Report

Lake Ettamogah Winter Storage Dam Design Review

29 JANUARY 2014

Prepared for

Norske Skog Paper Mills (Australia) Ltd Albury

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Abbreviations

Abbreviation	Description
DSC	Dam Safety Committee
LEWSD	Lake Ettamogah Winter Storage Dam
PMP	Probable Maximum Precipitation
PMF	Probable Maximum Flood
ANCOLD	Australian National Committee of Large Dams
PLL	Potential Loss of Life
PAR	Population at Risk
FFC	Fallback Flood Capacity
AEP	Annual Exceedance Probability
HDPE	High Density Polyethylene
PGA	Peak Ground Acceleration
SRC	Seismology Research Centre
OBE	Operating Basis Earthquake
MDE	Maximum Design Earthquake
AHD	Australian Height Datum
ACC	Albury City Council
BOM	Bureau of Meteorology

Executive Summary

In November 2010, Norske Skog were requested by the NSW Dam Safety Committee (DSC) to undertake the following assessments regarding the Lake Ettamogah Winter Storage Dam:

- Re-assess the consequence categories considering the new developments downstream of the dam
- Undertake a new flood study and include in the next Surveillance Report
- Have a stability analysis of the dam carried out within the next 12 months and forward the results.

On behalf of the Norske Skog Paper Mills (Australia) Ltd., URS have prepared this report to address these requests. Subsequent to being engaged to complete this work, the NSW DSC informed URS that a stability analysis would not be required. However as part of the geotechnical assessment of the site, URS had proposed to undertake a piping assessment. This component of work was completed as part of this study.

Consequence Assessment

The consequence category was determined in accordance with ANCOLD *Guidelines on the Consequence Categories for Dams* (ANCOLD, 2012). The incremental Population at Risk (PAR) and the Potential Loss of Life (PLL) values for both PMF and sunny day scenarios are show below:

- PMF
  - PAR – 161
  - PLL – 4
- Sunny Day
  - PAR – 208
  - PLL – 2

Note: The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

The two dam break scenarios correspond to a severity of damage and loss using the ANCOLD *Guidelines On The Consequence Categories For Dams* (ANCOLD, 2012) of **Medium** and in conjunction with the PAR and PLL results the Consequence Category for both flood and sunny day scenarios for LEWSD is **High C**.

For both the PMF flood failure and the Sunny Day Failure the consequence category is dependant of the amount of property development downstream. If there is a large amount of downstream property development it is likely that the consequence category could change from the current High C to a High A.

The ANCOLD Guidelines on Selection of Acceptable Flood Capacity for Dams (2000) recommend a Fallback Flood Capacity (FFC) of between a 1 in 10,000 AEP event and a 1 in 100,000 AEP event for a High C consequence category dam. The current flood capacity of the dam meets the ANCOLD FFC for a High C consequence category dam.

The flood inundation maps were updated as part of assessing the consequence assessment. All the inundation maps can be found in Appendix G

Piping Assessment

Three key failure modes were identified as part of the piping assessment. The probability of these failure modes were assessed using the procedures recommended in the ‘Piping Toolbox’. The key failure modes, their estimated annual probabilities of failure and their contribution to the total annual probability of failure through piping are provided in Table 7-1.

Table E-1 Summary of Piping Related Failure Modes

Failure Mode	Annual Probability of Failure	Contribution
ETT-F1 Piping through the embankment (Normal and Earthquake)	$6.3x10^{-7}$	57%
ETT-F2 Piping along the outlet conduit (Normal)	$3.1x10^{-8}$	3%
ETT-F3 Piping through the foundation (Normal)	$4.5x10^{-7}$	40%
TOTAL	**$1.1x10^{-6}$**	**100%**

ETT-F1 Piping through the embankment and ETT-F3 are assessed to present the greatest contribution to the annual probability of failure through piping, with a combined 97%. ETT-F2 is estimated to have a probability of failure of greater than an order of magnitude below ETT-F1 and ETT-F3.

The probability of piping failure is low which is consistent with the embankment design, construction and materials.

Introduction

1.1 Background

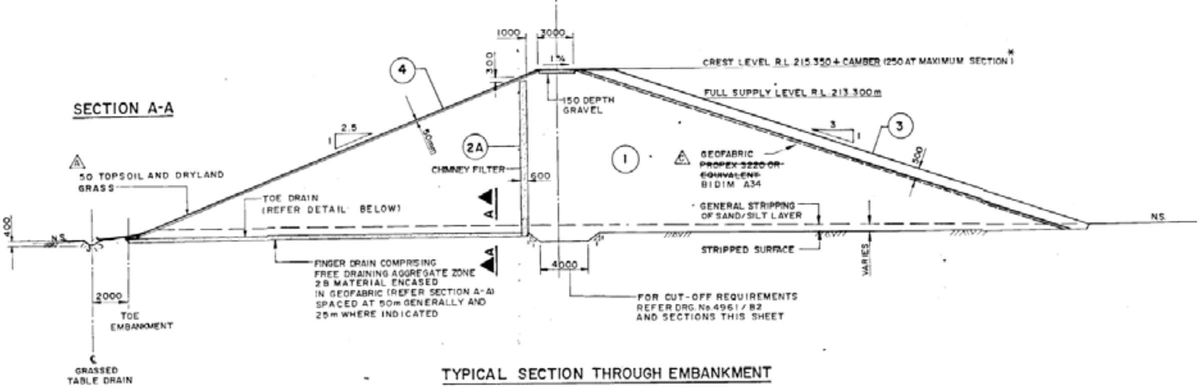
Lake Ettamogah Winter Storage Dam (LEWSD), which was earlier known as Maryvale Winter Storage Dam, is located approximately 12.5km North-East of the Albury Town Centre and 1.5km West of Somerset Road, Table Top NSW. The dam was constructed in 1994 and its function is to provide storage capacity for excess process water from the Norske Skog paper mill during the winter months. During the summer months this water is used to irrigate the company's pine plantation located in the nearby surroundings. An aerial image of the reservoir is provided in Plate 1-1 below.

Plate 1-1 Lake Ettamogah Winter Storage Dam Aerial Image

LEWSD is a 13m high earthfill embankment dam with a vertical chimney filter and blanket filter. The embankment is 1,030m long and has a 3H:1V upstream slope and a 2.5H:1V (H:V) downstream slope. The crest width is 3m and is capped with road base. The material for the embankment was sourced from site, refer to Figure 1-1 for LEWSD general arrangement section.

1 Introduction

Figure 1-1 LEWSD - General Arrangement Section



The outlet works for the dam comprise a concrete encased HDPE lined conduit with an upstream control valve operated from a tower in the reservoir. The outlet works also have downstream control. The dam has a 300m long, 90 m wide concrete crested spillway with a grass lined chute. The dam has limited contributing catchment with a catch drain around the perimeter of the reservoir to divert catchment flows up to the 1 in 100 AEP flow.

Based on the 2009 Surveillance report it is understood that:

- The dam has previously been assessed as a Significant Hazard Category dam for both flood and sunny day;
- The dam has been assessed to have a flood capacity equivalent to the Probable Maximum Flood (PMF) based on the original hydrology completed as part of the dam design in 1993-1994;
- There are seven piezometers located along the downstream face of the embankment and 11 crest settlement points along the centreline of the dam crest; and
- The dam is in good condition and there is no evidence of unusual performance.

  Note: The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

The catchment area of the dam is approximately 9.8 km^2 of rural farmland.

Table 1-1 contains summary characteristics of the winter storage dam.

Table 1-1 Lake Ettamogah Winter Storage Dam Characteristics

Embankment Crest Elevation	215.35 m AHD
Length of Embankment	1,000 m
Capacity at Embankment Crest Level	3,867,000 m^3
Spillway Crest Elevation (FSL)	213.3 m AHD
Length of Spillway	300 m
Width of Spillway	90 m
Capacity at Spillway Crest Level	2,100,000 m^3
Embankment Height	13 m
Catchment Area	9.8 km^2

1.2 Objectives of Project

The key objectives of the project were to:

- Perform static stability analysis of the embankment;
- Carry out a piping assessment for the dam;
- Develop Elevation-Discharge and Elevation-Storage relationships for the storage;
- Undertake dam break modelling to determine the extent of downstream flood inundation;
- Develop inundation maps and undertake consequence assessments; and
- Determine the ANCOLD consequence category.

During the course of this investigation the Dam Safety Committee advised that a stability analysis was not required (refer to Appendix A) as one was conducted during the design of the dam. The existing stability analysis is provided as Appendix B.

1.3 Report Layout

Section 2 - Piping Assessment

- Assessment of potential for piping failure of the dam.

Section 3 - Storage Characteristics

- Development of stage-storage and stage-discharge relationships and estimation of the PMF reservoir level.

Section 4 - Existing Flood Study

- Explanation of flood study previously completed of the Eight Mile Creek catchment (URS, 2012).

Section 5 - Dam Break Modelling

- Methodology, parameters and results of the HEC-HMS, MIKE 11 and TUFLOW dam break and no fail modelling scenarios.

Section 6 - Consequence Category Classification

- Method and tables used in the ANCOLD *Guidelines on the Consequence Categories for Dams* (ANCOLD, 2012) and the determined consequence category.

Section 7 - Conclusions and Recommendations

- Conclusions from this investigation and any actions to be taken by Norske Skog Paper Mill relating to the LEWSD.

Piping Assessment

2.1 Data Review

2.1.1 Geological Setting

2.1.1.1 Regional Geology

The 1:50,000 geological map of Hume (provided as Figure 2-1) indicates that LEWSD was constructed on surficial colluvial deposits from the Quartenary period over Silurian to Ordovician aged Bethanga Gneiss. The colluvial deposits are described as hillwash and scree deposits, red to yellow silt, sand and poorly graded gravel. The Bethanga Gneiss is described as medium to coarse grained, biotite-rich and strongly contorted with sedimentary xenoliths.

The 1:250,000 geological map of Tallangatta (Adamson, Browne et al 1966 and Adamson & Loudon, 1966) categorise the area as Rhyolites, Tuff, Quartzite and Slate of Devonian to Silurian age.

The regional geology described in the Design and Construction Manual (Willing and Partners, 1993) is broadly consistent with the categorisation based on the geological maps. Willing and Partners (1993) describe the regional geology as Paleozoic bedrock forming the highlands, with colluvium and alluvium deposits along the major drainage systems. Willing and Partners (1993) describe the bedrock as fine-grained biotite granite East of the site and quartz feldspar porphyry to the West.

2.1.1.2 Dam Geology

Prior to construction of the dam the subsurface conditions adjacent to the streamline and maximum section were assessed to be sandy, silty topsoil overlying depositional sandy, gravelly clays, overlying highly weathered quartz feldspar porphyry. The highly weathered quartz feldspar porphyry was encountered at depths ranging between 1.9 m towards the left abutment and 8.0 m towards the right abutment. Adjacent to the creek, the weathered rock was found to be approximately 3.0 m deep.

Surface mapping undertaken prior to construction of the dam indicated the presence of alluvial layering with some units of high plasticity clay and others of clayey gravel, fine to medium grained and rounded.

Based on the information provided by Willing and Partners (1993), Table 2-1 describes the understanding of the typical subsurface profile of the dam adjacent to the creek-line.

Coffey Partners International drilled 18 boreholes to shallow depth (2 m) between 30/6/93 and 5/7/93 along the dam centreline and within the proposed borrow. Beneath surficial silty, sandy, clay layers, the material was typically found to be high plasticity sandy clay. Coffey also excavated 24 test pits, in unknown locations at the site between November 1993 and January 1994. Test pits were excavated to a depth of 3-4 m. The material encountered was typically logged as a layer of silty, sandy clay overlying layers of sandy clay and clay of medium to high plasticity. These descriptions are in general agreement with that described by Willing and Partners (1993).

2 Piping Assessment

Table 2-1 Subsurface profile of dam adjacent to creek-line

Depth	Unit	Description
0 – 0.35 m	Topsoil	Roots and rich organics in the upper 150 mm over sandy silt, light grey soft and wet.
0.35 – 0.7 m	Slopewash	Sandy clay, medium plasticity, mottled yellow, brown and light grey, stiff and friable.
0.7 – 2.0 m	Alluvium	Clayey sands and clayey gravel or clean gravels, medium sized pea gravel, rounded quartz, manganese, lithic, red brown.
2.0 – 3.0 m	Colluvium	Sandy clay, medium plasticity, grey brown, some gravel, hard.
3.0 m +	Highly weathered rock	Clayey sand or sandy clay of low to medium plasticity.

2.1.1.3 Foundation Preparation

Design drawing 4961/B3, Rev C, marked as ‘Work as Executed’ and the technical specification described the foundation treatment as such:

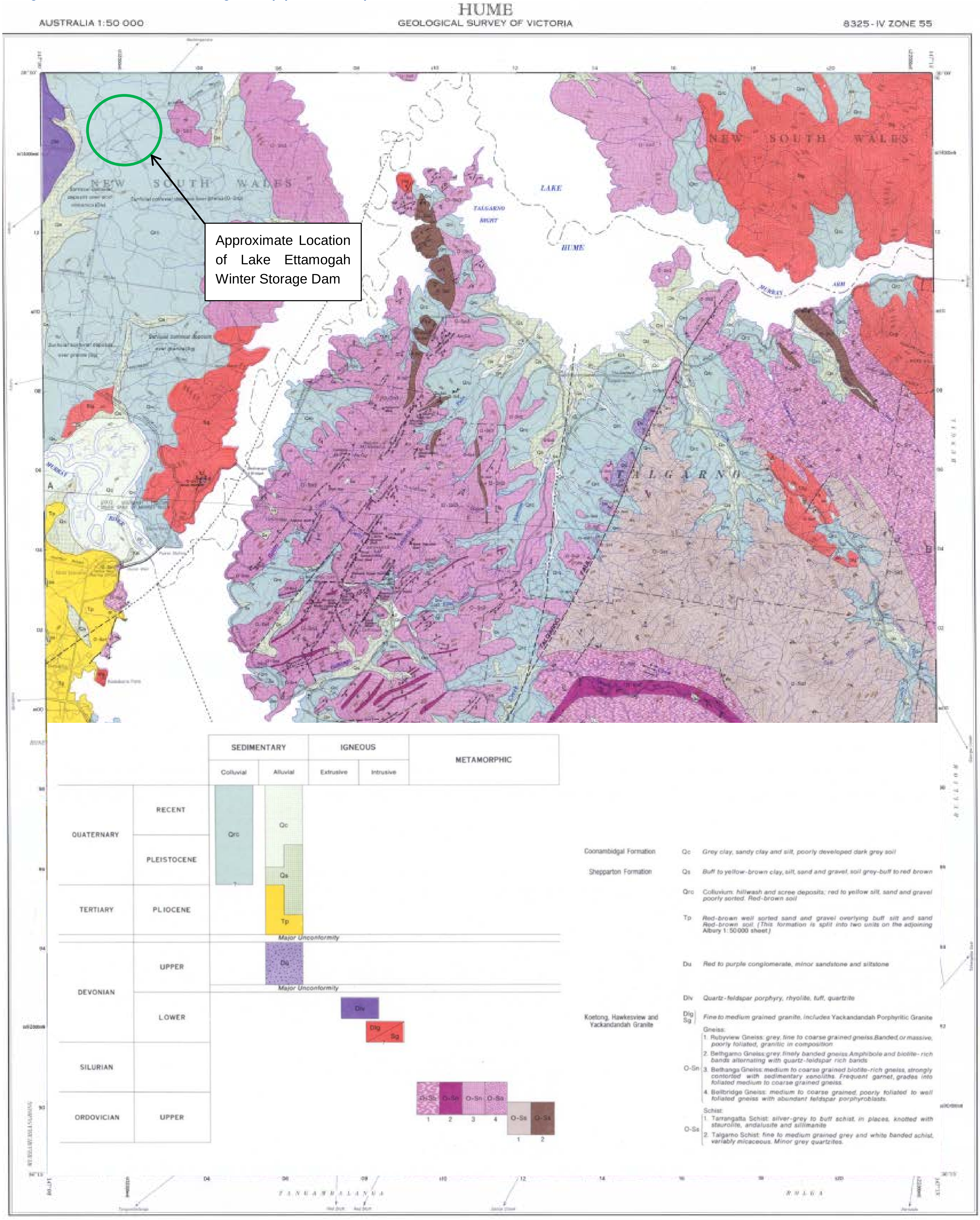
General stripping of the sandy and silty surficial layer over the entire foundation area to a minimum of 1.0 m. The sandy and silty layer is inferred to mean the layer of topsoil described in Table 2-1.

- The cut-off was shown to be 0.3 m below the base of the Alluvium (and presumably into the Colluvium) near the maximum section. The Superintendent was to approve the final depth of the cut-off over the 150 m length of where Alluvium was expected to be encountered.
- For the remainder of the embankment, the cut-off was designed to be 1.0 m below the stripped surface and 0.2 m into the Colluvium layer.
- The foundation was to be compacted with a tamping foot roller of 6 tonnes mass. Areas of soft material were to be removed and replaced with compacted clay.

In discussions with Mr. Richard Rodd (Consultant during construction), it is understood that the cut-off was excavated into residual clay below all lenses of the prior stream bed. This is inferred to be the material classified as ‘highly weathered’ rock, however, described as clayey sand or sandy clay of low to medium plasticity in the Design and Construction Manual (Willing and Partners, 1993). Mr. Rodd also indicated that sandy lenses were encountered adjacent to the natural streamline, where the cut-off was taken to a depth of approximately 3 m below the stripped surface.

No foundation grouting was undertaken.

Figure 2-1 Hume 1:50,000 Geological Map (O'Shea, 1979)



2.1.2 Embankment Materials

2.1.2.1 Embankment Arrangement

Lake Ettamogah Winter Storage Dam (LEWSD) is a 13m high homogeneous earthfill (Zone 1) embankment with a Zone 2A vertical chimney filter. The embankment is approximately 1,030m long and has a 3H:1V upstream slope and a 2.5H:1V downstream slope. The crest width is 3m and is capped with road base. A cut-off was constructed and is understood to have been excavated to residual soil, as described in Section 2.1.1.3. Design drawings (stamped as Works as Executed) show the chimney filter extending to the base of the cut-off. The top of the chimney filter is shown to terminate 0.7 m below Dam Crest Level at RL 214.65 m.

Finger drains were provided at approximately 50 m centres to allow drainage of the chimney filter and were constructed from Zone 2B material, wrapped in geotextile (Bidim A34). A toe drain was also constructed to direct seepage flow to the outlets. The toe drain was comprised of 2A material towards the abutments and 2B near the maximum section.

Slope protection was provided on the upstream face with Zone 3 Rip Rap.

2.1.2.2 Zone 1

Zone 1 material was sourced from the spillway excavation and the borrow within the reservoir (Coffey, 1993b) outside of a 50 m buffer zone from the upstream embankment toe (Willing and Partners, 1993). The Zone 1 material was specified to have a minimum liquid limit of 30% with the grading provided in Table 2-2. Ten particle size distributions for material sampled from the borrow were undertaken by GHD in February 1993. As shown in Figure 2-3 the sampled material met the gradation specification for Zone 1 material. Furthermore, the liquid limit of the material sampled from the borrow ranged between 44 to 66%, in excess of the specified minimum of 30%.

Table 2-2 Zone 1 Grading Specification

Sieve Size (mm)	% Passing
9.5	≥95
2.36	≥90
0.6	≥80
0.075	≥65

The specification called for the Zone 1 material to be compacted in 200 mm horizontal layers to a dry density of 98% standard compaction and with a moisture content of between 1% dry and 2% wet of optimum.

Potential issues relating to the dispersivity of the earthfill were raised during design. Dispersion testing conducted in the proposed borrow locations indicated that the more dispersive soils were dominant below 1.5 m depth and typically produced Emerson dispersion test results of between Class 2-3 in distilled water (Coffey, 1993b). Of note is that when the marginally dispersive soils were tested with wastewater (presumably of similar chemical properties to the effluent from the Norske Skog Paper Mill) the earthfill was found to be less dispersive with Emerson values in the range of 5-6 (Coffey, 1993b).

2 Piping Assessment

Due to the presence of potentially dispersive materials within the borrow, a regime of dispersion testing was suggested every 200 m^3 (Coffey, 1993b). The specification required that material placed upstream of the chimney filter or within 2 m of the downstream face have a minimum Emerson Class of 3. Dispersion testing was undertaken by Coffey during construction of the dam between 29/11/93 and 23/3/94 in-line with AS12893.8.1. Samples were taken from a number of sources including (but not limited to):

- Upstream fill;
- Downstream fill;
- Core trench;
- Borrow area; and
- Dam foundation.

Of the 594 test results available, 562 (approximately 95%) were classified as Emerson Class 3. The remaining 32 results were classified as Class 2 (Coffey, 1994). The majority of Emerson Class 2 results were sampled from the downstream shoulder fill and were considered to meet the specification. For Emerson Class 2 earthfill sampled within the core trench, borrow or upstream fill, the material was removed (if already placed on the bank) and stockpiled for future use in the downstream shoulder.

2.1.2.3 Zone 2A

The Zone 2A fine filter was specified to comprise a non-plastic mixture of angular gravelly sand, processed from slightly weathered to fresh rock. Compaction was to be undertaken in 300 mm lifts with a vibrating plate or whacker plate to achieve a nominal density index of 65%.

The specified grading envelop of the Zone 2A is provided in Table 2-3.

Table 2-3 Zone 2A Grading Specification

Sieve Size (mm)	% Passing
9.5	100
4.75	80-100
2.36	60-100
1.18	40-80
0.6	20-58
0.425	12-47
0.15	0-22
0.075	0-5

The Design and Construction Manual (Willing and Partners, 1993) reported that the filter material was imported to site and met the requirements of the specification.

Particle size distributions conducted on Zone 2A material sampled from the stockpile on site, the toe drain and the chimney filter by Coffey in December 1993 and January 1994, indicate that the contractor had difficulty meeting the grading specification. Of the 10 test results available, 7 contained excessive fines (<0.075mm) of between 6-7 %. Of those that did meet the specification, 2 were reported as 5% fines.

2 Piping Assessment

2.1.2.4 Zone 2B

The Zone 2B fine filter was specified to comprise free-draining gravel consisting of hard, durable rock particles. Like the 2A material, the 2B was to be compacted in 300 mm lifts to achieve a nominal density index of 65%.

The specified grading envelop of the Zone 2B is provided in Table 2-4.

Table 2-4 Zone 2B Grading Specification

Sieve Size (mm)	% Passing
26.5	100
19.0	40-100
13.2	0-90
9.5	0-30
6.7	<2

The Design and Construction Manual (Willing and Partners, 1993) reported that the filter material was imported to site and met the requirements of the specification.

One particle size distribution was available from construction for the Zone 2B. The material was sampled from the Readymix stockpile on site on the 9/12/93 and tested within the specified grading envelop.

2.1.3 Review of Filter Grading Curves

In order to understand the effectiveness of the filter material in LEWSD, to inform the piping assessment process, an analysis of the existing filter material was undertaken. The main purpose of this assessment was to consider the effectiveness of the existing Zone 2A chimney filter to prevent migration of the Zone 1 earthfill.

2.1.3.1 Internal Stability Assessment

Internal instability (or suffusion) is the process whereby finer particles of a soil are washed out with the coarser fraction of the soil remaining. This is of concern in the context of filters as an internally unstable filter, which may otherwise have a particle size distribution expected to stop erosion of the base soil, could be rendered ineffective after the loss of the finer fraction. Internal instability can also lead to clogging of the filter and hence reduced permeability.

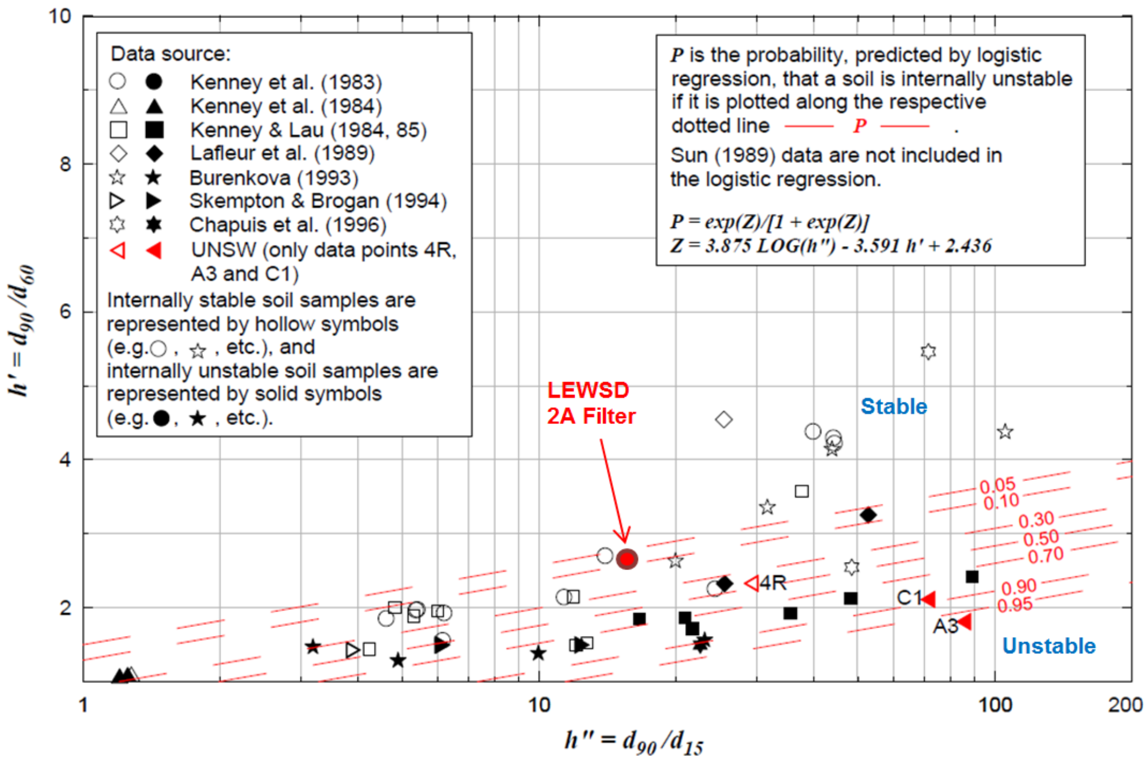
If the finer fraction (point of inflection of a grading curve) of the soil constitutes greater than 40% of the total soil mass, internal instability is not considered possible as the coarse particles will ‘float’ in the finer particles (‘Piping Toolbox’; Reclamation, USACE, URS and UNSW, 2008). Typically the fine fraction of the filter material for sampled Zone 2A material from LEWSD is in the range of 40-50%, as shown in Figure 2-3, indicating that internal instability of the Zone 2A material is unlikely.

The LEWSD Zone 2A filter material was also assessed for internal stability using the method described in Wan and Fell (2004) as presented in Figure 2-2. The results indicate that the Zone 2A filter material has a probability of being internally unstable ranging between 0.05 to 0.1. This is indicative of having a low potential for internal instability.

2 Piping Assessment

Based on this assessment, it is concluded that the Zone 2A filter material is unlikely to experience internal instability/ suffusion and therefore it is acceptable to assess the filter compatibility of the materials without having to adjust for the possible loss of fines associated with suffusion.

Figure 2-2 Internal Stability Check (<10% non-plastic fine) - Wan and Fell (2004)

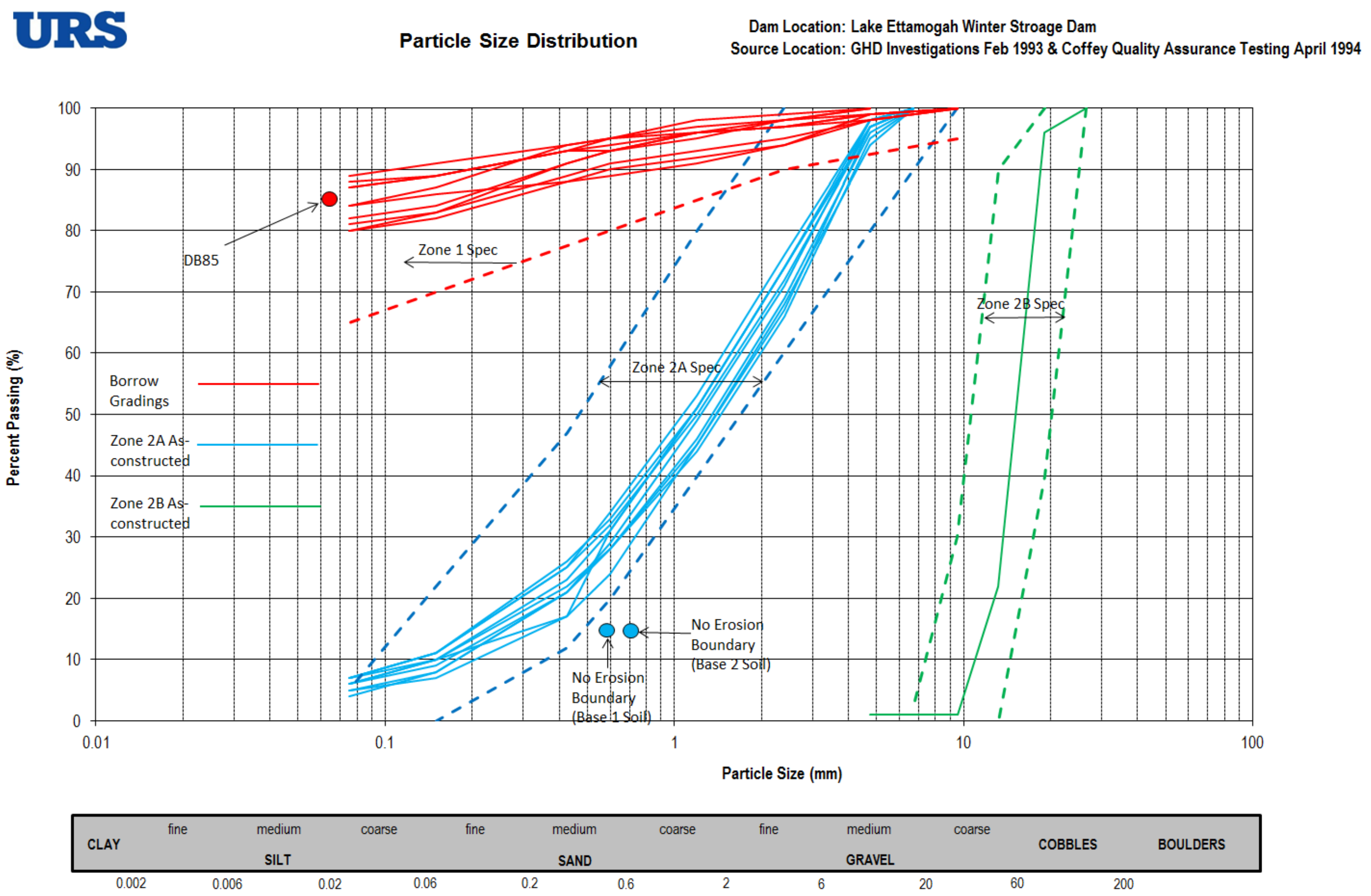


2.1.3.2 *Erosion Boundaries*

Using the method outlined in the ‘Piping Toolbox’, the continuing erosion boundary for the Zone 1 earthfill was assessed. Although no as-constructed grading curves were available for the Zone 1 material, particle size distributions were available for material sampled from the earthfill borrow prior to construction. The grading of these borrow samples indicates that the Zone 1 material is likely to classify as borderline Category 1 and 2 base soil, according to Foster and Fell (2001) i.e. some gradings have greater than 85% fines (<75 µm). The no erosion criteria for base 2 soils is DF_{15} of ≤ 0.7 mm and the no erosion boundary for base 1 soils is $9xDB_{85}$ (approximately 0.6 mm in this case). Both these boundaries are shown in Figure 2-3, along with available as-constructed grading for the Zone 2A filter. The DF_{15} of the Zone 2A material ranges from approximately 0.2 to 0.4 mm and hence falls within the ‘no erosion’ boundaries for a base 1 and 2 soil. On this basis, the likelihood of erosion occurring due to the filter material not being compatible with the Zone 1 is assessed to be highly unlikely. Accordingly, the likelihood of continuing erosion occurring is governed by the probability of the filter material holding a crack. This is discussed further in 2.4.1.3.

2 Piping Assessment

Figure 2-3 No Erosion Boundary Assessment



2.2 Identification of Failure Modes

To assess the susceptibility of LEWSD to piping, the 'Piping Toolbox' (Reclamation et al, 2008) was employed to estimate the probability of dam failure associated with piping failure modes.

All conceivable piping related failure modes were initially considered and short-listed for the event tree analysis. Failure modes that were considered to have negligible contribution to the probability of failure of the dam were excluded from further analysis.

The criteria by which failure modes were excluded were:

- Where engineering analyses showed the dam component satisfies normally accepted design criteria, (e.g. adequate factors of safety for slope stability);
- Where the likelihood of the initiating event was considered to be very low or inconceivable; and

Appendix C summarises the failure modes that were identified and also records the reasons for their inclusion or exclusion from the event tree analysis.

The failure modes screening identified three key potential failure mechanisms for LEWSD as described below in Table 2-5.

Table 2-5 Key failure modes analysed

Potential Failure Modes Talbingo Dam		Initiating Event
ETT-F1	Piping through embankment	Normal and Earthquake
ETT-F2	Piping along outlet conduit	Normal
ETT-F3	Piping through foundation	Normal

2.3 Event Tree Analysis

Event trees were developed for each failure mode identified in Table 2-5 and are presented as Appendix D. The event trees decompose the failure path of the dam into a series of steps that were then assigned probabilities based on the methodology described in the 'Piping Toolbox' (Reclamation et al, 2008).

2.3.1 Loading Partitions

2.3.1.1 Reservoir Loading Partitions

As the PMF reservoir level is estimated to reach RL 214.30 m (refer to Section 4), approximately 0.35 m below the top of the chimney filter and 1.05 m below dam crest level, reservoir levels were not partitioned for this analysis. In the event that reservoir levels were partitioned, the difference in calculated hydraulic gradients and postulated defect dimensions would be negligible. Furthermore, as there are no physical differences in the embankment characteristics over the range of possible reservoir levels (i.e. between FSL at RL 213.30 m and the PMF reservoir level of RL 214.30 m), the conditional probabilities for each potential reservoir level partition would be equivalent. The total probability of failure for flood induced piping would be the conditional probability of failure multiplied by the probability of the flood partition. The probability of failure due to piping under normal operating conditions (i.e. reservoir at FSL) would therefore be the dominant piping mechanism. Based on this, the probability of failure due to flood induced piping has not been assessed as part of this study.

2.3.1.2 Earthquake Loading Partitions

Earthquake loading events were partitioned into three peak ground acceleration (PGA) ranges. Details of the peak ground acceleration levels and their associated Annual Exceedance Probabilities (AEP) within the particular partition range are provided in Appendix D. The AEP of the earthquake loading partitions were obtained from the most recent seismology study for Hume Dam which was carried out by Seismology Research Centre (SRC) in 2010. Earthquake events with a PGA less than 0.045g were assumed to have negligible effect on the performance of the dam and hence were excluded from the analysis. The reservoir was assumed to be at FSL at the time of the earthquake event.

2.3.2 Annual Probability of Failure

The annual probabilities of failure were calculated using the annual probabilities of the loading partition and the conditional probability of failure for each loading partition for each of the failure modes. Table 2-6 below summarises the resulting annual failure probabilities for the best estimate scenario.

Table 2-6 Annual probability of failure

Potential Failure Modes LEWSD		Initiating Event	Annual Failure Probability
ETT-F1	Piping through embankment	Normal and Earthquake	6.3×10^{-7}
ETT-F2	Piping along outlet conduit	Normal	3.1×10^{-8}
ETT-F3	Piping through foundation	Normal	4.5×10^{-7}

The estimated annual probability of failure of LEWSD for all piping related failure modes is 1.1×10^{-6} per annum. The greatest contributors to this probability are piping through the embankment under normal and earthquake loading (ETT-F1) and piping through the embankment under normal conditions (ETT-F3). The annual probabilities of failure for these mechanisms are estimated to be 6.3×10^{-7} and $4.5 \times 10^{-}$, respectively. Piping along the outlet conduit (ETT-F2) is estimated to be greater than an order of magnitude lower than the ETT-F1 and ETT-F3.

2.4 Overview of Failure Modes

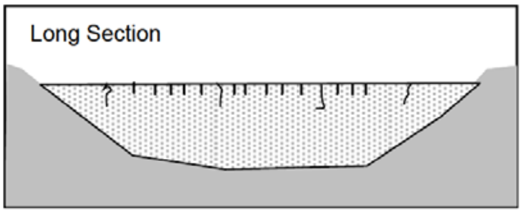
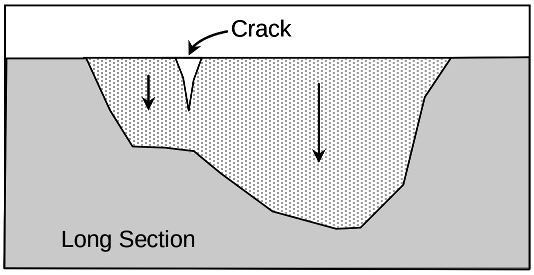
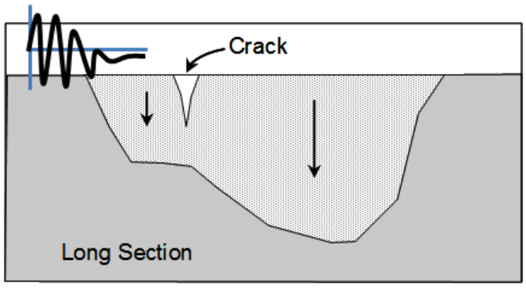
2.4.1 Piping through Embankment

2.4.1.1 Initiating mechanisms

Table 2-7 lists the analysed initiating mechanisms associated with the embankment piping failure modes. These mechanisms are discussed in the sections below. The initiating mechanism that was considered to be the most critical for LEWSD was piping through desiccation cracks in the upper part of the embankment.

2 Piping Assessment

Table 2-7 Initiating mechanism for piping

Initiating Mechanism	Sketch
Desiccation cracking	
Transverse cracking due to cross valley differential settlement	
Transverse cracks caused by earthquake	

Desiccation Cracking

This initiating mechanism involves drying and shrinking of the earthfill resulting in the development of cracks near the embankment crest. The maximum likely crack depth was assessed to be 3 m below the dam crest and approximately 1 m below FSL. This assessment takes into consideration the following factors in determining the likelihood of crack development:

- Pavement details of the crest: embankments with thick road pavement details are less likely to develop surface cracks compared to exposed earthfill. In the case of LEWSD, the gravel capping is expected to provide some protection against crack development.
- Climatic conditions: consistent periods of rainfall are less likely to allow shrinkage cracks to develop, when compared to arid climates. LEWSD was classified as experiencing a seasonal climate.
- Plasticity of the core: higher plasticity soil has the ability to absorb more water and hence is more prone to shrink/ swell behaviour. The earthfill material at LEWSD classifies as medium – high plasticity and hence is susceptible to crack development.

2 Piping Assessment

Cross Valley Differential Settlement

This initiating mechanism involves differential settlement of the earthfill due to large scale changes in the profile of the foundation, such as those associated with steep abutments or benches in the abutment, which could give rise to transverse cracking of the earthfill. The abutment profile at LEWSD is gentle and uniform and so a low probability of a continuous defect was adopted (1×10^{-4}).

Transverse Cracking Caused by Earthquake

The potential for earthquake induced cracking was assessed using the Pells and Fell (2002) empirical method. Pells and Fell (2002) developed a system of damage class categories based on a database of dams exposed to seismic activity. The result of their work is a system to predict crest settlement and longitudinal and transverse cracking (to a lesser extent) for dams under seismic loading.

Figure 2-4 shows the position of LEWSD on the damage class contours, and Figure 2-5 shows the maximum settlement and longitudinal crack widths according to Pells & Fell (2002).

Pells and Fell recommend crude relationships developed by *Fong & Bennett (1995)* to estimate transverse crack widths and depths. Fong & Bennett found that transverse crack depth was about 5 to 6 times the settlement depth, and that the ratio of transverse crack width to transverse crack depth was from 15 to 100, with an average of 40.

Estimated deformations as a result of earthquake have been estimated based on a design earthquake magnitude (M_w) of 7.5. Two Peak Ground Accelerations (PGA) were considered based on the seismic hazard assessment conducted for Hume Dam (Seismology Research Centre, 2010).

- Operating Basis Earthquake (OBE) with a PGA of 0.045 g and an AEP of 1 in 500
- Maximum Design Earthquake (MDE) with a PGA of 0.22 g and an AEP of 1 in 10,000

Based on an OBE:

- The results indicate likely crest settlements of 4 to 26 mm at the maximum embankment section.
- The likely maximum longitudinal crack width is estimated to be 30 mm.
- Transverse cracking is estimated to reach depths of 130 to 150 mm below the dam crest, with crack widths of around 3 to 4 mm.

Based on an MDE:

- The results indicate likely crest settlements of 26 to 65 mm at the maximum embankment section.
- The likely maximum longitudinal crack width is estimated to be 80 mm.
- Transverse cracking is estimated to reach depths of 320 to 390 mm below the dam crest, with crack widths of around 8 to 10 mm.

The results of this analysis are summarised in Table 2-8.

2 Piping Assessment

Figure 2-4 LEWSD plotted on Pells & Fell (2002) damage class contours

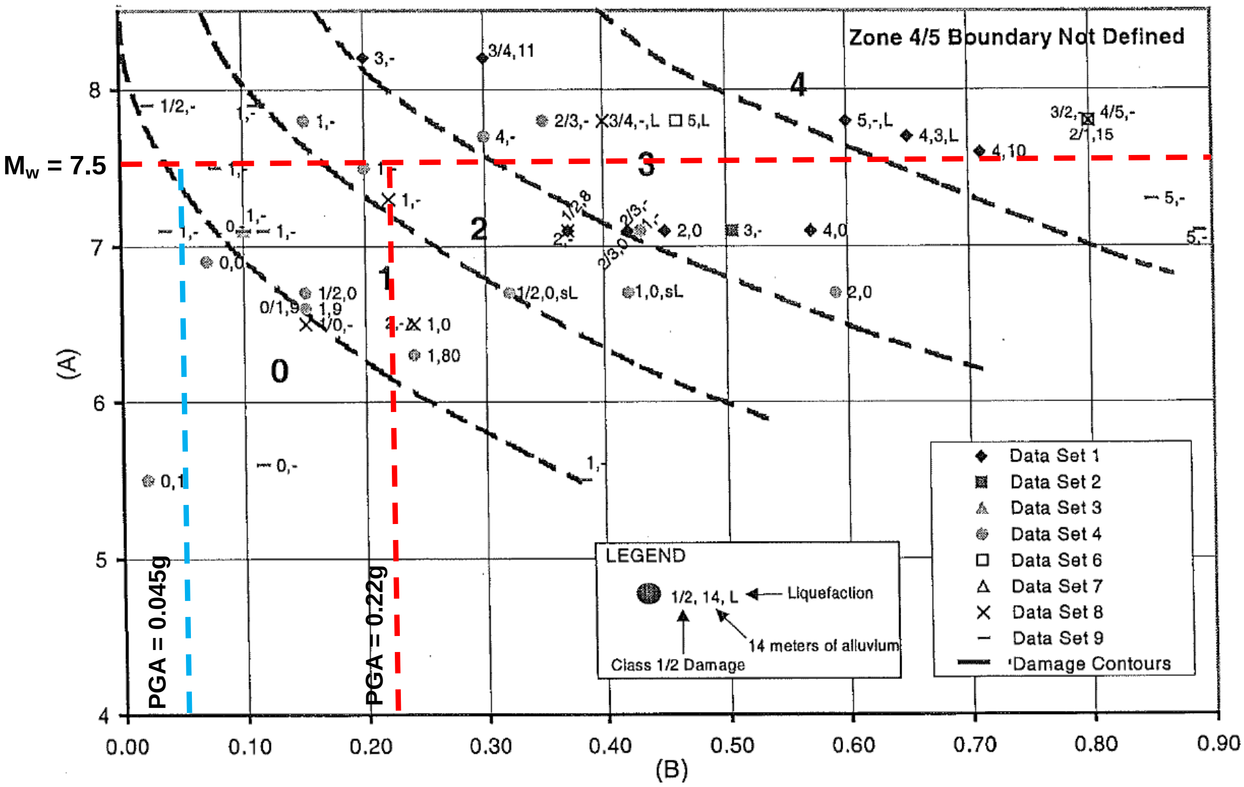


Figure 1 *Earthfill Dams – Contours of damage class versus earthquake magnitude and peak ground acceleration.*
(A) Earthquake Magnitude.
(B) Foundation Peak Ground Acceleration (as a fraction of acceleration due to gravity).
Note: Contours Drawn without consideration for cases that had liquefaction

Figure 2-5 LEWSD plotted on Pells & Fell (2002) Damage Classification System

Damage Class		Maximum Longitudinal Crack Width (1) mm	Maximum Relative Crest Settlement (2) %
Number	Description		
0	No or Slight	< 10 mm	<0.03
1	Minor	10 - 30	0.03 - 0.2
2	Moderate	30 - 80	0.2 - 0.5
3	Major	80 - 150	0.5 - 1.5
4	Severe	150 - 500	1.5 - 5
5	Collapse	> 500	> 5

(1) Maximum crack width is taken as the maximum width, in millimetres, of any longitudinal cracking that occurs.

(2) Maximum relative crest settlement is expressed as a percentage of the structural dam height.

Table 2-8 Pells and Fell (2002) Deformation Predictions

	Mw = 7.5	
	a_{max} = 0.045g	**a_{max} = 0.22g**
Damage Class	1 – Minor	2 – Moderate
Estimated Maximum Crest Settlement	4 to 26 mm	26 to 65 mm
Estimated Maximum Longitudinal crack width	30 mm	80 mm
Estimated Transverse crack depth	130 to 150 mm	320 to 390 mm
Estimated Transverse crack width	3 to 4 mm (range from 1 to 10mm)	8 to 10 mm (range 3 to 26mm)

This analysis indicates that piping failure of the embankment, initiated through a crack developed during an earthquake, is unlikely. This is because transverse cracks are not expected to extend below FSL, which is approximately 2.05 m below dam crest level (overtopping failure during an earthquake is also unlikely based on the estimated settlement). Nevertheless, the approach recommended in the 'Piping Toolbox' was adopted to quantify the probability of piping through the embankment under earthquake loading. As expected, the results of this assessment indicate a negligible likelihood of dam failure from this initiating mechanism, with an estimated annual probability of failure of 1.3×10^{-9}.

2.4.1.2 Erosion Initiates

As cross valley differential settlement and earthquake induced transverse cracking are not substantial contributors to the annual probability of failure through piping, only the probability of piping through desiccation cracks is discussed hereon.

Based on the factors discussed in Section 2.4.1.1, the maximum crack width at the embankment crest was estimated to be 20 mm. Although no cracks were observed on the crest of the dam, it is possible that cracks were obscured by the gravel capping layer. The estimated 20mm crack width at the top of the embankment was adopted to assess the probability of erosion initiating.

The earthfill material was assumed to be non-dispersive as considerable effort was taken to test borrow material for dispersivity (as discussed in Section 2.1.2.2) and place material of Emerson Class 2 downstream of the chimney filter. Zone 1 material was considered to have an average liquid limit of less than 65%, based on the testing conducting on the borrow material (GHD 2013) with Liquid Limits found to range between 44-66% (based on 10 results). The hydraulic gradient was estimated to be less than 0.1, with the reservoir level at FSL, resulting in a probability of erosion initiating of 0.005.

2.4.1.3 Erosion Continues

Based on the review of the Zone 2A filter gradings discussed in Section 2.1.3 the high fines content of the filter material discussed in Section 2.1.2.3, the conditional probabilities of erosion continuing were assessed to be dependent on the likelihood of the material holding a crack. In the event that a continuous transverse crack develops through the embankment, filters are designed to collapse into the void to limit the potential for erosion. Where filters have excessive fines (particularly plastic fines), are highly compacted and angular, they are less likely to collapse. For this reason, and based on the guidance provided in the 'Piping Toolbox', the probability of erosion continuing was adopted to be 0.005.

2.4.1.4 Erosion Progresses

The questions considered to assess the probability of erosion progressing were:

- Will the earthfill support a roof – a probability of 1.0 was adopted as the Zone 1 is Clay (CL-CH).
- Will crack filling action not stop progression – a probability of 1.0 was adopted as there are no upstream filter zones to fill cracks.
- Will flow not be restricted – this was assessed to be 1.0 as there are no upstream flow limiters.

2.4.1.5 Not Detect and Intervene

The time for a breach to develop was considered in assessing the likelihood of detecting an active piping incident. Based on the low gradient and characteristics of the earthfill, the breach development time was estimated to range between 12-24 hours. Given that the dam is only inspected twice weekly, it was considered likely that the piping incident would not be identified. Furthermore, there is limited opportunity to undertake mitigating measures, in the event that an incident is identified, due to the small drawdown capacity of the outlet works (750 mm diameter pipe). For these reasons, the probability of an active piping incident not being detected and intervention measures being ineffective was assessed to be 1.0.

2.4.1.6 Breach

The probability of the reservoir breaching was assessed to be 0.5 to account for the small storage and that the defect is only present in the upper part of the embankment. It is possible that the reservoir will fall below the invert of the defect, prior to full development of the breach.

2.4.2 Piping Along the Outlet Conduit

Conduits inherently have a tendency for contributing to a piping failure of an embankment. Approximately half of recorded piping failures can be attributed to the presence of conduits (Foster, Fell and Spannagle 2000). Characteristics of conduits that contribute to the initiation and progression of a piping failure include:

- Inability to adequately compact around the haunching of the conduit and difficulty in compacting within a trench;
- Possibility of seepage to infiltrate the conduit through a crack or joint, allowing the conduit to transport sediment;

2 Piping Assessment

- Possibility of cracks developing along the sides of an excavated trench due to differential settlement and shrinkage cracks and creating a pathway for concentrated seepage and piping to initiate;
- Flow out of a conduit through a defect or joint, leading to erosion along the conduit; and
- Ability of the conduit to support a 'roof' to enable continuing development of the piping mechanism rather than collapsing soil plugging the pipe.

Cut-off collars have traditionally been employed around conduits in embankments as a measure of reducing the potential for piping to initiate. The intent of the collars is to lengthen the seepage pathway, resulting in a reduction in the hydraulic gradient and hence seepage flow. In practice, issues have been encountered during construction in achieving an adequate level of compaction around the cut-off collars. This has the tendency to exacerbate a piping issue within an embankment.

The outlet conduit for LEWSD is a 750 mm diameter HDPE pipe, fully encased in reinforced concrete and with upstream valve control. The concrete encasement eliminates the difficulty in compacting around the haunches of the pipe and limits the issues associated with flow into and out of the conduit. Based on discussions with Mr. Richard Rodd (Consultant during construction), it is understood that the trench was excavated into residual clay and kept moist prior to backfilling with concrete. The result of this is a reduced chance of desiccation cracking developing in the trench.

Three concrete cut-off collars were constructed around the conduit, at 5 m spacing near the centreline of the dam. The concrete cut-off collars were excavated into the foundation and backfilled with concrete to the top of the concrete encasement, reducing the chance of a poorly compacted. The probability of a continuous defect being present, as a result of the outlet conduit, was assessed to be $7x10^{-4}$.

A filter diaphragm is also provided around the outlet conduit, downstream of the centreline of the dam. The filter diaphragm is well detailed, with an outlet provided for seepage flow and is connected to the chimney filter to provide a continuous barrier to erosion.

The assessment of the likelihood of each piping stage, leading to dam failure, is consistent with the descriptions provided in Sections 2.4.1.3 to 2.4.1.5, with the exception of the breach stage. As the mechanism is in the lower part of the embankment, it is considered unlikely that the reservoir will run out of water prior to full development of the breach. Therefore, the probability of breach was adopted to be 0.9 for piping along the outlet conduit.

2.4.3 Piping through the Foundation

As discussed in Section 2.1.1.3, it is understood that the cut-off trench was excavated to residual clay for the full length of the embankment. This implies that for piping to initiate through the foundation there needs to be:

a) Continuous defects in the shallow alluvial and colluvial deposits, day-lighting both upstream and downstream of the embankment;
b) A continuous defect in the residual soil, below the depositional material; and
c) Connectivity between all of the above defects.

The adopted probabilities for the various stages leading to dam failure via this mechanism are described below:

- The probability of a persistent, inter-connected defect being present throughout the foundation was adopted to be very low ($1x10^{-5}$) based on the conditions listed above. Given that the borrow was located within the reservoir, it is possible that excavations could have exposed the residual soil foundation, hence eliminating the requirement for inter-connecting defects between different strata for erosion to initiate. However, as a 50 m buffer from the upstream toe of the embankment was specified for the source of the borrow, the resulting hydraulic gradient and the probability of a continuous defect through the residual soil would be reduced. It is assessed that this scenario would likely produce an annual probability of failure of a similar order of magnitude to the calculated probability in Table 2-6 ;
- The likely size of the postulated defect was assumed to be 1-2 mm. With an estimated hydraulic gradient of between 0.1-0.2, the probability of erosion initiating was estimated to be 0.05 based on the guidance provided in the 'Piping Toolbox';
- As there is no foundation filter trench, the likelihood of erosion continuing was taken to be 1.0;
- Consistent with the reasoning described in Section 2.4.1.4 the probability of progression was adopted to be 1.0; and
- As described in Section 2.4.2 the probability of breach developing was estimated to be 0.9.

2.5 Assessment of Uncertainty

2.5.1 Relict Defects in the Foundation

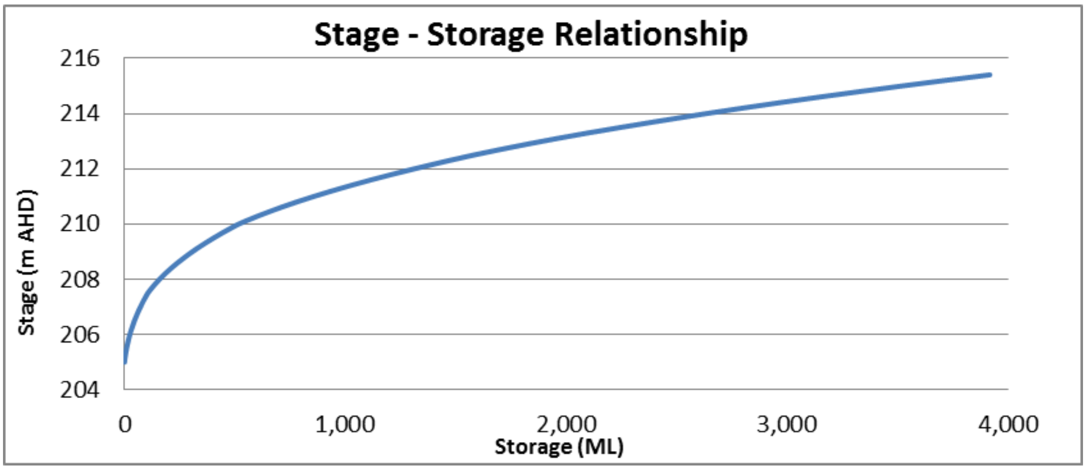
The key area of uncertainty is the assessment of the likelihood of persistent relict defects throughout the foundation. URS has not inspected any foundation conditions as part of this investigations and so have made an assessment of the likelihood of inter-connecting defects being present based on the understanding of the foundation conditions described in Section 2.1.1. The resulting failure mode, ETT-F3 piping through the foundation, is estimated to contribute approximately 40% to the annual probability of failure through piping, and so assumptions on the presence of the postulated defects are important in assessing the likelihood of dam failure. To provide a greater level of confidence in this assessment, investigation trenches could be excavated downstream of the embankment to identify presence of relict defects, however, given the assessed low likelihood of failure of the dam through piping, this is not considered warranted at this stage.

Storage Characteristics

3.1 Stage-Storage Relationship

The stage-storage relationship derived for the LEWSD was completed using the as constructed drawings and can be seen in Figure 3-1 below. The tabulated relationship can be found in Appendix E.

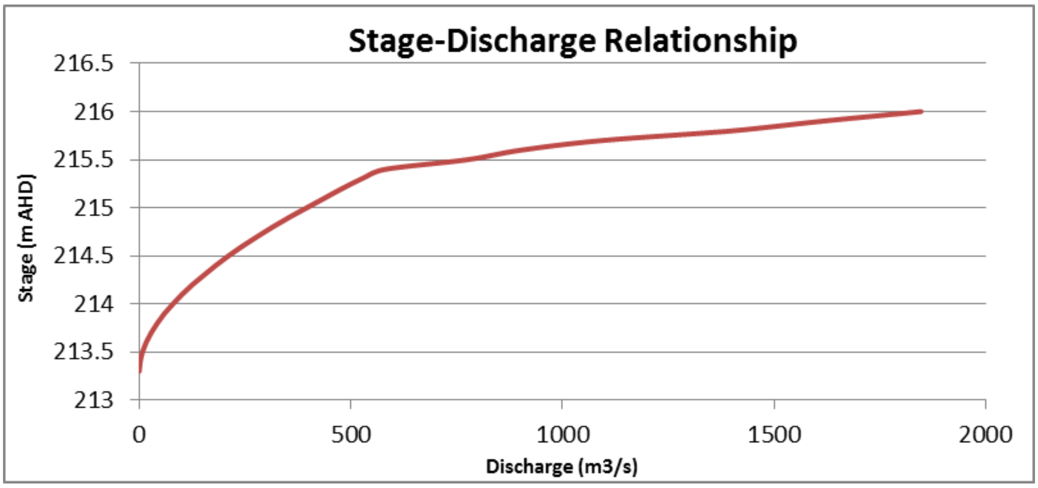
Figure 3-1 Stage-Storage Relationship



3.2 Stage-Discharge Relationship

he stage-discharge relationship in Figure 3-2 shows the discharge through the spillway and over the embankment crest. The stage-discharge curve for the spillway section was completed using LiDAR information along with HEC-RAS. The visible kink in the curve is due to the flow down the spillway overtopping the side and flowing into the eastern diversion drain, increasing the capacity of the spillway. The tabulated relationship can be found in Appendix F.

Figure 3-2 Stage-Discharge Relationship



Existing Flood Study

In March 2012, Albury City Council (ACC) commissioned URS to complete a flood study of the Eight Mile Creek catchment (URS, 2012). The study was carried out in accordance with the New South Wales (NSW) State Government Floodplain Development Manual (2005).

The hydrologic modelling completed in the Eight Mile Creek Flood Study (URS, 2012) was used to determine the inflow to the dam during the PMF event, along with the downstream tributary coincident floods.

Probable Maximum Precipitation (PMP) design storm envelopes were estimated according to the Generalised Short-Duration Method (BOM, 2003). Following a conservative approach a uniform spatial distribution was applied. Based on the location of the catchment PMP estimates are limited to storm durations of 3 hours and have a notional 10,000,000 year ARI. PMP estimates for storm durations greater than 3 hours were estimated according to the Generalised Southeast Australia Method (BOM, 1996). Calculation details for the PMP estimation are provided in the Eight Mile Creek Flood Study (URS, 2012).

Table 4-1 Final PMP Estimates (URS, 2012)

Duration (hours)	Final PMP Estimate (from envelope)
1	240
2	360
3	440
6	460
12	490
24	550
36	640
48	680
72	720
96	730

Hydrologic modelling was completed through XP-RAFTS to produce both total and local sub-catchment hydrographs, whilst the hydraulic modelling was undertaken in the MIKE11 software package, details of both can be found in the previously completed report (URS, 2012). The sum of three hydrographs which were routed through MIKE11 were input into HEC-HMS, which is discussed further in Section 5.1 below.

To be conservative, it was assumed that all sub-catchments upstream of the LEWSD would flow into the dam and that both the east and west diversion drains were 100% blocked. Although these drains have large capacity and there is no evidence of past blockage scenarios, the assumption that the drains will block during the PMP events will not have a significant impact of the outcome of the spillway capacity assessment. Since the reservoir has a freeboard of 1.05 meters during the PMF event there is little reason to see a problem arise in the future, however, it is recommended that in the next spillway capacity assessment a review of the capacity of the diversions is undertaken.

For the PMF scenario the critical duration was the 2.5 hour storm event and the maximum calculated inflow to LEWSD was 208m^3/s. The peak reservoir level reached for the PMF event was 214.30m AHD, which gives a 1.05 meter freeboard.

Dam Break Modelling

5.1 HEC-HMS Dam Break Modelling

The Hydrologic Modelling System developed by the U.S. Army Corps of Engineers Hydrologic Engineering Centre (HEC-HMS) was used to estimate the breach outflows resulting from the failure of the LEWSD investigation.

HEC-HMS produced a dam-break outflow hydrograph that was then inserted into the TUFLOW hydraulic models as the upstream boundary condition. HEC-HMS was compared with the dam break component of MIKE11 and the HEC-HMS results were more conservative.

The key failure mode modelled was a piping failure under both flood and sunny day conditions. As the dam can store the PMF with freeboard it was not considered necessary to model an overtopping failure.

5.1.1 Breach Flows

The critical duration hydrographs from XP-RAFTS (URS, 2012) for the PMF event were input into HEC-HMS along with dam breach parameters to estimate the breach hydrographs from LEWSD.

As the PMF event does not overtop the dam embankment, piping failure was modelled for the both the PMF and sunny day failure dam break scenarios. Dam breach parameters were estimated using the equations reviewed by the Office of the State Engineer Dam Safety Branch (2010). The Froehlich (2008) method was recommended for Large dams with High Storage Intensities, which is the situation for LEWSD. The adopted dam breach parameters are summarised in Table 5-1.

Table 5-1 LEWSD HEC-HMS Parameters

Breach Parameter	Value	Method/Explanation
Final breach bottom width	23.9m	*Froelich (2008)* Width of the bottom of the breach
Final breach bottom elevation	205.0 m AHD	Upstream and downstream toe of dam embankment
Breach side slope	0.7H:1V	Maximum likely angle of repose of embankment material
Piping initiation level	209.25 m AHD	Office of the State Engineer Dam Safety Branch (2010) recommends the piping elevation to be half the embankment height.
Piping coefficient	0.7	Office of the State Engineer Dam Safety Branch (2010) recommends the piping elevation of between 0.68 and 0.75 for 15.24m high embankments
Breach formation time	48 mins	*Froelich (2008)* Full breach width and depth developed in 48mins

Other characteristics relevant to the determination of dam break parameters are provided below.

Crest width: 3m

Upstream batter: 2.5H:1V

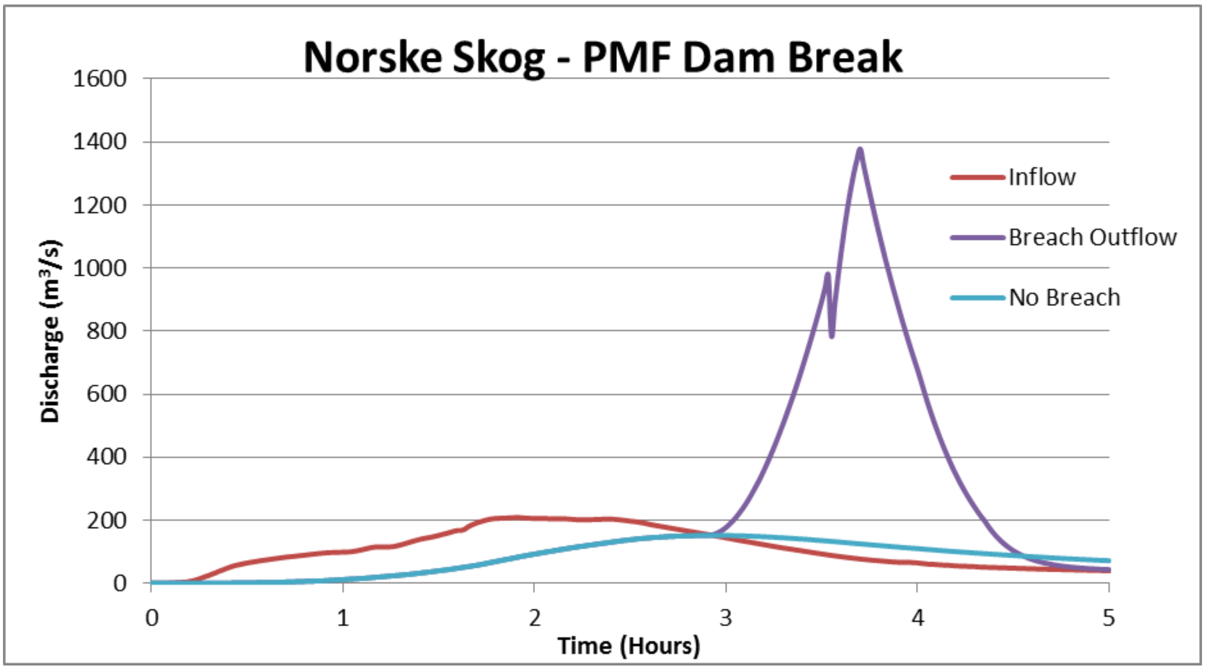
Downstream batter: 3.0H:1V

Embankment Material: Homogeneous earthfill (Zone 1) embankment with a Zone 2A vertical chimney filter.

5.1.1.1 PMF Failure and No Breach

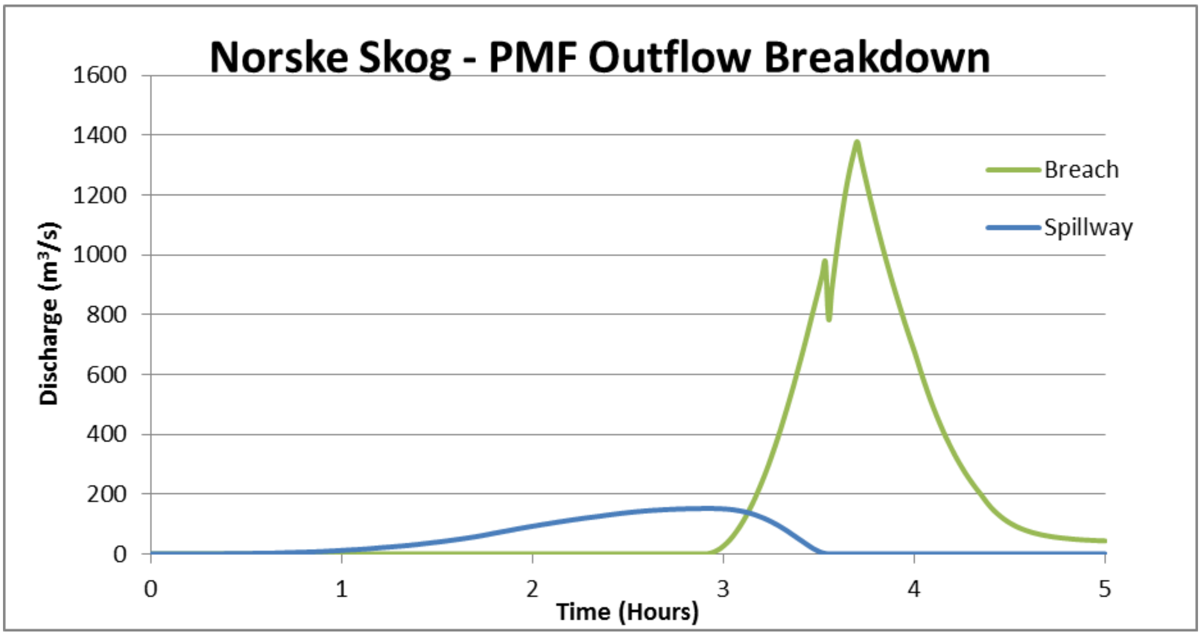
As stated above the PMF event does not overtop the dam embankment, the output from the HEC-HMS model was the hydrographs for the breach and no breach scenarios, these are shown in Figure 5-1.

Figure 5-1 PMF Outflow Hydrographs



The output from the HEC-HMS model has been separated into outflow from the spillway before the breach completely develops and then the flow through the embankment breach, these are shown in in Figure 5-2 below.

Figure 5-2 PMF Breach Outflow Breakdown

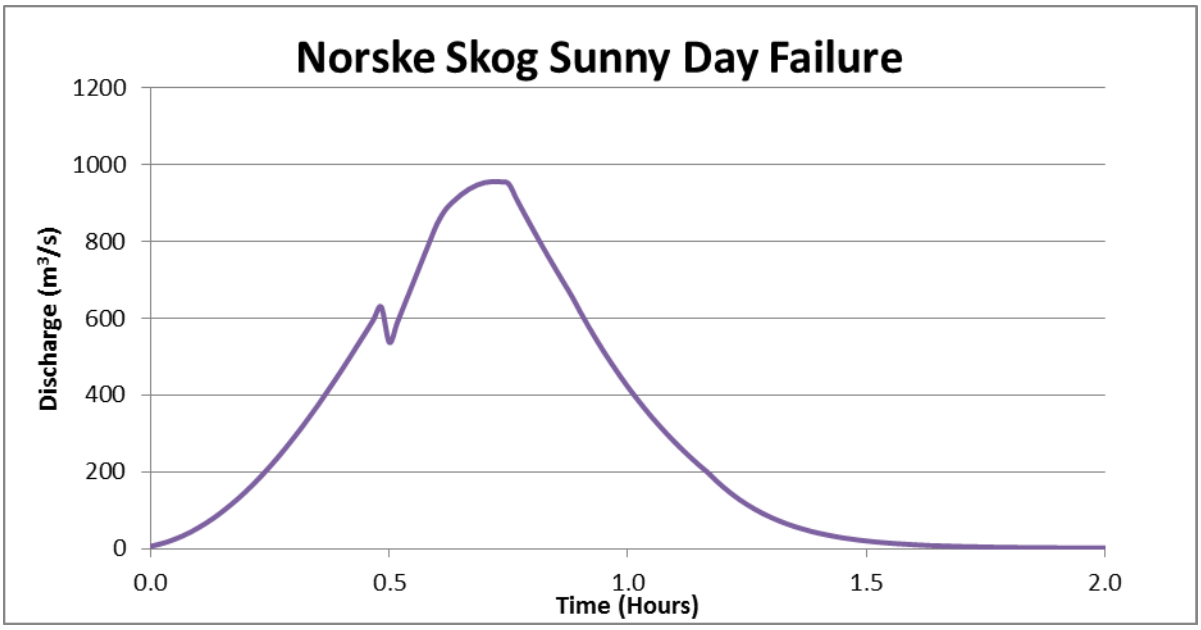


5.1.1.2 Sunny Day Failure

The sunny day failure of LEWSD also assumed a piping failure and the hydrograph obtained from HEC-HMS can be found in

Figure 5-3 below.

Figure 5-3 PMF Outflow Hydrographs



There are a number of limitations with this dam-break modelling approach which need to be considered. ANCOLD (2012) guidelines state that the accuracy of dam breach flow depth modelling is ±1m, which is taken under ideal conditions. Some of the other effects which are not taken into account, due to 2-dimensionsal cross-sectional modelling are rolling waves from breach scenarios and run-up around bends.

5.2 TUFLOW Models

TUFLOW was adopted as the flood routing software package for LEWSD, in lieu of a 1D model (such as MIKE 11), as a 2D model was considered more appropriate for use in all failure scenarios i.e. there is no single defined flow path and the downstream terrain results in multiple flow paths.

TUFLOW is a 1D/2D hydrodynamic model that was developed to simulate flood extents where flow patterns are poorly defined and/or unsteady. Modelling of flow in dynamic mode allows for the storage and attenuation effects of ponding and overland flow to be accurately modelled within local depressions of the ground surface.

A 5m gridded ground surface was adopted and this was developed in 12D based on LiDAR data provided. This ground surface is used by TUFLOW to determine the overland flow paths. Breach hydrographs were inserted into the models along a polygon of the assumed breach base width in the assumed failure locations.

Refer to Appendix G for inundation extents.

5.2.1 Surface Roughness

TUFLOW used the planning zones in order to assign a Manning's surface roughness to each specific zone. The adopted Manning's n values are consistent with the Melbourne Water Flood Mapping Guidelines (2010) and the values are listed in Table 5-2.

Table 5-2 Surface Roughness - Manning's n

Land Use	Manning's Coefficient
Residential - Rural (low density)	0.1
Residential - Medium/High density	0.2

5.2.2 Boundary Conditions

5.2.2.1 Inflow Hydrographs

The inflow hydrographs have been taken directly from the output hydrographs of the hydrologic modelling as discussed in Section 5.1.1. These hydrographs are located on the upstream ends of the model, in addition to contributing sub-catchments along the reaches discussed in Section 0.

5.2.2.2 Murray River Levels

The Murray River flood of 1917 has been used as the downstream boundary condition for the model. The level of the river during this flood was in the range of 157.5m – 158m AHD. Following a conservative approach a level of 158m AHD has been used across all storm events.

5.2.3 Model Checks

5.2.3.1 Monitoring of Flood Volumes

TUFLOW outputs the volume of water entering and leaving the model at every time step. This information, along with other parameters, were monitored throughout the model run in order to ensure that the model was running as intended and that the peak flood depth had been captured.

5.2.3.2 Plot Output Lines

A series of Plot Output (PO) lines were also included in the TUFLOW model. These lines give outputs of the flow at a particular location. These PO lines were used to ensure that the coincident flows were input correctly.

5.2.3.3 Model Errors

As shown in Table 5-3, the cumulative mass errors in these TUFLOW models are quite small and very consistent between models. The first 30 minutes of errors were not taken into consideration as this time was needed to allow the model to stabilise. Using a smaller grid size or shorter timestep when running the model would decrease these errors; however due to the fact that the irregularities are so small, it was assessed that this would have virtually no effect on the flood extent and depths of flooding and would significantly increase the model run time.

5 Dam Break Modelling

Table 5-3 Peak Cumulative Mass Error Range

Model Scenario	Peak Cumulative Mass Error Range (%)
PMF* Dam Break	1.0
PMF* No Fail	1.0
Sunny Day Failure	1.0

* The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

5.2.4 Coincident Flows

The critical duration hydrographs from MIKE 11 (URS, 2012) for the PMF event at downstream tributaries were obtained and 23 were input to the TUFLOW model in the appropriate locations to replicate the most realistic dam break scenario.

Consequence Category Classification

6.1 Introduction

ANCOLD *Guidelines on the Consequence Categories for Dams* (ANCOLD, 2012) allow each dam to be given a Consequence Category. This in turn relates to a Fallback Flood Capacity that the dam is required to retain and the required surveillance and monitoring for the dam. The term Consequence Category was previously known as Hazard Category in the *Guidelines on the Assessment of the Consequences of Dam Failure* (ANCOLD, 2000)

This section of the report contains:

- Explanation of the Population at Risk (PAR) and incremental PAR;
- Potential Damages and Losses;
- Assessment methodology of the Consequence Category; and
- Explanation of estimation of Potential Life Loss (PLL).

6.2 Incremental Population at Risk (PAR)

Following the production of the various inundation extents, the population at risk (PAR) was estimated. The PAR was assessed using GIS software, the Albury Local Environmental Plan 2010 and aerial imagery to estimate the number of properties that are inundated as a result of each flood scenario. In this analysis, a property was considered to be ‘at risk’ if the centroid of the place of residence was inundated by greater than 300mm.

The assumed PAR per property in Thurgoona, which is the suburb located downstream of LEWSD is 2.87. This value was based upon data from the Population and Household Forecasts (Albury City Council, 2012) occupancy rates for 2011.

Using the above value, the PAR for the flood events considered were assessed. The incremental PAR was estimated by subtracting the no failure scenario from the corresponding failure scenario.

It should be noted that for estimating Potential Life Loss (PLL), the total no fail PLL is determined and then subtracted from the breach scenario PLL to determine the incremental PLL. Refer to Section 6.5 for more information.

6.3 Economic Inundation Consequences

The economic consequences from inundation of buildings downstream of the spillway were assessed using SMEC (2009) unit damages for different property types. The assumed damages per property given different depths of flooding are shown in Table 6-1. By using the SMEC values for this study, URS has provided a consistent basis for comparing economic consequences for previous spillway capacity assessments.

6 Consequence Category Classification

Table 6-1 Damage Costs per Property

Property Type	Unit Cost
Residential house deep flooding	$ 100,000
Residential house shallow flooding	$ 80,000
Residential unit deep flooding	$ 80,000
Residential unit shallow flooding	$ 60,000
Industrial lot deep flooding	$ 150,000
Industrial lot shallow flooding	$ 100,000
Large industrial lot deep flooding	$ 250,000
Large industrial lot shallow flooding	$ 150,000
Primary School flooding	$ 300,000
Reserve pavilion	$ 10,000
Secondary school flooding	$ 500,000
Shopping centre flooding	$ 2,000,000

(Source: SMEC 2009)

The depth of flood inundation determines whether flood damages are mainly associated with the building structure, or fixtures and contents that will have to be replaced. For this investigation a shallow flood refers to the situation where a property is inundated by 300mm or less. A deep flood refers to a flood event that results in inundation of the property by greater than 300mm.

An investigation was undertaken to review whether the SMEC (2009) unit costs remained current and accurate for estimating economic consequences. A range of depth damage functions that have been developed in the US were reviewed to determine whether the damages estimated by SMEC remained appropriate. The depth damage functions (DDF) are available in 'Benefit-Cost Analysis' tool Version 4.5 (FEMA, 2012) developed by the Federal Emergency Management Agency (FEMA).

The DDFs relate flood damages (both building and contents) to a building's replacement value (building only). Rawlinsons Australian Construction Handbook (2012) was used to determine the rebuild cost of houses within the inundation extent. The assumptions for this investigation include:

- Residential damage factor for structure and contents adopted was 'USACE generic depth damage functions'
- Average residence area of 200m^2
- Houses downstream of all sites are medium standard, brick veneers with a rebuild value of $1,450/m^2

Table 6-2 shows the damage cost to residential properties with varying inundation depths.

Table 6-2 Flood Inundation Damage Cost (FEMA) – Residential

Inundation (m)	Cost
0.3	$ 62,000
1	$ 155,000
2	$ 251,000
3	$ 310,000

Comparisons between SMEC (2009) and the values obtained from FEMA are similar for a depth of inundation of 0.3m, which is approximately the height of the first floor level; however SMEC's values for 'deep flooding' are appropriate for perhaps 0.6 metres of flooding. The SMEC unit costs include no damages for greater depths of inundation.

Although URS consider that the inundation depth categorisation by SMEC could be refined, especially where catchments typically have inundation over 1.5 metres this approach is considered appropriate for a high level planning investigation.

6.4 Consequence Category Assessment

Based on the results of the dam break modelling the incremental population at risk was estimated for the dam at the current level of development. In conjunction with an assessment of the severity of damages and loss, and the incremental potential loss of life (PLL), Table 4 (ANCOLD, 2012) was used to determine the appropriate flood Consequence Category for the LEWSD. Consistent with the philosophy of ANCOLD *Guidelines On The Consequence Categories For Dams* (ANCOLD, 2012), the most conservative estimate of the severity of damages and loss was used to determine the appropriate flood consequence category for the dam.

6.5 Potential Loss of Life

The "*Flood Severity Based Method for Estimating Loss of Life*" published in "*A Procedure for Estimating Loss of Life Caused by Dam Failure*" (Graham, 1999), was used to assess the Potential Life Loss for the two breach scenarios. This method is herein referred to as the "Graham Method". It should be noted that this method is not appropriate for estimating Potential Life Loss associated with natural flooding where no breach occurs.

In accordance with the Graham Method, each investigation area required an assessment of:

- Flood Severity Category (Low, Medium or High);
- Warning Time Category (None, Some or Adequate); and
- Flood Severity Understanding (Vague or Precise).

The following sections detail the assessments completed in accordance with the Graham Method.

6.5.1 Flood Severity Category

The velocity and depth of floodwaters determine the flood severity. A general description of the flood severity categories defined by Graham is provided below.

- Low Severity – appropriate where no buildings are washed off their foundations (water depths less than ~ 3m)
- Medium Severity – applied to locations where homes are destroyed but trees or mangled homes remain for people to seek refuge on (water depths >3m)
- High Severity – for locations flooded by the near instantaneous failure of a concrete or earthfill dam (where the dam fails in seconds rather than minutes or hours)

The flood severity is determined by reviewing the estimated depths of flooding on the flood plain and the estimated velocity of water on the flood plain. All failure scenarios were regarded as being low severity with only localised areas with depths in excess of 3m, typically immediately downstream of failed embankments or in waterways and not inundating any properties.

6.5.2 Warning Time Category

Warning time is defined as the time available for the responsible authority (in this case the Police and the SES) to issue a warning prior to the arrival of peak flood waters. The warning time category for each scenario was assigned based on the following criteria:

- None when warning time is < 15mins.
- Some when waring time is 15 to 60mins.
- Adequate when warning time is > 60mins.

Due to the short reaches involved in the investigation it is assumed that less than 15 minutes warning is available to the population within the inundation extent.

6.5.3 Flood Severity Understanding

The flood severity understanding is divided into either a vague understanding or a precise understanding.

A vague understanding is described as the warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding. Conversely, a precise understanding implies that the warning issuers have witnessed a dam failure and then able to communicate appropriate warnings.

6.5.4 Fatality Rates and PLL Estimation

According to the Graham Method and the assessment described above, a fatality rate of 0.01 has been selected for the breach scenarios. This corresponds to a low flood severity with no warning. Refer to Figure 6-1 for the recommended fatality rates from Graham.

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)	
			Suggested	Suggested Range
HIGH	no warning	not applicable	0.75	0.30 to 1.00
	15 to 60	vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
		precise		
	more than 60	vague		
		precise		
MEDIUM	no warning	not applicable	0.15	0.03 to 0.35
	15 to 60	vague	0.04	0.01 to 0.08
		precise	0.02	0.005 to 0.04
	more than 60	vague	0.03	0.005 to 0.06
		precise	0.01	0.002 to 0.02
LOW	no warning	not applicable	0.01	0.0 to 0.02
	15 to 60	vague	0.007	0.0 to 0.015
		precise	0.002	0.0 to 0.004
	more than 60	vague	0.0003	0.0 to 0.0006
		precise	0.0002	0.0 to 0.0004

Source: Graham, W.J., 1999. *A Procedure for Estimating Loss of Life Caused by Dam Failure, DSO-99-06,* United States Department of Interior, Bureau of Reclamation, Sedimentation and River Hydraulics.

Figure 6-1 Graham Fatality Rates (Table 7 from (Graham, 1999))

6.5.5 Potential Life Loss for Natural Flooding Scenario

The current ANCOLD *Guidelines on Risk Assessment* (ANCOLD, 2003) state that the Graham method is not suitable for estimating Potential Loss of Life associated with natural flooding. This is due to the fact that the Graham method was developed based on data associated with dam failures and flash floods and did not include consideration of gradual flood level rises.

To estimate Potential Loss of Life for the non-failure cases a number of natural floods were reviewed. Hill (2007) has identified that a dam with a flood severity of ‘Low’, expected a fatality rate of 0.0002 (1 in 5,000), based on average fatality rates in the Dartmouth Flood Observatory, which is corresponding to the lower boundary of the Graham (1999) method. This fatality rate was adopted for the PMF no breach scenario.

6.6 Results

6.6.1 PMF Flood Scenario

6.6.1.1 Incremental PAR and PLL

The estimated incremental PAR, along with the PLL for the PMF dam breach scenario and no failure scenarios are shown in Table 6-3. The PLL estimates were determined using Graham (1999) for dam breach events and Hill (2007) for no fail events. The incremental PAR and PLL are estimated to be 161 and 4.15 respectively.

Table 6-3 Population at Risk and Potential Loss of Life for PMF* Scenario

	Population at Risk	Potential Loss of Life
No Fail	260	0.05
Dam Break	421	4.21
Incremental	161	4.15

* The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

6.6.1.2 Severity of Damage and Loss

The severity of damage and loss using the ANCOLD *Guidelines On The Consequence Categories For Dams* (ANCOLD, 2012) for this dam was assessed to be **Medium**. A summary of the assessment of severity of damage and loss is provided in Table 6-4 with a discussion on each category in the following sections.

Table 6-4 Severity of Damage and Loss for PMF* Scenario (ANCOLD 2012)

Category	Type	Limiting factor description
Total Infrastructure Costs	Minor	Damage to residential properties and infrastructure expected to be less than $10M
Impact on Dam Owner's Business	Medium	Community reaction and political implications: Severe widespread reaction
Health and Social Impacts	Minor	Human Health: <100 people affected
Environmental Impacts	Medium	Area of environmental impact less than 5km^2
Total outcome	**Medium**	**Most critical of above criteria**

* The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

6 Consequence Category Classification

Total Infrastructure Costs

Table 6-5 provides a breakdown of the estimated property damages as result of the flood scenarios.

Table 6-5 Cost Estimation for PMF* Scenario

	No Fail		Dam Break	
Property Type	No.	Cost	No.	Cost
Residential house deep flooding	86	$ 8,600,000	142	$ 14,200,000
Residential house shallow flooding	20	$ 1,600,000	42	$ 3,360,000
Thurgoona Country Club	13	$ 1,040,000	13	$ 1,040,000
Reserve pavilion	1	$ 10,000	1	$ 10,000
Dam Replacement Costs	0	$ -	1	$ 1,000,000
Totals	**120**	**$ 11,250,000**	**198**	**$ 19,610,000**

* The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

As shown in Table 6-5, the incremental dam breach cost is estimated to be $8.36 million which is due to an incremental number of houses becoming inundated. These properties which are inundated in a breach situation and not inundated during a natural flooding event are mainly in new development sites.

The ANCOLD 2012 guidelines consider the total infrastructure costs, with residential, commercial, infrastructure and dam replacement costs all contributing. While some roads and other infrastructure, such as railways, power and other utilities are potentially to be inundated as a result of an embankment failure, the incremental effect on this infrastructure is likely to be negligible due to the small increase in flood depths expected from the dam breach. For this reason, the major contributor to cost is expected to be the inundation of the residential properties and embankment rebuild. This is likely to be less than $10 million, which results in the dam being categorised as **Minor** for total infrastructure costs.

Impact on Dam Owner's Business

Due to the potential life loss and significant property and infrastructure damage expected there is likely to be a severe widespread reaction to failure of the embankment. This could manifest itself in political and hence regulatory implications imposed on Norske Skog. In respects to the impact on the dam owners business the LEWSD is assessed as being **Medium** according to the ANCOLD 2012 guidelines.

Health and Social Impacts

As the flow path doesn't run through any business or industrial areas, this is not a cause of concern. The only social impact is that the flood for both scenarios inundate Thurgoona Reserve, which is a local recreational facility. This results in a **Minor** rating being assigned for the Health and Social Impact.

Environmental Impacts

Due to the coincident flows incorporated downstream of the dam, the incremental area of impact was estimated as 1.2km^2, which categories the dam to have a Medium impact on the environment.

6.6.1.3 Consequence Category

The consequence category for LEWSD was determined using the ANCOLD *Guidelines on the Consequence Categories for Dams* (2012). The consequence category determined using these guidelines was **High C** for the PMF flood event based on a piping failure through the embankment when Table 4 in ANCOLD *Guidelines on the Consequence Categories for Dams* (2012) was adopted.

Sections 6.6.1.1 and 0 above discuss the estimated PLL and severity of damage loss adopted to determine the consequence category of the dam.

6.6.1.4 Required Flood Capacity

The ANCOLD *Guidelines on Selection of Acceptable Flood Capacity for Dams (2000) recommend a* Fallback Flood Capacity (FFC) of between a 1 in 10,000 AEP event and a 1 in 100,000 AEP event for a High C consequence category dam. Table 6-6 shows the existing flood capacity for the dam compared to the FFC for a High C hazard category dam.

Table 6-6 Comparison between current flood capacity and fallback flood capacity for PMF* scenario

IFHC Rating	Current Flood Capacity (AEP)	Fallback Flood Capacity (AEP)
High C	PMF*	Between 1 in 10,000 & 1 in 100,000

*The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

The current flood capacity of the dam meets the ANCOLD FFC for a High C consequence category dam. Figure 6-2 and Figure 6-3 below show the inundation maps for both the PMF breach and no breach scenarios. Areas inundated by green indicate an inundation depth of 300 mm or less. Areas inundated by red indicate a depth of greater than 300 mm.

All the inundation maps can be found in Appendix G.

6 Consequence Category Classification

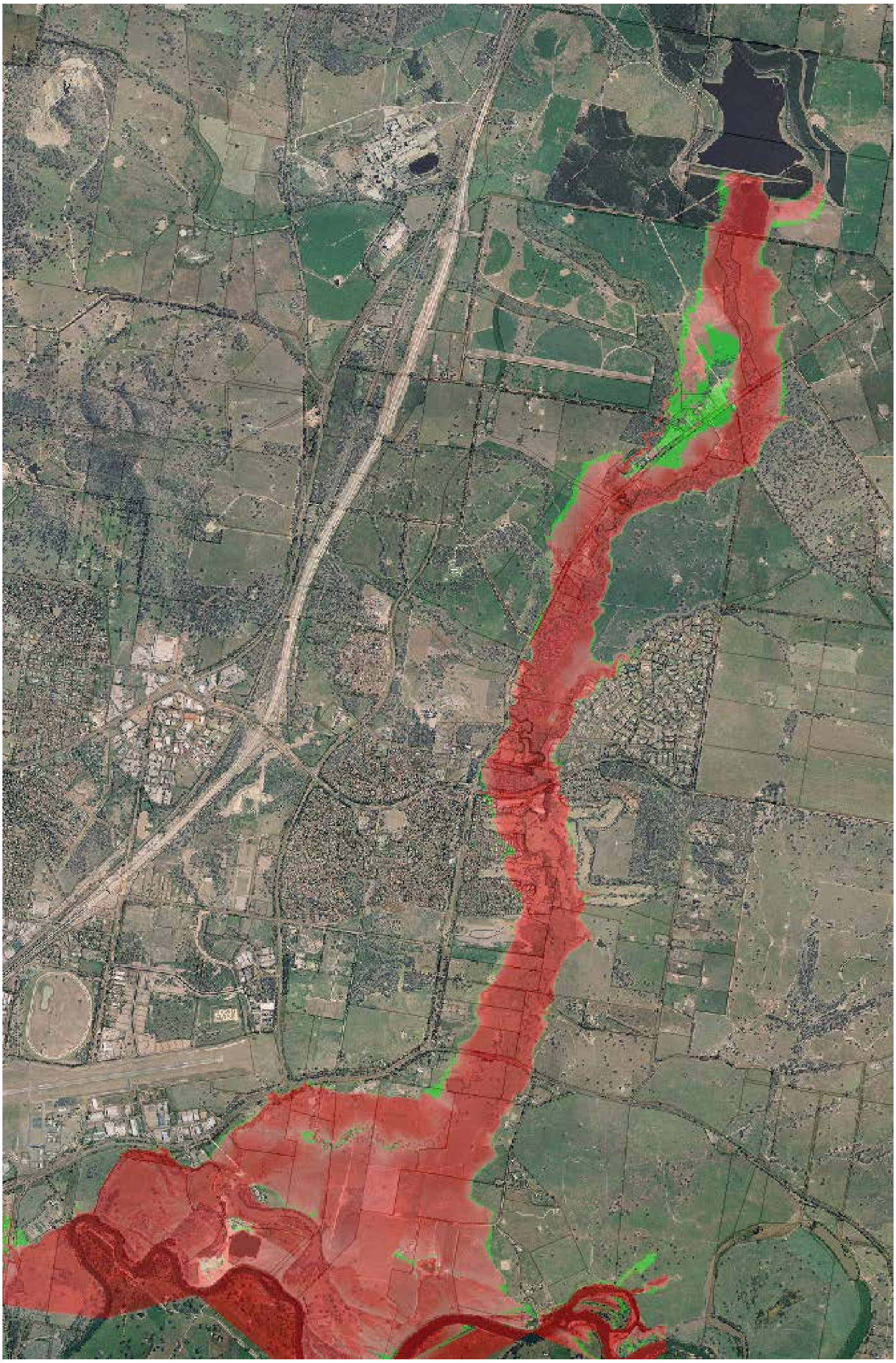


Figure 6-2 PMF Inundation - Dam Breach Scenario

6 Consequence Category Classification

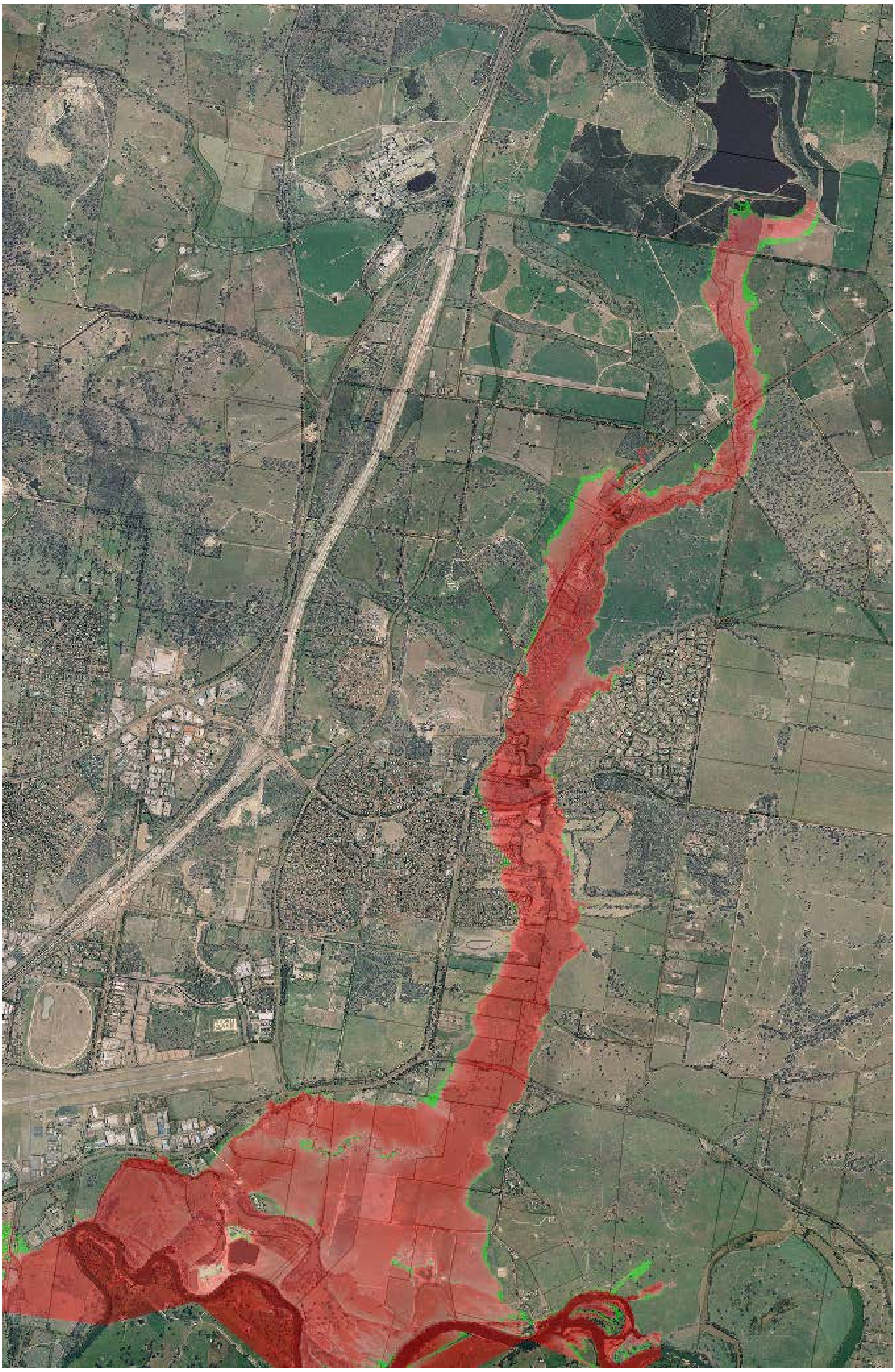


Figure 6-3 PMF Inundation - No Fail Scenario

6.6.2 Sunny Day Failure Scenario

6.6.2.1 Incremental PAR and PLL

The estimated incremental PAR, along with the PLL for the sunny day failure scenario and no failure scenarios are shown in Table 6-7. The PLL estimates were determined using Graham (1999) for dam breach events and Hill (2007) for no fail events. The incremental PAR and PLL are estimated to be 208 and 2.08 respectively.

Table 6-7 Population at Risk and Potential Loss of Life for Sunny Day Failure Scenario

	Population at Risk	Potential Loss of Life
No Fail	0	0.00
Dam Break	208	2.08
Incremental	208	2.08

6.6.2.2 Severity of Damage and Loss

The severity of damage and loss using the ANCOLD *Guidelines On The Consequence Categories For Dams* (ANCOLD, 2012) for this dam was assessed to be **Medium**. A summary of the assessment of severity of damage and loss is provided in Table 6-8 with a discussion on each category in the following sections.

Table 6-8 Severity of Damage and Loss for Sunny Day Failure Scenario (ANCOLD 2012)

Category	Type	Limiting factor description
Total Infrastructure Costs	Minor	Damage to residential properties and infrastructure expected to be less than $10M
Impact on Dam Owner's Business	Medium	Community reaction and political implications: Severe widespread reaction
Health and Social Impacts	Minor	Human Health: <100 people affected
Environmental Impacts	Medium	Area of environmental impact less than 5km^2
Total outcome	**Medium**	**Most critical of above criteria**

6 Consequence Category Classification

Total Infrastructure Costs

Table 6-9 provides a breakdown of the estimated property damages as result of the flood scenarios.

Table 6-9 Cost Estimation for Sunny Day Failure Scenario

	No Fail		Dam Break	
Property Type	No.	Cost	No.	Cost
Residential house deep flooding	0	$ -	68	$ 6,800,000
Residential house shallow flooding	0	$ -	6	$ 480,000
Thurgoona Country Club	0	$ -	13	$ 1,040,000
Dam Replacement Costs	0	$ -	1	$ 1,000,000
Totals	**0**	**$ -**	**88**	**$ 9,320,000**

As shown in Table 6-9, the incremental dam breach cost is estimated to be $9.32 million which is due to an incremental number of houses becoming inundated and dam replacement costs. This is likely to be less than $10 million, which results in the dam being categorised as **Minor** for total infrastructure costs.

Impact on Dam Owner's Business

Due to the potential life loss and significant property and infrastructure damage expected there is likely to be a severe widespread reaction to failure of the embankment. This could manifest itself in political and hence regulatory implications imposed on Norske Skog. In respects to the impact on the dam owners business the LEWSD is assessed as being **Medium** according to the ANCOLD 2012 guidelines.

Health and Social Impacts

As the flow path doesn't run through any business or industrial areas, this is not a cause of concern. There are no social impacts, resulting in a **Minor** rating being assigned for the Health and Social Impact.

Environmental Impacts

The total area of inundation for the Sunny Day Failure scenario is over 6.5km^2, of which approximately half is made up of land prone to natural flooding. Therefore the area of impact to the environmental is less than 5km^2, assigning LEWSD a Medium severity for Environmental Impacts.

6.6.2.3 Consequence Category

The consequence category for LEWSD was determined using the ANCOLD *Guidelines on the Consequence Categories for Dams* (2012). The consequence category determined using these guidelines was **High C** for the sunny day failure t based on a piping failure through the embankment.

Sections 6.6.2.1 and 6.6.2.2 above discuss the estimated PLL and severity of damage loss adopted to determine the consequence category of the dam for the Sunny Day Failure scenario.

6.6.2.4 Required Flood Capacity

The ANCOLD *Guidelines on Selection of Acceptable Flood Capacity for Dams (2000) recommend a* Fallback Flood Capacity (FFC) of between a 1 in 10,000 AEP event and a 1 in 100,000 AEP event for a High C consequence category dam. Table 6-10 shows the existing flood capacity for the dam compared to the FFC for a High C hazard category dam.

Table 6-10 Comparison between current flood capacity and FFC for Sunny Day Failure scenario

IFHC Rating	Current Flood Capacity (AEP)	Fallback Flood Capacity (AEP)
High C	PMF*	Between 1 in 10,000 & 1 in 100,000

* The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

The current flood capacity of the dam meets the ANCOLD FFC for a High C consequence category dam.

The inundation map of the Sunny Day Failure is shown in Figure 6-4 below. Areas inundated by green indicate an inundation depth of 300 mm or less. Areas inundated by red indicate a depth of greater than 300 mm.

6 Consequence Category Classification

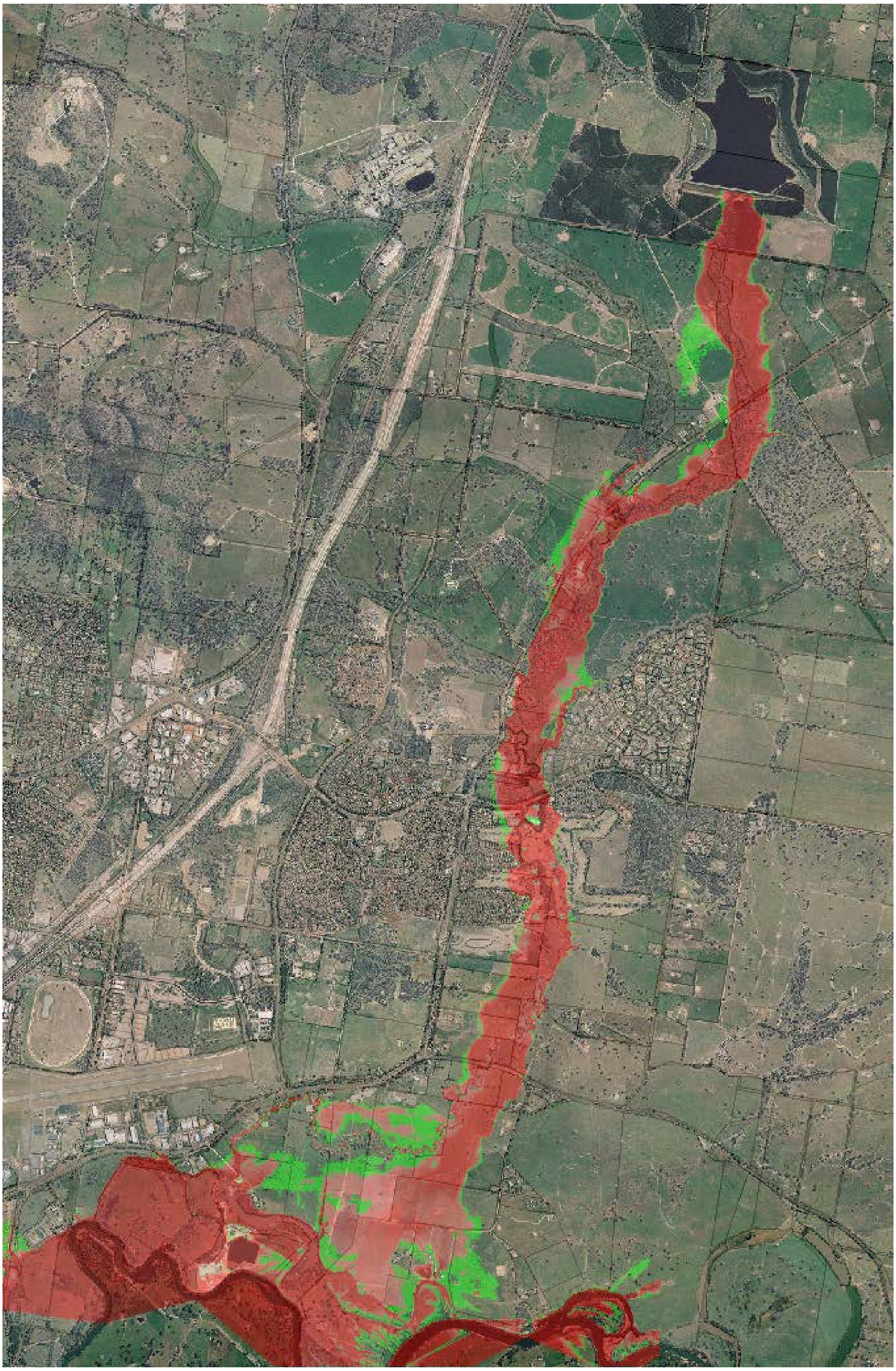


Figure 6-4 Sunny Day Failure Inundation

6.7 Future Development

The consequence category for both the PMF flood failure and the Sunny Day Failure are dependent on the development of the properties downstream. The Albury City Council (Albury City Council, 2010) developed a local environmental plan in 2010 which was to determine the zones in which residential development could take place.

If some of the General Residential (R1) zones are developed, especially higher upstream near the intersection of Table Top Road and Williams Road, it is likely that the consequence category could change from the current High C to a High A.

Although the spillway capacity of the dam is the PMF event and a High A event will require a Flood Fallback Capacity of the PMP Design Flood the situation of a High A consequence category dam will entail much greater maintenance and surveillance. Refer to Table 6-11 to see some of the requirements for higher Consequence Categories.

Table 6-11 Requirements for various Consequence Categories

Situation	Previous Study (Rodd, 2009)	Current Study (URS, 2013)	Future*
Consequence Category	Significant	High C	High A
DSC Report Type	Type 3	Type 2	Type 1
Surveillance Personnel	Dam owner	At least one experienced dam/surveillance engineer	At least two experienced dam/surveillance engineers
Telemetered monitoring required	No	Yes	Yes

*Estimated if development of R1 zones are completed. An assessment on the consequence category will have to be completed to verify this.

The DSC report types, which extend from Type 1 to Type 3, differ in the detail of reporting along with the surveillance personnel required to complete the report. The Dam Safety Committee's (DSC) *'Surveillance Reports for Dams'* (NSW Dam Safety Committee, 2010) have details on the different report formats and surveillance teams required for the various types of reports.

As stated in the DSC's *'Emergency Management for Dams'* (NSW Dam Safety Committee, 2010) "owners of Extreme and High Consequence Category dams are to have in place automatic telemetered monitoring of the storage level in their dams (and preferably rainfall and seepage as well)." It is also stated that "the DSC also required the owners of remotely located Extreme and High Consequence Category embankment dams to consider the practicalities of installing telemetered tailwater/seepage monitoring devices to maximise warning times potential piping incidents at these dams."

It is recommended by URS that a Spillway Capacity Assessment be completed once every 5 years to keep an up-to-date consequence category of LEWSD. This is in line with the recommendations by the NSW Dam Safety Committee in the *'Operation and Maintenance for Dams'* (NSW Dam Safety Committee (a), 2010) of an assessment made "5 yearly for Extreme and High A Consequence Category dams ranging out to 10 yearly for Significant Consequence Category dams".

Conclusions and Recommendations

7.1 Piping Assessment

Three key failure modes were identified as part of the piping assessment. The probability of these failure modes were assessed using the procedures recommended in the 'Piping Toolbox'. The key failure modes, their estimated annual probabilities of failure and their contribution to the total annual probability of failure through piping are provided in Table 7-1.

Table 7-1 Summary of Piping Related Failure Modes

Failure Mode	Annual Probability of Failure	Contribution
ETT-F1 Piping through the embankment (Normal and Earthquake)	$6.3x10^{-7}$	57%
ETT-F2 Piping along the outlet conduit (Normal)	$3.1x10^{-8}$	3%
ETT-F3 Piping through the foundation (Normal)	$4.5x10^{-7}$	40%
TOTAL	**$1.1x10^{-6}$**	**100%**

ETT-F1 Piping through the embankment and ETT-F3 are assessed to present the greatest contribution to the annual probability of failure through piping, with a combined 97%. ETT-F2 is estimated to have a probability of failure of greater than an order of magnitude below ETT-F1 and ETT-F3. The probability of piping failure is low which is consistent with the embankment design, construction and materials.

7.2 Hydrology

In March 2012, Albury City Council (ACC) commissioned URS to complete a flood study of the Eight Mile Creek catchment (URS, 2012). The study was carried out in accordance with the New South Wales (NSW) State Government Floodplain Development Manual (2005).

The hydrologic modelling completed in the Eight Mile Creek Flood Study (URS, 2012) was used to determine the inflow to the dam during the PMF event, along with the downstream tributary coincident floods. The LEWSD has PMF flood capacity with 1.05 meter freebaord.

7.3 Dam Break Modelling

The Hydrologic Modelling System developed by the U.S. Army Corps of Engineers Hydrologic Engineering Centre (HEC-HMS) was used to estimate the breach outflows resulting from the failure of the LEWSD investigation.

As the PMF event does not overtop the dam embankment, piping failure was modelled for the both the PMF and sunny day failure dam break scenarios. Dam breach parameters were estimated using the equations reviewed by the Office of the State Engineer Dam Safety Branch (2010).

The peak outflow for the PMF dam break scenario was 1,378 m^3/s, whilst the PMF no fail scenario had a peak discharge of 152 m^3/s through the spillway channel. The sunny day failure had a peak outflow of 957 m^3/s.

TUFLOW was adopted as the flood routing software package for LEWSD, in lieu of a 1D model (such as MIKE 11), as a 2D model was considered more appropriate for use in all failure scenarios i.e. there is no single defined flow path and the downstream terrain results in multiple flow paths.

Coincident floods downstream of the dam were considered for both the PMF breach and no breach scenarios. The critical duration hydrographs from MIKE 11 (URS, 2012) for the PMF event at downstream tributaries were obtained and these 23 hydrographs were input to the TUFLOW model in the appropriate locations to replicate the most realistic dam break scenario.

7.4 Consequence Category Classification

The consequence category was determined in accordance with ANCOLD *Guidelines on the Consequence Categories for Dams* (ANCOLD, 2012). A summary of the Population at Risk (PAR) and the Potential Loss of Life (PLL) values for both PMF and sunny day scenarios can be seen in Table 7-2.

Table 7-2 Summary of PAR and PLL results

	PMF*		Sunny Day Failure	
Scenario	PAR	PLL	PAR	PLL
No Fail	260	0.05	0	0.00
Dam Break	421	4.21	208	2.08
Incremental	**161**	**4.15**	**208**	**2.08**

* The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

The two dam break scenarios correspond to a severity of damage and loss using the ANCOLD *Guidelines On The Consequence Categories For Dams* (ANCOLD, 2012) of **Medium** and in conjunction with the PAR and PLL results the Consequence Category for both flood and sunny day scenarios for LEWSD is **High C**.

Table 7-3 Consequence Category Results

Dam Crest Flood (AEP)	Peak Outflow at PMF* (m^3/s)	Current Flood Capacity (AEP)	Consequence Category	Fallback Flood Capacity (AEP)
N/A	152	PMF*	High C	Between 1 in 10,000 & 1 in 100,000

* The PMF is based on the AEP of the PMP, which is estimated as the 1 in 10,000,000 AEP event.

For both the PMF flood failure and the Sunny Day Failure the consequence category is dependant of the amount of property development downstream. If there is a large amount of downstream property development it is likely that the consequence category could change from the current High C to a High A.

URS recommend that a Spillway Capacity Assessment be completed once every 5 years to keep an up-to-date consequence category of LEWSD. This is in line with the recommendations by the NSW Dam Safety Committee in the *'Operation and Maintenance for Dams'* (NSW Dam Safety Committee (a), 2010) of an assessment made "5 yearly for Extreme and High A Consequence Category dams ranging out to 10 yearly for Significant Consequence Category dams".

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Limitations

URS Australia Pty Ltd (URS) has prepared this report in accordance with the usual care and thoroughness of the consulting profession for the use of Norske Skog and only those third parties who have been authorised in writing by URS to rely on this Report.

It is based on generally accepted practices and standards at the time it was prepared. No other warranty, expressed or implied, is made as to the professional advice included in this Report.

It is prepared in accordance with the scope of work and for the purpose outlined in the contract dated 7th December 2012.

Where this Report indicates that information has been provided to URS by third parties, URS has made no independent verification of this information except as expressly stated in the Report. URS assumes no liability for any inaccuracies in or omissions to that information.

This Report was prepared between November 2013 and January 2014 and is based on the conditions encountered and information reviewed at the time of preparation. URS disclaims responsibility for any changes that may have occurred after this time.

This Report should be read in full. No responsibility is accepted for use of any part of this report in any other context or for any other purpose or by third parties. This Report does not purport to give legal advice. Legal advice can only be given by qualified legal practitioners.

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It is the responsibility of third parties to independently make inquiries or seek advice in relation to their particular requirements and proposed use of the site.

Any estimates of potential costs which have been provided are presented as estimates only as at the date of the Report. Any cost estimates that have been provided may therefore vary from actual costs at the time of expenditure.

Appendix A Dam Safety Committee Email

Purss, Cameron

From: Charles Navaratne <charles.navaratne@damsafety.nsw.gov.au>
Sent: Tuesday, 27 August 2013 2:21 PM
To: Purss, Cameron; Steve Knight
Cc: malcolm.alexander@norskeskog.com
Subject: RE: Lake Ettamogah Winter Storage Dam Construction Report

Dear Cameron,

There are many Reports available with DSC regarding the above dam.

They are;

1. Supplimentary Submission to the Commission of Inquiry – August 1992 by GHD
2. Geotechnical Report – April 1993 by GHD
3. Hydrology Report – May 1993 by GHD
4. Preliminary Design Report – May 1993 by GHD
5. Preliminary Sketch Plan Report – July 1993 by Willing & Partners
6. Geotechnical Studies Volume 1 – August 1993 by Coffey
7. Geotechnical Studies Volume 2 – August 1993 by Coffey - (This Report contains a Stability Analysis)
8. Seismic Assessment of Embankment – November 1993 by Coffey
9. WAE Drawings & Construction Certificate

But a Construction Report is not available.
Also it was noted that a Stability Analysis has been carried out in 1993.

Therefore DSC would like to help you with available reports and would like to inform that a **Stability Analysis** is no longer required.

Regards

Charles Navaratne
Small Dams Engineer
NSW Dams Safety Committee
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From: Purss, Cameron [mailto:cameron.purss@urs.com]
Sent: Friday, 23 August 2013 3:50 PM
To: charles.navaratne@damsafety.nsw.gov.au
Cc: Hannan, Elliot
Subject: Lake Ettamogah Winter Storage Dam Construction Report

Dear Charles,
URS are currently undertaking an investigation on the Lake Ettamogah Winter Storage Dam (aka Maryvale Winter Storage Dam) on behalf of Norske Skog Australia.

The investigation includes an assessment of the consequences of dam failure and a stability analysis.

Richard Rodd, who was involved in the original construction and authored the most recent Type 2 surveillance document on the dam, has advised that the original construction report was submitted to the Dam Safety

Committee, following completion of the dam in 1994. This document is important to enable URS to undertake the investigation.

Are you able to confirm if this document is available?

Regards,

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Appendix B Existing Stability Analysis

corresponding to a total seepage loss through the embankment of about 6 kl/day. Actual seepage losses should be significantly less than this, with a high possibility of not being measurable, as it is considered unlikely that steady state flow conditions will be established under normal operation of the dam and the permeability parameters adopted for design are considered to be conservative.

Seepage through the embankment will be collected in the chimney filter which will drain via finger drains at about 50m centres towards the downstream toe as shown in Figure 5. The finger drain will comprise slotted UPVC pipe wrapped in geotextile and encased in filter sand placed in trenches excavated into the stripped surface. Due to the length of the embankment and the recommendations on assessment of seepage contained in ANCOLD guidelines, each pipe should be brought outside the downstream toe so that individual seepage losses can be monitored. In order to maintain separation of chimney and tow drain seepages, the PVC pipe should be unslotted within the toe drain, to the details shown on Figure 5. As seepage is expected to be small, a measuring weir or similar on each outlet is not considered to be warranted. A small surface drain is provided to transport the seepage to the creek line. Pipes should enter the drain at such a level that a small container can be placed under the pipe to measure flows as required.

A toe drain is also provided to control the egress of seepage entering the downstream shell of the embankment via the foundation. The width of the toe drain varies with the embankment height as shown in Figure 6. The toe drain can be constructed from either free draining gravel materials wrapped in a geotextile or Zone 2A material. Simple rockfill outlets are provided, as it is not considered that segmentalising the toe drain (as for the chimney filter flow) is warranted.

5.4 Stability Analysis

Stability analyses have been carried out at the maximum section using the design cross section presented in Figure 4 and the material properties presented in Section 4.0. Based on the section presented in Figure 3, the alluvial clay soils in the foundation have been taken to be 2.5m thick. The analyses were carried out using the Coffey computer program COFSTAB and the Bishop simplified method of slices.

The following conditions were analysed for stability:

- end of construction, upstream and downstream slopes;
- steady state seepage, upstream and downstream slopes;
- rapid drawdown from FSL to empty, upstream slope, assuming steady state seepage conditions had been previously established and that no dissipation of excess pore

pressures occurred during drawdown; a pore pressure change equal to the change in vertical total stress was assumed;

- steady state seepage with earthquake, upstream and downstream slopes; earthquake conditions were analysed using the pseudo-static method of analysis with horizontal and vertical accelerations of 0.1g and 0.05g, respectively, where "g" is the acceleration due to gravity.

Undrained strength parameters were adopted for the end of construction analyses and drained strength parameters and effective stress analyses were adopted for the other cases. The results of the analyses are presented in Appendix D and sumarised below, together with the corresponding recommended minimum accepted factors of safety.

RESULTS OF STABILITY ANALYSIS

Analysis Condition	**Calculated Factor of Safety**	**Recommended Minimum Accepted Factor of Safety**
End of Construction - Upstream	2.54	1.5
End of Construction - Downstream	2.40	1.5
Steady State Seepage - Upstream	2.42	1.5
Steady State Seepage - Downstream	1.50	1.5
Rapid Drawdown - Upstream	1.49	1.3
Steady State Seepage + Earthquake - Upstream	1.33	1.2
Steady State Seepage + Earthquake - Downstream	1.17	1.2

Calculated factors of safety are equal to or greater than the recommended minimum factors of safety for all conditions except earthquake loading of the downstream face. It must be noted that the pseudo static method of analysis is at best approximate, and it is considered that the calculated value of the factor of safety for the downstream face (1.17) is sufficiently close to the recommended value to be accepted. Resistance to earthquake loading in this embankment is provided by:

- the absence of potentially liquefiable materials in the foundations or embankment;

- freeboard at the time of the earthquake likely to be at least 2m;

- static factors of safety of critical failure surfaces involving the crest at least 1.5 under conditions expected prior to the earthquake;

- a large thickness of clay materials constructed from CL/CH and CH clays to promote self-healing should a crack develop in the core.

5.5 Instrumentation

Instrumentation is based on the recommendations contained in the ANCOLD guidelines for small, high hazard dams, taking into account the expected behaviour of the dam. Recommended instrumentation comprises:

- Two cased and capped groundwater monitoring holes, one located in the east abutment (where the weathered rock is closer to the surface) and one located near the creek line. These holes will be additional to the present groundwater monitoring holes on the site. The holes should be drilled to a depth of about 10m, incorporate slotted PVC casing, be sealed near the surface to prevent infiltration of surface water and capped with a metal cap to reduce the potential for damage.

- Crest settlement points at about 200m intervals along the length of the embankment.

- Monitoring of seepage from the chimney filter as discussed in Section 5.3 above.

- A weir installed in the creek downstream of the dam to assess total seepage flow.

Installation of pore pressure monitoring piezometers is not considered necessary as it is unlikely that steady state flow conditions will ever be established. Should other indications, eg an increase in flow from a seepage pipe, be noted then standpipe or Casagrande type piezometers could be installed from the crest of the embankment at the location of interest at that time.

ANCOLD guidelines generally recommend monitoring of instrumentation at weekly intervals for small, high hazard dams. It is recommended that this interval be maintained for observation of the seepage outlet pipes, but extended intervals can be adopted for crest settlements and groundwater observations once the dam behaviour is established and after the first surveillance inspection.

APPENDIX D

RESULTS OF STABILITY ANALYSES

Coffey Partners International Pty Ltd

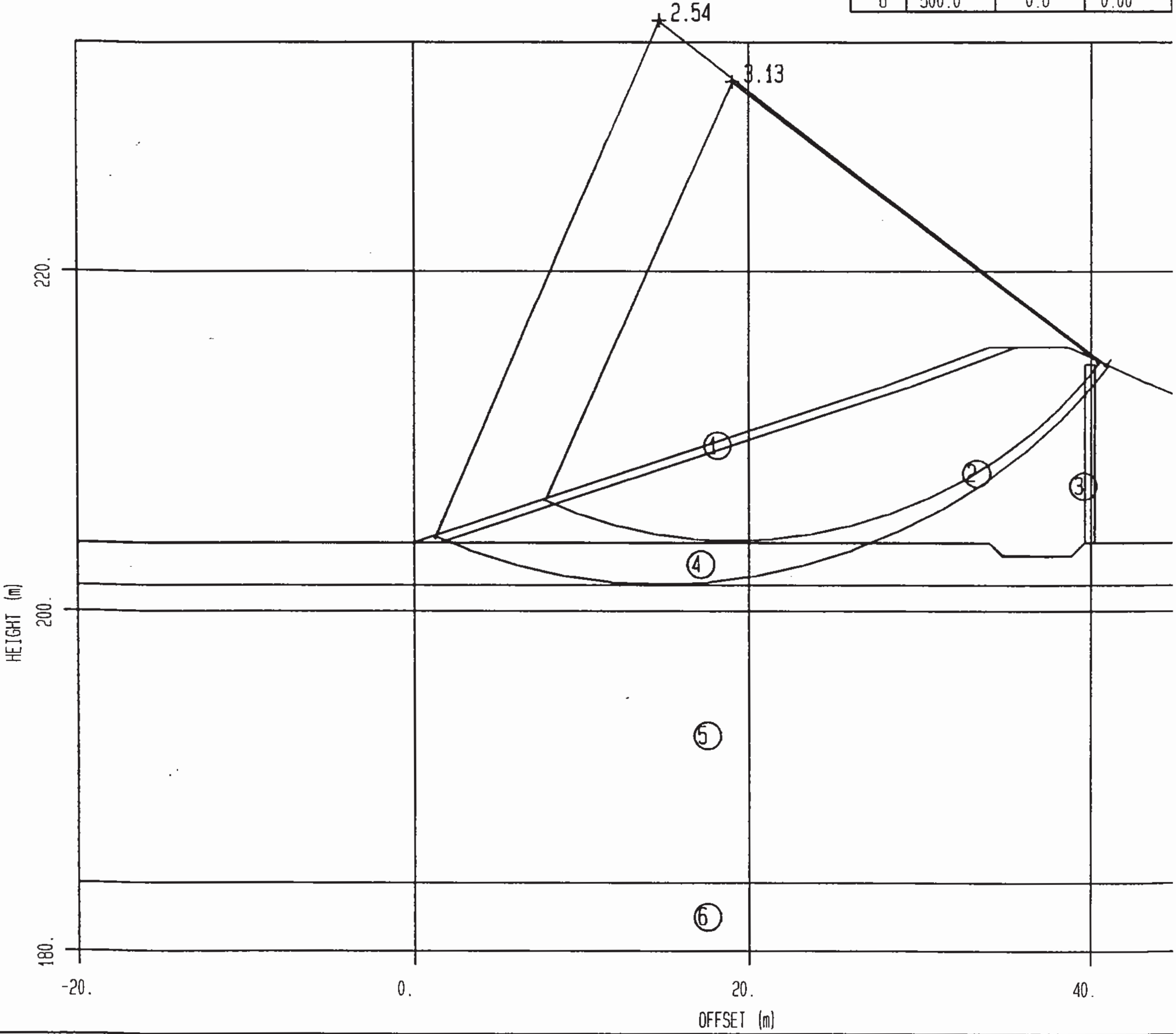
C5863/1-AC

 T89 252 M01 COFSTAB VEF

Upstream Face Stablity
End of Construction

LAYER NO.	COHESION (kPa)	FRICTION ANGLE (deg)	PORE PRESSURE CODE
1	0.0	40.0	0.00
2	75.0	0.0	0.00
3	0.0	30.0	0.00
4	75.0	0.0	0.00
5	100.0	0.0	0.00
6	500.0	0.0	0.00

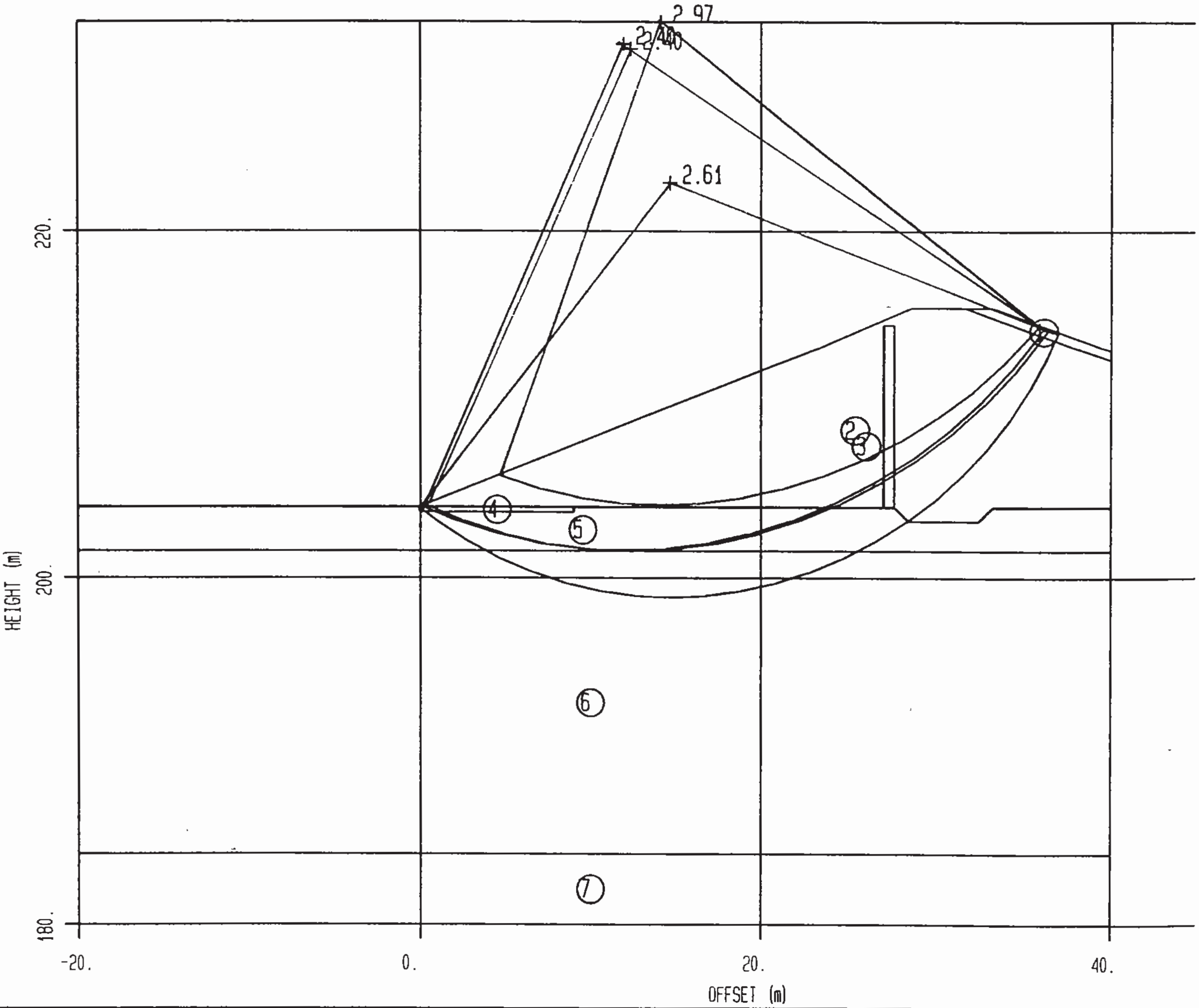
THE ANALYSIS METHOD IS SIMPLIFIED BISHOP



Downstream Face Stability
End of Construction

LAYER NO.	COHESION (kPa)	FRICTION ANGLE (deg)	PORE PRESSURE CODE
1	0.0	40.0	0.00
2	75.0	0.0	0.00
3	0.0	30.0	0.00
4	0.0	30.0	0.00
5	75.0	0.0	0.00
6	100.0	0.0	0.00
7	500.0	0.0	0.00

THE ANALYSIS METHOD IS SIMPLIFIED BISHOP

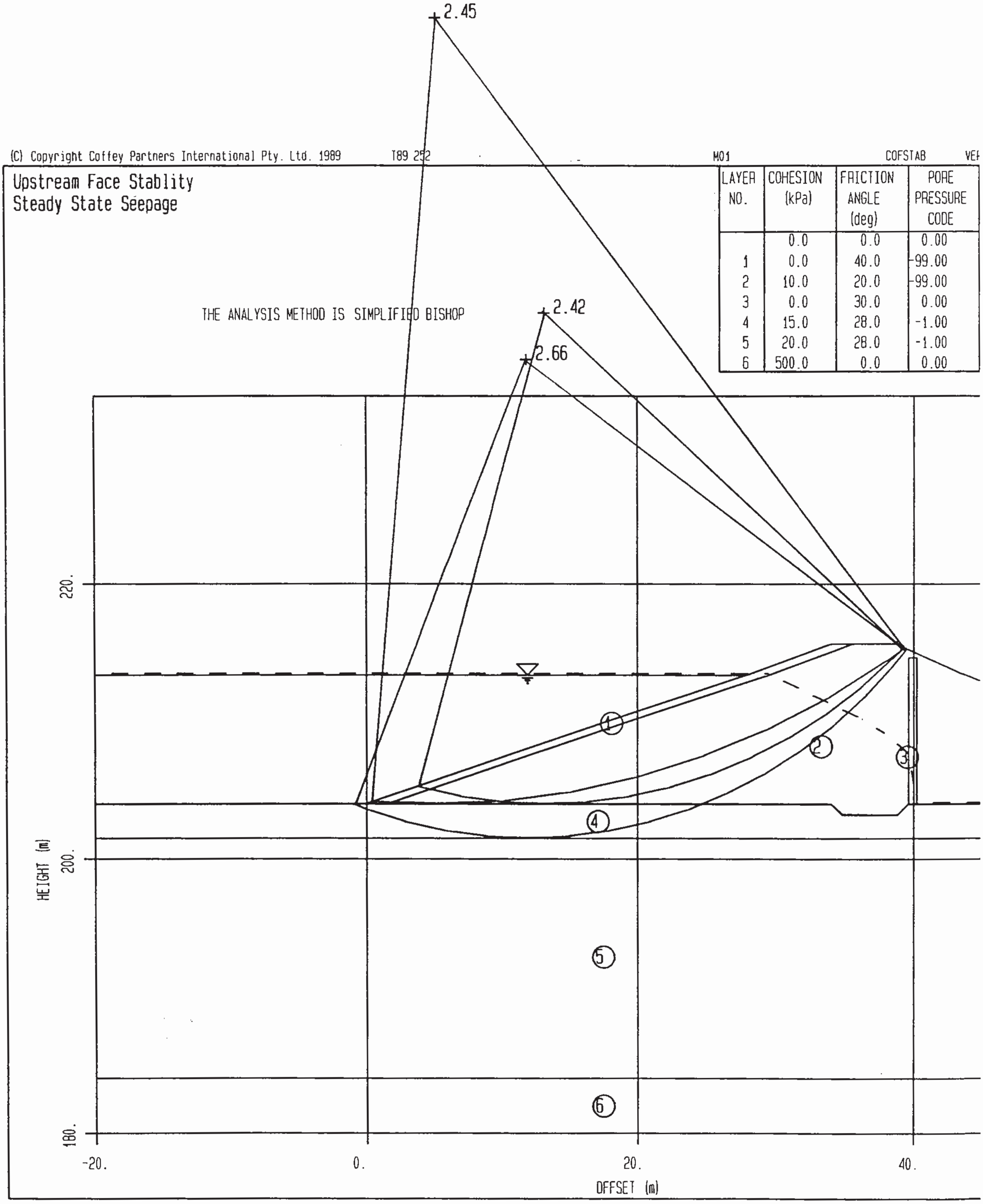


Coffey Partners International Pty Ltd

C5863/1-AC

 T89 252 M01 COFSTAB VEI

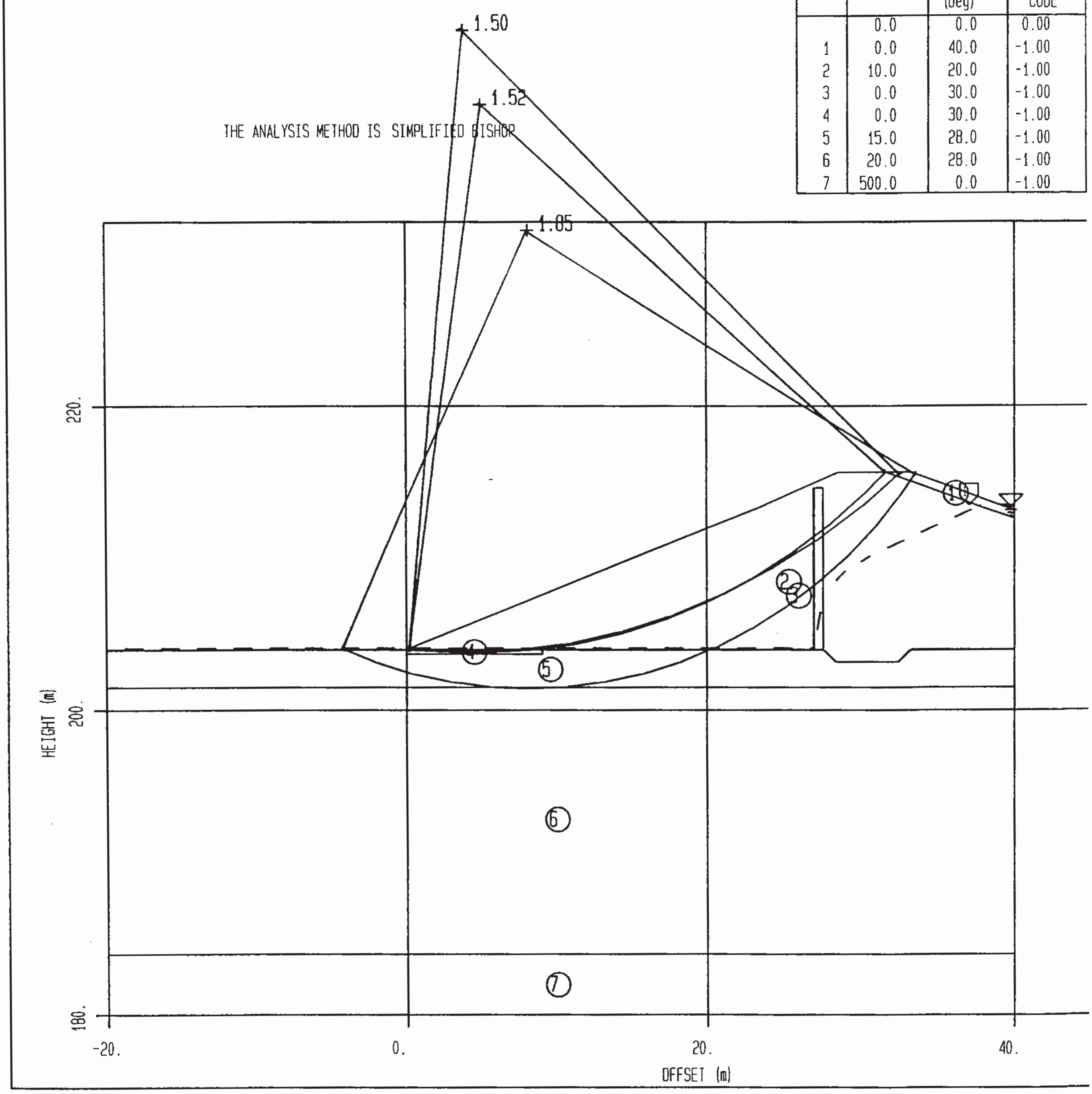
LAYER NO.	COHESION (kPa)	FRICTION ANGLE (deg)	PORE PRESSURE CODE
	0.0	0.0	0.00
1	0.0	40.0	-99.00
2	10.0	20.0	-99.00
3	0.0	30.0	0.00
4	15.0	28.0	-1.00
5	20.0	28.0	-1.00
6	500.0	0.0	0.00

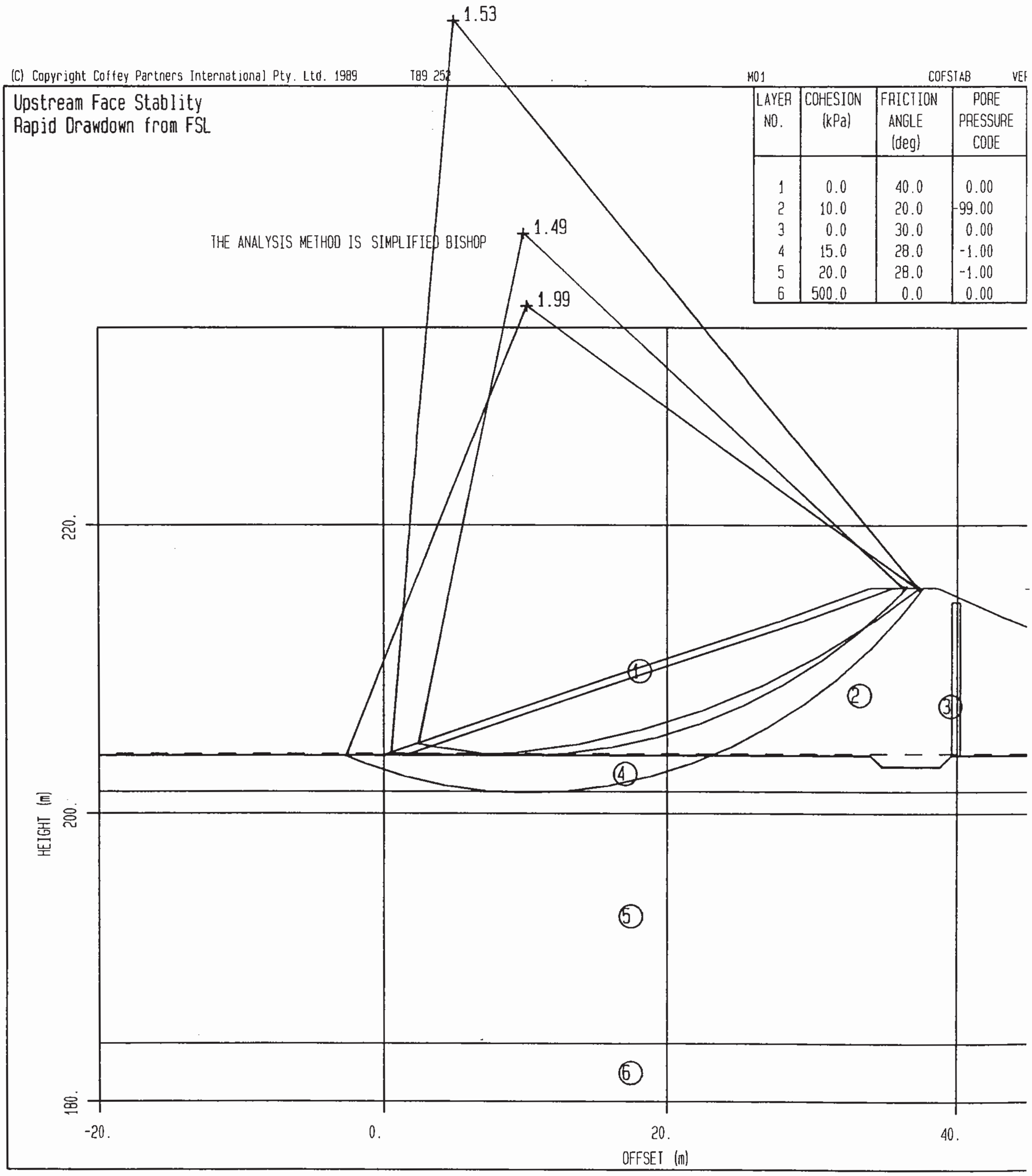


 T89 252 M01 COFSTAB VEF

Downstream Face Stability
Steady State Seepage

LAYER NO.	COHESION (kPa)	FRICTION ANGLE (deg)	PORE PRESSURE CODE
	0.0	0.0	0.00
1	0.0	40.0	-1.00
2	10.0	20.0	-1.00
3	0.0	30.0	-1.00
4	0.0	30.0	-1.00
5	15.0	28.0	-1.00
6	20.0	28.0	-1.00
7	500.0	0.0	-1.00





LAYER NO.	COHESION (kPa)	FRICTION ANGLE (deg)	PORE PRESSURE CODE
1	0.0	40.0	0.00
2	10.0	20.0	-99.00
3	0.0	30.0	0.00
4	15.0	28.0	-1.00
5	20.0	28.0	-1.00
6	500.0	0.0	0.00

Coffey Partners International Pty Ltd

C5863/1-AC

Upstream Face Stablity
Steady State Seepage + Earthquake

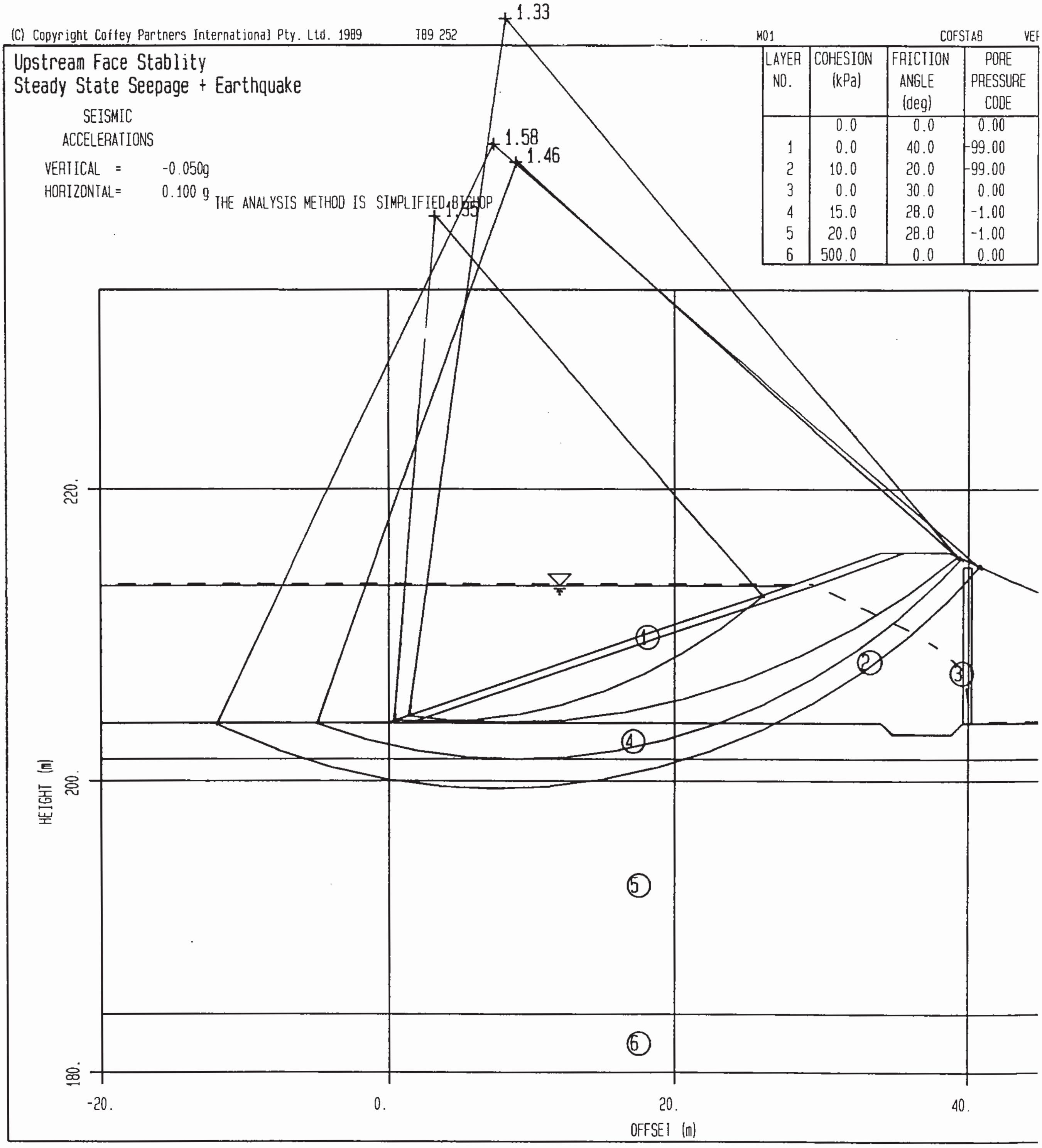
SEISMIC ACCELERATIONS

VERTICAL = -0.050g

HORIZONTAL= 0.100 g

THE ANALYSIS METHOD IS SIMPLIFIED BISHOP

LAYER NO.	COHESION (kPa)	FRICTION ANGLE (deg)	PORE PRESSURE CODE
	0.0	0.0	0.00
1	0.0	40.0	-99.00
2	10.0	20.0	-99.00
3	0.0	30.0	0.00
4	15.0	28.0	-1.00
5	20.0	28.0	-1.00
6	500.0	0.0	0.00



Coffey Partners International Pty Ltd

C5863/1-AC

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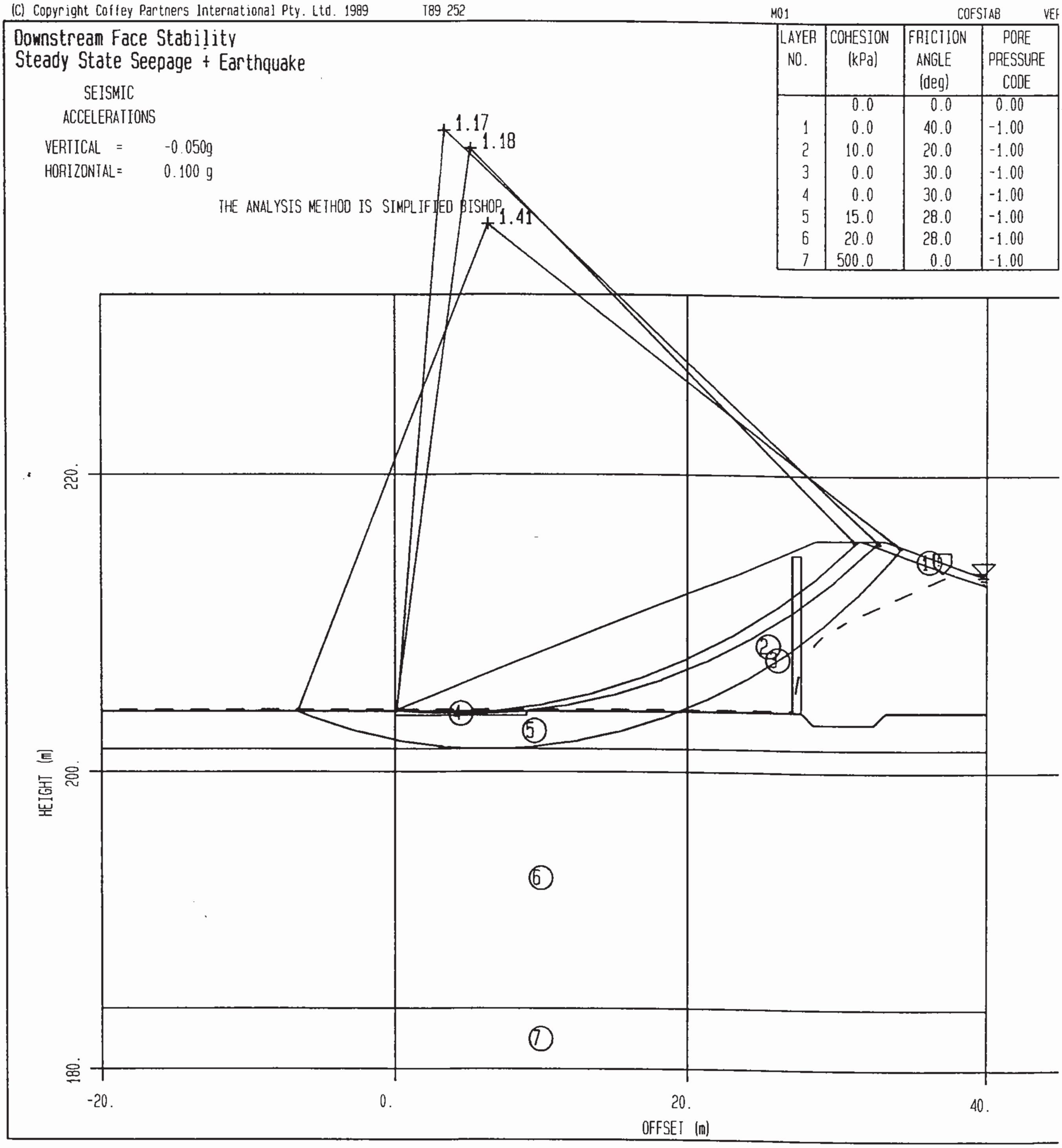


LAYER NO.	COHESION (kPa)	FRICTION ANGLE (deg)	PORE PRESSURE CODE
	0.0	0.0	0.00
1	0.0	40.0	-1.00
2	10.0	20.0	-1.00
3	0.0	30.0	-1.00
4	0.0	30.0	-1.00
5	15.0	28.0	-1.00
6	20.0	28.0	-1.00
7	500.0	0.0	-1.00

C

Appendix C Failure Modes Screening

Appendix C - Failure Modes Screening

Potential Failure Mode		Included/ Excluded	Summary Comments
P1	Piping through embankment	Included	• Zone 1 material Emerson Class 3 upstream of chimney filter and as low as 2 downstream of filter. • Zone 1 material compacted in 200 mm lifts to 98% standard compaction at 1% dry to 2% wet of optimum moisture content. Good documentation showing general conformance to spec (exclude initiating mechanism of poorly compacted layer). • Minimum Liquid Limit of 30% required in specification. Test results on borrow material indicated Liquid Limits in the range of 46% and 74%, but generally less than 66% (Willing and Partners 1993). • Zone material classified as Sandy Clay to Clay soils of medium to high plasticity • 2A filter provided from base of cut-off foundation to RL 214.65m (0.7m below dam crest) 2.5 m downstream of the embankment centreline. Minimum thickness of 2A filter was 0.6 m. • FSL at RL 213.30 m and PMF reservoir level RL 214.30 m. • Minimum Operating Level (MOL) at RL 206.7 m (invert of pipe outlet structure). • Outlet capacity provided through 750 mm diameter pipeline with upstream and downstream valves. Maximum velocity was designed to be 0.5 m/s (Willing and Partners, 1993), translating to a discharge of approximately 0.44 m3/s. Estimated time to drawdown reservoir from FSL to MOL = 54 days (assuming no additional inflows). • PSD conducted during construction on Zone 2A indicated high fines content of typically between 6-7%. Specification called for a maximum of 5% passing the 0.075mm sieve. • Filters compacted in 300 mm lifts using vibrating plate compactor. • Gentle abutment slopes. • Gravel capping on crest. • No trees present in vicinity of embankment, exclude piping through tree roots • Inspections twice weekly • Width of valley ~ 150m, dam height 13m – exclude cross valley arching mechanism as W/H >2 **Included for event tree analysis as ETT-F1**
P2	Piping along outlet conduit	Included	• Pipe placed in 1.5 m wide trench beneath stripped foundation surface and near the maximum embankment section. Trench backfilled with concrete. • Three concrete cut-off collars constructed at 5 m spacing near the embankment centreline. • 1.0 m wide Zone 2A filter diaphragm was provided downstream of dam centreline and concrete cut-off collars (same alignment as chimney filter). Filter diaphragm extended 2.5 m below concrete encasement and 3.0 m either side of concrete encasement. A Zone 2A drainage outlet was also provided downstream of the filter diaphragm. **Included for event tree analysis as ETT-F2**
P3	Piping adjacent to spillway walls	Excluded	• No embankment/ spillway interface • Spillway excavated through Eastern ridge, approximately 120 m from the left abutment **Excluded from further analysis as mechanism is not feasible**

Appendix C - Failure Modes Screening

Potential Failure Mode		Included/ Excluded	Summary Comments
P4	Piping through foundation	Included	• Borrow within reservoir. 50 m exclusion zone from upstream toe of embankment. • Surficial sandy silt material stripped. • Cut-off understood to be taken to residual soil (based on conversations with Richard Rodd). Residual soil described as clayey sand or sandy clay or low to medium plasticity. • Foundation materials (overlying residual soil) typically comprised of sandy, gravelly clay of alluvial/ colluvial origin. Foundation materials typically more dispersive at depths greater than 1.5 m. • Design drawings show chimney filter taken to base of cut-off trench. • No foundation grouting undertaken. • Piezometers installed downstream of embankment show pore water pressure approaching surface level during period of high reservoir level. **Included for event tree analysis as ETT-F3**
P5	Piping from embankment into foundation	Excluded	• Foundation material typically classified as same material as embankment fill i.e. sandy clay to clay. **Excluded from further analysis due to very low likelihood**

Appendix D Event Trees

LAKE ETTAMOGAH WINTER STORAGE DAM

EVENT TREE ANALYSIS

13 September 2013

Potential Failure Modes and Initiating Events

Structure: LAKE ETTAMOGAH WINTER STORAGE DAM

Potential Failure Modes		Initiating Event
ETT-F1	Piping through embankment	Normal & EQ
ETT-F2	Piping along outlet conduit	Normal
ETT-F3	Piping through foundation	Normal

Event Tree- Output Table

Structure: LAKE ETTAMOGAH WINTER STORAGE DAM

Potential Failure Modes		Initiating Event	Annual Failure Probability
ETT-F1	Piping through embankment	Normal & EQ	6.3E-07
ETT-F2	Piping along outlet conduit	Normal	3.1E-08
ETT-F3	Piping through foundation	Normal	4.5E-07
			1.1E-06

Cross check with Sum sheet **Match**

Potential Failure Modes Summary

LAKE ETTAMOGAH WINTER STORAGE DAM

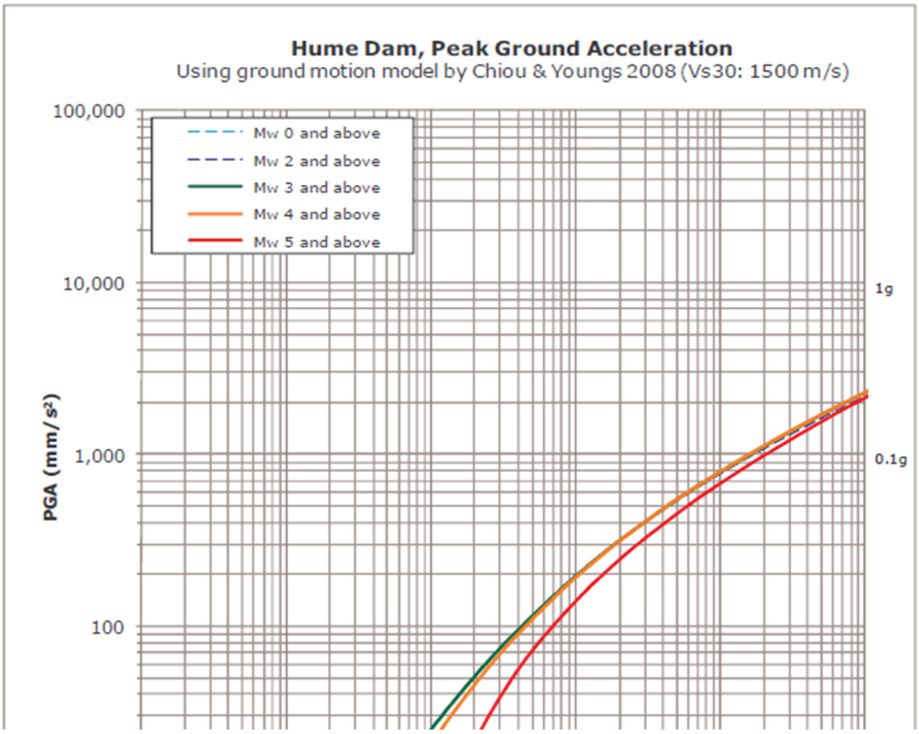
	FSL	EQ Loading			All Loadir
	Normal	EQ1	EQ2	EQ3	
	0.00 to 0.045g	0.045g to 0.13	0.13 to 0.22g	> 0.22g	Total Annual Probability of Failure
Failure Mode					
ETT-F1 Piping through embankment - Normal & EQ	**6.3E-07**	**1.1E-07**	**1.1E-07**	**1.1E-05**	**6.3E-07**
ETT-F1a Mechanism - Cross valley differential settlement	1.3E-09	1.1E-07	1.1E-07	1.1E-05	2.6E-09
ETT-F1b Mechanism - Desiccation cracking	6.3E-07				6.2E-07
ETT-F2 Piping along outlet conduit - Normal	**3.2E-08**				**3.1E-08**
ETT-F3 Piping through foundation - Normal	**4.5E-07**				**4.5E-07**
Annual Probability of Loading Event	1.0E+00	1.6E-03	3.0E-04	1.0E-04	
					1.1E-06

Earthquake Partitions

Structure: LAKE ETTAMOGAH WINTER STORAGE DAM

Earthquake Event		PGA (mAHD)	Description	AEP	Annual Probability of Reaching the Partition
		0.00	Reservoir Level RL 213.30m (FSL)	1.00E+00	
Normal	0.00 to 0.045g				9.98E-01
		0.045g	OBE (1 in 500 AEP)	2.00E-03	
EQ1	0.045g to 0.13				1.60E-03
		0.13	1 in 2500	4.00E-04	
EQ2	0.13 to 0.22g				3.00E-04
		0.22g	MDE (1 in 10,000 AEP)	1.00E-04	
EQ3	> 0.22g				1.00E-04
				TOTAL	1.00E+00

Ref Adopted to be the same as for Hume Dam seismic assessment (SRC 2010)



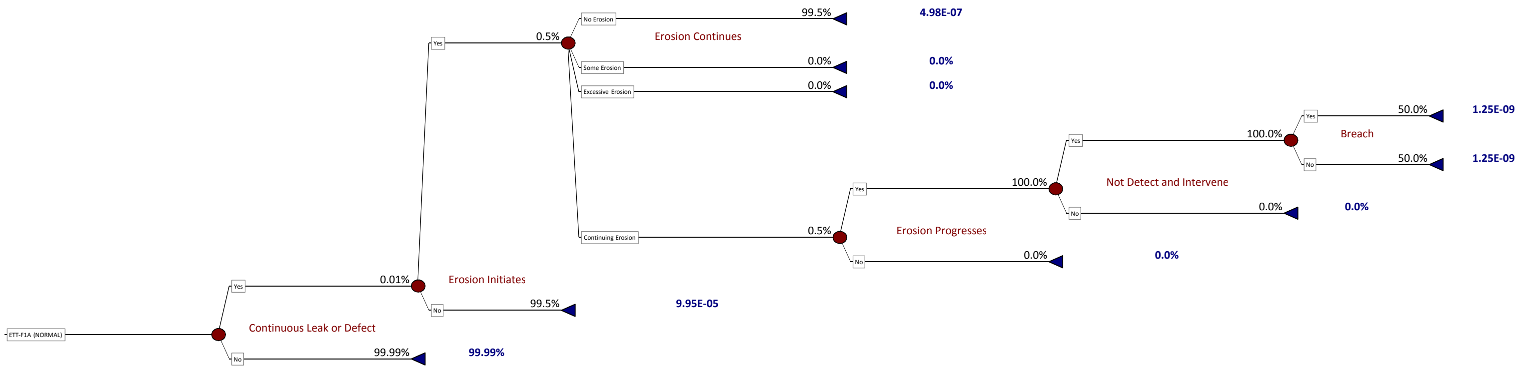
Failure Mode For

ETT-F1	**Piping through embankment**	**Normal & EQ**
ETT-F1a	**Mechanism**	**Cross valley differential settlement**

Probability Table

Partition		Continuous leak or defect?	Erosion initiates	Erosion Continues		Erosion Progresses	Not Detect and Intervene	Breach	Conditional Probability
Normal	0.00 to 0.045g	0.0001	0.005	No Erosion	0.995	0	0	0	0.00E+00
		0.0001	0.005	Some Erosion	0	0	0	0	0.00E+00
		0.0001	0.005	Excessive Erosion	0	0	0	0	0.00E+00
		0.0001	0.005	Continuing Erosion	0.005	1	1	0.5	1.25E-09
EQ1	0.045g to 0.13	0.01	0.005	No Erosion	0.995	0	0	0	0.00E+00
		0.01	0.005	Some Erosion	0	0	0	0	0.00E+00
		0.01	0.005	Excessive Erosion	0	0	0	0	0.00E+00
		0.01	0.005	Continuing Erosion	0.005	1	0.9	0.5	1.13E-07
EQ2	0.13 to 0.22g	0.01	0.005	No Erosion	0.995	0	0	0	0.00E+00
		0.01	0.005	Some Erosion	0	0	0	0	0.00E+00
		0.01	0.005	Excessive Erosion	0	0	0	0	0.00E+00
		0.01	0.005	Continuing Erosion	0.005	1	0.9	0.5	1.13E-07
EQ3	> 0.22g	0.05	0.1	No Erosion	0.995	0	0	0	0.00E+00
		0.05	0.1	Some Erosion	0	0	0	0	0.00E+00
		0.05	0.1	Excessive Erosion	0	0	0	0	0.00E+00
		0.05	0.1	Continuing Erosion	0.005	1	0.9	0.5	1.13E-05

Event Tree



Probability Table References.

Continuous leak or defect?	Erosion initiates	Erosion Continues	Erosion Progresses	Not Detect and Intervene	Breach
Uniform and gentle abutment profile	table 5.24 crack at crest = 1mm	Filter exhibits high fines content 6-7% and compacted (assume non-plastic). Table 10.2 adopt Pr continuing erosion = 0.005	Table 11.1 homogenous embankment of clay to sand clay. Will hold a roof. Pr = 1	Table 12.2 CL-CH, low gradient - rapid erosion	Table 13.3. Defect in upper part of embankment, small storage. Assume reservoir falls below invert of pipe quickly (<1 day). Adopt Pr =0.5
Slopes less than 30 degrees	assume 1mm crack at FSL also		Table 11.2 homogeneous earthfill no crack filling action. Pr = 1	Table 12.3 Likely breach time medium - rapid	
dam less than 15m (13m)	Gradient <0.1		Table 11.3 no flow limitation	Table 12.1 Likely breach time 12-24hrs	
Table 5.1	Table 5.33 adopt Pr = 0.005			Inspections twice weekly. Assume Pr = 0.25 of detecting piping incident i.e. 0.75 that won't	
(3*1)+(2*1)+(1*1) = 6	EQ1 and 2 Table 5.39 maximum crack width = 20mm Table 5.25 assume 1mm at FSL Table 5.33 adopt Pr = 0.005			Limited drawdown capacity. Table 12.8 adopt Pr = 0.9 that not able to intervene if detected	
Assume above POR, Table 5.2 pr = 0.0001	EQ3 Table 5.39 maximum crack width = 50mm Table 5.25 assume 10mm at FSL Table 5.33 adopte Pr = 0.1			Pr = 0.75+0.25*0.9 = 0.975 (adopt Pr = 1)	
From Figure 5.8, EQ1 and 2 are damage class 1 (table 5.39 pr = 0.01)				Assume will conduct detailed inspection following earthquake (pr detect = 1)	
From Figure 5.8, EQ3 is damage class 2 (table 5.39 pr = 0.05)					

URS

Failure Mode For

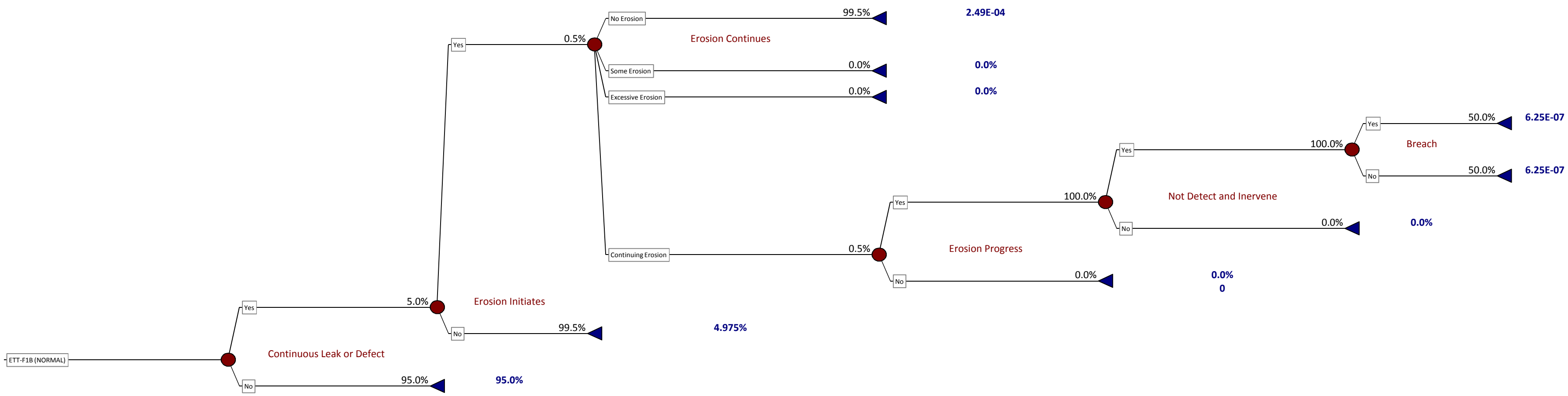
ETT-F1 Piping through embankment Normal & EQ

ETT-F1b Mechanism Desiccation cracking

Probability Table

Partition	Continuous leak or defect?	Erosion initiates	Erosion Continues		Erosion Progresses	Not Detect and Intervene	Breach	Conditional Probability
Normal 0.00 to 0.045g	0.05	0.005	No Erosion	0.995	0	0	0	0.00E+00
	0.05	0.005	Some Erosion	0	0	0	0	0.00E+00
	0.05	0.005	Excessive Erosion	0	0	0	0	0.00E+00
	0.05	0.005	Continuing Erosion	0.005	1	1	0.5	6.25E-07

Event Tree



Probability Table References.

Continuous leak or defect?	Erosion initiates	Erosion Continues	Erosion Progresses	Not Detect and Intervene	Breach
Table 5.11	Table 5.24 - crack width = 20mm @ crest. No cracks observed during inspection however, cracks could be obscured by gravel capping	Filter exhibits high fines content 6-7% and compacted (assume non-plastic). Table 10.2 adopt Pr continuing erosion = 0.005	Table 11.1 homogenous embankment of clay to sand clay. Will hold a roof. Pr = 1	Table 12.2 CL-CH, low gradient - rapid erosion	Table 13.3. Defect in upper part of embankment, small storage. Assume reservoir falls below invert of pipe quickly (<1 day). Adopt Pr =0.5
gravel surface assume to be at least 150mm thick (2)	Table 5.25 crack width at FSL 1-2mm		Table 11.2 homogeneous earthfill no crack filling action. Pr = 1	Table 12.3 Likely breach time medium - rapid	
seasonal climate (2)	Gradient <0.1		Table 11.3 no flow limitation	Table 12.1 Likely breach time 12-24hrs	
medium to high plasticity earthfill (LL 65) (3)	Table 5.33 adopt Pr = 0.005			Inspections twice weekly. Assume Pr = 0.25 of detecting piping incident i.e. 0.75 that won't	
RF*LF = (3*2)+(2*2)+(1*3) = 13				Limited drawdown capacity. Table 12.8 adopt Pr = 0.9 that not able to intervene if detected	
Pr = 0.05 Table 5.12				Pr = 0.75+0.25*0.9 = 0.975 (adopt Pr = 1)	
From table 5.13 maximum likely depth = 3m				Assume will conduct detailed inspection following earthquake (pr detect = 1)	

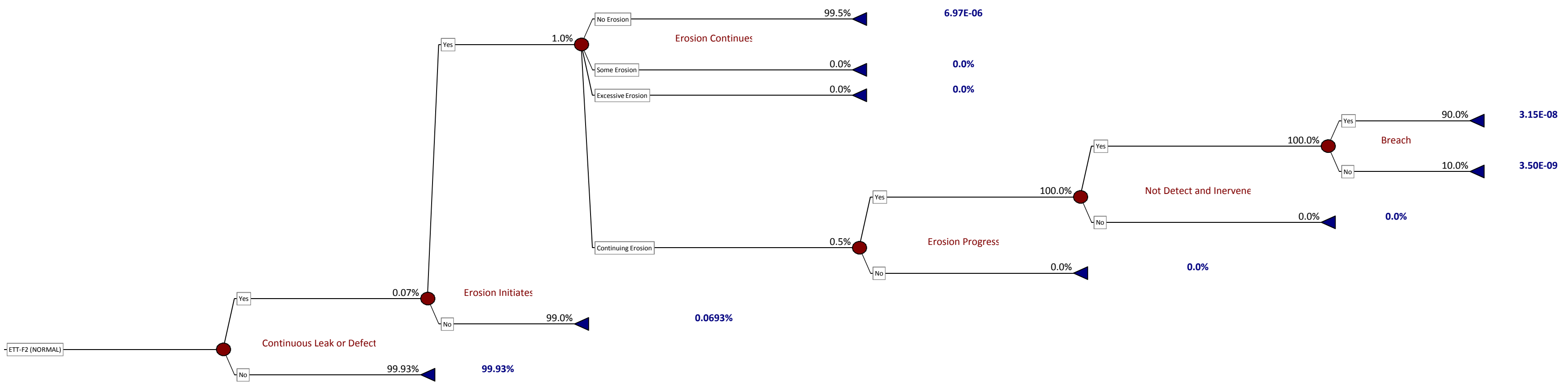
Failure Mode For

ETT-F2 Piping along outlet conduit Normal

Probability Table

Partition		Continuous leak or defect?	Erosion initiates	Erosion Continues		Erosion Progresses	Not Detect and Intervene	Breach	Conditional Probability
Normal	0.00 to 0.045g	0.0007	0.01	No Erosion	0.995	0	0	0	0.00E+00
		0.0007	0.01	Some Erosion	0	0	0	0	0.00E+00
		0.0007	0.01	Excessive Erosion	0	0	0	0	0.00E+00
		0.0007	0.01	Continuing Erosion	0.005	1	1	0.9	3.15E-08

Event Tree



Probability Table References.

Continuous leak or defect?	Erosion initiates	Erosion Continues	Erosion Progresses	Not Detect and Intervene	Breach
Table 6.11	Table 6.29	Filter exhibits high fines content 6-7% and compacted (assume non-plastic). Table 10.2 adopt Pr continuing erosion = 0.005	Table 11.1 homogenous embankment of clay to sand clay. Will hold a roof. Pr = 1	Table 12.2 CL-CH, low gradient - rapid erosion	Table 13.3. Defect in lower part of embankment. Assume reservoir will not fall below pipe. Adopt Pr =0.9
concrete encased pipe with vertical sides (2)	spec called for 200mm lifts however assume 300mm was done around conduit.	Well detailed filter diaphragm around conduit	Table 11.2 homogeneous earthfill no crack filling action. Pr = 1	Table 12.3 Likely breach time medium - rapid	
3 cutoff collars at 5m spacing. Cutoff collars shown to not extend above top of encasement so likely reasonable compaction achieved. Also as cutoff collars only in centre part of dam, could likely to have good compaction elsewhere (2)	Good compaction records. Compacted to 98% standard max at 1% dry to 2% wet of optimum moisture content		Table 11.3 no flow limitation	Table 12.1 Likely breach time 12-24hrs	
compacted to 98% standard max at 1% dry to 2% wet of optimum moisture content (1)	adopt factor of 0.005			Inspections twice weekly. Assume Pr = 0.25 of detecting piping incident i.e. 0.75 that won't	
trench width 1.5m (conduit 750mm) (3)	therefore defect size 1-2mm			Limited drawdown capacity. Table 12.8 adopt Pr = 0.9 that not able to intervene if detected	
RF*LF = (3*2)+(2*2)+(2*1)+(2*3) = 18	Table 5.33			Pr = 0.75+0.25*0.9 = 0.975 (adopt Pr = 1)	
Table 6.12 below POR Pr = 0.0007	Pipe IL 204.0m (9.3m head at FSL)			Assume will conduct detailed inspection following earthquake (pr detect = 1)	
	length = 65.4				
	gradient = 0.14				
	adopt Pr = 0.01				

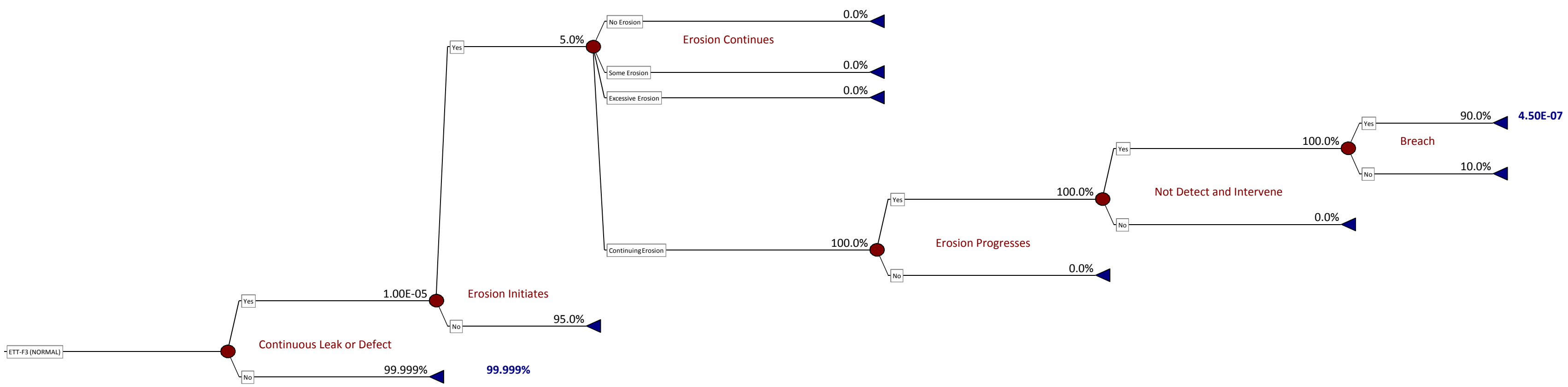
Failure Mode For

ETT-F3 Piping through foundation Normal

Probability Table

Partition		Continuous leak or defect?	Erosion initiates	Erosion Continues		Erosion Progresses	Not Detect and Intervene	Breach	Conditional Probability
Normal	0.00 to 0.045g	0.00001	0.05	No Erosion	C	0	0	0	0.00E+00
		0.00001	0.05	Some Erosion	C	0	0	0	0.00E+00
		0.00001	0.05	Excessive Erosion	C	0	0	0	0.00E+00
		0.00001	0.05	Continuing Erosion	1	1	1	0.9	4.50E-07

Event Tree



Probability Table References.

Continuous leak or defect?	Erosion initiates	Erosion Continues	Erosion Progresses	Not Detect and Intervene	Breach
Cutoff understood to be taken to residual soil	assume 1-2mm defect	No filter trench or blanket	Table 11.1 homogenous embankment of clay to sand clay. Will hold a roof. Pr = 1	Table 12.2 CL-CH, low gradient - rapid erosion	Table 13.3. Defect in foundation. Adopt Pr =0.9
erosion through cutoff considered in erosion through embankment as chimney filter shown in drawings to extend full depth of cutoff.	Residual soil described as sandy clay or clayey sand of low to medium plasticity. Assume foundation is dispersive based on Coffey 1993 encountering more dispersive material beyond 1.5m		Table 11.2 homogeneous earthfill no crack filling action. Pr = 1	Table 12.3 Likely breach time medium - rapid	
erosion through foundation is therefore considered through residual soil described as clayey sand or sandy clay of low to medium plasticity	Table 5.35		Table 11.3 no flow limitation	Table 12.1 Likely breach time 12-24hrs	
Require approx 70m continuous defect in residual soil and daylighting through aluviual layers and daylighting at the embankment toe. i.e. requires a defect in the alluvium/colluvium to line up with a continuous defect in the residual soil.	gradient = 13/70 = 0.18			Inspections twice weekly. Assume Pr = 0.25 of detecting piping incident i.e. 0.75 that won't	
Adopt probability of continuous defect in shallow alluvium/colluvium = 0.01 adopt probability of continuous defect in residual soil of 0.001 Probability of continuous defect daylighting from U/S to D/S = 0.01*0.001 = 1E-05	Adopt pr = 0.05			Limited drawdown capacity. Table 12.8 adopt Pr = 0.9 that not able to intervene if detected	
				Pr = 0.75+0.25*0.9 = 0.975 (adopt Pr = 1)	
				Assume will conduct detailed inspection following earthquake (pr detect = 1)	

E

Appendix E Stage-Storage Relationship

Appendix E - Stage-Storage Relationship

Lake Ettamogah Winter Storage Dam	
Stage (m AHD)	Storage (ML)
205	0.0
205.1	1.4
205.2	3.0
205.3	4.9
205.4	7.0
205.5	9.3
205.6	11.8
205.7	14.5
205.8	17.5
205.9	20.7
206	24.1
206.1	27.8
206.2	31.7
206.3	35.9
206.4	40.3
206.5	44.9
206.6	49.8
206.7	54.9
206.8	60.3
206.9	65.9
207	71.8
207.1	78.0
207.2	84.4
207.3	91.0
207.4	98.0
207.5	105.1
207.6	114.6
207.7	124.7
207.8	135.3
207.9	146.5
208	158.2
208.1	170.5
208.2	183.4
208.3	196.8
208.4	210.8
208.5	225.4
208.6	240.6
208.7	256.4
208.8	272.8
208.9	289.7
209	307.3
209.1	325.4
209.2	344.2
209.3	363.6
209.4	383.6
209.5	404.2
209.6	425.4
209.7	447.2
209.8	469.7
209.9	492.8
210	516.5
210.1	545.4
210.2	575.5
210.3	606.7
210.4	639.0
210.5	672.5
Stage (m AHD)	Storage (ML)
210.6	707.2
210.7	743.0
210.8	780.0
210.9	818.1
211	857.3
211.1	897.7
211.2	939.2
211.3	981.9
211.4	1025.7
211.5	1070.6
211.6	1116.7
211.7	1163.9
211.8	1212.2
211.9	1261.7
212	1312.2
212.1	1363.9
212.2	1416.8
212.3	1470.7
212.4	1525.8
212.5	1582.0
212.6	1641.3
212.7	1702.1
212.8	1764.4
212.9	1828.2
213	1893.5
213.1	1960.5
213.2	2029.3
213.3	2100.0
213.4	2173.2
213.5	2247.2
213.6	2322.4
213.7	2398.7
213.8	2476.4
213.9	2555.5
214	2636.0
214.1	2718.0
214.2	2801.4
214.3	2886.4
214.4	2972.8
214.5	3060.6
214.6	3149.9
214.7	3240.8
214.8	3333.1
214.9	3426.9
215	3522.0
215.1	3618.6
215.2	3716.6
215.3	3816.2
215.4	3917.3
215.5	4020.0
215.6	4124.3
215.7	4230.0
215.8	4336.9
215.9	4445.1
216	4554.5

F

Appendix F Stage-Discharge Relationship

Appendix F - Stage-Discharge Relationship

Lake Ettamogah Winter Storage Dam	
Stage (m AHD)	Discharge (m3/s)
213.3	0
213.4	2.7
213.5	7.7
213.6	16.5
213.7	28.4
213.8	43.0
213.9	59.7
214	79.7
214.1	101.5
214.2	125.7
214.3	153.2
214.4	181.0
214.5	210.8
214.6	243.5
214.7	278.6
214.8	315.4
214.9	354.2
215	395.8
215.1	437.6
215.2	479.4
215.3	525.1
215.4	581.6
215.5	775.2
215.6	902.8
215.7	1094.5
215.8	1399.3
215.9	1613.3
216	1846.9

Appendix G Inundation Maps

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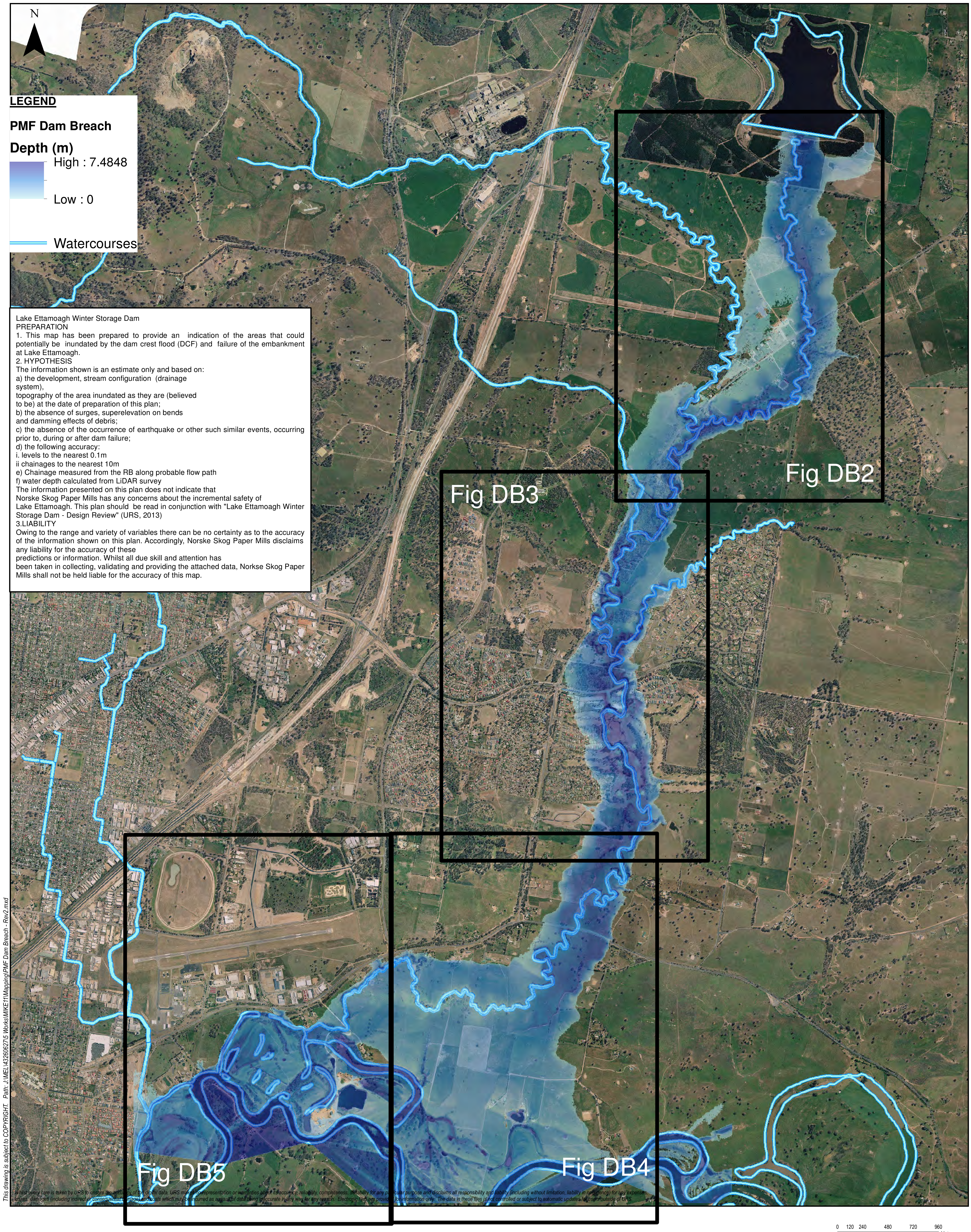

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LAKE ETTAMOGAH WINTER STORAGE DAM
DESIGN REVIEW

PMF DAM BREACH

URS

Figure: **DB1**

File No: **PMF Dam Breach - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013** Rev. **B** A3

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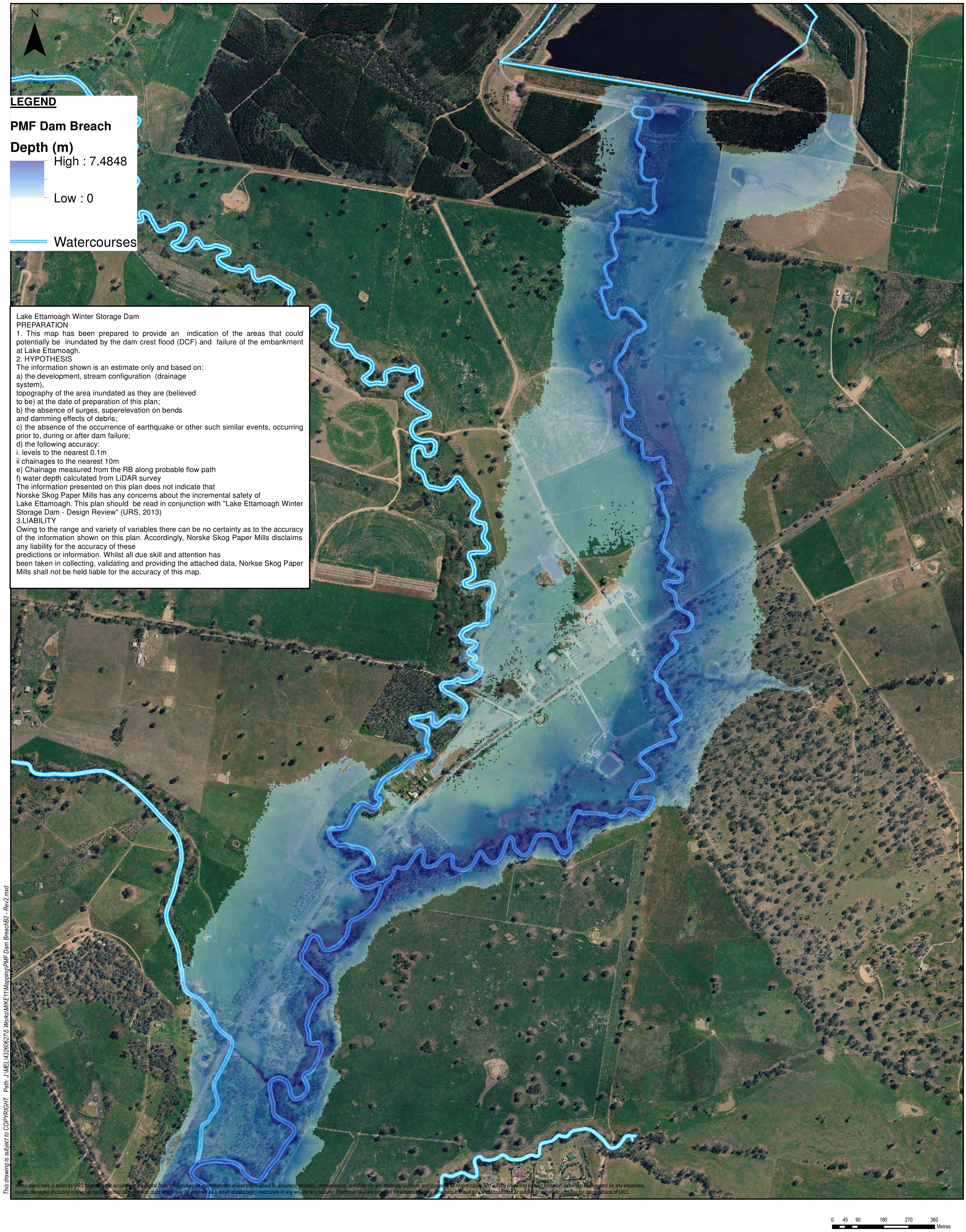

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PMF DAM BREACH
DB2

URS

Figure: **DB2**

File No: **PMF Dam BreachB2 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Rev. **B** A3

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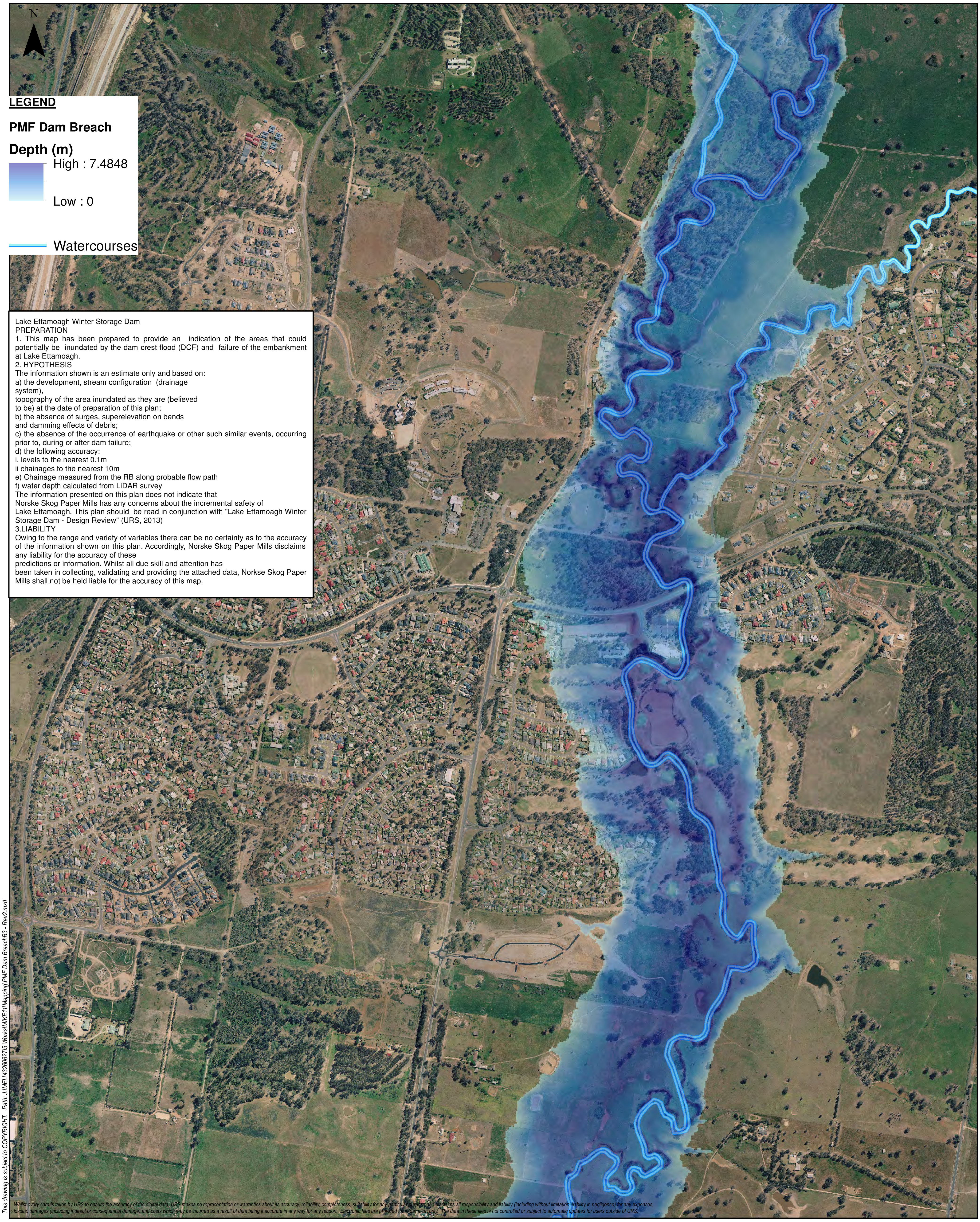

0 45 90 180 270 360
Metres

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PMF DAM BREACH
DB3

URS

Figure: DB3

File No:**PMF Dam BreachB3 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Rev. **B** A3

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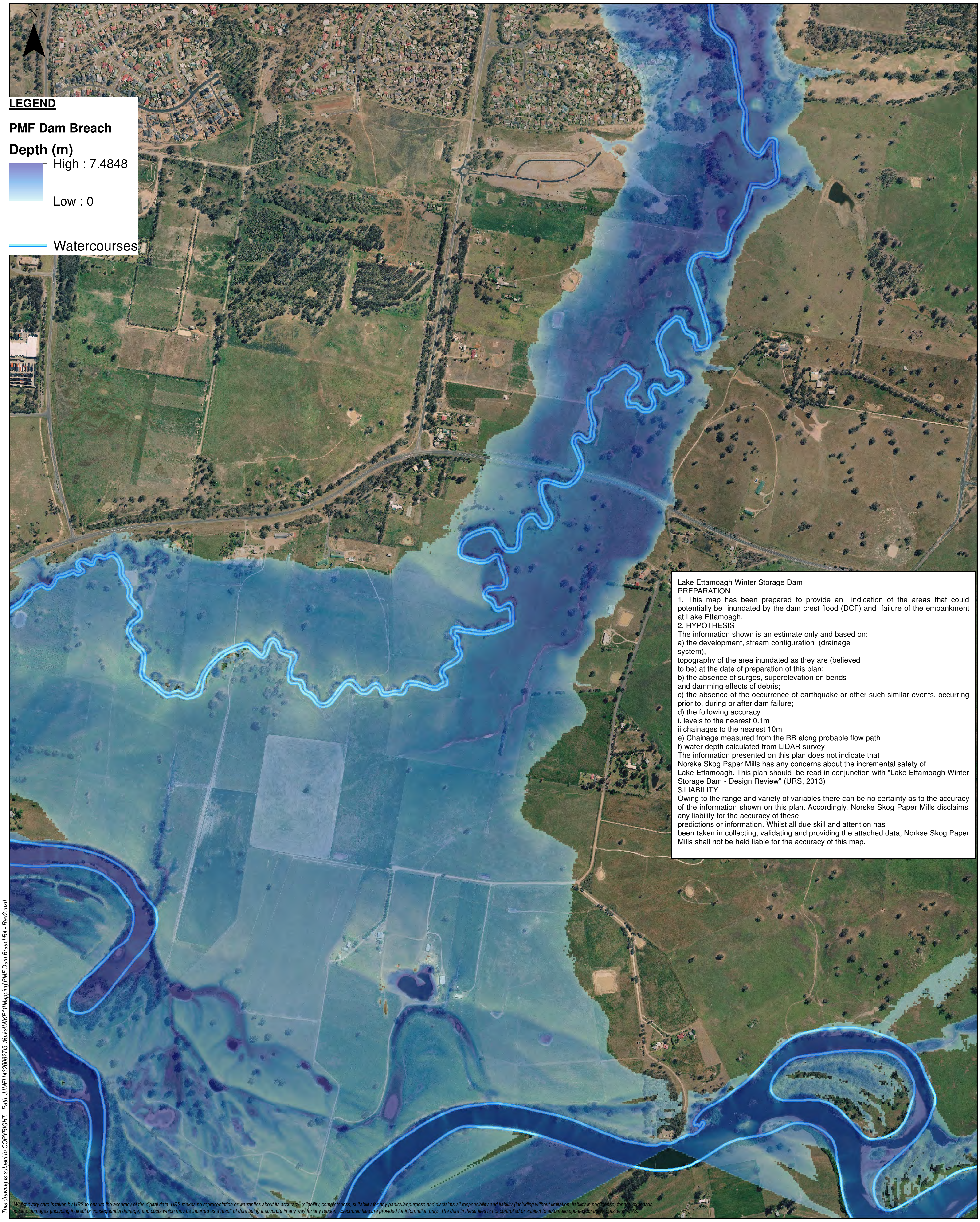

0 45 90 180 270 360 Metres

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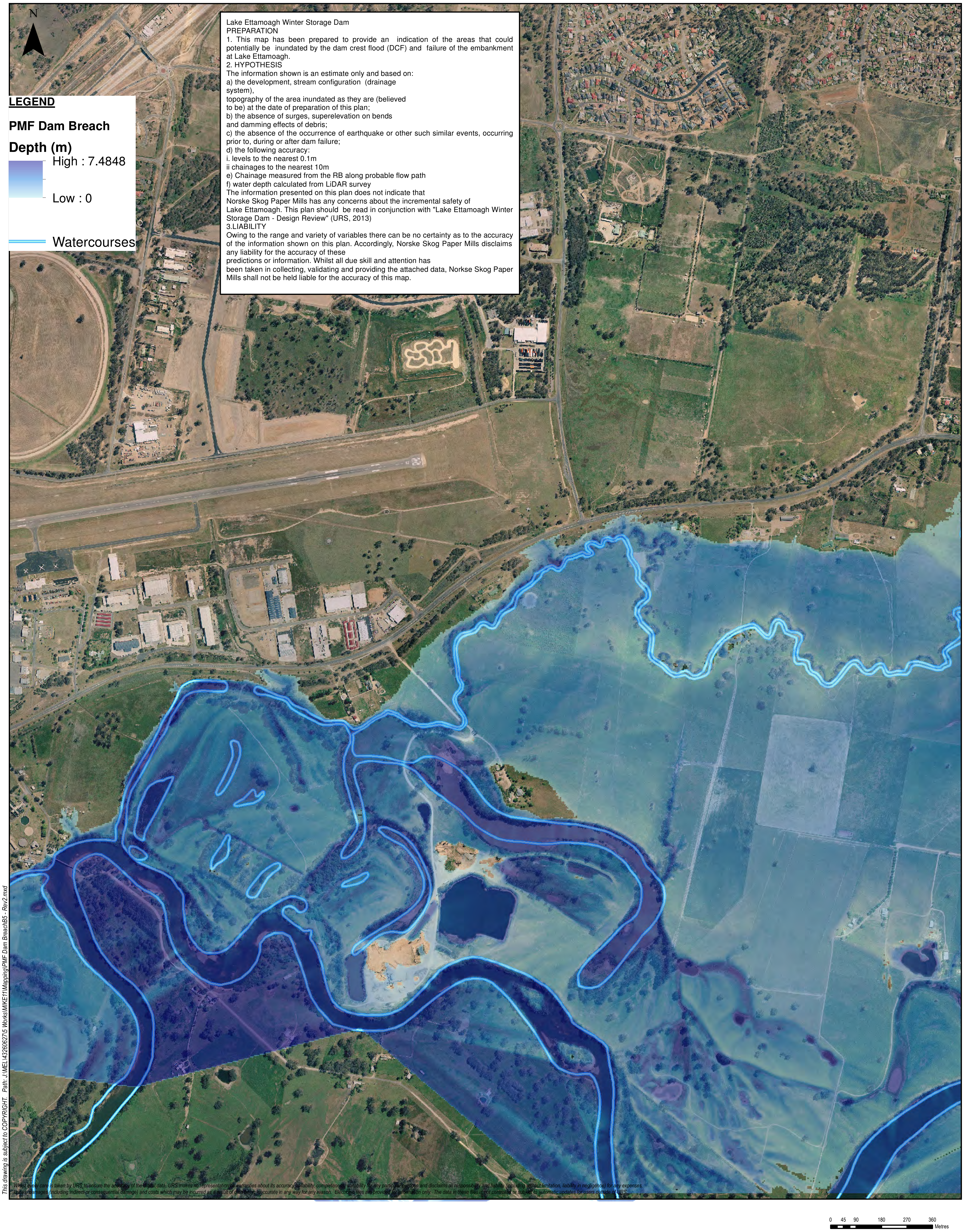
PMF DAM BREACH
DB4

URS

Figure: **DB4**

File No:**PMF Dam BreachB4 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Rev. **B** A3



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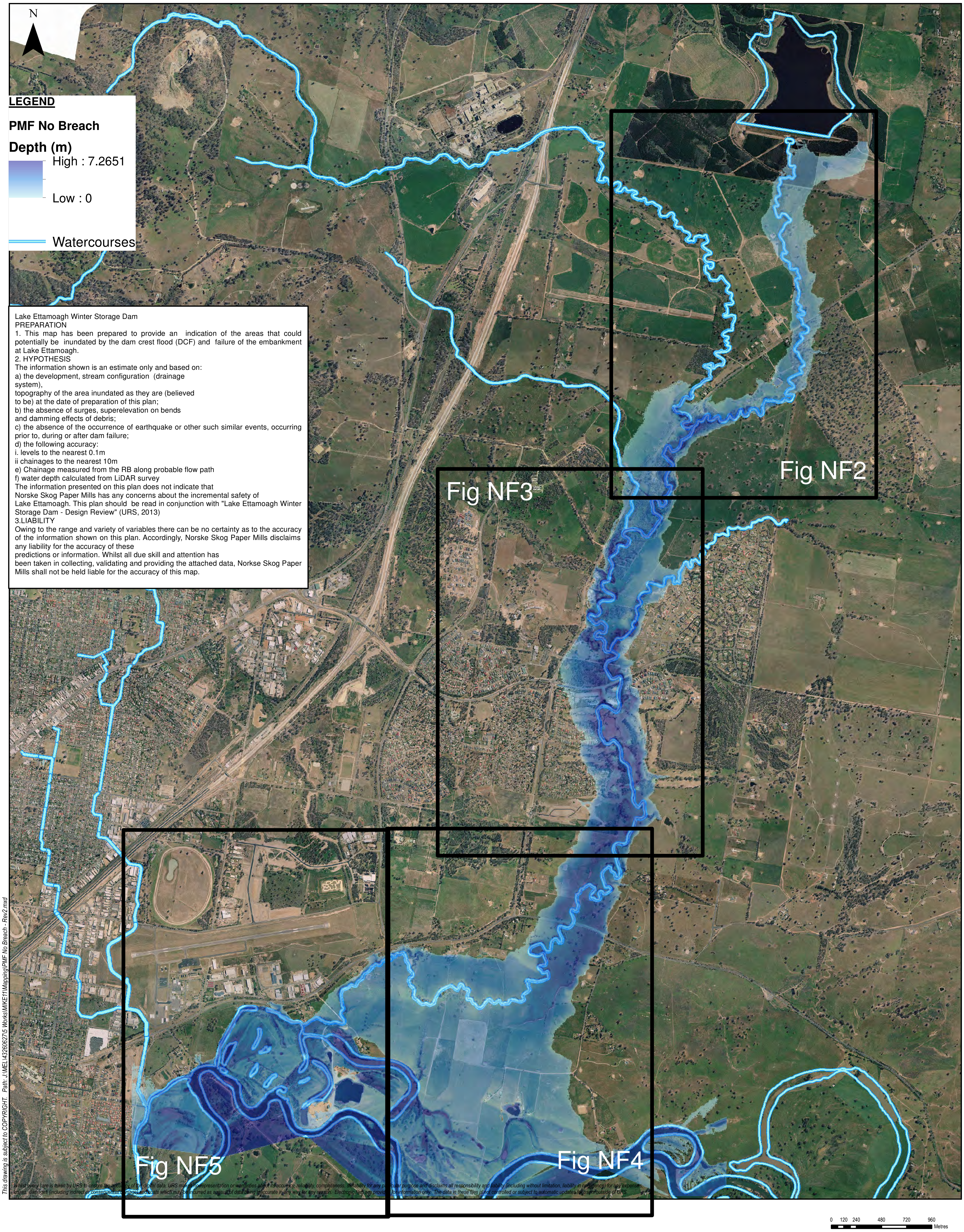
PMF DAM BREACH
DB5

URS

Figure: **DB5**

File No: **PMF Dam BreachB5 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013** Rev. **B** A3



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DESIGN REVIEW

PMF NO FAIL

URS

File No: **PMF No Breach - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Figure: **NF1**

Rev. **B** A3

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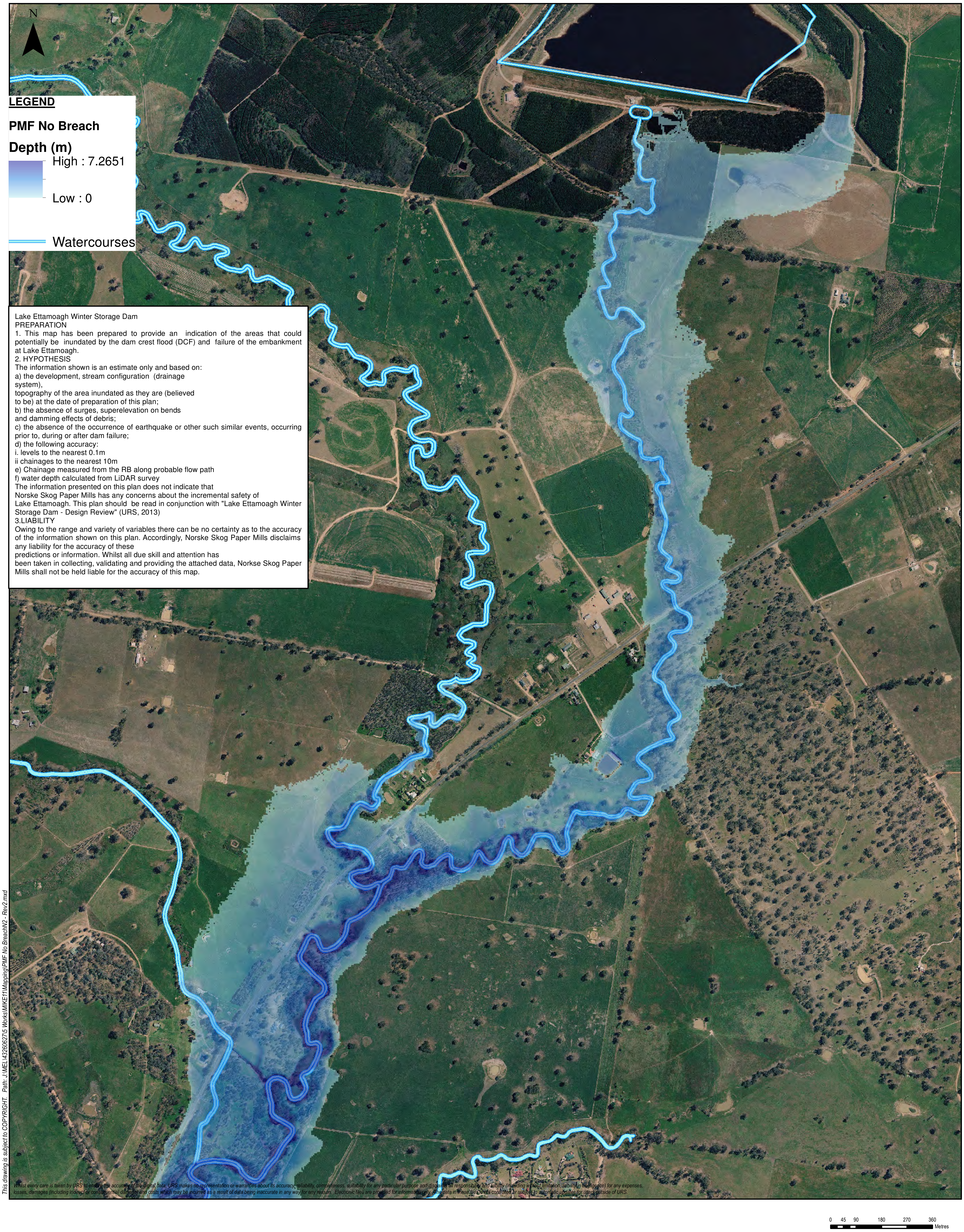

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DESIGN REVIEW

PMF NO FAIL
NF2

URS

Figure: **NF2**

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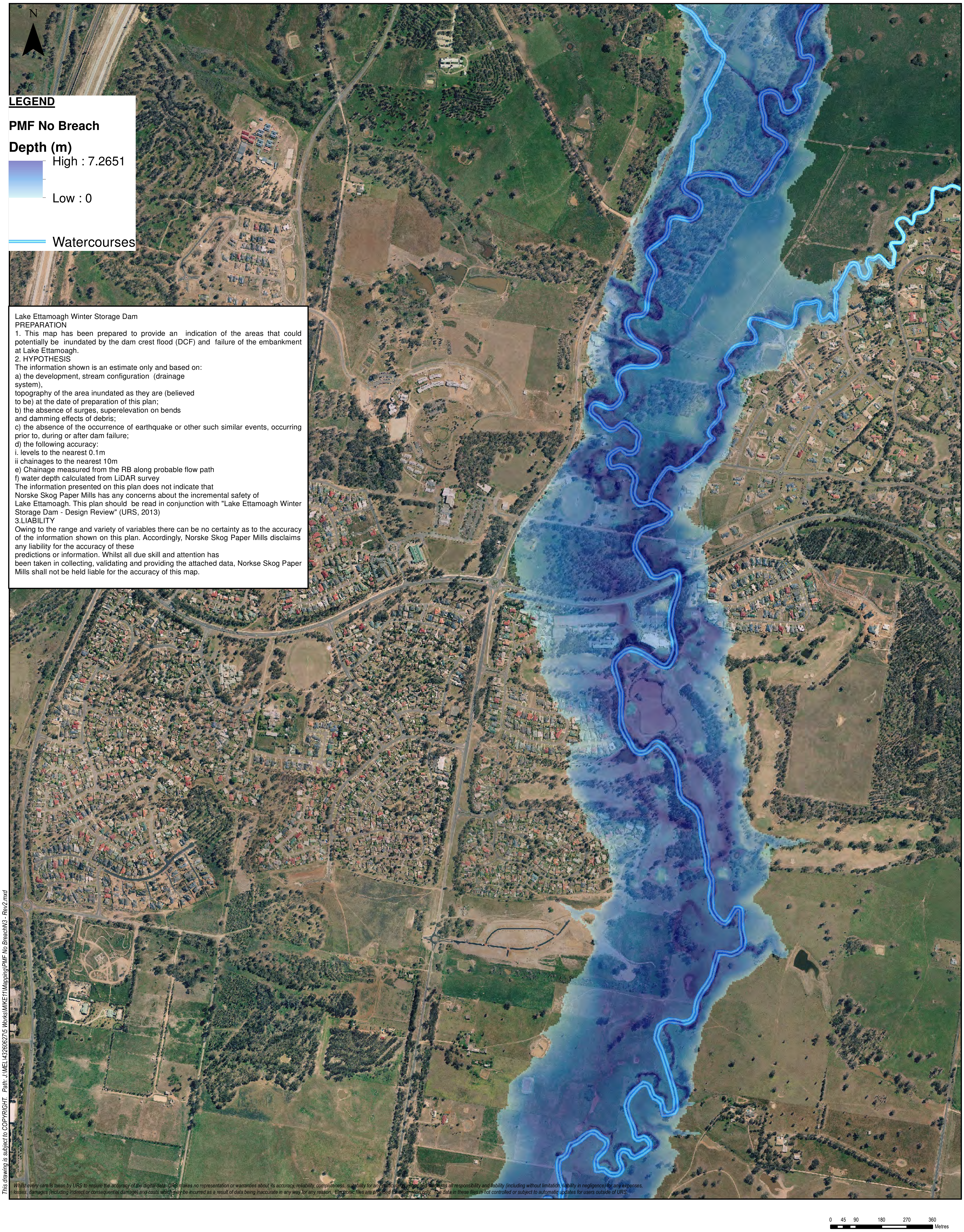

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PMF NO FAIL
NF3

URS

Figure: NF3

File No:**PMF No BreachN3 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Rev. **B** A3

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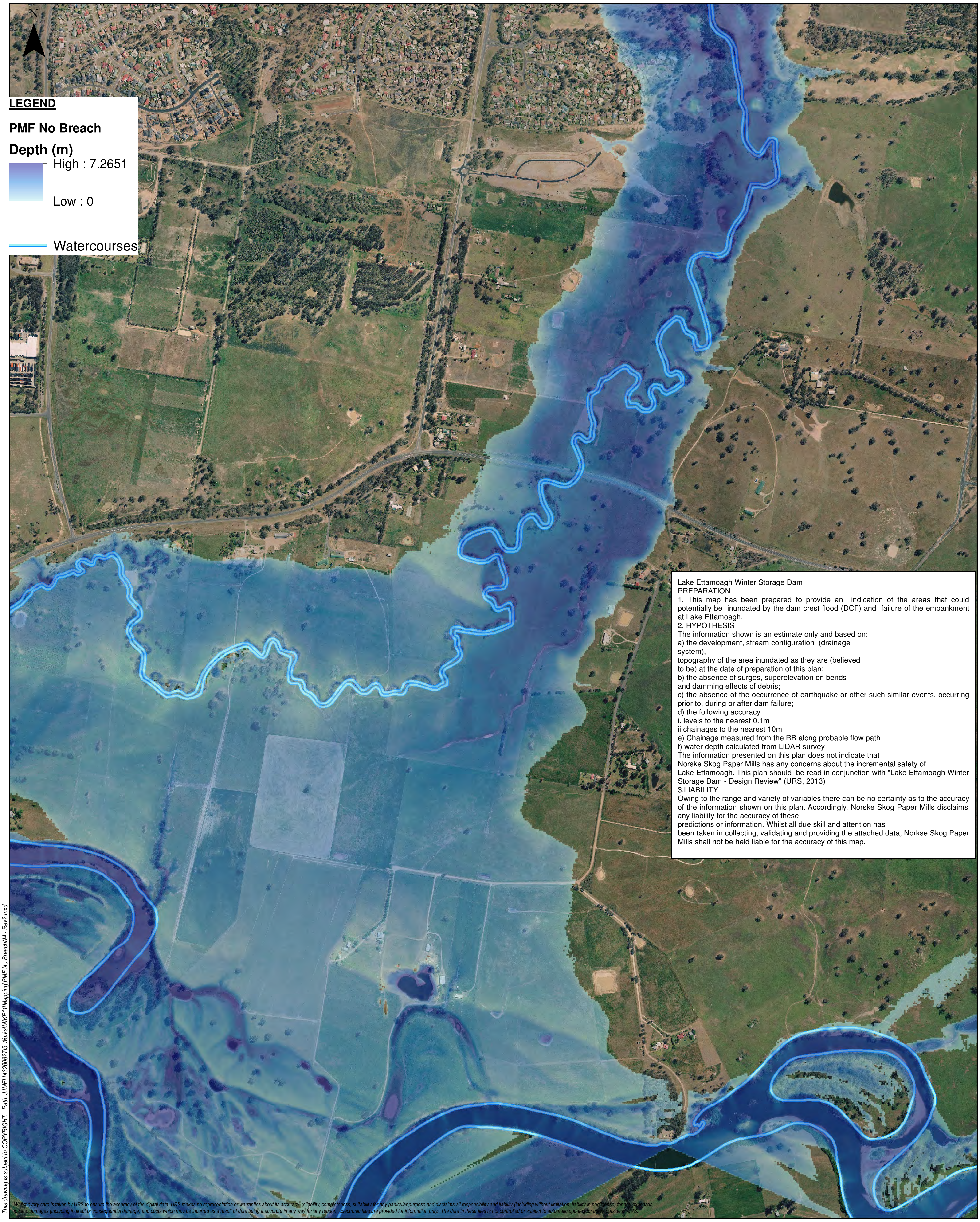

0 45 90 180 270 360 Metres

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DESIGN REVIEW

PMF NO FAIL
NF4

URS

Figure: **NF4**

File No: **PMF No BreachN4 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013** Rev. **B** A3

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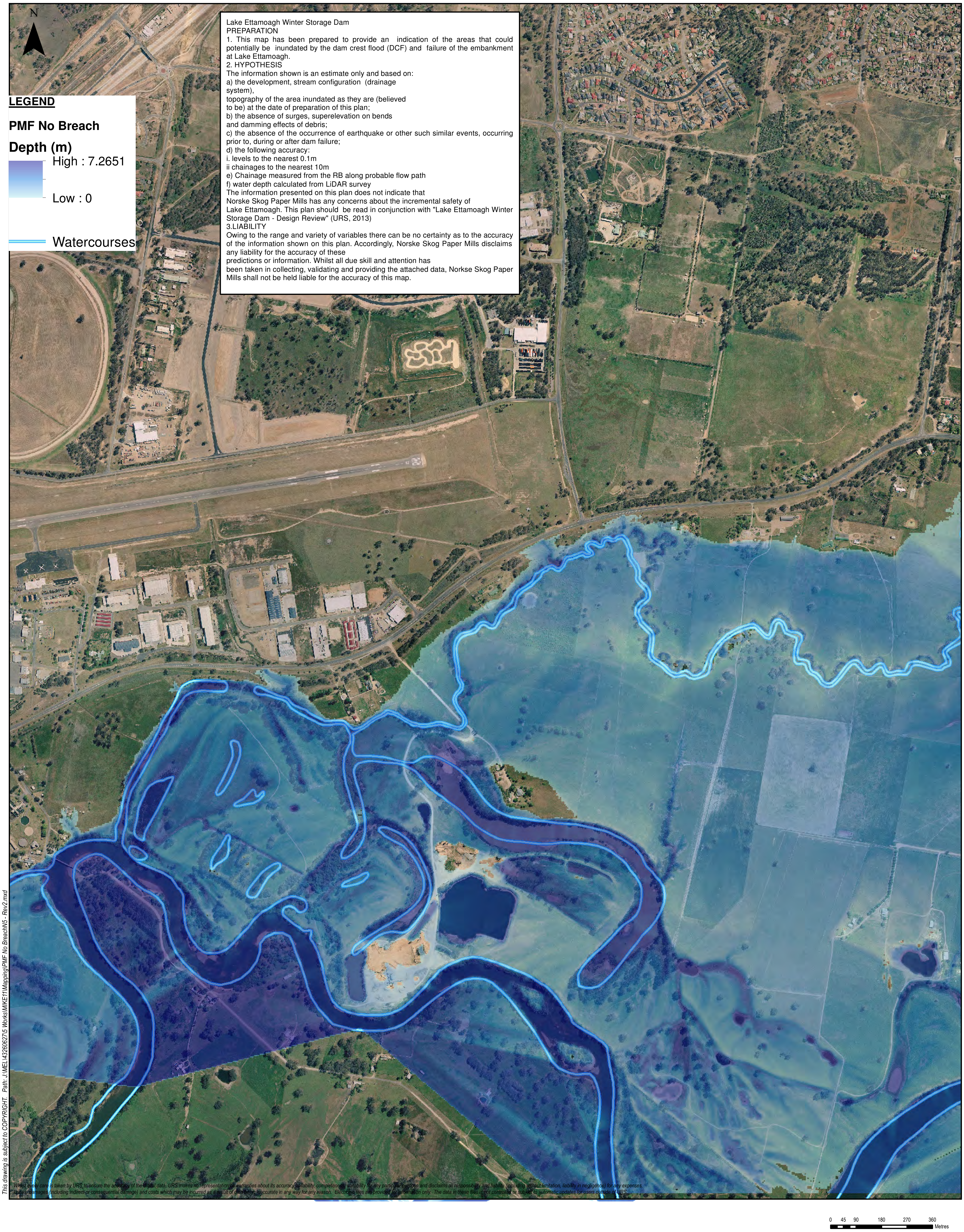

Lake Ettamoagh Winter Storage Dam
PREPARATION
1. This map has been prepared to provide an indication of the areas that could potentially be inundated by the dam crest flood (DCF) and failure of the embankment at Lake Ettamoagh.
2. HYPOTHESIS
The information shown is an estimate only and based on:
a) the development, stream configuration (drainage system),
topography of the area inundated as they are (believed to be) at the date of preparation of this plan;
b) the absence of surges, superelevation on bends and damming effects of debris;
c) the absence of the occurrence of earthquake or other such similar events, occurring prior to, during or after dam failure;
d) the following accuracy:
i. levels to the nearest 0.1m
ii chainages to the nearest 10m
e) Chainage measured from the RB along probable flow path
f) water depth calculated from LiDAR survey
The information presented on this plan does not indicate that Norske Skog Paper Mills has any concerns about the incremental safety of Lake Ettamoagh. This plan should be read in conjunction with "Lake Ettamoagh Winter Storage Dam - Design Review" (URS, 2013)
3.LIABILITY
Owing to the range and variety of variables there can be no certainty as to the accuracy of the information shown on this plan. Accordingly, Norske Skog Paper Mills disclaims any liability for the accuracy of these predictions or information. Whilst all due skill and attention has been taken in collecting, validating and providing the attached data, Norkse Skog Paper Mills shall not be held liable for the accuracy of this map.

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LAKE ETTAMOGAH WINTER STORAGE DAM
DESIGN REVIEW

PMF NO FAIL
NF5

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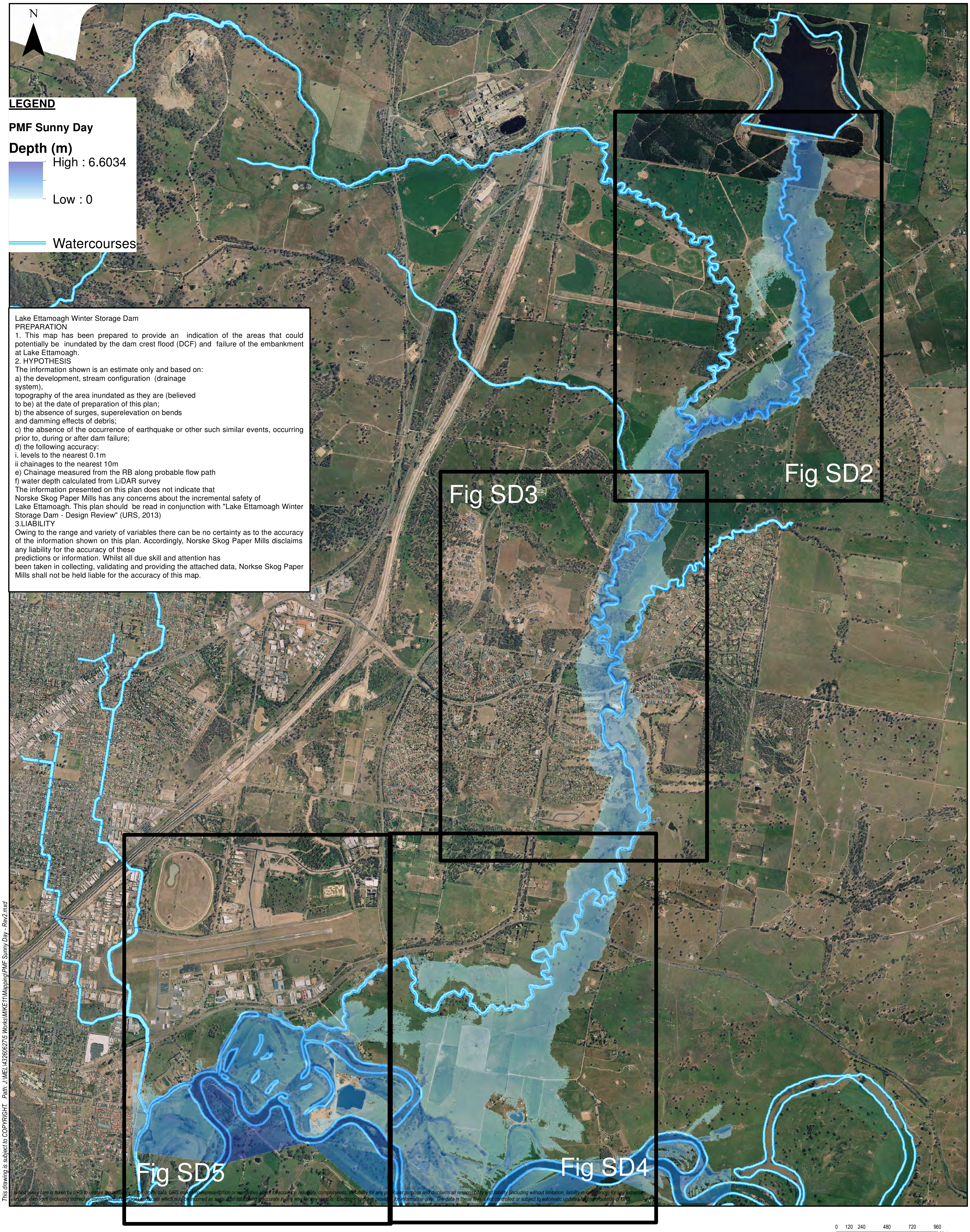
Figure: **NF5**

File No: **PMF No BreachN5 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013** Rev. **B** A3

J:\MEL\43260627\5 Works\MIKE11\Mapping\PMF Sunny Day - Rev2.mxd



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DESIGN REVIEW

SUNNY DAY FAILURE

URS

Figure: **SD1**

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Rev. **B** A3

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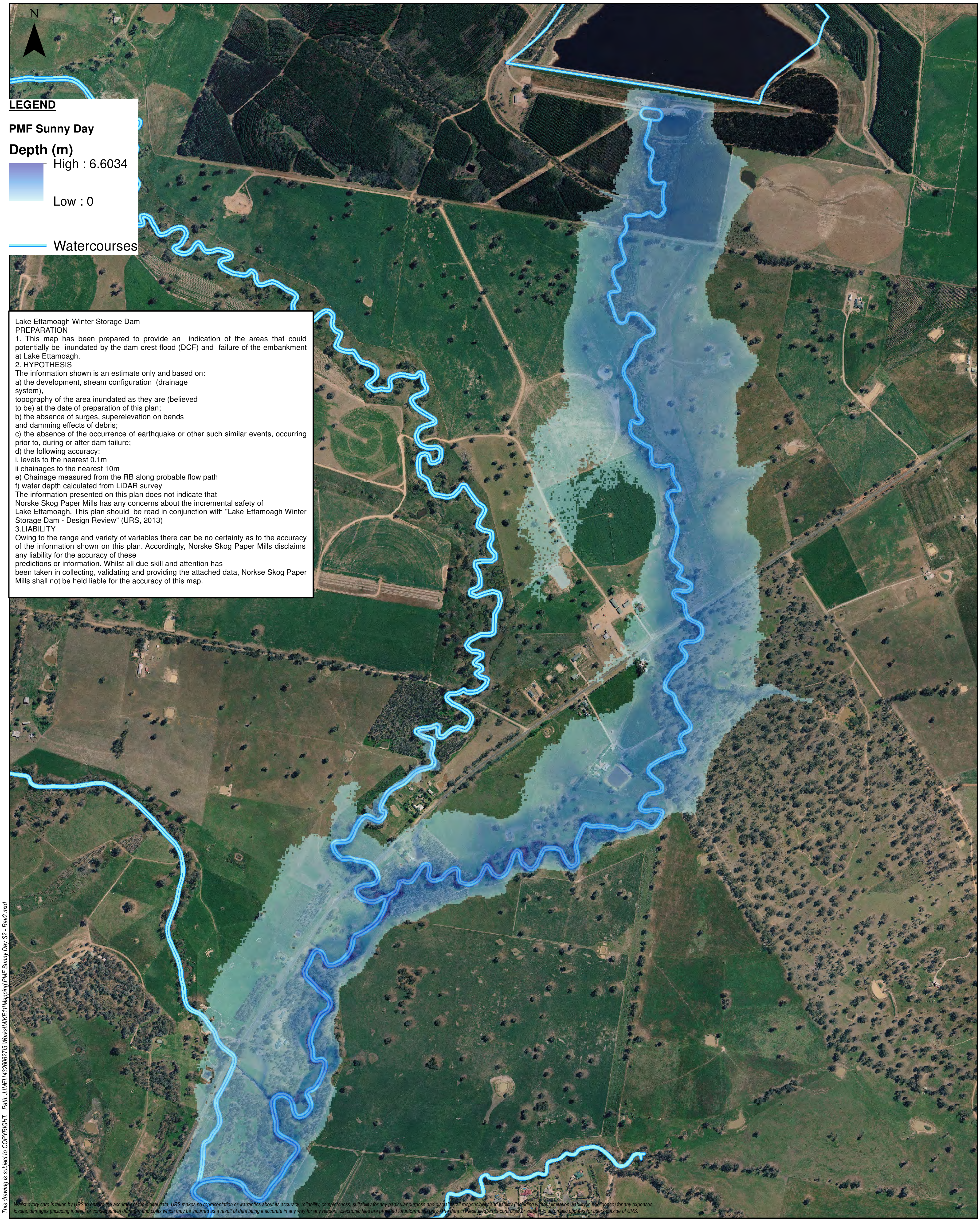

0 45 90 180 270 360
Metres

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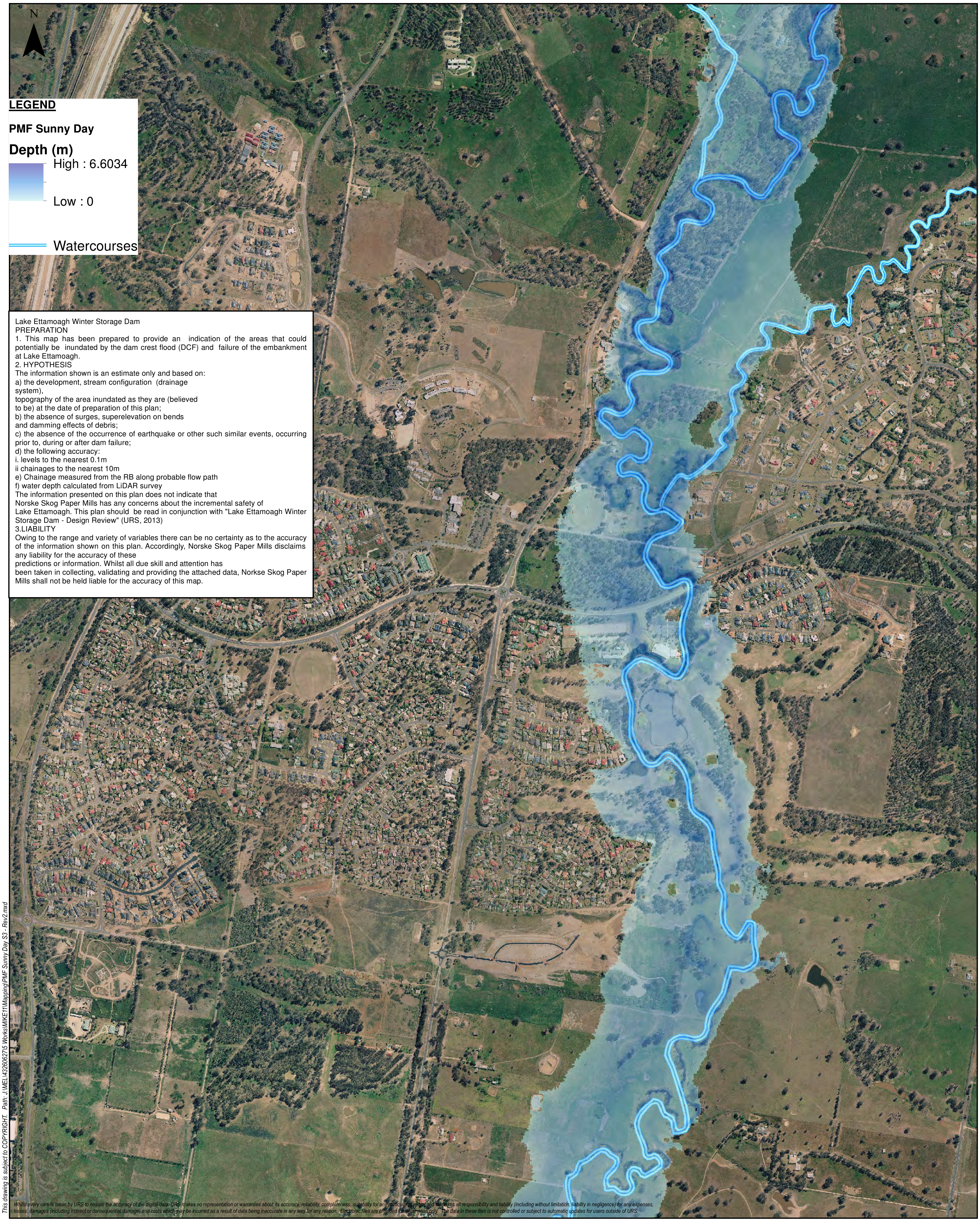
LAKE ETTAMOGAH WINTER STORAGE DAM
DESIGN REVIEW

SUNNY DAY FAILURE
SD2

URS

Figure: **SD2**

File No: **PMF Sunny Day S2 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013** Rev. **B** A3



Lake Ettamoagh Winter Storage Dam
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LAKE ETTAMOGAH WINTER STORAGE DAM
DESIGN REVIEW

SUNNY DAY FAILURE
SD3

URS

File No: **PMF Sunny Day S3 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Figure: **SD3**
Rev. **B** A3

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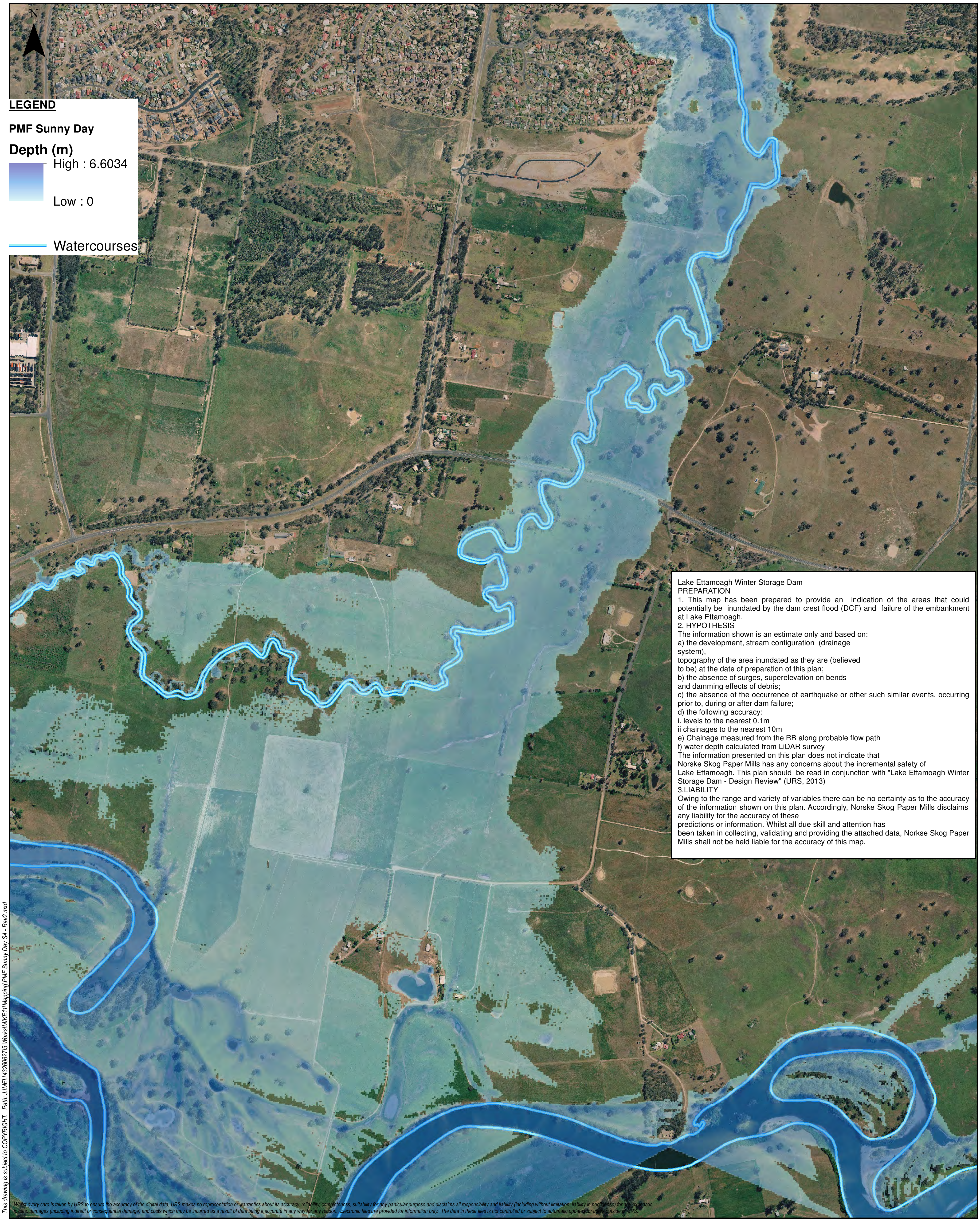

0 45 90 180 270 360
Metres

SUNNY DAY FAILURE
SD4

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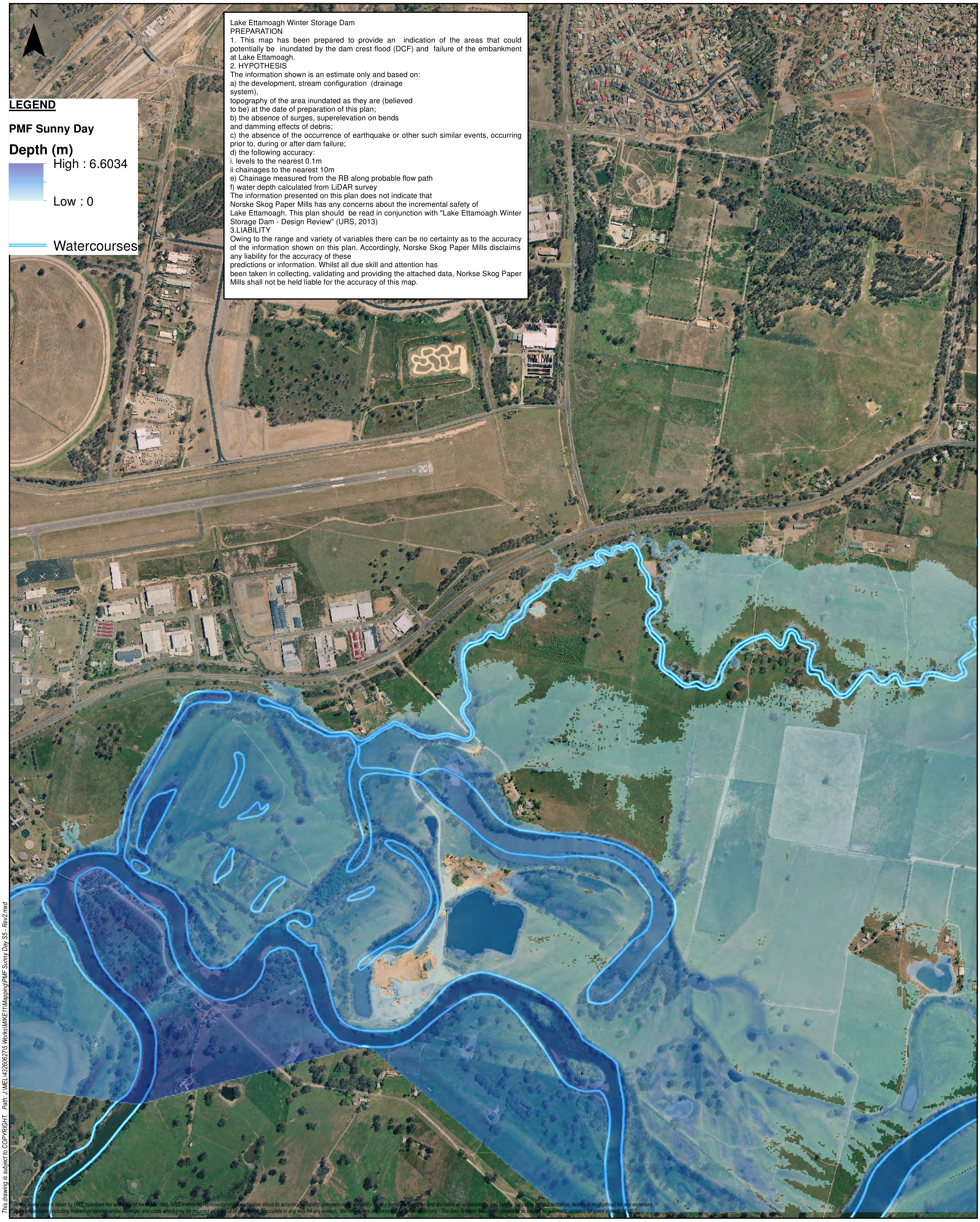
LAKE ETTAMOGAH WINTER STORAGE DAM
DESIGN REVIEW

URS

File No: **PMF Sunny Day S4 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Figure: **SD4**
Rev. **B** A3



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3.LIABILITY
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LAKE ETTAMOGAH WINTER STORAGE DAM
DESIGN REVIEW

SUNNY DAY FAILURE
SD5

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Figure: **SD5**

File No: **PMF Sunny Day S5 - Rev2.mxd** Drawn: **TB** Approved: **EH** Date: **10/12/2013**

Rev. **B** A3

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