Executive summary

Talison Lithium Limited (Talison) is proposing to increase throughput at the Greenbushes operation from the existing 4.7 Mtpa to 9.5 Mtpa of spodumene ore, which will produce up to 2.3 Mtpa of lithium mineral concentrate. Further expansions include remining and retreatment of TSF1 tailings.

An option study was carried out to manage the tailings production due to these expanded processing activities. The study concluded that although TSF1 and TSF2 would continue to be developed to store tailings, an additional tailings storage facility would be required to facilitate the increased tailings production forecasted (GHD, 2017). This report documents the detailed design of the new dual cell TSF4 and associated infrastructure.

TSF4 will be located south of TSF 1 and will use part of the TSF 1 south wall as containment.

A dam break assessment was undertaken to assess the potential consequences of failure. The consequence category was assessed as High B (ANCOLD, 2012) and the DMIRS hazard category was Category 1.

The design for the external walls uses the centreline construction method with a vertical clay core and waste rockfill for downstream zones. A starter dam up to 15 m high will provide for approximately the first two years operation, followed by 3 m raises at approximately yearly intervals. The final crest level was designed as RL 1295 m, resulting in a maximum embankment height of approximately 45 m.

The TSF 4 design was divided into two cells using a divider embankment built of mine waste and using the centreline construction method. The eastern cell as designed such that the decant pond will be against the northern TSF1 embankment, which will be covered by a clay blanket to act as a water barrier. The western cell only partly rests against TSF 1 and was designed to have a central decant. Decant water will be recovered by land based pumps with floating suctions.

Internal toe drains were included in the design to control seepage, reduce the amount of saturated tailings near the external embankments and the reduce the phreatic line. It is not necessary to provide a liner on the floor of the storage due to the presence of clay in the foundation and therefore low expected rates of seepage.

A detailed geotechnical investigation was completed to support the design of TSF4. A soft layer of material was encountered in the foundation around the southern embankment. The stability under post earthquake conditions was assessed to be dependent on the properties of the soft layer. It is recommended that the additional mine waste required for the downstream part of the future raises above the starter embankment is continuously placed to provide additional weight and facilitate consolidation of the foundation.

It is critical that the actual degree of ongoing consolidation is monitored regularly using the three piezometers in the F4 low strength, soft foundation layers. Further piezometers will be required in both F4 and F5 foundations before construction commences to check if consolidation of the critical foundation zones are at or higher than the rate assumed for the design.

This report is subject to and must be read in conjunction with the limitations set out in Section 1.4.
# Table of contents

1. Introduction ............................................................................................................................................. 1
   1.1 Project information ........................................................................................................................ 1
   1.2 Purpose of this report ..................................................................................................................... 1
   1.3 Previous studies ............................................................................................................................. 3
   1.4 Limitations ....................................................................................................................................... 3
2. Basis of design ........................................................................................................................................... 4
   2.1 General ................................................................................................................................................. 4
   2.2 Key assumptions .............................................................................................................................. 4
   2.3 Tailings production .......................................................................................................................... 4
   2.4 Storage requirements ....................................................................................................................... 5
   2.5 Seismicity .......................................................................................................................................... 6
3. Site characterisation ............................................................................................................................... 7
   3.1 Climate ................................................................................................................................................. 7
   3.2 Topography and surface water ....................................................................................................... 7
   3.3 Geology ............................................................................................................................................... 7
   3.4 Hydrogeology ..................................................................................................................................... 8
4. Site conditions .......................................................................................................................................... 10
   4.1 Land use ........................................................................................................................................... 10
   4.2 Existing TSF landforms ................................................................................................................. 10
   4.3 Powerlines ...................................................................................................................................... 11
   4.4 TSF1 retreatment plant .................................................................................................................. 11
   4.5 Site access ..................................................................................................................................... 12
5. Geotechnical investigations .................................................................................................................. 13
   5.1 Site investigation 2018 ..................................................................................................................... 13
   5.2 Boreholes ......................................................................................................................................... 13
   5.3 Test pits ........................................................................................................................................... 13
   5.4 CPTu .................................................................................................................................................. 14
   5.5 Sterilisation drilling ......................................................................................................................... 14
   5.6 Summary of investigations ............................................................................................................. 15
   5.7 Discussion on soft layer material ................................................................................................. 16
6. Dam break and consequence assessment ............................................................................................. 17
   6.1 General .............................................................................................................................................. 17
   6.2 Definitions ....................................................................................................................................... 17
   6.3 Previous assessment (2014) .......................................................................................................... 18
   6.4 Dam break assessment for TSF4 ................................................................................................. 19
   6.5 Consequence assessment for TSF4 ............................................................................................ 23
   6.6 DMIRS Hazard category ................................................................................................................. 24
7. Groundwater impact assessment .......................................................................................................... 26
7.1 General .............................................................................................................. 26
7.2 TSF Domain ....................................................................................................... 26
7.3 Source of impact and potential pathways .......................................................... 26
7.4 Groundwater risks and management ................................................................... 26

8. Tailings characterisation .............................................................................................. 28
8.1 Tailings behaviour .............................................................................................. 28
8.2 Tailings segregation ........................................................................................... 28
8.3 Geotechnical properties ..................................................................................... 28
8.4 Tailings and waste rock geochemistry ............................................................... 29

9. Deposition schedule ..................................................................................................... 31
9.1 General .............................................................................................................. 31
9.2 Starter embankment .......................................................................................... 31
9.3 Raise schedule .................................................................................................. 32

10. Embankment design ................................................................................................... 33
10.1 General .............................................................................................................. 33
10.2 Location ............................................................................................................. 33
10.3 Site preparation ................................................................................................ 33
10.4 Embankment materials ...................................................................................... 34
10.5 Embankment geometry ...................................................................................... 35
10.6 Seepage assessment ........................................................................................ 39
10.7 Stability assessment .......................................................................................... 41
10.8 Seismic assessment ........................................................................................ 52
10.9 Settlement estimation ....................................................................................... 55
10.10 Freeboard ....................................................................................................... 55
10.11 Underdrainage ................................................................................................ 58
10.12 Monitoring instrumentation ................................................................................ 59

11. Water balance .............................................................................................................. 60
11.1 General .............................................................................................................. 60
11.2 Data .................................................................................................................... 60
11.3 Key Assumptions ............................................................................................... 61
11.4 Results ............................................................................................................... 61

12. Deposition and water reclaim ....................................................................................... 63
12.1 General .............................................................................................................. 63
12.2 Decant location optimisation ............................................................................. 63
12.3 Deposition modelling ......................................................................................... 65
12.4 Tailings production data .................................................................................... 65
12.5 Modelling criteria ............................................................................................. 66
12.6 Storage capacity and staged development ....................................................... 66
12.7 Beach development and pond migration ......................................................... 66
12.8 Typical beach profile ......................................................................................... 69
12.9 Tailings delivery pipeline .................................................................................. 70
12.10 Return water ...................................................................................................... 70
13. Surface water management ................................................................................. 71
  13.1 Downstream toe drain ............................................................................... 71
  13.2 Sedimentation pond ................................................................................... 71
  13.3 Embankment crest ..................................................................................... 72
14. Construction methodology ................................................................................... 73
  14.1 General ....................................................................................................... 73
  14.2 Starter embankment .................................................................................. 73
  14.3 Subsequent raises ..................................................................................... 74
15. TSF1/2 seepage management .............................................................................. 76
16. Closure design .................................................................................................... 78
17. Safety in design .................................................................................................. 79
18. References ........................................................................................................... 80

Table index

Table 2-1  Tailings production schedule (annual tonnes) (June 2018) ....................... 5
Table 2-2  Storage requirements ............................................................................. 6
Table 2-3  Summary of seismic design parameters ............................................... 6
Table 3-1  Monthly average climate data ............................................................... 7
Table 5-1  CPTu location details ......................................................................... 14
Table 5-2  Estimated permeability from groundwater wells ............................... 15
Table 6-1  Summary of dam break scenarios ......................................................... 22
Table 6-2  Consequence categories based on PAR (ANCOLD, 2012) .................... 23
Table 6-3  Severity of damages and losses ............................................................ 24
Table 6-4  Hazard rating and heights for TSF categories (DMP, 2013) ............... 25
Table 8-1  Tailings laboratory testing .................................................................... 29
Table 10-1  Hydraulic conductivity parameters used for seepage analyses .......... 40
Table 10-2  Vertical seepage flows into foundation ............................................. 41
Table 10-3  Recommended FOS and shear strength parameters ......................... 43
Table 10-4  Material unit weight .......................................................................... 45
Table 10-5  Material parameters for short-term loading conditions .................... 45
Table 10-6  Material parameters for long-term loading conditions ...................... 45
Table 10-7  Material parameters for post-seismic loading condition ................. 46
Table 10-8  Cell 1 Stability analysis results ......................................................... 47
Table 10-9  Cell 2 Stability analysis results ......................................................... 48
Table 10-10 Deformation analysis results .............................................................. 55
Table 10-11 Stiffness parameters adopted ................................................................. 55
Table 10-12 Settlement estimates ........................................................................ 55
Table 10-13 Freeboard criteria ............................................................................. 57
Table 10-14 Key TSF levels .................................................................................. 57
Table 10-15 Seepage sumps ................................................................................. 59
Table 11-1 Rainfall and evaporation data ............................................................. 61
Table 11-2 Water balance assumptions ............................................................... 61
Table 12-1 Alternative decant pond options against Base Case ....................... 65
Table 12-2 Tailings deposition input parameters ............................................... 66
Table 12-3 Starter embankment storage capacity .............................................. 66
Table 13-1 Toe drain design parameters ............................................................. 71

**Figure index**

Figure 1-1 Talison locality plan (existing) .......................................................... 2
Figure 3-1 Groundwater under TSF4 ................................................................. 9
Figure 4-1 Existing TSF landforms adjacent to TSF4 ....................................... 10
Figure 4-2 Powerline arrangement ................................................................. 11
Figure 4-3 Indicative TSF1 retreatment plant location (by others) .......... 12
Figure 4-4 Indicative access to site (by others) ................................................. 12
Figure 6-1 Dam break failure locations .......................................................... 20
Figure 6-2 Breach formation (Knight and Froehlich, 2014) ............................ 22
Figure 9-1 Storage capacity curve .................................................................. 32
Figure 10-1 Plasticity chart for fine grained soils around TSF4 site .......... 34
Figure 10-2 PSD from TSF2 geotechnical investigation ................................. 35
Figure 10-3 Typical cross-section of TSF4 perimeter embankment .......... 37
Figure 10-4 Typical cross-section of TSF4 divider embankment ................. 38
Figure 10-5 Proposed TSF1 lining ................................................................. 39
Figure 10-6 Final embankment geometry example - Cell 1 South ............... 41
Figure 10-7 Cell 1 South, starter embankment, post seismic stability analyses .... 49
Figure 10-8 Cell 2 North, starter embankment, post seismic stability analyses .... 49
Figure 10-9 Sensitivity graph for cohesion in Foundation Layer F4 in Cell 1 South post seismic case ................................................................. 50
Figure 10-10 Sensitivity graph for cohesion in Foundation Layer F4 in Cell 2 North post seismic case ................................................................. 50
Figure 10-11 Cell 1 South post seismic stability with downstream berm .......... 51
Figure 10-12 Cell 2 North post seismic stability with downstream berm................................. 51
Figure 10-13 Starter embankment post seismic stability after 2 years of consolidation ...... 52
Figure 10-14 Third lift with strength gains from starter embankment load only ............... 52
Figure 10-15 Tailings PSD results (GHD, 2014) ................................................................. 53
Figure 10-16 Normalised deformation curve ....................................................................... 54
Figure 10-17 Decant pond locations for starter embankment ............................................. 56
Figure 10-18 Definition of Freeboard for TSF (DMP, 2015) ................................................ 56
Figure 10-19 IFD Data for Greenbushes WA (BoM) ........................................................... 57
Figure 11-1 Schematic representation of water balance .................................................... 60
Figure 11-2 Monthly Water Volume for Dry and Wet Conditions .................................... 62
Figure 11-3 Decant Pumping Rate for Average, Wet and Dry Conditions ....................... 62
Figure 12-1 Base Case – centralised decant pond location ............................................. 63
Figure 12-2 Alternative decant pond location options ..................................................... 64
Figure 12-3 Tailings and pond in July 2019 ..................................................................... 67
Figure 12-4 Tailings and pond in December 2019 ............................................................ 67
Figure 12-5 Tailings and pond in June 2020 ................................................................. 68
Figure 12-6 Tailings and pond November 2020 .............................................................. 68
Figure 12-7 Tailings and pond December 2021 .............................................................. 69
Figure 12-8 Typical beach profile and pond location ....................................................... 69
Figure 13-1 Catchment areas ......................................................................................... 71
Figure 14-1 Minimum bench width ................................................................................ 73
Figure 14-2 Possible staging of downstream mine waste rock ....................................... 74
Figure 14-3 Maximum recommended height of mine waste rock placed ahead ............ 74
Figure 14-4 Raise of first Zone 1A embankment ............................................................ 74
Figure 14-5 Filling of void between Zone 1A and mine waste ........................................ 74
Figure 14-6 Raise of second Zone 1A embankment ....................................................... 75
Figure 14-7 Filling of void between Zone 1A and mine waste (2) ................................... 75
Figure 14-8 Raise of mine waste 2 raises ahead of Zone 1A .......................................... 75
Figure 14-9 Staged raising to final embankment profile .................................................. 75
Figure 15-1 Starter embankment existing seepage trench location ............................... 76
Figure 15-2 Final embankment existing seepage trench location .................................... 77
Appendices

Appendix A - Drawings
Appendix B – Basis of Design
Appendix C – Dam break plans
Appendix D – Tailings laboratory test certificates–
Appendix E – Seepage and stability analyses
Appendix F – Safety in design register
1. **Introduction**

1.1 **Project information**

Talison Lithium Limited (Talison) is proposing to expand the existing Greenbushes Lithium Mine located within the Shire of Bridgetown – Greenbushes approximately 250 km south of Perth in Western Australia. A site locality plan is shown as Figure 1-1.

The project expansion will increase throughput at the Greenbushes operation from the existing 4.7 Mtpa to 9.5 Mtpa of spodumene ore, which will produce up to 2.3 Mtpa of lithium mineral concentrate. The project expansion will involve the following:

- expansion and merging of the three existing open pits
- extension of the Floyds waste rock landform
- development of additional catchment dams below the Floyd's waste rock landform
- establishment of a new tailings storage facility to accommodate increased tailings production
- construction and operation of new infrastructure including a new mine services area, explosive storage facilities, a new crushing circuit and two new processing plants

The spodumene ore body comprises two grades of ore, chemical grade (96%) and technical grade (3%). The chemical grade processing will be ramped up between financial year 2019 (FY19) and FY23 and proceed at full production until end of FY38. The technical grade will continue to be processed at the same rate as currently and is expected to be depleted by FY33.

A separate processing plant will be commissioned to process the re-mined TSF1 tailings. The TSF1 re-mining is planned to be processed between FY20 and FY26. During this period, TSF1 will be unavailable for tailings deposition due to mechanical tailings re-mining activities.

The site base survey is provided in Map Grid of Australia (MGA) however, the elevations are in Mine Datum, 1000 m above Australian Height Datum (AHD).

1.2 **Purpose of this report**

This report documents the detailed design of the new tailings storage facility, TSF4, required to meet the increased tailings production rates. The detailed design drawings for TSF4 are included in Appendix A.

The TSF4 design was generally carried out in accordance with the requirements of the Department of Mines, Industry Regulations and Safety (DMIRS), Australian National Committee on Large Dams (ANCOLD), Australian Standards and other relevant standards.
Figure 1-1 Talison locality plan (existing)
1.3 Previous studies

In November 2017, GHD completed a tailings storage options study for the expanded production and potential re-mining of TSF1 (GHD, 2017). The study concluded that the existing tailings storage facilities (TSF1 and TSF2) would not provide sufficient storage and that a new facility (TSF4) would be required should the expansion proceed. A site location to the south of TSF1 was selected as the preferred location for TSF4.

In early 2018, Talison provided GHD an updated production schedule that included the re-mining of TSF1 as well as the possible tailings production from the adjacent Global Advanced Metals (GAM) plant. GHD completed a study to refine the requirements for TSF4 by updating the previous options study and developing a concept design for TSF4 (GHD, 2018 a). GHD recommended a dual cell facility raised using a centreline construction methodology to be adopted for TSF4 and that the concept design presented in the study be refined based on the findings of a geotechnical site investigation, completed in parallel to the detailed design.

1.4 Limitations

This report has been prepared by GHD for Talison Lithium Limited and may only be used and relied on by Talison Lithium Limited for the purpose agreed between GHD and the Talison Lithium Limited as set out in Section 1.2 of this report.

GHD otherwise disclaims responsibility to any person other than Talison Lithium Limited arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible. The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer Section 2.2 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report based on information provided by Talison Lithium Limited and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the location of buildings, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this report.

Site conditions (including the presence of hazardous substances and/or site contamination) may change after the date of this Report. GHD does not accept responsibility arising from, or in connection with, any change to the site conditions. GHD is also not responsible for updating this report if the site conditions change.
2. **Basis of design**

2.1 **General**

The Basis of Design (BOD) (May 2018) for TSF4 detailed design is included in Appendix B. The BOD included the following:

- Scope of work and design objectives
- Battery limits
- Design standards and criteria
- Tailings properties
- TSF location and arrangement
- Deposition modelling
- Dam break and consequence assessment
- Hydrology and water balance
- Groundwater assessment
- Embankment design
- Drainage systems
- Return water system
- Preliminary closure plan

Additional basis of design items that were developed during the detailed design or updated from the original BOD document (May 2018) are detailed in this chapter.

2.2 **Key assumptions**

The following key assumptions were made as part of this detailed design:

- TSF1 and TSF2 will receive a portion of the Chemical Grade tailings so as to limit deposition rate to a maximum 2.5 m/year beach rate of rise, and the remainder of this stream will report to TSF4
- The full production of TSF1 retreatment tailings production will report to TSF4
- The full production of Tech Grade tailings production will report to TSF4
- The 3D model of the embankments and associated facilities is based on survey data provided by Talison on 24 May 2018
- A tailings beach slope of 1% is assumed based on survey of TSF2 tailings beach
- Relocation of the existing overhead powerlines will be undertaken by others and in various stages over the development of TSF4

2.3 **Tailings production**

The tailings production schedule provided by Talison for previous design stages was used to develop the BOD. A revised schedule showing a significantly increased production rate was provided by Talison during the early stages of the detailed design (email correspondence on 1 June 2018 – file name: 2018 Scenario Revision_v4.xlsx), and the detailed design was based on this revised schedule. The tailings production schedule used in the detailed design is summarised in Table 2-1.
Tech Grade and TSF1 retreatment tailings total production was assumed to report directly to TSF4 from FY19. Chem Grade tailings production was assumed be distributed between TSF2 and TSF4 to the end of FY25, after which the production will be distributed between TSF1, TSF2 and TSF4 until TSF1 and TSF2 reach capacity.

The TSF4 detailed design does not consider the tailings production from the adjacent Global Advanced Metals (GAM) plant.

**Table 2-1 Tailings production schedule (annual tonnes) (June 2018)**

<table>
<thead>
<tr>
<th>FY (1 July to 30 June)</th>
<th>Technical Grade</th>
<th>Chemical Grade</th>
<th>TSF1 Retreatment</th>
<th>Annual Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>FY17</td>
<td>206,636</td>
<td>1,199,050</td>
<td></td>
<td>1,405,686</td>
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<tr>
<td>FY18</td>
<td>201,599</td>
<td>1,498,520</td>
<td></td>
<td>1,700,119</td>
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<tr>
<td>FY19</td>
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<td>3,027,294</td>
<td>850,000</td>
<td>4,080,480</td>
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<tr>
<td>FY20</td>
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<tr>
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<td>4,696,966</td>
<td>1,700,000</td>
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<tr>
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<td>6,194,786</td>
<td>1,700,000</td>
<td>8,096,154</td>
</tr>
<tr>
<td>FY23</td>
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<td>1,700,000</td>
<td>8,778,954</td>
</tr>
<tr>
<td>FY24</td>
<td>201,829</td>
<td>6,877,125</td>
<td>850,000</td>
<td>7,928,954</td>
</tr>
<tr>
<td>FY25</td>
<td>201,829</td>
<td>6,877,125</td>
<td></td>
<td>7,078,954</td>
</tr>
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</tr>
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<td></td>
<td></td>
<td>5,590,663</td>
</tr>
</tbody>
</table>

2.4 Storage requirements

The distribution of the annual tailings production provided in Table 2-1 assumed the following:

- TSF2 will continue to receive tailings from Chem Grade production to a maximum rate of rise of 2.5 m/year
- TSF2 will reach design capacity in FY34 (no further deposition into TSF2 after this date)
- TSF1 will receive tailings from Chem Grade production to achieve a maximum rate of rise of 2.5 m/year from FY25 when TSF1 remining is expected to be complete
- TSF1 will reach design capacity in FY37 (no further deposition into TSF1 after this date)
- TSF4 will receive the full Tech Grade and TSF re-mining production as well as the remaining balance from Chem Grade production following distribution to TSF1 and TSF2 as described above.

Due to the changes in the production annually and the distribution assumptions listed above, the predicted deposition rate into TSF4 varied annually as presented in Table 2-2. The total storage capacity required was estimated to be approximately 68 Mt or 49 Mm³ (based on an average dry density of 1.4 t/m³).
<table>
<thead>
<tr>
<th>FY (1 July to 30 June)</th>
<th>Balance to TSF4 (tonnes)</th>
<th>Balance to TSF4 (m³)</th>
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<td>FY19</td>
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<tr>
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</tr>
<tr>
<td>FY21</td>
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</tr>
<tr>
<td>FY22</td>
<td>6,087,000</td>
<td>4,348,000</td>
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<tr>
<td>FY23</td>
<td>6,786,000</td>
<td>4,847,000</td>
</tr>
<tr>
<td>FY24</td>
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</tr>
<tr>
<td>FY34</td>
<td>4,113,000</td>
<td>2,938,000</td>
</tr>
<tr>
<td>FY35</td>
<td>4,215,000</td>
<td>3,011,000</td>
</tr>
<tr>
<td>FY36</td>
<td>6,877,000</td>
<td>4,912,000</td>
</tr>
<tr>
<td>FY37</td>
<td>5,591,000</td>
<td>3,993,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>68,196,000</strong></td>
<td><strong>48,711,000</strong></td>
</tr>
</tbody>
</table>

### Table 2-2 Storage requirements

#### 2.5 Seismicity

As part of the feasibility study for TSF2, GHD carried out a site specific seismic hazard assessment (GHD, 2015). The seismic design parameters from the 2015 assessment are summarised in Table 2.3. The results of the 2015 assessment will be used for the detailed design of TSF4 as per the BOD.

The Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) (now called the Safety Evaluation Earthquake (SEE)) loading will be determined using ANCOLD Guidelines (2012).

The site seismic study recommended a Magnitude 5.3 earthquake with epicentre distance of approximately 20 km and a Magnitude 6.0 with epicentre distance of 20 km to be adopted as OBE and MDE, respectively.

#### Table 2-3 Summary of seismic design parameters

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Rock PGA (g)</th>
<th>Magnitude for Design Event</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000 years (OBE)</td>
<td>0.07</td>
<td>5.3</td>
</tr>
<tr>
<td>10,000 years (MDE/SEE)</td>
<td>0.25</td>
<td>6.0</td>
</tr>
</tbody>
</table>

The selection of the design event was mainly based on the assumption that the probability of exceedance of the design earthquake magnitude at the chosen return period (i.e. 1,000-year return period and 10,000-year return period) is less than 50%.

The Maximum Credible Earthquake (MCE) for the project site was estimated to be a Magnitude 7.5 earthquake with an epicentre distance of 20 km. This estimate corresponds with the recommendations of previous studies (e.g. Brown and Gibson, 2004 and Burbidge 2012). The MCE is relevant only for closure considerations.

Seismically induced deformations are estimated using the methods outlined in ANCOLD, 2012.
3. Site characterisation

3.1 Climate

Bureau of Meteorology (BoM) rainfall records have been maintained at the town of Greenbushes since 1893 (Station 009552). According to the rainfall data, the lowest annual rainfall of 405 mm occurred in 2012 and the highest of 1,687 mm occurred in 1917. The mean annual rainfall for the last 125 years was 840 mm.

The monthly averages for rainfall and evaporation are shown in Table 3-1 and are based on the historical data from 1893.

Table 3-1 Monthly average climate data

<table>
<thead>
<tr>
<th>Month</th>
<th>Average Rainfall (mm)</th>
<th>Average Evaporation* (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>18</td>
<td>196</td>
</tr>
<tr>
<td>Feb</td>
<td>10</td>
<td>159</td>
</tr>
<tr>
<td>Mar</td>
<td>26</td>
<td>132</td>
</tr>
<tr>
<td>Apr</td>
<td>49</td>
<td>79</td>
</tr>
<tr>
<td>May</td>
<td>114</td>
<td>47</td>
</tr>
<tr>
<td>Jun</td>
<td>127</td>
<td>32</td>
</tr>
<tr>
<td>Jul</td>
<td>158</td>
<td>36</td>
</tr>
<tr>
<td>Aug</td>
<td>128</td>
<td>54</td>
</tr>
<tr>
<td>Sep</td>
<td>101</td>
<td>81</td>
</tr>
<tr>
<td>Oct</td>
<td>50</td>
<td>123</td>
</tr>
<tr>
<td>Nov</td>
<td>37</td>
<td>154</td>
</tr>
<tr>
<td>Dec</td>
<td>22</td>
<td>187</td>
</tr>
<tr>
<td>Total</td>
<td>840</td>
<td>1280</td>
</tr>
</tbody>
</table>

*Evaporation data used is the adjusted to Morton evaporation over shallow lakes

3.2 Topography and surface water

The TSF4 site includes two natural peaks located on the southwest and southeast portions of the site to elevations of approximately RL 1270 m. The natural ground forms a valley which is located along the centre of the site to a lower bound elevation of approximately RL 1250 m.

The north of the TSF4 site is mostly bounded by existing TSF1 embankment. TSF4 is designed to utilise the existing topography and adjacent TSF where possible.

There is no significant flowing surface water on the site; however, the surface water generally drains south via the valley along the centre of the site.

3.3 Geology

The site geology is described in detail in the Geotechnical Investigation Report (GHD, 2019) and a summary is provided below.

The 1:250,000 ‘Collie’ geological map indicates that the proposed TSF4 site is located in an area of variable geology. The elevated areas of the site are capped with early Tertiary aged laterite, which is described on the map as being a mix of massive caprock and pisolithic gravel with minor lateritised sand. The lower lying areas of the TSF site are mapped as Tertiary aged ‘old’ alluvial deposits comprising strongly lateritised conglomerates, sand and clay.
The surficial deposits are underlain by Archean basement rocks comprising granofelsic and amphibolitic greenstones. The granofelsic rocks are finely banded and granular with gneissic to schistose textures and the amphibolitic rocks are massive to schistose.

The ore body is made up of a series of pegmatite dykes trending northwest over a 7 km strike which is cross-cut with dolerite dykes. The rocks are locally faulted and sheared west of the ore body (GHD, 2019).

The residual soil profile below the TSF site reflects the variations in basement rock geology across the site, however the general profile observed comprises gravelly soils and lateritic duricrust overlying saprolite and saprock. The contact between the upper bauxite zone and the saprolite is commonly abrupt. The saprolite thickness depends on the basement rock type but is generally in the range of 15 to 20 m thick above granitic rocks, but can be up to 40 m over more mafic basement rocks such as dolerites and amphibolites. The saprolite is generally mottled and weakly to moderately cemented by iron oxides in the upper part, but becomes increasing bleached with depth.

A low strength layer was intersected in the saprolite profile during the investigation. The layer appears to be geologically controlled and is present at the vertical boundary between the pallid, leached residual soils and a lower layer of more strongly coloured soils. Laterally the layer appears to occur above an area of mafic bedrock which occurs along, and to the east, of the centre dividing embankment of the TSF

3.4 Hydrogeology

GHD prepared a Hydrogeological Investigation Report (GHD, 2018b). The findings of the investigations indicated the TSF4 site is underlain by 20 to 30 m of weathered rock, comprising clays with low permeability which limit migration of water into the deeper aquifers. Generally, where water yields are intersected, the groundwater is at depth between the boundary of clays and fresh rock. The potentiometric surface of the basement aquifer is generally above the clay zone indicating a confined aquifer.

Shallow groundwater (average RL 1250 m) was observed to be flowing southwards away towards lower terrain of the TSF4 site and released to environment as shown on Figure 3-1.

A groundwater monitoring network was proposed in areas surrounding the TSF4 footprint, aimed at monitoring water quality and seepage/goundwater levels throughout the geological profile.

The existing and proposed tailings liquor are of high enough water quality to indicate that the impact due of seepage on environment will be low.
Figure 3-1  Groundwater under TSF4
4. Site conditions

4.1 Land use

The majority of the TSF4 site is situated on a brownfield site used for agriculture. The west and east areas of the site extend into greenfield areas of state forest. Site preparation will consider the removal of fencing and reshaping of disturbed uneven ground in brownfield areas and clearing of forestry in the greenfields areas.

4.2 Existing TSF landforms

The TSF4 site is in close proximity to the closed and rehabilitated TSF3 located approximately 100 m west of the proposed TSF4 site extent.

The active TSF2 is located to the north of the TSF4 site. Existing seepage from the TSF1/2 common embankment is collected in a drain in the area between TSF2 and TSF4 as shown in Figure 4-1. The design will consider maintenance of this seepage path/drain (refer to Section 15).

The inactive TSF1 site is located north of and adjacent to the TSF4 site as shown in Figure 4-1. The southern embankment slopes of TSF1 are largely rehabilitated. TSF4 design will consider the condition of the TSF1 downstream slope which may comprise loosely placed and coarse material (large boulders). The downstream slope will require to be prepared and lined with low permeability engineered fill to enable it to be used as a containing embankment for TSF4.

Figure 4-1 Existing TSF landforms adjacent to TSF4
4.3 Powerlines

Existing powerline alignments are shown on Drawing 61-37226-C003. The existing powerline that runs along the downstream toe of TSF1 is scheduled to be removed (by others) prior to construction stage of TSF4.

The existing powerline that runs diagonally through the south-eastern portion of the TSF4 site will be realigned so that it runs along the east and south sides of TSF4 as shown on Figure 4-2.

![Figure 4-2 Powerline arrangement](image)

The junction located to the eastern edge of the TSF4 site (Figure 4-2) is to remain in place temporarily to provide power to the TSF1 retreatment plant. Therefore, the initial alignment of the TSF4 embankment assumed the junction will remain until it can be relocated to its long-term location outside the final TSF4 embankment footprint as shown in Figure 4-2. The relocation of this junction was assumed to be completed prior to Raise 1 (Year 3).

A section of the powerline in the north-east corner of Cell 1 is proposed to be installed underground as shown on Drawing 61-37226-C005. This powerline alignment is indicative only and will be designed by others. The powerline installation depth below ground should consider the underdrainage outlet to the seepage sump in this location. Alternatively, the underground powerline alignment could be located around the eastern side of the sump to avoid any potential elevation conflicts below ground level.

4.4 TSF1 retreatment plant

TSF1 retreatment plant is proposed to be located outside the north-west corner of the TSF4 site. The indicative location is provided in Figure 4-3 and the final design and locations is by others.
4.5 Site access

The main access to the site for the construction fleet will be from the north-east of the TSF4 site as indicatively shown in Figure 4-4. The powerline in this section will be buried to allow mine fleet to enter the site. The ground on the eastern side of TSF1 will require improvement to provide access (by others).
5. **Geotechnical investigations**

5.1 **Site investigation 2018**

Between May and June 2018 and GHD conducted geotechnical investigations to inform the design of TSF4. The field work included a series of boreholes and test pits, and representative samples were obtained for geotechnical testing. The outcomes of the investigations are described in the Geotechnical Investigation Report (GHD, 2019).

A low strength layer was encountered in several of the boreholes located at the southern side of the TSF4 site which were considered critical to the TSF4 embankment design. Additional investigations were undertaken in the area to characterise and determine strength of the potentially low strength layers with a higher degree of confidence. In July 2018, GHD completed the additional investigations including cone penetrometer tests with pore pressure measurement (CPTu) and dilatometer tests (DMT). In September 2018, three additional boreholes were drilled to allow tube sampling of identified low strength layer.

The 2018 geotechnical investigation locations are presented on Drawing 61-37226-C002.

5.2 **Boreholes**

Sixteen boreholes were drilled across the TSF4 site and Standard Penetrometer Tests (SPTs) were undertaken in granular and cohesive soils to estimate in-situ strength properties. Three groundwater boreholes were drilled for hydrogeological investigations and opportunity was taken to produce samples of the identified low strength layer.

The boreholes indicated the site typically has 1.5 m to 4.3 m of gravel/sand soil material or clayey gravel underlain by 4-6 m of very stiff to hard, brown, high plasticity clay.

A soft to firm, white and brown, high plasticity silty clay to clayey silt layer was encountered beneath the stiff to hard clay layers described above. This layer was estimated to have significantly lower strength parameters based on SPT results (N60 values below 8).

Highly to slightly weathered dolerite and amphibolite rock was encountered from approximately 13.5 m below ground level and slightly weathered to fresh rock typically encountered between 20 to 25 m below ground level. Bands of diorite and gneissic diorite were intersected in some boreholes and varied from extremely to slightly weathered.

Falling head tests were conducted in standpipe piezometers and in casing during drilling to estimate the permeability of the soils encountered.

Detailed logs are presented in Geotechnical Investigation Report (GHD, 2019).

5.3 **Test pits**

Thirty-eight test pits where excavated to depths varying between 1.5 to 5.6 m across the TSF4 site to understand the subsurface profile and obtain representative samples for classification testing.

The topsoil thickness across the majority of the site is about 0.6 to 1.5 m Topsoil was not encountered in some sandy locations to the south of the site.

Topsoil was generally overlying sandy/clayey gravel to depths of 0.6 to 5.3 m. Very stiff to hard, lateritised sandy/gravelly clay was encountered in the majority of holes below the sandy gravel. Test pits were generally excavated to refusal upon hard clay (lateritic) to termination depths from 1.5 m to 4.1 m below ground level.

Detailed logs and photographs are presented in Geotechnical Investigation Report (GHD, 2019).
5.4 CPTu

The aim of the CPTu was to determine properties of the silty clay to clayey silt layer encountered in the boreholes on the southern side of the site with potential low strength zones as described in Section 5.2.

The CPTu locations were pre-drilled to a depth between 5 and 7 m through material not readily penetrable by CPTu probes and cased using PVC casing. Additional pre-drilled holes were cased to larger diameters at selected locations to conducted DMTs. The details of each CPTu location is presented in Table 5-1. Raw data is included in Geotechnical Investigation Report (GHD, 2019).

Table 5-1 CPTu location details

<table>
<thead>
<tr>
<th>CPT ID</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT1</td>
<td>Predrilled and cased to 7 m.</td>
</tr>
<tr>
<td>CPT2</td>
<td>Predrilled and cased to 5 m.</td>
</tr>
<tr>
<td>CPT3</td>
<td>Predrilled and cased to 5 m. VWP installed in adjacent DMT hole tip at 5.9 m (refer to Drawing 61-37226-C002). DMT refusal at about 6 m.</td>
</tr>
<tr>
<td>CPT4</td>
<td>Predrilled and cased to 5 m</td>
</tr>
<tr>
<td>CPT5</td>
<td>Predrilled and cased to 5 m VWP installed in adjacent DMT hole tip at 7.2 m (refer to Drawing 61-37226-C002).</td>
</tr>
<tr>
<td>CPT6</td>
<td>Predrilled and cased to 5 m</td>
</tr>
<tr>
<td>CPT7</td>
<td>VWP installed in CPT hole tip at 6.5 m (refer to Drawing 61-37226-C002) Predrilled but not cased to about 4.5 m. CPT readings from 4.5 m to 5.0 m is expected to be sandy material that caved in from the top of the hole.</td>
</tr>
<tr>
<td>CPT8</td>
<td>CPT8 was predrilled and cased with PVC pipe and taped at the base of the hole. However, the PVC casing in CPT8 appeared to be backfilled with approximately 0.5 m of sand causing the casing to be pushed into the hole. CPT did not proceed.</td>
</tr>
<tr>
<td>CPT9</td>
<td>Predrilled and cased to 5 m It is assumed CPT9 was not pre-drilled deep enough due to limited access to the location and replacement drill holes were not drilled in time. CPT did not proceed.</td>
</tr>
<tr>
<td>CPT10</td>
<td>Predrilled and cased to 5 m</td>
</tr>
<tr>
<td>CPT11</td>
<td>CPT11 test location was cancelled due to the test area being waterlogged following high rainfall events.</td>
</tr>
<tr>
<td>CPT12</td>
<td>It is assumed CPT12 was not pre-drilled deep enough due to limited access to the location and replacement drill holes were not drilled in time. CPT did not proceed.</td>
</tr>
<tr>
<td>CPT13</td>
<td>Predrilled and cased to 5 m</td>
</tr>
</tbody>
</table>

The soft to firm silty clay was encountered in the all CPTs that were undertaken. The thickness of the soft to firm silty clay layers encountered in the area ranged from approximately 2 to 9 m.

The CPT plots were used to characterise and estimate the strength of the soft to stiff silty clay based on soil behaviour type (SBT). The CPT plots and interpreted data is included in Geotechnical Investigation Report (GHD, 2019).

5.5 Sterilisation drilling

Talison carried out sterilisation drilling to confirm that there are no ore bodies beneath the site. Although primarily aimed at identifying the underlying rock types, the geological logs were made available to GHD. The logs generally correlated with the findings from the site geotechnical investigations.
5.6 Summary of investigations

5.6.1 Subsurface profile

The typical subsurface profile comprises 0.6 to 1.5 m thick topsoil overlaying approximately 2 m of lateritic silty/clayey and sandy gravel. In low-lying areas approximately 1 m thick layer of sand, loose, fine to coarse, grey was encountered along the valley.

Below the upper soils lateritised saprolite – gravelly clay/clayey gravel layer was encountered at depths of 1.5 to 6.0 m across the site. This material is expected to have high strengths based on SPT refusal on this material. This layer is typically overlaying the soft to stiff silty clay layer. Soft to firm silty clays were encountered in the low-lying areas (along the valley) in the centre of the site and to the south of the site.

Bedrock was encountered at depths of approximately 12 to 24 m below ground level.

Laboratory testing and test certificates are available in Geotechnical Investigation Report (GHD, 2019).

5.6.2 Permeability

Hydrogeological groundwater wells were constructed at three locations across the site to various depths. Three wells (shallow, intermediate and deep) were constructed at each location as shown on Drawing 61-37226-C002. A summary of the estimated permeability and brief geological description of the tested layers is presented in Table 5-2.

Table 5-2 Estimated permeability from groundwater wells

<table>
<thead>
<tr>
<th>Hole ID</th>
<th>Depth (m)</th>
<th>Geological unit</th>
<th>Permeability (m/s)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>GW 01S</td>
<td>0.3 – 3.5</td>
<td>Laterite</td>
<td>3 x 10^-7</td>
<td>Clay rich with fine to medium grained quartz sand. dry firm to hard.</td>
</tr>
<tr>
<td>GW 01I</td>
<td>6 – 10</td>
<td>Saprolite</td>
<td>6 x 10^-7</td>
<td>Clay rich with fine to medium grained quartz sand. dry firm to hard.</td>
</tr>
<tr>
<td>GW 01D</td>
<td>18 – 22</td>
<td>Bedrock</td>
<td>4 x 10^-7</td>
<td>Metamorphic bedrock – fine grained, mica-rich, dark grey, highly foliated.</td>
</tr>
<tr>
<td>GW 02S</td>
<td>0.5 - 4</td>
<td>Laterite / saprolite</td>
<td>1 x 10^-7</td>
<td>Sandy clay to silty sand</td>
</tr>
<tr>
<td>GW 02I</td>
<td>13 - 17</td>
<td>Saprolite</td>
<td>7 x 10^-7</td>
<td>Clay – Grey brown, grey mica rich with some red-brown mottling. Minor fine grained quartz sand. Slightly cohesive, soft, slightly moist. Minor fragments of highly weathered foliated rock present,</td>
</tr>
<tr>
<td>GW 02D</td>
<td>26 - 28</td>
<td>Bedrock</td>
<td>5 x 10^-4</td>
<td>Metamorphic bedrock – fine grained, mica rich, dark grey with brown iron staining, vertical foliation, possible amphibolite.</td>
</tr>
<tr>
<td>GW 03S</td>
<td>3.5 - 7</td>
<td>Saprolite</td>
<td>1 x 10^-7</td>
<td>Clay – Grey to brown mottled with red through, firm, traces of gravel. Sandy lens 2.5-3.0 m. Becoming blue grey with minor mottled brown. Becoming very soft, moist from 6 m.</td>
</tr>
<tr>
<td>GW 03I</td>
<td>8.5 – 11</td>
<td>Saprolite</td>
<td>7 x 10^-7</td>
<td>Saprolite – brown to blue grey, mottled with red. Very stiff to hard (banding). Blue grey material noted to be softer clay, brown material hard/lithified.</td>
</tr>
<tr>
<td>GW 03D</td>
<td>11 - 13</td>
<td>Bedrock</td>
<td>2 x 10^-6</td>
<td>Metamorphic bedrock – Dark grey, medium grained crystalline rock, slightly foliated, possible amphibolite.</td>
</tr>
</tbody>
</table>
A falling head permeability test was carried out on samples of soil recovered from TP20 and TP35. The samples were compacted to 98% of the standard maximum dry density at 2% wet of the optimum moisture content, as determined by AS 1289.5.1.1. The tests returned permeability results ranging from $4.2 \times 10^{-9}$ to $2.3 \times 10^{-6}$ m/s.

### 5.6.3 Discussion on soft layer material

GHD completed the initial geotechnical investigations in June 2018 (Boreholes and test pits). This investigation identified low strength zones beneath the very stiff to hard lateritic clays in the valley located in the southern portion of the TSF4 site. Further investigations were undertaken in July 2018 (CPTu). Although four hole locations were inaccessible, the others confirmed that deeper foundations had a low strength for an embankment length of 500 m with the worst impact over 200 m length. Strength parameters of this lower strength zone were estimated from CPT data for the stability analysis and confirmed by the Dilatometer in situ strength testing. The results compare reasonably with the more general results from earlier investigation.

The nature of the foundation is such that pore pressures are expected to increase during rapid loading, particularly in the highest sections of the embankment (15 m embankment height in approximately six months) and therefore undrained strengths were used for short term and post seismic stability analyses of the starter embankment. As the area will subsequently be loaded progressively in ten lifts of 3 m each over the life of the facility (20 years), the low strength materials identified in the southern valley were assumed to consolidate particularly under rockfill loading and gain strength as the facility is raised. Observations during CPT testing indicate that consolidation could be achieved in the project life time. Therefore, the stability of the final embankment cross sections was assumed to follow drained behaviour for the long term case. Calculations show that the ultimate height embankment is stable under these conditions. Stability under earthquake was assessed using strengths reduced by 20% for post seismic case due to strain weakening during cyclic loading.

However, there is a contradiction in that the material descriptions and related permeability estimates from field falling head tests indicate that the material has low permeability and may not adequately consolidate during the life of embankment construction. The actual rate of consolidation can be monitored by observing pore pressure build up and dissipation in this zone during ongoing construction.

Three vibrating wire piezometers have been installed in this soft zone to monitor the pore pressures during early construction (loading). Additional vibrating wire piezometers (minimum of 2) should be installed during construction in similar soil profiles encountered. This instrumentation will indicate how the foundation is behaving in response to loading. Other monitoring instrumentation will include survey markers to monitor settlement.

Due to core losses and limited boreholes in the relevant area, only one undisturbed sample of the soft layer was recovered as part of the initial geotechnical investigation. This sample was tested in the laboratory to provide a better understanding of the behaviour of this material under load. However, it is prudent to make decisions concerning timing and ultimate stability on more than one laboratory test. Additional boreholes were drilled in October 2018 to obtain additional undisturbed samples of the soft layer. The impact of the consolidation testing on the construction of the embankment is expanded on in Section 10.7.7.
6. Dam break and consequence assessment

6.1 General
A dam break and consequence assessment was undertaken for TSF4. The results of the dam break assessment were used to determine the ANCOLD Consequence Category, DMIRS Hazard Category and prepare flood inundation maps.

The TSF consequence category was determined for different guidelines and requirements as follows:

- ANCOLD Consequence Category (Guidelines on Tailings Dams, ANCOLD, 2012)
- DMIRS TSF Category (Guide to the preparation of a design report for tailings storage facilities (TSFs), DMP, August 2015).

The design implications were set based on the most conservative requirements of the above.

6.2 Definitions
ANCOLD Guidelines (ANCOLD, 2012) define the terms used in this dam break analysis and consequence assessment as follows:

**Consequence category**
The consequence category is the classification of adverse consequences resulting from a dam failure, based upon the severity of potential damage and loss in conjunction with Population at Risk (PAR) or Potential Loss of Life (PLL). The will vary according to the type, location and mechanism of the dam failure.

Consequence categories provide a basis for determining the design and management requirements for dams.

**Dam break**
A dam break is the uncontrolled release of the contents of a dam through collapse of the dam or some part of it.

**Dam break PAR**
The dam break PAR is the total PAR minus the PAR affected by a natural flood event immediately prior to the dam break.

**Flood failure**
A flood failure event is the failure of the dam during a flood. The assessment of the Flood Failure case is based on the difference between the consequences of a natural rainfall and flood event without the dam break and the consequences of the superimposed dam break flood resulting from the failure of the dam during that event.

**Incremental PLL**
The incremental PLL is the estimated PLL for a flood event with dam failure minus the estimated PLL for the same flood event without dam failure.

**Population at risk (PAR)**
The population at risk includes all people who would be directly exposed to flood waters assuming they took no action to evacuate.
This includes dwellings, works sites and other places where people assemble, as well as itinerants who are travelling through the dam breach affected zone. The PAR may vary according to the time of day, day of the week and season.

**Potential loss of life (PLL)**

The potential loss of life is the part of the PAR that could lose their lives in the event of a dambreak event.

**Sunny day failure (SDF)**

A sunny day failure is a failure of the dam with its storage at Full Supply Level (FSL) without concurrent flood flow either into or downstream of the dam, e.g. a failure due to piping through the embankment or a failure due to a seismic event.

**Severity of damages and losses**

The Guidelines on Consequence Categories for Dams (ANCOLD, 2012) groups the severity of damages and losses due to a dam break into four categories:

- Total infrastructure costs
- Impact on dam owners' business
- Health and social impacts
- Environmental impacts

The impact on the dam owners' business, and the environmental impacts, particularly the potential for the release of elevated elements in the tailings were considered to be the primary impact in determining the Consequence Category.

The severity of damages and losses is assessed using Table 6-3.

### 6.3 Previous assessment (2014)

#### 6.3.1 Dam break assessment

**General**

GHD completed a dam break assessment on the water dams and tailings dams at the site in 2014 (GHD, 2014). The 2014 study included hydraulic analysis to determine the flood inundation area. The study considered failure of either TSF1 or TSF2 under both sunny day and flood failure conditions.

The U.S. Army Corps of Engineers Hydraulic Engineering Centre’s River Analysis System (HEC-RAS) (version 4.1.0) was used to simulate the breach of the dams and route the resulting flood to the Blackwood River. HEC-RAS was used to simulate one-dimensional unsteady flow river hydraulics calculations through a full network of open channels.

To estimate the volume of tailings released during an embankment breach a number of simplifying assumptions were required. An annual tailings deposition rate in TSF2 was assumed to be approximately 600,000 tonnes per annum with an insitu density of approximately 1.5 t/m³ (TSF1 was in care and maintenance and has been since 2005). The tailings were assumed to be relatively coarse and consolidate rapidly. It was assumed that the previous six months of deposition could be mobilised during a dam breach, i.e. 300,000 tonnes. It was assumed that only the top portion of the tailings (assumed to be 3 m) were able to be mobilised during a failure, as the tailings deposited lower in the structure would have consolidated.

A sunny day failure event was not considered the critical case, as the volume of tailings were lower than the amount that would be mobilised during a flood failure.
Flood failure of TSF1

For the Flood Failure conditions, a Probable Maximum Precipitation (PMP) storm event of 750 mm over the surface area of the TSF was applied. The PMP was calculated using the Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method (BoM, 2003).

TSF1 has an area of 2 km². It is estimated that 1,500,000 m³ to 2,000,000 m³ could be released from TSF1 during a flood failure event. A peak flow of approximately 150 m³/s was estimated for a breach in the embankment.

Flood failure of TSF2

Approximately 500,000 m³ to 1,000,000 m³ of tailings were estimated to be released from TSF2, based on a surface area of approximately 0.5 km².

TSF2 was considered likely to fail along the west or south embankments only, as the east and north sides were constrained by TSF1 and the mine respectively.

The flow from a breach in the embankment is likely to cross Maranup Ford Road and continue into Cowan Brook Dam. It was assumed that Cowan Brook Dam would be overtopped.

6.3.2 Consequence assessment

Population at risk

For failure of TSF1, the buildings shown on the aerial imagery at the time of the study (2014) to the east and south of the TSF were likely to be within the flood inundation area. Based on this, the PAR was estimated to be between 1-10 persons.

For failure of TSF2, the PAR was assumed to be the same as for overtopping of Cowan Brook Dam. This was calculated to be between 1-10 persons.

The itinerant PAR for a breach of TSF1 along the eastern embankment could include road users travelling along South Western Highway or vehicles travelling around Pit C1. The itinerant PAR for TSF1 was considered to be less than one.

Maranup Ford Road is located to the west of TSF2 and could contribute an itinerant PAR for a breach along the west embankment. The itinerant PAR for Maranup Ford Road was considered to be <1.

Severity of damages and losses

Assessments of the severity of damages and losses from a breach of either TSF1 or TSF2 were undertaken in accordance with ANCOLD (2012). The severity of damages and losses for both cases were set as “Medium”, primary due to the impacts on the business.

Consequence category

Based on a PAR of 1-10 persons and a severity of damages and losses of Medium, the Consequence Category was set as Significant for both TSF1 and TSF2.

6.4 Dam break assessment for TSF4

6.4.1 Modelling method

Two-dimensional hydraulic modelling was undertaken using the MIKE 21 software programme to simulate the progression of the flood wave by routing the outflow hydrographs downstream. The modelling determined water surface elevations, velocities and flood wave travel times based on the topography and surface roughness inputs.
A time-step of 0.3 seconds was used and the model was run for as long as necessary to route the outflow hydrographs.

6.4.2 Survey data

The following survey data was used in the dam break modelling:

- LiDAR survey Talison mine area and surrounds with a vertical accuracy of +/- 0.5 m and a horizontal accuracy of +/- 0.25 m.
- DEM data from Landgate which encompassed the wider region with an accuracy of +/- 10 m.

The flood model was setup with the combined survey data and by using the more accurate data where there was an overlap.

The surface levels between the discrete points of the raw LiDAR survey in the town-site area were linearly interpolated to provide a ground surface model.

6.4.3 Failure locations

The terrain surrounding TSF4 is generally hilly with well-defined valleys. Any flow from a failure is likely to be concentrated in these valleys.

Three failure locations were selected for modelling based on the surrounding topography as follows:

- South failure into the major valley beneath the TSF4 footprint. Tailings will flow into this valley in the case of a failure along the southern embankment
- West failure through the northern or western embankments, towards TSF2
- East failure through the eastern embankment, towards the South Western Highway.

Failure into TSF1 was not considered as the crest level of TSF1 is planned to be at a higher elevation than TSF4 for the full life of the facility.

The dam break failure locations are shown in Figure 6-1.
6.4.4 Modelling assumptions

The following assumptions were used for the dam break modelling. Actual site information was used where available, and conservative assumptions were made to account for uncertainties in assumed parameters.

- An average Manning’s number (n) of 0.037 was used for the model area. The flood inundation areas are predominantly river valleys or forest.
- A discharge – height boundary was used at the downstream of the model. The model calculated the boundary conditions using Manning’s number of 0.03 in conjunction with the river bed slope.
- The tailings were modelled to behave as water to be conservative.
- The tailings beach slope was assumed as 1%
- The final design crest level was used for TSF4 (RL 1295 m) and TSF1 (RL 1300 m)
- TSF4 was assumed to act as a single cell and the total volume for both cells was used in the modelling. This is conservative and considers a failure of the internal dividing embankment.
- The actual volume of tailings was determined based on the design. The total volume of stored tailings was approximately 46,000,000 m³ for TSF4
- The volume of additional tailings from a cascade failure of TSF1 into TSF 4 was approximately 6,000,000 m³, based on the storage volume above TSF4.
- 50% of the tailings would be released in a failure, representing 85% of known tailings dam failures. It was considered overly conservative to assume a higher percentage of released tailings given the tailings were assumed to act as water.
- 100% of the decant pond and/or flood volume would be released in a failure during the first time step.
- The size and volume of the normal decant pond was determined assuming the pond was central in each cell and 200 m from all embankments.
- The design storm event for a flood failure was set as the Probable Maximum Precipitation (PMP) event. The intensity of the PMP was 750 mm (GHD, 2014) and a coefficient of 1.0 was used to determine the runoff.

6.4.5 Dam break scenarios

Four dam break scenarios were considered as follows:

- Sunny Day Failure
- Flood Failure
- Sunny Day Failure plus cascade failure from TSF1
- Flood Failure plus cascade failure from TSF1

The preliminary modelling outputs showed the tailings and water flow were confined to the valleys surrounding the TSF, and the results were less sensitive to the scenarios or input parameters. As a result, the inundation areas, and therefore impacts from a failure were limited to areas close to and within the mine area and the downstream valleys for all scenarios. As a result, the flood failure mode was used as the basis for determining the consequences of failure for each of the failure locations as this was considered worst case, and detailed modelling of the cascade failure scenario was not undertaken.

A summary of the scenarios is provided in Table 6-1.
Table 6-1  Summary of dam break scenarios

<table>
<thead>
<tr>
<th>Scenario number</th>
<th>Failure Direction</th>
<th>Failure Mechanism</th>
<th>Decant pond</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Southern</td>
<td>Sunny Day</td>
<td>Maximum design decant pond, 200 m from all embankments</td>
</tr>
<tr>
<td>B</td>
<td>Eastern</td>
<td>Sunny Day</td>
<td>Maximum design decant pond, 200 m from all embankments</td>
</tr>
<tr>
<td>C</td>
<td>Western</td>
<td>Sunny Day</td>
<td>Maximum design decant pond, 200 m from all embankments</td>
</tr>
<tr>
<td>D</td>
<td>Southern</td>
<td>Sunny Day plus Cascade from TSF1</td>
<td>Maximum design decant pond, 200 m from all embankments for both TSF1 and TSF4</td>
</tr>
<tr>
<td>E</td>
<td>Southern</td>
<td>Flood failure plus Cascade from TSF1</td>
<td>Maximum design decant pond, 200 m from all embankments for both TSF1 and TSF4 PMP storm event</td>
</tr>
<tr>
<td>F</td>
<td>Eastern</td>
<td>Flood failure plus Cascade from TSF1</td>
<td>Maximum design decant pond, 200 m from all embankments for both TSF1 and TSF4 PMP storm event</td>
</tr>
<tr>
<td>G</td>
<td>Western</td>
<td>Flood failure plus Cascade from TSF1</td>
<td>Maximum design decant pond, 200 m from all embankments for both TSF1 and TSF4 PMP storm event</td>
</tr>
</tbody>
</table>

6.4.6 Breach characteristics

The breach formation of the embankment was based upon Model C of Knight and Froehlich (2014) as shown in Figure 6-2. This model represents a breach, which is formed at the top of the embankment and propagates down the bank over the breach formation time.

![Figure 6-2 Breach formation (Knight and Froehlich, 2014)](image)

The breach base elevation was set at the foundation level at each failure location. The breach width was approximately 125 m.

An empirical formula for estimating breach formation time based on the height of the breach and the volume of water above the breach base was developed by Knight and Froehlich (2014). Breach formation times of 30 to 60 minutes were estimated using this empirical formula.

These times aligned well to breach formation times of historical dam failures (typically between 60 and 90 minutes for Flood Failure and Sunny Day Failure modes respectively based on historical data).

6.4.7 Outflow hydrographs

Outflow hydrographs were developed for each of the scenarios listed in Table 6-1. The time-step used for all scenarios was 0.3 seconds. Sensitivity analyses were conducted on the time step and it was found that there was little to no difference in the calculated flows with smaller time steps.
The breach flow was calculated using the broad crested weir equation with a coefficient of 1.7. The width and height of the breach were determined based on the breach characteristics.

6.4.8 Results

The dam break inundation plans for each of the scenarios are provided Appendix C, which show the maximum flow depths, maximum flow velocities.

The maximum flow depths and velocities at the downstream extents of the model were influenced by the downstream Q-h boundary conditions and the modelled depths and velocities adjacent to this boundary may not be representative.

6.5 Consequence assessment for TSF4

6.5.1 General

The Consequence Category is determined in accordance with the ANCOLD Guidelines (ANCOLD, 2012) using the population at risk.

A summary of the framework used to assign a Consequence Category is presented in Table 6-2. The Consequence Category is then used to set standards for the design, management and operation of the facility.

Table 6-2 Consequence categories based on PAR (ANCOLD, 2012)

<table>
<thead>
<tr>
<th>Population at Risk</th>
<th>Severity of Damage and Losses</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minor</td>
</tr>
<tr>
<td>&lt;1</td>
<td>Very Low</td>
</tr>
<tr>
<td>≥1 to 10</td>
<td>Significant (Note 2)</td>
</tr>
<tr>
<td>≥10 to &lt;100</td>
<td>High C</td>
</tr>
<tr>
<td>≥100 to &lt;1,000</td>
<td>Note 1</td>
</tr>
<tr>
<td>≥1,000</td>
<td>Note 1</td>
</tr>
</tbody>
</table>

Note 1: With a PAR in excess of 100, it is unlikely damage will be minor. Similarly, with a PAR in excess of 1,000 it is unlikely damage will be classified as medium.

Note 2: Change to ‘High C’ where there is potential of one or more lives being lost.

The Severity of Damage and Loss categories used in Table 6-2 are detailed in the ANCOLD Guidelines (ANCOLD 2012).

6.5.2 Inundation areas

The areas that would be subject to inundation for a failure of TSF4 were assessed to be:

- Open pit (C1), South Western Highway (two locations) and other mine infrastructure for the eastern failure
- Cowan Brook Dam, TSF2, TSF3 and other mine infrastructure for western failure
- Approximately 20 buildings, including dwellings and sheds along the banks of the valley
- Farm land along the valleys
- Minor roads
- Minor road bridges
6.5.3 Population at risk

The population at risk located in the dam break inundation zone was estimated to be in the range of 10-100 persons based on the inundation areas. There may be itinerant population including road users and people using the streams within the valleys.

6.5.4 Severity of damages and losses

An assessment of the severity of damages and losses to key infrastructure and personnel within the mine site was carried out in accordance with “Table 2: Severity of damages and losses” (ANCOLD, 2012). A summary of severity level of damages and losses is presented in Table 6-3.

The major impacts of a dam failure were assessed to be loss of production, environmental impacts, reputational impacts and the costs of remediation. A failure could result in at least a three-month interruption to the business. There could be further delays due to government restrictions on restarting production.

As a result, a severity level of “Major” was assigned to TSF4.

Table 6-3 Severity of damages and losses

<table>
<thead>
<tr>
<th>Damage type</th>
<th>Severity level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total infrastructure costs</td>
<td>Major</td>
</tr>
<tr>
<td>Impact on dam owner's business</td>
<td>Major</td>
</tr>
<tr>
<td>Health and societal impacts</td>
<td>Major</td>
</tr>
<tr>
<td>Environmental Impacts</td>
<td>Major</td>
</tr>
<tr>
<td>Highest level of severity of damage and loss</td>
<td>Major</td>
</tr>
</tbody>
</table>

6.5.5 Consequence category

With the total PAR estimated to be 10-100 persons, the potential of one more lives to be lost, and a severity of damages and losses classification of “Major”, the TSF4 was assigned a consequence category of “High B” (ANCOLD, 2012) (refer to Table 6-2).

6.6 DMIRS Hazard category

The TSF category was set as per the DMIRS Code of Practice – Tailings storage facilities in Western (DMP, 2013). The category is assigned in respect to the TSFs hazard rating and highest embankment height.

Given that the TSF4 embankments will be greater than 15 m in height, the TSF was assigned a category of ‘Category 1’ (Table 6-4).
Table 6-4  Hazard rating and heights for TSF categories (DMP, 2013)

<table>
<thead>
<tr>
<th>Maximum embankment or structure height</th>
<th>Hazard rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High</td>
</tr>
<tr>
<td>&gt; 15 m</td>
<td>Category 1</td>
</tr>
<tr>
<td>5 - 15 m</td>
<td>Category 1</td>
</tr>
<tr>
<td>&lt; 5 m</td>
<td>Category 1</td>
</tr>
</tbody>
</table>

Notes:

- Cross-valley TSFs or those that block or significantly impede flow in natural drainage paths should be treated as Category 1 TSFs, regardless of the embankment height.
- In-pit TSFs are categorised assuming an embankment height of less than 5 m. In-pit TSFs extended by constructing a perimeter embankment are categorised based on the embankment height.
- For thickened discharge facilities and “dry” stacked tailings, the maximum stack height is used in lieu of embankment height.
- Integrated landforms should be classified according to the height of the retained tailings.
7. **Groundwater impact assessment**

7.1 **General**

A site wide groundwater water impact assessment was completed in 2018 (GHD, 2018 b). The groundwater and surface water impact assessment of the TSFs are described in this chapter.

7.2 **TSF Domain**

The TSF domain incorporates the existing TSFs (TSF1, TSF2 and TSF3), together with the proposed TSF4. The existing TSF structures comprise unlined cells, constructed on the pre-existing surface level, bound by retaining embankments. Drainage structures are constructed around the embankments to intercept seepage. Seepage is directed to low points and pumped to the water storage dams.

The proposed southern toe of the TSF4 embankment will be keyed-in to the underlying clay through excavation of the shallow sands to expose the underlying weathered clay basement. Drainage structures will intercept seepage water, which will be directed/pumped to water storage dams, similar to the existing TSFs (e.g. Clear Water Pond).

7.3 **Source of impact and potential pathways**

Seepage occurs from the tailings facilities as a result water used to slurry and deposit the tailings, in addition to rainfall infiltration.

The water quality of the seepage is a reflection of the tailings/slurry deposition water circuit and any leachable constituents derived during weathering of the tailings in-situ. The existing tailings water quality is extensively monitored. The results generally do not indicate acidic conditions, elevated dissolved metals and salts (except lithium).

The drilling and investigations indicate that the TSF4 footprint is underlain by 15 to 25 m of low permeability clay (weathered basement), which will restrict the downwards migration of tailings derived water.

Seepage collection at the toe of the tailings facility will be constructed to manage any seepage through the embankments and foundations.

7.4 **Groundwater risks and management**

Seepage impacts from the proposed TSF4 (and existing facilities TSF1 and TSF2) to the groundwater and surface water systems should be manageable given the following:

- the prevalence of low permeability clays which underlie the site should hydraulically isolate the underlying groundwater system
- seepage collection at the toe of the tailings facility should limit migration of seepage into the surface water environment.
- excluding lithium and arsenic, the water quality of the existing and proposed facilities indicates that the waters exhibits a neutral pH with low metal concentrations. The elevated concentrations of lithium and arsenic in the circuit waters will be improved through removal of lithium and arsenic from a new water treatment plant.

Although the risks are considered low, a suitable surface and groundwater monitoring program should be developed to:

- assess groundwater levels and potential mounding in areas surrounding the facility
- monitor baseline water quality and potential impacts to the groundwater system beneath the facility
- monitor the surface water environments which may be subject to seepage discharge from the facility.

To achieve the aims, a groundwater monitoring bore network should be designed and installed in areas surrounding the perimeter and the toe of the TSF4, together with appropriate dedicated surface water monitoring locations/infrastructure.
8. Tailings characterisation

8.1 Tailings behaviour

Tailings physical and geotechnical properties were generally based on the observations and test work completed for the existing TSF2 supported by some additional sampling and test work. The sandy tailings deposited close to the perimeter embankment of TSF2 were previously identified to be potentially liquefiable as detailed in the tailings seismic assessment in Section 10.8.3.

8.2 Tailings segregation

The classification of the tailings stored in TSF2 varies as shown from the results of the 2014 CPTu and laboratory testing (Section 8.3.1). Tailings near the deposition points along the embankment comprised generally sands with silts (SP), with increasing fines content towards the decant pond away from the deposition points, to sandy silts to silts (SM/ML). This confirms that the tailings were segregating along the tailings beach.

Similar segregation behaviour is expected to take place during TSF4 tailings deposition however, the design assumed less segregation will take place during the initial filling of the starter embankment due to the high rate of rise and topography within the impoundment. To facilitate uniform tailings beach development, some impoundment reshaping is proposed (refer to Section 10.3.3).

8.3 Geotechnical properties

8.3.1 Previous tailings investigations (2014)

The interpretation of the 2014 CPTu results indicated that the coefficient of permeability of the tailings adjacent to the embankment varied between $10^{-3}$ m/s and $10^{-5}$ m/s, which correlate well with the fine sand with silt (SP) characteristics of the tailings. The permeability coefficient of the tailings further away from the western embankment was found to reduce to values between $10^{-6}$ to $10^{-9}$ m/s, which is consistent with the comparatively finer grading (SM/ML) of these tailings.

8.3.2 Recent tailings laboratory testing

As part of detailed design, three tailings samples were obtained as described below:

- Sample 1 (CGP PP470/471 D/C) – This disturbed sample was obtained from the tailings pump at the Chem Grade plant and represents the mixed particle size distribution before segregation. The characterisation and strength parameters from this sample are representative of the tailings during initial filling of the cells due to the topography where significant segregation may not be achieved due to the short beach.

- Sample 2 (GB TSF2 Sample near Crest) – This disturbed sample was obtained from the tailings beach immediately adjacent to the embankment of TSF2. The characterisation and strength parameters from this sample are representative of the sandy tailings beach where the centreline-raise will extend over the tailings beach.

- Sample 3 (GB TSF2 Sample 7 m from Crest) – This disturbed sample was obtained from approximately 7 m along the tailings beach from the embankment of TSF2. The characterisation shows the segregation and permeability of the tailings beach at the toe of the centreline raise toe.

The samples were tested for classification, permeability, particle density and shear strength tests and the results are presented in Table 8-1. The laboratory test certificates are included in Appendix D.
### Table 8-1 Tailings laboratory testing

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Particle density (t/m³)</th>
<th>% fines (silt)</th>
<th>Moisture (%)</th>
<th>Atterberg Remoulded density (t/m³) – for triaxial and permeability tests</th>
<th>Triaxial</th>
<th>Permeability (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CGP PP470/471 D/C</td>
<td>2.62</td>
<td>50</td>
<td>189.2 (slurry sample)</td>
<td>Non plastic 1.2</td>
<td>3.6 x 10^{-5}</td>
<td></td>
</tr>
<tr>
<td>GB TSF2 Sample near Crest</td>
<td>2.79</td>
<td>10</td>
<td>8</td>
<td>-</td>
<td>1.4</td>
<td>4.9 x 10^{-5}</td>
</tr>
<tr>
<td>GB TSF2 Sample 7m from Crest</td>
<td>2.66</td>
<td>28</td>
<td>23.8</td>
<td>-</td>
<td>1.4</td>
<td>2.0 x 10^{-5}</td>
</tr>
</tbody>
</table>

#### 8.3.3 Engineering properties

The following tailings geotechnical properties were used for the design based on previous investigations and laboratory testing as appropriate:

- Tailings classification – non-plastic fine sand with silt (SP) to sandy silt (SM/ML)
- An average tailings particle density of 2.69 t/m³ was adopted based on an average particle density of 13 samples representative samples obtained from the TSF2 tailings beach during the 2014 investigations, which is also consistent with the three samples tested recently (Section 5).
- Settled dry density – 1.40 t/m³ based on initial and final dry densities of 1.2 t/m³ and 1.7 t/m³ (settlement tests). This is also consistent with the estimated insitu densities based on comparing survey data and production rates.
- Coefficient of consolidation C_v (m²/year) – 1.0 m²/year based on two consolidation tests
- Estimated permeability (close to embankment) – 10^{-3} m/s to 10^{-5} m/s range based on TSF2 CPTu (2014)
- Estimated permeability (further from embankment) - 10^{-6} m/s to 10^{-9} m/s range based on TSF2 CPTu (2014)

#### 8.4 Tailings and waste rock geochemistry

##### 8.4.1 General

GHD undertook an Acid Metalliferous Drainage (AMD) assessment to demonstrate the leaching potential of the future tailings and waste rock (Ref. GHD 2018 c). The outcomes of this assessment are summarised in the following sections.

##### 8.4.2 Acidic drainage risk

Given the low sulphur content in the proposed waste rock, the risk that the waste will produce adverse quantities of acid is considered low (average sulphur 0.006%). The observed occurrence of carbonate within the proposed waste rock (amphibolite) should be sufficient to neutralise the production of the low quantities of acid which may be produced. The inferred neutralising capacity of the waste rock is supported by former assessments of the formerly mined rock, comprising: a) water quality seepage from the existing waste rock facility (16 years), b) long term kinetic testing (seven waste rock samples); and c) static AMD tests (51 waste rock samples).
The risk that the tailings will produce adverse quantities of acid is considered low, given the low sulphur content (average 0.04%). Following processing, the risk of acid production is further reduced given that tailings are subject to the addition of bicarbonate to lower the pH prior to tailings deposition. In support, a former assessment of the tailings seepage water (long-term data 1997 to 2014) indicates that the tailings drainage waters are close to neutral and are moderately buffered (bicarbonate).

### 8.4.3 Metalliferous drainage risk

A large number of elements analysed within the ore and waste rock (total 25), support that a small number of elements are elevated compared to a standard Geochemical abundance indices (GAI) and may present a source of environmental concern if mobilised during leaching of the ore and tailings.

### 8.4.4 Saline drainage risk

Former studies relating to long term data (1997 to 2014), indicates that seepage from the existing tailings storage facilities possess stable major-ion concentrations. Given the continuity of mineralogy between the formerly mined ore and the proposed ore, the risk of elevated concentrations of saline drainage deriving from the future tailings is considered low.

Former studies relating to seepage/drainage derived from the existing waste rock dump indicates stable electrical conductivity values from 2,500 to 3,500 uS/cm over the long term (1997 to 2013). Given the continuity of mineralogy between the formerly mined waste rock and the proposed waste rock, the risk of elevated concentrations of saline drainage driving from the proposed waste rock dump is considered low.

Although the risk of AMD leaching from the proposed waste rock and tailings is considered low, the understanding of risk is primarily based on the former assessments of the formerly mined waste rock and existing tailings facilities. Consequently, the AMD risk of the proposed ore/tailings and waste rock should be confirmed through the following specific AMD laboratory testing:

- Acid neutralising capacity (ANC)
- Net acid generation (NAG)
- Sulphur speciation
- Metals (testing for comprehensive suite - 52 metals)
- Leach testing (major-ions, pH, EC, metals)
- Gross alpha and beta

The numbers of samples obtained for specific laboratory analysis should be sufficient to statistically support conclusions relating to the AMD risk and/or requirements for any additional further AMD testing and analysis (e.g.: kinetic testing requirements).
9. Deposition schedule

9.1 General
The indicative storage capacity was developed based on the total tailings production rates for TSF4. The maximum TSF4 impoundment was determined based on-site constraints and the storage capacity curve was developed using the combined storage capacity and depositional areas of both cells. The storage capacity curve provides the basis to optimise the TSF4 starter embankment elevation.

9.2 Starter embankment
The storage capacity curve for the combined two-cell TSF4 is presented as Figure 9-1. This shows the selected embankment elevation for the starter embankment will provide the following:

- Approximately 3 years of storage capacity
- Tailings beach area of more than 1 Mm² at capacity
- Rate of rise initially over 4 m/year reducing to 3.5 m/year when nearing starter capacity

The starter embankment elevation was set at RL 1265 m. The subsequent raises are proposed to be annual 3 m high raises to a final embankment crest elevation of RL 1295 m. However, the rate of rise curve indicates a rate of 4 m/year in some periods during high production rates (FY22 and FY23 – refer to Table 2-1). The detailed design of subsequent raises may need to consider 4 m high raises for the periods of high production to avoid multiple raises within one year. This will be subject to review of the drainage and consolidation behaviour of the tailings beach during deposition.

The starter embankment of TSF4 will be constructed and commissioned in two stages. First, the construction of Cell 1 to the extent shown on Drawing 61-37226-C005 will be completed. Cell 1 will be commissioned and receive tailings for nominally one year to allow adequate time to construct Cell 2 to the extent shown on Drawing 61-37226-C006. Cell 2 will then be filled to crest level within six months and deposition will then revert back to Cell 1. A detailed deposition plan has been developed for the starter embankment to show the migration of the decant pond and deposition scheduling (refer to Section 12).
The indicative raise schedule presented in Section 9.3 shows the TSF1 raises are planned to remain ahead TSF4 in elevation, based on the TSF1 conceptual design. This schedule should be reviewed in the detailed design of raises of TSF1 based on observed conditions.

Due to the site topography, Cell 2 will be raised first during the final filling of Cell 1 as shown Section 9.3. Cell 2 will always be raised ahead of Cell 1.
10. Embankment design

10.1 General

TSF4 is designed to be operated as a two separate cells to allow drying time prior to the subsequent raise construction and to facilitate consolidation.

10.2 Location

The site selection study for TSF4 was completed in the option study report (GHD, 2017).

The northern part of TSF4 is largely bounded by the southern embankment of the existing TSF1. TSF4 footprint extents was limited by the following site constraints:

- Existing tenement boundary to the south of the site
- Existing TSF3 to the west of the site
- Proposed waste dump to the north east of the site

A minimum 100 m buffer was maintained between proposed TSF4 embankment structures and the existing site constraints. The TSF4 eastern embankment design was aligned with the existing TSF1 eastern embankment.

10.3 Site preparation

10.3.1 Sterilisation drilling and geotechnical investigations

Sterilisation drilling across the entire TSF4 footprint was completed in June 2018. The sterilisation drill holes were backfilled with cuttings and gravel which may be disturbed/uncovered during the foundation preparation. Following clearing, grubbing and topsoil stripping, all borehole locations will be inspected by a geotechnical engineer. Any voids found on inspection will be backfilled with bentonite as required.

10.3.2 Foundation preparation

Clearing, grubbing and stripping topsoil of the TSF4 base to the extent of the clearing areas are shown on Drawing 61-37226-C003. The base of the foundation and impoundment will be ripped and compacted prior to excavation of any trenches.

The potentially liquefiable poorly graded grey sands (refer to Section 10.8.1) encountered on the surface or just below the topsoil in low lying areas along the valley will be removed from within the footprint of the perimeter and dividing embankment.

10.3.3 Impoundment reshaping

The sandy gravel and gravelly clay material found beneath the topsoil is considered to be suitable material for reshaping requirements of the TSF4 impoundment.

Following the construction of the starter embankments, the floor of both cells will require some backfilling of low points prior to installation of underdrainage systems (for underdrainage details refer to Drawings 61-37226-C005, 61-37226-C006 and Section 10.11). This will prevent rapid filling of low spots with slowly consolidating tailings.

All backfilling areas will be graded towards the centre of the cells to prevent ponding against the embankment.
10.4 Embankment materials

10.4.1 Zone 1A – Clay Embankment fill

The clay encountered at depths less than 5 m below ground level is considered suitable for Zone 1A construction material where required (refer to classification and compaction laboratory test results). Alternate sources for Zone 1A material were identified across the site.

The clay materials required for Zone 1A embankment fill will be sourced from the within the TSF4 impoundment and other locations across the site, such as mine stripping where suitable clayey material has been identified. Soil classification for the fine grained soil identified across the site as suitable for Zone 1A was completed as part of previous investigations. Classification results against acceptance criteria for 16 samples of available material are presented on Figure 10-1.

![Plasticity chart for fine grained soils around TSF4 site](image)

Figure 10-1 Plasticity chart for fine grained soils around TSF4 site

10.4.2 Zone 2A - Drainage blanket

A drainage blanket is proposed downstream of the clay core to limit any potential development of phreatic surface within the clay zone as described in Section 10.11.3.

The sand filter material required for the starter embankment will be sourced from within TSF2 impoundment from borrow areas close to the perimeter embankment. Suitable sandy material was identified during the 2014 geotechnical investigations within the impoundment adjacent to the perimeter embankment. The grading curves for available material are presented in Figure 10-2.
10.4.3 Mine waste rock

Mine waste rock from cut backs will be hauled from the pit expansion to the north-west of TSF4. The mine waste production rate is anticipated to be significantly greater than the minimum required for the raise schedule. The design allows flexibility for placement of the mine waste zone and the minimum elevation of the mine waste zone should remain at least two lifts ahead of the Zone 1A raise construction as shown on Drawing 61-37266-C033. Additional filling in horizontal layers beyond the minimum profiles given can be placed by the mine fleet as required.

10.4.4 General rock fill

General rock fill will be sourced from mine waste stockpiles. This zone will form a wider crest to provide access along the dividing embankment during raises. Alternative suitable materials can be considered for this zone if required.

10.5 Embankment geometry

10.5.1 Perimeter embankment

Both centreline and downstream construction methodologies were considered during the concept design phase. A centreline construction methodology was recommended due to various factors such as construction methodology, site boundary constraints and cost.

The typical perimeter embankment cross section is presented in Figure 10-3. The initial construction comprises a clay starter embankment (Zone 1A) with mine waste placed downstream. The starter embankment will be keyed into the hard clay foundation with a 4 m wide cutoff trench. An upstream toe underdrainage system will be constructed to lower the phreatic surface within the tailings and ensure the tailings are adequately drained for subsequent embankment raise construction.

The starter embankment will be constructed with a minimum mine waste rock crest width to allow for development of future raises in the stages to the minimum profiles shown on Drawing 61-37226-C033.
The waste rock downstream minimum slope of 3(H):1(V) will be adopted and the clay zone of the embankment will be constructed with upstream and downstream slopes of 2(H):1(H). To provide long term drainage of the Zone 1A starter embankment and foundation, a geofabric wrapped sand blanket was included on the foundation beneath the mine waste zone.

The clay zone (Zone 1A) will be constructed in nominally 3 m lifts and the mine waste rock profiles will remain a minimum of two lifts ahead of the clay zone.

Select mine waste (clean waste rock with <5% fines) layer will be placed in all areas where the embankment footprint extends to form a minimum 3 m thick drainage layer along the base of the mine waste.

Safety windrows 0.5 m in height were included to allow light vehicle access. Depending on the plant used to place the waste rock materials, windrows of greater height may be required on the mine waste crests.
Figure 10-3  Typical cross-section of TSF4 perimeter embankment
### 10.5.2 Divider embankment

The divider embankment will be constructed using staged centreline raising methodology. A typical cross section is presented on drawing 61-37226-C008. The initial construction comprises a clay starter embankment (Zone 1A) with general rockfill placed on the eastern side. The starter embankment will be keyed into the hard clay foundation with a 4 m wide cutoff trench. An underdrainage system was provided to lower the phreatic surface within the tailings and ensure the tailings surface is adequately drained for subsequent embankment raise construction.

Each subsequent lift will be nominally 3 m high and will be constructed along the same centreline with a 9 m wide clay zone. Cell 2 will be raised ahead of Cell 1 and therefore the raising of the dividing embankment will be undertaken in two stage as shown in alpha labels in Figure 10-4. The clay zone has therefore been designed to be on the Cell 2 side although Cell 1 will be commissioned first. The staged raising of the dividing embankment allows for continuous tailings deposition during construction by moving the pipeline to the raised zone as required.

A slope of 2(H):1(V) was included for the rockfill and both sides of the clay embankment.

Safety windrows 0.5 m in height were included to allow light vehicle access. Depending on the plant used to place materials, windrows of greater height may be required.

![Figure 10-4 Typical cross-section of TSF4 divider embankment](image-url)
10.5.3 TSF1 lining

Northern section of the proposed TSF4 will be located against the existing TSF1 southern embankment. To provide separation between facilities and control potential TSF1 seepage into series of future drains, a minimum 7.5 m wide clay lining is proposed as per drawing 61-37226-C009.

The lining will be locally widened to 15 m adjacent to divider embankment’s Cell 1 side due to the rockfill zone close to the Cell 1 decant pond (refer to Section 10.5.2).

The clay facing will be keyed into the hard clay foundation with a 4 m wide cutoff trench. A toe underdrainage system was included to lower the phreatic surface within the tailings.

Each subsequent raise will be 3 m high with a slope of 3(H):1(V) to match existing TSF1 geometry as presented on Figure 10-5.

![Figure 10-5 Proposed TSF1 lining](image)

Safety windrows 0.5 m in height were included to allow light vehicle access. Depending on the plant used to place materials, windrows of greater height may be required.

10.6 Seepage assessment

10.6.1 General

Seepage modelling was conducted to estimate the phreatic surface through the embankment for different cross-sections and to estimate the requirement for an underdrainage system. Geostudio Seep/W computer software was used to conduct the seepage analyses.

10.6.2 Material parameters

Permeability characteristics for the embankment materials consisting of clay and mine waste rock and the tailings and foundation materials were estimated using available geotechnical investigation test results and typical material parameters used for similar materials around the site based on previous field investigations.

Hydraulic conductivity parameters for the different materials assigned are presented in Table 10-1.
### Table 10-1  Hydraulic conductivity parameters used for seepage analyses

<table>
<thead>
<tr>
<th>Material</th>
<th>Cell 1 NE Hydraulic conductivity k (m/sec)</th>
<th>Cell 1 S Hydraulic conductivity k (m/sec)</th>
<th>Cell 2 NW Hydraulic conductivity k (m/sec)</th>
<th>Cell 2 S Hydraulic conductivity k (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay materials (Zone 1A)</td>
<td>1 x 10^{-8}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mine waste rock</td>
<td>1 x 10^{-4}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandy silt tailings</td>
<td>5 x 10^{-7}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandy tailings</td>
<td>1 x 10^{-4}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation Layer 1 (top)</td>
<td>3 x 10^{-7}</td>
<td>3 x 10^{-7}</td>
<td>3 x 10^{-7}</td>
<td>1 x 10^{-6}</td>
</tr>
<tr>
<td>Foundation Layer 2</td>
<td></td>
<td>6 x 10^{-7}</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation Layer 3</td>
<td>2.5 x 10^{-7}</td>
<td>2.5 x 10^{-7}</td>
<td>2.5 x 10^{-7}</td>
<td>2.5 x 10^{-7}</td>
</tr>
<tr>
<td>Foundation Layer 4</td>
<td>3 x 10^{-6}</td>
<td>2.5 x 10^{-7}</td>
<td>3 x 10^{-6}</td>
<td>2.5 x 10^{-7}</td>
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<tr>
<td>Foundation Layer 5</td>
<td>3 x 10^{-6}</td>
<td>2.5 x 10^{-7}</td>
<td>3 x 10^{-6}</td>
<td>2.5 x 10^{-7}</td>
</tr>
<tr>
<td>Foundation Layer 6</td>
<td>3 x 10^{-6}</td>
<td>2.5 x 10^{-7}</td>
<td>3 x 10^{-6}</td>
<td>2.5 x 10^{-7}</td>
</tr>
<tr>
<td>Bedrock</td>
<td></td>
<td></td>
<td>5 x 10^{-8}</td>
<td></td>
</tr>
</tbody>
</table>

#### 10.6.3 Scenarios considered

The starter and final embankments for all sections were modelled with and without upstream toe underdrainage to consider the impacts on the phreatic surface within the tailings and embankment and the seepage into the environment.

The seepage analysis results are included in Appendix E (refer to Figures 1.01 – 1.04 and Figures 2.01 – 2.04).

#### 10.6.4 Seepage results and discussion

The seepage analyses show a high phreatic surface is likely to develop against the perimeter embankment without an upstream toe underdrainage system. The seismic assessment of the sandy tailings was previously identified as potentially liquefiable (Section 10.8.3) which could cause settlement and slumping of the raised embankments. The underdrainage system will minimise the liquefiable zone that could cause slumping. Monitoring information will be required to confirm the operation of the underdrains and whether additional internal drainage is required for future raises. The underdrainage was sized to handle flows for the final raise, and additional underdrainage is not likely to be required for intermediate raises.

The interaction of phreatic line with stability is discussed in Section 10.7.

The material permeabilities for the foundation, embankment and tailings were assumed based on recent geotechnical investigations. Monitoring of actual seepage flows will be required and the seepage analysis should be updated based on the monitoring data from the initial filling of TSF4.

The estimated flows into the underdrainage from the seepage analysis were used to size the underdrainage sumps (refer to Section 10.11.5).

The estimated vertical seepage into the natural ground varies based on the topography and subsurface conditions at each section. The estimated vertical seepage flows over the total impoundment area of 1.4 sq km are presented on Table 10-2. This equates to less than 0.3 m/year vertical seepage rate into the foundation. The seepage is significantly higher for the cases without upstream toe underdrainage system due to the increased pressure head within the tailings.

The seepage flow estimates for the final embankment considers the topography by applying a lower head in areas where the natural ground is significantly higher.
Table 10-2  Vertical seepage flows into foundation

<table>
<thead>
<tr>
<th></th>
<th>Starter Embankment (m³/year)</th>
<th>Final Embankment (m³/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>With upstream toe underdrain</td>
<td>96,000</td>
<td>168,000</td>
</tr>
<tr>
<td>Without upstream toe underdrain</td>
<td>120,000</td>
<td>394,000</td>
</tr>
</tbody>
</table>

10.7 Stability assessment

10.7.1 General

Slope stability analyses were conducted using Geostudio Slope/W software in conjunction with Seep/W. Limit equilibrium computer models were developed adopting the Morgenstern-Price method of slices for all analyses. The figures showing the stability analysis results are included in Appendix E.

10.7.2 Model geometry and cases

Four typical cross sections were analysed for the various foundation and topographic profiles encountered on site. The sections were taken at locations where the embankment is locally at its highest. The foundation layers modelled were based on the material profiles encountered in the geotechnical boreholes closest to the cross sections. The cross sections analysed are described as follows and a typical example is shown in Figure 11-6:

- Cell 1 – South (CPT07)
- Cell 1 – North-east
- Cell 2 – South (CPT04 and CPT05)
- Cell 2 – North-west

![Figure 10-6 Final embankment geometry example - Cell 1 South](image-url)
The Factors of Safety (FOS) and shear strength parameters recommended by ANCOLD Guidelines on Tailings Dams (2012) are shown in Table 10-3. This table describes the failure slopes analysed for each loading case.

For post seismic cases, the target FoS is 1.1, being the mid-range of that described in ANCOLD Guidelines. The ANCOLD range is related to the confidence in the data being used for relevant analyses. This is considered further in Section 10.7.6.