

Appendix B: History of NTSF

Table of Contents

Executive Summary	1
B1. Introduction	2
B1.1 Background	2
B1.2 Reports and Data	2
B1.3 Nomenclature	2
B2. Design and Construction	3
B2.1 Overview	3
B2.2 Stage 1	3
B2.3 Stage 2A and Stage 2B	6
B2.4 Stage 3	7
B2.5 Stages 4 to 9	8
B2.6 Stage 10	10
B2.7 Buttress Construction	11
B2.7.1 Stage 1 Buttress Construction	12
B2.7.2 Stage 2 Buttress Construction	14
B3. Geotechnical Investigations	18
B3.1 Foundations & Construction Materials	18
B3.1.1 Pre-Construction Investigations	18
B3.1.2 STSF Investigations	19
B3.1.3 2017 Investigation	20
B3.1.4 2018 Investigations	20
B3.2 Tailings	21
B3.2.1 Overview	21
B3.2.2 2004 Investigations	21
B3.2.3 2013 Investigations	21
B3.2.4 2016 Investigations	21
B3.2.5 2017 Tailings Investigation	23
B4. Analyses	25
B4.1 Design Parameters	25
B4.2 Results of Analyses	26
B4.2.1 Overview	26
B4.2.2 Stage 1	27
B4.2.3 Stage 2	27
B4.2.4 Stages 3 to 6	27
B4.2.5 Stage 7	28
B4.2.6 Stage 8	28
B4.2.7 Stage 9	28
B4.2.8 Stage 10	28
B5. Inspections and Surveillance Monitoring	30
B5.1 Overview	30
B5.2 Displacement	30
B5.2.1 Terrestrial Monitoring	30
B5.2.2 Satellite Monitoring	33
B5.3 Piezometers	39
B5.3.1 Stage 1	39
B5.3.2 Stage 2	39
B5.3.3 Stage 6	39
B5.3.4 Stage 9	40
B5.3.5 Stage 10	40

B5.3.6 Discussion	40
B5.4 Seepage	44
B6. Audits & Third-Party Reviews	46
B6.1 Overview	46
B6.2 KCB 2012	46
B6.3 GHD 2016	46
B6.4 Golder 2016	47
B6.5 Golder 2017	47
B6.6 KCB 2017	47
B6.7 CHPL ITRB 2018	48
B7. Tailings Management	49
B8. NTSF Embankment Failure	53
B8.1 Pre- Failure Condition	53
B8.1.1 Overview	53
B8.2 9 th March Time Line	53
B8.2.1 07:30	53
B8.2.2 09:00	54
B8.2.3 11:00	56
B8.2.4 12:00	56
B8.2.5 14:30	56
B8.2.6 16:00	57
B8.2.7 18:45	57
B8.3 Post Failure Condition	58
B8.3.1 10 th March 2018	58
B8.3.2 11 th March 2018	58
B8.3.3 19 th April 2018	58

List of Tables

Table B2-1: Summary of Design & Construction	3
Table B3-1: Summary of pre-construction Investigations	18
Table B3-2: Summary of drillhole information	18
Table B3-3: Summary of core trench test pits in vicinity of slump	19
Table B3-4: Summary of Tailings Investigations	21
Table B3-5: Scope of Golder's 2016 testing	22
Table B3-6: Summary of 2016 Index Testing	22
Table B3-7: Summary of 2016 Shear Strength Testing	22
Table B3-8: Summary of 2017 tailings investigation	23
Table B3-9: Scope of 2017 laboratory testing	24
Table B3-10: Summary of 2017 index testing	24
Table B3-11: Summary of 2017 shear strength testing	24
Table B4-1: Summary of NTSF Design Parameters	25
Table B4-2: URS Undrained strength ratio profile based on 2013 CPT (2014-002)	26
Table B4-3: ATCW tailings strength parameters (2017-001)	26
Table B5-1: Annual inspections	30
Table B5-2: Stage 2B piezometers	39
Table B5-3: Details of 2013 VWP	40
Table B5-4: Details of 2017 VWP	40
Table B5-5: Comparison of 2017 and 2013 VWP data	41
Table B8-1: Personnel who observed embankment cracking on 9 March 2018	53

List of Figures

Figure B2-1: Stage 1 Starter Embankment (2018-025)	4
Figure B2-2: Stage 2A and 2B Embankments (2018-026).....	7
Figure B2-3: Stage 3 Embankment (2018-027).....	7
Figure B2-4: Stage 4 to 9 Embankment Configuration (2018-028).....	9
Figure B2-5: Stage 4 to 9 Embankment Detail (2015-002).....	9
Figure B2-6: Stage 1 and 2 Buttress Construction (2018-029).....	11
Figure B2-7: Stage 1 Buttress Static FOS for Southern Embankment (2017-001).....	12
Figure B2-8: Stage 1 and 2 Buttress Static FOS for Southern Embankment (2017-001).....	12
Figure B2-9: Approximate position of the western tip head of Stage 1 Buttress	13
Figure B3-1: Multistage CIU triaxial test data from NTSF, STSF and URC investigations	20
Figure B5-1: NTSF Southern Embankment Prisms – Staging and displacement vectors	31
Figure B5-2: NTSF Southern Embankment – Ground elevation and vertical displacement	31
Figure B5-3: Southern Embankment – Displacement vectors normalized for NTSF height	32
Figure B5-4: Southern Embankment – Vertical displacement normalized for NTSF height	32
Figure B5-5: Prism 6 – Comparison of survey and SMM data	35
Figure B5-6: Prism 31 – Comparison of survey and SMM data	35
Figure B5-7: InSAR surface movement measurements at Ch 2000	35
Figure B5-8: Airbus DS average velocity map for NTSF (01/17 to 02/18) (2018-032)	36
Figure B5-9: Airbus DS average velocity map for NTSF (01/17 to 09/18) (2018-033)	36
Figure B5-10: Typical InSAR displacement time series trends for the NTSF.....	37
Figure B5-11: InSAR rate of displacement related to Buttress 1 construction progress.....	38
Figure B5-12: CPTu pore water pressure gradients	41
Figure B5-13: Longitudinal profile of piezometric surface with time	42
Figure B5-14: VWP piezometer measurements showing stages of Buttress 1 construction	44
Figure B5-15: Ch1800 drain flow and NTSF decant pond level.....	45
Figure B7-1: NTSF showing spillway and decant pond	49
Figure B7-2: Stage 10 spigot arrangement (May 2017).....	50
Figure B7-3: NTSF spigot opening sequence between July 2017 and 9 March 2018.....	51
Figure B7-4: Tailings beach and decant pond levels	52

List of Annexures

Annexure BA Figures

- Figure B1 – Cadia NTSF Timeline 1995 to 2018
- Figure B2 – Cadia NTSF Timeline Dec 2017 to Mar 2018
- Figure B3 – Cadia NTSF Timeline 8 to 9 March 2018
- Figure B4 – Orthophoto Map – March 9, 2018
- Figure B5 – Annotated Orthophoto Map March 9, 2018
- Figure B6 – Orthophoto Map March 10, 2018
- Figure B7 – Annotated Orthophoto Map March 10, 2018
- Figure B8 – Orthophoto Map March 14, 2018
- Figure B9 – Annotated Orthophoto Map March 14, 2018
- Figure B10 – NTSF Previous Investigation Locations
- Figure B11 – NTSF Plan with CPT Locations

Annexure BB Nomenclature

Annexure BC Stage 10 Construction Program

Annexure BD Monitoring Information

Annexure BE Otus SSM Report

Annexure BF ATCW 9 March 2018 Inspection Report

Executive Summary

Historical information relevant to the Northern Tailings Storage Facility (NTSF) has been reviewed in detail and summarized in this appendix, under the headings of:

- Design and Construction
- Geotechnical Investigations
- Analyses
- Inspections and Surveillance Monitoring
- Audits and Third Party Reviews
- Tailings Management
- NTSF Embankment Failure

The timeline for the NTSF is presented as three Microsoft Project Gantt Charts in Annexure BA, each at a different time scale. The time scales of the three charts are:

- Figure B1 1995 to March 2018
- Figure B2 January 2017 to March 2018
- Figure B3 March 8, 2018 to March 14, 2018

For convenience each activity in the Gantt chart references a sub-section in this Appendix, which provides full details and in some cases photographs.

Orthophotos of the NTSF in the vicinity of the embankment failure are also included in Annexure BA. Orthophotos are provided for the following dates:

- March 9, 2018 Pre-failure condition,
- March 10, 2018 Following initial failure
- March 14, 2018 Following secondary failure

Annotated orthophotos are also included in Annexure BA for these dates and they also reference photographs and sub-sections in the appendix.

B1. Introduction

B1.1 Background

Cadia Valley Operations (CVO) is a gold/copper mining and processing complex 25 km south of the town of Orange, central west NSW. Cadia Holdings Pty Ltd (CHPL), a wholly owned subsidiary of Newcrest Mining Limited (NML), is the owner and operator of CVO.

The CVO complex comprises the Cadia Hill, Ridgeway and Cadia East mines, minerals processing facilities and associated infrastructure. Mining commenced in 1998, with current approvals taking the project through to 2031.

At the time of the NTSF embankment failure, there were two operational tailings storage facilities (TSF) at CVO; the Northern TSF (NTSF) and the Southern TSF (STSF). Both TSF embankments were constructed across the former Rodds Creek, the NTSF being at the upstream location and the STSF at the downstream location. The location of the NTSF and STSF are shown on Figure 1-2 in the Main Report.

Construction of Stage 1 of the NTSF was completed in 1998, while construction of Stage 1 of the STSF was completed in November 2001. By mid-2007, tailings and decant water impounded by the STSF had commenced to encroach on the downstream toe of the NTSF.

The NTSF is a Prescribed Dam under the requirements of the NSW Dams Safety Act 1978, with the NSW Dam Safety Committee (DSC) being the administering authority. At the time of the failure, the NTSF was assigned a Consequence Category of Significant with an environmental approval for a final crest level of 779 mAHD.

In the late afternoon of Friday 9 March 2018, following the identification of cracks on the dam crest earlier in the day, a slump occurred on the western side of the southern embankment of the NTSF.

B1.2 Reports and Data

The NTSF has been in operation for approximately 20 years and has been raised on average every two years. Together with the STSF, numerous reports have been prepared on investigations, design, construction, surveillance, monitoring, audits, operation etc. Documentation relevant to both the NTSF and STSF was assembled on a CHPL SharePoint site. Internal project document referencing appears in Appendix K in the format YYYY- 001 etc.

B1.3 Nomenclature

Nomenclature pertaining to the NTSF is provided in Annexure BB. Information provided in this annexure relates to:

- Co-ordinate systems;
- Level datum;
- Magnetic declination;
- Embankment chainage; and
- Embankment setout.

B2. Design and Construction

B2.1 Overview

Initial construction of the NTSF commenced in August 1997 to a height of 50m. Since then, the TSF has been raised eleven times, with the most recent raising being Stage 10 which was commenced in 2017.

A summary of the design and construction details is provided in Table B2-1. Details provided in the following sections are based on design and construction reports completed for each stage.

Table B2-1: Summary of Design & Construction

Stage	Crest Level (mAHD)	Max Height (m)	Construction Type	Design By (1)	Construction Completed	Construction Reference (Annexure BB)
1	700.0	50	Conventional	KP	May 1998	1998-001
2A	707.0	57	Downstream	WC	Aug 2000	2000-002
2B/1	710.5	60.5	Downstream	URS	May 2002	2002-001
2B/2	714.0	64	Downstream		June 2003	2003-001
3	718.5	68.5	Centreline	URS	Nov 2005	2006-001
4	723.0	73	Upstream	URS	Oct 2008	2008-001
5	729.0	79	Upstream	URS	Aug 2011	2011-001
6	732.0	82	Upstream	URS	Dec 2012	2013-001
7	735.0	85	Upstream	URS	Feb 2014	2014-001
8	738.0	88	Upstream	AECOM	Oct 2015	2015-001
9	741.0	91	Upstream	AECOM	Dec 2016	2017-006
10	744.0	94	Upstream	ATCW	Mar 2018 ⁽²⁾	NA

Notes:

- (1) KP; Knight Piesold, WC; Woodward Clyde, ATCW; ATC Williams. Woodward Clyde was acquired by URS who were subsequently acquired by AECOM.
- (2) Stage 10 was incomplete at the time of the NTSF Failure

B2.2 Stage 1

The Stage 1 starter embankment is an earth and rockfill dam with a maximum height of 50m. The dam was designed by Knight Piesold and Pells Sullivan and Meynink (PSM) for the CVO EPCM contractor Bechtel Minproc Joint Venture. At the time, the final design had the starter embankment being raised a further six times using modified centerline construction methods to a final height of RL741 (91m high). On completion of Stage 7, it was proposed that the downstream batter would be rehabilitated to a slope of 3H :1V with two 10m wide benches (1997-001).

The Stage 1 embankment design comprised a 1680m long embankment (Chainage 1500 to 3180) with a 16m wide crest at RL 700 mAHD. A 5m wide sloping clay core, thickening to 12m at the base, (Zone A) was bounded by rockfill shoulders (Zone B) with upstream and downstream slopes of 1.5H:1V. A 15m wide transition/filter (Zone C1) was provided between the clay core and downstream rockfill shoulder of the embankment.

During construction Rodds Creek was diverted through the dam foundation via a 1350mm diameter Helcore steel conduit which extended for a distance of 240m from an upstream earthfill cofferdam to the downstream embankment toe. The conduit was concrete encased through the clay core and the filter/ transition zone and subsequently pumped with controlled low strength material (CLSM) on completion of the embankment construction.

To control runoff during construction and to remove water collected within the upstream rockfill shoulder, as well as groundwater from natural springs in the Rodds Creek channel, a sediment dam (crest RL666) was integrated into the upstream face of the embankment. The sediment dam includes a 10m wide zone of drainage gravel (Zone C2) extending between RL661 and RL665. The gravel layer is drained by two parallel 150mm diameter slotted ABS pipes that discharge to the downstream via a 150mm GI pipe set into the concrete encasement to the Helcore diversion conduit.

A license condition of the NTSF was that seepage from the TSF should be no greater than that which could be expected from a TSF with a 1m thick clay liner with a permeability of $1 \times 10^{-9} \text{m/s}$. To achieve this criterion, a clay blanket was constructed upstream of the dam in Rodds Creek and a 1m thick clay layer at RL658 was constructed between the sediment dam and the clay core.

The general arrangement of the Stage 1 starter embankment is shown on Figure B2-1.

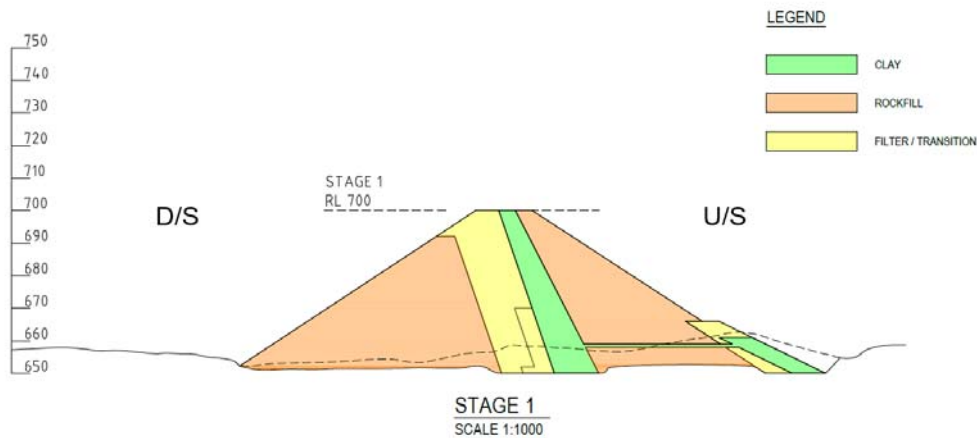


Figure B2-1: Stage 1 Starter Embankment (2018-025)

Material specified as Zone B was to be igneous rock (monzonite) with a maximum particle size of 600mm, won from the open pit and placed in 1.25m lifts, compacted with a minimum of 5 passes of a 10 tonne static weight vibrating roller. During construction it became apparent that the volume of monzonite was limited, and placement was constrained by double handling. As a consequence, the upstream shoulder was retained as Zone B (monzonite) while the downstream shoulder was changed to Silurian sedimentary rock (Zone D), with a maximum particle size of 300mm placed in 650mm lifts and compacted in the same manner as Zone B.

The Silurian sedimentary rockfill, designated as Zone D, was initially considered inferior to the igneous rock (Zone B) and as a consequence, the Stage 1 design was modified to incorporate a downstream waste rock berm (Zone B1), with a crest width of 32m at RL690 mAHD, and a downstream slope of 1.35H:1V. Zone B1 consisted of monzonite directly hauled from the open pit (2000-001).

Subsequently, direct shear tests on Zone D material indicated that it met the requirements of the original design, and placement of Zone B1 was postponed even though it formed part of the Stage 2 embankment. The direct shear tests on Zone D material are not reported in the construction documentation.

The Woodward Clyde technical specification for Zone B1 required the following:

- Removal of “any materials of stiff or lesser consistency” from the foundation,
- A maximum particle size of 1m;
- Spread in layers no more than 5m thick; and
- Trafficking with Cat 793 dump trucks and High Energy Impact Compaction.

A major change to the design during the construction, was the inclusion of filter / transition Zone C3 as an L shaped zone at the base of Zone C1. Whereas Zone C1 was < 300mm Silurian sedimentary rock visually selected in the open pit, Zone C3 was crushed to < 80mm, placed in 300mm loose layers and compacted with 1 pass of a 10 tonne static weight vibrating roller. The base leg of Zone C3 was 10m wide and extended up the abutments to RL695 as a 1m thick layer, while the upstand leg was 5m wide and extended full height to RL670 and then 3m height between RL670 and RL680.

During construction concerns were raised regarding the particle size distribution of Zone C1. Test excavations in this material indicated that voids were filled, while large scale slot tests indicated that the core was not subject to piping. Details of the Stage 1 construction materials is provided in Appendix D.

Excavation beneath the clay core and transition/ filter was undertaken to expose hard residual soil or extremely weathered rock. Where potentially permeable material was identified in the downstream side of the core trench, the core trench was widened (on the downstream side) by 6m and blanketed with Zone A. Zone C3 filter was then extended over the widened section of core trench.

Areas of core trench requiring special treatment including ripping of rock, widening, hand clean-off using compressed air and dental concrete were noted between:

- Chainage 2390 to 2400
- Chainage 2100 to 2200
- Chainage 1530 to 1725

Although basalt was intersected in the core trench between Chainage 1530 and 1725, pre-construction investigations (1997-001) indicated that the basalt bedrock extended to Chainage 1875, whilst probing of the core trench (1997-002) with a further thirty one (31) test pits suggested that the basalt extended to at least Chainage 2140.

No survey information is provided in the construction report on the depth of the core trench. However, based on test pits excavated at 50m intervals along the core trench, PSM recommended that, with the exception of the creek bed, the core trench be excavated to no more than 1.6m depth. In the creek bed a depth of 4m was indicated.

Although the shoulders of the embankment were designed to be founded on hard clay or extremely weathered rock, a large proportion of the clay beneath the downstream shoulder was excavated for use in the clay core. The extent of stripping and earthfill borrow areas is shown on the 1998 aerial photography (Photo B2-1).

Pneumatic piezometers were installed at five cross-sections, upstream, downstream, beneath and within the clay core and the transition/filter zone (1998-001).



Photo B2-1: Stage 1 NTSF extents showing stripping (1998 Aerial Photograph) (1998-002)

B2.3 Stage 2A and Stage 2B

The Stage 2A embankment was a 7m downstream raise and comprised a zoned earth and rockfill embankment with a 5m wide sloping core and a crest length of 1980m. The core was keyed into the top of the Stage 1 core and extended onto the abutments beyond the Stage 1 footprint. The upstream and downstream shoulders of the embankment are monzonite rockfill, compacted to a batter slope of 1.5H:1V (2000-002).

A 1m wide filter zone comprising sand sized material was constructed downstream of the core, while a transition zone was provided between the filter and downstream rockfill zone.

Stage 2 and subsequent stages used the following nomenclature for embankment zones.

- Zone 1 Clay core
- Zone 2A Fine Filter
- Zone 3A Transition Zone (Rockfill) – 600 mm layers
- Zone 3B Rockfill (Upstream and Downstream Shoulders) 1250 mm layers
- Zone 3C Rockfill (Buttress)
- Zone 3D Working Platform

The Stage 2B embankment was also designed as a 7m downstream raise with zoning identical to Stage 2A, however Stage 2B was constructed in two separate lifts, Stage 2B/1 and Stage 2B/2, each of 3.5m height.

In preparation for the Stage 2A raising, construction of the rockfill shoulder, started during Stage 1, recommenced in January 2000. The width of the rock shoulder was increased to 40m, to suit safe operation of the mine dump trucks and to provide a suitable base width for the 14m Stage 2 raising. In addition, the downstream slope was flattened from 1.35H : to 1V to 1.5H:1V.

Where the Stage 2 was extended beyond the Stage 1 footprint, the core trench was extended and the downstream shoulder was founded on very stiff to hard residual soil. Clay was sourced from borrow areas within the storage area, the Stage 2A downstream shoulder and from the right abutment 200m downstream of the dam (2002-001).

The general arrangement of the Stage 2A and 2B embankments are shown on Figure B2-2.

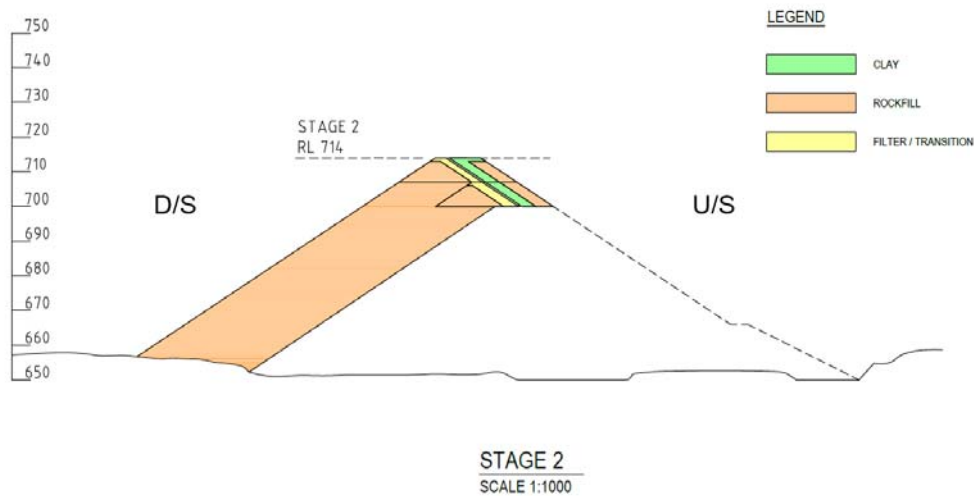


Figure B2-2: Stage 2A and 2B Embankments (2018-026).

B2.4 Stage 3

Stage 3 comprised a 4.5m high zoned earthfill embankment comprising a 3m wide central clay core (Zone 1), a 3m wide transition zone (Zone 3A) and upstream and downstream rockfill shoulders (Zone 3B). Where the embankment was constructed over tailings, a rockfill working platform (Zone 3D) was provided, while the core was keyed into the top of the Stage 2B/2 core (2006-001).

The upstream batter slope was designed at 2H:1V, while the downstream was 1.5H:1V consistent with the previous stages. The crest width was reduced from 14m in previous stage to 9m for Stage 3 and subsequent stages.

The general arrangement of the Stage 3 embankment is shown on Figure B2-3.

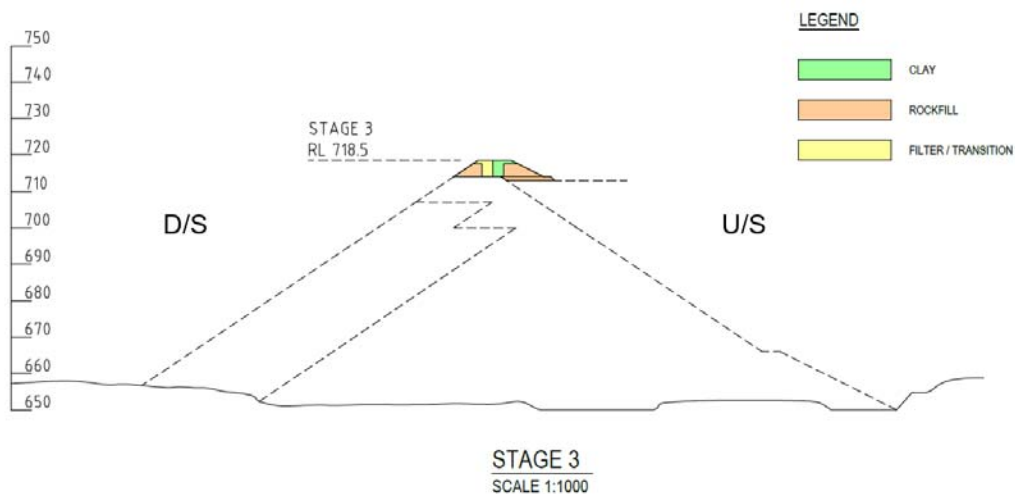


Figure B2-3: Stage 3 Embankment (2018-027).

An underdrain system consisting of a slotted collection pipe encapsulated within a filter blanket was provided over the full length of the upstream toe of the Stage 3 embankment. Outlet pipes were provided from the collection pipe to the downstream rockfill batter at 200m intervals between Chainage 1800 and 3600. The outlet pipes were concrete encased through the clay core with a filter sand plug immediately downstream of the concrete encasement (2006-001).

The NTSF Stage 3 arrangement is shown in 2006 aerial photography (Photo B2-2).



Photo B2-2: 2006 Aerial photography showing Stage 3 arrangement (2006-002)

B2.5 Stages 4 to 9

Stages 4 to 9 were upstream raises to the existing embankment with individual heights ranging between 3m and 6m and crest widths of 9m. With the exception of Stage 5 (6m height), upstream and downstream batter slopes were 2 H:1V. In the case of Stage 5, the downstream batter slope was flattened to 2.5 H:1V (2009-001).

Preparatory to the construction of each stage, a working platform of mine waste (Zone 3D) was pushed out over the tailings surface. The base of Stages 4 to 9 comprised low permeability clay (Zone 1), 0.8m thick, connecting with a 3m wide upstream clay face. The remainder of each embankment shell comprised compacted high strength igneous waste rock (Zone 3B), with a transition zone (Zone 3A) of finer well graded mine waste between the clay and rockfill.

Prior to the placement of the clay base to a particular stage, a 4m wide strip of geofabric was placed over the interface between the existing crest and the working platform. The purpose of the geofabric was to provide protection against tailings migration should a through-going crack develop at the potential "hinge point" between the old and new embankments. Each stage overlapped the crest of the previous stage leaving a 6m wide berm which was used for access.

During the construction of each embankment stage, tailings were being discharged during the construction. This required careful scheduling of works and in the case of Stage 5, the embankment was built to 3m height over the full length then raised to the final height of 6m.

Whereas the prior embankments had been constructed using civil earthworks contractors Stage 4 and onwards were constructed directly by Newcrest using direct plant hire. Quality control on Zone 1 was provided by an independent NATA registered soils testing laboratory, while the designer undertook site inspections on regular occasions.

In the case of Stage 8, four (4) underdrains were located at the downstream side of the working platform at Chainages 300, 600, 900 and 1173. The underdrains consisted of a 5m x 1.2m x 0.2m thick layer of drainage gravel encapsulated in geofabric with a slotted drainage pipe discharging through the Stage 7 crest (2015-001).

The general arrangement of the Stage 4 to 9 embankments is shown on Figure B2-4 while the detail of each stage is shown as Figure B2-5.

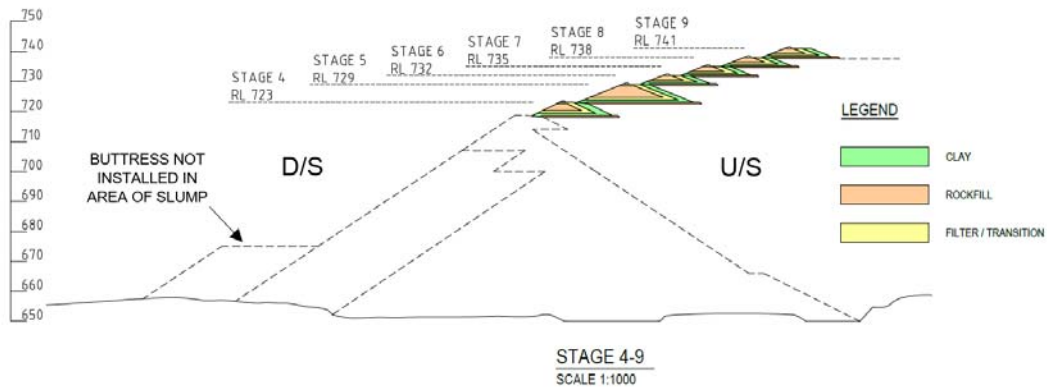


Figure B2-4: Stage 4 to 9 Embankment Configuration (2018-028).

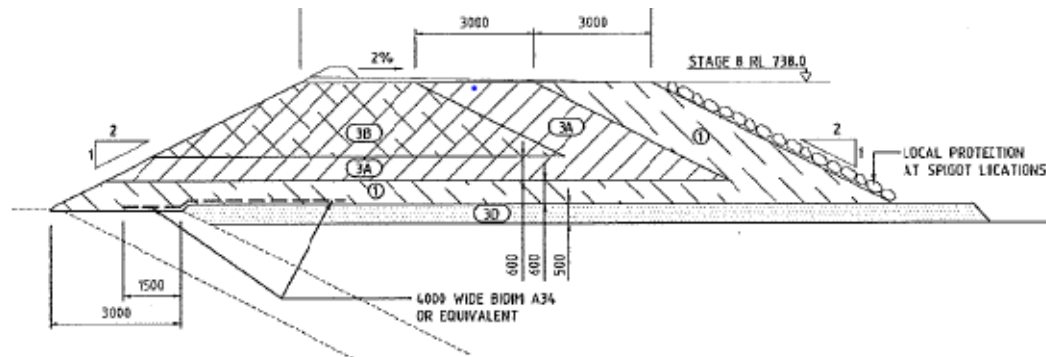


Figure B2-5: Stage 4 to 9 Embankment Detail (2015-002).

In mid-2007, prior to the construction of the Stage 4 embankment, a 35m wide berm of igneous mine waste was placed through the STSF decant pond at the toe of NTSF (2013-002). Over the following years, the berm was progressively raised (in 1.2m lifts) and lengthened to keep it above the STSF decant pond level.

At the time of construction, it was considered that the berm would serve several purposes.

- It provided the capability to easily provide a weighting berm for the NTSF embankment in the event that detailed design of future raises found that this to be necessary.
- It provided an additional haul route from the clay borrow areas on the eastern side of the STSF storage area.
- It provided an alternative future location for the STSF decant pumps.

The berm was not designed, nor specific foundation preparation undertaken and was only included in Stage 7 and subsequent stability analyses.

Photo B2-3, taken in 2010, shows the completed Stage 4 embankment and the Stage 5 working platform under construction. Other points to note with respect to this photograph are:

- Tailings have overflowed the Stage 4 crest (reported in Stage 5 Construction Report) (2011-001).
- The downstream berm across the upper reaches of the STSF has been completed.

- Tailings are starting to accumulate in a depression at the western end of the berm. The tailings originate from a STSF tailings pipeline break pressure overflow.



Photo B2-3: Downstream of NTSF embankment (2010) (2010-001)

B2.6 Stage 10

The Stage 10 embankment is a 3m high zoned earthfill embankment with similar zoning to the Stage 4 to 9 construction with the following subtle differences (2017-001):

- Crest level at RL744m.
- Upstream and downstream batters of 2 H:1V;
- A 6m wide geotextile at the interface between the working platform and Stage 9 embankment.

The Stage 10 embankment construction commenced on 27 February 2017 and generally advanced from the northwest (Chainage 0) to the south and east, progressing through the construction of working platform (Zone 3D), clay blanket, Zone 3A & 3B rockfill and upstream clay. Weekly plans indicate that the Stage 10 embankment construction in the vicinity of the slump was essentially complete by the end of July 2017 (Photo B2-4). As with the Stage 4 to 9 construction this work was undertaken by Newcrest using direct hire.

A complete program of Stage 10 construction is provided in Annexure BC.



Photo B2-4: Proposed Stage 10 Construction WE 02 August 2017 (2017-008)

B2.7 Buttress Construction

ATCW's interpretation of cone penetration tests (CPTu), undertaken as part of the 2017 geotechnical investigation indicated an overall strength profile lower than that used in the analysis of previous embankment stages and a long term static Factor of Safety (FOS) less than that recommended by both ANCOLD and NSW DSC. As a consequence, ATCW recommended the construction of two buttresses to achieve acceptable FOS under both static and seismic loading (2017-009).

The two buttresses, referred to as Stage 1 Buttress and Stage 2 Buttress are shown on Figure B2-6.

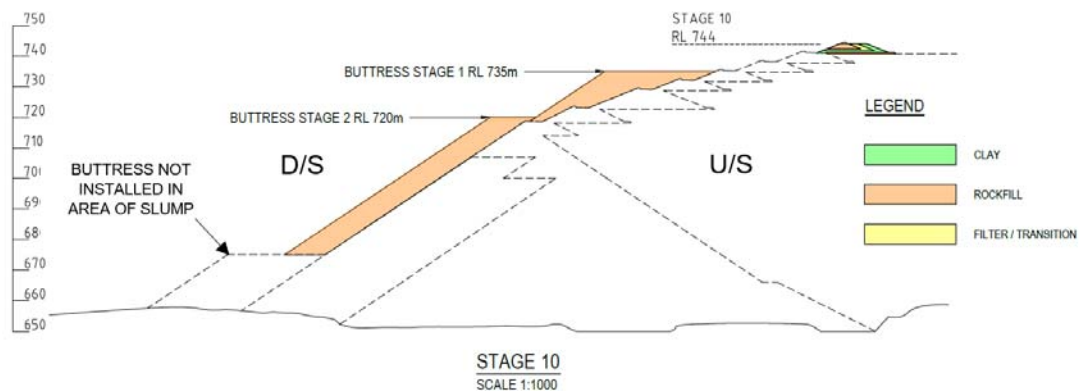


Figure B2-6: Stage 1 and 2 Buttress Construction (2018-029).

The Stage 1 Buttress was designed by ATCW to increase the static short term (undrained) FOS of a potential slip surface extending through the upstream raises as shown in Figure B2-7, while the Stage 2 Buttress was designed to increase the static long term (drained) FOS of a potential slip surface extending through the foundations as shown in Figure B2-8.

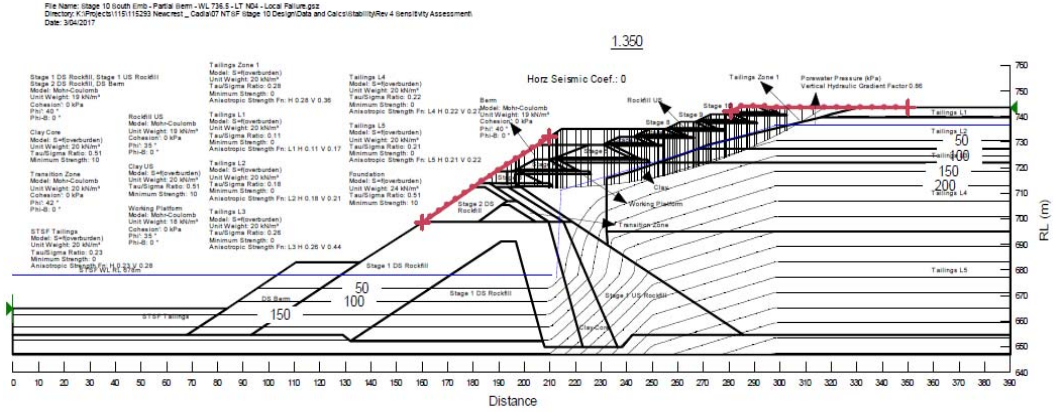


Figure B2-7: Stage 1 Butress Static FOS for Southern Embankment (2017-001).

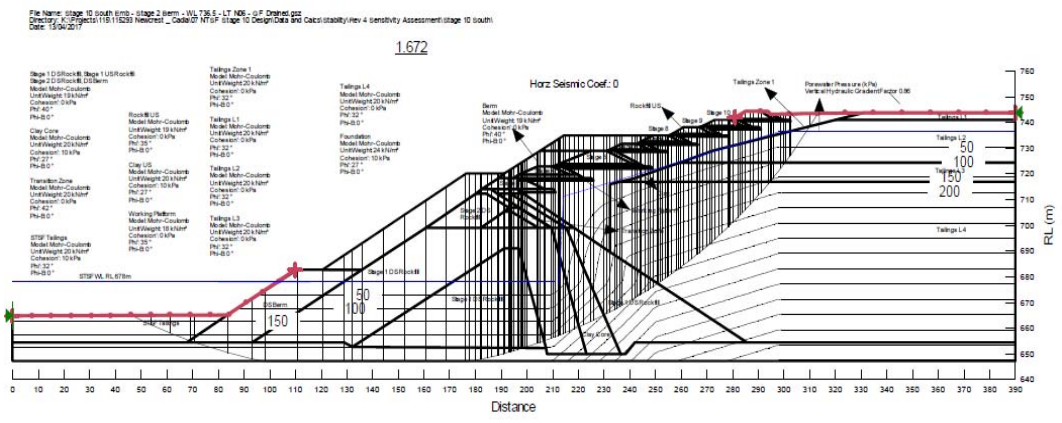


Figure B2-8: Stage 1 and 2 Butress Static FOS for Southern Embankment (2017-001).

B2.7.1 Stage 1 Butress Construction

The Stage 1 Butress extends from the Stage 3 crest (RL718.5) to the Stage 7 crest (RL735) and was designed to be constructed in three lifts using rockfill (Zone 3C) with a maximum particle size of 1.5m and a downstream batter slope of 1.5 H:1V (2017-001).

The three lifts were:

- Stage 3 to Stage 5 crest.
- Stage 5 to Stage 6 crest
- Stage 6 to Stage 7 crest.

Although there was no restriction on the rate of placement of Lift 1, a two-week resting period was required between Lift 1 and 2 and between Lift 2 and 3. Four standpipe piezometers (P8, P8A, P9 and P10), which were subsequently converted to VWP, were installed to monitor pore pressure increase in the tailings during the Stage 1 Butress construction.

Construction of the Stage 1 Butress commenced mid July 2017 and was in progress at the time of the NTSF failure. A tip head was initially developed from the northern and western sides of the NTSF, followed by an eastern tip head (from the eastern end of the NTSF Southern Embankment) from early December 2017.

Although records are available indicating the proposed volume of material to be placed in the Stage 1 Buttress each week, there is no construction information on the tip head location or lift progress. Notwithstanding this, high resolution geo-referenced satellite imagery on the following dates:

- 8 December 2017 (2018-001)
- 13 January 2018 (2018-002)
- 13 February 2018 (2018-003)
- 9 March 2018 (2018-004)

was used to provide an accurate location of the tip-head, while low resolution satellite imagery was used to provide an approximate location of the tip head over the whole construction period. Proposed placement rates were then cross-checked against the tip-head locations and buttress volumes and were found to be in reasonable agreement.

The advance of the three stages of the western tip-head between 1 December 2017 and 9 March 2018 are shown in Figure B2-9. Symbols show dates of high resolution satellite photographs.

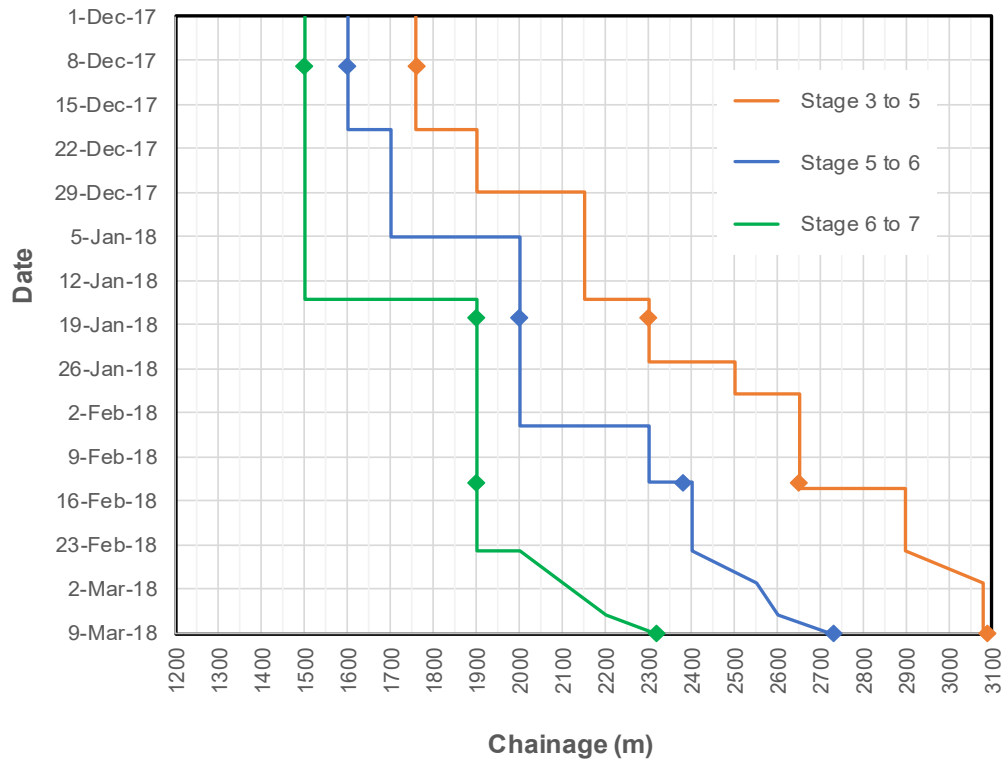


Figure B2-9: Approximate position of the western tip head of Stage 1 Buttress

Satellite imagery showing the Stage 1 Buttress construction on 18 January 2018 is presented as Photo B2-5.



Photo B2-5: Stage 1 Buttress construction on 18 January 2018 (2018-002)

Figure B2-9 indicates that the Stage 1 Buttress construction occurred through the NTSF failure zone (Chainage 1800 to 2200) around the following dates.

- Lift 1 9 December 2017 to 14 January 2018
- Lift 2 5 January 2018 to 3 February 2018
- Lift 3 18 January 2018 to 6 March 2018

At the time of the NTSF embankment failure, Lift 3 of the Southern Embankment Stage 1 Buttress had progressed to Chainage 2300 at which point it began to ramp down to Lift 2. The exception to this was an area around piezometer P9 (Chainage 2150) that remained at the Stage 5 crest level (RL729) and had not been backfilled awaiting VWP installation.

B2.7.2 Stage 2 Buttress Construction

The Stage 2 Buttress was designed as a 15m wide buttress extending from natural ground level at the toe of the NTSF at a slope of 1.5H:1V to RL721. Local widening of the Southern Embankment downstream berm was recommended between Chainage 1600m and 3100m to provide safe access for machinery.

The ATCW Stage 10 Design Report does not specify a sequence for construction of the buttresses but notes that “the sequencing of construction may be adjusted to suit site access and the construction equipment available for the work” (2017-001). It is understood (Peter Lord pers. comm.) that the decision to proceed with Stage 1, prior to Stage 2, was made in consultation with ATCW as the Stage 1 buttress provided the largest increase in the FOS.

Up to the time of NTSF embankment failure, the Stage 2 Buttress had been placed on an ad hoc basis along some of the Western Embankment and had commenced at Chainage 2100 on the Southern Embankment ramping up to a reasonably level platform at ~RL689 extending from Ch2300 to the east.

In April 2017 ATCW (2017-002) prepared a memorandum identifying the stripping required at the toe of the NTSF required for the Stage 2 Buttress placement. This advice was based on the excavation of nine (9) test pits which identified four areas requiring stripping, the locations of which are shown on Photo B2-6.

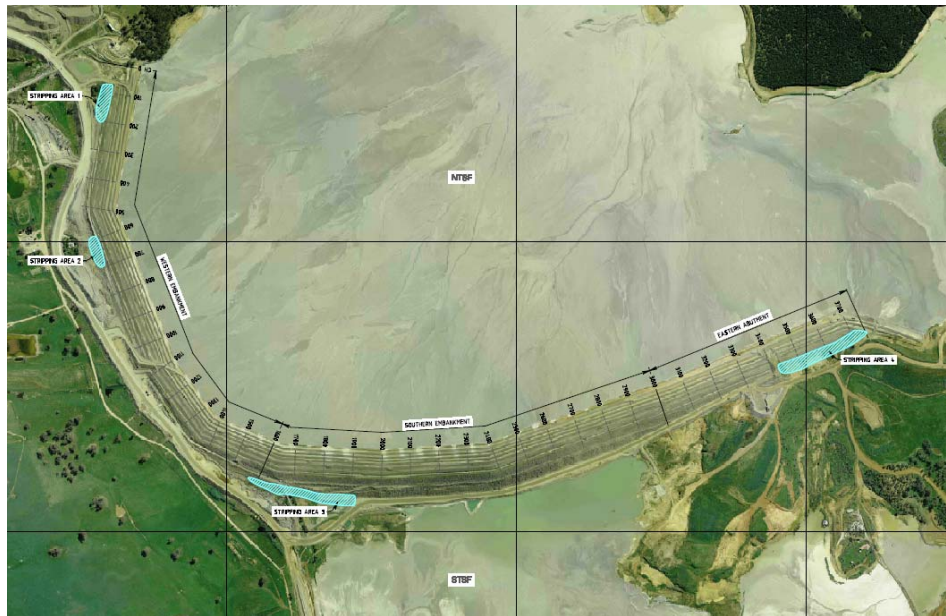


Photo B2-6: ATCW Stage 2 Buttress foundation stripping recommendations (2017-002)

Of particular interest to the NTSF Embankment Failure investigation is Stripping Area 3, where in excess of 4m of tailings had accumulated in a depression (previously noted in Section B2.5). The extent of the tailings can be more clearly seen in Photo B2-7 after the depression had been partially excavated to make more room for additional tailings overflow.



Photo B2-7: Aerial photo of tailings filled depression (circa 2012) (2012-001)

Although stripping at the western end of Area 3 commenced in late September 2017, removal of the tailings at the toe of the NTSF embankment (between Chainage 1930 and 2030) did not commence until 5th January 2018. Prior to excavation, the tailings surface in this area was at RL 681. By 13th January 2018, tailings removal was essentially complete and the underlying residual soil / extremely weathered volcanoclastics had been exposed. As the insitu materials continued to deteriorate on further excavation (described as becoming “wet and spongy”), a test pit was excavated on 16th January 2018 in an attempt to identify a suitable foundation.

Photo B2-8 shows the exposed foundation on 18th January 2018, with excavated test pit at the western end of the excavation and seepage in foreground.

Photo B2-9, taken looking to the east along the excavation, shows a 3 to 5m wide bench of tailings left against the downstream toe of the NTSF, with the underlying insitu material exposed for 1.0 to 1.5m below the tailings. Sample PL1 BS1 was taken from the insitu material at the base of the excavation.

Apart from further removal of tailings and residual soils to the west, and the accumulation of a small amount of seepage, the excavation remained in this condition, with an estimated base at RL676, until the time of failure on 9th March 2018 (Photo B2-10).



Photo B2-8: Exposed foundation on 18th January 2018 (2018-010)



Photo B2-9: Excavated face at toe of NTSF on 17th January 2018 (2018-011)

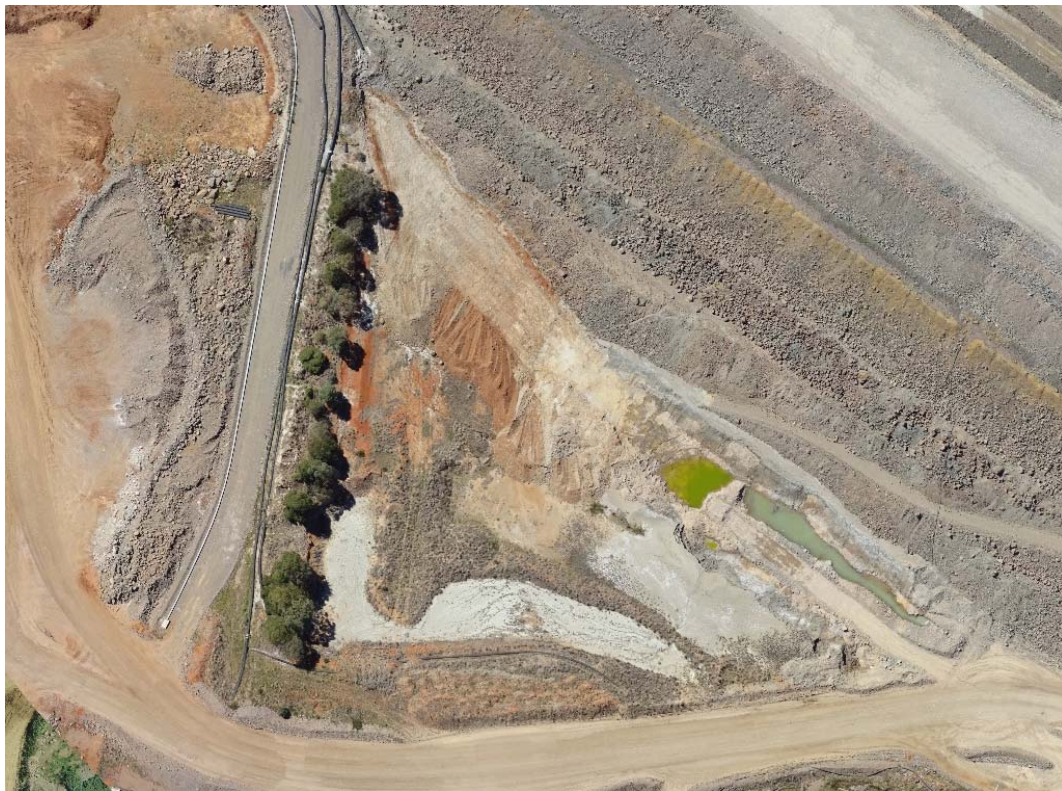


Photo B2-10: Toe excavation on 9th March 2018 (2018-004)

B3. Geotechnical Investigations

B3.1 Foundations & Construction Materials

B3.1.1 Pre-Construction Investigations

Geotechnical investigations undertaken prior to the construction of the NTSF are summarised in Table B3-1, while the locations are shown in Figure B10 (Annexure BA).

Table B3-1: Summary of pre-construction Investigations

Year	Consultant	Drillholes	Test Pits	Comments	Reference (Annexure BB)
1995	WC	9	64	Foundation & Storage	1995-001
1997	PSM	10	23	Foundation & Storage	1997-001
1997	PSM	-	31	Stage 1 Foundation	1997-002

Notes:

PSM Pells Sullivan & Meynink
WC Woodward Clyde

Although details of the site geology and the results of these investigations are provided in Appendix C, a brief summary of the main strata intersected are given in Table B3-2.

Table B3-2: Summary of drillhole information

Hole ID	Approximate Chainage	By	Total Depth (m)	Soil (m)	Basalt (m)	Paleosoil (m)	Volcaniclastics (m)	Sedimentary Strata (m)
BH109	240	PSM	10.0	3.0				7.0
BH22	645	WC	23.5	17.0			6.5	
BH100	740	PSM	20.4	9.0			11.4	
BH108	1015	PSM	20.4	5.7				14.7
BH21	1210	WC	15.5	1.0			14.5	
BH107	1470	PSM	20.0	5.2	0.9	1.8		12.0
BH106	1750	PSM	15.8	4.3	7.9	1.9	1.7	
BH20	1950	WC	15.0	1.0	14.0			
BH101	2090	PSM	20.6	3.3			17.3	
BH102	2330	PSM	14.8	2.6			12.3	
BH17	2480	WC	32.2	5.0			27.2	
BH103	2690	PSM	21.3	2.9			18.4	
BH19	3265	WC	14.6	1.0			13.6	
BH21	3490	WC	15.5	1.0			14.5	
BH104	3730	PSM	20.4	2.2			18.2	
BH105	240	PSM	20.6	9.3			11.3	
BH23	645	WC	7.0	1.0				6.0

Falling head permeability tests and lugeon permeability tests were completed in a number of drillholes.

Immediately prior to construction (June 1997) thirty-one (31) test pits were excavated along the dam axis to assess the depth of the cut-off trench. Logs are only provided for a number of test pits, however Table B3-3 provides a summary of test pit details in the vicinity of the slump. These test pit results are not completely consistent with previous investigations.

Table B3-3: Summary of core trench test pits in vicinity of slump

Chainage	Test Pit	Depth (m)	Rock Type Recorded
1700	TP229	1.9	
1750	TP228	2.4	
1800	TP227	2.0	Basalt
1850	TP226	>7.0	
1900	TP225	3.7	
1950	TP224	2.0	Basalt ?
2000	TP223	4.9	
2050	TP222	2.8	
2100	TP221	1.4	
2140	TP220	1.4	Basalt ??
2180	TP219	2.7	
2220	TP218	1.9	
2270	TP217	3.3	Andesite

At the pre-construction stage laboratory testing included the following:

- Atterberg & Linear Shrinkage;
- Particle Size Distributions;
- Emerson Class;
- Moisture density tests (Standard Compaction);
- Point Load Index (I_{s50}) tests;
- Four (4) multistage CIU triaxial tests on samples from test pits, re-compacted to 98% of Standard Maximum Dry Density (SMDD); and
- Four (4) constant head permeability tests.

B3.1.2 STSF Investigations

In 2000, Woodward Clyde undertook geotechnical investigations for the Lower Rodds Creek (LRC) Tailings Dam and the Upper Rodds Creek (URC) water dam (2000-003). Seven (7) drillholes and forty-one (41) test pits were completed for the LRC TSF, now known as the Southern TSF (STSF). Three (CIU) triaxial tests and ten permeability tests on samples from the STSF were used to supplement the NTSF design data.

Effective stress data from the NTSF, STSF and URC investigations is summarised in Figure B3-1. Lower bound Mohr Coulomb parameters are $c' = 10\text{kPa}$, $\phi' = 22.2^\circ$, while average strength parameters for the NTSF and STSF are $c' = 18\text{kPa}$, $\phi' = 26.7^\circ$.

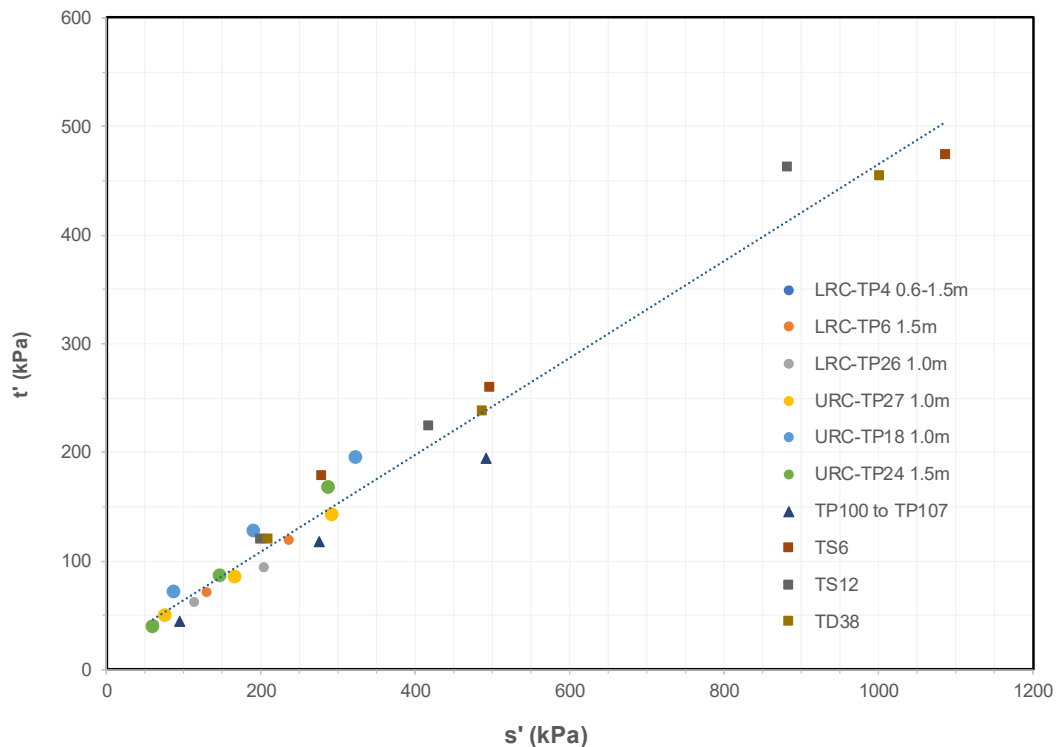


Figure B3-1: Multistage CIU triaxial test data from NTSF, STSF and URC investigations

B3.1.3 2017 Investigation

Between 28th and 29th March 2017, ATCW excavated nine test pits along the downstream toe of the NTSF to assess the foundation conditions and stripping requirements for the Stage 2 Buttress (2017-002). These test pits are discussed in Section B2.7.2, while the location of these test pits are shown in Figure B10 (Annexure BA). As these test pits were excavated to confirm stripping depths, no laboratory testing was undertaken.

B3.1.4 2018 Investigations

Between the 13th and 14th February 2018, ATCW drilled a series of auger holes at five (5) locations around the NTSF and STSF (2018-005). The locations of the holes are shown in Figure B10 (Annexure BA). The purpose of the investigations was to supplement the Stage 11 design of the NTSF and update the existing knowledge regarding the properties of the clay foundations.

In general, two holes were drilled using solid flight augers at each location. The first hole was used to log the materials encountered and probe the depth of the clay while the second was to recover 75mm diameter undisturbed samples of the clay. At two locations, additional holes were drilled either to recover additional samples or the first hole encountered refusal at shallow depth.

In addition to Atterberg Limit, particle size distribution (by hydrometer) and specific gravity tests, testing of undisturbed samples included the following:

- Three (3) oedometer consolidation tests.
- Four (4) undrained monotonic triaxial compression tests following anisotropic consolidation (CKU).

Details of this testing will be discussed in Appendix D.

B3.2 Tailings

B3.2.1 Overview

The investigation of NTSF tailings have been undertaken on four separate occasions as indicated in Table B3-4.

Table B3-4: Summary of Tailings Investigations

Year	Consultant	CPT	Vane Shear	Comments	Reference (Annexure BB)
2004	URS	3	4	NTSF Stage 3 Design	2004-001
2014	URS	6		NTSF Stage 8 Design	2014-002
2016	Golder	-		Thickener underflow	2016-001
2017	ATC Williams	11	15	NTSF Stage 10 Design	2017-001

B3.2.2 2004 Investigations

In August 2004, prior to the Stage 3 design, URS completed three (3) cone penetration tests (CPT's) to a maximum depth of 16 m. Access was provided via causeways constructed onto the tailings beach. Hard copies of the CPT plots are included in the Stage 3 Design Report (2004-001) together with four (4), hand vane shear strength profiles to 4.5m depth.

Although the Stage 3 Design Report provides a summary of tailings parameters, no laboratory test results are included in the report. Tailings parameters provided include:

- Description Silty fine sand with 10% clay
- Specific gravity 2.67
- Dry density 1.5 t/m³.
- Void ratio 0.78
- Permeability 10⁻⁷ and 5x10⁻⁹m/s (Rowe Cell consolidation; 2001)

B3.2.3 2013 Investigations

In February 2013, URS commissioned CPTS (2014-002) to undertake a further six (6) CPT with pore pressure measurement (CPTu). Although six CPTu were completed, it would appear that duplicate tests were completed at three locations (N1, N2 and N3). The duplicate test at each location was essentially for pore water pressure dissipation tests (PWPD). The locations of these tests are shown on Figure B11 (Annexure BA).

Plots of undrained shear strength ratio (S_u/σ'_v) versus depth, interpreted from the CPTu, are included in the Stage 8 Design Report, however, there is no basis provided for the interpretation.

The six (6) CPTu probes and ten (10) PWPD tests have been re-interpreted and the results are included in Appendix E.

B3.2.4 2016 Investigations

In 2016, Golder Associates was commissioned by CVO to conduct laboratory testing on a number of samples taken from the thickener underflow. The type and number of tests undertaken are summarized in Table B3-5.

Table B3-5: Scope of Golder's 2016 testing

Purpose	Test Type	Number
Index Tests	Atterberg Limits	2
	Particle Specific Gravity	2
	Particle Size Distribution	2
Settling	Undrained settling	2
Consolidation	Slurry Consolidometer	2
Strength	Consolidated Undrained Triaxial Compression (CAU)	2
	Monotonic Direct Simple Shear (DSS)	2

Golder Associates received three samples from CVO, numbered TH006, TH2003 and TH602. TH006 and TH2003, were combined in equal proportions to form a composite C1, while TH602 was mixed to form sample C2.

These tests are discussed in Appendix E, however Table B3-6 and

Table B3-7 provides a summary of the results.

Table B3-6: Summary of 2016 Index Testing

Test	Parameter	Unit	TH006	TH2003	C1	C2
PSD	< 75µm	%	66	64	65	82
	< 2µm	%	15	12	14	15
Atterberg Limits	LL	%			20	23
	PL	%			15	15
	PI	%			5	8
Specific Gravity Gs					2.69	2.92

Table B3-7: Summary of 2016 Shear Strength Testing

Parameter		Unit	C1		C2	
			DSS ⁽¹⁾	TX ⁽²⁾	DSS	TX
Vertical Effective Stress	σ_v'	kPa	500	416	500	504
Mean Effective Stress	p'	kPa		305		335
Geostatic Stress Ratio	K_o	-		0.60		0.50
Peak Undrained Strength	s_u	kPa	159	104	136	154
Undrained Strength Ratio	S_u/σ_v'	Peak	0.32	0.25	0.27	0.31
		CSL		0.23		0.21
Consolidated Void Ratio	e_c	-	0.55	0.53	0.64	0.65

Notes:

- (1) Direct Simple Shear Test
- (2) Triaxial Test

Hydraulic conductivities from the slurry consolidometer ranged from approximately 1×10^{-8} to 5×10^{-9} m/s across the range of stresses tested. These are consistent with those of a low plasticity silt.

Based on the shear strength testing, Golder made the following general observations regarding the behaviour of the C1 and C2 materials:

- If in a normally consolidated saturated state, both C1 and C2 are likely to exhibit contractive behavior on shear – i.e. undrained conditions will be the controlling static stability scenario.
- The contractive state of the material, combined with its index properties suggest that should cyclic liquefaction occur, or a static liquefaction trigger eventuate, significant post-liquefaction strength reduction is likely.

The range of peak undrained strengths obtained in the testing (0.25 to 0.32) appear to be generally consistent with the lower range of values inferred by URS (2015) from CPTu's in the NTSF.

B3.2.5 2017 Tailings Investigation

Prior to the construction of Stage 10, ATC Williams completed a tailings investigation program at ten locations numbered N01 to N10 in January and February 2017. Testing and sampling was subcontracted to IGS of Brisbane (2017-010).

Eight locations were situated on clay and rockfill 'fingers' offset approximately 30m from the Stage 9 Embankment Crest, while three locations (N08, N09 and N10) were situated on the Stage 5 berm. At the latter locations holes were pre-drilled through the Stage 5 and 4 embankments.

Investigation locations are shown on Figure B11 (Annexure BA), while a summary of the type and number of tests is provided in Table B3-8.

Table B3-8: Summary of 2017 tailings investigation

Investigation Type	Test Location (Prefixed by N)										
	01	02	02A	03	04	05	06	07	08	09	10
Stage 9 Fingers	✓	✓	✓	✓	✓	✓	✓	✓			
Stage 5 Berm									✓	✓	✓
CPTu Depth (m)	38.4	39.4	15.0	51.6	58.7	65.5	51.8	28.6	24.4	25.0	23.6
CPTu Dissipation Tests	2	2	1	3	4	4	3	2	2		1
Shear Wave Velocity Depth (m)	36	38		48	57	63	45	27			
Vane Shear Tests	5				4			6			
Undisturbed Samples (63mm)	1			2	2	2		4			
Piezometer Depth (m)	10	10		10	16	16	16	16			

The number and type of laboratory tests completed on both disturbed and undisturbed samples are indicated in Table B3-9.

Table B3-9: Scope of 2017 laboratory testing

Test Purpose	Test Type	Number
Index Testing	Soil Moisture Content	24
	Atterberg Limits	18
	Particle Size Distribution	18
	Particle Density	20
Dry Density	Field Dry Density	22
Strength Testing	Anisotropically Consolidated Undrained Triaxial (CAU)	5

Although test results will be discussed more fully in Appendix E, summaries are provided in Table B3-10 and Table B3-11.

Table B3-10: Summary of 2017 index testing

Test Purpose	Measurement	Value
Specific Gravity		2.72
Grading	Percent passing 75 μ m	60
Atterberg Limits	Liquid Limit (%)	23
	Plastic Limit (%)	16
	Plastic Index	7
Classification	Low plasticity Sandy Silt (ML) to Sandy Clay (CL)	

Table B3-11: Summary of 2017 shear strength testing

Parameter		Unit	N01	N05	N07 ¹	N07 ²	N07 ³
			4.2 m	10.4 m	20.3 m	20.3 m	20.3 m
Vertical Effective Stress	σ_v'	kPa	82	152	254	371	787
Lateral Effective Stress	σ_3'	kPa	30	59	109	208	332
Geostatic Stress Ratio	K_o	-	0.37	0.39	0.43	0.56	0.42
Consolidated Void Ratio	e_c	-	0.67	0.60	0.58	0.57	0.56
Consolidated Dry Density	γ_d	kN/m ³	16.6	17.0	17.4	17.6	17.8
Mean Effective Stress	p_r'	kPa	50	154	251	476	483
Deviator Stress	q_r'	kPa	84	261	375	772	783

Deviator and Cambridge Mean Effective stresses reported in Table B3-11 are values measured on termination of the test and are not at failure.

B4. Analyses

B4.1 Design Parameters

Limit equilibrium stability analyses were completed for all stages of construction. Design parameters adopted for the analyses are summarized in Table B4-1, while a brief overview of the analyses completed is provided in the following sections.

Table B4-1: Summary of NTSF Design Parameters

Zone	Parameter	Stage 1	Stage 2	Stage 3 - 5	Stage 6 - 8	Stage 9	Stage 10
Foundation	γ_b	24	24	24	24	24	24
	c'	5	5	5	25	25	10
	Φ'	24	24	24	26.5	27	27
	Undrained					22+0.7 σ	0.51 σ
Clay Core Zone A Zone 1	γ_b		19.4	19.4	20	20	20
	c'	5	5	5	10	10	10
	Φ'	22	22	22	26	26	27
	Undrained						10+0.51 σ
Filter / Transition Zone C Zone 2	γ_b		20	20	20	20	20
	c'		0	0	0	0	0
	Φ'		42	42	42	42	42
Rockfill Zone B Zone 3A, 3B	γ_b	19	19	20	20	20	20
	c'	0	0	0	0	0	0
	Φ'	40	40	40	40	40	40
Rockfill Zone B1 Zone 3D	γ_b			19	19	19	19
	c'			0	0	0	0
	Φ'			30	30	30	35
Working Platform	γ_b			18	18	18	18
	c'			0	0	0	0
	Φ'			35	35	35	35
Stage 1 & 2 Buttress Rockfill	γ_b			19	19	19	19
	c'			0	0	0	0
	Φ'			40	40	40	40
Insitu Tailings	γ_b		16		16	16	20
	c'		0		0	0	0
	Φ'		30		30	30	32
	Undrained				0.25 σ	(1)	(2)
Liquified Tailings	c'	0	0				-
	Φ'	5	0				-
	Undrained	-		0.16 σ	0.16 σ		0.05 σ

Notes

- (1) Refer Table B4-2
 (2) Refer Table B4-3.

Table B4-2: URS Undrained strength ratio profile based on 2013 CPT (2014-002)

Probe depth range (m)	RL range	Shear strength ratio S_u/σ_v'	Inferred stage of deposition
1 - 9	731 – 722	0.50	Stages 5, 6
9 - 14	722 - 717	0.25	Stages 4
14 - 24	717 - 707	0.30	Stages 3, 2B
24 - 32	707 - 699	0.40	Stages 2A
>32	< 699	0.25	Stage 1

Table B4-3: ATCW tailings strength parameters (2017-001)

Western Embankment					Southern Embankment			
Layer	Top (m)	Base (m)	S_u/σ_v'		Top (m)	Base (m)	S_u/σ_v'	
			H	V			H	V
1	739.9	738.0	0.39	1.59	737.0	734.5	0.14	0.28
2	738.0	737.7	0.08	0.13	734.5	724.5	0.26	0.34
3	737.7	727.7	0.26	0.29	724.5	717.0	0.19	0.24
4	727.7	720.6	0.28	0.35	717.0	691.2	0.20	0.26
5	720.6	688.5	0.25	0.29				

B4.2 Results of Analyses

B4.2.1 Overview

General comments relating to the analyses completed for Stages 1 to 9 are listed below;

- Only the stability of the maximum embankment section was considered.
- Limit equilibrium stability analyses were completed using Slope/W in conjunction with circular failure surfaces. Bishop's and Spencer's methods of analysis were used although this was not always noted. Non circular failure surfaces were not considered.
- Seepage analyses were not undertaken. The phreatic surface in the upstream tailings was assumed based on drainage at the Stage 3 upstream toe.
- The Stage 3 Design Report indicated an upstream stability FOS greater than 2.0 at end of construction. Based on this analysis and considerable experience of previous centerline and upstream raises of the NTSF, end of construction stability of the upstream face was assumed for Stages 3-9.
- A peak ground acceleration of between 0.08 and 0.075g was adopted for design
- A deep seated failure surface through the foundations was not considered as a failure mode for Stages 8 or 9.
- A tailings strength profile based on the 2013 CPTu data was adopted for the Stage 8 and 9 designs.

B4.2.2 Stage 1

Analyses undertaken for Stage 1 (1997-001) indicated the following:

- End of Construction FOS > 1.1 for pore pressure ratio in core < 0.5.
- Long Term Stability FOS \approx 1.4 for deep seated failure.
- Earthquake loading FOS < 1.1 for peak ground acceleration = 0.34g

Deemed satisfactory as the design acceleration for design earthquake (1 in 500 AEP, 0.08g) is much less than 0.34g.

- Post seismic FOS > 1.5

Analyses for the final arrangement with a downstream slope of 1V:3H were

- Long Term Stability FOS > 1.5.
- Earthquake loading FOS < 1.1 for peak ground acceleration = 0.2g
- Post seismic FOS > 1.1

B4.2.3 Stage 2

Analyses undertaken for Stage 2 (2000-001) indicated the following:

- End of Construction FOS = 1.1 to 1.2 depending on water pressure in core.
- Long Term Stability FOS = 1.4 to 1.5 depending on phreatic surface.
- Earthquake loading Yield acceleration calculated as 0.2g.

Displacements not significant and failure extremely unlikely as design earthquake is much less than 0.2g.

- Post seismic FOS > 1.37

B4.2.4 Stages 3 to 6

A site specific seismic assessment undertaken as part of the Stage 3 design (2004-001) indicated a 1 in 500 AEP peak ground acceleration (pga) = 0.075g. A dynamic site response was undertaken using SHAKE96 and time histories from the seismic assessment. Analysis indicated that liquefaction for Stages 3 to 6 would not occur under the design earthquake.

Stability analyses undertaken at the time of the Stage 3 design indicated FOS = 1.33 to 1.46 for the Stage 6 design depending on tailwater conditions. These marginal FOS highlighted the potential need for a downstream weighting berm, which was subsequently constructed by the time of the Stage 6 design.

Analyses undertaken for Stage 3 to 4 indicated the following:

- End of Construction FOS > 2.0
- Long Term Stability FOS = 1.57

Analyses undertaken for Stage 3 to 6 indicated the following:

- Long Term Stability FOS = 1.65 – with berm and STSF tailings
FOS = 1.61 – with berm and STSF decant only
FOS = 1.44 – upstream raises & undrained tailings
- Earthquake loading FOS = 1.23 (USACE Screening Method)

B4.2.5 Stage 7

The updated requirement of Guidelines on Tailings Dams (ANCOLD, 2012) were considered as part of the Stage 7 design. For previous stages, a peak ground acceleration of 0.075g had been adopted and this was considered to be consistent with the MDE requirements of ANCOLD for a Significant Hazard TSF (1 : 1000 AEP).

Analyses undertaken for Stage 7 design (2013-006) indicated the following:

- Long Term Stability FOS = 1.65 – global with effective stress parameters
FOS = 1.52 – global with undrained tailings
FOS = 1.44 – upstream raises & undrained tailings
- Earthquake loading FOS = 1.23 (USACE Screening Method)

B4.2.6 Stage 8

For the Stage 8 design (2014-002) liquefaction analysis (CPeT-IT & CLiq V1.7) using 2013 CPT data indicated FOS against liquefaction generally greater than unity with a low potential for liquefaction.

Deformation analysis using Swaisgood, Pells and Fell and Makdisi & Seed ranged up to 0.2m. This crest displacement was considered negligible as the elevation difference between the crest and decant pond was typically 6m.

Analyses undertaken for Stage 8 indicated the following:

- Long Term Stability FOS = 1.56 – global with undrained tailings
FOS = 1.63 – upstream raises & undrained tailings
- Earthquake loading FOS = 1.20 (USACE Screening Method)

B4.2.7 Stage 9

Analyses undertaken for Stage 9 (2015-002) indicated the following:

- Long Term Stability FOS = 1.65 – global with undrained tailings
FOS = 1.54– upstream raises & undrained tailings
- Earthquake loading FOS = 1.08 (USACE Screening Method)

B4.2.8 Stage 10

General comments on the Stage 10 stability analyses (2017-001) completed by ATCW are:

- Maximum height sections were analysed, for both the Southern and Western Embankment.
- Only circular failure surfaces were considered; the method of limit equilibrium analysis is not reported.
- Seepage analyses were not undertaken and phreatic surfaces for the analysis of each section were based on piezometer data.
- An anisotropic tailings strength profile was adopted based on the 2017 CPTu investigations. Twentieth percentile values were used for horizontal tailings strength and fifty percentile values were adopted for the vertical strength profile.
- A peak ground acceleration, PGA =0.08g, was adopted for the 1 in 500 year AEP Maximum Design Earthquake (MDE).

- Stability analyses were completed for the Stage 1 Buttness only (Partial Section) and the Stage 1 and Stage 2 Buttness (Full Section) geometries.

Minimum FOS for the various analyses are presented below for the **Partial Section**:

- End of Construction FOS = 1.33 – through upstream raises
 FOS = 1.38 – through foundations

Minimum FOS for the various analyses are presented below for the **Full Section**:

- End of Construction FOS = 1.52 – through upstream raises
 FOS = 1.47 – through foundations
- Long Term Stability FOS = 2.57 – through upstream raises
 FOS = 1.67 – through foundations
- Earthquake loading FOS = 1.23 – through upstream raises
 FOS = 1.29 – through foundations
- Post Seismic Stability FOS = 1.20 – through upstream raises
 FOS = 1.46 – through foundations

B5. Inspections and Surveillance Monitoring

B5.1 Overview

Since the initial construction, inspections of the NTSF have been undertaken on a regular basis ranging from per shift, weekly, monthly, annual and five yearly comprehensive inspections. Table B5-1 provides a list of annual inspections.

Table B5-1: Annual inspections

Report	Level	By	Date	Reference (Annexure BB)
Surveillance Report 2000	Comprehensive	WC	Feb, 2000	2000-004
Surveillance Report 2006	Comprehensive	URS	Mar, 2007	2007-001
Surveillance Report 2007-2008	Intermediate	URS	Dec, 2008	2008-002
Surveillance Report 2009	Intermediate	URS	Jan, 2010	2010-002
Surveillance Report 2010-2012	Intermediate	URS	Feb, 2013	2013-003
Surveillance Report 2013	Intermediate	URS	Mar, 2014	2014-003
Surveillance Report 2014	Intermediate	URS	Mar, 2015	2015-003
Interim Surveillance Report 2016	Intermediate	AECOM	May, 2016	2016-002
Surveillance Report 2016	Intermediate	ATCW	Oct, 2016	2016-003
Surveillance Report 2017	Comprehensive	ATCW	Nov, 2017	2017-003

The most recent comprehensive inspection was completed by ATC Williams in 2017. This is a detailed report and has been drawn upon for information contained in the following sections, except where pertinent observations have been made in other reports.

Surveillance monitoring of the NTSF has included:

- Piezometers;
- Seepage; and
- Crest displacement beacons.

The monitoring data is discussed in the following sections.

B5.2 Displacement

B5.2.1 Terrestrial Monitoring

B5.2.1.1 Data

Initially reference pins were concreted into the crest of each raise to monitor both horizontal and vertical movement. However, as the pins were surveyed using GPS, it was found that the data was unreliable and an alternative method was implemented.

From October 2013, survey prisms grouted into large boulders on the dam crest were used for monitoring deformation. The location of the prisms was measured using a total station situated at one of four base stations (referred to as pillars). Prisms located on the Southern Embankment were measured from Pillars 9, 50 and 51, while those on the Western Embankment were measured from Pillar 56.

Prisms were located on the crest of Stages 4, 5 and 7, however, so as to avoid disturbance to the prism locations, they were not placed in position until the following embankment stage had been constructed. In all ten (10) prisms were placed on the Western Embankment and thirty-two (32) on the Southern Embankment. The location of monitoring points is shown in Annexure BD whilst survey data used in the analysis is provided in Appendix E (2018-006).

Between January 2017 and November 2017, prisms were progressively removed to facilitate Stage 1 Buttress construction.

The relative position of prisms along the Southern Embankment, together with the staging of installation and horizontal displacement vectors for the period October 2013 to November 2017 are shown in Figure B5-1. The vertical displacement and original ground elevation for these prisms are shown in Figure B5-2.

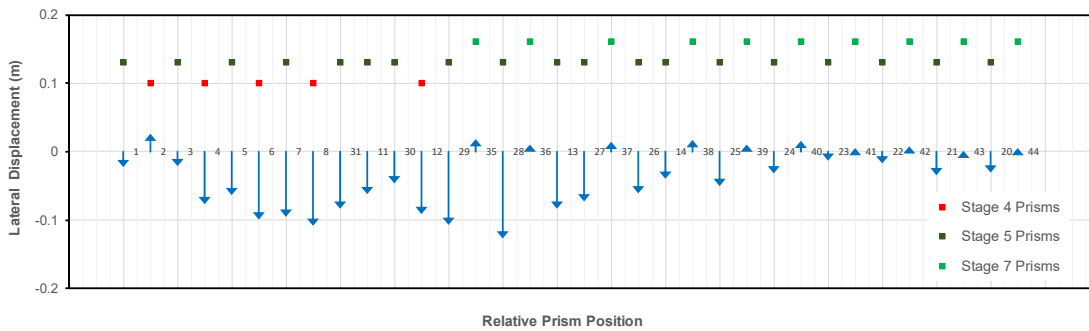


Figure B5-1: NTSF Southern Embankment Prisms – Staging and displacement vectors

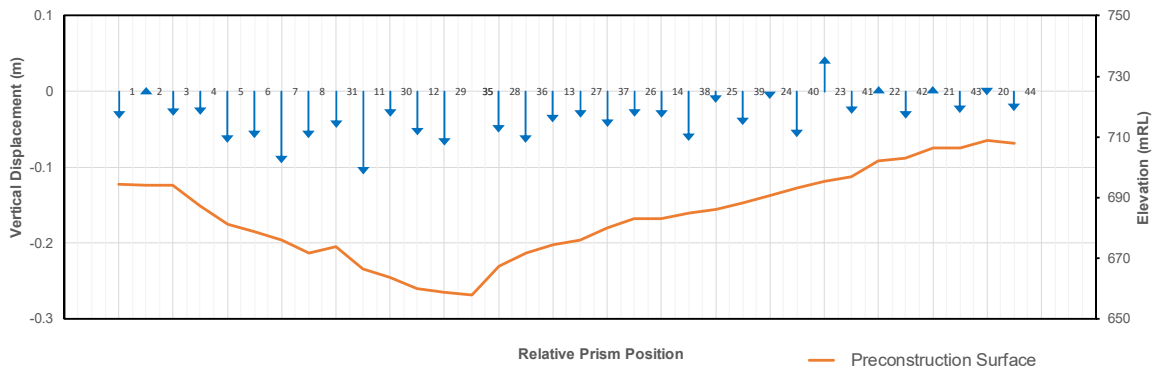


Figure B5-2: NTSF Southern Embankment – Ground elevation and vertical displacement

It would be expected that displacement measurements normalized for embankment height would provide reasonably consistent values for at least vertical displacement. Normalization of results with respect to embankment height was based on a crest elevation of RL729, with height divided by 100.

Horizontal displacement vectors normalized for embankment height are shown in Figure B5-3 while vertical displacement vectors normalized for embankment height are shown on Figure B5-4.

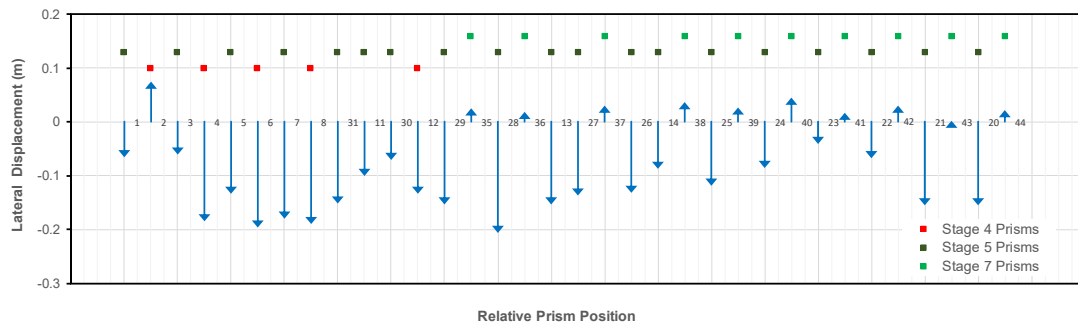


Figure B5-3: Southern Embankment – Displacement vectors normalized for NTSF height

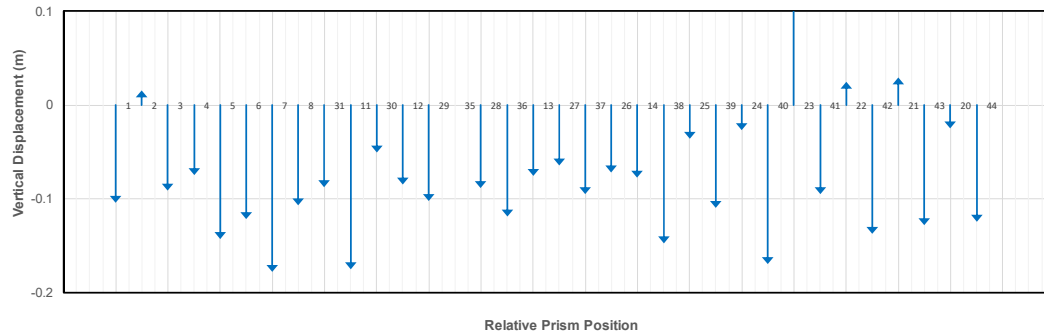


Figure B5-4: Southern Embankment – Vertical displacement normalized for NTSF height

Embankment cracking was observed in the NTSF crest between prisms #4 and #7, while the maximum embankment height is in the vicinity of prisms #35 and #28.

The following observations can be made regarding the displacement monitoring.

- Vertical Movement
- South Wall movement is generally less than 100mm with the largest measurement 109mm.
- South Wall measurements from Pillar 9 (prisms 1 to 8 and 13) appear consistent while measurements from Pillars 50 and 51 are less consistent.
- South Wall measurements normalized for embankment height are more consistent. Notwithstanding this, measurement in the vicinity of the slump are above the average.
- West Wall measurements (Annexure BD) range between +19mm and -28mm and are erratic.
- Lateral Movement
- Except for Stage 7 prisms (which show small and erratic movement), movement is generally down valley with a maximum movement of the order of 100mm.
- Prism movement on the eastern side of the South Wall is generally small and correlates reasonably well with embankment height.
- Prism movement on the western side of the South Wall does not correlate well with embankment height. Without normalization, down valley movement at prisms #4 to #8 is similar to movement at the maximum embankment section,
- The effect of normalization on down valley movement is less pronounced than the effect on vertical movement.
- Prism movement on the West Wall (Annexure BD) is generally very small.

B5.2.1.2 Conclusions

Terrestrial survey of prisms located on the Stage 4, 5 and 7 crests indicate a down-valley movement that is not inconsistent with an earth and rockfill embankment of the height of the NTSF including upstream raises. However, down-valley movement in the vicinity of the slump (normalized for embankment height), is larger than the remainder of the NTSF and comparable with the maximum embankment section, suggesting a potential underlying weakness in the embankment or foundation.

B5.2.2 Satellite Monitoring

B5.2.2.1 Background

Following the NTSF embankment failure, Newcrest engaged Otus Intelligence Group Pty Ltd (Otus) to derive surface movement measurements (SMM) in the vicinity of the NTSF using historical satellite Synthetic Aperture Radar Interferometry (InSAR) data.

As the primary contractor, Otus engaged Airbus DS Geo Australia P/L to derive the SMM using data from the Sentinel-1 European satellite. Although data from this satellite may not be the most accurate going forward, it was found to have the longest historical time series.

Initially thirty-five InSAR scenes at 12 day intervals acquired between 13th January 2017 and 25th February 2018 were processed using the Small Baseline (SBAS) method. The SBAS method makes use of small spatial baselines (spatial orbital separation) and small temporal baselines (time between data acquisitions) to provide a high sampling rate while maintaining spatially dense deformation mapping. Sentinel-1 maintains an orbital tube width of ± 50 m and a nominal revisit time interval of 12 days, satisfying the SBAS criteria.

The SBAS algorithm used to analyse the Cadia Area of Interest (AOI) data uses the distributed scatterer approach rather than the persistent scatterers interferometry (PSI). The SBAS algorithm is more appropriate where higher non-linear deformation (e.g. strong acceleration) is anticipated.

Subsequently Otus were engaged to extend this acquisition period until 5th September 2018 to investigate potential ongoing deformation of the NTSF.

An output from the analyses was web based access to the pixel data (deformation and velocity) and pixel data in ArcGIS format which enabled data to be extracted at specific locations and manipulated. The initial InSAR data (up to 25th February 2018) is provided in Appendix K (2018-031) while the subsequent data (up to 5th September 2018) is provided in Appendix K (2018-007).

The following is an overview on the Otus report, included as Annexure BE, and provided based on the initial InSAR analysis.

B5.2.2.2 Technical Constraints

The limitations of the InSAR method and the Sentinel-1 data are briefly listed below:

- Whereas the movements from terrestrial survey can be resolved into three components (x,y,z) the surface movements measurements (SMM) associated with the InSAR data from a single satellite can only be resolved in a single direction in the satellite line-of-sight (Annexure BE).
- Movement (between any two scenes) cannot be resolved if it is greater than the wavelength of radar (3.5 to 5 cm). When large movement occurs, there is a 'lack of coherence' and no data is provided for the particular pixel.
- The theoretical precision of Sentinel -1 velocity measurements is approximately 1-2 mm per year. The precision of individual time series points can be given as approximately 3-5 mm.

- Sentinel-1 data is C-band medium resolution and not optimal for high resolution monitoring. TerraSAR-X high-resolution data is more suitable for high resolution measurements but was not available for the period of interest.
- Although the inherent resolution of the InSAR data used is 5m x 20m, these are not discrete cells but overlap each other resulting in over-sampling. The result is that the value for a single 20m x 20m pixel is the composite of a number of radar returns from surrounding cells. Therefore, a coherent response for surface movement may be provided for a single pixel, even if there is a loss of coherence on some returns from a number of cells within the pixel.
- Random errors may result from strong atmospheric disturbances, changes in soil moisture and/or air temperature. These may be eliminated by selecting a local reference with known stability within the AOI.

B5.2.2.3 Comparison with Terrestrial Monitoring

InSAR SMM data was compared with terrestrial survey of prisms located on the NTSF. Graphs for Prism 6 (located within the slump) and Prism 31 (located outside the slump) are presented as Figure B5-5 and Figure B5-6 respectively. In both cases, the SMM time series prior to 8th January 2018 indicates a similar rate of movement to the terrestrial survey. However, after 8th January 2018, the rate of movement in the vicinity of the slump (Prism 6) increases while outside the slump (Prism 31) the rate of movement remains relatively constant.

As the SMM data cannot be resolved into the three components of movement and can only be compared with terrestrial measurements in terms of trend, vertical scales on these figures are different so as to demonstrate the similarity of trend.

B5.2.2.4 Deformation at slump

InSAR surface movement time series taken at the crest, mid height and toe in the vicinity of the slump (Ch2000) are presented in Figure B5-7. Data at the crest shows an increased rate of movement from 8th January 2018. A small increase in the rate of movement was observed at mid height, while none was apparent at the toe. The exception to this is data from 25th February 2018, which presents as a down kick on most, if not all data across the Cadia area. This regional down kick was attributed to abnormal atmospheric conditions on the particular day and was subsequently removed by reprocessing the data based on reference point location nearby.

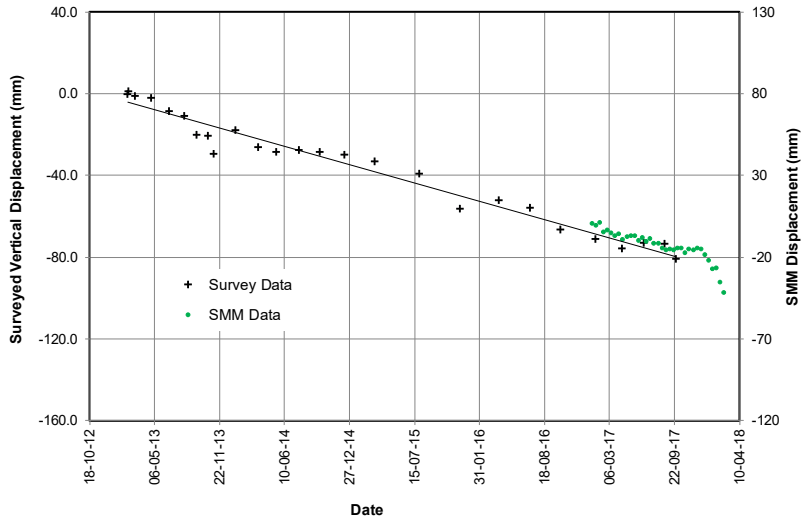


Figure B5-5: Prism 6 – Comparison of survey and SMM data

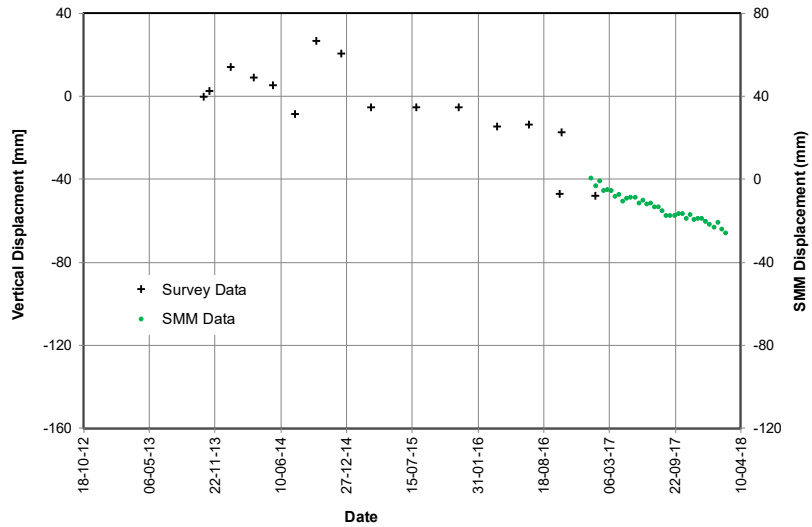


Figure B5-6: Prism 31 – Comparison of survey and SMM data

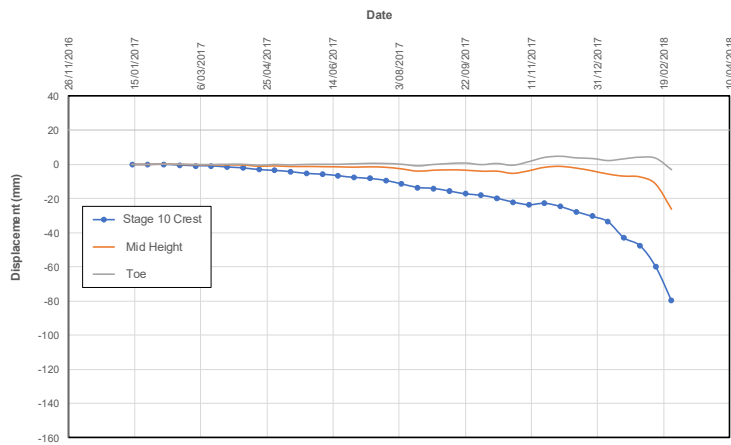


Figure B5-7: InSAR surface movement measurements at Ch 2000

B5.2.2.5 Long Term InSAR Trends

An average velocity map of the NTSF embankment, showing movement velocity in millimeters per year, was an output from the data processing by Airbus DS. An average velocity map provided for the period 13th January 2017 and 25th February 2018 is shown as Figure B5-8 while one for the period 13th January 2017 and 5th September 2018 is shown as Figure B5-9. Red areas on these maps show a relatively faster rate of movement.

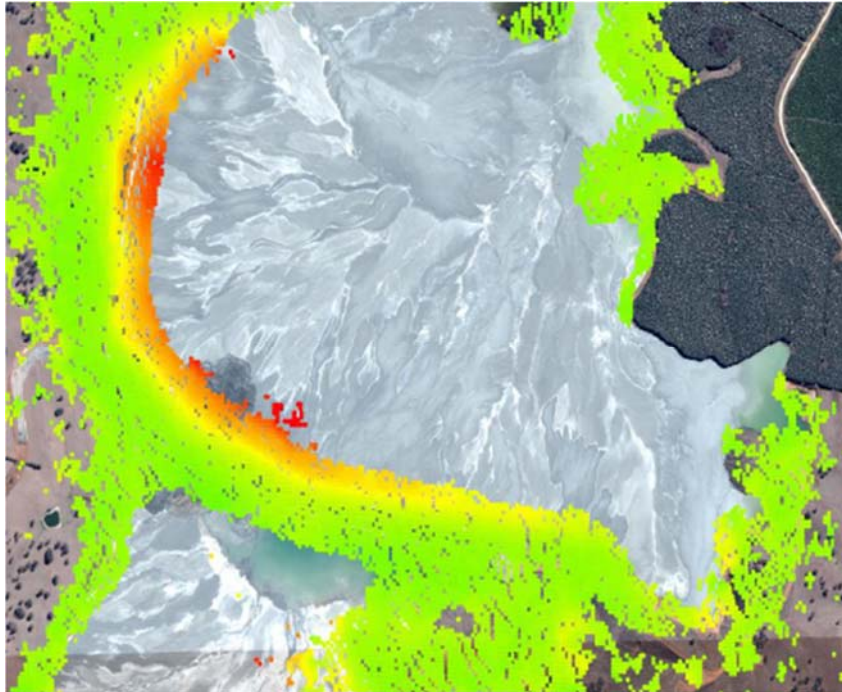


Figure B5-8: Airbus DS average velocity map for NTSF (01/17 to 02/18) (2018-032)

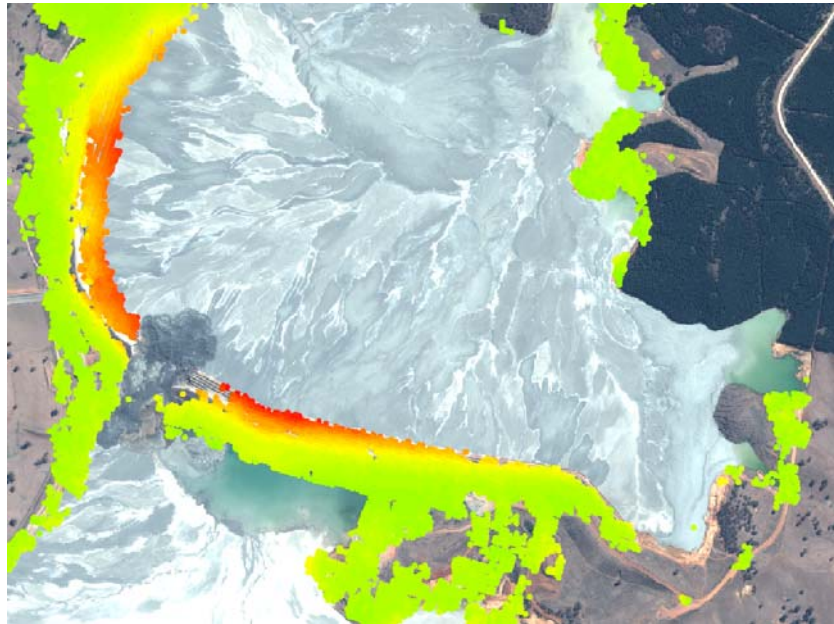
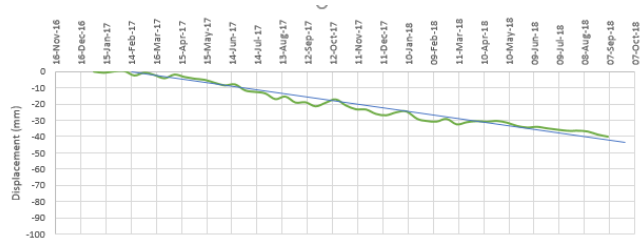


Figure B5-9: Airbus DS average velocity map for NTSF (01/17 to 09/18) (2018-033)

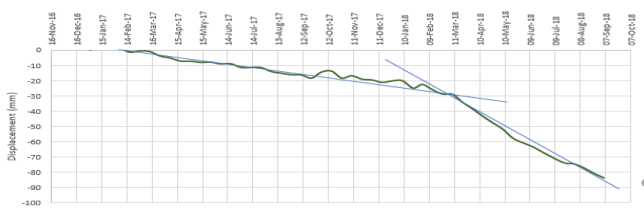
A comparison of Figure B5-8 and Figure B5-9 indicates that the increased rate of movement (red area) was not confined to the slump but continued after the event and moved to the east.

The reason for this phenomenon was investigated further by interrogating the time series of pixels at regular (100m) intervals around the Stage 8 crest. It was found that the time series fitted three trends, a uniform or one segment trend, a two segment or a three segment trend as shown on Figure B5-10. In addition, it was found that slope (rate of displacement) of the first segment of all three series and third segment of the three segment time series were all similar.

One segment time series



Two segment time series



Three segment time series

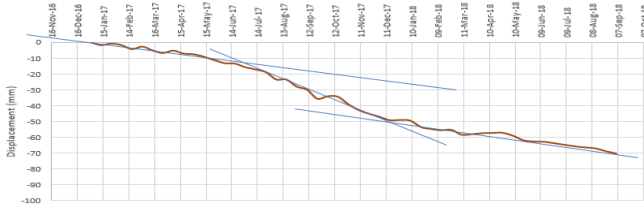


Figure B5-10: Typical InSAR displacement time series trends for the NTSF.

B5.2.2.6 Conclusions

Figure B5-11 shows a close relationship between the Buttress 1 construction and an increased rate of displacement measured using the InSAR time series. In addition, where the depth of tailings below the Stage 5 embankment (Stage 4 embankment crest) is less than ~15m the increase in the rate of displacement is either negligible or of limited duration.

Although there is a close relationship between an increase in the rate of displacement measured by InSAR and the Buttress 1 construction together with the depth of tailings beneath Buttress 1, the rate of displacement measured in the immediate vicinity of the slump after the Buttress 1 construction is larger than that recorded elsewhere on the NTSF, including where the tailings are significantly deeper.

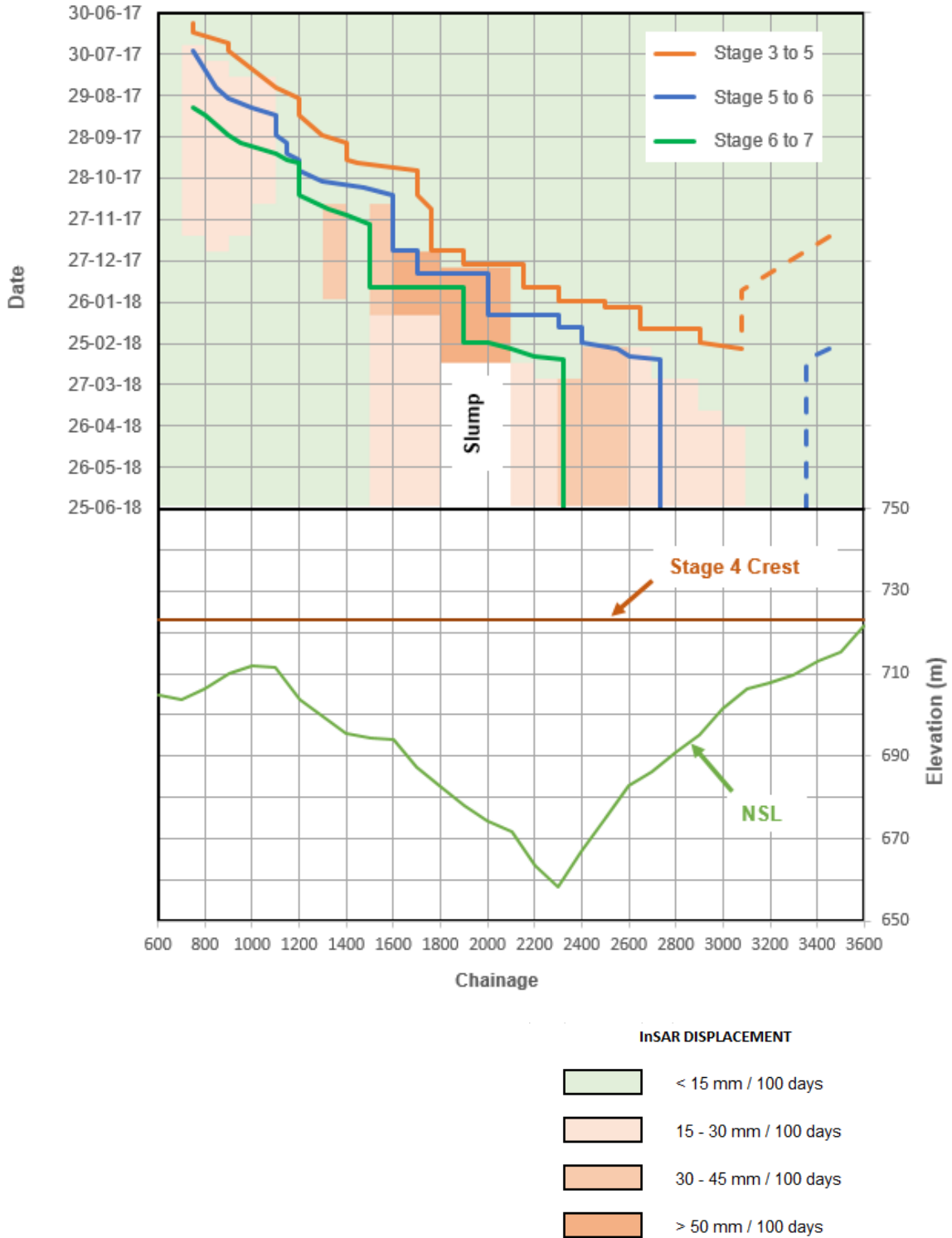


Figure B5-11: InSAR rate of displacement related to Buttress 1 construction progress

B5.3 Piezometers

B5.3.1 Stage 1

Twenty-three (23) pneumatic piezometers were installed in the Stage 1 embankment, between Ch2200 and Ch2700. The piezometers were mostly located at the foundation interface across the upstream rockfill, core and transition zones and within the core at RL665 and RL680. Based on the piezometer data, the 2000 surveillance report concluded that the dam was functioning satisfactorily.

B5.3.2 Stage 2

Following the Stage 2B/1 construction, pneumatic piezometers were installed in the tailings beach to provide information on the beach development and tailings consolidation properties. Piezometer locations are shown in Table B5-2 and in Annexure BD. At the time of the 2017 report, shaded piezometers were considered to be operational.

Table B5-2: Stage 2B piezometers

Chainage	Offset	Mine Grid Location		Tailings Elevation (mAHD)	Designation	Tip Elevation (mAHD)
		Easting	Northing			
1500	10	16257.15	17235.96	705.28	P1500/1	705.2
2200	5	16606.78	17216.54	705.59	P2200/1	705.5
	30	16603.59	17242.02	705.46	P2200/2	705.4
	30	16603.59	17242.02	705.46	P2200/3	702.4
	60	16598.20	17271.43	705.24	P2200/4	705.2
	100	16589.04	17310.12	704.96	P2200/17	704.9
2500	5	16906.13	17235.02	704.82	P2500/1	704.7
	30	16904.96	17260.96	704.71	P2500/2	704.6
	30	16904.96	17260.96	704.71	P2500/3	701.6
	60	16903.22	17291.74	704.50	P2500/4	704.0
	100	16901.55	17332.68	704.38	P2500/5	704.0
2750	10	17142.92	17319.01	705.10	P2750/1	705.0
3000	10	17380.37	17398.43	705.33	P3000/1	705.0

Initially the piezometer results were inconclusive, however at the time of the Stage 4 design, the piezometric surface was 6 to 8m below the tailings surface and 2 to 4m below the decant pond level, reducing to 10m below the tailings surface at the time of the Stage 5 design. The Stage 2 pneumatic piezometers have had a checkered life, being extended and refurbished on a number of occasions. Data from the pneumatic piezometers is considered to be generally unreliable.

B5.3.3 Stage 6

As part of the 2013 CPTu investigations, VWP's were installed at Chainages 700, 1600 and 2500 with piezometer tips between RL695 and 707mAHD respectively. The three piezometers were designated NTSF WP1, WP2 and WP3 and installation details are provided in Table B5-3. Locations are shown in Annexure BD.

Table B5-3: Details of 2013 VWP

Designation	Mine Grid Location		Tailings Elevation (mAHD)	Chainage	Tip Elevation (mAHD)	Water Level 30/6/17
	Easting	Northing				
NTSF WP1	15637.2	18039.4	731.77	700	706.77	725.99
NTSF WP2	16115.8	17326.2	731.67	1600	696.67	722.15
NTSF WP3	16903.7	17289.2	730.67	2500	694.66	711.49

B5.3.4 Stage 9

As part of the 2017 geotechnical investigations, a further seven VWP's were installed into the tailings surface. Installation details are provided in Table B5-4 while historical VWP data is provided in Annexure BD.

Table B5-4: Details of 2017 VWP

Designation	Mine Grid Location		Tailings Elevation (mAHD)	Chainage	Tip Elevation (mAHD)	Water Level 30/6/17
	Easting	Northing				
VWP-N01	15671.9	18044.8	740.68	800	730.68	734.50
VWP-N02	15872.4	17572.4	741.16	1200	731.16	735.14
VWP-N03	16112.3	17361.6	740.86	1650	730.86	735.19
VWP -N04	16531.0	17306.7	740.39	2150	724.39	731.28
VWP-N05	16894.5	17326.9	740.08	2500	724.08	727.80
VWP-N06	17424.2	17506.8	739.02	3000	723.02	730.83
VWP -N07	18077.8	17767.1	738.96	3800	722.96	733.09

B5.3.5 Stage 10

Four standpipe piezometers were installed through the Stage 5 embankment crest to monitor pore water pressure in the tailings during the construction of the Stage 1 Buttress. The standpipes numbered P8, P8A, P9 and P10 were installed with VWP.

B5.3.6 Discussion

Pore pressure dissipation tests (PWPD) completed as part of both the 2013 and 2017 CPTu investigations also provide important information regarding the piezometric conditions in the NTSF.

Equilibrium pore pressures from the PWPD tests, plotted against depth on Figure B5-12, indicate a pressure gradient below hydrostatic at a number of test locations. The pressure gradient is closest to hydrostatic along the Western Embankment (N02, N03, N1, N2) and well below hydrostatic in N05 and N3 (Chainage 2500 Southern Embankment). With the exception of N05 and N3 where the inferred water level is ~8m, other tests generally indicate water levels between 3 and 4m below the tailings surface.

A pressure gradient below hydrostatic can also be observed where VWP, installed at a similar chainages but different depths, indicate a piezometric head difference less than the installed level difference. However, the differences indicated in Table B5-5 are much greater than that which could be anticipated from the PWP data (Figure B5-12).

Table B5-5: Comparison of 2017 and 2013 VWP data

2017 Piezometer	2013 Piezometer	Installed Level Difference (m)	Piezometric Head Difference (m)
VWP-N01	NTSF WP1	23.9	8.5
VWP-N02	NTSF WP2	34.2	13.0
VWP-N03	NTSF WP3	29.4	16.3

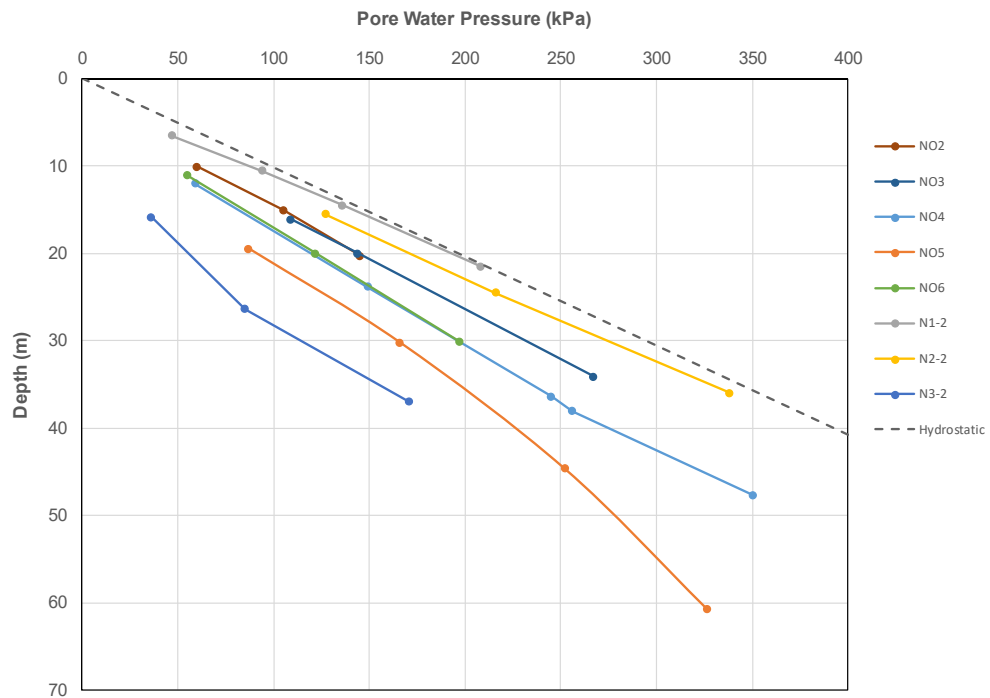


Figure B5-12: CPTu pore water pressure gradients

The deeper groundwater surface in the vicinity of Ch2500 can also be seen in a longitudinal profile of the piezometric surface (Figure B5-13). This and the pressure gradient less than hydrostatic can most likely be attributed to downward drainage towards the Stage 1 underdrainage system, installed between Ch2300 and 2600 to assist in the consolidation of the tailings.

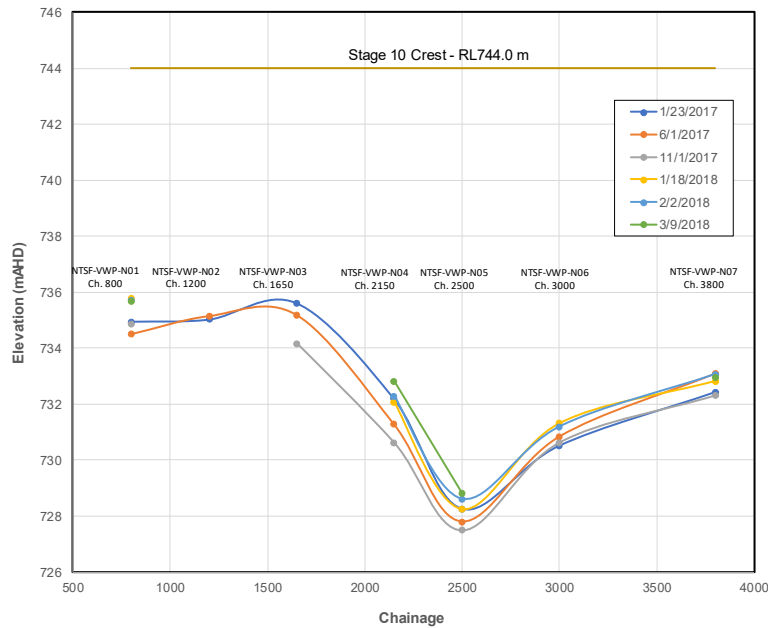


Figure B5-13: Longitudinal profile of piezometric surface with time

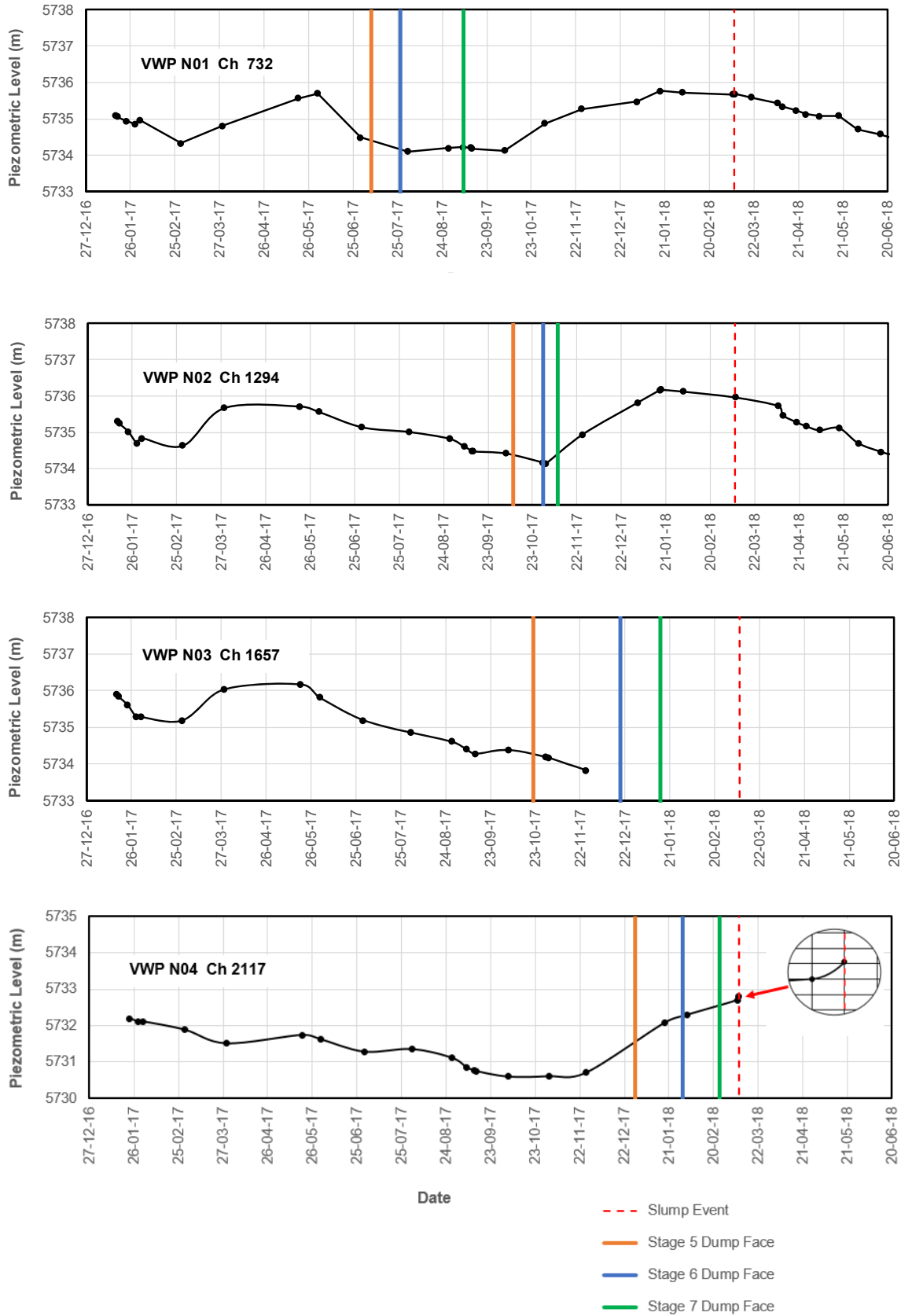
Key points noted in the NTSF Comprehensive Surveillance Report (ATCW, 2017) regarding the piezometer data are:

- The large variation in interpreted VWP piezometric levels (RL711 to 736 mAHD) is due to their spatial location and depth of installation.
- Levels have generally increased in accordance with the increasing pond level but at a reduced rate.
- Both vibrating wire piezometers and pneumatic piezometers show a similar trend in level, but not a consistent variation.
- Piezometers on the western side of the NTSF indicate piezometric levels generally higher than the southern embankment.
- Deeper piezometers (and pore water pressure dissipation tests undertaken as part of the 2017 CPTu program) indicate a reduction in pore pressure trend which has been interpreted to be the result of downward drainage, with a pressure head that is less than hydrostatic

The piezometer data file is provided in (2018-021).

Piezometer measurements for VWP installed in January 2017 (VWP N01 to N07) are provided in Figure B5-14. Also shown on the piezometer records is the progress of the Stage 5, 6 and 7 of the Buttress 1 construction (refer to Figure B5-11) and the timing of the slump event. Although the NTSF Stage 10 raise was constructed during 2017, piezometers surprisingly show a static or declining trend until the commencement of the Buttress 1 construction when a noticeable increase in level was observed around the time of or shortly after the commencement of the construction. The exception to this is VWP N03 and N06 which ceased to operate prior to the buttress construction and N07 which was located beyond the buttress construction.

Also shown on Figure B5-14 is a small (0.11m) rise in piezometer VWP-N04 on the day of the slump.



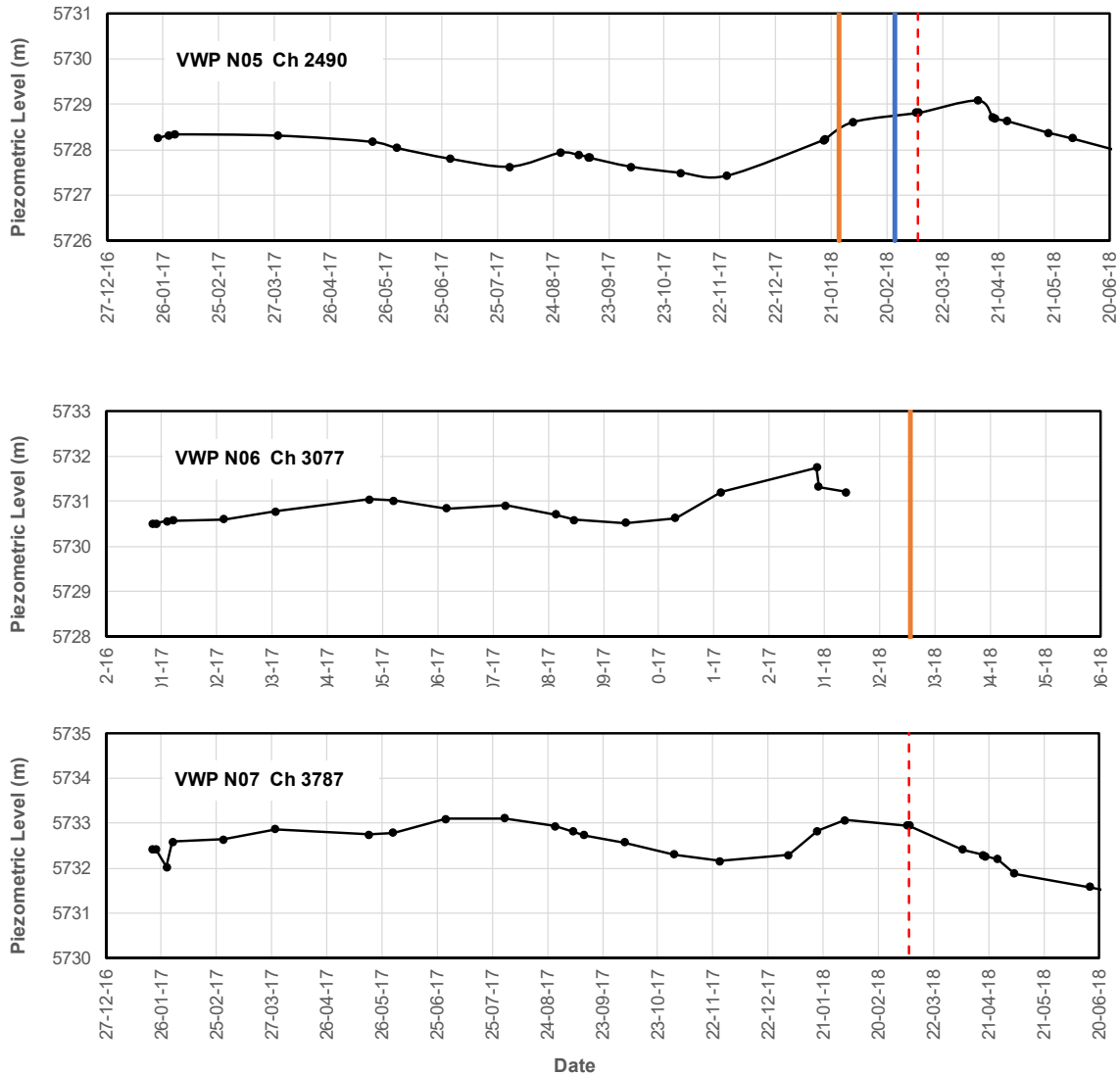


Figure B5-14: VWP piezometer measurements showing stages of Buttress 1 construction

B5.4 Seepage

As the STSF decant pond and tailings had begun to encroach on the toe of the NTSF since 2006, there has been limited opportunity to observe and monitor seepage from beneath the Southern Embankment of the NTSF. However, the 2000 Comprehensive Surveillance Report (2000-004) noted minor seepage emerging from the right abutment rockfill toe at Chainage 2200 (RL670) and downstream of the toe on the left abutment at Chainage 2850 (RL670-680). In both cases this seepage reported to sediment dams that can be seen in Photo B2-1.

A drainage system was installed at the upstream toe of the Stage 3 embankment with outlets to the downstream face at 200m intervals between Chainage 1800 and 3600. Underdrains were also installed at five locations below the Stage 5 Western Embankment and at four locations below the Stage 8 Western Embankment. The drain outlet locations are shown in Annexure BD.

The Stage 3 drains have remained dry, except for the western drain (Chainage 1800). Although seepage from this drain was noted for some time, it was not until a pipe (to toe of the NTSF) was attached to the drain, that accurate measurements of drain flow were made. Drain flow measurements between early 2015 and February 2018 (excluding a couple of outliers) are plotted on Figure B5-15 and range between 30 and 50 litres/minute. A trend line through this data

indicates that the flow possibly commenced in mid-2006 and prior to the NTSF Embankment failure a 10 l/min increase in the flow was observed for each 5m rise in the decant pond level. The seepage data file is provided in Appendix K (2018-030).

At the time of the June 2017 Surveillance Inspection (2017-003), ATCW noted a small flow from the Stage 5 Drain 4 (Chainage 1000) on the Western Embankment.

Semi-permanent wet spots have been noted on a number of berms (URS, 2014 & ATCW, 2016) on both the Southern and Western Embankments. The locations of these wet spots are shown in Annexure BD (2017-003).

As most of the wet spots appear to dry out during dry weather, it has been concluded that they most likely result from rainfall runoff and infiltration into the rockfill collecting at low points.

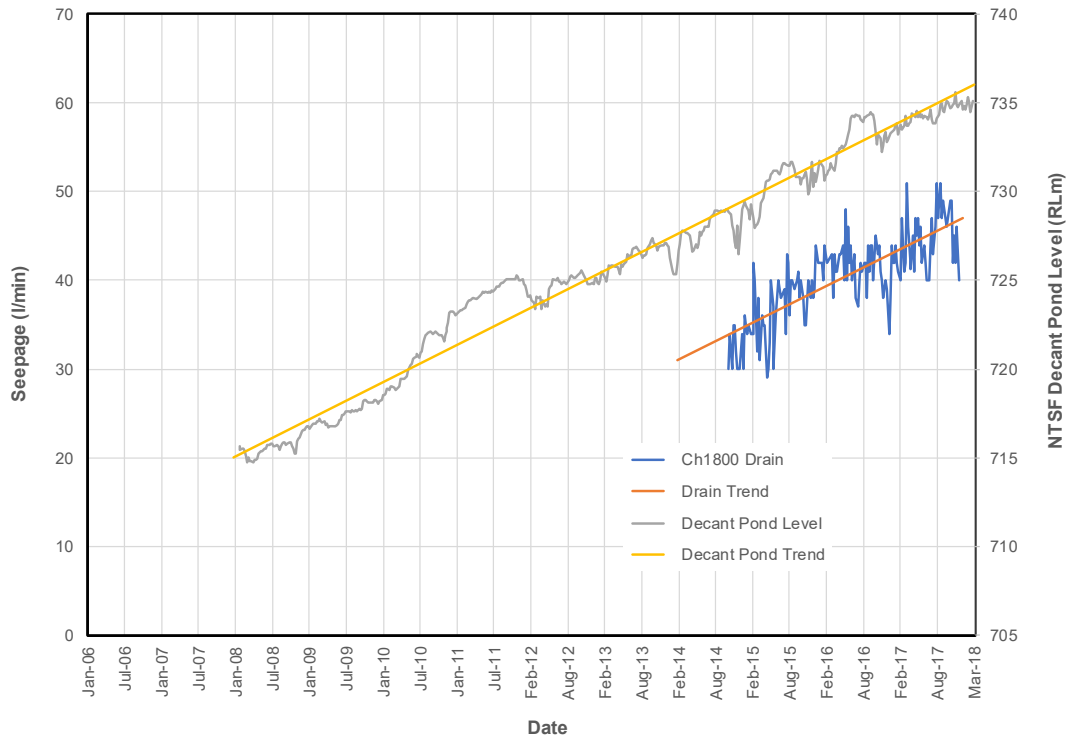


Figure B5-15: Ch1800 drain flow and NTSF decant pond level

Foundation stripping for the Stage 2 Buttress in late 2017 / early 2018 exposed seepage in the vicinity of Chainage 1700 and 2000 (2018-004). Where the seepage was not affected by tailings, it was clear. Green grass to the south-west of the Chainage 1700 haul road indicated that seepage in this area had been ongoing for some time.

B6. Audits & Third-Party Reviews

B6.1 Overview

Audits and third party reviews were undertaken for the NTSF, STSF and various water dams across CVO. Audits and reviews of particular relevance to the NTSF are listed below and summarized in the following sections.

- 2012 KCB
- 2016 GHD and Amberley Management
- 2016 Golder Associates
- 2017 Golder Associates
- 2017 KCB
- 2018 CHPL ITRB

B6.2 KCB 2012

An audit of the NTSF and STSF was undertaken by Len Murray of KCB in August 2012. The review covered governance, containment and monitoring, compliance and technical issues. Technical issues raised in the audit were;

- Consequence category to be confirmed by dam break analysis;
- Additional CPT data required to assess potential for cyclic and static liquefaction;
- Additional monitoring required;
- Geochemistry and physical properties of tailings need to be better understood; and
- Additional testing of foundation clays required.

Due to Newcrest re-structuring shortly after the draft review report (2012-002) was prepared, the KCB review was not finalized. However, it is pertinent that Len Murray notes the following:

“Some of the early site investigation reports note the presence of a highly plastic clay layer in the foundation with liquid limits above 50%. It is not clear what the extent of this layer is. However, the upcoming design review should include the assessment of this layer and also if possible obtain a sample for residual friction angle testing. Clays with liquid limits above 50% can have very low residual drained strengths.”

B6.3 GHD 2016

An audit of the two water dams and the two TSF was undertaken by GHD and Amberley Management in May 2016 (2016-004). High priority recommendations of the review are:

- Carry out dam-break modelling, review ANCOLD Consequence Category and design parameters against ANCOLD requirements;
- Peer review designs;
- Establish “Comprehensive” inspections; and
- Install additional piezometers and set trigger levels.

Medium priority recommendations are listed below:

- Determine tailings density using aerial survey and check filling rates;
- Review earthquake loadings and seismic design;
- Review phreatic line and incorporate into stability analyses;

- Review design parameters and undertake further testing if required;
- Observe and test seepage on berms;
- Review pipe corridors and mark spigots with identifying numbers;
- Record density tests in a continuous log and calibrate nuclear densitometer;
- Continue dust trials;
- Daily inspection sheet to be completed;
- Seepage points to be clearly identified;
- Determine relationships between seepages, rainfall, spigot operation etc.;
- Improve format of monthly reports;
- Review inspection reports and determine trends;
- Improve emergency response plan and undertake training; and
- Develop closure plans and determine impact on future designs.

B6.4 Golder 2016

Golder Associates reviewed the stability analyses previously undertaken by URS without undertaking separate analysis. Comments on the analyses included:

- The need for a more robust assessment of tailings strength;
- Stability analyses should consider non-circular and block failure;
- The undrained strength of foundation materials need to be confirmed; and
- The need to review the site specific seismic hazard assessment and adopt a PGA (with amplification) consistent with an updated Consequence Category.

Golder recommended a program of investigation, testing and analysis (2016-004).

B6.5 Golder 2017

In March 2017 Golder commented on stability analyses undertaken by ATCW. Although Golder acknowledged the need for buttressing, they disagreed with ATCW, in the following areas:

- Position of phreatic surface; and
- Consequence Category.

Golder stressed the need for further analysis regarding the timing of the buttress construction and the stability of final embankment configuration (2017-007).

B6.6 KCB 2017

Len Murray of KCB was commissioned to validate the tailings profiles and strength parameters adopted by ATCW and review slope stability analyses and identify potential data gaps. Mr Murray undertook a site visit in September 2017 and prepared a site visit report dated 14 September 2017 (2017-005).

Following an initial response to the Site Visit report, ATCW provided a comprehensive response to the comments in a memorandum dated 11 October 2017 Ref 115293.07-M009 Rev0 (2017-004). KCB were subsequently engaged to provide a peer review of the Stage 10 Design Report and documentation to ensure that it complied with current best practice.

Excluding comments relating to field testing procedures, KCB key recommendations of the report dated 6 November 2017 are listed below:

- Review sensitivity of design to Zone 1 tailings extent;
- Set up and calibrate a seepage model;
- Check sensitivity of design to different cone calibration factor;
- Seismic design should use current parameters;
- More detailed analysis of cyclic liquefaction required;
- Analysis of static liquefaction urgently required;
- Review and revise design criteria in context of current ANCOLD;
- Analyse worst case construction sequencing;
- Review strain compatibility of materials and non-circular failure mode;
- Analyse additional critical sections and check for consistency with ultimate design and closure requirements;
- Once liquefaction analysis is complete, review assumptions and undertake a 2D numerical deformation analysis; and
- Check location and condition of foundation clays as dam construction may have loaded foundation clays into a normally consolidated state.

KCB's major concerns related to the static liquefaction assessment (not completed) and seismic liquefaction assessment (inadequate). Further analyses were recommended to assess the following:

- Impact of stiff clay in foundations;
- Ultimate dam stability;
- Stability under flood conditions; and
- Strain compatibility.

B6.7 CHPL ITRB 2018

Following the KCB 2017 review, CHPL convened an Independent Technical Review Board comprising Dr Bruce Brown, Dr Andy Fourie and Mr Len Murray. A presentation on the Stage 11 design was made to the ITRB by ATCW on 6 December 2017 via a teleconference. At the time of the NTSF embankment failure, the CHPL ITRB were scheduled to make a site visit.

The CHPL ITRB provided initial comments on 2 January 2018 (2018-008). The initial ITRB comments are summarized below:

- Review tailings critical state parameters;
- Assume tailings are contractive unless proven otherwise;
- Complete Cyclic DSS testing of tailings to develop a liquefaction triggering curve;
- Update seismic hazard assessment to include return periods up to 1 : 10,000;
- Review stability in light of above;
- Check stress path for previous and existing loading conditions;
- Review undrained stress ratios of foundation clays; and
- Confirm intermediate stages for the buttress construction are stable.

B7. Tailings Management

The most recent NTSF Operations and Maintenance Manual (31933-018 Rev 6) was prepared by URS in July 2014 for the Stage 7 Raise (2014-004). The O&M Manual provides high level advice on operating constraints, operation of the underdrainage system, spillway, piezometers and crest displacement survey points as well as inspection monitoring and maintenance requirements.

Constraints placed on the operation of the NTSF include:

- The embankment, above the Stage 2 crest (RL714), is not designed to store water.
- The decant pool should be kept as small as practicable, commensurate with achieving an acceptable quality of decant.
- Tailings deposition should be carried out to minimise variations in tailing beach level where it intersects the retention embankment. This will assist to maximise the temporary flood storage capacity available without the decant pool extending out to the retention embankment.
- The dam safety adviser is to be notified should the decant pool approach within 250 m of the retention embankment at any point.

Tailings deposition is sub aerial, using multiple spigots from a header pipeline (630mm) running along the embankment crest, with the tailings beach falling to a decant pool at the upstream end of the storage (Figure B7-1). A rail mounted pumping system that can be moved up as the tailings and decant water level rises is used to decant the supernatant.

The emergency spillway for the NTSF is an unlined earth channel cut through the left abutment. Storage is designed to handle probable maximum precipitation with spillway discharge.



Figure B7-1: NTSF showing spillway and decant pond

Tailings deposition is managed through two stages:

- **Deposition planning** which co-ordinates mill waste to be deposited, filling the TSF and construction of the upstream embankments raises.
- **Deposition management** which is undertaken on a day to day basis. Using the deposition plan for guidance it also matches operational and dust suppression requirements.

Deposition planning is achieved by splitting the dam wall into zones, each containing five to seven discharge spigots (Figure B7-2). The Enhanced Production Scheduler (EPS) program allows the sequencing of mill waste production, tailings discharge and embankment construction through a set of rules. These rules include linking areas that are finished being constructed to waste tonnes for filling and rules that prevent wall construction from taking place on tailings that has not been consolidating for at least 60 days. EPS provides a rolling monthly schedule of construction and deposition zones that includes target tonnes for embankment construction as well as zones to be targeted for deposition.

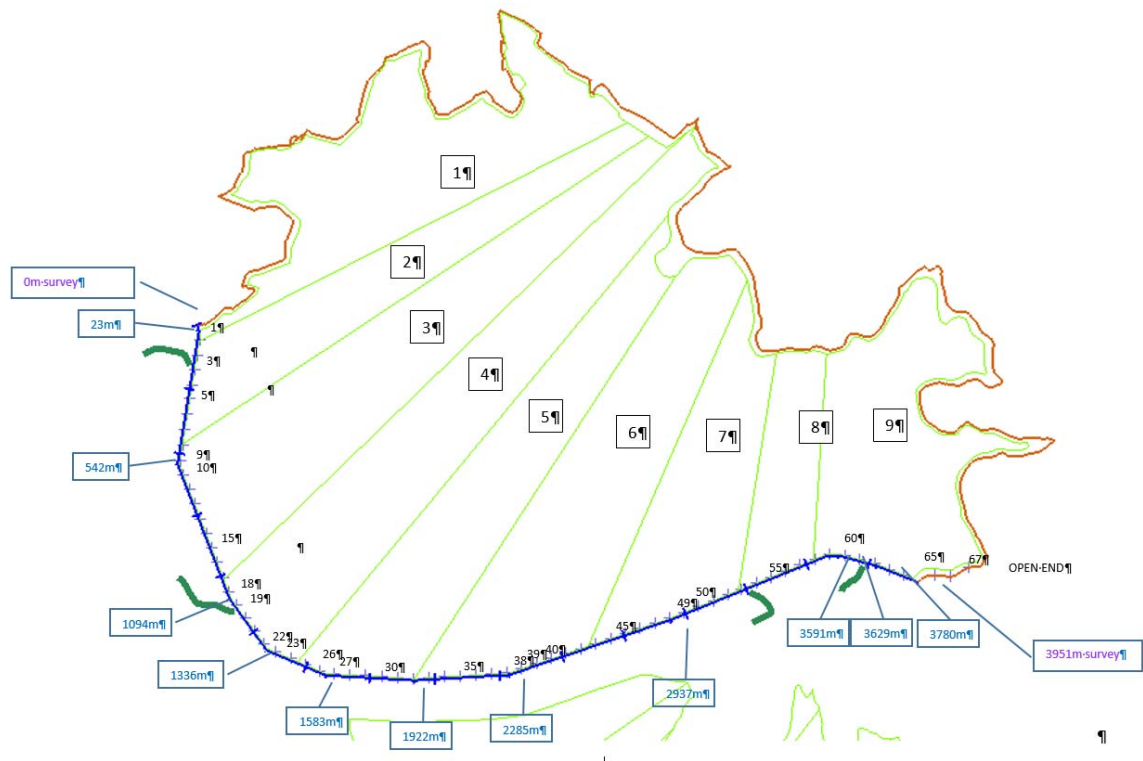


Figure B7-2: Stage 10 spigot arrangement (May 2017)

General constraints placed on the tailings discharge operation are listed below:

- Three (3) to six (6) spigots (depending on throttling) open at any one time;
- Spigots are opened using a sweeping routine to minimize concentrated flow of tailings;
- Deposition limited to a maximum of 200mm of tailings at any one time;
- Deposition within 500mm of crest closely monitored, with a limit of 300mm;
- Dust sweeping is undertaken to ensure the tailings surface is maintained in a moist condition to suppress dust generation, particularly between October and February.

The NTSF spigot opening sequence between July 2017 and 9 March 2018 is shown as Figure B7-3, highlights the sweeping nature of the tailings discharge. In Figure B7-3 dots indicate open spigots. The NTSF embankment failure is located between Spigots 28 and 32.

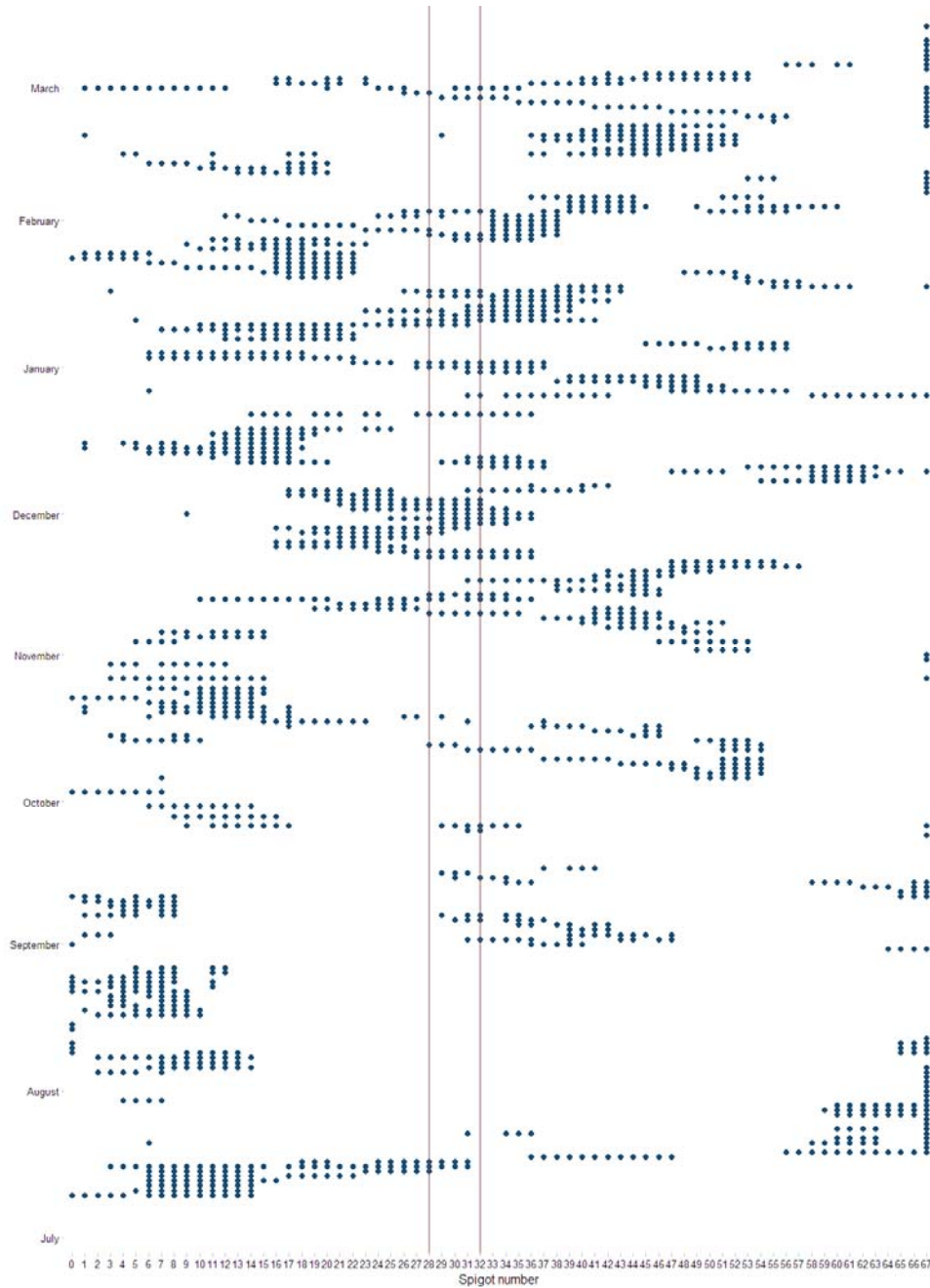


Figure B7-3: NTSF spigot opening sequence between July 2017 and 9 March 2018

Two concentrators are in operation at CVO; the second concentrator being commissioned in 2003. Waste from Concentrator 1 reports to thickeners TH006 and TH2003, while waste form Concentrator 2 (a finer grind) reports to thickener TH602. Historically, thickeners TH006 and TH2003 (Con 1) have been discharged to the NTSF, while thickener TH602 (Con 2) is only discharged to the NTSF when the STSF is being bypassed.

Since 2008, the level of the tailings beach adjacent to the NTSF embankment has been surveyed. Initially this was at six monthly intervals but has reduced to bi-monthly and monthly in recent years. The surveyed level of the tailings beach at any one time can vary between 1m and 3m over the length of the embankment. However, to provide an indication of the NTSF fill rate, the surveyed level of the tailings beach, in the vicinity of Chainage 1950, together with the east bay decant pond level are plotted on Figure B7-4.

The tailings modelling software package, Muck 3D, was used to model the TSF fill rate prior to 2008 using the NTSF raise schedule (with a maximum beach elevation of 0.5 m below the dam crest), an assumed settled dry density of 1.5t/m³ and a uniform tailings profile along the embankment, 0.5m below crest level. Figure B7-4 shows the following:

- Good agreement between the modelled and surveyed levels;
- A tailings rate of rise of ~1.9m /year prior to 2010;
- A tailings rate of rise of ~2.4m/year after 2010; and
- An average beach slope of ~ 0.3%.

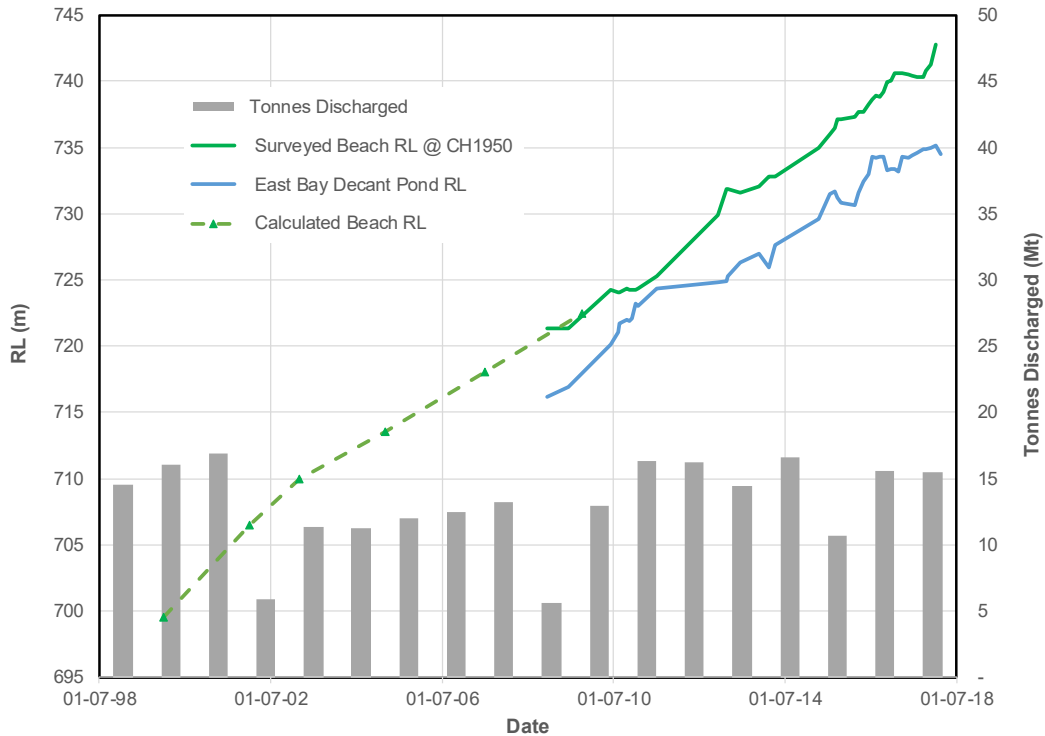


Figure B7-4: Tailings beach and decant pond levels

The increased rate of rise after 2010 corresponds to an increase in the tailings discharged to the NTSF from 11.54 Mtpa to 14.83Mtpa.

B8. NTSF Embankment Failure

B8.1 Pre- Failure Condition

B8.1.1 Overview

Prior to the embankment failure, the Stage 1 Buttress had been constructed from the west to Chainage 2300. From the design height at Chainage 2300, the buttress ramped down to the Stage 5 crest level over approximately 600m. The exception to this was a small area between Chainage 2050 and 2220, which was benched down to the Stage 6 and Stage 5 crest levels where a VWP was to be installed.

The Weekly Plan commencing 8 March 2018 (2018-034) indicates that the Stage 1 buttress construction changed to the eastern dumping face, advancing towards the west and a VWP was to be installed on 9 March 2018.

Cracking was first noticed on the morning of 9th March 2018 and inspections were made during the day by personnel listed in Table B8-1. The following section provides a chronology of events assembled from statements made by those noted in Table B8-1. Reference should be made to Figure B5 (Annexure BA) for the location of features and photographs taken on 9th March 2018.

Table B8-1: Personnel who observed embankment cracking on 9 March 2018

Name	Role	Company	Statement
Larry Wright	Mine Technician	Newcrest	(2018-006)
Nick Emms	Surface Operations Supervisor	Newcrest	
Peter Lord	Specialist Tailings Dams	Newcrest	(2018-020)
Peter Udy	Surface Operations Superintendent	Newcrest	(2018-017)
Steven Roberts	Tailings Area Supervisor	Newcrest	(2018-018)
Jason Ingham	ERT Officer	Newcrest	
Peter Sharpe	General Manager	Newcrest	
Lindsay Potts	Mining Manager	Newcrest	
David Cuello	Geotechnical Engineer	Newcrest	
Genevieve New	Dam Engineer	ATCW	(2018-009)
Travis Small	Site Surveyor	Newcrest	(2018-019)

B8.2 9th March Time Line

B8.2.1 07:30

On the morning of the 9 March 2018, Larry Wright travelled to the NTSF to install a VWP into Standpipe P9 on the Stage 5 crest at Chainage 2150. While unwinding the piezometer cables onto the ground, Larry noticed that cracks 1 to 1.5 m long had formed approximately across the crest. Larry investigated the next crests up (Stage 6 and 7) and found additional cracks that were much longer and wider. Larry advised Nick Emms that further investigation was warranted and returned to Standpipe P9 where he observed that the cracks had widened in the short time that he had been away.

A photograph of cracking taken at 07:48 on 9 March is presented as Photo B8-1.



Photo B8-1: Photograph of cracking on Stage 5 crest taken at 07:48 (2018-012)

B8.2.2 09:00

At approximately 9:15 am, Nick Emms arrived at Standpipe P9, and inspected the cracks. Nick then escalated the situation to Peter Lord as well as Peter Udy and Steven Roberts. Peter Lord, Peter Udy, and Steven Roberts arrived at the scene at approximately 9:35 am. They inspected the cracking on the Stage 7 crest, and observed them to be 100 m long, and 10-20 mm wide. Peter Lord then phoned ATC Williams Dam Engineer; Genevieve New and explained the situation. Genevieve and Peter agreed that the situation should be escalated further, and that all operations were to cease in the area. Peter then arranged for Genevieve to get on the first flight from Melbourne.

Peter Lord then called Jason Ingham as per the Dam Safety Emergency Plan (DSEP) and arranged for barricading to the area. After reviewing the DSEP, Peter Lord escalated the situation to a White Alert, pending further development, and removed all working personnel from the area. Peter Udy, Peter Lord and Steven then travelled back to the administration building and escalated the situation to Peter Sharpe; General Manager of CVO at 10:30 am.

Photographs of the cracking on the Stage 8 crest taken between 09:35 and 09:40 are presented as Photo B8-2 and Photo B8-3.



Photo B8-2: Stage 8 crest cracking looking west (2018-013)



Photo B8-3: Stage 8 crest cracking looking east. (2018-014)

B8.2.3 11:00

At approximately 11:00 am, Peter Sharpe, Peter Lord, Peter Udy and David Cuello travelled back to the NTSF to complete another inspection. During this time, they inspected the Stage 8 crest as well as the Stage 9 and 10 crests. There were no signs of deformation or cracking on either the Stage 9 or 10 crests. Subsequently, they inspected the embankment toe and observed some cracking on an old access ramp and also in the excavation at the toe of the dam. They had also observed a rock on the ramp that had been freshly dislodged onto the ground. Peter Lord and Peter Udy then organised for Steven Roberts to mark the cracking on the Stage 1 Buttress and Stage 8 crest with white paint to see if there was any movement, as well as measuring the Vibrating Wire Piezometers in the area.

The marked cracks can be clearly seen in drone photography taken at 16:00 and marked on Figure B5 (Annexure BA).

B8.2.4 12:00

At 12:00 pm, Peter Udy and Lindsay Potts re-inspected the area and observed that the cracks had advanced in length and width, and that they could hear rocks falling into the cracks, signifying that the cracks were continuous and deep.

B8.2.5 14:30

Genevieve New arrived on site at 2:30 pm and Genevieve, Dave Cuello and Peter Lord travelled to the NTSF and inspected the affected areas. On this trip, Peter Lord observed that the cracks had developed since the last inspection. They then inspected the toe of the embankment and observed that a large section of haul road at the toe had cracked and heaved. At this time, small rocks began to roll down the downstream face. It was then decided to remove all personnel from the area and for survey prisms to be placed on the downstream face of the NTSF.

Photographs taken at the embankment toe between 15:50 and 16:30 are presented as Photo B8-4 and Photo B8-5.



Photo B8-4: Cracking and heaving of haul road at Chainage 2060 (2018-015)



Photo B8-5: Cracking of excavated face in tailings at Chainage 2000 (2018-009)

A site visit record provided by Genevieve New as memorandum 115293.18 NTSF M013 is included as Annexure BF. Key observations included in this memorandum are:

- Cracking on Stage 1 Buttress extended diagonally from Stage 7 crest to edge as a 20m wide band of closed, 'en eschelon' cracks 5 to 10m long and 1 to 2m apart;
- The crack pattern on the Stage 7 crest was comprised of up to 10 individual cracks and was curved at either end;
- Cracks on Stage 7 crest were open 5 to 10cm, with no vertical displacement;
- Rocks falling into the Stage 7 crack could be heard after 15:00;
- Cracking on Stage 8 crest similar to Stage 7, but no sound could be heard;
- No cracking observed on Stage 9 crest and none reported by others on Stage 10;
- Longitudinal crack extended by 20m (2 to 5cm wide) on western side;
- Rupture in haul road on eastern side of borrow area at toe was 20m long;
- Significant cracking on excavated batter in old borrow area at toe; and
- Rocks could be heard rolling down batter near end of site visit at 16:30.

B8.2.6 16:00

Aerial images and topographic data were obtained at 16:00hrs using the CVO drone to overfly the NTSF in the area of cracking. The 16:00 orthophoto is included as Figure B4 in Annexure BA and forms the base to the annotated orthophoto presented as Figure B5.

B8.2.7 18:45

At approximately 6:45 pm, Travis Small travelled to the NTSF to survey prisms on the NTSF. At this time, Travis observed that the NTSF had slumped, and reported the failure to Peter Udy, who informed Lindsay Potts. The Dam Safety Emergency Plan was escalated and local residents in the vicinity of Panuara Road were evacuated.

B8.3 Post Failure Condition

B8.3.1 10th March 2018

An orthophoto of the slump taken at 10:00 on 10th March 2018 (2018-023) is provided as Figure B6 in Annexure BA, while an annotated version of this orthophoto is included as Figure B7.

Key features to note regarding the 10th March orthophoto are:

- A brown marker horizon at the Stage 2 crest level is reasonably continuous around the face of the slump;
- The eastern side of the slump appears to have rotated around a hinge point and has left an access ramp on the dam face and Drain 2 (Chainage 2000) in a relatively undisturbed condition.
- Apart from some tailings 'boils' at the toe of the slump, the tailings are contained within a perimeter of rockfill;
- Coarse rockfill associated with the Stage 1 Buttress can be clearly seen,
- Some sand boils are evident near the centre of the slump; and.
- A pump house adjacent to the access road has been moved 100m.

B8.3.2 11th March 2018

At 19:21 hrs on 11th March 2018, undisturbed tailings behind the 10th March failure surface, slumped and the liquefied tailings flowed over the southern face and the eastern corner of the slump.

Apart from some minor slumping of tailings that ensued, the slumped surface has remained relatively stable. This is reflected in the 14th March orthophoto (2018-024) (Figure B8 of Annexure BA) which is annotated as Figure B9.

Since 14th March 2018, the slump has remained relatively stable, with only minor degradation of the tailings face behind the slump being recorded.

B8.3.3 19th April 2018

During the ITRB site visit between 16th and 20th April 2018, the slump was accessed for the first time on the afternoon of 19th April, after conducting a comprehensive risk assessment. The following photos, taken on the afternoon of the 19th highlight some key features of the slump.

Photo B8-6. Surface of runout on southern side of slump had an appearance of "ropey lava" with a dried and cracked surface.

Photo B8-7: Backslope of slump with sand boils in foreground. The secondary slump on 14th March was concentrated on the eastern side of the backslope.

Photo B8-8: Left abutment showing well-constructed and clear embankment zoning. The overlapping Z pattern of the embankment clay can be seen in this photograph.

Photo B8-9: Eastern side of slump showing basal clay layer to Stage 4 or above. The basal clay layer in this photo has been broken into slabs, with underlying geotextile and working platform.



Photo B8-6: Ropey and cracked tailings from secondary runoff.



Photo B8-7: Backslope of slump with sand boils in foreground.



Photo B8-8: Left abutment showing well-constructed and clear embankment zoning.



Photo B8-9: Eastern side of slump showing basal clay layer to Stage 4 or above.

Annexure BA

Figures

Figure B1 – Cadia NTSF Timeline 1995 to 2018

Figure B2 – Cadia NTSF Timeline Dec 2017 to Mar 2018

Figure B3 – Cadia NTSF Timeline 8 to 9 March 2018

Figure B4 – Orthophoto Map – March 9, 2018

Figure B5 – Annotated Orthophoto Map March 9, 2018

Figure B6 – Orthophoto Map March 10, 2018

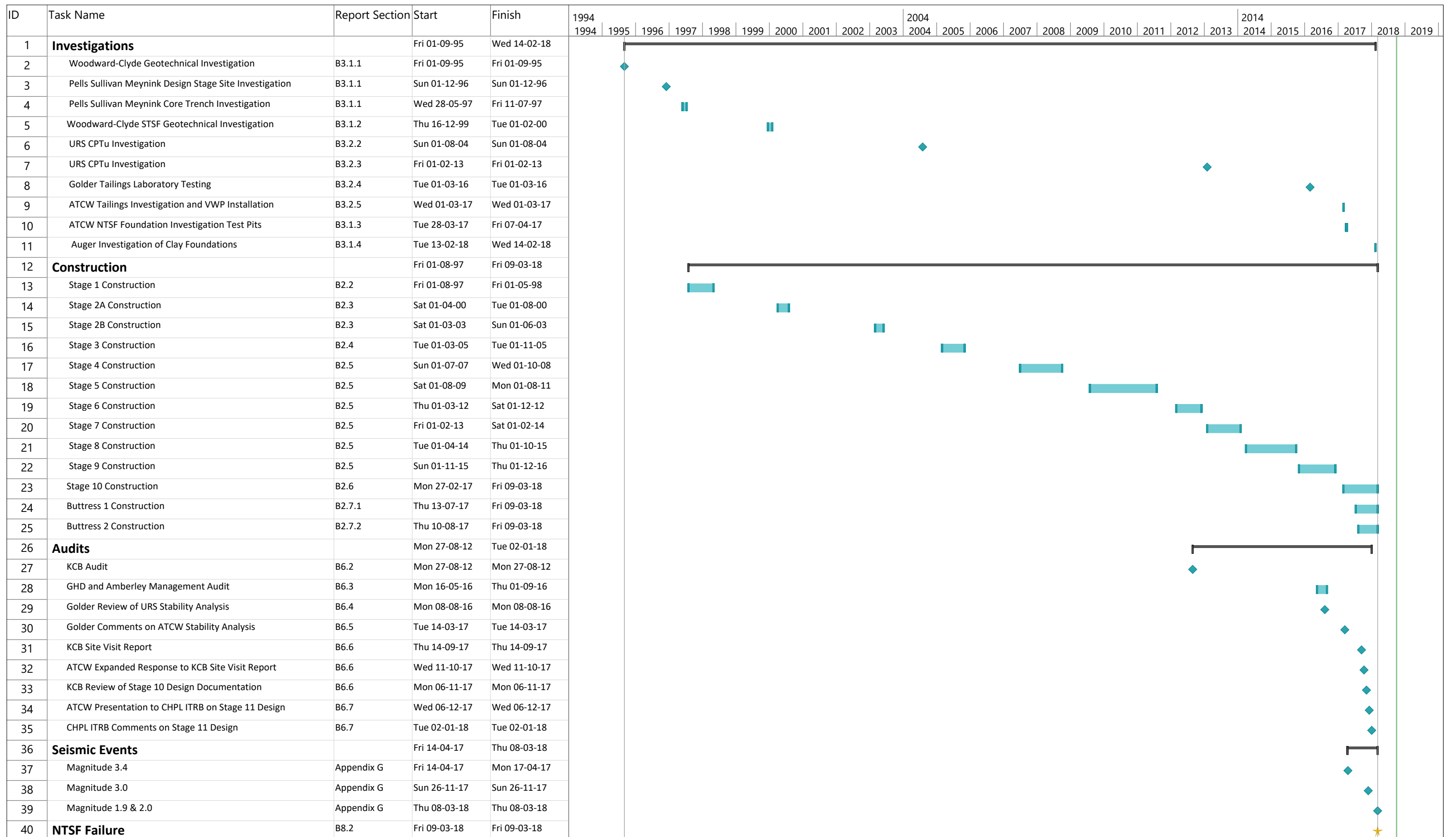
Figure B7 – Annotated Orthophoto Map March 10, 2018

Figure B8 – Orthophoto Map March 14, 2018

Figure B9 – Annotated Orthophoto Map March 14, 2018

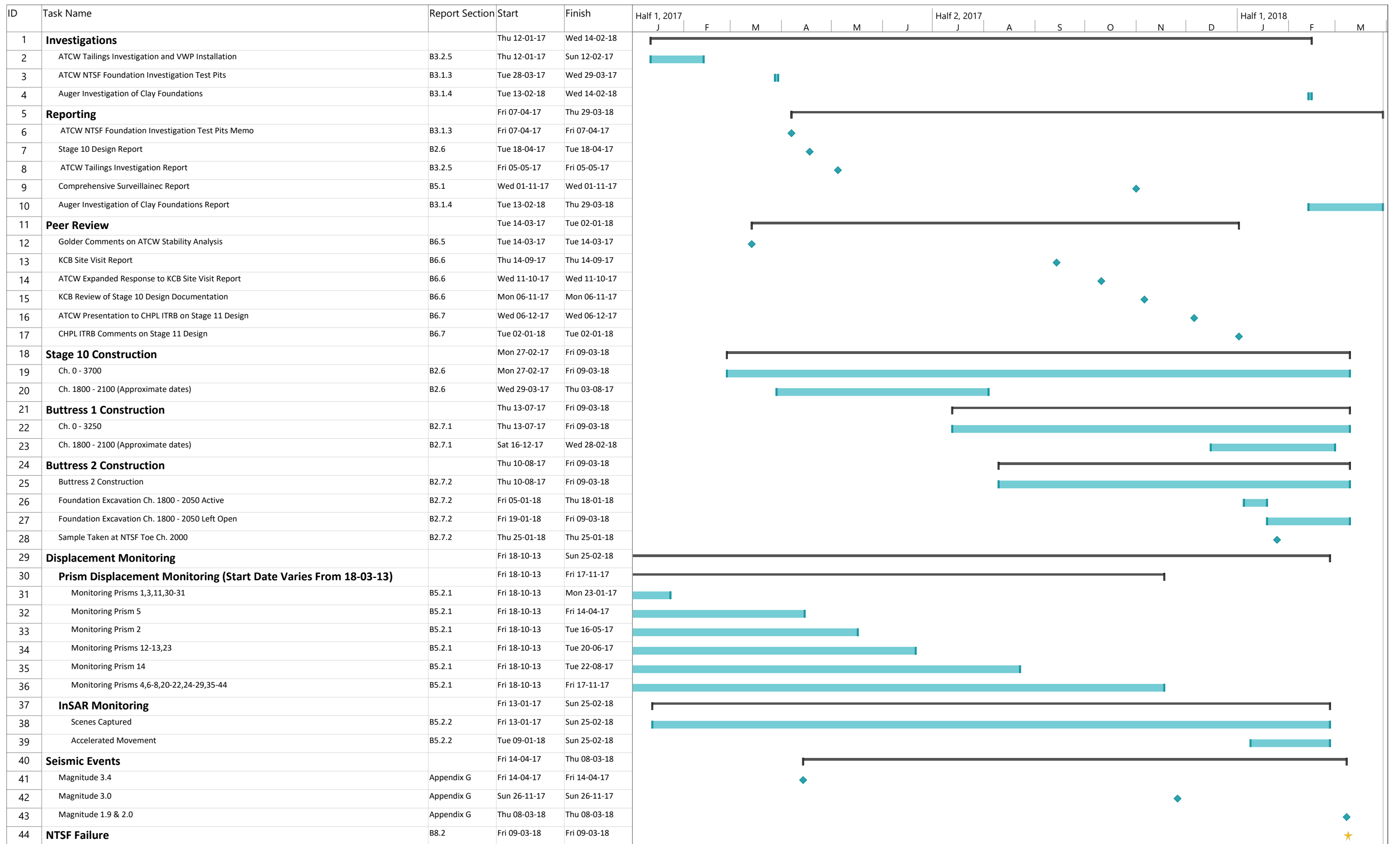
Figure B10 – NTSF Previous Investigation Locations

Figure B11 – NTSF Plan with CPT Locations



Report on NTSF Embankment Failure
Appendix B
Cadia NTSF Timeline - 1995 to 2018
Figure B1

Manual Task [Task bar] Manual Milestone [Milestone diamond] Manual Summary [Summary bar]



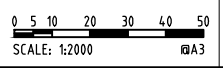
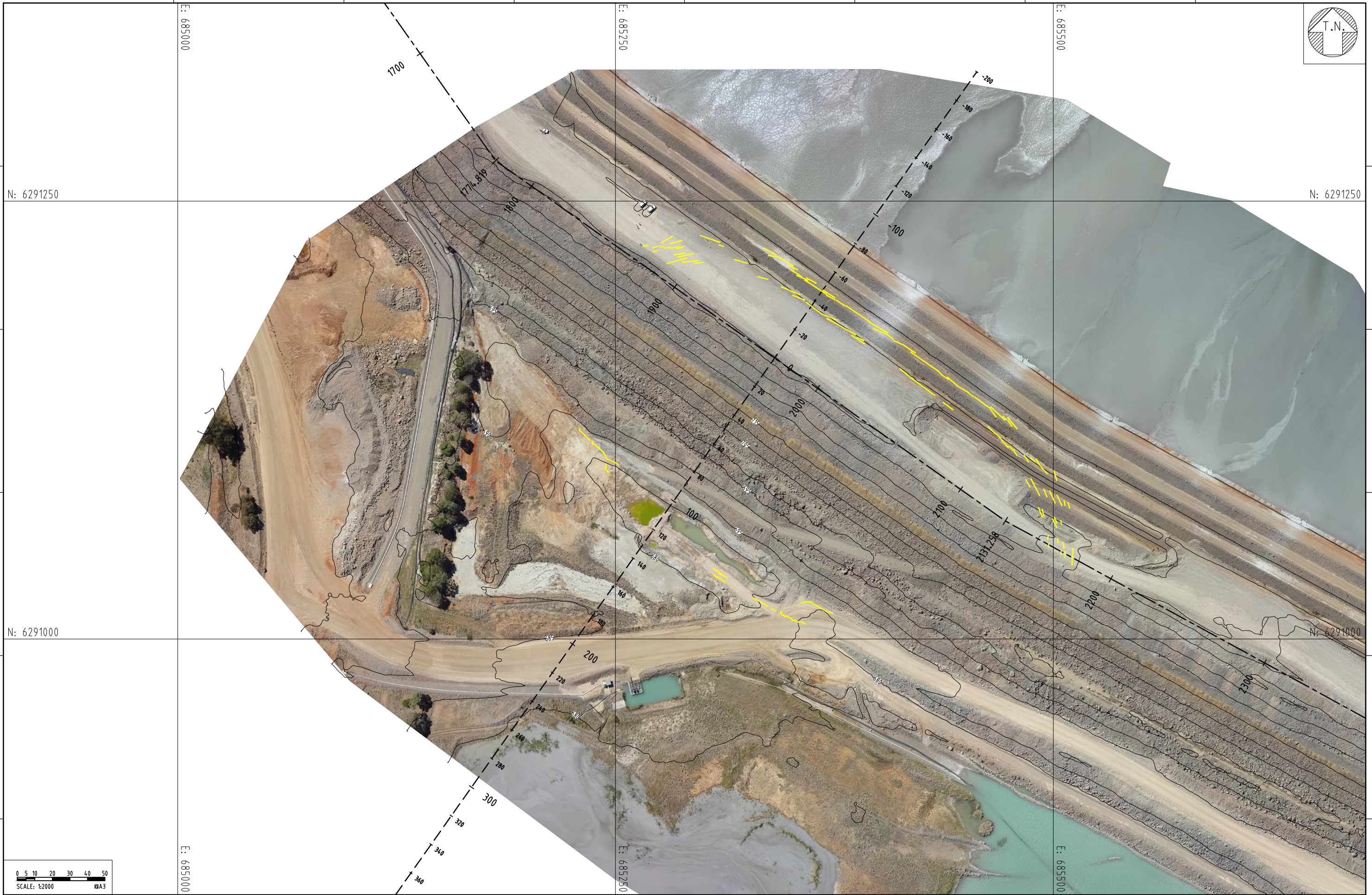
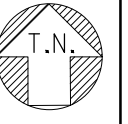
Report on NTSF Embankment Failure
Appendix B
Cadia NTSF Timeline - Dec 2017 to Mar 2018
Figure B2

Manual Task [Bar] Manual Milestone [Diamond] Manual Summary [Bracket]

ID	Task Name	Report Section	Start	Thu 08 Mar				Fri 09 Mar				Sat 10 Mar								
				12 AM	3 AM	6 AM	9 AM	12 PM	3 PM	6 PM	9 PM	12 AM	3 AM	6 AM	9 AM	12 PM	3 PM	6 PM	9 PM	12 AM
1	Stage 1 Buttress Construction from Eastern Tiphead	B2.7.1	Thu 08-03-18 6:30 AM																	
2	Cracks first noticed	B8.2.1	Fri 09-03-18 7:30 AM																	
3	Situation escalated. Designers notified and work ceased. Site barricaded.	B8.2.2	Fri 09-03-18 9:35 AM																	
4	Detailed inspection of Stage 9 and 10 crests and toe - cracking observed at toe. Cracks marked.	B8.2.3	Fri 09-03-18 11:00 AM																	
5	Cracks widening and rocks heard falling into cracks.	B8.2.4	Fri 09-03-18 12:00 PM																	
6	Designer arrives at site and inspects area. Haul road at toe heaving and rocks falling down face. Personnel removed from area.	B8.2.5	Fri 09-03-18 2:30 PM																	
7	Drone flight.	B8.2.6	Fri 09-03-18 4:00 PM																	
8	Survey returns to site and observes embankment has slumped.	B8.2.7	Fri 09-03-18 6:45 PM																	

Report on NTSF Embankment Failure
Appendix B
Cadia NTSF Timeline - 8 to 9 March 2018
Figure B3

Manual Task  Manual Milestone  Manual Summary 



- LEGEND**
- MAPPED CRACKS
 - 1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
 - 2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
 - 3. VERTICAL DATUM CORRESPONDS TO AHD.
 - 4. AERIAL PHOTO AND CONTOURS COMPLETED BY NEWCREST ON THE 9TH OF MARCH 2018.

ORTHOPHOTO MAP - MARCH 9, 2018

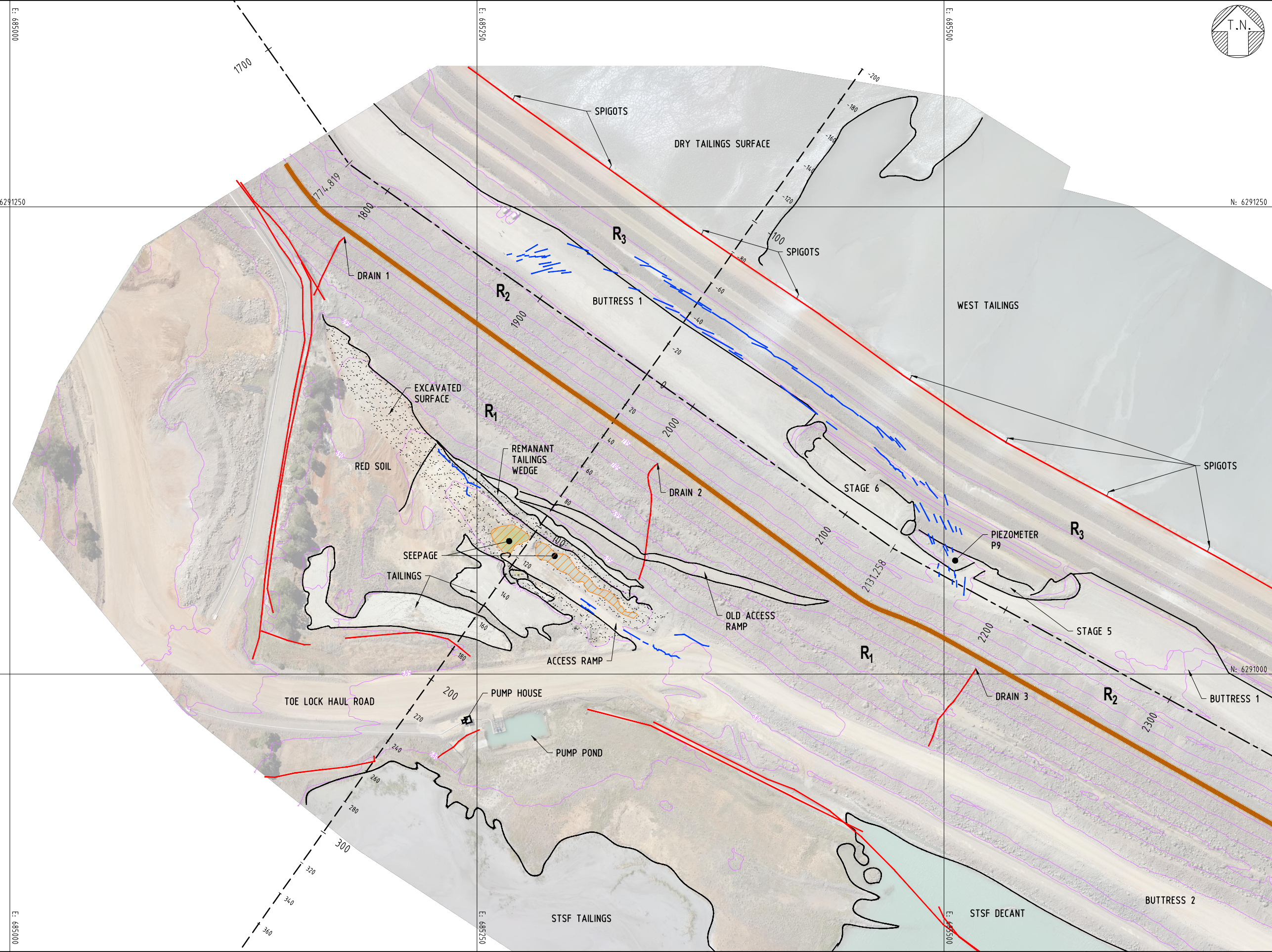
CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B4

DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-272-0101	

LEGEND

R ₁	STAGE 2 ROCK FILL
R ₂	COARSE ROCK FILL
R ₃	ROCK FILL
M ₁	MIXED ROCKS AND EARTHFILL
M ₂	FILL MIXED WITH TAILINGS
S ₁	SLUMPED TAILINGS 10/03/2018
S ₂	SLUMPED TAILINGS 14/03/2018
T ₁	INTERBEDDED TAILINGS
T ₂	FINER TAILINGS
(Blue dashed line)	MAPPED CRACKS
(Green line)	TOE OF SLUMP
(Brown line)	BROWN MARKER HORIZON
(Black line)	SCARP
(Red line)	PIPES
(Square with X)	PUMP HOUSE
(Blue wavy area)	PONDED WATER
(Orange wavy area)	SEEPAGE
(Dotted area)	EXCAVATED SURFACE
(Star symbol)	SAND BOIL
(Arrow symbol)	SEEPAGE
(Black arrow)	DIRECTION OF MOVEMENT



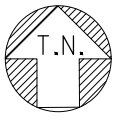
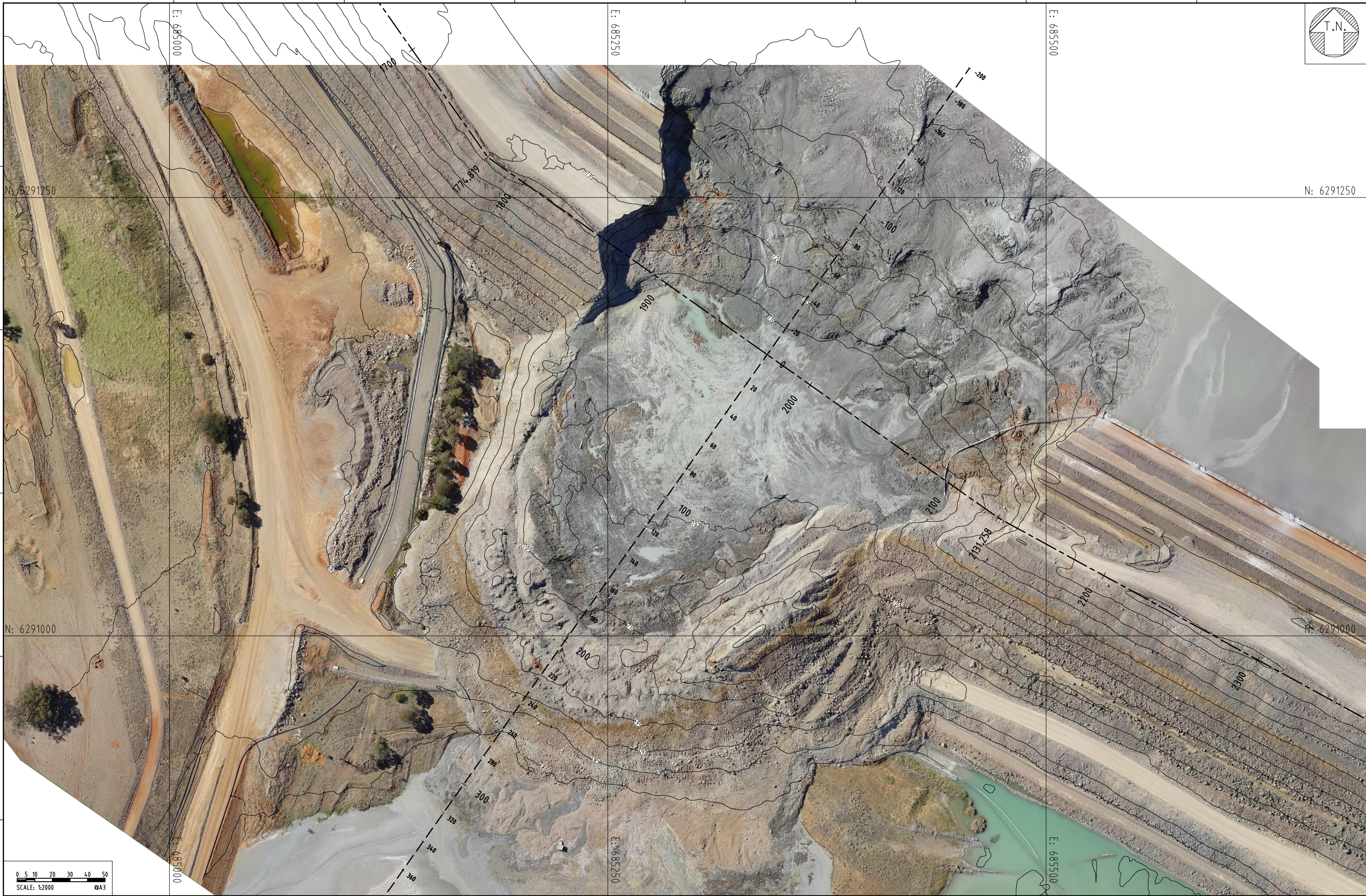
- NOTES**
1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
 2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
 3. VERTICAL DATUM CORRESPONDS TO AHD.
 4. AERIAL PHOTO AND CONTOURS COMPLETED BY NEWCREST ON THE 9TH OF MARCH 2018.

ANNOTATED ORTHOPHO MAP - MARCH 9, 2018

CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B5

DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-272-0201	



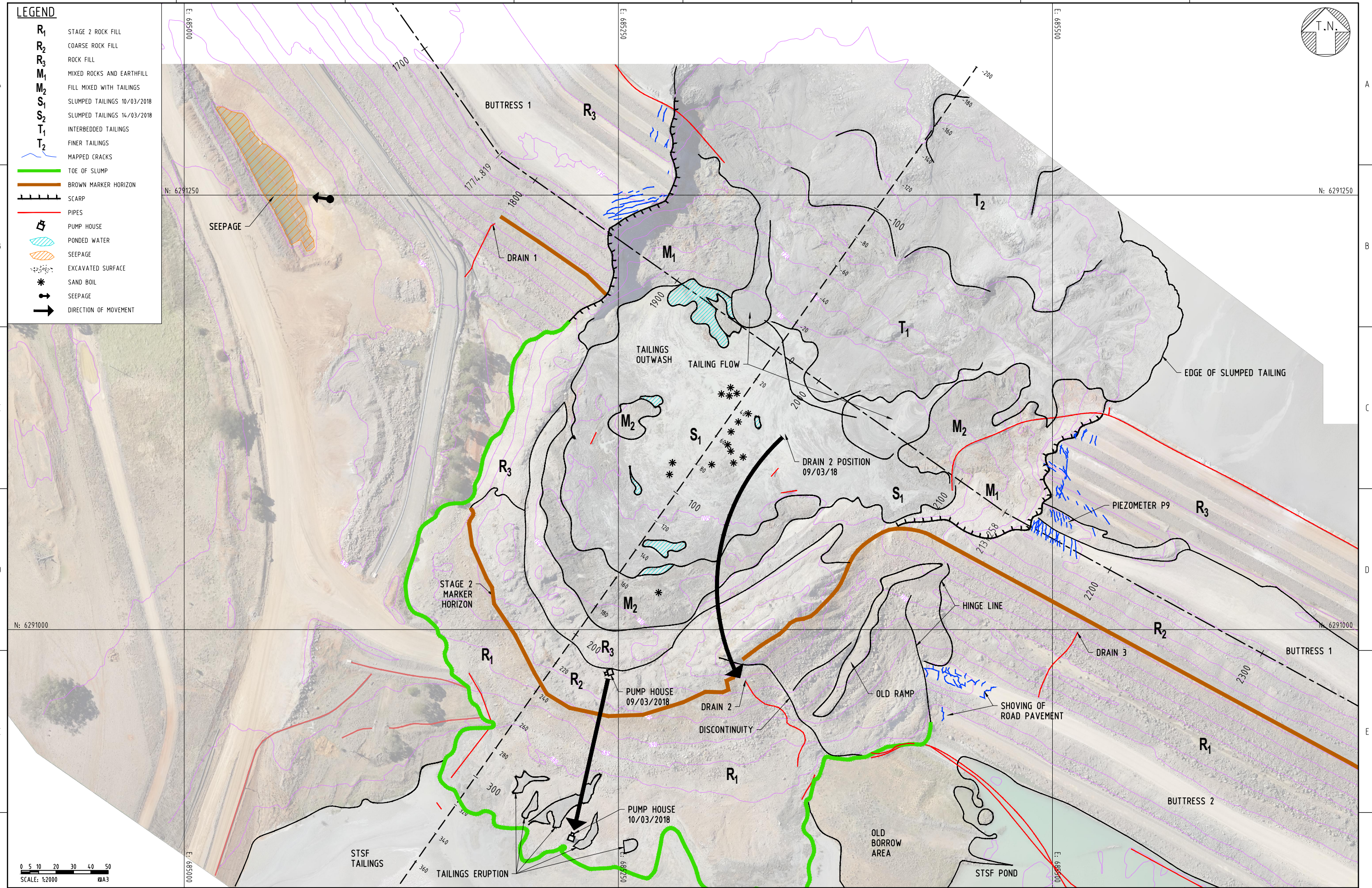
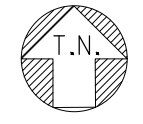
- NOTES**
1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
 2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
 3. VERTICAL DATUM CORRESPONDS TO AHD.
 4. AERIAL PHOTO AND CONTOURS COMPLETED BY NEWCREST ON THE 10TH OF MARCH 2018.

ORTHOPHOTO MAP - MARCH 10, 2018

CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B6		
DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-272-0102	

- LEGEND**
- R₁** STAGE 2 ROCK FILL
 - R₂** COARSE ROCK FILL
 - R₃** ROCK FILL
 - M₁** MIXED ROCKS AND EARTHFILL
 - M₂** FILL MIXED WITH TAILINGS
 - S₁** SLUMPED TAILINGS 10/03/2018
 - S₂** SLUMPED TAILINGS 14/03/2018
 - T₁** INTERBEDDED TAILINGS
 - T₂** FINER TAILINGS
 - MAPPED CRACKS
 - TOE OF SLUMP
 - BROWN MARKER HORIZON
 - SCARP
 - PIPES
 - PUMP HOUSE
 - PONDED WATER
 - SEEPAGE
 - EXCAVATED SURFACE
 - SAND BOIL
 - SEEPAGE
 - DIRECTION OF MOVEMENT

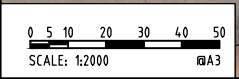
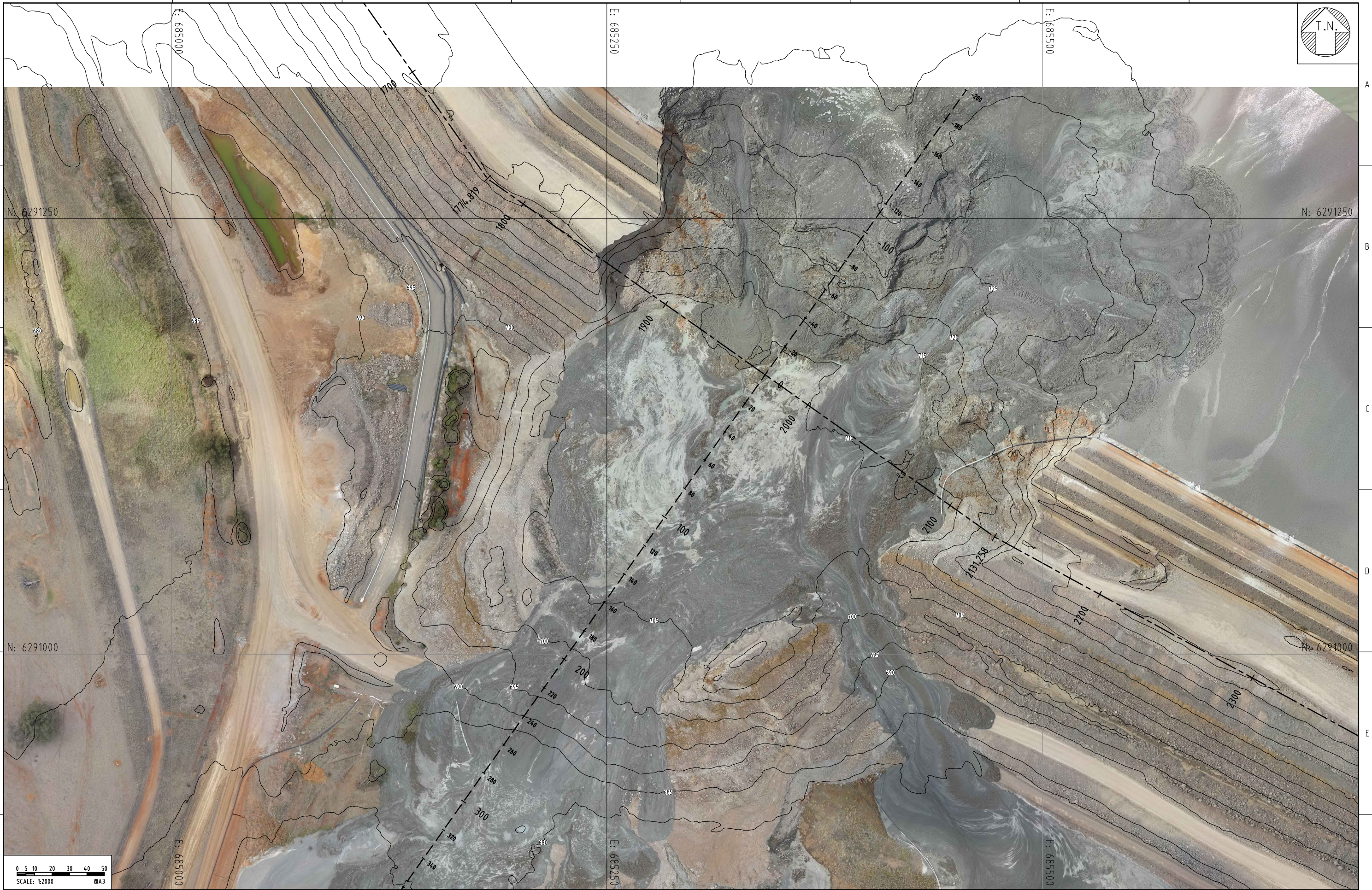
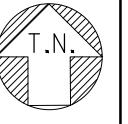


- NOTES**
1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
 2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
 3. VERTICAL DATUM CORRESPONDS TO AHD.
 4. AERIAL PHOTO AND CONTOURS COMPLETED BY NEWCREST ON THE 10TH OF MARCH 2018.

ANNOTATED ORTHOPHO MAP - MARCH 10, 2018

CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B7		
DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-272-0202	



- NOTES**
1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
 2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
 3. VERTICAL DATUM CORRESPONDS TO AHD.
 4. AERIAL PHOTO AND CONTOURS COMPLETED BY NEWCREST ON THE 14TH OF MARCH 2018.

ORTHOPHOTO MAP - MARCH 14, 2018

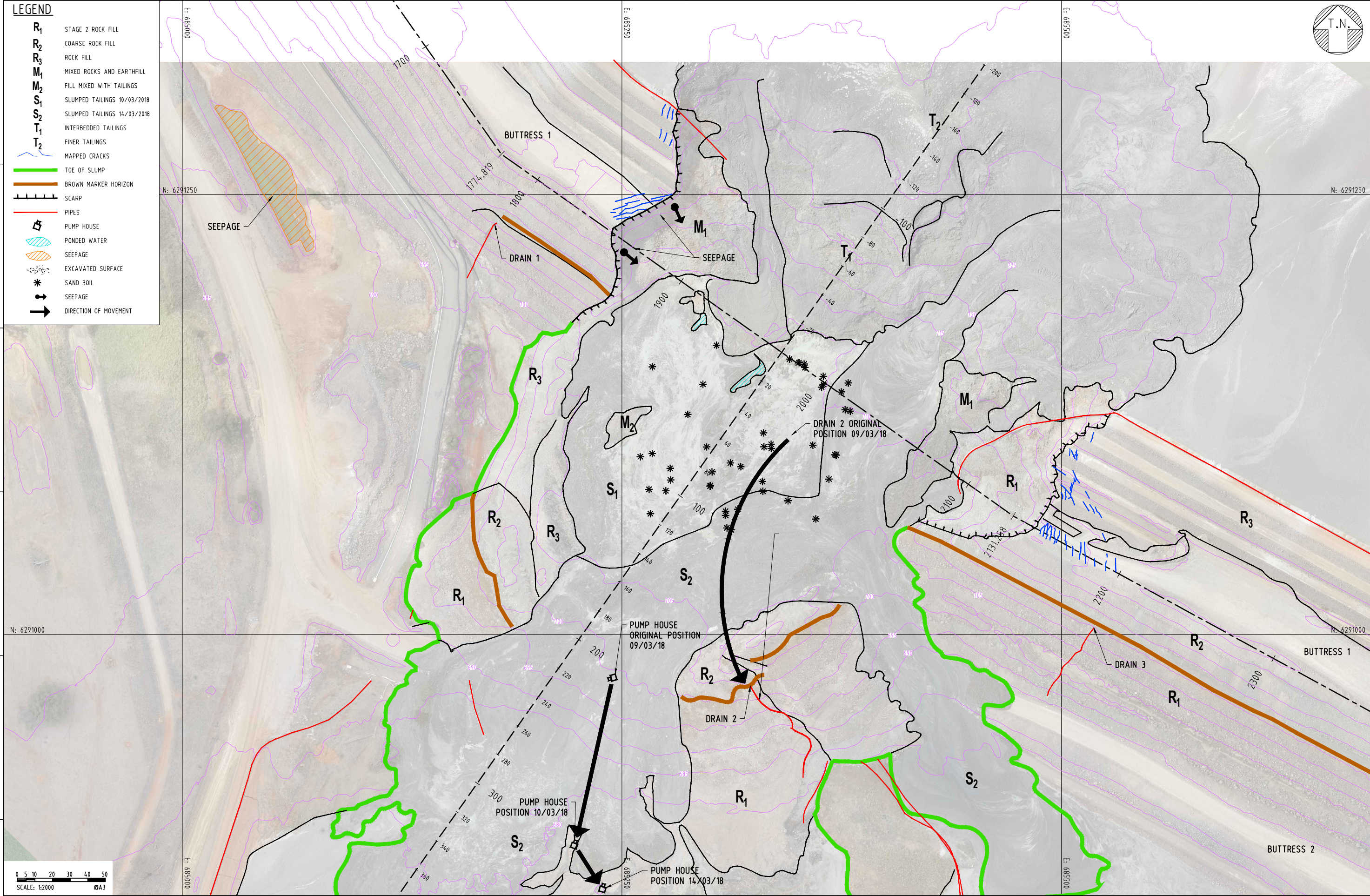
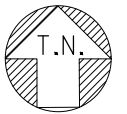
CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B8

DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-272-0103	

LEGEND

R ₁	STAGE 2 ROCK FILL
R ₂	COARSE ROCK FILL
R ₃	ROCK FILL
M ₁	MIXED ROCKS AND EARTHFILL
M ₂	FILL MIXED WITH TAILINGS
S ₁	SLUMPED TAILINGS 10/03/2018
S ₂	SLUMPED TAILINGS 14/03/2018
T ₁	INTERBEDDED TAILINGS
T ₂	FINER TAILINGS
(Blue dashed line)	MAPPED CRACKS
(Green line)	TOE OF SLUMP
(Brown line)	BROWN MARKER HORIZON
(Black line)	SCARP
(Red line)	PIPES
(Square with X)	PUMP HOUSE
(Blue hatched area)	PONDED WATER
(Orange hatched area)	SEEPAGE
(Dotted area)	EXCAVATED SURFACE
(Asterisk)	SAND BOIL
(Arrow with dot)	SEEPAGE
(Black arrow)	DIRECTION OF MOVEMENT



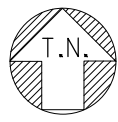
- NOTES**
1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
 2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
 3. VERTICAL DATUM CORRESPONDS TO AHD.
 4. AERIAL PHOTO AND CONTOURS COMPLETED BY NEWCREST ON THE 14TH OF MARCH 2018.

ANNOTATED ORTHOPHO MAP - MARCH 14, 2018

CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B9

DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-272-0203	



NOTES

1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
3. VERTICAL DATUM CORRESPONDS TO AHD.
4. LIDAR SURVEY COMPLETED BY AAM ON THE 19TH OF MARCH 2018.

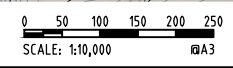
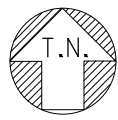
LEGEND

- ◆ PELLIS SULLIVAN MEYNINK (DECEMBER, 1996) DRILLHOLE
- PELLIS SULLIVAN MEYNINK (DECEMBER, 1996) TEST PIT
- ◆ WOODWARD-CLYDE (SEPTEMBER, 1995) DRILLHOLE
- WOODWARD-CLYDE (SEPTEMBER, 1995) TEST PIT
- ◆ NEWCREST DRILLHOLE
- NEWCREST TEST PIT
- ◆ ATCW (APRIL, 2017) DRILLHOLE
- ◆ ATCW (MARCH, 2018) TEST PIT

NTSF PREVIOUS INVESTIGATION LOCATIONS

CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B10		
DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-270-0301	



- NOTES**
1. ALL DIMENSIONS, ELEVATIONS AND COORDINATES ARE IN METERS, EXCEPT WHERE INDICATED OTHERWISE.
 2. HORIZONTAL DATUM CORRESPONDS TO GDA 94 MGA ZONE 55.
 3. VERTICAL DATUM CORRESPONDS TO AHD.
 4. LIDAR SURVEY COMPLETED BY AAM ON THE 19TH OF MARCH 2018.

- LEGEND**
- "2013 URS INVESTIGATION" CPT
 - "2017 ATCW TAILINGS INVESTIGATION" CPT AND VWP

NTSF PLAN WITH CPT LOCATIONS

CADIA NTSF FAILURE INDEPENDENT TECHNICAL REVIEW BOARD

FIGURE B11

DESIGNED IG	DRAWN PK	APPROVED
DATE 13-02-2019	DWG. No. H356804-00000-22A-270-0302	

Annexure BB

Nomenclature

BB.1.1 Co-ordinate Systems

A number of co-ordinate systems have been used in relation to the NTSF. These are:

- ISG A NSW co-ordinate system used for the initial investigations.
- Mine Grid Used for design of all stages of NTSF and STSF.
- GDA94 Co-ordinate system based on Mapping Grid of Australia Zone 55 (MGA55) and used for historical aerial photography and post failure aerial photography and LiDAR.

Unless indicated to the contrary, the MGA co-ordinate system has been used in this report.

The Cadia Mine Grid is rotated 19° clockwise from Magnetic North. As the magnetic declination is approximately 12° east of True North, the Cadia Mine Grid is rotated approximately 31° clockwise from True North.

BB.1.2 Height Datum

To ensure that negative elevations were not encountered in the CVO underground operations, the Cadia Mine Local Height Datum was set at 5000 metres above the Australian Height Datum (AHD).

Unless indicated to the contrary, the Australian Height Datum (AHD) has been used in this appendix and elsewhere within the report.

BB.1.3 Digital Terrain Models

The following digital terrain models are available for the NTSF and STSF:

- Pre-construction ground surface model with a 2m contour interval based on photogrammetry.
- As – constructed survey. For each embankment stage, the embankment construction was surveyed and the design strings were superimposed on the pre-construction ground survey and previous construction. This model cannot be relied upon other than the specific embankment construction details.
- 2016 DTM. Lidar and aerial photography was taken of the NTSF and STSF in late 2016.
- Drone Data. Drone flights of the NTSF embankment failure were completed at 10:00hrs and 16:00hrs commencing at 16:00hrs on 9th March 2018. A digital terrain model and a georeferenced photomosaic were produced from each flight.
- 2018 DTM. LiDAR and aerial photography was obtained using a fixed wing aircraft on 17th March 2018. The data set has a specified vertical accuracy of 0.075m RMS and a horizontal accuracy of 0.1m RMS. Contours were supplied at 0.5m intervals.

BB.1.4 Dam Chainages

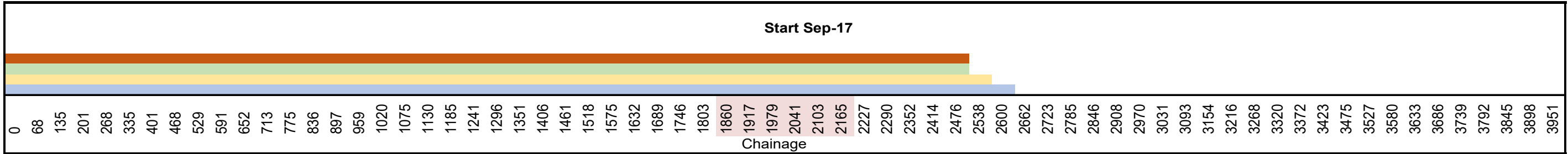
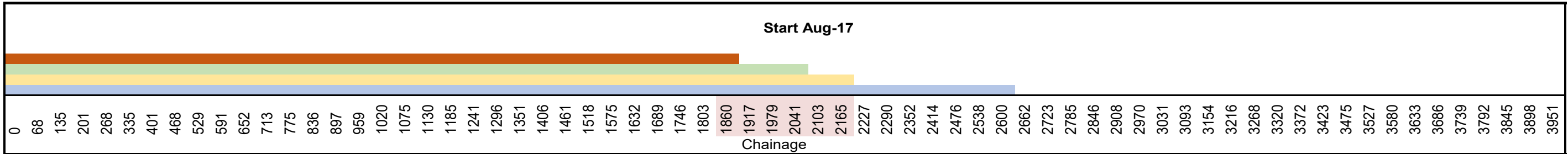
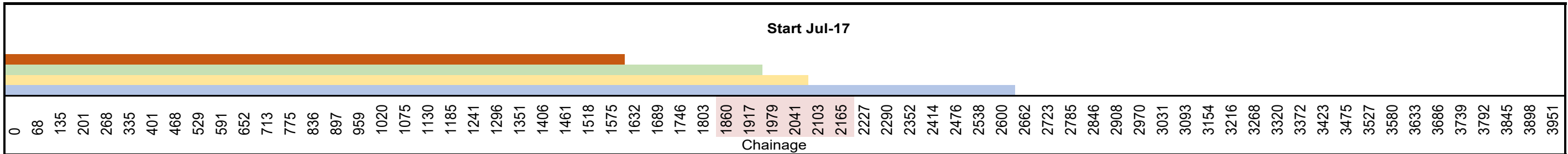
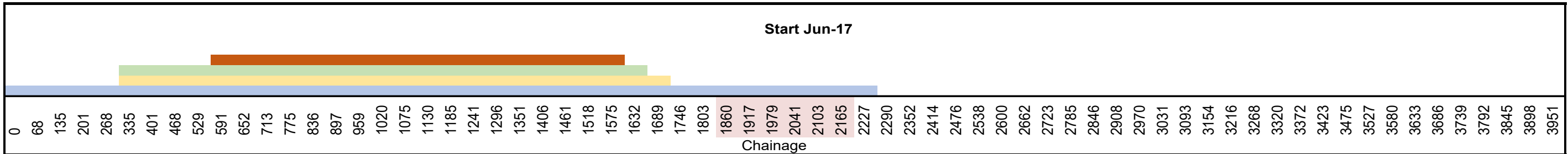
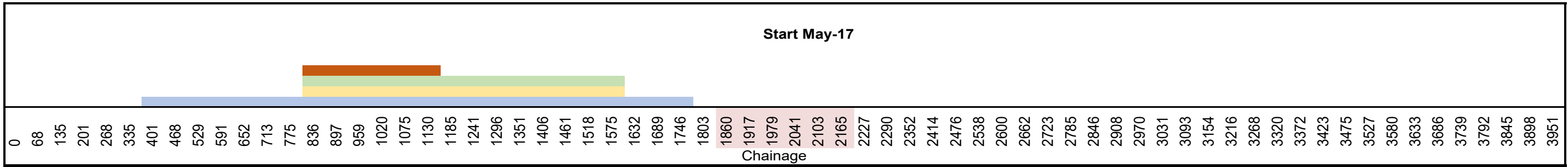
NTSF embankment chainages for Stages 1 to 9 are based on a common reference line with zero chainage at RL741 on the right abutment. The Set Out Line (SOL) is located at the upstream crest of the Stage 2 design.

As the Stage 10 crest is at RL744, zero chainage has been moved 100m to the north of Stages 1 to 9 and the reference line is the downstream crest of the Stage 10 embankment.

Unless indicated to the contrary, chainages in this report reference the Stage 1 to 9 SOL. this report.

Annexure BC

Stage 10 Construction Program



Upstream Clay
3a/3b Rockfill
Blanket
Working Platform

Slump Zone

Job number	H356804		
Ref	H356804-00000-22A-031-00XX		
By	TMY	IAG	12-Feb-19
Revision	0		12-Feb-19

Report on NTSF Embankment Failure

**Stage 10 Lift
Construction Timeline
May-Sep 2017**

Newcrest

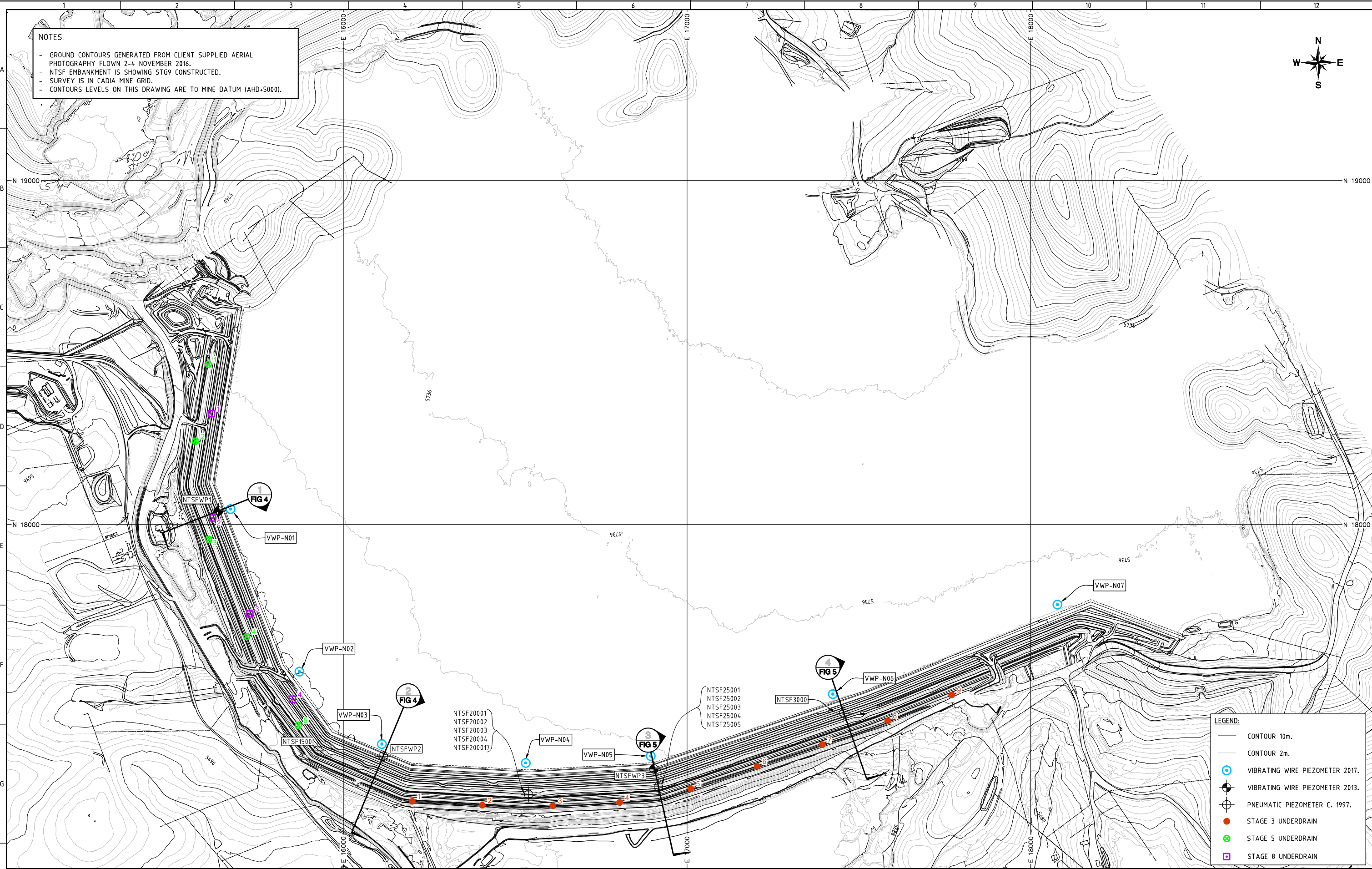
NTSF

ANNEXURE BC

Annexure BD

Monitoring Information

NOTES:
 - GROUND CONTOURS GENERATED FROM CLIENT SUPPLIED AERIAL PHOTOGRAPHY FLOWN 2-4 NOVEMBER 2016.
 - NTSF EMBANKMENT IS SHOWING STG9 CONSTRUCTED.
 - SURVEY IS IN CADIA MINE GRID.
 - CONTOURS LEVELS ON THIS DRAWING ARE TO MINE DATUM (AHD+5000).



LEGEND:

- CONTOUR 10m.
- CONTOUR 2m.
- ⊙ VIBRATING WIRE PIEZOMETER 2017.
- ⊙ VIBRATING WIRE PIEZOMETER 2013.
- ⊕ PNEUMATIC PIEZOMETER C. 1997.
- STAGE 3 UNDERDRAIN
- STAGE 5 UNDERDRAIN
- STAGE 8 UNDERDRAIN

A ISSUED FOR REVIEW		06.07.17	JM		
No.	DESCRIPTION	DATE	DRAWN	CHECKD	APPRD

SCALE:	1:10,000
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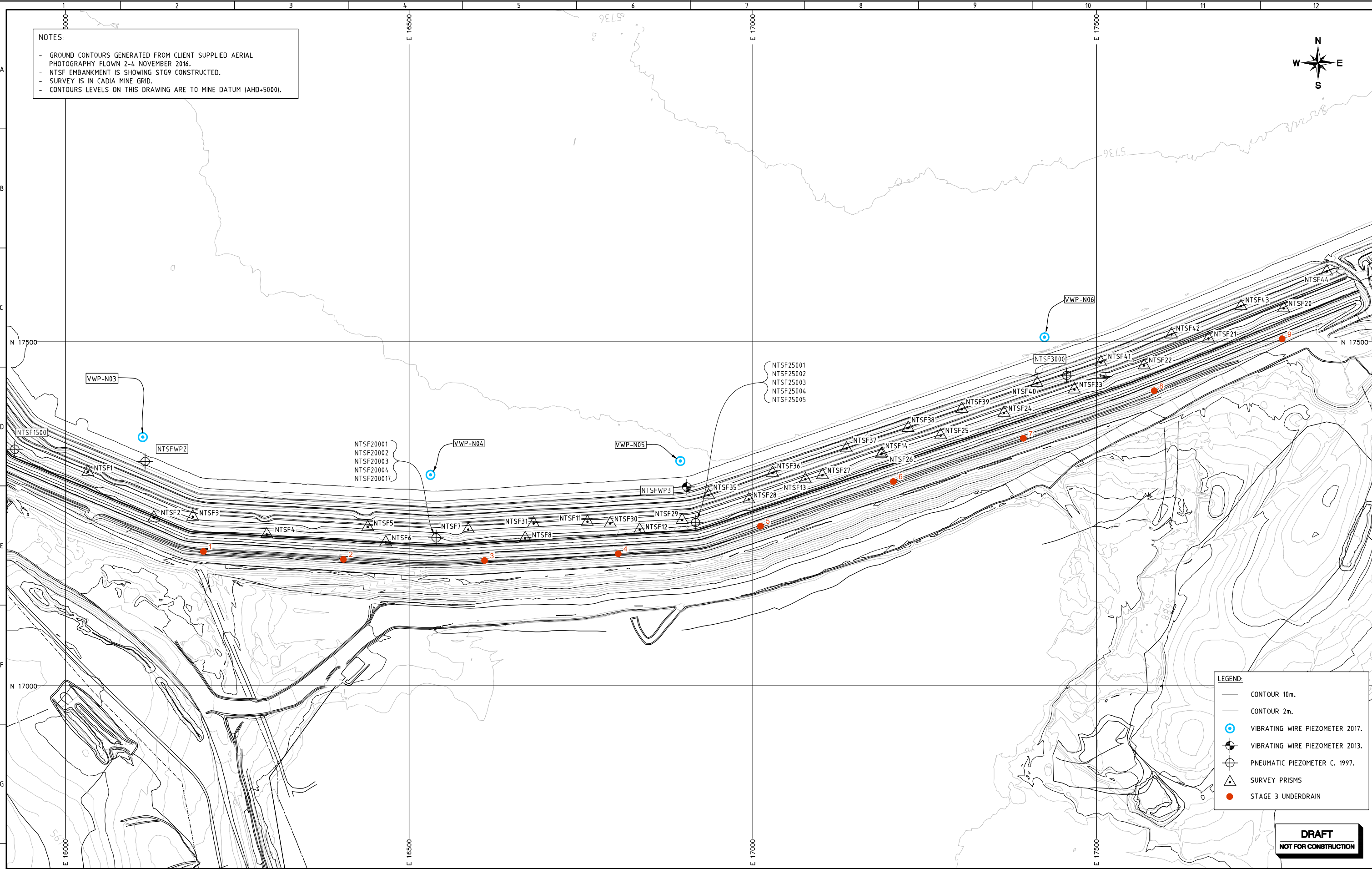
NEWCREST MINING LIMITED
CADIA VALLEY OPERATIONS

NORTHERN TAILINGS STORAGE FACILITY
GENERAL LAYOUT

DWG. No.		FIGURE 2	
SHEET SIZE	A3	Rev.	A
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SHEET 1 OF 1			

NOTES:

- GROUND CONTOURS GENERATED FROM CLIENT SUPPLIED AERIAL PHOTOGRAPHY FLOWN 2-4 NOVEMBER 2016.
- NTSF EMBANKMENT IS SHOWING STG9 CONSTRUCTED.
- SURVEY IS IN CADIA MINE GRID.
- CONTOURS LEVELS ON THIS DRAWING ARE TO MINE DATUM (AHD+5000).



LEGEND:

- CONTOUR 10m.
- CONTOUR 2m.
- ⊙ VIBRATING WIRE PIEZOMETER 2017.
- ⊙ VIBRATING WIRE PIEZOMETER 2013.
- ⊕ PNEUMATIC PIEZOMETER C. 1997.
- △ SURVEY PRISMS
- STAGE 3 UNDERDRAIN

DRAFT
NOT FOR CONSTRUCTION

				SCALE: 1:5,000	
				JOB No. 116298.12	
				DATE 11.07.17	
				DESIGN GN	
				DRAWN AC	
				CHECKED	
				APPROVED	
A	ISSUED FOR REVIEW	11.07.17	AC		
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NEWCREST MINING LIMITED
CADIA VALLEY OPERATIONS

NORTHERN TAILINGS STORAGE FACILITY
MAIN EMBANKMENT

DWG. No. **FIGURE 3**

SHEET SIZE	A3	Rev.	A
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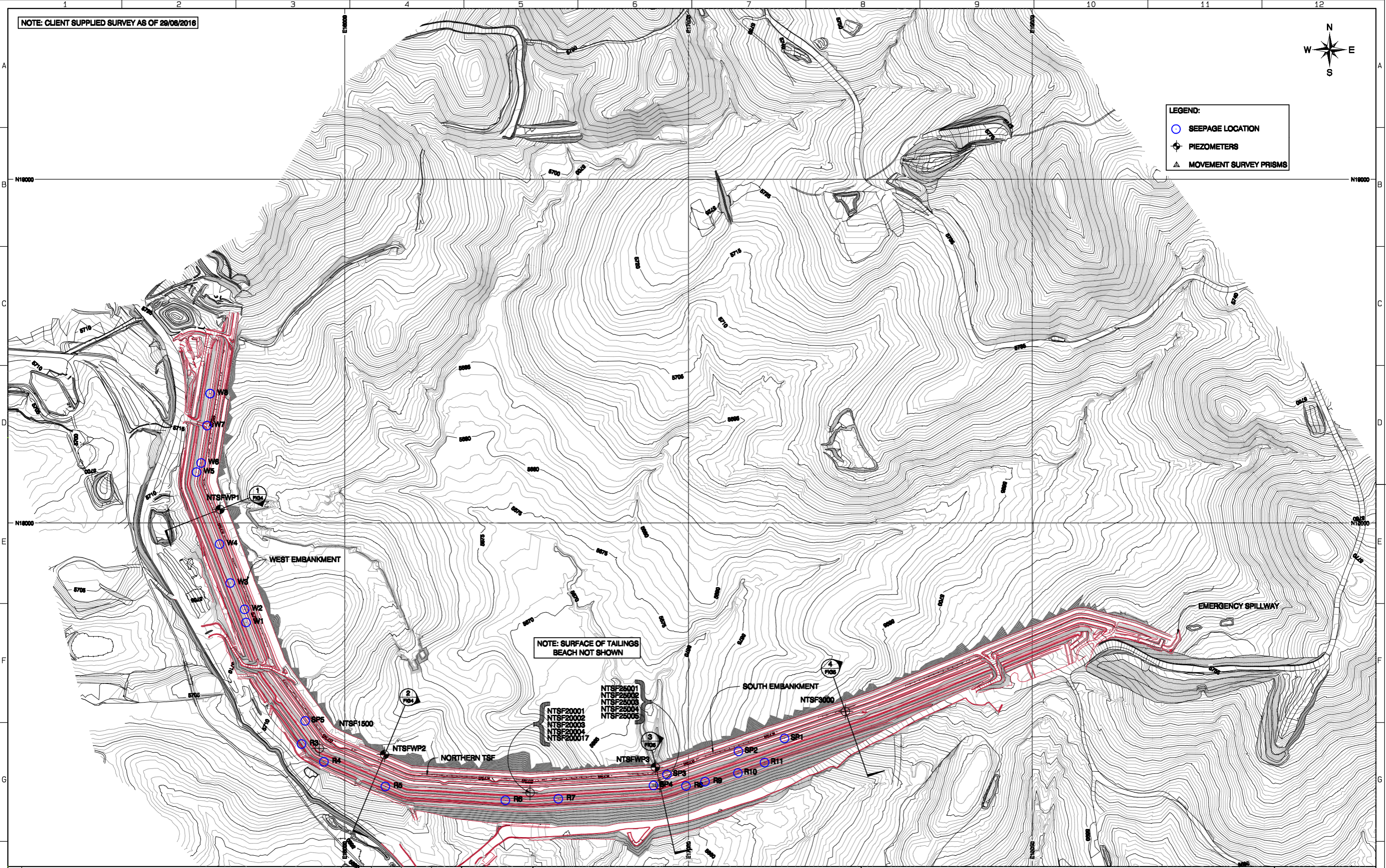
SHEET 1 OF 1

NOTE: CLIENT SUPPLIED SURVEY AS OF 29/08/2016



LEGEND:

- SEEPAGE LOCATION
- ⊕ PIEZOMETERS
- △ MOVEMENT SURVEY PRISMS



NOTE: SURFACE OF TAILINGS BEACH NOT SHOWN

SCALE	1: 5,000 @ A3
JOB No.	116288.04
DATE	29.08.2016
DESIGN	GN
DRAWN	SM
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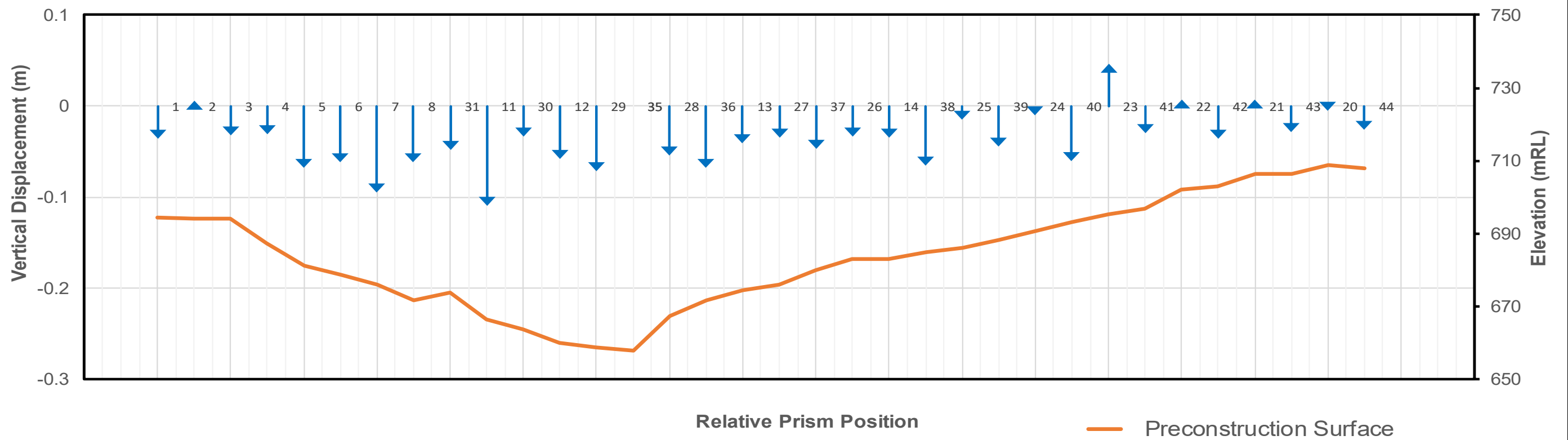
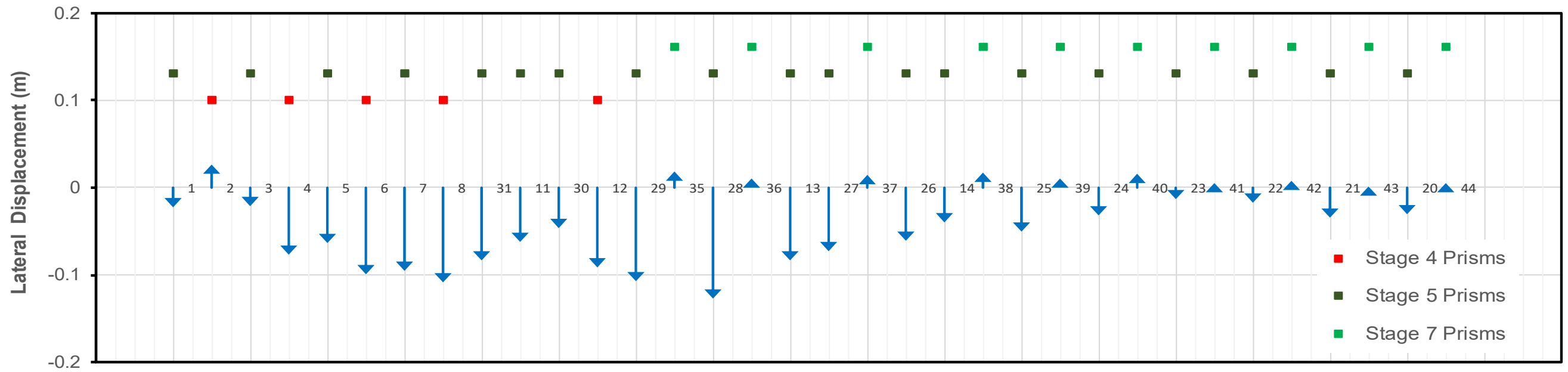
**NEWCREST MINING LIMITED
CADIA SURVEILLANCE 2016**

**NORTHERN TAILINGS STORAGE FACILITY
GENERAL LAYOUT**

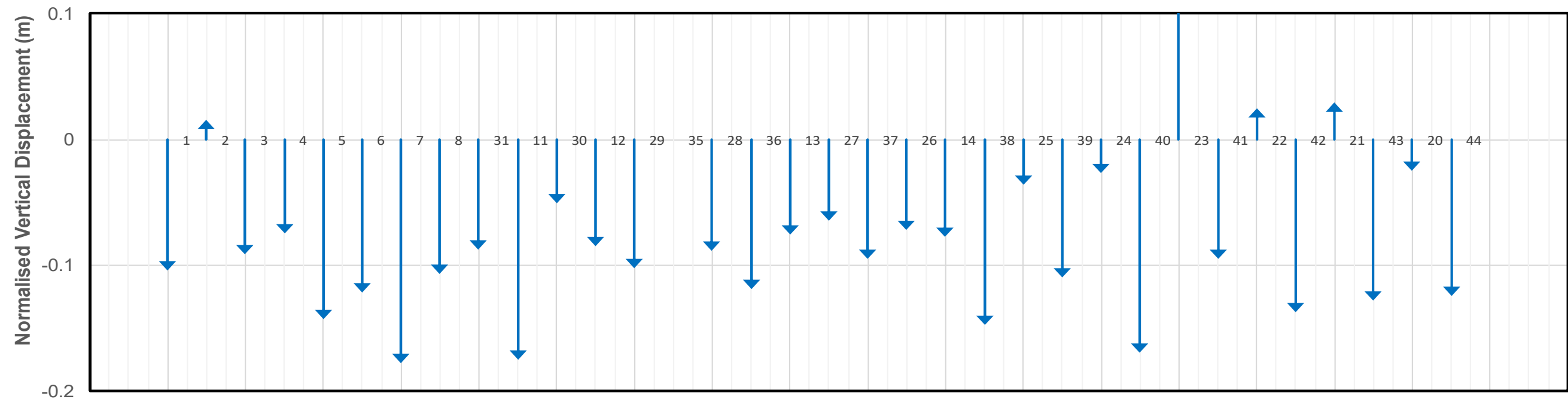
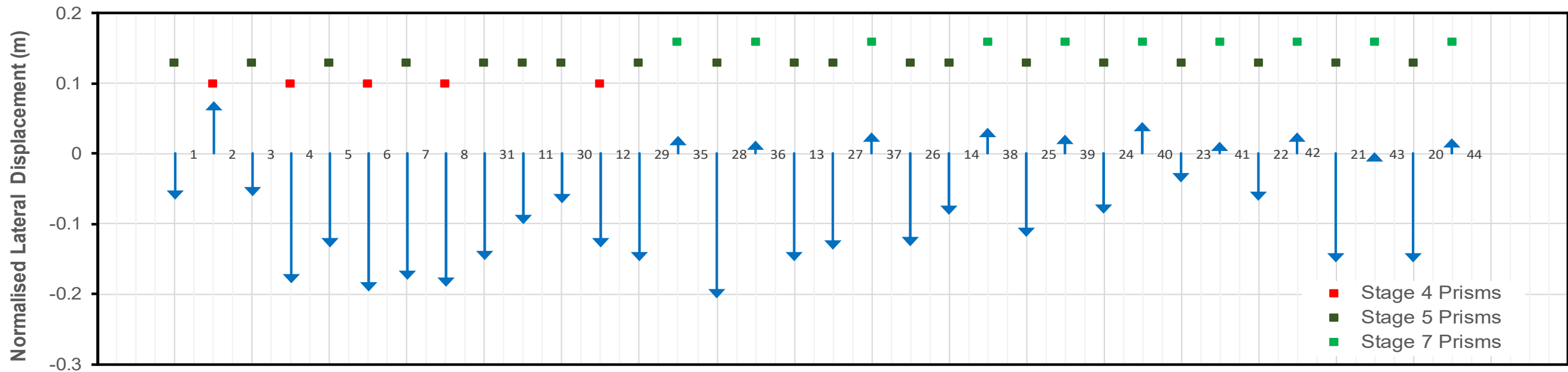
FIGURE 2

CLIENT No.	
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SHEET 1 OF 1	

No.	DESCRIPTION	DATE	DRAWN	CHECKD	APPRD
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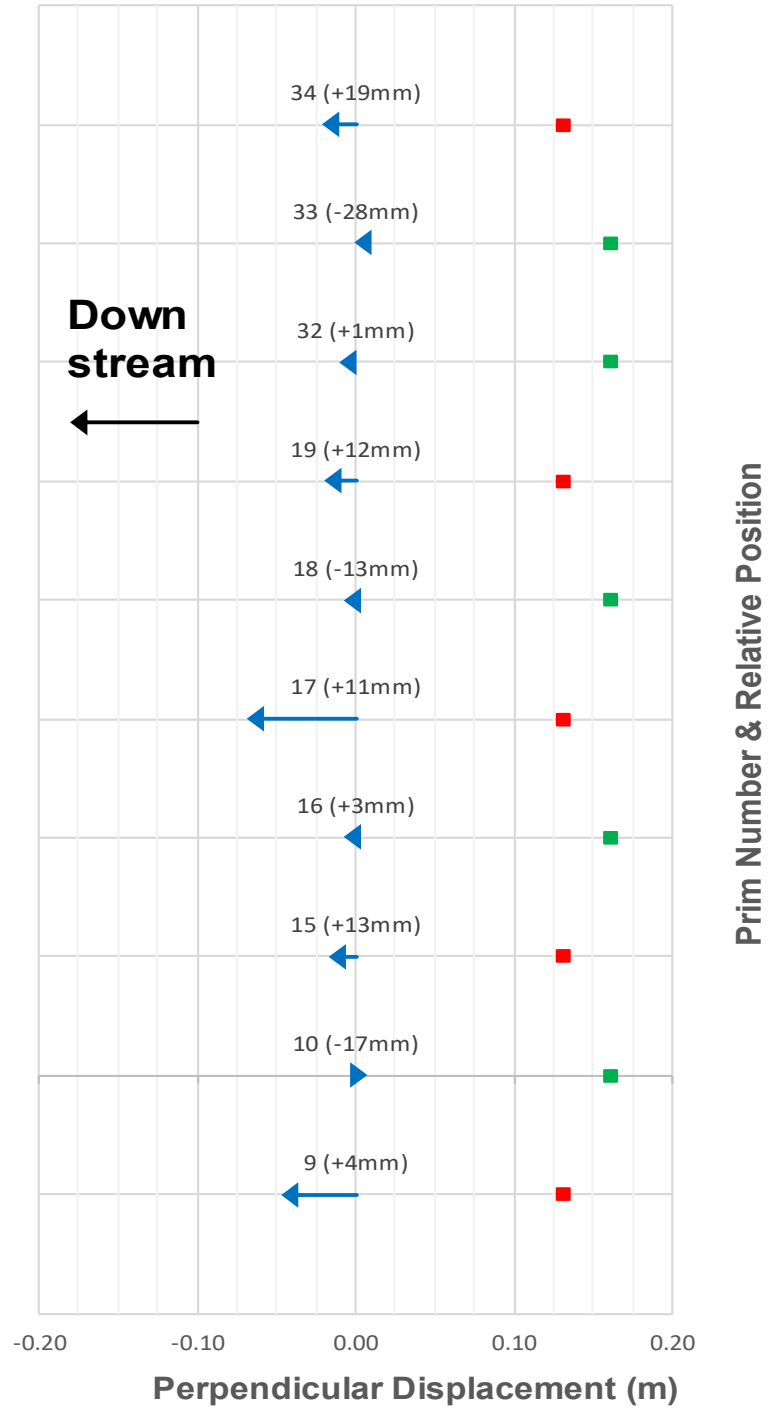
Job number	H356804			Report on NTSF Embankment Failure	Newcrest
Ref	H356804-00000-22A-230-0001-AP0B				Deformation Monitoring South Wall
By	TMY	IAG	15-Mar-19	Revision	
Revision	0				15-Mar-19



Relative Prism Position

Job number	H356804			Report on NTSF Embankment Failure	Newcrest
Ref	H356804-00000-22A-230-0001-AP0B				Cadia Valley Operations
By	TMY	IAG	15-Mar-19	Deformation Monitoring South Wall Normalized for Embankment Height	Annexure BD
Revision	0	15-Mar-19			

Perpendicular Displacement Vectors



Bracketted numbers indicate vertical displacement in mm

■ Stage 5 Prisms
■ Stage 7 Prisms



Location Plan
(Southern and Western Embankment Prism Locations)

Job number	H356804	Report on NTSF Embankment Failure	Newcrest
Ref	H356804-00000-22A-230-0001-APOB		Western Embankment Deformation Modelling
By	TMY IAG 15-Mar-19		Cadia Valley Operations
Revision	0 15-Mar-19		Annexure BD

Annexure BE

Otus SSM Report



Satellite Based Surface Movement Monitoring of Cadia Mine Facilities, AU

– Report –

Monitoring Period 01/2017 - 02/2018

**Version I 1.1
31 May 2018**

Airbus DS Geo Australia Pty Ltd

AIRBUS

Document Release

Project Manager:	Dr. Jan Anderssohn, Airbus	Signature:
Author:	Dr. Jan Anderssohn, Airbus	Signature:
Review:	Dr. Oliver Lang	Signature:
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Document Change Record

<i>Version</i>	<i>Date</i>	<i>Pages</i>	<i>Description</i>
11.0	31/05/2018	all	Document Set-Up and Issue

Report History

<i>Report</i>	<i>Monitoring Period</i>	<i>File Name</i>
-	-	-

CONTENT

1	EXECUTIVE SUMMARY	4
2	INTRODUCTION	6
3	TECHNICAL DETAILS	7
3.1	Data Acquisition and Selection	7
3.2	Surface Movement Monitoring (SMM)	9
3.2.1	<i>Processing Workflow</i>	9
3.2.2	<i>Satellite Viewing Geometry</i>	11
3.2.3	<i>Processing Conditions</i>	14
4	RESULTS AND ANALYSIS	17
4.1	Additional Analysis At The Tailing Dam	18
4.1.1	<i>Measurement Pixel Allocation to Prism Location</i>	20
4.2	Technical Constraints	24
4.3	Precision and Accuracy	25
4.4	Digital File Format	26
5	CONCLUSION.....	27
	ABBREVIATIONS.....	29
	REFERENCES.....	29
	LIST OF FIGURES.....	30
	LIST OF TABLES	31

1 EXECUTIVE SUMMARY

Otus Intelligence requested Airbus to derive surface movement measurements at the Cadia Mine facilities, Australia for the period January 2017 to February 2018. The area of interest (AOI) encompasses the surrounds of the Cadia Mine in central west New South Wales, approximately 250 kilometres west of Sydney. The overall processed area has a rural character with localized man-made mining infrastructure. The reason for the monitoring is a tailings pond dam failure on 9 March 2018. This study assesses the capability of SAR-Interferometry based on satellite data to derive precursors of dam collapse from space. The impact of movement to local infrastructure is intended to be monitored by satellite based Surface Movement Monitoring (SMM).

The applied SMM time series analysis technique, based on satellite observations, makes use of mid resolution data from the European satellite Sentinel-1 (S1) with the goal to derive surface movement measurements by exploiting satellite image data stacks interferometrically. The result is presented in the form of velocity movement maps, showing the average velocity per year of each measurement pixel identified within the area of interest (see Figure 1).

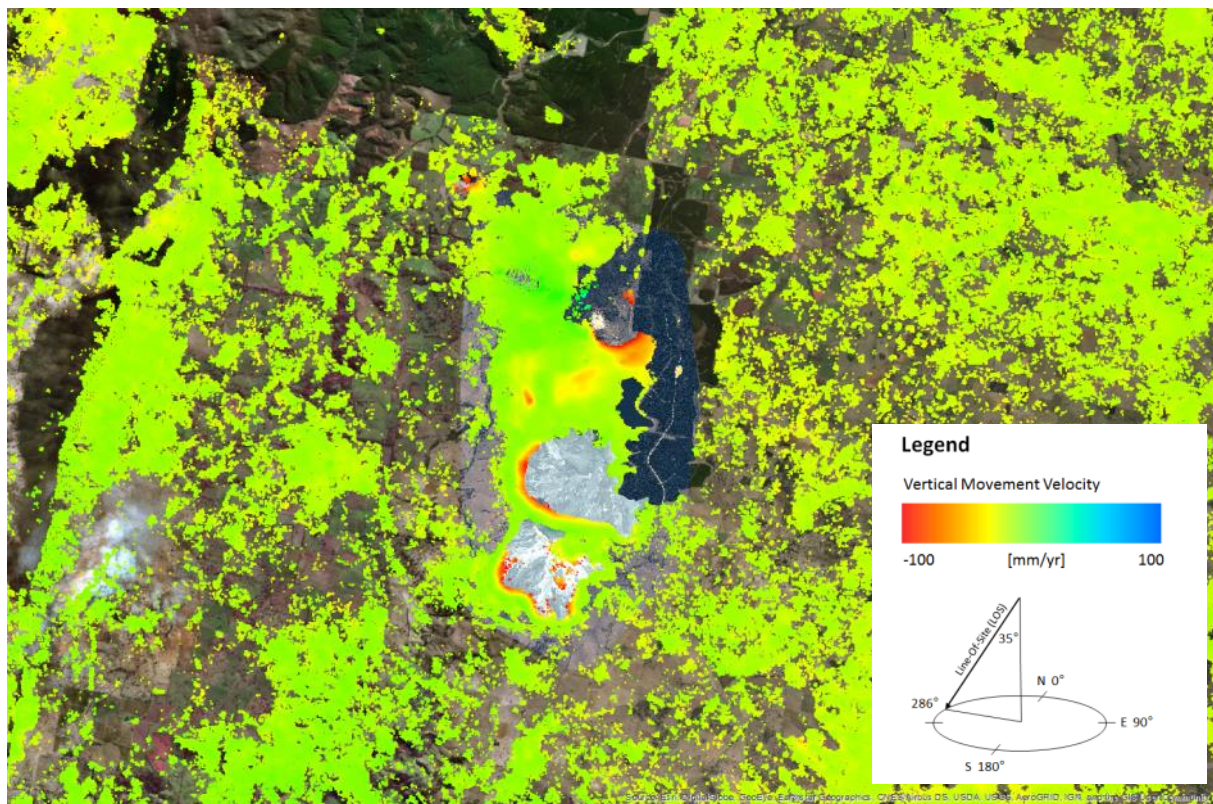


Figure 1: Surface movement map over Cadia Mine (01/2017 – 02/2018). © Airbus Defence and Space GmbH 2018.

The velocity map, as depicted in Figure 1 demonstrates, that measurements can be derived for a wide spread area consisting of a few settlements, mining infrastructure and sparsely vegetated to non-vegetated areas. In contrast, over regions of significant vegetation, such as cultivated agricultural fields and forests, no results could be derived.

The extended area of interest shows different zones of subsidence most likely triggered by mining activities. Since the location of the later breach of the dam is of main interest, this report focuses on dam deformation. The satellite based results reveal the usefulness of the technique as a precursor provision of a potential dam failure. At the end of 2017 the time series indicate an increase of subsidence. In addition to the averaged velocity, parameters like 'velocity' times 'deviation' product reveal dam zones at potential risk.

The comparison with local terrestrial measurements, undertaken by mining surveyors, suffers from a short temporal overlap between both measurement periods. The overall match between the two time series is good. Nevertheless, a reliable cross comparison cannot be provided due to the short overlap. Both measurements consistently show an ongoing deformation.

2 INTRODUCTION

Airbus Defence and Space was requested by Otus Intelligence to derive surface movement measurements from January 2017 to February 2018 over the Cadia Mine facilities, Australia. A tailings pond dam failed on 9 March 2018 and this study assesses the capability of Interferometry based on satellite synthetic aperture radar (SAR) data to derive precursors of the dam collapse.

This report presents the results of a SAR interferometric surface movement monitoring analysis using 35 Sentinel-1 Interferometric Wide-swath (IW) mode satellite scenes acquired in descending viewing orbit. The area of interest (AOI) covers the surrounds of the Cadia Mine in central west New South Wales, approximately 250 kilometres west of Sydney. It comprises approximately an area of 90 km² (see Figure 2).

The observation of such a large area by applying ground-based measurement is challenging and expensive. Remote sensing methods, in particular interferometric Surface Movement Monitoring (SMM), offer a very valuable supplement. In this case hundreds of square kilometres could be measured in one process.

Section 3 discusses the technical details regarding the production of the surface movement: 3.1 describes the Sentinel-1 satellite data acquisition and selection, 3.2 provides the description of the SMM time series analysis processing method which has been applied.

Section 4 presents a surface movement velocity map for the study area regarding the time period from January 2017 to February 2018. The chapter is complemented with a summary of the specific characteristic of the study area and a short discussion of the findings. Further a comparison with local terrestrial measurements is documented in 4.1.1.

Section 5 concludes this report in a short summary.

3 TECHNICAL DETAILS

3.1 DATA ACQUISITION AND SELECTION

The entire study area (see Figure 2) is covered by a single Interferometric Wide-swath (IW) footprint of the Sentinel-1 satellite acquired in descending orbit.



Figure 2: Area of interest over with respect to the Cadia Mine (blue polygon). Background image © Google Earth, 2018.

In total, 35 scenes in VV polarization were chosen covering the time interval 13/01/2017 – 25/02/2018 in descending orbit direction (Figure 3). Table 1 and Table 2 show more details about the used Sentinel-1 scenes and related acquisition parameters. All data were quality checked prior to their usage for processing. Precise orbit data with maximum accuracy were used for processing. For the interferometric Sentinel-1 IW data processing a multi-looking factor of 1 x 4 was applied and the final results were geocoded to a ground pixel sampling of 15 m x 15 m. Airbus' WorldDEM™ digital elevation model was used in order to improve the initial height estimate.

Table 1: Sentinel-1 IW scene dates used for the analysis

No.	Date	No.	Date
1	13/01/2017	19	17/08/2017
2	25/01/2017	20	29/08/2017
3	06/02/2017	21	10/09/2017
4	18/02/2017	22	22/09/2017
5	02/03/2017	23	04/10/2017
6	14/03/2017	24	16/10/2017
7	26/03/2017	25	28/10/2017
8	07/04/2017	26	09/11/2017
9	19/04/2017	27	21/11/2017
10	01/05/2017	28	03/12/2017
11	13/05/2017	29	15/12/2017
12	25/05/2017	30	27/12/2017
13	06/06/2017	31	08/01/2018
14	18/06/2017	32	20/01/2018
15	30/06/2017	33	01/02/2018
16	12/07/2017	34	13/02/2018
17	24/07/2017	35	25/02/2018
18	05/08/2017		

Table 2: Acquisition parameters

Orbit direction	Relative Orbit	Incidence Angle	Polarization	Mode
Descending	45	~35°	VV	IW

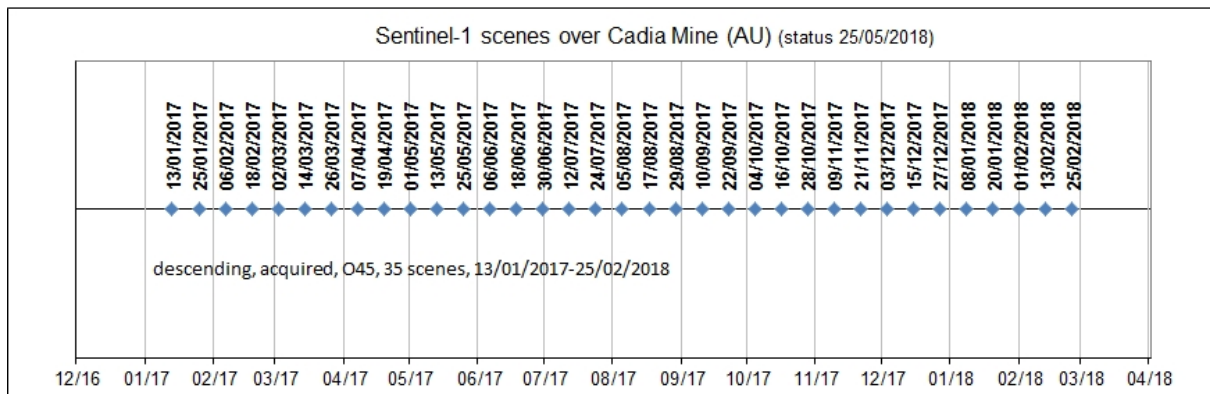


Figure 3: Scene time plot of the Sentinel-1 IW scenes (depicted as blue diamonds).

3.2 SURFACE MOVEMENT MONITORING (SMM)

Interferometric time series analysis exploits phase information of a number of Synthetic Aperture Radar (SAR) satellite data for the derivation of ground movement. A network of measurement pixels typically over wide areas is provided.

The Small Baseline (SBAS) method was chosen for the processing of the entire AOI. The SARscape software (Version No. 5.4) from SARMAP S.A. was used for processing.

The SBAS approach [Berardino et al. 2002] extends the technique presented in [Lundgren et al. 2001] and [Usai 2001] to the case of multiple short-baseline (SB) acquisitions via an easy and effective combination of all the available SB interferograms.

The term “small baselines” can be understood as small spatial (spatial separation between orbits) and temporal (time separation between acquisitions) baselines. The presented technique satisfies two key requirements: to increase the “temporal sampling rate” by using all the acquisitions included in the different SB interferograms and to preserve the capabilities of the system to provide spatially dense deformation maps, the latter being a key issue of conventional differential interferometry [Berardino et al. 2002]. Clearly, this latter requirement is related to the use of small baseline interferograms that limit the baseline decorrelation phenomena [Berardino et al. 2002].

For Sentinel-1 data an orbital tube width of ± 50 m is guaranteed and in combination with the C-band almost all Sentinel-1 data interferograms fulfil the requirement of small spatial baselines. Comparable to the Persistent Scatterer Interferometry (PSI) technique (e.g. [Ferretti et al. 2001]), an Atmospheric Phase Screen (APS) is employed and refined heights are estimated. The SBAS result can reveal non-linear surface movement time series and is suited for areas where short-term to mid-term interferograms display higher coherence than long-term one.

3.2.1 Processing Workflow

The workflow of Airbus' *surface movement monitoring* service consists of different processing sections and single steps. The interferometric processing core contains different time series analysis approaches (PSI, SBAS ...) to derive the optimum deformation estimation for different conditions at a given AOI. In this case, the SBAS method was chosen. The flowchart in Figure 4 gives an overview of the main sections and steps of the processing workflow.

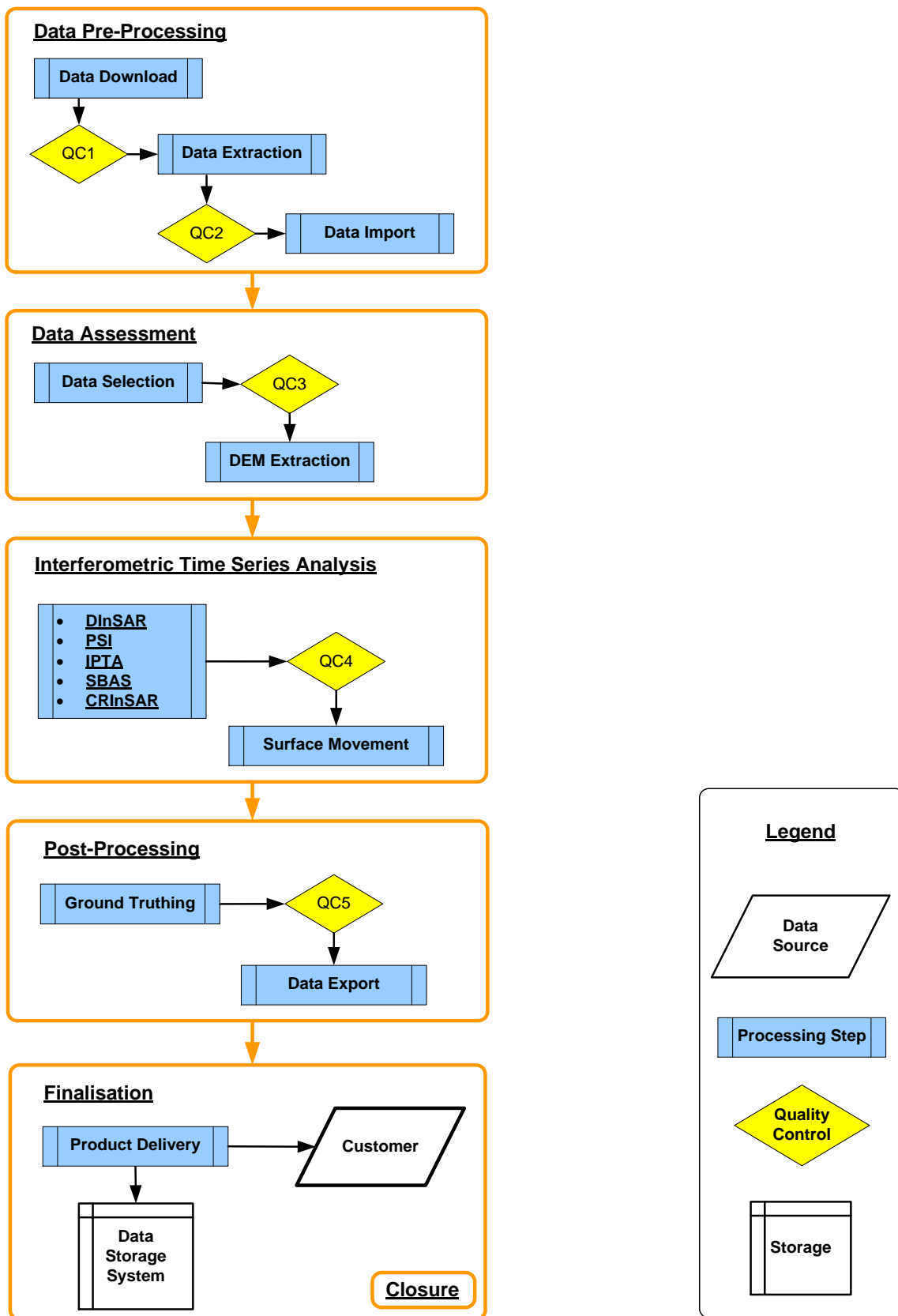


Figure 4: Generic surface movement monitoring workflow overview with quality control (QC) breakpoints in yellow. © Airbus Defence and Space GmbH 2017.

Primary input data sets are delivered by the Airbus Customer Service and downloaded via ftp server. After data extraction a data import into an internal software format is performed.

The data are then subject to a general assessment before processing. All useful data sets, which are free of errors, will subsequently be used for further processing. An external DEM is introduced into the workflow to correct the data for the topography.

After the pre-processing steps different interferometry approaches can be applied in order to derive ground deformation estimations. Depending on the given AOI conditions and specific interests the appropriate approach is selected.

After the Interferometry processing a post-processing is carried out to undertake a plausibility control and to integrate available ground truth information. As a last step in the post-processing the ground deformation analysis outcomes are exported to customer specific formats.

During the finalisation the *surface movement monitoring* outcomes are exported to customer-specific formats and delivered. Afterwards the project relevant data are prepared for a back-up system and finally stored. The processing workflow of *surface movement monitoring* processing is accompanied by a quality control (QC) procedure to ensure a high quality of the product.

3.2.2 Satellite Viewing Geometry

It has to be considered in the interpretation of the surface movement results that measurements are conducted within the one dimensional (1D) SAR-satellite viewing geometry: The real three dimensional (3D) surface movement phenomena are being projected into a 1D measurement into the satellite line-of-sight (LOS). A decomposition of the 1D LOS measurement into a 3D surface movement is generally not possible using only one satellite viewing geometry. Figure 5 depicts the viewing geometry in terms of a potential subsidence bowl in the study area.

The satellite measures the surface movement in line-of-sight of satellite (M_{LOS}) under a certain incidence angle. A 3D movement phenomenon is characterized by a combination of horizontal and vertical surface movement components. As shown in Figure 5, while having almost comparable vertical movement components (subsidence) values, the horizontal surface movement components for an exemplary subsidence bowl yield different amounts of M_{LOS} depending on their location on the bowl. Therefore, a simple transformation of LOS measurements into the vertical direction under the assumption of only vertical surface

movement introduces a certain inaccuracy. This issue cannot be overcome without any additional information about the real three dimensional surface movement components.

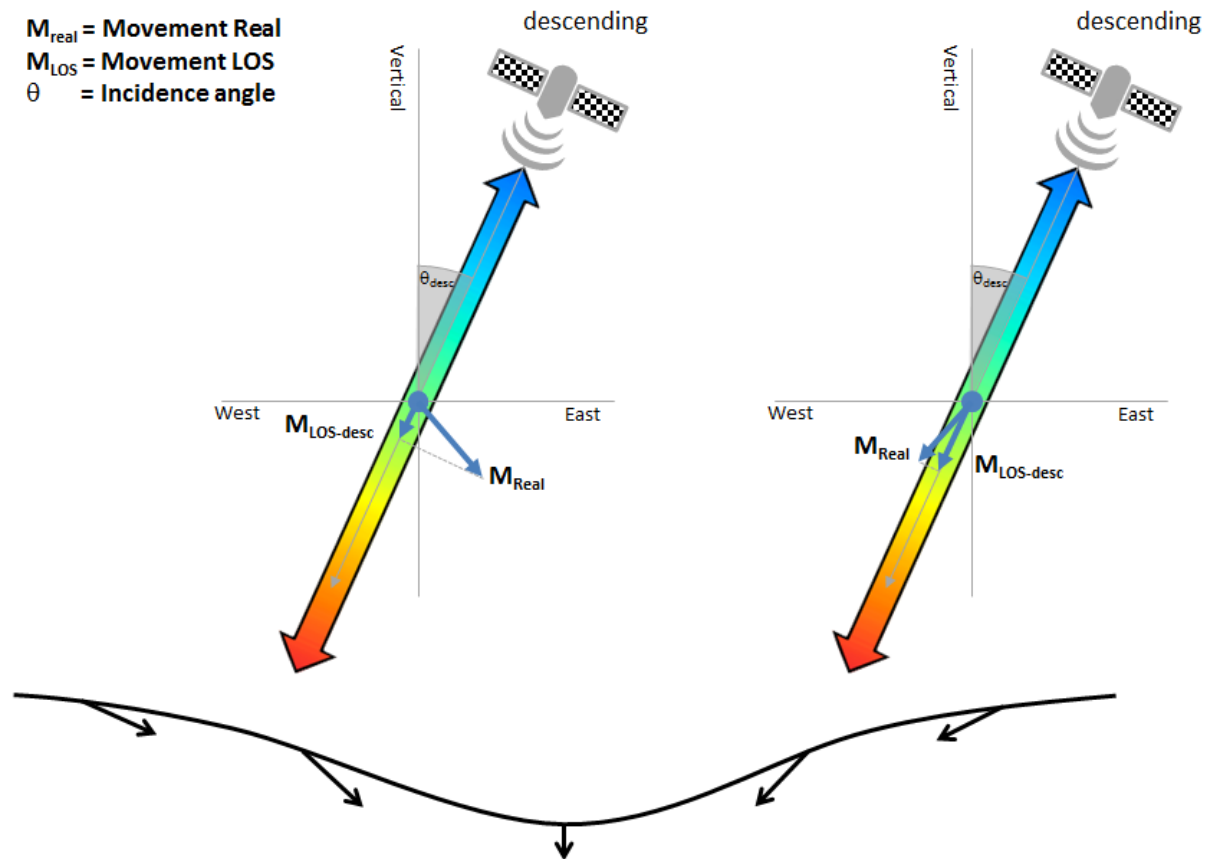


Figure 5: Exemplary descending viewing geometry for a subsidence bowl. The colour coding of arrows (line-of-sight of satellite) corresponds to the one of the velocity maps. © Airbus Defence and Space GmbH 2018.

One technical way to overcome this issue is the exploitation of two viewing geometries, i.e. ascending and descending, as depicted in Figure 6 and Figure 7. The merge of the ascending and descending M_{LOS} allows a decomposition of the real 3D surface movement into two components: vertical (M_V) and almost East-West directed horizontal (M_{EW}). The North-South component cannot be derived due to the polar orbit of SAR-satellites and the related viewing geometry.

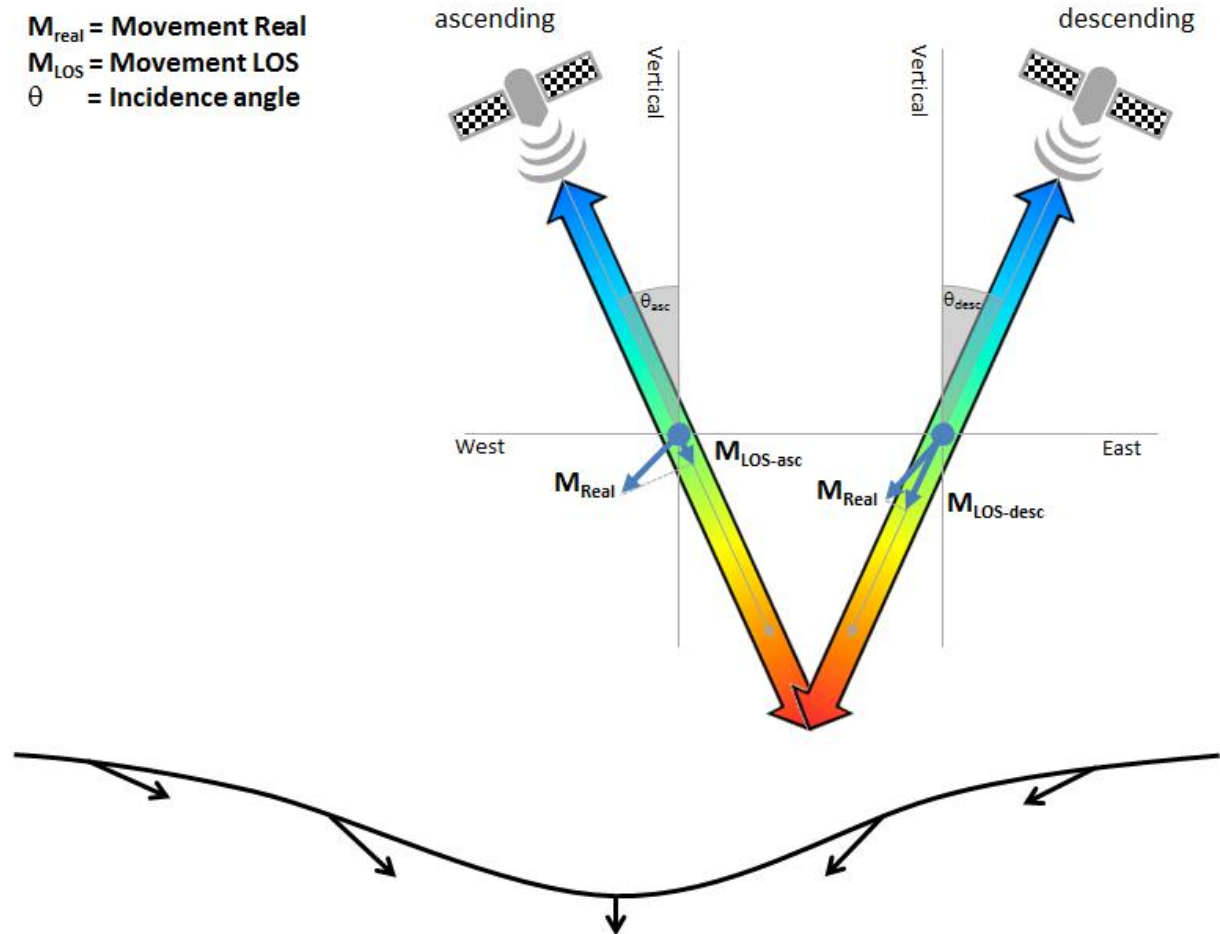


Figure 6: Ascending and descending viewing geometry in terms of an exemplary vertical and horizontal composed surface movement. The colour coding of arrows (line-of-sight of satellite) corresponds to the one of the velocity maps. © Airbus Defence and Space GmbH 2018.

Based on this technical explanation it is worthwhile to consider the acquisition of both ascending and descending viewing geometries over the study area to have movement estimates in both vertical and horizontal directions.

M_{real} = Movement Real
 M_{LOS} = Movement LOS
 θ = Incidence angle
 M_{EW} = Movement East-West
 M_{V} = Movement Vertical

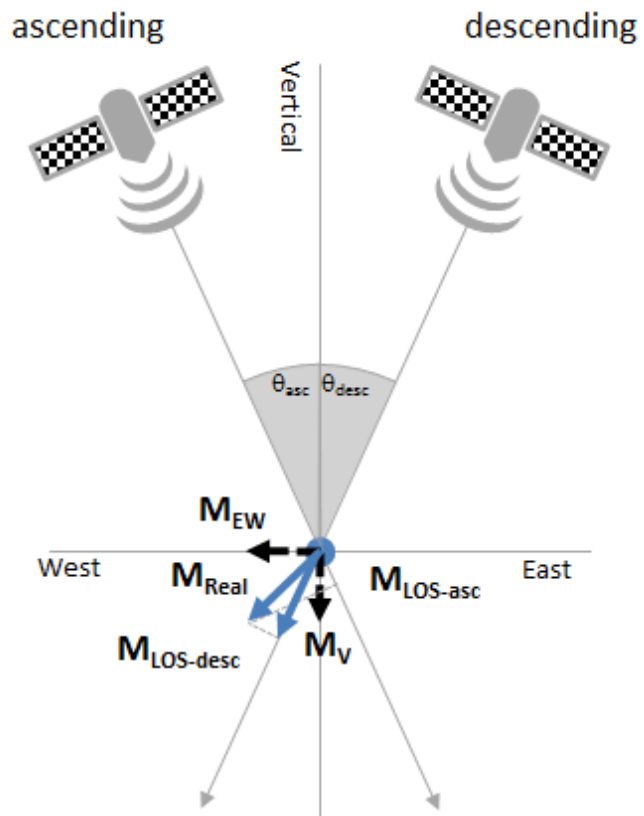


Figure 7: Sketch of ascending and descending merge that allows a decomposition of the real three dimensional surface movement into two components. © Airbus Defence and Space GmbH 2018.

3.2.3 Processing Conditions

The processing conditions for an interferometric time series analysis, based on the given satellite data and the interferometric characteristics (coherence) in the study area, can be qualified as good. The sparse vegetation of the surroundings of the Cadia Mine and the man-made mining infrastructure provides generally sufficient measurement pixels. The width of the tailings pond dam of few hundred meters is covered by a number of measurement pixels, sufficient for a detailed analysis.

General comment: Dams of a smaller scale may need higher resolution data for a sophisticated analysis. Generally, in areas with rural conditions (dense vegetation on agricultural land and grasslands, forests and plantations) no SMM results can be derived. Varying reflection situations over time prevent the interferometric analysis in this case (temporal decorrelation). In contrast, at naturally sparsely vegetated areas and rock

formations for example at mountain ridges or at bare ground areas (e.g. mining sites) surface movement monitoring may be applicable.

The Sentinel-1 scenes have been acquired within the nominal revisit time interval of 12 days. The scenes acquired at March 09 has not been used, because strong changes at the dam due to the dam failure. Such changes cause almost a total loss of measurement pixels at such a location.

A multi-looking factor of 1 x 4 has been selected in order to prepare the data for SBAS processing. Therefore, the intermediate processing results have a ground resolution of approximately 15 m x 15 m.

Table 3 provides a statistic summary of the interferometric-pairs network. Absolute (geometric) baselines have been limited by a threshold to avoid geometric decorrelation. The maximum temporal baseline has been limited to 60 days due to expectable strong movement previous the dam failure. Temporal decorrelation and loss of coherence accordingly, is therefore reduced.

Table 3: Interferometric network statistic.

Number of Input Images	Number of Interferograms	Minimum Absolute Baseline [m]	Maximum Absolute Baseline [m]	Minimum Temporal Separation [d]	Maximum Temporal Separation [d]
35	165	3	217	12	60

Based on the used Sentinel-1 IW scenes the following SBAS network connections result (see Figure 8). Each connection between two points represents one interferogram. Not all possible connections have been admitted. Some pairings have been rejected exceeding the threshold for absolute baselines. This time-position plot shows a normally distributed connection graph.

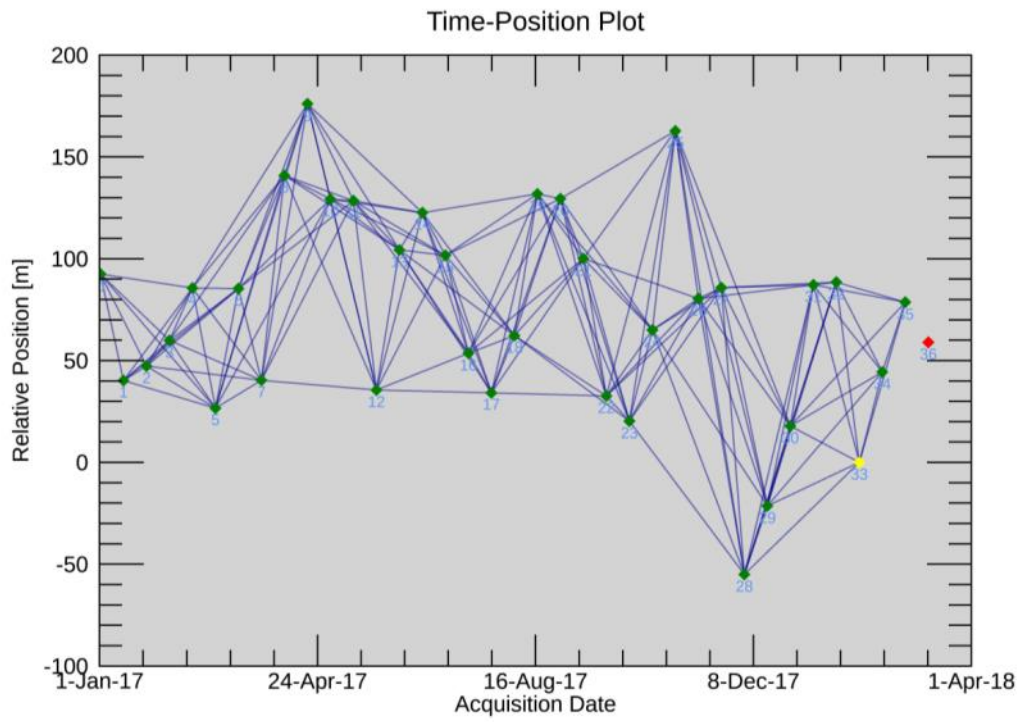


Figure 8: Time-Position plot of SBAS connection graph for the used scenes.

4 RESULTS AND ANALYSIS

The result of processing, as shown in Figure 9, is delivered as average velocity map in PDF file format in high resolution. A high overall measurement pixel density has been derived over regions with sparsely or dry vegetation, mining facilities and rock bare earth. Known constraints are present in areas with dense vegetation and farming, where no measurement pixels exist.

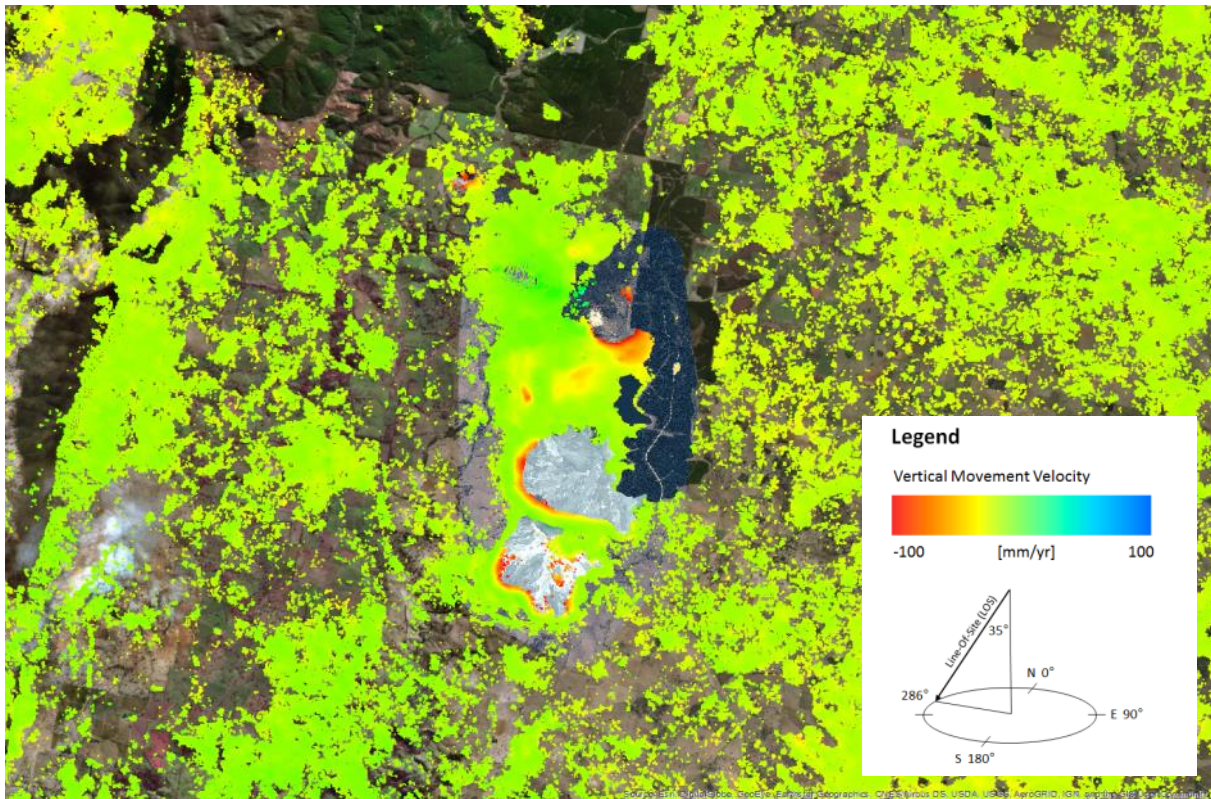


Figure 9: Surface movement map over Cadia Mine region (13/01/2017 – 25/02/2018). © Airbus Defence and Space GmbH 2018.

The movement values are measured in line-of-sight of the satellite and are projected into vertical direction in this map. The scenes have been acquired in descending geometry and the satellite looks roughly from East to West with an incidence angle of 35° (see legend of Figure 9). Due to this looking direction, movement measurements at hillslopes have to be interpreted with care (see also section 3.2.2).

Please note that the velocity values provided within the digital maps represent the average movement per year. As a consequence, velocities are provided in the unit 'millimetre per year'.

Table 4: Characteristics of results

Measurement Unit	Velocity in [mm/year]
Monitoring Period	13/01/2017 – 25/02/2018
Average Velocity	-3 mm/yr
Maximum Subsidence Velocity	-330 mm/yr
Maximum Uplift Velocity	+49 mm/yr
Measurement Pixel Density	1260 pixel/km ²

This SMM analysis provides a good overall coverage with measurement pixels. Only agricultural cultivated fields, water bodies and forest provide no measurements. Table 4 provides the characteristics of results.

The surface movement situation within the area of interest exhibits regional subsidence effects; especially over the (southern) tailings ponds and East of the open pit (see orange/red areas in Figure 9). Most of the elongated subsidence pattern parallel to the tailings pond dams is related to a drying process and the related shrinking of the tailings material. The dominating pattern of subsidence at the North cannot be interpreted because of missing additional information. Some small-scale subsidence patterns are linked to dumpsites. They likely indicate a typical compaction process of the dumped material.

Further detailed analysis of surface movement process at the breached tailings pond dam is given in the following section.

4.1 ADDITIONAL ANALYSIS AT THE TAILING DAM

Airbus has been asked to conduct additional analysis at the breached tailings pond dam. This section summarizes the results. In order to distinguish between adjacent measurement pixels representing shrinking tailings material inside the pond and the dam itself a mask has been defined as provided on the left-hand side of Figure 10. Centre lines along *Toe1*, *Stage7* and *Stage10* were provided by the client to be further analysed. Along these polylines the SMM measurement pixels within a search radius of 20 m were allocated and averaged to a single value. Figure 10 shows the surface movement map masked out for the dam and the associated averaged values along the polylines. Clearly visible are subsidence zones in the North and in the South, representing the location where the dam recently breached.

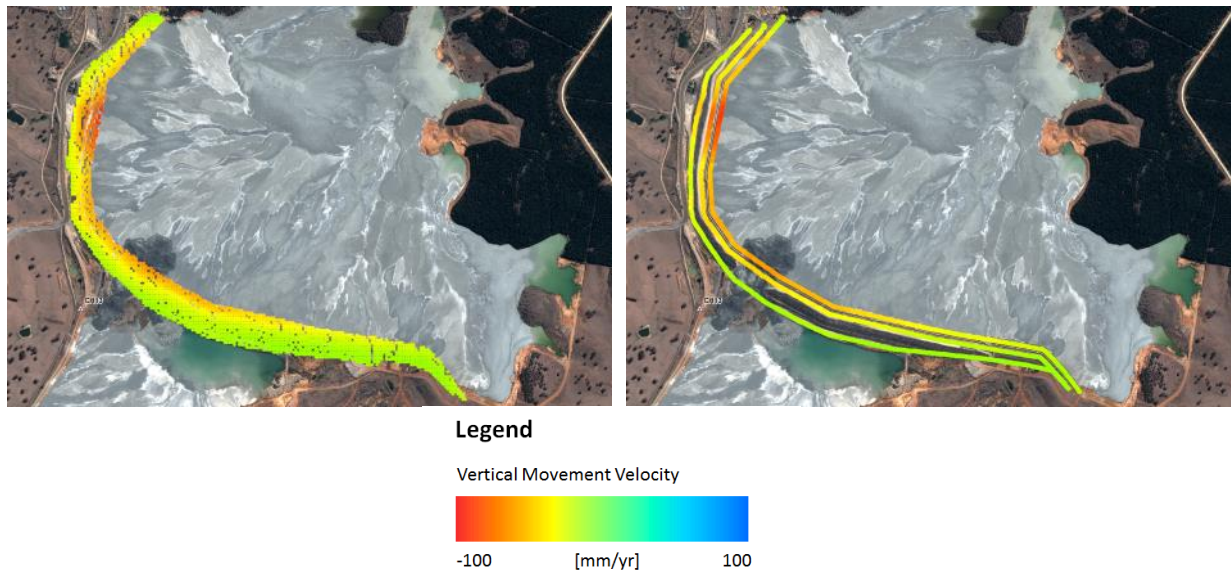


Figure 10: **Left:** Surface movement map (13/01/2017 – 25/02/2018) masked for the dam area only. **Right:** Allocated and averaged values along the polylines. Background Pléiades scene acquired 17/03/2018. © Airbus Defence and Space GmbH 2018.

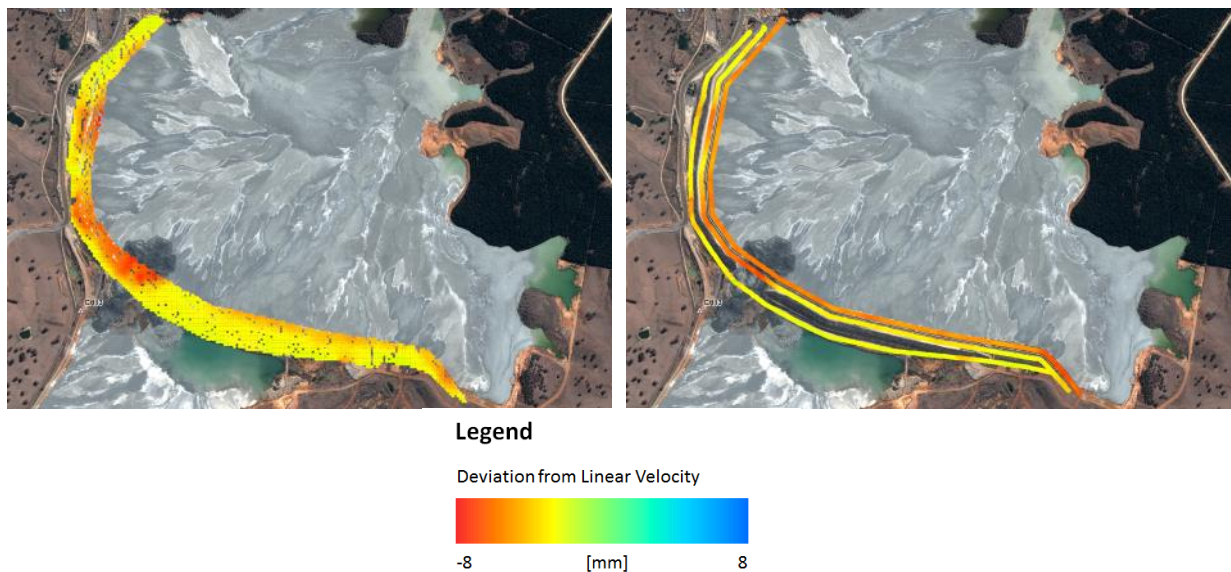


Figure 11: **Left:** Temporal deviation map (13/01/2017 – 25/02/2018). **Right:** Allocated and averaged values along the polylines. Background Pléiades scene acquired 17/03/2018. © Airbus Defence and Space GmbH 2018.

The analysis of the average velocity map (Figure 10) alone does not lead to a complete understanding of a critical deformation process. Therefore, an additional parameter has been extracted: The temporal deviation of the displacement values. These highlight those zones where the displacement values vary in time. This can be - among others - caused by accelerated movement behaviour. Figure 11 shows high deviation values (red zones) exactly at

the location where the dam failed in March 2018 and at a second location further north. In other words: the averaged velocity values may not necessarily reveal critical deformation behaviour in time. Hence, other parameters should be used for further analysis, like the deviation from linearity.

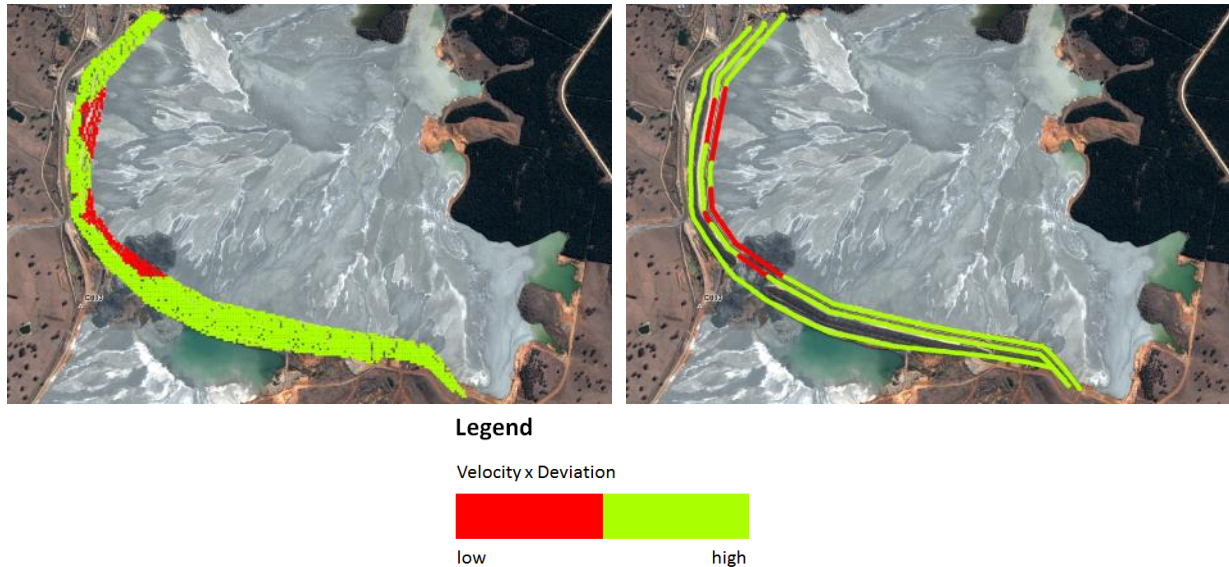


Figure 12: Left: Illustration of Velocity x Deviation product map (13/01/2017 – 25/02/2018). **Right:** Allocated and averaged values along the polylines. Background Pléiades scene taken 17/03/2018. © Airbus Defence and Space GmbH 2018.

As written above, large deviation values may be among others caused by movement changes in time. Such deviation values can be caused by several reasons, for example by strong varying (noise) time series at low level of velocity. To highlight areas characterized by strong movement (critical movement) and strong temporal movement deviation (acceleration) Figure 12 shows an illustration of the mathematical product of ‘velocity’ and ‘deviation’. This map classifies in red where strong and temporally variable deformation occurred. The zone of dam failure is clearly correlated. Other “red” zones further north are identified in addition.

4.1.1 Measurement Pixel Allocation to Prism Location

A comparison of SMM result with terrestrial survey measurements has been requested by the customer. Terrestrial survey measurements taken at prism locations called “NTSF” have been provided to Airbus. Figure 13 shows the location of prisms as well as the location of an observation pillar called CH13. Around the prism locations a buffer of 15 m radius was defined in order to allocate SMM measurement pixels lying inside this buffer. The averaged velocity of allocated measurement pixels per buffer is represented by a colour coded circles

in Figure 13. The related time series of this averaged SMM measurements are being used for further analysis.

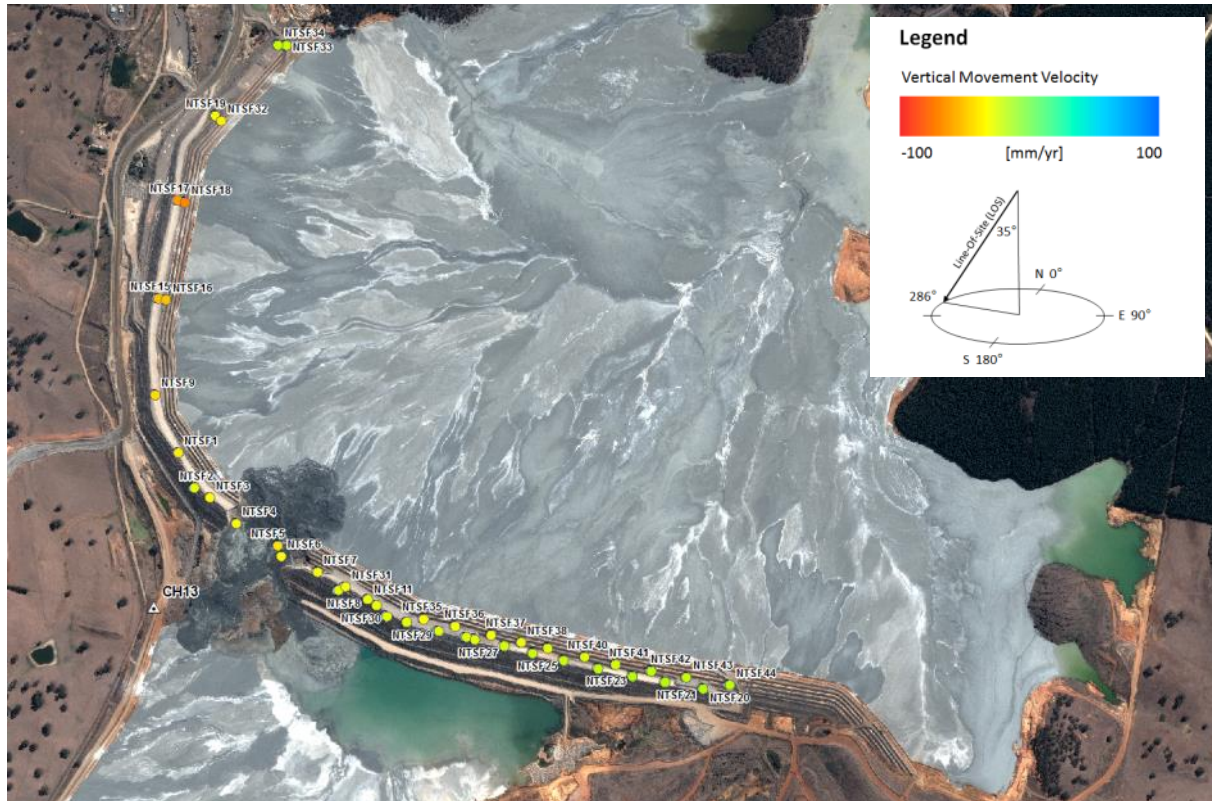


Figure 13: Allocated and averaged SMM velocity at prism locations (13/01/2017 – 25/02/2018). Background Pléiades scene taken 17/03/2018. © Airbus Defence and Space GmbH 2018.

To compare such different measurements techniques (space borne and terrestrial) several preparation steps have to be undertaken:

- 3D terrestrial deformation values have to be converted into satellite's line-of-sight (LOS). Considering satellite's incidence and azimuth angle the 3D terrestrial deformation can be transformed into the 1D-LOS.
- Secondly the different references of the relative measurements have to be taken into account. The terrestrial measurements have been observed with respect to *CH13*, whereas the SMM results are referred to alternative reference points. Hence the SMM time series of *CH13* has been subtracted from all SMM time series of all SMM measurement pixels. This yield a zero displacement at the *CH13* and an overall SMM result with reference to *CH13*. That means both measurements, space borne and terrestrial, refer to the same reference point namely *CH13*.

The prisms called *NTSF4*, *NTSF5* and *NTSF6* are located over the failure zone of the dam and therefore these are of special interest for further analysis. Figure 14, Figure 15 and Figure 16 show joined time series at these prism locations. Space borne and the terrestrial derived time series have a different character in terms of measurement noise. It seems the terrestrial survey results are characterized by higher variation in time (noise). The terrestrial observation is characterized by a lower measurement repeat frequency than the space borne data. Unfortunately, the terrestrial and space borne observations just have few temporally overlapping observations. Hence, it is difficult to correlate both. Nevertheless, all three joined time series show an overall match in terms of an ongoing subsidence during the observation period. The SMM measurements reveal a movement acceleration starting at the end of 2017. Such a sudden increase could potentially act as an indicator for a dam failure and may have been used as an alert indicator prior to the breach.

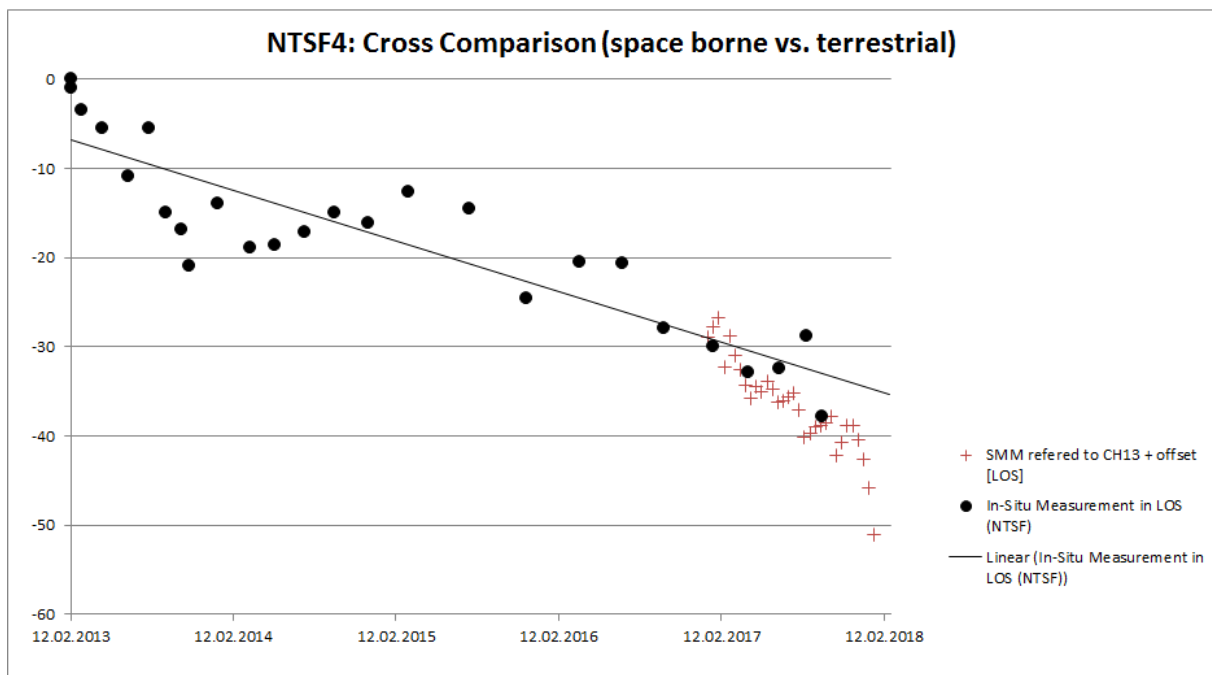


Figure 14: Plot of terrestrial survey results and allocated and averaged SMM velocity at prism NTSF4.

The relatively low measurement repeat frequency of the terrestrial surveying limits its capacity to act as an alert indicator. Sentinel-1 acquires scenes within a 12-day revisit interval, resulting in a significantly higher measurement frequency. Consequently, the surface movement monitoring technique can provide valuable input into an early warning system.

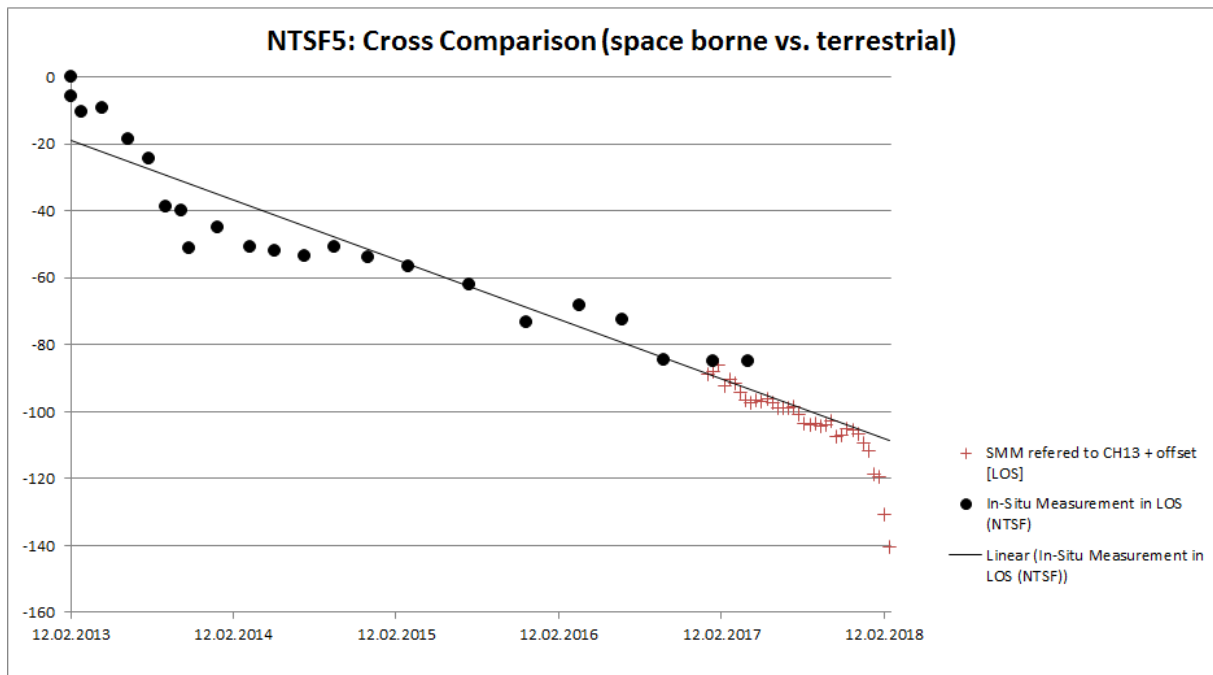


Figure 15: Plot of terrestrial survey results and allocated and averaged SMM velocity at prism NTSF5.

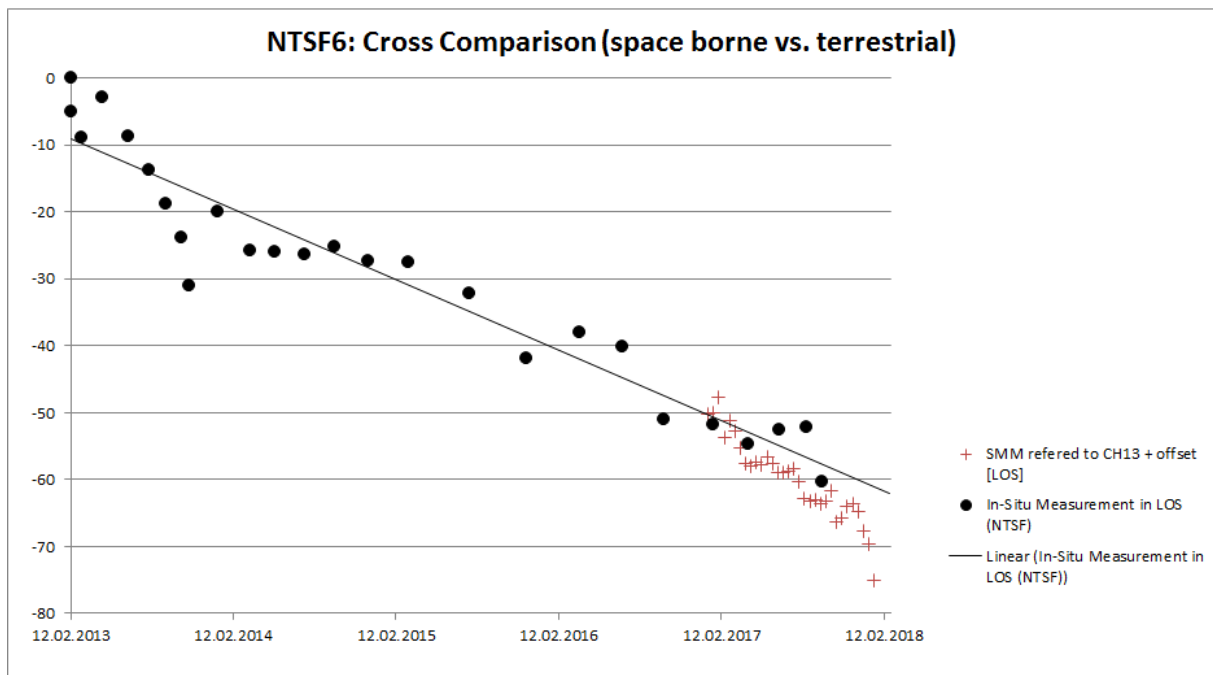


Figure 16: Plot of terrestrial survey results and allocated and averaged SMM velocity at prism NTSF6.

4.2 TECHNICAL CONSTRAINTS

For this proof-of-concept study Sentinel-1 descending scenes are used. The complex 3D deformation of the tailing dam is measured along the 1D-LOS direction (compare section 3.2.2). This circumstance has to be taken into account while analysing and comparing SMM results with alternative observations. Currently, Sentinel-1 scenes are being acquired in descending viewing geometry, only. No ascending Sentinel-1 scene acquisitions are currently available. The use of the high resolution TerraSAR-X and PAZ satellite constellation could overcome this constrain. Based on this constellation ascending and descending viewing geometry scenes could be acquired and jointly analysed. This would lead to a two-dimensional SMM result, providing vertical and east-west directed movement components.

4.3 PRECISION AND ACCURACY

The interferometric surface movement monitoring estimates generally an average annual (linear) movement rate, the velocity, from a number of time series points. The theoretical precision, i.e. the repeatability of measurements, of the velocity can be given to be about 1 - 2 mm per year, the precision of individual time series points about 3 - 5 mm (using Sentinel-1 data). For each measurement pixel, the precision of the velocity value is estimated and given in the column 'V_Precision' in the digital results file (see Table 5 below).

The accuracy, i.e. the difference between measurements and truth, of a time series point depends on several measurement parameters, such as data availability, surface signal reflection properties, temporal and spatial movement characteristics, processing method and the assumed stable reference points.

The accuracy can be estimated in comparison to true measurements, e.g. resulting from a terrestrial levelling campaign. However, under optimal conditions, e.g. in an urban environment, it can be shown that an accuracy of about 5 mm for a time series point (using Sentinel-1 data) can be achieved. Under 'normal' measurement conditions, the technique is estimated to produce an accuracy of about 1 cm for a time series point.

That is, in this result, only the precision (and not the accuracy) is given for the pixels' velocity: velocity \pm precision (e.g. -12.9 ± 1.6 mm/year).

SAR interferometry (InSAR) is a relative surveying method and its results have to be referenced to reference points (RPs) in order to derive absolute measurements. The RPs themselves are assumed to be stable. All other measurement pixels represent the movement values with respect to these defined zero movement locations. Actually one RP would be enough to refer the InSAR result, but from a technical point of view it is advisable to define several RPs within a small area of zero movement. Please refer to [Casu et al. 2006] regarding the theoretical increase of standard deviation, depending on the distance to the RPs, for single measurements within a time series: The longer the distance to the RP, the smaller the precision of a measurement.

The less the number of satellite scenes the less the accuracy: Normally millimetre accuracy can be expected when using more than 30 scenes.

4.4 DIGITAL FILE FORMAT

The interferometric time series results have been made available as a digital ESRI shape file (SHP format), where necessary due to file size limitations, delivered in a file geodatabase (GDB) format. Table 5 summarizes the information provided within the digital file.

Table 5: Columns and their Description of the Digital ESRI Shape File.

Fieldname	Data Type	Description
FID	Object ID	Identification number of shape file feature
Shape	Point	Geometry of shape file
X	Double	Location of the feature: East value using the reference UTM zone [m]
Y	Double	Location of the feature: North value using the reference UTM zone [m]
Velocity	Double	Average surface movement velocity [mm/year]: <ul style="list-style-type: none"> - Positive value: movement of measurement point toward the sensor, uplift of measurement point in vertical result, eastward movement of measurement point for horizontal result - Negative value: movement of measurement point from the sensor away, subsidence of measurement point in vertical result, westward movement of measurement point for horizontal result
V_Precision	Double	Precision corresponding to the average surface movement velocity at the measurement pixel [mm/year]
Deviation	Double	Mean deviation of displacement values to the average surface movement velocity at the measurement pixel [mm]
LOS_In	Double	Line-Of-Sight of satellite (incidence angle) [°]
D_YYYYMMDD	Double	Vertical surface movement at acquisition date YYYYMMDD (Y=Year, M=Month, D=Day) referred to the first acquisition date [mm] (displacement time series) <ul style="list-style-type: none"> - Positive value and negative value interpretation: see Velocity

5 CONCLUSION

This report presents interferometric surface movement monitoring results over Cadia Mine facilities, Australia conducted for the time period January 2017 to February 2018. 35 Sentinel-1IW scenes were used for the study which focusses on a 90 km² area of interest around Cadia Mine in central west New South Wales. Space borne surface movement results were being compared with local terrestrial measurements.

From a technical point of view a high number of interferometric measurement pixels were derived covering well the tailings pond dam and surrounding surfaces with sparsely vegetation during the observation period. Due to the relatively large size (width) of the dam compared to the satellite image resolution (15 m) Sentinel-1 turned out to provide a sufficient measuring point density. Higher resolution satellite scenes like TerraSAR-X are able to deliver even more measurement pixels and subsequently a more detailed situational picture.

Densely vegetated regions, like cultivated land and forests, yield no measurements due to permanent change of surface conditions and radar reflection properties.

The extended larger area of interest shows different zones of subsidence most likely triggered by mining activities. Satellite based monitoring is consequently a suitable approach ensuring a complete coverage of the area to monitor. A continuous understanding of surface movements supports a safe operation of the mining activities.

Since the location of the recent breach of the dam is of main interest, this report focuses on the dam's deformation. Based on the SMM results it can be concluded that this technique could have been used for the provision of an alert indicator with respect to the dam failure in March 2018. Generally, the surface movement monitoring approach can be used as valuable additional information of ongoing deformation processes at critical infrastructures. In addition to the averaged velocity further parameters like 'velocity' times 'deviation' were derived which highlight zones at potential risk. In combination with additional ground based information coming from the mine operation those results could be used to reduce future risks.

The comparison with local terrestrial measurements suffers from a short-term overlap between both measurement periods. The overall match between both time series is good. Nevertheless, a reliable cross comparison cannot be provided. Both measurements show consistently an ongoing deformation. The SMM results reveal an accelerating subsidence starting at the end of 2017.

Future SMM analysis of this site is advisable in order to understand better the ongoing deformation processes. SMM results in a pixel density and measurement frequency significantly higher than the existing terrestrial survey.

ABBREVIATIONS

2D	Two-Dimensional
3D	Three-Dimensional
AOI	Area of Interest
APS	Atmospheric Phase Screen
DEM	Digital Elevation Model
IW	Image Wide-swath
InSAR	SAR Interferometry
LOS	Satellite Line-Of-Sight (Looking Direction of the Satellite Sensor)
M_{EW}	Surface Movement in Horizontal East-West Direction
M_{LOS}	Surface Movement in LOS
M_V	Surface Movement in Vertical Direction
QC	Quality Analysis
RP	Reference Point
S1	Sentinel-1
SAR	Synthetic Aperture Radar
SB	Short-Baseline
SBAS	Small Baseline approach
SM	StripMap Mode
SMM	Surface Movement Monitoring
VV	Vertical Transmit – Vertical Receive Polarisation

REFERENCES

Berardino, P.; Fornaro, G.; Lanari, R. & Sansosti, E. (2002). A New Algorithm for Surface Deformation Monitoring Based on Small Baseline Differential SAR Interferograms, IEEE Transaction on Geoscience and Remote Sensing, 40, 2375-2383.

Casu, F.; Manzo, M. & Lanari, R. (2006) A quantitative assessment of the SBAS algorithm performance for surface deformation retrieval from DInSAR data Remote Sensing of Environment, 102, 195-210.

Ferretti, A.; Prati, C. & Rocca, F. (2001) Permanent Scatterers in SAR Interferometry, IEEE Transaction on Geoscience and Remote Sensing, 39, 8-20.

Lundgren, P.; Usai, S.; Sansosti, R.; Lanari, R.; Tesauro, M.; Fornaro, G. & Berardino, P. (2001). Modeling surface deformation observed with SAR Interferometry at Campeii Flegrei Caldera, J. Geophysical. Res., 106, 19355-19367.

Usai, S. (2001). A new approach for long term monitoring of deformation by differential SAR interferometry, PhD thesis, Delft Univ. Press, Delft, The Netherlands.

LIST OF FIGURES

<i>Figure 1: Surface movement map over Cadia Mine (01/2017 – 02/2018). © Airbus Defence and Space GmbH 2018.</i>	4
<i>Figure 2: Area of interest over with respect to the Cadia Mine (blue polygon). Background image © Google Earth, 2018.</i>	7
<i>Figure 3: Scene time plot of the Sentinel-1 IW scenes (depicted as blue diamonds).</i>	8
<i>Figure 4: Generic surface movement monitoring workflow overview with quality control (QC) breakpoints in yellow. © Airbus Defence and Space GmbH 2017.</i>	10
<i>Figure 5: Exemplary descending viewing geometry for a subsidence bowl. The colour coding of arrows (line-of-sight of satellite) corresponds to the one of the velocity maps. © Airbus Defence and Space GmbH 2018.</i>	12
<i>Figure 6: Ascending and descending viewing geometry in terms of an exemplary vertical and horizontal composed surface movement. The colour coding of arrows (line-of-sight of satellite) corresponds to the one of the velocity maps. © Airbus Defence and Space GmbH 2018.</i>	13
<i>Figure 7: Sketch of ascending and descending merge that allows a decomposition of the real three dimensional surface movement into two components. © Airbus Defence and Space GmbH 2018.</i>	14
<i>Figure 8: Time-Position plot of SBAS connection graph for the used scenes.</i>	16
<i>Figure 9: Surface movement map over Cada Mine region (13/01/2017 – 25/02/2018). © Airbus Defence and Space GmbH 2018.</i>	17
<i>Figure 10: Left: Surface movement map (13/01/2017 – 25/02/2018) masked for the dam area only. Right: Allocated and averaged values along the polylines. Background Pléiades scene acquired 17/03/2018. © Airbus Defence and Space GmbH 2018.</i>	19
<i>Figure 11: Left: Temporal deviation map (13/01/2017 – 25/02/2018). Right: Allocated and averaged values along the polylines. Background Pléiades scene acquired 17/03/2018. © Airbus Defence and Space GmbH 2018.</i>	19
<i>Figure 12: Left: Illustration of Velocity x Deviation product map (13/01/2017 – 25/02/2018). Right: Allocated and averaged values along the polylines. Background Pléiades scene taken 17/03/2018. © Airbus Defence and Space GmbH 2018.</i>	20
<i>Figure 13: Allocated and averaged SMM velocity at prism locations (13/01/2017 – 25/02/2018). Background Pléiades scene taken 17/03/2018. © Airbus Defence and Space GmbH 2018.</i>	21
<i>Figure 14: Plot of terrestrial survey results and allocated and averaged SMM velocity at prism NTSF4.</i>	22
<i>Figure 15: Plot of terrestrial survey results and allocated and averaged SMM velocity at prism NTSF5.</i>	23

Figure 16: Plot of terrestrial survey results and allocated and averaged SMM velocity at prism
NTSF6.....23

LIST OF TABLES

Table 1: Sentinel-1 IW scene dates used for the analysis..... 8
Table 2: Acquisition parameters 8
Table 3: Interferometric network statistic.15
Table 4: Characteristics of results18
Table 5: Columns and their Description of the Digital ESRI Shape File.26

Annexure BF

ATCW 9 March 2018 Inspection Report

Memorandum - Privileged and Confidential

To: Bob Thiele (Newcrest Mining Limited)
From: Genevieve New
Subject: Site Visit Record - Inspection of the Northern Tailings Storage Facility at Cadia Valley Operations, near Orange, NSW
Date: Friday 9th March 2018 Our Ref: 115293.18 NTSF M013 Draft

This site visit record is provided as an event log for the day of 9th March 2018 based on written notes and photos by Genevieve New, an Associate Engineer of ATC Williams Pty Ltd. The photographic record is provided after the text with captions.

EVENT LOG

Initial Notification

Received a phone call from Peter Lord of Newcrest on 9/3/2018 at 9:36 am:

Cracking 100 m long had been observed by an operator during an inspection on the downstream slope of the NTSF that morning. Cracking was open. Peter requested me to come to site to inspect. Peter also sent through some photos of the cracking.

Note: on 11/03/2018 Geoff Hewitt advised me that Larry Wright had made the initial observation of cracking. Larry had been called in from break to install the vibrating wire piezometer in Standpipe P9.

I recommended to Peter Lord that no personnel or vehicles should access the embankment crest or downstream area for normal work purposes. I also recommended that no further tailings deposition should occur in this area. I advised that access to this area should be restricted. I requested the most recent surveillance data to be supplied and for new piezometer readings to be obtained.

I provided regular updates to Peter Lord on my expected travel arrangements and times. I booked next flight on Fly Corporate at 12:20 pm, from Essendon Airport to Orange Airport, arriving at 1:50 pm. This was the quickest route to site. I phoned Peter Lord at the airport to confirm my arrival. I drove to site, arriving in car park at 2:20 pm. I sorted my files and then entered site via access control gates near the administration area.

During this period and onwards, I maintained contact with Keith Seddon, a Senior Principal at ATC Williams, for ongoing technical advice.

Arrival

Onsite at 2:30 pm I met with Peter Lord and David Cuello. I was immediately transferred by car to the NTSF with both Peter Lord and David Cuello.

Inspection of the Embankment

Inspection of NTSF started at 2:37 pm. I inspected the Stage 1 Buttress, Stage 7 crest, Stage 8 crest and Stage 9 embankment crest up until 3:49 pm. Area inspected was from: in-line with the STSF Western Embankment alignment, to within approximately 100m of Standpipe P9. The length of the affected embankment was estimated to be 200m.



The following observations were made:

Stage 1 Buttress Crest

1. Diagonal cracking was noted on the crest of the Stage 1 Buttress to the west, from the Stage 7 embankment crest edge to the outer edge of the Stage 1 Buttress. Cracks were 5-10 m long each and closed. Multiple cracks were noted parallel to each other, and offset from each other at approximately 1-2m apart. Overall crack pattern was wedge shaped (at edges of affected area). This crack area extended on this side for approximately 20 m.
2. New cracking had occurred since the surveyors had marked the cracks approximately 1-2 hours prior (time to be confirmed by CVO personnel as was done whilst I was not present).

Stage 7 Embankment Crest

3. Longitudinal cracking was noted from 0 (zero) to 4 meters from the downstream crest edge of Stage 7 embankment (towards the upstream side). Cracking was parallel with the dam alignment (= longitudinal).
4. Towards either end of the area of cracking, the crack pattern tended towards the downstream crest edge (slight circular pattern at edges). Multiple (up to 10) major cracks made up the overall shape.
5. During initial inspection no 'sound' could be heard from the cracks (knelt down and listened with ear to crack in two locations on this stage). However after a period of time, rock fall could be heard (estimate 3:00 - 3:20pm onwards) at intervals of approximately 2-5 minutes.
6. From visual inspection, the longitudinal cracks were open approximately 5-10 cm. There was no vertical offset at this time (opening was horizontal).
7. Crack depth from visual estimation was approximately 1m; however predicted crack depth was greater (unable to observe due to irregular crack surface).
8. Crest was predominantly dry. At a single location the surface was observed to be moist (of limited extent, 0.5m length, beneath rocks on the downstream batter of Stage 8).

Stage 8 Embankment Crest

9. Longitudinal cracking was noted at 3 m and 4 m from the downstream crest edge of the Stage 8 embankment towards the upstream side.
10. Cracking was open. From visual inspection cracks were open approximately 5-10 cm.
11. Crack depth from visual assessment was approximately 1 m; however predicted crack depth was greater (unable to observe due to irregular crack surface).
12. During initial inspection no 'sound' could be heard from the cracks (knelt down and listened with ear to crack in two locations on this stage). Crest was dry.

Stage 9 Embankment Crest

13. No cracking was noted on the Stage 9 embankment crest edge. The embankment crest, for length parallel to cracking on the lower embankments, showed no cracking. Crest was in excellent condition. Crest was dry.

Stage 10 Embankment Crest

14. I did not inspect the Stage 10 embankment crest. Verbal advice from Peter Lord was that no cracking had been observed on this crest. This correlated well to my observations of the Stage 9 embankment crest.

Discussions at Stage 1 Buttress Crest

When I arrived on the embankment, multiple site personnel were already on the Stage 1 buttress. More people arrived whilst I was there (in the latter time period) include Peter Sharpe.

Peter Lord had a copy of the Dam Safety Emergency Plan with him. We discussed the level of response based on the site observations.

During the time of the inspection a new crack developed to the right hand side (west), parallel with the embankment, extension estimated at 20m from previous area, open, 2 cm-5 cm, level (no vertical offset).

At this time I recommended that no personnel have access to the dam in this location (area of inspection). Movement was evaluated to be active. I stated that the embankment was not stable, and that the situation was developing.

CVO site action at status of dam: CON1 (concentrator) was switched off. No further tailings distribution occurred to the NTSF.

All personnel left the area.

Inspection of the Toe Area

After inspection of the embankment, Peter Lord advised that I should inspect the toe area. Cracking had been observed at the toe.

15. Inspection of Toe Lock area (downstream toe of the NTSF below area of cracking on downstream slope) from after 3:50 pm until approximately 4:30 pm.
16. Ground rupture was noted to the left (eastern side) of the old borrow pit at the toe of the dam. Rupture was approximately 20m long. I did not approach closer than 30m to this rupture for my own safety reasons (personal assessment).
17. On downstream face of area where tailings had been excavated out of the old borrow pit, significant irregular cracking was noted on batter slope (I was facing the dam).
18. Advice from Peter Lord was this cracking was new (not observed on the day before); cracking was observed in this area after cracking was observed on embankment of upper embankment.
19. I believe it was at this time a decision was made to switch off tailings discharge to STSF (CON2 (concentrator) was switched off; no further tailings deposition to the STSF). I was not a part of this decision and was not present at the location of this request. The decant pump station for the return water systems remained operating for both TSFs.
20. Rock fall could be heard from the toe of the dam. After hearing rock fall I called for relocation to a greater distance from the dam. I repeated this more loudly to ensure David Cuello and other (Peter Sharpe or Peter Udy - I do not recall) heard me. They had been walking within the danger zone towards the east. I made it clear this area was no longer safe for personnel access.
21. We retreated to the vehicles and drove to slightly higher ground to the west. We relocated to the side of the area of concern, however still within a potential failure zone. Further photos were taken by the group. At this time I took another photo for record (at end of text).
22. After a period of what seemed like 10 minutes (some rock fall was heard at a distance in this period by myself), someone in the group made the decision to relocate to a meeting room to debrief.

Emergency Response (Technical) Room

I was present in the Emergency Response (Technical) Room.

Whilst we were in this room the DSEP notification of external stakeholders was implemented by Newcrest.

By 7pm (estimated) the slumping of the embankment occurred. I was not informed at the time, however suspected something had occurred as multiple personnel were called out. Peter Lord informed me over the phone a short period after and sent me through a photo.

A group of Newcrest personnel then invited me to come along to attempt to inspect the area. It was nearing dark by this time. I observed the slump from the western side. We then drove to the northern lookout to continue to observe the area, which is where the rest of the personnel were.

From this point we returned to the Technical room to debrief.

End of Day (Departure)

I left site when it was dark.

I contacted both my office manager Mark Dillon and Keith Seddon with information on the site status.

A photo record is provided on the following pages for reference.

**Genevieve New
Associate Engineer
ATC Williams**

DRAFT

PHOTOGRAPHIC RECORD of 9/3/2018

EMBANKMENT CREST AREA

View along Stage 9 embankment crest, looking east, survey marker 7 in mid distance (IMG_2530)



Looking from Stage 9 crest down across Stage 8 and Stage 7 and Stage 1 Buttress, looking south-easterly (IMG_2531)



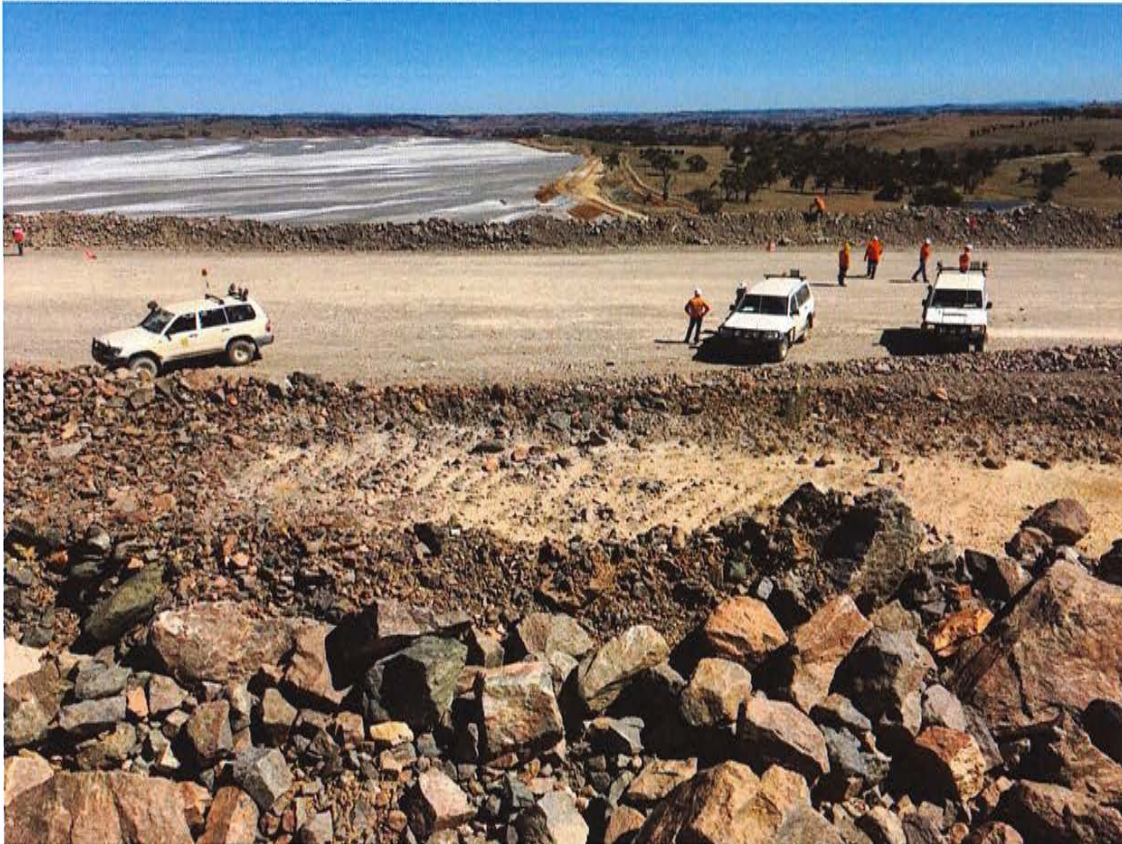
Western extent of cracking, survey marker 7, looking east along stage 9 (IMG_2533)



Western extent of cracking, survey marker 7, looking south (IMG_2534)



Looking from Stage 9 crest down onto Stage 8, Stage 7 and Stage 1 Buttress - this marks the western end of the cracking (IMG_2532)



From Stage 9 looking down onto Stage 8 crest, and Stage 7 embankment Stage 1 buttress below, looking south-east (IMG_2528)



View from Stage 9 looking downstream, visible cracking on Stage 8, towards eastern end of zone of cracking (IMG_2527)



From Stage 9 looking down onto Stage 8 Crest and Stage 1 Buttress below, looking southwest (IMG_2529)



Stage 8 Crest looking East from western extent of cracking, white paint indicates cracking (IMG_2535)



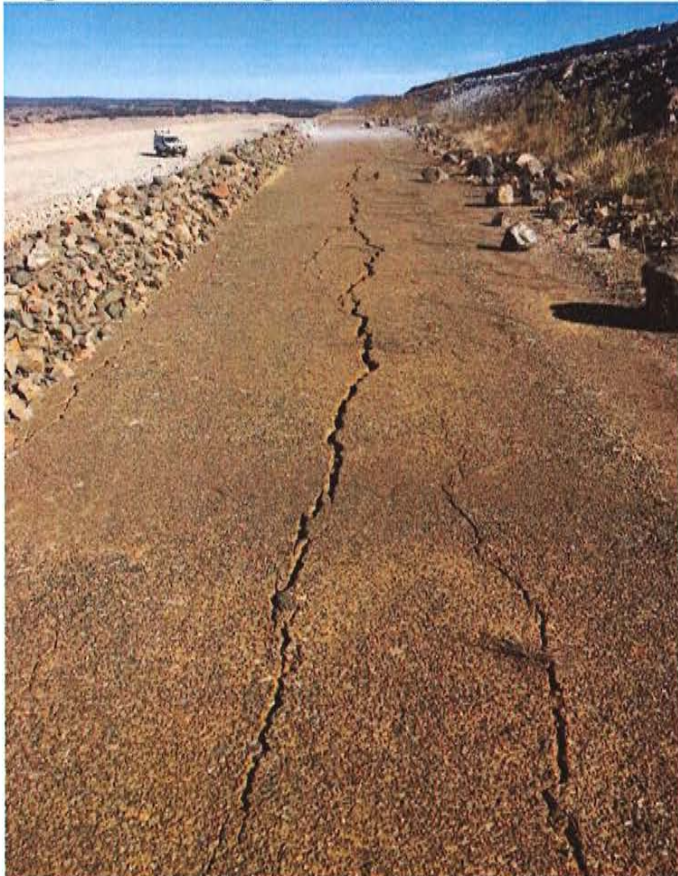
Stage 8 crest, close-up of crack opening (IMG_2526)



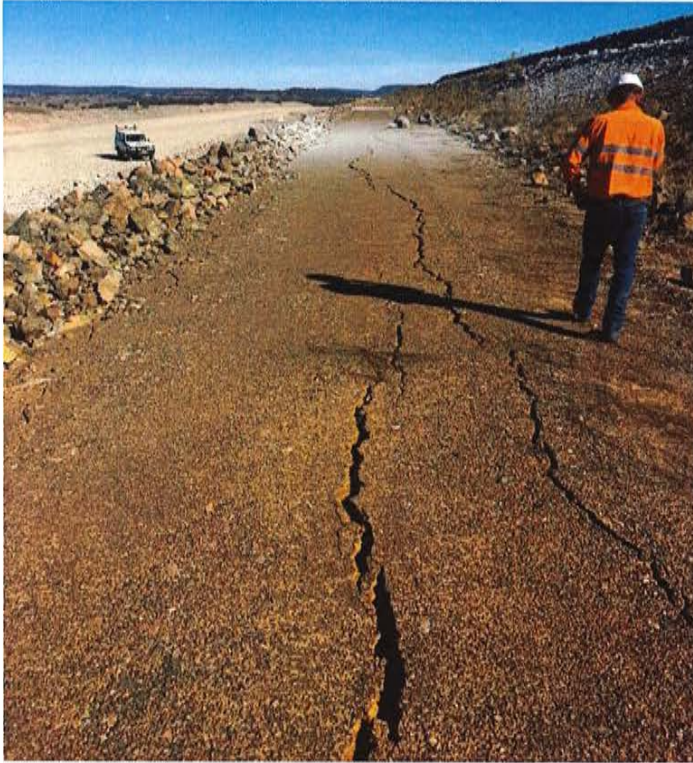
Stage 8 Crest Looking West, (IMG_2524)



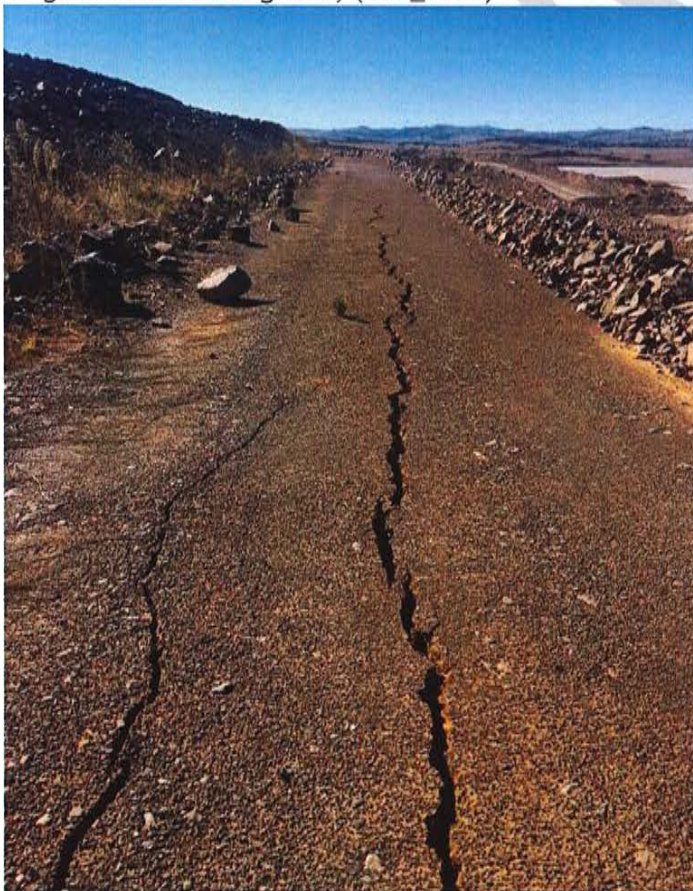
Stage 8 Crest Looking West, (IMG_2519)



Stage 8 Crest Looking West, (IMG_2521)



Stage 8 Crest Looking East, (IMG_2522)



DOWNSTREAM TOE

Initial approach to toe area, ground rupture noted at dam toe, looking north-easterly (IMG_2536)



DRAFT

Toe area, linear cracking on exposed batter, looking northwards (IMG_2537)



Toe area, linear cracking on exposed batter, looking westwards (IMG_2538)



Toe area, linear cracking on exposed batter, looking NNE (IMG_2539)



Toe area, linear cracking on access road (ground rupture), looking NE (IMG_2540)



Toe area, linear cracking on access road (ground rupture), looking Northwards (IMG_2541)



Toe area, linear cracking on access road (ground rupture), looking North-easterly (IMG_2542)



Toe area, linear cracking on access road (ground rupture), looking North-easterly (IMG_2543)



Relocated to a distance from the toe area, overall view looking Northeast (IMG_2544)



POST EMBANKMENT SLUMP RECORD

Photo looking easterly at side of slumped area (by Peter Lord)



Photo looking easterly at side of slumped area (picture correction applied for low light)



Photo looking over NTSF impoundment area post slump (picture correction applied for low light)

