DENILIQUIN COUNCIL

EDWARD RIVER AT DENILIQUIN FLOOD STUDY

FINAL

NOVEMBER 2014

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Project Edward River At Deniliquin Flood Study	**Project Number** 112002
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FOREWORD

The NSW State Government's Flood Policy provides a framework to ensure the sustainable use of floodplain environments. The Policy is specifically structured to reduce the impact of flooding in rural and urban areas. In addition, the Policy provides a means of ensuring that any new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the Government through four sequential stages:

1. Flood Study
- Determine the nature and extent of the flood problem.

2. Floodplain Risk Management
- Evaluates management options for the floodplain in respect of both existing and proposed development.

3. Floodplain Risk Management Plan
- Involves formal adoption by Council of a plan of management for the floodplain.

4. Implementation of the Plan
- Construction of flood mitigation works to protect existing development, use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

EXECUTIVE SUMMARY

A flood study has been carried out for the Deniliquin Local Government Area (LGA) in accordance with the NSW Government's Flood Policy. The flood study is aimed at determining design flood behaviour in the area. Design flood behaviour has been defined through the use of a flood frequency analysis and a 2D hydrodynamic model. Design flood levels will be used to assess the performance of the levee in Deniliquin, as well as identify peripheral flooding issues, as part of the Floodplain Risk Management Study.

Deniliquin has experienced irregular flooding over the past 150 years, resulting from high flows in the Edwards River. Major floods occurred in 1870, 1917, 1956 and 1975, inundating large sections of the town and the surrounding area. A levee system has been built in stages in the past 50 years to protect the town from flooding, the most recent stage of which was completed in 2012. The most recent modelling that covered the LGA was undertaken in 1984 and produced design levels on which the levee design was based.

Design flood behaviour was determined for events ranging from 20% to 0.5% Annual Exceedance Probability (AEP) as well as the Probable Maximum Flood (PMF). The analysis was made up of two parts: firstly, design discharges were derived from a flood frequency analysis, and secondly, a 2D hydraulic model was used to determine the flood level and velocity corresponding to those discharges.

Flood Frequency Analysis

The flood frequency analysis consisted of fitting a probability distribution to the historic record of flows at the river gauge in Deniliquin. In accordance with current practice, various scenarios were assessed that each represented a different historical record, and two probability distributions were tested for each scenario. These scenarios measured the sensitivity of the analysis to the gaps in the historical record and ensured no missing data was creating significant bias in the derived discharges. The adopted design discharges are given in Table 1.

Table 1: Estimated Design Flows

AEP (%)	Flow (m^3/s)	Flow (Ml/d)
10	998	86,200
5	1391	120,200
2	1861	160,800
1	2204	190,400
0.5	2425	209,500

Hydraulic Analysis

Design flood levels and extents were derived from a 2D hydraulic model that represented the floodplain in detail. The model covered the Deniliquin LGA and was based on the TUFLOW software. The features in the model, including the ground elevation, the river bathymetry, the levee height and alignment, and the hydraulic roughness were manually compiled using data from a variety of sources. The model was calibrated using three historical events (the floods of

1956, 1975 and 1993), which allowed for adjustment of model parameters. The design discharges were then used to construct design hydrographs, which, when applied to the model's upstream boundary, produced design flood levels and extents across the LGA. The model shows the 1% AEP flood to peak at 92.28 mAHD at the National Bridge and 92.51 mAHD at the location of the town gauge, generally similar to what was previously estimated. The height and extent of the 1% AEP peak flood level is shown on Figure 17.

The 1% AEP flood event was found to overtop the levee in North Deniliquin at three points. The overtopping is a result of the 1% AEP peak flood level being slightly higher than what was previously applied at those locations. The previous model did not capture the complexity of the flow behaviour in the vicinity of the overtopping, and the relatively low freeboard in North Deniliquin does not accommodate the inaccuracy.

The hydraulic model was also used to detail hydraulic behaviour, including the flood behaviour at key locations in the study area. In addition to design depths and levels, the study determined provisional hazard and hydraulic categories for two design events, and an estimate of the 1% AEP + 0.5 m extent. Rates of rise and evacuation routes were assessed for several points of interest in the study area, to assist planning emergency procedures.

The design flood levels produced by the current study will supersede the previous LGA-wide assessment, completed in 1984. The levels derived in the previous study used a different hydrological analysis method with a different historical record, and combined them with markedly dissimilar hydraulic analysis. As such, it is reasonable that the levels should change, and that design flood behaviour should be updated based on the increased length of hydrologic record and changes to assessment methods.

1. INTRODUCTION

Deniliquin has experienced severe flooding on several occasions since its settlement in the mid 19th century. The largest flood on record occurred in 1870, devastating the town and the surrounding land. Large floods then passed through the town in 1917 and 1931, before a makeshift levee was built in 1955 in the weeks leading up to the flood of that year. The levee essentially protected the town during that flood and the one of the following year, which was exceptionally large and inundated the Davidson Street area. Subsequent floods have not peaked as high as the 1956 event and the Davidson Street area, as well as the town, has been largely flood free.

A series of studies have examined the probability, impact and management of flooding in the town. The primary mitigation measure has been the construction and upgrading of the levee system. While the initial construction of the levee was based on historic flood levels, its current height is based on a 1% AEP flood level (determined in 1984) with an added freeboard. The freeboard is generally 0.5 m in South Deniliquin and 0.1 m in North Deniliquin (but up to 1 m in some sections), as per the recommendation of the 1997 study, that assessed freeboard height. Following identification of the land as being in a floodway area, the Davidson Street area's levee was not upgraded with other levees, and development on the land restricted. Various other studies have also been undertaken that examine flooding in the area.

At the time of writing, Deniliquin's most serious floods occurred over 50 years ago and the upgrade of the levee system has finished. It might be assumed that the levee has resulted in the decrease in major flood events; however, this is not the case. The hydrological record of the past 50 years shows no floods have occurred that would test the full capacity of the levee to withhold the river in flood. As the levee has no impact on the magnitude of flow in the river, the lack of extreme flood events and the levee's construction is a coincidence.

It is the role of the current study to re-assess the design flood behaviour in response to a number of changes that have occurred since the previous assessment. A number of assessments have been previously undertaken, however, the 1984 study is the most important as it was used to determine the height of the levee as it now stands. Since that study, an additional 30 years of hydrological data has become available, which allows for a more extensive analysis of the Edward River's behaviour. Furthermore, there have been significant advances in the hydrologic analysis methods used to estimate the design flows of the river, as well as large changes to how the hydraulic behaviour of the floodplain is modelled. All of these factors warrant a re-assessment of the design flood behaviour, particularly the 1% AEP level on which the levee is based. This report is divided into five sections:

1. **Background**. Describes the study area and the previous studies undertaken.
2. **Available Data**. Lists and summarises the previous work done in the study area.
3. **Flood Frequency**. Describes the methodology used to determine the design discharges.
4. **Hydraulic Analysis**. Describes the hydraulic model that was established for this study, including calibration, design flood results and sensitivity analysis.
5. **Review of Flood Risk.** Outlines the types of risk assessed as part of the current study, including hydraulic hazard, hydraulic categories and flooding iat points of interest.

2. BACKGROUND

2.1. Study Area

The Edward River is located in the Riverina region in the south-west of New South Wales. The River is an anabranch of the Murray River, running parallel to it for approximately 380 km before re-joining it at Wakool Junction. This study concerns the section of the Edward River in the Deniliquin Council Local Government Area, which is approximately 100 km^2 and has a 19 km long section of the Edward River and its floodplain, as shown on Figure 1. The river travels in a general north-west direction through the area, which varies in elevation from 80 mAHD to 100 mAHD. The area, like much of the Riverina region, is characterised by its very flat terrain, containing mostly agricultural and pastoral land. The town of Deniliquin (population approximately 8,000) lies on both sides of the Edward River, and has a number of properties in the floodplain itself.

2.2. Flood Behaviour

Heavy rainfall or snowmelt in the catchment, which extends as far east as the Great Dividing Range, causes flooding in the Edward River. The river system is complex in that the flow in the Edward River is made up of flows from not only the Murray River but also Tuppal and Bullatale Creeks, themselves anabranches of the Murray River. The peak flow of the Edward River at Deniliquin is strongly influenced by the extent of overlap of the timing of the peak flows coming from each of these sources. These inflows to the Edward River occur outside of the study area and so their relative magnitude and timing has not been assessed, specifically.

The section of the Edward River in the study area is characterised by a primary channel, a number of flood runners that leave and rejoin the river, a number of 'high flow' areas adjacent to the main channel which are inundated in moderate flooding. The land that the town itself is located on is generally above the 'high flow' areas and is only flooded in rare to extreme flood events. The main channel is approximately 70 m wide (± 30 m) with a bed elevation that drops by approximately 2.5 m within the study area. Most flood runners only transmit water during periods of high flow and are otherwise dry. One such flow path passes through the town centre but is effectively sealed off from the river due to the levee system.

The town's levee system has been constructed at various stages since a burst of construction prior to the 1955 flood. There are two main levees, one on each side of the Edward River. The levee on the south side is approximately 9 km long and varies between 91.82 and 93.83 mAHD. The levee on the north side (which does not include the informal levee between the Edward River and Brick Kiln Creek) is approximately 5.5 km long and varies between 92.05 and 93.56 mAHD. Other levees exist but are not maintained by council, including that around the Davidson St precinct (lowest height around 91.8 mAHD), that around the Mclean Beach Caravan Park (lowest height around 90.5 mAHD) and other small, private levees.

Stevens Weir is located 26 km downstream of the town and also affects the hydraulic behaviour of the river. The weir, constructed in 1935, has a backwater effect that extends past Deniliquin, as it holds water during low flow periods. Comparison of the water level at Deniliquin before and after the weir's construction shows that it prevents the water level from going below 2 m depth at the gauge. Currently, the rating table at Deniliquin does not apply until the gates of the weir are raised well above the water (R Brown 2012, pers. comm.).

2.3. Previous Studies

A number of studies have investigated flooding in and around Deniliquin. These were used to inform the methods of analysis of the current study, as well as to provide background on the levee system and the history of flooding. The studies have been categorised as either floodplain management studies or environmental assessments.

2.3.1. Floodplain Management Studies

Deniliquin Floodplain Management Study - Rankine and Hill, February 1984

The study made a comprehensive assessment of flooding behaviour in the area which was used to determine the height of the levee system that was subsequently built and completed in April 2012.

The flood frequency analysis carried out was based on a 'partial series' of annual flood maxima at the 'Edward River at Deniliquin' gauge. The partial series was taken as those events with stage greater than 5.90m (The SES defined 7.16 m as the moderate flooding threshold, in a 2009 SES plan it is listed as 7.2 m). The earliest event in the series is from 1867, and where there are gaps in the record (e.g. 1867-97, 1943-53) gauges near the Deniliquin site have been used (no further details are given). The report does not detail which rating table was used in the analysis, nor what fitting method was used on the partial series. Based on the figure in the report showing the historical record and the fitted discharge-probability curve, it appears that the study omitted the 1917 event (the second highest on record) from its analysis. The curve fitted passes through or near the majority of points, but does not pass near the 1917 point (were it to be added to the plot), suggesting the analysis did not include it, and it was not merely overlooked in plotting. Including the point would most likely raise the upper arm of the curve. The study estimated the 1 in 100 year ARI flow to be 2500 m^3/s (216,000 Ml/d) (generally equivalent to the 1% AEP flow).

Using the results of the flood frequency analysis, the study used a Standard Step Method of Backwater Analysis to determine the extent of flooding for design events of 20, 50 and 100 year ARIs. The report states that the initial water level used at the downstream end of the model did not converge during the simulation and that profiles were consequently estimated from the cross-section rating curves and the model was used as a check. These cross-section rating curves interpolated between recorded flood levels where possible, and then extended beyond the recorded levels using a prescribed method that uses a constant friction factor and slope value. Table 2 gives a summary of the design flood levels. The design flood levels will be superseded by the current study.

Table 2: Deniliquin Flood Plain Management Study - Design Levels

Location	1 in 20 Year ARI	1 in 50 Year ARI	1 in 100 Year ARI
Kyalite Park	92.12	92.55	92.86
National Bridge	91.58	92.08	92.33
Chippenham Park	90.96	91.31	91.59

Deniliquin Flood Protection Levee Study - Sinclair Knight Merz, July 1997

The study assessed the type and design of levee system necessary, including revising the estimate of the levee's freeboard. The study used the flood frequency analysis undertaken in the previous study (Rankine & Hill, 1984) and the design levels determined by that study. The study recommended a freeboard of 0.5 m for South Deniliquin and 0.1 m for North Deniliquin. The freeboard in both locations was assessed in terms of its components (wave action, spillways, levee types etc.), it's benefit from an economic viewpoint, and the community's needs. It concluded that the previously recommended 1 m freeboard was too high and should be lowered.

Edward-Wakool Rivers – Stages 1, 2, 3 – Flood Study Report - SMEC, May 2004

The study modelled the Edward-Wakool Rivers between Deniliquin and Liewah Station (more than 100 km of river), and used MIKE-11, a more advanced 1D hydraulic model than the HEC model previously used. The model was calibrated with the flood events of '93, '75 and '56, and validated using the '96 event. Different model parameters were used to represent the current and pre-developed states of the floodplain. The upstream boundary of the study area was the town of Deniliquin and the model used the gauged data from the town as an inflow.

Because the study only gave a broad-scale representation of the flow distributions and began the model in the town itself, the design results are not applicable for use in validating hydraulic modelling in the current study.

Hydraulics Analysis of the 100 year ARI Flood on South West Deniliquin - NSW Department of Commerce, 2008

Using HEC-RAS and the procedures established in the Rankine and Hill model, the study covered Edward River and its floodplain starting at Stockbridge and ending at Lawson Syphon. It found that if the existing levee is not extended to the Mulwala Canal, south-west Deniliquin would experience Low Hazard flooding during the 100 year ARI event (generally equivalent to the 1% AEP event), with a maximum ponding depth of around 0.28 m. It modified the model used by the 1999 Golf Course Levee Report and used Rankine and Hill (1984) modelling procedures.

2.3.2. Environmental Assessments

Three environmental assessments undertaken in the area were reviewed. They were:

- Environmental Impact Statement for the Construction of North Deniliquin Flood Levees (CMPS&F Environmental, 1994)
- South Deniliquin Levee Stage II and North Deniliquin Levee Stage II – Environmental Impact Statement (Kinhill, 1996)
- Deniliquin Floodplain Management – Statement of Environment Effects for the West Deniliquin Levee Bank (GHD, 2005)

The studies uniformly concluded that their respective sections of the levee system would have minimal impact on flooding patterns in the area. The Kinhill (1996) report mentions that there would be a minor increase in flood levels upstream of the levee due to its construction, referencing the Rankine and Hill (1984) study. Also, the CMPS&F (1994) report found that the minor relocation of a section of the North Deniliquin Drainage Channel would have no impact on the overall drainage system.

3. AVAILABLE DATA

Data was collected from various sources and was used for several components of the study. The data has been categorised as being historical flood data, topographic data, aerial photography or from community questionnaires. Historical flood data and community questionnaires were used to construct a hydrological history of the area for use in the flood frequency analysis and in calibrating the hydraulic model, while the topographic data and aerial photography was used to represent the floodplain and its various features in the hydraulic model.

3.1. Historical Flood Data

3.1.1. Stream Gauging

Several types of hydrologic data, including flood heights, rating curves and discharges, were available for stream gauging stations relevant to the study. Water levels have been taken at these sites and then converted to a discharge value via a rating table. Rating curves are developed using measurements of velocity and the channel cross-section, which are combined to give a discharge value. Rating curves were initially compiled from the same source as the water level records.

Streamflow records are available for three locations in the study area, the details of which are given in Table 3. The Edward River at Deniliquin gauge was moved in 1981 from the bridge to a point approximately 250 m upstream. It is possible that the new location has a different stage-discharge relationship. Based on the cross-section, which is significantly different, it could be assumed the relationship has changed. However, the gauged data taken since the move suggests the relationship to be unchanged. The location of the gauging stations are shown in Table 3.

Table 3: Streamflow Gauging Stations

Gauge Location	Gauge Number	Period of Operation
Escape from Mulwala Canal at Edward River*	409029	1940 to present
Edward River at Deniliquin*	409003	1896 to present
Wakool River at Offtake Regulator*	409019	1935 to present
Edward River downstream of Stevens Weir	409023	1935 to present
Edward River at Toonalook	409047	1979 to present
Tuppal Creek at Aratula River	409056	1985 to present
Bullatale U/S Edward River	409075	1991 to present

*Located within the study area

Streamflow data was also available for a further four locations to give a broader range of data for historical events of interest, also given in Table 3. ‘Edward River downstream of Stevens Weir’ is a further 10 km downstream from the study area whilst the other three are south and upstream of the study area near the confluence of Tuppal Creek and Edward River.

Although the gauge at Deniliquin (409003) began operation in 1896, the site summary report for the gauge includes information on floods prior to this. The report, a short document recording noteworthy information relating to the gauge, lists the flood levels for several major events, listing the 'River Discharge Register' as the source. According to the staff at the Deniliquin Office of Water, the River Discharge Register was a book found in an archive in Sydney that recorded discharges for different sites on the Murray, which were then copied into the site summary report. These flood levels, which include the largest flood on record, were included in the flood frequency analysis.

3.1.2. Flood Marks

Marks which record the peak height of the river at a particular location were available for the 1956 and 1975 flood events. Five were available for the 1956 event and twelve were available for the 1975 event. The marks were taken from two sources: eight flood marks for the 1975 event were from a map provided by Brian Mitsch & Associates (written as R.W. Veitch & B.L. Mitsch on the map) and the remaining flood marks were those used in the 1984 study, which have an unknown origin. The height and location of each flood mark is described in the calibration section (Section 5.4).

3.1.3. 'Flood History of Deniliquin'

The book 'Flood History of Deniliquin' provided extensive information about past floods in the form of scanned newspaper articles. The articles, as well as a series of photos, date back to 1870 and refer to floods as early as 1851. Of particular interest were those events not recorded at the Deniliquin gauge (either due to a gap in the record or them being pre-1896), as little information is available for these events. The book was from Deniliquin Library and was made available as a PDF file.

3.1.4. Flood Photography

Flood photography shows the extent of past floods and aids interpretation of model results, especially when modelling historical events. It may also show changes to the floodplain and the town over the past century. Photographs were included in the book 'Flood History of Deniliquin', as a series of aerial photographs showing the 1955 flood, and were provided by residents as part of the community consultation process.

3.2. Topographic Survey

Topographic survey data gives a detailed representation of the landforms in the study area and is a key component of the hydraulic model. The topographic data collected consisted of Light Detection and Ranging (LiDAR) data, surveys of cross-sections of the Edward River, dimensions of bridges and culverts, and the height and alignment of the levee system (both current and historical).

3.2.1. LiDAR

LiDAR data is used to build a three dimensional representation of the study area's topography. The data consists of a series of points spatially distributed across the study area, each of which has an elevation value. These points are used to create a triangular irregular network, from which a Digital Elevation Model (DEM) is then created. The DEM discretises the area into a grid of 1 m^2 cells, each having an elevation value. This data has a vertical accuracy of ±0.15 m and a horizontal accuracy of ±0.18 m (both within one standard deviation). LiDAR data used in this study was surveyed on June 19^{th}, 2012 by Photomapping Services Pty Ltd and covered the study area.

A second LiDAR set was used in order to extend the 2D model domain for use in modelling the PMF event. The LiDAR was captured in August 2001 as part of the Southern Murray Darling Basin LiDAR Project. A 15 m DEM was produced from the LiDAR, which was used to model a larger area than that covered the 2012 LiDAR described above. Some inaccuracies were found in the LiDAR data, namely long 'troughs' of slightly lower terrain that were a result of the collection and processing. Given the other uncertainties in estimating the PMF flood extent, it was not felt that the inaccuracies were significant.

3.2.2. River cross-sections

Cross sectional data of the Edward River consisted of nine cross-sections of the main channel, originally used in the Deniliquin Flood Plain Management Golf Course Levee Report (February 1999). The cross-sections were used to augment the DEM to include the river bed's bathymetry (a feature not captured in the LiDAR), and so only the centre portionof each cross-section was used. Their location is shown on Figure 2.

3.2.3. Bridges and Culverts

Survey plans of the National Bridge, which links North and South Deniliquin, were taken following construction of the bridge and provided by Roads and Maritime Services for the current study. Visual inspection was made of the remaining bridges in the study area (none of which cross the main channel)

Survey data of culverts in the town was limited to visual inspection of the culverts' dimensions and estimation of their invert levels based on the LiDAR data. Subsurface drainage has little to no effect on major flooding in Deniliquin and so a detailed survey was not warranted.

3.2.4. Levees

The height and alignment (both current and historical) of the various levees was derived from a variety of sources. The following list describes the data available for each levee. Data was taken from the current Operation and Maintenance manual, the 1991 levee audit, a GIS layer provided by Council which showed the alignment and height of an 'Old' and a 'New' levee, and the LiDAR data.

- **South and North Deniliquin**. This is the levee surrounding the town centre, south west of the National Bridge, and the town north-east of Brick Kiln Creek. Two states of development were modelled, the current levee (as of 2012) and a 'historical' version. The current levee is based on the alignment shown in the 2011 Operation and Maintenance Manual, as well as a GIS layer and additional field survey undertaken in April 2014 provided by council. The 'historical' state was based on the survey of the levee taken in 1991 for the study 'Audit of Flood Levee for New South Wales – Town of Deniliquin'. The report states that the levee was raised in 1975, but no further information was found.
- **Davidson Street**. The Davidson Street levee surrounds the section of land between the National Bridge and Brick Kiln Creek. The levee's current height was based on the LiDAR data. This levee's historic height, which was not included in the 1991 audit, was based on a GIS layer provided by Council with the elevation.
- **Mclean Beach Caravan Park and Private Levees.** No field survey data was available for the levee of the caravan park at Maclean Beach. The levee's current height was apparent in the LiDAR data and so this was used. The LiDAR also showed the location of private levees on properties outside of the town.

The alignment of the various levees is shown on Figure 2.

3.3. Aerial Photography

Aerial photography is used as a visual aid to clarify ambiguous features in the DEM and to show the location of key structures such as culverts, bridges and levees. Geo-referenced aerial photography was provided by Council at resolutions of 10 and 20 cm, and also as part of the LiDAR data set.

3.4. Community Questionnaires

As part of the flood study, Deniliquin Council sent out a questionnaire to Deniliquin residents aimed at gathering information on historical flooding and awareness of flooding in the town. By collecting individuals' experiences of flooding, a detailed picture could be constructed as to the extent and behaviour of historical floods. Of particular interest were major floods, including those in 1956, 1975 and 1993, as these were to be used for model calibration and validation. Figure 3 summarises the quantitative data from the questionnaires, including respondents' awareness and experience of previous floods.

124 residents returned completed questionnaires, with almost all respondents being aware of flooding in Deniliquin, and some having experienced it personally. Approximately 20% of respondents had performed mitigation works on their property, including temporary works such as sandbagging. A handful of residents gave detailed descriptions of the flood extent during the 1956 event, describing the level the flood came to relative to their property, and mitigation measures taken at the time. The events in 1975 and 1993 were also referred to, as well as a general sentiment that no severe flooding had occurred recently. The three respondents who

were not aware of flooding can be considered anomalous, with two of the three having lived in Deniliquin for less than ten years.

Out of the residents who responded, the number who experienced inundation was relatively low, given the history of severe flooding in Deniliquin. Two factors may have contributed to this; the low occurrence of extreme flood events in the last 50 years, and the ability of the levee to mitigate flood events since its construction. The locations of the respondents suggested that the length of residency was a significant factor, with those who experienced flooding and had lived in Deniliquin for a long period, living adjacent to those who had not experienced it and had a shorter period of residency. Although the questionnaire did not refer to specific events, it can be concluded that Deniliquin has not been subject to major floods in the past 10-20 years, at least in the areas surveyed. Generally, respondents who were affected by flooding were in two areas – south-west of the golf course and between Edward River and Brick Kiln Creek.

4. FLOOD FREQUENCY ANALYSIS

4.1. Overview

The flood frequency analysis undertaken as part of this flood study uses the original historic data and the analysis method prescribed by Australian Rainfall & Runoff (AR&R). It was felt that the lack of transparency of the method followed in previous studies, as well as possible errors in the analysis, meant it was necessary to re-examine the original sources rather than apply the data used in previous studies. In addition, a new analysis was warranted by the new years of record (1984 – 2011) that have become available since the last study.

Generally speaking, the analysis consisted of fitting a probability distribution to a truncated series of annual peak discharges. This method is recommended by AR&R and avoids the issues associated with using peak flood levels, which can be strongly influenced by changes to the floodplain. For example, the construction of the levee at Deniliquin has decreased the floodplain storage and so slightly higher peak flood levels would be expected, whereas the peak discharge will be unaffected.

The analysis was made up of two stages: constructing a time series of flood events at the Deniliquin gauge and applying a probability distribution to this time series. The first stage involved filling in gaps in the gauge record by using nearby gauges and estimating the height and discharge of events that occurred prior to the gauge's record. The second stage involves fitting different probability distributions to the data and describes how and why the design flood levels differ from previous studies.

4.2. Historical Time Series

The Deniliquin gauge has two significant gaps in the record where nearby gauges were used to estimate the historic levels. The gauge, which has a record of the river height from 1889 to the present, is located approximately 200 m upstream of the National Bridge. It was located at the National Bridge until 1981, when it was moved to the current site. The two significant gaps are from September 1943 to July 1953 inclusive and from 1903 to the end of 1912.

Nearby gauges were used to estimate if any events occurred during these periods which could be large enough to affect the flood frequency analysis. The nearby gauge 'Wakool River at Offtake Regulator' (no. 409019, 1935 to present) was used to estimate the magnitude of events in the gap by measuring the correlation the gauge has with flows at Deniliquin. No significant events were found and the two periods were deemed to be representative of the entire record.

The gauge at Albury, approximately 200 km upstream of Deniliquin and beginning in 1877, was used to estimate what events occurred before the historical record began. There is not a strong enough correlation between the gauges at Albury and Deniliquin to establish a relationship and extend the record at Deniliquin. However, a comparison of large events at both gauges showed that, generally speaking, a large flow at Albury will continue downstream to Deniliquin. The data given by the gauge at Albury was also compared to the events listed in the report ‘Murray River Flood Plain Management Study – Detailed Report’ (Reference 9), which, despite not covering the Edward River at Deniliquin, gave information on large events on the Murray River. The report confirmed the flood events recorded at the Albury gauge and did not list any extreme events not already known.

This correlation gave an upper and lower estimate for seven events in the missing period. The flood frequency analysis was then undertaken with and without these events to determine the sensitivity of the analysis to their presence.

4.3. Height-Discharge Conversion of Historical Time Series

The annual time series of historical flood levels was converted to a time series of equivalent discharges using a combination of existing rating tables and a rating table derived from the calibrated hydraulic model. In general, if events were in the range of the gauged data, the rating table produced by that gauged data was used to convert the height to a discharge. Additionally, the gauged data was required to be recorded during a comparable period of the town’s development. Based on these two criteria, all but five events were able to be converted using an existing rating table. The rating table was chosen based on its applicable date range; rating tables were taken from PINEENA Version 9.3.

The exception to the conversion method described above is the gauged data and subsequent rating curve from July 1931. Inaccuracies were found in the gaugings’ estimate of discharge, and they were therefore not considered representative of the stage-discharge relationship, and therefore the curve derived from the 1931 gaugings was not used to convert any historical heights. An assessment of the estimation method used for the gaugings found that the velocity in the channel was over-estimated, which lead to an over-estimation of discharge. The assessment is fully described in Appendix B.

For the five events that did not have an applicable rating table (as they either occurred prior to the first gaugings or were outside the range estimated by the rating tables), a rating table was derived from the calibrated hydraulic model. That is, the model was used to replicate the behaviour of the river as it was in the early 20th and late 19th centuries, and the model-produced stage-discharge relationship was then used to convert the historical peak flood levels to equivalent discharges. The hydraulic model was based on the TUFLOW software (Refer to Section 5 and was the model used for the design flood analysis as part of this study, excepting the changes made to the model to replicate the earlier topography. The model produced rating curve is shown on Figure 4. The figure also shows the rating curve produced for the current topography, and the gauged data before and after the construction of the levee, for the sake of comparison.

Table 4 lists the 30 highest recorded events at the Deniliquin gauge, most of which are used in the adopted scenario in the flood frequency analysis. The adopted scenario used in the flood frequency analysis only includes the three highest events prior to 1913. This is a result of the record being discontinuous prior to 1913 and is discussed further in the following section.

Table 4: 30 Highest Recorded Events at the Deniliquin Gauge

Year	Peak Flow (m^3/s)	Peak Flow (Ml/d)	Peak Height (m)	Year	Peak Flow (m^3/s)	Peak Flow (Ml/d)	Peak Height (m)
1870	2321	200,500	9.68	1958	855	73,900	8.16
1917	2189	189,100	9.63	1981	850	73,400	8.19
1956	1784	154,100	9.37	1973	841	72,700	8.10
1867	1640	141,700	9.27	1992	819	70,800	8.11
1931	1476	127,500	8.99	1880	806	69,600	7.8
1889	1405	121,400	9.09	1996	795	68,700	8.05
1975	1384	119,600	9.04	1952	759	65,600	7.65
1955	1283	110,900	8.95	1916	756	65,300	7.65
1939	974	84,200	8.26	1970	714	61,700	7.62
1993	964	83,300	8.48	1878	700	60,500	7.45
1909	943	81,500	8.19	1964	672	58,100	7.57
1906	893	77,200	8.06	1990	656	56,700	7.57
1974	884	76,400	8.15	1920	655	56,600	7.29
1894	883	76,300	8.03	1918	634	54,800	7.21
1921	856	74,000	7.95	1924	621	53,700	7.16

4.4. Probability Distribution

The time series used in the frequency analysis consists of the highest recorded value of discharge for each year of the record. Using a series of annual maximums lowers the risk of two successive peaks being dependent, and is recommended by ARR (2012). Observing the time series of monthly maximums showed that no year contained more than one major flood event, ensuring the annual series was not filtering out significant events.

As described in the previous section, the record of flood events at Deniliquin is made up of a period of continuous record, several earlier recorded events, and several events that are estimated to have occurred. Generally, a continuous record of around 100 years would be sufficient for use in a flood frequency analysis. However, in this case, inclusion of the earlier record is warranted given that four of the ten highest recorded floods occurred before the continuous record, including the highest flood on record in 1870. For this reason, several scenarios were tested, in order to determine what length of record was able to be approximated best by a probability distribution. The truncation points in the following scenarios were taken so as to remove the double curvature of the data; the probability distributions have a single curvature, so multiple distributions would be needed to fit the full set of data.

The scenarios are as follows:

1. Continuous record – Used 99 years of record (one gap filled) from 1913 to 2011
2. Continuous record (truncated at 18,300 Ml/d) – Used the same 99 years of record but only included events above 18,300 Ml/d.
3. Continuous record extended (truncated at 18,300 Ml/d) – Used the same truncated 99 years of data but included two additional events from 1906 and 1909.
4. Continuous record (truncated at 18,300 Ml/d) + 1867, 1870, 1889 events – Used the 99 years of record (truncated) as well as the three highest previous floods, from 1867, 1870 and 1889.

5. All recorded and estimated events (truncated at 31,000 Ml/d) – Used all events (recorded and estimated) since 1867 that were above 31,000 Ml/d.
6. Continuous record (truncated at 31,000 Ml/d) +1867, 1870, 1889 events – Used the 99 years of continuous data but only included events above 31,000 Ml/d, as well as the three highest previous floods (1867, 1870 and 1889).

A Bayesian maximum likelihood approach was used to fit a specified probability distribution to each of the scenarios. Two probability distributions were used; the Log-Pearson III (LP3), which is commonly used in flood frequency analysis, and the Generalized Extreme Value (GEV) distribution, which is a more recently developed family of probability distributions that combine the Gumbel, Frechet and Weibull families of distributions. Flike (file version 5) was used to apply the Bayesian maximum likelihood approach.

4.5. Results

Of the above scenarios, the fourth (4) was found to give the best results and its design flood discharge estimates were adopted. The scenarios were assessed on the basis of how well they could be approximated by a probability distribution, as well as the length of record they were based on (a longer record being better). Comparing the two probability distributions, the LP3 distribution was found to have better confidence intervals than the GEV. The LP3 fit of the adopted scenario is shown on Figure 5. Table 5 lists the time-series of annual peaks that comprised the scenario.

Table 5: Truncated Annual Series Used in Adopted Scenario

Year	Peak Flow (m^3/s)	Peak Flow (Ml/d)	Peak Height (m)		Year	Peak Flow (m^3/s)	Peak Flow (Ml/d)	Peak Height (m)
1870	2321	200,500	9.68		1918	634	54,800	7.21
1917	2189	189,100	9.63		1924	621	53,700	7.16
1956	1784	154,100	9.37		1960	609	52,600	7.33
1867	1640	141,700	9.27		1936	573	49,500	6.98
1931	1476	127,500	8.99		1932	550	47,500	6.89
1889	1405	121,400	9.09		1946	538	46,500	6.84
1975	1384	119,600	9.04		1951	505	43,600	6.69
1955	1283	110,900	8.95		1915	484	41,800	6.61
1939	974	84,200	8.26		2000	477	41,200	6.70
1993	964	83,300	8.48		2010	451	39,000	6.56
1974	884	76,400	8.15		1953	441	38,100	6.40
1921	856	74,000	7.95		1934	430	37,200	6.34
1958	855	73,900	8.16		1942	366	31,600	5.94
1981	850	73,400	8.19		1923	358	30,900	5.88
1973	841	72,700	8.10		1991	323	27,900	5.71
1992	819	70,800	8.11		1926	315	27,200	5.52
1996	795	68,700	8.05		1971	312	27,000	5.59
1952	759	65,600	7.65		1986	291	25,100	5.46
1916	756	65,300	7.65		1995	279	24,100	5.35
1970	714	61,700	7.62		2011	264	22,800	5.21
1964	672	58,100	7.57		1989	254	21,900	5.12
1990	656	56,700	7.57		1935	247	21,300	4.91
1920	655	56,600	7.29		1984	213	18,400	4.71

The flood frequency analysis produced a 1% AEP flow of 2204 m^3/s (190,400 Ml/d), lower than the previous estimate of 2500 m^3/s (216,000 Ml/d) from the 1984 flood study. The 5% and 2% year AEP flows were 1391 and 1861 m^3/s respectively (see Table 6). The decrease in the design discharges is primarily a product of the ~30 years of additional record, during which no floods occurred that would change the top end of the probability distribution. The modified analysis method, the inclusion of the 1917 event and the exclusion of the 1931 gaugings were also contributing factors to the change in the design discharges.

Table 6 Design Flows

AEP (%)	Discharge (m^3/s)	Discharge (Ml/d)
20	600	51,800
10	998	86,200
5	1391	120,200
2	1861	160,800
1	2204	190,400
0.5	2425	209,500

4.5.1. Use of Expected Probability for 1% AEP Flow

The 1% AEP flow was based on the ‘expected probability’ estimate, while the other design flows were based on the ‘expected parameter’ estimate. Both types of estimate are outputs of the flood frequency analysis, and the choice between them depends on what the intended use of the design discharge is. Generally speaking, an expected parameter estimate is chosen when the discharge is used as an arbitrary standard based on previous experience, while an expected probability estimate is more applicable to design for a site where underestimation of the design flood level would have large penalties, such as overtopping of a levee. In Deniliquin, the levee system is based on a 1% AEP flood level and is designed to protect the majority of the town in such an event. The importance of this structure has meant the expected probability estimate has been used for the 1% AEP discharge.

4.5.2. 0.5% AEP Flow

The 0.5% AEP flow has been estimated using the same flood frequency analysis used to estimate the more frequent design events. Although ARR 87 does not recommend using a flood frequency analysis to estimate design events above a 1% AEP, the increased length of record and updated estimation techniques qualify the analysis for estimating rarer events. Specifically, the 145 years of data available in this instance is greater than almost all record lengths available in 1987 (the date of the standard’s publication). Secondly, Bayesian techniques developed since the recommendation allow the non-continuous record to be included in the analysis, lengthening the period of record. For example, the 1870 flood is estimated as having a 1 in 242 year probability, an estimation that was not previously possible.

4.5.3. PMF Flow

The discharge for the PMF was approximated by tripling the ‘expected parameter’ estimate of the 1% AEP flow, which comes to 6499 m^3/s (3 x 2166 m^3/s), or 561,500 Ml/d (3 x 187,100

Ml/d). This represents an approximation of the largest flood that could conceivably occur in the area. As the estimate of its discharge is coarse relative to other methods, no estimate has been made of the probability of the event.

The estimate of the PMF discharge at Deniliquin may be revised in the near future, as it is expected that the dam-break analysis currently being completed for Hume Dam will include an estimate of PMF flows downstream of the dam.

5. HYDRAULIC ANALYSIS

5.1. Choice of Model

The Edward River produces a variety of flow behaviours, becoming more complex during flood events as the floodwaters spread beyond the main channel. The behaviour of the river changes significantly as it transitions between the main channel, the various flood runners, the high flow areas, and through urban areas with sub-surface drainage. The simulation of the interaction between these topographical features requires a fully two-dimensional (2D) flow model.

The TUFLOW modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water flow equations in both one and two dimensions. The TUFLOW software is produced by WBM BMT Pty Ltd (Reference 13). TUFLOW has been widely used for a range of similar projects both internationally and within Australia. The model is capable of dynamically simulating complex overland flow regimes. It is especially applicable to the hydraulic analysis of flooding in rural areas which is typically characterised by long-duration events with complex overland flow regimes. TUFLOW effectively models a combination of super-critical and sub-critical flow behaviour making it very suited to the assessment of flood control works and the overtopping of these.

For the hydraulic analysis of a complex system of flood runners a 2D model such as TUFLOW provides several key advantages when compared to a 1D model. For example, in comparison to a 1D approach, a 2D model can:

- provide localised detail of any topographic and/or structural features that may influence flood behaviour,
- better facilitate the identification of potential flood runners, floodways and flooding ‘hot spot’ areas.
- Inherently represent the available flood storage within the 2D model geometry.

Importantly, a 2D hydraulic model can better define the spatial variations in flood behaviour across the study areas. Information such as flow velocity, flood levels and hydraulic hazard can be readily mapped in detail across the model extent. This information can then be easily integrated into a GIS based environment for mapping and presentation purposes. Furthermore, the TUFLOW software provides a more flexible modelling platform to properly assess the impacts of any ‘hot spot’ management strategies for the study area.

5.2. Model Configuration

The TUFLOW model for Deniliquin is comprised of hydrologic data applied to a schematisation of the area’s topography. The primary input in representing the floodplain is the Digital Elevation Model (DEM), a grid of values that represent the topography of the area. This is then modified to include structures and processes not captured by the DEM, for example, culverts and subsurface structures, various levees, the cross-sectional geometry of the river, and the hydraulic roughness of the terrain. The hydrologic behaviour is described at the boundaries of the model – in this case a hydrograph at the upstream boundary and a stage-time relationship at the downstream boundary.

The TUFLOW model layout is shown on Figure 6, including the up and downstream boundaries, the model DEM, and 1D components, the section of the river that is based on the surveyed cross sections and the bridge structures. Each of these is described in detail in the following section.

5.2.1. Topography

Light Detection and Ranging (LiDAR) data was used to construct a 1 m DEM covering an area of approximately 220 km^2, centred on Deniliquin. This DEM was then re-sampled to a resolution of 10 m to meet the computational constraints of the model. A resolution of 10 m was considered an acceptable compromise between greater spatial resolution and reasonable computational runtime. The main channel of the Edward River is approximately 70 m wide and a number of flood runners on the floodplain are around 40 m wide, which allows both to be adequately represented with a 10 m resolution.

Because LiDAR data is processed to exclude non-ground points (e.g. buildings, trees or water surfaces), several areas in the DEM required modification to accurately represent the topography from a hydraulic perspective. These include farm dams and other small-scale reservoirs, flood runners adjacent to the main channel, and the Edward River itself. For the former, the DEM was raised to simulate the reservoirs being full, whilst the flood runners were ‘stamped’ into the grid in sections where their true depth had not been captured. The geometry of the main channel was inserted into the DEM by creating a Triangulated Irregular Network (TIN) based on nine cross-sections of the channel, originally used in the Deniliquin Flood Plain Management Golf Course Levee Report (February 1999). The channel geometry was uniform to the extent that, although the cross-sections only covered the middle 8 km of the 18 km stretch of river in the TUFLOW model domain, the end cross sections could be repeated to create a TIN that covered the full 18 km. The downstream cross-section was also compared to the next available cross-section in the MIKE-11 model used in the Edward-Wakool study (Reference 4).

5.2.2. Structures

5.2.2.1. Bridges

The National Bridge, which is part of the Cobb Highway and is located near the centre of Deniliquin, was represented using a layered flow constriction shape file. This designates an area for which the flow is constricted, e.g. the area under the bridge, where the bridge piers and abutments impede the river. The extent to which the flow is constricted is a function of the height of the river. For example, there is greater flow restriction when the water is at the level of the bridge deck than when it is below it. There are four such layers: underneath the bridge deck, at the bridge deck, at the bridge railing, and above the bridge railing. The bridge dimensions were based on survey drawings provided by Roads and Maritime Services.

The bridge crossing Brick Kiln Creek and the small bridge immediately after the National Bridge were also included in the model as layered flow constriction shapes.

5.2.2.2. Levees, Roads and Irrigation Channels

Lines of raised ground in the form of levees, roads and irrigation channels, form embankments that significantly affect the flood extent of large flood events that spill over the floodplain, which is otherwise flat and mostly without defined drainage paths. Because the crest of these features is much more narrow than the 2D model resolution (1 m or less versus 10 m), the crest elevation accuracy is reduced when the model re-samples the 1 m grid. This is caused by the re-sampling taking the average of 100 1 m^2 cells when determining the height of one 100 m^2 cell (10 m x 10m), of which the ‘crest’ cells are a small fraction. To account for this effect, and ensure the proper crest elevation is represented in the model, lines were inserted that raised the model terrain along any prominent raised features on the floodplain (such as levees, roads and irrigation channel banks).

The levee system that surrounds parts of Deniliquin was represented via modification of the DEM. The height and alignment of the levee (both present and historical) was taken from several sources, namely the data in the Levee Operation and Maintenance Manual (2011), the Deniliquin Levee Audit Report (1991), field survey undertaken by Council in April 2014 and the 1 m LiDAR for areas not covered by the surveys. Although the levee is well represented in the original 1 m resolution DEM, re-sampling it to 10 m resulted in sections of the levee losing their original elevation. A line representing the alignment and elevation of the levee was specified, and grid cells falling under this line were raised to better represent the structure. The current levee system includes a series of removable panels which are installed during a flood event. The levee, as represented in the model, assumed these panels were in place and represented the crest height accordingly.

The configuration of the levee in the hydraulic model was based on the assumption that when the levee overtops, it will do so without any structural failure. That is, it is assumed that once the river reaches a high enough level to overtop the crest of the levee, for example, in the 1% AEP event, the structure of the levee will remain intact. Other assumptions may be assessed as part of a floodplain risk management study for the area.

The survey data used to build the aforementioned DEM is shown on Figure 2 and includes the LiDAR survey, river sections, the various levees and the National Bridge.

5.2.3. Culverts and Channels

A series of culverts in the town were represented in the model as 1D elements nested in the 2D model domain. The culverts connect the remnants of a now urbanised flood runner. As the area they lie in is within the bounds of the levee, they only function in severe flooding, and are more important for local drainage. Given the insignificance of the culverts as a flow structure (from a flooding perspective) it was sufficient to estimate their invert levels and dimensions from LiDAR data and visual inspection.

The floodplain around Deniliquin contains several irrigation channels, the largest of which is Mulwala Canal, which passes underneath the Edward River via the Lawson Syphon. Although these channels carry a significant quantity of water, they do not influence the flood behaviour as a drainage line, as they are separated from the river during a flood. Either the channels are elevated above the river and water is taken from the river via a pump, the channels are connected to the river but use a floodgate that is closed during a flood, or they are not connected to the river (in the vicinity of Deniliquin), such as the Mulwala Canal. The features that influence the flood behaviour in the form of an obstruction are included in the model as detailed in Section 5.2.2.2.

5.2.4. Hydraulic Roughness

The frictional resistance the water experiences as it moves over a surface, known as hydraulic roughness, is represented in the model via each computational grid cell having a Manning's n-value. This simulates the difference in how the flood flows through a dense urban area compared to a densely vegetated area, which will have a higher Manning's n-value. The Manning's n-values were initially based on previous investigations and experience and were then adjusted, within reasonable limits, as part of the model calibration. Their spatial variation was based on analysis of aerial photography. The adopted values for design event modelling are listed in Table 7 while Figure 8 shows their spatial distribution.

Table 7: Manning's n-values

Description	Manning's n-value
Pasture/Grassland	0.06
Industrial	0.05
Dense Trees	0.08
Dense Urban	0.04
Rural residential	0.06
Swamp	0.04
Golf Course	0.04
Medium to Light Vegetation	0.06
Watercourses	0.03

5.2.5. Hydraulic Boundaries

The upstream boundary of the hydraulic model, which consists of a discharge time-series, uses both historical and design events as input. The historical events were used for calibration and validation of the model and were from 1956, 1975 and 1993. Each hydrograph has a single peak and a duration of approximately 13 days, as shown on Figure 9. The time-series data has been taken from Gauge No. 409003 ('Edward River At Deniliquin'), which, despite being 12 km downstream of the upstream boundary, gives an accurate representation of each flood event, as there is minimal attenuation between the two locations. The 1993 event was then used to create a hydrograph for each of the design events, by scaling its magnitude to the peak design flows found in the flood frequency analysis (see Section 5.5.1. for further information)

The downstream boundary of the hydraulic model is approximately 10 km downstream of the town and used a stage-time relationship. There is no record of stage in the vicinity of the downstream boundary and so an estimate was made. The estimate was an interpolation of the recorded height at the National Bridge gauge and the next gauge, No. 409023 ('Edward River Downstream of Stevens Weir'), which is 24 km downstream of Deniliquin. The large distance between the two gauges means that only a rough estimate can be made. In this respect, the stage value is not intended to strictly recreate the behaviour of the river at this point but rather to function in a way that ensures stable behaviour in the model without influencing the majority of the model domain.

5.2.6. PMF Model

The PMF event was modelled using a larger model area that incorporated a further 7.5 km of river at the downstream end. It was found that the initial model extent (that used for the other design events), did not have adequate floodplain area to contain the PMF event. The extended domain covers more area downstream and to the north of Deniliquin, areas where the PMF tends to spread out significantly. The larger model extent and its features are shown on Figure 7. The model followed the same principles in its schematisation, and contained the following features:

- A 15 m grid resolution. The resolution was made coarser to keep the model run-time reasonable (the area was approximately doubled),
- An extended bathymetric section, based on extrapolating the existing survey,
- Additional roads and irrigation channel embankments in the added downstream area, as well as roads and embankments that were not previously adjacent to flood extents,
- Wider upstream and downstream boundaries, with similar schematisation to that used in the non-PMF model (discharge-time relationship at the upstream boundary, stage-time relationship at the downstream),
- A wider upstream boundary, with a similar discharge-time relationship.

5.3. Calibration Events

Three events were chosen for use in calibrating the hydraulic model – the flood events of 1956, 1975 and 1993. These events are ranked as the third, seventh and tenth largest recorded events respectively. They were chosen based on them being the three largest events in the past 60 years (larger, previous events would have insufficient peripheral data) and their spread across the 60 year period (there was a large event in 1955, but this would be quite similar to the 1956 event in terms of the development on the floodplain).

Flooding on the Edward River is characterised by a slow increase in the river height over a period of several weeks, culminating in the peak flood level. The long duration over which the flood event occurs, combined with the large area covered in the hydraulic model, would result in a model simulation time of approximately 2 weeks per run, were the entire event to be run. To incorporate this build-up period while avoiding modelling it each time, it was modelled once per historical event and the flood extent across the floodplain was taken at the point 10 days prior to the peak and used as an initial water level grid in the peak period model run.

5.3.1. 1956 Event

5.3.1.1. Model Configuration

The 1956 flood at Deniliquin peaked at 9.37 m (91.80 mAHD) on the 17th of July that year. The peak level was converted to a discharge of 1756.2 m^3/s (151,700 Ml/d), using a stage-discharge relationship (rating table No. 76). The preceding 10 days of flow is used in the hydraulic model to simulate the event, as well as 48 hours following the flood peak, in order to allow the flow to move throughout the floodplain.

An extended simulation was used to determine the extent of the flood at the beginning of the simulation time, which was then applied via a grid of initial water levels. Velocity, depth and flow information from the extended simulation was not used, as they were only applicable to the DEM of the model they were generated from, and could therefore not be applied easily to different historical scenarios. The extended simulation covered the 30 days leading up to the peak flow. An initial water level grid was used which was approximately 92.9 mAHD at the upstream boundary and 86.1 mAHD at the downstream boundary. To approximate the velocities of the floodplain and the channel at the start of the model run, a 24-hour warm up period was included in the model run.

The model was amended in several areas to remove the changes that have occurred since 1956. The Mannings 'n' values were changed to reflect the evolution of land use, using aerial photos taken in 1955 as a guide. The levee was lowered to its former height, an estimate of which was made based on survey taken in 1991. Additionally, following initial model runs, the topography of the floodplain between the golf course and the National Bridge was changed to reduce the choke point occurring in this area. A comparison of modelled and observed flood profiles showed that the choke was over-represented in the model. The changes entailed widening Tarangle Creek, lowering the small peninsula of land on the river bank opposite Edwardes Street, and lowering the levee around Davidson Street.

5.3.1.2. Calibration Data

Five flood marks were available that represent the peak water level of the 1956 event. The flood marks, shown in blue on Figure 10, cover a 6 km stretch of the river and range from 92.387 to 91.045 mAHD in height. Further knowledge of the event comes from anecdotal evidence and newspaper reports from the time. These sources gave the following information:

- The main part of town was, on the whole, not inundated. The Wyatt Street levee was not overtopped. There was inundation in the vicinity of the golf course, as a rescue was made from a home surrounded by water near Memorial Park.
- The section of land between the Edward River and Brick Kiln Creek was severely flooded. Water was flowing over Davidson Street. There is no record of the depth, beyond that the Edward River Hotel had several feet of inundation, a photo shows a house with water to the roof level and that boating became a necessary means of transport to cross between North Deniliquin and the main part of town. Photos from the time show that the area was extensively flooded but do not give exact dates.

- The extent of flooding in North Deniliquin (north of Brick Kiln Creek) is not well-documented. The newspaper from the time reported that homes in North Deniliquin had been evacuated. The fact that there were crossings between North Deniliquin and the main part of town, as well as reports that emergency shops and a post office were set up in North Deniliquin, suggest the area was only partially inundated, if at all.
- Water spread overland in an easterly direction (opposite to the flow of the river) immediately south of Wakool Road near Racecourse Road and Burton St.
- The house at 215 Waring St in South Deniliquin was inundated with water.

5.3.2. 1975 Event

5.3.2.1. Model Configuration

The 1975 flood peaked at 9.04 m (91.47 mAHD) on the 5th and 6th of November, 1975. This level corresponded to a discharge of 1380.6 m^3/s (119,300 Ml/d) (derived from rating table No. 105). As with the 1956 event, the 10 days leading up to the peak were modelled, as well as the 48 hours following. As with the other historical events, an extended simulation was used to determine the initial water level, which was then applied via a grid of initial water levels. The extended simulation covered the 45 days leading up to the peak flow. This grid had a water level of 92.4 mAHD at the upstream boundary and 86.0 mAHD at the downstream boundary.

The Mannings 'n' values applied were based on the current extent and nature of urbanised land in Deniliquin, as no aerial photography (or equivalent data) of the town from around 1975 was available. The model DEM represented the various levees with the same height and alignment as those used in the 1956 simulation.

As with the 1956 event, floodmarks suggested that the current knowledge of the 1975 topography is incomplete and that this was causing a choke point in the model upstream of the National Bridge. The changes made to the '1956' model terrain were replicated, including widening Tarangle Creek, lowering the peninsula opposite Edwardes Street, and lowering the Davidson Street levee.

5.3.2.2. Calibration Data

Twelve flood marks were available that represent the peak water level of the 1975 event, shown on Figure 10. The flood marks cover a 6 km stretch of the river and range from 91.83 to 90.72 mAHD in height. Anecdotal evidence and newspaper reports give several pieces of information:

- North Deniliquin experienced some inundation, with the tennis courts near Brick Kiln Creek being covered with water before the flood peaked.
- Water came close to overtopping (or may have overtopped) a makeshift levee at the end of Burton Street.
- A small bridge on Memorial Drive, under construction at the time, was inundated.

5.3.3. 1993 Event

5.3.3.1. Model Configuration

The 1993 flood peaked at 8.48 m (90.91 mAHD) on the 18th of October, 1993, which is equivalent to a discharge of 963.8 m^3/s (83,300 Ml/d) (converted using rating table No. 110). The 9 days prior to the peak were included in the modelled hydrograph as well as the 72 hours following the peak. Compared to the 1975 and 1956 events, which had hydrographs of similar shape, the 1993 event had a less pronounced peak. That is, the several days prior to the peak flood level had flows similar to the peak flow. The extended simulation used to determine the initial water level of the event covered the 29 days leading up to the flood peak.

As with the 1975 event, the Manning's 'n' values were based on the current land use and the height and alignment of the various levees was based on survey of the levee system in 1991.

5.3.3.2. Calibration Data

No floodmarks were available for the 1993 event, and so the river height could only be compared to the community's responses to the questionnaire, which were also limited. The questionnaires rarely mention the 1993 event specifically, or if they do, do not refer to the flooding of any house or road in the area. From the general response, it can be ascertained that the Davidson Street area was not flooded and that the levee was not breached in any location.

5.4. Calibration Results

A range of parameters in the model were adjusted in order to calibrate the model. Adjustments were made in the following areas.

1. Hydraulic Roughness, particularly for the main channel of the river and the different riparian zones adjacent to the main channel. The sensitivity of varying roughness along the length of the main channel was also assessed.
2. The initial water level used. Following preliminary results, a 'warm-up' simulation was used to determine the initial water level (described in Section 5.3.1.1, for example)
3. The sensitivity to varying water level at the downstream boundary.
4. The sensitivity to varying levee heights in sections of the levee where the historical height of the levee is not definitively known.

Following this calibration process the model showed a strong fit to observed flood behaviour for the three calibration events. Results were compared to the recorded height at the town gauge, the set of flood marks for the 1956 and 1975 events, and historical reporting of each event in newspapers at the time.

5.4.1. 1956 Event

The TUFLOW model showed reasonably good correspondence to observed flood levels, both in terms of the gauge height recorded at the National Bridge, and the five observed flood marks in the town. Figure 11 shows the modelled and observed river heights for the 1956 event at the

National Bridge. As can be seen, the model achieves a very close fit for the week leading up to the flood peak, before deviating by around 0.15 m at the peak. For the four days around the peak, the river height was above 91.75 m. At this level, large areas of the Davidson Street precinct were inundated, as well parts of North Deniliquin, and the flow would be impeded by the urbanised floodplain (for example, buildings and raised roads). Given the limited data describing the topography of these areas in 1956, the fit is considered to be reasonable.

Figure 12 shows the modelled longitudinal profile compared to the five observed flood marks. The difference between the modelled and observed levels is given in
Table 8. The figure shows that the overall gradient of the flood profile is well replicated, with the observed levels dropping by 0.22 m/km and the model dropping by 0.20 m/km. The largest difference is at the flood mark recorded at the golf course, where the model is 0.24 m higher. In general, the model contains a slight drop in water level as the river enters the natural choke that occurs due to the Davidson Street area, which is not apparent in the flood marks. This is most likely a result of the flood mark not being on the main channel, and therefore being susceptible to localised influences. Its location is on Tarangle Creek (the creek that runs through the golf course), which most likely changed during the golf course's construction, changing the point at which the mark can be compared to the main channel. Historical survey data of the creek was not available, beyond parish maps which suggest it was once a more prominent waterway. The remaining points are around 0.1 m different, which is considered adequate given the event occurred over 50 years ago.

Table 8: Comparison to Observed Flood Levels - 1956 Event

Location	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
Rose Street	92.39	92.46	-0.07
Golf Course	92.07	92.31	-0.24
National Bridge	91.80	91.97	-0.17
Chippenham Park Road	91.22	91.29	-0.07
Harfleur Street & Wyatt Street	91.05	91.19	-0.14

Figure 13 shows the peak flood depth and height contours for the 1956 calibration event. The figure shows that the majority of the town centre (the area south of the National Bridge) was not flooded, save for small areas at either end of the urban area. Comparison to aerial photos from 1955 show these areas, that is, north-west of Wyatt Street and near Ross Street, were not developed in 1955 and so are unlikely to be reported on in the newspaper and in recounts of the event (although the account of flooding on Waring Street is matched). The Davidson Street precinct is almost completely inundated in the model, with depths ranging from 0.1 to 1 m, which is similar to what was reported at the time. North Deniliquin on the north side of Brick Kiln Creek is not inundated in the urban areas, but is surrounded by inundation of around 0.4 m. There is some evidence to support this in the form of an aerial photo taken in 1956 which shows inundation in this area. However, complete isolation of the area is not reported in newspapers from the time, and may be a possible inaccuracy of the model. The results show correspondence to the account of water flowing to the south of Wakool Road, with inundation occurring around the peak and then retracting in a westerly direction.

5.4.2. 1975 Event

Compared to the 1956 event, the 1975 event shows a better fit to the observed data. A comparison of the observed and modelled gauge height, shown on Figure 11, shows the modelled height to be within 0.1 m of the observed height for almost all of the event's duration. The modelled peak is 0.1 m higher than the observed level, which is considered a good fit.

Figure 12 and Table 9 show the modelled flood profile compared to twelve recorded flood marks. The profile shows a good match (around 0.05 m difference) to the observed levels downstream of National Bridge and the overall gradient of the river is very similar to the floodmarks. Similar to the 1956 event, the model comes furthest from replicating the flood mark at the golf course, although it is not as large a difference as for the 1956 event. As with the earlier event, the discrepancy is unexplained, beyond that the point is not from the main channel and the creek's function is likely to have changed during construction of the golf course.

Table 9: Comparison to Observed Flood Levels - 1975 Event

Location	Observed Level (mAHD)	Modelled Level (mAHD)	Difference (m)
Dick Street & Wick Street	91.83	91.92	-0.09
Golf Course	91.76	91.90	-0.13
Memorial Drive	91.65	91.75	-0.10
National Bridge	91.50	91.58	-0.08
National Bridge	91.49	91.58	-0.09
Macauley St & Riverside Dr	91.33	91.41	-0.08
Fowler Street	91.11	91.18	-0.07
Burton Street	90.95	90.96	-0.01
Chippenham Park Road	90.95	91.05	-0.10
Harfleur Street	90.9	90.86	0.04
Harfleur Street	90.83	90.86	-0.03
Edward Street	90.72	90.78	-0.06

Relative to the 1956 event, the 1975 event was confined to the main channel and the high flow zones, as shown on Figure 14. The figure shows general agreement between modelled flows and the newspaper reporting of the time. The Davidson Street tennis courts have 0.5 m of inundation, Burton Street came within 0.75 m of inundation and the bridge on Memorial Drive was under more than 2.5 m of water at the flood peak.

5.4.3. 1993 Event

The observed river height was closely matched by the model for the 1993 event, despite parameters not being further adjusted to calibrate the model. That is, no model parameters were adjusted in response to the calibration results. The event, which was modelled using topography very similar to what currently exists (the levee is higher and more extensive now compared to 1993), reached a lower peak than the two other calibration events and did not flood any roads or houses. Because less calibration data was available for the event, the Manning's 'n' used in the model was not adjusted from the 1975 model configuration (in this way the flood was closer to a validation event).

Figure 11 shows the observed and modelled river height at the gauge (which was moved to a new upstream location in 1981). The height is well replicated for the duration of the event, being around 0.15 m lower in the rising limb and less than 0.01 m higher at the peak flow.

Figure 15 shows the modelled extent of the 1993 event. While a significant part of the study area is inundated, the river was too low to exceed the overbank height or parts of a levee withheld the flow. Most of the flood runners are active, including Brick Kiln Creek.

5.5. Model Configuration – Design Flood Events

5.5.1. Hydrograph Shape

The design discharge determined by the flood frequency analysis represents the peak flow during a flood event on the river. This discharge is then combined with the hydrograph shape of a historical flood to produce a design hydrograph, which is used as input in the hydraulic model. There is no prescribed method for choice of hydrograph shape, beyond that it should be representative of the flood behaviour of the river.

A comparison of the eight largest floods for which hydrographs were available was used to assess hydrograph shape and volume at Deniliquin. The hydrograph shape of major floods on the river generally conform to the same shape and rate of rise, with a long rising limb leading up to the peak. They differ in whether they rise and then stay near the peak for several days (e.g. floods of 1931 and 1993), rise and then quickly recede from the peak (e.g. floods of 1975 and 1917) or rise more slowly but over a longer period leading to the peak (e.g. 1955 and 1956). The latter two categories have greater total volume, however, based on the long duration of the event (weeks) and the similarity in the total volume between shapes (20% maximum difference), the choice of shape is unlikely to have any significant effect on the peak flood level.

The 1993 shape was therefore chosen as being representative of the river, as it had a typical shape and is the most recent significant event. Once scaled, it resembles the 1956 and 1931 events for the majority of the hydrograph, whereas the 1975/1917 shape was relatively anomalous. The 1% AEP inflow hydrograph is shown on Figure 16.

5.5.2. Downstream Boundary and Initial Water Level

The height time-series of the downstream boundary was estimated by interpolating between the expected heights at Deniliquin and downstream of Stevens Weir. As with the calibration events, these are the locations of the two nearest gauges. In the absence of recorded water levels to interpolate between, the design hydrograph was used to estimate the level at both gauges based on their rating tables. Generally, the water level was between 2.5 and 3.5 m lower than the height at the National Bridge.

The initial water level was the same as that used for the 1956 calibration event. This initial water level grid was the highest of the three determined for the calibration.

5.6. Design Flood Results

Flood levels and extents for a range of design events were determined using the hydraulic model in combination with the design discharges produced by the flood frequency analysis. The flood frequency analysis produced estimates of the 20%, 10%, 5%, 2%, 1% and 0.5% AEP events, which were then assessed using the hydraulic model. Flood behaviour was also produced for the PMF event, which used a slightly different model schematisation (see Section 5.2.6).

5.6.1. 1% AEP Results

The 1% AEP flood peaked at 92.3 mAHD at the National Bridge and 92.5 mAHD at the location of the town gauge. Figure 17, which shows the peak depth and height of the event, shows that the South Deniliquin levee is not overtopped, but much of the Davidson Street area is inundated, and the North Deniliquin levee is breached in several points. Figure 24 shows the flood profile compared to the levee height (in South Deniliquin) and the river bed level, while Figure 25 shows the levee and design flood levels in North Deniliquin. Figure 16 shows the water level at the gauge over the duration of the event.

5.6.1.1. North Deniliquin Levee – Level of Protection

The levee that encloses North Deniliquin was designed to offer protection against events up to and including the 1% AEP flood. The majority of the levee was designed with a freeboard of 0.1 m (a section approximately 1.4 km long on the north-east side has a 1 m freeboard) and has a crest height based on the hydraulic analysis carried out as part of the Deniliquin Flood Plain Management Study (Reference 2). The freeboard was chosen following the Deniliquin Flood Protection Levee Study (Reference 3), which recommended a 0.1 m freeboard. The study found that a 0.1 m freeboard was most suitable as it considered several issues, including:

- It is necessary to evacuate elements of North Deniliquin when Davidson Street is inundated, due to access and water supply issues.
- The importance of access to the river for environmental and social reasons, which would be restricted by a higher freeboard.
- Lower damage costs associated with North Deniliquin.
- Geotechnical features in North Deniliquin which could cause seepage, meaning a higher freeboard may not offer more protection.

The current study found that the levee is overtopped in the 2% AEP event in three locations, each of which is only several metres wide. These locations are where survey has shown small dips in the levee crest height, and the height goes below the design crest height (for reasons unknown). The locations are:

1. Immediately upstream of the bridge over Brick Kiln Creek Bridge, where survey found the levee crest height to be 92.33 mAHD. This level is 0.05 m less than the 2% AEP peak flood level at that location.
2. At the south-west end of Box Street, where survey found the levee crest height to range

from 92.46 to 92.36 mAHD. This level is 0.1 m below the 2% AEP peak flood level at that location.

3. At the rear of 272 and 276 River Street, where survey found the levee crest height to dip to 92.44 mAHD. This level is 0.1 m below the 2% AEP peak flood level at that location.

In the 1% AEP event and larger, the North Deniliquin levee overtops at additional locations, and not solely as a result of localised dips in the existing crest height. The additional points at which the North Deniliquin levee is breached in the 1% AEP event are:

1. Near the west corner (near the intersection of Smart Street and the Cobb Highway), where there is a gap between the constructed levee and the natural levee the houses are built on.
2. Immediately upstream of the Brick Kiln Creek bridge.
3. A 600 m section located 300 m upstream of the Brick Kiln Creek bridge, parallel to River Street.

The levee height in these areas is generally set at a height of the 1% AEP level (as determined by the 1984 Floodplain Management Study) plus 100 mm.

The levee height and design flood level (both from the 1984 study and the current work) at the three locations is shown in Table 10 and on Figure 25 and Figure 26. The levee profile on Figure 25 shows the levee crest height compared to the range of design flood levels as well as the design height of the levee. This is the design level determined by the 1984 Floodplain Management Study, which does not include a freeboard. On the figure, the flood level profiles decrease up to around chainage 3700 m, at which point they then begin to increase. This is due to the profile being taken around the levee in a clockwise direction, and so the level increases along the north-east side of the levee when moving upstream. At Cobb Highway and Wanderer Street on Figure 25 there is a sudden increase in flood level of between 0.1 m and 0.6 m. This is due to these roads acting as hydraulic controls in large events, with the water pooling behind them until they are overtopped.

Table 10: North Deniliquin Levee Breaches

Location	Levee Height (mAHD)	1984 1% AEP Level	2013 1% AEP Level	Depth of Overtop (m)
1. West Corner	91.9*	92.02	92.1	0.2*
2. U/S Brick Kiln bridge	92.4†	92.35	92.7	0.3†
3. River Street	92.6 – 92.7†	92.42 - 92.53	92.7 – 92.8	0.1†

*Some sections in this area may require further confirmation, as current estimates are based on ALS data and scattered field survey between houses.

†This does not include the small dips in the crest level (see description of 2% AEP overtopping on previous page)

Except for Location 1, where the levee (or lack thereof)[1] is below the design level, the levee is breached as a result of a combination of the relatively small freeboard and as a result of a misapplication of the results from the previous modelling schematisation. The schematisation, which was considered sound practice at the time, models Davidson Street and the channels at

[1] Site inspection has found that the levee ends at Smart Street and there is a section of low ground south-east of Smart Street that does not contain a levee.

either end of it as a single cross-section. This results in a single design level being determined for this cross-section, that is, the 1% AEP level is the same at the National Bridge and at Brick Kiln Creek's bridge. Results from the 2D hydraulic model used in the current study suggests that for the 1% AEP event a gradient exists and there is a 0.35 m difference between the two locations. This is primarily a result of the river's sinuosity causing it to switch sides of the floodplain before the National Bridge, a feature which is not captured in the 1D model. Combined with this, the relatively low freeboard in North Deniliquin (100 mm) is not sufficient to accommodate any inaccuracy in the design flood level.

The overtopping of the levee in three locations in the 1% AEP event means the levee does not offer protection in this event. Although the crest elevation was based on the 1% AEP event, and the freeboard was chosen in order to protect against the 1% AEP event, the understanding of the flood gradient along the levee has been revised, and the relatively small freeboard is not enough to compensate for the changes. The floodplain risk management study that follows the current study should further assess the risk posed by the current levee height, as well as determine options for managing flood risk for the area.

5.6.1.2. South Deniliquin Levee – Level of Protection

The South Deniliquin levee was designed to offer protection against events up to and including the 1% AEP flood. It was designed with a freeboard of 0.5 m, except for at the NW end, which has a spillway and 0.2 m of freeboard, and the SE end, which has 1 m of freeboard. As with the North Deniliquin levee, the height, alignment and freeboard were determined as part of the Deniliquin Flood Plain Management Study (Reference 2) and the Deniliquin Flood Protection Levee Study (Reference 3), as well smaller studies which looked at smaller sections in more detail.

The current study found that the levee is not overtopped in the 1% AEP event, however, the freeboard was found to be less than 0.5 m in some sections (see Figure 24). While the levee was designed with a freeboard of 0.5 m, the peak flood level this was based on has been revised, which means the freeboard has changed in each location that the peak flood level has changed. Figure 24 shows the height of the levee compared to the 1% AEP peak flood level. As can be seen, the section between approximately Crispe St and National Bridge has less than 0.5 m between the levee and the flood level (The levee is 0.3 m higher at the closest point). There is also a section immediately downstream of the bridge where the levee is around 0.45 m higher than the flood level.

5.6.1.3. South Deniliquin Levee – Spillway Function

The South Deniliquin levee has a spillway located at its northern end that is designed to overtop before any other section of levee is breached, so as to provide a relatively safe and known breach. The spillway, which is 3.24 km long and begins at Poictiers Street, was designed with a freeboard of 0.2 m, while the rest of the South Deniliquin levee has a freeboard of generally 0.5 or 1 m. Unlike the rest of the levee, which is graded to correspond with the grade of the flood gradient, the spillway section has a constant height of 91.8 mAHD, except for the section between Poictiers St and Harfleur St, which is 91.86 mAHD. Two studies determined the alignment of the levee as it currently stands: the levee protection studies from 1997 (Reference 3) and 2008 (Reference 5), with the latter based partly on the former. The height of the spillway was proposed in the 1997 study and then revised in the 2008 study.

It was found that the spillway does not function as intended, in that it is not overtopped prior to the rest of the levee, and that when it does overtop, it is from water on the town side of the levee, flowing north-west and rejoining the floodplain. The spillway is not overtopped mainly as a result of the constant crest elevation, whereas the flood height decreases with the chainage of the levee. This is apparent on Figure 24, which shows the height of the levee compared to the 1% AEP peak flood height. The figure shows that the spillway is between 0.5 and 0.9 m above the flood level, which, coupled with the freeboard of less than 0.5 m in other sections (see Section 5.6.1.2), means the flood overtops at other locations before it does the spillway.

5.6.1.4. 1% AEP + 0.5 m

The area inundated by the 1% AEP peak flood level + 0.5 m ('1% + 0.5 m') has been estimated for use in flood planning, as shown on Figure 31. The topography of Deniliquin and the surrounding area made the estimation of this area less straightforward than in a typical catchment, where there is a well-defined 'valley' shape which confines the flood extent. In contrast, the terrain on either side of the Edward River is almost completely flat, meaning no features act as clear bounds on the floodwaters as they spread across the floodplain. Secondly, a conventional estimate of the 1% + 0.5 m extent assumes that an event may feasibly reach the extent estimated. In the case of Deniliquin, this would involve the water spreading laterally for up to ten kilometres from the river, which is very unlikely to occur, due to the finite volume of any event, despite the long duration of flood events. Thirdly, a conventional estimation method does not prescribe how the 1% + 0.5 m will spread around obstacles. The best example of this is the levee around South Deniliquin. Because there is no levee south-west of the town, there is no defined area that is protected by the levee (for events much larger than the 1% AEP event).

For these reasons, the area included in the 1% AEP + 0.5 m was estimated by simulating events larger than the 1% AEP, including the PMF. Firstly, an inflow was applied that would cause a flood level at the National Bridge 0.5 m greater than the 1% AEP peak at the same point. In this simulation, the levee around south Deniliquin was raised, as it was assumed the freeboard of the levee would protect the town. Similarly, the flood extent was taken when the water level above Lawson Syphon was 0.5 m above the 1% AEP peak (as it occurred significantly earlier than at the bridge). Using these results, the 1% + 0.5 m area was split into **four sections**, each of which had a different method to determine the 1% + 0.5 m extent. The areas are:

1. All area south of the Mulwala Canal/Lawson Syphon. The 1% + 0.5 m in this area is taken from the model simulation at the point when the water level at the syphon is 0.5 m higher.
2. North-west, north and east of the town. The 1% + 0.5 m in this area is taken from the model simulation at the point when the water level at the bridge is 0.5 m higher.
3. Downstream of the town, south of the river. The 1% + 0.5 m in this area is taken as the PMF extent in the area, as the PMF is approximately 0.5 m higher than the 1% AEP.

4. A small area between the third section and the town. In this area, the 1% + 0.5 m has been manually estimated by taking the area that is covered by the 1% peak flood level with 0.5 m added. The PMF could not be used in this area as it covers the entire area, including the town (whereas slightly west, the PMF is bounded by the Mulwala Canal)

5.6.1.5. 1% AEP + 0.1 m

In addition to the 1% AEP + 0.5 m, the area inundated by the 1% AEP + 0.1 m has been estimated. This may be used for comparison to the current flood planning level, which uses a level 0.1 m above the previous 1% AEP flood level. The 1% AEP + 0.1 m flood level is shown on Figure 32, which also shows the PMF extent and the 1% AEP + 0.5 m extent.

Estimation of the height and extent of the 1% + 0.1m was complicated by the same difficulties encountered in estimating the 1% + 0.5 m (see Section 5.6.1.4). For this reason, a similar approach was adopted, whereby the area was divided into sections, and the flood planning level in each section was estimated using flood levels from events larger than the 1% AEP. Specifically,

- the first section was the area located upstream of the levee at Harfleur and Wyatt Streets, which used the water level grid from a timestep from the 0.5% AEP simulation that was approximately 0.1 m greater than the 1% AEP peak, and
- the second section was the remaining area, and it used a water level grid from the PMF simulation that was approximately 0.1 m greater than the 1% AEP peak.

5.6.2. Other Design Events

Peak depths and levels were also produced for design events of 20%, 10%, 5%, 2% and 0.5% AEP, as well as the PMF. Figure 18 to Figure 23 show these results across the study area. Table 11 summarises the peak flood level at seven locations for each of the events. A short description of each event's flow behaviour is given below.

Table 11: Peak Flood Levels in Study Area for Design Events

	Peak Flood Level (mAHD)						
Location	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
National Bridge	89.3	90.9	91.6	92.1	92.3	92.4	93.1
Gauge Location	89.4	91.0	91.8	92.3	92.5	92.6	93.4
Brick Kiln Creek Bridge	89.5	91.1	91.8	92.3	92.5	92.7	93.3
River@Burton St	88.7	90.2	91.0	91.5	91.6	91.7	92.2
Tarangle Creek @Ross St	89.7	91.3	92.1	92.6	92.8	92.9	93.7
River@Lawson Syphon	90.7	92.1	92.7	93.2	93.4	93.6	94.7
River@Boggy Creek Rd	88.3	89.8	90.7	91.1	91.2	91.3	91.7

- The 20% AEP flood event does not spread far beyond the main channel of the river, except for several flood runners becoming active. For example Brick Kiln Creek transmits flow during the event, as well as Tarangle Creek and other small flowpaths.
- The 10% AEP event covers more of the high flow area (the vegetated areas adjacent to the floodplain), including a large section east of Carew St.

- The 5% AEP event breaches the Davidson St levee and spreads over the remaining high flow area, making the inundated area a flowpath running in the north-west direction. The Davidson Street Levee is overtopped by 0.1 m at a single point, where the levee elevation dips slightly. The 5% event also overtops the river banks immediately east of the end of Ochtertyre Street, spreading in a SE direction up until the levee, to a depth of around 0.2 m.
- The 2% AEP event inundates significantly more area than the more frequent events, as the water extends out of the high flow area and slowly spreads over the flat pastoral land, mostly downstream of the township. The Davidson Street levee is overtopped in several areas by up to 0.6 m, and the area is almost completely inundated at the flood peak. Almost all of the North and South Deniliquin levees are withstanding water at the flood peak, and the North Deniliquin levee is overtopped in three locations (see Section 5.6.1.1).
- The 0.5% AEP event has a flood extent not dissimilar to the 1% AEP event, and is around 0.15 m higher across the study area. The event inundates a section of land immediately south of the Mulwala Canal, flowing from west of Lawson Syphon up until Wirraway Drive. However, this inundation is dependent on the culverts beneath the Canal and also beneath the Cobb Highway, and as such will be quite different under a blockage scenario.
- The PMF event inundates almost the entire study area, as water spreads out from the main channel, including both sides of the Mulwala Canal. At the peak of the event, most of the study area is inundated to a depth of 1 to 3 m.

5.6.3. Comparison to 1984 Floodplain Management Study

A comparison of the current design levels to those determined in 1984 is pertinent given the basis of the levee system's crest height on those levels. The current study uses a revised 1% AEP design flow of 2204 m^3/s (190,400 Ml/d), 12% lower than the previous estimate of 2500 m^3/s (216,000 Ml/d). The 2% and 5% AEP events have decreased by 7% and 2% respectively. This would be expected to produce lower design levels than previously determined, particularly for the 1% AEP event. However, the levee in South Deniliquin and North Deniliquin (excluding Davidson Street) has been raised in the intervening 29 years, which would be expected to constrict flows and therefore raise flood levels, especially in the elevation range in the vicinity of the levee crest height. A third factor is the model schematisation, which has changed significantly, and has been shown to produce different results by way of incorporating more features of the floodplain (see Section 5.6.1).

Preliminary results show that 1% AEP design levels are virtually unchanged between National Bridge and Mclean Beach, while upstream of the bridge they are 0.1 to 0.2 m higher and downstream of Mclean Beach, 0.2 to 0.3 m lower. The level at the National Bridge is quite similar, with a difference of less than 0.1 and 4.5 km upstream of the bridge (the first cross-section in the earlier model) the new level is 0.07 m higher. In the section of the river downstream of the Mclean Beach Caravan Park the previous level is around 0.2 m higher.

The stage-discharge relationship (produced by either study) shows that the reduction to 2204

m^3/s (190,400 Ml/d) does not have a large impact as the gradient of the rating curve is relatively flat at that level. For example, reducing the flow to 1700 m^3/s (146,900 Ml/d) would only decrease the level by a further 0.35 m. The other differences in design levels are a result of the changed flood profile, which is a product of the model schematisation and the slightly modified topography. As identified previously, the current model's incorporation of more features, for example, the sinuosity of the main channel, storage areas on the floodplain, and local topographic features, can be expected to give a more accurate representation of the flow behaviour and flood profile.

Table 12 lists the height of the levee (both in South and North Deniliquin) at several locations as well as the 1% AEP level determined by the current study.

Table 12: Levee Height versus 1% AEP

	Location	**Levee Height**	**1% AEP Height**
South Deniliquin	Wakool Road	91.80	90.9
	Harfleur and Wyatt Streets	91.85	91.4
	Butler Street	92.40	91.9
	National Bridge	92.83	92.3
	Duncan Street	93.05	92.7
	Henry and Mitsch Streets	93.82	92.9
North Deniliquin	Davidson Street	92.33	92.6
	River and Yarra Streets	92.70	92.7
	Cobb Highway and Smart Street	92.26	92.1
	Augustus and Hyde Streets	93.47	92.4

5.7. Model Sensitivity

5.7.1. Climate Change

Human-induced climate change is expected to have (and to be having) an effect on rainfall intensities, and should therefore be incorporated in the assessment of design flood behaviour in a particular area. However, there is uncertainty over the ways in which climate change will manifest itself in Australia. In the case of flood estimation, there is uncertainty over how much rainfall intensities will increase by (in the long term), and how changes in other variables (e.g. evaporation and temperature) will influence runoff.

The impact of climate change on flood behaviour in the study area has been assessed by comparing the 1% AEP flood levels to those of the 0.5% AEP event. This comparison allows the sensitivity of the 1% AEP flood levels to the possible long term influences of climate change to be identified. This increases the estimated discharge from 2204 m^3/s (190,400 Ml/d) to 2425 m^3/s (209,500 Ml/d) (about 9%). Table 13 shows the increases in flood levels using the higher estimate (2425 m^3/s or 209,500 Ml/d).

Table 13: Climate Change Impact - 1% AEP vs 0.5% AEP Comparison

Location	1% AEP Level (mAHD)	Increase in level under 0.5% AEP event (m)
National Bridge	92.3	0.14
Gauge Location	92.5	0.13
Brick Kiln Creek Bridge	92.5	0.13
River@Burton St	91.6	0.1
Tarangle Creek @Ross St	92.8	0.13
River@Lawson Syphon	93.4	0.16
River@Boggy Creek Rd	91.2	0.07

The table shows the increase in flood levels will be between 0.07 and 0.16 m. The largest difference is near Lawson Syphon, where a channel perpendicular to the main channel has a greater impact with increasing flood level. The smallest difference is downstream of the town, while the river adjacent to the town itself has around 0.13 m difference.

5.7.2. Model Parameters

Model sensitivity tests were carried out for a range of model parameters. The aim of the analysis was to determine the impact that various model parameters have on design flood levels, and to gain general insight into the complexities of the modelling process. Knowledge of various parameters' influence on model results, as the well as the general model functioning, can then be used to inform flood risk management decisions.

The following alternative scenarios were simulated as part of the sensitivity analysis:

- The 1% AEP flow rate increased to 2722 m^3/s (235,200 Ml/d). This number is an upper estimate of the 1% AEP estimate produced by the flood frequency analysis, and is the value of the higher of the two confidence intervals shown on Figure 5. Although this confidence interval represents a highly unlikely estimate of the 1% AEP flow, it gives some indication of what may be considered an upper bound.
- Similarly, the 1% AEP flow rate decreased to 1814 m^3/s (156,700 Ml/d). This represents the lower of the two confidence intervals shown on Figure 5.
- The grid cell size decreased to 8 m (from 10 m).
- The adopted Mannings 'n' values increased by 20%
- The adopted Mannings 'n' values decreased by 20%
- The blockage of the National Bridge and other bridges increased to 50%.
- The downstream boundary water level raised by 1.2 m. This represents the upper bound of what water level can be estimated by interpolating between the historical water levels recorded at Stevens Weir and in Deniliquin.
- Similarly, the downstream boundary decreased by 3 m (the lower bound of the estimate).

The sensitivity of each of these parameters was measured by comparing the peak flood level at several locations in the study area. These results are shown in Table 14, which shows an increase in peak flood level as a positive value, and a decrease as negative.

Table 14: Sensitivity Results

	Difference in Peak Flood Level (m)							
Location	Increased 1% Flow	Decreased 1% Flow	8 m grid resolution	Increased 'n'	Decreased 'n'	50% blockage	+1.2 m boundary	-3 m boundary
National Bridge	0.24	-0.24	-0.02	0.17	-0.23	0.02	0.01	0.00
Gauge Location	0.24	-0.26	-0.02	0.15	-0.21	0.00	0.00	0.00
Brick Kiln Creek Bridge	0.22	-0.28	-0.04	0.15	-0.26	-0.02	0.00	-0.01
River@Burton St	0.17	-0.20	-0.03	0.12	-0.19	0.00	0.02	-0.01
Tarangle Creek @Ross St	0.24	-0.26	-0.02	0.16	-0.22	0.01	0.00	0.00
River@Lawson Syphon	0.32	-0.27	-0.01	0.21	-0.26	0.00	0.00	0.00
River@Boggy Creek Rd	0.12	-0.15	-0.02	0.08	-0.13	0.00	0.03	-0.02

Increasing the inflow resulted in higher flood levels across the study area, with smaller differences downstream of the town, where the floodplain is less confined and water spreads over a large area. The largest impact is at the Lawson Syphon, where a large section of canal is within the floodplain and perpendicular to the flow, and forms a larger obstruction as the water level rises and the floodplain widens. Similarly, the lower limit of the 1% flow gave flood levels around 0.25 m lower than the base case.

Increasing the model resolution to 8 m (from 10 m) slightly lowered the peak flood level across the study area. Given that the model has been calibrated using a fixed resolution (10 m), the calibration variables and some schematisation choices are partly a function of that cell size, and therefore the effects of changing cell size are not easily understood. That is, an 8 m model would most likely have slightly different roughness values (following its calibration) and so changing the cell size from 10 m to 8 m does not truly capture the relationship between cell size and peak flood level. Nevertheless, the decrease in flood level is quite small (around 0.03 m) which suggests low sensitivity.

Increased roughness represented the floodplain as being less efficient at conveying flow, resulting in higher flood levels, while 'smoother' roughness values decreased flood levels. Increased roughness caused an increase of around 0.15 m in peak flood level, while decreased roughness lowered levels by around 0.2 m. The largest difference in peak flood level was at Lawson Syphon, which, as mentioned, is a much more significant obstruction at higher flood levels (and vice versa).

A higher water level at the downstream boundary had a negligible effect on the peak flood level away from the boundary. The higher boundary caused less water to leave the model at each timestep, as there was a flatter hydraulic gradient near the boundary (compared to the base case). Similarly, a lower downstream boundary acted to force more water through the boundary, slightly lowering flood levels. The effects of both were negligible when comparing peak flood level at the town gauge.

6. REVIEW OF FLOOD RISK

6.1. Provisional Hydraulic Hazard Categories

Provisional hazard categories were produced for two design events (5% and 1% AEP) and are shown on Figure 27 and Figure 28 respectively. The two categories of hazard shown in the figures (high and low Hazard) are used to inform the management of flood risk in the study area, as they describe the severity of the flood at a certain location in terms of its depth and velocity. The provisional hydraulic hazard categories determined here are based on the method prescribed by the Floodplain Development Manual 2005, Appendix L.

Both figures show that the area of high hazard is fairly continuous, following the main channel and the high flow zone. Areas of low hazard are located on the fringe of the high flow zone in the 5% AEP event, and the wider floodplain in the 1% AEP design event. The 1% AEP also contains large areas of high hazard on the floodplain away from the main channel; these correspond to either small flood runners on the floodplain where the water is slightly deeper than surrounding land, or areas where floodwaters have pooled due to the embankments of roads and canals impeding the flow. A comparison to the 1% AEP peak flood depths shown on Figure 17 complements the location of the high hazard areas.

6.2. Hydraulic Categories

Hydraulic categories were also produced for two design events (5% and 1% AEP) and are shown on Figure 29 and Figure 30 respectively. Hydraulic categories describe the flood behaviour by categorising areas depending on their function during the flood event, specifically, whether they transmit large quantities of water (floodway), store a significant volume of water (flood storage) or do not play a significant role in either storing or conveying water (flood fringe). As with categories of hazard, hydraulic categories play an important role in informing floodplain risk management in an area.

Although the three categories of hydraulic function are described in the Floodplain Development Manual (Reference 11), their definitions are largely qualitative and the manual does not prescribe a method to determine each area. The Manual gives one indication of how to quantitatively differentiate floodway and flood storage, when it states that flood storage areas, when completely filled with solid material, will not raise peak flood levels by "*more than 0.1 m and/or would cause the peak discharge anywhere downstream to increase by more than 10%"*. The definition of hydraulic categories was determined by considering the velocity and depth in each cell in the model domain, and then applying the criteria that the floodway should be large enough so that blocking out non-floodway areas will only raise peak flood levels by around 0.1 m.

The use of velocity and depth to delineate areas of different hydraulic category follows the approach proposed by Howells et al. in their 2004 paper (Reference 12). At each grid cell, the peak velocity (v), peak depth (d) and their product (v*d) is considered, and the cell is categorised based on the following criteria.

1. *If both v*d > 0.25 and v > 0.25, then 'floodway'*
2. *If both v > 1 and d > 0.15, then 'floodway'*
3. *If neither of the above apply and d > 0.7, then 'flood storage'*
4. *Otherwise, 'flood fringe'.*

Applying these criteria produced an initial estimate of the hydraulic categories. The hydraulic model was then augmented to 'block out' all but the floodway, and the impact on the peak flood level was calculated. When the impact was significantly greater than 0.1 m, the floodway area was expanded, and the impact was re-calculated. The areas were expanded by first changing any 'islands' of non-floodway to floodway, that is, areas that are surrounded by floodway. Then flood runners were manually added to the floodway area, and their width was increased until they were sufficiently wide. Lowering the thresholds of v, d and v*d may also be used to select more area; however, this was not possible for the study area, as a number of features on the floodplain, including roads and irrigation canals, obstructed small flood runners, and so considering v, d or v*d does not produce any unbroken flood runner or flow path outside the high flow zone.

This iterative process produced a floodway that is approximately the smallest area that can be considered floodway while not having an impact significantly more than 0.1 m. The flood storage and flood fringe were the remaining areas. As the floodplain outside the 1% AEP floodway no longer has functioning flood runners or other features that are activated in a flood, the designation of flood storage was complicated somewhat. For example, outside the floodway, relatively deep areas are often where flows are detained behind a road, and not a feature that acts to store a significant volume of floodwater. One exception exists, where a series of channels still exists to the south-east of the town. These channels play a distinct role in the flood event, but do not convey enough flow to be designated floodway. Therefore, they have been designated as flood storage in the 1% AEP event (for depths greater than 0.7 m), while the remaining floodplain is classified as flood fringe. In the 5% AEP event, areas with greater than 0.7 m depth have been classified as floodway.

As a check, the percentage of flow conveyed by the designated floodway was measured at different sections of the floodplain. It was found that at the peak of the 1% AEP event, the area designated as floodway conveyed 97% of the flow at the National Bridge (with 3% of the flow passing outside the floodway, through North Deniliquin and to the north-east. Similarly, the floodway at Lawson Syphon conveyed 99% of the flow, and the floodway at Boggy Creek Road took 92%. Overall, the floodway conveyed more than 90% of the flow passing through the study area, and up to 99% in some sections.

The floodway's conveyance of the majority of the flow (in some sections, virtually the entire flow) is indicative of the topography of the floodplain around Deniliquin and the way in which it conveys floodwaters. The area between the established flood runners and the river (which is well approximated by the 5% AEP flood extent shown on Figure 20) conveys the majority of the flow, even in rare events. This is due to the remaining floodplain being extremely flat and having very few water courses. While 80% (of total flow) has been used to determine floodway in other studies, the floodplain's topography around Deniliquin allows no selection of a floodway that conveys 80% of total flow, while also satisfying the aforementioned encroachment analysis.

6.3. Points of Interest

Several locales in the study area were assessed for how floodwaters inundate them, including the rate of rise, inundation of access roads and other features, velocity of floodwaters, and the flood behaviour relative to the river height at the gauge (no. 409003). Figure 33 shows the nine locations designated as areas of interest in the brief, as well as the locations of particular occurrences in each area (e.g. where a road or property is inundated). These occurrences are described in Table 16 to Table 24. Hydraulic hazard in each area is described as the velocity-depth product (V.D), which is then related to a level of hazard using the categories given in Table 15. The table is proposed in ARR Project 10 and describes the risk to different groups of people, based on their height and mass product (H.M).

Table 15 Flow hazard regimes for children, infants and adults

<table>
<tr><th>DV (m^2s^{-1})</th><th>Infants, small children (H.M ≤ 25) and frail/older persons</th><th>Children (H.M = 25 to 50)</th><th>Adults (H.M > 50)</th></tr>
<tr><td>0</td><td>Safe</td><td>Safe</td><td>Safe</td></tr>
<tr><td>0 – 0.4</td><td rowspan="5">Extreme Hazard; Dangerous to all</td><td>Low Hazard[1]</td><td rowspan="2">Low Hazard[1]</td></tr>
<tr><td>0.4 – 0.6</td><td>Significant Hazard; Dangerous to most</td></tr>
<tr><td>0.6 – 0.8</td><td rowspan="3">Extreme Hazard; Dangerous to all</td><td>Moderate Hazard; Dangerous to some[2]</td></tr>
<tr><td>0.8 – 1.2</td><td>Significant Hazard; Dangerous to most[3]</td></tr>
<tr><td>> 1.2</td><td>Extreme Hazard; Dangerous to all</td></tr>
</table>

It should be noted that the rate of rise of floodwaters described for various locations in the following section is an estimate that is strongly tied to the hydrograph shape adopted for the design events modelled (see Section 5.5.1). Rates of rise will vary significantly with differently shaped hydrographs, and each flood event will have a unique hydrograph shape. The event the hydrograph shape was based on has an average rate of rise (in flood level) of 0.3 m per day in the 7 days leading up to the peak, and a maximum rate of rise of 0.6 m per day. In context, other historic events, have an average rise over the same period of between 0.1 and 0.3 m per day, and a maximum rate of rise of between 0.2 and 0.9 m per day.

The time taken for different areas to be isolated has been described as the number of days after the closest category of flood (i.e. Minor, Moderate and Major Flood) as defined in the 2011 Operations and Maintenance Manual for the levee.

6.3.1. Riverview Western

This area is on the north side of the river outside of the town, near Boggy Creek Road, and contains several properties adjacent to the river. The area is serviced by a single road (Boggy Creek Road) which runs north and connects with Dahwilly Road. There is a low point on the road a little over 1 km north of the river, where it crosses a flood runner.

Boggy Creek Road is first inundated at a gauge height of 8.39 m, which occurs in a 10% AEP event and above. In frequent flood events the water over the road rises around 0.1 m/day, whereas in rarer events it will be closer to 0.25 m/day. Flow across the access road is moderately hazardous for able-bodied adults (as per Table 15) at the peak of the 10% AEP event, with a V.D of up to 0.7. The peak V.D across the access road (where it crosses the flood runner) in the 1% AEP event is around 1.2, and as high as 1.9, which is extremely hazardous for an able-bodied adult to cross. Table 16 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

In a large enough flood (around 8.5 m at the gauge and above), the area enclosed by the horse-shoe shaped flood runner begins to be inundated. Flows will first spill out of the north end of the horse-shoe shaped flood runner, flowing south across the west half of the 'island' enclosed by the flood runner and the main channel. This western half will become inundated first, with the water flowing south, before the remaining dry land is inundated, at which point nearly all flow in the area will be travelling in a westerly direction (parallel with the main channel to the south).

Table 16: Flooding behaviour – 'Riverview Western'

Height (m)	Consequences	Point No.
8.39	Boggy Creek Road is inundated at its lowest point, where it passes over a flood runner, 1.1 km north of the river (89.6 mAHD)	1
8.88	The area enclosed by the horse-shoe shaped flood runner is around 50% inundated, including the road.	-
9.12	The access road is inundated to a depth of around 0.4 m at its lowest point.	1
9.40	The area enclosed by the horse-shoe shaped flood runner is almost completely inundated, save for some high ground, which the houses adjacent to the river are built on.	-

The area is first isolated when Boggy Creek Road is inundated, which occurs at a gauge height of 8.39 m, which is approximately 1.2 m above the 'Moderate Flood' level of 7.2 m. The rise in depth at the gauge from 7.2 m to 8.39 m takes approximately 3.5 days in the 10% AEP event.

6.3.2. Racecourse

This area is located on the western outskirts of town near Old Racecourse Road and contains several properties. It is largely flood free in the 1% AEP event, save for a small area to the east which floodwaters cover. In the PMF event, floodwaters flow from directly north of the area to eventually cross McCrabb Road and Francis Drive. When the area becomes inundated, there will be significant access issues, as most of the township will be flooded, and parts of the Mulwala Canal will likely be overtopped. This will connect the large inundated area to the south of the canal to the floodplain downstream of the town, isolating the remaining dry land. Rate of rise on Old Racecourse Road is up to 0.1 m/hour when it is first inundated, and 0.2 m per day after that. The rate of rise on McCrabb Road is around 0.2 m per day, while Francis Drive rises by around 0.05 m per day. Table 17 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

Table 17: Flooding behaviour – ‘Racecourse’

Height (m)	Consequences	Point No.
10.05	Water crosses Old Racecourse Road at a low section just east of intersection with Racecourse Road	2
10.73	Water crosses McCrabb Road	3
10.90	Water crosses Francis Drive	4

The area becomes isolated when Old Racecourse Road is inundated, which occurs at a height of 10.05 m at the gauge, which is approximately 0.9 m above the ‘Major Flood’ level of 9.2 m. The rise in depth at the gauge from 9.2 m to 10.05 m takes just over 1 day in the PMF event.

Hydraulic hazard in the area is low, with a V.D of up to 0.3 (low hazard to most) at the peak of the PMF event, except for a part of a remnant flood runner immediately south of Wakool Road, where the depth is slightly increased and the V.D is up to 0.7 (moderate hazard to adults).

6.3.3. Dahwilly

This is a large area spanning both sides of the river near the western end of the levee, containing urban residential areas. It is described here as three sub-areas: south of the levee (excluding Mclean Beach Caravan Park), Mclean Beach Caravan Park and north of the levee.

South of the River (excluding Mclean Beach Caravan Park)

On the south side of the river, the levee is not overtopped in the 1% AEP event and the areas inside the town are not inundated. On the outside of the levee there are several houses inundated to a shallow depth in the 1% AEP event, while others are raised above the natural ground level and are isolated. Properties outside of the levee in the area become isolated in the following order:

1. The land between Poictiers Street and Harfleur Street becomes inundated (gauge height of approximately 9.4 m). Approximately 3 properties isolated.
2. Around the same time, the land between Sloane Street and Ochtertyre Street is inundated. Approximately 2 properties isolated.
3. Most of the land outside the levee and west of Riverview Drive is inundated, save for small high points, on which most houses are built (gauge height of approximately 9.64 m). Approximately 30 properties isolated, including Big 4 Deniliquin Holiday Park.

Access issues for this area are complicated by the multiple exit routes from the area heading east into Deniliquin. Access roads to the areas in the above list are generally inundated soon after the area is inundated (as roads are generally slightly higher than surrounding land).

Hydraulic hazard in the area is low, with a V.D of up to 0.2 at the peak of the 1% AEP event, except for a higher V.D in a localised area between Harfleur Street and Poictiers Street where a local depression causes the water to pool to a greater depth, giving a V.D value of up to 0.9.

Mclean Beach Caravan Park

Mclean Beach Caravan Park has a levee that is approximately 90.5 mAHD at its lowest point. Once this is breached, the park becomes inundated and begins to transmit significant flow as the river rises. This occurs in a 10% AEP event at a gauge height of 8.50 m. This is 1.3 m above the 'Moderate Flood' level of 7.2 m. The rise from 7.2 m to 8.5 m takes 4 days in the 10% AEP event.

Hydraulic hazard in the area is significant, with a V.D of over 1.2 (highly hazardous to all body types) passing over most of the caravan park at the peak of the 1% AEP event.

North of the River

Access roads in the area north of Mclean Beach consist of a network of dirt roads, some of which are overtopped in frequent flood events. The nature of the network of roads, as well as the limited information on their function, means only a rough estimate of access issues can be made at this point, without further information on road heights and function. The inundation height has been given at two points on the access roads which appear to be critical. Dahwilly Road is north-west of the area and is first overtopped from water spreading northwards at a location just south of the rubbish tip, after which its inundation rises by up to 0.2 m per day in the 1% AEP event. Phylands Lane is inundated where a small flood runner passes over it flowing west, near the main channel. After this its inundation spreads quite quickly, as it runs perpendicular to the floodplain, with a rate of rise of up to 1 m per day in the 1% AEP event (depending on the shape of the hydrograph).

Hydraulic hazard in the area is significant, with a V.D of 0.4 over most of the area at the peak of the 1% AEP event (low hazard to adults, significant hazard to others) and a V.D of up to 1.5 in the flood runners that loop through the area (highly hazardous for all body types).

Table 18 gives the equivalent gauge height at these points (as well as Mclean Beach Caravan Park) and Figure 33 shows the location of the events described in the table.

Table 18: Flooding behaviour – 'Dahwilly'

Height (m)	Consequences	Point No.
6.71	Water crosses access road near Peuker Road (88.1 mAHD)	5
7.91	Water crosses Chippenham Park Road (89.9 mAHD)	6
8.50	Mclean Beach Caravan Park levee is overtopped.	7
8.76	Water crosses Phylands Lane (89 mAHD)	8
9.64	Water crosses Dahwilly Road (90.8 mAHD)	9

6.3.4. National Bridge

The area is centred on the National Bridge which joins Davidson St to the main part of Deniliquin. It includes the second, smaller bridge immediately east of the National Bridge, and the caravan park nearby, which is partially enclosed by the Davidson Street Levee. Once the caravan park area outside the levee is inundated, which has a rate of rise of 0.5 m per day in the 10% AEP event, the remaining part of the park is located on a peninsula of land, which itself is inundated in larger events, with a rate of rise of up to 1.3 m per day in the 1% AEP event.

The National Bridge is not inundated in the PMF event, while the adjacent levee on the South Deniliquin bank is not overtopped in a 0.5% AEP event. The bridge is around 5 m above the 1% AEP flood level, while the levee is around 0.6 m higher. The velocity in the main channel below the bridge is around 2 m/s. While the bridge is not inundated, the road to the east has shallow, low hazard inundation. Table 19 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

Hydraulic hazard in the area is high, with V.D of up to 1.7 in the 1% AEP event around where the Davidson Street levee is overtopped. Once over the levee, the hazard is not as high, with a V.D of up to 0.5 at the peak of the 1% AEP event (low hazard to adults, more significant to others).

Table 19: Flooding behaviour – 'National Bridge'

Height (m)	Consequences	Point No.
7.32	Deniliquin Riverside Caravan Park outside the levee first inundated	10
9.52	Small bridge overtopped (91.75mAHD)	11
9.53	Caravan Park inside the Davidson Street Levee inundated	12
9.76	'River Edward Hotel' levee breached (92mAHD)	13

6.3.5. Aerodrome

The area is centred on the aerodrome south of the town and does not contain residential areas. The area is not flooded in the 1% AEP event, nor is it adjacent to any floodwaters. In the PMF event, water spreads over the area south of the Mulwala Canal, breaking away from the main channel and high flow zone south of Lawson Syphon, with a rate of rise of 0.25 m per day in the PMF. This flow is bounded to the south by high ground and to the north by the canal and flows east to west. Closer to the PMF event peak, flows spread across the town and meet the north side of the canal, completely inundating the area, with a rate of rise of up to 0.3 m per day in the PMF. Table 20 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

Table 20: Flooding behaviour – 'Aerodrome'

Height (m)	Consequences	Point No.
10.25	Water crosses Wirraway Drive between Saleyards Rd and Mulwala Canal	14
10.42	Water crosses road leading to Aerodrome	15
10.62	Water crosses Wirraway Drive at low point SW of Cobb Hwy intersection	16
10.82	Water inundates Sports Stadium ground (near low point)	17

The sports stadium is not inundated until close to the PMF peak flood level (the floor level is not known so over-floor affectation cannot be determined for this study). It will be isolated in a PMF flood, as the town and area to the west will be inundated, and the area south of Mulwala Canal will be inundated, with the canal wall acting as a levee (wet side on the south). The ability of the canal wall to serve this purpose as a levee is not known, and there may be localised features which compromise this ability which have not been modelled, such as structural weak points, sections with potential seepage or local low points in the crest level.

The aerodrome itself is inundated by water banking behind the Mulwala Canal and slowly spreading south. The road leading to the aerodrome is overtopped several days before the peak of the PMF, limiting the ability of the aerodrome to be used in extreme floods.

Hydraulic hazard in the area is significant in the PMF event, when the area is completely inundated. The water banks across the south side of the Mulwala Canal, with a V.D of up to 0.9 (significant hazard to adults, extremely hazardous to others). On the north side of the canal the hazard is lower, with a V.D of around 0.1 (low hazard) except for a depression immediately north of the sports stadium, which has a V.D of up to 0.6.

6.3.6. Davidson Street

The area is north-east of the National Bridge and contains around 100 residential and commercial properties. The area has three distinct hydraulic functions during the flood event, with increasing levels of hazard. Initially, the river is lower than the levee and the floodwaters surround the area. Then, with large enough flows, the levee is breached near Jones Avenue and the area begins to fill with water, with more sections of the levee being overtopped as the river height increases. Once the water reaches the downstream side of the area, the area becomes part of the floodplain, transmitting the flow of the river. For example, at the peak of the 1% AEP event the entire area is conveying floodwaters, although with lesser velocity than the river, as the levee and buildings act to restrict the flow. (The time periods described will vary depending on the rate of rise of the hydrograph). The most hazardous areas are those where the levee is breached, with velocities of up to 4 m/s in the 1% AEP event.

In the 1% AEP event, the entire Davidson Street area is inundated at the peak flood level. The properties on either side of Davidson Street have water to a depth of between 0.5 m and 1.5 m, while Davidson Street is covered by around 0.75 m. The sewer pump station on Evans Street has a depth of inundation between 1.2 m and 1.6 m at the peak of 2% AEP event, and between 0.2 m and 0.6 m at the peak of the 5% AEP event. The effect of these depths on the station's functioning will require survey of the structure and the water level at which operating equipment is turned off.

Brick Kiln Creek becomes a significant flow path during the 1% AEP flood event, especially as part of the Davidson Street levee diverts flow into Brick Kiln Creek. The levee, along with that of North Deniliquin and the bridge between the two, reduces the capacity of the creek, causing a severe choke point. This is manifested in the steep hydraulic gradient through the narrow section, with a drop of 0.3 m over a 200 m section of the creek, and velocities of up to 2 m/s.

Table 21 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

Table 21: Flooding behaviour – 'Davidson Street'

Height (m)	Consequences	Point No.
9.18	Davidson Levee low point is overtopped (91.7 mAHD)	18
9.52	Second Davidson Levee low point is overtopped. (91.95 at caravan park)	19
9.62	Davidson Street Levee is overtopped at three points	-
9.62	Davidson Street low point is overtopped by 0.1 m (91.75 mAHD)	20
9.84	Davidson St is inundated to 0.5 m	20

Hydraulic hazard in the area is extreme, with V.D of up to 4 in the 1% AEP event around where the Davidson Street levee is first overtopped. Once over the levee, the hazard is not as high, with a V.D of up to 0.5 at the peak of the 1% AEP event (low hazard to adults, more significant to others).

6.3.7. Flanagans Lane

This area is north-east of North Deniliquin and contains several residential properties. In large events, flood waters spread out from the main channel and inundate the area. The floodwaters originate from the river spilling over the bank of the river around 3 km south of Flanagans Lane, which is followed by other breaches. The river width then spreads to the Riverina Highway, and the flow is directed north as it comes up against the North Deniliquin levee. The area acts as flood storage, as the waters slowly spread at low velocity. There are no well-defined flowpaths, causing the flood extent to fan out, guided by the raised ground of irrigation canals and roads. Rates of rise of floodwaters can be up to 0.8 m / day in the 1% AEP event (depending on the hydrograph).

In regards to the properties on Flanagans Lane, the water approaches from the south, banking across the irrigation canal along the lane, before it overtops the lane at two points: the intersection with Moonee Swamp Road, and a point around 500 m south-east of that intersection. The lane is then inundated across most of its length as the water spreads north, banking against the large irrigation canal about 500 m to the north.

The Ute Muster site is inundated from flood waters spreading north from around the Moonee Swamp Road/ Conargo Road intersection. In a large flood event, the site is almost entirely inundated over a period of 24 hours, while the low point on Conargo Road (see Table 22) is inundated at 0.75 m above the 'Major Flood' level. The rise from 9.2 m to 9.95 m at the gauge takes 3 days in the 1% AEP event. Floodwaters are then contained by the irrigation canal to the north and east of the site, causing waters to pool in the area, with a rate of rise of up to 0.6 m per day in the 1% AEP event (depending on the hydrograph). Evacuation out of the site appears to be the north along Conargo Road, as all land south and west of the site is inundated. Table 22 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

Table 22: Flooding Behaviour – 'Flanagans Lane'

Height (m)	Consequences	Point No.
9.85	Flanagans Lane and Moonee Swamp Road intersection is inundated (92.0 mAHD)	21
9.85	Flanagans Lane 500 m SE of Moonee Swamp Rd Intersection is inundated (91.9 mAHD)	22
9.95	Low point on Conargo Road on east side of Ute Muster site inundated (91.50 mAHD)	23

Hydraulic hazard near Flanagans Lane is generally low, with most of the area having a V.D of less than 0.05 at the peak of the 1% AEP event, save for very localised sections where the road is overtopped and the V.D is as high as 0.3. Similarly, at the Ute Muster site, the V.D is less than 0.05 at the peak of the 1% AEP event, with small sections of Conargo Road having a value of up to 0.5.

6.3.8. Eastern

This is a large area spanning both sides of the river near the eastern end of town, and includes the golf course. The area is centred on the river and its high flow area, which becomes fully inundated in a large flood event. On the South Deniliquin side of the river, the levee is not overtopped in large flood events, leaving half of the golf course not inundated. In the 1% AEP event, the buildings on the golf course and around Memorial Park are inundated with between 0.5 and 2 m of water with low velocity (around 0.5 m/s), which spreads out from the main channel and the flood runners, before flowing in a more uniform north-west direction, aligned with the floodplain. Further details of which buildings are inundated in which events is not able to be determined until floor level survey is taken.

Memorial Drive is inundated at a level of 7.94 m at the gauge (see Table 23), which occurs 0.74 m above the 'Moderate Flood' level of 7.2 m. The rise from 7.2 m to 7.94 m takes just over a day in the 10% AEP event. Rate of rise in the area is up to 0.5 m per day in the 1% AEP event.

The area in South Deniliquin near the golf course, which contains Dick Street, Harfleur Street, Henry Street, Burchfield Avenue, Ross Street, Packenham Street and Lucas Court, is not inundated in 1% AEP or 0.5% AEP events, but is inundated in the PMF event. Under the current levee height and alignment in South Deniliquin, overtopping will occur when the gauge height is approximately 10.42 m, at both Crispe Street and Edwardes Street at their north-east ends. This will be followed by other locations overtopping, and, in the PMF event, most of the urban area will be inundated within 24 hours. The area bounded by the levee, the remnant flood runner (now a lagoon) running diagonally through the town, and south-east of Napier Street will be affected soon after overtopping, as this section of levee is the first to overtop and the lagoon fills and restricts access to the area.

In North Deniliquin, the area around Coborro Street, Melon Street and River Street is affected by the North Deniliquin levee overtopping in the 1% AEP event and larger. In the 1% AEP, the levee is overtopped at several locations (see Section 5.6.1.1). The overtopping process in the 1% AEP event is shown on Figure 26. Once overtopped, water spreads across the area, eventually reaching the south-east part of North Deniliquin. At this point, access to the area is cutoff by flow surrounding the outside of the North Deniliquin levee (to a depth of around 0.8 m in the 1% AEP).

Hydraulic hazard in this area is varied, with a V.D of over 1.2 (extreme hazard to all) on the wet side of the levee (north and south) in the 1% AEP event, while in the same event North Deniliquin has a V.D of less than 0.1 (except for over parts of the levee that overtop). When the South Deniliquin levee is overtopped in a PMF event, the V.D is up to 0.6 (low hazard to adults, higher to others), except for in the remnant flood runner running diagonally through the town, which has a V.D of over 1.2 (extremely hazardous to all).

Table 23 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

Table 23: Flooding Behaviour – 'Eastern'

Height (m)	Consequences	Point No.
7.94	Memorial Drive inundated (near South Deniliquin Levee)	24
8.78	Buildings east of Memorial Park showground inundated	25
10.37	Crispe St Caravan Park inundated (from levee overtopping)	26
10.69	More than half of the town is inundated, access to dry land is cut off.	-
10.73	Most of the town is inundated, depth of water over the natural ground elevation is 0.1 – 0.6 m	-

6.3.9. Four Post

This area is south-east of the town, situated on Lawson Syphon Road and near the Mulwala Canal. It contains several properties. Flood waters in the area inundate the depressions on either side of Mulwala Canal, as well encroaching on the rear of a number of properties close to the high flow area of the river, off Lawson Syphon Road on Willow Drive, Pindara Lane, Amy Lane, Cooinda Lane (Point number 27 on Figure 33). South of the canal, there is around 1 m of water in the 1% AEP event (some of which passes through culverts underneath the canal) that isolates the properties in the area. North of the canal, Four Post Lane has just over 0.6 m of inundation and most properties are not flooded. A week before the peak of the 1% AEP event is reached, water in the area is rising at approximately 0.2 m/day, while closer to the peak it is around 0.05 m/day. Table 24 gives further description of flood behaviour and Figure 33 shows the location of the events described in the table.

The recreation camp on Greaves Road (YMCA Four Post Camp) is outside the study area and as such, the model domain does not extend to the area and flooding behaviour cannot be ascertained.

Hydraulic hazard in the area is varied, with a V.D of less than 0.1 at Lawson Syphon Road when it is inundated in the 1% AEP event. Closer to the channel, the small creek that runs past the end of Willow Drive, Pindara Lane, Amy Lane and Cooinda Lane has a V.D of over 1.2 (extreme hazard to all) in the 1% AEP event, and the area alongside the creek on the south-west side of it (approximately 100 m wide) has a V.D of around 0.4.

Table 24: Flooding Behaviour – 'Four Post'

Height (m)	Consequences	Point No.
8.68	Access road near Lawson Syphon inundated (92.2 mAHD)	28
9.39	Lawson Syphon Road inundated at low point, 550 m north of Mulwala Canal crossing (92.25 mAHD)	29
9.69	Lawson Syphon Road inundated at second low point, 350 m south of Mulwala Canal crossing (93.15 mAHD)	30

7. CONCLUSIONS

The current study has been carried out to determine the design flood behaviour in the Deniliquin LGA. The design flood levels are a revision of those determined by the floodplain management study completed in 1984, which were used as the basis of the town's levee's crest height. The re-assessment was warranted given the substantial changes to the hydrological and hydraulic analysis techniques that have occurred in the past 30 years, as well as the additional 30 years of hydrologic data.

The updated analysis techniques consisted of a flood frequency analysis that used more advanced fitting techniques and a more thorough assessment of the impact of large events that occurred prior to the continuous record. These changes, as well as the additional 30 years of record, resulted in a lower estimate of the design discharges compared to what was previously estimated. For example, the estimate of the 1% AEP discharge was reduced from 2500 m^3/s (216,000 Ml/d) to 2204 m^3/s (190,400 Ml/d).

A hydraulic model based on the TUFLOW software was used to determine the design flood behaviour. The model represented the topography of the floodplain in detail and was calibrated to three historical events. The model incorporated a number of features that were not included in the 1984 study and so produced a more detailed and accurate representation of the flow behaviour in the river. This was especially apparent in the vicinity of the town, where the main channel of the river moves from one side of the floodplain to the other, creating complex flow patterns around the Davidson Street area.

The height and extent of the 1% AEP design flood was determined using the calibrated hydraulic model. The significant reduction in the design discharge only translated to a small decrease in the design flood level in the town. The North Deniliquin levee was shown to be overtopped during the event, despite the reduction in the 1% AEP flood level. This is because the schematisation of the earlier model type (that of the 1984 study) was not detailed enough to capture the difference in flood height at either end of Davidson Street, and so the level at the North Deniliquin end was underestimated. Furthermore, the relatively low freeboard applied at North Deniliquin (0.1 m) does not accommodate the inaccuracies resulting from the model schematisation.

The hydraulic model was also used to map provisional hydraulic hazard and hydraulic categories for two design events (1% and 5% AEP) as well as to determine the area covered by the 1% AEP event + 0.5 m. The design flood behaviour was used to describe the flood behaviour in nine areas of interest, which will inform consideration of flood access issues and emergency planning.

The definition of new design flood levels and extents will allow for detailed assessment of the flood liability across the study area. This will include both the areas inside the north and south levees, and the areas outside of the levee where future development may occur. They will also provide the basis for further assessment of the levee's ability to protect against flood events.

8. ACKNOWLEDGEMENTS

WMAwater wish to acknowledge the assistance of Deniliquin Shire Council staff in carrying out this study. WMAwater has prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. This document does not necessarily represent the opinions of the NSW Government or the Office of Environment and Heritage.

Further assistance was gratefully received from the local Office of Water, including information and discussion of flood behaviour, gauging methods and early flood heights.

This report's cover photo is courtesy of user 'Mattinbgn' on Wikimedia Commons.

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Figures

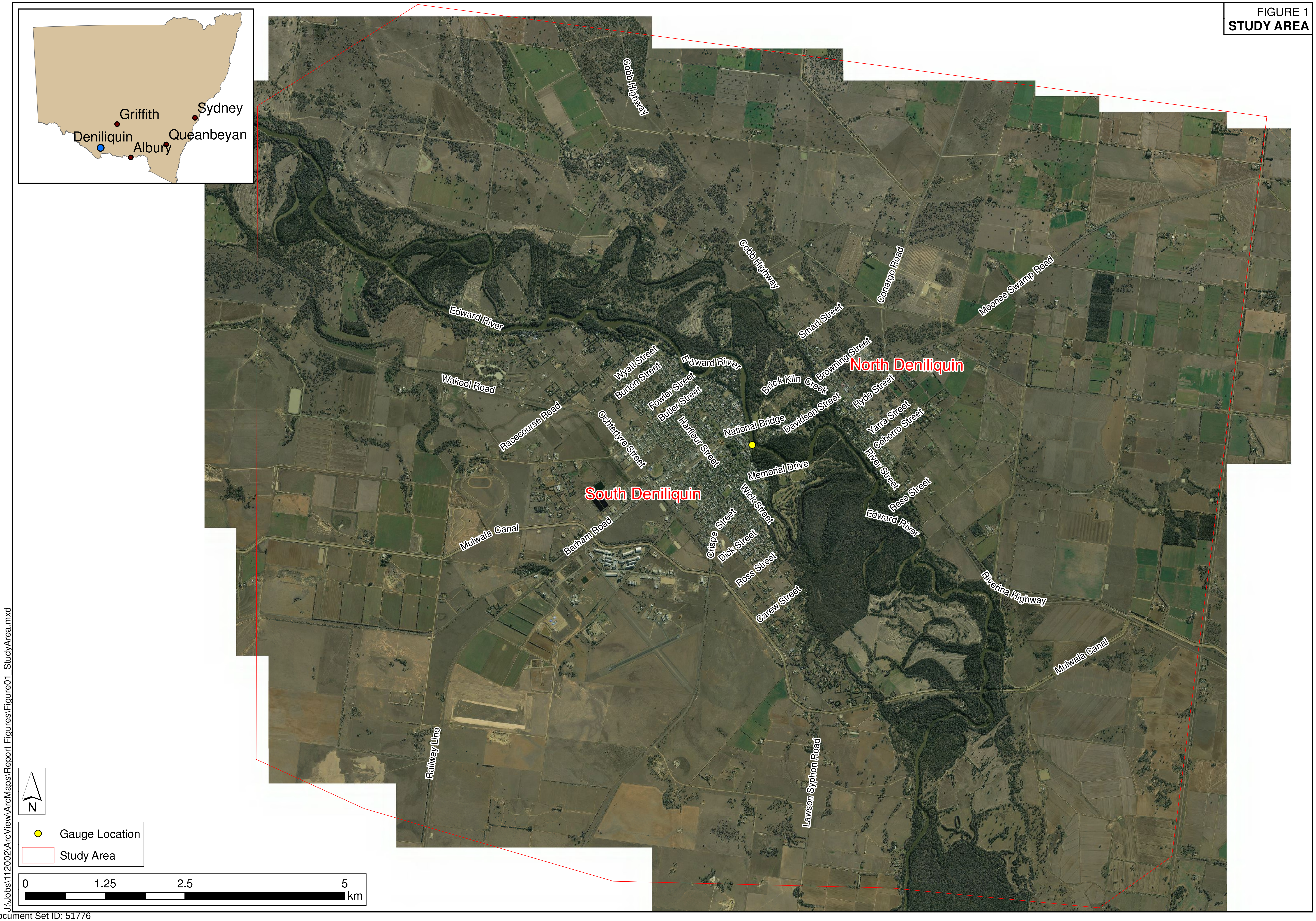

FIGURE 1
STUDY AREA

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FIGURE 2
AVAILABLE SURVEY DATA

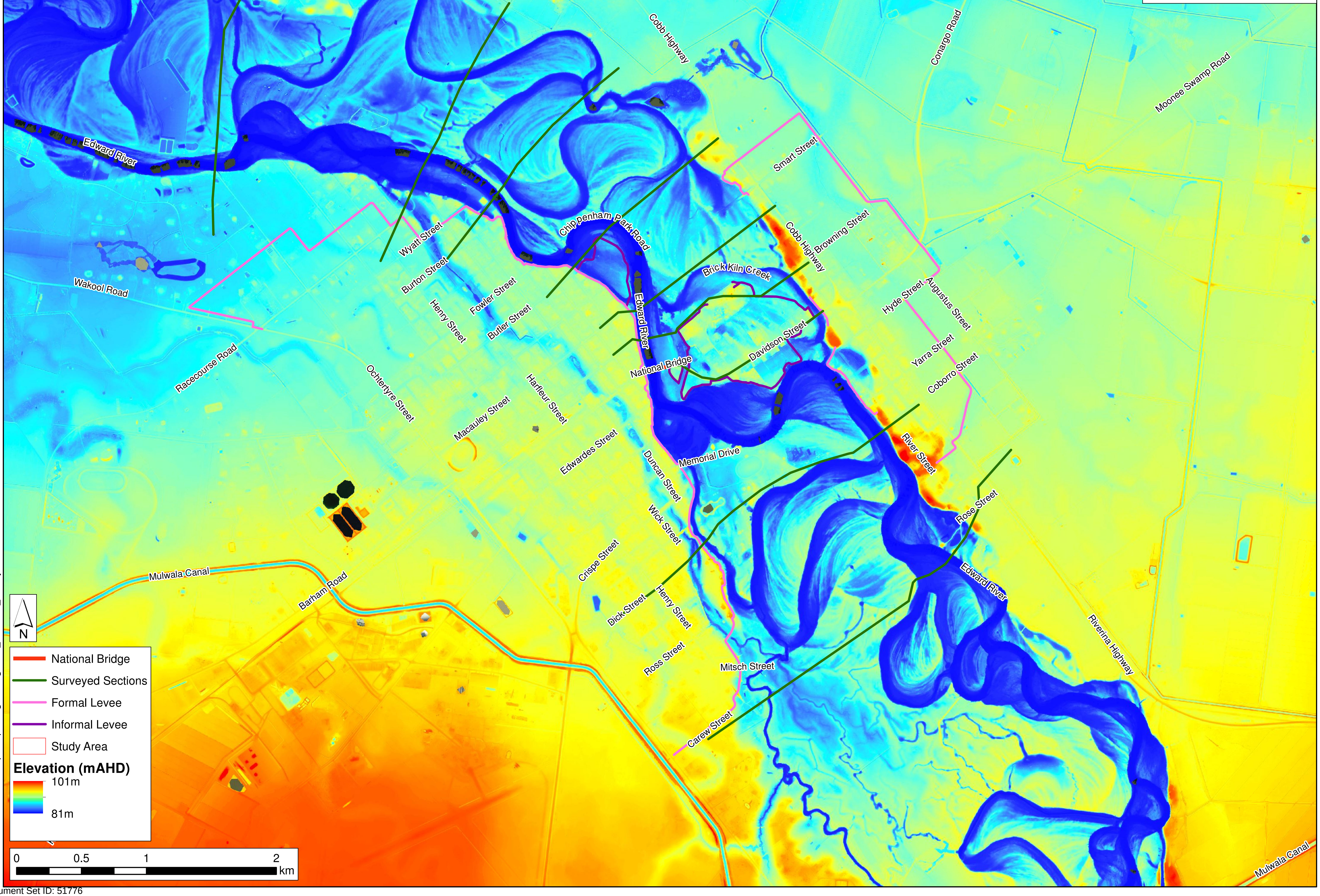

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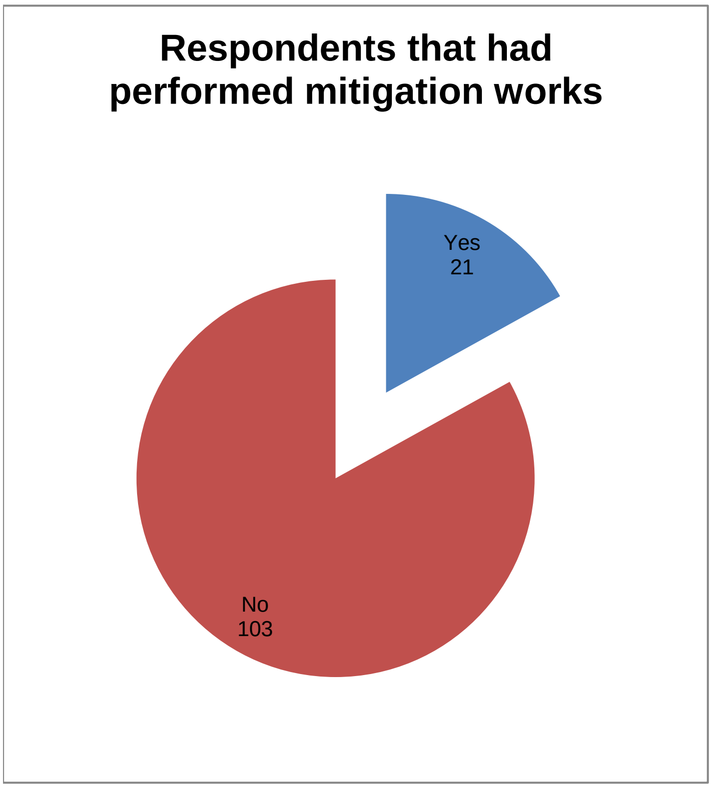
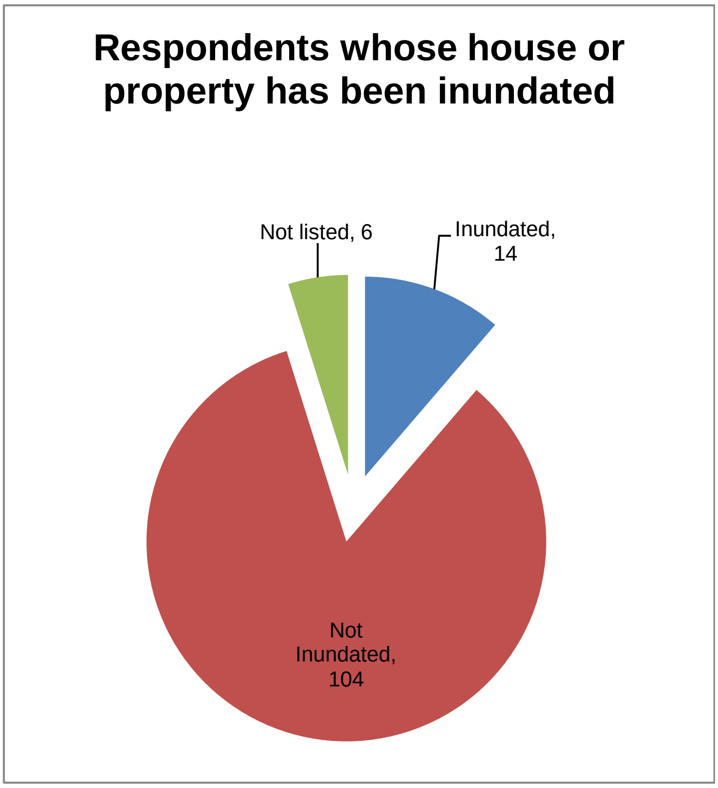
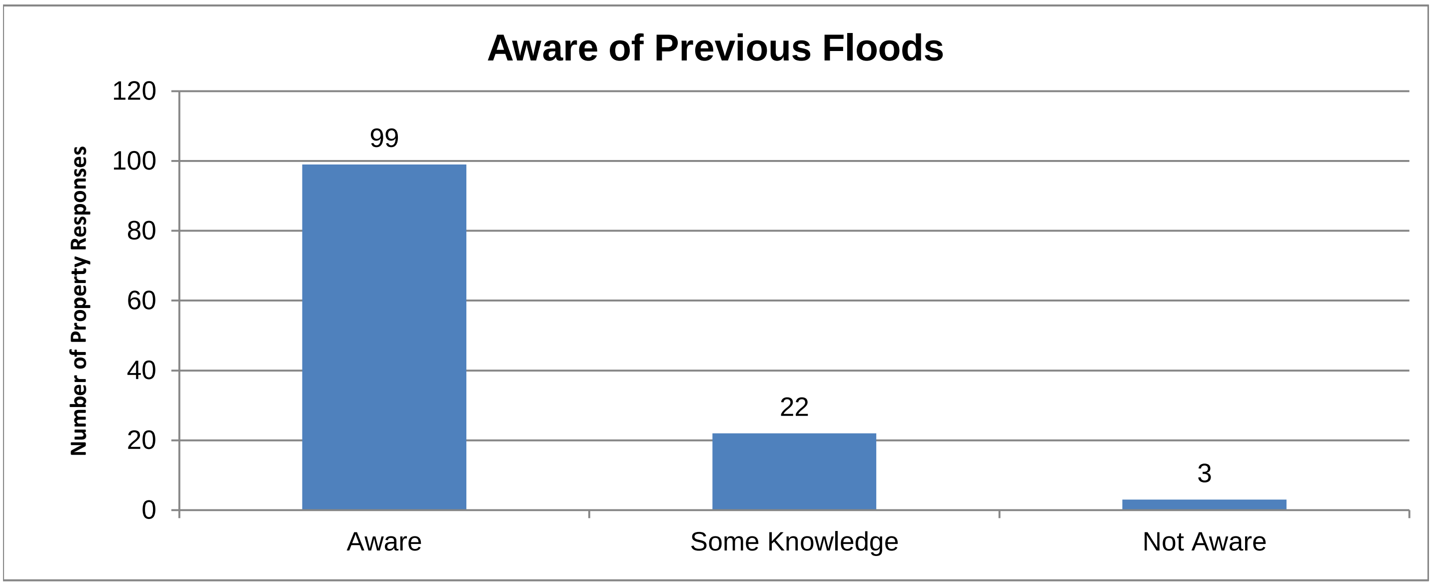
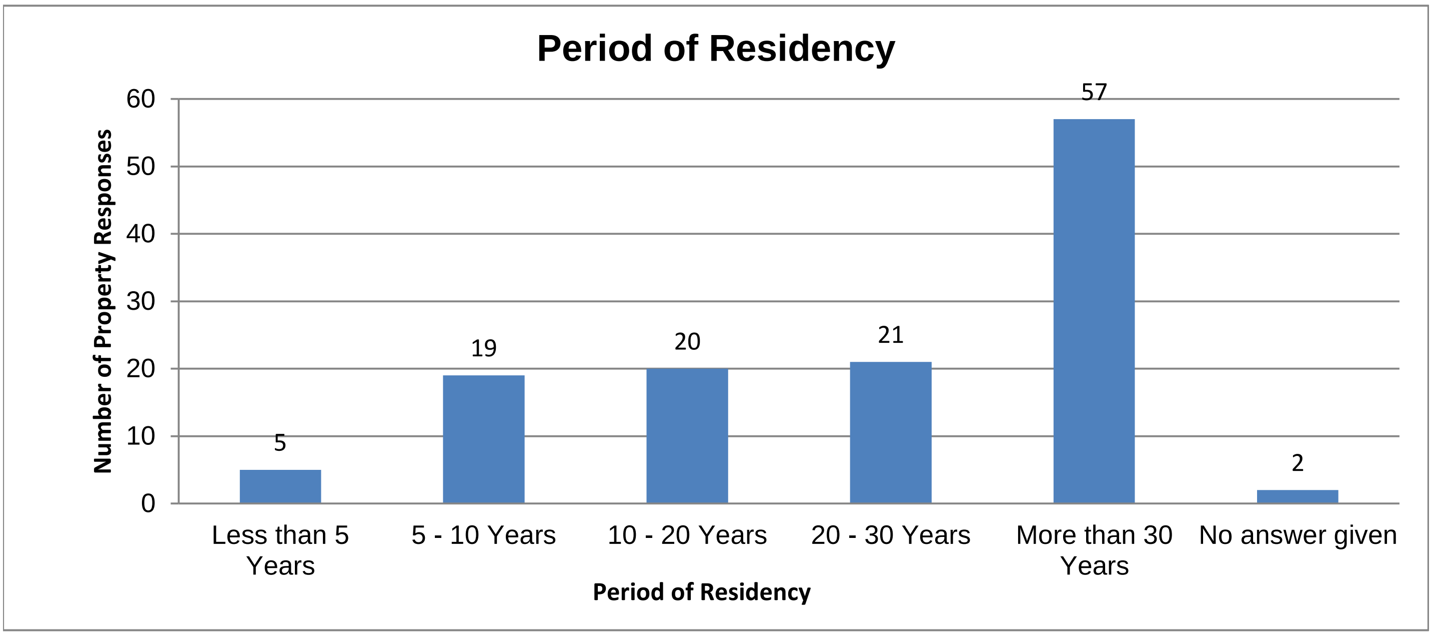
FIGURE 3

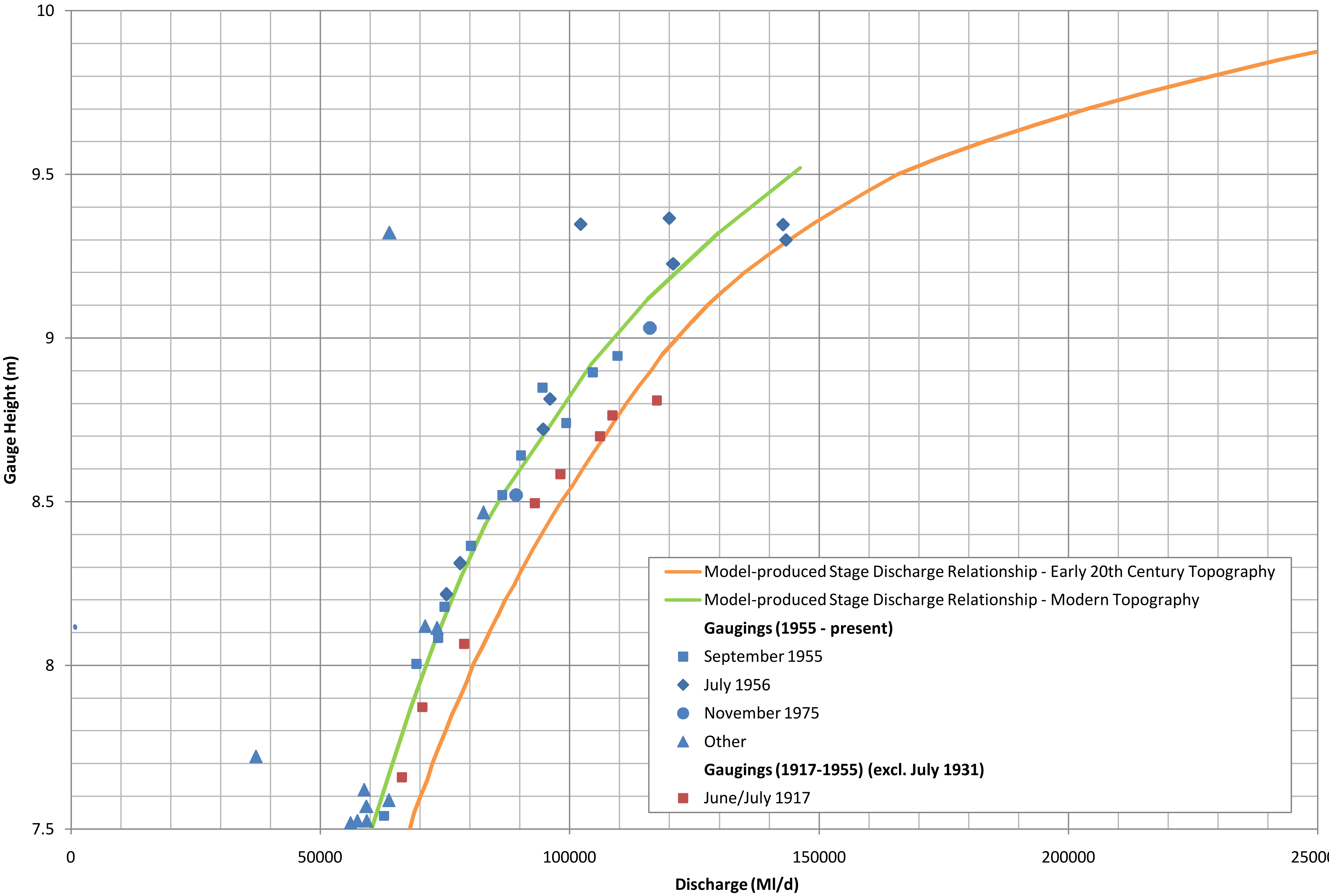
COMMUNITY CONSULTATION - ANALYSIS OF RESPONSES





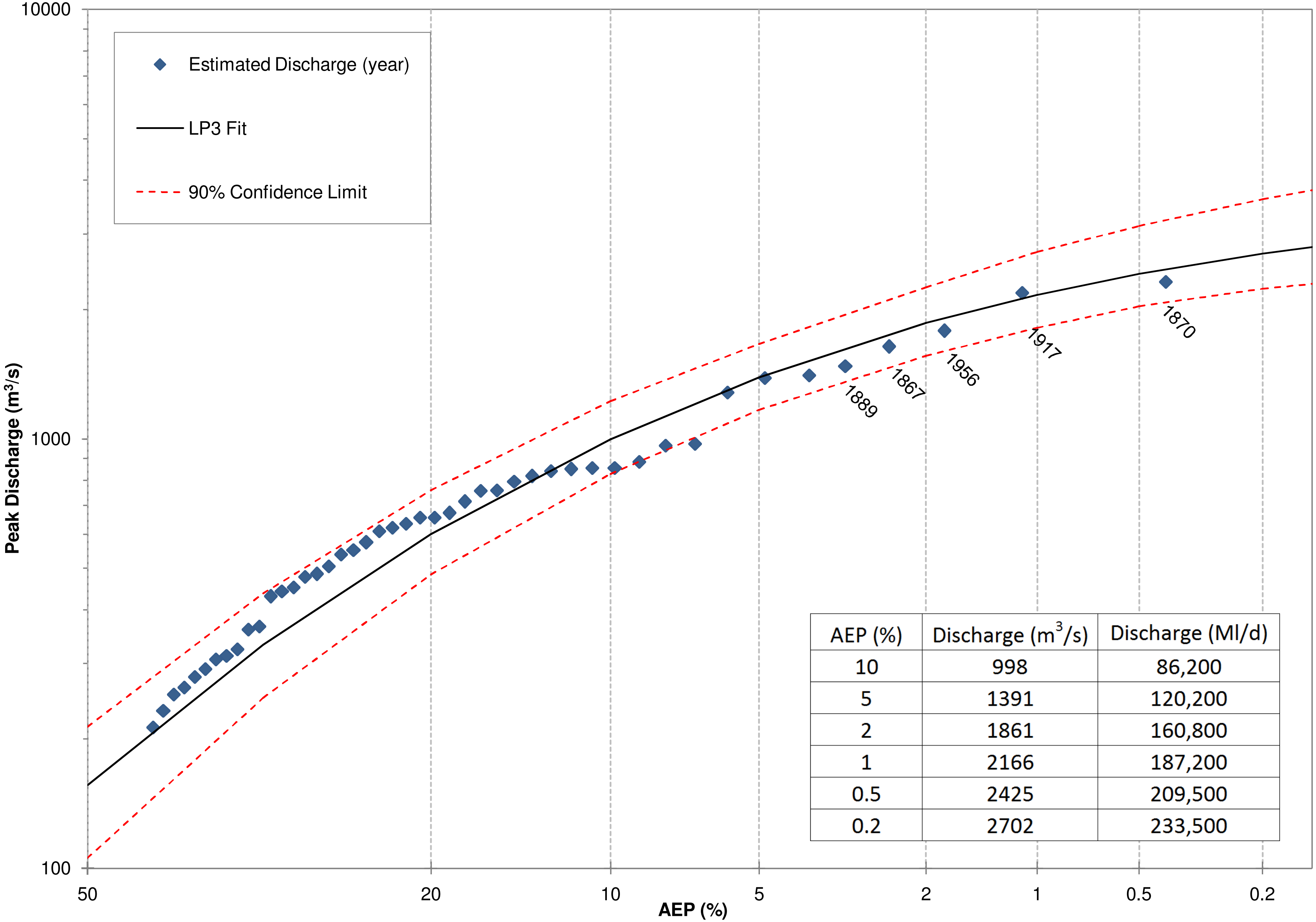






STAGE DISCHARGE RELATIONSHIPS - MODERN AND EARLY 20TH CENTURY TOPOGRAPHY

FIGURE 4



AEP (%)	Discharge (m^3/s)	Discharge (Ml/d)
10	998	86,200
5	1391	120,200
2	1861	160,800
1	2166	187,200
0.5	2425	209,500
0.2	2702	233,500

FIGURE 5
LP3 ANALYSIS
ADOPTED SCENARIO (CONTINUOUS RECORD PLUS THREE EVENTS)

FIGURE 6
TUFLOW MODEL LAYOUT

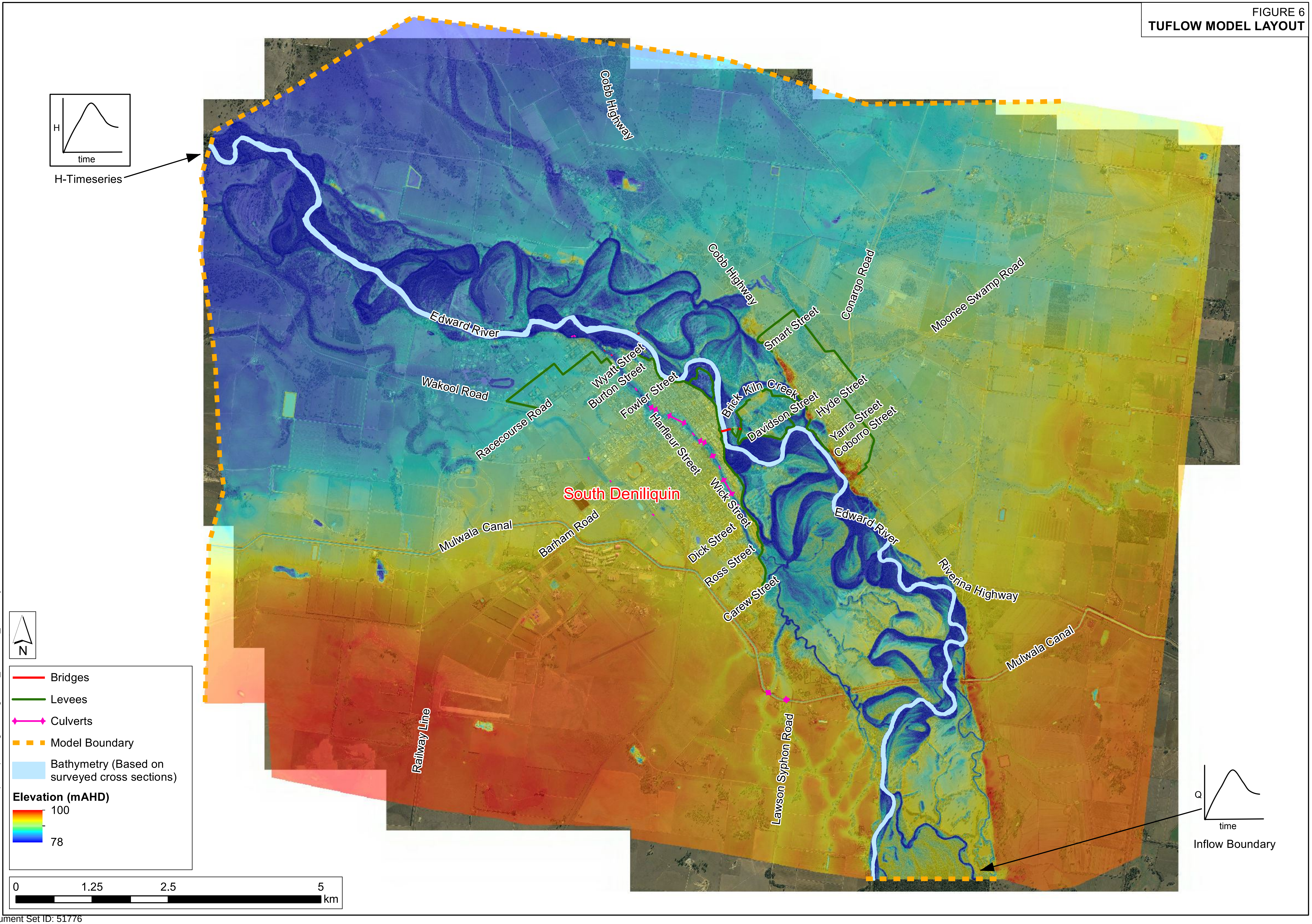


FIGURE 7
TUFLOW MODEL LAYOUT - PMF

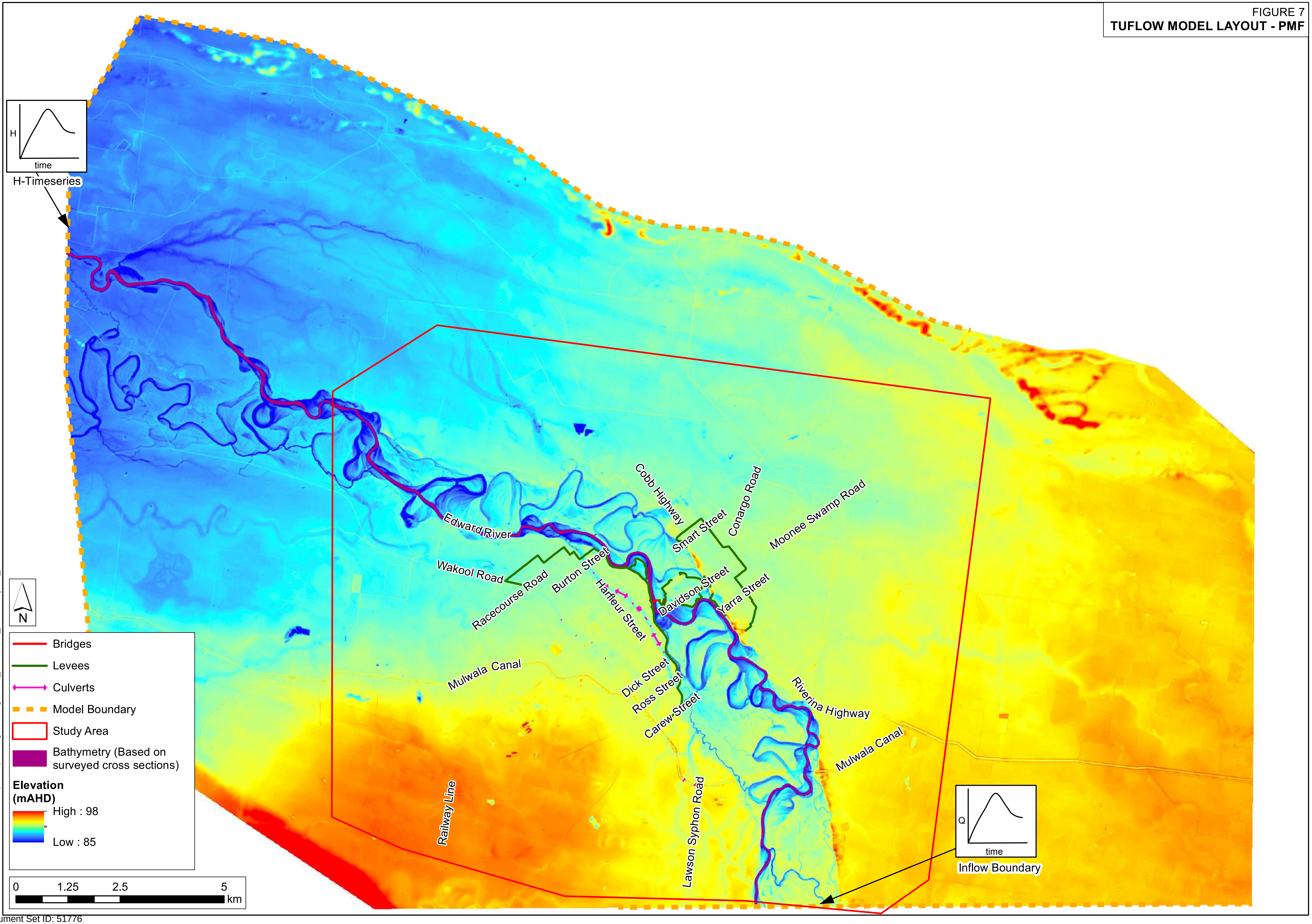


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FIGURE 8
MANNINGS 'N' LAYOUT

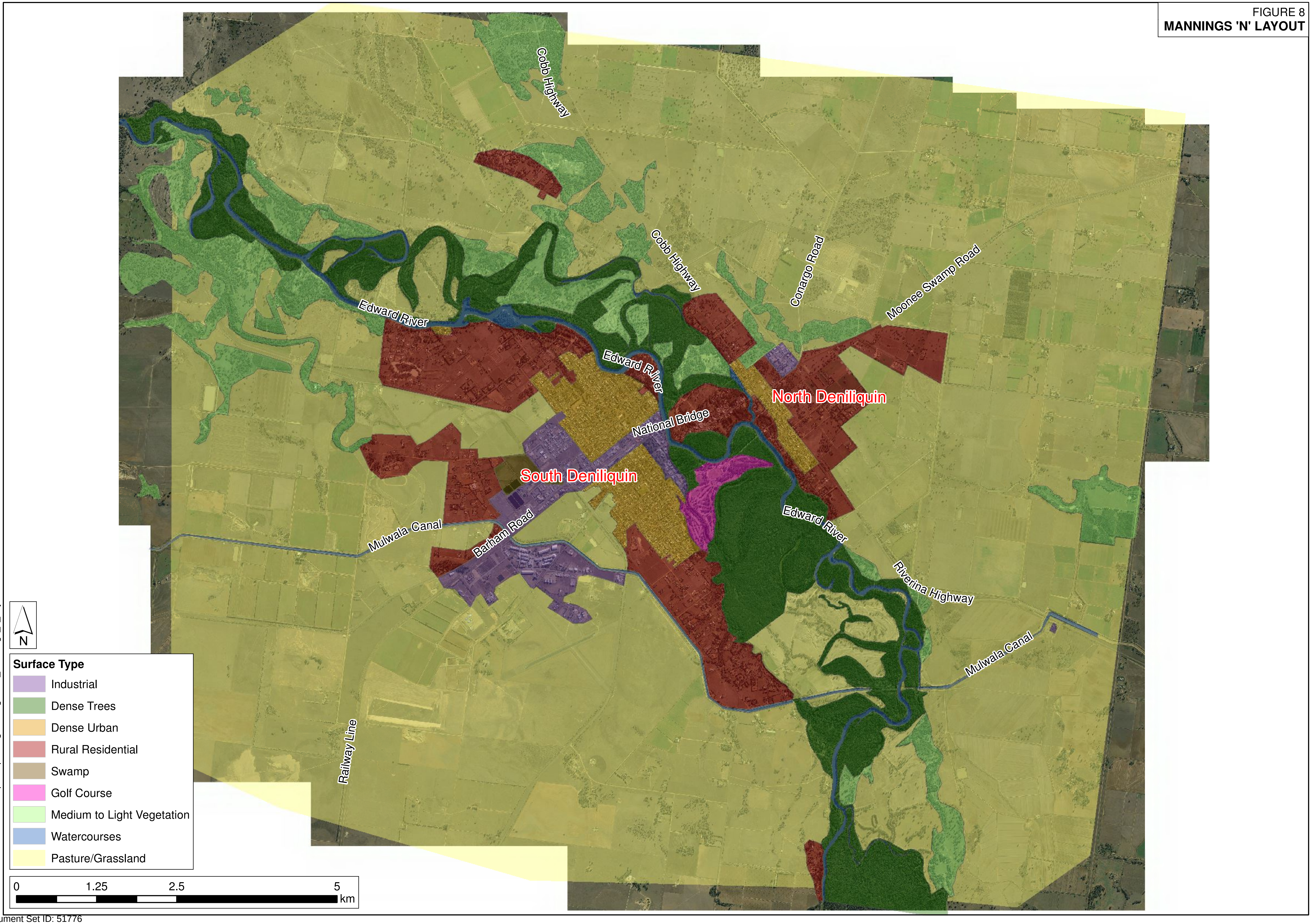

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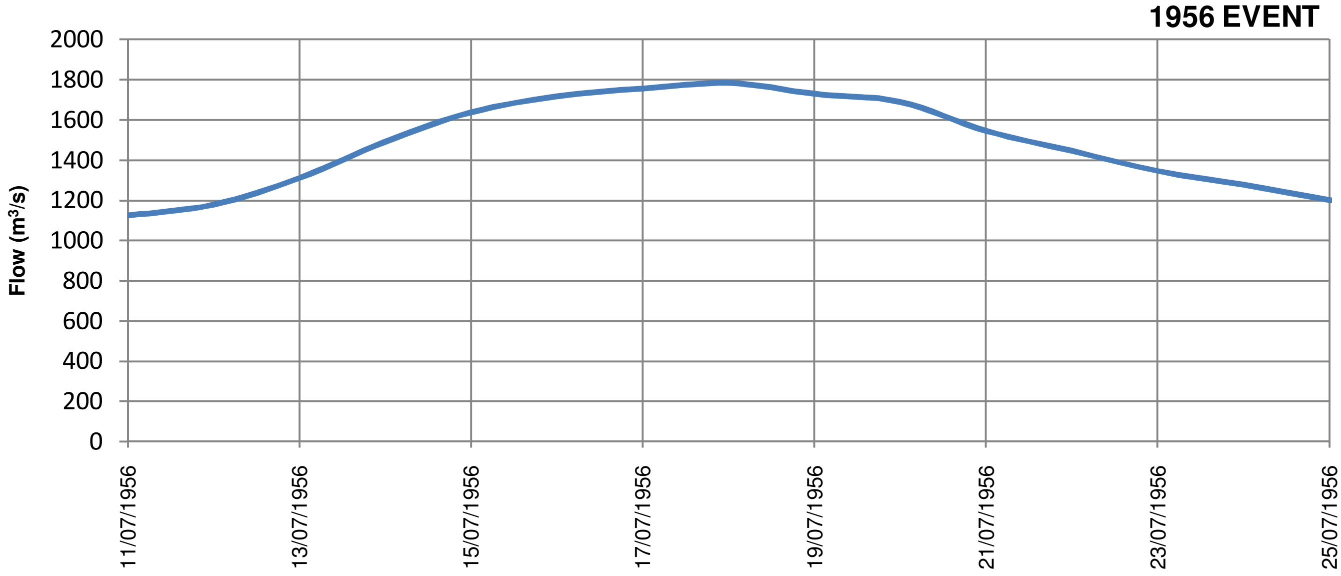
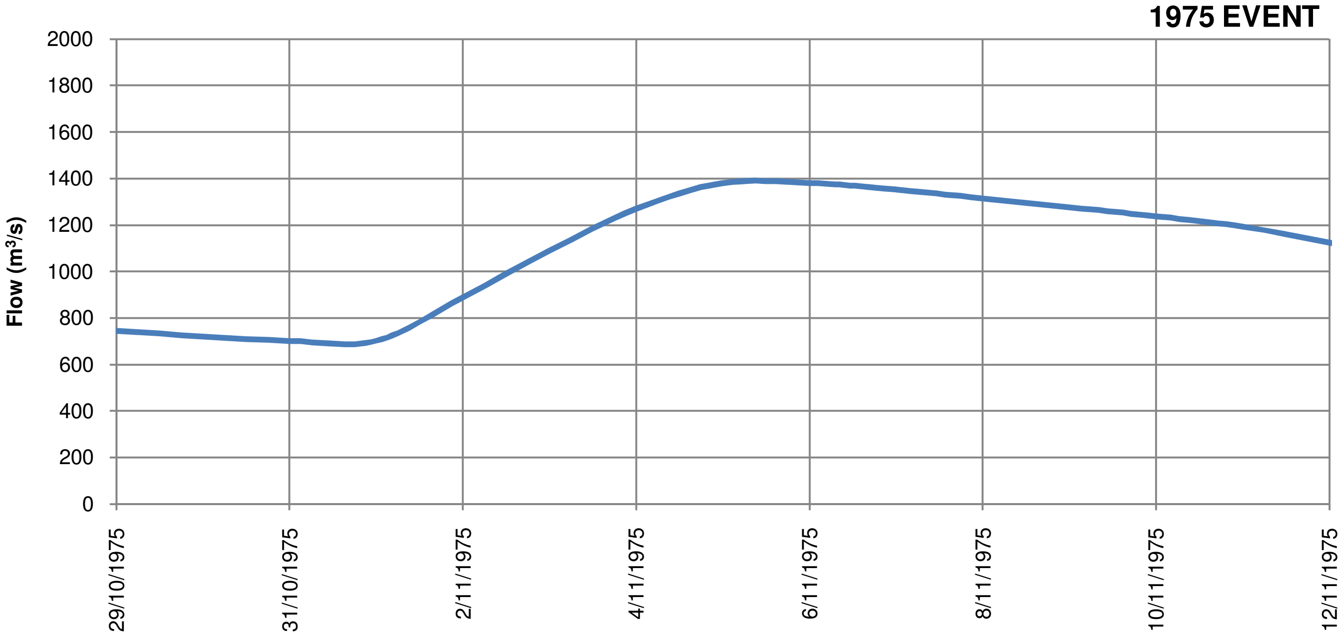
FIGURE 9

MODEL INFLOWS - CALIBRATION EVENTS





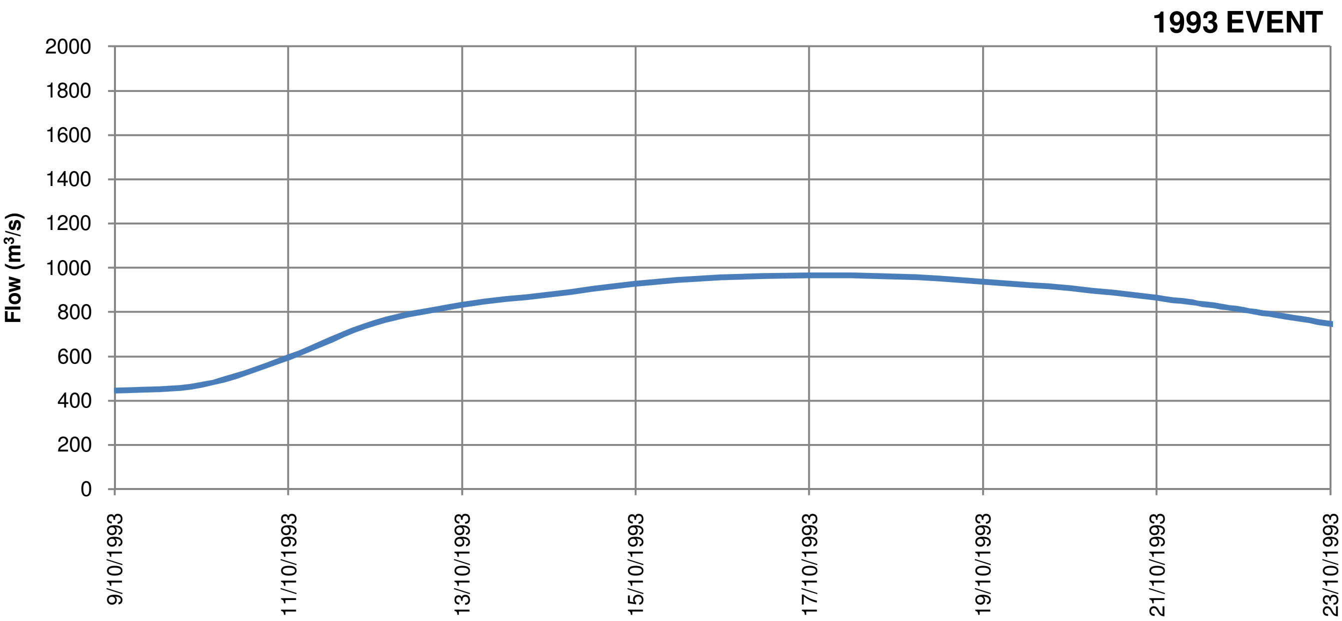


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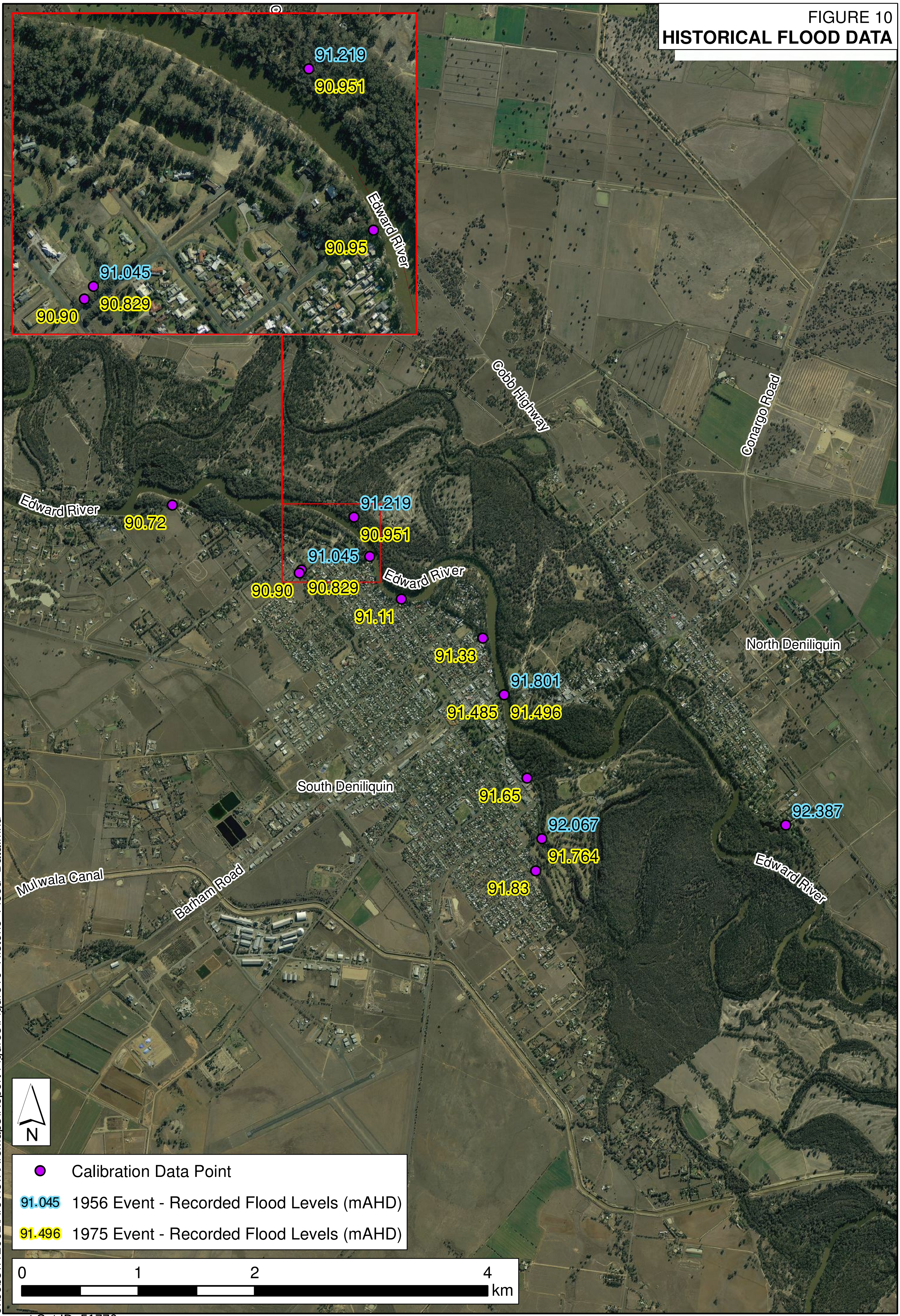
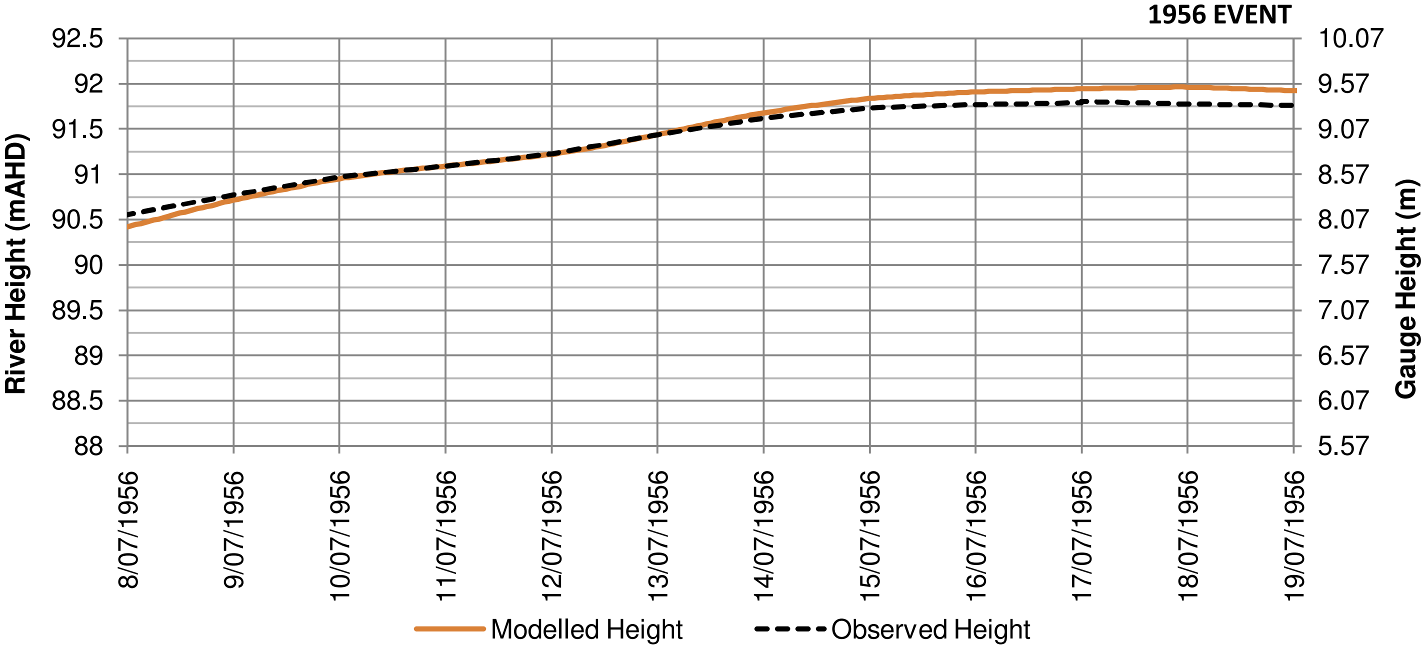
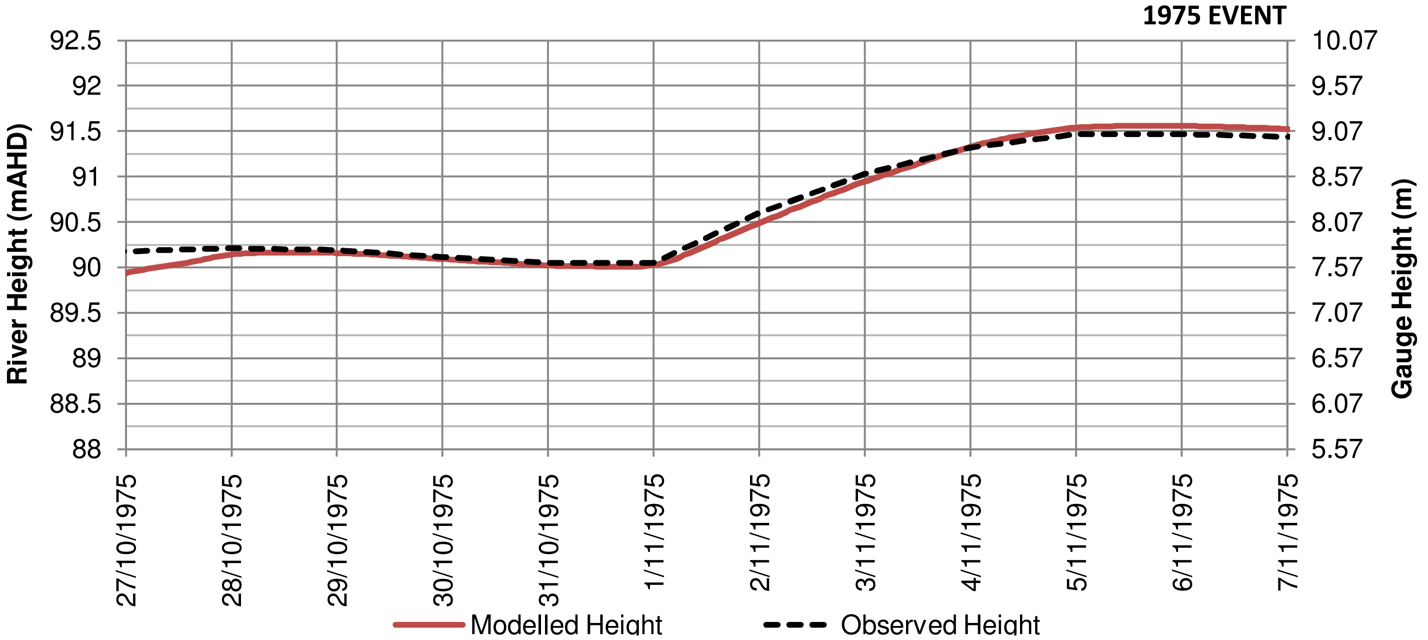
FIGURE 10
HISTORICAL FLOOD DATA

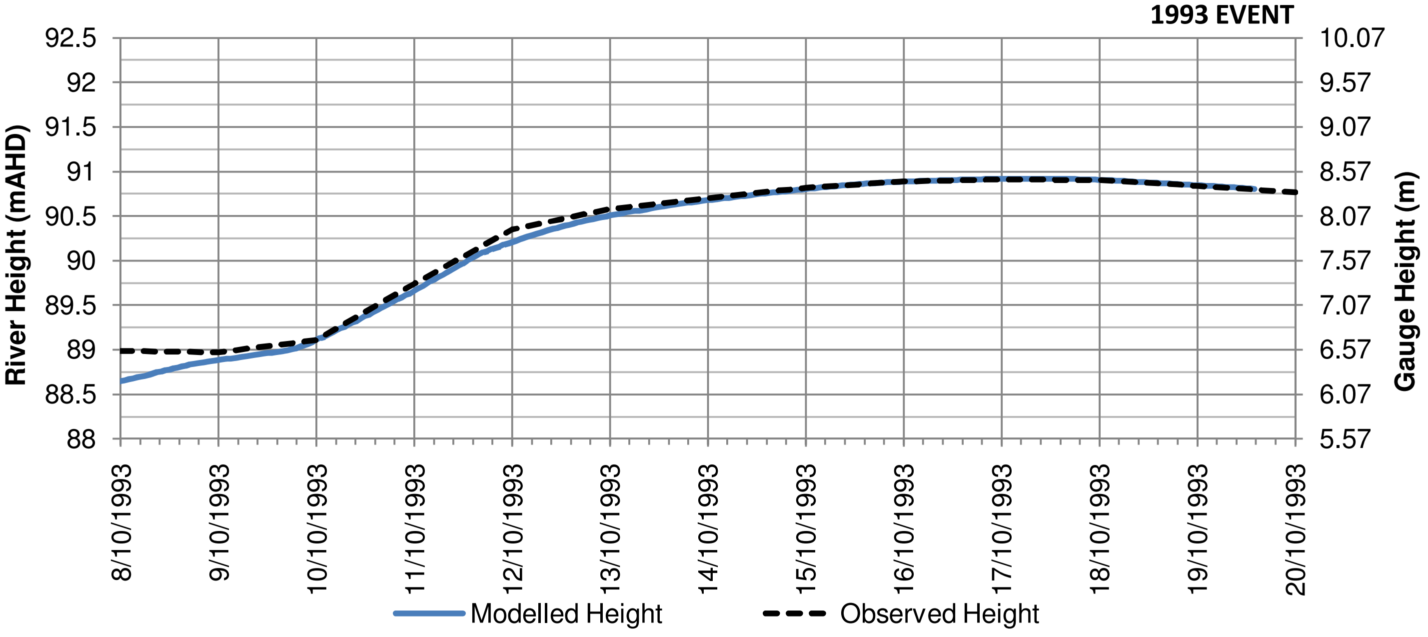


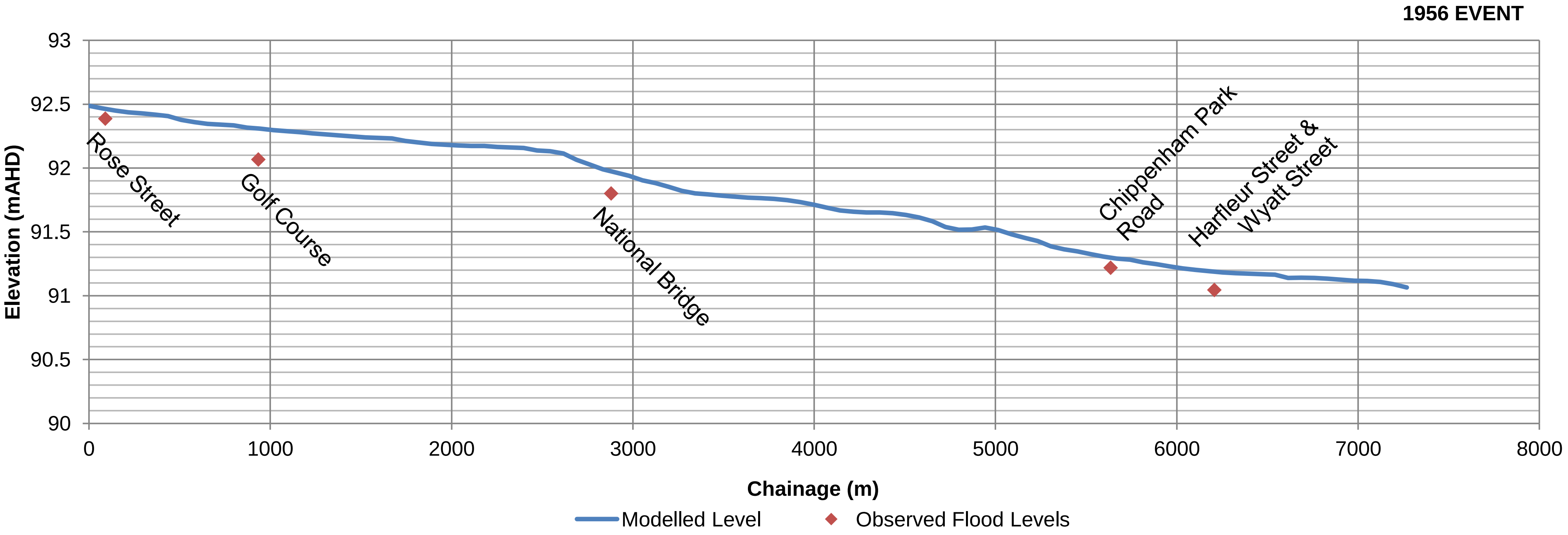
Flood Marks Sources: Brian Mitsch & Associates and 'Deniliquin Flood Plain Management Study' (1984)

FIGURE 11
WATER LEVEL HYDROGRAPH - CALIBRATION EVENTS









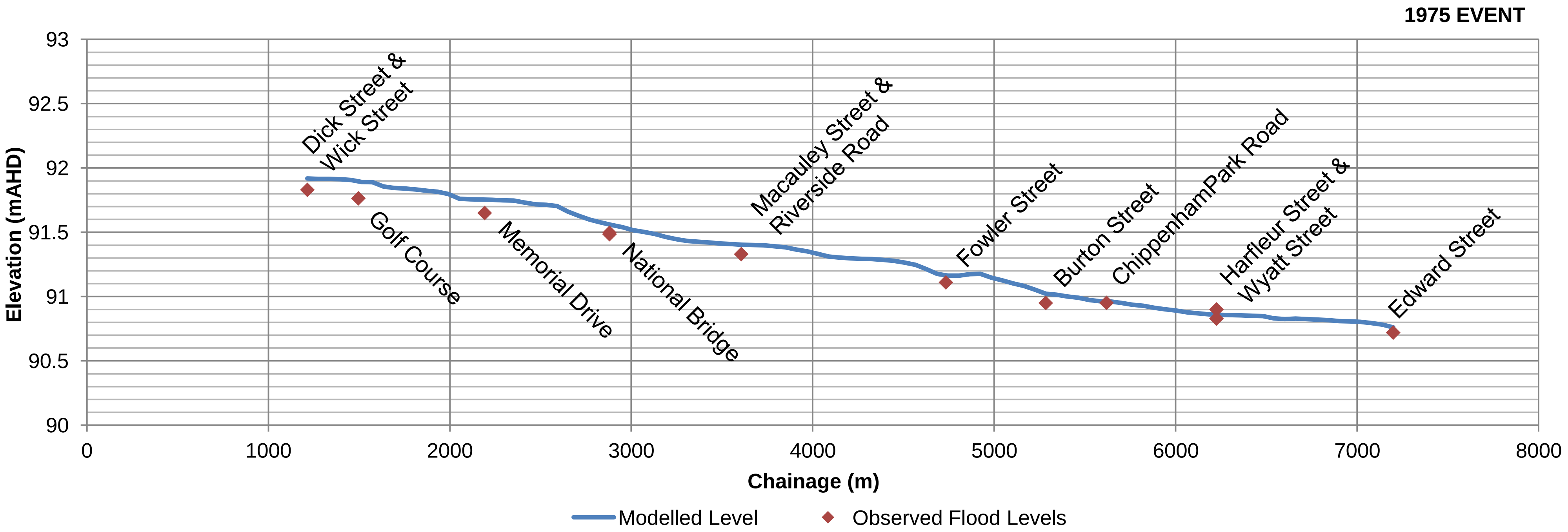


FIGURE 12
CALIBRATION EVENTS - PROFILES

FIGURE 13
PEAK FLOOD DEPTH AND LEVELS
1956 CALIBRATION EVENT

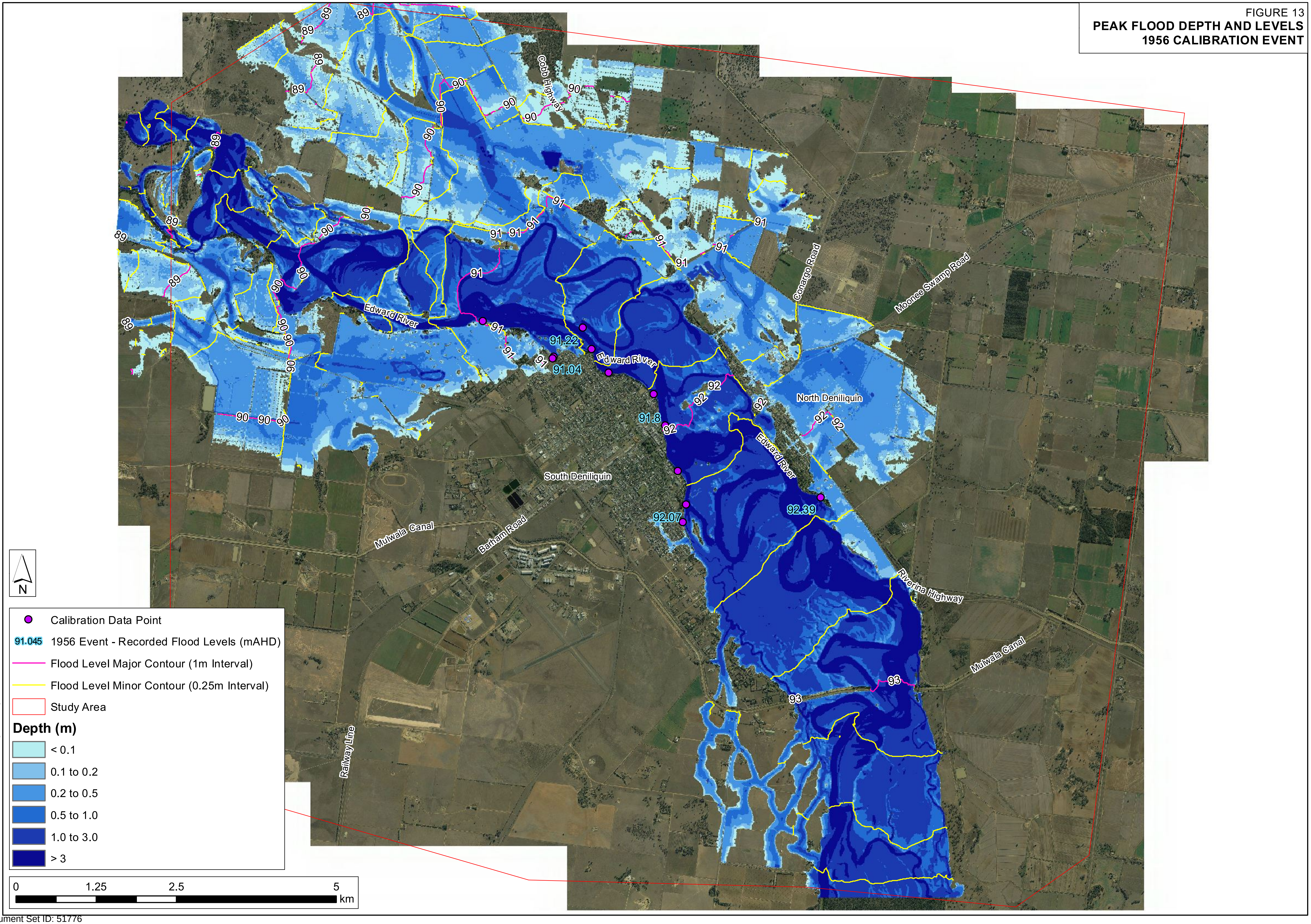


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FIGURE 14
PEAK FLOOD DEPTH AND LEVELS
1975 CALIBRATION EVENT

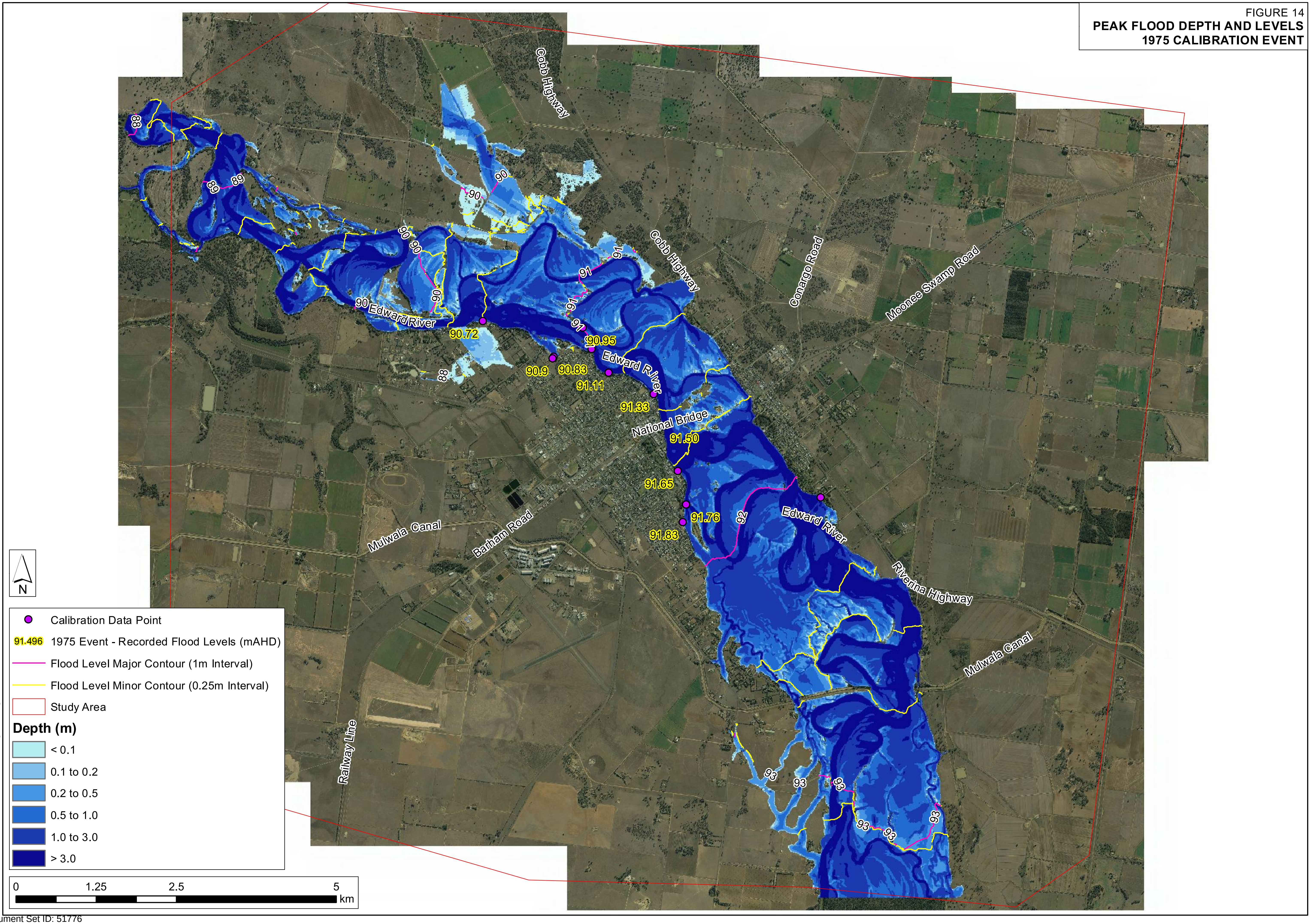

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FIGURE 15
PEAK FLOOD DEPTH AND LEVELS
1993 CALIBRATION EVENT

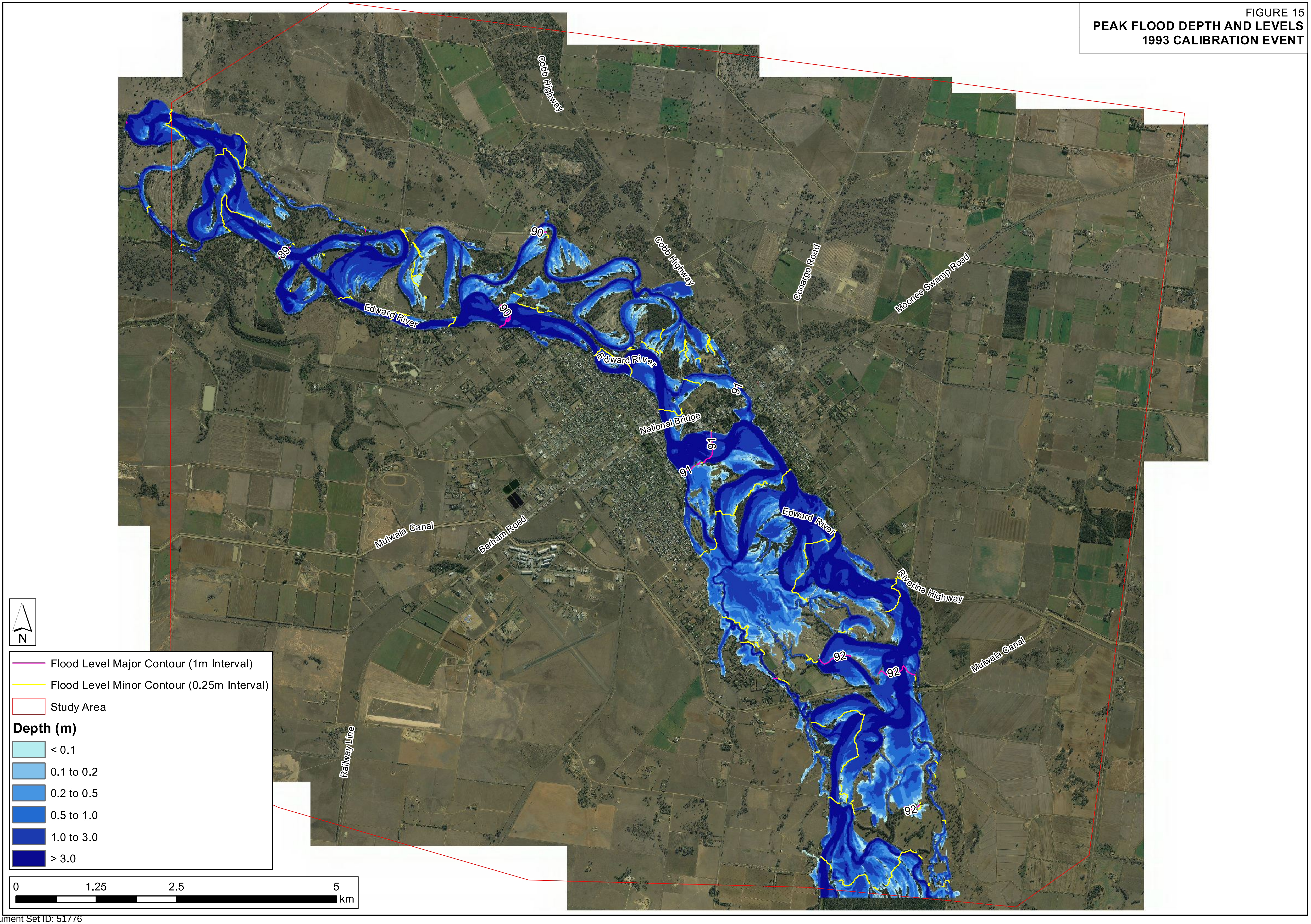

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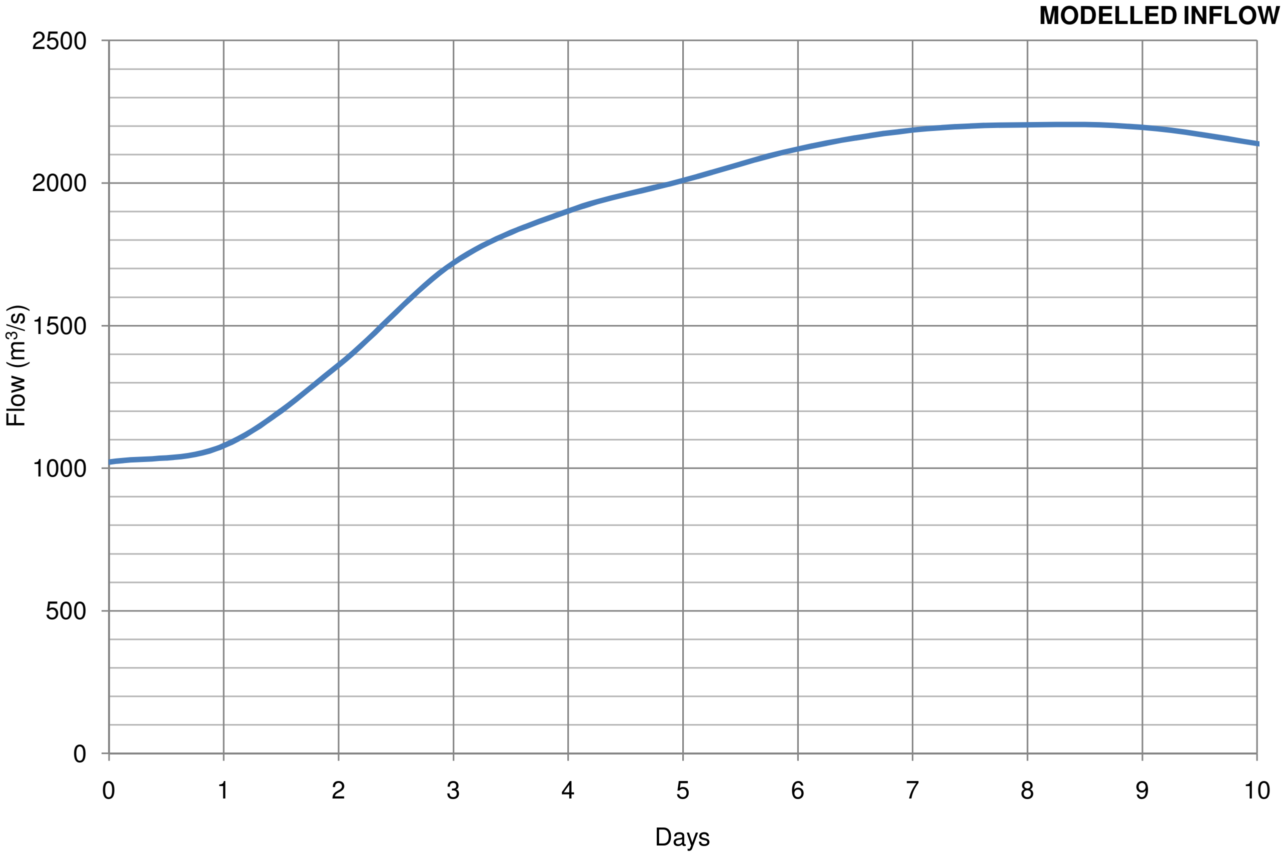
FIGURE 16
WATER LEVEL AT GAUGE AND INFLOW HYDROGRAPH
1% AEP DESIGN EVENT



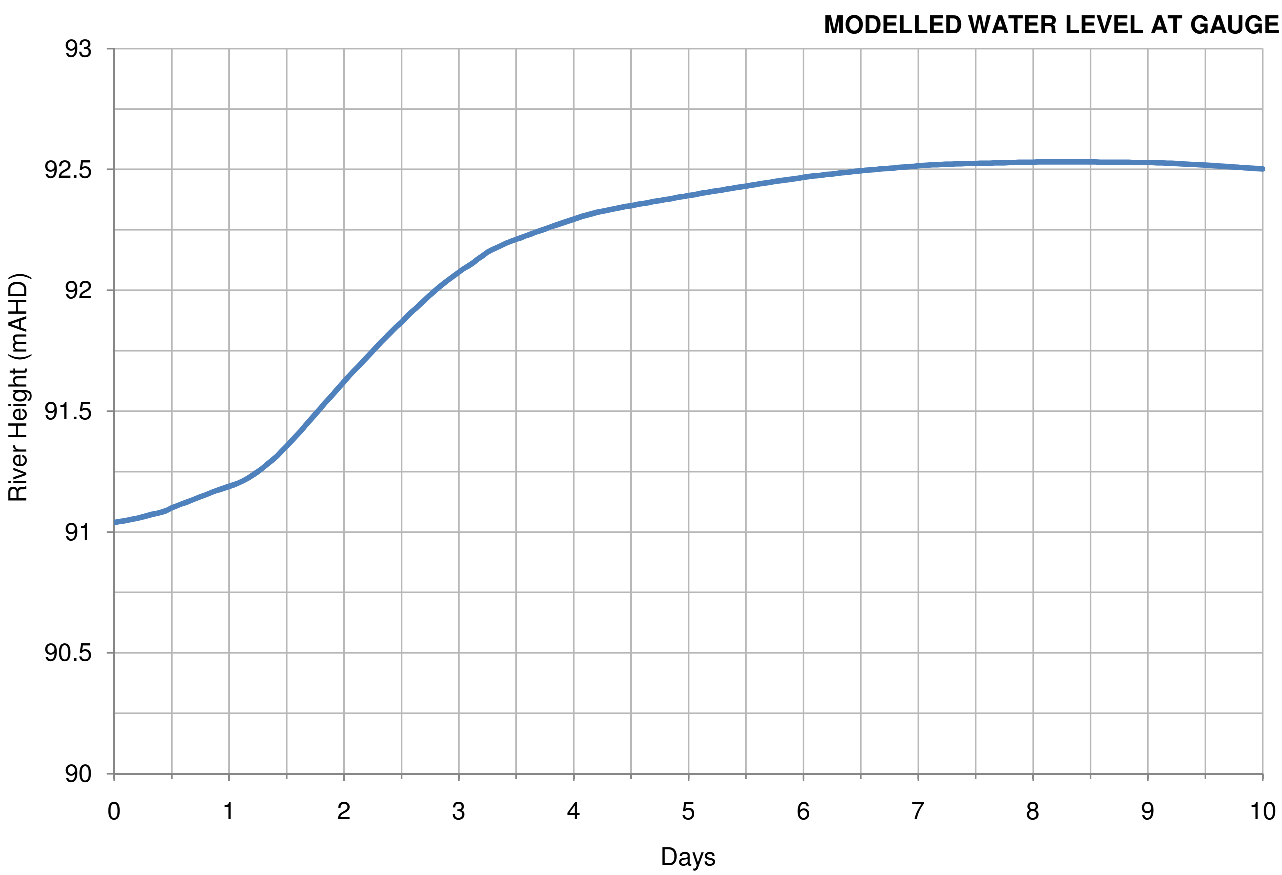
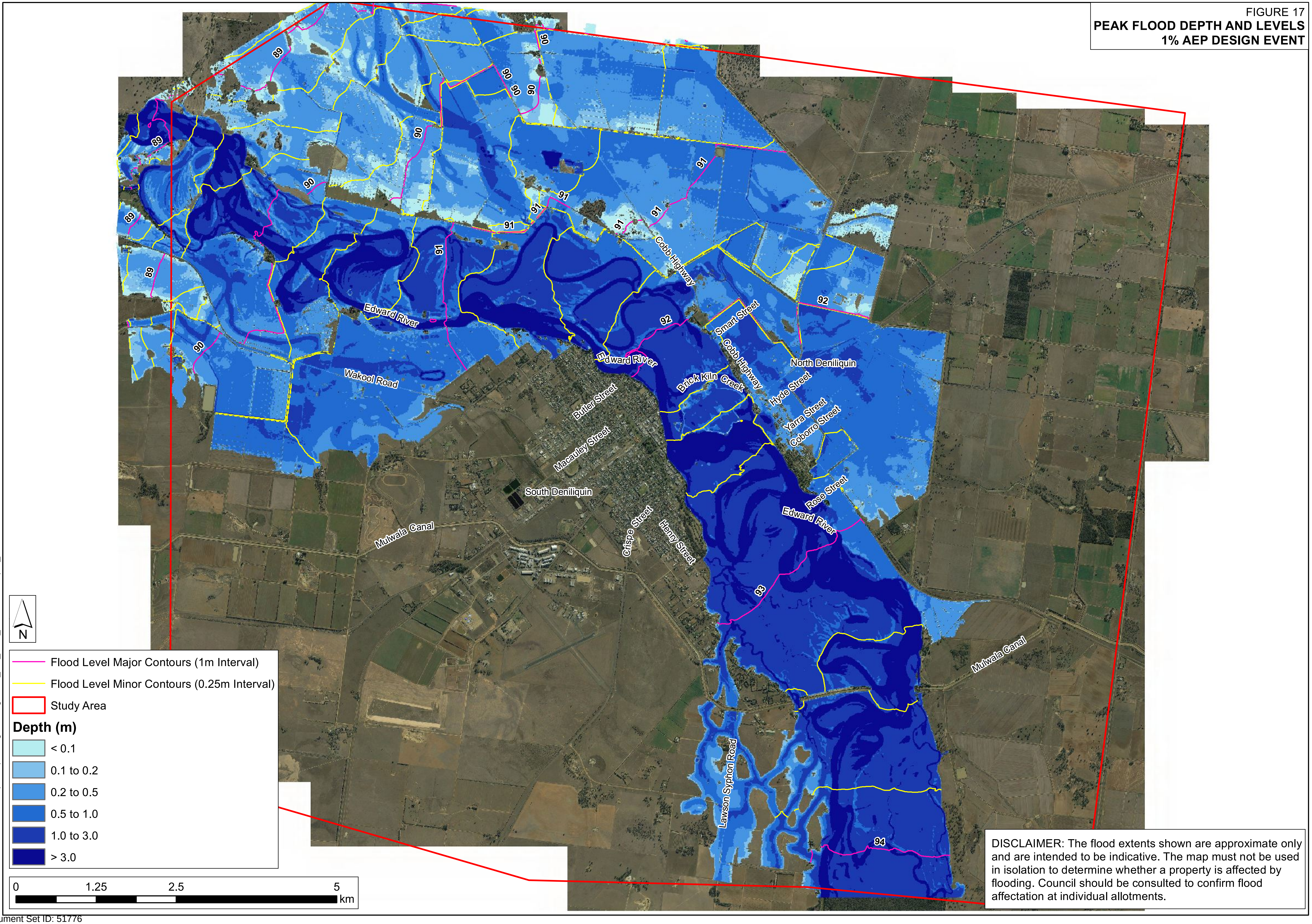


FIGURE 17
PEAK FLOOD DEPTH AND LEVELS
1% AEP DESIGN EVENT



DISCLAIMER: The flood extents shown are approximate only and are intended to be indicative. The map must not be used in isolation to determine whether a property is affected by flooding. Council should be consulted to confirm flood affectation at individual allotments.

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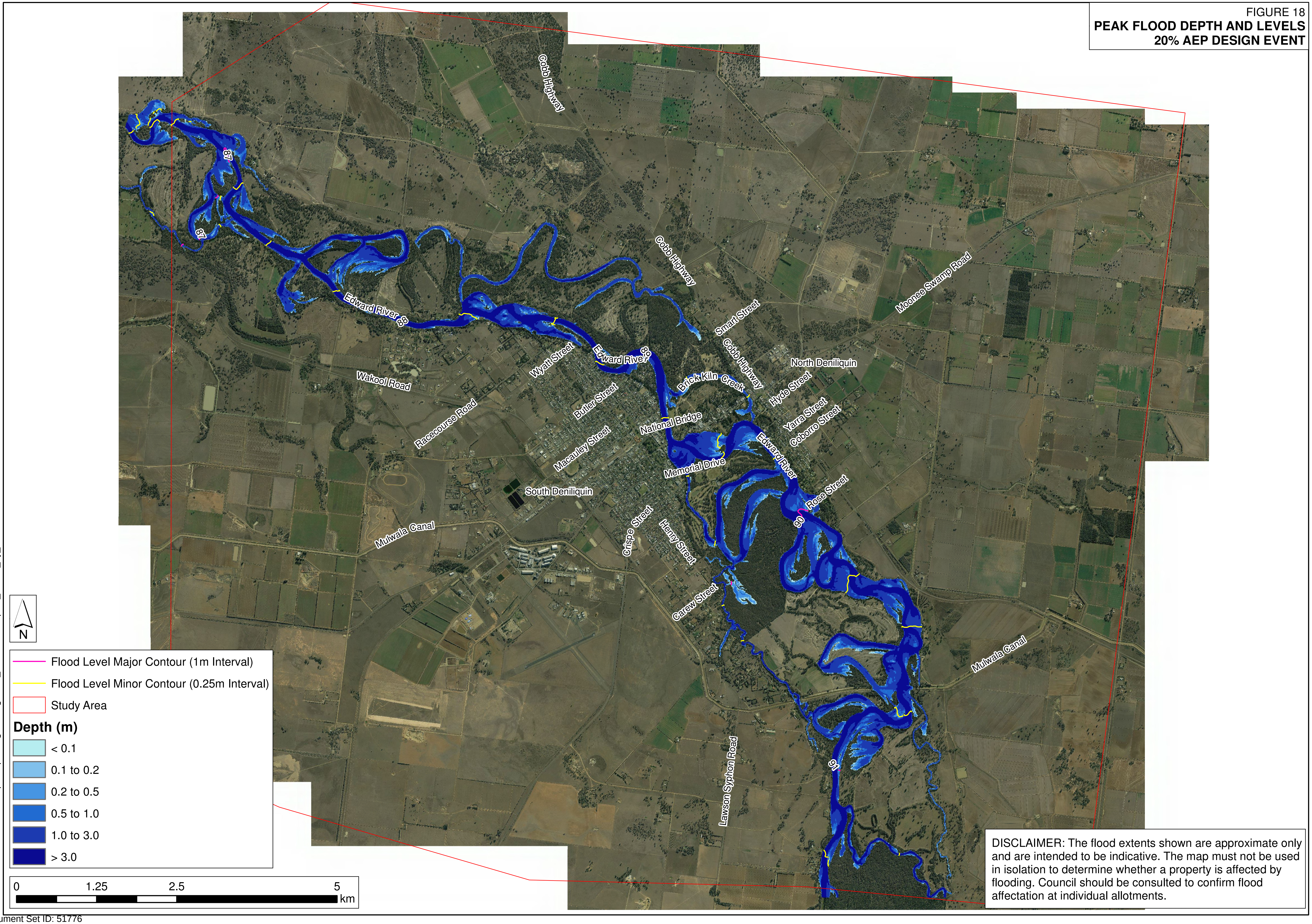
FIGURE 18
PEAK FLOOD DEPTH AND LEVELS
20% AEP DESIGN EVENT



DISCLAIMER: The flood extents shown are approximate only and are intended to be indicative. The map must not be used in isolation to determine whether a property is affected by flooding. Council should be consulted to confirm flood affectation at individual allotments.

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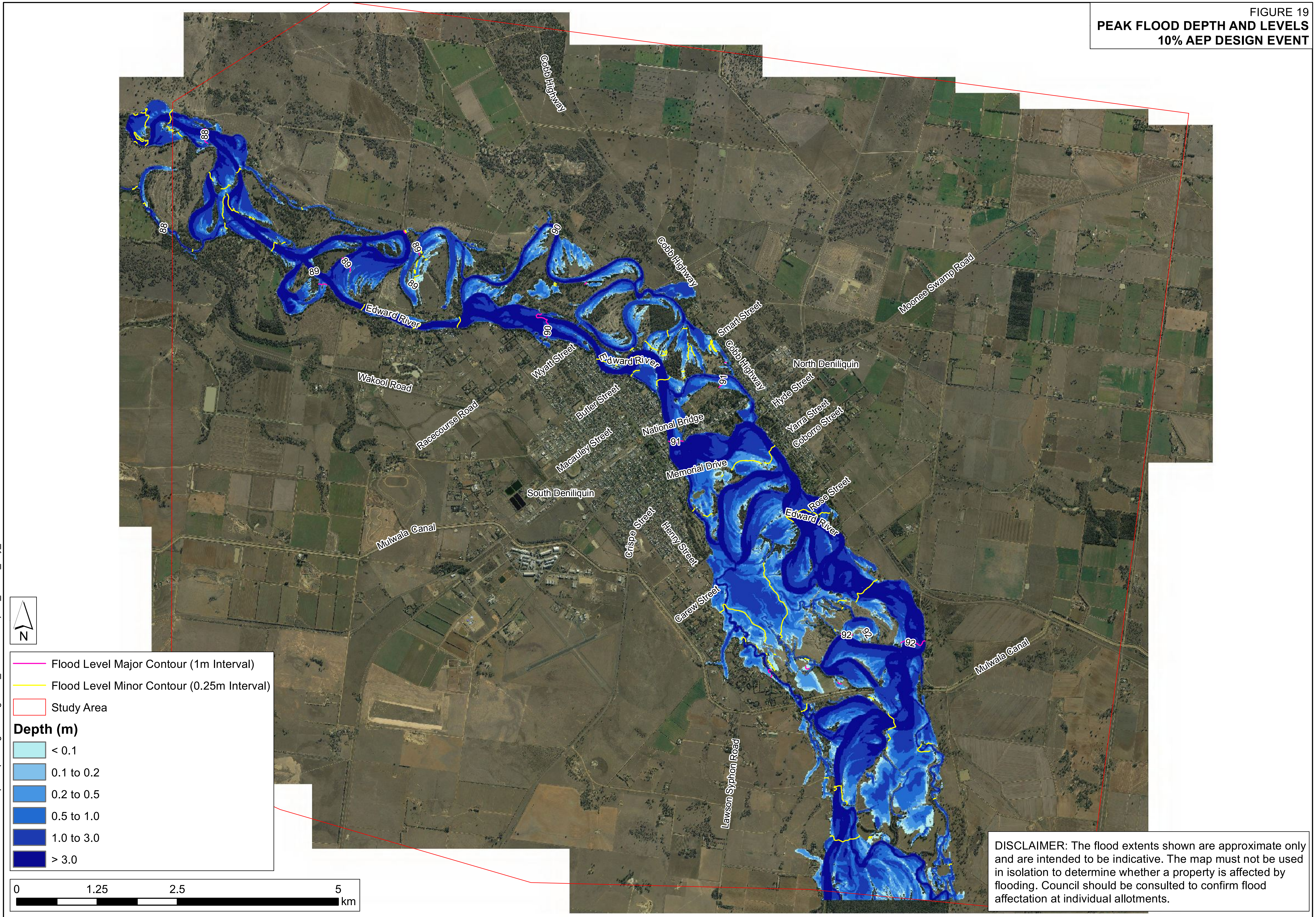
FIGURE 19
PEAK FLOOD DEPTH AND LEVELS
10% AEP DESIGN EVENT



DISCLAIMER: The flood extents shown are approximate only and are intended to be indicative. The map must not be used in isolation to determine whether a property is affected by flooding. Council should be consulted to confirm flood affectation at individual allotments.

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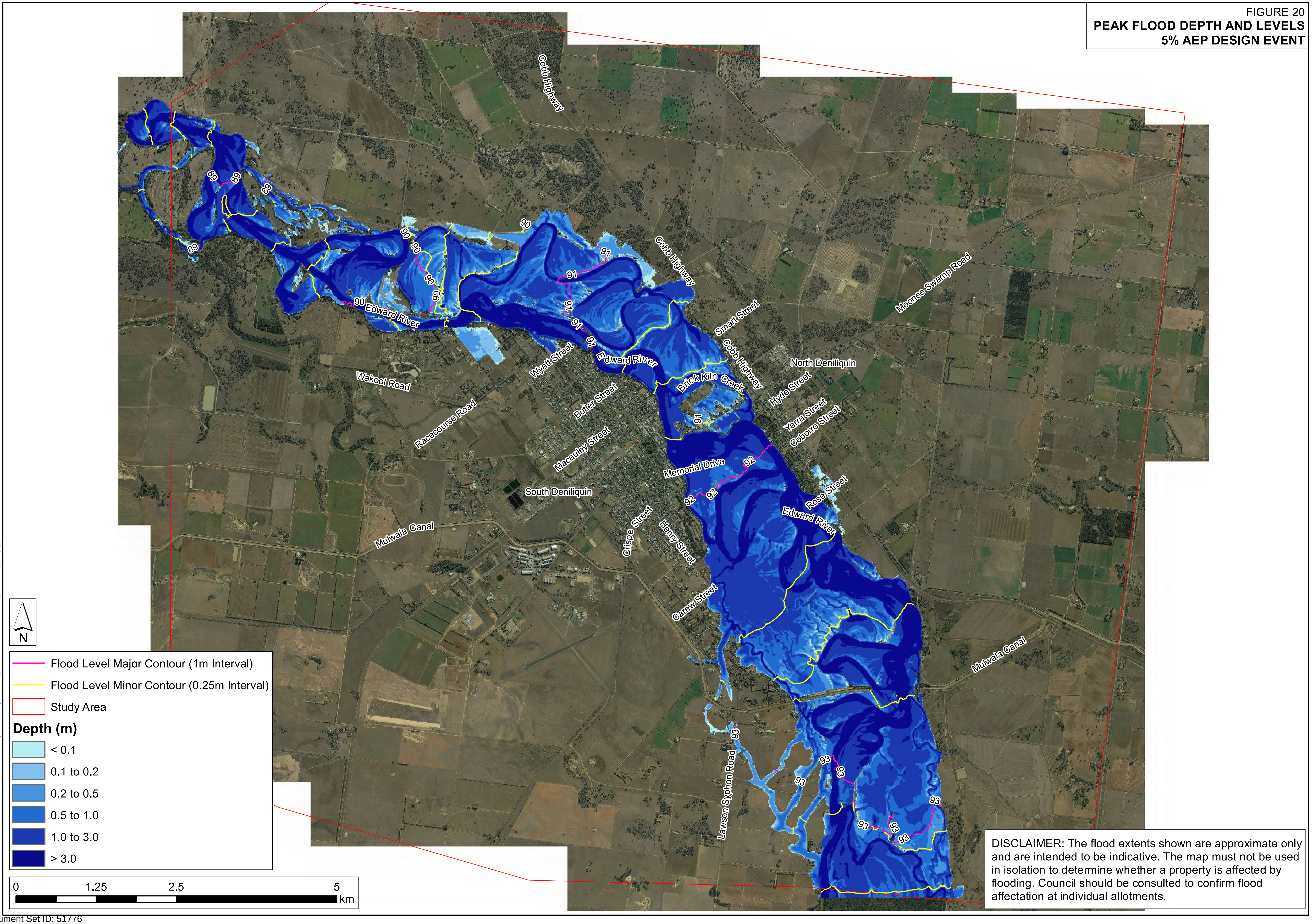
FIGURE 20
PEAK FLOOD DEPTH AND LEVELS
5% AEP DESIGN EVENT



DISCLAIMER: The flood extents shown are approximate only and are intended to be indicative. The map must not be used in isolation to determine whether a property is affected by flooding. Council should be consulted to confirm flood affectation at individual allotments.

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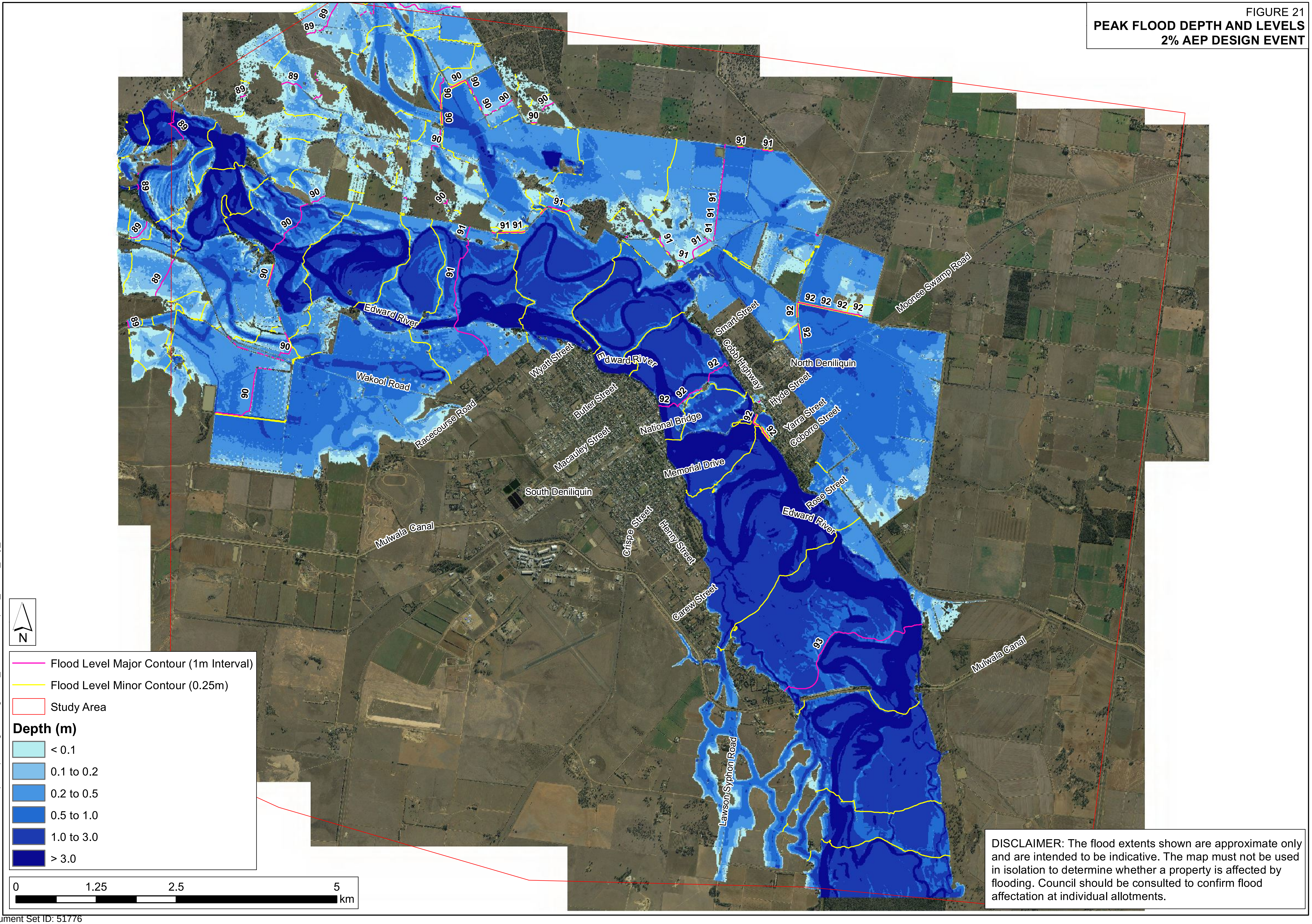
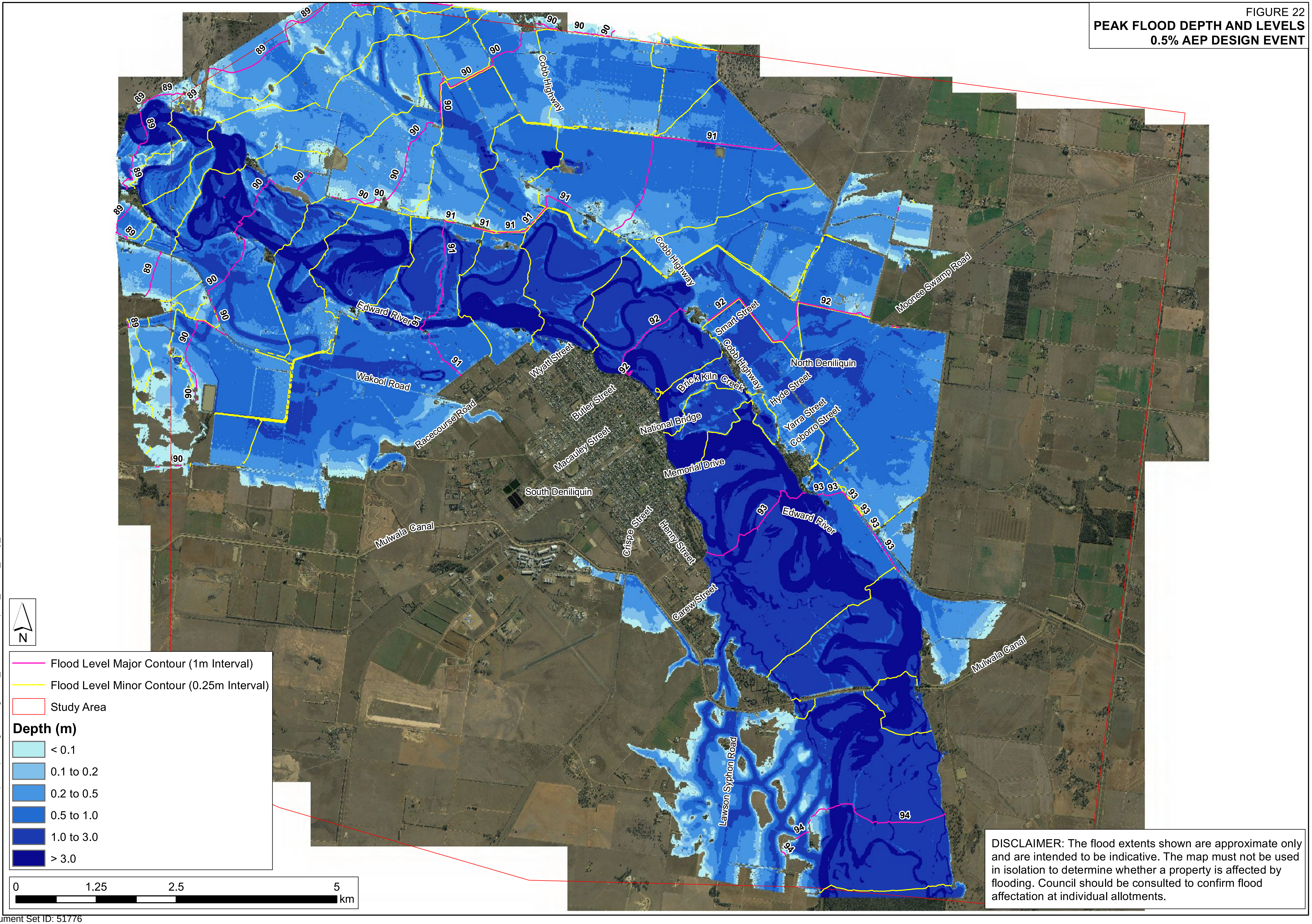
FIGURE 21
PEAK FLOOD DEPTH AND LEVELS
2% AEP DESIGN EVENT



DISCLAIMER: The flood extents shown are approximate only and are intended to be indicative. The map must not be used in isolation to determine whether a property is affected by flooding. Council should be consulted to confirm flood affectation at individual allotments.

FIGURE 22
PEAK FLOOD DEPTH AND LEVELS
0.5% AEP DESIGN EVENT



DISCLAIMER: The flood extents shown are approximate only and are intended to be indicative. The map must not be used in isolation to determine whether a property is affected by flooding. Council should be consulted to confirm flood affectation at individual allotments.

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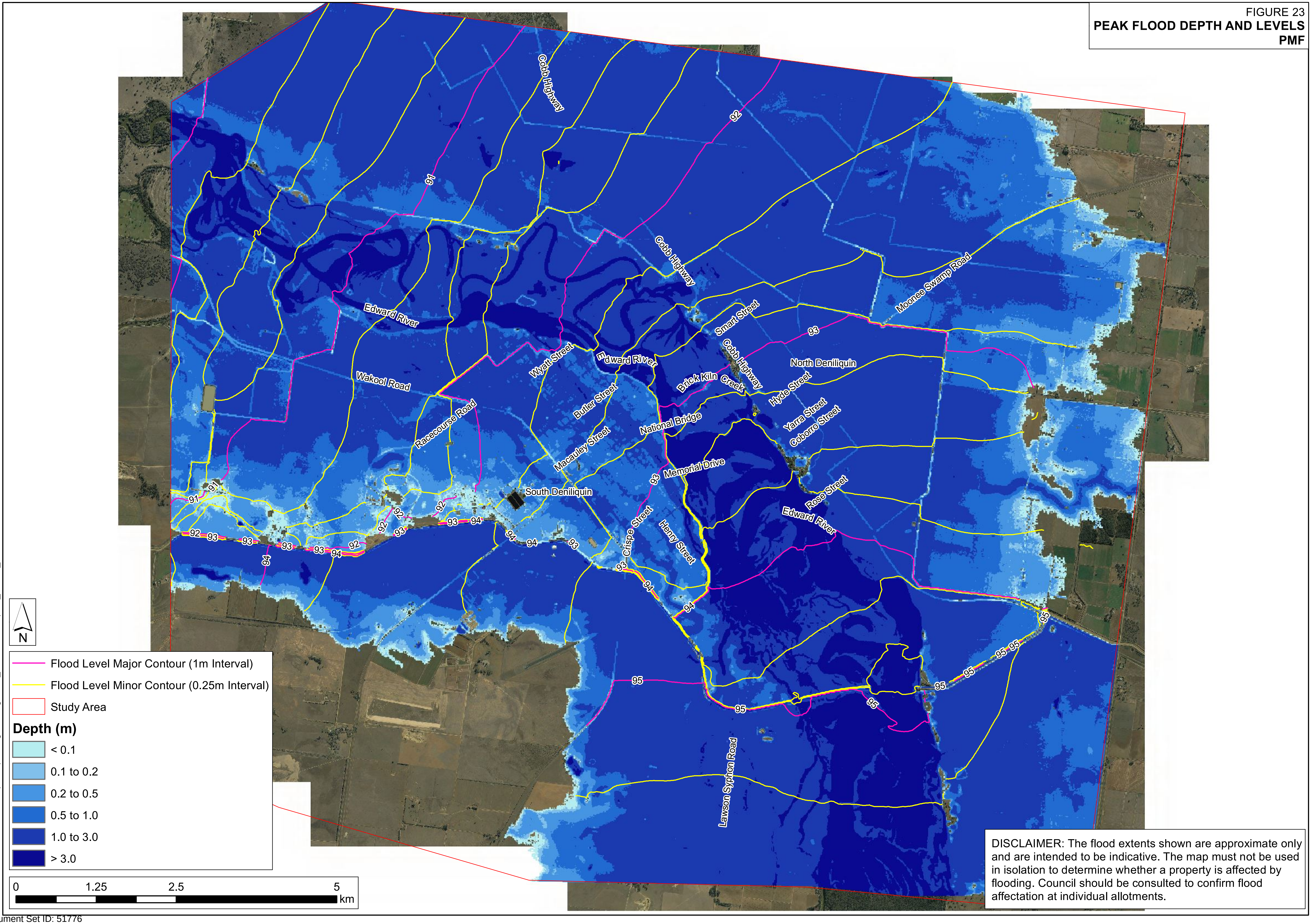
FIGURE 23
PEAK FLOOD DEPTH AND LEVELS PMF



DISCLAIMER: The flood extents shown are approximate only and are intended to be indicative. The map must not be used in isolation to determine whether a property is affected by flooding. Council should be consulted to confirm flood affectation at individual allotments.

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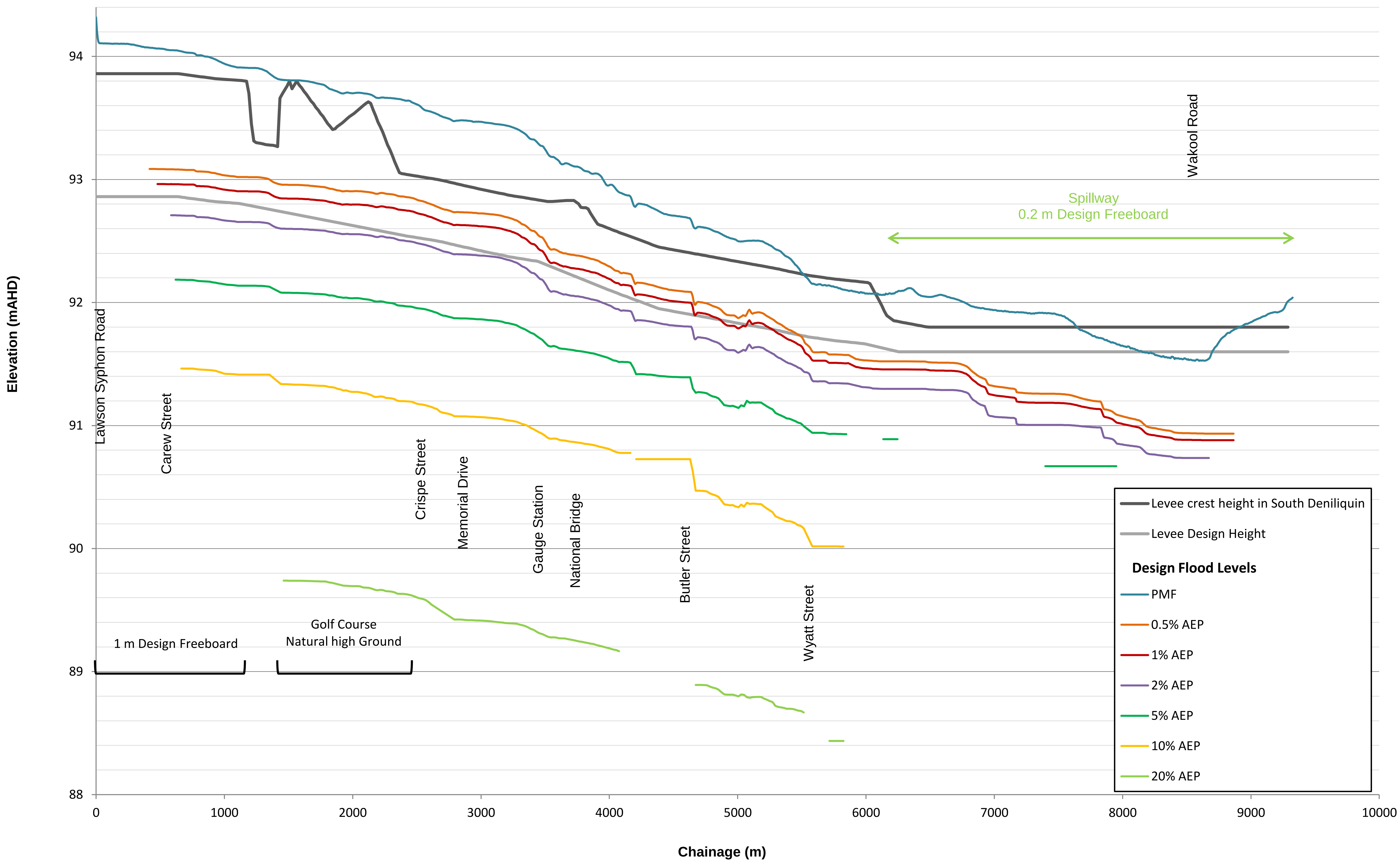
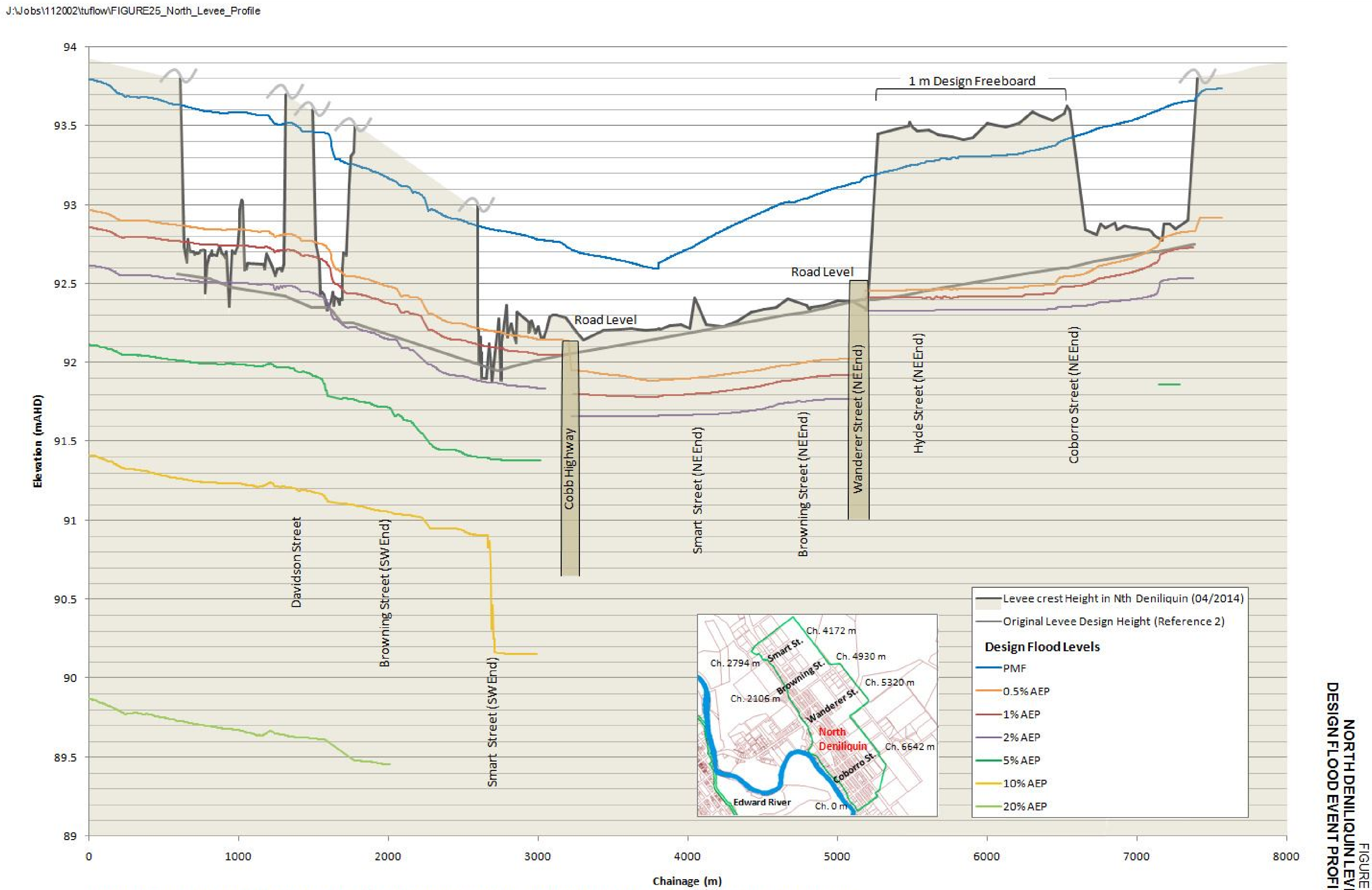


NOTE: The remainder of the levee has a 0.5 m design freeboard.

FIGURE 24
SOUTH DENILIQUIN LEVEE
DESIGN FLOOD EVENT PROFILE



NOTE: - Levee elevations above 94 mAHD are sections of natural high ground in North Deniliquin that form part of the levee structure.
- All other sections of the levee have a design freeboard of 0.1 m, except where shown above between chainages 5200 m and 6500 m.

FIGURE 25
NORTH DENILIQUIN LEVEE
DESIGN FLOOD EVENT PROFILE

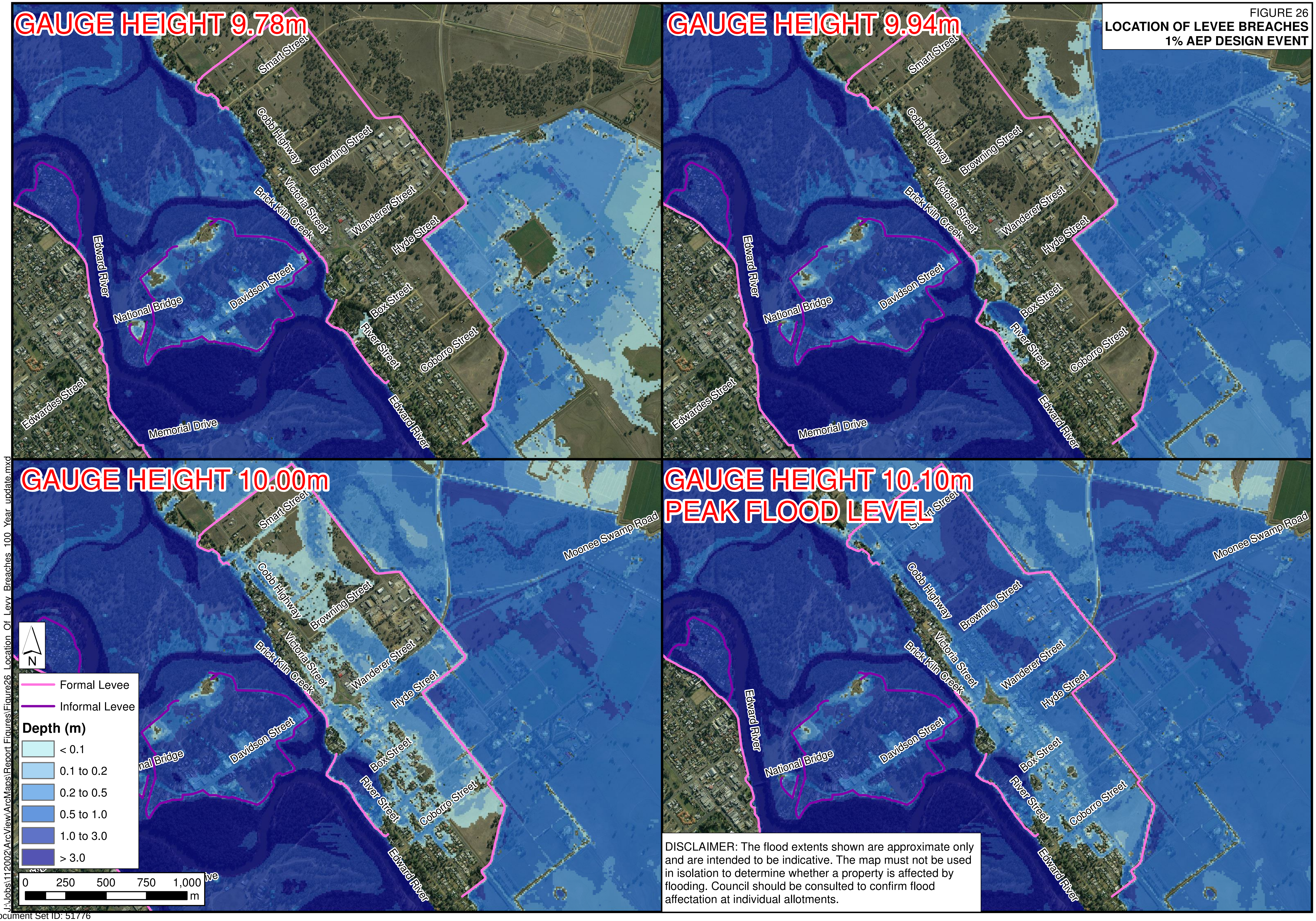


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FIGURE 27
PROVISIONAL HYDRAULIC HAZARD
5% AEP DESIGN EVENT

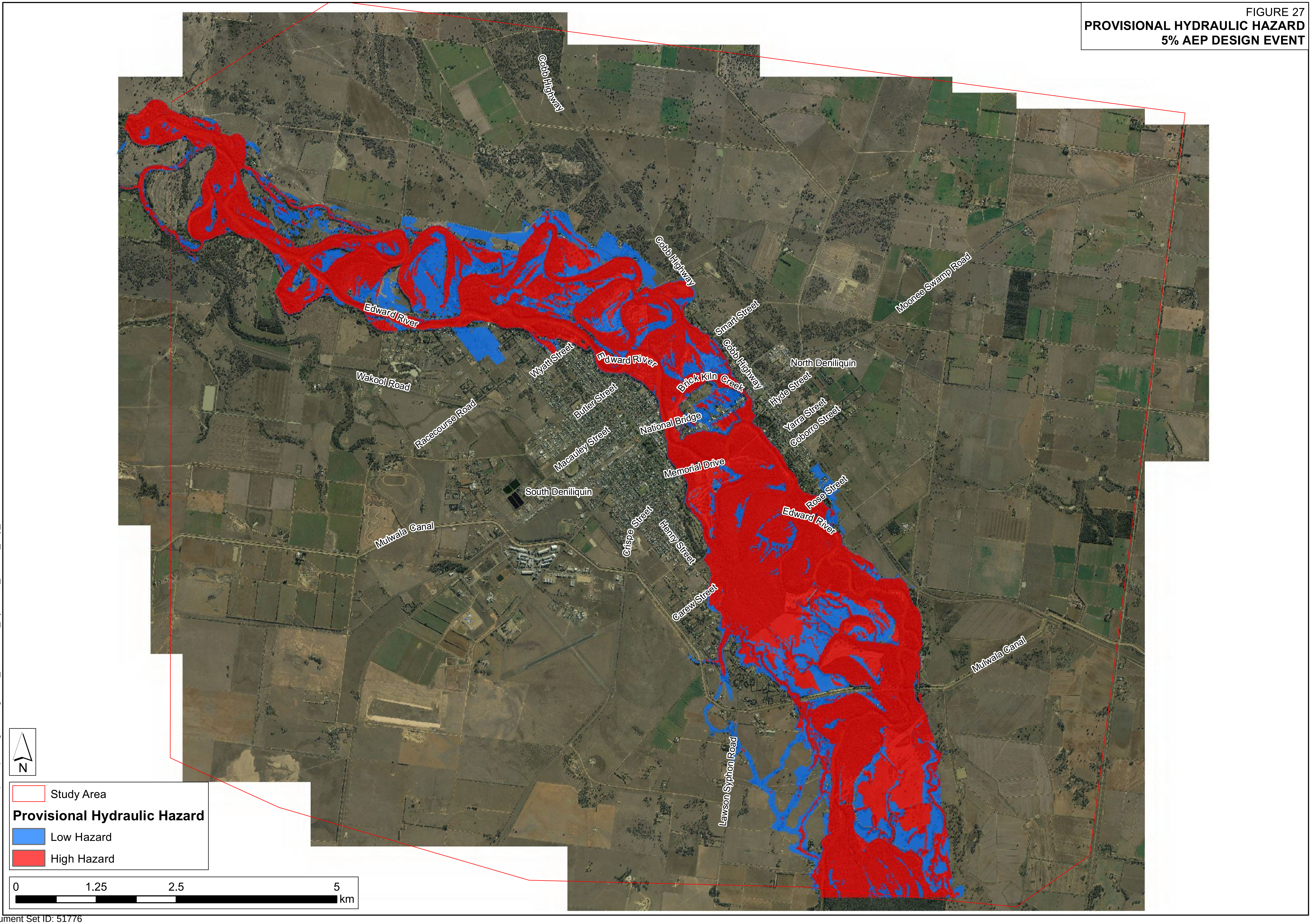

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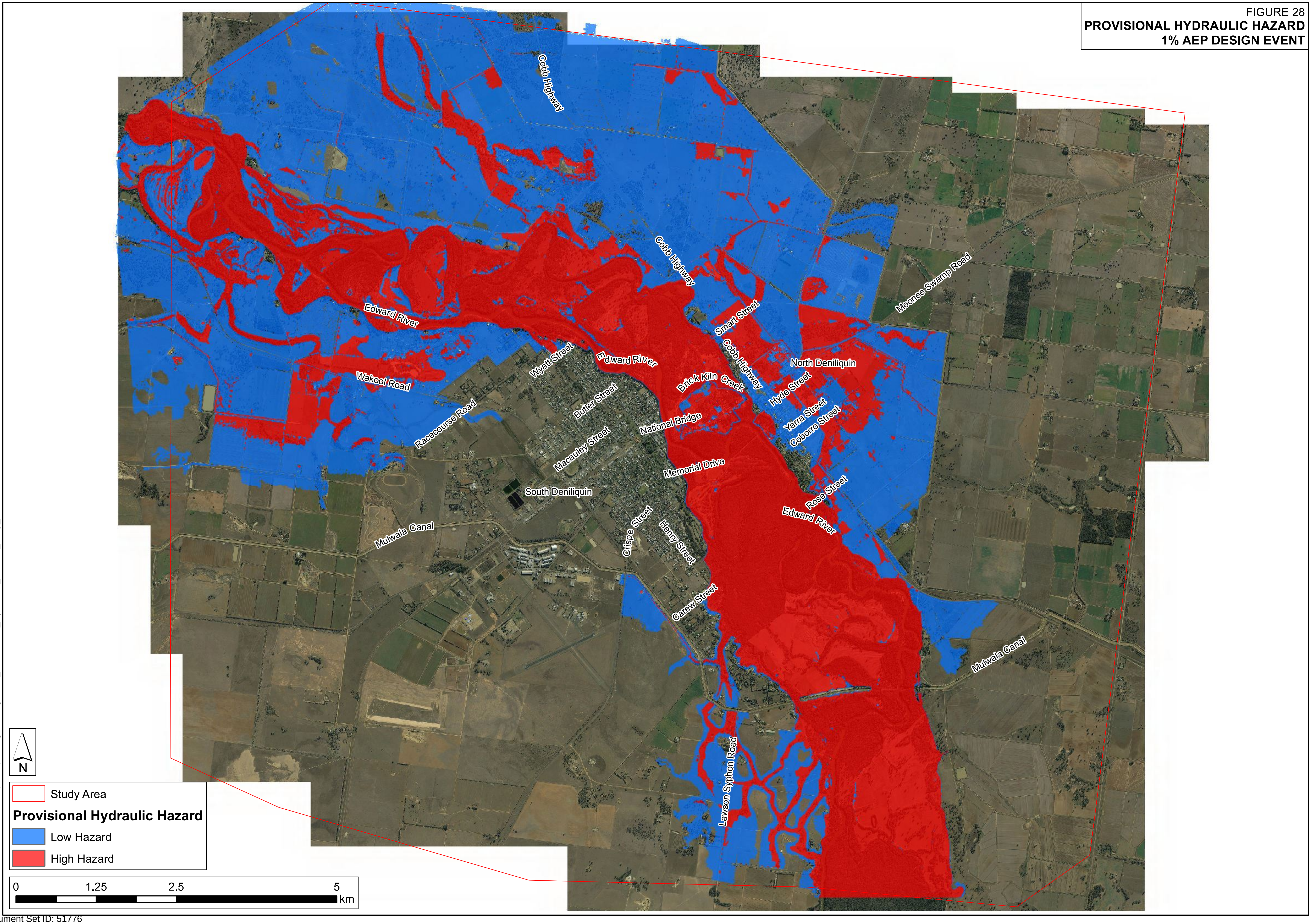
FIGURE 28
PROVISIONAL HYDRAULIC HAZARD
1% AEP DESIGN EVENT



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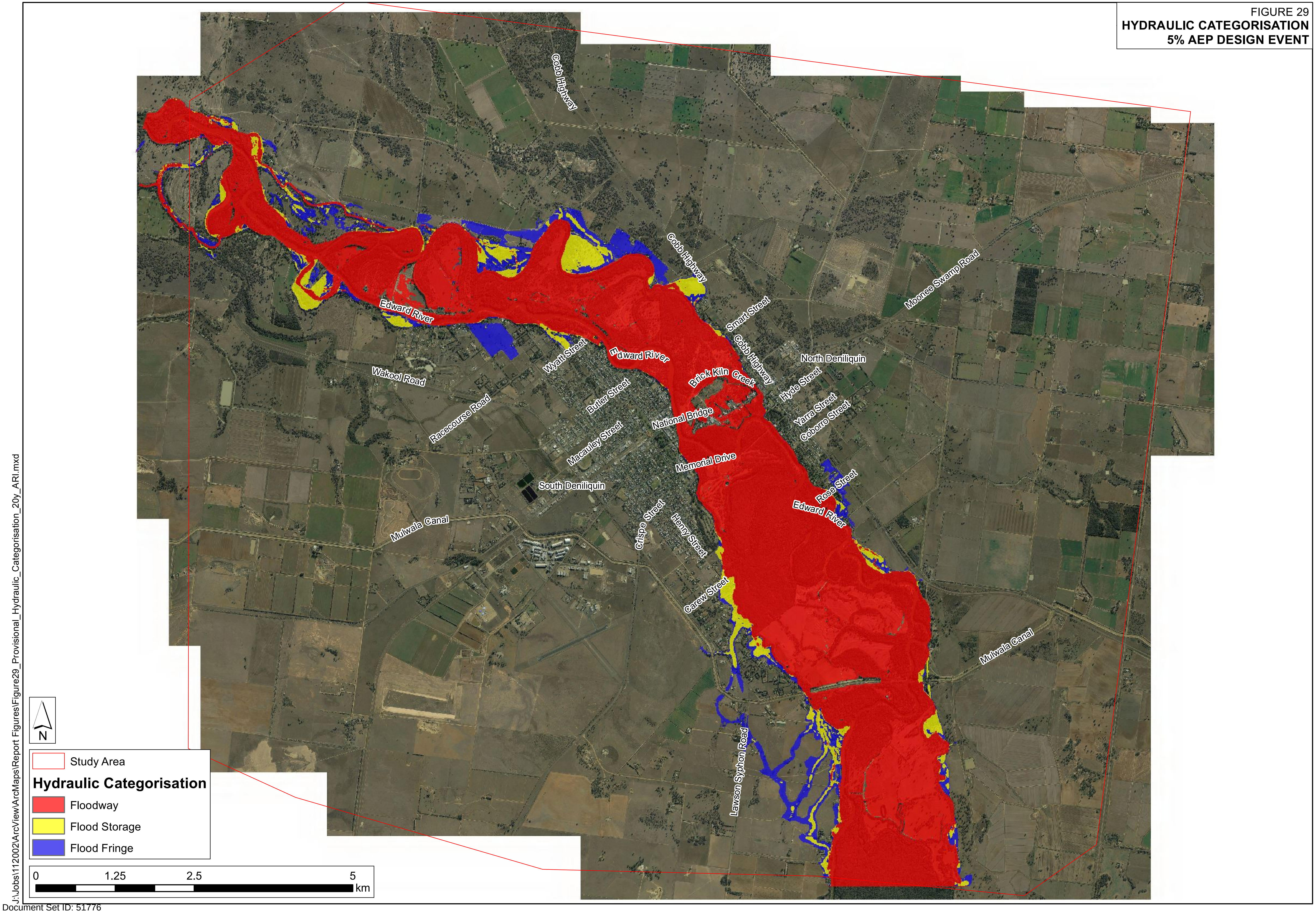

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FIGURE 30
HYDRAULIC CATEGORISATION
1% AEP DESIGN EVENT

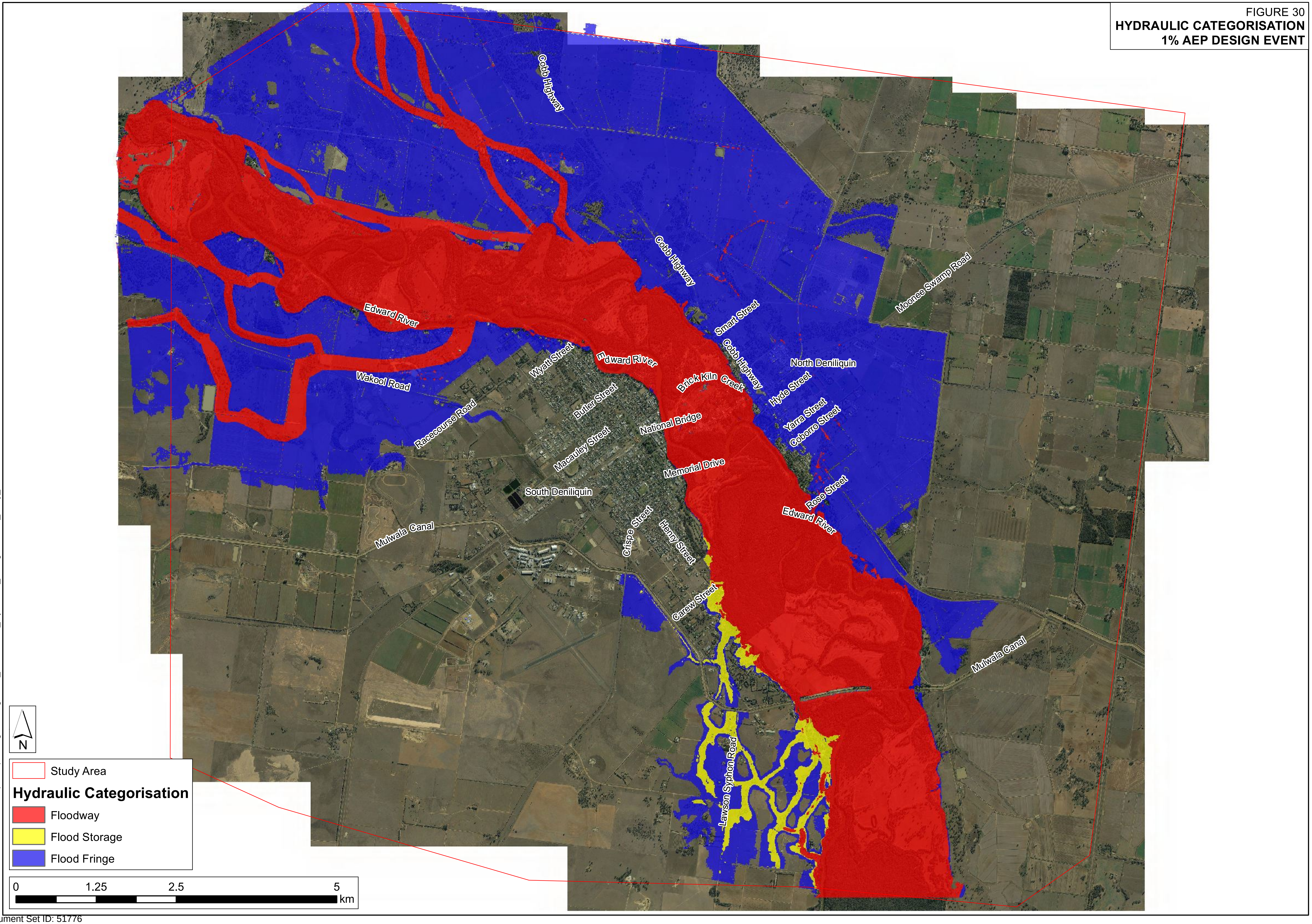

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FIGURE 31
PROVISIONAL FLOOD PLANNING AREA
1% AEP DESIGN LEVEL + 0.5m

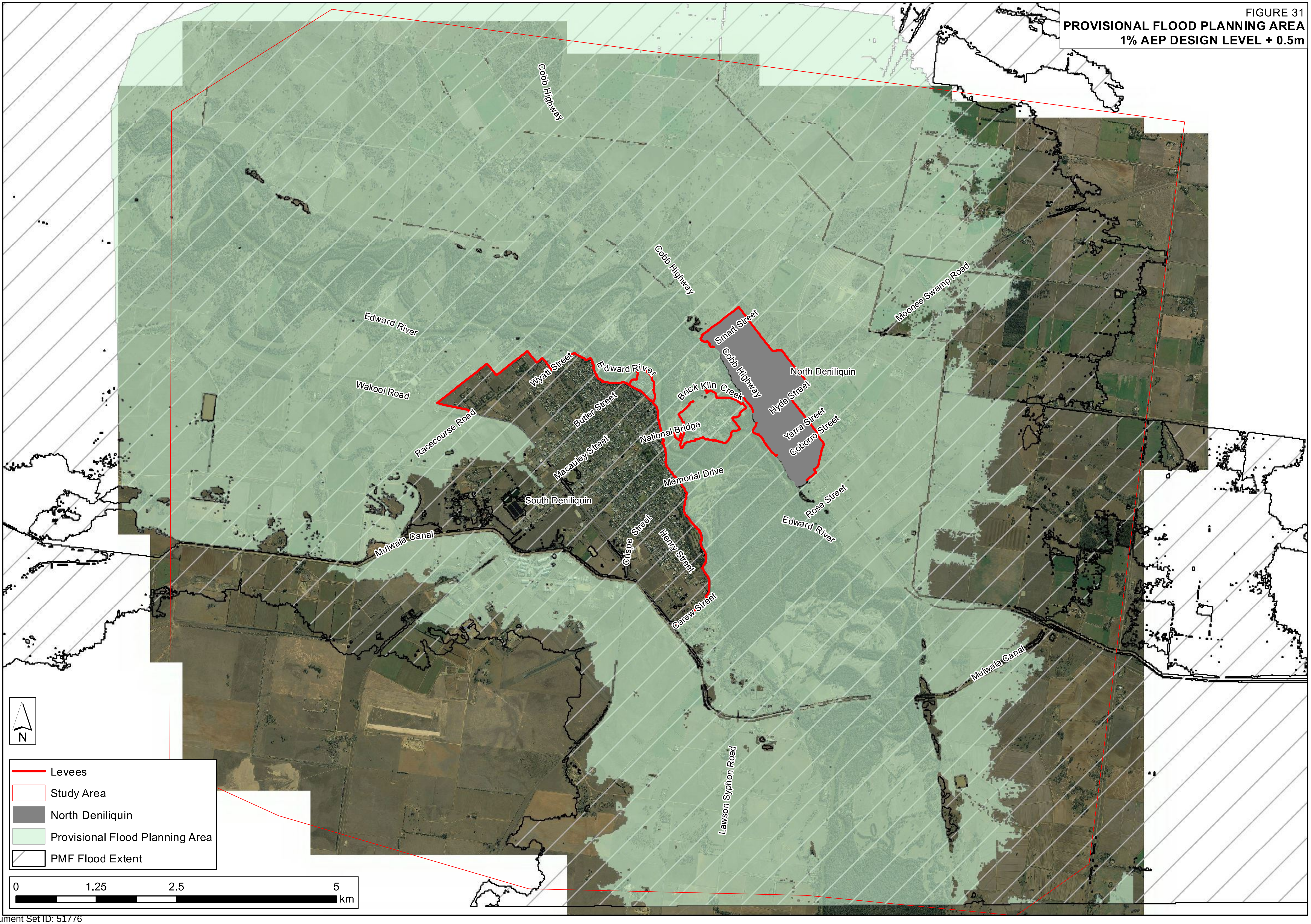

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FIGURE 32
1% AEP DESIGN LEVEL + 0.1m

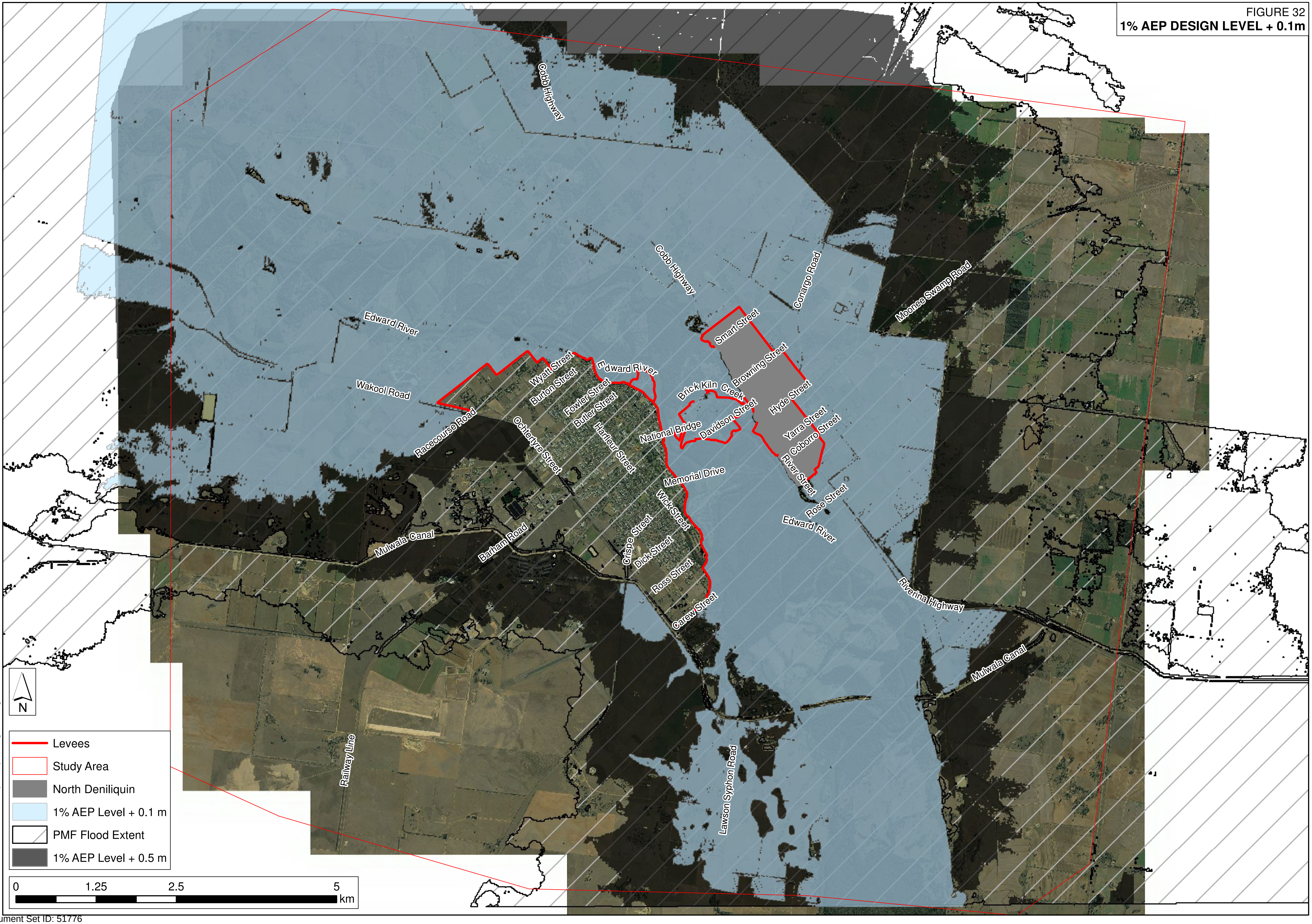

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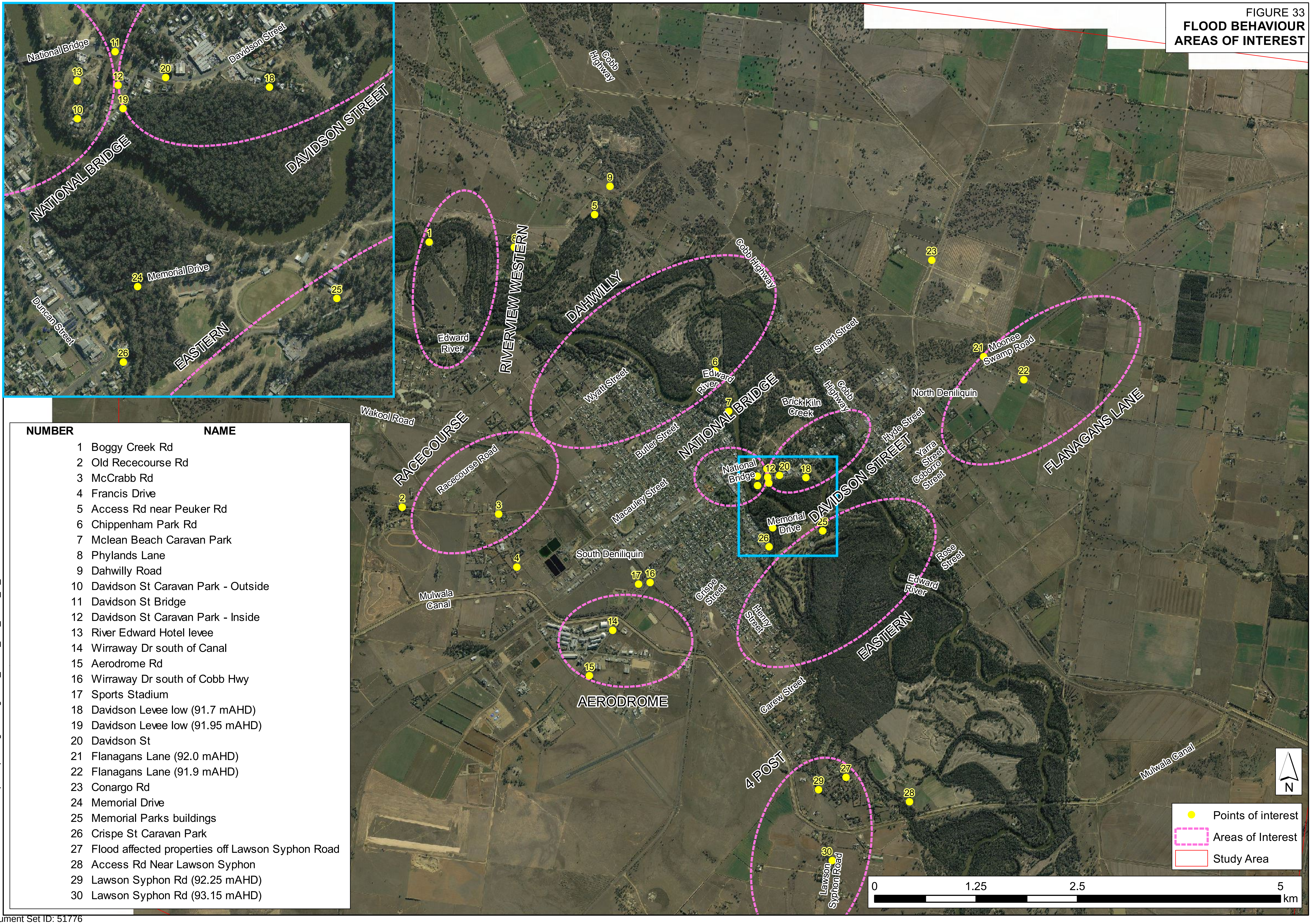
FIGURE 33
FLOOD BEHAVIOUR AREAS OF INTEREST



NUMBER	NAME
1	Boggy Creek Rd
2	Old Rececourse Rd
3	McCrabb Rd
4	Francis Drive
5	Access Rd near Peuker Rd
6	Chippenham Park Rd
7	Mclean Beach Caravan Park
8	Phylands Lane
9	Dahwilly Road
10	Davidson St Caravan Park - Outside
11	Davidson St Bridge
12	Davidson St Caravan Park - Inside
13	River Edward Hotel levee
14	Wirraway Dr south of Canal
15	Aerodrome Rd
16	Wirraway Dr south of Cobb Hwy
17	Sports Stadium
18	Davidson Levee low (91.7 mAHD)
19	Davidson Levee low (91.95 mAHD)
20	Davidson St
21	Flanagans Lane (92.0 mAHD)
22	Flanagans Lane (91.9 mAHD)
23	Conargo Rd
24	Memorial Drive
25	Memorial Parks buildings
26	Crispe St Caravan Park
27	Flood affected properties off Lawson Syphon Road
28	Access Rd Near Lawson Syphon
29	Lawson Syphon Rd (92.25 mAHD)
30	Lawson Syphon Rd (93.15 mAHD)

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Appendix A

APPENDIX A: GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.

redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.

disaster plan (DISPLAN) A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.

discharge The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).

ecologically sustainable development (ESD) Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.

effective warning time The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.

emergency management A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.

flash flooding Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.

flood Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.

flood awareness Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.

flood education Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.

flood fringe areas The remaining area of flood prone land after floodway and flood storage areas have been defined.

flood liable land Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).

flood mitigation standard The average recurrence interval of the flood, selected as part of the floodplain risk

	management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the flood liable land concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPLs are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the standard flood event in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can

	increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: ▪ the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or ▪ water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or ▪ major overland flow paths through developed areas outside of defined drainage reserves; and/or

- the potential to affect a number of buildings along the major flow path.

mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State•s rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
peak discharge	The maximum discharge occurring during a flood event.
Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.

probability A statistical measure of the expected chance of flooding (see AEP).

risk Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.

runoff The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.

stage Equivalent to water level. Both are measured with reference to a specified datum.

stage hydrograph A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.

survey plan A plan prepared by a registered surveyor.

water surface profile A graph showing the flood stage at any given location along a watercourse at a particular time.

wind fetch The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

Memorandum

TO: Julie Rogers, Richard Brown
CC: Marcus Walsh, Peter Nankivell
FROM: Erin Askew, Felix Taaffe
DATE: 4 October 2013
SUBJECT: Edward River Flood Study – Early Gaugings
PROJECT NUMBER: 112002

This memorandum details the quality assessment of early gauging points used in the Edward River Flood Study. The gaugings, taken in July 1931 at the National Bridge in Deniliquin, are instrumental in estimating the magnitude of early recorded flood events, which themselves are used to determine the design flows of the river to be used in the current Flood Study. Following a detailed assessment of the gauged data, it was concluded that the data had significant inaccuracies, and should therefore not be considered representative of the early flood behaviour. The inaccuracies arose from insufficient measurements being carried out as part of the gauging process.

Background

The 12 gaugings taken over a two week period in July 1931 represent a significant portion of what is known about the flood behaviour of the river at Deniliquin in the early 20th century. The data, shown in Figure 1, was taken during the flood event of that year, including the peak on the 7th of July. The next time the river was gauged at this level was in 1955 and 1956, by which time a levee had been partially constructed, changing the floodplain topography and subsequently flood behaviour. Because some of the highest events on record occurred before 1931 (including the highest and second highest recorded events in 1870 and 1917 respectively), the 1931 gaugings were thought to represent the best data for estimating the discharge of these early flood events.

A 2D hydraulic model of the river, based on the TUFLOW software, was not able to reproduce the 1931 flood behaviour. The model, which was set up as part of the Edward River Flood Study, was used to simulate the floodplain as it existed in the early 20th century. This included removing many topographic features that exist today, including several levees, the built up section around Davidson Street, the flowpath through the town that has been filled in, the National Bridge, the Mulwala Canal, and several other features. The stage-discharge relationship produced by the model fitted the early gauged data, except for that from 1931. The model-produced rating table and the gaugings are shown in Figure 1, as well as Rating Table No. 56, the established rating table for the period. The figure shows that behaviour is generally reproduced, except for the section above 8.6 m, at which point the 1931 gaugings show a distinct change in shape, showing more flow for a small increase in height.

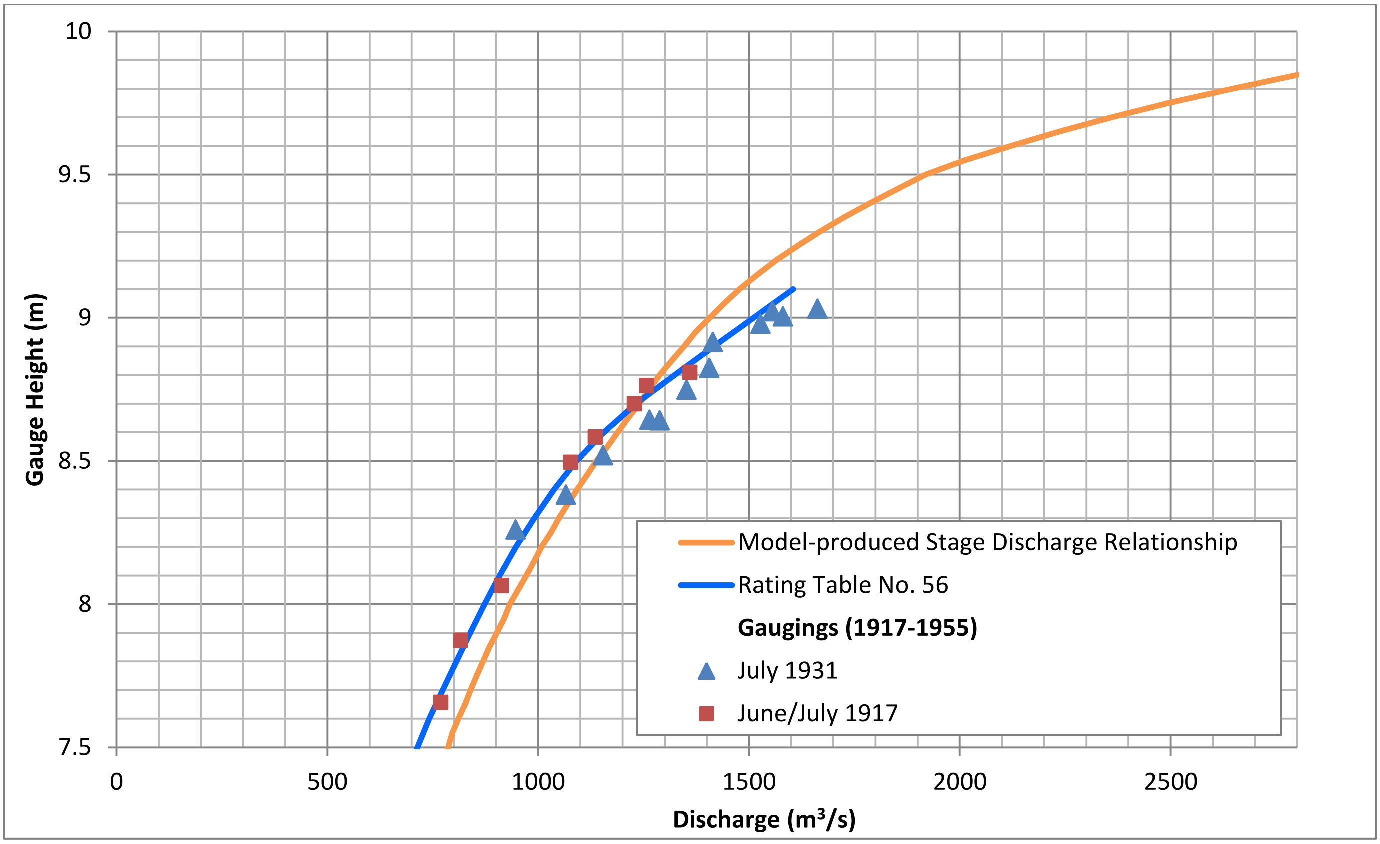


Figure 1 1917 and 1931 Gaugings with Model-produced Rating Table

The discrepancy between modelled and gauged flows lead to considering what possible floodplain features were not being incorporated in the model. Changes were made to the model that could potentially cause an increase of 270 m^3/s at a depth of 9 m (the discrepancy in Figure 1). This investigation was aided by the components that were gauged in 1931 as comprising separate flowpaths, as recorded on the gauging cards. Specifically, the total flow was made up of four components: the main channel, Brick Kiln Creek, over Davidson St, and the lagoon through the town. The flow distribution through these components is shown in Figure 2.

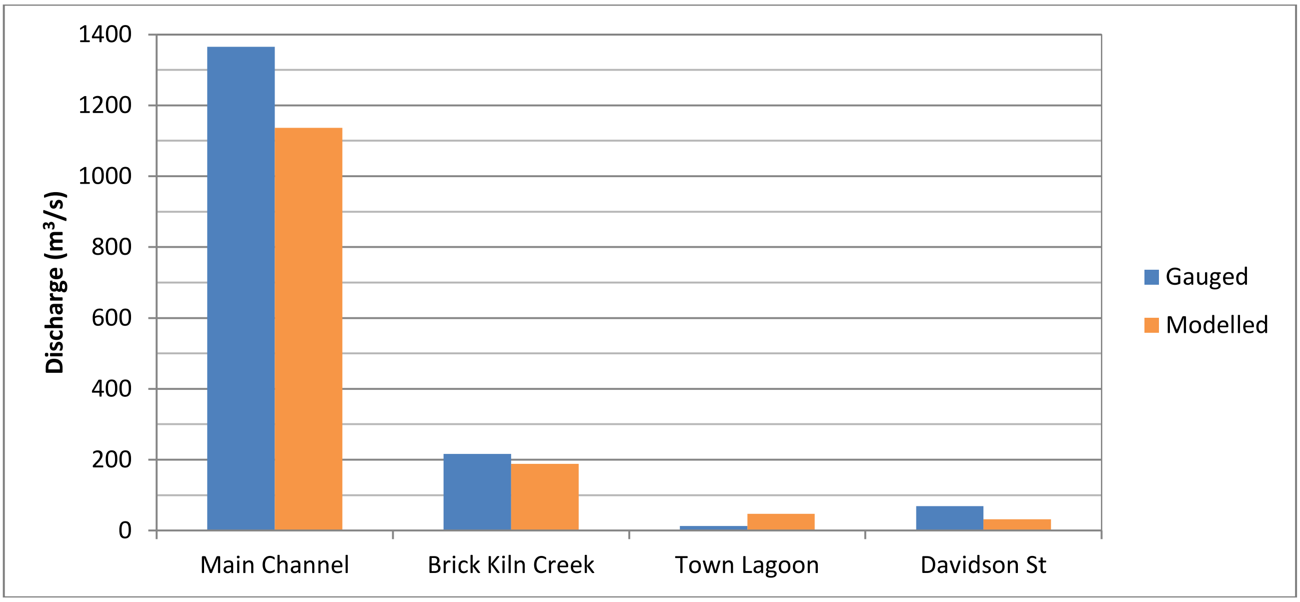


Figure 2 1931 Peak Flow Distribution - Gauged and Modelled

As can be seen in Figure 2, the 270 m^3/s discrepancy is almost solely contained in the difference between the gauged and modelled discharge in the main channel (it should be noted that the flow over Davidson St was not gauged, but rather estimated by an unspecified method that did not involve measuring velocity or depth). Because the discrepancy only exists above 8.6 m, the gauged data is indicating that at this depth, the main channel begins conveying significantly more flow, causing the stage-discharge relationship to 'flatten' out. This in turn indicates that there is a change in the channel cross-section at this point which allows the channel to transmit the additional flow. The only possible feature which could cause this is the 50 m wide overbank section on the east bank. However, both the model and the gauged data show that the velocity over this section is relatively low in the depth range being examined, and that the velocities were not large enough to cause the significantly increased flow.

The discrepancy was therefore isolated to the centre section of the main channel, which the gauged data showed to be faster moving than that of the model. Figure 3 shows the gauged and modelled velocity across the channel when the river is at the peak of the 1931 event. As can be seen, the velocity in the overbank sections (approximately chainage 100 to 160, and chainage 0) is relatively consistent between the two, whereas in the deep section of the channel the velocity is around 60% larger in the gauged data. This prompted further investigation into the estimation methods used in the 1931 data.

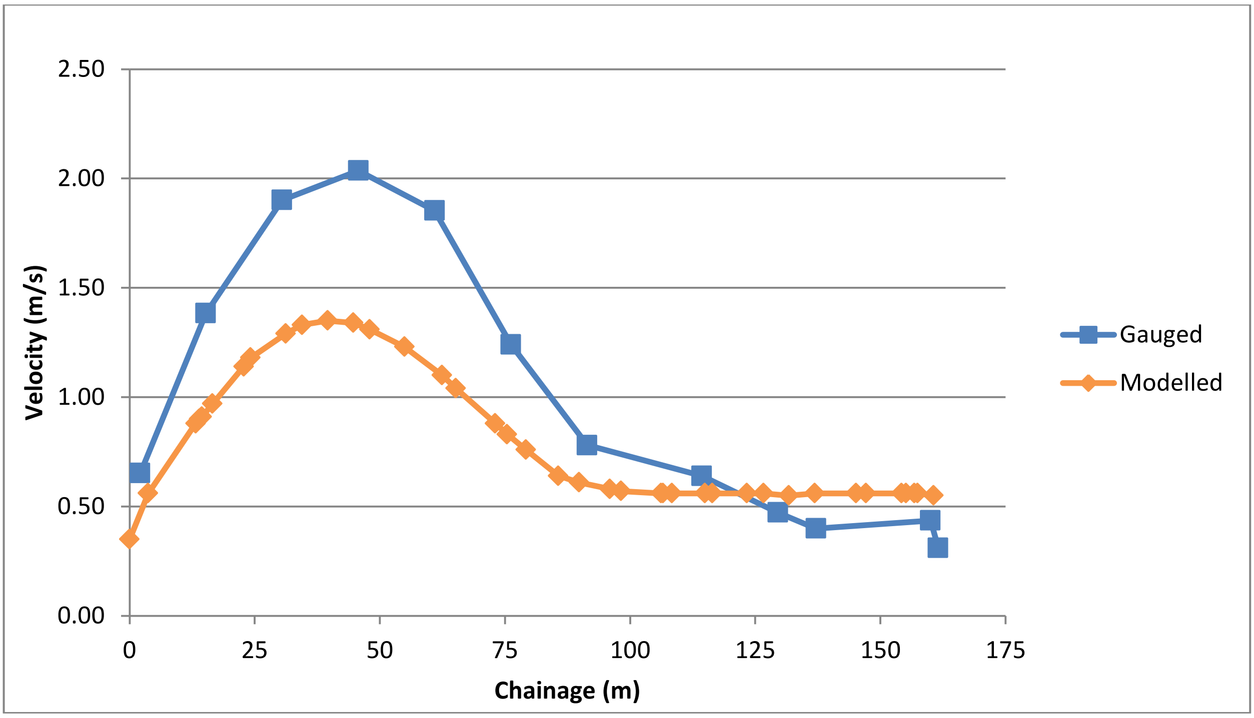


Figure 3 Gauged and Modelled Velocity Across the Main Channel at 1931 Peak Height

Assessment of Gauged Data from 1931

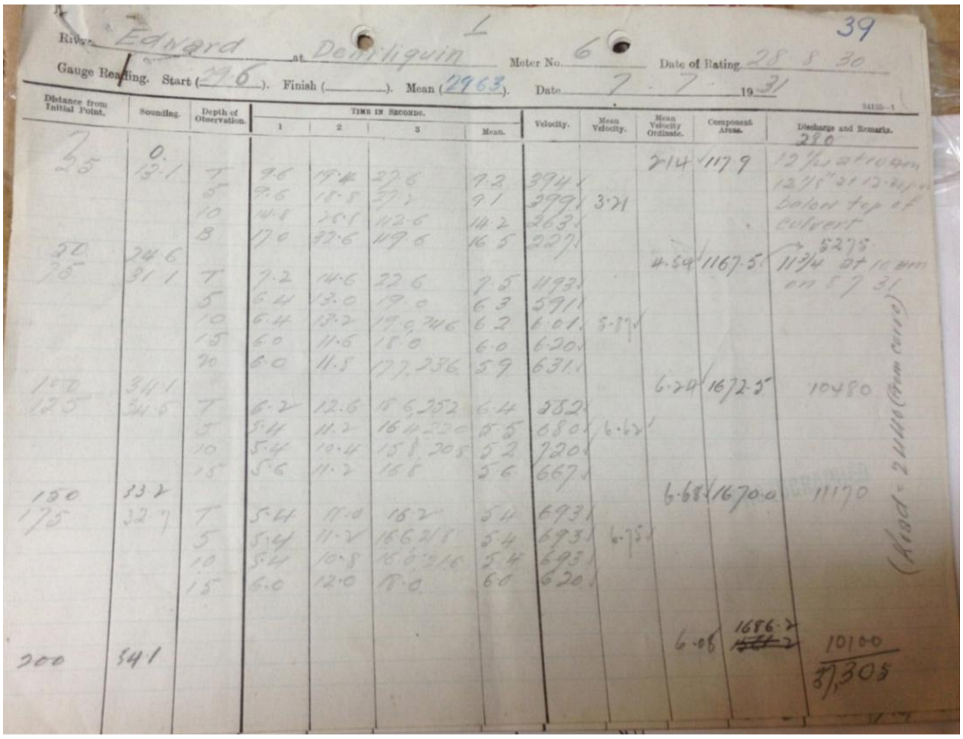
Method

To assess the accuracy of the 12 gaugings, the measurements and observations that are used to calculate the height and flow of the river were digitised and checked for possible errors. These measurements were available in their original form, recorded on a series of cards for each event, an example of which is shown in Figure 4 below. Copies of the cards were sourced from the NSW Office of Water in Tumut. The following types of error were considered:

- Arithmetic errors, arising from mistakes in the arithmetic used in the calculation steps. These were checked for by digitising the measurements and re-calculating each step using a spreadsheet.
- Measurement error, whereby measurements were incorrectly recorded during transcription. Isolated occurrences of measurement error were checked for by comparing each measurement to those around it. Systemic biases could not be checked for without a second source of data from the time.
- Instrument error, due to flaws in the instruments used to measure depth and velocity. Very little information was given on the type of instrument used, and as such this source of error could not be evaluated.

- Schematisation errors, a result of flawed assumptions in the method used to convert the measured velocities to a single flow. A spreadsheet was used to check each of the calculation steps for possible flaws.

The gauged data from the peak of the flood event, on July 7th, was used to assess the error. The estimated flow from this date represents the biggest discrepancy between the model produced rating table and the 1931 gaugings (approximately 270 m^3/s at a depth of 9.031 m). Any error found in this data was then checked for in the remaining dates.


River Edward at Deniliquin Meter No. 6 Date of Rating 28 8 30

Gauge Reading. Start (29.6). Finish (). Mean (29.63). Date 7 7 1931

Distance from Initial Point.	Sounding.	Depth of Observation.	Time in Seconds. 1	2	3	Mean.	Velocity.	Mean Velocity.	Mean Velocity Ordinate.	Component Areas.	Discharge and Remarks.

Figure 4 Example of Gauging Card

Results

Arithmetic errors were found to exist in sections of the calculation process; however, the resulting errors were relatively small and generally cancelled each other out. Using the raw measurements (i.e. the velocity measured across different depths and chainages) with corrected calculations did not produce a significantly different total flow estimate. Similarly, the data did not suggest there to be any isolated measurement error which could cause the discrepancy in the total estimated flow.

In the original method used to measure the velocity of the main channel's flow, the velocity was measured at around five different depths approximately every 15 m across the river. The average velocity of each 15 m interval was then found and, by multiplying it by the interval's area, the flow was estimated. However, an examination of the data found that the velocity was not measured to the full depth for a portion of the channel. For the four intervals in the middle of the channel (which conveys the majority of the flow), the velocity was only measured to between half and three quarters of the total depth. Figure 5 shows the channel cross section with the points at which the velocity was measured, as well as the magnitude of the velocity measurement.

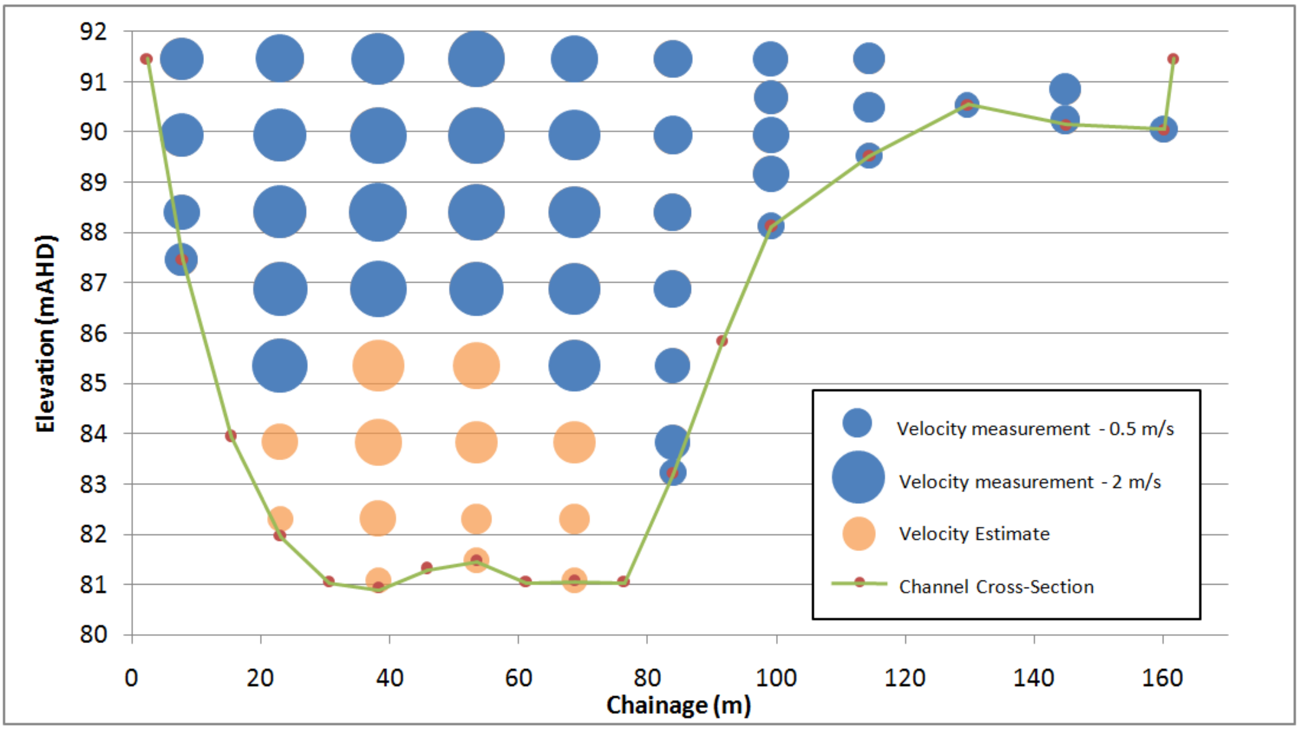


Figure 5 Location of Velocity Measurements Across Channel Cross-section – 1931 Peak Flow.

Not measuring the velocity to the full depth resulted in the average velocity in those sections being over-estimated. The exaggerated velocity estimate arose from the fact that the velocity in the upper section of the channel is typically higher than that closer to the bed, which experiences more friction effects. The flawed assumption is that the velocity in the top half is a good representation of the entire section.

The magnitude of this error was estimated by comparing the existing flow estimate with a re-calculated estimate that includes low velocities near the channel bed. The estimated channel velocities, shown in orange in Figure 5, are only an approximation and are based on observed velocity profiles in other rivers, reproduced in standard hydraulic texts. Table 1 shows the change in velocity for each of the four sections that were not measured to the full depth.

Table 1 Original and Re-calculated Velocity Estimates

Chainage (m)	Original Velocity Estimate (m/s)	Re-calculated Velocity (m/s)	% Decrease
23	1.80	1.48	18
38	2.02	1.60	21
53	2.06	1.51	27
69	1.65	1.32	20

Given that these four intervals contain around 60% of the total flow (including that in Brick Kiln Creek, over Davidson Street and through the town), it is apparent that the error in the total flow may be significant, at approximately 10-15%. Reducing the estimated flow by this amount results in the peak flow decreasing by around 250 m^3/s, which brings it in line with the stage-discharge relationship estimated by the 2D hydraulic model at this level. The re-estimated gauging is shown in Figure 6.

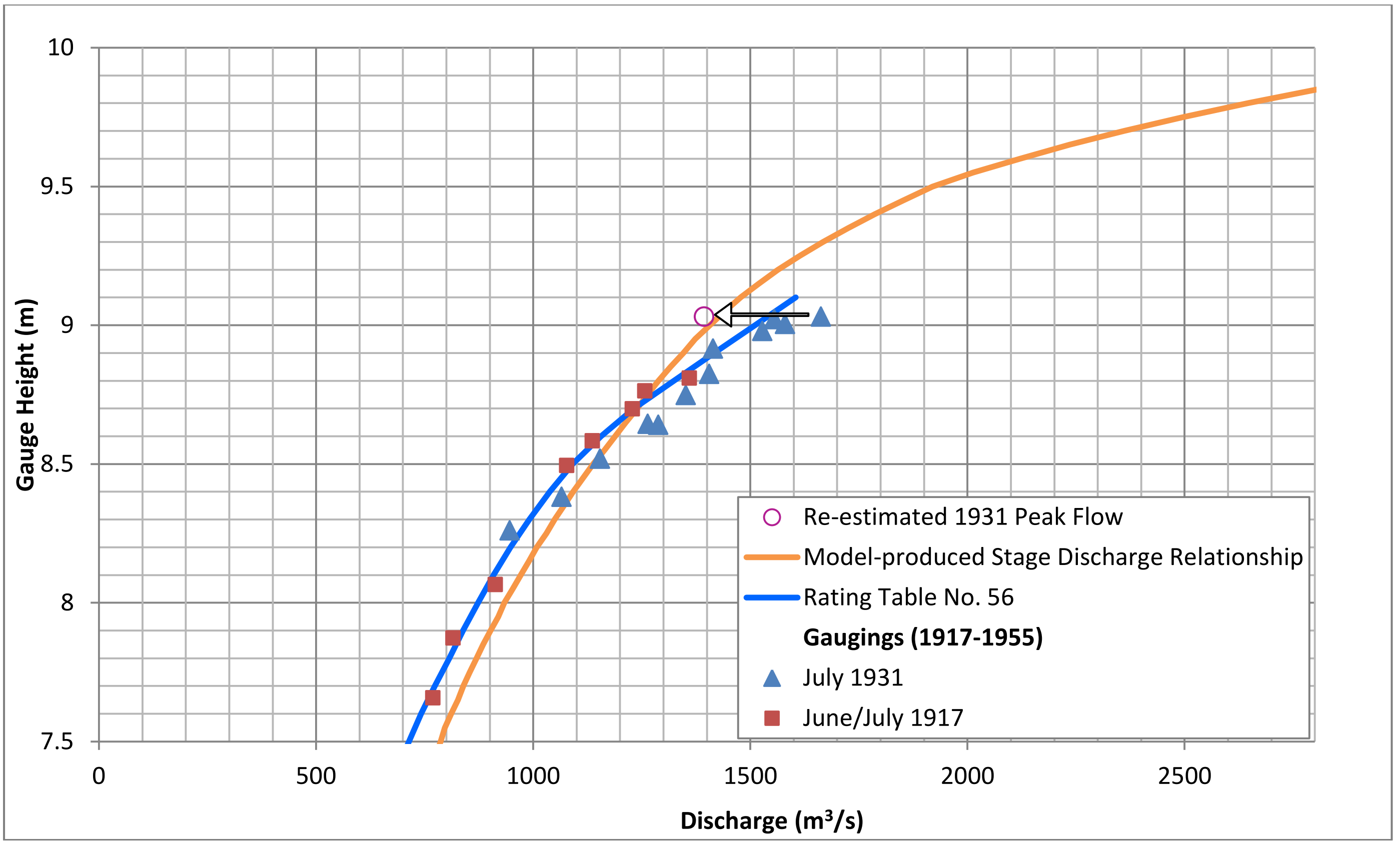


Figure 6 Gaugings and Rating Tables with Re-estimated 1931 Peak Flow Gauging

The gaugings from the three dates prior to the peak flow were then checked for the same error. It was found that the gaugings, from the 2nd, 3rd and 4th of July, all contained sections in the middle of the channel that were not fully measured. Specifically, each of the gaugings contained three sections in the middle of the channel that were only gauged to between 45% and 70% of the total depth, while the adjacent measurements, on the channel sides, were to the full depth. An estimate of the magnitude of error was not made for these gaugings.

Comparison to Other Gauged Data

The gauged data from July 1931 was then compared to that of other years to determine if the identified measurement error was confined to the 1931 gaugings. If the measurement error was not widespread, it could be concluded that the error is the reason the gaugings appear anomalous compared to the rest of the gauged data, and why the model was not able to replicate the data. The gaugings from the peak of the flood events of 1955 and 1993 were used for comparison.

The gauging from September 4th 1955 (the peak of the 1955 flood, at 8.946 m) was found to also contain insufficient measurements in the vertical cross-section; however, the effect on the total discharge estimate was not as severe as in 1931. The gauging in the main channel consisted of measurements taken every 20 ft across the river. The six measurements from 160 ft to 260 ft consisted of a single measurement at each, taken to a depth of 5 ft (out of a possible depth of around 33 ft). However, the effect on the total discharge was not as influenced by this error for three reasons:

1. The average velocity in the vertical cross-section is closer to the velocity at 5 ft than it is to the average velocity of the top half of the channel (what was measured in 1931). Therefore, despite less measurements being taken, the estimate was closer to the average velocity. The error is around 10%, compared to the 20% shown in the fourth column of Table 1.
2. The zone where the depth was not fully measured was only 100 ft wide (18% of the main channel flow width), compared to 150 feet in 1931 (28% of the main channel flow width).
3. The peak flood level was less than that of 1931, meaning the absolute value of the error was less.

Although the 1955 gauging was found to be imperfect in its estimation method, the magnitude of the error is less than that of 1931. When the 1955 flow is re-calculated using an estimate of the missing velocity measurements, the decrease is only around 100 m^3/s, which, given that it was already to the right of the trend shown in surrounding gaugings, means it does not shift significantly with respect to the overall trend. The re-estimated flow is shown in Figure 7.

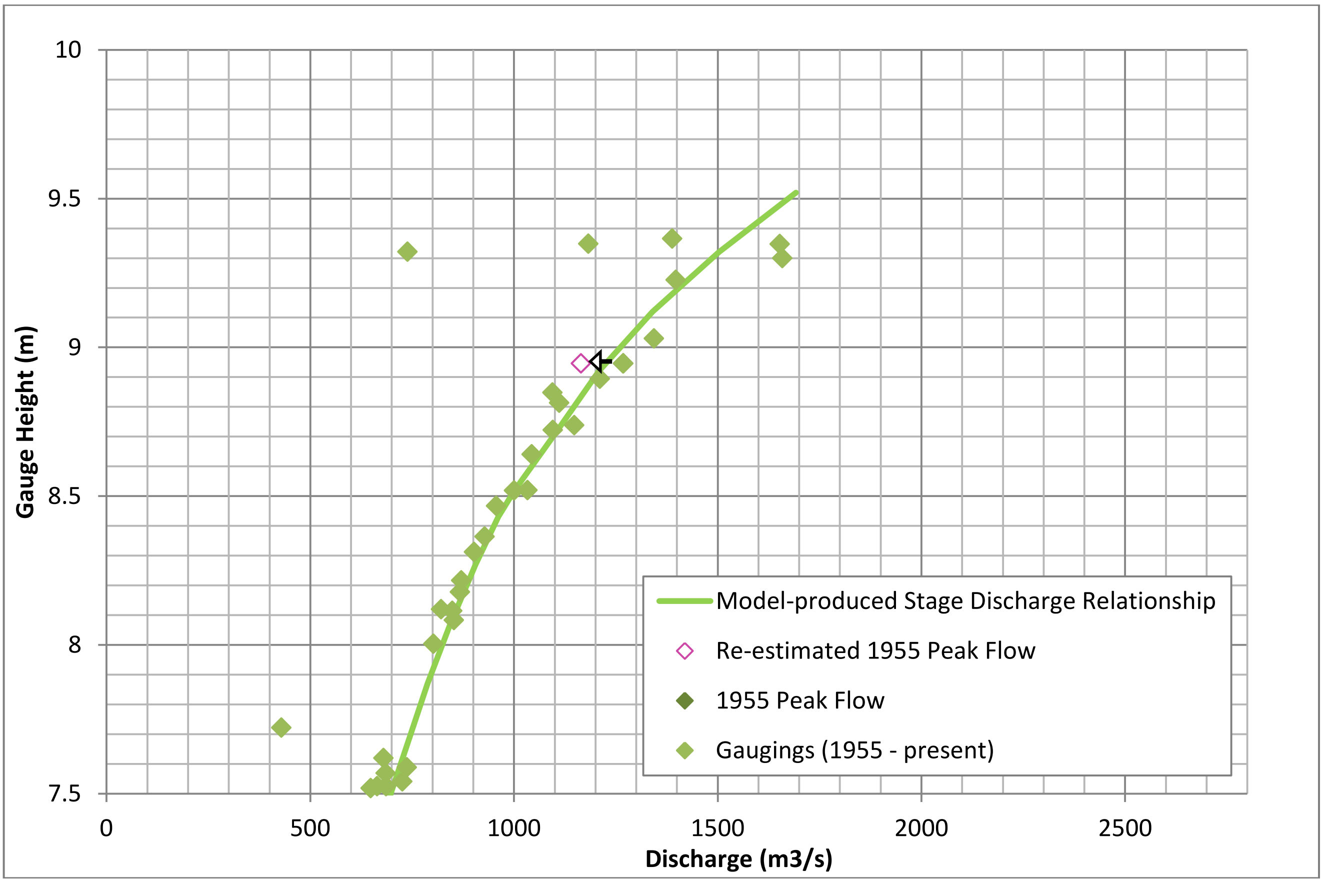


Figure 7 Gaugings and Rating Tables with Re-estimated 1931 Peak Flow Gauging

The gauging from October 16th 1993, taken at a level of 8.47 m, was found to have a complete representation of the velocity across the channel cross-section, as shown in Figure 8. Measurements were taken every 6 m across the channel to the full depth at each chainage. This produced the expected velocity profile (i.e. lower velocities closer to the river bed) and the estimation of total flow can be considered accurate. Furthermore, the velocity profile reinforces the estimates made to re-estimate the 1931 velocity profile.

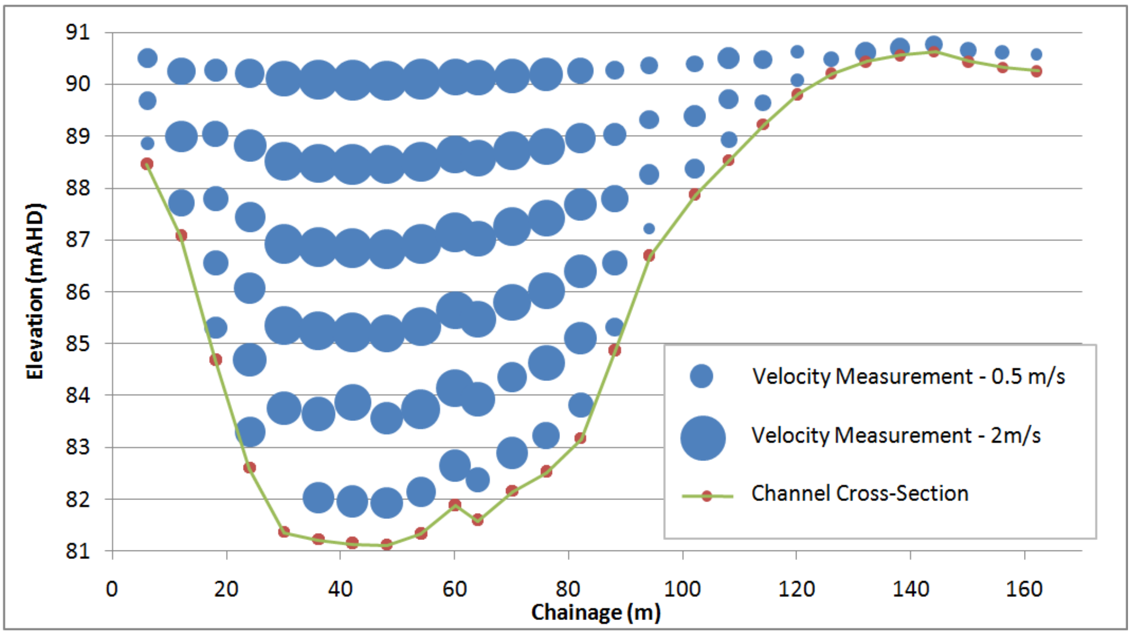


Figure 8 Location of Velocity Measurements Across Channel Cross-section – 1993 Peak Flow

Conclusions

Assessment of possible errors in the gauged data from the 1931 flood event found that flaws in the measurement process resulted in a significantly over-estimated discharge for each of the gaugings. By measuring only the top half of the channel in the main flow area, the velocity was estimated to be larger than it actually was, resulting in an over-estimation of discharge. The error was found to be localised to the July 1931 gaugings of those investigated, causing the gaugings to follow a different trend to that of the remaining gauged data for the station. The finding was in line with the results of the 2D hydraulic model set up for the area, which suggested that the river could not have conveyed the flow that was estimated by the gaugings, at the river height which was recorded.

The magnitude of this error means the 12 points from 1931 should not be considered representative of the stage-discharge relationship of the river in the early 20th century, and should therefore not be used in calibrating the 2D hydraulic model. The points could possibly be recalculated using the incomplete raw data by estimating the missing velocity, however, not enough is known of the actual velocity profile to do this, beyond an approximate estimation of the error.

In light of these amendments, the best estimation of the stage-discharge relationship for the early 20th century and late 19th century is the rating table produced by the 2D hydraulic model. The model incorporates known topographic features of the era, including some of the bathymetric data collected as part of the early gaugings.