River Murray flood mitigation planning: Assessment of flood consequences

DEWNR Technical report 2015/56

Government of South Australia
Department of Environment, Water and Natural Resources

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Foreword

The Department of Environment, Water and Natural Resources (DEWNR) is responsible for the management of the State's natural resources, ranging from policy leadership to on-ground delivery in consultation with government, industry and communities.

High-quality science and effective monitoring provides the foundation for the successful management of our environment and natural resources. This is achieved through undertaking appropriate research, investigations, assessments, monitoring and evaluation.

DEWNR's strong partnerships with educational and research institutions, industries, government agencies, Natural Resources Management Boards and the community ensures that there is continual capacity building across the sector, and that the best skills and expertise are used to inform decision making.

Sandy Pitcher
CHIEF EXECUTIVE
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Summary

The Department of Environment, Water and Natural Resources (DEWNR), the Renmark Paringa Council (RPC) and the State Emergency Service received a grant through the National Disaster Resilience Grant Scheme (NDRGS) to undertake a joint project to mitigate flood risk along the River Murray in South Australia, improve pre-planning for future upgrades of levees and increase community awareness and resilience to flood disasters.

Within DEWNR, the project is being coordinated by the Hazard Management Unit of the Regional Coordination Branch. Key components of the project are assessing the likelihood (frequency) of flooding along the River Murray, the consequences from that flooding and understanding the current condition and extent of levee banks. The Science, Monitoring and Knowledge Branch (SMK) of DEWNR have provided specialist technical advice and analysis for these project components, including hydrological analysis, flood modelling and Geographical Information Systems (GIS) spatial analysis, which are documented in this technical report.

Knowing the likelihood and consequences of flooding enables decision makers, including councils, state government and emergency services, to understand the level of risk to the community and the relative risk compared to other natural hazards and other communities. This information may also be used as an input to cost-benefit analyses to determine if mitigation works are justifiable.

The project components undertaken by SMK described in this report are:

- Updated flood modelling and mapping
- Flood frequency analysis to determine the likelihood of flooding
- Flood consequence assessment
- spatial analysis to produce a GIS layer of levee bank information.

The project area includes the River Murray between the South Australian (SA) border with Victoria/New South Wales and Wellington. Additional analysis was undertaken for the town of Renmark, due to its vulnerability to flooding as a result of being entirely located on the River Murray floodplain and encircled by levee banks.

Flood modelling and mapping

Updated flood modelling was undertaken to prepare flood inundation maps for the SA portion of the River Murray for a range of river flows (measured at the SA border), with additional mapping produced for the Renmark Paringa Council area. The following flows have been included in this technical study:

- <u>130 000 ML/d</u>
  This flow is the moderate flood threshold for the SA River Murray, and correlates to the flow at which properties would begin to be inundated in Renmark if the levee banks were not in place.
- <u>200 000 ML/d and 250 000 ML/d</u>
  200 000 ML/d is the major flood threshold for the SA River Murray. These two flows were selected for analysis since they are near the estimated 1 in 100 annual exceedance probability[1] (AEP) flood. These flows are also in the range at

[1] The likelihood of a flood occurring is expressed using Annual Exceedance Probability (AEP) which is the chance of a flood of that size being equalled or exceeded in any year. Average Recurrence Interval (ARI) is no longer the preferred term since it wrongly implies that flood events occur at regular or known or intervals. The AEP is approximate to the reciprocal of the ARI for ARIs of greater than 10 years. Thus, 100 year ARI is equivalent to 1% AEP or 1 in 100 AEP. In this report, AEP is expressed as a one in Y chance rather than a percent since it is more easily understood, in contrast to a Y year ARI

which levee bank overtopping is a risk in Renmark, and as such there may be substantial changes in damage for small changes in flow when levee banks begin to be overtopped.

- 341 000 ML/d
  This is the gauged flow of the 1956 flood, the largest flood on record for the SA River Murray. The 1956 flood level is commonly used for development planning purposes.

Flood mapping has been undertaken which shows the inundation that could be expected at these flows. The maps assume that inundation behind levee banks only occurs from overtopping and that the levee banks remain structurally sound during the flood event. The flood mapping also assumes no remedial actions are taken during the flood events such as sandbagging or raising low points in levee crests.

Additional modelling and mapping was undertaken for Renmark to assess the areas potentially inundated if levee banks failed prior to being overtopped. This was carried out for a flow of 250 000 ML/d, with all of the levee banks removed from the model to simulate multiple levee bank failure models. The 250,000 ML/d scenario was considered for this analysis since the inundation extent for flow scenarios below 250,000 ML/d generally does not reach the levee banks, and for the 1956 event the levee banks were overtopped in places.

The modelling undertaken to produce these maps is part of a more comprehensive series of flood mapping for the River Murray currently being published by DEWNR.

The flood maps have been used to calculate flood consequences. The maps have also been used to generate flood awareness materials, which is a separate project component not included in this report.

Flood frequency analysis to determine likelihood of flooding

A flood frequency analysis was undertaken to determine the likelihood of different sized floods occurring in the SA River Murray. Flood frequency analysis is a statistical analysis which usually uses long periods of flood records from river gauges. However, since the volume of dams and water extractions have steadily increased over time in the Murray–Darling Basin, gauging records cannot be used reliably as catchment conditions have changed. This was overcome by using hydrological modelling outputs produced by the Murray–Darling Basin Authority (MDBA) that recreated what flows would have occurred over the last 114 years if current catchment conditions were in place for the whole time. The recent modelling for the Basin Plan was used as the basis for the analysis since this is considered to best representation of future conditions. However, it is noted that any flood frequency analysis is a 'point in time' estimate only based on the best information available and these estimates may change in future.

The analysis concluded that the 1 in 100 AEP event has a flow rate of 226 000 ML/d, while the 1956 event was estimated to have an AEP of 1 in 250.

Flood consequence assessment

A flood consequence assessment is an estimate of the economic costs of flood damage from specified flood events. When tangible damaged estimates are combined with the probabilities of the selected events, the consequence assessment enables the annual average damage from flooding to be calculated. In this study, the flood inundation maps generated from modelling were used to provide water levels for the various flow scenarios. GIS analysis provided inundation depths and identified the types and number of properties and roads inundated. Depth-damage curves were derived and in combination with the inundation depths, the damage costs for each property, road and agricultural infrastructure for the extent of the SA River Murray and for specific areas of Renmark were determined. Additional analysis was undertaken for the Renmark area to determine the cost of damages within the town of Renmark protected by levee banks, and the cost of damages in Renmark under the levee bank failure scenario.

The assessment relied upon desktop GIS methods, Government of South Australia layers relating to infrastructure and property valuations and published data of flood damages. Due to the large area under consideration many simplifying assumptions were required to be made, such as setting a single value for floor levels above the ground. The assessment only included housing, commercial properties, roads, bridges and agriculture, and didn't include a range of infrastructure types such as hospitals, schools and treatment plants. Consequently, the calculated damages are considered broad-scale estimates only. The estimates could be further improved in future by on-ground verification of property types and floor levels.

For the River Murray floodplain between the border and Wellington, the following flood consequences were estimated to occur: Damage estimates are assessed using the Australian dollar value in 2012:

- 130 000 ML/d: $18 million, including inundation of approximately 20 houses and 900 shacks
- 200 000 ML/d: $41 million, including inundation of approximately 100 houses and 1300 shacks
- 250 000 ML/d: $110 million, including inundation of approximately 900 houses and 1400 shacks
- Repeat of 1956 flood: $225 million, including inundation of approximately 1800 houses and 1500 shacks.

Within the Renmark Paringa Council area only, the following flood consequences were estimated to occur:

- 130 000 ML/d: $10 000; limited infrastructure inundated at this flow
- 200 000 ML/d levees intact: $1.7 million as water level expected to be below levee sill levels
- 200 000 ML/d levee failure: by overtopping damages up to $8 million, including inundation of approximately 400 houses
- 250 000 ML/d: $16 million, including inundation of approximately 600 Houses as levees are expected to be overtopped
- Repeat of 1956 flood: $60 million, including inundation of approximately 1350 houses.

Using the findings of the likelihood and consequence assessments the annual average damages from flooding were calculated to be $4.3 million for the communities along the SA River Murray, and between $460 000 and $555 000 for the Renmark Paringa Council area, depending on the structural integrity of the levee banks.

Levee bank GIS layer

The Levee Bank Management Strategy GIS Layer has been developed to bring together a variety of information on levee banks, including flood inundation depth and extent, condition attributes, ownership and identification metadata, into a single spatial layer. It provides a robust tool that can assist in the planning and development of flood mitigation and management strategies.

The Levee Bank Management Strategy GIS Layer was generated by combining a GIS spatial layer identifying levees and associated crest height and other attributes such as condition, levee ownership and identification metadata with information on flood inundation depth and extent resulting from the hydraulic modelling of various flow scenarios. The levee banks around Renmark (and adjacent horticultural areas), and the lower Murray between Mannum and Wellington have been considered in this investigation.

1 Introduction

1.1 Purpose

Renmark Paringa Council (RPC), in partnership with the South Australian (SA) Department for Environment, Water and Natural Resources (DEWNR) and the State Emergency Service, are undertaking a project through the Natural Disaster Resilience Program for flood mitigation planning for the River Murray in South Australia.

The broader project aims to develop materials to educate and increase the resilience of River Murray communities with respect to flooding and to develop a management plan for rehabilitation of levee banks for the flood vulnerable community of Renmark. Deliverables of the project include updated flood mapping, a flood consequence assessment to inform risk assessments and cost benefit analysis of mitigation works, pre-planning work for upgrades of the Renmark levee banks and community education material.

Within DEWNR, the project is being coordinated by the Hazard Management Unit in the Regional Coordination Branch. The Science, Monitoring and Knowledge Branch (SMK) of DEWNR have provided specialist technical advice and analysis for a number of project components, including hydrological analysis, flood modelling, consequence assessment and Geographical Information Systems (GIS) spatial analysis, which are documented in this Technical Report.

A consequence assessment is an estimation of the cost of flood damages for a range of different sized flood events. Understanding the frequency and consequences of flooding enables decision makers to understand the:

- Level of flood risk a community is exposed to
- Relative risk flooding poses compared to other natural hazards
- Relative risk a community is exposed to in comparison to other communities.

Flood consequence information may also be used as an input to cost-benefit analyses to determine if mitigation works are justifiable.

1.2 Background

The River Murray is the largest river in South Australia and supports a range of social, economic and environmental values. Due to the river's use as an irrigation supply for agriculture, numerous towns have been developed adjacent to the River Murray, including the towns of Renmark, Berri, Waikerie, Morgan, Blanchetown, Murray Bridge, Mannum, and Wellington.

The River Murray has been observed to flood intermittently since settlement began, with the worst flood on record occurring in 1956. This flood resulted in the inundation of hundreds of homes (O'Gorman, 2012). During this event, many towns were protected to varying degrees by levee banks. Some of these levees were temporary, while some of the levees considered permanent in 1956 have since been removed or allowed to degrade over time. Due to increased development on the floodplain and different levels of flood protection by way of levees, it is reasonable to speculate that the consequences from a flood of the magnitude of the 1956 event occurring today would be very different.

The progressive construction of dams in the upper catchment of the River Murray and increasing extractions of water for agriculture and water supply has led to a reduction in the frequency of higher flows and floods in the River Murray. Community awareness of the potential for flooding and its consequences has been reduced due to the reduction in higher flow events. However, the consequences, including large-scale, severe damage to property, infrastructure and peoples livelihoods, of large flood events since 2008 in Australian catchments that also have substantial development (most notably the 2011 Queensland and Victorian floods), demonstrate that increased development does not eliminate the risk of substantial flood damage.

A flood mitigation study was undertaken for RPC in 2008 by engineering consultants Maunsell (now AECOM). This study undertook flood inundation mapping of the RPC area, including the towns of Renmark, Paringa and Lyrup, as well as a geotechnical condition assessment of levee banks. The study estimated the 1 in 100 AEP flood event to be a flow of 235 000 ML/d, while the 1956 flood (which was a flow of 341 000 ML/d) was estimated to have an AEP of 1 in 170. The study further identified that levee banks in the RPC area may be overtopped at flows of 220 000 ML/d, and that the condition of many banks were poor or average.

There are approximately 110 kilometres of levee banks between Mannum and Wellington with 67 kilometres managed by DEWNR. The remainder are privately owned. During the drought years of 2006 to 2010, low water levels downstream of Lock 1 led to levee banks adjacent to the river becoming damaged through drying out and slumping. This damage posed a risk of bank failure leading to water loss and inundation of agricultural land. In 2011, the former Department for Water (DFW) (now part of DEWNR) undertook a comprehensive condition assessment for all levee banks below Lock 1 for targeted remediation planning. The condition assessment found that in general, government-managed levee banks were in good condition and their performance satisfactory. The condition of privately owned banks was highly variable, with some in very poor condition. Levee banks that received forms of routine maintenance (all government and two private levee banks) were rated significantly higher than those that received no routine maintenance and were not accessible by vehicles. DEWNR has since undertaken minor remediation works on the government-owned banks, and is continuing to investigate options for remediation of the remaining banks. Outcomes from the condition assessment have been incorporated into the Levee Bank Management Strategy GIS Layer (See Section 7).

DFW undertook modelling and flood mapping of the 2011 flow event (90 000 ML/d) for the River Murray between Renmark and Wellington, and the 2012 peak (60 000 ML/d) between Morgan and Mannum. The inundation maps were published on the DFW website. DFW subsequently undertook modelling and mapping of a range of flows from 100 000 ML/d to 341 000 ML/d (1956 event) (Bloss et al., 2015). Some additional modelling was undertaken for this study and the modelled extents were used as basis for the assessment contained in this report.

Throughout this report, all flows refer to the measured flow to South Australia at the border with New South Wales and Victoria (commonly referred to as Flow to SA). For the purpose of this study, it has been assumed that the peak flow for individual events is the same for the length of the River Murray within South Australia since there are minimal tributary inflows downstream of the border. However, it is noted that some reduction of peak flow may occur due to seepage, evaporation and attenuation (flattening out of the flood peak as it moves down the river).

2 Flood inundation mapping

2.1 Methodology

Flood modelling has been used to develop maps of the expected extent of flood inundation for various flood events. These maps are used in the consequence assessment to estimate the number of properties likely to be affected by flooding and the depth to which they may be inundated.

Modelling has previously been undertaken by DEWNR (Bloss et al., 2015) for flows of 100 000, 120 000, 140 000, 160 000, 180 000, 200 000, 250 000, 300 000 and 341 000 ML/d for the River Murray between Murtho (upstream of Renmark) and Wellington. Of these, maps produced for 200 000, 250 000 and 341 000 ML/d have been used for this analysis. An additional run of 130 000 ML/d was undertaken to enable consequences to be determined for the flow that aligned with the moderate flood trigger for the SA River Murray in SA (DFW, 2011a).

The MIKE FLOOD hydraulic modelling software was used to model the River Murray between Murtho (upstream of Renmark) and Wellington. The model was calibrated to three historic flood events (1993, 1974 and 1956) by matching measured water levels at several locations along the river. The model simulates water level, velocity and inundation extent for a given flow upstream and water level downstream. Modelled extents (water levels) are then converted to GIS layers for further analysis. Details of the original MIKE FLOOD model configurations are presented in (Bloss et al., 2015)

The flood modelling and mapping follows similar techniques to that employed by Maunsell (2008) for the flood inundation modelling and mapping of the RPC area. The DEWNR modelling utilised a finer resolution grid (15 metres (m) instead of 30 m) and was based on a newer, finer resolution Digital Elevation Model (DEM) of the River Murray floodplain. The finer resolution DEM was generated using LiDAR capture between 2003 and 2008. Accordingly, in the RPC area the DEWNR modelling (Bloss et al., 2015) is considered to be an update to a higher resolution version of previous (Maunsell (2008)) modelling. Some difference in expected flood inundation extent predicted by the 2008 and 2014 modelling may be present due to different input data and model configuration.

The hydraulic modelling has assumed steady-state conditions, that is, the peak flow in the river persists for a sufficient period of time so that areas of inundation away from the main channel (such as anabranches and wetlands) have filled and water levels have stabilised. In reality, these backwaters may take several days to fill to the same level as the main river channel.

A further assumption made in the modelling process was that flooding behind levee banks only occurs from overtopping. Overtopping occurs when the river level is higher than the crest level of the levee bank. The flood maps show the area of land that would be inundated behind the levee bank in locations where overtopping was simulated to occur. This approach to modelling the inundation behind levee banks would overestimate the expected flood inundation extent if floodwaters pass through damaged sections of the levees slowly or actions were taken to constrain the extent of flooding. Consequently the maps illustrating levee overtopping do not represent the area that could be inundated if the levee bank was to fail due to other failure modes, such as piping or slumping.

An additional scenario was modelled with the levee banks removed to obtain an estimate of the number of properties in the RPC area that would be flooded if the levee banks failed when the river level was lower than the crest level of the banks. Levee banks were removed surrounding Renmark and Renmark West and North. This additional analysis was only undertaken for the 200 000 ML/d scenario. At 130 000 ML/d the extent of inundation does not reach the levee banks, while for flows of 250 000 ML/d and above floodwaters overtop the levee banks in multiple locations so the inundated area with and without levee banks is similar.

It is difficult to discern the available freeboard (the vertical distance between the modelled water level and levee crest) from the flood maps alone. The available freeboard is important to consider as this provides an additional factor of safety to allow for differences between simulated and actual water levels. Differences in simulated and actual water levels can occur due to model inaccuracies, differences between the peaks and durations of flood events and the effects of wind and waves.

2.2 Outcomes

Inundation extents for 130 000ML/d, 200 000 ML/d, 250 000 ML/d and 341 000 ML/d (1956 event) events were produced for the SA River Murray region and RPC area. An additional map was produced for the town of Renmark of the flood extent at 200 000 ML/d that shows both the expected inundation with levee banks in place (minimal since generally the crest levels are higher than river levels) and with levee banks removed (extensive flooding of Renmark). The map illustrating the flood inundation extent with the levee banks removed estimates the inundation extent if the levee banks fail when the river level is lower than the crest level of the banks. This map has been produced for community flood awareness purposes to demonstrate the critical role that the levees play in protecting the community from flooding.

2.3 Assumptions and limitations

Key assumptions and limitations of the hydraulic modelling that impact on the accuracy of the estimated flood extents and water levels are as follows:

1. Flood modelling and mapping is an estimation of the water levels and extent of inundation based on the input data and modelling tools available, and thus variation between actual events and the modelled predictions is expected to occur. The accuracy of the models and their outputs is strongly dependent on the input data and assumptions of the model, such as ground levels, flow and assumed downstream water levels. It is possible that small-scale elements of the channel or floodplain that may affect water levels or extent of inundation locally, such as floodplain obstructions or the effect of vegetation. Crest levels of levees in particular are sensitive to small variations in water level which in turn affects the estimated flow at which levees are predicted to overtop. Variation in water level may also occur due to the effects of waves (from wind or boats), and calibration, measurement or model errors.
2. Hydraulic modelling has assumed steady-state conditions. That is, the peak flow in the river persists for a sufficient period of time so that areas of inundation away from the main channel have filled and water levels have stabilised. In reality, these backwaters may take several days to fill to the same level as the main river channel. The steady-state model runs represent the maximum estimated extent of inundation for a specific flow scenario, as simulated by the models.
3. The model is not suitable to simulate levee bank collapse, as it uses a fixed land surface elevation (bathymetry) which does not change as a result of erosion or deposition during a flood event (excepting the modelled scenario where the levee banks were deliberately removed from the model bathymetry). This modelling approach therefore only simulates flooding behind levee banks as a result of overtopping. Consequently, the flood maps (other than the levee removal scenario for a 250 000 ML/d event) do not represent the area that could be inundated if the levee bank was to fail as a result of piping, slumping or other failure modes.
4. Extent of flooding behind levees has also been allowed to reach equilibrium conditions. In some cases this may be an overestimate of inundated area, if filling rates of areas behind levee banks are slow or actions were taken to constrain the extent of flooding.
5. Construction of temporary levee banks, sandbagging or other remedial actions during flood events are not included in the flood inundation modelling.

3 Likelihood assessment

3.1 Methodology

Assessing the frequency at which floods are expected to occur is an essential input to flood risk assessment and cost-benefit analysis for mitigation works. The estimated frequency of floods is also known as the event likelihood or probability of a flood. Flood frequency analysis is the process of fitting a probability distribution to a series of recorded flood peaks at a particular location, to determine the likelihood of particular flows occurring. Throughout this report, likelihood (or frequency) has been expressed using the term Annual Exceedance Probability (AEP). The AEP is the chance of a flood of a given or larger size occurring in any one year. AEP is usually expressed as a percentage, for example a flow with an AEP of 5% means that there is a 5%, or one in twenty chance, of that flow being met or exceeded in any one year. In this report, AEP is expressed as a one in Y chance rather than a percent since it is more easily understood. The use of the term Average Recurrence Interval (ARI) is not preferred since it can mislead the community about risk, and can be misinterpreted as meaning that a flood of a particular size is only exceeded at regular intervals (BoM 2013). However, the flood frequency analysis software used for this study still refers to ARI. For ARIs of greater than 10 years, the ARI is equivalent to the reciprocal of the AEP. For example, 20 year ARI is equivalent to 5% or 1 in 20 AEP, and 100 year ARI is equivalent to 1% or 1 in 100 AEP.

Flood frequency analysis is applicable for determining the likelihood of events occurring when (AR&R, 2006):

1. A long flood record exists
2. The flood record is homogenous or can be adjusted to near homogenous state (that is, the catchment conditions such as dams or urban development are similar throughout the flood record or the flows can be adjusted to take the changes into account)
3. A reliable rating curve exists to convert measured river heights to flows
4. The probability of the events of interest does not require extrapolation too far beyond the observed record length.

Since flow records began, considerable development has occurred within the Murray–Darling Basin such as the construction of large dams, as well as increasing extractions for irrigation and water supply. These developments in the catchment have reduced the size and frequency of floods but have also ensured that the river continues to flow during dry periods. Because of the changing catchment conditions, it is not possible to use the historical data series to generate a meaningful estimate of the current frequency of floods.

In order to account for changing catchment conditions, this study has used the outputs of modelling by the Murray–Darling Basin Authority (MDBA) as the input time-series for the flood frequency analysis. Hydrological models used by the MDBA are able to recreate what flows would have occurred over the last 114 years if current catchment conditions were in place for the whole time.

The MDBA undertook hydrological modelling of the Murray–Darling Basin to inform the development of the Basin Plan (MDBA, 2012). Under the Basin Plan, the volume of water consumption will be reduced, with this water instead used to reinstate high flow events in the river to provide water to wetlands, and to hold reserves of water for very dry years. The modelling linked together 24 existing state, Snowy Mountains Hydro and MDBA hydrological models into a single integrated modelling framework, to enable the modelling of flows throughout the basin under different water management scenarios. The modelling used historical climate records to model 114 years of climate from 1 July 1895 to 30 June 2009. Modelled scenarios included 'without development' (near-natural conditions with all infrastructure and extractions removed from the model), 'baseline' (current conditions) and Basin Plan scenarios, which included surface water extractions with reductions of 2400 GL, 2800 GL and 3200 GL across the basin (MDBA, 2012). Each Basin Plan scenario assumed constant infrastructure and water extraction conditions across the full 114 year climate period.

This study has used the time series from the Basin Plan 2800 GL reduction scenario (MDBA model run number 847) as the best estimate of future water management conditions in the Murray–Darling Basin (MDBA, 2012). Additional modifications are planned by the MDBA to further refine the hydrological modelling; however, at the time of this study this model represents the most appropriate time-series for future conditions.

Figure 3.1 demonstrates the changes in river flow to South Australia that can occur due to different catchment and water management conditions. The maximum annual Flow to SA is shown for the period 1989 to 2009 under three scenarios:

1. Modelled flows replicating what would have occurred prior to development such as dams and water extractions (without development),
2. Recorded flow measured at the gauge at the South Australian border (recorded historical)
3. Modelled flow with the new extraction limits and water management arrangements under the new Basin Plan (modelled Basin Plan).

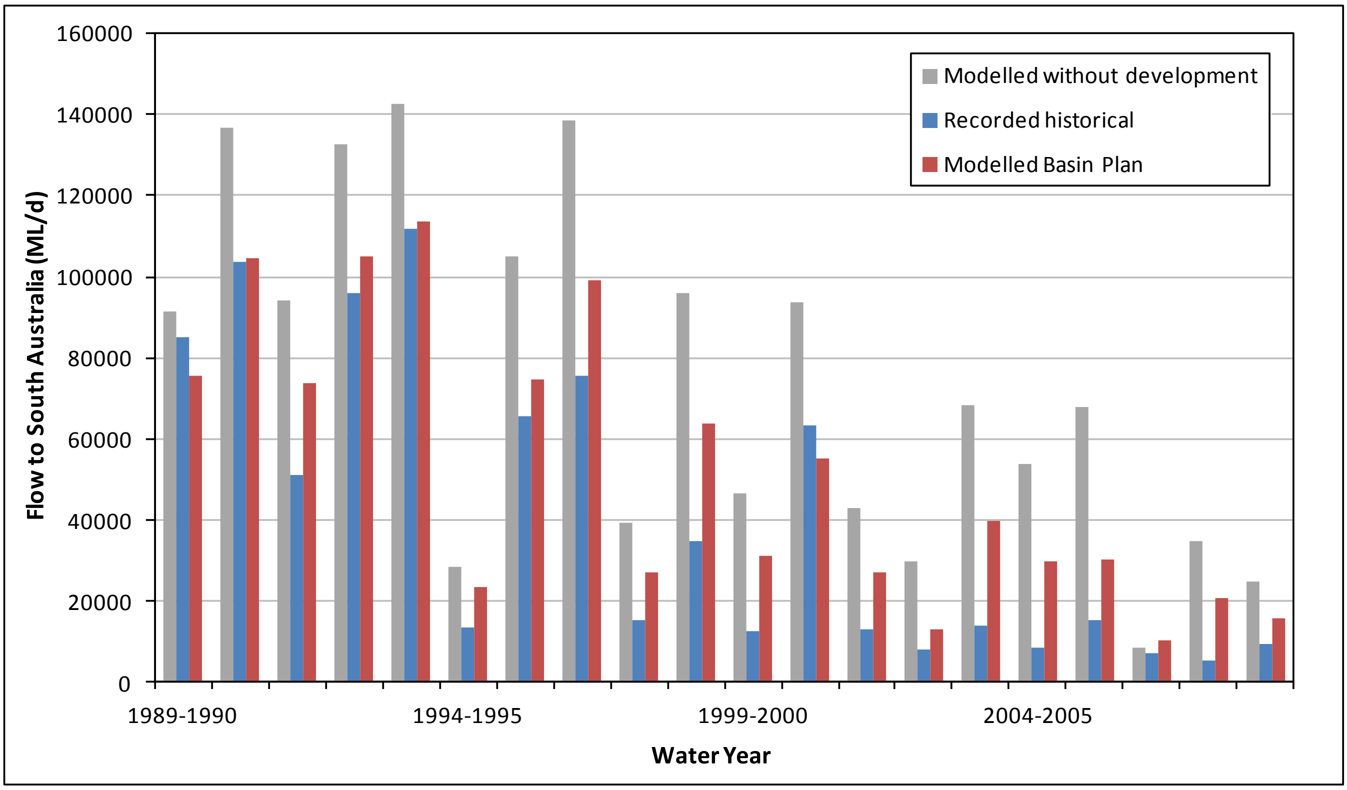

Figure 3.1 Maximum annual Flow to South Australia for period 1989 to 2009 under modelled and historical scenarios

As demonstrated by Figure 3.1, the following observations can be made when comparing modelled flow scenarios and the historical flow record:

- Apart from the extreme dry years, the maximum flow that occurs in any year is reduced compared to what would have occurred naturally ('without development' conditions). This is true for both the "recorded historical" and "modelled Basin Plan" scenarios.
- The "modelled Basin Plan" scenario typically increases the size and frequency of flows in the range of around 10 000 ML/d to 80 000 ML/d compared to what would have occurred without the Basin Plan. This can be attributed to planned environmental watering events for floodplain wetlands.
- The size of floods (greater than around 80 000 ML/d) are similar under the Basin Plan scenario compared to what occurs under current conditions.

As with any modelling, MDBA modelling contains some inaccuracies and uncertainties due to gaps in the data record, assumptions and simplifications (MDBA 2012). The models that have been linked together in the basin-wide framework have been calibrated to a range of flows, however it should be noted that these models have been primarily developed for water resource management and not flood forecasting. Sources of uncertainty include the modelling of some upper catchments using weekly or monthly time steps (such as Goulburn, Broken, Campaspe, and Ovens Rivers) (MDBA 2012), and the accuracy of rating curves to convert water levels into flows which have been used in the calibration of the models. Modelling undertaken for the Basin Plan did not incorporate projections of climate change.

The flood frequency analysis is based on the methodology contained within the draft Australian Rainfall and Runoff chapter on Estimation of Peak Discharge (AR&R, 2006). The current guideline for undertaking flood frequency analysis is contained in Australian Rainfall and Runoff (1987). The 1987 version recommends the use of the Log-Pearson 3 probability distribution for fitting a statistical distribution to rainfall records. A draft version of the new guideline was produced in 2006. The revised guidelines are similar to that of AR&R (1987), but now recommend the use of both Log-Pearson 3 (LP3) and Generalised Extreme Value (GEV) probability distributions. The revised guidelines also recommend the use of Bayesian methods for fitting the distribution, which enables the inclusion of historic floods outside the period of gauged record, even if the exact height or flow are unknown, as well as censoring lower flows within the gauged record because they may distort the fitting process. For this study, the software FLIKE was used (Kuczera, 2001), which applies a Bayesian fitting process as recommended by the draft Guidelines (AR&R, 2006).

A flood frequency analysis was previously undertaken as part of the Renmark Paringa Flood Mitigation Study (Maunsell 2008). Flood frequency estimates generated by that study are presented in Table 1 for comparison.

Table 3.1 Flood frequency analysis from Maunsell 2008

Annual Exceedance Probability (1 in Y)	Annual Exceedance Probability (%)	Flow at SA border (ML/d)
20	5	114 000
50	2	169 000
100	1	235 000
200	0.5	335 000
500	0.2	547 000

3.2 Input data

The Basin Plan data used as the basis for the flood frequency analysis is shown in Figure 3.2. The values used are the maximum flow occurring in each year (known as the annual series), ignoring smaller peaks that may occur in the year. When determining the maximum flow in each year, the water year is used (spanning dry season to dry season) rather than the calendar year. In South Australia, most floods in the River Murray occur during spring or summer while the lowest flows typically occur in autumn, so a water year of May to April has been used.

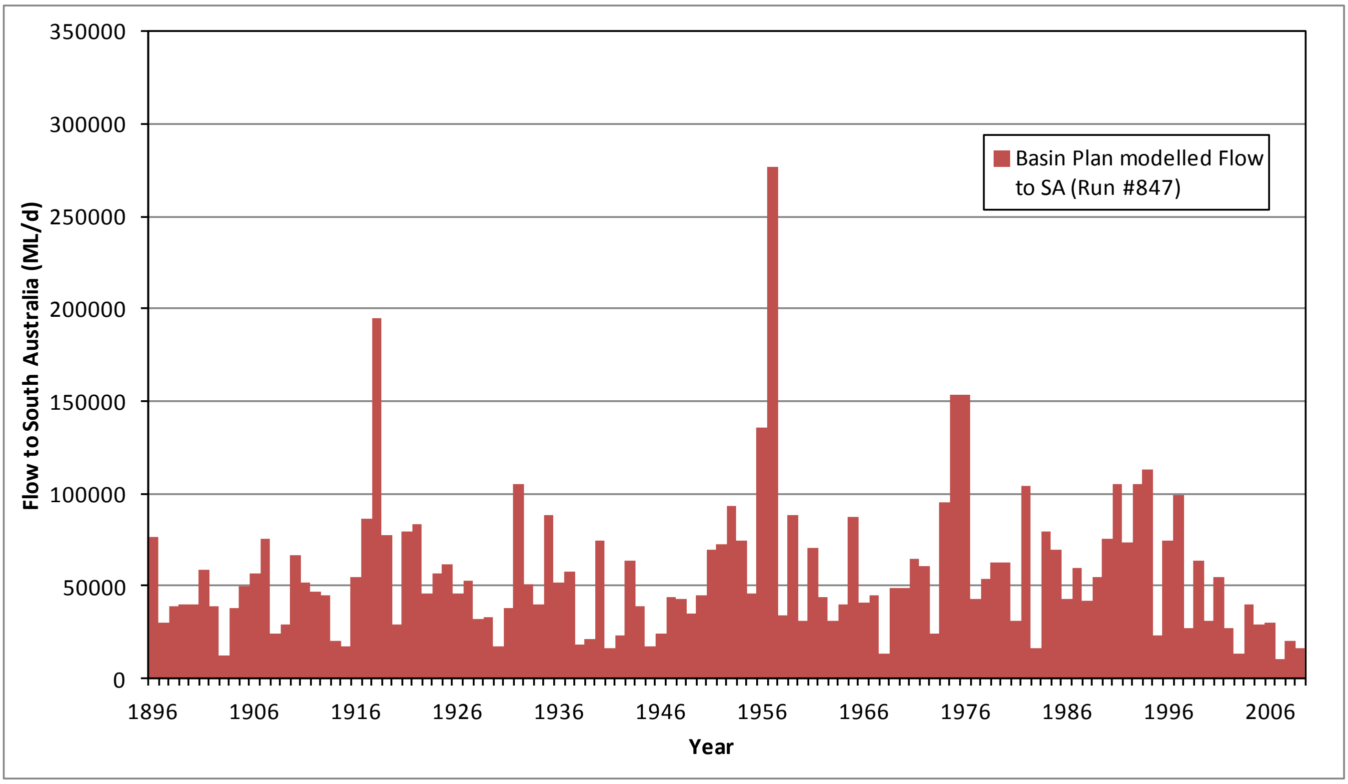

Figure 3.2 Modelled peak flow to South Australia with Basin Plan

It is interesting to explore what the hydrological modelling has predicted the size of floods would have been under the Basin Plan scenario compared to what would have occurred naturally. The MDBA's 'without development' model approximates near-natural river flows without any dams, weirs or extractions (run number 844).

Figure 3.3 shows a comparison of the peak annual flow between the Basin Plan modelling scenario and the 'without development' modelling scenario. If the models produced the same value for a given year, the points would be placed along the 1:1 line on the graph. However, nearly all of the points are placed below this line, indicating that in nearly all cases maximum annual flows in the Basin Plan scenario are less than what would have occurred naturally.

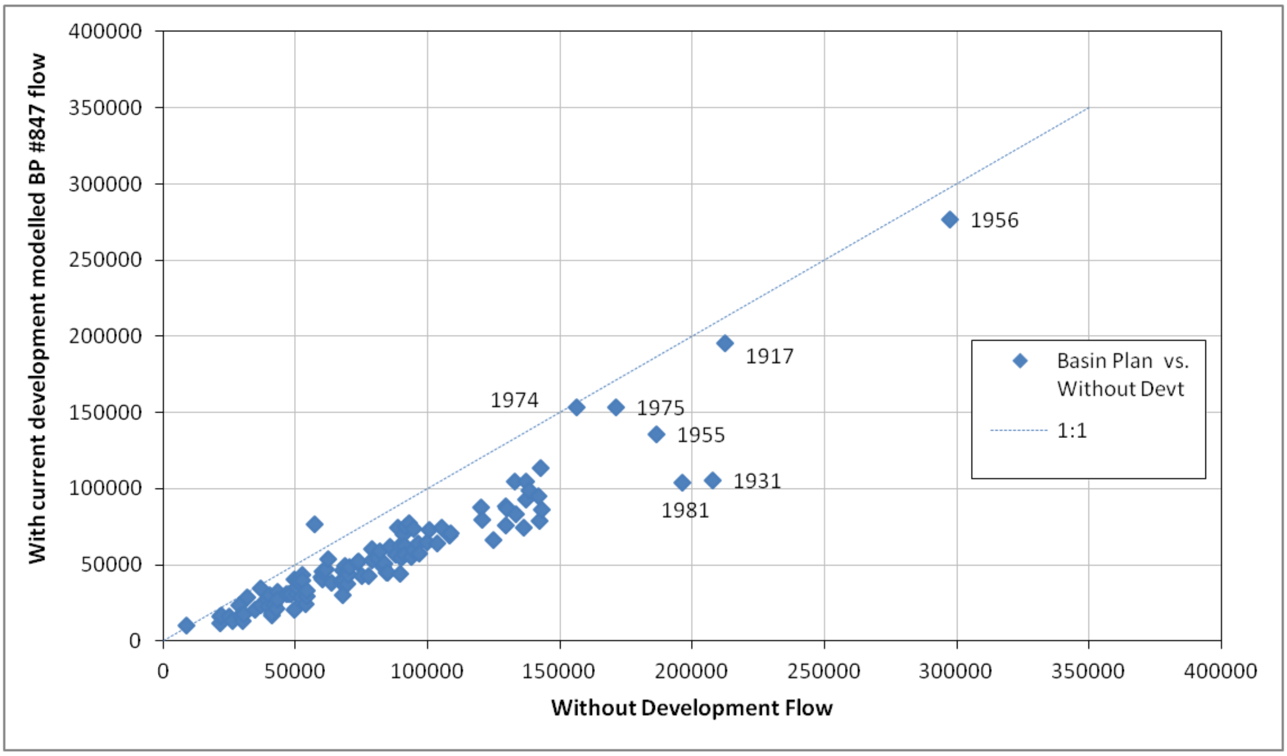

Figure 3.3 Effect of development on flows (ML/d) reaching South Australia

For the flood years of 1956, 1917, 1974 and 1975, the Basin Plan flow is only a small amount less than the 'without development' flow (indicated by points located close to the 1:1 line). This means that in these years, modern basin development (such as dams and irrigation) have not had a large flood mitigation impact. The largest event in the time series, the 1956 event, is mitigated slightly to 93% of the without development flow. However, for other flood years (1955, 1981 and 1931) there is a large reduction between 'without development' and Basin Plan flows. For example, the flood in 1931 would have been 208 000 ML/d, but modelling that uses the dams and water sharing rules in place under the Basin Plan predicts that the same rainfall would result in a flood of only 106 000 ML/d.

It is expected that generally the mitigating effect of dams is reduced as the size of the flood increases, as the flood storage capacity of the dams would be overwhelmed for very high rainfall events. The mitigating effect of dams is also linked to how much rain has fallen in the preceding year since this influences how full the dams are prior to the flood-producing rainfall event.

The MDBA modelling of without development conditions simulates the 1956 event to be 297 000 ML/d which is considerably less than the flow of 341 000 ML/d recorded during the event. As there were already regulating structures in place in the catchment during the 1956 event it would be reasonable to expected that the without development modelled flow would be higher than that recorded during the event.

Stephens (1977) indicates that estimates of flow at the South Australian border are based on gauging at Lock 9 and Lake Victoria Outlet, and this method has been in use since the 1930s. The water level measurements and subsequent extrapolation using the established rating curves and weir formula for calculating discharge, used during the 1956 flood, were shown to be consistent with measurements taken during the floods in the 1970s. It is considered that the recorded flow is likely to be more reliable due to the limitations in the modelling such as the use of monthly/weekly time steps in some catchments and uncertainty regarding accuracy of rating curves at very high flows to which the models were calibrated. Therefore, this analysis has used the recorded flow for the 1956 event of 341 300 ML/d in place of the modelled output of 297 000 ML/d.

One of the assumptions of the flood frequency analysis methodology is that the flow events are independent, that is, the size of a flood in given year is not affected by the size of floods in previous years. For most unregulated river catchments, the use of an annual series would mean that each event would be independent. However, in the Murray–Darling Basin there are numerous dams with large storage and flood mitigation capacity. A link has been observed between the mitigation of without development flows and annual basin inflows, suggesting that annual maximum events may not be truly independent. The alternative method for determining the probability of flow events is to use hydrological models with rainfall-runoff modelling of design storm events.

However, hydrological models of the Murray–Darling Basin are not able to produce flood estimates by this method due to its size and complexity. Therefore, flood frequency analysis is considered to be the most appropriate method available.

Flood frequency analysis of other rivers in Australia have based the approach on using ‘without development’ flow series and subsequently applied a reduction factor to account for the mitigating effects of dams (see for example WMA 2011). That method was not adopted in this case due to the highly variable degree of mitigation, as demonstrated in Figure 3.3.

One way to improve the accuracy of flood frequency analysis is to include more historical floods. Knowledge of large floods that occurred prior to the modelled period (pre 1895) was described in Maunsell (2008). A major flood occurred in 1870 with a flow of 318 000 ML/d (E&WS, 1987). However, there are patchy observations of flood heights recorded between 1870 and 1895. Maunsell (2008) provides a discussion of River Murray floods that occurred between 1870 and 1895 and how these can be included in a flood frequency analysis. A similar approach has been adopted for this analysis, the 1870 flood event has been included and the intervening years (1871 to 1895) have been counted as 24 years with maximum flows less than 130 000 ML/d.

3.3 Results

Flood frequency analysis was undertaken using the software FLIKE. The best fit to a probability distribution was determined by visual inspection (including how well points fit within the 5% and 95% percentiles), the closeness of fit to the largest events (e.g. 1956, 1931) and the width of confidence limits as a percentage of the flow estimate.

Several trials were undertaken such as with and without the inclusion of the 1870 historic flood event, various thresholds for censoring low flows, and the use of the Generalised Extreme Value (GEV) or the Log-Pearson 3 (LP3) probability distributions (PD).

Fitted probability models and estimated flow events are shown in Figures 3.4 to 3.7 for four trial combinations of probability model and flow events:

1. GEV PD with all modelled flows (GEV Complete)
2. LP3 PD with all modelled flows (LP3 Complete)
3. GEV PD with flows below 71 000 ML/d excluded (GEV Censored)
4. LP3 PD with flows below 71 000 ML/d excluded (LP3 Censored).

The censoring of low flows enables the probability distribution to achieve a better fit to higher flows without being overly influenced by the lower portion of the hydrograph, which are not of interest in flood frequency analysis. All four trials include the 1870 historical event (not shown on graphs), as this improved the fit of the probability distribution and width of confidence limits at the higher range.

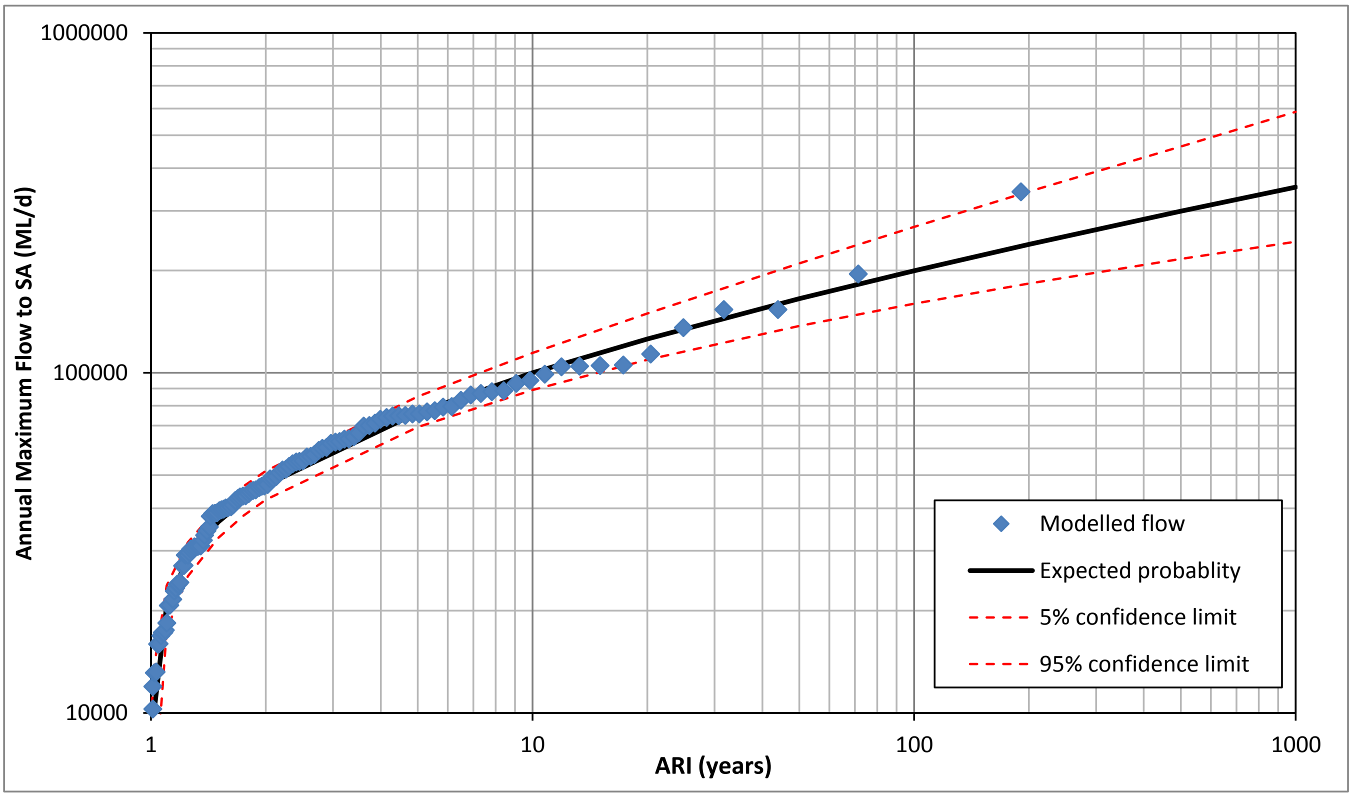

Figure 3.4 GEV probability distribution with entire 1875 to 2009 time series, including 1870 historic event

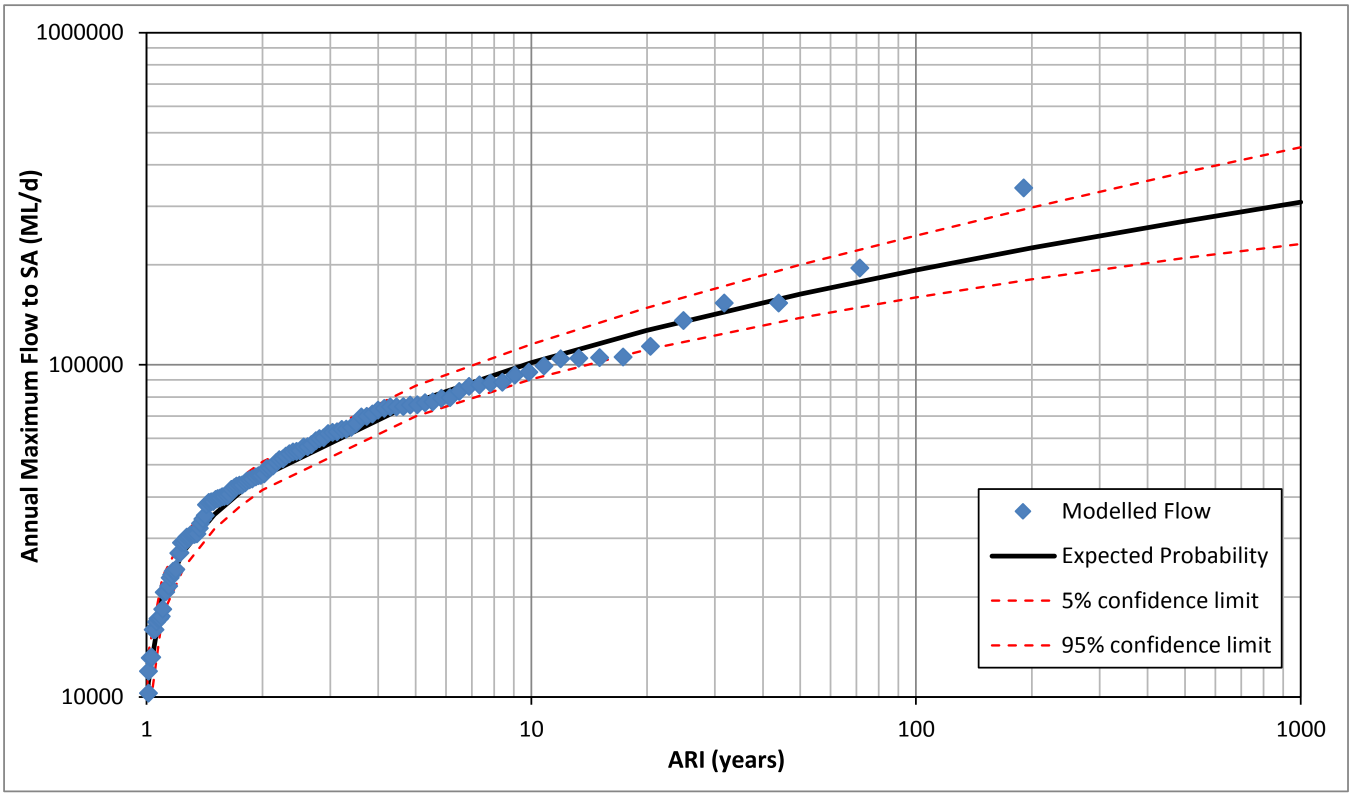

Figure 3.5 LP3 probability distribution with entire 1875 to 2009 time series, including 1870 historic event

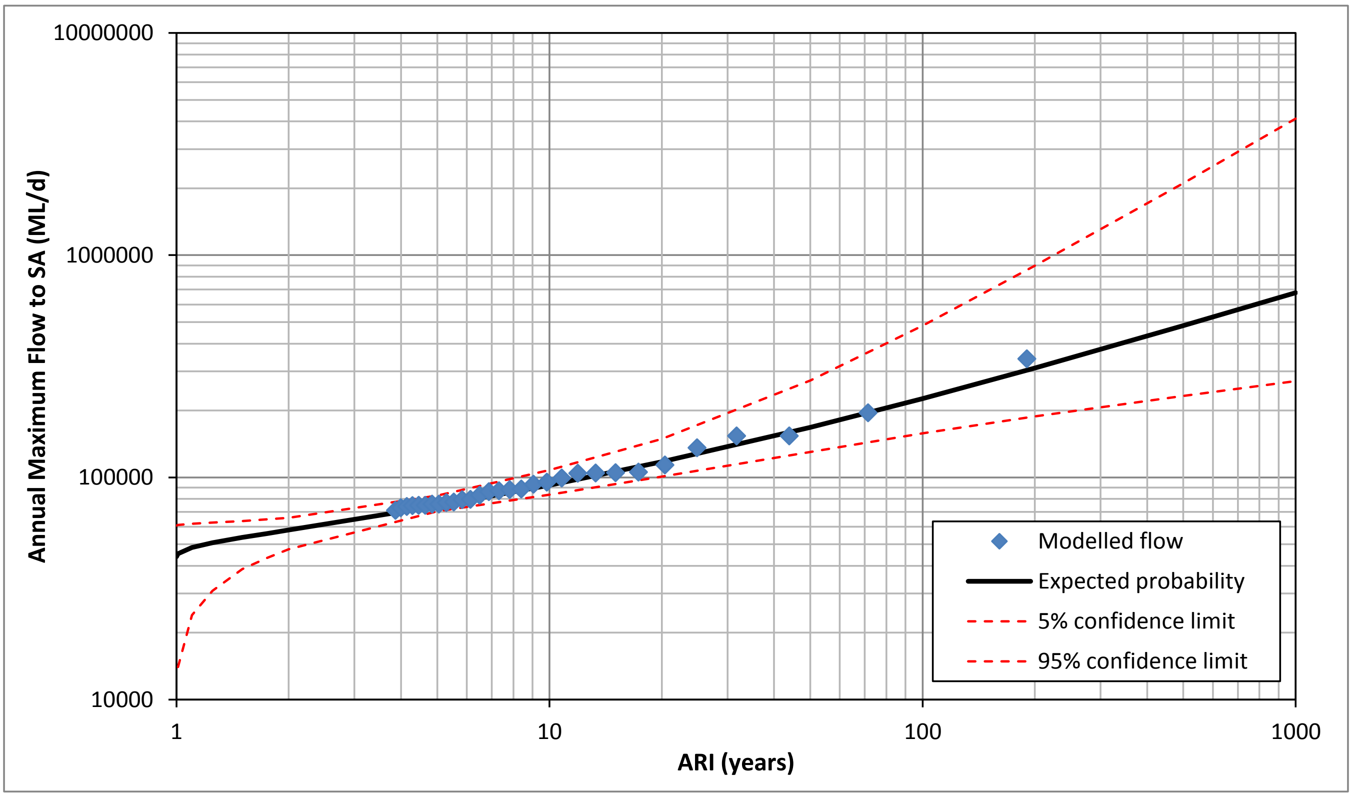

Figure 3.6. GEV probability distribution and flows below 71 000 ML/d censored, including 1870 historic event

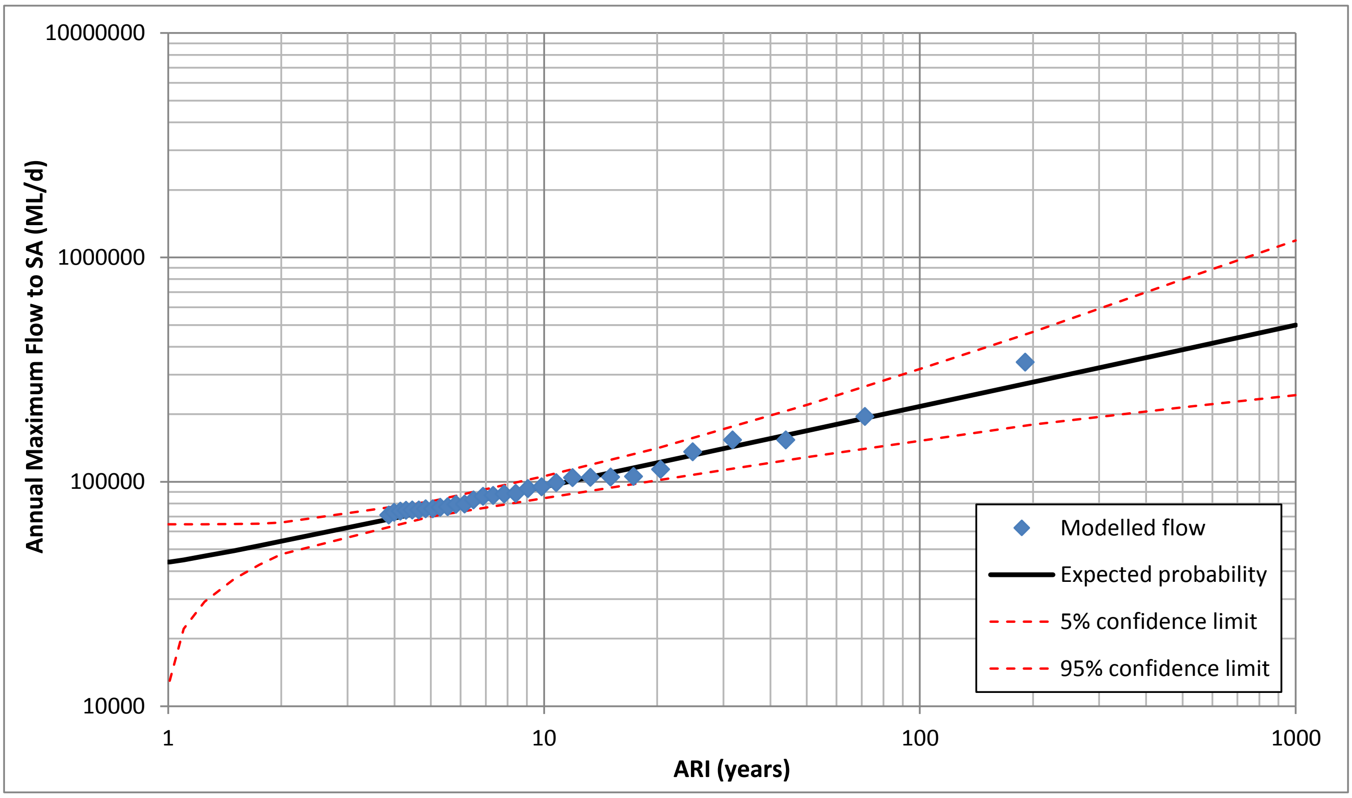

Figure 3.7. LP3 probability distribution and flows below 71 000 ML/d censored, including 1870 historical event

Table 3.2 Flow estimates from trials of different probability distributions and input data

AEP (1 in Y)	Flow estimate (ML/d)			
	GEV Complete	*LP3 Complete*	*GEV Censored*	*LP3 Censored*
20	126 000	127 000	117 000	121 000
50	165 000	163 000	168 000	169 000
100	200 000	193 000	226 000	217 000
200	239 000	225 000	310 000	278 000
500	299 000	271 000	481 000	388 000

It can be seen in Table 3.2 that the difference between the different trial combinations is small for AEPs within the length of the modelled data record, i.e. for AEPs more frequent than a probability of 1 in 100. The influence of the distribution fitted to the data had a greater influence on the flow estimated for events occurring with a lower AEP than this due, to the need extrapolate the distribution. Based on a visual inspection of the fit between the probability distribution and the expected probability line it was considered that the GEV Censored represented the best fit for predicting events rarer than 1 in 100 AEP. This approach was also selected as it provided the most conservative estimate of the flow occurring with a low AEP. The flood frequency estimates from this distribution are shown in Table 3.3 and Table 3.4 below.

Table 3.3 Flow estimates from flood frequency analysis

AEP (1 in Y)	Flow estimate (ML/d)	Confidence limits	
		5% Limit	*95% Limit*
20	117 389	100 731	148 868
50	168 128	130 189	273 030
100	226 446	157 630	482 318
200	310 530	187 888	894 995
500	481 406	232 444	2 109 178

Table 3.4 AEP estimates of selected flows

Flow (ML/d)	AEP (1 in Y)
130 000	26
200 000	75
250 000	125
341 000 (1956)	250

3.4 Discussion

The results of flood frequency analyses should be seen as an indicative 'point in time' estimate. Invariably, flow estimates will change in the future as rainfall-runoff modelling and analytical techniques improve and new data becomes available. The scarcity of very large events means that it is difficult to achieve a reliable 'fit' to the upper portion of the flow series, and the process itself also contains some subjectivity in terms of visually fitting modelled flow to probability distributions. The use of a 114 year record means that the estimated AEP is based on extrapolating a mathematical distribution and the values presented are influenced by this subjectivity around the distributions used and associated parameters.

The flow estimates provided are for Flow to South Australia. Some reduction in flow may occur between the border and downstream locations due to attenuation (reductions in peak due to water spreading out to wetlands and floodplain) and losses (seepage and evaporation). It is unlikely that flows will increase significantly between the SA border and Wellington since tributaries are small.

Based on modelling undertaken by the MDBA, new water management strategies under the Basin Plan will enhance flow events up to approximately 80 000 ML/d at the South Australian border using recovered environmental water, either by increasing the peak flow of an event or extending its duration. The MDBA modelling assumed that flow enhancement would not occur for larger events due to risks of flooding and reduced ability to control high flows using river infrastructure.

The Basin Plan modelling also assumes that the recovered water is used each year to deliver environmental events without being held in storage for extended periods. Consequently, the modelled frequency of major unregulated events is similar between current conditions and the Basin Plan scenario since average dam storage levels are similar. In reality, the frequency of floods will be affected by the timing of environmental releases and whether carryover of environmental water occurs. The effects of achieving the sustainable diversion limit, which is defined in the Murray–Darling Basin Plan (MDBA, 2012) through irrigation improvements rather than buyback of irrigator entitlements on dam storage levels and dam spills is unknown.

Flood mitigation is one way of reducing flood risk by reducing the likelihood of flow events. However, for many cities in Australia which are built on floodplains, flood mitigation is not able to entirely eliminate flood risk altogether. It is important to convey to the community that locations outside of particular design events or planning lines are not without risk or flood proof. For planning purposes, it is necessary to designate a level of risk that is acceptable to the community. Thus, many communities use the 1 in 100 AEP event, however in some cases the largest event on record is adopted. For planning purposes along the River Murray in South Australia, typically the 1956 flood extent is used. However, there are exceptions. In the RPC Development Plan, there are locations where the flood zone is delineated by levees built to the 1931 flood level, which is a lower standard of protection. Previous studies have estimated the AEP of the 1956 event to be 1 in 160 (E&WS, 1987) and 1 in 170 (Maunsell, 2008), while the flood frequency analysis in this study estimates it to be 1 in 250. This range highlights the uncertainty involved in estimating the probability of occurrence of very rare events.

In summary, the following uncertainties and assumptions related to the flood frequency analysis should be noted:

1. The MDBA hydrological modelling results used in this study contain some uncertainties, due to gaps in the data record, model assumptions and simplifications. These can for example be related to the representation of the entire Murray–Darling Basin resulting in fewer small-scale details in the model and to the purpose of the model being for water resources estimation and not flood forecasting.
2. This study has utilised the time series from the Basin Plan 2800 GL reduction scenario as the best estimate of future water management conditions in the Murray–Darling Basin.
3. The flood frequency analysis methodology assumes that the flow events are independent, that is, the size of one event is not affected by the size of other events. In the Murray–Darling Basin there are numerous dams with large storage and flood mitigation capacity, suggesting that annual maximum events may not be truly independent.
4. The scarcity of very large events means that it is difficult to achieve a reliable fit to the upper portion of the flow series, and the process itself also contains some subjectivity in terms of visually fitting modelled flow to probability distributions. The use of a 114 year record means that the estimated AEP above approximately 1 in 100 is based on extrapolating a statistical distribution and the values presented are influenced by this subjectivity around the distributions used and associated parameters. For the smaller flood events, for example 1 in 20 the confidence bounds are narrower, as there are more events to fit the statistical distribution to.

4 Consequence assessment methodology

4.1 Overview of flood consequence assessment

A flood consequence assessment is an estimate of the economic costs of flood damage from specified flood events. This information is useful for understanding the level of risk to the community, as well as being an input to cost-benefit analyses to determine if mitigation works are justified. .

In-line with the standard approach in estimating flood damage costs, a distinction has been made between three types of flood damage in this report (Reed Sturgess and Associates, 2000) (Figure 4.1). The three types of flood damage are:

- **Direct (tangible) damages** comprise the physical impact of the flood, for example, damages to structure and contents of buildings, agricultural enterprises and regional infrastructure.
- **Indirect (tangible) damages** comprise losses from disruption of normal economic and social activities that arise as a consequence of the physical impact of the flood; for example, costs associated with emergency response, clean-up, community support, as well as disruption to transport, employment and commerce.
- **Intangibles**, or non-market impacts, comprise losses which cannot be quantified in monetary terms (since market prices cannot be used). For example, loss in biodiversity or increased stress levels for residents following a major flood event affecting their homes.

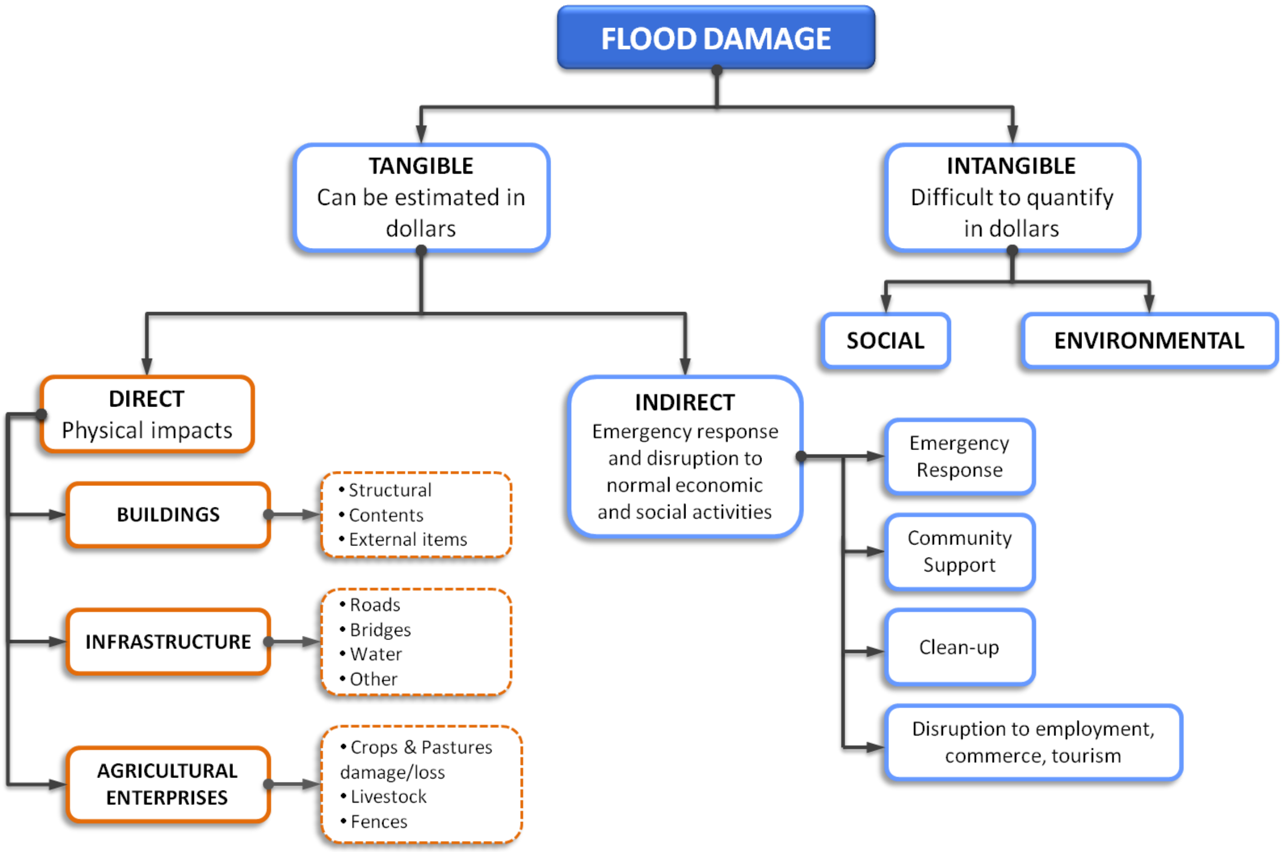

Figure 4.1 Types of flood damage costs (Read Sturgess and Associates, 2000, p. 12)

This assessment only includes direct, tangible flood damages, such as loss of property that can be represented as a monetary value (Orange boxes in Figure 4.1). It does not attempt to quantify intangible or indirect damages, for example, loss of life and mental anguish.

The steps involved for determining the total value of damages from flooding is shown in Figure 4.2. Each of these steps is described in more detail in following sections.

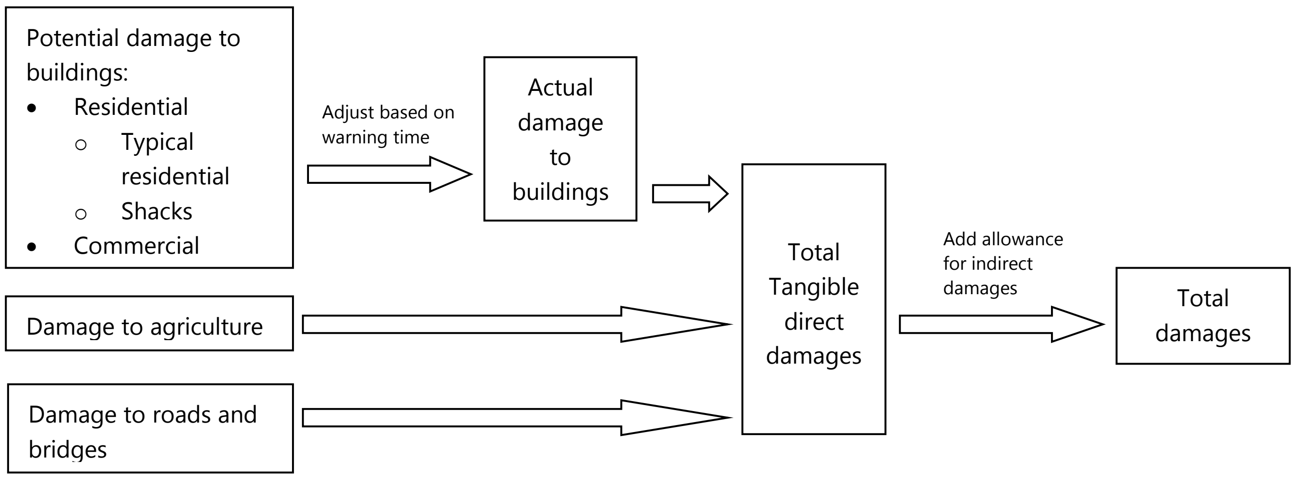

Figure 4.2 Process for estimating flood consequences

4.2 Scenarios

Tangible flood damages have been estimated for the following damage scenarios:

1. The whole of the River Murray floodplain between the South Australian border and Wellington
2. Renmark Paringa Council area only
3. Renmark Paringa Council area only, but assuming that the levee banks have failed. This can be used for future analysis to determine if it is economically feasible to repair levee banks that are in poor condition.

The first two scenarios included levee banks in place, and only overtopping of the levee banks occurs if applicable. For Scenario 3 the levee banks were removed from the model, to simulate the inundation expected if the levee banks failed completely due to mechanisms occurring when the water level is below the levee crest (e.g. slumping or piping).

4.3 Information sources

In this study, the flood inundation maps generated from hydraulic modelling (described in Section 3) were used to provide water levels and flood extents for the various flow scenarios considered. GIS spatial analysis was used to compare modelled flood heights with ground level and property data to calculate inundation depths and identify the types and number of properties and roads inundated. This information was used to determine the damage costs for properties, road and agricultural infrastructure for the South Australian River Murray floodplain and for the Renmark Paringa Council area.

As the size of the flood increases, the amount of damage or disruption to normal activities increases while the probability of that event occurring decreases. The combination of the probability of flood event and the damage it causes, averaged over all years, and is known as the average annual damage. Ideally, the average annual damage should be calculated using a range of flood events up to the Probable Maximum Flood. For this assessment, limits of available flood event data and practicality reasons means that damages have been calculated for four flood events. It has been assumed that damages from other flow events can be calculated by interpolating between these events. The events chosen for the flood damage assessment are Flow to South Australia of 130 000 ML/d (corresponding to the moderate flood level trigger), 200 000 ML/d and 250 000 ML/d (corresponding to the modelled flows nearest to the 1 in 100 AEP flood and for which there is a risk of flooding in Renmark) and 341 000 ML/d (corresponding to the largest flood on record).

The methodology and costing adopted for this study have largely been drawn from documentation developed for Victoria and Queensland (Reed Sturgess & Associates, 2000; URS, 2002 and Queensland Government, 2002). No single approach was fit-for-purpose in this study due to the availability of appropriate data to fit the documented methods. Different methods, from the above references, for calculating damage costs for residential, commercial and agricultural properties, and roads/bridges were adopted commensurate with the data available for the River Murray in South Australia. The damages estimates for the different components were all converted to Australian dollar value in 2012 for consistency.

The following GIS layers of infrastructure data and property data were used to identify inundated buildings, property, infrastructure and roads:

- 2008 Land Use mapping to determine the area of agricultural land inundated. Data sourced from DEWNR's GIS Database in December 2012. Layer name, GIS.ADMIN.Landuse_South Australia_2008. Refer to 'Guidelines for land use mapping in Australia: principles, procedures and definitions, prepared by the Bureau of Rural Sciences, Commonwealth of Australia 2006'.
- Road spatial data to determine the length of road inundated. Ownership information is also included in this spatial layer, which has been used to determine whether a road is Council owned. This data was originally sourced from BTI Branch in December 2012, but is now available through DEWNR's GIS Database. Layer name, TOPO.Roads.
- Structure point spatial layer generated using ArcGIS via a visual inspection of houses, shacks and various structures visible in 2008 and 2011 aerial photography of the River Murray. Data prepared in 2012. Layer available through DEWNR Science, Monitoring and Knowledge Branch.
- River Murray Properties Valuation Data supplied by the Land Services Group, Department of Planning, Transport and Infrastructure in December 2012. As this information is continuously updated, contacting the Land Services Group is recommended if data is further required.

The structure point layer and property valuation data were analysed to determine the approximate value of residential properties (including shacks showing inundation extent for 130 000 ML/d, 200 000 ML/d, 250 000 ML/d and 341 000 ML/d) at risk of inundation. For commercial properties a different approach was required (described in Section 4.5).

Flood intelligence layers previously developed for the DEWNR "Implications of Environmental Water Delivery – Feasibility Investigation" (Cetin et al., 2013) have been further developed to include inundation information for council assets at risk.

4.4 Potential damages to residential properties

Many methods for estimating flood damages for residential properties relate cost to depth of overfloor inundation, that is, the greater the depth of flooding is through a house, the more costly it is to repair. There is a lack of recent published flood damage costs for individual properties despite the number of major flooding events in recent years. It was therefore difficult to source up-to-date residential flood damage costs from the available literature. The damage curves cited in URS (2002) were adopted for this study. The URS (2002) damage curves relate depth of overfloor inundation to the percentage of insured contents and structure value damaged as a result of the inundation. The depth of overfloor inundation is calculated by subtracting the floor level of structure from the modelled water level at that location. These curves are shown in Figure 4.3.

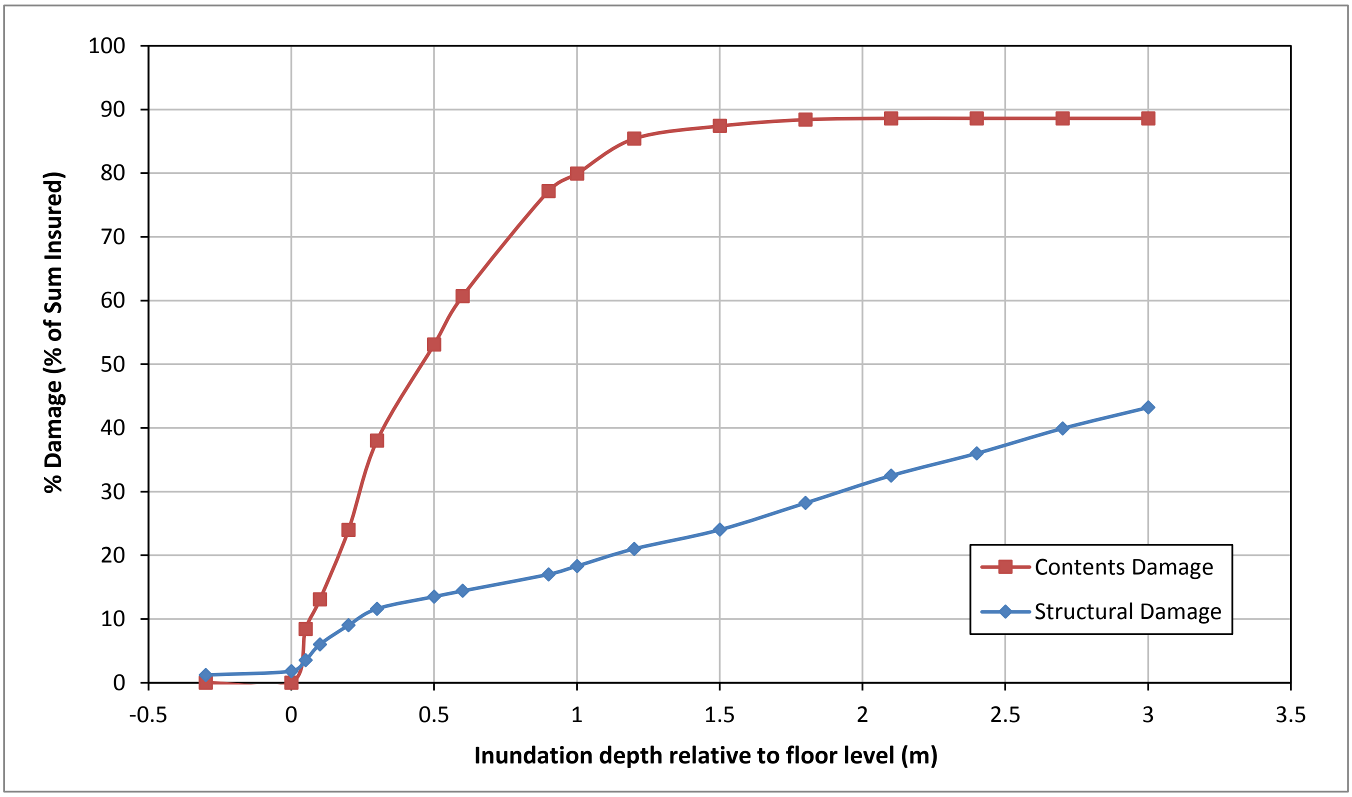

Figure 4.3 Structural and contents damage (URS 2002 from preliminary figures supplied by R. Blong, Risk Frontiers – Natural Hazard Research Centre)

There are over 3000 holiday shacks located within the SA River Murray floodplain, the majority of which are in the Mid-Murray Council area between Morgan and Mannum. Shacks have been built on the floodplain since the early twentieth century and are predominately high-set buildings due to the frequency of flooding. Since the 1990s, building requirements for new development requires that habitable floor levels are elevated above the 1956 flood level with only limited enclosure of the ground floor of high-set buildings to allow free passage of flood waters. Some shacks constructed prior to this time are built on ground level or have enclosed ground floor areas. While these structures are referred to as shacks, many newer buildings are well-appointed holiday homes. Figure 4.4 shows a photograph of typical shacks taken during the high flow event in early 2011.

Figure 4.4 Shacks inundated in 2011 high flow event (93 000 ML/d at SA border; 79 000 ML/d at Lock 1)

The structure point layer distinguishes structures likely to be shacks (generally located within the 1956 flood line and between Morgan and Mannum) and other minor buildings and structures such as sheds and caravans. Residences and shacks have been considered separately in this assessment since typically shacks are highset, the structure and contents are lower in value (due to their use as temporary not permanent residences) and owners are more flood aware.

For residences, the floor level was assumed to be 0.3 m above ground level. For shacks, the floor level was assumed to be 3.6 m above ground level.

For structural damage, the total value of each structure was obtained from the property valuation dataset. The structural value was assumed to be capital value less site value. Capital value is the value of the land including all improvements permanently attached to the ground (such as buildings and sheds). The Capital value is used by rating authorities as the basis for the levying of rates, taxes and other imposts. The value is determined annually and based on the analysis of market evidence. Site value is the value of the land including site improvements (such as levelling, retaining walls and clearing of timber) but excluding structural improvement (GOSA Housing, Property and Land website, viewed April 2013).

For contents value, the Government Insurance Office, Insurance online contents assessor tool, was used to obtain typical values for the Riverland region (GIO, 2013). This tool takes into account the location (post code) of properties, the size of residential property and typical occupancy to estimate the costs to replace contents of an individual residence. From the use of this tool, a contents value of $70 000 was adopted for residences, while a contents value of $30 000 was used for shacks since these are usually holiday homes and not permanent residences.

4.5 Potential damages to commercial properties

Damages for commercial properties are very difficult to estimate since contents of commercial and industrial properties can vary enormously between different business types. In particular, stock such as perishables, electrical items or paper-based products is particularly vulnerable (Gissing, 2001). The method documented for assessing commercial property damage in URS (2002) was adopted, which relates flood damages on a per square metre basis to depth of overfloor inundation. This method utilises estimates of total direct damages derived by Gissing (2001) rather than separate contents and structure damages due to the vast variation in the value of contents. Available datasets of properties in the study area did not contain information on floor area so an assumed floor area of 200 square metres was adopted for each property for the analysis. The Gissing (2001) cost estimated were converted to represent the dollar value for financial year 2012–13 using the consumer price index (CPI) values published by ABS (2012). The adopted relationship for depth of inundation to damages is shown in Figure 4.5.

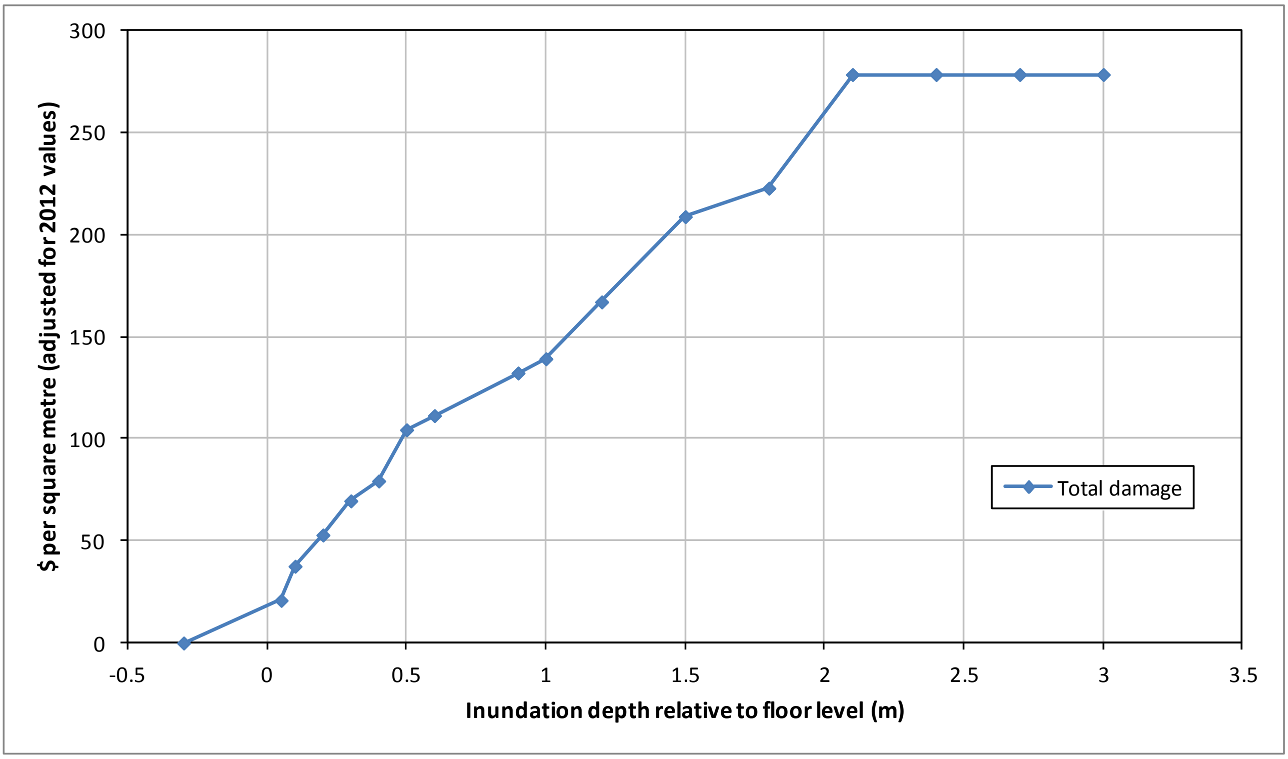

Figure 4.5. Total commercial damage in dollar value for FY 2012–13 (converted from URS 2002, based on Gissing 2001)

4.6 Adjusting potential damages for buildings to actual damages

The damage estimates provided in the previous sections indicate potential tangible damages, that is, the amount of damage that could occur if no actions were taken to reduce the amount of damage, such as residents placing valuables up high or removing to other locations. Studies have shown that the actual damages are usually less, with the reduction in damages dependent on the level of flood awareness of the community and the amount of warning time received.

There is generally four to six weeks warning time ahead of flooding in the SA River Murray. For communities with warning times of greater than 12 hours, the ratio of actual to potential damages is estimated to be 0.4 for experienced communities (those that have experienced flooding in the past five years), and 0.7 for inexperienced communities (Reed Sturgess & Associates, 2000). This assessment has assumed that the shack communities are experienced and flood aware since they experience more frequent flooding due to the their location lower on the floodplain. The remainder of the community is assumed to be inexperienced. The adjustment does not apply for roads and bridges and agriculture, and it has also been assumed that substantial mitigation works such as raising or erecting temporary levees would not be undertaken.

A higher ratio of actual to potential damages was used for the levee bank failure scenario for the RPC area, since it was considered likely that under a levee failure scenario only a few hours warning would be received. Assuming a warning time of between two and 12 hours and an inexperienced community, a ratio of 0.8 has been used (Reed Sturgess & Associates, 2000).

4.7 Damages to agricultural properties

The cost of flood damage for agricultural properties was estimated using the methodology described in the Rapid Appraisal Method for Floodplain Damages (Reed Sturgess and Associates, 2000). This method provides estimates of agricultural damages based on area, time of year and duration of inundation. The method has been developed for Victoria, but for this study it was assumed that crop types, values and seasons are also applicable to the River Murray floodplain in SA due to the similar crop types, climate and this was the best available data at the time of this study. Damages have been updated to 2012 values using CPI (ABS, 2012). Damage estimates have been used for inundation durations greater than one week. The mapping of crop types available in the Rapid Appraisal method to the crop types identified as inundated on the floodplain (defined by the Agricultural Land Use Management (ALUM) classification) used is in Table 4.1. Damages by crop type and month are shown in Figure 4.6.

Table 4.1 Alignment of Rapid Appraisal Method categories and ALUM classifications

Rapid Appraisal Method categories	Corresponding ALUM classification
Dryland pastures	Grazing modified pastures
Irrigated pastures	Irrigated modified pastures
Dryland broadacre crops	Cropping
Irrigated broadacre crops	Irrigated cropping Irrigated land in transition
Vegetables	Irrigated seasonal horticulture Seasonal horticulture
Grapes Flood sensitive orchard	Irrigated perennial horticulture*

* For irrigated perennial horticulture average value for Grapes and Flood sensitive orchards have been adopted.

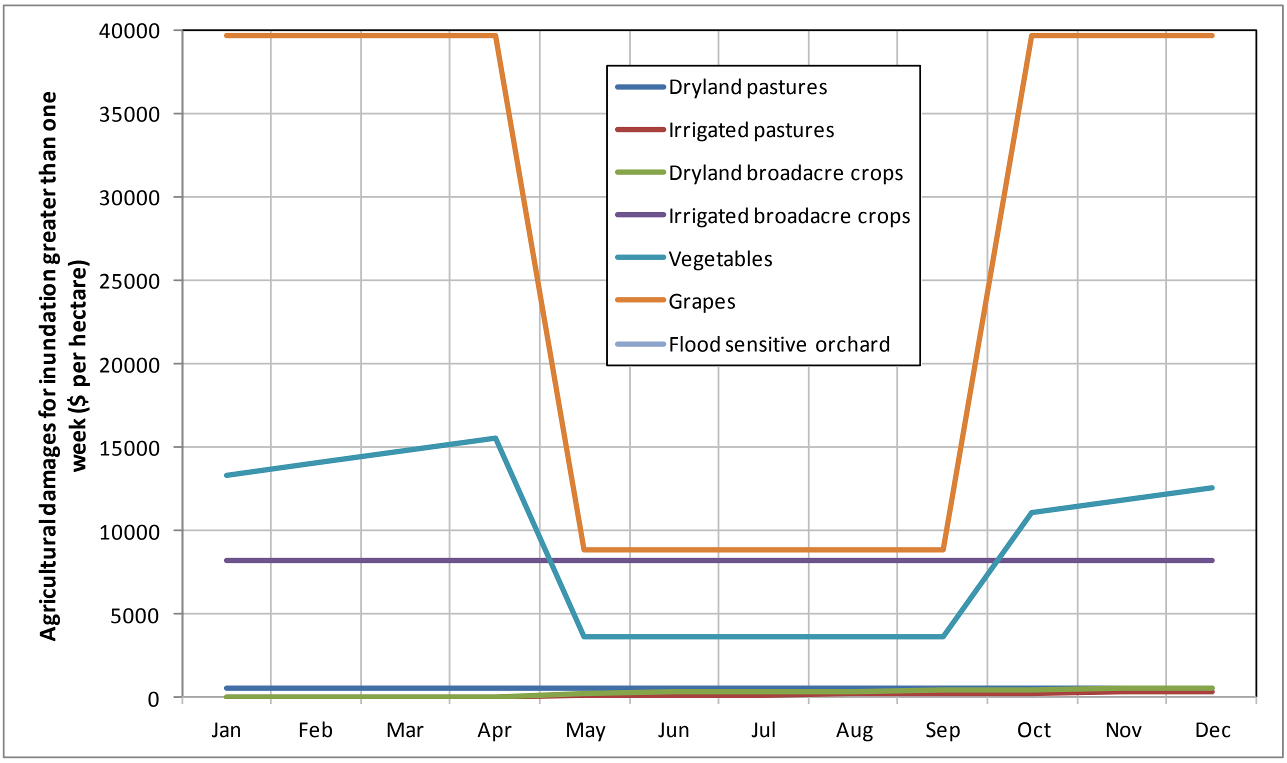

Figure 4.6. Agricultural damages from Rapid Appraisal Method for Floodplain Management (Reed Sturgess & Associates, 2000, p28, Table 3-8).

4.8 Damages to roads and bridges

Repair costs for roads and bridges have been sourced from Reed Sturgess and Associates (2000) and adjusted to 2012 dollar values. The values are expressed on a per kilometre basis, taking into account the initial costs of temporary repairs and the subsequent damage to roads and bridges that become apparent after a prolonged time after the flood event. These are presented in Table 4.2. The unsealed roads value was used for both public unsealed roads and public tracks.

Table 4.2 Unit damages for roads and bridges (expressed as dollars per kilometre of road inundated) (Reed Sturgess & Associates, 2000, adjusted to 2012 dollar values)

Road type	Initial roads repair ($)	Subsequent deterioration of roads ($)	Initial bridge repair and subsequent increased maintenance ($)	Total cost to be applied per km of road inundated ($)
Major sealed roads	47 100	23 500	16 200	86 800
Minor sealed roads	14 700	7400	5200	27 200
Unsealed roads	6600	3300	2400	12 300

4.9 Allowance for indirect damages

Indirect damages are those that are a secondary consequence of the damage caused by flooding and include costs such as emergency response, clean-up and disruption of business and employment. Indirect damages are estimated to be 30% of direct damages (Reed Sturgess & Associates, 2000). The percentage adopted is expected to be representative of the possible range of indirect damages as a proportion of the direct damages, for example BTE (2002) indicate a range of 25–40% is typical.

4.10 Average annual damages

As the size of the flood increases, the amount of damage or disruption to normal activities increases while the probability of that event occurring decreases. The Average Annual Damage (AAD) is an indicative measure of the long term average flood damages to an area, and is equivalent to the total damage caused by all floods over a long period of time divided by the number of years, assuming that the population and extent of development does not change (CSIRO, 2000). AAD is useful for evaluation of mitigation measures and determining relative priorities between flood risk areas.

The AAD can be determined by plotting a curve of the damage caused by various annual events against the probability of that event occurring (a consequence-probability curve). The AAD is then equal to the area under the curve (CSIRO, 2000). Theoretically, the consequence-probability curve should be a continuous curve developed from analysis of all events, but this is unrealistic in practice and typically only a selection of events is considered.

4.11 Discussion

The consequence assessment should be considered indicative of the likely magnitude of flood damages rather than a precise assessment since only desktop methods were utilised and many assumptions were required in the adopted methodology relating to the extent and nature of damages.

Ideally, flood damage estimates would be based on each individual building or property characteristics (e.g. floor height, area, type of structure, type of building materials, specific contents costs) to give the most accurate assessment possible of the potential costs associated with flooding. However, current information of this kind is difficult to source using desktop methods, and thus well-established relationships between flood damage costs and a particular property attribute (e.g., depth of over-floor flooding) have been used to fill knowledge gaps. Future flood consequence assessments for regional centres in high risk flood zones along the SA River Murray would benefit from on-site surveys collecting information on floor heights, and building classification.

As much of the information in the literature on flood damage costs is at least 10 years old, this study has taken these costs, such as the cost of repairing infrastructure, residential contents values, and costs of replacing crops and pastures, and accounted for 2012 dollar values using CPI as a measure of inflation. In reality, these costs are likely to be higher depending on other economic factors that are of influence at present (e.g. export market price of stock or seeds, current insurance premiums).

The main assumptions and limitations of the flood consequence analysis are as follows:

1. The assessment doesn't include the damage costs to major infrastructure such as schools, hospitals, government buildings and council infrastructure. Typical costs of flood damage to these infrastructure types were unable to be sourced for this study. A qualified insurance assessor may need to be consulted if this information is required.
2. The total damages estimates for only four flood events were calculated: flow to South Australia of 130 000 ML/d, 200 000 ML/d, 250 000 ML/d, and 341 000 ML/d. To estimate damages for other flow scenarios, it has been assumed that a line can be fitted through the damage curve calculated from the four flood events and extrapolated outside these events as required.
3. This assessment only includes direct, tangible flood damages, such as loss of property that can be represented as a monetary value. It does not attempt to quantify intangible or indirect damages, for example, loss of life and mental anguish. Indirect, tangible costs such as emergency response and disruption to normal economic and social activities were assumed to be 30% of the estimated direct costs. The estimate of costs did not specifically allow for higher indirect costs related to, for example, the possible inaccessibility to the Renmark hospital, the temporary closure of water treatment plants or long-term loss to the economy resulting from a reduction in for example tourism or agriculture.
4. The methodology and costing adopted for this study have largely been drawn from documentation developed for Victoria and Queensland. No single approach was fit-for-purpose in this study due to data constraints. Different methods for calculating damage costs for residential, commercial and agricultural properties, and roads/bridges were adopted that were commensurate with the data available for the SA River Murray.
5. The methods for calculating damage costs are for potential tangible damages which are considered to be the damages possible if no additional actions are taken. Subsequently, a further adjustment is undertaken to convert potential damages to actual damages based on warning time and community experience and awareness. For communities with warning times of greater than 12 hours, the ratio of actual to potential damages is estimated to be 0.4 for experienced communities and 0.7 for inexperienced communities (Reed Sturgess & Associates, 2000). This assessment has assumed that the shack communities are experienced and the remainder of the community is assumed to be inexperienced. The adjustment does not apply for roads and bridges and agriculture.
6. Due to the nature of this study being a large-scale desktop study, the cost estimates could be improved by including site surveyed data on individual building or property characteristics to give the most accurate assessment possible of the potential costs associated with flooding.
7. As much of the information in the literature on flood damage costs is at least 10 years old, this study has taken these costs, such as the cost of repairing infrastructure, residential contents values, and costs of replacing crops and pastures, and accounted for 2012 dollar values using CPI as a measure of inflation.

5 Consequence assessment for South Australian River Murray floodplain

5.1 Setting

The River Murray floodplain is defined as the extent of the 1956 flood, the largest recorded flood in history. In this analysis, the extent of the river between the border and Wellington has been considered.

New development on the floodplain is controlled by the *River Murray Act 2003*, the *Development Act 1993* and council development plans. Development within the floodplain is generally limited to recreation facilities, river management infrastructure, shacks and agriculture, particularly between Mannum and Wellington on reclaimed floodplain protected by levee banks. However there are some houses on the floodplain that pre-date modern development controls. Additionally, some areas of development that are outside of the historical 1956 flood line have now been shown to be potentially at risk of flooding by a 1956 size event due to the removal or degradation of levee banks.

5.2 Potential damage to residential properties

Damage estimates for residential properties and shacks are presented in Table 5.1 and Table 5.2. Damage costs for the shacks affected at each flow event is higher than residential damages costs despite the assumption that these properties are built higher from ground level. This is due to more shacks being affected by flooding than residential properties during the more common 130 000 ML/d and 200 000 ML/d flood events. This results in a higher damage cost estimate for shacks. These are direct tangible costs only and the results are in dollar value for financial year 2012–13.

Table 5.1 Estimated potential cost of damage to residential properties (excluding shacks)

Flow event	Estimated number of residential properties affected	Estimated cost of structural damage	Estimated cost of contents damage	Total cost of residential damages
130 000 ML/d	22	$510 000	$570 000	$1 080 000
200 000 ML/d	104	$2 100 000	$3 300 000	$5 400 000
250 000 ML/d	942	$9 200 000	$15 300 000	$24 500 000
341 000 ML/d	1843	$28 000 000	$57 000 000	$85 000 000

Table 5.2 Estimated potential cost of structural damage to shacks

Flow event	Estimated number of shacks affected	Estimated cost of structural damage	Estimated cost of contents damage	Total cost of residential damages
130 000 ML/d	867	$1 300 000	$150 000	$1 450 000
200 000 ML/d	1305	$5 500 000	$4 000 000	$9 500 000
250 000 ML/d	1396	$13 000 000	$11 500 000	$24 500 000
341 000 ML/d	1477	$35 000 000	$30 000 000	$65 000 000

5.3 Potential damage to commercial properties

Damage estimates for commercial properties are shown in Table 5.3.

Table 5.3 Estimated potential cost of damage to commercial properties

Flow event	Estimated number of commercial properties affected	Estimated cost of total damage
130 000 ML/d	19	$450 000
200 000 ML/d	36	$1 200 000
250 000 ML/d	97	$2 100 000
341 000 ML/d	155	$4 100 000

5.4 Potential damage to agriculture

Damage estimates for agriculture are shown in Table 5.4.

Table 5.4 Estimated potential cost of damage to residential properties (excluding shacks)

Flow event	Estimated number of agricultural properties affected	Area (ha)	Estimated cost of total damage
130 000 ML/d	686	14 700	$10 500 000
200 000 ML/d	977	19 267	$18 600 000
250 000 ML/d	1631	23 600	$48 000 000
341 000 ML/d	2048	26 300	$79 000 000

5.5 Potential damage to roads and bridges

Damage estimates for roads and bridges are shown in Table 5.5. The unsealed roads value was used for both public unsealed roads and public tracks.

Table 5.5 Estimated potential cost of damage to roads and bridges

Flow event	Estimated length of road inundated (km)				Estimated cost of total damage
	Major public sealed roads and bridges	*Public sealed roads*	*Public unsealed roads*	*Public tracks*	
130 000 ML/d	3	31	56	11	$1 900 000
200 000 ML/d	9	78	96	21	$4 400 000
250 000 ML/d	24	143	123	29	$7 900 000
341 000 ML/d	31	193	136	30	$10 000 000

5.6 Total damages

The summation of damages for each event, including adjustment for actual damages and indirect damages is shown in Table 5.6, Table 5.7, Table 5.8 and Table 5.9.

Table 5.6 Estimate of flood damages for 130 000 ML/d event

Description	Damages estimate	
Residential buildings	$1 080 000	
Shacks	$1 450 000	
Commercial Properties	$450 000	
Subtotal, adjusted for ratio of actual to potential damages	**$1 650 000**	**Direct Tangible**
Agriculture	$10 500 000	
Roads and bridges	$1 900 000	
Subtotal	**$14 050 000**	**Indirect Tangible**
Indirect damages (@ 30% of direct costs)	$4 215 000	
Total damages	**$18 265 000**	

Table 5.7 Estimate of flood damages for 200 000 ML/d event

Description	Damages estimate
Residential buildings	$5 400 000
Shacks	$9 500 000
Commercial Properties	$1 200 000
Subtotal, adjusted for ratio of actual to potential damages	**$8 420 000**
Agriculture	$18 600 000
Roads and bridges	$4 400 000
Subtotal	**$31 420 000**
Indirect damages (@ 30% of direct costs)	$9 430 000
Total damages	**$40 850 000**

Table 5.8 Estimate of flood damages for 250 000 ML/d event

Description	Damages Estimate
Residential buildings	$24 500 000
Shacks	$24 500 000
Commercial Properties	$2 100 000
Subtotal, adjusted for ratio of actual to potential damages	**$28 420 000**
Agriculture	$48 000 000
Roads and bridges	$7 900 000
Subtotal	**$84 320 000**
Indirect damages (@ 30% of direct costs)	$25 300 000
Total damages	**$109 620 000**

Table 5.9. Estimate of flood damage for 341 000 ML/d event (1956)

Description	Damages Estimate
Residential buildings	$85 000 000
Shacks	$65 000 000
Commercial Properties	$4 100 000
Subtotal, adjusted for ratio of actual to potential damages	**$88 370 000**
Agriculture	$79 000 000
Roads and bridges	$10 000 000
Subtotal	**$177 370 000**
Indirect damages (@ 30% of direct costs)	$52 210 000
Total damages	**$230 580 000**

5.7 Average annual damages

The AAD curve developed for the River Murray floodplain (border to Wellington) for events considered in this assessment (130 000 ML/d, 200 000 ML/d, 250 000 ML/d and 341 000 ML/d) is shown in Figure 5.1. In order to determine the events that begin to incur damages, the experiences from recent high flow events have been considered. The event in 2012 peaked at 60 000 ML/d at the border, and 54 000 ML/d at Lock 1, with no reports of damage. The 2011 event peaked at 94 000 ML/d at the border and 79 000 ML/d at Lock 1, which did cause some damage to properties. Based on these events, the threshold for when damage starts to occur was expected to occur between 60 000 ML/d and 80 000 ML/d, and it has been assumed that flood damage begins to be incurred when the River Murray flow exceeds 71 000 ML/d.

AAD calculated as the area under the curve shown in Figure 5.1. , with the AAD is calculated to be $4 313 000 (2012 values). The interpolation between the four flow events considered will influence the value of ADD calculated. However, the substantial break point in the curve near the 200 000 ML/d event (AEP of 1.33%) has been captured, and while further modelled events would improve the shape of the curve, and hence the AAD calculated, this is not expected to have a substantial effect on the results.

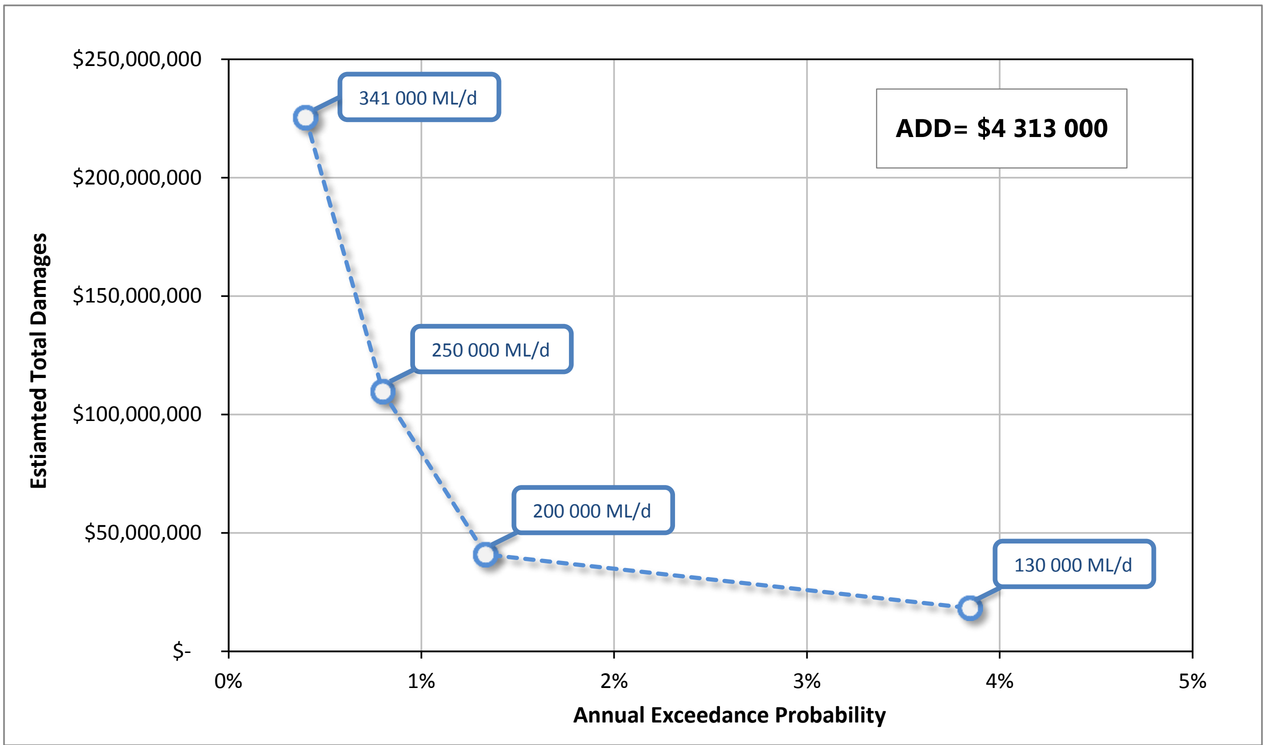

Figure 5.1 Annual average damage curve for River Murray floodplain

6 Consequence assessment for Renmark Paringa Council area

6.1 Setting

The Renmark Paringa Council (RPC) is a local government area of approximately 921 km^2 and is home to an estimated population of 9,800 people. The area has three major settlements being Renmark, Paringa and Lyrup. The towns are located on the banks of the SA River Murray, near its border with New South Wales and Victoria, and approximately 260 km north-east of Adelaide. The Renmark Paringa district economy is centred on irrigated primary production, with viticulture being the major industry, supported by an expanding citrus, stone fruits and almond industry, all relying on irrigation water supplied by the River Murray. Dryland farming is also a major primary industry in the district. Tourism plays a major part in Renmark's economy with the River Murray the predominant tourist attraction.

The RPC and the surrounding rural properties are situated on the floodplains of the River Murray and are vulnerable to flooding to varying extents. The most significant flood event on record was in 1956, where large areas of floodplains along the River Murray were inundated and Renmark experienced severe flooding of parts of the township and surrounding areas. The town of Renmark, which is situated on the floodplain between the River Murray and its anabranch Bookmark Creek was spared from complete inundation by levee banks constructed around the town. Lyrup and Paringa, which are situated on sloping ground on the banks of the river, had the lower portions of their towns protected by smaller networks of levee banks.

The Jane Eliza canal development, constructed in the 1980s and 1990s on the northern outskirts of Renmark, used imported fill to raise the ground level of the residential lots to be higher than the 1956 flood level (Maunsell, 2008).

Despite the levee banks surrounding Renmark (shown in Figure 6.1), there is a risk of flooding if the levees are overtopped by large flood events. As a result, a focused consequence assessment was conducted for the RPC area. Names for levee banks have been taken from the Renmark Paringa Flood Mitigation Study (Maunsell, 2008).

The same methodology as described in Section 4 for the estimation of damage costs from flooding for different flow events has been applied specifically to the Renmark Paringa Council region. An additional flow scenario of 200 000 ML/d assuming failure of levee banks (by overtopping, not other scenarios such as slumping, piping, etc.) was also assessed. Consequences from levee failure was only assessed for 200 000 ML/d. At flows of 250 000 ML/d and above the modelling shows widespread overtopping of levee banks. There are no properties affected by levee failure at 130 000 ML/d.

Figure 6.1 The levee banks surrounding Renmark (Sourced from Figure 32, Maunsell, 2008)

6.2 Potential damage to residential properties

In the RPC area, all houses are treated as permanent residential properties, not shacks. Damage estimates for residential properties are presented in Table 6.1. No properties were affected from flooding for the 130 000 ML/d event.

Table 6.1 Estimated potential tangible direct damage to residential properties

Flow event	Estimated number of residential properties affected	Estimated cost of structural damage	Estimated cost of contents damage
130 000 ML/d	0	$0	$0
200 000 ML/d	1	$3 000	$0
250 000 ML/d	623	$3 370 000	$5 260 0000
341 000 ML/d	1353	$15 700 000	$37 700 000
With levee failure			
200 000 ML/d	402	$1 460 000	$1 830 000

6.3 Potential damage to commercial properties

Damage estimates for commercial properties are shown in Table 6.2.

Table 6.2 Estimated potential cost of damage to commercial properties

Flow event	Estimated number of commercial properties affected	Estimated cost of total damage
130 000 ML/d	0	$0
200 000 ML/d	0	$0
250 000 ML/d	38	$153 000
341 000 ML/d	80	$1 300 000
With levee failure		
200 000 ML/d	36	$98 000

6.4 Damage to agriculture

Damage estimates for agriculture are shown in Table 6.3.

Table 6.3. Estimated potential cost of damage to agriculture

Flow Event	Estimated Number of Agricultural Properties Affected	Area (ha)	Estimated Cost of Total Damage
130 000 ML/d	7	14	$1 900
200 000 ML/d	28	155	$1 250 000
250 000 ML/d	116	392	$4 740 000
341 000 ML/d	142	485	$6 000 000
With levee failure			
200 000 ML/d	78	284	$2 770 000

6.5 Damage to roads and bridges

Damage estimates for roads and bridges are shown in Table 6.4. The unsealed roads value was used for both public unsealed roads and public tracks.

Table 6.4. Estimated potential cost of damage to roads and bridges

Flow event	Estimated length of road inundated (km)				Estimated cost of total damage
	Major public sealed roads and bridges	*Public sealed roads*	*Public unsealed roads*	*Public tracks*	
130 000 ML/d	0	0	1	0	$5 600
200 000 ML/d	0	1	2	0	$52 800
250 000 ML/d	4	25	3	1	$1 100 000
341 000 ML/d	5	43	4	1	$1 720 000
With levee failure					
200 000 ML/d	2	15	3	1	$630 000

6.6 Total damages

The summation of damages for each event, including adjustment for actual damages and indirect damages is shown in Table 6.5, Table 6.6 and Table 6.8

Table 6.5. Estimate of flood damages for 130 000 ML/d event

Description	Damages estimate
Residential buildings	$0
Commercial properties	$0
Subtotal, adjusted for ratio of actual to potential damages	**$0**
Agriculture	$1 900
Roads and bridges	$5 600
Subtotal	**$7 500**
Indirect damages (@ 30% of direct costs)	$2 250
Total damages	**$9 750**

Table 6.6. Estimate of flood damages for 200 000 ML/d event

Description	Damages estimate
Residential buildings	$3 000
Commercial properties	$0
Subtotal, adjusted for ratio of actual to potential damages	**$2 200**
Agriculture	$1 250 000
Roads and bridges	$52 800
Subtotal	**$1 305 000**
Indirect damages (@ 30% of direct costs)	$391 000
Total damages	**$1 700 000**

Table 6.7. Estimate of flood damages for 250 000 ML/d event

Description	Damages estimate
Residential buildings	$8 700 000
Commercial properties	$153 000
Subtotal, adjusted for ratio of actual to potential damages	**$6 200 000**
Agriculture	$4 700 000
Roads and bridges	$1 100 000
Subtotal	**$12 000 000**
Indirect damages (@ 30% of direct costs)	$3 600 000
Total damages	**$15 600 000**

Table 6.8. Estimate of flood damages for 341 000 ML/d event (1956)

Description	Damages estimate
Residential buildings	$53 400 000
Commercial properties	$1 300 000
Subtotal, adjusted for ratio of actual to potential damages	**$38 300 000**
Agriculture	$6 000 000
Roads and bridges	$1 720 000
Subtotal	**$46 000 000**
Indirect damages (@ 30% of direct costs)	$13 800 000
Total damages	**$59 800 000**

Table 6.9. Estimate of flood damages for 200 000 ML/d event with levee failure

Description	Damages estimate
Residential buildings	$3 290 000
Commercial properties	$98 000
Subtotal, adjusted for ratio of actual to potential damages	**$2 710 000**
Agriculture	$2 770 000
Roads and bridges	$630 000
Subtotal	**$6 110 000**
Indirect damages (@ 30% of direct costs)	$1 830 000
Total damages	**$7 940 000**

6.7 Average annual damages

Two AAD curves were developed for the Renmark Paringa Council region for events considered in this assessment (130 000 ML/d, 200 000 ML/d and 341 000 ML/d, and 200 000 ML/d with levee failure) and are shown in Figure 6.2. The lower curve indicates the average annual damages that might be expected if the levee banks were structurally sound and flooding only occurring due to overtopping of the banks. The upper curve indicates that the damages that would occur if the levee banks were in poor condition and failed during a large flood event. It is impossible to predict whether the levee banks would fail during a flood, thus the AAD has been calculated to be a range of $460 000 to $555 000, subject to the structural integrity of the levee banks.

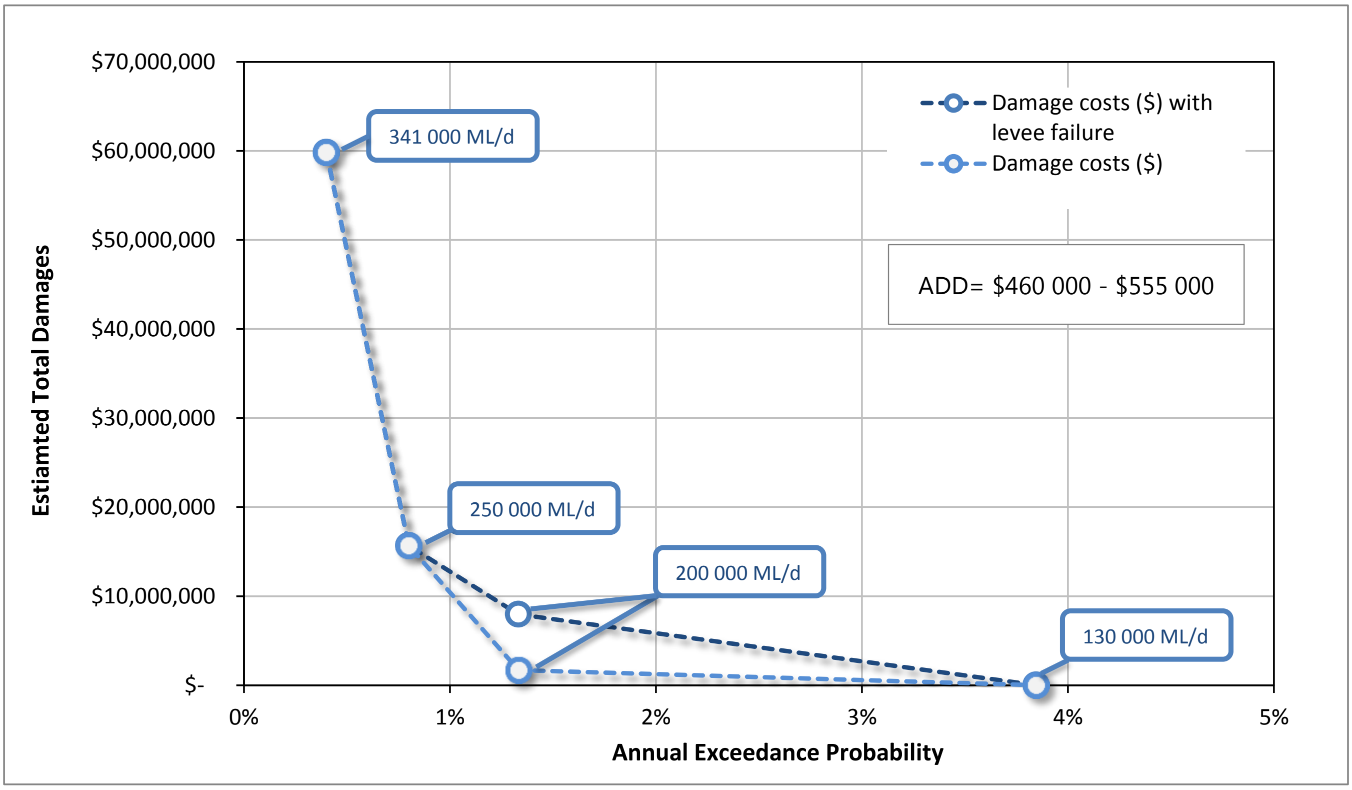

Figure 6.2 Annual average damage curves for Renmark flood consequence analysis

7 Levee bank GIS layer

7.1 Overview

The Levee Bank Management Strategy GIS Layer has been developed to bring together a variety of information on River Murray levee banks, including depth and extent of flood inundation, condition attributes, ownership and location details into a single spatial layer. It is intended to be a used as a tool to assist in the planning and development of flood mitigation and management strategies.

The layer consolidates existing information on the two major levee bank systems on the SA River Murray:

- Renmark Paringa Council levee banks
- Lower River Murray levee banks between Mannum and Wellington.

The layer was generated by combining a GIS spatial layer identifying levees and associated crest height and other attributes such as condition, levee ownership and identification metadata with information on flood inundation depth and extent resulting from the hydraulic modelling of various flow scenarios.

The methodology and information sources for development of the GIS layer and a summary of levee bank condition attributes are described in the following sections.

7.2 Information sources

Sources of levee condition information were:

- Condition and ownership information for the lower Murray levees has been sourced from Golder Associates 'Lower Murray Levee Banks Condition Assessment and Remediation Options' report delivered 30 August 2011 completed on behalf of the Department for Water. Characteristics that affect levee condition identified in the report include cracking, slumping, erosion, crab holes, animal burrowing, tree damage, seepage, vegetation and infrastructure, and have been incorporated as attribute information into the Levee Bank Management Strategy GIS Layer.
- Condition and ownership information for the Renmark Levees has been sourced from the 'Renmark Paringa Flood Levee Opinion of Probable Construction / Rehabilitation Costs' report prepared by AECOM on behalf of the Renmark Paringa Council in May 2011. The report documents the condition of the levees at Renmark and evidence of damage due to insufficient compaction, scouring, gullying or piping. Condition of scour protection has also been documented. These attributes have been incorporated into the Levee Bank Management Strategy GIS Layer.
- The 'Renmark Paringa Flood Mitigation Study' prepared by Maunsell/AECOM on behalf of Renmark Paringa Council in September 2008, documents levee condition and the approximate maximum flow at which the levee would provide protection. This information has been compared with the maximum flow levels prior to overtopping produced from MIKE FLOOD modelling completed by DEWNR for this project at the approximate location of the Renmark levees. The results are comparable for the majority of levees.

Crest height information for the Renmark levee banks has been extracted from the 2008 River Murray Stitched Digital Elevation Model (DEM) using ESRI® ArcGIS. Each levee was individually digitised to generate minimum and maximum elevations or heights for each levee. A similar process has been previously carried out by DFW for an assessment of lower River Murray levee banks in 2010.

Flood height information for the levees around Renmark as well as between Mannum and Wellington has been extracted from the flood modelling undertaken by DEWNR including the heights for the 100 000, 120 000, 130 000, 140 000, 160 000, 180 000, 200 000, 250 000, 300 000 and 341 000 ML/d flow scenarios. The expected flood heights for the Mannum to Wellington section of the study area for various flow scenarios were also cross-checked against results from previous internal DEWNR modelling, which used MIKE 11 modelling, high resolution LiDAR and aerial photography (unpublished DEWNR study).

The information from these sources has been incorporated where possible into the Levee Bank Management Strategy GIS Layer. The data was used as a starting point to determine whether each levee banks are at risk of breaching as a result of high flows and/or poor condition.

7.3 Methodology

The risk of overtopping for each section of levee bank has been determined by comparing levee crest heights and modelled water levels using the available information sources. Risk is defined as both overtopping or water level is within 1 m of the crest. It has been assumed that no actions are taken to increase the height of the banks prior to a flood peak arriving such as by sandbagging or filling. For some levee banks, there are variations in crest levels and isolated low points. The lowest crest height in the available information has been used in the analysis without consideration of where the overtopping would occur or the likelihood that actions would be taken to raise these low points. Furthermore, the analysis does not take into account freeboard requirements, that is, the difference in water level between the water level and crest height which acts as a factor of safety to allow for differences between predicted and actual water levels (due to both forecast and model inaccuracies), the effects of wind and waves and variations in crest level.

In the lower River Murray, there can be considerable variation in water levels for the modelled flow scenarios due to wind conditions and the water level in Lake Alexandrina, which is in turn influenced by tide and river mouth conditions during flood events. Furthermore, there are some discrepancies between different modelling approaches previously undertaken by DEWNR and DFW and how these incorporate uncertainty in predicted water levels. A conservative approach has been taken in this case to give a worst case scenario estimation on those levees that are overtopped, particularly at 130 000 ML/d. The potential variation in water level has been incorporated into water level estimates for the River Murray; accordingly levee banks for the lower Murray have only been described as at risk of overtopping (As seen in Table 7.2) since it is difficult to reliably model at which flow this might occur. A levee bank has been listed as at risk where the difference between modelled water levels and crest height is less than one metre.

The risk of failure of earthen levee banks increases with increasing depth of water retained by the levee bank. Failure can occur by several possible failure modes, such as piping (seepage), slumping and erosion. The geotechnical condition of levee banks has been listed where the water level is within one metre of the levee crest to highlight where levee banks are predicted to be retaining a significant depth of water and are at increased risk of failure.

For the Renmark levee banks, poor condition is assigned where a minimum of one of the four damage conditions is noted as High Damage in the condition descriptions contained in 'Renmark Paringa Flood Levee Opinion of Probable Construction/Rehabilitation Costs Report' (AECOM, 2011) accordingly, average condition is assigned where a minimum of one of the four damage conditions is noted as Medium Damage. For the lower River Murray levee banks, available geotechnical advice on levee bank condition is more complex than that reported for the RPC levee banks. The "Poor Condition" categories include cracking, slumping, erosion, crab holes, animal burrowing, tree damage, seepage, vegetation and infrastructure. No "scale" of condition (e.g. high, low, very poor, moderate etc.) was included in the available information, only a Yes/No indication against the Poor Condition categories. Therefore, those levees that were identified as having four or more Yes scores for a given poor condition category are reported.

7.4 Outcomes

Overtopping risk and geotechnical condition information has been summarised for the 130 000, 200 000, 250 000 and 341 000 ML/d flood event scenarios for the RPC and lower River Murray levee banks in Table 7.1 and Table 7.2 .

Table 7.1. Summary of risk of overtopping and geotechnical condition for Renmark levees

Flood event (ML/d)	Levee predicted to be overtopped	Geotechnical condition for levees with flood level within 1 metre of crest
130 000	Block D Bank (Northern section)	Bookmark Avenue Bank North (Poor) Kulnine Street Bank (Poor) Growers Dist. Bank (Average) Lower Crescent Bank (unknown) Crescent West – Natural Surface (N/A) Sonneman's Lane Bank (Good)
200 000	Block D Bank (Northern section) Lower Crescent Bank Growers Dist. Bank Crescent West – Natural Surface Sonneman's Lane Bank Bookmark Avenue Bank North	Hospital Bank (Poor) Block D Bank (Southern section) (Poor) Crescent Bank (Poor) Kulnine Street Bank (Poor) Hale Street Bank (Poor) Bookmark Avenue Bank South (Average) Murray Avenue – Natural Surface (N/A) Tolarno Street Bank (Good) Sturt Highway Bank (Good) S.A.H.T. Bank (Good)
250 000	Lower Crescent Bank Block D Bank (Northern section) Crescent West – Natural Surface Growers Dist. Bank Sonneman's Lane Bank Bookmark Avenue Bank North Bookmark Avenue Bank South Kulnine Street Bank Murray Avenue – Natural Surface Crescent Bank Hale Street Bank	Hospital Bank (Poor) Block D Bank (Southern section) (Poor) Block E Bank (Poor) Renmark South Bank (Average) Old No.2 PS Bank (Average) Angroves Bank (Average) Tolarno Street Bank (Good) Sturt Highway Bank (Good) S.A.H.T. Bank (Good)
341 000	Lower Crescent Bank Block D Bank (Northern section) Crescent West – Natural Surface Growers Dist. Bank Bookmark Avenue Bank South Sonneman's Lane Bank Crescent Bank Bookmark Avenue Bank North Kulnine Street Bank Murray Avenue Natural Surface Hale Street Bank Hospital Bank Tolarno Street Bank S.A.H.T. Bank Sturt Highway Bank	Block E Bank (Poor) Old No.2 PS Bank (Average) Renmark South Bank (Average) Angroves Bank (Average)

Table 7.2 Summary of risk of overtopping and geotechnical condition for Lower River Murray levees

Flood event (ML/d)	Levee at risk of overtopping	Geotechnical condition for levees with flood level within 1 metre of crest
130 000	Burbridge North Burbridge South Baseby Wellington Marina Mypolonga Glen Lossie Neeta North Cowirra Neeta South Toora Jervois	Mobilong (Poor) River Glen (Poor) Wall Flat (Poor) Wellington (Poor) Woods Point (Poor) McFarlane (Poor) Placid Estate Long Flat Burdett Long Island Yiddinga Paiwalla Westbrook Pompoota Swanport Monteith Kilsby
200 000	All levees except McFarlane	McFarlane (Poor)
250 000	All levees except McFarlane	McFarlane (Poor)
341 000	All levees	Nil

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