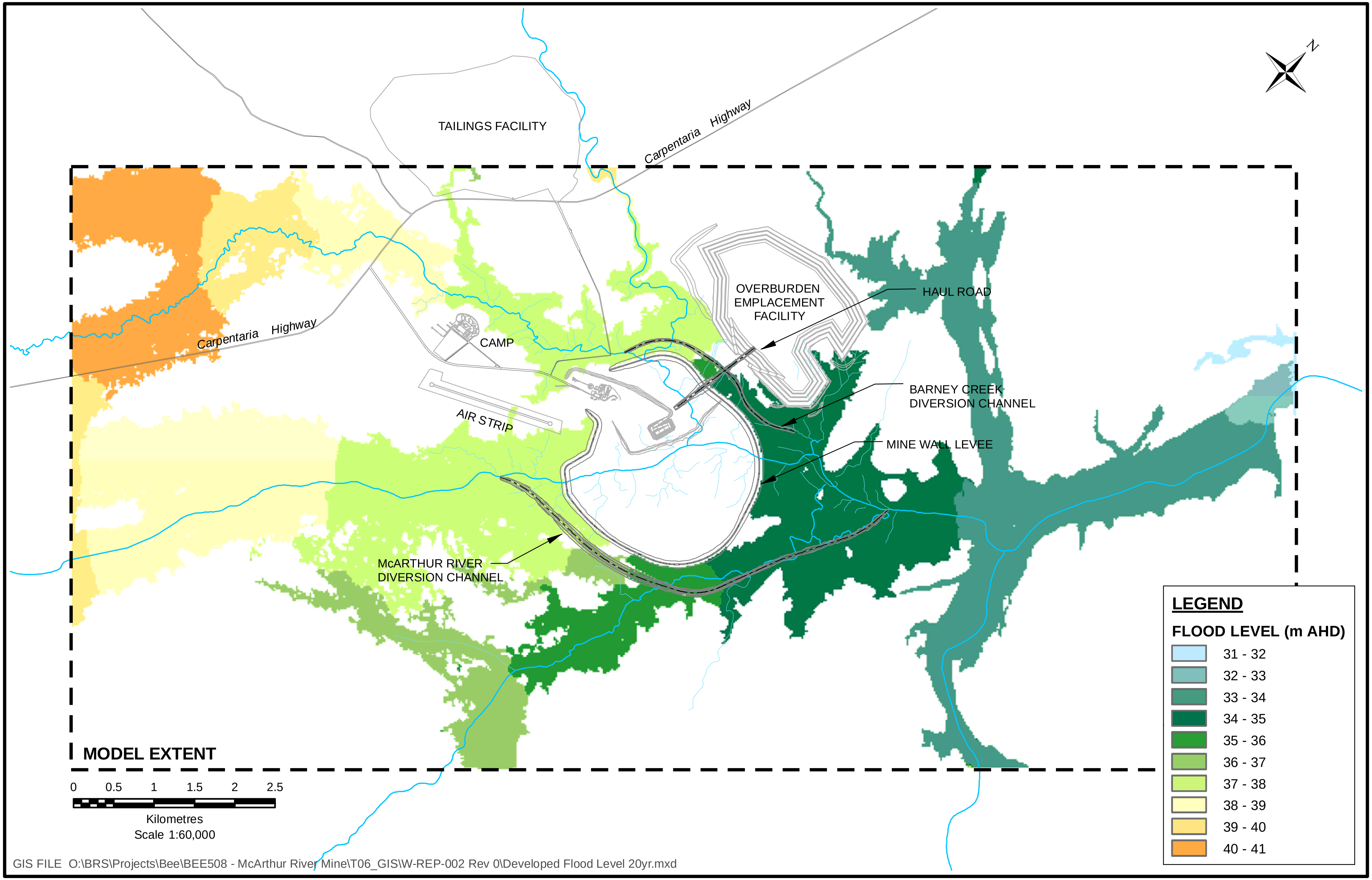

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Figure 4.12
McARTHUR RIVER 20 YEAR ARI EVENT
DEVELOPED CONDITIONS - PEAK FLOOD LEVEL

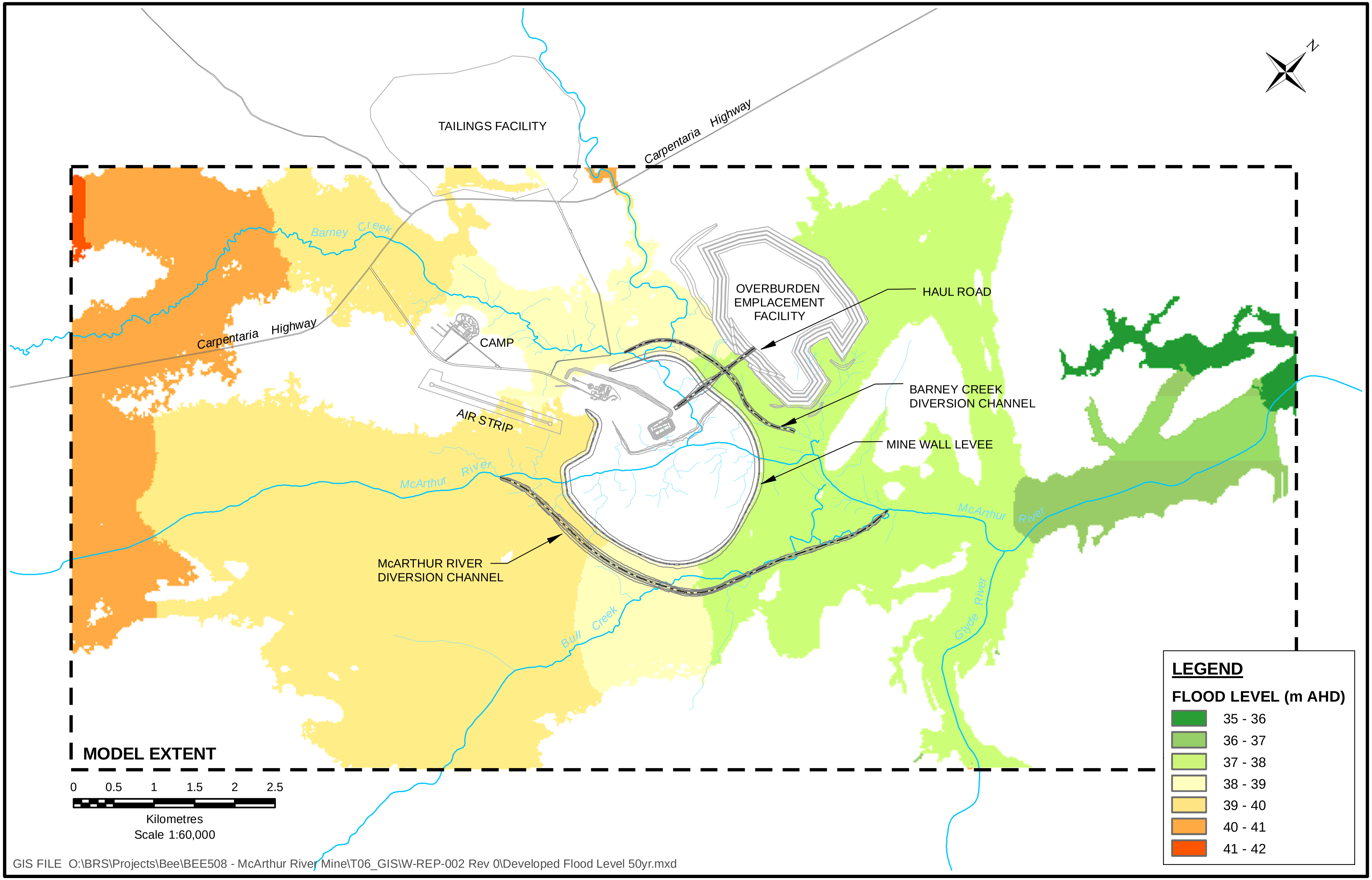

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Figure 4.13
McARTHUR RIVER 50 YEAR ARI EVENT
DEVELOPED CONDITIONS - PEAK FLOOD LEVEL

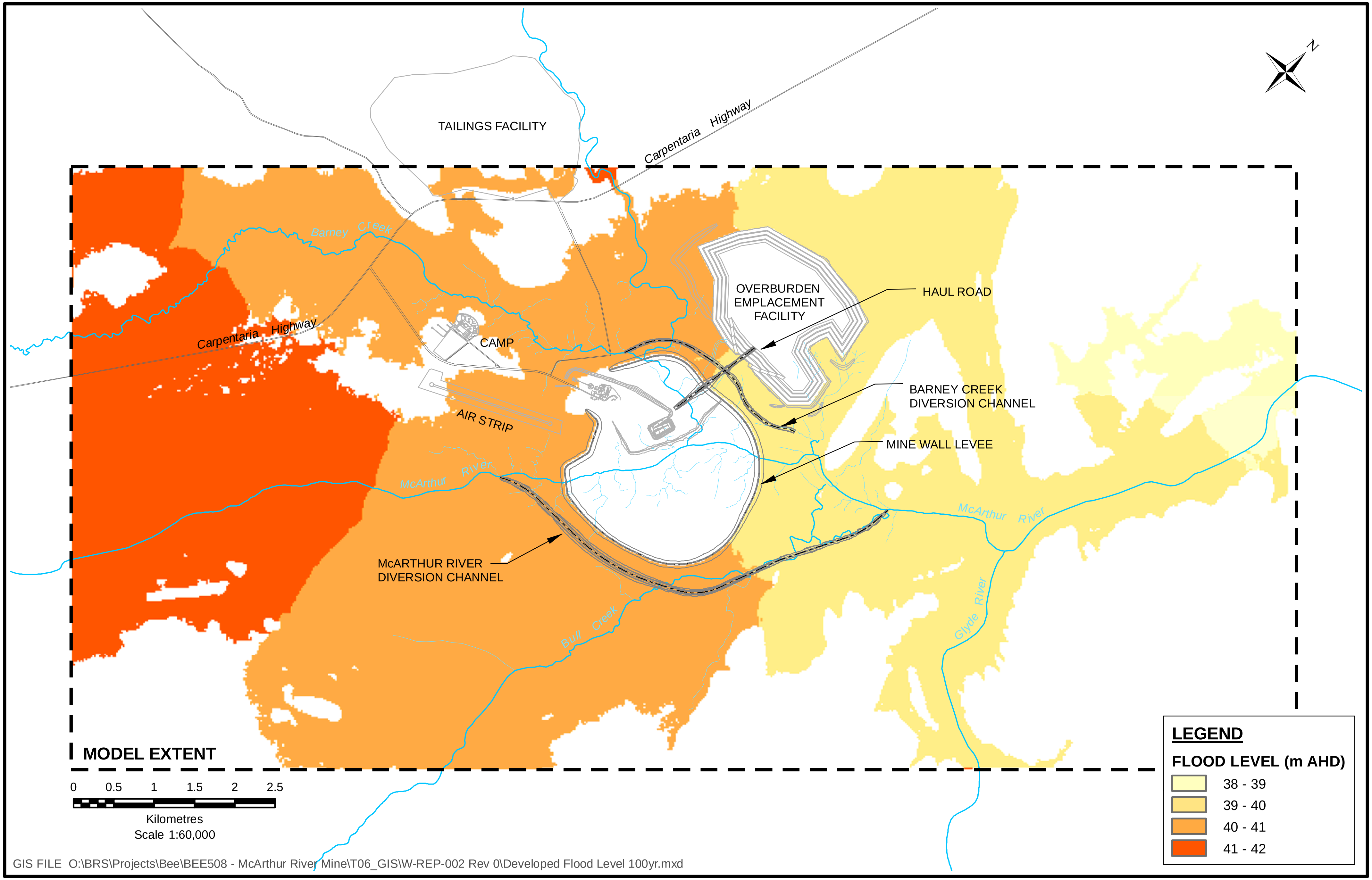

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Figure 4.14
McARTHUR RIVER 100 YEAR ARI EVENT
DEVELOPED CONDITIONS - PEAK FLOOD LEVEL

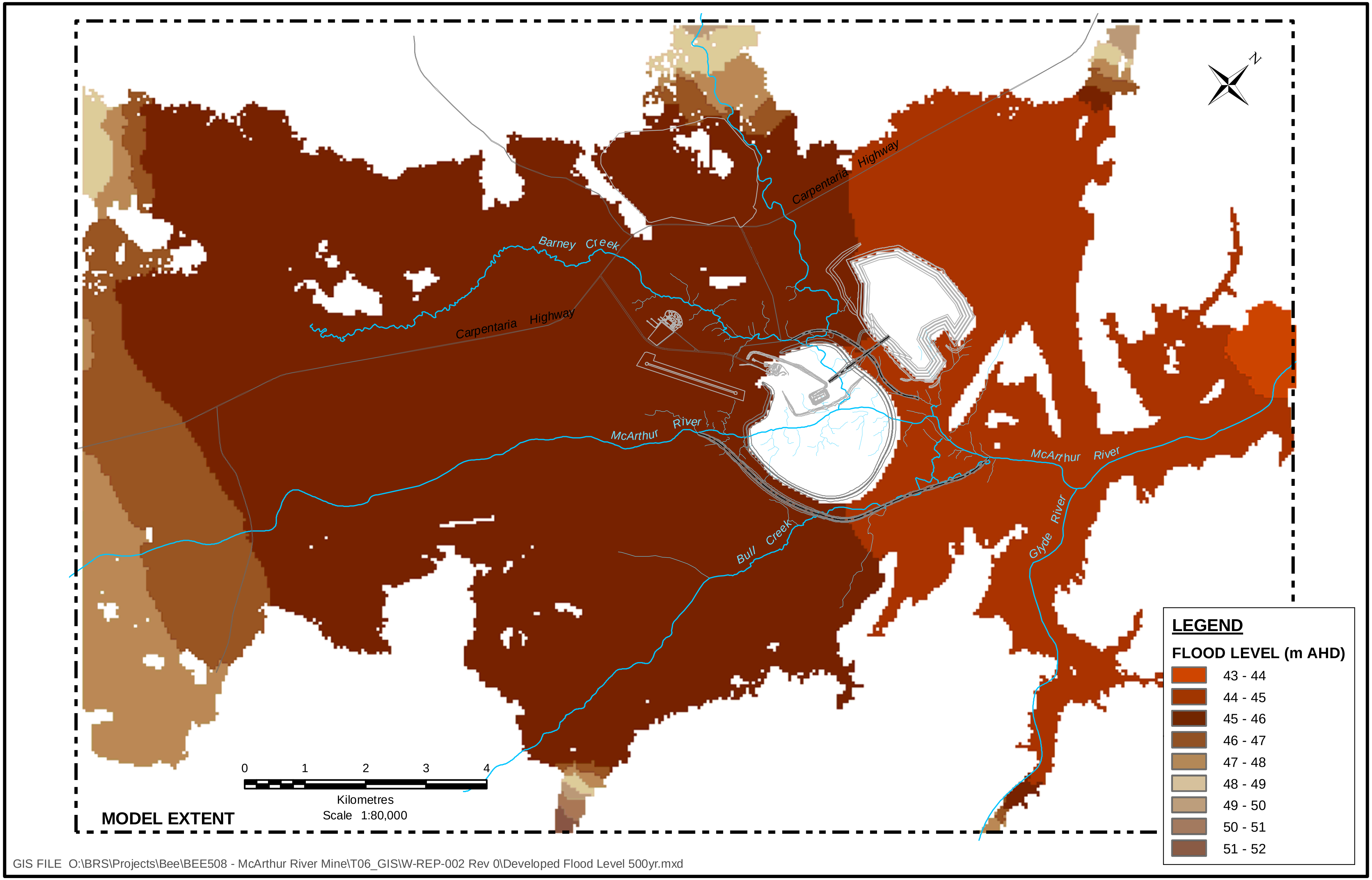

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Figure 4.15
McARTHUR RIVER 500 YEAR ARI EVENT
DEVELOPED CONDITIONS - PEAK FLOOD LEVEL

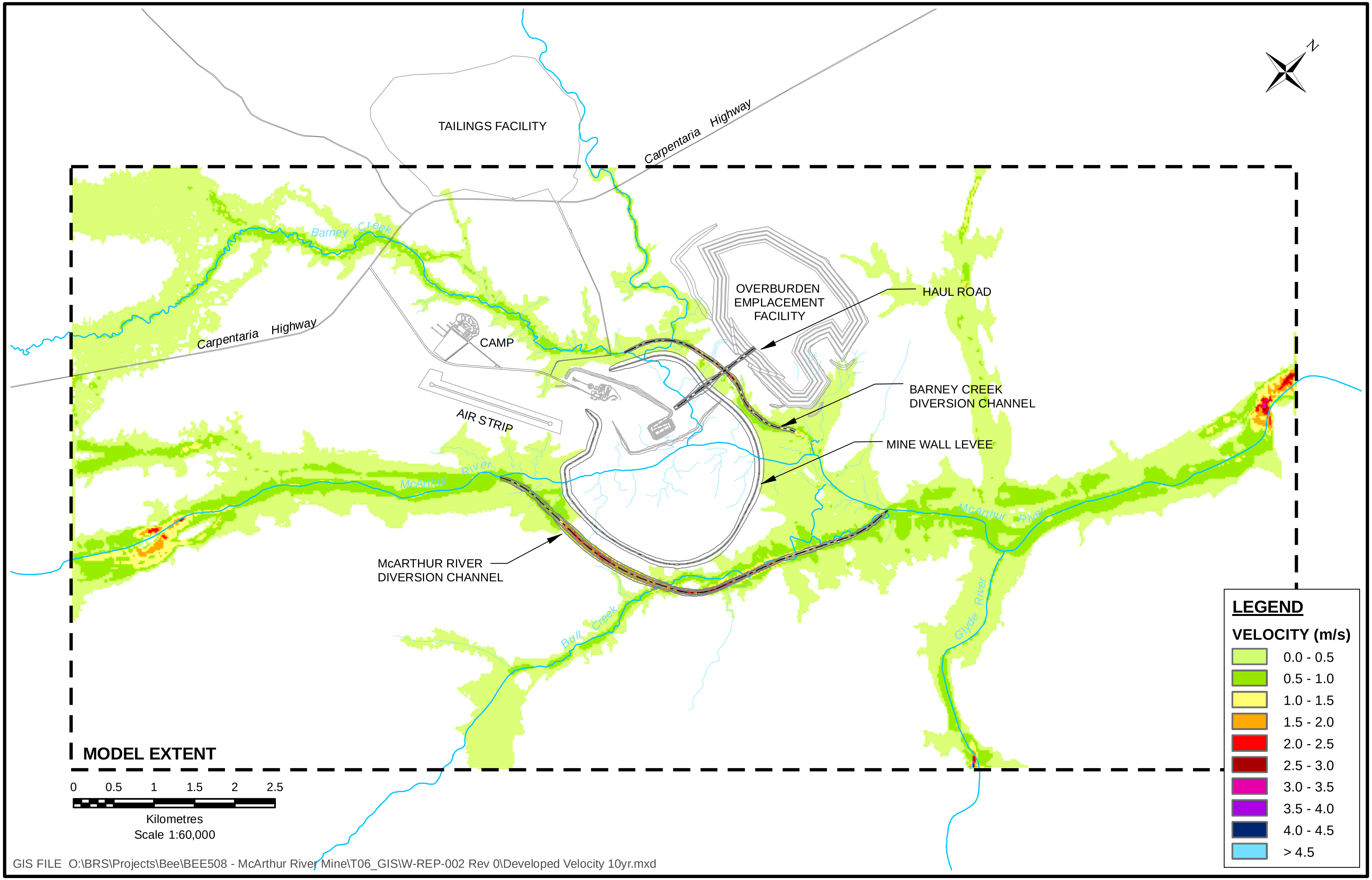

Figure 4.16
McARTHUR RIVER 10 YEAR ARI EVENT
DEVELOPED CONDITIONS - MAXIMUM VELOCITY

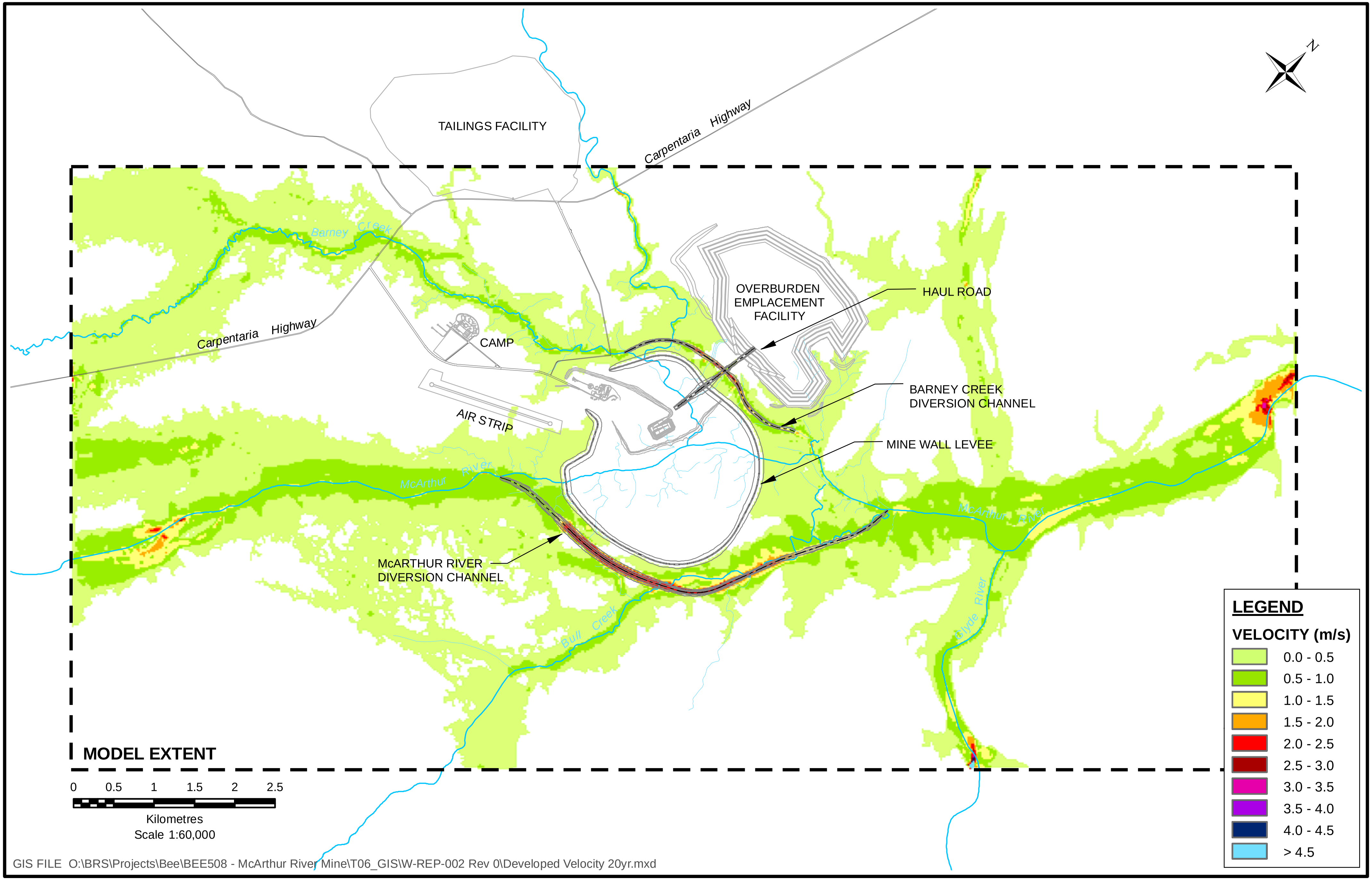

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Figure 4.17
McARTHUR RIVER 20 YEAR ARI EVENT
DEVELOPED CONDITIONS - MAXIMUM VELOCITY

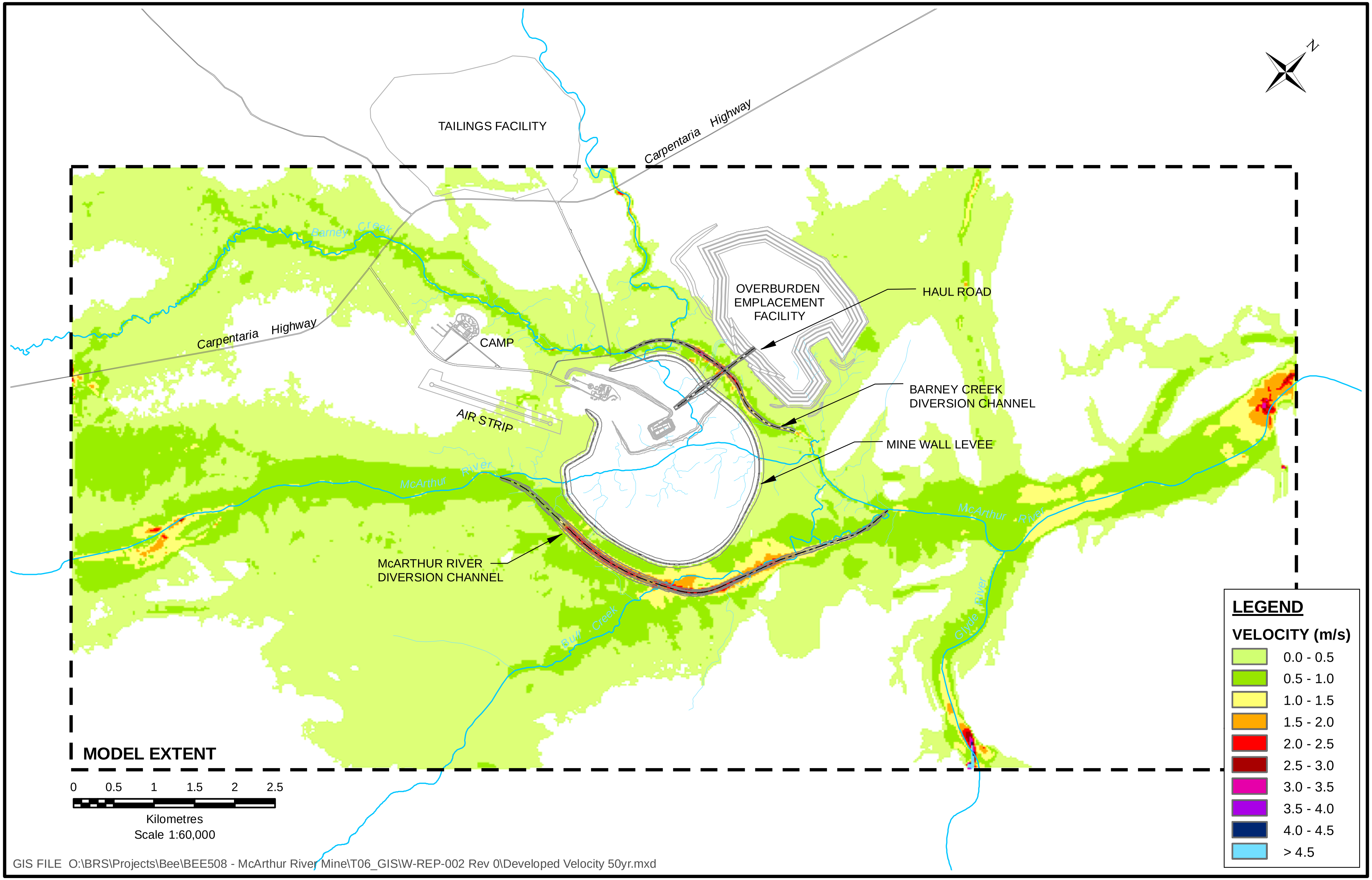

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Figure 4.18
McARTHUR RIVER 50 YEAR ARI EVENT
DEVELOPED CONDITIONS - MAXIMUM VELOCITY

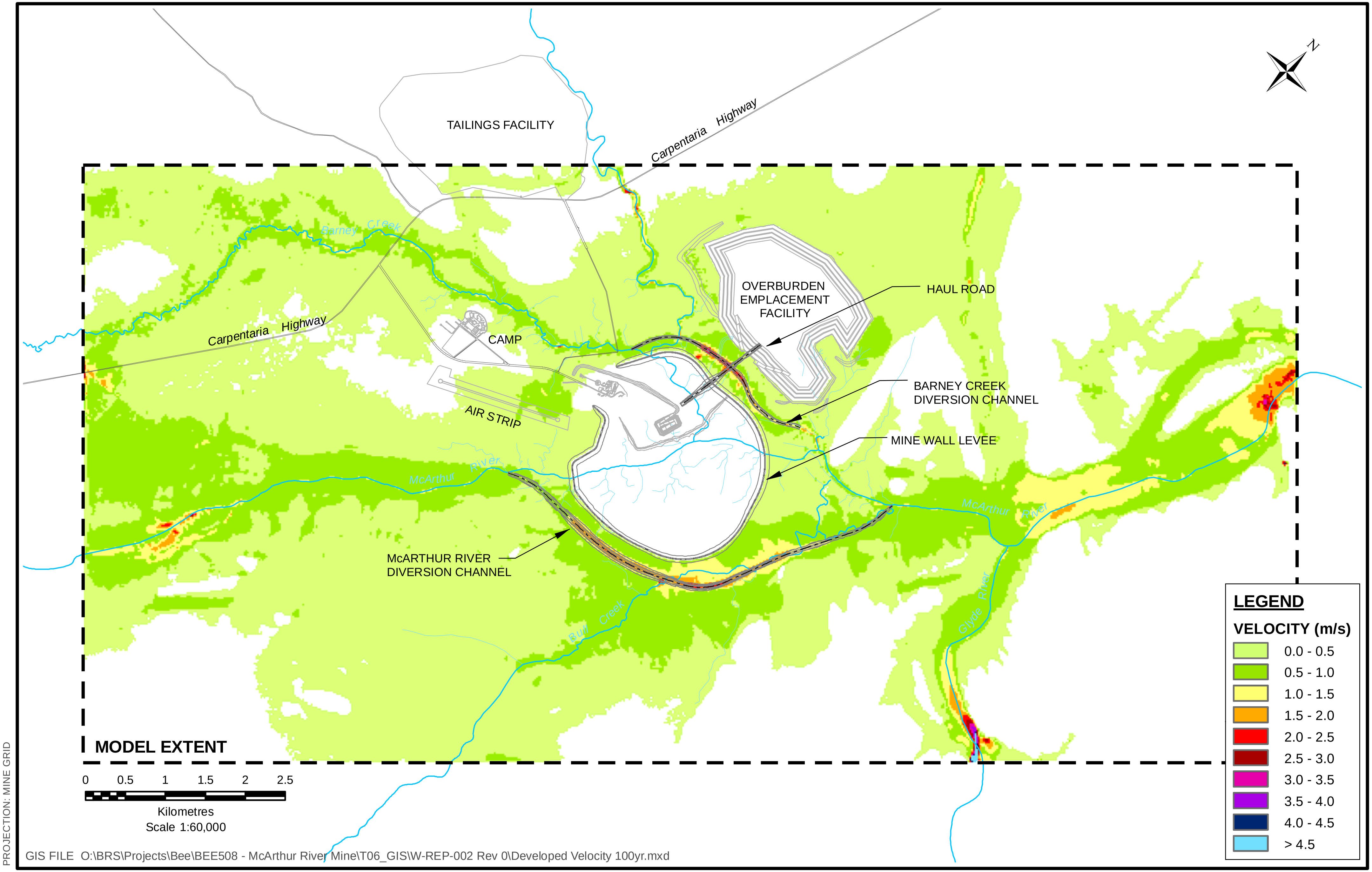

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Figure 4.19
McARTHUR RIVER 100 YEAR ARI EVENT
DEVELOPED CONDITIONS - MAXIMUM VELOCITY

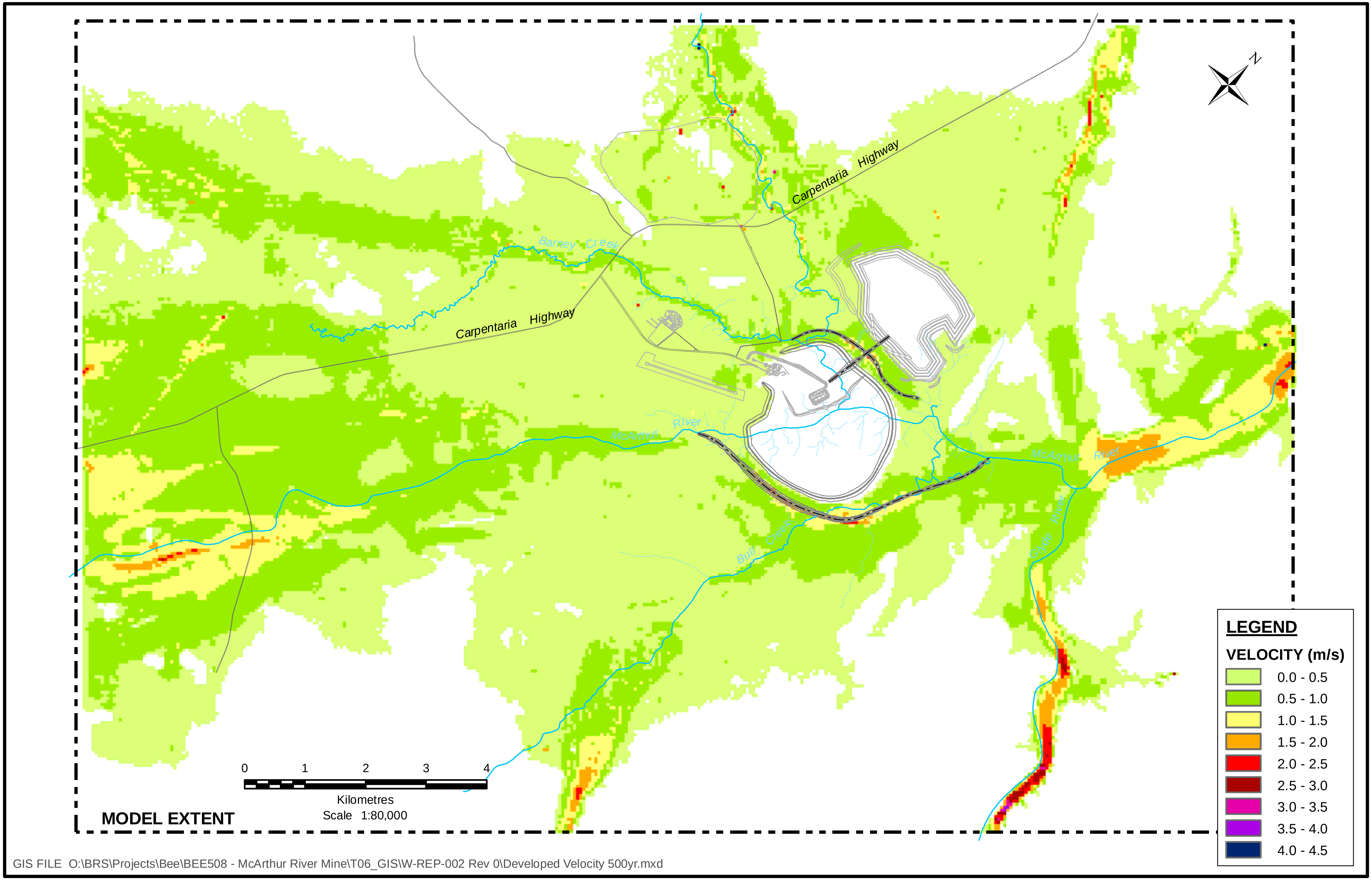

Figure 4.20
McARTHUR RIVER 500 YEAR ARI EVENT
DEVELOPED CONDITIONS - MAXIMUM VELOCITY

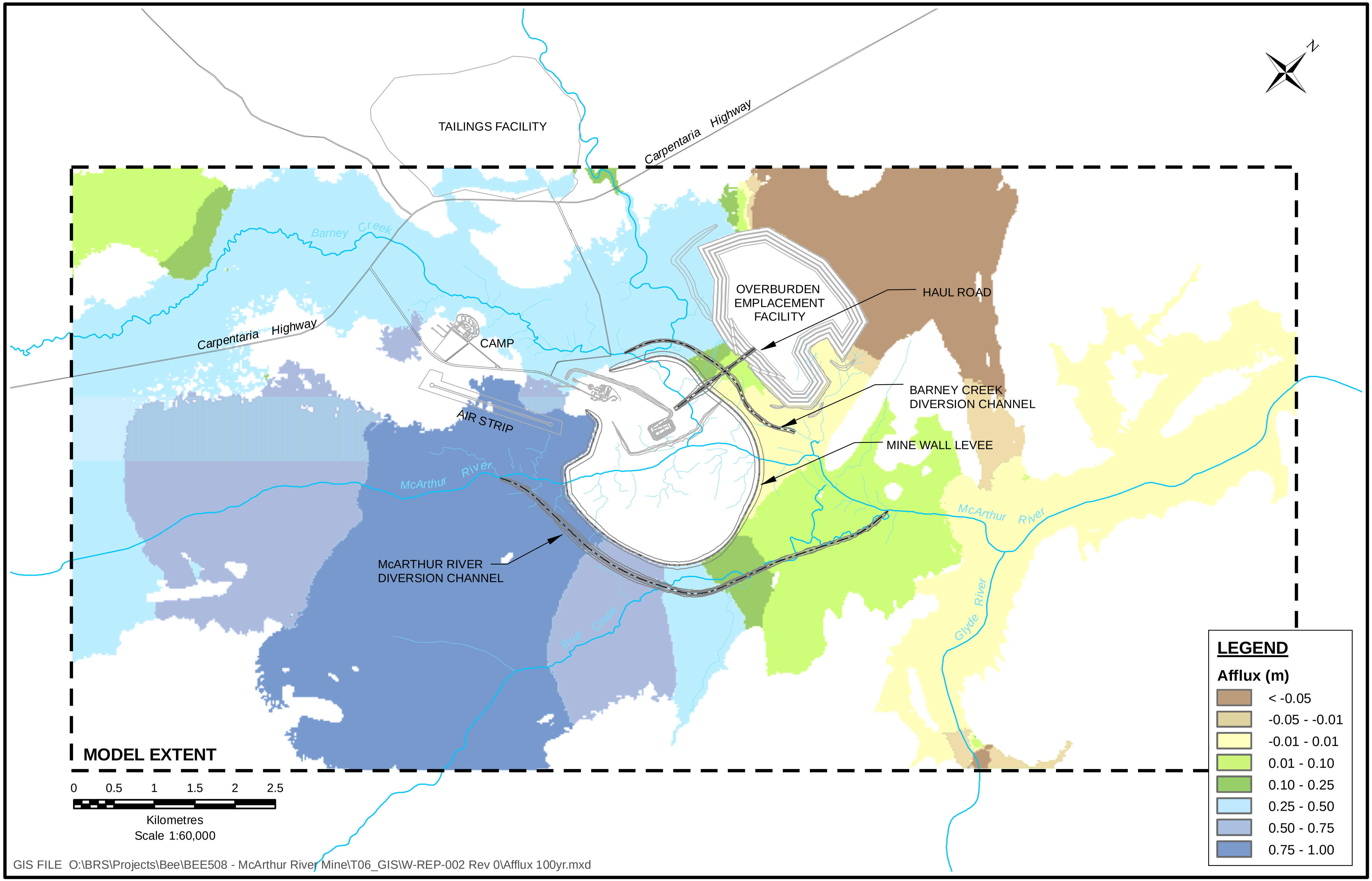

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Figure 4.21
McARTHUR RIVER 100 YEAR ARI EVENT
DEVELOPED CONDITIONS - WATER SURFACE AFFLUX

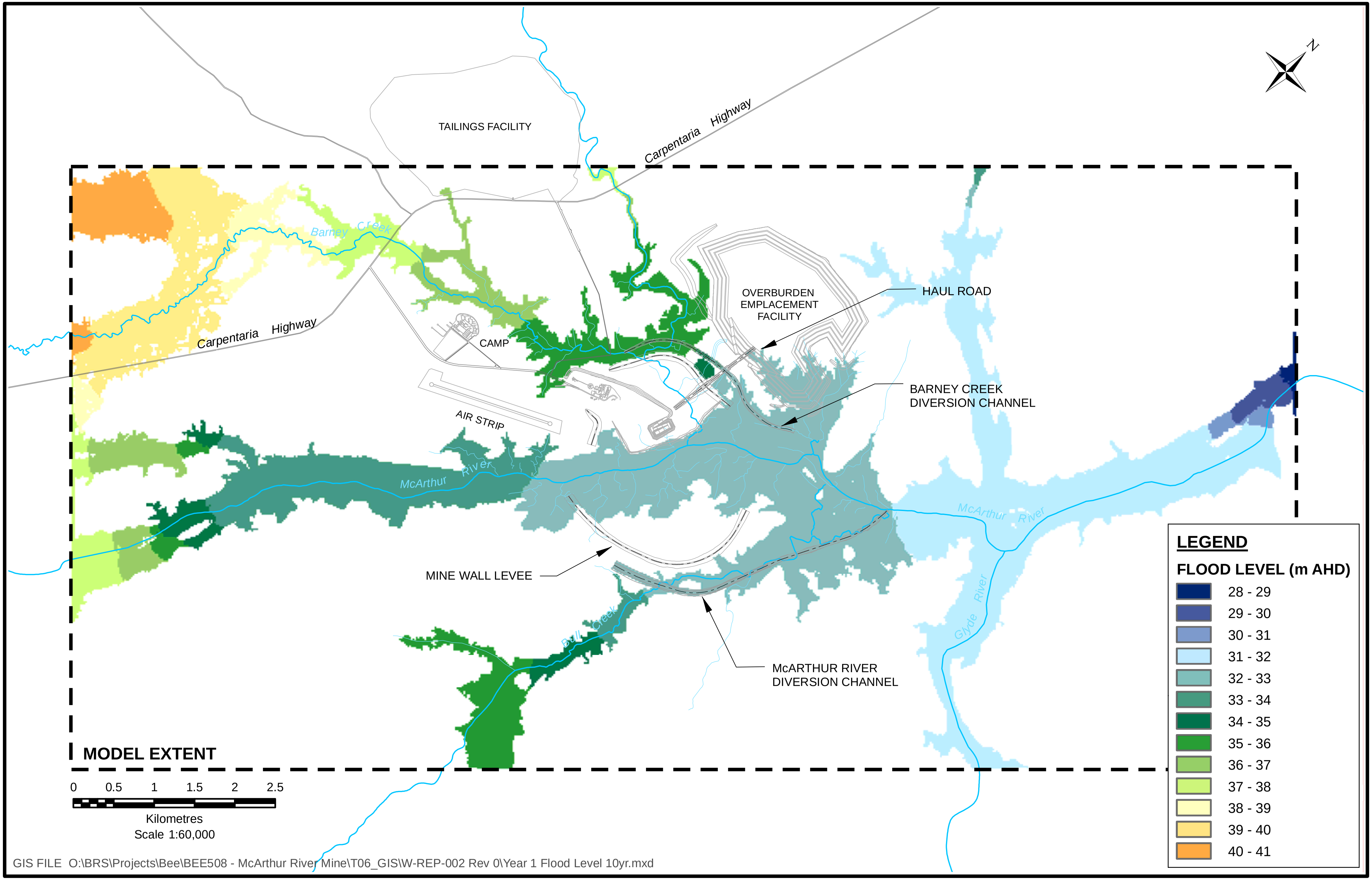

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Figure 5.1
McARTHUR RIVER 10 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - PEAK FLOOD LEVEL

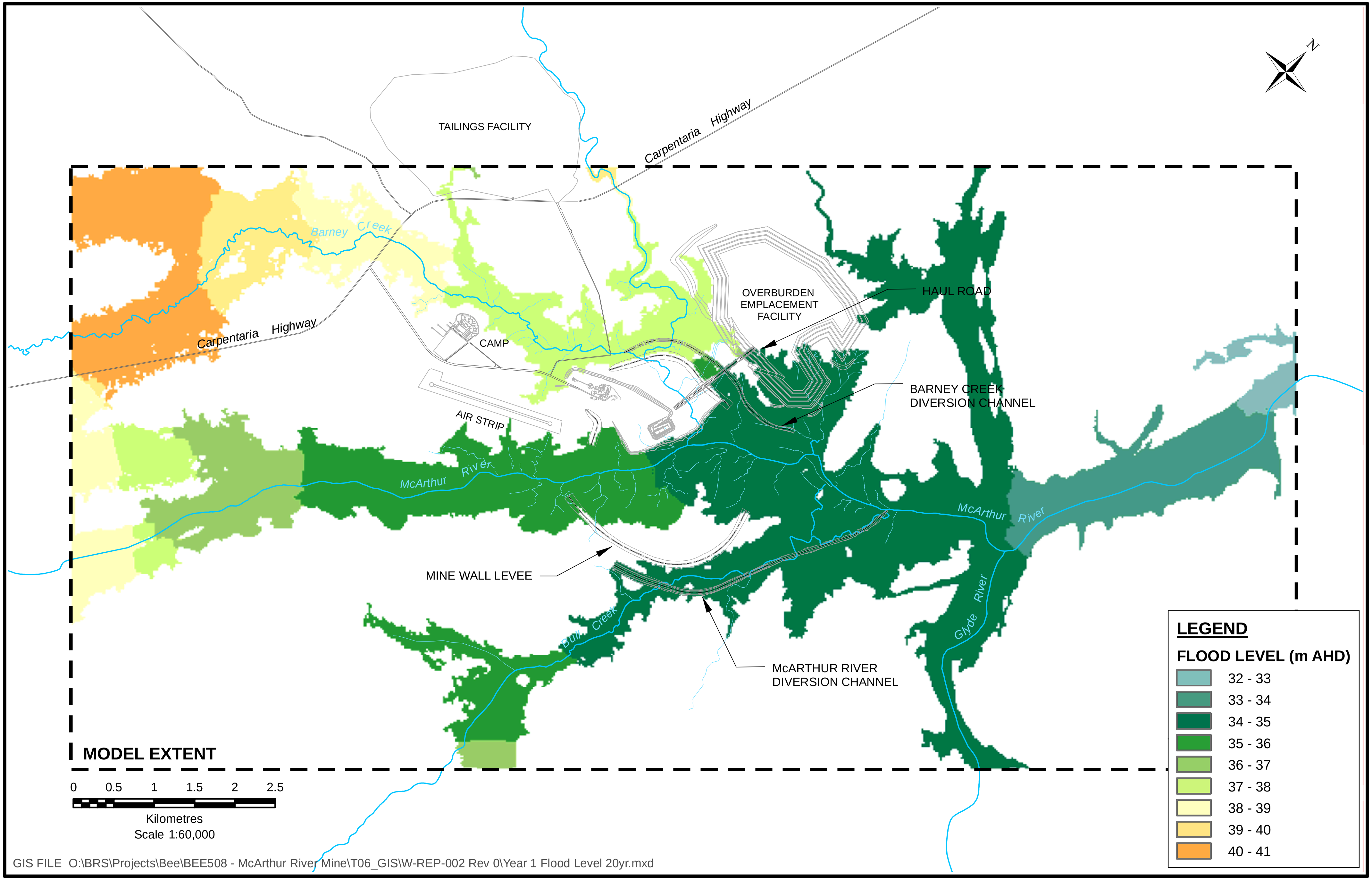

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Figure 5.2
McARTHUR RIVER 20 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - PEAK FLOOD LEVEL

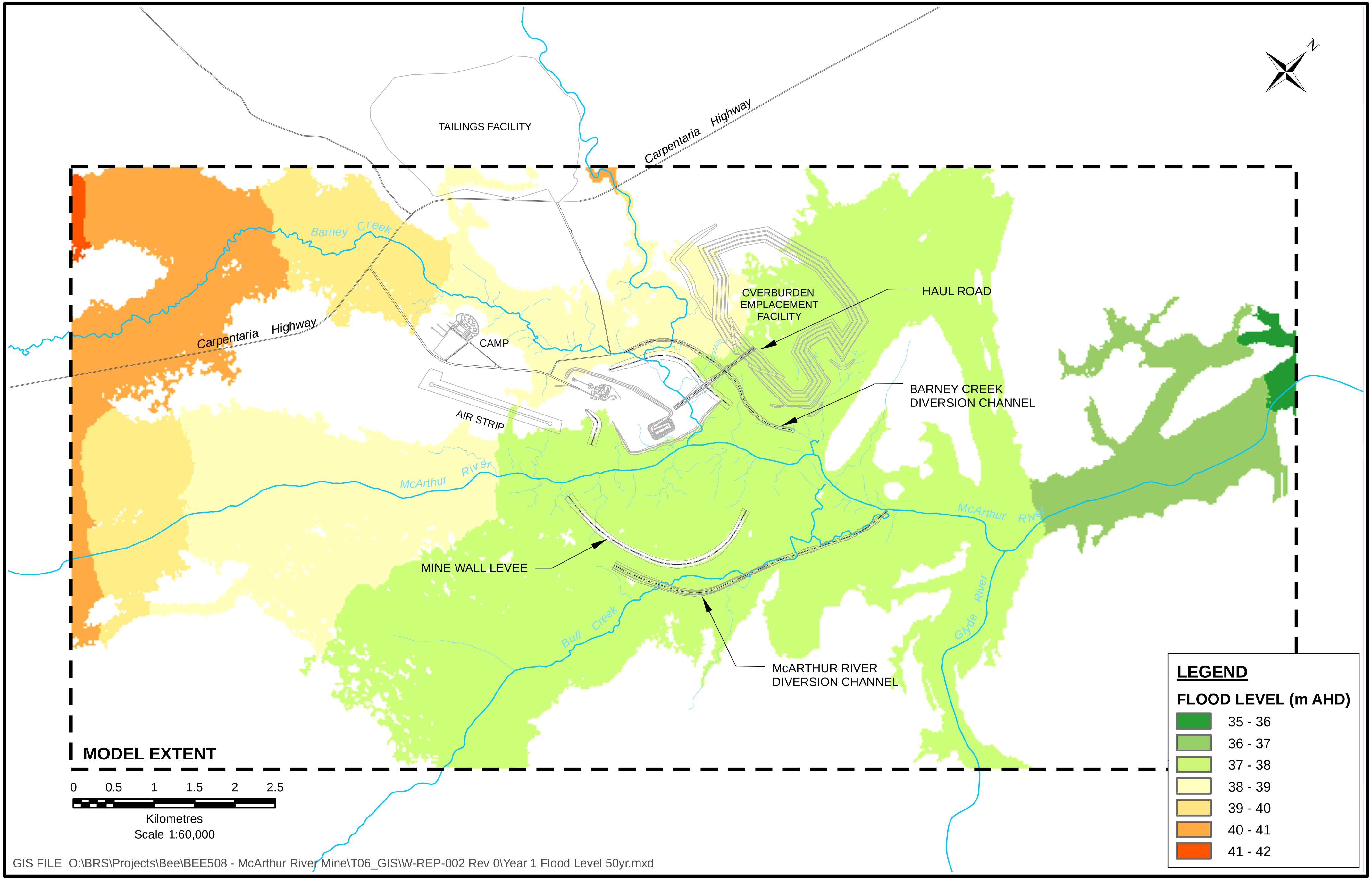

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Figure 5.3
McARTHUR RIVER 50 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - PEAK FLOOD LEVEL

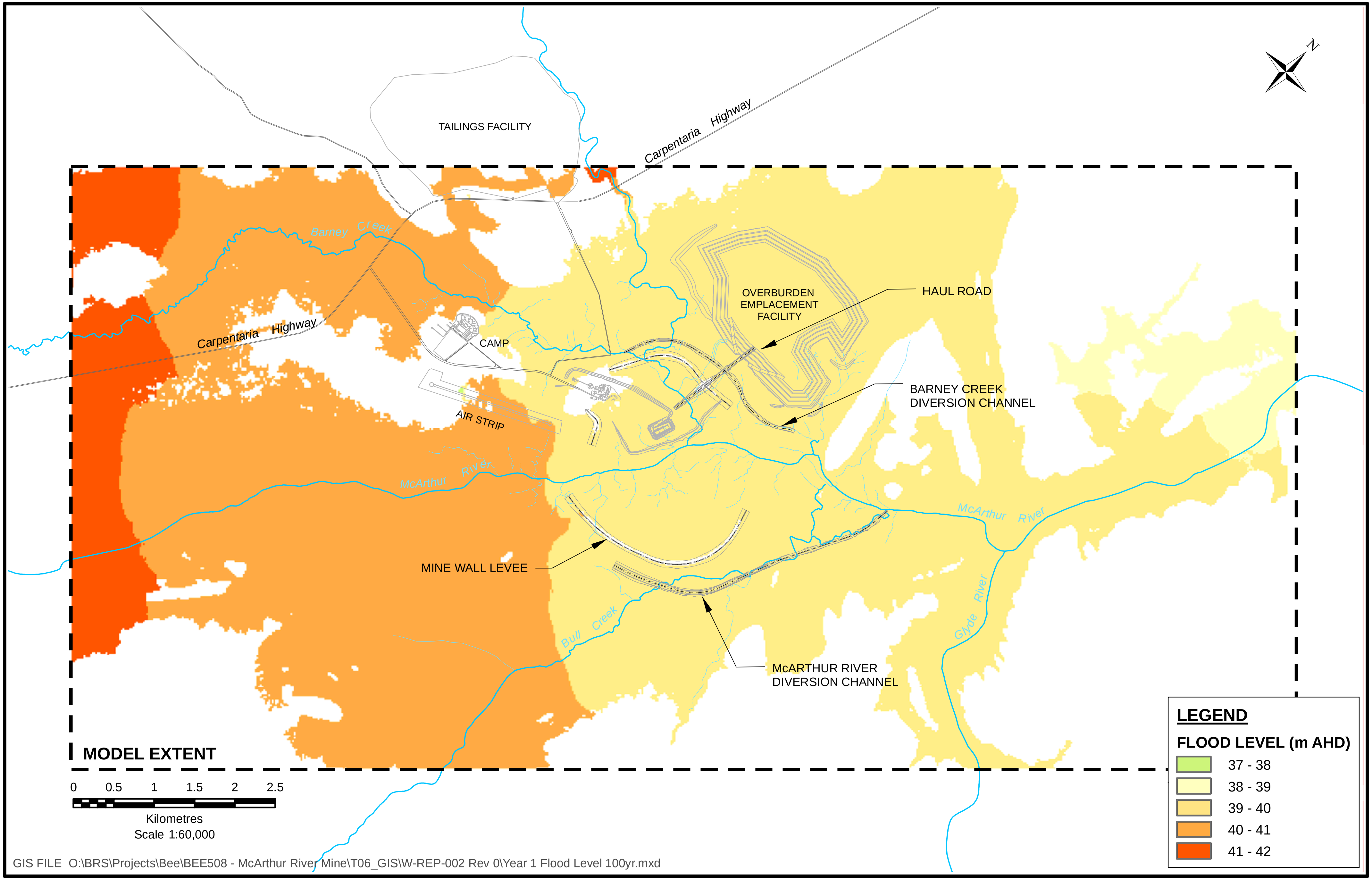

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Figure 5.4
McARTHUR RIVER 100 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - PEAK FLOOD LEVEL

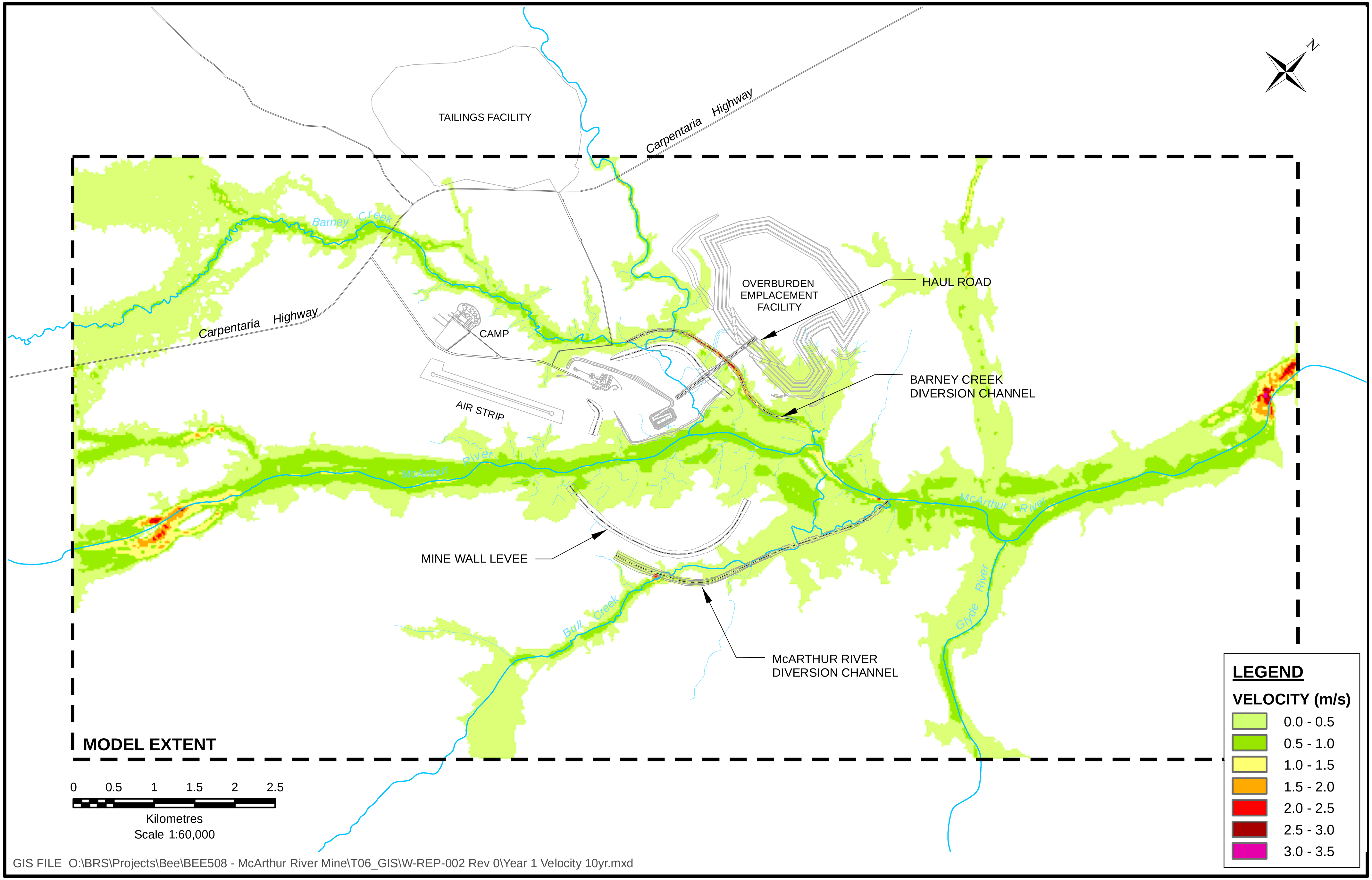

Figure 5.5
McARTHUR RIVER 10 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - MAXIMUM VELOCITY

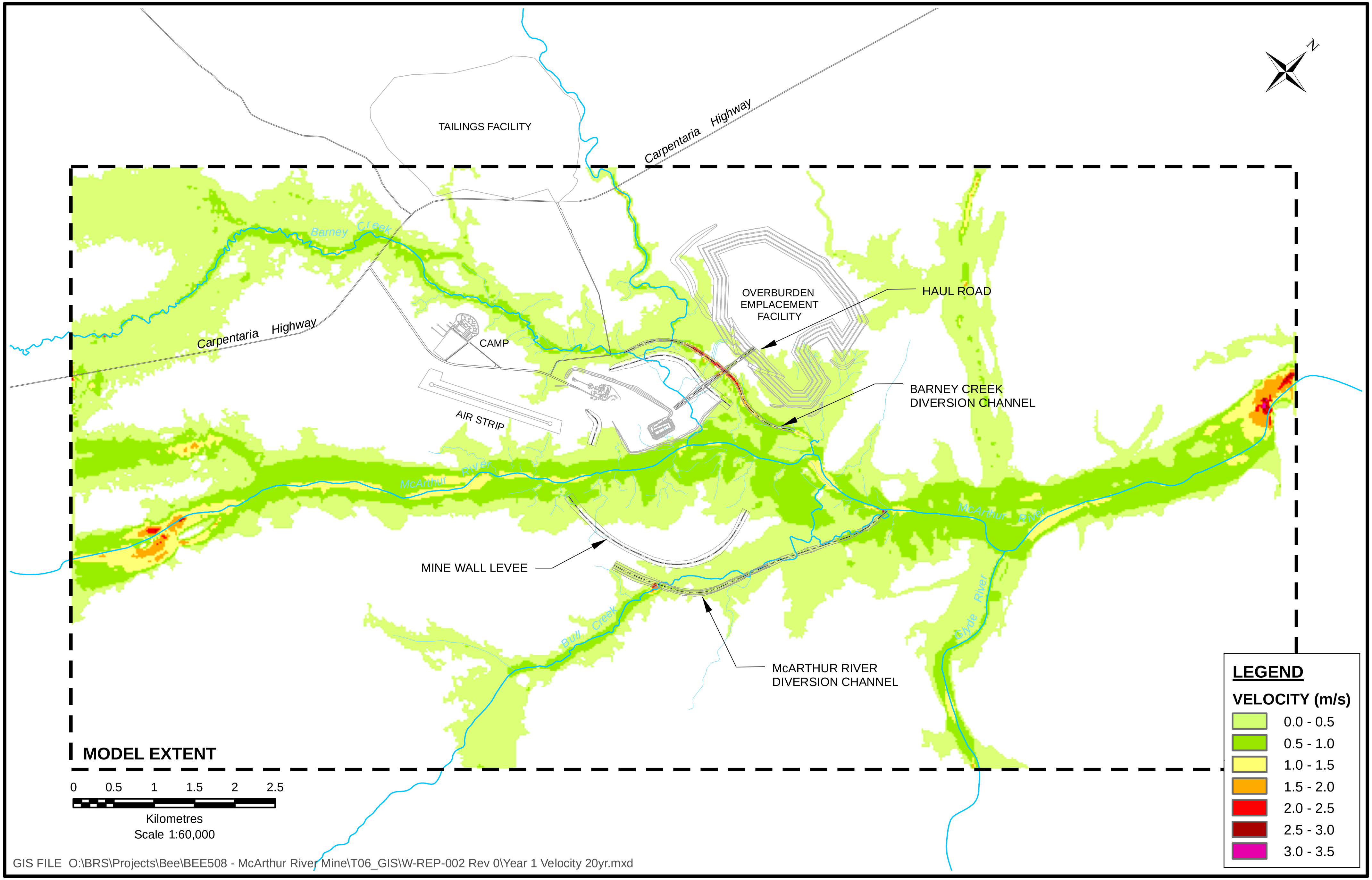

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Figure 5.6
McARTHUR RIVER 20 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - MAXIMUM VELOCITY

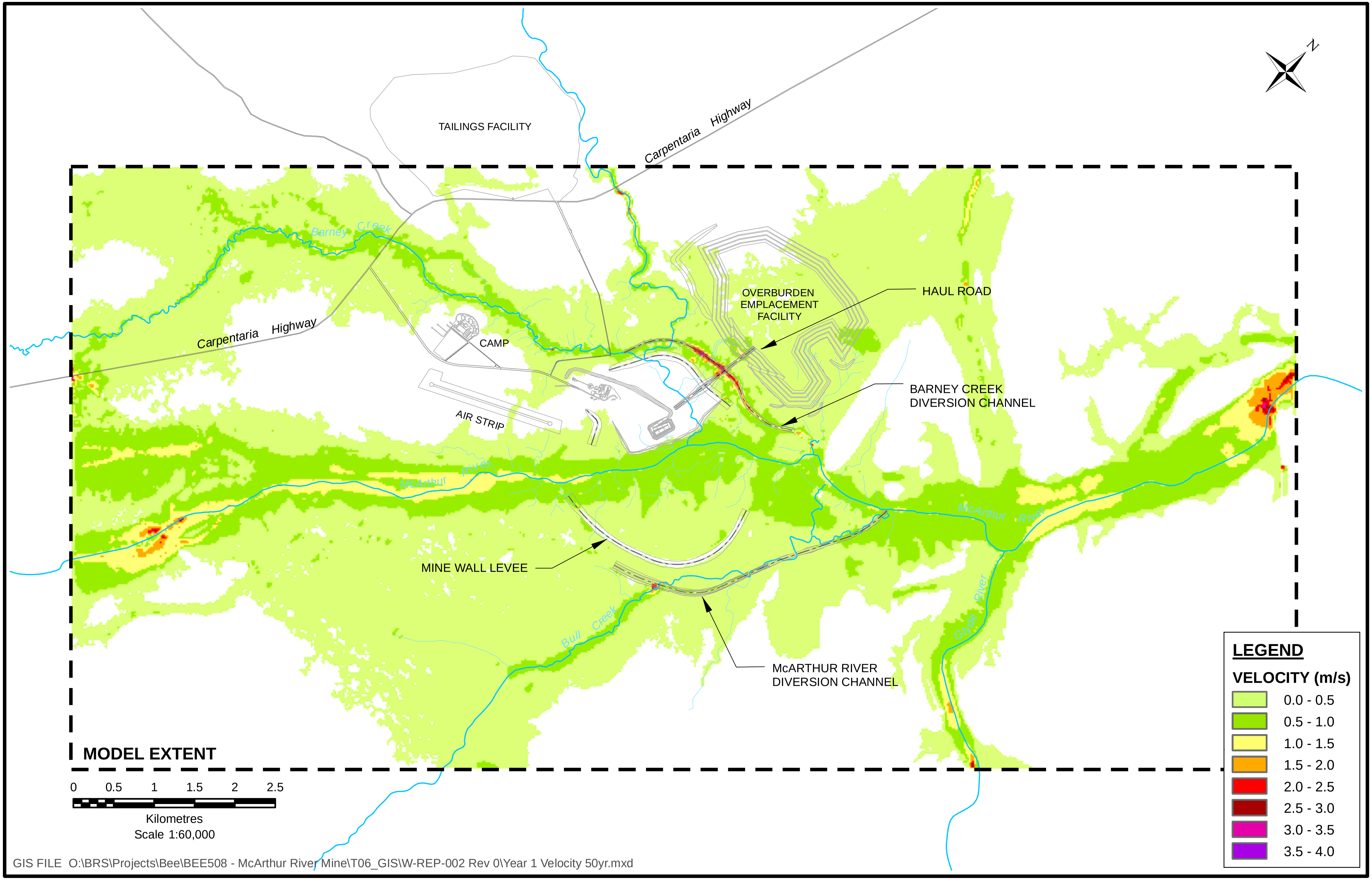

Figure 5.7
McARTHUR RIVER 50 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - MAXIMUM VELOCITY

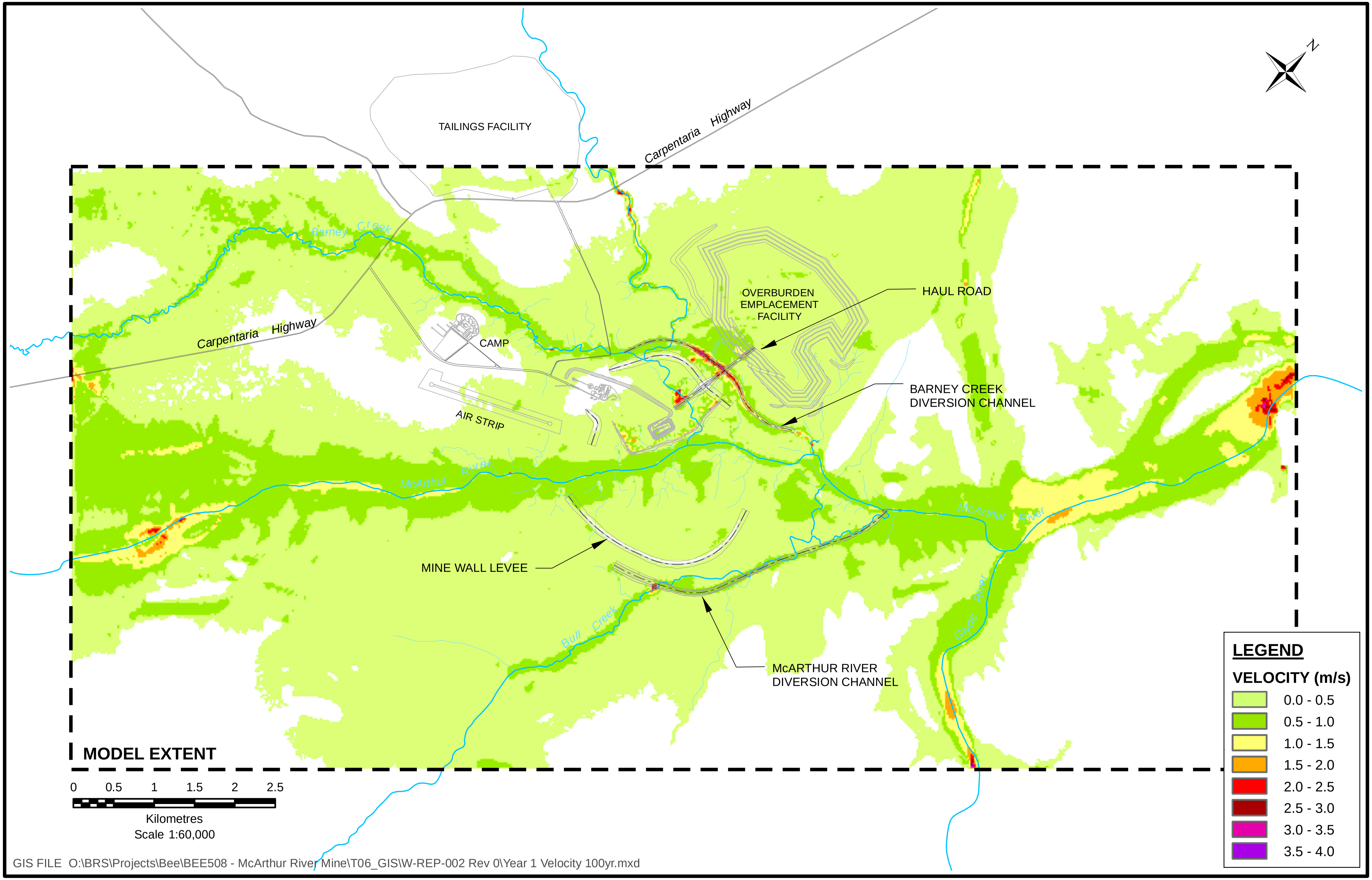

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Figure 5.8
McARTHUR RIVER 100 YEAR ARI EVENT
YEAR 1 CONSTRUCTION - MAXIMUM VELOCITY

Appendix B

ADDENDUM REPORT TO GEOTECHNICAL INVESTIGATION REPORT 04632206-009

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ADDENDUM REPORT TO

REPORT 04632206-009
GEOTECHNICAL INVESTIGATION
DETAILED FEASIBILITY STUDY
McARTHUR RIVER MINE EXPANSION PROJECT
McARTHUR RIVER, NORTHERN TERRITORY

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Electronic Copy - Connell Hatch

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1.0 INTRODUCTION

Golder Associates' (Golder) report 04632206-009[1] described a geotechnical investigation for the proposed flood protection works for the McArthur River Mine Expansion Project. This report is an addendum to that.

The proposed expansion would covert the mine to an open cut operation with a single pit intersecting the McArthur River channel. The pit would be protected by 7 km of ring levee and a 5 km river diversion channel to the south. Subsequent to the base report, Kellogg Brown & Root Pty Ltd (KBR) undertook design of the flood protection works including the levee and diversion channel[2].

Connell Hatch are reviewing those designs for the project owner, Xstrata Zinc, in response to regulatory issues with which Golder have had no involvement. This addendum describes analysis and assessment work requested by Connell Hatch to address specific geotechnical matters raised as part of the review. The work requested was as follows.

Embankment Stability

(1) Stability analyses were to be carried out for the embankment section shown on the KBR Drawing No. BEE508-C-DWG-303-A, with the maximum expected embankment height, for the following conditions:

(a) At the end of construction of the embankment.

(b) Downstream slope with full flood condition (500 year ARI)

(c) Upstream slope with rapid drawdown condition.

(d) Under seismic loading.

(e) Sensitivity analysis was to be undertaken to assess the impacts of variation in the clay core strength either due to material variability or poor construction practices.

Item (b) required an assessment of the likely phreatic surface that could develop within the embankment, by transient seepage analysis.

---

1 *Geotechnical Investigation, Detailed Feasibility Study, McArthur River Mine Expansion Project, McArthur River, Northern Territory, Golder Associates report to Xstrata, April 2005.*

2 Drawings *BEE508-C-DWG-301-D, 302-D*, and *303-A*, Kellogg Brown & Root Pty Ltd, 2006
McArthur River Mine Expansion Project, Design Basis Report, Kellogg Brown & Root Pty Ltd, April 2006
McArthur River Mine Expansion Project, Technical Specification, Kellogg Brown & Root Pty Ltd, April 2006

(2) An assessment of potential cracking of the clay core was to be undertaken, and the impacts of this cracking on both the upstream and downstream stability considered, particularly under flood or heavy rainfall conditions where the cracks could fill with water.

(3) Design criteria for acceptable factors of safety were to be developed from relevant reference material such as Australian National Committee on Large Dams (ANCOLD) Guidelines. The results of the stability analyses undertaken were to be compared to these design criteria.

Piping Risk Assessment

A piping risk assessment was to be undertaken, considering the potentially dispersive nature of the clay core materials and the 1 in 500 year ARI flood event. This risk assessment was to consider the likely event that deep cracking of the upper sections of the embankment will occur. Comment was to be provided regarding whether the piping risk is acceptable, with reference to applicable technical guidelines. If the piping risk was considered to be unacceptable, additional defensive design measures were to be considered and the risk assessment rerun to demonstrate that an acceptable risk is achieved.

Inspection and Monitoring

An outline was to be provided of routine inspection and monitoring that should be undertaken to ensure that the integrity of the flood protection bund is maintained over the design life.

These matters are design issues properly the province of the levee designers. As geotechnical consultants, Golder provided geotechnical assistance and advice as part of the design process, so we consider it reasonable to comment on these geotecbnical design related matters as requested.

2.0 EMBANKMENT STABILITY

2.1 Stability Criteria

Stability criteria in dam and embankment design are conventionally expressed as limit state equilibrium factors of safety[3]. ANCOLD has not revised its published general criteria for embankment dam stability factors of safety since 1969[4]. The criteria then were:

End of construction condition	1.3 to 1.5
Steady seepage - downstream stability	1.5
Rapid drawdown upstream stability	1.25 to 1.3
Initial filling	1.3 to 1.5

Fell et al[5] give an up to date local and international perspective, which differs little from ANCOLD 1969:

End of construction condition	1.3
Steady seepage - downstream stability	1.5
Rapid drawdown - upstream stability	1.3
Maximum flood downstream stability	1.5, or 1.3 if crest zone is not free-draining

For both, conservative (eg lower-quartile) material strengths are to be adopted.

For earthquake loading, practice now accepts a minimum pseudo-static factor of safety of 1.0 without more elaborate analysis[6], provided significant pore pressure rise or strain-weakening of materials are not anticipated. With additional deformation analysis, factors of safety lower than 1.0 may be accepted.

Levees are not dams, nor are mining structures necessarily equivalent to civil structures in terms of acceptable design risk levels. Nevertheless, we think stability factors of safety of similar general order to the above are appropriate for this major levee, in view of their apparent high failure consequence.

---

3 Which are actually strength reduction factors at the theoretical onset of failure.

4 *Current Technical Practices for Design, Construction, Operation and Maintenance of Large Dams in Australia*, Australian National Committee on Large Dams, June 1969

5 *Geotechnical Engineering of Dams*, R Fell, P MacGregor, D Stapledon and G Bell, Balkema, 2005

6 *Guidelines on Design of Dams for Earthquake*, Australian National Committee on Large Dams, 1998

2.2 Sections and Parameters

We understand that it is proposed to build the levee cross-section progressively over two annual dry seasons. In the first season, just the central part of the cross-section comprising the Zone 1A sealing zone and the Zone 2 upstream rockfill zone will be constructed (KBR Drawing BEE508-C-DWG-303-A). In the subsequent season the outer mine waste rock shells will be added.

During this construction sequence, the levee is expected to be exposed to one wet season without the support of the outer rock shells.

For stability analysis, we selected critical sections as follows:

1. General Section - near CH2000 m, about the highest section of levee away from the river channel crossings. Crest RL44 m, natural surface RL26 m, height 18 m.

   1a. Temporary condition without the outer rock shells

   1b. With the outer rock shells in place

2. River crossing about CH2550 m, near the highest section across the McArthur River channel. Crest taken as RL44 m, natural surface RL19 m, height 25 m.

   2a. Temporary, without outer rock shells

   2b. With rock shells

Material parameters used were generally as per the base report, vis:

Table 1 Stability Analysis Parameters

Case	Parameter	Levee		Foundation	
		Zones 2 & 3 Rockfill	Zone 1A Compacted Clay	Clay	Alluvial Sand and Gravel
Permeability	k_h	1e-3 m/s	1e-7 to 1e-8 m/s	1e-8 m/s	1e-4 m/s
	k_v	1e-3 m/s	1e-7 to 1e-8 m/s	1e-9 m/s	1e-5 m/s
Long Term	Φ'	45°	28°	25°	33°
	c'	0	8 kPa	5 kPa	0
Short Term	$Φ_u$	45°	0°		33°
	c_u	0	100 kPa	5	0
Sensitivity Long Term	Φ'	45°	26°	25°	
	C'	0	0 kPa	5 kPa	0
Under seismic loading	$Φ_u$	45	0	25°	33°
	c_u	0	100 kPa	5 kPa	0

We adopted an undrained strength for compacted clay of 100 kPa for the end of construction and earthquake cases, based on experience with similar compacted clay fills.

2.3 Transient Seepage Analysis

Unlike dams, most levees are only subjected to short term high water levels. Consequently the "steady seepage" condition of embankment dam engineering often does not apply. We were advised the MacArthur River levees are unlikely to be subjected to a large flood of duration longer than about three weeks, more likely shorter. In that time, for the cross section as designed, conventional steady seepage conditions are unlikely to be established.

To obtain pore pressures for the "maximum flood" condition, we instead analysed the transient seepage response of Section 1a to a triangular flood hydrograph peaking at the 500 year ARI flood level, with an arbitrary 1 week rise time and 2 week fall time. The results are shown on Figures 3 & 4 for clay fill zone permeabilities of 10^{-7} and 10^{-8} m/s respectively.

The phreatic surfaces reached are relatively low, well below what would be expected from steady seepage analysis. These transient analyses assume that Zone 1A is uncracked and free of construction defects. Cracks or defects could result in a higher pore pressures further downstream within the section.

2.4 Stability Analysis

2.4.1 End of Construction

Properly engineered fill embankments of the order of 20 m high on firm foundations do not suffer end of construction failures. Figures 1 & 2 show limit equilibrium analyses for Section 1a for the end of construction condition, with the expected high factors of safety, well in excess of the criteria (Section 2.1).

2.4.2 Maximum Flood

Figures 5 & 6 show downstream limit equilibrium analyses for the temporary Section 1a condition and transient seepage from the 500 year ARI flood level (Section 2.3). The probability of such a large flood occurring during the temporary exposure window is of the order of 0.2%.

Tbe calculated factors of safety for this case (both ~1.4) reflect the drained strength adopted for the Zone 1A fill without seepage induced pore pressures, because the transient seepage analysis did not propagate seepage that far within the flood rise and fall time. The resulting factors of safety meet the modern "maximum flood" criteria (Section 2.1), even for this temporary condition.

Levee design should give consideration to the possibility that higher pore pressures may develop than those analysed if there are cracks or construction defects in the sealing zone, or possibly via untreated permeable foundation zones.

For the longer term condition with the rock shells in place, the section is calculated to be suitably stable even if conservative steady seepage conditions are assumed, as shown on Figure 6 for Section 1b (F ~ 2.0).

2.4.3 Rapid Drawdown

The levee cross-section will experience relatively rapid drawdown following most major flood events, but the design incorporates an upstream permeable zone to resist that.

The calculated factor of safety for a conservative rapid drawdown assumption is ~1.9 for the temporary condition without the outer rock shells (Section 1a - Figure 8), and 2.7 for with the outer rock shells (Section 1b - Figure 9).

2.4.4 Seismic Loading

The Australian Standard AS 1170.4-1993 "Minimum Design Loads on Structures - Part 4: Earthquake Loads", contains a seismic acceleration coefficient map of the Northern Territory for an AEP of 1:475. This AEP is generally accepted as appropriate for structures with a design life of up to about 50 years. The mine has a 25 year design life, and it is assumed that suitable closure strategies for the levee will be implemented to negate the impacts of earthquake loading beyond that time. The McArthur River area falls within (bedrock peak) acceleration coefficients of 0.04 and 0.05g. For the purpose of conservative assessment, a peak bedrock acceleration coefficient of 0.1g and a magnitude (M_w) of 6.0 were adopted.

a) Pseudo Static Analysis

A pseudo-static limit equilibrium analysis of Section 1a was undertaken following generally the procedure described in the ANCOLD guidlelines[6]. This approach is considered reasonable where significant pore pressure rise or strain-weakening of materials are not anticipated (Section 2.1). That is the case for the proposed levee embankment materials. For the foundation materials, the potential for pore pressure rise causing liquefaction is considered in b) below.

Using the adopted undrained parameters and a seismic coefficient equal to half the adopted bedrock peak acceleration (following the Corps of Engineers approach in the guidelines), we obtained a minimum factor of safety of 1.7, well in excess of the criteria (F=1.0, Section 2.1).

b) Liquefaction Assessment

A liquefaction assessment was carried out based on the method of Youd et al[7].

Ground parameters used were from Golder Associates report 04632206-009[1]. SPT 'N' values in the sands and silty sands encountered in the boreholes were greater than 10 and fines contents of sands sampled and tested from the boreholes and test pits was not less than 5%. Sands encountered in the boreholes were generally encountered below 2 or 3 m from the surface. For the purpose of the assessment, an 'N' value of 5, 0% fines and 3 m of overburden material were adopted.

The calculated FOS against liquefaction (i.e. the ratio of resisting forces to destabilising forces) was 1.3. Liquefaction is not anticipated.

2.5 Results Summary

Stability analysis results are summarised in Table 2.

Table 2 Stability Analysis Results

Case	Factors of Safety – Sections 1 & 2		
	Target	a) Temporary Condition	b) Permanent Condition with Rock Shells
Section 1 – General Section			
1. End of Construction	1.3	1.7 - 2.0	nc*
2. Maximum Flood	1.3	1.4	2.0
3. Rapid Drawdown	1.3	1.9	2.5 - 2.7
4. Earthquake	1.0	1.7 - 2.1	nc
Section 2 – River Channel			
2. Maximum Flood	1.5	nc	2.0
3. Rapid Drawdown	1.3	1.9	2.5

* Not calculated.

7 *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*, T. L. Youd (Chair), I. M. Idriss (Co-Chair), R D. Andrus, I Arango, G Castro, JT Christian, R Dobry, W D Liam Finn, LF Harder Jr., ME Hynes, K Ishihara, JP Koester, SSC Liao, WF Marcuson III, GR Martin, JK Mitchell, Y Moriwaki, MS Power, PK Robertson, RB Seed, and KH Stokoe II, Journal of Geotechnical and Geoenvironmental Engineering V127 N10, 2001.

2.6 Clay Core Cracking

A probable defect type within the sealing zone of the levee is shrinkage cracking due to desiccation in service from the initial placement moisture content[8].

The local dry season climate has large moisture deficit (evapotranspiration vs rainfall). The seasonal moisture content variation depth is probably at least 4m, from a vegetated natural surface[9]. An elevated structure like a levee embankment may suffer additional desiccation due to increased exposure, but may suffer less desiccation if vegetation is excluded and if the surface is rockfill, not fine-grained fill[10]. There would appear to be considerable scope for desiccation and cracking to perhaps 4 or 5 m depth below the crest of the levee. Although the fill may re-swell during the wet season due to incident rainfall, cracks may not completely close before a flood against the levee occurs, leaving a potential flow path through the levee.

Such flow would appear to be of little consequence to basic batter slope stability as analysed, but has the potential to lead to breach of the structure by a variety of piping mechanisms (some of which include local induced instability Section 3.0).

Because serious cracking appears likely to be confined to the near-crest area, a design response may be to provide internal filtration in that area, say by means of a strip of geotextile between the sealing zone and the downstream rock shell over say the upper 10 m of bank.

8 Which practice usually keeps relatively high to maximise fill placement density and minimise fill permeability.

9 AS2870-1996

10 Because rockfill cannot support soil-suctions, so capillary rise is limited.

3.0 PIPING RISK ASSESSMENT

3.1 Basis

The brief was to undertake a piping risk assessment, "*considering the potentially dispersive nature of the clay core materials and the 1 in 500 year ARI flood event. [The] risk assessment should consider the likely event that deep cracking of the upper sections of the embankment will occur.*"

Comment was to be provided regarding whether the piping risk is acceptable, with reference to applicable technical guidelines. If the piping risk is considered to be unacceptable then additional defensive design measures were to be considered and the risk assessment rerun to demonstrate that an acceptable risk is achieved.

We followed the basic procedure for piping risk assessment described in Foster et al[11]. This is not a failure risk assessment in accordance with the ANCOLD *Guidelines on Risk Assessment*[12], rather it considers one particular failure mechanism and attempts to broadly quantify the resulting probability of breach, by accumulating subjective judgements of individual contributing event probabilities.

3.2 Fault-Event Tree

Appendix A contains fault-event trees constructed for piping failure of the proposed MacArthur River Levee.

In this system, the starting point is the probability that the retained water level will reach the height range quoted at the first branch at least once in any given year. The notion is that it will be sufficient, for the purposes of piping breach development, for the retained water level to reach the "failure range" at least once, however briefly. There is evidence to support this argument for embankment dams (where water levels rarely fall quickly from the annual high), but it may be rather conservative for a levee. The annual probability so obtained is then multiplied by the tree of fault-event probabilities to obtain the breach probability for each combination, and the overall breach probability is obtained by summing the combinations.

We view the assigned event tree probabilities at each stage as essentially arbitrary, despite claims to the contrary[11]. They are, however, intended to at least be internally consistent, so that the overall relativity may be reasonably meaningful.

11 *Estimation of the Probability of Failure of Embankment Dams by Internal Erosion and Piping using Event Tree Methods*, MA Foster, R Fell, R Davidson and CF Wan, ANCOLD Bulletin No 121, August 2002.

12 *Guidelines on Risk Assessment*, Australian National Committee on Large Dams, 2003.

Calculated breach probabilities were as follows:

Case	Annual Probability		
	Piping through levee	**Piping through foundation**	**Total ***
No rock shells, no internal filters	0.045% ~1 in 2,200	0.017% ~1 in 5,800	0.06% ~1 in 1,600
With rock shells, no internal filters	0.03% ~1 in 3,400	0.017% ~1 in 5,800	0.05% ~1 in 2,000
With rock shells, internal filtration	0.001% ~1 in 100,000	0.017% / 2 ˇ ~1 in 11,600	0.01% ~1 in 10,000

ˇ arbitrarily halved * totals are rounded

3.3 Engineering Comment

The condition with the levee in place without the rock shells is a short-term construction condition, for which a relatively high probability of breach might be tolerated. Even for the permanent condition without internal filtration, the calculated probability of breach is relatively low, compared with, say, the annual probability of overtopping, which is approximately 1 in 500.

4.0 MONITORING AND SURVEILLANCE

4.1 General

Monitoring and surveillance requirements for embankment dams are described in the ANCOLD guideline[13]. Levees are not dams, but it can be argued that monitoring and surveillance needs are generally similar, at least for major levees protecting high value assets. Failure of the proposed MacArthur River Mine levee could lead to major economic loss, potential loss of life, and potential significant environmental impact. Consequently, an appropriate monitoring and surveillance plan should be designed and implemented.

The requirements depend on[5,13]:

i) The consequences of failure

This is a high hazard levee. Failure could lead to major economic loss (probably loss of the mine), potential loss of life (particularly if failure were to occur quickly and without warning from surveillance), and potential significant environmental impact (debris would be driven into the pit not out to the surrounding environment, but some contaminated water could be expected to escape on the falling flood leg).

ii) The type of dam

The design is for a zoned earth levee with flat batters (in its final form), but no internal filtration.

iii) The nature of the foundations

Some permeable alluvial foundation materials are present. Initial seepage cutoff proposed is not to full-depth, but may be augmented to full depth subsequently.

iv) The size of dam

At up to ~25 m high, the proposed levee would fall into the "large dam" category, were it a dam.

[13] *Guidelines on Dam Safety Management*, Australian National Committee on Large Dams, 2003.

v) Known deficiencies

There are no known deficiencies, but given the long levee crest length (7 km), it will be challenging to construct the entire levee without any "built-in" deficiencies.

vi) Identified failure modes

The piping failure risk is discussed in Section 3.0.

vii) The water level

For much of its life the water level against the levee will be nil. Requiring intensive monitoring or surveillance under such conditions appears unnecessary. In this monsoonal climate, nearly all cases of high flood level against the levee will occur in the wet season, and in most of those there will be at least a brief (one or two day) warning. It may be reasonable to schedule surveillance for, say:

- on an annual basis before the start of the wet,
- at a lower level continuing through the wet season, and
- more intensively at the onset of a flood event.

4.2 Requirements

Monitoring refers to things that are measured by survey or by instrumentation, and *surveillance* refers to things that are observed by a documented inspection program. Given the relatively high hazard associated with this levee (potential large economic loss, potential loss of life[14], potential environmental impact), we believe that both are warranted here - surveillance to look for obvious defects or developing problems, and monitoring to check performance against expectation. The monitoring concern is principally pore pressure, so we recommend piezometers only. We suggest:

Surveillance:

i) An annual professional dam engineering inspection and review of data prior to the onset of each wet season.

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[14] Though there will be no actaul "lives at risk" according to the common dam safety definition, which generally requires that there be an occupied dwelling house within the incremental or sunny day dambreak flood footprint.

ii) Documented weekly inspection of the full length of the crest and downstream toe by mine engineering or experienced mine technical staff through the wet season.

iii) Documented daily inspection of the full length of the crest and downstream toe during actual flood events above a trigger level, to be set.

Monitoring:

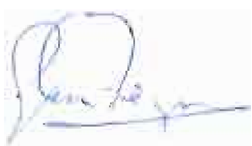
i) Three sections instrumented with piezometers in both the embankment and the foundation, for a total of say 25 tips, all data-logged six-hourly.

5.0 IMPORTANT INFORMATION

Your attention is drawn to the document - "Important Information about your Geotechnical Engineering Report", which is included in Appendix B of this report. This document has been prepared by the ASFE *(Professional Firms Practicing in the Geosciences)*, of which Golder Associates is a member. The statements presented in this document are intended to advise you of what your realistic expectations of this report should be, and to present you with recommendations on how to minimise the risks associated with the groundworks for this project. The document is not intended to reduce the level of responsibility accepted by Golder Associates, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

We would be pleased to answer any questions about this important information from the reader of this report.

GOLDER ASSOCIATES PTY LTD



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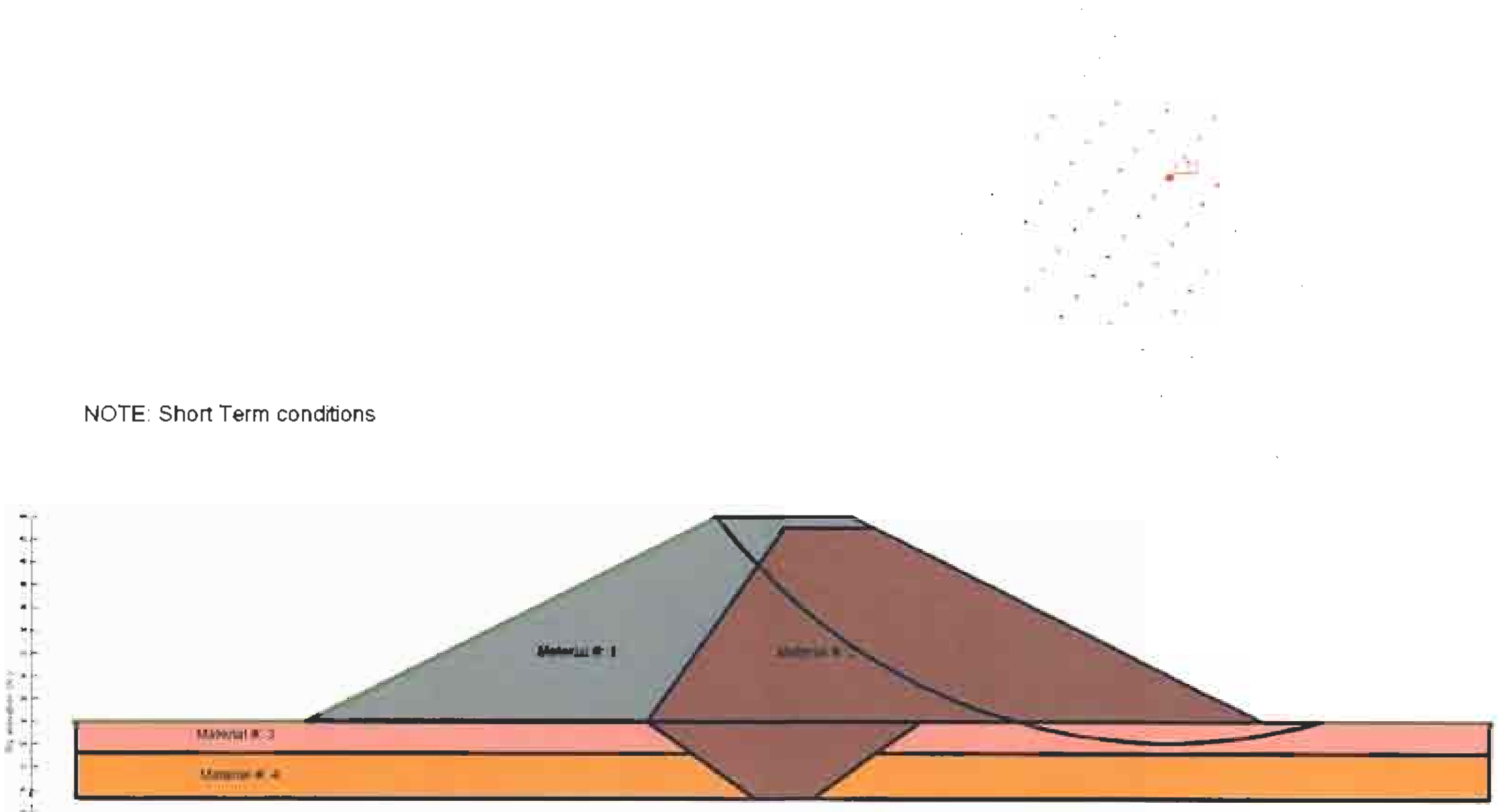
Associate

NOTE: Short Term conditions



Material # 1
Description: Rock Fill
Model: MohrCoulomb
Wt: 20
Cohesion: 0
Phi: 45
Piezometric Line: 0

Material # 2
Description: Compacted silty/Sandy CLAY Fill
Model: MohrCoulomb
Wt: 19
Cohesion: 100
Phi: 0
Piezometric Line: 0

Material # 3
Description: Natural Very Stiff to Hard CLAY
Model: MohrCoulomb
Wt: 19
Cohesion: 5
Phi: 25
Piezometric Line: 0

Material # 4
Description: Alluvial sands and gravel
Model: MohrCoulomb
Wt: 19
Cohesion: 0
Phi: 33
Piezometric Line: 0

Golder Associates	CLIENT: XSTRATA ZINC		PROJECT: MacArthur River Expansion Project	
	DRAWN: VAL	DATE: 22/05/06	TITLE: Section 1a – Downstream Stability End of Construction	
	CHECKED: GSF	DATE: 5/06		
	SCALE: Not to scale		PROJECT No: 06632038	FIGURE No: 1 / REV No: 1 / A4

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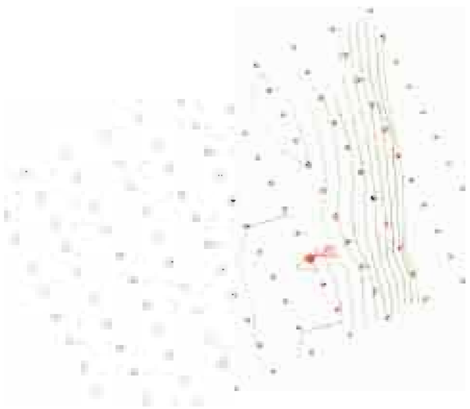
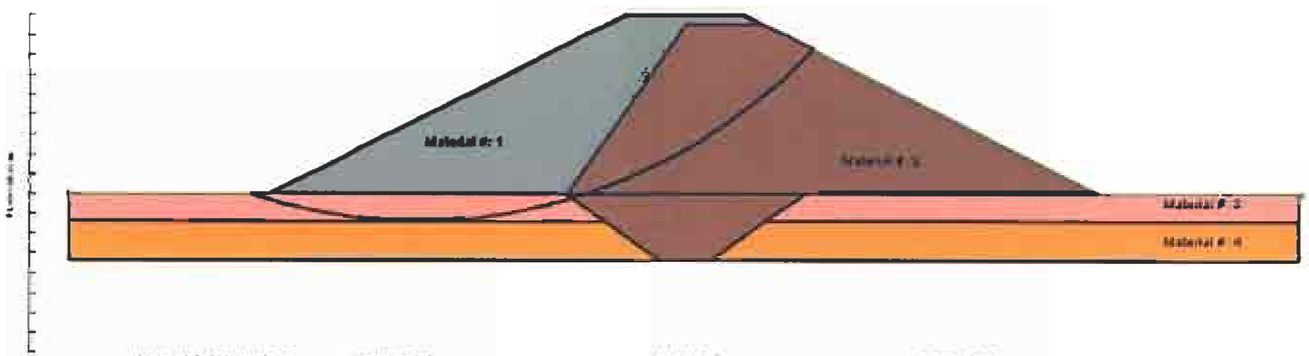
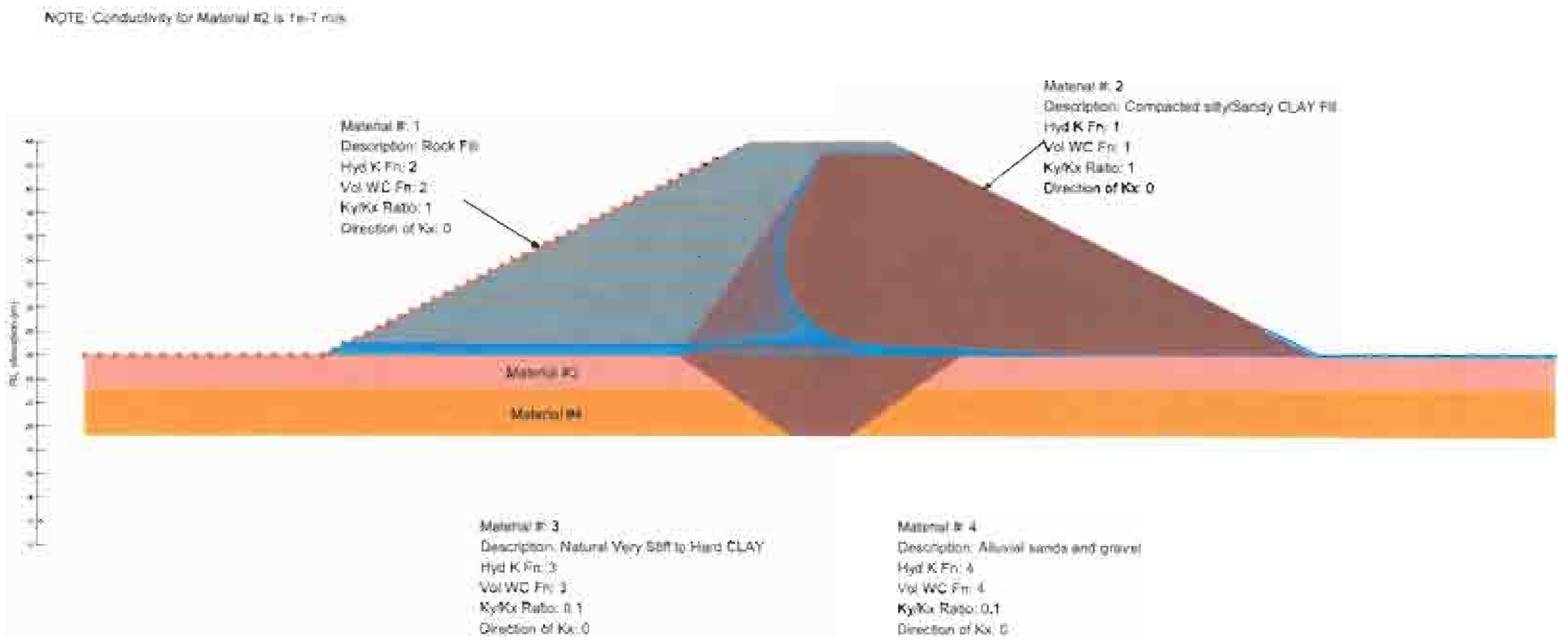


NOTE Short Term Conditions



Material #: 1Material #: 1	Material #: 2	Material #: 3	Material #: 4
Description: Rock Fill	Description: Compacted silty/Sandy CLAY Fil	Description: Natural Very Stiff to Hard CLAY	Description: Alluvial sands and gravel
Model: Mohr/Coulomb	Model: Mohr/Coulomb	Model: Mohr/Coulomb	Model: Mohr/Coulomb
Wt: 20	Wt: 19	Wt: 19	Wt: 19
Cohesion: 0	Cohesion: 100	Cohesion: 0	Cohesion: 0
Phi: 45	Phi: 0	Phi: 25	Phi: 33
Piezometric Line: 0	Piezometric Line: 0	Piezometric Line: 0	Piezometric Line: 0Material #: 4

Golder Associates	CLIENT XSTRATA ZINC			PROJECT MacArthur River Expansion Project	
	DRAWN VAL	DATE 22/05/06		TITLE **Section 1a – Upstream Stability End of Construction**	
	CHECKED GSF	DATE 5/06			
	SCALE Not to scale			PROJECT No 06632038	FIGURE No 2 / REV No 1 / A4



Golder Associates	CLIENT: XSTRATA ZINC		PROJECT: MacArthur River Expansion Project	
	DRAWN: VAL	DATE: 22/05/06	TITLE: **Section 1a : $k_h = 10^{-7}$ m/s Transient Seepage Analysis**	
	CHECKED: GSF	DATE: 5/06		
	SCALE: Not to scale		PROJECT No: 06632038	FIGURE No: 3 · REV No: 1 · A4

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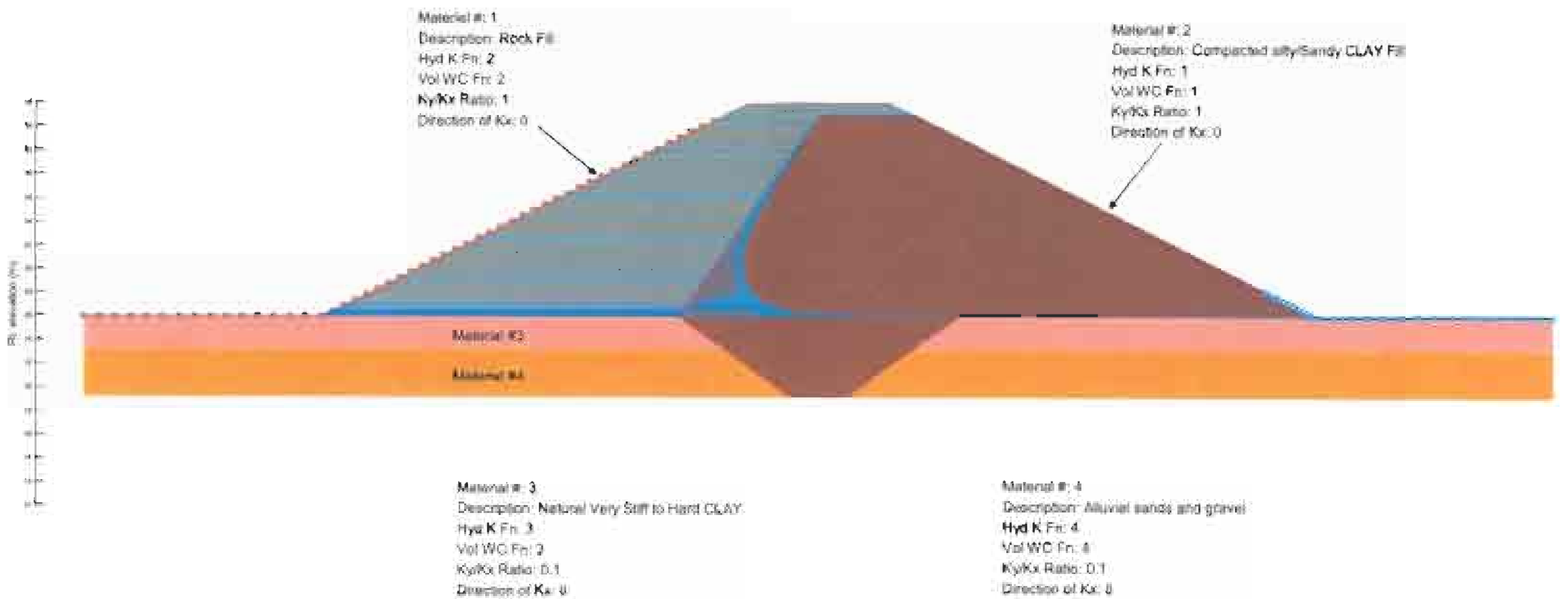
NOTE: Conductivity for Material #2 is 1e-8 m/s



Golder Associates	CLIENT: XSTRATA ZINC		PROJECT: MacArthur River Expansion Project	
	DRAWN: VAL	DATE: 22/05/06	TITLE: Section 1a : $k_h = 10^{-8}$ m/s Transient Seepage Analysis	
	CHECKED: GSF	DATE: 5/06		
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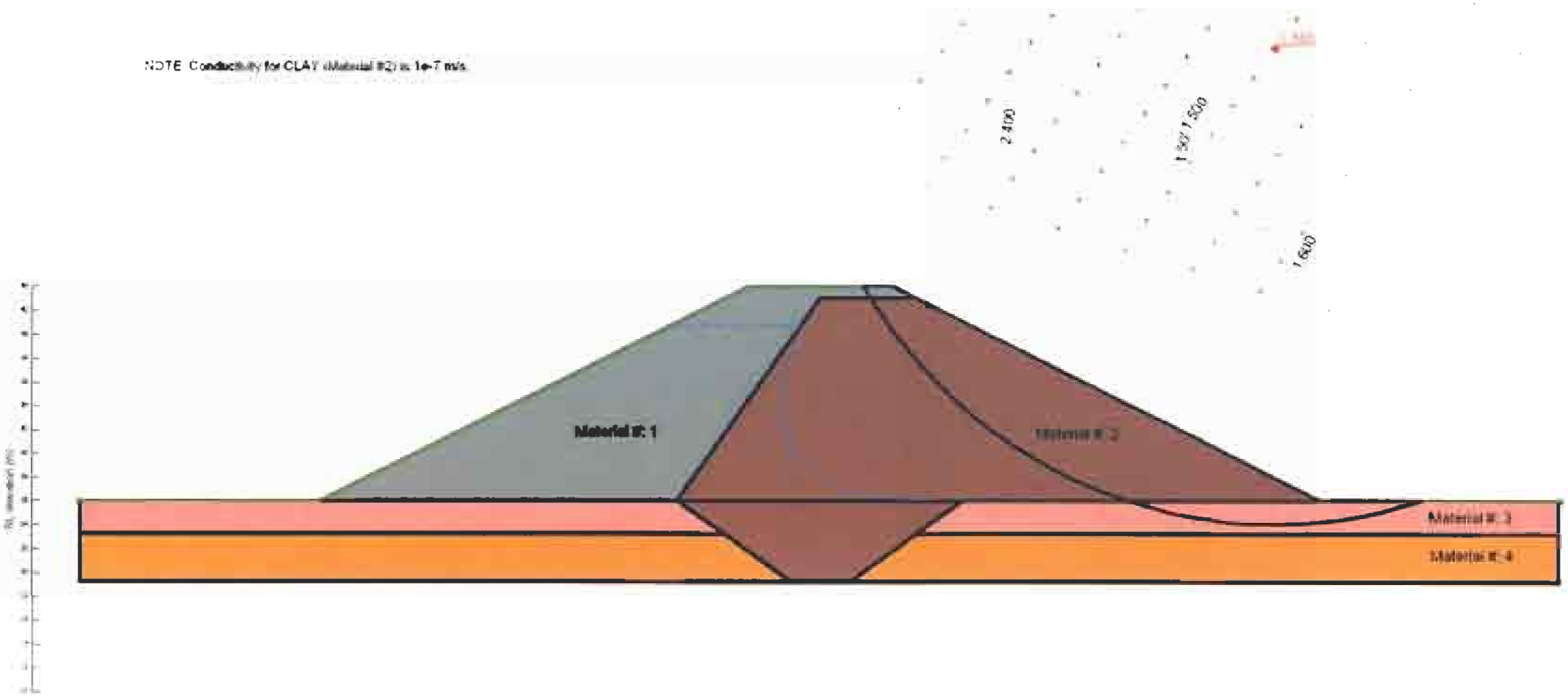
NOTE: Conductivity for CLAY (Material #2) is 1e-7 m/s



Material #: 1Material #: 1
Description: Rock Fill
Model: MohrCoulomb
Wt: 20
Cohesion: 0
Phi: 45
Piezometric Line: 0

Material #: 2
Description: Compacted silty/Sandy CLAY Fill
Model: MohrCoulomb
Wt: 19
Cohesion: 4
Phi: 28
Piezometric Line: 0

Material #: 3
Description: Natural Very Stiff to Hard CLAY
Model: MohrCoulomb
Wt: 19
Cohesion: 5
Phi: 25
Piezometric Line: 0

Material #: 4
Description: Alluvial sands and gravel
Model: MohrCoulomb
Wt: 19
Cohesion: 0
Phi: 33
Piezometric Line: 0Material #: 4

Golder Associates	CLIENT: XSTRATA ZINC		PROJECT: MacArthur River Expansion Project
	DRAWN: VAL	DATE: 22/05/06	TITLE: **Section 1a – Downstream Stability Transient Seepage : $k_h = 10^{-7}$ m/s**
	CHECKED: GSF	DATE: 5/06	
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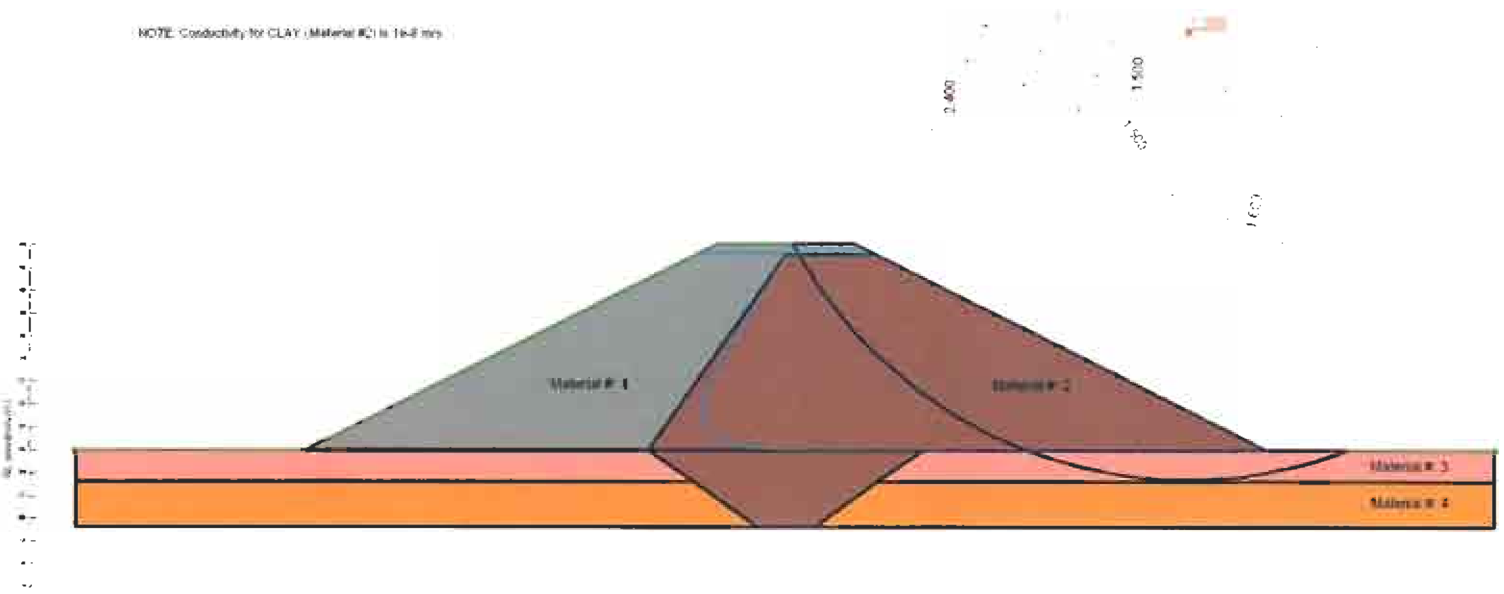
NOTE: Conductivity for CLAY (Material #2) is 1e-8 m/s



Material # 1	Material # 2	Material # 3	Material # 4
Material #: 1Material # 1	Material # 2	Material # 3	Material # 4
Description: Rock Fill	Description: Compacted silty/Sandy CLAY Fill	Description: Natural Very Stiff to Hard CLAY	Description: Alluvial sands and gravel
Model: MohrCoulomb	Model: MohrCoulomb	Model: MohrCoulomb	Model: MohrCoulomb
Wt: 20	Wt: 19	Wt: 19	Wt: 19
Cohesion: 0	Cohesion: 8	Cohesion: 5	Cohesion: 0
Phi: 45	Phi: 28	Phi: 25	Phi: 33
Piezometric Line: 0	Piezometric Line: 0	Piezometric Line: 0	Piezometric Line: 0Material # 4

Golder Associates

CLIENT	XSTRATA ZINC	PROJECT	MacArthur River Expansion Project
DRAWN	VAL	DATE	22/05/06
CHECKED	GSF	DATE	5/06
TITLE	Section 1a : Downstream Stability Transient Seepage – $k_h = 10^{-8}$ m/s		
SCALE	Not to scale	PROJECT No	06632038
FIGURE No	6	REV No	1
A4			

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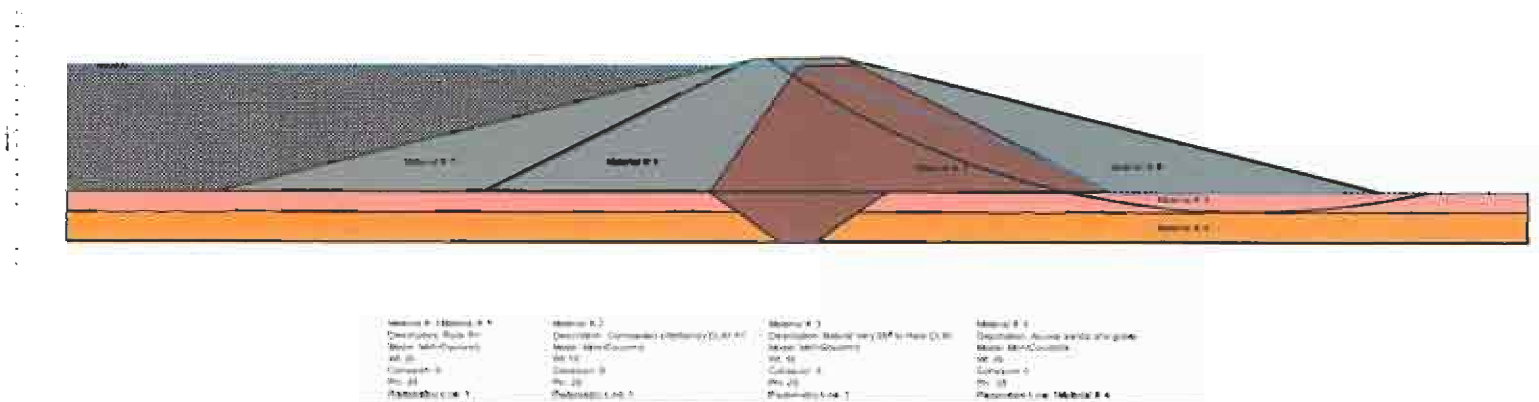

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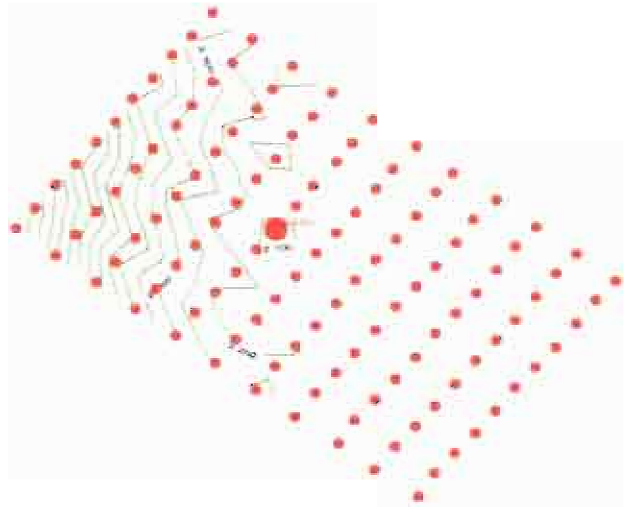
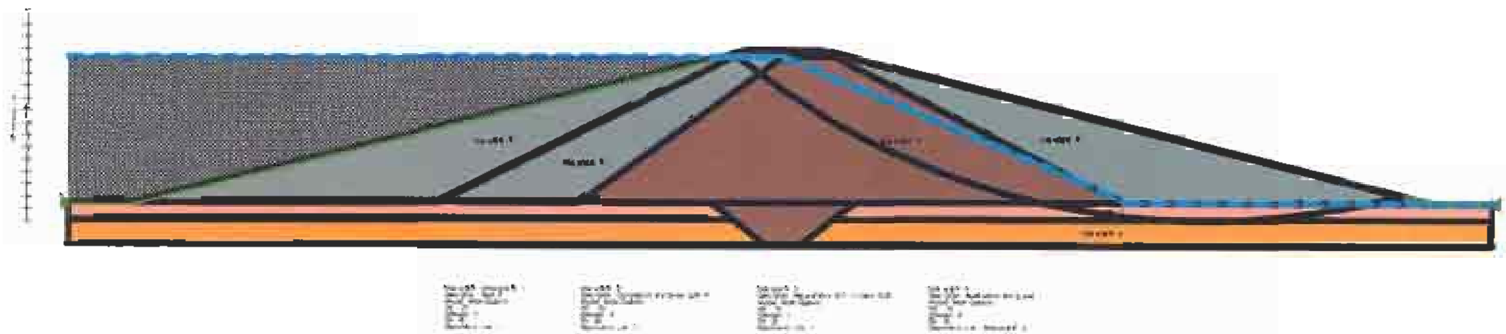


NOTE: Creek Crossing Section



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A4