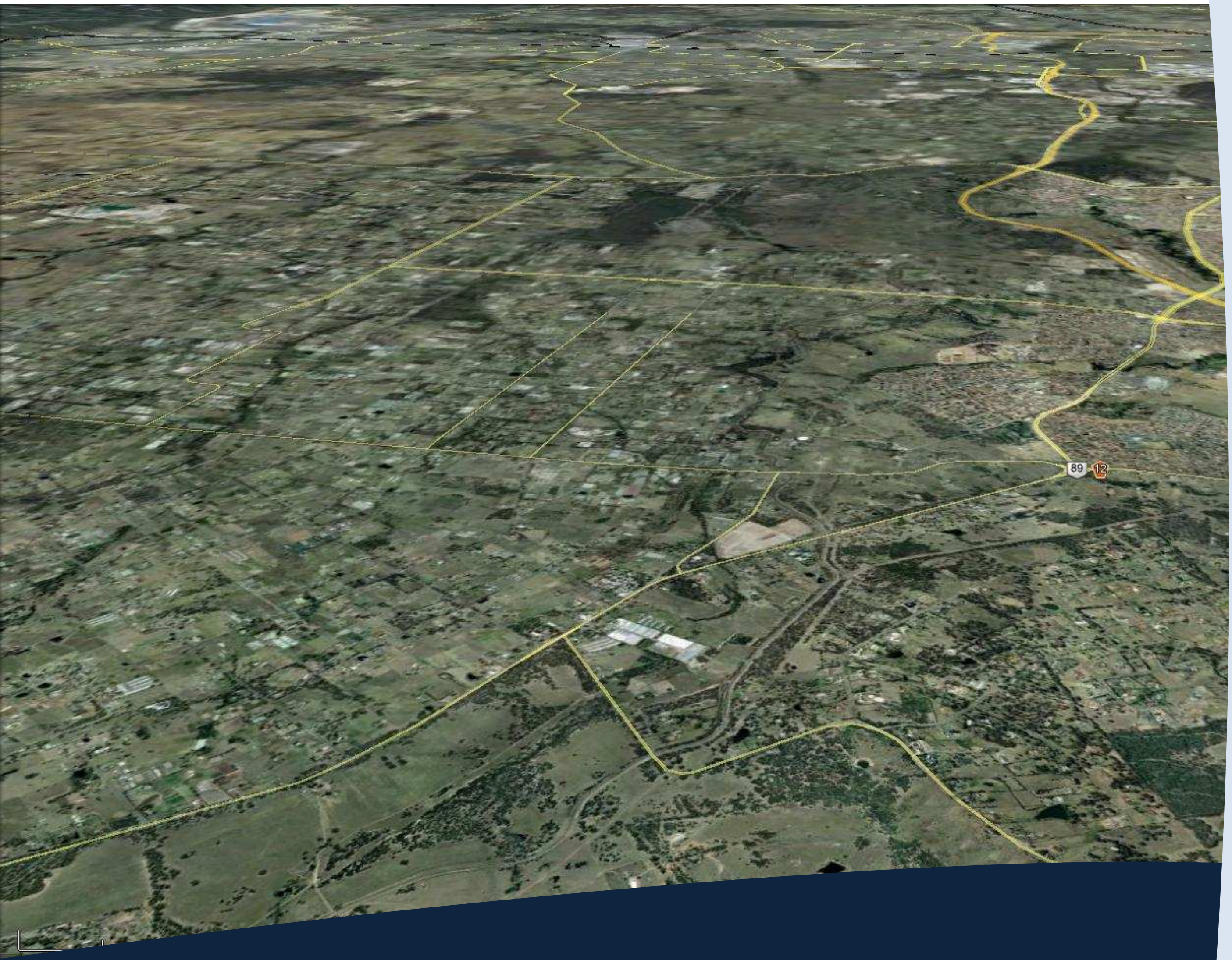

Water Cycle Management Report
East Leppington

Project No: 600319

Prepared for Department of Planning & Infrastructure

July 2013

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Cover Image: Oblique Aerial View of East Leppington (Source: Google earth Pro, Accessed 13 February 2013)

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APPENDICES

Note: The Appendices were prepared following completion of the various technical modelling tasks required by the Water Cycle Management Plan. The dates shown in the footers, therefore relate to the date for completion of each task with the results informing the outcomes set out in the report .

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1 INTRODUCTION

1.1 Preamble

This report was originally prepared for the Department of Planning and Infrastructure in June 2012 and documents the proposed strategy for water management for the rezoning of the East Leppington precinct. As part of the rezoning process the WCM was subject to public and stakeholder review during the exhibition period following the original issue of the report. At this time comments were raised by Campbelltown City Council and Liverpool City Council regarding the approach adopted to hydrological modelling and a subsequent sensitivity assessment was undertaken to determine the sensitivity of estimated runoff to alternate model parameters. The findings of the sensitivity assessment were documented in a letter report in October 2012. The sensitivity assessment showed that the alternate model parameters had minor impacts on the estimated flood behaviour in East Leppington. Stormwater discharges increased, however the requirements for water quantity management required minimal modification and flood levels were not increased to a significant degree.

A peer review of the proposed Water Cycle Management strategy was commissioned by DP&I in December 2012. The peer review of the WCM strategy recommended the following actions:

1. The proposed design of the floodway between Denham Court Road and Camden Valley Way should be modelled in more detail. Allowance should be made within the floodway for proposed assets such as bioretention basins so that a more accurate representation of the planned measures is modelled.

2. The results of two hydraulic model packages were provided in the original WCM report, being TUFLOW for flood behaviour and XPSWIMM for basin sizing. It was recommended that only results of the TUFLOW modelling be provided to avoid potential confusion. This subsequently required further modelling so that the TUFLOW modelling provided sufficient data to enable basin sizing without the reliance on XPSWIMM.

3. A more detailed discussion was recommended to explain how the upstream and downstream boundaries are treated to ensure that no flood affectation is experienced outside of the Precinct.

4. Recommendations were made to demonstrate how the biofiltration areas estimated by the water quality modelling have been accommodated in the bioretention basins shown in the ILP; and

5. The above recommendations should be addressed in an updated WCM report that reflects the latest version of the ILP and includes the sensitivity assessments of October 2012 as the final flood mapping and results.

These recommendations have been actioned and are documented in this updated version of the WCM report. It is expected that a letter of confirmation would be provided from the peer reviewer to confirm that the outcomes of the peer review process have been satisfactorily addressed.

1.2 Background

In 2005 the NSW Government identified two regions, one in Sydney's northwest and one in Sydney's southwest, of largely undeveloped land as the potential location for development of new communities. These two growth areas are capable of accommodating 500,000 people and have been named the North West Growth Centre and the South West Growth Centre respectively. Each growth centre is divided into a number of Precincts that will drive the staged development of each Growth Centre.

In order to prioritise and facilitate the development of the Precincts within the Growth Centres the NSW Government passed *State Environmental Planning Policy (Sydney Region Growth Centres)* 2006 (referred to as 'Growth Centres SEPP'). The Growth Centres SEPP primarily expedites the Precinct planning and rezoning processes that most developments are required to undergo in accordance with the *Environmental Planning and Assessment Act 1979 (EP&A Act)*. The Growth Centres SEPP establishes the planning rules and objectives for the Growth Centres.

Each Precinct is required to undergo a Precinct Planning Process which brings together State Government agencies and local councils to coordinate the provision of infrastructure and social services within each of the precincts. Integral to this stage is the assessment of appropriate land use options within each of the Precincts (e.g. key transport routes, residential housing, commercial areas, biodiversity conservation). As such the Precinct Planning Process involves detailed investigations into environmental constraints which will help determine the development potential within the precincts. The Precinct Planning Process is integral to the control and management of development to ensure these aims are met.

Ultimately the constraints identified within a Precinct are combined to prepare an Indicative Layout Plan, which is placed on public exhibition along with supporting documents (the Precinct Planning Package). Following receipt of submissions, the Minister for Planning and Infrastructure may approve the Precinct Planning Package and rezoning of the land within the Precinct where appropriate. Following rezoning, Development Applications may then be lodged.

East Leppington is adjacent to the Austral Leppington Precincts where Cardno has undertaken a Water Cycle Management Strategy for the Department of Planning and Infrastructure (Cardno 2011). The hydrology and hydraulic modelling for this study was informed by the "Austral Floodplain Risk Management Study & Plan", report Version 5.0, prepared for Liverpool City Council, September (Parrens 2003). The Parrens 2003 study continued with modelling from a number of preceding flood studies using RAFTS and HEC RAS models calibrated to historical flood events. Cardno adopted the hydrological modelling assumptions of the previous studies and used ground level data from the HEC RAS models made available by Liverpool Council.

1.3 Scope

Cardno was engaged by the Department of Planning and Infrastructure (DP&I) to undertake a Water Cycle Management Study for the East Leppington Precinct. This Water Cycle Management Report considers the flood and stormwater behaviour in the East Leppington precinct in order to identify appropriate flood extents, a strategy for stormwater quantity and quality management for the development of the Precinct and the subsequent detailed works required to give effect to the strategy.

1.4 Study Area

The precinct of East Leppington is located southwest of Sydney on the Cumberland Plain and covers an area of approximately 463ha. Its location within the Southwest Growth Centre is shown in **Figure 1-1**.

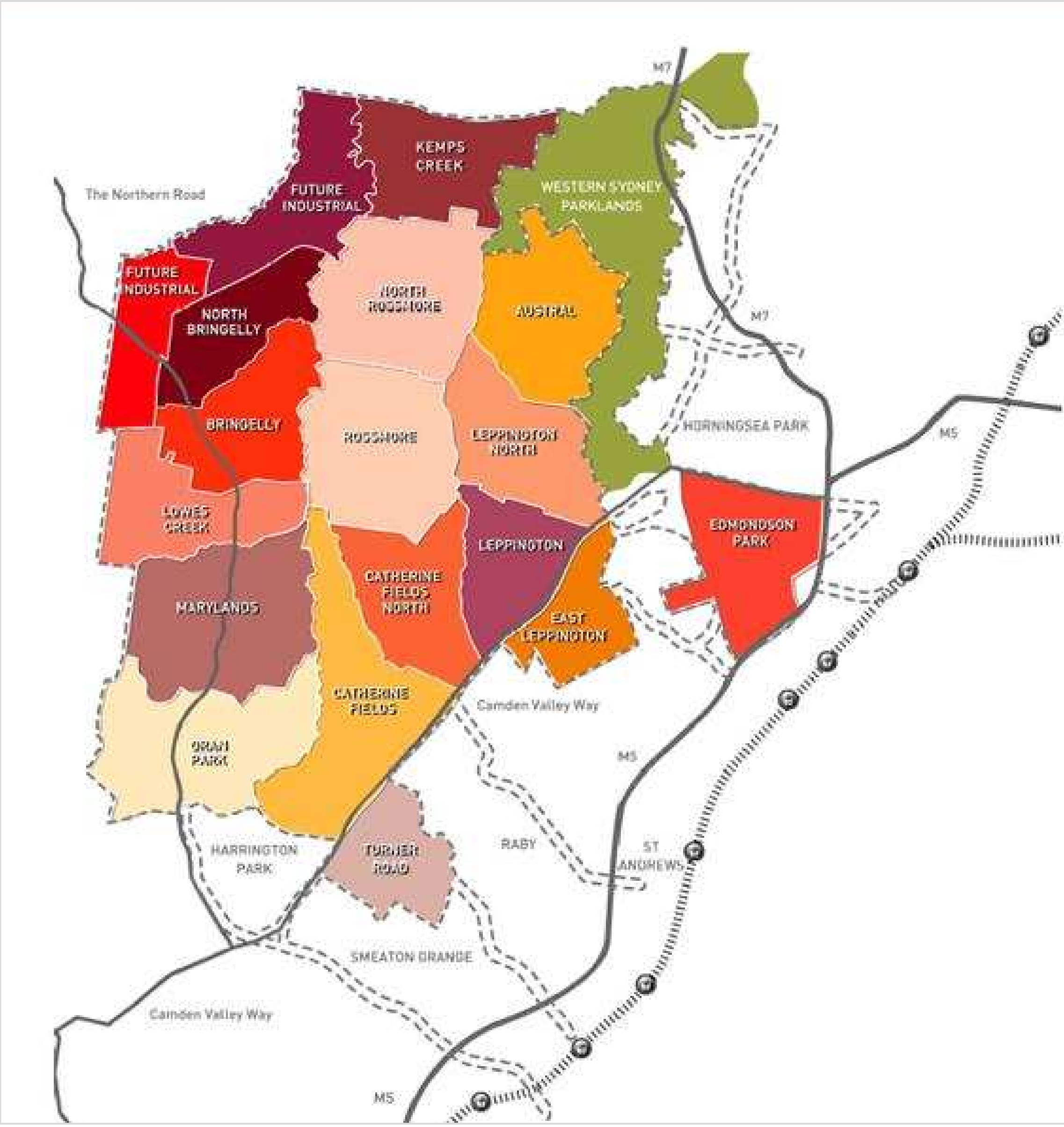


Figure 1-1: Location of Southwest Growth Centres (Department of Planning & Infrastructure, 2010)

The study area is bound to the northwest by Camden Valley Way and St Andrews Road to the west. Its north eastern boundary is formed by Denham Court Road. The study area is dominated by cleared land with clusters of trees. The main land use is agriculture. A number of small dams are spread across the study area in drainage depressions.

There are two main Creeks within the study area. College Creek located along the eastern boundary and Bonds Creek flowing from southwest to northeast centrally through the precinct. Bonds Creek has a number of tributaries flowing from the south which are included in this analysis.

A Sydney Catchment Authority (SCA) canal flows from southwest to northeast in direction and bisects the precinct. Topographical information indicates surface water in the east of the precinct is conveyed to College Creek, with the remaining being conveyed to Bonds Creek. A number of existing culverts convey overland flow beneath the SCA canal. The proposed study area is shown in **Figure 1-2** and indicates the main watercourses within the catchment.

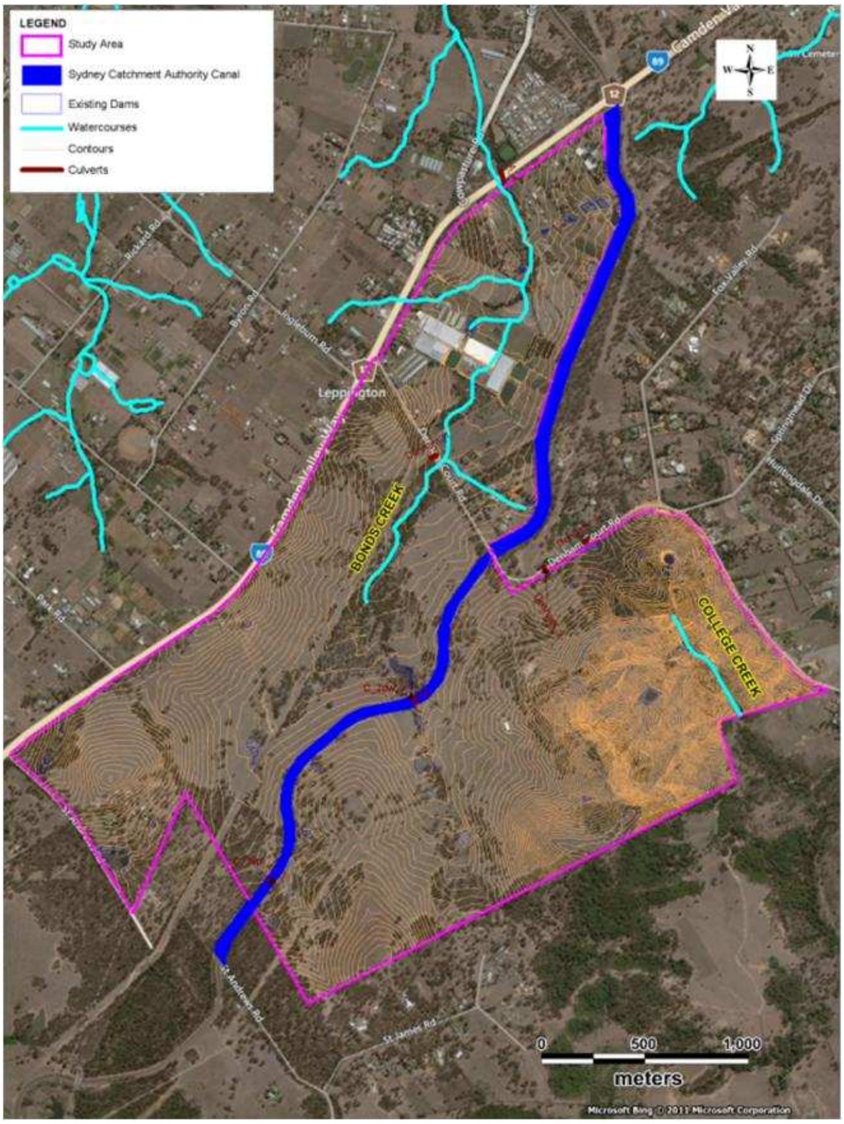


Figure 1-2: Study Area

2 CONSULTATION

2.1 Stakeholder Consultation

In preparing the Water Cycle Management Strategy, Cardno consulted with the Departments' ecology consultant (Eco-Logical Australia) and NSW Office of Water (NOW) which included an on-site meeting with NOW. It was agreed that an integrated approach to riparian corridor rehabilitation, water quality, flow attenuation and flood management would be adopted including the following:

- Flood affectation would be managed by cut and fill within the precinct and removal of existing dams along watercourses such that flooding would be confined to proposed riparian corridors;
- Disturbance of existing vegetation within the nominated riparian corridors would be avoided where possible.

The above principles are further detailed in the Eco-Logical Australia report. Subsequent to these discussions, DP&I advised that the Strahler Stream Order and Waterway Classification System is to be adopted for the East Leppington Precinct due to a change in government policy. This system is described below.

2.2 Strahler Stream Order and Waterway Classification System

The Strahler system is based on waterways being assigned an "order" according to the number of additional tributaries associated with each waterway and indicates the complexity of a system.

The stream orders described in **Table 2-1** is similar, but are not to be confused with, the definitions of three categories that define environmental objectives under the RCMS.

The Core Riparian Zone (CRZ) is the land contained within and adjacent to the channel. The intent is to ensure that the CRZ remains, or becomes vegetated, with fully structured native vegetation. The width of the CRZ is measured from the top of the highest bank on both sides of the watercourse.

Table 2-1: Recommended CRZ widths as defined by the Strahler Method

Types of Watercourse	CRZ Width (m)
Any first order watercourse and where there is a defined channel where water flows intermittently	10
Any permanently flowing first order watercourse, or Any second order watercourse, and Where there is a defined channel where water flows intermittently or permanently	20
Any third order watercourse or greater watercourse and where there is a defined channel where water flows intermittently or permanently. Includes estuaries, wetlands and any parts of rivers influenced by tidal waters.	20 ~ 40

Applying the Strahler system results in the following corridor widths, including an allowance for approximate channel width:

- 25m for the tributary of Scalabrini Creek, located in the southwest corner of the Precinct;
- 25m on Bonds Creek upstream of the confluence; and
- 45m on the Bonds Creek tributary;
- 45m on Bonds Creek downstream of the confluence.

Preliminary modelling was undertaken to assess the adequacy of the reduced riparian corridor widths for the purpose of flow conveyance. Results indicated significant increases in flood levels would be expected in some parts of the corridors with constriction in the floodway to the minimum riparian corridor extents.

Further assessment indicated the following minimum drainage corridor widths would be appropriate. These were subsequently analysed in detail as part of this assessment:

- 50m on the Bonds Creek tributary; and
- 80m on Bonds Creek upstream and downstream of the confluence.

For the other smaller watercourses that would have been first order watercourses prior to European settlement, they have been disturbed by land clearing, filling and diversion to such an extent that they no longer resemble a recognisable watercourse. In these cases no defined channel is evident and intermittent flows are carried in man-made drainage channels and/or piped drainage systems. As such some of the watercourses that would have fallen under Strahler first order are not included in the stream network. Flows within these catchments would be conveyed by piped drainage and overland flows in the road corridor under developed conditions.

2.3 Retarding Basins

Retarding (detention) basins are required in order to attenuate peak flows expected during the 100 year ARI event under post-development conditions to pre-development levels.

As part of the consultation process Campbelltown City Council and Department of Planning and Infrastructure requested that the retardation basins be located on-line. This was due to the stream classification using Strahler guidelines allowing on-line basins which are more efficient in terms of land-take and consolidate maintenance within the drainage corridor. Water quality treatment however is to remain off-line, such that runoff is treated prior to discharge to the watercourses.

An assessment of on-line basins has been undertaken as part of this investigation, except for the catchment draining to the tributary of Scalabrini Creek within Camden LGA, where an off-line basin is suggested.

3 HYDROLOGY

The hydrological analysis consisted of the following:

- Assembly of **xprafts** rainfall/runoff models of the existing catchment condition and the post-development scenario.
- Estimation of runoff under the 2yr ARI, 20 yr ARI, 100 yr ARI, 500yr ARI and PMF event under pre-development conditions;
- Estimation of runoff in the 2 yr ARI and 100 yr ARI events under post-development conditions to allow preliminary sizing of basins to limit peak flows to pre-development levels; and
- Export hydrographs from the hydrological model to the hydraulic model of the floodplain.

Hydrological modelling was then updated using revised rainfall loss rates as detailed in **Table 3-1**.

Table 3-1: Revised rainfall loss rates

Condition	Initial Loss (mm/hr)	Continuing Loss (mm/hr)
Impervious Areas in up to 100 year ARI events	1	0
Pervious Areas in up to 100 year ARI events	15	2.5
Impervious and Pervious Areas in greater than 100 year ARI events	0	0

In addition to these rainfall losses a BX factor = 1.0 was adopted.

3.1 Existing Conditions

The **xprafts** model was run to estimate design storm hydrographs for input to the TUFLOW floodplain model.

The **xprafts** model layout is included in **Appendix A.1** and identifies the subcatchment layout and node names.

The estimated peak flows at all locations within the study catchment for storm durations ranging from 30mins to 18 hours are summarised in **Appendix A.1**.

3.2 Developed Conditions

Under developed conditions an overall average fraction imperviousness of 0.7 has been assumed. This has been calculated based on a breakdown of land use within the Precinct and fraction imperviousness based on Campbelltown City Council's DCP. Further information is given in **Appendix C**.

The estimated peak flows at all locations within the study catchment for storm durations ranging from 30mins to 18 hours are summarised in **Appendix A.1**.

As expected a comparison of existing and developed conditions disclosed that the proposed development generally increases peak flows along Bonds Creek and its tributary. In general the critical storm burst duration for the 2 year ARI event is 9 hours while it is 2 hours for the 20 year and 100 year ARI events.

3.3 Retarding Basin Assessment

A hydrological assessment of retardation basin options was undertaken. The aim of the assessment was to limit as far as possible, the 2 yr ARI and 100 yr ARI peak flows downstream of the proposed development areas to no greater than the peak flows under existing conditions.

Potential sites for on-line retarding basin were identified at the following locations:

- Upstream of the Sydney Catchment Authority canal on the tributary to Bonds Creek;
- Upstream of Denham Court Road on Bonds Creek;
- Upstream of Camden Valley Way on Bonds Creek; and
- And at an additional two locations along Bonds Creek.

The potential on-line retarding basin locations are shown in **Figure 3-1**.

It is noted that for the tributary of Scalabrini Creek located in the southwest corner of the Precinct, an off-line Basin R1 has been assumed due to topography being less suitable for an online basin. The basin would be situated adjacent to the creek and Camden Valley Way accepting flows from its local catchment and including a biofiltration area. The methodology used for design of a typical off-line basin is outlined in **Appendix A.3**.

3.3.1 Basin Sizing Methodology

A 1D hydraulic model was initially created using channel sections to represent Bonds Creek and its tributary in order to test the indicative performance of the on-line basin approach. The channel sections were based on the proposed floodway widths with filling on the sides of the floodway up to the 100 year ARI flood level to represent the expected developed condition.

Existing and developed condition 2 year and 100 year ARI flows were exported from the **xprafts** model and input into the 1D hydraulic model. Retarding basin storage volumes and basin outlets comprising two stage culverts were sized to attenuate the peak 2 year and 100 year ARI flows under developed conditions to pre-development levels.

The basin configuration was guided by the following design objectives:

- Locate the basin on-line within the floodway;
- Limit the amount of earthworks required to construct the basin. This was achieved by including the basin bund without excavation of existing floodplain topography where possible.
- Soften and vegetate the basin structures so that they complement the riparian vegetation and habitat. And as far as practical make use of existing topography by the use of an outlet structure which creates temporary ponding within the drainage corridor upstream of the structure;

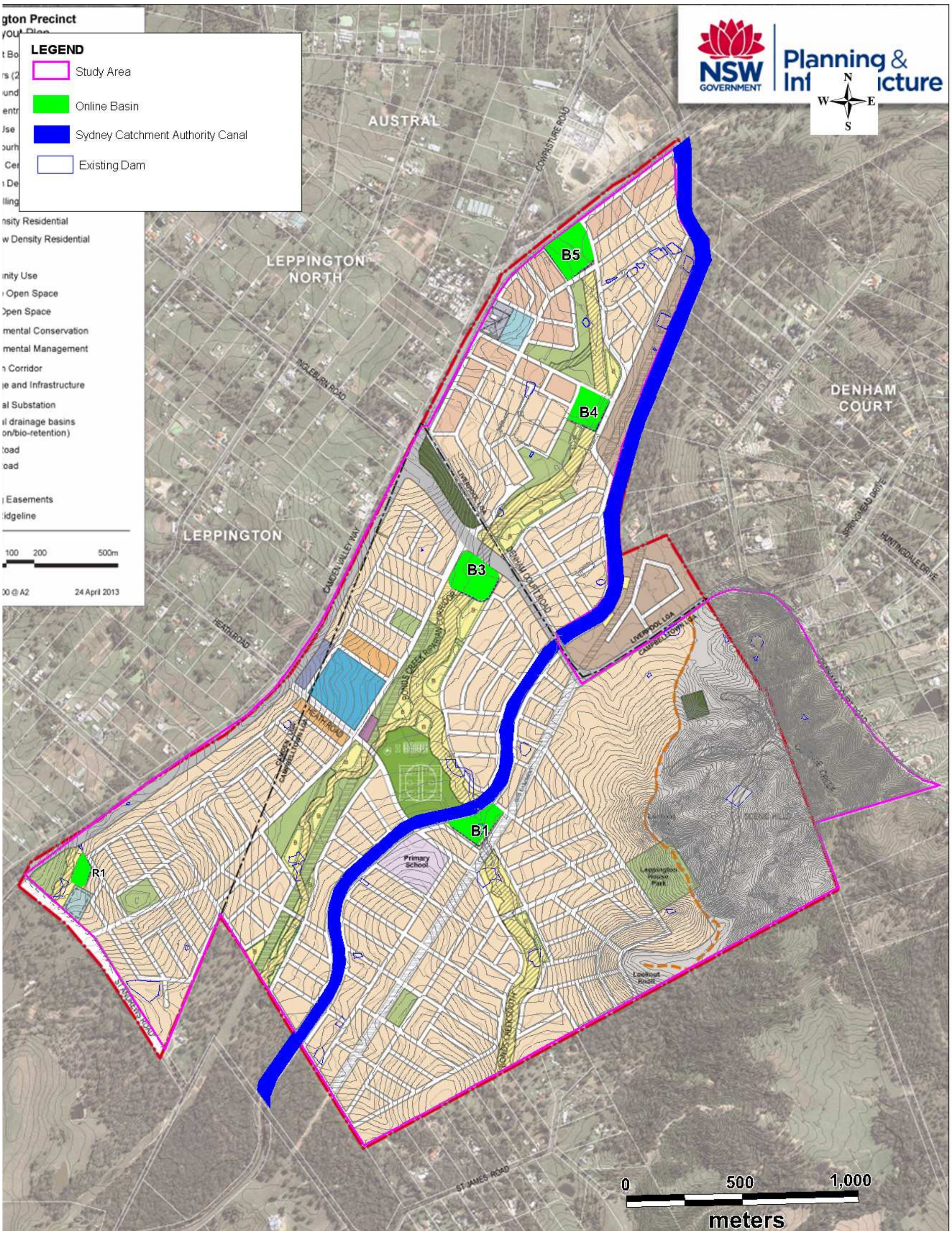


Figure 3-1: Proposed On-line Retarding Basin Locations

- Adopt maximum batter slopes of 1 (V) : 4 (H) in order to minimise the impact of the basin embankment on existing vegetation, and;

- Use of a two stage outlet structure on grade to attenuate the peak 2 year and 100 year ARI flows under developed conditions to pre-development levels.

Various basin outlet configurations were assessed using the 1D hydraulic model. Once the initial size of the hydraulic outlet and height of the basin embankment were determined these basins were then included in the 1D/2D TUFLOW floodplain model to confirm or otherwise the performance of the basins.

The 1D model included generic topography for the creeks whereas the 2D floodplain model reflects the existing ground terrain including an ill-defined drainage line of varying dimensions. It was therefore expected that the peak flows estimated using the 1D model would be higher than those estimated by the 2D floodplain model. It would also be expected that the basin structures sized using a 1D modelling approach would produce different results when modelled in 2D TUFLOW model.

Consequently in order to assess if the peak 2 year and 100 year ARI flows under developed conditions are no greater than under existing levels approach it was necessary to compare the flow hydrographs calculated by TUFLOW at key locations under existing and developed conditions.

The peak flow estimated by the 2D TUFLOW model with and without basins are summarised in **Table 3-4**. It is shown that there is an increase in peak flow when comparing the existing and developed conditions. Thus it is considered prudent to include on-line basins within the drainage corridor to ensure that flood behaviour is not adversely impacted by urban development. Interestingly the post-development peak flows from TUFLOW in **Table 3-4** exhibit smaller increases as a result of urbanisation than that estimated by **xprafts**. Factors that contribute to the difference in peak flows between TUFLOW and **xprafts** are listed below:

1. Topographic features such as swamps depressions and floodplain storage are not replicated in **xprafts.** Therefore it is predicted that there are storage effects in TUFLOW that are contributing to natural retardation of overland flow.
2. The Precinct is segregated by the SCA canal that naturally retards overland flow upstream in existing conditions. In the developed condition future drainage improvements would route flows under/over the canal to avoid stormwater contamination to drinking water. This leads to a quicker response in the generation of runoff, particularly in the first half hour of the storm duration where a drainage network would route the flow directly to the floodway.
3. In the developed condition there is filling proposed of the floodway fringe that reduces floodway width. This could increase flow velocity and thus contributing to the quick response of the impervious areas of the sub-catchments.

Appendix A.2 includes hydrographs of the flow extracted upstream and downstream of the proposed retarding basins. It is shown that the existing peak flow commonly occurs around 6 hours after the start of runoff in the 2 year ARI 9hr event and the 1.5 – 2 hours after the start of runoff in the 100 year 2hr event. It is evident that the total volume of flow, for the developed condition with basins, has increased and the peak flow has decreased. This is a result of urbanisation increasing volume from the impervious surface runoff through a rapid response in the first hour of the storm and continually thereafter. It should be noted that there are undeveloped sub-catchments remaining upstream of the Precinct that contribute to the peak of the storm following where the initial loss of the pervious surfaces reduces the generation of runoff in the first hour.

It is shown that the basins are retarding flows back to existing conditions in the 2yr 9hour event leading to flows being slightly over retarded in the 100yr ARI 2hr event. While it is noted that it may not be necessary to retard the 100 year ARI peak discharges below existing flows, consideration needs to be given to ensuring that existing flood levels downstream are not exceeded. It was found through multiple iterations that the flood levels are sensitive to the overall volume of runoff where the increase in flow volume released from the hydraulic structure of a retarding basin contributes to an increase in flood level. It is then necessary to retard the 100 year ARI runoff below the peak flow of existing conditions in order to observe existing flood levels. The hydraulic structures of the retarding basins have then been sized to throttle the peak flows of the 100 year ARI below existing conditions in order to control afflux and to avoid adverse impacts downstream caused by volumetric increases

Basin B1 located upstream of the SCA canal easement has been configured so that it would not allow overtopping of the canal in the 100 year ARI event. In addition the basin has been sized to include sufficient freeboard such that the existing crossing under the canal can be retained.

Table 3-2: Proposed Basin Outlet Details

Basin ID	Description	100yr Outlet	2yr Outlet
B1	This basin was located on-line upstream of the SCA canal with its own detention bund and multi-level outlet independent of the canal and its existing culverts. The basin modelled did not include excavation of the floodplain, however this may be desirable during detailed design to reduce the estimated developed flood level. Typical levels are shown in **Table 3-3**. This basin over retards post development flows in order to ensure that no overtopping of the SCA canal is experienced in developed conditions.	3.55m (W) x 1.0m (H)	3.4m (W) x 0.75m (H)
B3	This basin was located on-line with the new Denham Court Road crossing embankment and culverts to act as the detention bund with multi-level outlet control. No excavation of the floodplain is proposed. Re-alignment of a short length of the creek may be required as the existing flowpath at the crossing is shifted to the west. The existing Denham Court Road crossing is to remain in the interim to allow access to properties to the north.	30m (W) x 0.5m (H)	15m (W) x 0.5m (H)
B4	On-line basin located within the proposed open channel connecting the Denham Court Road and Camden Valley Way crossings. The basin includes a widening of the open channel to maximise storage volume. A road crossing is proposed downstream which could act as the detention bund in a similar manner to that proposed for B3.	23m (W) x 0.75m (H)	10m (W) x 1.0m (H)
B5	On-line basin located directly upstream of Camden Valley Way (CVW) with a detention bund independent of the CVW	5 x 2.1m (W) x 1.2m (H)	4 x 2.1m (W) x 1.2m (H) RCBCs

	road embankment and proposed bridge. This basin includes a widening of the proposed open channel and a multi-level outlet structure.	RCBCs	
R1	Off-line basin located at the base of the drainage network and outlet of future overland flowpaths of the urban space. The basin included excavation of the natural surface and a multi-level outlet structure.	1m (W) x 0.25m (H)	1m (W) x 0.25m (H)

***Table 3-3*: Proposed Basin Details**

Basin ID	Outlet IL (m AHD)	Peak Depth in 100yr ARI (m)	Bund Level (m AHD)	Indicative 100yr Storage Volume (m^3)	Peak Depth in 2yr ARI (m)	Indicative 2yr Storage Volume (m^3)
B1+	94.1	2.82	97.0	31,150	1.5	11,300
B3+	85.03	1.84	87.60	19,500	1.30	3,000
B4	81.71	2.0	84.20	27,540	1.10	15,150
B5	77.16	3.62	80.90	62,879	1.95	33,872
R1	TBC	0.85	TBC	2,832	0.65	1,844

+ these basins are modelled by inclusion of a detention structure on-line which temporarily stores floodwaters over the ground terrain of the detail survey DTM.

Table 3-4: Estimated 2yr and 100yr ARI Peak Flows extracted downstream of proposed basins

Basin ID	Reference * Location	100yr 2hr Peak Flow (m^3/s)			2yr 9hr ARI Peak Flow (m^3/s)		
		Existing	Developed (No Basin)	Developed (with Basin)	Existing	Developed (No Basin)	Developed (with Basin)
B1	**A-DS**	11.7	15.6	9.0	5.6	13.1	4.7
B3	**B-DS**	46.0	46.6	37.4	16.2	17.6	15.7
B4	**C-DS**	46.7	51.8	40.1	17.0	19.4	16.9
B5	**D-DS**	52.1	63.6	43.4	20.2	24.0	20.6
R1	**Low point of local catchment**	0.8	3.1	0.8	0.3	1.5	0.3

** see Figure A-2 (Appendix A) for indication of reference locations*

4 HYDRAULICS

Given the availability of ground survey data for the study area, the approach that was adopted was to assemble a 2D hydrodynamic model of the existing watercourses and floodplain using TUFLOW. The TUFLOW model was based on a Digital Elevation Model (DEM) which was created using the available digital survey data.

The advantages of a 2D approach included the ability to:

- Represent spatial planting strategies in overbank areas in the 2D model far more accurately than in a 1D model;
- Present flood levels, extents and velocity fields in spatial plots which are more readily understood by stakeholders; and
- Identify flood extents more accurately rather than relying on linear interpolation between 1-D cross sections.

The hydraulic modelling was undertaken using a 4m x 4m grid. The TUFLOW model samples points from a terrain model constructed using ground survey undertaken by Lockley Land Title Solutions, July 2007. The adopted downstream boundary conditions for Scalibrini and Bonds Creeks were water level time series extracted from the TUFLOW model assembled for the Austral Leppington North (ALN) Flooding Assessment Strategy (Cardno, 2011).

4.1 Model Calibration and Verification

The hydraulic models assembled for the 1990 South Creek Flood Study and the 2012 Flood Study for South Creek (WMAwater) were calibrated against the 1988 historical flood.

A comparison of the flood levels estimated by the ALN TUFLOW model and results of the Upper South Creek Flood Study (WMAWater, 2012) is included in **Table 4-1** and shows that consistent results were achieved downstream of Bringelly Road.

The accuracy of the East Leppington flood model was also assessed against the flooding predicted by the TUFLOW models prepared for the Austral and Leppington North Precincts (Cardno 2011). It was found from the TUFLOW modelling that the flood extent and depth results are generally lower than those predicted for previous studies such as Perrens Consultants (2003) "*Austral Floodplain Risk Management Study & Plan*", Report Version 5.0, prepared for Liverpool City Council.

Table 4-1: Comparison of Flood Levels downstream of Bringelly Road (m AHD)

	Kemps Creek			Bonds Creek		
	20 year ARI	100 year ARI	PMF	20 year ARI	100 year ARI	PMF
WMAWater	74.1	74.2	74.8	73.6	73.7	74.4
Cardno	74.1	74.2	74.8	73.5	73.8	74.7

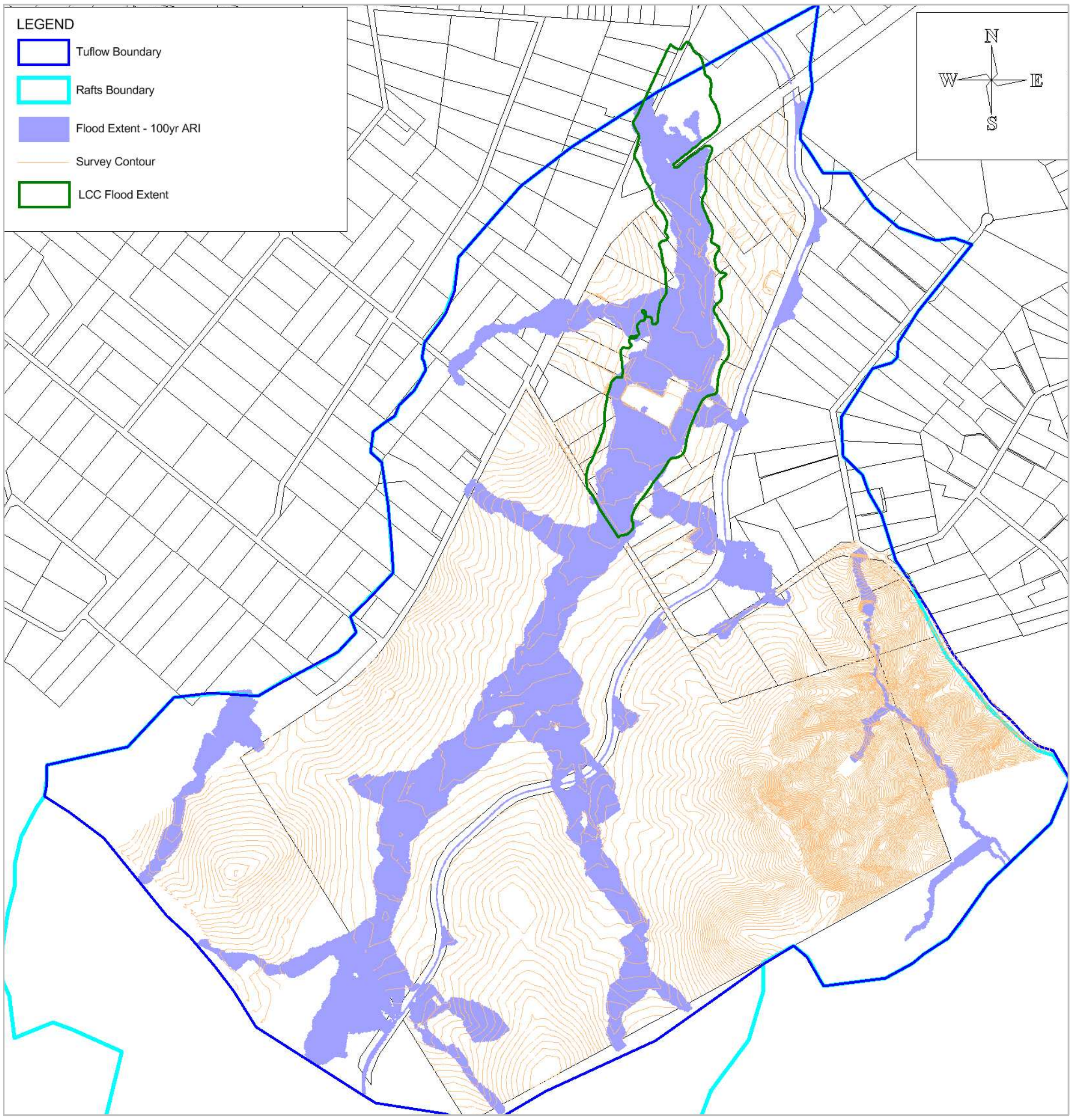


Figure 4-1: Comparison of 100 year ARI Flood Extents under Existing Conditions

The flood extents predicted by the TUFLOW model were also compared with 100 year ARI flood extents provided by Liverpool City Council (see **Figure 4-1**).

The 100 yr ARI flood extents estimated in this study are less than that identified by the Liverpool City Council studies. The possible reasons for the differences in these results are as follows:

- The previous LCC study used 1D cross sections of Bonds Creek based on survey undertaken in the early 1990's that were used to assemble a HEC-2 model at the time. No further ground survey was undertaken to inform the 2003 study. In the case of the current study the ground survey adopted for TUFLOW modelling purposes is more recent than the ground survey used to inform the LCC study and includes any changes which have occurred on the floodplain in the intervening years;

- The floodplain was classified into four hydraulic roughness categories in the TUFLOW model whereas two categories only were adopted in the 2003 HEC-2 model; and/or
- The discharges adopted in the 2003 HEC-2 model differ from the discharges calculated in the 2D floodplain model due to hydraulic routing and storage effects (which were not included in the steady-state HEC-2 model).

Based on the finding that consistent flood levels are estimated at Bringelly Road, and that the flood extents were in good agreement with the flood extents in the 2011 WCM report, it was concluded that the East Leppington model is suitable for assessing the impact of planned development on flooding.

The approach which was adopted to hydraulic modelling and the assessment of impacts was as follows:

- The floodplain model was extended approximately 1km downstream of the study area to eliminate any influence of the adopted downstream boundary condition The downstream model boundary condition was extracted from the ALN hydraulic model, also undertaken by Cardno, in the form of water level time series data at a section located downstream of Cowpasture Road.
- The TUFLOW model was run to estimate flood levels, flood extents, flood velocities and flood hazards during the 20 year ARI, 100yr ARI and PMF critical duration events under existing landuse conditions;
- The TUFLOW model was modified to represent the development scenario that may incorporate filling of sections of the floodplain and the creation of a floodway between Denham Court Road and Camden Valley Way. The modified TUFLOW model was run to estimate flood levels, flood extents, flood velocities and flood hazards during the 20 year ARI, 100yr ARI, 500 yr ARI and PMF critical duration events under post-development conditions;
- The impact of the proposed development on existing 100 year ARI flood levels was assessed with particular emphasis on levels on the downstream side of Denham Court Road and Camden Valley Way. Any impacts on existing flood levels upstream of Denham Court Road are within the area of development and would be managed by filling developable land to manage the flood risk, and
- Assess the impact of climate change by increasing rainfall intensity in the 100 year ARI storm by 30%.

4.2 Existing Conditions

The following existing hydraulic structures were identified and included in the TUFLOW model:

- 3 x 1.6m x 1.1m box culverts under Denham Court Road;
- 4 x 1.5m x 1.2m box culverts under Camden Valley Way;

- 3 x arches each with 2.1m base width and 1.32m height beneath the Sydney Catchment Authority canal (southwest of Denham Court Road);
- 2 x 300mm dia. pipes beneath the Sydney Catchment Authority canal (near St. Andrews Road)

Hydrographs for each of the storm durations summarised in **Appendix A.1** were exported from the **xprafts** model and imported into the TUFLOW model. These models were then run to determine the critical storm burst duration. It was found that the critical storm burst duration is either 2 hours or 9 hours depending on the location within the Precinct. In the upper reaches of the floodways the 2 hour storm is critical (upstream of Denham Court Road) while the 9 hour storm is critical downstream of Denham Court Road. On this basis the hydraulic models were run for the 2 hour and 9 hour durations and the results were compared to identify the peak flood levels for any given ARI.

The estimated flood levels, flood extents, flood velocities and flood hazards under Existing Conditions are plotted in the flowing Figures given in **Appendix B**:

- The estimated flood extents for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.1 – B.3**;
- The estimate peak flood depths for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.4 – B.6**;
- The estimated peak flood velocities for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.7 – B.9**;
- The estimated peak velocity x depth for the 20 year ARI, 100 year ARI and PMF events are shown in Figures **B.10 – B.12**; and
- The estimated flood hazard for the 20 year ARI, 100 year ARI and PMF events are shown in Figures **B.13 – B.15**.

It is shown in the existing flood extents that overland flows are included for various drainage depressions to give a general understanding of overland flow behaviour. Overland flows within these drainage depressions are not modelled in detail, for example, Camden Council has advised that there are a number of flumes and/or drainage structures under/over the SCA canal.

The purpose of the existing flood modelling was to determine flood behaviour and to identify the requirements for hydraulic design under developed conditions. For areas adjacent to the SCA canal the overland flows that appear under existing conditions would be conveyed in formalised drainage and overland flow systems. Furthermore, the existing flumes have limited capacity and could not be relied on under post-development conditions. In locations, such as at minor SCA canal crossings, detailed hydraulic design will need to be undertaken to ensure that stormwater flows that could potentially spill into the canal are directed elsewhere. Thus it is considered that the existing condition flood results give a reasonable indication of flood behaviour for the purposes of this rezoning investigation.

4.3 Developed Conditions

The TUFLOW model was modified to represent planned works including the re-alignment of a section of the floodway, local filling to confine floodwaters within the corridor, construction of a number of on-line retarding basins and upgrading of the following hydraulic structures:

- Existing 4 x 1.5m x 1.2m BCs under Camden Valley Way upgraded to a bridge with minimum 60m wide opening as part of the upgrade of Camden Valley Way; and
- Existing crossing near intersection of Camden Valley Way/St Andrews Road upgraded to 2 x 3.6m x 1.2m BCs.

The details on the retarding basin bunds and outlet structures which were included to the TUFLOW model are given in **Tables 3.1** and **3.2**.

Hydraulic structures were included in the existing condition model based on findings of the ground survey and verified by measurements taken during the site inspections. Several road upgrades are proposed in the Precinct with associated crossing augmentation. **Table 4-2** details the various structures included in the TUFLOW model, basin structures are detailed in **Table 3-2**. It was assumed that suitable detailed design contingencies will be made to limit the potential for blockage. Hence the results for Existing Conditions and Developed Conditions are based on nil blockage. The potential impact of blockage is discussed in **Section 4.7**.

The estimated flood levels, flood extents, flood velocities and flood hazards under Developed Conditions are plotted in the flowing Figures given in **Appendix B**:

- The estimated flood extents for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.16 – B.18**;
- The estimate peak flood depths for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.19 – B.21**.

Table 4-2: Hydraulic Structures included in the TUFLOW models

Location	Existing Conditions	Developed Conditions	Blockage Factor (refer Figure B.43)
Camden Valley Way (Bonds Creek crossing)	4 x 1.5m x 1.2m BCs under Camden Valley Way	3 x 18m Span Bridge	20% (based on findings from AR&R Project 11)
Camden Valley Way (Scalibrini Creek crossing)	2 x 3.6m x 1.2m BCs	2 x 3.6m x 1.2m BCs	Not Assessed – 50% Blockage factor is considered as part of the CVW design
Denham Court Road	3 x 1.6m x 1.1m BCs	1 x 3.9m (w) x 2.4m (h) BC plus 3 x 3.6m (w) x 1.2m (h) BC	50%
SCA canal crossing	3 x arches each with 2.1m base width and 1.32m height	3 x arches each with 2.1m base width and 1.32m height	50%

- The estimated peak flood velocities for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.22 – B.24**;
- The estimated peak velocity x depth for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.25-B.27**;
- The estimated provisional flood hazard for the 20 year ARI, 100 year ARI and PMF events are shown in **Figures B.28 – B.30**;
- The estimated flood extent, depth, velocity, VD and provisional hazard for the 500 year ARI event is shown in **Figures B.31 – B.35**;
- Water level differences between developed and existing conditions for 100 year ARI are shown in **Figure B.36**;
- Water level differences between 100 year ARI plus 30% rainfall intensity increase (climate change) and 100 yr ARI both under developed conditions are shown in **Figure B.37**, and;
- Water level differences between 100 year ARI plus blockage and 100 yr ARI with nil blockage both under developed conditions are shown in **Figure B.38**.

The results show that there is, in general, an increase in flood levels upstream of Denham Court Road. The level of increase varies from 0.01 to 1 m. Throughout the floodway the increase is approximately 0.0 m - 0.2m with pronounced increases within basins. The basins represent locations, usually at a road crossing, where a detention bund is retarding post development flows and increasing flood levels. It is shown in **Section 3.3** that the maximum depth of floodwater is 2.1m in Basin B5. The basin is to be configured with bunds built on the natural topography of the floodway, therefore applying 0.5m of freeboard gives a maximum bund height of 2.6 m.

The increases in flood levels upstream of Denham Court Road are not considered to be problematic, as they occur within the Precinct and within a single landholding and would only occur as a result of filling to contain the flood extents. It is expected that filling would raise ground levels to the extent that any feeeboard requirements would be met.

Any increases due to the construction of basins are also not considered to be problematic, as these basins would only be constructed as part of development within that part of the Precinct. Basins would be appropriately designed to provide freeboard between flood levels in the basins and adjoining development.

It is noted that the hydraulic modelling outlined in this Report allows for flow conveyance through the proposed drainage corridor, the retarding basins and bioretention basins identified in this Report.

4.4 Flood Impacts

Within the study area there is a section of land between Denham Court Road and Camden Valley Way (located in the Liverpool LGA) that may not be developed at the same time as the land upstream of Denham Court Road (located in the Campbelltown LGA).

Table 4-3: Comparison of 100 year ARI Flood Levels under Existing and Developed Conditions

Location	Existing Conditions	Developed Conditions
Downstream of Camden Valley Way (Bonds Creek crossing)	79.48	79.02
Downstream of Denham Court Road	86.32	86.35
Upstream of SCA canal crossing	94.43	96.73*

** - The Developed Condition flood level is increased to above existing levels by the Basin B1. This level could be reduced through detailed design of the basin to include excavation of the floodplain to achieve the required storage volume to lower the peak water level.*

Thus it is prudent to assess a potential interim developed condition where the Campbelltown LGA is developed only. This may be investigated by comparing the flood levels downstream of the new Denham Court Road under various conditions.

It has been advised that the existing road corridor will be retained in the interim condition and that it is planned to deviate Denham Court Road by constructing a new road and crossing on the upstream side of the existing road and crossing. Thus for reference purposes the flood levels on the existing road corridor are used, which is downstream of the new road alignment.

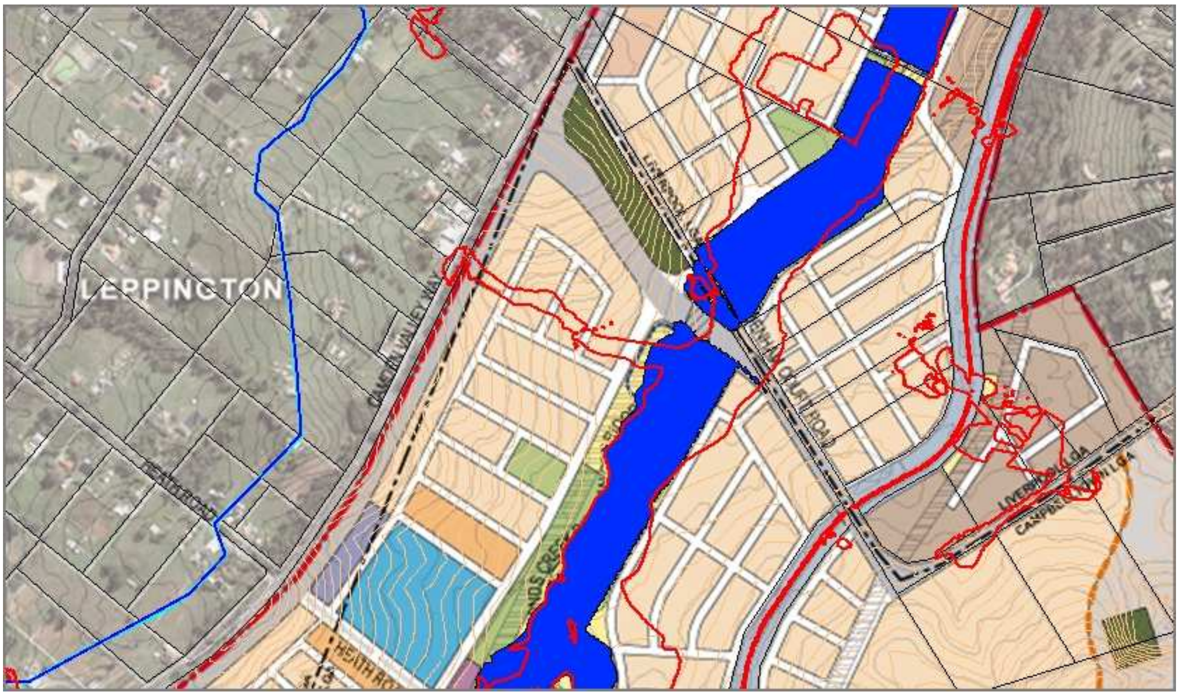


Figure 4-2: Comparison of 100 yr ARI Flood Extents at Denham Court Road

As indicated in **Figure B.36**, there are some flood level impacts on the old alignment of Denham Court Road and in the floodway downstream. As shown in the **Figure 4-2** (where the Existing Conditions flood extent is shown as a red line while the flood extent under Developed Conditions is shaded in blue) the floodway and road crossing will be re-aligned slightly to the west of the existing floodway. Hence the re-alignment of floodway and road crossing is the cause of the assessed flood level impacts even though the flood level remains unchanged between existing and developed conditions as shown in **Table 4-3**.

The existing Denham Court Road crown level and culvert has been retained in the Developed Condition model downstream of the new road and culvert because it is likely that it would be retained for a period to provide access to existing properties while development occurs upstream. It is expected that the road structure will remain in place for the foreseeable future, perhaps indefinitely as a pedestrian access. If the existing road is ever removed a detailed design and flood assessment should be undertaken to assess any impacts that may arise and design them out if necessary. If the road is taken out in the future it would be expected to have minimal impact to the operation of the basin upstream, because the road does not act as a hydraulic control due to the low level crown, broad width of the low point and the small size of the culvert. Impacts would be mitigated by ensuring downstream channel capacity is maintained and would not be expected to be significant in either extent or cost.

Likewise **Figure B.36** shows that there is both a slight increase of 0.01-0.1m and a decrease of up to 0.2 m on the downstream side of Camden Valley Way. It is noted that the slight increase is over approximately 30m and could be controlled by tail out works and/or bunding of the channel downstream of the Camden Valley Way.

As shown in **Figure 4-3** the existing flood behaviour would be modified by the construction of Basin B5 and the upgrade of Camden Valley Way. Consequently the flood impacts are attributed to a re-alignment of the flows in this area.

In the upstream reaches of the precinct there would be also minor changes to the alignment of flows as a result of filling on the edges of the corridors as shown by **Figure 4-4**. The narrowing of the 100 yr ARI flood extents by the development is minor however some creek training works may need to be designed and constructed to limit any impacts on upstream properties to acceptable levels.

Figure 4-3: Comparison of 100 year ARI Flood Extents at Camden Valley Way

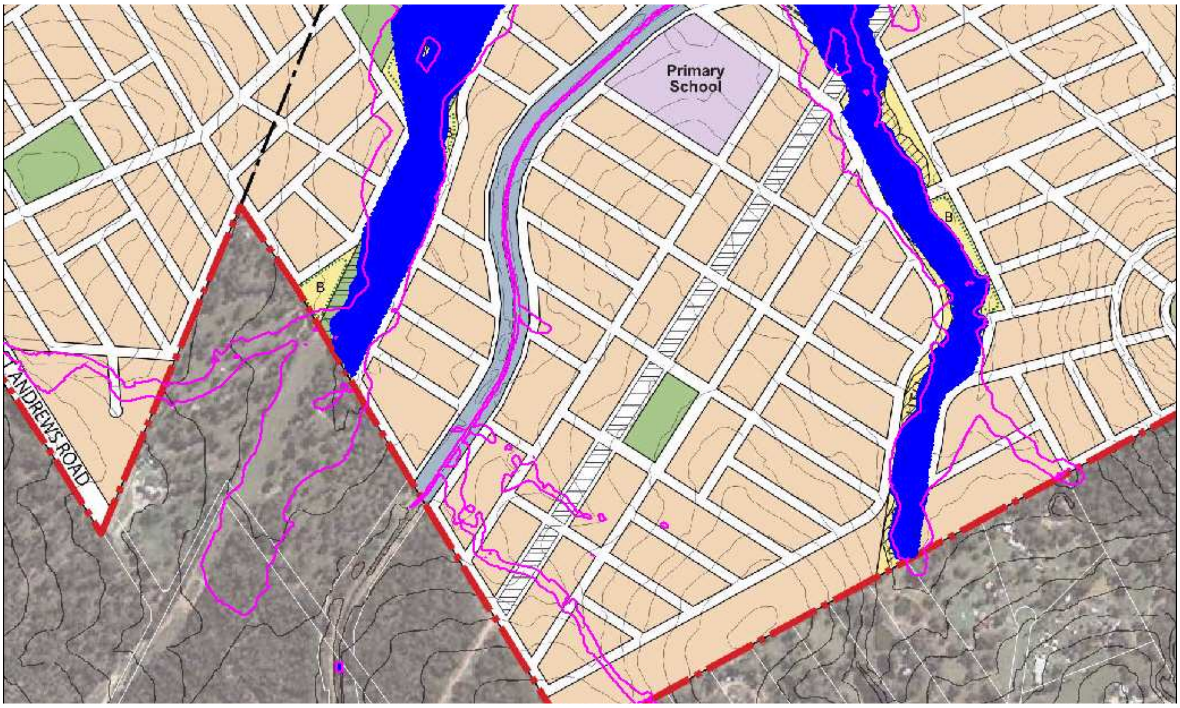


Figure 4-4: Comparison of 100 year ARI Flood Extents in the upstream reaches of the Precinct

4.5 Floodplain Management

The flood extents under Existing Conditions vary throughout the precinct from approximately 75m to 300m in width north of Denham Court Road in part as a result of man-made modifications of the existing terrain including filling of sections of the floodplain and the construction of farm dams on the drainage lines.

In order to appropriately manage flooding an assessment of the impact of re-shaping of the floodplain within the precinct was undertaken.

As part of the ultimate development of the precinct including the land within the Liverpool LGA, it is proposed that a broad channel would be excavated into the existing floodplain and vegetated to formalise the floodway. There are a number of obstructions within the existing floodplain that would be removed as part of the development. Under Developed Conditions a constructed floodway was included in the TUFLOW model from Denham Court Road to Camden Valley Way**.** This floodway includes suitable geometric variation to accommodate bio-retention measures and retarding basins. The width of the floodway is on average 120 m and the depth is approximately 1.2 m. A Manning roughness value of 0.07 was adopted across the full width of the floodway to represent extensive re-vegetation within this corridor.

The concept geometry of this floodway is given in **Figure 4.5.** A tributary to the floodway is shown on the ILP on the western side, upstream of Basin B5. This tributary has not been included in the TUFLOW model and the assumption is that the flows arriving from the tributary would arrive in the floodway via a pipe and associated overland flowpath. If the tributary is developed into an open channel then it is expected that flows from it would be routed through a natural channel and arrive into the floodway slightly retarded in comparison to what has been included in TUFLOW.

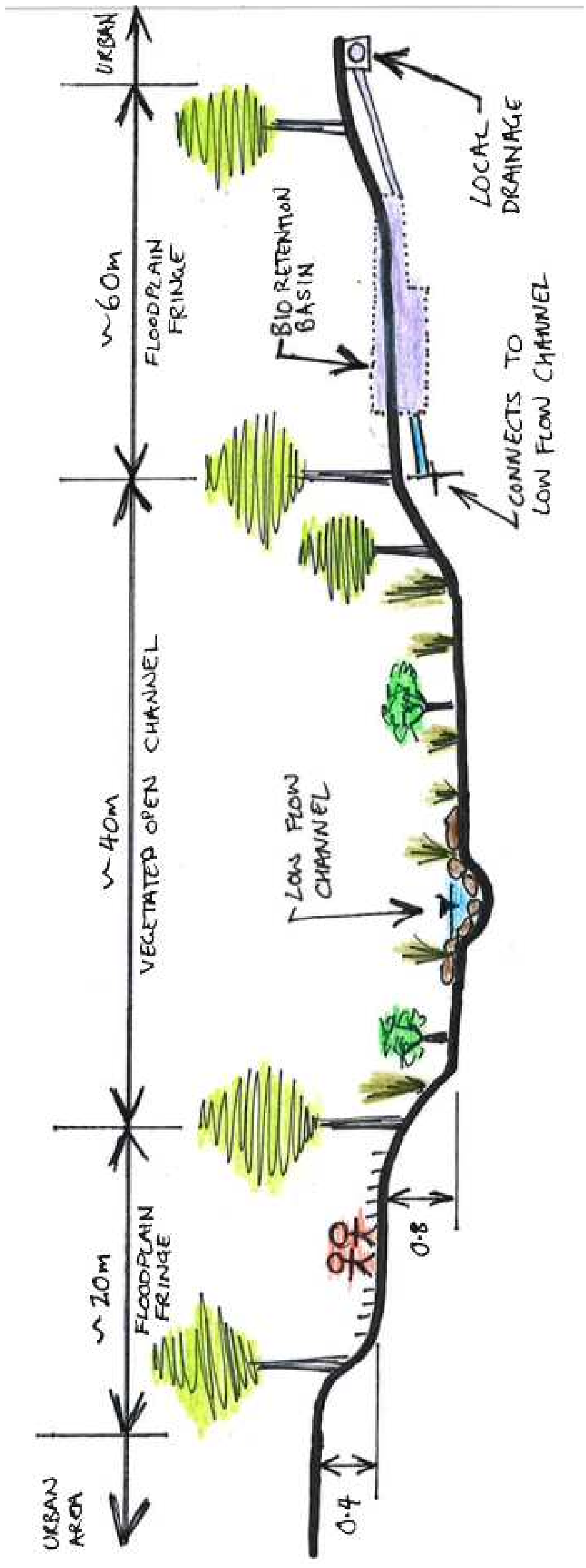


Figure 4-5: Concept Geometry of Floodway between Denham Court Road and Camden Valley Way

Table 4-4: Approximate Pipe Flow Capacities (m^3/s)

Pipe Diameter (mm)	1% Grade	3% Grade	5% Grade
600	0.7	1.2	1.5
750	1.2	2.0	2.8
900	2.0	3.5	4.5
1050	3.0	5.2	7.0

Figure B.2 also identifies a number of minor tributaries of Bonds Creek which drain across Camden Valley Way from the East Leppington Precinct. Under Developed Conditions with 100 yr ARI peak flows vary from $0.8m^3/s$ to $6.6m^3/s$ depending on location. These flows would be typically conveyed by a combination of piped drainage and overland flow on proposed roads. **Table 4-4** indicates potential capacities provided by various pipe sizes.

As Camden Valley Way is to be upgraded, a number of large diameter culverts are proposed to convey flows from these minor tributaries. These culverts are designed to convey flows up to the 100 year ARI event in order to maintain a high level of serviceability for Camden Valley Way.

Within the Precinct, it is not proposed to design piped drainage to convey flows up to the 100 year ARI event but rather to design the piped drainage to convey flows to the standard required by relevant Council guidelines and for excess flows to be conveyed overland on roads and/or overland flowpaths as appropriate.

The proposed drainage network and surcharge arrangements would be subject to further assessment at the detailed design stage. However, the existing grades between Camden Valley Way and Bonds Creek provide significant scope to allow surcharge arrangements which do not negatively impact on upstream culvert capacity. These would include appropriately sized surcharge pits either directly downstream of the crossings or at a suitable location within roadways. Alternatively they could consist of a tail out arrangement with entry pits for design flows and overland flow provisions for major flows.

4.6 Climate Change Assessment

The potential impacts of climate change on 100 yr ARI flood behaviour were assessed increasing the 100 year ARI by rainfall intensities by 30%. Hydrographs for the critical storm durations were exported from the **xprafts** model and imported into the TUFLOW model. A comparison of the impact of climate change on 100 yr ARI design flood levels is shown in **Figure B.37**. It will be noted that the design flood levels increase significantly at the floodway crossings. The largest local increases would be 0.33m at the SCA canal, 0.18m at Denham Court Road and 0.91m at Camden Valley Way.The increases are less significant in the floodway upstream of Denham Court Road.

4.7 Culvert Blockage Assessment

A sensitivity assessment of the impact of culvert blockage was undertaken based on the degrees of blockage given in **Table 4-2**. It was assumed that the proposed road crossings and basin outlets would include measures to reduce the risk of blockage and no blockage was included at these locations. The impact of partial blockage on 100 yr ARI flood levels is given in **Figure B.38**. The results show that the 100 yr ARI flood levels are most sensitive to blockage at the SCA canal where an increase in flood levels of more than 0.5m is predicted. Negligible differences and/or reductions in 100 yr ARI flood levels are predicted elsewhere.

4.8 Emergency Management Strategy

An assessment of the potential to manage residual flood risk was undertaken by assessing the ability of the public to seek refuge from floodwaters and to evacuate if needed during extreme floods up to the PMF. For the most part it is expected that the public would not require evacuation because the duration of flooding in the study area is typically less than 6 hours for events greater than the 100 year ARI event. Longer duration storms would not create flood levels of the same magnitude as the peak duration events and therefore long duration flooding as a result of extreme events if of no concern for emergency management. Some habitable parts of the floodplain are affected by the PMF and suitable provisions should be made so that the public can safely escape flood inundation if necessary. It is recommended that these areas should be developed with muster points close to the flood prone area and safe egress pathways so that in the event of an extreme flood any affected members of the public can quickly take refuge until the flood subsides. This approach is consistent with emergency management protocols for the Camden and Campbelltown LGAs (SES, 2010).

Development of safe egress routes should take into consideration pedestrian and vehicular safety with three velocity x depth criteria used as follows:

Table 4-5: Velocity x Depth Criteria

Velocity x Depth (m^2/s)	Comment
≤ 0.4	Typically adopted as a limit of stability for pedestrians
0.4 – 0.6	Unsafe for pedestrians but safe for vehicles if overland flood depths do not exceed 0.2 m (approx.)
> 0.6	This is typically adopted as a limit of stability for vehicles

The peak velocity x depth experienced during the PMF is plotted onto the ILP in **Figure B.27** while the provisional flood hazard is shown in **Figure B.30**.

It has been advised that Camden Valley Way is to be upgraded by RMS with the proposed road to be above the 100 year ARI flood levels. Notwithstanding this upgrade it is not expected that the Camden Valley Way would serve as an evacuation route during the PMF. This is confirmed further when referring to the modelling results in **Figure B.30** showing that the road crossing is subject to flooding with high provisional hazard in the PMF. The Precinct is split by Bonds Creek, its main tributary and the SCA canal with road crossings proposed at various locations. In determining safe egress from an extreme flood, it is necessary that the proposed road layout take into consideration the ability for resident to access a local road and make their way out of the floodplain without the need to cross neighbouring properties.

4.9 Update of the ILP in the Liverpool LGA

Following the flood modelling and mapping of model results there have been some minor changes to the layout of the floodway and surrounding land uses in the Liverpool LGA. These changes are documented in ILP version 12.6 and involve the following with respect to flood behaviour:

1. Realignment of the western side of the floodway directly downstream of Denham Court Road
2. Relocation of the road crossing directly downstream of Basin B4 further to the north
3. Realignment of the eastern side of the floodway in the vicinity of Basin B5

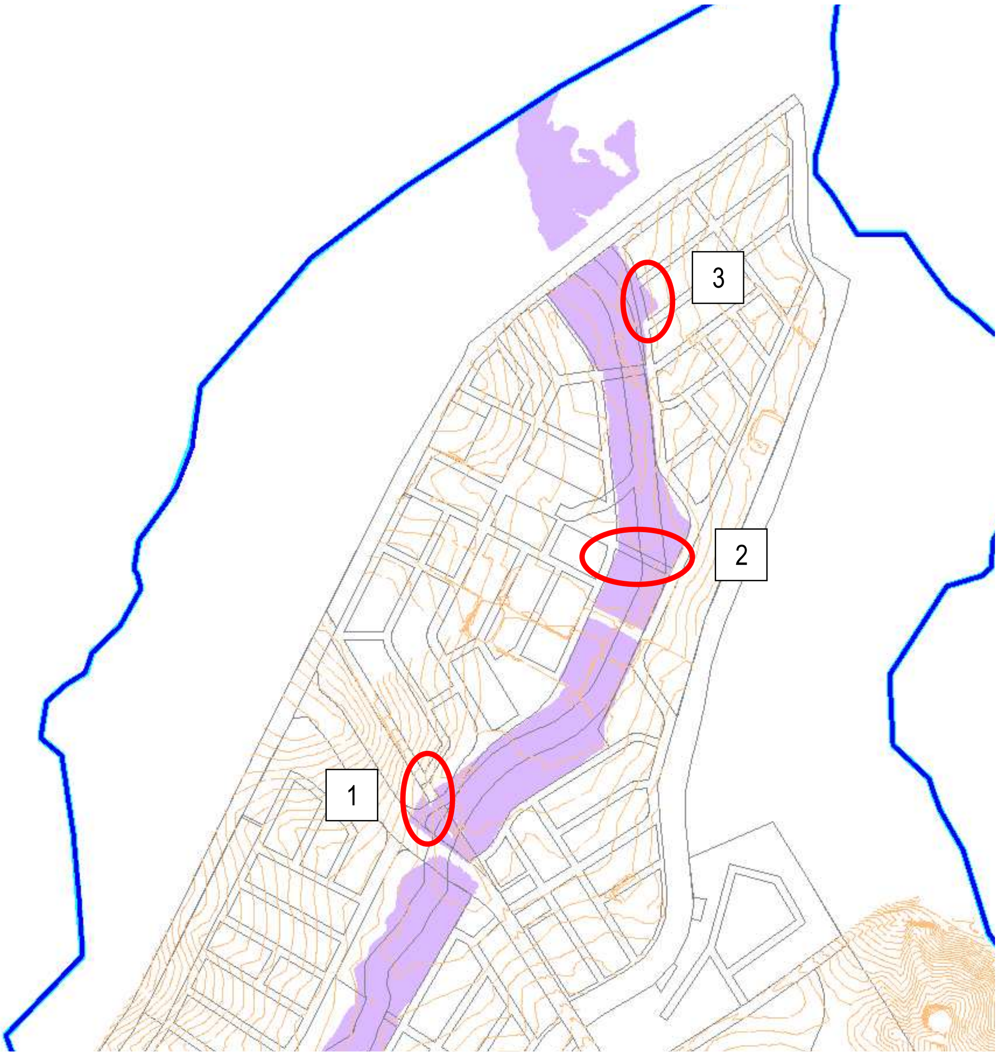
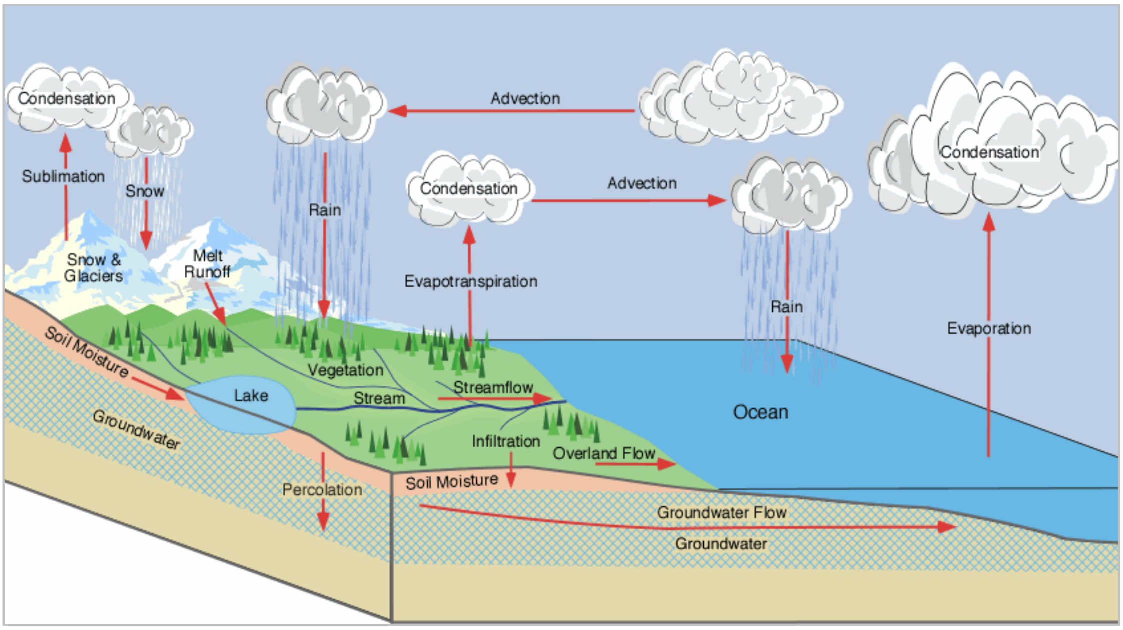


Figure 4-6: 100 year ARI flood extent of ILP 8.4 overlaid onto ILP 12.6

These changes would require minor adjustment to the geometry of the floodway and basin design from that modelled. The results of this study indicate that there is sufficient flexibility to include these adjustments considering that this segment of the precinct is to be modified by construction of an open channel and two on-line retarding basins. The adjustments would involve change to the shape of the floodway geometry to ensure that appropriate allowance is made for hydraulic capacity in the open channel and would be incorporated into the detailed design process. In addition, Basin B4 would be relocated slightly further north so that the proposed road crossing of Bonds Creek would act as the hydraulic control for the basin, in a similar manner to that proposed for Denham Court Road. Basin B5 would require slight adjustment of the footprint whilst maintaining the storage volume through additional depth elsewhere. The area on the floodway fringe would be filled to create flood free land for urban development and flexibility in the final levels of the filling would also allow for such changes to the ILP. It is therefore concluded that the changes included in the ILP version 12.6 would not adversely affect flood behaviour and would be able to be managed as part of the detailed design process ordinarily required as development within the precinct progresses. As such, the amended basin locations have been incorporated into ILP version 12.6.

5 WATER SENSITIVE URBAN DESIGN STRATEGY

The purpose of a Water Sensitive Urban Design (WSUD) strategy is to identify suitable methods for the management of stormwater in a sustainable manner that is integrated with other aspects of the water cycle management plan. This is aligned with the principles of WSUD where the urban development considers its implications for the total water cycle (refer **Figure 5.1**).



(Source: http://www.physicalgeography.net)

Figure 5-1 Total Water Cycle

The strategy will inform where water management controls are to be located in the Indicative Layout Plan (ILP) and document requirements for the preparation of a site specific Development Control Plan (DCP).

This WSUD strategy focuses on better ways of managing the available water resources by looking beyond the traditionally separate consideration of water supply, wastewater and stormwater services.

The objectives of the WSUD strategy are to:

- Integrate stormwater controls into open space and drainage corridor to allow combined water management and recreation uses;
- Soften the structural elements of stormwater controls to increase visual amenity and allow for embellishment of the landscape;
- Manage stormwater quantity to ensure that peak flows during the 2 yr and 100 yr ARI storm events are no greater than pre-development conditions;
- Manage stormwater quality to ensure that pollutants are reduced to levels according to best management practice; and
- Consolidate stormwater quality and quantity controls in order to control construction costs and reduce allocation of valuable land for water management purposes.

5.1 Stormwater Quality Management

It is proposed to manage stormwater quality within the East Leppington Precinct using a treatment train approach as shown in **Figure 5-2**.

Potential stormwater quality management measures are outlined in the **Table 5-1.**

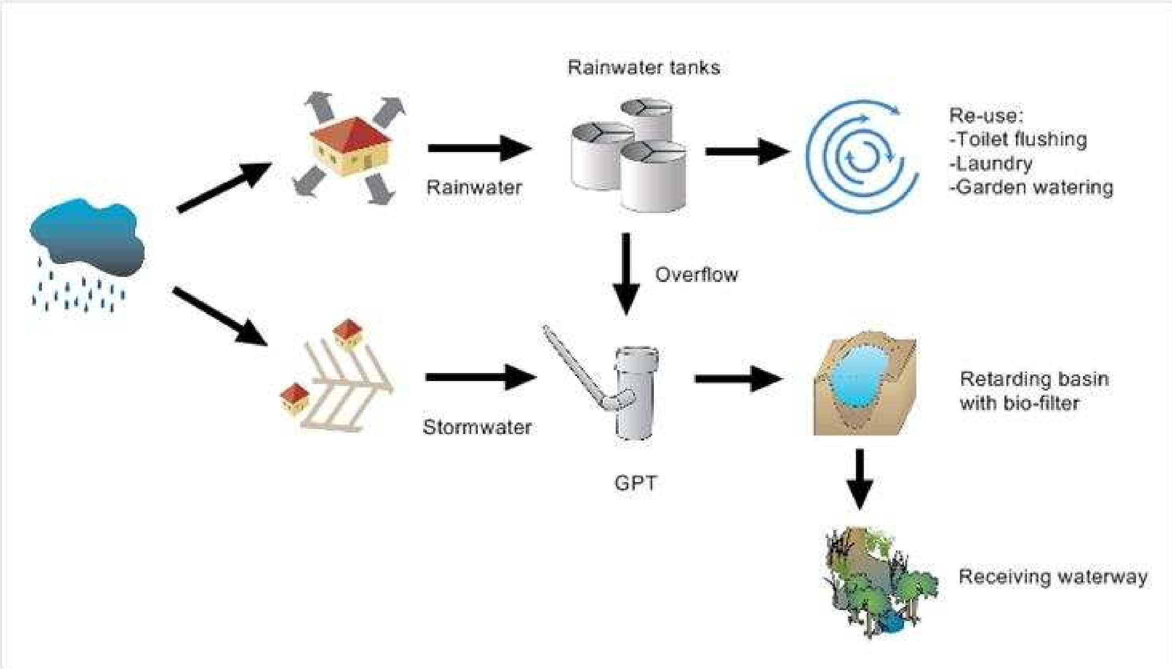


Figure 5-2: Stormwater Treatment Train

Table 5-1: Stormwater Management Measures

Element	WSUD measure	Description
Rainwater	Rainwater Tanks	Reduce potable water demand by supplying reclaimed water for toilet flushing, laundry use, garden irrigation around homes and irrigation of dedicated passive recreational areas in excess of the BASIX requirement for potable water consumption reductions.
Stormwater	Gross Pollutant Trap (GPT)	Neighbourhood scale control of gross pollutants, suspended solids and phosphorous in purpose designed devices. Propriety products are most appropriate for underground drainage systems and trash racks/deflectors are most appropriate for the inlets to retarding basins.

Element	WSUD measure	Description
	Bioretention basins	Bioretention basins have been proposed to control stormwater quantity at the confluence of local drainage lines and perennial streams. The basins will incorporate a GPT at the inlet and a bio-filter area at the low point to treat low flows from frequent storms. The bioretention system will be sized to meet best practice targets for TSS, TP and TN and would discharge to the existing waterway/floodplain.
	Stormwater Harvesting	Stormwater is a resource that can be harvested and re-used for open space irrigation. Opportunities to harvest treated stormwater exist from the bioretention systems in the sub-catchments. Further treatment may be required prior to storage and re-use. These opportunities may be explored further during future stages of the planning process.
Groundwater	Infiltration	Retarding Basins would be lined with locally available clay soils in a manner that allows for infiltration, where appropriate. Further geotechnical investigations will inform the appropriate composition of the liner to complement surrounding soils. In some instances, such as in high salinity areas, infiltration is not preferred.

Note – Rainwater tanks have not been considered as a stormwater quality treatment device

5.2 Preliminary Sizing of Treatment Measures

The effectiveness of stormwater quality improvement measures has been assessed using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC), Version 5 software. MUSIC is the industry standard water quality analysis software used for conceptual design of treatment measures.

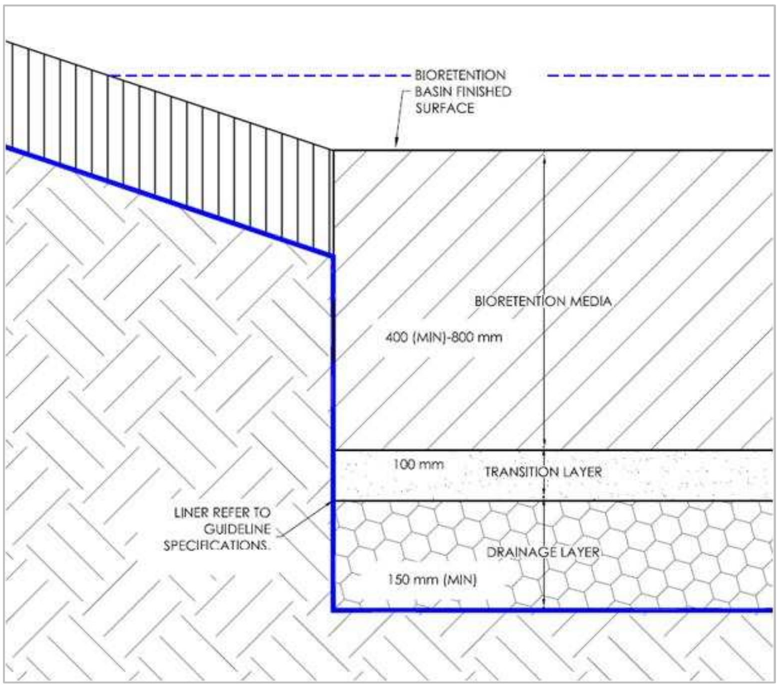
Target reduction of stormwater pollutants will be in accordance with best management practice as follows:

- 90% capture of gross pollutants;
- 85% reduction of total suspended solids;
- 65% reduction of total phosphorous; and
- 45% reduction of total nitrogen.

Bio-retention systems offer a suitable method of stormwater treatment for the development. An investigation of the soils found in the Leppington area has identified that salinity is moderate to high (Cardno, 2011). Therefore the soils are not conducive to infiltration, retaining water or supporting certain types of vegetation.

Soils would require treatment with gypsum to improve their ability to hold water and support plants. Therefore the precincts are more suited to the use of bio-retention than constructed wetlands. In general the constructed wetlands require a far greater land take to achieve the pollutant reduction targets. In addition wetlands require the retention of water to support plant growth, not a suitable approach considering the saline soils of the area.

Bio-retention systems operate by passing runoff through prescribed filtration media planted with specific vegetation which provides stormwater treatment through filtration, extended detention and biological uptake. Extended detention above the basin finished surface controls the volume of stormwater to be treated. A typical cross section through a bioretention system is shown in **Figure 5-3**.



Source: Sydney Metropolitan CMA, Typical Drawings for WSUD

Figure 5-3: Typical Bio-retention Detail

A key criterion is the selection of the bio-retention filter media to provide sufficient hydraulic conductivity and sufficient water retention to support vegetation growth with a minimum 400mm deep filter required for plant establishment. The transition layer separates the bioretention media from the drainage layer below. The drainage layer contains perforated pipes which convey treated stormwater to the drainage system.

The sizing of water quality management measures was undertaken using MUSIC and is described in **Appendix C.** The required bio-retention area for each sub-catchment is outlined in **Table 5-2** below with catchment delineation included in **Appendix C**. The basin sizes and filter area requirements have been interpreted into provisional basin footprint in the Draft ILP.

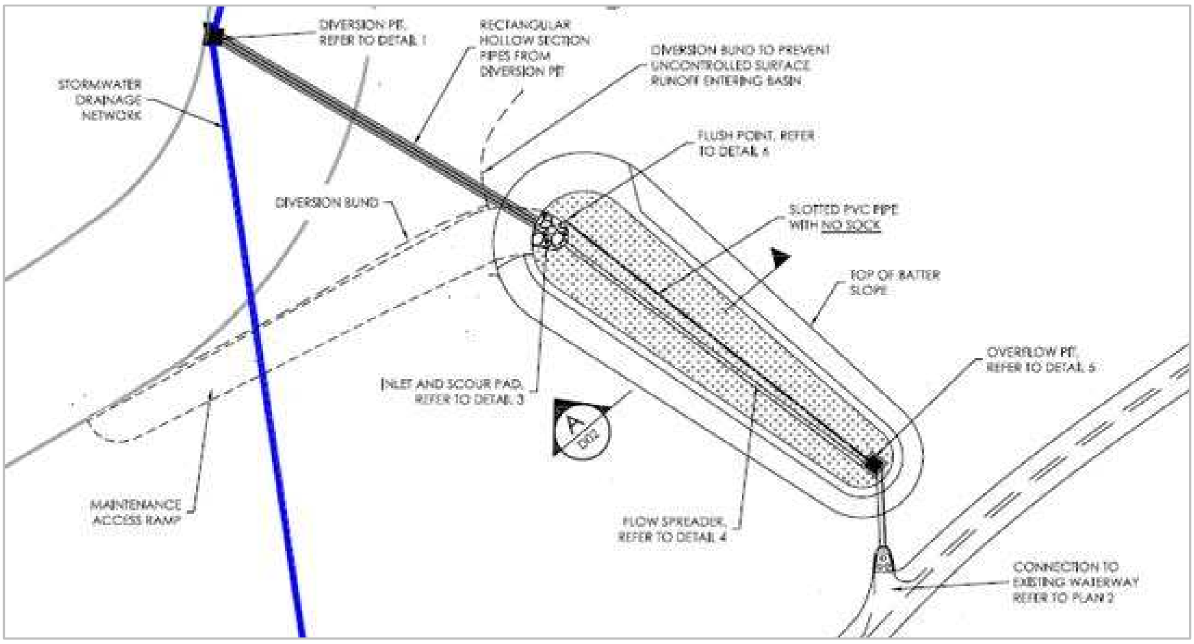
Table 5-2 MUSIC Water Quality Modelling Results Summary

Catch ID	Area (ha)	Filter Area (m^2)	ILP Basin ID	Reduction %			
				TSS	TP	TN	GP
EX-1	14.6	438	BR11	91.8	65.3	59.2	100
EX-2	9.0	269	CLB8	91.3	65.6	55.9	100
EX-3	13.4	402	BR10	92.2	65.3	61.9	100
OB2-1	16.6	499	BR10	92.4	65	61.2	100
OB2-2	14.9	447	BR8	92.5	65.3	62.7	100
OB2-3	13.0	391	BR9	91.7	65.4	59	100
OB2-4	8.2	245	BR9	91.2	65.3	58.6	100
OB2-5	6.7	202	BR9	93.4	65.1	61.4	100
OB2-6	5.7	171	BR7	88.5	65	55.9	100
OB2-7	10.7	-	-	-	-	-	-
OB2-8	3.5	105	BR9	91.4	65.5	58.4	100
OB3-1	6.5	196	CLB7	90.3	65.8	57.2	100
OB3-2	6.6	199	CLB7	89.6	65.4	56.5	100
OB3-3	21.8	653	BR12	92.5	65	60.9	100
OB3-4	4.0	-	-	-	-	-	-
OB4-1	7.5	224	CLB7	92	65	57.6	100
OB4-2	5.6	169	CLB6	93.2	65.1	62.1	100
OB4-3	12.0	359	CLB7	90.6	65.5	59	100
OB4-4	14.9	448	CLB6	92.3	65.5	60.3	100
OB4-5	4.2	-	-	-	-	-	-
OB4-6	14.0	421	CLB6	93.8	65	65	100
OB5-1	2.5	74	BR5	91.8	65.1	61.6	100
OB5-2	4.4	133	BR4	90.9	65.1	59.3	100
OB5-3	7.1	212	BR5	-	-	-	-
OB5-4	9.8	293	CLB5	93.3	65.2	59.2	100
OB5-5	17.3	520	BR6	93.8	65.2	61.4	100
OB5-6	11.5	344	CLB4	92.1	65.2	60.1	100
OB5-7	15.4	-	-	-	-	-	-
OB6-1	13.5	406	BR2 / BR3	92.8	65	62.8	100
OB6-2	10.0	300	BR2 / BR3	94.1	65.2	65.7	100
OB6-3	11.8	354	BR2 / BR3	91.8	65	60.1	100
OB6-4	17.5	525	BR13	92.1	65.4	60.9	100
OB6-5	8.2	-	-	-	-	-	-
OB7-1	10.5	316	BR1	89.6	65.1	54.8	100
OB7-2	14.7	441	CLB1	90.1	65.7	59	100
OB7-3	12.6	365	CLB3	90.3	65.4	58.2	100
OB7-4	4.7	142	CLB2	94.1	65.2	62.3	100
OB7-5	7.1	-	-	-	-	-	-

5.3 Typical Bio-retention System Details

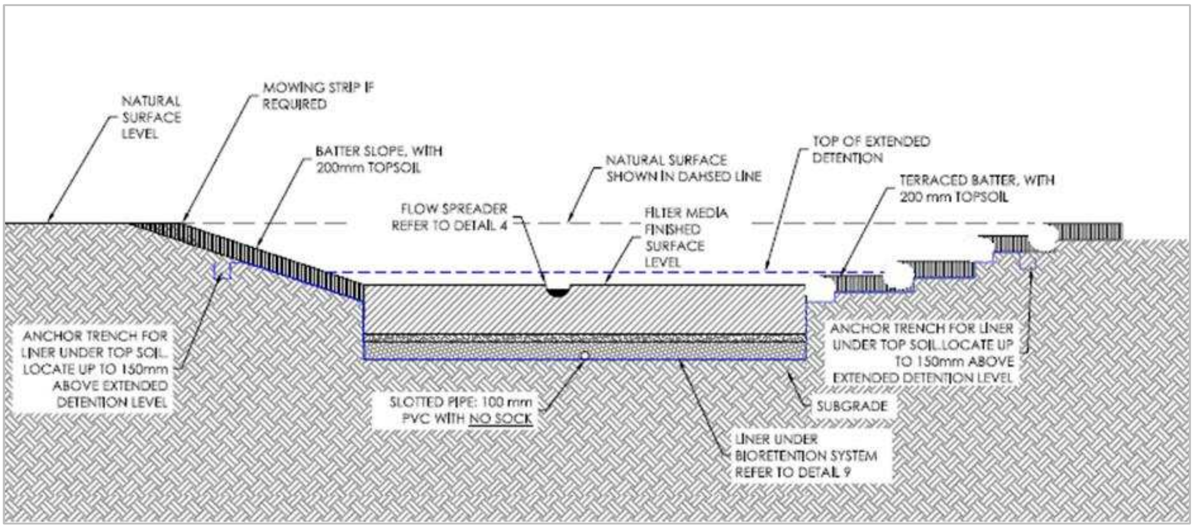
Bio-retention measures may take the form of basins, swales and tree pits depending on contributing catchment size. Bio-retention system locations are included in **Figure C.1** and are indicative only at this stage. There may be scope to co-locate bio-retention systems within retarding basins, where possible.

Typical details for bio-retention measures are outlined in the following Figures.



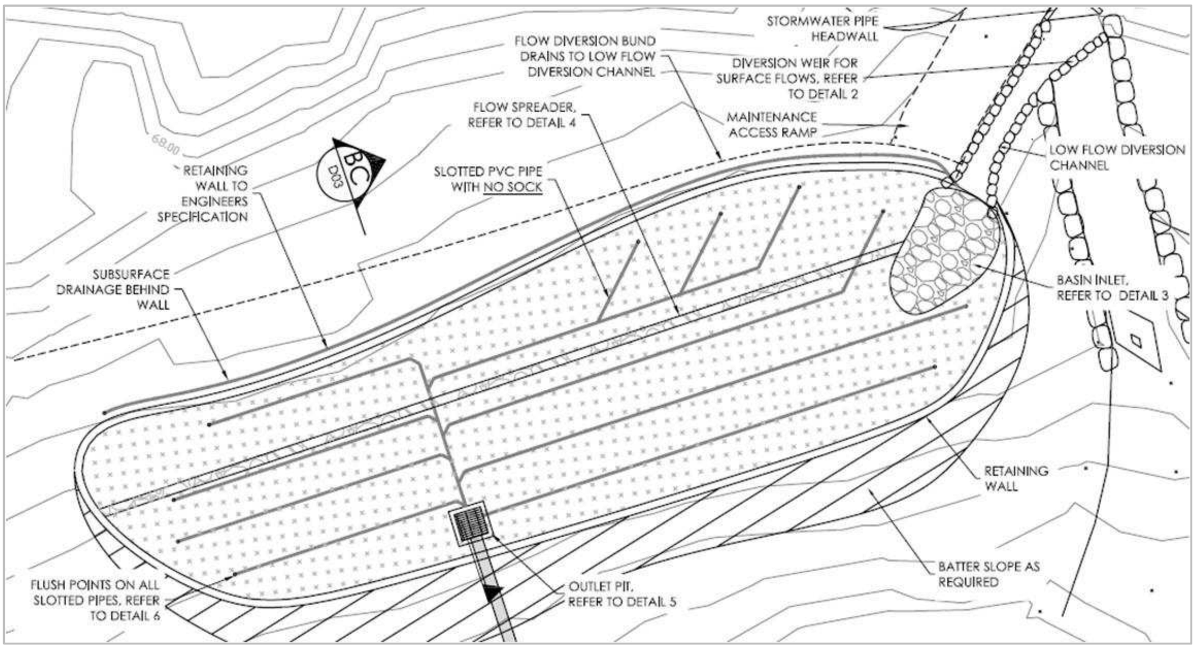
Source: Sydney Metropolitan CMA, Typical Drawings for WSUD

Figure 5-4: Typical Bio-retention Layout – Flat Terrain



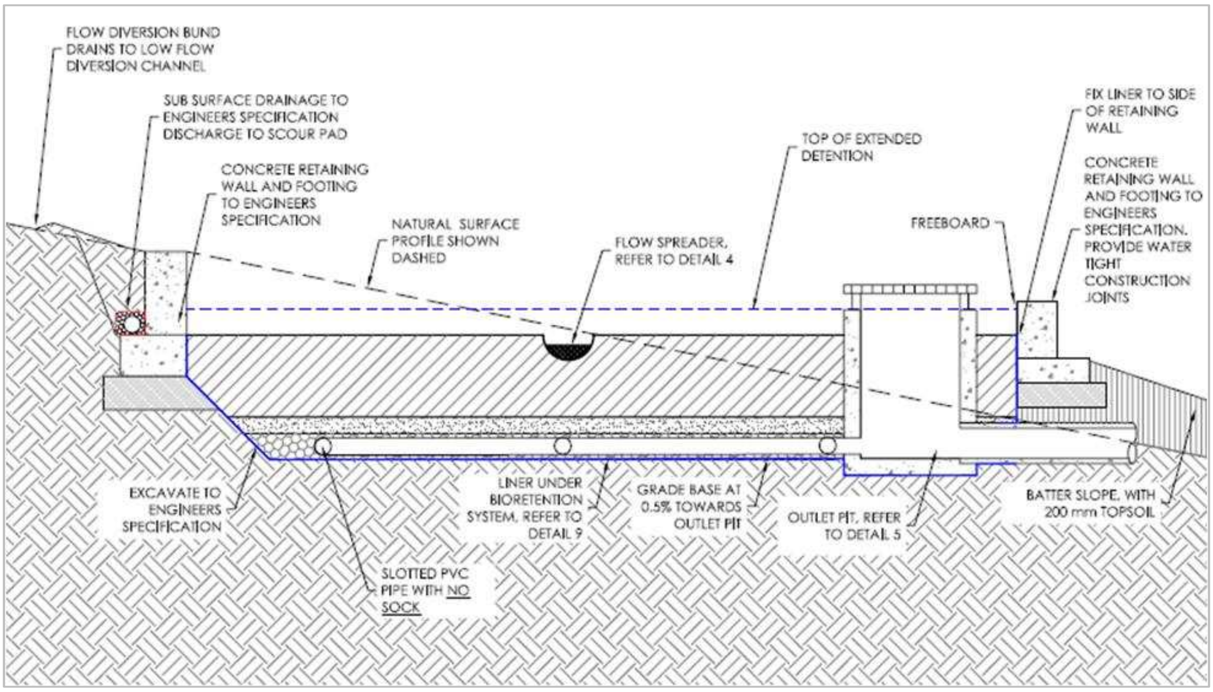
Source: Sydney Metropolitan CMA, Typical Drawings for WSUD

Figure 5-5: Typical Bio-retention Detail - Flat Terrain



Source: Sydney Metropolitan CMA, Typical Drawings for WSUD

Figure 5-6: Typical Bio-retention Layout – Steep Terrain



Source: Sydney Metropolitan CMA, Typical Drawings for WSUD

Figure 5-7: Typical Bio-retention Detail - Steep Terrain

5.4 Implementation

The implementation of the above water management strategy requires land acquisition and a range of works including drainage corridors, detention basins and bio retention basins. Appendix D includes a map which outlines the items required. Estimated quantities for land acquisition and the range of works discussed have been estimated by Cardno and can be found in Appendix D. In considering the construction of bioretention basins it should be noted that filter areas described in this report are the minimum areas of filter media required to meet the treatment criteria and determined by modelling. The filter areas identified in Table 5-2 are the minimum areas required in order to meet the necessary reductions of Total Suspended Solids, Total Phosphorous and Total Nitrogen. Whilst these areas are theoretically sufficient to provide treatment to the specified levels, numerous factors will impact on the filter's efficiency, these include:

- Reduced in field hydraulic conductivity of media during the life of facilities due to siltation, construction tolerances, vegetation build up and non-uniform hydraulic flow.
- Varying catchments and treatment rates to suit urban design and topographic constraints.

Given these factors and the need to ensure sufficient allocation of space, it is recommended at the early planning stages to allow for approximately twice the theoretical filter area. The filter area however only comprises a small component of the water quality treatment facilities with the whole area required consisting of batters, overflow spill ways, stilling basins, vehicular access, piped drainage infrastructure. Taking topography into account this typically results in a total area of facilities of approximately 6 times the recommended filter area, or, typically 3% of the catchment. Refer in Appendix C for typical layout of bio-retention facilities and relationship to filter area.

5.5 Operations and Maintenance

The operation of WSUD measures is reliant on periodic maintenance to ensure that various elements of the measure are in good working order. WSUD measures comprise, for the most part, natural materials which can be quickly degraded by high volumes of stormwater. Stormwater can contain gross pollutants and sediment that can smother filtration media, plants and drainage structures. In addition stormwater can also reach high velocities that can cause scour and erosion.

Gross Pollutant Traps (GPTs) need to be regularly maintained to remove captured pollutants. Often these devices are located underground and can be neglected if maintenance routines are not observed. Failure to maintain GPTs can exacerbate stormwater pollution by potentially releasing nutrients that are bound to sediments captured in GPTs.

In light of these issues it is recommended that the WSUD measures be included in the public domain so that they are visible to the public and are accepted as part of the landscape. Segregation of WSUD measures with fencing and dense peripheral vegetation can lead to the measure becoming isolated and neglected. Integration of the WSUD measures and the open spaces should promote regular maintenance to ensure that the amenity of the public open space. Local land care groups can also be encouraged to take responsibility for local assets and to share maintenance duties with Councils.

The construction period is one of the main threats to fouling of WSUD measures if the construction is not staged in a way that will protect the measures. Release of sediments into stormwater during construction is common and although soil and water management controls are put in place, they are often neglected and fail during storms. The following recommendations are made to protect the measures from fouling during construction:

- Locate the WSUD measure off-line until the commissioning phase of the development. This will ensure that any stormwater generated during construction is routed around the WSUD measures;
- Delay landscaping of the WSUD measures to the final stages of construction to reduce the risk of surface degradations and plant loss; and
- Temporarily create a small inlet zone to retarding basins and bio-filters that will accept small amounts of local stormwater during construction. This will allow plants to establish in the greater area of the basin/filter without risk of fouling.

The design life of the WSUD measures is highly dependent on the maintenance regime. If a maintenance regime is followed then the life of the WSUD elements will be maximised and a reliable level of pollutant capture will be achieved. Note that an establishment period will be required to ensure that any vegetation included in the WSUD measure is healthy and robust. A vegetation management plan should be provided with the detailed design of measures such as retarding basins and bioretention systems that includes full details on the procurement and establishment of plants.

A maintenance schedule for proposed WSUD measures is outlined in **Table 5-3**.

Table 5-3: WSUD Maintenance Schedule

WSUD Measure	Maintenance Actions	Frequency	Waste Management	Responsible Party
Rainwater Tanks	Clean out first flush device of any sediment and debris build up.	Quarterly or after each storm event of 10mm in rainfall depth or more	Dispose of in-organic material to waste disposal facility.	Property manager / Owner
	Drain tank and clean sediment/organic matter from tank base	Bi-annually	Re-use organic material in separate gardens or landscaped areas	
Gross Pollutant Traps	Remove collected pollutants	Quarterly or after each storm event of 20mm in rainfall depth or more	Dispose of in-organic material to waste disposal facility.	Council
	Check inlet and outlet structures for signs of blockage	Annually	Dispose of in-organic material to waste disposal facility.	
	Replace filter mesh (if included in device)	Every 5 years	Nearest waste disposal facility	
Retarding Basins	Remove collected pollutants on the surface	Quarterly or after each storm event of 20mm in rainfall depth or more	Dispose of in-organic material to waste disposal facility. Use organic material as mulch.	Council / Landcare Group
	Check surfaces for any signs of erosion or displacement of surface treatments/vegetation	Quarterly or after each storm event of 20mm in rainfall depth or more for the first 24 months and annually thereafter.	No waste – collect dislodged materials and re-use.	
	Replace damaged plants	Annually	Re-use organic material in separate gardens or landscaped areas	
	Check integrity of basin inlet and outlet structures and replace scour protection where necessary	Annually or after each storm event of 100mm or more	Re-use organic material in separate gardens or landscaped areas, replace rock where appropriate.	Council
	Check integrity of basin walls and make appropriate structural repairs where necessary	Annually or after each storm event of 100mm or more	No waste – collect dislodged materials and re-use.	

WSUD Measure	Maintenance Actions	Frequency	Waste Management	Responsible Party
Bio-retention Systems	Remove pollutants collected on the surface	Quarterly or after each storm event of 20mm in rainfall depth or more	Dispose of in-organic material to waste disposal facility. Use organic material as mulch.	Council / Landcare Group
	Flush stand pipes of bio-filter	Half yearly or after each storm event of 20mm in rainfall depth or more	Collect material flushed into stormwater pits and re-use as mulch.	
	Check surfaces for any signs of erosion or displacement of scour protection/soil/mulch	Quarterly or after each storm event of 20mm in rainfall depth or more for the first 24 months and annually thereafter.	No waste – collect dislodged materials and re-use.	
	Replace damaged plants	Annually	Re-use organic material in separate gardens or landscaped areas	
	Replace filtration media	Every 5 years as a minimum or up to 20 years as a maximum depending on pollutant load from the catchment.	Dispose of in-organic material to waste disposal facility. Use organic material as mulch.	Council

6 CONCLUSIONS

This Water Cycle Management Report has been prepared as part of the Precinct Planning process for East Leppington within the Southwest Growth Centre.

Stream classification is based on the Strahler Stream Ordering and Waterway Classification System and has been adopted using information provided by Department of Planning and Infrastructure and Campbelltown Council. The Strahler Stream Ordering and Waterway Classification System assigns an "order" based on the number of tributaries associated with each waterway. The recommended corridor widths are 50 m for Bonds Creek upstream of the main tributary, 50 m for the main tributary itself and 80 m for Bonds Creek downstream of the main tributary. For the tributary of Scalabrini Creek it is expected to be contained within the existing topography.

There are other minor watercourses which have been disturbed and modified over time. Flows within these catchments will be conveyed by piped drainage and overland flowpaths within developed areas. Camden Valley Way is to be upgraded with culverts proposed to convey existing flows for the 100 year ARI event with a 10% increase in rainfall intensity as an allowance for climate change. Drainage within the Precinct would be designed in accordance with the relevant Council guidelines. Appropriate surcharge arrangements at major crossings will be incorporated at the detailed design stage.

Retarding basins are required in order to maintain the existing flood behaviour downstream of the Precinct. It has been requested by Campbelltown City Council and the Department of Planning and Infrastructure that retarding basins be located on-line in order to reduce land take and associated future maintenance costs. An off-line basin is proposed on the tributary of Scalabrini Creek in the southwest of the Precinct. Basin locations, required storage and outlet configurations have been identified as part of this assessment. These recommendations have been incorporated into the Draft ILP.

Hydrologic modelling was undertaken for the 20 year ARI, 100 year ARI, 500 year ARI and PMF events with hydraulic modelling undertaken using a TUFLOW 1D/2D flood routing model. The flooding assessments indicate increases in 100 year ARI flood levels upstream of Denham Court Road ranging from 0.01m to 1m with the largest increases at proposed basin locations. In general, an increase in flood levels ranging from 0m to 0.2m is evident throughout the proposed floodway. These increases can be managed within the Precinct without adversely affecting landholdings within, or adjacent to, the Precinct.

A portion of the Study Area within the Liverpool LGA may not be developed at the same time as those areas upstream of Denham Court Road. Retarding Basin B3 has been designed in order to attenuate peak flows during the 100 year ARI event to below pre-development levels and achieve negligible changes to flooding downstream of Denham Court Road.

Under ultimate developed conditions, a constructed naturalised channel along with two on-line basins are proposed within the Liverpool LGA and extends from Denham Court Road to Camden Valley Way. The flooding assessment indicates general decreases in flood depth would be expected with a significant reduction in flood extents contained within the proposed channel in comparison to existing conditions.

Water quality treatment is proposed using bio-retention systems throughout the Precinct. Water quality modelling indicates that a minimum of 0.3% of catchment area is required for the filter area of bio-filtration. The sizing of bio-retention basins has been discussed in Section 5.4 and Appendix C. Proposed bio-retention systems will be off-line and will treat runoff prior to discharge into the watercourses. Some of these systems being co-located within on-line detention basins where feasible. The total area of water quality facilities required would be approximately 3% of the catchment area taking into consideration topography, embankments, inlet/outlet structures, design tolerances and access facilities.

This study has demonstrated that flood behaviour and water quality can be appropriately managed within the East Leppington Precinct and can be accommodated within the proposed ILP 12.6. Work items included in the Section 94 Plan that have been recommended by this WCM are summarised in **Table 6-1**.

Table 6-1: Work Items Summary

	CAMDEN	CAMPBELLTOWN	LIVERPOOL
	Area (m2)	Area (m2)	Area (m2)
Drainage			
Bio-retention basins			
BR1			5,554
BR10		4,218	
BR11		3,186	
BR12		4,355	
BR13			2,000
BR2			5,547
BR3			5,090
BR4		6,870	
BR5		5,198	
BR6		2,554	
BR7		2,572	
BR8		4,529	
BR9		2,427	
Bio-retention basins Total		**35,910**	**18,192**
Bio-retention basins (co-located)			
CLB1			2,690
CLB2			4,545
CLB3			2,915
CLB4		3,006	
CLB5		3,362	
CLB6		1,924	
CLB7		1,677	
CLB8	2,425		
Bio-retention basins (co-located) Total	**2,425**	**9,969**	**10,149**
Drainage Infrastructure			

D10		1,572	
D11		3,652	
D12		2,942	
D13		3,078	
D14		624	
D3		709	554
D4		2,779	
D5		4,531	
D6		6,037	
DE1		1,113	135
DE2		5,058	
DE3		2,600	
Drainage Infrastructure Total		**34,696**	**689**
Drainage lands			
RC	13,149	31,321	13,531
RC1			61,434
RC2		133,409	478
RC3	5,633		
Drainage lands Total	**18,782**	**164,730**	**75,443**
Offline Detention Basins			
R1	2,560		
Offline Detention Basins Total	**2,560**		
Online Detention Basins			
B1		4,637	
B3		6,959	
B4			1,088
B5			2,197
Online Detention Basins Total		**11,596**	**3,285**
Drainage Total	**23,767**	**256,901**	**107,758**
Grand Total	**23,767**	**256,901**	**107,758**

Offline Detention Basins (1 Campbelltown):

- *R1 (Campbelltown)*

Online Detention Basins (2 Campbelltown, 2 Liverpool)

- *B1 (Campbelltown)*
- *B3 (Campbelltown)*
- *B4 (Liverpool)*
- *B5 (Liverpool)*

7 REFERENCES

Cardno (2011) "Austral and Leppington North Precincts, Riparian Corridor and Flooding Assessment Main Report", prepared for NSW Department of Planning and Infrastructure.

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