Chief Minister, Treasury and Economic Development Directorate (CMTEDD)

Sullivans Creek Flood Study and Kenny Stormwater PSP Design

Flood Study Final Report

March 2015

Executive summary

The Chief Minister, Treasury and Economic Development Directorate (CMTEDD) engaged GHD to undertake a flood study for the Sullivans Creek catchment. The project included development of flood models for Sullivans Creek and specified tributary stream/floodways in order to determine flooding characteristics for Sullivans Creek under present catchment conditions.

Subsequent analysis was also specified to assess future development in the suburb of Kenny and the proposed detention basin configuration to offset increased flooding impact from the development. At the time of undertaking the modelling and reporting, the development boundaries and constraints for Kenny had not been finalised by the Economic Development Directorate. The basin size and configuration is therefore likely to require review, however the design requirement to not exacerbate the 1% Annual Exceedance Probablity event (also known as the 1 in 100 year Average Recurrence Interval event) should remain valid if the retarding basin changes.

The Sullivans Creek catchment is located in Canberra on the northern side of Lake Burley Griffin (LBG) and is approximately 52 km^2 in area. It includes the proposed suburbs of Kenny and Throsby, and traverses approximately 12 km through Mitchell, Lyneham, O'Connor and Turner, before passing through the Australian National University campus and discharging into Lake Burley Griffin.

It contains a mix of urban, open space, and conservation land uses. A large part of the catchment consists of mature inner north residential suburbs. The main alignment is dominated by concrete channels which were designed to convey stormwater quickly. In the northern reaches of the catchment, Sullivans Creek passes through a variety of open spaces, such as Southwell Park, Yowani Golf Club, Thoroughbred Park and Exhibition Park in Canberra (EPIC). The catchment contains a number of ponds and basins designed to improve water quality and provide flood attenuation.

This report describes the flood study including the dam break studies and Consequence Category Assessments for the existing basin at Southwell Park, and the proposed basin at Kenny. A hydrological model (using RORB) was developed for the estimation of flood hydrographs for a range of historic and design events, up to and including the Probable Maximum Flood (PMF).

Assessment was undertaken for the 10%, 1%, 0.2%, 0.01% Annual Exceedance Probability (AEP) events (10, 100, 500 and 10,000 year ARI) and the PMF under existing catchment conditions. Flood mapping representing the proposed Kenny development was produced to downstream of Morisset Road. This report is subject to, and must be read in conjunction with, the limitations set out in Section 1.3 and the assumptions and qualifications contained throughout the Report.

Key features of the Sullivans Creek catchment include pervious alluvial and fractured rock layers which contain shallow aquifers which strongly influence the runoff characteristics of the northern part of the catchment, and the retarding basin at Southwell Park.

Calibration and validation of the hydrological and hydraulic models was undertaken using a combination of historical events (February 2002, December 2010 and February 2011) and Flood Frequency Analysis of the annual maximums.

Existing condition flooding

Flood maps for the exisiting condition are contained in Appendix B for the various Annual Exceedance Probabilities.

10% Annual Exceedance Probability Flooding (1 in 10 year Average Recurrence Interval)

Modelling indicates that existing condition 10% AEP flood waters will generally be conveyed by the channels, culverts and bridges without significant surcharging into surrounding areas.

Overtopping of the roadway is predicted at:

- Miller Street on the O'Connor channel,
- opposite the Morisset Wetland on Flemington Road (to the north of the culvert crossing), and
- over Thurbon Road and Riggall Place in the Lyneham sporting precinct.

Overland flow is also predicted around:

- Haig Park (Masson Street and the northern end of Macleay and Watson Streets)
- behind Duffy Street at the bend of the Dickson Channel near Madigan Street.
- a small amount of breakout is observed through ANU.

Flooding in all these areas is estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual.

Six crossings are likely to be inundated due to 10% AEP flooding (refer Appendix C for locations):

- S09-OC02 O'Connor driveway
- S09-OC04 Coolibah Footbridge
- S09-OC05 Miller Street
- S15 Thurbon Road (Rigall Place)
- S23 Flemington Road DS Morrisset Pond
- S25 Old Well Station 2

1% AEP flooding (1 in 100 year ARI)

Modelling indicates that 1% AEP flood waters will largely be conveyed by the channels, culverts and bridges with some flooding of surrounding areas. The accuracy of the flooding extent has not been ground truthed and there may be instances where the extent is limited by levees, walls, or other obstructions not included in the Digital Elevation Model supplied.

A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Condamine Street.

Overtopping of Flemington Road is predicted opposite the Morisset Wetland (to the north of the culvert crossing), at Heffernan and Darling Streets in Mitchell, east along Sandford Street and south along Flemington Road. A small area of road in Exhibition Park is also predicted to be flooded.

On the Dickson Channel flooding is predicted through a number of properties on Duffy Street at the bend near Madigan Street, a small amount of flooding is predicted where Majura Avenue crosses the Dickson Channel, and to the north of the channel from the Dickson District Playing Fields. At Cowper Street flooding is also observed to breakout to the south of the channel, and also at Challis and De Burgh Streets.

Thurbon Road and Riggall Place in the Lyneham sporting precinct are predicted to be flooded. Flooding is predicted due to limitations in the trunk underground drainage in Lyneham, including Challis Street, Northbourne Avenue, Oliver Street and Goodwin Street. Flooding is also observed around the trunk drainage in David Street (Turner).

Overland flow is observed around Haig Park (Stawell Street, Ormond Street, Masson Street, Hackett Gardens, Miller Street, Barry Drive and the northern end of Macleay and Watson Streets). Breakout is also observed at the upstream end of ANU.

Flooding in these areas is generally estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual with the exception of some flooding within the Southwell Park Retarding Basin.

Twenty two (22) crossings are likely to be inundated by the 1% AEP flooding:

- S04 ANU Biology
- S06 Barry Drive
- S09-OC02 O'Connor driveway
- S09-OC04 Coolibah Footbridge
- S09-OC05 Miller Street
- S11-D01 Dickson Lyneham Footbridge
- S11-D02 Deburgh Street
- S11-D04 Challis Street
- S11-D05 Cowper Street
- S11-D06 Dumaresq Footbridge
- S11-D07 Hawdon Footbridge
- S11-D10 Majura Avenue
- S11-D11 Duffy Footbridge
- S13 Fox Place Footbridge
- S15 Thurbon Road (Rigalli Place)
- S16 Yowani CC FP1
- S17 Yowani CC FP2
- S21-M02 Winchcombe Road Overpass
- S21-M03 Heffernan Overpass
- S23 Flemington Road DS Morrisset Pond
- S25 Old Well Station 2
- S26 Old Well Station 1

0.2% AEP flooding (1 in 500 year ARI)

Modelling estimates that not all of the 0.2% AEP flood is able to be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. Nearly all major crossings observed are subject to flooding with the exception of Parkes Way at the outlet of Sullivans Creek.

There is significant flooding predicted at ANU downstream of Barry Drive. There is also significant flooding predicted in residential areas between Barry Drive and David Street in Turner. A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Stawell Street. For the Dickson channel, significant amount of breakaway flooding is predicted to occur at the upstream end of the channel until it re-joins the Dickson channel near Cowper Street. Flooding is also predicted to occur along the entire length of the Dickson channel, with several dwellings on the north and south sides of the channel being affected.

The Southwell Park fuse-plug is predicted to activate with significant flooding occurring within the sports precinct. Flooding at Randwick pond is predicted to occur and flooding continues south, through the Canberra Racecourse area and overtops the Barton Highway. Overtopping of Flemington Road is predicted opposite the Morisset Wetland (to the north of the culvert crossing). Flooding is also predicted to occur along the Mitchell channel.

Localised flooding from pipe networks is predicted to occur throughout the catchment from this size event. An area of interest is near Stirling Avenue in Watson, where breakout flooding is predicted to occur through EPIC and along Federal Highway from Stirling Avenue to Barton Highway. Several residential dwellings are predicted to be flooded parallel to the Federal Highway.

Flooding in these areas is generally estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual. Flooding along the floodplains of Sullivans Creek channel is estimated with a high hazard category, however very few residential areas lie within this area. There are some areas (generally on roads) where flooding has been categorised as medium to high hydraulic hazard. This is mostly observed for flooding on roads in the suburbs of Turner and O'Connor. Breakout flooding south of Randwick pond, in areas of the Canberra Racecourse and just upstream of the Barton Highway have been categorised with medium to high hazard as well.

0.01% AEP flooding (1 in 10,000 year ARI)

Modelling estimates that 0.01% AEP flood waters will not all be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. Nearly all major crossings observed are subject to flooding with the exception of Parkes Way at the outlet of Sullivans Creek.

There is significant flooding predicted at ANU with flood extents bounded by Daley Road to the west and Childers Street and Ellery Crescent to the east. There is also significant flooding predicted in residential areas between Barry Drive and David Street in Turner, with flood extents bounded by Froggatt Street to the west and Moore Street to the east. A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Stawell Street.

For the Dickson channel, a significant amount of breakaway flooding is predicted to occur at the upstream end of the channel as well as flooding along the entire length of the Dickson channel. This flooding extends to Antill Road and Bonython Street to the north near the Dickson District Playing Fields.

The Southwell park fuse-plug is predicted to activate with significant flooding occurring within the sports precinct. It is predicted that there would be significant flooding south of Randwick pond, at Canberra Racecourse and further south which overtops the Barton Highway. Overtopping along a significant length of Flemington Road is predicted due to breakout flooding coming from the east and north. Flooding is also predicted to occur along the Mitchell channel. Within EPIC, flooding is predicted mainly from Old Well Station Road and Morisset Road, as well as breakaway from pipe networks near Stirling Road. The breakaway at Stirling Road is also predicted to flood along Federal Highway as well as several residential dwellings parallel to the Federal Highway.

Many areas of flooding are generally estimated to fall into the high hydraulic hazard category as defined by the NSW Flood Development Manual. This includes most of the flooding along the main Sullivans Creek channel. However, breakaway flooding from the O'Connor and Dickson channels are estimated to fall into the low hazard category, with the exception of a few flooding roads with a high category.

Probable Maximum Flood inundation

Modelling indicates that PMF flood waters will exceed the capacity of the channels, culverts and bridges, with many areas predicted to be affected. All crossings are subject to flooding including Parkes Way at the outlet of Sullivans Creek.

The predicted flood extent spans approximately 0.5 km to 1 km wide along the main Sullivans Creek channel.

There is significant breakaway flooding predicted from the O’Connor and Dickson Channels. There is also breakaway predicted from Stirling road, which causes flooding along Federal Highway and residential areas parallel to it.

Most areas of flooding are generally estimated to fall into the high hydraulic hazard category as defined by the NSW Flood Development Manual. This includes most of the flooding along the main Sullivans Creek channel. However, there are some areas of breakaway flooding from the O’Connor and Dickson channels which are estimated to fall into the low hazard category, with the exception of a few flooding roads with a high category.

Sensitivity analysis

Flood maps for the various sensitivity scenarios and AEP’s are provided in Appendix D

Increased rainfall intensity due to climate change was simulated (30% increase). The impact on flood extents was greatest in the smaller channels like the O’Connor, Dickson and Mitchell Channels. For the 10% AEP, increases ranged from around 0.1 - 0.4 m in Kenny and parts of the Dickson and Mitchell Channel, 0.5 m near the racecourse to 0.2 - 0.4 m in Sullivans Creek, 0.2 m in the Lyneham Wetland, 0.6 m at Southwell Park and up to 0.8 m in parts of the O’Connor Channel. For the 1% AEP, increases ranged from around 0.1- 0.3 m in Kenny, 150 - 400 mm in Sullivans Creek, 0.5 m in the Lyneham Wetland, 0.7 m at Barry Drive, 0.75 m at Southwell Park and up to a metre in parts of the Mitchell and O’Connor Channels. Localised increases of 0.6 m to in excess of a metre were observed in areas in and around the Dickson Channel.

Application of blockage factors (in line with Australian Rainfall and Runoff - Project 11) at the Southwell Park inlets (50%) and Majura Avenue (75%) did not have a significant effect on flood levels around Southwell Park. A 75% blockage of the Majura Avenue bridge increased flood levels and caused additional breakaway flooding to the west and northwest.

Increasing the Manning’s ‘n’ roughness values resulted in increased flood levels in many areas but did not significantly change the flood extents due to the terrain. New minor flow paths are initiated in some areas, such as through Kenny and downstream of Barry Drive,

The tailwater level for the downstream boundary condition of the hydraulic model was increased by up to a metre, and only affected flood levels from the lake up to the weir control downstream of Fellows Rd for both the 10% and 1% AEP.

A brief sensitivity analysis on the Southwell Park fuse-plug failure assumptions was undertaken in both the flood flow and and flood level models. The impact was less significant for the 0.04% AEP (1 in 250 year ARI) and PMF. However, changes in assumptions could potentially have a significant impact on the rare / extreme events between the 0.04% AEP and PMF.

Southwell Park breach modelling

Maps for Southwell Park breach modelling are contained in Appendix G

The Southwell Park retarding basin stores approximately 250 ML below the “starter chute” spillway level, and has a fuse-plug spillway which is designed to progressively fail laterally and via overtopping in events rarer than the 1% AEP. The level at which the fuse-plug fails, and the time it takes to fail are critical assumptions which affect the flood levels in the basin and the flood behaviour downstream.

Further investigation of the fuse-plug current condition and its likely operation is recommended.

The lowered embankment crest level where the bike path crosses to the east of the outlet culverts results in a Dam Crest Flood of 0.4% AEP, a higher AEP (more frequent event) than if the main crest level had been maintained.

A number of embankment breach scenarios were simulated for the 0.4% AEP, 0.01% AEP and PMF. Breaches to the west of the fuse-plug were found to be critical, with large affected areas (300 mm or more increase in flood levels due to the breach) and concentrations of Population at Risk on busy Mouat Street, at the Lyneham Motor Inn, St Ninian's Uniting Church, Brindabella Christian College and Lyneham Primary, along with residential dwellings on Mouat Street, Brigalow Street, Boyd Street and Lewin Street. The Comprehensive Category Assessment for Southwell Park ranks the basin as a "High A" dam, and remedial measures will need to be undertaken so that it can safely pass the PMPDF (Probable Maximum Precipitation Design Flood).

It is recommended that a Dam Safety Emergency Plan be prepared and implemented for the basin to reduce the risk of loss of life in the event of the basin breaching. The gauging station at Southwell Park (410772) should be upgraded to ensure an alarm mechanism prior to possible breach of the fuseplug and embankment overtopping. Incorporation into a flood warning system is recommended.

Kenny development and Kenny basin breach modelling

Flood maps for Kenny development are contained in Appendix F, and for the Kenny Basin breach refer to Appendix H.

The hydrology (RORB) model was updated to reflect the proposed development, with the increase in impervious areas increasing peak flows. To attenuate these back to the existing model peak flows for the 10% and 1% AEP events, preliminary sizing of the storage and staged outlet with high level spillway was undertaken, and shown to not increase flood peaks downstream up to and including the 1% AEP.

The preliminary design of the basin is estimated to store 53 ML to the spillway level, and 134 ML at crest level. Breaches were estimated and simulated for the critical duration (1 hour) 0.01% AEP and PMF events at the proposed Kenny basin. No significant affected area (increase in flood levels of greater than 300 mm due to breach) was produced by any of the breaches. With no Population at Risk or Potential Loss of Life within the affected area if a "Medium" Severity of Loss and Damage is assumed the basin is given a preliminary Consequence Category of "Low". According to the NSW Dam Safety Committee guidance, DSC3B, a "Low" Consequence Category dam has an Acceptable Flood Capacity of between the 1% and 0.1% AEP flood.

Subsequent breach modelling was undertaken for the 1% AEP flood, as this is expected to have a greater incremental effect than the 0.01% AEP.

The possibility of a failure of the proposed Kenny Basin causing a cascade failure of the Southwell Park basin was not investigated in detail, as under NSW Dam Safety Committee requirements modelling and assessment is only required for areas where flood levels are increased by more than 300 mm in the event of a failure. With likely upgrades to Southwell Park, the downstream basin is not expected to be sensitive to the increase in volume which would occur in the event of a failure of the Kenny Basin, but this should be reassessed at the design stage.

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1. Introduction

1.1 Purpose of this report

The Chief Minister, Treasury and Economic Development Directorate (CMTEDD) engaged GHD to undertake a flood study for Sullivans Creek, located in Canberra, ACT. The brief included development of hydrological and hydraulic models for Sullivans Creek and specified tributary stream/floodways in order to determine flooding characteristics for Sullivans Creek under present catchment conditions.

A subsequent analysis was also specified to assess future development and proposed detention basin configuration to be incorporated within future development to offset flooding impacts in and around the suburb of Kenny.

For this study flooding is defined as mainstream flooding from Sullivans Creek and the nominated tributary creeks and channels. Mainstream flooding consists of flows within these waterways, and flow which has broken out of the waterways where they do not have sufficient capacity. Some areas where the overland flow paths and drainage networks diverge have had flow applied to aid in a better estimate of flow distribution but are not considered mainstream flooding. These are marked on the flood maps.

1.2 Methodology

This report presents the following methodology:

- Development of a hydrological model suitable for the estimation of flood hydrographs for a range of design events and basin breach hydrographs, up to and including the Probable Maximum Flood (PMF).
- Development of a two-dimensional hydraulic model, and subsequent flood mapping of Sullivans Creek, Mitchell Channel, Dickson Channel, O'Connor Channel and Kenny for the existing catchment conditions.
- Assessment of the "developed" scenario including the proposed Kenny development concept design and the proposed detention/retarding basin to offset the proposed development.
- Assessment of subsequent flood impacts of the proposed Kenny development within the catchment (checking peak flows downstream and flood mapping to Morisset Road).

1.3 GHD Disclaimer and limitations

This report has been prepared by GHD for Chief Minister, Treasury and Economic Development Directorate (CMTEDD) and may only be used and relied on by CMTEDD for the purpose agreed between GHD and the CMTEDD as set out in section 1.1 of this report.

GHD otherwise disclaims responsibility to any person other than CMTEDD arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the investigation was completed prior to preparation of the report.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

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1.4 Data

For the purposes of undertaking the flood study, and to calibrate models, the following key data was sourced:

- Concurrent rainfall and flow data for a number of flood events to enable calibration and verification of the hydrological model parameters;
- Rainfall data included:
  - Pluviographic rainfall data at 6 minute intervals to provide information on historic storm temporal patterns
  - Daily rainfall data to provide spatial distribution of rainfall events;
- Flow gauge data, including gauge history and rating curves, to determine hydrographs of flood events
- GIS data or drawings of key drainage assets
- Topographic survey data (LiDAR flown July 2004 and additional ground survey) to enable appropriate representation of the terrain within the hydrologic and hydraulic models
- Cadastre information (supplied 07/08/2013) and aerial photography (edited 13/07/2012) to assist with the estimation of impervious fractions (for the hydrology) and roughness values (reach type for the hydrologic model and Manning's n for the hydraulic model).

1.5 Assumptions

The assumptions made are discussed along with the methodologies in the relevant sections of the report. While some limited common sense checking of provided data has been undertaken, in general it has been assumed that the data and information provided is correct.

2. Background

2.1 Adopted standards and guidelines for flood studies

At the time this work was undertaken there were a number of revisions being made to the standard approach to flood studies. Relevant sections of various standards and guidelines were adopted as outlined below.

2.1.1 Australian Rainfall and Runoff (ARR)

ARR was under review at the time of this study with a number of reports at various stages of drafting. While some consideration was given to a number of these only limited parts of the proposed updates were adopted, such as blockages and Areal Reduction Factors.

2.1.2 TAMS design code

Guidance was taken with regard to:

- Impervious fractions and initial RORB values (k_c and runoff-coefficients);
- Mannings 'n' values for channels and overland flow; and
- Retarding basin design guidelines.

2.1.3 BOM IFD curves

While revised IFD (Intensity-Frequency-Duration) information is now available from the Bureau of Meteorology (BOM), the traditional version consistent with ARR 1987 was adopted as it remains consistent with remainder of the design storm methodology and produced results which were generally consistent with the flood frequency analysis undertaken. The 2013 IFD values are very similar but slightly lower as the data incorporates rainfall to the start of 2010, which includes the 10 year drought (hence lowering values), but not the major events from late 2010 and 2012. It is understood there are still revisions of IFD and temporal patterns occurring as well as a new revision of the CRC-Forge method about to commence.

2.1.4 ANCOLD and NSW Dam Safety Committee guidelines

The basin breach modelling and subsequent Consequence Category Assessment was undertaken in accordance with the NSW Dam Safety Committee information sheets DSC3A and DSC3B (2010) and DSC General Notice about Retarding and Detention Basins (2013), ANCOLD Guidelines on the Consequence Categories for Dams (2012) and ANCOLD Bulletin 97: Dam Break Breach Mechanisms (1994). Loss of Life calculations followed Graham (1999), with flood severities for itinerants defined in line with the draft ARR Chapter 6: Safety Design Criteria.

2.1.5 Bureau of Meteorology Bulletin 53

Probably Maximum Precipitation (PMP) rainfall, temporal patterns and spatial distribution were derived in accordance with the Generalised Short Duration Method (GSDM), as set out in the Bureau of Meteorology's Bulletin 53. The GSDM temporal pattern was also used for events with an AEP of less than 1%.

2.2 Catchment description

The Sullivans Creek catchment is located in Canberra on the northern side of Lake Burley Griffin and is approximately 52 km^2 in area. It includes the suburbs of Kenny and part of Throsby (as yet undeveloped), Turner, Dickson, Downer, Hackett, Watson, Braddon, Bruce, Ainslie, Kaleen, Harrison, Franklin, Mitchell, O'Connor, Lyneham, Acton, part of the CBD, and Australian National University (ANU) as shown in Figure 2-1 . It contains a mix of urban, open space, and conservation land uses.

A large part of the catchment consists of mature inner north residential suburbs. The main stormwater alignment is dominated by concrete channels which were designed to convey stormwater quickly, thereby minimising the land take required. In the northern reaches of the catchment, Sullivans Creek passes through a variety of open spaces, such as Southwell Park, Yowani Golf Club, Thoroughbred Park and Exhibition Park in Canberra (EPIC). Sullivans Creek is approximately 13 km in length and discharges to Lake Burley Griffin after passing through the ANU.

The catchment contains a number of wetlands and basins, designed to provide water quality and flood attenuation benefits respectively. These include the Dickson, Morisset, Randwick and Lyneham wetlands, a large dry retardation basin at Southwell Park, and a large on-line Gross Pollutant Trap (GPT) at Barry Drive in Turner.

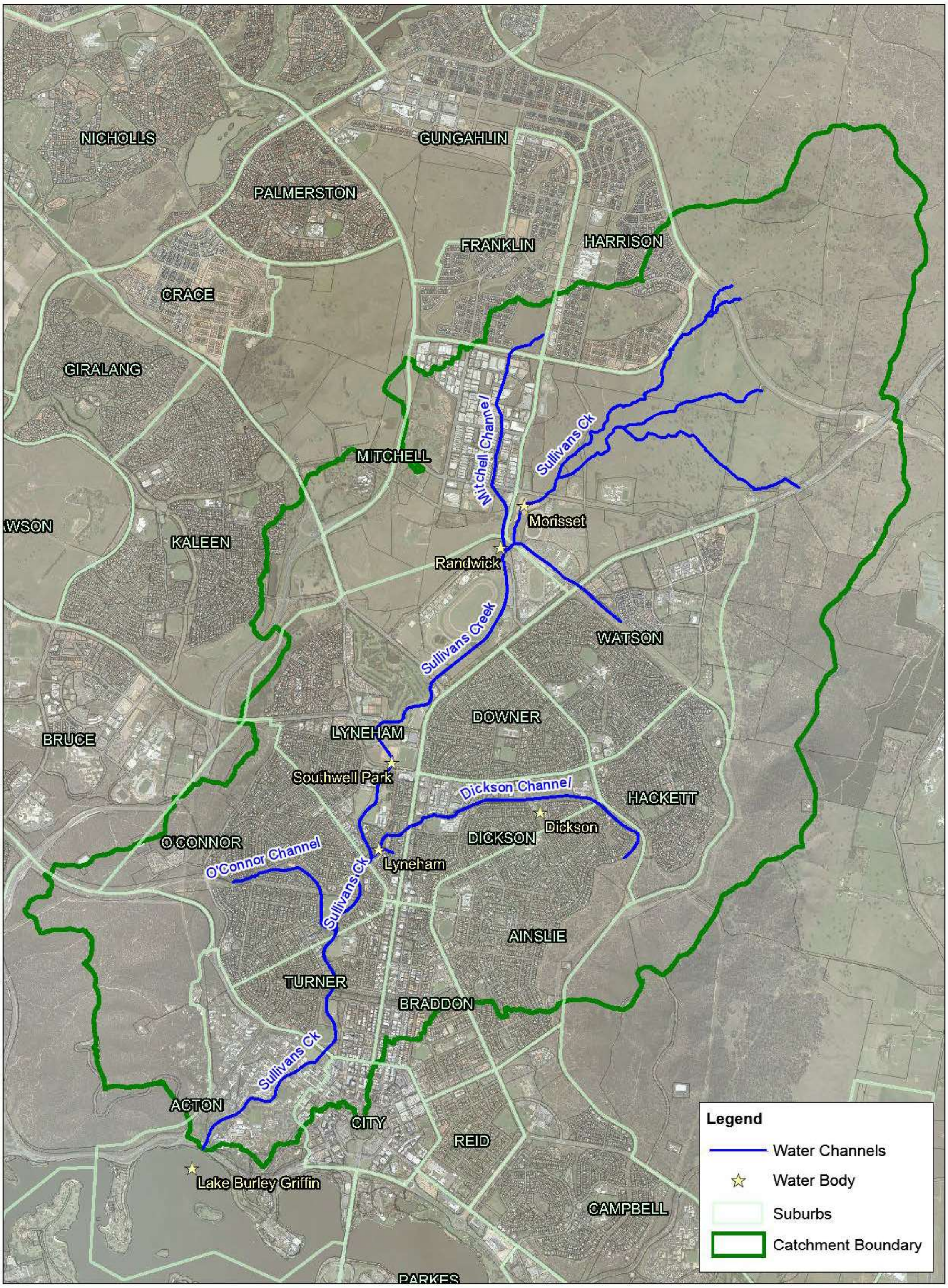

Figure 2-1 Catchment overview map

2.2.1 Southwell Park

Southwell Park is a sporting precinct located in Lyneham, approximately halfway down the Sullivans Creek catchment (Figure 2-1). It is bounded by an embankment along Mouat Street on the southern side to form a retarding basin. Two low level culverts exit the basin under Mouat Street (Figure 2-2), with a high level outlet pit connecting to a third culvert to convey higher flows (Figure 2-3). Above this level a spillway activates through the access-way and there is also a fuse-plug spillway which is designed to fail in two parts, one quickly and one more slowly if flood levels reach 572.2 m AHD.

Significant changes were made to the configuration of this precinct following a study by Cardno undertaken for ESDD (now EPD) in 2011. Flow now enters the basin through a 4200 mm wide by 1540 mm high box culvert under the precinct, which was open channel prior to works in 2011. The high level outlet level was lowered from the original design level of 571.2 m AHD to 570.57 m AHD and the western (archery) field was also lowered by approximately half a metre in 2012 to provide more storage. There is a water level gauging station (410772) located in the neck of a pond between the 4200 mm inlet box culvert and the twin outlet culverts, with a reference level of 567.02 m AHD (Figure 2-4). Further discussion of the hydraulic modelling of Southwell Park can be found in Section 5.8.

Figure 2-2 Southwell Park low level outlet culverts (looking downstream from the gauging pond)

Figure 2-3 Southwell Park basin high level outlet pit

Figure 2-4 Southwell Park inlet culvert and gauge location (in centre)

2.2.2 Delayed Upper Catchment Response (DUCR)

Previous investigations (Cardno 2011 and AECOM 2013) have discussed the ground conditions in the upper Kenny portion of the Sullivans Creek catchment. The area is understood to include relatively shallow aquifers in alluvium and fractured rock, with some waterlogged areas characteristic of a high water table and groundwater discharge.

While the characteristics of these systems and consequently their response is expected to vary throughout the catchment it was found that adoption of higher losses in the area previously identified by Cardno as the "pedoderm" area did improve the representation of historic flood peaks. The area shown in Figure 2-5 was assumed to exhibit the response resulting from these characteristics based on Figure 2.1 of the Draft Kenny Pond and Floodway Feasibility Study undertaken by Cardno Young in 2011. Whilst not all of these subareas were identified by Cardno, the flow from them passes through the areas delineated by Cardno.

The characteristic response of this area has subsequently been referred to in this report as the Delayed Upper Catchment Response (DUCR) since from a surface water perspective the area exhibits relatively little immediate runoff although appears to generate groundwater discharge resulting in additional base flow in the later stages of particularly the longer events. The DUCR area does not represent a distinct boundary between differing geologies or land use, rather it is an approximation of changes in the average catchment characteristics based on observations by others and the recognition that it improved the ability of the calibrated model to represent the peaks of the calibration events at the two calibration locations.

Cardno estimated that between 20 and 40 mm of rainfall was required in the upper parts of the catchment to generate any flow above Flemington Road, although the basis of this is not described, and information on the rate of infiltration and impact of antecedent conditions was not detailed. A detailed investigation of the hydrogeology of these areas is outside of the scope of this investigation, although some understanding of their response and influence has been gained from analysis of the calibration events. The calibration events provide a limited understanding of the effect of the DUCR for relatively small events. In the absence of detailed knowledge of the behaviour of the DUCR in extreme events conservative assumptions have been made, as outlined in Section 4.

As the hydrologic model does not include the ability to model the groundwater process, the delayed event discharge evident in the tail of the historic hydrographs is underestimated by the adopted hydrologic model. The apparent ground water response evident in the historic hydrographs is relatively fast from a groundwater perspective and indicative of a relatively pervious, wet and highly responsive groundwater system. Although the surface hydrology provides a good approximation of the peak surface water response it is likely that a better match particularly of recession on the longer events could be achieved using an integrated model capable of modelling the surface and groundwater processes concurrently. The current study objectives and available data do not warrant the additional complexity of an integrated ground and surface water model however future investigations should reconsider the benefits and costs of such an approach.

2.3 Previous studies

The Sullivans Creek catchment has been the subject of a number of previous modelling studies, including the reports listed below. References to these reports are included within the relevant sections of this report as appropriate.

- Sullivans Creek and Tributaries Flood Study, Ecowise Environmental, 1996
- Sullivans Creek Flood Study, Ecowise Environmental, 2000
- Upper Sullivans Creek Flood Study, Ecowise Environmental, 2002
- Review of Infrastructure Planning for Sullivans Creek Catchment, Bill Guy & Partners, 2003
- Lyneham Sports & Recreation Precinct Stormwater 2D Modelling Report, Cardno Young, 2009
- Kenny Pond and Floodway Feasibility Study, Cardno Young, 2011
- Kenny Supplementary Flood Modelling, Cardno Young, 2013.

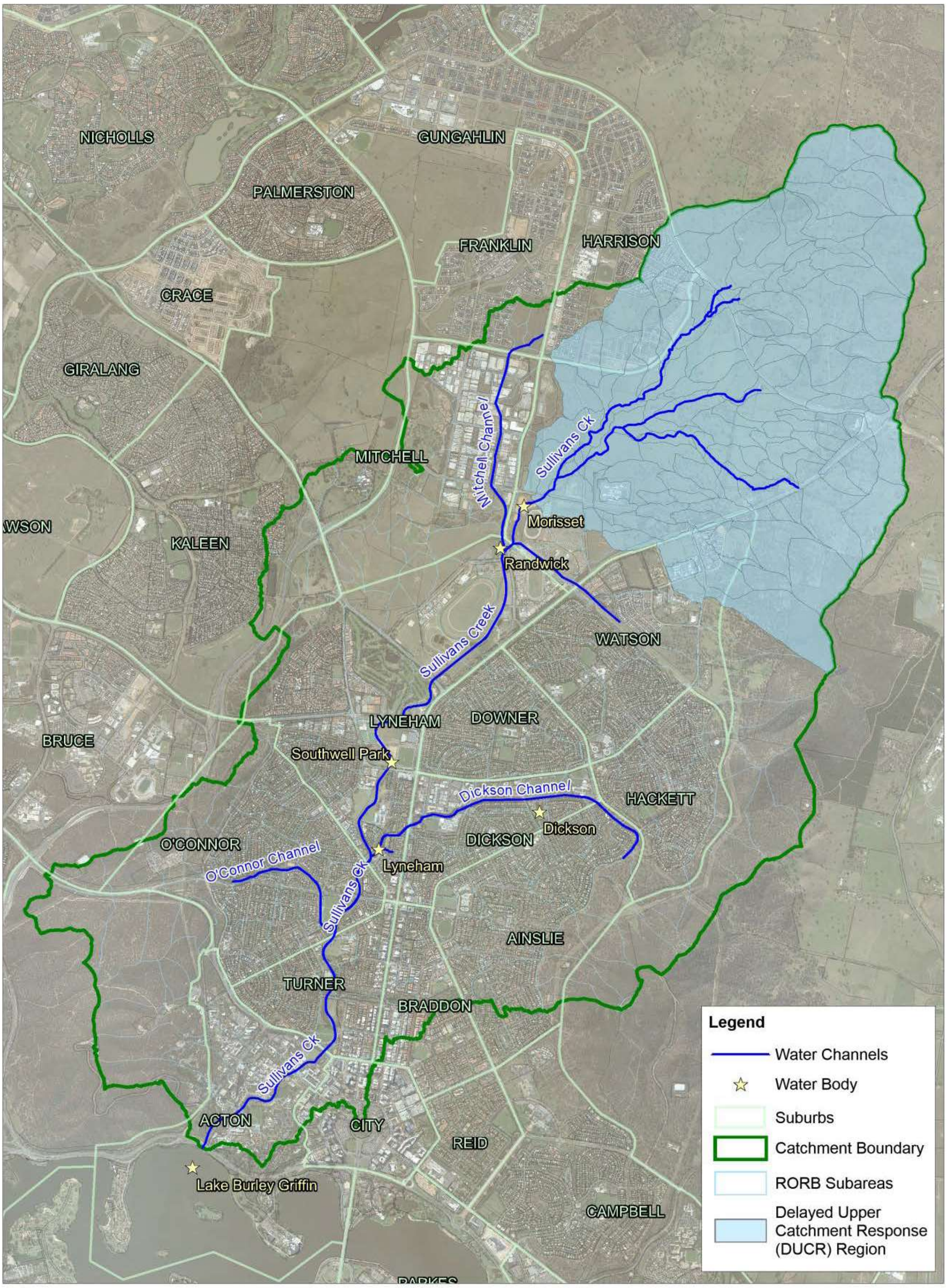

Figure 2-5 RORB subcatchments – shaded subareas exhibiting a Delayed Upper Catchment Response (DUCR)

3. Gauge data

3.1 Flow data

Two flow gauges are located within the Sullivans Creek catchment, namely Barry Drive (410775) and Southwell Park (410772). These gauges:

- Are located along the main channel of Sullivans Creek
- Have recorded a number of flood events, but not major events
- Have rating tables to enable estimation of flows from the recorded levels
- Have data with concurrent daily and continuous pluviographic rainfall data in and adjacent to the catchment.

Flow gauge information is provided below in Table 3-1.

Table 3-1 Flow gauge records

Gauge Number	Gauge name	Period of record	Operational Dates	Comments
410775	Barry Drive Gauge	27 years	01/08/1986 to Present	Gauge zero (datum for rating table) is at 559.819 m AHD. Situated immediately upstream of Barry Drive bridge.
410772	Southwell Park Gauge	34 Years	01/08/1979 to Present	Gauge zero is at 567.02 m AHD. Situated within the Southwell Park basin upstream of the outlet culverts. Note that there is also a secondary gauge location with the same number located downstream of Mouat Street on the same channel, operated by a different service provider for National Capital Authority

3.1.1 Rating tables

Gauges record levels, which are then converted to flows using a rating curve. Rating tables and gaugings at the two flow gauge locations were examined in order to ascertain the validity of the rating curves and the flows reported by the gauges for particular events. The existing rating curves for the gauges are shown below in Figure 3-1 and Figure 3-2.

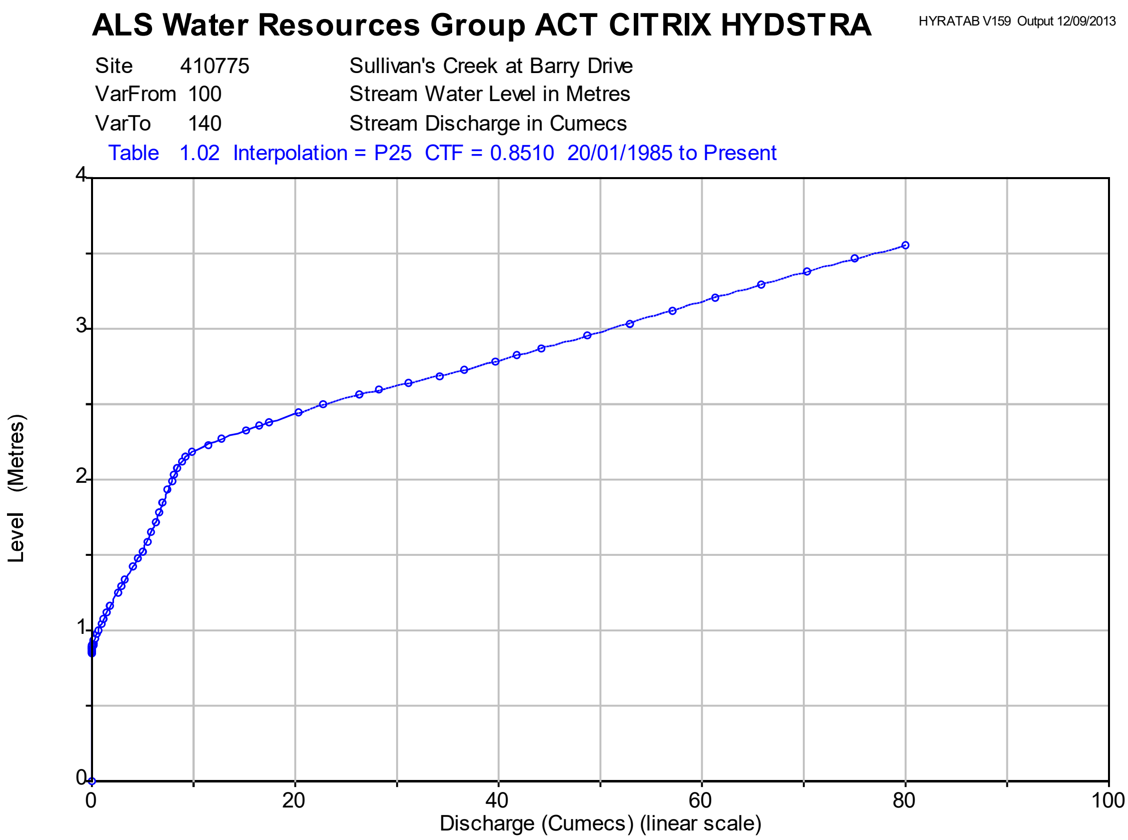

Figure 3-1 Barry Drive rating curve

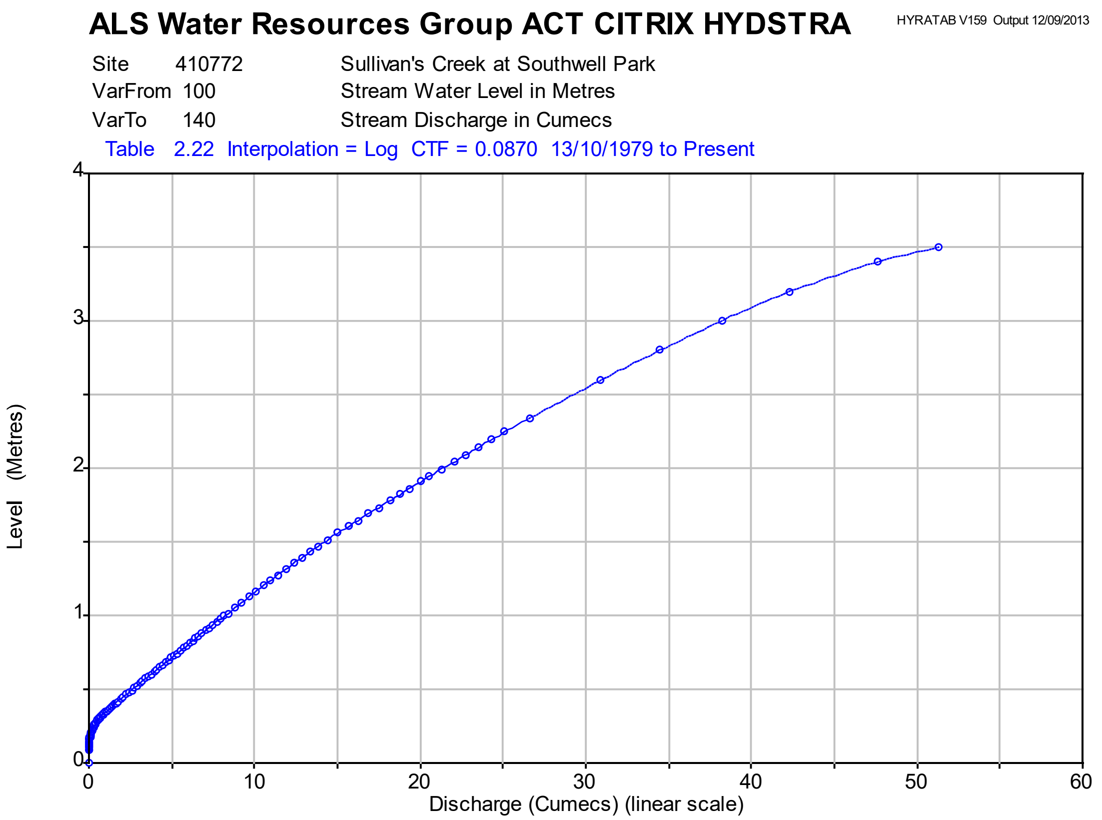

Figure 3-2 Southwell Park rating curve

The Southwell Park gauge (410772) is located at a concrete flume at the downstream end of a small pond at RL 567.02 mAHD. It measures water level which at low levels can be converted to flows through the constriction. Once water overtops the edges of this structure, current practise has been to estimate the discharge by linear extrapolation of the existing rating curve. However this approach is not expected to be accurate. The largest gauged flow for the Southwell Park is approximately 26 m^3/s, and above this point the rating table at Southwell Park had been extrapolated linearly.

The Barry Drive gauge (410775) is located at the weir on the upstream side of the bridge crossing. The largest gauged flow for the Barry Drive is 20.3 m^3/s and above this point the rating table at Barry Drive had been extrapolated.

3.1.2 Flood frequency assessment

A flood frequency analysis was previously produced for the lower Sullivans Creek Flood Study (Ecowise, 2002) using records available up to late 2001. A revised flood frequency analysis was produced for this project taking into account the additional 12 years of available recorded data at the gauges between 2001 and mid 2013. The results of the revised flood frequency analysis based on the flows provided are shown in Figure 3-3 and Figure 3-4.

A summary of the revised estimated peak flows is also provided below in Table 3-2. In general, the peak flow estimates have been revised down from previous (Ecowise, 2002) estimates, taking in to account the additional 12 years of data. The rating curves are extrapolated above approximately 20 m^3/s for Barry Drive and 26 m^3/s for Southwell Park, and hence there is low confidence in these results as all flows shown in the table are within the extrapolated range for Barry Drive, and extrapolated beyond the 20% AEP for Southwell Park.

Table 3-2 Revised Flood Frequency Analyses (m^3/s)

AEP (%)	Barry Drive			Southwell Park		
	5% ile	Best Fit	95% ile	5% ile	Best Fit	95% ile
50	43.9	36.7	30.7	19.2	16.8	14.7
20	55.4	49.6	44.4	27.5	24.0	21.0
10	63.9	55.5	48.2	33.2	28.6	24.7
5	74.0	59.7	48.2	39.4	32.9	27.4
2	87.4	63.7	46.4	48.6	38.2	30.1
1	98.9	67.2	45.6	56.4	42.1	31.5
0.5	106.0	67.6	43.1	64.9	45.9	32.5
0.2	116.9	69.3	41.1	77.4	50.9	33.4

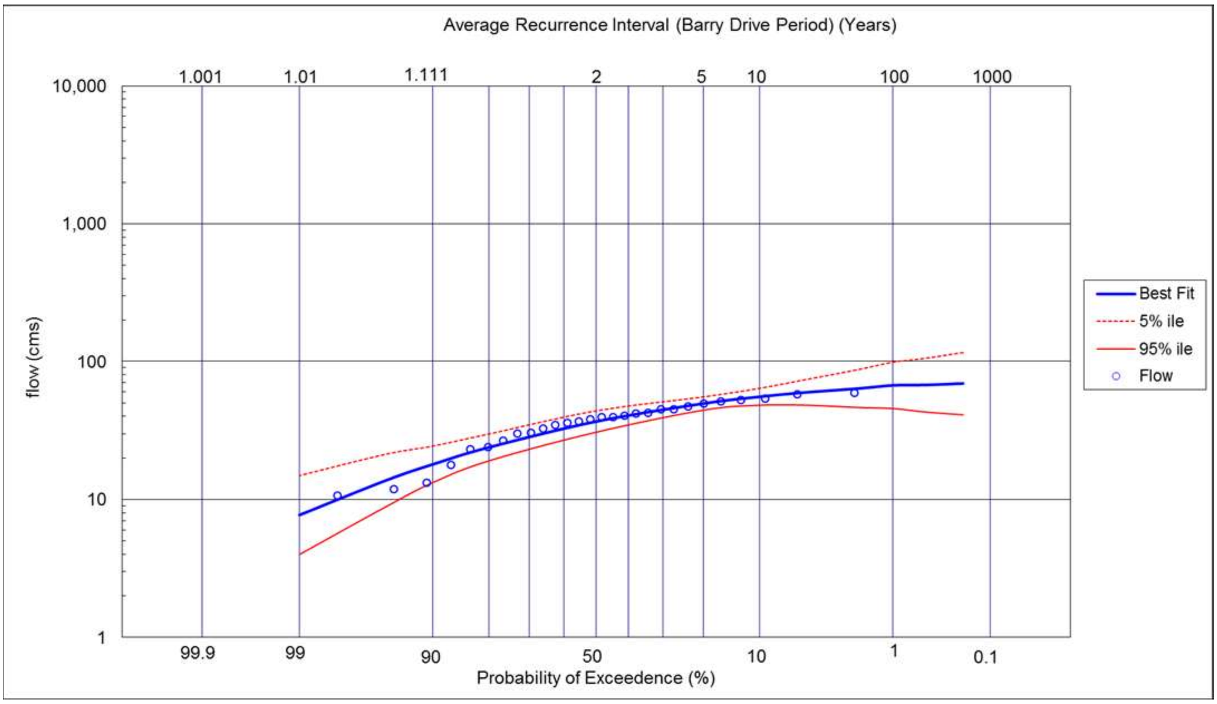

Figure 3-3 Revised Flood Frequency Analysis - Barry Drive

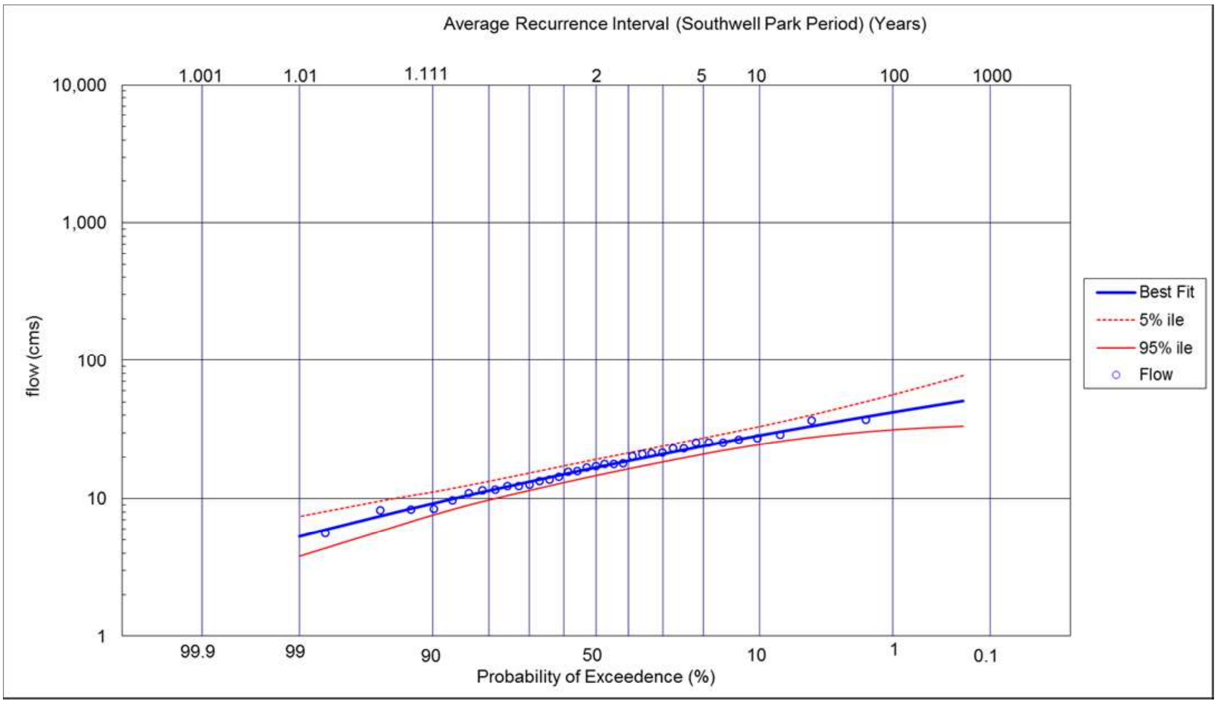

Figure 3-4 Revised Flood Frequency Analysis - Southwell Park

Given the uncertainty associated with the extrapolated rating table an alternative flood frequency analysis was also conducted using FLIKE on the gauge level data, and can be found in Section 9.

3.1.3 Calibration events

The project brief requested a revised calibration or validation based on available streamflow data, including recent floods in December 2011 and December 2010. The ranking of annual maximums at Barry Drive and Southwell Park is shown in Table 3-3. Although December 2010 and December 2011 were specifically mentioned in the brief, larger flows were recorded in February 2011 than December 2011. Large flows were also recorded at both gauges in February 2002. Surveyed flood height data at two locations (Barry Drive and Condamine Street) were recorded and available for December 2010. Based on the above information it was decided to calibrate the model to the following events:

- 16 February 2002
- 2 December 2010
- 4 February 2011.

Table 3-3 Ranking of maximum annual flows at Sullivans Creek gauges

Ranking	Barry Drive	Southwell Park
1	2002	1992
2	1992	1981
3	2009	1995
4	1989	2007
5	1995	2011
6	1993	2002
7	2011	1993
8	2010	2010
9	2007	2005
10	2004	2012

3.2 Rainfall data

Three rainfall gauges are located within the Sullivans Creek catchment area, and were analysed to estimate the temporal variation in the rainfall patterns for the calibration events. Details for these gauges are provided below in Table 3-4. There are also a number of daily rainfall gauges from BOM, within and external to the catchment, which were used to estimate the spatial distribution of rainfall depths for the calibration events.

Table 3-4 Available pluviograph data

Gauge number	Gauge name	Operational Dates	Comments
570942 (ALS)	Lyneham - Southwell Park	1985 - Present	Some gaps in data set
570813 (ALS)	Turner - Barry Drive	1987 – Present	Some gaps in data set
570981 (ALS)	Watson - Racecourse	1981 - Present	Some gaps in data set

3.2.1 February 2002 storm event

Overview

This rainfall event started on the 16 February at 6:30 pm (EST) and lasted less than an hour. During this event, a total of 32.2 mm was recorded at Southwell Park in 42 minutes with a peak intensity of 114 mm/hr. A total of 16 mm had been recorded in the previous 5 days leading up to this event.

Based on the Canberra IFD tables, the recorded intensities would place this event at between a 2 year and 5 year ARI design rainfall intensity event as shown in Figure 3-5.

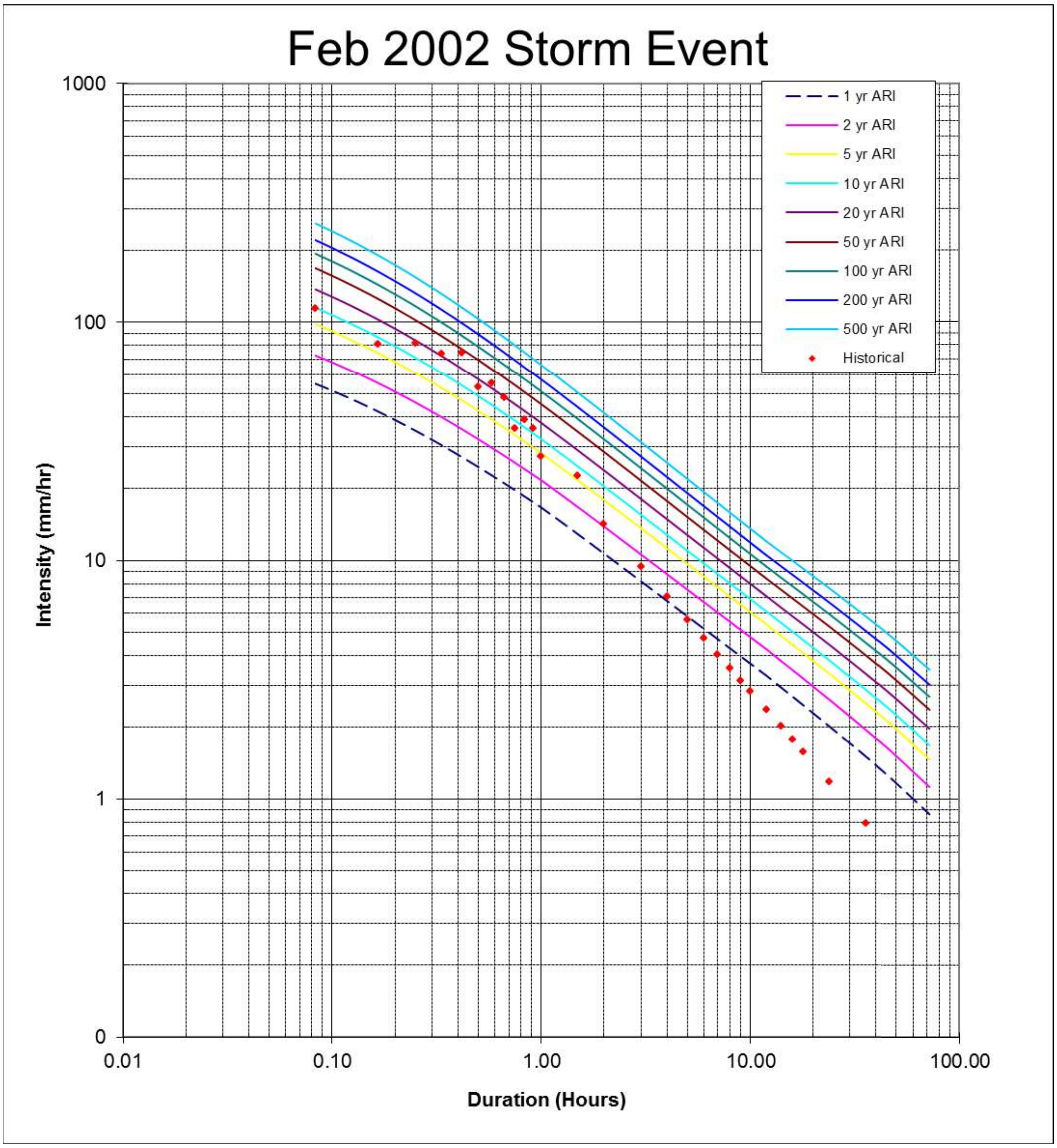

Figure 3-5 IFD curve for 16th February 2002 event (Southwell Park)

Spatial variability

To understand the spatial distribution of the rainfall on the catchment an analysis of daily rainfall records was undertaken. Daily rainfall data from within the catchment and surrounding rainfall stations was collected and is presented in Figure 3-6. This figure indicates that rainfall totals for the event tended to increase towards the south and were lower in the upper portions of the catchment. The rainfall patterns were derived from the pluviograph data for Watson at Racecourse and at Southwell Park as indicated by the shading.

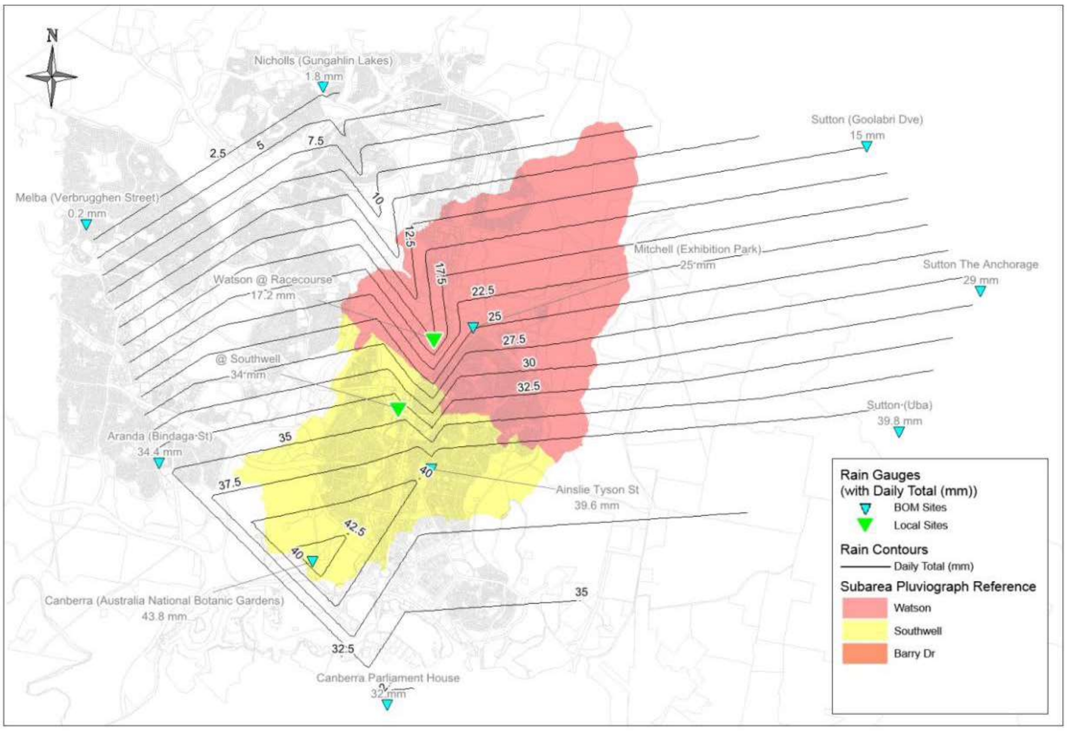

Figure 3-6 Spatial distribution of 16th February 2002 rainfall event.

3.2.2 December 2010 storm event

Overview

Rainfall was recorded off and on for a number of days around the 2 December 2010 event. Peak rainfall intensities at Southwell Park and Barry Drive occurred late on 28 November, with the peak rainfall intensity at Racecourse at 5:42 pm (EST) on 2 December. The water level gauges at Southwell Park and Barry Drive peaked at 6:18 pm and 6:36 pm on 2 December respectively. Whilst small amounts of rainfall were recorded earlier in the day and large amount subsequently, peak flows can be attributed to the rainfall which occurred between approximately 4 pm and 6:18 pm on 2 December. In the 24-hour period 65 mm fell at Southwell Park, 43 mm at Barry Drive and 63 mm at Racecourse. Of this a total of 34 mm was recorded at Southwell Park from approximately 4-6 pm with a peak intensity of 68 mm/hr. A total of 70.6 mm had been recorded in the previous 5 days leading up to this event. A summary of the 2 December 2010 rainfall event is in Table 3-5.

Table 3-5 Summary of 2nd December 2010 rainfall event

Rainfall period	Southwell Park	Barry Drive	Racecourse
Total over 24-hour period (mm) to 9 am 3/12/2010	65.2	42.86	62.8
Total between 4:00 and 6:12 pm 02/12/2010 (mm)	34	13.16	36
Peak intensity between 4:00 and 6:12 pm 2/12/2010 (mm/hr)	68	16.9	106
Sum of daily totals 28/11 – 2/12	112.4	101.14	94.4
Daily total to 9 am 04/12/2010	1.8	1.32	1.4
Daily total to 9 am 05/12/2010	11.6	20.3	9.6

Based on the Canberra IFD tables, the recorded intensities would place this event at between the 2 ARI and 10 ARI design rainfall intensities at Racecourse and Southwell Park, and less than a 1 year ARI at Barry Drive (Figure 3-7).

The recorded levels indicate that this event was just above a 3 year ARI at Barry Drive, and just above a 5 year ARI at Southwell Park. Refer to Section 9.1 for the flood frequency analysis based on gauge levels.

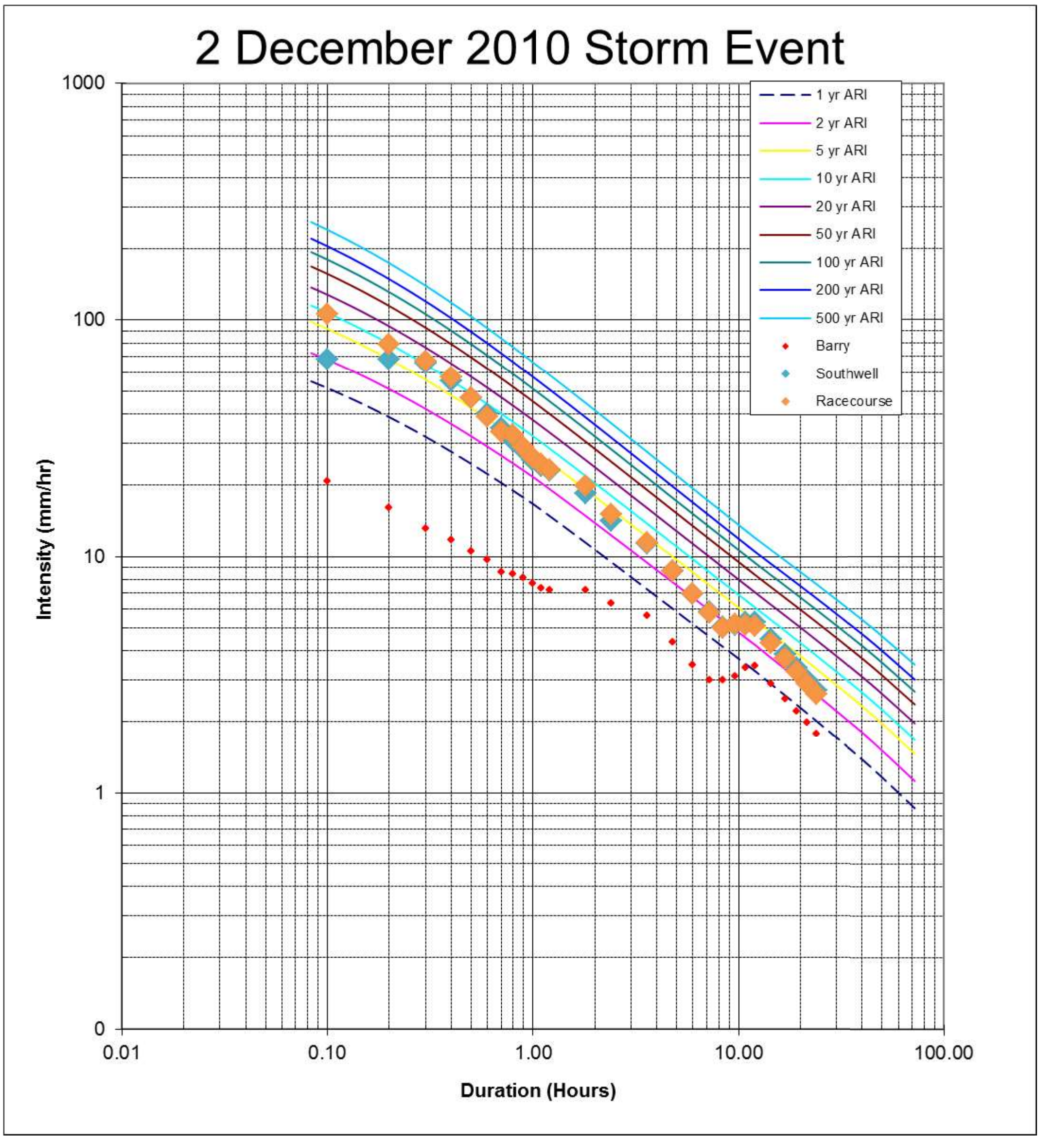

Figure 3-7 IFD curve for 2nd December 2010 rainfall event

Spatial variability

To understand the spatial distribution of the rainfall on the catchment an analysis of daily rainfall records was undertaken. Daily rainfall data from within the catchment and surrounding rainfall stations was collected and is presented in Figure 3-8 indicates that rainfall totals for the event tended to decrease from the upstream section of the catchment in the North-East to the downstream section of the catchment to the South-West. The temporal patterns were derived from the pluviograph data for Watson at Racecourse, at Southwell Park, and for Turner at Barry Drive as indicated by the shading.

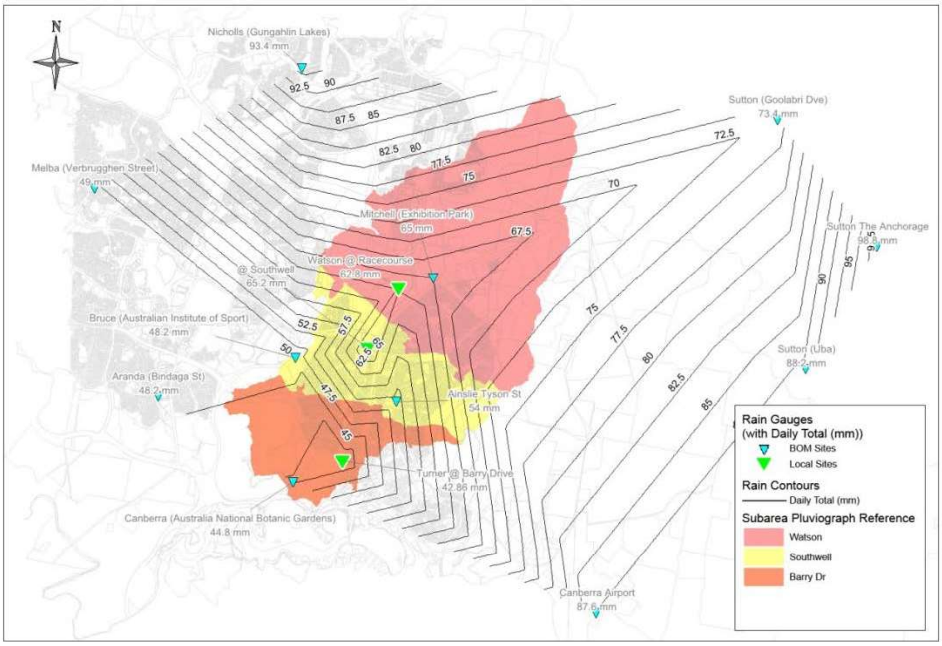

Figure 3-8 Spatial distribution of 2nd December 2010 rainfall event

3.2.3 February 2011 storm event

This event occurred on the 4th February 2011 at 2:30pm (EST), with 38.8mm of rainfall in 2 hours, with the highest intensity being 122mm/hr (12.2mm in 6 minutes). Spatial distribution is shown in Figure 3-9.

Spatial variability

To understand the spatial distribution of the rainfall on the catchment an analysis of daily rainfall records was undertaken. Daily rainfall data from locations within the catchment and surrounding rainfall stations was collected and is presented in Figure 3-9. This figure indicates that rainfall totals for the event tended to decrease towards the south and were higher in the upper portions of the catchment. The temporal patterns were derived from the pluviograph data for Watson at Racecourse, at Southwell, and Turner at Barry Drive as indicated by the shading.

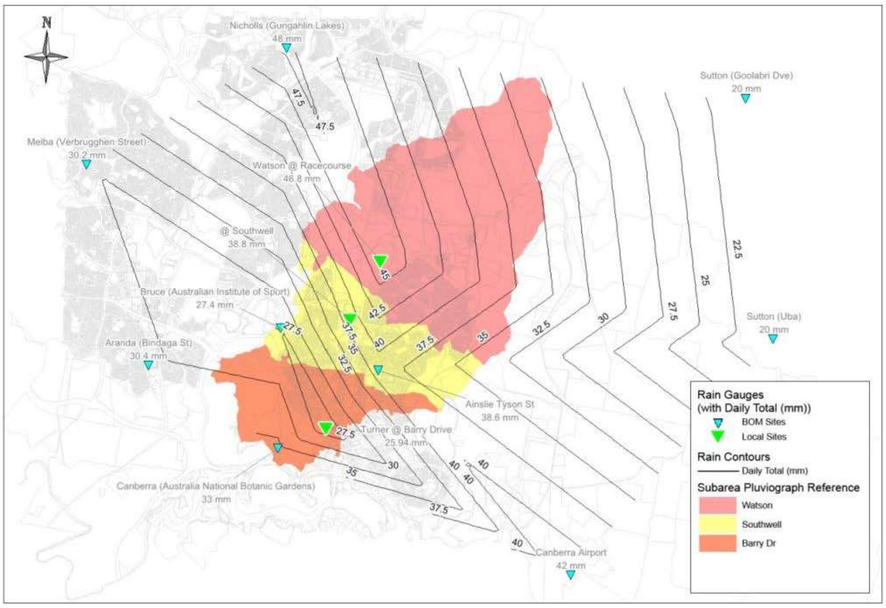

Figure 3-9 Spatial distribution of 4th February 2011 rainfall event

4. Hydrologic model establishment (RORB)

4.1 General

The hydrology for the Sullivans Creek Flood Study was developed using the RORB hydrological model.

RORB is a general runoff and stream flow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce runoff hydrographs at any location. It can also be used to design retarding basins and to route floods through channel networks (RORB 6 User Manual).

The main capabilities of this model with respect to this study include:

- Representation of temporal and spatial variation of rainfall and losses
- A number of calibration parameters to adjust model response to better match historic records of catchment response
- The generation of design hydrographs
- Ability to represent the effect of retarding basins
- Representation of base flow (although like RAFTS the model does not include the ability to model groundwater processes explicitly).

The model was setup to produce flood hydrographs for Sullivans Creek and adjoining floodways and channels upstream of the confluence with Lake Burley Griffin.

4.2 Impervious fractions

Impervious fractions were defined using the Land Use Area polygons (provided by ACTLIC) in combination with the TAMS guidelines. The adopted values by land use type are given in Table 4-1. , Manual overrides were also made as deemed necessary.

Table 4-1 Base impervious fraction by land use area

Land use area	TAMS guideline value	Adopted value	Comment
Residential	0.45 – 0.60	0.45	Spot checks of residential areas suggest maximum value of 0.45 is appropriate based on cadastre and building footprint layers and a 10% allowance for paving in line with TAMS guidelines. An impervious fraction of 0.7 was assumed for local roads in these calculations
Commercial	0.7	0.7	As recommended by EDD
Industrial	0.9	0.9	As recommended by EDD
Major roads		0.5	Based on spot checks of aerial photography.
Urban open space		0.05	

4.3 Configuration

Configuration of the RORB model included:

- Refinement of the sub catchments delineated in previous studies to suit locations where inflows were required in the TUFLOW hydraulic model. Catchments generally delineate key drainage paths and the drainage network. This resulted in 320 sub catchments;
- Catchment parameter determination, namely sub catchment area, reach lengths and slope
- Event rainfall and concurrent flood data compilation for calibration
- Design rainfall determination for generating design storm rainfall events for the 10%, 1%, 0.2% and 0.01% AEP together with the Probable Maximum Flood (PMF)
- The percentage of impervious catchment for each sub-area was estimated from aerial imagery in conjunction with the TAMS Design Guidelines
- The RORB model was established with four interstation areas which allows for the parameters to be refined throughout the catchment. This variation in parameters was used to change the initial loss and runoff coefficients for the upper catchment which has previously been identified the DUCR region.

Catchment maps and sub-catchment delineation are provided in Appendix A.

The RORB hydrological model was setup to provide estimates of both historic and design flows for input into the TUFLOW hydraulic model. The RORB model is used primarily to characterise the catchment and to convert rainfall into runoff hydrographs for routing within the TUFLOW model. Upstream of the hydraulic model, some routing in the upper subareas was undertaken in RORB. While flows are still routed within the RORB model to provide flow estimates directly from the RORB model for the purpose of preliminary calibration, such routing is relatively coarse and considered less reliable than the routing within the more detailed TUFLOW model.

4.3.1 Losses for the Delayed Upper Catchment (DUCR) response

The Kenny report (Cardno, 2011) quotes an initial loss of 20-40 mm in the upper catchment due to the geological characteristics before flow is observed. The behaviour of the DUCR region in larger events has not been documented and is therefore relatively unknown. Losses in the 20-40 mm range are discussed in the model calibration section.

4.3.2 Representation of the top of the Dickson Channel

The drain at the top of the Dickson Channel is believed to have approximately a 1% AEP capacity. Excess flows breaking away from this will flow overland in a generally westerly direction towards Sullivans Creek, or in a north-westerly direction to re-join the Dickson Channel further downstream. Diversions were configured in the RORB model so that half of the flow in excess of the 1% AEP (based on the peak existing flow with ARFs applied) was added to the subarea to the west of the drain, and half routed along the Dickson Channel.

5. Hydraulic model establishment (TUFLOW)

The flood conveyance through Sullivans Creek was calculated using the TUFLOW hydraulic model.

Hydraulic modelling was undertaken using TUFLOW version 2013-12-AB-iDP-w64 (Intel double precision 64-bit). TUFLOW is a hydrodynamic model used for simulating one-dimensional (1D) and two-dimensional (2D) flows. The model is based on the solution to the free-surface shallow water flow equations. The TUFLOW model consists of a 2D (TUFLOW) domain representing the catchment terrain, including open channels and some structures. A 1D (ESTRY) network represents the pipe systems and some of the smaller structures. A set of boundary conditions links the 1D and 2D elements and other boundary conditions apply of the calculated RORB inflow hydrographs and the downstream water levels. Variable 'z shapes' allow the height of the fuse-plug spillway and other embankment locations at Southwell Park to vary over time once a trigger level is reached.

A TUFLOW model was generated with a cell size of 5m by 5m for the 2D region to be mapped. The cell size was chosen as a compromise between accuracy of the DTM data, simulation run time, model stability and accuracy of the results. Initial testing indicated that a smaller cell size would improve the representation of the narrower sections of the channels and hence a finer resolution 3m by 3m grid was adopted for modelling of events up to and including the 1% AEP event. To avoid excessive model run times, the 5m square grid was retained for the extended duration runs used to estimate the duration of inundation and for the larger events (larger than 1% AEP) where conveyance in the smaller channels was relatively insignificant.

The model extent is shown in Figure 5-1.

5.1 2D model terrain

A Digital Terrain Model (DTM) was compiled to describe the topography of the Sullivans Creek catchment. This data included the following data sets, obtained by GHD in descending order of accuracy:

- Proposed road design for Morisset Road as included in the Cardno XP-SWMM model
- Road design for Horse Park Drive as included in the Cardno XP-SWMM model
- Proposed redevelopment design for Lyneham Sports and Recreation Precinct
- Terrestrial site survey of Morisset Road
- Terrestrial site survey of the Kenny development area
- Terrestrial site survey of the Throsby development area
- Terrestrial site survey of the ANU Campus
- 3m gridded LiDAR survey of the catchment supplied by ACTLIC
- 2m topographic contours.

Terrestrial survey was commissioned for key features where data was missing, such as bridges, and particular areas where the LiDAR data is either significantly out-dated or considered to not adequately represent the topography of the area. Differential GPS survey points were also taken at Southwell Park, Cowper St, ANU weir and the Lyneham Wetlands weir where initial model runs indicated more detailed data was required than represented in the 3m grid data supplied.

It should be noted that the ACT Government does not possess LiDAR for the north-eastern corner of the catchment; however the availability of terrestrial survey covers the required 2D modelling extents. The 2m contours were adopted for the development of the DEM outside these modelling extents.

This investigation assumed that the designs for road upgrades to Horse Park Drive, Morriset Road and Lyneham Sports and Recreation Precinct would be considered as part of the "Existing Scenario".

No detailed bathymetric data was available for the outlet into Lake Burley Griffin. It is believed that the depth under Parkes Way is approximately 2.6 m, based on a previous model (Cardno). Depths were interpolated between the downstream side of the ANU weir and Parkes Way. It may be worth undertaking an up to date bathymetric survey of Sullivans Creek outlet if further detailed work is required.

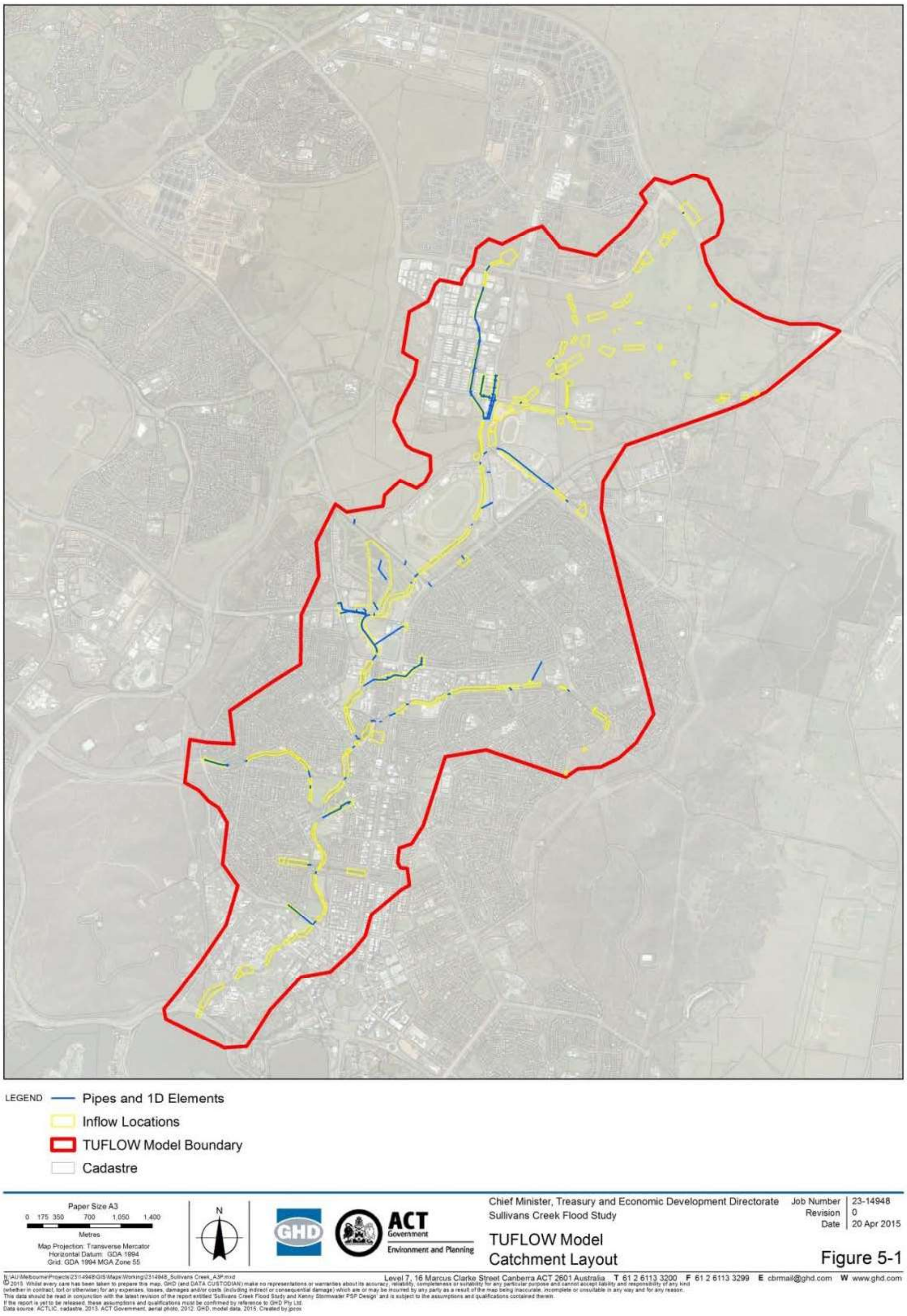

Figure 5-1 TUFLOW model layout

5.2 Roughness coefficients

Based on aerial photography and site inspections, hydraulic roughness coefficients were spatially defined. The table below lists general roughness assumptions made.

Table 5-1 Manning's 'n' roughness assumptions

Feature	Manning's 'n' Value
Sealed Roads	0.02
Concrete lined drains	0.015
Waterways and waterbodies including grassed swales	0.03
Parks and well maintained open space	0.04
Developed areas (residential, commercial, industrial)	0.10
Lightly vegetated areas	0.03
More densely vegetated areas	0.10
GPTs and very rough areas	0.30

5.3 Representation of channels

The representation of channels was generally undertaken in the 2D domain. There are limitations of such an approach when trying to represent channel features in the 2D domain which are not substantially bigger than the grid size, in particular the conveyance on the diagonal of such channels can be underestimated. To reduce the effect of this a combination of gully lines and averaging of grid values was undertaken. The alternate approach of using 1D channels within the 2D domain was considered. Given the resolution of the underlying data and issues with the interface between the 1D and 2D domain (potential to over or under count the conveyance of narrow channels and the potential for stability issues) it was decided that the adopted 2D gridded representation was preferable.

5.4 Representation of buildings

In general a composite Manning's 'n' was applied to developed areas rather than separating buildings out from the surrounding block. This approach means that flood extents are likely to be more representative in areas of re-development than if buildings had been represented explicitly. At the request of CMTEDD, buildings in the Dickson town centre were "blocked out" to better estimate velocity and depth. The building footprints from the ACTLIC database (06/05/2014) were raised by 3m, with several other buildings manually digitised based on the aerial photograph (13/07/2012). The buildings included are indicated in Figure 5-2 below.

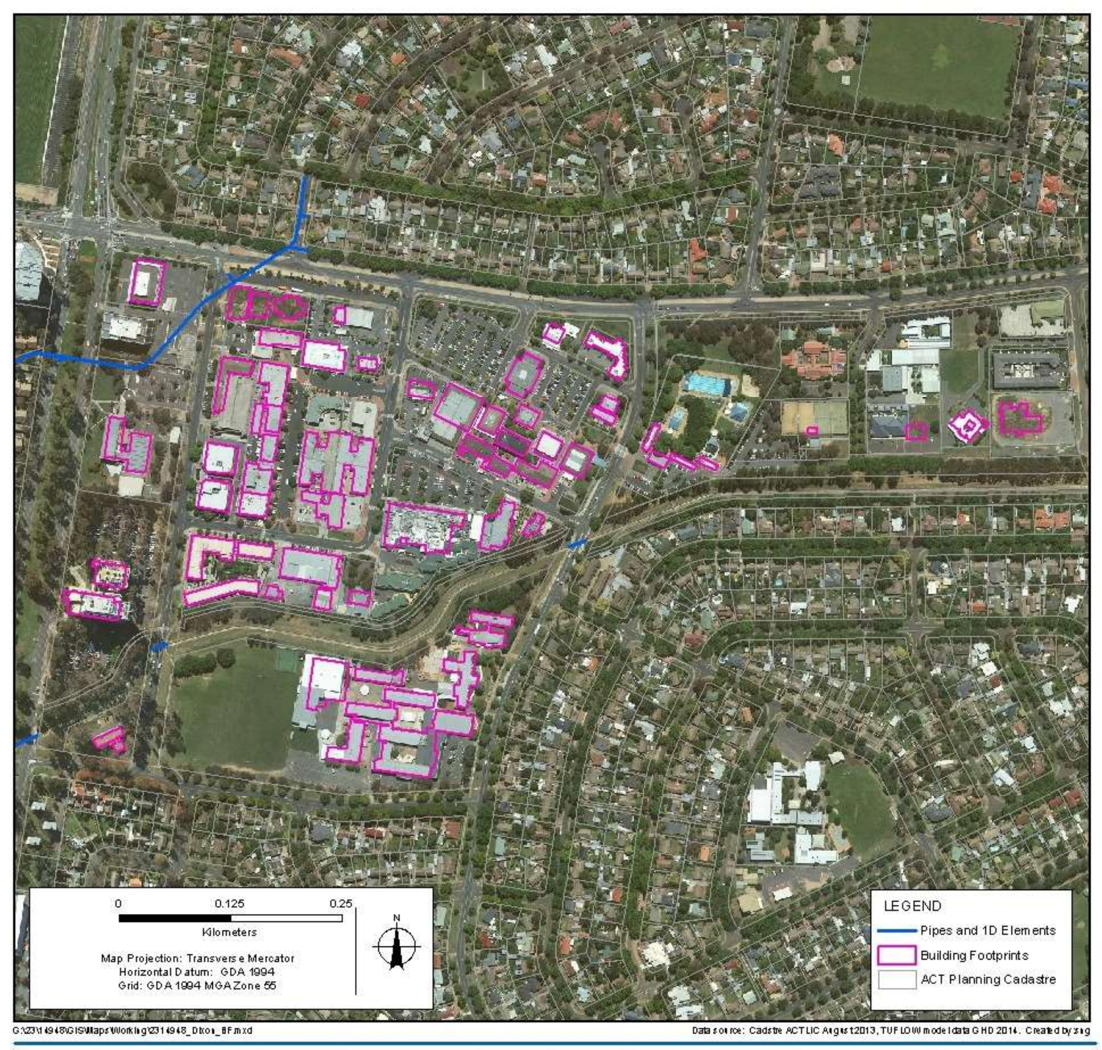

Figure 5-2 Dickson building footprints

5.5 Boundary conditions

5.5.1 Inflow locations

The flood hydrographs output by the RORB model were configured as inflows for all sub catchments draining to the main drainage network for Sullivans Creek. Inflows were applied along the channels to be mapped, except in cases where significant trunk drainage or other features would result in a flow split (e.g. pipe and overland flows in separate directions), or there is significant attenuation or flood storage. Areas where inflows are applied to side branches, and may contribute to overland flow along with mainstream flooding are marked on the flood maps in Appendix B.

5.5.2 Downstream boundary conditions

Each event had a fixed tailwater condition at Lake Burley Griffin applied as the downstream boundary condition. The level applied depended on the event in question, and was based on the downstream rating curve and event flows adopted in the 2000 Ecowise modelling.

The level at Lake Burley Griffin applied to the TUFLOW model as a downstream boundary condition is shown by event in Table 5-2.

Table 5-2 Lake Burley Griffin levels assumed as downstream boundary conditions

Event	Assumed lake level (m AHD)
1% or greater AEP	555.93
0.4% AEP	556.33
0.2% AEP	556.61
0.01% AEP	557.18
PMF	558.90

5.6 Structures

Bridges and hydraulic structures within the model extents were configured using a combination of 1D and 2D structures using terrestrial survey and the ACT Government stormwater network information (“Chapman Plans”) and previous modelling. Where data was missing or inconsistent, the details were checked using a combination of available imagery and confirmation in the field by local resources. Invert levels were assumed to be approximately 0.9m below the ground surface level where no detailed survey data was available.

Where details were unknown, minor footbridges were assumed to have:

- Deck level based on surrounding DEM
- Deck thickness 200 mm
- Handrail height 1.0 m, with 5% blockage for rails

Minor retarding basins within development areas north west of Kenny (north of Wells Station Drive) were not included in either RORB or TUFLOW due to:

- Scale of works compared to the contribution of flow to the main catchment, particularly during larger events
- Insufficient detail provided within the DEM to represent these features within either RORB or TUFLOW.

Culvert crossings through Horse Park Drive and Federal Highway bordering Kenny were not modelled in RORB or TUFLOW as contours suggest there is little storage upstream of these roadways, and the majority of the flow can either pass through the culverts during the 10% AEP event, or will overtop the road crest and be overland flow during the larger events, without significant attenuation.

1D network details (stormwater drainage and structures) were extracted from the XPSWMM model (supplied by Cardno), and HEC-RAS model data (Ecowise 2000) and GIS data provided by the Environment and Planning Directorate (EPD). Where there were discrepancies between the sets of data, the GIS data was generally adopted over the XPSWMM model data. The exception is for drainage near Morisset Rd/Flemington Rd where XPSWMM model data was adopted as this represented additional drainage following the proposed upgrade of Morisset Rd.

The 1D and 2D structures that were modelled in TUFLOW are summarised in Table 5-3.

Table 5-3 Summary of 1D and 2D structures modelled

Reach	1D structures	2D structures
Sullivans Creek below Horse Park Drive to confluence with Mitchell channel	Horse Park Drive Old Well Station Road Morisset Road Flemington Road	Morisset Wetlands and Weir
Mitchell channel	Well Station Drive Vicars Street Callan Street Lysaght Street Baillieu Court Heffernan Street Winchcombe Court Sandford Street Underground drainage below channel	Randwick Wetlands
Sullivans Creek (Mitchell channel to Dickson channel)	Randwick Road Drainage along Flemington Rd Paceway Exhibition Aspinall Road culvert Trunk underground drainage network from Federal Highway near Stirling Avenue Racecourse box culverts Barton Highway Golfcourse access bridge Southwell Park underground culverts Drainage network from Mouat Street Drainage network from Northbourne Avenue Southwell Park Outlet culverts and pit Drainage network from Pigot Street Goodwin Street	Southwell Park Basin Footbridge at Lyneham basketball courts Footbridge near Fox Place

Reach	1D structures	2D structures
Dickson channel	Majura Avenue Underground network from Antill Street under Dickson District Playing Fields Dickson Wetlands outlet system Cowper Street Challis Street Northbourne Avenue De Burgh Street Lyneham Wetlands GPT culverts Low flow culvert from Lyneham Wetlands	Footbridge near Kellaway Street Footbridge near Calvert Street Footbridge near Dickson College Footbridge near Dickson District Playing Fields Footbridge at Dickson Wetlands Dickson Wetlands Footbridge near Hawdon Street Footbridge near Dickson Aquatic Centre Access way bridge at Lyneham Wetlands inlet Lyneham Wetlands Footbridge across Lyneham Wetlands Owen Crescent at Lyneham Wetlands
Sullivans Creek (Dickson Channel to O'Connor Channel)	Wattle Street Macarthur Avenue	
O'Connor Channel	Drainage network from intersection of Dryandra and Fairfax Streets Miller Street Macarthur Avenue O'Connor Oval access from Pedder Street	Footbridge near Banksia Crescent Footbridge near Moorhouse Street
Sullivans Creek downstream of O'Connor Channel to Lake Burley Griffin	Drainage network from Towns Crescent through Turner School and Playing Fields McCaughey Street (Haig Park) Masson Street Barry Drive Drainage network from north side of Barry Drive through Willow Oval	David Street Condamine Street Footbridge near Bent Street (Haig Park) Stepping stones downstream of Barry Drive Footbridge near Sports Union Extension Union Court Footbridge near Library and Arts Centre Fellows Road Ward Road Parkes Way (x2)

5.7 Wetlands

Further details on the wetlands modelled in TUFLOW are summarised in Table 5-4.

Table 5-4 Wetland summary

Wetland	Location	Date constructed	Comment
Dickson	Southern side of the Dickson Channel from the Dickson District Playing Fields	Sept 2010 – Oct 2011	Not included in RORB model as offline and does not significantly affect flows above 2-year ARI
Lyneham	Near the confluence of Dickson Channel and Sullivans Creek in Lyneham.	Sept 2010 – May 2012	
Morisset	At the intersection of Morisset Road and Flemington Road in Kenny	2005	
Randwick	At the intersection of Randwick Road and Flemington Road in Mitchell	Dec 2008 – July 2010	

5.8 Southwell Park

Details for Southwell Park were taken from several sources including design drawings, and survey. The failure mode for the fuseplug was required for extreme event analysis and required researching old information documented by [redacted], and also contacting the original designer, [redacted].

According to [redacted] (2008) "*Its concept was to develop a retaining embankment around the downstream edge of Southwell Park along a boundary road – then the Barton Highway (embankment length approximately 800m, a crest = AHD 572.75 and with a storage capacity of 280 Ml for the 100 year flood event – 40% of the total runoff volume), with multiple spillways for regulation up to the 150 year ARI, while the PMF was assessed as 484 cumecs and would peak at AHD 572.67. A primary spillway was developed to regulate the frequent flows, which consisted of a constrained lined channel profile/culvert (up to 5 year ARI = 26 cumecs with twin 2.4m x 1.8m box culverts) plus a higher level intake culvert, AHD 571.2. A fuseplug secondary spillway arrangement was provided for handling rarer flows over the then Barton Highway/ boundary road; to operate/breach at the 150 year ARI flood level (AHD 572.2) and to contain the PMF at a level below the long embankment crest. The fuseplug consisted of a lowered crest at AHD 572.4 (generally 0.35m below the long embankment crest at AHD 572.75) and three components: one being a starter concrete chute (IL 572.9) to warn motorists of the possible fuseplug operation; an initiating fuseplug on the LHS (length of 5m with a crest level AHD 572.1, designed to fail in a short time of several minutes); and with a lateral component on the RHS (crest AHD 572.4) with both components constrained within a concrete batter at each end with a total fuseplug length of some 60m. The fuseplug has an impervious crest and upstream batter section with a pervious downstream section; the initiating LHS fuseplug component has a full pervious batter to permit quick failure, while the RHS component has both a pervious section and an impervious batter to slow down failure. So the fuseplug would progressively fail by headward erosion starting from the LHS and moving laterally to the RHS; it would be able to discharge the PMF in conjunction with the three primary spillway culverts. As well, it would regulate the outflow discharge of rare flood events in between the primary spillway outflow and the PMF. The basin has been designed specifically to give maximum peak flow reductions for flood events between the 50-150 year frequencies by storing the peak of such flows. For the 100 year ARI the flow reduction at the basin for the existing conditions is from 106 cumecs to 58 cumecs, while the assessed period to empty a 100 year storm is 6 hours. The operation of the fuseplug was intended to be such that outflow would not exceed inflow and/or flows without the basin - undoubtedly brave words. In fact, the retarding basin has not had to operate over its 30 years of life. There have been some minor flood events such as an approximate 50 year ARI rural catchment rainfall event, and several periods when the lined channel has run a banker. The Southwell Park Retarding Basin was an economic development, which protected downstream urban development, avoided major bridge augmentation and lengthy channel augmentation. It was the classical means of providing flood protection for the existing development and an urbanising upper catchment. The multiple spillway arrangement was a well-planned and innovative low cost solution. With upper catchment urbanisation, two other retarding basins were contemplated and one basin has now been constructed (in 2005). "*

[redacted] provided copies of original drawings from a design report by Willing and Partners (1979) as well as estimates of the design failure mode used.

5.8.1 Key parameters

The key Southwell Park basin parameters are summarised in Table 5-5.

Table 5-5 Southwell Park Basin key parameters

Parameter	Value	Source	Comment
Embankment crest length	800 m	[redacted]	
Embankment crest level	572.75 m AHD 572.6 m AHD	[redacted] DGPS	Assumed settlement has occurred
Fuse-plug crest level	572.4 m AHD 572-572.2 m AHD	[redacted] DGPS	
Higher level intake culvert	571.2 m AHD 570.57 m AHD	[redacted] DGPS	Lowered following Cardno study
Main outlet culverts	Twin 2.4 m x 1.8 m	[redacted]	
Starter chute	571.9 m AHD 571.8 m AHD	[redacted] DGPS	
Initiating fuse-plug IL	572.1 m AHD 572.0 m AHD (low-point at 571.8 m AHD)	[redacted] DGPS	Assumed predominant level governs failure initiation.
Level at which fuse-plug designed to activate	200 mm above sill level	Willing and Partners,1979	Equates to 572.2 m AHD
Initiating fuse-plug length	5 m	[redacted]	
Initiating fuse-plug failure time	Several minutes, 2 minutes	[redacted] Goyen (pers. com)	
Natural surface level below fuseplug/ embankment	570.3 m AHD	[redacted]	
Main outlet culvert IL	567.0 m AHD	DGPS	
Time for main fuse-plug to erode	Approximately 2 hours for lateral failure, quicker when overtopping occurs	Willing and Partners,1979	Vary failure time with event
Total fuse-plug length	60 m	[redacted] DGPS	
Basin volume	280 ML	[redacted]	
Basin volume at spillway level (571.8 m AHD)	250.6 ML	12d calculations	
Basin volume at assumed fuse-plug failure level (572.2 m AHD)	364.6 ML	12d calculations	
Basin volume at main embankment crest level (572.6 m AHD)	517.0 ML	12d calculations	

5.8.2 Fuse-plug spillway

It is understood that the fuse-plug spillway was designed such that the main embankment would not overtop as the lateral failure of the fuse-plug is intended to allow sufficient discharge, whilst still being less than the flows were the basin not present ([redacted] 2008). The design report (Willing and Partners, 1979) gives a failure time of two minutes for the initiating section, triggered by flood levels 200 mm over the sill. The main section of the fuse-plug was designed to be select fill on the upstream side, and gravel and sand on the downstream side ([redacted] personal coms). A lateral failure rate of 0.4 m/min for the main section is given and notes that overtopping failure will dominate in larger events, leading to a faster failure time. Assuming the side slopes of the initiating section of the fuse-plug are representative of the concrete batters which would constrain the fuse-plug breach, side slopes of 2 H: 1V were adopted. Based on the volume of water impounded between 572.3 m AHD (100 mm above the main section) and natural surface level, along with various fuse-plug dimensions the overtopping failure time was estimated using a number of empirical equations. The estimates range from 20 minutes (MacDonald Langridge-Monopolis) to 75 minutes (Froehlich). It should be noted however that these equations are based on data from a limited number of spontaneous failures rather than an engineered fuse-plug.

For modelling purposes the fuse-plug was configured in five sections as shown in Figure 5-3 to represent the intended lateral failure. The first section is the gravel "initiating fuse-plug", whilst the remaining length of the main fuse-plug is divided into four sections to simulate progressive failure. The first section fails in one quarter of the total failure time, the second in half the total failure time, the third in three quarters of the total failure time, whilst the fourth section takes the entire time to fail completely.

Figure 5-3 Southwell Park fuse-plug spillway

Given the uncertainty around failure times a sensitivity analysis was undertaken on the breach time of the fuse-plug whilst the model was under development to assess the impact on maximum flood levels upstream and downstream of the basin. Three failure times were simulated; 20 minutes, 60 minutes and 90 minutes. These results are presented in the sensitivity analysis section in Table 8-2.

5.9 Model calibration

5.9.1 Methodology

The RORB model was initially calibrated by variation of model parameters to obtain a reasonable fit of the calculated hydrograph to the measured hydrograph in RORB. The parameter k_c is the main means of achieving a fit. The parameter, k_c can be decreased to increase the hydrograph peaks and decrease the lag time. Conversely, increasing k_c does the opposite. In addition to k_c, which affects hydrograph timing, varying the loss parameters provides a method of adjusting the volume of rainfall which becomes runoff. A further means of altering the catchment response is using the “m” parameter (a measure of the catchment’s non linearity) however use of this parameter for calibration is less common. Once a reasonable fit was obtained in RORB, both the RORB and TUFLOW models were run in sequence to test the calibration of the model.

5.10 Initial RORB parameter selection

5.10.1 Regional k_c parameters

A number of regional estimates for the determination of k_c are available in the literature and in the Australian Rainfall and Runoff (ARR, 2001). A number of these are offered within the RORB model for use as a guide. The TAMS guidelines suggest the RORB default equation be utilised. For the Sullivans Creek RORB model, various regional estimates of k_c parameters are tabulated in Table 5-6 below.

Table 5-6 Regional RORB kc estimates

Method	k_c Estimate
Eastern NSW (Kleemola) (Eqn 3.20, ARR (Book V) $k_c = 1.22\ A^{0.46}$	7.52
Australia Wide – Dyer (1994) data (Pearse et al, 2002) $k_c = 1.14\ Dav$	9.46
Australia Wide – Yu (1989) data (Pearse et al, 2002) $k_c = 0.96\ Dav$	7.96
RORB Default – Eqn 2.5 (RORB Manual) $k_c = 2.2A^{0.5}\ (Qp/2)^{0.8-m}$	15.87

Initial RORB simulations showed that the TAMS design parameters were not appropriate for the Sullivans Creek catchment based on the available observed data. The simulated peaks were considerably higher than those observed, and significant time lag was also produced compared to the recorded hydrographs.

A k_c value of 14.0 was adopted which best suited the Sullivans Creek catchment, along with typical “m” value of 0.8.

5.11 Representation of recent features in calibration events

Although the chosen calibration events are reasonably recent they occurred prior to construction of a number of features now in existence. To better match conditions at the time the existing conditions model was modified in a number of ways as summarised in Table 5-7.

Table 5-7 Model changes for calibration

Key Catchment Feature	2002 calibration event	2010 calibration event	2011 calibration event
Randwick Wetland	Not Included	Included	Included
Southwell Park Sports Precinct including associated remodelling of open channel and storage	Not Included	Not Included	Not Included
Dickson Wetland	Not Included	Not Included	Not Included
Lyneham Wetland	Not Included	Not Included	Not Included

5.12 Adopted parameters and calibration results

The adopted RORB model parameters for the calibration events are summarised in Table 5-8 through

Table 5-10. The RORB and TUFLOW results for each event are plotted along with the gauge (flow and height) data in Figure 5-4 to Figure 5-9.

Table 5-8 Adopted RORB parameters for 2002 calibration event

	DUCR Area (u/s Kenny)	u/s Southwell Park	u/s Barry Drive	d/s Barry Drive
Initial Loss (IL mm)	30	0	11	10
Runoff Coefficient (RC)	0.2	0.2	0.2	0.2

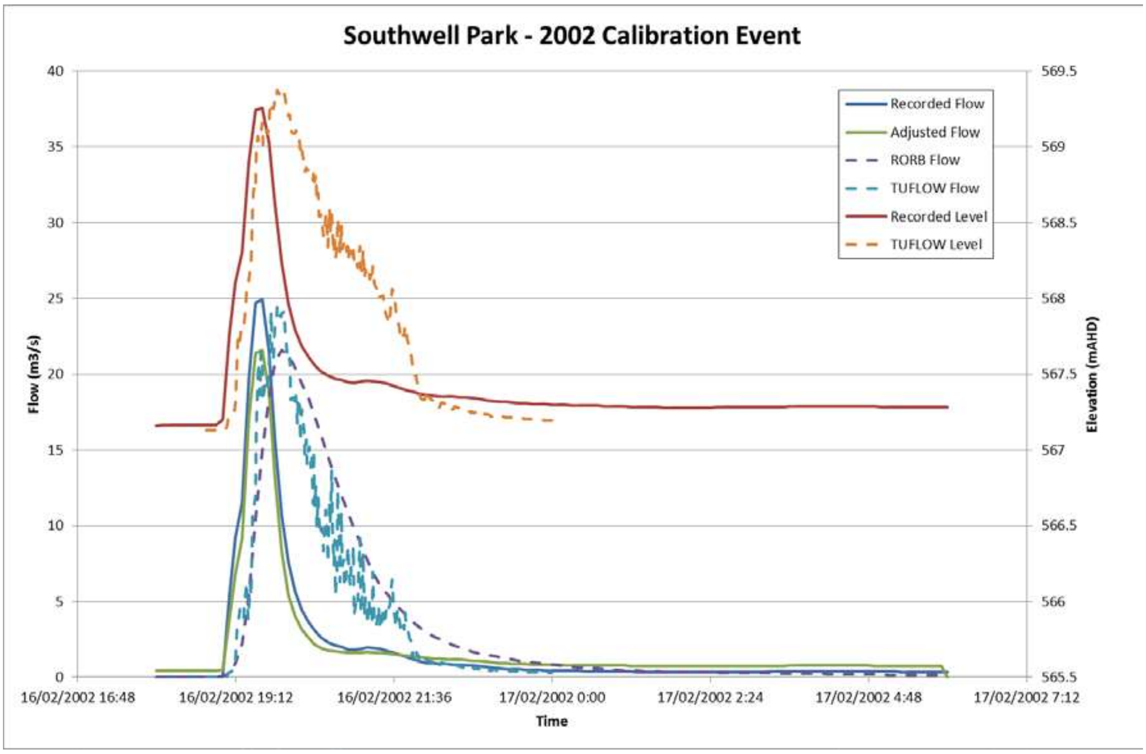

Figure 5-4 TUFLOW results at Southwell Park for 2002 calibration event

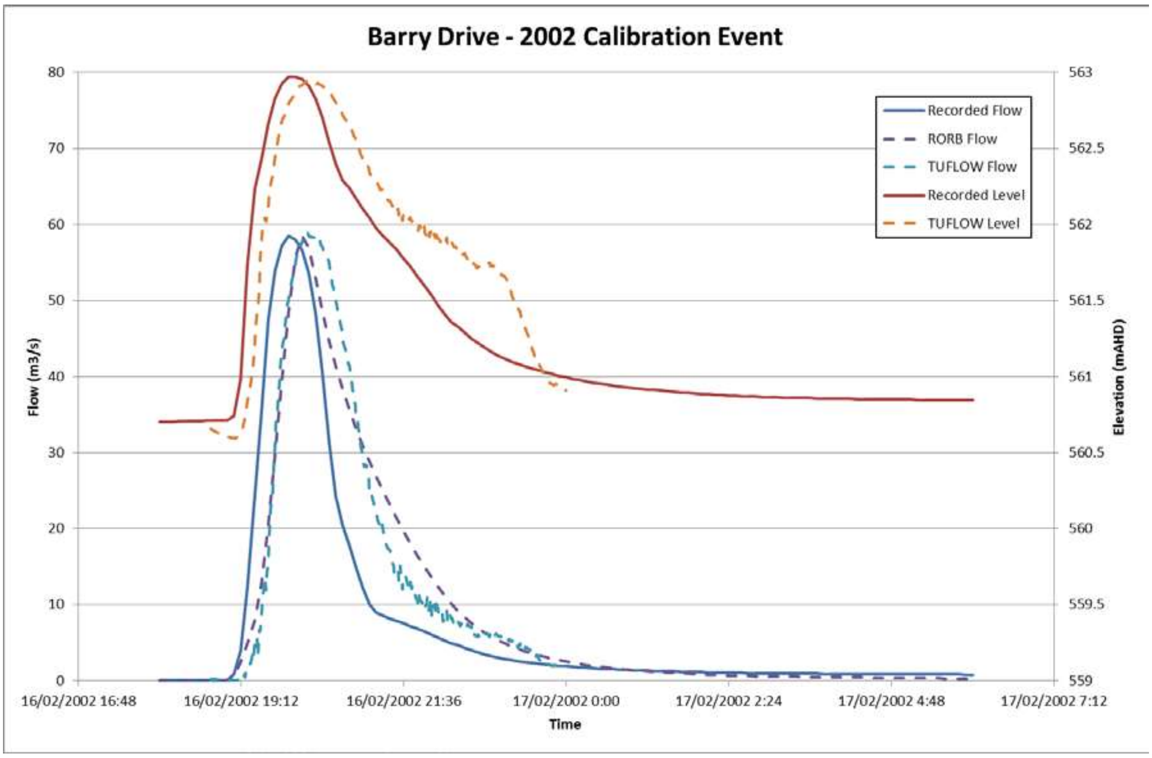

Figure 5-5 TUFLOW results at Barry Drive for 2002 calibration event

Table 5-9 Adopted RORB parameters for December 2010 calibration event

	DUCR Area (u/s Kenny)	u/s Southwell Park	u/s Barry Drive	d/s Barry Drive
Burst 1 Initial Loss (IL mm)	40	20	0	0
Burst 1 Runoff Coefficient (Rc)	0.2	0.2	0.2	0.2
Burst 2 Initial Loss (IL mm)	0	0	0	0
Burst 2 Runoff Coefficient (Rc)	0.2	0.2	0.2	0.2
Burst 3 Initial Loss (IL mm)	0	0	0	0
Burst 3 Runoff Coefficient (Rc)	0.2	0.2	0.2	0.2

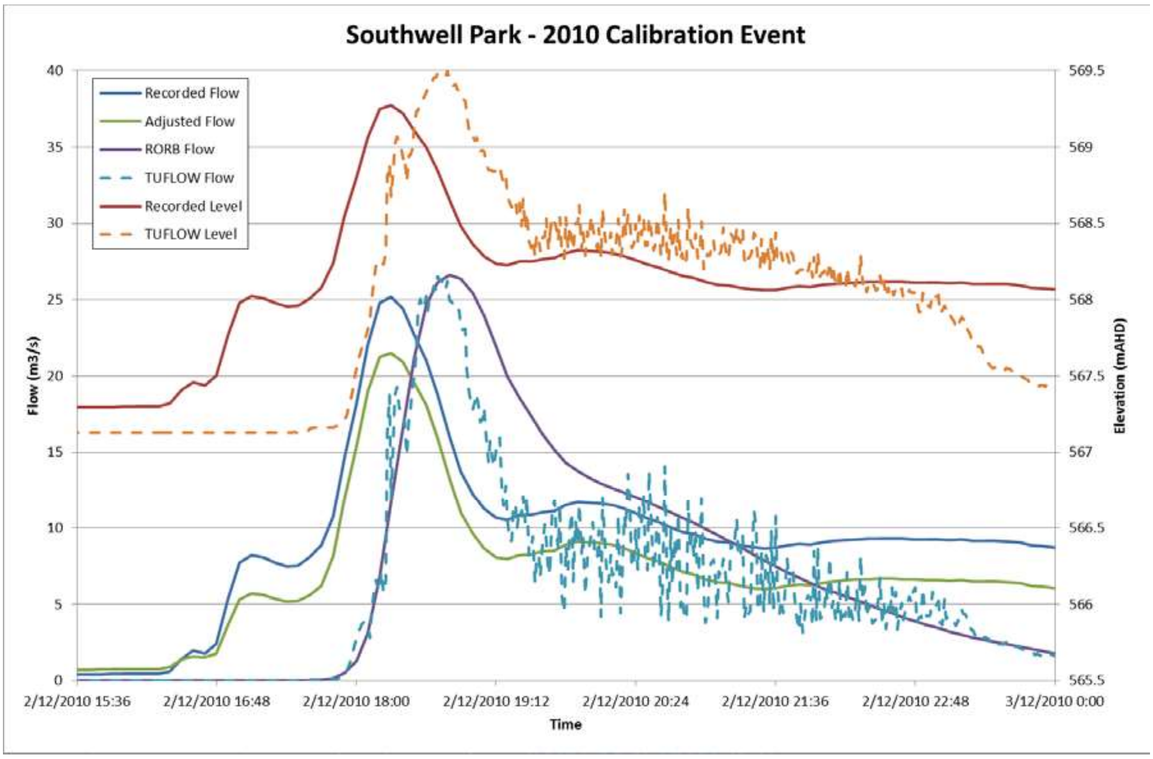

Figure 5-6 TUFLOW results at Southwell Park for 2010 calibration event

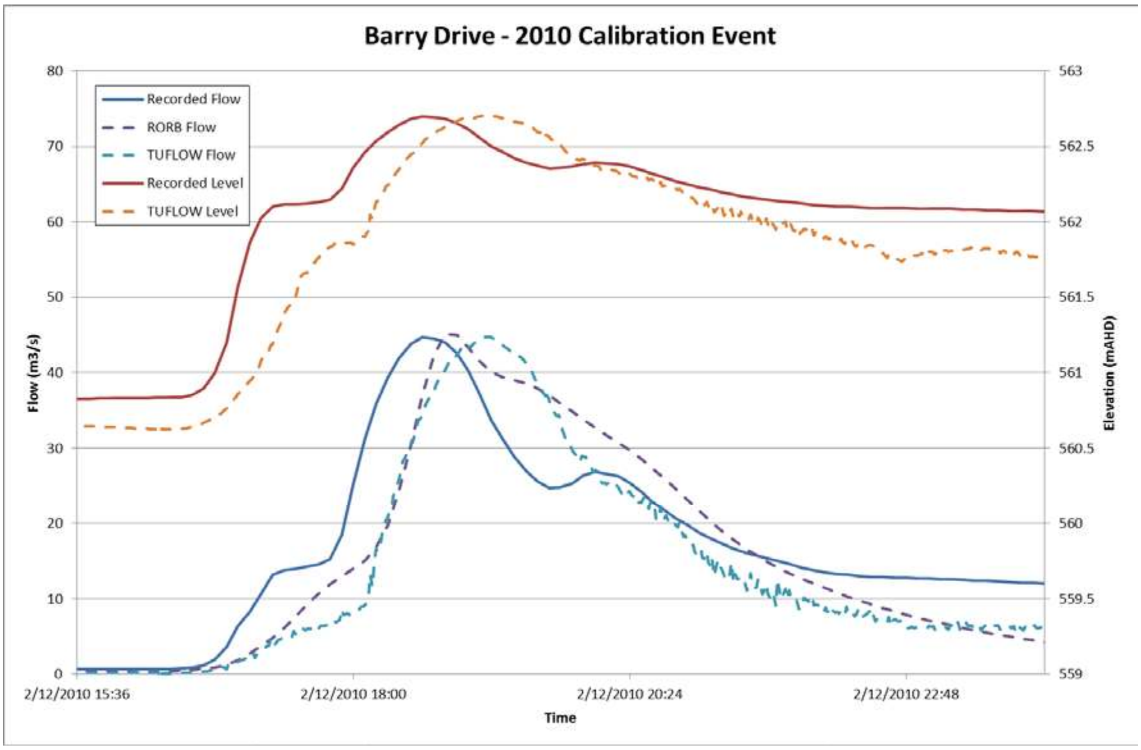

Figure 5-7 TUFLOW results at Barry Drive for 2010 calibration event

Table 5-10 Adopted RORB parameters for February 2011 calibration event

	DUCR Area (u/s Kenny)	u/s Southwell Park	u/s Barry Drive	d/s Barry Drive
Burst 1 Initial Loss (IL mm)	30	15	10	5
Burst 1 Runoff Coefficient (Rc)	0.2	0.2	0.2	0.2
Burst 2 Initial Loss (IL mm)	30	13	8	5
Burst 2 Runoff Coefficient (Rc)	0.2	0.2	0.2	0.2

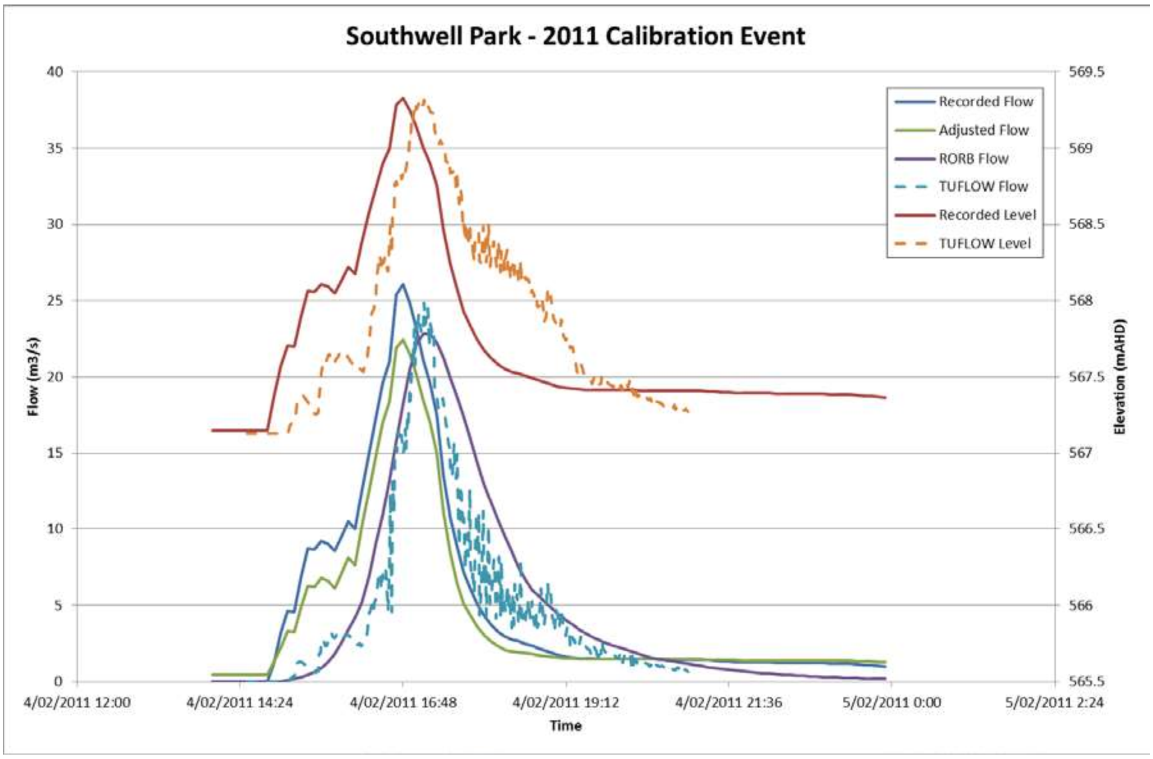

Figure 5-8 TUFLOW results at Southwell Park for 2011 calibration event

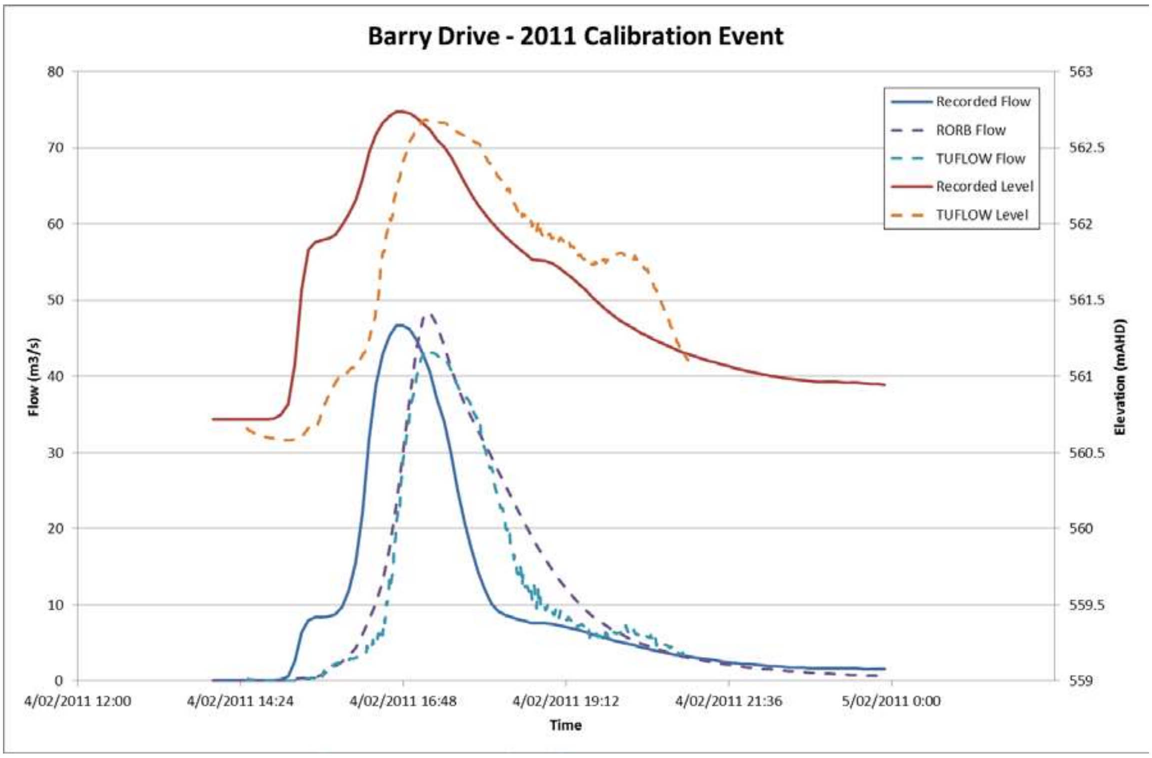

Figure 5-9 TUFLOW results at Barry Drive for 2011 calibration event

5.12.1 Discussion of results

2002

Results obtained from the TUFLOW simulation for the 2002 rainfall event reveal that the predicted levels at Southwell Park are 0.15 m above the recorded levels and peak flow is approximately 3.4% lower than the recorded flow at the Southwell gauge. The peak flow generated from RORB for the same event is lower than the recorded flow however very similar to flows adjusted using the gauge levels and rating curve from the Cardno study “adjusted flow”.

Similar results are generated at Barry Drive for the 2002 event where the TUFLOW level is 0.03m below the recorded level whilst the peak flow is 1.1% less than the recorded peak flow.

Based upon the 2002 calibration results, the fit of the adopted RORB parameters for the 2002 calibration event indicate that it is a good fit for TUFLOW modelling.

2010

In addition to the 2002 calibration event, the 2010 event has also been calibrated to generate a set of RORB parameters that not only match the 2002 parameters, but also generate a good fit to the recorded results at both the Southwell Park and Barry Drive gauges.

Based upon the adopted 2010 parameters and the subsequent results, the TUFLOW peak water level at the Southwell Park gauge is 0.23m above the recorded level and the peak flow is 5.4% higher than the recorded flow. Peak flows are comparable as they are of a similar magnitude but a timelag is apparent. It is noted that beyond the peak there are some instabilities within the TUFLOW water level and flow results however these are not considered to be significant as a considerable amount of time has elapsed since peak conditions.

The TUFLOW results at Barry Drive upon visual inspection yield a good fit between the recorded and predicted results. The TUFLOW peak water level at Barry Drive is the same as the recorded level and the peak flow is effectively the same as recorded peak flow at the Barry Drive gauge. The timing of the peak flow occurs at similar times albeit the TUFLOW results are lagged by approximately half an hour.

Based upon the 2010 calibration results, the fit of the adopted RORB parameters for the 2010 calibration event indicate that it is a good fit for TUFLOW modelling.

Two flood height observations were made within the catchment for the December 2010 flood, one at Barry Drive and one at nearby Condamine Street. For Barry Drive this was the same as the maximum gauge level. For Condamine Street the recorded level was 565.4 m AHD, approximately 1 m higher than the simulated maximum.

While a small amount of this discrepancy is likely to be the result of velocity head; that is, a comparison of a water surface level (modelled) with a total energy line (observed debris mark) this is still a sizeable discrepancy. There are many potential reasons for such a discrepancy including uncertainties in modelling both hydrologic and hydraulic, measurement errors, uncertainty of debris marks, wave action, effects of blockages, or potential datum issues. A single outlier like this would not necessarily raise concern if there were many other levels which were a closer match.

In the absence of additional data it is difficult to be conclusive about this comparison other than it indicates a level of uncertainty. Given the good match at Barry Drive two crossings downstream, the discrepancy is most likely to be due to inaccuracies in the debris mark level although a localised variation in the model remains a possibility.

2011

TUFLOW results based on the adopted calibration parameters for the 2002 and 2010 events have resulted in good fits to the recorded levels at both the Southwell and Barry Drive gauges. By comparing an additional calibration event to the 2002 and 2010 events based on the RORB parameters, predicted results that match the recorded results will not only strengthen the validity of the RORB parameters that are adopted, but also verify the consistency of the recorded data.

TUFLOW results generated from the RORB parameters adopted for the 2011 calibration event show that the peak water level at Southwell Gauge is 0.17 m higher than the recorded level whilst the peak flow is 4.6% lower than the recorded level. Similar characteristics observed within the 2002 and 2010 calibration results as the time lag of the peak flows in the 2011 calibration event is once again prevalent.

The peak water level at Barry Drive is 0.18 m above the recorded gauge level and peak flow is 7.1% lower than the recorded peak flow. Predicted peak flow at the Barry Drive gauge is delayed by approximately half an hour which is again consistent with the other calibration event results.

Based upon the 2002, 2010 and 2011 calibration results, the TUFLOW results generated from the calibrated RORB parameters indicate a good fit to the recorded levels at both Barry Drive and Southwell Park gauges. Thus the model and its parameters have been adopted for all subsequent TUFLOW modelling.

6. Design event parameters

6.1 Rainfall losses / runoff coefficients

The following initial loss parameters were adopted based on the calibration events, and have been adjusted for rarer/ larger events.

From the calibration process runoff coefficients were used for the 10% and 1% AEP events.

Book VI ARR99 makes the following comments on loss models: The specific recommendations in Section 4.3 apply to loss parameters for the initial loss-continuing loss (IL-CL) model, as a large body of relevant experience has been accumulated over many years. However, other loss models may be used if they can be shown to be more appropriate in the specific situation. The use of a proportional loss (PL) model for continuing losses implies that the actual depth of losses increases with increasing rainfall magnitude. This is not very plausible in the range of Large to Rare and Extreme events.

For extreme flood estimation it is generally not recommended that the use of the IL-PL model be used, unless you have experience values to define a gradual increase in the runoff coefficient with increasing event magnitude. Therefore, for larger events a continuing loss model was used. The initial loss and continuing loss used for the larger events were estimated based on best matching volumes for the 1% AEP event (for all locations for the 3-hour duration)

In general the losses adopted within the catchment are larger than those considered typical for an urban area. Various reasons have been considered for this; including the fact that much of the development predates the ARR major and minor drainage concept, so that sags in roads may result in water ponding up to a threshold (Cardno 2011); the underlying geology, types and size of gardens as well as maturity of trees and shrubs.

Given that large flows have not been recorded in the catchment (maximum flow estimate of 58.6 m3/s at Barry Drive and 36.9m3/s at Southwell Park), how it will respond in larger events when saturation levels change infiltration rates, cannot be reliably informed by historical data. Log-log interpolation was therefore used to increase the runoff coefficient and decrease the continuing loss as the AEP decreased, as shown in Figure 6-1 and Figure 6-2 .

Table 6-1 Initial losses for design storms (excluding PMF)

	DUCR Area (u/s Kenny)	u/s Southwell Park	u/s Barry Drive	d/s Barry Drive
Initial Loss (mm) - all design storms except PMF	30	10	10	10
Initial Loss (mm) – PMF only	0	0	0	0

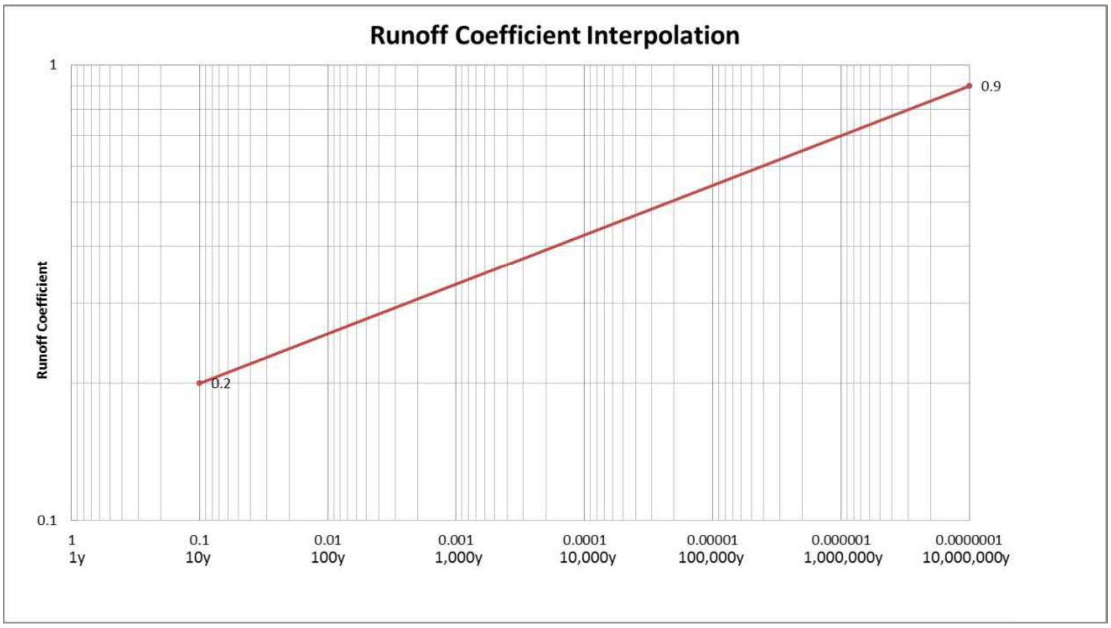

Figure 6-1 Runoff coefficient interpolation

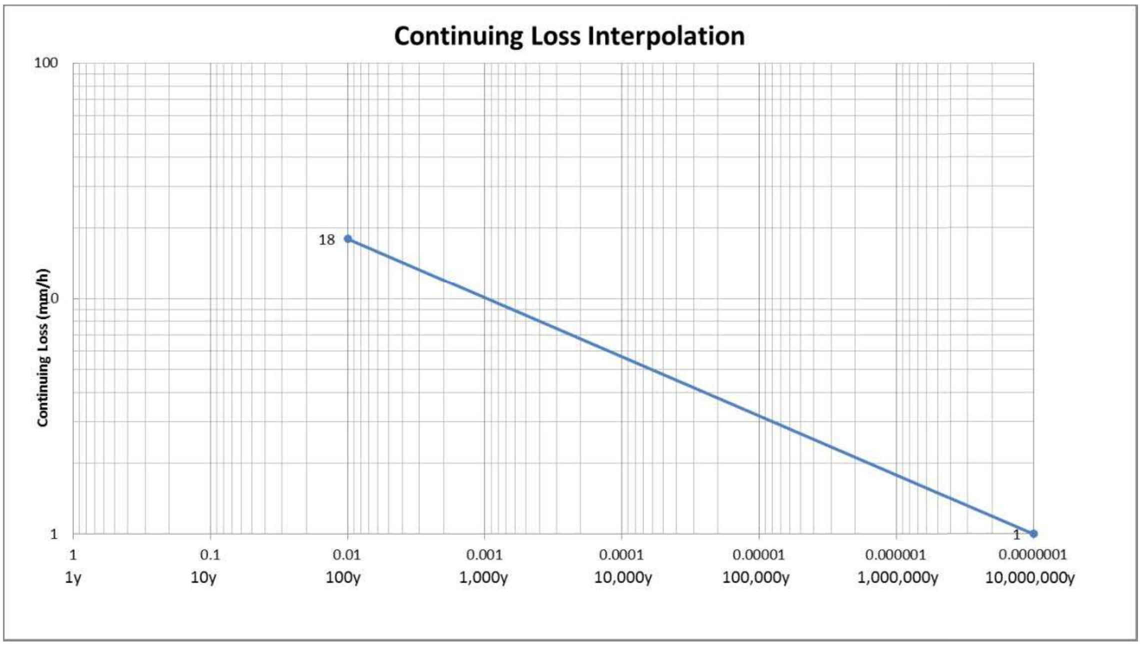

Figure 6-2 Continuing loss interpolation

The runoff coefficients and continuing loss parameters adopted for each design event are shown Table 6-2.

Table 6-2 Runoff coefficients and continuing losses

AEP	Runoff co-efficient	Continuing loss (mm/hr)
10%	0.2	NA
1%	0.26	NA
0.2%	NA	12.0
0.01%	NA	5.7
PMF	NA	1

6.2 Design rainfall

Design rainfall events were derived in accordance with the procedures in the Australian Rainfall and Runoff, Region 2 (ARR), 2001. The Intensity Frequency Duration (IFD) parameters adopted for the Sullivans Creek catchment were generated by input of co-ordinates on the Bureau of Meteorology website, and are generally in accordance with those listed in the TAMS design guidelines as shown below in Table 6-3.

Table 6-3 Sullivans Creek IFD Parameters

Parameter	Adopted value
2yr 1hr (ARI, duration)	21.90
2yr 12hr (ARI, duration)	4.24
2yr 72 hr (ARI, duration)	1.14
50yr 1hr (ARI, duration)	43.02
50yr 12hr (ARI, duration)	7.98
50yr 72 hr (ARI, duration)	2.25
Skew	0.24
F2 Value	4.28
F50 Value	15.57
Zone	2

6.3 Areal Reduction Factors (ARF)

Point rainfall magnitudes were estimated using IFD rainfall from the Bureau of Meteorology. The areal rainfall for the design flood event was derived using areal reduction factors (ARF). Areal reduction factors are needed to allow for the fact that larger catchments are less likely to experience a given storm intensity simultaneously over the whole catchment than smaller catchments.

The interim short duration equations for NSW and the ACT were adopted for the 10%, 1% and 0.2% AEP events, following the current recommendations contained in ARR Revision Project 2 Spatial Patterns of Rainfall (April 2013). Largely due to data availability the ARF analysis in Australia has been based on daily rainfall data with the result that the “interim” ARF equation for short durations (less than 18 hours) are still substantially based on overseas analysis and data.

To estimate the flow at a given location, an ARF is derived for the specific design event (duration and AEP) and the specific upstream area. The point rainfall intensities on all subareas are factored by the ARF and the flow reaching this point is estimated. The correct application of such a procedure over a large area of interest is a significant challenge. The rigorous approach would involve undertaking a large numbers of hydraulic simulations with various ARFs. Although possible, such an approach is resource intensive and ultimately only a step wise approximation of an approximate but continuous relationship. The approach taken on this project was to plot catchment area against critical duration based on a simulation using point rainfall. For each critical design event (duration and AEP) the spread of contributing areas was assessed and an ARF derived for the mean area. The results were then rerun in the RORB model with ARF for each design event (duration and AEP) derived using the characteristic area (based on the mean contributing area of those locations where that event produced the critical results).

The adopted approach is considered an appropriate approximation given the inherent uncertainty and implicit assumptions in the ARFs. It should be noted that the adopted approach is expected to have the following bias and limitations:

- It is likely that some locations will experience under or overestimates relative to a site specific estimate
- While the derivation of ARFs is purely an assessment of meteorological processes, this analysis will be biased by timing and routing effects within the model
- Areas with significant storage are likely to have longer critical durations and be assigned larger characteristic areas than measured
- During the initial iteration the ARFs for the longer durations are based on larger areas and are more significant, whereas there is a general trend that the critical duration reduces for many areas.

The large initial loss on the DUCR area created two distinct patterns with regards to critical durations, meaning the longer durations were critical for smaller areas in the DUCR area than elsewhere. To limit the effect of this characteristic, the DUCR areas were calculated separately. In practice this meant that a second set of RORB runs were undertaken with different ARF's applied. One set was appropriate for the DUCR area, the other set for the downstream area.

6.4 Probable Maximum Flood (PMF)

The Generalised Short Duration Method (GSDM) was applied for the PMF, with a 5-hour limit. Spatial distribution was applied in accordance with Bulletin 53. The average rainfall depths are given in Table 6-4.

Table 6-4 Average PMP rainfall depths

Event duration	Average total rainfall depth (mm)
15 minute	120
30 minute	170
45 minute	220
60 minute	260
90 minute	340
2 hour	390
2.5 hour	440
3 hour	480
4 hour	540
5 hour	590

6.5 Temporal patterns

The ARR Zone 2 temporal patterns were adopted for the 10% and 1% AEP events, while the GSDM temporal patterns was adopted for events less frequent than 1% AEP.

7. Design flood behaviour

7.1 Overview

The following simulations as indicated in Table 7-1 were run in accordance with the brief and proposal.

It includes the events of AEP 10%, 1%, 0.2% and 0.01%, as well as the PMF. The 10% and 1% AEP events were each run for 20 durations ranging from 10 minutes to 72 hours. For events above the 1% AEP, durations were run up to 5 hours, being the limit for the GSDM.

Flood maps for a range of events have been produced from an envelope of durations and are presented in Appendix B.

Table 7-1 Hydraulic model simulation overview

Event	Standard durations	Basin breach modelling	Climate change 30%	High Roughness sensitivity	Low Roughness sensitivity	Blockage sensitivity	DS boundary condition - +1m	DS boundary condition – +0.5m
10% AEP	X 20	NA	X 4	X 4	X 4	X 4	X 1	X 1
1% AEP	X 20	NA	X 4	X 4	X 4	X 4	X 1	X 1
0.2% AEP	X 10 (assuming 5-hour limit for GSDM)	NA	NA	NA	NA	NA	NA	NA
0.01% AEP	X 10 (assuming 5-hour limit for GSDM)	X 4 (No breach Southwell, Breach Southwell, No breach Kenny, Breach Kenny)	NA	NA	NA	NA	NA	NA
Southwell Park Basin DCF	NA	X 2 (Breach and no breach for critical duration)	NA	NA	NA	NA	NA	NA
Kenny Basin AFC	NA	X 3	NA	NA	NA	NA	NA	NA
PMF	X 10 (assuming 5-hour limit for GSDM)	X 7 Southwell critical duration breach and no breach, Kenny critical duration breach and no breach, Kenny cascade breach, Kenny failure no southwell failure.	NA	NA	NA	NA	NA	NA

7.2 Summary of 10% AEP flooding

Modelling estimates that existing condition 10% AEP flood waters will generally be conveyed by the channels, culverts and bridges without affecting surrounding areas. Overtopping is predicted at Miller Street on the O'Connor Channel, opposite the Morisset Wetland on Flemington Road (to the north of the culvert crossing) and over Thurbon Road (Rigall Place) in the Lyneham sporting precinct. Overland flow is also observed around Haig Park (Masson Street and the northern end of Macleay and Watson Streets) and behind Duffy Street at the bend of the Dickson Channel near Madigan Street. A small amount of breakout is observed through ANU. Flooding in all these areas is estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual.

7.3 Summary of 1% AEP flooding

Modelling indicates that 1% AEP flood waters will largely be conveyed by the channels, culverts and bridges without affecting surrounding areas. A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Condamine Street. Overtopping of Flemington Road is predicted opposite the Morisset Wetland (to the north of the culvert crossing), at Heffernan and Darling Streets in Mitchell, east along Sandford Street and south along Flemington Road. A small area of road in Exhibition Park is also predicted to be flooded. On the Dickson Channel flooding is predicted through a number of properties on Duffy Street at the bend near Madigan Street, a small amount of flooding is predicted where Majura Avenue crosses the Dickson Channel, and to the north of the channel from the Dickson District Playing Fields. At Cowper Street flooding is also observed to breakout to the south of the channel, and also at Challis and De Burgh Streets.

Thurbon Road (Rigall Place) in the Lyneham sporting precinct is predicted to be flooded. Flooding is predicted around the trunk underground drainage in Lyneham, including Challis Street, Northbourne Avenue, Oliver Street and Goodwin Street. Flooding is also observed around the trunk drainage in David Street (Turner).

Overland flow is also observed around Haig Park (Stawell Street, Ormond Street, Masson Street, Hackett Gardens, Miller Street, Barry Drive and the northern end of Macleay and Watson Streets). Breakout is also observed at the upstream end of ANU.

Flooding in all these areas is estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual.

7.4 Summary of 0.2% AEP flooding

Modelling estimates that 0.2% AEP flood waters will not all be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. Nearly all major crossings observed are subject to flooding with the exception of Parkes Way at the outlet of Sullivans Creek.

There is significant flooding predicted at ANU downstream of Barry Drive. There is also significant flooding predicted in residential areas between Barry Drive and David Street in Turner. A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Stawell Street. For the Dickson channel, significant amount of breakaway flooding is predicted to occur at the upstream end of the channel until it re-joins the Dickson channel near Cowper Street. Flooding is also predicted to occur along the entire length of the Dickson channel, with several dwellings on the north and south sides of the channel being affected.

The Southwell park fuse-plug is predicted to overtop with significant flooding occurring within the sports precinct. Flooding at Randwick pond is predicted to occur and flooding continues south, through the Canberra Racecourse area and overtops the Barton Highway. Overtopping of Flemington Road is predicted opposite the Morisset Wetland (to the north of the culvert crossing). Flooding is also predicted to occur along the Mitchell channel.

Localised flooding from pipe networks is also generally predicted to occur throughout the catchment. An area of interest is near Stirling Avenue in Watson, where breakout flooding is predicted to occur through EPIC and along Federal Highway from Stirling Avenue to Barton Highway. Several residential dwellings are predicted to be flooding parallel to Federal Highway.

Flooding in these areas is generally estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual. Flooding along the floodplains of Sullivans Creek channel is estimated with a high hazard category, however very few residential areas lie within this area. There are some areas (generally on roads) where flooding has been categorised as medium to higher hydraulic hazard. This is mostly observed for flooding on roads in the suburbs of Turner and O'Connor. Breakout flooding south of Randwick pond, in areas of the Canberra Racecourse and just upstream of the Barton Highway have been categorised with medium to high hazard as well.

7.5 Summary of 0.01% AEP flooding

Modelling estimates that 0.01% AEP flood waters will not all be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. Nearly all major crossings observed are subject to flooding with the exception of Parkes Way at the outlet of Sullivans Creek.

There is significant flooding predicted at ANU with flood extents bounded by Daley Road to the west and Childers Street and Ellery Crescent to the east. There is also significant flooding predicted in residential areas between Barry Drive and David Street in Turner, with flood extents bounded by Frogartt Street to the west and Moore Street to the east. A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Stawell Street.

For the Dickson channel, significant amount of breakaway flooding is predicted to occur at the upstream end of the channel as well as flooding along the entire length of the Dickson channel. This flooding extends to Antill Road and Bonython Street to the north near the Dickson District Playing Fields.

The Southwell park fuse-plug is predicted to overtop with significant flooding occurring within the sports precinct. It is predicted that there is significant flooding south of Randwick pond, at Canberra Racecourse and further south which overtops the Barton Highway. Overtopping along a significant length of Flemington Road is predicted due to breakout flooding coming from the east and north. Flooding is also predicted to occur along the Mitchell channel. Within EPIC, flooding is predicted to occur which mainly comes from Old Well Station Road and Morisset Road, as well as breakaway from pipe networks near Stirling Road. The breakaway at Stirling road is also predicted to flood along Federal Highway as well as several residential dwellings parallel to the Federal Highway.

Many areas of flooding are generally estimated to fall into the high hydraulic hazard category as defined by the NSW Flood Development Manual. This includes most of the flooding along the main Sullivans Creek channel. However, breakaway flooding from the O'Connor and Dickson channels are estimated to fall into the low hazard category, with the exception of a few flooding roads with a high category.

7.6 Summary of PMF flooding

Modelling estimates that PMF flood waters will not all be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. All major crossings observed are subject to flooding including Parkes Way at the outlet of Sullivans Creek.

Flood extents span approximately 0.5 km to 1 km wide along the main Sullivans Creek channel.

There is significant breakaway flooding predicted from the O’Connor and Dickson Channels. There is also breakaway predicted from Stirling road, which causes flooding along Federal Highway and residential areas parallel to it.

Most areas of flooding are generally estimated to fall into the high hydraulic hazard category as defined by the NSW Flood Development Manual. This includes most of the flooding along the main Sullivans Creek channel. However, there are some areas of breakaway flooding from the O’Connor and Dickson channels which were estimated to fall into the low hazard category, with the exception of a few flooding roads with a high category.

7.7 Inundation of key bridge and culvert structures

A total of fifty four (54) key waterway crossings were observed which included road as well as pedestrian crossings. A plan showing the location of these crossings is included inAppendix C, C-01.

More detailed flooding characteristics of these crossings are tabulated in Appendix C. C-02.

Tables for the 10%, 1%, 0.2%, 0.01% AEP and PMF include:

- Peak upstream and downstream flood depth
- Peak upstream and downstream velocity
- Flood gauge level at Southwell Park and Barry Drive streamflow gauges when the crossing is flooded

A summary of which crossings are predicted to be inundated during the design rainfall event are summarised after the tables in in Appendix C, C-02.

8. Sensitivity analysis

A number of runs were undertaken to assess the sensitivity of various parameters, including climate change, Southwell Park fuse-plug failure assumptions, Manning's 'n' roughnesses and the downstream tailwater level, as outlined in following sections. Maps showing the results from the sensitivity simulations are included in Appendix D.

In general four critical durations were simulated for each sensitivity analysis. These were selected based on the histograms of total areas they were critical for, and are listed in Table 8-1 below.

Table 8-1 Durations simulated for sensitivity analysis

AEP	Durations simulated
10%	2h, 6h, 3h, 48h
1%	1h, 3h, 48h, 2h

8.1 Hydrology

8.1.1 Future climate impacts on rainfall

A 30% increase in rainfall intensity was simulated in line with the maximum value suggested by NSW DECC "Practical considerations of Climate Change". RORB IFD parameters were adjusted to reflect the 30% increase in rainfall intensity, and the resulting flows simulated in the TUFLOW model.

Increasing rainfall by 30% has resulted in increased flooding extents in the 10% and 1% AEP but with a greater impact in the smaller channels like the O'Connor, Dickson and Mitchell Channels.

For the 10% AEP increases ranged from around 0.1-0.4 m in Kenny and parts of the Dickson and Mitchell Channel, 0.5 m near the racecourse to 0.2-0.4 m in Sullivans Creek, 0.2 m in the Lyneham Wetland, 0.6 m at Southwell Park and up to 0.8 m in parts of the O'Connor Channel.

For the 1% AEP increases ranged from around 0.1-0.3 m in Kenny, 150-400 mm in Sullivans Creek, 0.5 m in the Lyneham Wetland, 0.7 m at Barry Drive, 0.75 m at Southwell Park and up to a metre in parts of the Mitchell and O'Connor Channels. Localised increases of 0.6 m to in excess of a metre were observed in areas in and around the Dickson Channel.

8.2 Hydraulics

8.2.1 Southwell Park fuse-plug spillway failure

Although investigation of fuse-plug spillway was not originally scoped the assumptions about the failure time of the fuse-plug spillway are shown to have a significant impact on the flood levels in the Southwell Park basin in larger events. Some preliminary results from early runs during model development are shown in Table 8-2 below.

Table 8-2 Fuse-plug failure time sensitivity analysis (early TUFLOW results)

AEP	Maximum flood level with 20 min fuse-plug failure (m AHD)	Maximum flood level with 60 min fuse-plug failure (m AHD)	Maximum flood level with 90 min fuse-plug failure (m AHD)	Maximum flood level with no fuse-plug failure (m AHD)
0.05%	572.25	572.41	572.50	572.93
0.01%	572.9	573.0	573.0	573.25
PMF (Entire Sullivans Creek catchment)	574.0	574.0	574.0	574.2

The speed at which the fuse-plug fails is most critical in events where the fuse-plug level is reached, but the main embankment is not overtopped, and potentially changes the AEP of the Dam Crest Flood (DCF). Further discussion on this issue can be found in Section 11.

8.2.2 Manning's roughness assumptions

Two alternative sets of Manning's 'n' roughness parameters were used as a sensitivity analysis, one at the lower end of the range indicated in the TAMS guidelines, and one at the upper end. The values applied are given in Table 8-3 below.

Table 8-3 Values for Manning's 'n' sensitivity analysis runs

Manning's 'n'	Adopted	Low	High
Roads	0.02	0.012	0.024
Concrete lined drains	0.015	0.011	0.018
Waterways and waterbodies	0.03	0.025	0.045
Parks and well maintained open space	0.04	0.03	0.05
Developed areas (residential, commercial, industrial)	0.1	0.05	0.2
Lightly vegetated areas	0.03	0.025	0.05

Increasing the Manning's 'n' roughness values does not significantly change the flood extents although it does result in increased flood levels in many areas. New minor flow paths are initiated in some areas, such as through Kenny and downstream of Barry Drive.

8.2.3 Bridge and culvert blockage

A blockage scenario was run with blockages in accordance with the ARR Project 11 - Blockage of Hydraulic Structures (2013) for the 1% AEP events. The locations and extent of blockages modelled are listed in Table 8-4.

Table 8-4 Blockage assumptions for 1% AEP sensitivity run

Location	Percent blocked
Southwell Park inlet culvert	50%
Majura Avenue	75%

Majura Avenue

Blockage at Majura Avenue changes flood behaviour in the area for both the 10% and 1% AEP, with flood depths increasing by up to 0.4 m.

In the 10% AEP a 75% blockage is predicted to cause flow to breakout around the western side of the bridge, flow along Majura Avenue for a short distance before flowing down the layby and back towards the channel opposite Officer Crescent. In the 1% AEP event a 75% blockage of the Majura Avenue bridge is predicted to engage new flow paths west along Majura Avenue and along Dutton Street, as well as overland through properties to re-join the Dickson Channel, or flow into the Dickson Wetland from the east.

Southwell Park inlets (trash rack)

For the both the 10% and 1% AEP the base case has flooding on the surface above the culverts. Blockage at the inlets to Southwell Park has not a resulted in significant change in the flooding extents for both 10% and 1% AEP within the area. However there is a slight increase in flood depth ranging from 0.01-0.1 m. The minor impact is likely because ponding at the Southwell Park basin is the control, rather than the culvert capacity.

8.2.4 Downstream boundary condition

Two sensitivity analysis scenarios were run for the tail water level at Lake Burley Griffin for the 1% and 10% AEP events. One scenario increased the base lake water level of 555.93 mAHD by 0.5 m (556.43 m AHD) and another scenario by 1 m (556.93 (m AHD). Since Sullivans Creek is largely controlled by a weir just downstream of Fellows Rd, the changes to the tail water level at the lake is expected to only impact the creek catchment up to the weir control. Only the 3 hour duration event was run as it was the critical event for that area.

As expected, flood levels were only affected from the lake up to the weir control downstream of Fellows Rd for both the 10% and 1% AEP.

9. Model validation

The TUFLOW model results were checked against a variety of information to verify the TUFLOW model's performance.

9.1 Gauge level flood frequency analysis

The FLIKE extreme value analysis package, recommended for flood frequency analysis in the ARR updates, was used to analyse the gauge level data for the Barry Drive and Southwell Park gauges To check the sensitivity of results, maximums from all years (including partial records) and only full years were analysed. Results of the flood frequency analysis (FFA) are given in Table 9-1 below and plotted in Figure 9-1 for Barry Drive, and Figure 9-2 for Southwell Park.

Table 9-1 Flood Frequency Analysis (gauge levels)

Gauge	ARI	FLIKE all years	FLIKE full years	RORB	TUFLOW
Southwell	10	569.53	569.55	569.4	569.13
Barry	10	562.89	562.90		562.63
Southwell	100	570.32	570.33	570.78	570.72
Barry	100	563.14	563.17		563.17

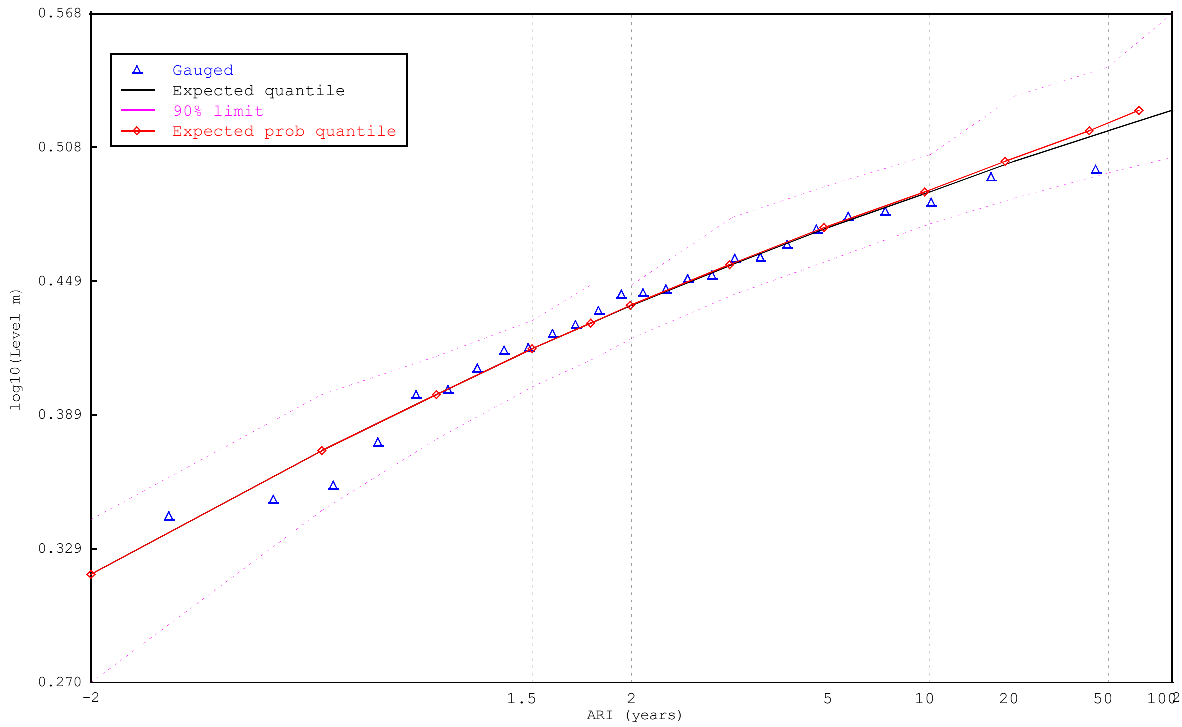

Figure 9-1 Barry Drive gauge level FFA (full years only)

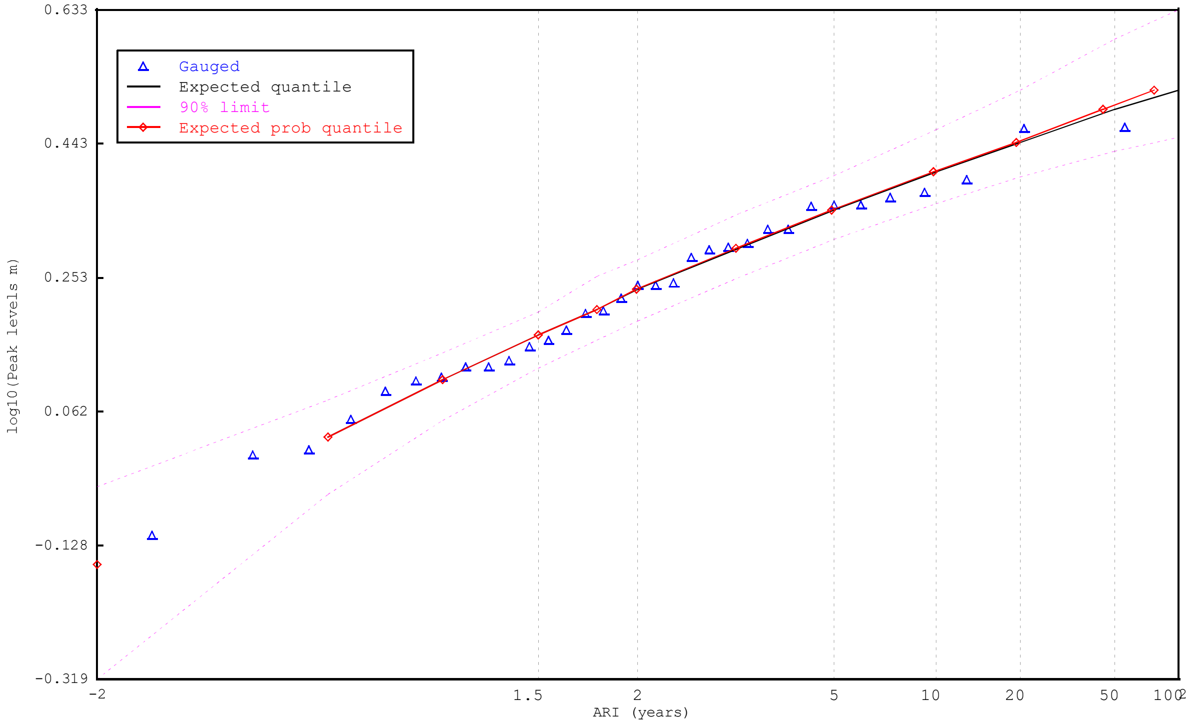

Figure 9-2 Southwell Park gauge level FFA (full years only)

The rating curves derived from the hydraulic model at Southwell Park and Barry Drive were compared against the established curves to check the TUFLOW model's performance at these locations.

9.2 Southwell Park and Barry Drive rating curves

A detailed comparison of the model results and gauging information is provided in0.

The modelled rating curves are in general considered to be an improvement on the extrapolated curves currently being used. It is suggested that future rating curves should include the recorded values at the lower levels and rely on the modelled results for the higher flows until better information becomes available. If more accuracy is desired a more detailed computation fluid dynamics (CFD) analysis could be undertaken to provide increased confidence in the relationships for a wide range of operational levels.

9.3 TUFLOW and HECRAS rating curve comparison

A detailed comparison of the TUFLOW and HECRAS hydraulic model results was undertaken and is presented in 0.

Comparisons have been made where possible and in general the results are reasonably consistent. There are some discrepancies although these are typically the result of discrepancies in the comparison locations and model capabilities.

10. Kenny planned development scenario

10.1 RORB modelling

The development for Kenny provided by CMTEDD was used to update impervious fractions to reflect the planned development. These values were included in a developed case RORB model and Areal Reduction Factors (ARF's) calculated using the methodology described in Section 6.3. The expected development and impervious fractions used for the proposed development can be found in Appendix F, Figure F-00a and F-00c for Kenny, and F-00b for Throsby.

The RORB model was then used for preliminary sizing of the Kenny Development Retarding Basin which was required to maintain 1% and 10% AEP existing peak flows. The location of the basin was chosen where the two tributaries within Kenny area would meet, with the embankment provided just downstream of the confluence to provide for the require storage volume.

Using the LDA supplied survey file (07286.01_DT_001_RevB 22/03/213) as a reference for levels, the downstream end of the basin outlets were tied into the existing level of 586.17 m AHD with a minimum grade of 0.5%. Basic earthworks were modelled in 12d to provide storage and remove the ridge between the eastern and western tributaries so that flow from the western tributary had a path to the assumed outlet location on the eastern branch. Approximately 7700 m^3 of cut was made within the basin compared to an estimated embankment fill volume in the order of 10,000 m^3.

Table 10-1 summarises the basin inflows, outflows and peak level from RORB. The peak flows within RORB with the development and basin in place were compared to the existing peak flows to check the impact of changes to runoff volumes and timing. The results at select locations are summarised in Table 10-2.

For consistency with the existing conditions approach, ARFs used for the developed case TUFLOW modelling were calculated using the methods described in Section 6.3. This results in a slightly different ARF to that calculated for the Kenny basin catchment area alone, and thus the basin outflow results differ in Table 10-1 and Table 10-2.

Table 10-1 Kenny Basin preliminary design inflows and outflows (RORB results basin catchment ARF)

Annual Exceedance Probability	Existing peak flow (m^3)	Developed peak inflow (m^3)	Developed peak outflow (m^3)	Developed peak basin stage (m AHD)
10% AEP	5.84 (6hr)	6.91 (6hr)	5.83 (6hr)	588.02
1% AEP	17.44 (6hr)	20.15 (3hr)	16.85 (6hr)	588.67
0.01% AEP	278 (1.5hr)	280.2 (1.5hr)	278.1 (1.5hr)	589.59

Table 10-2 RORB flow comparison of existing and future Kenny development including basin (ARFs as per methods described in Section 6.3)

Location	RORB reference	AEP	Existing	Developed
Outlet of proposed Kenny Basin	Peak flow KennyRB Inflow (existing)* Outflow Kenny Basin (developed)	10%	6.326	6.31
		1%	18.43	17.26
Confluence at the Kenny western and eastern tributaries	Peak flow Conf at KP1	10%	6.511	6.491
		1%	18.66	17.65
Outlet of Southwell basin	Peak flow Southwell RB Outlet	10%	24.86	24.88
		1%	40.01	37.61
Sullivans Creek upstream of the Dickson branch confluence	Peak flow Sull us of dickson Conf	10%	39.69	39.69
		1%	75.53	75.53
Barry Drive gauge point	Peak flow Barry Drive Gauge	10%	50.37	50.37
		1%	98.24	98.24
Sullivans Creek catchment outlet	Peak flow end of model	10%	53.75	53.75
		1%	102.3	102.3

* Estimated basin outflow for the existing case is from a combined flow from the two tributaries upstream of the proposed basin location.

10.2 TUFLOW modelling

The following modifications were made to the TUFLOW model for the purpose of assessing the proposed Kenny development:

- Inclusion of a main floodway through Kenny
- Inclusion of preliminary retarding basin design.

The events of AEP 10%, 1%, 0.2% and 0.01%, as well as the PMF were run for a range of durations. The 10% and 1% AEP events were each run for 20 durations ranging from 10 minutes to 72 hours. For events above the 1% AEP, durations were run up to 5 hours, being the limit for the GSDM.

10.3 Flood maps

Kenny flood maps from the TUFLOW modelling for the events of 10%, 1%, 0.2% and 0.01% AEP, as well as the PMF, have been produced from an envelope of durations and are presented in Appendix F. Maps F-01 to F-15.

11. Southwell Park breach modelling

11.1 Methodology

For each event to be assessed the critical duration was selected based on the maximum basin flood level from the RORB output.

Table 11-1 Critical duration based on maximum flood level in Southwell Park basin (RORB)

Event	Critical storm duration	Peak basin level (mAHD)	Peak inflow for critical duration event (m^3/s)
0.4% AEP (Dam Crest Flood)	90 minute	572.21	136
0.01% AEP	2 hour	572.77 - 572.92	494
PMF (Spatial distribution centred on the Southwell Park basin catchment)	90 minute	573.96 - 574.09	2048

The lower range peak basin level represents the RORB model adopting a completely failed fuseplug above the trigger level, the higher with no fuseplug failure.

These events were then simulated in TUFLOW for the no breach scenario. The TUFLOW model included the failure of the fuse-plug spillway.

The embankment breach times and sizes were estimated in line with ANCOLD Bulletin 97 with adaptation for the detention basin design and configuration of spillways and outlets. The scenarios in Table 11-2 were simulated.

Table 11-2 Flood scenarios to be assessed for Southwell Park Consequence Category Assessment

Scenario number	Flood	No Basin Failure	With failure of Southwell Park Basin only
S1	Southwell Park Dam Crest Flood (DCF)	X	
S2	Southwell Park Dam Crest Flood (DCF)		X
S3	0.01% AEP	X	
S4	0.01% AEP		X
S5	PMF (Southwell Park critical duration and spatial distribution)	X	
S6	PMF (Southwell Park critical duration and spatial distribution)		X

Due to the complexity of the fuse-plug, varying crest heights and breach locations the adopted parameters were then simulated in TUFLOW directly using “variable z shapes” to change the elevation of the embankment over time, rather than using a programme such as FLDWAV to calculate a hydrograph for input into the TUFLOW model.

11.2 Failure modes and locations

Without any geotechnical studies or work-as-executed drawings being available for the main Southwell Park embankment to guide where the embankment is most likely to fail, a number of possible failure locations were considered:

- To the south of the archery field from where the crest becomes well defined to the western end of the fuse-plug (approximately 120 m)
- From the east of the access-way to where the crest becomes less well defined (approximately 60 m)
- At the low point in the crest where the bike path crosses to the east of the outlet.

The failure of the fuse-plug spillway at Southwell Park is included in both the embankment breach and no embankment breach scenarios. Failure modes for the embankment are indicated in Table 11-3.

Table 11-3 Adopted embankment breach failure modes and assumed fuse-plug failure times

Event	Embankment failure mode	Fuse-plug failure time (main section)
DCF	Piping	2 hours (no overtopping)
0.01% AEP	Overtopping	60 minutes
PMF	Overtopping	20 minutes

11.3 PMF hydrology for Southwell Park

Spatial distribution of PMP rainfall was undertaken centred on the Southwell Park catchment. An initial loss of 0 mm and continuing loss of 1 mm/hr was applied.

The 1.5 hour storm was shown to be critical for basin inflow, stage and outflow, as shown in Table 11-4.

Table 11-4 Southwell Park PMF summary (hydrology)

Storm duration	Average total rainfall depth (mm)	Peak inflow at Southwell Park (m^3/s)	Peak outflow at Southwell Park (assuming instantaneous fuse-plug failure at 572 m AHD) (m^3/s)	Peak outflow at Southwell Park (assuming no fuse-plug failure) (m^3/s)	Peak basin level at Southwell Park (instantaneous failure at 572 m AHD)	Peak basin level at Southwell Park (no FP failure)
15 minute	120	836	753	738	572.99	573.12
30 minute	170	1237	1176	1168	573.33	573.47
45 minute	220	1609	1516	1505	573.61	573.74
60 minute	260	1825	1713	1703	573.77	573.9
90 minute	**340**	**2048**	**1943**	**1936**	**573.96**	**574.09**
2 hour	390	1958	1881	1876	573.9	574.04
2.5 hour	440	1865	1812	1807	573.85	573.99
3 hour	480	1771	1719	1714	573.77	573.91
4 hour	540	1563	1531	1528	573.62	573.76
5 hour	590	1405	1380	1379	573.5	573.64

11.4 Dam Crest Flood (DCF) estimation

Excluding areas designed as spillways such as the starter chute or fuse-plug the low-point on the embankment crest is at the point the bike path crosses to the east of the outlet culverts. The crest level at this point was recorded by DGPS as being 572.25 m AHD.

A number of AEPs between the 1% and 0.2% were simulated in RORB to estimate the AEP of the Dam Crest Flood. Rainfall depths for these were determined in accordance the procedures for interpolating between the 1% AEP and the PMP as outlined in with Book 6 of ARR. GSDM temporal patterns were adopted for events of less than 1% AEP and ARFs were applied (refer Section 6.3).

The critical basin water level was plotted on a log-normal scale for each AEP as indicated in Figure 11-1. Two different sets of assumptions were made in the stage discharge curve adopted in the RORB model to assess the sensitivity of the DCF to the operation of the fuse-plug spillway. One assumed no fuse-plug failure, and the second instantaneous failure of the entire fuse-plug when the water level in the basin reaches 572 m AHD (at the level of the initiating section sill rather than 200 mm above it as stated in the design report). At maximum water levels close to the design trigger level of 572.2 m AHD the intended failure of the fuse-plug is most closely represented by the no failure assumption. This is because the intention is for the main section to fail laterally over the course of two hours when there is not significant overtopping of the main fuse-plug section to accelerate this, and the peak water levels is reached shortly after the trigger level.

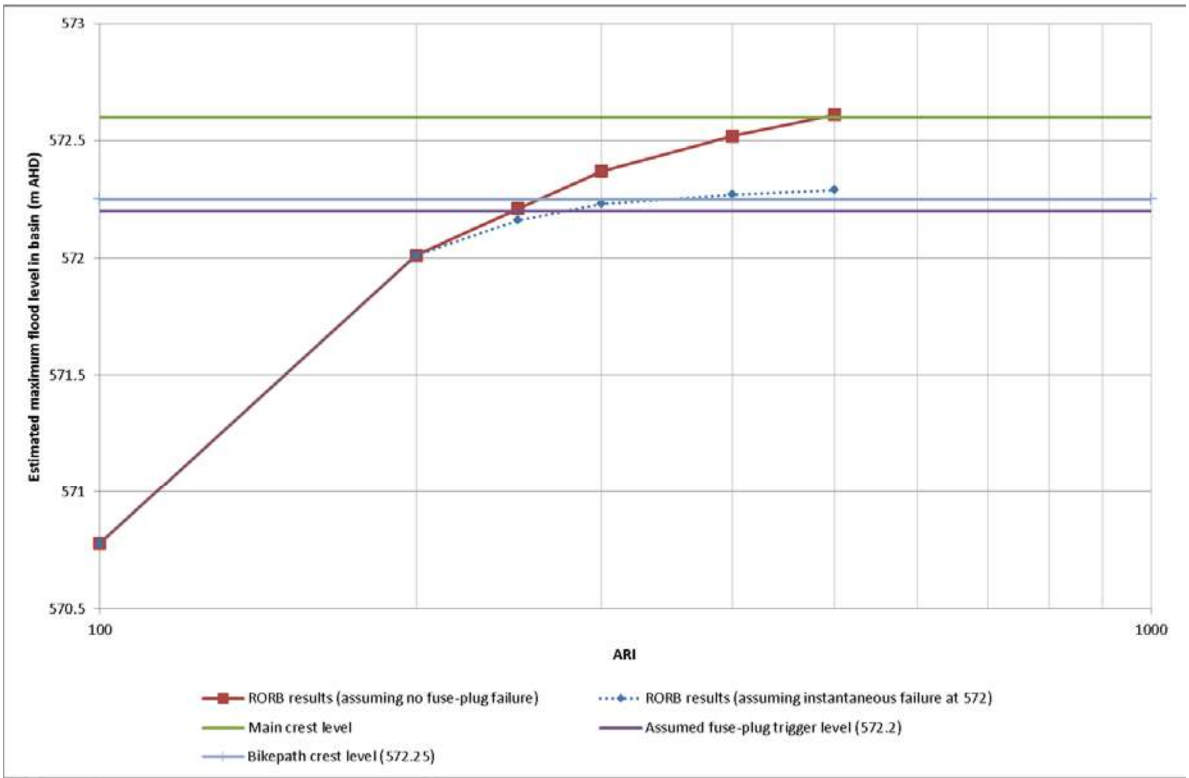

Figure 11-1 Southwell Park basin stage vs ARI for DCF estimation

As shown on the plot, using the no fuse-plug failure assumption the DCF is estimated to be the 250-year ARI (0.4% AEP). Using the less valid assumption of instantaneous fuse-plug failure at 572 m AHD would only increase this to something less than the 350-year ARI. A DCF of 0.4% AEP was adopted, with a peak basin level of 572.21 from RORB. The corresponding critical duration is 2 hours with a peak inflow of136 m^3/s.

The critical factor in estimating the DCF is the lowered crest level where the bike-path crosses east of the outlet culverts, rather than the operation of the fuse-plug. If the crest level was higher relative to the fuse-plug, then fuse-plug failure assumptions would have a greater bearing on the result.

11.5 PMF Breach formation parameters

The duration and degree of overtopping that an embankment might withstand before failing is unpredictable. ANCOLD Bulletin 97 quotes a number of criteria such as 900 mm for over an hour or 600 mm for up to one day but also suggests that for overtopping failures it is generally conservatively assumed that the embankment begins to fail with any overtopping.

It was assumed that the overtopping breach of the main crest would initiate at a level of 572.7 m AHD (100 mm above the crest). Based on the volume and head of water at this time, breach times and sizes were predicted for the following methods:

- MacDonald Langridge Monopolis method (in line with the recommendations of ANCOLD Bulletin 97)
- Froehlich (for comparison);
- Bureau of Reclamation (for comparison)
- Von Thun Gillette (for comparison).

During the time it takes an overtopping breach to form, the flood level in the basin continues to rise, such that the volume and head at the time of failure would actually be larger. The breach parameters were estimated by iterating on volume and head until the water level at the breach time matched that specified for the breach formation. The range of predicted breach parameters are summarised in Table 11-5.

Table 11-5 Predicted PMF breach parameter ranges

Breach Parameter	Minimum	Maximum
Breach base width (m)	6	340
Breach development time (min)	5	93
Peak breach flow (m3/s)	118	2187

The parameters (breach size and formation time) obtained are presented in Appendix G, Table G-1 and Figure G-15-1.

The breach size calculated using Macdonald Langridge Monopolis's method is larger than the length of embankment identified in possible failure locations, as is that given by Joy's equation quoted in ANCOLD Bulletin 97 (breach top width of 149 m but only based on the failure of five dams). The total length of embankment identified at each of the locations in Section 11.2 was therefore assumed to fail.

These large breaches do not satisfy the Singh and Scarlatos geometry checks specified in ANCOLD Bulletin 97 (ratios of breach top width to breach base width, and breach top width to breach height based on analysis of 52 historical dam failures). ANCOLD recommends breaches should meet the geometry checks except in special circumstances.

The maximum breach base size which would fall within their range was calculated as being 21 m, with a top width of 25 m for the full height of the embankment, or a 13 m base and 15 m top width at the location where the bike-path crosses. The larger breaches were simulated to provide a possible upper bound, which could occur if multiple breaches formed sufficiently close together to collapse into one another. , As constructed drawings were not available to discount atypical breach geometry. The breaches simulated in the hydraulic model are summarised in Table 11-6.

Table 11-6 PMF Breach parameters simulated

Simulated breach base width	Basis	Simulated breach time	Basis	Basin flood stage at time of failure
13 m	Singh and Scarlatos acceptable geometry at bike path location (bottom of breach 571.2 m AHD)	12 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	573.12 m AHD
21 m	Singh and Scarlatos acceptable geometry	17 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	573.59 m AHD
57 m	Access-way and where crest is no longer well defined to east of gauging pond. Macdonald Langridge Monopolis and Joy predict breaches larger than this. Bottom of breach 571 m AHD. Ds slope 1/10	21 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM	573.82 m AHD
116 m	Distance between where crest is well defined to 570.3 m AHD and western end of fuse-plug. Macdonald Langridge Monopolis and Joy predict breaches larger than this.	29 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM	574.10 m AHD

Note: The effective modelled breach sizes are multiples of the TUFLOW cell size (5 m). Given the scatter and magnitude of breach size predictions this rounding effect is not likely to have a significant impact on the Consequence Category Assessment.ANCOLD bulletin 97 also suggests modifying the breach parameters as appropriate to err on the conservative side. These recommendations include increasing the size of the breach if the discharge continues at high levels long after being fully formed, and decreasing the breach size or increasing the breach development time to reflect the reduced erosive capacity if the flow is dominated by tail-water.

11.6 0.01% AEP breach formation parameters

It was assumed that an overtopping breach of the main crest would initiate at a level of 572.7 m AHD (100 mm above the crest), or at 572.4 m AHD at the bike path location. Based on the volume and head of water at this time breach times and sizes were predicted for the MacDonald Langridge Monopolis method in line with the recommendations of ANCOLD Bulletin 97. Froehlich, the Bureau of Reclamation and Von Thunn Gillette parameters were also calculated for comparison. The various breach formation times and parameters are provided in Appendix G,Table G-2 and Figure G-15-2.

During the time it takes the overtopping breach to form the flood level in the basin continues to rise, such that the volume and head at the time of failure would actually be larger. By iterating on volume and head until the water level at the breach time matched that specified for the breach formation, the sets of breach parameters estimated were obtained and are provided in Table 11-7 and Table 11-8.

The operation of the fuse-plug spillway means that less water is expected to flow through the breach than if there was no fuse-plug failure. Based on the relative sizes of the breach predicted by Macdonald Langridge-Monopolis (after ANCOLD and DERM) and the size of the fuse-plug breach the volume of water through the breach was reduced, leading to a reduced breach size.

Table 11-7 Predicted 0.01% AEP breach parameter ranges

Breach Parameter	Minimum	Maximum
Breach base width (m)	5	181
Breach development time (min)	5	75
Peak breach flow (m3/s)	84	1433

The larger breaches do not satisfy the Singh and Scarlatos geometry checks specified in ANCOLD Bulletin 97 (ratios of breach top width to breach base width, and breach top width to breach height based on analysis of 52 historical dam failures). ANCOLD recommends they should except in special circumstances. The maximum breach base size which would fall within their range was calculated as being 21 m, with a top width of 25 m for the full height of the embankment, or a 13 m base and 15 m top width at the location the bike-path crosses. The larger breaches were simulated to provide a possible upper bound, which could occur if multiple breaches formed sufficiently close together to collapse into one, especially in the absence of as-constructed drawings being available to suggest that an a typical breach was unlikely to form. The breaches simulated in the hydraulic model are summarised in Table 11-8.

Table 11-8 0.01% AEP Breach parameters simulated

Simulated breach base width	Basis	Simulated breach development time	Basis	Basin flood stage at time of failure (m AHD)
13 m	Singh and Scarlatos acceptable geometry at bike path location (bottom of breach 571.2 m AHD)	12 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	572.66
21 m	Singh and Scarlatos acceptable geometry	17 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	572.96
57 m	Access-way and where crest is no longer well defined to east of gauging pond. Macdonald Langridge Monopolis and Joy predict breaches larger than this. Bottom of breach 571 m AHD	21 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM	572.98
61 m	Macdonald Langridge Monopolis within distance between where crest is well defined to 570.3 m AHD and western end of fuse-plug. Accounting for flow through fuse-plug	24 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM	572.99

Note: The effective modelled breach sizes are multiples of the TUFLOW cell size (5 m). Given the scatter and magnitude of breach size predictions this rounding effect is not likely to have a significant impact on the Consequence Category Assessment.

11.7 DCF breach formation parameters

It was assumed a breach would initiate when flood levels reached 150 mm below the bike path crest of 572.25 (572.1 m AHD), ie before fuse-plug failure would occur. For the critical duration storm event of 90 minutes this level is reached at 2.15 hours, which is after the rainfall has stopped. As with the other events the breach formation parameters were calculated according to Macdonald Langridge-Monopolis, Bureau of Reclamation, Von Thun Gillette and Froehlich with the range of predictions shown in Table 11-9. The result for each of the methods is in Appendix G, Table G-3 for the 0.4% AEP breach used.

Table 11-9 Predicted DCF breach parameters ranges (piping)

Breach Parameter	Minimum	Maximum
Breach base width (m)	4	85
Breach development time (min)	4	60
Peak breach flow (m^3/s)	57	919

The Macdonald Langridge Monopolis parameters were adapted for the breach location to the south of the outlet where the embankment has a milder slope and is not as high. The breaches simulated are outlined in Table 11-10.

Table 11-10 DCF breach parameters simulated (piping)

Simulated breach base width	Basis	Simulated breach time (Macdonald Langridge-Monopolis after ANCOLD/ DERM)
13 m	Singh and Scarlatos acceptable geometry at bike path location (bottom of breach 571.2 m AHD)	12 min
21 m	Singh and Scarlatos acceptable geometry	17 min
49 m	Macdonald Langridge-Monopolis after ANCOLD/ DERM	22 min
40 m	Access-way and where crest is no longer well defined to east of gauging pond using Macdonald Langridge Monopolis after ANCOLD/ DERM Bottom of breach 571 m AHD	18 min

Note: The effective modelled breach sizes are multiples of the TUFLOW cell size (5 m). Given the scatter and magnitude of breach size predictions this rounding effect is not likely to have a significant impact on the Consequence Category Assessment.

11.8 Representation of buildings

Initial model runs were carried out using the 5 m grid base model. The area affected by breaches (greater than 300 mm incremental depth) was then delineated and buffered by 75 m. Building footprints in this area were manually digitised using the supplied aerial photography, and the maximum ground level of each extracted from the DEM. The floor level was then set to 300 mm above this. The Manning's 'n' of the building footprint was increased to 1, and the surrounding block to 0.05. Buildings that were represented in this manner have the building footprint highlighted in the flood maps in Appendix G.

11.9 Results

The model results were post-processed to create maps of depth, flood level contours, velocity, velocity depth products and areas of greater than 300 mm incremental depth ('affected zones'). Hydrographs were also generated at selected locations. These are included in Appendix G for various scenarios. The results of the breach modelling are discussed under the corresponding event heading in the following sections.

11.9.1 250-year ARI (90 min storm duration)

No breach (fuse-plug failure only)

For the no breach scenario flooding from the starter chute and fuse-plug is observed along Mouat Street to the east of the intersection of Brigalow Street. Some flooding occurs around the north-eastern and eastern side of the Lyneham Motor Inn, and the eastern edge of Brindabella Christian College. Flooding occurs to the east of the channel, including properties on Mouat Street and Goodwin Street. Based on the assumption of floor levels being 300 mm above the maximum ground level of the footprint no floors are predicted to be flooded.

Breach scenarios (no fuse-plug failure)

In the 21 m base width 17 minute breach and 49 m 22 minute breach scenarios flooding extends along Mouat Street from approximately number 187 and south of the intersection with Archibald Street respectively, down Brigalow Street to Lyneham Primary School, up Boyd Street and along Lewin Street from Boyd Street to Glover Street. A large number of properties are flooded. Within the affected zone buildings at the Lyneham Motor Inn, St Ninians Church and Brindabella Christian College are predicted to have wet floors.

In the 40 m 18 minute breach flood levels to the east of the channel on Mouat Street and Goodwin Street are predicted to rise. Between five and ten residential properties within the affected zone are predicted to have wet floors

A comparison of no breach and breach hydrographs at the basin crest in Figure 11-2 shows that peaks occur significantly earlier in the breach scenarios than when fuse-plug failure occurs in the no breach case.

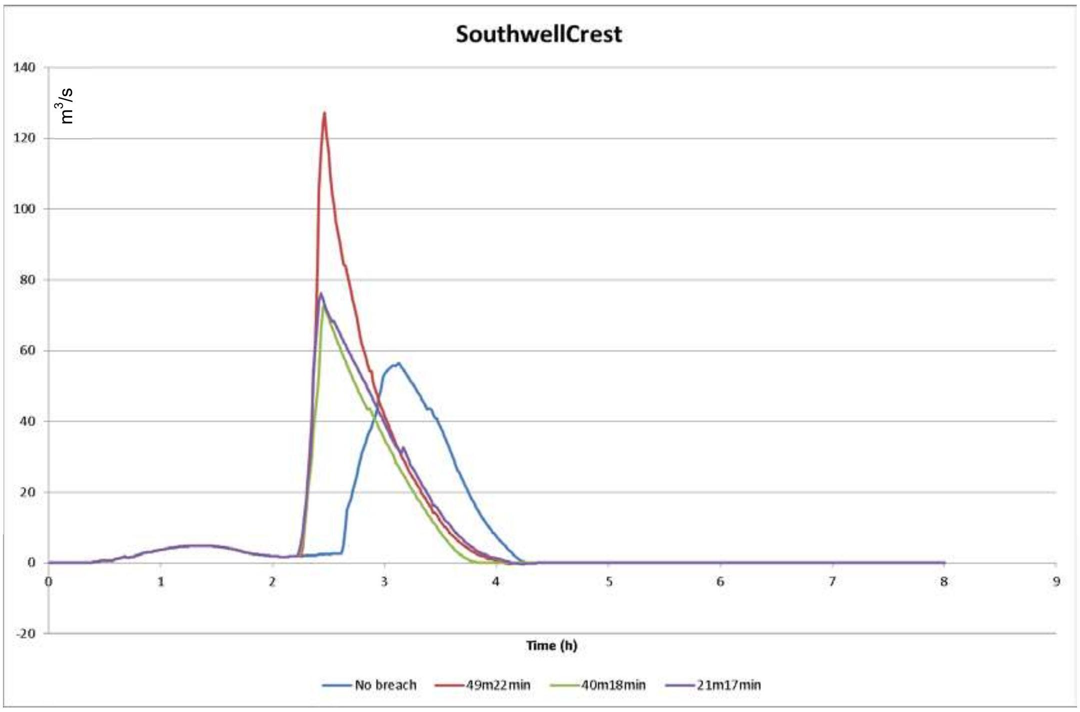

Figure 11-2 Southwell Park DCF overland outflow hydrographs

The peak flows at the Southwell Park basin crest (including the starter chute and fuse-plug locations) are summarised in Table 11-11 below.

Table 11-11 Dam crest flood peak overland outflows (excludes culverts)

Scenario	Peak flow at Southwell Park crest (including spillway and fuse-plug) (m^3/s)	Flow through breach area (m^3/s)	Comment
No breach (fuse-plug failure only)	56	NA	
49 m breach	127	127	
40 m breach	72	71	
21 m breach	76	75	

The extent of incremental depth of over 300 mm was calculated, and is shown on the maps in Appendix Gand summarised in Table 11-12.

Table 11-12 Summary of main affected areas by DCF breach scenario

Breach scenario	Size of affected zone (m^2)	Comment
49 m breach to west of fuse-plug	76,035	Includes Lyneham Motor Inn, St Ninian's Uniting Church, Brindabella Christian College, Lyneham Primary School.
21 m breach to west of fuse-plug	60,098	Includes Lyneham Motor Inn, St Ninian's Uniting Church, Brindabella Christian College, Lyneham Primary School.
40 m breach to south of outlet culverts	10,777	Includes Mouat Street and properties between the eastern side of the channel and Goodwin Street, and to the north of the park access.

11.9.2 10,000-year (2-hour)

No breach (fuse-plug failure only, embankment overtopped)

Significant flooding occurs along Mouat Street, Brigalow Street, Boyd Street, Lewin Street, Goodwin Street and Oliver Street. A number of floors within the affected zones are flooded, assuming these are 300 mm above the highest ground level on the building footprint. These include residential properties on Mouat Street, Goodwin Street, Brigalow Street and Lewin Street, along with the Lyneham Motor Inn, St Ninian's Uniting Church and Brindabella Christian College.

Breach scenarios (including fuse-plug failure)

Flood levels within already flooded areas increase, along with the velocities. In places this results in a significant increase in flood hazard on roads based on the velocity depth product, and in additional flooded floors. The largest affected area results from the 116 m wide breach to the west of the fuse-plug, which is deemed to be too conservative and has not been analysed further. The 61 m wide breach in the same area has the second largest affected zone. The breach to the west of the fuse-plug according to Singh and Scarlatos geometry is smaller.

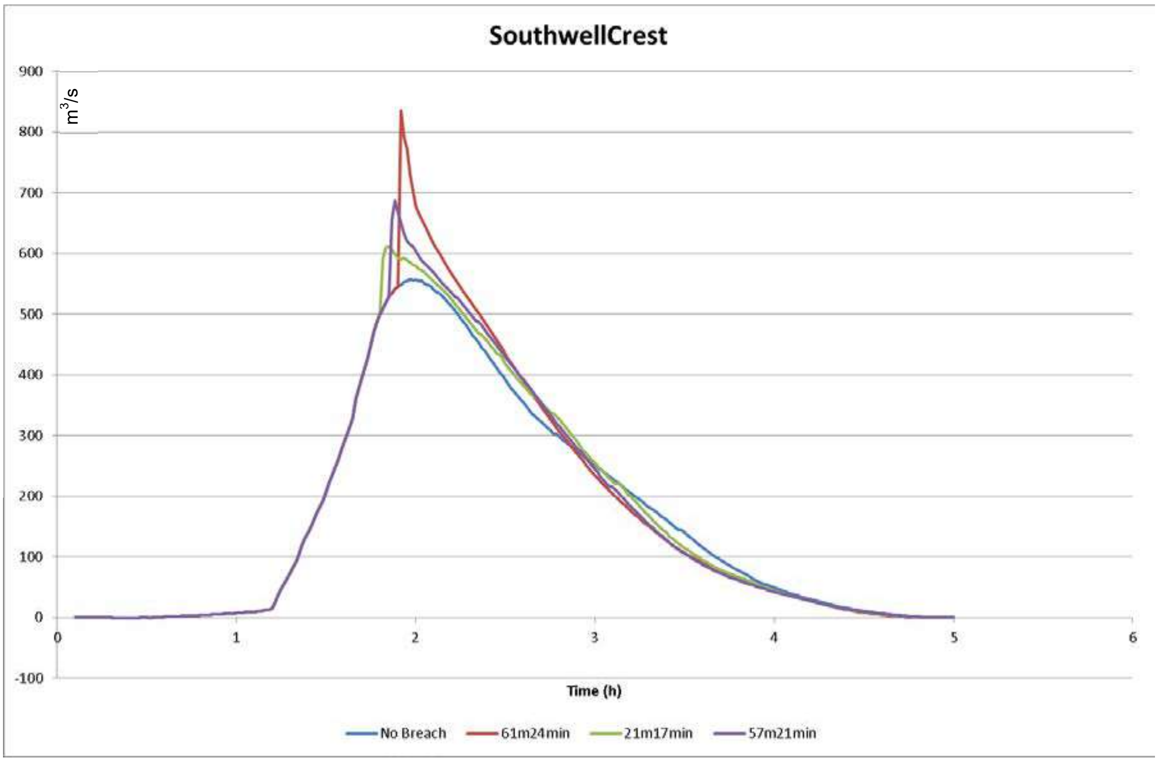

Figure 11-3 Southwell Park 0.01% AEP overland outflow hydrographs

Table 11-13 0.01% AEP flood peak overland outflows (excludes culverts)

Scenario	Peak flow at Southwell Park crest (including spillway and fuse-plug) (m^3/s)	Peak flow through breach area (m^3/s)	Comment
No breach (fuse-plug failure only)	558		
61 m breach (including fuse-plug failure)	835	372	Overall overland outflow 277 m^3/s higher than no breach peak
57 m breach (including fuse-plug failure)	612	234	Overall overland outflow 54 m^3/s higher than no breach peak
21 m breach (including fuse-plug failure)	687	145	Overall overland outflow 130 m^3/s higher than no breach peak

Table 11-14 Summary of main affected areas by 0.01% AEP flood breach scenario

Breach scenario	Size of affected zone (m^2)	Comment
17 min breach and fuse-plug failure	10,777	
21 min breach and fuse-plug failure	22,047	Includes Mouat Street and properties to the east of the channel including a number on Mouat Street, Goodwin Street and Oliver Street.
24 min breach and fuse-plug failure	82,156	

11.9.3 PMF (90 min)

There is widespread flooding below Southwell Park in the PMF. The area affected by a breach in the PMF (flood levels 300 mm or greater above the no breach maximum) is very small in all cases, and does not include buildings or roads. Hydrographs at the breach locations show that there is increased flow due to the breaches, but the overall outflow from the basin does not increase significantly, as shown in Figure 11-4 to Figure 11-6 and Table 11-15.

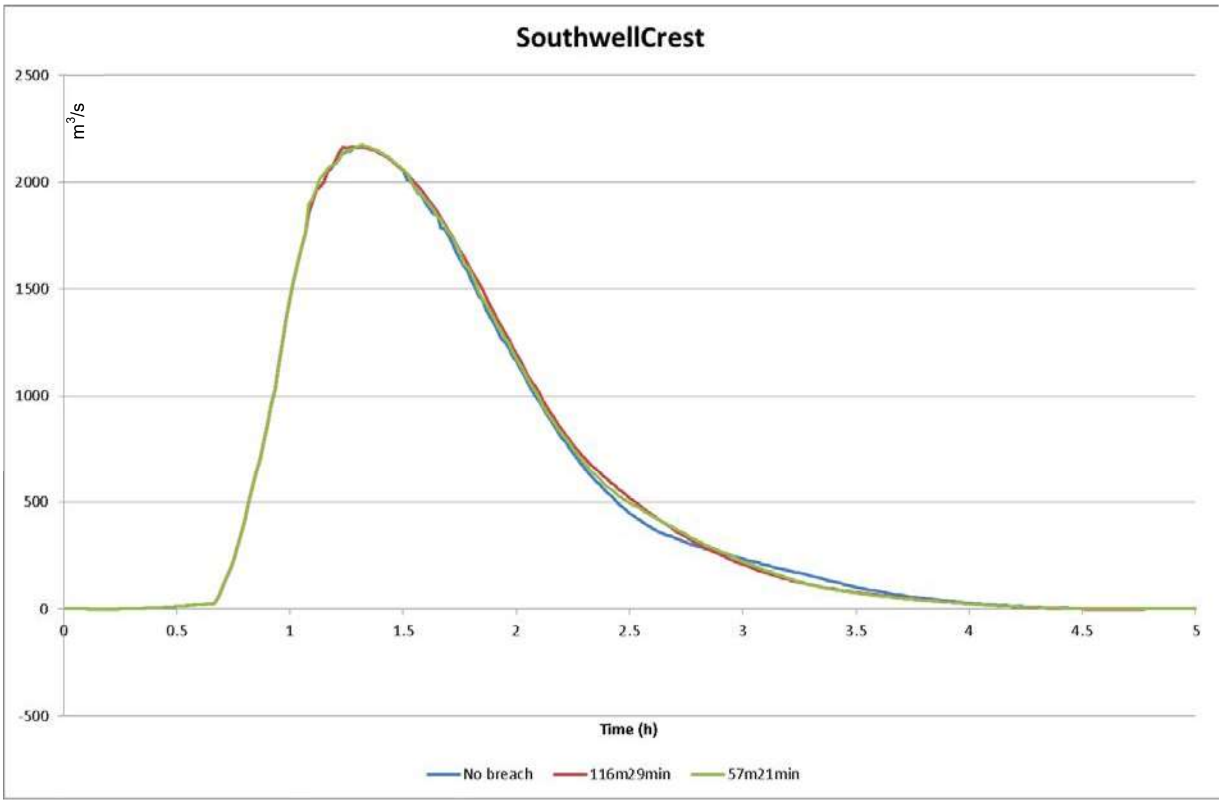

Figure 11-4 Southwell Park crest outflow for PMF

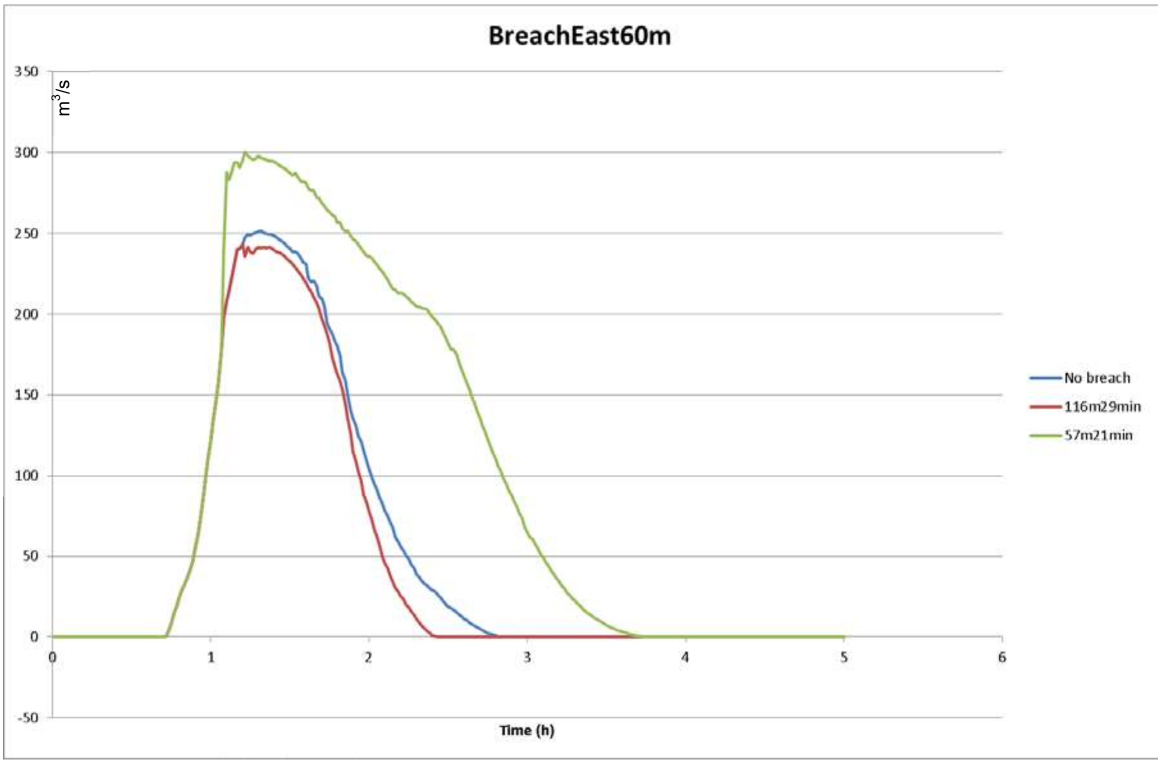

Figure 11-5 PMF outflow at eastern breach location (to south of outlet)

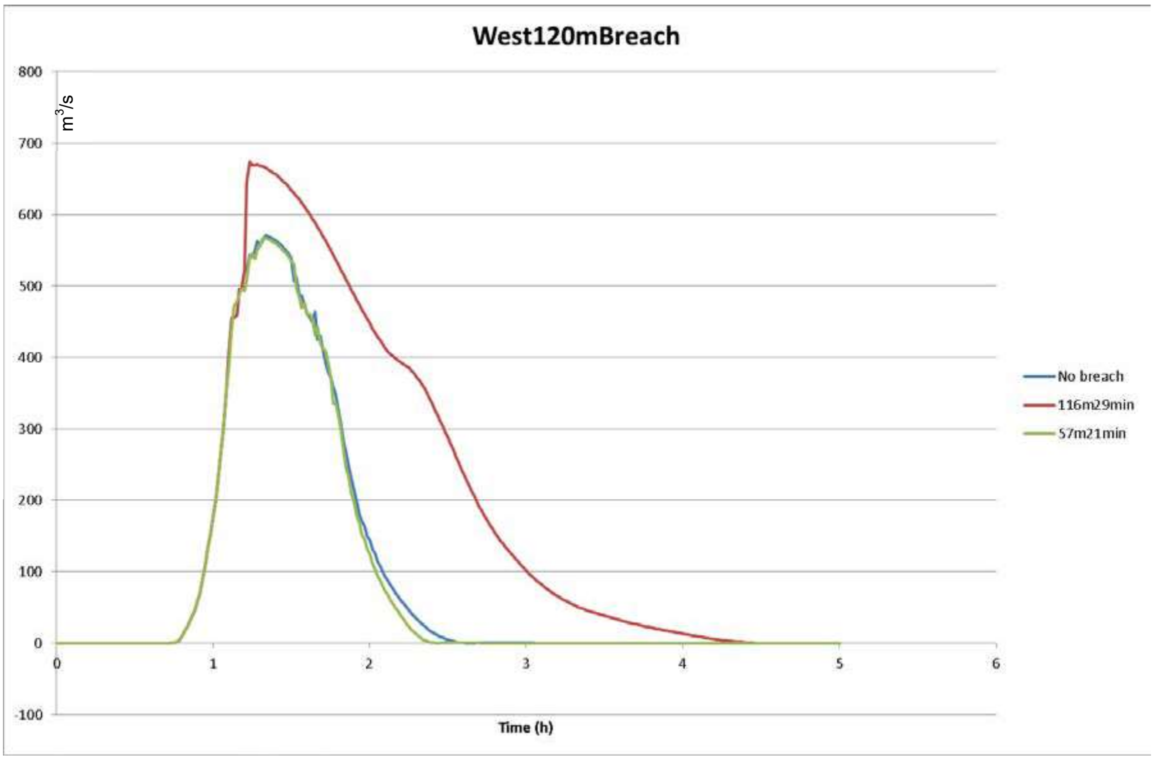

Figure 11-6 PMF outflow at breach location to west of the fuse-plug

Table 11-15 PMF flood peak overland outflows (excludes culverts).

Scenario	Peak flow at Southwell Park crest (including spillway and fuse-plug)	Peak flow through western breach area	Peak flow through eastern breach area	Comment
No breach (fuse-plug failure only)	2165	572	252	
116 m breach and fuse-plug failure	2165	674	243	
57 m breach and fuse-plug failure	2173	567	300	

Due to the fact that the affected zones are insignificant no further analysis of the PMF was undertaken.

12. Southwell Park basin consequence category assessment

12.1 Introduction

A comprehensive level assessment has been carried out to determine the Flood Consequence Category for Southwell Park Retarding Basin. The consequence category assessment was undertaken based on the ANCOLD Guidelines on Consequence Categories for Dams (November 2012) with the modifications outlined in DSC Guidance Sheet DSC3A - Consequence Categories for Dams (updated May 2014)

In accordance with the 2012 ANCOLD guidelines this assessment is required to be based on both the Dambreak Population at Risk (PAR) and the Incremental Potential Loss of Life (PLL) estimates in conjunction with the severity of the damage and loss resulting from the dam failure.

Southwell Park Basin is a “dry” detention basin. Therefore the Sunny Day consequence category of the basin is assessed to be “**Very Low**”.

12.2 Methodology

This assessment includes the evaluation of the consequences due to failure of the Southwell Park Basin embankment. In the event of failure of the basin embankment the flood wave will travel south through Lyneham. The development affected depends on the breach location and size, and may include areas of Mouat Street, Goodwin Street, Oliver Street, Brigalow Street, Boyd Street and Lewin St, including apartments, semi-detached residential dwellings, detached residential dwellings, the Lyneham Motor Inn, St Ninians Uniting Church, Brindabella Christian College, and Lyneham Primary School.

The 2012 ANCOLD Guidelines require that the evaluated flood cases include the dam crest flood (DCF), PMF and a flood event of lesser magnitude than DCF if estimation of incremental Potential Loss of Life (PLL) is likely to increase. In view of this the following flood and breach scenarios were assessed for the basin:

- DCF (0.4% AEP/ 1 in 250 ARI 90 minute flood event) with fuse-plug failure but no embankment failure and with embankment failure prior to fuse-plug failure (no fuse-plug failure)
- 0.01% AEP / 1 in 10,000 ARI (2-hour flood event) with fuse-plug failure with and without embankment failure
- PMF (90 min duration event) with fuse-plug failure, with and without embankment failure. As there is no significant affected area for the PMF no further analysis was undertaken for this case.

Flood inundation maps and associated results included inAppendix G were utilised in the assessment of PAR and PLL and the evaluation of severity of damage and losses due to the dambreak. The dambreak study included a series of sensitivity analysis on the size and location of each breach. For the purpose of this assessment the breaches formed to the west of the fuse-plug with the expected geometry (estimated using Singh and Scarlatos) were used to assess the most likely Population at Risk (PAR), along with larger breaches using Macdonald Langridge Monopolis parameters to indicate an upper bound (refer Section 11)..

The breaches of each flood scenario adopted in this assessment are as follows:

- 0.01% AEP with a 64 m breach to the west of the fuse-plug (breach at 1.92 hours);
- 0.01% AEP with a 21 m breach to the west of the fuse-plug (breach at 1.8 hours);
- 0.4% AEP (DCF) with a 49 m breach to the west of the fuse-plug (breach at 2.15 hours prior to fuse-plug failure and once rain would have stopped and people would likely be outdoors or moving again); and
- 0.4% AEP (DCF) with a 21 m breach to the west of the fuse-plug (breach at 2.15 hours, prior to fuse-plug failure and once rain would have stopped and people would likely be outdoors or moving again).

Based on DSC correspondence with dam owners (and GHD) dated 20th June 2013 [Ref 3], the PAR and PLL estimates were carried out within the potential inundation area at which the incremental depth due to dam break flooding exceeds 300 mm. For ease of reference this has been referred to as the affected zone. The affected zone varies in extent and location for each breach.

12.3 Population at Risk (PAR) estimate

The total population at risk (PAR) comprises the non-itinerant residents in the inundated buildings and the itinerant population in the form of vehicle and pedestrian traffic along the roads within the affected zone. For the purposes of this assessment it has been assumed that as per the current situation there are no alarms on Southwell Park or gates across Mouat Street to close the road when the fuse-plug activation level is reached.

12.3.1 Non-itinerant population at risk

The flood mapping was used to identify residences and buildings in the potential inundation zone. The depth of flooding at each building was estimated for each scenario based on the TUFLOW hydraulic model results (refer to Appendix G).

In lieu of surveyed floor levels, which although preferred were not available, a floor level of 300 mm above the maximum level on the building footprint on the supplied 3m DEM was assumed.

The number of residents/occupants in the buildings within the inundation area varies depending on the type of building and the use thereof. The information on the number of occupants was estimated based on the type of each property. Assumptions have been made on the distribution of the occupants in the properties with either permanent, temporary or infrequent residents (refer to Table 12-1 and Table 12-2).

Due to the low exposure times of students being outside, it has been assumed that all persons will be in the buildings when assessing the schools for PAR.

Table 12-1 Estimated number of non-itinerants in properties identified within affected zone (residential dwellings)

Type	Number of people	Basis
Number of residents per dwelling	2.1	2011 Census QuickStats quotes an average of 2.1 people per household in Lyneham (Released at 11:30 AM (AEST) 28/03/2013)
Estimated day time distribution	0.8	Census data suggests approximately 60% of adult population in workforce or studying. Assumed 40% of population may be home during the day.
Estimated night time distribution	2.1	Assumed maximum population numbers will occur during night time.

Table 12-2 Estimated number of Non-Itinerants in properties with temporary occupation within the affected zones

Location [(1)]	Description	Temporary / Infrequent (Visitors)		
		Weekends/ Holidays (daytime)	Weekday (daytime)	Night time (Weekdays and weekends)
39 Mouat Street	Lyneham Motor Inn[(2)]	12	12	90
150 Brigalow Street	St Ninian's Uniting Church	80	10	0
136 Brigalow Street	Brindabella Christian College[(3)]	0	650	0
68 Brigalow Street	Lyneham Primary	0	388	0

Notes:

1) The location of the buildings are shown in Appendix G
2) 60 room motel, assuming average 1.5 people per room
3) In 2011 census 156 residents of Lyneham stated they belonged to the Uniting Church. Assumed approximately half this number may attend any given service, including parishioners from neighbouring areas. According to the church's website the chapel holds approximately 130 people. On weekdays assumed 10 people may be on the grounds at any time for mothers group, clothing sales etc
4) Brindabella Christian College from MySchool website, School Facts 2013: 592 total enrolments, 42 teaching staff and 16 non-teaching staff
5) Lyneham Primary School from MySchool website, School Facts 2013: 356 total enrolments, 27 teaching staff and 5 non-teaching staff.

Table 12-3 Summary of non-itinerant Population at Risk by flood event and time category

Flood event/ scenario	Weekday (no breach)	Weekday (with breach)	Weekday (pre breach)	Night time (no breach)	Night time (with breach)	Night time (pre-breach)	Weekend (no breach)	Weekend (with breach)	Weekend (pre-breach)
0.01% AEP 21 m 17 min breach	18	24.8	0	60	87.6	0	88	94.8	0
0.01% AEP 61 m 24 min breach	256.0	578.6	14.0	60.0	121.2	30.0	88.0	107.6	84.0
0.4% AEP 21 m 17 min breach	0.0	12.0	0.0	0.0	15.0	0.0	0.0	82.0	0.0
0.4% AEP 21 m 17 min breach	0.0	151.0	0.0	0.0	75.0	0.0	0.0	90.0	0.0

12.3.2 Itinerant Population at Risk

Travellers on roads

Based on flood mapping, the major roads which are located within the inundation area were assessed for the estimation of itinerants at risk in each flood and breach scenario. The traffic data on these roads was provided by the client, as follows:

- Mouat Street : Traffic counts from Roads ACT (2013)
- Brigalow Street: Traffic counts from Roads ACT (2013).

Lewin Street and Boyd Street were assumed to predominantly service local traffic, i.e the properties serviced by Aston Street, Earle Street, Dyson Street, Longstaff Street and Boyd Streets off Archibald, McKennal, Wattle and Brigalow Streets (approximately 250 properties). The average number of vehicles per dwelling in Lyneham at the 2011 census was 1.4, assuming each makes two trips a day on Lewin Street and/or Boyd Street these streets will have approximately 700 vehicle movements a day.

Based on the Floodplain Development Manual, New South Wales (NSW Public Works 1986) and the Floodplain Development Manual (NSW Department of Infrastructure Planning and Natural Resources, 2005) an upper bound for both depth (0.3 m) and velocity (2.0 m/s) is considered for vehicles to remain stable. In this assessment a flood depth of 0.3 m or greater was considered as hazardous to itinerants travelling in a vehicle.

Table 12-4 includes the estimated number of vehicles at risks in the flood/breach scenarios for the Southwell Park Basin.

Table 12-4 Estimated Number of Vehicles at Risk in the Flood / Breach scenarios

Flood/Breach Scenario	Inundated Road (1)	Daily traffic volume (2)	Total Length of Inundation (m)	Estimated Travel Time (s) (3)	Estimated No. of Vehicles at Risk(4)
0.01% AEP (17 min breach)					
without Dambreak	Mouat Street	23067	190	17.1	4.57
	Brigalow Street	7400	140	12.6	1.08
	Boyd street	700	80	7.2	0.01
	Lewin Street	700	180	16.2	0.01
with Dambreak	Mouat Street	23067	190	17.1	4.57
	Brigalow Street	7400	140	12.6	1.08
	Boyd street	700	80	7.2	0.01
	Lewin Street	700	180	16.2	0.01
0.01% AEP (24 min breach)					
without Dambreak	Mouat Street	23067	320	28.8	7.69
	Brigalow Street	7400	240	21.6	1.85
	Boyd street	700	80	7.2	0.01
	Lewin Street	700	340	30.6	0.02
with Dambreak	Mouat Street	23067	320	28.8	7.69
	Brigalow Street	7400	240	21.6	1.85
	Boyd street	700	80	7.2	0.01
	Lewin Street	700	340	30.6	0.02
0.4% AEP (DCF) 17 min breach					
without Dambreak	Mouat Street	23067	130	11.7	3.12
with Dambreak	Mouat Street	23067	300	27	7.21
	Brigalow Street	7400	380	34.2	2.93
	Boyd street	700	80	7.2	0.01
	Lewin Street	700	120	10.8	0.01
0.4% AEP (DCF) 22 min breach					
without Dambreak	Mouat Street	23067	190	17.1	4.57
with Dambreak	Mouat Street	23067	370	33.3	8.89
	Brigalow Street	7400	450	40.5	3.47
	Boyd street	700	80	7.2	0.01
	Lewin Street	700	190	17.1	0.01

Notes:
1) The roads located within the affected zone shown in Appendix G .
2) Daily traffic volume data provided by Roads ACT or estimated as described previously.
3) The exposure time for itinerants at risk of flooding is assumed to be the time it would take a vehicle travelling at a speed of 40 km/h to travel through the length of the inundated section within the affected zone (300 mm or greater incremental depth). No allowance was made for stopping at traffic lights or intersections.
4) The number of vehicles at risk was estimated based on the traffic flow data and the assumed travel time ie the average estimated number of vehicles on the road in the affected zone.

12.3.3 Total Population at risk

Based on Queensland Department of Environment and Resource Management (DERM) Guidelines for Failure Impact Assessment of Water Dams, June 2010 [Ref 4], it was assumed that a flood inundation depth of 0.3 m or greater will result in persons within the inundation zone being at risk.

Table 12-5 shows the total PAR for Southwell Park Basin. These results are derived from the tables provided in Section 12.3.1 and Section 12.3.2 of this report.

Table 12-5 Estimated Total PAR

Flood/Breach Scenario	PAR		Total PAR
	Non-itinerants (1)	Itinerants (2) (3)	
0.01% AEP (17 min breach scenario)			
without Dambreak	88 (60 night time)	11	99
with Dambreak	95(88 night time)	11	106
Incremental	7 (28 night time)	0.00	7 (28 night time)
0.01% AEP (24 min breach scenario- weekday daytime)			
without Dambreak	256	19	275
with Dambreak	579	19	598
Incremental	323	0	323
0.4% AEP- DCF (17 min breach scenario weekday daytime			
without Dambreak	0	6	6
with Dambreak	82	20	102
Incremental	82	14	96
0.4% AEP- DCF (22 min breach scenario weekday daytime)			
without Dambreak	0	9	9
with Dambreak	151	26	177
Incremental	151	17	168

Notes:
1) Based on DSC3A, the contribution of exposure factors were ignored in the assessment of non- itinerant PAR. There non-itinerant PAR was therefore estimated as the largest total population that is exposed at any one time on a regular basis.
2) An average of two people per vehicle was assumed in this assessment.
3) Based on DERM Guideline, the buildings inundated by flood depths less than 0.3m were excluded from this estimate.
4) For the 0.01% AEP 17 minute breach the maximum with Dambreak PAR occurs at a different time of day to the maximum incremental PAR, and thus both sets have been reported.

12.3.4 Dambreak PAR

The Dambreak PAR is the difference between the Total PAR and the PAR affected by the natural flood immediately prior to the dambreak. The purpose of the Dambreak PAR is to capture the number of people directly affected as a result of the dambreak event, thereby excluding those that have already been affected by natural flooding. To determine those people who can be discounted from the Dambreak PAR, the 2012 ANCOLD Guidelines [Ref 1] have suggested the following criteria:

- PAR exposed to pre-dambreak flood waters where the product of the depth (D) and velocity (V) of the flood waters is greater than 0.6 m^2/s. In determining this value of DV, D_{max} (peak flood depth) should not exceed 1.2 m or V_{max} should not exceed 1.5 m/s at the point of reference.
- PAR who have conclusively adequate warning time (at least 12 hours) of the event or have been subjected to pre-dambreak flooding for a period of at least 12 hours and are within an area which is covered by a demonstrably effective and reliable Dam Safety Emergency Plan (or Emergency Action Plan) that includes inundation mapping and provides for safe evacuation of areas above the dam break flood level.

A summary of the Dambreak PAR estimated for Southwell Park Basin is shown in Table 12-6 below.

Table 12-6 Dambreak PAR

Flood Scenarios	Non-Itinerants		Itinerants		Total Dambreak PAR (2)
	Pre-Dambreak Flood(1)	Post-Dambreak Flood)	Pre-Dambreak Flood(1)	Post-Dambreak Flood	
0.01% AEP flood (17 minute breach scenario)	0	95	11	11	95
0.01% AEP flood I (24 minute breach scenario)	14	579	19	19	565
0.4% AEP (DCF 17 minute breach scenario)	0	82	1	20	102
0.4% AEP flood (DCF 22 minute breach scenario)	0	151	3	25	172

Notes:
1) Pre dambreak PAR comprises the persons who are affected by the natural flood immediately prior to the dambreak only
2) Total Dambreak PAR is the sum of non-itinerants and itinerants exposed to the pre-dambreak flood subtracted from those exposed to the post dambreak flood.

12.4 Potential Loss of Life (PLL) estimate

Based on Graham's method the Potential Loss of Life (PLL) in an inundation zone is estimated by applying recommended fatality rates to the Total Population at Risk downstream of the dam. These fatality rates vary depending on the flood severity, warning time to populations at risk downstream of the dam and the flood severity understanding. The fatality rates used in the dambreak flood scenarios are derived from Graham's paper 1999 [Ref 7], whilst the fatality rates used in the flood scenarios without dambreak are as recommended by 2012 ANCOLD Guideline

12.4.1 Flood Severity and Understanding

Based on the updated DSC3A Guidance sheet on Consequence Categories for Dams [Ref 2], the following criterion should be used for the assessment of the flood severity at buildings:

- Flood severity at buildings "Low" where Depth × Velocity (DV) < 4.6 m^2/s OR Flood Depth (D) < 3m
- Flood severity of at buildings "Medium" where DV ≥ 4.6 m^2/s AND D ≥ 3m.

In this assessment, the severity of the flood at each location was assessed based on the maximum flood depth (D) and DV values obtained at each property from the results of the dambreak study.

The severity classifications given above are not appropriate for pedestrians or vehicles (if caught in a 3 m high flood wave or at 4.6 m^2/s any pedestrians or occupants of a car would be unlikely to survive). Severity for roads was therefore defined in line with the NSW FDM and ARR so that:

- Flood severity for roads "Low" where Depth x Velocity < 0.6 m2/s or Flood Depth (D) < 0.3 m
- Flood severity for roads "Medium" where Depth x Velocity 0.6- 0.8 m2/s AND D > 0.3 m
- Flood severity for roads "High" where Depth x Velocity > 0.8.

Due to the low exposure times of students being outside, and for simplicity it has been assumed that all persons will be in the buildings when assessing the schools for PLL.

12.4.2 Warning time

No Dam Safety Emergency Plan (DSEP) is known to exist for Southwell Park Basin. For the present consequence category assessment, the following assumptions have been made to estimate the warning time:

- As there are no alarms set on the Southwell Park gauge, it is likely that authorities will be notified of the potential risk of flooding when flow is observed from the starter chute. It is assumed that a depth of the flow over the Southwell Park starter chute of 200 mm is required for this to be noticed..
- It is assumed it will take members of the public 15 min to alert authorities.
- From the time that authorities are notified, it is assumed that warnings will take 1 hour during the day time and an hour and half during the night time before occupants of a potentially inundated dwelling are instructed to evacuate or roads are closed.

The warning time is estimated from the time it takes the flood wave to reach a specific location in the inundation zone, minus the time required to raise the evacuation alert.

For the 0.01% AEP (2-hour event) the starter chute level (571.83 m AHD) is reached at 1.10 hours, and is exceeded by 200 mm at 1.15 hours.

Based on the breach hydrographs included in Appendix G "No warning Time" was assumed for the 10,000 ARI without and with dambreak scenarios for the following reasons:

- The impact of the 10,000 ARI rain event is significant and the flood levels will rise by 0.3 m across the potential inundation areas within less than an hour after the Amber Alert is triggered (i.e. the flood water reaches 571.83 m AHD) in the basin).
- The flood wave would already reach the properties and roads before basin embankment breaches.

For the 0.4% AEP (90-min event) the starter chute level (571.83 m AHD) is reached at 1.68 hours, and is exceeded by 200 mm 20 minutes later. If an alarm is not raised until either of these levels are reached, based on the previous assumptions there is not sufficient warning time for the affected areas and "No warning time" has been adopted. Refer to Appendix G for the breach hydrographs for the 0.4% AEP scenarios assessed

The PAR is immediately downstream of the dam and the warning time from the commencement of a dambreach will always be limited. Implementation of a DSEP however, including an appropriate flood warning system, would serve to improve the warning time of imminent overtopping events (i.e. a likely cause of dam failure) and would enable early evacuation on account of the impending risk. This would reduce the potential loss of life in the event of dam failure.

12.4.3 Potential Loss of Life

Based on the Graham's method 1999 – Table 7 fatality rates were adopted given the flood severity of "Low" and "No warning" in each dambreak event. Based on ANCOLD Guidelines 2012 the fatality rates in the low end of the range used for dambreak events were adopted for natural flooding in this assessment. Therefore a fatality rate of 0.0002 was adopted where the flood severity was assessed to be "Low" (refer to Table 12-7).

The fatality rates were multiplied by PAR to determine the PLL value. The PAR values used in this assessment the non-itinerants and itinerant. The results of this assessment are shown in Table 12-8 below.

Table 12-7 Assumptions Used in Estimate of PLL

Flood/ Breach scenario	Flood Severity	Warning Time (min)	Flood Understanding	Adopted Fatality Rates (1)	
				Suggested Estimates	Suggested Range
without Dambreak	Low	No Warning	Not Applicable	0.0002	----
	Medium	No Warning	Not Applicable	0.03	
	High	No Warning	Not Applicable	0.3 (assume lower bound from dam break values due to failure of fuse-plug)	
With Dambreak	Low	No Warning	Not Applicable	0.01	0.0 to 0.02
	Medium	No Warning	Not Applicable	0.15	0.03 to 0.35
	High	No Warning	Not Applicable	0.75	0.3 to 1

Table 12-8 Summary of Potential Loss of Life (PLL) Estimates

Flood Scenario [1]	Estimated PLL (Graham 1999)		
	Suggested Estimates	Lower Bound Estimates	Upper Bound Estimates
0.01% AEP (17 min breach scenario)			
Without Dambreak	3.11	0.03	0.03
With Dambreak	9.42	3.39	13.19
Incremental	6.31	3.36	13.16
0.01% AEP (24 min breach scenario)			
Without Dambreak	5.08	0.05	0.05
With Dambreak	20.10	5.73	30.67
Incremental	15.02	5.68	30.62
0.4% AEP- DCF (17 min breach scenario)			
Without Dambreak	0.00	0.00	0.00
With Dambreak	5.00	0.01	7.39
Incremental	5.00	0.01	7.39
0.4% AEP- DCF (22min breach scenario)			
Without Dambreak	0.02	0.01	0.01
With Dambreak	10.31	0.03	15.01
Incremental	10.29	0.02	15.00

Table 12-9 Summary of Potential Loss of Life (PLL) estimates

Flood Scenario (1)	Estimated PLL (Graham 1999)	
	Suggested Estimates (0.3 m or greater above assumed floor level)	Suggested Estimates (all wet floors)
Without Dambreak	3.11	3.11
With Dambreak	9.42	9.85
Incremental	6.31	6.74
Without Dambreak	5.08	5.15
With Dambreak	20.10	20.31
Incremental	15.02	15.17
Without Dambreak	0.00	0.00
With Dambreak	5.00	6.78
Incremental	5.00	6.78
Without Dambreak	0.02	0.02
With Dambreak	10.31	12.50
Incremental	10.29	12.48

12.5 Severity of damage and loss

The severity of damage and loss due to Southwell Park Basin embankment failure for each flood scenario was assessed using Tables in Appendix B – Selection of Severity of Damage and Loss - in the 2012 ANCOLD Guidelines on the Consequence Categories for Dams [Ref 1]. This assessment is based on the potential damages and losses including total infrastructure costs, impact on dam owner's business; health and social impacts.

A flood severity of "Major" was estimated for impacts due to the failure of basin embankment for all the flood scenarios evaluated in this assessment. The details of this assessment are provided in Appendix G.

12.6 Sunny day consequence category assessment

Southwell Park Basin is a "dry" detention basin. Therefore the Sunny Day consequence category of the basin is assessed to be "**Very Low**".

12.7 Flood consequence category assessment

As mentioned previously, the 2012 ANCOLD Guidelines [Ref 1] require that the consequence category assessment be based on either the Dambreak PAR or Incremental PLL in conjunction with the severity of damage and loss relating to the affected area.

For a severity of damage and loss of "Major", the Flood Consequence Categories for Southwell Park Basin have been assessed based on the estimated Dambreak PAR in Table 12-6, in compliance with the DSC3A. The results of this assessment are presented in Table 12-10.

The assessment of the consequence category based on Loss of Life (LOL) was carried out using the PLL figures given in Table 12-8 in compliance with the ANCOLD Guidelines on the Consequence Categories for Dams, October 2012 -Table 3 [Ref 1]. The results of this assessment are presented in Table 12-10.

Where differences exist between the Consequence Category based on PAR, and that based on PLL, PLL is to have primacy in accordance with the 2012 ANCOLD guidelines. This does not affect this assessment as both the PAR and PLL assessments have produced the same result.

Table 12-10 Summary of consequence category assessment

Flood Scenario	Severity of Damage and Loss	Estimated Dambreak PAR (1)	Consequence Category based on PAR(2)	Estimated PLL (3	Consequence Category based on LoL(2)	Suggested Consequence Category
PMF	Major	<1 (no significant affected area)	Significant	Major	<1 (no significant affected area)	Significant
0.01% AEP (17 min breach scenario)	Major	94.8	High B	6.31	High A	High A
0.01% AEP (24 min breach scenario)	Major	564.6	High A	15.02	High A	High A
0.4% AEP (17 min breach scenario)	Major	101.8	High A	5.00	High A	High A
0.4% AEP (22 min breach scenario)	Major	172.4	High A	10.29	High A	High A

Notes:
1) Refer to Table 12-6.
2) Based on a severity of damage and loss of “Major”.
3) Refer to Table 12-8.

12.7.1 Sensitivity analysis

A number of sensitivity analyses were carried out to investigate the impact of various assumptions made in the assessment of the consequence category of Southwell Park Basin based on LOL. Variations in the following factors were evaluated as part of this sensitivity assessment:

- Ground Levels: Inclusion of the buildings with wet floor in estimate of PLL in 0.4% and 0.01% AEP events.
- Reducing St Ninian's PAR to 30 with an exposure factor of 2/24 (2 hours out of 2 days at 12-hours a day)
- Reducing the assumed driving speed to 20km/hr on all affected roads, and adding a stoppage time of 30 seconds to travel times on Brigalow and Mouat Streets to account for traffic lights. This increases travel times and in turn vehicle exposure times.

The results of this sensitivity analysis are presented Table 12-11 below.

Table 12-11 Results of sensitivity analysis on PLL brackets

Description	Consequence Category Assessment (1) Based on Variation in		
	Inclusion of all wet floors in PAR	Reducing St Ninian's PAR	Increases in travel times
0.01% AEP	No change to bracket	No change to bracket	No change to bracket
0.04%	No change to bracket	PLL for 17 min breach reduces to 4.33 (lower bracket) As there is no change to the 0.01% AEP bracket there is no change to the overall category.	No change to bracket

The described changes to assumptions would not change the Flood Consequence Category Assessment for the Southwell Park basin.

12.8 Acceptable flood capacity

The NSW Dam Safety Committee's guideline on Acceptable Flood Capacity, DSC3B [Ref 8] requires that a basin with a Flood Consequence Category (FCC) of **High A** safely pass flood events with an Annual Exceedance probability (AEP) of up to the PMPDF.

If the fuse-plug operates as assumed in this analysis (refer Sections 0 and 11), Southwell Park Basin would pass the 0.4% AEP flood before the flood reaches the dam crest level at the bike path (i.e. no dry freeboard). Therefore the current estimate of flood discharge capacity of Southwell Park Basin does not satisfy the DSC and ANCOLD requirement on Acceptable Flood Capacity for a "High A" consequence category dam.

We do however also note that there may be an incremental PAR / PLL associated with operation of the fuse-plug, however this has not been assessed at this stage. The suitability of the fuse-plug spillway should be critically reviewed when the remedial measures to pass the AFC are determined.

A DSEP and flood warning system should also be implemented to reduce the risk to loss of life in the event of dam failure.

In DSC3B the NSW DSC requires immediate notification when a High A dam does not pass the 0.01% AEP. It is recommended that this be discussed with the ACT regulator as soon as possible.

13. Kenny basin breach modelling

13.1 Methodology

For each event the critical duration event was selected based on the maximum basin flood level from the RORB output.

Table 13-1 Critical duration based on maximum flood level in proposed Kenny basin

Event	Basin used for critical duration	Critical storm duration
0.01% AEP	Kenny	1 hour
0.01% AEP	Southwell Park	2 hour
PMF (spatial distribution centred on the Kenny basin catchment)	Kenny	1 hour
PMF (spatial distribution centred on Southwell Park)	Southwell Park	1.5 hour
Acceptable Flood Capacity (AFC) 1% AEP	Kenny	2 hour

These events were then simulated in TUFLOW for the no breach scenario.

The breach time and size were estimated in line with ANCOLD Bulletin 97 with adaptation for the detention basin design and configuration of spillways and outlets. The scenarios in Table 13-2 were simulated.

Table 13-2 Flood scenarios to be assessed for Kenny basin Consequence Category Assessment

Scenario number	Flood	No Basin Failure	With failure of Kenny Basin only
S1	0.01% AEP (1 hour)	X	
S2	0.01% AEP (1 hour)		X
S3	0.01% AEP (2 hour)	X	
S4	0.01% AEP (2 hour)		X
S5	PMF (Kenny basin critical duration and spatial distribution)	X	
S6	PMF (Kenny basin critical duration and spatial distribution)		X
S7	PMF (Southwell Park critical duration and spatial distribution)	X	
S8	PMF (Southwell Park critical duration and spatial distribution)		X
S9	Kenny basin AFC	X	
S10	Kenny basin AFC		X

13.2 Failure modes and locations

The breach scenarios were simulated in TUFLOW directly using “variable z shapes” to change the elevation of the embankment over time. Two possible failure locations were considered:

1. To the north of the spillway, where water is able to build up to the embankment crest
2. At the southern end of the spillway near the outlets / low point.

Failure modes for the embankment are indicated in Table 13-3.

Table 13-3 Adopted breach failure modes

Event	Failure mode
0.01% AEP	Piping
PMF	Overtopping
AFC	Piping

13.3 PMF hydrology for Kenny basin

Spatial distribution of PMP rainfall was undertaken centred on the Kenny basin catchment. The 1 hour storm was shown to be critical for basin inflow, stage and outflow. An initial loss of 0 and continuing loss of 1 mm/hr was applied.

The 1 hour storm was shown to be critical for basin inflow, stage and outflow, as shown in Table 13-4 below. When simulated in TUFLOW, the peak level for the 1 hour duration was 590.31 m AHD.

Table 13-4 Kenny PMF summary (hydrology)

Storm duration	Total spatially distributed rainfall depth (mm)	Peak inflow at Kenny basin (m^3/s)	Peak outflow at Kenny basin (m^3/s)	Peak basin level at Kenny basin (m AHD)
15 minute	120	576.8	571.7	590.1
30 minute	170	806.1	800	590.35
45 minute	220	959	952.9	590.49
60 minute	**260**	**982.2**	**976.5**	**590.51**
90 minute	340	971.6	966.1	590.5
2 hour	390	890.2	888.1	590.43
2.5 hour	440	830.1	827.6	590.38
3 hour	480	774.8	772.4	590.32
4 hour	540	672.5	668	590.21
5 hour	590	601.4	594.9	590.13

For the PMP rainfall based on the spatial distribution centred on Southwell Park, the 1.5 hour duration was shown to be critical as described in Section 11.3. The maximum flood level in the proposed Kenny basin for this event is 590.24 m AHD.

13.4 PMF breach formation parameters

The duration and degree of overtopping that an embankment might withstand before failing is unpredictable. ANCOLD Bulletin 97 quotes a number of criteria such as 900 mm for over an hour or 600 mm for up to one day but also suggests that for overtopping failures it is generally conservatively assumed that the embankment begins to fail with any overtopping.

It was assumed that the overtopping breach of the main crest would initiate at a level of 590.1 m AHD (100 mm above the crest). Based on the volume and head of water at this time, breach times and sizes were predicted for the following methods:

- MacDonald Langridge Monopolis method (in line with the recommendations of ANCOLD Bulletin 97)
- Froehlich (for comparison)
- Bureau of Reclamation (for comparison)
- Von Thun Gillette (for comparison).

During the time it takes an overtopping breach to form, the flood level in the basin continues to rise, such that the volume and head at the time of failure would actually be larger. The breach parameters were estimated by iterating on volume and head until the water level at the breach time matched that specified for the breach formation.

A summary of breach parameters (breach size and formation time) for the PMF events is summarised in Table 13-5. Detailed breach parameters have been included from Table H-1 to Table H-4 and Figure H-1 to Figure H-4of Appendix H.

Table 13-5 PMF summary breach parameters

Event	Breach location	Min breach width (m)	Max breach width (m)	Min breach formation time (min)	Max breach formation time (min)
PMF 1 hour	North of spillway	3.9	13.9	3	56
PMF 1 hour	At spillway	6.8	11.7	6	35
PMF 1.5 hour	North of spillway	3.7	12.5	3	54
PMF 1.5 hour	At spillway	6.6	11.5	6	34

For the location to the north of the spillway, breaches larger than 12 m base width do not satisfy the Singh and Scarlatos geometry checks specified in ANCOLD Bulletin 97 (ratios of breach top width to breach base width, and breach top width to breach height based on analysis of 52 historical dam failures). ANCOLD recommends breaches should meet the geometry checks except in special circumstances. For those that did satisfy the geometry checks, the breach widths at both locations were adjusted to match the 5m cell size of the TUFLOW model. For example, a 3 m breach would effectively be a 5 m breach or a 9 m breach would effectively be 10 m breach. Given the scatter observed in predicted breach sizes, these small adjustments to the predicted sizes are not considered significant. The breaches simulated in the hydraulic model for the PMF 1 hour and 1.5 hour event are summarised in Table 13-6.

Table 13-6 PMF 1 hour and 1.5 hour breach parameters simulated

Simulated breach base width	Basis	Simulated breach development time	Basis	PMF 1 hour Basin flood stage at time of failure (m AHD)	PMF 1.5 hour basin flood stage at time of failure (m AHD)
5 m	Low range Singh and Scarlatos acceptable geometry at north side of spillway (bottom of breach 588.6 m AHD)	11 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	590.28	590.14
10 m	High range Singh and Scarlatos acceptable geometry at north side of spillway (bottom of breach 588.6 m AHD)	14 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	590.30	590.19
10 m	Singh and Scarlatos acceptable geometry t spillway (bottom of breach 587.2 m AHD)	21 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	590.28	590.24

13.5 0.01% AEP breach formation parameters

For the 0.01% AEP event, it was assumed a breach would initiate when flood levels reached 750 mm below the crest of 590 m AHD (i.e at 589.25 m AHD). Given that the 0.01% AEP 2 hour event was critical at Southwell Park, the event was also simulated. For the critical duration storm event of 1 hour this level is reached at 0.7 hours. For the 2 hour event, this level is reached at 0.98 hours. As with the other events, the breach formation parameters were calculated according to Macdonald Langridge-Monopolis, Bureau of Reclamation, Von Thun Gillette and Froehlich methods as detailed in Table H-5 to Table H-8 of Appendix H.

Table 13-7 summarises the breach parameters for the 0.01% AEP events.

Table 13-7 0.01% AEP summary breach parameters

Event	Breach location	Min breach width (m)	Max breach width (m)	Min breach formation time (min)	Max breach formation time (min)
0.01% AEP 1 hour	North of spillway	1.6	7.1	2	36
0.01% AEP 1 hour	At spillway	2.8	9.2	5	26
0.01% AEP 2 hour	North of spillway	1.4	6.9	2	34
0.01% AEP 2 hour	At spillway	2.4	9	4	25

As with the PMF breaches, the breach widths were adjusted to be consistent with the 5 m cell size of the TUFLOW model, and meet the Singh and Scarlatos geometry checks. Refer to Section 13.4 for more detailed explanation. The breaches simulated in the hydraulic model for both 0.01% 1 hour and 2 hour events are summarised in Table 13-8.

Table 13-8 0.01% AEP Breach parameters simulated (1-hour and 2-hour storm duration)

Simulated breach base width	Basis	Simulated breach development time	Basis	Basin flood stage at time of failure (m AHD)
5 m	Singh and Scarlatos acceptable geometry at north side of spillway (bottom of breach 588.6 m AHD)	11 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	589.25
10 m	Singh and Scarlatos acceptable geometry at north side of spillway (bottom of breach 588.6 m AHD)	14 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	589.25
10 m	Singh and Scarlatos acceptable geometry t spillway (bottom of breach 587.2 m AHD)	21 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	589.25

13.6 Results

The model results were post-processed to create maps of depth, flood level contours, velocity, velocity depth products and areas of greater than 300 mm incremental depth ('affected zones'). Hydrographs were also generated at selected locations. These are included in Appendix H for various scenarios. The results of the breach modelling are discussed under the corresponding event heading in the following sections.

13.6.1 PMF

For the both the PMF 1 hour and 1.5 hour event, there is widespread flooding below the proposed Kenny basin. The area affected by a breach (flood levels 300 mm or greater above the no breach maximum) for both events is very small in all cases (refer Appendix H). Hydrographs at the breach locations show that there is some increased flow due to the breaches, but the overall outflow from the basin does not increase significantly. Results for the PMF 1 hour event is shown in Figure 13-1 and

Table 13-9 and results for the PMF 1.5 event is shown in Figure 13-2 and Table 13-10.

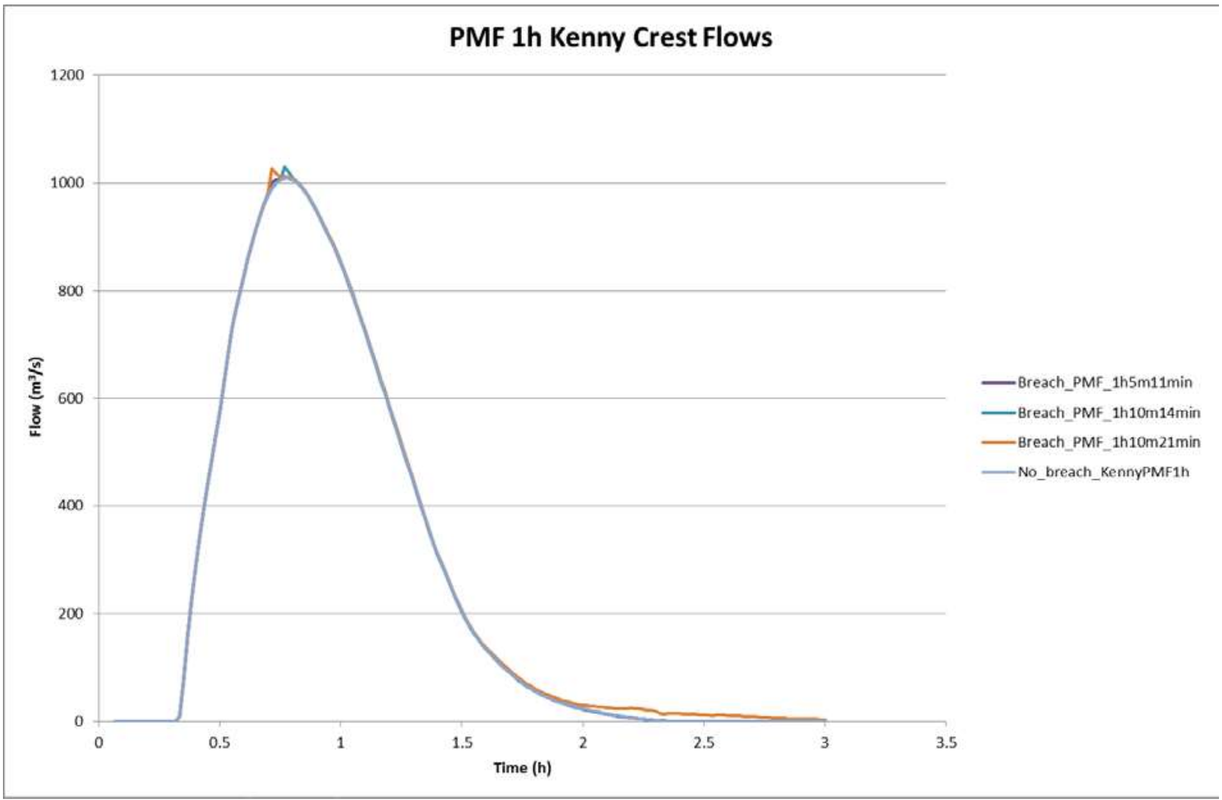

Figure 13-1 Kenny basin PMF 1 hour overland outflow hydrographs

Table 13-9 PMF 1 hour flood peak overland outflows (excludes culverts)

Scenario	Peak flow at Kenny basin crest (m^3/s)	Peak flow through breach (m^3/s)
No breach	1009	
5m breach at north side of spillway	1009	19.5
10m breach at north side of spillway	1030	36.5
10 m breach at spillway	1026	97

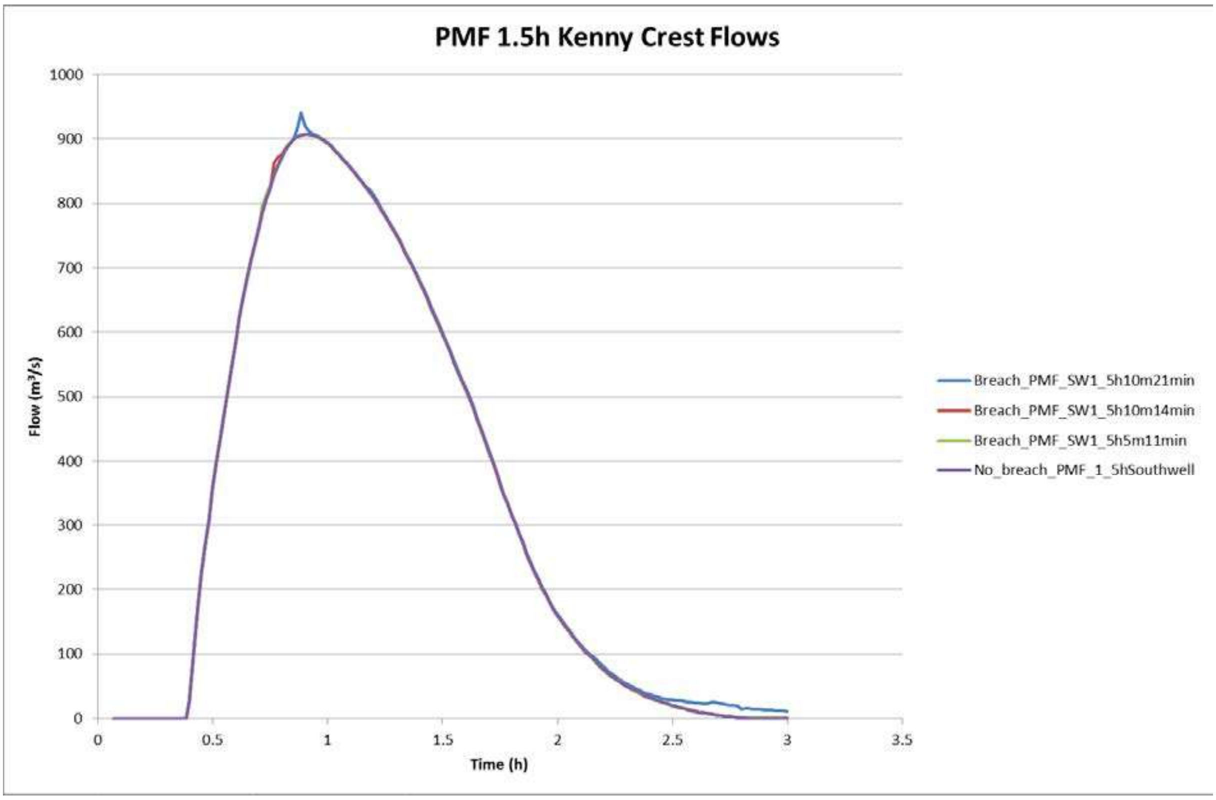

Figure 13-2 Kenny basin PMF 1.5 hour overland outflow hydrographs

Table 13-10 PMF 1.5 hour flood peak overland outflows (excludes culverts)

Scenario	Peak flow at Kenny basin crest (m^3/s)	Peak flow through breach (m^3/s)
No breach	908	40.6
5m breach at north side of spillway	908	17.8
10m breach at north side of spillway	908	32.5
10 m breach at spillway	941	94.2

In addition, flood volume downstream of the breach near Barton Highway was also considered and there is very little increase in volume due to the breaches as summarised in Table 13-11 and Table 13-12. Therefore, the breaches modelled for the PMF are unlikely to have any effect on Southwell Park.

Table 13-11 PMF 1 hour flood volume near Barton Highway

Scenario	Peak flow PMF 1 hour (m^3/s)	Volume PMF 1 hour (ML)
No breach	1868	24395
5m breach at north side of spillway	1869	24395
10m breach at north side of spillway	1870	24393
10 m breach at spillway	1877	24458

Table 13-12 PMF 1.5 hour flood volume near Barton Highway

Scenario	Peak flow PMF 1 hour (m^3/s)	Volume PMF 1 hour (ML)
No breach	1860	31481
5m breach at north side of spillway	1860	31482
10m breach at north side of spillway	1860	31482
10 m breach at spillway	1867	31521

As the results show that affected zones are insignificant, no further analysis of the PMF was undertaken.

13.6.2 0.01% AEP

There is widespread flooding below the proposed Kenny basin for 0.01% AEP 1 hour critical event as well as the 2 hour event. The area affected by a breach (flood levels 300 mm or greater above the no breach maximum) for both events is very small in all cases (refer Appendix H). Hydrographs at the breach locations show that there is some increased flow due to the breaches for the events, but the overall outflow from the basin does not increase significantly. Results for the 0.01% AEP 1 hour event is shown in Figure 13-3 and Table 13-3, and results for the 0.01% AEP 2 hour event is shown in Figure 13-4 and Table 13-4.

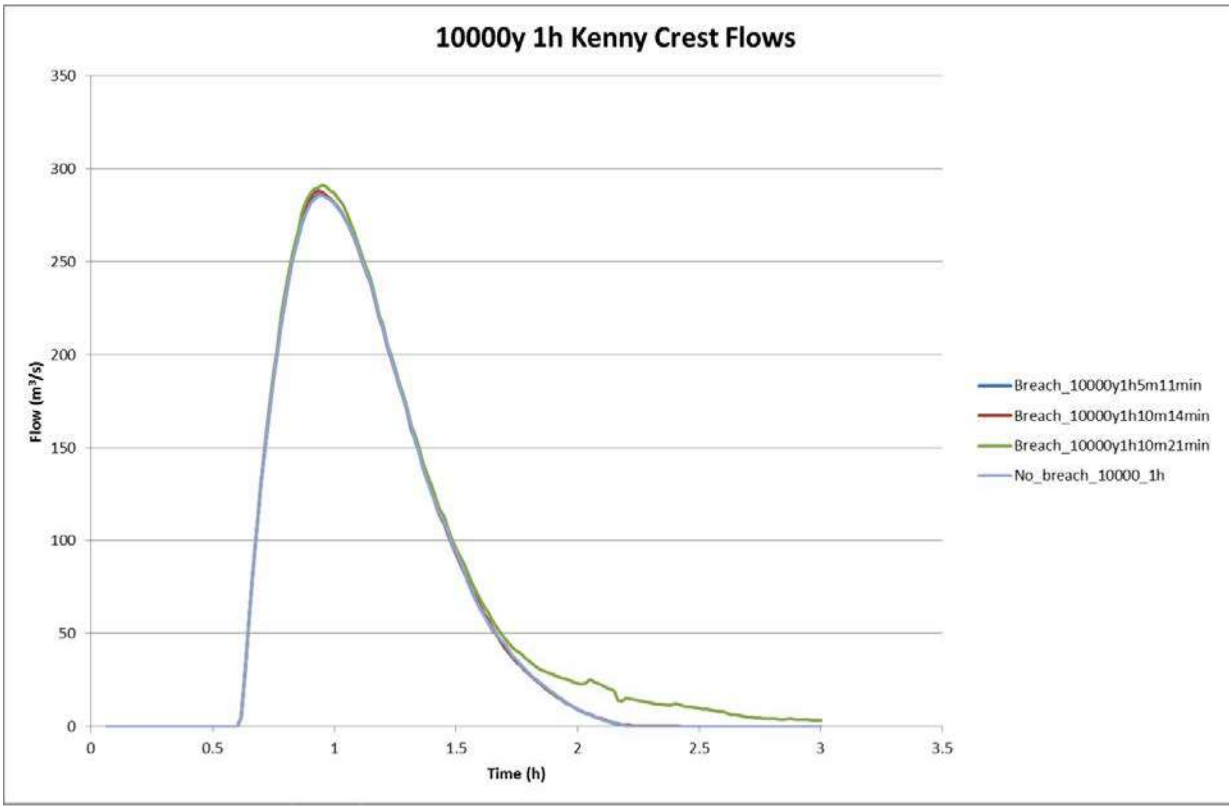

Figure 13-3 Kenny basin 0.01% AEP 1 hour overland outflow hydrographs

Table 13-13 0.01% AEP 1 hour flood peak overland outflows (excludes culverts)

Scenario	Peak flow at Kenny basin crest (m^3/s)	Peak flow through breach (m^3/s)
No breach	285	
5 m breach north side of spillway	286	5.7
10 m breach north side of spillway	288	13.0
10 m breach at spillway	291	52.1

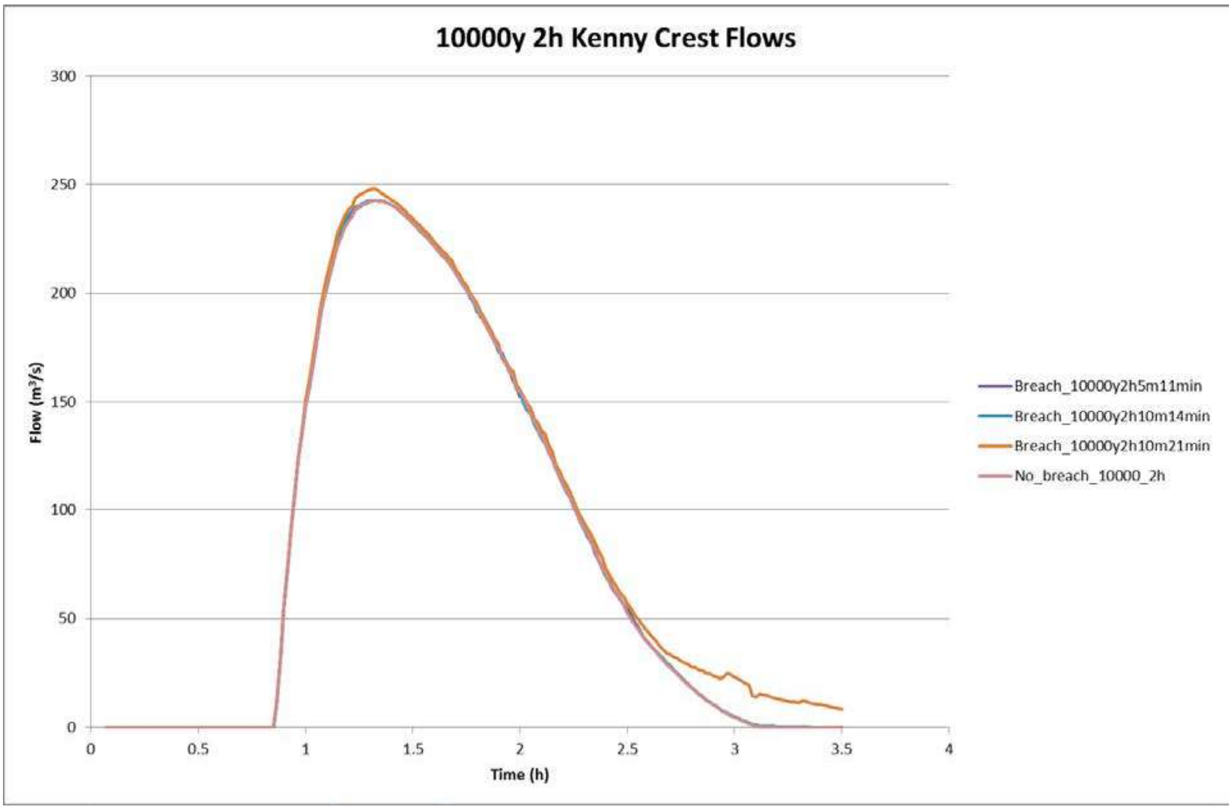

Figure 13-4 Kenny basin 0.01% AEP 2 hour overland outflow hydrographs

Table 13-14. 0.01% AEP 2 hour flood peak overland outflows (excludes culverts).

Scenario	Peak flow at Kenny basin crest (m^3/s)	Peak flow through breach (m^3/s)
No breach	242	
5 m breach north side of spillway	243	5.2
10 m breach north side of spillway	243	10.8
10 m breach at spillway	248	49.5

In addition, flood volume downstream of the breach near Barton Highway was also considered and there is very little increase in volume due to the breaches as summarised in Table 13-15 and Table

13-16. Therefore, the breaches modelled for the 0.01% AEP is unlikely to have any effect on Southwell Park.

Table 13-15 0.01% AEP 1 hour flood volume near Barton Highway

Scenario	Peak flow 0.01% AEP 1 hour (m3/s)	Volume 0.01% AEP 1 hour (ML)
No breach	508	7701
5m breach at north side of spillway	508	7702
10m breach at north side of spillway	508	7700
10 m breach at spillway	513	7769

Table 13-16 0.01% AEP 2 hour flood volume near Barton Highway

Scenario	Peak flow 0.01% AEP 2 hour (m3/s)	Volume 0.01% AEP 2 hour (ML)
No breach	526	10747
5m breach at north side of spillway	526	10750
10m breach at north side of spillway	526	10751
10 m breach at spillway	529	10800

As the results show that affected zones are insignificant, no further analysis of the 0.01% AEP was undertaken.

13.7 Kenny basin preliminary Consequence Category Assessment and AFC

The breach modelling of the proposed Kenny basin showed no significant affected areas. For the 0.01% AEP and PMF breach modelling there was no area of greater than 300 mm increase in flood level due to the breach beyond the Kenny development boundary.

Even though there is no significant affected area, it is possible that a failure of the proposed Kenny Basin could have a Medium severity of loss and damages in terms of “Effect on continuing credibility” and “Community reaction and political implications”. The construction of the basin would create disturbance, and the affected areas are expected to be within the construction footprint. It was therefore assumed that within the affected areas there would be no heritage values, water supplies for stock and fauna, impact on ecosystems or rare and endangered species which would be damaged or contaminated by the breach.

According to DSC3A, where the Population At Risk and Potential Loss of Life are less than one, and the Severity of Loss and Damages are “Medium” a dam has a “Low” consequence Category. DSC3B requires that a “Low” consequence category dam safely passes the 1% to 0.1% AEP flood event.

13.7.1 Kenny 1% AEP breach modelling

On the basis that a 1% AEP flood breach is likely to have a greater affected area than a 0.1% AEP flood breach, the decision was made to model the 1% AEP flood as the AFC breach.

13.7.2 1% AEP breach formation parameters

From the proposed Kenny development TUFLOW modelling the maximum flood level in the basin for the 1% AEP flood is 588.6 m AHD. Therefore the breach location at the north side of the spillway with a base level of 588.6 m AHD was not considered.

It was assumed a breach would initiate when flood levels reached 300 mm below the spillway crest of 588.8 m AHD (i.e at 588.5 m AHD). For the critical duration storm event of 2 hour this level is reached at 1.87 hours. As with the other events, the breach formation parameters were calculated according to Macdonald Langridge-Monopolis, Bureau of Reclamation, Von Thun Gillette and Froehlich methods as detailed in Table H-9 of Appendix H. The breach sizes ranged from -0.4 m to 6.8 m breach formation time from 3 minutes to 21 minutes for the breach location at the spillway. The negative breach base width from the Macdonald-Langridge Monopolis method suggests that is not enough volume and/or head to form a breach. The breach widths from the other methods were considered relative small compared to the cell size of the model of 5 m. Therefore it was decided to model breaches of 5 m or 10 m provided they satisfied Singh and Scarlatos geometry checks. The breaches simulated in the hydraulic model are summarised in Table 13-17.

Table 13-17 1% AEP Breach parameters simulated

Simulated breach base width	Basis	Simulated breach development time	Basis	Basin flood stage at time of failure (m AHD)
10 m	Singh and Scarlatos acceptable geometry at spillway (bottom of breach 587.2 m AHD)	21 min	Macdonald Langridge-Monopolis after ANCOLD/ DERM based on breach size	588.5

13.7.3 1% AEP breach results

The breach modelled for the 1% AEP 2 hour event resulted in very small affected areas, with the limit of 300 mm flood level increase confined within the actual breach zone.

14. Summary and conclusions

This report describes the flood study component of the project, including the dam break studies and Consequence Category Assessments for the existing basin at Southwell Park, and the proposed basin at Kenny. A RORB hydrological model was developed for the estimation of flood hydrographs for a range of historic and design events, up to and including the Probable Maximum Flood (PMF). These flows were input into TUFLOW two-dimensional hydraulic models, and flood mapping of Sullivans Creek, Mitchell Channel, Dickson Channel, O'Connor Channel and Kenny subsequently undertaken for the 10%, 1%, 0.2%, 0.01% AEP events and the PMF in existing catchment conditions. Flood mapping representing the proposed Kenny development was produced to downstream of Morisset Road. This report is subject to, and must be read in conjunction with, the limitations set out in Section 1.3 and the assumptions and qualifications contained throughout the Report.

Key features of the Sullivans Creek catchment include pervious alluvial and fractured rock layers which contain shallow aquifers, which heavily influence the runoff characteristics of the northern part of the catchment, and the retarding basin at Southwell Park.

There are two stream-flow gauges within the Sullivans Creek catchment, one at Southwell Park in Lyneham, and a second at Barry Drive in Turner. Although the rating curves are extrapolated above a limited number of gauging points, the data from these two gauges allowed calibration and validation of the hydrological and hydraulic models using a combination of historical events (February 2002, December 2010 and February 2011) and Flood Frequency Analysis of the annual maximums.

14.1 Existing condition flooding

Flood maps for the exisiting condition are contained in Appendix B for the various Annual Exceedance Probabilities.

14.1.1 10% AEP flooding

Modelling indicates that existing condition 10% AEP flood waters will generally be conveyed by the channels, culverts and bridges without affecting significant surcharging into surrounding areas. Overtopping is predicted at Miller Street on the O'Connor Channel, opposite the Morisset Wetland on Flemington Road (to the north of the culvert crossing) and over Thurbon Road (Riggall Place) in the Lyneham sporting precinct. Overland flow is also observed around Haig Park (Masson Street and the northern end of Macleay and Watson Streets) and behind Duffy Street at the bend of the Dickson Channel near Madigan Street. A small amount of breakout is observed through ANU. Flooding in all these areas is estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual.

Six crossings are likely to be inundated due to 10% AEP flooding (refer Appendix C for locations):

- S09-OC02 O'Connor driveway
- S09-OC04 Coolibah Footbridge
- S09-OC05 Miller Street
- S15 Thurbon Road (Rigall Place)
- S23 Flemington Road DS Morrisset Pond
- S25 Old Well Station 2.

14.1.2 1% AEP flooding

Modelling indicates that 1% AEP flood waters will largely be conveyed by the channels, culverts and bridges with limited flooding of surrounding areas. The accuracy of the flooding extent has not been ground truthed and there may be instances where the extent is limited by levees, walls, or other obstructions not included in the Digital Elevation Model supplied.

A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Condamine Street. Overtopping of Flemington Road is predicted opposite the Morisset Wetland (to the north of the culvert crossing), at Heffernan and Darling Streets in Mitchell, east along Sandford Street and south along Flemington Road. A small area of road in Exhibition Park is also predicted to be flooded. On the Dickson Channel flooding is predicted through a number of properties on Duffy Street at the bend near Madigan Street, a small amount of flooding is predicted where Majura Avenue crosses the Dickson Channel, and to the north of the channel from the Dickson District Playing Fields. At Cowper Street flooding is also observed to breakout to the south of the channel, and also at Challis and De Burgh Streets.

Thurbon Road (Rigall Place) in the Lyneham sporting precinct is predicted to be flooded. Flooding is predicted around the trunk underground drainage in Lyneham, including Challis Street, Northbourne Avenue, Oliver Street and Goodwin Street. Flooding is also observed around the trunk drainage in David Street (Turner).

Overland flow is also observed around Haig Park (Stawell Street, Ormond Street, Masson Street, Hackett Gardens, Miller Street, Barry Drive and the northern end of Macleay and Watson Streets). Breakout is also observed at the upstream end of ANU.

Flooding in all these areas is estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual.

Twenty two (22) crossings are likely to be inundated by the 1% AEP

- S04 ANU Biology
- S06 Barry Drive
- S09-OC02 O'Connor driveway
- S09-OC04 Coolibah Footbridge
- S09-OC05 Miller Street
- S11-D01 Dickson Lyneham Footbridge
- S11-D02 Deburgh Street
- S11-D04 Challis Street
- S11-D05 Cowper Street
- S11-D06 Dumaresq Footbridge
- S11-D07 Hawdon Footbridge
- S11-D10 Majura Avenue
- S11-D11 Duffy Footbridge
- S13 Fox Place Footbridge
- S15 Thurbon Road (Rigalli Place)
- S16 Yowani CC FP1
- S17 Yowani CC FP2
- S21-M02 Winchcombe Road Overpass
- S21-M03 Heffernan Overpass
- S23 Flemington Road DS Morrisset Pond

- S25 Old Well Station 2
- S26 Old Well Station 1.

14.1.3 0.2% AEP flooding

Modelling estimates that not all of the 0.2% AEP flood will be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. Nearly all major crossings observed are subject to flooding with the exception of Parkes Way at the outlet of Sullivans Creek.

There is significant flooding predicted at ANU downstream of Barry Drive. There is also significant flooding predicted in residential areas between Barry Drive and David Street in Turner. A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Stawell Street. For the Dickson channel, significant amount of breakaway flooding is predicted to occur at the upstream end of the channel until it re-joins the Dickson channel near Cowper Street. Flooding is also predicted to occur along the entire length of the Dickson channel, with several dwellings on the north and south sides of the channel being affected.

The Southwell park fuse-plug is predicted to activate with significant flooding occurring within the sports precinct. Flooding at Randwick pond is predicted to occur and flooding continues south, through the Canberra Racecourse area and overtops the Barton Highway. Overtopping of Flemington Road is predicted opposite the Morisset Wetland (to the north of the culvert crossing). Flooding is also predicted to occur along the Mitchell channel.

Localised flooding from pipe networks is also generally predicted to occur throughout the catchment. An area of interest is near Stirling Avenue in Watson, where breakout flooding is predicted to occur through EPIC and along Federal Highway from Stirling Avenue to Barton Highway. Several residential dwellings are predicted to be flooding parallel to Federal Highway.

Flooding in these areas is generally estimated to fall into the low hydraulic hazard category as defined by the NSW Flood Development Manual. Flooding along the floodplains of Sullivans Creek channel is estimated with a high hazard category, however very few residential areas lie within this area. There are some areas (generally on roads) where flooding has been categorised as medium to higher hydraulic hazard. This is mostly observed for flooding on roads in the suburbs of Turner and O'Connor. Breakout flooding south of Randwick pond, in areas of the Canberra Racecourse and just upstream of the Barton Highway have been categorised with medium to high hazard as well.

14.1.4 0.01% AEP flooding

Modelling estimates that 0.01% AEP flood waters will not all be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. Nearly all major crossings observed are subject to flooding with the exception of Parkes Way at the outlet of Sullivans Creek.

There is significant flooding predicted at ANU with flood extents bounded by Daley Road to the west and Childers Street and Ellery Crescent to the east. There is also significant flooding predicted in residential areas between Barry Drive and David Street in Turner, with flood extents bounded by Frogartt Street to the west and Moore Street to the east. A significant amount of breakaway flooding is observed from the O'Connor Channel, which causes fairly widespread overland flooding until re-joining Sullivans Creek near Stawell Street.

For the Dickson channel, significant amount of breakaway flooding is predicted to occur at the upstream end of the channel as well as flooding along the entire length of the Dickson channel. This flooding extends to Antill Road and Bonython Street to the north near the Dickson District Playing Fields.

The Southwell park fuse-plug is predicted to activate with significant flooding occurring within the sports precinct. It is predicted that there is significant flooding south of Randwick pond, at Canberra

Racecourse and further south which overtops the Barton Highway. Overtopping along a significant length of Flemington Road is predicted due to breakout flooding coming from the east and north. Flooding is also predicted to occur along the Mitchell channel.

Within EPIC, flooding is predicted to occur which mainly comes from Old Well Station Road and Morisset Road, as well as breakaway from pipe networks near Stirling Road. The breakaway at Stirling road is also predicted to flood along Federal Highway as well as several residential dwellings parallel to the Federal Highway.

Many areas of flooding are generally estimated to fall into the high hydraulic hazard category as defined by the NSW Flood Development Manual. This includes most of the flooding along the main Sullivans Creek channel. However, breakaway flooding from the O'Connor and Dickson channels are estimated to fall into the low hazard category, with the exception of a few flooding roads with a high category.

14.1.5 PMF flooding

Modelling estimates that PMF flood waters will not all be conveyed by the channels, culverts and bridges, with many areas predicted to be affected. All major crossings observed are subject to flooding including Parkes Way at the outlet of Sullivans Creek.

Flood extents span approximately 0.5km to 1 km wide along the main Sullivans Creek channel.

There is significant breakaway flooding predicted from the O'Connor and Dickson Channels. There is also breakaway predicted from Stirling road, which causes flooding along Federal Highway and residential areas parallel to it.

Most areas of flooding are generally estimated to fall into the high hydraulic hazard category as defined by the NSW Flood Development Manual. This includes most of the flooding along the main Sullivans Creek channel. However, there are some areas of breakaway flooding from the O'Connor and Dickson channels which were estimated to fall into the low hazard category, with the exception of a few flooding roads with a high category.

14.1.6 Sensitivity analysis

Flood maps for the various sensitivity scenarios and AEP's are provided in Appendix D

Increased rainfall intensity due to climate change was simulated (30% increase). The impact on flood extents was greatest in the smaller channels like the O'Connor, Dickson and Mitchell Channels. For the 10% AEP increases ranged from around 0.1-0.4 m in Kenny and parts of the Dickson and Mitchell Channel, 0.5 m near the racecourse to 0.2-0.4 m in Sullivans Creek, 0.2 m in the Lyneham Wetland, 0.6 m at Southwell Park and up to 0.8 m in parts of the O'Connor Channel. For the 1% AEP increases ranged from around 0.1-0.3 m in Kenny, 150-400 mm in Sullivans Creek, 0.5 m in the Lyneham Wetland, 0.7 m at Barry Drive, 0.75 m at Southwell Park and up to a metre in parts of the Mitchell and O'Connor Channels. Localised increases of 0.6 m to in excess of a metre were observed in areas in and around the Dickson Channel.

Application of blockage factors (in line with ARR project 11) at the Southwell Park inlets (50%) and Majura Avenue (75%) did not have a significant effect on flood levels around Southwell Park. A 75% blockage of the Majura Avenue bridge increased flood levels and caused additional breakaway flooding to the west and northwest.

Increasing the Manning's 'n' roughness values resulted in increased flood levels in many areas but did not significantly change the flood extents due to the terrain. New minor flow paths are initiated in some areas, such as through Kenny and downstream of Barry Drive.

The tailwater level for the downstream boundary condition of the hydraulic model was increased by up to a metre, and only affected flood levels from the lake up to the weir control downstream of Fellows Rd for both the 10% and 1% AEP.

A brief sensitivity analysis on the Southwell Park fuse-plug failure assumptions was undertaken in both RORB and TUFLOW. The impact was less significant for the 0.04% AEP and PMF. Changes in assumptions could potentially have a significant impact on the rare / extreme events between the 0.04% AEP and PMF however.

14.1.7 Southwell Park

Maps for Southwell Park breach modelling are contained in Appendix G

The Southwell Park retarding basin stores approximately 250 ML below the “starter chute” spillway level, and has a fuse-plug spillway which is designed to progressively fail laterally and via overtopping in events rarer than the 1% AEP. The level at which the fuse-plug fails, and the time it takes to fail are critical assumptions which affect the flood levels in the basin and the flood behaviour downstream. Further investigation of the fuse-plug and it’s likely operation is recommended. The lowered embankment crest level where the bike path crosses to the east of the outlet culverts results in a Dam Crest Flood of 0.4% AEP, a higher AEP than if the main crest level had been maintained. A number of embankment breach scenarios were simulated for the 0.4% AEP, 0.01% AEP and PMF. Breaches to the west of the fuse-plug were found to be critical, with large affected areas (300 mm or more increase in flood levels due to the breach) and concentrations of Population at Risk on busy Mouat Street, at the Lyneham Motor Inn, St Ninian’s Uniting Church, Brindabella Christian College and Lyneham Primary, along with residential dwellings on Mouat Street, Brigalow Street, Boyd Street and Lewin Street. The Comprehensive Category Assessment for Southwell Park ranks the basin as a “High A” dam, and remedial measures will need to be undertaken so that it can safely pass the PMPDF. It is recommended that a Dam Safety Emergency Plan be prepared and implemented for the basin to reduce the risk of loss of life in the event of the basin breaching.

14.1.8 Kenny development and Kenny basin breach modelling

Flood maps for Kenny development are contained in Appendix F, and for the Kenny Basin breach refer to Appendix H.

The RORB model was updated to reflect the proposed development, with the increase in impervious areas increasing peak flows. To attenuate these back to the existing model peaks for the 10% and 1% AEP events preliminary sizing of the storage and staged outlet with high level spillway was undertaken, and shown to not increase RORB flood peaks downstream up to and including the 1% AEP.

The preliminary design of the basin is estimated to store 53 ML to the spillway level, and 134 ML at crest level. Breaches were estimated and simulated for the critical duration (1-hour) 0.01% AEP and PMF events at the proposed Kenny basin. No significant affected area (increase in flood levels of greater than 300 mm due to breach) was produced by any of the breaches. With no Population at Risk or Potential Loss of Life within the affected area if a “Medium” Severity of Loss and Damage is assumed the basin is given a preliminary Consequence Category of “Low”. According to the NSW Dam Safety Committee guidance, DSC3B, a “Low” Consequence Category dam has an Acceptable Flood Capacity of between the 1% and 0.1% AEP flood.

Subsequently breach modelling was undertaken for the 1% AEP flood, as this is expected to have a greater incremental effect than the 0.01% AEP.

The possibility of a failure of the proposed Kenny Basin causing a cascade failure of the Southwell Park basin was not investigated in detail, as under NSW Dam Safety Committee requirements modelling and assessment is only required for areas where flood levels are increased by more than 300 mm in the event of a failure. With likely upgrades to Southwell Park the downstream basin is not expected to be sensitive to the increase in volume which would occur in the event of a failure of the Kenny Basin, but could be reassessed at that stage.

15. References

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Appendices

Appendix A – RORB Hydrology

RORB (*Laurenson et al. 2010*) is a non-linear rainfall runoff and streamflow routing model for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be subdivided into subareas, connected by a series of conceptual reach storages. Design storm rainfall is input to the centroid of each subarea. Specified losses are then deducted, and the excess routed through the reach network.

The model is also able to simulate:

- Lakes, retarding basins and similar storages
- Concentrated and/or distributed inflows and outflows.

Catchment layout is in the Appendix directory or by selecting the hyperlink below:
(Note: In the pdf use ALT Left-Arrow to return to the document location)

Appendix A-01. RORB Model: Catchment Layout

Appendix B - Flood Maps Existing Scenario

This information is provided on the basis that the ACT Chief Minister, Treasury and Economic Development Directorate and GHD do not accept responsibility for any loss or damage alleged to be suffered by anyone as a result of publication of these maps and the associated notations, or as a result of use or misuse of the information herein.

These maps are derived from hydrologic and hydraulic modelling which while detailed is a coarse simplification and approximation of real life. It involves the necessary adoption of a large number of assumptions and simplifications. As a consequence all modelling results require appropriate interpretation, with an understanding of the particular strengths and weaknesses inherent in the adopted modelling approach. Various methods are used to understand and make appropriately conservative allowances for the uncertainties which exist in both the hydrologic and hydraulic modelling processes including undertaking sensitivity analysis, risk analysis and applying appropriate freeboard to estimated levels.

1. Local flooding of other areas, or in excess of the extents shown, may occur. The extent of flooding shown relates to flooding from mapped reaches and does not generally include flooding along non mapped drains including council, private and other small drainage systems

2. Some parts of the flood extents shown were modelled solely for the purpose of estimating flows at locations of interest

3. Flood information shown relates only to floodwaters emanating from the catchments of the drainage systems shown, and does not include any allowance for flood events occurring within downstream waterways, or other streams or drains. In the lower reaches of most drainage systems, greater flood levels, depths and/or velocities than those shown may result from flood events on downstream waterways. Flood levels greater than those shown may occur:

   – Upstream of the upstream mapping limits of each mapped drain; or

   – Along other Council or private drains, either within or outside the extents of land liable to flooding shown.

4. The approximate extents of land liable to flooding have been based on survey data available at the time of preparation. The exact extent of flooding for individual properties can only be determined by a licensed surveyor.

5. Local increases in flood extents, levels, depths and/or velocities shown may result from local factors such as drain blockages and local obstructions to overland flows such as fences, buildings and cars.

6. One percent probability flood extents are expected to be reached during a flood event with a 1 in 100 chance of being equalled or exceeded in any one year (often referred to as a 100 year average recurrence interval flood) under the physical and topographic conditions prevailing at the time of preparation. This flood may occur more than once per year.

7. Ten percent probability flood extents shown are expected to be reached during a flood event with a 1 in 10 chance of being equalled or exceeded in any one year (this is similar but not identical to a 10 year average recurrence interval flood) under the physical and topographic conditions prevailing at the time of preparation. This flood may occur more than once per year.

8. Floods greater than the one percent flood can occur. During such floods an area greater than that shown would be inundated. Conversely, properties within the area shown can be affected by floods of lesser magnitude.

9. The smaller, more frequent events, eg 10 year ARI results, are produced using a hydraulic model established primarily for the purpose of modelling the 100 year ARI event, a fact which should be considered in their interpretation

10. The extents should be taken as indicative only. It is likely that the flooding experienced during a significant rainfall event may differ to that indicated on these maps

11. Cadastral boundaries, street names and drain locations are as supplied and have not been independently verified.

12. The modelling has been designed to produce representative flood extents associated with the mainstream flooding of the nominated waterways the extents shown in other areas may over or underestimate the actual flooding from a given event. For example:

13. Locations without flood extent may be liable to flooding from sources which have not been explicitly modelled

14. Portions of the mapped flood extent are produced from areas of the modelling designed to enable appropriate distribution of flows. Where this distribution is undertaken in the hydrologic (RORB) model these extents are not shown. Where this distribution is undertaken in the hydraulic (TUFLOW) model these extents are shown. These areas are identified by notes on the maps as “Inflow Polygons Off Main Channels” and “Pipes and 1D Elements Off Main Channels”. In smaller events in particular the extents shown in these areas may be excessive due to the incomplete modelling of underground drains. Likewise actual flooding in these areas may be larger than shown due to additional flooding sources which have not been explicitly modelled.

Plots are in the Appendix directory or by selecting the hyperlink below:
(Note: From the pdf document use ALT Left-Arrow to return to the main document location)

Appendix C – Inundation of Key Bridges/Culverts

C.1 Bridge Locations

The following plan shows the crossing locations referred to in Section 8.3 of the report.

Appendix C-01. Key Bridge & Culvert Locations

C.2 Inundation of key bridge and culvert structures

The following tables contain TUFLOW model results for the 10%, and1% AEP flood levels and velocities for upstream and downstream of each of the key bridge and culvert structures.

The full set of tables for the 10%, 1%, 0.2% 0.01% AEP and PMF events for key bridge and culvert structures, including level, velocity, duration of inundation, and stream gauge level are contained in:

Appendix C-02. Bridge inundation tables and listing

Appendix C2a. Key road and bridge inundation information 10% AEP (10 year ARI) – Level and Velocity

ID	Description	Channel Invert (mAHD)	Road Level (mAHD)	Channel	Type	10% AEP US Peak Flood Level (mAHD)	10% AEP DS Peak Flood Level (mAHD)	10% AEP US Peak Velocity (m/s)	10% AEP DS Peak Velocity (m/s)
S01	Parkes Way	552.7	562.0	Sullivans	Road	556.0	555.9	0.9	1.1
S02	Ward Footbridge	553.9	559.3	Sullivans	Road	557.4	557.4	1.1	1.0
S03	Fellows Road	556.0	560.5	Sullivans	Road	558.7	558.7	2.0	1.6
S04	ANU Biology	557.2	560.4	Sullivans	Pedestrian	559.8	559.5	2.4	3.8
S05	Union Footbridge	558.4	562.5	Sullivans	Road	560.4	560.3	1.9	2.3
S06	Barry Drive	560.5	563.2	Sullivans	Road	562.6	561.3	0.6	1.2
S07	Masson Street	561.3	565.0	Sullivans	Road	563.4	563.3	2.2	1.7
S08	Condamine Street	561.9	565.6	Sullivans	Road	564.2	564.2	3.0	1.4
S09	David Street	563.1	566.5	Sullivans	Road	565.2	565.0	1.9	3.0
S09-OC01	O'Connor Footbridge	564.9	567.2	O'Connor	Pedestrian	566.2	566.2	2.2	2.2
S09-OC02	O'Connor driveway	566.4	568.1	O'Connor	Road (driveway)	568.1	567.5	1.8	1.7
S09-OC03	O'Connor Macaurthur Road	567.9	569.7	O'Connor	Road	569.1	568.8	2.3	2.2
S09-OC04	Coolibah Footbridge	574.4	575.8	O'Connor	Pedestrian	575.3	575.2	4.1	4.3
S09-OC05	Miller Street	578.9	581.1	O'Connor	Road	580.9	580.2	2.1	3.5
S09-OC06	Bauhinia Street	582.8	584.1	O'Connor	Road	583.0	582.9	0.3	1.0
S10	Macarthur Avenue	563.7	567.3	Sullivans	Road	566.0	565.9	1.0	1.0
S11	Wattle Street	564.4	568.3	Sullivans	Road	566.6	566.5	3.9	1.3
S11-D01	Dickson Lyneham Footbridge	566.8	569.0	Dickson	Pedestrian	568.4	568.1	3.0	4.0
S11-D02	Deburgh Street	567.2	569.7	Dickson	Road	569.2	568.7	1.1	2.0
S11-D03	Northbourne Avenue	568.4	571.2	Dickson	Road	570.0	569.7	2.3	1.9
S11-D04	Challis Street	569.8	572.6	Dickson	Road	571.8	570.9	2.1	4.2
S11-D05	Cowper Street	574.2	576.9	Dickson	Road	576.3	575.6	2.2	5.0
S11-D06	Dumaresq Footbridge	575.3	577.1	Dickson	Pedestrian	576.9	576.9	6.0	5.3
S11-D07	Hawdon Footbridge	580.9	582.8	Dickson	Pedestrian	582.6	582.2	5.1	3.4
S11-D08	Dutton Footbridge	583.4	585.5	Dickson	Pedestrian	585.0	584.9	4.5	4.7
S11-D09	Majura Avenue DS Footbridge	589.1	591.3	Dickson	Pedestrian	590.2	590.3	5.2	4.4
S11-D10	Majura Avenue	591.0	593.6	Dickson	Road	593.0	592.7	2.0	4.4
S11-D11	Duffy Footbridge	596.4	598.2	Dickson	Pedestrian	597.6	597.5	6.8	7.0
S11-D12	Phillip Avenue Footbridge	610.1	611.2	Dickson	Pedestrian	610.3	610.0	3.3	4.1

ID	Description	Channel Invert (mAHD)	Road Level (mAHD)	Channel	Type	10% AEP US Peak Flood Level (mAHD)	10% AEP DS Peak Flood Level (mAHD)	10% AEP US Peak Velocity (m/s)	10% AEP DS Peak Velocity (m/s)
S12	Goodwin Street	565.1	568.3	Sullivans	Road	566.9	566.8	1.3	1.0
S13	Fox Place Footbridge	565.8	567.9	Sullivans	Pedestrian	567.7	567.6	2.1	2.0
S14	Mouat Street	567.1	570.7	Sullivans	Road	569.1	568.7	2.0	2.0
S15	Thurbon Road	568.0	570.3	Sullivans	Road	570.7	570.5	1.5	1.5
S16	Yowani CC FP1	569.2	571.0	Sullivans	Pedestrian	570.9	570.8	1.7	2.2
S17	Yowani CC FP2	569.3	571.1	Sullivans	Pedestrian	570.9	570.9	2.0	1.9
S18	Yowani CC FP3	569.7	572.8	Sullivans	Pedestrian	571.4	571.2	0.8	0.6
S19	Barton Highway	570.0	574.6	Sullivans	Road	571.7	571.7	1.8	1.1
S20	Federal Footbridge	573.3	575.9	Sullivans	Pedestrian	574.8	574.7	0.7	0.5
S21	Randwick Road	576.9	579.9	Sullivans	Road	578.3	578.3	1.7	2.5
S21-M01	Sandford Overpass	581.3	584.3	Mitchell	Road	583.5	582.7	1.4	1.8
S21-M02	Winchcombe Road Overpass	582.7	585.3	Mitchell	Road	584.9	584.5	1.2	1.3
S21-M03	Heffernan Overpass	583.0	585.8	Mitchell	Road	585.2	584.9	1.6	1.9
S21-M04	Ballieu Overpass	583.8	586.6	Mitchell	Road	585.7	585.5	1.3	4.5
S21-M05	Lyasaght Overpass	584.9	587.7	Mitchell	Road	586.7	586.5	1.4	2.7
S21-M06	Callan Overpass	586.3	589.2	Mitchell	Road	587.6	587.0	0.5	0.6
S21-M07	Vicars Overpass	592.6	595.1	Mitchell	Road	593.9	0.0	0.7	0.0
S21-M08	Well Station Drive Overpass	595.2	598.5	Mitchell	Road	597.2	594.5	0.3	1.5
S22	Flemington Road	577.2	581.2	Sullivans	Road	578.5	578.5	2.4	3.4
S23	Flemington Road DS Morrisset Pond	0.0	580.5	Sullivans	Road	581.6	579.5	0.1	2.3
S24	Morisset Road	582.0	585.2	Sullivans	Road	582.3	582.2	1.4	0.6
S24-01	Morisset Road (detention centre)	588.7	589.7	Off Sullivans	Road	588.4	588.9	0.3	1.0
S25	Old Well Station 2	586.3	586.5	Sullivans	Road	586.6	586.4	0.2	0.5
S26	Old Well Station 1	587.0	588.3	Sullivans	Road	588.3	587.4	1.3	0.7
S27	Horse Park Drive	605.2	609.5	Sullivans	Road	606.3	606.3	0.6	1.6

*Note: Flood maps are based on the 3m grid model which indicates flooding. However, it is only marginal flooding, and does not flood in the 5m model

Appendix C2b. Key road and bridge inundation information 1% AEP-(100 year ARI) Level and Velocity

ID	Description	Channel Invert (mAHD)	Road Level (mAHD)	Channel	Type	1% AEP US Peak Flood Level (mAHD)	1% AEP DS Peak Flood Level (mAHD)	1% AEP US Peak Velocity (m/s)	1% AEP DS Peak Velocity (m/s)
S01	Parkes Way	552.7	562.0	Sullivans	Road	556.0	555.9	1.8	2.2
S02	Ward Footbridge	553.9	559.3	Sullivans	Road	558.0	558.0	1.6	1.5
S03	Fellows Road	556.0	560.5	Sullivans	Road	559.1	559.1	3.4	2.7
S04	ANU Biology	557.2	560.4	Sullivans	Pedestrian	561.0	560.4	2.6	4.4
S05	Union Footbridge	558.4	562.5	Sullivans	Road	561.4	561.3	2.1	2.5
S06	Barry Drive	560.5	563.2	Sullivans	Road	563.4	562.5	1.1	4.3
S07	Masson Street	561.3	565.0	Sullivans	Road	563.7	563.6	2.8	2.9
S08	Condamine Street	561.9	565.6	Sullivans	Road	564.8	564.8	3.0	1.6
S09	David Street	563.1	566.5	Sullivans	Road	565.6	565.4	1.7	3.0
S09-OC01	O’Connor Footbridge	564.9	567.2	O’Connor	Pedestrian	566.6	566.5	2.3	2.3
S09-OC02	O’Connor driveway	566.4	568.1	O’Connor	Road (driveway)	568.4	567.8	1.8	2.1
S09-OC03	O’Connor Macaurthur Road	567.9	569.7	O’Connor	Road	569.6	568.9	2.2	1.8
S09-OC04	Coolibah Footbridge	574.4	575.8	O’Connor	Pedestrian	576.2	575.8	4.4	4.4
S09-OC05	Miller Street	578.9	581.1	O’Connor	Road	581.3	580.5	2.1	3.9
S09-OC06	Bauhinia Street	582.8	584.1	O’Connor	Road	583.1	583.1	0.4	1.9
S10	Macarthur Avenue	563.7	567.3	Sullivans	Road	566.7	566.3	1.1	1.3
S11	Wattle Street	564.4	568.3	Sullivans	Road	567.2	567.0	3.8	1.6
S11-D01	Dickson Lyneham Footbridge	566.8	569.0	Dickson	Pedestrian	568.8	568.3	3.1	4.2
S11-D02	Deburgh Street	567.2	569.7	Dickson	Road	569.7	569.1	1.3	3.6
S11-D03	Northbourne Avenue	568.4	571.2	Dickson	Road	570.6	570.0	2.7	1.8
S11-D04	Challis Street	569.8	572.6	Dickson	Road	572.6	572.6	2.1	4.8
S11-D05	Cowper Street	574.2	576.9	Dickson	Road	576.9	576.0	1.9	5.5
S11-D06	Dumaresq Footbridge	575.3	577.1	Dickson	Pedestrian	577.3	577.1	5.5	5.5
S11-D07	Hawdon Footbridge	580.9	582.8	Dickson	Pedestrian	583.1	582.5	5.6	3.8
S11-D08	Dutton Footbridge	583.4	585.5	Dickson	Pedestrian	585.4	585.1	4.5	4.6
S11-D09	Majura Avenue DS Footbridge	589.1	591.3	Dickson	Pedestrian	590.8	590.7	6.0	4.4
S11-D10	Majura Avenue	591.0	593.6	Dickson	Road	593.6	593.0	2.2	4.3
S11-D11	Duffy Footbridge	596.4	598.2	Dickson	Pedestrian	598.4	597.8	6.6	7.5
S11-D12	Phillip Avenue Footbridge	610.1	611.2	Dickson	Pedestrian	611.1	610.8	7.0	8.6
S12	Goodwin Street	565.1	568.3	Sullivans	Road	567.7	567.4	1.4	1.4

ID	Description	Channel Invert (mAHD)	Road Level (mAHD)	Channel	Type	1% AEP US Peak Flood Level (mAHD)	1% AEP DS Peak Flood Level (mAHD)	1% AEP US Peak Velocity (m/s)	1% AEP DS Peak Velocity (m/s)
S13	Fox Place Footbridge	565.8	567.9	Sullivans	Pedestrian	568.1	568.0	1.9	2.3
S14	Mouat Street	567.1	570.7	Sullivans	Road	570.8	569.1	2.1	2.3
S15	Thurbon Road	568.0	570.3	Sullivans	Road	571.4	571.1	1.5	2.0
S16	Yowani CC FP1	569.2	571.0	Sullivans	Pedestrian	571.5	571.4	2.0	3.0
S17	Yowani CC FP2	569.3	571.1	Sullivans	Pedestrian	571.5	571.5	2.3	2.2
S18	Yowani CC FP3	569.7	572.8	Sullivans	Pedestrian	572.3	571.8	0.8	0.8
S19	Barton Highway	570.0	574.6	Sullivans	Road	572.7	572.6	2.0	1.4
S20	Federal Footbridge	573.3	575.9	Sullivans	Pedestrian	575.7	575.4	0.7	0.5
S21	Randwick Road	576.9	579.9	Sullivans	Road	578.9	578.9	1.8	2.5
S21-M01	Sandford Overpass	581.3	584.3	Mitchell	Road	584.1	582.9	1.8	2.4
S21-M02	Winchcombe Road Overpass	582.7	585.3	Mitchell	Road	585.4	584.8	1.3	1.3
S21-M03	Heffernan Overpass	583.0	585.8	Mitchell	Road	586.0	585.4	1.9	1.9
S21-M04	Ballieu Overpass	583.8	586.6	Mitchell	Road	586.5	586.1	1.4	4.1
S21-M05	Lyasaght Overpass	584.9	587.7	Mitchell	Road	587.6	586.8	1.6	2.9
S21-M06	Callan Overpass	586.3	589.2	Mitchell	Road	588.0	587.7	1.3	0.6
S21-M07	Vicars Overpass	592.6	595.1	Mitchell	Road	594.5	0.0	0.7	0.0
S21-M08	Well Station Drive Overpass	595.2	598.5	Mitchell	Road	597.7	594.7	0.5	2.0
S22	Flemington Road	577.2	581.2	Sullivans	Road	579.3	579.3	4.7	4.0
S23	Flemington Road DS Morrisset Pond	0.0	580.5	Sullivans	Road	581.7	579.9	0.3	2.3
S24	Morisset Road	582.0	585.2	Sullivans	Road	582.6	582.6	1.6	1.0
S24-01	Morisset Road (detention centre)	588.7	589.7	Off Sullivans	Road	588.6	589.1	0.6	1.5
S25	Old Well Station 2	586.3	586.5	Sullivans	Road	586.7	586.6	0.5	0.7
S26	Old Well Station 1	587.0	588.3	Sullivans	Road	588.4	587.9	1.4	0.8
S27	Horse Park Drive	605.2	609.5	Sullivans	Road	606.8	606.8	0.7	1.7

Appendix D - Sensitivity Assessment Maps

To be read in conjunction with notes from Appendix B.

Plots are in the Appendix directory or by selecting the hyperlink below:
(Note: From the pdf document use ALT Left-Arrow to return to the main document location)

Appendix D-01. Sullivans Creek Existing Catchment Flood Extent: 10% AEP (10y ARI) Climate Change 30% Scenario

Appendix D-02a. Sullivans Creek Existing Catchment Flood Extent: 10% AEP (10y ARI) High Roughness Scenario

Appendix D-02b. Sullivans Creek Existing Catchment Flood Extent: 10% AEP (10y ARI) Low Roughness Scenario

Appendix D-03a. Sullivans Creek Existing Catchment Flood Extent: 10% AEP (10y ARI) Blockage Scenario

Appendix D-03b. Existing Catchment Flood Extent: 10% AEP Southwell Park 50% Blocked

Appendix D-03c. Existing Catchment Flood Extent: 10% AEP Majura Ave 75% Blocked

Appendix D-04a. Sullivans Creek Existing Catchment Flood Extent: 10% AEP (10y ARI) 1m Increase Lake Level Scenario

Appendix D-04b. Sullivans Creek Existing Catchment Flood Extent: 10% AEP (10y ARI) 0.5m Increase Lake Level Scenario

Appendix D-04c. Existing Catchment Flood Extent: 10% AEP 1m Increase Lake Level

Appendix D-04d. Existing Catchment Flood Extent: 10% AEP 0.5m Increase Lake Level

Appendix D-05. Sullivans Creek Existing Catchment Flood Extent: 1% AEP (100y ARI) Climate Change 30% Scenario

Appendix D-06a. Sullivans Creek Existing Catchment Flood Extent: 1% AEP (100y ARI) High Roughness Scenario

Appendix D-06b. Sullivans Creek Existing Catchment Flood Extent: 1% AEP (100y ARI) Low Roughness Scenario

Appendix D-07a. Sullivans Creek Existing Catchment Flood Extent: 1% AEP (100y ARI) Blockage Scenario

Appendix D-07b. Existing Catchment Flood Extent: 1% AEP Southwell Park 50% Blocked

Appendix D-07c. Existing Catchment Flood Extent 1% AEP Majura Ave 75% Blocked

Appendix D-08a. Sullivans Creek Existing Catchment Flood Extent: 1% AEP (100y ARI) 1m Increase Lake Level Scenario

Appendix D-08b. Sullivans Creek Existing Catchment Flood Extent: 1% AEP (100y ARI) 0.5m Increase Lake Level Scenario

Appendix D-08c. Existing Catchment Flood Extent: 1% AEP 1m Increase Lake Level

Appendix D-08d. Existing Catchment Flood Extent: 1% AEP 0.5m Increase Lake Level

Appendix E – TUFLOW Model Comparisons

E.1 Rating table comparison at gauges locations

An understanding of how well the TUFLOW model is performing relative to other models and gauged information can be gained by taking the results at the established gauge locations and plotting them against the recognised rating curves. Graphs detailing this comparison for both Southwell Park and Barry Drive are presented on the following pages.

Results are shown for both the 3m and 5m TUFLOW grids obtained from the 100 year ARI 3 hour simulation and also for the 3 hour PMF simulation. To enable presentation of the large flows from the PMF run as well as comparison with the relatively frequent and smaller flows the graphs are produced on a log scale and a truncated natural scale respectively.

The TUFLOW results for both the 3m and 5m grids at Southwell Park indicate a more restricted basin outlet than the established rating curve for the Ecowise gauge and the 1980s-1990s gaugings. In this regard the results are more consistent with the recent modelling by Cardno. This may represent a systematic modelling bias (both Cardno and GHD modelling used TUFLOW although with independent representations), it is also feasible that there has been a change in the physical system and or a bias in the gauge measurements.

The TUFLOW results for Barry Drive are generally a good match with the established Ecowise gauge rating curve, noting that the small hysteresis (the larger flows occurring at lower levels on the rising limb of the hydrograph) evident in the hydraulic modelling results are not included in the Ecowise gauge rating curve.

Appendix E-01. Ratingt table comparison

E.2 Comparison with HECRAS model

The existing HECRAS models created by Ecowise in 2001 were spatially referenced to assist in the comparison with the current investigation as per the image in Figure E-1 below.

Comparisons where undertaken at all locations where some reasonable comparison could be made between the HECRAS model and the current TUFLOW model. TUFLOW results for both the 3m and 5m grids were produced for comparison using the 3 hour 100 year ARI results and the 3 hour PMF results respectively.

The relative locations of the TUFLOW Print Out lines and the HECRAS cross section lines are provided in the following map set Appendix E- 02a.

Following this map set are rating curves presented in order along Sullivans Creek followed by Dickson and then O’Connor Channels (Appendix E-02b to Appendix E- 02d figures).

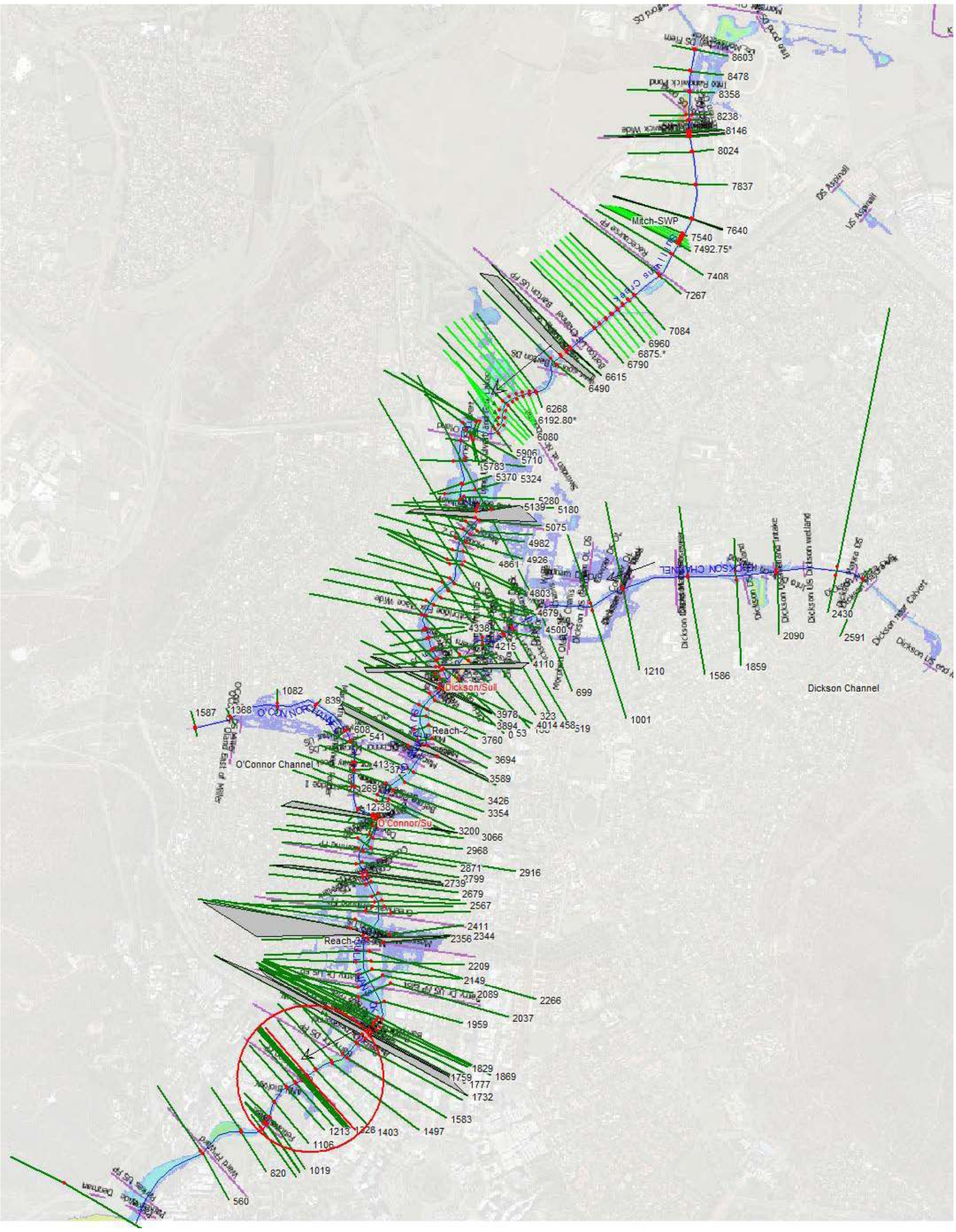

Figure E-1 Existing HECRAS model comparison

Appendix E-02a. HECRAS & TUFLOW comparison locations

Appendix E-02b. Sullivans Creek rating tables: HECRAS, TUFLOW 3m & TUFLOW 5m

Appendix E-02c. Dickson Channel rating tables: HECRAS, TUFLOW 3m & TUFLOW 5m

Appendix E-02d. O'Connor Channel rating tables: HECRAS, TUFLOW 3m & TUFLOW 5m

Appendix F – Flood Maps Developed Scenario

To be read in conjuction with Appendix B.

Plots are in the Appendix directory of the electronic report data files supplied, or by selecting the hyperlink below:
(Note: From the pdf document use ALT Left-Arrow to return to the main document location)

Appendix F-00a. Kenny proposed

Appendix F-00b. Throsby development

Appendix F-00c. Kenny proposed impervious fractions

Appendix F-01. Sullivans Creek Developed Catchment Flood Extent: 10% AEP (10y ARI) Depth & Levels

Appendix F-02. Sullivans Creek Developed Catchment Flood Extent: 10% AEP (10y ARI) Velocities

Appendix F-03. Sullivans Creek Developed Catchment Flood Extent: 10% AEP (10y ARI) Hazard Category

Appendix F-04. Sullivans Creek Developed Catchment Flood Extent: 1% AEP (100y ARI) Depth & Levels

Appendix F-05. Sullivans Creek Developed Catchment Flood Extent: 1% AEP (100y ARI) Velocities

Appendix F-06. Sullivans Creek Developed Catchment Flood Extent: 1% AEP (100y ARI) Hazard Category

Appendix F-07. Sullivans Creek Developed Catchment Flood Extent: 0.2% AEP (500y ARI) Depth & Levels

Appendix F-08. Sullivans Creek Developed Catchment Flood Extent: 0.2% AEP (500y ARI) Velocities

Appendix F-09. Sullivans Creek Developed Catchment Flood Extent: 0.2% AEP (500y ARI) Hazard Category

Appendix F-10. Sullivans Creek Developed Catchment Flood Extent: 0.01% AEP (10000y ARI) Depth & Levels

Appendix F-11. Sullivans Creek Developed Catchment Flood Extent: 0.01% AEP (10000y ARI) Velocities

Appendix F-12. Sullivans Creek Developed Catchment Flood Extent: 0.01% AEP (10000y ARI) Hazard Category

Appendix F-13. Sullivans Creek Developed Catchment Flood Extent: PMF Depth & Levels

Appendix F-14. Sullivans Creek Developed Catchment Flood Extent: PMF Velocities

Appendix F-15. Sullivans Creek Developed Catchment Flood Extent: PMF Hazard Category

Appendix G – Southwell Park breach modelling

Table G-1 Southwell PMF breach parameters (overtopping). (Base at 570.3 m AHD)

Breach formation time (min)	Breach base width (m)	Time breach assumed to occur (hours from start of event)	Estimated flood level in basin when breach occurs (m AHD)	Method	Empirical equation predicted breach peak m^3/s excluding simultaneous inflow	Comment
39	250	1.38	574.17	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	666-2187	
20	338	1.06	573.77	Macdonald-Langridge Monopolis (after Wahl	590-1939	
5	6	0.82	572.99	Bureau of reclamation	119	
11	11	0.92	573.27	Von Thun Gillette		
93	20	2.28	573.12	Froehlich	118	After peak

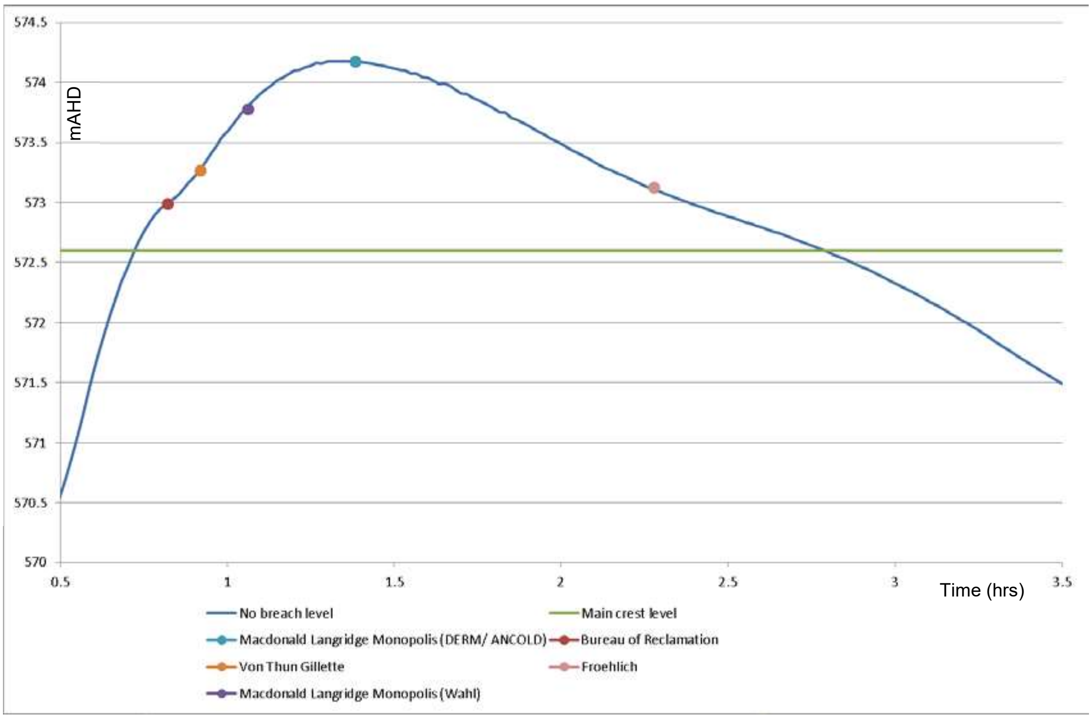

Figure G-15-1 Breach points along the PMF hydrograph using different methods

Table G-2 Southwell 0.01% AEP breach parameters (overtopping). (Base at 570.3 m AHD)

Breach formation time (min)	Breach base width (m)	Time breach assumed to occur (hours from start of event)	Estimated flood level in basin when breach occurs (m AHD)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
29	113	1.98	572.99	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	436-1433	Breach size ignores flow through failed fuse-plug.
16	181	1.76	572.93	Macdonald-Langridge Monopolis (after Wahl)	423-1392	Breach size ignores flow through failed fuse-plug.
24	61	1.91	572.99	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	332-1093	Proportion of volume flowing through breach and fuse-plug based on relative sizes.
5	5	1.58	572.77	Bureau of reclamation	102	
11	10	1.68	572.85	Von Thun Gilette		
75	18	2.775	572.65	Froehlich	84	After peak

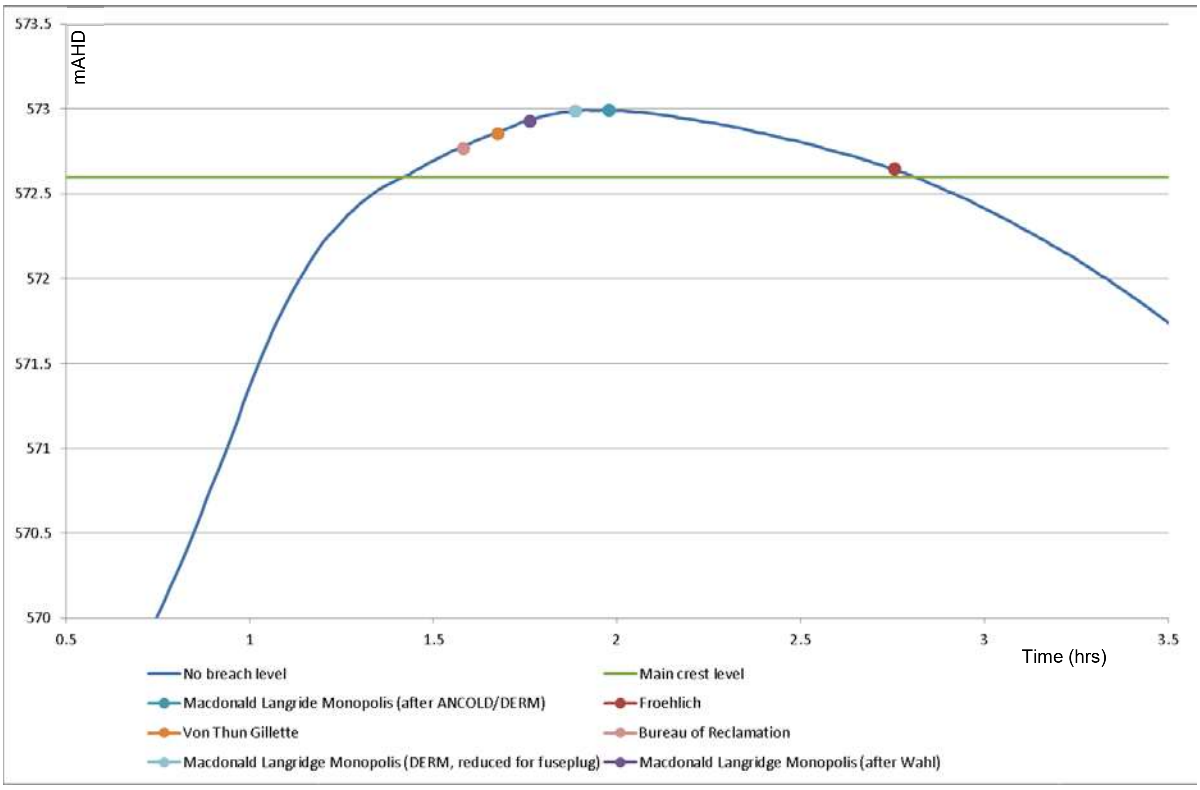

Figure G-15-2 Breach points along the 0.01% AEP hydrograph using different methods.

Table G-3 Southwell 0.4% AEP breach parameters (piping). (Base at 570.3 m AHD)

Breach formation time (min)	Breach base width (m)	Method	Empirical equation predicted breach peak m^3/s excluding simultaneous inflow	Comment
22	49	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	279-914	
10	85	Macdonald-Langridge Monopolis (after Wahl)	279-.914	
4	4	Bureau of reclamation	63	
8	8	Von Thun Gilette		
60	10	Froehlich	57	

Flood maps - Southwell Park breach modelling

(0.4% AEP or 1 in 250 yr ARI, and 0.01% AEPor 1 in 10,000 yr ARI)

Plots are in the Appendix directory or by selecting the hyperlink below:
(Note: From the pdf document use ALT Left-Arrow to return to the main document location)

Appendix G-01. Southwell Park breach modelling: Full no breach extent (0.4% AEP)

Appendix G-02. Southwell Park breach modelling: 0.4% AEP reporting points

Appendix G-03a. Southwell Park breach modelling: 0.4% AEP depth without breach

Appendix G-03b. Southwell Park breach modelling: 0.4% AEP velocity without breach

Appendix G-03c. Southwell Park breach modelling: 0.4% AEP VxD without breach

Appendix G-04a. Southwell Park breach modelling: 0.4% AEP depth before breach

Appendix G-04b. Southwell Park breach modelling: 0.4% AEP velocity before breach

Appendix G-04c. Southwell Park breach modelling: 0.4% AEP VxD before breach

Appendix G-05a. Southwell Park breach modelling: 0.4% AEP depth 22 min breach

Appendix G-05b. Southwell Park Breach Modelling: 0.4% AEP velocity 22 min breach

Appendix G-05c. Southwell Park breach modelling: 0.4% AEP VxD 22 min breach

Appendix G-06a. Southwell Park breach modelling: 0.4% AEP depth 17 min breach

Appendix G-06b. Southwell Park breach modelling: 0.4% AEP velocity 17 min breach

Appendix G-06c. Southwell Park breach modelling: 0.4% AEP VxD 17 min breach

Appendix G-07. Southwell Park breach modelling: Full no breach extent (0.01% AEP)

Appendix G-08. Southwell Park breach modelling: 0.01% AEP reporting points

Appendix G-09a. Southwell Park breach modelling: 0.01% AEP depth without breach

Appendix G-09b. Southwell Park breach modelling: 0.01% AEP velocity without breach

Appendix G-09c. Southwell Park breach modelling: 0.01% AEP VxD without breach

Appendix G-10a. Southwell Park breach modelling: 0.01% AEP pre 24 min breach depth

Appendix G-10b. Southwell Park breach modelling: 0.01% AEP pre 24 min breach velocity

Appendix G-10c. Southwell Park breach modelling: 0.01% AEP pre 24 min breach VxD

Appendix G-11a. Southwell Park breach modelling: 0.01% AEP pre 17 min breach depth

Appendix G-11b. Southwell Park breach modelling: 0.01% AEP pre 17 min breach velocity

Appendix G-11c. Southwell Park breach modelling: 0.01% AEP pre 17 min breach VxD

Appendix G-12a. Southwell Park breach modelling: 0.01% AEP 24 min breach depth

Appendix G-12b. Southwell Park breach modelling: 0.01% AEP 24 min breach velocity

Appendix G-12c. Southwell Park breach modelling: 0.01% AEP 24 min breach VxD

Appendix G-13a. Southwell Park breach modelling: 0.01% AEP 17 min breach depth

Appendix G-13b. Southwell Park breach modelling: 0.01% AEP 17 min breach velocity

Appendix G-13c. Southwell Park breach modelling: 0.01% AEP 17 min breach VxD

Appendix G-14. Southwell Park breach modelling: Full no breach extent (Southwell PMF)

Hydrographs

Appendix G-15. DCF Hydrographs – Flow in m^3/s

Appendix G-16. 10,000y Hydrographs- Level in mAHD

The maximum depths and velocities may occur at different parts of the building, and thus the actual instantaneous maximum velocity depth product may not be equal to the product of the maximum depth and the maximum velocity.

Appendix G17a Buildings within inundated area affected by failure of basin embankment (0.4% AEP flood event with 21 m 17 min breach)

			Without Dambreak			With Dambreak			Pre Dambreak			
Building reference	Address	Assumed floor level	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Incremental depth
1	39 MOUAT STREET	570.96	-	0.00	0.00	0.06	0.20	0.02	-	0.00	0.00	-
2	39 MOUAT STREET	571.12	-	0.00	0.00	0.04	0.07	0.03	-	0.00	0.00	-
3	39 MOUAT STREET	571.08	-	0.00	0.00	0.17	0.68	0.20	-	0.00	0.00	-
4	39 MOUAT STREET	570.76	-	0.00	0.00	0.22	0.29	0.06	-	0.00	0.00	-
7	39 MOUAT STREET	571.22	-	0.00	0.00	0.31	0.59	0.14	-	0.00	0.00	-
8	39 MOUAT STREET	571.10	-	0.00	0.00	0.28	0.20	0.07	-	0.00	0.00	-
9	150 BRIGALOW STREET	570.46	-	0.00	0.00	0.65	0.75	0.10	-	0.00	0.00	-
11	136 BRIGALOW STREET	570.85	-	0.00	0.00	0.07	0.40	0.19	-	0.00	0.00	-
220	136 BRIGALOW STREET	570.77	-	0.00	0.00	0.12	0.43	0.18	-	0.00	0.00	-
221	136 BRIGALOW STREET	570.93	-	0.00	0.00	0.14	0.67	0.44	-	0.00	0.00	-
222	136 BRIGALOW STREET	570.80	-	0.00	0.00	0.16	0.87	0.16	-	0.00	0.00	-

Appendix G17b Buildings within inundated area affected by failure of basin embankment (0.4% AEP flood event with 49 m 22 min breach)

			Without Dambreak			With Dambreak			Pre Dambreak			
Building reference	Address	Assumed floor level	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Incremental depth
1	39 MOUAT STREET	570.96	-	0.00	0.00	0.45	0.68	0.17	-	0.00	0.00	-
2	39 MOUAT STREET	571.12	-	0.00	0.00	0.22	0.31	0.15	-	0.00	0.00	-
3	39 MOUAT STREET	571.08	-	0.00	0.00	0.36	0.67	0.27	-	0.00	0.00	-
4	39 MOUAT STREET	570.76	-1.04	0.03	0.00	0.34	0.29	0.08	-	0.00	0.00	1.39
7	39 MOUAT STREET	571.22	-	0.00	0.00	0.55	0.83	0.29	-	0.00	0.00	-
8	39 MOUAT STREET	571.10	-0.59	1.02	0.05	0.62	0.88	0.20	-0.95	0.03	0.00	1.21
9	150 BRIGALOW STREET	570.46	-	0.00	0.00	0.81	0.75	0.16	-	0.00	0.00	-
11	136 BRIGALOW STREET	570.85	-	0.00	0.00	0.22	1.03	0.24	-	0.00	0.00	-
14	136 BRIGALOW STREET	570.34	-	0.00	0.00	0.18	0.15	0.06	-	0.00	0.00	-
113	141 BRIGALOW STREET	571.10	-	0.00	0.00	0.00	0.14	0.06	-	0.00	0.00	-
220	136 BRIGALOW STREET	570.77	-1.06	0.02	0.00	0.33	0.83	0.27	-	0.00	0.00	1.39
221	136 BRIGALOW STREET	570.93	-	0.00	0.00	0.30	0.62	0.21	-	0.00	0.00	-
222	136 BRIGALOW STREET	570.80	-	0.00	0.00	0.26	1.04	0.25	-	0.00	0.00	-

Appendix G17c Buildings within inundated area affected by failure of basin embankment (0.01% AEP-flood event with 21 m 17 min breach)

			Without Dambreak			With Dambreak			Pre Dambreak			
Building reference	Address	Assumed floor level	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Incremental depth
1	39 MOUAT STREET	570.96	0.44	0.49	0.07	0.78	0.50	0.17	0.20	0.49	0.04	0.33
2	39 MOUAT STREET	571.12	0.30	0.20	0.04	0.65	0.29	0.11	-0.23	0.11	0.02	0.35
3	39 MOUAT STREET	571.08	0.34	0.48	0.14	0.69	0.50	0.20	0.07	0.48	0.09	0.35
7	39 MOUAT STREET	571.22	0.35	0.38	0.19	0.76	0.58	0.31	0.15	0.25	0.19	0.41
8	39 MOUAT STREET	571.10	0.43	0.17	0.11	0.92	0.49	0.37	0.25	0.13	0.07	0.50
9	150 BRIGALOW STREET	570.46	0.94	0.21	0.14	1.27	0.23	0.26	0.09	0.57	0.04	0.33
85	175 MOUAT STREET	571.93	-	0.00	0.00	0.09	0.35	0.16	-	0.00	0.00	-
86	173 BRIGALOW STREET	571.93	-	0.00	0.00	0.08	0.19	0.12	-	0.00	0.00	-
87	108 LEWIN STREET	571.92	-	0.00	0.00	0.04	0.10	0.05	-	0.00	0.00	-
90	169 BRIGALOW STREET	571.74	-0.07	0.44	0.06	0.23	0.44	0.26	-0.33	0.38	0.03	0.30
92	167 BRIGALOW STREET	571.55	0.10	0.12	0.05	0.41	0.38	0.07	-0.31	0.07	0.00	0.31
93	165 BRIGALOW STREET	571.55	0.07	0.56	0.10	0.40	0.52	0.27	-0.46	0.01	0.00	0.33
95	163 BRIGALOW STREET	571.62	-0.05	0.07	0.01	0.30	0.26	0.05	-	0.00	0.00	0.35
96	161 BRIGALOW STREET	571.62	-0.07	0.32	0.03	0.29	0.46	0.09	-0.68	0.01	0.00	0.36
97	161 BRIGALOW STREET	571.64	-0.09	0.11	0.02	0.24	0.40	0.17	-	0.00	0.00	0.33
99	157 BRIGALOW STREET	571.39	0.07	0.16	0.07	0.42	0.64	0.12	-	0.00	0.00	0.35
100	159 BRIGALOW STREET	571.39	0.10	0.35	0.16	0.46	0.60	0.31	-	0.00	0.00	0.36
103	149 BRIGALOW STREET	571.44	-0.10	0.17	0.04	0.20	0.23	0.12	-	0.00	0.00	0.30
104	147 BRIGALOW STREET	571.44	-	0.00	0.00	0.13	0.21	0.10	-	0.00	0.00	-
105	86 LEWIN STREET	571.55	-0.31	0.06	0.02	0.01	0.14	0.05	-	0.00	0.00	0.32
106	88 LEWIN STREET	571.55	-0.30	0.36	0.04	0.03	0.35	0.08	-	0.00	0.00	0.33
107	82 LEWIN STREET	571.41	-0.22	0.07	0.03	0.09	0.11	0.08	-	0.00	0.00	0.31
108	84 LEWIN STREET	571.41	-0.21	0.25	0.03	0.11	0.53	0.05	-	0.00	0.00	0.32
109	145 BRIGALOW STREET	571.09	0.14	0.36	0.03	0.44	0.47	0.07	-	0.00	0.00	0.30
111	78 LEWIN STREET	571.43	-	0.00	0.00	0.04	0.11	0.08	-	0.00	0.00	-
112	80 LEWIN STREET	571.43	-0.25	0.06	0.01	0.06	0.15	0.09	-	0.00	0.00	0.31
152	33 LEWIN STREET	571.31	-0.18	0.06	0.01	0.12	0.18	0.07	-	0.00	0.00	0.30
153	31 LEWIN STREET	571.31	-0.19	0.02	0.00	0.12	0.03	0.01	-	0.00	0.00	0.31
155	35 LEWIN STREET	571.25	-	0.00	0.00	0.20	0.38	0.22	-	0.00	0.00	-
156	37 LEWIN STREET	571.30	-0.14	0.05	0.01	0.17	0.13	0.03	-	0.00	0.00	0.31
159	39 LEWIN STREET	571.47	-0.31	0.06	0.01	0.02	0.46	0.12	-	0.00	0.00	0.33
163	41 LEWIN STREET	571.44	-0.27	0.02	0.00	0.07	0.21	0.03	-	0.00	0.00	0.34
265	3 BOYD STREET	571.60	-0.19	0.17	0.02	0.14	0.45	0.33	-	0.00	0.00	0.33
267	1 BOYD STREET	571.60	-0.19	0.24	0.01	0.13	0.28	0.04	-	0.00	0.00	0.33
269	2 BOYD STREET	571.44	-0.10	0.29	0.03	0.21	0.44	0.06	-	0.00	0.00	0.30
271	4 BOYD STREET	571.44	-0.08	0.10	0.04	0.22	0.24	0.16	-	0.00	0.00	0.30

Appendix G17d Buildings within inundated area affected by failure of basin embankment (0.01% AEP-flood event with 61 m 24 min breach)

			Without Dambreak			With Dambreak			Pre Dambreak			
Building reference	Address	Assumed floor level	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Incremental depth
1	39 MOUAT STREET	570.96	0.44	0.49	0.07	0.92	0.50	0.24	0.33	0.49	0.05	0.48
2	39 MOUAT STREET	571.12	0.30	0.28	0.15	0.77	0.37	0.17	-0.01	0.21	0.03	0.47
3	39 MOUAT STREET	571.08	0.34	0.48	0.15	0.83	0.51	0.24	0.21	0.48	0.15	0.49
7	39 MOUAT STREET	571.22	0.35	0.38	0.19	0.98	0.52	0.40	0.29	0.51	0.19	0.63
8	39 MOUAT STREET	571.10	0.43	0.80	0.40	1.26	1.85	1.79	0.37	0.69	0.35	0.83
9	150 BRIGALOW STREET	570.46	0.94	0.21	0.14	1.39	0.24	0.33	0.49	0.21	0.10	0.45
11	136 BRIGALOW STREET	570.85	0.34	0.81	0.22	0.65	0.81	0.28	-0.27	0.04	0.01	0.32
13	136 BRIGALOW STREET	571.21	0.06	0.41	0.11	0.41	0.64	0.41	-0.30	0.30	0.11	0.35
82	179 MOUAT STREET	571.91	0.02	0.21	0.04	0.33	0.50	0.14	-	0.00	0.00	0.31
83	177 MOUAT STREET	571.91	0.01	0.24	0.06	0.33	0.66	0.28	-0.11	0.12	0.06	0.32
84	110 LEWIN STREET	572.08	-0.24	0.14	0.02	0.07	0.17	0.10	-0.51	0.05	0.01	0.31
85	175 MOUAT STREET	571.93	-0.10	0.28	0.06	0.29	0.99	0.56	-0.17	0.10	0.04	0.40
86	173 BRIGALOW STREET	571.93	-0.18	0.14	0.04	0.27	0.96	0.57	-0.36	0.10	0.03	0.45
87	108 LEWIN STREET	571.92	-0.19	0.09	0.03	0.19	0.37	0.23	-	0.00	0.00	0.39
90	169 BRIGALOW STREET	571.74	-0.05	0.44	0.06	0.41	0.56	0.34	-0.12	0.39	0.05	0.46
91	171 BRIGALOW STREET	571.74	-0.03	0.16	0.03	0.42	0.71	0.25	-0.11	0.15	0.03	0.45
92	167 BRIGALOW STREET	571.55	0.10	0.12	0.05	0.57	0.53	0.12	-0.05	0.10	0.01	0.48
93	165 BRIGALOW STREET	571.55	0.07	0.56	0.10	0.56	0.59	0.32	-0.11	0.34	0.04	0.49
94	102 LEWIN STREET	572.01	-0.43	0.07	0.01	0.02	0.38	0.13	-	0.00	0.00	0.45
95	163 BRIGALOW STREET	571.62	-0.05	0.07	0.01	0.45	0.37	0.11	-0.25	0.08	0.01	0.51
96	161 BRIGALOW STREET	571.62	-0.07	0.32	0.03	0.43	0.61	0.16	-0.26	0.28	0.03	0.50
97	161 BRIGALOW STREET	571.64	-0.09	0.11	0.02	0.38	0.52	0.23	-	0.00	0.00	0.47
99	157 BRIGALOW STREET	571.39	0.07	0.16	0.07	0.54	0.85	0.17	-0.35	0.02	0.00	0.47
100	159 BRIGALOW STREET	571.39	0.10	0.35	0.16	0.59	0.81	0.35	-0.20	0.28	0.05	0.49
103	149 BRIGALOW STREET	571.44	-0.10	0.17	0.04	0.30	0.27	0.13	-0.63	0.06	0.01	0.40
104	147 BRIGALOW STREET	571.44	-0.12	0.10	0.03	0.27	0.30	0.18	-0.61	0.02	0.00	0.39
105	86 LEWIN STREET	571.55	-0.31	0.06	0.02	0.10	0.32	0.21	-	0.00	0.00	0.41
106	88 LEWIN STREET	571.55	-0.30	0.36	0.04	0.12	0.34	0.14	-	0.00	0.00	0.42
107	82 LEWIN STREET	571.41	-0.22	0.07	0.03	0.18	0.15	0.11	-	0.00	0.00	0.40
108	84 LEWIN STREET	571.41	-0.21	0.25	0.03	0.20	0.58	0.06	-	0.00	0.00	0.41
109	145 BRIGALOW STREET	571.09	0.18	0.36	0.04	0.55	0.58	0.11	-0.36	0.02	0.00	0.37
110	143 BRIGALOW STREET	571.09	0.16	0.32	0.03	0.52	0.50	0.08	-0.38	0.09	0.00	0.37

			Without Dambreak			With Dambreak			Pre Dambreak			
Building reference	Address	Assumed floor level	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Max flood depth (m)	Velocity (m/s)	Max DV m^2/s	Incremental depth
111	78 LEWIN STREET	571.43	-0.27	0.15	0.06	0.12	0.19	0.13	-	0.00	0.00	0.39
112	80 LEWIN STREET	571.43	-0.25	0.06	0.01	0.14	0.22	0.13	-	0.00	0.00	0.40
113	141 BRIGALOW STREET	571.10	0.12	0.27	0.09	0.48	0.48	0.28	-0.38	0.06	0.01	0.36
114	139 BRIGALOW STREET	571.10	0.11	0.13	0.05	0.45	0.39	0.07	-0.38	0.06	0.01	0.34
115	135 BRIGALOW STREET	571.11	0.06	0.37	0.21	0.37	0.57	0.29	-0.49	0.14	0.01	0.31
116	137 BRIGALOW STREET	571.11	0.08	0.41	0.14	0.40	0.57	0.17	-0.49	0.02	0.00	0.33
117	76 LEWIN STREET	571.27	-0.12	0.22	0.02	0.25	0.27	0.05	-	0.00	0.00	0.37
118	74 LEWIN STREET	571.27	-0.12	0.06	0.03	0.24	0.32	0.20	-	0.00	0.00	0.36
119	133 BRIGALOW STREET	571.10	0.01	0.14	0.08	0.32	0.21	0.18	-	0.00	0.00	0.32
125	70 LEWIN STREET	571.12	0.01	0.22	0.05	0.34	0.28	0.10	-	0.00	0.00	0.33
126	72 LEWIN STREET	571.12	0.02	0.16	0.06	0.35	0.45	0.18	-	0.00	0.00	0.34
132	64 LEWIN STREET	571.39	-0.32	0.11	0.05	0.02	0.30	0.22	-	0.00	0.00	0.34
142	60 LEWIN STREET	571.32	-0.32	0.07	0.01	0.02	0.13	0.05	-	0.00	0.00	0.33
152	33 LEWIN STREET	571.31	-0.17	0.07	0.01	0.20	0.22	0.12	-	0.00	0.00	0.37
153	31 LEWIN STREET	571.31	-0.17	0.07	0.01	0.19	0.20	0.12	-	0.00	0.00	0.37
155	35 LEWIN STREET	571.25	-0.10	0.07	0.02	0.28	0.43	0.25	-	0.00	0.00	0.38
156	37 LEWIN STREET	571.30	-0.14	0.05	0.01	0.26	0.49	0.07	-	0.00	0.00	0.39
159	39 LEWIN STREET	571.47	-0.31	0.06	0.01	0.11	0.47	0.13	-	0.00	0.00	0.42
163	41 LEWIN STREET	571.44	-0.27	0.02	0.00	0.16	0.27	0.06	-	0.00	0.00	0.43
164	43 LEWIN STREET	571.63	-0.43	0.27	0.02	0.01	0.42	0.08	-	0.00	0.00	0.44
220	136 BRIGALOW STREET	570.77	0.51	0.34	0.14	0.82	0.34	0.16	0.18	0.34	0.14	0.31
221	136 BRIGALOW STREET	570.93	0.42	0.29	0.22	0.81	0.39	0.27	-0.02	0.11	0.03	0.39
222	136 BRIGALOW STREET	570.80	0.46	0.56	0.12	0.77	0.55	0.21	0.12	0.57	0.05	0.31
265	3 BOYD STREET	571.60	-0.19	0.17	0.02	0.25	0.43	0.31	-	0.00	0.00	0.45
267	1 BOYD STREET	571.60	-0.19	0.24	0.01	0.24	0.31	0.07	-	0.00	0.00	0.44
269	2 BOYD STREET	571.44	-0.08	0.29	0.03	0.31	0.48	0.07	-0.58	0.00	0.00	0.39
271	4 BOYD STREET	571.44	-0.08	0.10	0.04	0.32	0.34	0.17	-0.64	0.04	0.01	0.40

Notes:
1) The location of the buildings are indicated in the associated maps.

Appendix H - Kenny Breach Modelling

Plots are in the Appendix directory or by selecting the hyperlink below:
(Note: From the pdf document use ALT Left-Arrow to return to the main document location)

Table H-1 PMF 1 hour (Kenny basin spatial distribution) Kenny basin breach parameters (overtopping) north side of spillway

Breach formation time (min)	Breach base width (m)	Time breach assumed to occur (hours from start of event)	Estimated flood level in basin when breach occurs (m AHD)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
15	13.9	0.78	590.31	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	178-588	
3	3.9	0.59	590.17	Bureau of reclamation	52	
9	9.0	0.69	590.27	Von Thun Gillette		
56	10.1	1.46	589.49	Froehlich	37	After peak. Overtopping does not occur for full breach formation time.

Table H-2 PMF 1 hour (Kenny basin spatial distribution) Kenny basin breach parameters (overtopping) at spillway

Breach formation time (min)	Breach base width (m)	Time breach assumed to occur (hours from start of event)	Estimated flood level in basin when breach occurs (m AHD)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
20	7.9	0.86	590.29	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	257-846	
6	6.8	0.64	590.23	Bureau of reclamation	156	
11	11.1	0.72	590.29	Von Thun Gillette		
35	11.7	1.11	590.11	Froehlich	85	After peak.

Table H-3 PMF 1.5 hour (Southwell Park spatial distribution) Kenny basin breach parameters (overtopping) north side of spillway

Breach formation time (min)	Breach base width (m)	Time breach assumed to occur (hours from start of event)	Estimated flood level in basin when breach occurs (m AHD)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
15	12.5	0.85	590.20	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	171-563	
3	3.7	0.59	589.95	Bureau of reclamation	48	
9	8.8	0.68	590.13	Von Thun Gillette		
54	9.9	1.43	590.06	Froehlich	35	

Table H-4 PMF 1.5 hour (Southwell Park spatial distribution) Kenny basin breach parameters (overtopping) at spillway

Breach formation time (min)	Breach base width (m)	Time breach assumed to occur (hours from start of event)	Estimated flood level in basin when breach occurs (m AHD)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
19	7.3	0.85	590.24	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	250-822	
6	6.6	0.63	590.06	Bureau of reclamation	149	
11	10.9	0.72	590.16	Von Thun Gillette		
34	11.5	1.10	590.21	Froehlich	82	

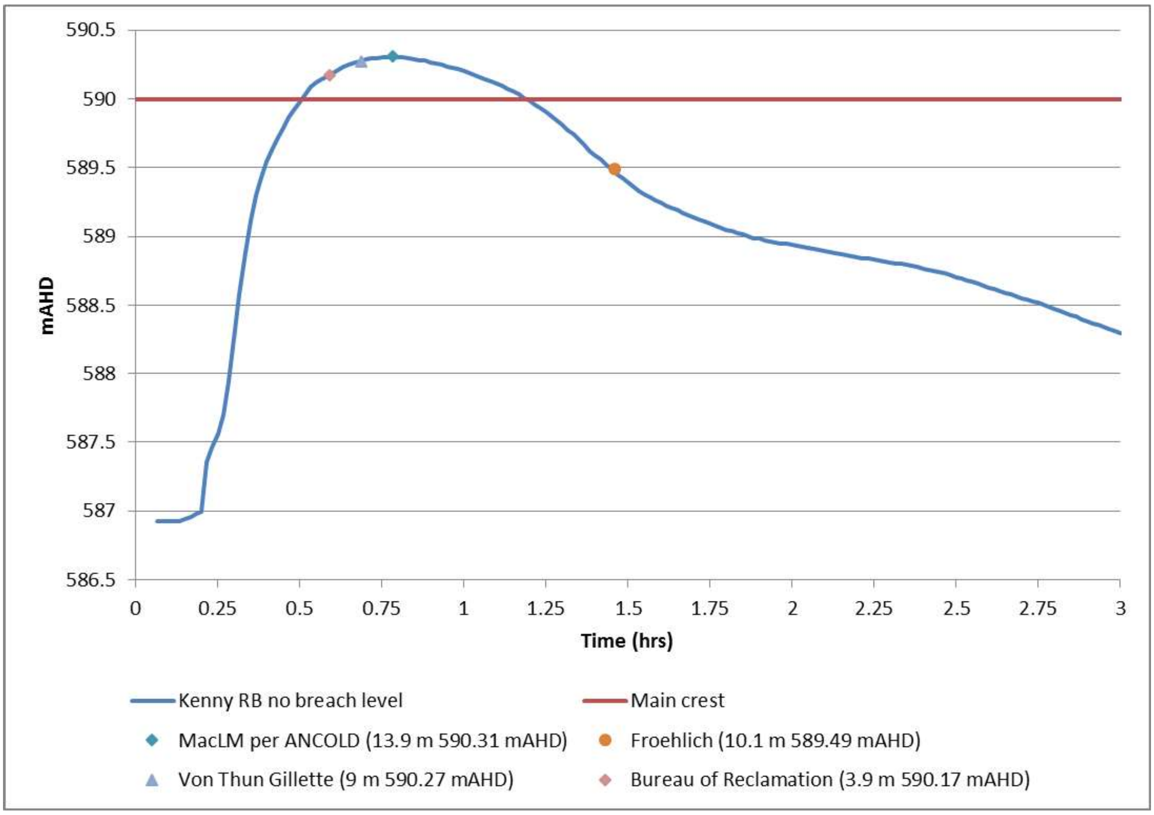

Figure H-1 Breach points along the PMF 1 hour hydrograph using different methods (breach at north side of spillway)

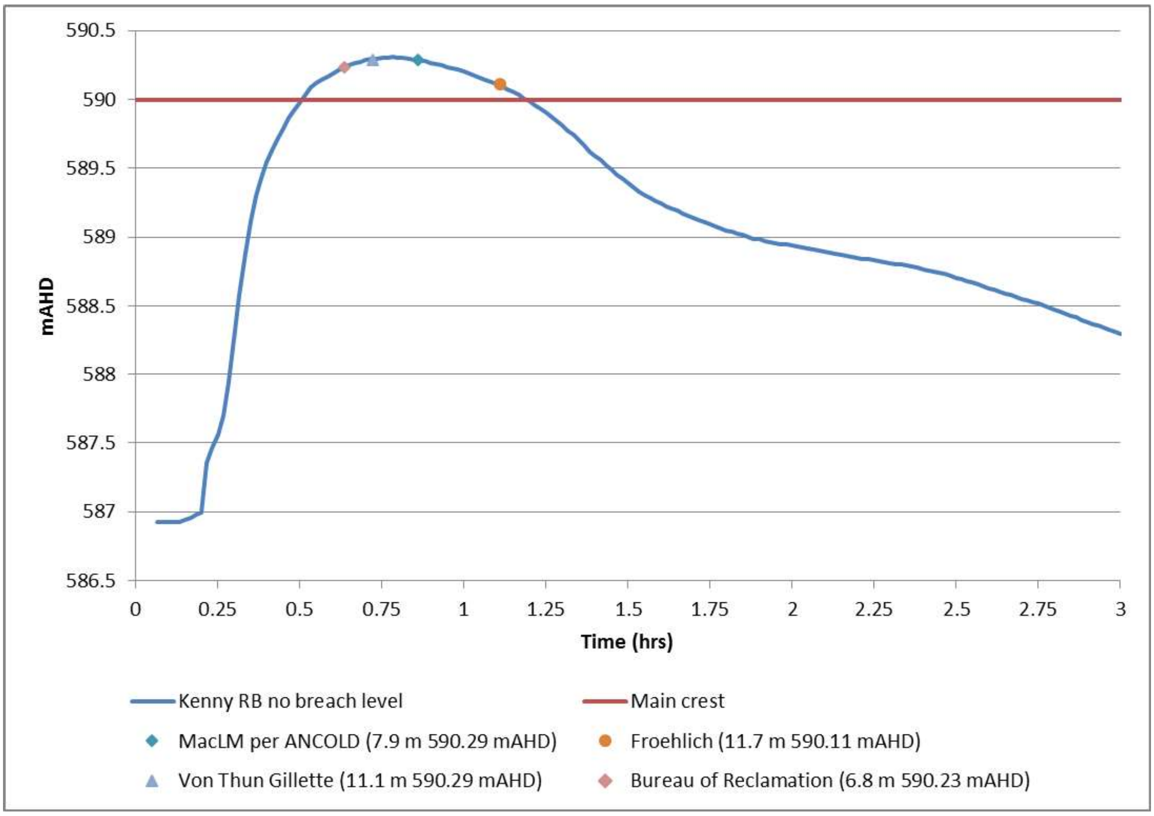

Figure H-2 Breach points along the PMF 1 hour hydrograph using different methods (breach at spillway)

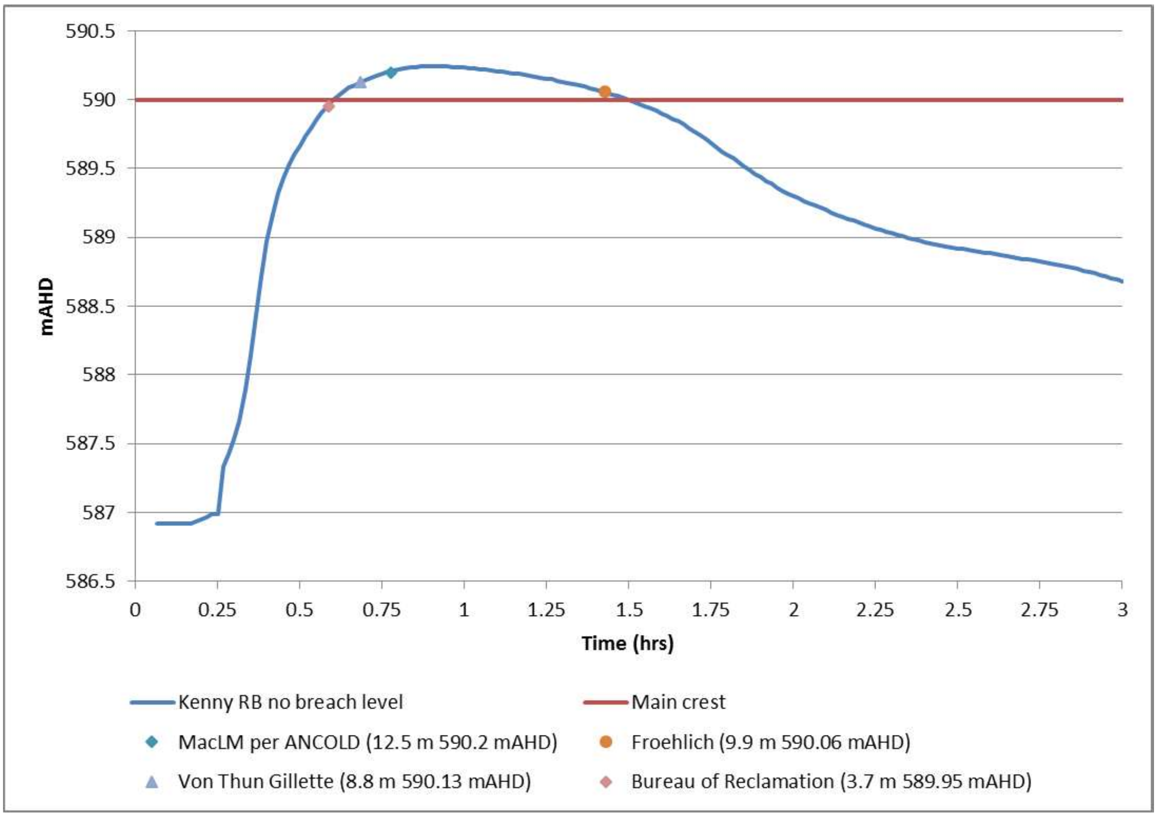

Figure H-3 Breach points along the PMF 1.5 hour hydrograph using different methods (breach at north side of spillway)

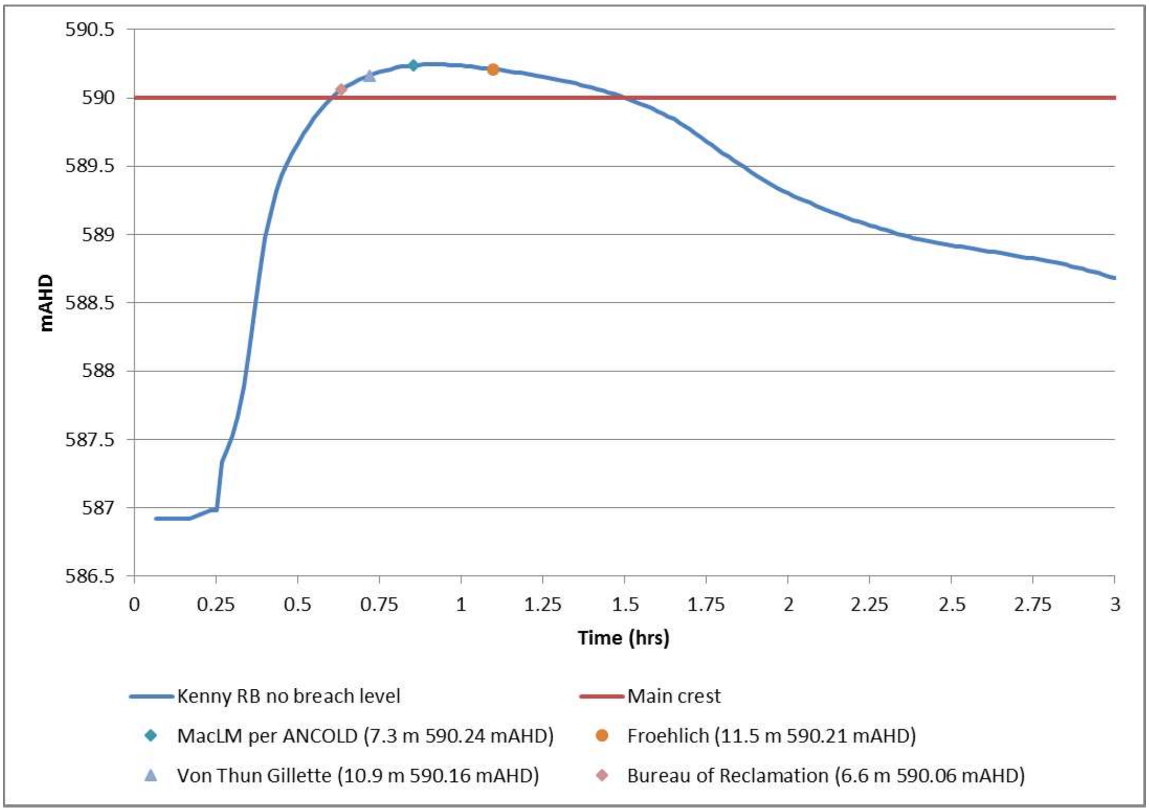

Figure H-4 Breach points along the PMF 1.5 hour hydrograph using different methods (breach at spillway)

Table H-5 Kenny basin 0.01% AEP 1 hour breach parameters (piping) north side of spillway

Breach formation time (min)	Breach base width (m)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
10	3	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	100-331	
2	1.6	Bureau of reclamation	17	
8	7.1	Von Thun Gillette		
36	5.0	Froehlich	14	

Table H-6 Kenny basin 0.01% AEP 1 hour breach parameters (piping) at spillway

Breach formation time (min)	Breach base width (m)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
15	2.8	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	183-604	
5	4.5	Bureau of reclamation	93	
10	9.2	Von Thun Gillette		
26	6	Froehlich	51	

Table H-7 Kenny basin 0.01% AEP 2 hour breach parameters (piping) north side of spillway

Breach formation time (min)	Breach base width (m)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
9	2.3	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	92-305	
2	1.4	Bureau of reclamation	15	
8	6.9	Von Thun Gillette		
34	4.8	Froehlich	12	

Table H-8 Kenny basin 0.01% AEP 2 hour breach parameters (piping) at spillway

Breach formation time (min)	Breach base width (m)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
15	2.4	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	176-580	
4	4.3	Bureau of reclamation	87	
10	9.0	Von Thun Gillette		
25	5.9	Froehlich	48	

Table H-9 Kenny basin 1% AEP breach parameters (piping) at spillway

Breach formation time (min)	Breach base width (m)	Method	Empirical equation predicted breach peak m3/s excluding simultaneous inflow	Comment
10	-0.4	Macdonald-Langridge Monopolis (after ANCOLD/ DERM)	104-344	not enough volume and/or head to form a breach
3	1.7	Bureau of reclamation	36	
9	6.8	Von Thun Gillette		
21	4.0	Froehlich	21	

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