Goulburn Broken Catchment Management Authority

Berrigan Shire and Moira Shire

Murray River Regional Flood Study

Dicks/Seppelts levees to downstream of the Ulupna Creek confluence

Study Report

Report No. J150/R02 Final

November 2011

WATER TECHNOLOGY
WATER, COASTAL & ENVIRONMENTAL CONSULTANTS

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Front cover: Remnants of a previous levee at Dixons Bend (Source: Water Technology)

Acknowledgments

The Murray River Regional Flood Study (Dick's Spillway to downstream of the Ulupna Creek confluence) has been prepared drawing on the support and contribution of a number of groups and individuals. Without this support the study would not have so readily achieved its outcomes and delivered upon the study objectives.

Water Technology's study team acknowledges the contributions of the following people and agencies:

- Guy Tierney, Goulburn Broken Catchment Management Authority
- Graham Henderson (formerly of Berrigan shire)
- Paula Toovey (formerly of Moira Shire)
- Victorian State Emergency Service
- New South Wales State Emergency Service
- Department of Environment and Climate Change (NSW)
- Department of Sustainability and Environment (Vic)

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1 INTRODUCTION

This report documents the technical investigations undertaken as part of the Murray River Regional Flood Study.

The Murray River Regional Flood Study is an investigation of flood behaviour (height, depth, extent, velocity) and risk for an area along the Murray River from the Dicks/Seppelts Levee Spillway to downstream of Ulupna Island. The study area includes the towns of Cobram, Barooga and Tocumwal. Figure 1-1 shows the study area for the Murray River Regional Flood Study.

Key components of the study include:

- Topographic survey: defines floodplain terrain
- Hydrologic analysis: determines frequency and magnitude of flood flows
- Hydraulic analysis: assesses flood behaviour
- Flood mapping: prepares mapping for flood height, extent and depth
- Flood response planning: prepares flood response plans for relevant agencies
- Land use planning: provide flood behaviour information for determination of flood related planning requirements
- Performance of existing mitigation schemes: assesses the level of service that are currently provided by the mitigation schemes
- Structural mitigation measure assessment: identifies possible mitigation measures

The study was undertaken by a study team led by Water Technology on behalf the Goulburn Broken Catchment Management Authority (GBCMA), Moira Shire Council (MSC) and Berrigan Shire Council (BSC).

The study was funded under the Natural Disaster Risk Management Studies Programme by the Australian, Victorian and New South Wales Governments with contributions from the Moira and Berrigan Shires. The study provides a foundation for co-ordinated floodplain management along the Murray River between the various stakeholder agencies.

The study team was led by Water Technology with sub-consultants Michael Cawood and Associates, and LICS (subsequently part of Sinclair Knight Mertz) providing specialist input.

The structure of this report is as follows:

- Section 2 – provides a description of key waterway and floodplain features
- Section 3 – reviews available data
- Section 4 - describes the community consultation process
- Section 5 – outlines approach and outcomes from the hydrologic analysis
- Section 6 – discusses the hydraulic analysis for the existing conditions
- Section 7– outlines the existing performance of structural mitigation measures and potential structural mitigation augmentation

- Section 8– details a range of non-structural mitigation measures
- Section 9– summarises the key conclusions and recommendations

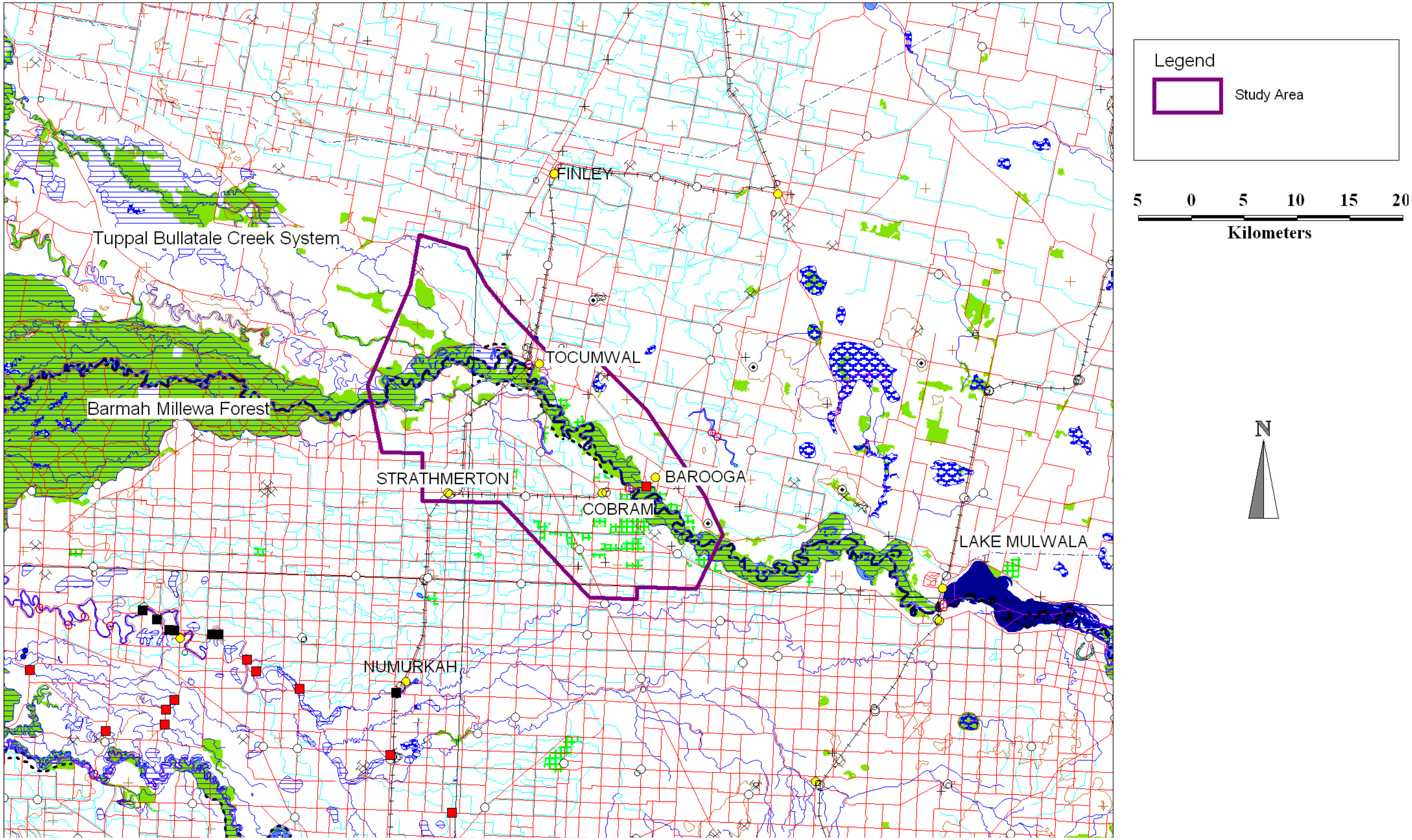

Figure 1-1 Study area

2 STUDY AREA FEATURES

The study area contains the first extensive floodplain reach along the Murray River downstream of Albury. The study area has experienced numerous floods since European settlement, with major events occurring in 1870, 1917, 1931, 1956, 1974, 1975 and 1993.

Immediately downstream of Yarrawonga, the floodplain is relatively confined. The area adjacent to Dick's levee is a natural lower floodplain section between sand hills. Across this section, significant flow breaks out from the river once the channel capacity is exceeded. These floodplain flows, under natural floodplain conditions, are likely to extent to the south and west in vicinity of Strathmerton. This extensive pattern of flooding was observed during the 1917 flood event.

During the 1975 event, extensive sandbagging was required along Dick's levee to prevent overflow (RWCV et. al. 1986). As part of subsequent flood mitigation works for Cobram, Dick's levee was raised and reinforced.

Considerable irrigation infrastructure (channels and drains) have been constructed across the Victorian floodplain since the 1940's. This infrastructure is likely to control flood behaviour across the floodplain.

An extensive rural levee system flanks the Victorian floodplain from Cobram to Yielima, a distance of some 51 km (RWCV et. al. 1986) known as the Public Works Department "PWD" levee. Recent flood mitigation works adjacent to Cobram have raised and strengthened the levee, known as the Cobram town levee. This raised levee provides flood protection for Cobram and was designed to protect the town from a 100 year ARI magnitude flood including 600 mm freeboard. Downstream of the Cobram town levee, the "PWD levee" (Public Works Department) provides a lower level of protection to the rural floodplain. The PWD levees were first constructed in 1895 (RWCV et. al. 1986). The levees have been breached and re-instated following major events in 1916, 1917, 1956 and 1975 (RWCV et. al. 1986). During the 1975 flood, major levee breaches occurred at Brentnalls, Dixons Bend, and Cleaves. In recent times, the GBCMA has strengthened the PWD levees at a number of locations that were identified as major priorities from a levee audit in 1996 (CMPS&F). This included locations of major levee breaches that occurred in the 1975 flood and an 8 km length of levee from the Cobram town levee to Greens Lane at Koonoomoo. The protection offered by the PWD levee varies along its length. Any breakouts at Dixons Bend flow along Sheepwash Creek to re-join the Murray River via Ulupna Creek on the southern side of Ulupna Island.

Adjacent to Dick's Levee, a natural low floodplain section occurs on the New South Wales side at Seppelts levee. This levee marks the upstream end of the Barooga Cowal Depression. The Barooga Cowal Depression flows generally parallel to the current Murray River course. Effluent flows from the river along the Barooga Cowal Depression can give rise to flooding in Barooga and Tocumwal.

Along the New South Wales side, the Barooga levee between Barooga and Tocumwal provides protection to both rural areas and to Tocumwal. This is a substantial levee and is up to 4 m in height. The Tocumwal town levee consists of several levee segments extending from the golf course to downstream of the road bridge, and was designed to protect the town from a 100 year ARI magnitude flood including 600 mm freeboard.

Around Ulupna Island, a 14 km long levee provides varying degrees of flood protection. Generally, the protection is lower than the PWD levee sections, with overtopping having occurred in the 1974 and 1975 events.

Downstream of Tocumwal on the New South Wales/ north side of the river, two significant effluent flow paths leave the Murray, Tuppal and Bullatale Creeks. Extensive private and public levees have been constructed in this area that influence the distribution of overbank flows.

Figure 2-1shows the key study area features.

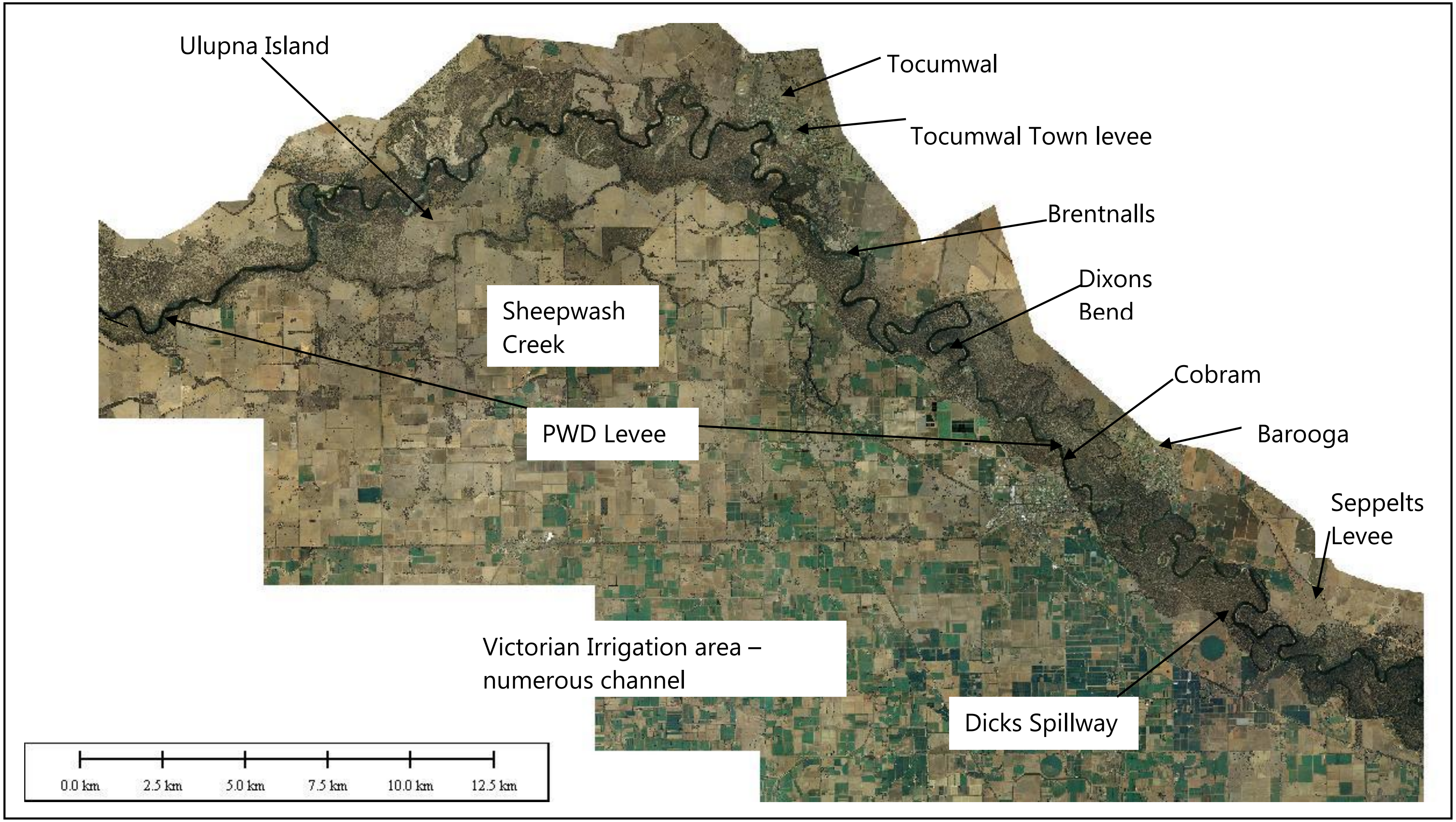

Figure 2-1 Study area features

3 AVAILABLE INFORMATION

3.1 Previous Studies

An extensive flood study and flood mapping investigation jointly undertaken by Rural Water Commission of Victoria and the Water Resources Commission of New South Wales was completed in 1986. This investigation considered the Murray River from Lake Hume to the South Australian border. The investigation collated historical flood information, assessed flood magnitude and prepared indicative 100 year ARI flood maps. The flood maps largely utilised observed flood levels and profiles with an additional buffer.

This previous investigation provided valuable and extensive background information and descriptions of flooding for the current study.

GBCMA provided working files from the Rural Water Commission of Victoria, containing details of numerous flood level investigations for locations throughout the study area.

These resources have been reviewed and drawn upon as necessary to provide background, context and verification of the current study approach and outcomes.

3.2 Hydrologic data

There are two key long-term streamflow gauging stations of relevance to the study, as listed in Table 3-1.

Table 3-1 Details of Streamflow Gauges

Gauge Number	Station Name	Catchment Area (km^2)	Data Type	Length Of Record
409025	Murray River @ Yarrawonga (Downstream Of Weir)	27,300	Mean Daily Flows	2/1/1938 - 30/11/1960
			Daily Maximum Instantaneous	1/12/1960 - 1/12/2004
409202	Murray River @ Tocumwal	29,008	Mean Daily Flows	2/1/1908 - 9/12/1974
			Daily Maximum Instantaneous	10/12/1974 – 1/12/2005

A detailed discussion of the available streamflow data is provided in Section 5.2.

3.3 Topographic data

3.3.1 Overview

There were two major sources of topographic information gathered during the course of the investigation, these being:

- Aerial Laser Scanning (ALS)
- Field Survey

Following the collection and processing of the topographic information, a detailed Digital Terrain Model (DTM) was developed as the basis for the establishment of a hydraulic model of the study area. The sources of the topographic information are discussed in more detail below.

3.3.2 Aerial Laser Scanning

The main source of topographic information utilised in the development of the hydraulic model was the topographic survey data collected for the Murray Darling Basin Commission using the Light Detection and Ranging (LiDAR) airborne remote sensing technique (also known as Aerial Laser Scanning, ALS). This technique allows for the collection of detailed topographic data over large areas. The raw LiDAR data has an average spacing of 2.4 m for points on the ground, with a vertical accuracy of 0.15 m Root Mean Square Error (RMSE) and horizontal accuracy of 1.0 m RMSE. The raw LiDAR data was processed and interpolated onto 1 m and 10 m cell grids. Overall, the digital elevation model developed from the LiDAR data provided excellent topographic detail of the study area from which to base the hydraulic model.

3.3.3 Field Survey

Extensive field survey was undertaken by Sinclair Knight Merz (formerly LICS). The field survey component included:

- Structure survey (arrangement, type, number, inverts & photographs) for syphons and subways. Total: 56 syphons/subway structure.
- Opportunistic elevation survey for channel banks at road crossings (particularly Channel 1 and 2)
- Opportunistic elevation survey for drain banks at road crossings (particularly Drain 3 and 5)
- Structure survey (arrangement, type, number, invert & photographs) for culverts/bridges along Goulburn Valley Highway between Murray Valley Highway and the Murray River. We understand there are 6 culvert/bridge structures along the Goulburn Valley Highway in above section.
- Structure survey (arrangement, type, number, invert & photographs) for culverts/bridges along Tocumwal – Strathmerton Railway between Strathmerton and the Murray River. Total: 12 culverts

Appendix A contains general arrangements drawing of the field survey undertaken in this study.

4 CONSULTATION

4.1 Overview

During the study, consultations undertaken focused around the following two groups:

- Key personnel – Goulburn Broken CMA, Berrigan shire and Moira Shire
- Technical steering committee

Further details of the consultation are provided in the following sections.

4.2 Key personnel – Goulburn Broken CMA, Berrigan Shire and Moira Shire

Throughout the course of the study, regular progress updates were provided to Goulburn Broken CMA, Berrigan Shire and Moira Shire via telephone discussions, emails and informal meetings.

Draft study reports and flood mapping outputs were reviewed by Goulburn Broken CMA, Berrigan Shire and Moira Shire.

In addition, a presentation was made to the Berrigan Shire Council on 30 November 2006.

4.3 Technical steering committee

The technical steering committee comprised of officers from the Goulburn Broken CMA, Berrigan Shire, Moira Shire, Department of Sustainability and Environment (DSE), Department of Environment, climate change and Water (DECCW), Victorian State Emergency Service (VICSES), New South Wales State Emergency Service (NSWSES) and Goulburn Murray Water (GMW). Also, a number of local landholders, with particular experience of flood events, were part of the technical steering committee.

Four meetings with the technical steering committee were conducted. A bus tour of the study area was undertaken in conjunction with the second meeting. A number of local landholders provided commentary on the nature of flooding during the 1975 and 1993 events.

5 HYDROLOGIC ANALYSIS

5.1 Background

This section documents the hydrologic analysis undertaken as part of the Murray River Regional Flood Study. The key aim of the hydrologic analysis is the determination of design flood hydrographs for input to the hydraulic analysis. The primary input (inflow) point for the hydraulic analysis is upstream of the study area, the Murray River at Yarrawonga. As such the hydrologic analysis focused on design flood hydrograph estimation at Yarrawonga. For this study, estimates of the design flood hydrographs for 1 in 10 year to 1 in 500 year ARI events are provided.

The observed flood behaviour in the study area is dependent on flood characteristics such as peak flow, flood volume and duration (i.e. hydrograph shape). History suggests that flood events with similar peak flows but different flood volumes and durations can result in significantly different flood behaviour. Hence consideration of peak flows, flood volumes and durations is an important aspect of the hydrologic analysis.

The contributing catchment for the Murray River to Yarrawonga is approximately 27,300 km^2. The catchment area can be broken into three sub-catchments, the Upper Murray River (above Lake Hume), the Kiewa River and the Ovens River. Flood flows in the study area can arise from varied contributions from these three sub-catchments.

Two general approaches were considered for the determination of the design flood hydrographs:

- Rainfall based approaches
- Streamflow based approaches (flood frequency analysis, FFA)

Rainfall based approaches utilise historical and/or design rainfall with a runoff routing model (e.g. RORB) to yield estimates of flood flows (flood hydrographs) for a range of magnitudes (ARIs). The use of this approach requires assumptions to be made about the temporal and spatial variation of rainfall input to the runoff routing model. The contributing catchment to the study area is large, hence the assignment of appropriate rainfall temporal and spatial patterns can be difficult and would be accompanied with a high degree of uncertainty. Subsequently, the suitability of the rainfall based approach is limited for this application.

Streamflow based approaches analyse available streamflow data to assess flood characteristics (peak flow and volume). A streamflow based approach relies on the length and reliability of observed streamflow data. In this approach we assess the individual flood characteristics (peak flow and volumes) separately and combine these individual characteristics to yield design flood hydrographs. A reliable streamflow record length of around 100 years is available within the study area, thus facilitating and providing some confidence in the use of streamflow based approaches for this study.

Australian Rainfall and Runoff (ARR) (IEAust 1999) provides a methodology for the derivation of flood hydrographs from frequency analyses of peak flow and flood volume. This methodology is underpinned by the assumption that a design flood hydrograph for a given ARI has a peak flow and flood volume with the same ARI.

The key components of the ARR methodology are summarised as follows:

- Peak flow frequency analysis: evaluates the frequency and magnitude of peak flows.

- Flood volume frequency analysis: evaluates the frequency and magnitude of flood volumes.
- Flood event rank comparison: assesses the relative rank of peak flows and flood volumes from selected historical events.
- Peak flow – flood volume ratio: determines the peak flow to flood volume ratios from selected historical and design flood events.
- Historical flood hydrograph selection: examines historical flood hydrographs with peak flow – volume ratios similar to the design flood events and selects representative historical flood hydrographs suitable for use as design flood hydrographs.
- Design flood hydrograph scaling: determines design flood hydrographs by scaling representative historical flood hydrographs.

The peak flow and flood volume frequency analyses are detailed in Sections 5.3 and 5.4 respectively. The remaining components are summarised in Section 5.5. A discussion of the key issues arising from the hydrologic analysis is provided in Section 5.6.

As highlighted earlier, the hydrologic analysis focused on the determination of design flood hydrographs for the Murray River at Yarrawonga. Accordingly the reporting focuses on the design estimates at Yarrawonga with a summary of design estimates provided at Tocumwal to enable comparison with previous studies.

5.2 Available streamflow data

This section describes the available streamflow data from gauges and estimates of design stream flows from previous studies. As discussed, this hydrologic analysis has adopted a streamflow based approach. The robustness of the design estimates from this approach relies on the length and reliability of the available streamflow data. Streamflow gauges were established in the early 1900's providing nearly 100 years of data.

The study area has been the subject of numerous flood related investigations in the past that are a useful information source for streamflow data and observed flood behaviour.

The structure of this section is as follows:

- Section 5.2.1: outlines the streamflow data collected by various New South Wales and Victorian government agencies
- Section 5.2.2: briefly summarises the previous key flood investigations and available information
- Section 5.2.3: discusses the reliability and suitability of the available streamflow data

5.2.1 Agency gauged streamflow data

Two streamflow gauges of direct relevance to this analysis are located within or adjacent to the study area. Table 5-1 summarises the details of these two streamflow gauges.

Table 5-1 Available agency gauged streamflow data

Gauge	Station Name	Operator/ Contractor	Catchment Area (km^2)	Data Type	Length Of Record
409025	Murray River @ Yarrawonga (Downstream of Weir)	Murray Darling Basin Commission / Department of Natural Resources (NSW)	27,300	Mean Daily Flows	2/1/1938 - 30/11/1960
				Daily Maximum Instantaneous	1/12/1960 - 1/12/2004
409202	Murray River @ Tocumwal	Department of Sustainability & Environment (Victoria) / Thiess Environmental	29,008	Mean Daily Flows	2/1/1908 - 9/12/1974
				Daily Maximum Instantaneous	10/12/1974 - 1/12/2005

Both the Yarrawonga and Tocumwal gauges have a lengthy record of streamflow. For both gauges the initial periods of measurements were undertaken using a manually read staff. This measurement technique yields only mean daily flow. Following the installation of continuous recorders from around the 1960's, measurement of instantaneous flow data was possible.

Streamflow data for the Murray River at Yarrawonga from 1938 to 1960 has been disaggregated manually to mean daily flow from cumulative flow data. Advice from New South Wales Department of Natural Resources indicates that the data may contain human 'typographical' errors due to the manual transcription process (Rod Kerr NSW DNR, pers. comm. December 2006).

Overtopping of the levees adjacent to Cobram can and has led to significant flow across the Victorian floodplain. This floodplain flow effectively bypasses the streamflow gauge at Tocumwal. As a result, the higher flows at Tocumwal should be treated with caution.

The streamflow data, outlined in Table 5-1, was obtained from the respective operator/contractor for use in this analysis.

Further comments on the available data are provided in Section 5.2.3.

5.2.2 Previous Studies

Murray River Flood Plain Management Plan (1986)

The Murray River Flood Plain Management (MRFPM) (RWCV et. al. 1986) study evaluated 1 in 20, 1 in 50 and 1 in 100 year ARI peak flows at both the Yarrawonga and Tocumwal streamflow gauges.

At Yarrawonga, the MRFPM study (RWCV et. al. 1986) documents a peak flow frequency analysis for the period following the construction of Yarrawonga Weir in 1938. No specified end date to the period of record employed in the analysis was documented, however it can be assumed that records up to the study date (1985-1986) were used. This peak flow frequency analysis yielded a 1 in 100 year ARI estimate of 410,000 ML/d. However, the MRFPM study adopted the estimate of the 1917 flood, 390,000 ML/d, as the 1 in 100 year

ARI peak flow. No clear justification was provided for the adoption of the 1917 peak flow as the 1 in 100 year ARI/1% AEP probability event. Further, no discussion is provided as to why the 1917 peak flow was not included in the peak flow frequency analysis.

For Tocumwal, the MRFPM study (RWCV et. al. 1986) highlights the low reliability of the streamflow data for large floods due to the possible bypassing of the gauge. The MRFPM study (RWCV et. al. 1986) cites this low reliability as cause not to undertake a peak flow frequency analysis at Tocumwal. The MRFPM study (RWCV et. al. 1986) employs a correlation analysis using Yarrawonga peak flows to estimate peak flows at Tocumwal. The results of this correlation analysis were provided without further details on the nature of the analysis. Further, MRFPM study (RWCV et. al. 1986) indicates that *" … a separate analysis of recorded peak flows at Tocumwal predicts a 1% flow of 272,000ML/d …"*. It is unclear as to the nature of the separate analysis.

Table 5-2 shows the design peak flow estimates from the MRFPM study (RWCV et. al. 1986).

Table 5-2 MRFPM (RWCV et. al. 1986) Design peak flow estimates

ARI (years)	Yarrawonga Design peak flow (ML/d)	Tocumwal Design peak flow (ML/d) *
1 in 20	235,000	210,000
1 in 50	325,000	290,000
1 in 100	390,000	340,000

* - does not include bypassed flows

Victorian State Rivers and Water Supply Commission and New South Wales Department of Water Resources working files

The GBCMA provided a number of working files from the Victorian State Rivers and Water Supply Commission (SR&WSC) and New South Wales Department of Water Resources (DWR). These various calculations and correspondence related to flooding within the study area. The working files appear to date from the late 1970's to early 1980's.

The files contained two annual historical peak flow series at Yarrawonga. From the available files, the exact derivation of these two peak flow series is unclear. Significant differences occur in peak flow estimates for several large flood events between the two series. For the purposes of this analysis, the two series are labelled SR&WSC-A and SR&WSC-B respectively. Further discussion of these peak flow series are provided in Section 5.2.3. A listing of the two series is provided in Appendix B.

A daily hydrograph for the month of October 1917 is contained in the working files, with a peak flow of 125,000 cubic feet per second (306,000 ML/d) recorded (which is consistent with the SR&WSC-A data set). This hydrograph allows the estimation of flood volume for the 1917 flood event. Further discussion of the 1917 hydrograph is provided in Section 5.2.3. The SR&WSC/DWR working files do not contain any further streamflow data for the Murray River at Tocumwal.

An analysis of the historic flood events for the Murray River at Yarrawonga is useful in that it provides insight if not data into relative flood flow magnitudes observed in the past. The study team was supplied historic flood height information from the GBCMA. The three largest observed events in the Murray River can be ranked based on peak gauge height. The 1870 flood clearly stands out as the largest flood on record. However, the 1867 and 1917 are more difficult to distinguish from gauge heights with the 1867 flood generally accepted as the larger of the two. An estimate of discharge is available for the 1917 event. However, flood flow estimates for the two largest floods, 1870 and 1867, cannot be derived due to a lack of appropriate rating curve data. Through the hydraulic analysis for this study an estimated rating curve at Tocumwal has been developed (refer to Section 6.5).

5.2.3 Discussion

Three annual peak flow data sets are available for the Murray River at Yarrawonga. These three data sets, for the purpose of this analysis, are referred to as follows:

- Agency gauged data: Streamflow data outlined in Section 5.2.1. Annual peak flow extracted based on calendar years. Available for period 1938 – 2004.
- SR&WSC-A: Peak flow from SR&WSC/DWR working files, as outlined in Section 5.2. Available for period 1905- 1979.
- SR&WSC-B: Peak flow from SR&WSC/DWR working files outlined in Section 5.2. Available for period 1905- 1979.

The design peak flow estimates from a flood frequency analysis are heavily influenced by the reliability of the streamflow data used, and in particular by large flood events. Table 5-3 and Figure 5-1 provide a comparison of estimated peak flows for several large flood events from the three available data sets.

Table 5-3 Murray River at Yarrawonga: Comparison of estimated peak flows for significant flood events (1905-1979)

Flood event	Peak flow (ML/d)		
	Gauged agency data	SR&WSC-A	SR&WSC-B
1917	N.A.	306,000	390,000
1931	N.A.	179,000	210,000
1955	181,000	181,000	171,000
1956	204,000	208,000	193,000
1958	157,000	163,000	157,000
1970	184,000	187,000	166,000
1973	142,000	140,000	140,000
1974	196,000	285,000	193,000
1975	234,000	431,000	280,000

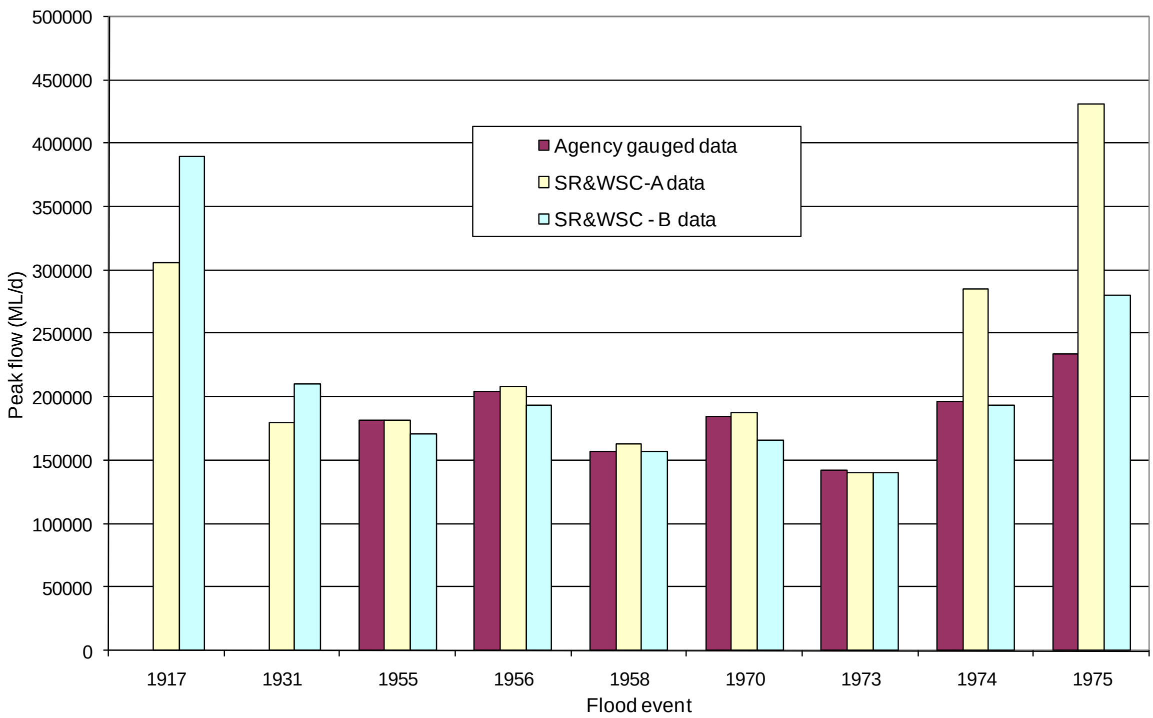

Figure 5-1 Murray River at Yarrawonga: Comparison of estimated peak flows for significant flood events (1905-1979)

Considerable variation in peak flow estimates occur for the 1917, 1974 and 1975 events between the three data sets. These three flood events are amongst the four largest recorded events for the Murray River at Yarrawonga. This variation has a significant impact on the magnitude of the design peak flow estimates derived from a flood frequency analysis.

Examination of the SR&WSC-A data set reveals several inconsistencies, with the 1974 and 1975 peak flows being significantly larger than the estimates in the other two data sets. The SR&WSC-A 1975 flow estimate is considerably larger than two of the three estimates for the 1917 event. The 1917 flood event is considered as being the largest recorded event in terms of peak flow (RWCV et. al. 1986). Conversely, the SR&WSC-A 1917 estimate is considerably less than the two other estimates.

Further comparison of the three data sets is provided in Appendix B.

The inconsistencies in the SR&WSC-A data noted above raise concerns over the reliability of this information and any frequency analysis based on it. It was therefore considered that the inconsistencies in the SR&WSC-A data were sufficient to set aside this information from any further analysis. However, it is noted that definitive examination of the derivation of the data set is difficult due to the lack of available documentation.

No peak flow estimate for 1917 is available from the agency gauge data set. Hence no comparison is possible. Generally the peak flows are similar for the agency gauged data and the SR&WSC-B data. Notable differences in peak flow occur in 1975 and 1970. The lack of available documentation prevents a thorough investigation of any underlying reasons for these differences.

The agency gauged data is the most recently derived data set and may contain revisions since the derivation of the SR&WSC-A and SR&WSC-B data. However, the documentation of any revisions was not available. Given that this is the most recent derivation, the agency gauged data was adopted for the Murray River at Yarrawonga over the period 1938 to 2004.

As a number of significant flood events occurred prior to 1938, inclusion of these in the flood frequency analysis was considered desirable. As discussed above, the inconsistencies contained in the SR&WSC-A data raised concerns over the reliability of this information and in the absence of any other data source, the SR&WSC-B data set was adopted for the period 1905-1937. It is recognised that the reliability of the SR&WSC-B data set is difficult to define, however the inclusion of the pre - 1938 period was considered worthwhile for the additional length of record provided.

The 1870 flood event has been documented to be larger than the 1917 flood (RWCV et al 1986), however no peak flow estimates are available. The occurrence of the 1870 and 1917 flood events underscore the importance of longer periods of stream flow data in the peak flow frequency analysis. Additional discussion of the 1870 flood event is provided in Section 3.5.

The analysis of flood volumes requires a time-series of daily flows to enable accumulation of flow over a given time period. The SR&WSC-A and SR&WSC-B data sets contain annual maximum peak flows only. Hence the SR&WSC-A and SR&WSC-B data sets are unsuitable for use in a flood volume analysis. The agency gauged data set was therefore employed for the flood volume analysis at both Tocumwal and Yarrawonga. As the agency gauge data at Yarrawonga is only available for the period from 1938 to 2004, the flood volume analysis was limited to this period.

The daily hydrograph for the month of October 1917 allows the estimation of the flood volume associated with this event. The peak flow for this hydrograph is 306,000 ML/d and is line with the SR&WSC-A data set. As discussed, the reliability of the SR&WSC-A data set is considered questionable. Given this uncertainty, the absolute flood volume from the daily hydrograph was considered unsuitable for direct inclusion in the flood volume frequency analysis. However, the relativity between the peak flow and flood volume was considered useful in providing guidance on the variation of hydrograph shape. Further discussion on the use of the 1917 daily hydrograph is provided in Section 5.5.2.

The three largest observed events in the Murray River can be ranked based on peak gauge height. The 1870 flood clearly stands out as the largest flood on record with a gauge height of 125.4 m AHD at Yarrawonga. The 1867 and 1917 events are difficult to distinguish based on gauge heights as both are estimated to be approximately 124.9 m AHD. An estimate of discharge is available for the 1917 flood based on an old rating curve. However, flow estimates for the two largest floods, 1870 and 1867, cannot be derived due to the lack of a robust rating curve applicable at these times. This makes their inclusion in any peak flow flood frequency analysis problematic.

5.3 Peak flow frequency analysis

5.3.1 Overview

From the data sets outlined in Section 5.2.1, a series of annual maximum peak flows was determined at both Tocumwal and Yarrawonga.

A peak flow frequency analysis involves the fitting of a probability distribution to observed series of annual maximum peak flows. In this analysis the following probability distributions were trialled:

- Log-Normal Distribution (LP3)
- Generalised Pareto (GP)

It was found that overall the Generalised Pareto (GP) distribution provided the best fit to the peak flow series at both Yarrawonga and Tocumwal. The results from the GP distribution were therefore adopted for the study. To enable comparison with the previous MRFPM study (RWCV et. al. 1986), results from the LP3 distribution are also presented.

Expected probability estimators were applied in this study. The parameters of the probability distributions in this study were estimated using a Bayesian framework.

5.3.2 Yarrawonga

Table 5-4 summarises the design peak flow estimates from the frequency analysis at Yarrawonga.

As discussed in Section 5.2.3, no peak discharge data exists or can be reasonably derived for the 1870 and 1867 historical flood events. This typically means that these flow events must be excluded from any conventional flood frequency analysis. However a technique is available whereby ungauged floods may be included in the flood frequency analysis. The use of 'Censored Flows' allows the inclusion of historical flood events for which no gauged discharge exists, by considering the number of floods in the pre-gauging period greater than a threshold discharge. This method has been implemented in this analysis to establish the impact of the 1867 and 1870 historic floods on the flood frequency estimates for Yarrawonga. The threshold discharge chosen was 390,000 ML/d as both floods are regarded as being larger than the 1917 event.

Table 5-4 Design peak flow estimates at Yarrawonga (409025)

ARI (years)	1905-2004 LP3 Dist. (ML/d)	1905-2004 GP dist. (ML/d)	1938-2004 GP dist. (ML/d)	1905-2004 plus 1870,GP dist. (ML/d)	1905-2004 plus 1867 & 1870,GP dist. (ML/d)	MRFPM 1986,LP3 (ML/d)
10	178,000	186,000	152,000	193,000	215,000	--
20	240,000	236,000	190,000	251,000	292,000	235,000
50	334,000	300,000	236,000	328,000	406,000	325,000
100	416,000	346,000	269,000	387,000	445,000	390,000
200	507,000	392,000	300,000	448,000	527,000	--

Figure 5-2 shows the LP3 and GP distributions (based on the record 1905 -2004) along with the observed peak flow data for the Yarrawonga gauge. For clarity, the LP3 estimates are shown in green with the GP estimates shown in red.

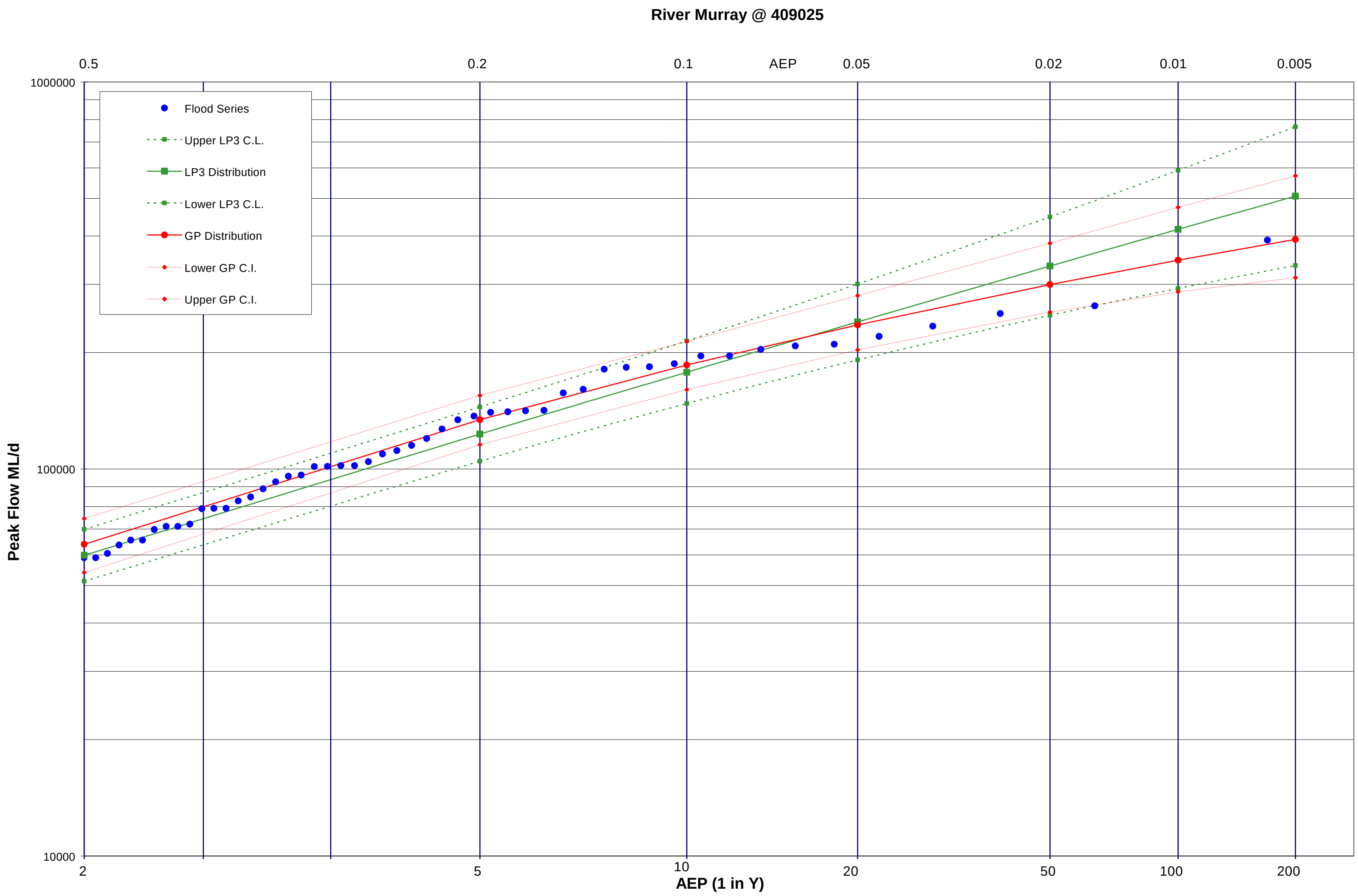

Figure 5-2 Murray River at Yarrawonga Annual Peak Flow Flood Frequency Analysis (Streamflow record 1905-2004)

The LP3 distribution predicts design flood magnitudes at Yarrawonga that are consistent with those developed as part of the MRFPM (RWCV et. al. 1986) study. The GP distribution, without the inclusion of the 1867 and 1870 events, predicts significantly lower estimates than those of the MRFPM study for the 1 in 50 year ARI and greater events. However, the GP distribution fits the observed data significantly better than the LP3 distribution. The inclusion of the 1867 and 1870 events for the GP distribution considerably increases the design peak flow estimates.

Limiting the analysis to the period 1938 to 2004, yields significantly lower design peak estimates for both the distributions. This is due to the exclusion of the significant flood events in 1906, 1909, 1917, 1922 and 1931. The 1 in 100 year ARI GP distribution peak flow increases from 269,000 ML/d to 346,000 ML/d with inclusion of the 1905-1937 data. This is a 28% increase for the 1 in 100 year ARI peak flow estimate. Further, the inclusion of the 1870 event increases the 1 in 100 year ARI peak flow estimate to 387,000 ML/d. The inclusion of both the 1867 and 1870 events increases the 1 in 100 year ARI peak flow estimate to 445,000 ML/d. The variation in design flows due to the period of record analysed is further highlighted by determining approximate average recurrence intervals from the GP distributions for 3 large historical flood events, as displayed in Table 5-5.

Table 5-5 Approximate historical event recurrence interval at Yarrawonga

Event	Estimated Peak flow (ML/d)	Approximate average recurrence interval for GP distribution (years)		
		Period 1905-2004 plus 1867 and 1870	Period 1905-2004	Period 1938-2004
1917	390,000	~100	~200	> 500
1956	204,000	~ 15	~ 17	~30
1975	234,000	~17	~20	~50

The study steering committee conducted lengthy discussion and debate regarding the appropriate peak flow estimates to be adopted. A consensus position was reached to adopt the peak flow estimates from the GP distribution with the inclusion of the 1870 event. The adopted peak design flow estimates at Yarrawonga are shown in Table 5-6.

Table 5-6 Adopted Design peak flow estimates at Yarrawonga (409025)

Average Recurrence Interval (years)	1905-2004 plus 1870 GP distribution ML/d
10	193,000
20	251,000
50	328,000
100	387,000
200	448,000
500	528,000

Further discussion on the effect of the data period employed is provided in Section 5.6.

5.3.3 Tocumwal

Table 5-7 summarises the design peak flow estimates from the frequency analysis at Tocumwal with the MRFPM (RWCV et. al. 1986) design peak flow estimates provided for comparison.

Table 5-7 Design peak flow estimates at Tocumwal

Average Recurrence Interval, (years)	LP3 Distribution ML/d	GP Distribution ML/d	MRFPM 1986, (LP3) ML/d
10	128,000	152,000	--
20	163,000	189,000	210,000
50	215,000	236,000	290,000
100	257,000	269,000	340,000
200	304,000	300,000	--

Figure 5-3 depicts the two distributions along with the observed data for the Tocumwal gauge. For clarity, the LP3 estimates are shown in green with the GP estimates shown in red.

Figure 5-3 shows that neither the LP3 and GP distribution fit the higher flows (> 50 year ARI) well. As discussed, there is considerable uncertainty surrounding the streamflow data at Tocumwal for higher events. This is due the considerable flow across the Victorian floodplain, outside the PWD levee. Given this uncertainty in the streamflow data, design flow estimates at Tocumwal were not used in this study.

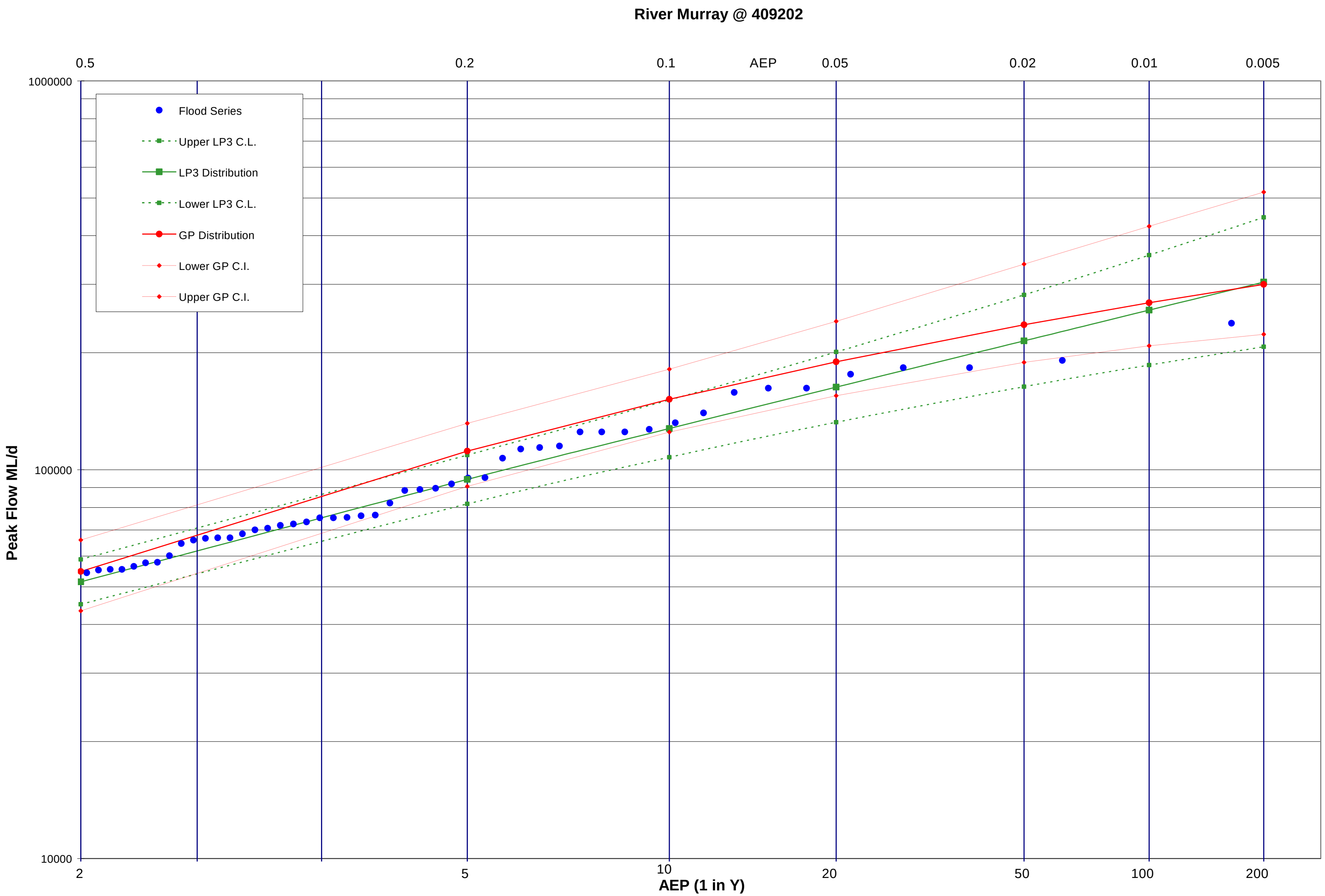

Figure 5-3 Murray River at Tocumwal Annual Peak Flow Flood Frequency Analysis

5.4 Flood volume frequency analysis

5.4.1 Overview

From the data sets outlined in Section 5.2.1, flood volume series were determined at both Yarrawonga and Tocumwal. The flood volumes were evaluated over three durations; 14 days, 21 days and 28 days. As discussed in Section 5.2.3, the agency gauged data period was used in the flood volume analysis, Yarrawonga 1938 -2004 and Tocumwal 1908 – 2005.

Similar to the peak flow frequency analysis, the following probability distributions were trialled:

- Log-Normal Distribution (LP3)
- Generalised Pareto (GP)

It was found that the Generalised Pareto (GP) distribution provided the best fit to the flood volume at both Yarrawonga and Tocumwal. The results from the GP distribution are presented along with the estimates from the LP3 distribution for comparison.

Expected probability estimators were applied in this study. The parameters of the probability distributions were estimated using a Bayesian framework.

As noted, three flood volume durations were assessed. The choice of these durations was founded on consideration of durations for a number of large observed floods. The selected durations were found to generally bracket the observed flood durations and were considered appropriate for this analysis. The sensitivity of the modelled flood behaviour to the event duration is discussed in Section 6.4.6.

5.4.2 Yarrawonga

Table 5-8 summarises the design flood volumes for the three selected event durations at Yarrawonga.

Table 5-8 Design flood volume estimates at Yarrawonga (409025)

Average Recurrence Interval (years)	LP3 distribution			GP distribution		
	14 Day Vol. (ML)	21 Day Vol. (ML)	28 Day Vol. (ML)	14 Day Vol. (ML)	21 Day Vol. (ML)	28 Day Vol. (ML)
10	1,467,000	1,977,000	2,423,000	1,514,000	2,032,000	2,519,000
20	1,948,000	2,614,000	3,199,000	1,831,000	2,446,000	3,049,000
50	2,681,000	3,578,000	4,380,000	2,192,000	2,913,000	3,660,000
100	3,316,000	4,411,000	5,404,000	2,427,000	3,214,000	4,061,000
200	4,028,000	5,341,000	6,555,000	2,633,000	3,476,000	4,417,000

Figure 5-4shows the frequency analysis for the 14 day flood volume with the observed data for the Yarrawonga gauge. For clarity, the LP3 estimates are shown in green with the GP estimates shown in red.

Appendix B contains the frequency curves for the 21 day and 28 day volumes.

Comparison of the two distributions to the observed data for the 14 day volume, reveals that the GP distribution fits the higher volume events better. As such, the GP distribution estimates were adopted in this study.

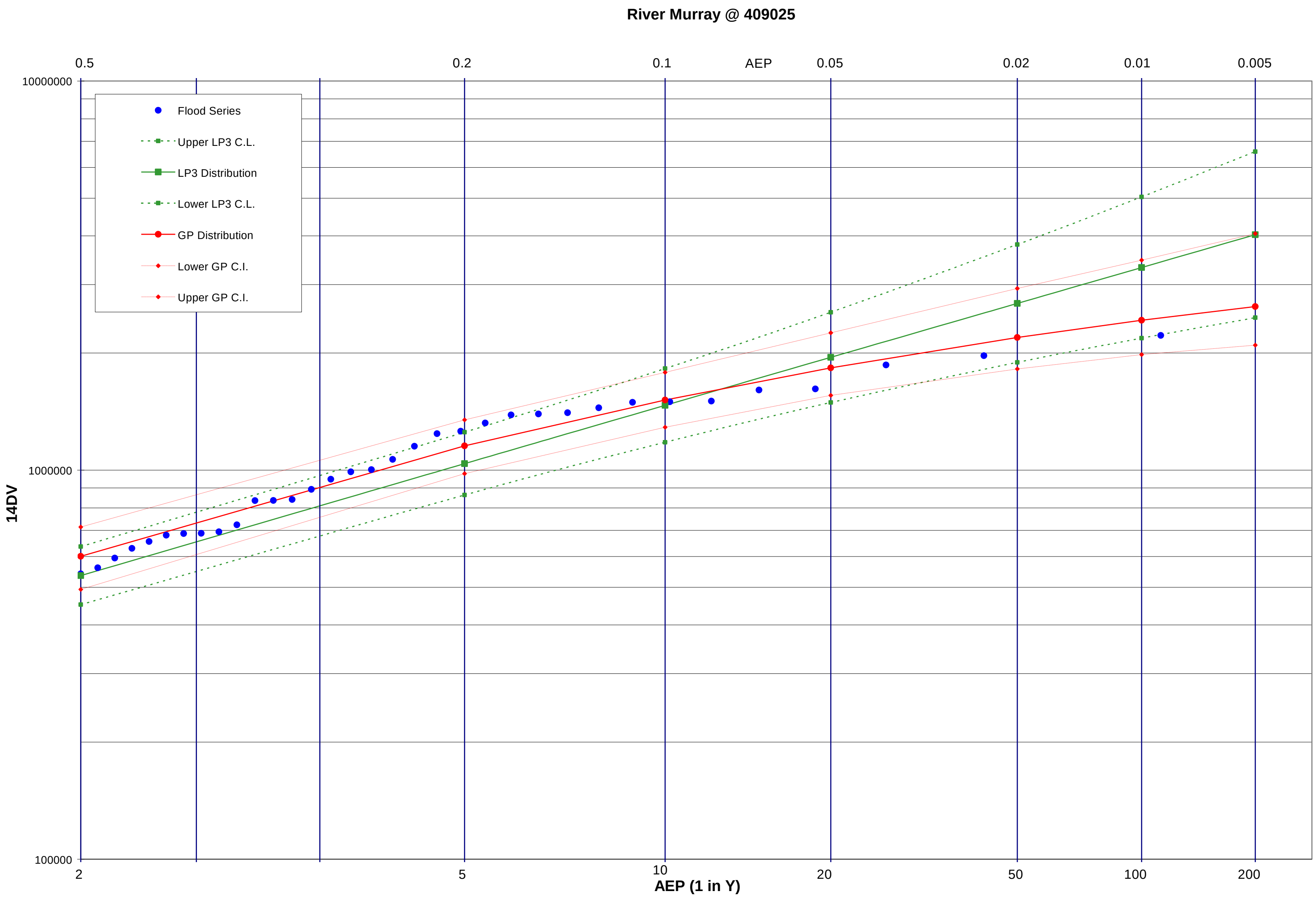

Figure 5-4 Murray River at Yarrawonga,. 14-day volumes flood frequency analysis

5.4.3 Tocumwal

Table 5-9 summarises the design flood volumes for the three selected event durations at Tocumwal.

Table 5-9: Design flood volume estimates at Tocumwal (409202)

Average Recurrence Interval (years)	LP3 Predicted Flow			GP Predicted Flow		
	14 Day Vol. (ML)	21 Day Vol. (ML)	28 Day Vol. (ML)	14 Day Vol. (ML)	21 Day Vol. (ML)	28 Day Vol. (ML)
10	1,321,000	1,799,000	2,238,000	1,430,000	1,934,000	2,411,000
20	1,593,000	2,135,000	2,644,000	1,664,000	2,237,000	2,781,000
50	1,959,000	2,577,000	3,176,000	1,897,000	2,533,000	3,141,000
100	2,246,000	2,918,000	3,581,000	2,030,000	2,699,000	3,340,000
200	2,544,000	3,267,000	3,994,000	2,135,000	2,827,000	3,492,000

Figure 5-5shows the 14 day flood volume frequency analysis along with the observed data for the Tocumwal gauge. For clarity, the LP3 estimates are shown in green with the GP estimates shown in red.

As for the volumes at Yarrawonga, the GP distribution provided a better fit than the LP3 distribution for the higher volume events

Appendix B contains the frequency curve for the 21 day and 28 day volume.

5.4.4 Discussion

The GP distribution provided a better fit to the flood events at Yarrawonga and Tocumwal, particularly for the higher volumes (> ~ 30 year ARI).

A comparison of the design flood volume estimates at Yarrawonga and Tocumwal revealed higher estimates at Yarrawonga. It would be expected, in the absence of significant inflow/outflows between Yarrawonga and Tocumwal, that the flood volume estimates would be similar. The differences in volume estimates are considered to reflect the uncertainty in streamflow data at both sites.

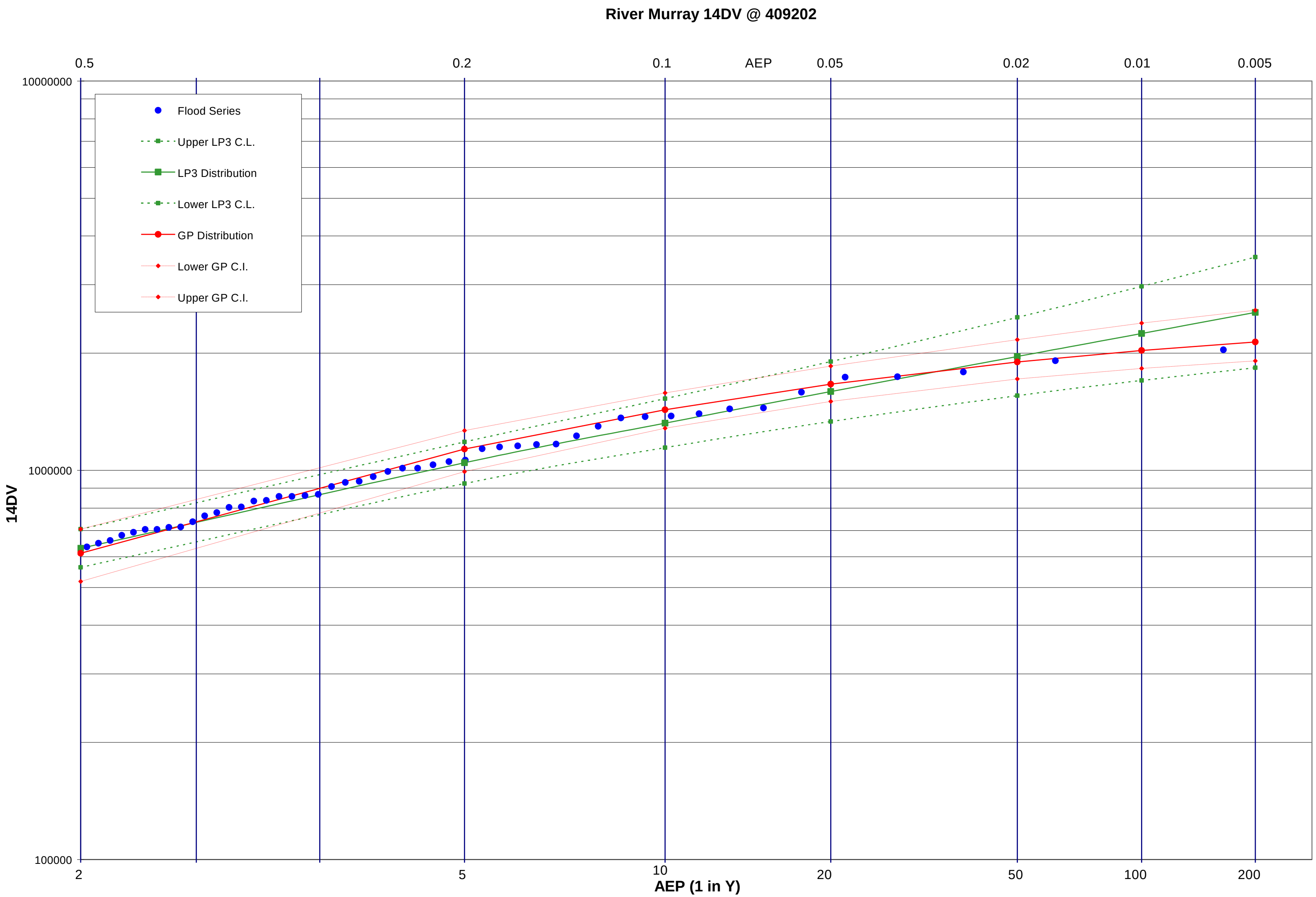

Figure 5-5 Murray River at Tocumwal 14 day volume flood frequency analysis

5.5 Design flood hydrograph derivation

5.5.1 Flood event rank comparison

To apply the ARR methodology, reasonable coincidence between the rank of a given flood event in the peak flow and flood volume series is required. For this analysis, the largest 15 peak flows were selected from the agency gauged data at Yarrawonga (1939-2004). The period 1939-2004 was chosen to provide a concurrent period for both the peak flow and flood volumes series. The rank in the 14 day flood volume series for these largest 15 peak flow events was then determined. It was found that for these 15 flood events the annual maximum peak flow and 14 day volume occurred for the same flood event within the year.

Table 5-10, Table 5-11 and Table 5-12 show the comparison of rank for the peak flow and flood volume series at Yarrawonga for 14, 21 and 28 day durations respectively.

Table 5-10 Peak flow – 14 day flood volume rank comparison at Yarrawonga (1938 -2004)

Flood Event	Peak Flow (ML)	14 Day Volume (ML)	Rank In Peak Flow Series	Rank In Flood Volume Series
1975	234,000	1,863,000	1	3
1956	204,000	2,219,000	2	1
1974	196,000	1,503,000	3	6
1970	184,000	1,493,000	4	8
1993	183,000	1,393,000	5	11
1955	181,000	1,968,000	6	2
1958	157,000	1,404,000	7	10
1973	142,000	1,606,000	8	5
1996	141,000	1,617,000	9	4
1952	140,000	1,445,000	10	9
1992	137,000	1,386,000	11	12
1981	127,000	1,498,000	12	7
1964	109,000	1,258,000	13	14
1990	104,000	1,240,000	14	15
1939	101,000	1,321,000	15	13

Table 5-11 Peak flow – 21 day flood volume rank comparison at Yarrawonga (1938 -2004)

Flood event	Peak flow (ML)	21 day volume (ML)	Rank in peak flow series	Rank in flood volume series
1975	234,000	2,398,313	1	3
1956	204,000	3,001,803	2	1
1974	196,000	2,198,328	3	5
1970	183,000	1,931,363	4	8
1993	183,000	1,738,247	5	13
1955	181,000	2,677,292	6	2
1958	157,000	1,764,206	7	12
1973	142,000	2,198,184	8	6
1996	141,000	2,236,660	10	4
1952	141,000	1,866,174	9	11
1992	137,000	1,893,073	11	10
1981	127,000	2,042,580	12	7
1964	109,000	1,682,993	13	15
1990	104,000	1,712,316	14	14
1939	102,000	1,899,178	15	9

Table 5-12 Peak flow – 28 day flood volume rank comparison at Yarrawonga (1938 -2004)

Flood event	Peak flow (ML)	28 day volume (ML)	Rank in peak flow series	Rank in flood volume series
1975	234000	2977958	1	3
1956	204000	3731629	2	1
1974	196000	2821389	3	4
1970	183000	2384476	4	8
1993	183000	2161929	5	13
1955	181000	3312836	6	2
1958	157000	2163181	7	12
1973	142000	2647407	8	6
1996	141000	2677040	10	5
1952	141000	2218041	9	11
1992	137000	2375310	11	9
1981	127000	2493682	12	7
1964	109000	2050586	13	15
1990	104000	2151386	14	14
1939	102000	2346534	15	10

From the above three tables, there is considered a reasonable coincidence of ranks between the peak flow and flood volume events. The following comments are made:

- the same 15 events are reflected in the peak flow and flood volumes series
- the 1975 flood event is the largest event by peak flow and the third largest event by flood volume
- the 1956 flood event is the largest event by flood volume and the second largest event by peak flow
- the 1970 and 1993 events have peak flow rank considerably higher than their flood volume ranks.
- The 1996 and 1973 events have peak flow rank considerably lower than their flood volume ranks.

The coincidence between peak flow and flood volume rank is considered sufficient for the purposes of this analysis and the application of the ARR methodology.

5.5.2 Peak flow – flood volume ratio

For the 15 flood events, listed in Table 5-10, the ratios of the peak flow to volume were determined to assess the shape of the flood hydrograph (i.e. peakiness). The 14 day flood volume was converted to an average daily flow over a 14 day period, and then divided into the peak flow. In addition, the October 1917 daily hydrograph yielded a peak flow and 14-day volume. The 1917 peak flow from the daily hydrograph was 306,000 ML/d and differs from the 390,000 ML/d used in the peak flow frequency in Section 5.3.2, however the peak flow – flood volume ratio was considered useful and warranted in the comparison of historical event hydrographs.

Table 5-13 displays the peak flow – flood volume ratios for the 15 largest events for the agency gauged data period (1938-2004) plus the 1917 event from the SR&WSC working files (SR&WSC-A data set). The events in Table 5-13 are listed in descending order based on peak flow.

The magnitude of the peak flow – volume ratio reflects the peakiness of the flood hydrograph i.e. the higher the ratio,' the peakier the flood hydrograph. Higher peak flow – volume ratios occurred for the 1917, 1975, 1993 and 1974 events with lower ratios occurring for the 1939, 1981 and 1990 events. The mean ratio for the 15 historical events plus 1917 is 1.43 with the median ratio at 1.33.

Similar peak flow to flood volume ratios were determined from the design estimates of the peak flow and flood volume. As the flood volume analysis was conducted using the period 1939 to 2004, the design peak flow estimates from an analysis on this same period were used to determine the peak flow – volume ratios.

Table 5-14 shows the peak flow – flood volume ratios for the design events.

Table 5-13 Historical peak flow – flood volume ratio at Yarrawonga (1938 -2004)

Flood Event	Peak Flow (ML/d)	14 Day Volume (ML)	Average Daily Flow for 14 Day Volume (ML/d)	Peak Flow – Volume Ratio	21 Day Volume (ML)	Average Daily Flow For 21 Day Volume (ML/d)	21 Day Peak Flow – Volume Ratio	28 Day Volume (ML)	Average Daily Flow for 28 Day Volume (ML/d)	28 Day Peak Flow – Volume Ratio
1975	233,761	1,863,000	133,000	1.76	2,398,000	114,000	2.05	2,977,000	106,000	2.20
1956	203,677	2,219,000	159,000	1.28	3,002,000	143,000	1.42	3,731,000	133,000	1.53
1974	195,818	1,504,000	107,000	1.82	2,198,000	105,000	1.87	2,821,000	101,000	1.94
1970	183,687	1,494,000	107,000	1.72	1,931,000	92,000	2.00	2,384,000	85,000	2.16
1993	183,012	1,394,000	100,000	1.84	1,738,000	83,000	2.21	2,162,000	77,000	2.37
1955	181,096	1,968,000	141,000	1.29	2,677,000	127,000	1.42	3,313,000	118,000	1.53
1958	157,090	1,404,000	100,000	1.57	1,764,000	84,000	1.87	2,163,000	77,000	2.03
1973	141,722	1,606,000	115,000	1.24	2,198,000	105,000	1.35	2,647,000	95,000	1.50
1996	141,395	1,617,000	116,000	1.22	2,237,000	107,000	1.33	2,677,000	96,000	1.48
1952	140,556	1,445,000	103,000	1.36	1,866,000	89,000	1.58	2,218,000	79,000	1.77
1992	136,877	1,386,000	99,000	1.38	1,893,000	90,000	1.52	2,375,000	85,000	1.61
1981	126,830	1,499,000	107,000	1.18	2,043,000	97,000	1.30	2,494,000	89,000	1.42
1964	109,350	1,258,000	90,000	1.22	1,683,000	80,000	1.36	2,050,000	73,000	1.49
1990	104,423	1,241,000	89,000	1.18	1,712,000	82,000	1.28	2,151,000	77,000	1.36
1939	101,533	1,322,000	94,000	1.08	1,899,000	90,000	1.12	2,346,000	84,000	1.21

Table 5-14 Design peak flow – flood volume ratio at Yarrawonga (1938 -2004)

Average recurrence interval (years)	Peak flow 1939 - 2004 (GP distribution) (ML/d)	14 day volume (ML)	Average daily flow for 14 day volume (ML/d)	Peak flow – volume ratio	21 day volume (ML)	Average daily flow for 21 day volume (ML/d)	Peak flow – volume ratio	28 day volume (ML)	Average daily flow for 28 day volume (ML/d)	Peak flow – volume ratio
10	152,000	1,514,000	108,000	1.40	2,032,000	97,000	1.57	2,519,000	90,000	1.69
20	190,000	1,831,000	131,000	1.45	2,446,000	116,000	1.63	3,050,000	109,000	1.74
50	236,000	2,192,000	157,000	1.51	2,913,000	139,000	1.70	3,660,000	131,000	1.81
100	269,000	2,427,000	173,000	1.55	3,214,000	153,000	1.76	4,061,000	145,000	1.85
200	300,000	2,633,000	188,000	1.59	3,476,000	166,000	1.81	4,418,000	158,000	1.90

For the design events, the peak flow – volume ratio is seen to increase with ARI. The design ratios generally span between the median and mean historical event ratios. This consistency provides confidence in the design ratios.

Figure 5-6 displays the peak flows – 14 day flood volumes for both historical and design flood events. Also included in Figure 5-6 is the 1917 flood event using the peak flow and 14 day volume based on the SR&WSC-A data set. The design flood events shown are derived from the frequency analysis for" the period 1938 to 2004. The scatter of the historical peak flows – 14 day flood volumes highlights the variation in the relationship of peak flow to flood volume i.e. hydrograph shape.

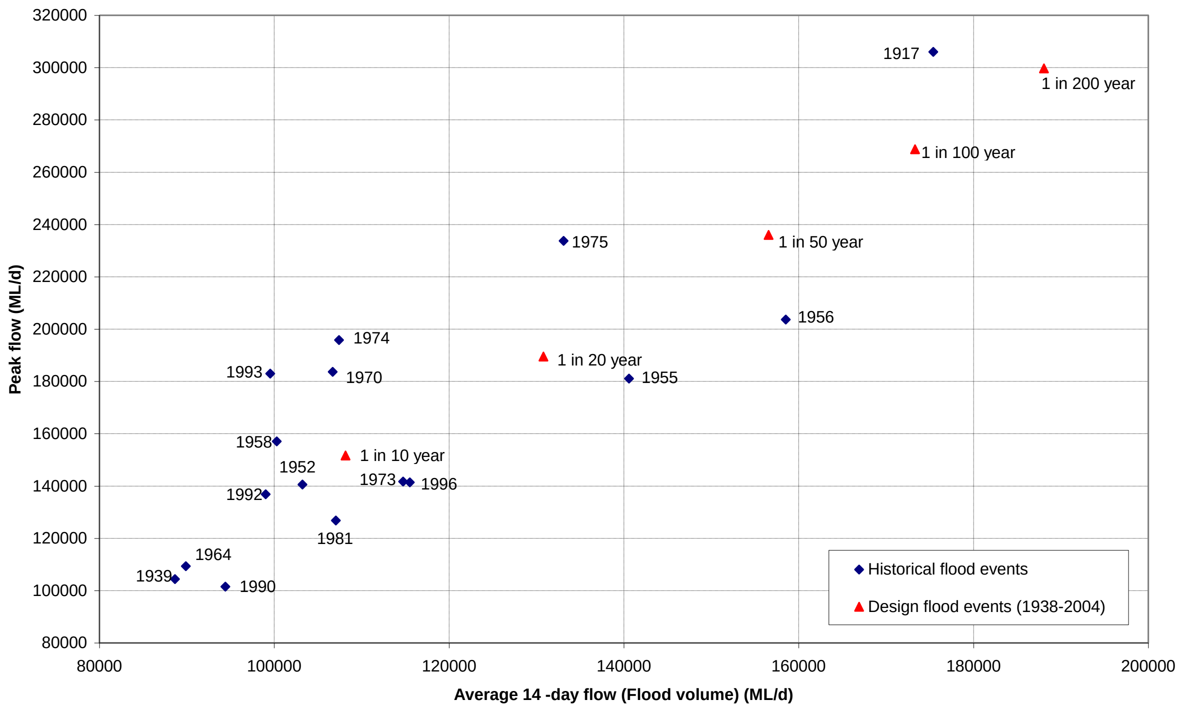

Figure 5-6 Historical and design peak flow – 14 day flood volume ratio at Yarrawonga (1938 -2004)

It should be noted that the peak flow – volume ratios for the two significant historical floods, 1956 and 1975, were 1.28 and 1.76 respectively. These ratios differed considerably from the design event ratios (1.40 – 1.59). The variation in these ratios reflects the nature of these two events. The 1956 event had a long duration (i.e. lower peak flow – volume ratio) and the 1975 event was of shorter duration with a high peak flow (i.e. higher ratio). This variation in peak flow – volume ratio compared to design peak flows, is further highlighted by examination of the approximate ARI for the peak flow and flood volumes as follows:

- 1956 event peak flow ARI ~ 30 year and 14 day volume ARI ~ 50 years
- 1975 event peak flow ARI ~ 50 year and 14 day volume ARI ~ 20 years

It should be noted the above approximate ARIs are based on the frequency analysis for the period 1938 to 2004 and differ from the ARI determined from the frequency analysis of 1905-2004.

The spread of flow-volume ratios reflects the natural variability inflows within Australian rivers and as such is not unexpected.

5.5.3 Historical flood hydrograph selection

Examination of the historical flood ratios in Table 5-13 revealed that the peak flow – volume ratios for the 1952, 1958 and 1992 events, 1.36, 1.57 and 1.38 respectively, were closest to the design event ratios.

The 1992 flood hydrograph contained a prolonged period of relatively constant flow prior to the rise and occurrence of the peak flow. It was considered that the 1992 flood hydrograph shape did not representt what could be considered to be a typical flood hydrograph shape. The 1952 and 1958 historical flood hydrographs were considered to be more representative of typical flood hydrographs.

Listings of the 1952, 1958 and 1992 historical flood hydrographs are provided in Appendix A.

Design flood hydrographs were computed by scaling historical flood hydrographs with similar peak flow – volume ratios to the design event ratios shown in Table 5-14.

As discussed, the 1952 and 1958 flood events had similar flow-volume ratios to the design flood ratios, hence the following historical events were adopted as representative hydrograph shapes for scaling to become design flood hydrographs:

- 1952 flood hydrograph: 10 and 20 year ARI design events
- 1958 flood hydrograph: 50, 100, 200 and 500 year ARI design events

5.5.4 Design flood hydrograph scaling

The representative historical hydrographs were scaled by the ratio of the historical to design peak flows. The design peak flows adopted for scaling were from the frequency analysis using the GP distribution and the period 1905-2004 plus 1870, as listed in Table 5-4. Table 5-15 displays the scaling factor for the historical hydrographs to yield the design flood hydrographs. The representative historical flood event employed for the scaling is provided in brackets.

Table 5-15 Design peak flow – flood volume ratio at Yarrawonga (1938 -2004)

Average recurrence interval (years)	Historical peak flow from representative flood event (ML/d)	Design Peak flow 1905 – 2004 plus 1870 (GP distribution) (ML/d)	Peak flow scaling factor
10	140,500 (1952)	193,000	1.37
20	140,500 (1952)	251,000	1.79
50	157,090 (1958)	328,000	2.09
100	157,090 (1958)	387,000	2.46
200	157,090 (1958)	448,000	2.85
500	157,090 (1958)	528,000	3.36

The use of the design peak flow estimates for the period 1905 to 2004 plus 1870 implies an assumption that the design peak flow – volume ratios determined for the period 1938 to 2004 would be similar to those for the longer period used in the peak flow frequency analysis. The validity of this assumption is unable to be rigorously tested given the absence of suitable data for use in the flood volume frequency analysis prior to 1938.

The 14 day volumes contained in the scaled design flood hydrographs were determined and are provided in Table 5-16.

Table 5-16 Adopted design flood hydrograph 14 day volume at Yarrawonga

Average recurrence interval (years)	Scaled Design hydrograph – 14 day volume (ML)
10	1,913,000
20	2,427,000
50	2,681,000
100	3,092,000
200	3,504,000
500	4,120,000

The above 14 day volumes differ from the 14 day volume estimates based on the frequency analysis outlined in Section 5.4.2. This difference is due to scaling based on the design peak flow estimates from the period 1905-2004 and the gap between the design peak flow and the flow peak of the historical event used in the hydrograph scaling process.

Figure 5-7 displays the 1952 and 1958 historical flood hydrographs, and the adopted scaled design flood hydrographs

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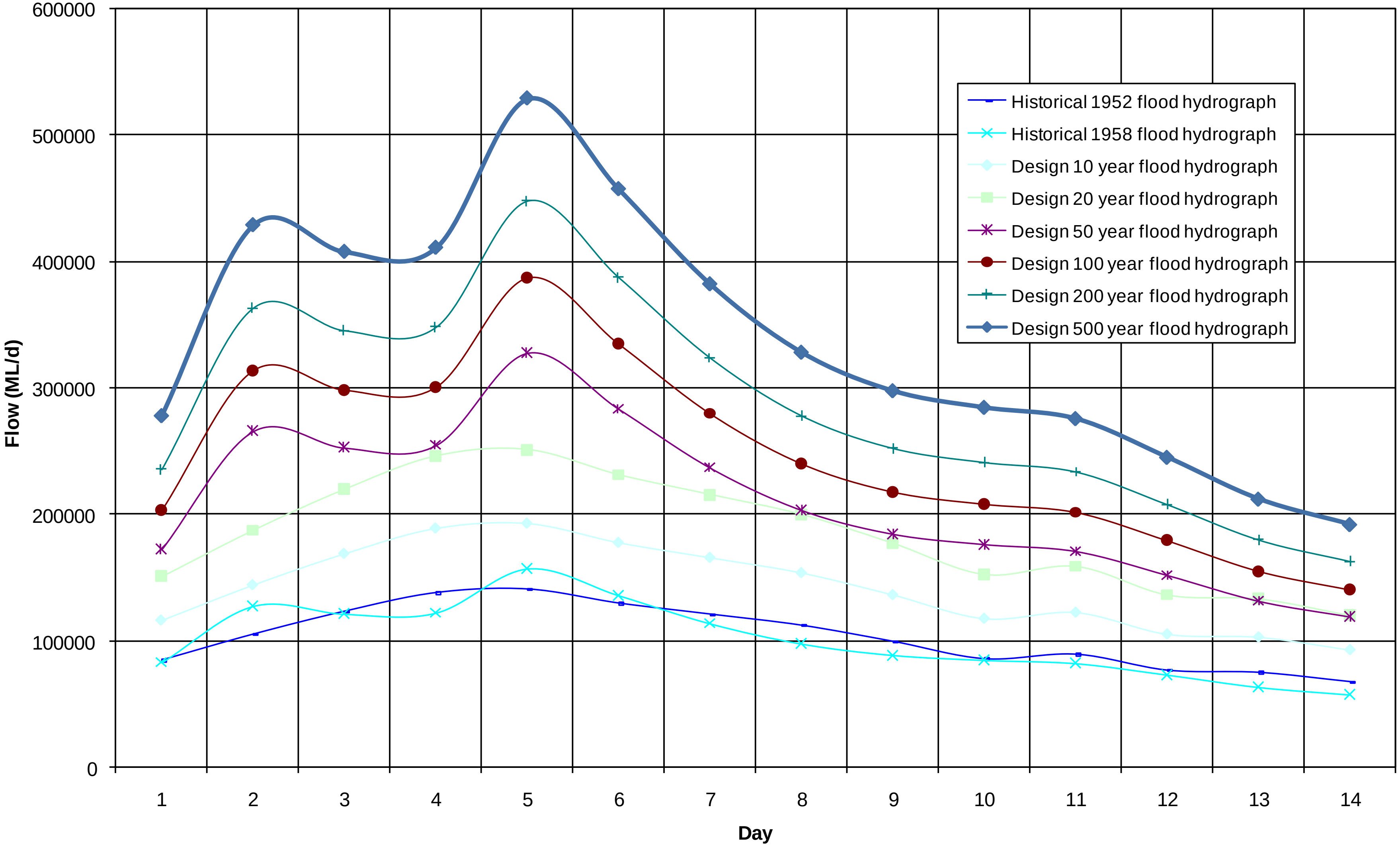

Figure 5-7 Historical and design flood hydrographs for the Murray River at Yarrawonga

5.6 Discussion

The reliability or confidence surrounding the peak design flow estimates at Yarrawonga is influenced by a number of factors which are discussed below.

The peak flow estimates at Yarrawonga are highly dependent on the period of record employed in the frequency analysis. A number of significant flood events occurred in the early gauge period (1905-1937). The reliability of the peak flow data derived from the early gauge record (1905-1937) is difficult to establish.

Another important factor influencing the frequency analysis is the significant water resources development that has occurred in the Upper Murray catchment from 1930's i.e. construction of Hume Dam in 1930's and Dartmouth Dam in the 1970's. It is likely that this development has reduced the magnitude of flooding, particularly for more frequent events, say up to 1 in 20 year. However, the reduction in flood magnitude for larger flood events would be limited. The MRFPM study (RWCV et al 1986) suggests that for the 1917 flood event the impact of Hume Dam would be negligible.

Given these above factors, it is evident that considerable uncertainty surrounds the peak flow estimates at Yarrawonga. This study has applied a different probability distribution (GP) to observed peak flow series than used in previous studies. The GP distribution is considered to better fit the observed data and is appropriate for adoption in this study.

The study considered two periods of streamflow data, 1905 to 2004 and 1938 to 2004. Despite the uncertainty in the reliability of the early period peak flows, the peak flow estimates derived from the longer period are considered more appropriate. Further it is considered that the exclusion of the early period (1905-1937) would result in an unreasonable reduction in peak flow estimates for the 1 in 50 year ARI and greater events.

5.7 Climate change considerations

The study area and contributing upstream catchment has been subject to a drying trend since 1960 with mean annual rainfall decreasing by around 15 -30 mm per decade (CSIRO & BoM 2010).

Climate change modelling suggests that rainfall in winter, spring and autumn will decrease by 20%-50% by 2050, with an increase in summer rainfall of 10-20% from a relatively low current summer rainfall (DECCW 2008).

DECCW (2008) provides the following comments on future flood behaviour under the influence of Climate Change:

"Due to the increase in summer rainfall there is a risk that flood-producing rainfall events are likely to become more frequent and more intense in the wetter La Niña years. Whether these changes lead to an increase in flood levels depends upon the existing catchment conditions and the water levels in the major storages at the time of actual events. The risk of protection measures such as levees being overtopped is likely to increase with an associated risk to life and property. Changes to short and intense rainfall events are likely to increase flooding from smaller urban streams and urban drainage systems." p 3

As noted above, there are a number of factors that may influence the computed design flood magnitudes. It is unclear how climate change may ultimately impact design flood magnitudes into the future.

This study has considered a range of flood magnitudes from 10 year ARI to 500 year ARI. Comparison of the flood behaviour between these events can provide insight into the sensitivity of flood impacts to climate change influences.

It is recommended that as understating of climate change influences on large flood events in the Murray River improves, sensitivity analysis of the changes in design flows is considered.

6 HYDRAULIC ANALYSIS

6.1 Overview

The hydraulic analysis determined flood behaviour for the Murray River floodplain from Dicks/Seppelts levees to downstream of the Ulupna Creek / Murray River confluence. The flood behaviour was assessed for the 10, 20, 50, 100, 200 and 500 year ARI flood events. Design flood hydrographs for the Murray River at Yarrawonga, outlined in Section 5, were utilised as inflows for the hydraulic analysis. The sensitivity of flood behaviour to several levee failure scenarios was assessed.

The extensive nature of the floodplain requires the application of a two-dimensional (2D) hydraulic model. The linked one-dimensional and two-dimensional unsteady hydraulic model, MIKEFLOOD, was the principal tool for the hydraulic analysis.

For this present study, a two-dimensional (2D) MIKE 21 model has been set up to model the overall floodplain flows. A coupled one-dimensional (1D) MIKE 11 model has been utilised to explicitly model waterway (bridge and/or culvert) crossings within the study area.

The MIKEFLOOD model parameters were calibrated through comparison of the modelled and observed flood levels with historical inflow flood hydrographs as an input. Once calibrated, the MIKEFLOOD model was applied to estimate design flood behaviour (levels and extents) with design inflow hydrographs as an input.

This section details the input data, methodology and outputs for the hydraulic analysis. The structure of the section is as follows:

- Hydraulic model development – details the construction of the MIKEFLOOD model structure (Section 6.2)
- Hydraulic model calibration – details the selection of calibration events and calibration of model parameters (Section 6.3)
- Design flood modelling – summaries the estimation of design flood levels and velocities with the calibrated MIKEFLOOD model (Section 6.4)

6.2 Hydraulic model development

6.2.1 Overview

The development of a detailed DTM and subsequent construction of a hydraulic model of the study area enables the flood and hydrodynamic behaviour of the study area to be simulated in detail. Hydrodynamic conditions varying from historical flood events to the simulation of hypothetical "design" flood events can be modelled to investigate the pattern of flooding behaviour within the study area. These conditions can be applied to both the existing floodplain geometry, and geometries that have been altered to represent changes eg. flood mitigation measures, proposed developments or historical floodplain conditions.

6.2.2 Hydraulic Model Software

The hydraulic model of the study area has been undertaken utilising the Danish Hydraulic Institute's (DHI) MIKE FLOOD modelling software. MIKE FLOOD is a state-of-the-art tool for floodplain modelling that has been formed by the dynamic coupling of DHI's well proven MIKE 11 river model and the MIKE 21 fully two-dimensional modelling system. This dynamic coupling extends the capability of MIKE 21 to include the following:

- A comprehensive range of hydraulic structures (including weirs, culverts, bridges, etc.)
- Ability to accurately model sub-grid scale channels
- Ability to accurately model dambreak or levee failures

For the present study, a two-dimensional (2D) MIKE 21 model has been set up to model the overall floodplain flows. A coupled one-dimensional (1D) MIKE 11 model has also been utilised to explicitly model waterway bride and culvert crossing within the study area.

Further information on MIKE FLOOD can be found at:

http://www.dhigroup.com/Software/WaterResources/MIKEFLOOD.aspx

6.2.3 Model Structure

The basis of the two-dimensional hydraulic model is the topographic grid which is based on aerial laser survey, bathymetric data and a significant amount of field survey. Field surveys included embankments (channels and levees), culverts and bridges within the study area. A 30 m grid resolution of the greater study area was utilised for the hydraulic model.

The choice of an appropriate grid resolution for the hydraulic modelling was determined to a large extent by the significant depths (>5 m) encountered in the river resulting in a high wave celerity (~15 m/s) which impacts on the maximum allowable time-step and hence model simulation times. In addition, significant flow velocities (~2 m/s) are also predicted in the river during large flood flows. These velocities dictated the computational time step adopted for the hydraulic model to ensure the maximum model Courant number would provide a stable and accurate model solution. A finer resolution than the adopted 30 m grid would require a correspondingly smaller computational time step, resulting in excessively long simulation times. Given the nature of the study topography it was considered that the chosen grid spacing provides a good resolution of the physical characteristics of the river and floodplain.

Bridge and culvert crossings within the study area were modelled as MIKE 11 structures and dynamically coupled with the two-dimensional model. Head loss through the structures could therefore be modelled explicitly within the model.

As noted, levee and irrigation embankment crest elevations were collected/surveyed and then entered or stamped into the hydraulic model topography.

The variation in hydraulic roughness within the study area was schematised as a grid, representing various land forms or uses, e.g. open grassland, paved surfaces, buildings, thick vegetation etc. The hydraulic roughness grid was based principally on aerial ortho-photography and visual inspection undertaken during field visits. Hydraulic roughness values adopted for the two-dimensional hydraulic model are summarised in Table 6-1. Roughness values were initially adopted based on literature and previous experience with similar flood models. These values were then validated during the hydraulic model calibration process and any necessary adjustments made.

Table 6-1 Hydraulic Roughness Parameters

Topography Class	▪ Manning's "n"
Murray River channel and major anabranches (bed & banks)	0.035
Overbank riparian corridor (within levees)	0.065- 0.07
Vegetated floodplain areas and waterways (e.g. Sheepwash Creek)	0.07
Cleared floodplain (outside levee)	0.04

6.3 Hydraulic model calibration

6.3.1 Approach

The calibration process requires systematically comparing the hydraulic model's representation of flooding in the study area with observed flooding behaviour. This process may incorporate comparisons between gauged stream flows, observed maximum flood levels, areas of inundation as shown in aerial photography and eyewitness recounts of flooding behaviour. Where the model does not adequately represent the observed behaviour, the reason for the discrepancy is identified and inputs into the model are adjusted as required.

The hydraulic model developed for this study is based on current topographic data and flooding behaviour is therefore influenced by the current topography. As such, the ability of the hydraulic model to simulate observed historical flood behaviour is affected by changes to the topography subsequent to the flood event being modelled.

Calibration of the model was primarily based on matching the modelled flood levels with a number of observed flood levels throughout the study area. This was achieved through a combination of fine tuning of the factors describing head loss through the major bridge and culvert structures, and some minor adjustment to the roughness parameters.

Due to uncertainty in the hydraulic efficiency of waterway structures, care was taken not to adjust various model parameters outside acceptable ranges in order to 'force' an acceptable calibration fit. In this respect, it is noted that calibration was readily achieved with standard model parameter values.

The historical events, October 1975 and October 1993, were used as the principal hydraulic model calibration events.

The following section discusses the hydraulic model calibration for the two events.

6.3.2 October 1975

The October 1975 flood event reached a peak flow at Yarrawonga of approximately 234,000 ML/d, estimated to be a 17 year ARI event. Significant levee failures occurred at Brentnalls, Cleaves, and Dixons Bend. The levee failures led to considerable flow across the Victorian floodplain. GBCMA provided some general description of the nature of the levee breaches (timing and extent). It is understood that this information was sourced from local

SR&WSC officers and landholders. The levee failures were included in the hydraulic model as time varying structures based on this information.

The October 1975 event provides a good assessment of the hydraulic model's ability to simulate flood events in excess of the levee capacity.

The Victorian Flood Database (VFD) contained 178 observed flood levels from the October 1975 event. The majority of the observed flood levels were located along the PWD levee between Cobram and the Goulburn Valley Highway Bridge (Tocumwal). The strength of the model calibration was reflected by the breakdown of flood levels differences (modelled flood level - observed flood level), as seen in Figure 6-1.

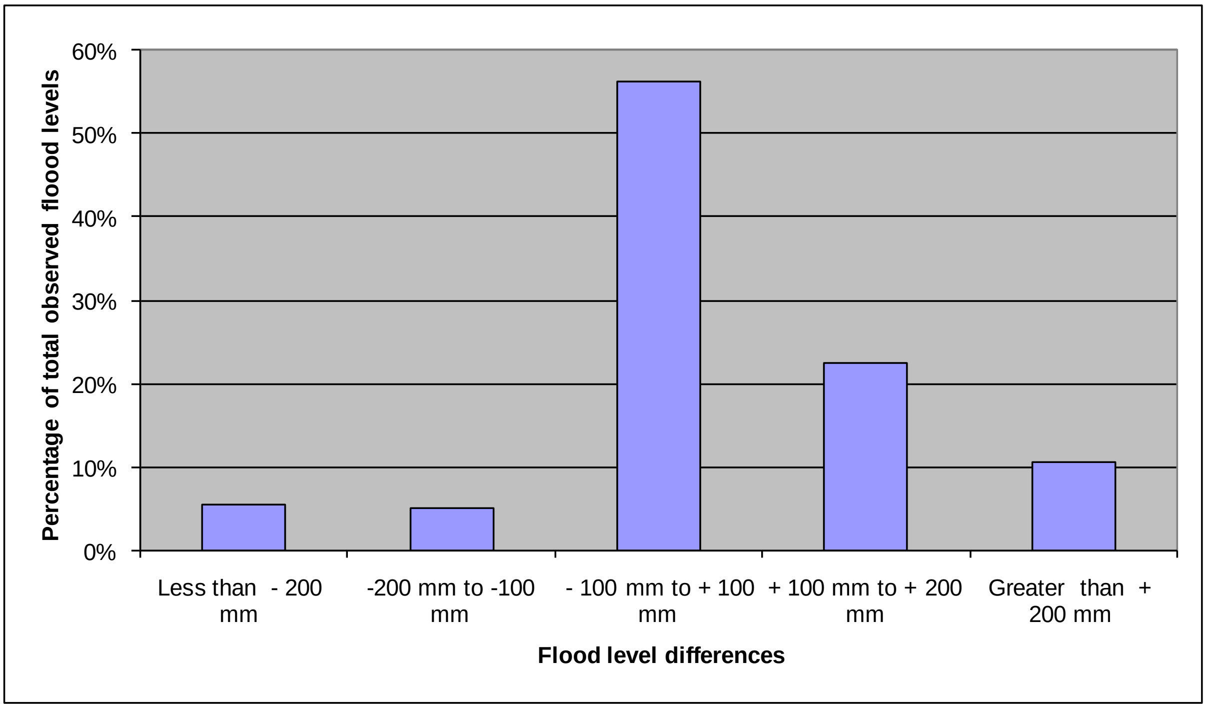

Figure 6-1 October 1975 – Hydraulic model calibration – flood level difference breakdown

This indicates that around half (56%) of the modelled flood levels were within +/- 100 mm, and 84% of modelled flood levels were within +/- 200 mm of the observed flood levels.

Figure 6-2 displays the modelled October 1975 flood extent, and flood level differences. The dot colour reflects the difference between the modelled and observed flood levels. The dark and light green dots indicate observed flood levels are under-estimated, the orange dots indicate the modelled and observed flood levels were within 0.1 m and the red and purple dots indicates observed flood levels were over-estimated.

The following general comments regarding the modelled flood behaviour are provided:

- Upstream of Dixon's Bend: The modelled 1975 flood levels are within +/- 0.1 m.
- Brentnalls to Tocumwal: The modelled 1975 flood levels are generally up to 0.2 m higher than observed levels
- Sheepwash Creek adjacent to Brentnalls: Two modelled 1975 flood levels (light green points) are up to 0.2 m lower than observed levels, with an additional adjacent modelled flood level point low by 0.5 m (dark green)

- Newell Highway to Immediately downstream of Railway: The modelled 1975 flood levels are within 0.15 m
- Adjacent to Ulupna Island: The modelled 1975 flood levels are generally within 0.1 of observed levels.
- Downstream of Ulupna Island: The modelled 1975 flood levels within 0.1 m.

Given the uncertainty in the levee failures during the 1975 flood, it is considered that a good fit with observed levels for the 1975 flood was achieved.

As an independent check the Goulburn Broken CMA plotted historic 1975 flood level contours (obtained from the Victorian Flood Database (VFD)) against the modelled contours at 0.2 m intervals. These flood contours, including observed spot 1975 flood heights, can be found in six sheets and are presented in Appendix C. The 1975 historic flood level contours developed for the Murray Darling Basin Commission (MDBC) in the mid-1990s took into account some of the anomalies in the observed 1975 flood heights. This provides another perspective for the calibration. The following comments were made by Guy Tierney of GBCMA.

> Upstream of Cobram observed 1975 flood levels are limited. The fit is very good until 2.5 km upstream of Cobram. No flood levels are available and comparisons with the fitted historic flood contours show the modelling to be 0.2 to 0.3 metres higher. The modelling does indicate head loss over the Yarrawonga-Cobram Road (also known as Barooga-Cobram Road) and its causeway which is not picked up in the historic flood contours. Therefore, the modelling results are considered good, and represent "today's" conditions. This has implication to Cobram Town levee freeboard.
>
> Downstream of the Yarrawonga-Cobram Road comparisons become very good all the way to Torgannah Road, Koonoomoo.
>
> Between Torgannah Road to the Goulburn Valley Highway the modelling is some 0.1 higher. This can be explained from the Victorian levee breach assumptions. Also the modelling closer to the Goulburn Valley highway and its causeway and Tocumwal levee shows complex flooding patterns. The modelling of "today's" conditions is considered good.
>
> Sheepwash Creek modelling is underestimating by 0.2 to 0.3 m. Further downstream, in the Ulupna floodplain the modelling results are mostly good.
>
> Downstream of the Goulburn Valley Highway, the modelling tends to under estimate flood heights along the river floodplain by 0.1-0.2 m. Closer to the downstream end of the model flood heights are under estimated by 0.3 m.
>
> Overall the calibration represents a good fit of the 1975 observed data.

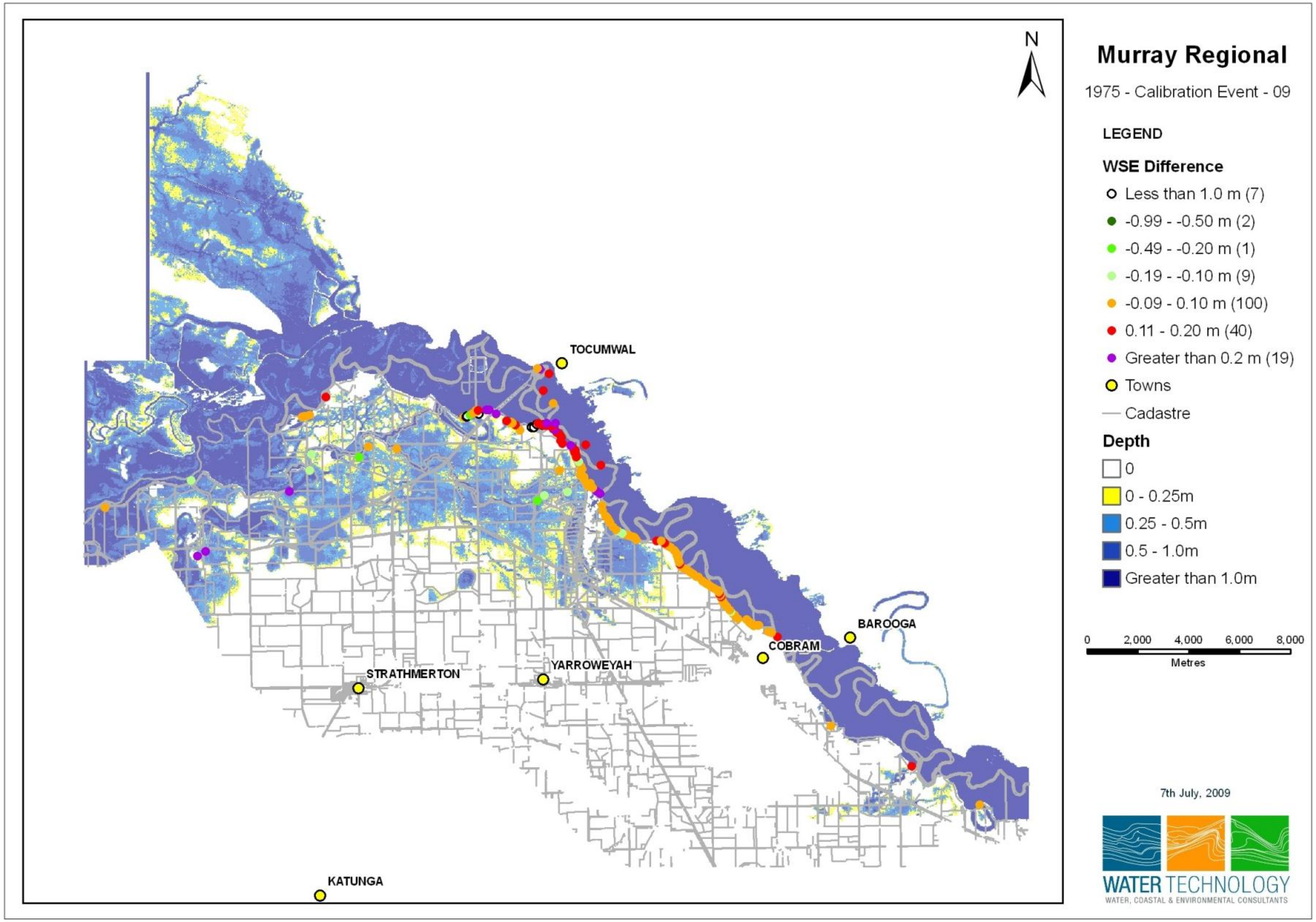

Figure 6-2 October 1975 – Hydraulic model calibration – Flood level comparison

6.3.3 October 1993 calibration

The October 1993 flood event reached a peak flow at Yarrawonga of approximately 183,000 ML/d, estimated to be a 9 year ARI event, based on the adopted flood frequency. The October 1993 event was contained between the Cobram/PWD levee on the Victorian side, and the Barooga/Tocumwal levee on the New South Wales side. No overflow occurred at Dick's levee. As such, the October 1993 event provides a good assessment of the hydraulic model's ability to simulate flood events within the levees.

The Victorian Flood Database (VFD) contained 15 observed flood levels from the October 1993 flood. The strength of the model calibration was reflected by the breakdown of flood levels differences (observed flood level -modelled flood level). A total of 3 from 15 points were within 100 mm, and 7 of the 15 points were within 200 mm.

Figure 6-3 displays the maximum modelled October 1993 flood extent and comparison of the modelled versus observed maximum flood levels. The dot colour reflects the differences between the modelled and observed flood levels. The dark and light green dots indicate observed flood levels are under-estimated, the orange dots indicate the modelled and observed flood levels were within 0.1 m and the red and purple dots indicate observed flood levels were over-estimated. Unfortunately 1993 observed flood levels are only available near Dixon's bend and downstream. .

The comparison of modelled and observed flood levels was considered good for the October 1993 event.

6.3.4 Limited Verification to 1917

There have been significant changes to the floodplain since 1917, and comparisons should be treated with caution.

Comparisons (by the Goulburn Broken CMA) have been carried out with respect to declared 100 year ARI flood level contours (based on 1917 flood levels), and the modelled 100 year ARI flood contours determined in Section 6 under two conditions, with levee breaches and without. The following observations are made by GBCMA:

- Cobram and upstream areas are good, generally within 0.1 m.
- Downstream of Cobram the modelling is generally 0.3 to 0.4 lower than declared with the Victoria Levee breached and generally significantly more agreement without breaches within 0.05 to 0.2 m.
- Generally a good match to observed 1917 flood levels at Dixons Bend, Sheepwash Creek with the levee breach model.

Comparisons have been made against the declared 100 year ARI flood contours that were manually interpreted and drawn. The shape of the manual contours can be significantly different compared to the model generated contours. The manually drawn contours are subject to a significant degree of personal judgement and represent a best estimate. The hydraulic model generated contours are based on a continuous two-dimensional water surface that in most cases is likely to be more realistic (taking into account momentum effects around bends for example).

Overall the verification against the 1917 flood information was considered to be good.

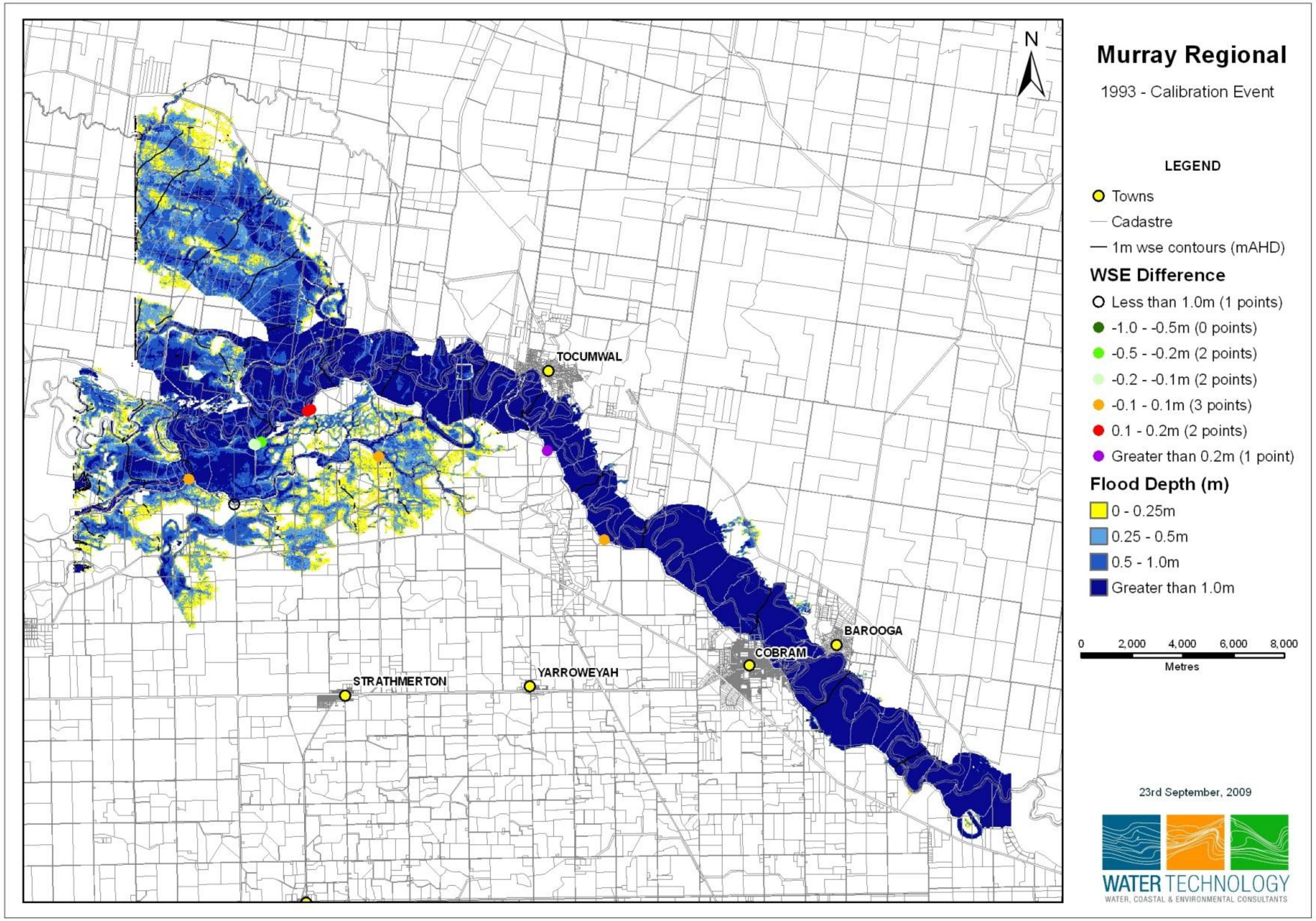

Figure 6-3 October 1993 – Hydraulic model calibration – Flood level comparison

6.4 Design flood behaviour assessment

6.4.1 Overview

Design flood levels and inundation extents were determined using the calibrated MIKEFLOOD model for the 10, 20, 50, 100, 200 and 500 year ARI floods. The design hydrographs for the Murray River at Yarrawonga, determined by the hydrologic analysis, were used as inflows at the upstream model boundary.

As discussed in Section 2, during significant flood events, the levees throughout the study area are subject to possible failure or breaching. Levee breaching can have significant effects on flood behaviour. In order quantify the potential variation in flood behaviour due to levee breaching, the following three levee failure scenarios were considered:

- Levee overtopping without failure (No levee failure)
- Victorian levee failure
- New South Wales levee failure
- Victorian irrigation channel removal

The following sections outline the assumptions for each of the above scenarios and discuss the key differences in flood behaviour between the modelled scenarios. A detailed discussion of the flood behaviour is provided in Section 7.2.

To assess the sensitivity of flood behaviour to flood volume, the hydraulic model was run with a 28 day 100 year flood hydrograph as an inflow at Yarrawonga (the standard simulations used a 14 day hydrograph).

6.4.2 No levee failure

This scenario assumes that no levees are breached. However, the levees can overtop when the adjacent flood level exceeds the levee crest height. Under these assumptions, flood levels, within the levees along the river, will tend towards an upper limit (i.e. this scenario produced the highest flood levels along the river, inside the levees). This scenario should be employed when assessing the available freeboard for levees designed to provide 100 year ARI protection.

Design flood maps for the 10, 20, 50, 100, 200 and 500 year ARI events, under the "No levee" scenario are provided in the accompanying map atlas. Design flood maps for the "no levee" scenario are provided at the following two scales:

- Regional: 1:25,000 – 6 sheets across the study area
- Township: 1:10,000 – Single sheet: Cobram-Barooga, Yarroweyah, Koonoomoo, Tocumwal & Strathmerton.

6.4.3 Victorian levee failure

Historical levee failures have occurred along the PWD levee at Dixon's Bend, Brentnalls and Cleaves. Further, levee failures have occurred at numerous locations around Ulupna Island.

For this scenario, concurrent failures of the PWD levee were considered at the following locations:

- Dixon's Bend and upstream some 1000 m
- Brentnalls
- Ulupna Creek southern bank – four locations

This scenario reflects the upper limit of flood level and extent across the Victorian floodplain. The levee failures were assumed to occur for the 20, 50, 100, and 200 year ARI events.

Design flood maps for the 20, 50, 100, and 200 year ARI events, under the "Victorian levee failure" scenario are provided in the accompanying map atlas. These maps are provided at 1:25,000 scale.

6.4.4 New South Wales levee failure

The New South Wales levees are generally constructed to provide additional protection above the protection level achieved by the PWD levee. No historical evidence of significant levee failures has come to light during the course of this study.

However, to reflect the upper limit of flood level and extent across the New South Wales floodplain, concurrent failures were considered at the following locations:

- Seppelts levee
- Barooga levee

The levee failures were assumed to occur for the 100 year ARI event.

Design flood maps for the 100 year ARI event under the "New South Wales levee failure" scenario are provided in the accompanying map atlas. These maps are provided at 1:25,000 scale.

Figure 6-4 displays a comparison of 100 year ARI flood extents for the three scenarios described above (No levee failure, Victorian levee failure, .New South Wales levee failure).

6.4.5 Victorian irrigation channel removal

Irrigation infrastructure was constructed across the Victorian floodplain during the 1930's. The infrastructure consists of a network of earthen channels and drains. Typically, spoil from the channel construction was placed either side of the channels/drains, forming raised banks.

To assess the influence of the irrigation infrastructure on flood behaviour, the hydraulic model topography was revised to remove the following channels/drains:

- Main Channel No. 1 and No. 2
- Branch channel No. 6/1 and No. 3/7/2

Figure 6-5 displays a comparison of 100 year ARI flood extents with and without the Victorian irrigation infrastructure.

Generally, there were modest increases in flood extents with the selective channel removal conditions. However, there is a reduction from the existing conditions to the south-east of Cobram, adjacent to the Murray Valley Highway. The removal of the Main Channel No. 1

allowed additional flow to the west in this area. In turn, this reduced the flooding along the Murray Valley Highway towards the south of Cobram.

Relatively limited differences in flood extents reflect the considerable overtopping of levees/channels under the existing conditions in the 100 year ARI event. It is considered likely that for events less than the 100 year ARI event, the influence of irrigation channels on flood behaviour would be greater.

6.4.6 Flood hydrograph volume sensitivity

The design flood hydrographs, as discussed in Section 5.5, were based on a 14 day flood volume. A 28 day 100 year ARI design flood hydrograph was evaluated, and applied as inflow to the hydraulic model. The flood behaviour for the 28 day flood hydrograph was assessed for the "No levee failure" and the "Victorian levee failure" scenarios.

Figure 6-6 displays the flood level difference for the No levee failure scenario between the 14 and 28 day flood hydrographs. A positive difference indicates an increase in flood level for the 28 day event compared to the 14 day event. Upstream of Tocumwal, the differences were generally within +/- 10 mm, suggesting that extending design flood duration beyond 14 days has little impact on peak flood levels.

6.4.7 Discussion

The three scenarios considered reflect a range of potential flood extents across the Victorian and New South Wales floodplains, and along the Murray River corridor. Figure 6-4 compares the 100 year ARI flood extents from the three scenarios considered.

As expected, the Victorian levee failure scenario yields the larger flood extent across the Victorian floodplain, and likewise the New South Wales levee failure scenario yields the larger flood extent across the New South Wales floodplain.

Typically, the Victorian levee failure scenario produced increases in flood depths by up to 50 mm across the Victorian floodplain (compared to the "no levee failure" scenario). These limited increases in flood depth are due to the significant overtopping of the rural (PWD) levees that occur in the "no levee failure" scenario for the 100 year ARI event.

The New South Wales levee failure yielded a considerable increase in flood extent from the no failure scenario. Considerable flooding through Tocumwal would occur under the New South Wales failure scenario with flood depths up to 1 m. These considerable changes in flood behaviour reflect the absence of overtopping of the New South Wales levees except for Seppelts levee.

It should be noted that at several locations, in particular along the southern edge of the Victorian floodplain, the modelled flood extent was limited to the available topographic data. In these locations flooding is likely to extend beyond the limit of the hydraulic model. Flood inundation maps display a "limit of mapping" annotation where this is the case. ***It is recommended, as additional topographic data for the Victorian floodplain becomes available, consideration is given to the extension of the study area and re-modelling/mapping of these areas.***

Figure 6-7 displays 100 year ARI flood level contours at 1 m intervals representing the upper bound results of the above scenarios. The Map Atlas displays flood contours at 200 mm intervals.

The comparison of 14 and 28 day flood hydrographs revealed minor changes in flood levels, generally within +/- 10 mm. Given these minor differences, the design flood mapping based on the 14 day is considered acceptable for the purposes of this study.

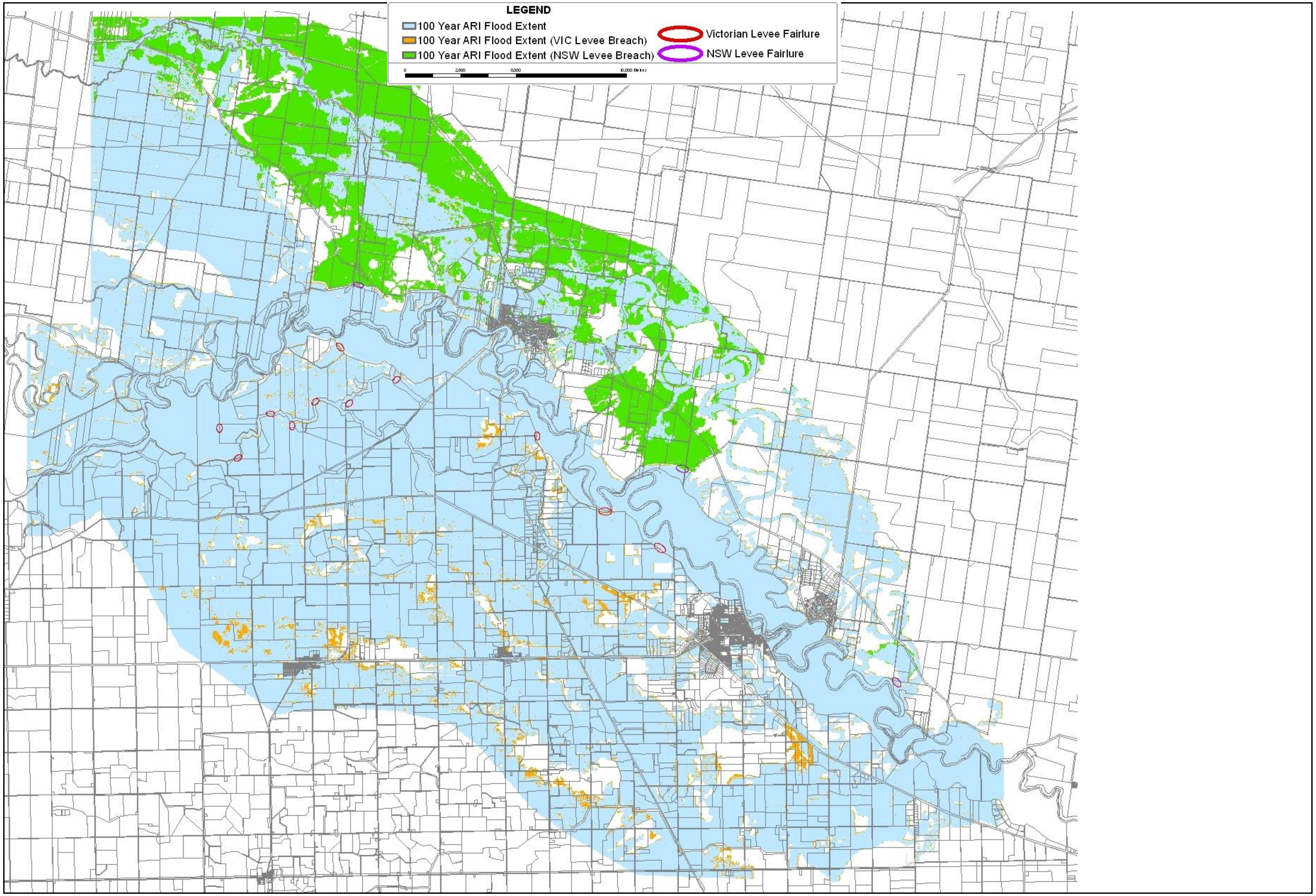

Figure 6-4 Design 100 year flood map – Flood extent comparison for levee failure scenarios

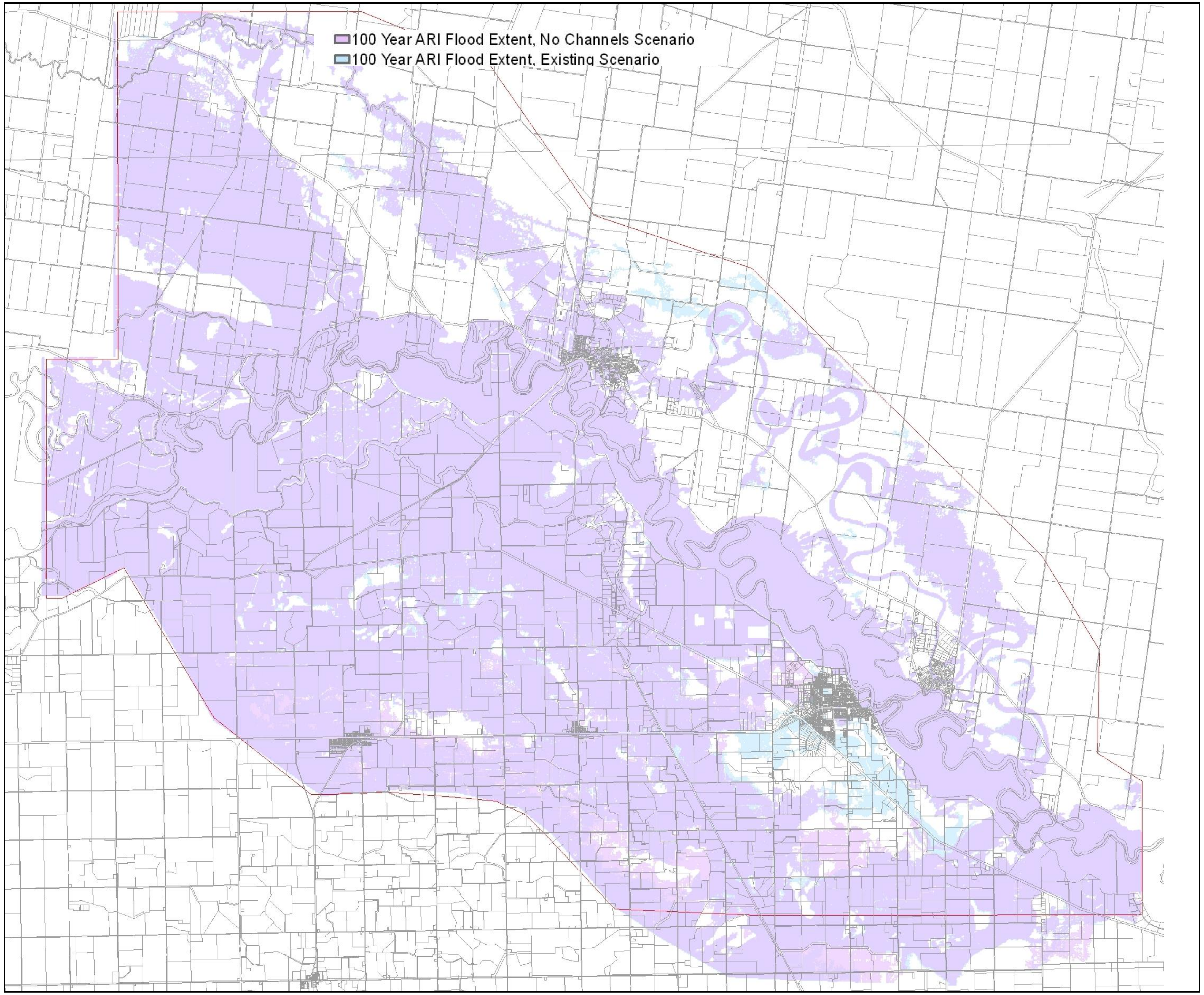

Figure 6-5 Design 100 year flood map – Flood extent comparison for Victorian irrigation infrastructure removal

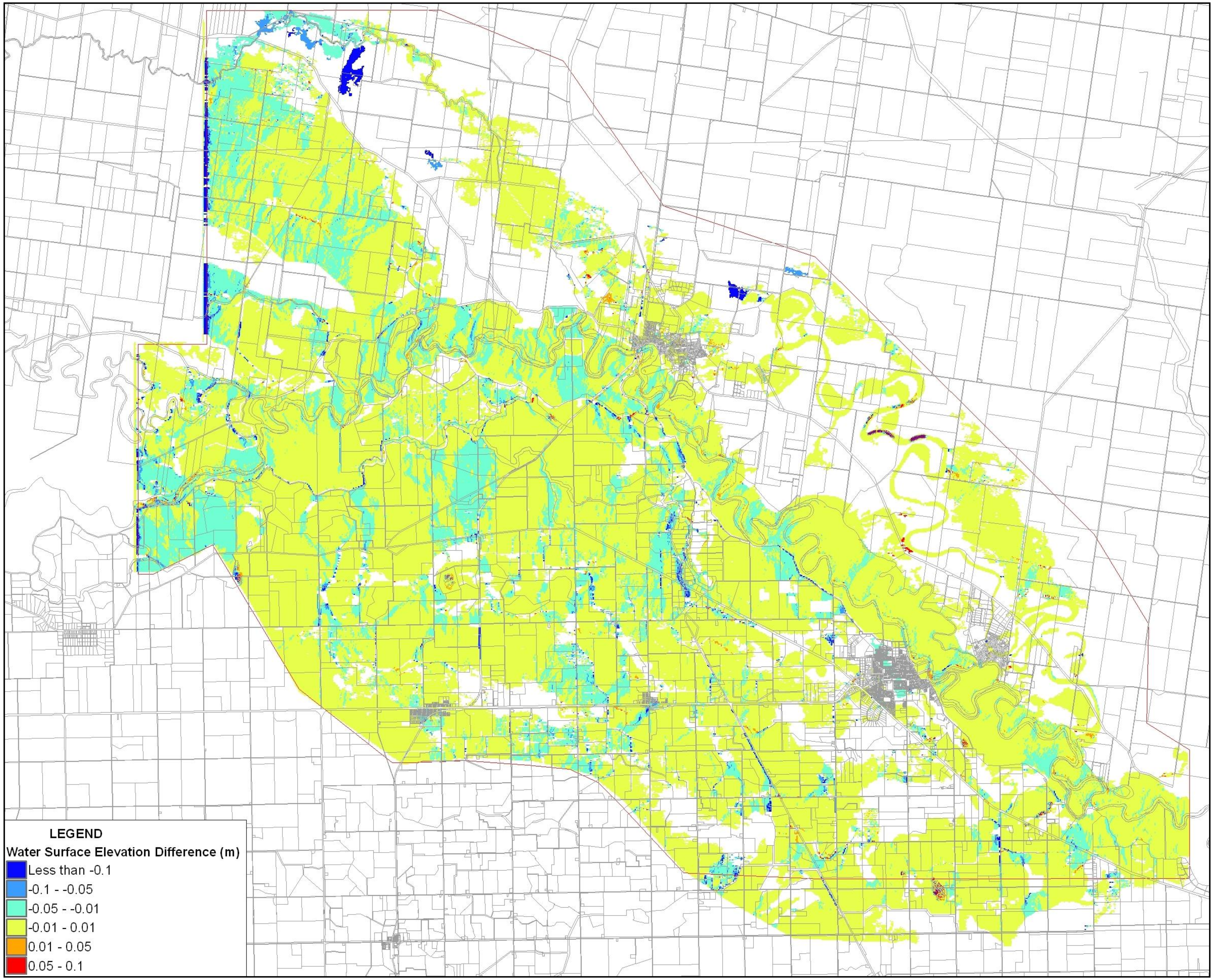

Figure 6-6 Design 100 year flood map – Flood extent comparison for 28 day flood hydrographs – no levee failure

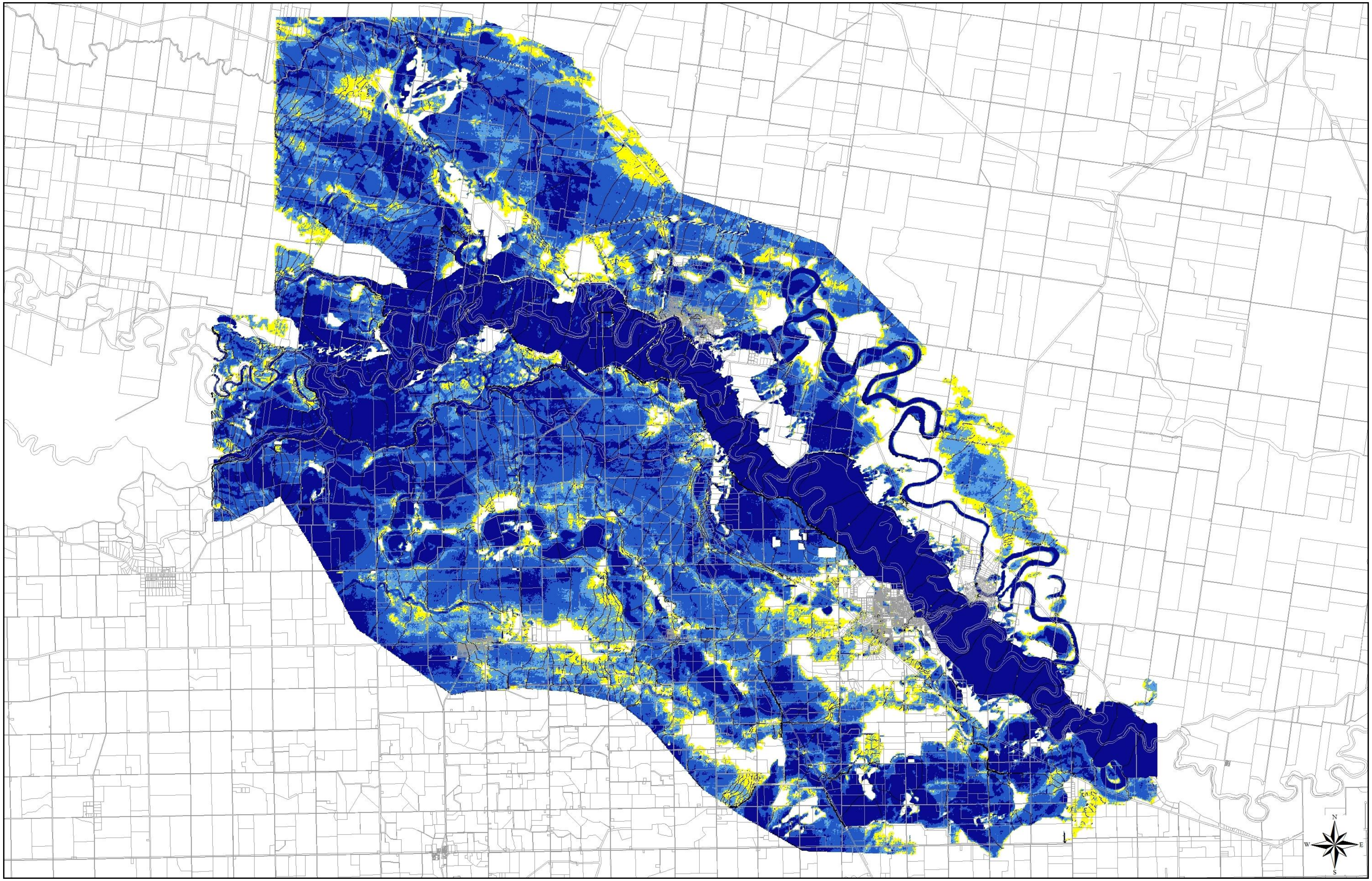

Figure 6-7 Design 100 year flood map – 100 year level contours (Maximum envelope)

6.5 Theoretical Rating Curves at Tocumwal Gauge

From the hydraulic analysis, a modelled rating curve at Tocumwal was derived, as shown in Figure 6-8 and Table 6-2. For the derivation of the modelled rating curve, the location of the Tocumwal gauge was taken as immediately upstream (~ 25 m) of the Tocumwal bridge on the Victorian bank.

The derived rating assumed no levee failure, just overtopping. If levee failure occurred, there would be additional flow across the floodplain for a given gauge height at the Tocumwal gauge. The derived modelled rating curve tends towards an upper limit of stage for a given flow.

The current rating curve, sourced from the Victorian Water Data Warehouse, is also shown on Figure 6-8 for comparison.

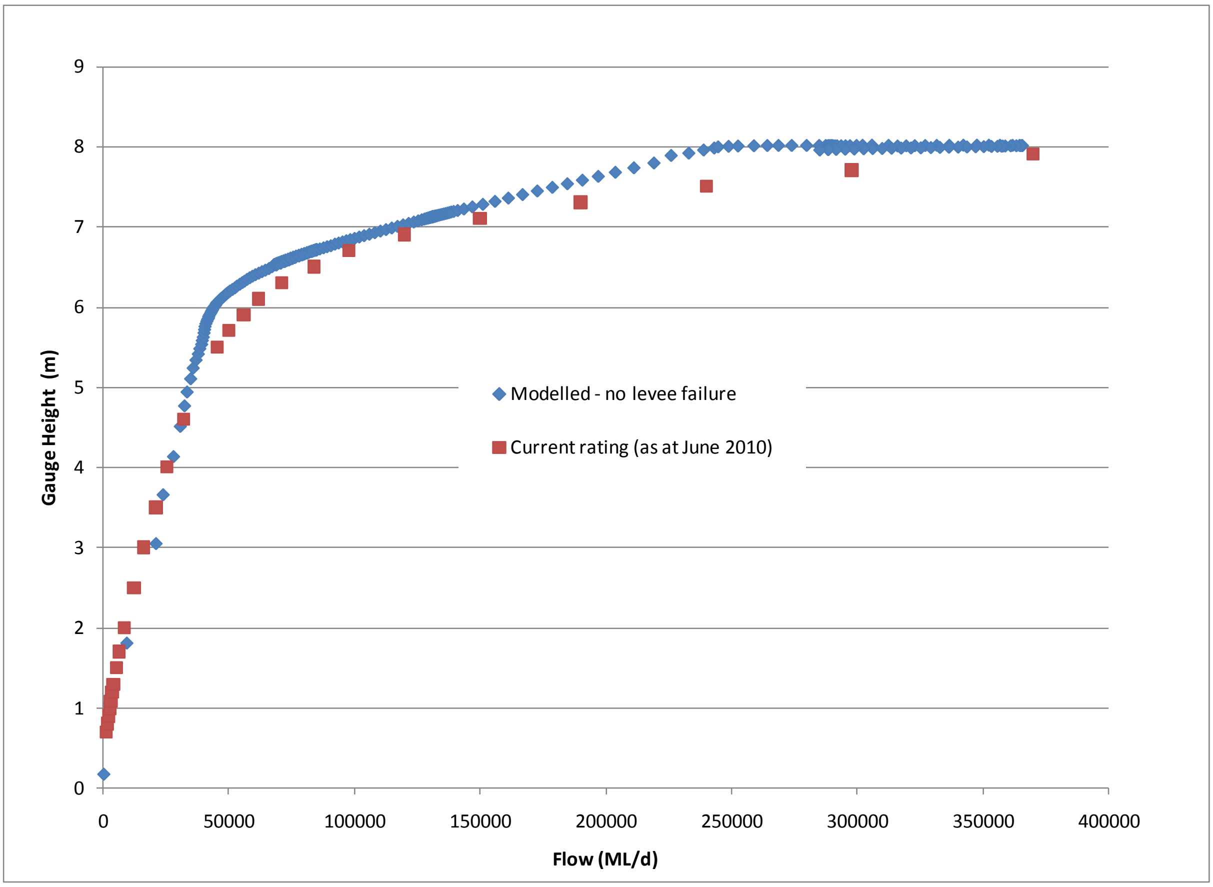

Figure 6-8 Murray River at Tocumwal – Modelled rating curve

Table 6-2 Murray River at Tocumwal – Modelled rating curve

Gauge Height (m)	Flow (ML/d)
7.0	116,239
7.1	127,954
7.2	140,349
7.3	153,406
7.4	166,366
7.5	179,540
7.6	192,871
7.7	206,219
7.8	219,367
7.9	228,013
8.0	347,405

The modelled rating curve shows little increase in gauge height as the flow increases beyond about 240,000 ML/d. This reflects the overtopping of the upstream PWD levee and the additional flow across the Victorian floodplain. This behaviour highlights the relative insensitivity of flood levels along the river downstream of Cleaves as the PWD levee overtopping occurs.

7 STRUCTURAL MITIGATION MEASURES ASSESSMENT

7.1 Overview

This section discusses the flood behaviour with a focus on the performance of the existing structural mitigation measures, identifies potential augmentation to the existing measures and potential new mitigation measures.

As discussed in Section 2, a number of structural mitigation measures, mainly levees, have been constructed within the study area. Also, particularly on the Victorian floodplain, irrigation infrastructure plays a role in flood protection. The key existing mitigation measures include the following:

- Victoria
    - Cobram Town scheme
    - Public Works Department (PWD) levees
- New South Wales
    - Seppelts Levee
    - Barooga Levee
    - Tocumwal Town levee

The level of flood protection afforded by a levee/embankment is defined by the Flood Planning Level (FPL). The FPL is the design flood level plus an allowance for freeboard. The design flood level applied depends of the nature of the built assets to be protected. Generally, for the protection of urban areas the 100 year ARI event is adopted. The freeboard allowance is afforded to ensure this level of protection is achieved over the life of the structure. For flood protection of urban areas, a 600 mm allowance is generally adopted. However, this freeboard depends of the structural nature of the levee.

For the urban area of Tocumwal and Cobram, GBCMA advised that the FPL was adopted as the 100 year ARI flood level plus 0.6 m freeboard allowance. No formal FPL has been adopted for the protection of rural areas (PWD levee). However, GBCMA advised a 300 mm freeboard is generally applied in rural areas.

The following sections assess the performance of the above existing mitigation measures against the adopted FPL.

7.2 Existing structural mitigation schemes

The level of flood protection offered by the existing mitigation scheme levees was assessed against the 100 year ARI flood levels (from the no levee failure scenario). The performance was graded using the following criteria:

- Flood level more than 600 mm below levee crest (this is the performance measure used for Cobram and Tocumwal Town levees)
- Flood level more than 300 mm below levee crest (this is the performance measure used for non-town areas)
- Flood level less than 300 mm below levee crest but no overtopping

- Flood level less than 300 mm above levee crest (Overtopping)
- Flood level more than 300 mm above levee crest (Overtopping)

Figure 7-1 displays the existing flood protection levels.

The above criteria were provided to identify possible low points in the levee crest heights, and to guide further geotechnical/structural investigations.

Appendix D contains longitudinal profiles of the levee crest and design flood levels. These plots show the indicative freeboard/overtopping for the range of design flood events assessed.

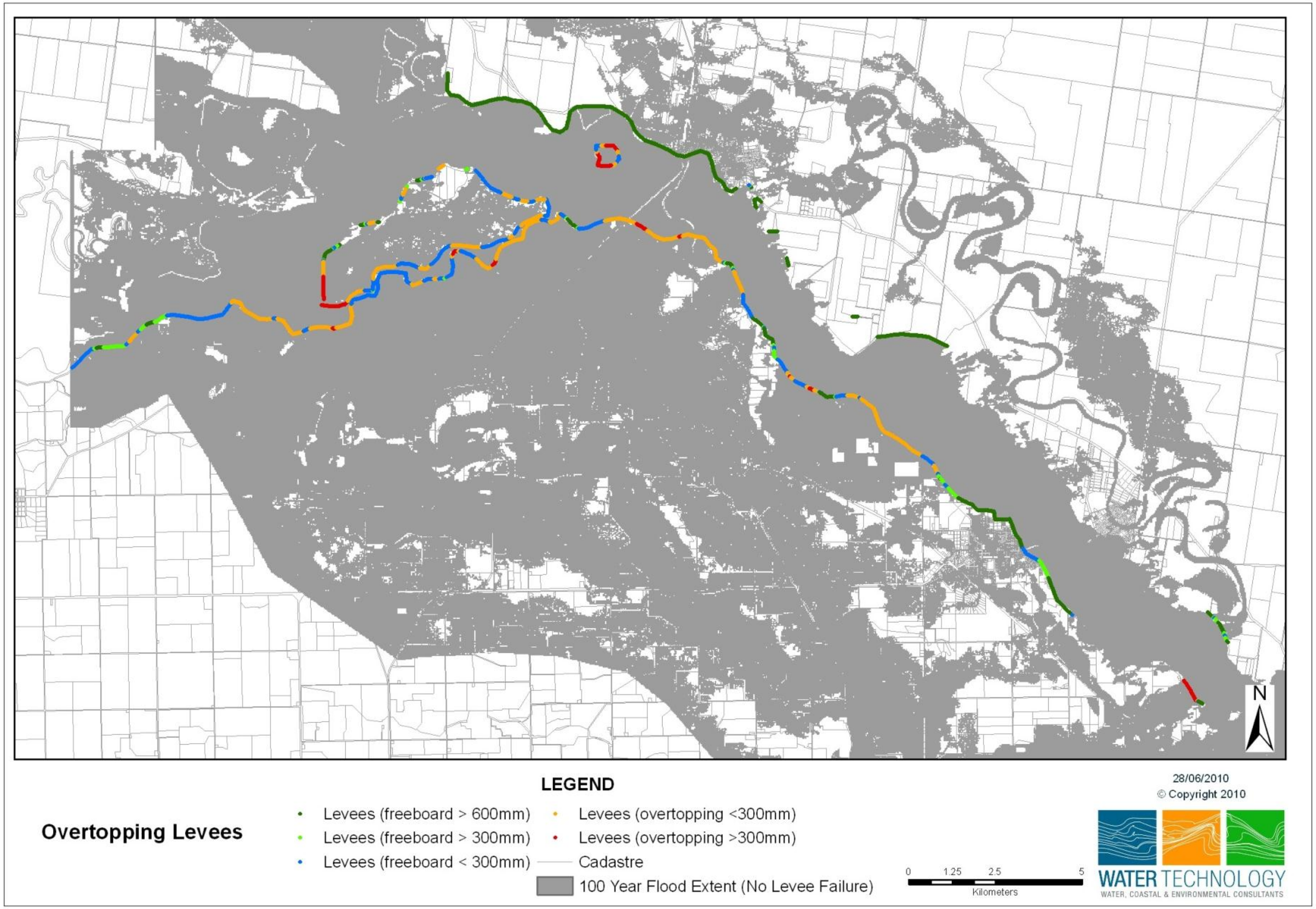

Figure 7-1 Existing flood protection

7.2.1 Cobram Town Scheme

The Cobram Town Scheme was designed and constructed following a number of investigations in the 1980's and 1990's which led to a document known as the Approved Water Management Scheme under the *Water Act, 1989*. Key elements of the scheme include:

- Dick's spillway: This spillway allows flooding to breakout to a natural lower lying floodplain. The breakout reduces the flow in the river and in turn the flood levels along the river through Cobram. The spillway has been reinforced to withstand overtopping, with a non-erodible top layer.
- Levee adjacent to Wyatt Road (Cavagna's levee): A short section (some 300 m) of earthen levee prevents breakout towards Pullar Road.
- River Road: Along River Road, elevated allotments act as a levee to the south of Scenic Drive. A concrete wall (Densons levee), landscaped as the front property fence, provides flood protection between Scenic Drive and Barooga Road. Temporary flood barriers are required across Barooga Road.
- Town levee: This levee extends from Barooga Road to near Harris Road. Adjoins the PWD levee.
- The design standard (FPL) adopted in the Water Management Scheme for Cobram was the 100 year ARI flood event with a standard freeboard of 600 mm for earthen levees. Levees built to this standard provide a Nominal Flood Protection Level for floods up to the 100 year ARI event.

Figure 7-2 shows flood behaviour and freeboard/overtopping for the 100 year ARI flood event along the Cobram Town levee and Dicks Spillway.

Overflow at Dick's spillway commences for flows greater than 20 year ARI (251,000 ML/d) at Yarrawonga. For the 100 year ARI flood event, the depth of overtopping is up to 450 mm.

Along River Road, the elevated allotments are greater than 600 mm above the 100 year ARI flood level. For Densons Levee (concrete wall), the freeboard is greater than 300 mm. Temporary barriers (gates) across the allotment access are required. ***It is recommended that the arrangements for the placement of the temporary barriers are documented in the Flood Emergency Plan for Moira Shire.***

At Barooga Road, the placement of temporary barriers is required as part of the scheme to achieve the appropriate freeboard. ***It is recommended that the arrangements for the placement of the temporary barriers are documented in the Flood Emergency Plan for Moira Shire.***

The Town levee, downstream of Barooga Road, has a freeboard generally above the 100 year ARI flood levels of greater than 600 mm. However, the freeboard is reduced to about 250 mm for the segment from Harris Road to approximately 500 m upstream of Harris Road, adjacent to the treatment plant.

The design practice for earthen levees in Victoria is to provide a 600 mm freeboard above the design flood. In this case, the design flood for the Cobram town is the 100 year ARI flood event. Consequently the Town levee does not provide protection up to the nominal flood protection level. Comparing the current levee crest to the design flood levels reveals a nominal flood protection level of 20 year event (i.e. 600 mm freeboard is available for the 20 year event).

It is recommended that the GBCMA and Moira Shire review this levee segment to assess the degree of the compromise to the levee integrity. This matter requires closer attention to determine consequences of reduced freeboard and the type of levees involved.

The 100 year flood mapping indicates flow paths along the Murray Valley Highway towards the southern limits of Cobram. Discussions with GBCMA (Guy Tierney pers. comms) confirm there are community concerns about the potential for flooding along the Murray Valley Highway. The 100 year ARI flood mapping shows inundation along two flow paths through Cobram. A flowpath along the Murray Valley Highway affects residential properties in the vicinity of William, High, Sydney Station, Murray and Punt Streets. A further flow path crosses Campbell Road adjacent to Dudley Park Lane, and then continues north along Acadia Street, Gregory Street, Thomson Street, through Cobram Secondary collage grounds, across Karook Street, and affects Gorton Street, Nicolina Street, Irene Street and Grasso Drive.

Section 7.3 discusses potential mitigation measures to protect against flows along the Murray Valley Highway.

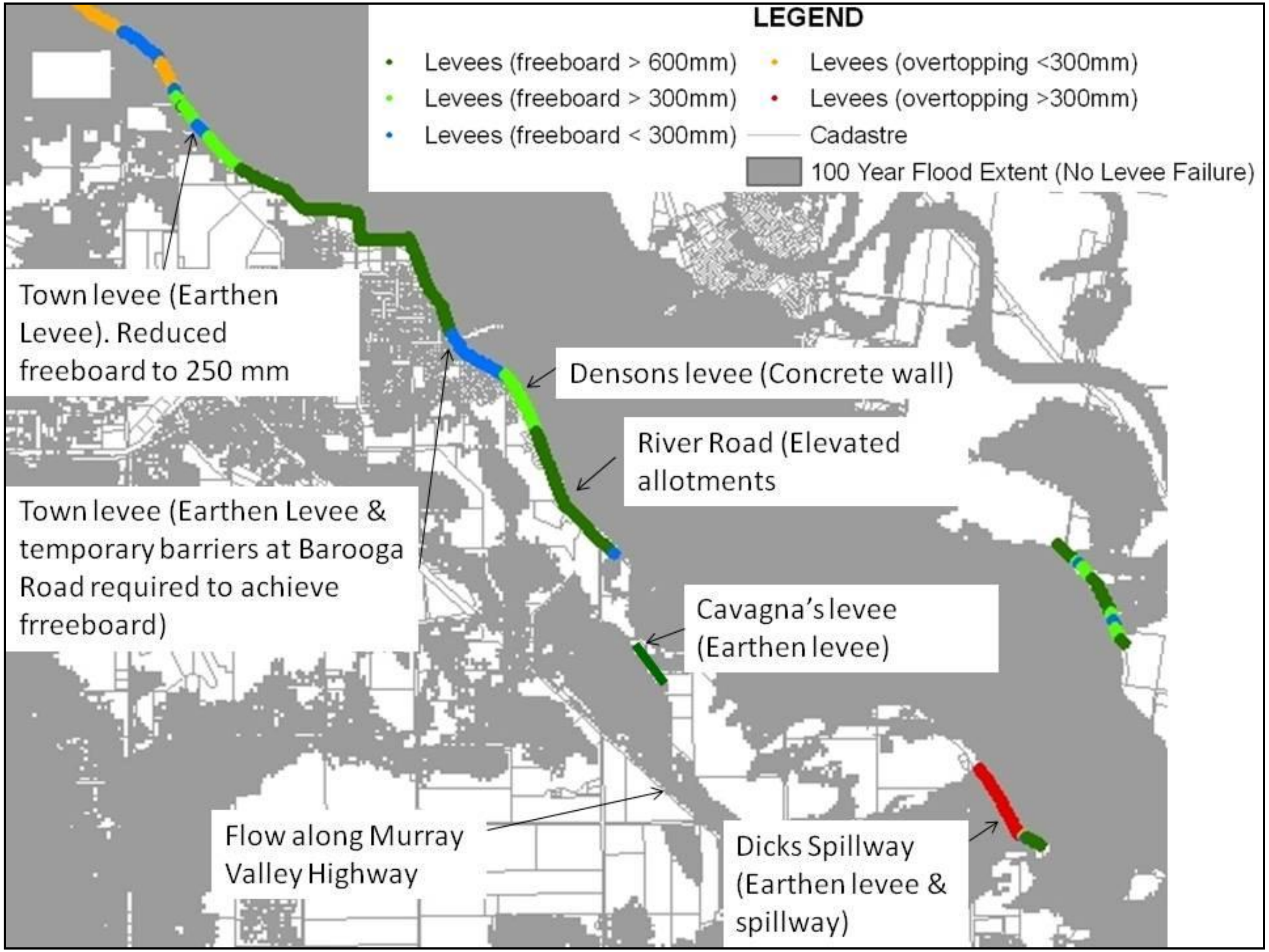

Figure 7-2 Cobram town scheme – levee performance

7.2.2 PWD levee

The PWD (Public Works Department) levee extends from Cobram to Piree Creek in Yielima located some 50 km downstream. The freeboard above the 100 year ARI flood level varies considerably, with some sections having a freeboard greater than 300 mm, while other sections are overtopped by greater than 300 mm. The following discusses the flood performance for various sections of the PWD levee.

Cobram to Cleaves

From the end of the Cobram Town Levee, near Harris Road, to the Dixon's Bend (near Smith Road), the freeboard is less than 300 mm for the 100 year ARI flood event. There is overtopping of the PWD levee by up to 300 mm for a section of 1300 m upstream from Dixon Bend. A short section of levee at Cleaves is overtopped by up to 300 mm.

The PWD levee at Dixon's Bend and Cleaves has suffered significant damage during major flood events. At Cleaves, the PWD levee abuts a natural sand hill. Major strengthening of the PWD levee has been undertaken at Dixon's Bend and Cleaves by the GBCMA since 2000 (Guy Tierney pers. comms).

Overtopping of the PWD levee upstream of Dixons Bend commences for flow greater than a 20 year ARI flood (251,000 ML/d) at Yarrowonga. At Cleaves, the overtopping commences for flows greater a 50 year ARI flood (328,000 ML/d) at Yarrowonga.

Figure 7-3 shows the freeboard along the PWD levee between the Harris Road (Cobram) and Cleaves.

A maintenance program is required to underpin the integrity of the PWD levee, and to preserve the capital investment made in the recent upgrading. The study team recommends that the GBCMA, in conjunction with the Moira Shire and the Department of Sustainability and Environment (Victorian Government) establish a suitable maintenance program. This action is seen by the study team as essential.

Breakouts at Dixon Bend travel via Torgannah Lagoon, and cross the Goulburn Valley Highway near Koonoomoo, continue via Sheepwash Creek to return via Ulupna Creek.

Augmentation (raising) of the PWD levee would lead to increases in flood levels for minor events, contained between the levees. These increases would reduce freeboard for both the New South Wales and Victorian levees.

The study team does not recommend raising the PWD levee, between Cobram and Cleaves. However, as discussed above, a maintenance program is required to underpin the current level of flood protection.

Figure 7-3 shows flood behaviour and freeboard/overtopping along the PWD levee between Harris Road (Cobram) and Cleaves. Detailed longitudinal profiles of levee crest and design flood level heights are shown in Appendix D.

Cleaves to Ulupna Creek confluence

The PWD levee abuts a natural sand hill adjacent to Torgannah Road, with Cleaves located to the east and Brentnalls to the west. A number of effluent streams once exited the main river channel near Bretnalls. These effluent streams have been infilled with sand. The presence of sand provides a preferential flow path under the levee and lessens the levee's integrity. Significant levee failure occurred at Brentnalls during the 1975 flood event. Major strengthening of the PWD levee has been undertaken at Brentnalls by the GBCMA since 2000 (Guy Tierney pers. comms).

The PWD levee provides protection for the 20 year ARI flood event (251,000 ML/d at Yarrawonga). For larger events, upstream levee overtopping/failures results in flooding behind the PWD levee through this section.

A maintenance program is required to underpin the integrity of the PWD levee, and to preserve the capital investment made in the recent upgrading. The study team recommends that the GBCMA, in conjunction with the Moira Shire and the Department of Sustainability and Environment (Victorian Government) establish a suitable maintenance program. This action is seen by the study team as essential.

Augmentation (raising) of the PWD levee would lead to increases in flood levels for minor events, contained between the levees. These increases would reduce freeboard for both the New South Wales and Victorian levees along the river, within the study area.

The study team does not recommend raising of the PWD levee, between Cleaves to Ulupna Creek confluence. However, as discussed above, a maintenance program is required to underpin the current level of flood protection.

Figure 7-4 displays the flood behaviour and freeboard/overtopping along the PWD levee between Cleaves and the Ulupna Creek confluence.

7.2.3 Ulupna Island

The Ulupna island levee provides protection for floods up to and including the 10 year ARI event (193,000 ML/d at Yarrawonga). The levees were not overtopped in the 1993 flood (183,000 ML/d Yarrawonga). Overtopping of the levee commences at the north east end of the island during a during a 20 year ARI flood event (251,000 ML/d at Yarrawonga). Significant failures/overtopping occurred during the 1975 event.

A maintenance program is required to underpin the integrity of the Ulupna Island levees. The study team recommends that the GBCMA, in conjunction with the Moira Shire and the Department of Sustainability and Environment (Victorian Government) establish a suitable maintenance program. This action is seen by the study team as essential.

Augmentation (raising) of the Ulupna Island levee would lead to increases in flood levels for minor events, contained between the levees. These increases would reduce freeboard for both the New South Wales and Victorian levees along the river, within the study area.

The study team does not recommend raising of the Ulupna Island levee. However, as discussed above, a maintenance program is required to underpin the current level of flood protection.

Figure 7-5 displays the flood behaviour and freeboard/overtopping along the Ulupna Island levee.

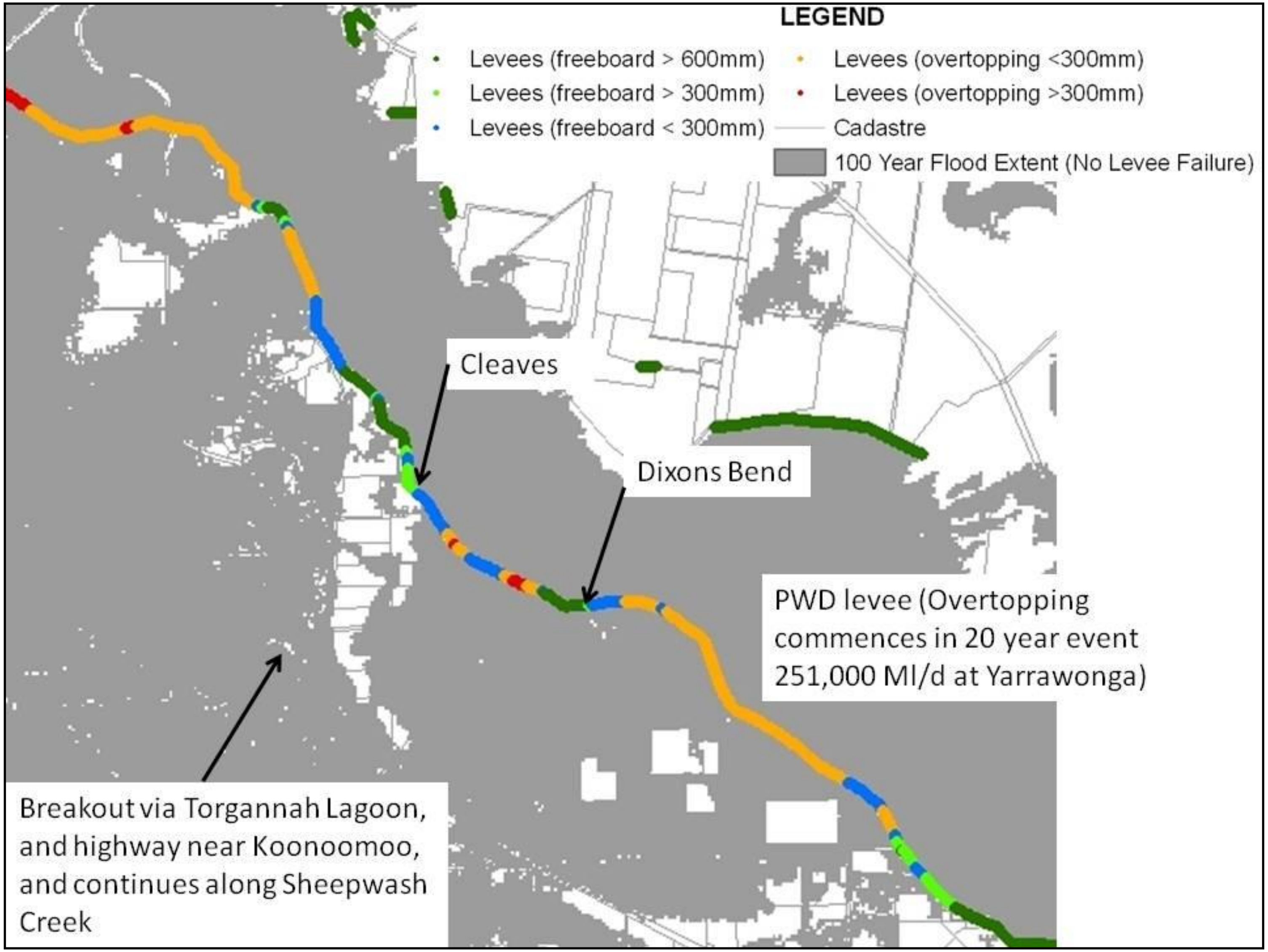

Figure 7-3 PWD levee – levee performance- Harris Road to Cleaves

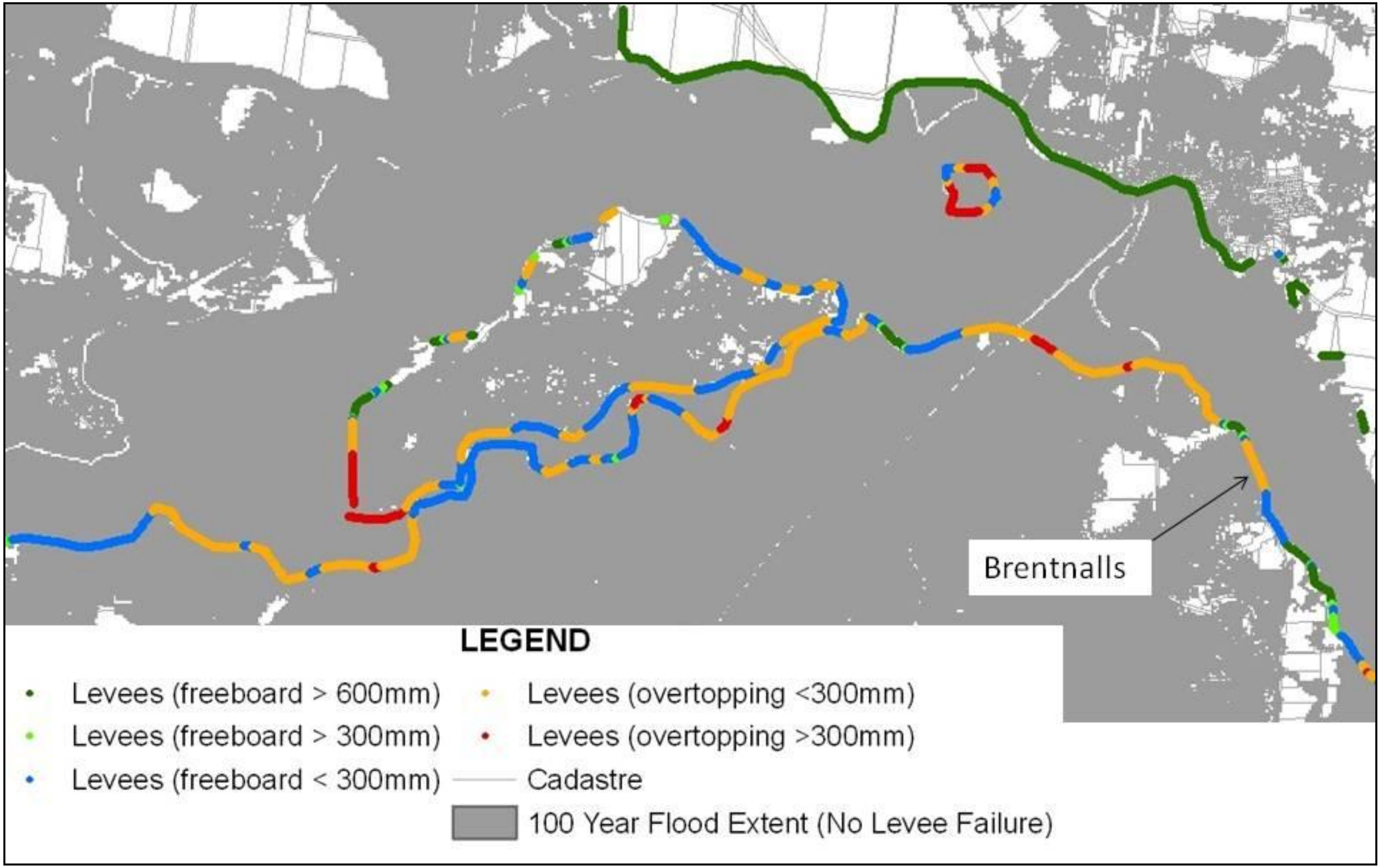

Figure 7-4 PWD levee – levee performance –Cleaves to Ulupna Creek confluence

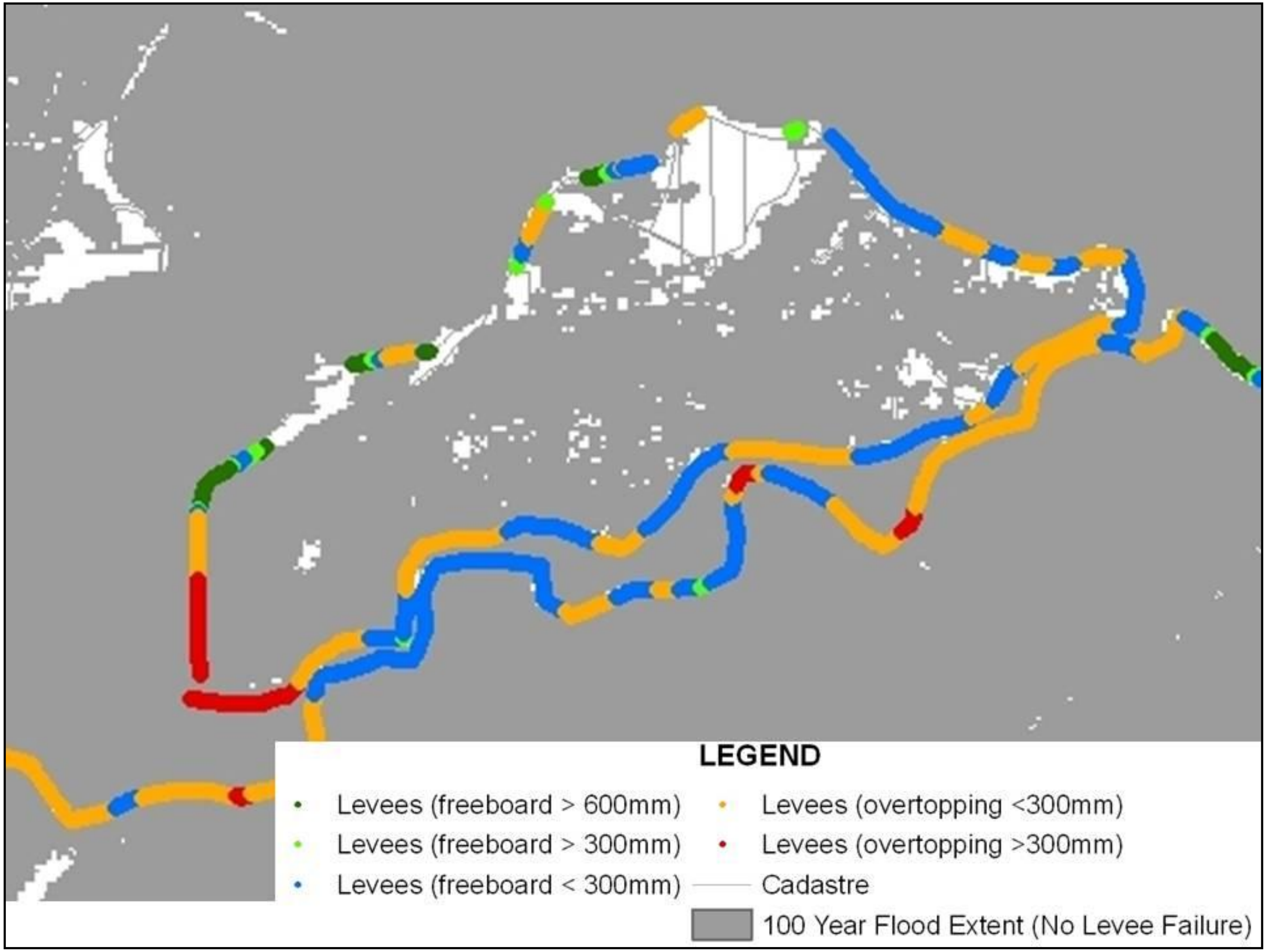

Figure 7-5 Ulupna Island levee –levee performance

7.2.4 Seppelts and Barooga Levee

An earthen levee, adjacent to Seppelts Road aims to prevent flow entering the Barooga Cowal depression. During the 100 year ARI flood event, the levee is overtopped. Also the levee is outflanked upstream (about 500 m) where the natural terrain is lower than the levee crest.

Once overtopped, flow continues along the Cowal adjacent to Mulwala Barooga Road. Limited flow affects properties adjacent to Cowal through the township of Barooga (Hughes Street). At the corner of Berrigan Road and Mulwala Barooga Road, the flow in the Cowal can continue to the north-west, generally within the Cowal depression. The flow continues to Tocumwal and affects properties along the eastern limit of Tocumwal. Affected properties are located in Marian Drive, Thurburns Road, Quicks Road and Babingtons Road.

The current levee crest elevation at Seppelts is generally around 116.8 m AHD, and the low lying area upstream is around 116.6 m. This compares to the 100 year ARI flood level of 117.3 m AHD. The 1975 flood event reached a height of 116.8 m AHD in this vicinity.

The augmentation/extension of the Seppelts Levee would prevent flows along the Barooga Cowal affecting Barooga and Tocwumal, and is further discussed in Section 7.3.

The Barooga levee is located some 7 kilometres downstream from Barooga (near Smithers Road). This levee prevents breakout and protects Tocumwal from overbank flooding from upstream. The levee is about 2 km in length and reaches up to 4 m in height. The levee has greater than 600 mm freeboard in the 100 year ARI flood event (387,000 ML/d). Given this degree of freeboard, the study team considers raising of the Barooga Levee is unwarranted.

A maintenance program is required to underpin the integrity of the Barooga levee. The study team recommends that the Berrigan Shire, in conjunction with relevant New South Wales Government agencies establish a suitable maintenance program. This action is seen by the study team as essential.

The 100 year ARI flood level adjacent to the Cobram – Barooga Bridge is around 116.2 m AHD. At this level, flooding encroaches on properties along Collie Street and Golf Course Road (adjacent to Vermont Street).

Figure 7-6 displays the flood behaviour and freeboard/overtopping along the Seppelts and Barooga levee.

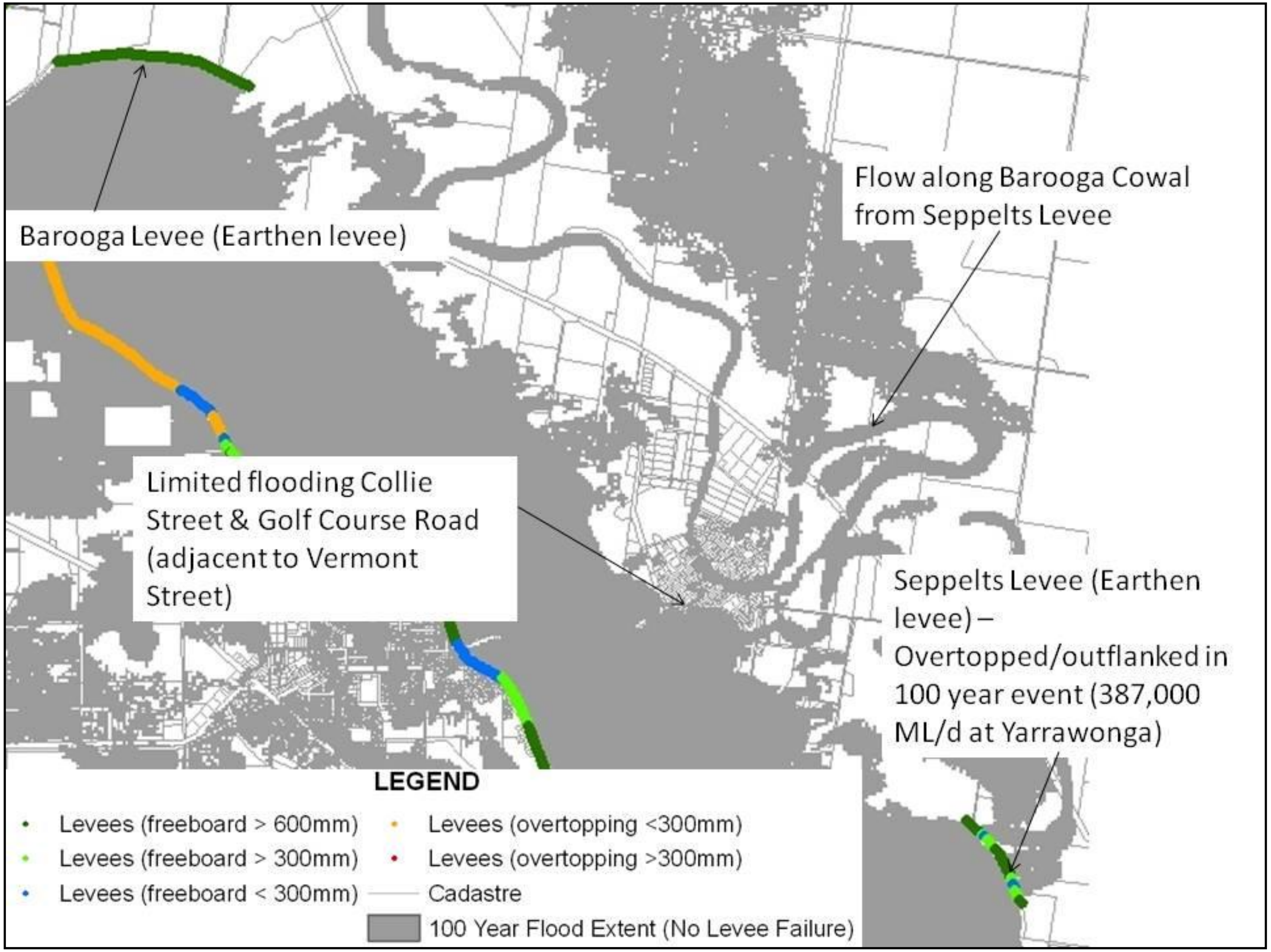

Figure 7-6 Seppelts and Barooga levee –levee performance

7.2.5 Tocumwal

The flood mitigation scheme for Tocumwal consists of the following five levee elements:

- Levee No. 1
- Cemetery Levee
- Levee No. 2
- Levee No. 3
- Levee No. 4

Figure 7-7 displays the flood behaviour and freeboard/overtopping for the five levee elements.

The levee elements, 1, 2 ,3 & 4 have a freeboard greater than 600 mm in the 100 year event. The Cemetery levee requires the placement of temporary barriers to achieve the freeboard requirement (600 mm).

Berrigan Shire (Graham Henderson pers. Comms 14/8/2008) advised that a section of Barooga Road (part of Levee No 2) adjacent to the golf course has been raised to 113.20 m AHD to provide 600 mm free board. These works were funded under the Natural Disaster Mitigation Program in June 2008.

Recent upgrading of the Tocumwal flood mitigation scheme was undertaken by Berrigan Shire in 1999/2000. Given this degree of freeboard, the study team considers raising the Tocumwal levees, a part of the proposed above works, is unwarranted.

A maintenance program is required to underpin the integrity of the Tocumwal flood mitigation scheme. The study team recommends that the Berrigan Shire, in conjunction with relevant New South Wales Government agencies establish a suitable maintenance program. This action is seen by the study team as essential.

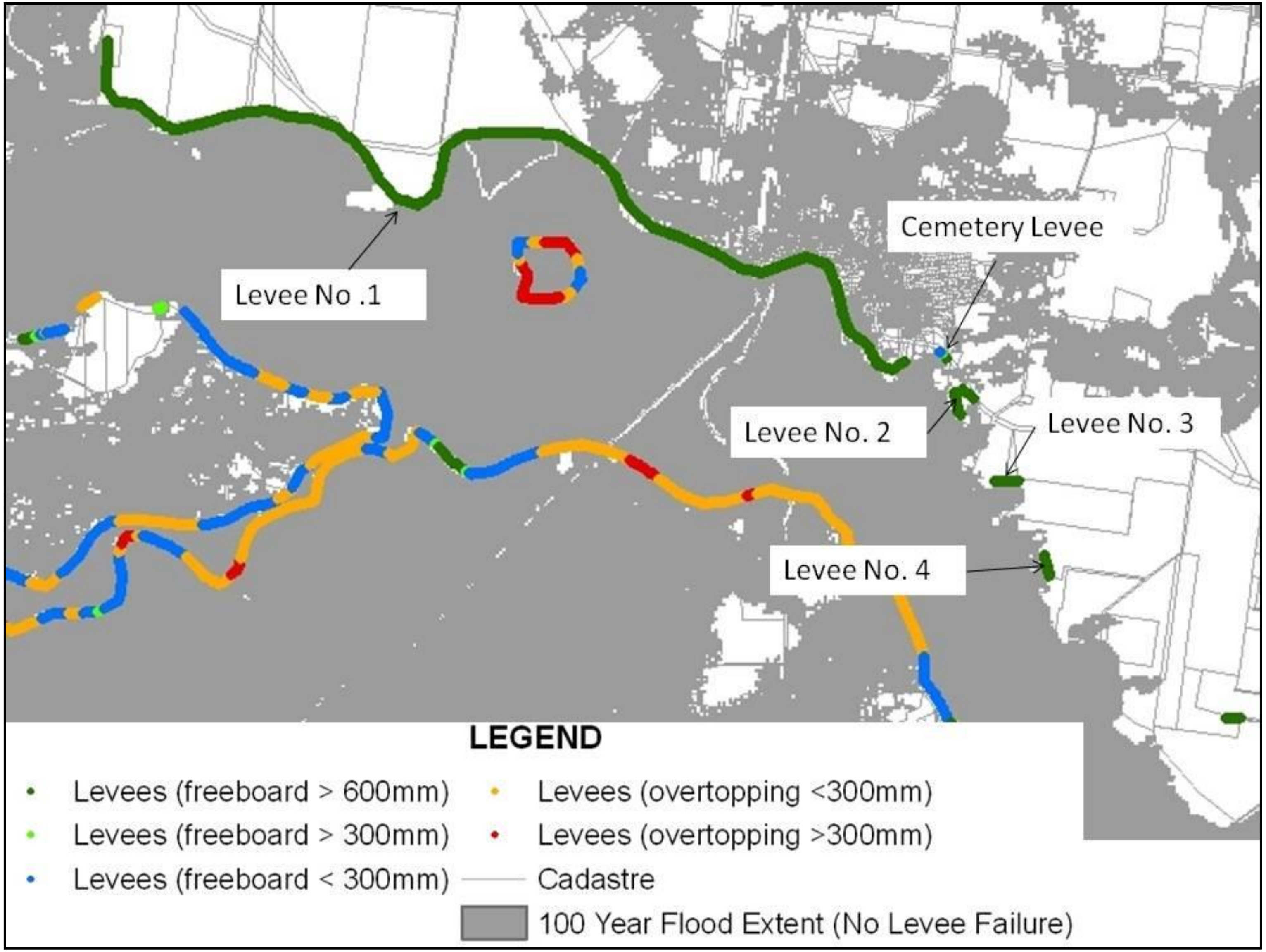

Figure 7-7 Tocumwal flood mitigation scheme –levee performance

7.2.6 Dicks Spillway - Sandbagging

A review of flood behaviour, adjacent to Dicks Spillway, shows in the 20 year ARI event (Flow at Yarrawonga 251,000 ML/d, Gauge Height 8.5 m) overtopping commences at Dicks Spillway. The depth of this overtopping is up to 0.1 m. Also, a breakout occurs some 1500 m upstream from Dicks Spillway. Refer to Figure 7-8.

Flood modelling was undertaken to assess the change in flood levels if sandbagging prevented overtopping at Dicks Spillway for a 20 year ARI event. This flood modelling revealed that there were no significant changes (less than 0.005 m) in flood levels from Dicks Spillway to the Cobram-Barooga Bridge for the 20 year ARI event. The area behind Dicks Spillway, adjacent to Cemetery Road, experiences similar flood behaviour as for the no-sandbagging scenario as this area is inundated by the upstream breakouts, with flow across the Murray Valley highway, and then backwatering behind Dicks Spillway as shown in Figure 7-8.

The assessment of the sandbagging arrangements indicates that sandbagging at Dick Spillway results in no changes to the flood behaviour, for the 20 year ARI event, in the Murray River or in the area behind Dicks Spillway.

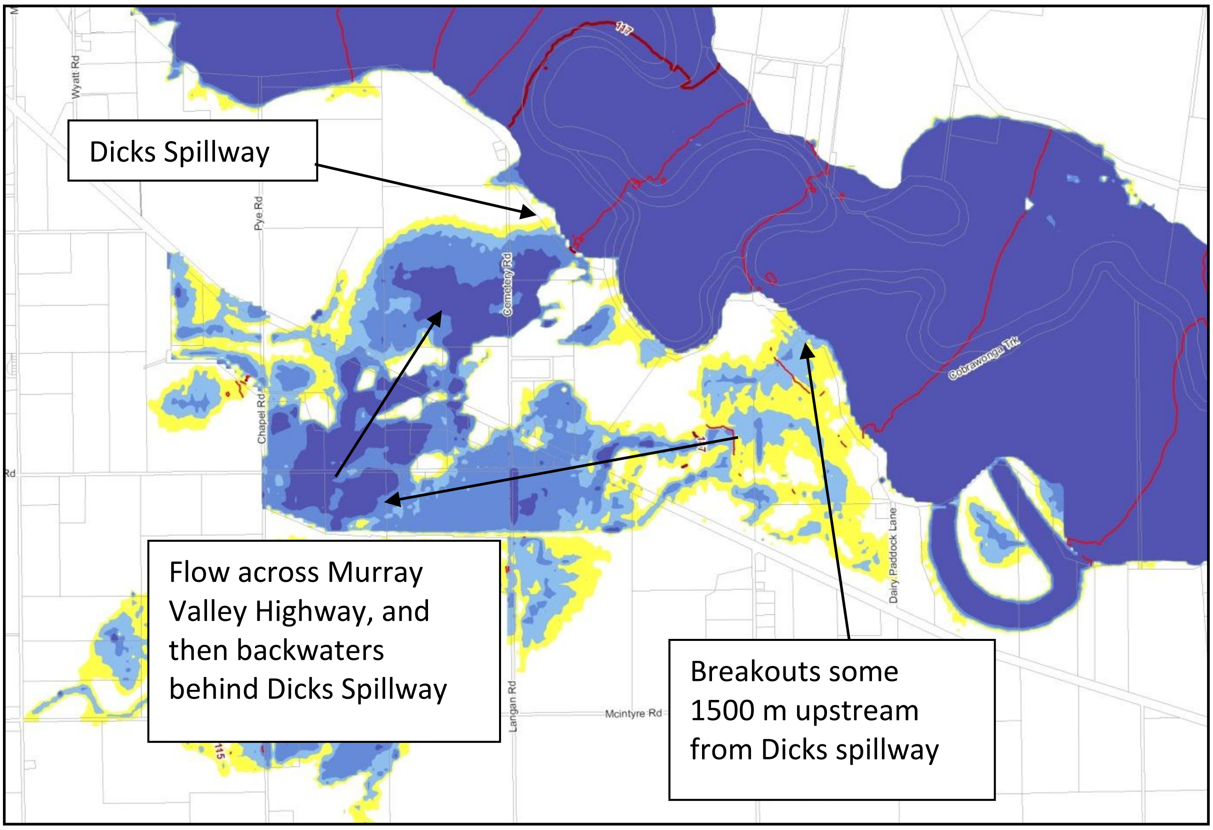

Figure 7-8 Dick's Spillway – Flood behaviour – 20 year ARI event

7.3 Potential structural mitigation augmentation

The potential structural mitigation measures indentified include:

- Seppelts Levee - Augmentation/extension: To prevent flows along the Barooga Cowal.
- Murray Valley Highway to the south of Cobram: Flooding occurs along the Murray Valley Highway. Use of temporary flood barriers.
- Investigate upgrading town levees of Cobram and Tocumwal to meet adopted freeboard standards above the 100 year ARI design flood.

8 NON-STRUCTURAL MITIGATION MEASURES ASSESSMENT

8.1 Overview

This section discusses a range of non-structural mitigation measures, which includes land use planning, flood warning and flood response.

8.2 Revised flood related provisions and overlays delineation

8.2.1 Moira Shire (Victoria)

The current Moira Planning Scheme applies the Land Subject to Inundation Overlay (LSIO) and the Rural Floodway Overlay (RFO). The current LSIO and RFO extents are provided in Appendix D.

The current LSIO extents are based on the 1 in 100 year ARI flood extent estimated from the Murray River Floodplain Management Study (RWCV et al 1986). The current RFO is intended to delineate land subject to higher flood risk.

The existing conditions hydraulic analysis, discussed in Section 6, provides considerable refinement of the current LSIO and RFO.

The Floodway Overlay is defined according to the guidelines provided by DNRE (1998b). The guidelines provide three approaches to the delineation of FO as follows:

- Flood frequency
- Flood depth
- Flood hazard

For **flood frequency**, DNRE (1998b) suggest areas that flood frequently and for which the consequences of flooding are moderate or high, should generally be regarded as floodway. The 10 year ARI flood extent was considered an appropriate floodway delineation option. Using the 10 year ARI event definition limits the FO delineation to the river corridor between the levees.

Flood hazard combines the flood depth and flow speed for a given design flood event. DNRE (1998b) suggest the use of Figure 8-1 for delineating the floodway based on flood hazard. The flood hazard for the 1 in 100 year ARI event was considered for this study. Figure 8-1 displays the flood hazard criteria for floodway delineation.

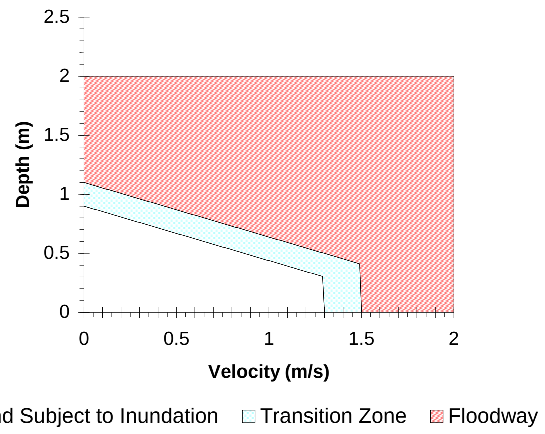

Figure 8-1 Floodway overlay flood hazard criteria

For **flood depth**, regions with a flood depth in the 1 in 100 year ARI event greater than 0.5 m were considered as FO based on the flood depth delineation option. The 100 year ARI extent for the Victorian levee failure scenario was adopted as the basis of the application of the flood depth criteria.

As outlined in Section 6.4, 100 year ARI extents were mapped for four floodplain arrangements, no levee failure, Victorian levee failure, New South Wales levee failure and the removal of the Victorian irrigation infrastructure.

The study team recommends that the GBCMA and Moira Shire adopt the maximum extents from the Victorian levee failure (as defined in Section 6.4.3) and the removal of the Victorian irrigation infrastructure scenarios as the revised LSIO delineation. The adoption of the maximum envelope serves to recognise residual flood risk following floodplain development.

Figure 8-2 displays the proposed/draft Moira Shire LSIO and FO delineation

The study team recommends that Moira Shire adopt the draft LSIO and FO as the basis for a Planning Scheme Amendment. Further, the study team recommend that GBCMA provide the appropriate assistance to Moira Shire to enable the timely adoption of the Planning Scheme Amendment.

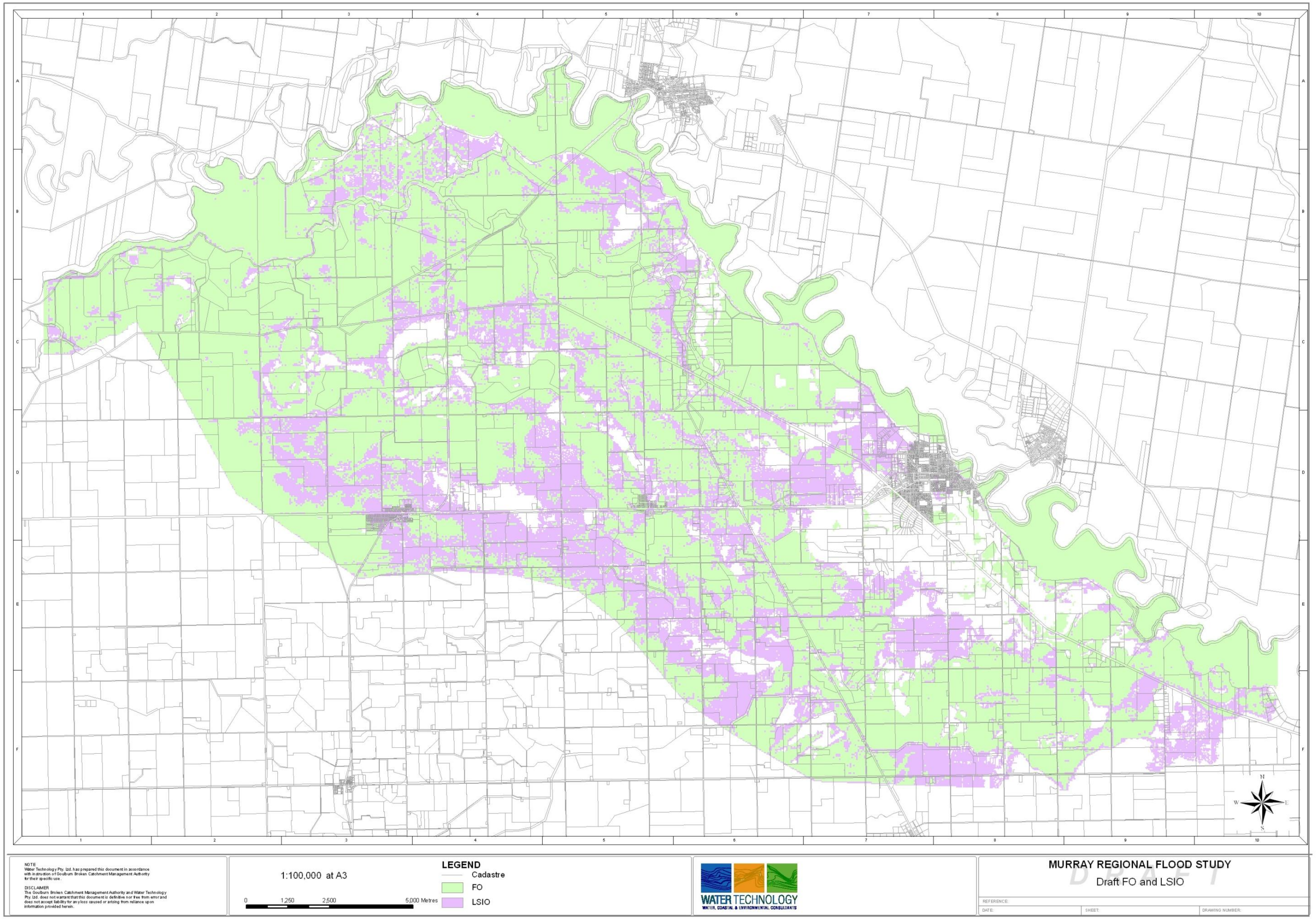

Figure 8-2 Moira Shire – Draft FO and LSIO delineation

8.2.2 Berrigan Shire (New South Wales)

The Floodplain Development Manual (NSW Government 2005) sets the policy framework for the management of flood liable land. This study addresses the data collection and flood study components of the Floodplain Risk Management Process, as defined in the Floodplain Development Manual (NSW Government 2005).

This study mapped flood extents and depths for design floods from 1 in 10 to 1 in 500 year ARI. The determination of the flood planning level (FPL) is able to be assessed by Berrigan Shire and relevant New South Wales Government agencies on the basis of the flood mapping outputs.

The study team recommends that Berrigan Shire, in conjunction with relevant New South Wales Government agencies determine appropriate FPLs.

Further, the study's mapping outputs underpin the determination of the six categories of flood-prone land (NSW Government 2005):

- Low Hazard - Flood Fringe
- Low Hazard - Flood Storage
- Low Hazard - Floodway
- High Hazard - Flood Fringe
- High Hazard - Flood Storage
- High Hazard - Floodway

The study team recommends that Berrigan Shire, in conjunction with relevant New South Wales Government agencies delineate the flood-prone land categories.

8.3 Flood forecasting and warning

The Bureau of Meteorology provides flood warnings for the Murray River at Yarrawonga Weir (downstream). The current flood warning categories are defined in Table 8-1.

Table 8-1 Murray River at Yarrawonga- flood warning categories

Location	Minor	Moderate	Major
Murray River at D/S Yarrawonga Weir – Height (m)	6.4	6.7	7.8
Murray River at D/S Yarrawonga Weir – Flow (ML/d)	82,000	98,000	182,000

VFWCC (2005) identified flood warning system development priorities throughout Victoria and ranked the Murray River catchment sixth on a list of ten priority catchments. The key elements for the Murray River catchment were:

- Opportunities to improve the lead time of forecast outflows from Lake Hume and other storages.
- Development of a service level agreement identifying key locations and services needs

- Existing data collection networks
- Local data management arrangements
- Existing warning dissemination arrangements
- Opportunities to improve flood awareness and preparedness at individual and agency level

The study team recommends that relevant Victorian and New South Wales Government agencies, in conjunction with local authorities, establish a framework to address the key elements arising from VFWCC (2005) affecting the study area.

8.4 Flood response

Flood response for Moira Shire (Victoria) is outlined in the Moira Municipal Emergency Management Plan (MEMP) and the accompanying Flood Sub-plan.

A revised Moira Shire sub-plan has been developed by Michael Cawood and Associates, and includes relevant information on local flood behaviour and intelligence from the existing conditions hydraulic analysis.

The study team recommends that the study outcomes form the basis of a revised Flood Sub-plan as an integral part of the Moira Shire MEMP.

For the New South Wales floodplain, the NSW SES requested revised flood intelligence based on the outcomes of this study. Michael Cawood and Associates has prepared revised flood intelligence for use by NSW SES and Berrigan Shire.

The study team recommends that the outcomes of this study form the basis of revised flood intelligence for use by Berrigan shire and the NSW SES.

9 STUDY CONCLUSIONS AND RECOMMENDATIONS

This section summarises the conclusions and recommendations arising from this study.

Hydrologic analysis

The study team applied a rigorous approach to the determination of design flood hydrographs for the study area. The study team acknowledges there is some uncertainty surrounding the design flood estimates developed by this study, primarily due to uncertainties in the historical flow record.

Hydraulic analysis

Formal calibration of the hydraulic model was limited by the extent_ of flood level information available (particularly for 1993) and uncertainty around historic levee failures. The study team undertook broad validation of the modelled flood extents through community consultation and a comparison to flood levels.

A key factor influencing _model sensitivity and results is levee failure. The consideration of three levee failure scenarios provides reasonable bounds around the likely range of flood behaviour.

The study team acknowledges considerable uncertainty surrounding the modelled flood extents given the necessary assumptions related to levee breaches. Given the unpredictable nature of levee failures, no two floods (even if the flows were identical) would produce the same inundation pattern within the study area.

It is recommended, as additional topographic data for the Victorian floodplain becomes available, consideration is given to the extension of the study area.

Structural mitigation measures assessment

The study team recommends the following actions:

- *PWD levee: GBCMA, in conjunction with the Moira Shire and the Department of Sustainability and Environment (Victorian Government) establish a suitable maintenance program. This action is seen by the study team as essential.*
- *Ulupna Island levee: GBCMA, in conjunction with the Moira Shire and the Department of Sustainability and Environment (Victorian Government) establish a suitable maintenance program. This action is seen by the study team as essential.*
- *Barooga levee: Berrigan Shire, in conjunction with the relevant New South Wales Government agencies, establish a suitable maintenance program. This action is seen by the study team as essential.*
- *Tocumwal Town levee: Berrigan Shire, in conjunction with the relevant New South Wales Government agencies, establish a suitable maintenance program. This action is seen by the study team as essential. Also, Berrigan Shire, in conjunction with the relevant NSW Government agencies assess the feasibility to provide the standard freeboard of 600 mm above the 100 year ARI design flood levels.*
- *Seppelts Levee - Augmentation/extension: Berrigan Shire, in conjunction with the relevant New South Wales Government agencies investigate the feasibility of the augmentation and extension of Seppelts Levee to prevent flows along the Barooga Cowal.*

- *Murray Valley Highway to the south of Cobram: Moira Shire, in conjunction with the GBCMA, assess the feasibility of the use of temporary flood barriers to limit flooding along the Murray Valley Highway.*
- *Cobram Town Levee: Moira Shire, in conjunction with the GBCMA, assess the feasibility to provide the standard freeboard of 600 mm above the 100 year ARI design flood levels.*

Land use planning

Victoria:

- *The study team recommends that the GBCMA and Moira Shire adopt the maximum extents from the Victorian levee failure (as defined in Section 6.4.3) and the removal of the Victorian irrigation infrastructure scenarios as the revised LSIO delineation.*
- *The study team recommends that Moira Shire adopt the draft LSIO and FO as the basis for a Planning Scheme Amendment. Further, the study team recommend that GBCMA provide the appropriate assistance to Moira Shire to enable the timely adoption of the Planning Scheme Amendment.*
- *The study team recommends that Moira Shire, through referrals to the GBCMA, should apply appropriate minimum floor levels (100 year ARI design flood level plus freeboard) for new dwellings within the mapped 100 year ARI flood-extent.*

New South Wales

- *The study team recommends that Berrigan Shire, in conjunction with relevant New South Wales Government agencies determine appropriate FPLs.*
- *The study team recommends that Berrigan Shire, in conjunction with relevant New South Wales Government agencies delineate the flood-prone land categories.*
- *The study team recommends that Berrigan Shire, with support of NOW, should apply appropriate minimum floor levels (100 year ARI design flood level plus freeboard) for new dwellings within the mapped 100 year ARI flood-extent*

Flood Warning

VFWCC (2005) identified flood warning system development priorities throughout Victoria and ranked the Murray River catchment sixth on a list of ten priority catchments. The key elements for the Murray River catchment were:

- Opportunities to improve the lead time of forecast outflows from Lake Hume and other storages.
- Development of a service level agreement identifying key locations and services needs
- Existing data collection networks
- Local data management arrangements
- Existing warning dissemination arrangements
- Opportunities to improve flood awareness and preparedness at individual and agency level

The study team recommends that relevant Victorian and New South Wales Government agencies, in conjunction with local authorities, establish a framework to address the key elements arising from VFWCC (2005) affecting this study area.

Flood Response

The study team recommends that the outcomes of this study form the basis of a revised Flood Sub-plan as an integral part of the Moira MEMP.

The study team recommends that the outcomes of this study form the basis of revised flood intelligence for use by Berrigan shire and the NSW SES.

10 REFERENCES

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APPENDIX A TOPOGRAPHIC SURVEY

SKM

Murray River Regional Flood Study

LA10724

SURVEY REPORT

- Progressive
-

Murray River Regional Flood Study

SURVEY REPORT

- Progressive
- 17/11/2005

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Survey Report

SKM

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SINCLAIR KNIGHT MERZ

Websters & Horshoe Lagoon– Survey Report PAGE 2

Survey Report

Contents

1.1 Introduction

The Goulburn Broken CMA, in association with Berrigan and Moira Shire Councils is conducting a Murray River Regional Flood Study from Dicks/Seppelts Levees to Murray River and Ulapna Creek Junction. The study aims to understand the nature of flooding in this area during a repeat of a 100-year ARI type flood under the current infrastructure configuration. As part of this study, a computer model will be developed to simulate flood behaviour for a range of flood magnitudes. The computer model requires details of structures (siphons/subways/culverts) and embankments (channels/drains/road). A field survey will be undertaken to gather the required details.

1.2 Scope of Survey

The survey will roughly cover the area between Cobram, Strathmerton and Tocumwal. Details of important structures such as siphons, subways and culverts will be surveyed.

As such an extensive survey, covering the land existing within the irrigation networks (channels / drains) through the 35km^2 area will be commenced. The survey largely based on RTK GPS, will aim to capture and provide details of existing structures and impediments to flood waters.

1.3 Site Briefing

Due to the location of many significant features over private lands, letters are included to be given to enquiring resident or landholders.

1.4 Methodology

The large proportion of the area will be surveyed by RTK GPS (10-50mm accuracy) where possible, aiming to gather information on the majority of road crests and embankments (that provide significant obstruction / influence to flood paths). Other features such as Siphons, Subways, Culverts and Bridges, where existing will be captured by similar methods. Where RTK GPS is not suitable, Total Stations, and conventional surveying techniques will be employed.

Data will be post processed and reduced in appropriate software packages, which include Trimble Geomatics Office, Leica Ski Pro, and Terramodel. Control checks will be performed amongst many numerous existing marks (as identified below). Further Static GPS results are to be recorded for verification of data, using local Victorian based GPSnet and the Australian Regional GPS network reference data.

Reduced data is to be collated in Terramodel in which it will be organised by SKM CAD convention. Output is XYZ ASCII, with survey point numbers included.

Geocomp Codes have been adopted:

102 – Top Bank
103 – Bottom Bank
301 – Open Lined Drain
104 – Natural Surface
401 – Road Crest Level
403 – Road Edge
303 – Water Level
305 - Headwall
324 – Drain Invert
605 – Bridge Abutment
606 – Bridge Extents
629 – Conc Slab
801 – Rail Line

1.5 Datum

Horizontal: MGA GRID 94 Zone 55 (GDA94)

Vertical: Australian Height Datum (AHD83)

1.6 Control Observed

GPS "here" coordinates were used and then later fitted to at least three reliable benchmarks.

VERTICAL (AHD)

Bench Mark	AHD (known)	AHD(adopted)
SR66L61	110.484	110.47
SR67L34	110.075	110.07
SR74A13	110.548	110.55
PM 127 STRATHMERTON	110.154	110.17
SR76E68.01	110.116	110.14
SR72R44	112.794	112.801
SR80U24C	110.745	110.755
SR74A22	112.798	112.826
SR75A3	113.959	114.039
SR70E61	112.208	112.191
SR76E64	112.158	112.164
SR76E63		113.88
SR66L15	114.066	114.048
SR16E68	110.116	110.138
SR79P33	117.267	117.25
SR79P19	116.761	116.75
SR66I37	116.519	116.553

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HORIZONTAL (MGA 55)

Point_ID	CONTROL EAST	CONTROL NORTH	ADOPTED MGA94 ZONE 55 EAST	ADOPTED MGA94 ZONE 55 NORTH
PM 128 STRATHMERTON	363575.563	6023188.861	363575.57	6023188.88
SR66L65	364134.2	6026177.8	364134.21	6026177.85
PM127 STRATHMERTON	367439.153	6028415.148	367439.12	6028415.11
PM 129 YARR0WEYA	367524.149	6029506.593	367524.16	6029506.56

See *Cobram_GPS.xls* for full details.

1.7 Temporary Bench Marks

Temporary Bench-Marks (TBMs) were placed

Name	Site	East	North	RL
RM DA	RAIL BRIDGE D	366176.43	6032102.39	110.06
RM DB	RAIL BRIDGE D	366158.19	6032082.83	110.11
STN D	RAIL BRIDGE D	366209.85	6032135.97	109.61
TBM STOKES	NEAR BASE STOKES RD	367383.50	6033391.42	110.72
TBM MURRAY	MURRAY RIVER BRIDGE (TOCUMWAL)	369785.81	6035698.30	115.80
TBM1	SEE LOCALITY	369446.33	6034932.41	112.10
TBM2	SEE LOCALITY	369105.57	6034334.89	111.94
TBM3	SEE LOCALITY	369905.65	6033834.19	112.35

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TBM4	SEE LOCALITY	369966.35	6033641.89	111.47
TBM R1	RAIL BRIDGE K	368964.26	6034835.66	112.06
TBM R2	RAIL BRIDGE K	369006.95	6034871.32	112.04
TBM R3	RAIL BRIDGE L	368539.079	6034447.601	111.906
TBM R4	RAIL BRIDGE L	368470.114	6034386.024	111.936

See Cobram.dwg for Locality Plan

1.8 PHOTO REFERENCES

Rail Bridges		**JPEG**
Bridge A		1640 - 1641
Bridge B		1638
Bridge C		1636
Bridge D		1633 - 1634
Bridge E		1632
Bridge F		1637
Bridge G		1635
Bridge H		1619 - 1620
Bridge I		1624 - 1625
Bridge J		1621 - 1623
Bridge K		1613-1615
Bridge L		1616-1618

Road Bridges		**JPEG**
Bridge 1		1601-1603
Bridge 2		1604-1606
Bridge 3		1607-1609
Bridge 4		1610-1612
Bridge 5		1628
Bridge 6		1626 - 1627
Bridge 7		1629

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2.0 STAGE ONE STATUS

Survey Report from 29/9/05

1. *Status at 29/9/05*
2. *Future Survey*
3. *Output*
4. *Contact Numbers*

1. STATUS (Mark Boyce)

a) The Rail from Strathmerton to the Murray River (at Tocumwal) has been surveyed, except for the reach from Bridge H to midway between (at palm tree) Bridges J and K
(All Bridges/Culverts and spot levels on the top of rail have been done).

Remaining survey is Bridges K and L and some spot levels on rail.

b) The Tocumwal Road (Goulburn Valley Highway) has been surveyed for spot levels from the Murray River South (approx. 3km) to the Levee.
- All bridges have coordinated TBMs in place for Total Station Survey

- Bridges 5 and 6 Culvert 1A have been surveyed only (see General Arrangement)

- TBMs 4 and 5 north and south of the levee can be used for the following survey south of the levee to Koonoomoo. Culverts have been found and a list (& running distances from Mywee Road) is shown on the photocopy of Vicroads. Note most (if not all) of these culverts south of the levee probably do not need to be surveyed (check with Water Tech).

c) The drainage Subuary and Channel Siphon/Drain X-sec has been surveyed near the south end of the rail survey.

d)
   i. Total Station (Geodimeter) data has been downloaded but **not** reduced;
   ii. GPS (Leica) data has been downloaded but **not** reduced (wait for more survey control)

e) Existing digital data

2. FUTURE SURVEY
    i. Remaining Bridges on main Goulburn Valley Highway (i.e. Bridges 1, 2, 3, &4)
    ii. Road levels (& culverts?) on G.V. Hwy (south of Levee) to Koonoomoo
    iii. Rail Survey (see Item (a) of Status)
    iv. All remaining Subways (i.e. Drain pipe under a Channel – see below)
    v. All remaining Cross Sections at siphons (i.e. A Cross Section above the channel pipe siphon in the bed of the degression/drain – see below)
    vi. Road crest levels
    vii. Channel crest levels
    viii. Road Cross Sections at Channels (see below)

2.1 STAGE ONE OUTPUT

a) An XYZ ASCII file that includes road, subway, siphon, levee, and survey marks surveyed between 21 September and 21 October 2005. This file includes survey point numbers and Geocomp feature codes;
b) A Corresponding AutoCAD DWG file.

3.0 STAGE TWO STATUS

Status 24/10/2005 (Sam Griffiths)

The Western / Central extents of road/channel surveys have been completed. This has included the capture of siphons/subways (where found) and major channels crossing roads.

Status 17/11/2005 (Sam Griffiths)

Road bridges and Rail bridges have all been surveyed – with Rail bridge K & Road bridge 1 coordinated via static GPS post processing – R1, R3, R4 (R2 – CA position)

Post processing of Base 2 revealed a 15 cm elevation error against AUSPOS – this is probably expected due to Geoid coordination of ARGN stations. The data has been checked against existing control.

All channel sections, siphons and subways have been completed and road levels captured throughout project areas.

All existing survey data files (DAT, GSI, & DC) have been downloaded and reduced.

Data currently resides in separate day files and has yet to be merged into an overall file.

Description data on siphons / subways has not been digitally recorded, and need to be appended to data. It would be best to wait until all data has been merged.

Control checks have been performed on all new existing data captured post 8/11.

Only TGO CSV outputs for data post 8/11.

Bridge Survey

All bridges (Bridge A to Bridge L) are now surveyed. Accompanying sketches are available as field notes.

Control has been brought in as per requirements of Mark Boyce report. Digital data for these bridges resides in Terramodel (fully reduced files) – a few further GPS rail shots exist in TGO file 15/11.

Photo's are referenced via excel document in project folder.

Status 02/12/2005 (Byron Starkey)

All of the above data has been collated into a Terramodel file, and an ASCII points coordinate listing has been produced in comma separated format. The listing has been enhanced with a field for pipe diameter (where possible to survey) and JPEG image number referencing.

All road and rail bridge sketches have been scanned and compiled with corresponding JPEG images into Adobe PDFs. Twelve out of the eighteen bridge sketches have been drawn up in Microstation, but are not to scale and do not include point numbers. The usefulness of this format is being assessed at present.

An AutoCAD locality file had been produced for the extent of survey.

3.1 STAGE TWO OUTPUT

a) An XYZ ASCII file that includes all road, subway, siphon, levee, and survey marks surveyed. This file includes survey point numbers and Geocomp feature codes;

b) Rail/Road bridge General Arrangement PDFs. These files combine digital photos and field sketches of these structures into Adobe PDFs. (see below for the typical format);
c) All relevant digital photos taken as part of the survey;
d) An AutoCAD (DWG) locality plan with all the data included (including bridges).

General Arrangement Sketches/Digital photo (typical output):

Murray River Regional Flood Study

[Cross Section]

[Digital Photo]

Surveyed by SKM

APPENDIX 1

CONTACT NUMBERS

- Byron Starkey (SKM) Mobile 0421 897 633, Office 9508 6122
- Simon Nazaretian (SKM) Mobile 0409 149 275, Office 9508 6111
- Sam Griffiths (SKM) Mobile 0417 476 699, Office 9248 3561
- Steve Muncaster (WaterTech) Office 9558 9366
  Email Steve.Muncaster@watech.com.au

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APPENDIX 2

Accuracy of Data

0.05m Vertical

0.5m Horizontal

References: ICMS (2002) Standards and Practices for Control Surveys, Publication No 1

Description	Horizontal	Vertical
Between TBM's at each site	Class E	Class LD
Between TBM's and GDA (taken as nearest PSM)	Class E, lower accuracy may be acceptable subject to discussion	Class D, Lower level may be acceptable subject to discussion

APPENDIX 3

Project Directory

I:\LAHM\Projects\LA10724\Technical

DESCRIPTION	LOCATION
Survey Report	\Technical\10724_report.doc
SMES LOCALITY	\Technical\All PMs.doc
ORIGINAL REPORT-MARK BOYCE	\Technical\SurveyStatus_and_brief.doc
LOCALITY OCTOBER 2005	\Technical\BASE2.dgn
CHANNELS – REFERENCED TO BASE 2	\Technical\COBRCHLS.DGN
DRAINS – REFERENCED TO BASE 2	\Technical\COBRDRNS.DGN
BENCH MARKS (REF TO BASE 2)	\Technical\GMW_BM.DGN
RAW GPS DATA	\LEICA GPS\GPS_RAW_DATA\
REDUCED GPS DATA	\LEICA GPS\2005-10-19.XLS
GEODIMETER RAW DATA	\GEOCOMP\072401-LAB.AGA
GEODIMETER RAW DATA	\GEOCOMP\072402-LAB.AGA
GEODIMETER RAW DATA	\GEOCOMP\072403-LAB.AGA
TSP 1200 GSI FILES	\Technical\Survey_Data\GSI
TGO FILES (18/10 – 16/11)	\Technical\Survey_Data\TGO
STATIC GPS POST PROCESSING	\Technical\Survey_Data\Static_GPS
SITE PHOTOS / BRIDGE PHOTOS	\Technical\Photos

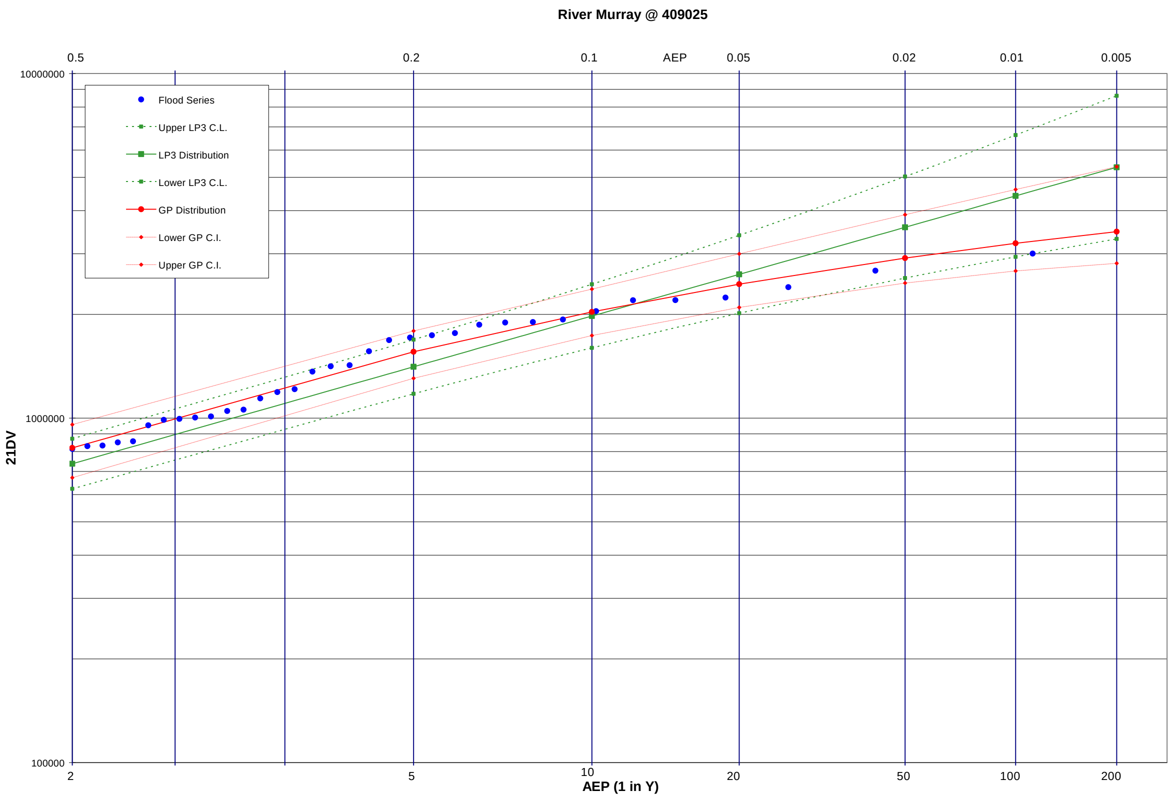
APPENDIX B HYDROLOGIC ANALYSIS

Murray River at Yarrawonga (Downstream of weir) – peak flow data

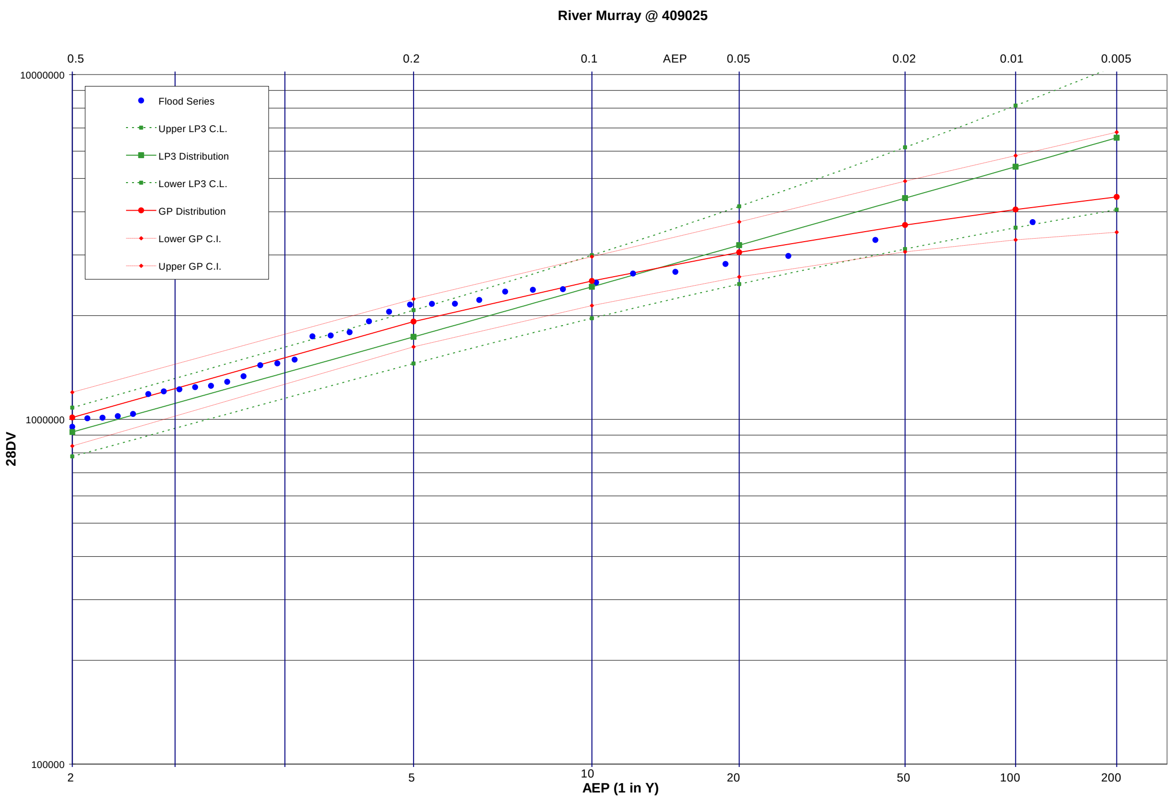
	Adopted peak flow data		Agency gauged data	SRWSC-A	SRWSC-B	Agency gauged data		
Year	Peak flow ML/d	Source	Peak flow ML/d	Peak flow ML/d	Peak flow ML/d	14 day vol (ML)	21 day vol (ML)	28 day vol (ML)
1905	220000	SRWSC-B		179000	220000			
1906	264000	SRWSC-B		210000	264000			
1907	45500	SRWSC-B		41000	45500			
1908	48000	SRWSC-B		43000	48000			
1909	252000	SRWSC-B		190000	252000			
1910	72000	SRWSC-B		61000	72000			
1911	58900	SRWSC-B		54000	58900			
1912	160500	SRWSC-B		135000	160500			
1913	47000	SRWSC-B		42000	47000			
1914	15700	SRWSC-B		14000	15700			
1915	115000	SRWSC-B		90000	115000			
1916	111500	SRWSC-B		98000	111500			
1917	390000	SRWSC-B		306000	390000			
1918	102000	SRWSC-B		79000	102000			
1919	32000	SRWSC-B		29000	32000			
1920	134000	SRWSC-B		108000	134000			
1921	196000	SRWSC-B		159000	196000			
1922	208000	SRWSC-B		36000	208000			
1923	92600	SRWSC-B		73000	92600			
1924	187000	SRWSC-B		159000	187000			
1925	53200	SRWSC-B		53000	53200			
1926	78900	SRWSC-B		65000	78900			
1927	51200	SRWSC-B		39000	51200			
1928	63600	SRWSC-B		66000	63600			
1929	44000	SRWSC-B		42000	44000			
1930	65500	SRWSC-B		58000	65500			
1931	210000	SRWSC-B		179000	210000			
1932	119800	SRWSC-B		109000	119800			
1933	58900	SRWSC-B		61000	58900			
1934	102000	SRWSC-B		95000	102000			
1935	65500	SRWSC-B		67000	65500			
1936	140000	SRWSC-B		123000	140000			
1937	20500	SRWSC-B		17000	20500			
1938	18400	SRWSC-B		15000	18400			
1939	101533	Agency gauged data	101533	102000	153500	1321640	1899178	2346534
1940	12942	Agency gauged data	12942	13000	14500	181157	309076	452127
1941	17493	Agency gauged data	17493	18000	21000	202405	278885	370681
1942	71024	Agency gauged data	71024	71000	89000	834588	1140067	1452471
1943	33910	Agency gauged data	33910	34000	43000	443907	647156	873929
1944	9615	Agency gauged data	9615	10000	9900	124580	180239	239666
1945	16722	Agency gauged data	16722	17000	20000	170967	246260	313896
1946	95735	Agency gauged data	95735	96000	131000	1151382	1562444	1924304
1947	56467	Agency gauged data	56467	57000	65000	680172	1010289	1333457
1948	58962	Agency gauged data	58962	59000	66000	655487	855812	1021323
1949	58962	Agency gauged data	58962	59000	66000	722988	1002974	1239068
1950	41959	Agency gauged data	41959	42000	49600	541256	759784	951008
1951	84652	Agency gauged data	84652	85000	102000	989456	1363660	1739772
1952	140556	Agency gauged data	140556	141000	140000	1445314	1866174	2218041
1953	79147	Agency gauged data	79147	79000	84100	1002704	1413019	1788471
1954	42473	Agency gauged data	42473	43000	44700	443410	651638	805836
1955	181096	Agency gauged data	181096	181000	171000	1968365	2677292	3312836
1956	203677	Agency gauged data	203677	208000	193000	2219293	3001803	3731629
1957	16201	Agency gauged data	16201	16000	18400	205522	371339	556911
1958	157090	Agency gauged data	157090	163000	157000	1404143	1764206	2163181
1959	26927	Agency gauged data	26927	27000	29700	348632	471729	573773
1960	101445	Agency gauged data	101445	105000	108000	1065239	1423317	1749744
1961	19036	Agency gauged data	19036	19000	21200	156415	217693	307904
1962	18511	Agency gauged data	18511	19000	20500	226590	318563	408712
1963	30330	Agency gauged data	30330	31000	34000	279957	360502	444788
1964	109350	Agency gauged data	109350	112000	115000	1258390	1682993	2050586
1965	28469	Agency gauged data	28469	29000	30900	266712	349096	470004
1966	47387	Agency gauged data	47387	52000	52900	512035	728207	845742
1967	12356	Agency gauged data	12356	16000	15400	355671	612473	842357
1968	48947	Agency gauged data	48947	50000	51200	458016	583897	660057
1969	44057	Agency gauged data	44057	44000	49600	594108	832292	1036206
1970	183687	Agency gauged data	183687	187000	166000	1493711	1931363	2384476
1971	82708	Agency gauged data	82708	87000	89000	839975	1048066	1183039
1972	22072	Agency gauged data	22072	23000	13800	183906	250300	315932
1973	141722	Agency gauged data	141722	140000	140000	1606373	2198184	2647407
1974	195818	Agency gauged data	195818	285000	193000	1503798	2198328	2821389
1975	233761	Agency gauged data	233761	431000	280000	1863404	2398313	2977958
1976	22833	Agency gauged data	22833	23000	23000	273884	387240	510162
1977	13065	Agency gauged data	13065	16000	16000	155110	213718	264479
1978	56295	Agency gauged data	56295	52000	52000	629272	813116	939673
1979	56926	Agency gauged data	56926	56000	56000	534506	828645	1010846
1980	20991	Agency gauged data	20991			221122	292772	358326
1981	126830	Agency gauged data	126830			1498602	2042580	2493682
1982	16392	Agency gauged data	16392			170968	247072	322039
1983	56031	Agency gauged data	56031			560768	849434	1006758
1984	60465	Agency gauged data	60465			694086	988009	1250873
1985	35853	Agency gauged data	35853			411704	506157	588647
1986	79093	Agency gauged data	79093			835739	1057357	1221839
1987	22417	Agency gauged data	22417			268039	393807	485468
1988	36950	Agency gauged data	36950			348122	451594	551119
1989	56578	Agency gauged data	56578			686478	952655	1204526
1990	104423	Agency gauged data	104423			1240814	1712316	2151386
1991	71056	Agency gauged data	71056			892062	1212878	1489836
1992	136877	Agency gauged data	136877			1386140	1893073	2375310
1993	183012	Agency gauged data	183012			1393576	1738247	2161929
1994	21115	Agency gauged data	21115			265512	377328	472279
1995	69775	Agency gauged data	69775			687716	994301	1284810
1996	141395	Agency gauged data	141395			1617109	2236660	2677040
1997	14076	Agency gauged data	14076			168546	241296	319376
1998	96431	Agency gauged data	96431			511826	636913	755173
1999	17994	Agency gauged data	17994			224268	316071	388677
2000	88858	Agency gauged data	88858			947053	1189260	1435654
2001	14910	Agency gauged data	14910			181726	285875	404516
2002	16240	Agency gauged data	16240			211453	317105	422487
2003	45567	Agency gauged data	45567			346508	449225	532005
2004	35225	Agency gauged data	35225			286287	356681	427210

Murray River at Tocumwal– peak flow data

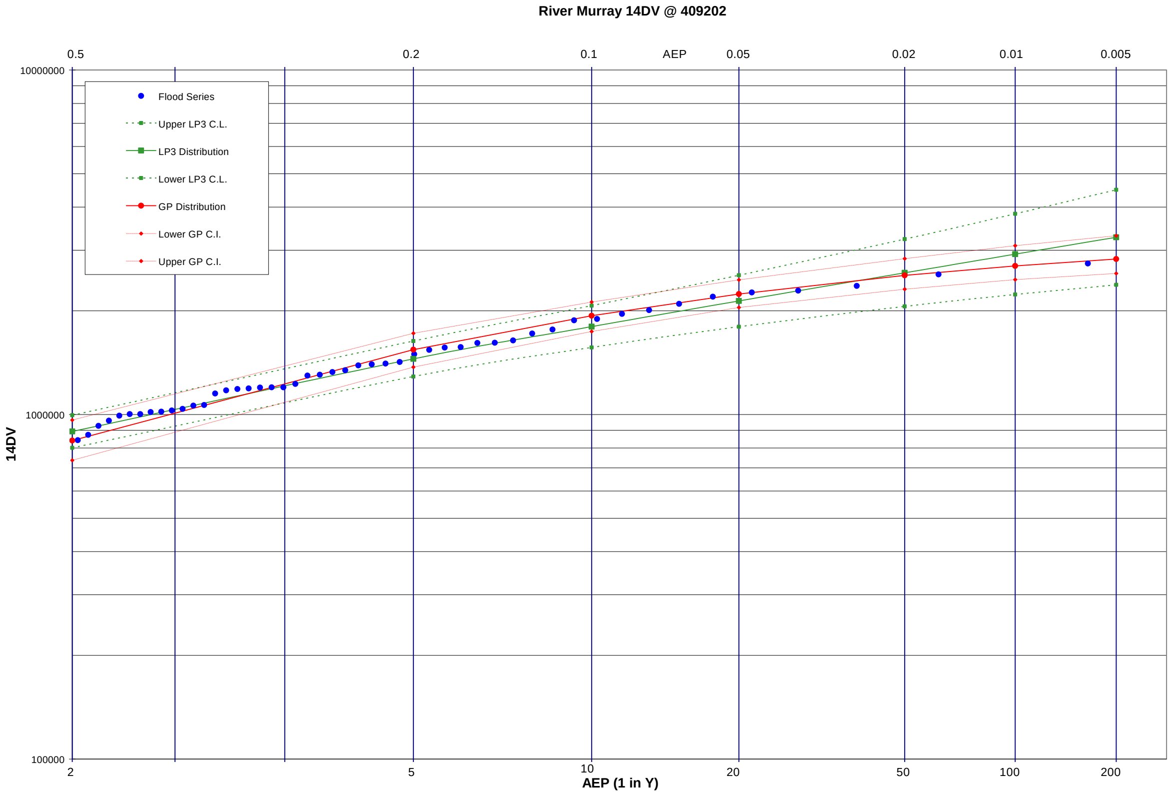
	Adopted peak flow data		Agency gauged data		
Year	Peak flow ML/d	Source	14 day vol (ML)	21 day vol (ML)	28 day vol (ML)
1908	31800	Agency gauged data	370400	511000	656900
1909	125000	Agency gauged data	1061600	1495100	1905300
1910	49800	Agency gauged data	692800	1018800	1267700
1911	41000	Agency gauged data	532600	780800	990300
1912	70000	Agency gauged data	763600	1002800	1225300
1913	30300	Agency gauged data	362400	487700	601200
1914	8350	Agency gauged data	99190	136850	173840
1915	66600	Agency gauged data	833600	1196500	1577700
1916	71900	Agency gauged data	908300	1304400	1613300
1917	191000	Agency gauged data	1789100	2260500	2603100
1918	65900	Agency gauged data	805100	1198500	1584200
1919	23300	Agency gauged data	292800	441200	567500
1920	76400	Agency gauged data	857000	1226200	1602200
1921	125000	Agency gauged data	1135200	1539800	1918500
1922	32400	Agency gauged data	404300	571400	719400
1923	73400	Agency gauged data	866800	1189800	1486900
1924	125000	Agency gauged data	1052500	1296700	1546700
1925	48700	Agency gauged data	540100	739800	931000
1926	64600	Agency gauged data	778400	1150000	1487300
1927	39400	Agency gauged data	455100	665100	849700
1928	56400	Agency gauged data	680800	959700	1204000
1929	34200	Agency gauged data	411300	562400	692800
1930	54300	Agency gauged data	635600	837000	1058800
1931	162000	Agency gauged data	1911000	2550500	3167900
1932	88900	Agency gauged data	1011900	1344200	1604900
1933	45900	Agency gauged data	573800	842200	1114900
1934	75200	Agency gauged data	930200	1404800	1812900
1935	55200	Agency gauged data	660000	926400	1165100
1936	82100	Agency gauged data	1012400	1388500	1710700
1937	16500	Agency gauged data	218700	324800	421200
1938	14600	Agency gauged data	165580	227250	298320
1939	107000	Agency gauged data	1377700	1958700	2418200
1940	13500	Agency gauged data	209900	349300	497100
1941	17900	Agency gauged data	203400	285770	375570
1942	66800	Agency gauged data	857000	1198800	1534500
1943	36100	Agency gauged data	482200	703800	943200
1944	9870	Agency gauged data	127010	185890	247670
1945	15900	Agency gauged data	164120	237850	301990
1946	95200	Agency gauged data	1164600	1563900	1944600
1947	55400	Agency gauged data	715000	1060900	1403000
1948	55400	Agency gauged data	649500	872500	1055000
1949	57600	Agency gauged data	737600	1038300	1297700
1950	42700	Agency gauged data	560800	793900	1003200
1951	75400	Agency gauged data	962700	1327500	1761000
1952	114000	Agency gauged data	1225000	1640600	1977300
1953	75200	Agency gauged data	993100	1420400	1797200
1954	38200	Agency gauged data	435000	641200	803600
1955	158000	Agency gauged data	1734400	2362400	2939400
1956	183000	Agency gauged data	2040000	2743200	3420000
1957	15900	Agency gauged data	209660	371660	558060
1958	113000	Agency gauged data	1167100	1614900	1992700
1959	24800	Agency gauged data	321800	434100	528600
1960	89600	Agency gauged data	1033200	1398300	1712900
1961	17900	Agency gauged data	155940	216050	349490
1962	17800	Agency gauged data	218000	308800	393900
1963	27100	Agency gauged data	273200	356400	442680
1964	95400	Agency gauged data	1147900	1568500	1936600
1965	25800	Agency gauged data	243500	316000	422770
1966	41100	Agency gauged data	503100	717900	832400
1967	15700	Agency gauged data	414500	646300	838600
1968	41100	Agency gauged data	426600	556800	648100
1969	46700	Agency gauged data	577900	826800	999700
1970	162000	Agency gauged data	1296600	1718300	2173500
1971	76100	Agency gauged data	803200	1016400	1192200
1972	19700	Agency gauged data	188410	256590	323600
1973	127000	Agency gauged data	1445000	2008000	2462600
1974	183000	Agency gauged data	1437500	2094800	2657500
1975	238000	Agency gauged data	1738800	2287700	2863300
1976	21900	Agency gauged data	271900	385800	527100
1977	12300	Agency gauged data	145930	202930	251140
1978	50100	Agency gauged data	600400	790300	927200
1979	47100	Agency gauged data	516300	789800	976300
1980	21100	Agency gauged data	221800	295880	365500
1981	115000	Agency gauged data	1372600	1893400	2332600
1982	15500	Agency gauged data	172400	248500	324000
1983	53500	Agency gauged data	569900	842000	1007600
1984	60100	Agency gauged data	704600	1002200	1270100
1985	35900	Agency gauged data	411700	511600	595800
1986	72500	Agency gauged data	836900	1065300	1232100
1987	21800	Agency gauged data	265100	389900	483500
1988	34300	Agency gauged data	353400	457200	554700
1989	57800	Agency gauged data	713100	991600	1252600
1990	91900	Agency gauged data	1154700	1612200	2040900
1991	68400	Agency gauged data	861100	1174900	1441900
1992	132000	Agency gauged data	1362600	1875600	2360700
1993	176000	Agency gauged data	1397700	1765000	2190700
1994	21400	Agency gauged data	259800	368700	458600
1995	66800	Agency gauged data	704700	1027300	1327600
1996	140000	Agency gauged data	1587000	2197500	2651900
1997	13400	Agency gauged data	158020	234920	309820
1998	70700	Agency gauged data	487000	615800	736000
1999	17600	Agency gauged data	218800	309300	379990
2000	88400	Agency gauged data	936300	1185400	1430000
2001	14500	Agency gauged data	182200	288400	413700
2002	15800	Agency gauged data	208700	312300	415900
2003	39400	Agency gauged data	348700	454100	541200
2004	32000	Agency gauged data	287200	361900	434500
2005	28400	Agency gauged data	343200	488300	636800



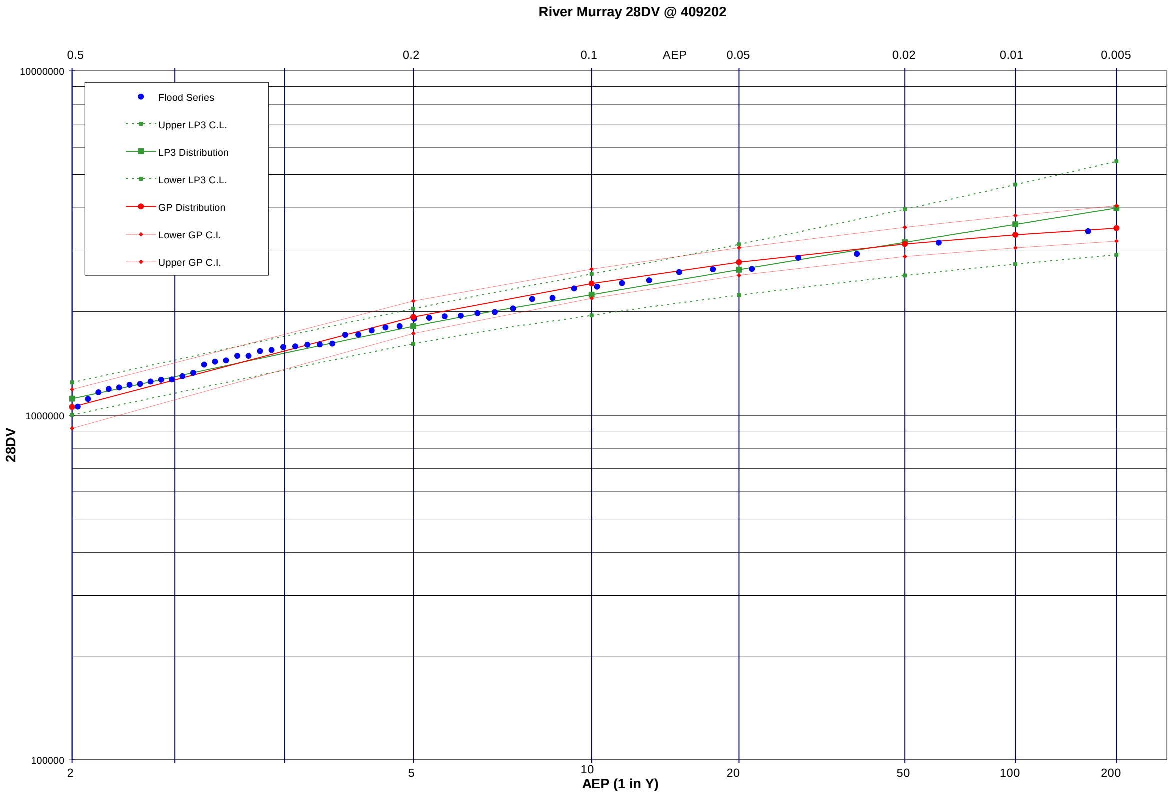
Yarrawonga 21 Day Volume Flood Frequency Analysis



Yarrawonga 28 Day Volume Flood Frequency Analysis



Tocumwal 21 Day Volume Flood Frequency Analysis



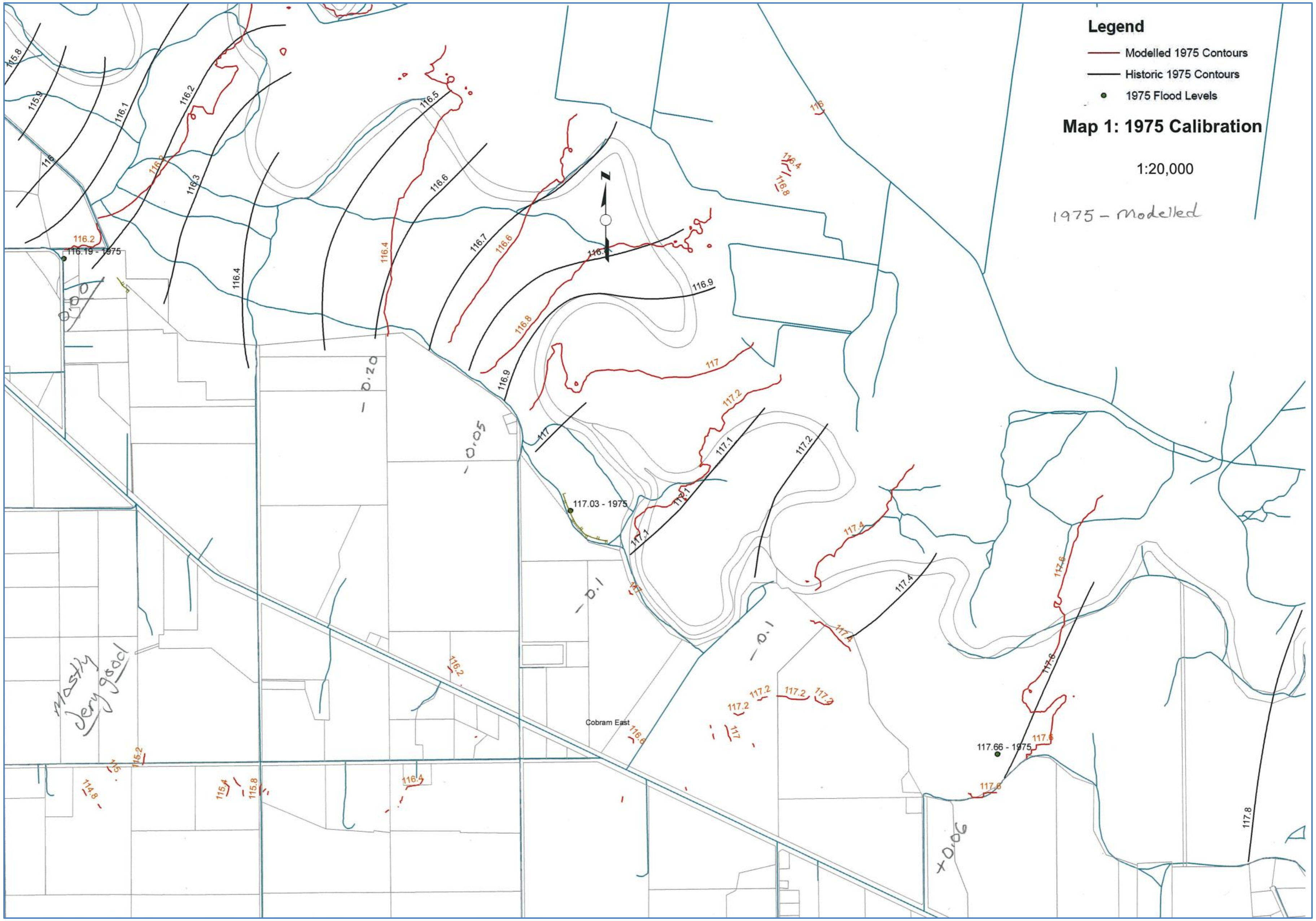
Tocumwal 28 Day Volume Flood Frequency Analysis

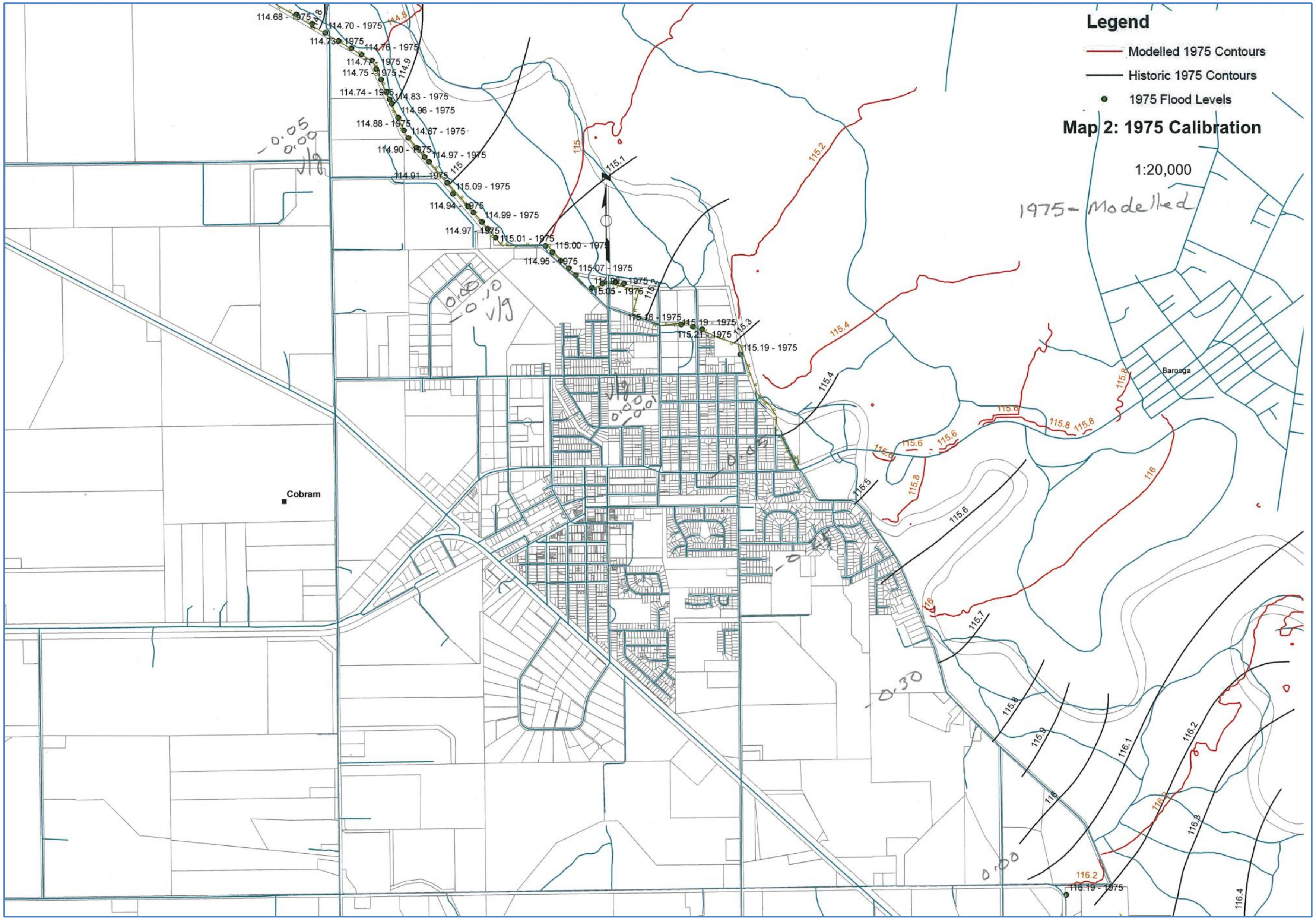
Scaled design hydrographs

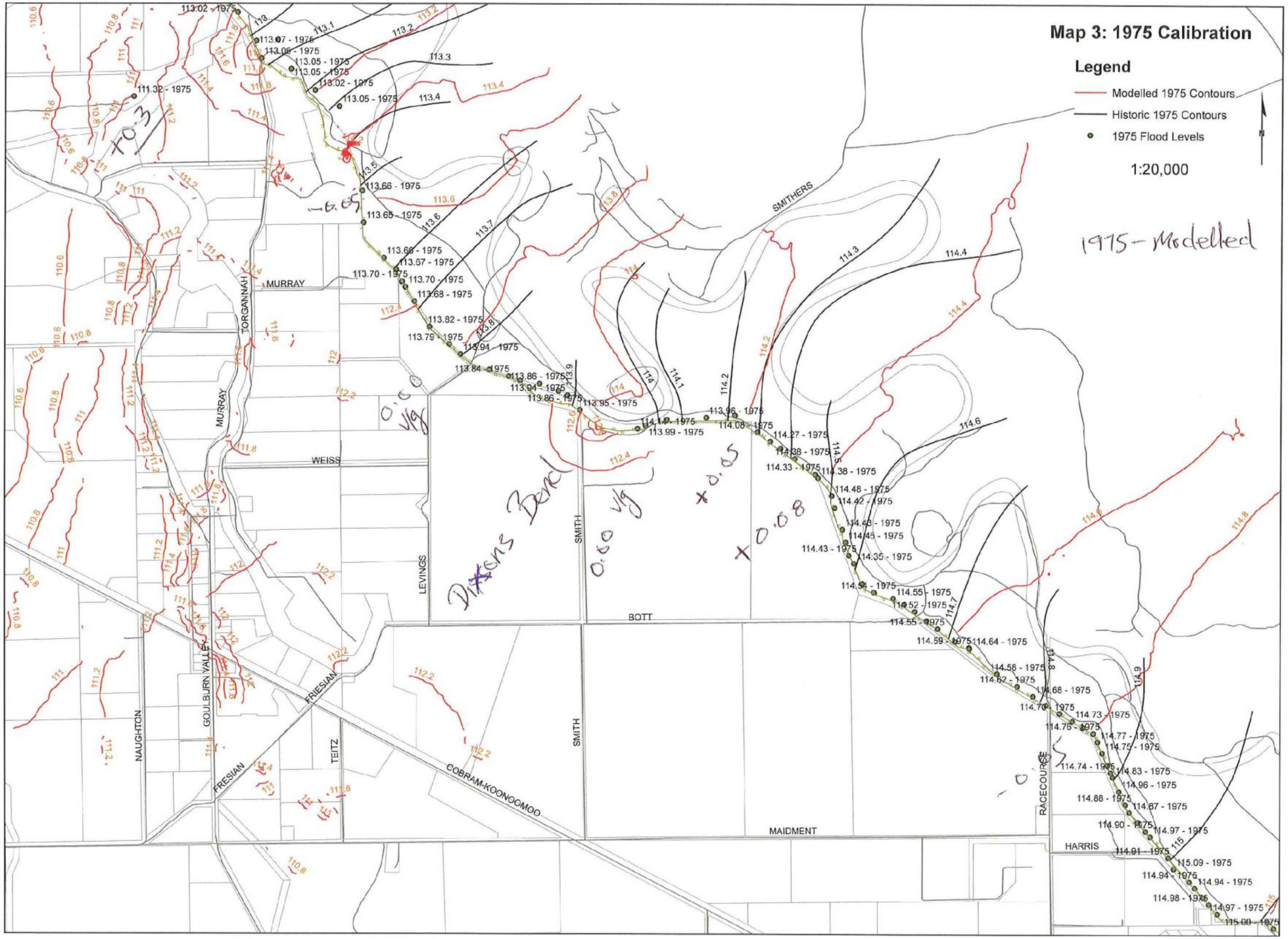
Day	Historical 1952 hydrograph ML/d	Design flood hydrograph 10 year ML/d	20 year ML/d	Historical 1958 hydrograph ML/d	Design flood hydrograph 50 year ML/d	100 year ML/d	200 year ML/d
1	84529	111859	141929	82535	157620	181789	205957
2	104958	138893	176230	127339	243184	280472	317761
3	123063	162851	206628	121083	231237	266693	302150
4	137742	182277	231276	122045	233073	268811	304549
5	140556	186000	236000	157090	299999	345999	391999
6	129424	171269	217309	135797	259336	299101	338866
7	120739	159775	202726	113475	216706	249935	283163
8	111931	148120	187937	97420	186047	214574	243101
9	99209	131285	166576	88383	168787	194668	220548
10	85508	113154	143572	84475	161325	186062	210798
11	89055	117848	149528	81843	156298	180264	204229
12	76455	101175	128372	72749	138931	160234	181536
13	74743	98909	125497	62938	120195	138625	157055
14	67403	89196	113173	56971	108799	125482	142164

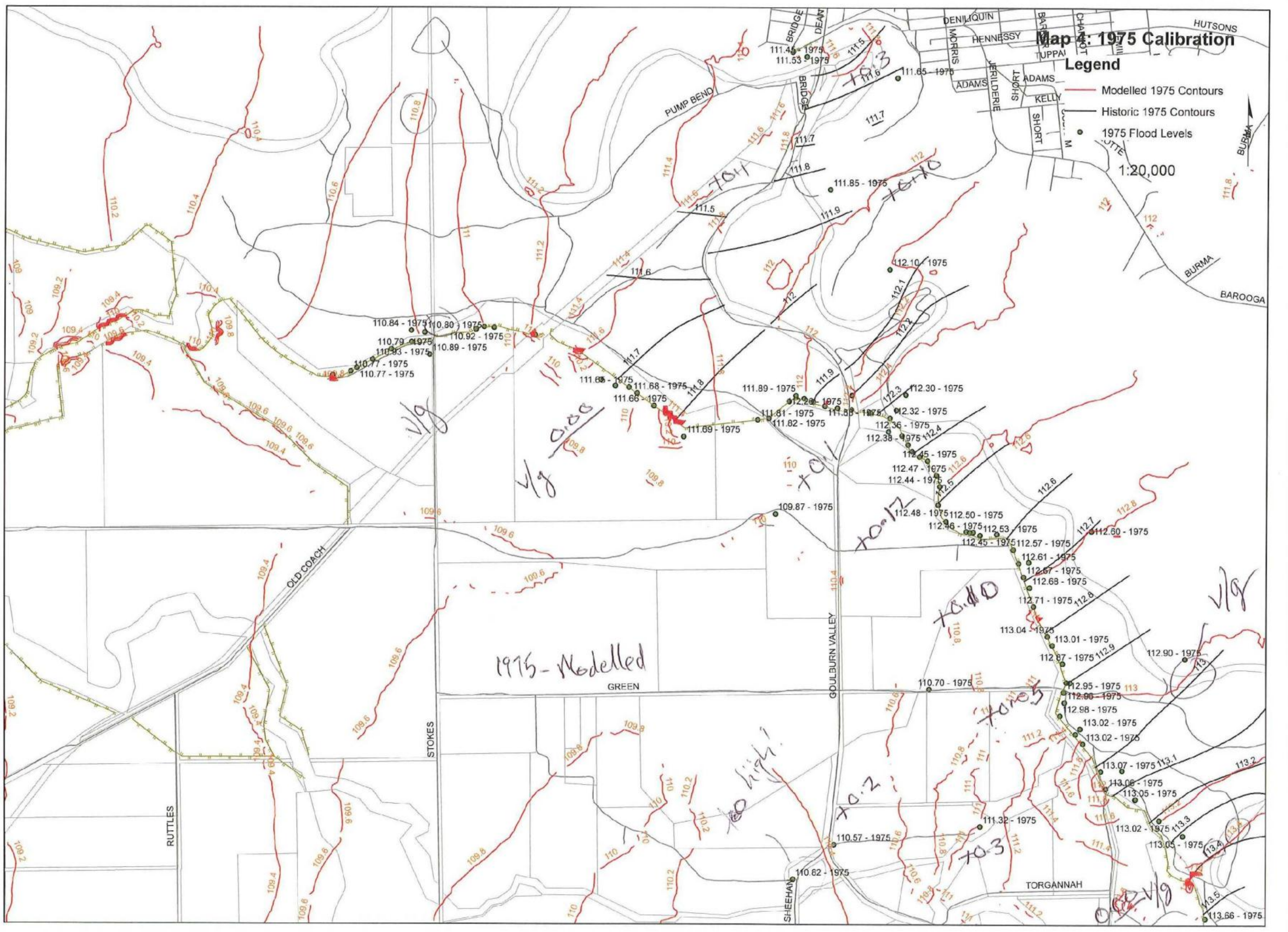
APPENDIX C 1975 MODELLED AND OBSERVED FLOOD LEVEL

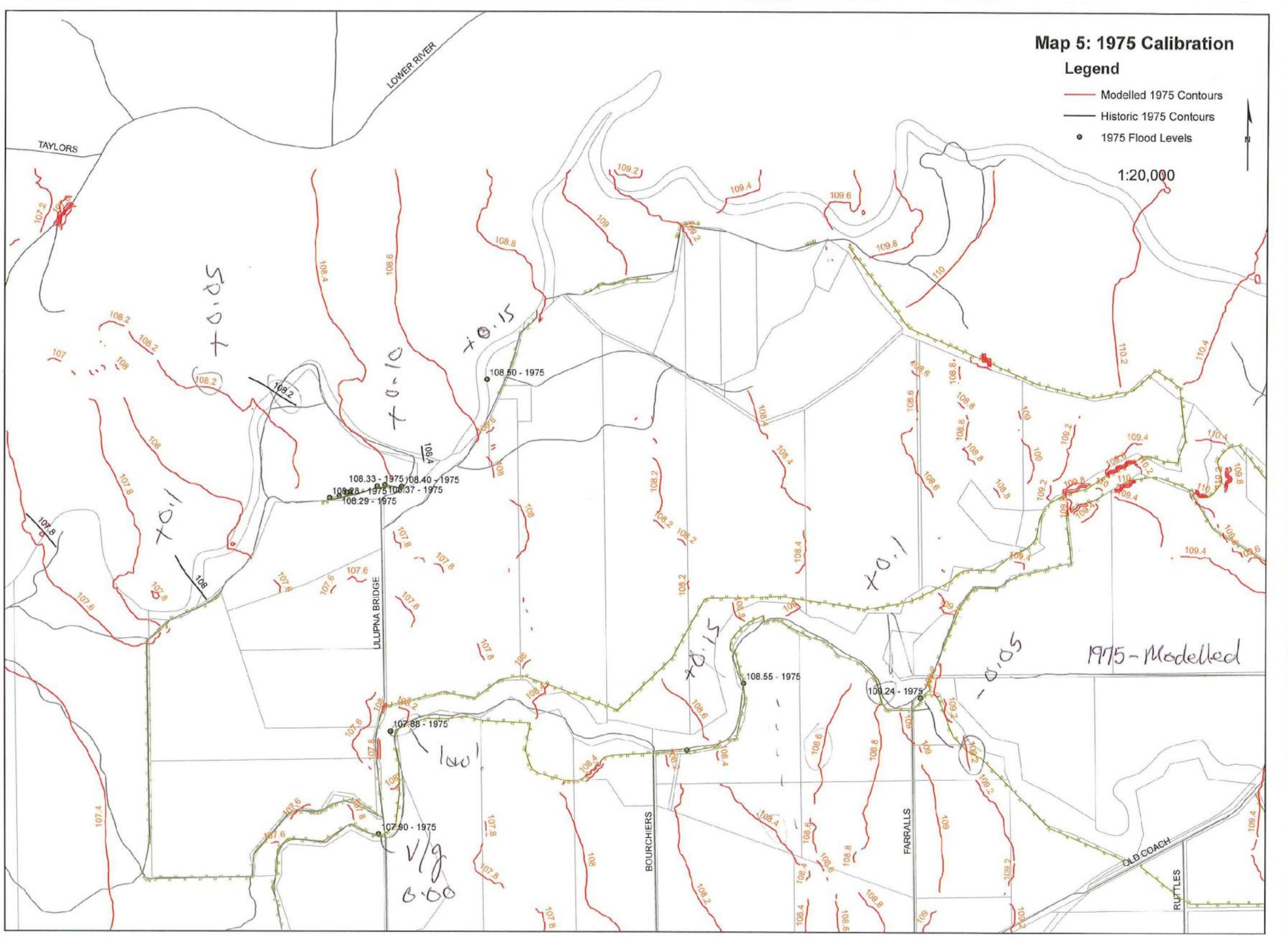
This appendix contains a comparison of modelled and observed October 1975 flood levels undertaken by Guy Tierney Goulburn Broken Catchment Management Authority. The annotations on the maps were made by Guy Tierney.

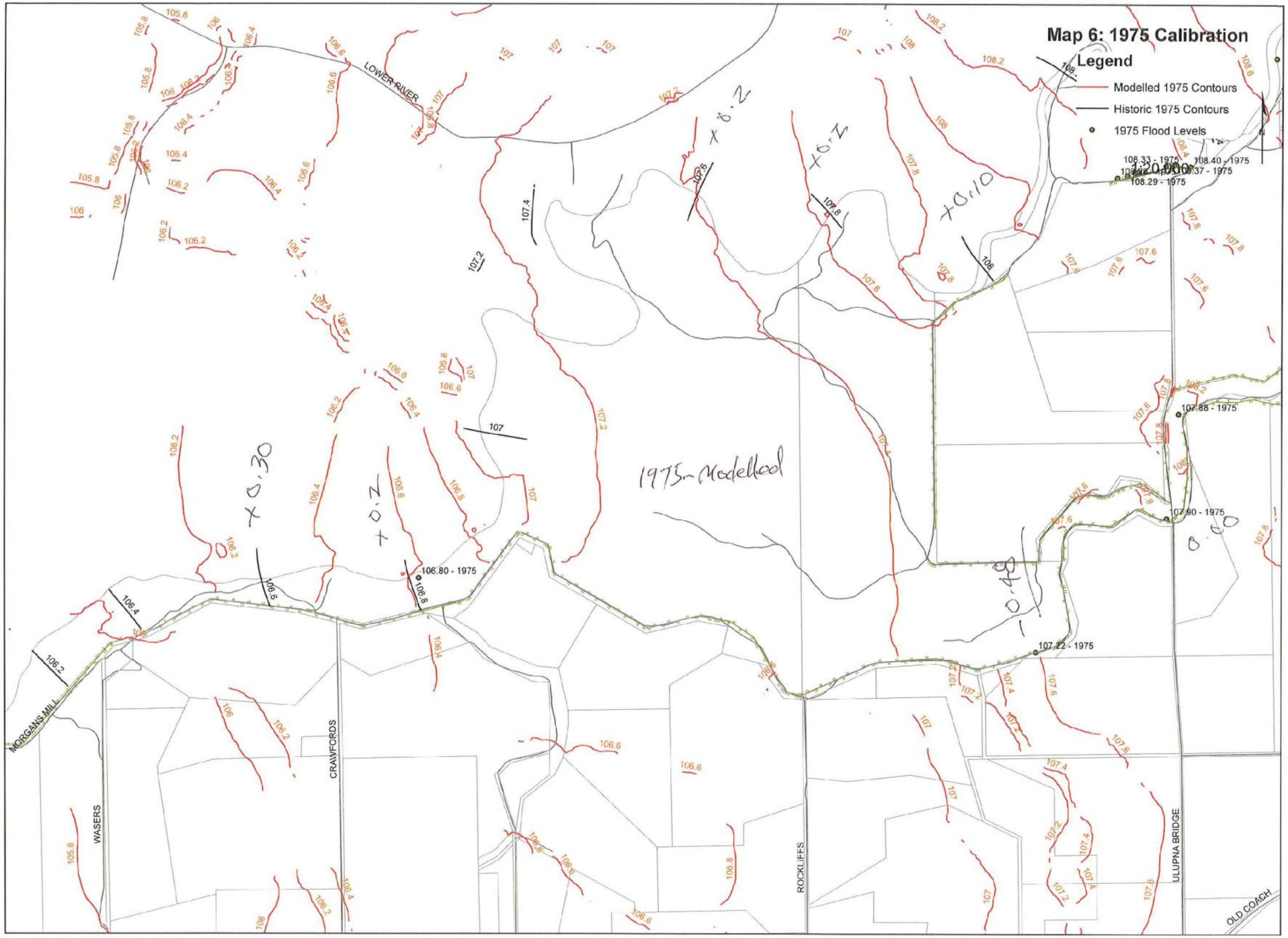

Legend
Modelled 1975 Contours
Historic 1975 Contours
1975 Flood Levels
Map 1: 1975 Calibration
1:20,000
1975 – Modelled
116.19 - 1975
117.03 - 1975
117.66 - 1975
Cobram East
Mostly Very good


Legend
Modelled 1975 Contours
Historic 1975 Contours
1975 Flood Levels
Map 2: 1975 Calibration
1:20,000
1975 - Modelled
Cobram
Barooga
114.68 - 1975
114.70 - 1975
114.73 - 1975
114.76 - 1975
114.77 - 1975
114.75 - 1975
114.74 - 1975
114.83 - 1975
114.96 - 1975
114.88 - 1975
114.87 - 1975
114.90 - 1975
114.97 - 1975
114.91 - 1975
115.09 - 1975
114.94 - 1975
114.99 - 1975
114.97 - 1975
115.01 - 1975
115.00 - 1975
114.95 - 1975
115.07 - 1975
115.05 - 1975
115.16 - 1975
115.19 - 1975
115.21 - 1975
115.19 - 1975
116.19 - 1975
-0.05
0.00
-0.10
-0.30
0.00


Map 3: 1975 Calibration
Legend
Modelled 1975 Contours
Historic 1975 Contours
1975 Flood Levels
1:20,000
N
1975 – Modelled
Dixons Bend
SMITHERS
MURRAY
TORGANNAH
WEISS
LEVINGS
SMITH
BOTT
MAIDMENT
RACECOURSE
HARRIS
NAUGHTON
GOULBURN VALLEY
FRIESIAN
TEITZ
COBRAM-KOONOOMOO
+0.3
-0.05
0.00 v/g
+0.05
+0.08


Map 4: 1975 Calibration
Legend
Modelled 1975 Contours
Historic 1975 Contours
1975 Flood Levels
1:20,000
1975 - Modelled
DENILIQUIN
GOULBURN VALLEY
OLD COACH
GREEN
STOKES
RUTTLES
SHEEHAN
TORGANNAH
PUMP BEND
BAROOGA
BURMA
HUTSONS

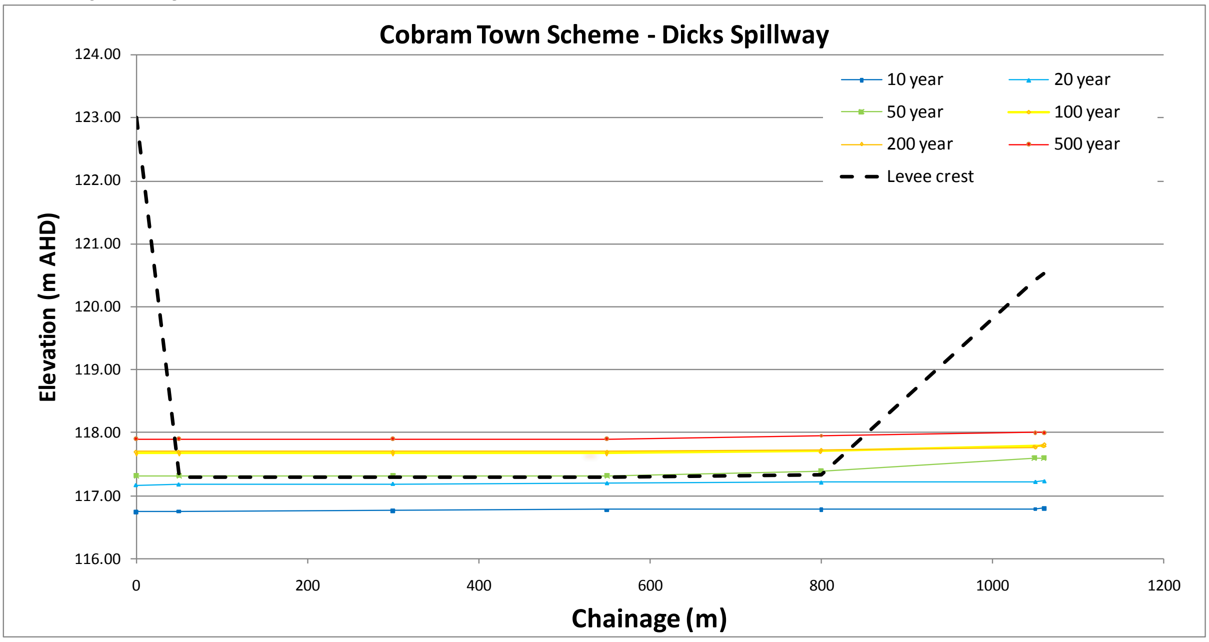

Map 5: 1975 Calibration
Legend
Modelled 1975 Contours
Historic 1975 Contours
1975 Flood Levels
1:20,000
N
LOWER RIVER
TAYLORS
ULUPNA BRIDGE
BOURCHIERS
FARRALLS
OLD COACH
RUTTLES
108.50 - 1975
108.33 - 1975
108.40 - 1975
108.37 - 1975
108.29 - 1975
108.55 - 1975
109.24 - 1975
107.88 - 1975
107.90 - 1975
+0.05
+0.10
+0.15
+0.1
+0.15
+0.1
-0.05
1975 - Modelled


Map 6: 1975 Calibration
Legend
Modelled 1975 Contours
Historic 1975 Contours
1975 Flood Levels
1:20,000
LOWER RIVER
MORGANS MILL
WASERS
CRAWFORDS
ROCKLIFFS
ULUPNA BRIDGE
OLD COACH
106.80 - 1975
107.22 - 1975
107.88 - 1975
107.90 - 1975
108.29 - 1975
108.33 - 1975
108.40 - 1975
1975 - Modelled
+0.30
+0.2
+0.10
-0.48

APPENDIX D FLOOD LEVEL AND LEVEE CREST PROFILES

Cobram Town Scheme

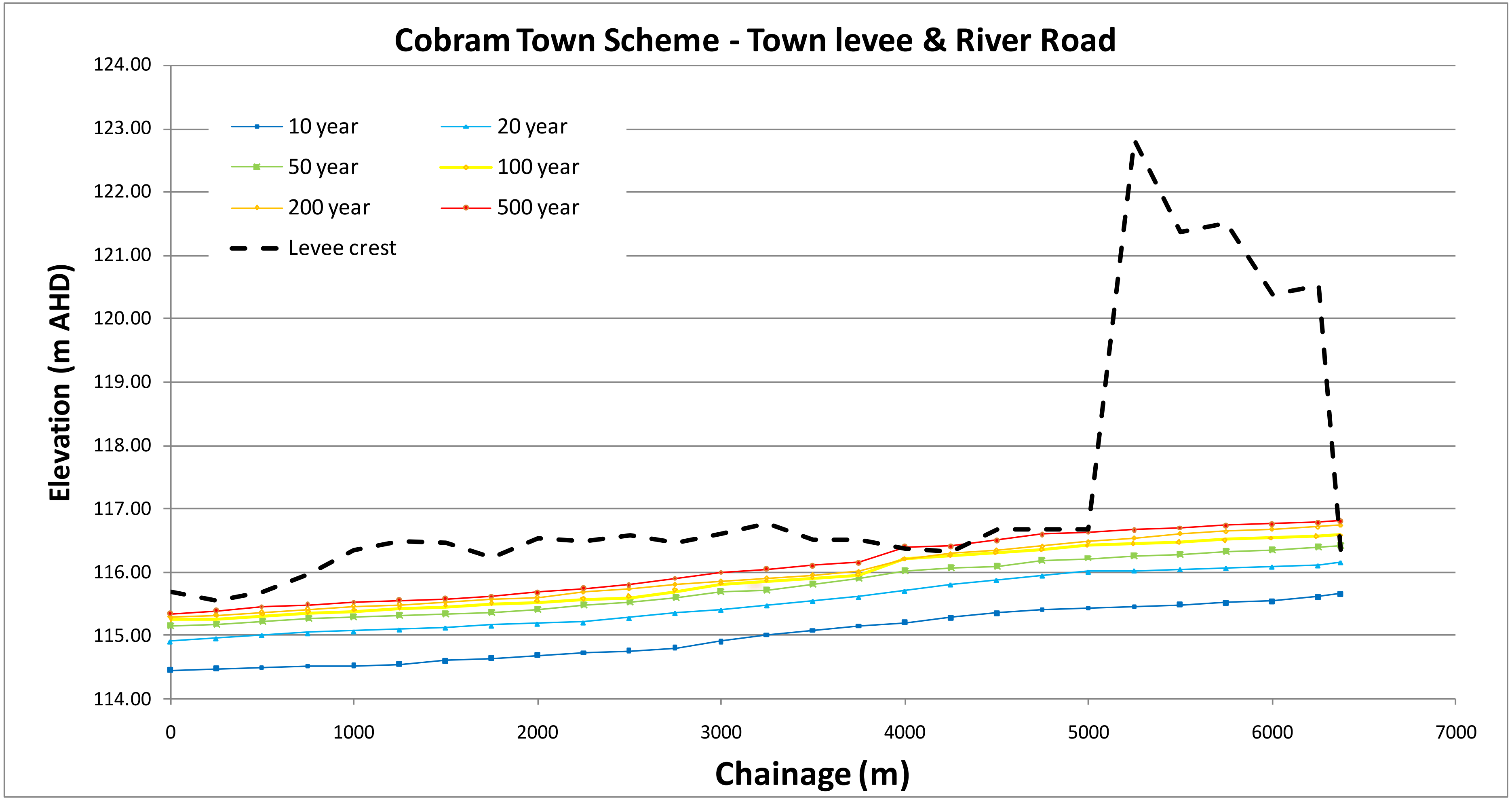
Dicks spillway

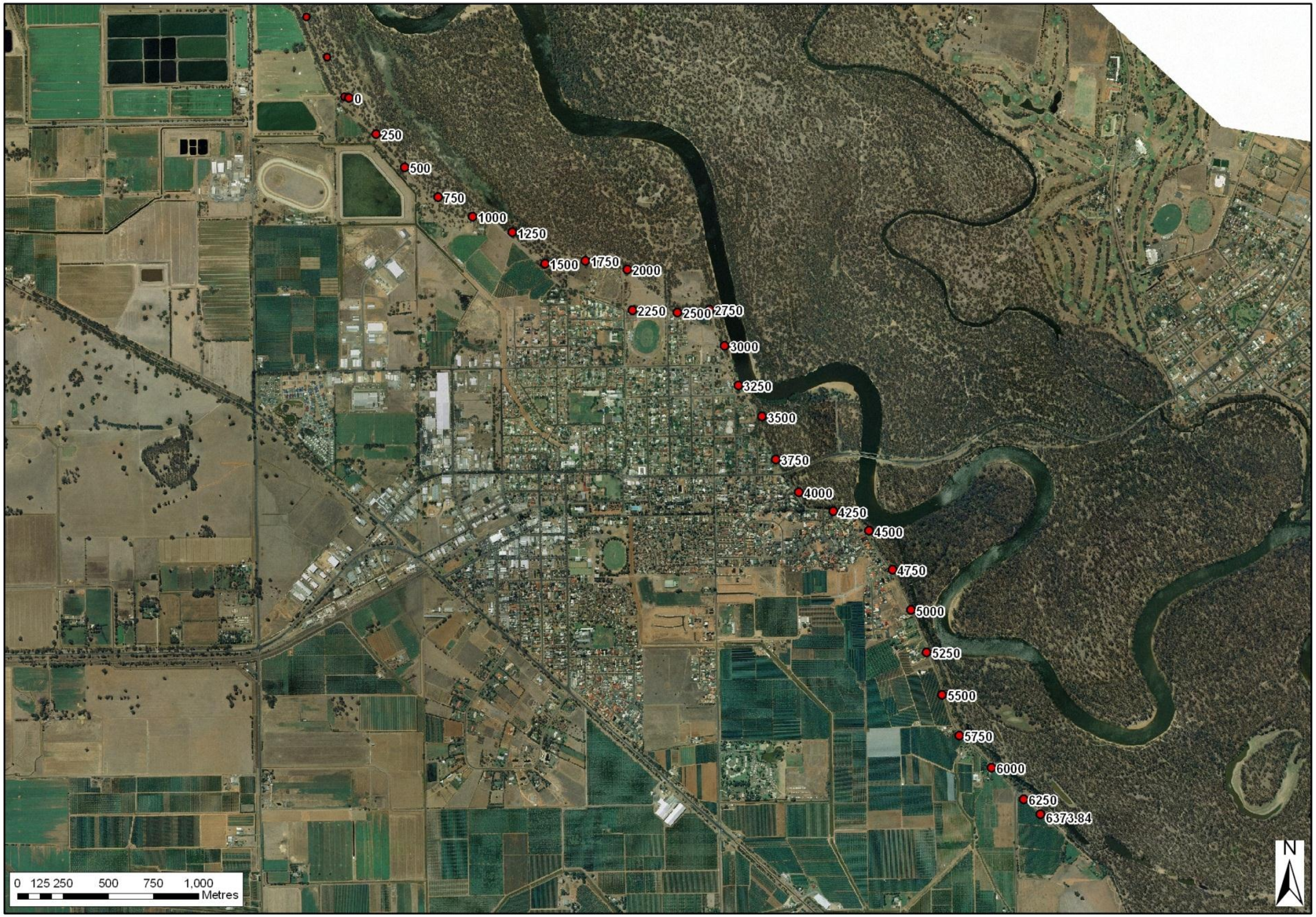


Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	116.74	117.17	117.32	117.67	117.70	117.90	123.00
50	116.75	117.18	117.32	117.67	117.70	117.90	117.30
300	116.76	117.19	117.32	117.67	117.70	117.90	117.30
550	116.77	117.20	117.32	117.67	117.70	117.90	117.30
800	116.78	117.21	117.40	117.70	117.73	117.95	117.33
1050	116.79	117.22	117.60	117.79	117.77	118.00	120.41
1060	116.80	117.23	117.60	117.79	117.82	118.00	120.53

Town Levee and River Road

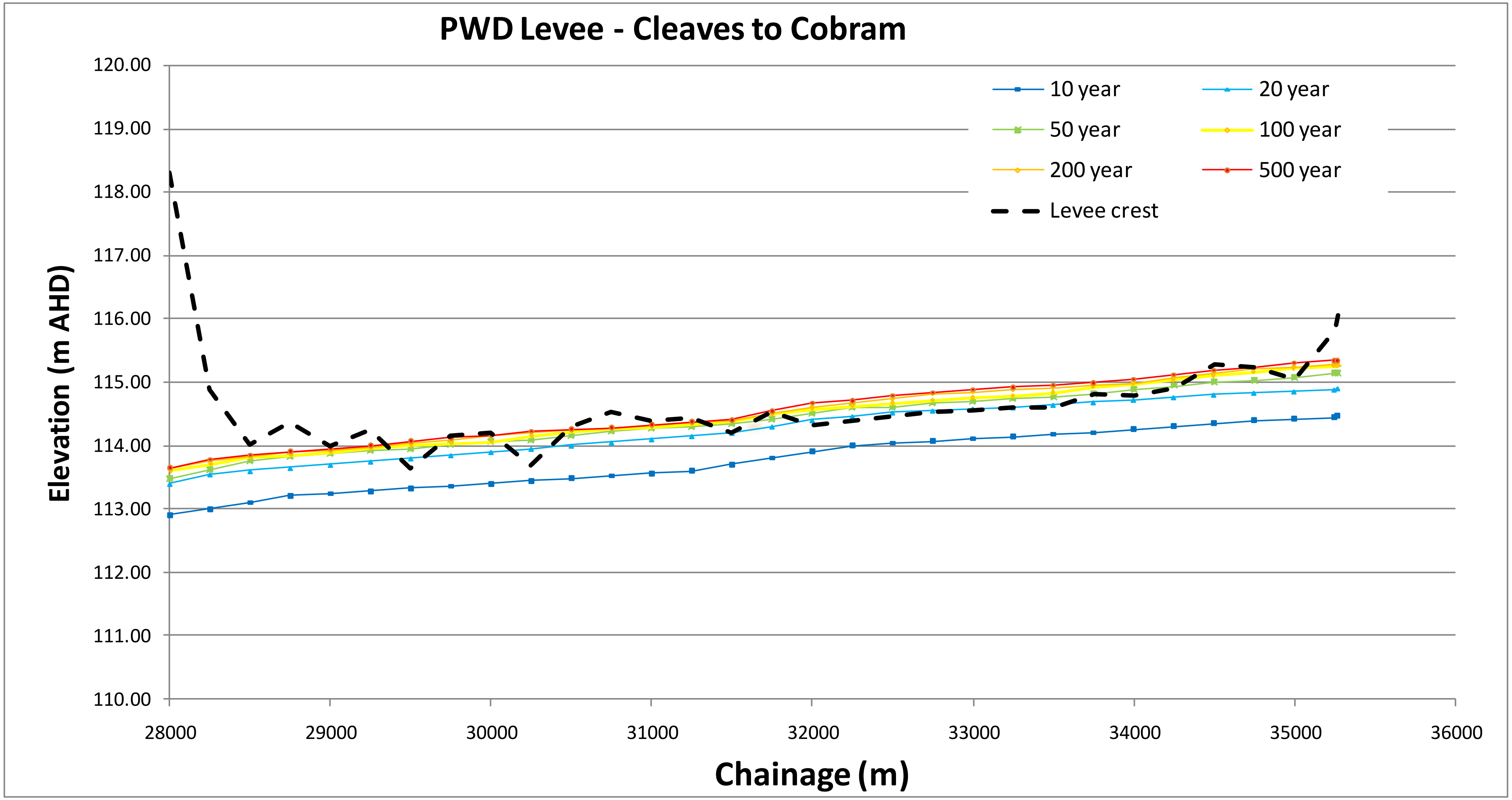
Chainage (m)	10 year	20 year	50 year	■ 100 year	■ 200 year	■ 500 year	■ Levee crest
0	114.45	114.90	115.15	115.25	115.29	115.34	115.69
250	114.47	114.95	115.18	115.29	115.32	115.40	115.54
500	114.49	115.00	115.22	115.32	115.37	115.45	115.69
750	114.51	115.03	115.27	115.35	115.42	115.49	115.96
1000	114.53	115.06	115.28	115.38	115.46	115.52	116.33
1250	114.55	115.09	115.31	115.42	115.49	115.56	116.49
1500	114.60	115.12	115.34	115.45	115.52	115.58	116.46
1750	114.64	115.15	115.37	115.49	115.57	115.62	116.21
2000	114.68	115.18	115.40	115.53	115.60	115.68	116.53
2250	114.72	115.20	115.47	115.56	115.68	115.74	116.47
2500	114.76	115.27	115.52	115.60	115.73	115.80	116.57
2750	114.80	115.35	115.60	115.70	115.80	115.90	116.45
3000	114.90	115.40	115.68	115.80	115.85	115.99	116.59
3250	115.00	115.47	115.72	115.85	115.90	116.05	116.76
3500	115.07	115.54	115.80	115.90	115.95	116.10	116.50
3750	115.14	115.60	115.91	115.95	116.00	116.15	116.50
4000	115.20	115.70	116.03	116.20	116.20	116.40	116.37
4250	115.27	115.80	116.07	116.25	116.28	116.41	116.32
4500	115.34	115.87	116.10	116.30	116.34	116.50	116.66
4750	115.40	115.94	116.18	116.35	116.40	116.59	116.67
5000	115.43	116.00	116.22	116.41	116.47	116.63	116.67
5250	115.45	116.02	116.26	116.44	116.53	116.67	122.82
5500	115.48	116.04	116.29	116.47	116.60	116.70	121.37
5750	115.50	116.06	116.32	116.50	116.63	116.73	121.51
6000	115.53	116.08	116.36	116.53	116.67	116.76	120.37
6250	115.60	116.10	116.40	116.56	116.70	116.79	120.52

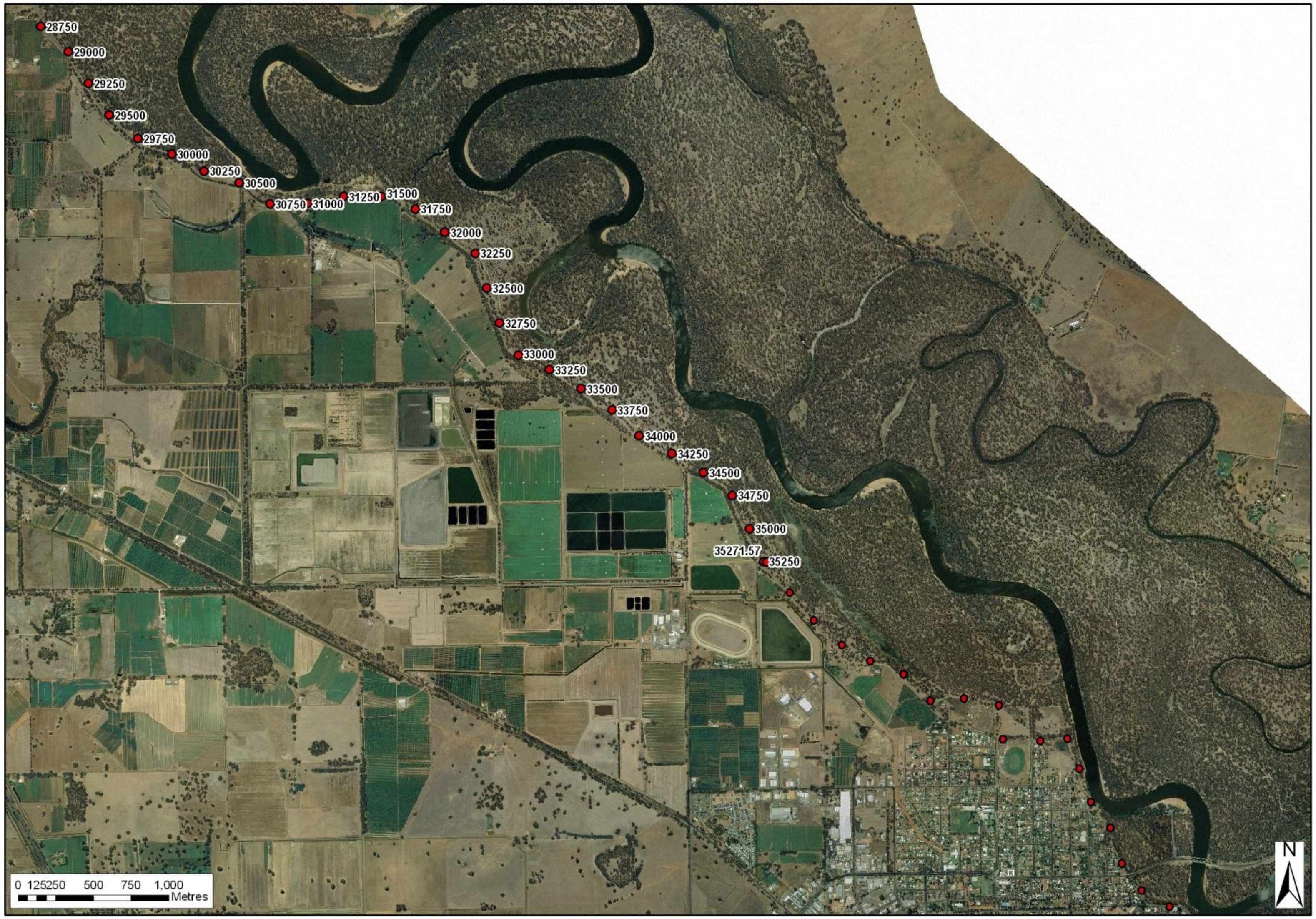

Cobram Town Scheme - Town levee & River Road
Elevation (m AHD)
Chainage (m)
124.00
123.00
122.00
121.00
120.00
119.00
118.00
117.00
116.00
115.00
114.00
0
1000
2000
3000
4000
5000
6000
7000
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


0
250
500
750
1000
1250
1500
1750
2000
2250
2500
2750
3000
3250
3500
3750
4000
4250
4500
4750
5000
5250
5500
5750
6000
6250
6373.84
0 125 250 500 750 1,000
Metres
N

PWD Levee
Cobram to Cleaves

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
28000	112.90	113.40	113.48	113.61	113.63	113.64	118.30
28250	113.00	113.55	113.63	113.70	113.75	113.77	114.88
28500	113.10	113.60	113.77	113.81	113.83	113.84	114.01
28750	113.20	113.65	113.83	113.85	113.88	113.90	114.36
29000	113.24	113.70	113.88	113.90	113.92	113.95	114.00
29250	113.28	113.75	113.92	113.95	113.97	114.00	114.24
29500	113.32	113.80	113.96	114.00	114.04	114.06	113.64
29750	113.36	113.85	114.02	114.03	114.09	114.12	114.17
30000	113.40	113.90	114.06	114.05	114.15	114.16	114.20
30250	113.44	113.95	114.10	114.15	114.21	114.22	113.68
30500	113.48	114.00	114.16	114.23	114.24	114.25	114.29
30750	113.52	114.05	114.23	114.27	114.28	114.28	114.53
31000	113.56	114.10	114.28	114.30	114.32	114.32	114.38
31250	113.60	114.15	114.31	114.34	114.36	114.37	114.44
31500	113.70	114.20	114.34	114.38	114.39	114.40	114.20
31750	113.80	114.30	114.43	114.50	114.50	114.54	114.53
32000	113.90	114.40	114.52	114.58	114.61	114.67	114.31
32250	114.00	114.46	114.60	114.62	114.67	114.70	114.38
32500	114.04	114.52	114.62	114.67	114.74	114.78	114.47
32750	114.07	114.55	114.67	114.71	114.82	114.83	114.53
33000	114.11	114.58	114.71	114.75	114.85	114.88	114.56
33250	114.14	114.60	114.74	114.78	114.88	114.91	114.61
33500	114.18	114.64	114.78	114.83	114.92	114.95	114.61
33750	114.20	114.68	114.83	114.92	114.96	114.98	114.81
34000	114.25	114.72	114.89	114.98	114.99	115.04	114.78
34250	114.30	114.76	114.95	115.03	115.07	115.11	114.90
34500	114.35	114.80	115.00	115.10	115.14	115.18	115.27
34750	114.40	114.83	115.04	115.16	115.21	115.23	115.23
35000	114.42	114.85	115.09	115.22	115.24	115.30	115.04
35250	114.45	114.88	115.14	115.25	115.29	115.34	115.82
35272	114.47	114.90	115.15	115.25	115.29	115.34	116.05

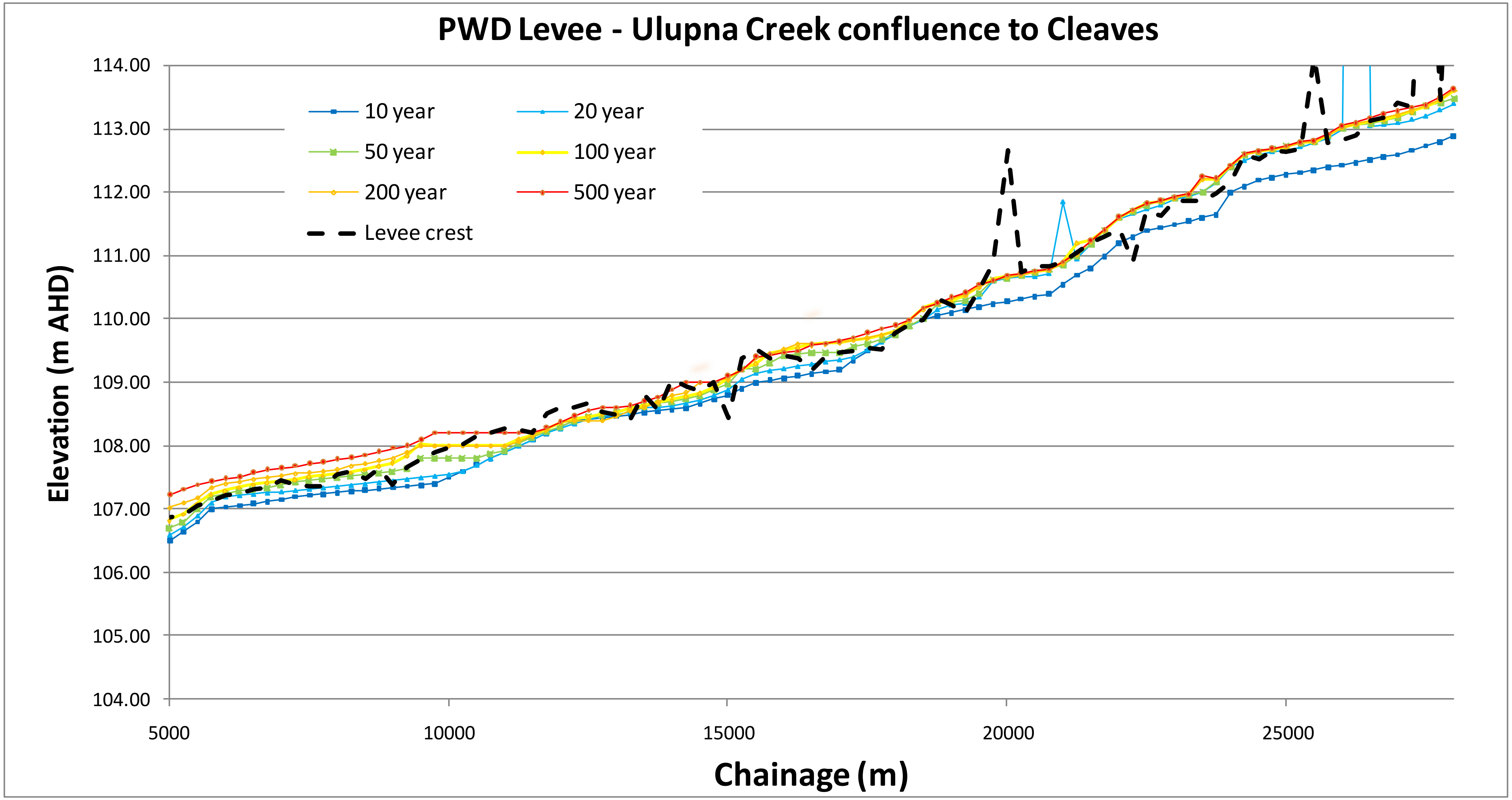

PWD Levee - Cleaves to Cobram
Elevation (m AHD)
120.00
119.00
118.00
117.00
116.00
115.00
114.00
113.00
112.00
111.00
110.00
28000
29000
30000
31000
32000
33000
34000
35000
36000
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest

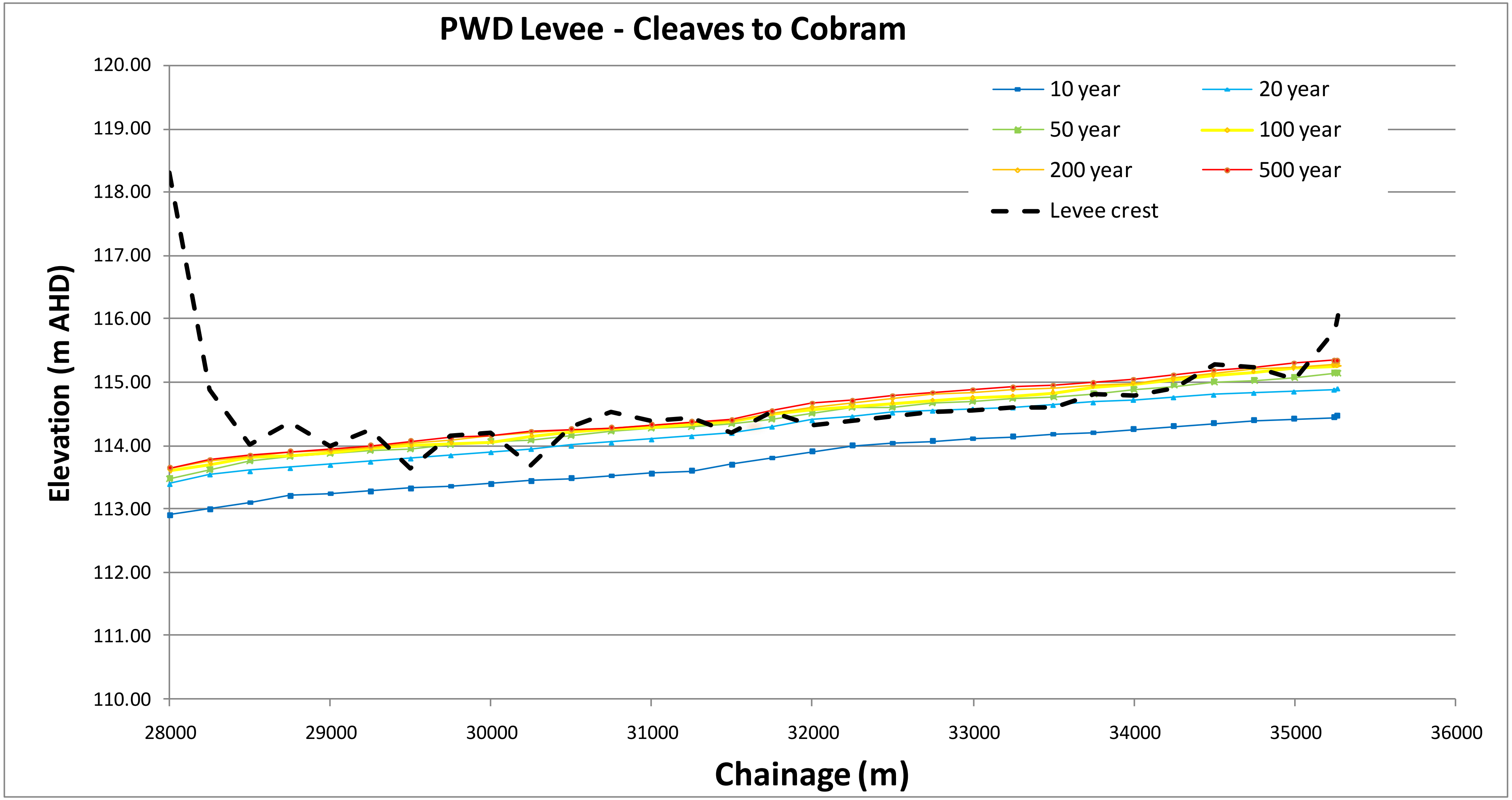

28750
29000
29250
29500
29750
30000
30250
30500
30750
31000
31250
31500
31750
32000
32250
32500
32750
33000
33250
33500
33750
34000
34250
34500
34750
35000
35271.57
35250
0 125250 500 750 1,000
Metres
N

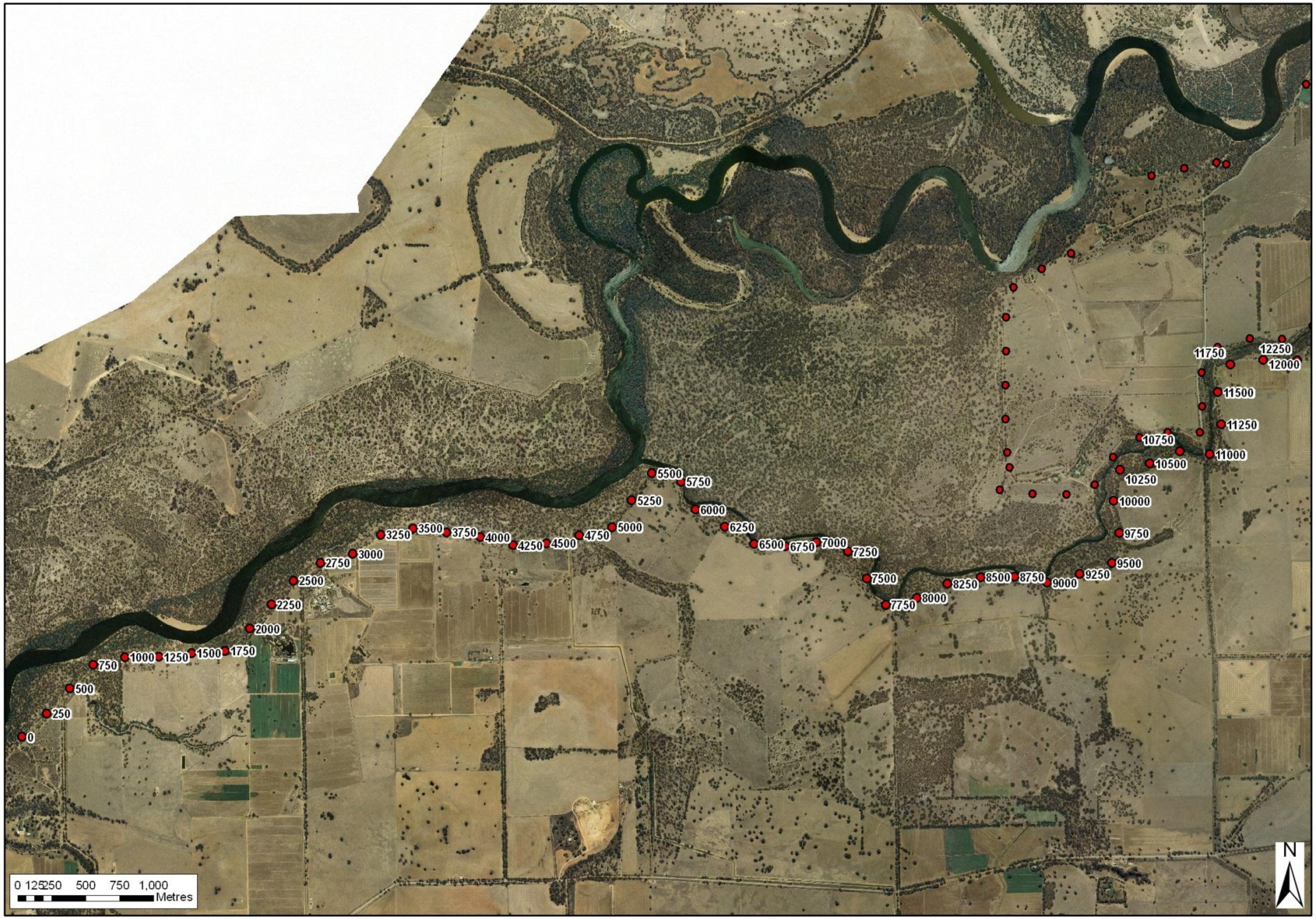
Cleaves to Ulupna Creek confluence

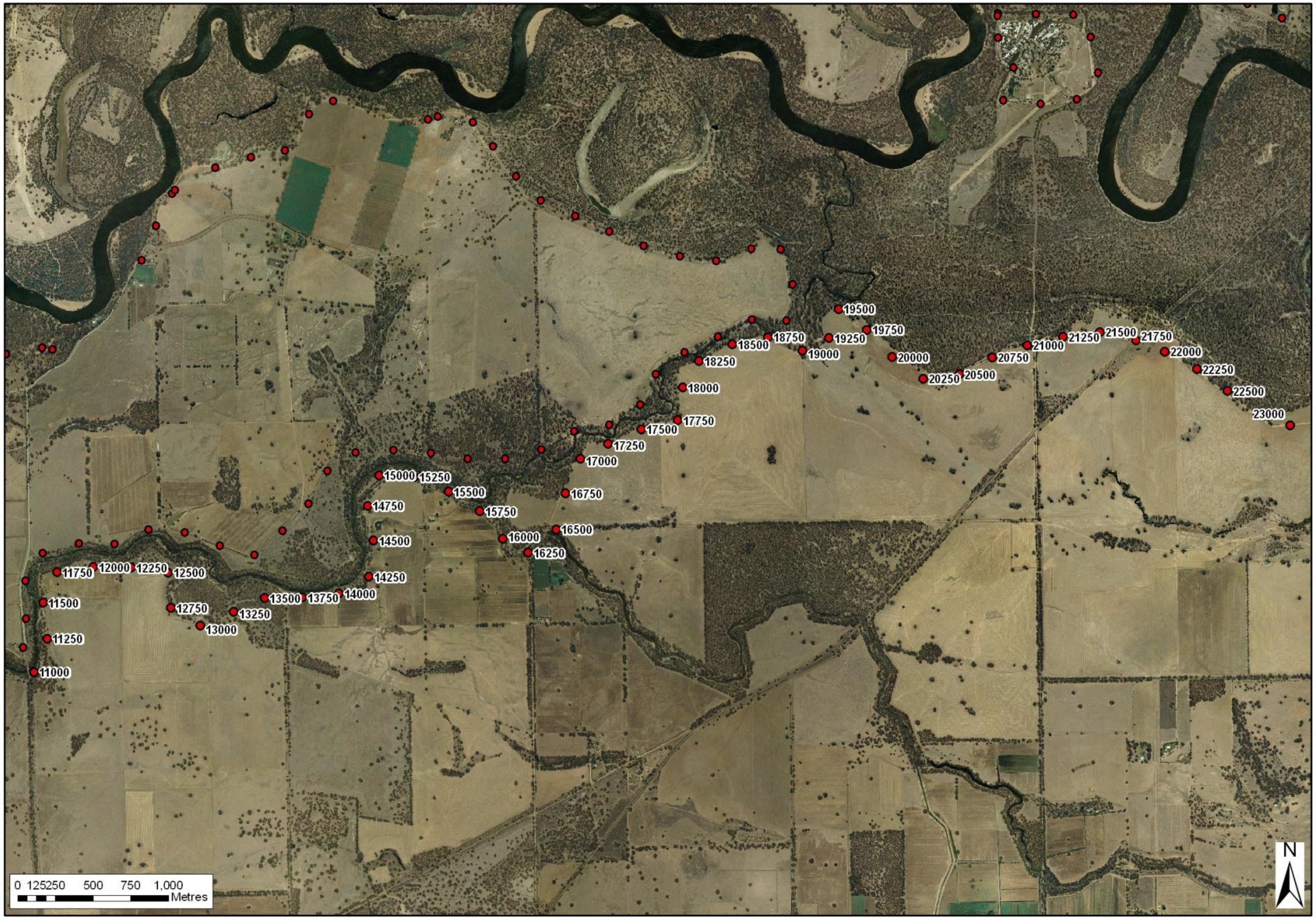
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
4250	106.40	106.43	106.45	106.64	106.79	107.03	106.67
4500	106.43	106.48	106.52	106.71	106.90	107.10	106.75
4750	106.46	106.54	106.61	106.81	106.96	107.18	106.82
5000	106.50	106.60	106.70	106.82	107.02	107.22	106.87
5250	106.65	106.73	106.80	106.92	107.10	107.31	106.94
5500	106.80	106.90	107.00	107.08	107.18	107.38	107.07
5750	107.00	107.10	107.20	107.23	107.34	107.43	107.15
6000	107.03	107.20	107.23	107.29	107.40	107.48	107.24
6250	107.06	107.22	107.28	107.34	107.43	107.51	107.26
6500	107.09	107.24	107.31	107.39	107.47	107.57	107.31
6750	107.12	107.26	107.34	107.42	107.50	107.62	107.36
7000	107.15	107.28	107.38	107.45	107.53	107.65	107.47
7250	107.20	107.30	107.42	107.47	107.56	107.68	107.40
7500	107.22	107.32	107.45	107.51	107.58	107.72	107.38
7750	107.24	107.34	107.47	107.53	107.60	107.75	107.38
8000	107.26	107.36	107.50	107.56	107.63	107.78	107.55
8250	107.28	107.38	107.52	107.58	107.68	107.81	107.60
8500	107.30	107.40	107.54	107.62	107.72	107.85	107.48
8750	107.32	107.43	107.56	107.68	107.77	107.90	107.65
9000	107.34	107.45	107.58	107.73	107.80	107.96	107.39
9250	107.36	107.48	107.63	107.84	107.90	108.00	107.68
9500	107.38	107.50	107.80	108.02	108.00	108.10	107.79
9750	107.40	107.53	107.80	108.00	108.00	108.20	107.92
10000	107.50	107.55	107.80	108.00	108.00	108.20	107.98
10250	107.60	107.60	107.80	108.00	108.00	108.20	108.03
10500	107.70	107.70	107.80	108.00	108.00	108.20	108.17
10750	107.80	107.80	107.87	108.00	108.00	108.20	108.21
11000	107.90	107.90	107.92	108.00	108.00	108.20	108.28
11250	108.00	108.00	108.03	108.10	108.07	108.20	108.27
11500	108.10	108.10	108.12	108.17	108.15	108.21	108.22
11750	108.20	108.20	108.23	108.25	108.27	108.29	108.53
12000	108.30	108.27	108.30	108.35	108.37	108.38	108.60
12250	108.40	108.35	108.40	108.42	108.40	108.47	108.61
12500	108.43	108.43	108.45	108.46	108.40	108.55	108.68
12750	108.45	108.47	108.49	108.49	108.40	108.60	108.55
13000	108.48	108.50	108.52	108.52	108.47	108.60	108.49
13250	108.50	108.54	108.57	108.58	108.55	108.64	108.42
13500	108.53	108.60	108.62	108.63	108.60	108.70	108.84
13750	108.55	108.61	108.67	108.68	108.70	108.76	108.57
14000	108.58	108.64	108.70	108.72	108.80	108.88	109.06

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
14000	108.58	108.64	108.70	108.72	108.80	108.88	109.06
14250	108.60	108.67	108.74	108.78	108.85	109.00	108.94
14500	108.67	108.73	108.78	108.83	109.00	109.00	108.88
14750	108.74	108.80	108.89	108.92	109.00	109.00	109.00
15000	108.80	108.89	108.97	109.05	109.07	109.10	108.46
15250	108.90	109.05	109.20	109.20	109.20	109.20	109.39
15500	109.00	109.15	109.20	109.30	109.39	109.41	109.54
15750	109.04	109.19	109.30	109.45	109.46	109.43	109.39
16000	109.07	109.22	109.42	109.50	109.53	109.47	109.44
16250	109.11	109.26	109.45	109.54	109.61	109.49	109.39
16500	109.14	109.29	109.47	109.59	109.62	109.60	109.21
16750	109.18	109.33	109.48	109.61	109.62	109.61	109.42
17000	109.20	109.36	109.48	109.63	109.63	109.65	109.49
17250	109.35	109.40	109.56	109.67	109.67	109.70	109.50
17500	109.50	109.52	109.61	109.70	109.70	109.78	109.56
17750	109.65	109.64	109.68	109.75	109.75	109.84	109.54
18000	109.80	109.76	109.74	109.80	109.80	109.90	109.80
18250	109.90	109.90	109.90	109.98	109.98	109.98	109.93
18500	110.00	110.00	110.00	110.18	110.15	110.17	110.01
18750	110.05	110.15	110.25	110.25	110.25	110.25	110.34
19000	110.10	110.22	110.27	110.27	110.33	110.35	110.24
19250	110.15	110.25	110.30	110.38	110.40	110.42	110.10
19500	110.20	110.35	110.40	110.52	110.50	110.55	110.48
19750	110.24	110.60	110.61	110.62	110.62	110.60	110.89
20000	110.28	110.64	110.65	110.67	110.66	110.69	112.66
20250	110.32	110.68	110.69	110.70	110.70	110.72	110.75
20500	110.36	110.67	110.74	110.74	110.75	110.76	110.85
20750	110.40	110.72	110.78	110.78	110.78	110.79	110.83
21000	110.55	111.85	110.86	110.88	110.88	110.90	110.92
21250	110.70	110.95	111.00	111.20	111.02	111.04	111.06
21500	110.80	111.20	111.19	111.23	111.23	111.24	111.21
21750	111.00	111.40	111.40	111.40	111.40	111.41	111.30
22000	111.20	111.60	111.60	111.60	111.60	111.61	111.42
22250	111.30	111.67	111.70	111.71	111.72	111.72	110.92
22500	111.40	111.74	111.80	111.82	111.83	111.82	111.70
22750	111.45	111.80	111.85	111.86	111.87	111.87	111.63
23000	111.50	111.90	111.90	111.91	111.92	111.92	111.87
23250	111.55	111.95	111.95	111.96	111.97	111.97	111.87
23500	111.60	112.01	112.01	112.20	112.22	112.25	111.86
23750	111.65	112.15	112.17	112.21	112.21	112.22	111.99
24000	112.00	112.40	112.40	112.42	112.42	112.43	112.19
24250	112.10	112.50	112.60	112.60	112.60	112.61	112.59
24500	112.20	112.60	112.64	112.64	112.64	112.65	112.53
24750	112.24	112.64	112.68	112.68	112.68	112.69	112.67
25000	112.28	112.68	112.72	112.72	112.72	112.73	112.64
25250	112.32	112.72	112.78	112.78	112.78	112.79	112.70
25500	112.36	112.78	112.80	112.80	112.80	112.81	114.14
25750	112.40	112.85	112.89	112.90	112.91	112.92	112.79
26000	112.44	113.00	113.00	113.02	113.03	113.05	112.83
26250	112.48	133.02	113.05	113.07	113.08	113.10	112.91
26500	112.52	113.05	113.09	113.11	113.13	113.18	113.13
26750	112.56	113.07	113.13	113.15	113.18	113.24	113.18
27000	112.60	113.10	113.17	113.21	113.24	113.29	113.43
27250	112.67	113.15	113.27	113.27	113.31	113.34	113.36
27500	112.74	113.20	113.37	113.35	113.38	113.38	117.54
27750	112.80	113.30	113.40	113.45	113.47	113.49	113.48
28000	112.90	113.40	113.48	113.61	113.63	113.64	118.30


PWD Levee - Ulupna Creek confluence to Cleaves
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest
Elevation (m AHD)
114.00
113.00
112.00
111.00
110.00
109.00
108.00
107.00
106.00
105.00
104.00
5000
10000
15000
20000
25000
Chainage (m)


PWD Levee - Cleaves to Cobram
Elevation (m AHD)
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest
120.00
119.00
118.00
117.00
116.00
115.00
114.00
113.00
112.00
111.00
110.00
28000
29000
30000
31000
32000
33000
34000
35000
36000


0
250
500
750
1000
1250
1500
1750
2000
2250
2500
2750
3000
3250
3500
3750
4000
4250
4500
4750
5000
5250
5500
5750
6000
6250
6500
6750
7000
7250
7500
7750
8000
8250
8500
8750
9000
9250
9500
9750
10000
10250
10500
10750
11000
11250
11500
11750
12000
12250
0 125 250 500 750 1,000
Metres
N


11000
11250
11500
11750
12000
12250
12500
12750
13000
13250
13500
13750
14000
14250
14500
14750
15000
15250
15500
15750
16000
16250
16500
16750
17000
17250
17500
17750
18000
18250
18500
18750
19000
19250
19500
19750
20000
20250
20500
20750
21000
21250
21500
21750
22000
22250
22500
23000
0 125250 500 750 1,000
Metres
N


21000
21250
21500
21750
22000
22250
22500
22750
23000
23250
23500
23750
24000
24250
24500
24750
25000
25250
25500
25750
26000
26250
26500
26750
27000
27250
27500
27750
28000
28250
28500
28750
29000
29250
29500
29750
30000
30250
30500
30750
31000
31250
31500
31750
32000
32250
0 125250 500 750 1,000
Metres
N

Ulupna Island levee – Levee 1

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	107.35	107.45	107.62	107.62	107.70	107.86	107.06
250	107.37	107.52	107.67	107.68	107.74	107.90	107.22
500	107.39	107.60	107.72	107.72	107.79	107.93	107.08
750	107.40	107.68	107.74	107.76	107.85	107.96	107.40
1000	107.50	107.72	107.76	107.80	107.90	108.00	107.64
1250	107.60	107.79	107.79	107.87	107.95	108.04	107.61
1500	107.70	107.80	107.88	107.94	108.00	108.09	107.95
1750	107.90	107.90	108.00	108.03	108.05	108.13	108.34
2000	108.00	108.00	108.05	108.08	108.12	108.18	108.13
2250	108.10	108.10	108.12	108.13	108.18	108.20	108.08
2500	108.20	108.20	108.22	108.24	108.24	108.26	108.21
2750	108.25	108.27	108.30	108.33	108.32	108.36	108.23
3000	108.30	108.34	108.38	108.40	108.40	108.42	108.32
3250	108.40	108.40	108.44	108.45	108.44	108.49	108.53
3500	108.45	108.45	108.48	108.50	108.51	108.56	108.58
3750	108.48	108.50	108.53	108.57	108.57	108.63	108.65
4000	108.52	108.55	108.62	108.64	108.65	108.72	108.36
4250	108.56	108.65	108.70	108.72	108.76	108.81	108.78
4500	108.60	108.72	108.78	108.79	108.85	108.90	108.89
4750	108.70	108.80	108.88	108.90	108.94	108.98	109.13
5000	108.80	108.90	108.98	109.00	109.00	109.07	109.04
5250	108.87	109.00	109.11	109.13	109.13	109.16	109.05
5500	108.94	109.10	109.23	109.25	109.26	109.27	109.22
5750	109.00	109.20	109.32	109.36	109.38	109.39	109.22
6000	109.07	109.25	109.41	109.44	109.46	109.50	109.47
6250	109.12	109.30	109.46	109.50	109.54	109.60	109.67
6500	109.20	109.35	109.51	109.57	109.62	109.64	109.70
6750	109.30	109.40	109.56	109.62	109.68	109.71	109.63
7000	109.40	109.50	109.61	109.70	109.74	109.77	109.68
7250	109.60	109.60	109.72	109.77	109.82	109.89	109.94
7500	109.80	109.85	109.89	109.90	109.97	109.96	109.65
7750	110.00	110.00	110.10	110.12	110.12	110.12	110.03
8000	110.05	110.15	110.24	110.27	110.25	110.27	110.21
8250	110.05	110.25	110.33	110.38	110.37	110.37	110.41
8500	110.05	110.35	110.41	110.42	110.43	110.43	110.51
8750	110.05	110.40	110.41	110.42	110.43	110.43	110.43
9000	110.00	110.30	110.34	110.35	110.35	110.37	110.27
9250	109.95	110.20	110.24	110.26	110.26	110.29	110.31
9500	109.90	110.17	110.18	110.20	110.20	110.23	110.14
9750	109.90	110.14	110.14	110.17	110.17	110.17	110.23
10000	109.80	110.10	110.11	110.13	110.13	110.13	110.04
10250	109.80	110.07	110.08	110.09	110.09	110.10	110.12
10500	109.80	110.03	110.04	110.06	110.06	110.06	110.10
10750	109.70	110.00	110.01	110.02	110.02	110.03	110.04
11000	109.60	109.90	109.89	109.91	109.91	109.93	109.93
11213	109.55	109.80	109.78	109.81	109.81	109.81	109.91

Ulupna Island levee – Levee 2

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	107.85	107.90	107.93	107.98	108.00	108.04	108.82
250	107.80	107.82	107.87	107.90	107.93	108.02	107.95
500	107.60	107.65	107.78	107.80	107.87	108.00	107.74
750	107.40	107.50	107.60	107.69	107.80	107.88	107.48
1000	107.35	107.45	107.60	107.65	107.80	107.87	107.56
1250	107.35	107.45	107.60	107.65	107.78	107.86	107.28
1500	107.35	107.45	107.60	107.65	107.75	107.86	107.31
1750	107.35	107.45	107.60	107.65	107.73	107.85	107.25
1859	107.35	107.45	107.59	107.65	107.72	107.85	107.16

Ulupna Island levee – Levee 3

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	108.10	108.20	108.27	108.35	108.35	108.32	109.75
250	108.20	108.30	108.38	108.40	108.40	108.42	108.35
500	108.27	108.37	108.45	108.48	108.48	108.49	108.48
569	108.28	108.38	108.46	108.49	108.49	108.50	110.52

Ulupna Island levee – Levee 4

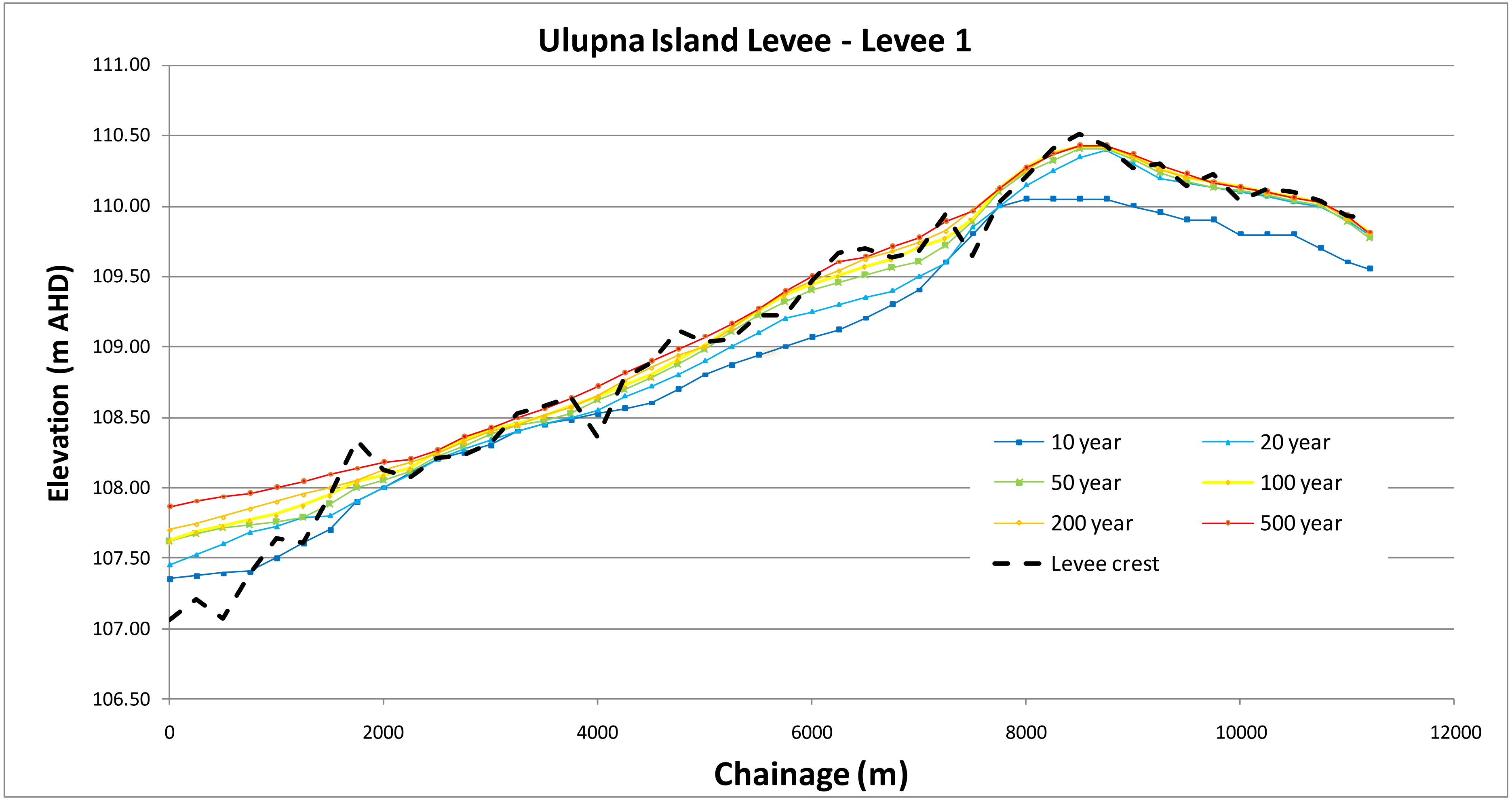
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	108.50	108.60	108.67	108.70	108.70	108.71	109.16
250	108.53	108.60	108.72	108.76	108.76	108.76	108.70
500	108.56	108.70	108.78	108.80	108.80	108.80	108.71
529	108.60	108.70	108.79	108.80	108.80	108.81	109.82

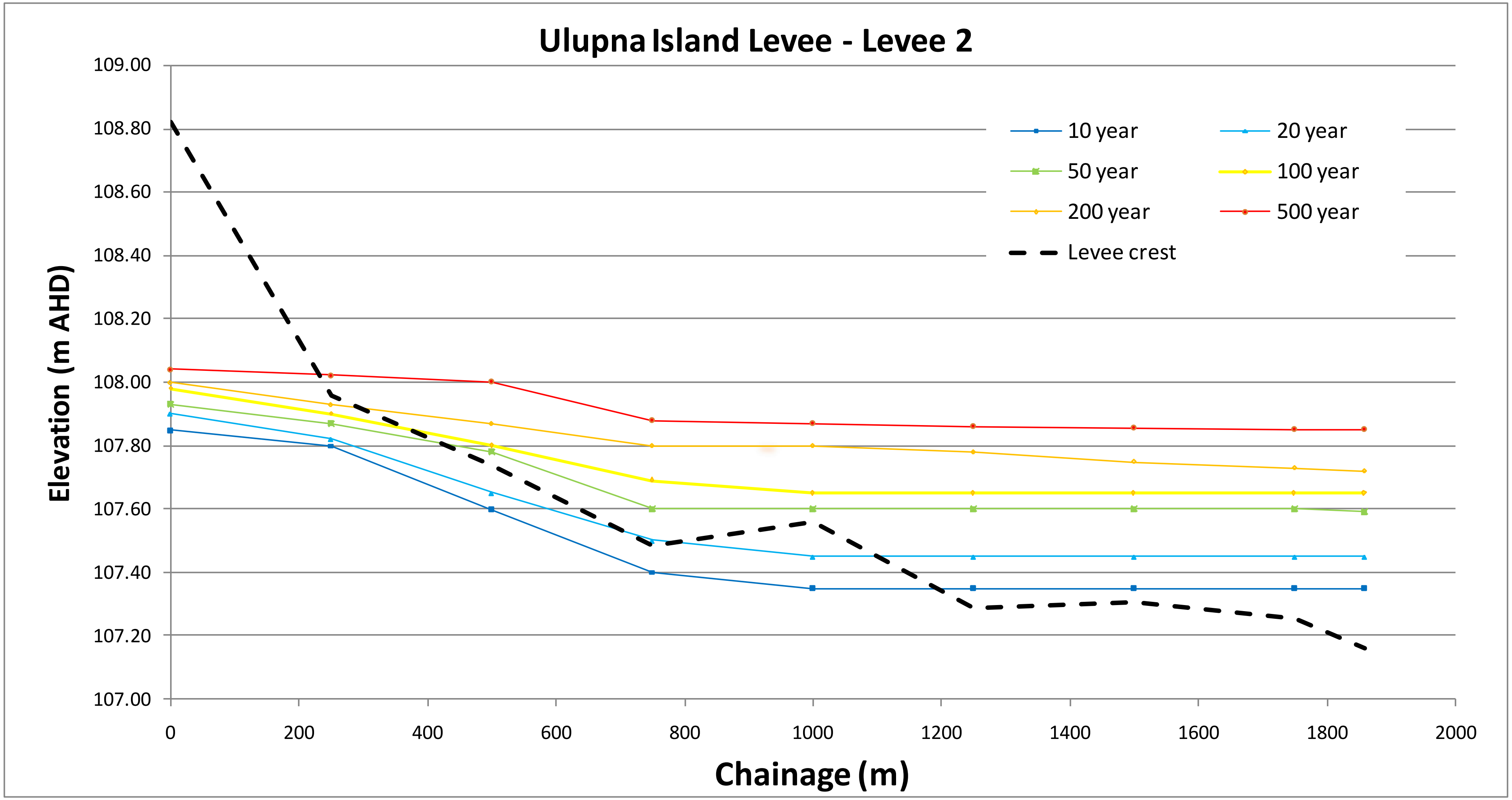
Ulupna Island levee – Levee 5

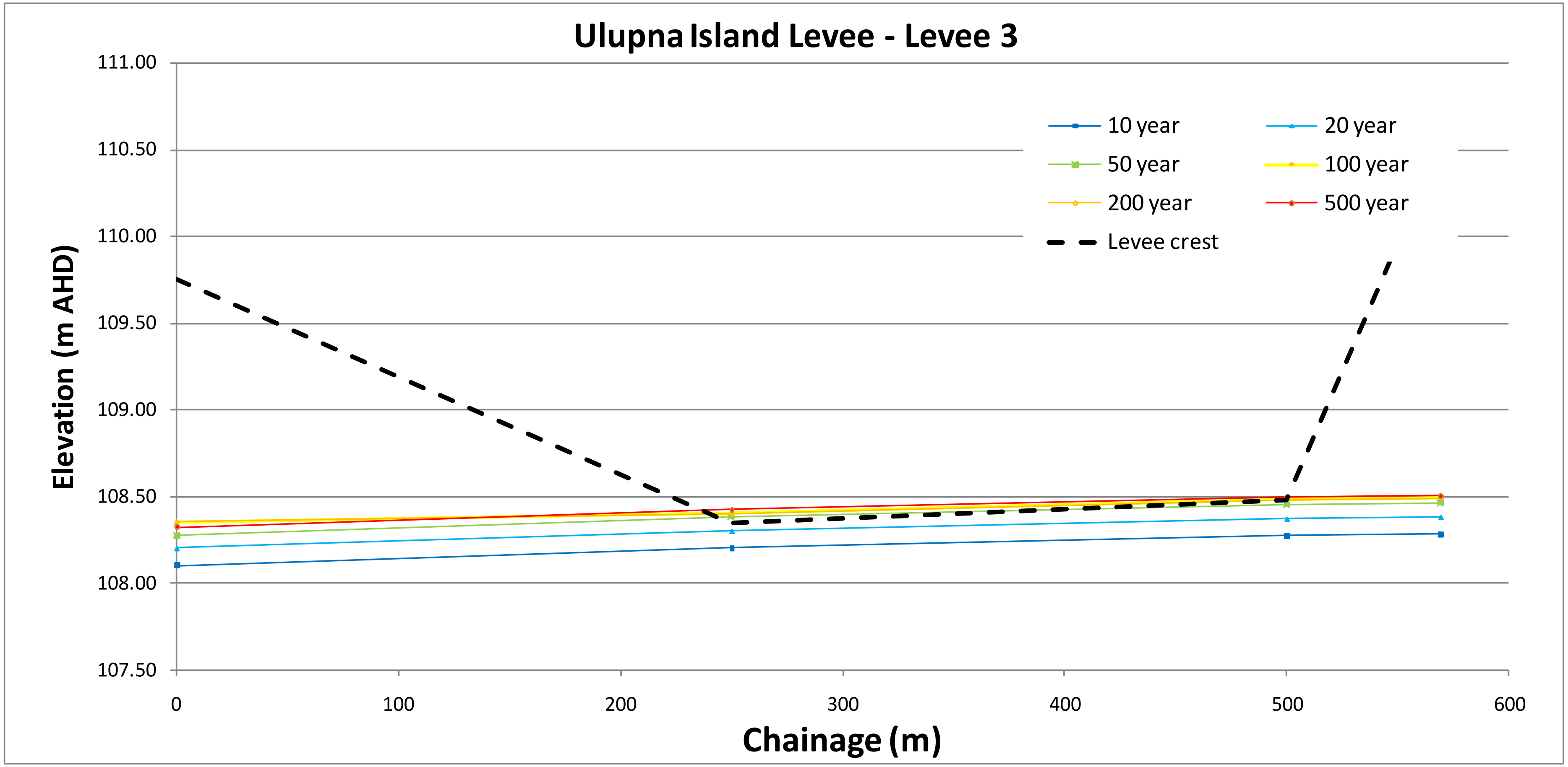
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	108.70	108.80	108.92	108.90	108.90	108.94	110.14
250	108.80	108.90	109.00	109.01	109.01	109.04	109.00
483	108.90	109.00	109.08	109.09	109.09	109.12	109.26

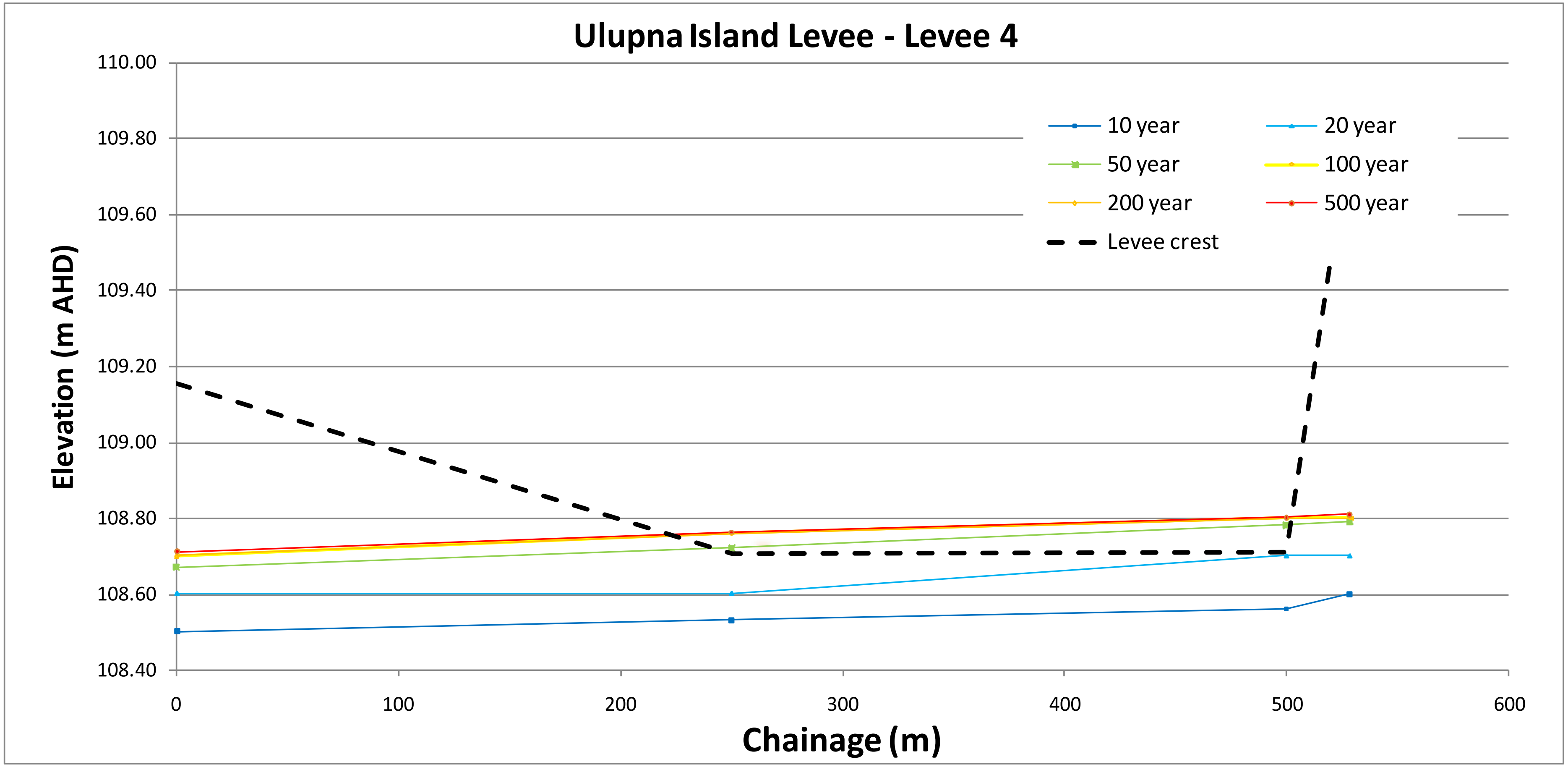
Ulupna Island levee – Levee 6

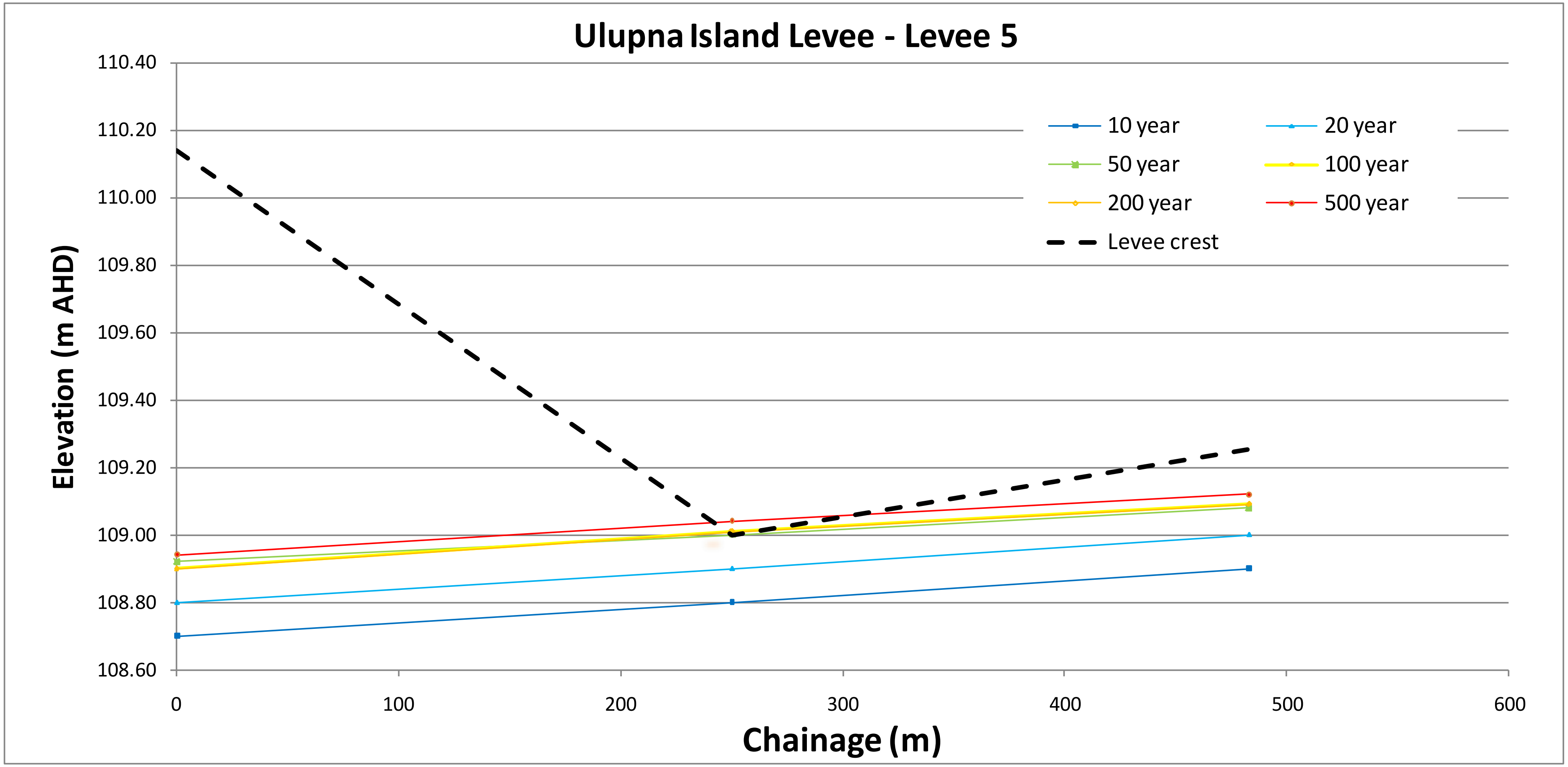
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	109.00	109.20	109.20	109.22	109.22	109.21	109.00
219	109.20	109.35	109.38	109.40	109.40	109.40	109.16

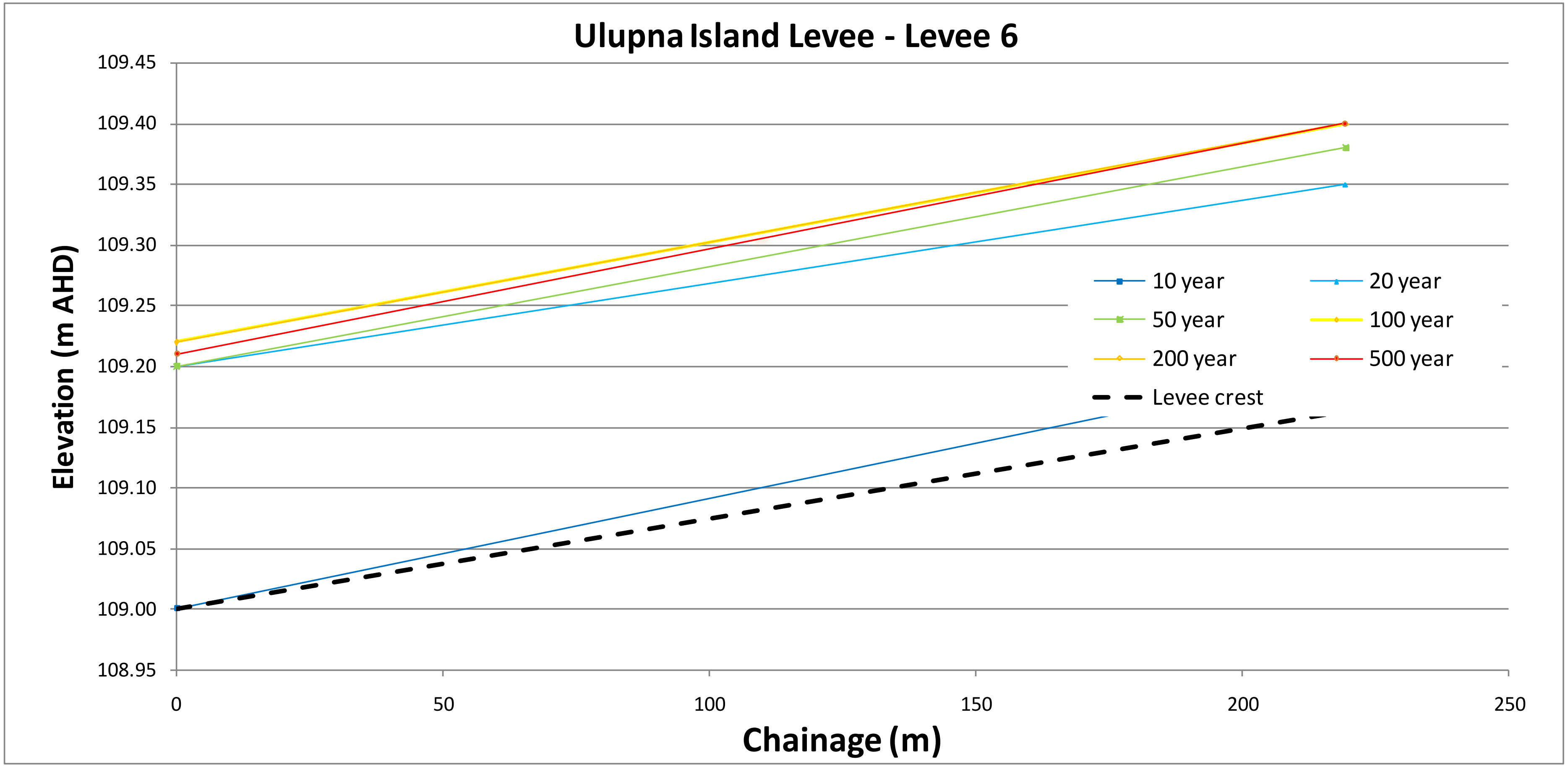

Ulupna Island Levee - Levee 1
Elevation (m AHD)
111.00
110.50
110.00
109.50
109.00
108.50
108.00
107.50
107.00
106.50
0
2000
4000
6000
8000
10000
12000
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Ulupna Island Levee - Levee 2
Elevation (m AHD)
109.00
108.80
108.60
108.40
108.20
108.00
107.80
107.60
107.40
107.20
107.00
0
200
400
600
800
1000
1200
1400
1600
1800
2000
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Ulupna Island Levee - Levee 3
111.00
110.50
110.00
109.50
109.00
108.50
108.00
107.50
Elevation (m AHD)
0
100
200
300
400
500
600
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Ulupna Island Levee - Levee 4
Elevation (m AHD)
110.00
109.80
109.60
109.40
109.20
109.00
108.80
108.60
108.40
0
100
200
300
400
500
600
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Ulupna Island Levee - Levee 5
Elevation (m AHD)
110.40
110.20
110.00
109.80
109.60
109.40
109.20
109.00
108.80
108.60
0
100
200
300
400
500
600
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Ulupna Island Levee - Levee 6
Elevation (m AHD)
109.45
109.40
109.35
109.30
109.25
109.20
109.15
109.10
109.05
109.00
108.95
0
50
100
150
200
250
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest

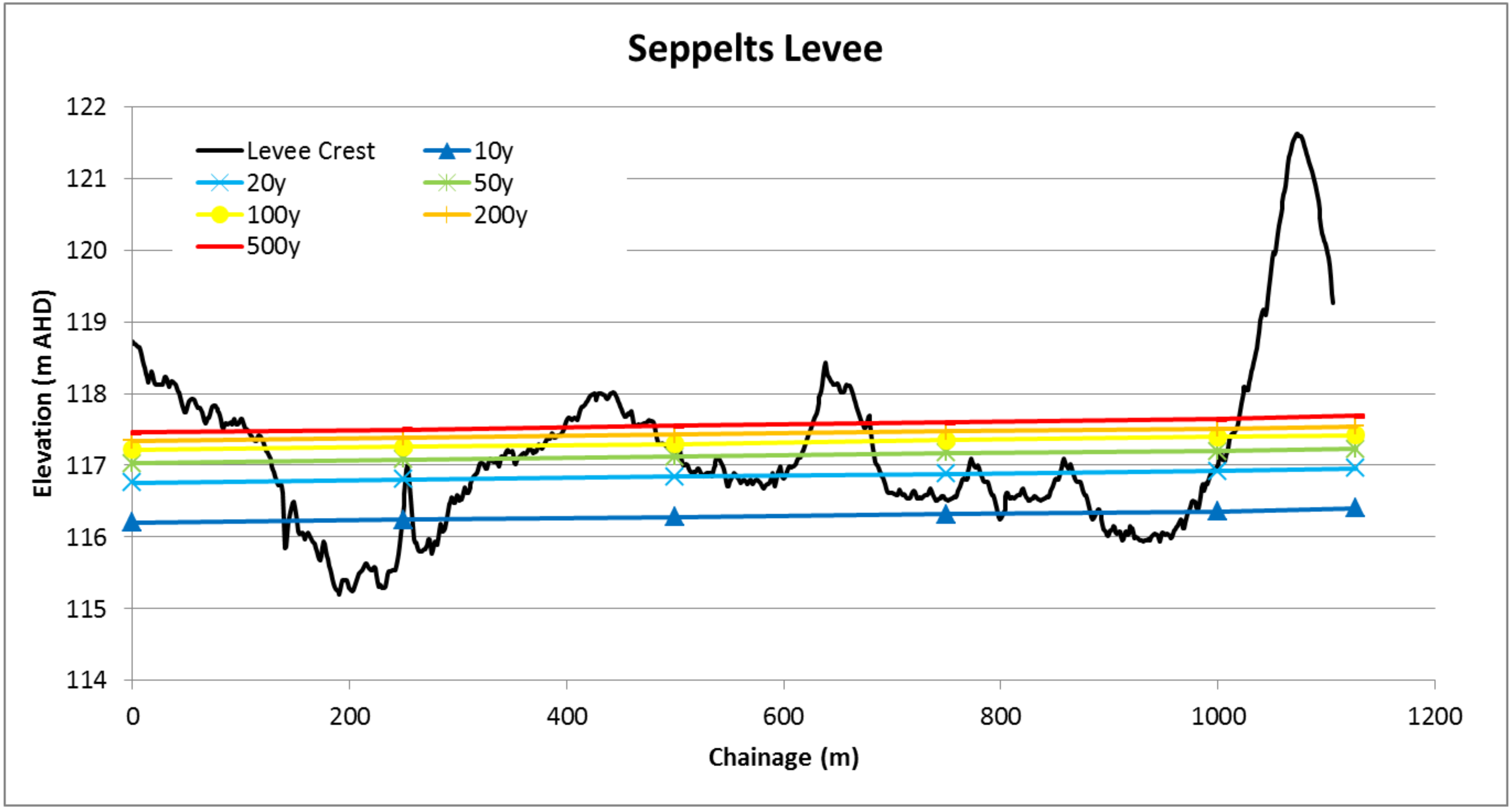
219.4
0
66.47
0
11212.69
11000
10750
10500
10250
10000
9750
9500
9250
9000
8750
8500
8000
8250
7750
7500
7250
7000
6750
6500
6250
6000
5750
5500
5250
5000
4750
4500
4250
4000
3750
3500
3250
3000
2750
2500
2250
2000
1750
1500
1250
1000
750
500
250
0
482.84
250
0
528.51
500
250
0
0
250
500
0
250
500
750
1000
1250
1500
1750
1858.59
0
250
500
0 125 250 500 750 1,000
Metres
N

Seppelts Levee

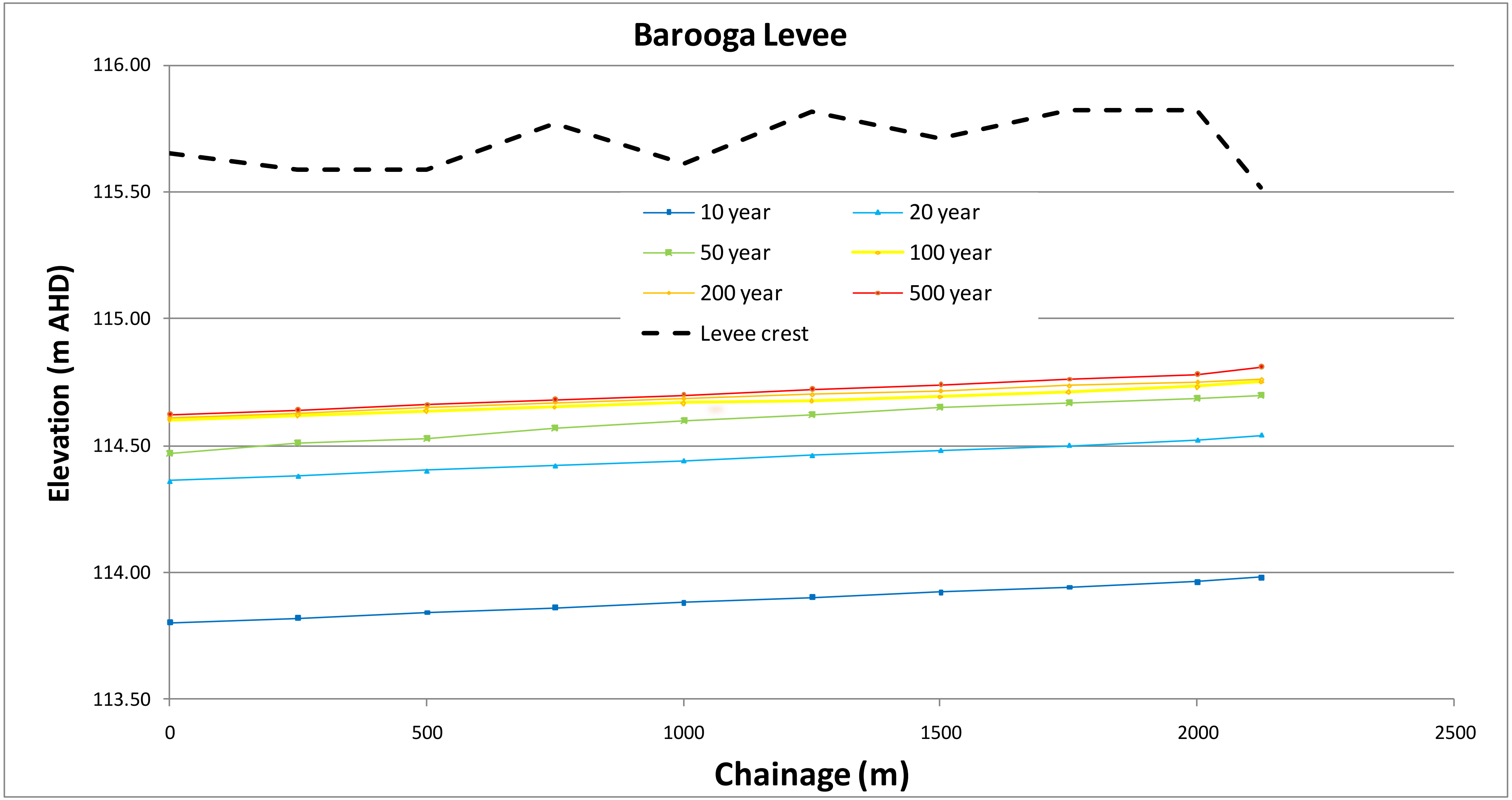
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	116.20	116.76	117.03	117.22	117.34	117.46	120.02
250	116.24	116.80	117.08	117.26	117.39	117.50	117.45
500	116.28	116.84	117.12	117.30	117.43	117.55	118.05
750	116.32	116.88	117.17	117.35	117.47	117.60	118.11
1000	116.36	116.92	117.20	117.40	117.51	117.64	117.11
1127	116.40	116.96	117.23	117.42	117.54	117.69	120.59

Barooga Levee

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	113.80	114.36	114.47	114.60	114.61	114.62	115.65
250	113.82	114.38	114.51	114.62	114.63	114.64	115.59
500	113.84	114.40	114.53	114.64	114.65	114.66	115.59
750	113.86	114.42	114.57	114.65	114.67	114.68	115.77
1000	113.88	114.44	114.60	114.67	114.69	114.70	115.62
1250	113.90	114.46	114.63	114.68	114.70	114.72	115.82
1500	113.92	114.48	114.65	114.69	114.72	114.74	115.71
1750	113.94	114.50	114.67	114.71	114.74	114.76	115.82
2000	113.96	114.52	114.69	114.73	114.75	114.78	115.82
2124	113.98	114.54	114.70	114.75	114.76	114.81	115.52


Seppelts Levee
Levee Crest
10y
20y
50y
100y
200y
500y
Elevation (m AHD)
122
121
120
119
118
117
116
115
114
0
200
400
600
800
1000
1200
Chainage (m)


0
250
500
750
1000
1127.29
0
125
250
500
Metres
N


Barooga Levee
116.00
115.50
115.00
114.50
114.00
113.50
Elevation (m AHD)
0
500
1000
1500
2000
2500
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


0
250
500
750
1000
1250
1500
1750
2000
2124.22
0 125 250 500 750 1,000
Metres
N

Tocumwal Levee

Levee 1

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	108.20	108.60	108.60	108.61	108.65	108.67	115.21
250	108.40	108.70	108.80	108.85	108.78	108.82	110.73
500	108.60	108.80	109.00	109.06	109.00	109.05	110.76
750	108.80	108.90	109.10	109.17	109.15	109.20	110.83
1000	108.90	109.00	109.20	109.27	109.25	109.30	110.31
1250	109.00	109.20	109.30	109.32	109.48	109.49	110.41
1500	109.20	109.40	109.41	109.60	109.60	109.61	111.00
1750	109.40	109.80	109.81	109.82	109.81	109.82	111.02
2000	109.60	109.82	109.82	109.84	109.83	109.84	111.09
2250	109.62	109.84	109.84	109.86	109.85	109.86	110.63
2500	109.64	109.86	109.86	109.89	109.87	109.88	111.20
2750	109.66	109.88	109.88	109.91	109.92	109.90	111.23
3000	109.68	109.90	109.90	109.94	109.91	109.92	111.27
3250	109.70	109.92	109.93	109.96	109.95	109.95	111.31
3500	109.72	109.94	109.97	109.98	109.97	109.98	111.36
3750	109.74	109.96	110.10	110.00	110.05	110.08	111.27
4000	109.76	110.00	110.20	110.20	110.22	110.23	111.44
4250	109.80	110.20	110.28	110.30	110.31	110.31	111.56
4500	110.20	110.39	110.39	110.40	110.41	110.42	111.60
4750	110.25	110.42	110.45	110.46	110.46	110.47	111.65
5000	110.30	110.45	110.50	110.53	110.53	110.50	111.71
5250	110.35	110.48	110.55	110.60	110.60	110.55	111.82
5500	110.40	110.51	110.60	110.63	110.63	110.60	111.87
5750	110.42	110.54	110.66	110.66	110.66	110.64	112.04
6000	110.44	110.60	110.73	110.80	110.80	110.68	112.05
6250	110.46	110.80	110.80	110.84	110.84	110.75	112.23
6500	110.48	110.83	110.85	110.88	110.88	110.82	112.33
6750	110.50	110.86	110.90	110.91	110.91	110.88	112.23
7000	110.52	110.89	110.93	110.95	110.95	110.95	112.32
7250	110.60	110.92	110.98	110.97	110.97	111.00	112.26
7500	110.67	110.95	111.05	111.03	111.07	111.07	112.16
7750	110.75	111.00	111.13	111.13	111.14	111.13	112.21
8000	110.80	111.10	111.20	111.20	111.20	111.20	112.23
8250	110.90	111.20	111.28	111.30	111.30	111.31	112.39
8500	111.00	111.30	111.32	111.40	111.40	111.41	112.47
8750	111.10	111.40	111.40	111.48	111.47	111.48	112.60
9000	111.20	111.50	111.52	111.52	111.53	111.54	112.79
9250	111.30	111.60	111.60	111.60	111.61	111.62	112.79
9500	111.40	111.80	111.85	111.87	111.88	111.85	112.66
9750	111.44	111.85	112.00	112.00	112.01	112.02	112.61
10000	111.48	111.90	112.03	112.04	112.06	112.07	112.91
10250	111.52	111.95	112.05	112.06	112.10	112.10	113.01
10500	111.56	112.00	112.08	112.09	112.13	112.13	113.33
10750	111.60	112.10	112.18	112.19	112.18	112.17	113.19
11000	111.80	112.20	112.30	112.31	112.32	112.30	114.01
11250	111.90	112.40	112.42	112.43	112.44	112.43	114.16
11500	112.00	112.42	112.48	112.49	112.49	112.50	113.55
11715	112.10	112.44	112.49	112.49	112.51	112.51	114.98

Cemetery levee

Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	112.10	112.42	112.46	112.47	112.47	112.48	113.95
30	112.10	112.42	112.46	112.47	112.47	112.48	113.51
90	112.10	112.42	112.46	112.47	112.47	112.48	113.51
91	112.10	112.42	112.46	112.47	112.47	112.48	112.46
103	112.10	112.42	112.46	112.47	112.47	112.48	112.46
118	112.10	112.42	112.46	112.47	112.47	112.48	112.46
119	112.10	112.42	112.46	112.47	112.47	112.48	113.87

Levee 2

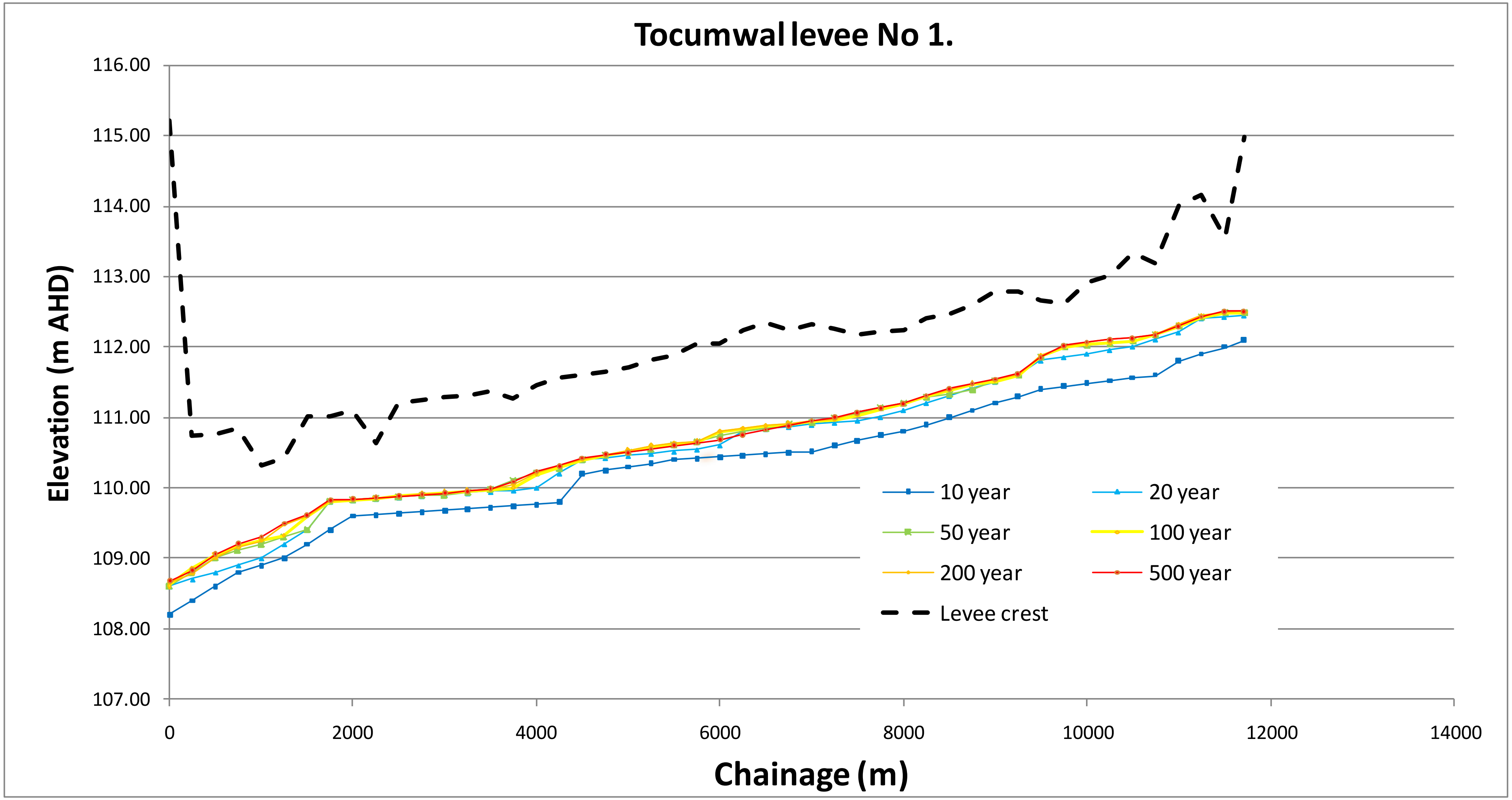
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest (as August 2008)
0	112.12	112.45	112.52	112.53	112.53	112.56	115.64
95	112.12	112.45	112.52	112.53	112.53	112.56	113.18
144	112.12	112.45	112.52	112.53	112.53	112.56	113.20
170	112.12	112.45	112.52	112.53	112.53	112.56	113.20
226	112.12	112.45	112.52	112.53	112.53	112.56	113.20
326	112.12	112.45	112.52	112.53	112.53	112.56	115.94
361	112.12	112.45	112.52	112.53	112.53	112.56	114.00
452	112.12	112.45	112.52	112.53	112.53	112.56	113.20
500	112.12	112.45	112.52	112.53	112.53	112.56	115.63
578	112.12	112.45	112.52	112.53	112.53	112.56	113.00

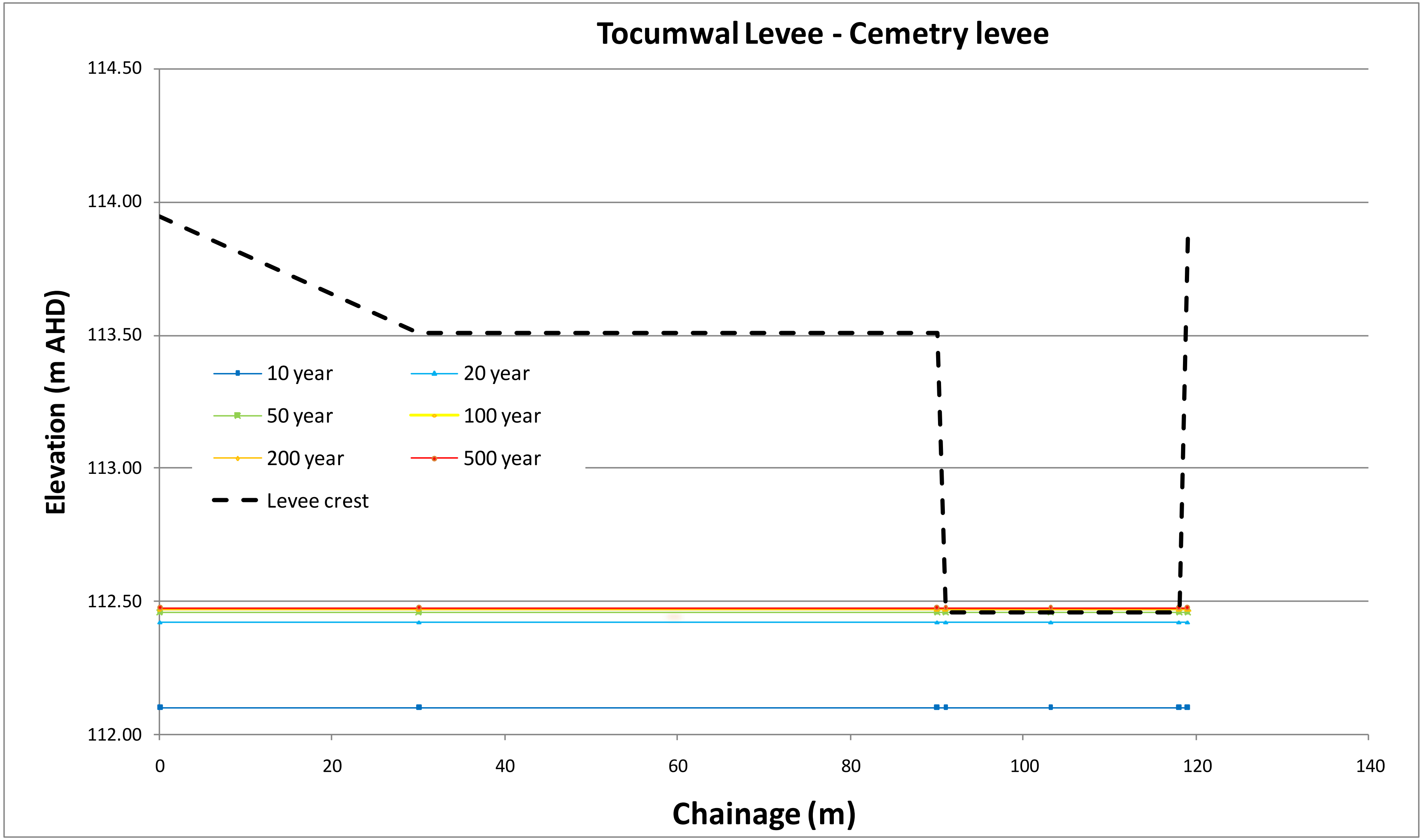
Levee 3

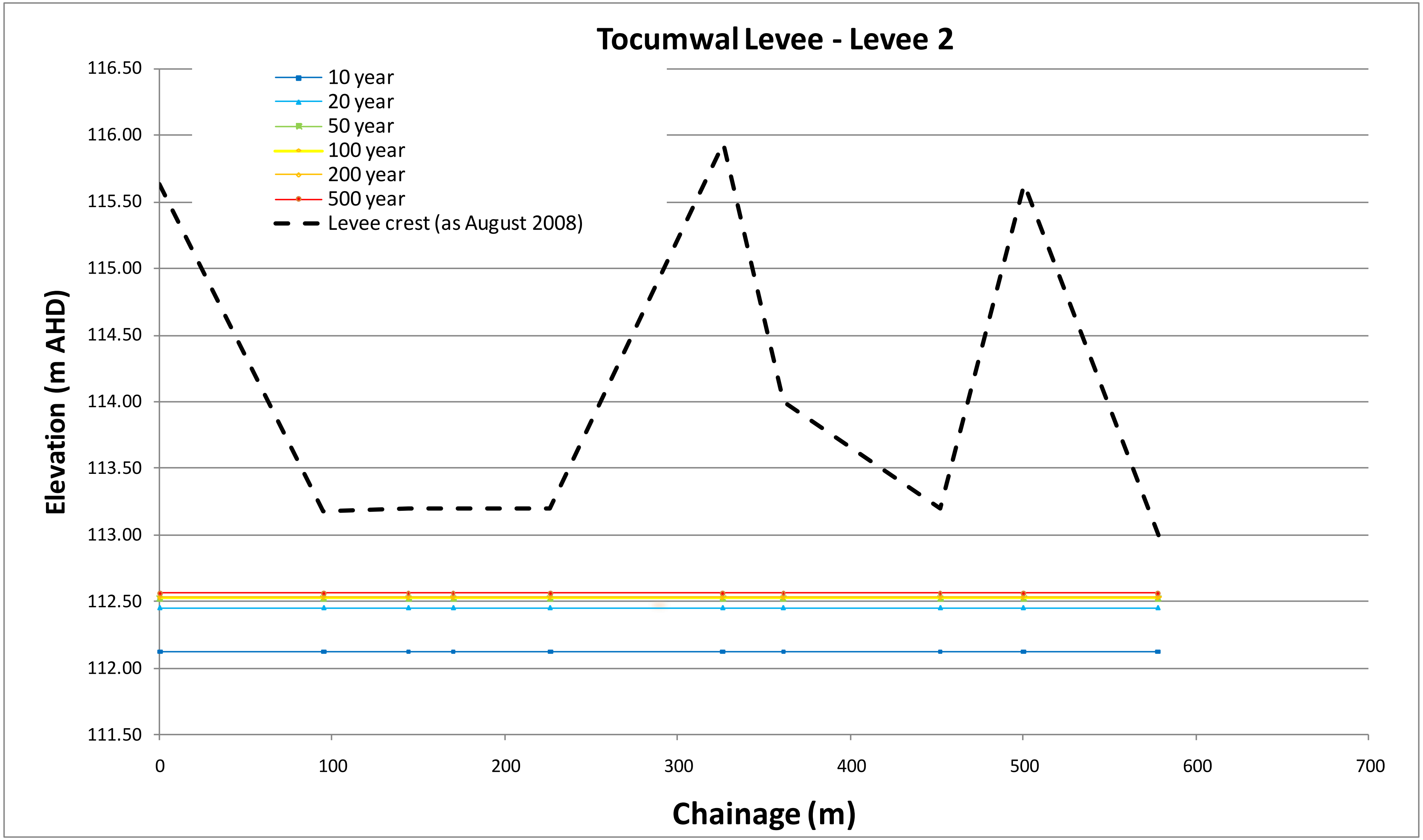
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	112.40	112.80	112.82	112.82	112.85	112.86	113.79
250	112.40	112.80	112.88	112.86	112.91	112.93	113.79
271	112.40	112.80	112.89	112.87	112.92	112.93	113.79

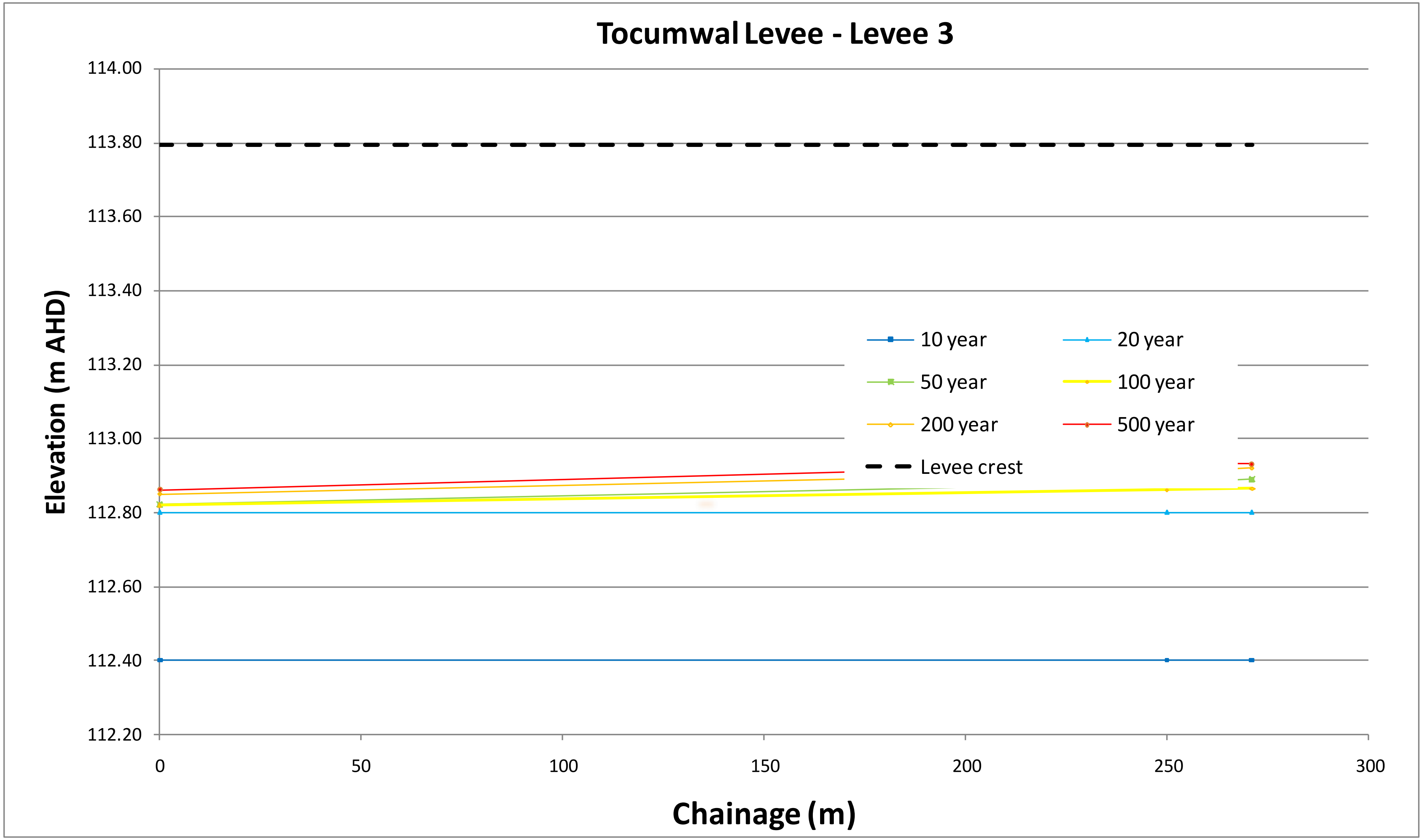
Levee 4

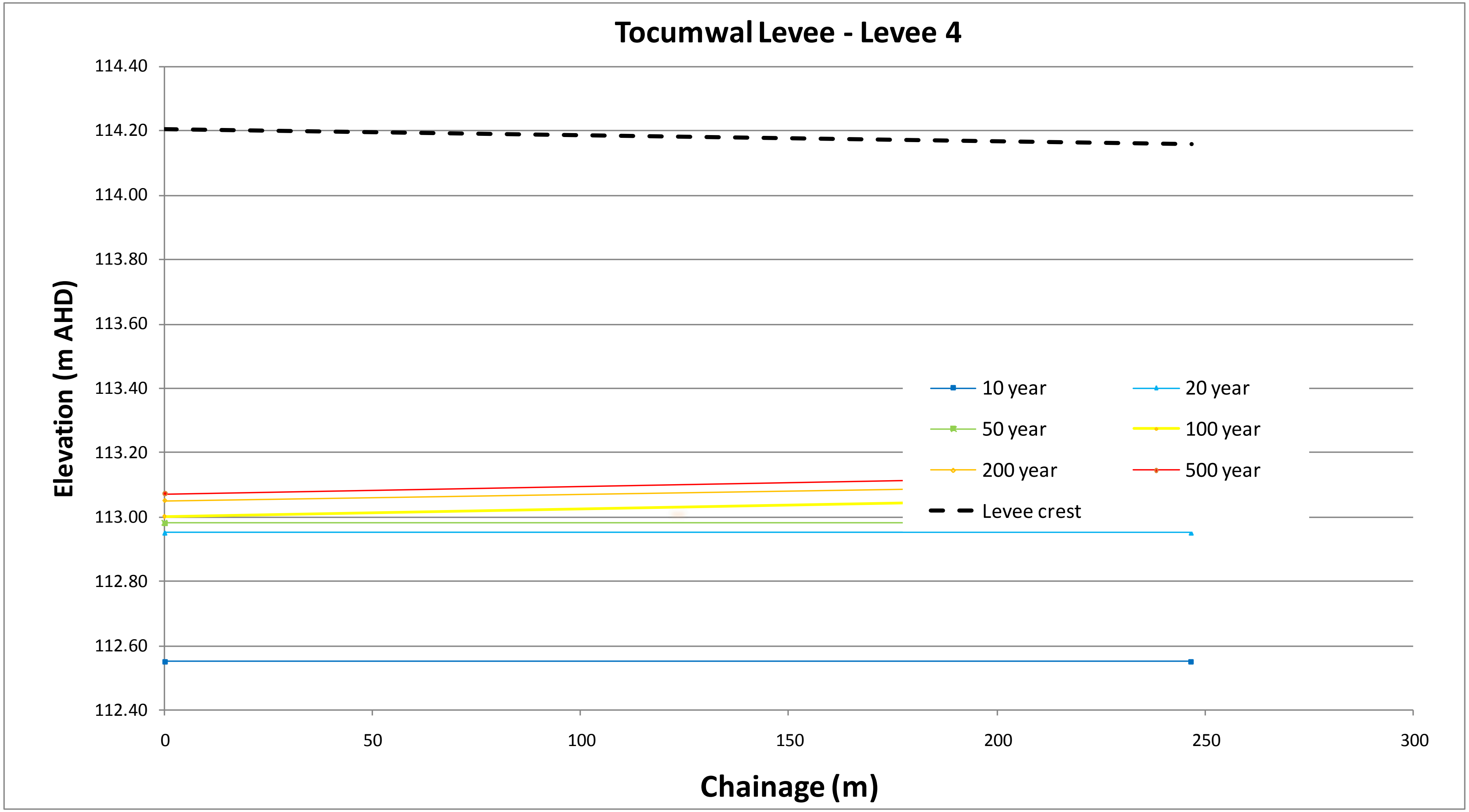
Chainage (m)	10 year	20 year	50 year	100 year	200 year	500 year	Levee crest
0	112.55	112.95	112.98	113.00	113.05	113.07	114.20
247	112.55	112.95	112.98	113.06	113.10	113.13	114.16

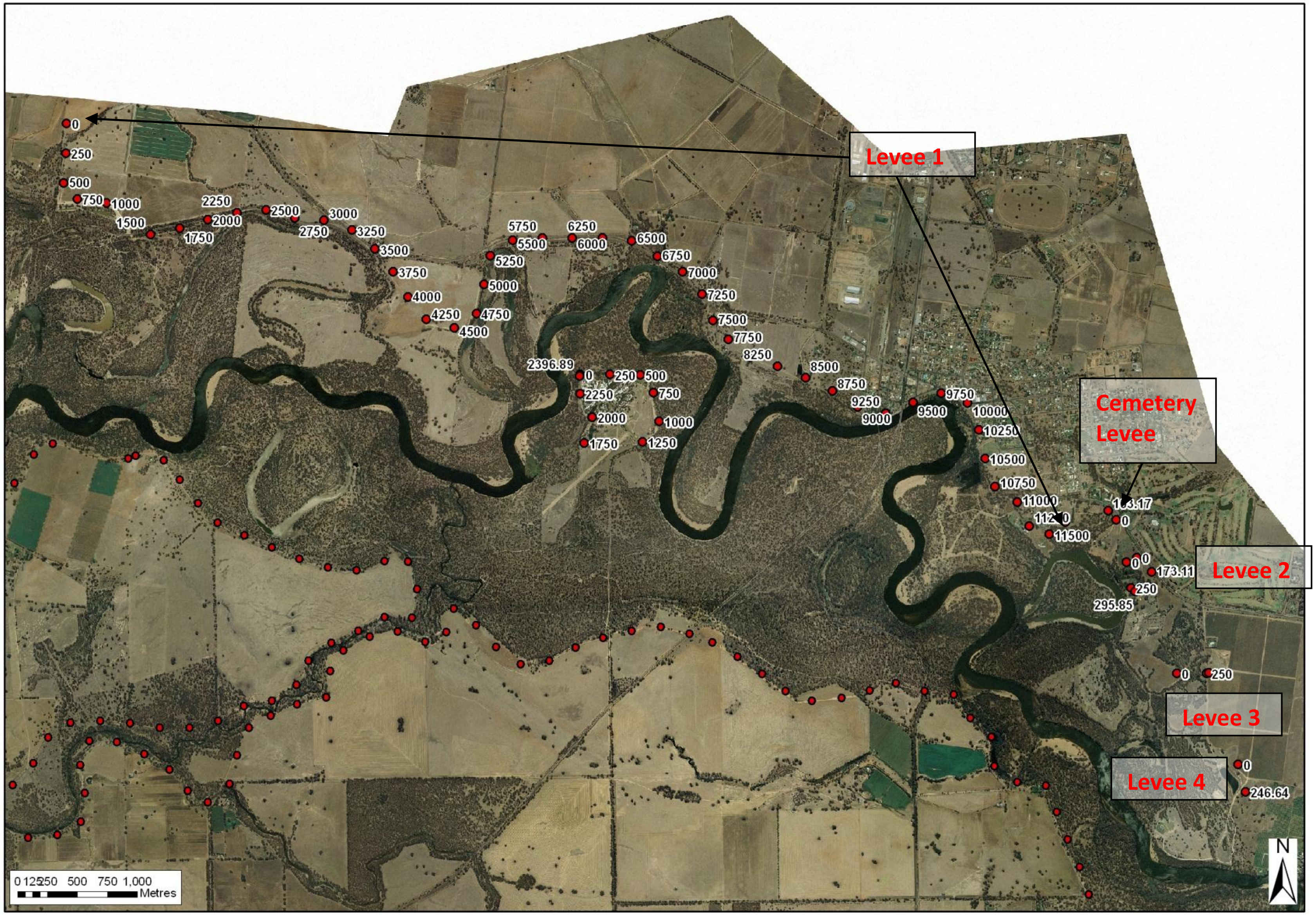

Tocumwal levee No 1.
116.00
115.00
114.00
113.00
112.00
111.00
110.00
109.00
108.00
107.00
Elevation (m AHD)
0
2000
4000
6000
8000
10000
12000
14000
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Tocumwal Levee - Cemetry levee
114.50
114.00
113.50
113.00
112.50
112.00
Elevation (m AHD)
0
20
40
60
80
100
120
140
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Tocumwal Levee - Levee 2
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest (as August 2008)
Elevation (m AHD)
116.50
116.00
115.50
115.00
114.50
114.00
113.50
113.00
112.50
112.00
111.50
0
100
200
300
400
500
600
700
Chainage (m)


Tocumwal Levee - Levee 3
Elevation (m AHD)
114.00
113.80
113.60
113.40
113.20
113.00
112.80
112.60
112.40
112.20
0
50
100
150
200
250
300
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Tocumwal Levee - Levee 4
Elevation (m AHD)
114.40
114.20
114.00
113.80
113.60
113.40
113.20
113.00
112.80
112.60
112.40
0
50
100
150
200
250
300
Chainage (m)
10 year
20 year
50 year
100 year
200 year
500 year
Levee crest


Levee 1
Cemetery Levee
Levee 2
Levee 3
Levee 4
N
0 125 250 500 750 1,000 Metres