PUSHBACK 8 SOUTH – A CASE STUDY IN PIT SLOPE MANAGEMENT

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ABSTRACT

Pushback 8 South is the final cut back on the south wall of the East Pit at BHPBilliton's Mt Whaleback iron ore mine in the Pilbara region of Western Australia. Development of the cutback commenced in 1999 and is now nearing completion with ultimate pit walls up to 300m high.

This paper describes the geotechnical, structural and hydrogeological setting of PB8S, reconciliation between design and slope performance and the slope engineering techniques used to effectively manage the risks associated with the slope. These techniques include slope buttressing and sequential mining with waste backfilling, as well as a detailed program of displacement and hydrogeological monitoring, structural mapping and slope stability assessment. During the course of slope formation recent developments in remote slope displacement monitoring using radar and remote structural mapping using terrestrial photogrammetry have also been applied.

Aspects of the PB8S slope management plan are relevant to many large scale open pit slopes. This paper provides geotechnical practitioners with information which may be helpful in managing geotechnical risk in similar environments.

1.1 INTRODUCTION

The Mt Whaleback open pit mine, operated by BHPBilliton Iron Ore Pty Ltd (BHPBIO), is situated at Newman, approximately 1200 km north of Perth in the Pilbara region of Western Australia. Total material movement from the mine is approximately 120 million tonnes (Mt) per annum and increasing.

The Mt Whaleback mine has been in operation for more than 30 years with the pit being developed in a series of pushbacks. The mining operation is conventional truck and shovel, with drilling and blasting performed from 15 m benches with blastholes drilled by large rotary drill rigs, with holes in the size range 251 mm to 311 mm diameter. Blasted material is normally excavated in full 15 m benches or in smaller increments (3 m or 5 m flitches) depending on the allocated loading equipment.

The ultimate pit, approximately 5.5 km long by 2 km wide and extending to depths of over 500 m (34 mined benches), is being developed as a series of pushbacks which form either interim or final pit walls. Pushback 8 South (PB8S) forms the final pushback on the south wall of the east pit, is up to 300 m high and is developed predominantly in folded and faulted shales, which form the footwall to the orebody. Development of

PB8S commenced in 1999 and is now nearing completion. Approximately 20 Mt of high grade hematite ore and 38 Mt of waste have been mined from this pushback since commencement. Once PB8S is complete, the East Pit will be used to store mine waste rock generated as other areas of the pit are mined.

2.1 GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

The Mt Whaleback orebody and therefore the development of the pit are stratigraphically controlled. Pit walls are developed in a range of rock types comprising banded iron formation (BIF), shales and associated chert, in a structurally complex environment. The complex structural geology and stratigraphy of the Mt Whaleback deposit dictate slope stability. Experience within the shale rock units, which form the majority of the PB8S slopes, confirms that a detailed knowledge of the stratigraphy and structural geology is essential for successful pit wall design.

2.1.1 Stratigraphy

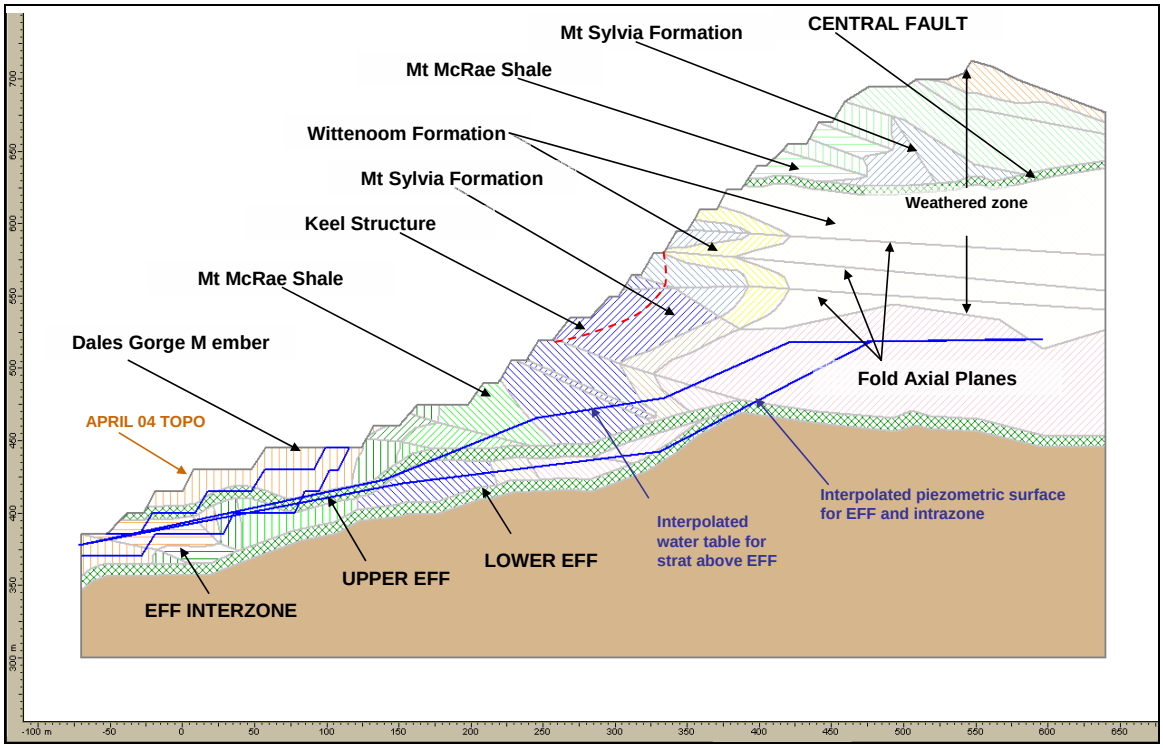
Referring to Figure 2.1, the major rock units that form the footwall to the iron enriched zone of the Brockman Iron Formation at Mt Whaleback, and which largely form the pit walls in PB8S, are the Mt McRae Shales, Mt Sylvia Formation and Wittenoom Formation.

The Mt McRae Shales form the immediate footwall to the orebody and consist of the Upper Shale (stratigraphically directly below the orebody), the Nodule Zone and the Lower Shale. The Upper Shale is often fissile and sheared, with an intact rock strength ranging from weak where sheared, to strong in undisturbed fresh rock, using the ISRM strength description convention. In the weathered zone the shales are kaolinitic, weak to moderately strong and exhibit increased bedding fissility. The Nodule Zone and Lower Shale are well bedded with strong intact rock strength in fresh rock. A number of chert bands are present in the Lower Shale with strengths in the very strong range.

The Mt Sylvia formation consists of a massive but relatively thin, very strong to extremely strong BIF, interbedded chert/shale units and a well jointed (blocky) siltstone unit. Intact rock strengths in the chert/shales and siltstones range from moderately strong to strong.

The upper member of the Wittenoom Formation is intersected in PB8S and consists of calcareous shales and minor cherts. Rock strengths are similar to those in the Mt McRae shale but bedding and jointing are generally more frequent, resulting in a poorer quality rock mass.

Figure 2.1: Typical Stratigraphy in PB8S Area



2.1.2 Structural Geology

Several phases of deformation dictate the structural environment in PB8S (Figure 2.1). The main folding phase (F2) includes large and small scale open to layer parallel folds with axial planes striking approximately east-west and plunging to the west. Two large, low angle faults – The Central Fault (CF) and Eastern Footwall Fault (EFF) – cut the main F2 fold set which, where they dip at shallow angles to the north and are daylighted in the pit wall, form potential planes of weakness along which sliding may develop. A set of faults dipping between 30° and 40° to the northwest (known as 320 Faults as their dip direction is on a bearing of approximately 320°) and having continuities between 20 m and 100 m and small throws (~1m) also influences slope stability. The shear strength along these structures, where developed in shale, is similar to that of the shale bedding planes.

The other major influence on batter and multiple bench stability is bedding plane orientation. This is particularly important in the shale units where planar polished bedding surfaces are common. Bedding plane shear strengths in the shale units are often planar, polished and very weak with friction angles in the 18° to 25° range for small (laboratory scale) specimens, with little cohesion.

At the commencement of PB8S in 1999 the geometry of the folding and large low angle faults was known with some confidence as a result of extensive face mapping on previous pushbacks and geotechnical and structural drilling programmes. However, as mining progressed it became apparent that the presence of 320 Faults had not been well

known and the identification of additional 320 Faults has been the principal change to the geological/structural model since mining commenced.

A critical component of the successful management of PB8S has been the high quality structural model for the area. This model is computer based and allows the three dimensional interaction of structure with the pit slopes to be readily assessed. This model has been developed by the BHPBilliton Structural Geology Group at Mt Whaleback. Updates of the PB8S model, based on three dimensional terrestrial photogrammetric pit wall mapping, have been undertaken regularly up to two or three times a year over the life of the pushback.

2.1.3 Hydrogeology

Trials conducted in 1994 to investigate the effectiveness of sub-horizontal drain holes in the footwall shale units had indicated that depressurisation of the footwall shales was possible. These trials also indicated that recharge during the wet season, between January and May, can be significant.

The level of depressurisation achieved is a critical factor influencing the many structurally controlled failure mechanisms identified in PB8S. Sub-horizontal drain holes have been installed along the entire length of the lower benches of PB8S as access has become available. Where possible, long (250 m to 300 m) drain holes have been drilled through the PB8S ore from the preceding pushback at 20 m to 40 m spacings. A number of both horizontal and vertical single and nested piezometers have been installed during the mining of PB8S, the data from which suggest that a complex hydrogeological regime exists. This may be approximated by a relatively non-transmissive shale rock mass above the EFF subject to water table generated pore pressures and a more transmissive EFF and intra fault zone subject to lower, separate piezometric surfaces (see Figures 2.1 and 2.2).

The current hydrogeological model has water draining towards the north (out of the slope) along the EFF into the readily dewatered ore in the base of the pit. This is aided by the action of the horizontal drain holes, with minor underdrainage of the relatively non-transmissive shales overlying the fault zone.

Further complicating the hydrogeological regime are rainfall induced peaks in piezometric pressures following heavy rainfall events (Figure 2.3). Very rapid increases in piezometric heads of up to 70 m have been recorded in the centre of the pushback after heavy rainfall events followed by more gradual returns to pre-rainfall levels. The possibility of surface water ingress past the piezometer seals is being investigated where these pressure peaks are unusually high and occur very rapidly.

In general terms the depressurisation of PB8S has been partially successful. Significant ground water pressures are still acting at the deepest, western end of the pushback with pressure spiking above the design depressurisation target of the UEEF during the wet season.

Figure 2.2: Piezometric Monitoring at the Toe of PB8S

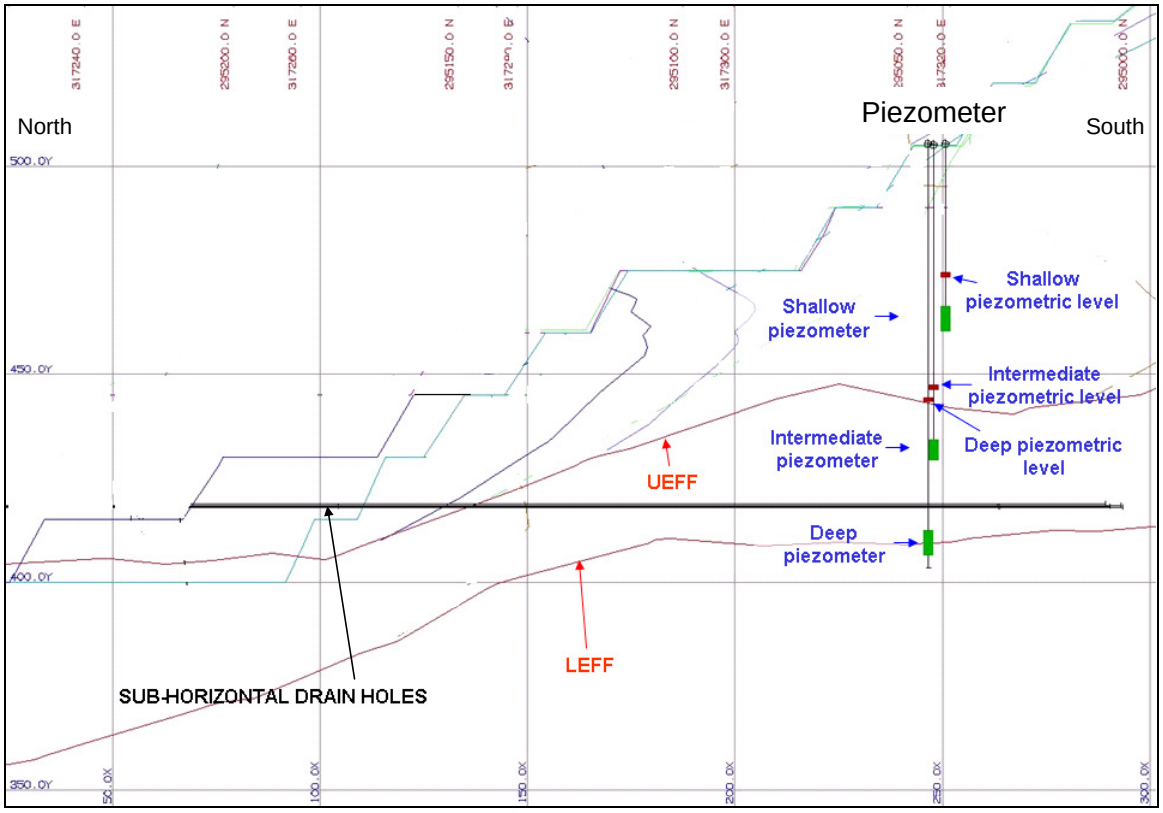
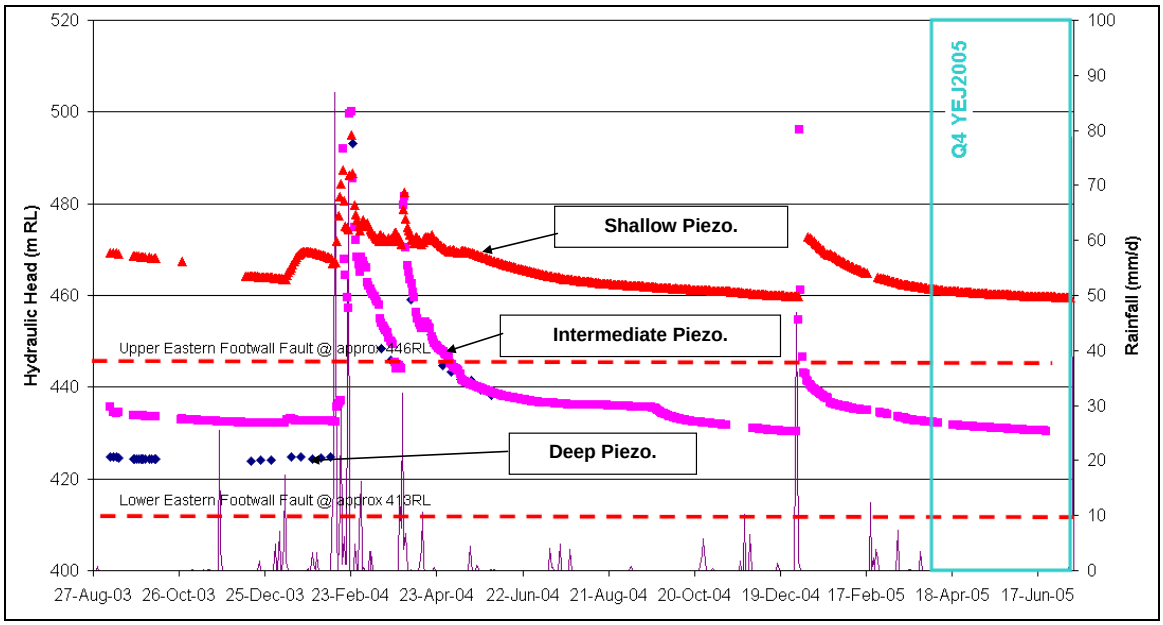


Figure 2.3: Water Level Data from Nested Piezometer showing reaction to seasonal rainfall events



2.1.4 Geotechnical Model

A number of potential failure mechanisms and paths exist in PB8S and include Keel structures[1], 320 Faults, the Upper and Lower EFF and low-angle bedding. The likelihood of instability developing by these mechanisms has been assessed using two dimensional limit equilibrium methods. Hence, since 1999, the development of the geotechnical model for PB8S has, to some extent, been driven by the functionality and input requirements of the software used, at the time, to undertake these stability analyses. An important characteristic of the sedimentary rock masses at Mt Whaleback is the anisotropic nature of the rock mass shear strength introduced by the persistent and weak bedding planes, particularly within the shale units. That is, shear strength parallel to bedding will be dictated by the shear strength of the bedding planes and strength normal to bedding will be dictated by the rock mass. As such, all stability analyses involving the bedded rock units at Mt Whaleback are now undertaken using anisotropic rock strength models, which are readily available in a number of popular limit equilibrium analysis programmes. The relationship describing the transition from rock mass to bedding plane strength is currently being investigated with the aid of numerical experiments using UDEC[2].

The Barton defect shear strength criterion and the Hoek-Brown rock mass strength criterion are used to establish strength parallel and normal to bedding respectively. Input parameters for these models have been determined by laboratory testing, back analysis of existing slopes and instabilities, and pit wall mapping. An undisturbed rock mass is assumed in the estimation of rock mass shear strength parameters using the Hoek-Brown criterion, as these parameters better approximate actual slope performance where large scale (overall slope) failure mechanisms are under investigation,.

In determining bedding plane shear strengths, an understanding is required of the differing scales of folding and how these will be modelled. Planar, continuous bedding surfaces exhibit lower strengths in shear, as compared to undulating surfaces or surfaces exhibiting small scale parasitic folding. Tight drilling control and face mapping in the East Pit has allowed folding at the bench scale and larger to be modelled in the geological/structural model and has allowed this to be incorporated explicitly in limit equilibrium stability analyses. Bedding undulations and folding at smaller scales are accounted for by means of the Joint Roughness Coefficient (JRC) – at the appropriate scale – in the Barton defect shear strength criterion. Face mapping has allowed appropriate JRC values to be determined at various scales for the various rock materials and on the various fold limbs.

Fault strengths have been estimated based on back analysis of a number of multi-bench instabilities that have previously occurred on the south wall of the East Pit. However, very large, fault controlled, overall slope instabilities are not available for back analysis.

[1] Keel structures are formed at fold hinges where the overturned limb dips into the slope, towards the south, and the normal limb of the fold dips to the north, forming potential sliding surfaces where bedding is undercut by the pit wall, as illustrated in Figure 2.1
[2] UDEC: Universal Distinct Element Code developed by Itasca Consulting Group.

3.1 SLOPE DESIGN AND PERFORMANCE

3.1.1 Design

A number of slope designs have been proposed for the south wall of the East Pit after a large scale, multi-bench instability developed on this wall in 1989. The strategy adopted by BHPBIO for mining the remaining ore in the East Pit required a full re-strip of the slope. A design implementing this strategy – PB8S - was developed, which would mine out all the known 320 Faults. The overall slope configuration would follow moderate north-dipping bedding in the lower slopes with multiple batter wall angles of approximately 38° in the upper slopes.

A number of potential pit wall instability mechanisms were identified in this design including:

- overall slope failure along the EFF and through the upper rock mass;
- multiple batter failures along the EFF and bedding in the Mt McRae shales at the toe of the slope;
- multiple batter failures along keel structures in shale as a result of undercutting bedding at the toe of the keel structure; and,
- batter scale failures in all units where bedding is undercut.

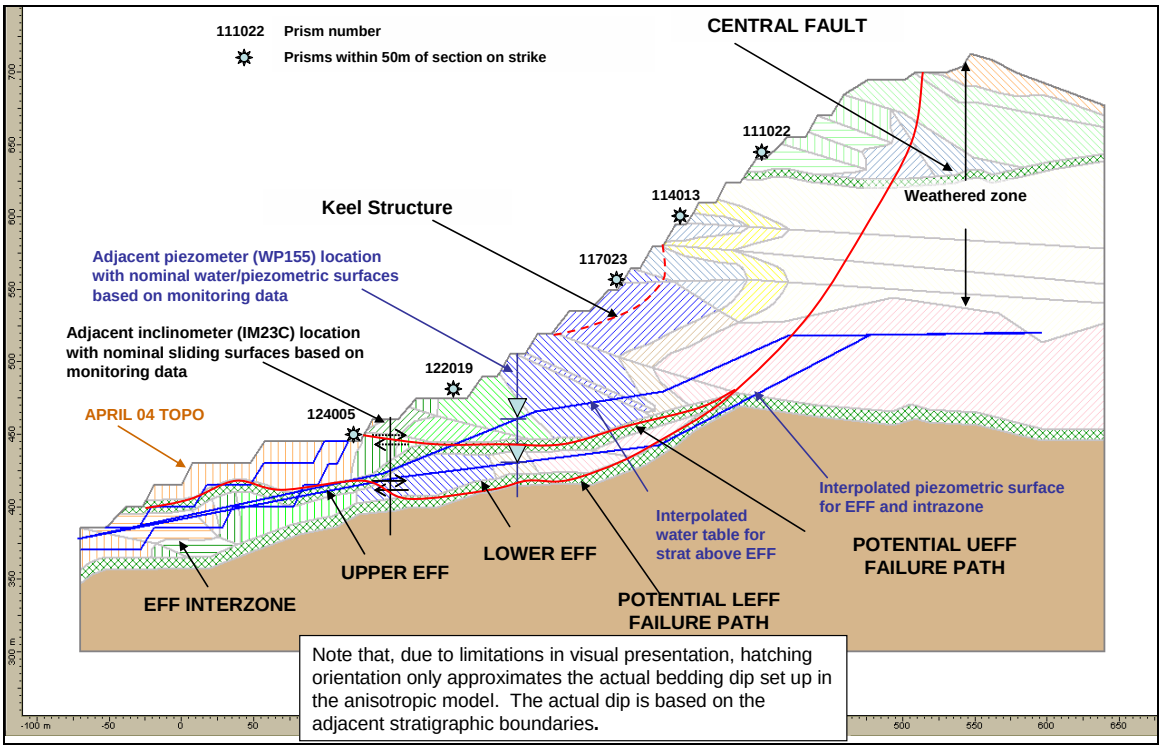
Figure 3.1 presents a typical section through PB8S illustrating the geology and potential instability mechanisms.

Analysis undertaken in 1999 indicated that implementation of the ultimate PB8S design would result in low factors of safety (FoS) for several instability mechanisms involving sliding along the EFF in the toe of the ultimate slope. Options considered to improve slope stability in these areas were over-mining of the toe of the slope (an unloading option) or leaving either permanent or temporary toe buttresses, providing confinement to the major fault structures at the slope toe. Over-mining the toe would push the entire slope further to the south and require significant additional waste mining. Two dimensional limit equilibrium stability analyses determined that maximum buttress dimensions of the order of 50 m wide on up to four benches high would be required, depending on the degree of depressurisation achieved behind the slope.
Leaving permanent buttresses would sterilise significant quantities of ore. However, the use of temporary buttresses, to be recovered on completion of PB8S, would maximise ore recovery, and this was the adopted approach.

Re-assessment of the buttress requirements was undertaken in late 2003, prior to the final buttress design being implemented. This assessment used the piezometric monitoring data available at that time to establish the ground water levels to be used in the stability modelling. These analyses, in conjunction with the requirements to maintain minimum mining widths and access, resulted in the current design, in which 3.5 Mt of ore remains in 30 m to 60 m wide buttresses on the lower six benches of the pit (Figure 3.2).

Subsequently, a 2004 review of the stability of the slope indicated that it was performing largely as anticipated. However, associated analysis of the effects of buttress removal indicated that these would need be recovered using a specialised mining approach in order to maintain the long-term stability of the slope and infrastructure at the slope crest whilst the ore in the buttresses are mined. This approach comprises, , waste fill buttresses, which have been designed and are being incrementally developed as the ore buttresses are removed at the toe of the slope.

Figure 3.1: Representative Section through PB8S Illustrating Potential Overall Slope Failure Mechanism



A mining and filling schedule has been developed to remove the buttresses from the west towards the east. This schedule is designed to daylight only limited sections of the EFF at any one time before the void created by removal of the ore buttress is back filled by the waste buttress. The exact mining schedule followed will be dictated by the performance of the slope during buttress extraction. Figure 3.3 illustrates the buttress mining and waste filling operation as at November 2005.

Slope performance to date has been good, with only one, 30 m high double bench slip (known as the "Feb13, 2004" slip, estimated to be 5 m thick and 20 m along strike, and approximately 8 kt), and several bench scale instabilities of very limited strike length developing to date.

Figure 3.2: Buttress Layout, PB8S

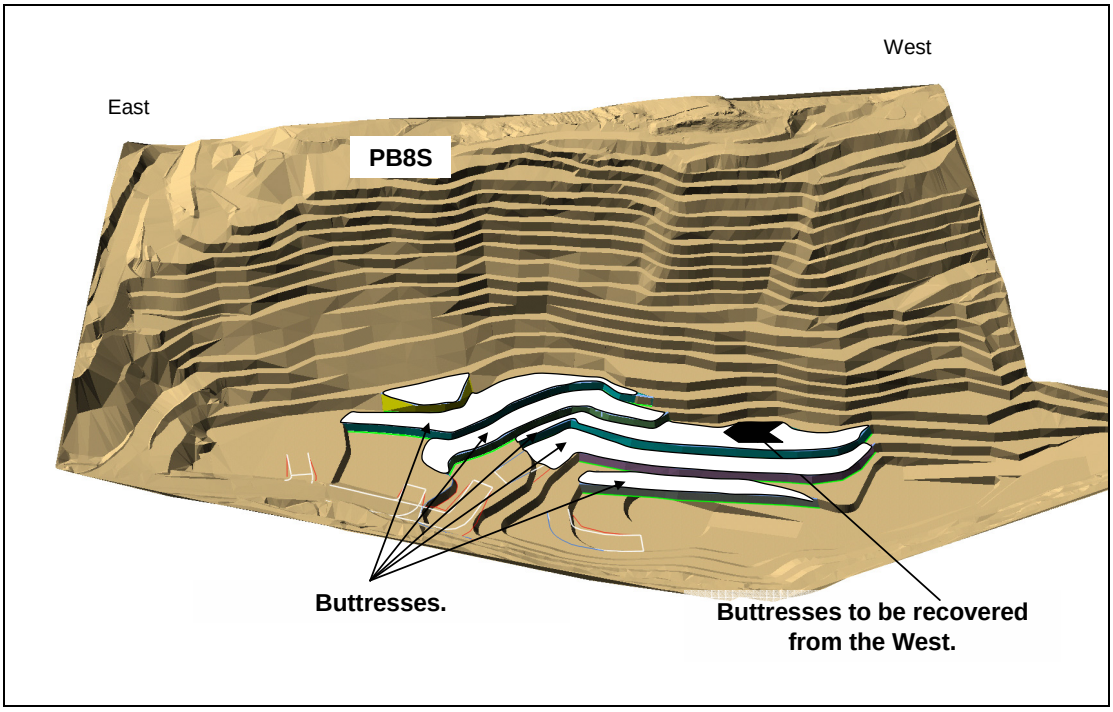
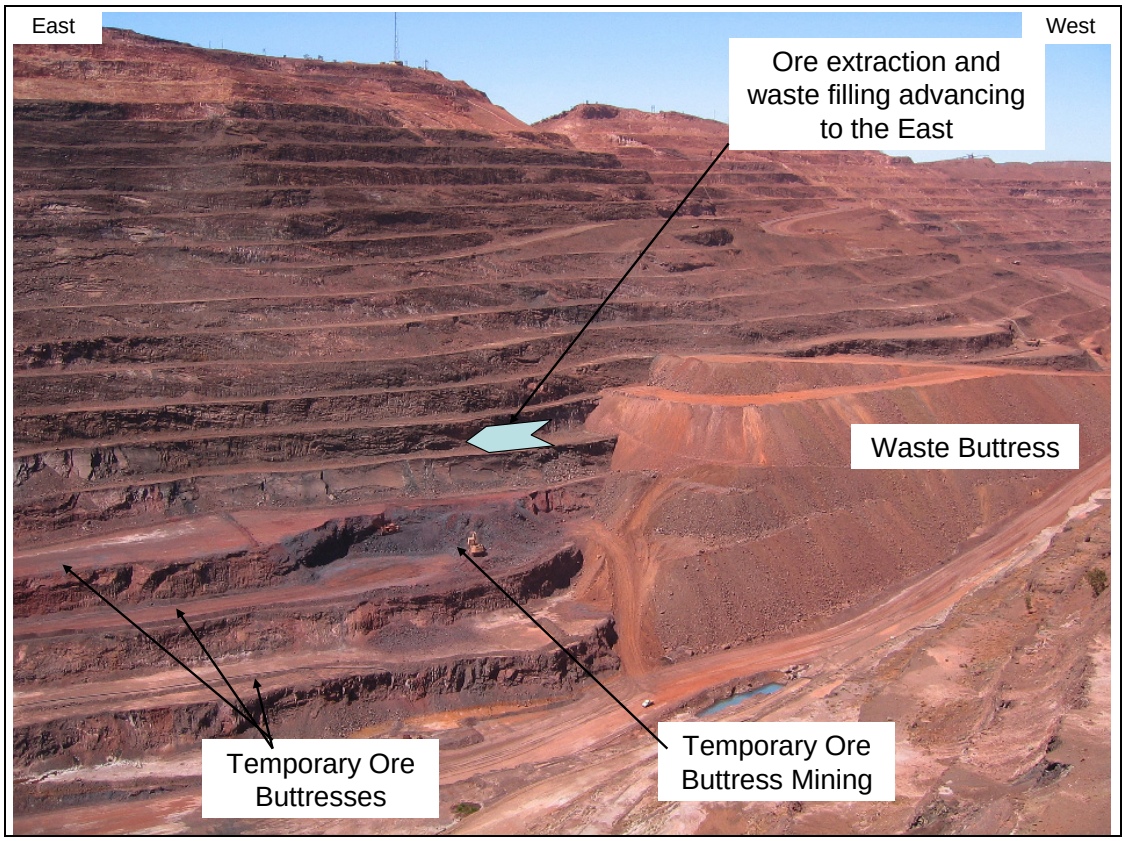


Figure 3.3: Temporary Ore Buttress Mining and Easte Backfilling, Nov.2005



3.1.2 Slope Performance

3.1.3 Monitoring

In addition to monitoring by visual inspections, comprehensive displacement monitoring has been implemented using survey prisms and inclinometers. Currently, during extraction of the buttresses and the mining of the lower benches, ground movement radar (Slope Stability Radar - SSR) has also been employed.

3.1.3.1 Survey Prism Monitoring

The current PB8S prism monitoring network consists of prisms read automatically approximately every six hours. Data are fed back to the geotechnical engineering and pit control offices by telemetry. Slope displacement monitoring shows movement over much of PB8S and significant effort has been made to determine the mechanisms causing this movement and the consequent risks. Figure 3.1 illustrates a representative section through PB8S showing the position of a number of prisms on the slope. Displacement records for these prisms are presented in Figure 3.4 and show the following:

- Long term movement rates of 0.02 mm/day on the upper slope (prisms 111022 and 114013).
- Long term movement rates of 0.03 mm/day in the centre of the slope through a keel structure (prism 117023).
- Short term movement rates of between approximately 0.1 mm/day and 0.2 mm/day on the lower benches of the slope in closer proximity to the EFF (prisms 122019 and 124005).
- All movement rates have slowed as the rate of mining in the lower benches has slowed.
- Some correlation between rainfall events and periods of increased movement, particularly where relatively shallow, smaller scale, instability mechanisms develop lower in the slope in proximity to the EFF.

Figure 3.5 illustrates the displacement data produced by a prism in close proximity to (but not on) the double bench slip of Feb.13, 2004 (prism 120010). Displacement rates of 0.2 mm/day were recorded over the five months prior to the adjacent instability which was triggered by 82 mm of rain fall. Displacement rates of 2.2 mm/day were recorded at the time of collapse.

Regarding mechanisms causing movement, the measured ground displacements and the time and rate of movement over the majority of the slope are compatible with the initial response of a rock mass to excavation (that is elastic rebound, relaxation and dilation of the rock mass) for large open pits (Zavodni, 2000). However, monitoring results prior to the Feb 13 slip indicate that, where multi-batter scale instability is controlled by weak planar structures, displacement rates similar to those expected as an initial response to mining may result in slope collapse. Total displacements of only 40 mm were recorded prior to the Feb 13 slip.

Figure 3.4: Prism Monitoring Results Adjacent to the Feb 13 Slip

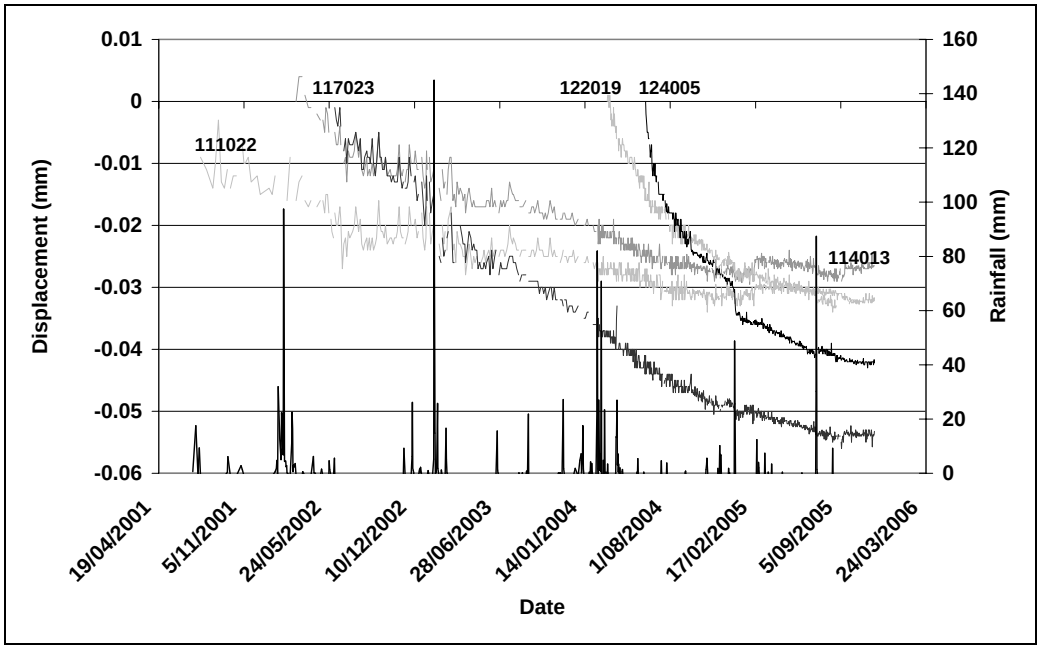
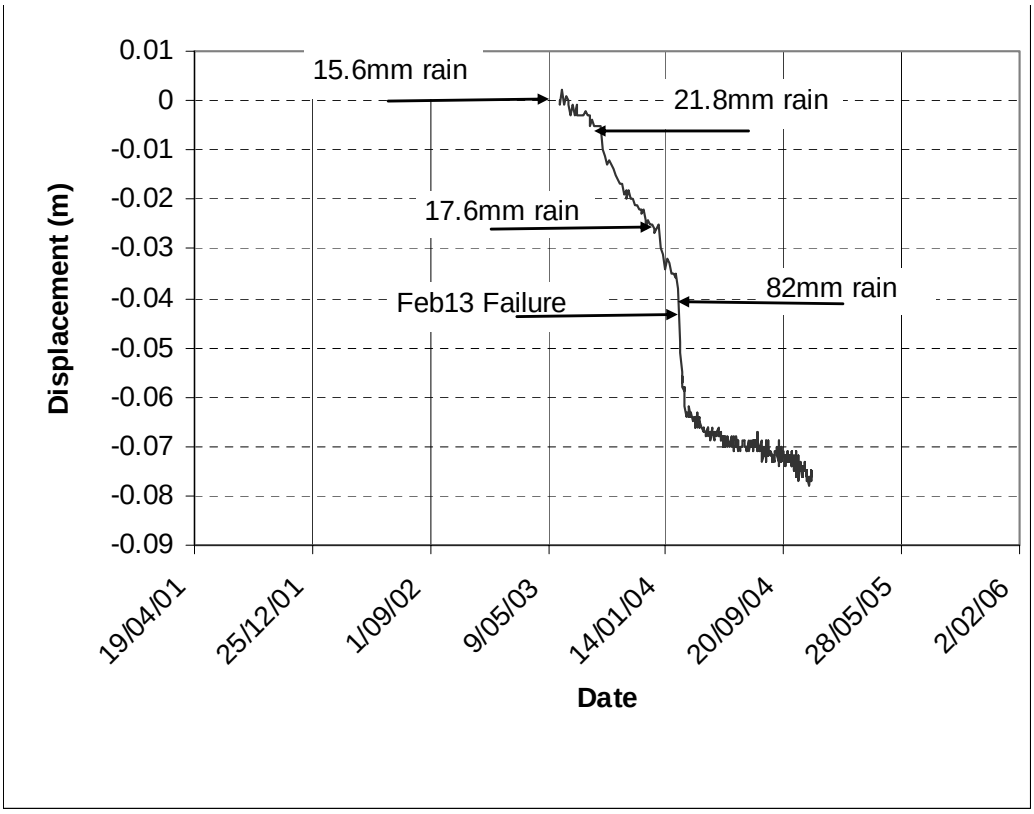


Figure 3.5: Prism Monitoring Results Adjacent to the Feb 13 Slip



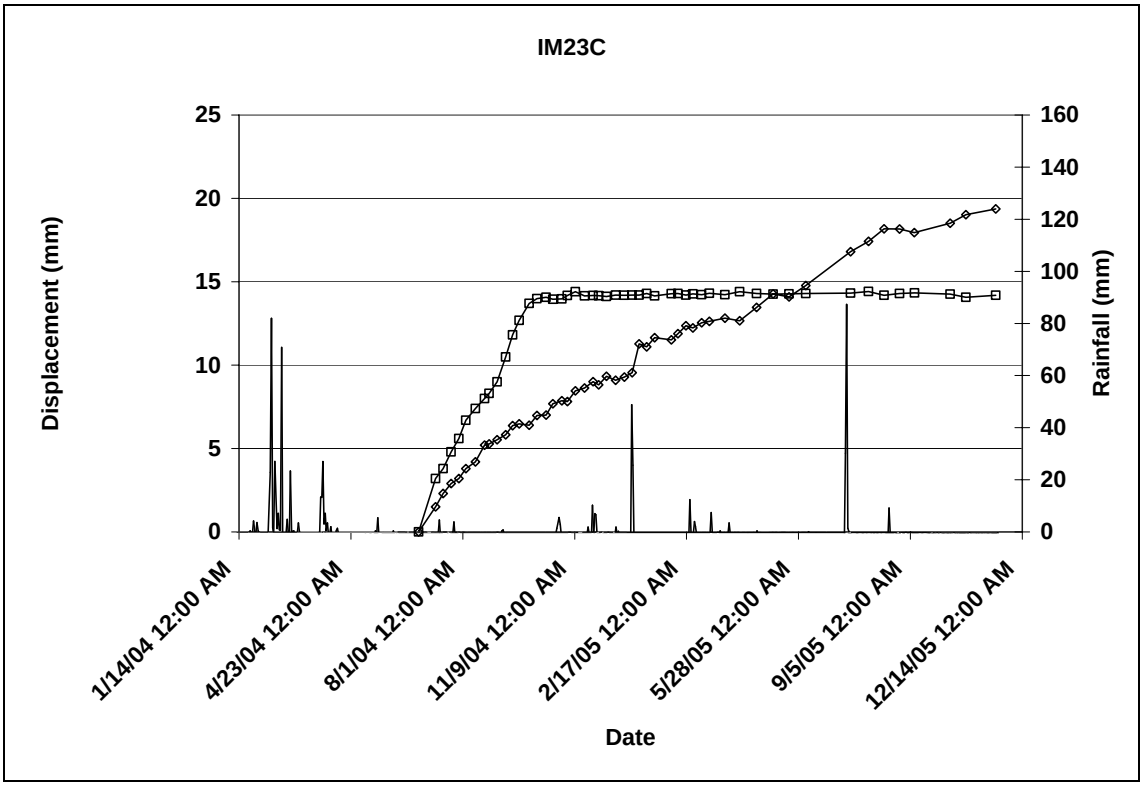
An important aspect of slope displacement monitoring is the identification of movements that may be indicative of the onset of failure or collapse. To this end the prisms in PB8S have been grouped depending on the identified mechanism and scale of

instability likely to cause movement in that particular sector of the slope. Daily, weekly and quarterly monitoring reviews are performed, based on an assessment of movement of prisms within these groups

3.1.3.2 Inclinometers

A total of seven inclinometers have been installed to various depths on PB8S, and are principally designed to monitor displacement on the 320 Faults and the major keel structures as well as the EFF. Inclinometers are manually read once every week or two, depending on previous movements and location on the slope. Movement has been recorded on all but one of the instruments, with the greatest movement being recorded in holes which intersect the EFF in the central area of the PB8S (i.e. approximately mid way along the strike of the pushback, close to the section shown in Figure 3.1). These movements show a strong correlation with large rainfall events and mining rates. Figure 3.6 illustrates typical movements to date for these instruments.

Figure 3.6: Typical Inclinometer Displacements



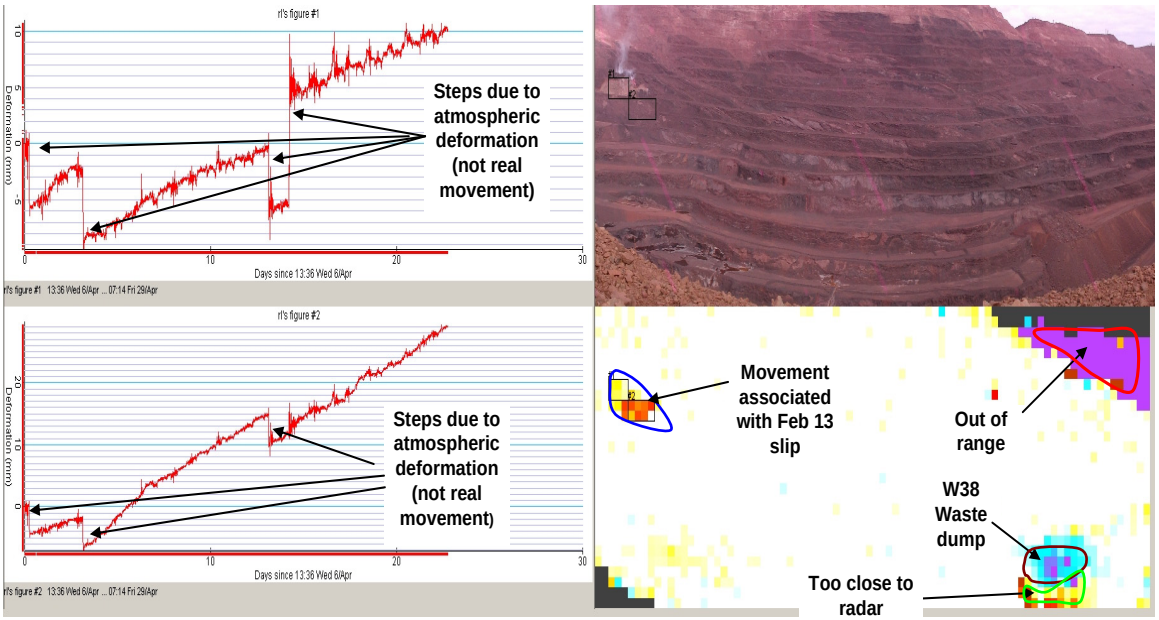
3.1.2.3 Slope Stability Radar (SSR)

During mining of the lower benches of PB8S, Slope Stability Radar (SSR) has been employed to provide additional monitoring of the safety hazard associated with possible bench to multi bench instability triggered by mining or rainfall. The principal advantages of the radar over prism monitoring are its coverage of the complete area of slope being monitored and shorter intervals between scans (15 minutes versus six

hours). To date, the radar monitoring results have identified ongoing movement only in the area of the Feb. 13 slip.

The SSR is operated in a safety critical mode. That is, critical displacement levels are set, at which movement alarms will be triggered in the pit control and geotechnical offices. Figure 3.7 presents an example of the SSR monitoring. In practice, alarm trigger levels have been set after consideration of the type and size of failure expected and the need to prevent numerous false alarms due to atmospheric interference.

Figure 3.7: Example of a SSR Scan Set Illustrating Movement and Interpretation of Scan Image



3.1.2.4 Reconciliation Between Design and Performance

The detailed 2004 geotechnical review and re-assessment of PB8S was undertaken following the Feb. 13, 2004 slip. This re-assessment used updated hydrogeological, structural and geotechnical data from monitoring and mapping undertaken since the commencement of the pushback in 1999. Two dimensional limit equilibrium analyses were undertaken, again using the anisotropic rock mass strength model, adjusted to account for the generally oblique strike of the stratigraphy relative to the likely failure surface. This strength modification accounted for the minor component of rock mass shear strength that would be generated by cross cutting bedding on strike.

This review indicated that the lowest factors of safety (FoS) for large scale overall slope instability for the buttressed wall geometry are in the central sector of PB8S. FoS at the eastern and western ends of the pushback are estimated to be higher. These results correlate well with the slope displacement monitoring (prisms and inclinometers), which shows the greatest movements in the central sector of PB8S. The review also indicated that critical failure surfaces follow the EFF (Figure 3.1), a conclusion supported by the inclinometer monitoring results.

Stability analyses indicate that there is a likelihood of instability developing after removal of the ore buttresses in the central sector of PB8S. The risks posed by this potential instability are being mitigated through the use of tightly controlled mining schedules, comprehensive slope monitoring (SSR) and construction of the waste fill buttress.

4.1 Conclusions

PB8S is the final cutback on the south wall of the East Pit of the Mt Whaleback mine. Stage 1, mining of the ore reserve in PB8S, has been successfully completed and Stage 2, mining of the ore buttresses, has recently commenced. The principal factors contributing to the success of PB8S are as follows:

- A comprehensive and accurate structural/geological model developed from tight drilling control and face mapping, with regular updates during mining.
- Implementation of comprehensive hydrogeological and slope monitoring programmes, which have allowed slope performance to be assessed as well as allowing the geotechnical and slope stability models to be calibrated; and failure mechanisms to be confirmed.
- Rigorous geotechnical analysis and slope design based on the most reliable geotechnical model achievable with the available data.

Whilst there is potential for instabilities developing during buttress extraction, the risks will be mitigated by the implementation of strictly controlled mining and waste buttressing schedules, linked to the monitored performance of the slope.

5.1 Acknowledgements

The permission of BHPBilliton Iron Ore to publish this paper is greatly appreciated. Mining is a team effort and involves input all along the process chain, from the design and long term planning engineers, to the short term planning and drill and blast engineers, and finally the operations technicians and their supervision. This contribution to PB8S is acknowledged In particular the input of Janos Ronaszeki of BHPBIO's Structural Geology Group, and of Aquaterra Consulting's hydrogeologists, is acknowledged. The authors would also like to acknowledge the significant input to the project by Mr Tim Sullivan, BHPBIO's geotechnical review consultant.

6.1 References

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