ORE BODY 25 PIT 3

Homestead Creek Management at Closure

Prepared for: BHP Billiton Iron Ore

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EXECUTIVE SUMMARY

Homestead Creek drains an area of approximately 300 km^2 and enters the Fortescue River just downstream of Ophthalmia Dam. The main channel of the creek is located approximately 4 km north of the Newman townsite at its closest point. For approximately 6 km of its length, Homestead Creek runs adjacent to BHP Billiton Iron Ore's (BHPBIO) Orebody 25 mining operations. Homestead Creek also passes by the Orebody 23 operation near the Marble Bar Road crossing.

Pit 3 of OB25 currently extends to within 60 m of the Homestead Creek main channel. The existing flood bunding is designed to protect the pit from flooding during the operational life of the pit; it is not intended for closure conditions in its current form. This study investigates the hydrologic interactions related to the Homestead Creek and OB25 catchment areas and identifies key areas of concern related to post-closure drainage management. The study includes an assessment of the probability that Pit 3 will capture Homestead Creek during a post-closure period of up to 10,000 years and includes recommendations for preventing creek capture.

During extreme flood events, the Homestead Creek floodplain is comingled with the Whaleback Creek floodplain. Tetra Tech was tasked with conducting a hydrologic analysis of the Homestead and Whaleback Creek catchment areas along with a hydraulic assessment of flow conditions in the vicinity of OB25. An erosion assessment was performed to estimate the extent of head cut migration in the event of flood flows overtopping the banks and discharging into Pit 3. Options were developed for preventing creek capture at closure and ensuring a long-term, stable land form. This report outlines the approach and findings of the hydrologic assessment, hydraulic modelling, site visit, and erosion investigations.

Hydrologic modelling of Homestead Creek yields a 100-year ARI peak discharge of 649 m^3/s and a PMF peak discharge of 3,911 m^3/s. Additional flow contribution from Whaleback Creek is assessed under the unlikely scenario of coincident peak flows in both creeks. The hydraulic analysis determined that the existing flood bund would overtop only in the event of a coincident PMF peak flow in both Homestead and Whaleback Creeks. Without ongoing maintenance during the post-closure timeframe, the existing flood bund would be expected to deteriorate over time; in order to reflect this condition, a hydraulic analysis was performed under the assumption of complete failure of the bund to the insitu floodplain elevation. Under that scenario, Homestead Creek would begin to overtop and spill into Pit 3 at the approximate peak discharge level associated with a 50-year Average Recurrence Interval (ARI) event in the Homestead Creek catchment.

A series of 100-year to 10,000-year ARI flood events overtopping the channel banks and spilling into the pit was modelled to determine head cut migration rates over a 10,000-year duration. The assessment determined that a single 100-year ARI event would result in sufficient head cut migration to capture the thalweg. The flow volume captured by the pit during the event would be sufficient to raise the pit lake elevation from the long-term groundwater table elevation to the level of the ground surface. With the water surface equalised between the creek and the pit, erosion would be slowed during subsequent events until the pit lake water surface recedes back to the long-term groundwater table elevation. Following creek capture and recession of the pit lake elevation to the groundwater table, the entire low-flow discharge of Homestead Creek would be routed into the pit. The probability of this condition occurring under the stated set of assumptions is essentially 100%.

A comparison of available historical aerial photographs showed no discernible change in the path of the creek thalweg in recent years. Observations of the vegetation along the channel banks and the geomorphological characteristics of the system indicate that the potential for creek migration appears to be more likely to occur to the south than to the north toward the pit. The overall potential for large-scale lateral migration appears to be relatively low in this reach of Homestead Creek. Because head cut erosion from creek flows entering the mining pit is effectively a certainty over the 10,000-year assessment period, lateral migration is not assessed quantitatively.

Options for improving the stability and durability of the existing flood bund include rock armouring, seepage protection, increased width, or the use of partial or complete pit backfill. A closure levee would need to be located outside of the potentially unstable pit edge zone as determined by geotechnical analyses. A hardened low point and spillway structure designed for overtopping flows may be warranted to prevent the unpredictable failure point that might result from a closure levee with a constant crest gradient. Closure designs need to address erosion risks where concentrated flow enters the pit as well as enclosed basins that have the potential to build up head and fail catastrophically if overtopped. Critical areas of concern are presented in the report figures.

1.0 INTRODUCTION

1.1 Background

Orebody 25 (OB25) is located along the Eastern Ridge of the Ophthalmia Range approximately 8 km northeast of Newman. Mining at OB25 began in 1989, with an extension project approved in 2006. Mining of the extension is expected to continue until at least 2030. A Decommissioning and Rehabilitation Plan (DRP) was prepared in 2010 (BHPBIO 2010). The DRP describes control measures for the management of landforms and infrastructure during the decommissioning and closure of the project.

The proximity of OB25 to Homestead Creek presents a significant closure concern; Homestead Creek, which drains an area of approximately 300 km^2, passes within 60 m of the ultimate pit shell of OB25 and enters the Fortescue River just downstream of Ophthalmia Dam. The Homestead Creek floodplain is comingled with the Whaleback Creek floodplain, which contributes to flood flows near OB25 under extreme flood conditions.

The existing flood bunding is designed to protect the pit from flooding during the operational life of the pit; it is not intended for closure conditions in its current form. Homestead Creek overbank flows that enter the pit have the potential to erode the pit wall and form a head cut that could capture the thalweg of the creek, diverting all subsequent low flows into the mining pit. In response to these concerns, Tetra Tech was tasked with conducting a hydrologic analysis of the Homestead and Whaleback Creek catchment areas along with a hydraulic assessment of flow conditions in the vicinity of OB25.

Figure 1 shows the relative locations of OB25, Homestead Creek, Whaleback Creek, and the Fortescue River. Figure 2 shows the individual mining pits of OB25 and mining infrastructure relative to the Homestead Creek channel.

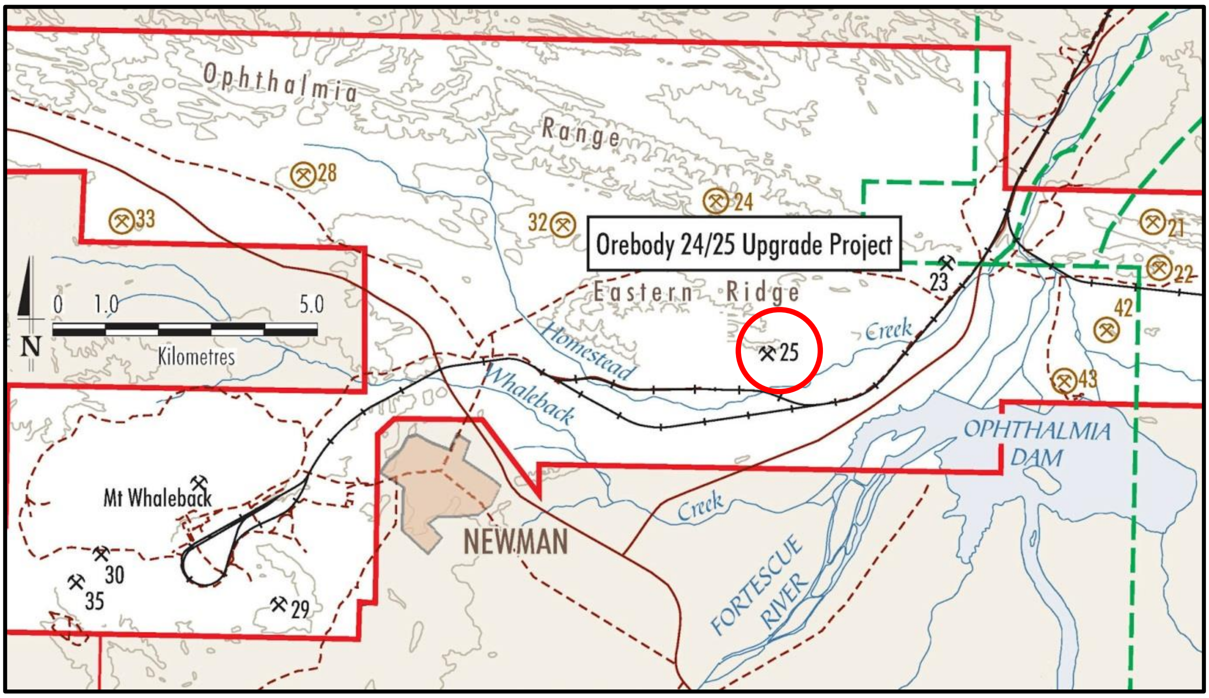


Figure 1. Regional mining tenements showing OB25 and Homestead Creek vicinity (BHPBIO 2010)

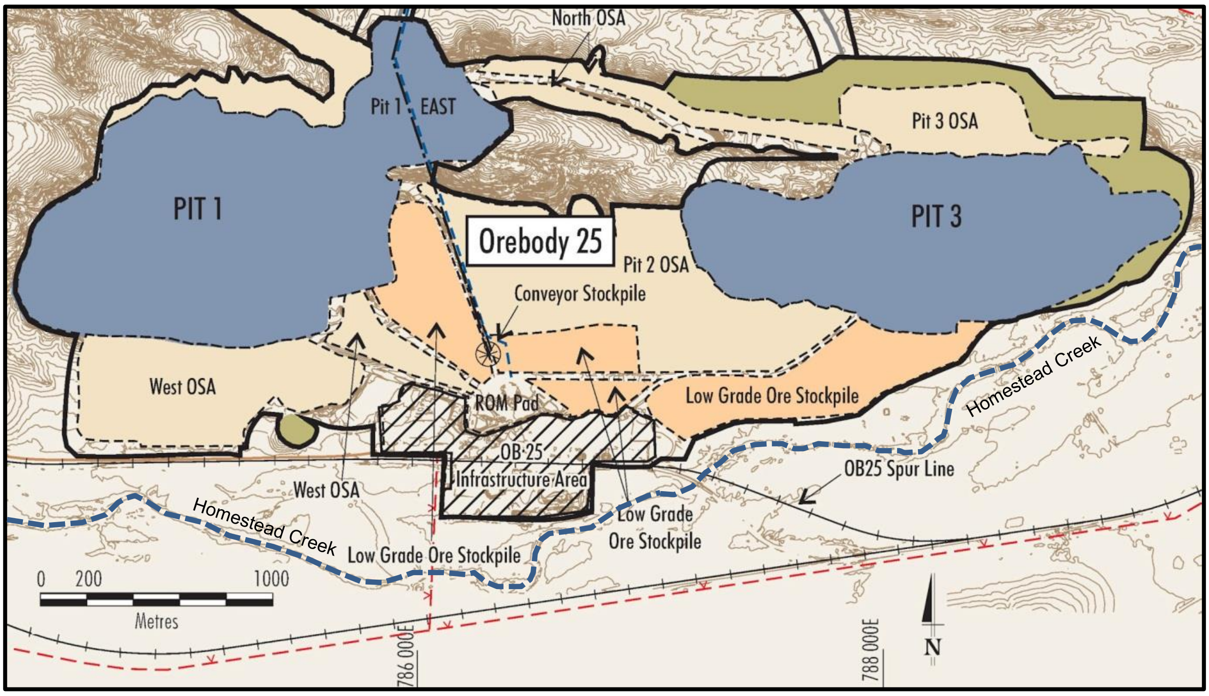


Figure 2. Homestead Creek and Orebody 25 mining pits (BHPBIO 2010)

1.2 Project Purpose

As stated in the Pilbara Closure and Rehabilitation Regional Management Strategy (CRRMS) (BHPBIO 2014), surface water hydrology assessments of mine sites should include a surface water model that is used to ensure that engineered landforms are stable over the long term. Modelling assists in predicting post-closure surface water quantity and quality and assessing the likelihood that mine voids will capture creek lines, or that major climatic events will result in damage to surface water controls that may in turn impact future groundwater/surface water interactions and hence, long term water balances.

Figure 3 shows a Western Australian example in which a flood bund failed after heavy rains in February 2014, diverting the adjacent creek flow into the open pit. The dewatering and pit wall stabilisation required after such a scenario can force an operational halt that lasts for many months. If floodplain flows spill into an open pit in the post-closure timeframe when no maintenance activities are undertaken, a head cut may form and capture the entire creek flow, trapping all future bed load and all low flows indefinitely. The outputs of the surface water models are used as inputs into closure planning strategy, and are used in the early operational stages of a mine to subsequently inform conceptual closure landform design and options assessments for solutions to prevent creek capture.

Model iterations are expected to evolve over the life of mine commensurate with increased data availability to validate the assumptions and increase statistical confidence, thereby improving the final closure strategy and landform detailed design. A major legacy risk for mine voids, commensurate with the hydrological and hydrogeological environment, is the formation of pit lakes. Depending on the costs, benefits, and impacts, available options for mine void closure may include a permanent pit lake (with no significant backfill), partial pit backfill below the water table, partial pit backfill to a specified height above the water table, or complete backfill to the natural surface. An assessment of the long-term geotechnical stability of the pit walls is also critical to the strategic decision. This project provides details regarding the probability of pit capture that will inform further closure planning.

Figure 3. Overflow into open mining pit following heavy rainfall February 2014 in Western Australia

1.3 Report contents

This report presents the existing hydrologic interactions related to the Homestead Creek and OB25 catchment areas and identifies key areas of concern related to post-closure drainage management. The study includes an assessment of the probability that Pit 3 will capture Homestead Creek during a post-closure period of up to 10,000 years and includes recommendations for preventing creek capture. This report covers the background assumptions, approach, and results of the following analyses:

- Homestead Creek and Whaleback Creek Hydrology
  - Catchment and subcatchment delineation, flood frequency analysis, and regional flood frequency methods for estimation of peak flows
  - RORB modelling for estimation of flood hydrographs
  - CRC FORGE analysis for extreme peak flows exceeding 100-year ARI
  - GTSMR analysis for determination of Probable Maximum Precipitation (PMP) and Probable Maximum Flood (PMF)
- Hydraulics
  - HEC-RAS modelling of Homestead Creek/Whaleback Creek interaction
  - HEC-RAS modelling of Homestead Creek flow conditions adjacent to OB25 with calibration to observed events
  - Lateral weir modelling for determination of discharges into OB25 Pit 3
- Erosion and Sedimentation
  - Sediment transport modelling of erosion from overflow into OB25 Pit 3
  - Qualitative geomorphic assessment of lateral migration potential
- Options development
  - Identification of areas of drainage concern for mine site
  - Identification of options for stabilising a closure bund

2.0 PREVIOUS REPORTS

2.1 Previous Reports

As listed below, five existing hydrological reports related to Homestead Creek and OB25 were reviewed in the context of this study. Geotechnical, hydrogeological, and mine planning reports were also reviewed for relevant content.

2.1.1 Drainage Engineering – OB25 Railroad Spur

This report was prepared by Golder Associates for MPDV and was published in September 2008 under Report #07764135-R01. The report presents hydrologic studies conducted to estimate water levels during extreme flood events, with the results used to assist in the design and sizing of culverts, levees, and sacrificial embankments for a railroad spur with two Homestead Creek crossings. The peak design discharges were adopted from Halpern Glick Maunsell (HGM 2000). The estimates are based on Flavell's Regional Method.

Table 1. Design Flood Discharges (m3/s) for Homestead Creek (HGM 2000)

2-year ARI	5-year ARI	10-year ARI	20-year ARI	50-year ARI	100-year ARI
31	65	103	160	283	436

Hydraulic results from a one-dimensional HEC-RAS model are presented for the 20-year and 50-year ARI events. A set of 8 CMP culverts with a diameter of 3000 mm was modelled at each Homestead Creek crossing. The culverts have a 20-year ARI peak discharge conveyance capacity. Levees along the overbanks are designed to route flows through the culverts, with excess flows routed over the sacrificial sections of the spur line. The design also includes specifications for rock armouring to resist hydraulic forces.

2.1.2 OB25 Homestead Creek Flood Study – Selection Phase

This report was submitted by RPS Aquaterra to BHPBIO on 2 April 2012 as Document Number 1496B 005a. The report presents HEC-RAS hydraulic modelling results with Homestead Creek flood depths and velocities adjacent to the OB25 development area. The model adopts the 436 m^3/s 100-year ARI discharge from HGM (2000).

Two scenarios are modelled: 1) the existing (2012) condition and 2) the proposed condition with an extension of OB25 Pit 3 into the northern floodplain of Homestead Creek. Under the proposed condition, flow is obstructed by development to within 50 m of the creek centreline. The encroachment of the development results in localised water surface elevation increases of up to 0.5 m, with localised velocity increases in velocities of up to 70%.

The report also presents concept designs for a bund to protect OB25 Pit 3 from flooding under 100-year ARI discharges. The conceptual design is for a 3 m high, 23 m wide bund with side slopes between 2H:1V and 3H:1V. Recommendations include buffer zones at five locations to avoid localised increases in depth and velocity. Rock armouring would not be required so long as the buffer zones are adopted. The report notes that the 100-year ARI design will not be adequate for closure.

2.1.3 OB25 Homestead Creek Flood Study – Selection Phase

This report was submitted by RPS Aquaterra to BHPBIO on 2 May 2012 as Document Number 196B 005b. The report updates the findings from the earlier Selection Phase report with reduced areas of impact for the extension of OB25 Pit 3. Localised changes in depth and velocity are similar to those estimated in the initial report for impacted areas.

2.1.4 OB37 Preliminary Surface Water Assessment

This report was submitted by RPS Aquaterra to BHPBIO on 12 April 2012 as Document Number 1496C 003a. The report presents the interaction between the Whaleback Creek and Homestead Creek floodplains for OB37, which is located just south of OB25. Modelling of the interaction area showed that up to 35% of Whaleback Creek overflows to the north of a split point in Whaleback Creek in the 100-year ARI event, running along the southern side of the Newman-Port Hedland railway that contains the Homestead Creek floodplain to the north. The report identifies locations with potential overtopping of the rail line and flow through railroad culverts into the Homestead Creek floodplain. The report figures include floodplain delineations and flow directions for the split flow scenario.

2.1.5 Orebodies 23, 24, and 25: Surface Water Review

This report was submitted by Aquaterra to BHPBIO on 19 Dec 2007 as Document Number 68/B2 107b. The report provides subcatchment delineations and general descriptions of drainage patterns across OB23, OB24, and OB25. The report also presents recommendations to minimise ponding, reduce uncontrolled runoff into pits, and mitigate other flood risks. Recommended measures include pit perimeter flood protection, diversion channels, culverts, and bunding. Although the overall drainage paths presented in this report are somewhat applicable to the current condition, significant changes to the mine site infrastructure, pit dimensions, and overburden storage areas (OSAs) at OB25 have occurred since 2007, rendering some of the catchment delineation details no longer relevant.

3.0 SITE VISIT

3.1 Route

A site visit was conducted 21 January 2014. Attendees were BHPBIO's Iain Rea and Rebecca Wright and Tetra Tech's Krey Price. Figure 4 shows the site visit route relative to the Homestead Creek and Whaleback Creek catchment boundaries.

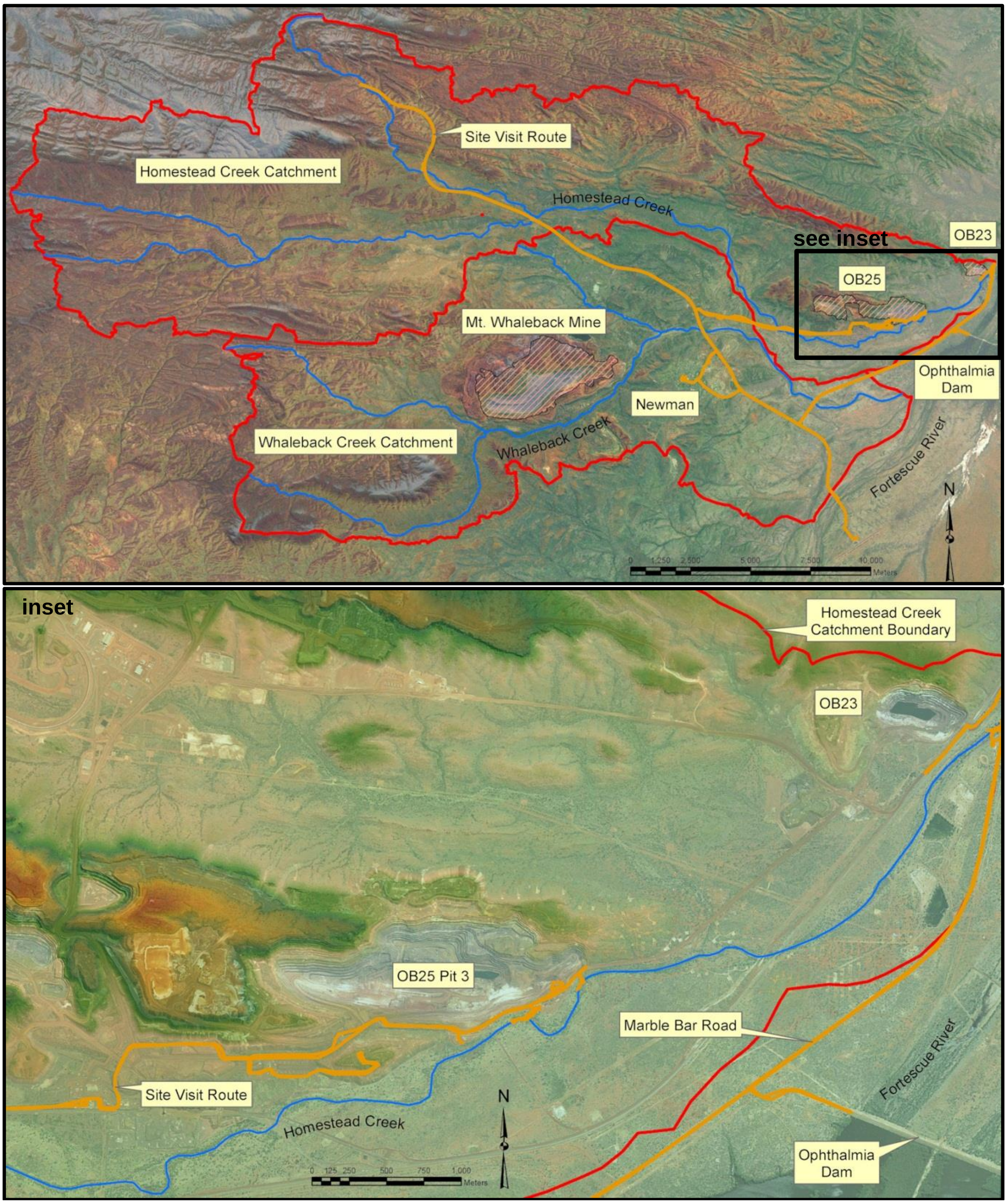


Figure 4. 21 January 2014 site visit route

Selected photographs from the site visit are included in Appendix A, with photo locations presented in Figure A-2.

The site visit included the following areas:

- Upper Homestead Creek catchment at several roadway crossings
- Whaleback Creek at the Great Northern Highway crossing
- Ophthalmia Dam
- Homestead Creek at Marble Bar Road
- OB25 Pit 3 and selected waste rock dumps
- Homestead Creek at OB25 Pit 3

3.2 Sediment Sampling

During the site visit, Homestead Creek was accessed in the two areas closest to the outer extent of OB25 Pit 3. Pebble counts were taken of the armour layer along the main channel, and bank and bed material substrate samples were collected for sieve analysis. Laboratory results are included in the erosion and sedimentation chapter and Appendix E.

3.3 Water level measurements

The site visit coincided with a heavy rainfall event that resulted in localised flooding at the mine site and floodway flows at Marble Bar road. Water levels and timing of staff gauge readings were noted to recreate a flood hydrograph for calibration of model results. A hydrologic analysis of the event is summarised in the following chapter with details included in Appendix B.

4.0 HYDROLOGY

This chapter outlines the hydrologic modelling approaches undertaken to estimate peak flow rates and hydrographs associated with extreme flood events in the Homestead Creek and Whaleback Creek catchment areas.

4.1 Hydrological Setting

Homestead Creek and Whaleback Creek are located within the Fortescue River catchment upstream of the Fortescue Marsh. Figure 5 shows the relative location of the catchment areas to Newman and the major Northwest Australian watercourses.

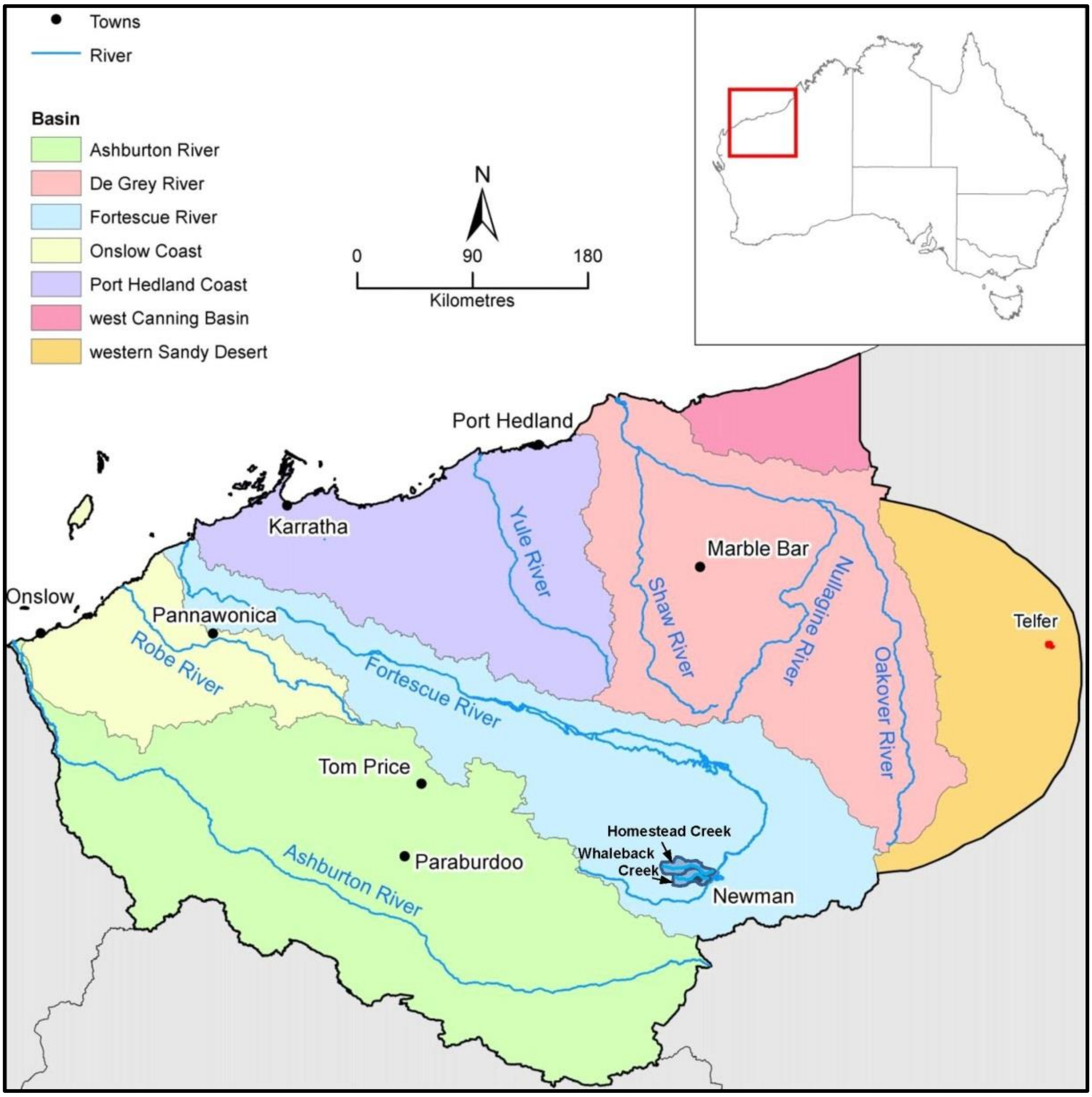


Figure 5. Fortescue River and other Pilbara Region catchments

4.2 Catchment Delineation

As shown in Figure 6, the Homestead Creek catchment area was broken into 8 subcatchments ultimately draining to the Fortescue River downstream of Marble Bar Road. Homestead Creek drains approximately 305 km^2, with a total main channel length of approximately 50 km. Whaleback Creek was broken into 7 subcatchments ultimately draining to Ophthalmia Dam. Whaleback Creek drains approximately 215 km^2, with a total main channel length of approximately 35 km.

Mine pit catchments (1,116 ha for Mount Whaleback, 47 ha for OB23, and 320 ha for OB25) were delineated and removed from the model. Some of the mine pit runoff would be directed back into the channels as part of dewatering efforts; the additional discharge would affect flow volumes unaccounted for by the computed catchment area, however, peak flow rates would not likely be affected since excess water pumped from the mining pits would tend to occur after peak flows in the tributaries have passed.

Subcatchment area designations are shown in Figure B-2 in the Hydrology Appendix. Subcatchment centroids, reach lengths, and drainage areas are shown in Figure B-3.

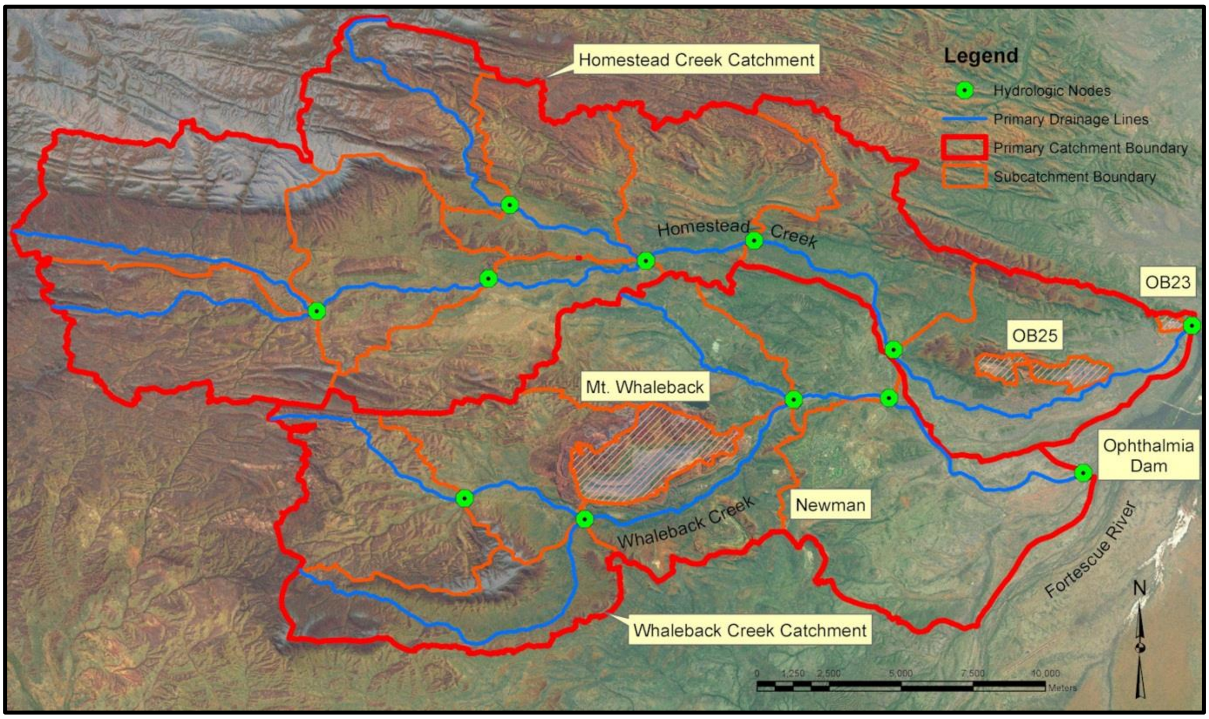


Figure 6. Homestead Creek and Whaleback Creek Subcatchment Delineation

4.3 Flavell's Method

Flavell's Method (2012) was applied to the Homestead Creek and Whaleback Creek catchments. The method has been updated since it was applied in the 2000 HGM study. Table B-1 shows the computations and peak flow results for 2-year through 100-year ARI flood events. The Homestead Creek peak flow at the Fortescue River outlet is calculated as 550 m^3/s. This discharge is approximately 25% higher than the estimate previously cited by HGM. The Whaleback Creek peak flow at Ophthalmia Dam is calculated to be 450 m^3/s.

4.4 Flood Frequency Analysis

Flood frequency analyses were conducted for the available period of record using the following flow gauges:

- Fortescue River at Newman. Gauge #708001. Period of record: 1980-present.
- Marillana Creek at Flat Rocks. Gauge #708011. Period of record: 1967-present.

Peak flow rates from the 22 January 2014 flood event were acquired from the Department of Water and added to the previously obtained records. This event resulted in a peak discharge of 797 m^3/s at Fortescue River (approximately 20-year ARI event) and 84 m^3/s at Marillana Creek (approximately 2-year ARI event).

The 100-year ARI peak flows are estimated as 1,980 m^3/s for Fortescue River and 1,710 m^3/s for Marillana Creek. Plots and tables of the flood frequency analysis are included in Figure B-35 and B-36 and Table B-7 in the Hydrology Appendix.

4.5 RORB Modelling

4.5.1 Schematic drainage network

Figure B-1 shows a schematic RORB catchment network for Homestead and Whaleback Creeks with nodes, inflow areas, and reaches. Length and area measurements used in the model are shown in Figure B-3. The model joins Whaleback Creek and Homestead Creek for the convenience of keeping all computations in a single model; the junction is hypothetical only since both creeks reach the Fortescue River separately without joining.

4.5.2 Loss rates

Initial and constant loss rates were applied as recommended in Australian Rainfall and Runoff (ARR) (Engineers Australia 1998). ARR includes loss rates through the 50-year ARI event. The 100-year loss rate is extrapolated from the given rates. A sensitivity analysis using 100-year ARI initial loss rates varying from 10 mm to 20 mm is provided in Table B-5.

Table 2.Initial loss rates (Engineers Australia, 1998)

2-year ARI	5-year ARI	10-year ARI	25-year ARI	50-year ARI	100-year ARI
22	40	52	47	32	[10]

Extreme floods in excess of the 100-year ARI flood use a loss rate of 0 mm, representing a saturated antecedent condition, which is generally reasonable for extreme cyclonic events typical of the Pilbara. All model runs use a constant continuing loss of 5 mm per hour in accordance with ARR recommendations for the Pilbara Region.

4.5.3 Storativity coefficient K_c

Hydrographs were run for a variety of K_c values using accepted, industry-standard methodologies. In addition to regional equations, Flavell's recommended approach of using 0.96 d_{av} or 1.14 d_{av} was applied. The 1.14 d_{av} approach, yielding a K_c value of 33.84, produced results most in line with the average predicted discharges using regional, rational, and other methods.

4.5.4 Critical duration

With very few exceptions, the 24-hour duration event produced the highest peak discharges at each node and return interval. A 24-hour duration was thus adopted as the storm duration for consistency between all model runs.

4.5.5 Other parameters

The non-linearity parameter *m* is set at 0.8 in accordance with standard practice. The Zone 7 temporal pattern was applied with localised Intensity-Frequency-Duration data from the Bureau of Meteorology (BOM).

4.6 CRC FORGE

Table B-2 in the Hydrology Appendix presents the CRC FORGE areal rainfall quantiles and extrapolated areal rainfall quantiles to PMP.

4.7 GTSMR

Figure B-22 in the Hydrology Appendix includes the PMP Method Selection worksheet for the Generalised Tropical Storm Method Revised (GTSMR). Location information and PMP values are included in Figure B-23.

4.8 Summary of Adopted Peak Discharges

Figure 7 shows a summary of extreme flood hydrographs applicable to Homestead Creek adjacent to OB25. Figures B-4 through B-13 show rainfall excess hyetographs and runoff hydrographs computed in RORB for the 5-year through 100-year ARI storm events. Results are shown for the selected node locations shown in Figure B-3. Figures B-14 through B-21 show extreme flood hydrographs for the 100 through 10,000-year ARI flood using the same node locations.

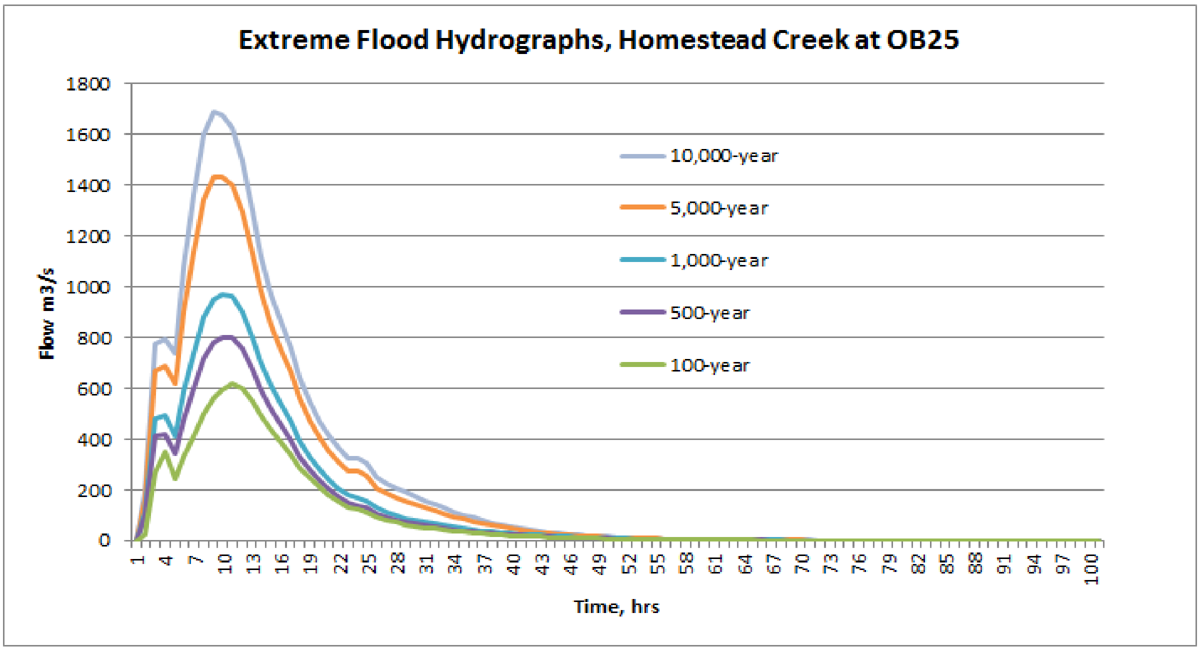


Figure 7. Summary of extreme hydrographs at OB25

4.9 Calibration

The site visit was undertaken during a heavy rainfall event, providing an opportunity to calibrate observations to modelled results. Figure B-24 shows the rainfall record at Newman Airport for the period of record from 1971 to present. As shown in Figure B-25, the maximum daily rainfall during the 20-22 January 2014 event was 60 mm, measured from 9 am on 21 January to 9 am on 22 January. According to BOM Intensity-Frequency-Duration (IFD) data (see Figures B-26 and B-27), this corresponds to a 2-year ARI precipitation event. 114 mm was recorded over the 3-day event, corresponding to a 72-hour rainfall with an ARI of just under 5 years.

Since daily readings at this gauge are tabulated from 9 am to 9 am, 30-minute readings were downloaded to determine the actual maximum rainfall intensity. A 24-hour precipitation depth of 80 mm was recorded from 9 pm on 20 January to 9 pm on 21 January. This corresponds to a 5 year rainfall depth.

In comparison, the rainfall gauge at Marillana Station received almost 200 mm of rainfall over the same period, corresponding to an ARI of 10 to 20 years. Marillana Creek at Flat Rocks recorded 110 mm, corresponding to a 5-year rainfall, with a 2-year runoff. These comparisons demonstrate the highly variable nature of the same rainfall event across the Pilbara Region.

IFD data from BOM are included for the 1987 data set (currently in use) as well as the 2013 data set (pending approval). A comparison is shown in Table B-3. Whilst the updated values are not recommended for current modelling efforts, it is notable that the comparison indicates that a 10% increase may be expected once the updated values are adopted.

A rating curve was developed for Homestead Creek at Marble Bar Road to estimate culvert and weir flow at various depths of flow. Because this area is very close to the Fortescue River, backwater effects can have a substantial impact on the conveyance capacity associated with a given water surface elevation. Figure B-28 shows the rating curve under the assumption of normal depth conditions downstream, whereas Figure B-29 shows a rating curve under backwater conditions from the downstream Fortescue River. Photographs of the flood event (Figure B-32 and B-33) show significant backwater conditions at higher stage levels. The submerged weir routine represented in the rating curve gives a peak flow of approximately 85 m^3/s for the maximum recorded stage reading. This is well in excess of the estimated 5-year ARI peak discharge that might be expected given the rainfall intensities.

Because the selected storm pattern in the RORB model is front-loaded (heaviest rainfall intensities at the beginning of the storm), the discharges are very sensitive to the initial loss parameter. Varying the initial loss parameter from 40 mm to 20 mm yields a 5-year discharge equivalent to the 85 m^3/s computed discharge. A comparison of runoff volumes (See Table B-4) using a 20 mm initial loss likewise gives equivalent results to the RORB model. As a comparison to previous studies, the RORB model yields a peak 100-year ARI Southern Creek discharge of 111 m^3/s, approximately twice the previous BHPBIO estimate of 58 m^3/s. The RAFTS model previously applied may have used a higher initial loss than the relatively conservative 10 mm adopted in the RORB model.

Volumetric comparisons were also conducted for the Homestead and Whaleback Creek computed hydrographs, and the flow volumes were found to give reasonably close representations of the runoff fraction measured in nearby gauged catchments.

5.0 HYDRAULICS

The flows presented above were incorporated into a hydraulic model to determine velocities, depths, and overflow rates associated with each of the modelled flood events along Homestead Creek and in the Pit 3 vicinity. Hydraulic results are shown in Appendix C. Note that cross section and profile views are shown with extreme vertical exaggeration.

5.1 HEC-RAS Model Setup

A one-dimensional, step-backwater hydraulic model was created in the software package HEC-RAS (USACE 2010) to model Homestead Creek and Whaleback Creek from the Newman townsite to the Fortescue River.

5.1.1 Cross sections and surfaces

Figure 8 shows a plan view of the cross section locations that were cut through the 3-dimensional surfaces representing the existing and ultimate conditions. Metadata for the topographic files used in the model is provided in Appendix G. Because the interest of this study is in flows exceeding the 100-year ARI, the main channel was delineated along the main direction of the flood flows (without necessarily following the individual bends of the low flow channel) and cross sections were bent to generally remain perpendicular to the direction of flow in extreme flood conditions. Bends were also incorporated into sections bounding railway and road crossings. The cross sections extend across both the Homestead Creek and Whaleback Creek floodplains to allow consistent modelling and viewing of overflows and interactions between the floodplains.

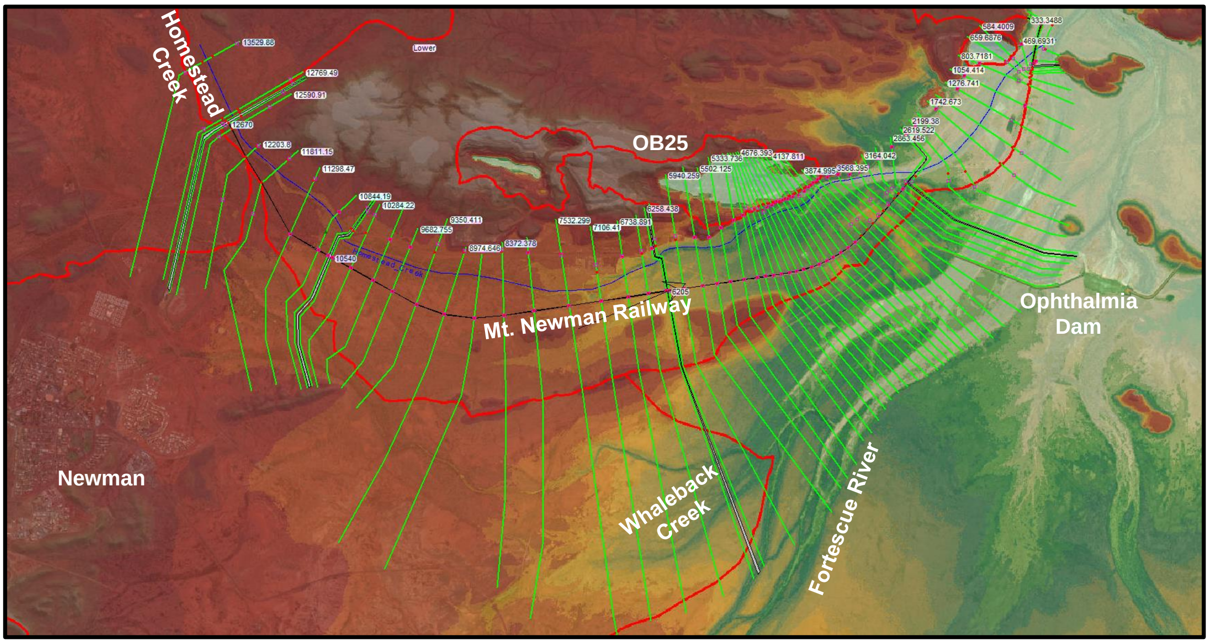


Figure 8. HEC-RAS Geometry Layout

5.1.2 Geometric features

Five bridge/culvert crossings were added to the model:

- Railway loop pipe culverts
- OB25 rail spur pipe culverts (upstream crossing)

- OB25 rail spur pipe culverts (downstream crossing)
- Mt Newman railway trestle
- Marble Bar Road floodway and concrete box culverts

Culvert data (invert elevations, material, diameter) are derived from Golder (2008). The railroad spur is modelled as 8 x 3000 CMP culverts with a length of 30 m at the upstream crossing and 20 m at the downstream crossing. The railway trestle dimensions were estimated from aerial photographs of the abutments. The Marble Bar floodway dimensions were derived from the 5-m DEM surface with field measurements of the box culverts taken from the site visit.

The Mount Newman railway embankment was modelled as a lateral weir allowing flow to pass from north to south (Homestead to Whaleback Creek) and from south to north (Whaleback to Homestead Creek). Some drainage culverts are located across the Mount Newman railway, allowing flow to pass both to the north and to the south across the railway, depending on the relative timing of the flood peaks. Because this study considers only extreme flood flows, the culverts are assumed to be blocked by debris or contributing only a small percentage of the overall discharge and are thus ignored in the model.

Another lateral weir was added to the model to represent the existing access road adjacent to Pit 3. The lateral weirs are located along the bank stations with a weir width of 5m using a standard broad-crested weir equation. Flow optimisation is enabled to allow downstream peak flows to be reduced by the flow rate entering te pit. For closure conditions, the embankment was assumed to be removed, with the existing floodplain ground surface modelled as a lateral weir.

5.1.3 Discharge

The model uses peak discharges derived from the 2-year through PMF hydrographs (see Figures B-4 through B-21). Discharges were modelled as steady-state flows. Flows were run using subcritical regime (downstream to upstream). Flood levels in the Fortescue River above and below Ophthalmia Dam were used to represent the downstream boundary condition.

5.2 HEC-RAS Model Results

5.2.1 Depth and Velocity

Figures C-4 to C-19 in the Hydraulic Appendix show the floodplain extents, water depths, and velocities for the 100-year to 10,000-year ARI flood events. Depth and velocity grids are shown at 5-m intervals. Results are shown for the existing condition (24 March 2014) and the ultimate condition (with the access road removed to the floodplain elevation). The maximum 100-year ARI depths in Homestead Creek adjacent to Pit 3 are approximately 4 m. The creek banks are generally about 2 m high, with 2 m of depth in the floodplain or along the access road embankment. Velocities are generally less than 2 m/s in the same reach.

Overflows over the top of the railway embankment to the south only occur in extreme events (on the order of the 10,000-year ARI). During these overtopping conditions, less than 5% of the discharge is conveyed across the top of the railway embankment. The contributing flow from Whaleback Creek is thus ignored in the Homestead Creek model.

Under existing conditions, only the 10,000-year ARI event spills over the access road to the north into the pit. The overtopping occurs at a single cross section, and the amount of time during peak flow conditions is insufficient to allow erosion of a complete breach.

Under future conditions, with the bund removed, all flows in excess of the 20-year ARI event spill into the pit.

Figure E-26 shows the modelled discharge over the floodplain into Pit 3 relative to the mainstem discharge using the lateral weir function in the HEC-RAS ultimate conditions model. This is the flow rate that would occur at the point of peak flow, assuming the edge of the pit remains at the elevation of the adjacent floodplain. This relationship was used to scale hydrographs for entering unsteady flows into the sediment transport model (covered below).

Figure 9 shows the Stage-Area and Stage-Volume relationship for Pit 3. As a comparison to the flood volumes, the 100-year ARI 24-hour storm runoff volume is approximately 30 GL, which would nearly fill the pit with an initially empty pit. A final groundwater elevation of 500 m RL is assumed once the water table has recovered over the long-term, post-closure period (Snowden 2012). This leaves a storage area of approximately 7.5 GL between the pit lake elevation and the Homestead Creek floodplain elevation. The 100-year ARI storm hydrograph would fill this volume prior to the peak if the entire discharge were routed to the pit (i.e., pit capture had already occurred). The void volume would be filled well after the time of peak flow if the embankment remained in place and only lateral weir flow entered the pit.

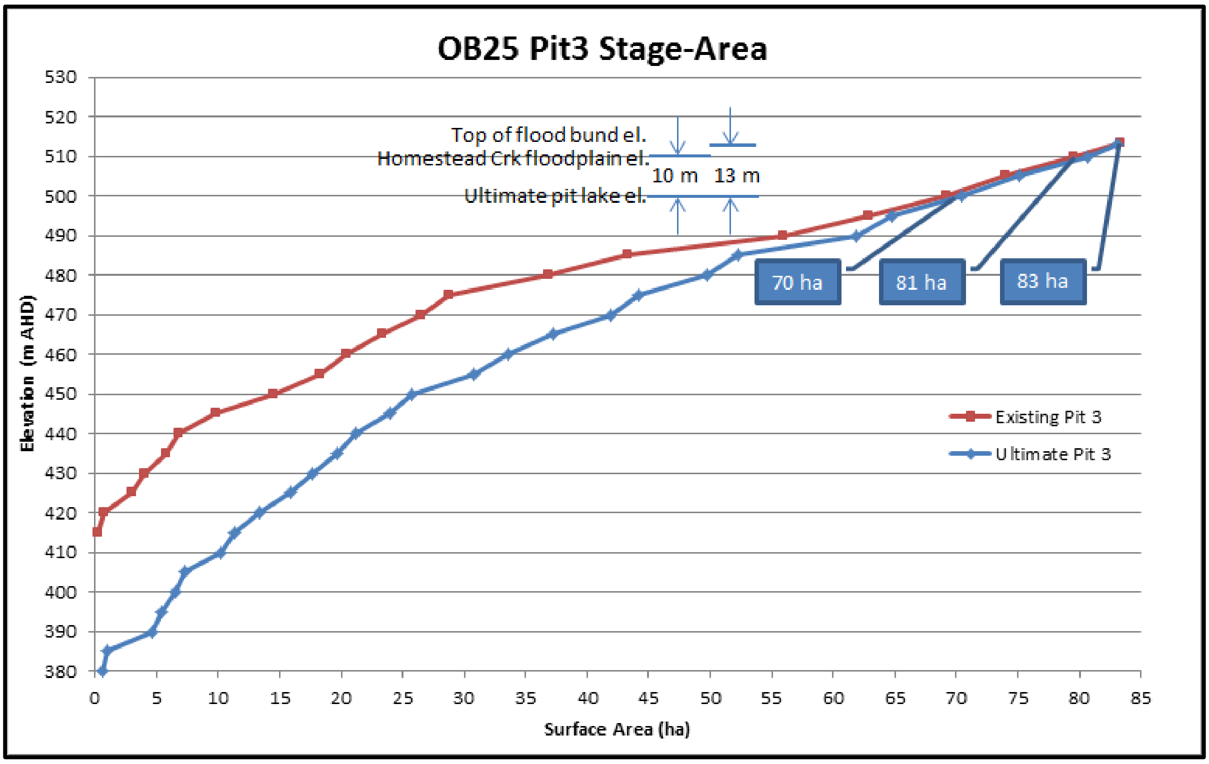
5.2.2 Sensitivity Analysis

A Manning's roughness coefficient (n-value) of 0.035 was used for the channel, with a value of 0.050 used to represent the overbanks. According to the HEC-RAS manual (USACE 2010), a value of 0.035 is appropriate for a channel with weeds and stones. Horizontal variation in n values can be applied to reflect changes in vegetation cover and other localised effects; however, for consistency with previous studies, a single value is used as a composite representation of the channel roughness along the entire project reach.

Because this model targets extreme floods (greater than 100-year ARI), the channel definition is wider than that used in previous models (intended for 100-year ARI and below) and a larger percentage of the flood flow is conveyed in the overbanks. As a result, the actual roughness of the composite channel may be slightly higher than 0.035. If the n-value is underestimated, the results will be more conservative for velocities and less conservative for depths.

In order to quantify this effect, a sensitivity analysis was performed with n-values ranging from 0.025 to 0.05 to represent the composite roughness of the entire floodplain between the railway spur and the main Newman-Port Hedland railway. The difference between the high and low roughness runs is shown in profile and section view in Figures C-23 to C-25. Up to 1 m of vertical change in water surface elevation is observable between the runs. Velocities vary by up to 50%. It is unlikely that the composite roughness would approach either of these extremes; however, some features (patches of heavy vegetation, channel bends, roadway/railway embankments, etc.) could cause the water surface elevation and velocity to vary on this order in localised areas.

Variations in the downstream boundary condition can also affect the water surface elevation and velocity results. A sensitivity analysis was performed on the downstream boundary condition to represent the range of possible water surface elevations in the Fortescue River. In an extreme case, the backwater effect travels upstream approximately 2,000 metres. Figure C-26 shows a long section profile of the downstream reach of the channel. The constriction caused by the Mt. Newman Railway trestle forms a boundary upstream of which the backwater effect is negligible. Conditions at the site are generally unaffected by the downstream boundary condition assigned in the model.



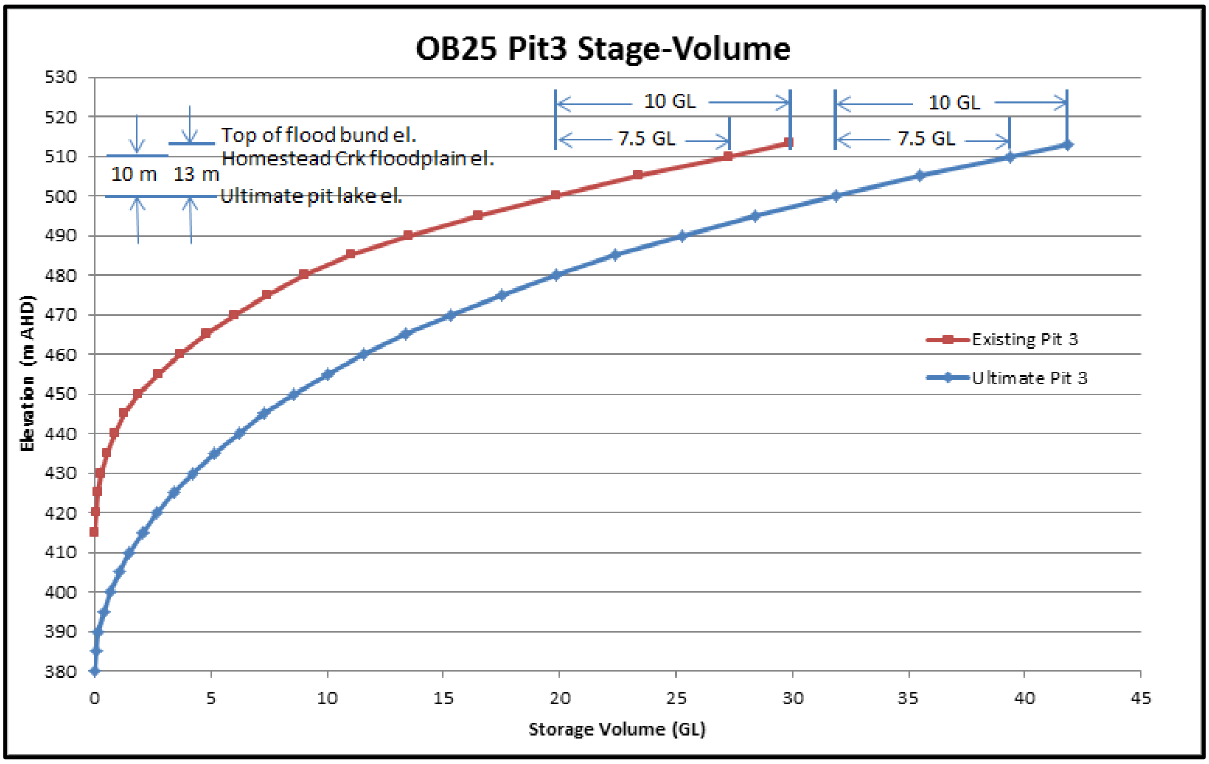


Figure 9. Stage Area and Stage Storage Curves for OB25 Pit 3 Existing and Ultimate Condition

6.0 EROSION AND SEDIMENTATION

6.1 Geology and Geotechnical

Figure 10 shows the OB25 Pit 3 South Wall geology in section view (Snowden 2012).

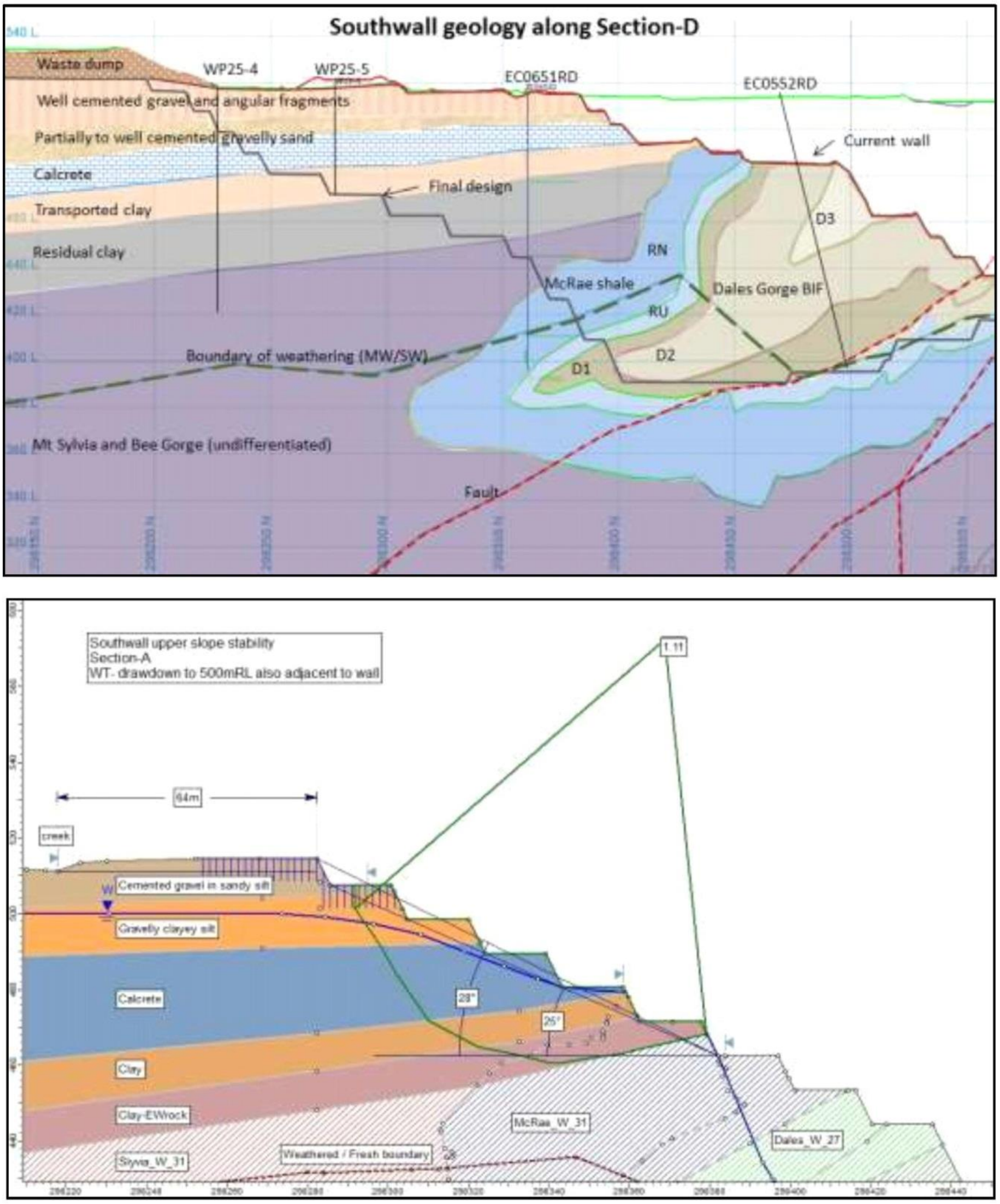


Figure 10. Geological and geotechnical slope stability of OB25 Pit 3 South Wall (Snowden 2012)

Cross section locations are shown in Figure F-2 along with the additional slope stability sections nearest to the overflow point. As shown in Figure 10, the material along the edge of the pit shell requires a relatively mild slope (25%) to achieve a safety factor of 1.2. This represents a typical operational safety factor, which would likely need to be increased for closure conditions, resulting in a milder post-closure ultimate slope. A 1.5 safety factor may be more reasonable for closure conditions. These analyses also reflect a dry pit with a suppressed groundwater table. A stability analysis with a wetted toe would be needed to determine post-closure pit wall stability.

According to the publication *Safety Bund Walls around Abandoned Open Pit Mines* (WA DIR 1997) at closure a safety bund would need to be located a minimum of 10 m from the edge of the potentially unstable edge zone. The operational pit walls appear to meet the final slope criteria; however, additional analyses would be needed to confirm the final stable slope under closure conditions. Figure 11 is derived from the recommendations in the DIR document.

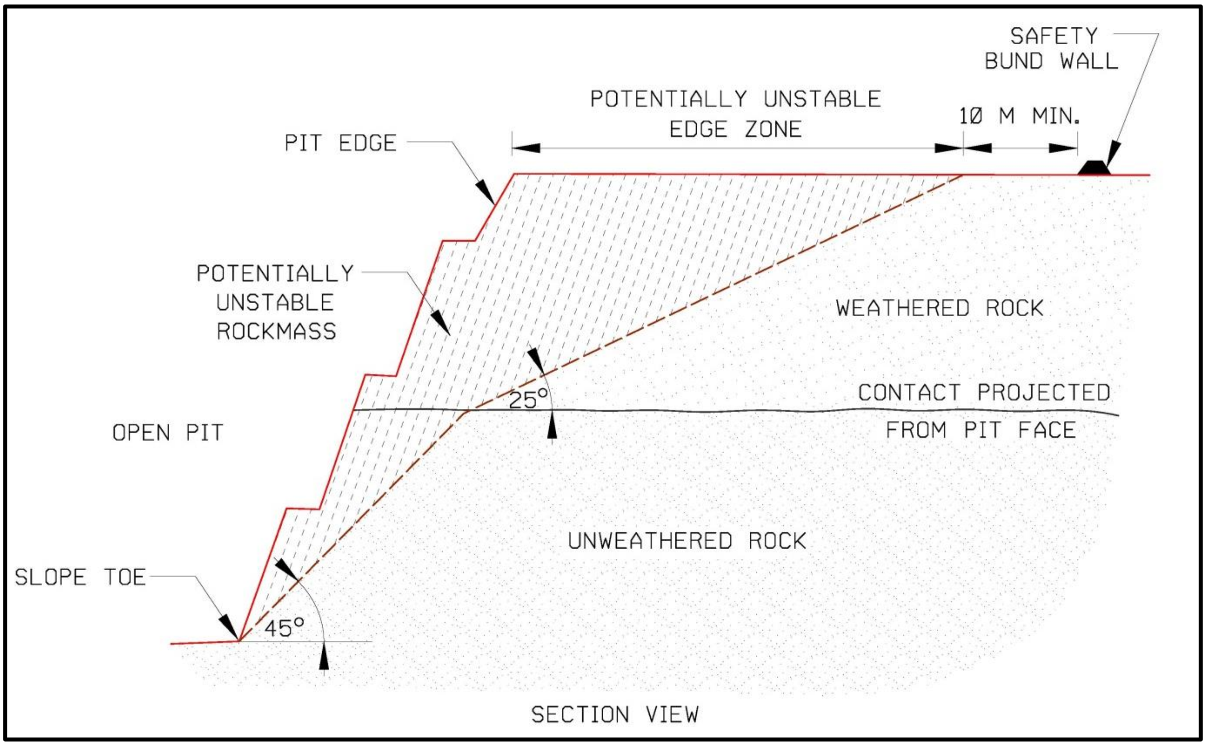


Figure 11. Safety Bund Location (WA DIR 1997)

Figure F-5 shows a cross section taken through the deepest part of the ultimate pit shell where Homestead Creek is in close proximity to the outer edge. As shown in the figure, the ultimate pit shell includes a grade break at approximately 460 m RL. This may indicate the presence of more stable material in the lower pit void; however, the boundary of weathering is generally shown in the Snowden report as being below the grade break. As shown in Figure F-5, if a 25 degree slope were applied extending from the toe of the pit, the intersection with the existing ground surface would extend into the main channel of Homestead Creek.

The geological and geotechnical data confirm the sediment transport modelling assumption that the tertiary sediments (detritals in the bore logs) are cemented and erosion resistant.

6.2 Historical Aerial Photo Analysis

Figure 12 shows a comparison of the 2003 aerial photograph with the 2014 photograph in the vicinity of OB25. The comparison shows no discernible change in the thalweg location. Some changes in vegetation are apparent. If earlier aerial photographs become available, a more thorough comparison could be made, particularly if a photograph prior to 1999 could be identified. The largest rainfall on record occurred in 1999 and a detailed aerial photograph comparison could identify event-based changes in flow path, banks, and vegetation. As shown in Figures C-4 to C-11, historical low-flow channels and meander bends are apparent from an analysis of the available LiDAR data.

6.3 Geomorphology

A photographic record of the geomorphic features upstream, downstream and within the proposed pit area is included in Appendix A. Based on sediment sampling efforts, site observations, and a review of the available topography and photography, historical lateral migration of the Homestead Creek channel appears to be very limited in the vicinity of Pit 3. No evidence of significant changes in channel planform was identified in the topography, and the material in the channel banks does not appear to have been disturbed in the recent past since it contains pedogenic calcium carbonate (calcrete) accumulations. The absence of bar and scroll topography that is characteristic of laterally meandering channels, along with observations of the size of the trees growing on the banks (surrogate for stability) and the presence of the calcium carbonate in the banks indicates that the channel banks have not shifted by more than a few meters over the past several centuries. Historical overbank channels that are likely the result of flood-induced in-channel sediment deposition and avulsions are discernible in LiDAR-based topographic mapping; however, these channels are likely very old. Whilst long-term changes in the bed profile are not likely due to the presence of both ferricrete and calcrete (groundwater derived) cementation of the valley-fill sediments, some localiSed lateral change would be expected over a 10,000-year post-closure duration. Localised scour adjacent to an armoured levee section would have the potential to undermine the structure and lead to lateral migration of the channel. As such, scour protection is recommended to below the depth of the existing Homestead Creek channel thalweg.

This conclusion is consistent with findings from studies conducted on other channels in the Pilbara Region (Tetra Tech, 2012). Considering the relatively long time period that is encompassed in the closure period, very rapid changes to the channel alignment as a result of localized sediment deposition during cyclonically-induced flooding (i.e., channel avulsions) are a real possibility (Graf, 2002; Tooth and Nanson, 2004; Harvey et al., 2014). However, it is much more likely that creek capture would occur as a result of overtopping lateral discharges and headward migration of channel incision, and not as a result of changes to the channel planform or erosion of the left (north) bank.

As with most of the small to medium drainages in the Pilbara, sediments derived from the catchment are primarily deposited on an alluvial fan well upstream of the site that grades out onto the floodplain of the receiving stream. Also in common with many of the smaller drainages in the Pilbara, the upstream fans are incised, which is indicative of a low present-day sediment supply from the watershed due to extensive armouring of the hillslopes by both bedrock and bedrock-derived colluvium (Schumm et al., 1987). Relatively low watershed sediment supply tends to reduce the potential for lateral migration since deposition on the inside of bends tends to drive erosion of the outer bank and hence channel migration (Van de Lageweg et al., 2014).

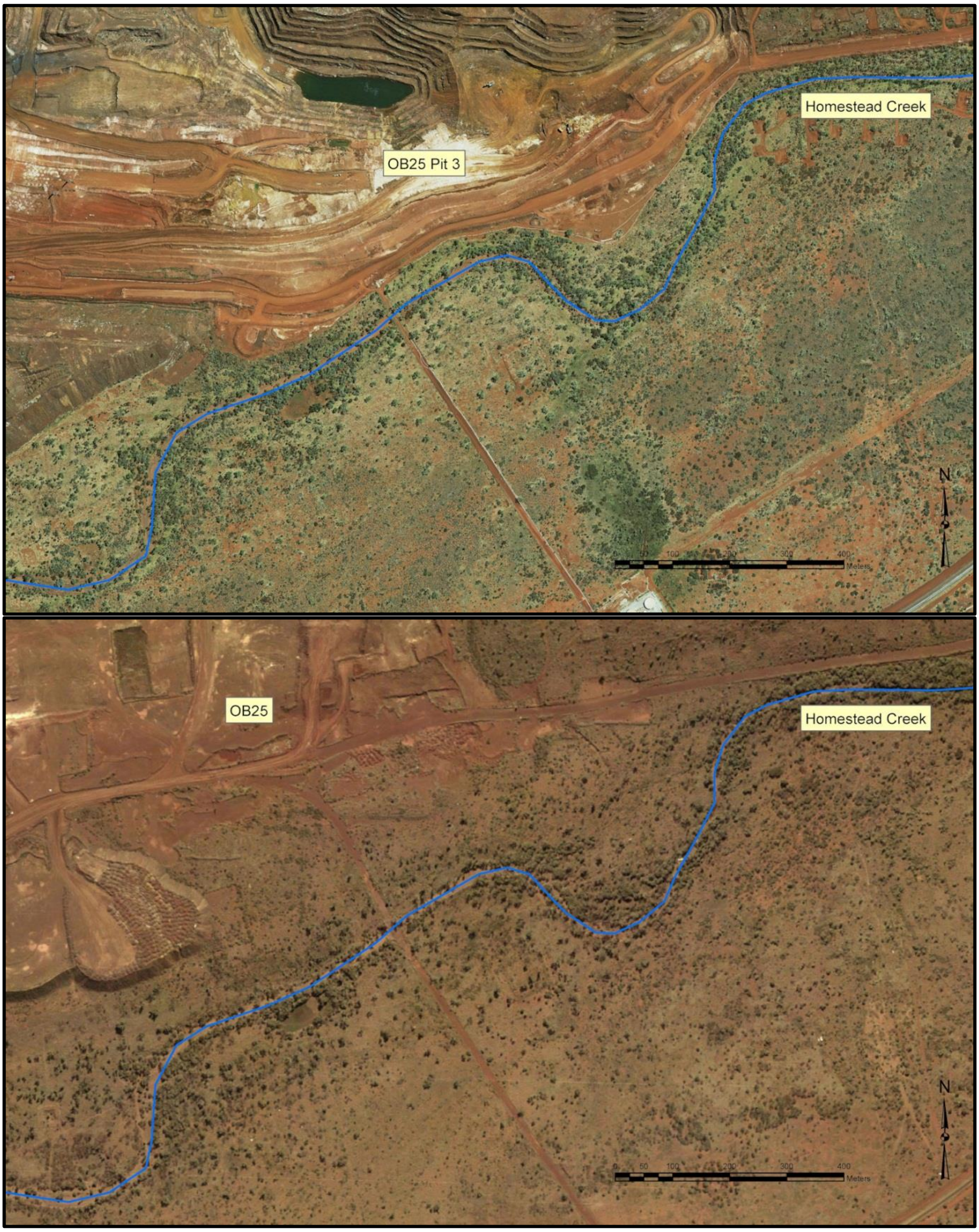


Figure 12. Comparison of 2003 and 2014 Aerial Photographs

6.4 Erosion Modelling

6.4.1 Substrate material

Sediment sampling was conducted on 21 January 2014 to characterise the size of the bed and bank material. Two samples of the armour material that provides the morphological framework of the channel were collected from the channel boundary material in the vicinity of the locations where Homestead Creek is nearest to the Pit 3 South Wall under closure conditions using the pebble count technique (Figure E-3). The sampling also included two bulk samples of the subsurface bed material in the vicinity of the two pebble counts. A single bulk sample of the channel bank material was also collected from the left (north) bank near the location of Pebble Count #1. Gradation curves from the sampled material indicate that the armour material is primarily gravel, with a median grain size (D_{50}) of about 25 mm, whilst the gradation of the finer material in the bulk bed samples has a D_{50} of between 5 and 10 mm and includes between 15 and 30 percent sand. The material sampled from the bank is much finer, with a median grain size of about 0.1 mm; about half of this material is silt or clay. For purposes of conducting the sediment-transport modelling, a representative gradation curve of boundary materials was developed by averaging the gradations of the pebble counts and bulk samples collected from the bed; the resulting gradation has a D_{50} of about 18 mm (Figure E-3).

6.4.2 Model development

Topographic surfaces used in the sediment-transport analysis are listed in Appendix G. The Homestead Creek capture analysis model was conducted by developing a headcut (sediment-transport) model of the north overbank flow path that would deliver lateral overflows into Pit 3. The analysis is based on the assumption that the current alignment of Homestead Creek will be similar in the future, even though there is some potential for the channel to change its location as a result of lateral migration, thereby either reducing or increasing the lengths of the overbank flow paths depending on whether the channel migrates towards or away from the pit. To analyse the potential for capture, a sediment-transport model of the approximately 340 m-long lateral overflow reach was developed using HEC-6T (version 08u; MBH, 2010), a one-dimensional mobile boundary sediment-transport model. The model extends from about 600 metres upstream from the current location of Homestead Creek along the primary flow path of the mainstem creek and across the location where headcutting would most likely result in creek capture (i.e., the location where the creek is nearest to the pit wall), to the toe of the proposed pit wall under closure conditions (Figure E-2). The model includes 10 cross sections between the creek and the pit toe with an average spacing of about 32 metres and 4 cross sections upstream from the creek to ensure the upstream boundary condition did not affect the model results through the headcut reach. To improve the definition through the area of interest, 9 interpolated sections were added between the top of the pit wall and the creek using the HEC-RAS cross-section interpolation feature.

The model geometry under closure conditions was developed by cutting the cross sections from the raster file of the closure conditions topography discussed in Appendix G. At the top of the pit (Model River Chainage 267.7), a small, 3 metre-wide by 0.1 metre deep pilot channel was inserted to convey very low flows, thereby limiting the lateral extents of erosion when significant headcutting would not occur. Manning's n-values of 0.05 were used along the entire extents of the cross sections.

Model input hydrology was based on the predicted overbank flow hydrograph relationship shown in Figure C-27. The analysis was conducted by developing simulations for the computed lateral overflow hydrographs from the mainstem model at the 100-, 500-, 1,000-, 10,000-year ARI floods and the Probable Maximum Flood (PMF) event. The water-surface elevation representing the downstream boundary condition was initially set to 500 metres based on BHPBIO estimates of the long-term, rebounded groundwater elevation. It should be noted that the water-surface elevation in the pit would increase due to the lateral inflows from Homestead Creek, and this would result in reduced erosion rates to varying degrees for each of the events. The effects of this were evaluated for selected events as discussed in the results section.

As discussed above, the representative bed material gradation for the modelled reach was developed by computing the average gradation of bed material surface and subsurface samples collected by Tetra Tech at two locations along the mainstem of Homestead Creek. In the absence of site-specific subsurface sediment data, it was assumed that the bed material gradations from the main stem would be reasonably representative of the sediment gradations along the alluvial deposits through which the headcut would form. A review of the hydrogeology boring logs (Appendix D) indicates that the alluvial materials near the surface that makes up the modelled bed sediments are typically Haematite gravels, so this assumption appears reasonable. A specific gravity of 2.65 was used for the bed material, corresponding to a specific weight of about 26 kN/m^3 for the individual sediment particles. A bulking factor of 0.56 was used to estimate the specific weight of the deposited material (about 14.6 kN/m^3).

6.4.3 Lateral limits

The lateral limits of the bed sediment reservoir (the limits of the existing alluvial deposits that could be subjected to erosion) were initially set to a width of about 100 metres along the assumed location of the headcut. Initial test runs indicated that for the 100-year ARI flood event simulation, creek capture would occur in less than 4 hours of a single event. The lateral limits were subsequently located along the portion of the cross-sections that represent the maximum width of creek capture (about 570 metres) based on the portion of the capture cross section that represents the mainstem channel. This assumption results in longer estimates of time to creek capture since more narrow lateral erosion limits would result in higher rates of incision and therefore more rapid headcut migration. A depth of 6 metres was used to define the bottom of the mobile bed sediment reservoir based on information from the hydrogeology boring logs (Appendix D). Bed material sediment supply to the upstream end of the modelled reaches was assumed to be negligible because bed material sediment concentrations in the overbanking flows from the mainstem of Homestead Creek will be very low.

6.4.4 Flow duration and time steps

The overall flow duration associated with the hydrologic input is broken down into smaller computation intervals to insure the effects of changes in bed geometry are appropriately accounted for in the hydraulic computations. As discussed in the HEC-RAS User's Manual (USACE 2010), "…smaller computation increments will increase (model) run time, re-computing geometry and hydraulics too infrequently (e.g., computation increments that are too large) is the most common source of model instability." For this study, the computation interval was determined using procedures outlined in the Corps' Guidelines for the Calibration and Application of Computer Program HEC-6 (USACE 1992). These time-steps were then further reduced for flows that resulted in oscillations in the predicted gradation of the active layer. The resulting time steps range from 30 seconds for flows that exceed 28.3 m^3/s to 60 minutes for flows that are less than 0.3 m^3/s (Table 3).

Table 3. Computation Increments used for the sediment-transport simulations.			
Minimum Flow (m^3/s)	Maximum Flow (m^3/s)	Computation Increment (minutes)	Computation Increment (seconds)
0	0.3	60	3600
0.3	1.4	30	1800
1.4	2.8	15	900
2.8	5.7	6	360
5.7	14.2	3	180
14.2	28.3	1.000	60
28.3	566.4	0.500	30

6.4.5 Constant boundary condition

Results from the simulations with the constant water-surface elevation in the pit (500 m RL) indicate that creek capture would occur during a single event of all the flood events analysed (Figures E-4a through E-4f). Relative to the start of runoff, creek capture would occur between 9 and 12 hours during the 100-year ARI event, between 6 and 9 hours during the 500- and 1,000-year ARI events, and between 3 and 6 hours during the remainder of the events. The predicted profiles also indicate that the headward migration of incision is reasonably approximated, and is highlighted by the predicted changes in cross sectional geometry. Along the top of the pit wall (River Chainage 267.7), the channel downcuts at a relatively constant rate during the rising limb of the hydrograph until the bed contacts the cemented layer (Figure E-5). As would be expected, the entire portion of the cross-section within the erosion limits erodes at this location. Midway between the pit wall and the creek (River Chainage 301.24), the downcutting is limited during the early portion of the simulations and becomes more significant during the rising limb of the hydrograph (Figure E-6). The cross section representing the point of capture along the main stem channel of the creek (River Chainage 341.74) shows minimal change until the point in the simulation when capture is indicated, and continued downcutting through the rising limb of the hydrograph (Figure E-7).

6.4.6 Varying boundary condition

The above scenarios reflect a constant water surface elevation in the pit as the downstream boundary condition. In reality, the pit lake elevation would increase with the inflow. The effect varies from negligible in lower flow scenarios to very significant in extreme flows. The pit water surface limits the vertical extent of erosion by inflows. Since the energy is absorbed in the turbulence of the water as the plunging flow reaches the pit lake, pit wall erosion is limited to within a few metres of the water surface, below which there is no discernible effect from the inflow. Whilst the rate of erosion would slow as the capacity of the storage area is reached (higher initial water level for each successive computational time step), the assumption of no backwater in the pit allows for some reasonable conservatism in the erosion assessment. Assessing the condition of a larger void (with a lower initial water level) is also warranted to account for drought conditions or to accommodate the chance of a major event occurring before the groundwater table has reached its long-term level.

To ascertain the sensitivity of the results to the boundary condition, simulations were developed to include a varying downstream water-surface elevation. The downstream water-surface elevations for these simulations assume that at the start of the simulation, the water-surface elevation in the pit is at 500 m RL based on BHPBIO estimates of the long-term groundwater elevation. For the remainder of the simulation, the water surface was increased based on the computed stage-volume curve for the pit under closure conditions (Figure 9). Under the assumption that the increased water-surface elevations would result in slower times to creek capture, both simulations were executed over two hydrographs (i.e., the 100-year ARI flood simulation includes two 100-year ARI flood hydrographs and the 1,000-year ARI flood simulation includes two 1,000-year ARI flood hydrographs).

Results for the 100-year ARI hydrograph simulation indicate that the creek is captured during the first 18 hours of the first hydrograph, and significant erosion of the creek occurs during the second hydrograph (Figures E-8 and E-9). Results from the 1,000-year simulation indicate that creek capture occurs during the first 12 hours of the first hydrograph, and continued headcutting occurs during the second hydrograph (Figure E-10).

7.0 OPTIONS ASSESSMENT

Given the findings of the erosion assessment that complete creek capture is effectively a certainty if floodplain flows enter the pit, options for preventing creek capture are presented. A closure levee designed for long-term stability could prevent creek capture entirely; however, several options for minimising or controlling pit inflow, stabilising the pit wall, or otherwise mitigating negative impacts of pit inflow at closure are available for further consideration.

7.1 General options for pit closure

Options for pit closure may be simplified into three conditions: 1) no backfill, 2) partial backfill, or 3) complete backfill. Any of these conditions could be undertaken with or without a closure levee or diversion structures in place. Scenarios involving a permanent pit lake may need further hydrogeological or water quality assessments; however, for the purposes of this study, permanent pit lake scenarios are assumed to be viable options for Pit 3. Figure 13 shows some typical options for closure scenarios involving open pits located in floodplain areas.

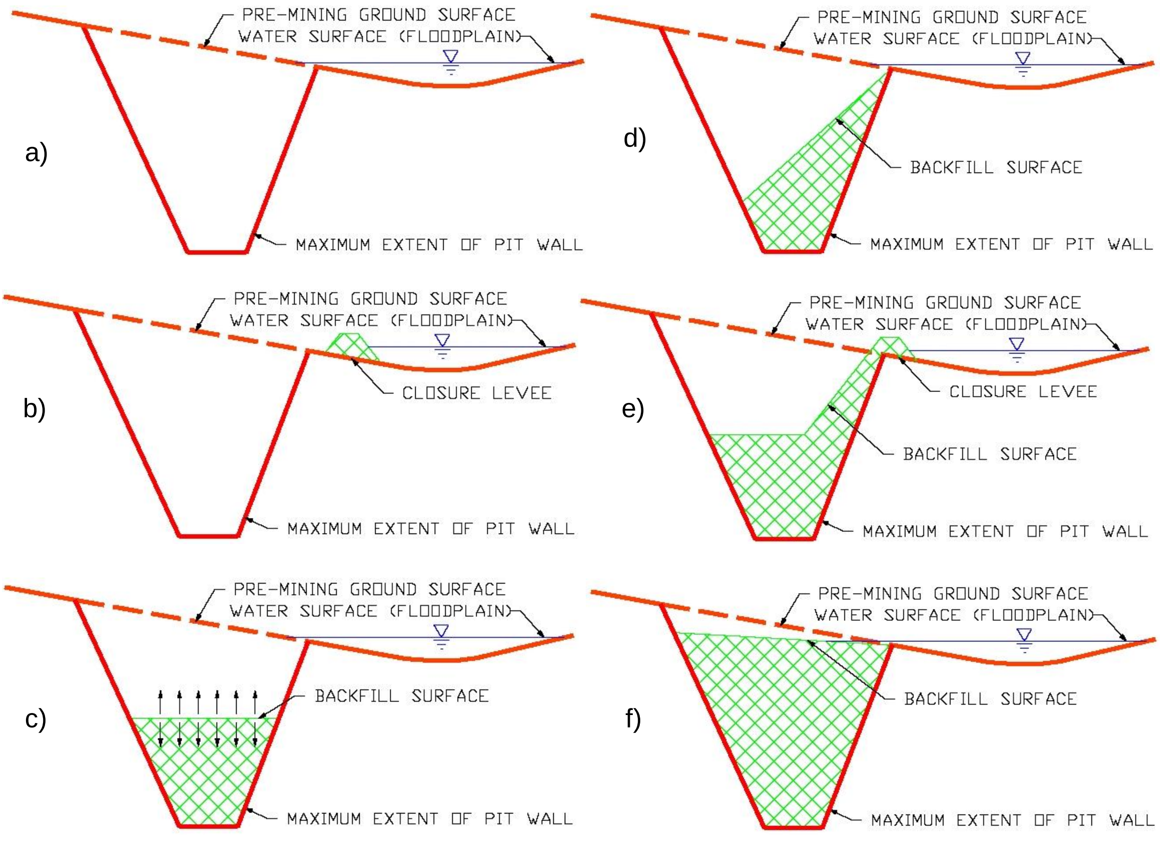


Figure 13. Schematic backfill sections for pits within floodplains

7.1.1 No backfill

Figures 13a and 13b show the no-backfill condition with and without a closure levee. Additional features may include spillways or other structures designed to control inflows and outflows or to stabilise the pit walls in order to prevent a geotechnical collapse from capturing the creek. This is generally the lowest-cost option unless extensive geotechnical work is needed to stabilise the closure pit wall.

7.1.2 Partial backfill

Figure 13c, d, and e show partial backfill scenarios that could range from very minor backfill to options that approach complete backfill. These scenarios could likewise include spillways or pit wall stabilisation options. Figure 13c shows a backfill level that can vary to be below, at, or above the recovered groundwater table. Placing waste material against the pit wall is one method used to slow the rate of collapse. The backfill may be graded to control the rate and mode of post-closure pit wall failure (Figure 13d and 13e). Because these scenarios do not necessarily prevent pit inflow, they can also be combined with a controlled, armoured spillway that could be placed in a single location or with an upstream and downstream channel to route flows into and out of the pit.

7.1.3 Complete backfill

The complete backfill condition restores the ground surface to pre-mining topography or to the minimum free-draining surface (Figure 13f). In general, the minimum free-draining surface achieves an equivalent environmental outcome for a lower cost and is thus preferred. In order to remain free-draining over a post-closure time period without the formation of depressions, backfill would need to account for long-term consolidation. A portion of the pit wall may remain exposed and would be expected to erode over time. Complete backfill eliminates the possibility of creek capture and the associated hydrologic and sediment transport issues, but this scenario is generally very expensive, particularly when waste rock sources are located a considerable distance from the pit.

7.1.4 Closure levees

As shown in Figure 13b and e, a closure levee can be combined with any of the backfill options. Levees can vary in scale and design life (the period after which the levee is likely to fail). In most circumstances, levees must be regarded as temporary structures, given that they will collapse or erode over time. This project assesses hydraulic impacts over a duration of up to 10,000 years, and no engineered structures are likely to have a design life of that magnitude.

The presence of a levee and water pressures behind the pit wall will increase the likelihood of and/or decrease the time to pit wall failure. To slow the collapse, a closure levee can be incorporated such that the back of the levee forms a partial backfill of the pit, as in Figure 13e above. In order to be considered a permanent structure, the levee could be constructed in the form of a waste rock dump that takes the place of a closure bund, becoming a permanent landform feature rather than an engineered hydraulic structure.

7.2 Conceptual Closure Levee Design

Although each of the above scenarios is a viable option for Homestead Creek and Pit 3, the closure levee is presented as the most realistic option for preventing creek capture and accounting for pit wall erosion over time. Further study regarding the pit lake and backfill options may be undertaken in the future to provide reasonable alternatives to the closure levee.

Because the existing roadway is located within the potentially unstable edge zone (Figure 11), the closure levee needs to be located further to the south (closer to Homestead Creek). In order to save material, however, the closure levee can build on a portion of the existing bund, as shown in Figure 15. A portion of the existing roadway would then become a sacrificial section which would be assumed to erode into the pit over time.

As shown in Figure 15 (and in the enlargement in Figure F-5), the closure levee would consist of the following dimensions:

- Height: 3m above existing ground (floodplain)
- Scour Depth 3m toedown (1m below thalweg elevation)
- Amour Rock ¼ tonne, 1m thick (median size 250kg, 550mm dia)
- Top Width 20m
- Side Slopes 3H:1V
- Lining Material Geosynthetic clay liner
- Key trench 1m deep
- Temp Riparian Buffer Zone 15 m
- Perm Riparian Buffer Zone 30 m

Snowden (2012) recommended a 21m minimum buffer zone to the riparian area. The proposed buffer zone allows some temporary impacts to the floodplain in order to construct the scour rock. This zone would be rehabilitated prior to closure.

In general floods are of relatively short duration in Homestead Creek, and sustained water depths against the levee are not likely; however, a geosynthetic clay seepage barrier is recommended to extend the life of the structure. A key trench is recommended to increase the flow path length and prevent piping along the interface between levee fill and insitu materials. Although a design life of 10,000 years cannot be guaranteed for any structure, these measures can extend the life significantly.

Levees and abutments generally require armouring to resist shear stresses and other hydraulic forces where channel velocities exceed 2 m/s, with size and thickness of rock protection determined based on AustRoads Waterway Design, 2013 (p51). The hydraulic results show that average Homestead Creek velocities in the Pit 3 vicinity generally fall under the 2 m/s threshold; however some localised effects may not be accounted for in the model. Also, any stone placed as armour rock protection may break down over time, particularly when time periods up to 10,000 years are considered. Some overdesign may be warranted given the issues associated with rock degradation.

Armour rock is proposed on the water side of the levees. Where levees terminate, armour rock keys into the waste rock dump to prevent erosion as flows spread out. Additional rock placement may be specified to account for loss and wastage. Rock armouring should be keyed in 3 metres below existing ground surface (1 m below the existing creek thalweg). The minimum specific gravity of the rock is recommended as 2.65. Rock strength should meet “very high” or “extremely high” strength as defined in AS1726. Tables 4 and 5 summarise the rock sizing requirements as presented in AustRoads design guides (2013). Strength requirements are shown in Table 6.

Table 4. Permissible velocities for standard rock classes (AustRoads 2013, MRWA 2006)

Velocity (m/s)	Class of rock protection (tonne)	Section thickness, *T* (m)
< 2	None	–
2.0–2.6	Facing	0.50
2.6–2.9	Light	0.75
2.9–3.9	¼	1.00
3.9–4.5	½	1.25
4.5–5.1	1.0	1.60
5.1–5.7	2.0	2.00
5.7–6.4	4.0	2.50
> 6.4	Special	–

Table 5. Rock Protection Requirements (Table 3.12, Austroads 2013)

Rock Class	Rock Size Diameter (m) *	Approximate Rock Mass (kg)	Minimum Percentage of Rock Larger than Rock Size in the Second Column	Typical Use (Examples Only)
Type A	0.20 0.10 0.075		0 50 90	Catchpit Surrounds
Type B1	0.30 0.20 0.10		0 50 90	Culvert Outlets
Type B (Facing)	0.40 0.30 0.15	100 35 2.5	0 50 90	Culvert Outlets
Light	0.55 0.40 0.20	250 100 10	0 50 90	Floodway Batters
Quarter Tonne	0.75 0.55 0.30	500 250 35	0 50 90	Floodway Batters
Half Tonne	0.90 0.70 0.40	1000 450 100	0 50 90	Floodway Batters
One Tonne	1.15 0.90 0.55	2000 1000 250	0 50 90	Floodway Batters
Two Tonne	1.45 1.15 0.75	4000 2000 500	0 50 90	Floodway Batters
Rock Pitching	0.40 x 0.40 x 0.20 0.15 x 0.15 x 0.15		60 100	Landscaped Slopes

Table 6. Rock Strength Requirements (Table A8, AS1726)

STRENGTH OF ROCK MATERIAL

Term	Letter symbol	Point load index (MPa) I_s50	Field guide to strength
Very high	VH	>3 ≤10	Hand specimen breaks with pick after more than one blow; rock rings under hammer
Extremely high	EH	>10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer

7.3 Quantities

Following are the estimated quantities for the primary construction materials needed for the closure levee:

- Select, compacted fill 90,000 m^3
- Geotextile or lining system 18,000 m^2
- ¼ tonne armour rock 15,000 m^3

Figure 15 shows the recommended cross section with typical dimensions.

7.4 Other Closure Drainage Considerations

At present it is unknown whether the railway embankment would remain in place at closure. If it is left in place, the railway spur would be assumed to fail at the sacrificial sections, but without long-term maintenance of the embankment, the actual failure location would not be predictable. If the railway embankment were not confining the discharge, a comingled floodplain would occur between Homestead Creek and Whaleback Creek. Removal of the railway embankment would reduce floodplain elevations and velocities adjacent to Pit 3.

Figure F-6 shows additional areas of concern that may need to be addressed prior to closure. Catchment areas draining into the pits were delineated using 1-m contour interval topography. Concentrated flow locations at the pit edge with contributing drainage areas of 1 ha or more are shown in the figure along with the flow paths. Also shown are basin areas with the potential of building up 2m or more of water depth, which could fail catastrophically if overtopped and eroded. These are based on the existing condition mine site with the ultimate condition pit shell. The draft closure landform designs may eliminate some of these concerns.

Whilst the modelling in this report focuses on OB25, the results cover OB23 as well. The findings may be extrapolated to OB23, which is shown to be more susceptible to pit flooding than OB25 under existing conditions.

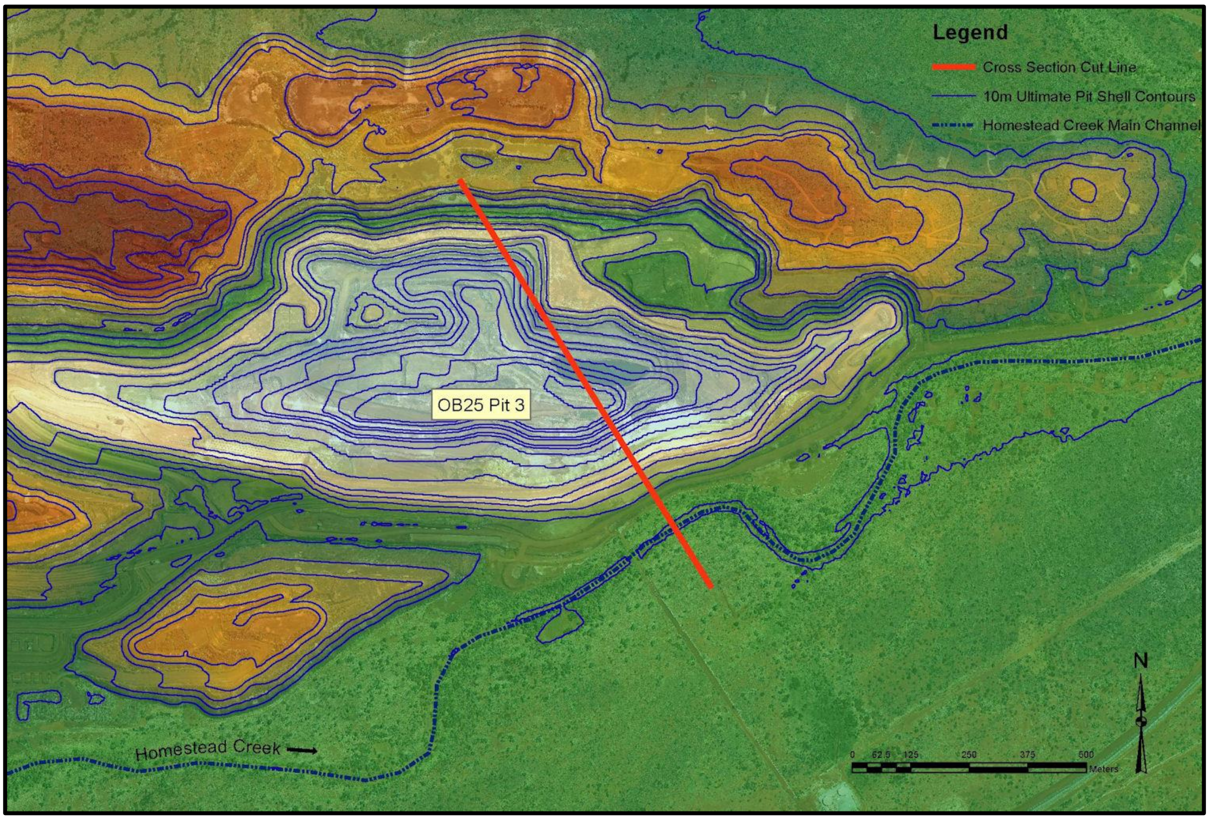


Figure 14. Cross Section Cut Location

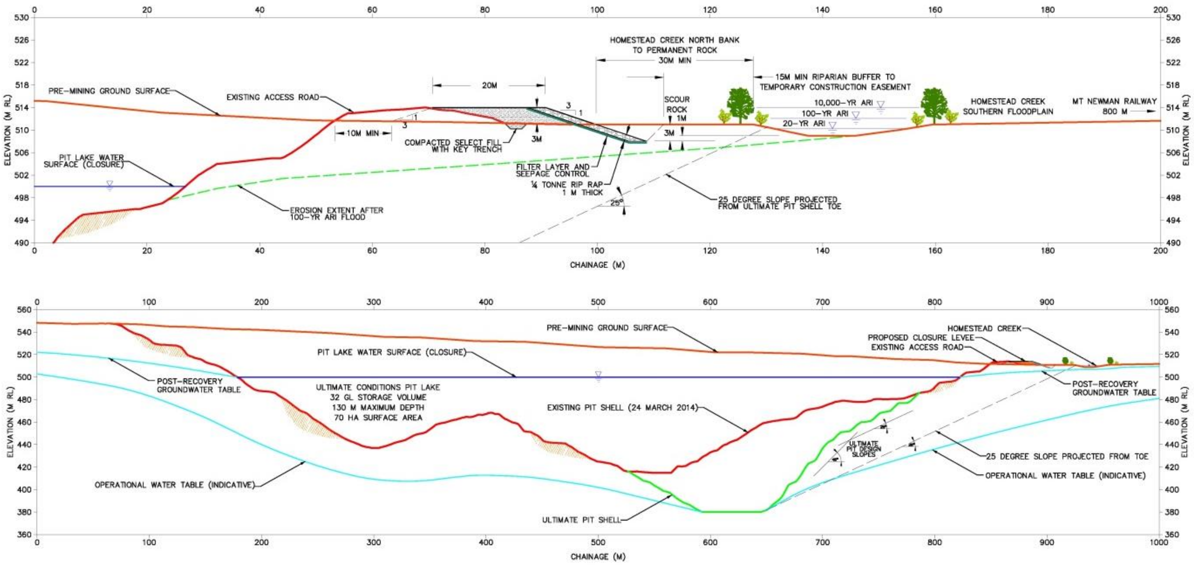


Figure 15. Closure Levee Concept Design (Enlargement included in Appendix F)

8.0 CONCLUSIONS

This report presents hydrologic and hydraulic assessments of Homestead Creek adjacent to OB25. With the existing access road removed the hydraulic modelling shows substantial overflows into Pit 3. Modelling indicates that a headcut would reach the thalweg of Homestead Creek within a single event exceeding the 100-year ARI flood. Following an event of this magnitude, all low flows and all sediment bed load from Homestead Creek would be routed into Pit 3 for an indefinite time period.

A proposed closure levee to prevent creek capture requires a geotechnically, geomorphically, and hydraulically stable design section. The hydraulic characteristics of the Homestead Creek channel flow do not preclude the use of a levee, even if it encroaches into the floodplain. A levee would, however, need to extend into the existing floodplain to remain clear of the zone of instability at the edge of the pit. The option of extending a levee toward the creek is technically feasible but may present unsurmountable environmental concerns. Options for mitigating creek capture concerns through the use of re-establishing drainage over pit backfill are presented qualitatively.

A closure bund with approximately 15,000 m^3 of ¼ tonne armour rock, 18,000 m^2 of a lining system, and 90,000 m^3 of compacted backfill would contain the flow and prevent entry into the pit. The recommended closure bund builds on the existing access road bund, with a sacrificial section of the existing bund assumed to erode into the pit over time.

Unless onsite infiltration tests demonstrate otherwise, a geosynthetic clay liner is recommended to reduce infiltration of surface water through the levee and potential piping of sediments into the pit. The lining system would also address geotechnical stability concerns related to the groundwater table. A risk assessment weighing liner material durability versus construction costs and the impacts of failure is recommended.

The erosion modelling in this study shows that creek capture over a 10,000-year duration is effectively a certainty without a closure levee. The proposed levee system would have a long design life; however, its permanence over a 10,000-year period cannot be guaranteed. Without ongoing maintenance, flow paths through the embankment can form by soil cracking, differential settlement, vegetation growth, uprooting of trees, rodent burrowing, pit wall failure, or other mechanisms. A single failure point could cause a breach during a relatively frequent event.

This analysis did not include a long-term geotechnical stability analysis of the pit wall. A geotechnical assessment suitable for closure timeframes and wetted toe conditions would be required for further assessment of engineering stability. Further studies are warranted to assess impacts, costs, and benefits of options involving partial or complete pit backfill if the relative risks of creek capture are deemed unacceptable.

9.0 REFERENCES

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APPENDIX A: SITE VISIT PHOTOGRAPHS

1. Fortescue River at Great Northern Highway	SAM_2908 – SAM_2917	WP_20140121_013 – WP_20140121_023
2. Homestead Creek at Great Northern Highway	SAM_2918 – SAM_2922	
3. Middle Cathedral Creek at Great Northern Highway	SAM_2939 – SAM_2935	SAM_2923
4. Upper Cathedral Creek at Great Northern Highway	SAM_2924 – SAM_2929	
5. Whaleback Creek at Great Northern Highway	SAM_2936 – SAM_2942	
6. Opthalmia Dam	SAM_2943 – SAM_2947	
7. Homestead Creek at Marble Bar Road	SAM_2948 – SAM_2966	DSC00039 – DSC00040
8. Homestead Creek at Ore Body 25 (See Zoomed Map)	**SAM_2973 – SAM_2974**	**DSC00007 – DSC00032**
9. Homestead Creek at railway crossing	DSC00033 – DSC00038	

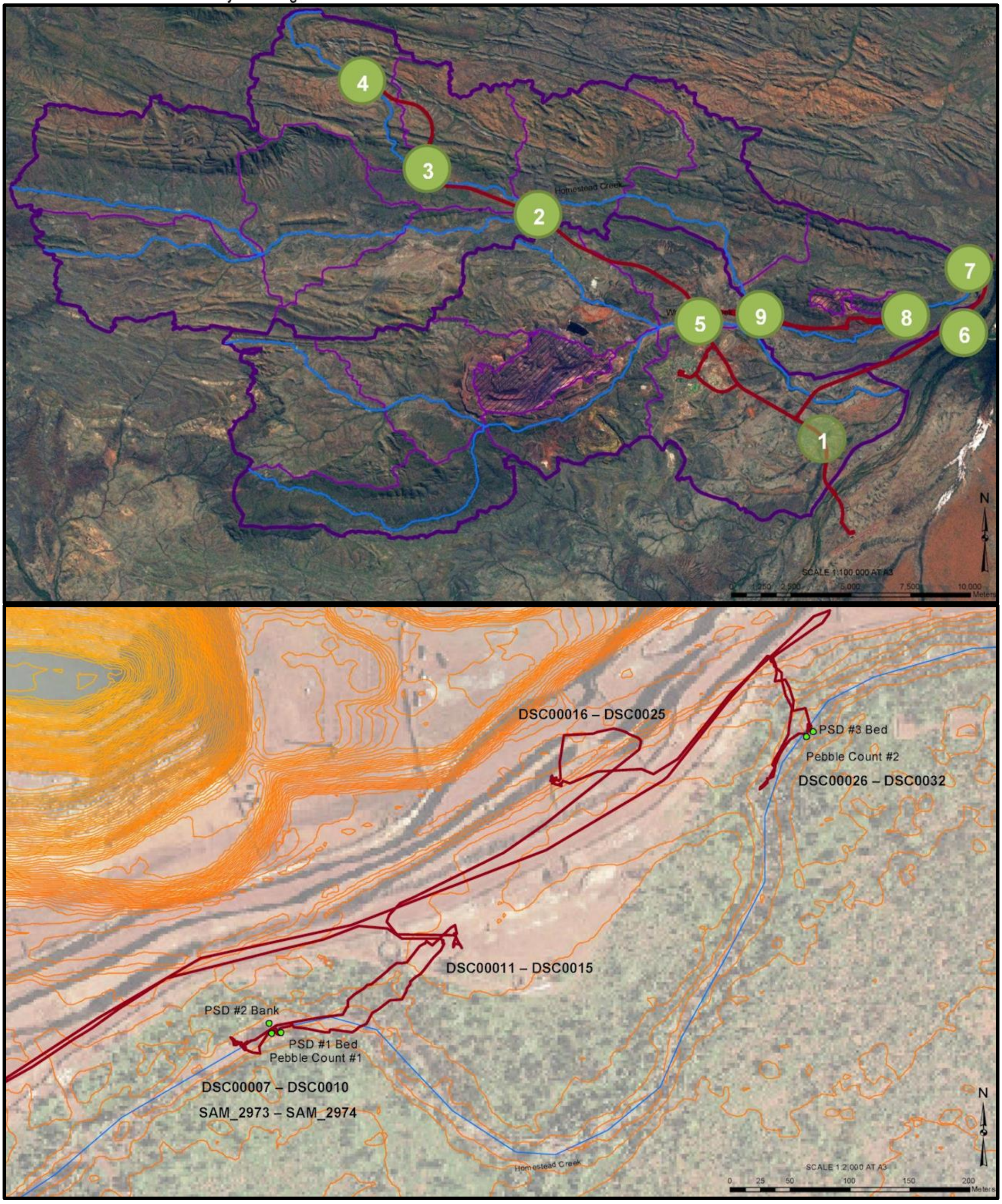


Figure A-1. Digital photo and sediment sample locations for 21 January 2014 site visit

Figure A-2. Upper Homestead Creek catchment (a-d), Homestead Creek at upstream railway spur crossing (e-f)

Figure A-3. Homestead Creek at upstream railway spur crossing (a), Homestead Creek at OB25 (b-f)

Figure A-4. Homestead Creek bank material (a), vegetation (b), bed material (c), cuttings (d), test pit (e), and Pit 3 south bank wall (f)

Figure A-5. Homestead Creek at Marble Bar Road (a-c), Fortescue River during 22 January 2014 flood event (d-e), inflow into Pit 3 (f)

Figure A-6. Homestead Creek at Great Northern Highway (a), Whaleback Creek at Great Northern Highway (b), Ophthalmia Dam (c)

Figure A-7. Homestead Creek at OB25 (a), Homestead Creek at OB23 (b), Homestead Creek at Marble Bar Road (c)

APPENDIX B: HYDROLOGY

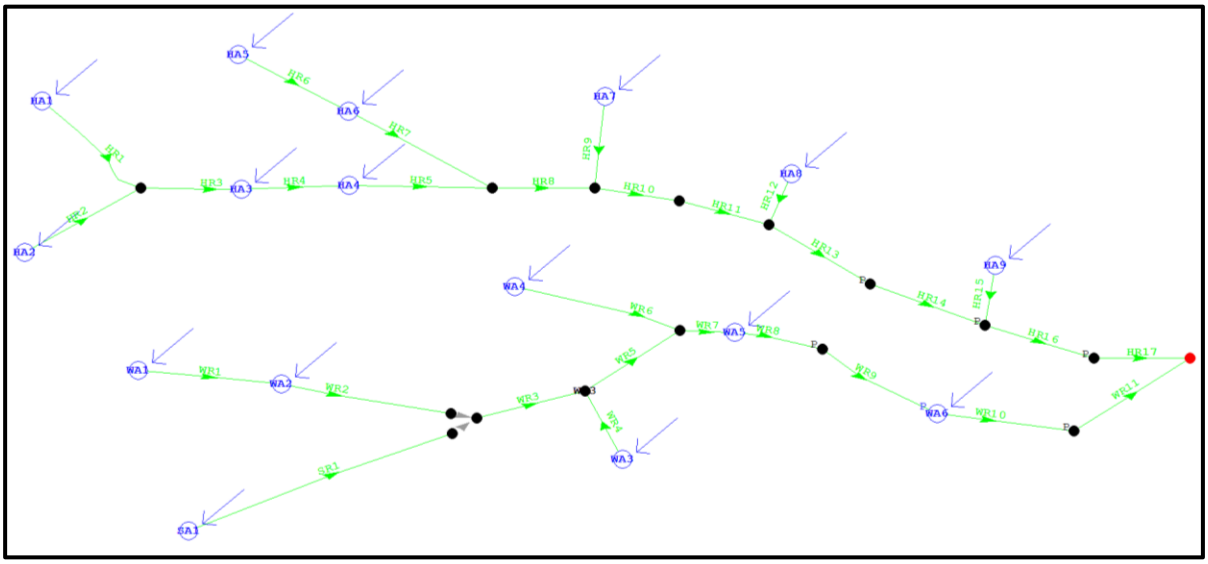


Figure B-1. RORB Schematic

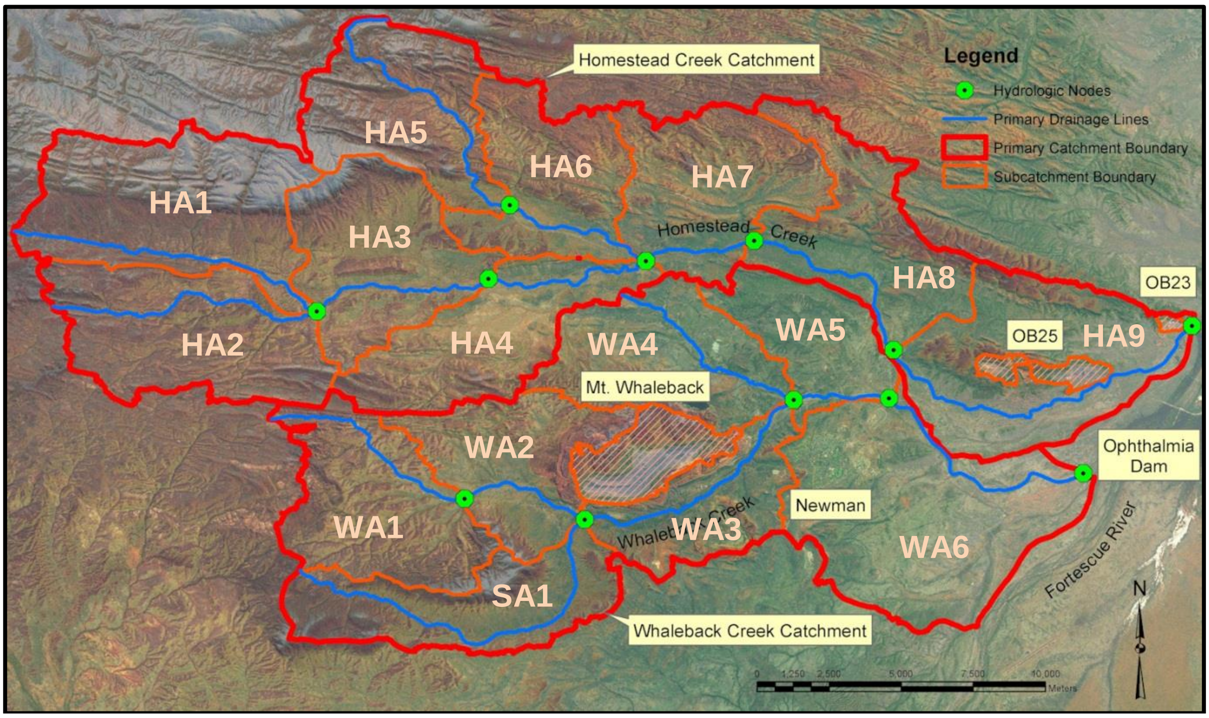


Figure B-2. Homestead Creek and Whaleback Creek subarea designations

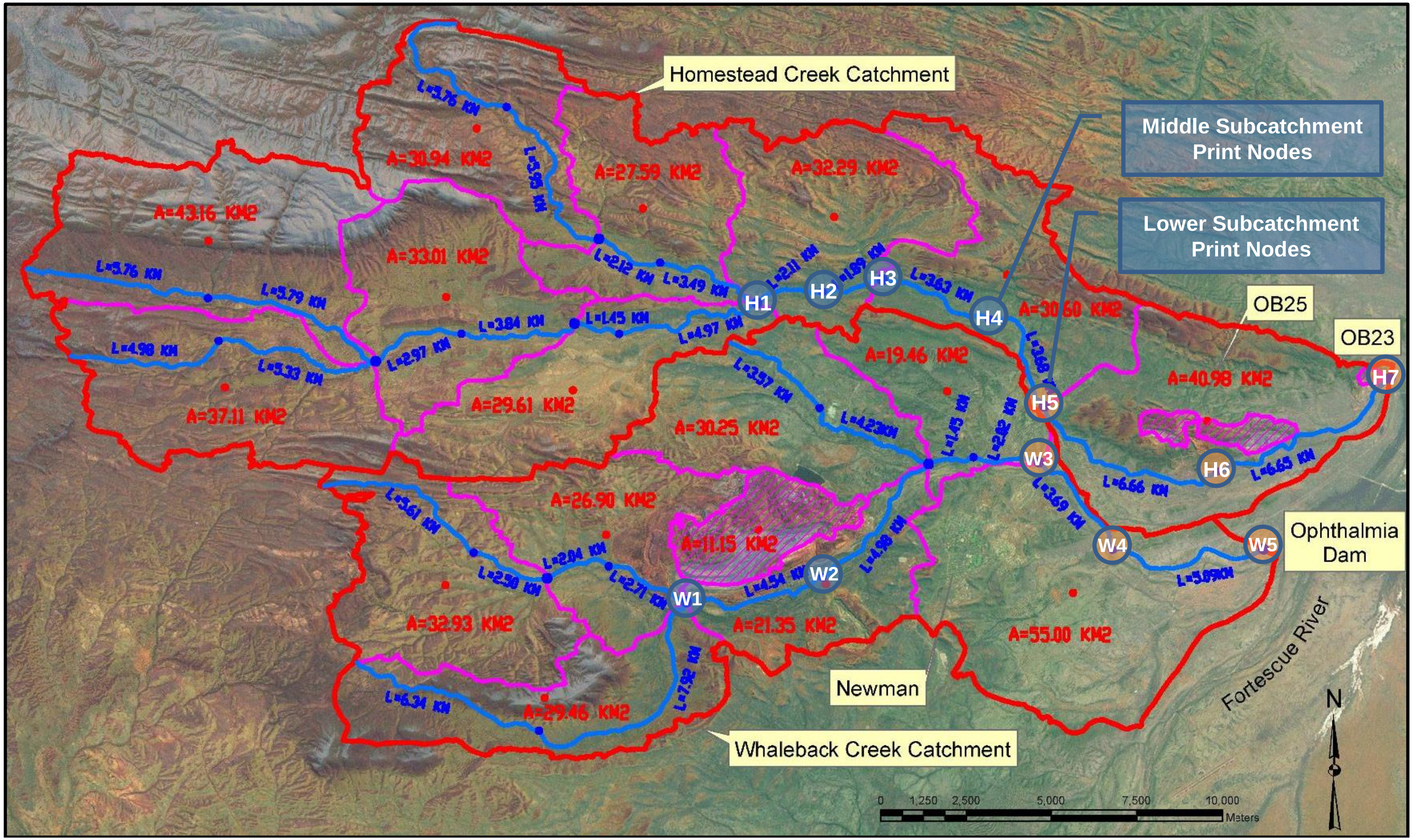


Figure B-3. Centroid locations, Subcatchment Areas, Reach Lengths, and Selected Print Nodes for RORB Model Development

Table B-1. Peak flow estimation using Flavell's Method (2012)

Node	Creek	Area	Length	Eq. Area Slope	Latitude	Longitude	$ASe^{0.5}$	L^2/A	FF	FF	Q_2	Q_5	Q_{10}	$Q_{20\ (a)}$	$Q_{20\ (b)}$	Q_{20}	Q_{50}	Q_{100}
		km^2	km	m/km	DD	DD			Q_{20}/Q_{50}	Q_{20}/Q_{100}								
H5	Homestead Creek	264.3	36.1	3.16	23.30	119.65	469.85	4.93	1.85	2.97	36.8	66.4	109	173	179	179	332	532
H7		305.3	49.4	2.84	23.30	119.65	514.48	7.99	1.86	2.98	23.8	51.4	96.8	185	160	185	345	552
W3	Whaleback Creek	160.4	25.9	2.75	23.30	119.65	265.91	4.17	1.84	2.94	27.8	46.9	73.2	110	120	120	221	353
W5		215.4	34.6	3.40	23.30	119.65	397.09	5.57	1.85	2.96	28.3	53.3	90.4	151	149	151	280	447

Table B-2. CRC-FORGE Areal rainfall quantiles and extrapolated areal rainfall quantiles to PMP

ARI	Duration (hours)							
	24	30	36	48	60	72	96	120
50	145	155	163	177	184	190	195	198
100	169	180	190	206	213	219	225	228
200	194	207	218	237	245	251	258	262
500	231	247	260	282	291	298	306	310
1,000	261	279	294	320	330	338	346	351
2,000	294	314	331	360	371	380	389	394
5,000	343	367	388	422	435	446	458	464
10,000	385	413	437	477	493	507	522	529
PMP	930	1020	1120	1300	1450	1610	1810	1900

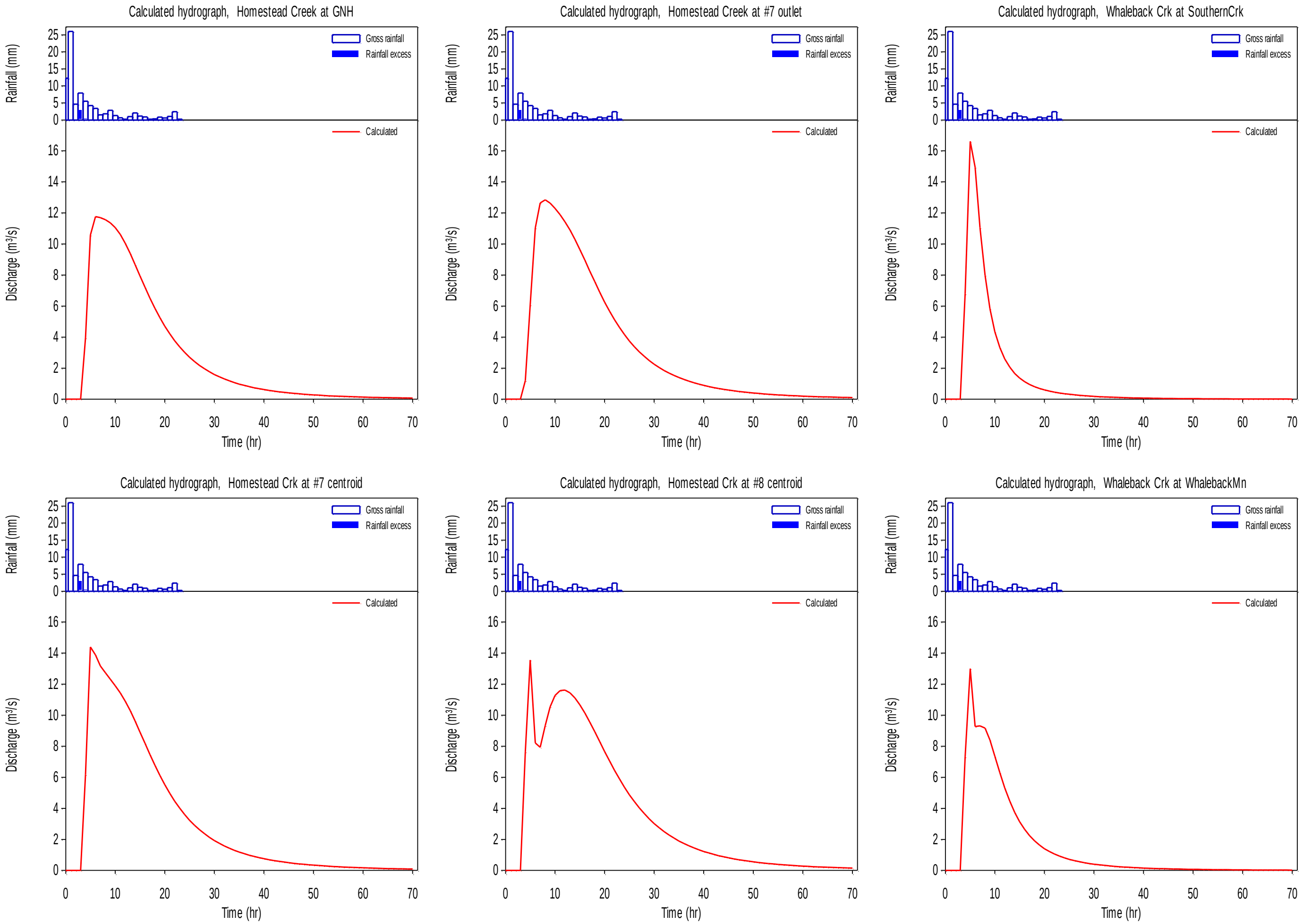


Figure B-4. 5-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Middle Subcatchments

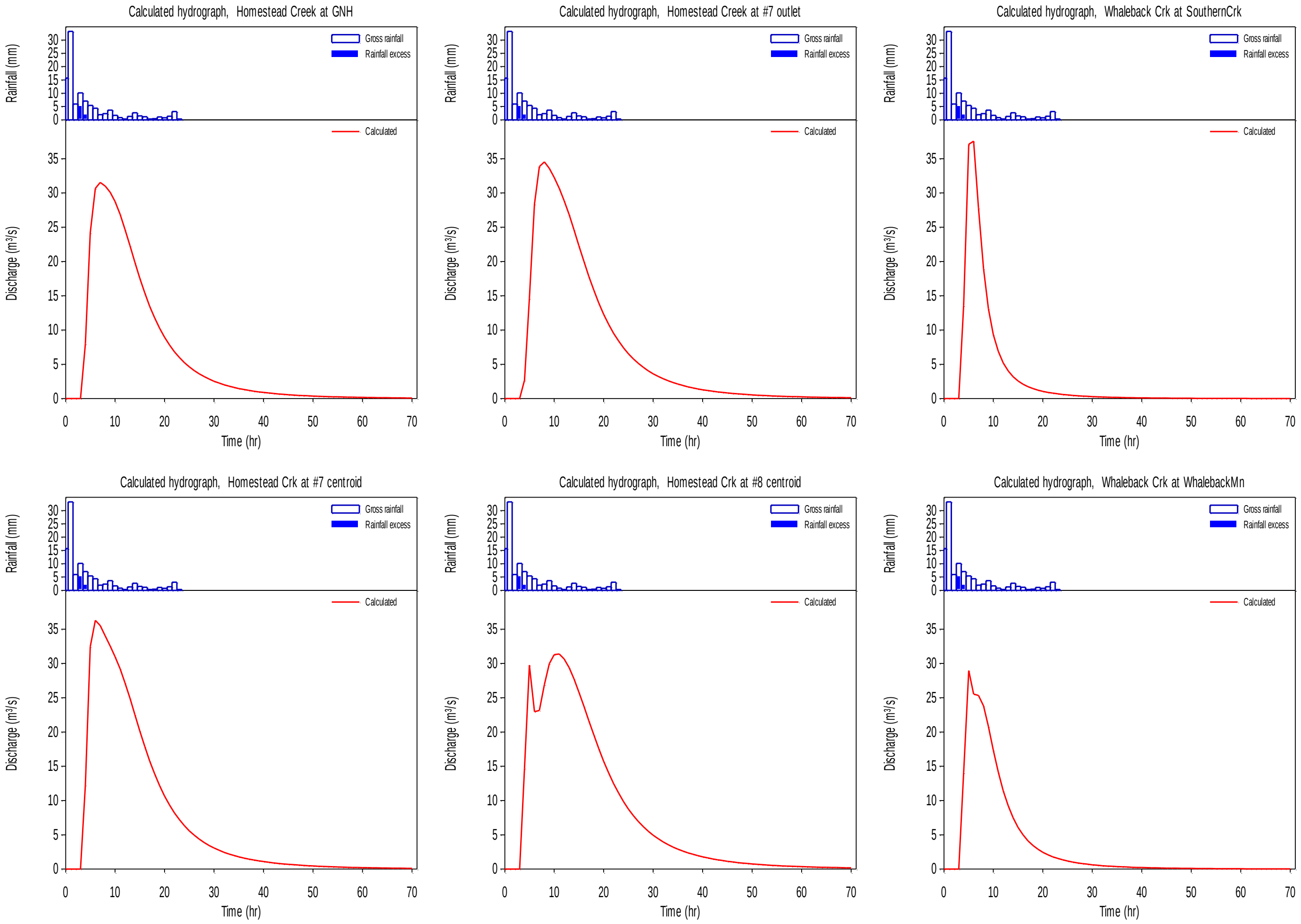


Figure B-5. 10-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Middle Subcatchments

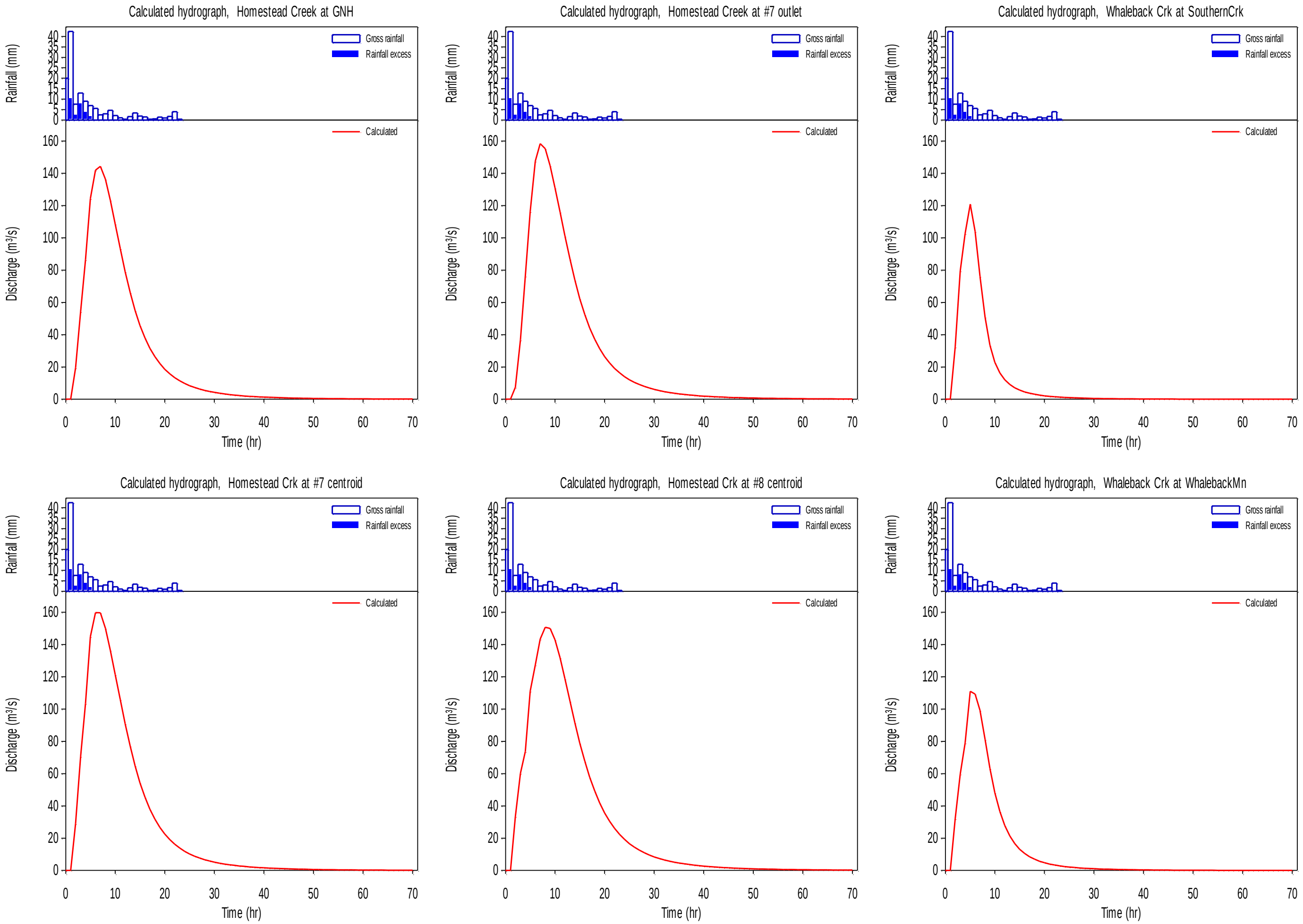


Figure B-6. 20-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Middle Subcatchments

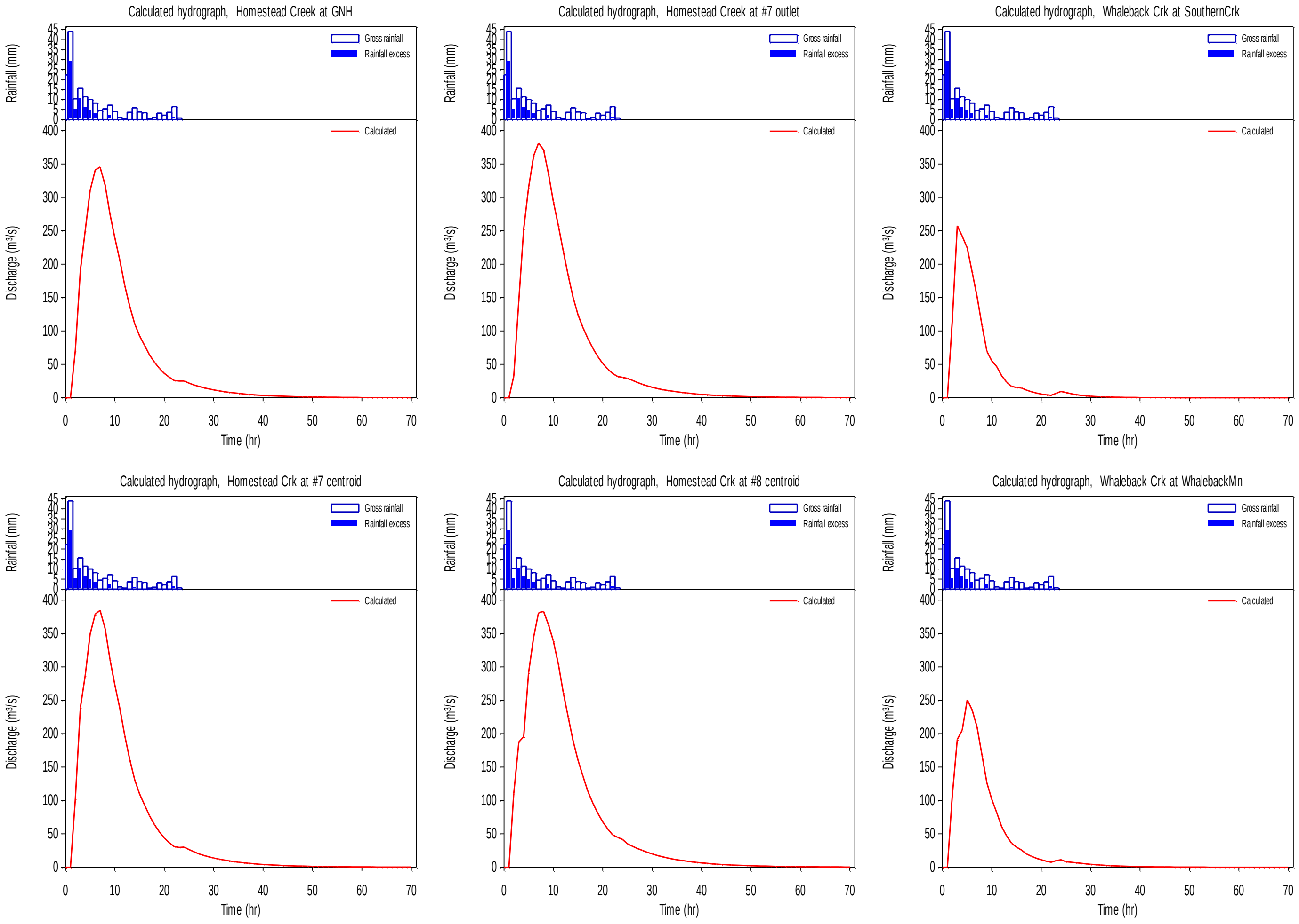


Figure B-7. 50-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Middle Subcatchments

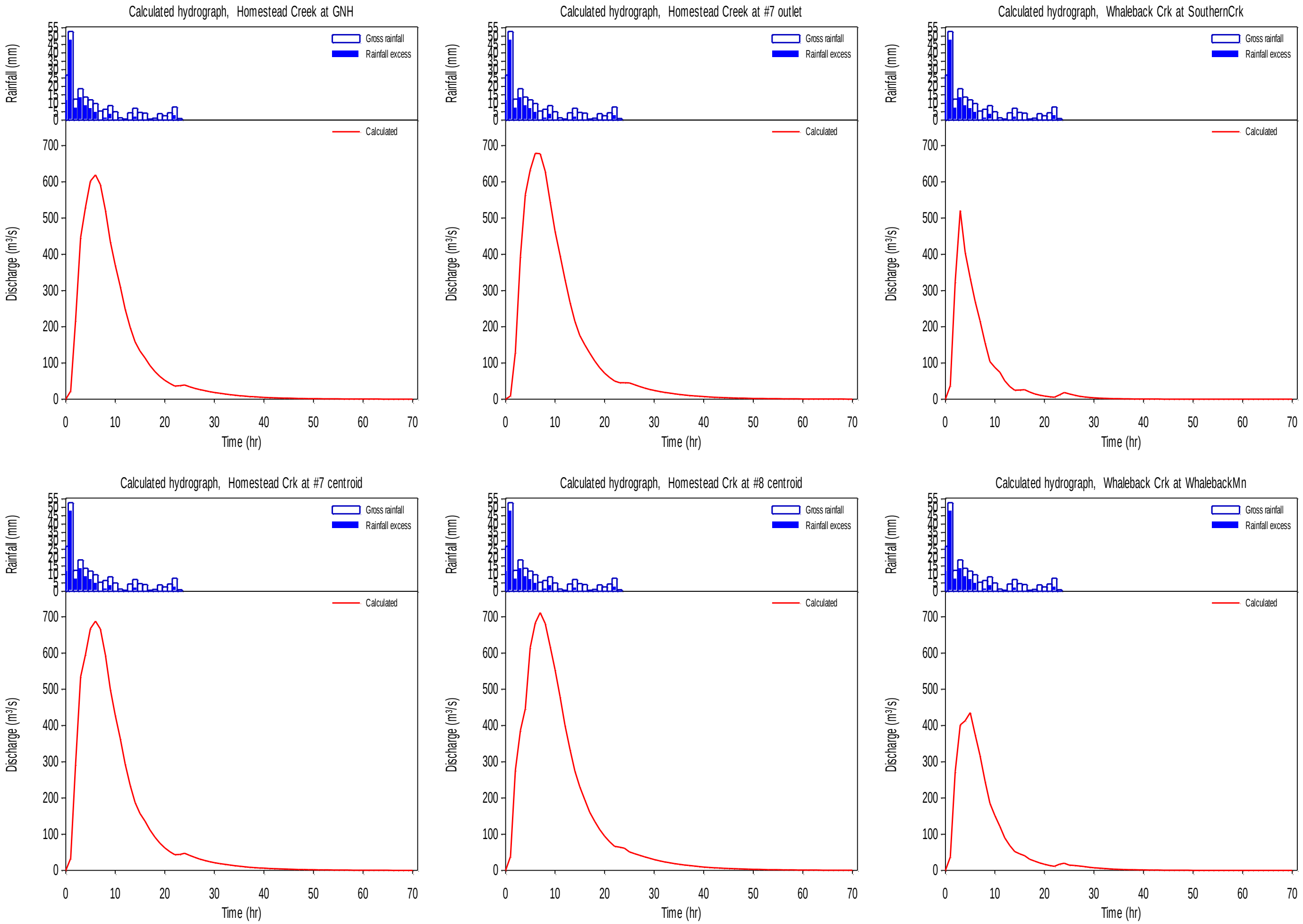


Figure B-8. 100-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Middle Subcatchments

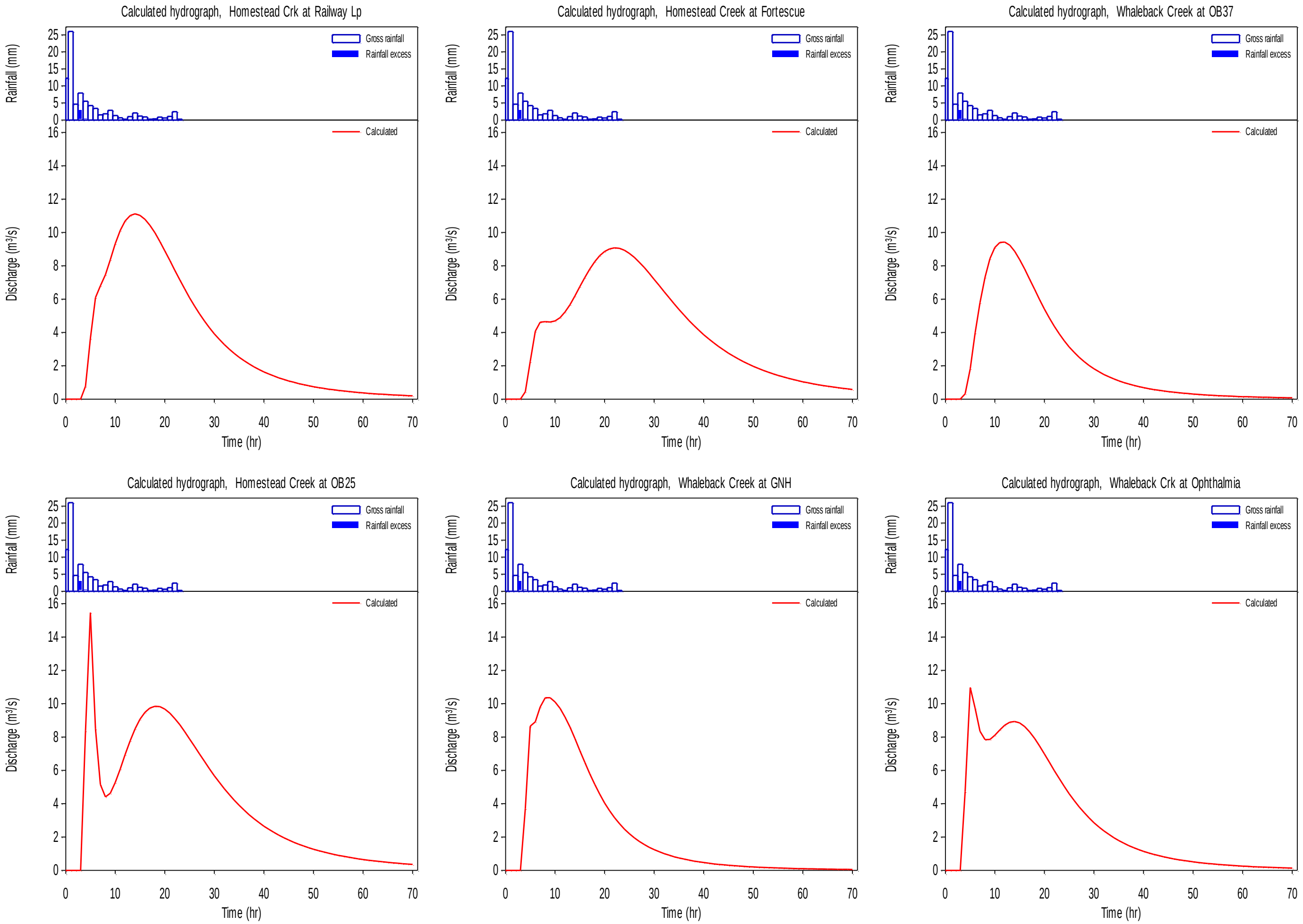


Figure B-9. 5-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Lower Subcatchments

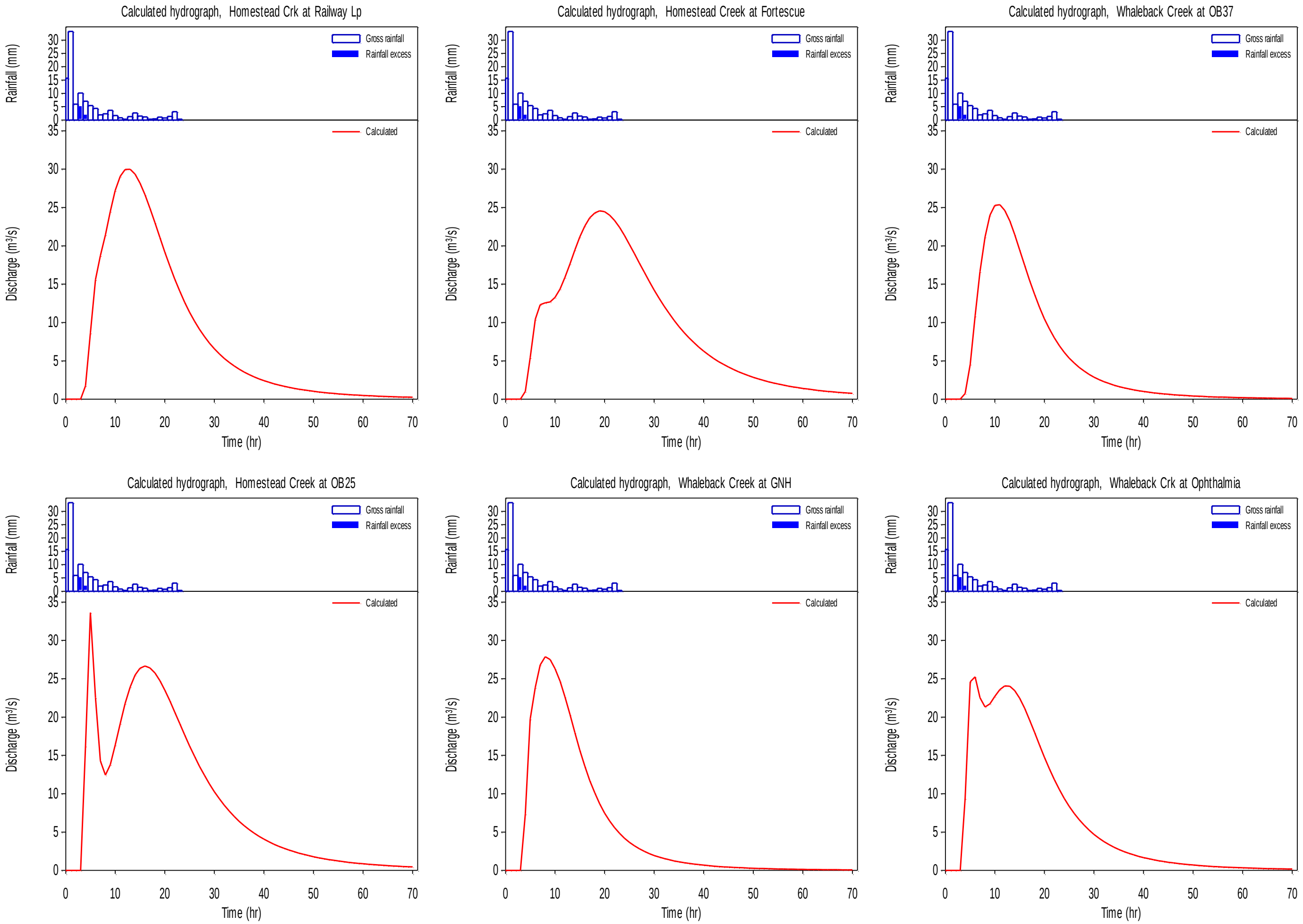


Figure B-10. 10-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Lower Subcatchments

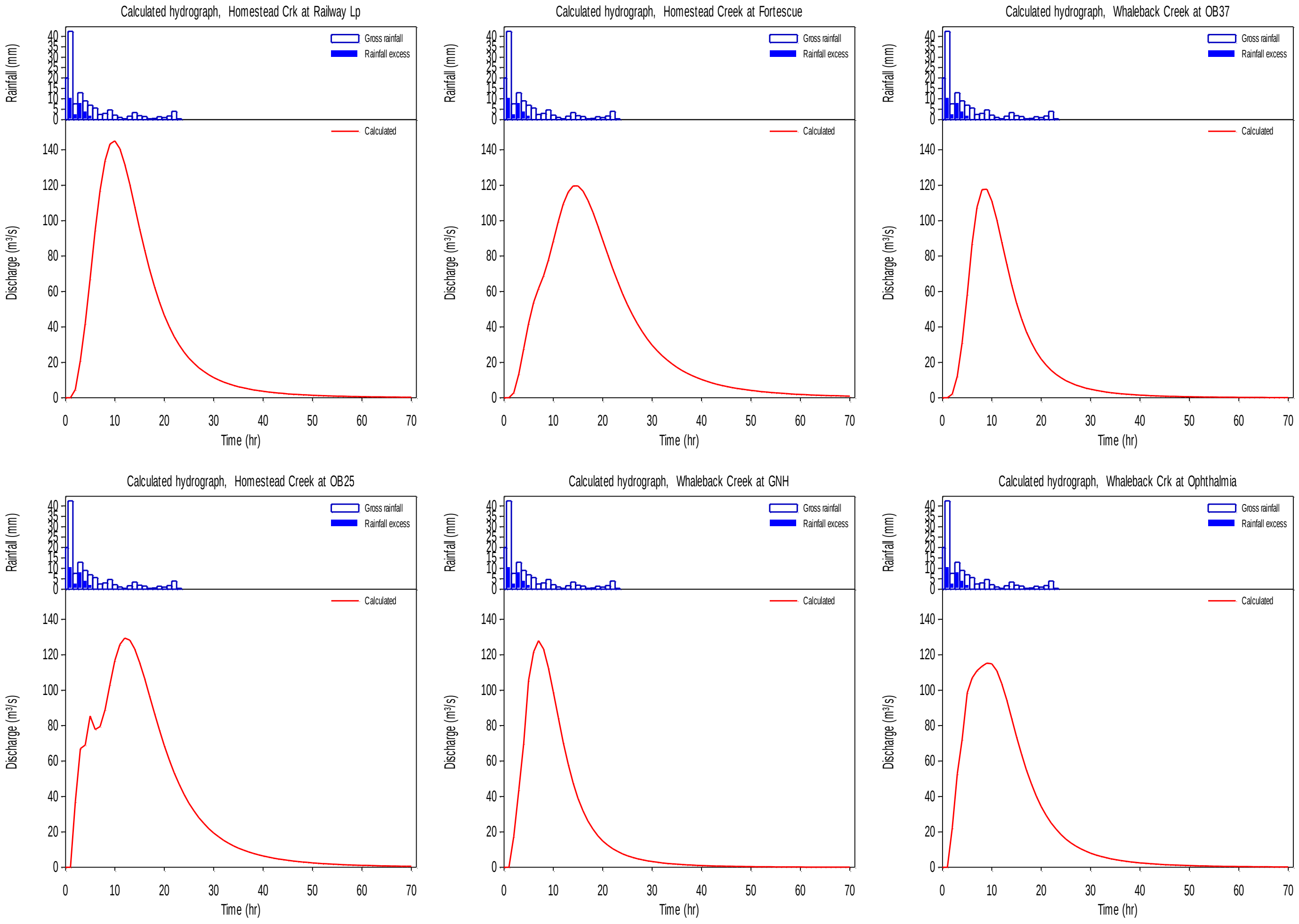


Figure B-11. 20-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Lower Subcatchments

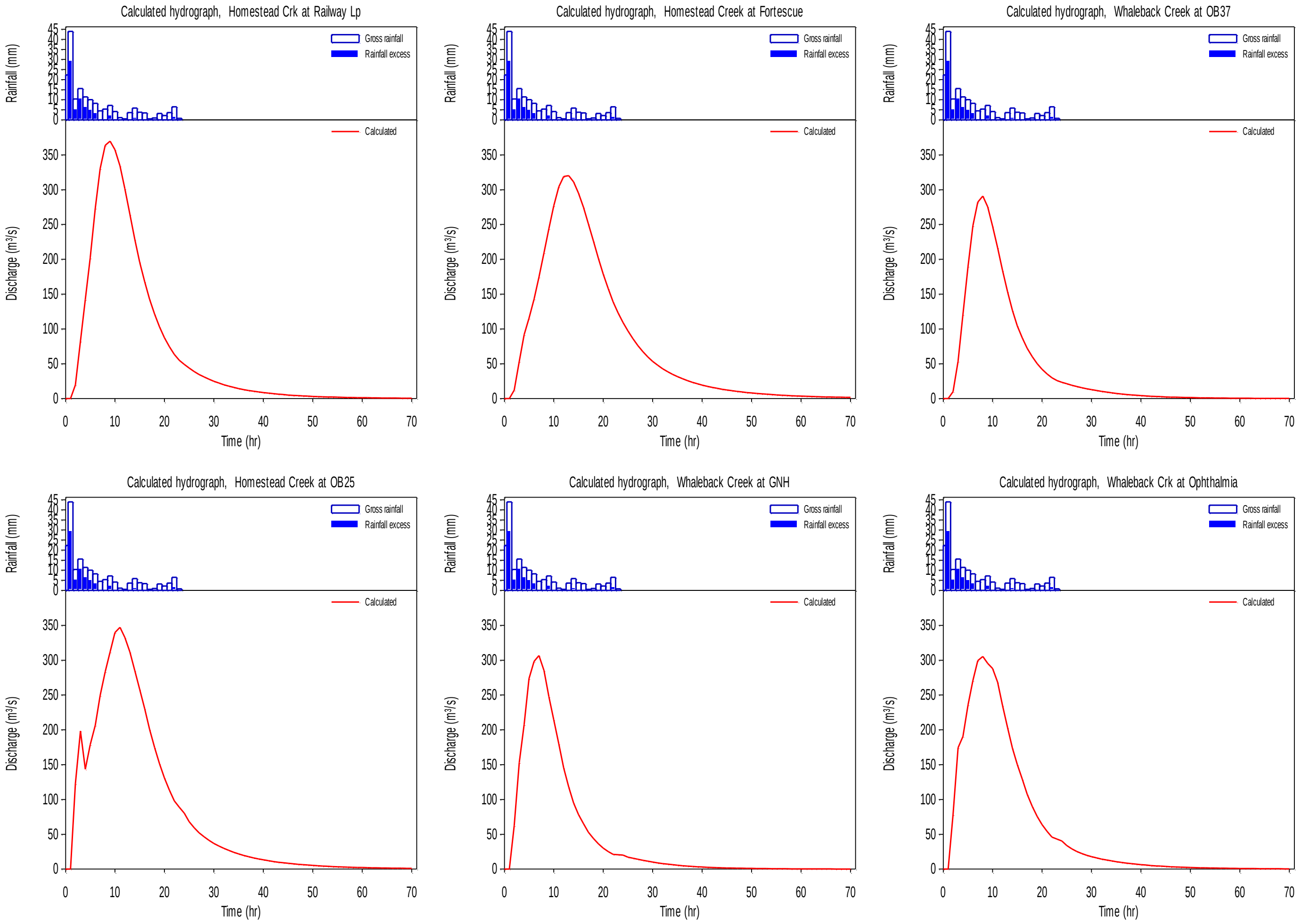


Figure B-12. 50-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Lower Subcatchments

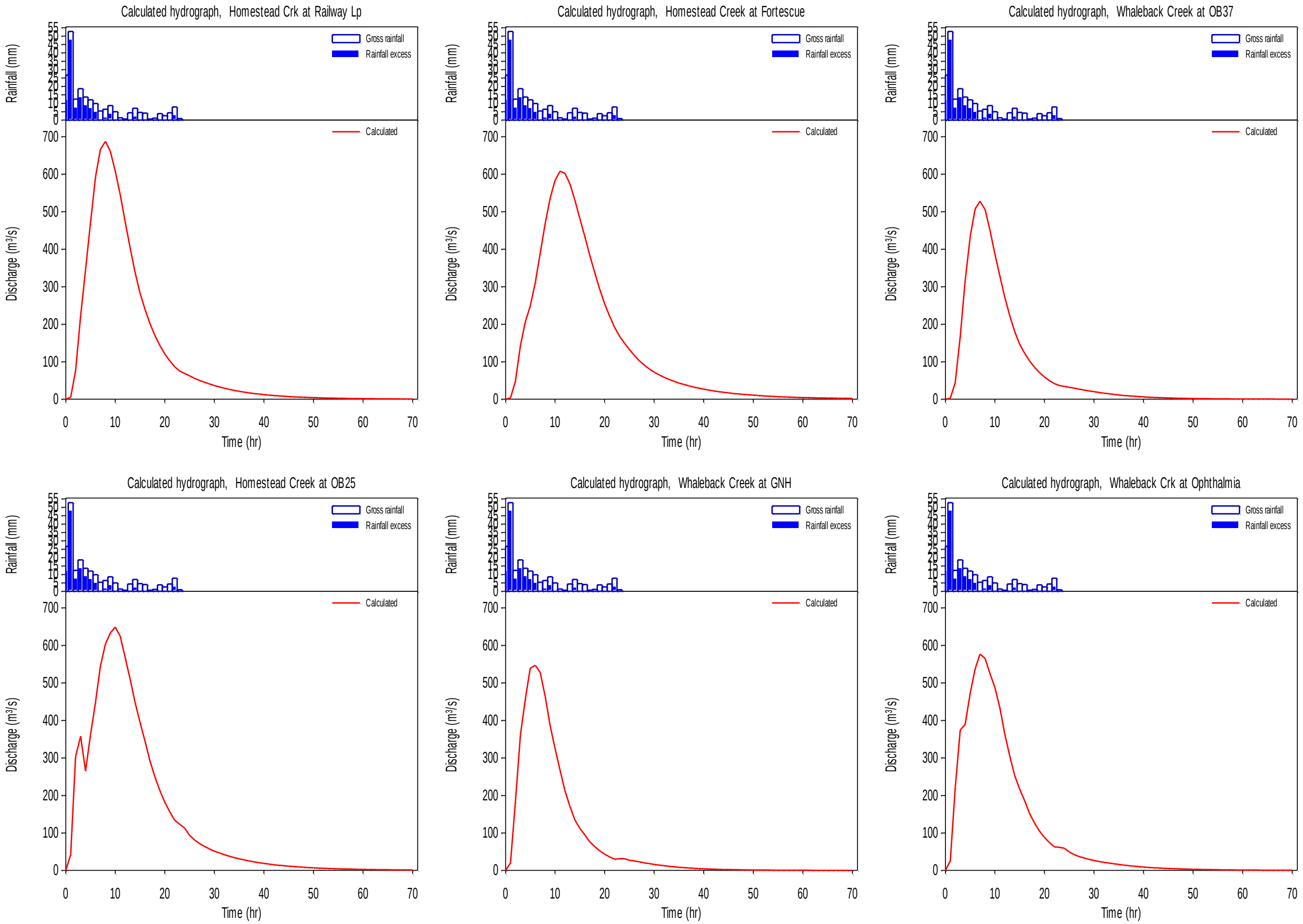


Figure B-13. 100-year ARI Excess Rainfall Hyetographs and Runoff Hydrographs, Lower Subcatchments

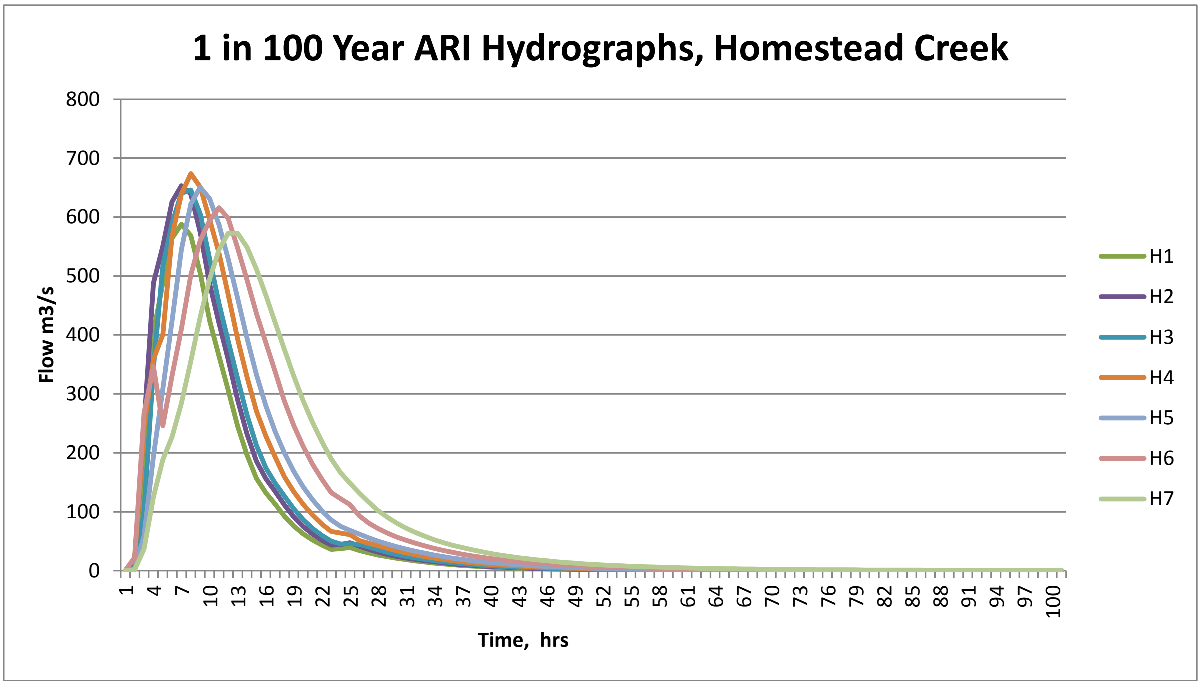


Figure B-14. 100-year ARI Homestead Creek Hydrographs

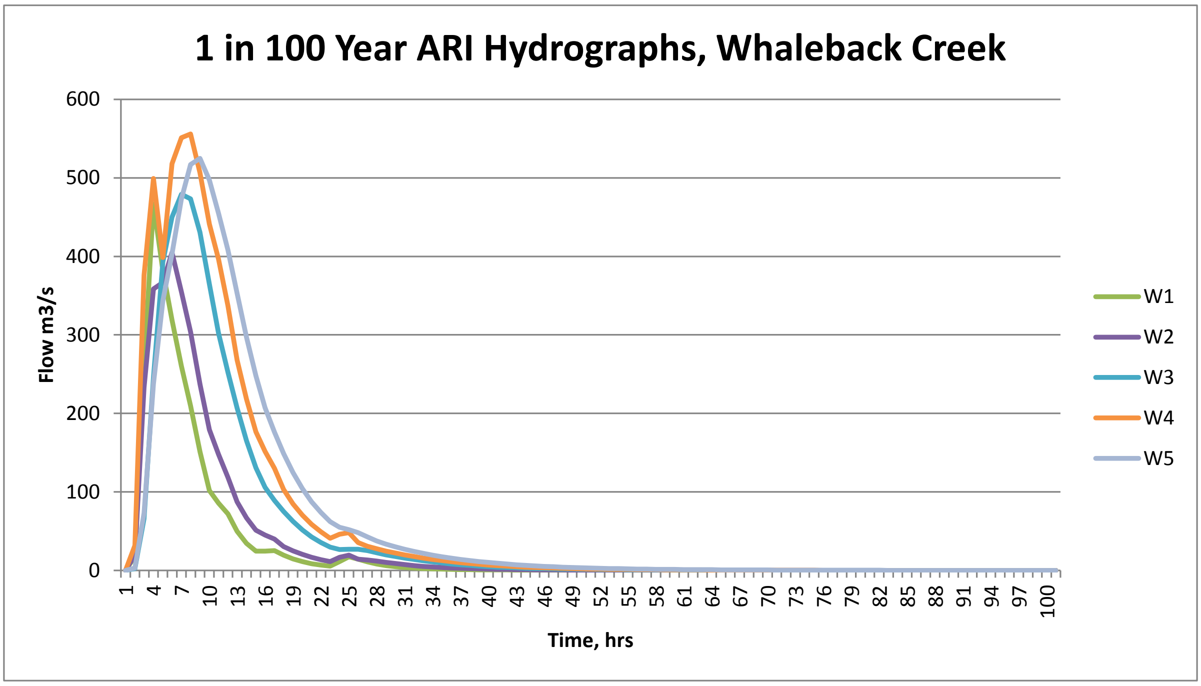


Figure B-15. 100-year ARI Whaleback Creek Hydrographs

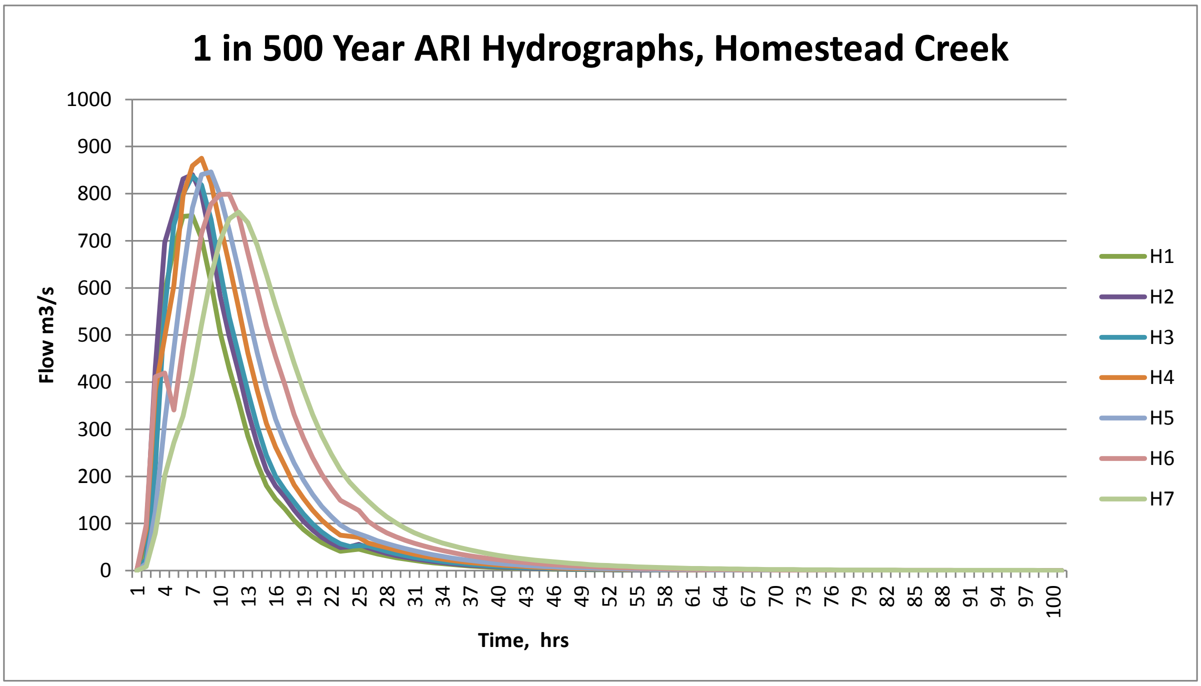


Figure B-16. 500-year ARI Homestead Creek Hydrographs

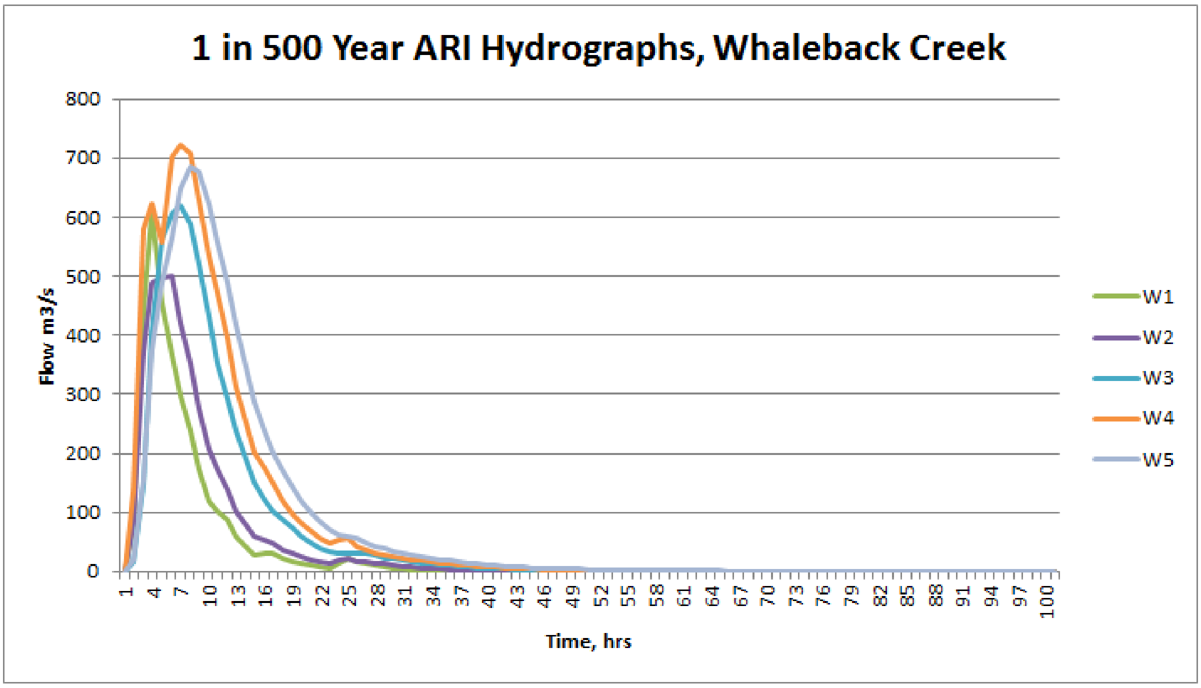


Figure B-17. 500-year ARI Whaleback Creek Hydrographs

TETRA TECH

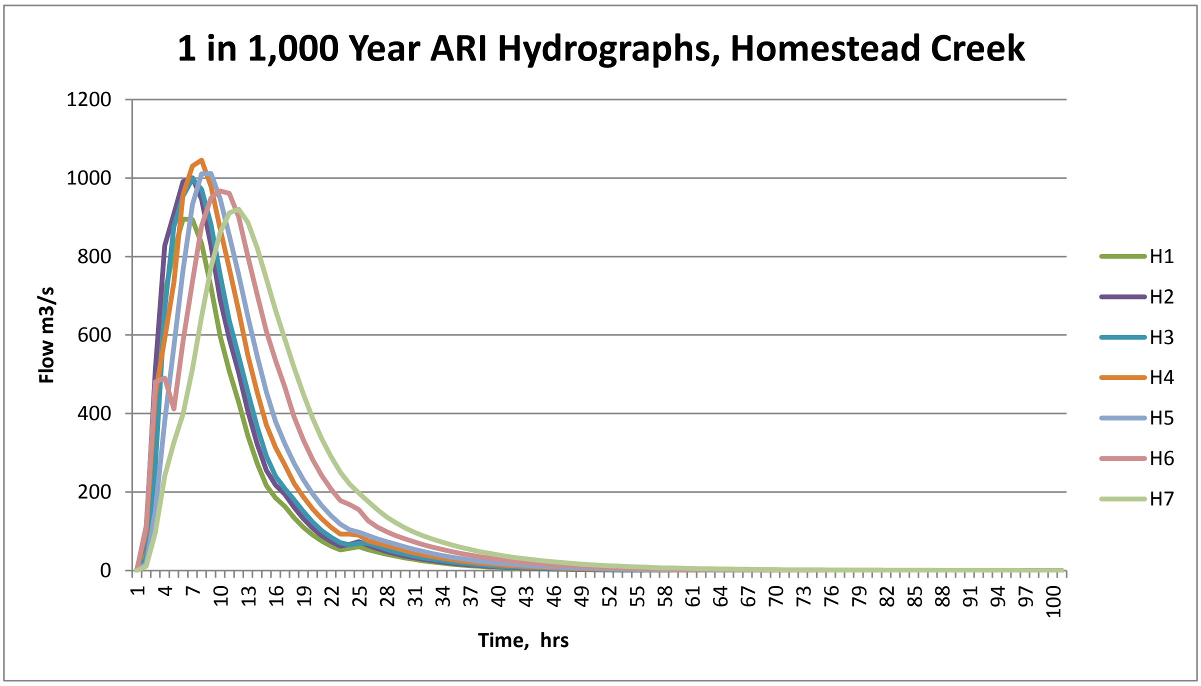


Figure B-18. 1,000-year ARI Homestead Creek Hydrographs

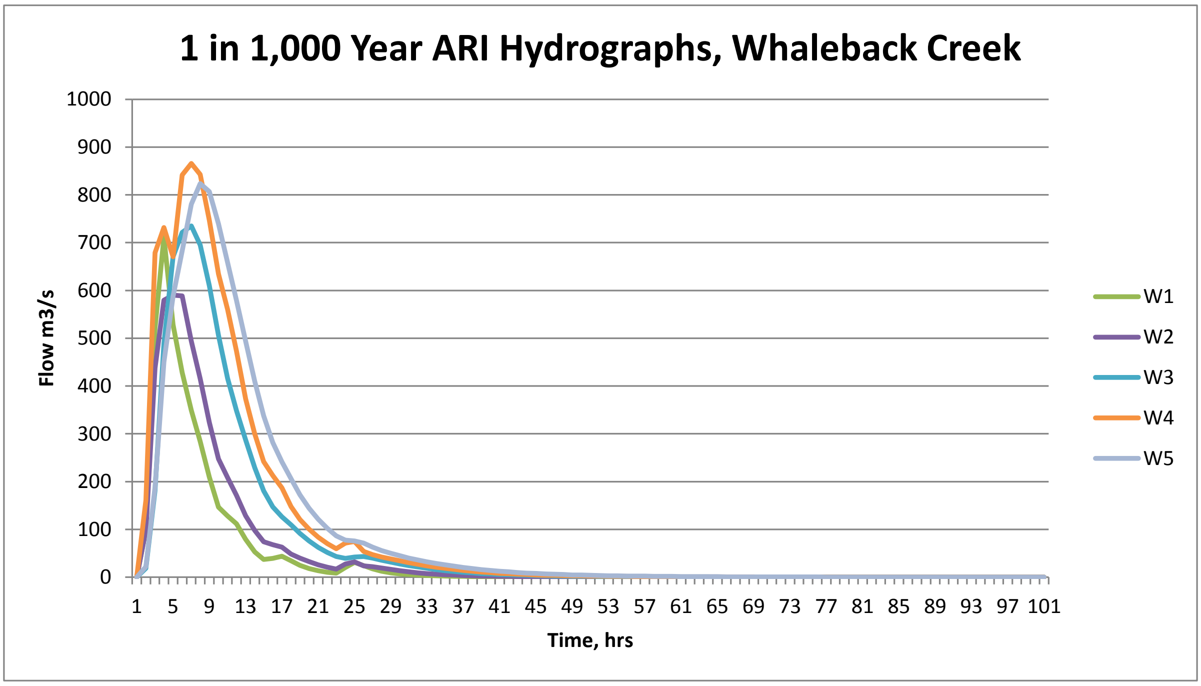


Figure B-19. 1,000-year ARI Whaleback Creek Hydrographs

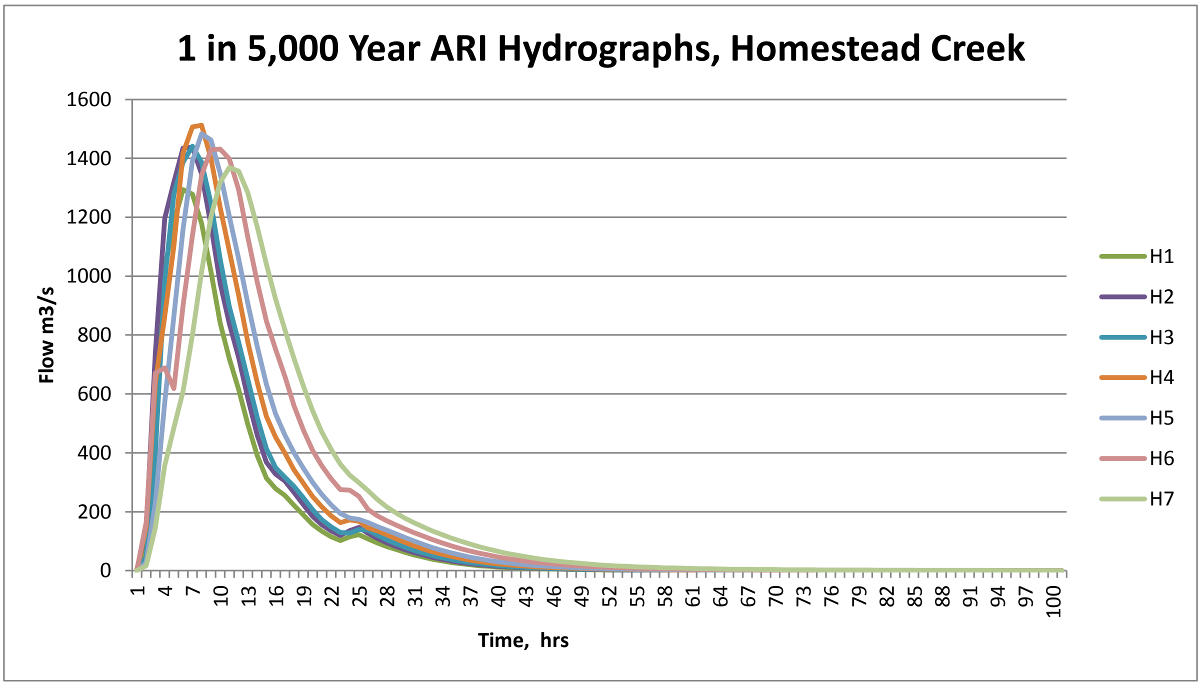


Figure B-20. 5,000-year ARI Homestead Creek Hydrographs

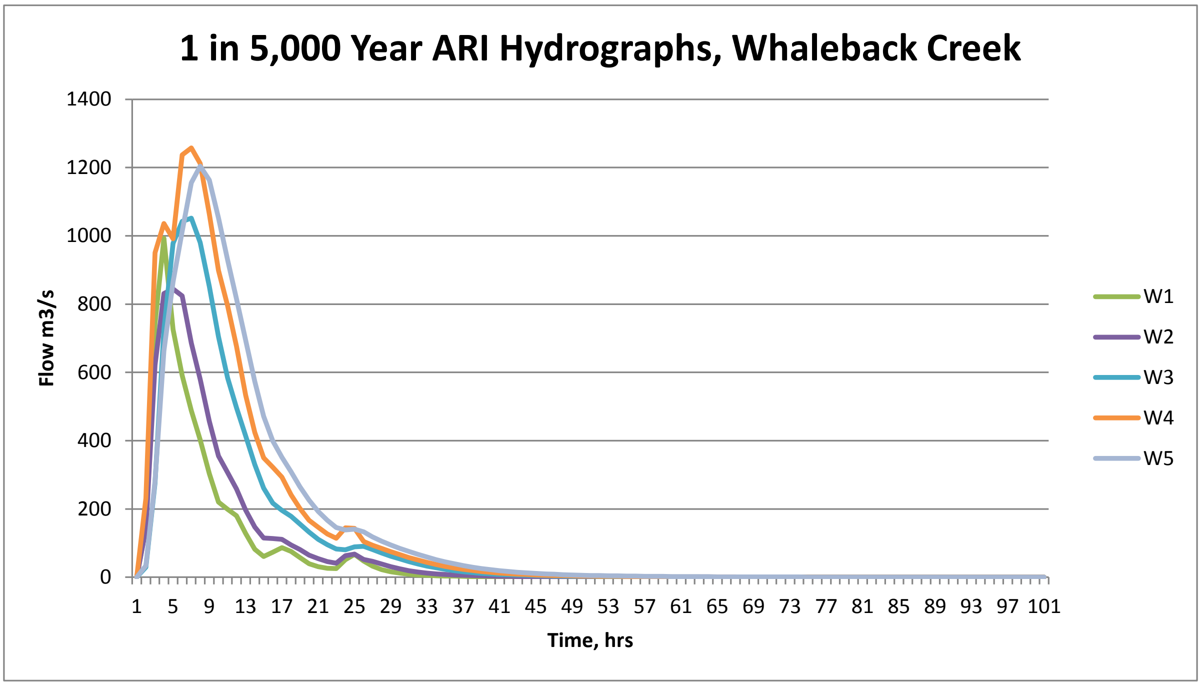


Figure B-21. 5,000-year ARI Whaleback Creek Hydrographs

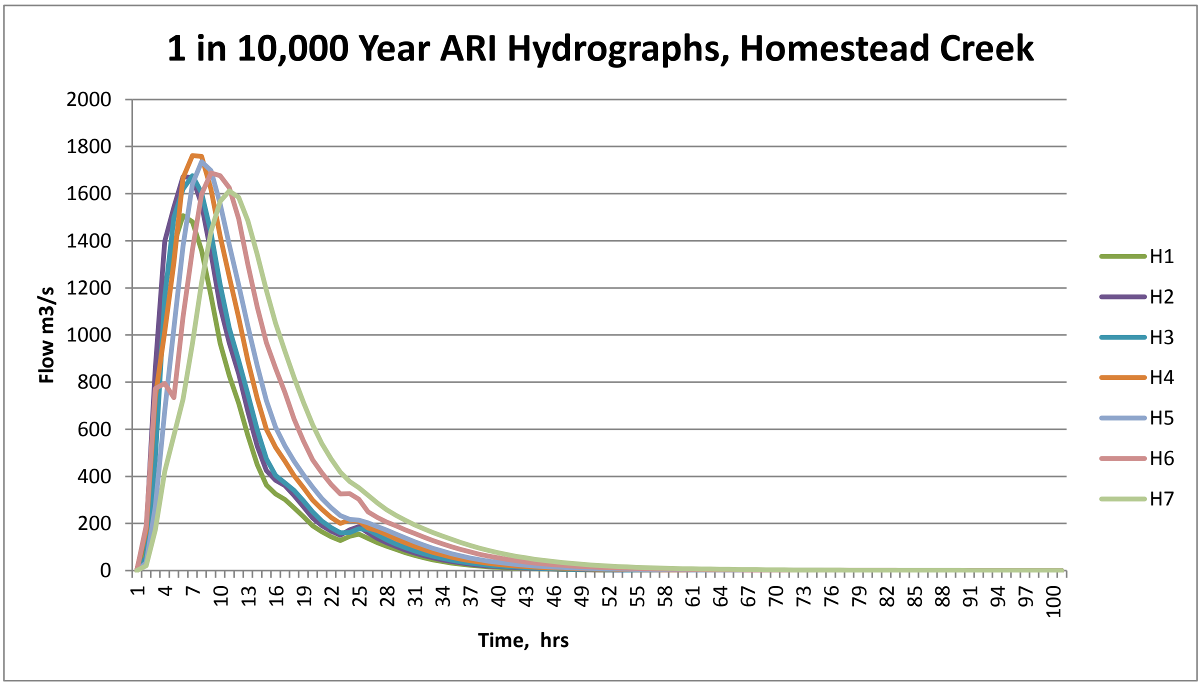


Figure B-22. 10,000-year ARI Homestead Creek Hydrographs

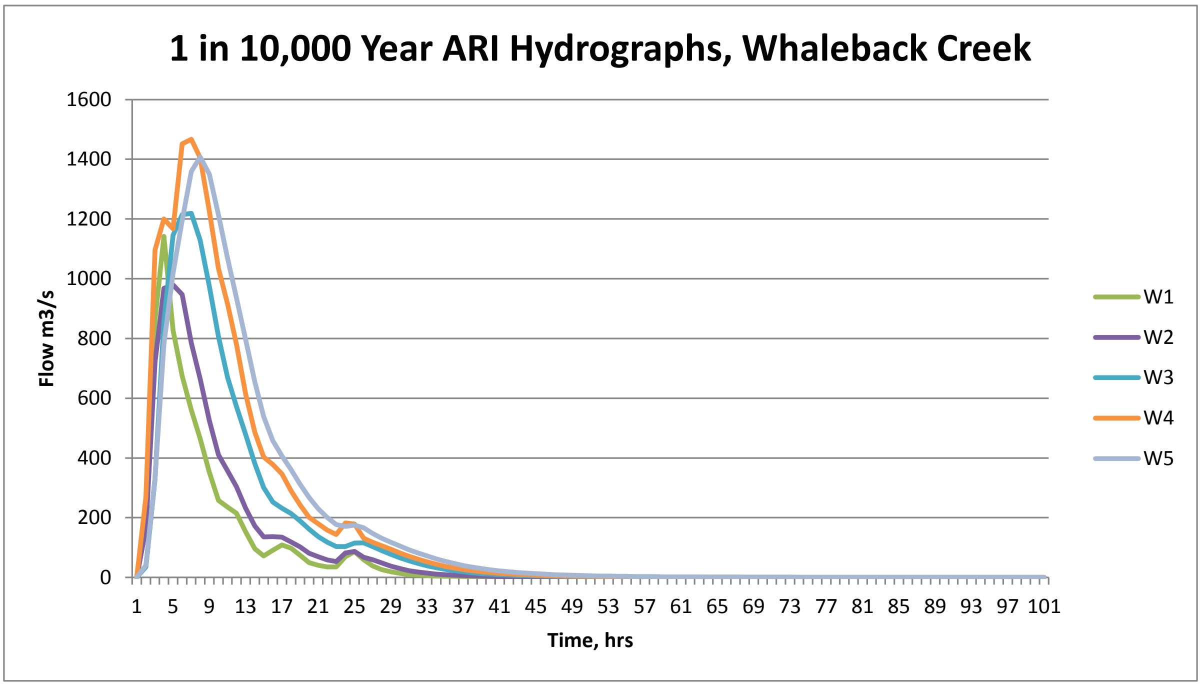
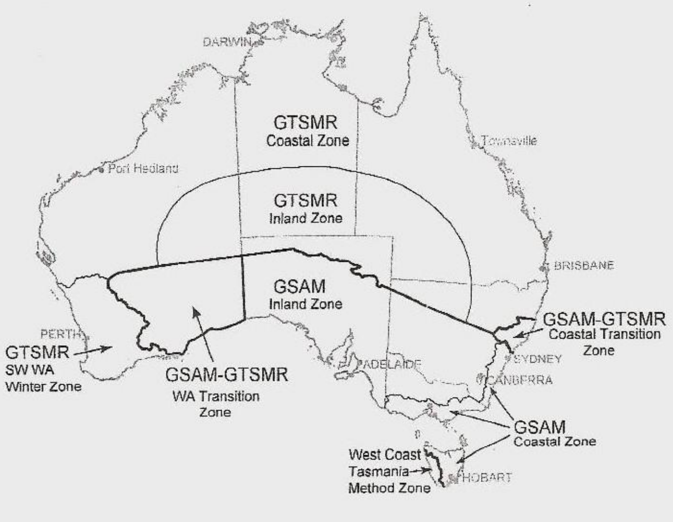


Figure B-21. 10,000-year ARI Whaleback Creek Hydrographs

WORKSHEET 1: PMP Method Selection

Catchment Name.. Catchment Area ......520·6 km²

LONG DURATION PMP



NB This diagram can is also available as a shapefile: [CD-ROM drive]:pmp_zones\zones_all.shp
or it can be printed on A3 paper from [CD-ROM drive]:documents\method_zones.pdf

CIRCLE THE ZONE IN WHICH THE CATCHMENT IS LOCATED:

GTSMR (Coastal)	GTSMR (Inland)
GTSMR (Coastal & SWWA)	Coastal Transition - GTSMR Coastal - GSAM Coastal
GSAM (Coastal)	WA Transition - GTSMR Coastal - GSAM Inland
GSAM (Inland)	WCTas

SHORT DURATION PMP (GSDM)

Is the catchment less than 1000km²? — NO → Short duration PMP estimates can **not** be calculated for the catchment

YES ↓

Is the catchment less than 500km² and south of 30°S? — NO → PMP estimates for up to 6 hours can be calculated using the **GSDM** for this catchment

YES → PMP estimates for up to 6 hours can be calculated using the **GSDM** for this catchment and can include **winter** estimates

PMP METHOD SUMMARY

Fill in the table below with the PMP method/s applicable to the catchment, referring to Table 1.1 for any additional information needed. NB: for the Transition zones, write separate entries for GTSMR and GSAM.

METHOD	ZONE	SEASON	DURATIONS
eg GTSMR	*Coastal*	*Annual*	*24-120 hours*
eg GSDM	*6 hours*	*Annual*	*1-6 hours*
GTSMR (Coastal)	Coastal	Annual	24-120 hours

WHAT NEXT?

GTSMR:	Calculate the PMP estimates for the catchment following the procedures in this guidebook
GSDM:	Calculate the PMP estimates for up to 6 hours following the GSDM (Bureau of Meteorology, 2003) guidebook (http://www.bom.gov.au/hydro/has/gsdm_document.shtml)
GSAM:	Contact the Hydrometeorological Advisory Service, Bureau of Meteorology
WCTas:	Contact the Hydrometeorological Advisory Service, Bureau of Meteorology

Figure B-22. PMP Worksheet

WORKSHEET 2: Generalised Tropical Storm Method Revised (GTSMR)

LOCATION INFORMATION

Catchment Name ...Homestead Creek........................ State ...WA.........

GTSMR zone(s) ...Coastal...

CATCHMENT FACTORS

Topographical Adjustment Factor	**TAF** = ...1.0............	(1.0 – 2.0)
Decay Amplitude Factor	**DAF** = ...0.895........	(0.7 – 1.0)
Annual Moisture Adjustment Factor	MAF_a = $EPW_{catchment}$/120.00	
Extreme Precipitable Water ($EPW_{catchment}$) = ...98.48	MAF_a = ...0.82.......	(0.4 – 1.1)
Winter Moisture Adjustment Factor (where applicable)	MAF_w = $EPW_{catchment\_winter}$/82.30	
Winter EPW ($EPW_{catchment\_winter}$) = ...	MAF_w = ...	(0.4 – 1.1)

PMP VALUES (mm) - Annual

Duration (hours)	Initial Depth (D_a)	PMP Estimate $=D_a x TAF x DAF x MAF_a$	Preliminary PMP Estimate (nearest 10mm)	Final PMP Estimate (from envelope)
1	Where applicable, calculate GSDM (Bureau of Meteorology, 2003) depths			
2				
3				
4				
5				
6				
12	(no preliminary estimates available)			
24	1274	935	930	930 3'-1020
36	1530	1123	1120	1120
48	1769	1298	1300	1300 60-1450
72	2190	1607	1610	1610
96	2460	1805	1840	1840
120	2590	1901	1900	1900

PMP VALUES (mm) – Winter (where applicable)

Duration (hours)	Initial Depth (D_w)	PMP Estimate $=D_w x TAF x DAF x MAF_w$	Preliminary PMP Estimate (nearest 10mm)	Final PMP Estimate (from envelope)
1	Where applicable, calculate GSDM (Bureau of Meteorology, 2003) depths			
2				
3				
4				
5				
6				
12	(no preliminary estimates available)			
24				
36				
48				
72				
96				

Figure B-23. GTSMR Worksheet

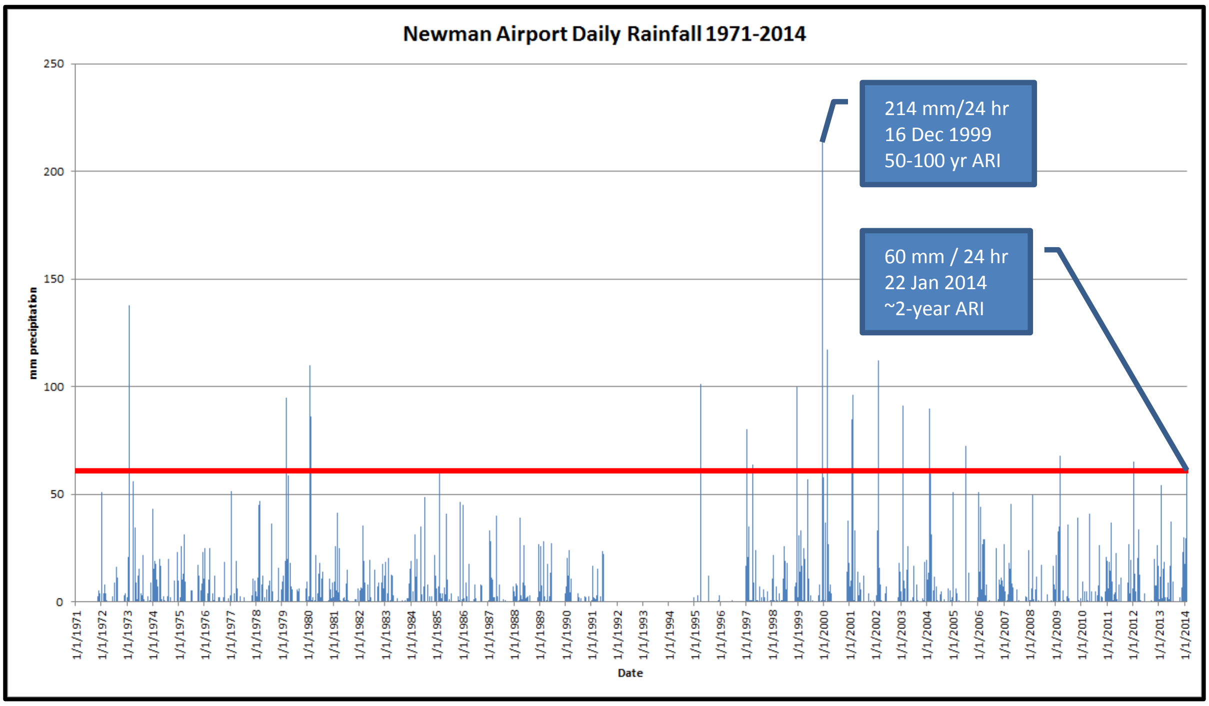


Figure B-24. Newman Airport Daily Rainfall Data 1971-2014

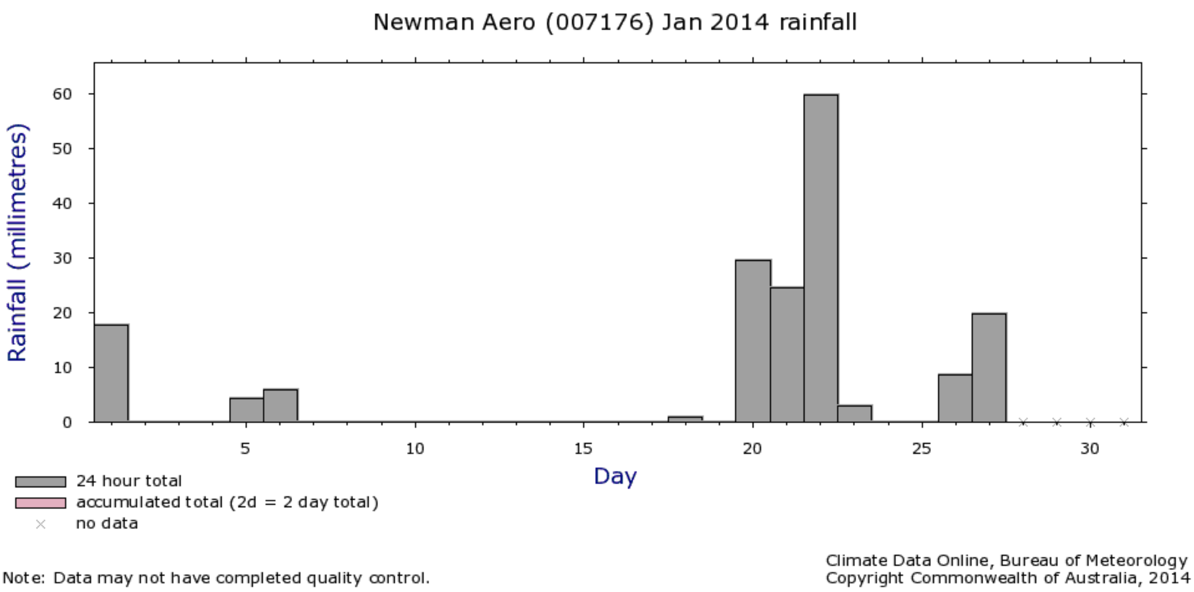


Figure B-25. Newman Airport January 2014 rainfall showing significant event 20-22 January

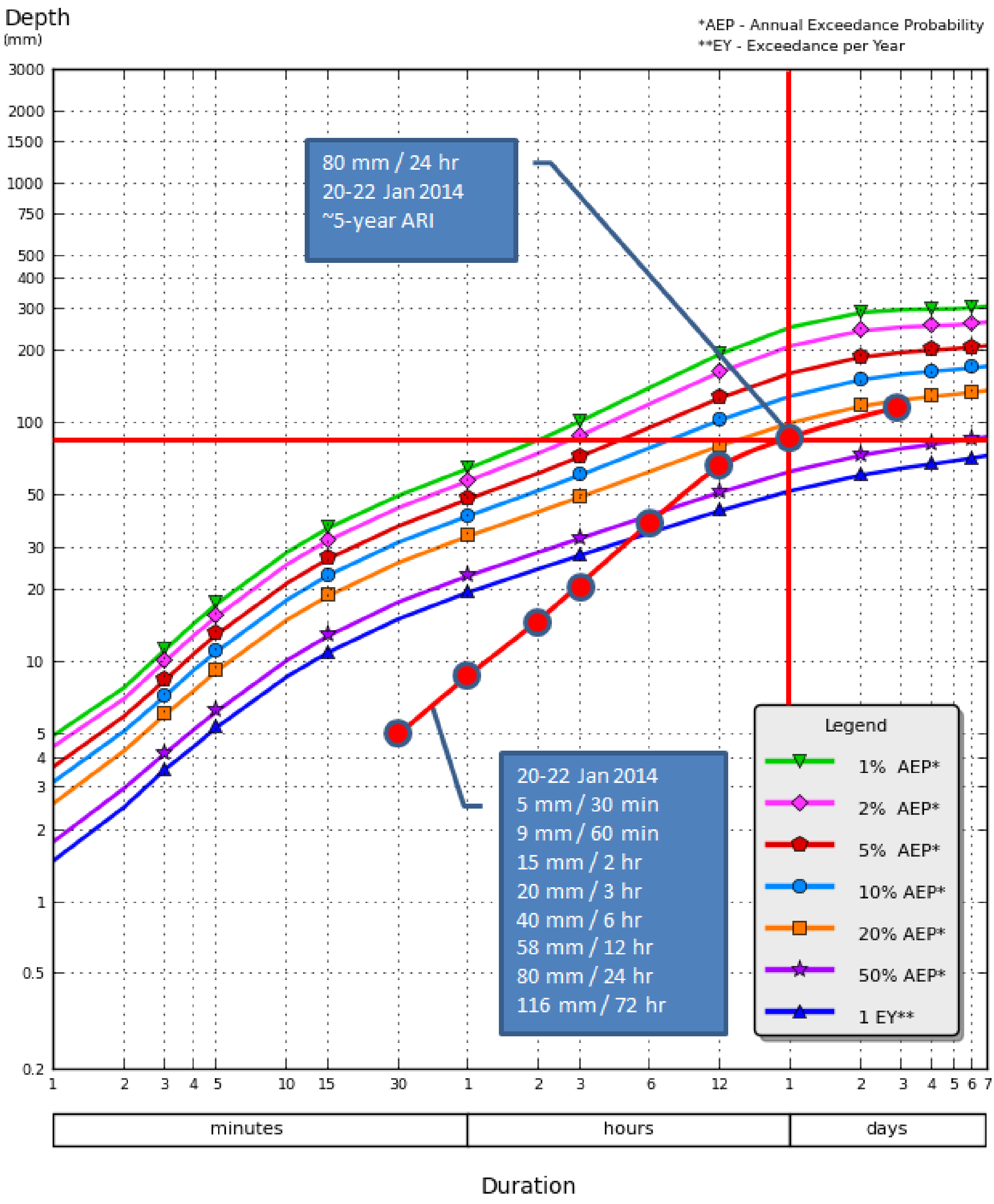


Figure B-26. Newman Airport Daily Rainfall Data 1971-2014 (BOM 2013)

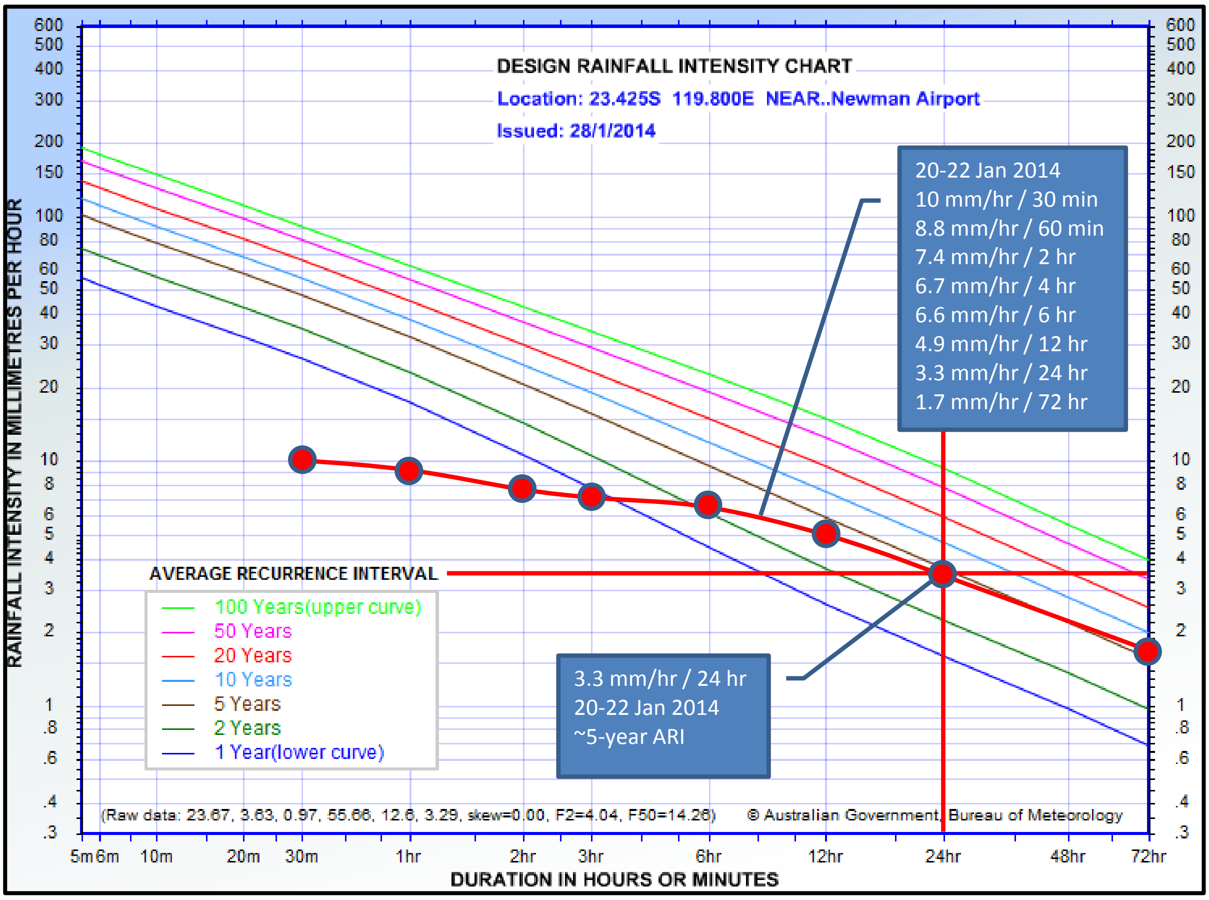


Figure B-27. Newman Airport Daily Rainfall Data 1971-2014 (BOM 1987)

Table B-3. Comparison of 1987 to 2013 Intensity-Frequency-Duration Data (BOM 1987, 2013)

Comparison of 1987 and 2013 BoM IFD data for Newman Airport, WA								
Duration		Comparison						
		1EY	50%	20%	10%	5%	2%	1%
Minutes	Hours	1-yr	2-yr	5-yr	10-yr	20-yr	50-yr	100-yr
5	0.083	13%	0%	7%	12%	11%	10%	9%
10	0.167	20%	7%	14%	19%	18%	17%	16%
30	0.5	14%	1%	8%	12%	11%	10%	9%
60	1	11%	-1%	3%	7%	6%	3%	2%
120	2	15%	0%	2%	5%	3%	1%	-1%
180	3	19%	3%	4%	5%	3%	1%	-1%
360	6	29%	11%	8%	9%	6%	3%	2%
720	12	36%	16%	12%	13%	10%	7%	7%
1440	24	35%	15%	13%	14%	12%	10%	10%
2880	48	28%	10%	10%	12%	10%	8%	8%
4320	72	28%	10%	9%	10%	7%	4%	3%
	10-20% decrease							
	0-10% decrease							
	0-10% increase							
	10-20% increase							
	20-40% increase							

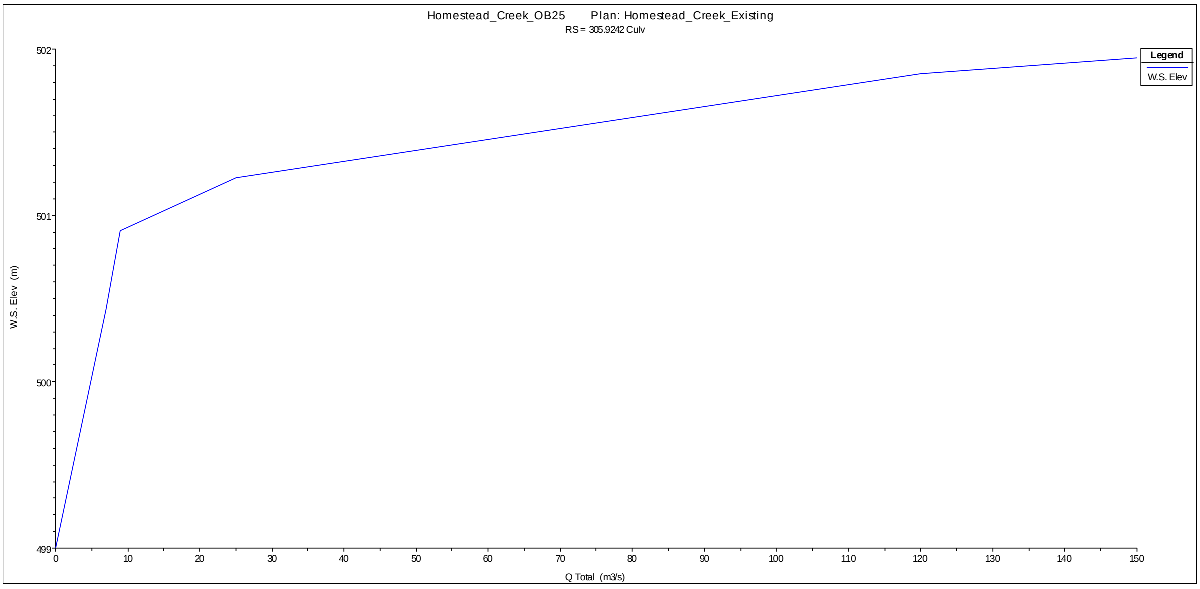


Figure B-28. Homestead Creek at Marble Bar Road Weir Rating Curve, no backwater

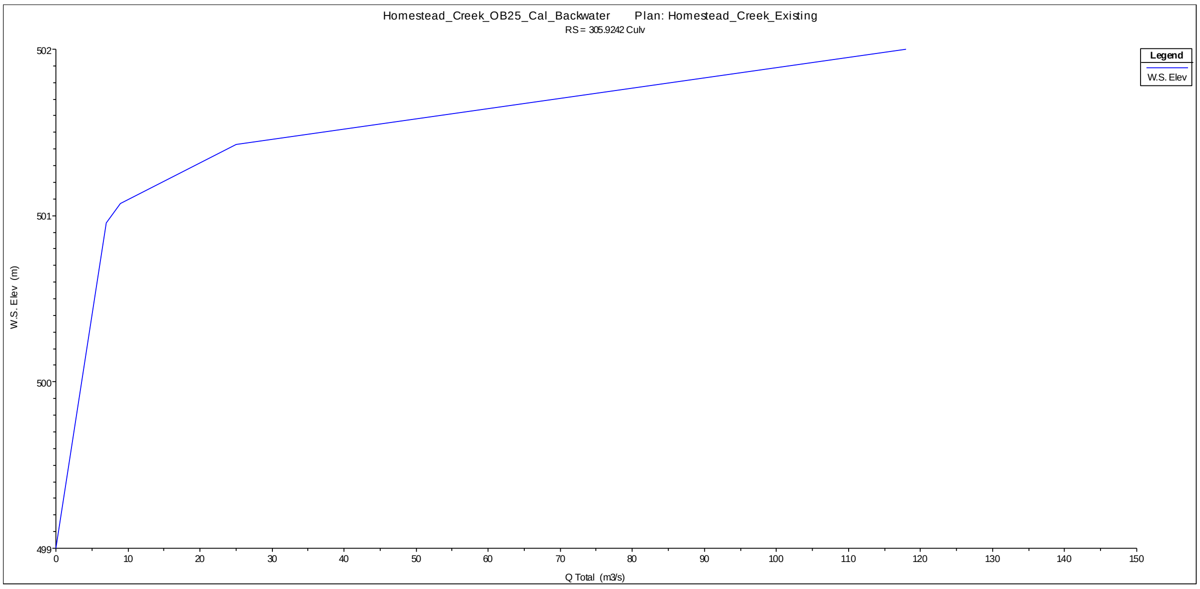


Figure B-29. Homestead Creek at Marble Bar Road Weir Rating Curve, with backwater

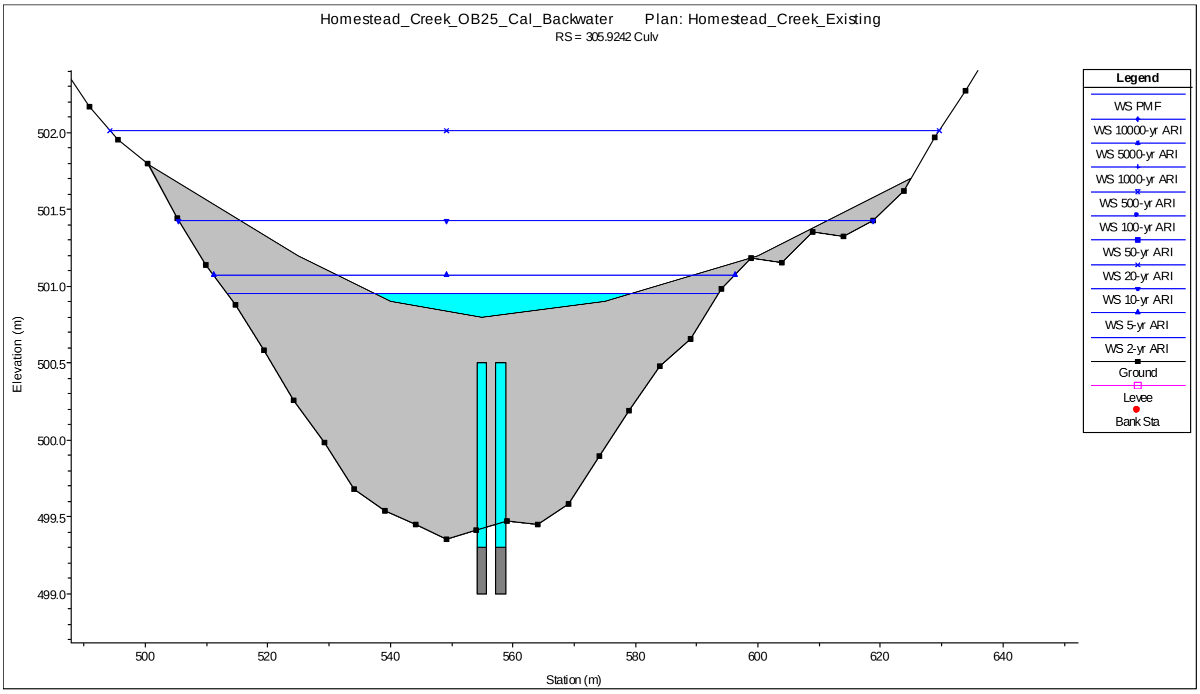


Figure B-30. Homestead Creek at Marble Bar Road Cross Section

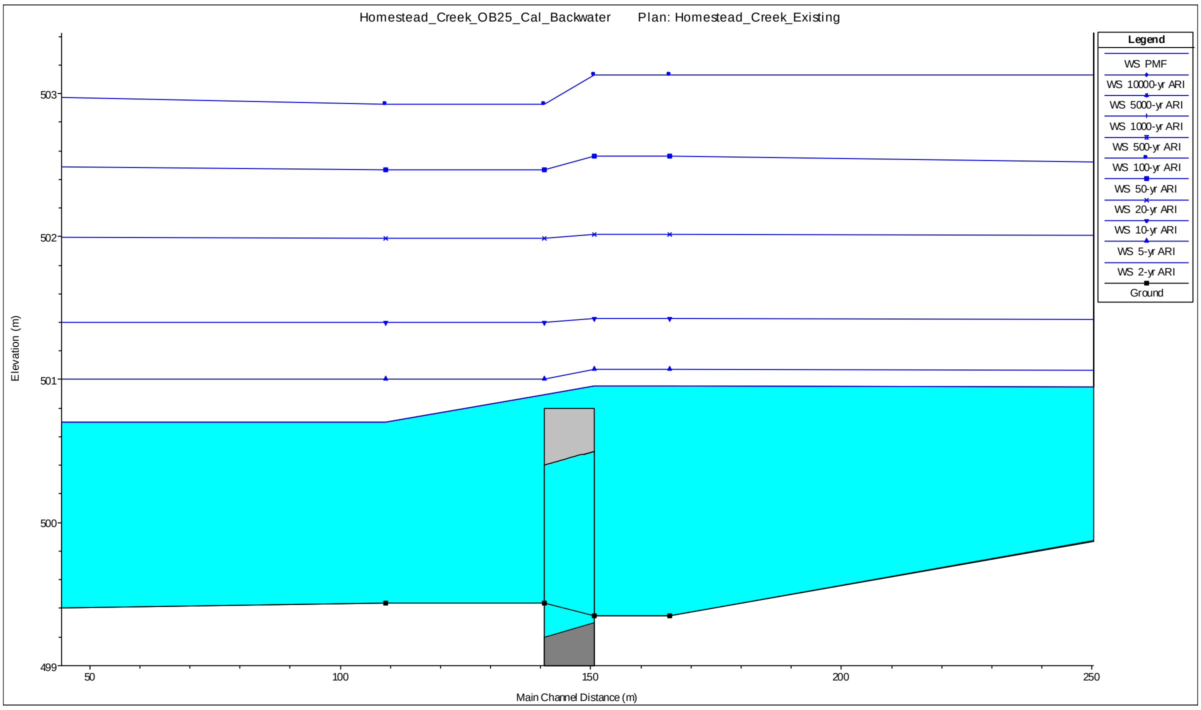


Figure B-31. Homestead Creek at Marble Bar Road Profile

Figure B-32. Homestead Creek at Marble Bar Road Floodway 21-22 January 2014

Figure B-33. Homestead Creek at Marble Bar Road Floodway 21-22 January 2014

Table B-4. Homestead Creek at Marble Bar Road Floodway Readings and Computed Discharge

Time of Reading	Stage Reading	Water Surface Elevation	Culvert Flow	Weir Flow	Total Flow	Flow Volume
	(m)	(M RL)	m^3/s	m^3/s	m^3/s	m^3
1/21/2014 15:00	0.00	499.4	0	0	0	-
1/21/2014 17:00	0.00	500.0	3	0	3	10,800
1/21/2014 20:30	0.45	501.3	5	12	17	126,000
1/21/2014 21:30	0.85	501.7	4	37	41	104,400
1/22/2014 3:00	1.10	501.9	3	95	98	1,376,100
1/22/2014 6:00	0.60	501.4	4	25	29	685,800
1/22/2014 13:00	0.20	501.0	5	7	12	516,600
1/22/2014 15:00	0.00	500.5	6	0	6	64,800
					Total:	2,884,500

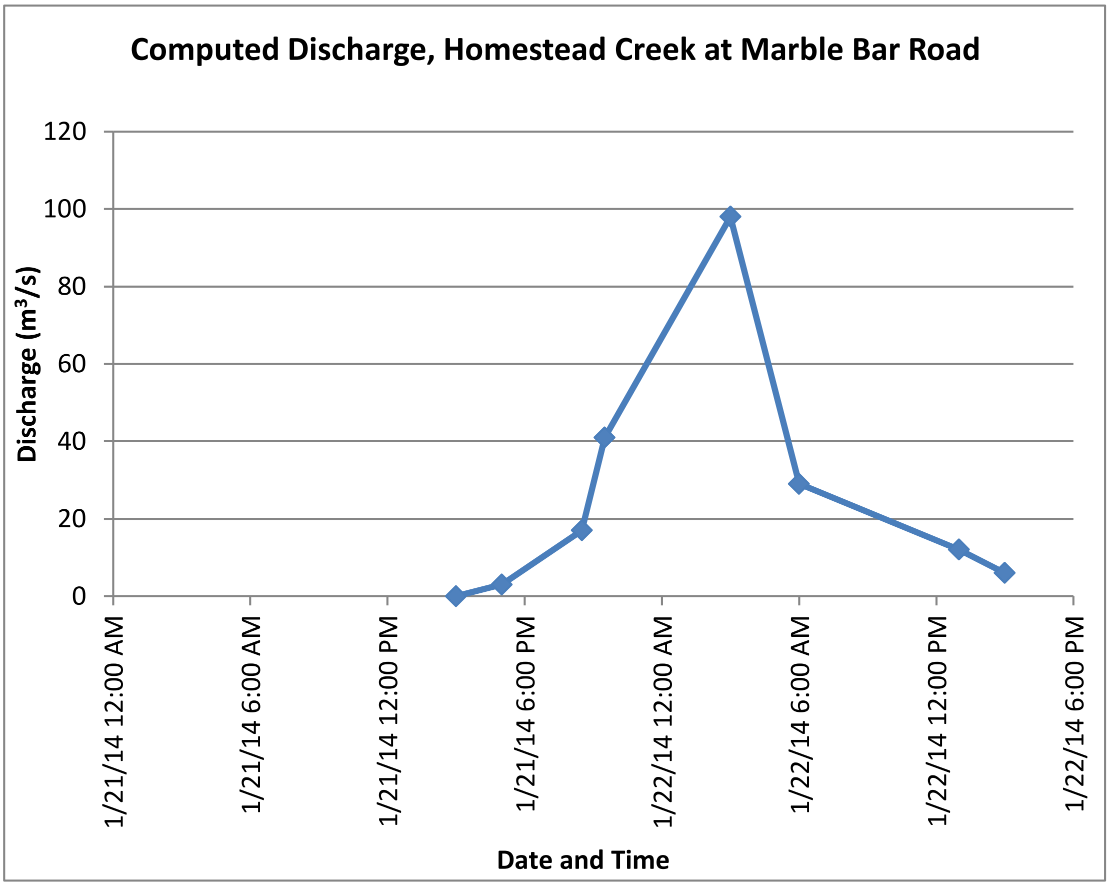


Figure B-34. Homestead Creek at Marble Bar Road Floodway Hydrograph 21-22 Jan 2014

Table B-5. Comparison of computed 100-year ARI discharges for varying loss rates and methods

Node	Peak discharge for 100-year ARI event (m^3/s)			
	10-mm loss	15-mm loss	20-mm loss	Proportional Loss
H1	619	588	556	403
H2	689	653	617	453
H3	679	646	614	449
H4	712	674	635	471
H5	688	651	612	459
H6*	**649**	**616**	**581**	**467**
H7	608	573	541	433
W1	497	467	435	298
W2	422	405	386	271
W3	507	479	452	331
W4	585	556	528	394
W5	553	525	496	370
* Peak discharge adjacent to OB25				

Table B-6. Summary of extreme peak flows

Node	Peak discharge for ARI event (m^3/s)					
	100-yr	500-yr	1,000-yr	5,000-yr	10,000-yr	PMF
H1	619	753	895	1,294	1,507	3,003
H2	689	841	1,000	1,438	1,672	3,480
H3	679	838	998	1,441	1,677	3,428
H4	712	875	1,045	1,512	1,762	3,727
H5	688	846	1,011	1,483	1,736	3,574
H6*	**649**	**799**	**967**	**1,431**	**1,688**	**3,911**
H7	608	761	921	1,368	1,611	3,898
W1	497	609	712	993	1,142	1,500
W2	422	501	590	844	979	1,757
W3	507	620	735	1,052	1,219	2,381
W4	585	723	865	1,258	1,467	3,126
W5	553	686	823	1,204	1,407	2,993
* Peak discharge adjacent to OB25						

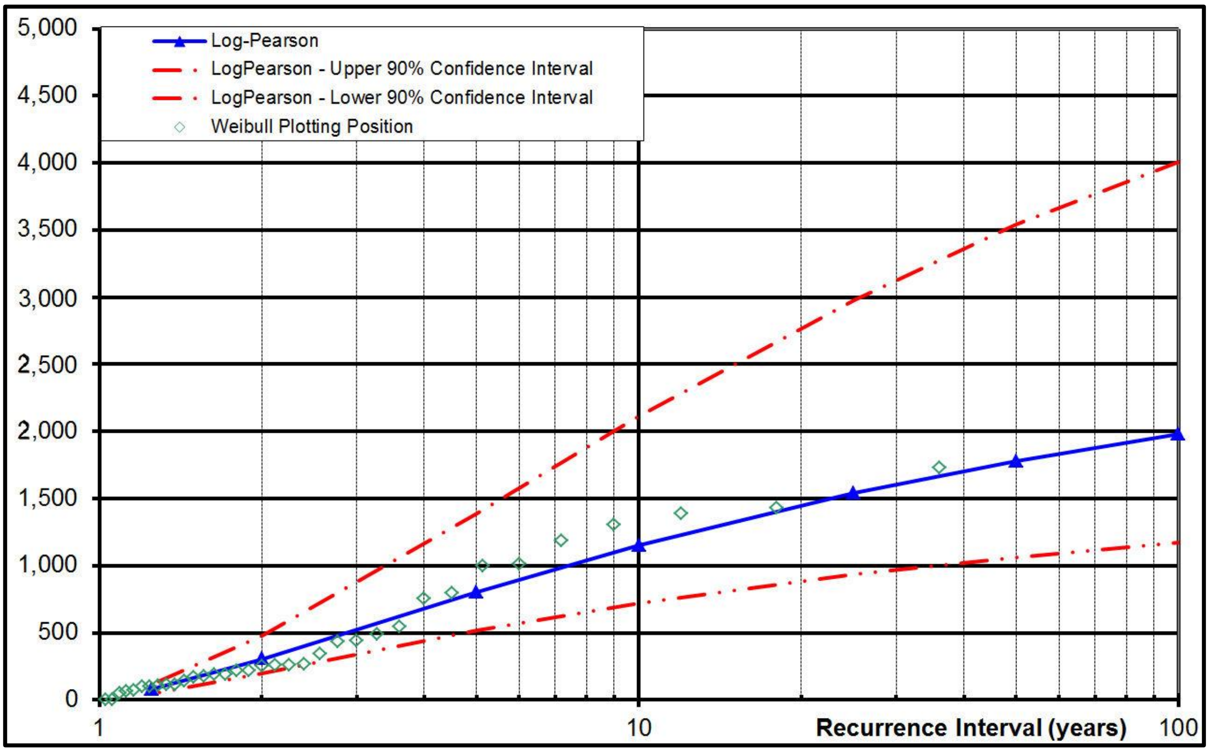


Figure B-35. Fortescue River at Newman Flood Frequency curve

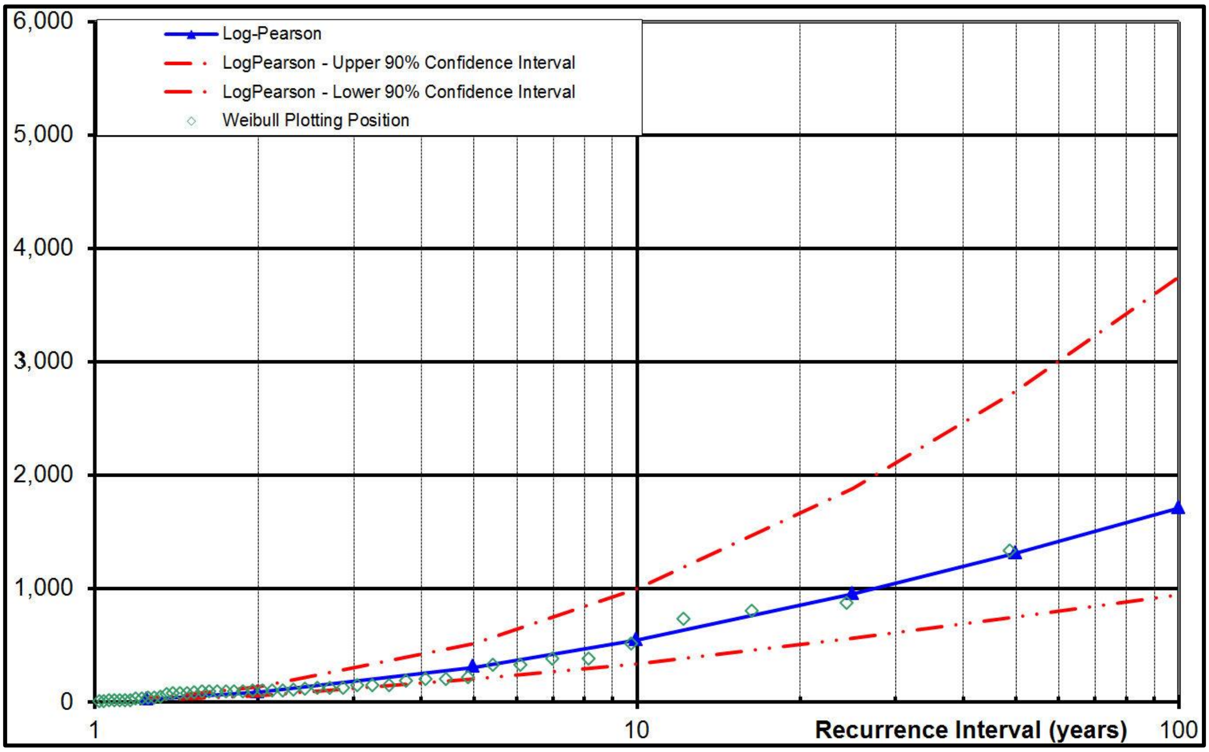


Figure B-36. Homestead Creek at Marble Bar Road Floodway Hydrograph 21-22 Jan 2014

Table B-7. Summary of extreme peak flows

Gauging Station>>	Marillana Creek at Flat Rocks	Fortescue River at Newman
Catchment Area (km^2)>>	1369	2822
2-year ARI peak Q (m^3/s)	85	305
5-year ARI peak Q (m3/s)	307	803
10-year ARI peak Q (m3/s)	547	1150
25-year ARI peak Q (m3/s)	949	1540
50-year ARI peak Q (m3/s)	1310	1780
100-year ARI peak Q (m3/s)	1710	1980
Node Location>>	Homestead Creek at Rail Loop	
	264	
2-year ARI peak Q (m^3/s)	27	58
5-year ARI peak Q (m3/s)	97	153
10-year ARI peak Q (m3/s)	173	219
25-year ARI peak Q (m3/s)	300	294
50-year ARI peak Q (m3/s)	414	339
100-year ARI peak Q (m3/s)	541	377
Node Location>>	Homestead Creek at Fortescue River	
	305	
2-year ARI peak Q (m^3/s)	30	64
5-year ARI peak Q (m3/s)	107	169
10-year ARI peak Q (m3/s)	191	242
25-year ARI peak Q (m3/s)	332	325
50-year ARI peak Q (m3/s)	458	375
100-year ARI peak Q (m3/s)	598	417
Node Location>>	Whaleback Creek at Ophthalmia	
	215	
2-year ARI peak Q (m^3/s)	23	50
5-year ARI peak Q (m3/s)	84	133
10-year ARI peak Q (m3/s)	150	190
25-year ARI peak Q (m3/s)	260	254
50-year ARI peak Q (m3/s)	359	294
100-year ARI peak Q (m3/s)	469	327

APPENDIX C: HYDRAULICS

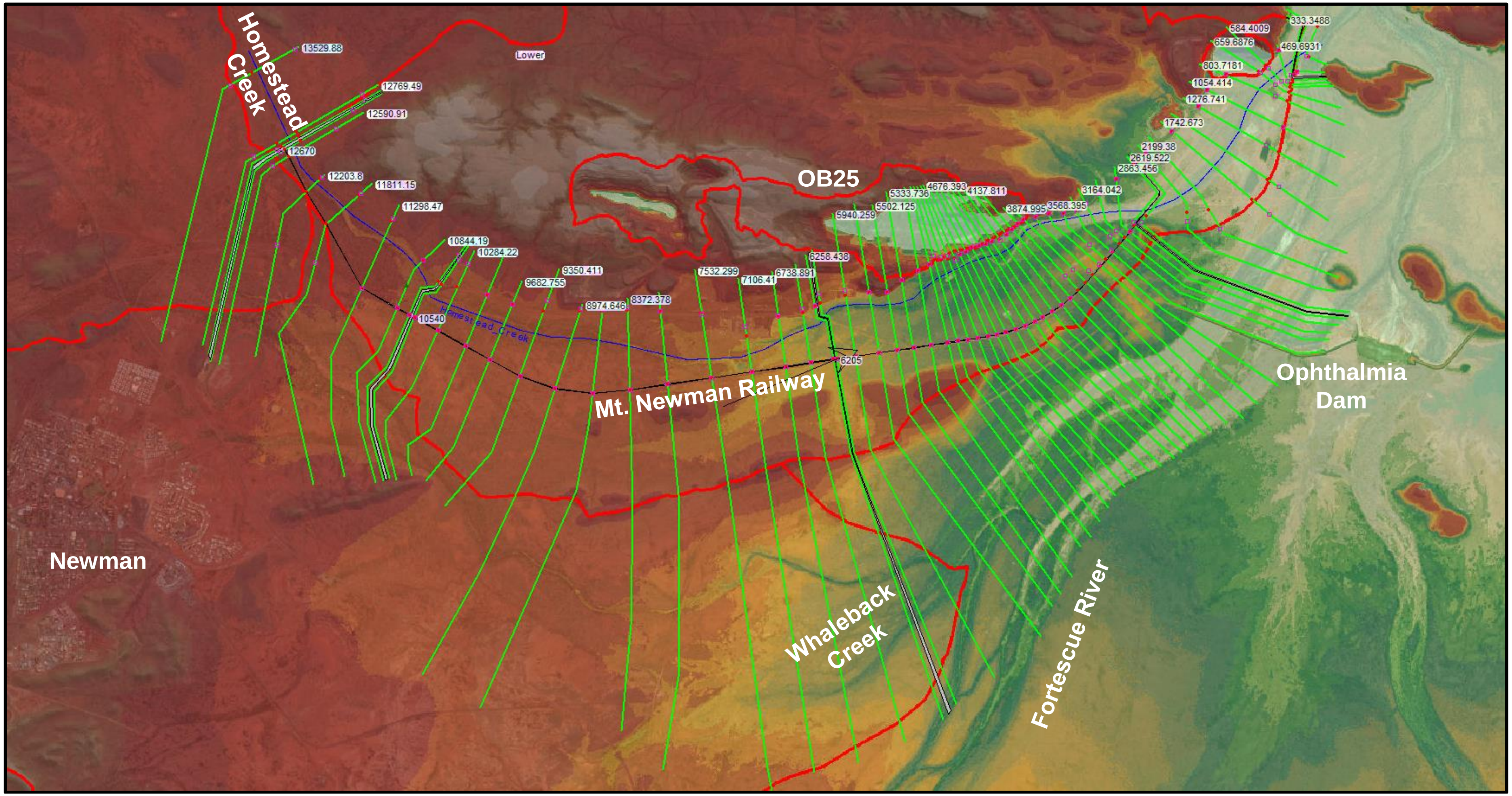


Figure C-1. HEC-RAS Geometry File Plan View

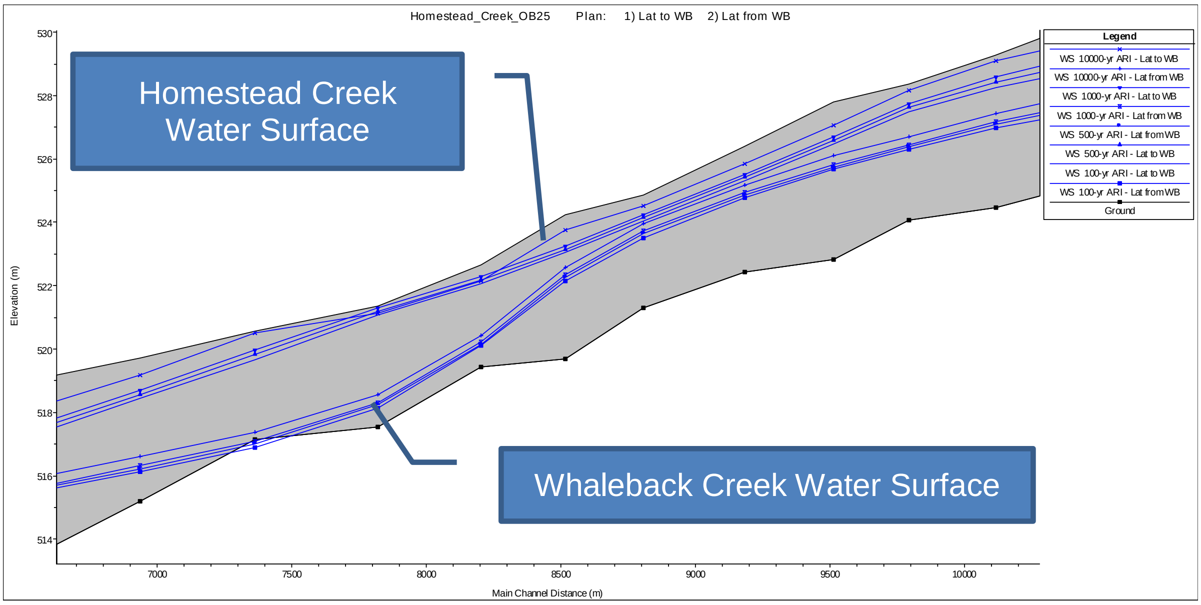


Figure C-2. Long-section profile of Whaleback and Homestead Creeks vs. railway embankment

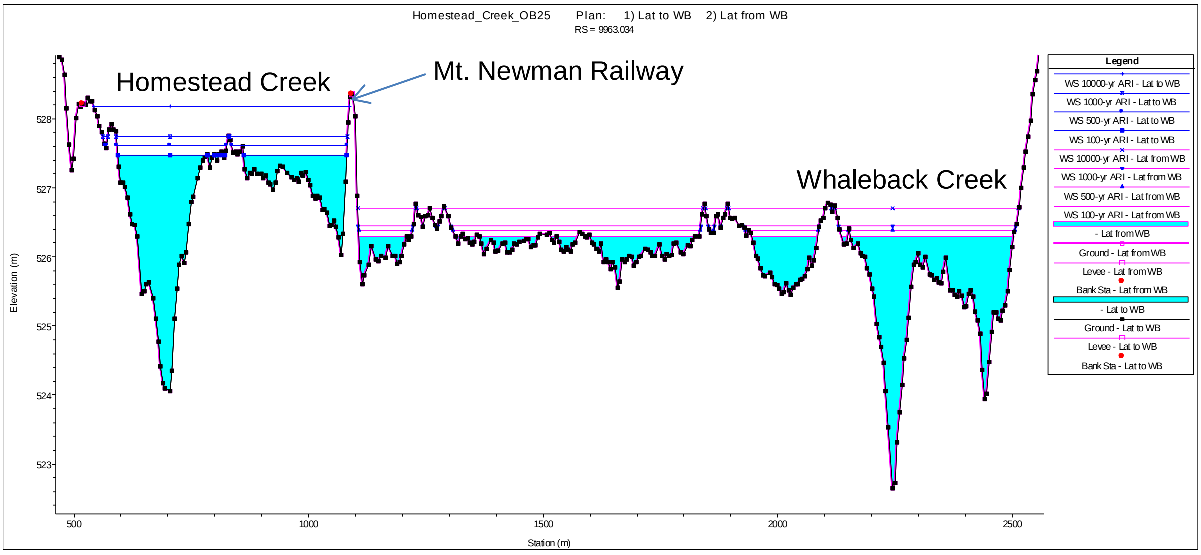


Figure C-3. Typical cross section showing relative levels of Homestead and Whaleback Creek

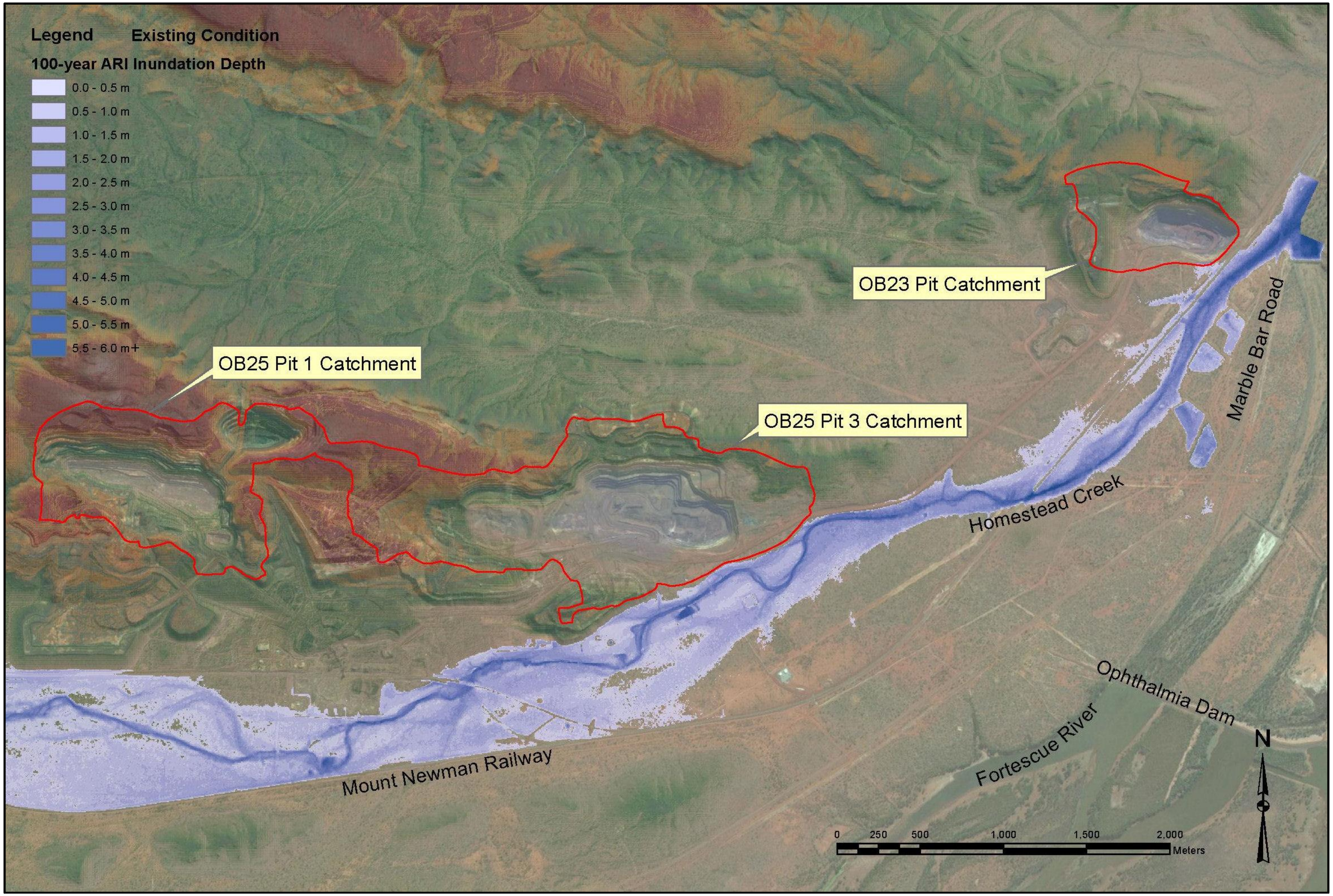


Figure C-4.100-year ARI Existing Conditions Inundation Depth

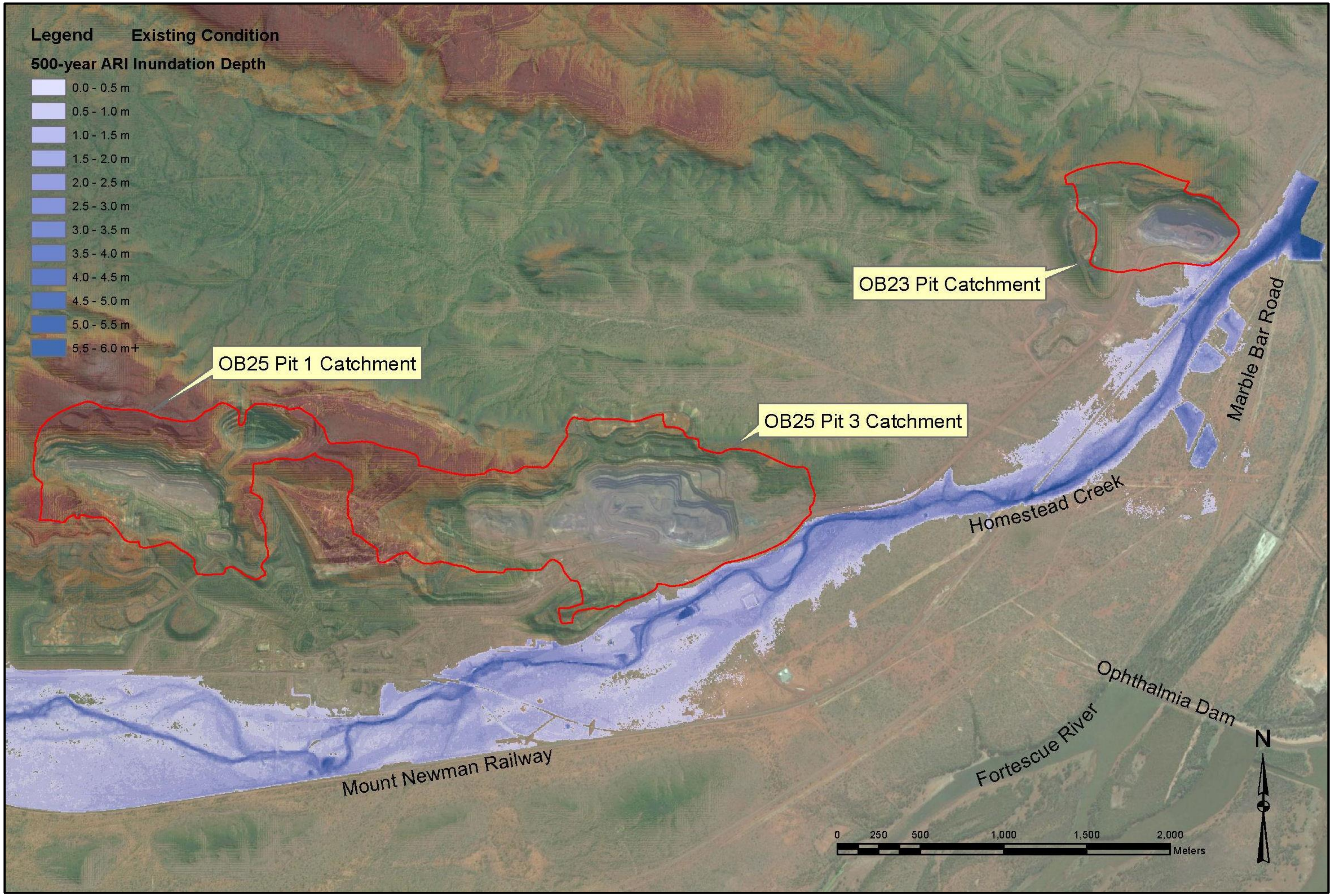


Figure C-4. 500-year ARI Existing Conditions Inundation Depth

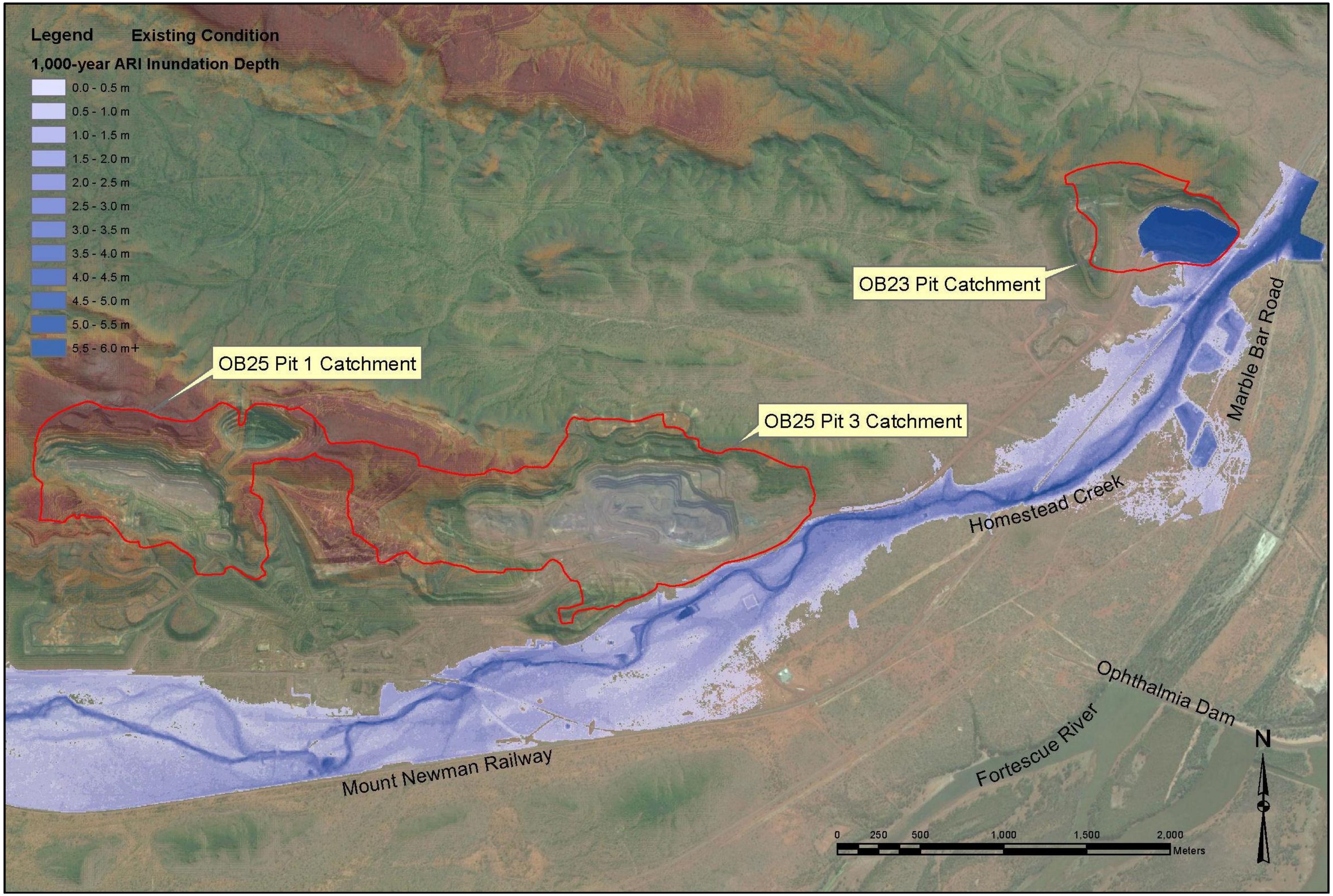


Figure C-6.1,000-year ARI Existing Conditions Inundation Depth

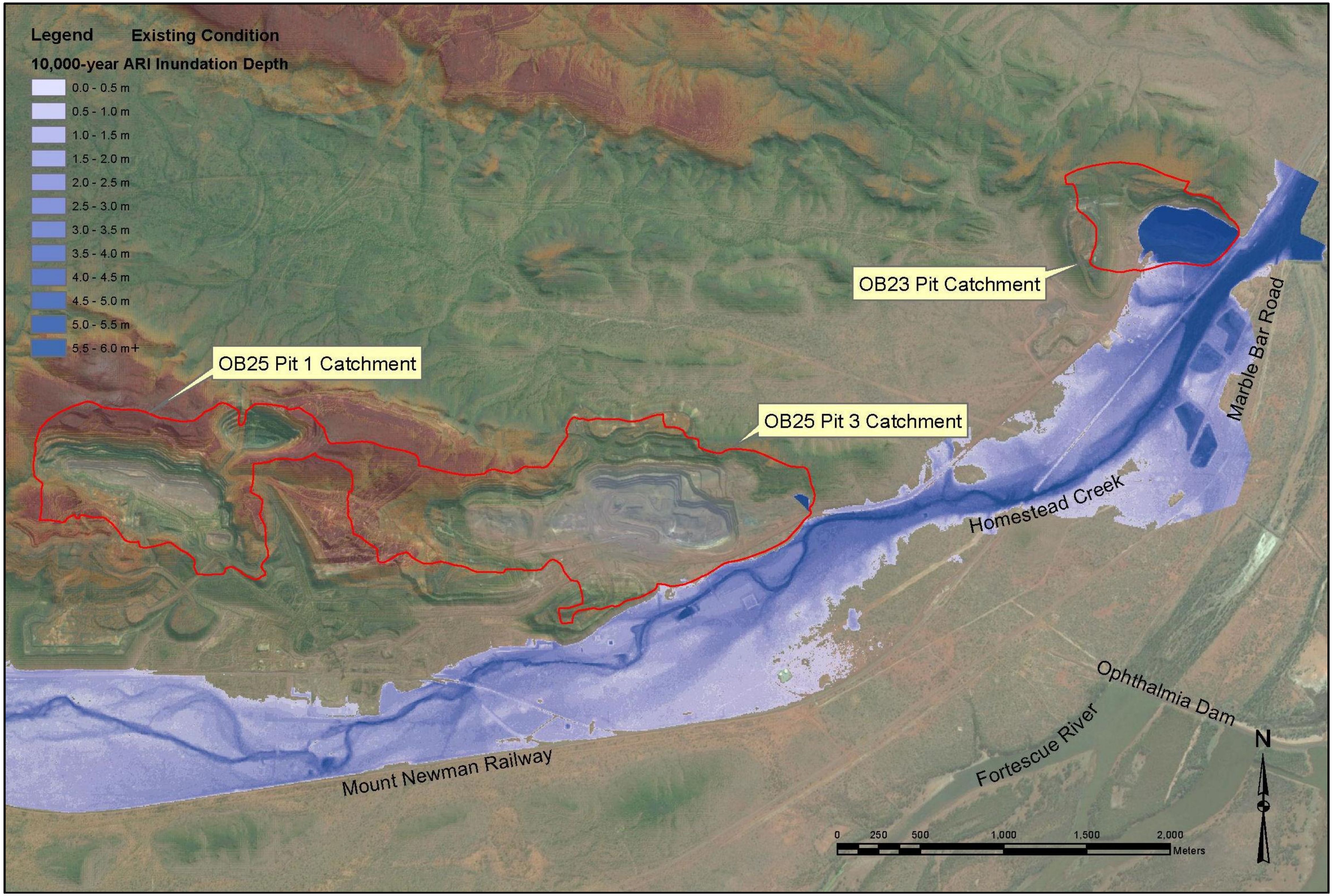


Figure C-7.10,000-year ARI Existing Conditions Inundation Depth

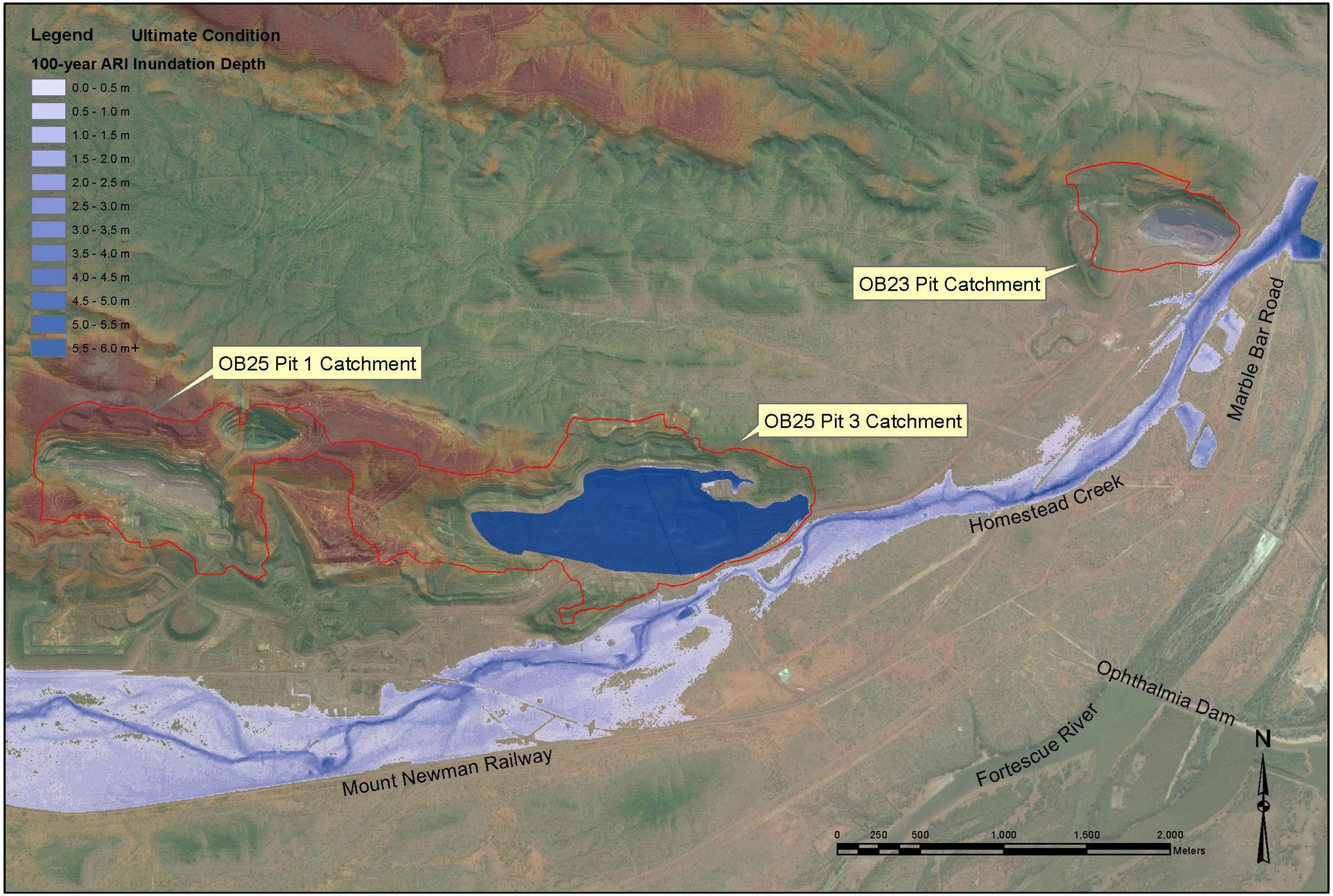


Figure C-8.100-year ARI Ultimate Conditions Inundation Depth

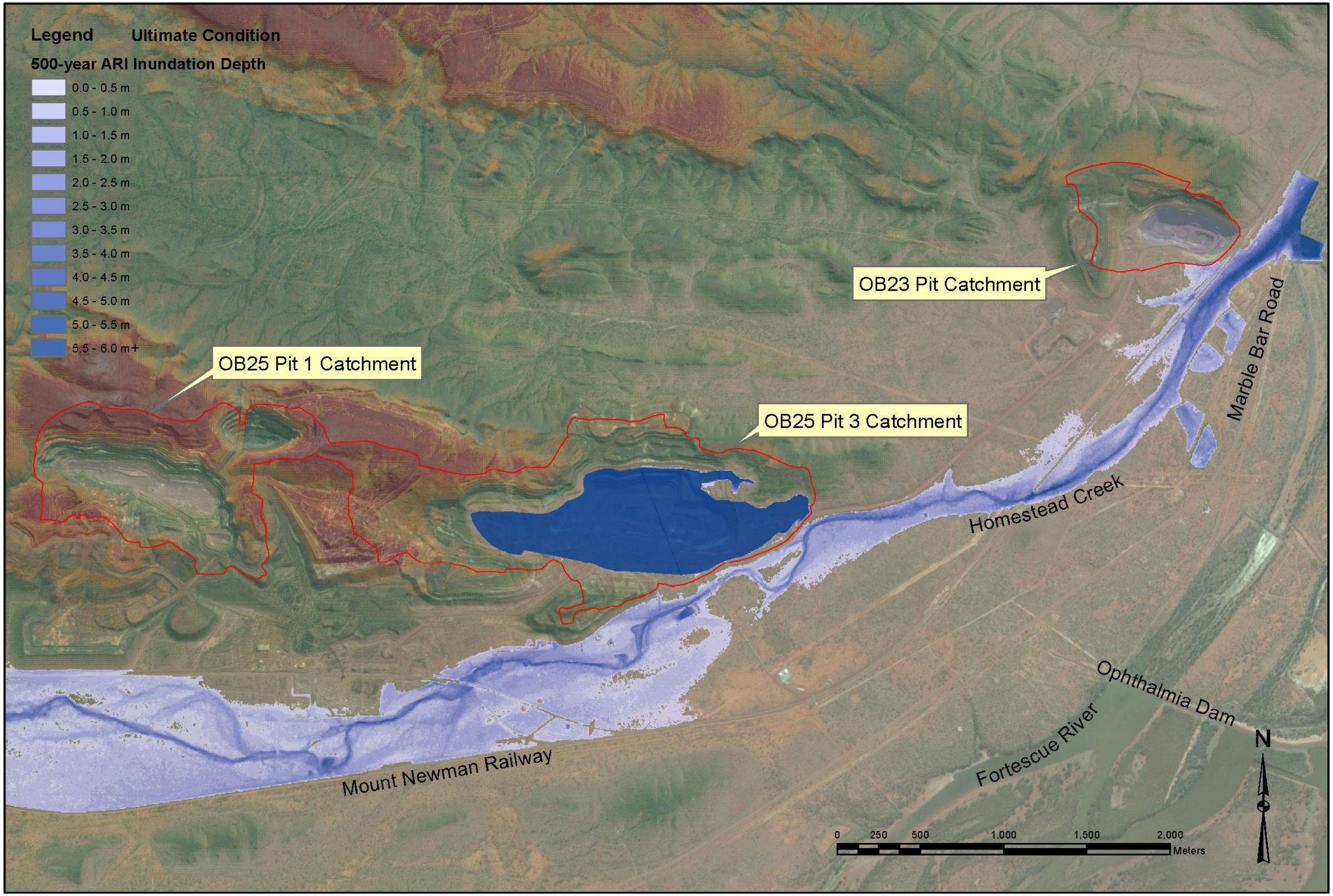


Figure C-9.500-year ARI Ultimate Conditions Inundation Depth

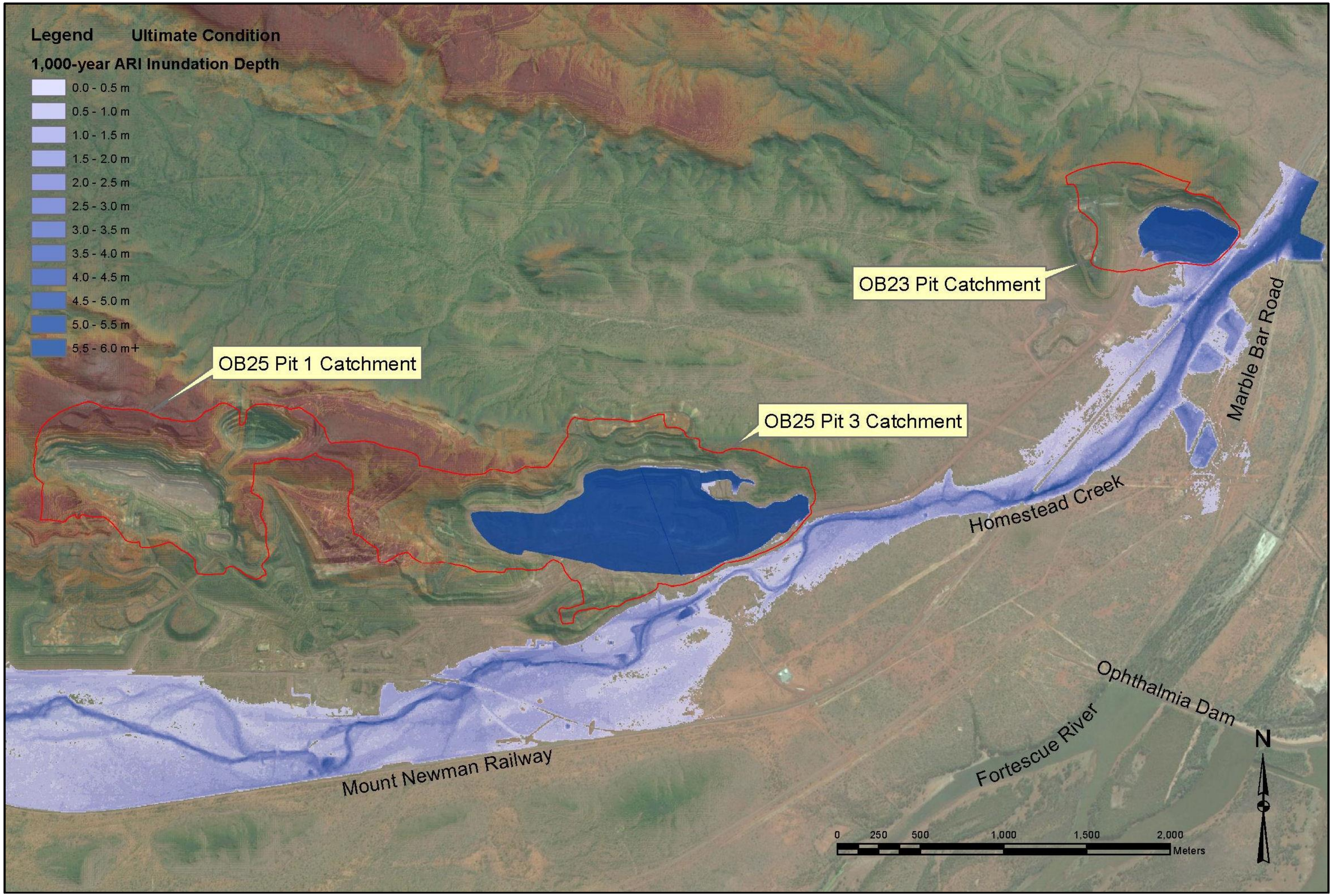


Figure C-10.1,000-year ARI Ultimate Conditions Inundation Depth

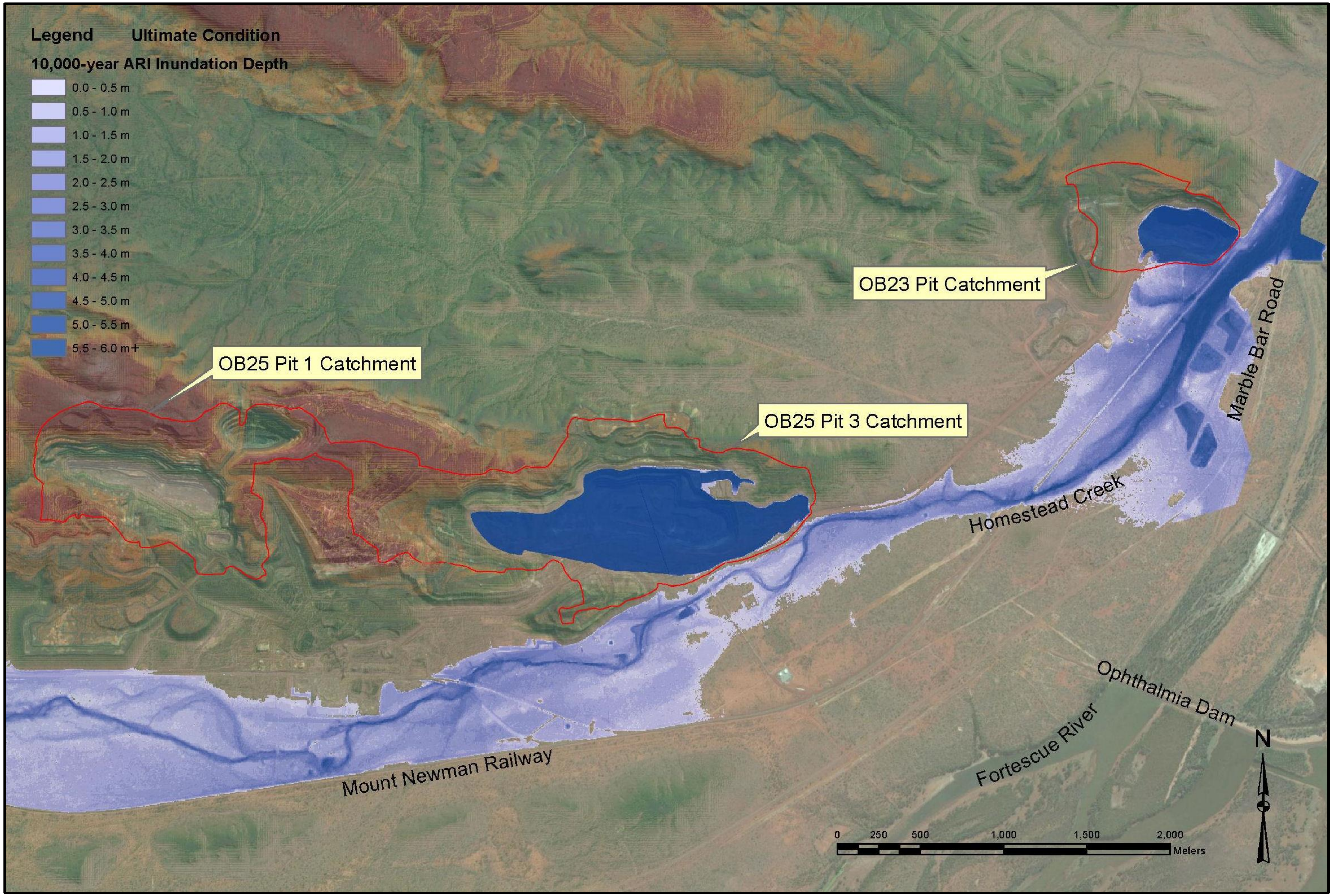


Figure C-11.10,000-year ARI Ultimate Conditions Inundation Depth

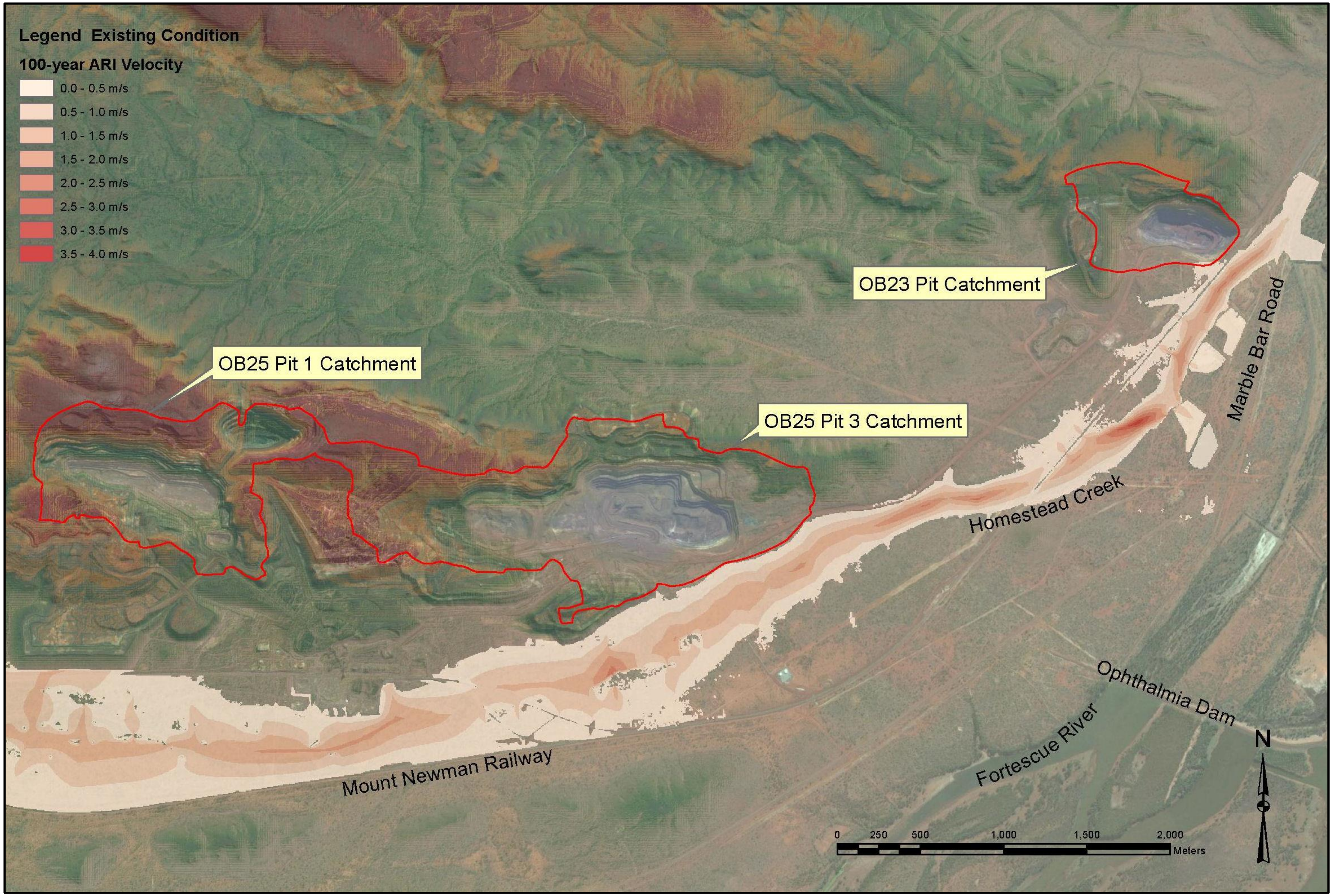


Figure C-12.100-year ARI Existing Conditions Velocities

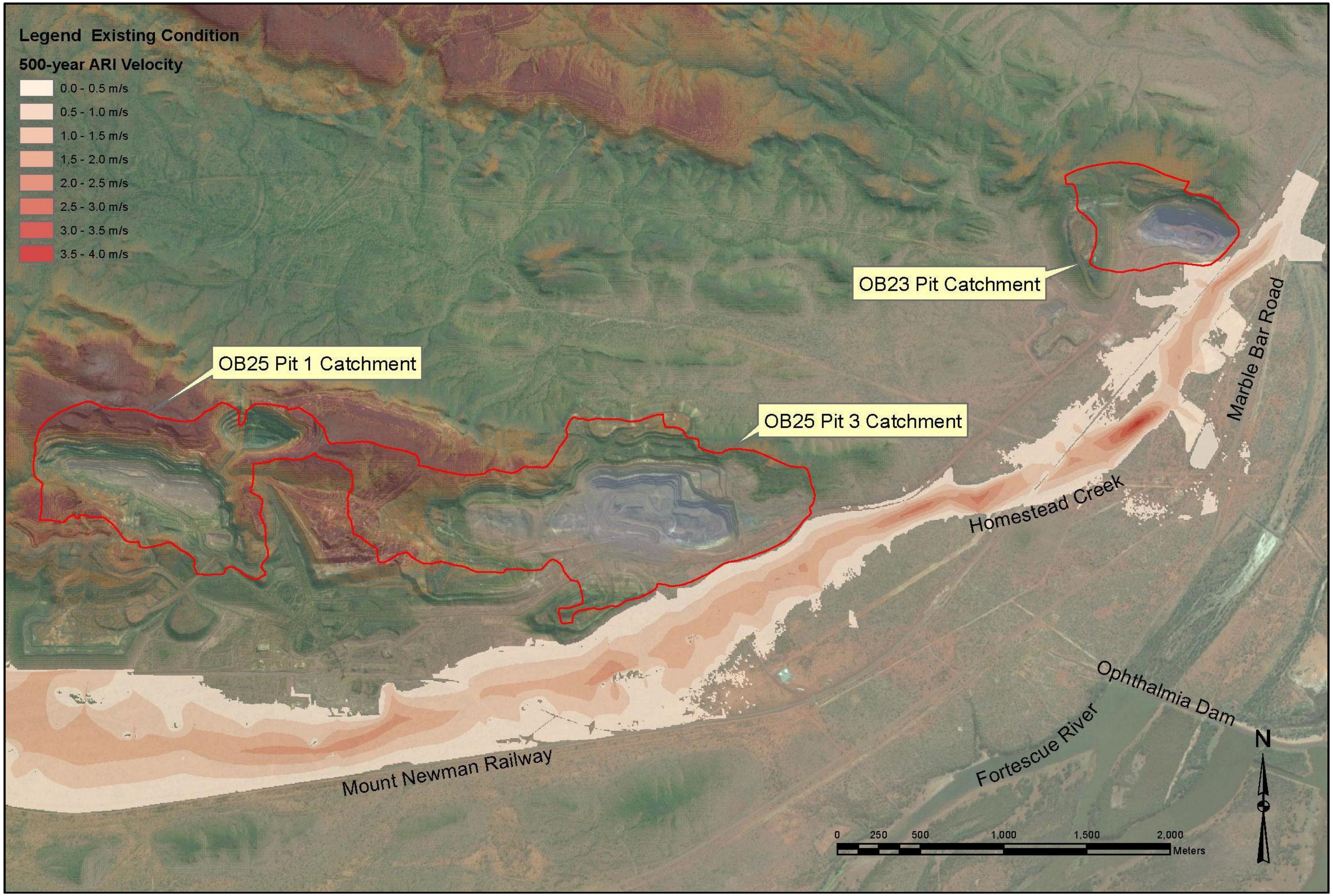


Figure C-13.500-year ARI Existing Conditions Velocities

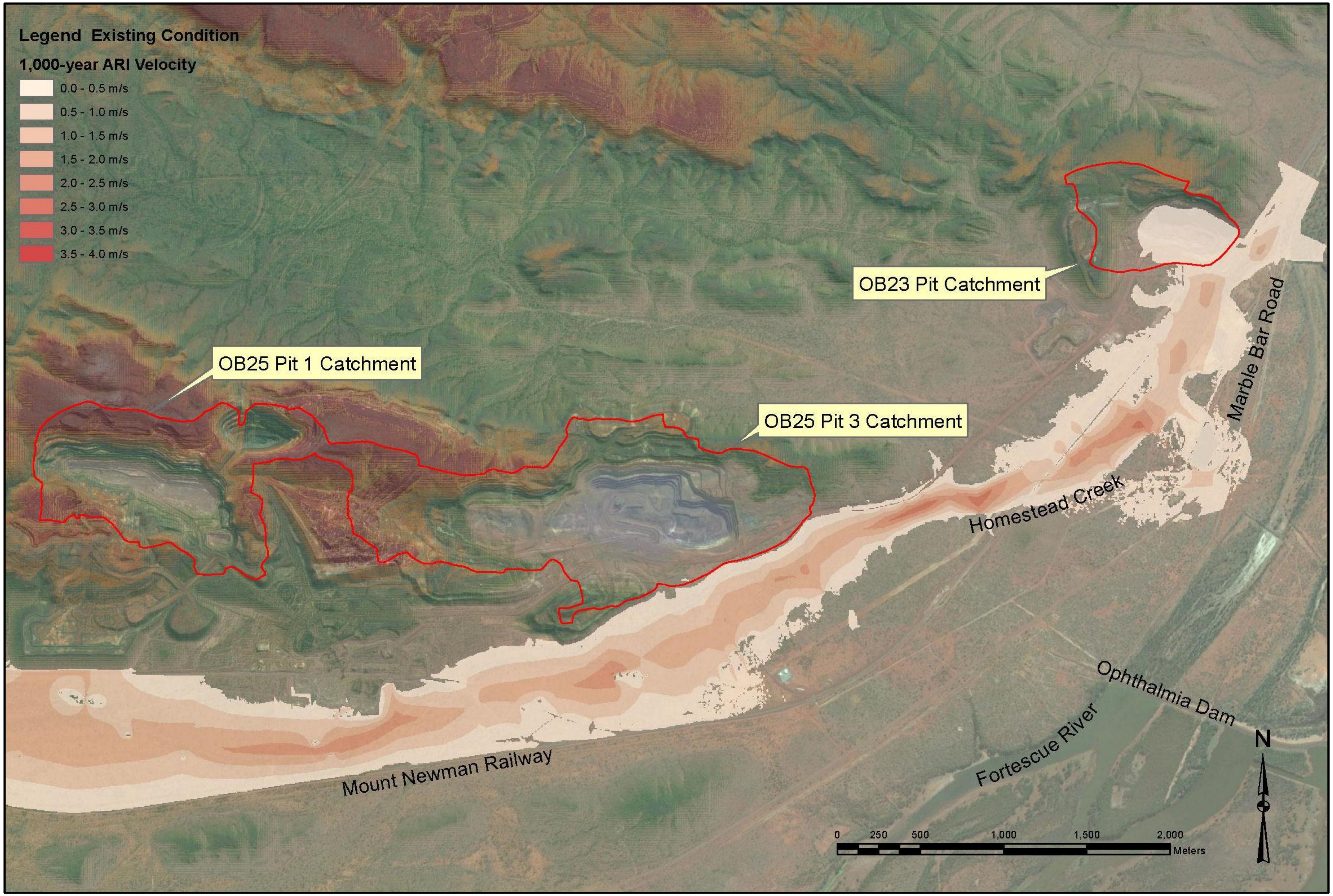


Figure C-14.1,000-year ARI Existing Conditions Velocities

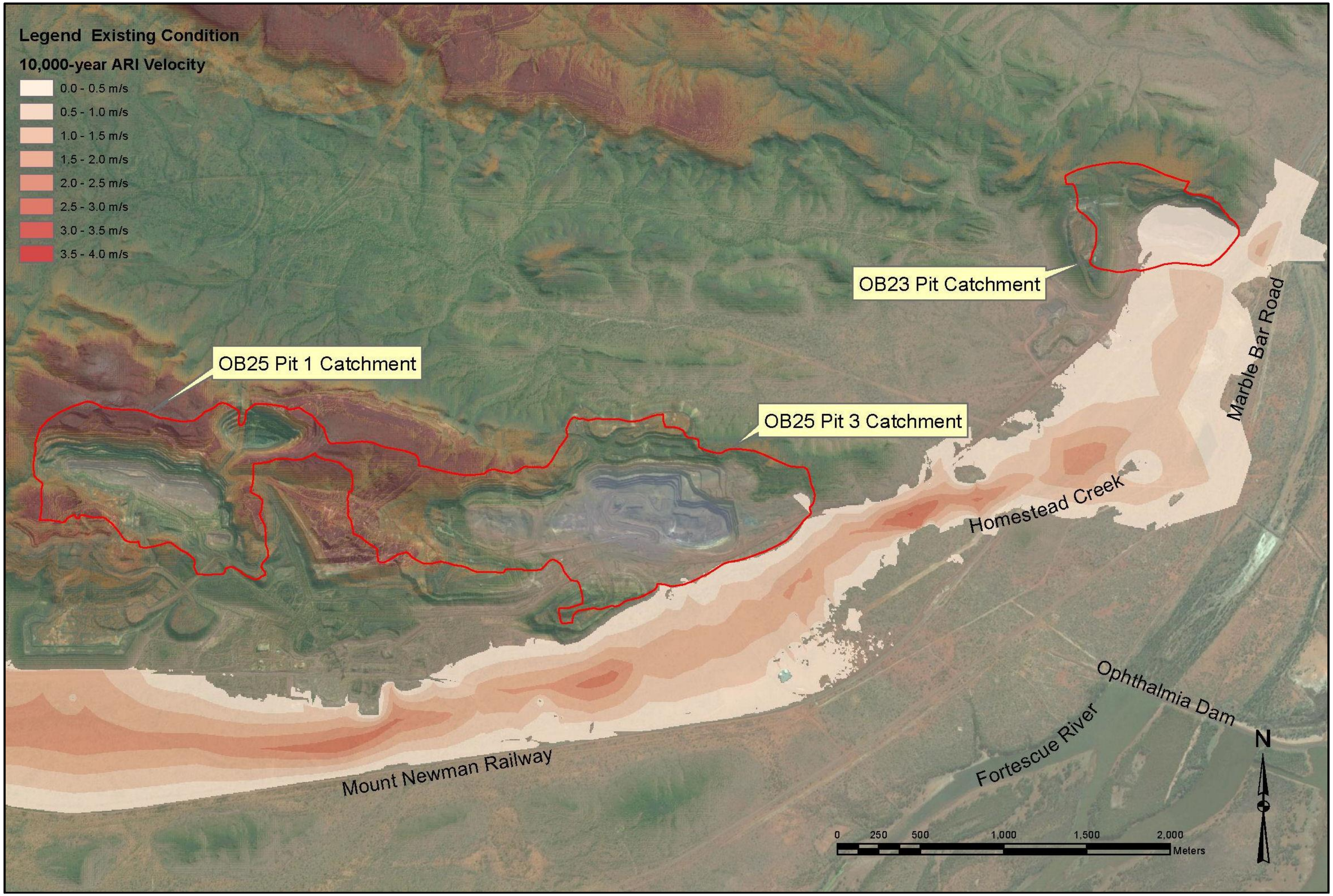


Figure C-15.10,000-year ARI Existing Conditions Velocities

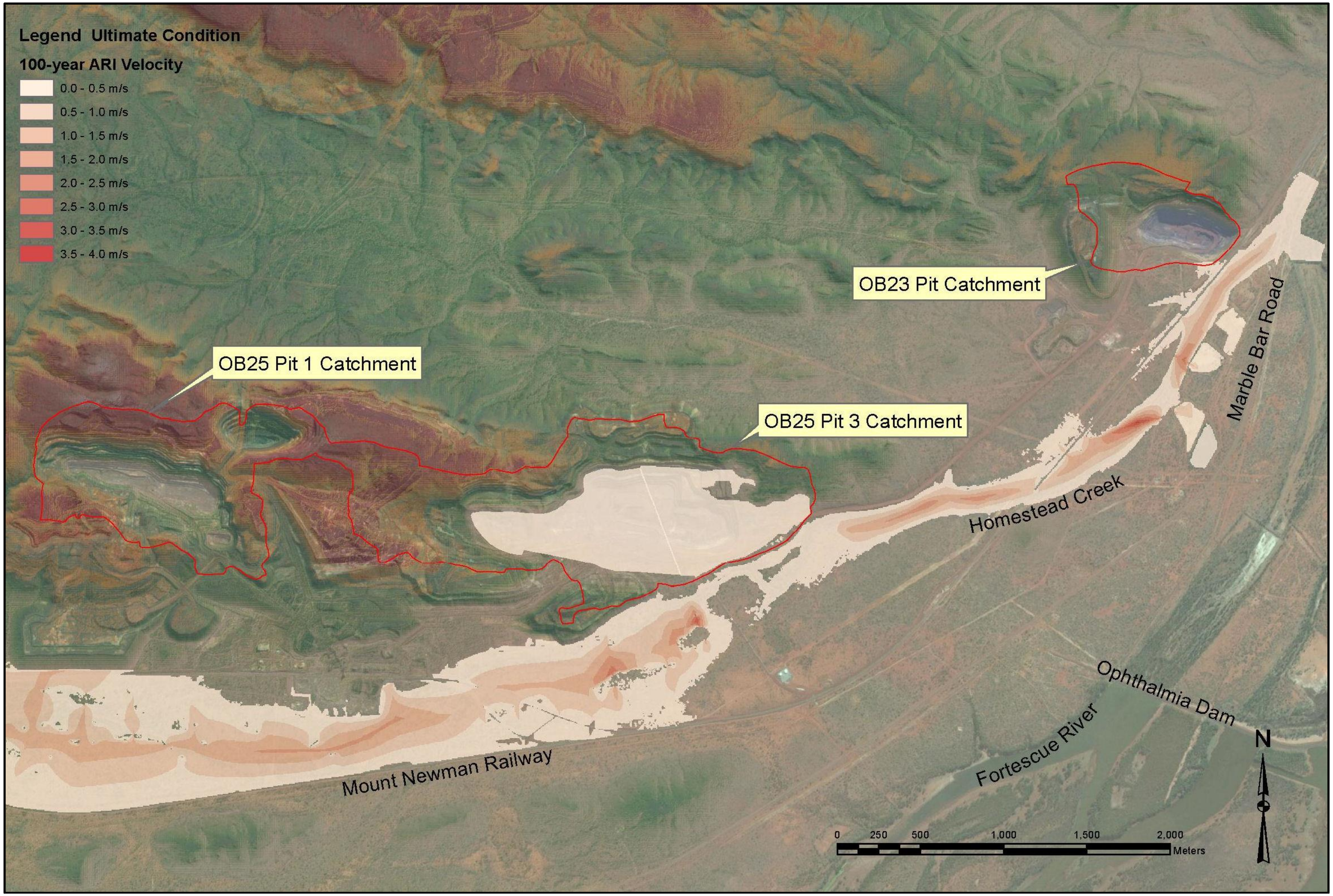


Figure C-16.100-year ARI Ultimate Conditions Velocities

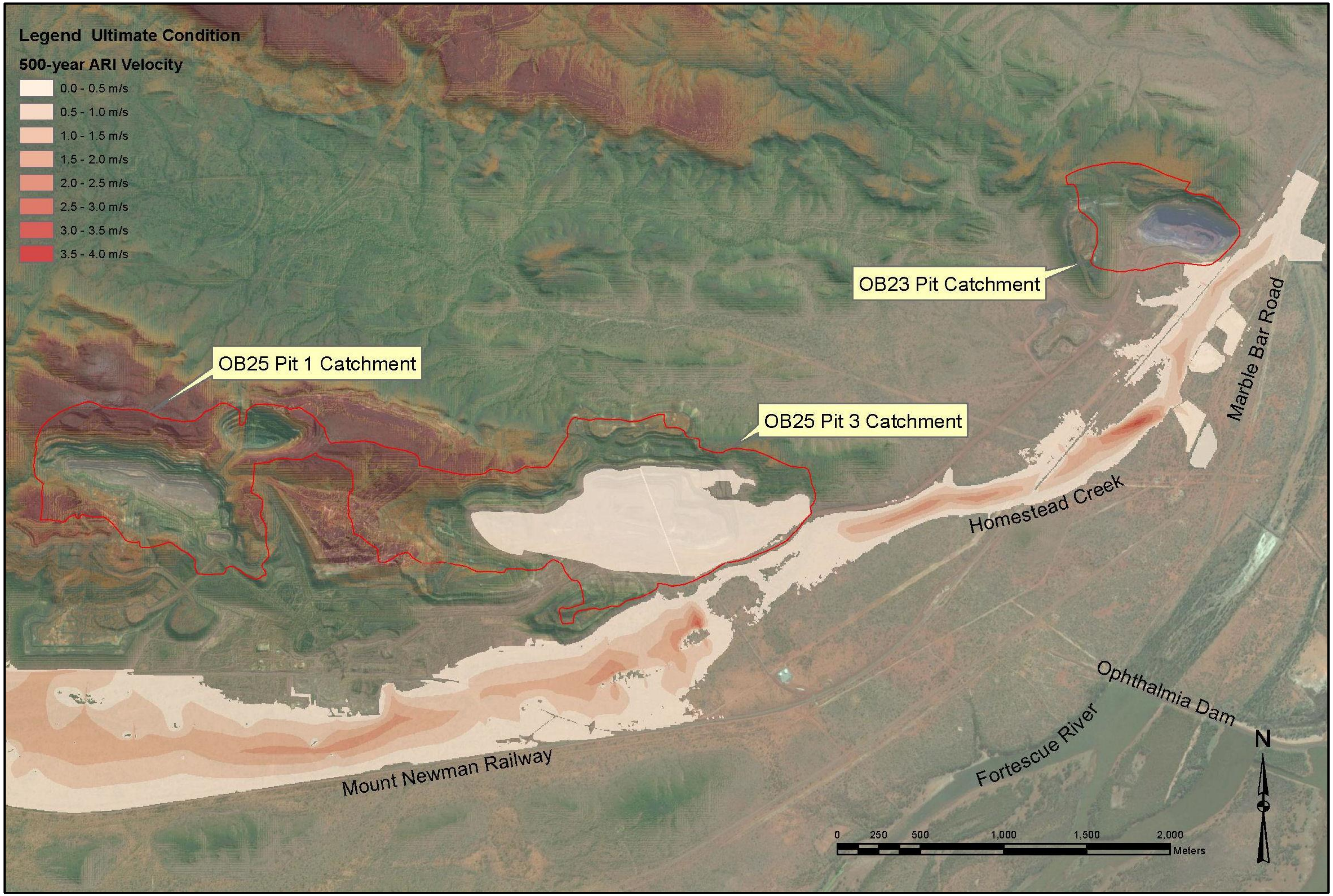


Figure C-17.500-year ARI Ultimate Conditions Velocities

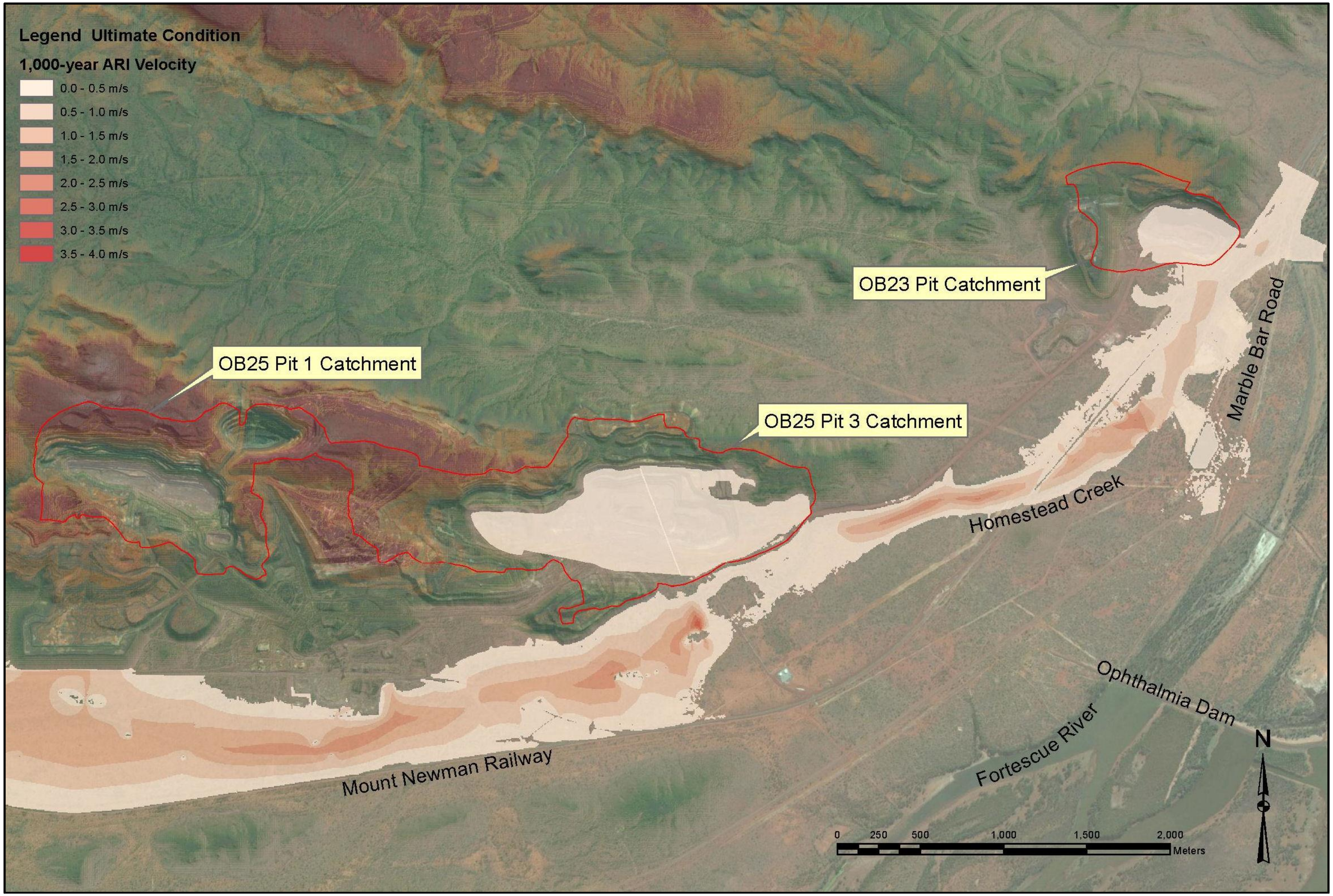


Figure C-18.1,000-year ARI Ultimate Conditions Velocities

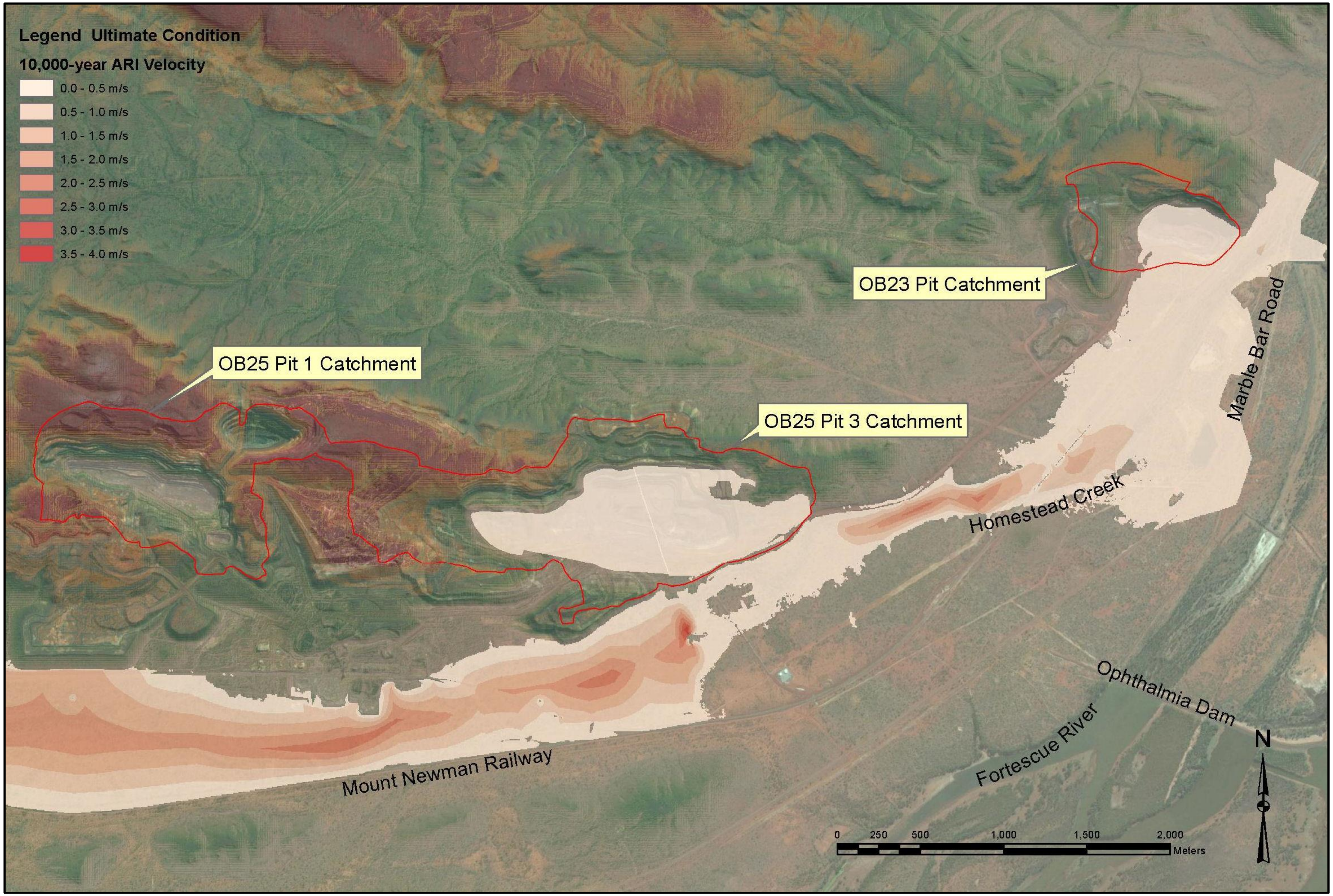


Figure C-19.10,000-year ARI Ultimate Conditions Velocities

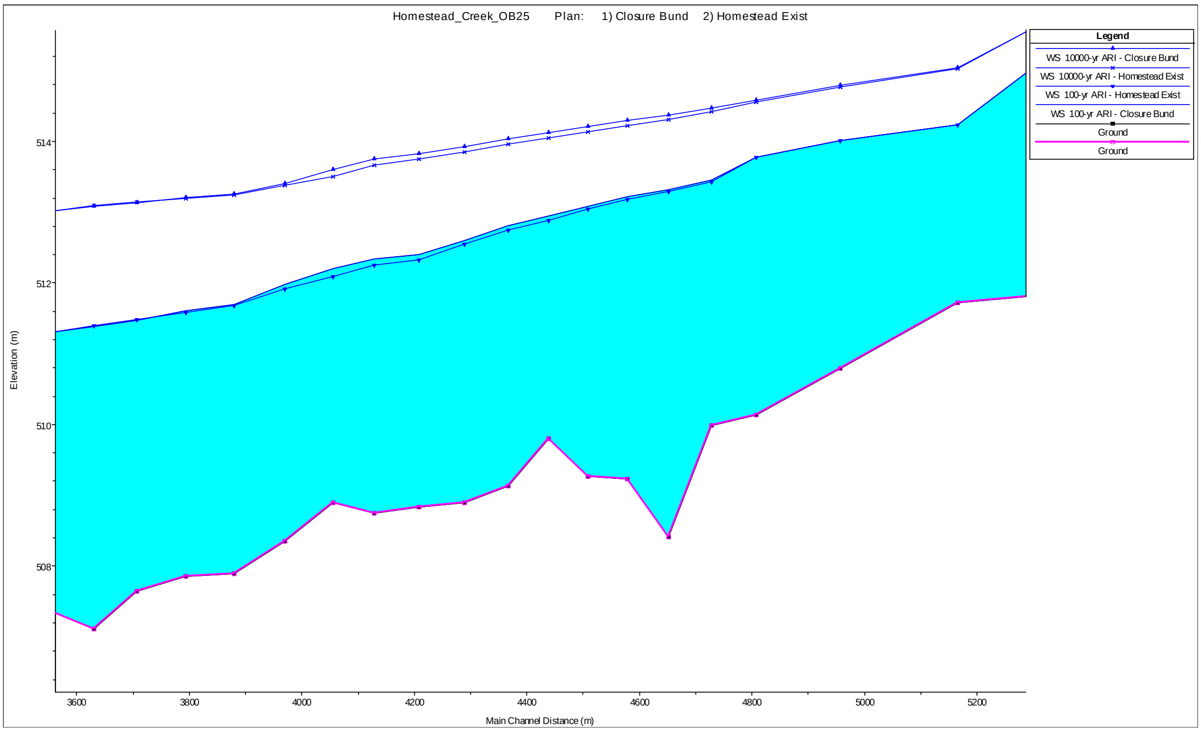


Figure C-20. Water surface profiles with and without 50-metre bund encroachment along Pit 3

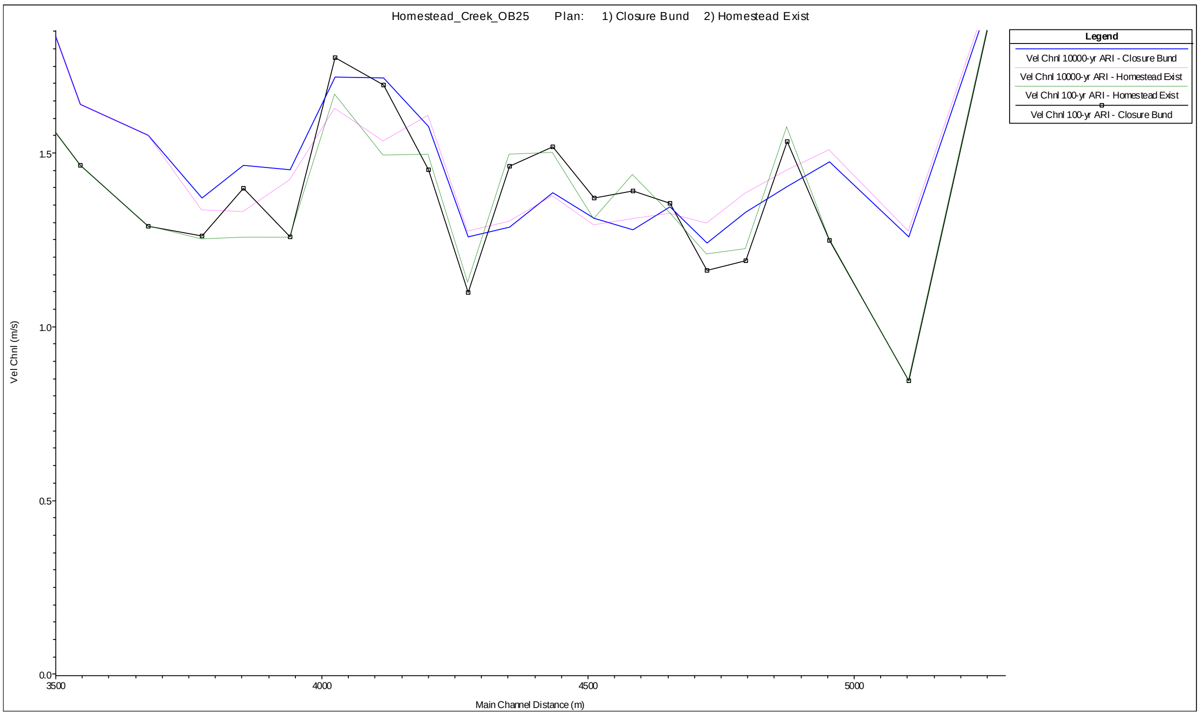


Figure C-21. Velocity profiles with and without 50-metre bund encroachment along Pit 3

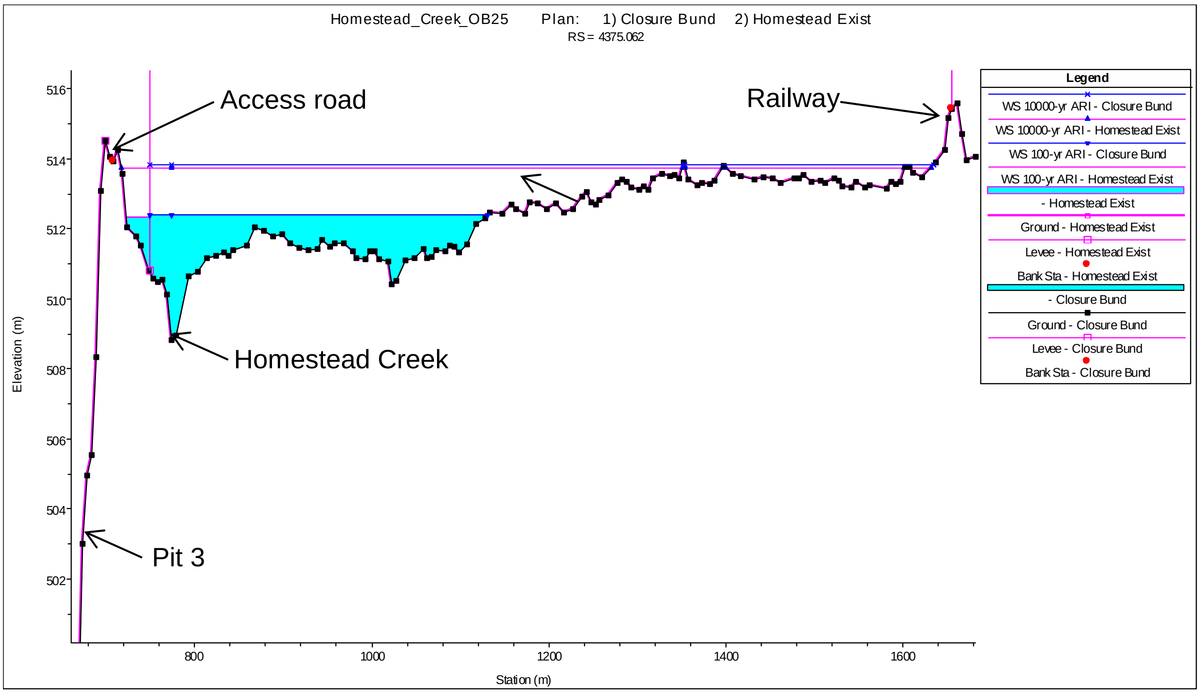


Figure C-22. Typical cross section showing 50-metre bund encroachment

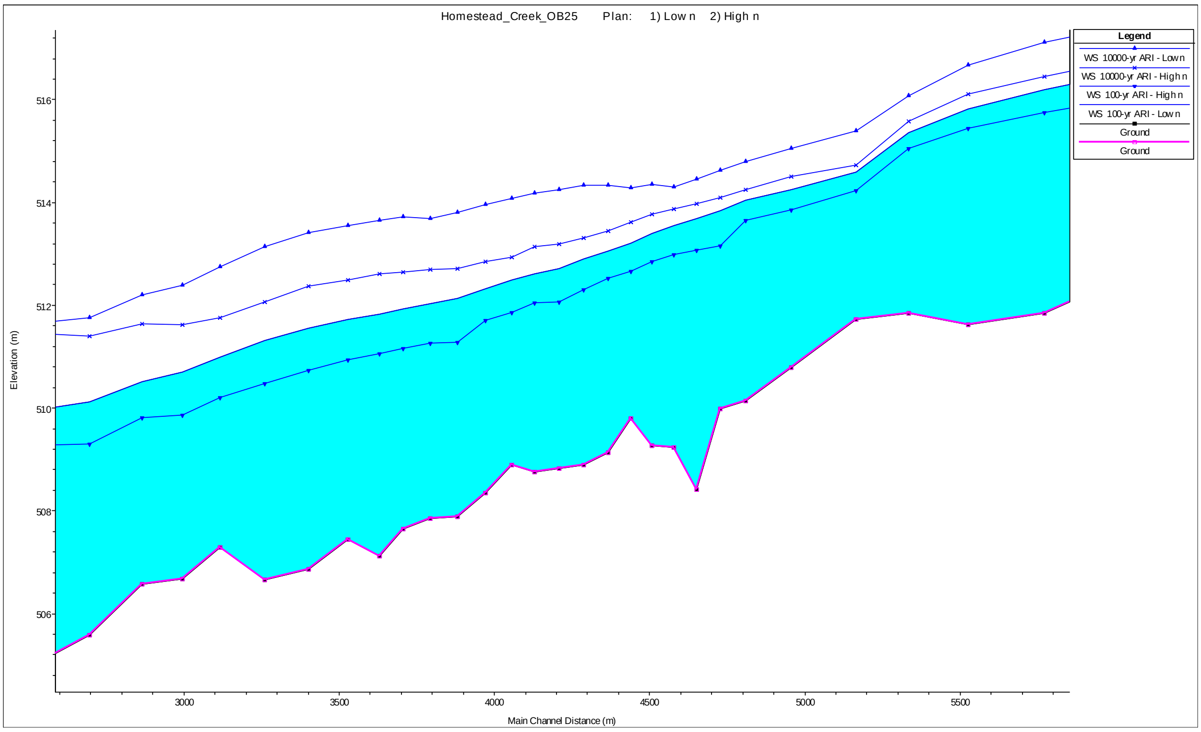


Figure C-23. Water surface profile for high and low roughness along Pit 3

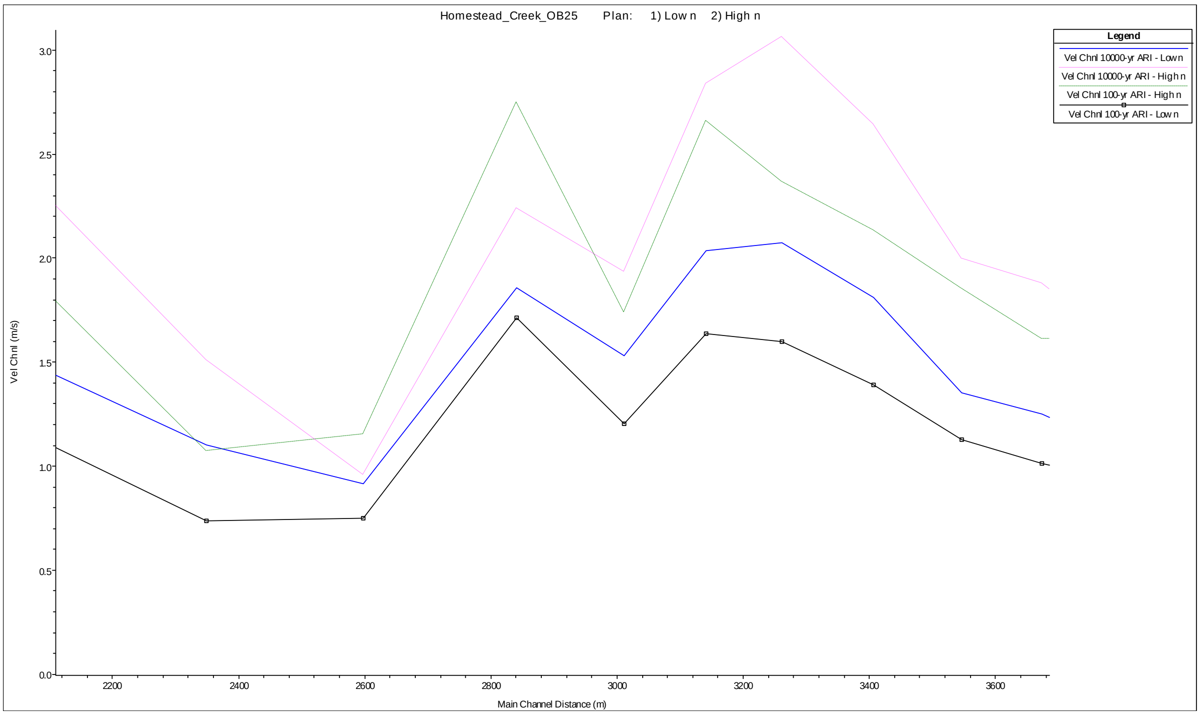


Figure C-24. Homestead Creek velocity profile for high and low roughness conditions along Pit 3

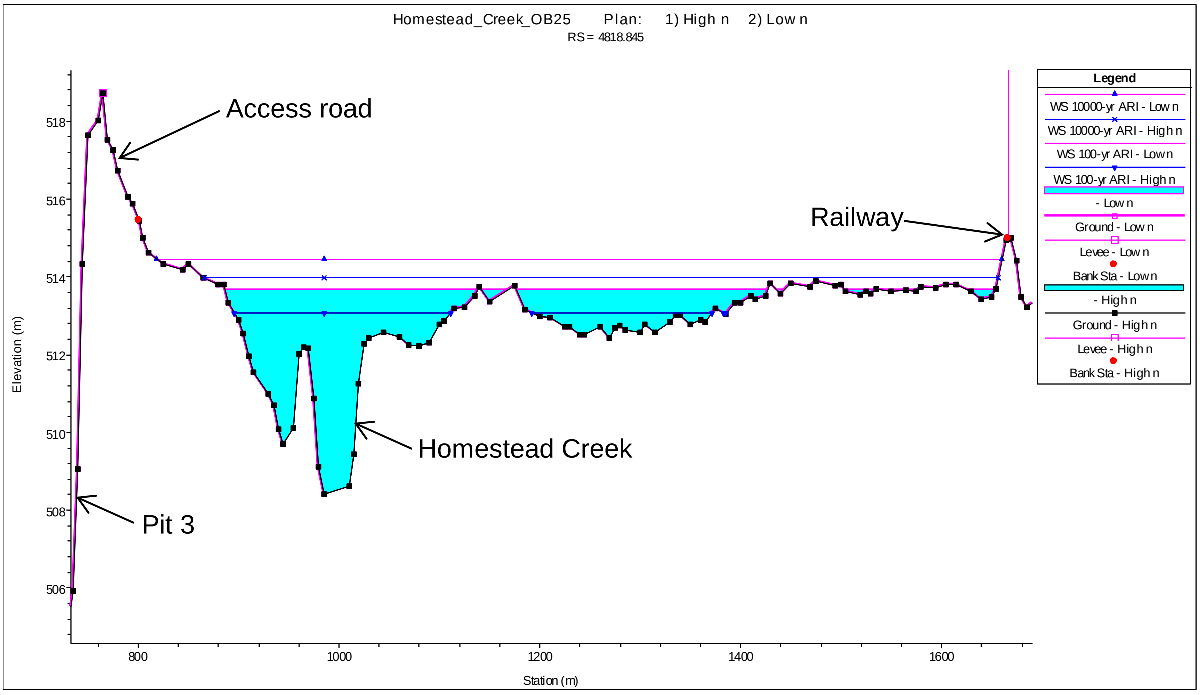


Figure C-25. Cross sectional view of water surface elevations for high and low roughness

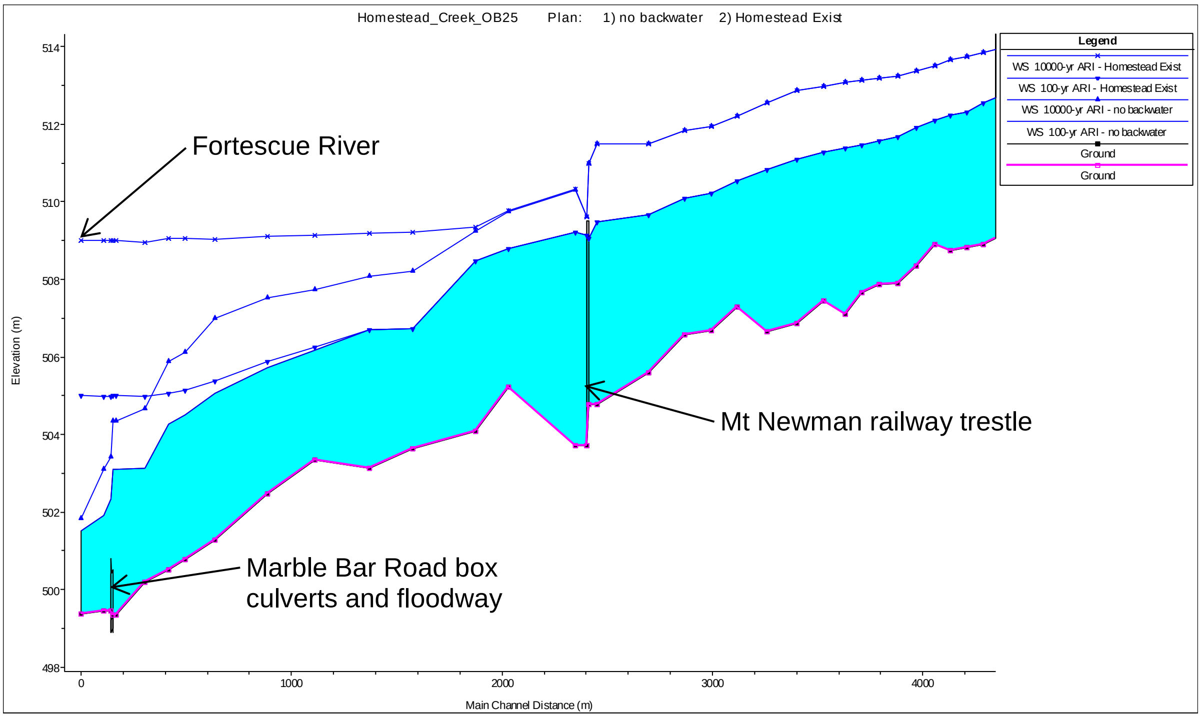


Figure C-26. Water surface profile showing backwater effect from downstream boundary condition

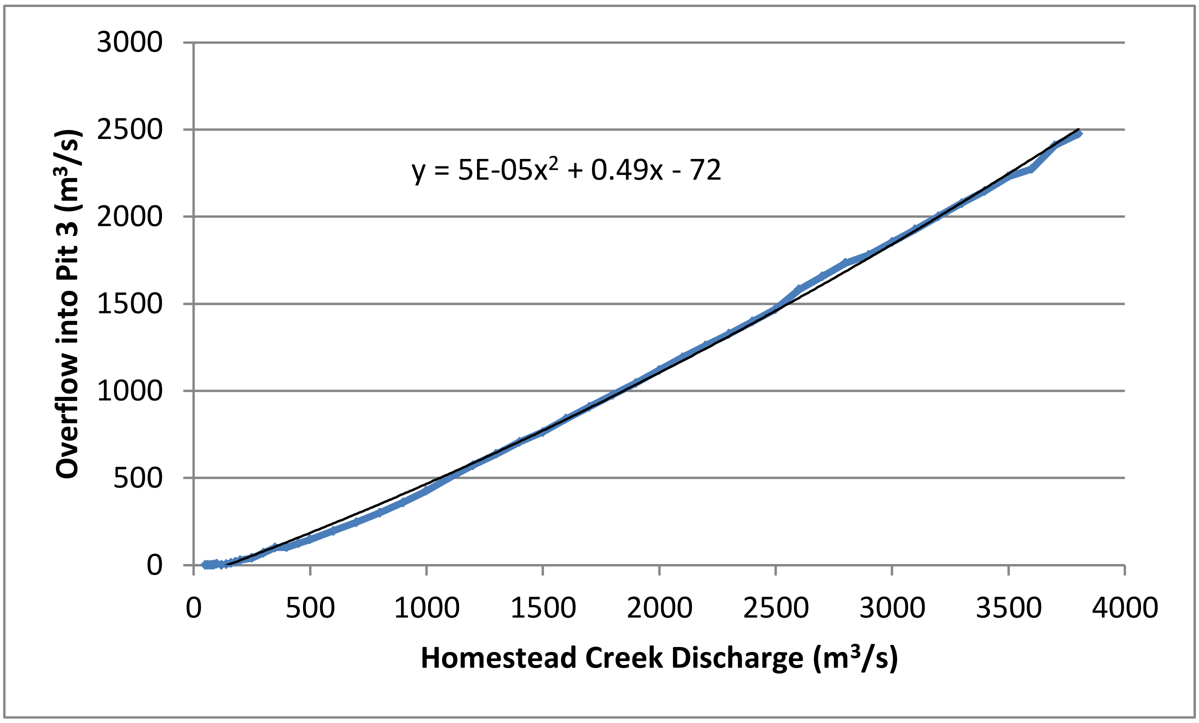


Figure C-27. Rating curve for lateral weir flow into Pit 3 vs. Homestead Creek main channel flow

APPENDIX D: GEOTECHNICAL

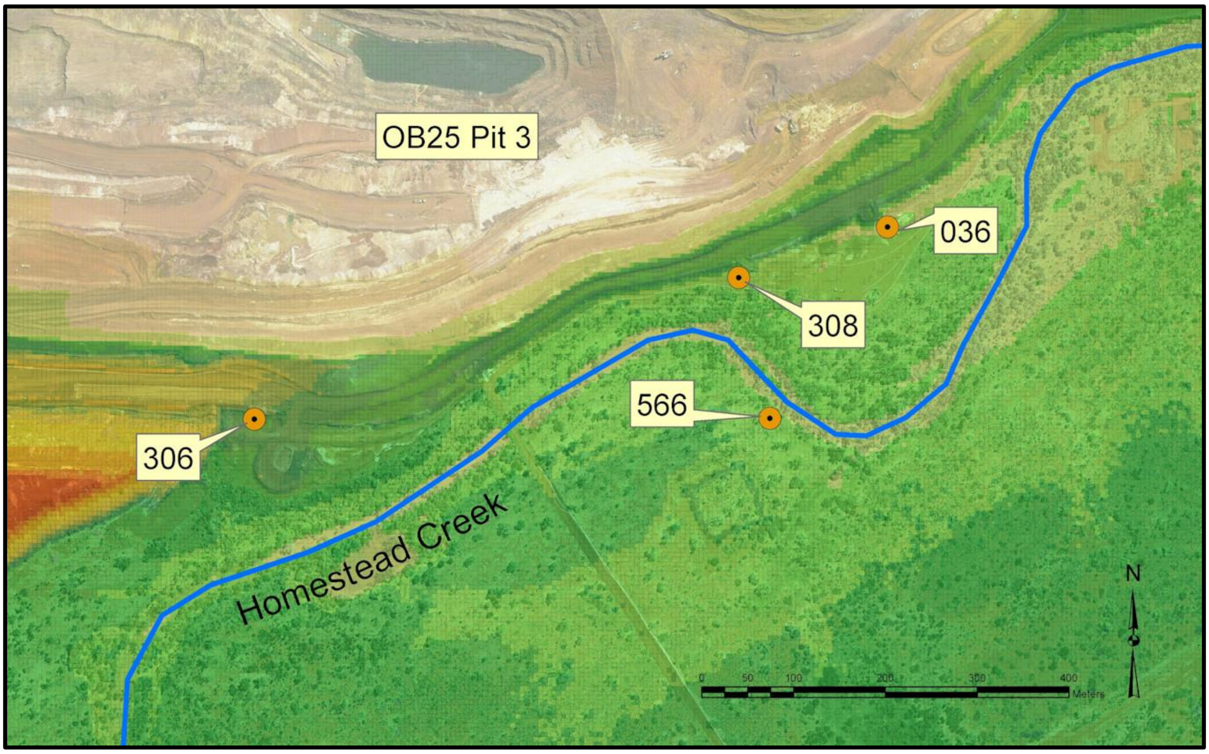


Figure D-1. Hydrogeological Borehole Locations in the vicinity of OB25 Pit 3

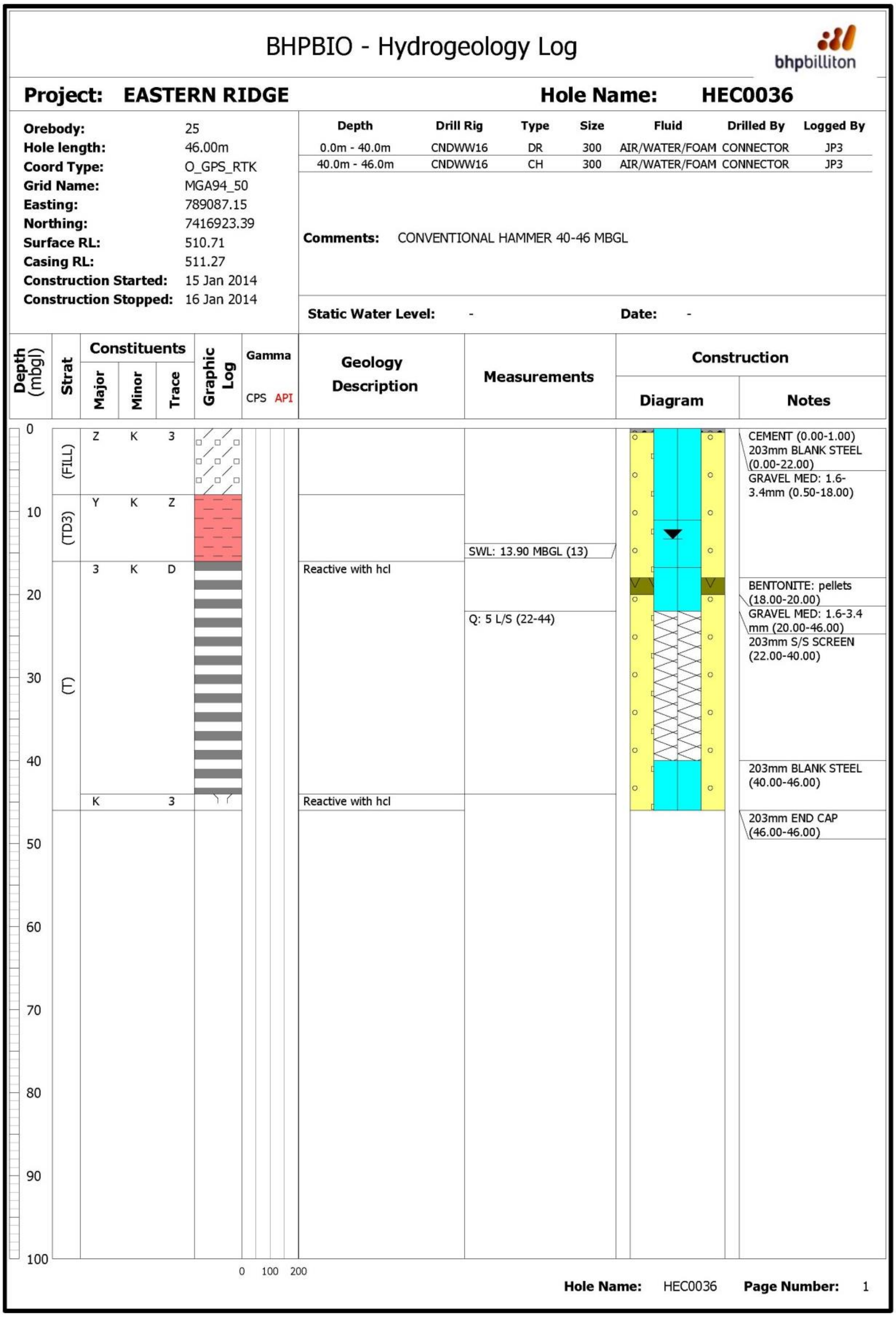


Figure D-2. Hydrogeology Log for HEC0036

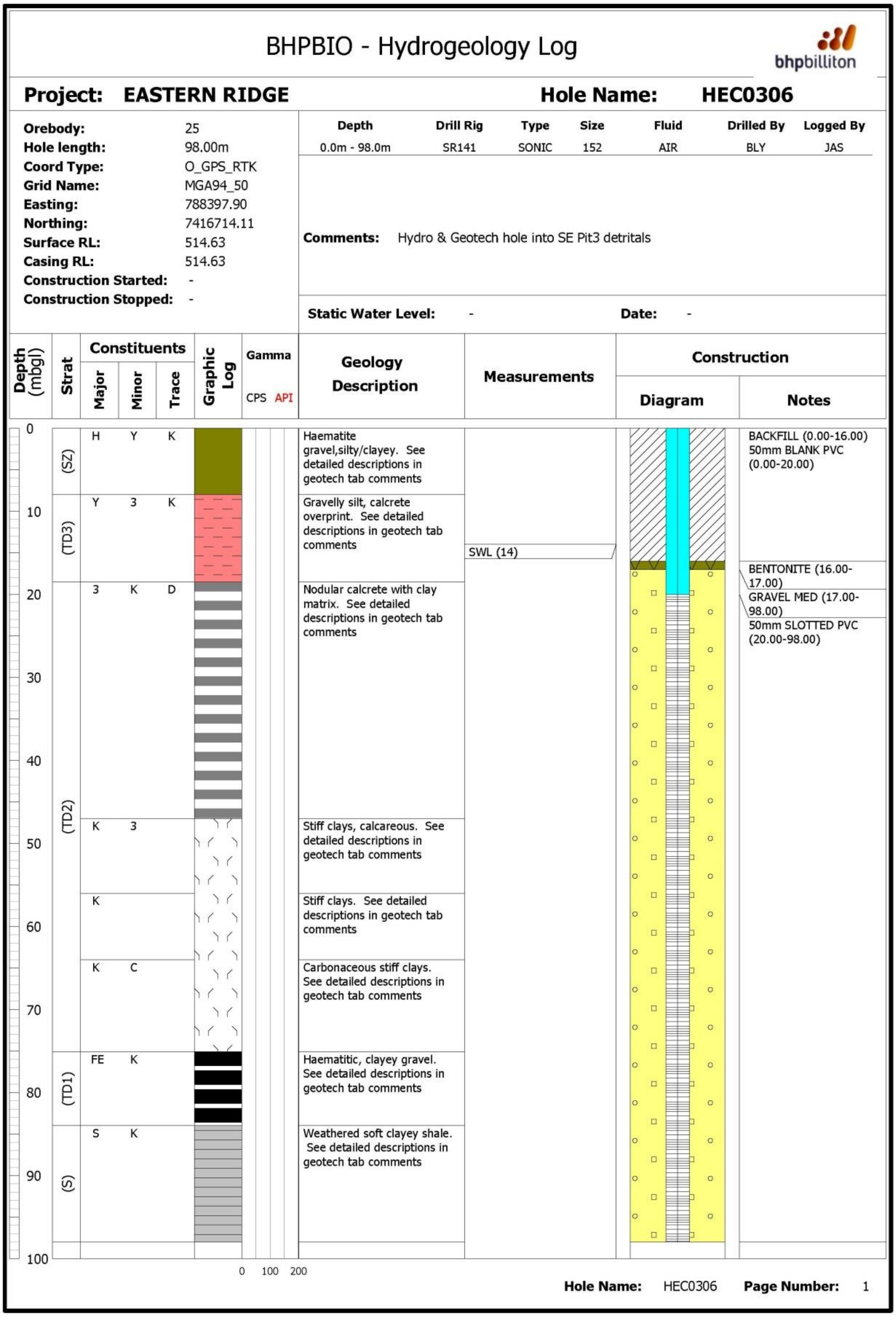


Figure D-3. Hydrogeology Log for HEC0306

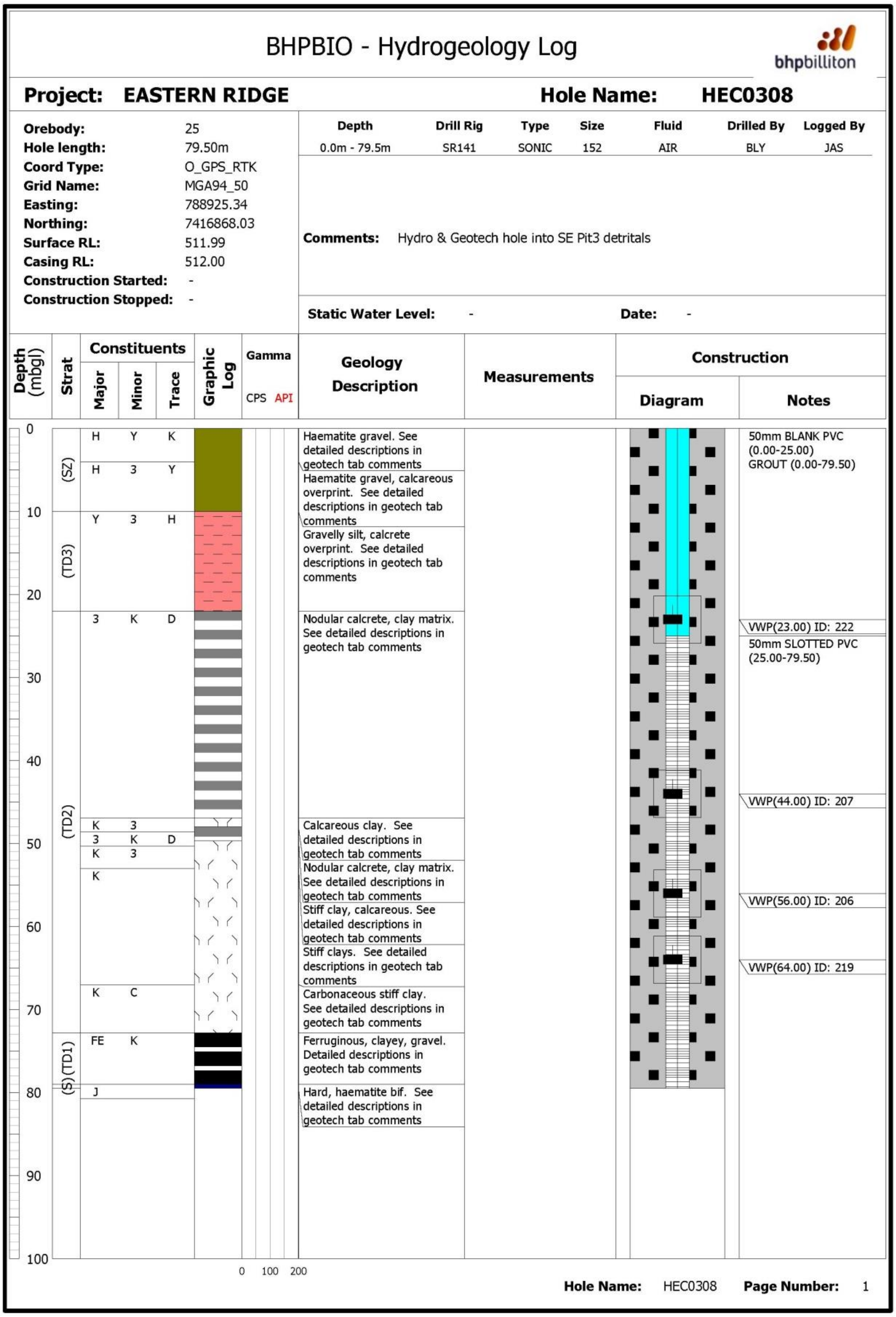


Figure D-4. Hydrogeology Log for HEC0308

BHPBIO - Hydrogeology Log

bhpbilliton

Project: EASTERN RIDGE **Hole Name: HEOP0566**

Orebody:	37
Hole length:	25.00m
Coord Type:	SURV
Grid Name:	MGA94_50
Easting:	788959.13
Northing:	7416714.95
Surface RL:	512.15
Casing RL:	-
Construction Started:	-
Construction Stopped:	-

Depth	Drill Rig	Type	Size	Fluid	Drilled By	Logged By
0.0m - 25.0m	WEST	UNKN	127		SWICK	AGC

Comments: HOLE ABANDONED Edit: Holename renamed from W0254. Authorised by Matthew Seeber on 17May2013.

Static Water Level: - **Date:** -

Strat	Constituents: Major	Minor	Trace
(UN)	K	Y0	Z
	Y	Z	
	Z0		
	Z	0Y	
	Z	0Y	
	0Y	3K	
	Z0Y	3K	
	3Y	K	
	3Y	KZ	
	ZYK	3	
	Z	K	3
	3	Z	
	3	Z	
	3		5
	3	5	
	3K	5	
	3	Z5	
	3	Z5	
	3	5	
	3	5	
	3	5	
	3K		
	3K		
	3K	Z	
	3K	Z	

Depth (mbgl): 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100

Graphic Log | Gamma (CPS API): 0 100 200 | Geology Description | Measurements | Construction: Diagram, Notes

Hole Name: HEOP0566 **Page Number:** 1

Figure D-5. Hydrogeology Log for HEOP0566

APPENDIX E: EROSION AND SEDIMENTATION

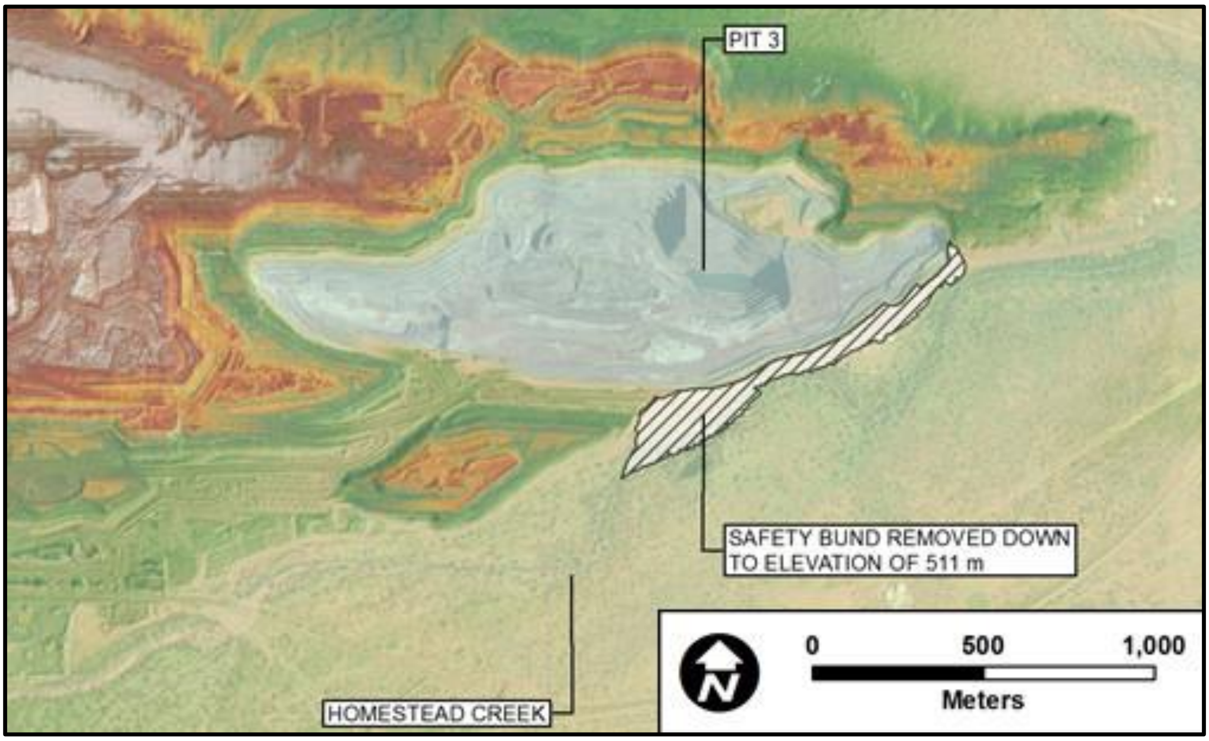


Figure E-1. Extent of bund removal for ultimate conditions model

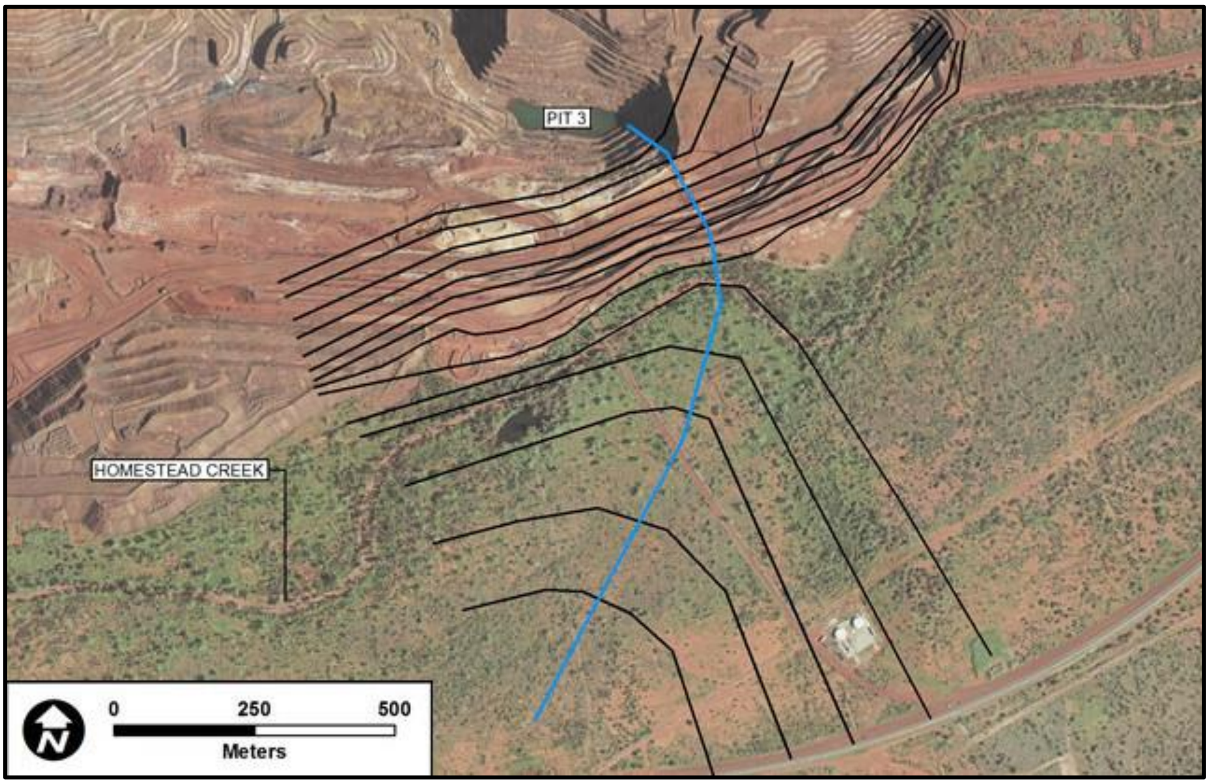
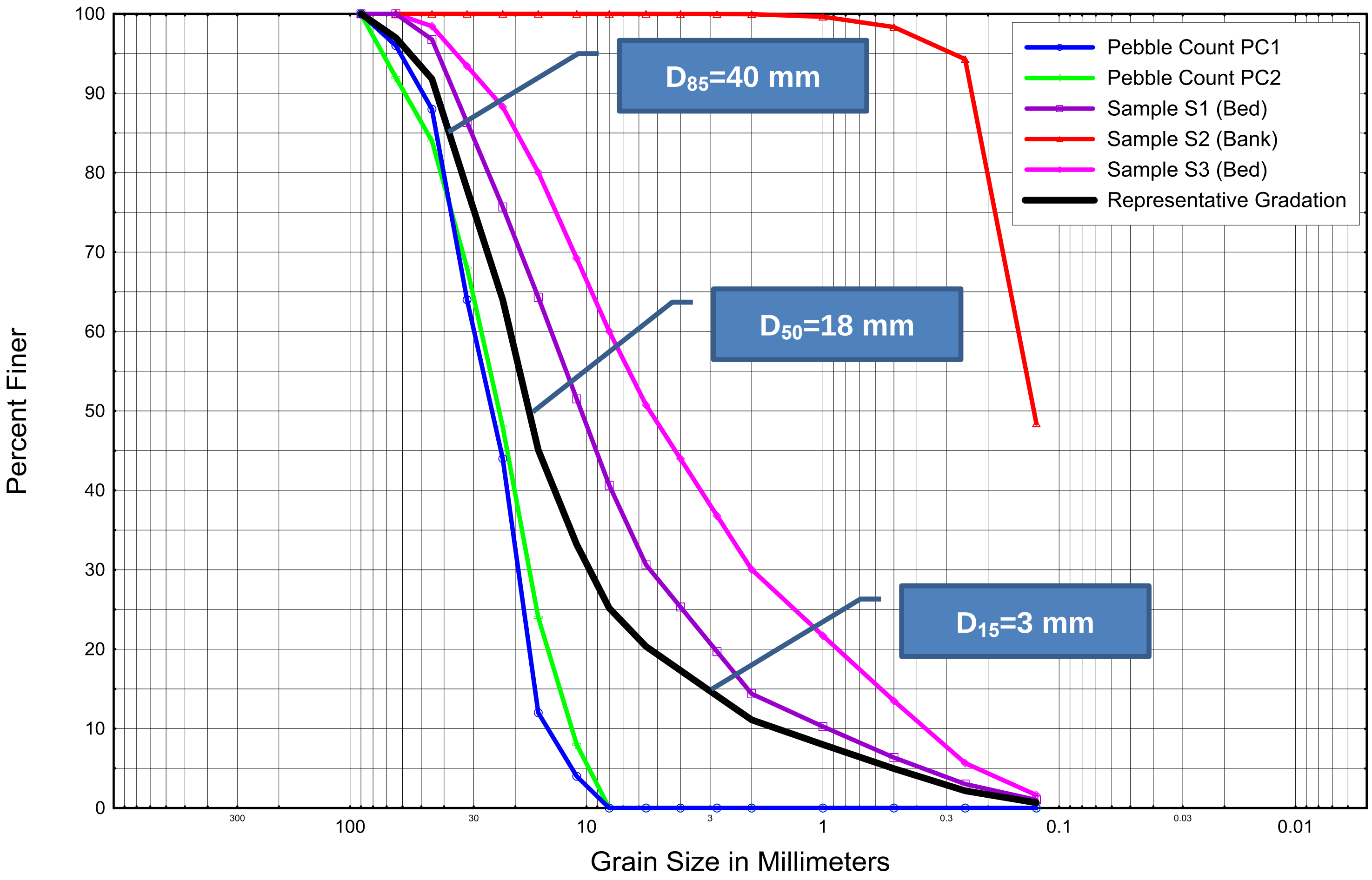


Figure E-2. Cross section locations for sediment transport/head cut model



BOULDERS	COBBLES	GRAVEL					SAND					SILT or CLAY
		VC	C	M	F	VF	VC	C	M	F	VF	

Figure E-3. Gradation curves for substrate samples taken on 21 January 2014 site visit

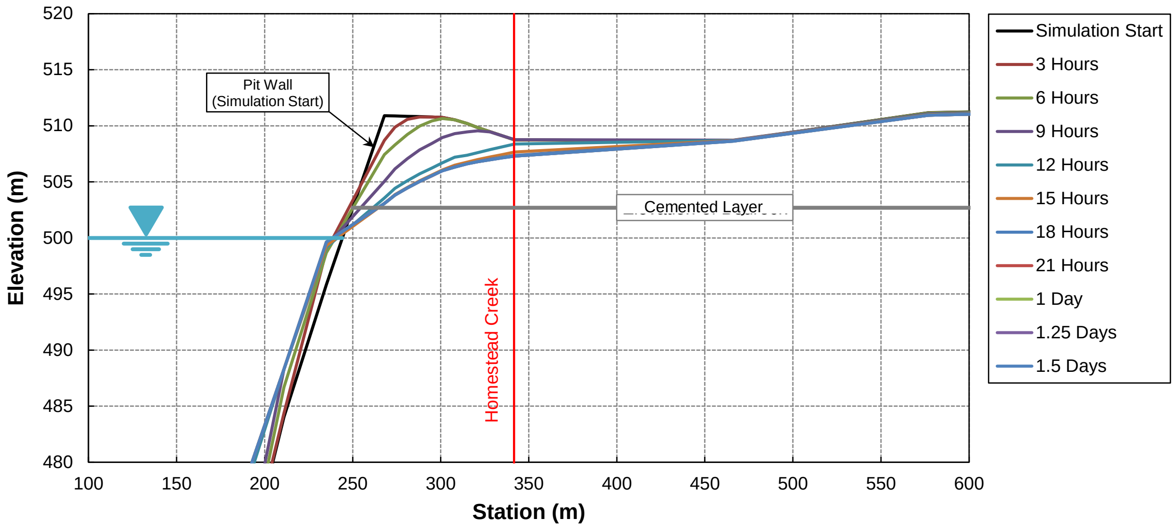


Figure E-4a. Predicted thalweg profile at various times during the simulation of the 100-year ARI flood event using a constant pit water-surface elevation of 500 metres.

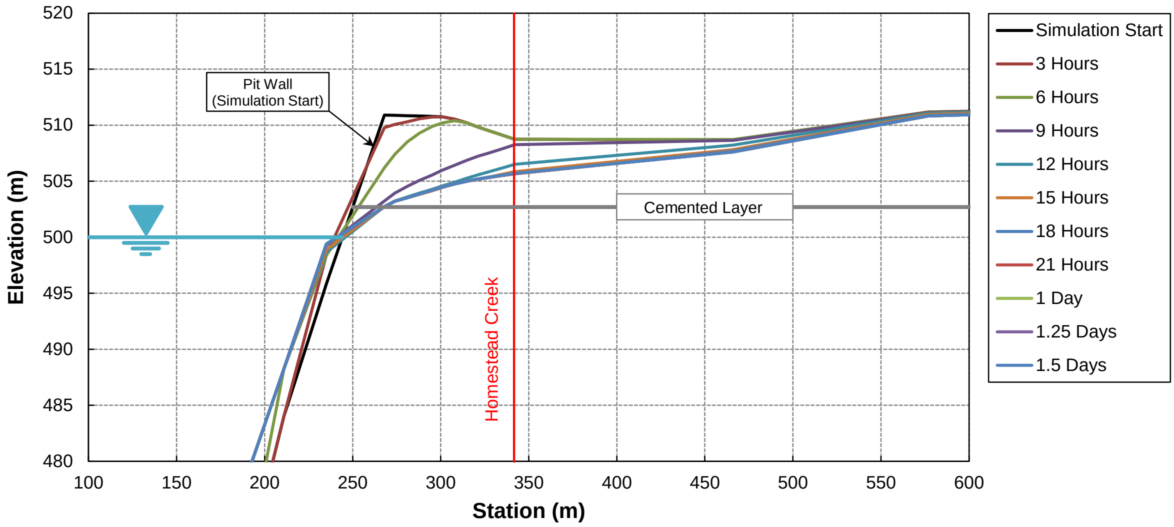


Figure E-4b. Predicted thalweg profile at various times during the simulation of the 500-year ARI flood event using a constant pit water-surface elevation of 500 metres.

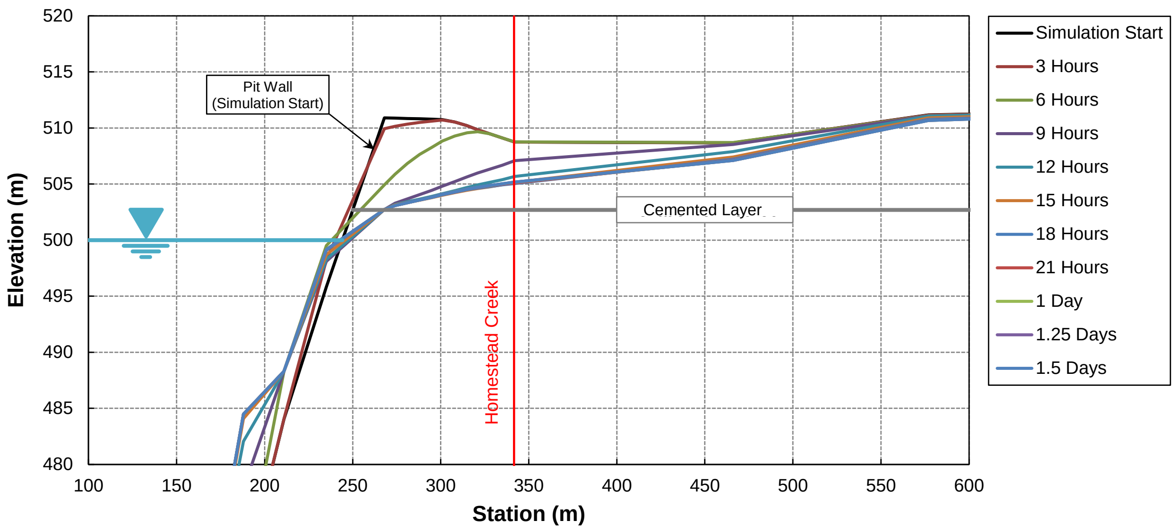


Figure E-4c. *Predicted thalweg profile at various times during the simulation of the 1,000-year ARI flood event using a constant pit water-surface elevation of 500 metres.*

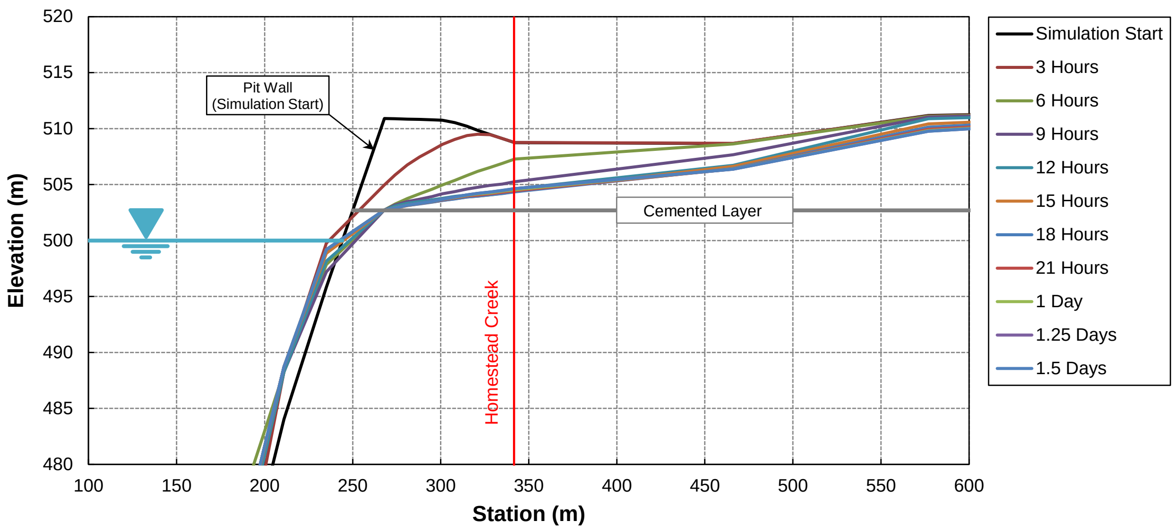


Figure E-4d. *Predicted thalweg profile at various times during the simulation of the 5,000-year ARI flood event using a constant pit water-surface elevation of 500 metres.*

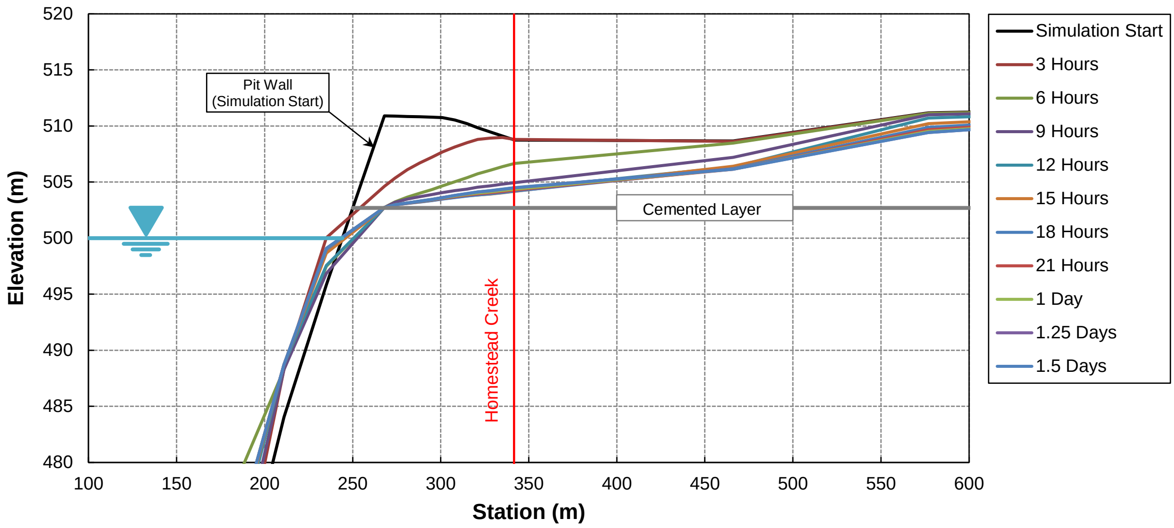


Figure E-4e. *Predicted thalweg profile at various times during the simulation of the 10,000-year ARI flood event using a constant pit water-surface elevation of 500 metres.*

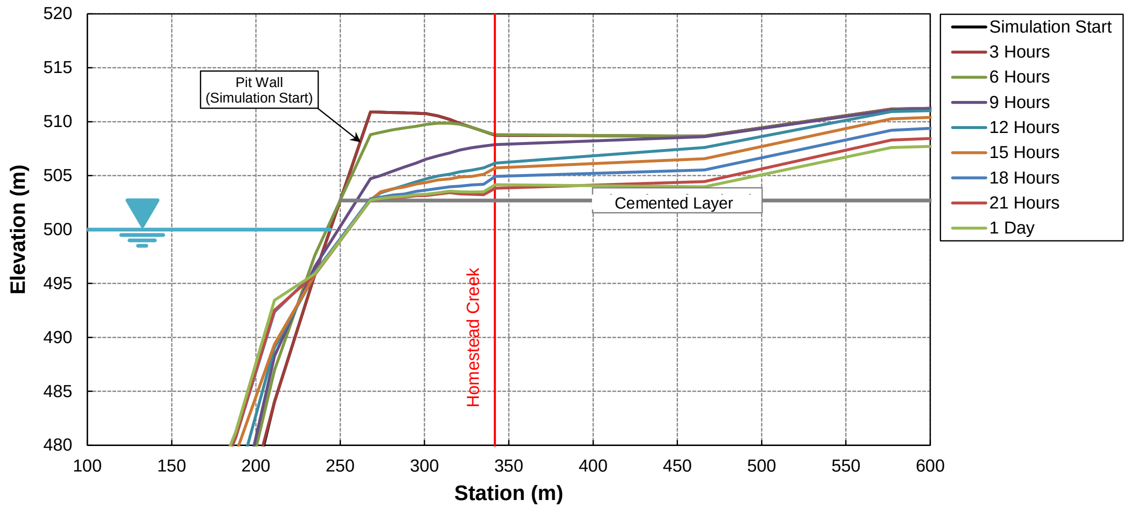


Figure E-4f. *Predicted thalweg profile at various times during the simulation of the Probable Maximum Flood event using a constant pit water-surface elevation of 500 metres.*

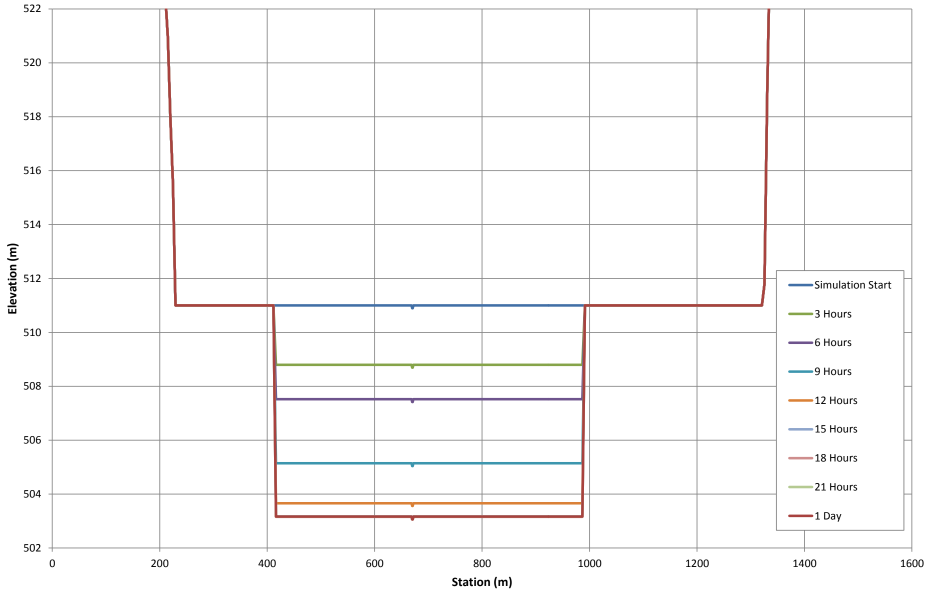


Figure E-5. Predicted cross-sectional geometry at various times during the simulation of the 100-year ARI event using a constant pit water-surface elevation of 500 metres at the pit wall (River Chainage 267.7).

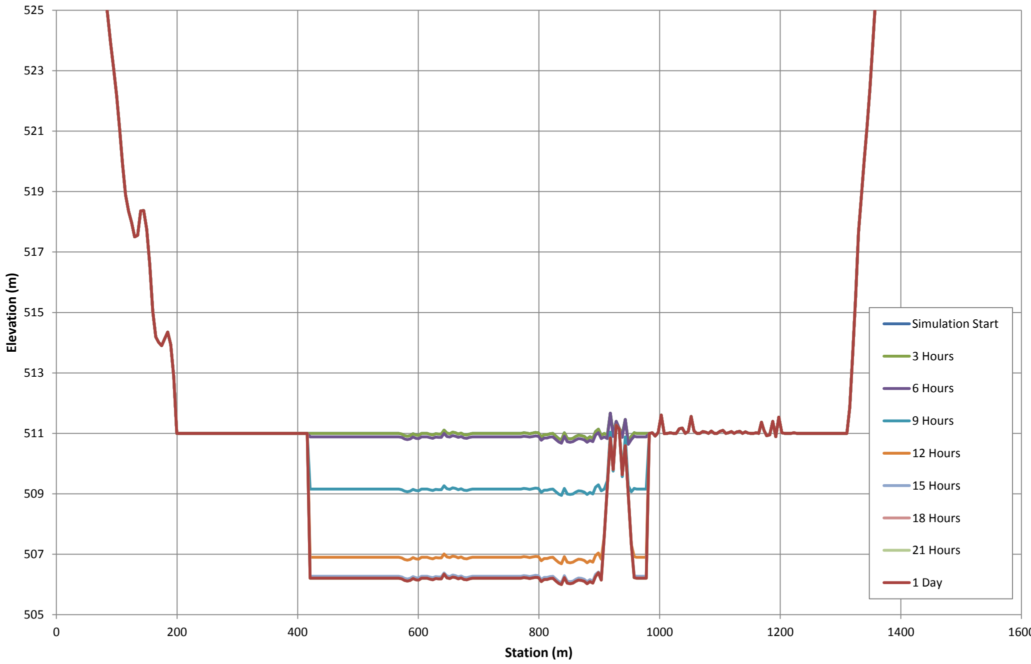


Figure E-6. Predicted cross-sectional geometry at various times during the simulation of the 100-year ARI event using a constant pit water-surface elevation of 500 metres midway between the pit wall and the point of capture (River Chainage 301.24).

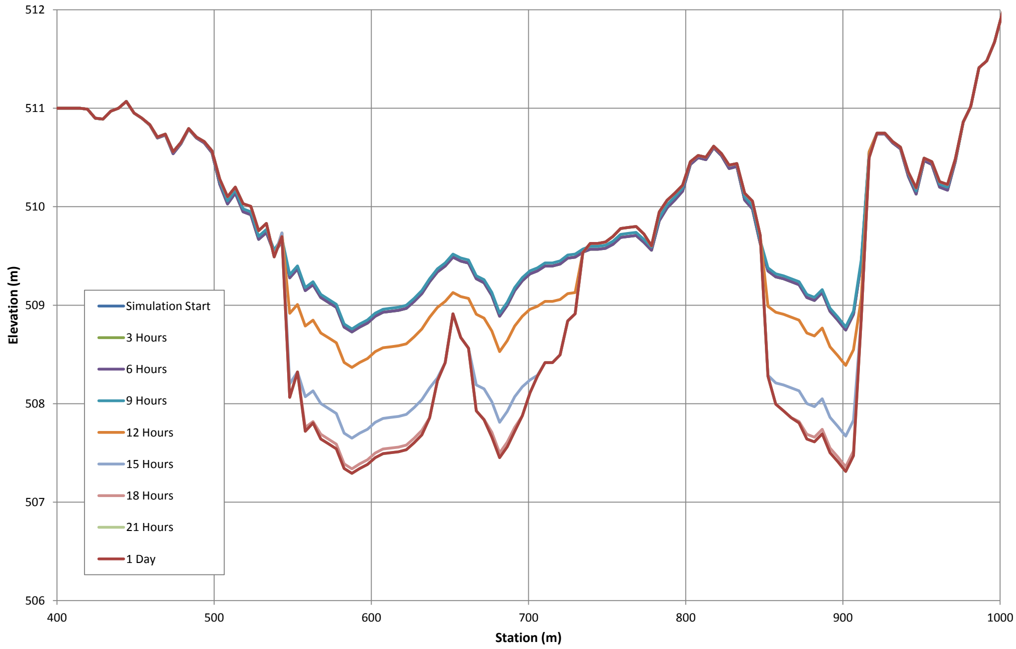


Figure E-7. *Predicted cross-sectional geometry at various times during the simulation of the 100-year ARI event using a constant pit water-surface elevation of 500 metres at the point of capture (River Chainage 341.74).*

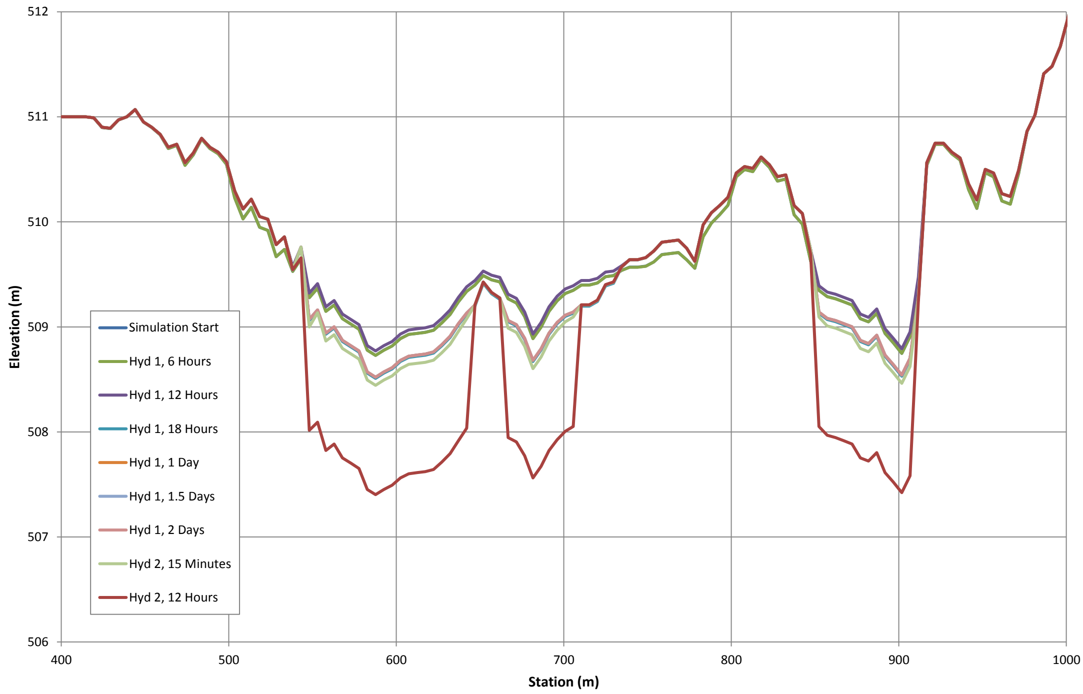


Figure E-8. *Predicted cross-sectional geometry at various times during the simulation of the 100-year ARI event using a varying pit water-surface elevation at the point of capture (River Chainage 341.74).*

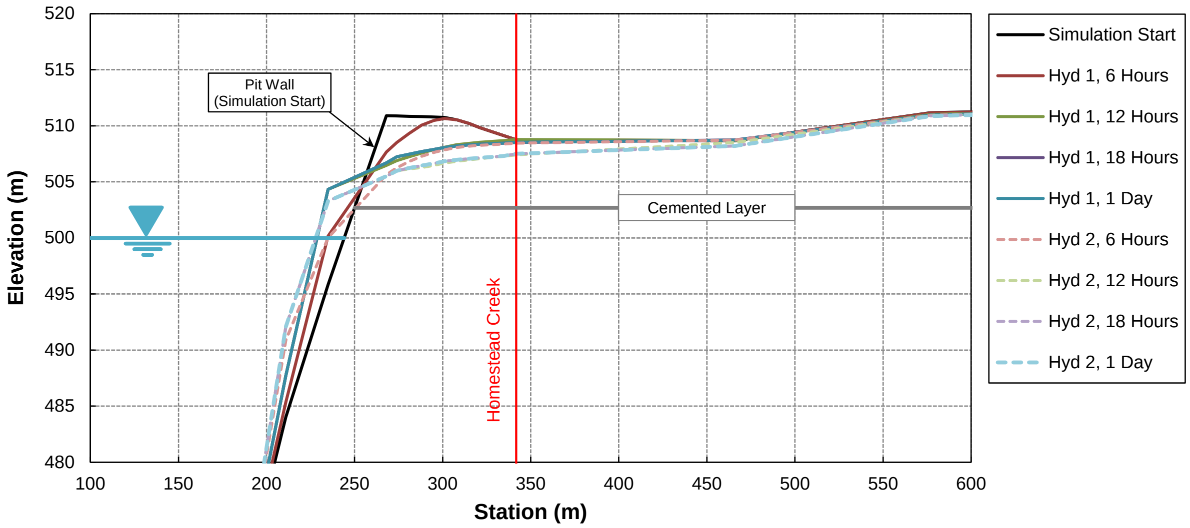


Figure E-9. Predicted thalweg profile at various times during the simulation of the 100-year ARI flood event using a varying pit water-surface elevation.

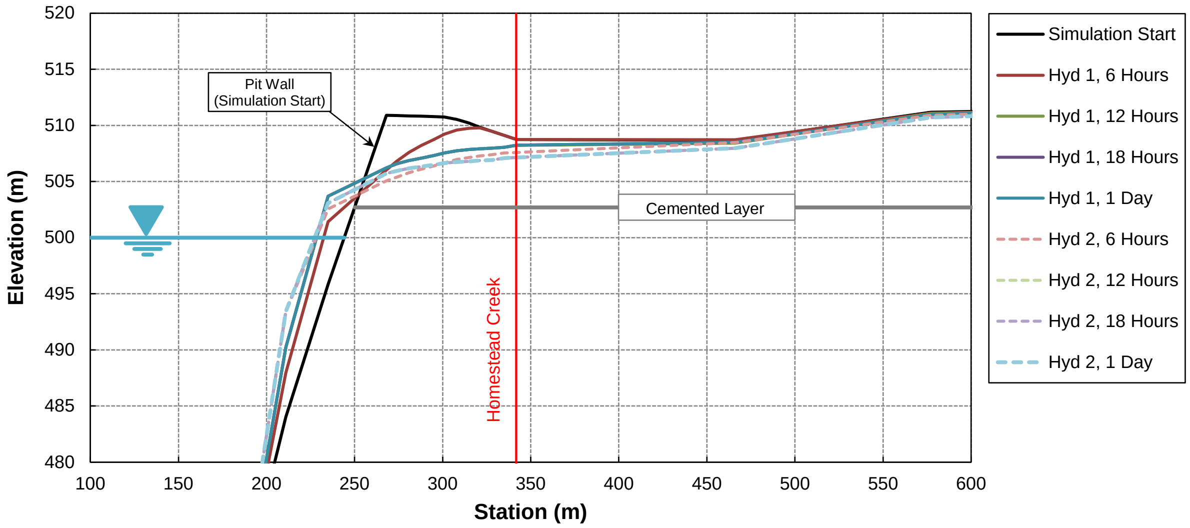


Figure E-10. Predicted thalweg profile at various times during the simulation of the 1000-year ARI flood event using a varying pit water-surface elevation.

APPENDIX F: CLOSURE OPTIONS AND CONCERNS

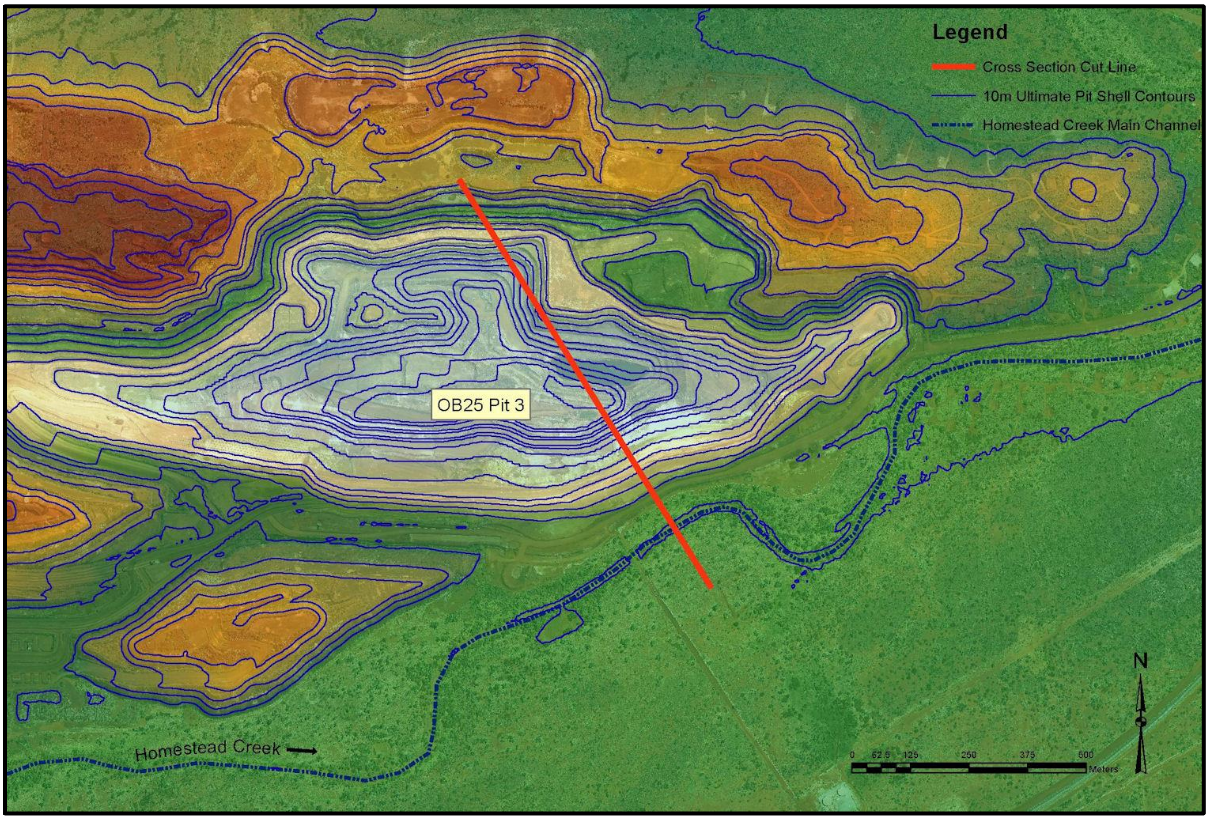


Figure F-1. Cross Section Cut Location for Figure F-5

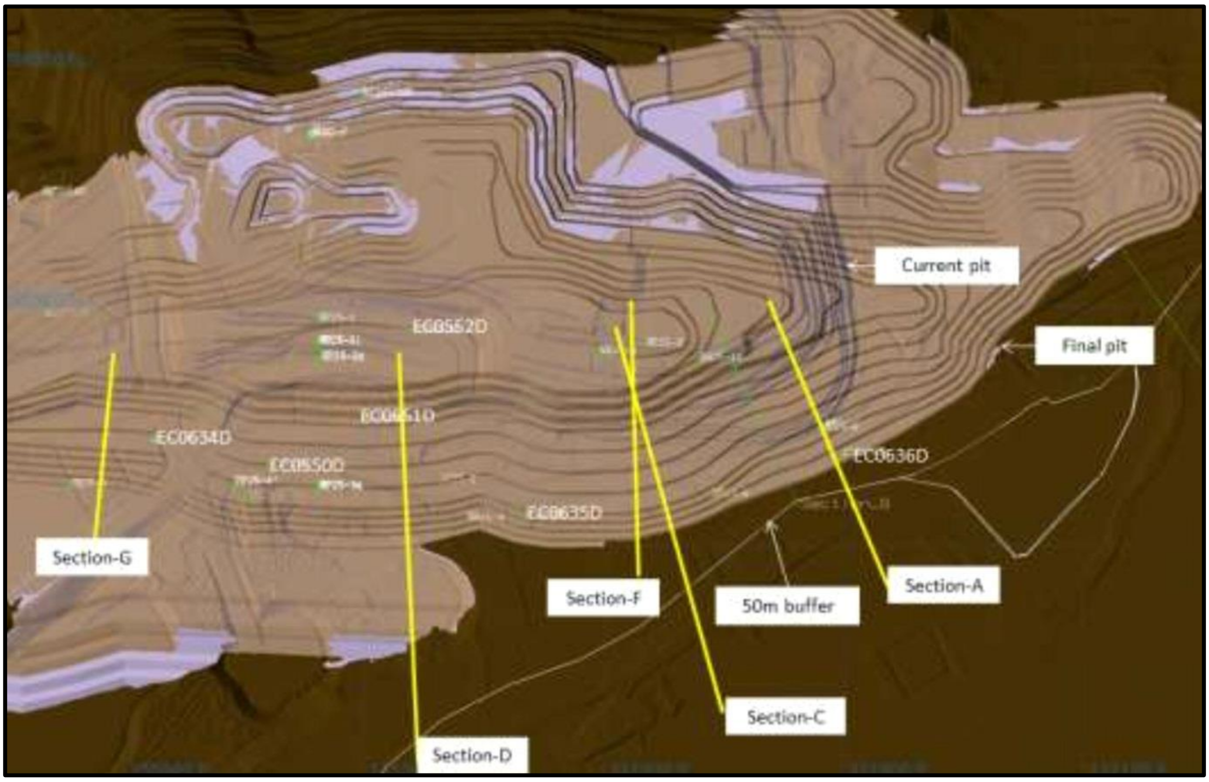


Figure F-2. Snowden Geotechnical Cross Section Cut Locations (2012)

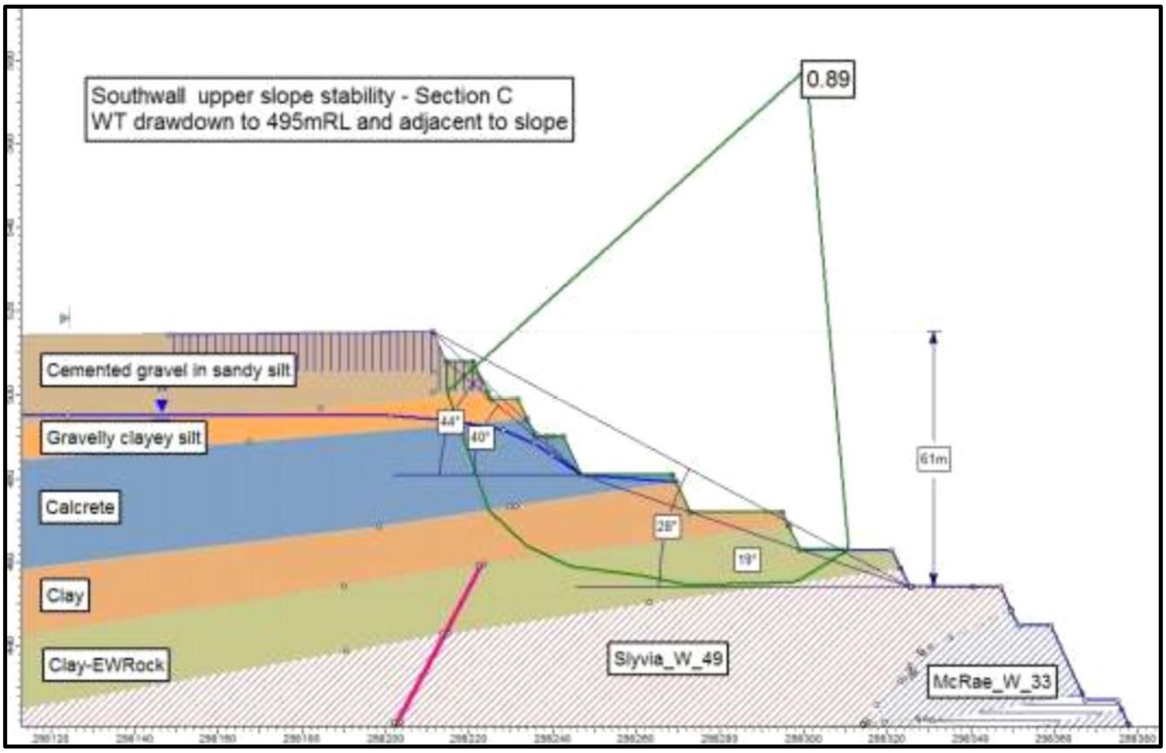


Figure F-3. Geotechnical Slope Stability Section C (Snowden 2012)

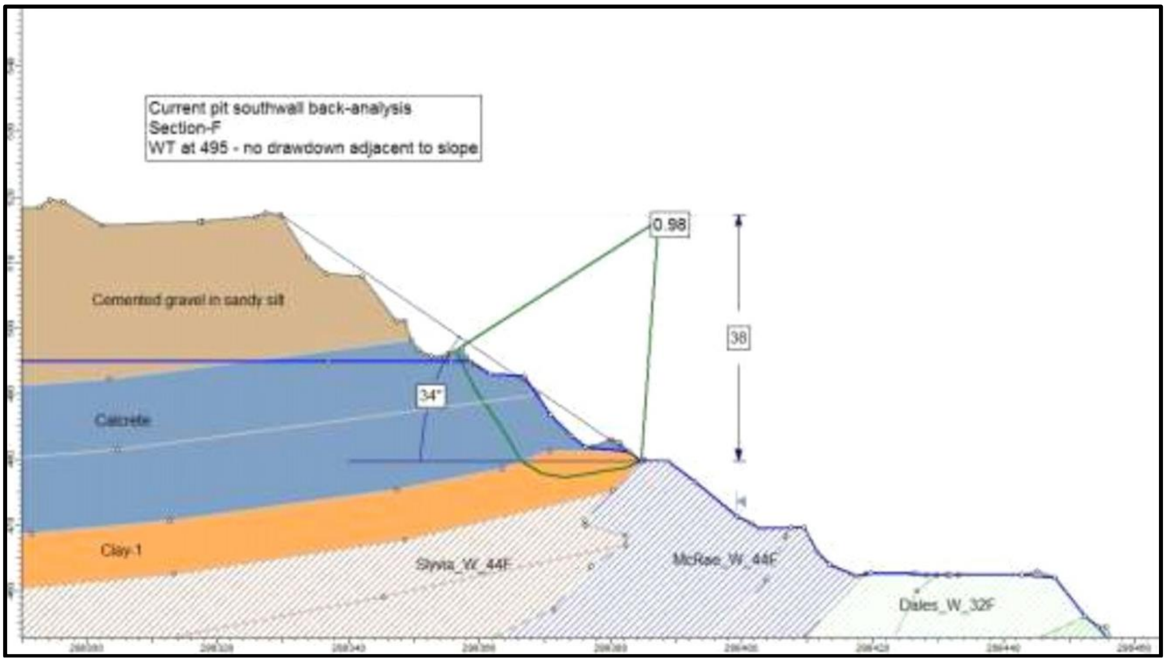


Figure F-4. Geotechnical Slope Stability Section F (Snowden 2012)

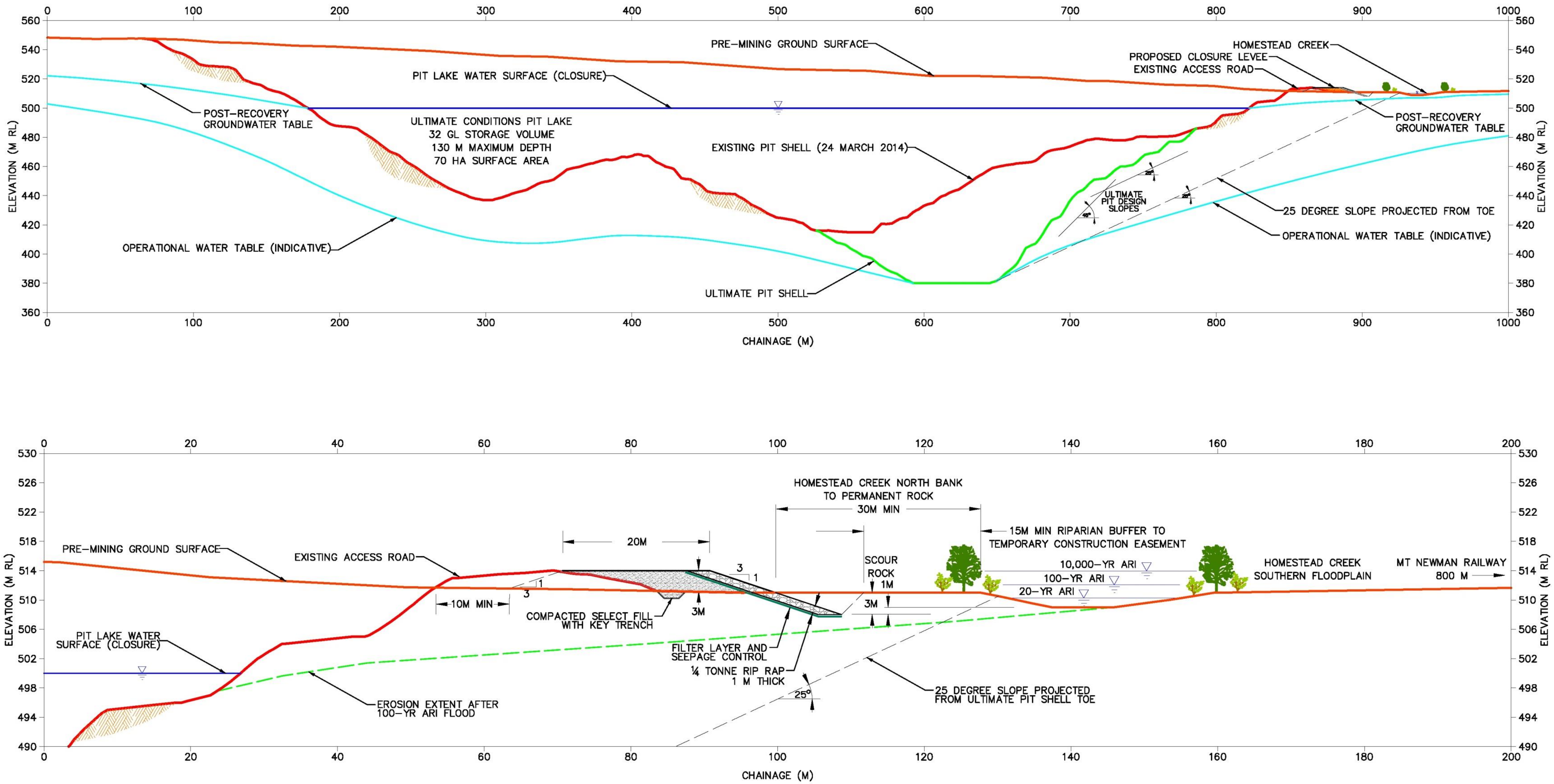


Figure F-5. Pit 3 and Levee Sections

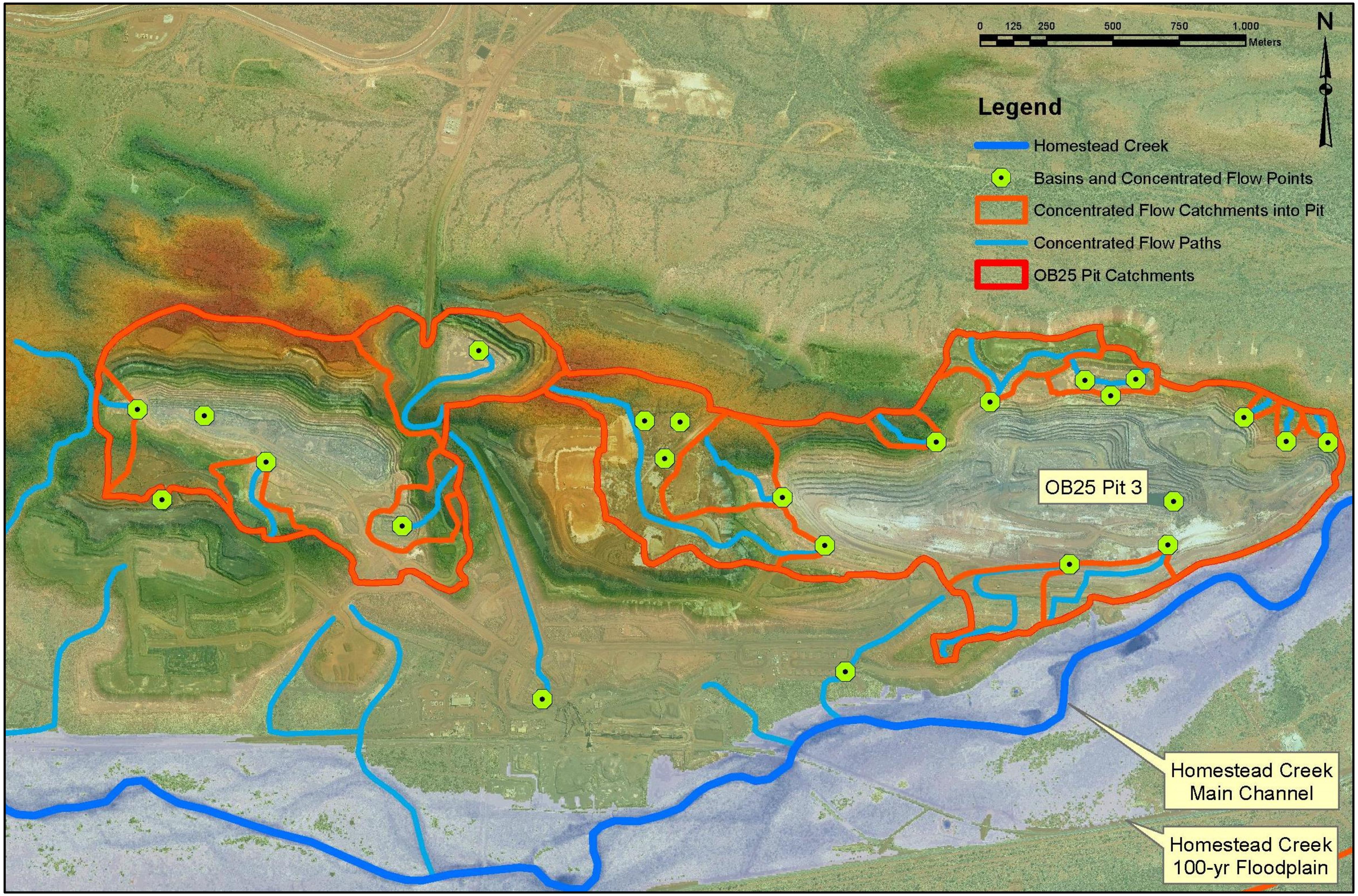


Figure F-6. Concentrated flow paths impacting pits and mine infrastructure

APPENDIX G: GEOSPATIAL METADATA

All CAD and GIS-based geospatial data used in project mapping are projected to the Australian National Grid GDA 1994 MGA Zone 50 Projection. The following files provided by BHPBIO were used in the modelling efforts for this project.

1.
File name: "HomesteadWhalebackCreek.tif"
Description: Geotiff grid file 5 m DEM, MGA Zone 50, AHD
Coverage Area: Entire Homestead Creek and Whaleback Creek catchments
Survey/Flight date: 2012 unknown date
Received: 3 Dec 2013 via ftp transfer from Matt Pomery

2.
File name: "HomesteadCreek_June2013_MGA50_AHD.tif"
Description: Geotiff grid file 5 m DEM, MGA Zone 50, AHD
Coverage Area: Entire Homestead Creek and Whaleback Creek catchments
Survey/Flight date: June 2013? (from file name)
Received: 27 Mar 2014 via USB transfer from Iain Rea

3.
File name: "OB25p3_rpdc.dxf"
Description: dxf break lines, MGA Zone 50, ADPH
Coverage Area: Pit 3
Survey/Flight date: Final pit shell (closure conditions)
Received: 27 Mar 2014 via e-mail from Kristen Risnes

4.
File name: "eph_2014_0101_ob25p3_surv_topo.dxf"
Description: dxf 1-m contour lines, ER94 Project Grid, ADPH
Coverage Area: Pit 3
Survey/Flight date: 1 Jan 2014? (from file name)
Received: via e-room 9 Apr 2013, file posted by Mike Bettison

5.
File name: "HomesteadWhaleback_MGA50_AHD_5m_20140324_32bit.tif"
Description: Geotiff grid file 5 m DEM, MGA Zone 50, AHD
Coverage Area: Entire Homestead Creek and Whaleback Creek catchments
Survey/Flight date: 24 March 2014
Received: 28 April 2014 via ftp transfer from Matt Pomery

6.
File name: "EasternRidge_r015_20140324_ER94.ecw"
Description: Georeferenced orthorectified colour aerial photograph
Coverage Area: Eastern Ridge project area
Survey/Flight date: 24 March 2014
Received: 28 April 2014 via ftp transfer from Matt Pomery

Following are the base mapping files used for the individual components of this study:

Hydrology:
Overall catchment and subcatchment boundary delineations were completed using a 1-metre contour interval shape file generated from the DEM in File #1. Whilst the topographic surface is outdated in the immediate vicinity of Pit3, the overall catchment delineations and RORB results are not significantly affected by updates to this file. Mine site catchment boundaries for OB23 and OB25 were delineated using 1-metre contour interval shape file generated from the DEM in File #5.

Existing Condition Hydraulics and Sediment Transport:
File #4 was shifted from the project grid to MGA Zone 50 and from ADPH to AHD (using the prj file data and vertical conversions listed below), then merged with File #2 to reflect the latest pit shell and flood bunding/access roads.

Closure Condition Hydraulics and Sediment Transport:
File #3 was shifted from ADPH to AHD (lowered by 3.155 m per) then merged with the surface generated for the existing condition hydraulics (combination of Files #2 and #4). The existing flood bund/access road was deleted from the surface to represent closure conditions with the bund assumed as failed/removed.

A 5-metre raster digital elevation model (DEM) of the existing conditions (24 March 2014) topography was developed and provided by BHPBIO, along with a separate 5-metre DEM of the topography in the vicinity of Pit 3 under closure conditions. The final closure conditions surface was prepared by merging the Pit 3 closure conditions raster file into the existing conditions raster image. Because the Pit 3 closure conditions raster file did not cover the area in the vicinity of the safety bund that would be removed at the time of closure, the elevations along the existing bund were manually adjusted down to an elevation of 511 metres (AHD) to represent the removed bund (Figure E-1). Finally, since the 24 March 2014 topography did not cover the entire extents of the Homestead Creek hydraulic model cross-sections along the very eastern edge of the surface, the raster image of the June 2013 topography was merged into the surface along this portion of the model domain.

Projection data:

PROJCS   Eastern Ridge Project Grid (ER94)

GEOGCS	GCS_GDA_1994
DATUM	D_GDA_1994
SPHEROID	GRS_1980",6378137.0,298.257222101
PRIMEM	Greenwich",0.0
UNIT	Degree",0.0174532925199433
PROJECTION	Transverse_Mercator
PARAMETER	False_Easting",446904.02
PARAMETER	False_Northing",2879827.84
PARAMETER	Central_Meridian",120.95
PARAMETER	Scale_Factor",0.9999
PARAMETER	Latitude_Of_Origin",0.0
UNIT	Meter",1.0

PROJCS	GDA_1994_MGA_Zone_50
GEOGCS	GCS_GDA_1994
DATUM	D_GDA_1994
SPHEROID	GRS_1980",6378137.0,298.257222101
PRIMEM	Greenwich",0.0
UNIT	Degree",0.0174532925199433
PROJECTION	Transverse_Mercator
PARAMETER	False_Easting",500000.0
PARAMETER	False_Northing",10000000.0
PARAMETER	Central_Meridian",117.0
PARAMETER	Scale_Factor",0.9996
PARAMETER	Latitude_Of_Origin",0.0
UNIT	Meter",1.0