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BUILDING A BETTER WORLD

DRAFT REPORT

Kintyre Flood Study

Floodplain Modelling and Protection Options

November 2011

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QUALITY ASSURANCE STATEMENT

PROJECT MANAGER:	REVIEWED BY:
Holly Taylor	**Tom Kerr**
PREPARED BY:	**APPROVED FOR ISSUE BY:**
HT, AO, NK, KS, HL, KA	**Gary Clark**

REVISION SCHEDULE

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2.0	15/11/2011	Floodplain Modelling and Protection Options	HT, AO, NK, KS	TK	GC

This document contains information about MWH, particularly about the culture of our organisation and our approach to business, which would be of value to our competitors. We respectfully request, therefore, that it be considered commercially sensitive.

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Draft Report for

Kintyre Flood Study

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1 Introduction

The Kintyre Uranium Deposit is located in the Eastern Pilbara of Western Australia and is approximately 90 km south of the Telfer Gold Mine. Original resource investigations were undertaken by CRAE and subsequently by Rio Tinto over a period extending from the 1980‟s to 2006. Rehabilitation of all disturbed areas and associated infrastructure was undertaken in 2002 by Rio Tinto Exploration (RTX). In August 2008 a joint venture consortium between Cameco Australia Pty Ltd (70%) and Mitsubishi Development Pty Ltd (30%) subsequently acquired the Kintyre deposit.

To follow on from previous works undertaken by Rio Tinto, Cameco is undertaking a Pre-Feasibility Study (PFS) to determine the viability of developing the Kintyre deposit. Hydrological studies will form a part of the supporting studies which will enable the potential for future development of the Kintyre Project to be assessed.

In July 2009 MWH Australia Pty Ltd (MWH) completed an assessment of the existing surface water information, undertaking a gap analysis and provided recommendations on how surface water management should proceed for the Kintyre Project. This involved the review of documents detailing surface water monitoring conducted from 1988 - 1992 and a field visit by MWH in May 2009 to the Kintyre Project site.

In September 2011 MWH was appointed by Cameco to undertake a flood study to determine the nature and extent of the potential flooding that could occur at Kintyre. This included the conceptual design and assessment of a flood protection embankment. This document presents the methods and outcomes of this work.

2 Scope of Work

This flood study has been prepared to assist Cameco in mitigating the impacts of major flooding of their mining and associated infrastructure. Using data collected during a field program (1988 – 1992), regional hydrological relationships and LIDAR (DTM) data, mathematical computer models were developed and interpreted to assess the potential for flooding in the catchment. The study objective was to define creek behaviour in terms of flows, levels and flooding behaviour for flood frequencies ranging between a 10 year interval (ARI) up to the probable maximum flood (PMF).

Due to the remote nature of the Kintyre site and the corresponding lack of hydrological data, particularly the absence of any historic data from large floods, the approach to assessing the potential nature and extent of flooding has relied on the development and interpretation of mathematical computer models. Flood behaviour was defined using a computer based hydrological model of the catchments and a hydraulic model of the streams and flood plain. The hydrological model was a runoff routing model, which was initially tested against recorded rainfall and runoff data, where observed data was not sufficient, regional design parameters were used. Design storms were then applied to the model to generate discharge hydrographs within the study area. These hydrographs constituted the upstream boundary and tributary inflow inputs to the hydraulic model.

A fully dynamic network hydraulic model was developed for the hydraulic analysis to account for the time varying effects of flows from the tributary streams and the routing effects of the floodplain storage. A two-dimensional model was chosen which allowed for the interaction of flows between the channel and the floodplain. The model was then used to produce water surface profiles, discharge hydrographs and average velocities of flow for the design events under existing conditions / base case.

A flood protection embankment has been proposed to provide additional flood protection to the Kintyre mine pit and important infrastructure. A conceptual design is presented as a part of this study. The conceptual embankment was incorporated into the hydraulic model and design flood scenarios were modelled to determine the embankment size required to prevent flooding of infrastructure and to assess downstream impacts.

The modelling process is illustrated in

Figure 2-1. The limitations of the models and the recommendations for further studies are highlighted in this report.

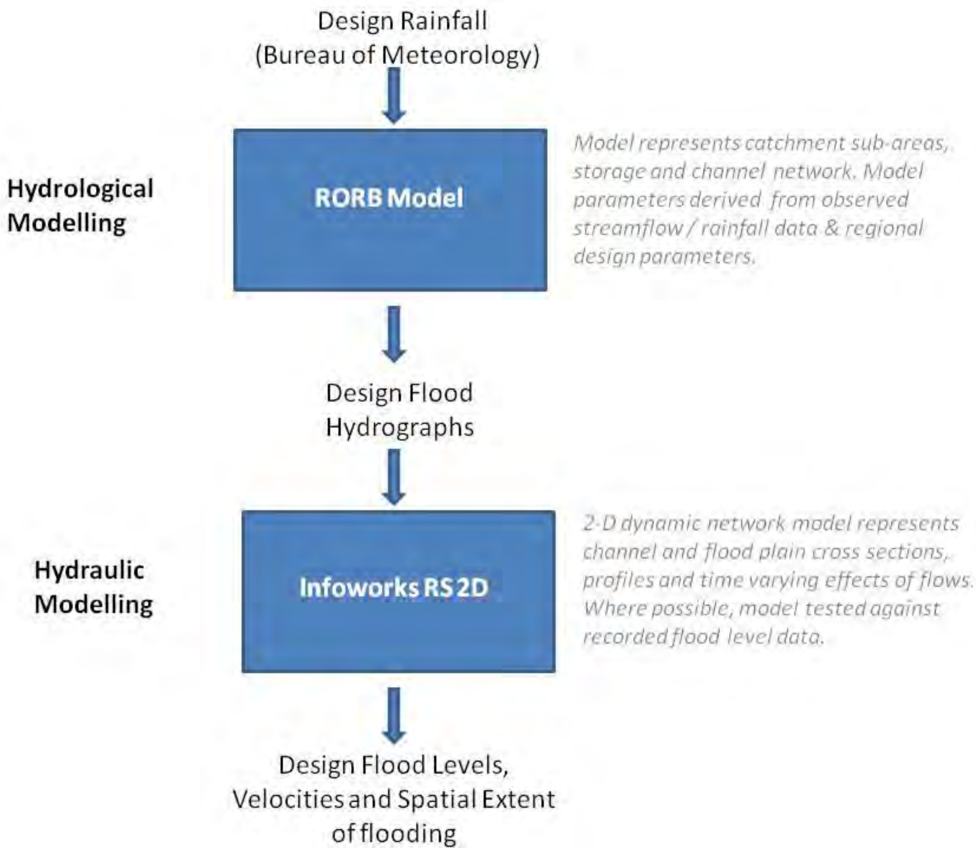

Figure 2-1 Flood Modelling Process

3 Catchment Characteristics

3.1 Location

The Kintyre Project is located in the remote East Pilbara region of Western Australia on the edge of the Great Sandy Desert. The Project is located approximately 1,200 kilometres north-east of Perth and 90 kilometres south of the Telfer mine site. A location map is shown in Figure 3-1.

Figure 3-1 Location Map of Kintyre Project

3.2 Climate

The Project is located in a semi-arid climatic zone subject to monsoonal influences. The region experiences a climate of extremes, where severe droughts and major floods can occur at close intervals.

The longest rainfall record for the region is located at the Telfer Climate station, where rainfall data is available from 1974 to present. The annual rainfall at Telfer is highly erratic ranging from 114 – 817mm; the long term average is 367 mm/year. Most rainfall occurs during the summer months as a result of scattered thunderstorms and occasional tropical cyclones. Mean monthly rainfall is presented in Figure 3-2. Very intense rainfall events mean large amounts of rainfall can fall in shorts periods; the highest daily rainfall recorded at Telfer was 199.6 mm in March 2004 (Bureau of Meteorology, 2011).

During the summer months (November to February), weather is characterised by the presence of hot low-pressure systems over the region resulting in clear skies and hot temperatures. The maximum and minimum daily temperatures at Telfer are contained in Figure 3-2.

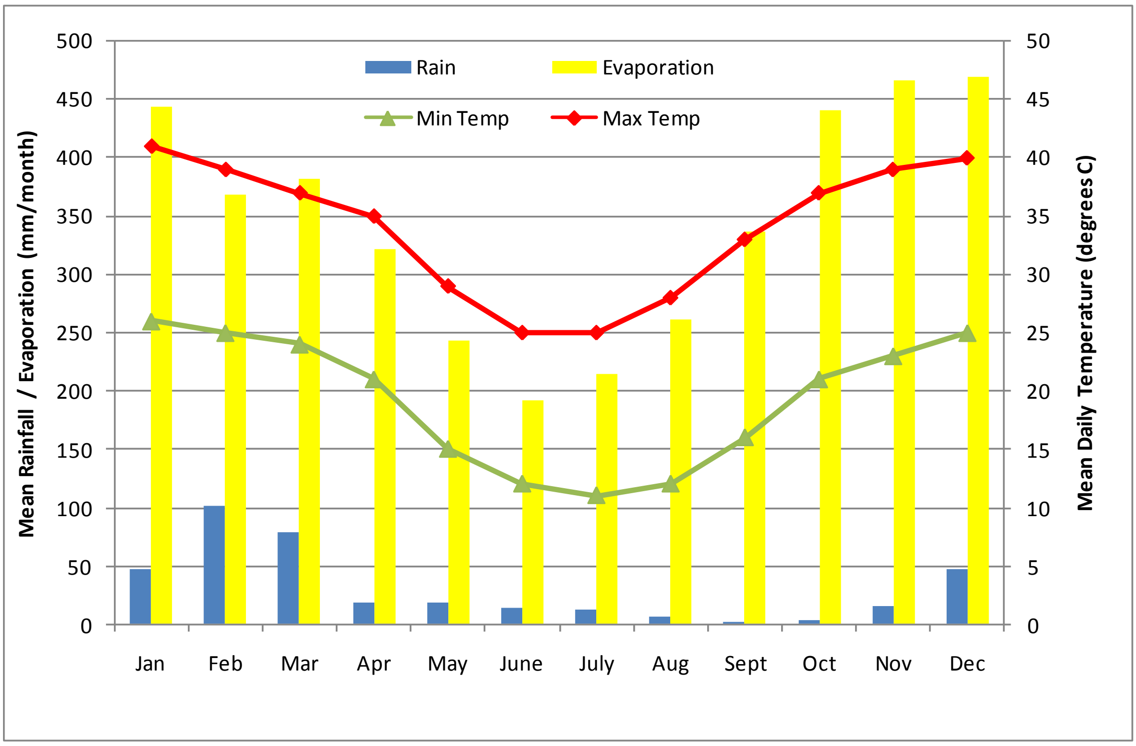

Figure 3-2 Climate Data at Telfer

Figure 3-3 contains the number of rain days greater than 50 mm at Telfer (1974 – 2009). Large rainfall events are most likely to occur during February and March which is related to the prevalence of tropical lows and cyclones in the region. Whilst floods can potentially happen at any time of year, historically the greatest flood potential has been in February and March.

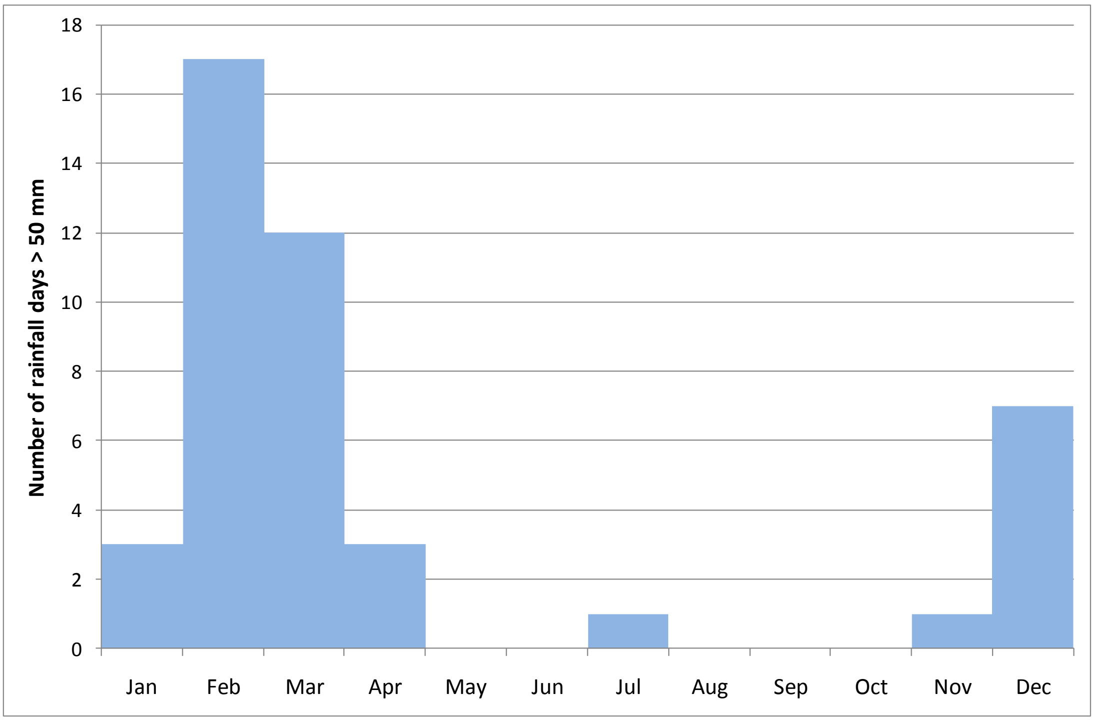

Figure 3-3 Number of rain days over 50 mm/day at Telfer

Evaporation in the region is the highest in Australia (

Figure 3-4). The average annual potential evaporation is over 4000 mm and is highest in the months from October to January (Figure 3-2); this greatly exceeds the average annual rainfall of 367mm.

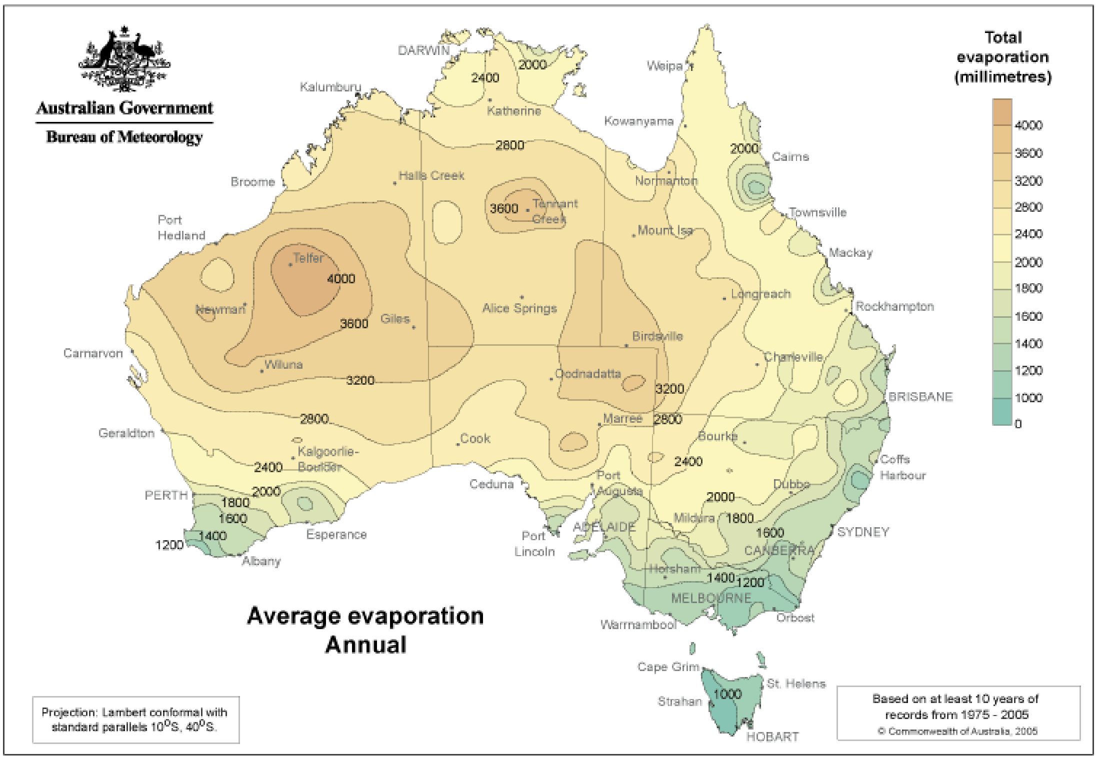

Figure 3-4: Average Annual Evaporation

Source: Commonwealth Bureau of Meteorology

3.3 Topography

The physiographic features of the Rudall region are generally related to glacial activity in the early Permian. Resistant rock types form plateaus while softer rocks form low rocky outcrops, surrounded by Aeolian sand. The plateau paleo-surface was formed by glacial erosion and the Permian Paterson Formation occurs as remnant mesa outcrops and as sedimentary infilling in broad U shaped valleys.

3.4 Hydrological Setting

The Project area is classified as being part of the Sandy Desert river basin within the Western Plateau drainage division (AWRC, 1975). The Sandy Desert River Basin is an internally draining basin (see

Figure 3-5).

The Kintyre project lies between the two tributaries of the Yandagooge Creek, referred to as the South Branch and the West Branch. The drainage in the upper reaches of the creeks occurs within relatively incised channels which widen to include significant flood plain storage in the area surrounding the Project area. The tributaries converge immediately downstream of the project site and flow north to the Coolbro Creek. Coolbro Creek then follows an easterly path into the Great Sandy Desert where the drainage eventually dissipates into the sandy environment. The Yandagooge Creek channels surrounding the Project area are well defined, approximately 1 – 2 metres deep and have coarse sand and gravel beds, characteristic of rivers in the Pilbara.

The creeks in the region are generally dry and flow only in response to heavy rainfall, when they may flow for several days. Semi-permanent surface water pools exist to the north of the Project area in the northern, central and southern creeks of the Coolbro Hills (Dames and Moore, 1996).

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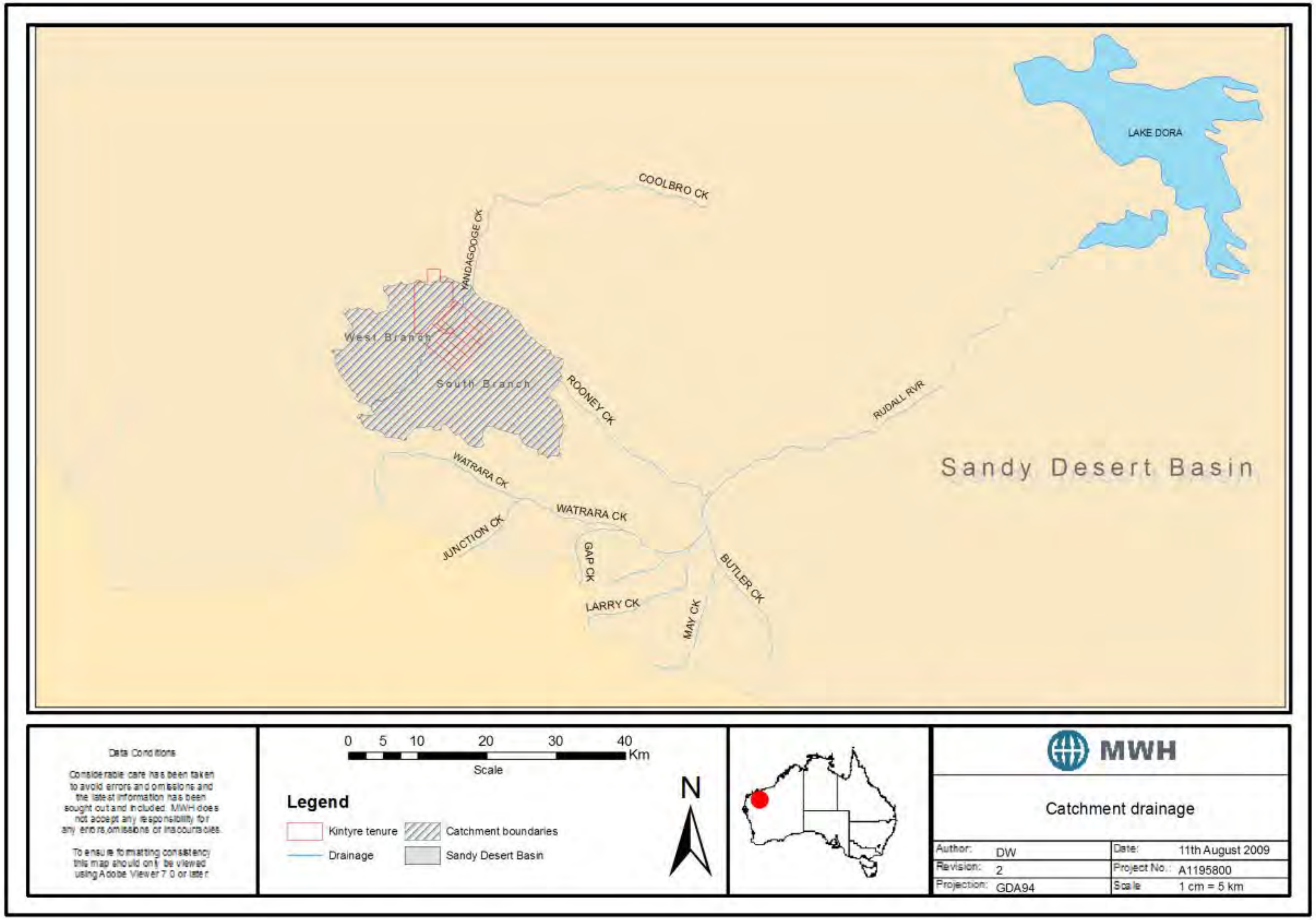

Figure 3-5: Major drainage in the Sandy Desert Basin

4 Data Review

4.1 Overview

There has been very little hydrological and meteorological data recorded at the Kintyre project site. The data that is available was recorded during previous mining investigations (primarily from 1988 – 1992). The Commonwealth Bureau of Meteorology weather station at Telfer (approximately 65 km north of Kintyre) has been operational since 1974 and has the longest rainfall and climate record in the region.

The data recorded at the site was not available in an electronic format and was not quality coded. The data relevant to hydrological modelling of the Yandagooge Creek, concurrent stream flow and rainfall record, has been collated and digitised for this flood study.

4.2 Catchment Delineation

The catchment size of the Yandagooge Creek is a critical determinant in the magnitude of floods at the Kintyre project site. Using topographical information, the catchment area of the South Branch has been assessed to be approximately 300km^2 and the West Branch approximately 170 km^2.

The major runoff generating areas are the sandstone and quartzite outcrops. The basic geology at Kintyre is presented in Figure 4-1. Previous hydrological investigation suggested that the more impermeable soil in the West Branch produces more runoff per unit area than the South Branch (Dames and Moore, 1996).

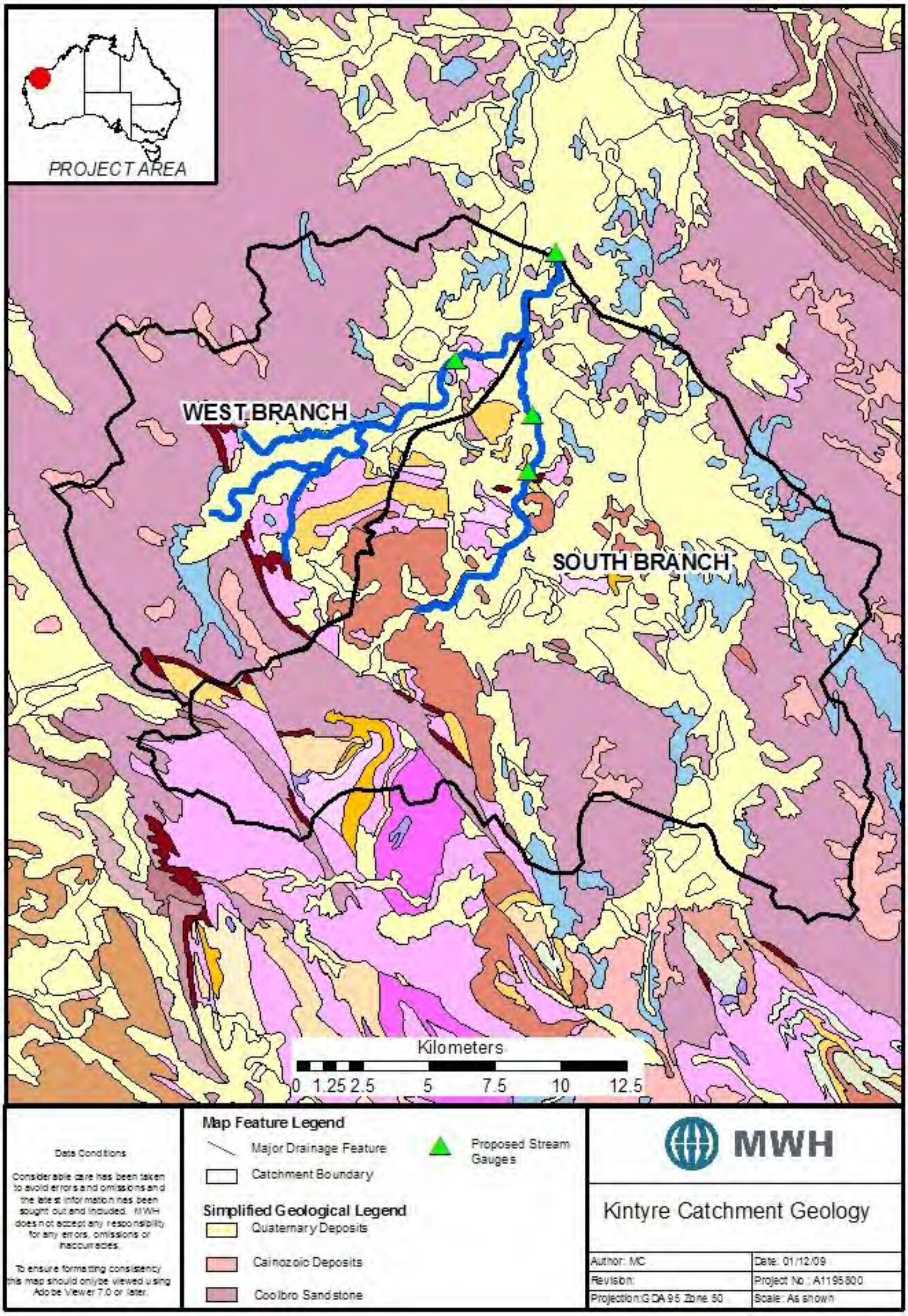

Figure 4-1 Kintyre Catchment Geology

4.3 Rainfall

A summary of the rainfall records near Kintyre is shown in Table 4-1 and a location map is contained in Appendix A - River & Rainfall Monitoring Stations, 1987-1992. The only high quality long-term rainfall record in the region is at Telfer, which only has a relatively short 36 year period of record. The longest site record was the daily data at Camp Tracy, whilst the most comprehensive (continuous data) record was from the pluviograph at the Central Weather Station (CWS). The continuous data set is of most relevance for rainfall – runoff modelling. A number of rainfall gauges were installed in 1990 – 1992 to assess the rainfall variability across the Kintyre tenement.

Table 4-1 Summary of rainfall record at Kintyre Project (1987 – 1992)

Station Name / No.	Frequency of record	Distance from Kintyre	Period of Record
013030 Telfer Aero	BoM daily	65 km	1974 - present
004103 Nifty Copper Mine	BoM daily	85 km	1996 – present Unreliable
Parnngurr	BoM daily	80 km	2004 – present Unreliable
Camp Tracy	Daily Read	*At site*	1987 - 1992
	Pluvio (continuous data)	*At site*	1996 – 1998 Unreliable
Central Weather Station (CWS)	Pluvio (continuous data)	*At site*	1988 – 1990
SE1, SE2, WS1, WN1, WN2	Monthly / Event based readings * Sites put in to investigate rainfall variability	*At site*	1990 - 1992

4.3.1 Kintyre Rainfall Record

The rainfall data collected at Kintyre (1987 – 1992) has been grouped into events and is contained in Table 4-2

Table 4-2 Rainfall Events recorded at Kintyre 1988 - 1992

Year	Event Number	Total Rainfall	Rainfall Station
Feb 1988	1	86.8	CWS
Feb 1988	2	57.8	CWS
Mar 1988	3	138.8	CWS
Jun 1988	4	138.8	CWS
Jun 1988	5	22.2	CWS
Dec 1988	6	17.0	CWS
Apr 1989	7	12.6	CWS
Jan 1990	8	79.6	CWS

Year	Event Number	Total Rainfall	Rainfall Station
Jan 1991	9	25.0	Camp Tracy
June 1991	10	40.8	Camp Tracy
Jan 1992	11	84.0	Camp Tracy
Mar 1992	12	35.0	Camp Tracy

Rainfall versus Average Recurrence Interval (ARI) can be characterised by a skewed log-normal distribution. The rate of recurrence of Intensity-Frequency-Duration (IFD) rainfall is represented by an Average Recurrence Interval (ARI). For example for a rainfall event having a 10 year ARI there will be a rainfall event of equal or greater magnitude once in 10 years on average.

The IFD curve for the Kintyre Project site was sourced from the Bureau of Meteorology (BoM) and is plotted, together with the rainfall events 1 to 8 from Table 4-2, in Figure 4-2. The figure shows that in 1988 there were two events with an ARI close to 10 years.

It is important to note that an ARI of, say, 100 years does not mean that the event will only occur once every 100 years. In fact, for each and every year, there is a 1% chance (a 1 in 100 chance) that the event will be equalled or exceeded (once or more than once).

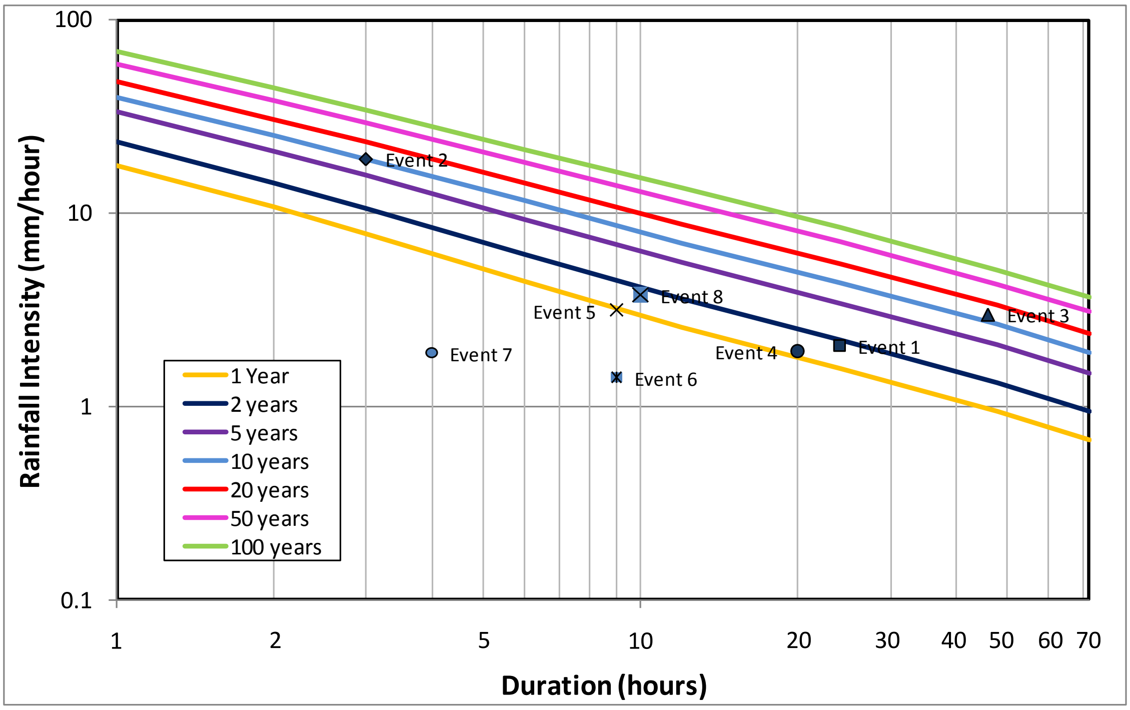

Figure 4-2 Recorded Rainfall Events (1 to 8) compared to Design Rainfall

4.3.2 Site Rainfall Variability

Rainfall in the Kintyre area has been noted to be extremely localised with downpours producing high rainfall in one area and little or no rain produced a few kilometres away (Dames & Moore, 1990). The rainfall recorded at one gauge may be localised and not representative of other parts of the catchment.

From 1990 – 1992 rainfall variability was investigated with the installation of five additional rain gauges (SE1, SE2, WS1, WN1 and WN2). A location map is shown in Appendix A - River & Rainfall Monitoring Stations, 1987-1992. Three rainfall events were captured during this period and the corresponding rainfall is shown in Figure 4-3. The data is too limited to discern any statistical trends however the data does show that; WN1 is consistently greater than the average rainfall and WN2 is consistently less that the average.

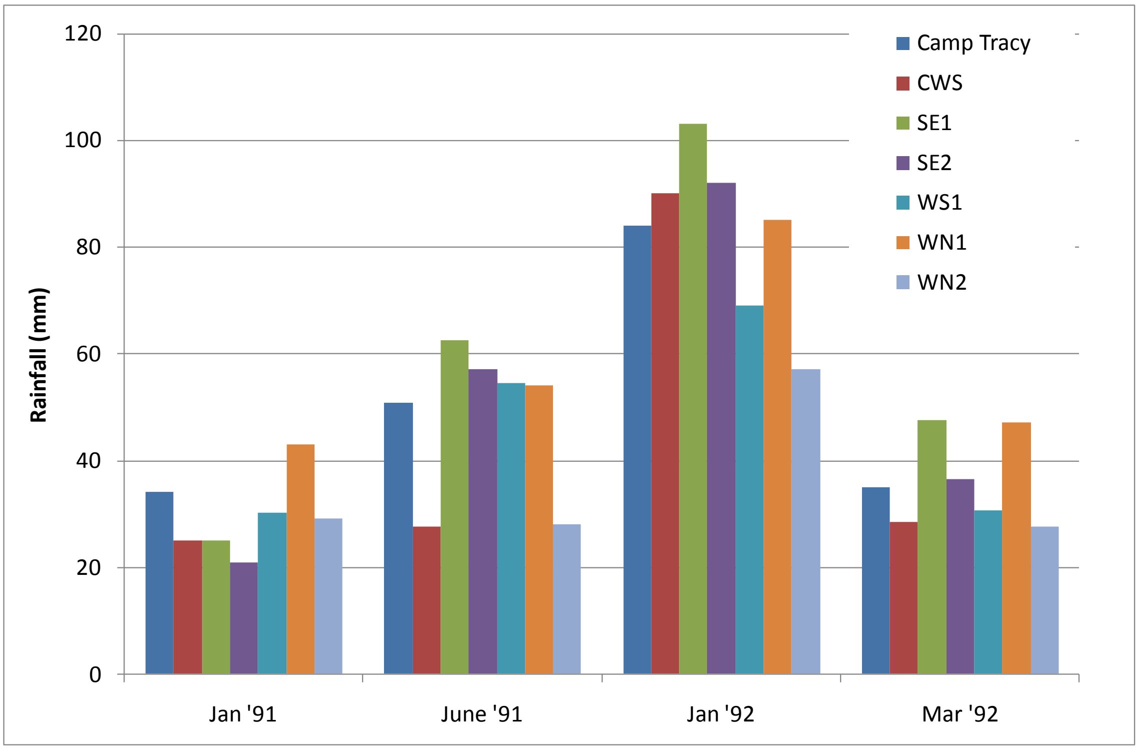

Figure 4-3 Rainfall variability at Kintyre 1991 - 1992

4.3.3 Telfer Rainfall Record

The Telfer climate station shows a good annual correlation to the rainfall recorded at Kintyre, for the short period of record available for comparison (Figure 4-4). The Telfer record is useful for looking at longer term regional rainfall trends.

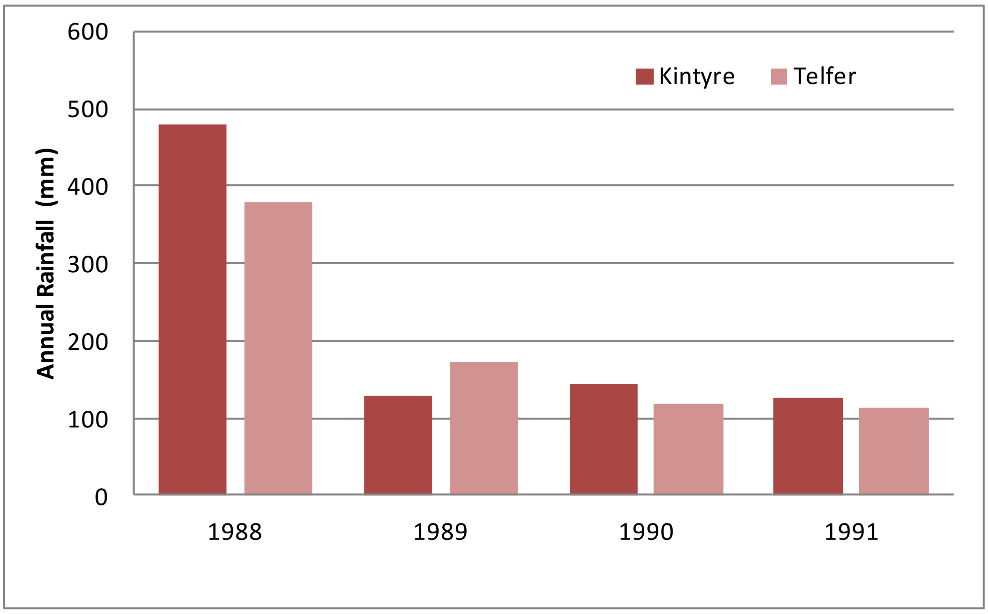

Figure 4-4 Comparison of rainfall at Telfer and Kintyre

The annual rainfall recorded at Telfer has been ranked from driest to wettest in Figure 4-5. The years for which data was collected at Kintyre are shown in yellow. These years were average or below average, with two of the years being the driest on record at Telfer (1990 and 1991). The stream flow data collected during this period also corresponds to below average flow, which makes it of limited value in flood analysis. The most useful data for flood analysis is collected during wetter years when the antecedent moisture in the catchment is higher than at other times.

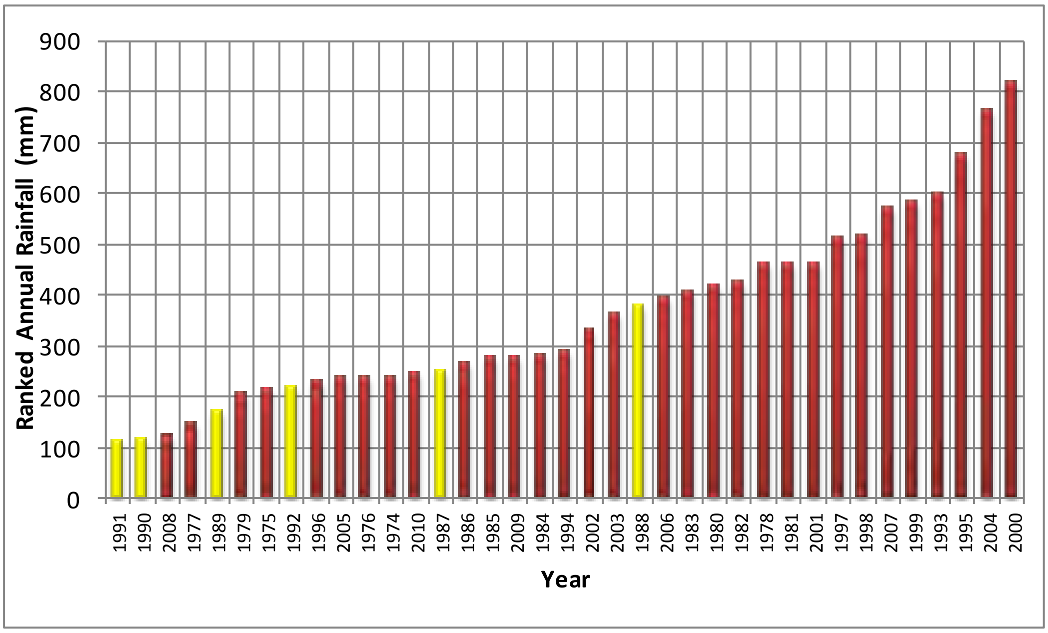

Figure 4-5 Ranked annual rainfall recorded at Telfer

4.3.4 March 2004 - Telfer Rainfall Event

The largest rainfall event recorded at Telfer was in March 2004 as a result of Cyclone Fay, which resulted in wide spread flooding. The road access to the town was cut for three months and a new causeway had to be constructed. Heavy rainfall was recorded along the track of the cyclone, with 372 mm of rainfall recorded in 3 days (see Table 4-3). The estimated ARI of this event and the embedded durations of 24 and 48 hours ranges from 366 to 939 years. There is no record of the stream flow at Telfer or Kintyre during this event.

Table 4-3: Cyclone Fay rainfall duration and ARI

Duration	Cyclone Fay Rainfall	Estimated ARI	Estimated AEP
24 hours	338mm	939 years	0.11%
48 hours	363mm	446 years	0.22%
72 hours	372mm	366 years	0.27%

Source (Parsons Brinckerhoff Australia Pty Ltd, 2004)

4.4 Level and Flow Data

There is very limited surface water data available for the Pilbara region. Technical difficulties, including the remoteness, the complex unconfined nature of flow and the mobility of sediment in creeks in the Pilbara has limited the development of gauging stations and the accuracy of some of the data that has been collected. The Sandy Desert River basin is not gauged by the Department of Water and there is no published data listed in the Australian Water Resources Station Catalogue.

The available data and local knowledge show that local drainages flow only during and for relatively short periods after significant rainfall. Runoff is highly variable and is, on average, a small proportion of rainfall but increases with higher and more prolonged rainfall. Intense cyclonic rainfall can produce major, widespread flooding particularly in the lower reaches of drainage lines. Sustained baseflow is generally negligible and the creeks typically recede rapidly and stop flowing soon after the cessation of rain.

River gauging stations recorded water levels in the South and West branches of the Yandagooge Creek from 1987 to 1992. The location of the stream gauging stations is contained in Appendix A - River & Rainfall Monitoring Stations, 1987-1992. Data loggers were intended to record all stream flow events during this period, however, the equipment did not always perform as intended and the data set is incomplete.

Table 4-4 contains the peak stream levels recorded in the South and West branches. The peak levels were collated from loggers, peak level indicators or estimated from the rising stage samplers. The corresponding flow has been estimated from discharge rating curves, developed using the HEC-2 computer model (Dames and Moore, 1991).

Twelve events were recorded, six of which occurred during 1988, which was the wettest year during monitoring; however, it corresponds to an average year when viewed within the context of long-term data at Telfer, indicating that more events or events of larger magnitude could be expected in wetter years.

Table 4-4: Streamflow Events in the Yandagooge Creek (1988 – 1992)

Year	Event Number	Total Rain	Maximum Level in South Branch (m)	Estimated Peak Flow South (m3/s)	Maximum Level in West Branch (m)	Estimated Peak Flow West (m3/s)
1988	1	86.8	0.41	4.2	1.23	28.1
1988	2	57.8	0.03 *	0	0.64*	5.4
1988	3	138.8	0.95	**30.1**	1.90	**84.4**
1988	4	138.8	0.35 -0.65*	14.7	0.95 – 1.35*	35.6
1988	5	66.2	0.35 -0.65*	2.8	0.95 – 1.35*	14.6
1988	6	17.0	0.00 ?	0.0	0.74 ?	7.8
1989	7	12.6	0.20 ?	0.4	0.52 ?	7.9
1990	8	43.0	0.05	0.0	0.04	0.0
1991	9	25.0	0.11	0.4	0.39	4.3
1991	10	40.8	0.38	4.1	0.35	3.5
1992	11	48.0	0.31	2.7	0.86	20.5
1992	12	35.0	0.32	3.0	0.40*	4.7

„* Data logger failure ? Raw time-series data was not available

The maximum streamflow was recorded during Event 3 in 1988.

Of the twelve flow events that were recorded, concurrent continuous rain and stream flow record is only available for three events (events 1, 3 and 8). The only events with sufficient quality data to use in the hydrological model calibration were Event 1 and Event 3; the hydrographs from these events are shown in Figure 4-6 and Figure 4-8.

4.4.1 Event 1 Data

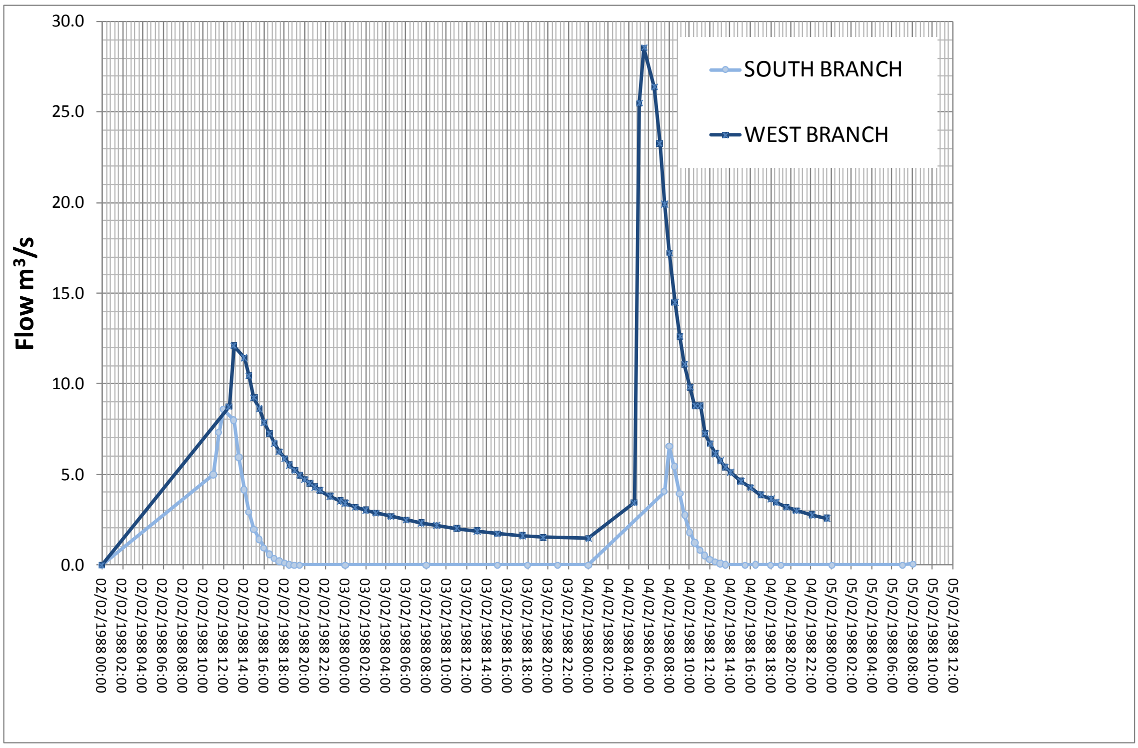

Figure 4-6 Hydrographs for Yandagooge Creek Event 1

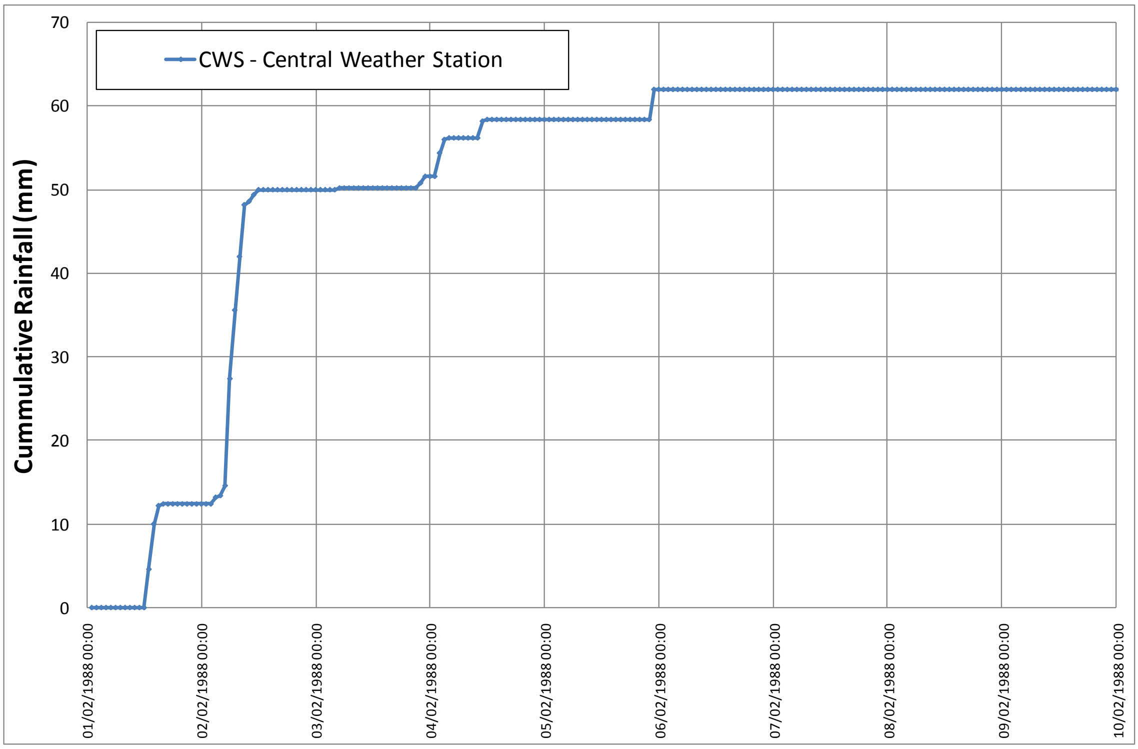

Figure 4-7 Cumulative rainfall data at CWS for Event 1

4.4.2 Event 3 Data

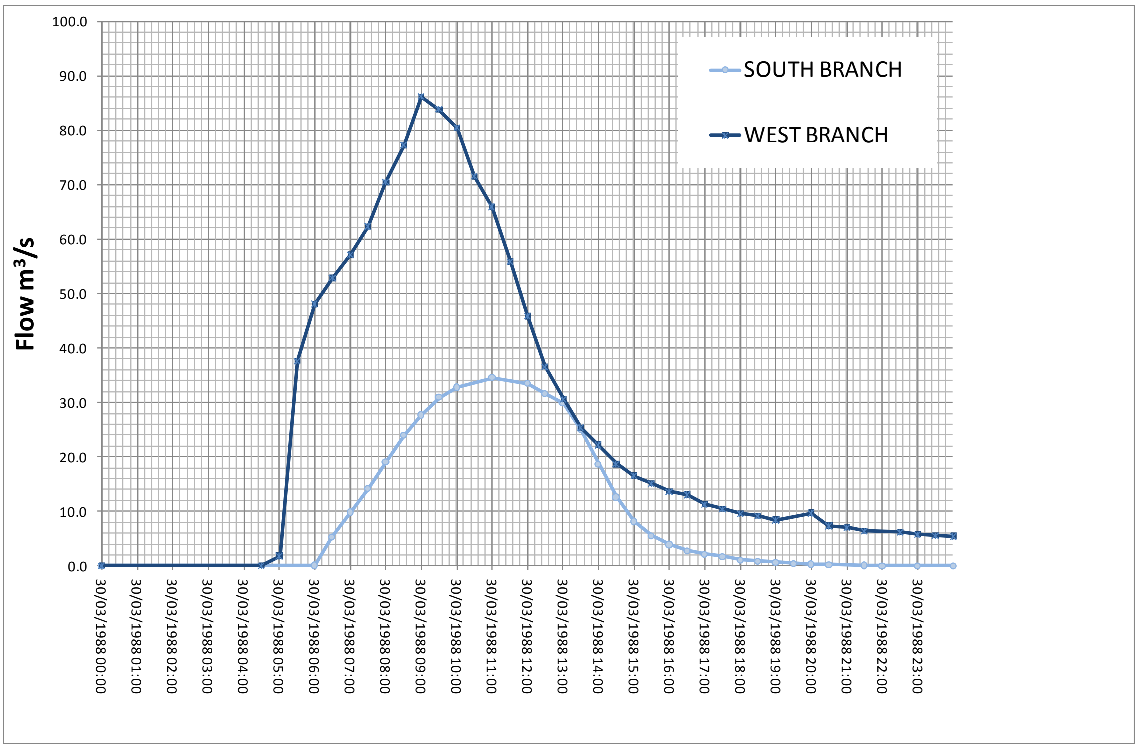

Figure 4-8 Hydrographs for Yandagooge Creek Event 3

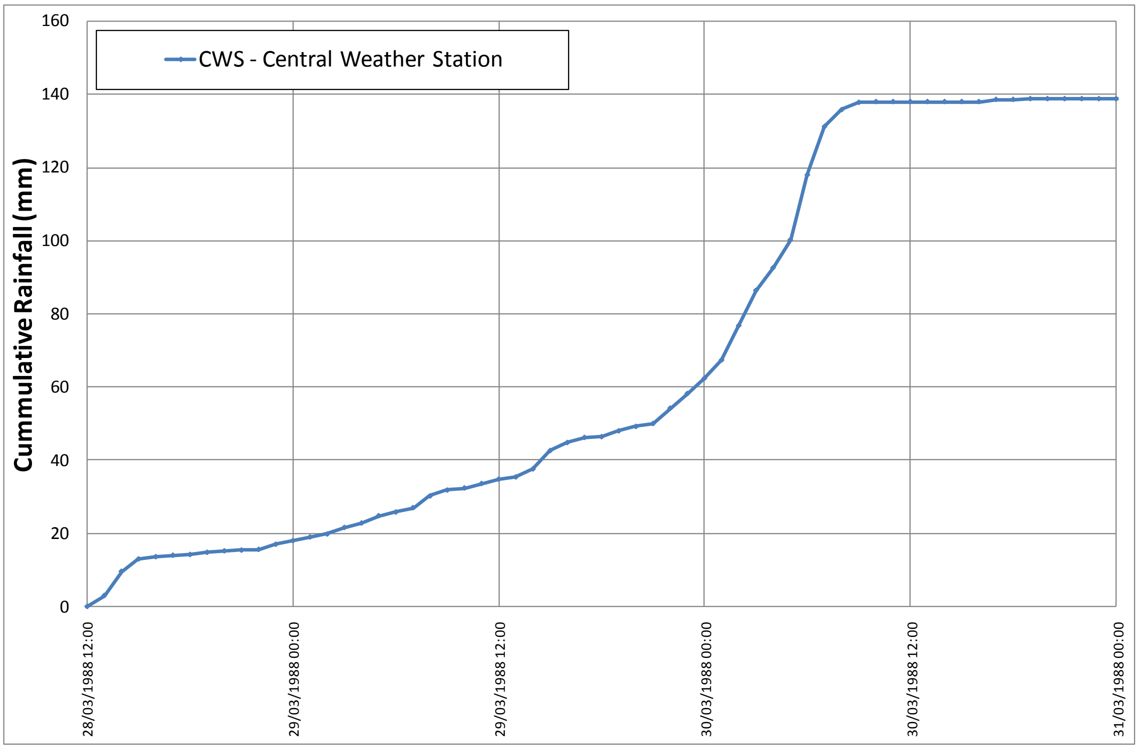

Figure 4-9 Cumulative rainfall data at CWS for Event 3

4.4.3 Telfer Event – March 2004

The rainfall recorded at Telfer in March 2004 is far in excess of any other rainfall events that have been recorded in the region. The cumulative rainfall plot of the event is shown in Figure 4-10. There is no record of the stream flow at Telfer or Kintyre during this event.

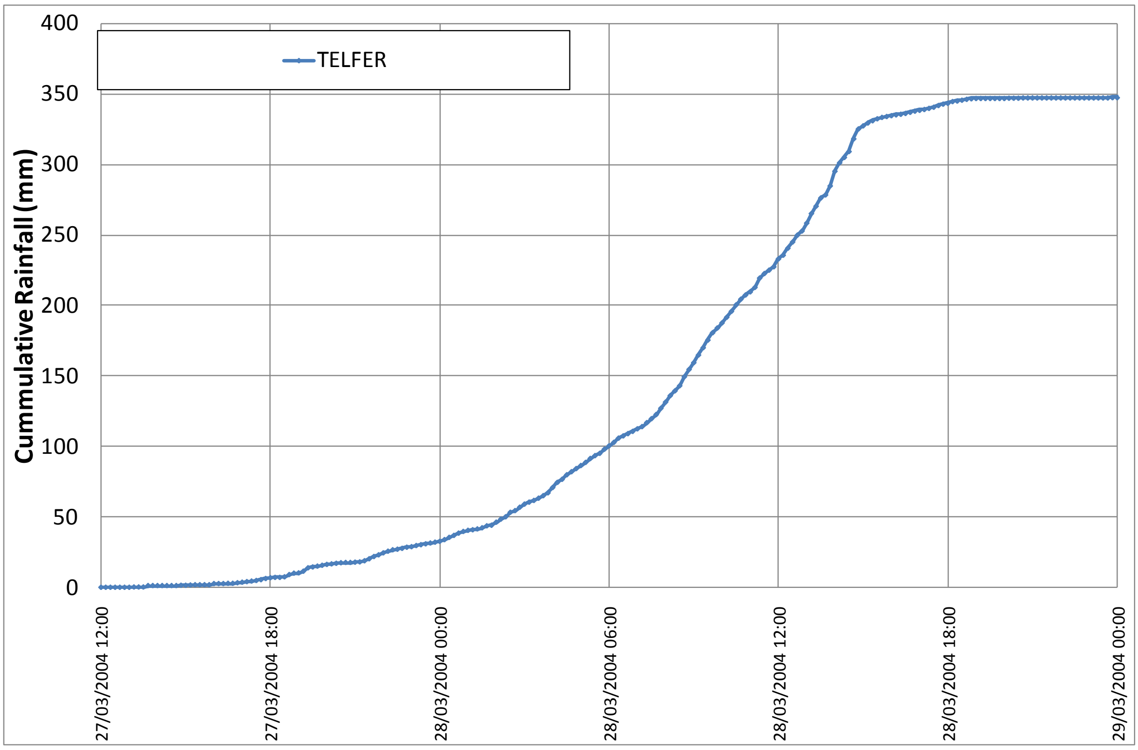

Figure 4-10 Cumulative rainfall data at Telfer in March 2004

4.4.4 Flood marks and anecdotal evidence

No anecdotal evidence of flooding or flood marks relating to specific floods in the Yandagooge Creeks was found during the course of this study.

Previous recommendations that flow level recorders and rising stage samplers be implemented (MWH, 2009), have not been actioned.

5 Hydrological Modelling

5.1 Introduction

An initial loss-continuing loss rainfall-runoff model was developed to estimate flood volumes and flows in the Yandagooge Creek. The model subtracted losses to evaporation and infiltration from rainfall to give rainfall-excess which was routed through the catchment and channel network to produce hydrographs. The model was used to determine design event peak flow hydrographs and investigate hydrological characteristics in the Yandagooge Creek at Kintyre.

5.2 Model Selection

The model was developed using *RORB* modelling software. *RORB* is an industry standard hydrological modelling package which has been widely used for hydrological design throughout Australia. *RORB* models can be set up with limited data, making it suitable for application for the Kintyre.

5.3 RORB Model Processes

5.3.1 Rainfall excess loss model

An initial loss – continuing loss model was used to determine the rainfall-excess. Initial loss is a threshold process where no runoff is assumed to occur until the initial loss capacity has been satisfied. The continuing loss is a constant loss rate. The continuing loss rate is a capacity rate of loss that occurs only if rainfall is equal to or greater than that rate. For less intense rainfall periods, the loss is equal to the rainfall.

5.3.2 Storage routing

The *RORB* model represents the channel network by a network of model storages with a similar arrangement to the actual river network. The purpose of representing the catchment as sub-areas is to model the storage effects within the catchment. Only the significant stream channels were explicitly modelled and the storage effects of smaller channels and overland flow were lumped in with the storage effects of the more significant channels. The sub-area rainfall-excess is assumed to enter the river network near the centroid of the sub area, where it is added to any existing flow in the channel and the combined flow is routed through the subarea.

The storage discharge relationship used to model the catchment storage effects in the *RORB* model is as follows:

Where is the volume of storage (m^3), is the outflow discharge (m^3/s), is a storage delay parameter and is a dimensional empirical coefficient.

The exponent is a measure of the catchment's non-linearity. When is set equal to unity the catchment's routing response is linear; the ordinates of the discharge hydrograph increase directly in proportion to the ordinates of the hyetograph of rainfall excess. A value of less than unity implies that the peak discharge increases at a proportionally greater rate than the rainfall intensity. In the absence of more catchment specific data, a value of 0.8 is commonly used for flood estimation, as most catchments tend to behave in a non linear fashion, at least for the minor and medium flood events (ARR- Volume 1, Book VI Section 5.4.7).

The storage parameter within the general storage equation is modified to reflect the catchment storage and the reach storage as follows:

Where is an empirical coefficient applicable to the catchment and stream network and is a dimensionless ratio called the relative delay time applicable to an individual reach storage

The relative delay time of a storage is defined as the ratio of its delay time at any given discharge to the total delay time at the same discharge of all channel reaches from the centroid of the area being modelled to the downstream end of the channel network. The relative time delay is calculated in the RORB program as follows:

Where is the relative delay time of storage i, is the length or reach represented by storage I (km), is the average flow distance in the channel network of sub area inflows (km) and is a factor depending on the type of reach (set at 1 for natural channels).

5.4 Model Calibration

5.4.1 Overview

The *RORB* model parameters were calibrated by fitting rainfall and runoff data from recorded events. Regional design values were adopted where there was insufficient data for calibration.

5.4.2 Calibration Events

The only historic rainfall-runoff events with sufficient continuous data to use in the hydrological model calibration were Events One and Three; the hydrographs from these events are shown in Figure 4-6 and Figure 4-8. Hydrographs are shown in Appendix C- RORB Model Calibration Hydrographs.

5.4.3 Stream Channel and Catchment Layout

Using topographic information the Yandagooge catchment was divided into model sub-areas representative of sub-catchment areas bounded by drainage divides.

Figure **5-1** shows the catchment network and sub-catchment areas used in the model.

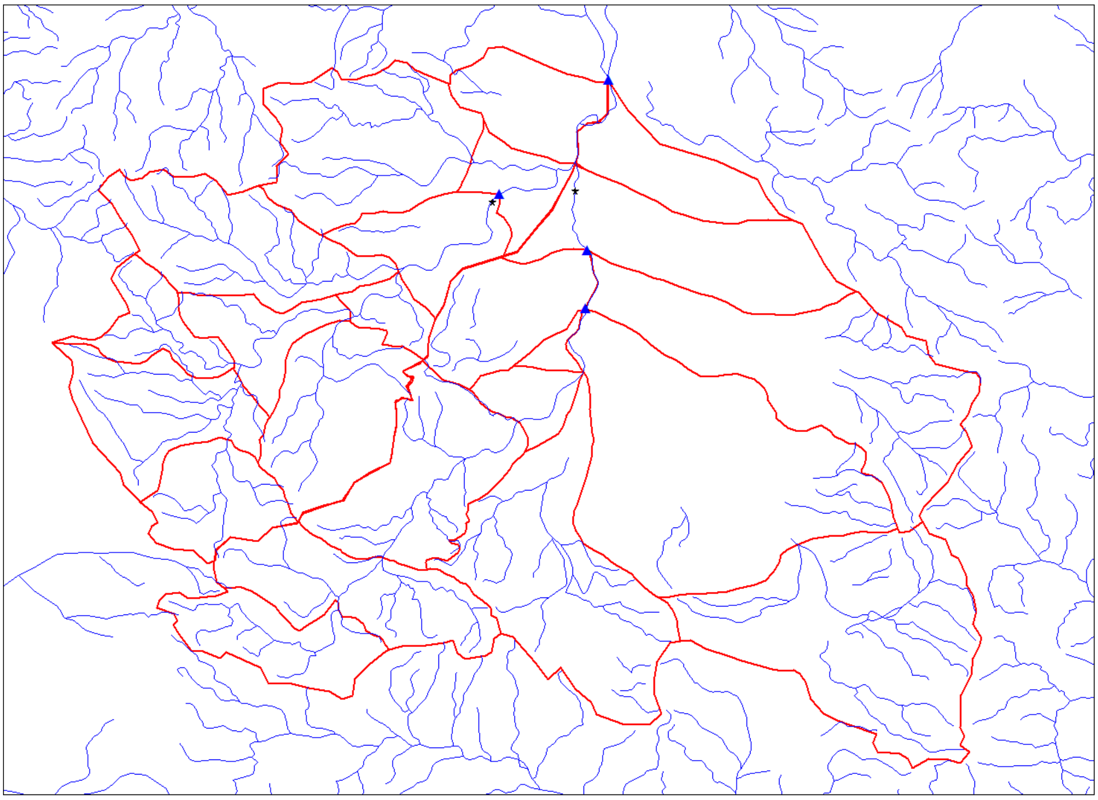

Figure 5-1 Sub-catchment layout

The area of the South Branch creek catchment is approximately 300 km^2 and the West Branch creek catchment area is approximately 170 km^2. Assuming similar hydrological characteristics and rainfall, it would be expected that, given the relative catchment areas, approximately twice as much volume runoff would be generated from the South Branch. This is not reflected in the available streamflow event data (Table 4-4) where the peak flow in the South Branch is generally less than half of the West Branch peak. Whilst relatively impermeable Coolbro Sandstone is dominant in the West Branch catchment, the geology in the South Branch is dominated by porous sands. It is likely that during smaller rainfall events, much of the rainfall in the eastern part of the catchment infiltrates into the sub-surface before surface runoff is produced or ponds in relatively flat areas rather than draining to the main channels.

The amount of infiltration in different sections of the catchment could not be quantified because of the lack of data. In larger flood events, infiltration losses will have a less significant impact on the overall flood flows than in smaller events. For the purpose of this study the entire South Branch catchment was assumed to contribute to surface runoff in the South Branch Creek.

5.4.4 Rainfall Representation

Ideally the hydrological model would have been set up to include multiple rainfall gauges in the catchment to reflect non-uniform rainfall distribution. However, continuous rainfall during the calibration period was only available for one rainfall gauge (Central Weather Station), therefore, rainfall in the model was assumed to be uniform in depth and temporal pattern over the entire catchment.

5.4.5 Rainfall losses

Where sufficient data is available, the initial loss and continuing loss parameters can be derived using catchment rainfall and runoff data, during the calibration process. It was necessary to adopt regional design parameters for the Yandagooge Creek model due to the lack of data.

Design values for rainfall losses have been derived in Australian Rainfall and Runoff (Pilgrim, 1987). The design values of initial loss vary with rainfall zone, flood frequency and the degree of non linearity assumed in the catchment flood hydrograph model. The design values for the *Pilbara* and *Arid Interior* rainfall zones are shown in Table 5-1.

Table 5-1 - Design initial loss and continuing loss values

ARI (years)	5	10	20	50
Pilbara – loam soils Median continuing loss = 5 mm/hr	40	52	47	32
Arid Interior – loamy soil Median continuing loss = 3 mm/hr	20	31	38	38

*Non Linear Model (m = 0.8)

The Kintyre project site lies within the A*rid Interio*r region, close to the boundary with the *Pilbara*, so both parameter sets were investigated during calibration.

5.4.6 Coefficients k_c and m of Storage-Discharge Equation

The empirical coefficients k_c and *m* are the principal parameters of the model. Where good quality historic rainfall and runoff data are available for a study area, the parameters are generally derived using a process of model calibration over a range of flood magnitudes.Given the limited Yandagooge catchment record, *m* was assumed to be 0.8 which is consistent with recommendations in (Pilgrim, 1987).

A calibration was undertaken to derive k_c for the Yandagooge catchment even though there was only one suitable calibration event (Event 3). Australian Rainfall and Runoff recommends that where rainfall and runoff records of at least one flood are available on a catchment, it is usually best to calibrate a given model by determining parameter values from the observed flows or to reproduce those flows. If the catchment is ungauged or no suitable flood data are available, parameter values must be estimated by transferring derived values from adjacent catchments, or by means of physical considerations or regional relationships.

Published regional relationships to determine k_c have been derived for Australia (ARR, 1987); for the Arid Interior/North West region of Western Australia, the following relationship is recommended:

$k_c = 1.06L^{0.87}S^{-0.46}$

Where *L* is the mainstream length (km) measured from the catchment outlet to the most remote point on the catchment boundary and *S* is the equal area stream slope (m/km).

The k_c value for the South and West branches of the Yandagooge Creek computed from the regional relationships is shown in Table 5-2.

Table 5-2 Regional Kc values for South and West Branch

	L	S	K_c
South Branch	28.2	3.2	11.5
West Branch	25.4	3.5	10.0

ARR suggests that these relationships may also be useful on a gauged catchment where only limited data are available for calibration, and design values may be selected considering both sources of information. However, regional relationships should be used with due caution, as most derived relationships have incorporated considerable scatter of the data from individual catchments.

5.5 Calibration Runs

The continuous rainfall and stream flow data from the Events 1 and 3 were input in to the RORB catchment model and the kc parameter was adjusted until the best fit between the observed stream flow and modelled stream flow could be achieved.

Event 1 was discarded because there was not sufficient rainfall correlated to the second hydrograph peak. This could have been due to erroneous data or the rainfall gauge not being representative of rainfall in other parts of the catchment.

Event 3 was the only historical event recorded at Kintyre for which there was sufficient data to undertake a RORB model calibration. The South Branch was not used in calibration due to the uncertainties associated with the amount of catchment area contributing to the runoff.

The results of the West Branch calibration are shown in Figure 5-2. The best fit was achieved using Pilbara loss design values, m equal to 0.8 and a k_c equal to 17. The modelled hydrograph produced a similar hydrograph peak, shape and volume to the observed hydrograph, although it lagged behind the observed stream flow by 2 hours.

The regionally derived parameter sets were also compared to the observed flow (Figure 5-3). The Arid Interior loss design values, m equal to 0.8 and a k_c equal to 10 produced the highest hydrograph peak and volume. To account for data and modelling uncertainties, this set of parameters was adopted in design to give a conservative estimate of flood peaks.

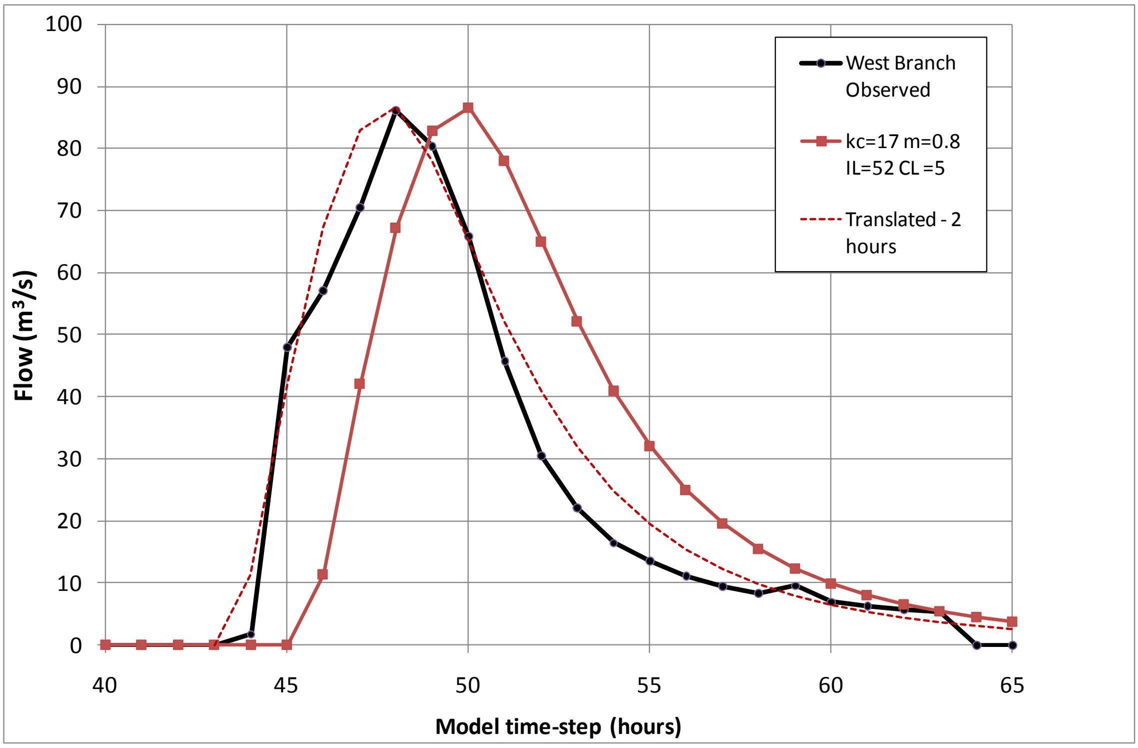

Figure 5-2 Calibration Hydrograph - adjusted k_c

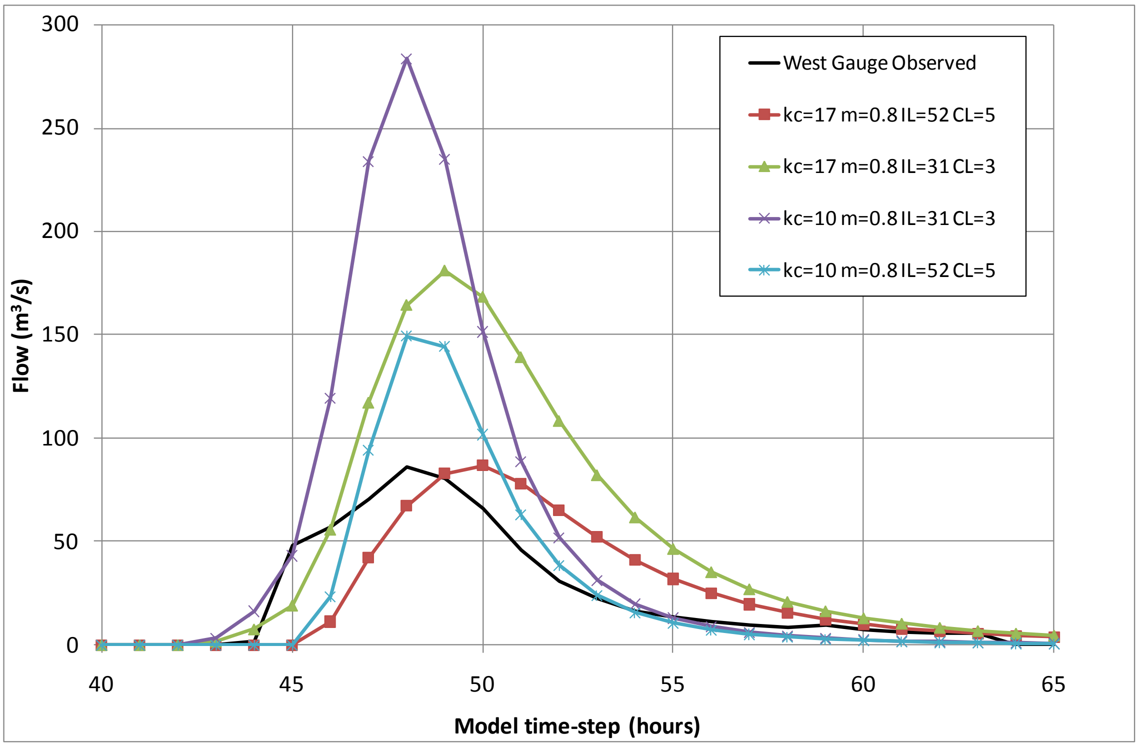

Figure 5-3 Calibration Hydrographs – regional parameter sets

5.6 Design Flood Estimation

5.6.1 Overview

The regional model parameters and the kc determined during calibration were used with design rainfall to estimate design hydrographs in the Yandagooge catchment. This enabled a probabilistic likelihood of occurrence to be attributed to the 10, 20 and 100 year ARIs.

5.6.2 Model Parameters adopted for Design Flood Estimation

Following the calibration runs and given the lack of catchment data it was decided to adopt two parameters sets in the design flood estimation. The first set of parameters was the "calibration" parameter set (Table 5-3) which was the set of parameters that gave the best fit with the observed hydrograph.

Table 5-3 Calibration Parameter Set

Parameter	Average Recurrence Interval (Years)			
	5	10	20	50
Initial Loss	40	52	47	32
Continuing Loss	5	5	5	5
kc	17	17	17	17
m	0.8	0.8	0.8	0.8

The second set of parameters which were adopted was the "regional" parameter set which generated the highest flood peak and volumes (Table 5-4). This set of parameters was adopted to give the most conservative set of peak flow values.

Table 5-4 Regional Parameter Set

Parameter	Average Recurrence Interval (Years)			
	5	10	20	50
Initial Loss	20	31	38	38
Continuing Loss	3	3	3	3
kc	10	10	10	10
m	0.8	0.8	0.8	0.8

5.6.3 Rainfall Intensity

5.6.3.1 IFD Curves

Design rainfall data were used to enable a probabilistic likelihood of occurrence to be attributed to different rainfall scenarios. An analysis of rainfall data from a single station is often unreliable, not temporally or spatially consistent, and should generally not be used for design purposes. Instead a set of accurate, consistent Intensity – Frequency – Duration (IFD) data have been derived for the whole of Australia using statistical procedures by the Bureau of Meteorology (Pilgrim, 1987).

- DURATION: refers to the period over which the rainfall occurs.
- FREQUENCY: refers to the regularity with which a rainfall event of a particular intensity and duration is likely to occur.
- INTENSITY: relates to the rainfall rate (in mm per hour). It is calculated by dividing the depth by the duration and is simply a measure of the 'heaviness' of the rainfall.

The IFD data for the Kintyre project site is shown in Figure 5-4.

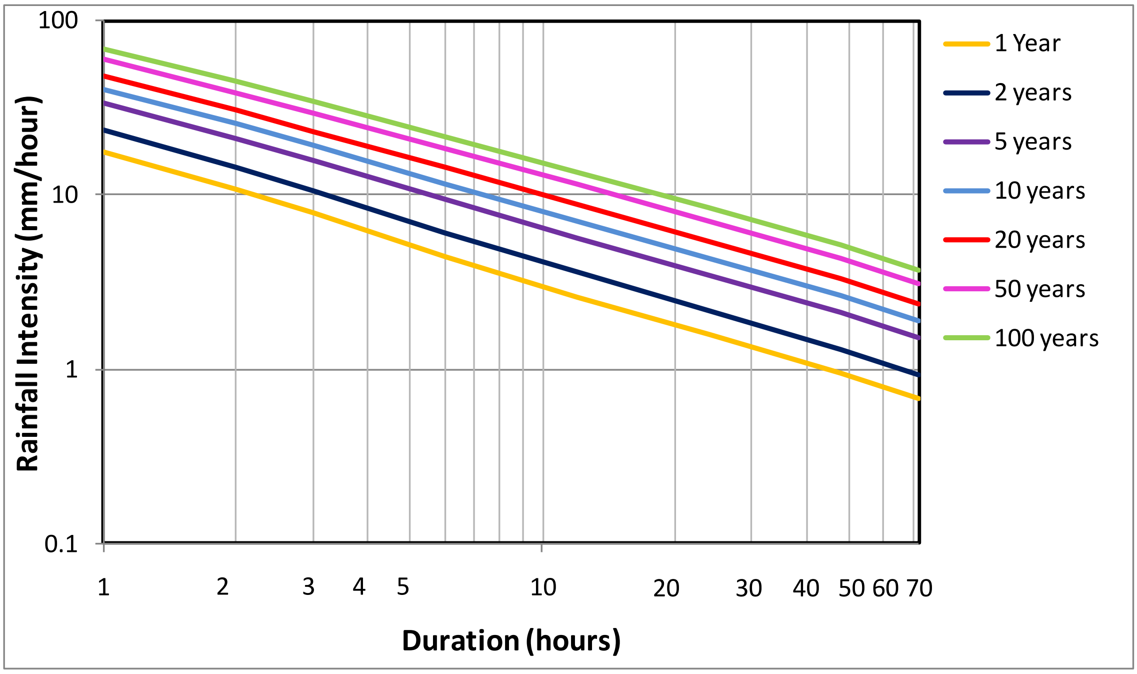

Figure 5-4 Intensity – Frequency – Duration Design Rainfall for Kintyre

5.6.3.2 CRC Forge Rainfall

The CRC-FORGE approach has been applied to Western Australian rainfall to derive seasonal and annual design rainfall estimates from an annual exceedance probability (AEP) of 1 in 50 to 1 in 2000 and for durations of between 24 and 120 hours (Durrant and Bowman, 2004).

The CRC-FORGE rainfall estimates for Kintyre are presented in Table 5-5.

Table 5-5 CRC-Forge Rainfall estimates for Kintyre

Duration	1:50	1:100	1:200	1:500	1:1000	1:2000
24	175.1	201.8	230.7	273.0	308.2	346.4
30	185.3	212.7	243.3	287.6	324.4	364.3
36	194	222.1	254.1	300.2	338.3	379.6
48	208.7	237.7	272.2	321.0	361.4	405.1
60	212.8	244.6	279.4	329.0	369.5	412.8
72	216.2	250.4	285.4	335.6	376.3	419.2
96	221	256.2	291.2	341.8	382.4	424.6
120	228.7	259.5	294.3	345.2	385.6	428

5.6.3.3 1,000 year ARI Flows

The 1,000 year rainfall depths for durations less than 24 hours were estimated using methodology described in ARR. The method involves interpolating between the PMP (see section 5.9)and the 100 and 50 year ARI rainfall depths. This gave a 1,000 year rainfall of 210mm. A plot of the interpolated values for a 6 hour rainfall event is shown in Figure 5-5.

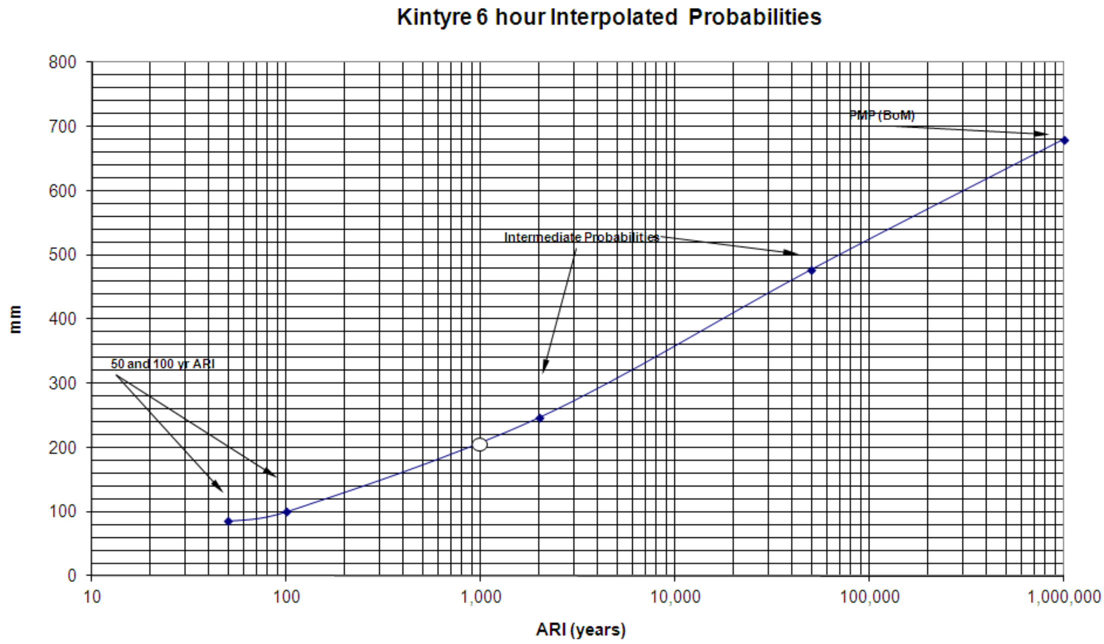

Figure 5-5: Interpolated Rainfall Probabilities

5.6.4 Critical Duration

The critical duration of a rainfall event is that which produces the highest peak flow. This duration will vary based on the size, layout and geology of the catchment. Hence a number of rainfall events with varying durations were run through the rainfall-runoff model. The range of durations covered was 6 hours to 72 hours.

5.6.5 Temporal Patterns

Temporal patterns were required to convert design rainfall depth with a specific ARI to a design flood of the same frequency. Temporal patterns were obtained from the recommended profiles for Zone 7 (Western Australia – Indian Ocean) in ARR Volume 2.The patterns vary in relation to ARI, with different patterns for events of recurrence interval less than or equal to 30-years and greater than 30-years. Example temporal patterns for the 100-year 24-hour and 72-hour rainfall events are shown in Figure 5-6 and Figure 5-7.

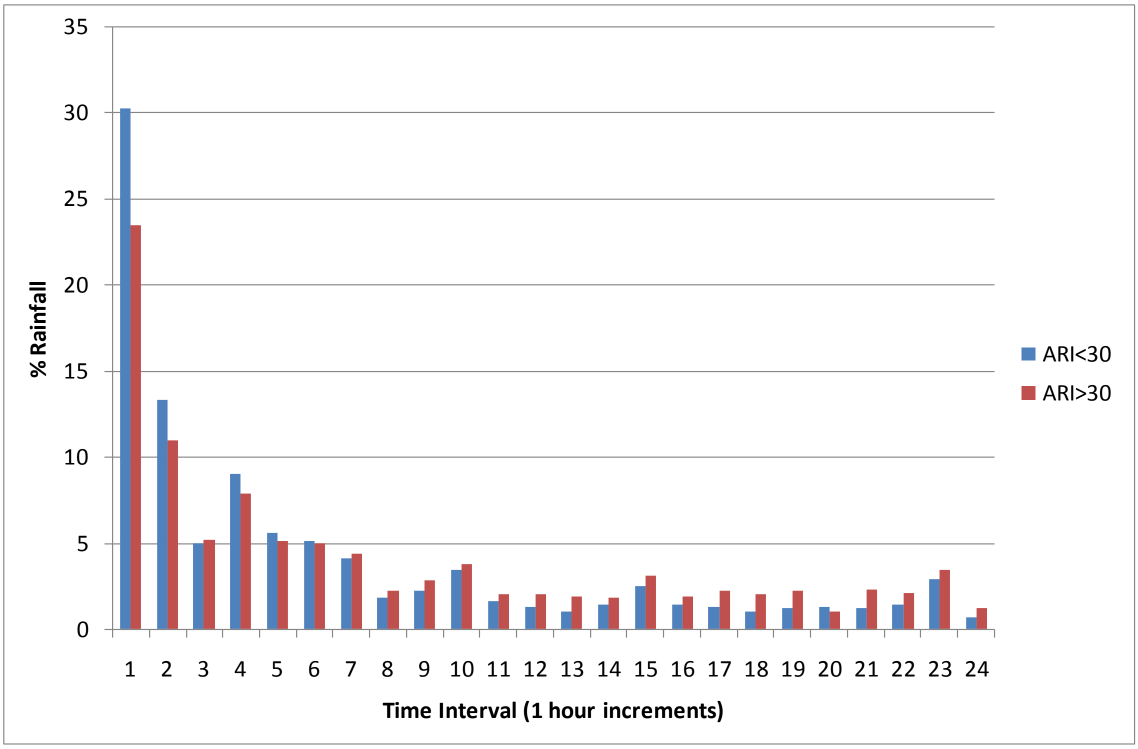

Figure 5-6 Temporal Patterns for Kintyre - 24 hour duration

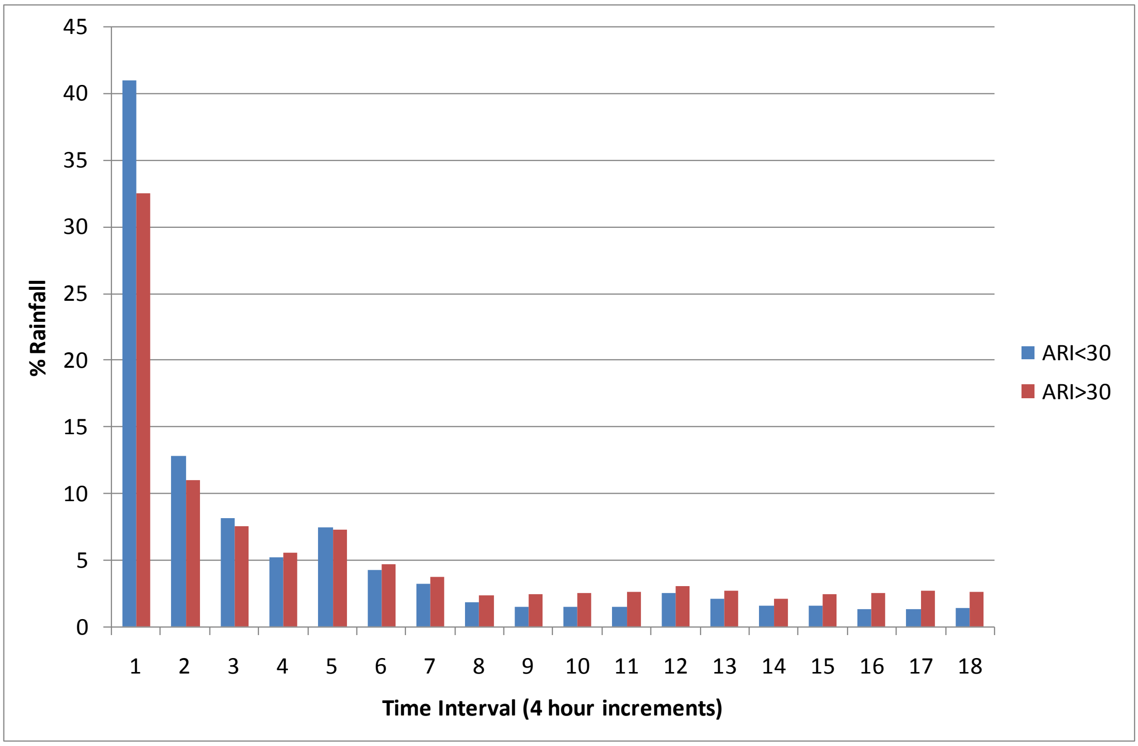

Figure 5-7 Temporal Patterns for Kintyre - 72 hour duration

The temporal pattern which is adopted can have a major effect of the computed flow. The Zone 7 temporal patterns have a significant fraction of the rainfall occurring in the first time interval. It should be noted that these temporal patterns are very sensitive to the depth of the initial loss value used. A comparison with other rainfall events indicated that the temporal patterns may attribute too much rainfall to the first time increment. In the absence of more detailed information, these temporal patterns have been adopted.

5.6.6 Telfer Event

For the purpose of comparison the 2004 Telfer rainfall event has been modelled. The temporal pattern for the observed rainfall event is shown in Figure 5-7.

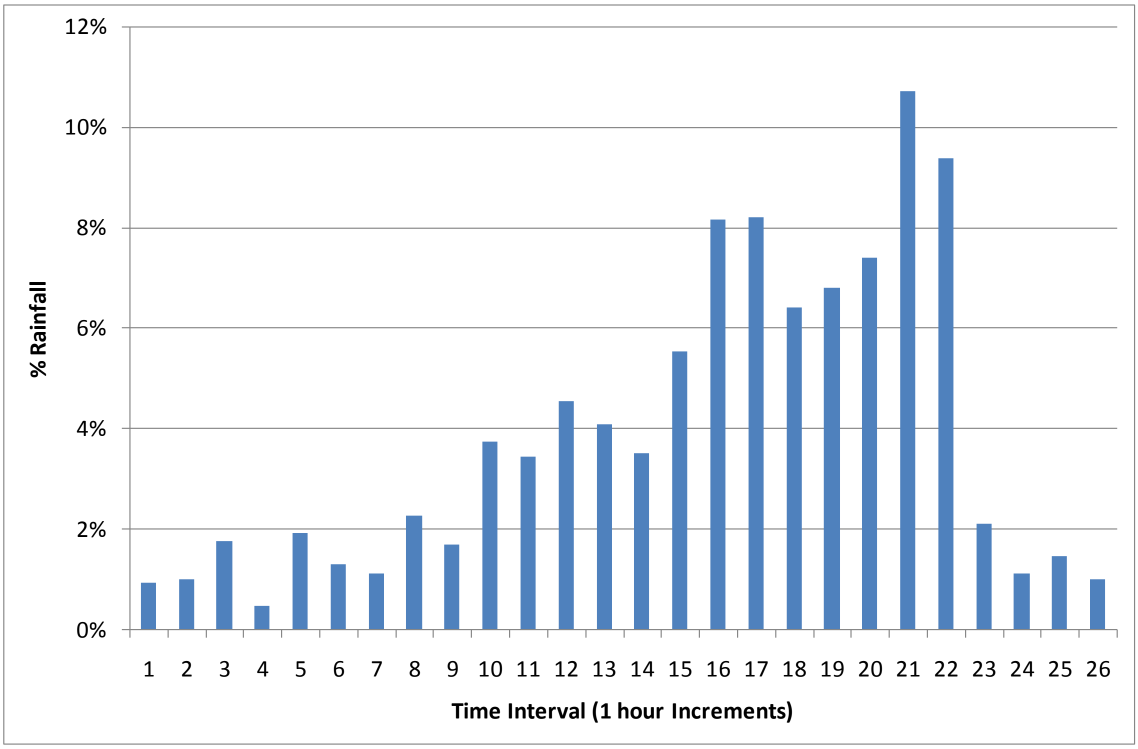

Figure 5-8 Temporal Pattern of rainfall during Telfer 2004 rainfall event

5.6.7 Design Flood Hydrographs

The design storm hyetographs for 10, 20 and 100 year ARIs and durations from 6 hours to 72 hours were derived using the IFD data (Figure 5-4) and ARR temporal patterns. These hyetographs were applied to the RORB model to obtain discharge hydrographs at the South Branch gauging station (downstream), West Branch gauging station and the creek confluence using the two parameters sets shown inTable 5-3 and Table 5-4.

The peak design flows and critical durations are contained in Table 5-6 and Table 5-7.

The design flood hydrographs are contained in Appendix F – RORB Hydrographs.

Table 5-6 Peak flood and critical duration South Branch Gauge

ARI	"Calibrated" parameters		Regional parameters	
	Peak Flow (m3/s)	Critical Duration (hr)	Peak Flow (m3/s)	Critical Duration (hr)
10 year	13	36	247	12
20 year	149	12	435	12
100 year	547	24	922	12

Table 5-7 Peak flood and critical duration West Branch Gauge

ARI	"Calibrated" parameters		Regional parameters	
	Peak Flow (m3/s)	Critical Duration (hr)	Peak Flow (m3/s)	Critical Duration (hr)
10 year	5	36	106	12
20 year	64	12	191	12
100 year	247	24	400	12

Table 5-8 Peak flood and critical duration Yandagooge Creek confluence

ARI	"Calibrated" parameters		Regional parameters	
	Peak Flow (m3/s)	Critical Duration (hr)	Peak Flow (m3/s)	Critical Duration (hr)
10 year	18	36	365	12
20 year	215	12	660	24
100 year	873	24	1447	12

The 1,000 year ARI peak flow generated by RORB was 3,530 m^3/s.

As expected from the calibration runs, the regional parameter set gave significantly higher peak flow estimates than the calibrated set. The proportional difference in peak flows was smaller as the ARI increased.

5.7 Telfer Event Hydrographs

The observed 2004 Telfer rainfall event was input into the hydrological model and the resultant hydrographs are contained in Appendix E - Telfer Hydrographs estimated using the calibration and design parameter sets. This event has been included in the hydrological modelling as it is a recent event, memorable for the Telfer community. The estimated peak flow at the confluence is approximately 3000 -3500 m^3/s, significantly higher than the estimated 1:100 design flood hydrograph which is consistent with the probabilistic assessment of the Telfer rainfall event (>350 year ARI).

5.8 Regional and Rational Peak Flow Estimates

Regional methods have been derived for estimating peak flows in ungauged catchments in the Pilbara. Peak flows in the South and West branches of the Yandagooge Creeks were estimated using the Index Flood (Regional) Method, as recommended in ARR. Estimates are based on catchment area and an average annual rainfall determined from regional isohyets provided in ARR. These two parameters were then entered into the recommended design equation for the Pilbara Region of Western Australia to calculate a peak flow for the 5-year event. 5-year peak flows were then used to estimate peak flows for other return periods by applying an appropriate frequency factor. A summary of the parameters used in the calculation is provided in Appendix D - Regional and Rational Method Parameters.

Peak flows for the Project area were also calculated using the Rational Method applicable to the Pilbara Region of Western Australia (ARR Vol.1, Book IV, Section 1.4.7). The rational method relates rainfall intensity for a given frequency, with the design flood magnitude of the same frequency, providing approximate peak flood flows. A summary of the parameters used in the calculation is provided in Appendix D - Regional and Rational Method Parameters.

Table 5-9 Comparison of peak flood estimation methods at South Branch Gauge

Method	10 yr ARI	20 yr ARI	100 yr ARI
Index Flood (Regional) Method	225	399	1279
Rational Method	443	695	2204
RORB run with calibrated parameters	13	149	547
RORB run with regional parameters	247	435	922

Table 5-10 Comparison of peak flood estimation methods at West Branch Gauge

Method	10 yr ARI	20 yr ARI	100 yr ARI
Index Flood (Regional) Method	121	214	684
Rational Method	221	441	1075
RORB run with calibrated parameters	5	64	247
RORB run regional parameters	106	191	400

A comparison between the results of the rainfall-runoff modelling, derived design floods and the regional methods is provided in Table 5-9 and Table 5-10. The peak flow estimates estimated using the rational method are much higher than other estimates. The RORB model with the calibrated parameters set gives considerably lower estimates of peak flow.

It should be noted that the majority of gauging stations in the Pilbara Region regions, data from which the regional methods have been derived, are poorly rated and have relatively short lengths of record. ARR recommends that flood estimates derived for these regions should be treated with caution, especially for higher average recurrence intervals and given that there is little data or the data are of poor quality. RORB and similar models should give better flood estimates than the Rational and Index Flood Methods (ARR).

5.9 Probable Maximum Precipitation

The probable maximum precipitation (PMP) is defined by the Manual for Estimation of Probable Maximum Precipitation (WMO,1986) as:

"...the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends."

The Bureau of Meteorology (BoM) was commissioned to provide an estimate of Probable Maximum Precipitation (PMP) for the Kintyre catchment. The BoM report is attached at Appendix J – PMP Report and a summary of the results of the assessment is listed in Table 5-11.

Table 5-11: PMP

Duration (hrs)	Final GTSMR PMP (mm)
1	280
3	530
6	680
12	820
24	1000
36	1200
48	1390
72	1720
96	1930
120	2030

5.10 Probable Maximum Flood

The Probable Maximum Flood (PMF) is the most severe flood that is likely to occur at a particular location. Such a flood would result from the most severe combination of critical meteorological and hydrological conditions.

Estimates of the Probable Maximum Flood within the Yandagooge Creek were calculated using three methods:

- RORB rainfall runoff model
- IAHS envelope curves
- Rational Method

Results using RORB and the BoM supplied PMP were adopted for input to the hydraulic model but were compared with the other methods to check the validity given the lack of calibration data available at the site. Although the RORB peak flow was around 16% higher than the other methods it was considered a more robust estimate of the maximum flood.

5.10.1 RORB Results

The PMP rainfall depths listed in Table 5-11 were input to the RORB model for durations of 3, 6 and 12 hours. Output from the model for these durations was 23,300, 17,600 and 11,600 m^3/s respectively.

The likelihood of a 3 hour time of concentration for the Kintyre catchment was considered low and so the 3 hour PMF was discarded. Various T_c formulae were used and gave an average time for the Kintyre catchment of 8.4 hours. A Tc of 3 hours for the Kintyre catchment would require average flow velocities of 2.7m/s for the period of the event which seems unlikely.

5.10.2 IAHS envelope Curves

The International Association of Hydrological Sciences (IAHS) has carried out studies on extreme floods throughout the world based on a "World Catalogue of Maximum Observed Floods". The maximum floods indexed have an envelope curve which was adapted to an equation given by Francou - Rodier:

$$Q = A^{1-6.4/10} \times 1{,}000{,}000$$

Where:

Q = Flood peak discharge (m^3/s)

A = Catchment area (km^2)

For the Kintyre catchment of 470 km^2 the equation yields a PMF of 14,250 m^3/s.

5.10.3 Rational Method

While not specifically applicable to large catchments the PMP was also used with the ARR Rational Method with Pilbara parameters. This gave a flow of 14,800 m^3/s.

5.10.4 Summary of PMF Estimates

Method	PMF m^3/s
RORB (6 hour PMP)	17,600
IAHS Envelope Curve	14,250
Rational Method (6 hour PMP)	14,800

5.11 Adopted Flows

The flow hydrographs derived using the RORB model adopted for input into the hydraulic model are summarised in Table 5-12.

Table 5-12: Adopted Flows and Rainfall

ARI	Rainfall Duration (hrs)	Rainfall (mm)	Flow (m^3/s)	Method
10	12	85	365	RORB (regional parameters)
20	24	131	660	RORB (regional parameters)
100	12	161	1,447	RORB (regional parameters)
1000	6	210	3,532	RORB (regional parameters)
PMP/PMF	6	680	17,600	RORB (regional parameters)

6 Hydraulic Modelling

6.1 Introduction

Due to the complex nature of floodplain dynamics a combined one-dimensional (1D) and two-dimensional (2D) hydraulic model was developed to determine potential flood magnitudes and impacts at the Kintyre project site.

The objectives of developing the floodplain hydraulics model were to:

- undertake estimation of flood levels and compare with the levels of the proposed mining infrastructure;
- produce flood maps of land inundation for floods of different magnitudes and probability of occurrence;

In considering the hydrology and hydraulics of the wider floodplain and catchment surrounding the mine site, the following characteristics were taken into account:

- The general topography of the catchments draining past the site;
- The soil and rock type of the ground and vegetation cover.

6.2 Model Description

Hydraulic modelling software was used to establish the magnitude and extent of flooding around the proposed project area. The software used was InfoWorks RS, a 1D/2D hydrodynamic modelling package developed by Innovyze (originally HR Wallingford). The software is internationally recognised as an accurate model for river system and floodplain analysis.

In 2009, initial modelling of the site was carried out in an entirely 1D environment for preliminary analysis and to identify widespread locations of flooding. The 2D component was then added to investigate direction and depth of flood flow in potential “high risk” areas. The 2D component is better suited for modelling flows through complex geometries and open ground where the characteristics of flow are difficult to assume. A combined model provides a faster run time (in the order of 4 to 8 hours) and enables a large number of runs to be tested and compared.

When the critical components of the Kintyre mine situation are confirmed, a model that is only 2D should be developed to provide more detailed information on depth, velocity and duration of flooding at all grid points.

The modelling of the proposed flood protection embankment was also completed. The extent of the combined 1D/2D model is shown in Figure 6-1.

For the 1D component of the model cross sections were derived from 0.5m contour interval information provided by Cameco. The contours were used to develop a Digital Terrain Map (DTM) for the 2D component of the model. Cross sections were positioned over the south and west branches of the Yandagooge Creek and downstream of the branch convergence.

Two flow-time (hydrograph) boundaries were used as the upstream boundaries for the hydraulic model. These boundaries were located on the upstream extent of the 0.5m contours on the west and south branches. The ground data available upstream of these positions was not detailed enough to provide a distinct channel within which flow could be directed.

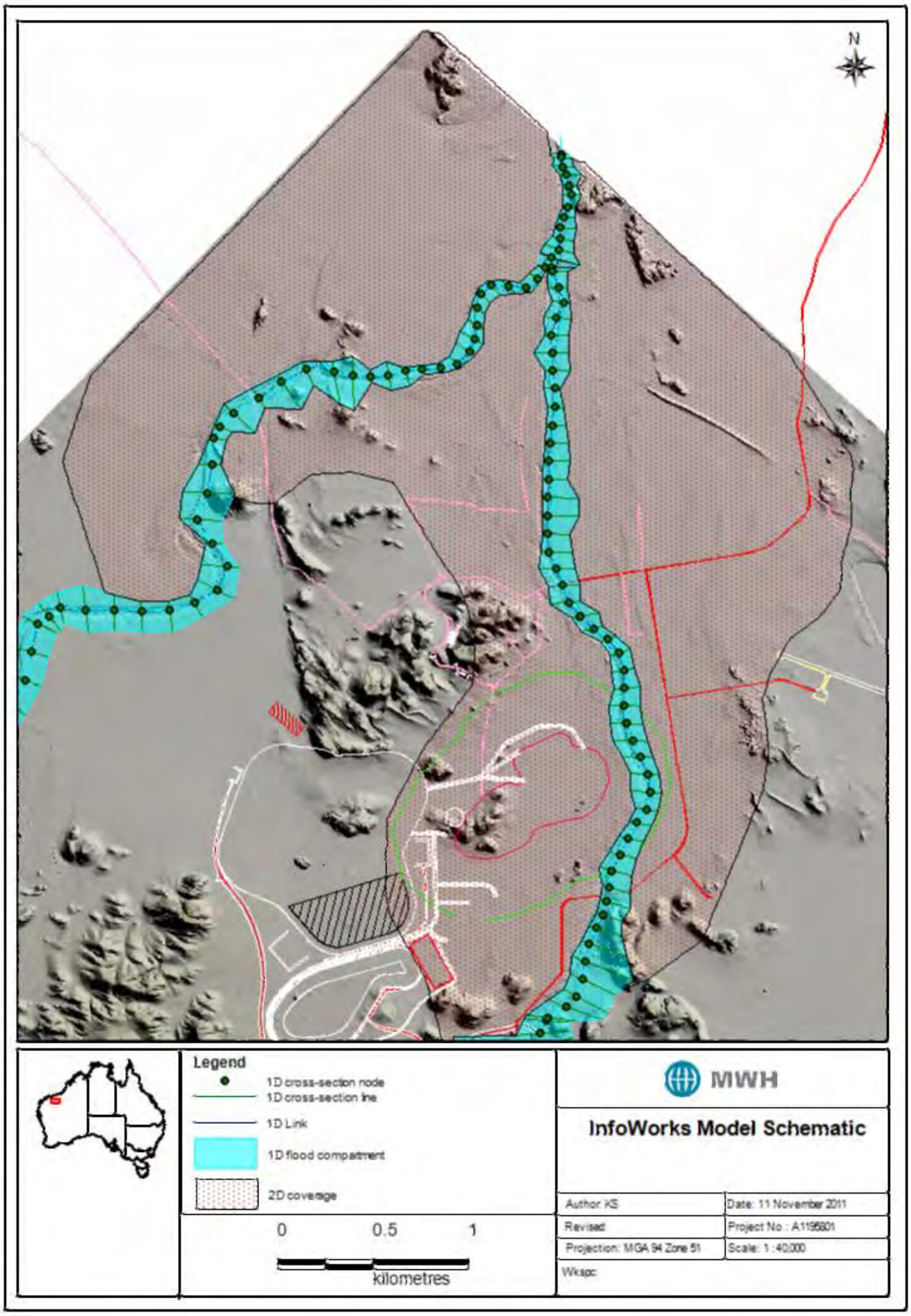

Figure 6-1: Extent of the Combined 1D/2D Hydraulic Model

Four calculated sub-catchment discharges were introduced into the model as distributed lateral inflows. The hydrographs generated for the flow-time boundaries and inputs were calculated using design rainfall input to RORB software as described previously (Section 5). At the downstream boundary of the model a flow-head relationship based on section data was applied. This downstream boundary is suitable for a catchment like Kintyre, with relatively flat downstream topography.

It should be noted that the 2D hydraulic model was developed without detailed field survey information of major creek cross sections, upstream and downstream creek characteristics and details of the confluences of waterways, which would improve the model delineation.

The catchment area of the South Branch is approximately 300km^2 and the West Branch covers approximately 170 km^2. The hydraulic model extends over an area of approximately 80 km^{2,} of which a quarter has been modeled as a 2D flood plain. The 2D floodplain consists of triangular elements, which were generated using the DTM created from the contour data provided by Cameco. The contour data was processed using Vertical Mapper into an ASCII grid format and subsequently imported into the hydraulic model.

An appropriate 2D triangular grid size for a regional model is 100m^2 to 1,000m^2. The average triangle size within the 2D model is approximately 320m^2, with a maximum set at 500m^2. The triangular size in proximity to the mine infrastructure was refined to 200m^2, to provide greater accuracy. The smaller the triangular area, the more triangles incorporated into the calculations and the longer the model run time.

Figure 6-2 shows the inputs used and steps followed to produce the flood maps generated by the model.

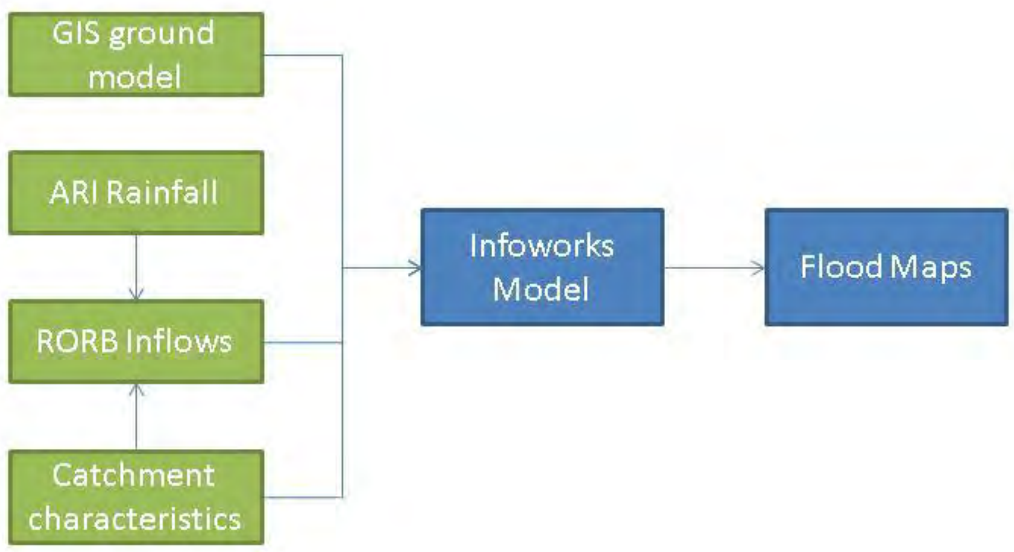

Figure 6-2: Modeling Inputs and Processes

6.2.1 Surface Roughness Data

Channel and floodplain roughness provides the primary resistive force which affects the flow of surface water. The channel roughness is the resistance due to the local boundary friction and is therefore best estimated through interpretation of the surface roughness. The 2D floodplain and creek cross sections in the model were assigned an appropriate bed resistance. The land cover and surface types were assigned through examination of aerial photographs and information gathered during an initial site visit in 2009.

The unit roughness values were composed of up to three component roughness values; surface material, vegetation and irregularities. These are combined to get the total unit roughness using the following equation:

$$n_l = [n^2_{sur} + n^2_{veg} + n^2_{irr}]^{1/2}$$

n_{sur} = bed, bank or floodplain surface material (sand, outcropping)

n_{veg} = vegetation

n_{irr} = irregularities (dead trees, pools, boulders)

Roughness categories used in the model are shown in Table 6-1 along with the associated model component (1D or 2D), and the layout of the roughness zones used in the 2D component of the model are shown in Figure 6-3. Photos of the site are included in Appendix B - Kintyre Photos (May 2009) for assessment of roughness.

Table 6-1 : Surface Roughness Categories

Category	Model Component	Roughness (n)
Bed 1	1D	0.037
Bed 2	1D	0.043
Bed 3	1D	0.05
Bank 1	1D	0.045
Bank 2	1D	0.049
Bank 3	1D	0.053
Bank 4	1D	0.051
Bank 5	1D	0.063
Bank 6	1D	0.049
Flood Plain 1	1D	0.048
Flood Plain 2	1D	0.052
Flood Plain 3	1D	0.063
Flood Plain 4	1D	0.038
Flood Plain 5	1D	0.054
Flood Plain 6	1D	0.05
Flood Plain 7	1D	0.051

Category	Model Component	Roughness (n)
Flood Plain 8	1D	0.047
Flood Plain 9	1D	0.072
Flood Plain 1V	2D	0.048
Flood Plain 2V	2D	0.052
Flood Plain 3V	2D	0.063
Flood Plain 4V	2D	0.043
Flood Plain 5V	2D	0.043
Flood Plain 6V	2D	0.043
Flood Plain 7V	2D	0.043
Flood Plain 8V	2D	0.043
Flood Plain 9V	2D	0.072

6.3 Sensitivity Testing

Sensitivity testing has not been carried out for the hydraulic model. Sensitivity to surface roughness is a process that could be undertaken after the engineering options are settled upon. Varying the surface roughness over a reasonable range of high and low roughness values will indicate changes in flood protection infrastructure such as embankment height and culvert sizing.

6.4 Calibration

No calibration of the 2-D InfoWorks models has been undertaken. It is recommended that accurate calibration of the model should be undertaken once monitoring equipment is installed and sufficient creek level and flow records have been collected.

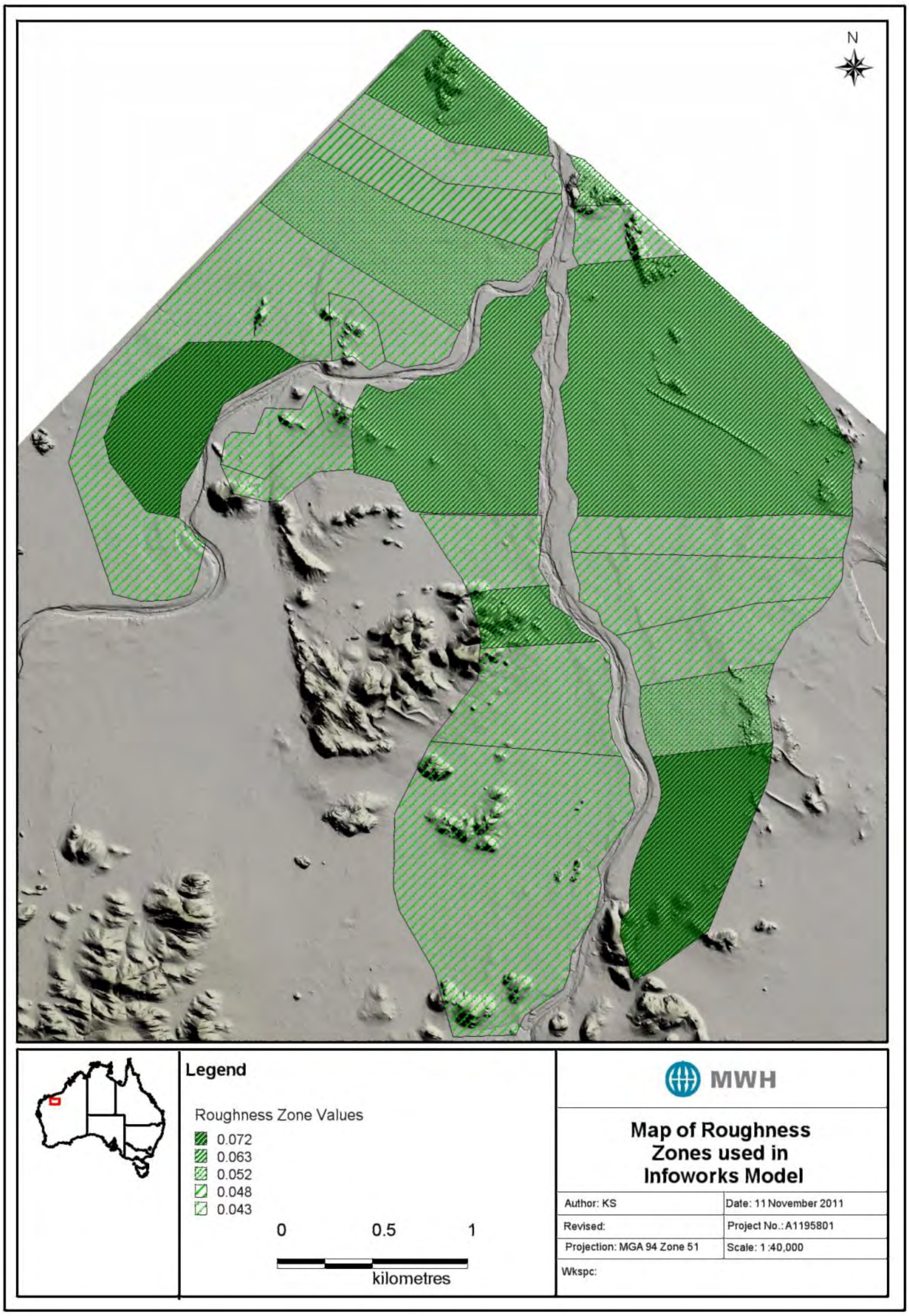

Figure 6-3: Roughness within the 2D model extent (unit roughness n_l)

6.5 Scenarios Modelled

The hydraulic model was used to simulate flows and levels for the 10, 20, 100, 1000 year ARI events and PMP for the base case scenario (pre- mining/ natural setting).In the second phase of the study the hydraulic model was used to simulate flows and flood levels for the same design flood events with the proposed flood protection embankment alignment in place. The results of the modelling are reported in Section 7.

7 Flood Extent Maps

7.1 Introduction

Two scenarios were modelled for each design flood event: the base case or pre mining scenario and the scenario with the proposed flood protection embankment in place.

7.2 Base Case Scenario Flood Extent Maps

A summary of the flood extent maps included in Appendix G – Base Case - Flood Extent Maps is given in Table 7-1. The maps show the peak depth at each grid cell during the flood with a colour sequenced legend indicating the parameter values from lowest to highest.

Table 7-1 List of Flood Extent Maps – Base Case

Event ARI (years)	Measured Parameter
10	Depth
20	Depth
100	Depth
1000	Depth
PMF	Depth

An example flood extent map is shown in Figure 7-1 and shows the peak flood depth during the 100 year ARI event with infrastructure, roads and the pit boundary. The heavy red line is the tenement boundary and the grey surface relief indicates flat land and rock outcrops. The flood extent map shows the modelled peak flood depth, with the darker blue following the main river channels where peak water depths are estimated to reach 2 to 10 metres. The graduated shades of blue represent the decreasing depth of flood inundation out to the lightest blue where inundation depths are only 0.1 to 0.5 metre.

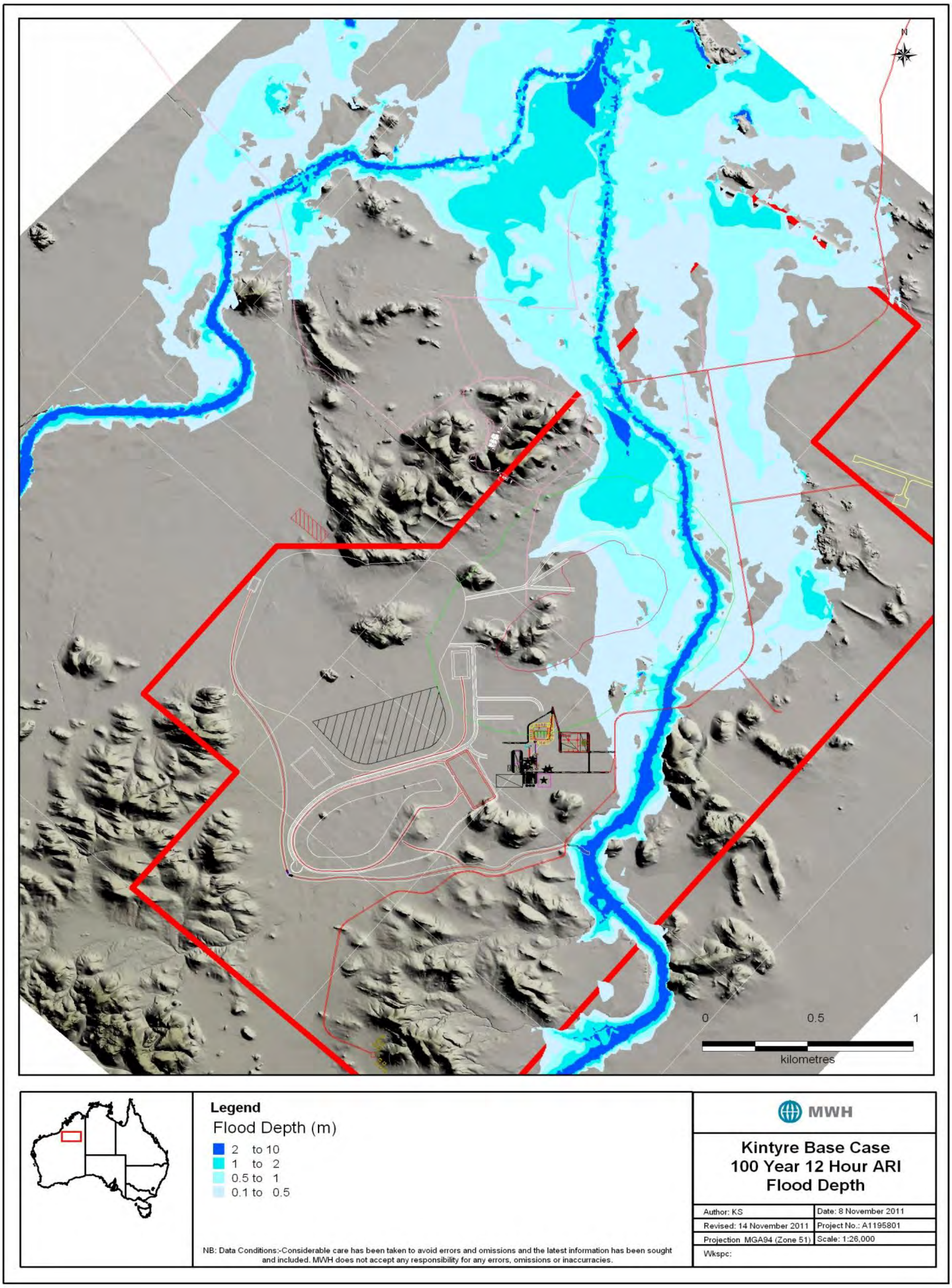

Figure 7-1 Base Case 100 Year 12 Hour ARI

7.3 Proposed Flood Protection Embankment

7.3.1 Introduction

Floodplain modelling indicates that the mine pit is unlikely to be at risk of flooding up to the 10 year ARI event (see flood maps in Appendix G – Base Case - Flood Extent Maps). For larger events, the mine pit will require some form of protection from South Branch Creek flows. In this study, an earth embankment levee has been assessed as an option for flood protection.

To date the modelling results indicate that flood flows from the West Branch of the creek are unlikely to be a flood risk to the mine. It is understood that rainfall that falls directly onto the mine site and is caught on the dry side of the embankment will be contained and treated on site.

The design of the embankment is to a conceptual stage. Issues associated with its alignment, and impact on mine and roading infrastructure will need further consideration at a later stage when a preferred concept is confirmed. This concept design does not take into account embankment foundation conditions or fill material characteristics and requirements.

7.3.2 Flood Protection Embankment Flood Extent Maps

Flood extent maps, incorporating the proposed flood protection embankment, were produced using the hydraulic floodplain model and are included in Appendix H - Flood Protection Embankment - Flood Extent Maps. The maps show the peak depth and velocity at each grid cell across the model extent with a colour sequenced legend. The list of flood map figures is given in Table 7-2.

Table 7-2 List of Flood Extent Maps – Flood Mitigation Case

Event ARI (years)	Measured Parameter
100	Depth
100	Velocity
1000	Depth
1000	Velocity
PMF	Depth
PMF	Velocity

7.3.3 Flood Protection Embankment Concept Design

The design of the flood protection embankment contained in this report is to a concept level and based on a desktop assessment. However, consideration has been given to generic modes of failure which could result in embankment failure. The main modes of embankment failure are:

- **Overtopping failure** – flood water elevations that exceed the embankment crest height will run over the crest and mine side flank potentially eroding the embankment material. The erosive effects are dependent on the depth of overtopping, speed and duration of the overflow, angle of side slope and construction materials used on the embankment surface.
- **Piping of fine material** – uplift pressures from water seepage will carry away material when the submerged weight of the materials is exceeded. Cavities may then form as particles are transported away, which in turn creates preferential flow paths and increases uplift pressures until the embankment may collapse from slumping or shear failure. This is more likely with long duration flood events but could occur during short duration events under unfavourable conditions or if a foundation or embankment weakness is exploited.
- **Slumping of embankment due to shear failure** – the resistance to sliding depends on the shear strength along any given sliding surface and when sliding forces exceed

the resistance, slumping can occur. This can happen in homogeneous materials over a circular surface or in boundary layers where there is stratification. The safety factor against a slumping failure depends on the shear strength of the bank materials and the fluid pressures that arise from seepage flows.

- **Penetrations or modifications to the foundations** - this failure generally occurs from the trenching of pipes or cables through the embankment and backfilling to a lesser standard with different materials. This can also occur from tree roots and burrowing animals. Excavations too close to the embankment toe or excessive blast episodes may cause weakness or preferential groundwater flow paths (cracks) to develop, eroding the fine material away. Pipelines can be installed and backfilled with care and excavations can be prohibited within 15-20m or more clearance of the embankment toe.
- **Creek attack eroding away embankment** – if the creek channel migrates its position towards the embankment during a storm or over a longer wet period the foundations can be undermined by channel flows. This can be prevented with creek works such as rock armouring and channel realignments. Additionally, the position of the embankment can be kept well clear of existing creek bed lines to provide a natural buffer.
- **Dispersion of colloidal clay material** – the embankment construction materials taken from site will need to avoid dispersive clay content as they expand and lose cohesion when they get wet. Embankment materials can be sampled and tested at the borrow sites to determine the risk of this type of soil in the embankment.
- **Foundation failure due to overloading** – generally only a problem with weak clay or peat foundations, and large embankments. This can be resolved by foundation investigation, undercutting poor materials and modification to the proposed embankment where required.
- **Surface wear and tear** – normal compaction with inert materials is sufficient to prevent surface erosion of bank material which could lead to a reduced crest level. Where embankment materials are exposed at vehicle crossings localised erosion can be expected, and maintenance or extra buffer material can be incorporated into the surface treatment at vehicle crossings.

7.3.4 Flood Protection Embankment Concept Profile

With consideration of the failure modes described in Section 7.3.3, the concept design is summarised below, with reference to the engineering drawings in Appendix I – Flood Protection Embankment - Conceptual Engineering Drawings :

- The embankment cross section has a 10m wide crest and 1 vertical to 3 horizontal side slopes to form a stable cross section. This provides a large mass through which the soil saturation surface will take a long time to pass through the embankment soils, thus preventing piping and slumping due to shear strength failure. The size of the embankment means that some damage can be sustained without failure during flood events and repairs carried out between events.
- The cross section is to be built up in compacted layers of 300mm thickness to create a high density and low voids ratio in the embankment. This provides a strong internal strength to resist slumping and shear failures, and reduces the risk of local weak points left during construction.
- The base of the embankment includes 500mm stripping of natural ground, proof rolling with a heavy roller to determine soft areas, undercutting of unsuitable materials and replacement with engineering materials. The foundation includes a centrally placed cut-off key to hinder subsoil groundwater movement, and a drainage blanket and pipe on the mine side to prevent a saturation of the cross section and foundation. The key and drainage measures mean that it would take a long duration event to develop saturated conditions, and the hydrology suggests that only short to medium length rainfall events will be critical in terms of flood depth and velocity.

- The alignment of the flood protection embankment has been selected from a desktop assessment of the pit edge and the creek channel. An offset of 150m from the pit edge has been chosen to keep an equal minimum offset to the creek channel. The alignment is buffered against pit workings and blasts on one side and allows a buffer of ground to the nearest part of the active creek channel. The upstream and downstream ends of the embankment levee merge into high ground. The length of the embankment is approximately 3.8km.
- Rock armouring of a section of the creek channel bank is proposed to reduce the risk of the channel migrating towards the embankment during a flood event. This is based on a review of the flood modelling results and the proximity of the creek channel to the embankment.
- The crest elevation profile is proposed to be 1m above the design standard flood level. The long section in Drawing C001, Appendix I – Flood Protection Embankment - Conceptual Engineering Drawings , indicates the crest profile needed to meet the probable maximum flow standard with embankment heights in the range of 2.2m to 5.8m.
- Mine infrastructure, access roading and the levee alignment will need to be merged together in a coherent design arrangement during further design.

Further design iterations are required to settle on the most convenient and effective levee alignment around the mine. The access road crossing of the south branch creek bed is required in further modelling iterations to accurately assess the backwater effects during large flow events.

Modelling indicates that for very large flows greater than 1,000 year ARI the creek adjacent to the upstream 1.5km of proposed levee is constrained by high ground, including the proposed levee, on both sides of the creek that causes higher flood depths compared to downstream. This is reflected in levee heights of up to 6m. In the reach adjacent to the lower 2.0km of the proposed levee, the floodplain width expands greatly and peak flood depths are less than the upstream section. This is reflected in smaller levee heights between 2.5m and 4.5m.

In general terms, the closer the levee alignment is to the creek channel the larger the levee height will be due to the ground elevation falling towards the creek channel and the need to contain the top water level of the flood event. Also, the risk of channel alignment migration towards the footprint is increased wherever the levee is close to the channel.

The road crossing of the creek bed in general terms will become an obstacle in the creek bed that will create a backwater effect or rise in the water level upstream of the crossing in large events.

7.3.5 Design Standard

The design standard adopted for the embankment crest profile will depend on the amount of risk of flooding Cameco is willing to take. The long section in Drawing C001, Appendix I – Flood Protection Embankment - Conceptual Engineering Drawings , indicates that the 1,000 year ARI design standard is 2m to 4m lower than the PMF standard.

The footprint width of a 6m high embankment is 46m from toe to toe assuming a 10m crest width and 1 vertical to 3 horizontal side slopes.

Further design investigation is required to determine the soil characteristics of the embankment materials and foundations. Depending upon available soil characteristics, the cross section could be reduced by narrowing the crest width and steepening the side slopes. The design standard flood level chosen for the project will possibly reduce the embankment height and earthworks volume.

8 Downstream Flow Impacts

8.1 Flow Regime

A comparison of model results and flood maps for the Base Case scenarios and the Flood Protection Embankment scenarios shows negligible impacts in terms of discharges, flood depths and velocity for events up to and including the 100 Year ARI event. Refer to flood maps in Appendix G and H.

For event magnitudes up to the 10 Year ARI event, modelling indicates that the flood embankment alignment has no impact on flow in the creek channels. The peak water level for this event was not high enough to intersect the flood protection embankment footprint. The mine therefore has no downstream impacts for events up to this size. The embankment is on relatively high ground and the available flood channel is large enough to pass the flow without being impacted by the proposed embankment.

For the 20 year ARI event, modelling indicates that the peak flood water depth would encroach on a quarter of the proposed pit ground surface area and the proposed flood protection embankment would hold back up to approximately 0.5m of flood depth. The mine therefore has a negligible impact on the downstream environment for events up to this size.

For the 100 year ARI event, the peak flood water depth covers half of the pit ground surface area and the proposed flood protection embankment would hold back up to approximately 1.0m of flood depth. The proposed flood protection embankment reduces the floodplain width at the closest point to the creek from 1.2km to 0.7km, and forces more flow onto the right bank overflow channels. Depths on the right bank floodplain are increased by around 0.25m to 0.5m for 2.5km downstream.

For the 1000 year ARI and PMF events, the flood protection embankment diverts significant flow away from the left bank area, out of the main channel and into a break-out overflow channel on the opposite bank (right bank). The proposed flood protection embankment reduces the floodplain width at the closest point to the creek from 1.5km to 0.8km, and forces more flow onto the right bank overflow channels. Depths on the right bank floodplain are increased by around 0.5m to 1.0m for 2.5km downstream. Such effects are large but they are associated with rare events.

At the downstream confluence of the west and south branches, modelling indicates that the flow depths and areas of inundation are very similar. This could be confirmed with a longer extension to the model downstream of the confluence.

The inclusion of the road crossing over the creek bed into the model will cause an increase in peak water depths upstream of the crossing due to it being an obstruction to very large flood flows. The location of the crossing will be critical to the areas of the floodplain that experience an increased flood water depth.

8.2 Potential Increased Sediment Runoff

Increased flow velocities around the embankment may cause localised scour and increased sediment load. With the encroachment of the proposed levee into the floodplain for events 20 year ARI and larger, the local velocities will be increased leading to higher scour forces. This would be rare and could be partly offset if stable vegetation along the banks can be retained during the mine operations. Potential scour over the mine footprint will be reduced due to the protection of the levee.

The right bank overflow path could be expected to pass more flood water during flood events of 100 year ARI and greater and will receive fine sediments that are deposited on the receding waters of the event as flow velocities stall and stop. This sediment would be expected to cover the ground where the mine site is on the left bank but it will be protected, and the right bank may incur more sediment.

Increased sediment volume may result where ground disturbance has occurred as a result of the proposed mining operation. Areas that are prone to elevated sediment runoff are downstream of waste rock dumps, stockpile areas and water pumped from the pits during flood events or dewatering if this is applicable. In the case of Kintyre, rainfall within the entire mine operation is understood to be contained within the mine site footprint with zero release to the surrounding environment.

Large areas of the Pilbara are predisposed to soil erosion because of their susceptible, often fine textured soils, land degradation (removal of vegetation that exposes the fragile soil structure) and the highly intense rainfall that is experienced. During a large rainfall event, the background mobilisation of natural sediments within the Yandagooge catchment is expected to be high.

The impact of the proposed flood protection embankment is expected to have a minimal impact in comparison to the high sediment loading from the natural surrounding environment in large rainfall events.

8.3 Environmental Impacts

The Kintyre Project lies between the two tributaries of the Yandagooge Creek which converge immediately downstream of the project site and flow north to the Coolbro Creek. The Coolbro Creek is an internally draining basin which dissipates into the sandy desert environment. Any changes to the natural hydrology in the Coolbro Creek will not impact the adjacent Rudall River National Park as the systems are not hydraulically connected.

Generally the mining operations will not significantly impact on the natural flow regime of the Yandagooge Creek with respect to the timing and volume of natural flow in the creek system. The proposed flood embankment would minimally impact the natural timing and magnitude of flows in the South Branch but not impact on the total volume of downstream flow.

An area of approximately 10 km^2, encompassing the mine pit and associated infrastructure, will be bunded and hydraulically disconnected from the surrounding environment. Stormwater that collects within this area will be treated and used for mine processing with zero release to the environment. It is estimated that this will result in a reduction of approximately 2 % of the total flow volume from the 470 km^2 upstream catchment.

9 Risk Considerations

The frequency of rainfall and flooding events has been represented by an average recurrence interval (ARI). It is important to note that an ARI of 100 years does not mean that the event will only occur once every 100 years. In fact, for each and every year, there is a 1% chance (a 1 in 100 chance) that the event will be equalled or exceeded (once or more than once).

The likelihood of a food event being exceeded over the operational lifetime of the mine is provided in Table 2-1. Risk can be illustrated with the following example: for an n =5 year design life for the design element, the probability (p) of encountering a T_r=100 year ARI rainfall event is given by:

In other words, there is a 5% chance that a 100 year ARI event (or greater) flood will occur in the next 5 years.

Table 4-2 Probability of Exceedance over mine life

Design Event	Risk of Exceedance			
	Likely 5 years mine life	Likely 10 years mine life	Likely 15 years mine life	Likely 20 years mine life
5 year ARI	67%	89%	96%	99%
10 year ARI	41%	65%	79%	88%
100 year ARI	5%	10%	14%	18%
1000 year ARI	0.5%	1%	1.5%	2%

10 Limitations

The models developed for the Yandagooge Creek to assess the flood potential at the Kintyre Project site have been developed using the best available catchment information. It should be noted that the data available is of a very limited nature and, as a consequence, the uncertainties should be considered when using the outputs contained in this report.

The key uncertainties associated with the rainfall- runoff and hydraulic modelling undertaken as a part of this study are:

- Absence of field data from actual major floods
- Limited rainfall data and the rainfall variability
- Uncertainties with the South Branch of the catchment and how it contributes to surface water runoff
- The use of regional rainfall and runoff relationships, as most relationships have been derived from limited data and incorporate considerable data scatter from individual catchments.
- The 2D hydraulic model was developed without detailed field survey information of major creek cross sections, upstream and downstream creek characteristics and details of the confluences of waterways.
- Peak velocity shown on the maps is likely to be underestimated in places due to averaging of velocity across the creek cross sections and over the typical 5m triangulation cell. MWH recommends adding 50% velocity to the mapping information for conceptual design purposes.
- The peak depth indicated in the flood extent maps is based on a ground model that has a reported error of plus or minus 1m at every point.
- The hydraulic model has not been calibrated against a significantly large flood. Accordingly MWH recommends the use of an adequate freeboard when using results for concept design.
- The current modelling has not included the effects of proposed creek crossings by the mine access road. Initial design concepts of the South Branch Creek crossing are understood to be a series of culverts on the creek bed with a capacity to pass the 20 year ARI flood event with overtopping for larger magnitude events. Peak flood levels upstream of the road crossing of the south branch creek are therefore likely to be higher than those modelled in this study.

11 Recommendations

The following section provides recommendations for tasks which could be undertaken to improve confidence in the results of hydraulic modelling and to reduce the level of uncertainty as the project progresses from conceptual to feasibility and definitive design stages.

MWH recommends that Cameco undertakes the following:

1. Install recording equipment to collect continuous surface water data and rainfall. A large event that is captured by the gauge equipment and field survey or photographic evidence could be used to re-calibrate the hydrological and hydraulic models with a greater level of confidence
2. Obtain high density aerial mapping data for design purposes and include this data into an updated ground model and hydraulic model.
3. Refine the model when a more detailed design of the levee alignment and road crossing of the south branch creek become available.
4. Undertake a detailed field survey of major creek cross sections, upstream and downstream creek characteristics and details of the confluences of waterways and incorporate into the hydraulic model.
5. For specific areas of interest in the mine layout include a higher proportion of grid cells and replace the 1D cross sections with more 2D elements. Better definition of velocities at points of interest can also be modelled through localised refining of the grid mesh.
6. Review the alignment of the flood protection embankment to ensure that the layout is optimised with respect to mine infrastructure and access roads.
7. Model the levee alignment in different locations to determine the effects on flows and levee height.
8. Model the access road creek crossing to assess the impacts of the backwater effect upstream of the crossing. This will increase the levee height required upstream of the crossing.
9. Undertake a risk assessment to determine a suitable design standard for the flood protection embankment and access road creek crossing.

12 Glossary

Annual Exceedance Probability (AEP): The chance of a flood of a given size (or larger) occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (i.e. a 1 in 20 chance) of a peak discharge of 500 m^3/s (or larger) occurring in any one year.

Average Recurrence Interval (**ARI**)**:** The frequency that a flood of a given size will recur on average. For example if a flood discharge has an ARI of 20 that means a flood of that size or greater will occur on average every 20 years.

Hydrograph: a plot of the variation of discharge with respect to time.

Hyetograph: a graphical representation of the variation of rainfall depth or intensity with time.

Probable Maximum Flood (PMF): The most severe flood that is likely to occur at a particular location. Such a flood would result from the most severe combination of critical meteorological and hydrological conditions.

Probable Maximum Precipitation (PMP): is defined by the Manual for Estimation of Probable Maximum Precipitation (WMO,1986) as:

"...the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends."

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Appendix A - River & Rainfall Monitoring Stations, 1987-1992

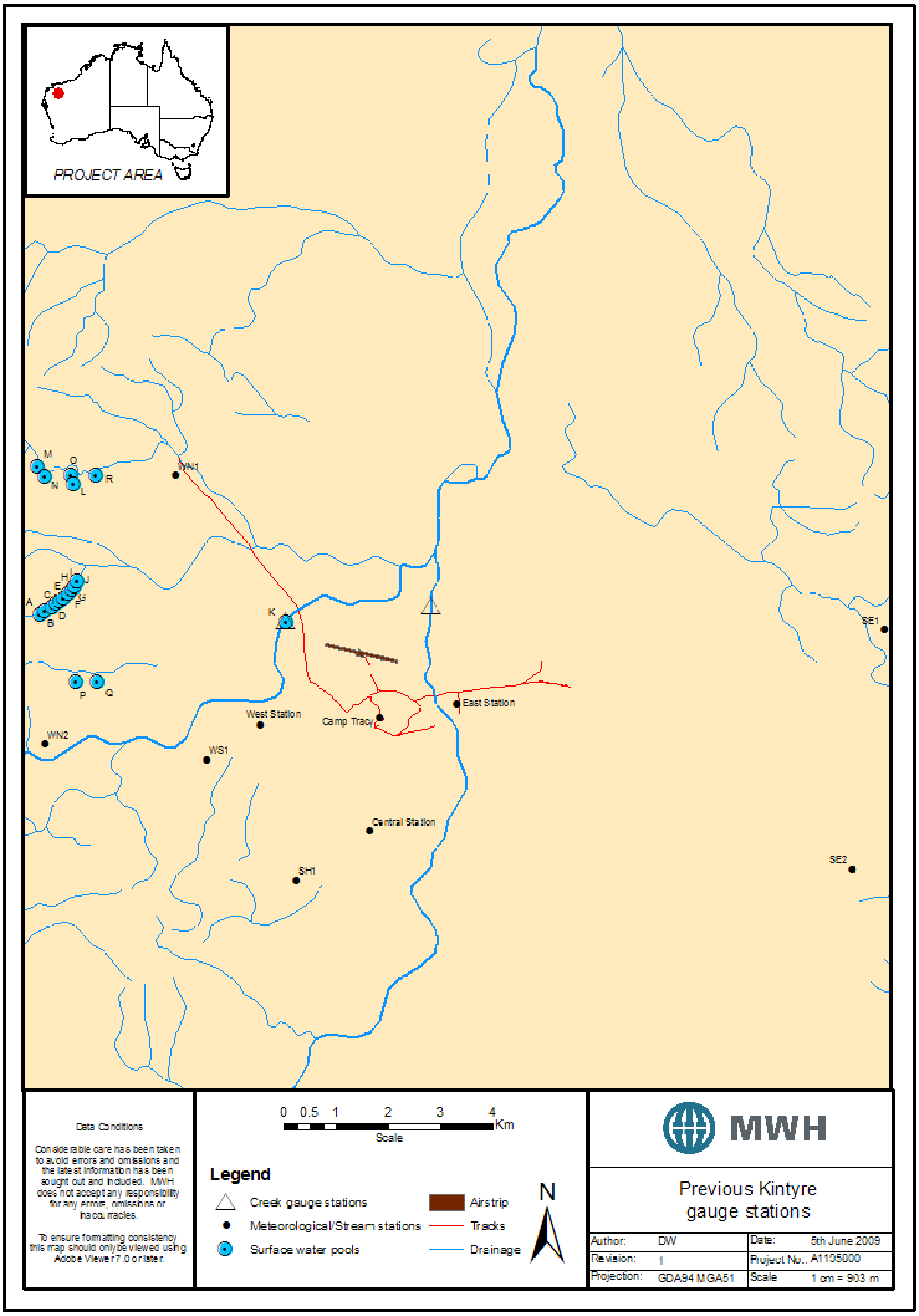


Figure 13-1 River and Rainfall Monitoring Locations

Appendix B - Kintyre Photos (May 2009)

Photo 1 Yandagooge South Branch.

Photo 2: Typical Creek Section; West Branch near North Bore crossing.

Photo 3: Red crossing; West Branch near North Bore crossing.

Photo 4: Typical sandy channel bed material.

Photo 5: Channel debris; indication of depth of creek flood flow, West Branch near North Bore crossing

Photo 6: Flood plain; South Branch near South Bore.

Appendix C- RORB Model Calibration Hydrographs

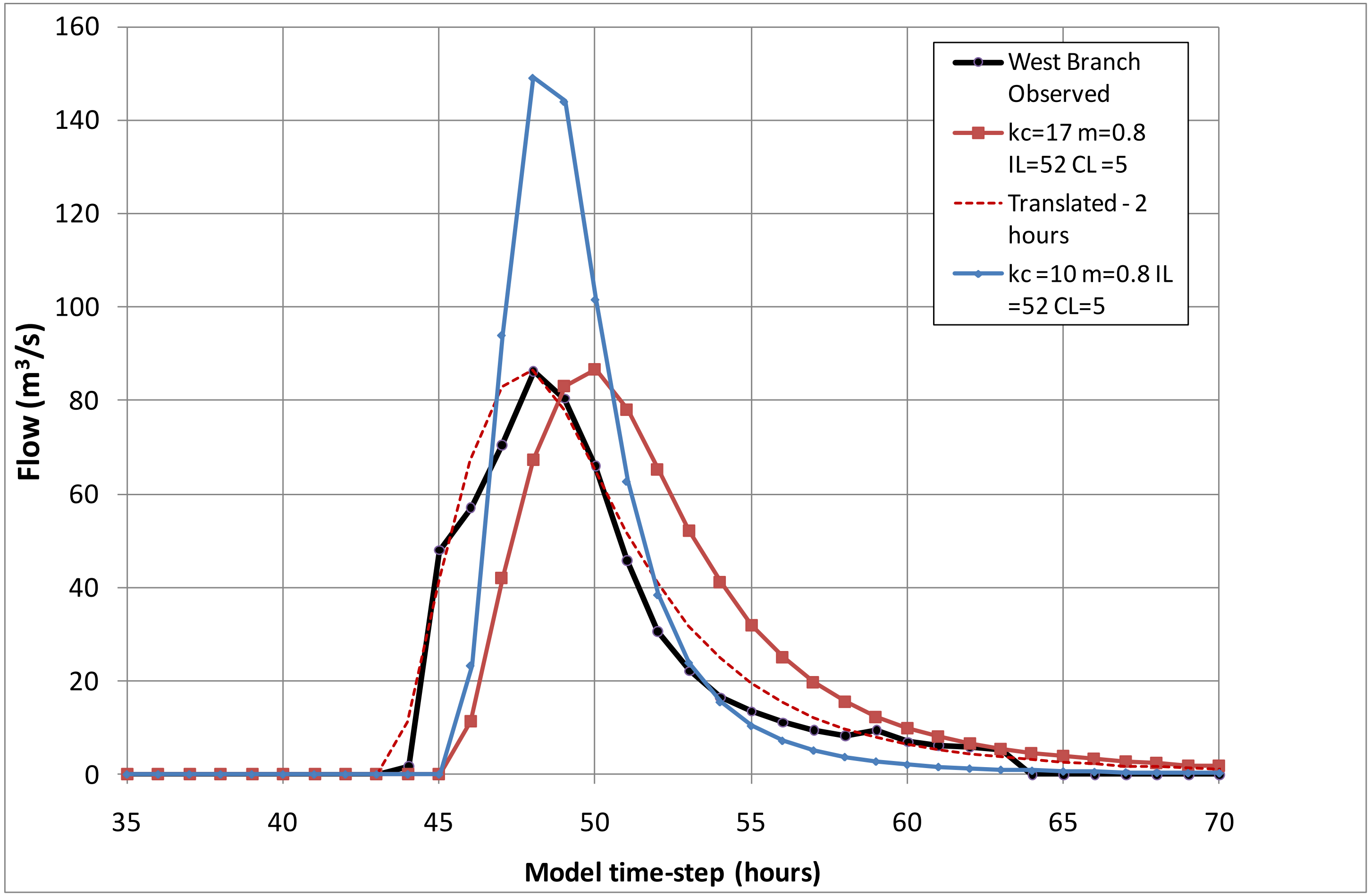


Figure 13-2 RORB Calibration Hydrograph - adjusted k_c

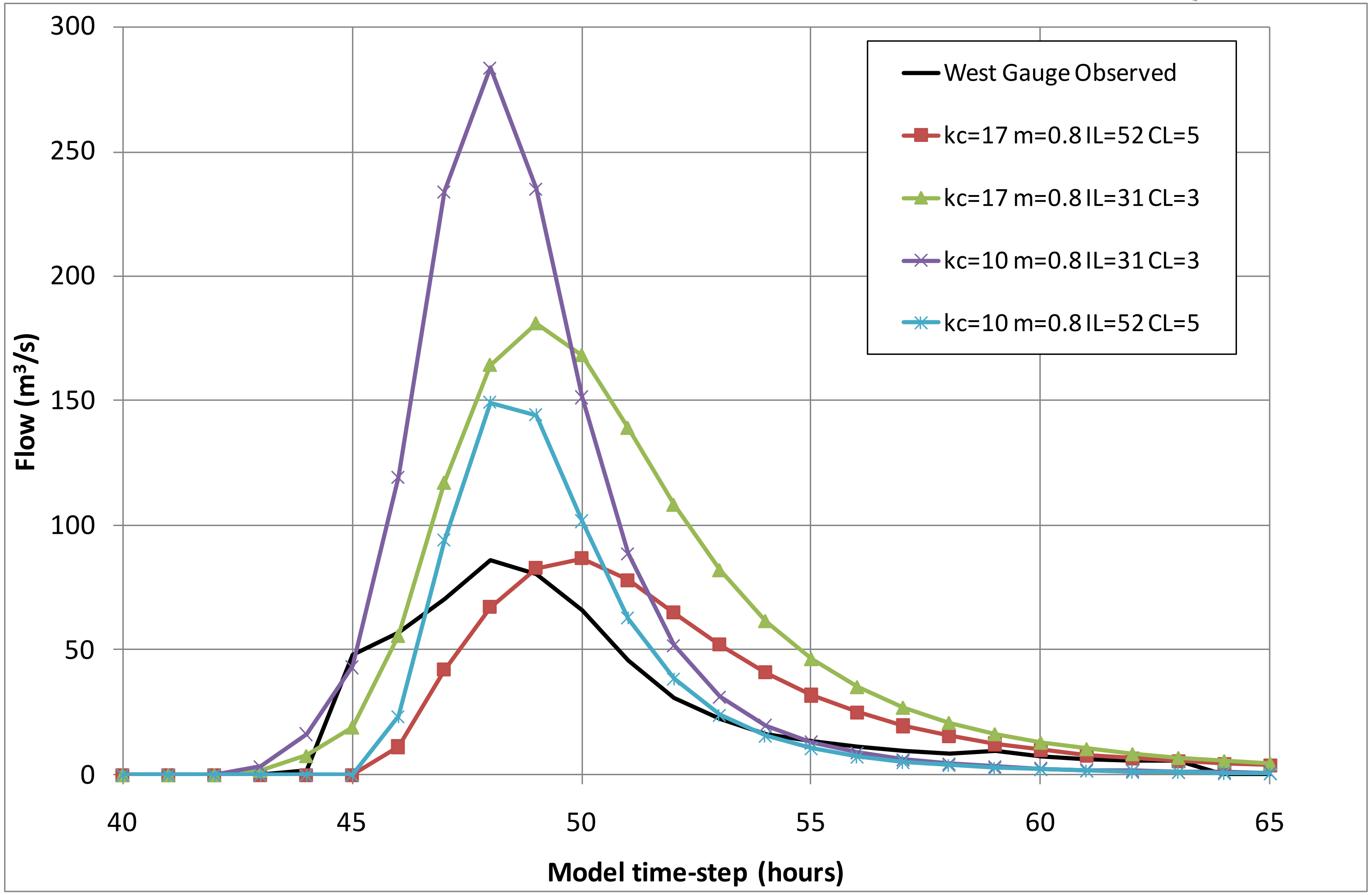


Figure 13-3 RORB Calibration Hydrograph - Regional parameter sets

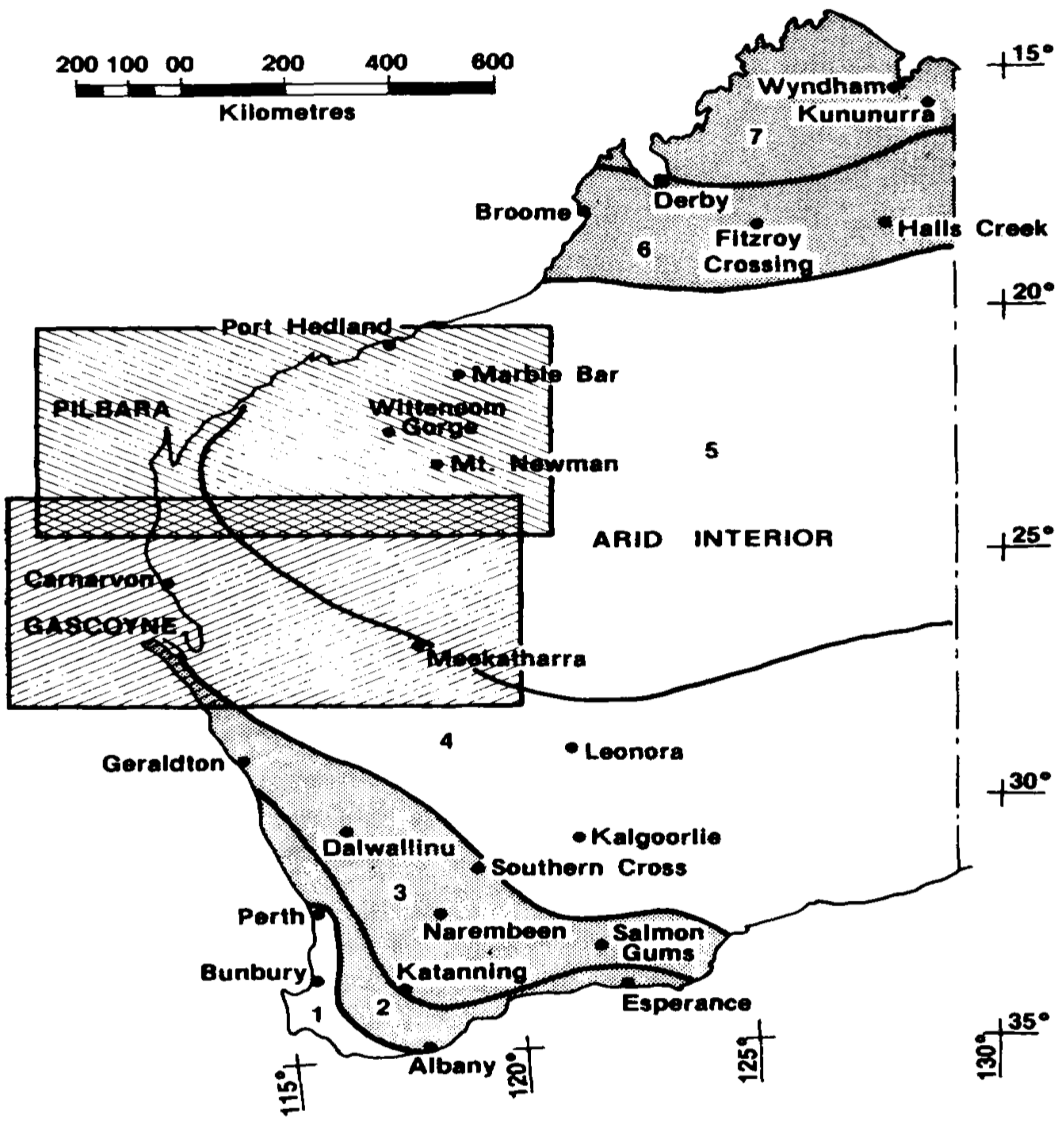
Appendix D - Regional and Rational Method Parameters

Table 13-1 Regional Method Parameters

Parameter	ARR Reference
A – Area (km^2)	Catchment areas delineated based on topographic data
P – Average Annual Rainfall (mm)	From Figure 5.8, Vol.2
Q_{100}/Q_5 – 2yr Frequency Factor	From Section 1.4.7, Vol.1, Book IV for catchments of sizes around $1km^2$, $10km^2$ and $100km^2$
Q_{10}/Q_5 – 10yr Frequency Factor	From Section 1.4.7, Vol.1, Book IV for catchments of sizes around $1km^2$, $10km^2$ and $100km^2$
Q_{100}/Q_5 – 100yr Frequency Factor	Extrapolated from frequency factors for lesser ARIs for catchments of sizes around $1km^2$, $10km^2$ and $100km^2$. Section 1.4.7, Vol.1, Book IV.

Table 13-2 Rational Method Parameters

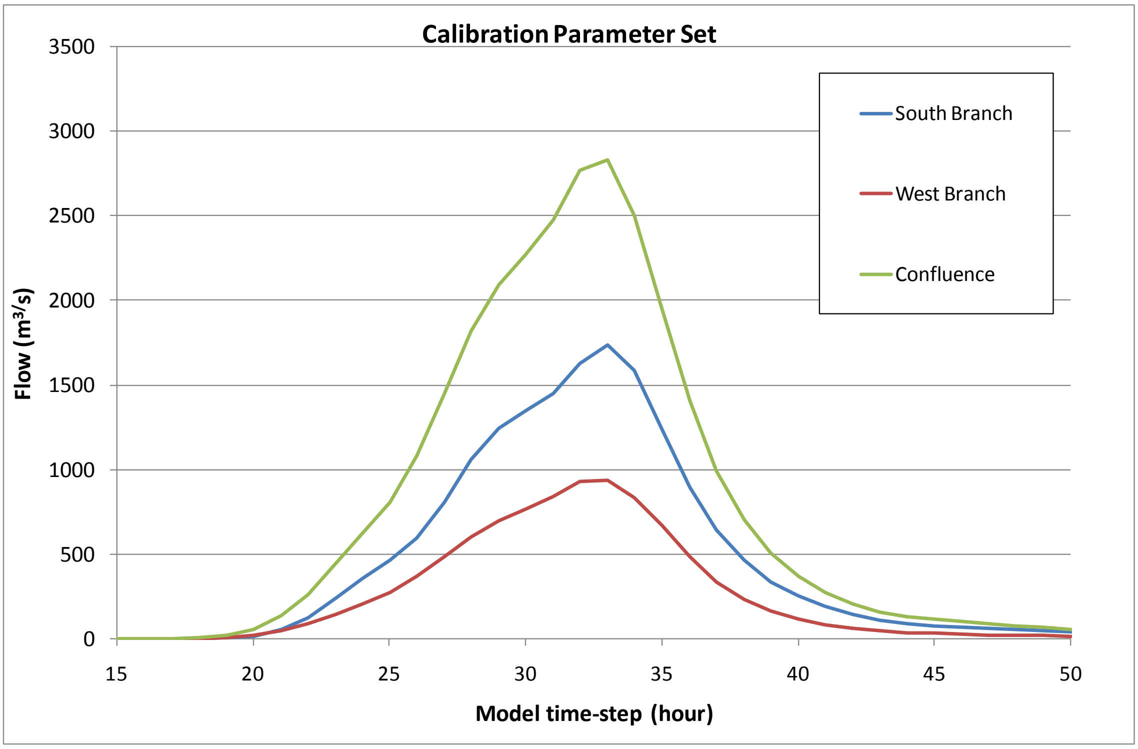
Parameter	ARR Reference
A – Area (km^2)	Catchment areas delineated based on topographic data
L – Mainstream Length (km)	Catchment outlet to most remote point on catchment boundary
t_c – Time of Concentration (min)	Calculated from catchment area using Equation 1.29, ARR Vol.1, Book IV, Section 1.4.7
I_{tc} – Rainfall Intensity (mm/hr)	From Kintyre IDF Table supplied by BOM (see Figure 5-4)
C_2 – Runoff Coefficient	Calculated from mainstream length using Equation 1.30, ARR Vol.1, Book IV, Section 1.4.7
C_2/C_2 – 2yr Frequency Factor	From frequency factors given in ARR Vol.1, Book IV, Section 1.4.7
C_{10}/C_2 – 10yr Frequency Factor	From frequency factors given in ARR Vol.1, Book IV, Section 1.4.7
C_{100}/C_2 – 100yr Frequency Factor	Extrapolated from frequency factors for lesser ARI"s given in ARR Vol.1, Book IV, Section 1.4.7

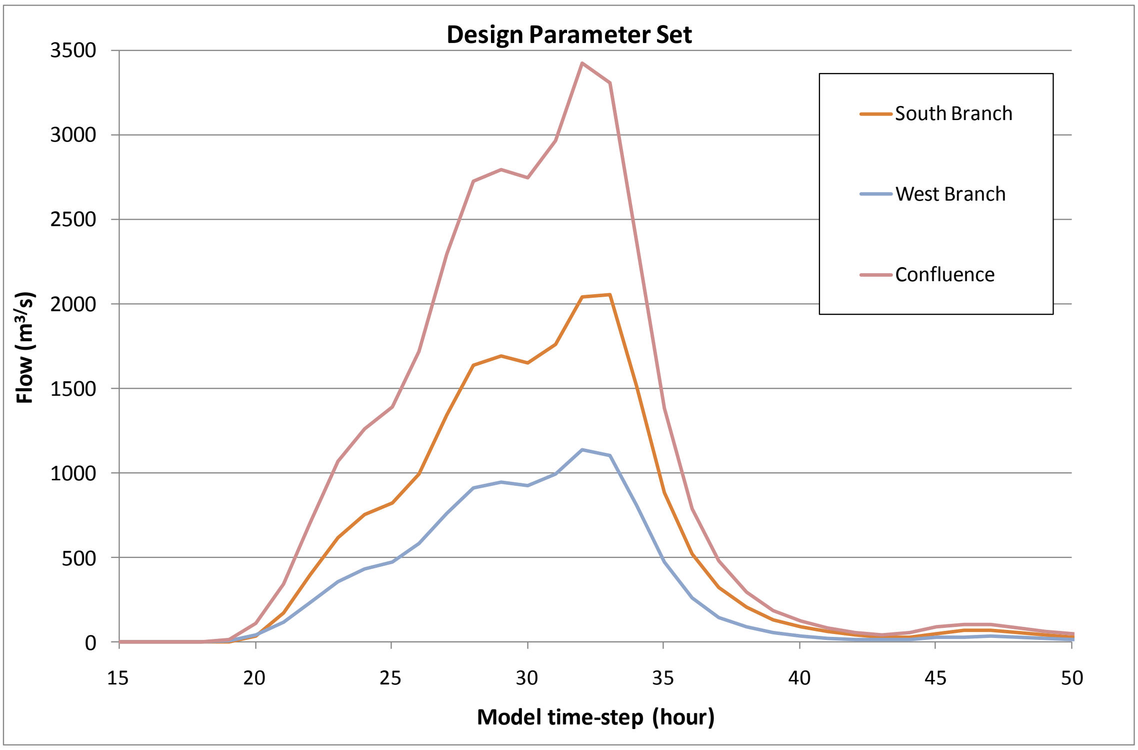


Region	Zone No.	Description	Average annual rainfall (mm)
South West	1	High winter rainfall	> 890
Wheatbelt	2	Medium winter rainfall	510 - 890
	3	Low winter rainfall	255 - 510
North West + Arid Interior	4	Arid zone	< 255
	5	Arid zone	< 380
Kimberley	6	Low summer rainfall	380 - 635
	7	Medium summer rainfall	635 - 1 400

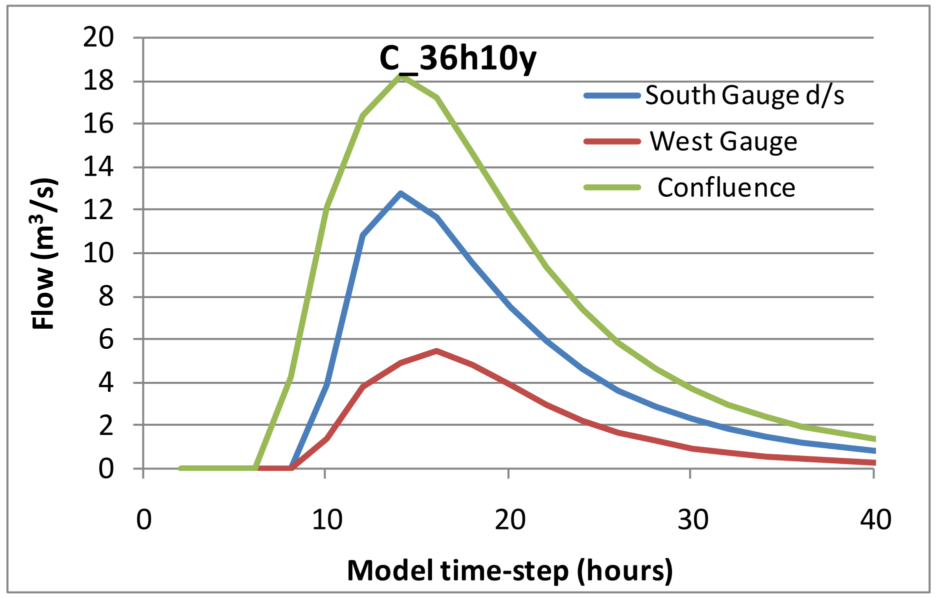
Figure 13-4 Regions of Western Australia

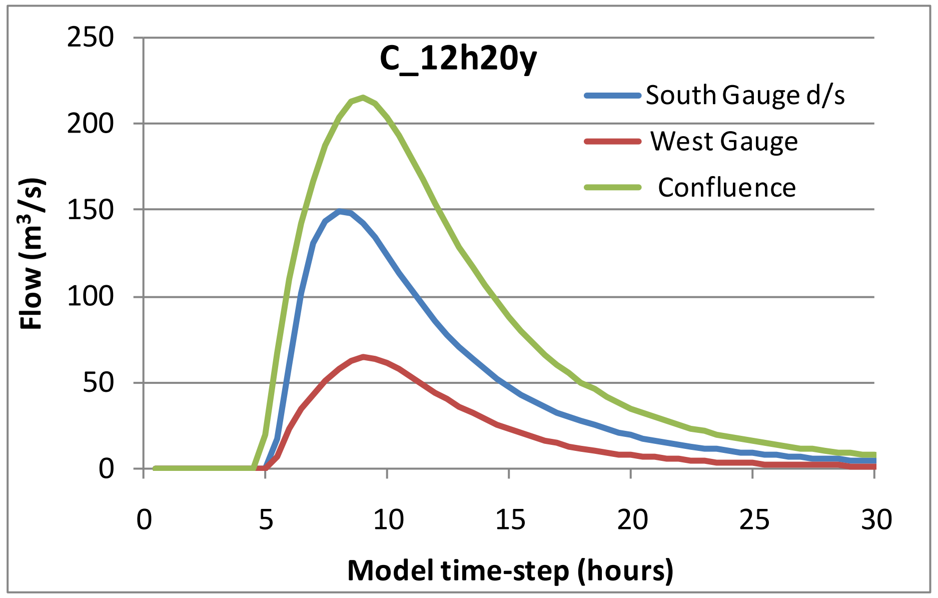
Appendix E - Telfer Hydrographs estimated using the calibration and design parameter sets

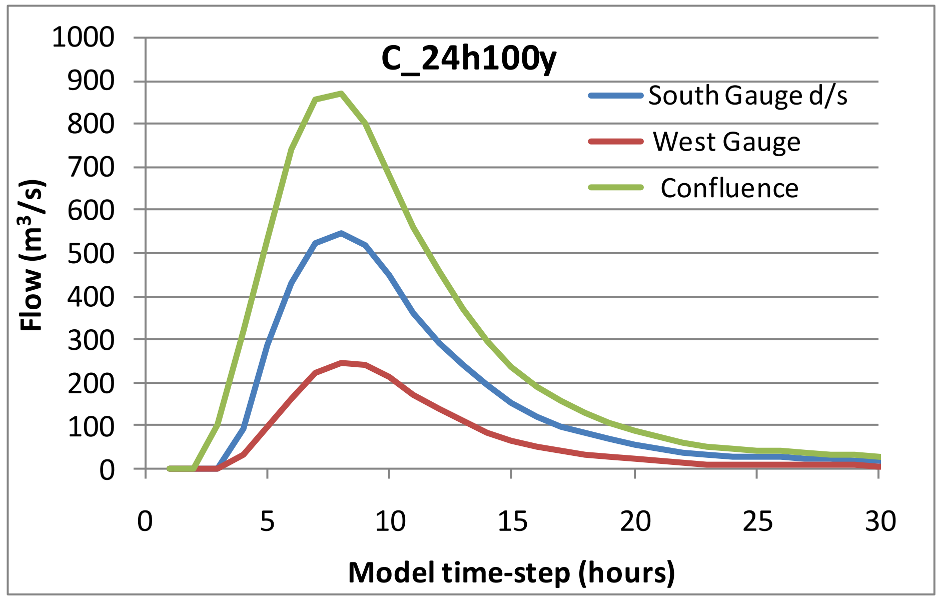


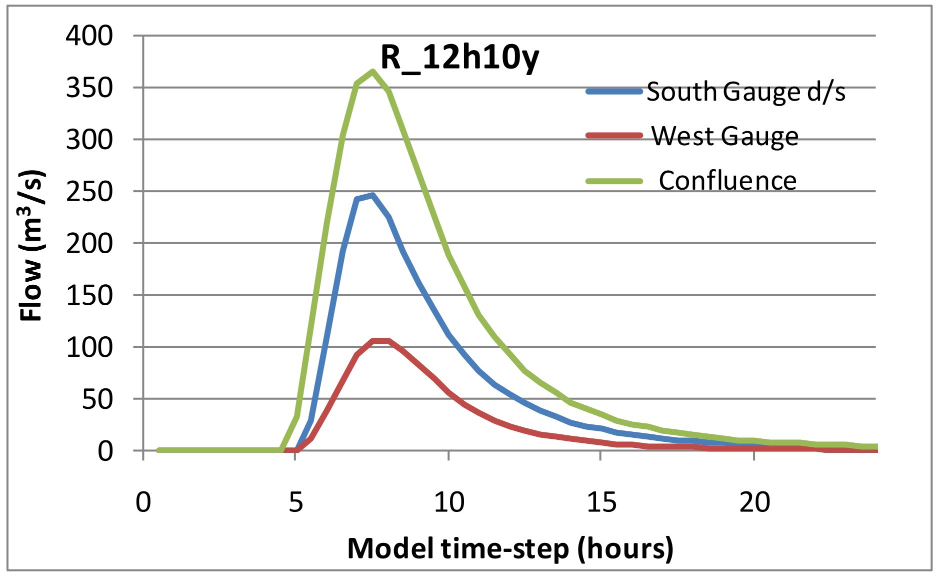


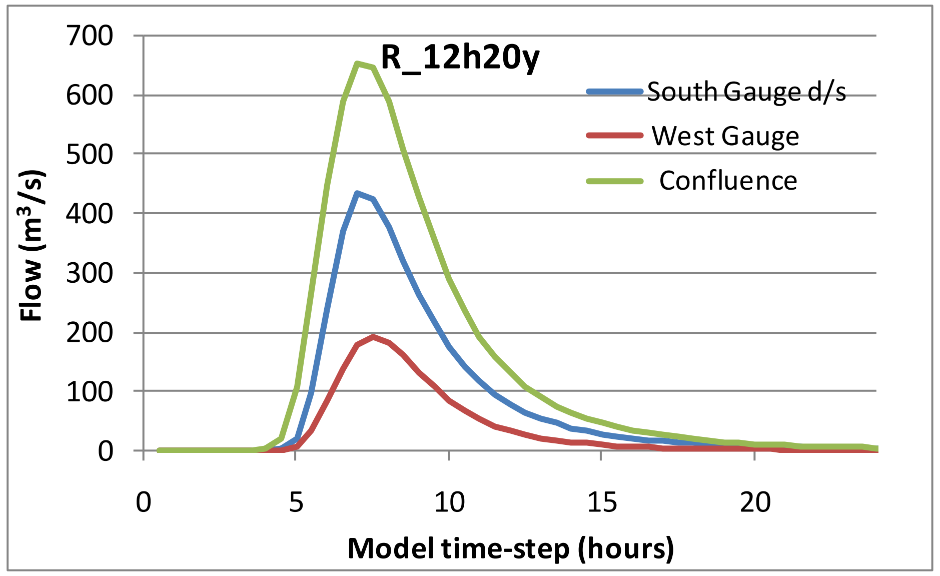
Appendix F – RORB Hydrographs

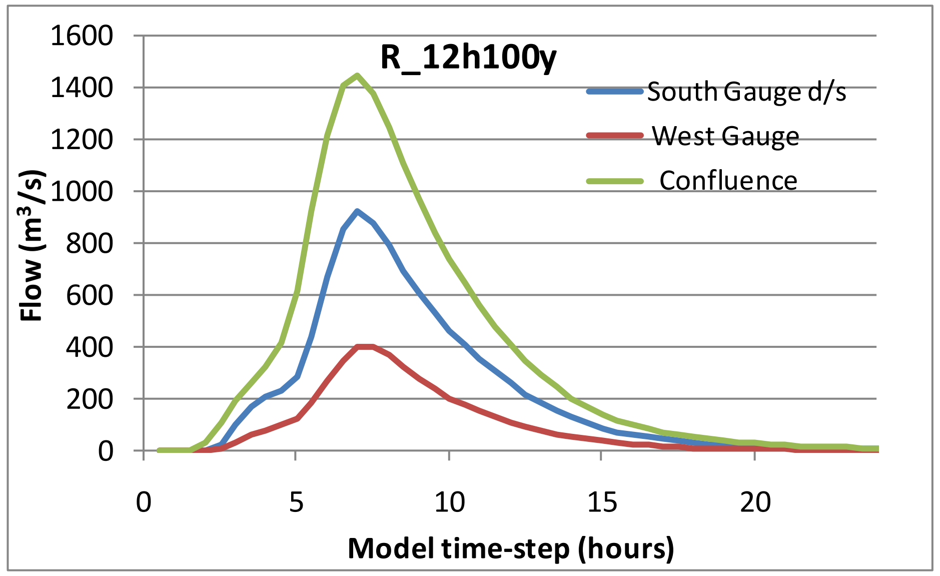


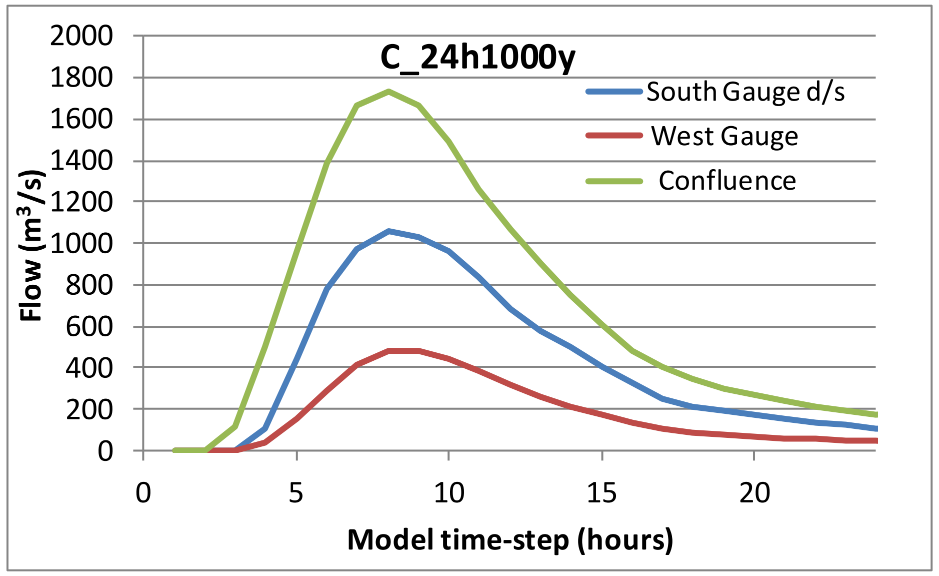


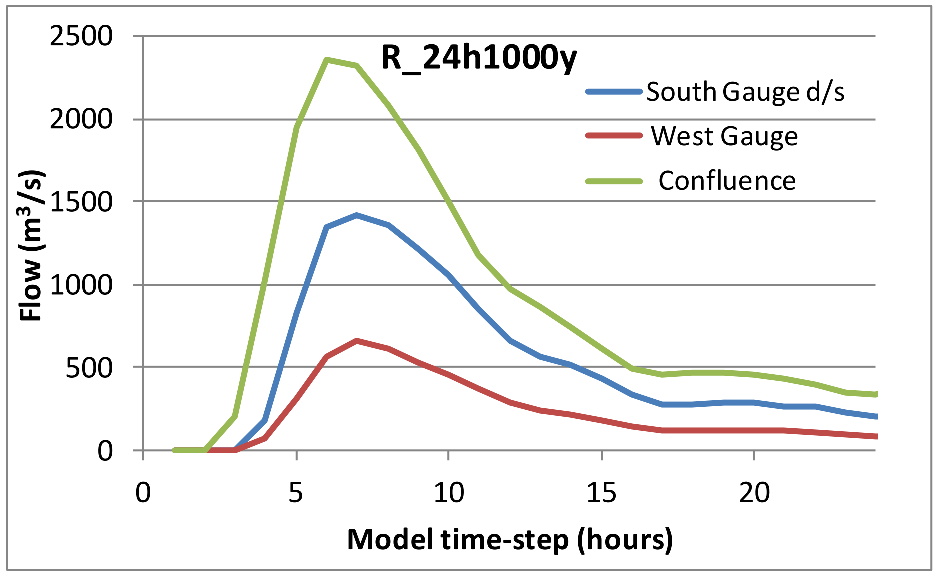




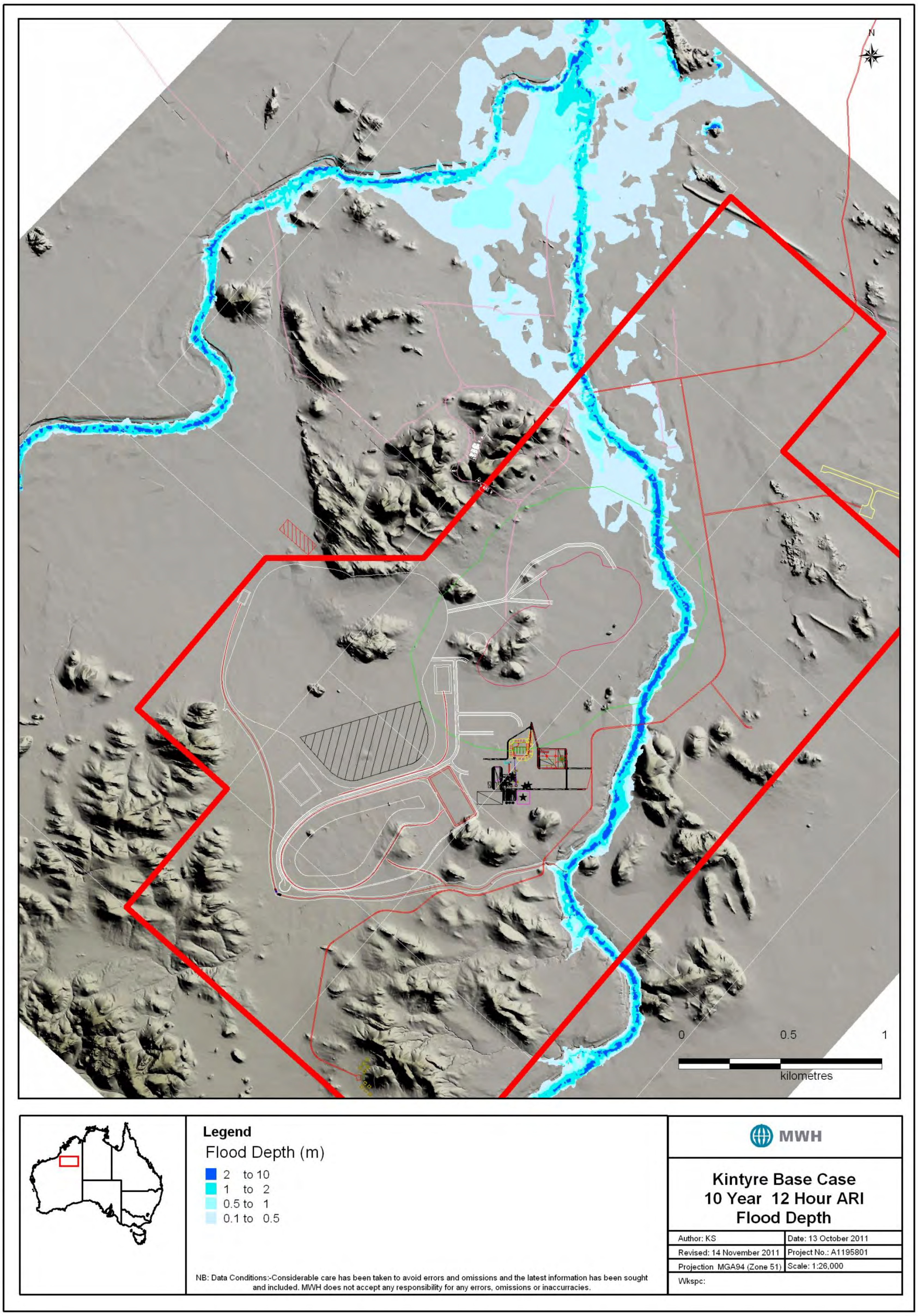


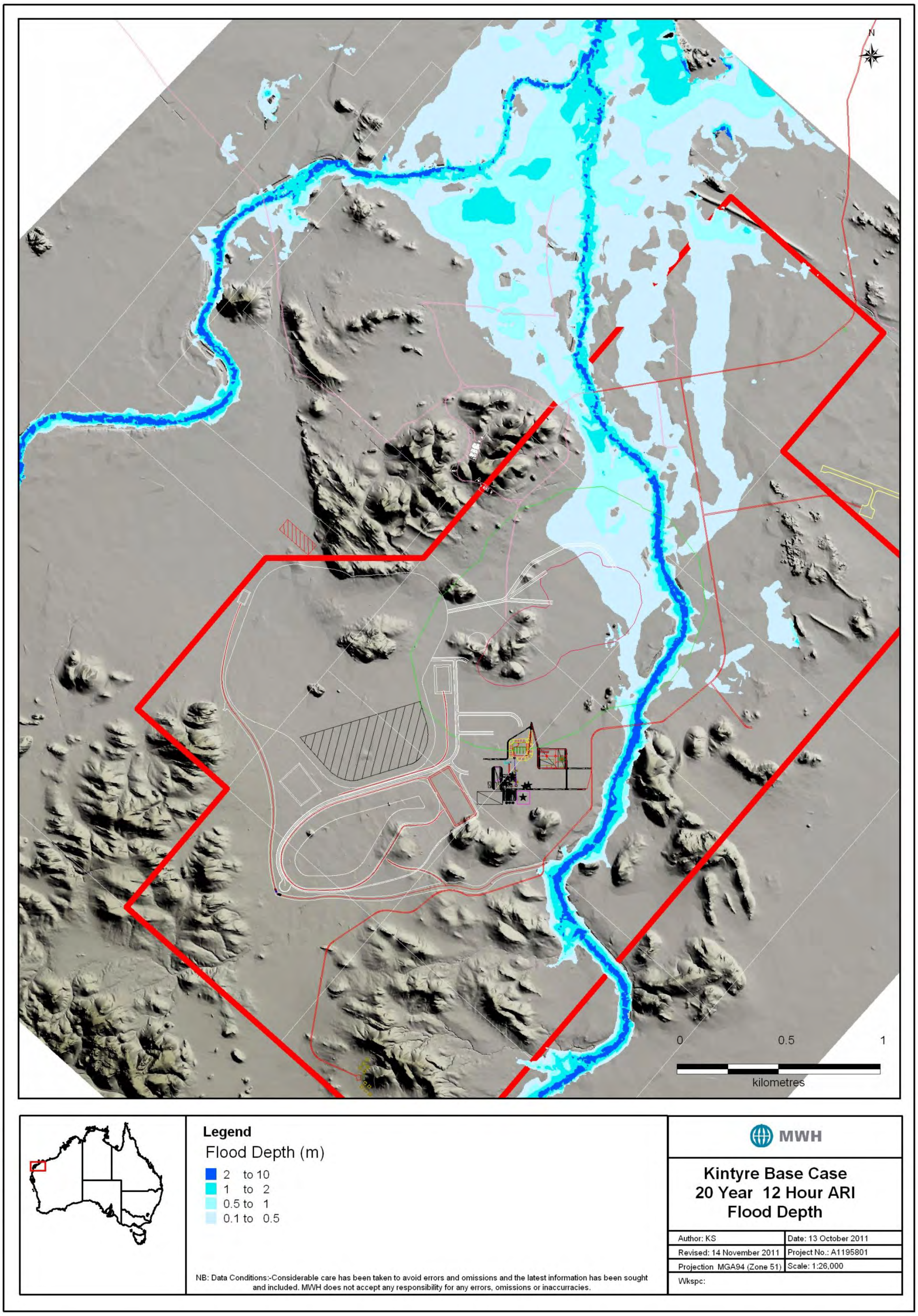



C_24h1000y
Flow (m³/s)
Model time-step (hours)
South Gauge d/s
West Gauge
Confluence


R_24h1000y
Flow (m³/s)
Model time-step (hours)
South Gauge d/s
West Gauge
Confluence

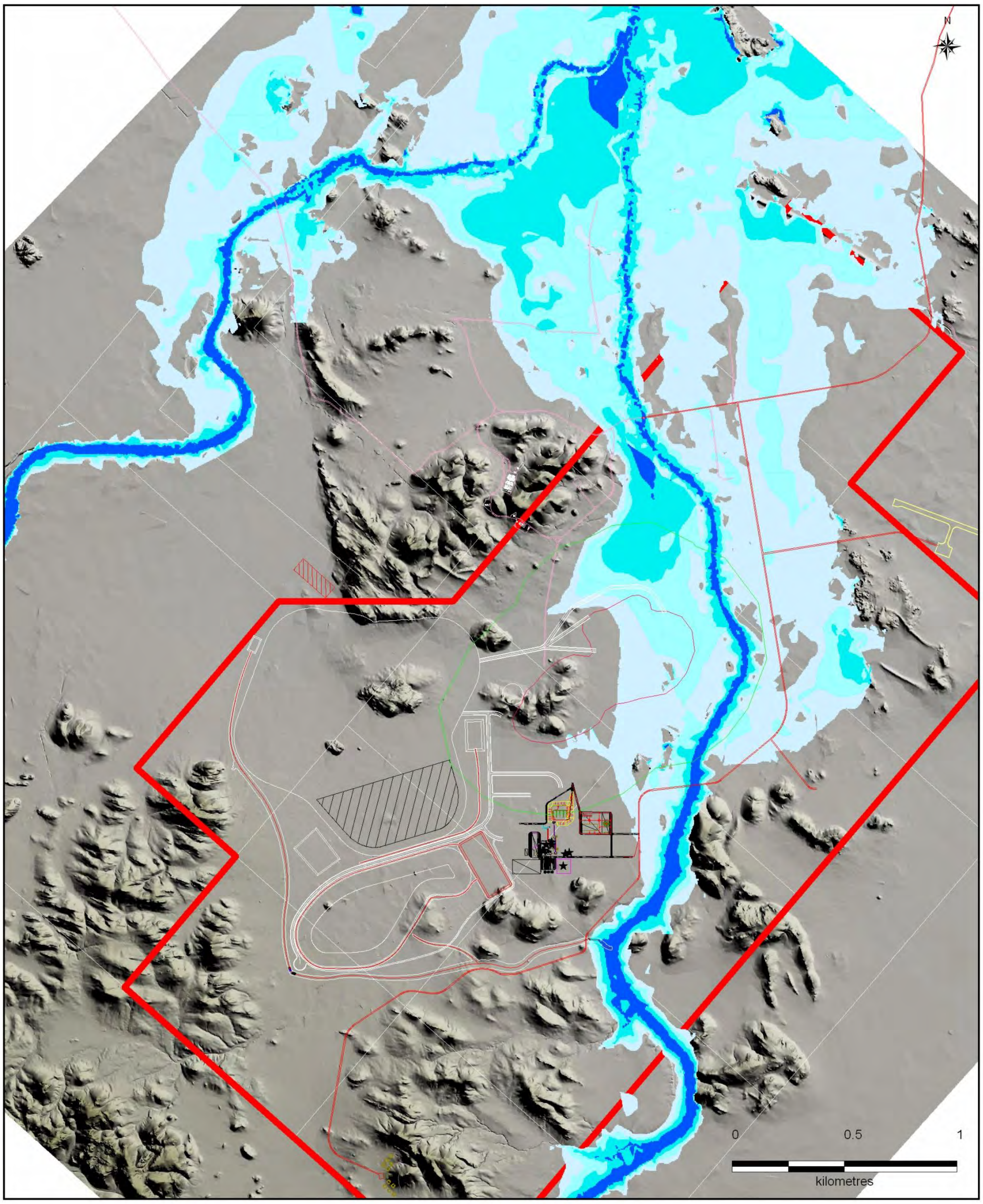
Appendix G – Base Case - Flood Extent Maps

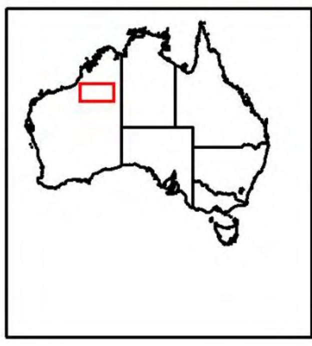




N
0
0.5
1
kilometres
Legend
Flood Depth (m)
2 to 10
1 to 2
0.5 to 1
0.1 to 0.5
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.
MWH
Kintyre Base Case
20 Year 12 Hour ARI
Flood Depth
Author: KS
Date: 13 October 2011
Revised: 14 November 2011
Project No.: A1195801
Projection MGA94 (Zone 51)
Scale: 1:26,000
Wkspc:

MWH





Legend

Flood Depth (m)

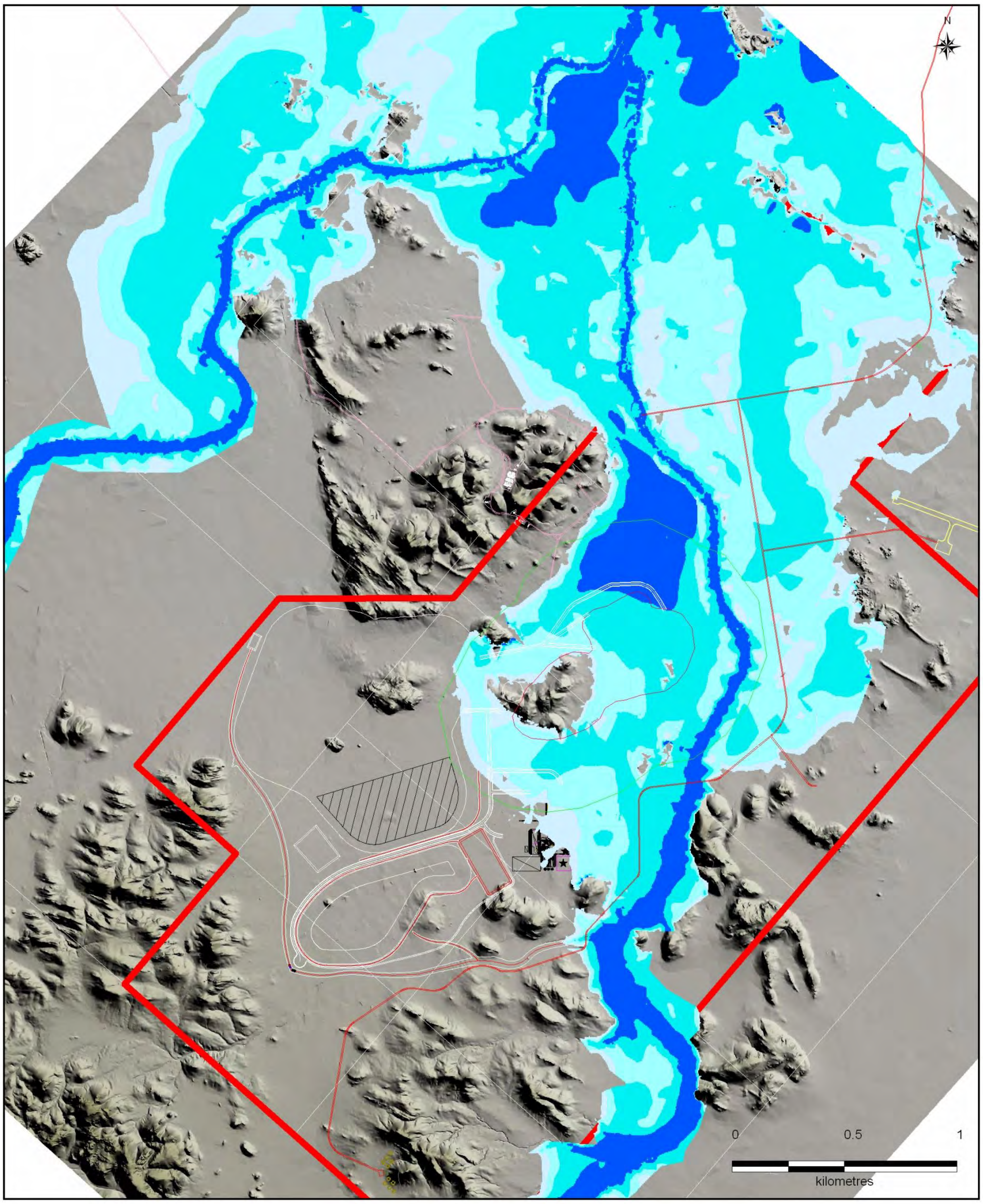
- 2 to 10
- 1 to 2
- 0.5 to 1
- 0.1 to 0.5

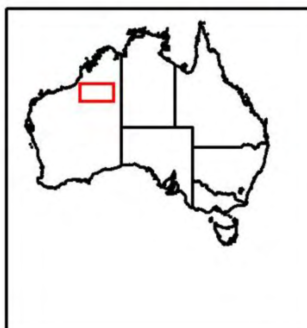
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

MWH

Kintyre Base Case
100 Year 12 Hour ARI
Flood Depth

Author: KS	Date: 8 November 2011
Revised: 14 November 2011	Project No.: A1195801
Projection MGA94 (Zone 51)	Scale: 1:26,000
Wkspc:	





Legend

Flood Depth (m)

- 2 to 10
- 1 to 2
- 0.5 to 1
- 0.1 to 0.5

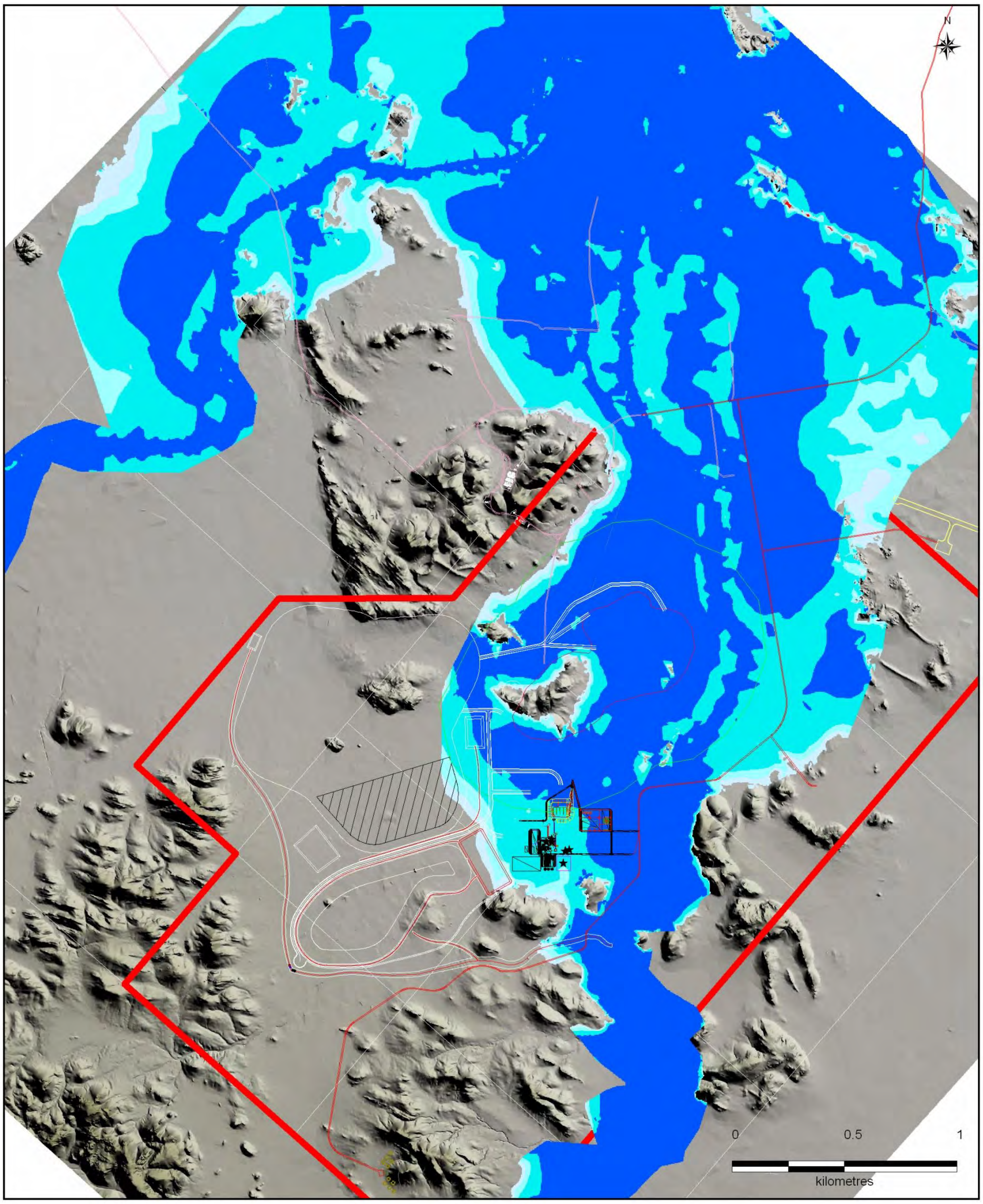
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

MWH

Kintyre Base Case
1000 Year 6 Hour ARI
Flood Depth

Author: KS	Date: 14 November 2011
Revised:	Project No.: A1195801
Projection MGA94 (Zone 51)	Scale: 1:26,000
Wkspc:	

MWH



Legend

Flood Depth (m)

- 2 to 10
- 1 to 2
- 0.5 to 1
- 0.1 to 0.5

NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

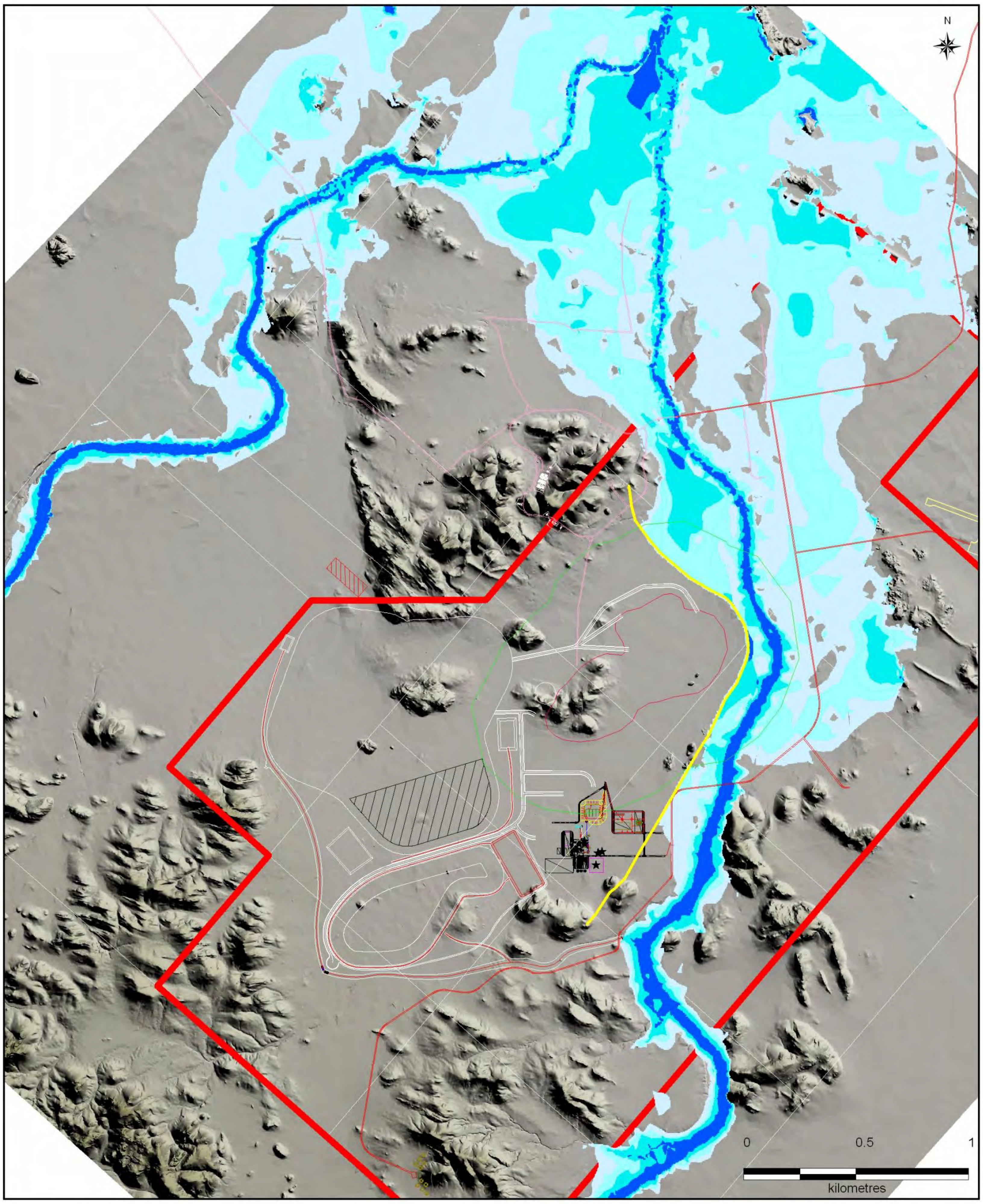
MWH

Kintyre Base Case
Probable Maximum Flood
6 Hour Duration
Flood Depth

Author: KS	Date: 14 November 2011
Revised:	Project No.: A1195801
Projection MGA94 (Zone 51)	Scale: 1:26,000
Wkspc:	

MWH

Appendix H - Flood Protection Embankment - Flood Extent Maps



Legend

Flood Depth (m)

- 2 to 10
- 1 to 2
- 0.5 to 1
- 0.1 to 0.5

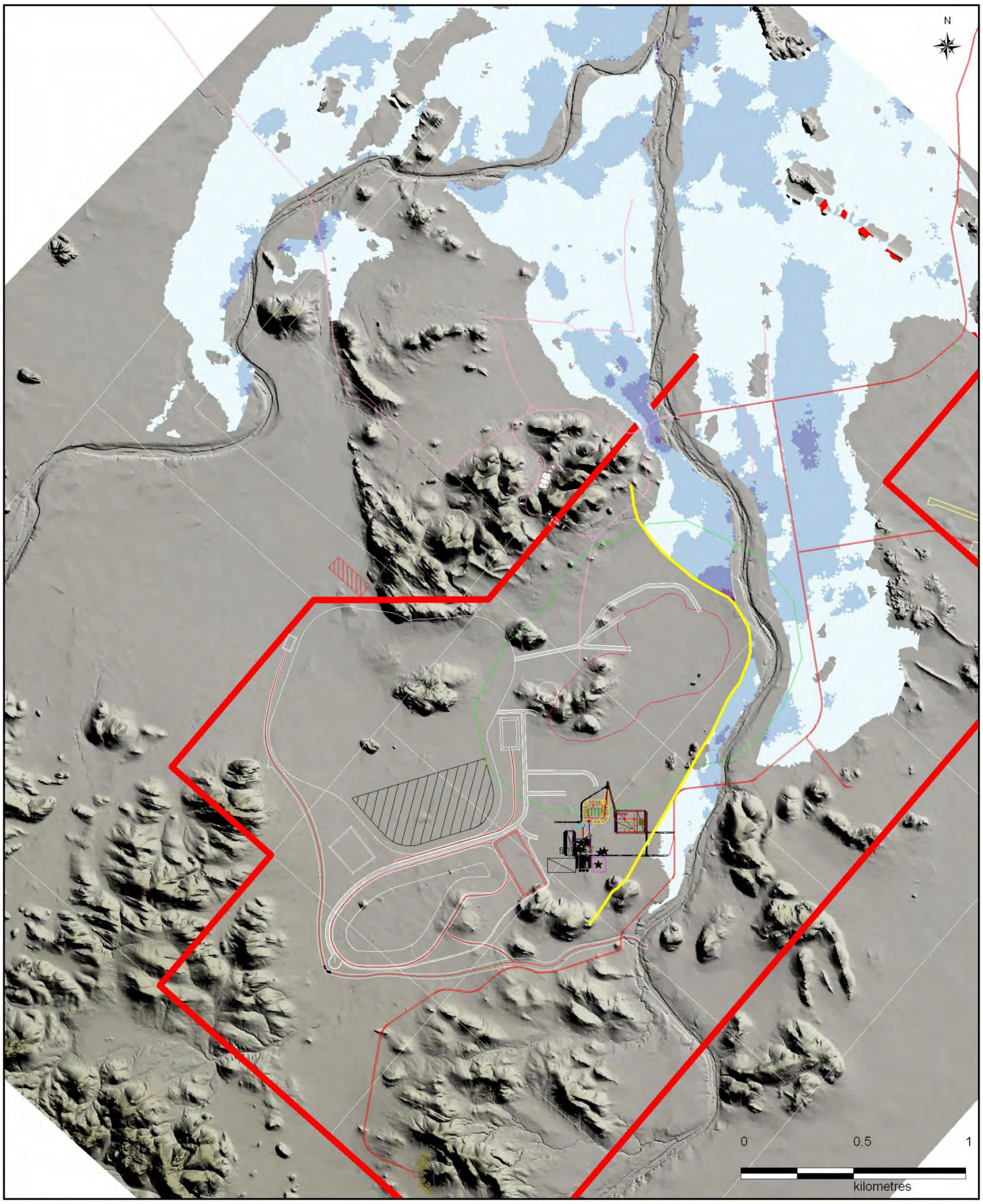
Flood Embankment

NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

MWH

Kintyre with Flood Embankment
100 Year 12 Hour ARI
Flood Depth

Author: KS	Date: 11 November 2011
Revised:	Project No.: A1195801
Projection: MGA 94, Zone 51	Scale: 1:26,000
Wkspc:	



Legend

Velocity (m/s)

- 3 to 50
- 2 to 3
- 1 to 2
- 0.5 to 1
- 0 to 0.5

Flood Embankment

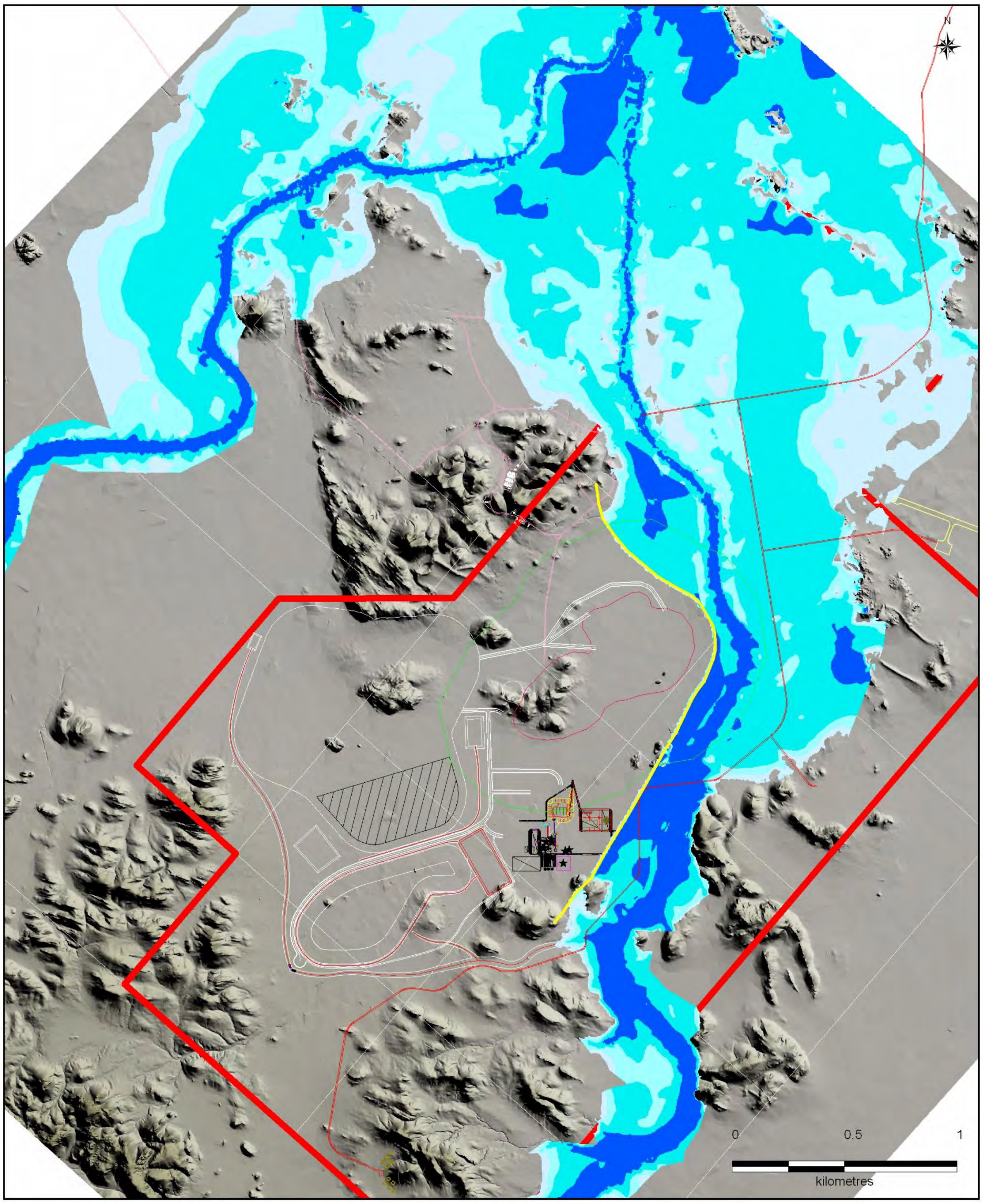
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

MWH

Kintyre with Flood Embankment 100 Year 12 Hour ARI Flood Velocity

Author: KS	Date: 11 November 2011
Revised:	Project No.: A1195801
Projection: MGA 94, Zone 51	Scale: 1:26,000
Wkspc:	

MWH



Legend

Flood Depth (m)

- 2 to 10
- 1 to 2
- 0.5 to 1
- 0.1 to 0.5

Flood Embankment

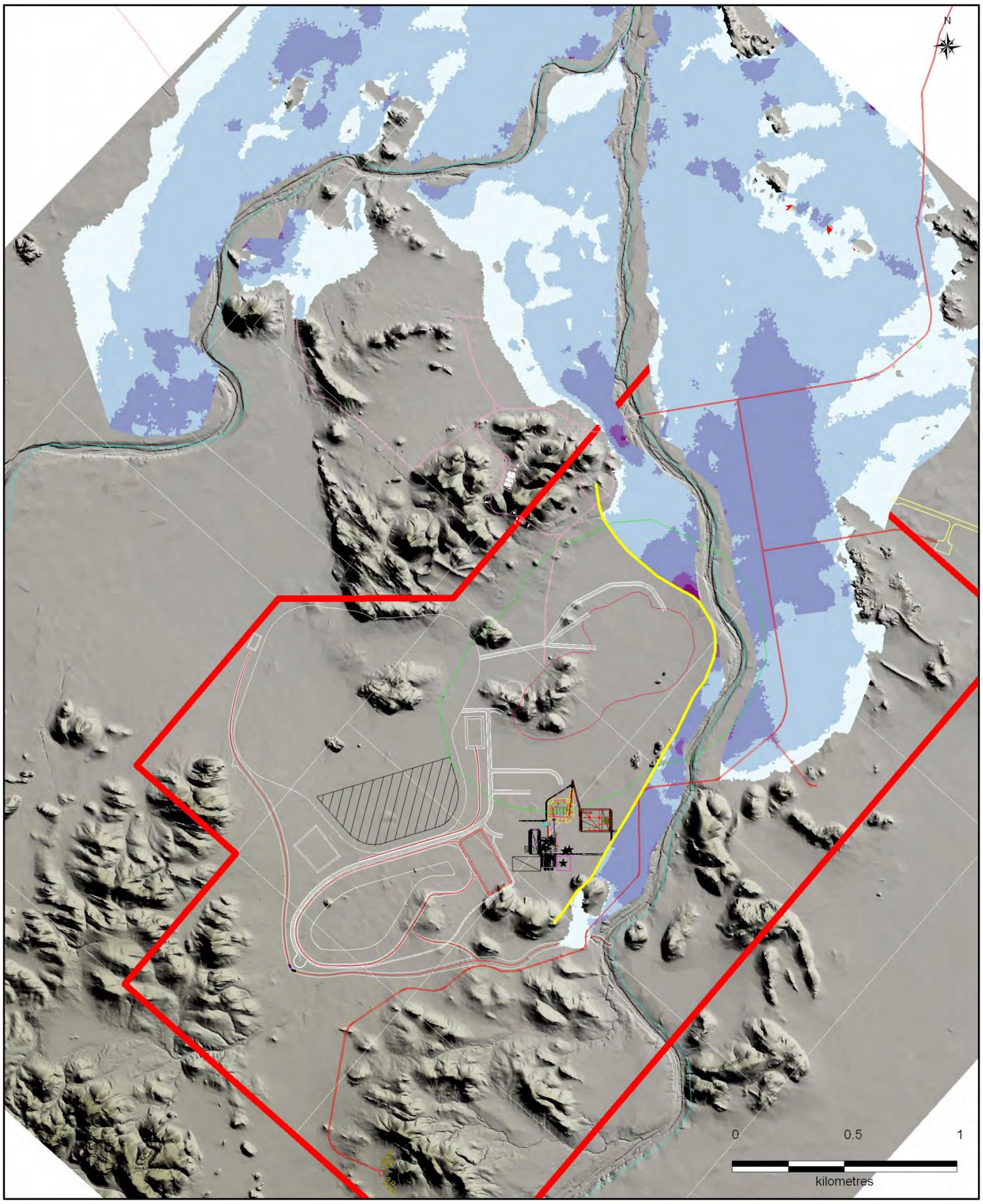
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

MWH

Kintyre with Flood Embankment
1000 Year 6 Hour ARI
Flood Depth

Author: KS	Date: 14 November 2011
Revised:	Project No.: A1195801
Projection MGA94 (Zone 51)	Scale: 1:26,000
Wkspc:	

MWH



Legend

Flood Velocity (m/s)

- 3 to 50
- 2 to 3
- 1 to 2
- 0.5 to 1
- 0 to 0.5

Flood Embankment

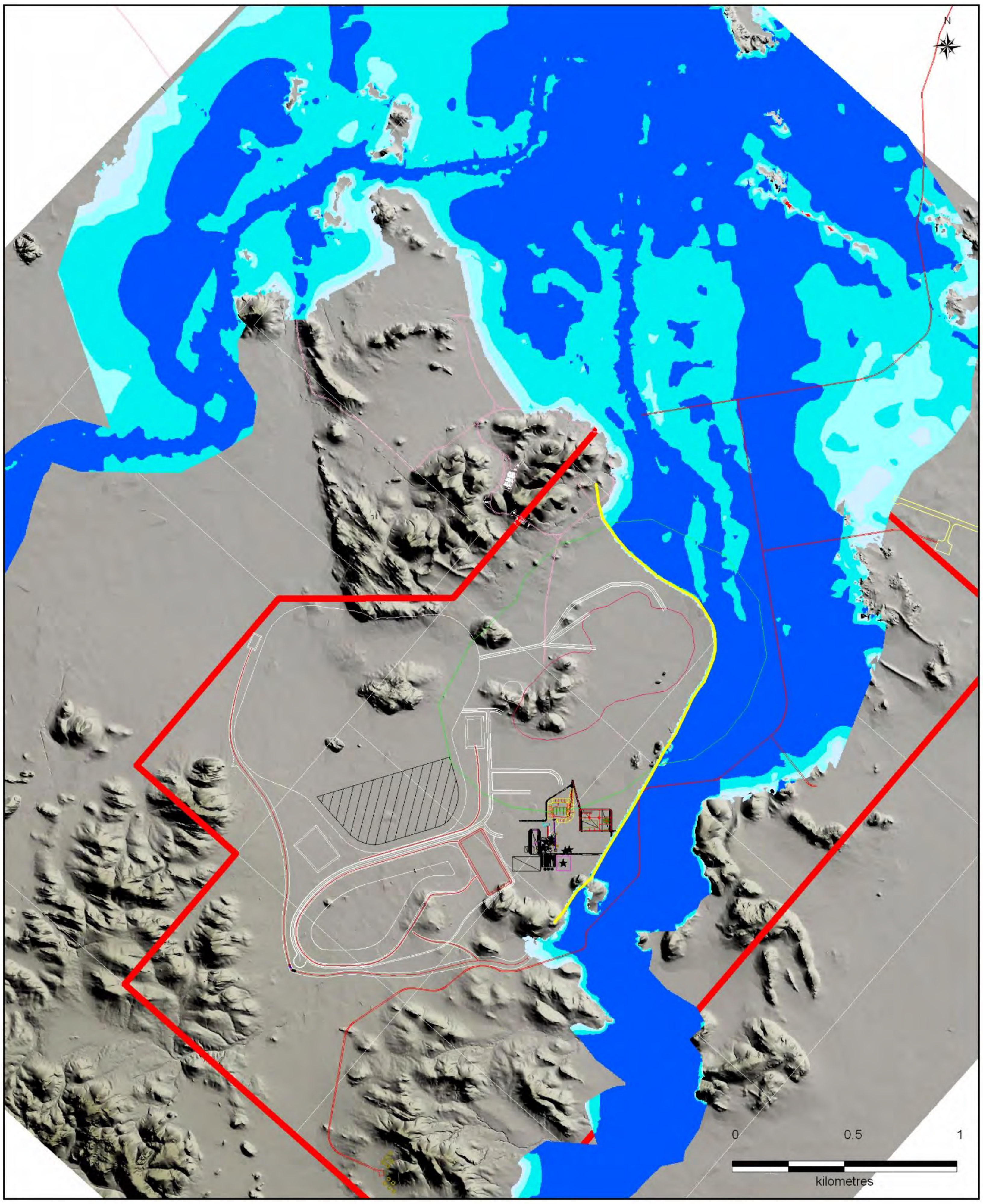
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

MWH

Kintyre with Flood Embankment
1000 Year 6 Hour ARI
Flood Velocity

Author: KS	Date: 14 November 2011
Revised:	Project No.: A1195801
Projection MGA94 (Zone 51)	Scale: 1:26,000
Wkspc:	

MWH



Legend

Flood Depth (m)

- 2 to 10
- 1 to 2
- 0.5 to 1
- 0.1 to 0.5

Flood Embankment

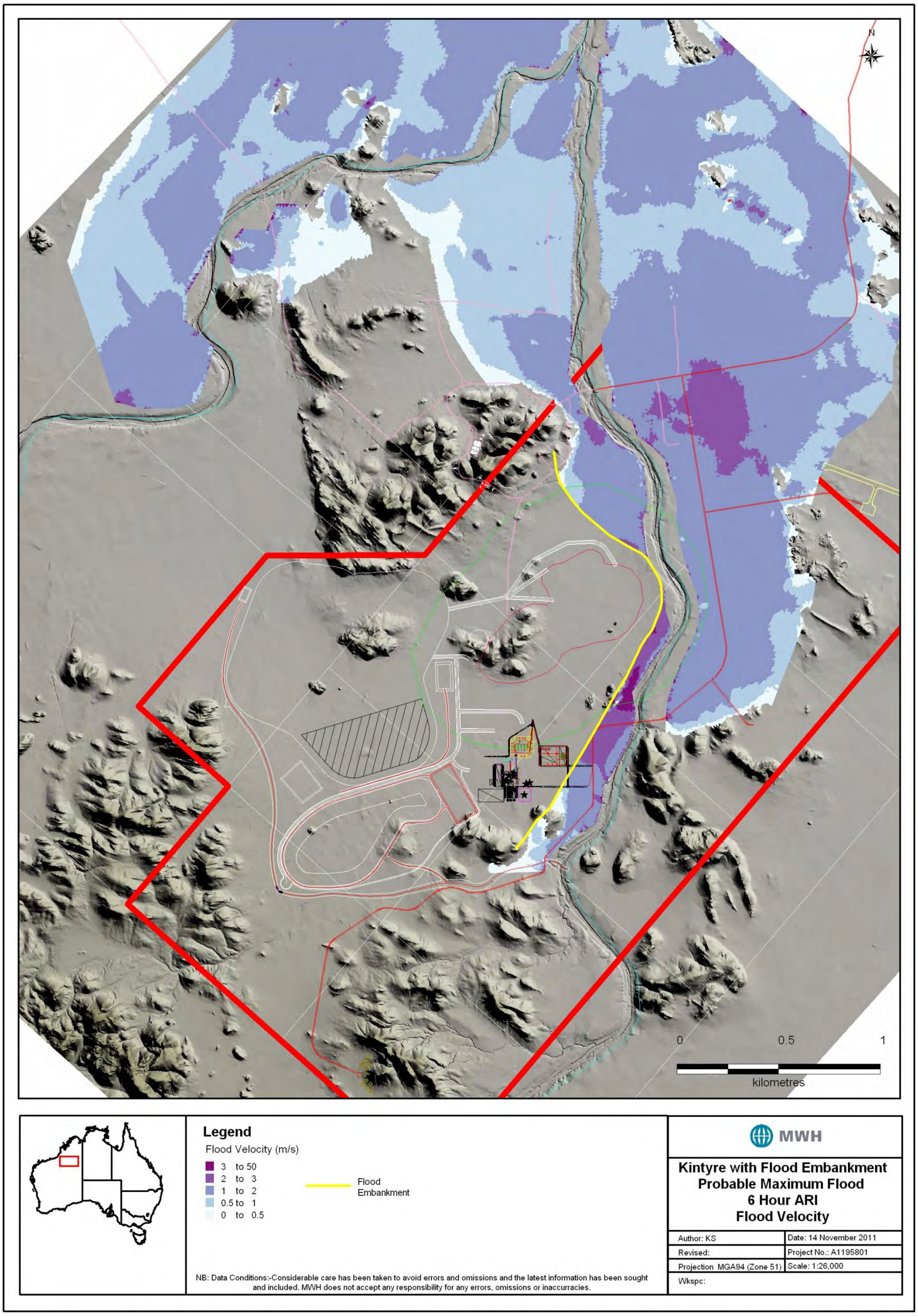
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.

MWH

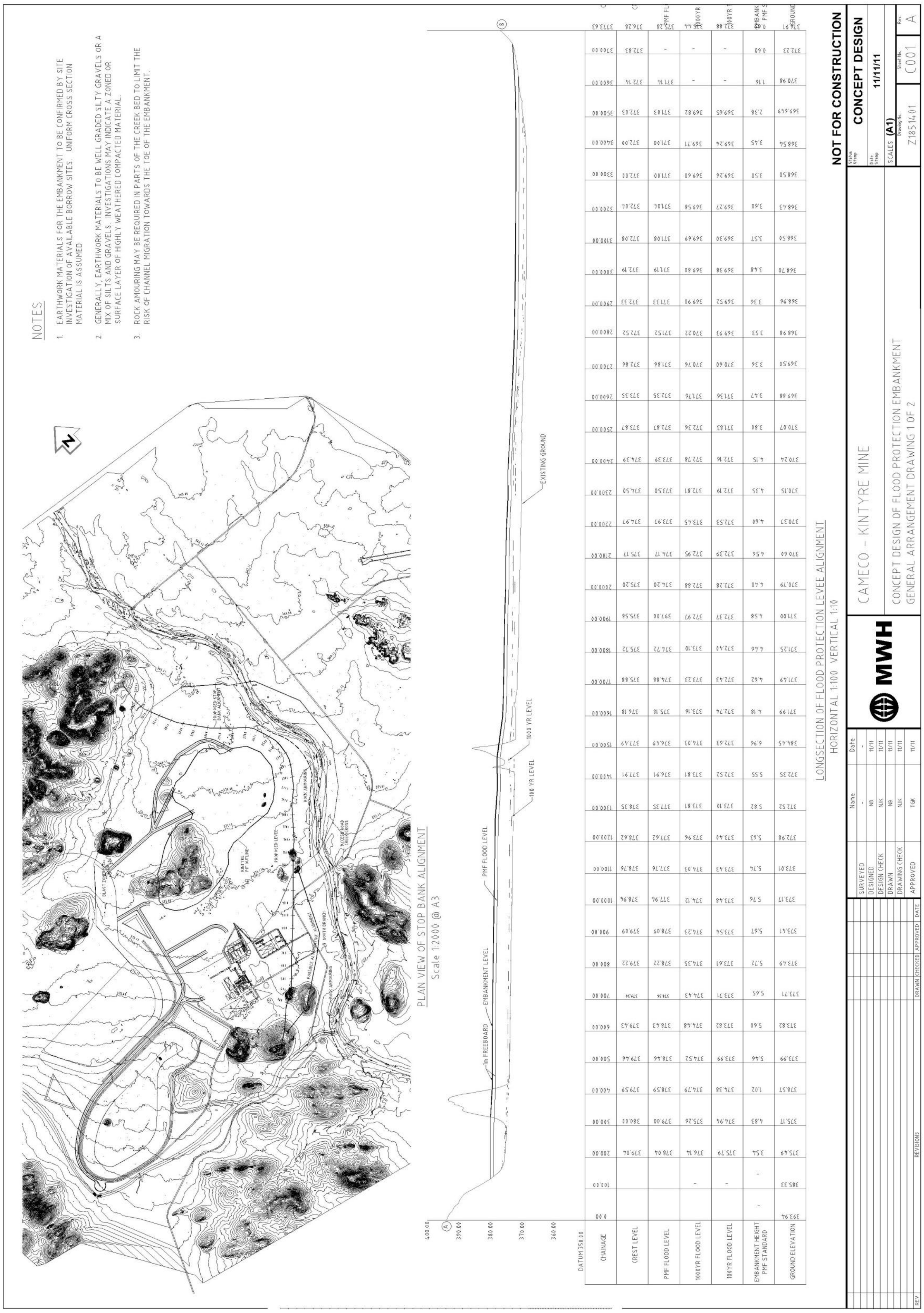
Kintyre with Flood Embankment
Probable Maximum Flood
6 Hour ARI
Flood Depth

Author: KS	Date: 14 November 2011
Revised:	Project No.: A1195801
Projection MGA94 (Zone 51)	Scale: 1:26,000
Wkspc:	

MWH


N
0
0.5
1
kilometres
Legend
Flood Velocity (m/s)
3 to 50
2 to 3
1 to 2
0.5 to 1
0 to 0.5
Flood Embankment
NB: Data Conditions:-Considerable care has been taken to avoid errors and omissions and the latest information has been sought and included. MWH does not accept any responsibility for any errors, omissions or inaccuracies.
MWH
Kintyre with Flood Embankment
Probable Maximum Flood
6 Hour ARI
Flood Velocity
Author: KS
Date: 14 November 2011
Revised:
Project No.: A1195801
Projection MGA94 (Zone 51)
Scale: 1:26,000
Wkspc:

Appendix I – Flood Protection Embankment - Conceptual Engineering Drawings



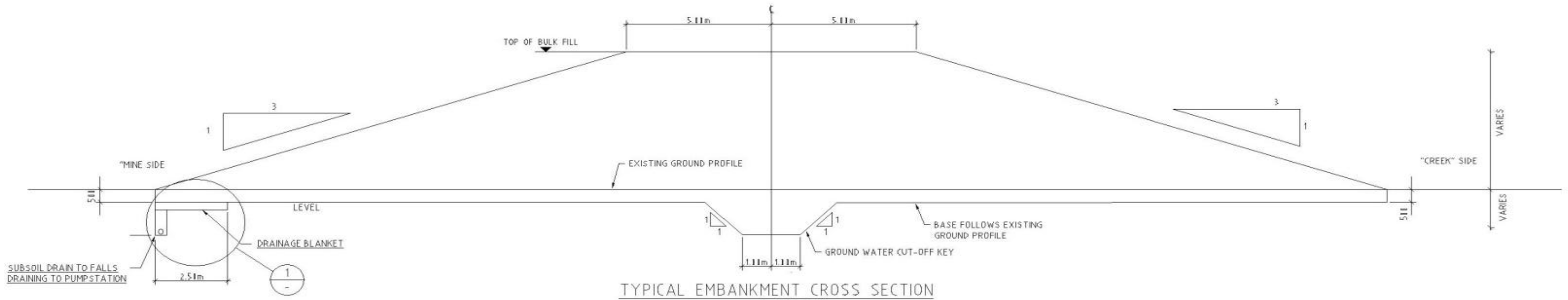
MWH

NOTES

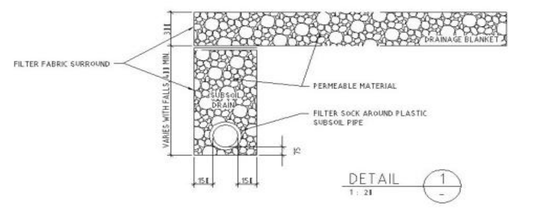
1. EARTHWORK MATERIALS FOR THE EMBANKMENT TO BE CONFIRMED BY SITE INVESTIGATION OF AVAILABLE BORROW SITES. UNIFORM CROSS SECTION MATERIAL IS ASSUMED

2. GENERALLY, EARTHWORK MATERIALS TO BE WELL GRADED SILTY GRAVELS OR A MIX OF SILTS AND GRAVELS. INVESTIGATIONS MAY INDICATE A ZONED OR SURFACE LAYER OF HIGHLY WEATHERED COMPACTED MATERIAL.

3. ROCK AMOURING MAY BE REQUIRED IN PARTS OF THE CREEK BED TO LIMIT THE RISK OF CHANNEL MIGRATION TOWARDS THE TOE OF THE EMBANKMENT.



TYPICAL EMBANKMENT CROSS SECTION



TYPICAL SUBSOIL DRAIN AND DRAINAGE BLANKET DETAIL

NTS

NOTE - RODDING EYES TO BE INSTALLED AT 50m INTERVALS BROUGHT TO SURFACE IN mPVC AND CAPPED.

NOT FOR CONSTRUCTION

	Name	Date
SURVEYED	-	-
DESIGNED	NB	11/11
DESIGN CHECK	NJK	11/11
DRAWN	NB	11/11
DRAWING CHECK	NJK	11/11
APPROVED	TGK	11/11

REV	REVISIONS	DRAWN	CHECKED	APPROVED	DATE

MWH

CAMECO - KINTYRE MINE

CONCEPT DESIGN OF FLOOD PROTECTION EMBANKMENT
GENERAL ARRANGEMENT DRAWING 2 OF 2

Status Stamp: CONCEPT DESIGN

Date Stamp: 11/11/11

SCALES (A1)

Drawing No.	Sheet No.	Rev.
Z1851401	C002	A

Appendix J – PMP Report