APPENDIX C

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APPENDIX C1

EIS pit slope design and rock storage facility report

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COMMERCIAL IN CONFIDENCE

Our Ref: PSM1002-110R
Date: 1 March 2011

BHP Billiton
Olympic Dam
Level 2, 55 Grenfell Street
ADELAIDE SA 5000

ATTENTION: MR. J. NAGEL
(Via email: James.Nagel@bhpbilliton.com)

Dear Sir,

RE: EIS PIT SLOPE DESIGN AND ROCK STORAGE FACILITY REPORT

Please find enclosed our EIS report regarding Pit Slope Design and the Rock Storage Facility.

We trust this report is in keeping with your requirements and will finalise the document on receipt of your comments.

For and on behalf of
PELLS SULLIVAN MEYNINK

Tim Sullivan

T.D. SULLIVAN

Distribution: 2 copies BHP Billiton
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BHP Billiton

EIS PIT SLOPE DESIGN AND ROCK STORAGE FACILITY REPORT

Report PSM1002-110R February 2011

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FIGURES

1. INTRODUCTION

This report presents a high level overview of the geotechnical slope design for a proposed open pit and rock storage facility at Olympic Dam (OD) in South Australia. The report is written at the request of BHP Billiton (BHPB) and follows discussions between BHPB and PIRSA.

The geotechnical slope design studies have been ongoing since 2006 and there is a very significant body of analysis and design. The geotechnical studies have been subject to three independent peer reviews between 2007 and 2008. This document summarises the key outcomes in the major principle design areas.

2. GEOTECHNICAL INVESTIGATION AND DESIGN STRATEGY

Because of the scale of the planned mine, the size of the pre-strip required, and the resultant financial commitment, the geotechnical study elements are required to be to a world-class standard and commensurate with the project's needs.

The final pit slopes were principally of significance for determining the location of infrastructure and possibly for their influence on overall pit development. The geotechnical studies were therefore focused on the payback period, notionally set at 15 years of ore production, on the principal risk areas in the early years of mining, and to identify any elements that could impact on achievement of the planned productivities.

An investigation strategy was formulated in early 2006 in conjunction with the BHPB Olympic Dam Geology group. The strategy entailed ten geotechnical sections which were selected to cover the Southern Mine Area (SMA). The aims of the sections were:

- To intersect the underground workings,
- To ensure at least one section was through Whenan Shaft,
- Sections to be oriented perpendicularly to the regional scale structure known as Mashers Fault Zone (MFZ),
- Sections to intersect the southern volcaniclastics (a known weak rockmass zone),
- Sections to be oriented perpendicularly to one of the known major structural fabrics (northwest-southeast),
- Sections approximately perpendicular to pit walls.

Drilling in 2006 was aimed at good coverage of the target area and definition of the geotechnical character of the main geological units of interest: e.g. Mashers Fault, volcaniclastics and haematite quartz breccia (HEMQ). Later drilling was a more targeted investigation of particular slopes and pits. The available borehole database, combined with pre-existing boreholes, is extensive with geotechnical logging for more than 500 holes.

Figure 1 below shows the location of the Starter Pit with the first four pushback stages. Overlain on this pit plan are the geotechnical sections and existing underground workings.

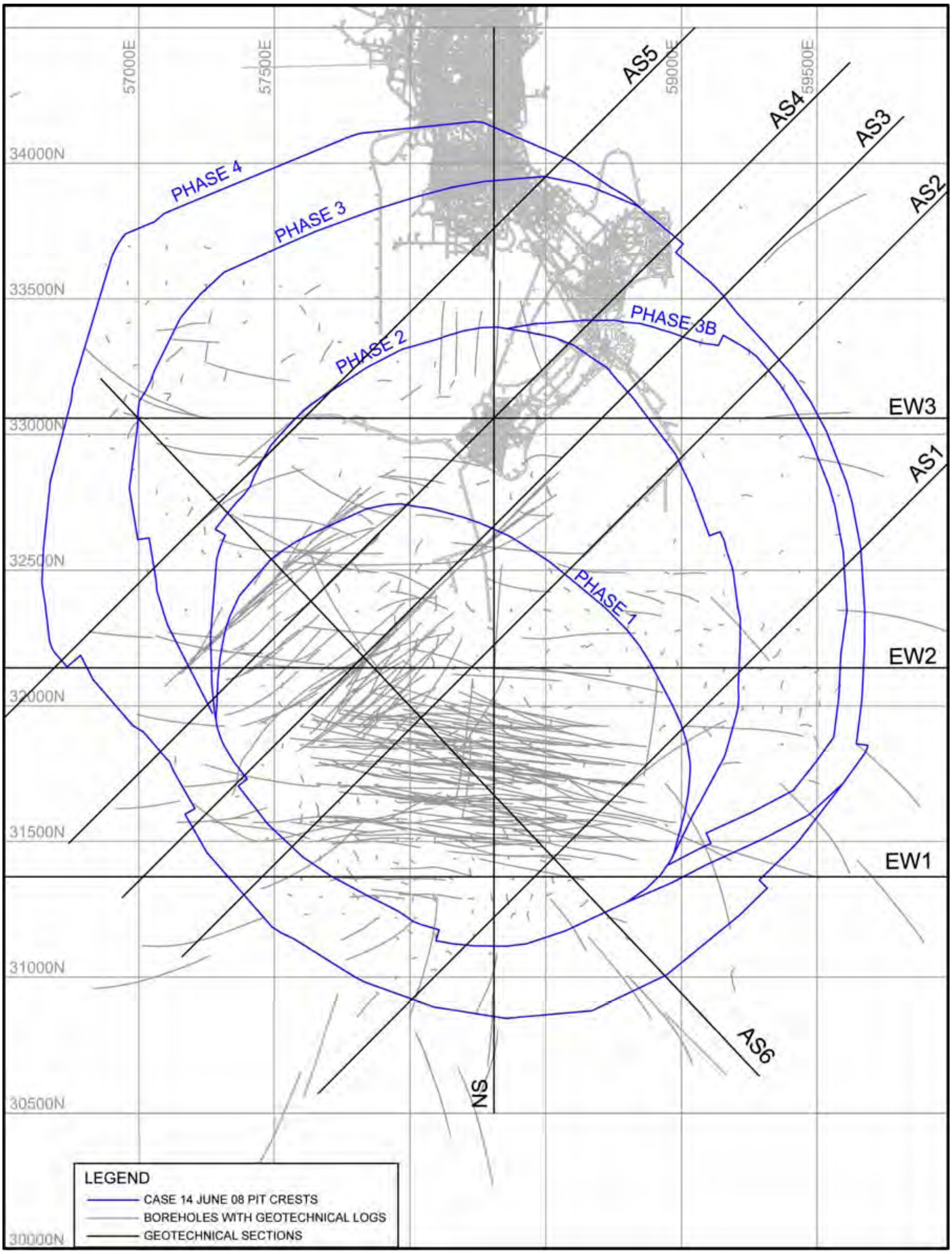


Figure 1 Location of planned open pit, existing underground workings, geotechnical borehole coverage and geotechnical investigation sections.

3. DATA

Geotechnical investigations undertaken to date, included:

1. Geotechnical drilling in ten key sections (AS1 – AS6, EW 1 - EW3, NS).
2. Detailed logging and data collection including: acoustic televiewer (ATV), core orientation, geotechnical logging, point load strength (PLS) testing and sampling.
3. Detailed geotechnical assessment of key sections.
4. Laboratory testing.
5. Rockmass strength assessment.
6. Probabilistic evaluation of rockmass properties for stability analyses.
7. Statistical evaluation of structural data for slope design in the Cover Sequence and Basement. These units are defined in Section 4 (Table 1).
8. Collation of information on in-situ stress.
9. Collation of geological and geotechnical information from the existing underground operation.
10. Modelling of open pit and underground infrastructure interaction.
11. Evaluation of Geometallurgical alteration models for use in geotechnical studies.
12. Development of a structural geological model.
13. Development of geological and alteration models including; lithology and alteration.
14. Hydrogeological investigations (BHPB and Schlumberger Water Services (SWS).

3.1. Drill Hole Coverage

Figure 1 below shows the location of holes with geotechnical logging compared to the Starter Pit. There is extensive coverage of the pit area, which is a considerably better situation than for most other open pits of this scale and at this stage of development.

Figure 1 also presents the locations of the ten geotechnical sections on which much of the interpretation, analysis and modelling have been based. This figure demonstrates that the sections provide good coverage for the pits.

3.2. Soil and Rock Mechanics Testing

The laboratory testing available from prior open pit studies (testing prior to 2006) comprised:

- Uniaxial Compressive Strength (UCS) tests – 492 (79 in Cover Sequence, 413 in Basement).

- Young's Modulus – 435 tests.
- Brazilian Tensile Strength tests – 163 tests.
- Direct Shear Tests – 35 tests (4 in Cover Sequence and 31 in Basement), although all tests were of poor quality.

Most of this pre-existing testing is located in the northern mine area (NMA), some distance from the proposed starter pit location. Also, there was no Triaxial testing in the pre-existing laboratory database. A comprehensive testing programme was developed to complement the existing database. The post 2006 testing program targeted the footprint of the proposed pit and all rock mass units. The testing comprised:

- Mineralogical analysis of the shear infill in the Cover Sequence, X-ray diffraction (XRD).
- Plasticity testing (Atterberg limits) of the shear infill in the Cover Sequence.
- Particle size distribution (PSD) analysis of the shear infill in the Cover Sequence.
- Direct shear testing of natural rock defects totalling 82 tests (40 in the cover sequence and 43 in the basement).
- UCS testing of rock core totalling 169 tests (73 in the Cover Sequence and 96 in the Basement).
- Brazilian tensile testing of rock core.
- Triaxial strength testing of rock core totalling 83 tests (40 in the Cover Sequence and 43 in the Basement).

The combined pre and post 2006 soil and rock testing databases provide the basis for input data to rock mass shear strength and for defect shear strength used in the stability assessment of structural data. These analyses are presented in following sections.

3.3. Structural Data

The structural database in the southern mine area comprises predominately borehole data. Both conventional core orientation and acoustic borehole imaging (ATV) was undertaken to obtain accurate orientation and character of the geotechnical defects encountered in the boreholes. A state of the art logging procedure was developed to capture high quality defect orientation data by combining the geotechnical logging, core photos and ATV data. Figure 2 provides an example of this ATV logging.

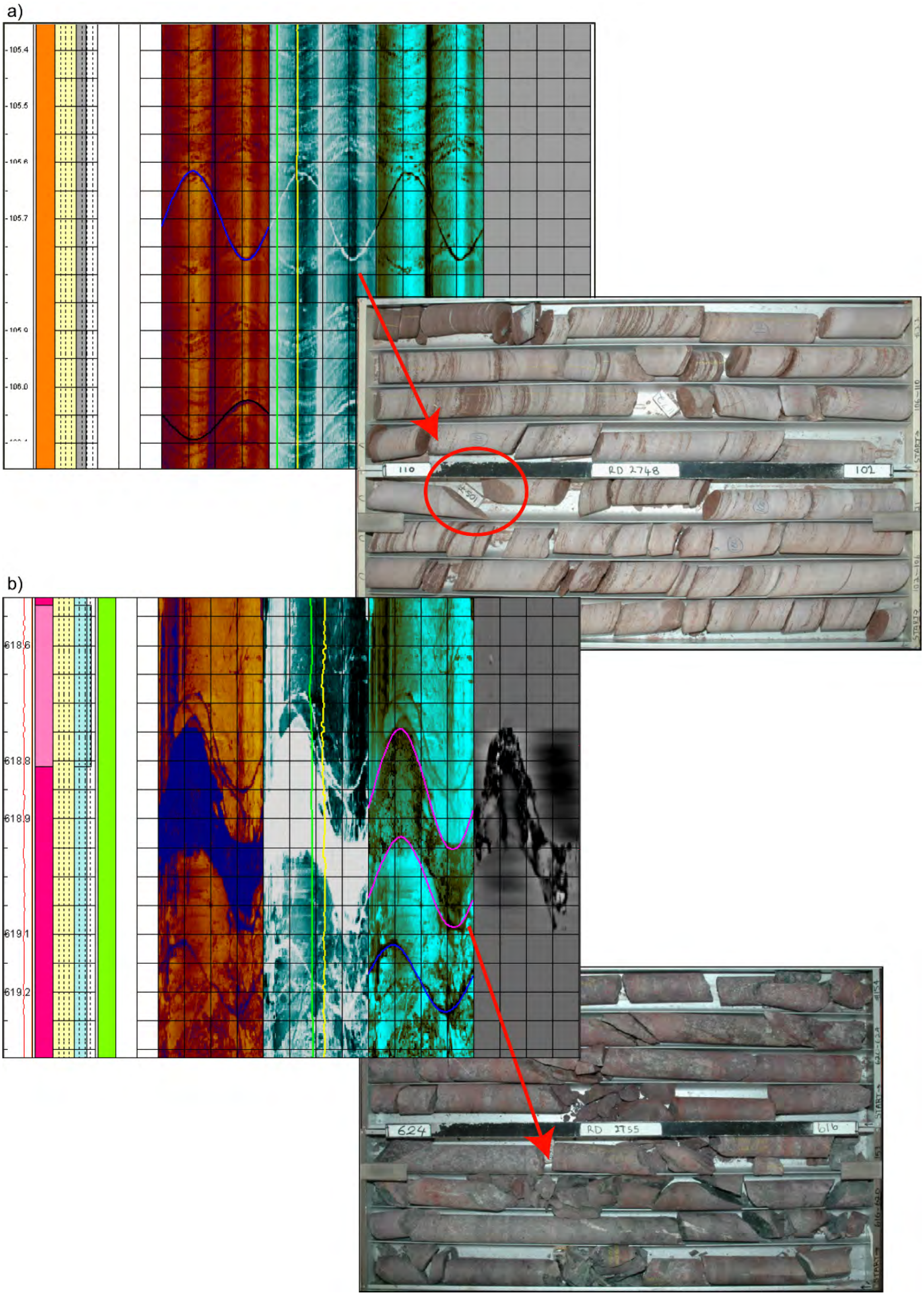


Figure 2: An example of acoustic borehole imaging (ATV) with a) bedding and a joint in the Cover Sequence, and b) shear in the Basement.

The resulting structural database is vast and comprises:

- 62,800 oriented structures from the ATV of 476 boreholes and
- Oriented core data - 19,500 structures from 285 boreholes

Figure 3 and Figure 4 below show the distribution of available ATV data in the Cover Sequence and Basement, respectively.

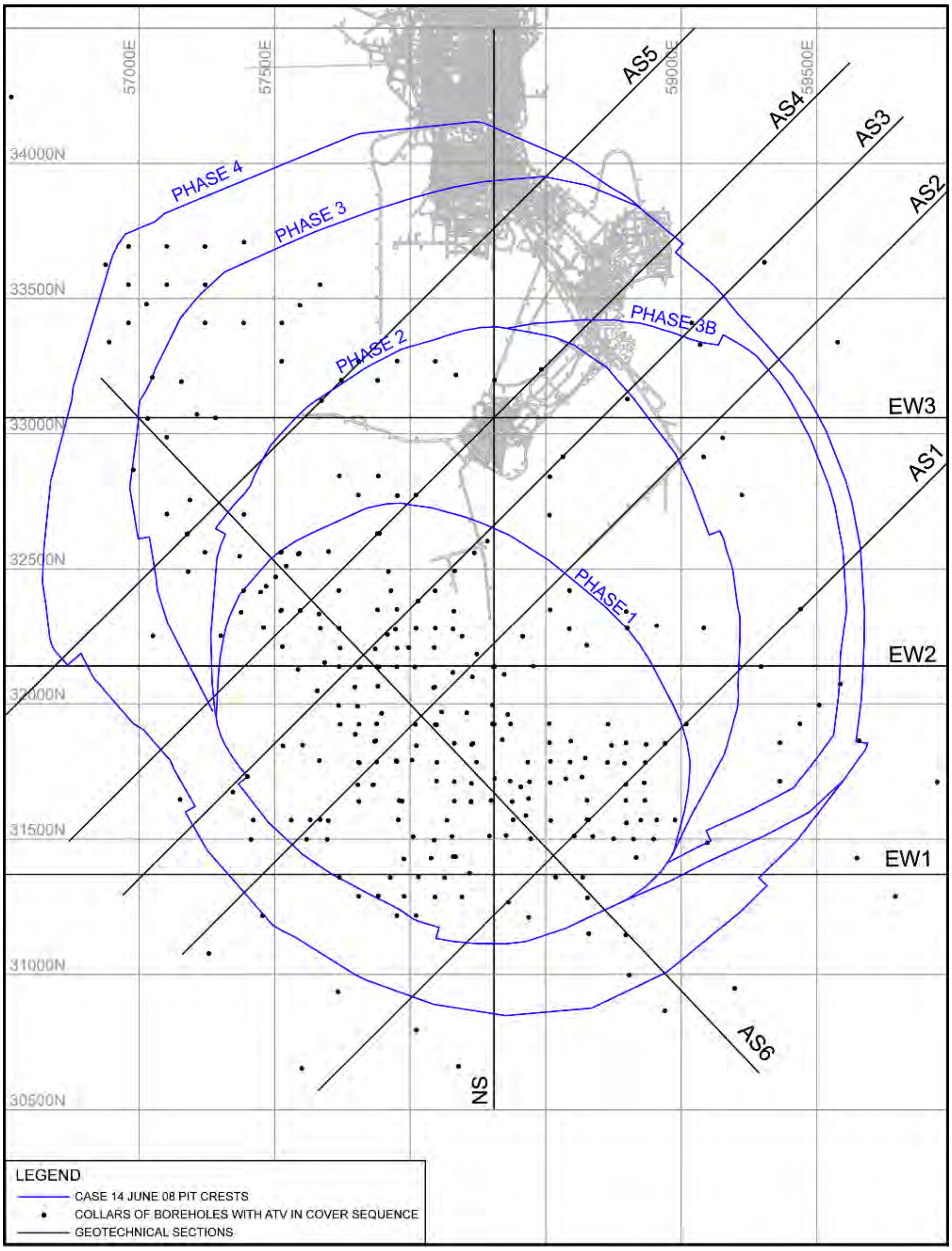


Figure 3: Boreholes with ATV in Cover Sequence

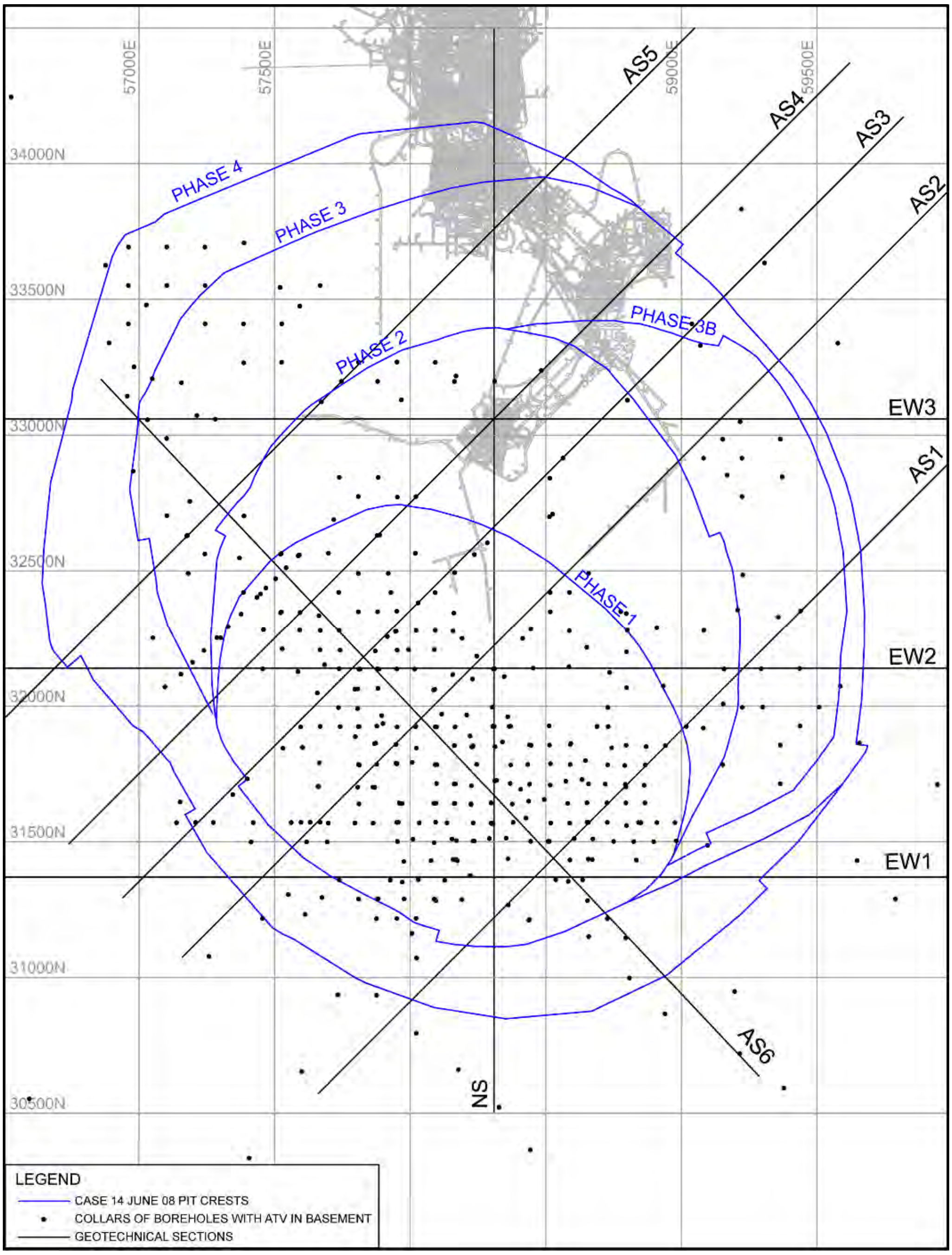


Figure 4: Boreholes with ATV in Basement

This structural data is managed via an Acquire™ database which can be visualised and interpreted in the three dimensional geological modelling package Vulcan™.

4. 2008 STRUCTURE MODEL

The structural model for OD SMA was developed by BHPB's geology department. The model is based primarily on borehole data but also contains mapping from the underground mine and regional interpretations. In summary, the resulting structural model available for geotechnical analysis comprises:

- A domain model for the Cover Sequence and the Basement. This model is based on volumes of rock with similar defect orientations and spacing. Figure 5 shows the four cover sequence structural domains (CSD1 – 4), while Figure 6 shows the 32 basement structural domains. Stereographs on these figures show all oriented structural data from ATV.
- Where possible, structures were correlated between adjacent boreholes in the SMA. Vulcan™ wireframes from these interpretations were developed and are available for geotechnical stability assessment. The wireframes are surfaces made up of triangles to simulate the shape of a three dimensional structure. Figure 7 shows the correlated structures for the cover sequence.
- Defect spacing models were created from borehole data to delineate zones of rock with closer fracture spacing.

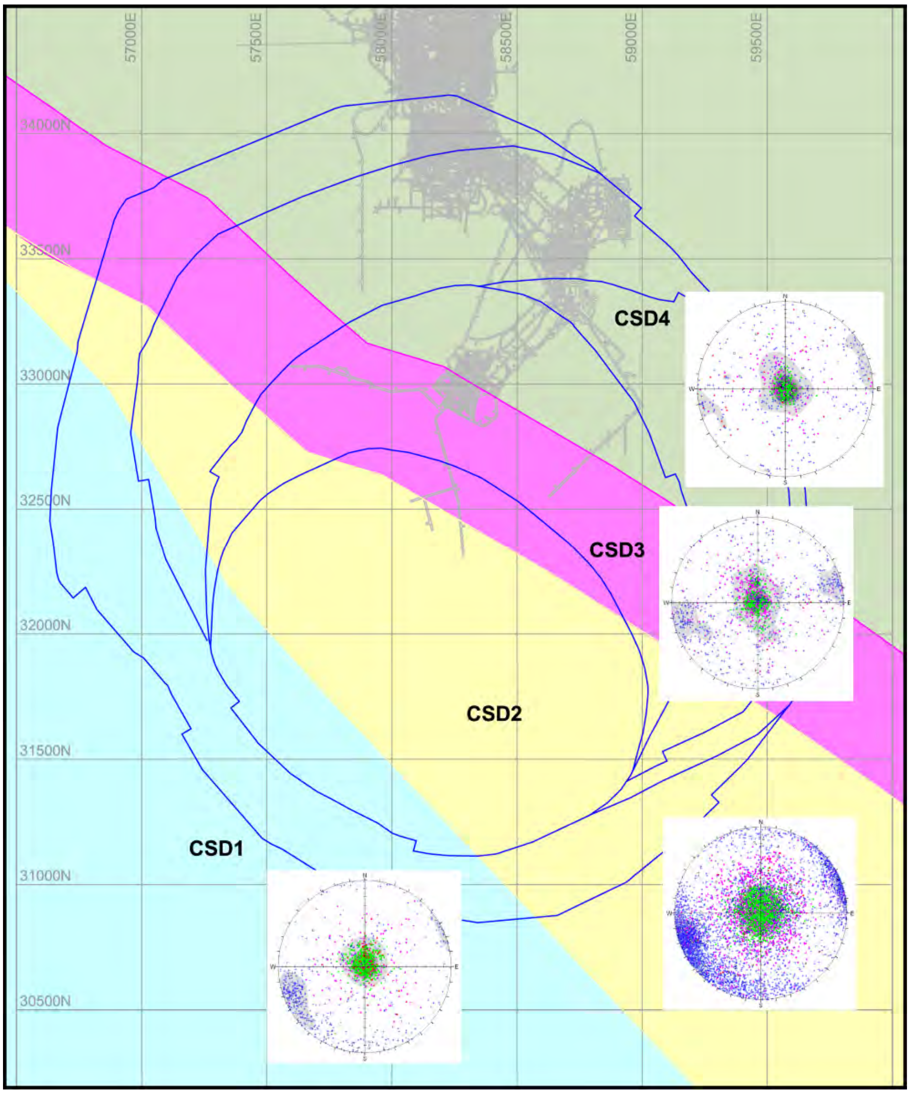


Figure 5: Cover Sequence structural domains, 2008. Stereographs show all structures from ATV for each domain.

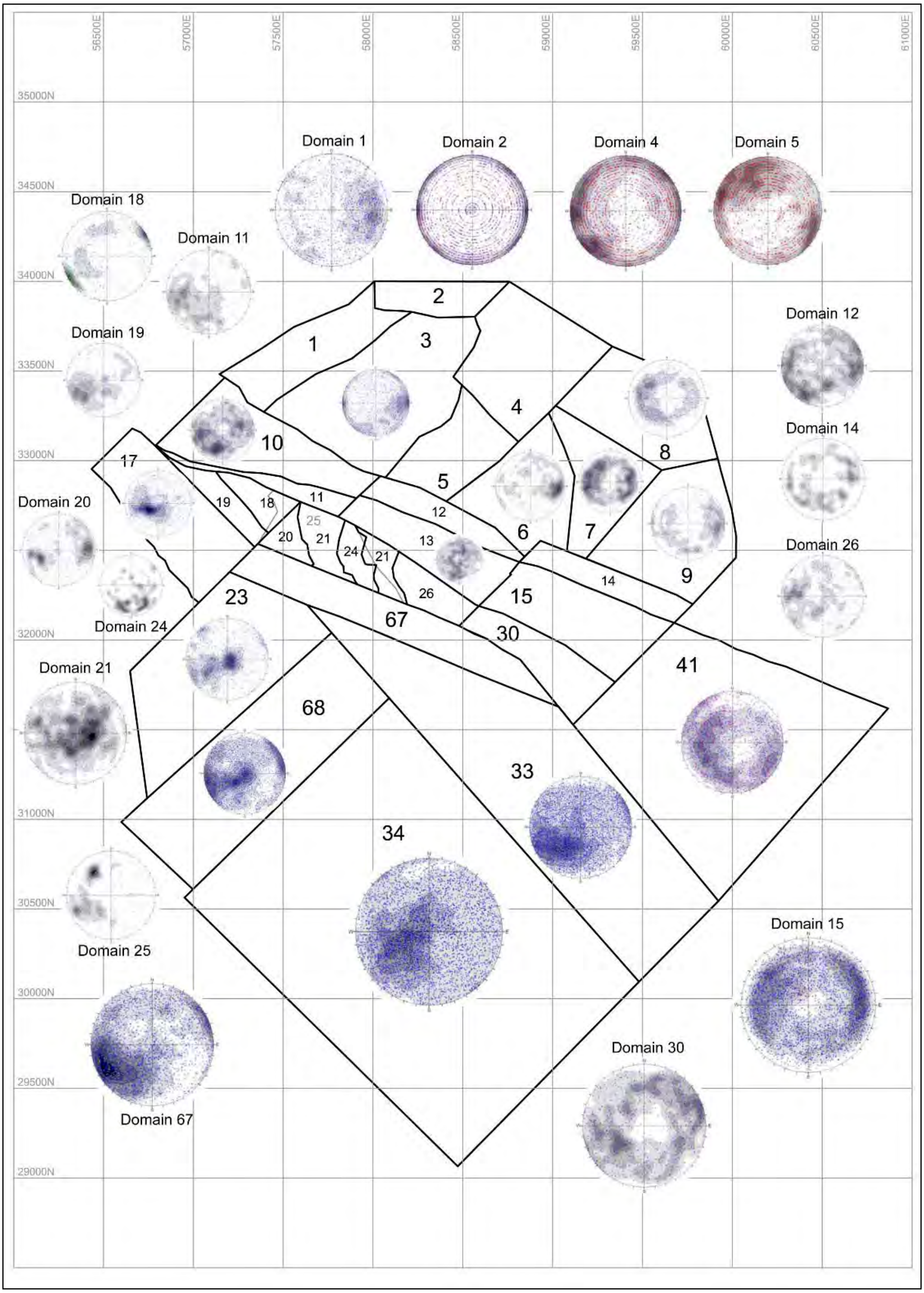
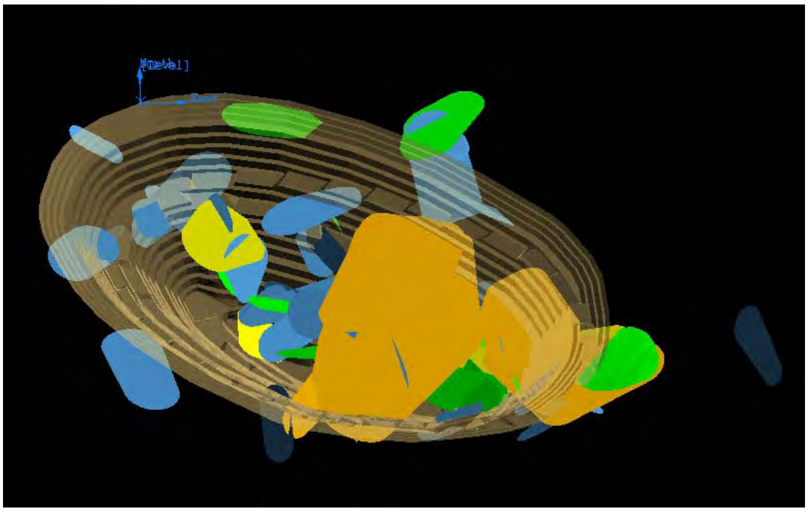


Figure 6: Basement structural domains, 2008. Stereographs show all structures from ATV for each domain.

a)



b)

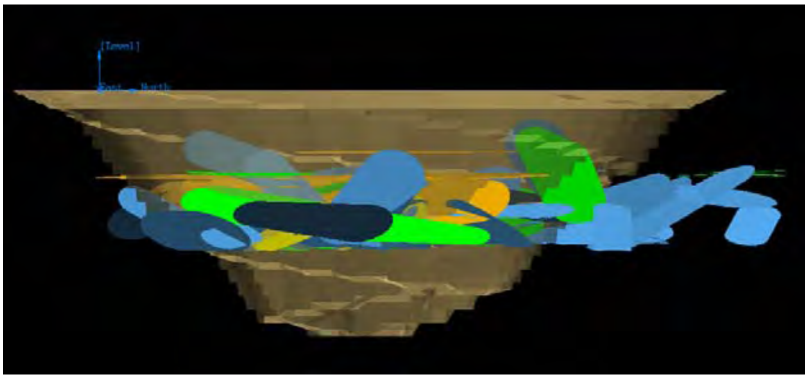


Figure 7: Correlated structures in the Cover Sequence relative to the starter pit a) oblique view b) cross-sectional view towards the west.

5. ROCK MASS MODEL

5.1. Introduction

For the purposes of the geotechnical study, the rock mass at OD has been divided into the Cover Sequence and Basement. Table 1 below presents a summary stratigraphic column showing the principal units.

The rock mass units capture lithological and structural differences. During the investigation it became evident that there were varying degrees of fracturing and deformation in the Cover Sequence and hence the geotechnical studies have also focused on the distribution and degree of this deformation.

TABLE 1
SIMPLIFIED STRATIGRAPHIC COLUMN SHOWING MAJOR UNITS AT OD

Surface +100mRL	ROCK UNIT	THICKNESS	DRY BULK DENSITY
	Sand & Clays (Aeolian)	0 m to 30 m thick	1.6 t/m^3 to 1.8 t/m^3
	Andamooka Limestone	20 m to 45 m thick	2.5 t/m^3 to 2.6 t/m^3
COVER SEQUENCE	Arcoona Quartzite	110 m to 170 m thick	2.45 t/m^3 to 2.6 t/m^3
(~315m to 360m thick)	Corraberra Sandstone	10 m to 25 m thick	2.3 t/m^3 to 2.45 t/m^3
Unconformity	Tregolana Shale	100 m to 150 m thick	2.7 t/m^3 to 2.75 t/m^3
(-215 to -360m RL) BASEMENT COMPLEX	Basement Units Granite >90% - Hematite <10% to Granite <10% - Hematite >90% plus Volcaniclastic Units, Felsic, Dolerite & Ultramafic Dykes	+2000 m thick	2.7 t/m^3 to 4.5 t/m^3

5.2. Cover Sequence Rock Mass Model

The Cover Sequence rock units were defined initially from the detailed geotechnical drilling along AS3 (see Figure 1) based on:

- The lithological units.
- Detailed logging and interpretation of seven closely-spaced geotechnical holes.
- High quality ATV data calibrated with core orientation and geotechnical logging.
- Field strength testing (Point Load Strength Index (PLSI)) but limited additional laboratory testing.

The other geotechnical sections were then reviewed. Variations in lithological character indicated that sub-division of some units was appropriate, resulting in eight units comprising:

- Surface soils and weathered Bulldog Shale (ZRS).
- Andamooka Limestone (ZAL): a high-strength rock with some leaching and vuggy texture; locally infilled with clay and rock fragments; limonite stained joints and defects, generally sub-vertical, irregular and rough.
- Arcoona Quartzite – Red (ZWAR): shale rich, a high-strength rock, with shale laminations and interbeds; closely-spaced bedding planes dominate; joints are irregular and rough with patchy coating. Bedding plane shears occur more frequently near the top of the unit.
- Arcoona Quartzite – Red (ZWAR): shale poor.
- Arcoona Quartzite – White (ZWAW): a high-strength rock with more quartzite and less shale than the unit above; as the frequency of shale beds increases, the defects in the rockmass change from bedding partings to joints; the joints are spaced at 1 m to 3 m and there are minor isolated bedding plane shears.
- Corraberra Sandstone (ZWC): a massive, high-strength sandstone that is locally leached and vuggy; below the aquifer section, the sandstone is iron-stained.
- Tregolana Shale (ZWT): upper transitional unit.
- Tregolana Shale (ZWT): a high-strength fissile shale with closely-spaced bedding partings that generally appear to become prominent on exposure. There are some joints, planar, rough and inclined although many truncate within the width of the core.

Characterisation of the Cover Sequence rockmass geotechnical units included:

- Fracture frequency.
- Rock Quality Designation (RQD).
- Intact rock strength.
- Percentage of total defect population that are faults and shears.
- Distribution of faults and shears through the rockmass.

5.3. Basement Rock Mass Model

A large number of basement lithologies are mapped at OD. However, many of these lithologies have similar geotechnical characteristics and for the purposes of this study are grouped together. This study defined fifteen rock mass units as presented in Figure 8. These units are described below:

- Granite (GRANITE 1; GRANITE 2; GRANITE 3-4; GRANITE 5-6) – Granite and re-cemented granite breccia. Typically hard massive rock with sericite and chlorite alteration along structures.
- Hematite Breccia (HEM BX1; HEM BX2-3; HEM BX4-5; HEM BXN) – Re-cemented granite and haematite breccia. A suite of rocks with varying amounts of haematite and brecciation. Typically hard massive rock with sericite and chlorite alteration along structures.
- HEMQ (HEMQ) - A suite of rocks with high amounts of haematite and varying amounts of brecciation. These are generally all very high strength rocks and widely jointed.
- Volcaniclastics (haematised) (VOLC BX, VOLC N, KHEMQ) - Volcanic and felsic breccias, which are variably haematised. These are generally all very high strength and widely jointed.
- Volcaniclastic Sediments (KASH, VASH KFMU) - Volcanic ash and sediments; well bedded and variably altered and or hematite cemented. Generally of medium strength and well jointed and or bed, except where there is extensive haematisation. Haematisation results in increased strength and decreased fracturing.
- DOLERITE (DOL) - A single pipe-like body in the eastern part of the SMA; of very high strength. This unit is not shown on Figure 8.
- ULTRAMAFIC and FELSIC DYKES (DYKES) - Thin units of varying age that intrude the Basement complex. Generally highly jointed and variably sheared rocks, with sericite, chlorite or haematite alteration. Except where there is extensive haematite alteration, these units could be likened to faults. Some of the very early dykes have a very thin irregular form. This unit is not shown on Figure 8

Figure 8 shows the distribution of the various rock mass units at -300m RL.

The Basement rocks at OD are of very high strength and widely to very widely jointed. The key lower strength and lower rockmass quality units are:

1. KASH, a medium-strength rock with bedding and jointing and variable sericite alteration.
2. Sericite-altered zones.
3. Low-strength friable granular zones in the HEMQ, locally termed ‘sooty’ by ODX.
4. An upper Basement layer immediately below the contact, which appears to have reduced strength and increased defect spacing. This is inferred to be an alteration and or weathering effect.
5. Dykes, which are jointed, sheared and altered, and on the larger scale, are expected to act like faults in the rockmass.

The distribution of KASH is relatively well known. Further definition of some of the other weak units will be included in the future works program.

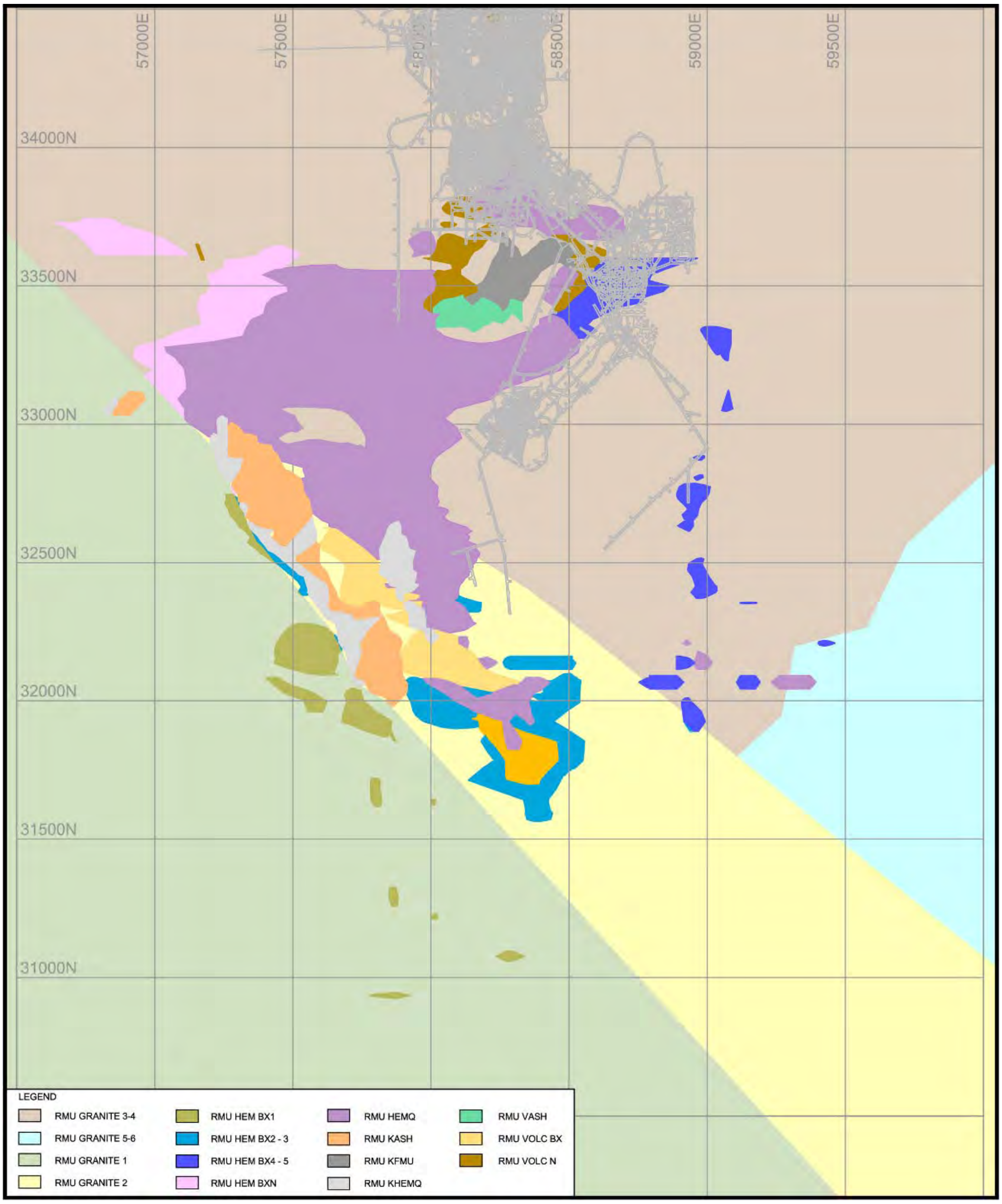


Figure 8: Basement rock mass units at -300m RL

5.4. Defect Shear Strengths

Defect shear strengths have been assessed using the results from the 82 direct shear tests of natural rock defects and are summarised in Table 2 below. The four lower strength defects are:

- Bedding plane shears in the Tregolana Shale.
- Bedding plane shears in the White Arcoona Quartzite.
- Shears in the Granite.
- Bedding in the volcaniclastics.

TABLE 2
FRICTION ANGLES – DIRECT SHEAR TESTING

ROCK TYPE	DEFECT TYPE	LOWER BOUND	UPPER BOUND	ADOPTED
ZWAR	Joints	32*	43	38
ZWAW	Joints	31	43	36
ZWC	Joints	33	43	39
ZWT	Joints	30*	46	37
ZWAW	Bedding	37	40	37
ZWC	Bedding & Shears	34	43	39
ZWAR	Bedding & Shears	24	33*	27
ZWT	Bedding & Shears	16	27*	22
HEMQ	Joints	33	45	39
KHEMQ	Joints	33	40	35
HEM & HEMH	Joints	28	56	42
Granite	Joints	26	49	39
KASH	Bedding	26	30	30
Granite	Shears	19	35*	20
HEMQ	Shears	39	42	39[1]
VHEM	Shears	37	37	37[1]

* Bounds modified based on majority of testing

[1] Based on limited testing

5.5. Rockmass Properties

5.5.1. Introduction

The rockmass properties for the different units in the SMA were derived from borehole core logging. To date, 190 holes have been incorporated into the rockmass model, comprising about 111,000 m of core.

5.5.2. Methodology – Cover Sequence

Initially, the geological units in the Cover Sequence were utilised. However, variations in character indicated that further subdivision was appropriate and resulting in seven key vertical units. The rockmass character data was then divided into deformed and undeformed, producing 14 units. The deformed zone is a relatively thin corridor that is coincident with a regional structure called Mashers Fault Zone (MFZ). This zone is still a good rock mass with a modest reduction in UCS and Geotechnical Strength Index (GSI) compared with the surrounding rock mass.

A summary of the rockmass parameters for the undeformed and deformed rock masses is presented in Table 3 and Table 4, respectively.

TABLE 3
COVER SEQUENCE – UNDEFORMED ROCKMASS PARAMETERS

ROCK UNIT	DESCRIPTION	UNIAXIAL COMPRESSIVE STRENGTH (MPa)		POINT LOAD STRENGTH INDEX (MPa)	MEAN RQD (%)	MEAN GSI
		MEAN	STANDARD DEVIATION			
ZG1	Andamooka Limestone (ZAL)	76	25	2.9	83	71
ZG2	Upper Red Arcoona Quartzite shale rich (ZWAR)	100	45	4.5	94	73
ZG3	Lower Arcoona Quartzite shale poor (ZWAR)	132	51	6.4	97	78
ZG4	White Arcoona Quartzite (ZWAW)	148	53	6.7	96	76
ZG5	Corrabera Sandstone (ZWC)	94	34	5.5	97	77
ZG6	Upper Tregolana Shale Transition – sandstone predominant (ZWT)	122	59	6.8	95	72
ZG7	Lower Tregolana Shale predominance of Chocolate Shale (ZWT)	90	27	5	92	73

TABLE 4
COVER SEQUENCE – DEFORMED ROCKMASS PARAMETERS

ROCK UNIT	DESCRIPTION	UNIAXIAL COMPRESSIVE STRENGTH (MPa)		POINT LOAD STRENGTH INDEX (MPa)	MEAN RQD (%)	MEAN GSI
		MEAN	STANDARD DEVIATION			
ZG1	Andamooka Limestone (ZAL)	50	16	4.4	67	61
ZG2	Upper Red Arcoona Quartzite - Shale Rich (ZWAR)	87	40	4	85	66
ZG3	Lower Arcoona Quartzite - Shale Poor (ZWAR)	80	40	5.57	89	69
ZG4	White Arcoona Quartzite (ZWAW)	95	27	4.2	81	68
ZG5	Corrabera Sandstone (ZWC)	78	17	4.13	88	70
ZG6	Upper Tregolana Shale Transition – Sandstone Predominant (ZWT)	112	25	5	76	65
ZG7	Lower Tregolana Shale - Predominance of Chocolate Shale (ZWT)	81	30	2.6	82	68

5.5.3. Methodology - Basement

The methodology adopted for the Basement was based on the geotechnical sections and commenced with Angled Section 3 (AS3, Figure 1). This section was considered the most important because it cut through the central part of the pit and is oriented normal to the volcaniclastics and MFZ. The geological wireframes on each section were used to define the broad lithological architecture. Hence the major rock types were provided with a spatial 'subdivision' based on geological interpretation.

The data and correlations were then reviewed and a number of units were combined after consideration of similarities in a number of parameters including GSI, UCS, PLSI and Triaxial Strength.

A summary of the rockmass parameters for each of the Basement units is included in Table 5 below.

TABLE 5
BASEMENT ROCKMASS PARAMETERS

ROCK UNIT	DESCRIPTION	STRENGTH UNIAXIAL COMPRESSIVE STRENGTH (MPa)		POINT LOAD STRENGTH (MPa)	MEAN RQD (%)	MEAN Gsi
		MEAN	STANDARD DEVIATION			
Granite 1	Granite south west of Ferenci Fault/ Volcaniclastics	144	68	6.53	99	81
Granite 2	Granite between Ferenci Fault and southern boundary of MFZ	109	18	5.15	99	79
Granite 3&4	Granite above haematite breccias in east	96	55	4.7	94	74
Granite 5&6	Granite below haematite breccias in east and north of MFZ southern boundary	138	70	6.18	96	77
HEM_BX1	Haematite breccias south-west of Ferenci Fault	172	90	7	99	80
HEM_BX2&3	Haematite breccias on north eastern margin of volcanic breccias	202	100	7	95	77
HEM_BX4&5	Haematite breccias, flat lying in the east and inclusive of minor HEMQ	222	90	NA	84	68
HEMQ	All HEMQ zones	149	90	4.4	96	76
KASH	Mixed Ash and Epiclastics	35	14	3.77	89	70
KHEMQ	Laminated Haematitic-Quartz sandstone/ siltstone	112	50	1.8	97	76
VOLC_BX	Volcanic Breccias, mainly HEMV & VHEM	103	60	6.65	96	77

5.5.4. Rock Mass Shear Strengths

A summary of the estimated rock mass shear strengths for both the Cover Sequence and Basement is included in Table 6 below. These strengths are based on the following factors:

- The Hoek Brown failure criterion (Ref. 1, Ref. 2) was adopted as the best estimator of rockmass strength.
- The Hoek Brown failure criterion requires two key parameters for assessing rockmass strength:
  - Intact Strength (UCS),
  - Geological Strength Index (GSI)
- The parameter m_i, which is a material constraint and is dependent of the actual rock type. Although there are published values for generic rock types, Hoek recommends triaxial testing to confirm this parameter.
- Hence, while the m_i was assessed using Hoek's recommended approach, experience has shown this is not robust and as such UCS and Brazilian testing were also used.
- The triaxial testing suggests no significant change in strength between weak and moderate alteration.
- All testing does suggest a decrease in UCS strength with increasing chlorite alteration.
- There is also a trend for decreasing m_i with increasing alteration.
- Trends and values were utilised in assigning appropriate m_i values to each of the granite domains.
- In domains with lower UCS and GSI, associated with a more altered rockmass, an m_i of 12 was adopted, which is in line with the lower bound of testing.
- In domains with higher UCS and GSI, associated with unaltered granite, an m_i of 22 was adopted, which is in line with upper bound of testing.

TABLE 6
ROCKMASS SHEAR STRENGTHS FOR COVER SEQUENCE AND BASEMENT

ZONE	ROCKMASS UNIT	GSI (Lower Quartile)	m_i	UCS MEAN (MPa)	COHESION (kPa)	FRICTION ANGLE (º)
COVER SEQUENCE UNDEFORMED ZONE	ZG1	65	12	76	550	61
	ZG2A	68	17	100	800	64
	ZG2B	71	17	100	1000	65
	ZG3	74	17	132	1800	64
	ZG4	71	36	148	1200	68
	ZG5	74	18	93	1300	60
	ZG6A	69	7	122	1700	48
	ZG6B	69	7	122	1700	47
	ZG7	70	7	80	1200	46
COVER SEQUENCE DEFORMED ZONE	ZG1	52	12	50	110	63
	ZG2A	57	17	65	230	58
	ZG2B	60	17	65	300	59
	ZG3	62	17	80	450	58
	ZG4	62	36	95	500	62
	ZG5	64	18	83	640	55
	ZG6A	63	7	100	900	43
	ZG6B	50	7	100	500	35
	ZG7	66	7	65	800	42
BASEMENT	Granite 1	78	22	135	2900	58
	Granite 2	76	19	109	2300	54
	Granite 3&4	70	12	95	1700	45
	Granite 5&6	75	22	135	2500	56
	HEM_BX 1	77	28	190	3300	61
	HEM_BX 2&3	73	28	190	2600	60
	HEM_BX 4&5	61	28	190	1600	53
	HEM_BX N	73	24	190	2700	58
	HEMQ	73	20	150	2900	55
	KASH	56	12	30	750	24
	KHEMQ	69	22	104	1600	51
	VOLC_BX	74	20	90	1900	51

6. GROUNDWATER

Hydrogeological studies for the open pit were undertaken by Schlumberger Water Services (SWS) (formerly known as Water Management Consultants). The hydrogeological studies were undertaken in concert with the geotechnical studies so that, for instance pore pressure modelling was undertaken on the same sections that were used for geotechnical stability and were focussed on rock mass units considered geotechnically significant.

The field component of the hydro geological studies comprised:

- Collation of existing measurements and abstractions.
- Installation of 89 nested vibrating wire piezometers as shown in Figure 9.
- Pump tests.
- Borehole permeability tests.

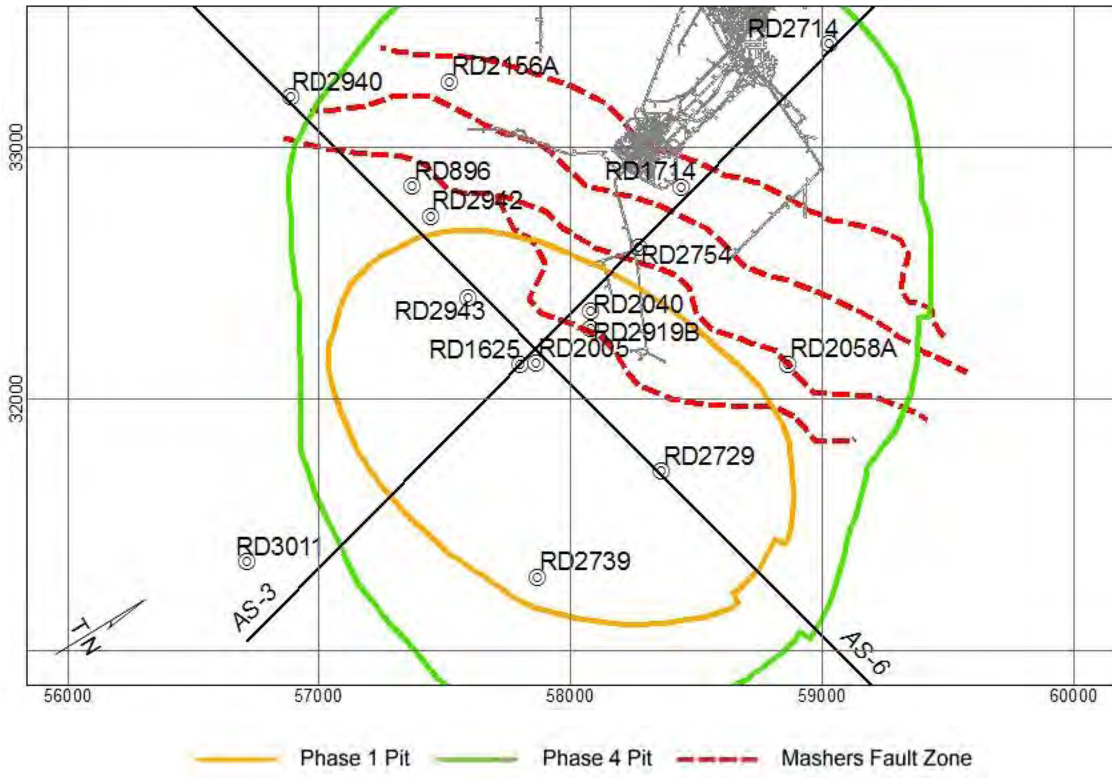


Figure 9: The location of nested vibtating wire piezometer installations

Three dimensional and quasi three dimensional models were then calibrated with historical and present day heads. These calibrated models were then used for forward prediction of pore pressures during pit development. Three different groundwater intervention cases were modelled. These cases were:

- P1 – No ground water intervention
- P2 – Perimeter ground water wells in the cover sequence.
- P3 –Depressurization drain holes in the Cover Sequence and the Basement rocks in addition to perimeter ground water wells in the Cover Sequence.

The purpose of these three scenarios was to demonstrate the incremental benefit of the dewatering and depressurisation efforts.

Pore pressure predictive models were produced at yearly intervals of pit development for typically the first 10 years of mining and then at longer interval thereafter. These pore pressure grids were then used in stability modelling. While stability modelling was undertaken for all three groundwater intervention cases, pit slope designs were based on intervention P3. An example of the output is presented in Figure 10 below for cross section AS3, assuming P3 intervention, at the time step of 2016 (last quarter).

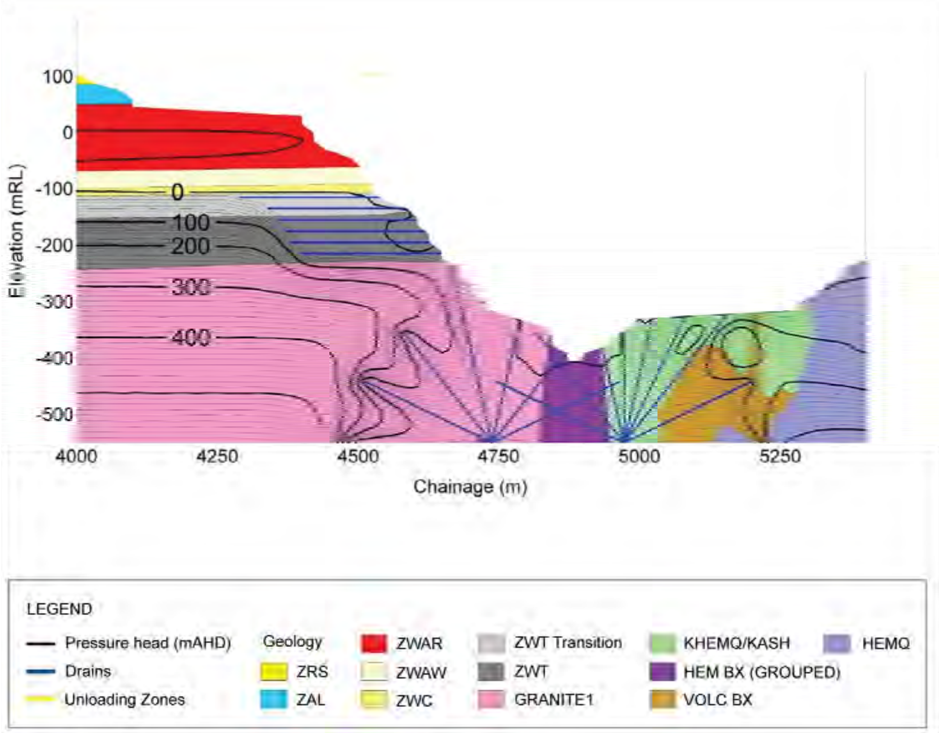


Figure 10: Section AS3, modelled case P3, fourth quarter 2016. Chainage 4000 about 57500mE.

7. STABILITY ANALYSIS

An extensive assessment of stability has been undertaken for the proposed open pit at Olympic Dam. The range of analysis includes:

- A range of rock mass properties,
- A range of pit geometries,
- A range of groundwater conditions,
- Investigation of a range of failure modes, and
- Multiple analytical techniques including hand calculations, Limit equilibrium, Finite element (2D & 3D), probabilistic methods for both rock mass and structural analysis.

The focus of the analysis has been to assess:

- Bench scale slope angles.
- Inter – ramp scale slope angles.
- Overall scale slopes angles.
- Impacts of pore pressure and to assess the depressurisation requirements.
- Sensitivities to assumptions including material strengths and pore pressures.
- Interactions with existing and proposed underground workings.
- Impacts and Interaction on key infrastructure including shafts and surface infrastructure including the proposed rock storage facility.

An overview of these analyses and key examples are presented in the following sections.

7.1. Stability Acceptance Criteria

The stability of the open pit slopes have been assessed at a minimum to achieve the stability criteria in Table 7 below.

TABLE 7
OLYMPIC OPEN PIT SLOPE DESIGN CRITERIA

SLOPE SCALE	FOS	PROBABILITY OF FAILURE FOR FOS ≤ 1
Bench	1.1	30 – 50%
Inter-ramp	1.2 – 1.3	3 – 5 %
Overall	1.3	1%

These are in keeping with other published criteria (Ref 1, Ref 2).

7.2. Conceptual Failure Mechanisms

The following conceptual failure mechanisms are considered to be those most likely to control stability in the open pit. These provide the basis for slope stability. Figures 11 and 12 present the likely mechanisms in the Cover Sequence and the Basement, respectively.

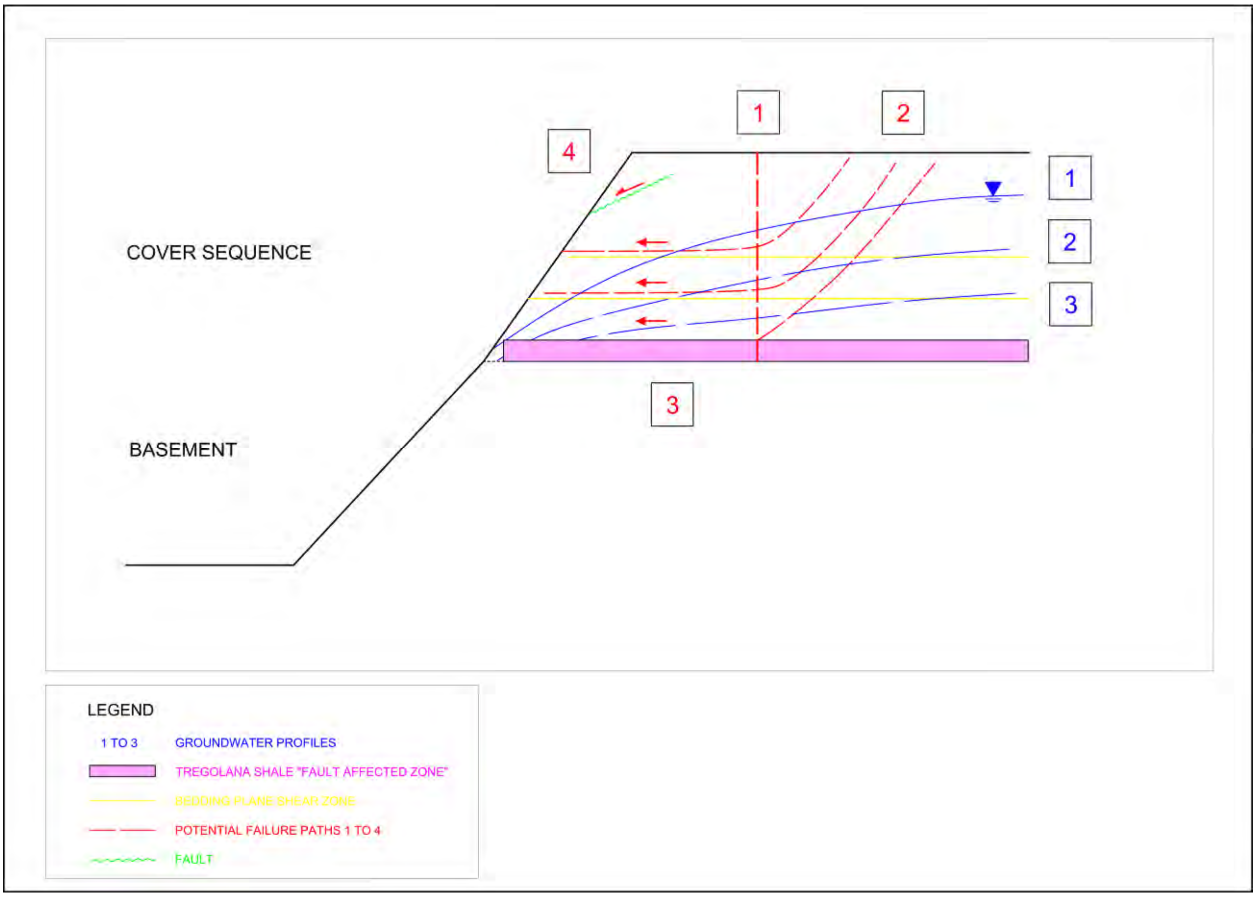


Figure 11: Conceptual failure mode in the Cover Sequence.

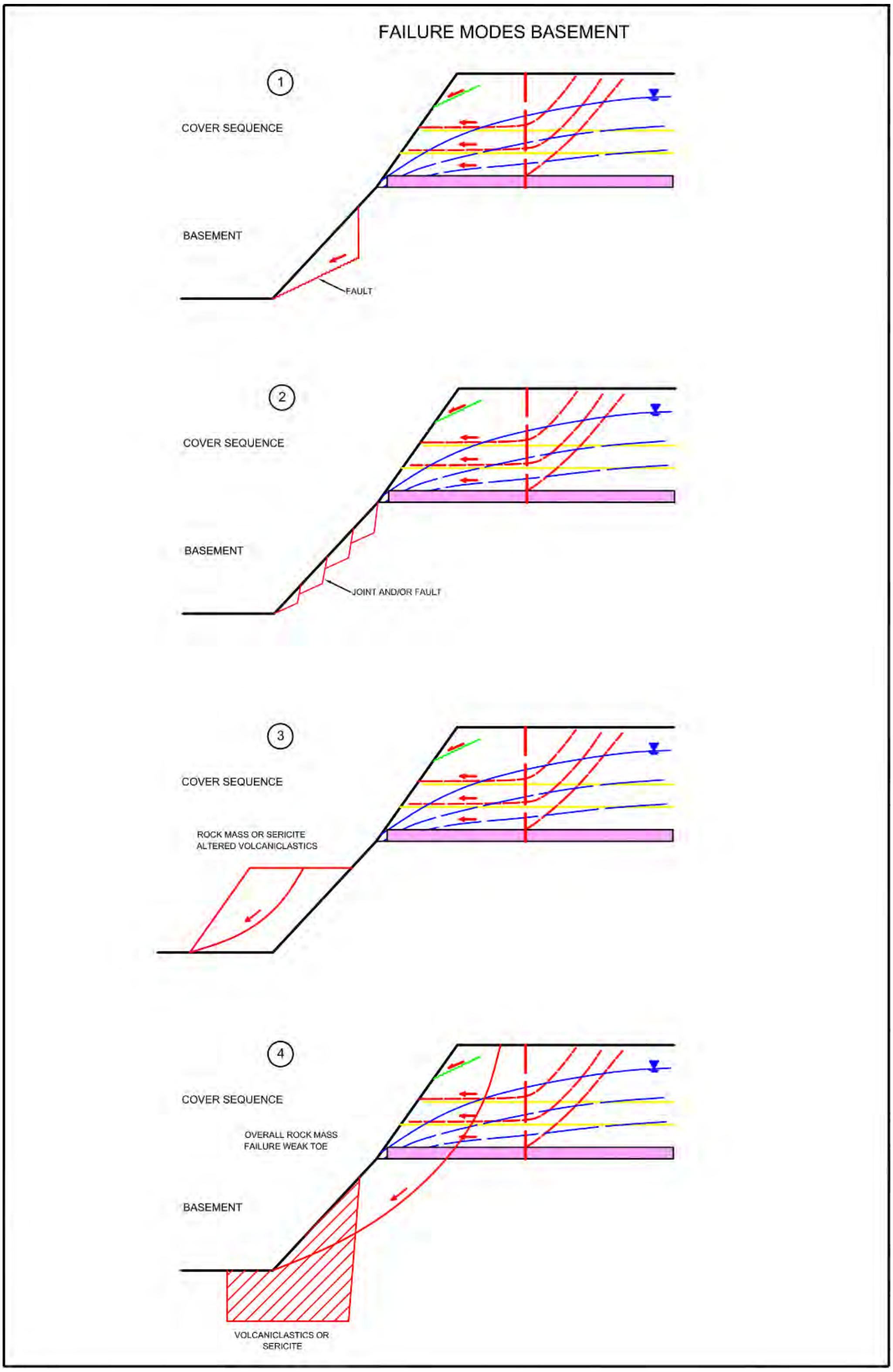


Figure 12: Conceptual failure modes in the Basement.

7.3. <u>Structure</u>

The design of slopes at Olympic Dam is predominantly controlled by structure. Extensive geotechnical analysis of the structural conditions has been undertaken based on the BHPB structural model.

7.3.1. Design Shear Strengths

The design defect shear strengths have been selected from laboratory testing (including direct shear, XRD, Atterberg limits and PSD) based on experience and engineering judgement. Defect strengths have been considered separately for the Cover Sequence and Basement. Defect type is the best predictor of shear strength. Defect types have been grouped into faults and shears; and joints and veins.

For the Cover Sequence the design defect shear strengths are:

- Faults and shears $c' = 0$, $\phi' = 22°$ and
- Joints and veins $c' = 0$, $\phi' = 35°$.

For the Basement the design defect shear strengths are:

- Faults and shears $c' = 0$, $\phi' = 20°$;
- Joints and veins $c' = 0$, $\phi' = 35°$, and
- Bedding (volcaniclastics) $c' = 0$, $\phi' = 30°$.

7.3.2. Methodology

Assessment of potential structural failures was carried out using kinematic and statistical methods. Kinematic analysis is a stereographic technique that assesses the critical failure mechanism and controlling defect sets. The method utilises the mean structural orientation of defect sets and adopted design shear strengths to assess the slope angle at which failure would occur for a given pit wall orientation.

After the critical failure mechanism is identified a statistical risk based analysis is carried out which takes into account the distribution of orientations and lengths within a defect set compared with just the mean orientation in a kinematic assessment. The assessments were carried out separately for the Cover Sequence and Basement.

The scale of the slope, the strength of the concentration of data and the nature of the defects defines which data set is the most relevant for the design. The slope design terminology used here is defined in Figure 13. The analyses were carried out separately for minor structures (joints and veins) and major structures (faults and shears) defects in both the Cover Sequence and Basement.

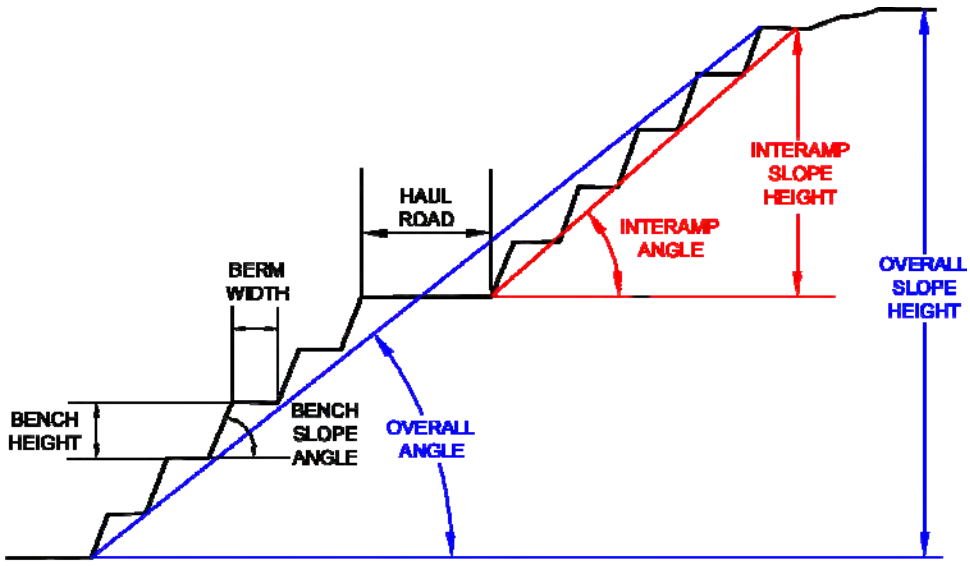


Figure 13: Slope Design Terminology

7.3.3. Cover Sequence

Statistical analysis of the borehole ATV structural data has been carried out for the Cover Sequence structural domains. The analysis results are summarised in Table 8 below and indicate:

- IRA angles in the range 55° – 65° are appropriate depending on the domain.
- Bench slope angles in range 85° – 90° are appropriate depending on the domain.

7.3.4. Basement

Statistical analysis of the borehole ATV structural data has been carried out for the Basement structural domains.

The results are summarised in Table 9 below and indicate:

- IRA angles in the range 50° – 65° are appropriate depending on the domain.
- Bench slope angles in range 75° – 85° are appropriate depending on the domain.

TABLE 8
STATISTICAL EVALUATION OF COVER SEQUENCE STRUCTURAL DATA (2007-2008 DRILLING)

DOMAIN	PIT WALL ORIENTATION	DEFECT TYPE		FAILURE MECHANISM P / W [1]	INDICATED SLOPE ANGLE (deg)	
		MINOR	MAJOR		BENCH	IRA
Southeast	315	•		P	90	
			•	P		65
Southwest	45	•		P	85	
			•	P		>65
Northwest	135	•		P	90	
			•	P		>65
Central	225	•		P	85	
			•	P		55

[1] P- Planar; W - Wedge

TABLE 9
STATISTICAL EVALUATION OF BASEMENT SEQUENCE STRUCTURAL DATA (2007-2008 DRILLING)

DOMAINS	PIT WALL ORIENTATION	DEFECT TYPE		FAILURE MECHANISM P / W [1]	INDICATED SLOPE ANGLE (deg)	
		MINOR	MAJOR		BENCH	IRA
East	270	•		P	75	
			•	P		55
Southeast	315	•		P	75	
			•	P		50
Southwest	45	•		P	85	
			•	P		65
Northwest (granite)	180°	•		P	85	
			•	P		65
Northwest (Volcaniclastics)	180°	•		P	80	
			•	P		60

[1] P- Planar; W - Wedge

7.4. Rock Mass

7.4.1. Overall Circular Failure

The overall rock mass capacity of the slopes has been assessed using Limit Equilibrium techniques. The rock mass capacity of both the Cover Sequence and Basement is typically high and the resulting factors of safety indicate more than adequate capacity as indicated in the example presented in Figure 14 below. There are some weaker rock masses of limited aerial extent including the volcaniclastic and highly altered granite breccia. While these units have less capacity they have a limited impact on overall stability.

Limited pseudo static seismic analysis has also been undertaken to date. Rock mass stability is generally acceptable with more detailed analysis proposed in future works.

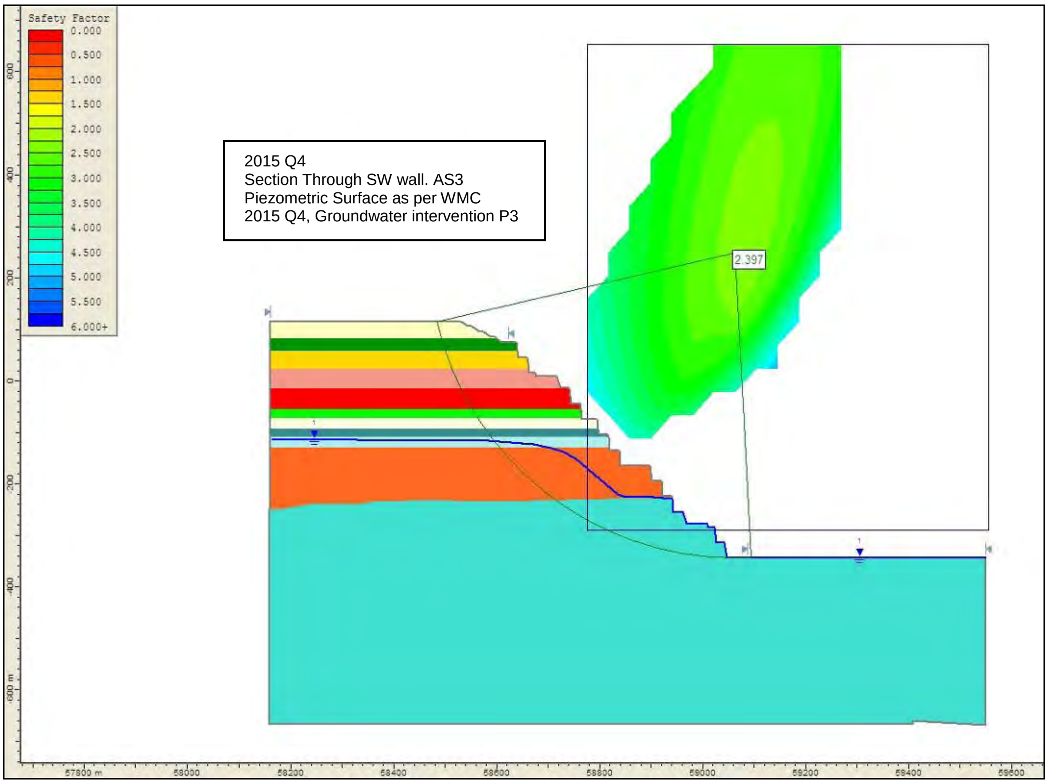


Figure 14: Overall slope stability for south west wall of starter pit (Sep 07)

7.4.2. Circular Failure of Cutbacks

The weaker units are exposed in cutback situations as shown below in Figure 15 and 16. These analyses indicate that for some geometries, where saturated conditions were modelled, marginal factors of safety were predicted. These analyses suggest that depressurisation of these rock mass units is required in order to achieve acceptable performance. Groundwater analyses suggest that this depressurisation is achievable with drain holes.

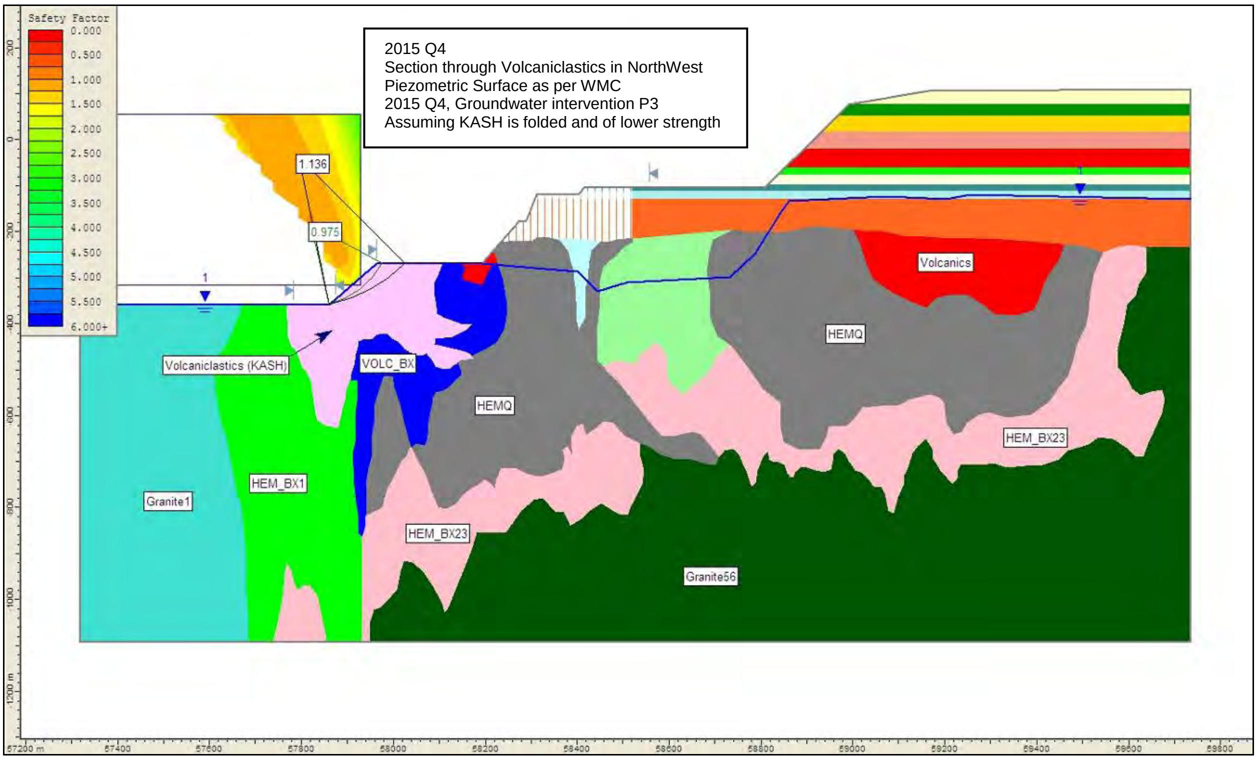


Figure 15: Deterministic Analyses – North East Volcaniclastics. Overall slope stability for south west wall of starter pit.

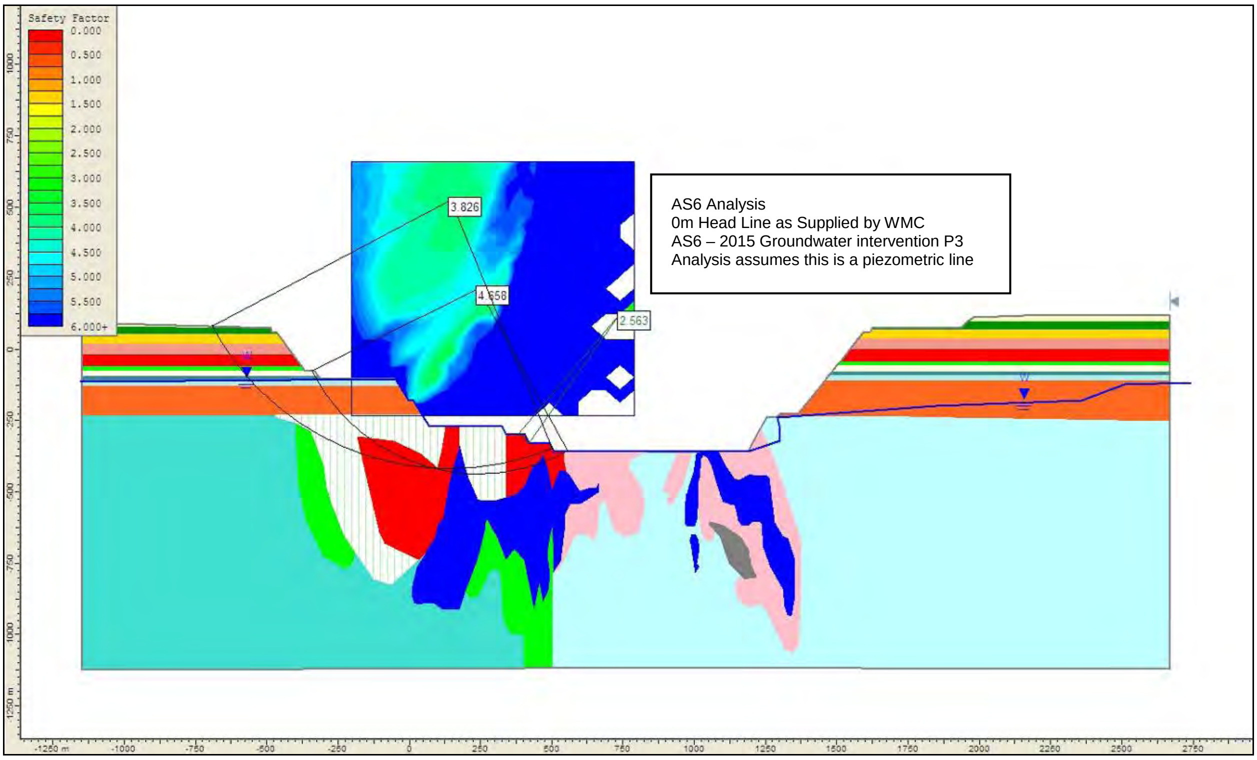


Figure 16: Deterministic Analyses – AS6 North West

7.5. Combined Rock Mass And Discrete Structure

It is considered that the critical mode of failure for the cover sequence is likely to comprise block sliding on shallowly dipping shears in Tregolana shale with failure through the rock mass above the shear. Analysis has been undertaken to assess this mode of failure as presented in Figure 17 below. The analysis indicates that acceptable factors of safety can be achieved for the design groundwater case P3.

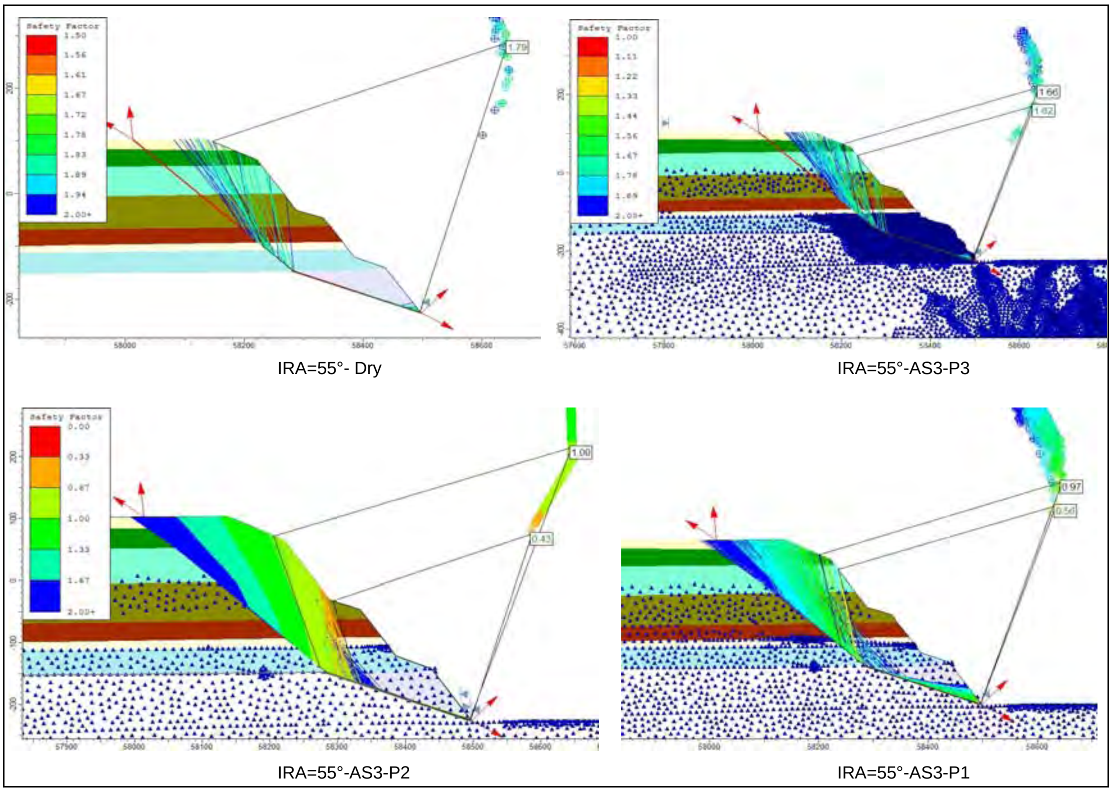


Figure 17: Limit Equilibrium Analysis Results for Model 2

7.5.1. Finite Element Modelling

Finite element modelling has been undertaken to confirm the results of the limit equilibrium analysis and to investigate a key geotechnical section in 2D using Phase2. Figure 18 shows an example output from this modelling.

This modelling has been used to:

- Confirm the failure mechanisms,
- Compare the estimate of factor of safety of limit equilibrium methods,
- Estimate displacements, and
- Estimate the impact of underground development on stability.

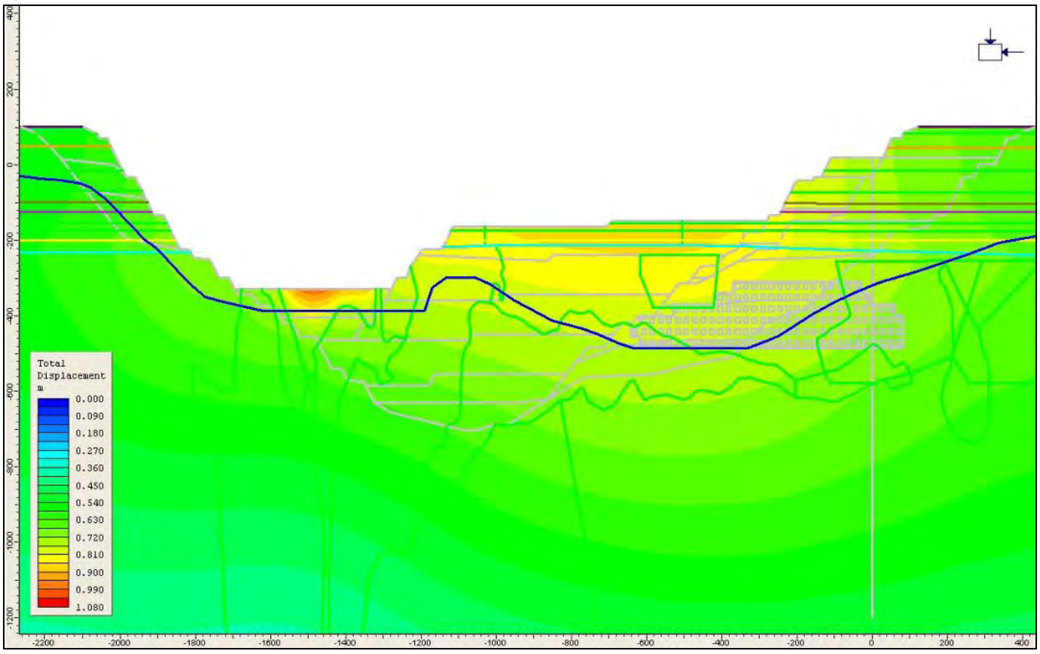


Figure 18: Finite element modeling - Section AS3 - Displacements at 2016 Q4

In additional pseudo coupled stress-displacement - pore pressure modelling was undertaken in 2D using Phase2. This approach was developed in house and used to understand the potential beneficial impact of unloading on depressurization of the slopes.

Three dimensional modelling of the Open pit and underground workings has been assessed using Abaqus finite element software. Abaqus is a general purpose, discontinuum/continuum, 3-D, non-linear, finite element analysis program designed specifically for analysis of problems where there is significant plasticity, high levels of deformation and large numbers of material discontinuities. An example of the output from modelling is presented in Figure 19 below.

a)

b)

c)

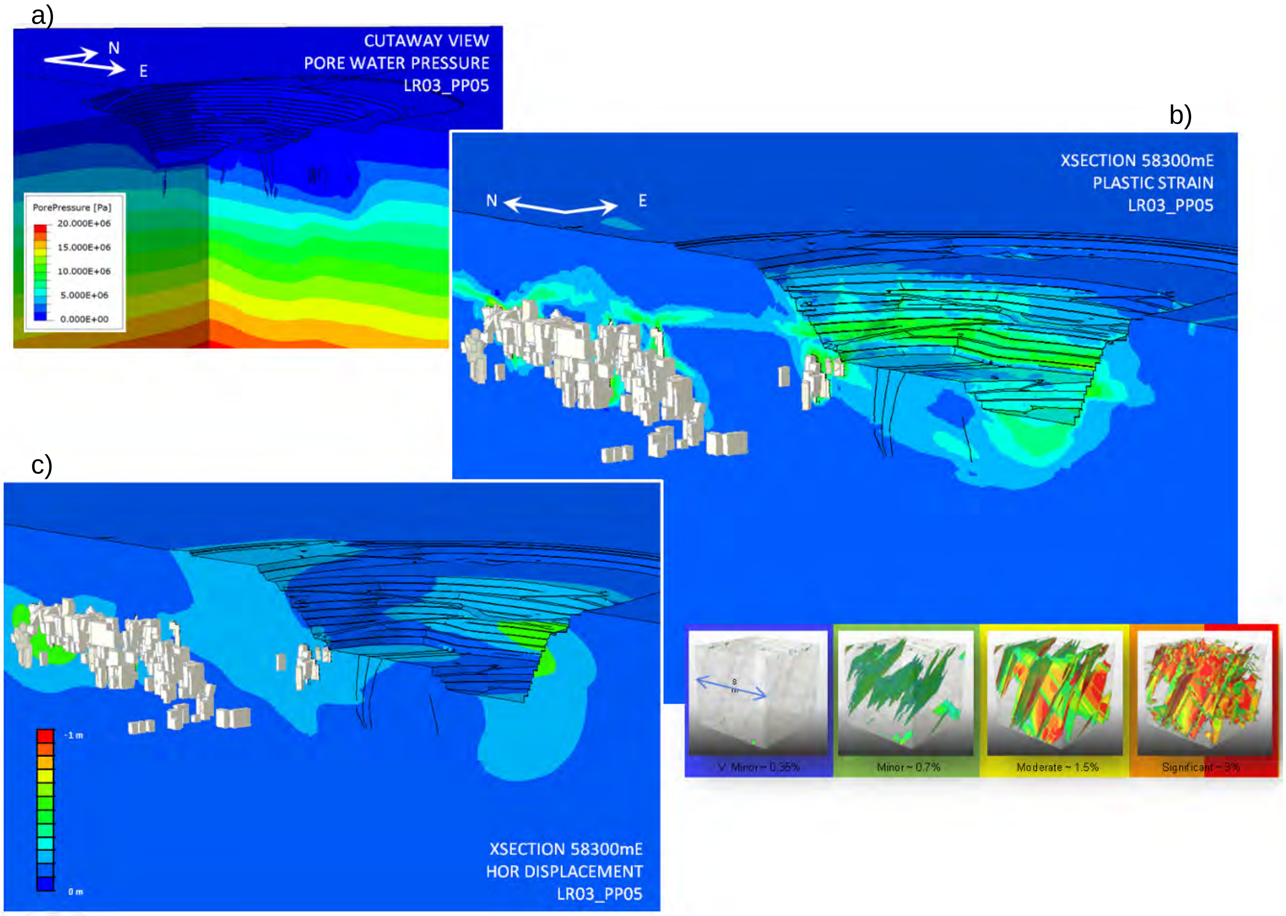


Figure 19: Cross Section of 3D model for 2018 58300mE, showing a) base case pore water pressure, b) damage and c) horizontal displacement.

8. ROCK STORAGE FACILITY

From a stability perspective the design of the Rock Storage Facility (RSF) is a comparatively simple assessment. The vast majority of the rock is of high strength to very high strength and will be relatively durable and sound. The exception to this is comparatively minor volumes of the shallow unconsolidated sediments and possibly some parts of the Tregolana Shale.

The design elements of the RSF are as follows:

- Rock fill will produce rill angles of approximately 35° - 40°.
- Lift heights of 50m are recommended except in the minor poorer units where the lift heights are to be 20-30m.
- Operation berms are proposed such that overall angles do not exceed 30°.
- Based on this configuration and published strengths appropriate for these materials, provides an overall FOS in excess of 1.2 as presented below in Figure 20.

- The rock fill is expected to be free draining such that pore pressure build up in the dump is considered highly unlikely.
- The dump foundation is above the water table.
- The dump foundation is predominately sand with some discontinuous sandy clay layers of a total depth ranging from a 1-20m deep overlying weathered limestone.
- Design earthquake loading of the RSF is expected to result in some displacement (predominately settlement) and be within tolerable limits.
- The offset between the RSF and the open pit is intended to produce a decoupled performance. This has been assessed by a review of a) credible failure paths in pit stability analysis and b) assessment of loading impacts from the dump on the pit as presented in Figure 21.

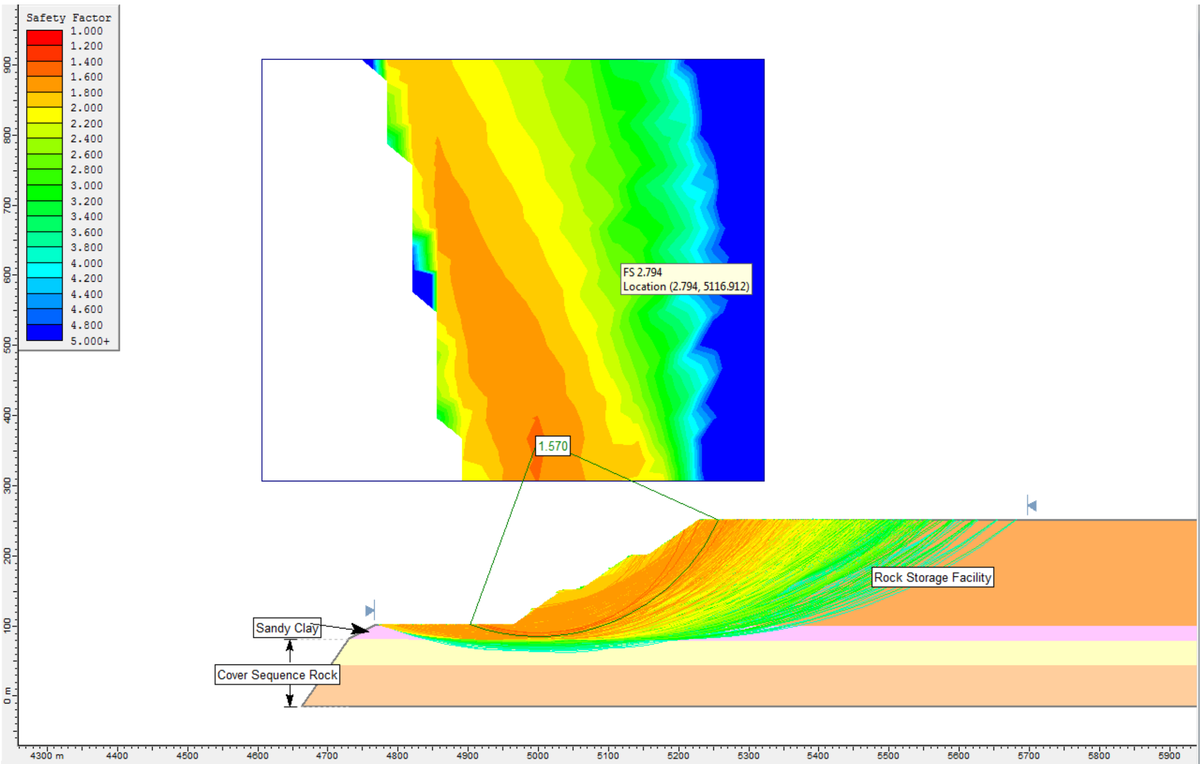


Figure 20: Stability of the Rock Storage Facility

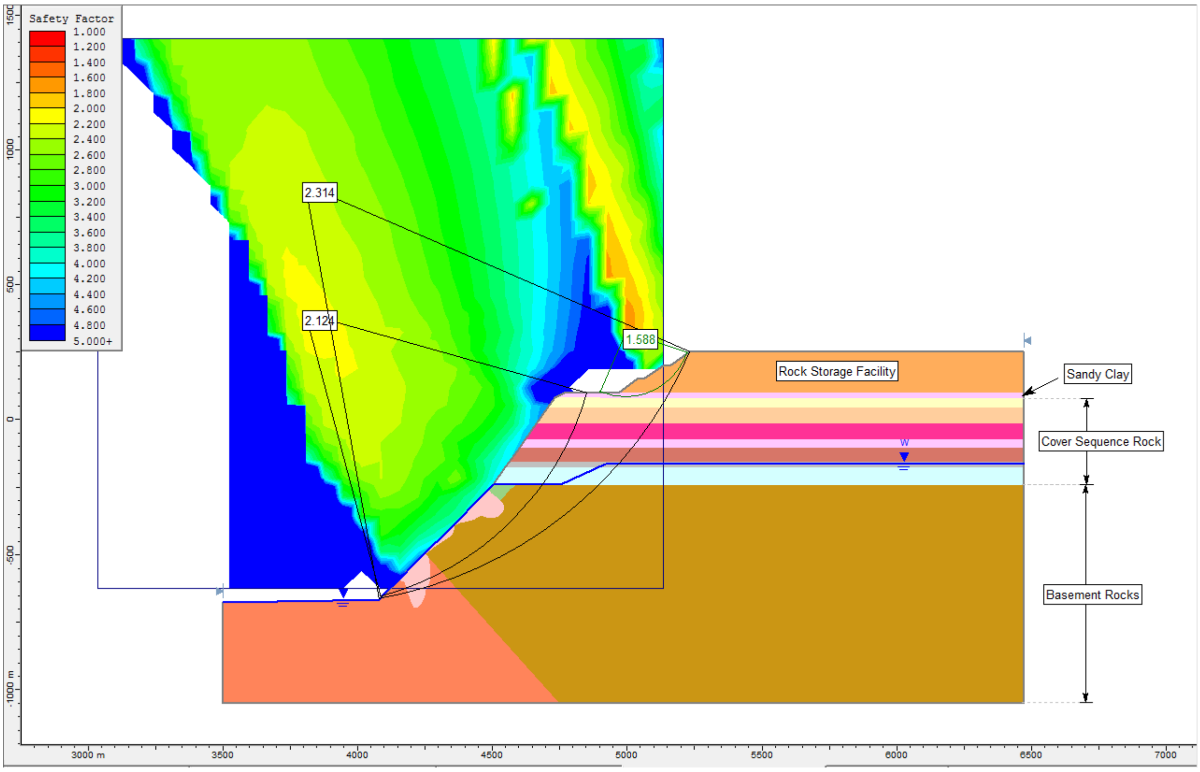


Figure 21: Impact of the rock storage facility on pit stability.

The rock mass strengths used in the RSF stability analyses (Figures 20 and 21) were presented Table 6. The strengths used for the rock fill and surface sands and clays (ZRS) are listed in Table 10 below.

TABLE 10
STRENGTHS USED IN RSF ANALYSIS

MATERIAL	UNIT WEIGHT (KN/M^3)	COHESION (KPa)	PHI (°)
Rock Fill	18	50	35
Surface Sands and Clays (ZRS)	18	5	25

For and on behalf of
PELLS SULLIVAN MEYNINK

MARK FOWLER

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1. Hoek, E. & Brown, E.T. (1997). Practical Estimates of Rock Mass Strength. *International Journal of Rock Mechanics and Mining Sciences.* Vol 34, No 8, pp1165-1186.
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3. T.D. Sullivan (Pells Sullivan Meynink Pty Ltd & University of NSW) (2006). Pit Slope Design and Risk – A View of the Current State of the Art. *The South African Institute of Mining and Metallurgy. International Symposium on Stability of Rock Slopes in Open Pit Mining and Civil Engineering.* Pgs 51-78
4. J. Read & P. Stacey (2009). Guidelines for Open Pit Slope Design. CSIRO Pub. Pg P235.

APPENDIX C2

Management of spent acid plant catalyst

C2.1 INTRODUCTION

To efficiently and effectively convert sulphur dioxide (SO_2) generated from both the smelting operation and the additional elemental sulphur burning into sulphur trioxide (SO_3), the high SO_2 off-gas is passed through a converter, which is essentially a tower containing multiple beds of a catalyst, as described in Section 5.5.4 of the Draft EIS. The resultant SO_3 is subsequently used to produce sulphuric acid within the absorption sections of an acid plant.

Typically, the catalyst used consists of either vanadium pentoxide or caesium-coated (6–9% by mass) silica rings of around 1–2 cm diameter. Periodically, the catalyst requires screening to maintain conversion efficiency as the silica rings break down, resulting in the compaction of the catalyst beds and a reduction in the surface area available to facilitate the conversion reaction. As discussed in Section 5.6.6 of the Draft EIS, the installed capacity of acid plant catalyst would be around 4.1 ML.

Routine maintenance is undertaken to screen the fine catalyst material from the gas converter beds, with the first bed being screened every three years, the second bed every six years and the third and fourth beds every nine to 12 years. On average, such screening is expected to generate around 225,000 litres of catalyst fines every three years, which initially would be transferred to lined steel drums and stockpiled for disposal or recycling.

BHP Billiton waste management practices are aligned with the waste hierarchy, and several reuse and recycling options have been, and in some cases continue to be, investigated for the treatment of spent acid plant catalyst fines, including:

- the on-site amalgamation of fines into a usable product suitable for reuse in a gas converter
- the reuse of the catalyst fines, as done in a smaller-capacity gas converter
- the return of the spent catalyst to the manufacturer for remanufacture into catalyst
- the transport of the catalyst to a vanadium producer for processing to a saleable vanadium product
- two methods of fixation, modification and stabilisation, whereby the vanadium pentoxide base material is rendered inert through either the modification, or phase change, of vanadium pentoxide to vanadium oxide, or the encapsulation of the catalyst material in a non-leachable matrix
- burial in landfill or charging into stopes as backfill material, together with cement aggregate fill (CAF).

These options are detailed further in the following sections.

C2.2 MANAGEMENT ALTERNATIVES

C2.2.1 RECYCLING AND/OR REUSE

The recycling and/or reuse of spent catalyst is considered the most environmentally sustainable disposal option. Four options were investigated: the on-site amalgamation of fines into a product suitable for reuse in a gas converter; the reuse of the catalyst fines as done in a smaller-capacity gas converter; the return of the spent catalyst to the manufacturer for remanufacture into catalyst; and the transport of the catalyst to a vanadium producer for processing to a saleable vanadium product.

The on-site amalgamation of fines into a reusable product is an untried and unproven technology, and a literature review failed to identify any information relating to the possibility of this method being successful. The amalgamation of materials, however, is a well-documented principle, and it is possible that a suitable binding agent could be found to enable this practice to be adopted within the Olympic Dam gas converter. Further work on this process, including amalgamation and efficiency trials, may be an area for future research.

The reuse of catalyst fines as done in a smaller converter was also considered, however the market for such material is not known, and no instances could be found of gas converter operators choosing to work in this manner. It is therefore not considered a viable alternative.

The recycling of catalyst fines by the manufacturer has been undertaken in some instances, generally under strict guidelines regarding the levels of contaminants in the spent catalyst materials, particularly concentrations of mercury. Sampling of the Olympic Dam catalyst indicates low concentrations of mercury, possibly making the catalyst suitable for this course of action. However, sending such a large quantity of spent catalyst to the manufacturing facility overseas poses cost and regulatory difficulties so significant that this course of action is unlikely to be effective.

Australia's only vanadium mining and processing operation closed as a result of prolonged low vanadium prices, eliminating the possibility of the vanadium pentoxide being reclaimed from the catalyst material for reuse.

C2.2.2 FIXATION, MODIFICATION AND STABILISATION

Fixation, modification and stabilisation are the processes by which the vanadium pentoxide base material, which is classified as a toxic substance, is rendered inert through either the modification, or phase change, of vanadium pentoxide to vanadium oxide, or the encapsulation of the catalyst material in a non-leachable matrix, which effectively renders the material safe. Two methods for the fixation of the catalyst were investigated: the charging of catalyst into a furnace, either on-site or externally; or the encapsulation of the material. Encapsulation effectively binds the spent catalyst in a proprietary magnesium oxide matrix, significantly reducing leaching and environmental exposure. The end product is suitable for use as road base or as a building material.

Discussions with encapsulation vendors established that some trials have been performed on wastes containing vanadium compound, however the success of these has varied, and they were typically not considered to be successful. Leaching trials indicated a reduced leachability, although this varied with the waste composition. The ratio of encapsulation base to waste is approximately 0.25, resulting in about 1,000 tonnes of inert material being produced from 735 tonnes of spent catalyst.

The waste is handled on-site, reducing the regulatory approvals necessary for this treatment method. However, a large encapsulation unit must be transported to site, incurring some traffic disruption and transport costs. Potential exposure to dusts during handling (feed to the encapsulation unit) exists, and would have to be considered further. The cost of treatment varies depending on the composition of the waste but it is considered expensive relative to other treatment methods. Some savings could be made if the material was used as a road base or hardstand, but the varied results of the leaching trials may result in the material being unsuitable for this use. For these reasons, this treatment and disposal method is not considered viable now.

A further option is the charging of the material into a furnace, reducing the vanadium pentoxide component of the catalyst to a glass oxide phase, effectively exhibiting zero leachability and exposure risk. This makes subsequent handling and disposal (if necessary) less complicated.

Trials at an Australian smelter indicate that the charging of the material into the sinter plant, followed by processing through the blast furnace, had little or no impact on the metallurgical process, apart from a rise in silica as a result of the smelting of some quartz packing. No information could be obtained about hygiene exposures, fugitive emissions or stack emission concentrations, all of which are areas of uncertainty. Similar trails at a Western Australian smelter indicate there was no increase in vanadium concentrations in dust, fugitive or stack emissions, and that most of the vanadium reported to the slag phase. No personnel exposure monitoring results could be obtained.

There are a number of key differences between the processes at the two smelters benchmarked above that add complexity to this issue for Olympic Dam. The most significant of these is that the electric furnace slag is recycled, both back into the furnace and through the slag milling circuit, which may result in a recirculating vanadium load in the smelter leading to process difficulties at later stages (i.e. at the refinery or slimes treatment plant). In general, it can be assumed that vanadium behaves as iron, and as such it is likely that vanadium in slag reprocessed via the slag mill would be directed to tailings. Another key difference is the nature of the flash furnace, which renders it unsuitable for the charging of large quantities of material due to the risk of adverse process effects such as bath foaming.

For this reason, the on-site charging of spent catalyst would most likely be done via the electric furnace, utilising its existing revert and coke charging bins. There are a number of exposure risks associated with this method, particularly that the spent catalyst would be transported to the charging bins through the existing uncovered conveyor system, increasing the risk of wind-blown particles escaping. Also of concern is the reliability of the gas cleaning system, a failure of which may result in unsmelted dust from the surface of the furnace bath being emitted unscrubbed through the bypass stack, further increasing exposures. When the gas cleaning system is operating, there is a risk that the bleed water for the venturi scrubber and the quench tower would contain elevated concentrations of vanadium compounds as a result of the dissolution of the material in the weak acid environment.

An option posed was to transport the spent catalyst materials to another smelter for treatment, however the potential legal implications and costs associated with this are considered prohibitive.

C2.2.3 BURIAL

The most obvious disposal option is burial in either the tailings storage facility (TSF), in landfill, or charging into stopes as backfill material together with cement aggregate fill (CAF).

Two options were identified for the disposal of the spent catalyst as backfill into empty stopes, as material mixed into the CAF during or before filling of the stope, or as drums trucked underground and added to the partially filled stope as space allowed. Of the two methods, the trucking can be discounted due to the unknown compressive strength of the partially or fully filled drums, which may result in the backfilled stope not having the required strength. Alternatives such as the trucking of drums underground, followed by the emptying of their contents into the stope, are not considered viable due to the increased risk of personnel exposure during handling.

The addition of spent catalyst during the production of CAF would be a more suitable alternative, the costs of which would be substantially less than those of the options described above, due to lower transport costs. The unknown compressive properties of the material could be further investigated to determine suitability as CAF, and a further cost saving could be derived from decreased CAF requirements. The process of catalyst addition to the CAF mix would need further investigation and engineering to ensure that exposure to dusting was minimised. In the event that the open pit developed beyond the 40-year timeframe assessed in the Draft EIS and reclaimed the underground workings, some spent catalyst-filled stopes would be exposed, the implications of which would require further investigation.

Disposal to an off-site storage and disposal facility is not considered a viable option. There are no storage facilities in South Australia licensed to take delivery of materials containing vanadium compounds such as spent catalyst. There is one facility at Tullamarine, Victoria, and a similar facility in NSW, however the interstate transport of wastes represents a significant regulatory hurdle.

The remaining landfill options involve excavating a trench or pit, and the disposal of the catalyst either in the existing steel drums, in corrosion-resistant poly-containers or without containment, or disposal to the TSF. There are two major methods of disposing of material to the TSF: placing the material in the TSF directly, or adding it to the process at some defined point and combining it with the tailings slurry piped to the storage facility.

The most appropriate location for the addition of spent catalyst to the tailings stream would be the disposal tanks immediately before the tailings pumps. The major consideration with the addition of spent catalyst to the tailings disposal stream is that the catalyst material has a higher specific gravity than the slurry (1.6 versus 3.3), meaning that the catalyst material may build up at the base of these tanks and clog the disposal pumps and lines. Other issues requiring consideration include the particle size of the spent catalyst (up to 20 mm square approximately), which may damage the slurry pumps and also might clog the lines to the TSF. The actual addition method would also require consideration; that is, how the material would be added to the tanks while minimising personnel exposure and the chance of clogging.

The addition of a crushing operation to the above method would eliminate the bogging and pump damage concerns, and may also promote increased or more effective mixing, however it is considered that the crushing operation itself, and the additional dusting, handling and machinery requirements it creates, make this option unattractive.

Regulatory approval has been granted in the past for other similar, albeit smaller, disposal operations at the TSF. In all cases, it is considered that the TSF has sufficient integrity and expanse that the addition of spent catalyst would represent no increase in environmental impact.

C2.3 CONCLUSION

As discussed in Section 5.6.6 of the Draft EIS, numerous options are being investigated for the reuse, recycling and/or disposal of spent catalyst, the details of which are presented here. The existing operation currently disposes of spent catalyst to the TSF, and it is likely that, in the absence of a viable, cost-effective recycling or reuse solution, this method would continue for the expanded operation.