Huntlee Project Trunk Stormwater and Flooding Assessment

Stage One Project Application

301017-00364

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HUNTLEE PROJECT TRUNK STORMWATER AND FLOODING ASSESSMENT
STAGE ONE PROJECT APPLICATION

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PROJECT 301017-00364 - HUNTLEE PROJECT TRUNK STORMWATER AND FLOODING ASSESSMENT

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1. INTRODUCTION

1.1 Background

Worley Parsons was engaged by Huntlee Pty Ltd to undertake a trunk stormwater and flooding assessment for two tributaries (*Tributary 1 and Tributary 2*) of Anvil Creek within the Huntlee Project area. The study area encompasses the Stage 1 release area (*approximately 360ha*) of the Huntlee development, as well as the northern extents of the existing townships of North Rothbury. The study was commissioned to provide an assessment of the stormwater treatment infrastructure required to meet stormwater management objectives as well as to determine flood extents for two tributaries of Anvil Creek within the study area. This study considers and incorporates where appropriate the outcomes of two studies previously conducted by Worley Parsons (*then trading as Patterson Britton and Partners*) titled "*Huntlee Project – Flood Study*" (*2007*) and "*Huntlee Project – Principles of Stormwater Management*" (*2007*). This study will be submitted in support of the Project Application for the Stage One Precinct of the Huntlee New Town Development.

1.2 Study Objectives

Worley Parsons were engaged to assess the stormwater and flooding constraints for the site. Accordingly, the following study objectives have been established:

- Assessment of the existing and developed state hydrology, including an assessment of the likely impacts of development on local and regional hydrology;
- Establishment of an appropriate detention policy to mitigate the effects of urban development on local and downstream flooding;
- Assessment of the flood behaviour over the site to determine the 100 year flood extents;
- Assessment of the potential flood impacts on adjoining and downstream properties as a result of the proposed development; and
- Collation the results from the engineering investigation and establish a stormwater and flooding management plan for the site.

It is noted that this assessment is specifically for the Stage One Project Application area. However, as the Stage One Project Application area makes up approximately 35% of the ultimate development (*excluding the large conservation area*), consideration has been given in this study to the ultimate Huntlee development footprint. This was undertaken to ensure sufficient allowances are made in the

Stage One Project Application for future stormwater and flooding infrastructure requirements. Only the stormwater and flooding assessment outcomes for the Stage One Project Application Area are formally reported in this study.

1.3 Site Description

The proposed Huntlee development site is located in the Hunter Valley approximately 500m south of Branxton. Refer to **Figure 1** for site locality. The proposed development master plan for the Huntlee Development is presented in **Figure 2.**

The natural drainage within the site is characterised by two ephemeral creeks flowing from south to north. Both creeks are unnamed tributaries of Anvil Creek, and are referred to as Tributary 1 (*eastern creek*) and Tributary 2 (*western creek*) in this report. Tributary 1 has a contributing catchment area of approximately 205ha which incorporates a portion of the Stage One Project Application area, including the Stage One Neighbourhood Centre. Tributary 2 has a contributing catchment area of approximately 315ha and incorporates parts of Stage One, including the Town Centre. Both Tributary 1 and 2 are currently in a degraded state with deep incising of the stream invert evident in certain sections as well as the presence of farm dams at regular intervals in both tributaries.

Current land use in both the Tributary 1 and Tributary 2 catchment areas consists predominantly of agricultural activities (*mainly cattle grazing*) and forested areas, with the decommissioned mine and the existing township of North Rothbury in the southern portion of the Tributary 2 catchment being the only significant exceptions.

1.4 Proposed Development

This Project Application relates to Stage One of the proposed Huntlee New Town Development which is located to the north and east of the existing township of North Rothbury. The proposed Stage One Project Application area is seeking approval for the following:

Subdivision for:

- Up to 1,900 residential lots, open space and community facilities in Village 1;
- The first stage of the mixed use Town Centre including up to 300 residential lots and employment lands (up to 68 hectares); and
- Up to 120 large lot residential lots.

Refer to **Figure 2** for the proposed development master plan.

The Roads and Traffic Authority (*RTA*) propose to construct a duel-carriage way freeway across the northern end of both catchments, however, this does not form part of the Huntlee development.

1.5 Previous Studies

A number of previous studies have been undertaken on the Huntlee site and two of these studies considered the current study area. All previous studies were undertaken at the concept plan level.

1.5.1 Huntlee Project – Trunk Stormwater and Flooding Assessment (WorleyParsons, 2008)

This study was commissioned to undertake a trunk stormwater and flooding assessment for two tributaries of Anvil Creek within the Huntlee Project area. The objectives of the study were to assess the existing and developed state hydrology and to mitigate the effects of the proposed urban development on local and downstream flooding. A stormwater management strategy was also developed to mitigate the potential water quality impacts of the proposed development on receiving waters. The results of the study were collated in order to establish a stormwater and flooding management plan for the site.

1.5.2 Huntlee Project – Flood Study (Patterson Britton and Partners, 2007)

This study was commissioned to undertake a flood assessment of Black Creek, Anvil Creek and tributaries of Anvil Creek within the Huntlee site. The objective of the study was to establish flood extents for the 100 year flood event and in doing so provide flooding constraints for the refinement of the development layout. The hydrologic analysis within the Anvil Creek Catchment was undertaken using Rational Method Calculations and as such is considered approximate only. Additionally, flood analysis was undertaken for the existing conditions only.

The current study aims to expand the findings of the "Huntlee Project – Flood Study" to incorporate hydrologic modelling using the *RAFTS* model and account for potential changes to the flow regime under the developed condition.

1.5.3 Huntlee Project – Principles of Stormwater Management (Patterson Britton and Partners, 2007)

This study was commissioned to provide stormwater treatment infrastructure sizings based on generally agreed methods of approximation. The study recommended stormwater treatment infrastructure based on landuses. Since the completion of this study, there have been a number of changes to the development parameters, in particular a move towards higher density development which tends to exclude the opportunity for conveyance controls such as swales. Additionally, catchment definition was undertaken for the existing condition only.

This study aims to refine the findings of the "Huntlee Project – Principles of Stormwater Management" to provide a preliminary sizing of stormwater control measures on a catchment by catchment basis using hydrologic and water quality models.

1.6 Relevant legislation and Guidelines

Australian Rainfall and Runoff

Australian Rainfall and Runoff (*AR&R*) is a document published in 1987 by the Institution of Engineers, Australia (IEAust). This document has been prepared to provide designers with the best available information on design flood estimation and is widely accepted as a design guideline for all flood and stormwater related design in Australia.

Australian Runoff Quality

Australian Runoff Quality (ARQ) is a document published in 2005 by IEAust which provides design guidelines for all aspect of water sensitive urban design (*WSUD*), including preventative measures, source controls, conveyance controls and end of line controls. Additionally, it provides guidance for water quality modelling as well as stormwater harvesting and re-use.

Floodplain Development Manual

The Floodplain Development Manual is a document published by the New South Wales State Government in 2005. The document details Flood Prone Land Policy which has the primary objective of reducing the impact of flooding and flood liability on individual owners and occupiers of flood prone property, and to reduce private and public losses resulting from floods. At the same time, the policy recognises the benefits from occupation and development of flood prone land.

Water Management Act 2000

The *Water Management Act 2000 (WMA)* is administered by the *Department of Water and Energy* (*DWE*). The act provides guidelines regarding development constraints and riparian setback for any controlled activity occurring within 40 meters from a river, lake or estuary. As the Huntlee project is assessed under the Part 3A permit, the development proposal is not legally bound to meet the requirements of the WMA. However, the WMA is considered best practise and has been applied in principle to the development proposal.

Council DCPs

The Huntlee site falls under the jurisdiction of both Singleton City Council and Cessnock City Council LGAs. While Council DCPs are primarily a detailed design guideline, the underlying principles have been adopted to ensure compliance can be achieved at the detailed design stage.

1.7 Available data

Both map and survey data was used to define catchments within the study area. The following information was used as part of this investigation:

- 10m contours (*provided by Department of Lands under license*);
- 2m contours (*provided by Department of Lands under license*);
- 1 m contours *(provided by Monteath & Powys);*
- Recent (2010) Aerial Laser Survey (ALS) of the Huntlee site (*giving accuracy to approximately ±150mm*); and
- A recent aerial photograph of the site.

It is noted that flood extents and stormwater requirements determined in this investigation are based on contour information derived from ALS survey of the area. A three dimensional land surface model was created from this contour information. It is noted that the contours may not provide an absolute resolution of the topography within the creekline corridors with features such as the creek channel not clearly defined in all areas. Hence, there is potential for minor inaccuracies in both the predicted flood level estimations and volumetric calculations of detention storage. However, it is considered that any inaccuracies are likely to be conservative in the case of flood modelling.

2. EXISTING CATCHMENT FEATURES

The natural drainage within the site is characterised by two ephemeral creeks flowing from south to north. Both creeks are unnamed tributaries of Anvil Creek, and are referred to as Tributary 1 (*eastern creek*) and Tributary 2 (*western creek*) in this report.

The following sections provide a brief description of both catchments and outline the proposed development land uses.

2.1 Tributary 1

Tributary 1 has a contributing catchment area of approximately 205ha which incorporates a significant portion of the Stage One Project Application area. Currently the proposed development area consists of grazing land, while the upper and lower reaches of the catchment (*external to the Huntlee site*) are predominantly forested.

The catchment has moderate grades ranging from 3% to 7%. Channel grades along Tributary 1 range from 1.0% to 1.5 %, with the upper reaches generally being steeper. A farm dam exists approximately half way down the reach. The channel is incised is some areas, which is most likely the result of cattle grazing and land clearing.

The Northern Railway embankment forms a downstream boundary. An existing culvert (*which has been identified as being undersized*) is the only hydraulic connection under the railway.

Refer to **Figure 3** for the catchment extents and topographic survey.

2.2 Tributary 2

Tributary 2 has a contributing catchment area of approximately 315ha and incorporates parts of the Stage One Project Application areas.

The existing land use in the catchment consists predominantly of agricultural land and forested areas, with the decommissioned mine and the existing township of North Rothbury in the southern portion of the catchment being the only significant exceptions.

The catchment features considerable variation in terrain, with grades ranging from 2% to 10%. Channel grades along Tributary 2 range from 0.4% to 2.5 %, with the upper reaches generally being steeper. Numerous farm dams exist throughout the catchment. The channel is incised is some areas, which is most likely the result of cattle grazing and land clearing.

The Northern Railway embankment forms the downstream catchment boundary. Two existing culverts (*which have been identified as being undersized*) are the only hydraulic connection under the railway.

Refer to **Figure 3** for the adopted catchment extents.

2.3 Hunter Expressway

Previous stormwater investigations for the site had referred to the Hunter Expressway as a future infrastructure project, however construction is now well underway and detailed information on the design of the expressway downstream of the Huntlee Project has been provided. The Contractor (Abigroup) has provided detailed plans and calculations for the culverts under the Expressway proposed for Tributaries 1 and 2, as well as proposed culvert upgrades for the existing rail embankment. Given the progress on the Expressway construction and expected completion in 2013, these previously proposed works have been considered as "existing" for the purposes of this stormwater investigation. Therefore the reduced storage volume upstream of the rail embankment, the expressway embankment/culverts and upgrade to the rail embankment culverts have been included in the existing scenario stormwater modelling and carried through to the developed scenario.

2.4 Baseline Water Quality Conditions

Typically, historical and recent local water quality data is used to assist in the assessment of the conditions of existing aquatic ecosystems. Whilst the water quality monitoring program for the site has not yet commenced, a qualitative assessment of the condition of the watercourses within and surrounding the proposed development is discussed below.

The ANZECC Guidelines (2000) distinguish three levels of aquatic ecosystem conditions;

(i) High Conservation/Ecological Value systems, effectively unmodified or other highly valued ecosystems.

(ii) Slightly to moderately disturbed systems, these have been impacted upon by human activities however integrity is largely maintained.

(iii) Highly disturbed systems, degraded ecosystems of low ecological value.

The Huntlee Stage 1 site has been the subject of long term farming and grazing, with many farm dams being constructed and also large areas of vegetation cleared along watercourses in some instances. This is predominately in the lower parts of the site and could be considered highly disturbed. In the upper parts of the site, some portions of the existing watercourses would be considered slightly to moderately disturbed, as generally riparian vegetation is intact, albeit farming activities and clearing have occurred in the nearby area.

Construction of the Hunter Expressway has also recently been in progress just downstream of the site, with a large embankment and culverts replacing the existing water courses. These works in conjunction with the existing railway embankment immediately downstream, add to the highly disturbed nature of the aquatic ecosystems in the lower parts of the site. Further downstream of the site is additional urban development as well as a golf course which directly adjoins one of the main site tributaries.

For the purpose of initially assigning target water quality values, the ecosystem has been defined as a "**slightly to moderately disturbed system**". This initial assessment would need to be reviewed during the baseline water quality monitoring program.

3. POTENTIAL SURFACE WATER IMPACTS

Collectively, over a number of development stages, it is proposed to develop over 400 ha of land between the Tributary 1 and Tributary 2 catchments. Development of this scale would introduce significant impervious area as well as create a more hydraulically efficient trunk drainage system.

Historically, urban development has had detrimental impacts on waterways through a number of mechanisms. The following extract from *Australian Runoff Quality (ARQ, IEAust, 2006)* summarises the key adverse impacts of urbanisation on waterways:

- Increased rate and volume of runoff;
- Increased frequency of high velocity flows;
- Increased rates of erosion, sedimentation and channelisation;
- Reduction and loss of riparian zones;
- Reduction and loss of in-stream habitat;
- Decreased water quality;
- Contamination of sediments;
- Introduction of barriers to the dispersal of biota and the loss of continuity between up-stream and downstream communities; and
- Reduced diversity of indigenous flora and fauna and the introduction of pests and weeds.

With respect to specific impacts on water quality, **Table 3-1** outlines the stormwater pollutants that are typically associated with urban land-uses and outlines the key sources of each pollutant.

Table 3-1 – Typical Urban Stormwater Pollutants and Key Sources

Urban Stormwater Pollutant	Key Sources
Gross Pollutants (litter)	General littering and illegal dumping
Total Suspended Solids (TSS)	Erosion and wash off from impervious areas
Total Nutrients, including Total Phosphorus (TP) and Total	Fertiliser application and atmospheric deposition

Nitrogen (TN)	
Faecal Coliforms	Pet droppings and sewer overflows
Oils, greases and petroleum hydrocarbons	Spills and leaks from motor vehicles
Metals	Wear of vehicle tyres and brake pads, corrosion of metal objects and leaks and spills of various chemicals.

The above items are typical of most urban development projects, more specific impacts of the proposed Huntlee development includes the removal of several minor branches of existing watercourses as well as the realignment of an existing third order stream. The removal of the minor streams allows the development footprint to be more accessible for pedestrians and vehicles and avoid fragmentation of the development. The proposed development aims to focus efforts on improving and maintaining the key watercourses through the site. Typically the minor streams proposed to be removed do not contain significant vegetation due to previous clearing from farming and in several cases contain farm dams.

In addition to the above consideration of impacts associated with the proposed urban development, construction works can also have significant impacts on surface water quality due to removal of vegetation and exposure of large areas of bare soil which can be eroded and transported downstream if not properly managed.

4. SURFACE WATER MANAGEMENT OBJECTIVES

The key objective of the Stormwater management plan is to minimise the impacts of the proposed development on watercourses within the subject site and downstream. In order to quantify this objective, specific targets for the management plan have been outlined below.

- Establishment of an appropriate detention policy to minimise the effects of urban development on local and downstream flooding, specifically no net increase in peak flows during storm events up to the 1:100 year ARI;
- Suspended Solids (TSS) 85% retention of the developed average annual load;
- Total Phosphorous (TP) 45% retention of the developed average annual load; and
- Total Nitrogen (TN) 45% retention of the developed average annual load.

The above would be adopted as design objectives when sizing stormwater controls. **Sections 5.3 to 5.6** discuss typical stormwater controls and provide indicative sizes for the various land-uses proposed as part of the Huntlee Project. It is noted that stormwater controls designed to achieve the above water quality treatment targets will also remove a substantial portion of the other stormwater pollutants listed in **Table 3-1** through bio-chemical processes.

In addition to the water quality control targets listed above, a range of preventative measures and source controls would also be integrated into the development design and ongoing management procedures.

It is noted that this assessment is specifically for the Stage One Project Application area. However, as the Stage One Project Application area makes up a significant portion of the ultimate development in the Tributary 2 catchment, consideration has been given to the ultimate development footprint. This was undertaken to ensure sufficient allowances are made in the Stage One Project Application for future stormwater and flooding infrastructure requirements. Only the results for the Stage One Project Application Area are formally reported in this study.

The following Sections outline the Hydrological and Water Quality Assessments undertaken to meet the objectives outlined above.

5. HYDROLOGICAL ASSESSMENT

A hydrologic assessment of the Tributary 1 and 2 catchments was undertaken to establish the existing state and developed state hydrology. This section outlines the methodologies implemented in the development of the hydrologic models and presents model results for the existing and developed state scenarios. Once the impacts of the development were determined, mitigation measures, such as detention basins, have been determined using the hydrologic model.

5.1 Discussion

A hydrologic assessment was undertaken to determine the likely impact of the proposed Huntlee development on the hydrology of the Tributary 1 and Tributary 2 catchments, which are both tributaries to Anvil Creek. Currently, the proposed development area consists predominantly of agriculture land and forested areas, apart from decommissioned mine and the existing township of North Rothbury in the southern portion of the Tributary 2 catchment.

With reference to **Figure 2**, the proposed Huntlee development will ultimately encompass a significant portion of the Tributary 1 catchment and nearly the entire Tributary 2 catchment. Additionally, the RTA proposes to construct a dual carriage-way freeway to the north of the proposed Huntlee site. The initial Stage One Project Application area encompasses a significant portion of the ultimate development area in the Tributary 2 catchment. Therefore, it is essential that the hydrologic assessment for the Stage One Project Application considers both the initial Stage One development as well as the ultimate catchment development.

This hydrologic assessment quantitatively assesses the impact of the proposed development on catchment hydrology through the development of hydrologic models of both the existing state and developed state catchments. Once the impact of the development is determined, mitigation measures, such as detention basins, have been determined using the hydrologic model.

5.2 RAFTS Hydrologic Model

The Runoff Analysis and Flow Training Simulation (*RAFTS*) software package was employed to quantify flood discharges from the Stage One and Town Centre catchments. *RAFTS* is a deterministic runoff routing model that simulates catchment runoff processes. It is recognised in *'Australian Rainfall and Runoff* (*AR&R 1987*), as one of the available tools for use in flood routing within Australian catchments.

RAFTS was chosen for this investigation because it has the following attributes:

- it can accommodate variations in catchment characteristics;
- it can accommodate stormwater controls such as detention basins; and
- it can be used to estimate discharge hydrographs at any location within a catchment.

5.2.1 Development of Hydrologic Model

A *RAFTS* hydrologic model was developed for the Stage One catchments, with all contributing areas upstream of the Northern Railway embankment included in the model. The hydrologic results from the *RAFTS* model were used for the following aspects of this study:

- To predict peak flows and storm hydrographs for a range of storm durations and average recurrence intervals (*ARI*) at locations within the study area, these results were used for hydraulic modelling used to assess the flood behaviour over the site.
- To assess the impact of the proposed development on the hydrologic regime at locations both within and external to the study area.
- To develop strategies to mitigate any adverse impacts to the hydrologic regime at locations within and external to the study area as a result of the proposed development.

To allow for a comprehensive impact assessment, a pre and post-development *RAFTS* model was established. Details of sub-catchment delineation, adopted model parameters and key model results are present in the following sections.

5.2.2 Adopted Model Structure

The Tributary 1 and 2 catchments were divided into sub-catchments differentiated on the basis of the alignment of the proposed development, as well as the homogeneity of land-use, vegetation and topography. Additionally, catchment boundaries were aligned so that the downstream points of sub-catchments coincided with the likely location of detention basins and inflow points for the proposed hydraulic model. To aid comparison of results and calculation of detention requirements, similar catchment configurations were adopted for both the existing and developed state models. However, in some cases the developed state catchment boundaries were slightly increased to account for a likely re-alignment of existing catchment boundaries resulting from road grading.

Figure 3 presents the adopted developed state catchment configuration. Adopted sub-catchment areas for both the existing state and developed state models are tabulated in **Appendix A.**

5.2.3 Model Parameters

The following methodologies were adopted in determining rainfall runoff parameters:

- **Impervious percentages** were applied based on the proposed land use categories as outlined below;
    - **Residential Areas**: incorporating residential roads and lots, which typically range between 450m^2 and 800m^2. It is assumed all residential areas are 65% impervious, with the following breakdown of land uses:
        - 45% roof area;
        - 20% road and carpark area; and
        - 35% garden, verge and lawn area.
    - **Town Centre/Mixed Use**: incorporating commercial areas, open space, schools and higher density residential dwellings. It is assumed all town centre/mixed use areas are 75% impervious, with the following breakdown of land uses:
        - 50% roof area;
        - 25% road and carpark area; and
        - 25% garden, verge and lawn area.
    - **Employment/Commercial**: incorporating larger scale commercial/employment development. It is expected that there would be significant areas of carparking within these catchments. It is assumed all employment/commercial areas are 85% impervious, with the following breakdown of land uses:
        - 55% roof area;
        - 30% road and carpark area; and
        - 15% garden, verge and lawn area.
- **Catchment roughness** values were determined based on vegetation cover (*determined by aerial photography*) for existing state models and proposed development surface for catchments within the proposed development extents.
- **Catchment Slopes** were digitised from survey information

- **Rainfall runoff parameters.** There is no known definitive information regarding initial and continuing loss rates in the Huntlee area. Hence, adopted rainfall loss rates were based on the recommended ranges outlined in the *RAFTS User Manual* and documented in *AR&R 1987*.

Table 5-1 presents the range of hydrologic parameters adopted for various sub-catchment land uses. A detailed list of all sub catchment parameters adopted is attached in **Appendix A.**

Table 5-1 – Existing State *RAFTS* parameters

Existing Urban Catchments		
	Impervious Surfaces	**Pervious Surfaces**
Initial Loss (mm)	1.5	10
Continuing Loss rate (mm/hr)	0	2.5
Catchment Roughness	0.025	0.035
Undeveloped Catchments		
	Rural Catchments	**Forested Catchments**
Initial Loss (mm)	15	15
Continuing Loss rate (mm/hr)	3.6	3.6
Catchment Roughness	0.05 - 0.08	0.08 - 0.1
Proposed Development Catchments		
	Impervious Surfaces	**Pervious Surfaces - Urban Catchment**
Initial Loss (mm)	1.5	10
Continuing Loss rate (mm/hr)	0	2.5
Catchment Roughness	0.015	0.025

Sub-catchment lag times were estimated based on average channel gradient and longitudinal channel distance. All sub-catchment lag times are attached in **Appendix A.**

5.3 Results

The *RAFTS* model was used to predict existing and developed state runoff hydrographs at all *RAFTS* nodes in the modelled catchment. For presentation purposes, the model results are reported at two select locations in both the Tributary 1 and Tributary 2 catchments. Each selected location is considered a parity point, whereby both the existing state and developed state models have a similar upstream contributing catchment area. Hence, comparison of model results at the parity locations allows for the likely impact of development on the catchment hydrology to be quantitatively assessed. Refer to **Figure 3** for parity point locations.

As discussed in **Section 5.1**, the hydrologic assessment considered the catchment holistically, with the key model results being compared at the downstream parity location. Hence, peak flow predictions at the downstream parity location (*rail embankment, at P1-6 and P2-6*) and parity point at the location of the proposed Basin 4 (*P2-5*) for both the existing state and developed state models are presented in **Table 5-2** below. Model results from all rainfall events assessed at these two parity locations are tabulated in **Appendix B**.

Table 5-2 – Model Results

ARI	**Tributary 1** *(RAFTS node P1-6)*		**Tributary 2** *(RAFTS node P2-5)*		**Tributary 2** *(RAFTS node P2-6)*	
	Existing State	Developed State *(no controls)*	Existing State	Developed State *(no controls)*	Existing State	Developed State *(no controls)*
	Peak flow (m^3/s) / *Critical Storm Duration*					
2 year	6.7	11.6	9.7	20.5	10.4	19
	540 min	*120 min*	*540 min*	*120 min*	*540 min*	*120 min*
5 year	9.9	16.3	14.1	27.6	14.92	23.7
	540 min	*120 min*	*540 min*	*90 min*	*540 min*	*120 min*
20 year	13.9	22.5	20.1	37.4	20.53	27.67
	540 min	*120 min*	*360 min*	*90 min*	*540 min*	*120 min*
100 year	20	29.6	30.2	48.8	25.55	31.5
	120 min	*120 min*	*120 min*	*90 min*	*120 min*	*120 min*

With reference to **Table 5-2**, modelling predicts that that the proposed development, with no detention controls, is likely to increase peak flows in both the Tributary 1 and Tributary 2 catchments over a range of *ARI* rainfall events. Hence, mitigation measures are required to minimise the impact of the development on the local and regional hydrology. These are discussed in **Section 5.4**

5.4 Mitigation Options

It is proposed to construct online detention basins to alleviate the increase in peak flow resulting from the proposed development. The following detention basins are proposed:

- **Tributary 1 Catchment**: It is proposed to construct two online detention basins along Tributary 1 with the following active storage volumes:
    - Basin 1 – 5,600 m^3; and
    - Basin 2 – 20,000 m^3.
- **Tributary 2 Catchment:** It is proposed to construct two on-line basins in the upper catchment with the following active storage volumes:
    - Basin 3 – 10,000 m^3;
    - Basin 4 – 71000 m^3.

The proposed basins in the Tributary 2 catchment have been sized considering the ultimate proposed development (*i.e. assuming future development area would be constructed*). Hence, the proposed basins are oversized for the initial Stage One Project Application.

Refer to **Figure 5** for proposed detention basin locations.

Additional stormwater controls are proposed for water quality control purposes. These controls would provide up to 20mm of retention storage per unit of impervious area within the proposed development. This equates to approximately 130m^3 of retention storage per hectare of residential (*65% impervious*) catchment. Refer to **Section 6** for more details. If empty at the start of the storm, the 20 mm of retention storage would provide a reduction in runoff volume as well as reduced peak flows, with significant reductions likely during the lower return period events such as the 2 year ARI storm. However, as the retention storage would take a number of days to empty, it has been conservatively assumed that they would be full at the beginning of a design storm, and have not been accounted for in the modelling accordingly.

The proposed detention basins were incorporated into the *RAFTS* model to account for the impact of the detention basins on catchment hydrology. Predicted peak flows at the downstream parity location (*rail embankment*) are presented for the existing state, developed state (*with no controls*) and developed state (*with controls*) in **Table 5-3** and **Table 5-4**.

Table 5-3 – Developed State – With detention basins (Tributary 1)

ARI	**Tributary 1** *(RAFTS node P1-6)*		
	Existing State	Developed State *(no controls)*	Developed State *(with controls)*
	Peak flow (m^3/s) / *Critical Storm Duration*		
2 year	6.7	11.6	6.6
	540 min	*120 min*	*540 min*
5 year	9.9	16.3	9.9
	540 min	*120 min*	*540 min*
20 year	13.9	22.5	13.9
	540 min	*120 min*	*540 min*
100 year	20.0	29.6	18.8
	120 min	*120 min*	*120 min*

With reference to **Table 5-3**, modelling indicates that the attenuation of flow provided by the proposed detention basins within *Tributary 1* would reduce peak flows at the Northern Railway embankment to at least existing state levels for all ARI rainfall events considered. Thereby complying with the water quantity objectives defined in **Section 4**.

The hydrologic results for *Tributary 2* are displayed below in **Table 5-4**, which highlights the impact with the proposed on-line detention basins in place.

Table 5-4 – Developed State – With detention basins (Tributary 2)

ARI	**Tributary 2** *(RAFTS node P2-5)*			**Tributary 2** *(RAFTS node P2-6)*		
	Existing State	Developed State *(no controls)*	Developed State *(controls)*	Existing State	Developed State *(no controls)*	Developed State *(controls)*
	Peak flow (m^3/s) / *Critical Storm Duration*					
2 year	9.7	20.5	10.4	10.4	19	11.0
	540 min	*120 min*	*120 min*	*540 min*	*120 min*	*120 min*
5 year	14.1	27.6	14.65	14.9	23.7	15.2
	540 min	*90 min*	*120 min*	*540 min*	*120 min*	*120 min*
20 year	20.1	37.4	19.19	20.5	27.7	19.7
	360 min	*90 min*	*120 min*	*540 min*	*120 min*	*120 min*
100 year	30.2	48.8	22.89	25.6	31.5	23.1
	120 min	*90 min*	*120 min*	*120 min*	*120 min*	*180 min*

Results show that the post development peak discharges can be reduced down to existing state peak discharges with the proposed on-line detention basins in place for the higher ARI events. However, peak discharges in the 2 and 5 year ARI storm event at P2-5 and P2-6 are slightly greater than those in the existing state. This is considered acceptable, given the fact that the modelling ignores the effect of proposed offline water quality control (bio-retention) basins that will effectively control peak discharges up to the 5 year ARI event. Furthermore, the design of outlet controls from the on-line basins within Tributary 2 can be further refined accordingly (*at a more detailed stage*) to further reduce peak discharges at the lower ARI events, as required.

5.4.1 Sensitivity Analysis for Climate Change Scenario

A sensitivity analysis was undertaken to model the impacts of climate change on peak discharges from the development site. To do this, the average rainfall intensities in the hydrologic model were increased by 10% and 30% for the critical storm duration for the 100 year ARI event. Results for the more conservative 30% rainfall intensity increase scenario suggest that the 100 year ARI peak discharge at P2-5 would increase from 30.2m^3/s (*Existing case*) to 40.7m^3/s (*Developed with controls + 30% increase in average rainfall intensity*). The 100 year ARI peak discharge at P2-6 would increase from 25.6m^3/s (*Existing case*) to 27.1m^3/s /s (*Developed with controls + 30% increase in average rainfall intensity*). However, an increase in average rainfall intensities of 10% is typically adopted for hydrologic models in the Hunter Valley. The case for 10% increase in rainfall intensity results in no significant change to peak discharge levels compared to existing conditions at the parity locations.

5.5 Impact on Downstream Flooding

Modelling undertaken as part of this hydrologic investigation has determined that the proposed detention basins are likely to provide sufficient attenuation of flow such that there is no net increase in peak flows or flood levels downstream of the site as a result of the development (*with exception of the 2 and 5 year ARI events in Tributary 2).*

It is noted that the proposed detention basins and water quality controls would not fully alleviate the increase in volume of runoff resulting from the introduction of impervious surfaces in the catchment during major rainfall events. The impact of the increased runoff volume on downstream flooding has been assessed through the modelling of the area downstream of basin 4 for the more critical Tributary 2 catchment. This incorporated culverts, currently under construction, which (Refer Fig, 4b,4c) convey stormwater downstream of the proposed Hunter Expressway (3 x 2.4m wide x 1.5m high RCBC) as well as upgraded culverts which will compliment existing culverts beneath the Northern Railway Embankment (additional 900mm diam. Pipe) . It is noted that headwater levels gained at the northern railway embankment were used as tailwater levels for the proposed Hunter Expressway culvert system. These results are outlined in **Table 5-5** below.

Table 5-5 – RAFTS results for level pool routing upstream of railway culverts

ARI	P2-6 (*Critical storm*)	*RAFTS node P2-6*		
		Existing	Developed (*no controls*)	Developed (*with controls*)
2 year	Q_{in}	10.61	22.2	11.22
	Q_{out}	10.4	19	11.02
	Level (mAHD)	32	32.97	32.05
	Volume (m^3)	2254	7341	2440
5 year	Q_{in}	15.3	29.6	15.8
	Q_{out}	14.9	23.7	15.2
	Level (mAHD)	32.5	33.7	32.52
	Volume (m^3)	4218	15528	4386
20 year	Q_{in}	21.6	40.0	20.79
	Q_{out}	20.53	27.7	19.74
	Level (mAHD)	33.2	34.6	33.06
	Volume (m^3)	8757	38388	8028
100 year	Q_{in}	32.8	54	25
	Q_{out}	25.6	31.5	23.1
	Level (mAHD)	34.14	35.6	33.6
	Volume (m^3)	22313	57026	13,923

The above table illustrates that the development (*with controls in place*) will have no significant impact on the downstream flood levels or peak discharge rates within the existing flood storage area immediately upstream of the railway embankment within Tributary 2.

6. STORMWATER QUALITY ASSESSMENT

This section outlines the proposed water quality control requirements for the Stage 1 Project Area.

6.1 Stormwater Quality Control Options

The following stormwater controls have been identified as suitable for implementation within the Stage 1 Project Area:

Preventative Measures – applied at the subdivision scale include:-

- **Minimising areas of Impervious Surfaces** - by reduced road widths and increased landscaping around dwellings.
- **Native Landscaping** – Native species are generally drought tolerant and do not require frequent fertilisation. As such, native landscaping should be encouraged in both public open space and on individual allotments. This will assist in minimising nutrient concentrations in storm water runoff.
- **Public Education** – information can be provided to residents to inform them of stormwater management issues and provide recommendations on how they can minimise their impact.

Source Controls - applied at the lot level include: -

- **Rainwater Tanks** - can be installed at the lot scale to capture roof water runoff for reuse for non-potable uses within the allotment. Rainwater tanks reduce the runoff volume from the allotment, reducing the size of downstream controls. Rainwater tanks will be installed at all dwellings that are not serviced by the proposed reticulated recycled waste-water network.
- **Permeable Pavers** - can be implemented into driveways, parking bays, car parks and on roads with low traffic loads. The pavers allow stormwater to infiltrate into the sub-base, where stormwater retention and treatment is provided. As permeable pavers can be integrated into the streetscape, they generally have no net land take and are therefore well suited to high density areas such as the proposed Town Centre.
- **Landscaping Features** – such as contour banks and soak-a-ways are suitable for rural residential lots where runoff from impervious surface can be directed onto grassed areas and slowly infiltrated.

Conveyance Controls - applied at the street level and include:

- **Bio-Filtration Swales** – can be integrated into the urban landscape in a longitudinal swale or a small basin area (*commonly referred to as rain gardens*). The bio-filtration areas would consist of vegetated areas with enhanced filtration media which would typically be 500-600mm deep. Filtered stormwater would be collected in an underlying subsurface drainage system and directed into roadside drainage. The bio-retention areas could be designed to provide 2 year retention storage, negating the need for downstream basins in some catchment areas; and

- **Road Side Swales** – can replace traditional kerb and guttering in residential and rural residential areas. Swales provide water quality and water quantity benefits as well as providing landscaping opportunities.

End of Line Controls - applied at the end of the stormwater system and include:-

- **Gross Pollutant Traps (*GPTs*)** – can be installed at the end of piped drainage systems. GPTs primarily remove gross pollutants (*i.e litter*) and coarse sediments and provide pre-treatment for other downstream controls.

- **Bio-Filtration Basins** – incorporate a similar concept to the bio-filtration swales, but are constructed on a larger scale at the end of a stormwater drainage line.

- **Constructed Wetlands** – can be implemented to treat stormwater prior to discharge into receiving waters. Wetlands would be designed to incorporate sediment fore bays, deep water zones and ephemeral (*'wet and dry'*) macrophyte beds to achieve maximum pollutant removal efficiencies. Extended and active detention can also be provided above the permanent pool to meet water quantity management objectives.

Stage 1 of the Huntlee development would incorporate varied land-uses including low density rural residential allotments, medium density residential allotments and higher density residential and commercial areas. **Table 6-2** discusses potential stormwater control strategies for the various land-uses.

Table 6-2 – Potential Stormwater Management Solutions for Various Land-Uses.

Land-use	Potential Stormwater Management Solutions
Residential	Impervious fractions in residential areas generally range from 50 to 70%, depending on the development densities. Given the limited space, opportunities for on-lot controls are generally limited to rainwater tanks and permeable pavers. However, landscaping features, such as contour banks and designated soak-a-way areas can also be incorporated with careful consideration. Conveyance controls such as bio-retention swales and basins (*also referred to as rain-gardens*) can be integrated in the streetscape and can generally achieve the water quality objectives. 'End of pipe' detention facilities are generally required for catchments where 100 year detention is required. If a traditional kerb and gutter approach is adopted, stormwater control would generally be achieved at the 'end of pipe' where constructed wetlands or bio-retention basins can be employed. Typically, GPTs are integrated in 'end of pipe' systems to provide removal of coarse sediments and litter. Implementation of upstream source and conveyance controls can significantly reduce the size of 'end of pipe' stormwater controls.
Rural Residential	Rural residential and large lot residential areas lend themselves to complete on-lot management of stormwater. This can be achieved through rainwater tanks for roofed areas and by allowing runoff from other impervious surfaces (*and any overflow from rainwater tanks*) to infiltrate in vegetated areas. Road ways can be effectively treated using roadside swales and bio-retention areas.
Town Centre	The Town Centre would incorporate a mix of high density urban allotment and commercial and light industrial land-uses. Impervious fractions in the Town Centre area are likely to be in the range of 80 to 90%. On-lot controls such as permeable pavers in parking areas and bio-retention areas integrated into the streetscape landscaping, can often achieve water quality management requirements. If 100 year detention is required, detention basins (*incorporating constructed wetlands*) would most likely be the most efficient treatment option. However, provision of upstream source controls can reduce the land take of the basins.
Open Space, Conservation Areas	Open space and conservation areas typically do not need any stormwater management controls. Initially, some rehabilitation may be required in areas which are degraded.

6.2 Proposed Stormwater Quality Control Strategy

The following 'end of pipe' water quality control strategies are proposed for the Stage 1 project application area:

- **Gross Pollutant Traps**: Proprietary Gross Pollutant Traps (*GPTs*) are considered primary water quality treatment devices and would be installed upstream of secondary treatment devices (such as water quality control ponds and bio-retention basins) to remove litter and coarse to medium sized sediments (*and attached metals, oil/grease and other contaminants*).

- **Bio-Retention Basins**: It is proposed to construct bio-retention basins at the edges of trunk drainage corridors. Bio-retention basins would consist of vegetated areas with enhanced filtration media which would typically be 500-600mm deep. Filtered stormwater would be collected in an underlying subsurface drainage system and directed into the main channel. Required retention storage would be provided above the filter media through ponding of water up to 500 mm deep. A high flow by-pass would be provided to prevent scouring within the basins.

- **Water Quality Control Ponds**: It is proposed to construct a large permanent pond as part of the Stage 1 neighbourhood centre and the Town Centre area. These ponds would be offline to the nearby drainage channel however, the Town Centre pond would be capable of receiving a portion of low flows from the upstream catchment.

Figure 5 indicates the proposed areas for the above end of pipe controls. **Section 6.4** establishes the required dimensions of the above water quality controls based on the results of the MUSIC model.

6.3 Stormwater Quality Model

In order to assess the effectiveness of the stormwater control strategy against the defined water quality objectives, it is necessary to model the proposed developed catchment and the water quality control structures. Computer software called MUSIC has been adopted as the tool to undertake this modelling. An outline of MUSIC modelling is included below.

6.3.1 MUSIC Modelling

MUSIC is a continual-run conceptual water quality assessment model developed by the Cooperative Research Centre for Catchment Hydrology (*CRCCH*). MUSIC can be used to estimate the long-term annual average stormwater volume generated by a catchment as well as the expected pollutant loads. MUSIC is able to conceptually simulate the performance of a group of stormwater treatment measures (*treatment train*) to assess whether a proposed water quality strategy is able to meet specified water quality objectives.

To undertake the water quality assessment, a MUSIC model was established for the Huntlee Stage 1 project area. The model was used to estimate the pollutant load generated from the development and estimate the indicative size of water quality controls required to meet the water quality targets defined in **Section 5.2.**

MODEL PARAMETERS

In order to establish a MUSIC model, rainfall and evaporation records in the vicinity of the Huntlee site were sought.

Rainfall

In order to develop a model that could comprehensively assess the performance of the proposed SWMP, the use of 6 minute pluviograph data was considered necessary. Rainfall records obtained from BoM Station 061174, located at Millfield, were used for MUSIC water quality simulations. The rainfall records at BoM Station 061174 extend from 1959 to 1980 and were reviewed to determine that the average annual rainfall depth is approximately 818mm. Observed rainfall between 1969 and 1973 was used for all MUSIC water quality simulations. This period was selected as it represents 5 consecutive years of approximate average rainfall.

Evaporation

Monthly areal potential evapotranspiration (*PET*) rates for the site was estimated from PET data provided by the Climate Atlas of Australia (BoM). The monthly average PET adopted for the MUSIC model are shown in **Table 5-3**.

Table 5-3 – Average Evaporation and Potential Evapotranspiration at the Huntlee Site

Month	Average Monthly Evaporation^ *(mm/month)*	Areal Potential Evapotranspiration *(mm/month)*
January	180	170
February	175	140
March	125	130
April	100	90
May	90	65

Month	Average Monthly Evaporation^ *(mm/month)*	Areal Potential Evapotranspiration *(mm/month)*
June	80	60
July	75	50
August	90	70
September	120	90
October	140	120
November	180	150
December	200	165

^ Evaporation from a Class evaporation pan.

Catchment Parameters

MUSIC simulates the generation, mobilisation and removal of the following pollutants:-

- Total Suspended Solids (*TSS*)
- Total Phosphorus (*TP*)
- Total Nitrogen (*TN*)

The pollutant loadings for each catchment are proportional to the land use and the impervious area fraction. The following three general urban surfaces were adopted for the MUSIC modelling:-

- Urban Roads
- Urban Roofs
- Other Urban pervious areas

The event mean concentrations (*EMCs*) for each of these land uses were derived from '*Urban Stormwater Quality: A Statistical Overview*' (*Duncan, February 1999*) and '*Australian Runoff Quality*' (*Engineers Australia, 2006*). Adopted EMCs for each land use are detailed in **Appendix D.**

6.4 MUSIC Model Results

An assessment was undertaken in MUSIC to determine the preliminary sizes of stormwater management controls required for the strategies discussed in **Section 6.2**. Stormwater controls are sized to provide the hydraulic residence time required to meet the water quality treatment targets stipulated in **Section 5.2**.

Table 5-4 presents the indicative sizes required for stormwater quality control devices based on *MUSIC* model results and retention requirements. The results have been broken down into the various sub-catchments to obtain a more detailed understanding of the sizes required in specific locations to assist in the development planning. Refer to **Figure 3** for illustration of the catchment locations.

Table 6-4 – Indicative Water Quality Treatment Control Sizing.

Catchment	GPT Required	Bio-Retention Area^^ (sqm)	End of Line Constructed Wetland ^
A1	-	-	-
A2	Y	2500	-
A3	-	-	-
A4	Y	3000	-
A5	Y	1500	-
A6	-	-	-
A7, A8	Y	2000	-
A9	Y	2500	-
A10	-	-	-
A11, A12	Y	2500	-
A13	Y	5500	-
A14	-	-	-
A15	Y	2500	-
A16	-	-	-
A17	Y	2500	-
A18	-	-	-
B1	-	-	
B2	Y	1950	
B3	-	-	
B4	Y	1050	Single downstream constructed wetland
B5	Y	2350	
B6	Y	1550	
B7	-	-	
B8	Y	1000	
B9	Y	1600	Assumed minimum wetland surface area
B10	Y	650	

B11	Y	4650	2.2Ha. Assumed permanent water volume 25,000m^3
B12, B13	Y	1400	
B14	Y	1800	
B15	Y	1600	
B16	Y	2975	
B17, B19, B31	Y	3200	
B18	-	-	
B20, B21	Y	-	
B22	-	-	-
B23, B26, B27	Y	4700	-
B24, B25	Y	3000	-
B28	-	-	-
B29	-	-	-
B30	-	-	-

^ The stated areas refer to treatment areas. The total footprint of the controls is likely to be in the order of 50% larger.

^^ Filter areas are likely to be approximately 50% of the stated bio-retention area.

The above water quality control structures would act to equal or exceed the level of water quality treatment required as outlined in **Section 5.2** as illustrated below;

1. ***Tributary 1:***

   - TSS Removal 89%
   - TP Removal 57%
   - TN Removal 45%

2. ***Tributary 2:***

   - TSS Removal 91%
   - TP Removal 64%
   - TN Removal 45%

It is noted that the MUSIC model does not account for the benefits of rainwater tanks. For the purposes of this report it was assumed that a third-pipe recycled water system would be implemented for the development and that rainwater tanks would not be required in addition to this system. If rainwater tanks are required to be adopted due to the absence of a third-pipe recycled water system, the resulting benefits can be incorporated into stormwater management control strategies at a later design stage.

It is noted that MUSIC simplifies a complex environment where many physical and bio-chemical processes can potentially influence the water quality. As MUSIC algorithms are based on observed average water quality performances (*which are highly variable*) it does not consistently represent a real scenario. All efforts have been made in this study to realistically represent the water quality scenario, however, the MUSIC results should be only considered as estimates of average conditions. As with any statistical representation, results could potentially be above or below average conditions. Hence, some degree of variability should be expected in the performance of the proposed SWMP.

6.5 Staging of Water Quality Treatment Measures

The constructed wetland at the downstream end of Anvil Creek Tributary 2 will be constructed as part of the Stage 1 development works. In addition at each stormwater outlet, a GPT and bioretention area needs to be constructed to manage water from the contributing catchments.

It is noted that the constructed wetland will initially be over designed and provide a higher level of treatment as not all of the upstream areas will be developed. However the deferral of the GPT and bioretention areas for the Stage 1 developed areas in this period is not possible as these measures are required to detain small, high frequency storm flows from the development and protect the watercourse from erosion due to higher frequency, higher velocity flows. In addition, the constructed wetland is the final stage in a stormwater treatment train and therefore relies on the upstream components to function correctly. Although rigorous sediment erosion controls will be in place during construction, the wetland and detention basin provides a measure of contingency during this period when the highest sediment loads are expected.

6.6 Water Quality Monitoring

At present, existing water quality data for the site is not available, however it is envisaged that at least six months prior to the commencement of construction works on site, a monitoring plan will be implemented in order to obtain a baseline data set for existing water quality conditions. During and after construction, water quality monitoring would continue to occur in order to assess the performance of temporary and permanent water quality improvement structures, which will allow refinement of the storm water management plan should it be required.

According to the ANZECC Guidelines, for a slightly to moderately disturbed lowland river system, the following target values for physical and chemical stressors should generally apply;

- Turbidity 6-50 NTU

- Salinity 125-2200 µS/cm
- pH 6.5-8.5
- Dissolved Oxygen 90-110 %
- Total Phosphorous 50 µg/L
- Total Nitrogen 250 µg/L
- Reactive Phosphorous 20 µg/L

For toxicant stressors, ANZECC Guidelines (2000) suggest that 95% protection level trigger values should be adopted for slightly to moderately disturbed ecosystems. Should water quality data suggest the ecosystem is more highly disturbed, the protection level trigger values may need to be re-visited. The specific values for this scenario are contained in Table 3.4.1 of the ANZECC Guidelines, and are for “soft” water. Conversion factors are to be applied for different levels of site specific water hardness, as per Table 3.4.4 of the ANZECC Guidelines (2000).

7. LOCALISED FLOODING ASSESSMENT

A flood hydraulics model was created to assess the 100 year ARI flood events over the Stage One Project Application area. Peak 100 year flow results from the *RAFTS* model established for this study (*refer to **Section 3***) were adopted in the hydraulic model. At the locations of detention basins, flood levels obtained from the RAFTS model were adopted, with HEC-RAS results provided for other general creek locations. It is noted that the flood levels and extents along the watercourses will need to continue to be developed and refined as detailed design of these areas continues.

7.1 Hydraulic Model Development

A hydraulic model simulates the movement of a flood wave through a river and its floodplain. The hydraulic model incorporates channel slope, roughness, and structures such as bridges and embankments.

The HEC-RAS software package was used to develop a hydraulic model of Tributaries 1 and 2. HEC-RAS is an integrated software package designed to enable one-dimensional river modelling using steady-flow, based on a single geometric representation of the stream network. It is the successor to the steady-flow *HEC-2 Water Surface Profiles* software, which has been used widely to simulate flood behaviour in river and channel systems, particularly where structures constrain free surface flow.

In its simplest application (*steady-flow simulations*), it automates the well known and respected *Standard Step Method* for backwater analysis.

The key downstream boundary for the Tributary 2 HEC-RAS model was modelled using the "level pool" routing method in the RAFTS model, to give a realistic representation of the headwater level upstream of the existing railway culverts. Previous studies simply adopted the steady-state backwater profile through the railway culverts, which generated a very conservative flood level upstream of the railway culverts. The current modelling for Tributary 2 reduces this effect by utilising the hydrodynamic effect of modelling the area upstream of the railway line as a "level pool" detention area within the RAFTS model, and then incorporating these results as a tailwater level for the remainder of the upstream HEC-RAS model profile.

7.1.1 Survey Data

Available topographic survey data provided (*refer to **Section 1.6***) was used to define the HEC-RAS cross sections. Refer to **Figure 4** for adopted cross-section locations. Considering hydraulic models are based on the topographical representation of the channel and flood plain, there is potential for inaccuracies in the predicted flood levels. However, any inaccuracies are likely to be conservative for this flooding assessment.

7.1.2 Channel and Floodplain Parameters

Manning's 'n' values are used to represent friction between water and a channel or floodplain. Generally, higher Manning's 'n' values imply increased friction and higher flood levels. As the proposed development landscape plan is currently at a conceptual level, the roughness of the proposed landform is uncertain. Hence, a Manning's 'n' roughness of 0.065 was generally applied for both channel and overbank regions in all cross-sections. This roughness is considered indicative of a moderately vegetated landscape, which is conservative considering the proposed landform is likely to incorporate pockets of native vegetation combined with areas of lawn and other recreational uses such as bike paths.

Contraction and expansion coefficients (*for evaluating creek transition losses*) of 0.1 and 0.3 were adopted for gradual contraction and expansion respectively. Higher coefficients of 0.3 and 0.5 were adopted for cross-section upstream and downstream of bridges and other structures. Ineffective flow areas were included in the model for ponded water in side channels or inactive flow areas on either side of the bridge and railway embankment.

7.1.3 Model Boundary Conditions

The culverts under the Main Northern Railway embankment are undersized and are likely to create a significant backwater during a 100 year flood. Hence, these culverts effectively act as the downstream control for flooding on the southern side of the rail embankment. As discussed above, in order to more accurately determine the downstream boundary conditions for Tributary 2, a "level pool" routing assessment was made (*using the RAFTS model*) of the natural detention basin formed by the railway culverts, which gave a peak 100 year water level of 34.14 m AHD for the critical storm duration. The HEC-RAS model was truncated south of the railway culverts, and this level adopted as a downstream boundary condition for the remainder of the upstream hydraulic model.

Upstream boundary conditions were estimated by applying normal depth conditions for the average channel grade in the upper reaches of each tributary.

7.2 Hunter Expressway Embankment

The construction Contractor for the Hunter Expressway adjacent to the Huntlee site, Abigroup, provided calculations and analysis for the culverts proposed under the road on both Tributary 1 and 2, as well as the downstream rail embankments. This provided additional information for expected flood levels between the Expressway and Rail embankment, as well as confirming that the proposed culverts under the Expressway had been sized with sufficient capacity to accommodate the existing

1:100 year peak flows without generating backwater which would impact on the Huntlee Development.

7.3 Results

Hydraulic model results were mapped against topographical survey data to determine the 100 year flood extents in Tributaries 1 and 2. Indicative flood planning levels were calculated by applying a 500mm freeboard to the predicted 100 year flood level. **Figure 4** maps both the predicted flood extents and the flood planning level extent. Additionally, areas which would require filling to be above the 100 year flood level are indicated in **Figure 4.**

Tabulated HEC-RAS output at each cross-section is attached in **Appendix C.**

7.4 Discussion

HEC-RAS modelling was undertaken to define the 100 year flood extent within Tributaries 1 and 2. As indicated in **Figure 4**, predicted flood extents would be generally confined within the designated drainage corridors. However, filling would be required in some areas (*particularly around detention basins*).

8. ASSESSMENT UNDER THE WATER MANAGEMENT ACT (2000)

The *Water Management Act 2000 (WMA)* provides guidelines regarding development constraints and riparian setback requirements for any controlled activity occurring within 40 meters from a river, lake or estuary. The proposed Stage One development has been assessed for compliance with the WMA.

8.1 Riparian Corridor Determination

The WMA specifies riparian corridor requirements based on the stream order. An assessment of stream order was undertaken in the "*Huntlee Project – Flood Study*" (*2007*). Refer to **Figure 6** from this report for stream order determination. As indicated in **Figure 6,** both Tributary 1 and 2 were determined to be first order streams in the upper reaches, becoming second order stream within a few hundred metres of the southern Project Application area boundary, with Tributary 2 becoming third order for the last 760m. This stream order determination has been adopted for this study and is presented in **Figure 6** in context with the proposed development layout.

The WMA stipulates that the riparian corridors should be determined through consideration of three riparian corridor zones, being the Core Riparian Zone (*CRZ*), Vegetated Buffer and Asset Protection Zone (*APZ*). The following sections discuss each of these zones in the context of the development proposal.

Core Riparian Zones (CRZ)

The CRZ refers to land contained within and adjacent to a water course. It is planned to provide the following CRZ setbacks where water courses are proposed to be retained on site:

- 1st order water courses would incorporate a 10 meter CRZ setback from the top of bank. A channel width of 5 meters is adopted resulting in a total CRZ width of 25 metres.

- 2nd and 3rd order water courses would incorporate a 20 meter CRZ setback from the top of bank. A channel width of 5 meters is adopted resulting in a total CRZ width of 45 metres.

The above setbacks are in accordance with the recommended CRZ widths in the WMA. **Figure 6** indicates the adopted CRZs within the Stage One Project Application area.

In accordance with the WMA, it is proposed to rehabilitate the CRZs, with the following rehabilitation measures proposed:

- Remove any exotic weeds from existing riparian corridors;

- Retain existing native riparian vegetation;
- Where required, enhance riparian vegetation to consist of a fully structured vegetation corridor, including ground cover, shrubs and trees;
- Where required provide bed and bank stabilisation; and
- Decommission all existing farm dams.

The above rehabilitation measures are likely to enhance the environmental function of the existing riparian corridors, which have been degraded from cattle grazing and land clearing. With reference to **Figure 6**, it is proposed to construct a number of detention basins within the CRZ.

Table 8-1 discusses the potential impacts of on-line basins on riparian functions and outlines proposed mitigation measures.

Table 8-1– Potential Impact of On-line Basins

Potential Impact on Riparian Functions	Proposed Mitigation Measures
Disruption of natural flow regime	The proposed online detention basins are for flood mitigation purposes only. They would provide an important function in mitigating peak flows during high return period rainfall events, minimising the impact on downstream flooding. As the basins would be designed for flood mitigation purposes, ponding would only occur when flows within the watercourse exceed the existing state, 2 year ARI peak flow rate. Water quality controls, located off-line from the water course would provide retention storage to mitigate the increased flow rates and runoff volume during lower return period rainfall events.
Disruption of natural geomorphologic functions such as sediment movement	As the proposed basins are not permanent water bodies, they are not likely to pose a barrier for sediment flow through the water course.
Introduction of barriers to the dispersal of biota and the loss of continuity between up-stream and downstream communities	It is possible to utilise open channel controls such as baffle weirs to allow for fauna connectivity through the basin controls. However, this is subject to further investigation at the detailed design stage.
Loss of riparian habitat	The basins would generally be constructed at proposed road crossings, resulting in no additional loss of riparian habitat due to the construction of the embankments (*assuming the proposed roads would be constructed*). Inundation behind the embankments would only temporarily occur during high return period rainfall events.

Vegetation Buffer

The vegetation buffer function is to protect the environmental integrity of the CRZ. The proposed vegetation buffer would contain offline bio-retention basins, which would be vegetated with indigenous riparian species. Measures would be undertaken to discourage access to the bio-retention basins.

Figure 6 indicates the proposed locality of bio-retention basins within the Stage One Project Application area.

Asset Protection Zone (APZ)

The APZ is a requirement of the NSW Rural Fire Service and is designed to protect assets from potential bushfire damage. A minimum 30 meter APZ has been assumed from the outer CRZ boundary. It is proposed that the APZ would contain stormwater infrastructure, roads and open space, but no building structures.

Figure 6 indicates the adopted APZ within the Stage One Project Application area.

In conclusion, the development proposal would:

- Provide CRZ and APZ setbacks in compliance with the WMA specifications;
- Construct managed bio-retention areas, vegetated with indigenous riparian vegetation, adjacent to the CRZ to provide a buffer between public areas and the CRZ;
- Increase the coverage of riparian vegetation within the Stage One Project Application Area, through retaining existing riparian vegetation and rehabilitating degraded areas;
- Decommission all online permanent water bodies (*i.e. existing farm dams*);
- Rehabilitate the existing channel where degraded;
- Construct five on-line detention basins (*for flood mitigation purposes only*) within the riparian corridors. The following mitigation measures would be adopted to address the potential impact of on-line stormwater controls on riparian functions:
    - Online basins would be designed to only detain water when flow exceeds the existing state 2 year ARI peak flow;
    - Online basins would not have a permanent water body;

- Detention basins would utilise proposed road crossings for embankments to minimise the loss of riparian vegetation areas; and

- Potential to implement open channel controls such as baffle weirs to allow for fauna connectivity through the embankment.

As discussed, the proposed online basins do not technically comply with the WMA (*2000*). However, with the proposed mitigation measures listed above, there would be no significant impact on the creekline corridors.

8.2 Farm Dams

It is proposed to decommission all existing farm dams within the Stage One Project Application area. Hence, an assessment of the existing harvestable rights is not required. With reference to the DWE website (*Farm dams in NSW*), the following dams are exempt from harvestable rights calculations:

- Dams for flood detention and mitigation. This applies to all proposed detention basins.

- Dams for the capture, containment and recirculation of drainage and/or effluent. This applies to all proposed water quality controls.

It is noted that no extraction is proposed from water quality controls for irrigation or other uses.

As all existing farm dams within the Stage One Project Application area are to be decommissioned, therefore a harvestable rights assessment of the existing farm dams is not required.

As discussed in **Section 8.1**, the development proposal would not include any online permanent water bodies in the form of detention basins, however, there would be one permanent water body (*Pond 1*) for water quality purposes.

8.3 Water Rights Assessment

It is proposed to construct three permanent water bodies in the Stage 1 Project Application Area (one water quality pond-adjacent Wine Country Drive, and two feature ponds-village centre). A water rights assessment was undertaken in order to determine whether these permanent water bodies would be required to be licensed under the WMA. Accordingly, an assessment was undertaken to determine the maximum dam volume permissible using the *Maximum harvestable right dam calculator* on the NSW Office of Water (*NOW*) website. Based on a multiplier of 0.075ML/ha and contributing catchment area of approximately 500ha (*catchment area of Tributary1 and 2, excluding the existing township of North Rothbury which partly falls within the Tributary 2 catchment area*). The results of the calculation are shown in **Table 8-2**.

Table 8-2 – Harvestable Rights Assessment

Proposed Water bodies	Project Application Area (ha)	Maximum Harvestable Rights (ML)	Proposed Permanent Water Volume (ML)
Pond 1 (adjacent Wince Country Drive), 2 & 3 (Feature ponds in Village Centre)	505	37.8	Pond 1 25ML Ponds 2 & 3 5-10ML

As indicated in **Table 8-2**, the proposed permanent water volume is less then that permitted using the maximum harvestable rights calculator. Hence, the proposed permanent water bodies will not be required to be licensed under the WMA. In addition, as the Pond 1 water body will be principally for water quality control, it should be exempt from harvestable rights calculations.

9. SALINITY AND GROUNDWATER MANAGEMENT

9.1 Salinity Management

The previous Douglas Partners Report (*Preliminary Geotechnical Assessment, Hunter Valley Project, 2003*) indicates that the site is generally classified as a high hazard site for salinity, however, recent experience by WorleyParsons indicates that more detailed assessment can sometimes result in significant spatial variation in salinity hazard (*from low to high*), depending on a number of factors including topography and type of underlying bedrock / soil. It is therefore recommended that prior to detailed design of the infrastructure and servicing (*including stormwater management controls / roadways etc.*) that further site specific salinity investigations are undertaken to categorise the level of salinity in various area of the site, in order to cost effectively manage the issue.

The Douglas Partners report indicates that generally the lower portions of the site are more likely to be susceptible to salinity, rather than ridgelines and slopes. Typical dryland salinity can also occur at the "break in slope", or where significant interference to natural subsoil drainage has occurred due to poor drainage practices.

Contrary to popular belief, the main source of salinity in these soil profiles is not "rising groundwater", but actually salts contained within the subsoils and underlying parent rock (*referred to as "B" & "C" Horizon materials respectively*). Surface soils ("*A" Horizon materials*) are relatively free of salinity due to leaching of the salts from the soil profile over time. Sound practices for the management of saline soils include the separation, sorting and stockpiling of these materials separately during earthworks, and then the replacement of the materials separately in their natural order (*i.e. B & C Horizon materials underlying A Horizon materials*). These practices prevent the exposure of saline soils at the surface, where vegetation and structures / services / roads are exposed to the higher saline levels. This also allows for the covering of the more saline materials with the less saline materials to maintain a free draining, relatively saline free surface soil that can support vegetation, and hence limit erosion. This practice is not dissimilar to the treatment of Acid Sulphate Soils, where potential Acid Sulphate Soil materials are kept below the water table etc.

The following general management strategies are listed that can be applied to the development to limit the impact of saline / sodic soils:

- maintaining the natural water balance and maintaining good drainage to prevent interference of good subsoil and surface drainage pathways, and to prevent rises in groundwater levels;
- avoiding disturbance or exposure of sensitive soils to minimise the potential for erosion;

- appropriate management of earthworks, particularly in higher salinity areas (*adjacent to creeklines*) – i.e. replacement of "A" Horizon material over the excavated subsoils, and appropriate application of gypsum / lime to manage dispersion and degradation of the soil would be appropriate. Similarly in filling operations for embankments, etc, subsoil should be used for embankment materials, possibly around an impermeable, treated core material, and overlaid by non saline "A Horizon" or imported topsoil, to allow vegetation planting (*turf*) on embankments. Avoidance of large trees on embankments would prevent windthrow uprooting more saline soils etc. If these practices are used for basins, then it is preferable that lining is not required. This could be examined by the geotechnical engineers at the detailed design stage, after more detailed investigation of the salinity risk.

- retaining or increasing appropriate native vegetation in strategic areas (*e.g. along creeklines and drainage paths*); and

- implementing building controls and engineering responses where appropriate.

Stormwater Drainage

Precautionary measures to reduce the potential for salinity and erosion problems may include:

- where possible, avoiding water collecting in low lying areas (*depressions, behind fill embankments or services trenches on the uphill sides of roads*) where these areas are proposed for construction of buildings or installation of infrastructure. This can lead to water logging of the soils, evaporative concentration of salts, and eventual breakdown in soil structure resulting in accelerated erosion. It is noted that stormwater management controls for the development of the site will require construction of ponds throughout the site to treat water quality and quantity. Such managed ponds need not pose a risk to buildings or infrastructure, provided they are sited away from this infrastructure.

- Where required, it is preferable that the stormwater quality ponds are created outside of areas with a moderate level of salinity and not downslope of these areas. The final location of the ponds along the corridors can be refined at the detailed design stage, once more details of salinity risk are identified.

- In the event that the stormwater basins are located within the areas of moderate salinity, consideration of the saline conditions should be taken into account in the detailed design. The most appropriate mitigation measures should be assessed on a site by site basis once the design of the basins has been completed and may include:

    - conditioning of the soil to be utilised within the embankment of the ponds, with gypsum / lime, to minimise the risk of structural degradation/erosion;

- careful control of compaction and moisture control during earthworks to ensure creation of a low permeability embankments to retard migration of saline water into the basin; and

- lining of the stormwater ponds with an appropriate liner (*such as HPDE*) where the results of further analysis preclude other more practical measures.

Roads and the shoulder areas should also be designed to be well drained, particularly with regard to drainage of surface water. There should not be excessive concentrations of runoff or ponding that would lead to waterlogging of the pavement or additional recharge to the groundwater system, particularly groundwater perched at the interface between soils and the often, relatively impermeable bedrock.

Surface drains should generally be provided along the top of all batters to reduce the potential for concentrated flows of water down slopes possibly causing erosion. Well-graded subsoil drainage should be provided at the base of all slopes where there are road pavements below the slope to reduce the risk of waterlogging.

Controls on final surface and road gradings (*minimum surface slope of 40 Horizontal (H):1 Vertical (V) is typically suggested*), in order to improve surface drainage and reduce ponding and waterlogging, which can lead to evaporation and salinity. It is noted, that owing to the flat terrain present in some parts of the site, a minimum surface slope in the order of 100 Horizontal (H):1 Vertical (V) is more likely to be achieved. Hence, careful detailing of surface and subsurface drawings will be required in the flatter areas of the site. This includes provision of adequate drainage at the base of service trenches, pavements, around surface structures and footings. It is important to avoid pooling or ponding of water on the surface or adjacent to concrete or steel structures, such as footings or services, to minimise the risk of structural degradation of these structures via contact with saline water.

Where possible materials and waters used in the construction of roads and fill embankments should be selected to contain minimal or no salt. In the event of use of imported materials of moderate (*or greater*) salinity and where salinisation could be a problem, a capping layer of either topsoil or sandy materials should be placed to reduce capillary rise, act as a drainage layer and also reduce the potential for dispersive behaviour in any sodic soils.

It may be necessary to mix gypsum / lime into filling containing sodic soils and cuts where sodic soils are exposed on slopes, to improve soil structure and reduce the potential for scour.

Salt tolerant species should be considered if re-planting close to gullies and creeks and in areas of moderate and greater salinity to reduce soil erosion and maintain the existing evapo-transpiration and groundwater levels.

Building and Infrastructure Construction

The extent of measures adopted during construction, in particular the concrete, masonry and steel requirements, should depend on the particular level of salinity, aggressivity or corrosivity at the actual site. Soil from specific building sites or services alignments in areas suspected to be moderately (or more) saline should be sampled, tested and classified for soil salinity, aggressivity and corrosivity to the proposed depth of excavation. Sampling should preferably be carried out by a geotechnical consultant at the same time the site is classified for soil reactivity (shrink – swell behaviour). These classifications would involve additional testing of soil or water samples for pH, electrical conductivity, sodicity, and possibly sulphates and chlorides. Once the testing has been carried out at the site specific areas, then the geotechnical and structural design engineers can specify appropriate design controls to limit the effects of saline soils on building and infrastructure construction.

Typical example measures include:

- bedding layer and membranes under concrete slabs to act as a moisture barrier and drainage layer to restrict capillary rise under the slab.
- use of high strength concrete or sulphate resistant cement.
- as an alternative to slab-on-ground construction, suspended slab or pier and beam construction can be considered.
- other industry standard measures can be considered to improve the durability of concrete in saline environments including adjustment of water / cement ratio, damp courses and water proofing agents, subsoil drainage in service trenches etc.

9.2 Groundwater Management

Little field investigation has been undertaken regarding groundwater levels or groundwater movement within the Stage One Project Application area. It is understood that some groundwater assessment has been undertaken in association with the nearby Ayrfield Colliery.

Regardless, it can be inferred that groundwater on the subject site would be expected to generally follow the natural surface drainage paths, with surface aquifers being generally confined over relatively impermeable unweathered rock strata.

There are no current plans to harvest existing groundwater resources at the site, and there will be limited potential to recharge to the groundwater aquifer, due to the nature of the clay soils on the site, and the introduction of higher levels of impermeable surfaces.

As discussed above, sound drainage practices are required to ensure that surface ponding does not occur, and that adequate through flow of near surface aquifers occurs, limiting the backing up of groundwater, possibly causing concentration of potential saline groundwater in the near surface soils.

10. PRELIMINARY DESIGN OF STORMWATER CONTROLS

A preliminary design of the required stormwater management controls was undertaken as part of this assessment. The design is based on the quantitative water quality and hydrologic investigations discussed in preceding sections. The design includes indicative sizing of water quality and quantity controls as well as a preliminary layout of road and lot drainage requirements for the Stage 1 Residential Release Area. The resulting preliminary stormwater management strategy and trunk drainage design is presented spatially in **Figure 5**.

10.1 Detention Basin Design

A preliminary design of the proposed detention basins was undertaken using 12D civil design software. The following design parameters were adopted:

- Where batters are required, typically a 1 in 6 batter slope was adopted (*approximately 1 in 4 batter slope for Basin 4*);
- Road crossings over the basin embankment were set at the peak 100 year level, and would act as an emergency spillway; and
- A 500mm freeboard was applied to minimum levels of surrounding roads and development areas upstream of the basin embankment. In some cases this required minor filling.

10.2 Construction Phase

During construction, sediment and erosion control structures would be designed and installed in accordance with the NSW Department of Housing "*Managing Urban Stormwater – Soils and Construction*" (*Blue Book*). Staging of the development will minimise impacts during construction. These controls will reduce any adverse impacts on receiving water quality during the construction stage. A detailed soil and water management plan will be required for each stage at a later design stage.

11. SUMMARY AND RECOMMENDATIONS

11.1 Hydrology

A hydrologic assessment was undertaken to determine the existing and developed state hydrological conditions and quantitatively assess the impact of the proposed development on the local and regional hydrology. The assessment considered the catchments holistically, and adopted the Northern Railway embankment as a point of reference for comparison of pre and post development hydrological results. This ensures that the appropriate critical storm durations were adopted for assessing downstream impacts.

It was determined that the proposed development would potentially increase both peak flows and runoff volume. Hence, a mitigation strategy incorporating five detention basins was adopted. It was determined that two detention basins, constructed on-line, would be required in the Tributary One catchment. As the Tributary 2 catchment is to be only partially developed under the Stage One Project Application, consideration was given to the ultimate mitigation requirements. It was determined that a third online detention basin would be required as part of the forthcoming Town Centre Project Application.

Modelling indicates that the proposed detention strategy would provide adequate attenuation of flow so that there is no increase in peak discharge from both Tributaries 1 and 2 during a range of Average Recurrence Interval (*ARI*) design rainfall events at the two downstream parity points.

Additional stormwater controls are proposed for water quality control purposes. These controls would provide up to 20mm of retention storage per unit of impervious area within the proposed development. If empty at the start of storm, the 20 mm of retention storage would provide a moderate reduction in runoff volume as well as reduced peak flows, with significant reductions likely during the lower return period events such as the 2 year ARI storm. However, as the retention storage would take a number of days to empty, it has been conservatively assumed that they would be full at the beginning of a design storm, and have not been accounted for in the modelling for peak flow attenuation accordingly.

Modelling predicts an increase in runoff volume, which could potentially exacerbate backwater flooding at the downstream Northern Railway culverts. An investigation was undertaken to determine the impact of the increase in runoff volume on backwater flooding.

The results of these investigations indicated that peak discharge rates and peak water levels downstream of the development would be no worse than under existing conditions for the critical duration, 100 year ARI event.

11.2 Flooding

A flooding assessment was undertaken to determine the 100 year ARI flood extent and flood planning levels within Tributaries 1 and 2.

The impact of the increased runoff volume on downstream flooding has been assessed through the modelling of the area upstream of the railway culvert using a "level pool" routing method within the RAFTS model. The model results suggest that the development (*with controls in place*) will have no significant impact on the flood levels within the existing flood storage area immediately upstream of the railway embankment.

As discussed previously, the existing culverts under the Northern Railway were adopted in the hydraulic model, with the modelling indicating potential for backwater flooding upstream of the railway embankment. This assessment considered in more detail the flow attenuation due to the natural detention basin formed by the topography and existing railway culverts. This saw a significant decrease in peak flow rates and levels from previous investigations for Tributary 2. Backwater effects from Tributary 1 do not affect the subject site

HEC-RAS modelling was undertaken to define the 100 year flood extent within Tributaries 1 and 2. Modelling indicates that a 100 year flood would be generally contained within the designated drainage corridors. However, filling would be required in some areas (*particularly around detention* basins).

Predicted 100 year flood extents and flood planning levels are presented in **Figure 4**.

11.3 Assessment under the Water Management Act

The Stage One development proposal was assessed for compliance with the *Water Management Act 2000* (*WMA*). The assessment concluded that the development proposal would:

- Provide CRZ and APZ setbacks in compliance with the WMA specifications;
- Construct managed bio-retention areas, vegetated with indigenous riparian vegetation, adjacent to the CRZ to provide a buffer between public areas and the CRZ;
- Increase the coverage of riparian vegetation within the Stage One Project Application Area, through retaining existing riparian vegetation and rehabilitating degraded areas;
- Decommission all online permanent water bodies (*i.e. existing farm dams*);
- Rehabilitate the existing channel where degraded;

- Construct three on-line detention basins (*for flood mitigation purposes only*) within the riparian corridors. The following mitigation measures would be adopted to address the potential impact of on-line stormwater controls on riparian functions:
  - Online basins would be designed to only detain water when flow exceeds the existing state 2 year ARI peak flow;
  - Online basins would not have a permanent water body;
  - Detention basins would utilise proposed road crossings for embankments to minimise the loss of riparian vegetation areas; and
  - Potential to implement open channel controls such as baffle weirs to allow for fauna connectivity through the embankment.

The proposed online basins do not technically comply with the WMA (*2000*). However, with the proposed mitigation measures listed above, there would be no significant impact on the creekline corridors.

11.4 Hunter Expressway & Rail Embankment

Both Tributary 1 and Tributary 2 discharge through culverts underlying the Hunter Expressway and Railway embankment to the north of the site.

As part of this study a more detailed investigation and liaison with the Hunter Expressway Contractor was undertaken. This assessment demonstrated a number of outcomes, as follows:

1. The previous “steady state” backwater assessment of flooding from the railway culverts was significantly overstated in previous reports;
2. The culverts on Tributary 1 and 2 proposed as part of the Hunter Expressway have been sized to accommodate the 1:100 year peak flows with minimal backwater effects;
3. The loss of flood storage due to the Hunter Expressway embankment upstream of the rail line and proposed changes to culverts through the rail embankment do not generate any backwater impacts on the Huntlee Development;
4. The proposed stormwater controls including detention basins limit downstream peak water levels and flows to below existing conditions for the critical duration 100 year event (*including 10% increase in rainfall intensity for climate change*);

11.5 Stormwater Quality

A water quality analysis was undertaken to develop a water quality control strategy in compliance with Government requirements and industry best practise. The strategy addresses both the potential impact of the development proposal on water quality as well as disturbances to the hydrologic regime during low return period rainfall events. Cessnock City Council has previously indicated that 'end of pipe' water quality controls were preferred over a water sensitive urban design approach. Hence, end of pipe controls were adopted accordingly.

The following water quality control strategies were developed for the site:

- **Bio-Retention Basins**: It is proposed to construct bio-retention basins at the edges of CRZ. Bio-retention basins would consist of vegetated areas with enhanced filtration media which would typically be 500-600mm deep. Filtered stormwater would be collected in an underlying subsurface drainage system and directed into the main channel. Up to 20mm of retention storage per unit area of impervious surfaces in the contributing catchment would be provided above the filter media through the ponding of water up to 500 mm deep. Proprietary Gross Pollutant Traps (*GPTs*) would be installed upstream of the bio-retention basins to remove litter and coarse to medium sized sediments. A high flow by-pass would be provided upstream of the GPTs to prevent scouring within the basins.

- **Water Quality Control Ponds**: In addition to the treatment of runoff from upstream developed catchments, the proposed permanent pond will serve as a visual feature, a focal point for community recreation and assist with the creation of a sense of place for the development.

  The permanent water quality pond is proposed to be located off line. This proposal requires the existing ephemeral creek line to be relocated around the extent of the new pond. The existing creek line is degraded and there is very little if any existing riparian vegetation. The relocated creek line will be extensively revegetated as part of the landscape plan and will provide a visual contrast to the permanent water body from the adjacent town centre development.

  Locating the pond offline will allow intermittent maintenance and cleaning to take place, with the ability to prevent inflows to the pond during these times. The configuration prevents high flows entering the pond which assists to avoid re-entrainment of pollutants stored in the pond, which can essentially be "stirred" up and flushed out of the pond during large storm events.

  It is proposed that during low flows, an engineering control will divert some flows into the water quality control pond and also allow some flows to continue within the creek to ensure a minimum flow regime is maintained for sustaining the riparian vegetation and associated aquatic fauna. This engineering control will be designed as part of the detailed infrastructure plans and will be prepared on advice from the ecologist in regards to the minimum required flow regime for the creek vegetation and associated biota.

A MUSIC water quality model was used to determine the required size of the water quality controls throughout the Stage 1 development. It is expected that the sizes of these control structures and locations will be developed further during future detailed engineering design stages.

12. REFERENCES

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2) New South Wales Government (*2005*), *'Floodplain Development Manual: the management of flood liable land'* ISBN 07313 0370 9

3) Willing & Partners Pty Ltd (*1996*), *'RAFTS-XP User Manual'*.

4) Bureau of Meteorology Website (*www.Bom.gov.au*)

5) Duncan, H. P. (February, 1999) '*Urban Stormwater Quality: A Statistical Overview*'.

6) Institution of Engineers, Australia (2006) '*Australian Runoff Quality*'

7) NSW Department of Water and Energy, 2002, 'Water Management Act 2002

8) Patterson Britton and Partners, August 2007, 'Huntlee Project- Flood Study'

9) Patterson Britton and Partners, August 2007, 'Principles of Stormwater Management

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11) Douglas Partners, 2003, 'Preliminary geotechnical Assessment, Hunter Valley Project, North Rothbury'

12) Department of Environment and Climate Change & CMA, 2007, 'Managing Urban Stormwater: Environmental Targets-Consultation Draft'

13) DWE Website: Farm dams in NSW*: http://www.naturalresources.nsw.gov.au /water/pdf/farmdams_do_you_need_licence.pdf*

14) WorleyParsons, 2008 - Huntlee Project – Trunk Stormwater and Flooding Assessment

Appendix A – RAFTS Parameters

Anvil Creek Tributary 1 - RAFTS Parameters (Existing State)

Subcatchment ID	Catchment Area				Slope	Roughness		Initial Loss		Continuing Loss	
	Pervious surfaces (ha)	Impervious Surfaces (ha)	Total (ha)	Percentage Impervious (%)	(%)	Pervious surfaces (n)	Impervious Surfaces (n)	Pervious surfaces (mm)	Impervious Surfaces (mm)	Pervious surfaces (mm/h)	Impervious Surfaces (mm/h)
A 1	40.1	0	40.1	0	7.7	0.08	NA	15	NA	3.6	NA
A 2	12.2	0	12.2	0	5.3	0.08	NA	15	NA	3.6	NA
A 3	3.1	0	3.1	0	1.0	0.06	NA	15	NA	3.6	NA
A 4	11.7	0	11.7	0	3.7	0.06	NA	15	NA	3.6	NA
A 5	6.0	0	6.0	0	5.4	0.08	NA	15	NA	3.6	NA
A 6	2.0	0	2.0	0	1.0	0.06	NA	15	NA	3.6	NA
A 7	8.7	0	8.7	0	3.6	0.06	NA	15	NA	3.6	NA
A 8	4.3	0	4.3	0	3.9	0.1	NA	15	NA	3.6	NA
A 9	9.6	0	9.6	0	6.0	0.06	NA	15	NA	3.6	NA
A 10	2.9	0	2.9	0	1.0	0.06	NA	15	NA	3.6	NA
A 11	13.5	0	13.5	0	4.0	0.06	NA	15	NA	3.6	NA
A 12	10.5	0	10.5	0	5.7	0.1	NA	15	NA	3.6	NA
A 13	4.5	0	4.5	0	1.0	0.06	NA	15	NA	3.6	NA
A 14	21.7	0	21.7	0	5.4	0.06	NA	15	NA	3.6	NA
A 15	8.9	0	8.9	0	5.5	0.06	NA	15	NA	3.6	NA
A 16	32.6	0	32.6	0	4.3	0.1	NA	15	NA	3.6	NA
A 17	4.2	0	4.2	0	5.0	0.06	NA	15	NA	3.6	NA
A 18	8.5	0	8.5	0	4.5	0.1	NA	15	NA	3.6	NA
Total	**205.0**	**0**	**205.0**								

Anvil Creek Tributary 2 - RAFTS Parameters (Existing State)

Subcatchment ID	Catchment Area				Slope	Roughness		Initial Loss		Continuing Loss	
	Pervious surfaces (ha)	Impervious Surfaces (ha)	Total (ha)	Percentage Impervious (%)	(%)	Pervious surfaces (n)	Impervious Surfaces (n)	Pervious surfaces (mm)	Impervious Surfaces (mm)	Pervious surfaces (mm/h)	Impervious Surfaces (mm/h)
B1	8.7	0	8.7	0	10.0	0.08	NA	15	NA	3.6	NA
B2	11.4	0	11.4	0	9.7	0.08	NA	15	NA	3.6	NA
B3	2.8	0	2.8	0	5.0	0.08	NA	15	NA	3.6	NA
B4	6.1	0	6.1	0	4.8	0.08	NA	15	NA	3.6	NA
B5	13.7	0	13.7	0	6.3	0.06	NA	15	NA	3.6	NA
B6	7.3	0	7.3	0	7.0	0.06	NA	15	NA	3.6	NA
B7	5.0	0	5.0	0	5.0	0.06	NA	15	NA	3.6	NA
B8	5.9	0	5.9	0	4.5	0.06	NA	15	NA	3.6	NA
B9	9.4	0	9.4	0	6.1	0.06	NA	15	NA	3.6	NA
B10	3.7	0	3.7	0	5.7	0.06	NA	15	NA	3.6	NA
B11	20.0	0	20.0	0	6.3	0.08	NA	15	NA	3.6	NA
B12	4.6	0	4.6	0	5.5	0.06	NA	15	NA	3.6	NA
B13	10.2	0	10.2	0	4.6	0.06	NA	15	NA	3.6	NA
B14	10.6	0	10.6	0	4.9	0.06	NA	15	NA	3.6	NA
B15	9.5	0	9.5	0	4.6	0.06	NA	15	NA	3.6	NA
B16	17.5	0	17.5	0	3.8	0.08	NA	15	NA	3.6	NA
B17	24.0	0	24.0	0	3.4	0.1	NA	15	NA	3.6	NA
B18	2.8	0	2.8	0	0.5	0.06	NA	15	NA	3.6	NA
B19	16.3	0	16.3	0	3.8	0.08	NA	15	NA	3.6	NA
B20	9.2	0	9.2	0	3.4	0.1	NA	15	NA	3.6	NA
B21	12.2	0	12.2	0	2.8	0.1	NA	15	NA	3.6	NA
B22	4.2	0	4.2	0	0.5	0.06	NA	15	NA	3.6	NA
B23	5.3	0	5.3	0	2.7	0.06	NA	15	NA	3.6	NA
B24	15.0	0	15.0	0	3.2	0.08	NA	15	NA	3.6	NA
B25	6.3	0	6.3	0	5.5	0.06	NA	15	NA	3.6	NA
B26	8.0	0	8.0	0	4.0	0.1	NA	15	NA	3.6	NA
B27	10.3	0	10.3	0	3.9	0.1	NA	15	NA	3.6	NA
B28	11.3	0	11.3	0	2.9	0.08	NA	15	NA	3.6	NA
B29	9.5	0	9.5	0	3.2	0.1	NA	15	NA	3.6	NA
B30	10.2	0	10.2	0	4.0	0.1	NA	15	NA	3.6	NA
B31	9.1	0	9.1	0	4.3	0.08	NA	15	NA	3.6	NA
RothE	1.8	1.8	3.6	50	7.0	0.035	0.015	10	1.5	2.5	0
RothW	5.4	5.4	10.7	50	7.0	0.035	0.015	10	1.5	2.5	0
Total	**307.3**	**7.2**	**314.4**								

Anvil Creek Tributary 1 - Developed State RAFTS Parameters

Subcatchment ID	Catchment Area				Slope	Roughness		Initial Loss		Continuing Loss	
	Pervious surfaces (ha)	Impervious Surfaces (ha)	Total (ha)	Percentage Impervious (%)	(%)	Pervious surfaces (n)	Impervious Surfaces (n)	Pervious surfaces (mm)	Impervious Surfaces (mm)	Pervious surfaces (mm/h)	Impervious Surfaces (mm/h)
A 1	40.1	0	40.1	0	7.7	0.08	NA	15	NA	3.6	NA
A 2	4.3	7.9	12.2	65	5.3	0.025	0.015	10	1.5	2.5	0
A 3	2.3	0.8	3.1	26	1.0	0.06	0.015	15	1.5	3.6	0
A 4	4.9	9	13.9	65	3.7	0.025	0.015	10	1.5	2.5	0
A 5	2.1	3.9	6.0	65	5.4	0.025	0.015	10	1.5	2.5	0
A 6	1.5	0.5	2.0	25	1.0	0.06	0.015	15	1.5	3.6	0
A 7	3	5.7	8.7	66	3.6	0.025	0.015	10	1.5	2.5	0
A 8	4.3	0	4.3	0	3.9	0.1	NA	15	NA	3.6	NA
A 9	3.4	6.2	9.6	65	6.0	0.025	0.015	10	1.5	2.5	0
A 10	2.2	0.7	2.9	24	1.0	0.06	0.015	15	1.5	3.6	0
A 12	10.5	0	10.5	0	5.7	0.1	NA	15	NA	3.6	NA
A 11	4.7	8.8	13.5	65	4.0	0.025	0.015	10	1.5	2.5	0
A 13	7.6	14.1	21.7	65	5.4	0.025	0.015	10	1.5	2.5	0
A 14	3.4	1.1	4.5	24	1.0	0.06	0.015	15	1.5	3.6	0
A 15	3.3	6.2	9.5	65	5.5	0.025	0.015	15	1.5	3.6	0
A 16	32.6	0	32.6	0	4.3	0.1	NA	15	NA	3.6	NA
A 17	1.1	3.2	4.2	75	5.0	0.025	0.015	10	1.5	2.5	0
A 18	8.5	0	8.5	0	4.5	0.1	NA	15	NA	3.6	NA
Total	**139.8**	**68.1**	**207.8**								

Anvil Creek Tributary 2 - Developed State RAFTS Parameters

Subcatchment ID	Catchment Area				Slope	Roughness		Initial Loss		Continuing Loss	
	Pervious surfaces	Impervious Surfaces	Total	Percentage Impervious		Pervious surfaces	Impervious Surfaces	Pervious surfaces	Impervious Surfaces	Pervious surfaces	Impervious Surfaces
	(ha)	(ha)	(ha)	(%)	(%)	(n)	(n)	(mm)	(mm)	(mm/h)	(mm/h)
B1	8.7	0.0	8.7	0	10.0	0.08	NA	15	NA	3.6	NA
B2	4.0	7.4	11.4	65	9.7	0.025	0.015	10	1.5	2.5	0
B3	2.5	0.3	2.8	11	5.0	0.025	0.015	10	1.5	2.5	0
B4	2.1	4.0	6.1	65	4.8	0.025	0.015	10	1.5	2.5	0
B5	4.8	8.9	13.7	65	6.3	0.025	0.015	10	1.5	2.5	0
B6	2.6	4.7	7.3	65	7.0	0.025	0.015	10	1.5	2.5	0
B7	4.5	0.5	5.0	10	5.0	0.025	0.015	10	1.5	2.5	0
B8	2.1	3.8	5.9	65	4.5	0.025	0.015	10	1.5	2.5	0
B9	3.3	6.1	9.4	65	6.1	0.025	0.015	10	1.5	2.5	0
B10	1.3	2.4	3.7	65	5.7	0.025	0.015	10	1.5	2.5	0
B11	7.0	13.0	20.0	65	6.3	0.025	0.015	10	1.5	2.5	0
B12	1.6	3.0	4.6	65	5.5	0.025	0.015	10	1.5	2.5	0
B13	7.7	2.6	10.3	25	4.6	0.025	0.015	10	1.5	2.5	0
B14	3.7	6.9	10.6	65	4.9	0.025	0.015	10	1.5	2.5	0
B15	3.3	6.2	9.5	65	4.6	0.025	0.015	10	1.5	2.5	0
B16	4.4	13.1	17.5	75	3.8	0.025	0.015	10	1.5	2.5	0
B17	3.6	20.4	24.0	85	3.4	0.025	0.015	10	1.5	2.5	0
B18	2.8	0.0	2.8	0	0.5	0.06	NA	10	NA	2.5	NA
B19	4.1	12.2	16.3	75	3.8	0.025	0.015	10	1.5	2.5	0
B20	2.3	6.9	9.2	75	3.4	0.025	0.015	10	1.5	2.5	0
B21	1.8	10.4	12.2	85	2.8	0.025	0.015	10	1.5	2.5	0
B22	4.2	0.0	4.2	0	0.5	0.06	NA	10	NA	2.5	NA
B23	0.8	4.5	5.3	85	2.7	0.025	0.015	10	1.5	2.5	0
B24	2.2	12.8	15.0	85	3.2	0.025	0.015	10	1.5	2.5	0
B25	6.3	0.0	6.3	0	5.5	0.06	NA	15	NA	3.6	NA
B26	1.2	6.8	8.0	85	4.0	0.025	0.015	10	1.5	2.5	0
B27	1.5	8.8	10.3	85	3.9	0.025	0.015	10	1.5	2.5	0
B28	5.7	5.7	11.3	50	2.9	0.025	0.015	10	1.5	2.5	0
B29	4.8	4.8	9.5	50	3.2	0.025	0.015	10	1.5	2.5	0
B30	5.1	5.1	10.2	50	4.0	0.025	0.015	10	1.5	2.5	0
B31	2.3	6.8	9.1	75	4.3	0.025	0.015	10	1.5	2.5	0
RothE	1.8	1.8	3.6	50	7.0	0.035	0.015	10	1.5	2.5	0
RothW	5.4	5.4	10.7	50	7.0	0.035	0.015	10	1.5	2.5	0
Total	**119.5**	**195.2**	**314.6**								

Anvil Creek Tributary 1 - Adopted Lagtimes

Flow Path		Length	Fall	Grade	Lag Time (min)	
From	To	m	m	%	Existing	Developed
A1	A3	400	6	1.5	5.4	5.4
A3	A6	270	3	1.1	4.3	4.3
A6	A10	350	5	1.4	4.9	4.9
A10	A14	350	4	1.1	5.5	5.5
A14	A18	255	4	1.6	3.4	3.4
A18	A16	355	7	2.0	4.2	4.2
A8	A6	720	16	2.2	8.0	6.2
A12	A10	500	12	2.4	5.4	4.1

Anvil Creek Tributary 2 - Adopted Lagtimes

Flow Path		Length	Fall	Grade	Lag Time (min)	
From	To	(m)	(m)	(%)	Existing	Developed
B1	P1	410	10.0	2.4	4.4	4.4
P1	P1	550	10.0	1.8	6.8	6.8
P2	P3	800	10.0	1.3	11.9	11.9
P3	P4	250	1.0	0.4	6.6	6.6
P4	P5	600	4.5	0.8	11.5	11.5
P5	P6	300	3.5	1.2	4.6	4.6
B8 & B5	P1	360	6.0	1.7	4.6	4.6
B9 & B6	P1	130	2.5	1.9	1.6	1.6
B12 & B14	P3	550	7.0	1.3	8.1	8.1
B15	P3	350	4.5	1.3	5.1	5.1
B19	P4	450	13.0	2.9	4.4	3.4
B20	B21	450	11.0	2.4	4.8	3.7
B26	B27	550	16.0	2.9	5.4	4.1
B27	B23	270	2.0	0.7	5.2	4.0
B25	B24	400	12.0	3.0	3.8	3.0
B30	B29	350	6.0	1.7	4.5	4.5
Roth W	P3	650	28.0	4.3	5.2	4.0
Roth E	P1	500	22.0	4.4	4.0	3.1
B31	P4	600	17.0	2.8	5.9	4.6

Adopted Detention Basin Details

Basin 1

Stage (m)	Storage (m^3/s)	Stage (m)	Discharge (m^3/s)
0.00	0	0.00	0.00
0.00	10	0.00	0.00
0.87	3266	0.87	2.38
1.10	4145	1.10	3.45
1.16	4368	1.16	4.89
1.50	5632	1.50	7.13

Basin 3

Stage (m)	Storage (m^3/s)	Stage (m)	Discharge (m^3/s)
0.00	0	0.00	0.00
0.00	10	0.00	0.00
0.63	3387	0.63	1.17
0.77	4130	0.77	1.64
1.01	5423	1.01	2.48
1.50	8091	1.50	3.98

Basin 2

Stage (m)	Storage (m^3/s)	Stage (m)	Discharge (m^3/s)
0.00	0	0.00	0.00
0.00	15	0.00	0.00
0.73	10226	0.60	5.13
0.77	10789	0.77	7.43
0.79	11041	0.90	10.49
1.50	20000	1.50	16.30

Basin 4

Stage (m)	Storage (m3/s)	Stage (m)	Discharge (m3/s)
0.00	0	0.00	0
0.50	363	0.37	1
1.00	1354	1.29	5
1.50	3697	2.08	10
2.00	7925	2.80	15
2.50	24956	3.85	20
3.00	48926	3.95	25
3.50	74352	3.98	30
3.60	79643	3.99	35
3.70	85026	4.02	40

Appendix B – RAFTS Results

RAFTS Parity Location		Tributary 1 P1-5			Tributary 2 P2-5		
		Existing State	Developed State		Existing State	Developed State	
			No Controls	With Controls		No Controls	With Controls
Storm ARI	Duration *(min)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*
2 yr	30 min	0.8	10.1	4.0	1.9	17.9	5.9
2 yr	45 min	1.9	10.2	4.7	3.2	18.0	7.8
2 yr	60 min	2.7	11.0	5.2	4.5	19.8	9.2
2 yr	90 min	3.4	11.2	5.4	5.7	20.3	9.9
2 yr	120 min	3.9	11.4	5.6	6.6	20.5	10.4
2 yr	180 min	4.0	8.3	5.0	6.6	17.0	9.8
2 yr	270 min	3.7	8.1	5.2	6.0	17.3	9.7
2 yr	360 min	4.8	7.2	5.2	7.8	16.1	10.4
2 yr	540 min	6.0	7.2	5.8	9.7	14.9	10.8
2 yr	720 min	5.3	7.7	5.2	8.3	13.8	9.7
5 yr	30 min	2.9	13.7	5.9	4.7	23.9	9.4
5 yr	45 min	4.5	13.9	7.1	7.3	24.7	11.6
5 yr	60 min	5.7	15.2	7.9	9.4	26.9	13.2
5 yr	90 min	6.6	15.0	8.0	10.9	27.6	14.1
5 yr	120 min	7.2	15.8	8.3	11.9	27.4	14.7
5 yr	180 min	6.9	11.7	7.4	11.5	23.0	13.7
5 yr	270 min	6.9	11.4	7.8	11.6	23.5	13.8
5 yr	360 min	8.0	10.0	7.8	13.1	21.8	14.3
5 yr	540 min	8.6	9.9	8.7	14.1	19.7	14.4
5 yr	720 min	8.1	10.6	7.8	12.6	18.2	13.0
20 yr	30 min	6.2	19.2	9.6	10.0	32.4	13.9
20 yr	45 min	8.6	19.2	11.1	13.7	33.8	16.6
20 yr	60 min	10.2	20.7	11.9	16.6	36.5	18.3
20 yr	90 min	11.0	20.6	12.0	18.4	37.4	18.8
20 yr	120 min	11.8	21.5	12.4	19.7	37.1	19.2
20 yr	180 min	11.1	16.4	11.1	18.1	31.3	18.4
20 yr	270 min	11.6	15.6	11.7	19.1	31.3	18.6
20 yr	360 min	12.2	13.8	11.7	20.1	28.8	18.8
20 yr	540 min	12.1	13.3	12.1	19.8	25.9	18.5
20 yr	720 min	11.5	14.5	11.8	18.0	24.0	17.2
100 yr	30 min	11.5	25.8	13.5	18.3	42.8	18.9
100 yr	45 min	14.6	25.6	15.3	23.7	45.9	20.9
100 yr	60 min	16.4	27.6	16.2	27.1	48.6	22.2
100 yr	90 min	17.0	27.1	16.3	28.8	48.8	22.6
100 yr	120 min	17.6	28.0	16.7	30.2	48.3	22.9
100 yr	180 min	16.1	21.3	15.0	26.4	40.4	22.1
100 yr	270 min	16.8	20.4	15.3	27.7	39.7	22.0
100 yr	360 min	16.8	18.1	15.4	27.6	35.9	22.2
100 yr	540 min	15.7	17.0	15.3	25.9	32.1	21.7
100 yr	720 min	15.1	18.3	14.7	23.7	29.8	20.3

RAFTS Parity Location		Tributary 1 P1-6			Tributary 2 P2-6		
		Existing State	Developed State		Existing State	Developed State	
			No Controls	With Controls		No Controls	With Controls
Storm ARI	Duration *(min)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*	Peak Flow *(m3/s)*
2 yr	30 min	0.9	10.1	4.1	1.7	15.5	6.1
2 yr	45 min	2.1	10.3	4.9	3.4	17.3	8.0
2 yr	60 min	3.0	11.1	5.5	4.8	18.7	9.3
2 yr	90 min	3.8	11.3	5.7	6.1	18.9	10.5
2 yr	120 min	4.3	11.6	5.9	7.0	19.0	11.0
2 yr	180 min	4.5	8.4	5.4	7.1	16.8	10.6
2 yr	270 min	4.3	8.4	5.5	6.6	17.4	10.6
2 yr	360 min	5.3	7.5	5.8	8.4	16.5	11.1
2 yr	540 min	6.7	7.7	6.6	10.5	15.5	11.5
2 yr	720 min	5.9	8.3	5.8	8.8	14.4	10.1
5 yr	30 min	3.1	13.9	6.1	4.9	19.7	9.4
5 yr	45 min	4.9	14.2	7.5	7.8	22.1	11.8
5 yr	60 min	6.2	15.7	8.4	9.8	23.1	13.3
5 yr	90 min	7.3	15.5	8.6	11.6	23.4	14.6
5 yr	120 min	8.0	16.3	9.0	12.7	23.7	15.2
5 yr	180 min	7.8	12.0	8.1	12.3	21.9	14.6
5 yr	270 min	7.6	11.9	8.5	12.3	22.1	10.6
5 yr	360 min	9.0	10.6	8.7	13.9	21.4	15.2
5 yr	540 min	9.9	10.9	9.9	15.0	19.9	15.2
5 yr	720 min	9.1	11.7	8.8	13.2	18.4	13.4
20 yr	30 min	6.7	19.7	10.2	10.4	24.1	13.6
20 yr	45 min	9.4	19.8	11.9	14.1	25.9	16.2
20 yr	60 min	11.3	21.6	12.9	16.8	27.0	17.9
20 yr	90 min	12.3	21.6	13.2	18.5	27.4	19.2
20 yr	120 min	13.2	22.5	13.7	19.8	27.7	19.7
20 yr	180 min	12.6	17.0	12.5	18.8	26.1	19.5
20 yr	270 min	12.8	16.8	12.9	19.1	26.0	19.5
20 yr	360 min	13.8	14.9	13.4	20.6	25.2	19.8
20 yr	540 min	13.9	14.9	13.9	20.6	24.0	19.5
20 yr	720 min	13.1	16.1	13.3	18.3	22.6	17.6
100 yr	30 min	12.6	26.9	14.6	17.9	28.2	18.3
100 yr	45 min	16.0	26.8	16.7	21.9	29.8	20.3
100 yr	60 min	18.2	29.1	18.0	23.7	30.8	21.6
100 yr	90 min	19.2	28.7	18.4	24.8	31.1	22.5
100 yr	120 min	20.0	29.6	18.8	25.6	31.3	23.1
100 yr	180 min	18.3	22.3	17.2	24.5	29.7	23.0
100 yr	270 min	18.8	22.2	17.3	24.4	29.4	22.6
100 yr	360 min	19.2	19.9	17.9	25.1	28.7	22.9
100 yr	540 min	18.2	19.1	17.7	24.5	27.2	22.4
100 yr	720 min	17.3	20.5	16.9	22.7	25.7	21.0

Appendix C - HEC-RAS output

HEC-RAS Output

Tributary 1

River Station *(m)*	Profile	Q Total *(m3/s)*	Min Ch El *(m AHD)*	W.S. Elev *(m AHD)*	E.G. Elev *(m AHD)*	E.G. Slope *(m/m)*	Frctn Slope *(m/m)*	Vel Chnl *(m/s)*	Flow Area *(m2)*	Top Width *(m)*	Froude # Chl
1128.25	100yr Flow	16.5	45.42	46.56	46.66	0.014	0.014	1.56	13	27.3	0.55
1174.8	100yr Flow	16.5	47	47.57	47.67	0.023	0.022	1.59	12	31.83	0.68
1192	100yr Flow	Bridge									
1211.78	100yr Flow	10.5	47.55	48.7	48.76	0.007	0.015	1.1	11	22.11	0.4
1236.85	100yr Flow	10.5	48.84	49.62	49.83	0.049	0.032	2.08	5	13.31	0.96
1306.75	100yr Flow	10.5	49	50.12	50.14	0.002	0.002	0.66	17	17.86	0.2
1376.25	100yr Flow	10.5	49.79	50.3	50.34	0.016	0.008	1.17	12	39.28	0.54
1446.26	100yr Flow	10.5	50.47	51.21	51.26	0.012	0.012	1.13	11	31.58	0.49
1531.76	100yr Flow	10.5	51.34	52.36	52.45	0.014	0.014	1.34	9	23.64	0.53
1567.24	100yr Flow	7.1	52.4	52.93	53.04	0.030	0.019	1.52	5	13.04	0.74
1578	100yr Flow	Culvert									
1588.61	100yr Flow	7.1	52.6	53.65	53.66	0.001		0.52	18	37.33	0.17
1603.41	100yr Flow	7.1	53.07	53.67	53.73	0.014	0.003	1.21	8	25.74	0.53
1657.52	100yr Flow	7.1	53.98	54.56	54.62	0.017	0.017	1.25	8	26.25	0.56
1718.72	100yr Flow	7.1	55	55.45	55.49	0.015	0.015	1.01	8	27.67	0.51
1773.63	100yr Flow	7.1	55.9	56.28	56.31	0.015	0.015	0.88	9	31.04	0.49
1804.43	100yr Flow	4.1	56.19	56.59	56.63	0.015	0.010	0.9	5	25.39	0.5
1819.64	100yr Flow	Culvert									
1834.84	100yr Flow	4.1	56.43	59.5	59.5	0.000		0.03	120	82	0.01
1979.07	100yr Flow	4.1	59.42	59.96	60.03	0.024	0.024	1.15	4	11.98	0.63
2065.46	100yr Flow	4.1	61.02	61.49	61.55	0.026	0.019	1.13	4	15.28	0.65
2206.78	100yr Flow	4.1	63.79	64.16	64.2	0.018	0.018	0.94	5	25.02	0.54

Tributary 2

River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
804.59	100yr Flow	36.9	36.2	37.46	37.24	37.48	0.004	0.81	62.44	136.47	0.29
850.03	100yr Flow	36.9	36.6	37.75	37.75	38.11	0.047	2.66	13.86	19.21	1
914.11	100yr Flow	36.9	37.3	39.04	38.45	39.14	0.008	1.39	26.46	25.18	0.43
966.89	100yr Flow	36.9	37.91	39.53		39.68	0.013	1.74	21.24	21.23	0.55
1019.51	100yr Flow	36.9	38.4	40.07		40.18	0.007	1.59	34.14	142.8	0.43
1057.88	100yr Flow	36.9	38.8	40.39	39.97	40.53	0.011	1.62	22.86	24.24	0.52
1079.46	100yr Flow	36.9	39.03	40.63		40.67	0.004	0.97	49.08	82.32	0.3
1113.77	100yr Flow	36.9	39.35	40.77		40.81	0.005	0.88	41.88	53.67	0.32
1239.65	100yr Flow	36.9	40.52	41.57		41.63	0.010	1.08	37.37	78.01	0.44
1278.01	100yr Flow	36.9	40.98	42.03		42.13	0.018	1.43	26.65	64.39	0.6
1318.69	100yr Flow	36.9	41.41	42.45		42.49	0.005	0.97	39.23	55.08	0.35
1426.11	100yr Flow	23.3	42.23	43.01	42.68	43.04	0.005	0.78	32.44	69.09	0.31
1548.56	100yr Flow	23.3	43.62	44.35	44.35	44.58	0.043	2.2	11.75	28.18	0.92
1654.95	100yr Flow	23.3	44.81	45.98	45.65	46.04	0.007	1.15	24.45	44.76	0.39
1766.72	100yr Flow	23.3	46.38	47.22		47.35	0.025	1.63	14.96	31.97	0.7
1861.16	100yr Flow	23.3	47.9	48.82		48.88	0.011	1.11	21.33	40.2	0.47
1954.75	100yr Flow	23.3	49.44	50.09		50.15	0.017	1.09	22.27	66.9	0.55
2005.62	100yr Flow	23.3	49.99	50.78		50.83	0.011	1.02	25.81	68.21	0.46
2028.4	100yr Flow	23.3	50.27	50.99		51.03	0.007	0.87	28.33	56.84	0.38
2061.18	100yr Flow	23.3	50.56	51.29		51.37	0.017	1.33	20.91	63.81	0.58
2144.87	100yr Flow	10.1	51.94	52.56		52.59	0.010	0.77	13.56	48.17	0.42
2248.15	100yr Flow	10.1	53.5	54.01		54.06	0.021	0.98	10.1	33.29	0.58
2336.8	100yr Flow	10.1	54.44	55.33		55.38	0.011	1.03	10.43	28.62	0.46
2414.53	100yr Flow	10.1	55.96	56.63		56.71	0.029	1.26	7.99	23.67	0.69
2440.94	100yr Flow	10.1	56.71	57.31		57.37	0.022	1.1	9.52	32.14	0.61
2464		Culvert									
2464.97	100yr Flow	10.1	57.37	60.02	57.86	60.02	0.000	0.09	132.48	89.55	0.02
2480.58	100yr Flow	10.1	57.76	60.02		60.02	0.000	0.12	105.19	77.48	0.03
2739.43	100yr Flow	3.1	62.18	62.75	62.56	62.77	0.009	0.61	5.05	17.79	0.37
2808.27	100yr Flow	3.1	63.77	64.12	64.12	64.22	0.074	1.39	2.23	11.55	1.01
2887.09	100yr Flow	3.1	65.84	66.42	66.27	66.46	0.015	0.82	3.77	12.95	0.49
2951.28	100yr Flow	3.1	67.82	68.26	68.26	68.35	0.073	1.38	2.25	11.81	1.01

FIGURE 1

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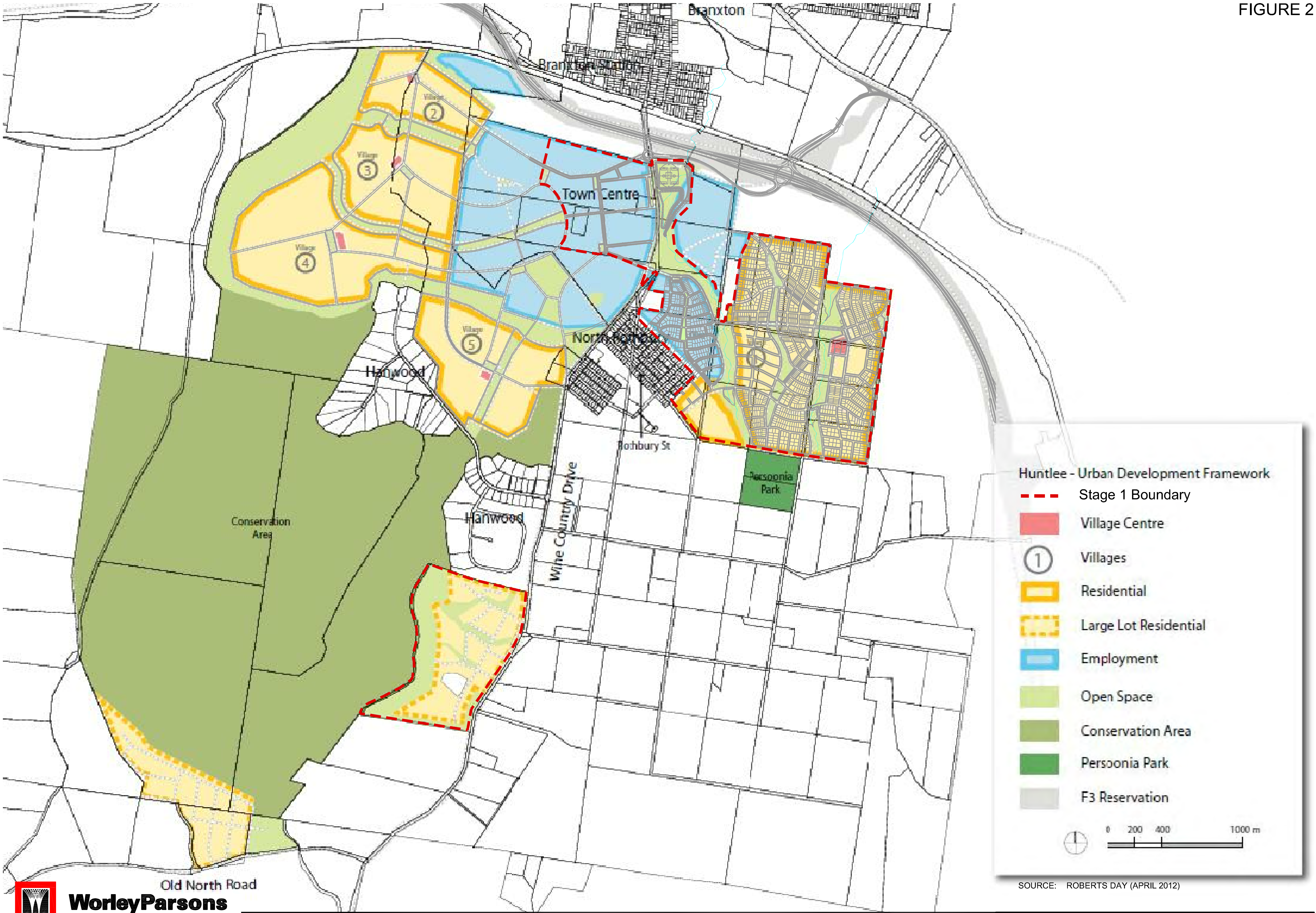
N:\301017\00364 Huntlee PA Village 1\CAD\TRUNK STORMWATER\FIGURE 1 – LOCALITY.dwg 16/12/2010

SITE LOCALITY PLAN



SOURCE: ROBERTS DAY (APRIL 2012)

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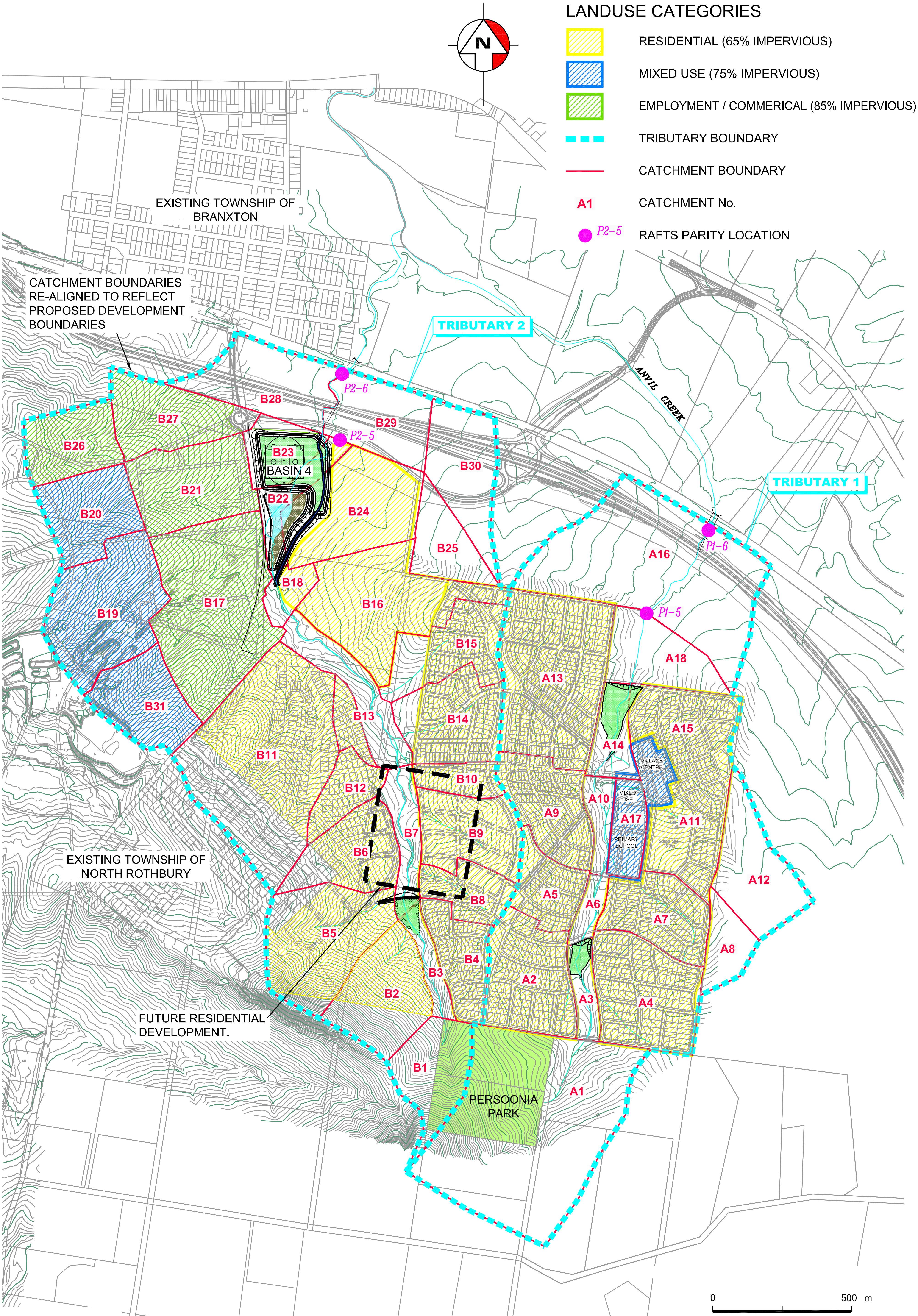
HUNTLEE MASTER PLAN

N:\301017\00364 Huntlee PA Village 1\CAD\TRUNK STORMWATER\FIGURE 2 - MASTER PLAN.dwg 21/09/2012

FIGURE 3



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DEVELOPED STATE
CATCHMENT PLAN

N:\301017\00364 Huntlee PA Village 1\CAD\TRUNK STORMWATER\FIGURE 3 - CATCHMENT PLAN.dwg 21/09/2012