PORT STEPHENS COUNCIL

MEDOWIE DRAINAGE AND FLOOD STUDY

FINAL REPORT

MAY 2012

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Project MEDOWIE DRAINAGE AND FLOOD STUDY	**Project Number** 29068
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MEDOWIE DRAINAGE AND FLOOD STUDY

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FOREWORD

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

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

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

1. Flood Study

- Determine the nature and extent of the flood problem.

2. Floodplain Risk Management

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

3. Floodplain Risk Management Plan

- Involves formal adoption by Council of a plan of management for the floodplain.

4. Implementation of the Plan

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

The current study is the first step in Council addressing flood risk within the Medowie area. In the ensuing stages of the process (i.e. the Floodplain Risk Management Study and Plan) the model described herein, along with results, can be utilised to assess the cost of flood damages and also to test proposed flood mitigation works/planning decisions.

Please note that ARI terminology will be used herein to describe events up to an ARI of 5 years, whilst for larger events, AEP terminology will be used. Generally use of AEP terminology is preferred, however, since whole numbers cannot be used for events smaller than the 5Y ARI (20% AEP), ARI is used for these.

For definitions of acronyms or technical terms, a glossary is provided at the rear of the document in Appendix A.

EXECUTIVE SUMMARY

Background

The Medowie area, located five kilometres to the north-east of Raymond Terrace in NSW, has in recent years undergone considerable residential development and potentially more may occur in the future. Rural and rural residential lots are being sub-divided for suburban lots and in some locations new development is proposed. A challenge that faces planners is that within both the Campvale and Moffats Swamp catchments which comprise Medowie, some areas are subject to inundation by flooding in relatively small flood events (1Y to 5Y ARI). A further complication is that the Campvale catchment is a drinking water supply catchment and so drainage solutions will need to take account of water quality impacts.

Areas upstream of Moffats Swamp have been developed relatively recently (in the past ten years) and, in some areas, it has been shown that houses may have been built adjacent to substantial overland flow paths. Council has subsequently moved to implement mitigation works and to date this work has been partially successful.

Campvale Swamp has historically exhibited two main flooding related issues. Firstly, areas upstream of Ferodale Road have been subjected to relatively short duration flooding of property and roads, with over floor level flooding in some cases (particularly along Kirrang Drive). Secondly, downstream of Ferodale Road flooding of the Campvale Drain Inundation Area (CDIA) can be sustained over weeks and tends to disrupt/damage agricultural activities rather than flood residences (although some residences are flood affected). Although a problem of real concern, as it is related to the duration of inundation rather than the peak flood level achieved it is noteworthy that this issue cannot be a focus of a flood study carried out under the NSW State Government Floodplain Risk Management Program. Inundation within the CDIA is aggravated by the restricted outlet capacity and by unmitigated upstream development, which by converting pervious areas to impervious areas removes catchment storage.

Summary of Method and Calibration/Validation

In order to assess the flood liability of Medowie, WMAwater have been given the task of carrying out a drainage and flood study for the Moffats and Campvale Swamp catchments. As part of this work various data sets have been collected, previous reports have been reviewed, a community survey has been carried out, calibration data has been collected and a hydraulic model has been built, calibrated, validated and then applied to design flood modelling. Calibration/validation work demonstrates a good match between the model and a number of reliable flood levels. It also demonstrated a close match to a wide range of observed flood depths collected during the extensive community consultation phase of the project.

Summary of Scenarios Run

Various model scenario runs were made in order to examine the impact on design flood levels.

The scenarios run in the modelling system were as follows:

- Future Development Scenario. For the 1%, 5% and 20% AEP events the impact of future planned land use has been modelled. Note that future land use modelled is as per Figure 9;
- Climate Change Assessment. The 1% AEP event has been assessed for climate change impact and this includes runs to assess how flood levels based on future development (as discussed above) are impacted by climate change;
- Blockage. For the 1% and 5% AEP events the impact of blocking the following five structures have been assessed:
    - Culvert at Ferodale Rd;
    - Culvert corner of Kula Rd and Kirrang Drive;
    - Kula Road (two culverts near Karwin Road);
    - Culvert at 754 Medowie Road; and
    - Drainage path on North-East corner of 35 County Close.
- Pump Outage. The impact of failure of the Campvale Water Pumping Station (WPS) was assessed on peak flood levels for a variety of events.

Design Flood Modelling Results

A range of design flood runs have been carried out in order to define the existing flood risk within the Medowie area. Runs have been carried out for the 2Y ARI and 20%, 10%, 5%, 2%, 1%, 0.5% AEP events as well as the Probable Maximum Flood (PMF). A variety of durations were utilised as the varied characteristics of the study area (urbanised short duration conveyance based issues upstream and volume sensitive mechanisms downstream) meant that very different durations achieved critical flood levels in different parts of the study area. For Campvale the 2, 9 and 72 hour duration events were critical in the upper, mid and lower parts of the study area respectively. It was observed that Campvale WPS has little impact on design peak flood levels downstream of Ferodale Road due to its low pumping capacity (approximately 23 mm of effective rainfall, i.e. rainfall not lost to interception, infiltration and evaporation can be pumped out in one day). Moreover, initial conditions within the drain (within a reasonable range) will not have a significant impact on peak flood levels upstream of Ferodale Road.

Results indicate that numerous properties will suffer some degree of inundation. Most flooding will occur, not on the fringes of the swamp areas in Campvale and Moffats, but rather in upstream areas where flows moving downstream are not able to be contained in overland flow paths and drains. A feature of such flood liability is that due to limited upstream catchment area flood depths are unlikely to change dramatically for events of different probability. That is, whilst property may be liable to flooding in relatively small events, flood depths will not dramatically increase in rarer events such as the 1% AEP event for example.

The actual number of houses impacted by over floor flooding cannot be discerned at this stage prior to the acquisition of the floor level survey. It is not anticipated that the number of houses inundated in the 1% AEP flood event would exceed 20 based on the results presented herein. Major flood liable locations are identified as follows:

- At the intersection of Kirrang Drive and Kula Road;
- In Ballat Close;
- Kirrang Drive near the Campvale Drain;
- Abundance Road south of the intersection with Lisadell Road;
- Isolated areas on James Road;
- Windeyer Place on the southern side which fringes Moffats Swamp; and
- Near the intersection of Potoroo Road and South St.

High hazard and areas designated as floodway (in the 1% AEP event) tend to be limited to defined flow paths and also those areas where water depth accumulates, such as the swamps. Few houses are impacted by high hazard flows or are likely to be found in defined floodways.

1. INTRODUCTION

In recent years near annual flooding events have shown that areas within the Medowie locale, both within the Campvale and Moffats Swamp catchments, are flood liable (refer to Figure 1 to see the study area). In order to define the flood liability Port Stephens Council (Council) have appointed WMAwater to carry out a Drainage and Flood Study. The Office of Environment and Heritage (OEH) as well as Hunter Water Corporation (Hunter Water) are providing assistance towards the study.

The study has established suitable hydrologic and hydraulic modelling tools, demonstrated their capacity to emulate local flood behaviour via calibration/validation (as data allowed) and then applied these tools to establish the existing flood risk for a range of design flood event probabilities in conjunction with a range of event durations. Other work carried out includes:

- Defining flood hazard and preliminary hydraulic categories as per Appendix L of the NSW Government's Floodplain Development Manual;
- Assessing the impact on flood risk of further development in Medowie as per the Medowie Strategy Plan March 2009;
- Assessment of the impact of climate change on flood behaviour (as per the NSW State Governments 2007 document);
- Preliminary assessment of potential impacts of blockage; and
- Assessment of the impacts of pump failure on design flood levels.

A range of sensitivity testing has been carried out. Sensitivity testing seeks to identify critical drainage infrastructure within the study area and also to establish the robustness or otherwise of the design flood predictions.

In the next stage of the study (the Floodplain Risk Management Study, which is outside the scope of this report) the model may be utilised to test preliminary mitigation measures.

This report details the study undertaken and in particular the model build, calibration and design flood results. The key elements of this report are:

- Presentation and review of available data;
- Review of previous relevant reports;
- The model development process;
- Calibration/validation of the model;
- Sensitivity testing;
- Definition of design flood behaviour;
- Definition of hazard and hydraulic categories; and
- Additional modelled scenarios investigating the impact of proposed future development, climate change, specific blockage scenarios and also the impact should the Campvale WPS fail to operate during an event.

A glossary of flood related terms is provided in Appendix A.

2. BACKGROUND

2.1. Introduction

Medowie is located approximately 25 km north of Newcastle and approximately 5 km north-east of Raymond Terrace. Figure 1 shows the proximity of the Medowie catchment to Grahamstown Reservoir. The most significant recent flood event that impacted residents occurred on the 14th-15th February 2009. This event caused nuisance flooding for residents at locations such as Lisadell Road, Kirrang Drive, Kula Road and Ballet Close (in the Campvale catchment) and County Close and Federation Drive (amongst others) in the Moffats Swamp catchment. The main impact for those that suffered flooding was inundation of roads and sheds. Not only did the event cause flooding of many properties (although no water entered any house) within both the Campvale and Moffats Swamp catchments, it also partially filled the Campvale Drain Inundation Area (CDIA, refer to Figure 1 for location) starting a period of continuous inundation that extended, according to comments by locals, a month into March. Despite the extensive flooding in the CDIA, residents report that the Campvale Drain was unable to deliver sufficient flow in order to allow the Campvale Water Pumping Station (Campvale WPS) to pump at peak capacity (around 5.4 m^3/s) throughout the period of continuous inundation. Small events subsequent to the February 14-15th event led to further runoff which “topped up” inundation and prolonged it.

The Medowie study area consists of two separate catchment areas: Campvale Swamp to the west and Moffats Swamp in the east (see Figure 1). The Campvale catchment contains a diverse group of landholders; the upper areas are home to natural forests, the middle reaches contain rural, rural/residential, residential, commercial and even industrial areas whilst the more downstream reaches are home to land uses that are substantially agricultural (cattle, horticulture, orchards, etc). Moffats Swamp is similarly composed in regards to the upper reaches (forested areas transitioning to residential areas) but in the downstream area contrasts with Campvale in that this area is entirely undeveloped. Heavy vegetation below an RL of approximately 9 mAHD is the norm in the Moffats Swamp catchment.

As small rural holdings and infill type development has occurred in Medowie (large rural holdings become sub-divided for urban residential projects) there is a perception in the community that the occurrence of nuisance flooding is increasing. A key issue in the Campvale catchment specifically is that whilst upstream landholders wish to transfer floodwaters downstream as efficiently and quickly as possible in order to avoid residential inundation, downstream holders of agricultural land feel doubly worse off. Not only can they not sub-divide their holdings for sale as residential blocks (as they are flood affected), but utilising them for agricultural purposes becomes difficult when they become inundated on a regular basis and stay inundated for long durations. The perception within the community being that upstream development exacerbates the likelihood of downstream land being inundated and once inundated keeps it under water for more days per year than otherwise might be the case.

Currently CDIA pondage and peak flood level behaviour is caused by the “pinch”. The pinch is a ridgeline approximately 1,500 m upstream of the Campvale WPS that hydraulically separates the Swamp from the pumping station. A man made channel links the pumping station and the Swamp. The location of the ridgeline is often referred to as the “pinch” by Medowie residents and this terminology will also be in this report.

The effect of the pinch is significant for all levels of flow. For example even as flood levels in the CDIA reach approximately 7 mAHD and the amount of flow through the pinch becomes sufficient to enable the Campvale WPS to operate at peak capacity, the pinch will continue to significantly attenuate peak flow. In the PMF a peak flow of approximately 180 m^3/s flows into the CDIA, however, the pinch limits flows to the downstream pumping station to approximately one eighth of this figure. Campvale WPS pumps sit at an invert level of 9.6 mAHD (Reference 12). Damage to the pumps due to flooding is not likely to occur as the PMF peak flood level is below the mentioned level.

A distinguishing characteristic of the Campvale catchment is that all water exiting Campvale catchment (barring evaporation and infiltration) must be pumped into Grahamstown Reservoir. The reservoir (which is the Hunter’s largest drinking water dam) is operated by Hunter Water. Hunter Water built the facility in 1962 in order to augment the area’s water supply.

Except when in flood, the Campvale and Moffats Swamps are separate catchments with respect to overland flow. Runoff from the Campvale Catchment will make its way into the Campvale Drain via a variety of upstream tributaries (many of which flow through urban residential areas) and then be detained in the CDIA until such time as the water can be either evaporated, infiltrated or pumped into Grahamstown Reservoir.

Runoff from the Moffats Swamp catchment flows via a series of tributaries, some of which traverse relatively dense urban development, into Moffats Swamp which is in a relatively natural state and is thickly vegetated. Unlike Campvale, four mechanisms exist for flow to leave Moffats Swamp. Three of the Moffats Swamp outlets move flow out of the Medowie area entirely whilst the fourth outlets to the Campvale catchment and into the CDIA (note that each of the outlets operates at different water levels and has varying stage-discharge characteristics).

2.2. Study Area

2.2.1. The Campvale Swamp Catchment and Trunk Drain

The Campvale catchment area is 20.5 km^2 and is bounded by a ridgeline lying to the east of Medowie Road, Richardson Road to the south and Grahamstown Reservoir to the west (see Figure 1).

The catchment rises in forest north of the township, drains south via the Campvale Drain into the CDIA, then south-west to the Campvale WPS which transfers water into Grahamstown Reservoir.

As a drinking water catchment, activities that affect water quality within the Campvale catchment area are regulated under the Hunter Water Regulation (2010) which seeks to minimise risks to drinking water quality.

The drainage system within the catchment consists of relatively ill-defined natural watercourses, open drains, pipes and pits, culverts and the downstream pumping station. Most of these elements are in Council's care and control and lie within road and drainage reserves, public reserves and drainage easements. However, some of these elements are controlled by other bodies such as Hunter Water.

Campvale Drain terminates at Campvale WPS which is owned by the Hunter Water. The pump station houses four pumps. Each pump has a maximum pumping capacity of approximately 1.35 m^3/s (or approximately 120 ML/day)[1]. The pumps are responsible for conveying the majority of all stormwater runoff from the catchment into Grahamstown Reservoir. The pump operation is automated and dependent on the water level at the off-take location.

The frequency, extent and period of flooding are the main concerns for the land owners in the CDIA and therefore of great concern to Council. Interviewed land holders highlight two principle issues exacerbating the severity (with respect to duration principally) of flooding of private land within the CDIA:

- The aforementioned "pinch" in the Campvale Drain limits flow arriving at the Campvale WPS. A consequence of this is that the Campvale WPS is not running at full capacity, yet ponded water remains above the pinch; and
- The limited capacity of the pumps at the Campvale WPS and/or the Hunter Water's reluctance to pump runoff into Grahamstown Reservoir due to water quality considerations. Note that given operation of the pumps is automatic this criticism seems baseless. It is also noteworthy that except when the stage height in the CDIA exceeds 7 m it is the pinch which limits outflow, not the pumps.

From a flooding perspective Campvale Swamp is relatively unique in that outflow from the catchment is limited by pump rate and does not scale. Due to this constraint there are two different types of flood affected residents within the wider Campvale catchment. Residents upstream of Ferodale Road are impacted by water moving downstream and residents in lower areas by inundation due to rising water as the swamp fills in large long duration flooding events. The lowest residents in Campvale are at 7.5 – 8.0 mAHD and depending on losses, the swamp can be filled to such heights by the 5% - 2% AEP event.

2.2.2. The Moffats Swamp Catchment

Moffats Swamp catchment is bound to the north and north east by a ridge running through Medowie's State Forest, to the west by a ridge line running east of Medowie Road and then down to the southern boundary at Richardson Road. The eastern boundary is defined by a previously mined sand barrier which adjoins the Tomago Sand Beds. The total area of the

[1] Note that at peak capacity the Campvale WPS can remove the equivalent of approximately 23 mm of runoff per day.

catchment is 15.7 km^2.

Complaints of flooding have been received from residents in the lower areas bordering the fringe of the swamp and properties along the County Close, Federation Drive and Settlers Close areas.

There are four outlets from Moffats Swamp (refer to Figure 1):

- Swan Bay - The majority of floodwaters currently exiting the swamp are through a trapezoidal shaped concrete spillway to the east with an invert level of RL 8.35 mAHD. This outlet flows to Racecourse swamp, which drains to Twelve Mile Creek and then Swan Bay;
- Campvale Swamp – a natural ridge between Moffats Swamp and Championship Drive operating when the swamp level rises above 9.5 mAHD;
- Salt Ash – A partially blocked triple box culvert with an invert level of 8.7mAHD; and
- Salt Ash – A natural saddle south of the concrete spillway draining into the nearby Moffats Ck and dispersing toward Salt Ash, activated only in high flows (approximately 10m AHD).

The availability of multiple outlet locations, and the fact that outlet capacity increases as stage increases in the swamp, make Moffats Swamp quite different to Campvale. Again though in Moffats as in Campvale, a large flood event (in the order to the 2% AEP event) can fill the swamp and cause low lying houses (lowest residential floor levels in Moffats Swamp are approximately 9.5 mAHD) to be impacted by rising water levels. However, the most likely flooding mechanism is as flowing water moves past/through properties on the way to the swamp proper.

3. DATA REVIEW

Various items of data as well as reports salient to the study have been collected and reviewed. This section provides a summary of the reports as well as a description of the various different forms of data utilised in the study.

3.1. Previous Studies

3.1.1. Reference 1

"Medowie Structure Plan: Preliminary Flooding, Drainage and WSUD Analysis". WBM Pty Ltd, 2006.

- Land use is rural open space/rural residential and pockets of urban residential;
- 2 and 9 hour events found to be two largest events with respect to flood levels;
- Conservative losses used in the study (initial loss of five and a continuing loss of zero for pervious areas);
- In-channel roughness is 0.035 and floodplain is 0.05;
- Base width of Campvale Drain varies from 5 to 9 m;
- 1% AEP level at Kula Road is 8.5 mAHD (approximately 0.5 m depth over road) and at Ferodale Road it's 8.4 mAHD (road invert level is approximately 7.9 mAHD);
- A lot of volume for storage in the drainage system (relative to catchment size) and hence a lot of attenuation in the system;
- Modelling of proposed development showed that the impact on flood levels is small [this needs to be seen in the light of the fact that the WBM report used very small losses for pervious areas however which very much skew results towards this outcome]. The current capacity of the drainage system generally is estimated to be in the order of 1 – 2 Y ARI; and
- Campvale Drain reach between 1,800 and 2,500 m contains a constriction (or "pinch") such that during flooding the area upstream of the pinch (the CDIA) will be flooded whilst downstream of the pinch flow will remain in-bank [location identified approximately 1500 m upstream of the Campvale WPS in this report].

3.1.2. Reference 2

"Medowie Drainage Study: Part 1 – Campvale Catchment". PSC, 1995.

- Since 1970's Medowie has undergone rapid residential and urban development;
- Flooding is a bigger issue in Campvale than Moffats Swamp;
- The Campvale Drain inundation area is below 8 m AHD;
- Catchment area is 18 km^2;
- Campvale WPS installed in 1962 – 4 multi stage SCADA (float) controlled pumps;
- A flooding event occurred in May 1994 and some residences were inundated. Peak flood level of 7.8 m at Ferodale Road gauge board [no time or date is given for the peak level observation];
- In February/August 1990 flooding also occurred and heights at Ferodale Road were 7.9 and 7.75 mAHD respectively [once again no time or date is given for the peak level observation although the February 1990 event was a multiple burst storm spread over

four days]. High water tables in 1990 and "flooding" problems varied from house to road flooding;

- Details from Table 3-1 indicate where historical flooding issues are most felt (see below);
- Sutton Park Estate is flood liable in the 1Y ARI event whilst other areas are more like 3 to 5Y ARI;
- Gauge board exists at Ferodale Rd;
- Feb 90 event was used for calibration. Ferodale Gauge board was used as was an observation of flooding behaviour at Ballat Close. In channel n of 0.035 and floodplain value of 0.10. Slightly underestimated peak flood height for calibration;
- Ferodale Rd is identified as the control for all upstream areas;
- Low losses used in hydrological modelling;
- 5.4 m^3/s is the maximum output possible at Campvale WPS; and
- Numbers 7 and 8 in Ballat Close have previously experienced some flooding.

Table 1: Flooding troublespots identified in Councils 1995 study

Table 3.1 - Medowie Flooding 1990

Site	Degree/Extent of Flooding	Likelihood of Re-occurrence
Kula Rd - Ryan Rd	• Road & property flooding. • This area also contributes a lot to flooding further downstream	HIGH
Fisher Rd	• Drain & property badly scoured	HIGH
North of Ferodale Rd	• Drain deficiencies worsen the above flooding in places	HIGH
South (downstream) of Ferodale Rd	• Extensive property flooding • Slow to drain away	MODERATE
Near corner of Abundance Rd & Lisadell Rd	• Property flooding unsatisfactory (no formal road drainage outlet) • Ground sodden for months	HIGH
North (upstream) of Ferodale Rd - Sutton Park Estate	• House septic road & property flooding at various locations • Long term septic soakage effects	HIGH
Kirrang Drive	• Upstream houses flooded • Fairly rapid rise in water levels	HIGH

3.1.3. Reference 3

"Medowie Drainage Study: Part 2 – Moffats Swamp Catchment".PSC, 2000.

- Prior to 1992 Moffats Swamp would discharge to Salt Ash and Campvale Swamp (sand dunes kept water out of Racecourse Swamp). Egress to Salt Ash via a triple cell RCBC with invert level at 9 mAHD;
- Reconfiguration followed a 1990 study (AWACS 1990) into Williamtown and Salt Ash flooding;

- As per recommendations a structure to transfer flow to Racecourse Swamp was put in place whilst the outlet to Salt Ash was blocked. Subsequent re-jig of this arrangement meant that flow to Racecourse Swamp could begin at 8.35 mAHD and that no flow will move to Salt Ash until flood levels reach 8.7 mAHD and higher;
- In a large flood there are effectively four outlets in total and these are:
  - Swan Bay – RL 8.35 mAHD, trapezoidal. Water moves from Moffats Swamp to Racecourse Swamp and then to Swan Bay;
  - Salt Ash – Tripe Box Culvert with Invert Level = 8.7 mAHD;
  - Campvale – natural ridge between Moffats and Campvale Swamps on Championship Drive inside the golf course at an invert level of 9.5 mAHD; and
  - Natural saddle south of concrete spillway draining into Moffats Swamp that is activated only during very high flows (approximately 10 mAHD).

3.1.4. Reference 4

"Boundary Road/Federation Drive Flooding Investigations".GHD, 2008.

- Investigated drainage at County Close/Federation Drive;
- Development may have encroached onto a natural overland low path;
- Flooding occurred 25th April 2008;
- Photos from Council show that numbers 27, 29, 31, 33, 35 and 37 on County Close are flood affected;
- Created a model and ran various flooding scenarios. Used pervious material losses of 5/1 (initial (mm) and continuing (mm/h) respectively) and then also tried 15/2.5 (for sensitivity);
- Roughness value of 0.15 used in Moffats Swamp; and
- Council proposed an earth bund (and subsequently built it) at rear of County Close at top height of 14.5 mAHD.

3.2. Topographic Data

3.2.1. ALS

Council have supplied WMAwater with a 2 m DEM derived from ALS. The ALS data was flown by Fugro Spatial Solutions Pty Ltd in January 2007. This data forms the foundation of the model build process. The 2 m grid has been aggregated into a 5 m DEM (using a mean value aggregation method) and it is this DEM which has then been used to inform the hydraulic model. The ALS data has an accuracy of +/- 0.15 m (for 67% of the data set) although where an area is heavily vegetated or inundated then the accuracy of ground strikes may be much worse.

3.2.2. Landform changes

Over time various changes have been made to the landform in Medowie. As numerous historical events are to be modelled in this study it is necessary for a timeline of such changes to be constructed and then heeded when modelling observed events. The timeline is shown below in Table 2.

Table 2: Timeline of landform changes in Medowie

Date	Works on the floodplain
After 1990	Western end of Richardson Road raised 500mm after 1990 flood.
Approximately 1993	Moffats Swamp outlet to weapons range. Invert of 8.35 mAHD. Outlet to Salt Ash blocked to 8.7 mAHD.
1994/1995	Ballat Close levee constructed.
1995	Changes to Medowie Road started due to construction of the Pacific Dunes Golf Course.
1996	Retarding basin levee upstream of Boundary Road.
1996/1997	Retarding basin in Medowie Road near Kindlebark Drive.
1997	Kirrang Drive culvert upgraded.
1998	Two retarding basins constructed. The first one on Evans Road near Kula Road, and the second one upstream of Evans Road.
2002/2003	Changes to South Street in the proximity of the Pacific Dunes Golf Course.
2002/2005	Construction of Pacific Dunes Golf Course.
2007	Ballat Close levee raised after June 2007 floods.
2009	Levee on County Close. Construction started after the February 2009 flood event. Completed between May and July 2009.

3.2.3. Cross-Sections

Cross-sections were obtained from the previous WBM study (Reference 3) and their location is shown in Figure 13. The cross-sections from the WBM study included floodplain details. This information was removed for the model build reported upon herein as the requirement was for in-bank cross-sections only with floodplain data coming from the previously mentioned ALS data set.

3.3. Structure Data

Numerous structures impact on drainage within the study area and this includes operated structures such as the Campvale WPS as well as more conventional structures such as bridges and causeways (refer to Figure 14). Structure information has come from the following sources:

- Council, in particular the 1995 and 2000 Council reports which addressed drainage issues in Medowie (References 2 and 3 respectively); and
- Hunter Water.

3.4. Rainfall Data

Medowie’s average annual rainfall is approximately 1150 mm with a distribution which leans towards late summer/autumn rain. This section outlines the rainfall data used in the calibration process and also examines general rainfall patterns in Medowie.

A number of rainfall stations have been identified as being reasonably proximate to the study area and data from these has been sourced in order to facilitate different elements of the study. The table below summarises the stations used and presented in Figure 2.

3.4.1. Pluviograph Data

There are two high resolution rainfall gauges proximate to Medowie; the Williamtown RAAF and Grahamstown pluviometers, operated by the Bureau of Meteorology (BOM) and Hunter Water respectively. Figure 2 shows the location of the gauges and details for each of the stations are summarised in the table below. The Williamtown gauge has a highly complete record and covers the main events of interest in the calibration/validation run set. In contrast the Grahamstown gauge has many data gaps and in fact provides no coverage for any of the historic events modelled herein. As such the Williamtown data has been used as the main source of rainfall data for the calibration/validation process.

Table 3: Proximate Pluviograph Stations

BoM Station Number	Station Name	Record Start Date	Record End Date
061078	WILLIAMTOWN RAAF	1/01/1953	17/11/2009
061311[2]	GRAHAMSTOWN (HUNTER WATER BOARD)	3/01/1975	31/1/2006

Figure 3 shows Intensity-Frequency-Duration (IFD) plots for the three principle rainfall events to be utilised in calibration/validation, i.e. the February 1990, June 2007 and February 2009 events. It also shows the IFD characteristics for the scaled rainfall for the June 2007 and February 2009 events[3].

The February 1990 rainfall event was characterised by rainfall intensities between 5% and 2% AEP for the 2 to 3 hour duration. The frequency of the rainfall intensity significantly increased as the event continued, exceeding the 1% AEP after 12 hours. This indicates that an event with reasonable intensity but of significant duration (rather than being critical for areas upstream of Ferodale Road) will have its worst impacts on those properties adjoining the CDIA.

The June 2007 and the February 2009 events were, however, of significantly lower duration relative to the February 1990 event. June 2007 scaled rainfall intensities are greater than 1% AEP between 30 minutes and 1 hour. For other durations of interest, namely 2 and 9 hours, the intensities were in the order of 5% AEP event.

The February 2009 event was the least intense of the three events for short durations (less than a 20% AEP event for 3 hours or less), however, after 6 hours rainfall intensity is greater than the June 2007 event peaking with an intensity between 2% to 1% AEP event at 12 hours.

In addition to collecting pluviograph data suitable for running calibration and validation events, proximate daily rainfall records have also been sought to confirm rainfall depths utilised in historical event modelling, in order to facilitate scaling of the data so to improve its representativeness. Additionally, it was also used to examine the seasonality of rainfall for the Medowie area.

[2] Data from the Grahamstown gauge is unavailable for any of the three historical events modelled herein and as such is not used in the Study.
[3] Rainfall scaling is discussed further in Section 4.3.

3.4.2. Daily Rainfall Data

Data for a number of proximate daily stations has been collected (Table 4). Daily data, due to its lack of resolution, is not directly useful in the calibration/validation process. It can be used however to check the general validity of the pluviograph data utilised.

Besides daily rainfall records collected from the BOM for seven proximate stations (see Figure 2) a reasonable daily record has also been supplied by local residents living within the Campvale Catchment at 7 Wade Close (see Figure 2). An inspection of the installation of the gauge indicates that it is placed reasonably (albeit not as per BOM guidelines with respect to height of installation and proximity to surrounding trees) and a comparison of gauge readings with surrounding daily stations indicates that the data seems valid.

Analysis of the 7 Wade Close daily record has been carried out in order to typify the seasonality of rainfall at Medowie area. 7 Wade Close data has been utilised as its location makes it likely to be the most spatially representative record available. Figure 4 shows the monthly mean rainfall for the gauge (record length 1981-2009 inclusive) as well as the monthly standard deviation. This data indicates the seasonality of the rainfall with the highest falls occurring in the months February to June. It is noteworthy that February has the highest mean rainfall as well as the highest standard deviation. This implies that rainfalls in February are more variable than for any other month and given the high base rainfall this implies that February would be a common month for flood events historically in Medowie. A review of past events finds that of six known historical events five of them occur in the February – June period.

Table 4: Proximate Daily Rainfall Stations

BoM Station Number	Station Name	Record Start Date	Record End Date	Record Length
061010	CLARENCE TOWN (GREY ST)	08/11/1895	30/11/2009	114
061031	RAYMOND TERRACE (KINROSS)	01/04/1894	31/12/2009	115
061055	NEWCASTLE NOBBYS SIGNAL STATION AWS	01/01/1862	25/01/2010	148
061072	TAHLEE (CARRINGTON (CHURCH ST))	01/04/1887	31/12/2009	122
061078	WILLIAMTOWN RAAF	1/09/1942	25/01/2010	68
061311	GRAHAMSTOWN (HUNTER WATER BOARD)	1/05/1971	31/08/2008	37
061395	TANILBA BAY WWTP	1/01/2002	31/12/2009	8

3.5. Pump/Water Level Data

The Campvale WPS is included in the model setup and this is done on the basis of data describing the pumps' performance (versus stage) supplied by the Hunter Water. The Campvale WPS has the capacity to move a maximum of approximately 480 ML per day which equates to a runoff depth of approximately 23 mm. The Campvale WPS will not influence design flood levels within the study area (particularly in areas upstream of Ferodale Road) and its inclusion in the model (the purpose of which is solely design flood estimation) is not critical. It may be critical however if future modelling seeks to address the issue of how inundation time in the CDIA might be impacted by drainage improvement works, an upgrade of the Campvale

WPS and/or a change to the rules which govern its current operation[4].

Figure 5 and Figure 6 show pump and water level data respectively for the Campvale WPS for the June 2007 event. Note that no data from the Campvale WPS is available for either the February 1990 or February 2009 events (as advised by the Hunter Water). This data was requested from Hunter Water but not supplied. Subsequent to provision of a draft final report to Hunter Water for review, it is apparent that such data is available. As stated previously pumping data is not critical for design flood behaviour and so a follow up data request has not been placed.

3.6. Ground Water Data

As per the Brief, an assessment is required as to whether or not ground water is likely to interact with surface flooding behaviour, particularly with respect to antecedent conditions for flooding events. To facilitate this available ground water data has been collected from Hunter Water. Numerous sites within the Medowie area were identified, and eventually three sites for which reasonably lengthy records exist have been selected for presentation and analysis and the locations of these are shown in Figure 2. This data is discussed further in Section 4.4 and plots of the data are shown in Figure 7. The table below summarises the length of data record available for the three stations identified in Figure 2. Please note that the resolution of the data is approximately monthly (sometimes once per two months approximately).

Table 5: Ground Water Records Utilised

Bore ID #	Site start	Depth to Water table (m)	Mean depth to Water table (m)	Standard Deviation (m)
SK4934	7/04/1976	11.6	4.8	1.0
SK5387	5/07/1978	10.2	3.0	0.9
SK3506	7/04/1976	8.6	2.4	0.9

3.7. Various GIS Layers

A variety of GIS layers have been supplied to WMAwater by Council. The layers supplied include the following:

- Cadastre (albeit without the attached database of resident names and addresses);
- The 2000 Local Environmental Plan (LEP);
- Aerial photographs;
- Drainage layers indicating the location of major pipes, open channels and natural watercourses; and
- Medowie Strategy Plan (future land use)

Land use data for the “existing” and “future” scenarios are shown in Figures 8 and 9 respectively.

[4]Any upgrade of the Campvale WPS would have to occur in tandem with an effort to improve the conveyance of the Campvale Drain which feeds it because at lower stage levels it does appear that restricted conveyance of the Campvale Drain is a limiting factor.

3.8. Medowie Soil Landscape

Given the relatively unique soils found in Medowie and the impact these have on rainfall losses, it is considered that a brief discussion of the Medowie soil landscape is warranted. Information presented herein comes from the Brief as well as the Draft Medowie Strategy.

Four main soil types are identified and these are as follows (in order of predominance):

- Residual soil landscapes – approximately 70-75% of the Campvale and Moffats Swamp catchments. Typically the upper areas, away from the swamps proper. The Brief describes the residual soil landscapes as red and yellow sandy clay structured loams. The Draft Medowie Strategy additionally describes residual soil types in the Local Government Area (LGA) as being typically marginally to highly sodic although no specific mention is made regarding the specific sodicity of soils in Medowie and based on the success of agricultural activities in the area, it seems that Medowie soils may be marginally sodic (if at all sodic). This is of relevance as sodic soils can lead to a breakdown of soil structure (crusting) and hence less infiltration;
- Aeolian landscapes – approximately 20% of the Campvale Catchment and 5% of Moffats Swamp catchment. Very much the lower areas (to south of Medowie, associated with Tomago Sand Beds) and areas below 6.5 m in the CDIA. This landscape consists of wet heath forest and sedge land overlaying sandy peat to sandy loam and then to sand. The fact that the CDIA can stay wet for weeks on end means that this soil landscape must reach a saturation point at which little further infiltration can occur and this accords with observations of hydrological behaviour by Campvale residents. The Draft Medowie Strategy notes that Aeolian soils are prone to waterlogging due to high water tables and this observation agrees with observations made by local residents as well;
- Swamp soils – approximately 25% of Moffats Swamp and very little in Campvale Swamp catchment. Limited to areas that experience very poor drainage consistently through the year. Typically highly acidic and highly sodic. Given this soil type is at the downstream end of Moffats Swamp it is less critical to considerations of likely losses in modelling undertaken; and
- Lacastrine Soil Landscapes – approximately 10% of the Campvale catchment. Limited to the main drainage path and associated floodplain from approximately 500 m upstream of Ferodale Road to one kilometre downstream of Ferodale Road. Consists of pedal loam coarse sandy topsoil overlaying mottled clay and this soil landscape is prone to waterlogging.

With respect to infiltration the following then can be said generally:

- Many of the soils found in the study area (Lacastrine, Swamp and Aeolian) are prone to water logging (via interaction with perched water tables/saturated sub-surface soils) but may account for high initial losses prior to reaching a water logged state; and
- Residual soils, presuming they are relatively non-sodic in Medowie, likely have an ability to absorb rainfall on a continuous basis given their partial sand and loam composition and good structure. For very high intensity storms it would be likely to see rainfall rates exceed infiltrative capacity however. The continuous infiltrative capacity of these soils could be indicated by field measurements given the importance of likely infiltration rates

to design flood outcomes, particularly in the Campvale catchment this is an overall recommendation, i.e. that soil testing for infiltration capacity be carried out over a range of representative areas for an extended period of time at regular intervals.

To more precisely state how the soils may absorb rainfall specific soil testing would be required as would infiltration tests at a large number of locations likely to encounter the various soil types. Generally however relatively large losses (compared to NSW standards of 15 mm initial loss and 2.5 mm/h continuing loss for example) could be expected in the study area.

3.9. Community Consultation

A community consultation programme was implemented with Council's assistance. The programme involved a number of steps and these were:

- A media release advising Medowie residents that a Flood Study was to be carried out, what the goals of the Flood Study were and indicating that those with any interest/information might contact the consultant and/or Council in order to communicate information;
- A questionnaire was then issued to certain residences that were, based on Council experience, likely to be impacted by drainage issues. A total of 508 questionnaires were mailed out to individual residences. Of the 508 issued approximately 17% have been returned. Those questionnaires returned have been compiled into a database so that the information contained within them can be better utilised in reporting as well as model calibration/validation exercises; and
- Council have mounted various items salient to the study on the Council's own website.

Please note that for completeness's sake a copy of the media release and questionnaire are provided in Appendix B. Also photos received from residents are included and have been catalogued on the basis of date of event. Figure 10 shows the location of residents who responded to the mailout. Also, charts summarising various features of the responses received from the community are shown in Figure 11.

Figure 11 shows a total of 88 residents responded to the questionnaire out of a total number issued of 508. This gives a return rate of 17% which is relatively high and indicates that local residents feel strongly flooding and drainage issues, although it could also indicate the targeting was accurate. The awareness of historical flooding events amongst respondents was very high with more than half of respondents recalling the February 2009 and June 2007 events. 26 of the 88 respondents claimed to have never been impacted by any flooding issues.

Major findings from the community consultation campaign are as follows:

- The February 1990, February 2009, April 2008 and June 2007 events are well remembered events during which a number of residents (a total of 62 respondents out of a total possible number of 88 respondents) experienced flooding in one form or another;
- Inundation of properties and roads in Medowie is a major issue for approximately forty households;

- 10% of the total of 62 respondents (who've experienced flooding) and only 1% of the total group surveyed) have actually had flood waters enter their home (water through the house etc);
- Rainfall events which precipitate drainage issues (i.e. inundation of private property) occur relatively often (once every two years at least);
- Within the study area there are at least two different types of "flooding" issues. Some residents, in upper areas, experience relatively brief and shallow inundation of their properties and access roads and this causes concern. Other residents, in the lower southern areas of the study area (primarily in the CDIA), are impacted most severely by the duration of flooding events, with multi-week inundation of the CDIA happening near annually. Whilst this is obviously a serious issue it is noteworthy that analysing/solving it does not lie with the Terms of Reference of this flood study nor with the overall goal of the NSW State Government FPRM program. It is noteworthy that solving the problem for upstream residents[5] in the Campvale catchment may actually result in pondage issues (but not necessarily design flood behaviour) worse in the CDIA and this is an important consideration moving forward into the next stage of the flood planning process which seeks to address mitigation. Note that the same issue does not exist in the Moffats Swamp catchment (although during flood events Moffats Swamp runoff can, if flood levels are high enough, flow into the CDIA exacerbating flooding there);
- Some residents believe Council is failing to meet its planning/infrastructure provision obligations in regard to the drainage issues in Medowie;
- Some residents are concerned that moving forward, expansion of urban areas in Medowie and further sub-division of existing rural and rural/residential land holdings will lead to flooding;
- Some residents believe that the Campvale WPS should be improved in order to solve the problem. These residents cite the age of the pumping equipment as being an issue with respect to its pumping capacity; and
- The capacity of the drain to deliver adequate flow to the Campvale WPS so that the Campvale WPS may maintain peak output was also questioned.

Community responses were accompanied by a large number of photographs (presented in Appendix C) and in some cases observed flood depths at known points. All of this information has been collated into a GIS layer so that it can be:

- Utilised by Council as needs be;
- Presented as a product of this study; and
- Utilised for calibration/validation purposes.

From data obtained during the community consultation process, a total of eight peak flood level marks were able to be surveyed for the June 2007 and February 2009 events. Photos from the community consultation programme which were used to identify these points are presented in Appendix D.

[5]Depending on what form the solution takes.

3.10. Compiled Calibration Data

A variety of calibration/validation data has been collected however much of it is of little use. The key data missing is data that would allow for a reliable assessment of runoff volumes, as well as known peak flood levels (with time and date attached) at key locations such as Ferodale Road. The data is summarised below in descending order of quality/utility with respect to calibration/validation:

- Eight surveyed peak flood level marks (as discussed above) derived from the community consultation process. The surveyed flood marks are mainly from the June 2007 event (six of the eight) but there are also two from the February 2009 event. These flood marks are presented as peak flood levels to Australian Height Datum. This is the best data available for calibration in this study. It is noteworthy, however, that as in any calibration/validation work the accuracy of the work is substantially impacted by the representativeness of available rainfall data, with regard to quantity as well as spatial representativeness;
- Recorded water levels at the Campvale WPS for the June 2007 event only. Whilst this is accurate time series data it is of limited utility since so much of the focus of the modelling is on the upper areas, many kilometres from the Campvale WPS. Also, levels at the Campvale WPS, once CDIA levels drop below approximately 7 mAHD, are ultimately dictated by the “pinch” for which ideal data does not exist. As such Campvale WPS has mainly been used to back calculate “pinch” characteristics;
- A single observation of peak level at the flood depth indicator located on Ferodale Road for the February 1990 event which has neither a time nor a date. The lack of time and date is extremely significant and very much undermines the usefulness of the observed peak level as the February 1990 event was a multi-burst event spread over four days. As such at least four bursts may have caused the observed level of flow and the use of this data is further complicated that at least two of the peaks occurred at times when it is unlikely that people were able to make observations (i.e. late at night and very early in the morning). This issue is further discussed in Section 5;
- Flood marks surveyed previously and supplied to WMAwater by Council (a total of six flood marks on Kirrang Drive were surveyed following the February 2009 event). Note that there appears to be some uncertainty in regards to the measuring of these flood marks (local overland flow versus main drain flow). This issue is further discussed in Section 5; and
- Indicative depths and locations gleaned from the community consultation process were not suitable for survey.

4. HYDROLOGICAL MODELLING

4.1. Introduction

This section will discuss the following topics:

- The direct rainfall approach to hydrology utilised in the modelling, including a comparison of the design peak flow estimates derived using the direct rainfall method with those achieved via more traditional techniques;
- The rainfall applied to the model in calibration/validation runs;
- The possibility of ground water interaction with surface water flooding in the study area;
- The stage/volume characteristics of the two swamp areas within the study (i.e. Campvale and Moffats Swamps); and
- The antecedent conditions for each of the three historical events utilised in the calibration/validation run set with a view to informing design losses.

4.2. Rainfall on Grid Considerations

Given that the direct rainfall approach is relatively new it is considered reasonable that some discussion of the method is presented herein, along with a consideration of the advantages and disadvantages of the method.

Many consulting studies done for both private and government clients, both in Australia and overseas, have been conducted using a direct rainfall approach. Also, within the literature on hydrological/hydraulic modelling there are examples of research which demonstrate the ability of this approach to emulate more established lumped conceptual hydrological models as well as more importantly to match observed data (see Reference 6 and Reference 2 for example).

The chief advantage of the approach is that:

- sub-catchments do not require delineation;
- flows do not need to be artificially applied to certain locations (distributed) as they would need to be given an approach that utilised separate hydrological/hydraulic models;
- routing is based on relatively high resolution topography and the full St Venant equations and so parameterisation of storage/routing processes is not necessary;
- no double routing of flows such as in a joint modelling system; and
- the approach lends itself to the final product which is of course mapped flood levels to inform planning decisions.

In summary whilst direct rainfall can be used to great advantage it is a relatively new method and as such it is best to corroborate the flows derived from the method against alternative methods (i.e. calibration/validation and comparison to other methods used to estimate design peak flow). As such the Section below presents a check of model predictions for design flow versus estimates by other, relatively well known methods.

4.2.1. Check of Rainfall on Grid Methodology

4.2.1.1. Introduction

In order to provide a check of the hydrological approach utilised in the Study, the 1% design peak flow for a test catchment located in Medowie is compared to peak design flow estimates derived by other more traditional methods.

4.2.1.2. Method

Direct rainfall on grid model estimates for peak flow, given application of the 1% AEP 2 hour storm, will be compared to design peak flow estimates derived from:

1. The Probabilistic Rational Method (PRM); and
2. WBNM modelling, i.e. application of traditional lumped conceptual runoff routing models.

Note that in the 2D model and in WBNM, an initial loss of 10 mm has been applied as has a continuing loss of 2 mm. Also, two WBNM results were generated using in the first instance default linearity and time lag parameters (i.e. 0.77 and 1.7 respectively) and in the second instance adjusting the time lag parameter to a smaller value to account for the relatively high mean slope of the test catchment (1.2%).

The outlet of the test catchment is located 500 m upstream of Lisadell Road within the Campvale catchment and the entire area can be described as cleared land which is currently used for pasture.

4.2.1.3. Results

Table 6: Comparison of 1% Peak Flow Estimates for Test Catchment

Peak Flow	Area (ha)	PRM	TUFLOW	WBNM (0.77/1.7)	WBNM (0.77/1.0)
500m US Lisadell Rd.	45	9.6	9.4	6.6	10.3

The results (refer to Table 6) demonstrate that peak flow estimates derived using the PRM, TUFLOW and WBNM are all in agreement, although producing a match with WBNM assumes a lower than default time lag value. Note that given the relatively high slope of the test catchment (mean slope of 1.2%), using a lower than default time lag value seems warranted, although the exact value suitable for use in Medowie would have to be derived via calibration. It is likely that the TUFLOW flow estimates are slightly conservative relative to the typical peak flow estimates that would be derived via WBNM or other similar lumped conceptual modelling.

4.2.1.4. Conclusion

The comparison has found that the direct rainfall approach appears to produce credible peak flow estimates that are certainly within the range of values that might be produced by alternative means. Whilst comparison to estimates made by the PRM and WBNM provide a good indication of the range of values that the direct rainfall approach should produce, a better test of the approach is perhaps calibration/validation of the model to historical events. This work has been carried out and is presented in Section 5 of this document.

4.3. Rainfall Scaling

The Williamtown pluviometer used to inform historical rainfall is four kilometres from the Study location (see Figure 2) and hence more proximate daily read stations either close to or within the study area are used to check, and if necessary scale, the Williamtown rainfall depths. This work is carried out for all of the historical events in the calibration/validation run set.

As a prelude to carrying out the scaling work, it was necessary to establish that there is a relationship (correlation) between the Williamtown, Grahamstown and 7 Wade Close gauges. An assessment of the correlation (using all available data and daily stations only) showed that the correlation between 7 Wade Close and Williamtown RAAF Daily Station was R^2 = 0.84. Other analysis also showed that the correlation between 7 Wade Close and Grahamstown is R^2 = 0.80 and the correlation between Grahamstown and Williamtown is R^2 = 0.84. These values of R^2 indicate a strong correlation between the gauges.

The rainfall scaling process is as follows. In the first instance an assessment is made as to whether or not scaling is required by comparing rainfall depth as recorded at Williamtown pluviometer with gauged depths at more proximate stations. Rainfall data from 7 Wade Close (given its location within the study area this data is given priority) has been compared to the Williamtown rainfall stations (both pluviometer and daily read), located approximately five kilometres to the south. Table 7 presents the comparison for days during the events used in the calibration/validation runs.

From Table 7 the first item of interest is perhaps that the two Williamtown gauges, although presumably located within reasonable proximity to one another, do not always agree with respect to observed rainfall depth[6]. The 1990 event shows a good match for the two Williamtown gauges but for the 2007 event the pluviometer appears to underestimate rainfall depth by 40% and by 23% for the 2009 event.

7 Wade Close shows a good match to the Williamtown daily data for the 1990 event but shows 24% more depth for the 2007 event and 34% more for the 2009 event. It is then of interest to compare 7 Wade Close estimates with those from other daily gauges as this may indicate whether the daily rainfall estimates larger than those recorded at Williamtown are credible.

[6] Pluviograph records utilised in the assessment and presented in Table 7 have been adjusted for comparability to daily rainfall records, i.e. presented as 9 am to 9 am records.

Table 7: Comparison of Limited Daily Stations to Williamtown Pluviometer (mm)

Date	7 Wade Close	Williamtown Daily (061078)	Williamtown Pluvio (061078)
1/02/1990	5.0	3.6	3.6
2/02/1990	25.0	23.8	24.8
3/02/1990	290.0	276	275.1
4/02/1990	190.0	174.6	184.8
5/02/1990	8.0	10.4	0.2
Total	**518.0**	**488.4**	**488.5**
8/06/2007	91.5	48.6	39.8
9/06/2007	175.0	147.0	104.4
10/06/2007	31.0	31.4	17.4
Total	**297.5**	**227.0**	**161.6**
14/02/2009	3.0	2.2	2.6
15/02/2009	205.0	134.8	110.8
16/02/2009	20.0	14.4	9.8
Total	**228.0**	**151.4**	**123.2**

Three particular records from 7 Wade Close presented in Table 7 diverge from the Williamtown daily data and so these values will be compared to other proximate daily stations. Firstly on 8/06/07 91.5 mm is recorded and this is almost double the value recorded at Williamtown. As can be seen in Table 8 for the same date 137 mm was recorded at Grahamstown, 101.4 mm at Tahlee and 122 mm at Clarence Town. Secondly on 9/06/07 175 mm was recorded at 7 Wade Place which diverged again from the Williamtown recording of 147 mm. From Table 8 it can be seen that Raymond Terrace recorded 199.4 mm and, of less relevance due to its distance from Medowie, Nobby Head in Newcastle had 209.8 mm. Thirdly on 15/2/09 205 mm was recorded at 7 Wade Place compared to the 134.8 mm recorded at Williamtown. Table 8 indicates that 195.4 mm fell in Clarence Town, 170.6 mm in Raymond Terrace and 183 mm in Tanilba Bay. It does seem then that the 7 Wade Place data estimates are, based on readings elsewhere by official BOM sites, reasonable. Given this, and 7 Wade Close location within the study area and also the lack of data for the February 2009 event at Grahamstown, 7 Wade Close has been used to scale rainfall recorded at Williamtown pluviometer station. Note that given the good match between 7 Wade Place and the Williamtown gauges no scaling was required for the 1990 event. Scaling was carried out for the 2007 and 2009 events however. Rainfall was increased by 41% for the 2007 event and 81% for the 2009 event.

Table 8: Comparison of Daily Stations

Date	7 Wade Close	Grahamstown (061311)	Williamtown (061078)	Raymond Terrace (061031)	Tanilba Bay (061395)	Tahlee (061072)	Clarence Town (061010)	Nobby Head (061055)
1/02/1990	5.0	3.8	3.6	3.0	No Data	0.0	0.0	1
2/02/1990	25.0	45.0	23.8	41.0	No Data	18.0	79.6	28.8
3/02/1990	290.0	235.0	276	218.0	No Data	206.6	156.4	251.6
4/02/1990	190.0	158.4	174.6	161	No Data	116.8	151.4	140.4
5/02/1990	8.0	18.0	10.4	16.2	No Data	0.0	14.0	2.2
Total	**518.0**	**460.2**	**488.4**	**439.2**	**-**	**341.4**	**401.4**	**424**
8/06/2007	91.5.0	137	48.6	75.6	90	101.4	122.0	21.6
9/06/2007	175.0	103	147.0	199.4	91	99.6	151.8	209.8
10/06/2007	31.0	26.4	31.4	22.0	3	26	22.2	18.8
Total	**297.5**	**266.4**	**227.0**	**297.0**	**184.0**	**227.0**	**296.0**	**250.2**
14/02/2009	3.0	No Data	2.2	3	7.0	1.2	5.6	1.0
15/02/2009	205.0	No Data	134.8	170.6	183.0	164.2	195.4	100.2
16/02/2009	20.0	No Data	14.4	0.0	16.0	7.4	17.4	10.6
Total	**228.0**	**-**	**151.4**	**173.6**	**206.0**	**172.8**	**218.4**	**111.8**

4.4. Ground Water Interaction

Within the Medowie area there is a heightened awareness of groundwater generally owing to the fact that Medowie is located (in part) within the Tomago Sand Beds which are an official source of drinking water utilised by Hunter Water. Unsurprisingly then, when contemplating flooding in Medowie, thoughts may turn to the ground water levels and as to whether or not there is a relationship between the ground water levels and flooding events.

Three ground water stations located within the study area with long term records have been identified. The location of these bore stations is shown in Figure 2 and a data summary for each of the bores is shown in Table 5. Also, the ground water levels over time have been plotted (see Figure 7).

In the context of the actual ground levels at each of the bores it would appear, from both the mean values and the time series shown, that ground water levels are at all times below the surface. The time series plot of ground water levels in Figure 7 indicates a positive correlation between the flood events of February 1990, June 2007 and February 2009. Groundwater levels rise as surface water flooding occurs. Based on a cursory examination it appears that the ground water levels have a strong correlation to surface water flooding events. It is noteworthy, however, that whilst groundwater levels rise, they do not at any stage reach the surface, although the bore hole south of the drain in Campvale does get within approximately 0.5 m on one occasion during 2008. In summary, whilst there is a definite relationship between long flooding event and groundwater levels, it does not appear that groundwater contributions to surface flooding events occur for any of the sites examined.

Based on the data reviewed then it seems unlikely that ground water contributes significantly to surface water within the Medowie area, particularly via some dynamic that contributes to surface water flooding.

4.5. Swamp Stage/Volume Characteristics

4.5.1. Campvale Swamp

The stage-volume characteristics of the CDIA have been assessed (shown on Table 9 Chart 1). This has been carried out using the highest resolution DEM (2 m grid) available. The results indicate that inundation over floor levels within the CDIA is a possibility via the mechanism of the swamp being filled by longer duration flood events[7].

Table 9: Campvale Swamp Stage Volume Relationship

Stage (mAHD)	Volume (m^3)	Runoff Equivalent[8] (mm)
5	34	0
5.5	7,267	0
6	58,923	3
6.5	438,655	21
7	1,598,688	78
7.5	3,427,840	167
8	5,700,000	278
8.5	8,366,000	408

Given Campvale Swamp's restricted outlet capacity the stage/volume characteristics of the swamp are of interest, particularly with respect to building floor levels. The lowest floor levels in Campvale start at approximately 7.5 mAHD (lowest floor levels appear to be on Abundance Road). The runoff equivalent required to achieve a level of 7.5 mAHD is 167 mm (~1% AEP 9 hour event).

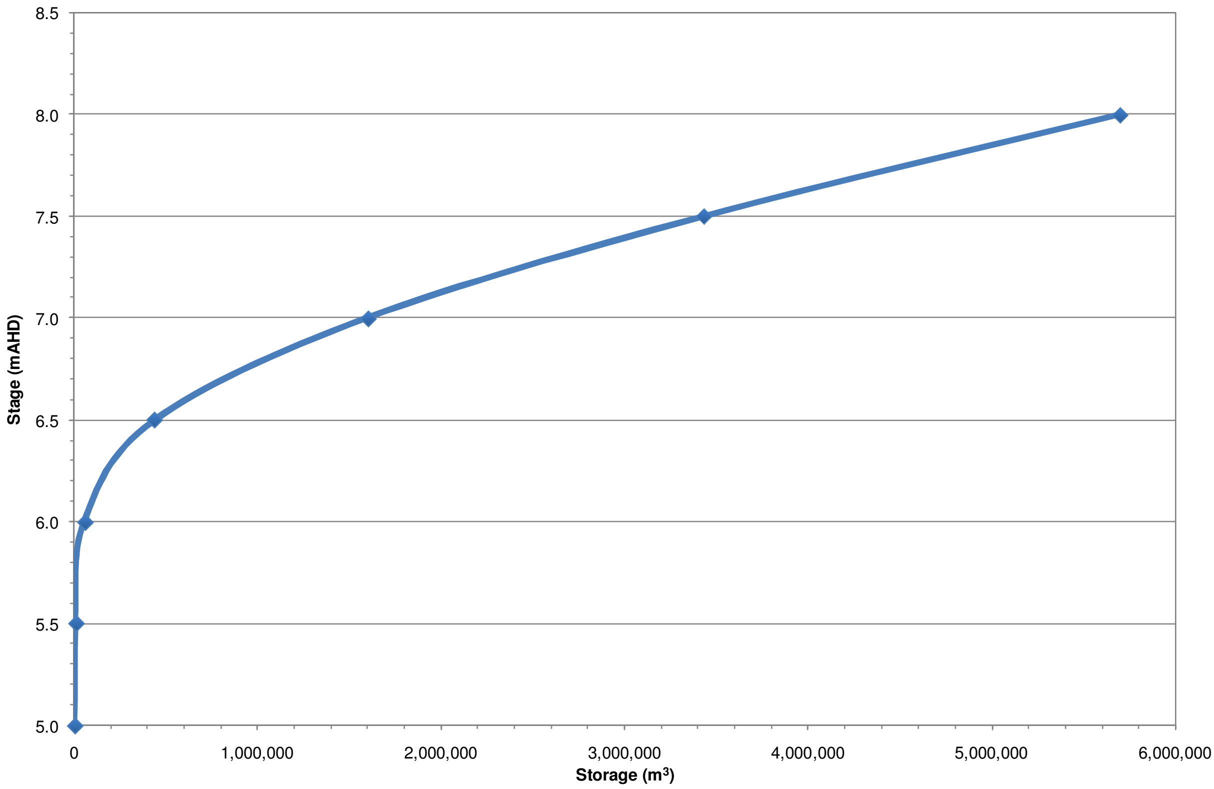
The 1% AEP 72 hour event for Medowie has an intensity of 5.5 mm/h and an overall depth of 396 mm. Given operation at 100% rate Campvale WPS will remove 69 mm of runoff but an assumption of less than 100% pumping may be more appropriate given start up delays as enough water needs to arrive at the Campvale WPS to achieve the trigger water level for all four pumps to operate, and given the "pinch" limiting inflows when CDIA levels are below 7 mAHD. The 2% AEP 72 hour event delivers 360 mm of rain and so would, given relatively wet conditions in the catchment prior to the event, be able to inundate the CDIA to a level approximately equal to 8 mAHD. The 5% AEP 72 hour event delivers 288 mm and as such it might be expected that the CDIA will experience a filling of the swamp up to 7.5 mAHD slightly

[7] Campvale Swamp can, during wet intervals and for to the lack of pumping capacity, average between 6 to 7 mAHD for extended periods.

[8] The runoff equivalent is the amount of rainfall required, minus losses to infiltration and depression storage, etc.

more often than once every 20 years. Two mechanisms exist then for flooding of flood liable property within the CDIA, i.e. flooding from flows moving downstream and also flooding from flows accumulated in the CDIA.

Chart 1: Campvale Swamp Stage - Storage Relationship



4.5.2. Moffats Swamp

The flood capacity of Moffats Swamp (prior to inundation of the lower building levels) is greater than that of Campvale Swamp mainly because the outlet capacity is so much greater. Presented below in Table 10 and Chart 2 is the actual tabulated stage/storage relationship for Moffats Swamp, and runoff depths in order to achieve such levels.

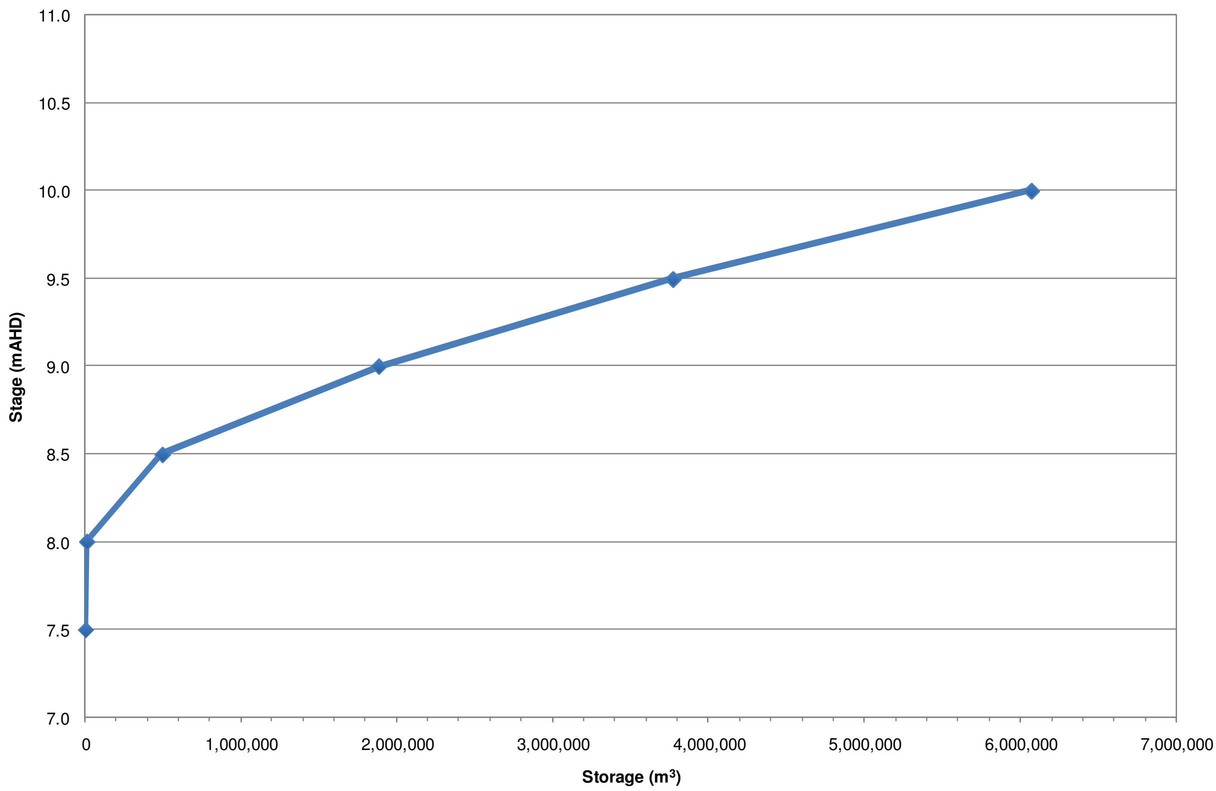
Table 10: Moffats Swamp Stage Volume Relationship

Stage (mAHD)	Volume (m3)	Runoff Equivalent (mm)
7.5	124	0
8	7,793	1
8.5	490,399	30
9	1,882,832	114
9.5	3,767,660	228
10	6,065,520	368

The lowest building floor levels identified in Moffats Swamp are in the order of 9.5 mAHD (approximately the peak flood level of the 1% AEP 24 hour event). This gives a larger runoff equivalent value (compared to Campvale) in Moffats prior to the inundation of the lowest building floor levels. Also outlet conditions for Moffats Swamp are vastly different to those at Campvale. In Moffats Swamp at 8.35 mAHD significant flows will move out of the catchment via the outlet at Racecourse Swamp and into Swan Bay via the spillway located there. At higher stages in the swamp, two further outlets will also transfer water out of the swamp, and overall, as stage increases so does the discharge out of the swamp. This is not the case in Campvale where discharge out of the CDIA is ultimately limited (for large flood events) by the Campvale WPS.

In summary Moffats Swamp is distinctly different to the CDIA in that as flow stage increases, the outlet flow capacity in Moffats increases, whilst in the CDIA the outlet capacity has a strict upper limit equal to the peak pumping capacity of the Campvale WPS. Additionally Moffats Swamp has more holding volume that Campvale with respect to minimum building levels.

Chart 2: Moffats Swamp Stage - Storage Relationship



4.6. Antecedent Conditions

Typically in a flood study the amount of rainfall prior to an historic flooding event may be examined in order to assess what the catchment condition was when the event began, i.e. relatively wet, dry or neutral, as this may influence the decision regarding what losses to apply in the calibration/validation work and then in subsequent design runs.

With regard to assessing the infiltration/rainfall losses there is a lack of data which hampers the work. A gauged time series of pump behaviour/stage is only available for one event (June 2007) for only one catchment (Campvale) and the data is downstream of the pinch. Ideal data for assessing how much of the event gauged rainfall converts to runoff would be a time series of water level upstream of the pinch, or ideally, a rated discharge gauge at Ferodale Road or further upstream. Such data is unfortunately unavailable and as such general rainfall prior to and during the calibration/validation events is discussed in order to ascertain whether or not significant losses were likely to have occurred during the events. A significant focus will be placed on the February 1990 event because it contained a large spatially consistent rainfall depth and because losses appear to have been quite substantial. Please note that in considering the antecedent conditions it may be of some help to examine the plot of pluviometer data shown for all historical events modelled in Figure 12, particularly in regards to the discussion of the February 1990 event.

4.6.1. February 1990 Event

The 1990 event occurred during the period February 1st – 5th 1990. The January prior had experienced below average rainfall with just 61 mm (mean January rainfall is 90 mm). Prior to the deluge that occurred on February 2nd, 5 mm fell on the last day of January and then 25 mm on the 1st of February. No gauged data exists to establish to what degree runoff had or had not begun to occur prior to the 2nd, which is when the first of the three or four major bursts of the storm fell.

What is known however is that the flood level in the CDIA did not exceed approximately 7.5 - 7.8 mAHD. If it had, houses in Abundance Road (lowest ground levels approximately 7.7 mAHD), would have had flood waters near or over floor level and this likely would have been reported. Also if the CDIA level had been much higher than 7.8 mAHD it seems likely that this would have been noted from Ferodale Road at least. Given that the February 1990 event contained 450-500 mm of rainfall[9] and a runoff equivalent of 167 mm will cause CDIA levels to reach 7.5 m (whilst a runoff equivalent of 278 mm will cause CDIA levels to reach 8 mAHD) losses must have been substantial. Some runoff from the long event was removed by Campvale WPS. However given the Campvale WPS peak capacity of 23 mm per day (runoff equivalent) and the fact that the main rainfall occurred over a period of 48 hours this may be as little as 30 mm when peak levels in CDIA were reached. By deduction losses over the event likely add to approximately 200 - 250 mm (i.e. 450 mm of rain, 30 mm was pumped and approximately 170-220 mm eventuated as runoff). Whether such substantial losses were due to the ability of the Aeolian soils in the CDIA to absorb large amounts of rainfall or were due to a combination of front end losses and reasonably high continuing losses (in the order of 5 mm/h) is not known as the timing of peak flood heights throughout the extended event are unknown. The distribution of the loss between initial and continuing is of great significance to the modelling however, particularly when the impact of future development is considered. A consideration of the soil landscapes (refer to Section 3.8) indicates that if the large losses in the event are attributable to continuing loss, then this continuing loss is occurring in the residual soil landscapes not in the other soil landscapes which are prone to waterlogging. If proposed development proceeds which was substantially reduces the ability of the mid to upper catchment residual soil

[9] Note the degree of confidence in the rainfall estimate is quite high. Between February 1st and February 5th 1990, 518 mm was recorded at 7 Wade Close, 488 mm recorded at Williamtown Daily, 489 mm at Williamtown Pluviometer, 460 mm at Grahamstown, 439 mm at Raymond Terrace (based on daily records presented in Table 8).

landscapes to absorb rainfall over events such as occurred during the February of 1990, inundation extent and level in the CDIA will likely be exacerbated.

4.6.2. June 2007 Event

Runoff for the June 2007 event began to flow to the Campvale WPS during the early morning of June 8th (based on gaugings taken at the Campvale WPS by Hunter Water). Monthly rainfall prior to the event had vary above and below average, depending on the month. Rainfall in March had been near average whilst the rainfall in April had been above average (180 mm versus the average of 110 mm). Rainfall in May had been below average with only 69 mm of rainfall falling relative to the mean figure of approximately 115 mm. Of particular interest is that whilst 56 mm of rain fell on May 10th, no more rainfall was recorded until June 7th.

29 mm was recorded as falling between 9 am on June 6th and 9 am on June 7th. The following 24 hours show that 91.5 mm fell (i.e. between 9am June 7th and 9 am June 8th). It appears based on the Campvale WPS gauging data (see Figure 6) that runoff began to arrive at the Campvale WPS from the morning of June 8th and so it is quite likely that at least 30 - 50 mm of rainfall was absorbed into the catchment prior to runoff occurring.

4.6.3. February 2009 Event

Daily rainfall records indicate that January rainfall was well below average at 19 mm (versus average rainfall for January of 90 mm). However, prior to the actual event which occurred on February 15th a total of 49 mm of rainfall had fallen over the preceding four days. As such it would be reasonable to say that the catchment was relatively wet prior to the occurrence of the main rainfall burst on February 15th. A rough estimate of losses prior to the event would be in the order of 30 - 50 mm.

4.7. Calibration/Validation Losses

Losses were applied to the input rainfall prior to model runs for each of the calibration/validation events. As such the rainfall depths applied are net of any losses and are more accurately described as rainfall excess. Table 11 shows rainfall loss values adopted for the calibration/validation events.

Table 11: Losses used in calculating rainfall excess

Event	Initial Loss (mm)	Continuing Loss (mm/h)
Feb 1990	135	5
June 2007	50	5
Feb 2009	30	5

As discussed in the previous section on antecedent conditions it is difficult to precisely establish what actual losses occurred in each of the calibration/validation events. Results do indicate that following high initial losses continuing loss rates stay reasonably high although, as discussed in the previous section, there is some uncertainty about this which a lack of gauged data makes difficult to resolve. Various combinations of losses were used and given the relative insignificance of the roughness values (refer to Section 5.2.4), losses became a significant calibration parameter. In the end some consistency for continuing loss was sought between the various runs and a value of 5 mm/h did optimise results generally. As discussed in the previous section, a high initial loss does seem reasonable, particularly in Campvale given the losses that seemed to have occurred in the February 1990 event.

Sensitivity runs discussed in Section 5 of this report examine what impact various losses have on design event modelled peak flood levels. Based on the calibration runs undertaken it does seem that upstream of Ferodale Road there is generally little sensitivity to the continuing loss rate used.

4.8. Design Losses

The initial loss used in design runs is 10 mm whilst the continuing loss used is 2 mm/h. These losses are based on Reference 1. The selected losses are low, particularly with regard to initial loss, however in the absence of better information (such as might be achieved if the catchment was gauged) it is best to take a conservative approach. It is important to note that whilst these losses will likely produce conservative design flood results they will not necessarily produce conservative estimates of the impact of urbanisation.

Note that design loss values have been scaled for each land use type, according to its imperviousness degree, based on the loss values previously mentioned (refer to Figures 8 and 9 for the land use type applied).

4.9. Probable Maximum Precipitation (PMP) Estimation

The PMP was derived using the Generalised Short-Duration Method (GSDM) (Reference

Reference 10). The GSDM is utilised for rainfall durations of up to 6 hours in small catchments (less than 1000 km^2).

Some of the parameters estimated for the derivation of the PMP are:

- The catchments were found to be topographically smooth;
- The Elevation Adjustment Factor was 1;
- The Moisture Adjustment Factor was 0.74; and
- The rainfall was spatially distributed using ellipses A, B and C.

Table 12 presents the estimated PMP values (rounded to the nearest 10 mm) for the three ellipses that enclose the Campvale and Moffats Swamps catchments.

Table 12: Estimated PMP depths

Time (h)	Ellipse A (mm)	Ellipse B (mm)	Ellipse C (mm)
0.25	170	150	130
0.50	250	220	190
0.75	310	280	250
1.00	360	330	310
1.50	420	380	360
2.00	460	430	400
2.50	500	450	430
3.00	520	470	450
4.00	570	520	500
5.00	620	560	540
6.00	650	600	570

It was appropriate to do the PMP estimation for both catchments at the same time, rather than independently, as in a very large flood event such as the PMF, Moffats flood flow does run into the Campvale catchment.

5. HYDRAULIC MODELLING

5.1. Introduction

The hydraulic model traditionally converts applied flow (discharge generated by a hydrological model) into flood levels and velocities. In the approach used herein where the hydraulic model also routes rainfall excess to create discharge (i.e. the traditional work carried out by hydrological models) the hydraulic model is the only model run. The hydraulic model in this study takes an applied rainfall depth (net of losses) and routes it to create flood extent, level and velocity information.

The hydraulic model used in this study is TUFLOW. TUFLOW is a finite difference grid based 2D/1D hydrodynamic model which uses the St Venant Equations in order to route flow according to gravity, momentum and roughness. TUFLOW is ideally suited to the study because it allows for the utilisation of breaklines, at differing resolution to the main grid, in order to ensure the correct representation of features which may affect flooding (features such as roads, embankments, levees etc). Additionally TUFLOW allows for the incorporation of 1D elements into the 2D domain and allows such elements to function dynamically within the 2D grid. This suits the study as it facilitates the inclusion of channelised flow within the context of a medium resolution 2D approach.

In this Section the model build will be discussed as well as the calibration/validation of the model.

5.2. Model Build Process

The model build process can be best visualised as a grouping of specific spatial layers of information. In running the model, use is made of these various layers of information. The most fundamental layer in the process is the DEM followed closely by other fine scale topographical data such as cross-sections, and also structure information. Figure 13 shows the DEM and the location of cross-sections.

Note that Moffats swamp, unlike Campvale, does not include a well defined channel and as such no cross-sections were needed for modelling of the Moffats catchment.

5.2.1. Breaklines

Breaklines have been described in order to ensure the correct representation of road crowns and other features such as levees which may impact on the routing of flood flows. The digitised breaklines are shown in Figure 14.

The breaklines are used to extract height data from topographical features represented in the high resolution grid. This ensures that controlling heights can be extracted from the best available topographical data set and then applied to the grid utilised in hydraulic modelling (which at 5 m is at a coarser resolution than the source DEM with its 2 m resolution).

5.2.2. Structures

Hydraulic structures have been implemented in the 2D model by a variety of means as described below. The location of structures can be seen in Figure 14:

- Culverts – culverts are inserted as 1D elements which are dynamically linked to the 2D model at an upstream and downstream location which may consist of one or more grid cells;
- Bridges – bridges are typically represented as a combination of an underflow structure (culvert) and an overflow structure (broad crested weir);
- Campvale WPS –represented approximately as a stage/discharge relationship which is in turn based on the switch on pumping rates/stage heights supplied by Hunter Water; and
- Dams/Retarding Basins – the dam wall is represented as a broad crested weir whilst if present, low flow drainage is represented by 1D elements linked dynamically to the 2D model.

5.2.3. Boundary Conditions

5.2.3.1. Rainfall Excess

As discussed in the previous section rainfall excess (rainfall minus losses) is applied to the hydraulic model. The loss values used are discussed in Section 4.7.

5.2.3.2. Downstream Boundaries

A number of boundaries are defined within the Campvale and Moffats Swamps areas. These are as follows:

- Campvale WPS – a stage/discharge boundary imitates the pumping station's four pumps which are height activated (Table 13). Note this is the only outflow boundary within the Campvale model domain. In order to enhance model stability the rather abrupt cut-in and out nature of the pump has been smoothed somewhat. This will have no noticeable impact on model results;
- Championship Drive – within the Pacific Dunes golf course Championship Drive is relatively low (invert level 9.5 mAHD) and is reported as being a location where flood flows from Moffats Swamp will potentially cross over into the Campvale Catchment. As such this location is a possible inlet boundary for the Campvale model whilst for the Moffats Swamp model it is a possible outlet boundary. Given the two models are run simultaneously as part of one larger model these possible cross-catchment flows are implicitly dealt with;
- Swan Bay Outlet – at Swan Bay a trapezoidal outlet exists with an invert level of 8.35 mAHD. On the downstream side of this structure the water level is set to a level approximately equal to the invert (plus 0.1 m) of the downstream channel. This ensures that outflow from Moffats Swamp is inlet controlled which is appropriate given that tidal

fluctuations will not impact on the capacity of the Swan Bay outlet; and

- Salt Ash Outlet – the Salt Ash outlet is modelled as three box culverts of dimension 2.4 m by 1.2 m. As per the Racecourse outlet the downstream water level is fixed at the level of the downstream channel plus 0.1 m to ensure that flow through the structure is inlet controlled. The invert of the Salt Ash outlet is 8.7 mAHD.

The locations of boundaries in the model setup can be seen in Figure 14.

Table 13 : Switch On/Off Stage Levels for the Campvale WPS

Pump #	Stage (on) (mAHD)	Stage (off) (mAHD)
1	5.45	4.90
2	5.60	5.02
3	6.10	5.18
4	6.50	5.30

5.2.4. Roughness

The roughness map is based on the land use data supplied by Council (see Figure 8). Roughness values ascribed to specific land uses are based on Chow (1959) as well as previous experience with roughness values in a 2D model and confirmed by the calibration process.

The different land use categories and the roughness values used to describe them are as described in Table 14 below. Note that, besides in main flow paths upstream of Ferodale Rd within Campvale, roughness values are likely to have little impact on results due to flow velocities being low, i.e. less than 1.5 to 2.0 m/s.

Table 14: Roughness Values Used in Modelling

Land Use	Manning's 'n' Roughness
High density trees	0.08
Creeks	0.035
Golf courses - Grass / Open areas	0.05
Quarry	0.05
Urban / housing properties	0.045
Pasture / cultivated land	0.05
Roads	0.02
Lakes / ponds	0.03
Open areas - low vegetation	0.04
Creeks with in-bank dense vegetation	0.08
Concrete culverts	0.015
Open drains in 2d	0.02

5.3. Calibration/Validation

The calibration and validation work has been carried out with three flood events. In the proceeding sections each of the events is briefly described as is available calibration data utilised in the work. Roughness values were not altered from values originally established. The chief calibration parameters used were rainfall losses and, in cases where rainfall data from Williamtown pluviometer was unlikely to be representative, rainfall scaling. Also the calibration/validation data provided ideal feedback into the model build process. Rainfall losses used in the events are summarised in Table 11.

5.3.1. Calibration Event – June 2007

The June 2007 event was chosen as the calibration event because it is a large event and has the most observations available. Community consultation responses indicate that during the June 2007 event some severe flooding occurred with three properties experiencing over floor flooding.

The June 2007 event was primarily limited to one extreme rainfall burst, although this was complemented by low intensity rainfall prior to and post the main burst (see Figure 12). As indicated by Figure 3 the scaled rainfall used for the June 2007 event is approximately a 10% AEP event 3 hour duration (which approximates to the duration of interest with respect to flooding upstream of Ferodale Road.

Calibration data available for the June 2007 event is as follows:

- Six surveyed flood marks (these are community identified flood marks that were able to be surveyed to mAHD);
- Gauged water level data at the Campvale WPS (the value of which is substantially lower by it being gauged data downstream, not upstream of the “pinch” and hence not representative of levels within the CDIA; and
- 11 flood depths observations collected during the community consultation process.

5.3.1.1. Surveyed Flood Marks

Surveyed flood marks were used as the primary source of calibration data. That is, model parameters were modified in order to optimise the match to these points. Given that roughness settings had little impact on flood levels, the main parameters modified were rainfall and rainfall losses. As discussed earlier in Section 4.3 the rainfall was scaled due to a significant discrepancy between the Williamtown pluviometer and locally gauged rainfall.

Table 15: 2007 Calibration – Comparison of observed and modelled flood heights

Respondents Address	Observed Height (mAHD)	Model Height (mAHD)
Yulong Oval	8.2	8.2
59 Kula Dr	13.6	13.6
8 Ballat Cl (near drain)	12.1	12.0
8 Ballat Cl (back of house)	12.0	11.9
8 Ballat Cl (front)	11.9	11.8
31 Lewis Dr	14.0	13.6

As can be seen in Table 15 and Figure 16 the match between the model and the observed points is very good. Three of the points are matched precisely whilst two vary by 0.1 m. Only one of the six is not matched well and this point occurs at 31 Lewis Drive. The reason for the mismatch, on close inspection, appears to be that the modelled ground height at 31 Lewis Drive is lower than the ground height as indicated by the flood mark survey. This same phenomenon is noted in the February 2009 event for several points on Kirrang Drive. It is likely that one of the following things has happened:

- the ALS data has measured a lower ground value at the location than surveyed;
- the ALS data for the location does not exist as trees obscured the reading and hence an interpolation has occurred over the ground resulting in a lower ground height; or
- in aggregating the original 2 m ALS grid the model has found (by averaging several ALS cells) a lower mean height for the ground at 31 Lewis Drive.

Although the flood height is not well matched at 31 Lewis Drive the model does still indicate flooding at the mismatched location and if the peak flood depth is used to assess over floor flooding (presuming a slab on ground) then above floor flooding would still be identified.

Overall it can be said that the match between modelled flood heights and observed shows an extremely good fit.

It is to be noted that many different model iterations were modelled and results indicated a general lack of sensitivity to changes to parameter value changes within reasonable ranges. A key issue was the input rainfall however and as discussed in Section 4.3 dealing with rainfall scaling, the applied rainfall utilised the Williamtown Pluviometer temporal pattern and then was scaled so that the overall rainfall depth matched the average of Williamtown and 7 Wade Place. A key calibration parameter was certainly the input rainfall in this case.

5.3.1.2. Campvale WPS Gauged Data

Figure 15 shows the match between observed stage at the Campvale WPS and modelled stage. Overall the match is not ideal however it is to be noted that Campvale WPS is a long way downstream of the more critical residential areas and also that the match shown is mainly a product of the modelling of the upstream pinch.

The peak observed level is 6.5 mAHD whilst the model produces a peak level of 6.85 mAHD within the modelled period. The overall shapes of the two stage plots is poor in that the observed data shows how the pinch fails to deliver enough discharge in order to keep Campvale WPS at peak pumping capacity in the early part of the event whilst the model fails to emulate this. The discrepancy is far from critical to the overall model's accuracy (particularly in more relevant upstream residential areas) as levels at Campvale WPS do not directly impact on flood levels upstream of Ferodale Road.

Overall, the results indicate a reasonable match considering the above. However, the quality of the match is reduced because flow through the pinch is likely to be 10-20% (on a base figure of approximately 6 m^3/s) too much for lower stages of flow.

5.3.1.3. Community Consultation Observed Depths

The community response points cover a wide area including both the Campvale and Moffats Swamps areas as well as both the areas upstream and downstream of Ferodale Road within the Campvale catchment. The accuracy of the points is suspect and hence they are described as indicative only. The accuracy is suspect because they are:

- Depths not flood heights. Depths are very difficult to match precisely as terrain is variable over very short distances (less than metres sometimes) whilst the model grid is based on a 4 m^2 ALS grid and is then aggregated to an (at minimum) 25 m^2 grid (5 m by 5 m);
-
- Unless people measure the depth, depth estimates are prone to error in judgment as flood waters are almost always opaque;
- Not surveyed; and also
- Have no exact location i.e. no coordinates were supplied, instead locations were described as either “in the shed”, out the back of the house, on the corner of the property, etc.

The match between the community response points and the modelled results is shown in Figure 16 and in Table 16 below.

Table 16: 2007 Calibration – Comparison of observed and modelled depths

Respondents Address	Observed Depth (m)	Model Depth (m)
5 Ballat Cl	0.3	0.2
6 Wellard Cl	1.0	0.3
673 Medowie Rd	1.0	0.7
56 Kula Rd	0.1	0.1
57 Kula Rd	0.1	0.1
8 Fairlands Rd (rear of property)	0.5	0.3
8 Fairlands Rd (rear of property)	0.6	0.4
60 Lisadell Rd (front of property)	0.3	0.2
4 Fairlands Rd	1.5	0.7
60 Lisadell Rd (front of property)	0.3	0.2
9 Kirrang Dr	0.5	0.4

Generally the depth comparison shows a good fit. Importantly for each location identified as being flooded in the community consultation data the model concurs. Two of the eleven observations are not well matched and besides model error this could be due to overestimated depth estimates or small sub-grid features (such as isolated depressions). Generally the match is good though and may be seen as a further endorsement of the models performance.

5.3.1.4. Peak Flood Height Profile for Main Drain

The peak flood level profile for the main drain, from Kula Road to the Campvale WPS (a drainage reach of approximately 7,800 m) shows only two calibration points and both of these are community consultation depth estimates (converted to levels using ALS data). As can be seen (Figure 20) the match to the approximate points is reasonable. Of more interest perhaps is the fact that flood levels up to 1.5 km upstream of Ferodale Road appear to be prone to backwatering and as such Ferodale Road can be described as a very significant control. This is mainly due to the limited capacity of the culverts beneath Ferodale Road.

5.3.2. Validation Event - February 2009

The February 2009 event was chosen for inclusion as a validation event because it was reasonably large but also because it was recent. Unlike the June 2007 event and the February 1990 event the February 2009 event mainly consists of relatively consistent low intensity rainfall spread over a period of approximately twelve hours (see Figure 12).

From the community consultation process 12 indicative depths were found and one of these was able to be surveyed (31 County Close). Additionally, immediately following the February 2009 event, Council surveyed eight flood marks on Kirrang Drive and this data is also available.

5.3.2.1. Surveyed Flood Marks

A total of nine surveyed flood marks exist and the match between these surveyed flood marks and modelled heights is shown below.

Table 17: 2009 Validation – Comparison of observed and modelled heights

Respondents Address	Observed Height (mAHD)	Model Height (mAHD)
13 Kirrang Dr*	8.7	8.4
5 Kirrang Dr*	8.7	8.2
3 Kirrang Dr*	8.6	8.4
Hardware Store*	7.7	8.3
8 Kirrang Dr*	8.2	8.2
9 Kirrang Dr (Garage)*	8.7	8.6
9 Kirrang Dr (Rear Shed)*	8.6	8.2
7 Kirrang Dr*	8.7	8.3
31 County Cl	13.5	13.4

*Observations and subsequent survey not carried out under auspices of this study

Of the nine surveyed flood marks:

- One is matched precisely;
- One modelled height is much higher than the observed value. It is noted that a similar value was apparently observed at the same location for the June 2007 event. In each instance the observed flood height is extremely low, indicating that Ferodale Road is not overtopped and that all flood flow moves through the culverts (given the location of the point immediately upstream of Ferodale Road). The figures (B1-B3) in Appendix B indicate how unlikely this scenario is and for this reason this surveyed flood mark should be ignored as likely being an incorrect observation (probably due to timing);
- Three are matched within 0.2 m (and two of the three match within 0.1 m) and

- Four of the nine flood marks are underestimated by between 0.3 and 0.5 m.

With respect to the mismatch between the four flood levels surveyed on Kirrang Drive and modelled results an inspection of model ground levels at the addresses shows that model ground levels are approximately 0.3 m below those achieved by survey. As such it is likely that one of the following issues exists:

- the ALS data has measured a lower ground value at the location;
- the ALS data for the location does not exist as trees obscured the reading and hence an interpolation has occurred over the ground resulting in a lower ground height; or
- in aggregating the original 2m ALS grid the model has found (by averaging several ALS cells) a lower mean height for the ground at the address is being used than that surveyed.

If grid levels in the model for the addresses mentioned in Kirrang Drive are adjusted by 0.3 m the match for the entire set of points (bar the obviously spurious point upstream of Ferodale Road) becomes quite good. It is noteworthy that these floodmarks are not likely not be associated with backwater, or flow in the main channel (at the rear of the residences) but rather they are storm water levels due to local overland flow water shedding off of Kirrang Drive and onto the properties (refer to Appendix C for comments made by local residents). This can be said with some confidence since the flood profiles on the main drain (see Figure 21) show that peak flood levels are:

- controlled by Ferodale Road (backwater effects from Ferodale Road is evident); and
- approximately 0.1 m higher than Ferodale Road peak heights only.

The February 2009 event was a relatively small event and the modelled peak flood level at Ferodale Road of 8.1 mAHD seems reasonable and ties in with results from previous studies, as well as other events modelled herein. For main drain levels to be in the order of 8.6 mAHD at Kirrang Drive, the Ferodale Road level would be 8.5 mAHD and this is extremely unlikely based on all data collected and reviewed to date.

Note that the apparent tendency of ground levels in the model to be underestimated relative to detailed land survey does require some further assessment prior to proceeding with the damages assessment to confirm that it is not a widespread issue likely to undermine above floor level flooding estimates in design runs.

5.3.2.2. Community Consultation Observed Depths

As with the June 2007 event, the community response points for the February 2009 event cover a wide area including both the Campvale and Moffats Swamp areas as well as both the areas upstream and downstream of Ferodale Road within the Campvale catchment. The accuracy of the points is suspect and hence they are described as indicative only (refer to points made in Section 5.3.1.3).

The match between the community response points and the modelled results is shown in Figure 17 and in Table 18.

Table 18: 2009 Validation – Comparison of observed and modelled depths

Respondents Address	Observed Depth (m)	Model Depth (m)
4 Fairlands Rd	1.5	0.6
31 County Cl (rear of property)	0.3	0.2
31 County Cl (swamp)	0.4	0.4
7 Kirrang Dr	0.2	0.1
8 Fairlands Rd (rear of property)	0.6	0.3
8 Fairlands Rd (rear of property)	0.5	0.2
57 Kula Rd	0.1	0.1
13 Kirrang Dr	0.2	0.1
6 Wellard Cl	0.3	0.1
673 Medowie Rd	0.3	0.5
18 Kula Rd	0.1	0.1

Overall the depth comparison shows a very good fit. Importantly, for each location identified in the community consultation as being flooded the model concurs. Only one of the eleven observations is not well matched[10]. Generally however the match is good and should be seen as an endorsement of the models performance and suitability for design flood estimation.

5.3.2.3. Peak Flood Height Profile for Main Drain

As evident in Figure 21, the key feature of the profile is once again the importance of Ferodale Road (and other upstream culverts such as the Kirrang Drive crossing) on upstream flood levels. Also it is of interest, as pointed out earlier, to note the height of the surveyed Kirrang Drive flood marks versus the main drain flood level.

5.3.3. Validation Event – February 1990

The February 1990 Event was chosen for inclusion in the validation process for the following reasons:

- With approximately 450 mm of rainfall over four days (most concentrated into 48 hours) it is a very large event (in excess of 1% AEP for durations exceeding 16 hours) and significantly different to the other modelled historical events which have total rainfall depths of below 300 mm;
- It was remembered by some community members; and
- A peak flood level for the event at Ferodale Road was available from the 1995 PSC study (Reference 2) (culvert lacking date/time information).

Being a large event with respect to rainfall depth meant that the February 1990 event provided a unique opportunity to assess how much rainfall is converted to runoff in Medowie, particularly in

[10]This same address showed a discrepancy between modelled and observed depth for the June 2007 event and so it may be that a minor drainage feature exists on the property which is beyond the resolution (and scope) of the model.

the Campvale catchment given its limited outlet capacity. This issue is discussed at length in Section 4.6.

The available validation data for the event consisted of the following:

- A peak flood level at Ferodale Road gauge board of 7.9 mAHD. Note that no date or time is provided for the observation and that the failure to assign a date or time to the observation becomes problematic when the rainfall (from Williamtown pluviometer) is examined as there are numerous large rainfall bursts within the event (see Figure 12); and
- Five depth observations derived from the community consultation process.

5.3.3.1. Peak Level at Ferodale Road

As stated above and earlier, the 1995 PSC study noted that an observation taken of the peak level at the Ferodale Road gauge board was 7.9 mAHD. Following carrying out extensive model runs, reviewing the 1990 rainfall (see Figure 12) and also looking at the likely timing of peak flow at Ferodale Road this observation becomes less credible (not as an observation per se but rather as an observation which claims to identify the peak flood level observed at the upstream side of Ferodale Road throughout the four day February 1990 event). The issues are as follows:

- As can be seen in Figure 12 the February 1990 event has numerous rainfall bursts, at least three of which (12 pm 2/2/90, 9 pm 3/2/90, 5 am 4/2/90) may produce credible claims to be the “peak” flood level at Ferodale Road (according to modelling carried out). The issue then becomes which one of the bursts produced the 7.9 mAHD observed although this is complicated by the fact that the gauge at Ferodale Road is not automated and the identity(s) of observer(s) are unknown;
- Relating to the above point the last two rainfall bursts produce peak flows at Ferodale Road at approximately 10 pm on 3/2/90 and 6 am on 4/2/90 so it is unlikely flood behaviour resulting from these bursts was observed by anyone; and
- Whilst the initial burst in the rainfall sequence which begins at around 10 am on 2/2/90 could have produced a peak water level at Ferodale Road that was observed, given the high rainfall losses that presumably occurred during the 1990 event (as discussed extensively in earlier sections) it’s unlikely that the level resulting at Ferodale Road from the first burst was actually as large as the flood levels produced by the later two bursts.

In summary then it is unlikely that the natural peak flood level at Ferodale Road for the February 1990 event was observed at all. Many runs have been carried out to investigate this issue (using a range of losses from initial loss/continuing loss of 45/5 to 350/5) and values produced at Ferodale Road range from 8.18 mAHD to 8.11 mAHD. Overall it appears the actual peak was higher than 7.9 mAHD but occurred at a time when observation was less likely.

As such no comparison is made between the model result and the observed peak value noted by the PSC 1995 study, it is noted rather that a peak flood level of 8.13 mAHD resulted from the third of the main rainfall bursts in the event (5 am on 4/2/1990 in Figure 12).

5.3.3.2. Community Consultation Observed Depths

As with the other community consultation derived observed depth data sets the results presented below (and in Figure 18) are indicative only due to the previously discussed limitations of the depth observations. The results do indicate however a reasonably good match and certainly where flooding was observed the model has matched this behaviour. Overall the results below indicate that once again the model is doing a reasonable job of emulating historical flooding behaviour.

Table 19: 1990 Validation – Comparison of observed and modelled depths

Respondents Address	Observed Depth (m)	Model Depth (m)
19 Wilga Rd	0.3	0.2
8 Fairlands Rd (rear of property)	0.5	0.3
8 Fairlands Rd (rear of property)	0.6	0.3
9 Kirrang Dr	0.5	0.8

5.3.3.3. Peak Flood Height Profile for Main Drain

The profile result for the 1990 event shows a good match with community consultation derived heights (converted from depth using ALS data) (see Figure 22).

Of greatest interest perhaps is that due to the large runoff depth during the event (approximately 220 mm) the level in the CDIA reaches approximately 7.8 mAHD, with water backwatering from the Campvale WPS. This profile result demonstrates the capacity of the CDIA to act as an enormous reservoir given enough runoff as the Campvale WPS is unable to ramp up its capacity as the CDIA level increases. It also demonstrates that even for relatively high levels in the CDIA that there is a lack of connectivity between flood behaviour upstream of Ferodale Road and the CDIA. For flooding purposes the two systems are quite separate albeit up to a point.

5.3.4. Calibration/Validation Summary

Generally the calibration/validation process shows that the model is able to emulate a range of reliable calibration points.

The main source of reasonably reliable calibration data are surveyed flood marks from the community consultation process and generally speaking, the model has done an excellent job of matching these. In specific instances significant difference is noted for limited points forming part of the surveyed flood mark set and where these differences have occurred some attempt at providing credible explanations for the divergence has been given. In particular the calibration/validation process has noted that modelled ground levels at residences do not always accord well with ground levels as determined by specific land survey, hence, is a timely reminder of the need to survey indicative house floor levels in order to carry out accurate and reliable damage assessment and work.

The model has also matched a relatively large and spatially diverse set of indicative flood depths relatively well with good matches in 20 out of 23 cases.

Unfortunately no gauged data exists (barring Campvale WPS data which, situated as it is downstream of the Campvale Drain "pinch" is not useful) as such data would be a great asset to the study.

The lack of an ideal calibration/validation dataset makes sensitivity testing of the model more important than it otherwise might be, so that some understanding can be gained of how design flood levels are likely to be impacted by changes in model parameters. The ensuing section presents sensitivity testing which has been carried out using the 1% AEP 2h event.

5.3.5. Sensitivity Runs

5.3.5.1. Introduction

As per the brief a number of sensitivity runs have been carried out in order to assess the sensitivity of the model to various parameters such as roughness and structure losses. The total list of runs as required from the brief is as follows (note that the 1% AEP 2h event has been used for sensitivity runs[11]):

- An increase in modelled roughness of 20%;
- A decrease in modelled roughness of 20%;
- An increase in the energy losses applied to hydraulic structures (culverts for example);
- Changes in the ground water height; and
- Higher losses used in determining rainfall excess.

[11] This run was used as it is a significant design event, likely to be close in terms of flow magnitude to the flood planning event, and also because of its short duration which means that the model is able to complete runs using this event in a reasonable time frame.

Of these only the ground water height run cannot be carried out because there is no relationship (based on a review of the data) between ground water levels over approximately 40 year record at 3 pertinent sites never reached ground surface.

5.3.5.2. Results

Table 20: Sensitivity Results (mAHD)

Location	Base run	Sens. 1	Sens. 2	Sens. 3	Sens. 4
		n' +20%	n' -20%	+50% structure losses	IL=15 CL=2.5
Lisadell Rd	8.24	8.25	8.22	8.24	8.22
Ferodale Rd	8.18	8.17	8.19	8.18	8.16
Ballat Cl	11.87	11.83	11.84	11.87	11.85
Campvale Drain	8.08	8.08	8.08	8.08	8.08
County Cl	13.34	13.37	13.21	13.34	13.31
Federation Dr	12.14	12.16	12.13	12.14	12.12
Moffats Swamp	9.46	9.48	9.44	9.46	9.43

5.3.5.3. Discussion

Sensitivity results indicate that changing roughness, structure losses or rainfall losses has an insignificant impact on model results for the modelled event, the 1% AEP 2h. Whilst changing roughness by 20% caused at most a 30 mm change in peak flood level, adjusting structure losses by 50% did not change flood levels at all. This is due to the fact that the structures assessed are all well below 1% AEP capacity, i.e. most flow is conveyed over the structure (over road) as is demonstrated by Figures 1-3 in Appendix E which compare culvert and road flows for Ferodale Road for the historical events modelled. Adjusting the rainfall losses from the base of 10/2 (initial loss (mm)/continuing loss (mm/h)) to 15/2.5 changed flood levels by only 30 mm which is trivial relative to the typical flood planning level freeboard of 500 mm.

The sensitivity testing provides confidence that as long as model parameters tested are chosen from a range of typical values design flood levels will closely approximate reality.

5.3.6. Scenario Runs

5.3.6.1. Introduction

As per the Brief a number of scenarios were required to be modelled. The runs carried out are as follows:

- Future Development Scenario. For three events, namely the 1%, 5% and 20% AEP events, the impact of future planned land use has been modelled. Note that future land use modelled is as per Figure 9;
- Climate Change Assessment. The 1% AEP event has been assessed for climate change impact and this includes runs to assess how flood levels based on future development (as discussed above) are impacted by climate change;
- Blockage. Five structures have been presumed blocked and these structures are the:
    - Culvert at Ferodale Rd;
    - Culvert at the corner of Kula Rd and Kirrang Drive;
    - Kula Road culverts (two culverts near Karwin Road);
    - Culvert at 754 Medowie Road; and
    - Drainage path on north-east corner of 35 County Close.
- Pump Outage. The impact of failure of the Campvale WPS was assessed on peak flood levels for a variety of events.

The details of the runs carried out were decided on in conjunction with the Technical Committee and that no runs were carried out until agreement had been secured.

In order to present the results of the various scenarios run, a number of points were established and peak flood levels at these locations were then extracted for all of the runs undertaken. This provides an ability to compare base runs with scenario runs and establish the impact of the assessed scenario on flood behaviour. Comparison points are described within the ensuing tables and Figure 19 shows point locations.

5.3.6.2. Results

Results are tabulated and shown below in Table 21 to Table 24. A summary of the results is provided below.

Climate Change

- For a 10% increase in rainfall intensities peak flood levels tend to increase by between 0.0 and 0.2 m. For a 30% increase in rainfall intensities the flood levels tend to increase by between 0.0 and 0.4 m. Neither impact is that drastic given that all new housing floor levels will be a minimum of 0.5 m higher than the defined 1% AEP levels, i.e. freeboard accounts for the variation easily. The impact of climate change on future development flood levels was also examined and results are similar to the results for the existing development scenario described above;
- Note that sea level rise was not examined in the modelling and this is because none of

the discharge locations from either Campvale or Moffats are exposed to the potential of being impacted by an increase in sea level. For example, the Moffats easterly discharge point into Swan Bay occurs at an invert level of 8.35 mAHD, and as such is unlikely to be impacted by any change in sea level expected to be in the order of 0.5 – 0.9 m.

Future Proposed Development

- A result of the modelling for the proposed future development as currently envisaged through the Medowie Strategy , flood levels are slightly higher in downstream locations where volume issues dominate flood behaviour characteristics (order of 0.1 m during the 1% AEP event) but generally there is no measurable impact (analysis rounded to nearest 100 mm). The impact identified (0.1 m) does seem to be widespread within the CDIA however. That is, future development as proposed, does, for the model scenario examined, exacerbate peak flood levels by approximately 100 mm within the CDIA;
- Whilst outside the extent of this study it is noteworthy that upstream development will likely impact on the principal issue for affected residents in the CDIA, i.e. long duration ponding; and
- It may be that future development (and the impact of it) is better examined through long term modelling (such as might be carried out during a water balance assessment). Certainly this would better target one of the main issues with additional development, i.e. whether or not inundation patterns in the CDIA are impacted. Of particular note are those events which residents in the CDIA are most worried about, i.e. long sequences of wet weather, typically occurring between February and July, which result in the CDIA being wet for extended intervals and denying property owners access and use of their own land.

Blockage

- As per the Brief scenarios have been modelled which examine the impact of blockage. Specifically five structures, listed in Section 5.3.6.1, have been blocked. Results generally demonstrate a lack of impact throughout the model domain however there are four locations at which some measurable impact is registered. The peak impact is 0.3 m located within the overland flow path (Moffats Swamp catchment) that did, prior to the erection of a levee there, impact on County Close residents. The general lack of impact is indicative of the fact that in the 1% AEP event much of the flow bypasses controlling structures. For example at Ferodale Road during the 1% AEP event the majority of flow moves across the road and not via the limited culvert capacity at that location. As such blocking the culvert tends to impact on peak flood levels very little (although likely to have a greater impact on lower flood levels). Note also that all model runs (base case) presumed that all structures were blocked by 25%.
- It is noteworthy that the impact of 0.3 m identified in the overland flow path upstream of County Close resulted from increasing the roughness within the overland flow path (as it interacts with the levee) by 100%. As such there is a strong case for maintaining this area relatively free of debris and vegetation;

Pump Failure

- Scenarios were also run examining the impact should the Campvale WPS fail. The results show, as was expected, that the pump has very little impact on peak flood levels outside of the CDIA. Levels are slightly impacted at the upstream face of Ferodale Road (0.1 m impact) but substantially affected up and down stream of the Campvale Drain pinch (impact on peak flood levels is 0.3 m).

Table 21: Future Development Scenario Run Results

	1% AEP - 9h			1% AEP - 72h			5% AEP - 9h			5% AEP - 72h			20% AEP - 9h		
Point # and Location	Existing Scenario (mAHD)	Future Scenario (mAHD)	Impact (m)	Existing Scenario (mAHD)	Future Scenario (mAHD)	Impact (m)	Existing Scenario (mAHD)	Future Scenario (mAHD)	Impact (m)	Existing Scenario (mAHD)	Future Scenario (mAHD)	Impact (m)	Existing Scenario (mAHD)	Future Scenario (mAHD)	Impact (m)
#0 - 500m D/S Boundary Rd	14.1	14.1	0.0	13.9	13.9	0.0	14.0	14.0	0.0	13.9	13.9	0.0	13.9	13.9	0.0
#1 - D/S Ryan Rd	12.3	12.3	0.0	12.1	12.1	0.0	12.2	12.2	0.0	12.0	12.0	0.0	12.0	12.0	0.0
#2 - U/S Kula Rd	9.6	9.6	0.0	9.5	9.5	0.0	9.5	9.5	0.0	9.4	9.4	0.0	9.5	9.5	0.0
#3 - D/S Ballat Rd	11.8	11.8	0.0	11.7	11.7	0.0	11.8	11.8	0.0	11.7	11.7	0.0	11.7	11.7	0.0
#4 - Cnr Karwin Rd + Kula	9.6	9.6	0.0	9.4	9.4	0.0	9.5	9.5	0.0	9.3	9.3	0.0	9.4	9.4	0.0
#5 - Back of 11 Kula Rd	8.7	8.7	0.0	8.5	8.5	0.0	8.6	8.6	0.0	8.4	8.4	0.0	8.4	8.4	0.0
#6 - U/S cnr Kula Rd + Kirrang Dr	10.4	10.4	0.0	10.3	10.3	0.0	10.3	10.3	0.0	10.3	10.3	0.0	10.3	10.3	0.0
#7 - Back of 19 Kirrang D	8.6	8.6	0.0	8.4	8.4	0.0	8.5	8.5	0.0	8.3	8.3	0.0	8.3	8.3	0.0
#8 - 100m U/S Kirrang Dr	8.5	8.5	0.0	8.3	8.3	0.0	8.4	8.4	0.0	8.2	8.2	0.0	8.3	8.3	0.0
#9 - 40m U/S Ferodale Rd	8.3	8.3	0.0	8.2	8.2	0.0	8.2	8.2	0.0	8.1	8.1	0.0	8.1	8.1	0.0
#10 - 200m U/S County Cl.	13.9	13.9	0.0	13.6	13.6	0.0	13.8	13.8	0.0	13.6	13.6	0.0	13.7	13.7	0.0
#11 - Back of 21 Federation Dr	13.2	13.2	0.0	13.1	13.1	0.0	13.2	13.2	0.0	13.0	13.0	0.0	13.1	13.1	0.0
#12 - County Cl drain	12.1	12.1	0.0	12.0	12.0	0.0	12.0	12.0	0.0	11.9	11.9	0.0	12.0	12.0	0.0
#13 - Drain U/S Coachwood	13.3	13.3	0.0	13.2	13.2	0.0	13.2	13.2	0.0	13.2	13.2	0.0	13.2	13.2	0.0
#14 - U/S cnr South St + Potoroo Blvd	15.1	15.1	0.0	15.0	15.0	0.0	15.0	15.0	0.0	14.9	14.9	0.0	15.0	15.0	0.0
#15 - Back 4 Raymond Cl	9.7	9.7	0.0	9.7	9.7	0.0	9.7	9.7	0.0	9.7	9.7	0.0	9.7	9.7	0.0
#16 - Back of 61 James Rd	9.9	9.9	0.0	9.8	9.8	0.0	9.8	9.8	0.0	9.8	9.8	0.0	9.8	9.8	0.0
#17 - U/S Culvert 754 Medowie Rd	12.6	12.6	0.0	12.5	12.5	0.0	12.5	12.5	0.0	12.5	12.5	0.0	12.5	12.5	0.0
#18 - 12 Sir Henry Parkes Av	21.6	21.6	0.0	21.6	21.6	0.0	21.6	21.6	0.0	21.6	21.6	0.0	21.6	21.6	0.0
#19 - U/S Pinch	7.1	7.2	0.1	8.1	8.1	0.0	6.9	6.9	0.1	7.7	7.7	0.0	6.6	6.6	0.0
#20 – D/S Pinch	7.1	7.2	0.1	8.1	8.1	0.0	6.8	6.9	0.1	7.7	7.7	0.0	6.5	6.5	0.0
#21–Near Swan Bay outlet	9.1	9.1	0.0	9.5	9.5	0.0	8.9	8.9	0.0	9.2	9.2	0.0	8.7	8.7	0.0

Table 22: Climate Change Run Results

	Existing Scenario					Future Scenario				
	1% AEP 9h	1% AEP 9h 10% Rainfall increase		1% AEP 9h 30% Rainfall increase		1% AEP 9h	1% AEP 9h 10% Rainfall increase		1% AEP 9h 30% Rainfall increase	
Point # and Location	Peak Level (mAHD)	Peak Level (mAHD)	Impact (m)	Peak Level (mAHD)	Impact (m)	Peak Level (mAHD)	Peak Level (mAHD)	Impact (m)	Peak Level (mAHD)	Impact (m)
#0 - 500m D/S Boundary Rd	14.1	14.1	0.0	14.2	0.1	14.1	14.1	0.0	14.2	0.1
#1 - D/S Ryan Rd	12.3	12.4	0.1	12.4	0.1	12.3	12.4	0.1	12.4	0.1
#2 - U/S Kula Rd	9.6	9.6	0.0	9.6	0.0	9.6	9.6	0.0	9.6	0.1
#3 - D/S Ballat Rd	11.8	11.8	0.0	11.8	0.0	11.8	11.8	0.0	11.8	0.0
#4 – Cnr Karwin Rd + Kula Rd	9.5	9.6	0.1	9.6	0.1	9.6	9.6	0.0	9.6	0.1
#5 - Back of 11 Kula Rd	8.7	8.8	0.1	8.9	0.2	8.7	8.8	0.1	8.9	0.2
#6 - U/S cnr Kula Rd + Kirrang Dr	10.4	10.4	0.0	10.4	0.0	10.4	10.4	0.0	10.4	0.0
#7 - Back of 19 Kirrang Dr	8.6	8.7	0.1	8.8	0.2	8.6	8.7	0.1	8.8	0.2
#8 - 100m U/S Kirrang Dr	8.5	8.5	0.0	8.6	0.1	8.5	8.5	0.0	8.6	0.1
#9 - 40m U/S Ferodale Rd	8.3	8.3	0.0	8.4	0.1	8.3	8.3	0.0	8.4	0.1
#10 - 200m U/S County Cl	13.9	13.9	0.0	14.0	0.1	13.9	13.9	0.0	14.0	0.1
#11 - Back of 21 Federation Dr	13.1	13.3	0.2	13.3	0.2	13.2	13.3	0.1	13.3	0.1
#12 - County Cl drain	12.1	12.1	0.0	12.2	0.1	12.1	12.1	0.0	12.2	0.1
#13 - Drain U/S Coachwood	13.3	13.3	0.0	13.3	0.0	13.3	13.3	0.0	13.3	0.0
#14 - U/S cnr South St + Potoroo Blvd	15.1	15.1	0.0	15.1	0.0	15.1	15.1	0.0	15.1	0.0
#15 - Back 4 Raymond Cl	9.7	9.7	0.0	9.7	0.0	9.7	9.7	0.0	9.7	0.0
#16 - Back of 61 James Rd	9.9	9.9	0.0	9.9	0.0	9.9	9.9	0.0	9.9	0.0
#17 - U/S Culvert 754 Medowie Rd	12.5	12.6	0.1	12.6	0.1	12.5	12.5	0.0	12.5	0.0
#18 - 12 Sir Henry Parkes Av	21.5	21.6	0.1	21.6	0.1	21.6	21.6	0.0	21.6	0.0
#19 - U/S Pinch	7.1	7.3	0.2	7.5	0.4	7.2	7.3	0.1	7.5	0.3
#20 – D/S Pinch	7.1	7.2	0.1	7.4	0.3	7.2	7.2	0.0	7.4	0.2
#21 – Near Swan Bay outlet	9.1	9.1	0.0	9.3	0.2	9.1	9.1	0.0	9.3	0.2

Table 23: Blockage Run Results

Point # and Location	1% AEP - 9h Existing Scenario (mAHD)	1% AEP - 9h Structure Blockage (mAHD)	1% AEP - 9h Impact (m)	5%AEP - 9h Existing Scenario (mAHD)	5%AEP - 9h Structure Blockage (mAHD)	5%AEP - 9h Impact (m)
#0 - 500m D/S Boundary Rd	14.1	14.1	0.0	14.0	14.0	0.0
#1 - D/S Ryan Rd	12.3	12.3	0.0	12.2	12.2	0.0
#2 - U/S Kula Rd	9.6	9.6	0.1	9.5	9.5	0.1
#3 - D/S Ballat Rd	11.9	11.9	0.0	11.9	11.9	0.0
#4 – Cnr Karwin Rd + Kula Rd	9.5	9.6	0.1	9.5	9.6	0.1
#5 - Back of 11 Kula Rd	8.7	8.7	0.0	8.6	8.6	0.0
#6 - U/S cnr Kula Rd + Kirrang Dr	10.4	10.4	0.0	10.3	10.4	0.1
#7 - Back of 19 Kirrang Dr	8.6	8.6	0.0	8.5	8.5	0.0
#8 - 100m U/S Kirrang Dr	8.5	8.5	0.0	8.4	8.4	0.0
#9 - 40m U/S Ferodale Rd	8.3	8.3	0.0	8.2	8.2	0.0
#10 - 200m U/S County Cl	13.9	14.2	0.3	13.8	14.1	0.3
#11 - Back of 21 Federation Dr	13.1	13.1	0.0	13.1	13.1	0.0
#12 - County Cl drain	12.1	12.1	0.0	12.0	12.0	0.0
#13 - Drain U/S Coachwood Dr	13.3	13.3	0.0	13.3	13.3	0.0
#14 - U/S cnr South St + Potoroo Blvd	15.1	15.1	0.0	15.0	15.0	0.0
#15 - Back 4 Raymond Cl	9.7	9.7	0.0	9.7	9.7	0.0
#16 - Back of 61 James Rd	9.9	9.9	0.0	9.9	9.9	0.0
#17 - U/S Culvert 754 Medowie Rd	12.5	12.6	0.1	12.4	12.5	0.1
#18 - 12 Sir Henry Parkes Av	21.5	21.5	0.0	21.5	21.5	0.0
#19 - U/S Pinch	7.1	7.1	0.0	6.9	6.9	0.0
#20 – D/S Pinch	7.1	7.1	0.0	6.8	6.8	0.0
#21–Near Swan Bay outlet	9.1	9.1	0.0	8.9	8.9	0.0

Table 24: Pump Outage Run Results

Point # and Location	1% AEP - 72h			5% AEP - 72H		
	Existing Scenario (mAHD)	Pumping Outage (mAHD)	Impact (m)	Existing Scenario (mAHD)	Pumping Outage (mAHD)	Impact (m)
#0 - 500m D/S Boundary Rd.	13.9	13.9	0.0	13.9	13.9	0.0
#1 - D/S Ryan Rd.	12.1	12.1	0.0	12.0	12.0	0.0
#2 - U/S Kula Rd.	9.4	9.4	0.0	9.4	9.4	0.0
#3 - D/S Ballat Rd.	11.9	11.9	0.0	11.9	11.9	0.0
#4 – Corner Karwin Rd. + Kula Rd.	9.4	9.4	0.0	9.3	9.3	0.0
#5 - Back of 11 Kula Rd.	8.5	8.5	0.0	8.4	8.4	0.0
#6 - U/S cnr Kula Rd. + Kirrang Dr.	10.3	10.3	0.0	10.3	10.3	0.0
#7 - Back of 19 Kirrang Dr.	8.4	8.4	0.0	8.3	8.3	0.0
#8 - 100m U/S Kirrang Dr.	8.3	8.3	0.0	8.3	8.3	0.0
#9 - 40m U/S Ferodale Rd.	8.2	8.3	0.0	8.1	8.1	0.0
#10 - 200m U/S County Cl.	13.6	13.6	N/A	13.6	13.6	N/A
#11 - Back of 21 Federation Dr.	12.9	12.9	N/A	12.9	12.9	N/A
#12 - County Cl. drain	12.0	12.0	N/A	11.9	11.9	N/A
#13 - Drain U/S Coachwood Dr.	13.2	13.2	N/A	13.2	13.2	N/A
#14 - U/S cnr South St. + Potoroo Blvd.	14.9	14.9	N/A	14.9	14.9	N/A
#15 - Back 4 Raymond Cl.	9.6	9.6	N/A	9.6	9.6	N/A
#16 - Back of 61 James Rd.	9.9	9.9	N/A	9.8	9.8	N/A
#17 - U/S Culvert 754 Medowie Rd.	12.5	12.5	0.0	12.5	12.5	0.0
#18 - 12 Sir Henry Parkes Av.	21.5	21.5	N/A	21.5	21.5	N/A
#19 - U/S Pinch	8.1	8.3	0.3	7.7	8.0	0.3
#20 – D/S Pinch	8.1	8.3	0.3	7.7	8.0	0.3
#21–Near Swan Bay outlet	9.5	9.5	N/A	9.3	9.3	N/A

5.4. Design Flood Results

Using three critical durations (2, 9 and 72 hour) the following design floods have been run; the 2Y ARI, 20%, 10%, 5%, 2%, 1% and 0.5% AEP events as well as the PMF.

Peak flood level/depth rasters have been developed for each of the runs. This involved merging maximum values from each of the critical durations run for each event.

Appendix D provides peak flood level, velocity and flow results in tabular form at key locations. Figure 23 shows the results for the 1% AEP event and this includes contours of peak flood levels. Figures 24 through to 30 then show peak flood extents/depths for all other modelled events. Figures 32 to 35 provide hazard data which is of particular importance for the 1% AEP event. Figures 36 and 37 show hydraulic categories for the 1% AEP and PMF events respectively. Although Reference 7 provides guidelines on determining hydraulic categories it does not explicitly define each category. Consultants and authorities use different approaches for this. For the purpose of this study hydraulic categories have been adopted according to Reference 11:

- Floodways: The product of Velocity * Depth > 0.25 m^2/s AND Velocity > 0.25 m/s OR Velocity > 1 m/s;
- Flood Storage: Land outside the floodway where Depth > 1.0 m; and
- Flood Fringe: Land outside the floodway where Depth < 1.0 m.

Figure 38 and 39 show flood profiles for all modelled design events for both the Campvale and Moffats Swamp areas. The main use of the profile data is that it indicates which events are likely to overtop roads. It is of interest to note however that peak flood level does not change markedly for different events modelled (although the difference is more pronounced in downstream areas such as the CDIA where volume dictates).

Results indicate that numerous properties will suffer some degree of inundation. Most flooding will occur not on the fringes of the swamp areas in Campvale and Moffats but rather will occur in upstream areas where flows moving downstream are not able to be contained in overland flow paths and drains.

The actual number of houses impacted by over floor flooding cannot be discerned at this stage prior to the acquisition of floor level survey. It is not anticipated that the number of houses inundated in the 1% AEP flood event would exceed 20 based on the results presented herein. Major flood liable locations are identified as follows:

- At the intersection of Kirrang Drive and Kula Road;
- In Ballet Close;
- Kirrang Drive near the Campvale Drain;
- Abundance Road south of the intersection with Lisadell Road;
- Isolated areas on James Road;
- Windeyer Place on the southern side which fringes of Moffats Swamp; and
- Near the intersection of Potoroo Road and South Street.

High hazard areas tend to be limited to defined flow paths and also those areas where water depth accumulates, such as the swamps. Few houses are impacted by high hazard flows or are likely to be found in defined floodways.

It is important to note that Moffats Swamp starts transferring flows into Campvale Swamp in events greater than 1% AEP 72 hour durations through the saddle located at Championship Drive, within the premises of the Pacific Dune golf course (refer to Figure 1).

5.5. Campvale WPS and Campvale Drain capacity

As mentioned before in Section 2.2.1 Campvale WPS has a peak pumping capacity of approximately 120ML/day which represents approximately 23 mm of runoff per day.

Campvale Drain's low slope can impede the transfer of flow from immediately downstream of Ferodale Road to the Campvale WPS. Primarily, however, the Campvale Drain "pinch" does not allow the Campvale WPS to operate at maximum capacity. The "pinch" is the most influential factor in exacerbating both peak flood levels and duration of inundation in the CDIA.

At CDIA stage heights of 7 mAHD flow through the pinch will begin to match the peak flow capacity of the Campvale WPS (~ 5.4 m^3/s). For larger inflows to the CDIA the pinch, whilst allowing flow downstream such that the pumps run at full capacity is nevertheless limiting, turning the CDIA into a de facto retardation basin. For example in the PMF event an inflow to the CDIA of approximately 180 m^3/s is converted into as flow moving downstream to the Campvale WPS of approximately 20 m^3/s.

It is highly recommended that the next stage of the FRMP investigate the impact of increasing the capacity of the pinch. Step One in that process must be detailed land based survey of the pinch, in the area immediately adjacent to, and including, the drain proper.

6. CONCLUSIONS

The flood study carried out for Medowie identifies flood liable land for a range of floods and finds that many properties and approximately 20 residences, are likely to be flood liable during the 1% AEP event. Recommendations going forward are as follows:

- In some locations existing drains are incapable of carrying 20 – 10% AEP (and in some cases even lower) flows. An example of this is the open drain which passes through Ballet Close from north to south. Enhancing the capacity of these trunk drainage systems should be examined as this may achieve a degree of flood relief for those properties which are exposed to flooding on a regular basis. Such an examination should however ensure that flooding issues are not simply transferred downstream;
- Houses in County Close which remain subject to inundation should be closely monitored. It is likely that local flows cause inundation of these low lying properties and minor works by Council may alleviate this circumstance for impacted residents;
- More generally there a number of locations where, due to the limited range of design flood behaviour (flood depths do not change markedly for variable AEP), minor works such as bunding could provide good levels of flood protection for individual properties. A good example of this is on Kirrang Drive (western side) where houses are currently impacted by flows from the swale at the front of their properties;
- Generally Council needs to recognise the main overland flow paths identified in the study and move to restrict development in these “corridors” in the future;
- In order to assess the issue in the CDIA with regard to upstream development potentially increasing flooding downstream of Ferodale Road, satisfactorily answer questions in the CDIA in regard to the impact of proposed development on the wet sequences which often deny property owners of access/enjoyment of their land, Council need to have a water balance type of exercise carried out. This will address the kinds of events which impact on inundation within the CDIA but which do not have any relevance to the definition of peak flood levels. It is noteworthy that during the Management Study process it will also be necessary to ensure mitigation works designed to alleviate flooding liability do not negatively impact on long term inundation for water sequences in the CDIA. The Management Study should consider a range of solutions but ensure that water quality is either maintained or improved;
- Council should take care not to raise those sections of roads currently inundated as doing so will directly impact on peak flood levels. This is due to the fact that much of the 1% AEP flow moves over roads rather than through corresponding culverts; and
- During the Management Study it is recommended that runs are carried out which examine the impact of removing the pinch which currently exacerbates the duration, extent and magnitude of inundation further upstream in the CDIA; and
- Pinch plus pump capacity enhancement runs be carried out. Note unless pinch works are modelled, nothing will be achieved by looking at pumping rate enhancement alone.

7. ACKNOWLEDGEMENTS

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