Appendix H – Surface Water Technical Report
Table of Contents

Section 1 Introduction ........................................................................................................................................... 1-1
  1.1 Project Description .................................................................................................................................. 1-1
  1.2 Purpose ................................................................................................................................................. 1-1
  1.3 Report Structure ................................................................................................................................... 1-1

Section 2 Surface Water Resources .................................................................................................................. 2-1
  2.1 Climate .................................................................................................................................................... 2-1
    2.1.1 Comparison between Data Sources ................................................................................................. 2-2
  2.2 Catchment Hydrology .............................................................................................................................. 2-3
    2.2.1 Water Balance Modelling of Catchment Hydrology ....................................................................... 2-6
  2.3 Mine Impact on Catchment Hydrology .................................................................................................. 2-7
  2.4 Water Management Network ................................................................................................................ 2-10

Section 3 Mine Site Drainage ............................................................................................................................ 3-0
  3.1 Stormwater Management ......................................................................................................................... 3-0
  3.2 Erosion and Sediment Control ............................................................................................................... 3-0
  3.3 Haul Road Cross-Drainage ....................................................................................................................... 3-2
    3.3.1 Rational Method Calculations .......................................................................................................... 3-2
    3.3.2 Culvert Sizing ..................................................................................................................................... 3-4

Section 4 Flood Assessment ................................................................................................................................... 4-1
  4.1 Hydrologic assessment .............................................................................................................................. 4-1
    4.1.1 Hydrologic Model Build .................................................................................................................... 4-2
    4.1.2 Regional Flood Frequency Analysis ................................................................................................. 4-5
    4.1.3 Hydrologic Model Calibration ........................................................................................................ 4-9
    4.1.4 Treatment of Extreme Rainfall Events ............................................................................................. 4-10
    4.1.5 Hydrologic Model Results ............................................................................................................... 4-10
  4.2 Hydraulic Assessment .............................................................................................................................. 4-12
    4.2.1 Modelling Software ........................................................................................................................ 4-12
    4.2.2 Survey Data ..................................................................................................................................... 4-12
    4.2.3 Model Setup ..................................................................................................................................... 4-12
    4.2.4 Boundary Conditions ....................................................................................................................... 4-13
    4.2.5 Methodology ................................................................................................................................... 4-14
    4.2.6 Results and Discussion ..................................................................................................................... 4-14

List of Figures

Figure 1-1 Bauxite Hills Mine Location ........................................................................................................ 1-2
Figure 2-1 Graph of average monthly rainfall and evaporation for Data drill ........................................... 2-2
Figure 2-2 Comparison of SILO Data to Gauge Data .................................................................................. 2-3
Figure 2-3 Ducie Basin Catchment Map ...................................................................................................... 2-5
Figure 2-4 Bauxite Hills Proposed Water Management Network .............................................................. 2-11
Figure 4-1 RORB Runoff Routing Model ...................................................................................................... 4-1
Figure 4-2 RORB Sub-catchment Delineation ............................................................................................. 4-4
Figure 4-3 Watson River (923001A) FFA output – LPIII Distribution ......................................................... 4-6
Figure 4-4 Dulhunty TM (926002A) FFA output – LPIII Distribution ............................................................ 4-6
Figure 4-5 Moreton TM (925001A) FFA output – LPIII Distribution .............................................................. 4-7
List of Tables

Table 2-1 Data drill average monthly rainfall and evaporation ................................................................. 2-1
Table 2-2 Gauge Information ....................................................................................................................... 2-2
Table 2-3 AWBM Calibration Parameters ................................................................................................... 2-6
Table 2-4 Water Balance Model Partitioning of Annual Rainfall .............................................................. 2-6
Table 2-5 AWBM Land Use Parameters ..................................................................................................... 2-7
Table 2-6 AWBM Land Use Water Budget Results .................................................................................. 2-8
Table 2-7 AWBM Land Use Partial Areas – Skardon River ..................................................................... 2-8
Table 2-8 Potential Impact on Water Budget – Skardon River .................................................................. 2-8
Table 2-9 AWBM Land Use Partial Areas – Namaleta Creek ................................................................. 2-9
Table 2-10 Potential Impact on Water Budget – Namaleta Creek ............................................................ 2-9
Table 2-11 Mine Water Demands ........................................................................................................... 2-10
Table 3-1 Local Catchment Areas ............................................................................................................. 3-2
Table 3-2 Coefficients of Runoff ............................................................................................................... 3-3
Table 3-3 Rational Method Peak Flow ....................................................................................................... 3-3
Table 3-4 Culvert Sizing ............................................................................................................................ 3-4
Table 4-1 CRC-FORGE Design Point Rainfall Intensities (mm/h) ............................................................. 4-2
Table 4-2 Uncalibrated RORB Peak Outlet Flow; Kc = 41.18, m = 0.8 ................................................... 4-3
Table 4-3 FFA Stream Gauge Details ......................................................................................................... 4-5
Table 4-4 FFA Flow Comparison to Uncalibrated RORB Output (m$^3$/s) .............................................. 4-8
Table 4-5 RORB Calibration to Regional Regression Equation Results .................................................. 4-9
Table 4-6 MIKE21 Model Dimensions ..................................................................................................... 4-13
Table 4-7 Tailwater Components – Storm Tide Condition ..................................................................... 4-13
Table 4-8 Peak Flood Depths at Locations of Interest ............................................................................. 4-15
Table 4-9 Peak Water Surface Elevations at Locations of Interest ........................................................... 4-15
Table 4-10 Peak Velocities at Locations of Interest ............................................................................... 4-15
Table 4-11 Tidal Levels at Locations of Interest ...................................................................................... 4-16

Appendices

Appendix A - Disclaimer and Limitations
Appendix B – Flood Mapping Results
Appendix C – Mine Drainage Plan
Document History and Status

<table>
<thead>
<tr>
<th>Revision</th>
<th>Date Issued</th>
<th>Reviewed By</th>
<th>Approved By</th>
<th>Date Approved</th>
<th>Revision Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>24 April 2015</td>
<td>MVW</td>
<td>MI</td>
<td>24 April 2015</td>
<td>DRAFT</td>
</tr>
<tr>
<td>B</td>
<td>05 May 2015</td>
<td>EOB</td>
<td>MI</td>
<td>15 May 2015</td>
<td>DRAFT</td>
</tr>
<tr>
<td>C</td>
<td>18 May 2015</td>
<td>MVW</td>
<td>MI</td>
<td>18 May 2015</td>
<td>FINAL</td>
</tr>
<tr>
<td>D</td>
<td>12 June 2015</td>
<td>MVW</td>
<td>MI</td>
<td>12 June 2015</td>
<td>FINAL</td>
</tr>
<tr>
<td>E</td>
<td>22 June 2015</td>
<td>MVW</td>
<td>MI</td>
<td>22 June 2015</td>
<td>FINAL</td>
</tr>
</tbody>
</table>

Distribution of Copies

<table>
<thead>
<tr>
<th>Version</th>
<th>Date Issued</th>
<th>Quantity</th>
<th>Electronic</th>
<th>Issued To</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>22 April 2015</td>
<td>1</td>
<td>PDF</td>
<td>Metro Mining Limited</td>
</tr>
<tr>
<td>C</td>
<td>19 May 2015</td>
<td>1</td>
<td>PDF</td>
<td>Metro Mining Limited</td>
</tr>
<tr>
<td>D</td>
<td>12 June 2015</td>
<td>1</td>
<td>PDF</td>
<td>Metro Mining Limited</td>
</tr>
<tr>
<td>E</td>
<td>22 June 2015</td>
<td>1</td>
<td>PDF</td>
<td>Metro Mining Limited</td>
</tr>
</tbody>
</table>

Printed: 23 June 2015
Last Saved: 23 June 2015 09:18 AM
File Name: Surface Water Assessment_RevE_20150618.docx
Authors: Tim McConnell; Evan O’Brien
Project Manager: Dr Craig Streatfield
Client: Metro Mining Limited
Document Title: Bauxite Hills – Surface Water Assessment
Document Version: Final
Project Number: BES140115.02
Section 1  Introduction

1.1  Project Description

Metro Mining Pty Ltd (Metro Mining) proposes to develop the Bauxite Hills Mine (the 'Mine') within Exploration Permit for Minerals (EPM) 15376 and 16899, located approximately 100 km north of Weipa on the Western Cape York, Queensland.

The bauxite resource lies in two main plateaus (referred to as BH1 and BH6) between the Skardon and Ducie Rivers, five kilometres from the existing port at Skardon River. The location of the Mine in the regional context is shown in Figure 1-1.

The Mine is a proposed open-cut bauxite mine, wholly located within three Mining Lease (ML) areas MLA 20676, MLA 20688 and MLA 20689. The Mine will involve the planned extraction of under two million tonnes per annum (Mtpa) of wet ore. Typical truck and shovel mining methods will be utilised to progressively develop the open-cut pits over 27 years. The mine development will include the development of haul roads and a barge loading facility with product ore to be barged to a trans-shipment point located approximately 12 km off shore in the Gulf of Carpentaria.

1.2  Purpose

The purpose of this report is to characterise the baseline surface water resources at the Project location and determine potential environmental impacts and mitigation measures. The report also aims to describe the mine water management infrastructure and processes, define the flood immunity of mine infrastructure and detail a mine site drainage concept.

1.3  Report Structure

The report structure and contents is as follows:

- **Section 2 Surface Water Resources**
  - Defines the Project climate and catchment hydrology;
  - Describes the likely mine operation impacts to catchment hydrology; and
  - Describes the mine water management network.

- **Section 3 Mine Site Drainage**
  - Describes stormwater management practices to be employed;
  - Determines the treatment of haul road crossing of watercourses; and
  - Describes the approach to erosion and sediment control.

- **Section 4 Flood Assessment**
  - Described the hydrologic modelling approach and hydrograph results;
  - Describes the hydraulic modelling approach and result; and
  - Discusses flood behaviour and potential impacts of mine processes on flooding.
Section 2  Surface Water Resources

2.1  Climate

Long term rainfall and evaporation data were collected from the SILO Climate Data website\(^1\) at the following coordinate location:

- Latitude: 11.80 degrees South
- Longitude: 142.10 degrees East

These coordinates are the location of Mine Pit 1.

SILO represents a gridded dataset based on records provided by the Bureau of Meteorology. The data is then processed to fill gaps in data and produce a spatially complete dataset. Table 2-1 and Figure 2-1 summarise monthly averages of the SILO long term data.

Table 2-1 Data drill average monthly rainfall and evaporation

<table>
<thead>
<tr>
<th>Month</th>
<th>Rainfall (mm)</th>
<th>Evaporation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>434.0</td>
<td>154.3</td>
</tr>
<tr>
<td>February</td>
<td>459.3</td>
<td>126.4</td>
</tr>
<tr>
<td>March</td>
<td>370.4</td>
<td>143.9</td>
</tr>
<tr>
<td>April</td>
<td>114.7</td>
<td>155.9</td>
</tr>
<tr>
<td>May</td>
<td>23.3</td>
<td>159.5</td>
</tr>
<tr>
<td>June</td>
<td>8.5</td>
<td>151.9</td>
</tr>
<tr>
<td>July</td>
<td>4.3</td>
<td>169.2</td>
</tr>
<tr>
<td>August</td>
<td>3.0</td>
<td>193.2</td>
</tr>
<tr>
<td>September</td>
<td>1.7</td>
<td>216.8</td>
</tr>
<tr>
<td>October</td>
<td>13.9</td>
<td>239.9</td>
</tr>
<tr>
<td>November</td>
<td>66.1</td>
<td>219.2</td>
</tr>
<tr>
<td>December</td>
<td>208.9</td>
<td>189.0</td>
</tr>
<tr>
<td><strong>Annual Average Total</strong></td>
<td><strong>1708.1</strong></td>
<td><strong>2119.2</strong></td>
</tr>
</tbody>
</table>

Some general trends can be observed from the SILO data such as:

- A distinct wet season between the months December to March with between 200mm to more than 450mm monthly average rainfall;
- A distinct dry season between the months April to November with less than 25mm mean monthly rainfall between the months May through October; and
- High evaporation rates showing an inverse trend to rainfall, reaching a trough in February and peaking in October.

### 2.1.1 Comparison between Data Sources

Due to the gridded and somewhat synthetic nature of the long term SILO data, a comparison with raw gauged data from sites within 100 km was prepared to assess:

- The validity of long term SILO climatic data; and
- Spatial variability of rainfall near the Project.

The gauging sites selected for comparison with the data acquired from SILO are shown below in Table 2-2.

<table>
<thead>
<tr>
<th>Gauge</th>
<th>Record Period</th>
<th>Location</th>
<th>Distance from Mine</th>
<th>Gauge Owner</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dulhunty River</td>
<td>25 years</td>
<td>Inland</td>
<td>35 km east</td>
<td>DNRM</td>
</tr>
<tr>
<td>Skardon River</td>
<td>5 years</td>
<td>Coastal</td>
<td>5 km north, 9km east</td>
<td>BOM</td>
</tr>
<tr>
<td>Bramwell</td>
<td>12 years</td>
<td>Inland</td>
<td>30 km south, 60km west</td>
<td>BOM</td>
</tr>
<tr>
<td>Weipa Eastern Ave</td>
<td>100 years</td>
<td>Coastal</td>
<td>100km south</td>
<td>BOM</td>
</tr>
</tbody>
</table>

A comparison of mean monthly rainfall values between the gauges listed in above and the SILO data, is presented in Figure 2-2. The graph indicates little spatial variability of rainfall and good
agreement between gauge records and data acquired through SILO. The only gauge that produces a significantly different data trend is the Skardon River gauge during the month of January. This variance is most likely due to the relatively small period of record (5 years) compared to other gauges such as Weipa Eastern Ave (100 years).

Figure 2-2 Comparison of SILO Data to Gauge Data

2.2 Catchment Hydrology

The majority of the Project area is located within the Skardon River catchment, which forms approximately 350 km² of the Ducie drainage basin and is bounded by the Ducie River Catchment to the South and the McDonald River catchment to the north (refer Figure 2-3). The Skardon River is tidally influenced and discharges to the Gulf of Carpentaria. A network of smaller ephemeral streams drain the upper reaches of the catchment. There are a series of swamps within the catchment; most notable with respect to the Project is Bigfoot Swamp, which is situated approximately 1.5 km west of the main haul road leading to the barge loading facilities.

The Project proposed pit locations are situated either side of the Skardon River on elevated Bauxite plateaus. Partial pit areas (BH6) and the camp facilities are proposed within the adjoining Namaleta Creek catchment to the south. The barge facilities are proposed on the bank of the southern Skardon River branch.

A rainfall-runoff relationship was established for the Ducie Basin through the calibration of Boughton’s Australian Water Balance Model (AWBM) parameters via the Rainfall Runoff Library platform (CRC for Catchment Hydrology). The AWBM aims to determine daily runoff from rainfall
and potential evapotranspiration data by conceptualising surface and baseflow stores and calculating the excess from the stores released as runoff.
Calibration of the catchment specific AWBM parameters was attempted at the Dulhunty TM gauging station\(^2\), which is the only gauging station available in the Ducie Basin. It is situated approximately 35 km east of the Project site. The calibration was achieved by providing the best fit between observed and calculated runoff over the calibration and verification periods.

The results of the AWBM parameter calibration are summarised below in Table 2-3, where:

- BFI is the baseflow index or ratio of baseflow to total flow;
- KBase is the baseflow recession constant where \((1 - KBase)\) multiplied by the baseflow store is the rate of depletion from the store contributing to total runoff;
- KSurf is the surface recession constant where \((1 - KSurf)\) multiplied by the surface store is the rate of depletion from the store contributing to total runoff;
- C1-C3 represent surface storage capacities; and
- A1-A3 represent partial areas of the C1-C3 storage capacities.

<table>
<thead>
<tr>
<th>AWBM Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>BFI</td>
<td>0.65</td>
</tr>
<tr>
<td>KBase</td>
<td>0.99</td>
</tr>
<tr>
<td>KSurf</td>
<td>0.68</td>
</tr>
<tr>
<td>C1 (mm)</td>
<td>17.3</td>
</tr>
<tr>
<td>C2 (mm)</td>
<td>177.1</td>
</tr>
<tr>
<td>C3 (mm)</td>
<td>354.2</td>
</tr>
<tr>
<td>A1</td>
<td>0.134</td>
</tr>
<tr>
<td>A2</td>
<td>0.433</td>
</tr>
<tr>
<td>A3</td>
<td>0.433</td>
</tr>
</tbody>
</table>

### 2.2.1 Water Balance Modelling of Catchment Hydrology

An AWBM was constructed within the GoldSim modelling environment with the objective of determining the likely partitioning of annual rainfall into evaporation, baseflow and surface runoff components. Historical daily rainfall and evaporation time series data (refer Section 2.1 for information on climate data used for the Project) were applied to the model and 125 simulations were run, each a single calendar year in length, covering the period 1889 to 2014. The results were analysed to create a probability distribution, the mean (50\(^{th}\) percentile probability) results of which are presented in Table 2-4.

<table>
<thead>
<tr>
<th>Water Budget Component</th>
<th>Mean Rainfall Segregation (% of total rainfall)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Evapotranspiration</td>
<td>53.3</td>
</tr>
<tr>
<td>Surface Runoff</td>
<td>20.3</td>
</tr>
<tr>
<td>Baseflow</td>
<td>26.4</td>
</tr>
</tbody>
</table>

2.3 Mine Impact on Catchment Hydrology

The mine impact on catchment hydrology was assessed via the AWBM method using the assessment of the “natural” catchment described under Section 2.1 as a baseline in which to measure impacts against. AWBM parameters were varied from the baseline to represent likely catchment characteristics exhibited by different land use types; namely, hardstand, open pit mining and rehabilitated areas. The natural AWBM catchment parameters were calibrated to recorded stream gauge and rainfall records as described in Section 2.1. AWBM parameters for hardstand, open pit mining and rehabilitation areas were applied based on industry-accepted values for these land types. In the absence of any recorded data against which to calibrate, conservatism was applied to the model parameters to ensure the full impact of mining was realised on the partitioning of rainfall into evaporative losses, surface runoff and base flow.

The AWBM parameters adopted for the various land use types are summarised in Table 2-5. The following trends, with respect to the natural catchment parameters, have been applied in deriving AWBM values for mine impacted areas:

- **Hardstand – Haul roads, mine camp and mine industrial areas:**
  - Increase in surface runoff and decrease in baseflow due to compaction of the ground surface and/or construction of impermeable surfaces; and
  - Decrease in surface storage capacity.

- **Open Pit Mining – Active mine areas excluding haul roads:**
  - Increase in baseflow due to storage of direct rainfall within open mine pits and infiltration into the bauxite layer; and
  - Increased evaporative losses due to absence of vegetation and ponding of water in open mine pits.

- **Rehabilitation – rehabilitated mine areas greater than 10 years old:**
  - Slight increase in baseflow due to the final void reducing surface runoff; and
  - Slight increase in surface storage capacity due to less consolidation of in filled void material.

<table>
<thead>
<tr>
<th>AWBM Parameter</th>
<th>Natural</th>
<th>Hardstand</th>
<th>Open Pit Mining</th>
<th>Rehabilitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF1</td>
<td>0.65</td>
<td>0.05</td>
<td>0.90</td>
<td>0.70</td>
</tr>
<tr>
<td>Kbase</td>
<td>0.99</td>
<td>1.00</td>
<td>0.99</td>
<td>0.99</td>
</tr>
<tr>
<td>Ksurf</td>
<td>0.68</td>
<td>0.10</td>
<td>0.10</td>
<td>0.68</td>
</tr>
<tr>
<td>C1 (mm)</td>
<td>17</td>
<td>5</td>
<td>5</td>
<td>20</td>
</tr>
<tr>
<td>C2 (mm)</td>
<td>177</td>
<td>10</td>
<td>10</td>
<td>200</td>
</tr>
<tr>
<td>C3 (mm)</td>
<td>354</td>
<td>40</td>
<td>500</td>
<td>400</td>
</tr>
<tr>
<td>A1</td>
<td>0.134</td>
<td>0.134</td>
<td>0.134</td>
<td>0.134</td>
</tr>
<tr>
<td>A2</td>
<td>0.433</td>
<td>0.433</td>
<td>0.433</td>
<td>0.433</td>
</tr>
</tbody>
</table>

The partitioning of annual rainfall for the various land uses is shown in Table 2-6. The table shows that open mine areas exhibit approximately a 30% increase in annual baseflow volume compared to natural catchments; correspondingly, there is a significant decrease in predicted surface runoff. Hardstand areas exhibit minimal baseflow and a more than two-fold increase in surface runoff.
compared to natural catchments. The rehabilitation land use shows an increase in baseflow and decrease in surface runoff when compared to natural catchments due to the final void left by the mining.

Table 2-6 AWBM Land Use Water Budget Results

<table>
<thead>
<tr>
<th>Water Budget Component</th>
<th>Natural</th>
<th>Open Pit Mining</th>
<th>Hardstand</th>
<th>Rehabilitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Evapotranspiration (%)</td>
<td>53.3%</td>
<td>58.6%</td>
<td>46.1%</td>
<td>55.3%</td>
</tr>
<tr>
<td>Surface Runoff (%)</td>
<td>20.3%</td>
<td>4.2%</td>
<td>51.3%</td>
<td>13.4%</td>
</tr>
<tr>
<td>Baseflow (%)</td>
<td>26.4%</td>
<td>37.2%</td>
<td>2.6%</td>
<td>31.3%</td>
</tr>
</tbody>
</table>

The partial areas of land use types at various stages of the mine life are summarised in Table 2-7 and 2-9 for Skardon River and Namaleta Creek, respectively. These were calculated based on the total local catchment area that drains to the mine affected areas. By applying the partial areas to the partitioning of annual rainfall for the various land uses shown in Table 2-6, an overall impact on the water budget due to mining activities could be estimated. The results of this assessment for the two drainage basins are shown in Table 2-8 and Table 2-10. The overall impact on the water budget is shown to be minor due to the small scale of the mine affected areas (i.e. hardstand, open pit and rehabilitation) relative to the local catchments in which they reside.

It is important to note that although the overall mine impact on the water budget is negligible on the local catchment scale, localised impacts during mining operations may be more pronounced. However, it is also likely that the partitioning of rainfall into runoff and baseflow will be recombined as total runoff re-entering the Skardon River/Namaleta Creek, thus reducing the total impact of varying recharge rates as a result of mining operations on the water environment and supported ecosystems. Both the Skardon River and Namaleta Creek are tidally influenced adjacent to the Project area and thus not as sensitive as an ephemeral or perennial freshwater system to variance in total runoff volume or duration of runoff entering the system.

Table 2-7 AWBM Land Use Partial Areas – Skardon River

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Pre-Mining</th>
<th>10-year mine plan</th>
<th>20-year mine plan</th>
<th>27-year mine plan</th>
<th>Post Mine Closure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural</td>
<td>103.9</td>
<td>97.3</td>
<td>92.6</td>
<td>82.9</td>
<td>89.5</td>
</tr>
<tr>
<td>Open Mining</td>
<td>-</td>
<td>5.8</td>
<td>4.6</td>
<td>3.1</td>
<td>-</td>
</tr>
<tr>
<td>Hardstand</td>
<td>-</td>
<td>0.8</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>Rehabilitation</td>
<td>-</td>
<td>-</td>
<td>5.8</td>
<td>10.4</td>
<td>13.5</td>
</tr>
<tr>
<td>Total</td>
<td>103.9</td>
<td>103.9</td>
<td>103.9</td>
<td>103.9</td>
<td>103.9</td>
</tr>
</tbody>
</table>

Table 2-8 Potential Impact on Water Budget – Skardon River

<table>
<thead>
<tr>
<th>Water Budget Component</th>
<th>Pre-Mining</th>
<th>10-year mine plan</th>
<th>20-year mine plan</th>
<th>27-year mine plan</th>
<th>Post Mine Closure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Evapotranspiration (%)</td>
<td>53.3%</td>
<td>53.5%</td>
<td>53.6%</td>
<td>53.6%</td>
<td>53.5%</td>
</tr>
<tr>
<td>Surface Runoff (%)</td>
<td>20.3%</td>
<td>19.6%</td>
<td>19.5%</td>
<td>19.3%</td>
<td>19.7%</td>
</tr>
<tr>
<td>Baseflow (%)</td>
<td>26.4%</td>
<td>26.8%</td>
<td>27.0%</td>
<td>27.0%</td>
<td>26.8%</td>
</tr>
<tr>
<td>Total (%)</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
</tbody>
</table>
### Table 2-9 AWBM Land Use Partial Areas – Namaleta Creek

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Pre-Mining</th>
<th>10-year mine plan</th>
<th>20-year mine plan</th>
<th>27-year mine plan</th>
<th>Post Mine Closure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural</td>
<td>47.2</td>
<td>47.2</td>
<td>46.1</td>
<td>43.9</td>
<td>43.9</td>
</tr>
<tr>
<td>Open Mining</td>
<td>-</td>
<td>-</td>
<td>1.0</td>
<td>2.3</td>
<td>-</td>
</tr>
<tr>
<td>Hardstand</td>
<td>-</td>
<td>-</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Rehabilitation</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.0</td>
<td>3.3</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>47.2</strong></td>
<td><strong>47.2</strong></td>
<td><strong>47.2</strong></td>
<td><strong>47.2</strong></td>
<td><strong>47.2</strong></td>
</tr>
</tbody>
</table>

### Table 2-10 Potential Impact on Water Budget – Namaleta Creek

<table>
<thead>
<tr>
<th>Water Budget Component</th>
<th>Pre-Mining</th>
<th>10-year mine plan</th>
<th>20-year mine plan</th>
<th>27-year mine plan</th>
<th>Post Mine Closure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Evapotranspiration (%)</td>
<td>53.3%</td>
<td>53.3%</td>
<td>53.4%</td>
<td>53.6%</td>
<td>53.4%</td>
</tr>
<tr>
<td>Surface Runoff (%)</td>
<td>20.3%</td>
<td>20.3%</td>
<td>20.0%</td>
<td>19.4%</td>
<td>19.9%</td>
</tr>
<tr>
<td>Baseflow (%)</td>
<td>26.4%</td>
<td>26.4%</td>
<td>26.6%</td>
<td>27.0%</td>
<td>26.7%</td>
</tr>
<tr>
<td><strong>Total (%)</strong></td>
<td><strong>100.0</strong></td>
<td><strong>100.0</strong></td>
<td><strong>100.0</strong></td>
<td><strong>100.0</strong></td>
<td><strong>100.0</strong></td>
</tr>
</tbody>
</table>
2.4 Water Management Network

A schematic of the proposed water management network for the Project is shown in Figure 2-4. The proposed water supply is via shallow and/or deep aquifer bores to meet a total annual demand of 200 ML. Assuming 275 days of operation per year and 20 hours of daily pumping time, a total yield of 10 L/s is required from the combined bores. Polyethylene storage tanks are proposed to buffer between supply from the bores and operational demand. The polyethylene tanks will include a peaking factor to accommodate temporary increases in water demand and to protect against irregularities in supply from the bores. A peaking factor of between 1 day and 1 week would equate to polyethylene tanks with a total storage of between 1 ML and 5 ML. The number of tanks required will be based on balancing the need to locate water storage near the water use versus trucking water to where it is used.

The potential breakdown of mine water demands which must be satisfied by the water supply system, is summarised in Table 2-9 the water supply system must satisfy. The majority of water use (150 ML/yr) is raw water for dust suppression of the dump station, haul roads and stockpiles, as well as for washdown of the crusher plant and conveyor system.

A potable water supply to the camp and mine industrial area of approximately 11 ML/yr is required to meet the standard outlined in the *Australian Drinking Water Standard Guidelines* (NHMRC, 2004). Field investigations and laboratory testing conducted by CDM Smith in November 2014 and March 2015 indicate that the shallow aquifer water quality is suitable for potable use. Chemical dosing may be required to control pH levels and provide disinfection. Two potable use water tanks will be required; one at the mine camp and the other at the mine industrial area. The main potable use tank will be located near the mine camp, as this is the main source of potable demand. A potable water pipeline or truck transport will be required to transport potable water to the storage tank located at the mine industrial area.

A sewage treatment plant is proposed to be located near the mine camp. Wastewater produced from the mine industrial area will be stored and periodically trucked and transported to the sewage treatment plant. Effluent and sludge waste streams will be appropriately treated and discharged to surface or used as mulching media, respectively.

Table 2-11 Mine Water Demands

<table>
<thead>
<tr>
<th>Description</th>
<th>Annual Demand (ML)</th>
<th>Water Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operations (crusher; truck fill for dust suppression)</td>
<td>150</td>
<td>Raw</td>
</tr>
<tr>
<td>Mine Camp (75 person camp)</td>
<td>10</td>
<td>Potable</td>
</tr>
<tr>
<td>Fire Fighting (poly tank spare capacity)</td>
<td>5</td>
<td>Raw</td>
</tr>
<tr>
<td>Mine Industrial Area (Workshop / Washdown)</td>
<td>35</td>
<td>Raw / Potable*</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>200</strong></td>
<td><strong>-</strong></td>
</tr>
</tbody>
</table>

*1 ML/yr potable supply to the Mine Industrial Area assuming 40 L/person/day
The water management network allows for potential reuse of water collected in sumps, ponds and slots. Allowance for reuse of water has not been incorporated into the demand analysis, however such an allowance would reduce the amount of water abstracted from bores. The main function of the sumps, ponds and slots is to capture sediment laden runoff for sediment removal prior to release to the existing environment. Oil/water separators are proposed for vehicle wash and workshop areas prior to release or reuse of water.

Fire water supply will be provided through storage in polyethylene tanks at suitable locations around the mine lease. A total of 5 ML has provisionally been included for the purpose of this water resources assessment. It is anticipated that these stores be replenished post use and that the total volume is available for firefighting activities during operations.
3.1 Stormwater Management

All stormwater runoff capturing devices, namely sediment ponds and drainage sumps, will be sized based on the 10 year Average Recurrence Interval (ARI), 24 hour rainfall event in keeping with The Department of Environment and Heritage Protection Stormwater Guideline (2014). The sediment pond functions to capture runoff generated from the stockpile/ore dump station area. Further provision has been provided for sediment capture via a wharf/conveyor drainage sump, which pumps return water to the sediment pond for treatment and possible reuse. The sump captures sediment laden runoff from the conveyor system, as a sediment pond cannot practically be located between the sediment producing source and the Skardon River receiving environment. A mine industrial area drainage slot is proposed to capture runoff from truck wash and workshop areas after oil has been separated. Bauxite is considered a non-hazardous material and thus water captured by the slot can either be reused within the mine industrial area or released to the environment under EA conditions.

Runoff from the network of haul roads will be captured in table drains and turned out to vegetated areas via spoon drains at regular intervals. Due to the generally flat topography it is not anticipated that the spoon and table drains will carry significant sediment load. In areas of steeper grade, sediment transport can be effectively managed by turning out the table drains more regularly before excessive velocities develop.

Where haul roads cross watercourses either a culvert or causeway arrangement will be provided. Sediment removal devices will be incorporated in the watercourse crossing design, where appropriate, to reduce sediment loads entering the system.

Mine pit areas are generally located on plateaus and thus are naturally inward draining. Due to the depth of the mine pits and fast infiltration rates through the bauxite layer, the mine pit areas act as a self-draining sediment trap for runoff from disturbed mine areas. Should the pit not be capable of containing the 10 year, 24 hour duration rainfall volume (as stipulated for sediment pond design) clean water diversion will first be considered; followed by provision of an appropriately sized sediment basin.

Several of the mine pits to the west of the Skardon River (BH6) have contributing clean water catchments. To separate clean and dirty water runoff, overburden from pit stripping activities will be used to construct a clean water diversion bund to the extents shown in Appendix C. Should there not be sufficient overburden to construct the clean water diversion bund to an appropriate height, an accompanying diversion drain will be excavated parallel to the bund on the upslope side of the bund.

3.2 Erosion and Sediment Control

Erosion and sediment control is critical to the successful management of surface waters. It is considered most effective if incorporated into initial project planning, reviewed at project commencement, and monitored during construction and mining activities.

---

Land disturbance caused by mining activities, including those proposed to be undertaken at Metro Mining, increases the potential for erosion and subsequent sediment transport. Higher erosion rates during construction work trigger the need to implement erosion and sediment control measures so as to meet locally accepted guidelines and, state and Commonwealth legislation, including the *Environmental Protection Act 1994* and the International Erosion Control Association (IECA). Increased erosion could be expected onsite during the wet season, however the mine itself is proposed to be operational only during the dry season. The erosion and sediment control devices, such as sediment ponds will be designed to freely flow under gravity and will be monitored by the wet season site caretakers.

Numerous activities that will be undertaken give way to greater rates of erosion than those which are naturally occurring, including, but not limited to; land clearing, soil stripping, mine excavation, and haul road construction. Elevated sediment concentrations could also be expected to occur in stormwater runoff from stockpile areas, and such runoff would be directed to appropriate sediment control structures.

Sediment laden runoff from the barge loading facilities has the potential to cause higher levels of turbidity in the Skardon River. Sediment from ship loading activities will be collected via a sump located at the shore and pumped back to the stockpile sediment dam for treatment.

Vegetation corridors will further assist in minimising sediment transport from disturbed areas to adjacent watercourses by slowing flow velocities and stabilising deposited sediment. Vegetation corridors and clearing set-back distances from watercourses will be employed in compliance with the relevant Queensland Government’s Regional Vegetation Management Codes.

Stripped topsoil will be maintained in accordance with IECA best practice guidelines, for use in the rehabilitation effort. Areas disturbed by mining activities will be progressively rehabilitated and at the end of mine life, mining related infrastructure will be decommissioned and rehabilitated back to an appropriate final land form, as agreed with the Traditional Owners and the land owners.

A certified erosion and sediment control plan, prepared by a Certified Professional in Erosion and Sediment Control, will be prepared prior to any construction or mining-related activities commencing.
3.3 Haul Road Cross-Drainage

The proposed haul road crosses a number of creeks and gullies, and therefore cross-drainage infrastructure, (such as culverts, floodways, or bridges) will be required to convey runoff beneath the haul road at such locations. In this section of the report the local catchments reporting to the haul road are examined, likely peak runoff values are calculated using the Rational Method, and concept culvert sizes are provided (refer Error! Reference source not found. and Appendix C). For the purpose of this assessment the two year ARI event was considered a sufficient culvert sizing design event to adopt considering that the operations are proposed for during the dry season. It is recommended that the road crossings be designed as a low flow culvert and floodway arrangement to efficiently pass flows greater than the two year ARI and reduce environmental impacts of filling within creek crossings.

3.3.1 Rational Method Calculations

The probabilistic Rational Method provides an estimate of peak discharge for a given design storm frequency, and is represented as follows:

\[ Q_y = \frac{(C_y \times I_y \times A)}{360} \]

Where:

- \( Q_y \) = Peak discharge (m\(^3\)/s);
- \( C \) = Coefficient of Runoff;
- \( I \) = Design Rainfall Intensity (mm/h);
- \( A \) = Catchment Area (ha); and,

The subscript ‘y’ denotes the particular ARI under consideration.

Catchment analysis was undertaken in ArcGIS to delineate the various local catchments reporting to the haul road. Nine concentrated flowpaths were identified as crossing the haul road, and nine corresponding contributing catchments were mapped. Catchment sizes are shown below in Table 3-1.

<table>
<thead>
<tr>
<th>Catchment Name</th>
<th>Area (ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>61</td>
</tr>
<tr>
<td>L2</td>
<td>83</td>
</tr>
<tr>
<td>L3</td>
<td>48</td>
</tr>
<tr>
<td>L4</td>
<td>122</td>
</tr>
<tr>
<td>L5</td>
<td>419</td>
</tr>
<tr>
<td>L6</td>
<td>3,419</td>
</tr>
<tr>
<td>L7</td>
<td>348</td>
</tr>
<tr>
<td>L8</td>
<td>140</td>
</tr>
<tr>
<td>L9</td>
<td>4,908</td>
</tr>
</tbody>
</table>
The Coefficient of Runoff ‘C’ and rainfall intensity ‘I’ were calculated following the guidance provided in the *Queensland Urban Drainage Manual* (QUDM). The Coefficient of Runoff is a dimensionless factor designed to account for the various natural processes that intercept or otherwise prevent precipitation from turning into runoff. It takes into account the degree of pervious surfaces in the catchment, the type of ground cover, and an estimate of the soil porosity. It varies in accordance with storm frequency (i.e. ARI) as per the equation below:

\[
C_y = F_y \times C_{10}
\]

Where:

- \(C\) = Coefficient of Runoff;
- \(F\) = Frequency Adjustment Factor;
- \(C_{10}\) = Coefficient of Runoff, 10 yr ARI design rainfall event; and,

The subscript ‘y’ denotes the particular ARI under consideration.

Coefficients of Runoff adopted for this analysis are presented below in [Error! Reference source not found.](#). The resultant Rational Method predicted peak flows for each catchment over a range of standard ARI events is presented in [Error! Reference source not found.](#).

### Table 3-2 Coefficients of Runoff

<table>
<thead>
<tr>
<th>ARI</th>
<th>(F_y)</th>
<th>(C_{10})</th>
<th>(C_y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 year</td>
<td>0.80</td>
<td>0.35</td>
<td>0.28</td>
</tr>
<tr>
<td>2 year</td>
<td>0.85</td>
<td>0.35</td>
<td>0.30</td>
</tr>
<tr>
<td>5 year</td>
<td>0.95</td>
<td>0.35</td>
<td>0.33</td>
</tr>
<tr>
<td>10 year</td>
<td>1.00</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>20 year</td>
<td>1.05</td>
<td>0.35</td>
<td>0.37</td>
</tr>
<tr>
<td>50 year</td>
<td>1.15</td>
<td>0.35</td>
<td>0.40</td>
</tr>
<tr>
<td>100 year</td>
<td>1.20</td>
<td>0.35</td>
<td>0.42</td>
</tr>
</tbody>
</table>

### Table 3-3 Rational Method Peak Flow

<table>
<thead>
<tr>
<th>ARI</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
<th>L7</th>
<th>L8</th>
<th>L9</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 year</td>
<td>3.2</td>
<td>4.5</td>
<td>2.5</td>
<td>5.7</td>
<td>13.1</td>
<td>59.4</td>
<td>9.7</td>
<td>5.3</td>
<td>74.9</td>
</tr>
<tr>
<td>2 year</td>
<td>4.3</td>
<td>6.0</td>
<td>3.4</td>
<td>7.7</td>
<td>17.6</td>
<td>80.3</td>
<td>13.0</td>
<td>7.1</td>
<td>101.3</td>
</tr>
<tr>
<td>5 year</td>
<td>5.8</td>
<td>8.1</td>
<td>4.5</td>
<td>10.2</td>
<td>23.7</td>
<td>108.5</td>
<td>17.6</td>
<td>9.5</td>
<td>137.2</td>
</tr>
<tr>
<td>10 year</td>
<td>6.7</td>
<td>9.3</td>
<td>5.3</td>
<td>11.9</td>
<td>27.5</td>
<td>126.6</td>
<td>20.4</td>
<td>11.0</td>
<td>160.2</td>
</tr>
<tr>
<td>20 year</td>
<td>7.9</td>
<td>11.1</td>
<td>6.3</td>
<td>14.1</td>
<td>32.7</td>
<td>151.5</td>
<td>24.3</td>
<td>13.0</td>
<td>191.8</td>
</tr>
<tr>
<td>50 year</td>
<td>10.1</td>
<td>14.0</td>
<td>7.9</td>
<td>17.8</td>
<td>41.6</td>
<td>193.4</td>
<td>31.0</td>
<td>16.6</td>
<td>245.3</td>
</tr>
<tr>
<td>100 year</td>
<td>11.6</td>
<td>16.2</td>
<td>9.2</td>
<td>20.7</td>
<td>48.2</td>
<td>201.8</td>
<td>35.8</td>
<td>19.2</td>
<td>256.0</td>
</tr>
</tbody>
</table>

---

3.3.2 Culvert Sizing

The culvert analysis program “HY-8” was utilised to calculate concept cross-drainage configurations. The analysis was conducted using the two year ARI peak flows presented in Section Error! Reference source not found., and under an assumed tailwater level of 1.25 m (representing mean high spring tide). Appendix C illustrates the culvert locations with respect to the mine plan. As shown in Appendix C and in Error! Reference source not found. below, there are two locations where a bridge structure is recommended because:

- The magnitude of the peak flow is such that an excessively large bank of culverts would be required; and
- The length of the crossing would require significant in-filling of the watercourse should a culvert and floodway arrangement be employed.

Table 3-4 Culvert Sizing

<table>
<thead>
<tr>
<th>Crossing</th>
<th>Approx. Coordinates (m East, m North)</th>
<th>Peak Flow (m³/s)</th>
<th>Barrels/Cells</th>
<th>Size (mm)</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>616950, 8697650</td>
<td>4.3</td>
<td>3</td>
<td>900</td>
<td>RCP</td>
</tr>
<tr>
<td>2</td>
<td>616570, 8696780</td>
<td>6.0</td>
<td>4</td>
<td>900</td>
<td>RCP</td>
</tr>
<tr>
<td>3</td>
<td>615972, 8695470</td>
<td>3.4</td>
<td>2</td>
<td>900</td>
<td>RCP</td>
</tr>
<tr>
<td>4</td>
<td>616035, 8692305</td>
<td>7.7</td>
<td>5</td>
<td>900</td>
<td>RCP</td>
</tr>
<tr>
<td>5</td>
<td>616800, 8692600</td>
<td>17.6</td>
<td>6</td>
<td>1200 x 1200</td>
<td>RCB</td>
</tr>
<tr>
<td>6</td>
<td>618565, 8693300</td>
<td>13.0</td>
<td>5</td>
<td>1200 x 1200</td>
<td>RCB</td>
</tr>
<tr>
<td>7</td>
<td>619945, 8693850</td>
<td>7.1</td>
<td>5</td>
<td>900</td>
<td>RCP</td>
</tr>
<tr>
<td>8</td>
<td>620200, 8694060</td>
<td>101.3</td>
<td></td>
<td></td>
<td>Bridge structure required</td>
</tr>
</tbody>
</table>

Note:
RCP = reinforced concrete pipe culvert, size in mm diameter.
RCB = reinforced concrete box culvert, size in mm width x height.
Coordinates for Map Grid of Australia Zone 54.

The analysis carried out above has been based on a proposed haul road alignment (as shown in Appendix C) that represents the “worst case” with respect to hydraulic impacts, that is to say, the natural surface level for this alignment is located at low elevation (ie. in estuarine areas), thus requiring a relatively large volume of fill. An alternative haul road alignment is currently under consideration. It has not been modelled as part of this assessment, but as it is predominately located on higher ground, the impacts are expected to be significantly less due to fewer and smaller waterway crossings.
This chapter details the flood assessment conducted for the Skardon River and tributaries with the aim of:

- Demonstrating flood immunity of critical mine infrastructure and haul roads; and
- Assessing impacts on flood behaviour as a result of mine construction.

### 4.1 Hydrologic assessment

Flood hydrographs produced from the hydrologic assessment are used as input to hydraulic model simulations (refer Section 4.2) to predict flood characteristics such as inundation depth and extent and flow velocities.

A rainfall-runoff model was constructed using RORB software, which is a streamflow routing program used to calculate flood hydrographs from rainfall and other catchment inputs such as catchment storage and rainfall losses. The program is aerially distributed, nonlinear and makes allowance for temporal distribution of rainfall.

The schematic in Figure 4-1 illustrates the RORB routing model structure.

![Figure 4-1 RORB Runoff Routing Model](image-url)
4.1.1 Hydrologic Model Build

The following input data was generated for running hydrologic simulations:

- Sub-catchment areas;
- Sub-catchment reach lengths;
- Design event rainfall depths; and
- Temporal pattern distribution of rainfall.

The sub-catchment delineation and river reach network for the Skardon River catchment is shown in Figure 4-2. In RORB, a rainfall excess is calculated at the centroid of each sub-catchment by subtracting rainfall losses. The sub-area rainfall excess (runoff) is then added to existing flow in the channel and routed through the downstream reach based on the storage function $S = kQ^m$.

Design rainfall events were derived from the CRC-FORGE application as shown in Table 2-1. CRC-FORGE enables the extrapolation of design rainfall estimates to extreme events up to and including the 2000 year ARI and as such has benefits over the Bureau of Meteorology Intensity-Frequency-Duration method. Temporal pattern distribution of rainfall was achieved using Zone 4, Australian Rainfall and Runoff (ARR87) patterns as is standard industry practice. Probable Maximum Precipitation (PMP) rainfall depth estimates (refer Table 2-1) were estimated using the Generalised Short Duration Method (GSDM) and Generalised Tropical Storm Method (GSTMR) as detailed by the Bureau of Meteorology.

<table>
<thead>
<tr>
<th>Event Duration (h)</th>
<th>5 year ARI</th>
<th>10 year ARI</th>
<th>20 year ARI</th>
<th>50 year ARI</th>
<th>100 year ARI</th>
<th>1000 year ARI</th>
<th>PMP</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75.3</td>
<td>82.8</td>
<td>93.7</td>
<td>108.5</td>
<td>123.8</td>
<td>186.6</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>32.8</td>
<td>36.4</td>
<td>41.5</td>
<td>48.4</td>
<td>55.2</td>
<td>83.2</td>
<td>160</td>
</tr>
<tr>
<td>6</td>
<td>19.2</td>
<td>21.4</td>
<td>24.5</td>
<td>28.8</td>
<td>32.8</td>
<td>49.5</td>
<td>105</td>
</tr>
<tr>
<td>12</td>
<td>11.3</td>
<td>12.7</td>
<td>14.6</td>
<td>17.2</td>
<td>19.6</td>
<td>29.5</td>
<td>840</td>
</tr>
<tr>
<td>18</td>
<td>8.8</td>
<td>9.9</td>
<td>11.5</td>
<td>13.7</td>
<td>15.6</td>
<td>23.5</td>
<td>980</td>
</tr>
<tr>
<td>24</td>
<td>7.4</td>
<td>8.4</td>
<td>9.7</td>
<td>11.6</td>
<td>13.3</td>
<td>20.0</td>
<td>1130</td>
</tr>
<tr>
<td>48</td>
<td>4.9</td>
<td>5.6</td>
<td>6.5</td>
<td>7.8</td>
<td>8.9</td>
<td>14.2</td>
<td>1580</td>
</tr>
<tr>
<td>72</td>
<td>3.8</td>
<td>4.3</td>
<td>5.0</td>
<td>6.0</td>
<td>7.0</td>
<td>11.9</td>
<td>1960</td>
</tr>
</tbody>
</table>

Initial simulations were run for standard ARI events (5 year to 100 year) and durations (1 hour to 72 hour) with results presented in Table 4-2. The default Kc value of 41.18 was initially applied for the uncalibrated simulations in the absence of catchment specific rainfall and streamflow data in which to calibrate the catchment parameters. The Kc value was later altered (refer Section 4.1.3) to attempt calibration to regional regression equation predicted peak discharges. As recommended by the RORB user manual the non-linearity exponent, $m$, was set to its default value of 0.8 in the absence of catchment specific calibration data. No initial or continual loss was applied to the uncalibrated model as these values are determined during calibration.

The 1000 year ARI and PMP rainfall events were not included in the pre-calibration simulation as these extreme events are beyond the credible limit of extrapolation of available calibration data (refer Section 4.1.4 for further information).
### Table 4-2 Uncalibrated RORB Peak Outlet Flow; Kc = 41.18, m = 0.8

<table>
<thead>
<tr>
<th>ARI (yr)</th>
<th>Critical Storm Duration (h)</th>
<th>Peak Outlet Flow (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>6</td>
<td>543</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>621</td>
</tr>
<tr>
<td>20</td>
<td>6</td>
<td>735</td>
</tr>
<tr>
<td>50</td>
<td>6</td>
<td>892</td>
</tr>
<tr>
<td>100</td>
<td>6</td>
<td>1,049</td>
</tr>
</tbody>
</table>
DISCLAIMER
CDM Smith has endeavoured to ensure accuracy and completeness of the data. CDM Smith assumes no legal liability or responsibility for any decisions or actions resulting from the information contained in this map.

GCS GDA 1994 Zone 54

Figure 4-2
RORB Sub-Catchment
Delineation

Legend
- River Reaches
- Catchment
- Pit Limit
- Mine Lease Area
4.1.2 Regional Flood Frequency Analysis

There is no stream gauge record for the Skardon River catchment against which to calibrate the RORB predicted hydrographs. Therefore, a regional flood frequency analysis (FFA) was conducted to attempt calibration.

Four stream gauges were selected for the regional FFA as detailed in Table 4-3. The table shows that all of the gauges have a significant period of record (>35 years) and all except the Moreton gauge are within 50 km from the coast. With the exception of the Dulhunty gauge, all remaining sites have significantly greater contributing catchment areas (by an order of magnitude larger) than the Skardon River catchment making area based regional analysis less representative.

Notwithstanding this, FFA of the gauges was performed using the software program FLIKE, fitted to an LPIII distribution, as shown in Figure 4-3 to Figure 4-5. The LPIII distribution is widely accepted for use in FFA and in this instance produced a better fit to observed data than the Generalised Extreme Value method. Censoring of low annual maximum flow values was performed to improve the LPIII distribution fit to data.

The result of the LPIII fit and the FFA is the assignment of an ARI to each annual maximum of recorded stream gauge data as well as the provision of predictive extrapolation to more extreme ARI events. The (90%) confidence limits defined by the LPIII fit to data are also produced and indicate the accuracy and reliability of the FFA.

Table 4-3 FFA Stream Gauge Details

<table>
<thead>
<tr>
<th>Station Name / Number</th>
<th>Period of Record</th>
<th>Censored Annual Maxima (year)</th>
<th>Catchment Area</th>
<th>Distance to Project Site</th>
<th>Distance to coast</th>
</tr>
</thead>
<tbody>
<tr>
<td>Watson River 923001A</td>
<td>40 years</td>
<td>1991; 1994; 2011</td>
<td>1,001 km²</td>
<td>152 km</td>
<td>40 km</td>
</tr>
<tr>
<td>Dulhunty TM 96002A</td>
<td>43 years</td>
<td>1970</td>
<td>332 km²</td>
<td>35 km</td>
<td>46 km</td>
</tr>
<tr>
<td>Moreton TM 925001A</td>
<td>56 years</td>
<td>1958; 1989; 1991; 1992; 1993</td>
<td>3,265 km²</td>
<td>98 km</td>
<td>90 km</td>
</tr>
<tr>
<td>Monument TM 927001B</td>
<td>35 years</td>
<td>1978</td>
<td>2,421 km²</td>
<td>72 km</td>
<td>23 km</td>
</tr>
</tbody>
</table>
Figure 4-3  
Watson River (923001A) FFA output – LPIII Distribution

Figure 4-4  
Dulhunty TM (926002A) FFA output – LPIII Distribution
Figure 4-5  Moreton TM (925001A) FFA output – LIII Distribution

Figure 4-6  Monument TM (927001B) FFA output – LIII Distribution
A summary of the flows predicted by the FFA in comparison to the uncalibrated RORB outputs is presented in Table 4-4. The Dulhunty Gauge, which is very similar in both catchment area and location to the Skardon River Catchment, produces FFA results much lower than the uncalibrated RORB results for the Skardon Catchment. This strongly indicates a conservatively high peak flow estimated by the uncalibrated RORB model, most likely due to:

- The omission of rainfall loss in the uncalibrated model; and
- The generic RORB Kc value not being representative of local catchment characteristics.

Table 4-4 FFA Flow Comparison to Uncalibrated RORB Output (m³/s)

<table>
<thead>
<tr>
<th>ARI</th>
<th>Dulhunty (332 km²)</th>
<th>Watson (1,001 km²)</th>
<th>Monument (2,421 km²)</th>
<th>Moreton (3,265 km²)</th>
<th>RORB uncalibrated (350 km²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 year</td>
<td>132</td>
<td>545</td>
<td>458</td>
<td>866</td>
<td>543</td>
</tr>
<tr>
<td>10 year</td>
<td>188</td>
<td>683</td>
<td>556</td>
<td>1,074</td>
<td>621</td>
</tr>
<tr>
<td>20 year</td>
<td>256</td>
<td>855</td>
<td>656</td>
<td>1,272</td>
<td>735</td>
</tr>
<tr>
<td>50 year</td>
<td>374</td>
<td>1,015</td>
<td>793</td>
<td>1,527</td>
<td>892</td>
</tr>
<tr>
<td>100 year</td>
<td>490</td>
<td>1,216</td>
<td>902</td>
<td>1,715</td>
<td>1,049</td>
</tr>
</tbody>
</table>

From Table 4-4 a regional trend between catchment area and flow can be established by fitting a regression trend line to the four gauge FFA results. Figure 4-7 shows the best fit regression to the 100 year ARI FFA estimates. The following regional regression equations were developed for predicting peak catchment discharge from the catchment area regression parameter.

\[ Q_5 = 2.67 A^{0.71} \]
\[ Q_{10} = 5.39 A^{0.64} \]
\[ Q_{20} = 10.59 A^{0.58} \]
\[ Q_{50} = 23.84 A^{0.50} \]
\[ Q_{100} = 46.04 A^{0.43} \]

; where \( Q_{ARI} \) is the discharge at the indicated ARI and \( A \) is the catchment area in square kilometres.
4.1.3 Hydrologic Model Calibration

A comparison of the RORB peak discharge calibration results against the values predicted by the regional regression equations presented in Section 4.1.2, is shown below in Table 4-5. A good fit to the regional analysis was achieved and is considered more representative of actual catchment characteristics than the uncalibrated model.

Based on these findings, calibration of the RORB model was achieved via:

- Increasing the Kc parameter from the default value of 41.18 to 48.5;
- Applying probability neutral initial loss values across the ARI events i.e. applying an initial loss that allows a specific ARI rainfall event to produce the same resultant ARI flood event; and
- Applying a continual loss of 10 mm/h across all ARI events.

Table 4-5 RORB Calibration to Regional Regression Equation Results

<table>
<thead>
<tr>
<th>ARI (yr)</th>
<th>RRE Discharge Skardon Catchment (m³/s)</th>
<th>RORB Peak Outlet Discharge (m³/s)</th>
<th>RORB Calibration Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Kc</td>
</tr>
<tr>
<td>5</td>
<td>166</td>
<td>178</td>
<td>48.5</td>
</tr>
<tr>
<td>10</td>
<td>230</td>
<td>241</td>
<td>48.5</td>
</tr>
<tr>
<td>20</td>
<td>311</td>
<td>321</td>
<td>48.5</td>
</tr>
<tr>
<td>50</td>
<td>435</td>
<td>426</td>
<td>48.5</td>
</tr>
<tr>
<td>100</td>
<td>564</td>
<td>563</td>
<td>48.5</td>
</tr>
</tbody>
</table>
By increasing the Kc parameter from the default value, which is the main calibration parameter within RORB and affects the treatment of flow attenuation in reaches, peak discharges were lowered to more closely match the RRE predicted results. As a result the time to peak increased.

The continual loss was set to 10 mm/h in order to calibrate the 100 year ARI event. This is a relatively high continual loss value compared to the default 2.5 mm/h value recommended in ARR (ARR87), however it is considered appropriate given the high permeability of the bauxite layer at the surface as evidenced by the regional Australian Water Balance Model calibration detailed in Section 2.1. Furthermore, by increasing the continual loss, peak discharges could be lowered to match RRE predictions without further delaying the hydrograph time to peak beyond what is reasonable, thus producing a more acceptable hydrograph shape.

Initial loss values were lastly applied to calibrate the 5 year, 10 year, 20 year and 50 year ARI events. Higher initial loss values were applied to more frequent events in keeping with a probability neutral approach between design rainfall and design flood. No initial loss was applied to the 100 year ARI event on the assumption that the antecedent conditions have saturated the catchment.

4.1.4 Treatment of Extreme Rainfall Events

The 1000 year ARI event was not included in the RORB calibration because the confidence limits obtained by the FFA for the 1000 year event show the predicted results to be less reliable. Furthermore, by adjusting calibration parameters to include extreme events the calibration of more frequent flood events necessarily become less accurate. In the interest of keeping a single set of calibration parameters across all ARI events only the 5 year to 100 year ARI events were included in calibration of RORB parameters.

For the abovementioned reasons, both the 1000 year ARI and PMP rainfall events were simulated using the calibrated Kc value achieved for the 5 to 100 year ARI event FFA predicted discharges as detailed in Section 4.1.3. No initial loss was applied to the 1000 year ARI or PMP rainfall event on the assumption that the antecedent conditions have saturated the catchment. A continual loss of 10 mm/h was applied to extreme rainfall events.

4.1.5 Hydrologic Model Results

RORB hydrographs produced by the calibration described in Section 4.1.3 and summarised in Table 4-5 were adopted as input to hydraulic model simulations. The resultant hydrographs produced at the Skardon River catchment outlet for the 5, 10, 20, 50, 100, and 1000 year ARI, and PMP critical duration storm events are shown in Figure 4-8

The hydrographs show an increase in peak discharge with increase in ARI. The hydrographs exhibit an early peak arising from local catchment runoff and tributaries closer to the outlet, followed by a larger peak as the main Skardon River flood wave reaches the outlet. A time to peak discharge at the outlet of approximately 13 to 16 hours can be observed for the 100 year through the 5 year ARI events.
The PMP predicted peak discharge is an order of magnitude greater than the 100 year event and has a likely ARI of approximately 1 in 3,000,000 years, plus or minus an order of magnitude, according to methods developed by Laurenson and Kuczera (1999)\textsuperscript{5}.

4.2 Hydraulic Assessment

The aim of the hydraulic assessment is to characterise the Skardon River system in the vicinity of the proposed project site. Specifically, it involves investigating the effects of rainfall runoff on flood levels and velocities so that any impacts upon infrastructure and the environment can be quantified. Hydrodynamic modelling is used to create maps showing flood extents, water depths and velocities, across a range of ARIs.

4.2.1 Modelling Software

MIKE 21 is a professional engineering software package containing a comprehensive modelling system for 2D free-surface flows. The numerical solution is based on the two-dimensional implementation of the St. Venant equations and can handle both sub-critical and super-critical flows. MIKE 21 is particularly useful in floodplain applications where the out of bank flow paths are poorly defined and where a traditional 1D model would fail to capture the complex flowpaths and transverse distribution of water levels and velocities that occurs on the floodplain.

4.2.2 Survey Data

The basis for the 2D flood model topographic grid is Airborne Laser Survey (ALS) data captured by surveyors Cameron Cottrell and Steen (CCS) on behalf of Metro Mining. The data were ratified by Metro Mining as appropriate to use for the purposes of this study, and were provided to CDM Smith in the form of a series of 1m ascii grids. A standard complication with this kind of survey is the treatment of water bodies. The laser cannot ‘see’ below the water, and observations are instead reflected back from the water surface. In the case of a large dataset which takes some time to capture, the water level is likely to vary during this time, particularly if multiple flights are carried out across a range of tidal conditions.

To overcome this inconsistency water areas are typically removed from the data and a false bottom elevation is applied. In this case, CCS modified the terrain data to give water bodies a uniform surface elevation of 0.4 m Australian Height Datum (AHD). This level is appropriately low considering the tidal condition used when mapping the maximum flood extents, as explained in Section 2.2.4.

4.2.3 Model Setup

The choice of grid cell size is a matter that requires careful thought. Too large, and important topographic detail can be missed; too small, and model run times can become unmanageable. Although it would seem logical that a smaller grid cell size is automatically superior to a large one, care must be taken to avoid the “water column’ effect, where the vertical length scale approaches or exceeds the horizontal length scale. Such a situation can lead to errors in the viscous shear stress and bed friction calculations and ultimately result in unrealistic estimates of velocity and water level distributions in the model. As a rule of thumb, the grid cell size should be no smaller than the expected maximum water depth.

For the aforementioned reasons, a twenty metre grid cell size was chosen. The 20 m grid size is larger than the maximum water depth for scenarios modelled and provides appropriate definition of the major topographic features (e.g. river channel definition) without resulting in overly long model run times. Details of the topographic grid are listed in Table 4-6.
**Table 4-6 MIKE21 Model Dimensions**

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grid Cell Size</td>
<td>20 m</td>
</tr>
<tr>
<td>Grid Orientation/Rotation</td>
<td>North up (i.e. zero degrees rotation)</td>
</tr>
<tr>
<td>Model extent (width x height)</td>
<td>1265 cells x 1150 cells</td>
</tr>
<tr>
<td>Model extent (km x km)</td>
<td>25.3 km x 22.8 km</td>
</tr>
<tr>
<td>Model Origin (Lower Left Corner)</td>
<td>604,310 m East; 8,683,210 m North</td>
</tr>
<tr>
<td>Map Projection</td>
<td>MGA, Zone 54</td>
</tr>
</tbody>
</table>

**4.2.4 Boundary Conditions**

In an estuarine river system such as that of the Project area, flood depths and velocities can be greatly influenced by the tidal level (i.e. the tailwater level). For this reason, two sets of tidal conditions were implemented in the model – a high tailwater condition to assess the likely maximum extents and depths of inundation, and a low tailwater condition to investigate maximum expected velocities.

The weather system that is most likely to create large scale flooding in the catchment is a tropical cyclone. In addition to the precipitation produced by a cyclone, the high wind speeds and large fetch lengths can create a significant increase in water level – referred to as a storm surge – in addition to the prevailing tidal condition. The high tailwater condition is therefore based on a large tide occurring coincidentally with a storm surge, and applied as a fixed level to the ocean boundary. The nearest available tidal plane data is from Weipa, which records a value of 2.15 m AHD for the highest astronomical tide (HAT).

With regards to the storm surge component, CDM Smith has adopted the same value (0.55 m) as WorleyParsons in their flood study of the adjacent Ducie River catchment, carried out for Metro Mining as part of the Pisolite Hills Project. Given that this data is derived from values recorded at Weipa, and that no other nearby records of storm surge are available, it is considered appropriate to adopt the same value for this study. The resulting components of storm tide are outlined below in Table 4-7.

**Table 4-7 Tailwater Components – Storm Tide Condition**

<table>
<thead>
<tr>
<th>Tailwater Components</th>
<th>Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest Astronomical Tide</td>
<td>1.60 m AHD</td>
</tr>
<tr>
<td>Storm Surge</td>
<td>0.55 m</td>
</tr>
<tr>
<td>Storm Tide</td>
<td>2.15 m AHD</td>
</tr>
</tbody>
</table>

For the low tailwater condition, the model is constrained by the fixed bottom elevation of 0.4 m AHD applied to water areas in the ALS grid (as explained in Section 2.2.2), as any tailwater condition should be higher than the assumed bottom of the channel to prevent numerical instabilities occurring in the solving engine. The next highest tide above the 0.4 m AHD level is that of “Mean Lower High Water”, with an elevation of 0.448 m AHD, and this was adopted as the low tailwater condition in the model.
4.2.5 Methodology

The aim of the hydraulic model simulation is to demonstrate the riverine flooding likely to occur in the Skardon River under various design flood event conditions.

To achieve this objective the following methodology was utilised:

- Extract the RORB hydrographs for design storm events ranging from the 5 year ARI to the 1000 year ARI, plus the Probable Maximum Precipitation (PMP) event. Fifteen locations were identified inside the MIKE21 grid at which to apply the hydrographs as point inflows;
- Set the boundary conditions. For each storm event, two simulations were created, one with the high tailwater condition, and the other with the low tailwater condition;
- Run each of the models for a sufficient time period to ensure that the flood peak had occurred and that water levels were receding at every point in the model domain;
- Extract from the model results files the maximum values for water depth (from the high tailwater simulation) and velocity (from the low tailwater simulation). It is important to note that the peak values do not occur simultaneously throughout the model domain, and that peak depth/velocity maps represent the highest values recorded across the entire simulation period. Similarly, the recorded values for peak depth and peak velocity at any particular model location do not necessarily occur simultaneously in time; and
- Convert the peak velocity and depth results in an ascii grid format for mapping and presentation.

4.2.6 Results and Discussion

The MIKE21 models were observed to be stable at a ten second time step. Each model was run for a 30 hour simulation time, which captured the bulk of the flood wave and peak water levels and velocities throughout the model domain. Results were processed to create maps showing depth and velocity maxima; these maps are found in Appendix B.

The maps show that the proposed pit locations are not at risk from riverine flooding, even under the PMF event. This is largely because the pits are generally situated on the bauxite plateaus with buffer distances between the pit shell and watercourse boundaries. Construction of the pits is not likely to interfere with the current floodplain processes, and will therefore cause no significant hydraulic impacts to the river in terms of changes to flows, water levels, or velocities.

The main impact of riverine flooding is to the haul road connecting the barge loading facilities. It is noted that the haul road has been designed to fit within the defined lease boundary. Negotiations with neighbouring lease holders have been initiated in order to locate the haul road in a less flood and tidally influenced zone.

Peak water depths, surface elevations, and velocities were extracted from the results at seven locations along the haul road; these are summarised below in Table 4-8, Table 4-9 and Table 4-10, respectively.
Table 4-8 Peak Flood Depths at Locations of Interest

<table>
<thead>
<tr>
<th>Haul Road Location</th>
<th>Coordinates (m East, m North)</th>
<th>Depth (m)*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5 yr ARI</td>
</tr>
<tr>
<td>Crossing Point A</td>
<td>620190, 8964050</td>
<td>1.75</td>
</tr>
<tr>
<td>Crossing Point B</td>
<td>618530, 8693330</td>
<td>1.67</td>
</tr>
<tr>
<td>Low Spot – BH6</td>
<td>616570, 8692530</td>
<td>1.33</td>
</tr>
<tr>
<td>Low Spot 1 – BH6 East to Port</td>
<td>616030, 8695570</td>
<td>1.81</td>
</tr>
<tr>
<td>Low Spot 2 – BH6 East to Port</td>
<td>616590, 8696810</td>
<td>1.41</td>
</tr>
<tr>
<td>Low Spot 3 – BH6 East to Port</td>
<td>617000, 8697750</td>
<td>1.08</td>
</tr>
<tr>
<td>Jetty (north-east corner)</td>
<td>617370, 8698490</td>
<td>1.39</td>
</tr>
</tbody>
</table>

*Depth is measured from datum 0.4 m AHD which represents the water surface elevation from processed survey data.

Table 4-9 Peak Water Surface Elevations at Locations of Interest

<table>
<thead>
<tr>
<th>Haul Road Location</th>
<th>Coordinates (m East, m North)</th>
<th>Water Surface Elevation (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5 yr ARI</td>
</tr>
<tr>
<td>Crossing Point A</td>
<td>620190, 8964050</td>
<td>2.24</td>
</tr>
<tr>
<td>Crossing Point B</td>
<td>618530, 8693330</td>
<td>2.24</td>
</tr>
<tr>
<td>Low Spot – BH6</td>
<td>616570, 8692530</td>
<td>2.24</td>
</tr>
<tr>
<td>Low Spot 1 – BH6 East to Port</td>
<td>616030, 8695570</td>
<td>2.23</td>
</tr>
<tr>
<td>Low Spot 2 – BH6 East to Port</td>
<td>616590, 8696810</td>
<td>2.22</td>
</tr>
<tr>
<td>Low Spot 3 – BH6 East to Port</td>
<td>617000, 8697750</td>
<td>2.22</td>
</tr>
<tr>
<td>Jetty (north-east corner)</td>
<td>617370, 8698490</td>
<td>2.22</td>
</tr>
</tbody>
</table>

Table 4-10 Peak Velocities at Locations of Interest

<table>
<thead>
<tr>
<th>Haul Road Location</th>
<th>Coordinates (m East, m North)</th>
<th>Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5 yr ARI</td>
</tr>
<tr>
<td>Crossing Point A</td>
<td>620190, 8964050</td>
<td>0.17</td>
</tr>
<tr>
<td>Crossing Point B</td>
<td>618530, 8693330</td>
<td>0.05</td>
</tr>
<tr>
<td>Low Spot – BH6</td>
<td>616570, 8692530</td>
<td>0.08</td>
</tr>
<tr>
<td>Low Spot 1 – BH6 East to Port</td>
<td>616030, 8695570</td>
<td>0.10</td>
</tr>
<tr>
<td>Low Spot 2 – BH6 East to Port</td>
<td>616590, 8696810</td>
<td>0.10</td>
</tr>
<tr>
<td>Low Spot 3 – BH6 East to Port</td>
<td>617000, 8697750</td>
<td>0.00</td>
</tr>
<tr>
<td>Jetty (north-east corner)</td>
<td>617370, 8698490</td>
<td>0.01</td>
</tr>
</tbody>
</table>

For each ARI listed in Table 4-8, the peak water depth can be taken as analogous to the fill height required to construct the haul road with the corresponding level of flood immunity. For example, at Crossing A, to construct the haul road with a 50 year ARI flood immunity would require the placement of almost 2 m of fill, bringing the finished road surface to a level above 2.45 m AHD. For a five year flood immunity the required fill height would be 1.75 m, and the finished surface elevation 2.24 m AHD. However, in the absence of detailed bathymetric data, the depths shown in Table 4-8 are measured from the water surface elevation datum applied to the topographical survey (refer Section 4.2.2) and thus may underestimate the total amount of fill required.

In all cases the peak velocities are quite low, as would be expected from an estuarine system with low stream energy. It is unlikely that river velocities will cause excessive scouring and sediment transport due to inundation of road surfaces or drainage infrastructure.
Noting that mining operations are scheduled to be carried only during the dry season, and that significant flooding is almost exclusively confined to the summer monsoonal period, constructing to high levels of flood immunity may not be warranted. However the regular tidal influence should still be taken into account when designing site infrastructure. At the same seven locations, Table 4-11 shows the expected conditions in the absence of riverine flooding, for both HAT and the representative storm surge.

**Table 4-11 Tidal Levels at Locations of Interest**

<table>
<thead>
<tr>
<th>Haul Road Location</th>
<th>Coordinates (m East, m North)</th>
<th>Depth (m)</th>
<th>Elevation (m AHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>HAT</td>
<td>Storm Tide</td>
</tr>
<tr>
<td>Crossing Point A</td>
<td>620190, 8964050</td>
<td>1.14</td>
<td>1.66</td>
</tr>
<tr>
<td>Crossing Point B</td>
<td>618530, 8693330</td>
<td>1.06</td>
<td>1.58</td>
</tr>
<tr>
<td>Low Spot – BH6</td>
<td>616570, 8692530</td>
<td>0.72</td>
<td>1.24</td>
</tr>
<tr>
<td>Low Spot 1 – BH6 East to Port</td>
<td>616030, 8695570</td>
<td>1.21</td>
<td>1.73</td>
</tr>
<tr>
<td>Low Spot 2 – BH6 East to Port</td>
<td>616590, 8696810</td>
<td>0.82</td>
<td>1.34</td>
</tr>
<tr>
<td>Low Spot 3 – BH6 East to Port</td>
<td>617000, 8697750</td>
<td>0.49</td>
<td>1.01</td>
</tr>
<tr>
<td>Jetty (north-east corner)</td>
<td>617370, 8698490</td>
<td>0.80</td>
<td>1.32</td>
</tr>
</tbody>
</table>

As the table shows, the tidal influence is fairly significant at the majority of locations. Ideally road infrastructure should be built no lower than the HAT, as tides near this level could be expected to occur at any time of year. Note that although HAT technically only occurs once every eighteen years, the calculated values for HAT are based on gravitational phenomena only and make no allowance for wind or wave setup, and thus these additional components of water level may cause a regular but large tide to impact on operations. Refer to Appendix B for a graphical representation of the HAT extents relative to project infrastructure.
Appendix A - Disclaimer and Limitations

This report has been prepared by CDM Smith Australia Pty Ltd (CDM Smith) for the sole benefit of Metro Mining Limited for the sole purpose of informing the Surface Water component of the environmental assessments being carried out as part of the Metro Mining project.

This report should not be used or relied upon for any other purpose without CDM Smith’s prior written consent. CDM Smith, nor any officer or employee of CDM Smith, accepts no responsibility or liability in any way whatsoever for the use or reliance of this report for any purpose other than that for which it has been prepared.

Except with CDM Smith’s prior written consent, this report may not be:

(a) released to any other party, whether in whole or in part (other than to Metro Mining Limited officers, employees and advisers);

(b) used or relied upon by any other party; or

(c) filed with any Governmental agency or other person or quoted or referred to in any public document.

CDM Smith, nor any officer or employee of CDM Smith, accepts no liability or responsibility whatsoever for or in respect of any use or reliance upon this report by any third party.

The information on which this report is based has been provided by Metro Mining Limited and third parties. CDM Smith (including its officer and employee):

(a) has relied upon and presumed the accuracy of this information;

(b) has not verified the accuracy or reliability of this information (other than as expressly stated in this report);

(c) has not made any independent investigations or enquiries in respect of those matters of which it has no actual knowledge at the time of giving this report to Metro Mining Limited; and

(d) makes no warranty or guarantee, expressed or implied, as to the accuracy or reliability of this information.

In recognition of the limited use to be made by Metro Mining Limited of this report, Metro Mining Limited agrees that, to the maximum extent permitted by law, CDM Smith (including its officer and employee) shall not be liable for any losses, claims, costs, expenses, damages (whether in statute, in contract or tort for negligence or otherwise) suffered or incurred by Metro Mining Limited or any third party as a result of or in connection with the information, findings, opinions, estimates, recommendations and conclusions provided in the course of this report.

If further information becomes available, or additional assumptions need to be made, CDM Smith reserves its right to amend this report.
FIGURE B-1

BAUXITE HILLS - PEAK DEPTH
5 YEAR FLOOD DEPTH MAP

Legend

Flood Depth (m)
- 0
- 0.00 - 0.25
- 0.25 - 0.50
- 0.50 - 0.75
- 0.75 - 1.00
- 1.00 - 1.25
- 1.25 - 1.50
- 1.50 - 1.75
- 1.75 - 2.00
- 2.00 - 2.25
- 2.25 - 2.50

Watercourse
Barge Loading Area
Haul Road
MLA Boundary
Pit Limit
Flood Hotspot

DATA SOURCE
MEC Mining;
QLD Government Open Source Data;
Australian Hydrological Geospatial Fabric (Geofabric) PRODUCT SUITE V2.1.1

CDM Smith has endeavoured to ensure accuracy and completeness of the data. CDM Smith assumes no legal liability or responsibility for any decisions or actions resulting from the information contained within this map.

GCS GDA 1994 MGA Zone 54

Scale A3 - 1:50,000

DESIGNER
CDM Smith

CLIENT
BAUXITE HILLS - PEAK DEPTH

11/06/15

©COPYRIGHT CDM SMITH
This drawing is confidential and shall only be used for the purpose of this project.
FIGURE B-4

**FLOOD DEPTH MAP**

**DATA SOURCE**
- MEC Mining
- QLD Government Open Source Data
- Australian Hydrological Geospatial Fabric (Geofabric) PRODUCT SUITE V2.1.1

**DESIGNER**
- CDM Smith
- www.cdm.com

**CLIENT**
- METRO MINING
- www.metalco.com

**DISCLAIMER**
CDM Smith has endeavoured to ensure accuracy and completeness of the data. CDM Smith assumes no legal liability or responsibility for any decisions or actions resulting from the information contained within this map.

**COPYRIGHT CDM SMITH**
This drawing is confidential and shall only be used for the purpose of this project.

**Legend**

- **Flood Depth (m)**
  - 0
  - 0.00 - 0.25
  - 0.25 - 0.50
  - 0.50 - 0.75
  - 0.75 - 1.00
  - 1.00 - 1.25
  - 1.25 - 1.50
  - 1.50 - 1.75
  - 1.75 - 2.00
  - 2.00 - 2.25
  - 2.25 - 2.50

- **Watercourse**
- **Barge Loading Area**
- **Haul Road**
- **MLA Boundary**
- **Pit Limit**
- **Flood Hotspot**

**Notes:**
- F:\1_PROJECTS\BES150115_Bauxite_Hill\GIS\DATA\MXD\FINAL\Flood Modelling\UPDATES\BES150115-Flood Maps PEAK DEPTH.mxd

**Scale:** 1:50,000 Scale @ A3 - 1:50,000

**GCS GDA 1994 MGA Zone 54**

**NORTH**

**W**

**E**

**2,000**

**1,000**

**500**

**0**

**Meters**

**Approved:**
- 11/06/15
- MD

**Checked:**
- 11/06/15

**Drawn:**
- 11/06/15

**Designed:**
- 11/06/15

**Copyright:**
CDM Smith has endeavoured to ensure accuracy and completeness of the data. CDM Smith assumes no legal liability or responsibility for any decisions or actions resulting from the information contained within this map.
DISCLAIMER
CDM Smith has endeavoured to ensure accuracy and completeness of the data. CDM Smith assumes no legal liability or responsibility for any decisions or actions resulting from the information contained within this map.

DATA SOURCE
MEC Mining; QLD Government Open Source Data; Australian Hydrological Geospatial Fabric (Geofabric) PRODUCT SUITE V2.1.1

FIGURE B-9
BAUXITE HILLS - PEAK VELOCITY
10 YEAR FLOOD DEPTH MAP

©COPYRIGHT CDM SMITH
This drawing is confidential and shall only be used for the purpose of this project.

DESIGNER CLIENT
1:50,000 Scale @ A3 - DESIGNED CHECKED APPROVED
- DJ 11/06/15
- EOB
- DAVE 11/06/15

F:\1_PROJECTS\BES150115_Bauxite_Hill\GIS\DATA\MXD\FINAL\Flood_Modelling\UPDATES\BES150115-Flood Maps PEAK VELOCITY.mxd

Draft

Legend

Flood Depth (m)
• Watercourse
• < 0.1
• 0.1 - 0.2
• 0.2 - 0.3
• 0.3 - 0.4
• 0.4 - 0.5
• 0.5 - 0.6
• 0.6 - 0.7
• 0.7 - 0.8
• 0.8 - 0.9
• 0.9 - 1
• 1 <

Barge Loading Area
Haul Road
MLA Boundary
Pit Limit
Flood Hotspot

BH1 MLA boundary
BH6 West MLA boundary
BH6 East MLA boundary
SKARDON RIVER
Legend

Flood Depth (m)
- < 0.1
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 - 0.6
- 0.6 - 0.7
- 0.7 - 0.8
- 0.8 - 0.9
- 0.9 - 1
- 1 <

Watercourse
Barge Loading Area
Haul Road
MLA Boundary
Pit Limit
Flood Hotspot

DATA SOURCE
MEC Mining;
QLD Government Open Source Data;
Australian Hydrological Geospatial Fabric (Geofabric) PRODUCT SUITE V2.1.1

DISCLAIMER
CDM Smith has endeavoured to ensure accuracy and completeness of the data. CDM Smith assumes no legal liability or responsibility for any decisions or actions resulting from the information contained within this map.

GCS GDA 1994 MGA Zone 54

APPROVED
Drawn
11/06/15

© COPYRIGHT CDM SMITH
This drawing is confidential and shall only be used for the purpose of this project.

This drawing is confidential and shall only be used for the purpose of this project.

Scale @ A3 - 1:50,000
GCS GDA 1994 MGA Zone 54

FIGURE B-10
BAUXITE HILLS - PEAK VELOCITY
20 YEAR FLOOD DEPTH MAP

F:\1_PROJECTS\BES150115_Bauxite_Hill\GIS\DATA\MXD\FINAL\Flood Modelling\UPDATES\BES150115-Flood Maps PEAK VELOCITY.mxd

CDMSmith.com

Metro Mining Limited

CLIENT

DESIGNER

DESIGNED
- 11/06/15
- EOB
- CHECKED
- EOB

DRAWN
- 11/06/15
- EOB
- CHECKED
- EOB

APPROVED
- 11/06/15
- EOB
- CHECKED
- EOB
BAUXITE HILLS - PEAK VELOCITY
1000 YEAR FLOOD DEPTH MAP

FIGURE B-13

DISCLAIMER
CDM Smith has endeavoured to ensure accuracy and completeness of the data. CDM Smith assumes no legal liability or responsibility for any decisions or actions resulting from the information contained within this map.

DATA SOURCE
MEC Mining; QLD Government Open Source Data; Australian Hydrological Geospatial Fabric (Geofabric) PRODUCT SUITE V2.1.1

Legend
Flood Depth (m)
- < 0.1
- 0.1 - 0.2
- 0.2 - 0.3
- 0.3 - 0.4
- 0.4 - 0.5
- 0.5 - 0.6
- 0.6 - 0.7
- 0.7 - 0.8
- 0.8 - 0.9
- 0.9 - 1
- 1 <

Watercourse
Barge Loading Area
Haul Road
MLA Boundary
Pit Limit
Flood Hotspot

Scale 1:50,000

GCS GDA 1994 MGA Zone 54

N
S
E
W

Notes:
Crossing 1:
- 3/900 dia. RCP (2 Year ARI Peak Flow = 4.3 m³/s)

Crossing 2:
- 4/900 dia. RCP (2 Year ARI Peak Flow = 6.0 m³/s)

Crossing 3:
- 2/900 dia. RCP (2 Year ARI Peak Flow = 3.4 m³/s)

Crossing 4:
- 3/900 dia. RCP (2 Year ARI Peak Flow = 4.3 m³/s)

Crossing 5:
- 5/900 dia. RCP Barrels (2 Year ARI Peak Flow = 7.7 m³/s)

Crossing 6:
- 5/900 dia. RCP (2 Year ARI Peak Flow = 4.3 m³/s)

Crossing 7:
- 5/1200 x 1200 RCBC (2 Year ARI Peak Flow = 13.0 m³/s)

Crossing 8:
- 5/1200 dia. RCP (2 Year ARI Peak Flow = 7.1 m³/s)

Crossing 9:
- Bridge structure required (2 Year ARI Peak Flow = 101.3 m³/s)

Crossing 10:
- Bridge structure required (2 Year ARI Peak Flow = 80.3 m³/s)

Crossing 11:
- Bridge structure required (2 Year ARI Peak Flow = 80.3 m³/s)

Please note that the diagram includes the locations of various road crossings and mine lease area boundaries, along with details on the types and sizes of culverts and other drainage structures proposed for different sections of the mine site. The data source for the map includes information from MEC Mining, QLD Government Open Source Data, and Australian Hydrological Geospatial Fabric (Geofabric) PRODUCT SUITE V2.1.1.