and along bedding planes was noted in 72% of the 29 boreholes observed. Shearing occurred in all the hangingwall observations. Shear resistance, or the ability to accommodate shear deformation without compromising support performance is therefore an important requirement of support tendons. However it should be noted that in

60% of the cases the amount of shear deformation was less than 10mm and was never observed to be greater than 30mm and that crack openings near the collar of the hole were 2mm to 4mm. Both these facts ease the achievement of shear resistance in tendons.

7) The combined effect of higher support density, support resistance and possibly tendon length in the test sections compared to the standard control sections, better restricted dilation in the 1m skin of the excavation. Support density tended to correlate better with sidewall deformation control than support resistance. As a result of the restricted skin movement, dilation was distributed deeper into the wall rock. The amount and position of this dilation appeared to be influenced by the length and bond characteristics of the tendon. For example higher bond strength resulted in dilation deeper in the sidewall while the greased tendons resulted in a more even distribution of the dilation.

8) Greater restriction in sidewall deformation induced larger deformation of the hangingwall where dilation was limited to the first 2m. The test section support reduced closure relative to the control sections by about 30%.

9) The limit of dilation in the hangingwall to the first 2m, although possibly influenced by the presence of the sill, agrees with the findings of Hepworth at Buffelsfontein Gold Mine. The inference is that under quasi-static loading at least, and for 3m wide tunnels, 4m or greater length tendons are superfluous in the hangingwall.

10) The sections where long tendons were installed had less deformation and better overall conditions than the sections with the same support density but with only 2m long tendons.

11) The damage to the tunnel consisted of relatively thin scaling and slabbing with little fallout. The Wiseman condition ratings for the down dip wall of the

tunnel were generally worse than those for the up dip wall which is in accordance with dilation measurements. This has been noted before but no explanation for this phenomenon has been established. However inelastic modelling also produced larger deformations in the down dip sidewall.

12) Both the FLAC and UDEC codes were used in attempts to back analyse the influence of support on tunnel deformation as monitored. It was found that the FLAC code was more sensitive to changes in support systems but that the UDEC code better simulated the measured deformations including the shearing. Overall the UDEC code is preferred for this type of modelling in fractured, bedded rock. However much further work is required in calibration of the model and in sensitivity studies of input parameters before the code can be considered as part of a tunnel and support design procedure.

13) On average the test sections deteriorated according to Wiseman ratings marginally more than the control sections. This was partly attributed to damage caused during installation of the tertiary support.

14) The low PPV's recorded at the site were insufficient to test the effectiveness of the various support systems being evaluated in controlling damage due to dynamic loading.

15) The monitored behaviour of the control sections, in ostensibly the same ground conditions, showed how variable rock mass behaviour can be. From this it is concluded that a large data base relating geotechnical parameters, stress changes and support to rock deformation should be established. It is envisaged that this could be achieved relatively inexpensively by the mines, co-ordinated by researchers, setting up an extensive but unsophisticated monitoring programme. This knowledge would facilitate the development and evaluation of a rational design procedure.

6. Proposed tunnel design methodology for highly discontinuous rock mass conditions

A conceptual tunnel and tunnel support design methodology has been formulated. The strategy for consequent detailed research in the continuation project will be to carry out the necessary work to validate and where appropriate modify the methodology taking into account the problematic ground conditions such as highly jointed or high stress rock mass conditions and to provide the required input parameters for the various steps in the design process.

The basis of the proposed tunnel design methodology has been described in the SIMRAC interim reports by Haile (1994) 'Analysis of previous investigations into brittle rock fracture and tunnel stability in highly stressed rock and proposed analysis of fracture development around tunnel excavations' and Haile et al.(1995) ' A proposed methodology for the research of tunnel stability and the development of excavation design criteria in a highly stressed environment'. A summary of this work will be covered here, for more detailed information reference should be made to the published SIMRAC interim reports.

Analysis of the brittle fracture of rock has been based on a review of the work conducted by other authors in this field of study. Of particular interest to this investigation was the analysis of the development of the fracture zone around an excavation. The failure of brittle rock is primarily due to the initial development of extension fractures parallel to the direction of the maximum applied stress and perpendicular to the minimum stress. These fractures may coalesce to form the characteristic shear failure plane which generally represents the ultimate failure of the material. This phenomenon may however be a function of the material modulus and triaxial stress state. The intensity of fracturing within a rock is considered to be a function of the mineralogy and grain size of the material, and would thus influence the degree of dilation of the rock mass into an excavation. Work previously conducted to investigate the extent of fracturing around tunnels within the South African mining environment has concentrated primarily on the analysis of the change in fracturing and displacements around the excavation brought about by subsequent increases in the stress field.

It is thus proposed within the scope of the continuation project to conduct detailed analysis of the fracture zone around an excavation with regard to the intensity, orientation of fracturing within different rock mass, stress and excavation environments in order to determine the strength and deformation modulus of the fractured rock and hence support requirements. The determination of the structure and mechanics of the fracture zone around an excavation will provide information and understanding which will allow the design of an appropriate support system on an engineering basis, to ensure the stability of the excavation in the defined rock mass environment.

The concept of a support design methodology for a tunnel excavation for a fractured or highly discontinuous rock mass was thus subsequently examined. The basis of the methodology was to address design questions which confront rock mechanic engineers in typical deep level mining environments (or discontinuous problematic ground conditions), and the design concept to be based on engineering design principles rather than empirical support selection tables.

Thus the conceptual design process would involve the following design steps:

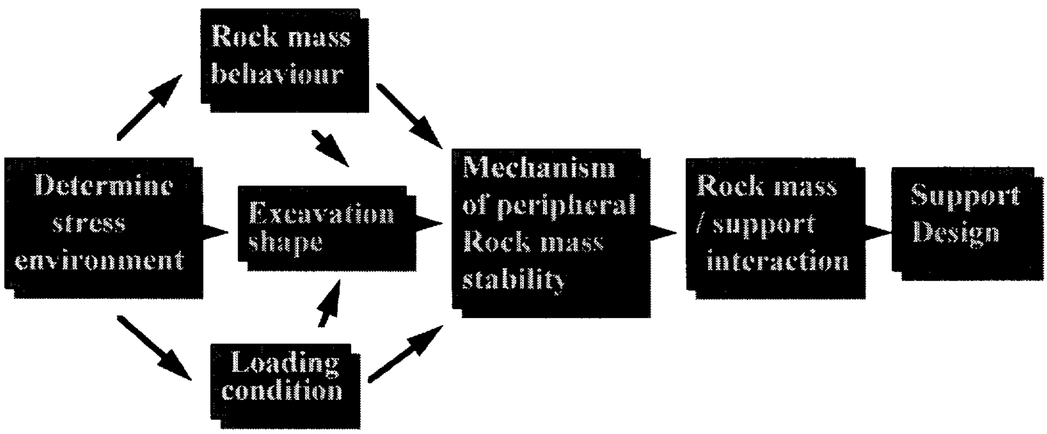
1) Determination of the design rock mass environment, whether the rock mass will be controlled by geotechnical structure in a low stress environment or stress controlled failure in a high stress environment.

2) Within a high stress environment to define the nature and extent of the fracture zone which will govern the structure of the fracture zone and dilation

of the rock mass into the excavation. In addition the influence of excavation shape on these parameters would be a further consideration.

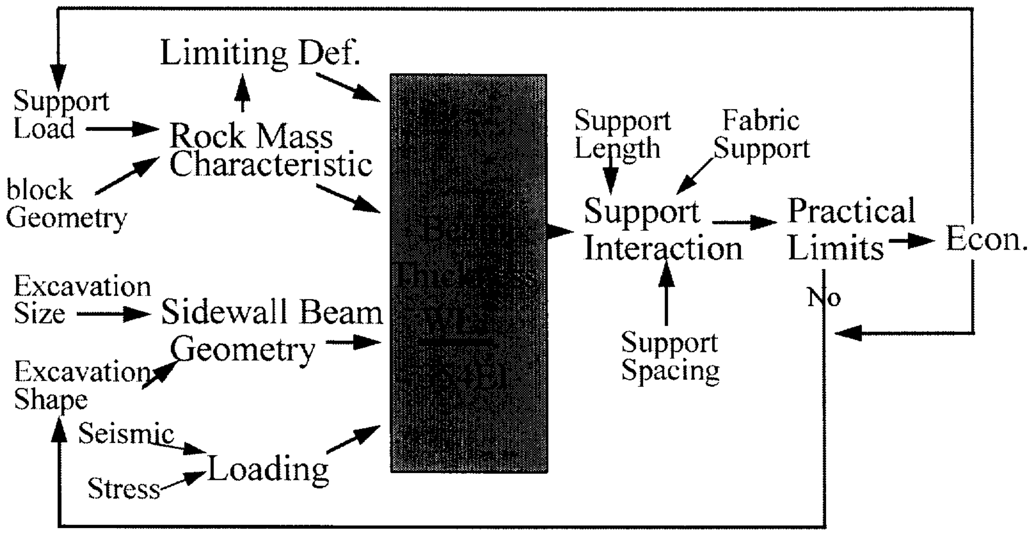
3) Criteria for the selection of support unit length, spacing, load and deformation requirements based on the anticipated rock mass structure (fractured / discontinuous) and deformation loading conditions (quasi-static or dynamic).

The following figure illustrates the overall envisaged design methodology



The basis of the design concept is the creation of a stable reinforced rock mass shell of appropriate thickness around the excavation by ensuring effective interaction of the support units. The rock mass shell is analysed as a series of hangingwall and sidewall beams or plates. The shell or beam concept has been proposed previously by numerous authors and design concepts such as Jaeger and Cook, Lang and NATM to ensure support interaction. Under the proposed design concept the required beam thickness is a function of the quality of the rock mass and the loading conditions under quasi static and dynamic conditions. The competency of the rock mass beam to control displacement is considered to be a function of the rock mass properties as defined by the rock mass type, structure and degree of confinement, and the beam shape as defined by the excavation profile. The design of the support system to create the required beam thickness is based on the support unit interaction. Current work is demonstrating that this interaction is a function of the support unit length, spacing and load, and the structure of the excavation peripheral rock mass. Thus for a given rock mass environment the support system may be designed on an engineering basis.

The following flow sheet indicates the design considerations and conceptual design charts for the analysis of sidewall beam stability.



Where $WL^3/354EI$ represents the engineering relationship for the analysis of the stability of a rectangular beam where W = loading, L = beam length, E = effective beam modulus and I = moment of inertia defining the beam geometry.

The principle of the design methodology is to establish the rock mass environment based on the rock mass structure and confinement as provided by the support system. This is considered to govern the rock mass characteristics, such as the effective modulus and the limiting deformation that the rock mass can sustain prior to unravelling, or critical strain. The excavation size and shape will govern the geometry of the beam with regard to its length and shape. The loading of the beam will be a function of gravity, stress or dilational loading, or seismic loading. These input parameters would be used to determine the required thickness of the rock mass shell to ensure excavation stability. The required thickness of reinforced rock would thus be a function of the interaction of the support units with the rock mass structure and thus the design of the support system. It is envisaged that this design methodology will give a more engineering emphasis to the design of excavation support systems.

The conceptual design methodology has been used to form the basis of the future research strategy. The formulation of this conceptual design methodology allows for the creation of a structured research strategy that will focus the research areas to address the design problems that face the industry in deep level and problematic mining environments.

7. Conclusions and recommendations of further studies

As a result of research activities that were carried out as the SIMRAC project GAP026, for the period from June 1993 to November 1995, the following important areas were investigated with consideration for implementation and areas of further research requirements:

- A foundation has been laid for the formulation of tunnel support requirements under a variety of geological and mining conditions in South African mines. In collaboration with the mine rock mechanics practitioners data obtained are to be used in development of a practical tunnel support design methodology

- The assessment of the cost of inadequate tunnel support, detailing areas in South African tunnelling where savings are possible, was produced.

- Eight groups of problems that are mainly responsible for the more serious damage to tunnels were identified and can be used by the mines to better plan any practical steps either to eliminate or at least reduce the severity of those problems.

- An analysis of the accident statistics on the mines produced quantitative and qualitative data that identified critical areas with regard to safety in tunnel excavations. 26 % of the FOG related fatal accidents in the Bushveld Complex take place in tunnels. A significant improvement of the areal coverage of the support used within the first 10 metres from the tunnel development end is needed in tunnels. Along that distance over 70 % of the FOG related fatalities are reported. In about 80 % of the cases the support capacity required to prevent the falls of ground was < 100 kN, and in 88 % of the reported fatalities the required length of the tendon support was < 1,5 m.

-The SAFEX computer code was identified as a suitable tool to perform tunnel stability analyses in a complex, geology controlled rockmass for use by rock mechanics practitioners. SAFEX software could be used to perform preliminary stability assessment and analysis of support requirements for Bushveld Complex tunnels. Since the influence of stress changes is not taken into account, the program is applicable mainly in lower stress environments where stability of tunnels is controlled by the geological discontinuities in the surrounding rock under the action of gravity loading and not by the stress field.

- Comparative analysis of the FLAC and UDEC codes was performed with an objective to establish which of the two inelastic codes is more appropriate for modelling tunnel behaviour in hard rock pervaded by fractures and bedding planes. This aspect of the project is evaluated on an on going basis as the computer codes are used for different aspects of analysis. UDEC inelastic code was found to simulate the discontinuous behaviour of the experimental tunnel in hard rock, pervaded by fractures and bedding planes, better than FLAC. It is therefore recommended that until a final tunnel design methodology is developed the UDEC code is used to obtain some insight into tunnel and support design. Although inelastic modelling methods were found to be potentially very useful, they need to be calibrated against field data on rockmass and support behaviour from test sites, case studies in the literature, and additional focused experiments should be monitored to obtain data on specific parameters identified by modelling to be inadequate.

- Usability of the Rockwall Condition Factor (RCF) was evaluated as a design criterion for the Bushveld Complex tunnels. It was concluded that the application of the RCF criterion within the Bushveld Complex was problematic due to the predominant influence of geotechnical structures on excavation stability, and the high variability of the rock mass environment to allow the application of global design criteria.

- A report on current tunnel support practices in gold and platinum mines was written. It contains information on various support elements and systems including their construction, mechanical behaviour and interaction with the surrounding rock. A discussion of the most important support characteristics of various systems in use is also included in the report.

- Two types of shear apparatus (single and double) were developed for laboratory testing of various rock tendons. Subsequently, a number of comparative shear tests on the most common support tendons have been carried out and performance characteristics under shear loading, both slow and rapid, were established and can be used by the mines in the selection of tendon support. In areas where shear deformations of the surrounding rock are expected either fully grouted rope support or yieldable cone bolts should be used. These two types of tendons can survive markedly larger shear deformations than conventional grouted tendons as illustrated in this report. The in situ monitoring of long cables indicated qualitatively the importance of consideration of shear deformation for support design under quasi-static conditions. Observations of rockburst damage within tunnels has also indicated shear failure of support elements.

- Preliminary tests conducted on polypropylene mesh indicate that their relatively low load carrying capacity (average 7,3 kN) as compared to conventional diamond wire mesh (>30 kN) , together with the "abrupt" mode of failure after reaching their peak strength could significantly limit their applicability as a large areal coverage tunnel support. Another important consideration in deciding on possible applications for polypropylene mesh should be an inherent vulnerability of that polymer material to the very high temperatures that can occur during underground fires.

- An apparatus to quantify field performance of mesh and lacing systems was developed and successfully tested underground. It can be used by the mines to establish force-deformation characteristics of any mine specific tendon/mesh/ lacing combinations, and hence in the design of support systems. The results of such testing can be used as input parameters into computer codes used to evaluate support systems.

- An apparatus to establish in-situ performance of various types of shotcrete was developed and is being tested underground. Field data obtained will be distributed through SIMRAC and could be used in the design of large areal coverage systems as well as in a calibration of the computer codes that are used to assess the effect of various total area support systems.

- An increase in the density of support units was found to correlate better with the reduction in sidewall dilation than an increase in support unit length or support resistance for the analysed site at Vaal Reefs.

- A conceptual design methodology has been developed for tunnel excavations that will form the basis of the future research strategy.

Recommended areas of future research include:

- Rockburst mechanisms in tunnels and how these influence support requirements need to be defined and explained. Non-linear dynamic modelling should be conducted together with analysing all the available field data on rockburst damage. Recently developed dynamic numerical codes will require calibration against peak ground velocities in tunnels, and analysis of rockbursts in tunnels.

- It is recommended that further research is carried out to obtain better understanding of the interaction between support elements and the rock. In addition, more data are needed on the influence of tunnel shape on stress fracturing, deformation and overall tunnel stability under high stress and rockburst conditions

- Better tunnel supports for extremely high stress and severe rockburst conditions as well as for low cohesion environments and strain bursting conditions have to be developed and tested

- Conduct further testing of support units and systems in the laboratory under simulated underground loading conditions and at underground test sites in situ.

These needs will be incorporated in the new research strategy under the conceptual design methodology as proposed previously.

8. SIMRAC Interim Report References

Wojno, L., Jager, A., "An overview of tunnel stability and support problems in South African mines" Miningtek Report GAP026, June 1994

Haile, A.T., "Literature review and proposed analysis of fracture development around tunnel excavations" SIMRAC Report GAP026, November 1994

Wojno, L., Jager A.J., "Identification of probable keyblock shapes and sizes for various known joint set and bedding combinations for drives and cross-cuts in Bushveld Complex mines", SIMRAC Report GAP026, December 1994

Haile, A.T., "An evaluation of the UDEC numerical code for the modelling of fracture and intact material behaviour" SIMRAC Report GAP024 / GAP026, December 1994

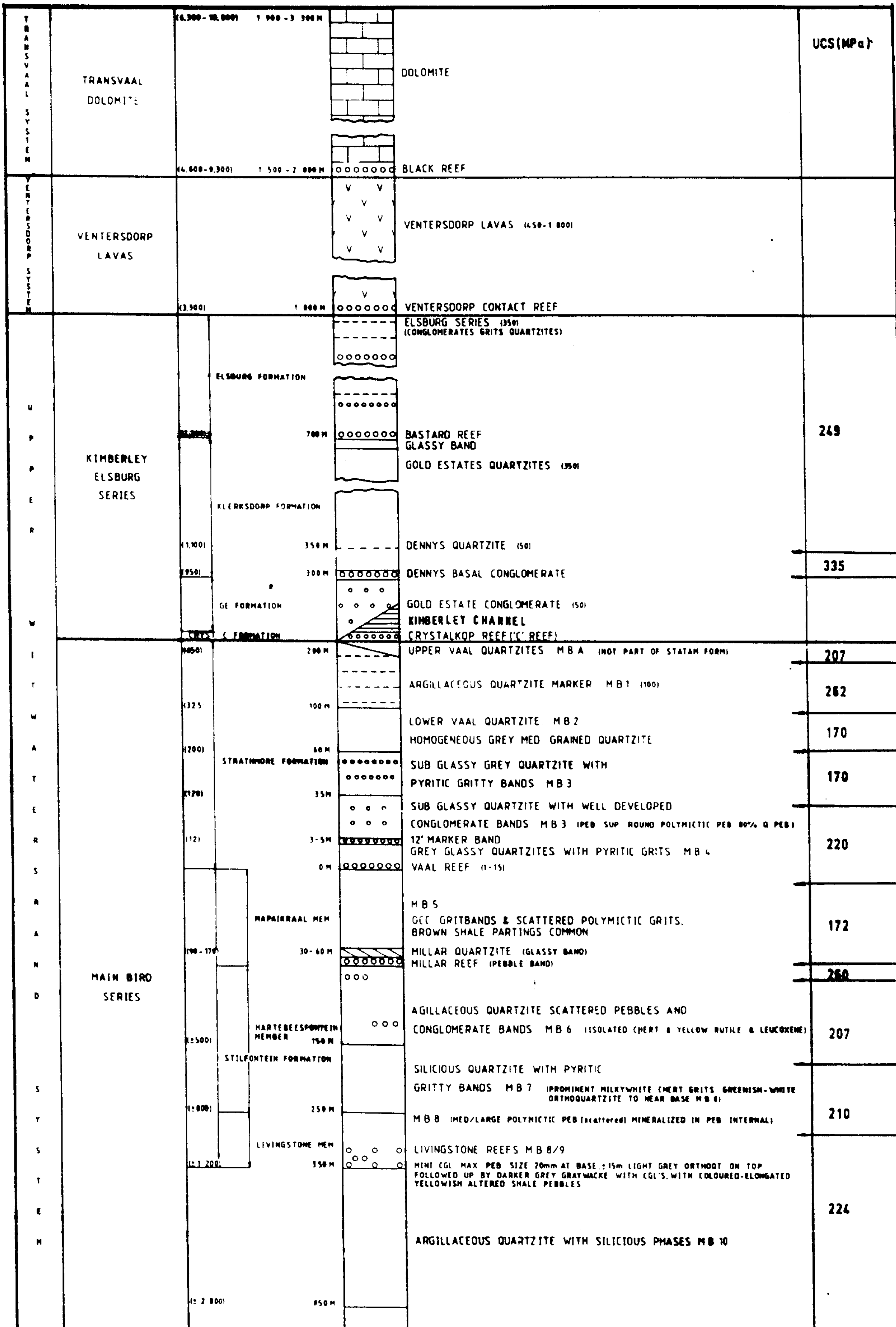
Haile, AT, Jager, A, Wojno, L, "A Proposed Methodology for the Research of Tunnel Stability and the Development of Excavation Design Criteria in a Highly Stressed Environment" SIMRAC Report GAP026, May 1995

Roberts, D.P., "Testing of mining tunnel support elements and systems for hard rock mines" MSc Thesis University of Natal, 1995.

Appendix A

Vaal Reefs long tendon evaluation site data

FIG: 22 IDEALISED GEOLOGICAL SUCCESSION FOR VAAL REEFS EXP. & MIN. CO.



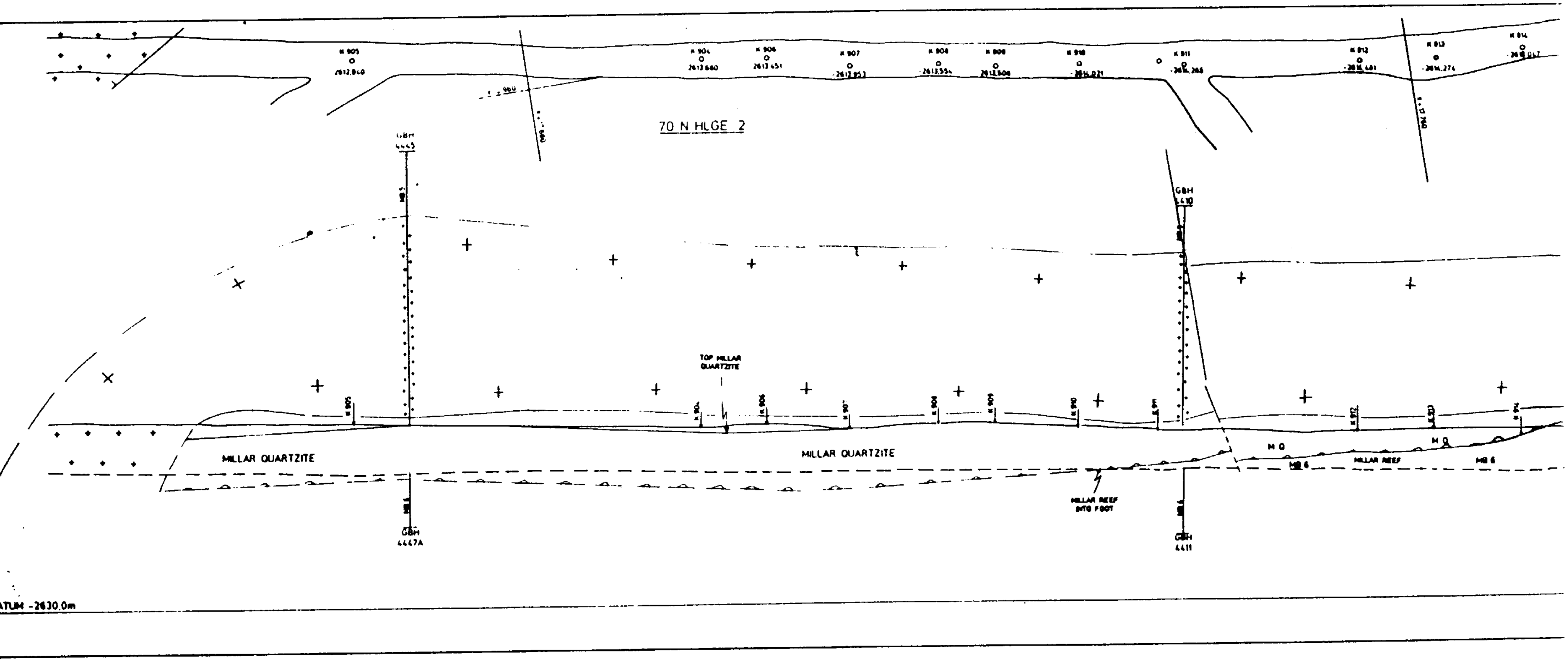
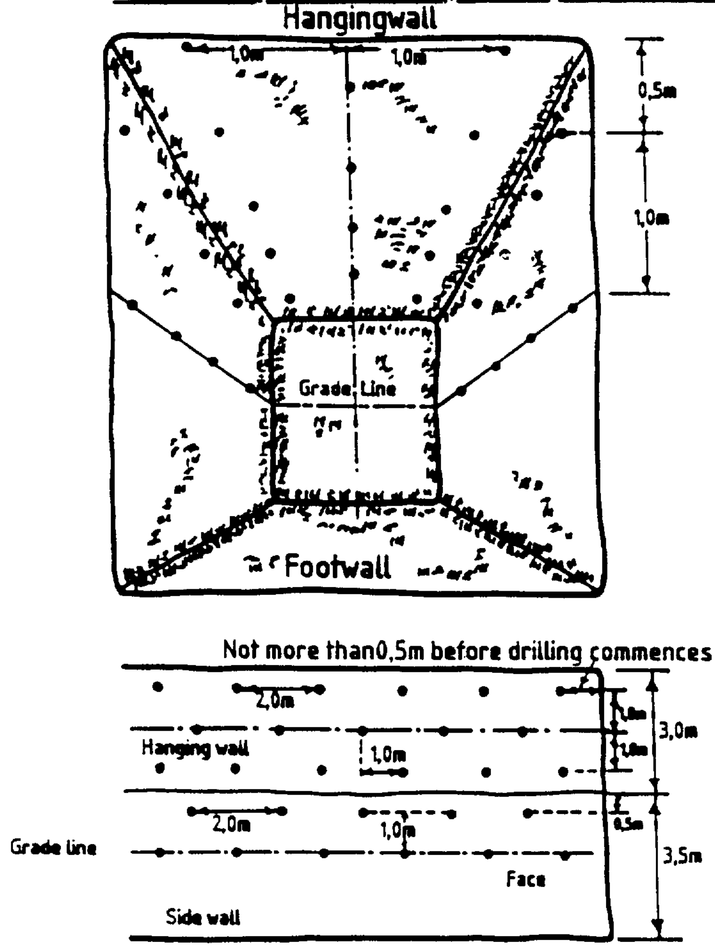


Figure 23: Plan and Section showing Geology of the Project Site.

Figure 28a: Roofbolt pattern for 3,5m x 3,0m Tunnels



Plan View

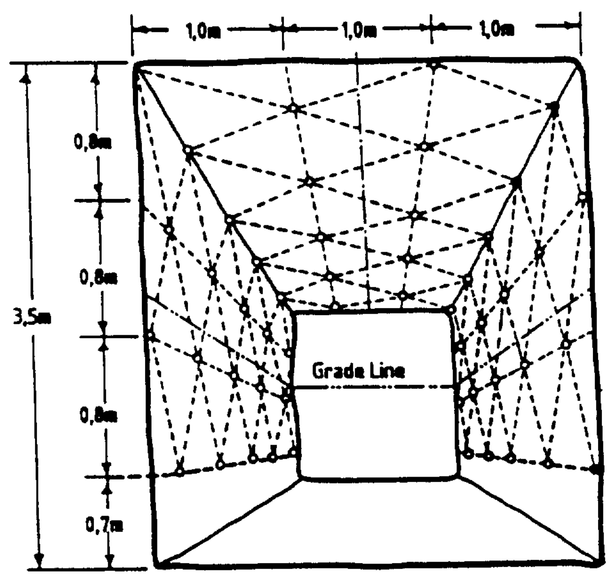
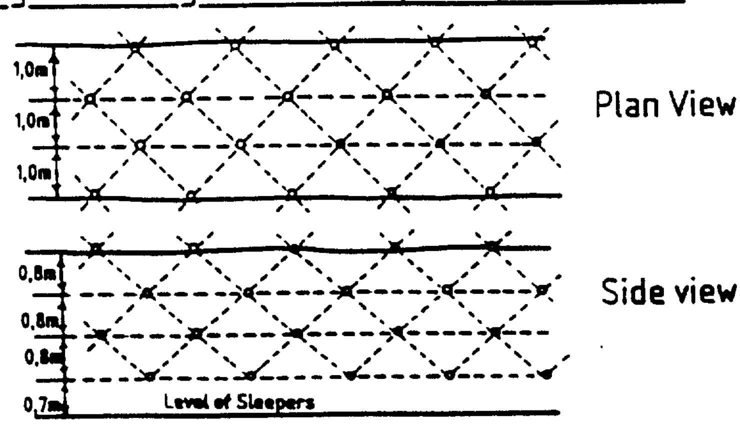


Figure 28b: Meshing and Lacing Pattern for 3,0m x3,5m Tunnels



Legend:

Sections 2,4,6,8,10 – Standard
Section 1 – Standard plus 2m 100kN clean Tendons
3 – 4m 200kN Greased Tendons
5 – 4m 380kN Straight clean Tendons
7 – 4m 380kN Mechanical clean Tendons
9 – 6m 200kN Greased Tendons

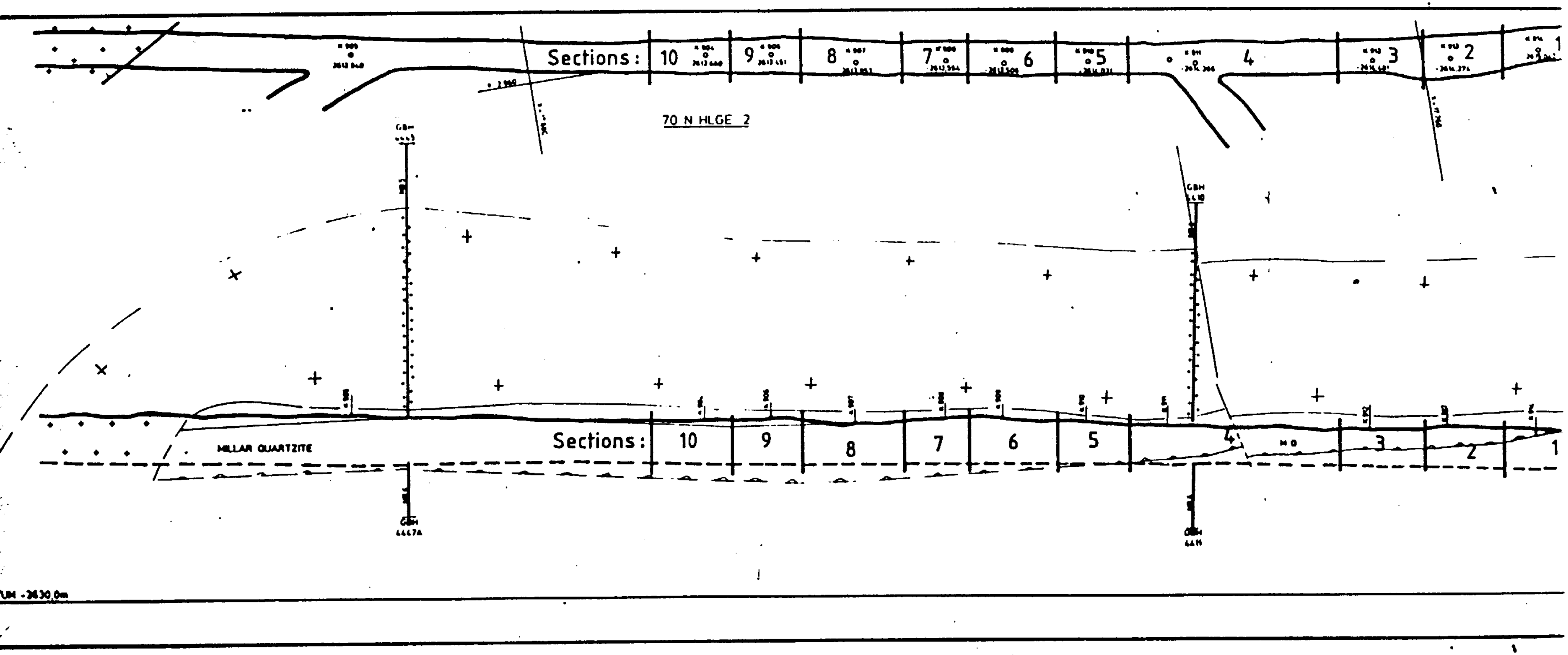
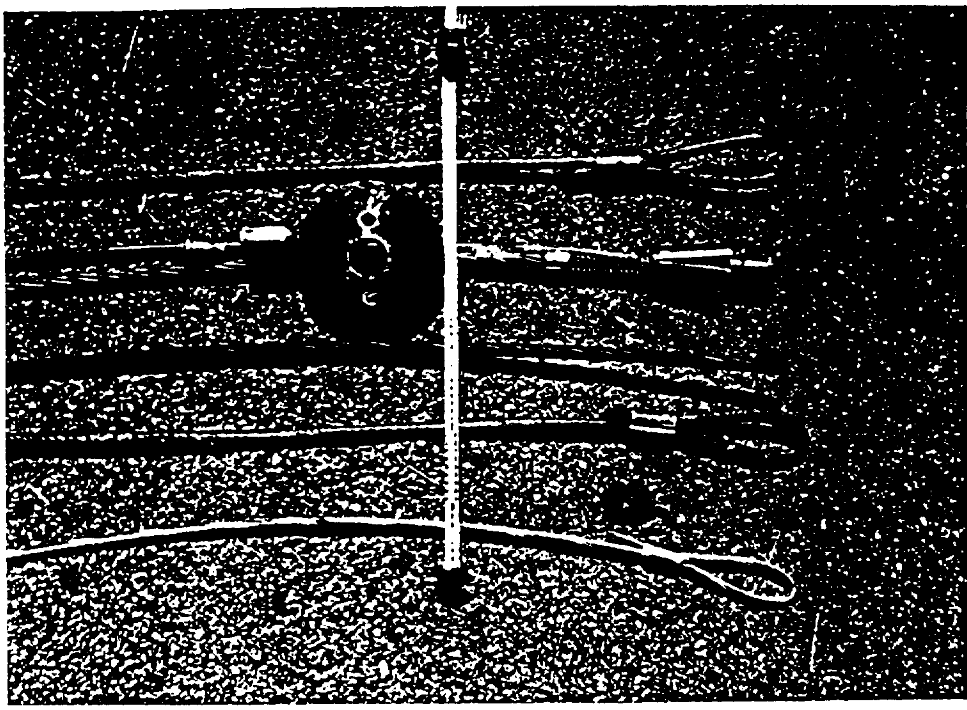


Figure 29 : Positions of the Monitored Tunnel Sections in the Project Site.

Figure 30 : Support Tendons similar to those used in the Project Tunnel.



a. 200 kN wire rope loop
b. 380 kN Mechanical anchors
c. 380 kN Straight tendon
d. 200 kN Wire rope loop
e. 100 kN Wire rope loop

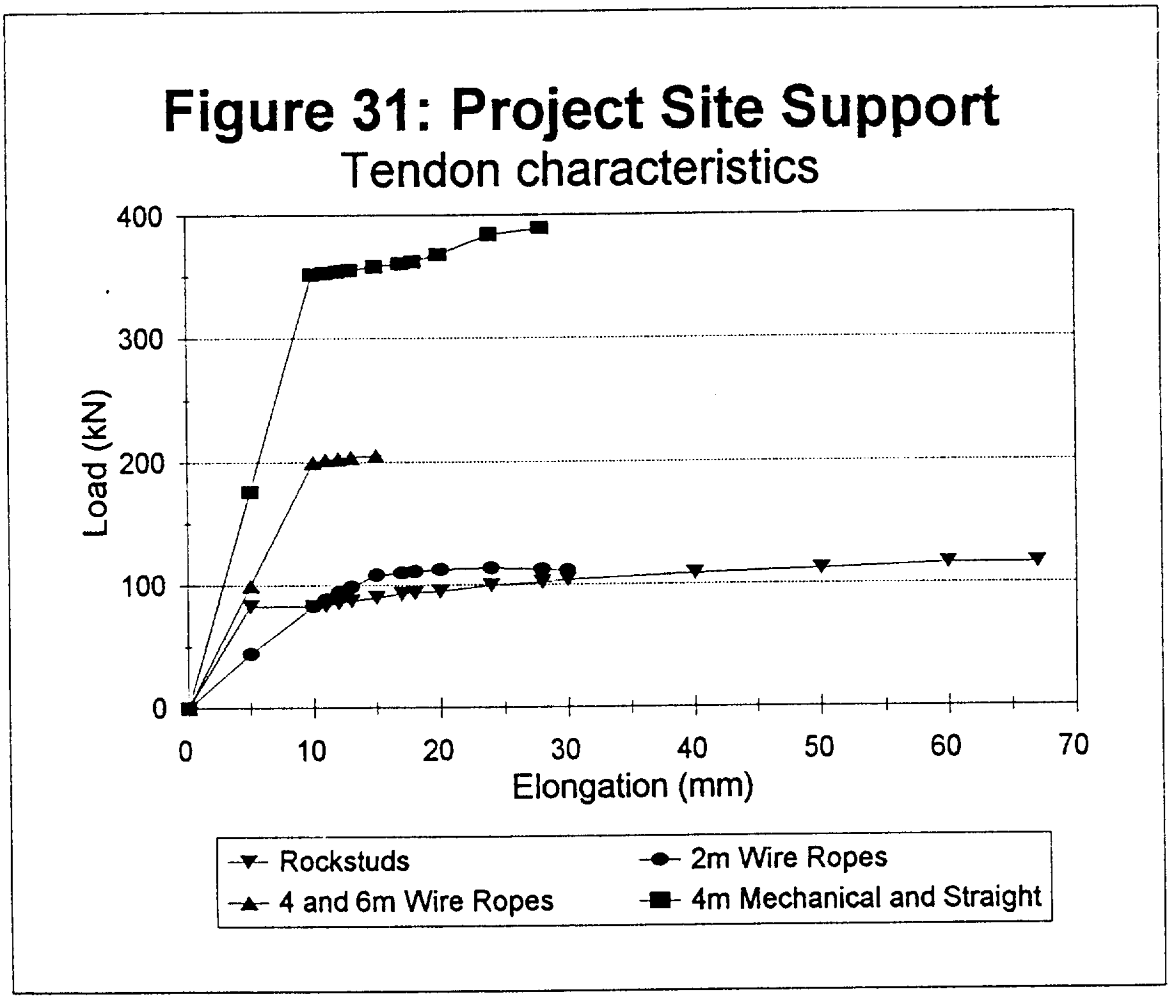


Figure 31: Project Site Support
Tendon characteristics
Load (kN)
400
300
200
100
0
0
10
20
30
40
50
60
70
Elongation (mm)
Rockstuds
2m Wire Ropes
4 and 6m Wire Ropes
4m Mechanical and Straight

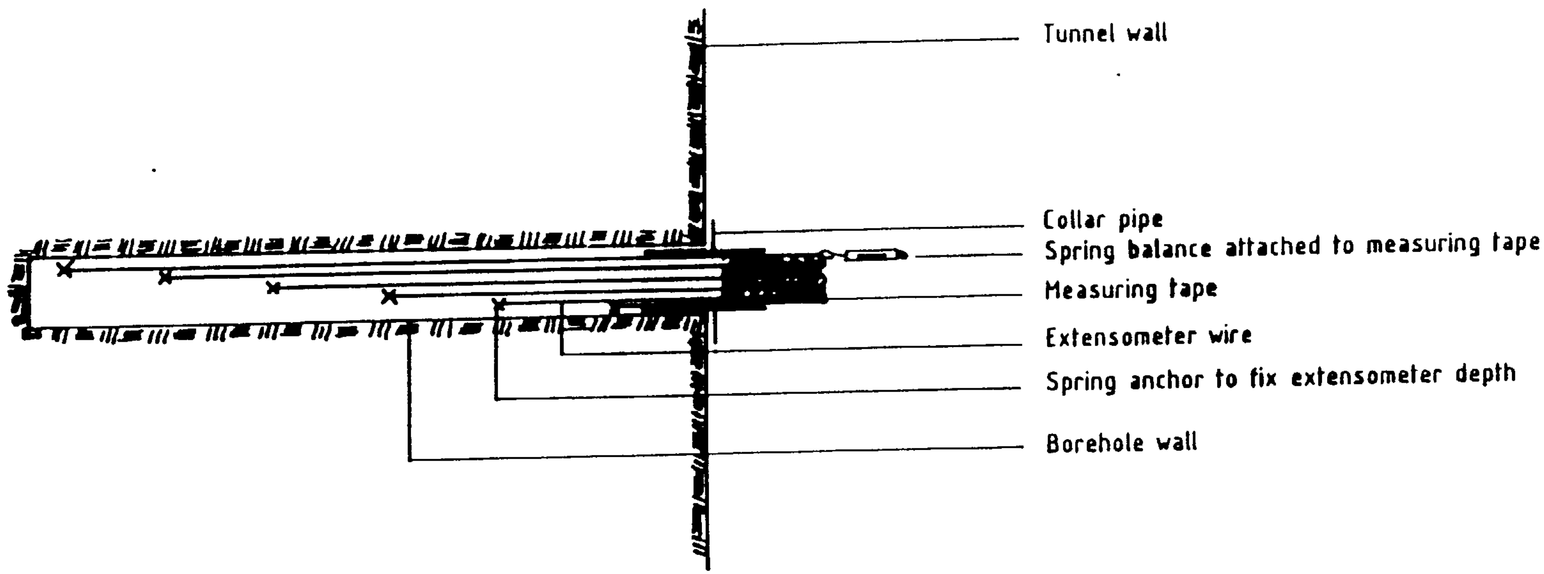


Figure 32: Sketch of the Extensometer method used in the experiment.

Figure 33: Section 1 stress history and deformation measurements

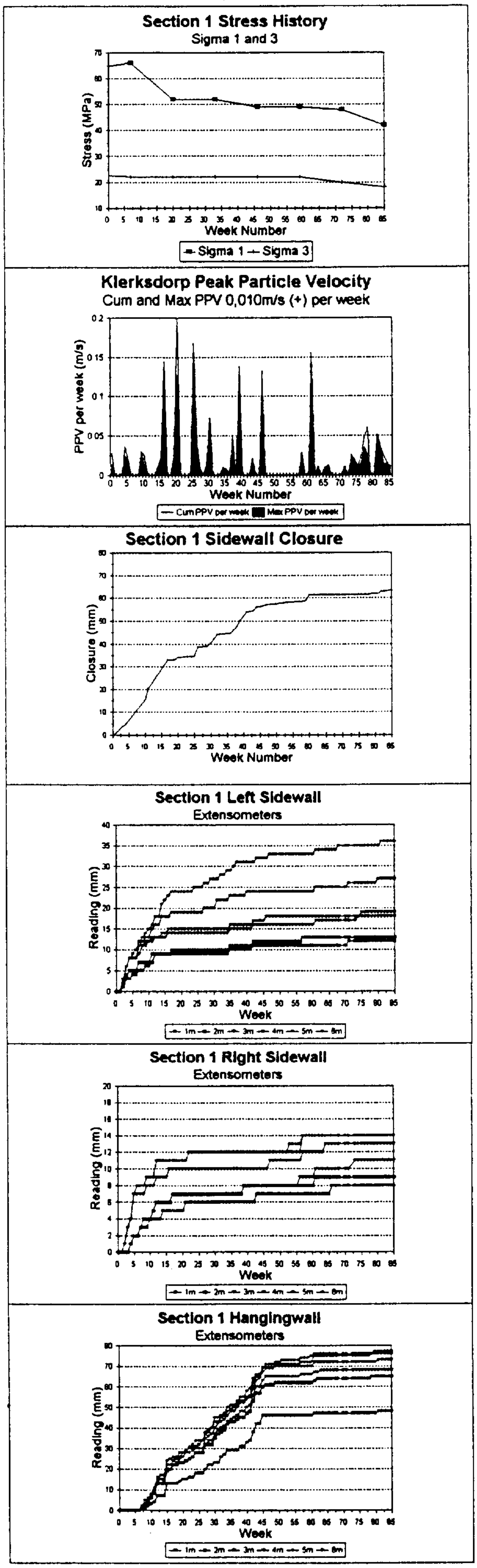


Figure 34: Section 2 stress history and deformation measurements

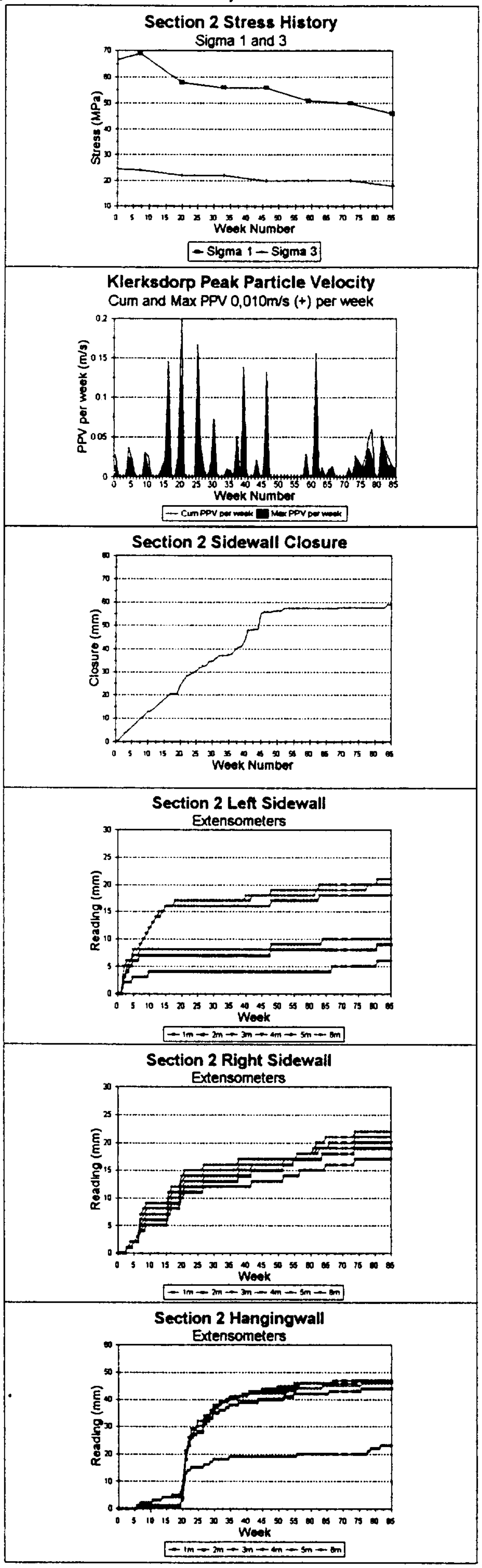


Figure 35: Section 3 stress history and deformation measurements

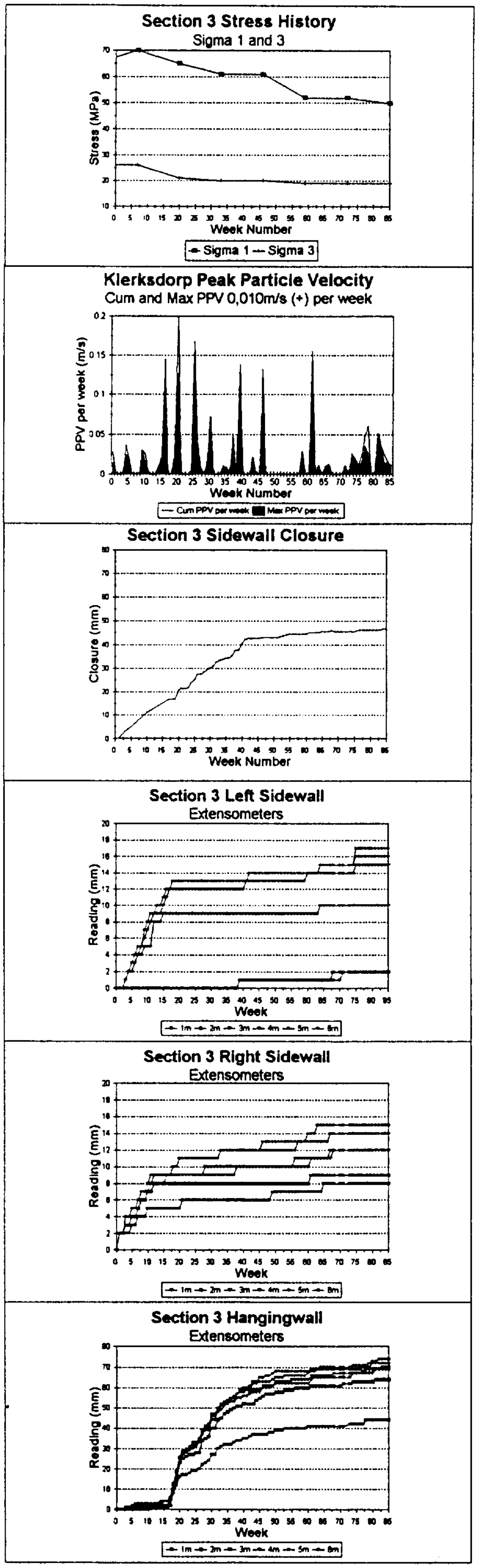
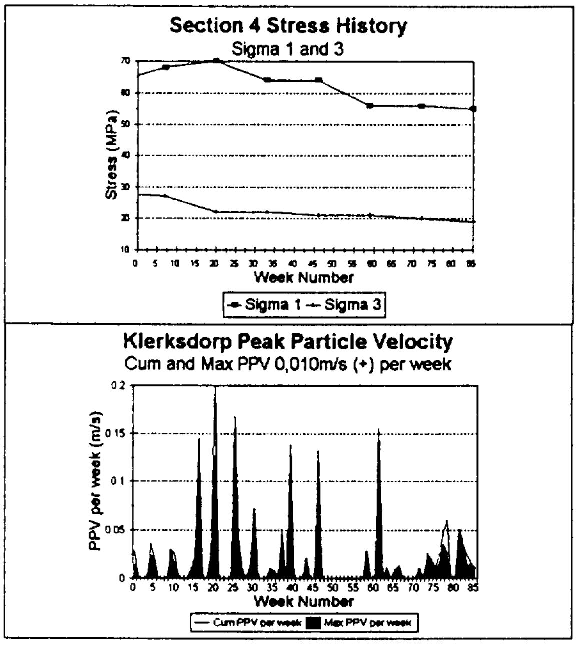


Figure 36: Section 4 stress history and deformation measurements



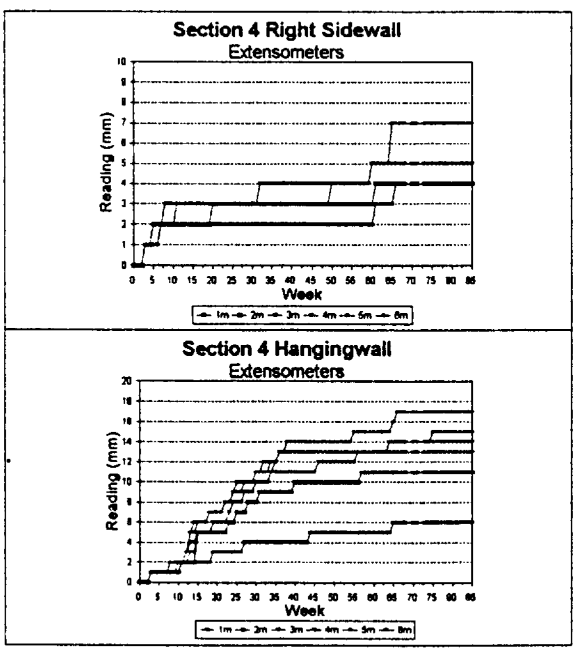


Figure 37: Section 5 stress history and deformation measurements

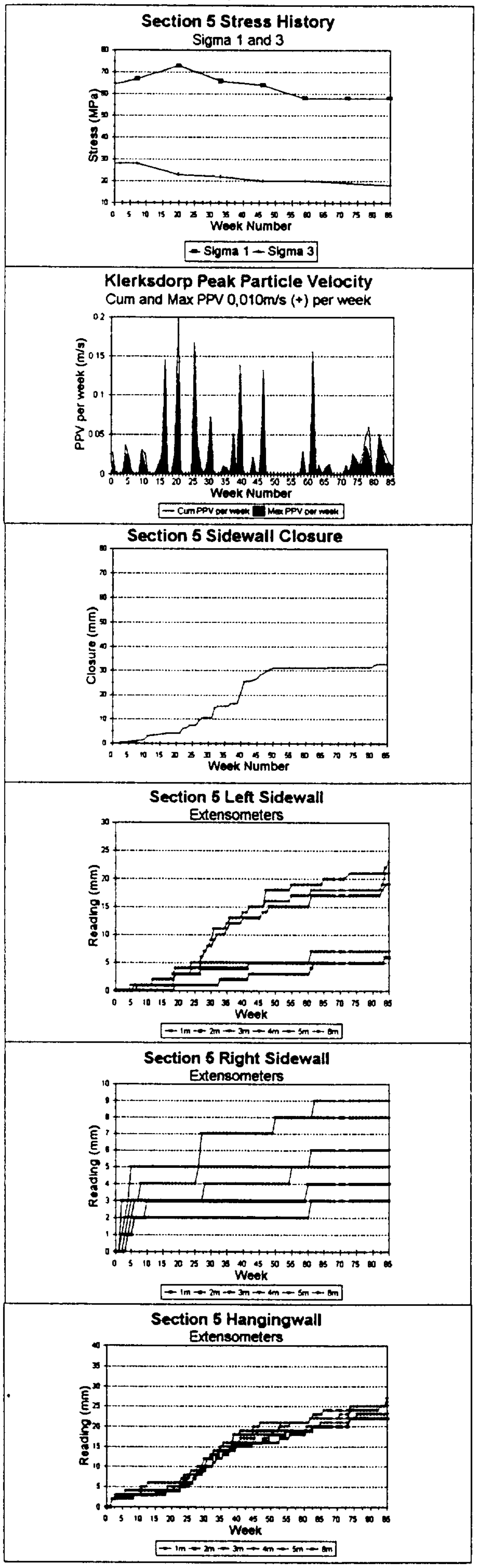


Figure 38: Section 6 stress history and deformation measurements

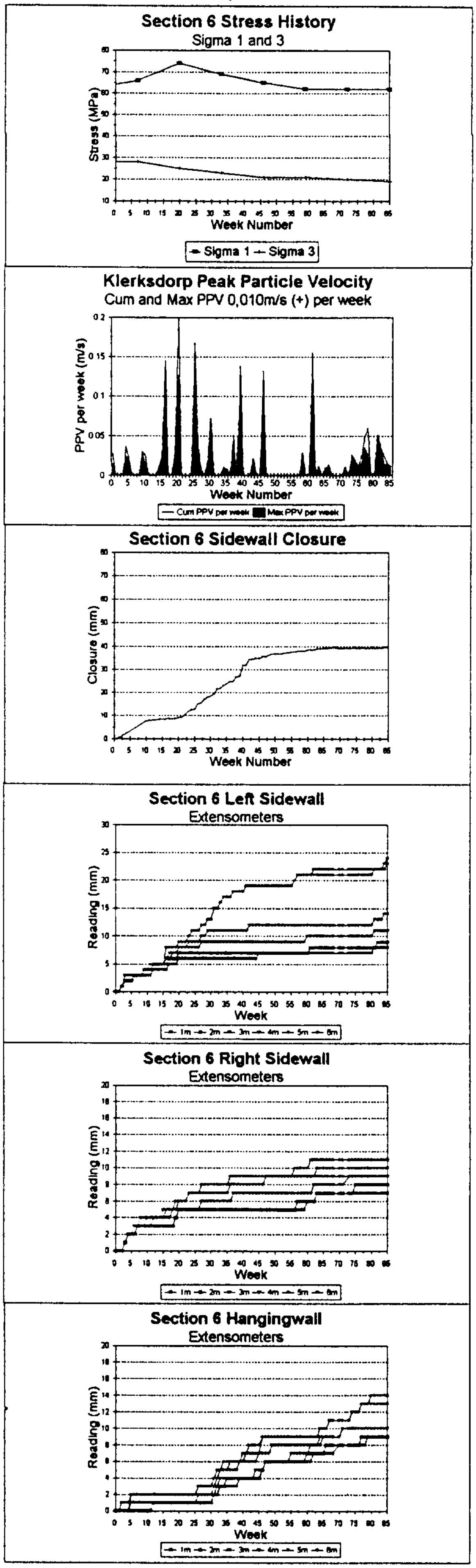


Figure 39: Section 7 stress history and deformation measurements

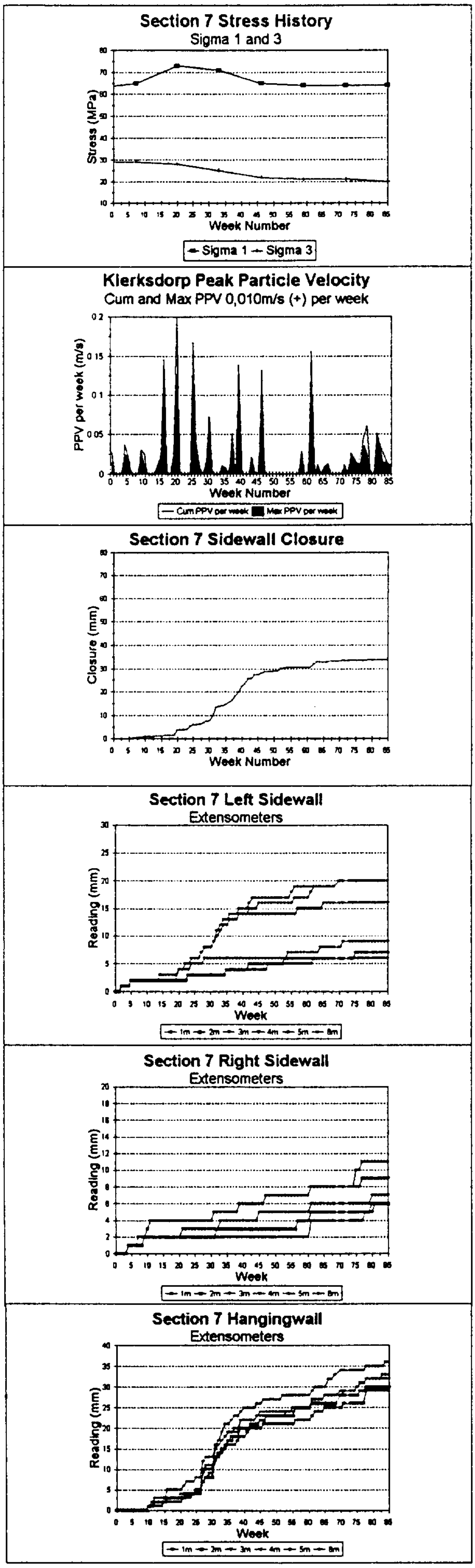


Figure 40: Section 8 stress history and deformation measurements

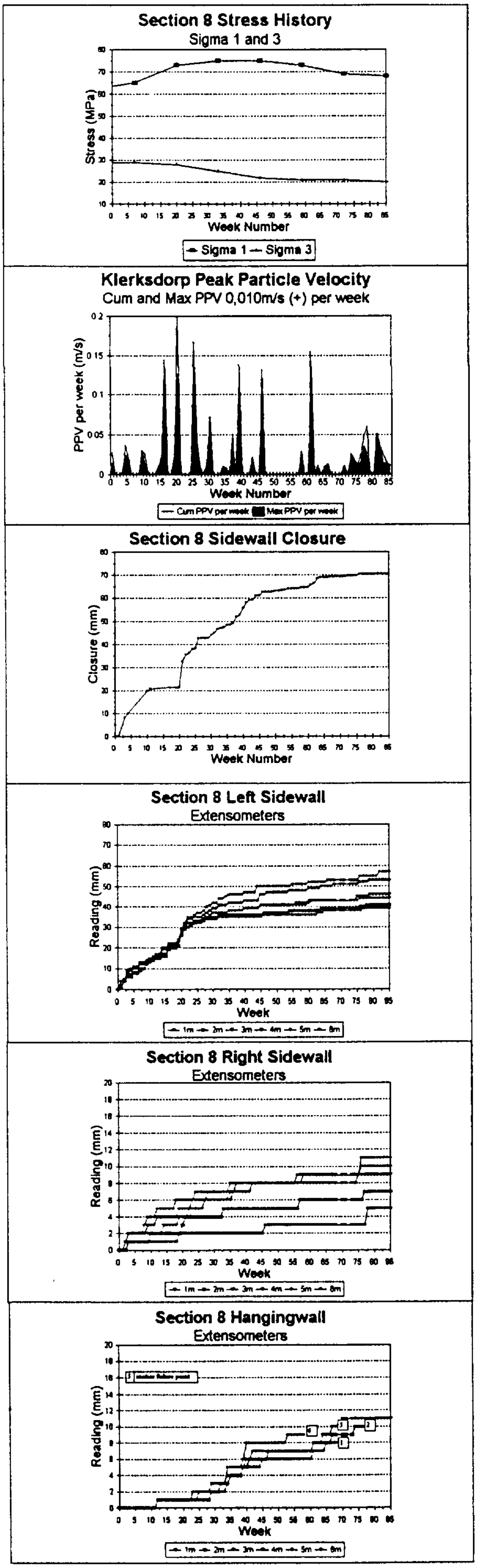


Figure 41: Section 9 stress history and deformation measurements

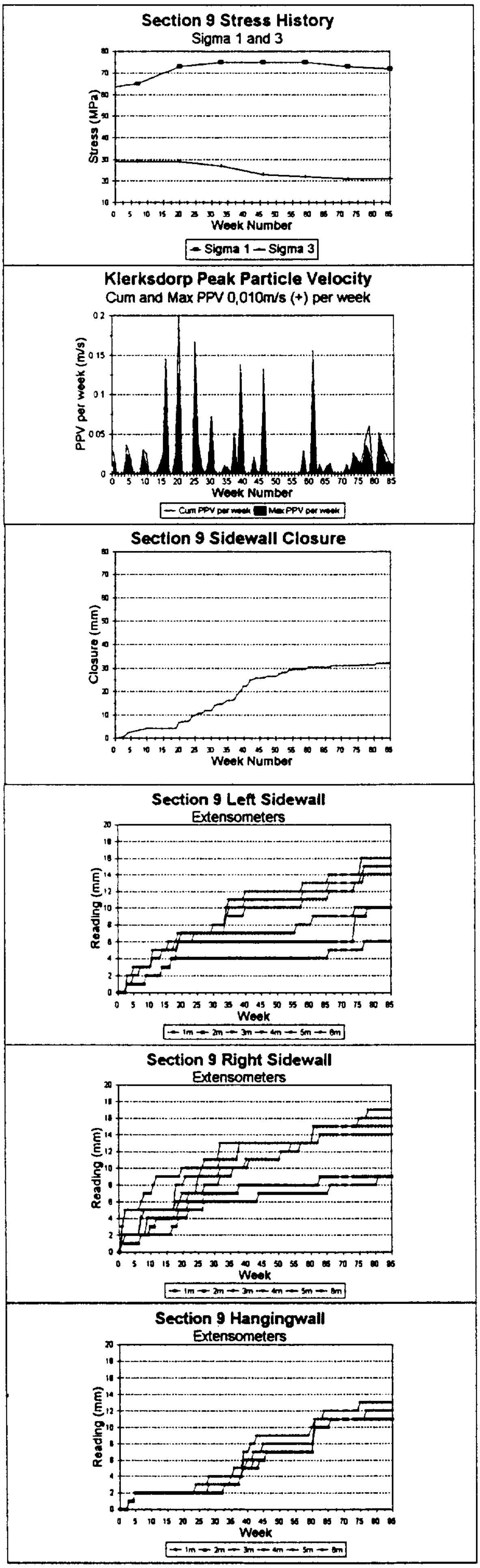
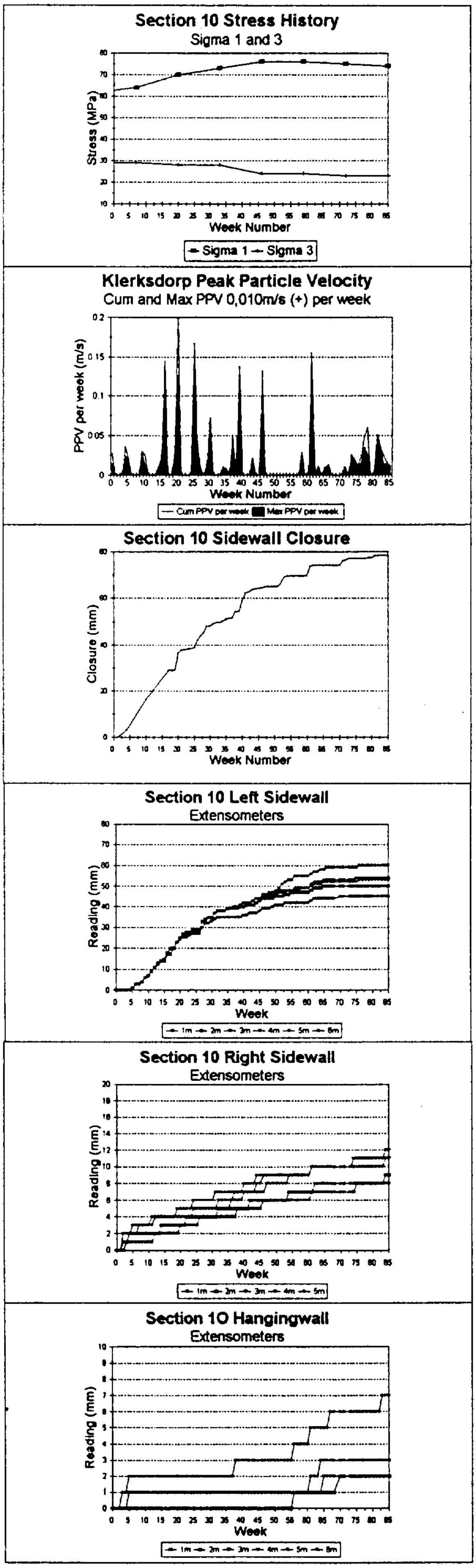


Figure 42: Section 10 stress history and deformation measurements



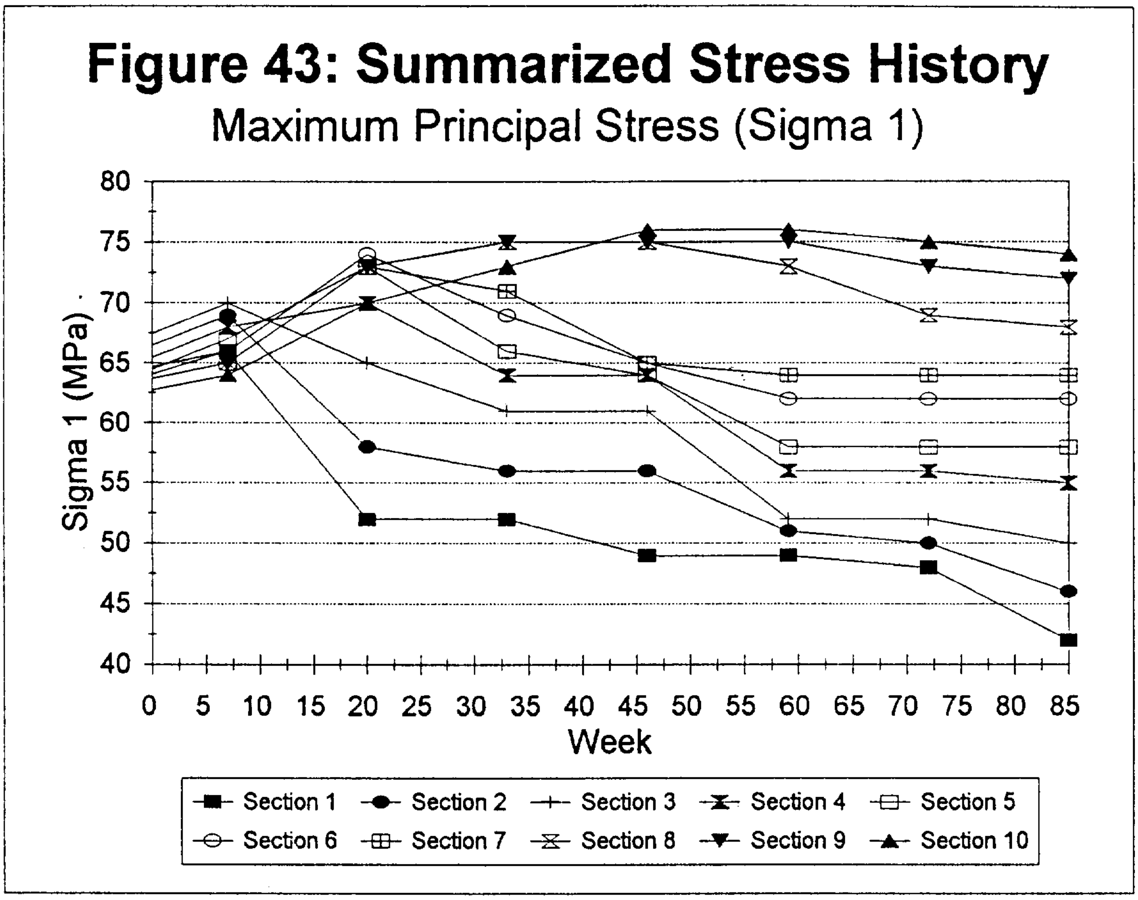


Figure 43: Summarized Stress History
Maximum Principal Stress (Sigma 1)
Sigma 1 (MPa)
80
75
70
65
60
55
50
45
40
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85
Week
Section 1
Section 2
Section 3
Section 4
Section 5
Section 6
Section 7
Section 8
Section 9
Section 10

Figure 44: Visual representation of Maximum Principal Stress dip and dip direction changes

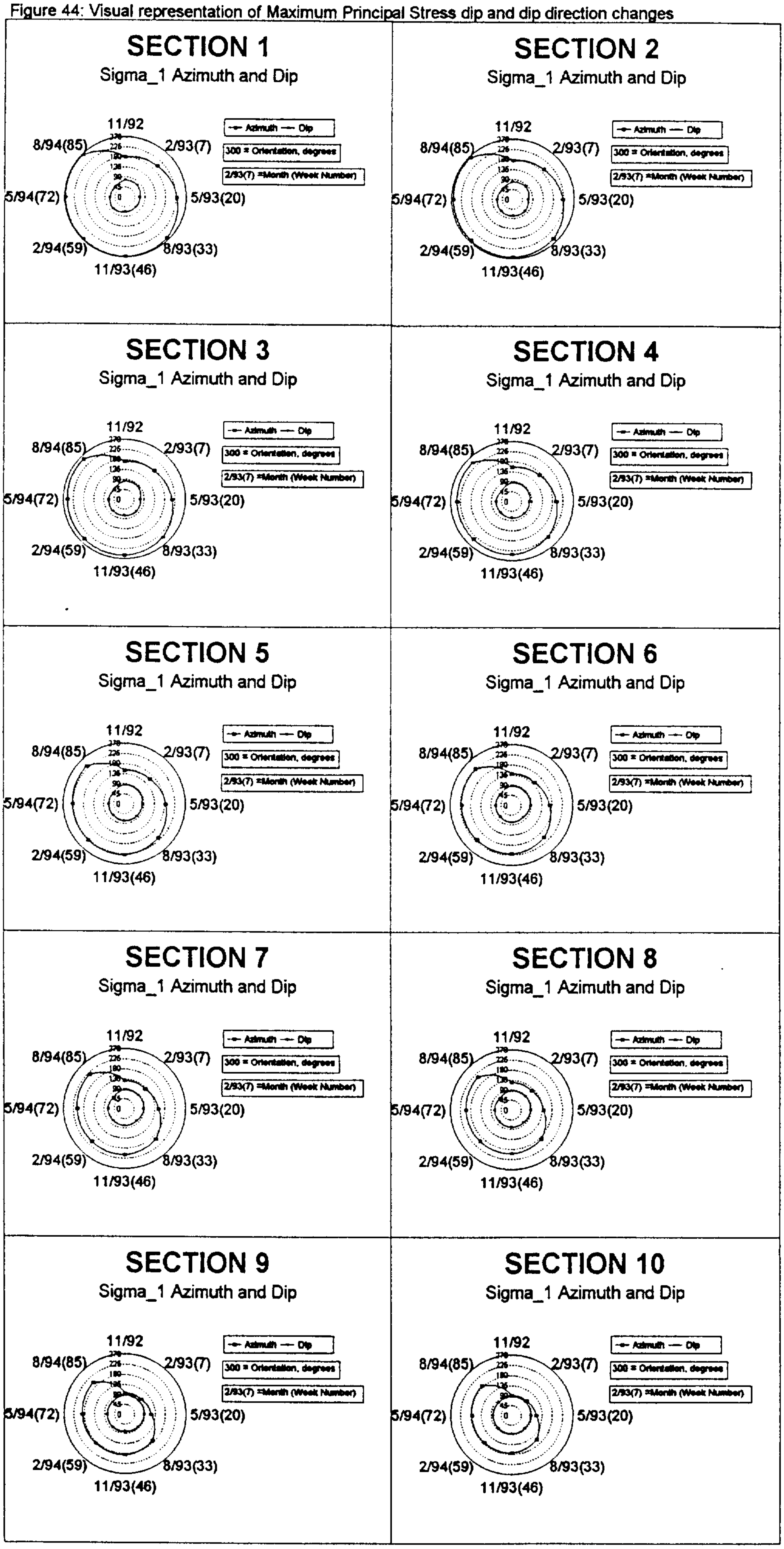


Figure 45: FLAC model set-up, plus zoomed image of last step view per support

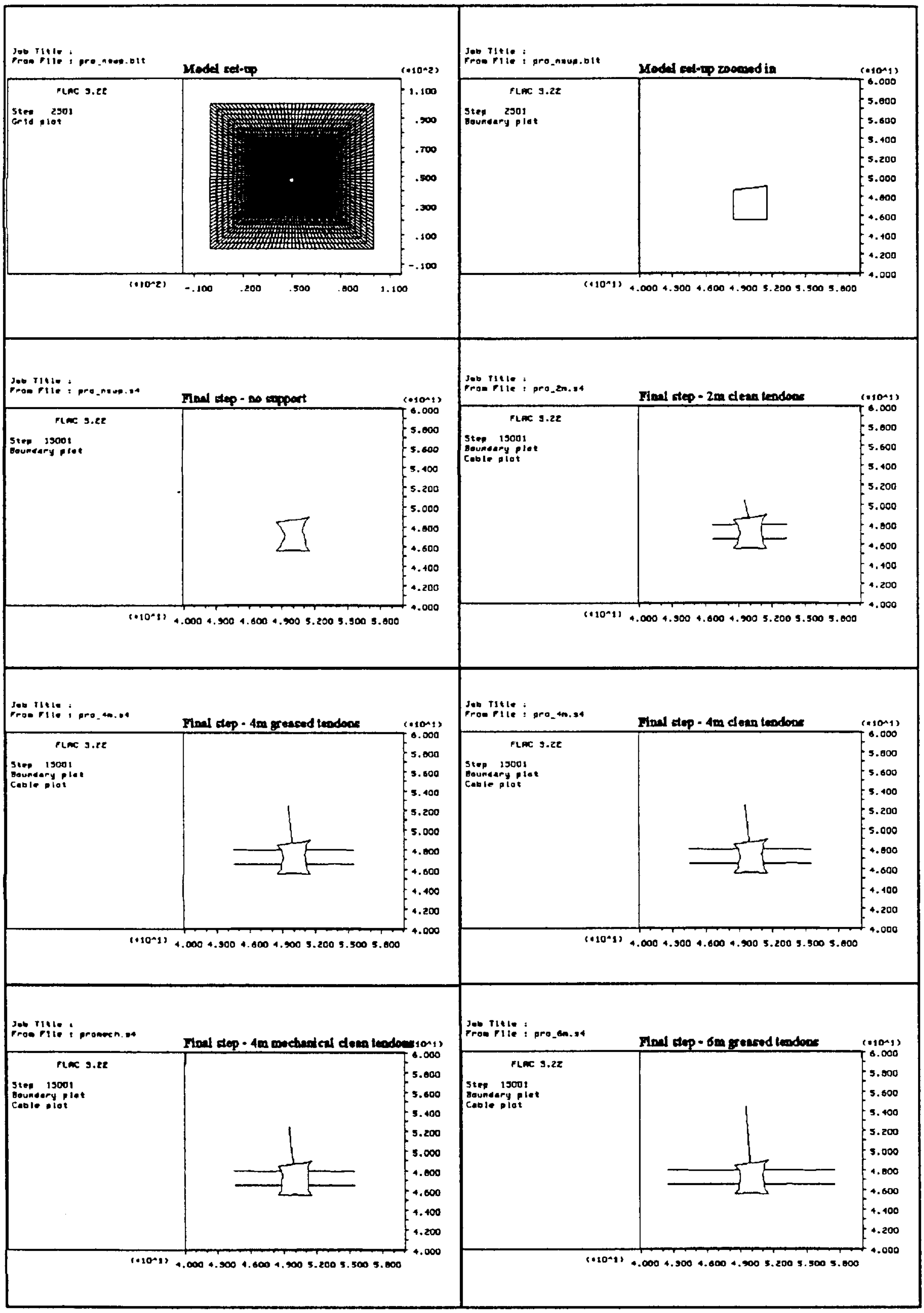


Figure 46: UDEC model set-up, plus last step view per support

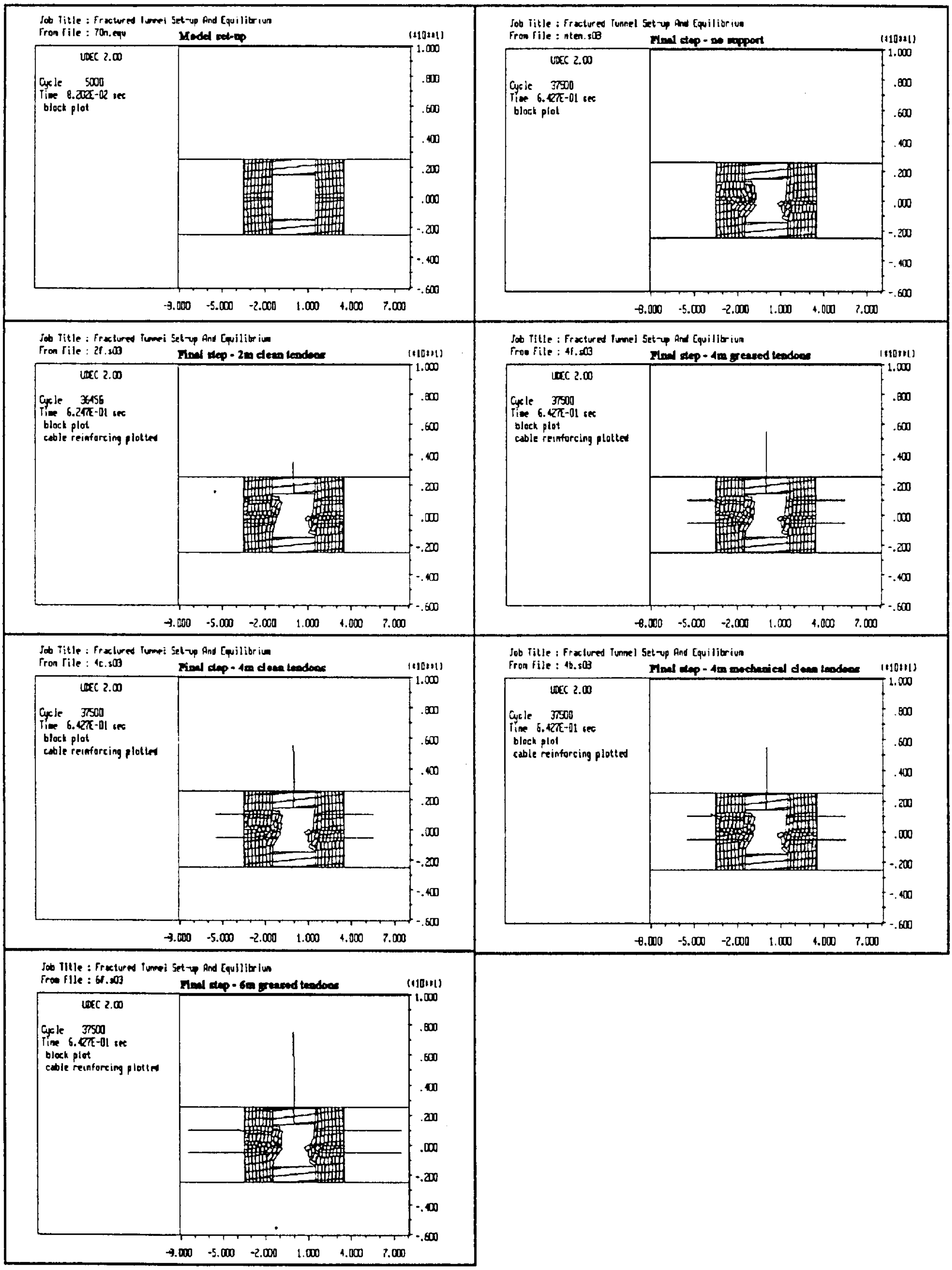


Figure 47: Dilations calculated using FLAC and UDEC

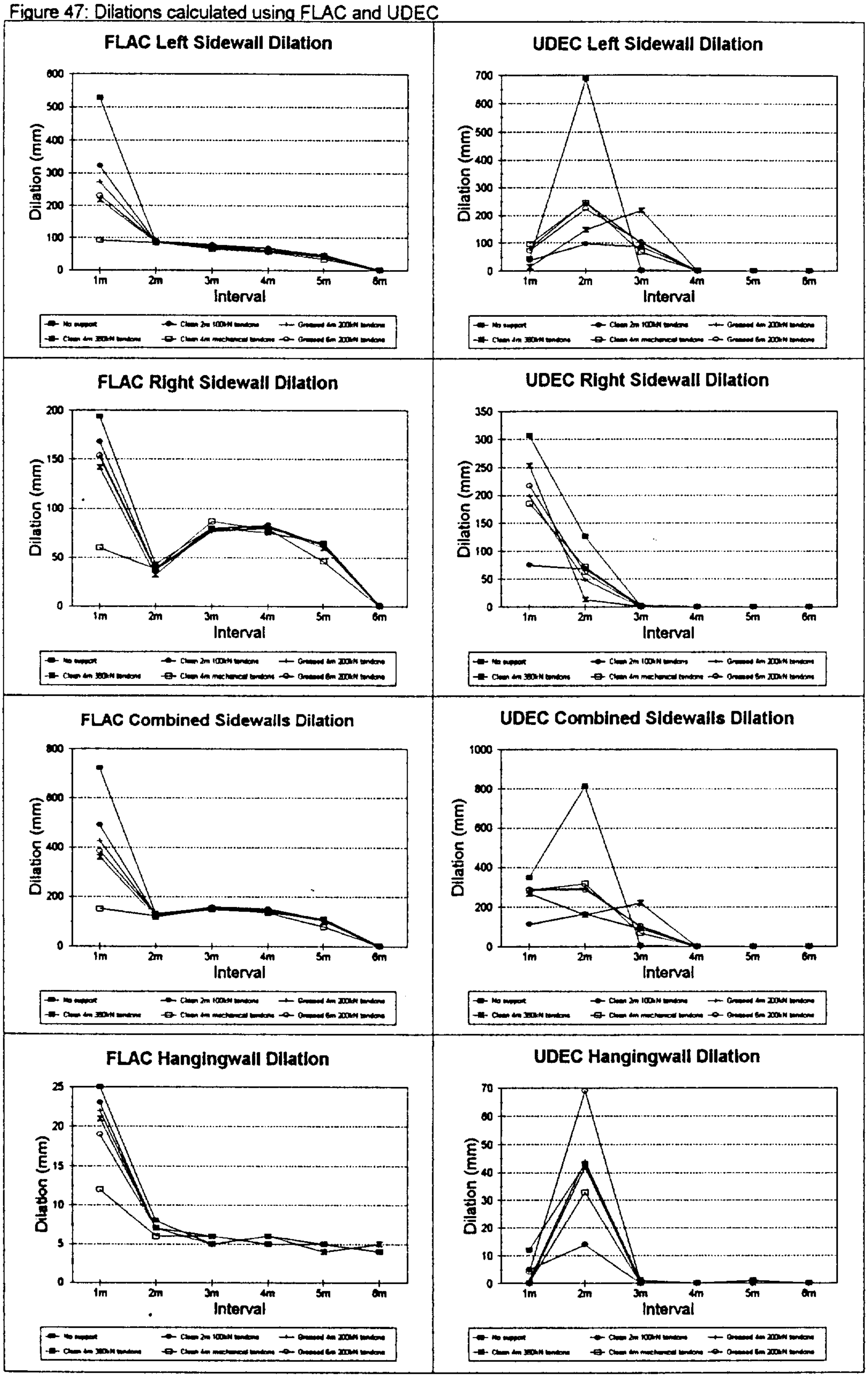


Figure 48: Summary of sidewall closure

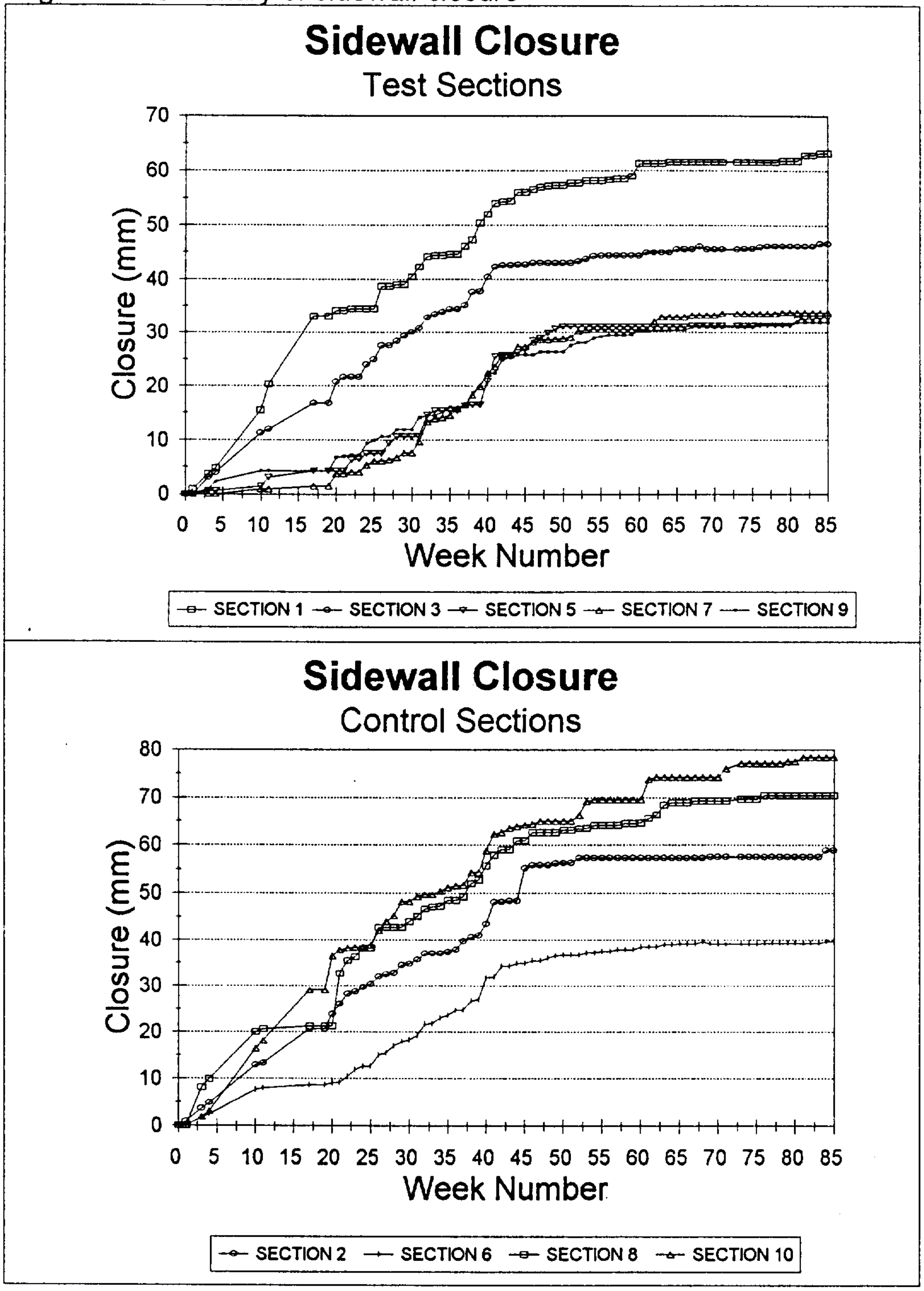


Figure 49: Summary of sidewall and hangingwall dilation

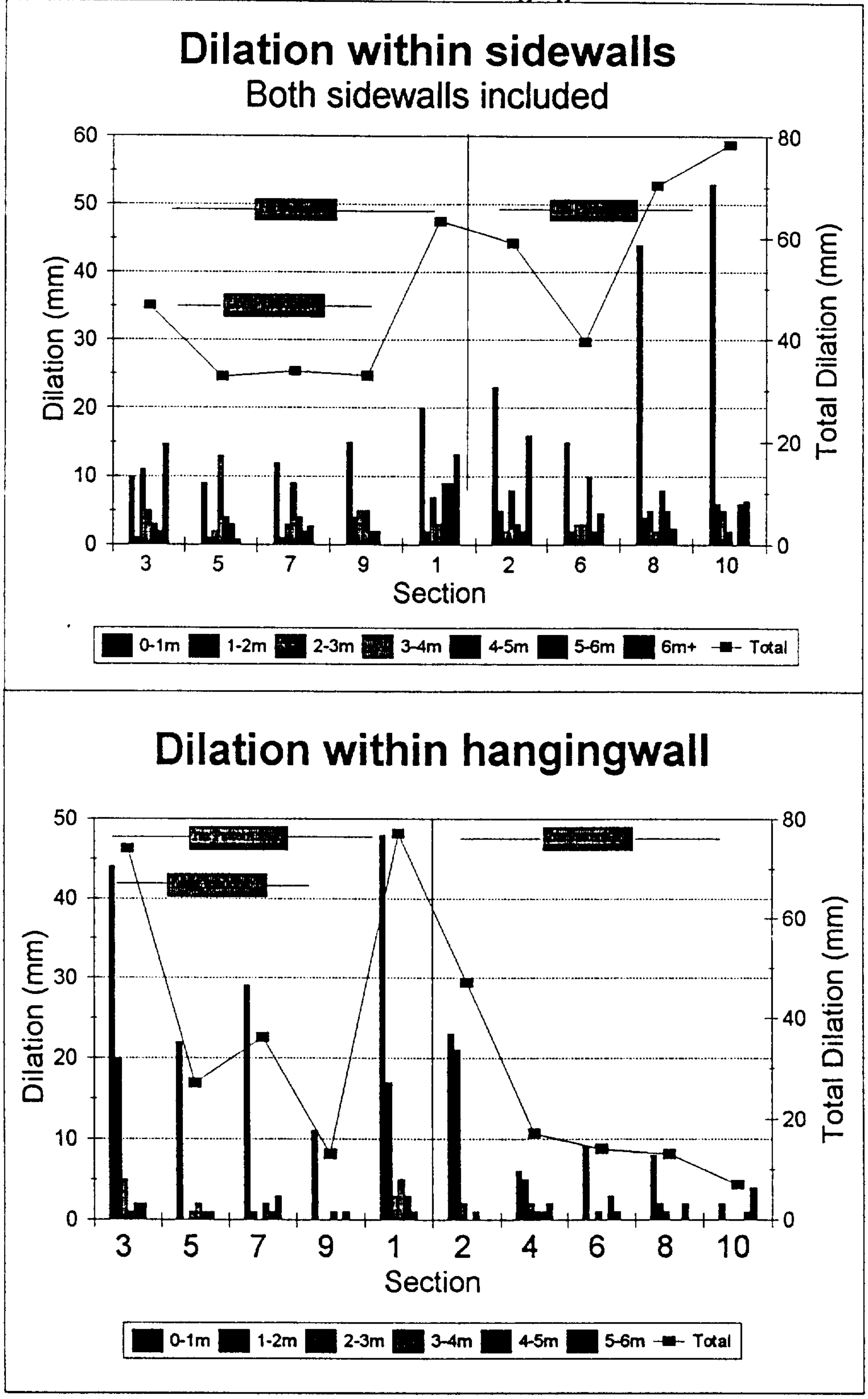


Figure 50: Summary of measured dilation per metre in each tunnel wall

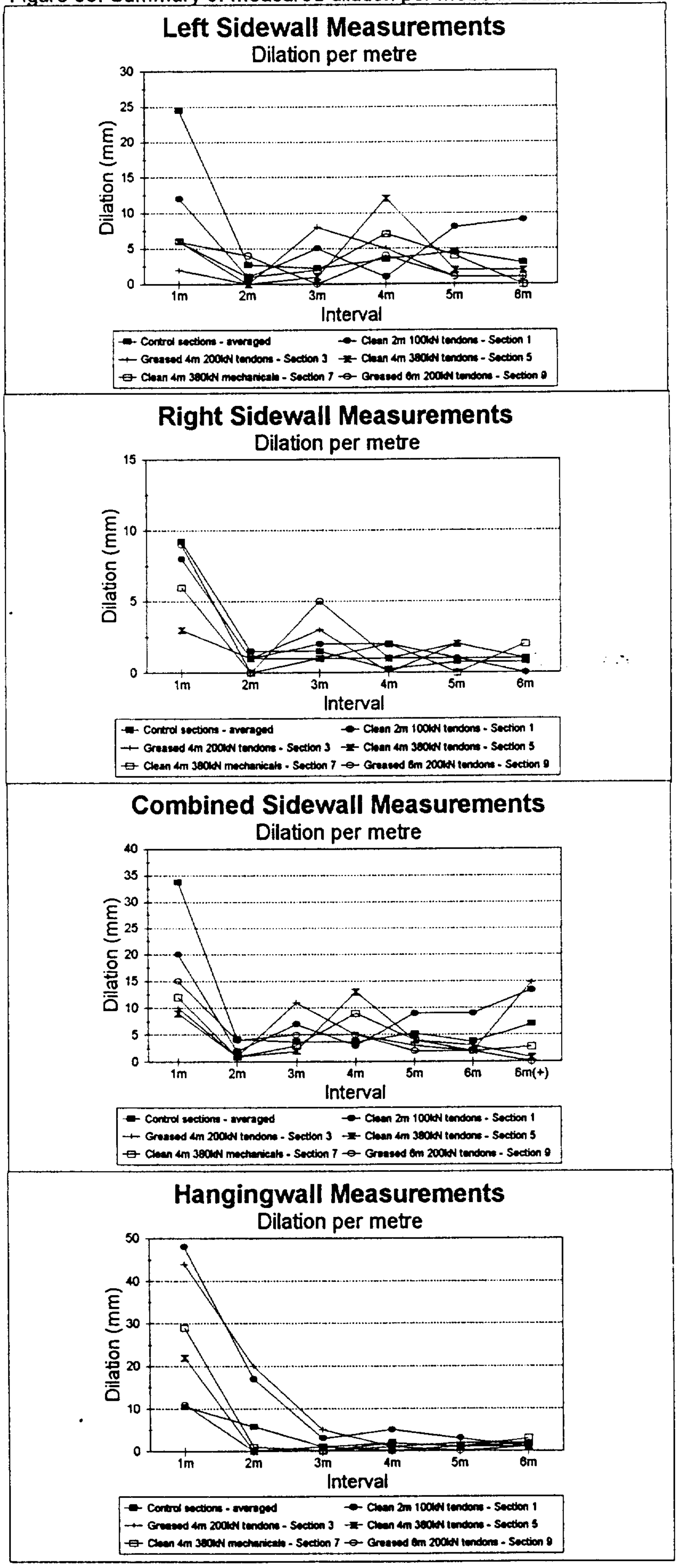


Figure 51: Tunnel condition rating for hangingwall, sidewalls and total

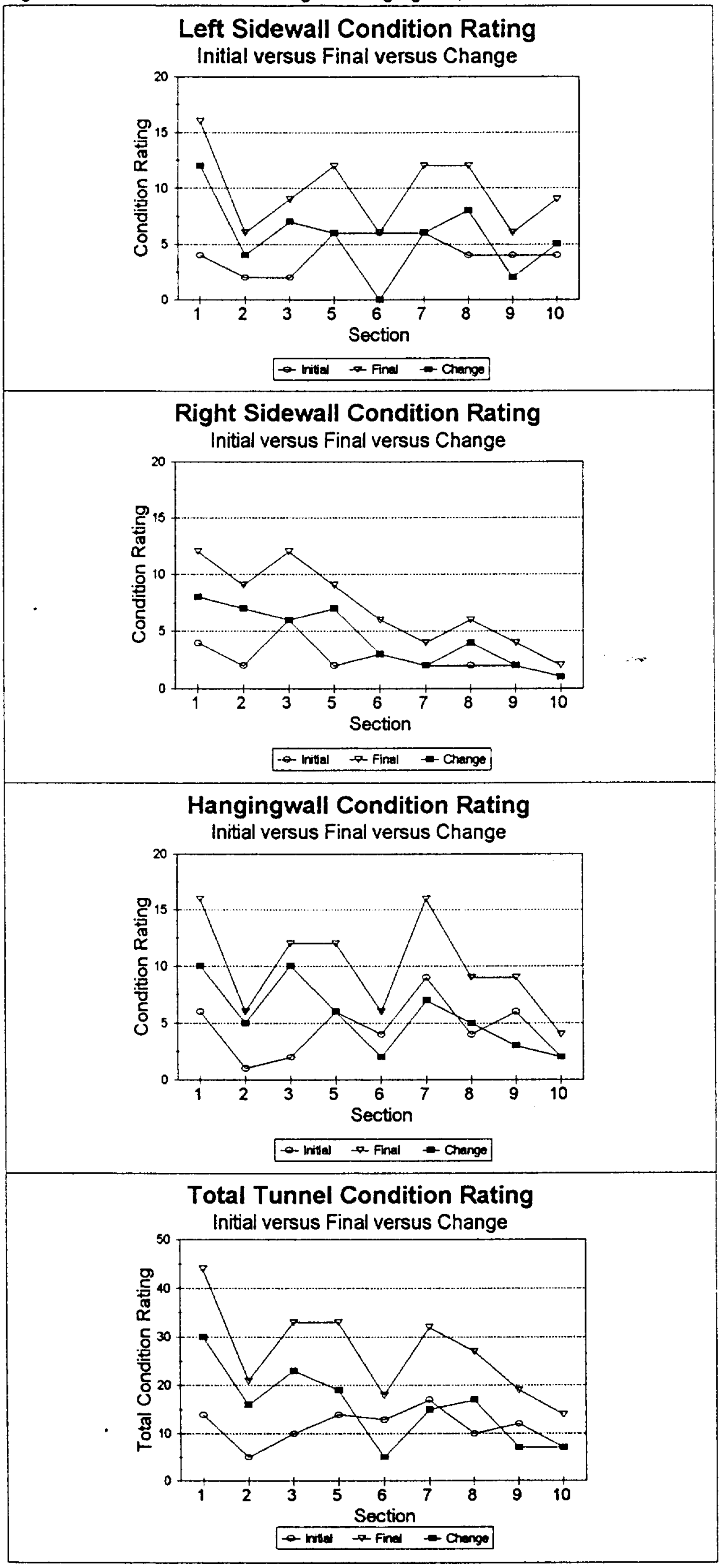


FIG: 52
VISUAL BOREHOLE CONDITION RATING AND REPRESENTATION OF BORESCOPE OBSERVATIONS

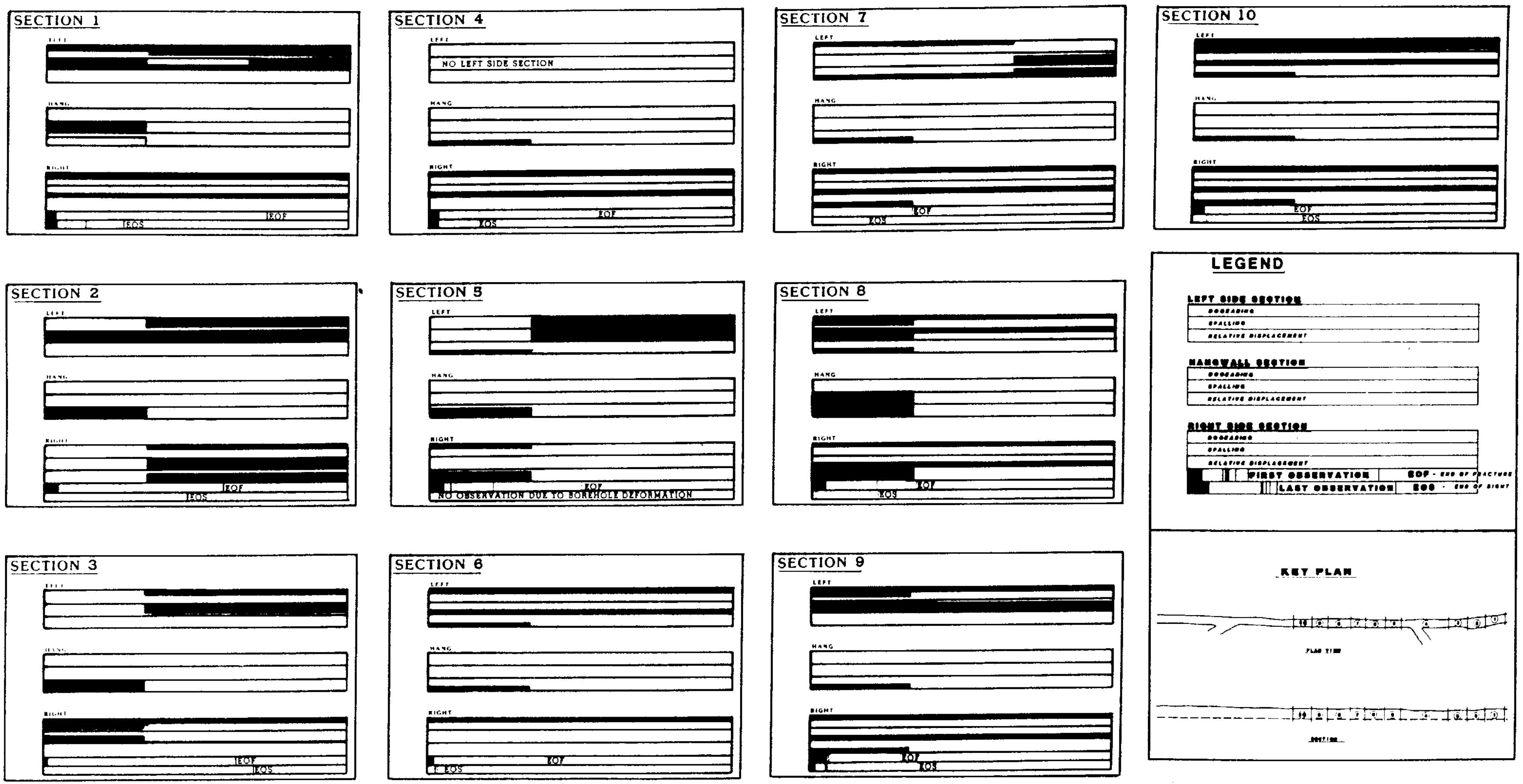


Figure 53: Visual inner sidewall condition rating

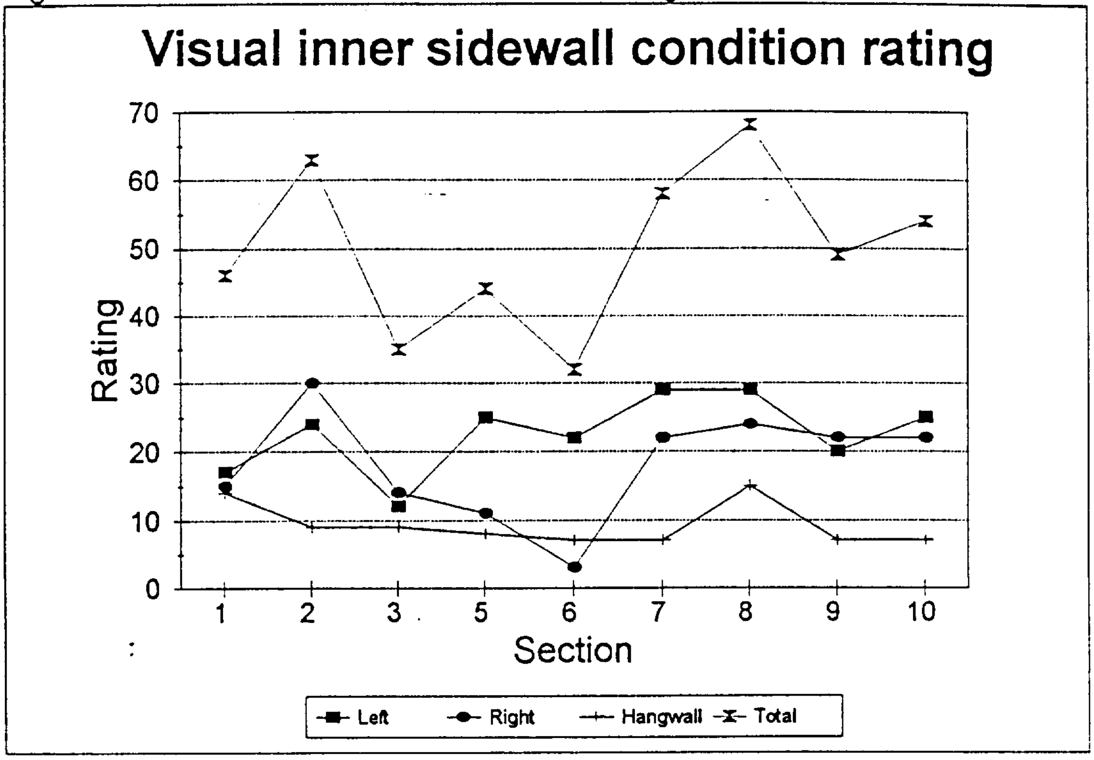


Figure 54: Photograph of observed shear displacements in the hangingwall

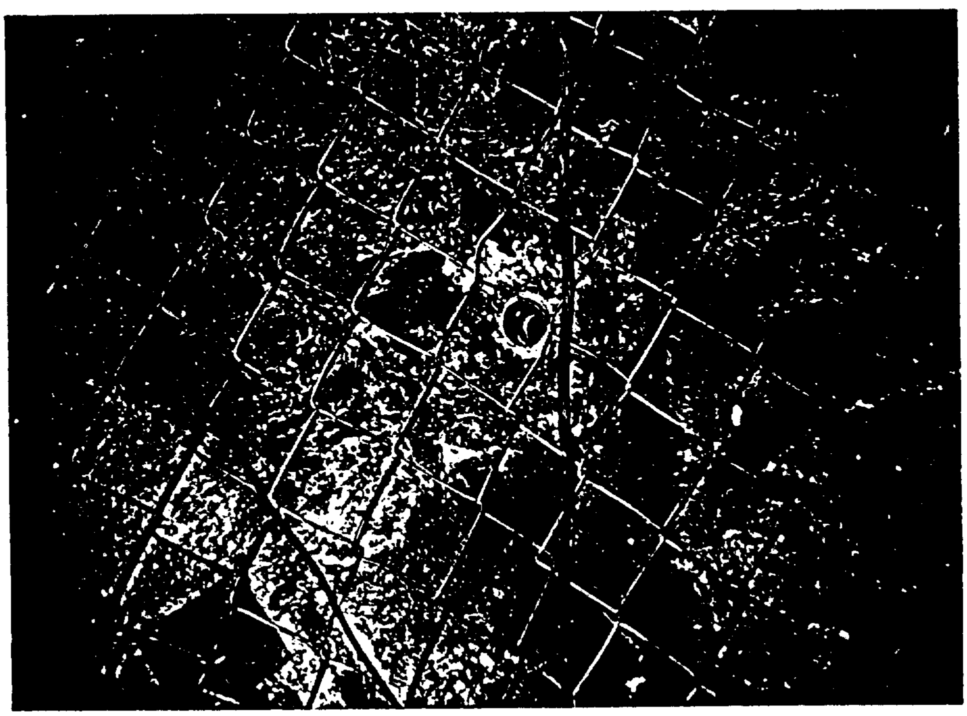
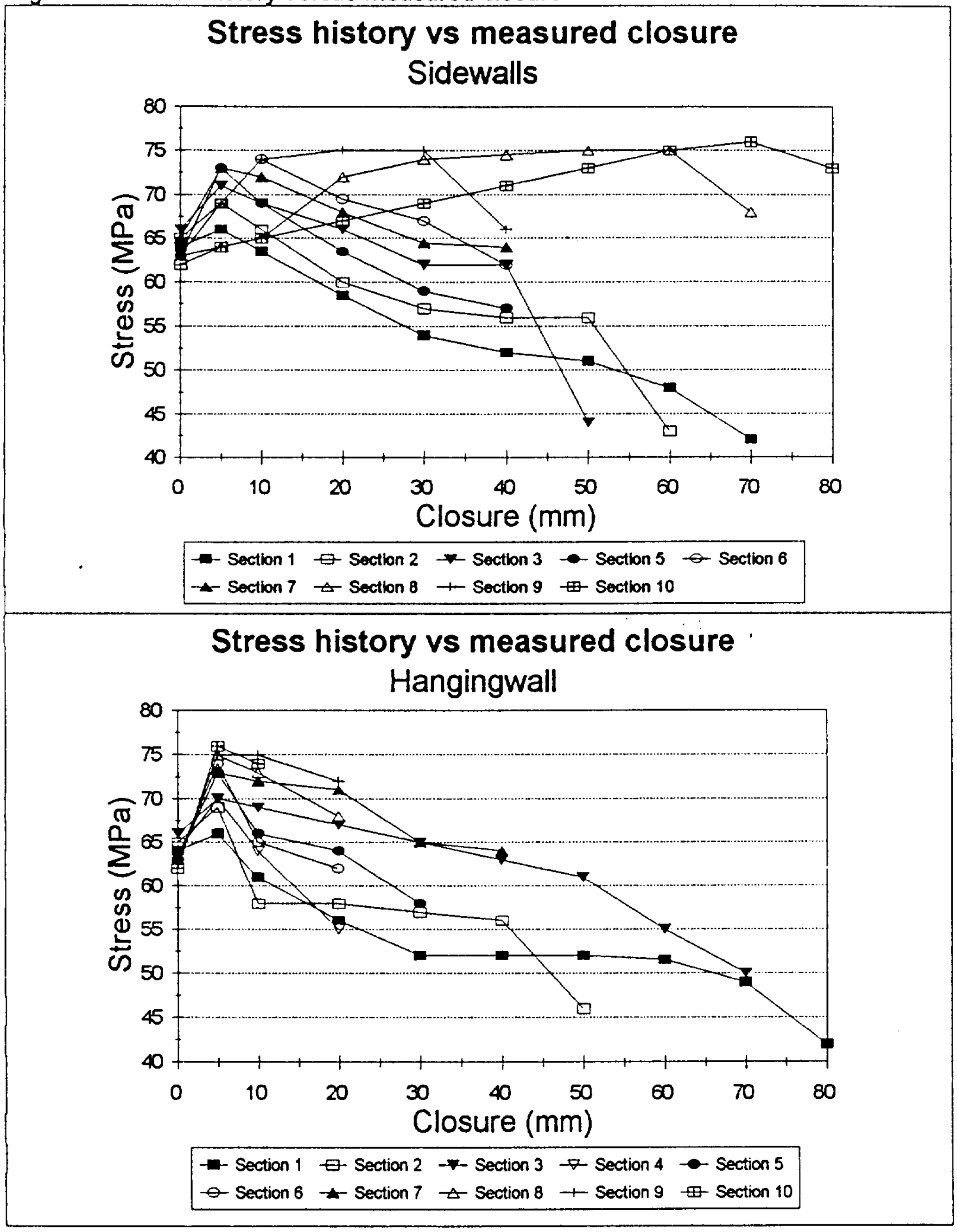


Figure 55: Stress history versus measured closure



Tables relating to data from the tunnel project

Table 6: Laboratory test results on core from the experimental site

Rock Type	Confining Pressure (MPa)	Compressive Strength (MPa)	Young's Modulus (GPa)	Poisson's Ratio
MB5	0 12,5 25,0	156,7 301,7 371,0	62.6	19
Millar	0 12,5 25,0	260,0 368,0 505,7	658	11
MB6	0 12,5 25,0	206,9 233,2 273,3	681	21
Sill	0 12,5 25,0	339,5 494,5 530.2	84	28

Table 7: Support type and density in the project site

Section; length; width; height; area	Support types	No of units per type	Tendons/m^2 per type: support resistance (kN/m^2)	Tendons/m^2 total : support resistance (kN/m^2)
1; 7,9; 3,2; 3,1; 74,2	1,5m rockstuds; double 2m pattern looped 2m wire ropes	21 91	0,28:28,3 1,22:122,6	1,51:150,9
2; 7,6; 4,5; 3,6; 88,9	1,5m rockstuds; 2m pattern looped 2m wire ropes	15 43	0,17:16,9 0,48:48,4	0,65:65,3
3; 7,5; 3,6; 3,6; 81,0	1,5m rockstuds; 2m patterns of looped 2m wire ropes and 4m greased looped wire ropes, for a double 2m pattern	16 46 45	0,20:19,7 0,57:56,7 0,56:111,1	1,32:187.5
4; 19,5; --; 3,8; 195,6	1,5m rockstuds; 2m pattern looped 2m wire ropes	58 93	0,30:29,7 0,48:47,5	0,77:77,2
5; 6,1; 3,4; 4,0; 69,5	1,5m rockstuds; 2m patterns of looped 2m wire ropes and 4m clean wire ropes, for a double 2m pattern	24 36 35	0,34:34,5 0,52:51,8 0,50:191,4	1,36:277,7
6; 8,1; 3,2; 4,8; 103,6	1,5m rockstuds; 2m pattern looped 2m wire ropes	19 36	0,18:18,3 0,35:34,7	0,53:53,1
7; 6,1; 3,4; 4,4; 74,4	1,5m rockstuds; 2m patterns of looped 2m wire ropes and 4m clean ropes, mechanically anchored, for a double 2m pattern	15 35 35	0,20:20,1 0,47:47,0 0,47:178,8	1,14:245,9
8; 9,0; 3,3; 4,3; 107,1	1,5m rockstuds; 2m pattern looped 2m wire ropes	16 48	0,15:14,9 0,45:44,8	0,60:59,7
9; 6,4; 3,5; 4,8; 83,8	1,5m rockstuds; 2m patterns of looped 2m wire ropes and 6m greased looped wire ropes, for a double 2m pattern	10 36 37	0,12:11,9 0,43:42,9 0,44:88,3	0,99:143,1
10; 7,0; 3,7; 4,5; 88,9	1,5m rockstuds; 2m pattern looped 2m wire ropes	15 37	0,17:16,9 0,42:41,6	0,58:58,5

Table 8: Description of support types in the project site

Type	Description (length; diameter; UTS; construction; material type)
Rockstud	1,5m; 14,5mm; 100kN; round rod, rolled thread, three leaf bail type mechanical anchor, 100mm square 4,5mm thick bearing plate, domed washer and hexagonal nut; high tensile steel
Short, looped wire rope	2,2m; 10mm; 100kN; round strand consisting of round, 10 of 2mm external, and 9 of 1mm internal wires; high tensile steel
Long, looped, greased, wire rope	4 and 6m; 16mm; 200kN; triangular strand consisting of round, 12 of 3mm external and 6 of 2,5mm internal wires; high tensile steel
Long, straight wire rope	4m; 18,5mm; 380kN; round strand consisting of round, 7 of 6mm wires; high tensile steel - same strand as in the mechanical anchors
Long, mechanically anchored wire rope	4m; 18,5mm; 380kN; round strand consisting of round, 7 of 6mm wires, three leaf remote release spring type anchor, 150mm square 12,5mm thick bearing plate, domed washer and retaining wedge tensioning assembly; high tensile steel

Table 9: PPV-rockburst relationships (Jesenak et al, 1993 and Ortlepp, 1993a)

Author(s)	Source	Damage term	PPV (m/s)
Kaiser et al, 1992	Mining	None Significant	0,05 0,40-0,80
Owen and Scholl, 1981	Earthquakes; civil works	None Major	0,20 0,90
Langefors and Kihlstrom,1963	Mining	Rockfall New fractures formed	0,30 0,60
Hedley, 1992	Mining	None Loose rock falls Rockfalls Severe damage	0,05 0,05-0,30 0,30-0,60 >0,60
McGarr and Bicknell, 1988	Mining	Near faults	2,00-4,00
Roberts and Brummer, 1988	Mining	Damage	110
Blake and Cuvelier, 1990	Mining	Light Serious	0,97 1,50
Van den Heever et al, 1984	Mining	Severe damage	0,26-3,50

Table 10: Rating considerations in using the Wiseman Tunnel Classification System (Wiseman, 1979)

Parameter	Description	Rating
Degree of fracturing	Little fracturing, no open fractures visible	1
	Localized slabbing, fractures open <10mm	2
	Medium sized slabs, slabbing depth <0,6m, fractures open <20mm	3
	Widespread slabbing, fractures open >20mm	4
	Walls broken into interlocked blocks	12
	Walls severely broken into loose blocks	13
	Walls intensely fragmented	14
Degree of movement	Little or no movement	1
	Infrequent fall out <0,3m	2
	Infrequent fall out <0,6m	3
	Frequent fall out 0,6m plus	4

Table 11: Section 1 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
1	Rock type: MB6 quartzite	2	2
	Joints/bedding: 9 layers, ave 0,3m with one weak 0,09m thick shale layer	5	5
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	2	4
	right	2	4
	hang	3	4
	Movement:		
	left	2	4
	right	2	3
	hang	2	4
	Position:		
	left	3	3
	right	3	3
	hang	2	3
	Total	31	42

Table 12: Section 2 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
2	Rock type: MB6 and Millar quartzite	2	2
	Joints/bedding: MB6 5 layers, ave 0,3m; Millar 4 layers, ave 1,0m with one weak 0,09m thick shale layer in mid-sidewall	4	4
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	2	3
	right	2	3
	hang	1	3
	Movement:		
	left	1	2
	right	1	3
	hang	1	2
	Position:		
	left	3	3
	right	2	3
	hang	3	3
	Total	25	34

Table 13: Section 3 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
3	Rock type: MB6 and Millar quartzite	2	2
	Joints/bedding: MB6 3 layers, ave 0,3m; Millar 3 layers, ave 0,8m with one weak 0,09m thick shale layer	3	3
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	2	3
	right	3	4
	hang	2	3
	Movement:		
	left	1	3
	right	2	3
	hang	1	4
	Position:		
	left	3	3
	right	2	3
	hang	3	4
	Total	27	38

Table 14: Section 4 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
4	Rock type: MB6 and Millar quartzite	2	2
	Joints/bedding: MB6 3 layers, ave 0,3m; Millar 4 layers, ave 0,7m	3	3
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	-	-
	right	2	2
	hang	2	2
	Movement:		
	left	-	-
	right	1	1
	hang	1	2
	Position:		
	left	-	-
	right	3	3
	hang	2	3
	Total	--	--

Table 15: Section 5 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
5	Rock type: Millar quartzite	1	1
	Joints/bedding: 5 layers, ave 0,7m	1	1
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	3	4
	right	2	3
	hang	3	4
	Movement:		
	left	2	3
	right	1	3
	hang	2	3
	Position:		
	left	2	3
	right	3	3
	hang	2	4
	Total	25	35

Table 16: Section 6 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
6	Rock type: Millar quartzite	1	1
	Joints/bedding: 5 layers, ave 0,7m	1	1
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	3	3
	right	3	3
	hang	2	3
	Movement:		
	left	2	2
	right	1	2
	hang	2	2
	Position:		
	left	2	3
	right	3	3
	hang	2	3
	Total	25	29

Table 17: Section 7 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
7	Rock type: Millar quartzite	1	1
	Joints/bedding: 6 layers, ave 0,5m	2	2
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	3	4
	right	2	2
	hang	3	4
	Movement:		
	left	2	3
	right	1	2
	hang	3	4
	Position:		
	left	2	3
	right	3	3
	hang	2	4
	Total	27	35

Table 18: Section 8 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
8	Rock type: Millar quartzite	1	1
	Joints/bedding: 5 layers, ave 0,7m	1	1
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	2	4
	right	2	3
	hang	2	3
	Movement:		
	left	2	3
	right	1	2
	hang	2	3
	Position:		
	left	3	3
	right	3	3
	hang	2	3
	Total	24	32

Table 19: Section 9 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
9	Rock type: Millar quartzite	1	1
	Joints/bedding: 8 layers, ave 0,5m	2	2
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	2	3
	right	2	2
	hang	3	3
	Movement:		
	left	2	2
	right	1	2
	hang	2	3
	Position:		
	left	2	3
	right	3	3
	hang	2	3
	Total	25	30

Table 20: Section 10 initial and final Wiseman ratings

Section	Geotechnical description - adapted Wiseman (1979) condition description	Initial Wiseman Rating	Final Wiseman Rating
10	Rock type: Millar quartzite	1	1
	Joints/bedding: 8 layers, ave 0,5m	2	2
	Orientation: strike	2	2
	Bedding dip: < 15 degrees	1	1
	Condition:		
	left	2	3
	right	1	1
	hang	2	2
	Movement:		
	left	2	3
	right	1	1
	hang	1	2
	Position:		
	left	3	3
	right	3	3
	hang	3	3
	Total	24	27

Table 21: Summary of the MINSIM-D stress history for each section

Month/year(week*)	8/92(-16)	11/92(-4)	2/93(7)	5/93(20)	8/93(33)	11/93(46)	2/94(59)	5/94(72)	8/94(85)
Mining step	Step 0	Step 1	Step 2	Step 3	Step 4	Step 5	Step 6	Step 7	Step 8
Section & parameter									
1 sig1	62	64	66	52	52	49	49	48	42
1 al1	78	77	75	64	64	68	77	70	64
1 az1	178	180	198	228	260	267	267	267	268
1 sig3	24	23	22	22	22	22	22	20	18
2 sig1	64	65	69	58	56	56	51	50	46
2 al1	83	80	81	68	75	75	75	71	65
2 az1	174	175	191	219	252	262	262	263	265
2 sig3	26	25	24	22	22	20	20	20	18
3 sig1	64	66	70	65	61	61	52	52	50
3 al1	84	83	84	71	75	73	73	71	65
3 az1	170	170	184	209	240	253	253	254	257
3 sig3	27	26	26	21	20	20	19	19	19
4 sig1	63	64	68	70	64	64	56	56	55
4 al1	87	85	86	78	76	75	72	71	67
4 az1	156	155	168	194	224	241	241	243	248
4 sig3	28	28	27	22	22	21	21	20	19
5 sig1	62	63	67	73	66	64	58	58	58
5 al1	87	86	86	80	77	76	71	70	69
5 az1	151	150	159	183	212	228	228	231	237
5 sig3	28	28	28	23	22	20	20	19	18
6 sig1	62	63	66	74	69	65	62	62	62
6 al1	87	87	86	83	73	76	72	72	71
6 az1	144	141	144	167	202	217	218	222	228
6 sig3	28	28	28	25	23	21	21	20	19
7 sig1	61	63	65	73	71	65	64	64	64
7 al1	88	87	87	85	75	76	73	74	72
7 az1	136	128	128	150	193	207	208	212	220
7 sig3	29	29	29	28	25	22	21	21	20
8 sig1	62	63	65	73	75	75	73	69	68
8 al1	88	87	87	83	77	77	76	76	76
8 az1	132	124	124	140	184	197	199	203	211
8 sig3	29	29	29	28	25	22	21	21	20
9 sig1	62	63	65	73	75	75	75	73	72
9 al1	88	87	86	84	80	78	80	80	80
9 az1	109	98	99	115	168	184	186	191	201
9 sig3	29	29	29	29	27	23	22	21	21
10 sig1	61	62	64	70	73	76	76	75	74
10 al1	88	87	86	84	81	83	83	84	83
10 az1	99	90	93	108	154	171	174	178	190
10 sig3	28	29	29	28	28	24	24	23	23

* Week relative to the start of the project
sig1 = maximum principal stress (MPa)
al1 = dip of the maximum principal stress (degrees from horizontal)
az1 = azimuth (dip direction) of the maximum principal stress (degrees clock wise from North)
sig3 = minor principal stress (MPa)

Table 22: Dilations and closure calculated using the FLAC inelastic computer programme

Section	Closure (mm)	Left sidewall							Right sidewall							Combined sidewalls							Hangingwall						
		1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total
No support	1258	528	88	75	61	46	2	800	194	43	80	75	65	1	458	722	131	155	136	111	3	1258	25	8	5	6	5	4	53
Clean 2m 100kN tendons	1032	321	89	78	68	46	0	602	168	37	80	83	62	0	430	489	126	158	151	108	0	1032	23	7	6	5	5	4	50
Greased 4m 200kN tendons	944	273	87	71	56	48	0	535	152	37	76	80	64	0	409	425	124	147	136	112	0	944	22	7	5	6	5	4	49
Clean 4m 380kN tendons	871	218	86	69	60	42	0	475	142	32	80	82	60	0	396	360	118	149	142	102	0	871	21	7	5	6	4	5	48
Clean 4m mechanical tendons	642	92	85	65	56	34	0	332	60	38	87	79	46	0	310	152	123	152	135	80	0	642	12	6	6	5	5	4	38
Greased 6m 200kN tendons	924	231	89	78	67	45	0	510	154	38	78	80	64	0	414	385	127	156	147	109	0	924	19	7	6	5	5	4	46

Table 23: Dilations and closure calculated using the UDEC inelastic computer programme

Section	Closure (mm)	Left sidewall							Right sidewall							Combined sidewalls							Hangingwall						
		1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total
No support	1166	42	687	1	0	0	0	730	306	126	2	0	0	0	434	348	813	3	0	0	0	1164	12	43	1	0	1	0	57
Clean 2m 100kN tendons	467	38	97	86	0	0	0	221	75	68	3	0	0	0	146	113	165	89	0	0	0	367	5	14	0	0	0	0	19
Greased 4m 200kN tendons	677	81	248	101	0	0	0	430	199	48	0	0	0	0	247	280	296	101	0	0	0	677	1	44	1	0	0	0	46
Clean 4m 380kN tendons	648	14	148	219	0	0	0	381	253	13	1	0	0	0	267	267	161	220	0	0	0	648	0	42	0	0	0	0	42
Clean 4m mechanical tendons	670	97	247	67	0	0	0	411	186	72	1	0	0	0	259	283	319	68	0	0	0	670	0	33	0	0	1	0	34
Greased 6m 200kN tendons	679	72	225	103	0	0	0	400	217	61	1	0	0	0	279	289	286	104	0	0	0	679	4	69	1	0	0	0	74

Table 24: Summary of PPV of 0.010(+) m/s measured at the project site, and McGarr PPV for the same Energy Magnitude and distance

Week	0	1	2	3	4	5	6	7	8	9
Cumulative Klerksdorp PPV (week)	0.027	0	0	0	0.036	0.021	0	0	0	0.030
Maximum Klerksdorp PPV	0.015	0	0	0	0.025	0.021	0	0	0	0.03
Maximum McGarr PPV	0.106	0	0	0	0.152	0.134	0	0	0	0.173
Week	10	11	12	13	14	15	16	17	18	19
Cumulative Klerksdorp PPV (week)	0.026	0	0	0	0.010	0.023	0.145	0	0	0.025
Maximum Klerksdorp PPV	0.013	0	0	0	0.01	0.023	0.129	0	0	0.025
Maximum McGarr PPV	0.099	0	0	0	0.080	0.145	0.471	0	0	0.152
Week	20	21	22	23	24	25	26	27	28	29
Cumulative Klerksdorp PPV (week)	0.199	0	0	0	0	0.167	0.041	0.009	0	0.014
Maximum Klerksdorp PPV	0.127	0	0	0	0	0.152	0.041	0.009	0	0.014
Maximum McGarr PPV	0.467	0	0	0	0	0.527	0.215	0.077	0	0.100
Week	30	31	32	33	34	35	36	37	38	39
Cumulative Klerksdorp PPV (week)	0.072	0	0	0	0.010	0.008	0	0.050	0	0.138
Maximum Klerksdorp PPV	0.072	0	0	0	0.01	0.008	0	0.05	0	0.117
Maximum McGarr PPV	0.314	0	0	0	0.083	0.071	0	0.245	0	0.440
Week	40	41	42	43	44	45	46	47	48	49
Cumulative Klerksdorp PPV (week)	0	0	0	0.021	0	0	0.132	0	0	0
Maximum Klerksdorp PPV	0	0	0	0.021	0	0	0.12	0	0	0
Maximum McGarr PPV	0	0	0	0.138	0	0	0.449	0	0	0
Week	50	51	52	53	54	55	56	57	58	59
Cumulative Klerksdorp PPV (week)	0	0	0	0	0	0	0	0	0.028	0
Maximum Klerksdorp PPV	0	0	0	0	0	0	0	0	0.028	0
Maximum McGarr PPV	0	0	0	0	0	0	0	0	0.163	0
Week	60	61	62	63	64	65	66	67	68	69
Cumulative Klerksdorp PPV (week)	0	0.155	0	0.011	0	0.009	0.013	0	0	0
Maximum Klerksdorp PPV	0	0.155	0	0.011	0	0.009	0.013	0	0	0
Maximum McGarr PPV	0	0.533	0	0.084	0	0.077	0.096	0	0	0
Week	70	71	72	73	74	75	76	77	78	79
Cumulative Klerksdorp PPV (week)	0	0.011	0.000	0.026	0.020	0.012	0.025	0.048	0.06	0
Maximum Klerksdorp PPV	0	0.011	0	0.026	0.02	0.012	0.014	0.036	0.026	0
Maximum McGarr PPV	0	0.084	0	0.155	0.131	0.089	0.103	0.196	0.154	0
Week	80	81	82	83	84	85				
Cumulative Klerksdorp PPV (week)	0	0.051	0.035	0.025	0.015	0.011				
Maximum Klerksdorp PPV	0	0.051	0.035	0.014	0.015	0.011				
Maximum McGarr PPV	0	0.247	0.190	0.101	0.105	0.085				

Table 25a: Summary of final closure and dilation measurements in the project site

Section	Closure	Left sidewall							Right sidewall							Combined sidewalls									Hangingwall						
	.	1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	L+R	6+	Total	1m	2m	3m	4m	5m	6m	Total
1	63.3	12	1	5	1	8	9	36	8	1	2	2	1	0	14	20	2	7	3	9	9	50	13	63	48	17	3	5	3	1	77
2	59	6	3	1	8	2	1	21	17	2	1	0	1	1	22	23	5	2	8	3	2	43	16	59	23	21	2	0	1	0	47
3	46.7	2	0	8	5	1	1	17	8	1	3	0	2	1	15	10	1	11	5	3	2	32	15	47	44	20	5	1	2	2	74
4	—	—	—	—	—	—	—	—	—	4	0	1	0	0	2	7	—	—	—	—	—	—	—	—	—	6	5	2	1	1	2	17
5	32.8	6	0	1	12	2	2	23	3	1	1	1	2	1	9	9	1	2	13	4	3	32	1	33	22	0	1	2	1	1	27
6	39.6	8	1	2	3	9	1	24	7	1	1	0	1	1	11	15	2	3	3	10	2	35	5	40	9	0	1	0	3	1	14
7	33.8	6	1	2	7	4	0	20	6	0	1	2	0	2	11	12	1	3	9	4	2	31	3	34	29	1	0	2	1	3	36
8	70.4	39	2	3	2	7	4	57	5	2	2	0	1	1	11	44	4	5	2	8	5	68	2	70	8	2	1	0	0	2	13
9	33	6	4	0	4	1	1	16	9	0	5	1	1	1	17	15	4	5	5	2	2	33	0	33	11	0	0	1	0	1	13
10	78.4	45	5	3	1	0	6	60	8	1	2	1	0	0	12	53	6	5	2	0	6	72	6	78	2	0	0	0	1	4	7

Table 25b: Summary of final closure and dilation measurements in the project site expressed in percentages

Section	Closure	Left sidewall							Right sidewall							Combined sidewalls								Hangingwall						
		1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	Total	1m	2m	3m	4m	5m	6m	6+	Total	1m	2m	3m	4m	5m	6m	Total
1	63.3	33	3	14	3	22	25	100	57	7	14	14	7	0	100	32	3	11	5	14	14	21	100	62	22	4	6	4	1	100
2	59	29	14	5	38	10	5	100	77	9	5	0	5	5	100	39	8	3	14	5	3	27	100	49	45	4	0	2	0	100
3	46.7	12	0	47	29	6	6	100	53	7	20	0	13	7	100	21	2	24	11	6	4	31	100	59	27	7	1	3	3	100
4	—	—	—	—	—	—	—	—	57	0	14	0	0	29	100	—	—	—	—	—	—	—	100	35	29	12	6	6	12	100
5	32.8	26	0	4	52	9	9	100	33	11	11	11	22	11	100	27	3	6	40	12	9	2	100	81	0	4	7	4	4	100
6	39.6	33	4	8	13	38	4	100	64	9	9	0	9	9	100	38	5	8	8	25	5	12	100	64	0	7	0	21	7	100
7	33.8	30	5	10	35	20	0	100	55	0	9	18	0	18	100	36	3	9	27	12	6	8	100	81	3	0	6	3	8	100
8	70.4	68	4	5	4	12	7	100	45	18	18	0	9	9	100	62	6	7	3	11	7	3	100	62	15	8	0	0	15	100
9	33	38	25	0	25	6	6	100	53	0	29	6	6	6	100	45	12	15	15	6	6	0	100	85	0	0	8	0	8	100
10	78.4	75	8	5	2	0	10	100	67	8	17	8	0	0	100	68	8	6	3	0	8	8	100	29	0	0	0	14	57	100

Table 26: Summary of the tunnel condition assessment using the Wiseman (1979) guidelines

		Section 1			Section 2			Section 3			Section 4			Section 5		
		5/93	4/94	8/94	5/93	4/94	8/94	5/93	4/94	8/94	5/93	4/94	8/94	5/93	4/94	8/94
Rock type		2	2	2	2	2	2	2	2	2	2	2	2	1	1	1
Discontinuities		5	5	5	4	4	4	3	3	3	3	3	3	1	1	1
Strike orientation		2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
Bedding dip		1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
ROCK MASS RATING		10	10	10	9	9	9	8	8	8	8	8	8	5	5	5
Relative F-factor		0.9	0.9	0.9	0.92	0.92	0.92	0.94	0.94	0.94	0.94	0.94	0.94	1	1	1
Fracturing	Left	2	3	4	2	2	3	2	3	3	—	—	—	3	4	4
	Right	2	3	4	2	2	3	3	4	4	2	2	2	2	3	3
	Hang	3	3	4	1	2	3	2	3	3	2	2	2	3	4	4
Movement	Left	2	3	4	1	2	2	1	3	3	—	—	—	2	3	3
	Right	2	2	3	1	2	3	2	3	3	1	1	1	1	2	3
	Hang	2	2	4	1	2	2	1	2	4	1	2	2	2	3	3
Position	Left	3	3	3	3	3	3	3	3	3	—	—	—	2	3	3
	Right	3	3	3	2	3	3	2	3	3	3	3	3	3	3	3
	Hang	2	3	3	3	3	3	3	3	4	2	3	3	2	3	4
DEFORMATION RATING		21	25	32	16	21	25	19	27	30	—	—	—	20	28	30
TOTAL RATING		31	35	42	25	30	34	27	35	38	—	—	—	25	33	35

		Section 6			Section 7			Section 8			Section 9			Section 10		
		5/93	4/94	8/94	5/93	4/94	8/94	5/93	4/94	8/94	5/93	4/94	8/94	5/93	4/94	8/94
Rock type		1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Discontinuities		1	1	1	2	2	2	1	1	1	2	2	2	2	2	2
Strike orientation		2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
Bedding dip		1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
ROCK MASS RATING		5	5	5	6	6	6	5	5	5	6	6	6	6	6	6
Relative F-factor		1	1	1	0.98	0.98	0.98	1	1	1	0.98	0.98	0.98	0.98	0.98	0.98
Fracturing	Left	3	3	3	3	4	4	2	4	4	2	3	3	2	3	3
	Right	3	3	3	2	2	2	2	3	3	2	2	2	1	1	1
	Hang	2	2	3	3	4	4	2	3	3	3	3	3	2	2	2
Movement	Left	2	2	2	2	2	3	2	3	3	2	2	2	2	2	3
	Right	1	2	2	1	1	2	1	2	2	1	1	2	1	1	1
	Hang	2	2	2	3	3	4	2	2	3	2	3	3	1	2	2
Position	Left	2	3	3	2	3	3	3	3	3	2	3	3	3	3	3
	Right	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	Hang	2	3	3	2	3	4	2	3	3	2	3	3	3	3	3
DEFORMATION RATING		20	23	24	21	25	29	19	26	27	19	23	24	18	20	21
TOTAL RATING		25	28	29	27	31	35	24	31	32	25	29	30	24	26	27

F = rock mass condition factor = rock type, discontinuities, bedding, orientation
= reduces with weaker rock and increased discontinuity
= six rock mass classes (ROCK MASS RATING) identified from summation of localized and detailed "rock type, discontinuities (bedding), orientation and strata dip"
= relatively reduced by 2% or 0.02 for diminishing competence of each rock mass class
= [Section, F] = [1, 0.90] ; [2, 0.92] ; [3, 0.94] ; [4, 0.94] ; [5, 1] ; [6, 1] ; [7,0.98] ; 8, 1] ; [9, 0.98] ; [10, 0.98]

FINAL CONDITION RATING

Section	1	2	3	5	6	7	8	9	10
Initial rating	14	5	10	14	13	17	10	12	7
Final rating	44	21	33	33	18	32	27	19	14
Change	30	16	23	19	5	15	17	7	7

Final condition rating = Sum of products of degree of fracturing and degree of movement for each tunnel wall
= (left sidewall fracturing * left sidewall movement) + (right sidewall fracturing * right sidewall movement) + (hangingwall fracturing * hangingwall movement)

Table 27 Summary of borescope observations

HOLE 1 RIGHT		
DATE	23/3/93	27/9/94
DEPTH	WIDTH	WIDTH
220	crushed	
240		crushed
250		2
280		2
285	1	
300		4
370		6
380	5	
390	1	
400		4
410	0 5	1
412	1	
470	1	
480		2
500		4
520		5
550		7
585	0 5	
600		5
694	3	
700		2
730		2
770		1
790		2
795	2	
800		1
810		1
820		2
840		1
880		2
910		1
930		4
1050		4
1130		2
1288	1	
1495	0 5	
1530		1
1578	0.1	EOS
2510	1	
4396	EOS	

HOLE 2 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	WIDTH
100		4
150		6
160	crushed	
290		4
296	1	
300		3
397	0.1	
480	0.1	
700	0.1	
860	0.1	
1064	0.1	
1379	2	
1480	1	
1620	0.1	
1872	0.5	
1880		4
2090	1	
2730		5
2850		1
3568	1	EOS
4329	EOS	

HOLE 3 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	WIDTH
100	crushed	7
120		2
265	1	
270		17
290	3	
300		1
318	2	
320		3
330	4	
380		4
385	2	
420	2	
472	2	
480		1
760	2	
770		2
950		1
1110		1
1650		1
3827	1	
4160	EOF	5
5400		EOS

HOLE 4 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	WIDTH
200		crushed
215	crushed	
220		1
770		1
950		1
980	1	EOS
2144	1	
2270	1	
3310	EOF	

HOLE 5 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	WIDTH
0		EOS
285	crushed	
297	5	
335	1	
405	0.1	
410	0 1	
490	1	
529	1	
1249	1	
3000	EOF	

HOLE 6 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	WIDTH
80		8
100	crushed	
111	1	.
112	4	
130		10
150		2
160		4
190		4
210		4
220	4	
250		4
300	1	1
380	0 2	EOS
495	0 5	
1680	1	
2310	EOF	

HOLE 7 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	DEPTH
290		1
550	2	
560	EOF	2
620		1
700		1
1100		2
2000		EOS

HOLE 8 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	DEPTH
80		5
90		5
100	crushed	
110		2
130		1
300	0 5	1
390		0 5
900		1
960		1
1140		1
1180		1
1320	0.3	
1340	EOF	2
2100		EOS

HOLE 9 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	DEPTH
100		crushed
110	crushed	
140		2
310		1
320		1
330	1	2
350	1 5	1
355	0.5	
375	2	
380	1	
390	1	1
400		1
420	0 5	1
690		1
1230		0 5
1340		0 5
1400		0 5
1730		1
1750	0 5	
1820	0.5	
1840	EOF	1
1970		0 5
2190		1
		EOS

HOLE 10 RIGHT		
	23/3/93	27/9/94
DEPTH	WIDTH	DEPTH
70		1
100		0 5
280	crushed	2
300		2
310		1
330	2	1
350		1
400		1
410		1
490		1
520		0 5
650		1
1200	1	
1240		1
1290	2	
1300		2
2000	2	
2150	EOF	3
		EOS

EOS = end of sight
EOF = end of visible fracturing

Table 28: Summary of inner sidewall visual condition rating

	DATE	DATE	DATE	DATE	RATING	RATING
SECTION	24/05/93	04/10/93	26/04/94	27/09/94	per wall	per section
1L	SD,MS,tl	MD,MS,tl	MD,MS,tl	SD,f,MD,mb,HS,fb	17	
1H	NIL	HS,f	MRD,HS,f	MRD,f,HS,f	14	
1R	SD,tl	SD,tl	SD,SS,tl	SD,SS,tl	15	46
2L	SD,HS,mb	MD,HS,mb	MD,HS,mb	HD,mb,HS,tl	24	
2H	NIL	MRD,f	HRD,f	HRD,f	9	
2R	SD,mb	SD,HS,mb	MRD,SD,HS,mb	MRD,SD,HS,mb	30	63
3L	SD,MS,m	SD,MS,m	SD,MS,m	SD,MS,mb	12	
3H	NIL	MRD,f	HRD,f	HRD,f	9	
3R	SD,fb	SD,SS,fb	SD,SS,fb	MD,MS,f,SD,SS,tl	14	35
4L	———	———	———	———	–	
4H	NIL	SRD,f	SRD,f	SRD,f	7	
4R	SD,tl	SD,tl	SD,SS,tl	SD,SS,tl	15	–
5L	SD,f;MS,b	SD,f;HS,b	SRD,f;HD,HS,mb	SRD,f;HD,HS,mb	25	
5H	NIL	SRD,f	SRD,f	MRD,f	8	
5R	NIL	SD,tl	SD,tl	SD,tl,MRD,f	11	44
6L	SD,SS,tl	SD,SS,tl	SD,SS,tl	SRD,f,SS,SD,tl	22	
6H	NIL	SRD,f	SRD,f	SRD,f	7	
6R	SD,tl	SD,tl	SD,tl	SD,tl	3	32
7L	SD,fm	SD,fm;MS,b	SD,SRD,fm;MRD,MS,b	SD,SRD,fm;MRD,MS,b	29	
7H	NIL	SRD,f	SRD,f	SRD,f	7	
7R	SD,tl	SD,tl;SS,b	SD,SS,tl	SRD,f,SD,SS,tl	22	58
8L	SD,SS,f	MD,HS,f	MD,HS,f;SD,MS,mb	SRD,MD,f,HS,f,SD,MS,mb	29	
8H	NIL	SRD,f	HRD,f	HRD,HS,f	15	
8R	NIL	SD,SS,tl	SD,SS,tl	HRD,f,SD,SS,tl	24	68
9L	SD,tl;SS,mb	MD,f;MS,mb	MD,HS,f;SD,MS,mb	MD,HS,f;SD,MS,mb	20	
9H	NIL	NIL	SRD,f	SRD,f	7	
9R	SD,tl	SD,tl	SD,SS,tl	SRD,f,SD,SS,tl	22	49
10L	SD,SS,tl	MD,SS,tl	MD,SS,tl	SRD,f,MD,SS,tl	25	
10H	NIL	NIL	SRD,f	SRD,f	7	
10R	SD,tl	SD,tl	SD,SS,tl	SRD,f,SD,SS,tl	22	54

LEGEND		Weighting	
	SD = Slight dogearing	1	
	MD = Moderate dogearing	2	
	HD = Heavy dogearing	3	
	SS = Slight spalling	4	
	MS = Moderate spalling	5	
	HS = Heavy spalling	6	
	SRD = Slight relative displacement = <25% shear across borehole	7	
	MRD = Moderate relative displacement =25 to 50% shear across borehole	8	
	HRD = Heavy relative displacement = 50 to 75% shear across borehole	9	
	f = Front	1	fm=2
	m = Middle	1	mb=2
	b = Back	1	fb=2
	tl = Total length	3	

Example: Rating per wall, say 10R = condition * distance = SRD,f,SD,SS,tl = (7*1)+(1*3)+(4*3) = 22
Rating per section, say Section 10 = 10L+10H+10R = 25+7+22 = 54

Rating criteria: Shearing is worse than spalling, which is worse than dogearing, when considering these effects on tunnel and support stability. Similarly, influence over the borehole total length is worse than for example the front and middle, which is worse than just the front.

Table 29a: Effect of stress increase versus decrease on the sidewall dilation

Section	Stress graph	Period (weeks)	Stress change (MPa)	Dilation (mm)	Stress rate (MPa/week)	Dilation rate (mm/week)	Dilation to stress rate (mm/MPa)	Ratio of stress increase to decrease rate	Dilation ratio for increasing to decreasing stress	Ratio of dilation-to-stress rate for increasing versus decreasing stress	Comment
1	increase	7	2	4.8	0.29	0.69	2.40				
	decrease	78	[illegible]	58.8	0.31	0.76	2.44	0.93	0.81	0.98	Rate of dilation for increasing vs decreasing stress is similar
2	increase	7	4	4.8	0.57	0.70	1.23				
	decrease	78	[illegible]	54.1	0.29	0.69	2.36	1.94	1.01	0.52	Rate of dilation for increasing stress is 0.52 that for decreasing stress, ie much lower
3	increase	7	4	4	0.57	0.57	1.00				
	decrease	78	[illegible]	42.7	0.26	0.55	2.14	2.23	1.04	0.47	Rate of dilation for increasing stress is 0.47 that for decreasing stress, ie much lower
5	increase	20	[illegible]	4.3	0.50	0.22	0.43				
	decrease	65	[illegible]	28.6	0.23	0.44	1.90	2.17	0.49	0.23	Rate of dilation for increasing stress is 0.23 that of decreasing stress, ie very much lower
6	increase	20	[illegible]	9.2	0.55	0.46	0.84				
	decrease	46	[illegible]	30.4	0.26	0.66	2.53	2.11	0.70	0.33	Rate of dilation for increasing stress is 0.33 that of decreasing stress, ie much lower
7	increase	20	[illegible]	3.8	0.50	0.19	0.38				
	decrease	65	9	30	0.14	0.46	3.33	3.61	0.41	0.11	Rate of dilation for increasing stress is 0.11 that of decreasing stress, ie very much lower
8	increase	33	[illegible]	47.1	0.36	1.43	3.93				
	decrease	52	7	23.3	0.13	0.45	3.33	2.70	3.19	1.18	Rate of dilation for increasing stress is 1.18 that of decreasing stress, ie slightly higher
9	increase	33	[illegible]	14.6	0.36	0.44	1.21				
	decrease	52	3	17.5	0.06	0.34	5.83	6.30	1.31	0.21	Rate of dilation for increasing stress is 0.21 that of decreasing stress, ie very much lower
10	increase	46	[illegible]	64.5	0.30	1.40	4.61				
	decrease	39	2	13.9	0.06	0.36	6.95	5.93	3.93	0.66	Rate of dilation for increasing stress is 0.66 that of decreasing stress, ie much lower

Table 29b: Effect of stress increase versus decrease on the hangingwall dilation

Section	Stress graph	Period (weeks)	Stress change (MPa)	Dilation (mm)	Stress rate (MPa/week)	Dilation rate (mm/week)	Dilation to stress rate (mm/MPa)	Ratio of stress increase to decrease rate	Dilation ratio for increasing to decreasing stress	Ratio of dilation-to-stress rate for increasing versus decreasing stress	Comment
1	increase	7	2	2	0.29	0.29	1.00				
	decrease	78	[illegible]	75	0.31	0.96	3.13	0.93	0.30	0.32	Rate of dilation for increasing stress is 0.32 that for decreasing stress, ie much lower
2	increase	7	4	2	0.57	0.29	0.50				
	decrease	78	[illegible]	46	0.29	0.59	1.96	1.94	0.50	0.26	Rate of dilation for increasing stress is 0.26 that for decreasing stress, ie much lower
3	increase	7	4	3	0.57	0.43	0.75				
	decrease	78	[illegible]	71	0.26	0.91	3.55	2.23	0.47	0.21	Rate of dilation for increasing stress is 0.21 that for decreasing stress, ie much lower
4	increase	20	6	7	0.30	0.35	1.17				
	decrease	65	[illegible]	10	0.23	0.15	0.67	1.30	2.28	1.75	Rate of dilation for increasing stress is 1.75 times that for decreasing stress, ie much higher
5	increase	20	[illegible]	6	0.50	0.30	0.60				
	decrease	65	[illegible]	21	0.23	0.32	1.40	2.17	0.93	0.43	Rate of dilation for increasing stress is 0.43 that of decreasing stress, ie much lower
6	increase	20	[illegible]	2	0.55	0.10	0.18				
	decrease	46	[illegible]	12	0.26	0.26	1.00	2.11	0.38	0.18	Rate of dilation for increasing stress is 0.18 that of decreasing stress, ie very much lower
7	increase	20	[illegible]	6	0.50	0.25	0.50				
	decrease	65	9	31	0.14	0.48	3.44	3.61	0.52	0.15	Rate of dilation for increasing stress is 0.15 that of decreasing stress, ie very much lower
8	increase	33	[illegible]	3	0.36	0.09	0.25				
	decrease	52	7	10	0.13	0.19	1.43	2.70	0.47	0.17	Rate of dilation for increasing stress is 0.17 that of decreasing stress, ie very much lower
9	increase	33	[illegible]	4	0.36	0.12	0.33				
	decrease	52	3	9	0.06	0.17	3.00	6.30	0.70	0.11	Rate of dilation for increasing stress is 0.11 that of decreasing stress, ie very much lower
10	increase	46	[illegible]	3	0.30	0.07	0.21				
	decrease	39	2	7	0.06	0.18	3.00	5.93	0.36	0.06	Rate of dilation for increasing stress is 0.06 that of decreasing stress, ie very much lower