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MONITORING AND MANAGEMENT OF A LARGE OPEN PIT FAILURE

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A research report submitted to the Faculty of Engineering and the Built Environment, University of Witwatersrand, Johannesburg, in partial fulfillment of the requirements of the degree of Master of Science in Engineering.

Perth, 2009

DECLARATION

I declare that this research report is my own, unaided work. It is being submitted for the Degree of Master of Science in the University of Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University

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November 2009

ABSTRACT

A large scale slope instability developed at an operating mine over two years, resulting in a 4.5 million tonne collapse in July 2004.

During this period the Geotechnical personnel monitored and inspected the slope to ensure that the safety of personnel and equipment was not compromised. Monitoring of the slopes was done using visual inspections, conventional survey methods and the use of the Slope Stability Radar. The details of the observations and the monitoring results are described in this project, as well as the methods used to try to predict the onset of failure. The Slope Strain method of predicting failure is evaluated.

An important part of the management of a failure is the control measures that are put in place. The control measures, and how they are escalated in reaction to an increasing risk, are discussed.

Certain trigger levels were put in place. Due to location of mining at time of collapse the evacuation of personnel based on the trigger levels was not required. The effectiveness of the different trigger levels is evaluated.

All slope deformation and slope failures behave differently. When no site specific historical data is available, the geotechnical practitioner relies on available literature to formulate guidelines and threshold levels for the monitoring, prediction of failure, and safe management of unstable slopes. The detailed case study described in this report is considered to be a valuable contribution to the literature in these fields. The data contained in the report have been presented in detail since they may be of value to other researchers and practitioners in the rock slope field.

This research is dedicated to Marlize, Marizanne and Stephanie - and the time we spent in Zambia

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1 INTRODUCTION

All natural and man-made slopes can be expected to record deformation, mainly due to the effect of time and stress. Man-made slopes are normally associated with a higher exposure of people near the slope, be it a road excavation or an open pit slope. The consequences of failure of a man-made slope can therefore be very high due to the possible injury to personnel as a result of the failure. In an open pit environment a failure can also have detrimental economic consequences. It is therefore important to be able to monitor the displacement of slopes and give advance warning of instability.

The time dependant behaviour of slopes needs to be understood before a slope can be monitored effectively. Various authors have developed models to describe this behaviour. There are also a number of techniques and systems available to monitor slope displacement. These displacement models and monitoring techniques will be used to describe the slope displacement recorded in an open pit mine.

The purpose of slope monitoring is to determine when the displacement will result in a failure that could pose a risk to men and equipment. Models that are used to predict failures will be described, as well as the methods used in the case study.

In an open pit environment where instability has been identified, this instability has to be managed properly to ensure the safety of personnel, while the maximum amount of ore is extracted before the slope collapse. A sound understanding of the behaviour of slopes, the failure mechanism causing the instability, adequate monitoring systems that produce reliable data and an understanding of failure prediction methods are key aspects that will contribute towards a well managed instability.

The purpose of this project is to describe the slope displacement model, monitoring techniques used and prediction and management of a large open pit slope instability.

In spite of the various displacement models, failure prediction techniques and monitoring systems, it is proposed that there is no 'one size fits all' approach to slope monitoring. Monitoring systems, threshold criteria and operational controls have to be reviewed on an ongoing basis, as new information becomes available.

1.1 Definition Of A Failure

The term "failure" can be used loosely and needs to be properly defined. Several authors make the distinction between a slope that has "failed" and a slope that has "collapsed" (Sullivan 1993; Zavodni 2001, Call 2001, Mercer 2006 and Sullivan 2007). Zavodni (2001) explained that the technical definition of a slope failure is when the driving stress exceeds the resisting stress and yielding movements develop. Call (1982) in Zavodni (2001) made the distinction between a theoretical and an operational failure. Theoretical failure is displacement beyond recoverable strain, if the rock is considered to be an elastic material. Operational failure is defined when "the rate of displacement is greater than the rate at which the slide material can be mined safely and economically, or the movement produces unacceptable damage to a permanent facility".

Mercer (2006) defined the terms "collapse", "functional failure" and "instability" as follows:

"Collapse" is defined as the complete overall loss of rock mass integrity and structure.

"Functional failure" is defined as the situation where a slope cannot perform the function for which it was intended. This implies that it does not necessarily involve overall collapse although localised sections of the structure may have collapsed. Examples would include haul roads and ramps.

"Instability" is defined as any other deformational movement or behaviour that does not involve collapse and/or functional failure.

The author has experienced a creep type "failure" of a low angle slope, where ore was mined at the bottom of the slope as the failed material slid down the slope. This could be done safely since the failure mechanism was well understood. The movement was continuously monitored based on a well developed Failure Management Plan that identified displacement trigger levels and associated responses. The equipment used and the geometry (available space at the pit bottom) were additional factors that were considered. This led to a distinction, at the specific mining operation, between "failure" and "collapse". Failure in this case would be similar to Call's (1982) definition of theoretical failure, while collapse would indicate operational failure. In this research this distinction between failure and collapse will be used. The term "instability" would refer to a situation where the time/ deformation curves would indicate increasing slope movement, but failure and/or collapse have not taken place.

1.2 Time Dependant Slope Movement

Sullivan (1993) suggested that basic structural geological concepts can be used to understand slope behaviour. He referred to the four time dependant strain effects on rocks that Spencer (1988) identified:

1) elastic and/or plastic creep
2) transient creep
3) steady state creep
4) accelerated strain prior to failure.

Based on these concepts, Sullivan (1993) divided horizontal movement of pit slopes into four stages, each with some approximate scales of movement :

1) elastic movements (mm in shallow or hard rocks; mm to m scale in deep and/or soil/soft rock)
2) creep movements (10s to 100s mm)
3) cracking and dislocation (0.2m to several m)
4) collapse (greater than 0.5m)

Sullivan (1993) expanded this idea and developed "ground reaction curves", where horizontal movement is plotted against depth (see Figure 1). Curve 1 represents a wedge or planar failure where the failure plane was exposed as the pit is deepened. The slope movement described by the author in this case study can be defined by curve 2, where a complex failure was triggered as the mine deepened. The movement resulted in a failure (collapse of the slope). Pure elastic movement is described by curve 4.

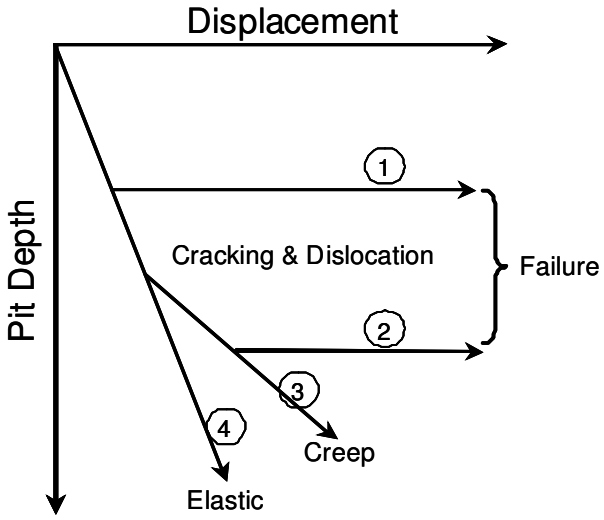


Figure 1 Ground Reaction Curves (after Sullivan 1993)

The displacement related to each stage depends on the type of rock mass that makes up the slope and the slope height. Elastic and creep movements normally do not affect mining operations since the movements are very small and/or the displacement rate very low. Operations are not necessarily affected during the "cracking and dislocation" stage, but some impact can be expected during the collapse stage.

Martin (1993) developed models to describe the time dependant behaviour of slopes based on his investigation of the deformation data and failure mechanisms of 6 case studies and a literature review. He defined three phases of deformation, i.e. (see Figure 2):

- Phase I : Initial response
- Phase II : Strain hardening
- Phase III : Progressive failure

According to Martin (1993) the slope deformation recorded during the initial response are adjustments in response to the removal of material from the slope. Although failure is not expected during this phase, displacements varying from a few centimetres to more than one meter can be recorded. The movement rates in this period decreases from an initial rate of several millimetres per day to almost no movement. Martin (1993) described the decrease in movement rate by means of a negative exponential relationship. He further acknowledged that a number of external factors affected the behaviour of slopes by assigning two constants in the equation which were a function of the rock mass quality, slope geometry mining rate and failure mechanism.

Martin (1993) suggested that a "locking up" of rock mass due to dilation occured during the strain hardening phase and that available shear strength on discontinuities or within the rock mass was mobilised. This results in increased stability. This phase can be identified by the short period of increased displacement rates in Figure 2, after which the movement decreases. According to the model the "Initial Response" and "Strain Hardening" phases are both part of a regressive behaviour phase of the slope. Several phases of strain hardening can be experienced by a slope, normally in response to mining. Martin (1993) described a number of events on the North Wall of the Palabora open pit between 1984 and 1989 that resulted in strain hardening.

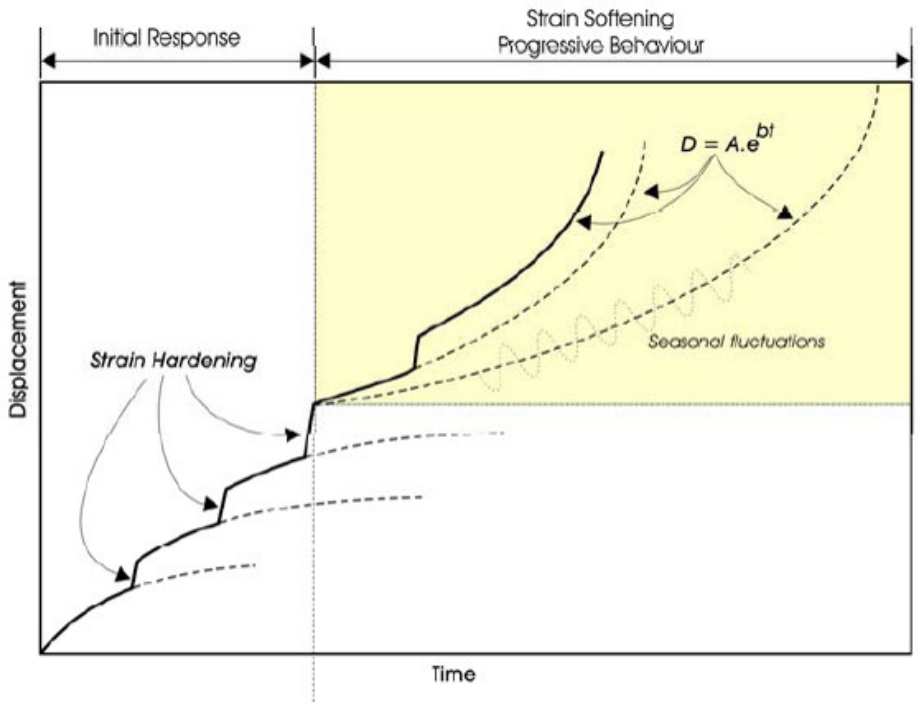


Figure 2 Martin's (1993) model for time dependant deformation behaviour (After Mercer 2006)

Martin (1993) contributed the "Progressive failure period" as a response to ongoing deformation experienced by the slope. The displacement keeps increasing during this period, with an associated increase in the movement rates. Strain softening can occur as a result of the decrease of shear strength due to the increased displacement. Without external forces the displacement will take place at a constant rate. These external events (e.g. blasting, rainfall) can lead to acceleration and eventually to a slope collapse.

Zavodni (2001) identified an "Initial Response" to mining as a result of elastic rebound, relaxation and/or dilation of the rock mass. During this phase no failure or defined failure surface is developed. Total displacement during this phase can vary from 150mm (competent rock) to 500mm (highly fractured, altered rock), as reported by Martin (1993) in Zavodni (2001), while movement rates between 0.1mm/day to 4mm/day can be expected. Total movement and movement rates depend on the rock mass quality. The movement recorded during this initial response should not affect

the slope stability. According to Sullivan (2007) this initial response phase includes the elastic movement and part of the creep movement he described in 1993.

The advanced stages of slope movement (cracking and dislocation and failure) described by Sullivan (1993) can be related to the "Regressive" and "Progressive" phases of time dependant movement of slopes, as proposed by Broadbent and Zavodni (1982) and Zavodni (2001), which are defined as follows (Figure 3):

"A regressive failure is one that shows short-term decelerating displacements cycles if disturbing events external to the rock are removed from the slope environment. A progressive failure, on the other hand, is one that will displace at an accelerating rate, to the point of collapse unless active and effective control measures are taken."

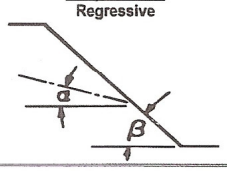
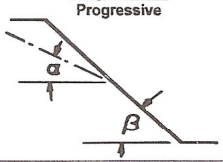
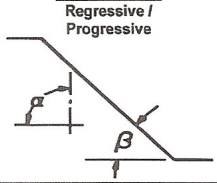
STRUCTURE ATTITUDE		PRIMARY STRUCTURE	
α = Mean Structure Dip or Plunge β = Slope Angle ϕ = Angle of Friction		SIMPLE CONTROL One or Two Surfaces	COMPLEX CONTROL Multiple Structures
Type I Regressive 	$\alpha < \beta$ $\alpha < \phi$	MINERAL CREEK Structural Control – Single Fault with Gouge Weight – 11 Million T Height – 50 M Mean Structure Dip Approx. 10° Overall Slope (β) Variable External Stimuli – Blasting, Water Unique Feature – Predictable & Regular Response to Blasting	Uncommon Type
Type II Progressive 	$\alpha < \beta$ $\alpha > \phi$	LIBERTY PIT Structural Control – 2 Intersecting Faults Weight – 7 Million T Height – 175 M Overall Slope (β) - 33° Mean Plunge (α) - 19° External Stimuli - ? Unique Feature – Regularity of Fault Surfaces & Attitudes	KIMBLEY PIT Structural Control – Single Joint System Weight – 1.1 Million T Height – 160 M Overall Slope (β) - 58° (Initial) Mean Structure Dip (α) - 65° External Stimuli - None Unique Feature – Induce Failure by Explosive Undercut, (β) 69° (Final)
Type III Regressive / Progressive 	$\alpha \leq \beta$ $\alpha \leq \phi$	BINGHAM PIT C+ BEDS Structural Control – Prominent Faults Dip into Slope Weight – 20 Million T Height – 260 M Overall Slope (β) - 39° Unique Feature – Extreme Sensitivity to Groundwater Levels – Low Rock Mass Strength	CHUQUICAMATA Structural Control – Multiple Fault Joints Weight – ? (Est. 1 Million T) Height – 151 M Overall Slope (β) - 42° Mean Structure Dip (α) - Various External Stimuli – Heavy Pit Blasting, Frequent Earthquakes, Pore Pressure Unique Feature – Rubble Confined Above Pit Bottom

Figure 3 Rock failure types based on structure / slope characteristics (after Broadbent and Zavodni 1982)

The original work by Broadbent and Zavodni (1982) was based on observations of several well documented failures that were observed. The stages were related to failure geometry, as depicted in Figure 3. Sullivan (1993) suggested another phase, namely a stick-slip type, which is characterised by sudden movements followed by periods of little or no movement. This is similar to the cyclical behaviour described by Martin (1993). The movement in the "slip" period is triggered by a specific event, for example rainfall or blasting. Martin (1993) contributed the "stick" period of the displacement to possible "strain hardening". The displacement curves showing the different stages are illustrated in Figure 4. In the author's experience is was difficult to distinguish between the regressive period of the Type 3 and the Type 4 (stick-slip) in Figure 4.

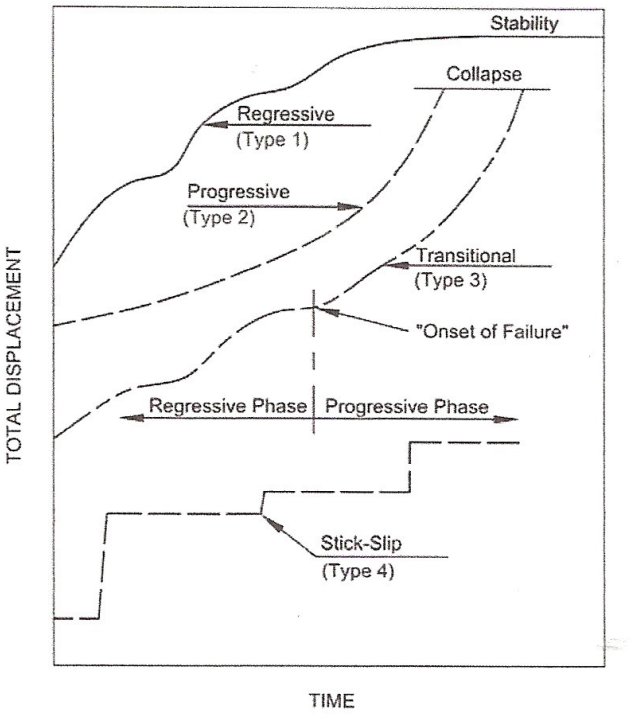


Figure 4 Typical time displacement curves (after Broadbent and Zavodni, 1982; and Sullivan, 1993)

The cycles of Type 1 regressive movement are experienced when the driving force temporarily exceeds the resisting force. The slope behaves regressively when the

movement decelerates before the next trigger event. These triggers are usually external forces such as blasting, rainfall, change in groundwater pressure or mining related (removal of buttress). According to Zavodni (2001) this type of slope movement normally becomes more stable as the ratio of driving force and resisting force decreases.

When a slope is recording increasing displacement rates, the challenge for the practitioner is to decide whether this period of increased rates is one of the cycles in the regressive phase, or whether it is the onset of a progressive phase. An increase in the movement rate, i.e. acceleration, is normally an indication of a progressive movement phase.

When the dip of the structure is steeper than the effective structure strength, Type 2 progressive slope movement (refer to Figure 3 and Figure 4) can be expected. Many large failures have started as regressive failures but developed into progressive failures, resulting in the Type 3 curve in Figure 4. Zavodni (2001) suggests that these types of failures need extensive monitoring and attention, since Type 1 conditions are normally controlled and Type 2 fails soon after exposure.

Sullivan (2007) highlighted some perceived shortcomings in the existing models as described above. The two most important criticisms were the fact that the current models did not indicate the large time periods over which acceleration is taking place. The other was the fact that the model implied that slopes record large displacements over long periods prior to failure. Because the focus of practitioners is the prediction and timing of failures, not enough emphasis is placed on the early stages of movement of the slope, i.e. the period prior to cracking and dislocation. Quite often there are more opportunities and scope for remedial measures during this period. Based on experience with recent failures, Sullivan (2007) also recommends the inclusion of post failure slope behaviour as an additional stage of slope deformation.

Sullivan (2007) expanded the models proposed by Broadbent and Zavodni (1982), Sullivan (1993) and Zavodni (2001) by dividing pit slope movement models into three periods, namely pre-failure movements, failure movements and post failure movements. The advantage of the updated classification system is that it allows for more detailed descriptions for the different periods of slope movement. The modified classification system is shown in Figure 5

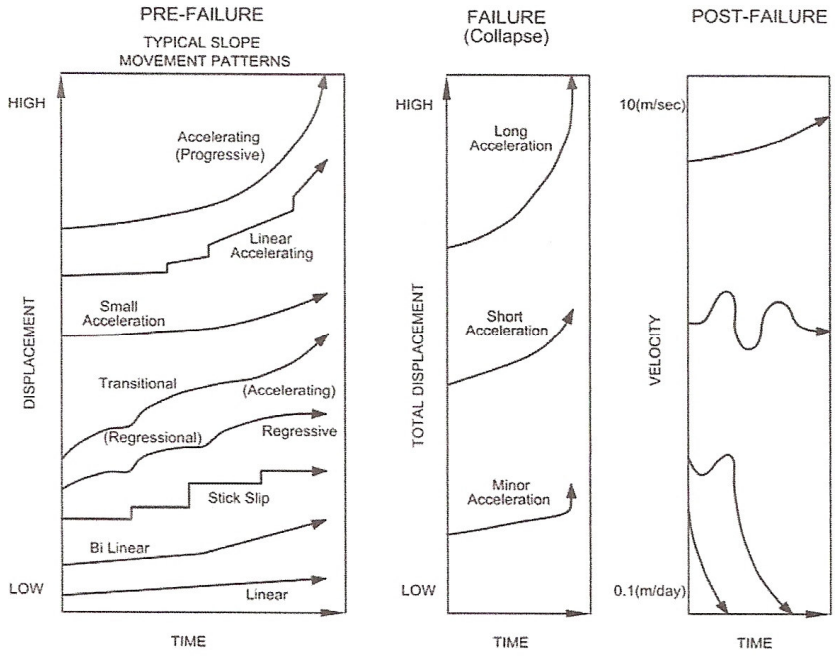


Figure 5 Modified pit slope failure classification system (after Sullivan, 2007)

Mercer (2006) reported on the outcome of a research project in which a total of 42 case studies and 83 failure events were reviewed. He concluded that no two failures are similar due to the fact that most failures are structurally controlled. Mercer went on to explain that the time scale involved with structurally controlled failure is normally within the life span of a pit slope. It is thus important to understand the time dependant behaviour of such a slope. The studies indicated that the peak and residual strengths of discontinuities are also time related.

From the case studies Mercer (2006) concluded that specific events triggered increases in deformation rates in slopes. In a mining environment these triggers are normally blasts or rain events. The author has experience of a number of collapses, of varying sizes, that were triggered by such events. Mercer (2006) referred to the rapid increase in the deformation rate following an event, as the initial response. This initial response is followed up by a quick reduction (decay) in movement rates and then a long period of slowly reducing steady state creep. Close to the collapse of the slope the behaviour of the decay cycle changes. The decay period gets longer (see Figure 7) and the slope behaviour eventually reaches a point where the deformation rate increases exponentially, until the slope finally collapses.

Mercer (2007) utilised the similarities in the rock mass deformation behaviour of the various case studies to build a model of deformation behaviour. He proposed that time and event dependant deformation can be grouped into 5 distinct stages of deformation. The following stages are based on deformation behaviour in terms of horizontal and vertical displacement and displacement rates:

- Stage 1 and 2 : Pre-collapse, primary and secondary rock mass creep: Mercer (2007) found that specific events can result in a sudden increase in deformation rate. In a mining environment these are normally rain or blast events. This peak in movement rate is followed by a period of decay (reduction) of the deformation rate until it eventually reduces to a steady state creep. During the initial stages (Stage 1) this recovery period is relatively short and steady state creep deformation is reached. During Stage 2, this recovery is slower and steady state creep is not reached. The deformation rates of the successive transient creep periods are also higher during Stage 2. Both Stages 1 and 2 are characterised by regressive behaviour.
- Stage 3 : Post onset of failure (OOF) to collapse. During this period the slope is experiencing continuous acceleration in the magnitude of deformation and thus increases in the movement rate, up to the point of collapse. During this stage the slope movement is progressive.

- Stage 4: Post-collapse behaviour. This is the period after collapse has taken place and before mining or recovery of the failure commences. The deformation during this period can be complex and Mercer (2007) identified six different modes of behaviour during this time.
- Stage 5 : Post mining/Recovery behaviour mode. The non-failed rock mass of the slope stabilises and the deformation modes recover.

The models proposed by Mercer (2006) is illustrated in Figure 6 and Figure 7.

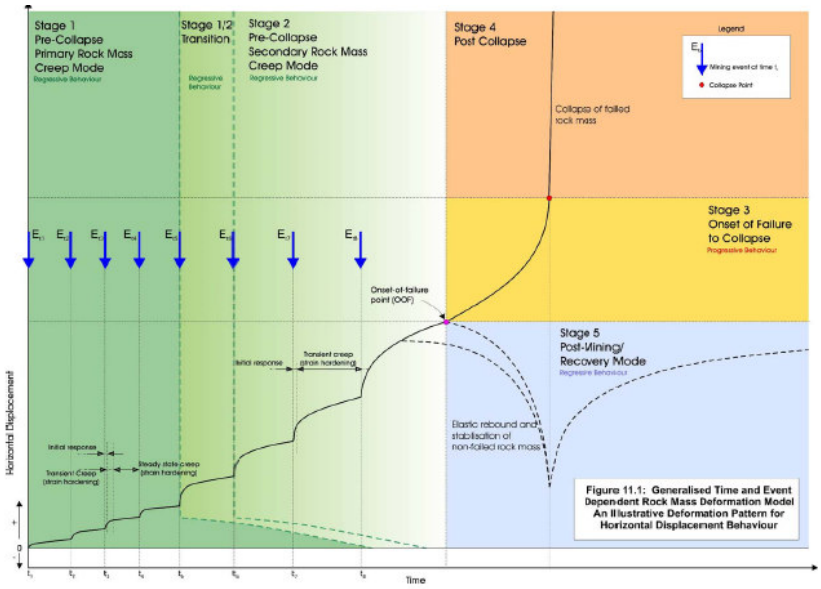


Figure 6 Generalised Time and Event Dependent Rock Mass Deformation Model. An Illustrative Deformation Pattern for Horizontal Displacement Behaviour (After Mercer 2006).

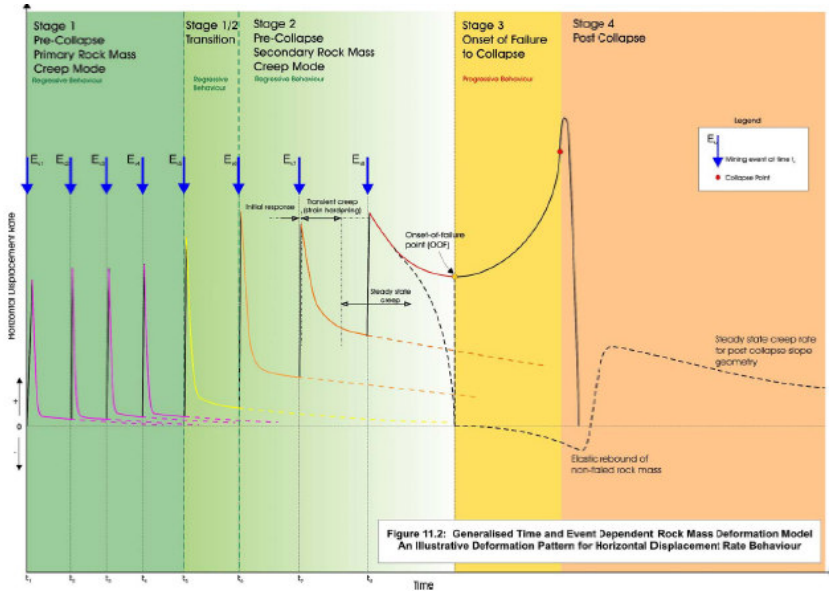


Figure 7 Generalised Time and Event Dependent Rock Mass Deformation Model. An Illustrative Deformation Pattern for Horizontal Rate Behaviour (After Mercer 2006).

Although all the proposed models have evolved and become more complex as more information became available and the behaviours were better understood, the following are common points in all the models:

- An initial response period during which slope movement is elastic and, with no further changes, no failure is expected and movement rates will eventually reduce to zero.
- Slopes can perform progressively (increased movement rates) or regressively, or in some, a combination of the two.
- In a mining environment movement is mostly triggered by external events such as blasting or rainfall.
- No two failures behave the same since the behaviour is a function of the rock mass conditions and structures.

1.3 Monitoring Methods

Many authors have noted that slope failures do not come without warning:

"… if a landslide comes as a surprise to the eye-witnesses, it would be more accurate to say that the observers failed to detect the phenomena which preceded the slide." Terzaghi (1950).
"…… slopes seldom fail without giving adequate warning". (Hoek and Bray, 1974).

A reliable monitoring system needs to be in place to record and identify slope deformation. Although the most obvious purpose of a monitoring system is safety related, slope deformation monitoring also enhances the understanding of slope behaviour and assists in the improving of designs (Hoek, Rippere et al 2001). The back analysis of slope deformations and mimicking of the results with numerical modelling can result in steeper slopes, with its associated economic implications, without compromising safety. (Du Plessis and Martin, 1991).

According to Hartman et al (1992) the objectives of pit slope monitoring should be :

1. For the safety of personnel and protection of equipment;
2. To provide early warning of an instability to allow plans to be modified to mitigate the effect of a failure.
3. To provide geotechnical information to assist with the understanding of the failure mechanism, to design remedial measures and to improve future designs. Stacey (2007) confirms the importance of monitoring in the design process.

Monitoring in an open pit should not only concentrate on surface movement, but systems should also be installed to monitor sub-surface movement. The timely collection and interpretation of the data, followed by distribution of the results, forms the complete slope monitoring system. There are numerous case studies available and various authors that have described monitoring systems for open pits.

The details of the monitoring systems described below are a compilation of descriptions by Little 2007, Jooste and Cawood (2007), Kayesa (2007), Stewart et al (2001), Flores and Karzulovic (2001) and Cahill and Lee (2006).

1.3.1 Visual inspection

Visual inspections are the first "line of defense" of the slope monitoring system. It is done by walking and physical inspection of all berms, haulroad and pit perimeters. During these inspections any new tension cracks, rock falls, slumping, heaving or other indication of instability should be noted. Photographs are a handy way of keeping notes and to monitor the condition of a specific area over time.

Inspections should be part of the routine tasks of the geotechnical section. The frequency of inspection depends on the conditions and risk of the area. Increased incidents of rock falls, a high rainfall event, new cracks or operations close to a high wall can be triggers for an increased frequency of inspections. Operators and pit supervisory personnel should receive some basic geotechnical training to increase their awareness of geotechnical matters. These extra eyes can assist in identifying possible indications of instability around the pit.

1.3.2 Crack monitors / surface extensometers.

One of the most basic forms of equipment is a crack monitor. This can be in the form of two pegs driven into the ground on either side of a crack, a wireline tripod with alarm system or an off the shelve crack meter system with alarms. One of the advantages of these basic systems is the ease of which measurements can be taken, which means that anybody in the pit can monitor movement. The more sophisticated crack meters can be linked to a telemetry system which can trigger an alarm in a control room or in the geotechnical office.

1.3.3 Survey monitoring

The conventional survey system consists of monitoring targets that were manually surveyed with a total station. However, this system has been replaced by robotic

systems at most mines, where the prisms are surveyed on a regular interval and the information transferred to the survey and/or geotechnical office, where the data is reviewed and analysed.

Conventional manual surveying takes a lot of time and labour, and is prone to human error. The advantages of an automated system are the continuous surveying, which produces more data, and the flexibility to survey high risk areas more frequently. The results are also more readily available than data from conventional surveying. Many of the automated systems also have an alarm system, where trigger levels can be set. When movement exceeds the trigger levels it will send out alarms as emails and text messages to mobile phones.

1.3.4 Radar Technology

The use of radar technology for the monitoring of open pit slopes was implemented in 2002 (GroundProbe, 2005). The technology uses differential interferometry to measure sub-millimetre movement of a slope. This is done by comparing the phases of the radar signals it receives from one scan to the next. Any phase difference that is recorded is converted to a millimetre measurement. The system can scan an entire face (as opposed to the discrete points of a survey system) and scans 24 hours per day, in all weather conditions. Data is transferred to the geotechnical office as it is collected – thus almost real time (Harris et al, 2006). The radar systems have alarm capabilities where an alarm will be activated when trigger levels have been exceeded.

The main disadvantage of the system is that the deformation history is lost when the unit is moved. After a new set-up the deformation starts from zero.

1.4 Prediction of Failures

Slope monitoring results are in most cases plotted as displacement or velocity against time. This data is investigated and analysed to identify a trend or a change in the trend. Once a change in the displacement trend of an operating slope has been

identified, the question that needs to be addressed is whether or not it would lead to "operational failure", and if so, when. The analysis of data can be done in various ways.

The most often used is the time-displacement curve, which works well in the initial stages of a developing instability. One of the classic examples of a failure prediction with time-displacement plots was recorded at Chuquicamata in 1969 (Kennedy and Niermeyer 1970), where a large slope failure was accurately predicted. These plots will also give an indication of whether the slope is experiencing regressive or progressive movement. A shortcoming of displacement versus time plots is the uncertainty of how much of the initial movement has not been recorded, because the operational difficulties related to the installation and maintenance of monitoring stations near active operational areas often results in the loss of the initial displacement record (Zavodni 2001).

The importance of the time-displacement curve is confirmed by Wylie and Munn (1978), but they also point out the importance of a correct scale for the axes. The author has also experienced that the selected scale of the plot can further complicates the interpretation of deformation plots. The same data can indicate an imminent failure or slow creep, depending of the scale of the plot. This is discussed in more detail in Section 2.6.6. Wylie and Munn (1978) continued to discuss the relationship between the frequency of monitoring and the rate of movement and the escalation of monitoring frequency and interpretation techniques as the rate of movement, or risk profile to the operations, changes.

When the displacement trend changes it requires some quantification of the amount of change. Movement rate is one of the more popular methods of quantifying displacement (Sullivan 1993) and is confirmed by Wylie and Munn (1978). Movement rates allow one to compare the displacement in various areas of the mine with each other. It can also be used as a decision making tool in the management of a

failed slope. Certain criteria can be established that will trigger management responses. Flores and Karzulovic (2001), Zavodni (2001) and Naismith and Wessels (2005) have described examples of trigger levels and associated management responses. A summary of the movement thresholds as described by some authors are given in Table 1.

Author	Movement thresholds	Actions Description
Martin (1993)	0.1mm/day (0.004 mm/hr)	Initial rock mass response
	0.2 to 2 mm/day (0.008 to 0.08 mm/hr)	Strain hardening
	10 - 100 mm/day (or more) (0.4 - 4.1mm/hr)	Progressive failure
Flores and Karzulovic (2001)	Less than 10mm/day (0.4mm/hr)	Conditions normal; no indication of instability
	10 - 30mm/day (0.4 to 1.25mm/hr)	More detailed monitoring required Appearance of cracks
	30 - 50mm/day (1.25 - 2.1mm/hr)	Potential for instability (if ongoing for longer than 2 weeks)
	More than 50mm/day (2.1mm/hr)	No mining allowed
Zavodni (2001)	0.1mm/day (0.004 mm/hr)	Initial response
	Less than 17mm/day (0.71 mm/hr)	No failure expected within 24hrs
	Less than 15mm/day (0.63 mm/hr)	No failure expected within 48hrs
	More than 50mm/day (2.1 mm/hr)	Indicates progressive failure (total collapse expected within 48 days)
	More than 100mm/day (4.2 mm/hr)	Clear mining area (Progressive geometry and progressive velocity)
	150mm/day (6.25 mm/hr)	Clear mining area (Regressive geometry)
Naismith and Wessels (2005)	84 mm/day (3.5mm/hr)	Alert : Increase monitoring assessments
	120 mm/day (5 mm/hr)	Alarm : Inform operations
	240 mm/day (10 mm/hr)	Scram : Pit evacuation
Roux, Terbrugge and Badenhorst (2006)	0.1 mm/day (0.004 mm/hr) for 3 days ; downward vertical movement	Red alert
	0.2mm/day (0.008 mm/hr)	Evacuate
	0.5 mm/day (0.02 mm/hr) for 10 days; horizontal movement	Orange alert
	1.0 mm/day (0.04mm/hr) for 3 days; horizontal movement	Red alert
	2.0 mm/day (0.08mm/hr) horizontal movement	Evacuate
Sullivan (2007)	0.1 - 0.25 mm/day (0.004 - 0.01 mm/hr)	Definite movement of slope related to shear of displacement on structures
	0.25 - 0.5 mm/day (0.01 - 0.02 mm/hr)	Likely to fail sometime in future
	1 mm/day (0.04 mm/hr)	High chance of failure
	More than 1.0 mm/day (>0.04 mm/hr)	Pre-failure collapse movements

Table 1 Displacement rate thresholds and related actions and / or descriptions

This range of thresholds is to be expected. Each slope has a unique geometry, rock mass conditions and structural setting. Threshold criteria are therefore very site specific, even slope specific, but the threshold levels listed in Table 1 can be used

(with caution) as guidelines when no site related data is available. In the author's experience thresholds that are based on back analyses of site – specific incidents will provide more reliable results.

Sarunic and Lilly (2006) suggested that the cusums technique can be used to assist to identify the inflection point where the slope deformation trend changes. The cusums technique involves plotting of the cumulative sum of the differences between a constant value and each data point in the sequence. Sarunic and Lilly (2006) explain the method as follows:

Let x_1, x_2, x_3… xn be the series of values measured in sequence.
Select a constant, K. The mean of the data set for which the analysis is being undertaken is often chosen as the value of K so that trends can be tracked relative to the mean (rather than some arbitrary) value.
Subtract K from each value in the sequence and then add the differences in a series of partial sums; that is:

$S_1 = x_1 - K;$

$S_2 = (x_1 - K) + (x_2 - K) = S_1 + (x_2 - K);$ *and*

$S_n = S_{n-1} + (x_n - K) = x_1 + x_2 + x_3 + \ldots + x_n - nK$

The S values represent a cumulative sum series (or cusum) and S is plotted versus position in the sequence.

The use of cusums is used in two case studies where the movement rate is used as the constant K value, since the average of total displacement or movement rates was not meaningless. The K value should be developed on site since all slopes and sites behave differently. As a guideline the value should relate to a threshold value at which a 'change of state' occurs (Sarunic and Lilly 2006).

Various authors (Small and Morgenstern 1991, Brox and Newcomen 2003 and Zavodni 2001) suggest high wall strain as an additional decision making tool to assess slope stability and predict time of failure. Certain threshold strain levels are

suggested to be indicative of the phase of deformation and can be used to evaluate the stability of the slope.

Brox and Newcomen (2003) argued that displacements do not reflect the total strain, caused by the displacement, that a slope has experienced. They define the high wall strain criterion as the total slope height divided by the total displacement experienced by the slope and expressed as a percentage, and suggest that this criterion can be used as an additional method to evaluate slope stability and predict time of failure. They recognised that the amount of strain a slope can accommodate before failure depends on the rock mass quality and type of failure mechanism and use the Bieniawski (1976) Rock Mass Rating (RMR) system to describe the rock mass quality.

A model was developed from the results of various case studies, and threshold strain levels suggested. This model will be used to test the applicability of the North Wall strain values to the levels suggested by Brox and Newcomen (2003).

In this project a failure in a large open pit will be discussed. The history of how the failure developed, the observations during each time period and the associated displacement plots will be discussed. Closer to actual collapse, the monitoring results and effect of operations on the monitoring will be explained. The various monitoring techniques, mining controls and actual collapse will be discussed. The case study will be concluded with a review of the effectiveness of threshold trigger levels and the various monitoring techniques.

2 CASE STUDY

2.1 Introduction

On 16 July 2004, at about 07:10 a failure of approximately 1.8 million m^3 (about 4.5 million tonnes) took place on the north wall of an open pit in Zambia.

This failure was the final product of a developing instability that was first identified when the benches in these areas were exposed in September 2002. The mine Geotechnical team closely monitored the development of the failure and implemented control measures that were improved and adjusted in reaction to observations and changing conditions.

The purpose of this report is to describe in detail the monitoring results that were obtained both by traditional methods and the Slope Stability Radar System (SSR), and to establish critical deformations and deformation rates that can be used to predict the onset of collapse in future slopes. It also describes the actions taken and control measures implemented to ensure safe mining below the unstable area.

The progression of the instability from onset to eventual collapse can be analysed in four distinct temporal phases. These were:-

- Prior to July 2003
- July 2003 to December 2003
- December 2003 to May 2004
- May 2004 to July 2004

2.2 Interpretation of survey results

When the first increase in movement was observed in July 2003, the total displacements versus time graphs were initially used to analyse the data. At that stage the survey intervals were approximately weekly. The total displacement rather than displacement rates curves were used because:

a. A graph displaying the movement rate since the previous survey ("since previous") was too erratic to identify a trend.
b. A graph displaying the movement rate since the implementation of the monitoring prism ("since original") could mask an increase in movement.

By identifying changes in the gradient of the displacement curves, periods with a similar rate would be identified and the rate calculated for that specific period and expressed as mm/day. As slope displacements increased, survey intervals were reduced and it became necessary to analyse the deformation in terms of velocity and acceleration to identify and respond to changes.

It is important to note that all movement rates quoted in this report refer to a rate since the previous survey event (instantaneous rate), i.e. displacement over a time period divided by the time period.

Figure 8 shows the location of the monitoring prisms in the area under discussion. The initial area of instability is defined by the smaller (yellow) shaded zone, while the final failure is indicated by the larger (brown) hatched area. The three vertical (green) lines are section lines 22E, 23E and 24E and are 120m apart. The thicker (red) contour lines are bench elevations and have a 15m-height difference.

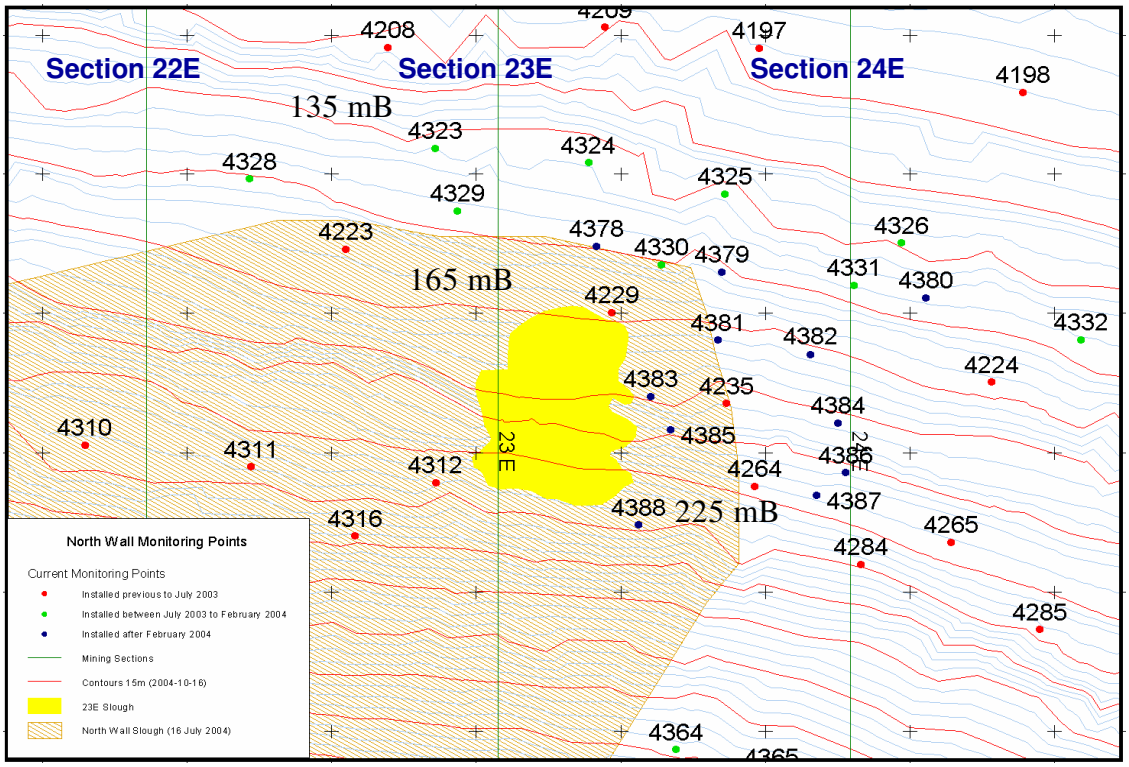


Figure 8 Monitoring prisms installed on North Wall : Red dots - Prior to July 2003; Green dots - between July 2003 and February 2004; Blue dots - After February 2004. Section lines are indicated.

As the movement rates increased, the survey monitoring frequency was also increased. When the surveys were done on a daily basis, the daily rates (i.e. "since previous") were used as a parameter to track and compare movement. With the further increase of the frequency to twice per day, the mm/day unit was still used. However, with the analysis of the data it would seem as if a mm/hour unit would have been a better option in this instance.

With the deployment of the Slope Stability Radar (SSR) the displacement information was received approximately every 15 minutes. Because of this increased frequency, mm/hour was used as the measuring parameter.

2.3 Pre-July 2003

The first indication of instability around the 23E section line area was observed in conjunction with the mining of the 180meter-Bench (mB) in September 2002. Sloughing was recorded on the crest of the 165mB and was attributed to localised, adverse dipping cleavage planes in the Shale With Grit (SWG) formation that had been disturbed as a result of the combination of blasting and relaxation. Near vertical, closely space cleavage planes at an acute angle to the bench face (< 30°) were observed on the crest. At the time of the failure a possible bench scale toppling mode of failure was proposed. This area is shown in Figure 9.

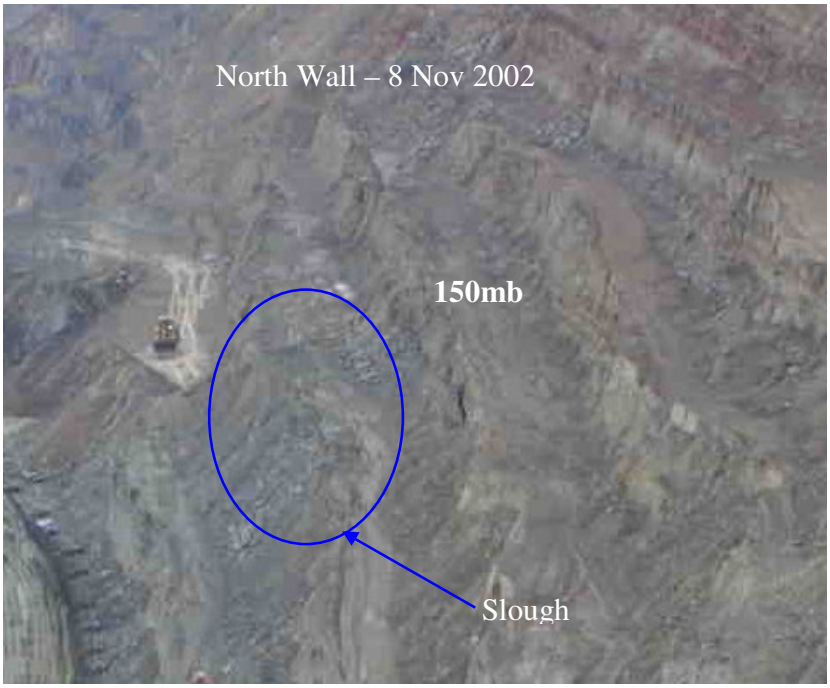


Figure 9 Photo taken on 8 November 2002, showing sloughing on the 165mB.

No further instability was recorded until localised sloughing from the crest of the 165-180mB was recorded at the end of February 2003. During February to May 2003 the failure progressively increased while mining continued. Sloughing was minor and did not interrupt mining operations. The sloughing was interpreted as

slope degradation typically found at the mine due to a combination of weathering and high seasonal rainfall.

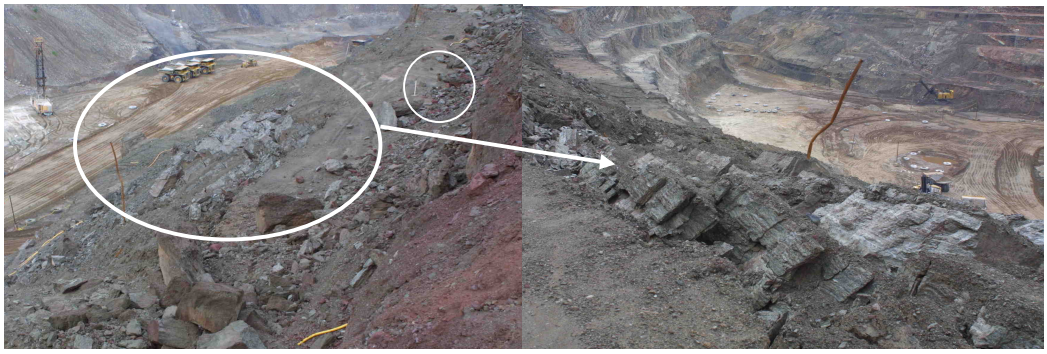


Figure 10 Photographs taken on 27 February 2003, showing the failure on the 165mB crest. Note the position of MP 4229 in the smaller circle.

Monitoring of the slough was carried out through visual inspections, crack monitoring and survey monitoring. Monitoring prisms (MPs) were installed on the 150-165mB benches at the end of 2002. Results of the survey monitoring during this period are shown in Figure 11 and Figure 12. MP 4229 was installed on the 165mB, directly behind the slough and is indicated in Figure 10. This MP will be referred to in the remainder of this report.

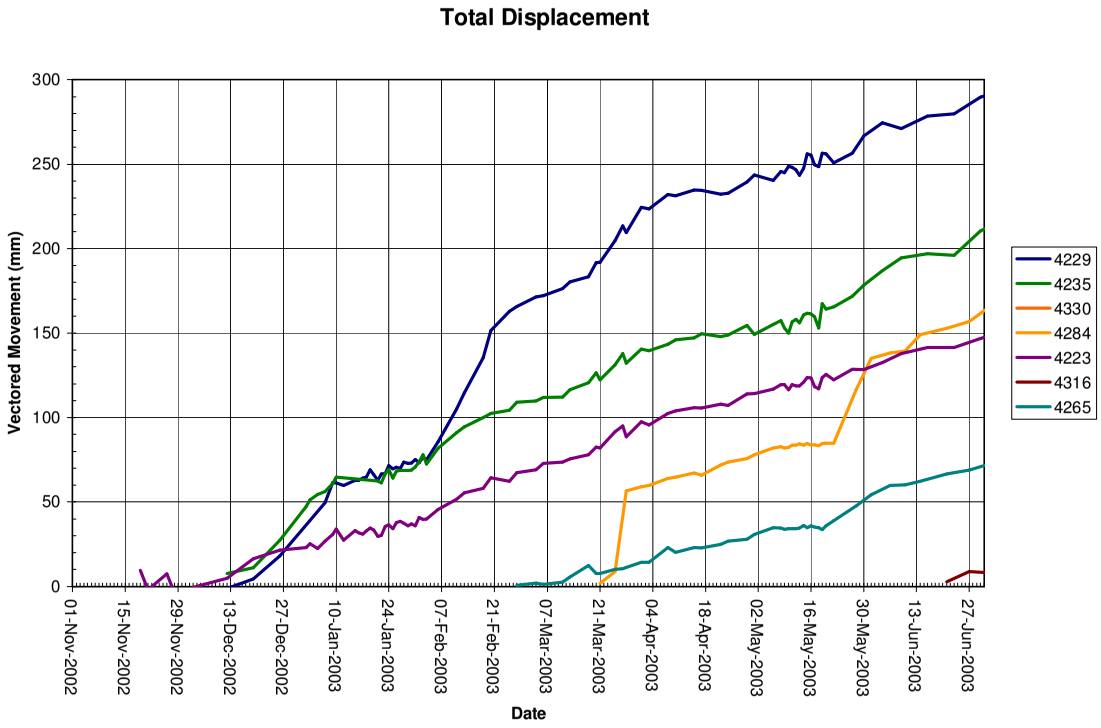


Figure 11 Displacement vs. Time and Movement rate graphs – Pre July 2003

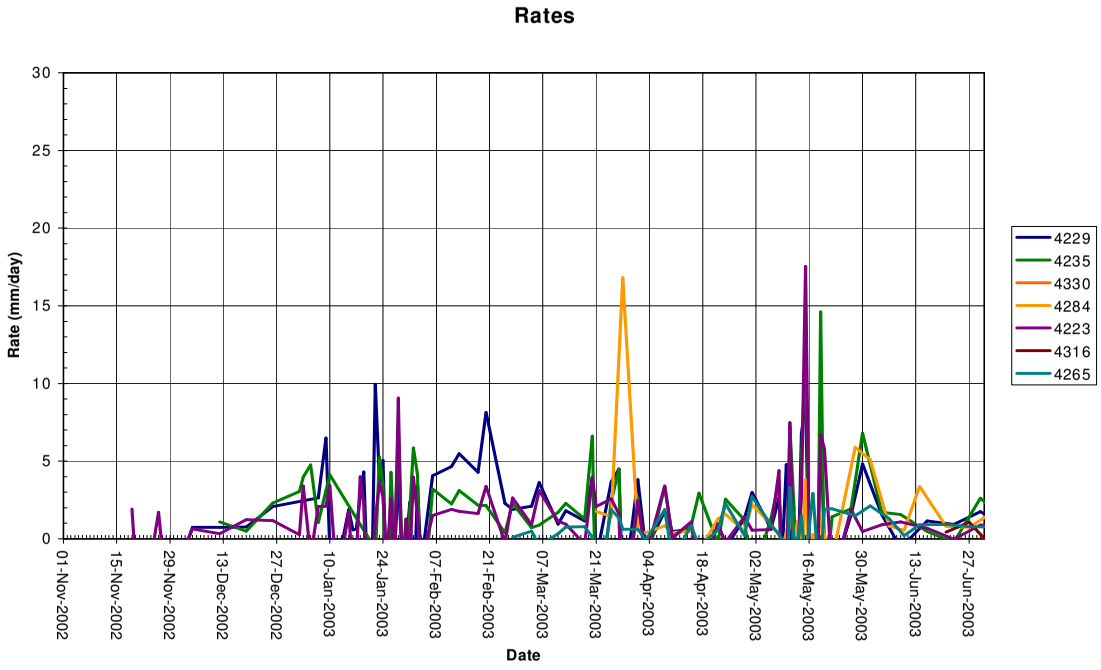


Figure 12 Displacement Rates prior to July 2003

Isolated incidents of increased movements were observed during this period, e.g. MP 4229 (dark blue line in Figure 11) around 20 February 2003. Rates did however recover rapidly to below 2mm/day on average. (Refer to Figure 11. The graphs are also reproduced as Fig A3 and Fig A4 in the appendix to show more detail). A rate of 2mm/day can be considered to be a natural "relaxation" background movement typical of the North Wall of the mine.

2.4 July 2003 – 10 December 2003

In July 2003 an above average increase in the displacement of some monitoring prisms was recorded.

The MPs that recorded the higher rates (MP 4229, 4235, 4284 & 4264) delineated a very specific area between section lines 23E to 24E and from the 150mB and to the bottom of the pit on the 255mB. This area was defined as the Area of Concern (AOC) on the north wall and was based on (a) displacement rates that were higher than those on the rest of the North Wall and (b) increased displacement rates were being observed over an area that indicated instability could be occurring on an inter stack scale rather than just bench scale. See Fig A1 for a photo illustrating the condition of the slope and position of MPs at 5 August 2003.

The increase in movement rates occurred in conjunction with the exposure of a weaker rock unit in the core of the fold on the 270-285mB. Figure 13 shows the simplified topography at that time and its relation to geology.

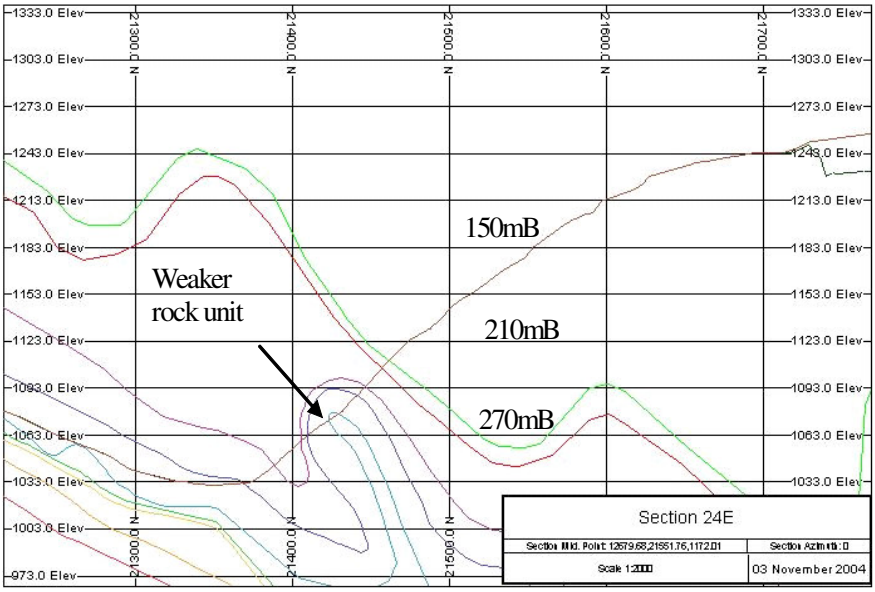


Figure 13 A section through the AOC indicating the geology.

Monitoring prism 4229, which was installed in November 2002, had recorded 295mm of total displacement by the end of June 2003. In discussions with consultants to the mine it was suggested that "failure is unlikely if the total movement in the slopes is less than 500mm…" (Terbrugge 2003). The consultants have been involved with the open pit for more than 29 years and have experienced similar types of failures on the north wall. Three months elapsed between the mining of the 165mB (September 2002) and the installation of the monitoring prism. At an average movement rate of about 12mm/day during this period, the total displacement on the 135mB could have been 483mm, i.e. very close to the perceived critical displacement.

The increase in movement rates was first recorded between 11 and 14 July 2003. The upper lift of the 285mB in the fold zone was exposed at the beginning of July 2003, while the bottom lift of this bench was mined on about 18 July immediately below the area of concern (Figure 14). At that time (approximately 17-18 July) another increase in rates was recorded.

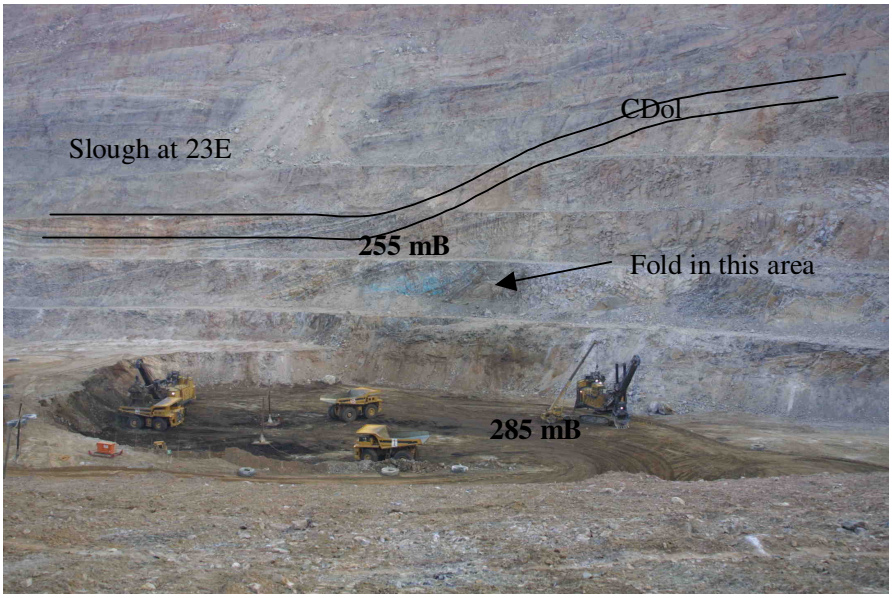


Figure 14 Photo taken on 18 July 2003. Shovel and trucks are on 285mB.

There is no obvious causal link between the elevation at which mining was taking place and the elevation at which movement was occurring. At the time it was thought that the movement was caused because of displacement on the vertical joint planes of the fold, which reduced the confinement of a band of weak talc dolomite, thus causing the upper benches to record movement.

2.4.1 Survey Results

During July and August 2003 the movement rates of individual prisms (MPs 4229, 4235, 4264, 4284 & 4331) increased from below 2mm/day to 6-10mm/day. A graph showing the increase in movement is shown in Figure 15. Graphs indicating the displacements and rates for this period are given in Fig A5 and Fig A6. Other MPs on the north wall recorded no increase in movement rates during the same period.

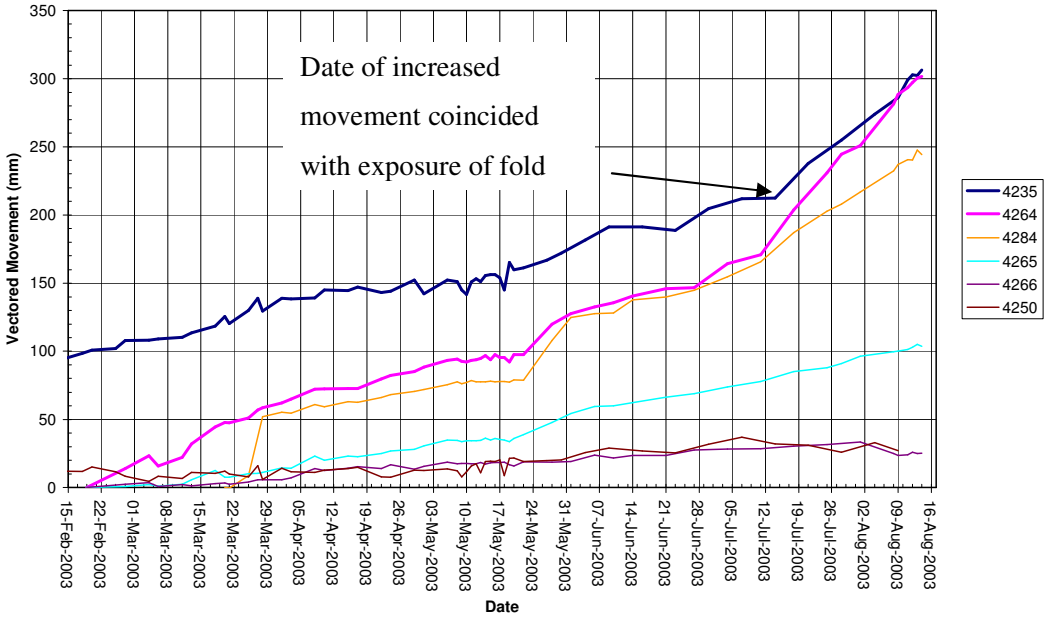


Figure 15 Displacement - time graphs of some MPs in Area of Concern

From the end of August the movement rates (MPs 4229, 4235, 4264, 4284 & 4331) again dropped to below 2mm/day. Movement rates were constant at this rate until December 2003. The decrease in movement rates coincided with the movement of mining activities towards the west of section line 21E and thus further away from the AOC. There is an obvious, but not fully understood, link between mining activity below the AOC and displacement. Mining further to the west appeared to have little or no influence on the movement rates.

By the beginning of December 2003 MP 4229 had recorded 584mm of total displacement. The perceived critical displacement defined in August had been exceeded. This was identified as a cause for concern and control measures were enhanced and formalised, as discussed in section 2.4.3. Refer to Fig A5 and Fig A6 in the appendix for details on the survey results.

During this period the prisms displayed cyclical behaviour as proposed by Broadbent and Zavodni (1982) and Sullivan (1993) – refer to Figure 16. While the behaviour of MP 4284 almost represent the "Stick-Slip" type of displacement, prisms MPs 4229 and 4235 behave more according to the Regressive (Type 1) behaviour as shown in Figure 4.

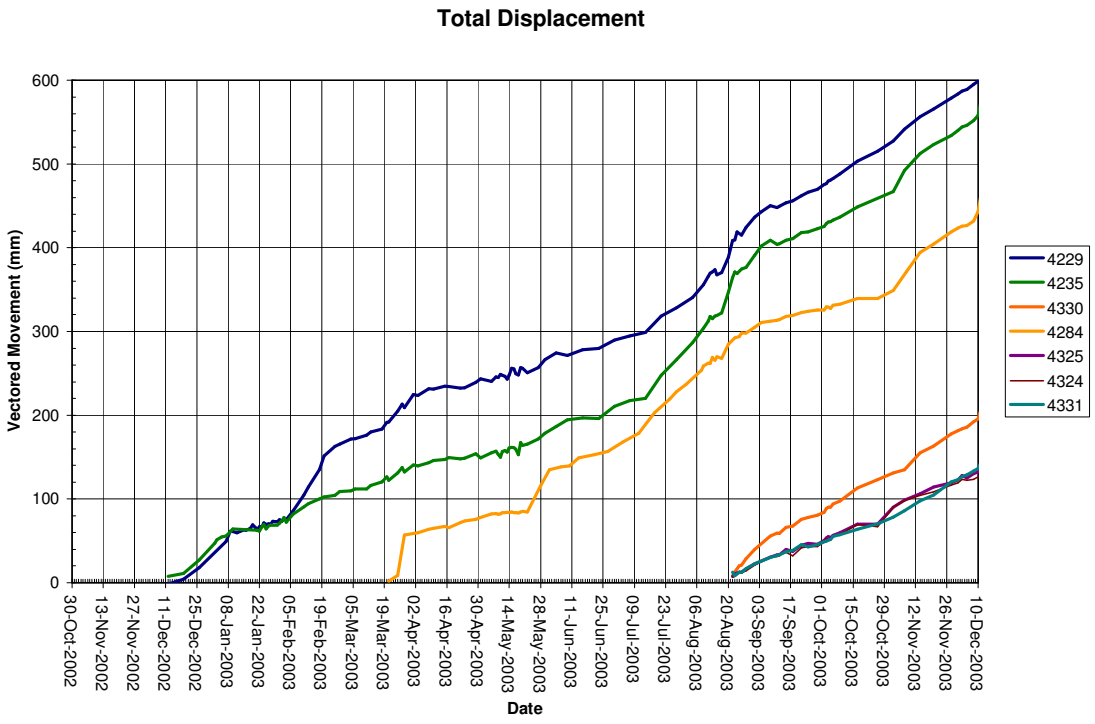


Figure 16 Displacement time graphs showing the different type of displacements during this period

2.4.2 Observations

Initially visual inspections of the north wall were conducted about once per month by walking and physically examining each bench. These took place from 90mB to 225mB but could not extend to lower benches due to lack of access. No monitoring prisms were installed below 240mB. Following the increase in movement rates in July 2003, the frequency of visual inspections was increased to weekly.

During the inspections completed in this period very few indications of instability or movement were observed. Although cracks existed on the benches, they were the result of sloughing between the 165 and 225mBs during February to May 2003. Virtually no indications of more recent deformation were observed during the inspections.

2.4.3 Actions

Various control measures were put in place at the beginning of August 2003 after the initial increased rates were recorded.

Survey and inspection frequencies were increased, additional monitoring prisms installed and personnel awareness of the situation increased. The details of the actions are described in an internal memorandum titled "*Status Report on North Wall Movement*", dated 20 August 2003.

FLAC modelling of the failure was also carried out by ITASCA (Leach 2003). The results of the modelling suggested a factor of safety of less than 1 for the model and indicated a potential 50m deep failure between the 120mB – 300mB. Different mining options for the 300 and 315mBs were evaluated. The modelling concluded that neither leaving a buttress nor mining according to original design would reduce further the FOS as the critical failure surface emerged at the 300mB above the more competent Feldspathic Quartzite (TFQ) horizon. No adjustments were thus made to the design.

At the beginning of December 2003 the level of awareness and operational controls for the mining below the AOC were further enhanced and formalised because of the following reasons:

- Onset of rainy season.
- Increase of movement rate of some monitoring prisms in the AOC.
- Mining operations resumed below the AOC on the 300mB.

- Total displacement recorded at that time had exceeded a perceived critical threshold (see section 2.4.1).

The control measures were described in a mine procedure which details the geotechnical and operational requirements that need to be in place, as well as control measures and reporting systems. It also describes the frequency of survey monitoring and visual inspections.

2.5 10 December 2003 – 19 May 2004

2.5.1 Survey Results

Towards the end of November 2003 the mining activities moved back to the eastern side of the mine, east of section line 24E and thus below the AOC. Mining activities were on the 300mB. As mining activities progressed towards the 22E-23E area, the movement rates of the monitoring prisms (MPs 4229, 4235, 4264, 4284 & 4330) in the AOC increased.

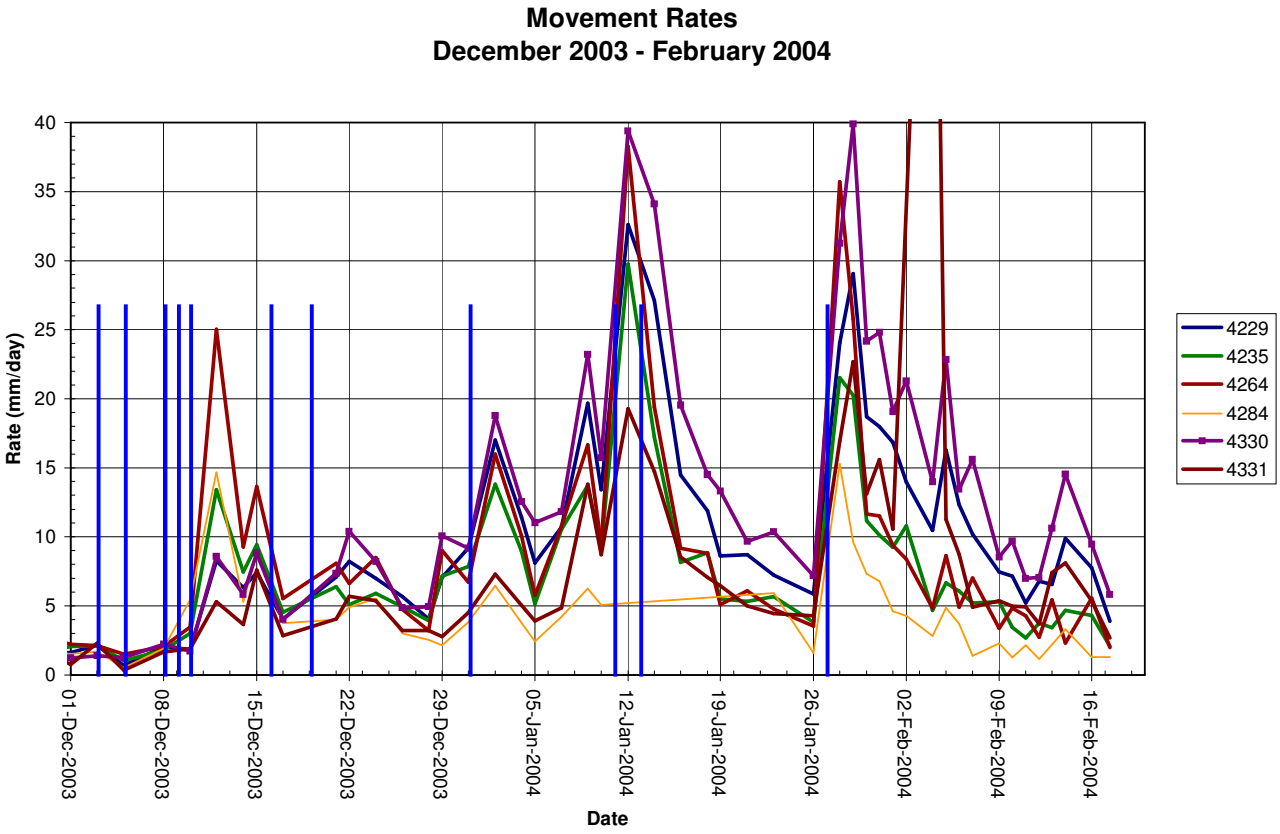


Figure 17 Movement Rates and Blast Events (blue vertical lines indicate blast events)

On 10 December a trim blast was taken on the 300mB (north wall) between sections 23.5E - 23.75E. The first peak in movement rates as a result of blasting was recorded following this blast. This is a very important observation as it is the first identified link between blasting and accelerated displacement. After the initial peak, the movement rates dropped again, but this time the average rates had increased to about 5mm/day as opposed to less than 3mm/day prior to the blast. Details of the movement rates and initial influence of blasting are shown in Figure 17.

Date of Blast	**Description**	**No of Holes**	**Effect on Survey Results**
3 Dec 03	South Wall Trim; East of 24.25E	18	No influence
5 Dec 03	South Wall Trim; East of 24.5E	16	No influence
8 Dec 03	North Wall Trim; 24.5 – 25E	16	Slight influence
9 Dec 03	Trim in AOC (23.25 – 24.5E)	15	Effect not clear
10 Dec 03	**Trim in AOC (23. 5 – 23.25E); Small slough on lower benches after blast**	**18**	**Peak in movement rates after blast.**
16 Dec 03	South Wall trim east of AOC	32	No influence. Rates dropping after blast
19 Dec 03	West of 23E (AOC)	17	No influence
29 Dec 03	North Wall Trim in AOC (22.75 – 23.25E)	14	Effect not conclusive (small jump same day)
31 Dec 03	**North Wall Trim and production in AOC (23 – 23. 5E)**	**27**	**Increase to around 15mm/day after blast**
11 Jan 04	**North Wall Trim and production in AOC (23.25 – 23.75E); Increased charges for use of front end loader (FEL) for production**	**26**	**Large peak to 30 – 40mm/day**
27 Jan 04	**North Wall Trim and production in AOC (23. 5 –24E); Increased charges for use of front end loader (FEL) for production**	**26**	**Large peak to 30 – 40mm/day**

Table 2 Blasts: December 2003 to 17 February 2004 (Blasts that influenced movement rates are in bold)

The relation between blasting and survey results is evident in Figure 17. Some of the blasts did not affect the survey results since they were not directly below the AOC. The details of the blasts in and adjacent to the AOC are summarised in Table 2.

Following the increase in movement rates due to blast events in December and January, the movement rates typically would "recover" to a rate similar to the pre-blasting rate. Towards the end of February and during March, the "recovery rate" was much slower and did not reduce to the pre-blast rates. Towards the end of March the movement rates were all below 10mm/day.

Three blasts in April, especially the blast on 14 April, had a significant influence on the slope movement. Instantaneous movement rates of more than 150mm/day were recorded – see Figure 18. The instantaneous rate may in fact have been higher as the survey measurement only took place several hours after blasting, by which time the rate of displacement was probably reducing.

If Figure 18 is compared with Figure 7 the similarity between the recorded behaviour of the slope and the Mercer model (Mercer, 2006) is evident. The blasts in April mark the change from a "Stage 1" type behaviour to a "Stage 3" type behaviour.

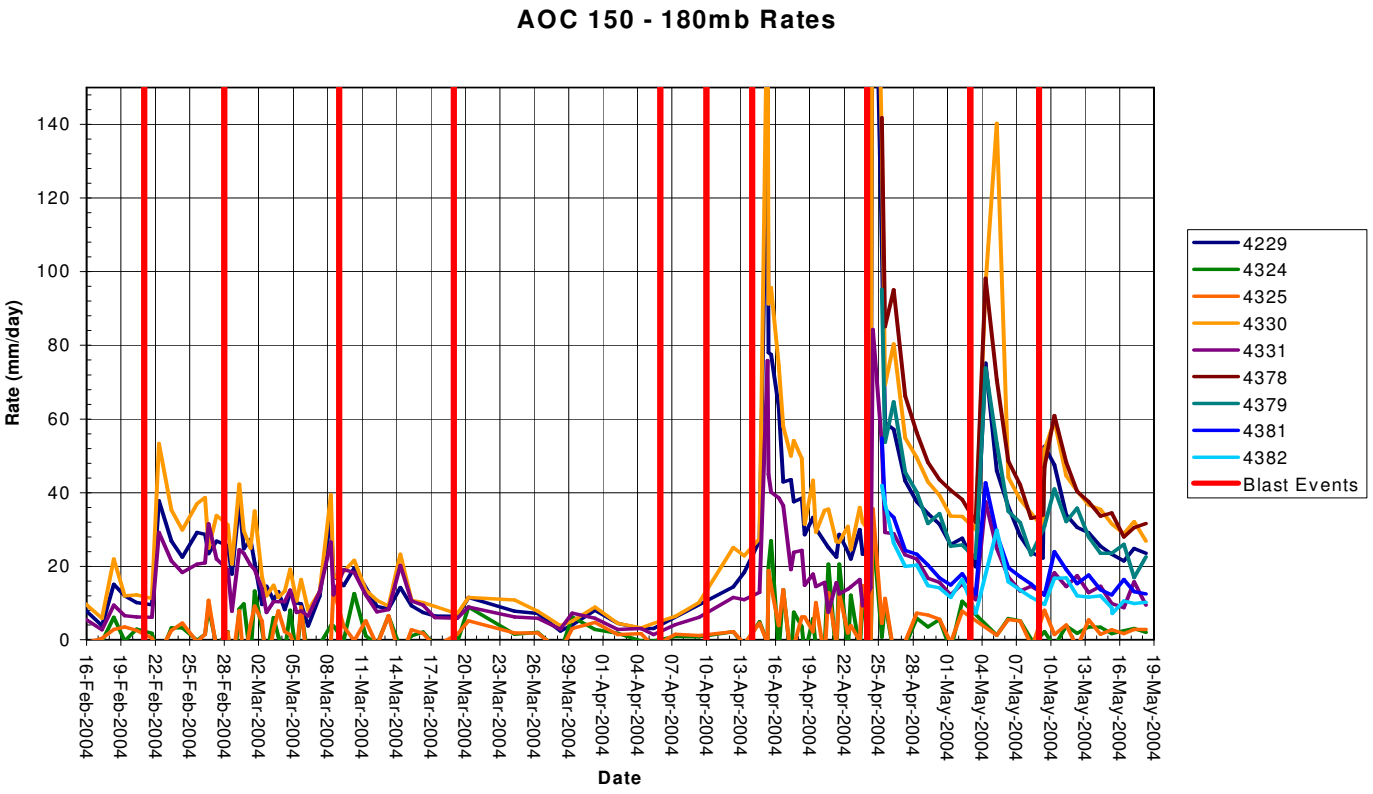


Figure 18 Movement Rates : 16 February to 20 May 2004

This movement rate is based on the total displacement recorded between two consecutive surveys, one done prior to and the other after the blast, approximately 15 hours apart. Although the survey rates did recover to lower rates, the "recovered rates" were significantly higher than those prior to the beginning of April 2004. For example, MP 4229 had an average movement rate of less than 10mm/day during most of March (excluding periods following a blast). After the mid-April peak, the movement rates of MP 4229 were never lower than 20mm/day. A graph of the movement rates for the period 10 December 2003 to 29 May 2004 can be seen in Fig A8.

During this period the total displacements recorded increased dramatically. MP 4229 reached 1000mm by early January, while MP 4330 (installed 20 August 2003)

reached 1000mm by 4 February. By mid May MP4229 had reached 3430mm and MP 4330 3627mm. Total displacements for this period are shown in Fig A7.

Towards the end of February 2004 the monitoring prisms on the 150 and 165mB recorded higher movement rates, and slower recoveries after a blast, than the other monitoring prisms. The cycle of increased rates after blasting followed by a recovery period continued for the remainder of this period.

2.5.2 Observations

Until the end of December very little physical manifestation of the recorded movements was evident on the slopes. At the end of December, a crack started to develop on the 150mB at the toe of the face. Movement on this crack was substantial and it was already 1m wide by mid February. More than 2.5m of movement was recorded by the crack monitoring measurements by mid April when measurements were suspended for safety reasons – see Figure 19.

Cracking was also evident on all the benches between the 150 and 210mBs, between section 24E and 23E. No indication of any movement or instability was observed above or either side of the AOC.

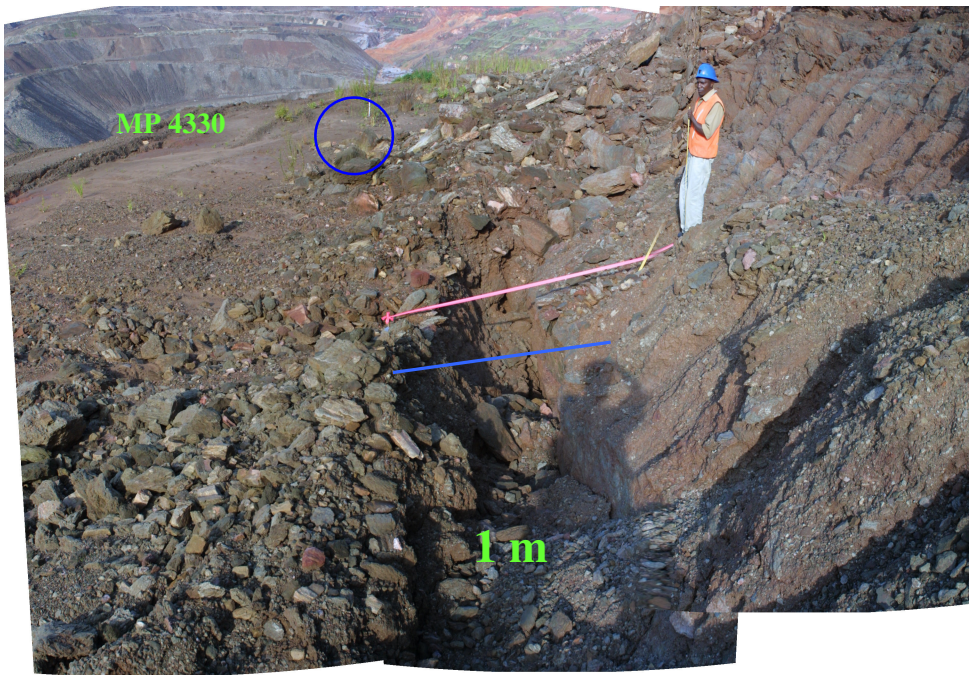


Figure 19 Crack on the 150mB at 28 January 2004. Note position of MP 4330

More incidents of sloughing of material had been observed since the beginning of January. The most noticeable slough was after the blast on 27 January, when a reasonable sized boulder was dislodged from the 150mB face and came to rest on the crest of the 165mB. ("*Notes on North Wall – 1 March 2004*").

2.5.3 Actions

Following the significant increase in movement rates on 28 and 29 January 2004, mining operations below the AOC were temporarily suspended to facilitate a thorough investigation. Following the investigation, mining resumed under enhanced control measures. These were added as amendments to the existing mine procedure.

The most significant control measures were:

- Mining operations were restricted to daylight hours only.
- Continuous survey monitoring of the Area of Concern. The surveying was done from the south wall. The surveyor was required to start the daily survey

with a conventional, detailed survey. Thereafter he took distance measurements only at regular intervals (starting off at 30 minutes) to the MPs in the AOC.

- A Geotechnical Observer was posted with the surveyor on the south wall. He would assist with the plotting of the survey results and the calculation of the movement rates. Should certain trigger levels be exceeded, he would follow certain protocols depending on the recorded rate.
- Following a rainstorm of sufficient intensity to cause temporary cessation of production operations, the go-ahead to resume operations would be given by the Senior Geotechnical Engineer (SGE) or the Group Geotechnical Engineer following examination of the face and survey monitoring records.
- Movement rate trigger levels, and related actions should levels be exceeded, were listed in detail in a mine procedure.

Mining continued under the enhanced controls stipulated in the updated KCM procedure as described above until mid April. The increased displacements following the blast on 16 April (as described in section 2.5.1) necessitated another critical evaluation of the trigger levels and controls. A consultant was on site to review the situation. As a result of the review, the trigger movement rate was revised from 40mm/day (1.6mm/hour) to 60mm/day (2.5mm/hour), additional monitoring prisms were installed and the procedure detailing the controls was amended to ensure effective evacuation of the affected area. (Terbrugge, April 2004).

2.6 20 May to 16 July 2004

At the end of April 2004 a proposal was made to implement the Slope Stability Radar System (SSR), developed by GroundProbe, in the open pit. This self contained, rapidly deployable system provided a 24-hour cover of the complete face (about 3500 pixels) and was able to discern average movement across a pixel with sub-millimetre accuracy at a range up to 850m. Deformations were measured at approximately 15minute intervals and transmitted via radio link to a central site where they were updated and displayed graphically.

The fact that the existing survey monitoring system only partly covered the area of concern was identified as a shortcoming in the safety procedure. The SSR offered significantly enhanced monitoring accuracy and a spread that afforded the mine the opportunity to improve safety, improve productivity and to enhance future slope designs.

The system was deployed at the mine on 19 May 2004, on the 225mB on section 23E on the south wall. Within the first couple of hours of monitoring it became evident that deformation was occurring over a substantial portion of the slope from 150mB to 315mB and from 21E to 24E, a much larger area than had been detected by survey and visual monitoring.

2.6.1 Radar Survey Results

The initial radar results for the north wall indicated two distinct areas of relative high displacements: the 150/165mB between 23E to 24 E and an area at about 21E, from the 270mB to 345mB– these areas are shown as dark red areas in the bottom graphic of Figure 20. The former coincided with the Area of Concern, as described earlier, while the second area was an area where rockfalls and sloughing had been recorded, but did not lie above an active mining area.

The colour coding in the GroundProbe software uses the convention of warmer colours representing greater amounts of movement towards the measuring point. Colder colours represent movement away from the measuring point. The blue polygon in Figure 20 (in photo and displacement plot) indicates the area that was identified as the AOC, based on the survey monitoring results and visual observations, while the red polygon is the area indicated by the SSR as recording movement.

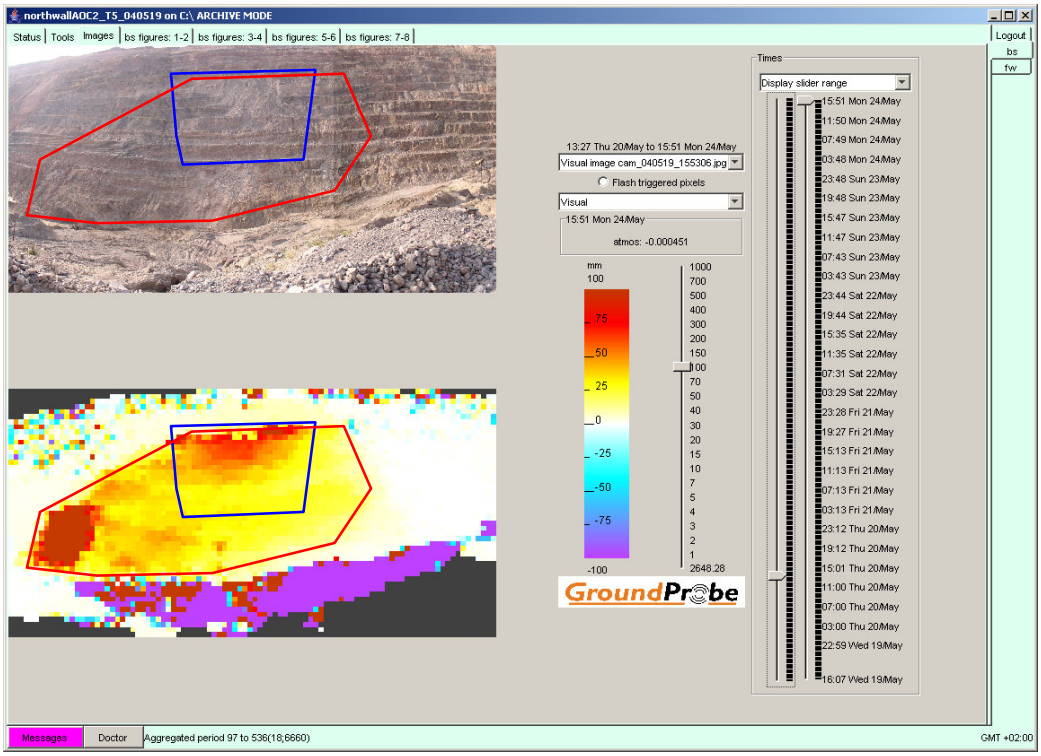


Figure 20 Initial Radar Survey Results-figure showing information that is relayed from the deployed unit to the central site. Warmer colours in the figure's bottom right indicate greater movements.

From these radar results it also became evident that, apart from the two "hot spot" areas described above, the area of movement on the north wall was much larger than expected. Although the displacements and movement rates are relatively small, an area of increased movement (yellow shaded area in Figure 20) can be distinguished from a background area of almost no movement, i.e. white shaded areas in Figure 20.

The radar results did thus confirm the survey monitoring results, both in magnitude and location. However, they did more than that. They also highlighted a specific area of high movement rate (21E) and a large area of the north wall that was recording some movement, albeit small. The 21E area was not identified by the survey monitoring due to a lack of monitoring prisms.

The yellow area in Figure 20 highlights the enhanced survey capabilities of the SSR. Although some individual monitoring prisms were surveyed and did record movement, the total area experiencing movement was never as clearly defined as in the SSR images. The SSR also covered an area on the lower slopes where very few monitoring prisms were installed.

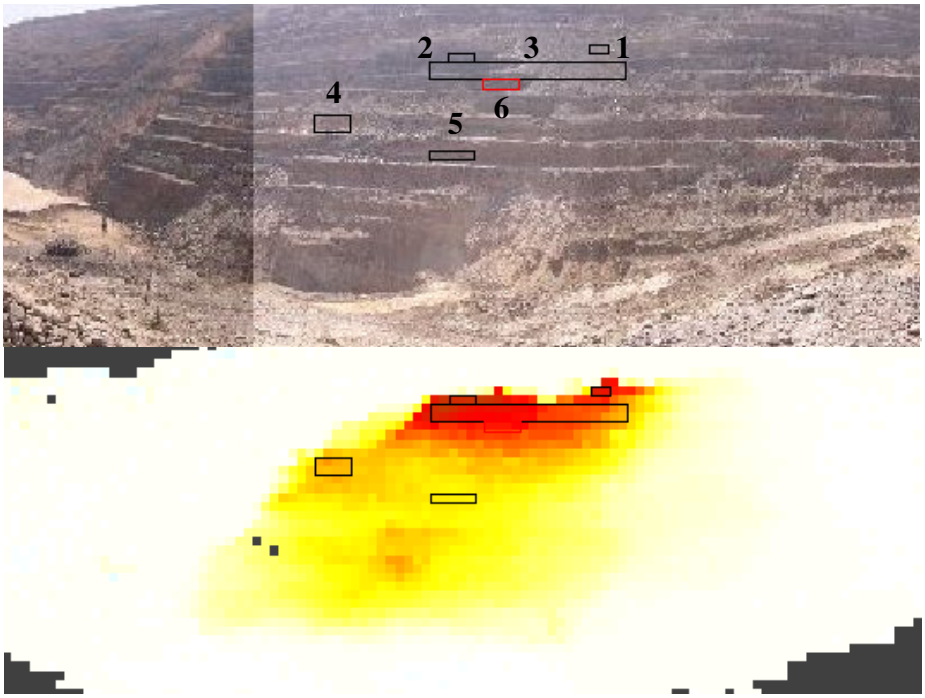


Figure 21 SSR results in June and July. Note the (red) area of high movement on the 150/165mB

The SSR indicated the highest movement rates of the scan area on the 150/165mB. The results from a graph drawn in the window labeled "3" in Figure 21 are shown in Figure 22. As can be seen from the graph, the movement rates in this area were fairly constant at about 2mm/hour (48mm/day) during June and the early part of July.

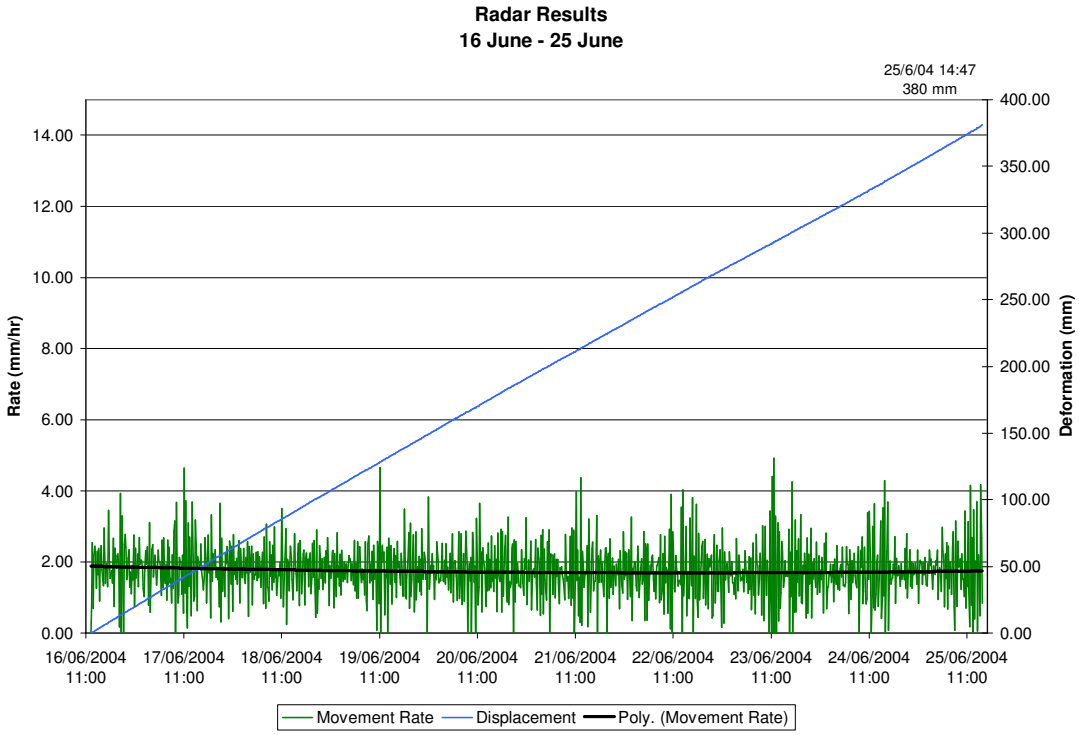


Figure 22 Radar Results – June 2004. (Refer to area "3" in Figure 21)

2.6.2 Observations

The north wall bench inspections were terminated in mid April due to safety concerns.

However, visual monitoring of the face from a distance was ongoing. Although it was more difficult to quantify bulk movement, observers were instructed to look for signs of local instability such as small sloughs, rolling rocks and dust. Most of the sloughing that was recorded occurred on the lower benches as a direct result of nearby mining activities causing disturbance of previously sloughed material.

2.6.3 Failure at Section line 21E

On Saturday, 22 May 2004, a collapse occurred at 21E, in the area indicated as being unstable by the SSR as described in Section 2.6.1. Unfortunately observers did not

routinely scrutinise the displacements in this area since this was outside the active mining area. The failure was also shortly after implementation of the SSR and personnel were still in the process of understanding the results and putting monitoring procedures in place. The huge increase in movement data (4 readings per hour as opposed to a maximum of 2 surveys per day) did take a while to get used to. The Geotechnical team was still in the process of developing protocols to handle the increase in data and to get familiar with the alarm systems of the SSR when the failure took place. No advance warning of this failure was thus issued. No injury or damage to equipment occurred due to the failure – the outcome was a delay in mining the 345mB whilst access was cleared.

A back analysis was carried out on data obtained from an analysis window that was located retrospectively to cover the area. The following observations were made from the back analysis (refer to Figure 23):

- The increase in movement rates can clearly be seen in Figure 23.
- The first time an instantaneous rate of 3.5mm/hour was exceeded was approximately 12 hours before the failure.
- The instantaneous rate exceeding a 5mm/hour trigger level would have triggered an alarm 6 hours before failure.
- An instantaneous trigger rate exceeding 10mm/hour would have meant 2 hours of warning.

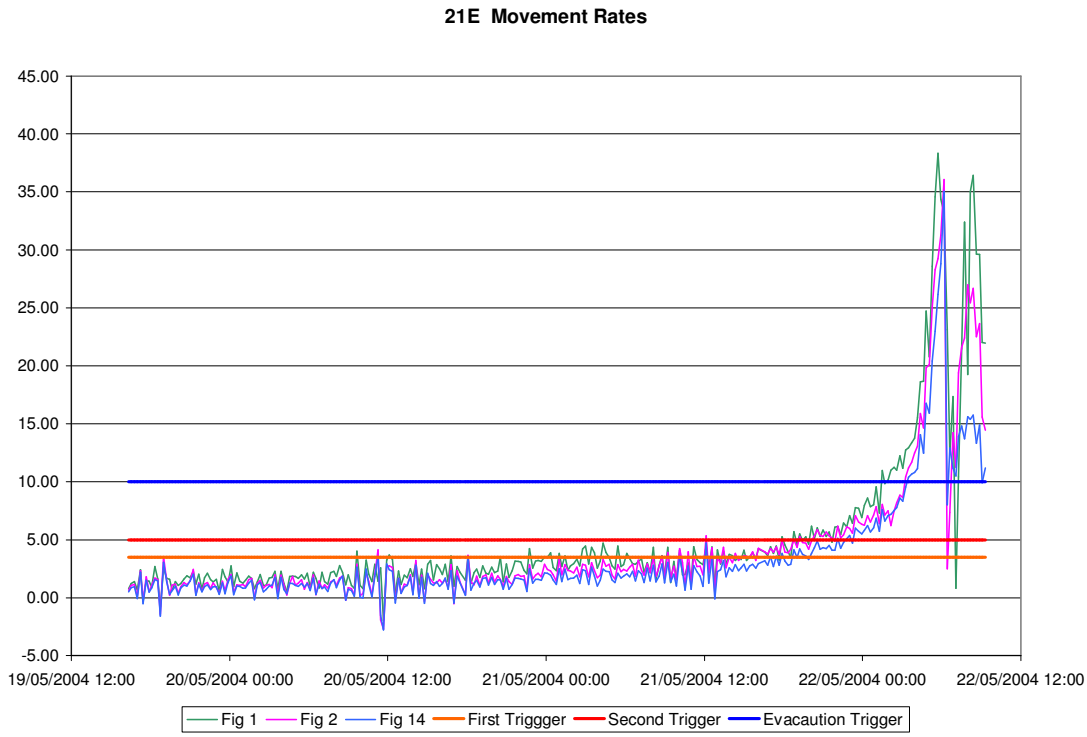


Figure 23 Back Analysis - 21E Failure figs 1, 2 and 14 refer to different windows placed retrospectively on the area

This information was used to establish the following Management responses to be invoked when mining in the area of concern.

Trigger Level	Response
3.5mm/hour	Increase frequency of rate calculations to every 30 minutes
5mm/hour	Increase frequency of rate calculations to every 15 minutes Inform Control Tower of Alarm status Inform Geotechnical Engineer on duty
10mm/hour	Evacuation from area

Table 3 Trigger levels derived from the 21e Failure

2.6.4 Radar Results Prior to Failure

The movement rate of about 2-3mm/hour continued until about 9 July, when it started to increase at a slow but steady rate (Figure 24). On Sunday 11 July the rates had increased to between 5 and 10mm/hr. On Monday 12 July the rates jumped to about 20mm/hr following blasting during the afternoon and continued between 20 – 30mm/hr until early in the morning of Thursday 15 July (about 02:00), when the rates accelerated until final slope collapse on Friday, 16 July at about 07:10. The increase in movement rates can clearly be seen in Figure 24. The average movement rate immediately prior to failure was around 100mm/hr. The radar results of the four days prior to failure are presented in Figure 25.

The total displacements are not recorded by the SSR. Every time the scan is interrupted and restarted (for example, when the system is moved or stopped), the displacement for the new scan is recorded from zero. However, the displacement of the 150/165mB (area #3 in Figure 21) as recorded by the SSR during the four days prior to failure was about 2800mm.

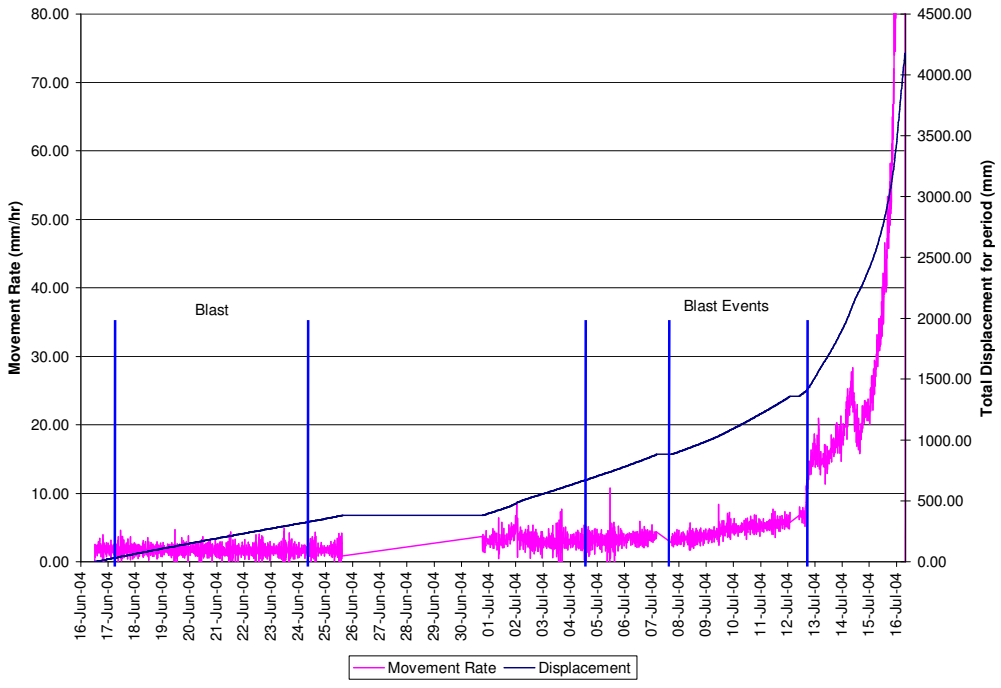


Figure 24 Radar Results : 16 June – 16 July. Blast events are indicated by blue lines.

The last displacement of MP 4229 was measured on 14 July. The total displacement recorded since November 2002 was about 8174mm, while MP 4330 recorded 7221mm of displacement between August 2003 and 7 July 2004. These two monitoring prisms were located at about 23.5E, thus directly above the "old slough" (February 2003) area and recorded the highest displacement of all the MPs on the slope. For displacement records of some of the other MPs refer to Table 4 and to Figure 8 for their locations on the North Wall.

MP	Date of 1st Survey	Bench	Section	Tot Displ	Date of last reading
4323	20/8/03	135	22.75	453	14/7/04
4324	20/8/03	135	23.25	467	7/7/04
4330	20/8/03	150	23.5	7221	7/7/04
4325	20/8/03	135	23.75	511	14/7/04
4328	20/8/03	150	22.25	319	7/7/04
4329	20/8/03	150	22.75	808	7/7/04
4331	20/8/03	150	24	2963	14/7/04
4223	14/11/02	165	22.5	1293	14/7/04
4229	26/11/02	165	23.25	8174	14/7/04
4235	5/12/2002	180	23.5	2934	14/7/04
4310	22/5/03	225	21.75	1762	8/7/04
4311	22/5/03	225	22.5	2064	8/7/04
4312	3/12/03	225	22.75	379	17/2/04

Table 4 Total displacements by MPs at time of failure. (MPs are listed by bench)

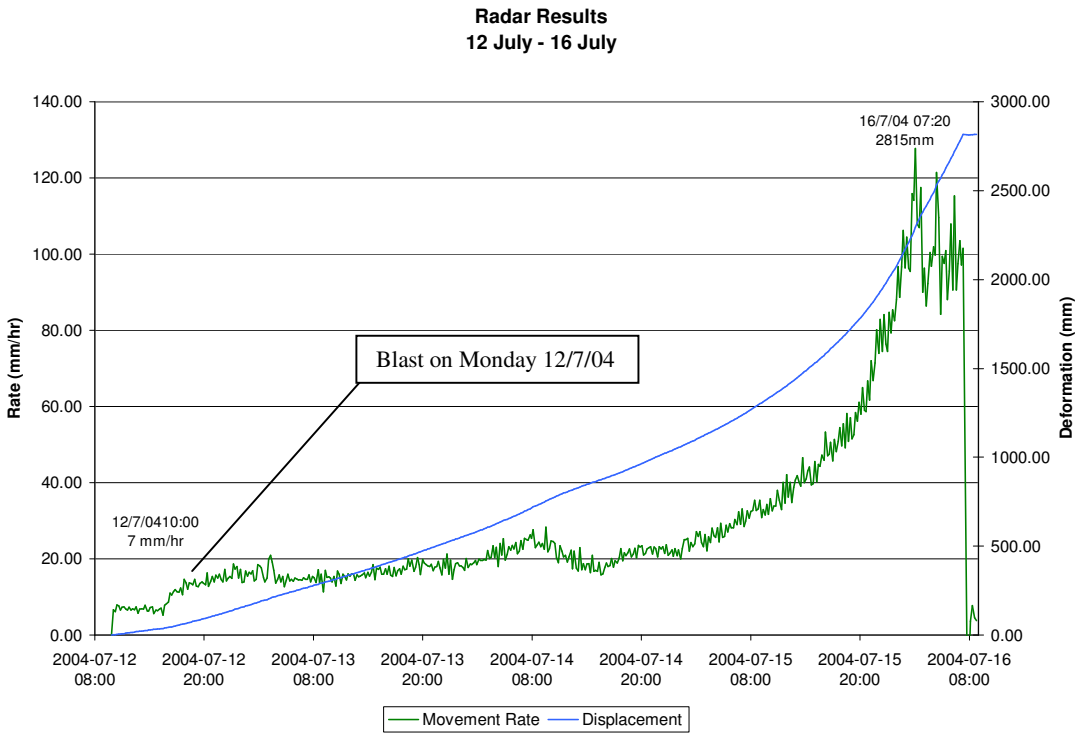


Figure 25 Movement Rates and Displacement in the four days prior to Failure

2.6.5 Evaluation of Trigger Levels.

During the time of the failure no mining was taking place directly below the Area of Concern. Although monitoring of the failure was ongoing, it was not necessary to act on exceedances of the trigger levels. However, the SSR records of the period shortly before and during the failure were used to evaluate the trigger levels determined earlier, as discussed in Section 2.6.3.

The information from four wall files (different scan periods) for the 150/165mB was grouped and is plotted in Fig A11. (The displacement values of the last three files were adjusted to provide a continuous record; otherwise the data for each file would have started at null displacement.)

The first time the 3.5mm/hour trigger was exceeded was on 16 June. However, due to the fluctuations in the movement rates it was agreed that at least 3 consecutive

exceedances were required to trigger the alarm. This would have been on 1 July. From 9 July the movement rates were continuously above 3.5mm/hour (see Fig A11). The evacuation alarm of 10mm/hour was first exceeded on 5 July. On 12 July a blast resulted in a movement rate above 10mm/hour, and the rate never recovered from this. The details of the exceedances of trigger levels are detailed in Table 5.

	1st Trigger			3 Consec. Triggers			Continuous		
	Date	**Days**	**Hrs**	**Date**	**Days**	**Hrs**	**Date**	**Days**	**Hrs**
3.5 mm/hr	16 June 19:28	29.5	708	1 July 17:40	14.6	349	9 July 08:20	6.9	167
5 mm/hr	19 June 11:07	26.8	260	9 July 16:30	6.6	158	11 July 17:40	4.6	109
10 mm/hr	5 July 11:00	10.8	260	12 July 16:15	3.6	87	12 July 16:15	3.6	87
10mm/hr (Adjusted)	5 July 11:00	10.8	260	14 July 03:31	3.3	79	14 July 18:42	2.7	66

Table 5 Table showing dates when trigger levels would have been exceeded. The warning of a failure this would have given is listed in days and hours.

From the above table it is clear that, if the trigger levels were to be applied to the movement rates on the 150/165mB prior to failure, adequate evacuation warning would have been given.

The blast on Monday, 12 July resulted in a jump in the movement rates (Figure 25) to above 10mm/hour. To further test the trigger levels, it was decided to eliminate the effect of the blast to see if the evacuation trigger would have been triggered early enough. Figure 26 shows the results when this jump is eliminated and it is assumed that the movement rates would have followed the same movement rates for the remainder of the period. The three consecutive readings exceeding the 10mm/hr trigger would have given 3.3days (79 hours) warning of collapse – refer to Table 5.

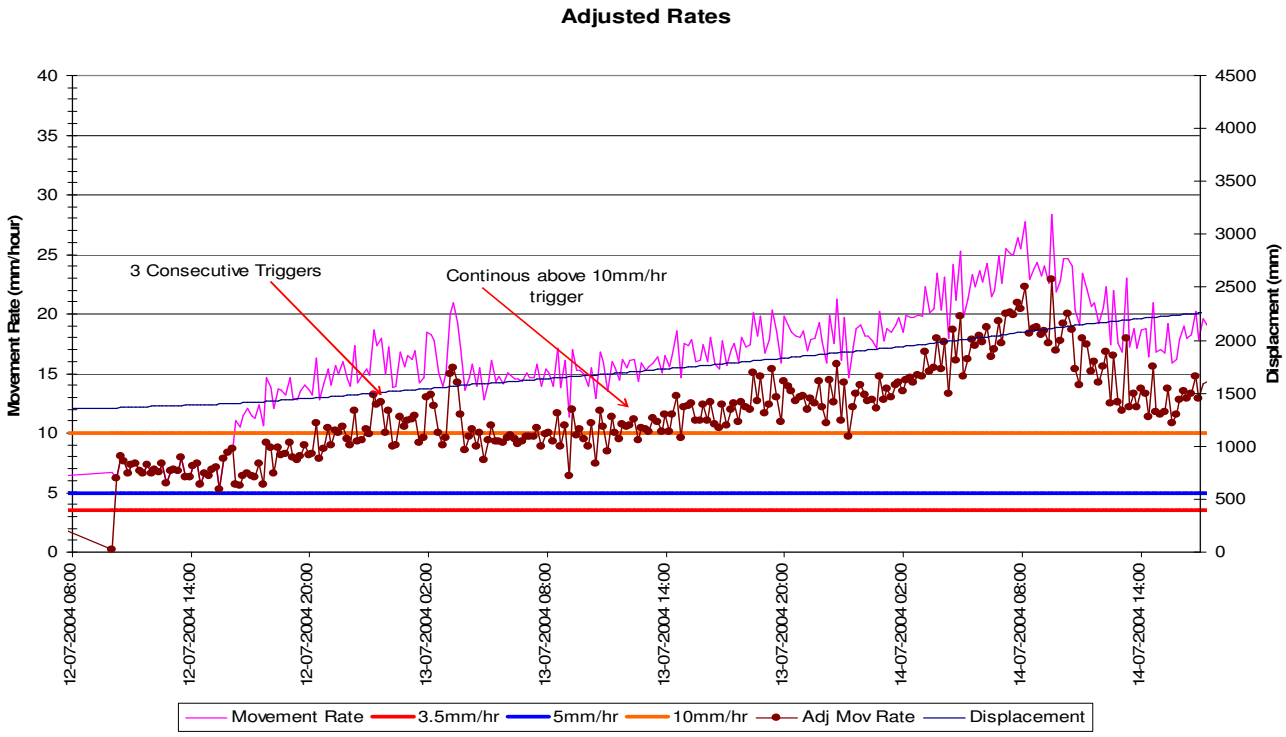


Figure 26 SSR Movement Rates and Adjusted movement rates for the period following the blast on Monday, 12th July. The various trigger levels are also indicated.

From the back analysis of the failure of 16 July and the 21E failure on 22 May (section 2.6.3) it would seem as if a 10mm/hr can be used as a warning of an imminent failure and thus be used as a final trigger level to initiate evacuation. However, it is important to note that this trigger level is based on limited information and should be used as a guideline and not in isolation. It is suggested that three consecutive triggers of 3.5mm/hr and 5mm/hr be used to increase frequency of rate calculations. The trends of movement rates should be closely monitored. Evacuation prior to a 10mm/hr trigger might be warranted if a rapid change in the movement rate (thus acceleration) is observed.

2.6.6 Prediction of time of failure.

When the initial increase in movement was observed in July 2003, time-displacement plots were used in the decision making process. The change in behaviour was first identified in such a plot.

One of the classical examples of a failure prediction with time-displacement plots was recorded at Chuquicamata in 1969 (Kennedy, 1970). The use of these types of plots to predict failure in this case study proved to be a problem. The displacements of a number of prisms are shown in Figure 27 and Figure 28. A dramatic increase in the displacement (e.g. MP 4330) can be seen in Figure 27. Based on this graph it would seem as if, by the end of January 2004, failure is imminent. However, Figure 28 is a plot of the same prisms showing displacements to the middle of July. This illustrates the effect of scale on a time-displacement curve: the "imminent failure" that was suggested in the early December curve can hardly be identified in the July curve.

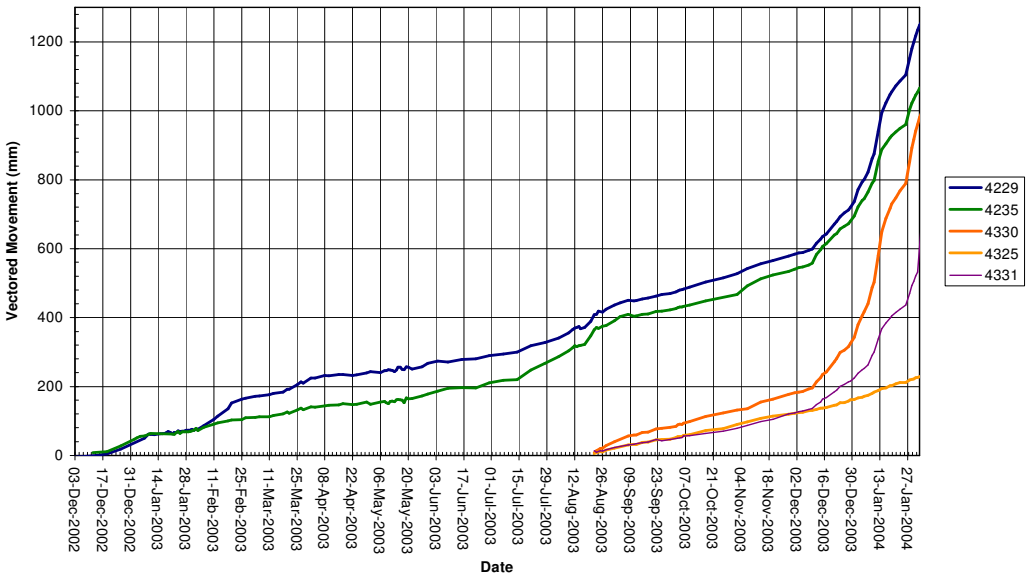


Figure 27 Time-displacement plot of prisms. Note dramatic increase in displacement during December 2003

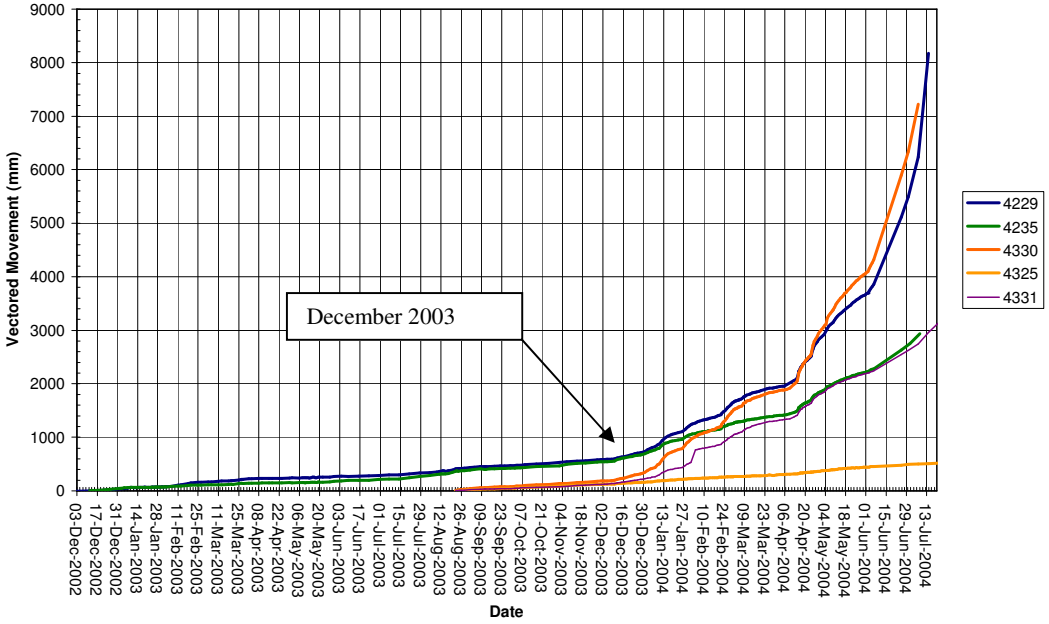


Figure 28 Time - displacement graph of the same prisms at a different scale.

The time-displacement graph is a handy tool, but should not be used in isolation. Once an increase in movement has been established, movement rate and increase in movement rate is a more accurate prediction and decision tool.

The effectiveness of the trigger levels was discussed in Section 2.6.5. Although the evacuation trigger level of 10mm/hour was exceeded on 12 July, personnel were not evacuated. Apart from the fact that operations were west of the Area of Concern, it was also acknowledged that the movement rates were not increasing. The evacuation of personnel was done early on 16 July. From the rapid change in movement rates it was evident that failure was imminent and personnel were evacuated as a precaution. Failure took place approximately 90 minutes after evacuation.

2.7 Failure Mechanism

Initial instability:

The initial bench instability that was reported between September 2002 and February 2003 developed slowly during this period and was attributed to bench scale instability

and an adverse dipping structure (refer to the yellow polygon labeled "23E Slough" in Figure 34). The increased movement that was identified by survey monitoring in July 2003 was mostly in the area directly above the "23 Slough" and was limited to a 120m wide area between section lines 23E and 24E and involved only two benches. The failure mechanism was toppling on a bench scale, as can be seen from the photos in Figure 10. Although the instability was triggered by a high rainfall season, the effect of the steeply dipping structure into the face cannot be ignored.

The period leading up to the failure

The re-activation of the failure in 2004 was related to the exposure of a fold structure and response to mining below the fold. The failure was complex and most likely a combination of two failure mechanisms, i.e. toppling and rock mass failure at the toe. The area between sections 23E and 24E continued to topple; an extension of the instability discussed above. Most of the recorded movement was on the 150 and 165mBs, and the cracks continued to open up – refer to Figure 19. Initially, only limited evidence of movement and/or instability was recorded on the lower benches. Evidence for the weakening of the rock mass is the fact that the onset of increased movement coincided with mining through a structure and exposing weaker material. During this stage (mid 2004) there was no evidence of instability further to the west.

The evidence for the toppling failure is the following:

- Visual inspections of the "23E Slough" showed evidence of toppling – refer to the photo on the right in Figure 10.
- Large displacements were recorded by prisms in this area.

It was only with the implementation of the SSR that movement to the west became evident, a few months prior to the failure. The full extent of the unstable area became clear after the initial scans. This is confirmed by the displacement records of prisms 4223 and 4329 that are tabulated in Table 7 and the displacement graphs in Fig A7. Both these prisms only recorded about 1000mm of movement by June 2004, while

the prisms east of 23E recorded between 2400 and 4700mm of movement (MPs 4235 and 4330). No physical evidence of toppling was evident in this part of the slope and it seems as if this was mainly a structurally controlled planar failure.

The failure

Figure 29 was taken during the failure. The failed area to the west (left hand) in the photo is more intact than the area to the right, although the large crack at the top of the failure confirms that failure has taken place Just before the material failed a loud "bang" was reported. After the failure a large, almost vertical, plane was exposed.



Figure 29 Photo of the failure. Note the more "disturbed" mass on the right, while the berms on the left are still relatively intact.

This would suggest that the eastern (right hand side in Figure 29) toppled and the accumulation of toppled material higher up the slope and the associated movement resulted in an increased interramp slope angle. This localised steepening increased the stress on the weaker material at the toe of the steepened section to a point where the rock mass in the toe failed. The loud bang could be when the release plane at the

back of the failure was formed and the slope to the west (left hand side of photo) failed.

The differential displacements on cross sections of the pit slope are depicted in the following figures. Time – Displacement graphs of different elevations were constructed from radar information. Data from the last week prior to failure was used.

From Figure 30 it is clear that the upper section of the failed area experienced the most displacement. Most of the slope recorded less than 1.5m of total displacement during the week, while the upper bench recorded almost 3.5m of movement during the same time. This evidence supports the overall toppling effect of the failure on the eastern section of the failure.

The displacement curves in Figure 31, i.e. a section further west, are however different: Curves 10 and 11 experience very little to no displacement. (Curve 10 is almost on the x-axis and difficult to see). Although this is expected since the curves fall outside of the failure, it is further evidence that the failure was structurally controlled: almost no displacement on the benches above the failure, i.e. behind the structure that formed the release plane at the back of the failure. The most displacement was recorded by Curves 12 and 13, which were at the crest of the failure. This is similar to what was observed in the section to the east (Figure 30). However, the zones on the wall described by Curves 15, 16 and 17 recorded more displacement than Curve 5 zone, i.e. the middle of the slope, a few days prior to failure. This suggests three zones of different movement: the crest of the failure (that recorded the most movement) due to toppling, the mid section, which recorded the least amount of displacement in the days leading up to the failure, and the toe section of the slope. If Curve 17 is examined closely it is interesting to note the jump in the displacement in the red oval. This jump occurred on 13 July, at about 21:00. This larger displacement lower down the section would suggest the weakening of the toe

prior to failure, and supports the idea of rock mass failure lower down the slope in this part of the failure.

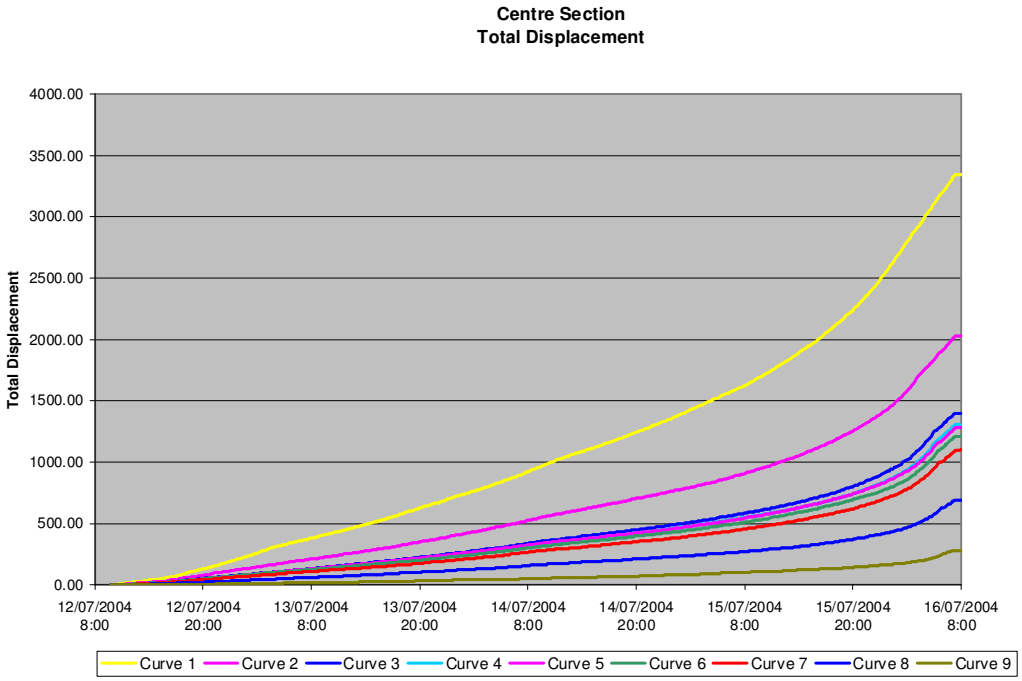


Figure 30 Time - Displacement graphs from radar results or the 'East section'. The different curves indicate the displacement at different elevations on the slope. Refer to Figure 32 for the location of the curve on the slope.

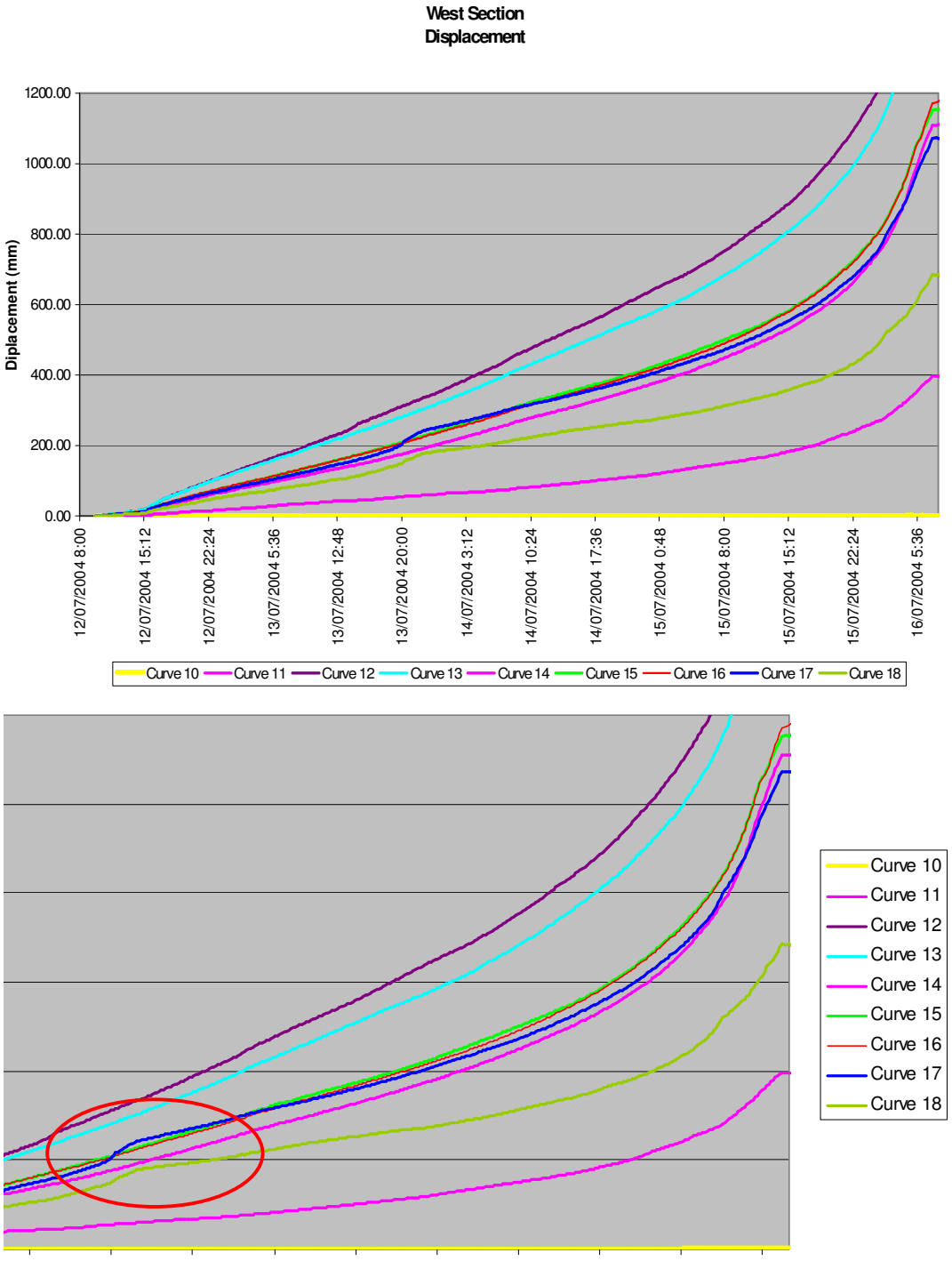


Figure 31 Time Displacements graph of the Western section, with a zoomed in section at the bottom. Note the jump in the displacement of Curve 17 in the red oval. Refer to the text for an explanation of the graphs.

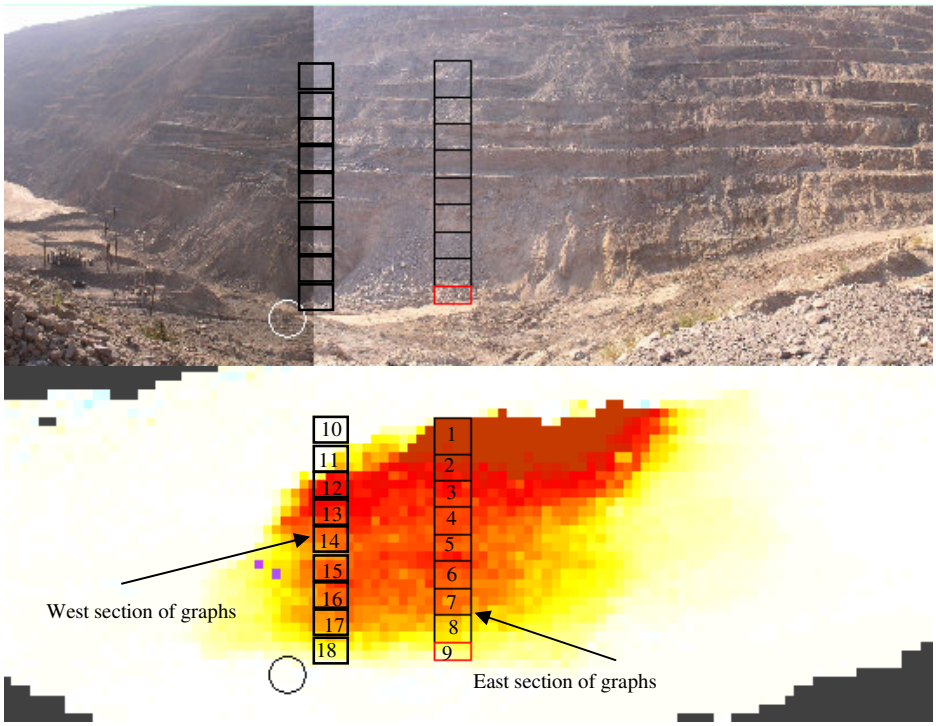


Figure 32 This figure indicates the locations of the areas for which displacement curves were constructed. The number relates to the curves in Figure 30and Figure 31.

The failure developed over a long period and the mechanism is complex. It would seem as if toppling, especially during the early stages, resulted in large movement at the crest of the failure (up to 7m – see Table 4). This movement caused a localised 'steepening' of the slope that led to a failure of the toe due to the loading by the mass of toppled material. Large structures within the wall acted as release planes at the time of the failure.

3 HIGHWALL STRAIN CRITERIA

The various methods available to predict the onset of failure and time of collapse were discussed in Section 1.3. The most frequently used method involves the use of displacement data and the plotting of displacement or velocity against time. That was also used in this case study, where the monitoring evolved from displacement /time graphs, to velocity/time graphs to changes to the velocity/time graphs (acceleration). Another method suggested by a few authors is to utilise the ratio of total displacement and slope height as an indication of an imminent failure. This method will now be evaluated against the information and experience gained with the north wall failure.

3.1 Background

Brox and Newcomen (2003) argued that displacements do not reflect the total strain, caused by the displacement, which a slope has experienced. They define the high wall strain criterion as the total slope height divided by the total displacement experienced by the slope and expressed as a percentage, and suggest that this criterion can be used as an additional method to evaluate slope stability and predict time of failure. They recognised that the amount of strain a slope can accommodate before failure depends on the rock mass quality and type of failure mechanism. Brox and Newcomen (2003) use the Bieniawski (1976) Rock Mass Rating (RMR) system to describe the rock mass quality.

In a planar or wedge type failure mechanism, failure can be expected at very low strain levels and the rock mass quality does not play an important role, although the orientation and strength of discontinuities play an important role in the failure. In a stepped-path failure the rock mass between the discontinuities has to fail before instability takes place, therefore a higher strain rate level than in the case of wedge and planar failure can be expected. When a toppling failure or rotational failure occurs large strain levels can be recorded before slope failure takes place, and the rock mass quality, especially at the toe of the slope, is important.

Brox and Newcomen (2003) developed a model from various case studies (Figure 33), and the suggested strain threshold levels are tabulated in Table 6. Figure 33 will be used to test the applicability of the North Wall strain values to the levels suggested by Brox and Newcomen (2003).

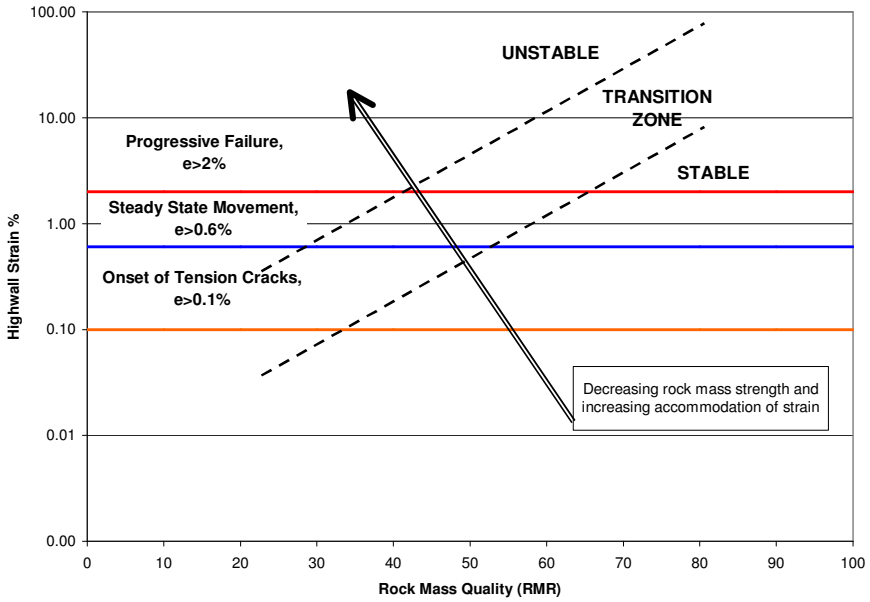


Figure 33 Strain model developed from case studies by Brox and Newcomen

Highwall Stability Stage	**Threshold Strain Level %**
Tension Cracks	~ 0.1
Progressive Movement	~ 0.6
Imminent Failure/Collapse	>2.0

Table 6 Strain levels suggested by Brox and Newcomen (2003)

Zavodni (2001) suggests that once 0.5% strain has been exceeded, displacement rates start to accelerate. Once slope strain exceeds 1 – 2% the displacement results in

collapse. Small and Morgenstern (1991) found that at a strain of between 0.6 to 1%, a failure becomes progressive.

3.2 North wall conditions for model

The failure took place in a rock mass with a RMR (Bieniawksi 1976) ranging between 35-45, with a strongly developed set of cleavage planes that dip into the slope. Near the toe of the failure is a weak rock mass (RMR below 35 and UCS lower than 10 MPa) that has historically been involved in a number of instabilities on the north wall.

The survey deformation history of monitoring prisms, circled in red in Figure 34 and tabulated in Table 7, was used in the strain calculations. Monitoring prisms 4223, 4229 and 4235 were installed relatively shortly after the bench, on which they were positioned, was mined out. Monitoring prisms 4328, 4329, 4330 and 4331 were installed in August 2003, thus almost a year after the bench was mined out. These prisms were installed to increase the density of survey coverage after an increase in movement rates was recorded. The other monitoring prisms in the area were installed at a much later stage when significant deformation had already taken place and were thus not used in the calculations.

	DISPLACEMENT								Pit Bottom		Slope
	4223	4224	4229	4235	4328	4329	4330	4331	Bench	Elev.	Height
Dec-02	54	59	105	97	54	54	105	59	195	1138	45
Jan-03	71	112	164	151	71	71	164	112	195	1138	45
Feb-03	93	129	215	186	93	93	215	129	210	1123	60
Mar-03	113	140	281	207	113	113	281	140	210	1123	60
Jun-03	180	171	379	306	180	180	379	171	255	1078	105
Sep-03	262	224	557	495	205	219	445	210	300	1033	150
Dec-03	390	267	736	692	322	336	613	334	300	1033	150
Jan-04	455	292	1124	974	354	395	1067	538	315	1018	165
Feb-04	503	312	1471	1209	377	451	1500	992	315	1018	165
Mar-04	529	343	1936	1427	397	490	2111	1392	315	1018	165
Apr-04	657	350	2338	1629	407	516	2552	1686	315	1018	165
May-04	922	378	3438	2162	457	782	4006	2214	330	1003	180
Jun-04	1046	419	3958	2367	484	872	4696	2414	330	1003	180
Jul-04	1331	440	8275	3020	499	988	7600	3134	330	1003	180

Table 7 Details of displacement, pit depth and slope height used in strain calculations.

In an effort to calculate the total displacement experienced by the different benches, the surveyed displacements were adjusted to cover the period between mining of the

bench and installation of the monitoring prisms. The details of the adjustments are listed in Table 8.

	Basis for adjustment	**Adjustment (mm)**
MP4223	Movement rate of first two month (0.52mm/day)	38
MP4229	Movement rate of first two month (1.17mm/day)	101
MP4235	Movement rate of first two month (1.13mm/day)	86
MP4328	Based on MP 4223	180
MP4329	Based on MP 4223	180
MP4330	Based on MP 4229	379
MP4331	Based on MP 4224	171

Table 8 Basis for adjustments to cover period between bench exposure and first survey of monitoring prisms

Monitoring prisms 4223, 4229 and 4235 were adjusted based on their respective movement rates for the first two months after installation. Because of the long period between mining of the bench and installation, monitoring prisms 4328, 4329, 4330 and 4331 were adjusted based on the displacement of monitoring prisms close to them. The adjustment (in millimeters) was added to the survey displacement for each monitoring point.

The displacements recorded by the monitoring prisms at given periods are tabulated in Table 7. Note that this is not the full survey history but summarised data. In order to calculate slope height, the bottom of the pit at the given period is also listed. The slope height was calculated from the crest of the failure (150mB) to pit bottom for each given period. The actual crest of the slope was at the 90mB elevation, but it was argued that the slope above the failure was not actively involved in the failure.

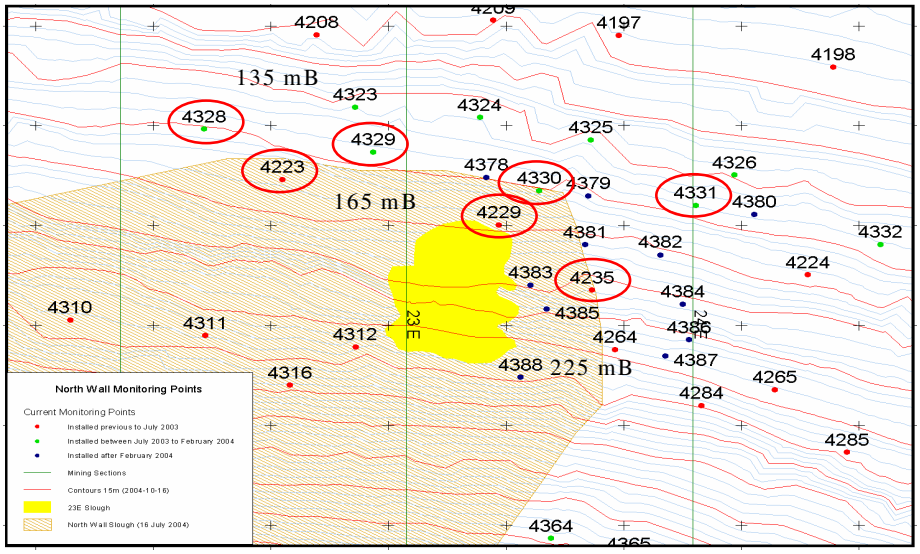


Figure 34 The monitoring prisms that were included in the strain analyses are circled in red.

The different stages as suggested by Brox and Newcomen will now be discussed in detail.

3.3 Crack forming

The first time cracks were observed in the area of the failure was in September 2002 on the 165mB (near MP 4229) when the 180mB was mined. However, these cracks were related to localised bench scale instabilities due to adverse dipping structures and not treated as part of the major failure.

	STRAIN LEVELS 0F MONITORING POINTS (%)							
	4223	4224	4229	4235	4328	4329	4330	4331
Dec-02	0.12	0.13	0.23	0.22	0.12	0.12	0.23	0.13
Jan-03	0.16	0.25	0.36	0.34	0.16	0.16	0.36	0.25
Feb-03	0.16	0.22	0.36	0.31	0.16	0.16	0.36	0.22
Mar-03	0.19	0.23	0.47	0.35	0.19	0.19	0.47	0.23
Jun-03	0.17	0.16	0.36	0.29	0.17	0.17	0.36	0.16
Sep-03	0.17	0.15	0.37	0.33	0.14	0.15	0.30	0.14
Dec-03	0.26	0.18	0.49	0.46	0.21	0.22	0.41	0.22
Jan-04	0.28	0.18	0.68	0.59	0.21	0.24	0.65	0.33
Feb-04	0.30	0.19	0.89	0.73	0.23	0.27	0.91	0.60
Mar-04	0.32	0.21	1.17	0.86	0.24	0.30	1.28	0.84
Apr-04	0.40	0.21	1.42	0.99	0.25	0.31	1.55	1.02
May-04	0.51	0.21	1.91	1.20	0.25	0.43	2.23	1.23
Jun-04	0.58	0.23	2.20	1.32	0.27	0.48	2.61	1.34
Jul-04	0.74	0.24	4.60	1.68	0.28	0.55	4.22	1.74

Table 9 Percentage Strain for Different Periods.

After the initial increase in movement rates in July 2003, some of the existing cracks did show some renewed movement. These cracks were limited to an approximately 100m wide area east of section line 23E, and include the following MPs: 4229, 4330 and 4235. This was the area where a toppling failure mechanism was thought to be the overriding mechanism – refer to Section 2.7. A major crack developed in December 2003 – as discussed in Section 2.5.2. The period June to December 2003 will be considered as the time when cracks associated with the failure developed. The strain levels recorded during this period are plotted in Figure 35. The June strain levels will be related to the toppling part of the failure, while the December levels should reflect the condition in the area where a planar failure was the failure mechanism. Refer to Table 9 for the details of the calculated strain.

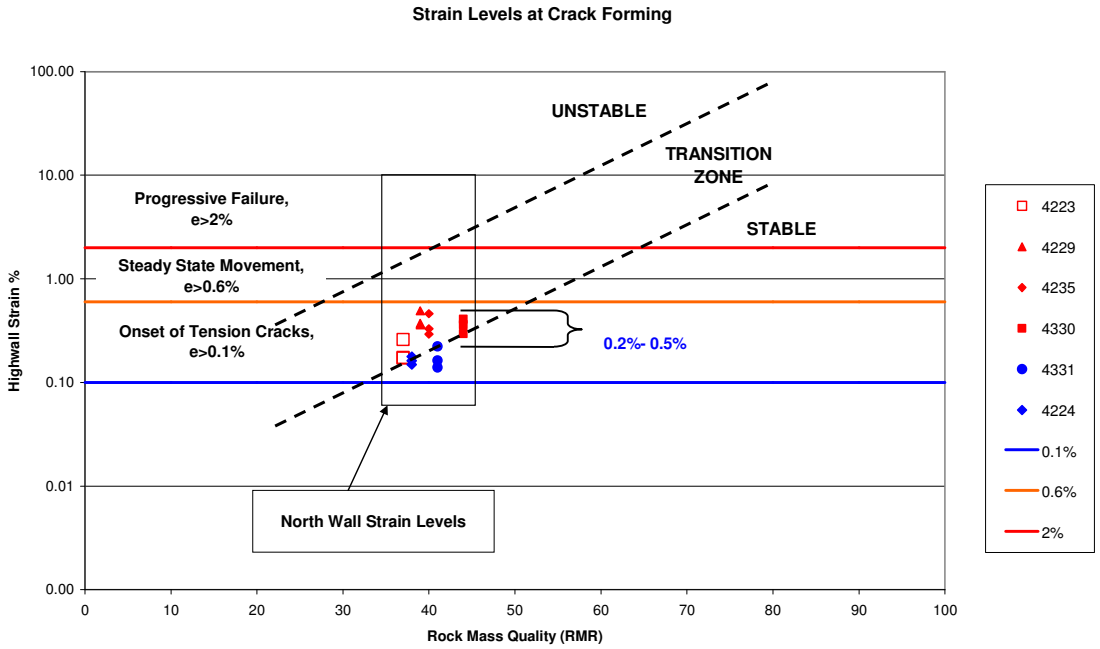


Figure 35 Crack forming strain levels in the North Wall

The strain levels in June in the toppling part of the failure (23E – 24E) vary between 0.3 and 0.4% strain. In the planar failure section of the slope (west of 23E), 0.2% strain was recorded during this period.

The complete range of strain levels associated with cracks forming varies from 0.2 to 0.5%, as indicated in Figure 35.

3.4 Progressive Movement

Zavodni (2001) suggests that 'progressive failure …. is one that will displace at an accelerating rate … to the point of collapse unless active and effective control measures are taken'. It was not easy to establish the onset of progressive movement of the north wall.

One criterion that can be used is the response to blasting, as discussed in section 2.5.1 – refer to Figure 36. Each peak in the graph represents a survey following a blast. The first recorded effect of blasting on movement rates occurred in December 2003, when the rates recorded a sharp increase but recovered within a short period (between survey intervals) to pre-blasting conditions. This pattern continued until about April 2004, when the recovery rates were much slower and did not reduce to the pre-blast rate.

This is especially evident in prisms 4229 and 4330 and representative of the area between section line 23E and 24E, the area that was considered as the toppling failure area. The strain levels of these two prisms in April are 1.4 and 1.5% respectively. The associated increase in total displacement is reflected in the time-displacement graph in Fig A7.

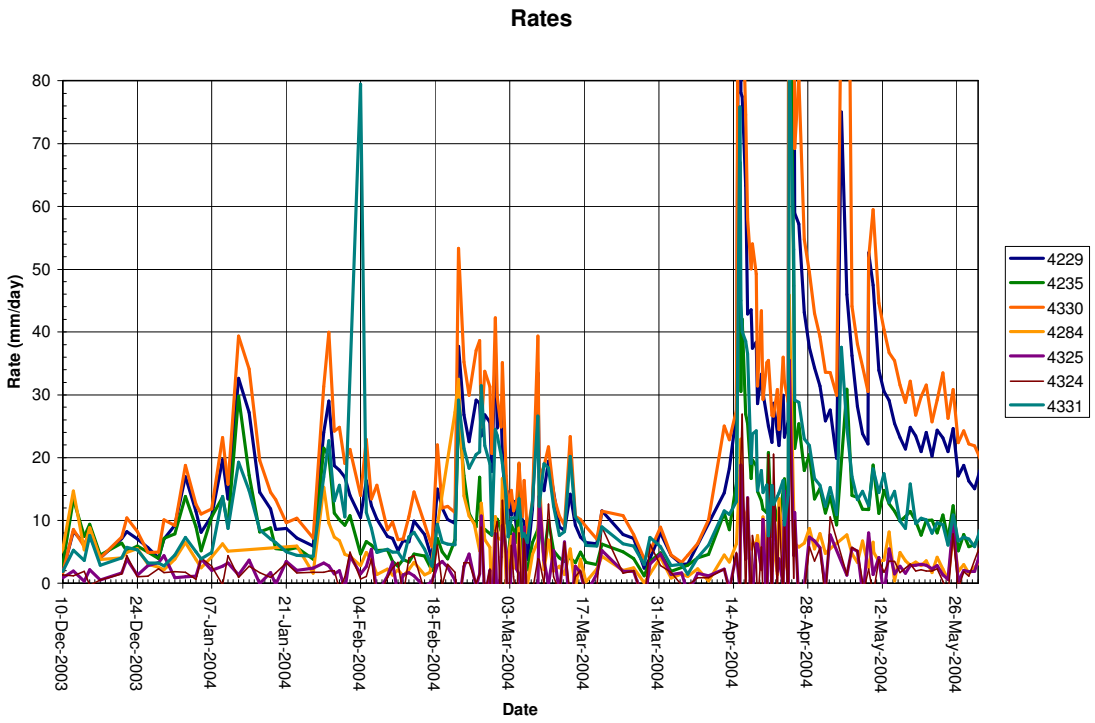


Figure 36 Effects of blasting on movement rates. The peaks represent the movement rates following a blast.

With the introduction of the Slope Stability Radar (SSR) in May 2004 the survey resources were utilised in other areas of the pit, and less survey data is available for the period after May 2004. The SSR data was thus analysed to identify the onset of progressive movement in the planar failure zone. Analyses of the SSR data indicate a relatively constant movement rate – see Figure 37. A slight increase in the movement rate can be identified in early July and could be interpreted as the onset of progressive movement. A more pronounced jump in the movement rate was triggered by a blast on 12 July. However, it can be argued that this blast only accelerated the failure and the "point of no return", i.e. onset of progressive movement, had already been reached at this stage. The end of June will thus be treated as the onset of progressive movement for the planar section of the failure.

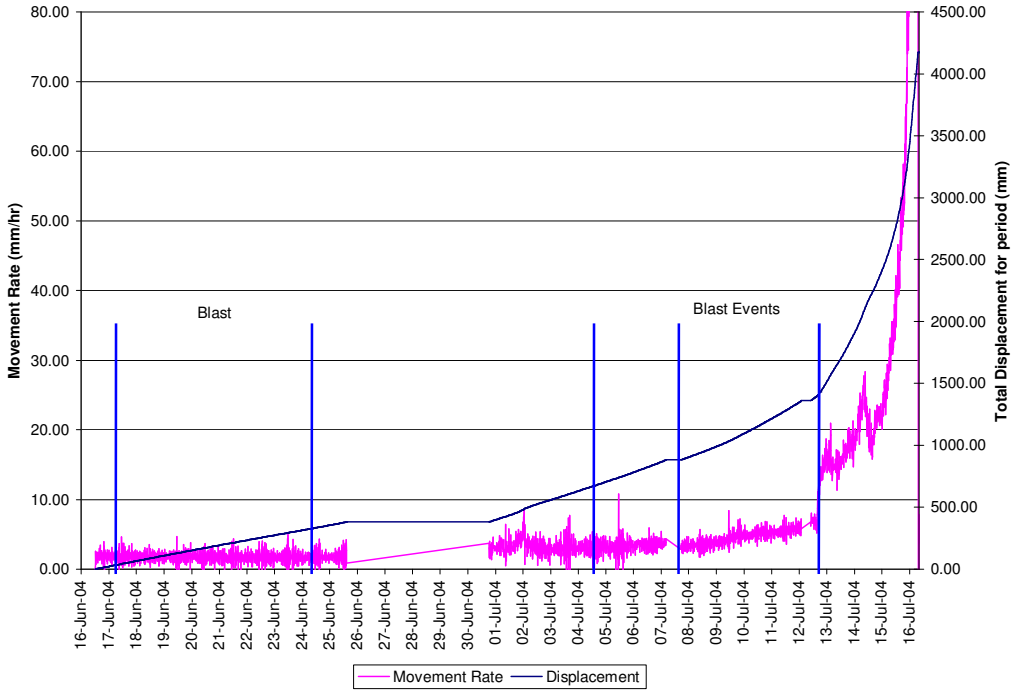


Figure 37 SSR data for the period 16 June to 16 July 2004. The smooth blue line indicates total displacement while the uneven purple line indicates movement rate in mm/hour

The calculated strain for the period April to June 2004 is plotted in Figure 38. Note the following:

- Prisms 4224 and 4328 recorded less than 0.3% strain. Both these prisms, especially 4224, are located away from the failure and, although they recorded some displacement due to the failure, should be considered as prisms in semi – stable ground.
- Prisms 4223 and 4329 recorded strain levels varying between 0.3 and 0.6% during this period. Both the prisms were located west of section line 23E, where planar failure was the overriding mechanism, and the onset of progressive movement in this area has been identified as late June early July 2004 based on the analyses of SSR results. The strain levels recorded by these prisms in June were 0.6 and 0.5% respectively.
- The strain levels recorded by prisms located between section lines 23E and 24E, for the period April to June 2004, varies between 1 and 2.6%. However, by

analysing the survey data and the response to blasting, it would appear as if the onset of progressive movement was in April 2004. The lower strain levels, i.e. 1.0 – 1.5%, in prisms 4330, 4229 and 4331 should thus be considered as the applicable strain levels.

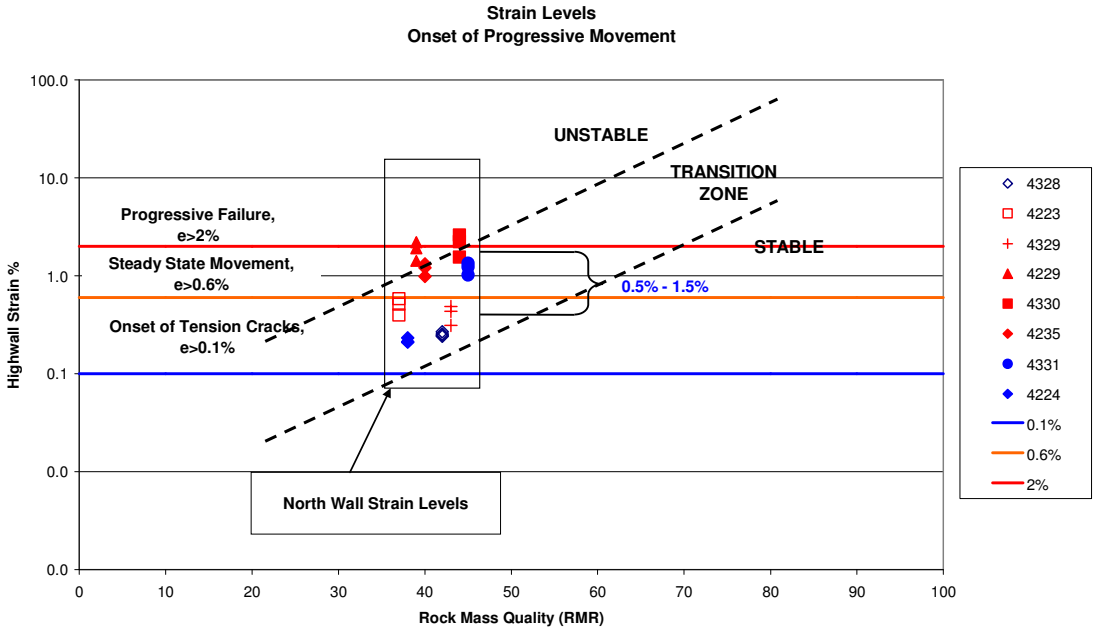


Figure 38 Strain rates at onset of progressive movement

In summary it would thus appear as if strain levels of between 0.5 and 0.6% (planar failure) and 1 to 1.5% (toppling failure) could be indicative of the onset of progressive movement.

3.5 Failure

The failure occurred on 16 July 2004. The last survey of the monitoring prisms was taken within the week prior to failure. The displacements are listed in Table 7. It is interesting to note that some monitoring prisms recorded up to 8m of total displacement, including the adjustments described earlier.

The failure mechanism described in Section 2.7 is a combination of toppling and planar failure. The strain levels recorded by the survey prisms in July (two days prior to failure) are shown in Figure 39 and in tabulated in Table 9.

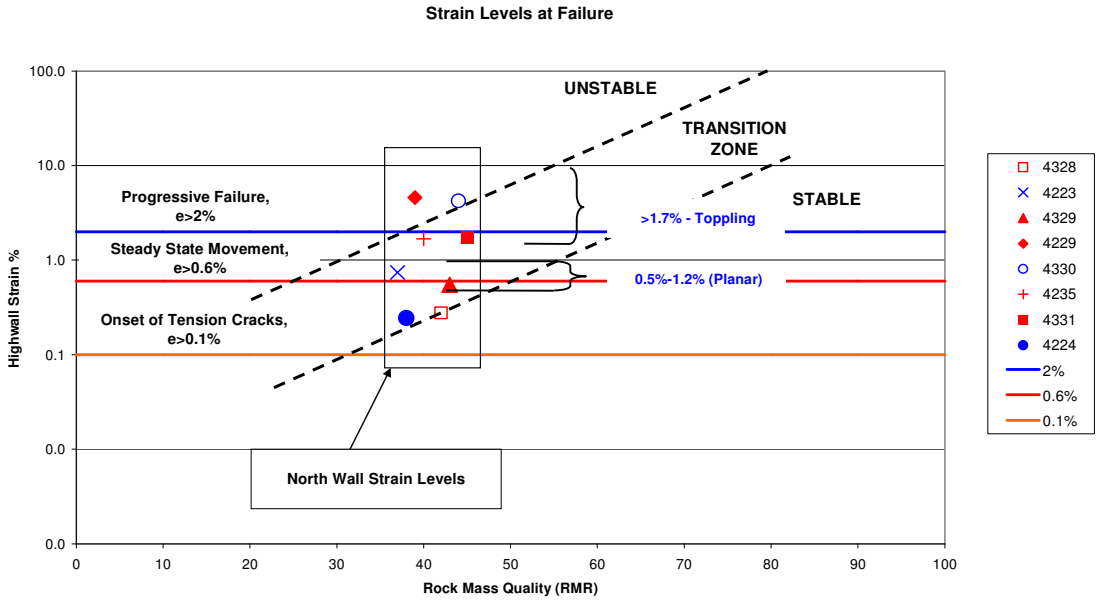


Figure 39 Strain rates at failure

According to Brox and Newcomen (2003) a toppling failure can result in relatively high strain value before collapse, while a plane/wedge failure has a much lower value. This would explain the wide range of strain levels at failure. Monitoring prisms 4229, 4235 and 4330 were located in an area where toppling was experienced. The strain levels for these prisms vary from 1.7 to 4.6%.

Monitoring prisms MP 4329 and MP 4223 were located further west where the planar failure was the main failure mechanism. The strain levels experienced by these two prisms are 0.5% and 1.7% respectively.

3.6 Results of Strain comparison

The High Wall Strain Level Criteria proposed by various authors were evaluated against survey results from monitoring of the north wall. The results are summarised

in Table 10. A distinction is made in the north wall results between the prisms that were located in an area that experienced a toppling failure mechanism and the prisms in a planar type failure.

Cracks formed in the north wall at strain levels that are higher than those proposed by Brox and Newcomen (2003), but in line with Zavodni's (2001) proposal. It was more difficult to identify the period when progressive movement commenced. However, it would seem as if the strain levels in the north wall during this stage of failure development were less than both the models.

	North Wall	**Brox and Newcomen (2003)**	**Zavodni (2001)**	
Highwall Stability Stage	**Threshold Strain Level, %**	**Threshold Strain Level, %)**	**Highwall Stability Stage**	**Threshold Strain Level, %**
Tension Cracks	Planar : 0.2 Toppling: 0.3–0.4	~ 0.1	Regressive movement	< 0.5
Progressive Movement	Planar: 0.5-0.6 Toppling: 1-1.5	~ 0.6	Increase in movement rates	> 0.5
Imminent Failure/Collapse	Planar: 0.5 - 0.7 Toppling : >1.7	>2.0	Slope acceleration (results in collapse)	1.0 - 2.0

Table 10 Suggested Strain Levels for the north wall, compared to those of other authors

During the failure phase the north wall strain associated with the planar failure is very similar to that during the progressive phase. The strain levels are also much lower that those suggested by Brox and Newcomen (2003) and Zavodni (2001). In the case of the toppling failure strain levels, the north wall levels are much lower than suggested by both models. The strain levels recorded by the north wall prisms for the two different modes of failure are summarised in Figure 40.

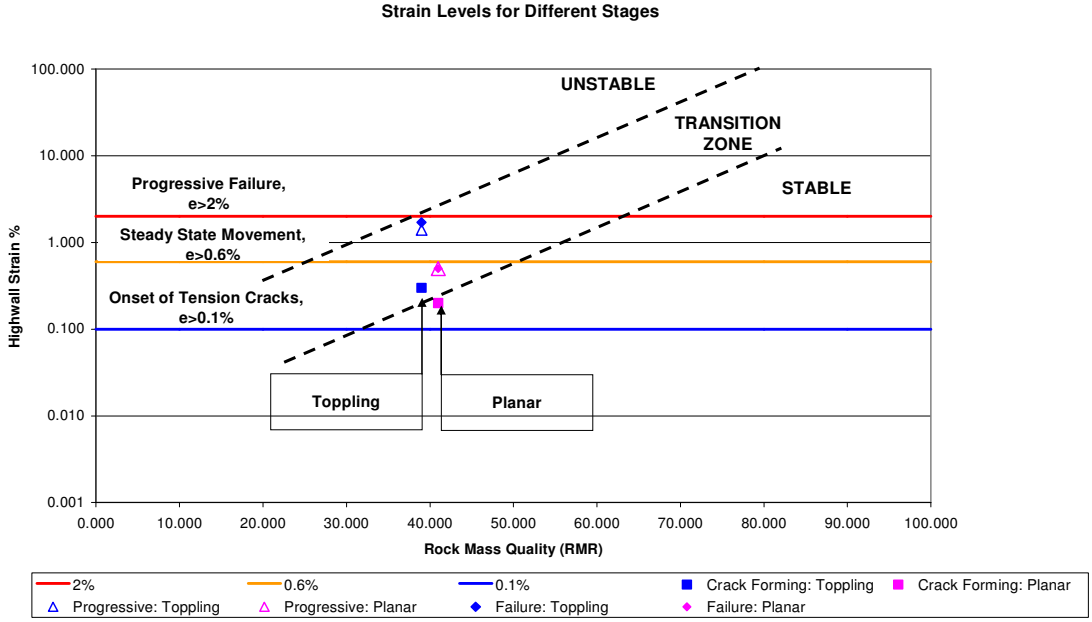


Figure 40 North wall strain levels for different stages

From Figure 40 the following is evident:

- All strain levels are within the transition zone of the Brox and Newcomen (2003) model.
- Cracks appeared at the same strain levels for both failure modes.
- There is a small difference between the strain levels at onset of progressive movement and failure, for both failure modes.
- Failure strain levels in both cases are lower than the strain levels proposed by the model.

Based on the above it is not recommended that the strain model be used for the prediction of failures. If more data can be collated and the model populated it might be possible to differentiate between the different stages. At this stage the strain levels are all within a relatively narrow zone. The visual inspection and monitoring results did provide more useful information and provided a better decision making tool than the strain models.

4 SUMMARY AND CONCLUSIONS

The development of the north wall failure at the mine site was closely monitored from August 2003 until eventual collapse in July 2004. During this time mining was active below the developing failure. Although other mining areas were available for ore extraction, the majority of the ore from the open pits was mined from this part of the part pit. Loss of ore would have had a detrimental effect on production. By close monitoring of the failure and adapting to changing conditions, all the ore was safely mined from below the "Area of Concern".

The monitoring systems were changed and improved with time. Survey monitoring started off with the weekly survey of prisms that were installed on a standard pattern. The frequency progressed through a daily survey to four distance readings per hour. Prism coverage was improved when additional prisms were installed based on visual inspections and survey results. The traditional survey methods were replaced with a state-of-the-art radar system, which provided almost real time slope surveys. The technology played an important role in the management of the failure and enabled the mine to recover all the ore from below the area of concern, in a safe environment.

Analyses of the survey results also evolved. Initially the results were analysed using time-displacement graphs. When displacement increased, instantaneous rates were found to be more effective. Trigger levels were determined from the back analyses of of a smaller failure provided. During the week prior to failure when the evacuation trigger level had been exceeded, the change in the movement rate was used to monitor the failure. Although some attempts were made to predict the time of the failure, this was not pursued due to the high risk involved with personnel working below the failure.

Managerial and operating controls were put in place and changed to mitigate the increasing levels of risk. An important part of the management strategy was to keep the workforce fully informed. A few years prior to this event a catastrophic failure killed a number of people in the same pit. The workforce was very nervous that this could happen again. Their confidence in the management of the situation was improved with the visual evidence of the thorough management of the situation, in addition to the regular feedback sessions. The visual evidence included increased visual inspections involving senior members of the Geotechnical team and improved monitoring systems. The Slope Stability Radar (SSR) played an important part in the latter.

Slope Strain was not used during the monitoring of the failure, but was subsequently evaluated as a failure prediction tool. It was found that the strain levels associated with the different stages of a developing failure were too close to each other and not distinctive enough to provide a practical predictive tool.

All of the above assisted in the decision making process to ensure the safe mining below a marginally stable slope. The movement trigger levels that were determined during the failure appear to be applicable to the slope conditions. They should however not be used blindly, but always in conjunction with visual inspections and careful analyses of the trend of movement rates

This case study has shown how trigger levels and monitoring systems can be escalated in response to increased slope instability. It is a good example of how systems and procedures could be adapted in response to the dynamic environment of a developing failure. This report is a valuable contribution to the literature dealing with the monitoring and management of slope instability, since the detailed information presented in the report may be of use to other researchers and practitioners in this field.

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APPENDICES

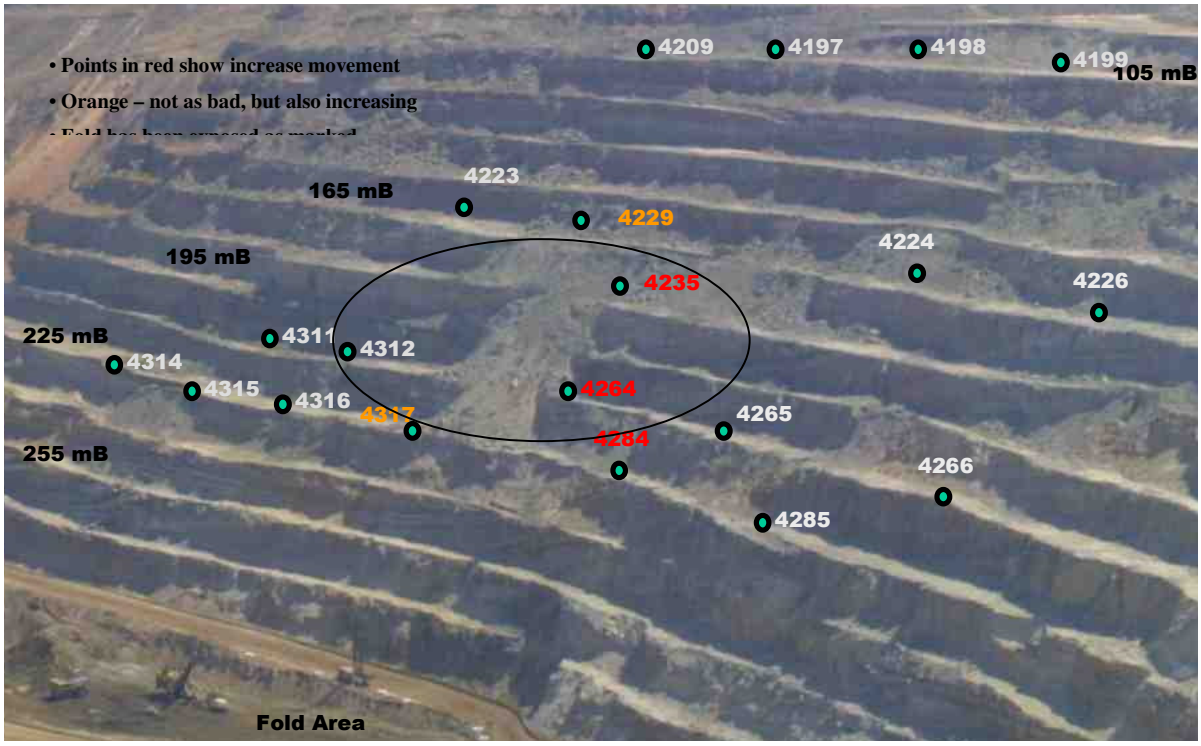


Fig A1 Area of Concern as at 5 August 2003. Note position of Monitoring prisms (those in red recorded increased movement rates)

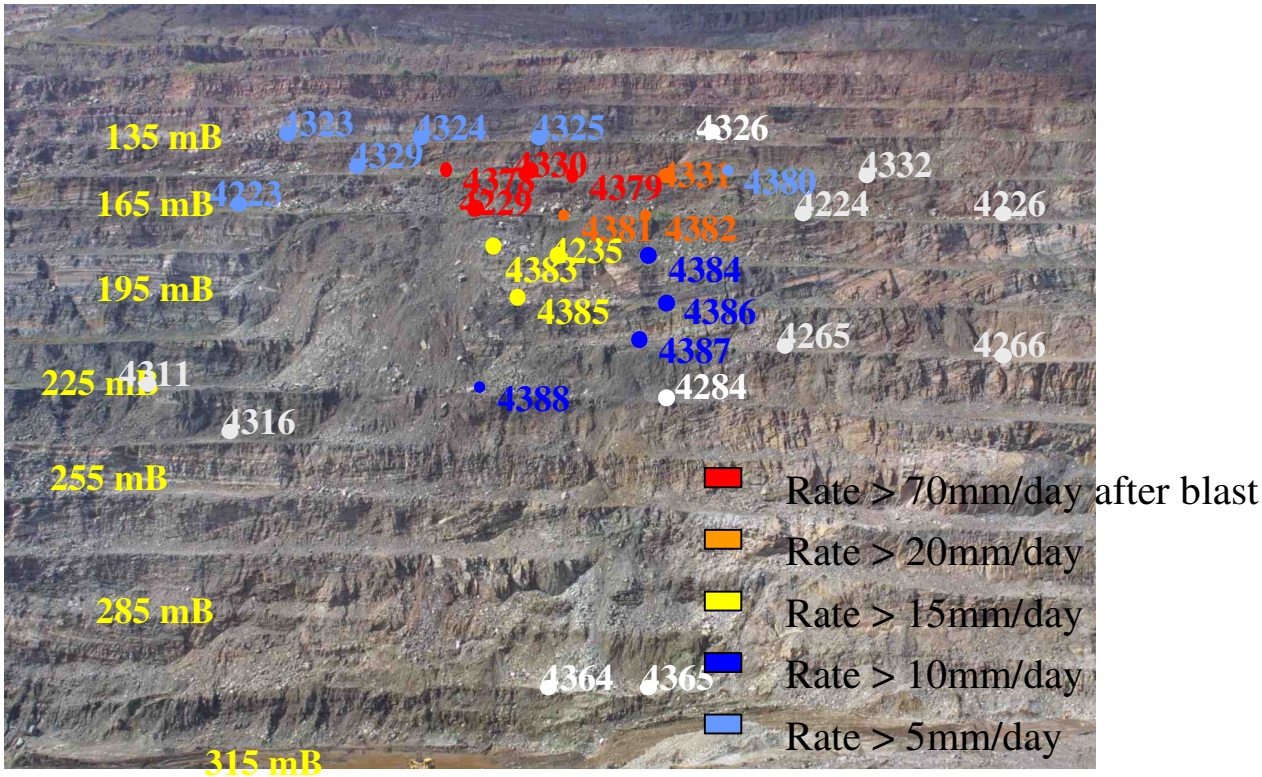


Fig A2 AOC on 29 April 2004.

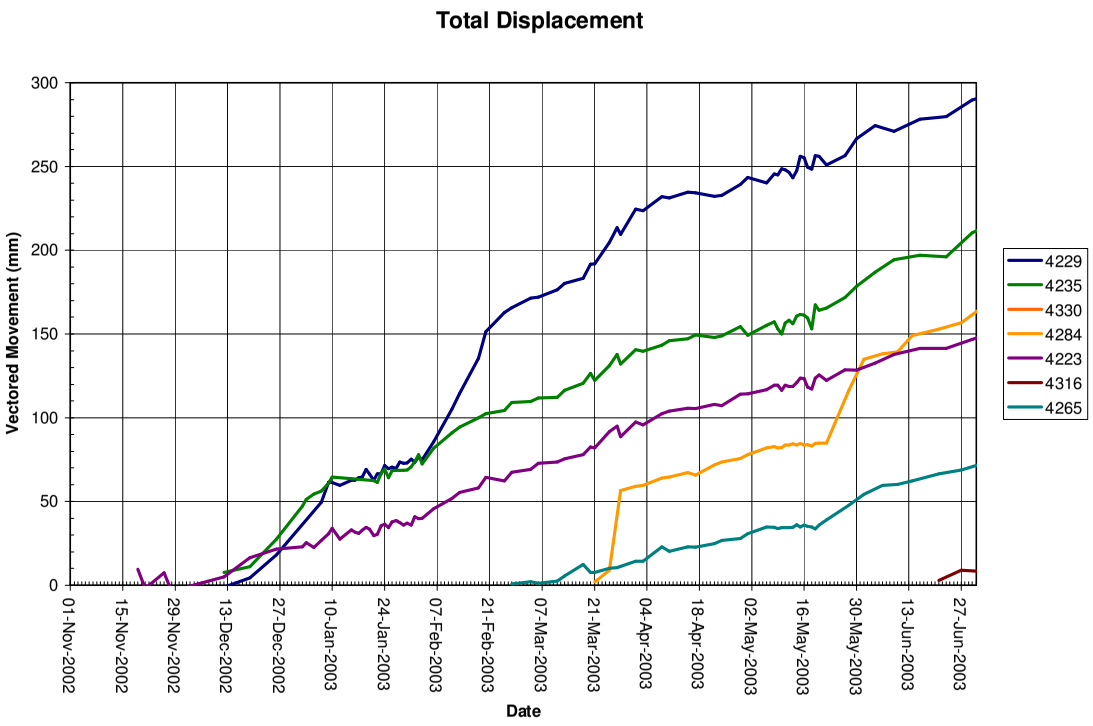


Fig A3 Total Displacement : Pre-July 2003

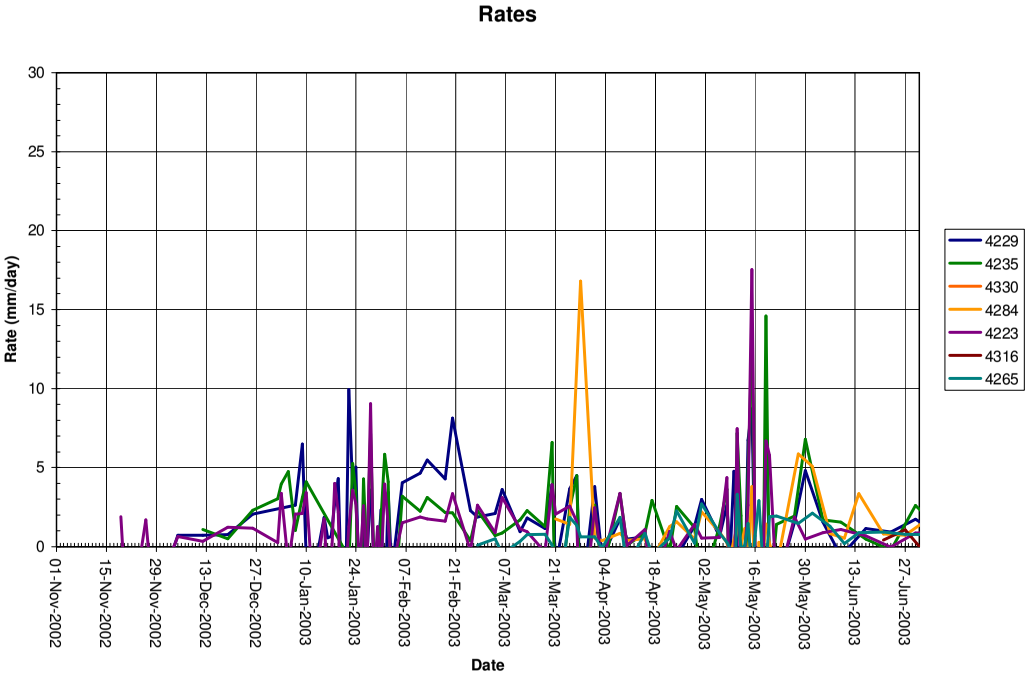


Fig A4 Movement Rate : Pre-July 2003

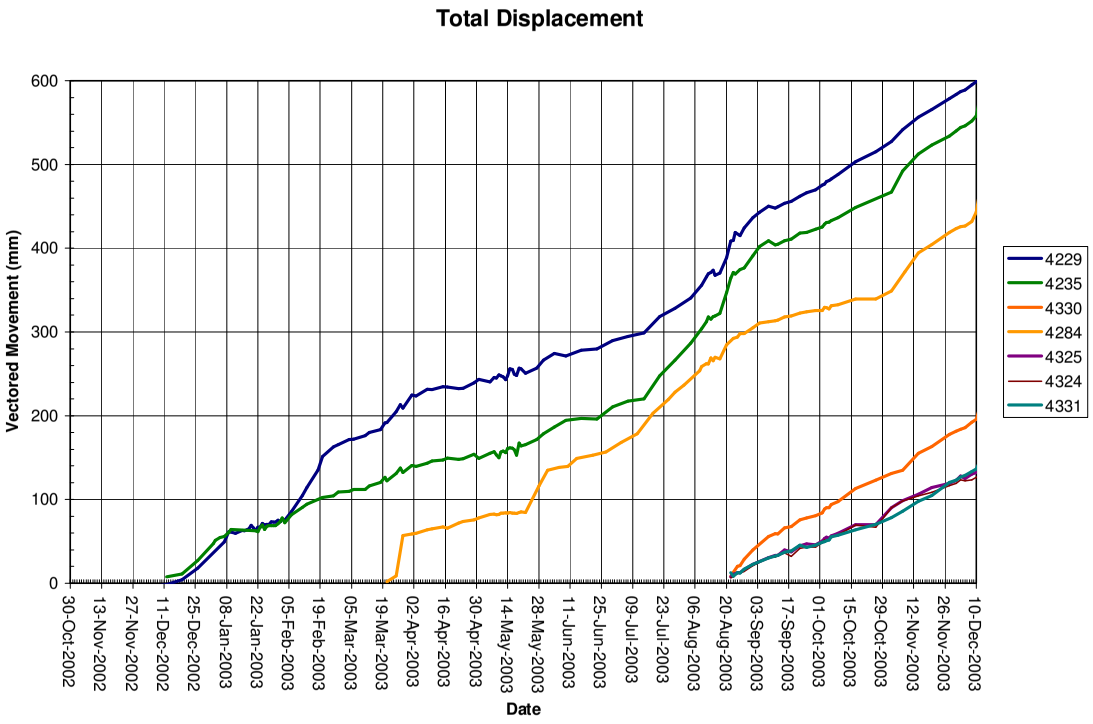


Fig A5 Total Displacement : July 2003 - 10 December 2003

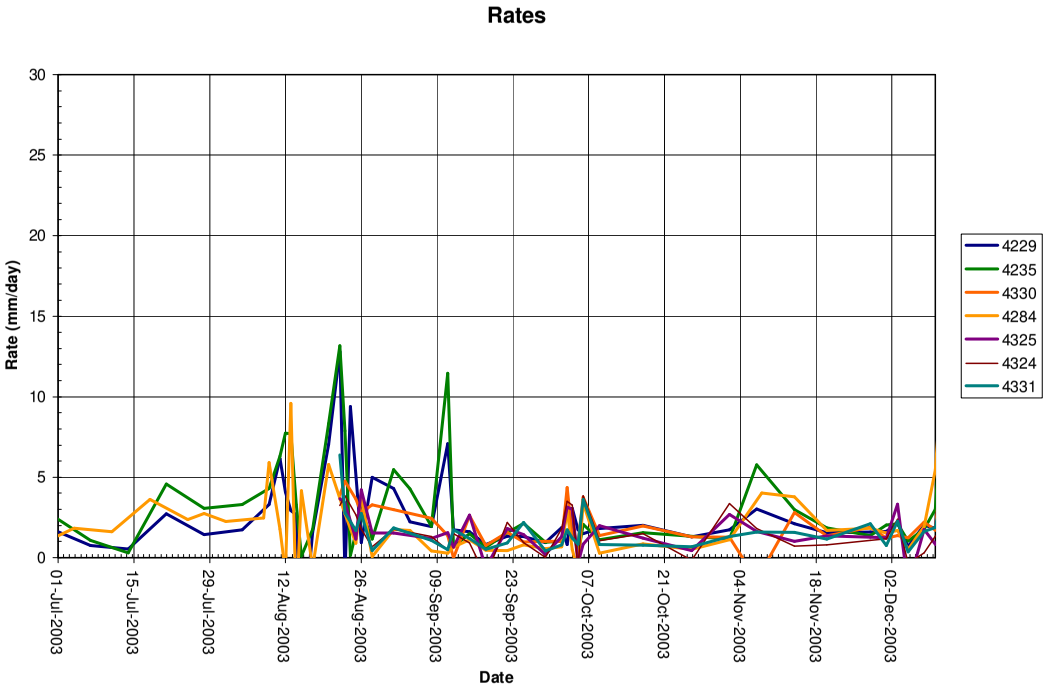


Fig A6 Movement Rate : July 2003 - 10 December 2003

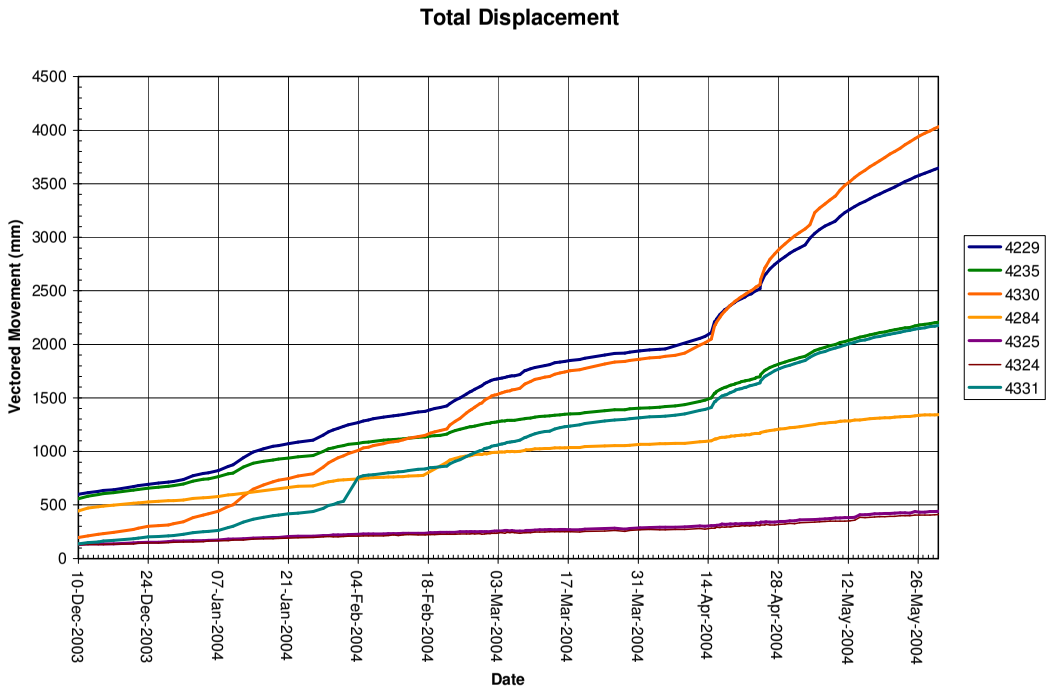


Fig A7 Total Displacement : 10 December 2003 - 19 May 2004

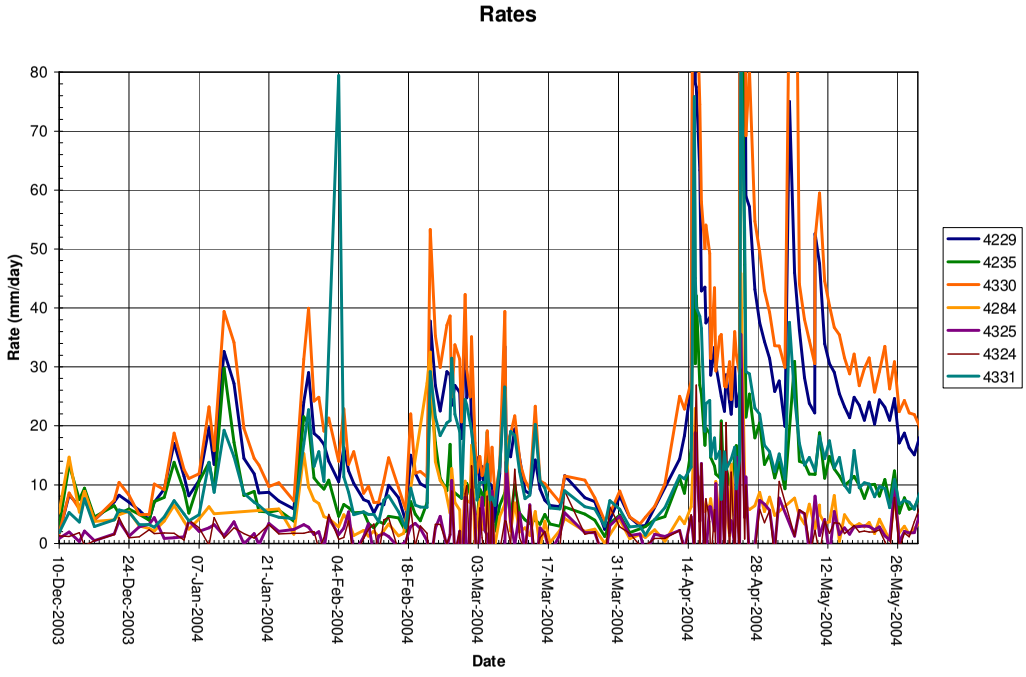


Fig A8 Movement Rate : 10 December 2003 - 29 May 2004

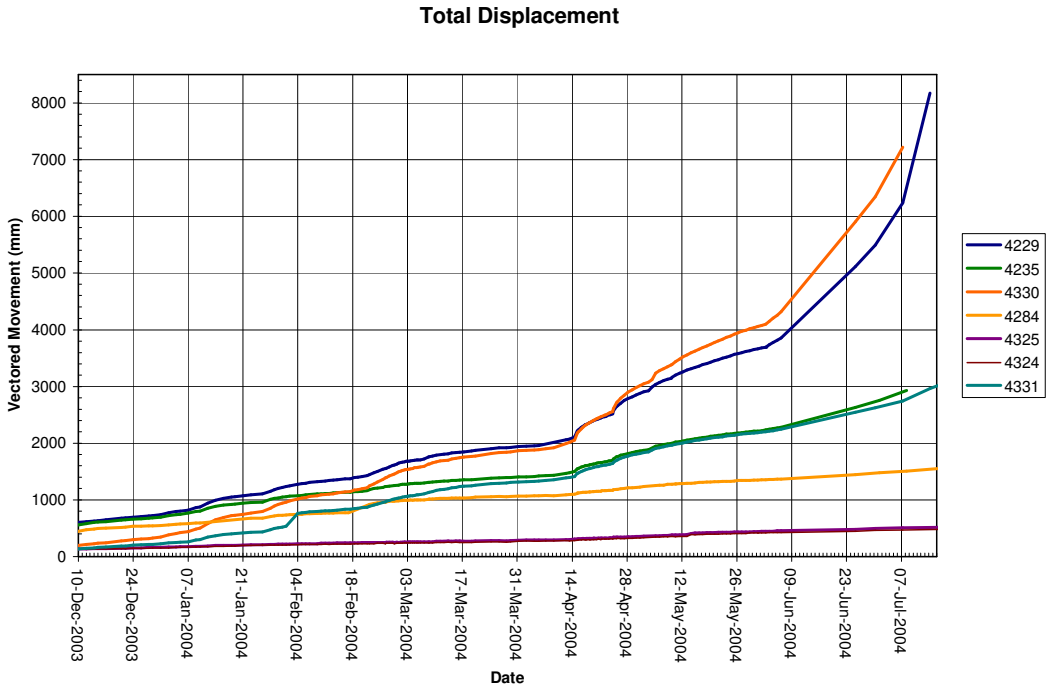


Fig A9 Total Displacement of monitoring prisms as at 7 July 2004

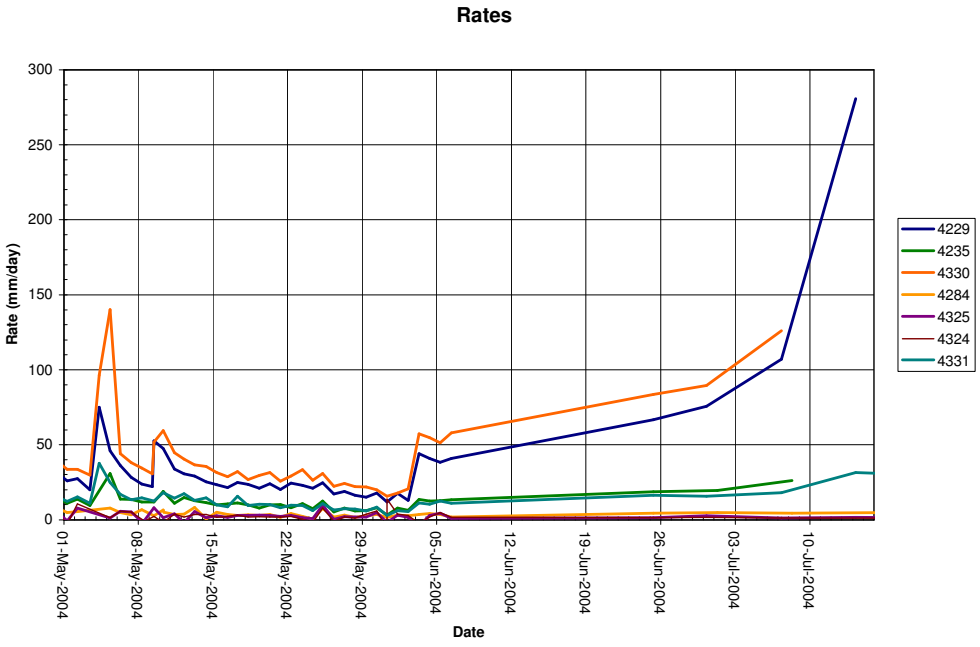


Fig A10 Movement Rates between May 2004 -7July 2004. Note that few surveys were done due to the deployment of the radar

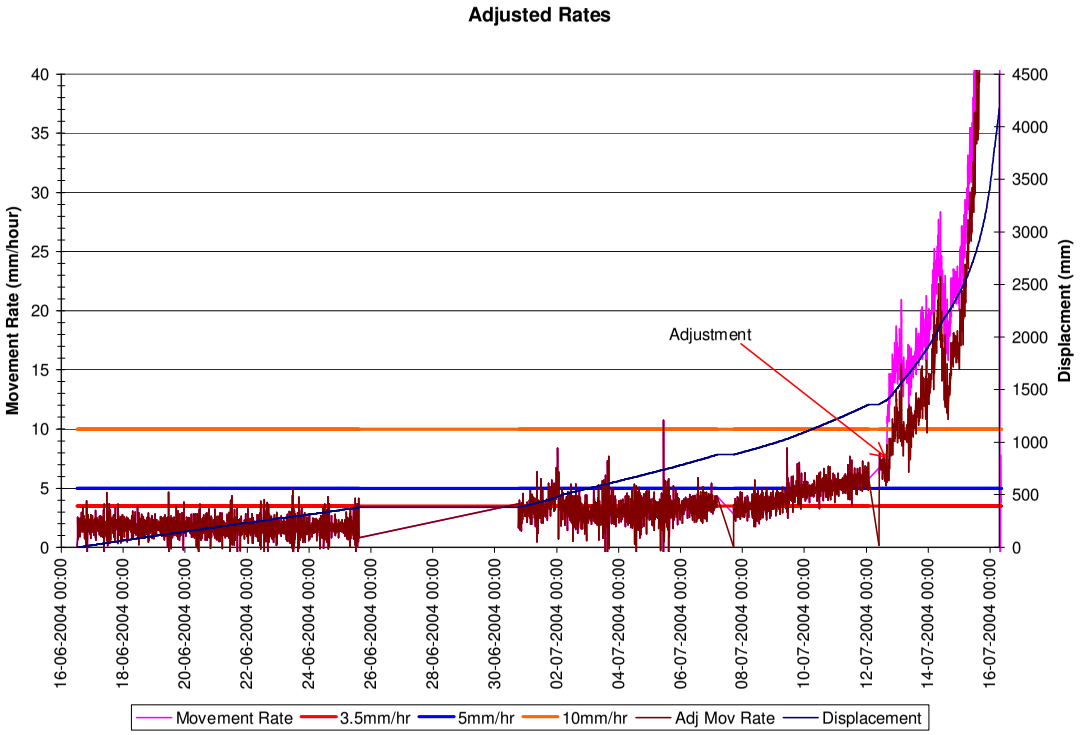


Fig A11 SSR results showing movement rates and total displacement for the period 16 June - 16 July. The adjusted rates are discussed in the text.