Claymore Urban Renewal Project

Water Cycle Report

Contents

1. Introduction

1.1. Scope of Work

The Claymore Urban Renewal Project (CURP) proposes to rejuvenate the existing NSW Department of Housing (DOH) Claymore Housing Estate by creating a new integrated community. The project will provide approximately 1,490 dwellings (including 100 seniors living units) to both new and existing residents, providing a safer and more aesthetically pleasing environment for the community. The redevelopment also creates an opportunity to assess and improve the water cycle management structures of the existing brownfield site.

This report details the procedures used and results obtained from analyses undertaken in developing the water cycle management to support the master planning Development Application (DA) for the CURP.

The purpose of the investigation is to:

- undertake a hydrologic, hydraulic and water quality assessment of the stormwater discharged from the site to demonstrate compliance with statutory requirements;
- identify appropriate measures to achieve the water quality and quantity statutory requirements and determine their location and land area required to implement the recommendations; and
- identify existing localised flood ‘hot spots’ and provide recommendations to rectify the situation.

The following analyses have taken into consideration the economical, engineering, environmental and social aspects of the works. Particular emphasis has been placed on protecting the environment and enhancing the biodiversity of the receiving water bodies and environment by implementing water sensitive urban design and best management practices.

2. The Physical Environment

2.1. The Site

The Claymore Estate area, which was built during the period 1979-1981, is a 125 hectare public housing estate located in the Campbelltown LGA, approximately 2km northwest of the Campbelltown CBD. The estate adjoins Eagle Vale to the north and west, the Hume Highway to the east, and Badgally Road to the south. The development area consists of approximately 1,100 cottages and townhouses that are part of a typical Radburn style subdivision layout.

The subject site is bisected by an overland flowpath which starts at a headwall from Drysdale Street and meanders west-east through the northern part of the development area. The flowpath then turns to the north-east of the site before eventually connecting flows beneath the Hume Highway via a large piped headwall (4 x 1800dia pipes). The flowpath grades at approximately 1% and includes a series of grassed detention basins which range in size from 6,000-16,000m^3. For the purpose of this report, we shall refer to this overland flowpath as the channel/ basin system. The following comments are also provided:

- A low flow piped system exists within the channel/ basin system. This piped system is typically 600dia but does increase to a 900dia at the north-east corner.
- Existing detention basins appear to have been previously designed and constructed to include staged storage. Here, piped outlets at embankment weirs typically increase to 1650dia to allow surcharge at downstream positions (refer section 5 and 6 for discussion).
- There are a series of existing stormwater piped outlets and flowpaths which convey surface flows from surrounding residential areas to the existing basins.

Throughout the estate there are also a number of parks and reserves; these areas together with the extensive watercourse provide approximately 29Ha of open space (Refer to Appendix A for the existing estate layout).

Figure 2.1 – Existing Site Layout

The overall development area can be split into 4 sub-catchments namely, (A) Western; (B) Central; (C) Eastern; and (D) Northern. Each of these sub-catchments is defined by existing crests and the above mentioned flowpath. Natural crests will be maintained as much as possible during proposed development in order to utilise existing basins and avoid significant earthworks (refer figures W02 and W04).

Figure 2.2 – Existing Catchment Plan

Northern Catchment – The natural crest to the north of the channel/ basin system runs generally along Emerald Drive, Alabaster Place and Tourmaline Street with catchment areas being residential. Most of these catchment areas remain unchanged from existing (external to development area) with piped discharges to basins being maintained.

Western Catchment – The western portion of the overall catchment area is a combination of existing open grassed areas, residential and schools (2). The majority of the catchment remains unchanged from the existing (external to development area).

Existing flowpaths and piped systems convey surface flows towards the channel/ basin system. The two primary connection points include:

(a) Through existing school and enter system at node N7.0 via an easement at corner of Crozier St; and
(b) At sag in Drysdale St via two existing 1800dia and 1350dia pipes (which appear to be the trunk stormwater lines for the upstream catchment), while an additional 600dia pipe allows flows to enter from the northern approach of Crozier Street.

Central Catchment – The existing central sub-catchment is brownfield and will be redeveloped as part of the proposed works. The sub-catchment is divided by 2 existing ridgelines and drains via numerous piped and overland flowpaths to the channel/ basin system (900dia, 1200dai etc.). The catchment is typically at 3-6%. The proposed development will include earthworks and regrading to redesign these flowpaths via new road network and connect to channel/ basin system through the water quality treatment train.

Eastern Catchment – The exiting eastern sub-catchment typically grades to the channel/ basin system via a trunk pipe and overland flowpath (N2.3-N2.1). This system includes 1450dia into 2 x 1650dia before discharging to channel/ basin system via 4 x 1800dia at a large outlet structure.

2.2. Data

2.2.1. Topography and Geology

Existing geological maps outline the CURP area to consist of Blacktown soils over weathered Ashfield shale of Triassic age, with isolated pockets of Hawkesbury Sandstone within the creek valleys. Typical characteristics of the soils include low fertility, tendency towards strongly acidic properties and prone to shrinkage and swelling.

Test pits through the detention basin system range in depth from 2.0m – 3.8m, all of which returned dry samples. This indicates that the water table lies at a depth of at least 2.0m but could be beyond 3.8m. Excavation of the proposed basin is expected to be no greater than 1.5m and will therefore not effect groundwater.

Additional on-site geotechnical investigations are being undertaken by others to confirm the site-specific geology.

Topographic information for the catchments was obtained from detailed survey data provided by Vince Morgan Surveyors.

2.2.2. Proposed Layout

The proposed road (including cross sections), lot and open spaces layout have been taken from the current proposed master plan documentation.

2.2.3. Rainfall Data

2.2.2.1. Rainfall Records

The water quality analysis requires historical rainfall data recorded, by a pluviograph station. The Lucas Heights pluviograph recording station has been used and is situated approximately 15km from the development site. Historical rainfall records for the area were obtained from the Bureau of Meteorology from the following station:

Station No.	Location	Records	Data Interval
066078	Lucas Heights ANSTO	Aug 1984 – Aug 1993	Daily

2.2.2.2. Intensity-Frequency-Duration (IFD)

The design IFD data for the site was obtained from Bureau of Meteorology Coefficients for Campbelltown listed in Council’s Engineering Design Guide for Development (2009). Probable Maximum Precipitation (PMP) was derived using the Bureau of Meteorology’s Generalised Short Duration Method (2003).

A summary of the rainfall intensities derived is shown in

Table 2.1 below.

Table 2.1 - Claymore Rainfall Intensities (mm/hr)

Storm Duration (minutes)	Annual Recurrence Interval (years)			
	0.25 (3-month)	5	20	100
10	34.7	103	132	171
15	29.1	86.3	111	143
20	25.4	75.4	96.9	125
25	22.7	67.4	86.7	112
30	20.7	61.3	78.9	102
45	16.5	49.0	63.1	81.4
60	13.9	41.4	53.3	68.9
90	10.8	32.2	41.6	53.9
120	9.0	26.8	34.7	45.0
180	6.8	20.6	26.7	34.7
270	5.2	15.7	20.5	26.7
360	4.3	13.0	17.0	22.2
720	2.7	8.3	10.9	14.3

2.2.4. Existing Utility Services

Existing utility service locations were derived from service utility plans and site survey information for gas, electricity, sewer, stormwater, telecommunications and water.

3. Design Controls

3.1. Australian Rainfall and Runoff – Volume 1 (2001)

Prepared by the Institution of Engineers, Australia Australian Rainfall and Runoff – *A Guide to Flood Estimation* was written to "provide Australian designers with the best available information on design flood estimation". It contains procedures for estimating stormwater runoff for a range of catchments and rainfall events and design methods for urban stormwater drainage systems.

According to the document, good water management master planning should take into account:

- hydrological and hydraulic processes;
- land capabilities;
- present and future land uses;
- public attitudes and concerns;
- environmental matters;
- costs and finances; and
- legal obligations and other aspects.

3.2. NSW Floodplain Development Manual (April 2005)

The NSW Government's *Floodplain Development Manual – the Management of Flood Liable Land (2005)* is concerned with the management of the consequences of flooding as they relate to the human occupation of urban and rural developments. The manual outlines the floodplain risk management process and assigns roles and responsibilities for the various stakeholders.

The manual applies to the development, in particular in Appendix L – *Hydraulic and Hazard Categorisation* for ensuring safe overland flow paths are provided (see Figure L1 below).

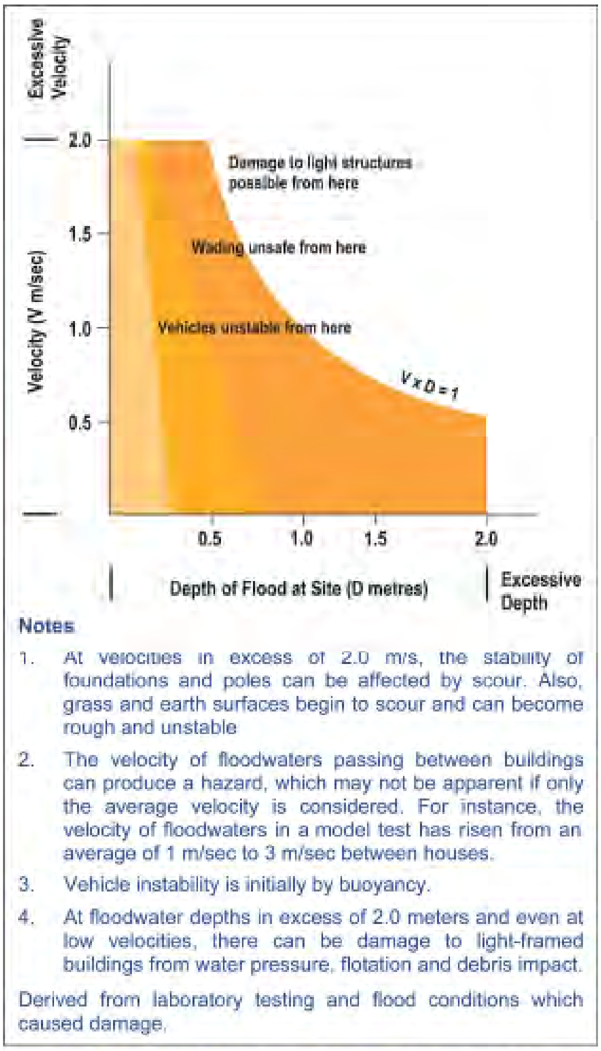

FIGURE L1 - Velocity & Depth Relationships

Figure 3.1 – Velocity Depth Relationships, FDM

Source*: NSW Floodplain Development Manual, 2004 (*Dept. of Infrastructure planning & Natural Resources*)*

3.3. NSW Department of Environment and Climate Change

The NSW Department of Environment and Climate Change (DECC), formerly the NSW Environment Protection Authority (EPA) has developed a set of guidelines known as the Managing Urban Stormwater (MUS) series. The set of guidelines includes:

- Managing Urban Stormwater: Council Handbook
- Environmental targets
- Managing Urban Stormwater: Source Control
- Managing Urban Stormwater: Soils & Construction
- Managing Urban Stormwater: Harvesting and Reuse

3.3.1. Managing urban stormwater: Environmental Targets

The NSW Department of Environment and Climate Change (DECC) encourages the principle of no net deterioration of water quality. Under its former name, the NSWEPA, the DECC published Managing Urban Stormwater: Environmental Targets, outlining recommended environmental targets for stormwater management in new urban developments. Among its recommendations are the following stormwater treatment objectives:

Table 3.1 – Stormwater Treatment Objectives for New Urban Areas from the Managing Urban Stormwater: Environmental Targets

Pollutant	Treatment Objective	Adopted Campbelltown City Council Treatment Objective
Gross Pollutant	90% retention of the annual average load for particles 0.5mm or less	95% retention of the annual average load for particles 0.5mm or less
Suspended Solids	85% retention of the annual average load	80% retention of the annual average load
Total Phosphorous	65% retention of the annual average load	45% retention of the annual average load
Total Nitrogen	45% retention of the annual average load	45% retention of the annual average load

The Campbelltown City Council Treatment objectives have been adopted for this report as they are specific for the Campbelltown City Council Area. These rates are also consistent with Australian Rainfall Quality (ARQ, 2006). Refer to section 7 for a detailed description of the analysis.

3.3.2. Managing urban stormwater: Source Control

The DECC guide, Managing Urban Stormwater: Source Control recommends the control of stormwater pollution at the source, rather than more traditional "end of line" systems that are unsightly and require high levels of ongoing maintenance. In this document, Water Sensitive Urban Design (WSUD) is described as "minimising the impacts of development on the total water cycle and maximising the multiple benefits of a stormwater system". It lists the main objectives of WSUD as:

- preservation of existing topographic and natural features;
- protection of surface water and groundwater sources;
- integration of public open space with stormwater drainage corridors, maximising public access; and
- passive recreational activities and visual amenity.

The broad principles of WSUD are listed as:

- minimising impervious area;
- minimising use of formal drainage systems (e.g. pipes);
- encouraging infiltration (where appropriate); and
- encouraging stormwater re-use.

3.3.3. Managing urban stormwater: Soils and Construction

Managing Urban Stormwater – Soils and Construction (4th edition, March 2004) are guidelines produced by the NSW Department of Housing to help mitigate the impacts of land disturbance activities on landforms and receiving waters by focusing on the removal of suspended solids in stormwater runoff from construction sites.

According to the guide, effective soil and water management during construction involves the following key principles:

- assess the soil and water implications of development at the subdivision or site planning stage (including salinity and acid sulphate soils);
- plan for erosion and sediment control concurrently with engineering design and before the land disturbance begins;
- minimise the area of soil disturbed;
- conserve topsoil for subsequent rehabilitation/revegetation;
- control surface runoff from upstream areas, as well as through the development site;
- rehabilitate disturbed lands as quickly as possible; and
- maintain soil and water management measures appropriately during, and after the construction phase until the disturbed land is fully stabilised.

3.4. WSROC Salinity Code of Practice

The Western Sydney Salinity Code of Practice was produced by the Western Sydney Regional Organisation of Councils (WSROC) to provide information on the current and best management practice for salinity management in the Western Sydney region. The document illustrates the methods used for assessing the salinity risk, recommended investigation methods and best management practices for managing salinity.

The guide lists the following key principles for salinity management:

- maintain natural water balance;
- maintain good drainage;
- avoid disturbance or exposure of sensitive soils;
- retain or increase vegetation in strategic areas; and
- implement building controls and/or engineering responses where appropriate.

3.5. BASIX

A water re-use assessment under the Building and Sustainability Index (BASIX) is outside of the scope of this report. However for water quality analysis a preliminary assessment has been undertaken to determine the approximate required rainwater tank volume per dwelling to achieve the minimum mandatory 40% efficiency rating for new dwellings (refer to section 7.3.1 for more details).

3.6. LANDCOM: Water Sensitive Urban Design Policy, DRAFT

Landcom's Water Sensitive Urban Design Policy has been prepared by Landcom to make sustainability objectives a key component of their many developments. The document provides targets and objectives for urban environments in regards to water conservation, pollution control and mitigation. The following table outlines Landcom's Target Objectives for new Greenfield developments:

Table 3.2 – LANDCOM WSUD Objectives

Objective		Baseline and Performance Target	Stretch Target
1	**WSUD Strategy**	(a) 100% of projects to have project-specific WSUD strategies.	
2	**Water Conservation**	Combination of water efficiency and reuse options – % reduction on base case.	
		(a) Single dwelling, no reticulated supply available: Baseline 40 % Performance 50+ % Stretch 65 %	
		(b) Single dwelling, reticulated supply available: Baseline 50 % Performance 65 % Stretch 75+ %	
		(c) Apartment, no reticulated supply available: Baseline 40 % Performance 50 % Stretch 60+ %	
3	**Pollution Control**	(a) 45% reduction in the mean annual load of Total Nitrogen (TN). (b) 65% reduction in the mean annual load of Total Phosphorus (TP). (c) 85% reduction in the mean annual load of Total Suspended Solids (TSS).	(a) 65% reduction in the mean annual load of Total Nitrogen (TN). (b) 85% reduction in the mean annual load of Total Phosphorus (TP). (c) 90% reduction in the mean annual load of Total Suspended Solids (TSS).
4	**Flow Management**	Maintain 1.5 year ARI peak discharge to pre-development magnitude Stream Erosion Index = 2.0	Maintain 1.5 year ARI peak discharge to pre-development magnitude Stream Erosion Index = 1.0

A combination of the above targets and mandatory guidelines from the relevant authorities has been used in this report and discussed in their relevant sections.

3.7. Campbelltown (Sustainable City) Development Control Plan 2009

An integral part of the master planning process for the CURP, the *Campbelltown (Sustainable City) DCP 2009* provides the necessary controls for the redevelopment of the site. Particular water management requirements include:

- compliance with Council's Engineering Design Guide for Development;
- compliance with the demands of the BASIX system; and
- adoption of the principles of WSUD (including a water cycle management plan).

3.8. Campbelltown City Council Engineering Design Guide for Development (2009)

Council's *Engineering Design Guide for Development*, which forms part of the larger *Campbelltown (Sustainable City) Development Control Plan (2009)*, sets out their requirements for the design of stormwater drainage for urban and rural areas. The Engineering Design Guide outlines the broad objectives of the policy of:

- retention of the natural stormwater system where possible;
- a high level of safety for all users;
- acceptable levels of amenity and protection from the impact of flooding;
- consideration given to the effect of floods greater than the design flood;
- a controlled rate of discharge to reduce downstream flooding impacts;
- protection of the environment from adverse impacts as a result of the development;
- maintenance of and enhancement of the regional water quality;
- sustainability of infrastructure; and
- economy of construction and maintenance.

The policy also provides detailed requirements for the hydrologic and hydraulic design and analyses of the proposed water management system including standard calculation factors and drawings.

4. Water Management Options

4.1. Water Quantity Management

4.1.1. Major/Minor Drainage System

The major/minor approach to street drainage is the recognised drainage concept for urban catchments within the Campbelltown City Council local government area.

“The minor system is the gutter and pipe network capable of carrying runoff from minor storms. The major system comprises the many planned and unplanned drainage routes which convey runoff from major storm to trunk drains, sometimes causing damage along the way.” [1] *Australian Rainfall and Runoff, 2001*. The major system also exists to cater for minor system failures.

The overall aim of the major/minor approach is to ensure that hazardous situations do not arise on streets and footpaths, and that all buildings in urban areas are protected against floodwaters.”[1]

4.1.2. Detention Basins

Detention basins temporarily detain stormwater runoff from urbanised catchments with the aim of reducing and attenuating the peak discharge at the outlet to reduce the risk of flooding to downstream lands as a result of a development. The storage volume may be above or below ground while discharges are accurately controlled via an orifice or throttled outlet pipe.

4.1.3. Rainwater Tanks

Rainwater tanks are sealed tanks designed to retain rainwater collected from roofs for subsequent re-use for toilet flushing, laundry or garden watering. Due to the uncertain nature of the rainwater supply, tanks are usually connected to mains water for “top-ups” in dry weather conditions.

4.2. Water Quality Management

4.2.1. Infiltration Devices

Consisting of a gravel bed and usually greater than 600mm depth, an infiltration device primarily removes sediments and attached pollutants (including nutrients, metals and other soluble pollutants) by filtration. They may be installed as conventional below ground trenches backfilled with filter media or beneath permeable paving and are designed to capture and treat the “first flush” volume of a rainfall event.

4.2.2. Gross Pollutant Traps

“Gross Pollutant Trap” is a term applied to either in-situ, or proprietary units that remove litter, vegetative matter and sediment. Although the numerous units fall under the under the one umbrella of gross pollutant traps, the actual mechanics of the different units vary, as do the achievable pollutant removal rates. GPTs come in a range of sizes, with the larger units able to effectively treat large catchment areas and high flow rates. They are usually sized based on

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[1] Australian Rainfall and Runoff 2001

their maximum treatable flow being equal to, or greater than the 3-month Annual Recurrence Interval (ARI) storm event (typically 50% of the 1-year ARI storm event) of the upstream catchment.

Table 4.1 – Typical Pollution Removal Rates of Water Quality Treatment Devices

Device	Coarse Sediment	Fine Sediment	Free Oil & Grease	Nutrients	Metals
Infiltration Devices*	50-80%	30-50%	30-50%	30-50%	30-50%
Bio-Retention Systems*	80-100%	30-50%	30-50%	30-50%	30-50%
Vegetative Filter Strips	50-80%	10-50%	10-50%	10-50%	10-50%
Pit Inserts	80-100%	40-60%	40-60%	40-60%	40-60%
Gross Pollutant Traps	60-90%	10-50%	-	10-50%	10-50%

* Assumes pre-treatment of stormwater runoff to remove gross pollutants and to minimise ongoing maintenance.

Source: WSUD – Technical Guidelines for Western Sydney (2004)

5. Water Quantity Modelling

The assessment of water quantity was completed using both hydrological and hydraulic modelling. Here, computer based models of the existing and proposed catchments were constructed using XP-RAFTS. Design storms were applied to these models to give estimates of the 100-year ARI discharges, which are examined in the following sections. Assessment of these models then allowed the determination of basin sizes and requirements. Assessment was also undertaken on the existing basin sizes.

As an overall check, the existing 100-year ARI results from XP-RAFTS (at the outlet) were then compared with empirical estimation techniques (Rational Method) as recommended by the Australian Rainfall and Runoff (I.E Aust, 2001).

Computer based, one-dimensional, steady flow hydraulic models were then constructed to represent both the proposed and the existing networks using HEC-RAS. The 100-year ARI discharges from the hydraulic model were then input into the hydraulic model to determine the respective flood levels and extent.

The probable maximum precipitation (PMP) was estimated using the Bureau of Meteorology's Generalised Short Duration Method (2003). Using a similar process to the 100-year, probable maximum flows (PMF) were estimated in XP-RAFTS, with flood levels and extent determined in HEC-RAS.

5.1. Model Parameters

The user data inputs required by XP-RAFTS include catchment areas and slopes, pervious and impervious areas, IFD rainfall statistics and hydrological losses. Guidelines for determining these parameters are provided in the Australian Rainfall and Runoff (I.E Aust, 2001) and are broken up as follows:

5.1.1. Slopes

In accordance with AR&R (I.E Aust, 2001), the slopes of the sub-catchments were generated using "equal area" method. The slopes for each of the catchments are listed in Tables B.1 to B.6 in Appendix B.

Proposed sub-catchment slopes for links and catchments were derived from the proposed master plan layout and grading (as of May 2011), while the existing slopes were developed from aerial contours.

5.1.2. Impervious and Catchment areas

The extent of impervious area within the existing catchment was digitally measured from aerial imagery. Impervious and catchment areas for each of the sub-catchments are included in Tables B.1 to B.6 in Appendix B

Similarly, the impervious areas within the proposed catchments were based upon the master plan density sketches supplied by Landcom.

Fraction impervious values for the proposed development were based on those values mentioned in Table 4.2 Campbelltown City Council Guidelines (CCC, 2009). Here footprint areas were digitally measured from the master plan layout and assigned the below mentioned values. An average of 75% impervious has been assigned to residential due to the proposed variances in lot sizes across the site. (Typically range from 450m^2-700m^2).

Table 5.1 – Percentage Impervious Areas for Various Land Uses

Land Use	Adopted % Impervious
Roads and Industrial Areas	75%
Commercial/ Industrial	100%
Residential Housing	75%
Parks/Grass Land	10%

Source: Derived from UPRCT estimates and CCC, 2009

5.1.3. Intensity-Frequency-Duration (IFD)

Rainfall intensities were used as described in Section 2.2.3

5.1.4. Rainfall Losses

The loss model adopted to estimate rainfall excess in the development of design flow hydrographs was the Initial Loss-Continuing Loss model.

As specified in Council's Engineering Design Guide for Development (CCC, 2009), the following initial – continual loss parameters were utilised within the model:

- an Initial Loss of 15mm and a Continuing Loss of 2.5mm/hr for pervious areas; and
- an Initial Loss of 1.5mm and Continuing Loss of 0mm/hr for impervious areas.

5.1.5. Land Use

The land use within the existing catchments is considered to be predominantly urban. This type of land use does have some effect on the runoff by providing some "resistance" to the flow. The effect is simulated in XP-RAFTS by a storage delay coefficient called "PERN". The following typical values are in accordance with Council requirements:

Table 5.2 – Adopted PERN 'n' values

Catchment Type	PERN 'n'
Developed (Impervious Portion)	0.015
Developed (Pervious Portion)	0.025 – 0.03

Source: Engineering Design Guide for Development (2009)

5.1.6. Hydraulic Roughness Parameters

Hydraulic roughness parameters for the overland flow paths were estimated based upon site visits and were applied in accordance with those recommended in AR&R. A Manning's roughness parameter of 0.035 was applied for all grassed areas (including verges) while 0.013 was applied for all road pavements. These also satisfy parameters set out in the Council guidelines.

5.1.7. B-Multiplier

The b-multiplier (b) used in RAFTS is usually determined by calibration against recorded floods. As discussed in Section 5.1.8, the value for b is then used in the standard equation S=bQn. Council Guidelines (Engineering Design Guide for Development) specify a b-multiplier of 1.0. This value was subsequently used in this study.

5.1.8. *RAFTS* Catchments

Hydrologic modelling was carried out using the XP-RAFTS software package (Version 6.5, XP Software, 2001). RAFTS is a non-linear runoff routing model that generates runoff hydrographs from rainfall.

A catchment is divided into a network of sub-catchments joined by links. The links represent natural watercourses, artificial channels, or pipes. The model divides each sub-catchment into 10 sub-areas. A sub-area is treated as a cascading non-linear storage governed by the relationship S=bQn. The coefficient ‘b’ is calculated from catchment parameters but can be calibrated to fit observed rainfall and streamflow data.

Rainfall is applied to each sub-area. Losses (representing infiltration, interception, etc) are subtracted from the rainfall and the excess is then converted into an instantaneous flow. This instantaneous flow is then routed through the sub-area storages to develop local sub-catchment hydrographs. Total flow hydrographs at various nodes in the drainage network are calculated by combining local hydrographs. Hydrographs are transported through the drainage network by time lagging or channel routing. Hydrographs may also be routed through the storage basins such as dams or detention basins.

5.1.9. Existing Catchment

As discussed in section 2.1, the existing overall catchment, was defined from aerial contours and is divided into four (4) sub-catchments namely (A) Western; (B) Central; (C) Eastern; and (D) Northern.

The proposed development site is 125 hectares, but also has contributing upstream areas, which give a total catchment size of approximately 231 hectares.

As described in Section 2.1, the pre-developed site has a constructed channel/ basin system which travels from west to east and conveys runoff from each of the 4 sub-catchments towards north-east before discharging under the Hume Highway. This watercourse consists of a series of detention basins which have been previously designed and constructed by others and will remain in the post-development scenario. Modelling in Section 5.1.10 & 5.3 will determine whether additional detention volumes and/ or control measures are required to ensure the overall discharge from the post development scenario does not exceed the overall site discharges in the existing scenario.

Each of the 4 catchments have further been divided into 32 sub catchments. These sub catchments ranged in size from 1.09 to 18.59 hectares (Refer to Tables B.1 to B.6 in Appendix B). Each of these sub-catchments naturally adjoin the system at various points and eventually discharge via a large headwall (4 x 1800dia) beneath the Hume Highway.

Figure W03 in Appendix A shows the existing catchment divisions, while Figures B.1 to Figure B.6 in Appendix B represent the existing networks within RAFTS. The division of catchments was based upon the overland flow paths and existing road and drainage networks. Overland flow paths generally match those specified by council. Some consideration of the proposed catchments was taken into account when developing the node network.

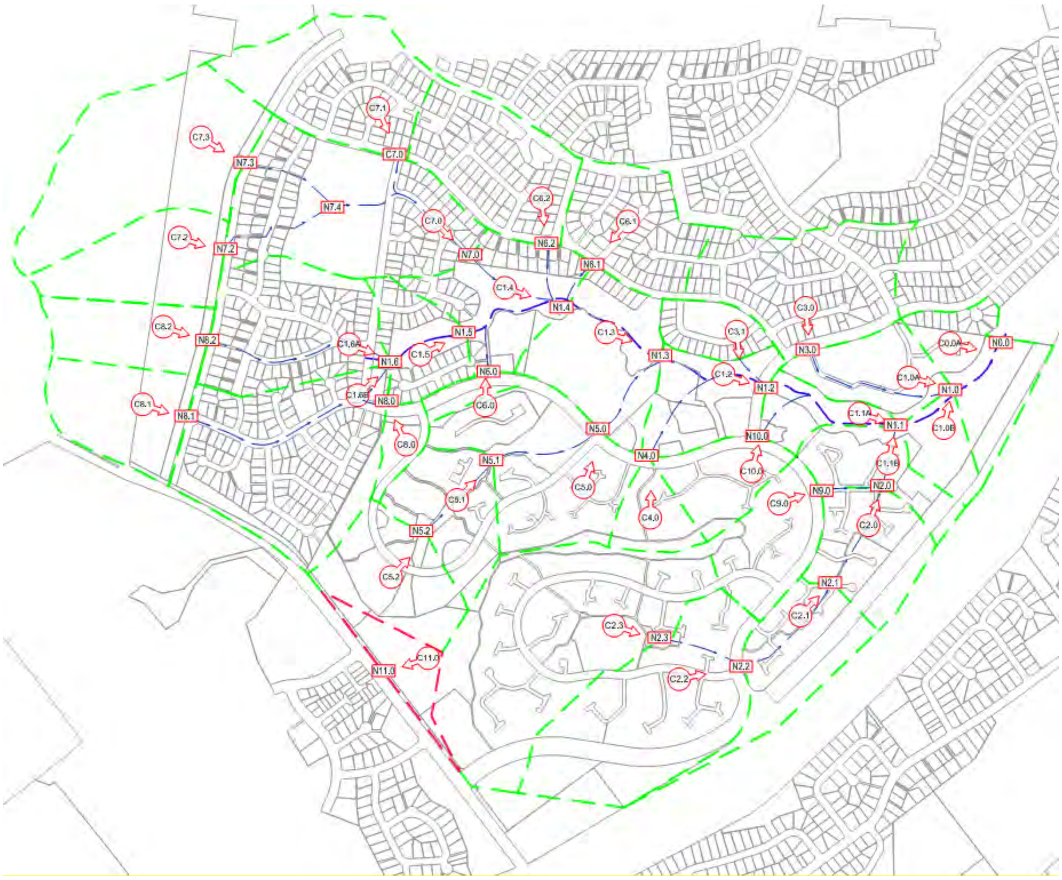

Figure 5.1 – Existing Catchment Plan

Site investigations have confirmed that the existing site has a combination of minor and major stormwater infrastructure in place to assist in conveyance of surface flows to their respective outlets.

The pre-developed RAFTS model was subsequently formulated by incorporating the following:

- “Catchment Nodes” were used to represent each of the 32 sub-catchments. Here, each node is representative of the catchment and is divided into both pervious and impervious values (refer Table in Appendix B);
- “Dummy Nodes” were used where two or more existing sub catchments joined, which allowed both inflow and outflow hydrographs to be assessed. Diversion links (with no lag time) were used to combine these inflow hydrographs; and
- Most of the links between nodes were modelled as channel routing links and are representative of the existing road profiles and low flow pipes. Sections were input from 12d as “HEC-2” and Manning’s ‘n’ values were estimated from site visits.

The following comments are also provided:

- The performance of existing detention basins was assessed within RAFTS. This included modelling of (a) the low level piped outlets; (b) high level overflow weirs; and (c) storage volumes.
- There is an existing flowpath from N2.3 to N1.1 which conveys peak flows from the eastern catchment towards the primary channel / basin system. This flowpath travels along both existing roadways and via a formalised reserve amongst houses (central

concrete path, trunk piped system, grassed overbanks to rear of properties). The trunk piped system includes a 1050dia pipe at the top of the catchment, which then becomes 2x 1650dia before discharging to the channel / basin system via a large concrete headwall structure and 4x 1800 dia.

Figure 5.2 – Existing 4 x 1800dia Culverts

- It is noted that localised ponding occurs directly in front of this outlet structure in the existing scenario for up to 100m. Evidence was found on site that there is a large amount of sedimentation within the existing pipes. Section 6.0 will discuss further for proposed regrading of the area.
- There is an existing sag within Drysdale St (Node N1.6) which collects all upstream catchments prior to discharge to the channel / basin system. A series of trunk pipes (1800dia, 600dia and 1350dia) convey flows from this sag to the basin. It is noted that there are no less than 5 large kerb inlet pits at the sag point in the road. Those flows which are not conveyed via piped system then travel overland between the houses.
- For the purposes of modelling the overland flow through these houses, a 50% blockage factor has been assumed on the piped system. The subsequent flowrate which is used in Section 6.0 is 3.6m^3/s as shown in Table 5.5.
- There is a small catchment area adjacent to Badgally Rd which bypasses the channel / basin system and drains to the South. This has been included in the overall site discharge via a bypass node N11.0.

Figure 5.3 – Existing Drysdale Rd Hazard Area

5.1.10. Proposed Catchment

Catchment division in the proposed scenario is similar to existing with 4 primary sub-catchments.

In developing the post-developed RAFTS models, the overall catchment was also further divided into 38 sub-catchments. These sub-catchments ranged in size from 1.09 to 14.22 hectares. Each sub-catchment was determined from the proposed master plan road layout and site grading, while consideration was given to retaining existing significant infrastructure wherever possible.

Figure W04 in Appendix A shows the sub-catchment division while Figures B.1 to B.6 in Appendix B illustrate the proposed RAFTS network.

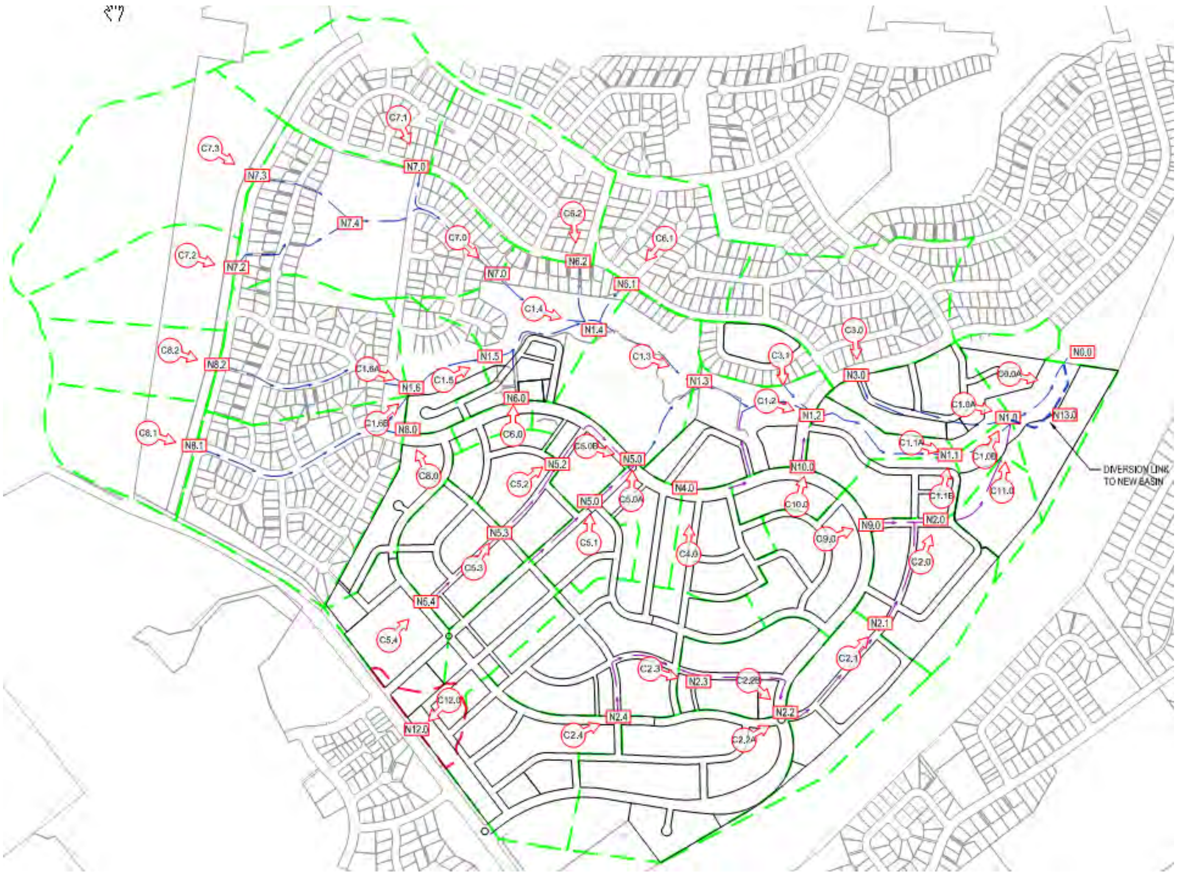

Figure 5.4 – Proposed Catchment Plan

As this project is primarily a brownfield development and includes upstream catchments which remain unchanged, some catchment areas have remained consistent with existing. This includes some road lines, impervious percentage, general catchment divisions, stormwater infrastructure and the like remaining. While redeveloped areas typically include modified flowpaths and increased housing density.

Most of the links were modelled as channel routing links and are representative of the road sections in the proposed master plan. Where considered practical, existing low flow pipes were maintained in the channel routing links while new pipes were also estimated / included along new roads where required.

Catchment areas, slopes and percentage impervious portions are tabulated in Appendix B.

The following comments are also provided:

- Catchment areas draining to existing Basins at Node N1.5 and N1.4 are typically unchanged from existing. Basin outlet configurations have consequently been maintained as per existing.
- Catchments directed towards the existing basins at N1.2 and N1.3 have typically been changed with increased impervious areas, piped systems and the like.
- The proposed basin configuration which has been included in the modelling includes an additional detention basin at the modified soccer field to the N-E (Node N13). This shall provide the additional storage volume which is required to satisfy statutory requirements. Refer Section 5.2 for discussion.

- The proposed basin has been modelled using a “diversion link” in order to allow a portion of flows into the basin. A staged storage outlet is then provided to allow flows to re-enter the channel system.

5.2. Management Strategies

5.2.1. Major/Minor System

The proposed drainage system will be major/minor system. The (minor) piped drainage system is to be designed to control nuisance flooding and enable effective stormwater management for the site. Council’s standard requires that the minor system be designed for a minimum 5 year ARI.

The major drainage system incorporates overland flow routes through proposed roads and has been assessed against the 100 year ARI design storm event, with general safety and flooding issues being addressed for events in excess of the 100 year ARI storm. If the major system cannot meet the safety and flooding criteria, the capacity of the minor system will need to be increased.

Inlet and culvert blockages were considered in the modelling with a 50% blockage factor across all culverts being applied in order to assess overland flow paths.

In order to assess the adequacy and safety of the major drainage system, channel routing links were used in RAFTS to model flow paths along roads and pathways, while lagging links were used elsewhere. Although negligible attenuation is expected along the roadways, channel routing was used in order to assess flow depths and velocities in major storm events. The channel cross-sections were based on the proposed road cross-sections in the master plan. Low flow pipes in channel links were based off 5year ARI and were assumed to operate at 100% during assessment. The proposed pipe drainage system may be designed with greater capacity than this if required. The capacity of the existing drainage system needs to be assessed at the detailed design stage.

5.2.2. Detention Basins

Detention Basins were included in the hydrologic model to ensure that there is no increase to peak flows exiting the overall development, which could potentially have adverse impacts on downstream properties.

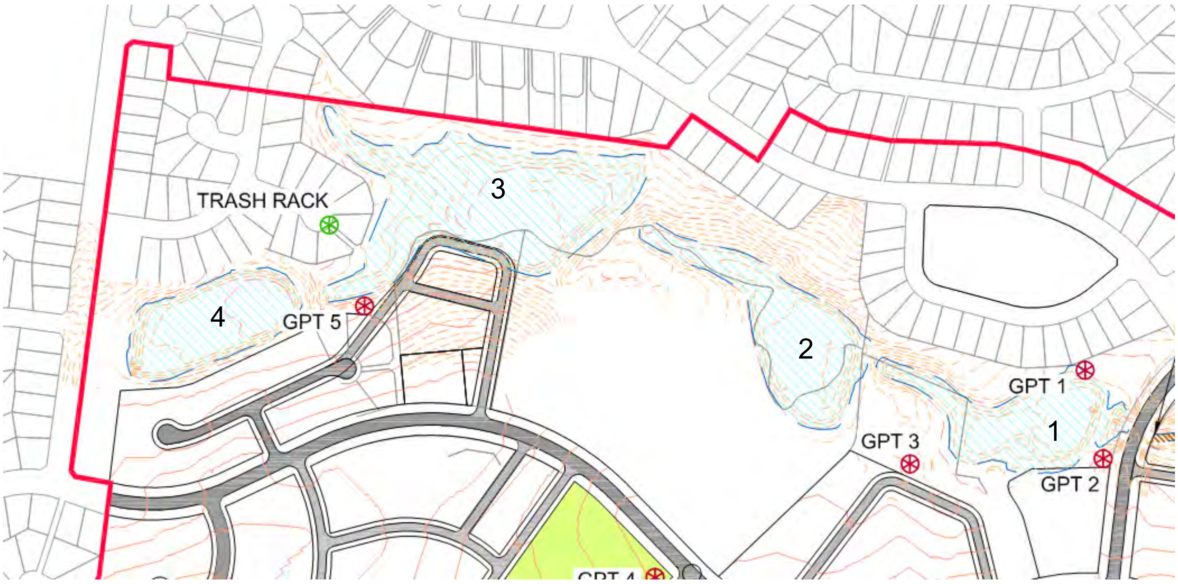

Figure 5.5 – Existing Detention Basins

Four (4) existing online detention basins were modelled along the length of channel using detailed survey information. Following an assessment of peak flowrates, an additional "offline" detention basin is proposed at Node N13.0 (at the existing modified soccer field) in order to decrease the peak flowrates generated from the proposed development.

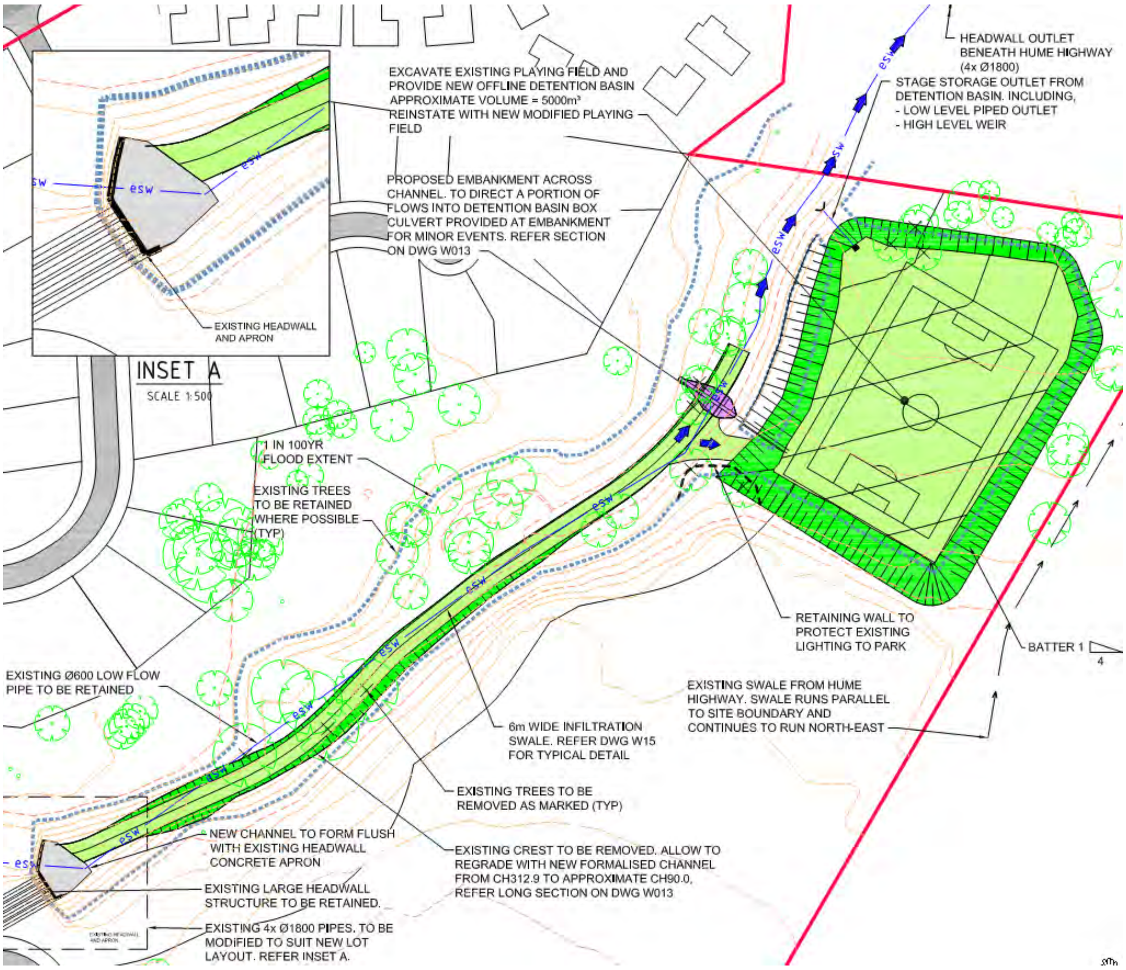

Figure 5.6 – Proposed Outlet Basin

The configuration of both the proposed control structure and basin storage is detailed on drawings W06 and W013 and includes the following:

- Basin storage;
    - Earthworks undertaken to lower the existing modified playing to approximate RL54.20 with minimum fall to outlet. Approximate volume of 5,000m^3 of storage has been provided to restrict post developed flow to existing;
    - Staged storage outlet with low flow pipe and high level control weir;
    - Maximum 1.2m depth of flood storage in accordance with recommendations by AR&R (I.E Aust, 2001); and
    - Construction of the proposed bas is proposed to be constructed as part of the stage 3A works, as increases in the impervious area are expected from the construction of this stage.

- Control weir / embankment installed within the existing channel, including:
    - Embankment formed to RL55.4 to ensure tailwater conditions are no longer imposed on the upstream 4x1800dia outlet structure at node N1.1;
    - Box culvert opening to be provided within the embankment in order to allow low flows to freely continue along channel system; and
    - Side entry weir (0.3m below embankment – RL55.10) to allow a portion of those flows which are above the 5 year minor event to enter the basin for controlled discharge.

All basins were modelled with a linear stage-storage relationship and used the default discharge equations within RAFTS. The design of the proposed basin initially incorporated the sizing of the piped outlet to satisfy the 5 year permissible discharge rate. A high level crested weir was then introduced within the basin at the 20 year ARI top water level to provide a secondary outlet and mirror the existing regime. The peak design flow (100yr ARI) is then discharged via a combination of the piped and weir outlets and conveyed along the watercourse downstream. It should be noted that more detailed modelling of the basins can be undertaken at the design stage when sufficient details are available to derive more accurate stage-storage and stage-discharge relationships.

The volumes required were refined by manual iteration until results showed that the total flows generated from the post-developed scenario did not exceed those in the pre-developed. The proposed basins and their volumes are shown in Table 5.3.

Table 5.3 – Modelled Detention Basins

Location (Catchment)	*RAFTS* Node	Volume (m^3)
Existing Basin 1	Node N1.5	6,000
Existing Basin 2	Node N1.4	12,420
Existing Basin 3	Node N1.3	13,850*
Proposed Basin 3*	Node N1.3	12,150
Existing Basin 4	Node N1.2	9,765
Proposed Basin 1	Node N13.0	5,000

*An extension of the existing Carter Place in the North West corner of the site is proposed. The extension works include creation of a new road segment extending over the existing 100 year ARI flood extent (refer drawing W09 for the existing flood extents). Refer section 4.3.2 for discussion.

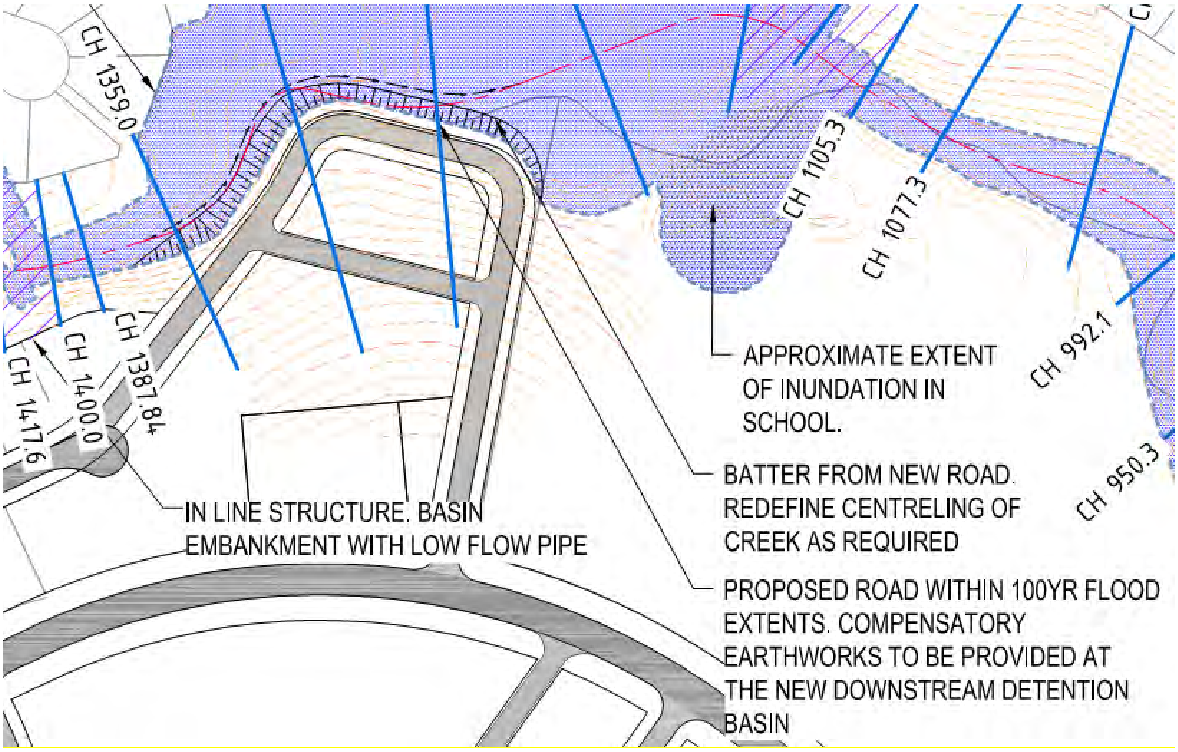

Figure 5.7 – Basin 3 Proposed Reduction

5.3. Results

5.3.1. Design Discharges

Design discharges were produced for a range of ARIs including the 5, 20, 50 and 100-year ARI events. Storm durations ranging from 10 minutes to 3 hours were modelled for each ARI, using AR&R temporal patterns, in order to identify the peak flow for each sub-catchment node. The design discharges for all of these events are shown in Appendix C.

5.3.2. Comparison of Post-developed and Existing Flows

The 100-year ARI flows for the post-development scenario are compared with existing conditions. From analysis, the critical storm duration for the 100 year ARI event at the outlet was 2 hours. However generally throughout the catchment the 1.5 hour duration storm is critical and has therefore been used for analysis in the HEC-RAS modelling. The shift to the critical 2 hour storm at the outlet is due to attenuation of flows caused by the proposed basin.

Comparative results are shown at various points in the site in Table 5.4 for the 100yr ARI event. Full results are included in Appendix C.

Table 5.4 - Comparison of Existing and Proposed 1 in 100 year ARI Flows

Existing		HEC-RAS Chainage (m)	Proposed	
Node	Flow (m^3/s)		Flow (m^3/s)	Node
N1.6	12.22	1628.9	12.19	N1.6
N1.5	13.01	1400	12.90	N1.5
N1.4	20.17	1105.3	20.95	N1.4
N1.3	17.45	874.4	19.47	N1.3
N1.2	13.69	641.6	15.27	N1.2
N1.1	24.89	312.9	25.95	N1.1
N1.0	30.58	100	32.55	N1.0
N0.0	31.09	9.63	28.55	N0.0

The basin sizes shown in Table 5.3 have been applied to the proposed models, subsequently allowing post-developed flows to be lower than existing at the outlets of the site.

The proposed peak discharge occurs during the 2 hour 100yr Storm event, the following is a comparison of the existing and proposed peak flow rates:

Table 5.5 - Comparison of Existing and Proposed 100 year ARI 120min Flows

Node	Existing	Proposed
N0.0	30.62m^3/s	29.76m^3/s
DUMMY (includes Bypass)	31.46m^3/s	30.10m^3/s

The basin 3 volume has been reduced in the proposed scenario due to the proposed extension of Carter Place in the North West corner of the site. The extension works include creation of a new road segment extending over the existing 100 year ARI flood extent (refer drawing W08 for the existing flood extents). In order to satisfy freeboard requirements the proposed road has been considered at a height greater than the 100yr flood level at this location (proposed flood level approximately RL69.07). The proposed road extends into the existing Basin 3 requiring minor earthworks to elevate the road, which will reduce the basin storage capacity by approximately 1,725m^3. It has been assumed that a batter at 1:4 will be provided from the edge of pavement to the basin floor. This will provide the worst case scenario for causing a reduction of storage in basin 3. This may be replaced with a retaining wall during the detailed design phase if required, which will improve storage within basin 3.

No additional compensatory earthworks within basin 3 have been proposed due to the preference to retain existing trees and open space in its current configuration. Minor regrading may be required to create a more defined basin invert.

As a result of the reduction in storage volume, a localised increase in flow rates throughout the main floodway in the order of 2m^3/s. HEC-RAS modelling of the proposed works has identified a proposed water level increase of approximately 40-80mm along the floodway in the vicinity of basin 3. The increase in flows has been offset with the proposed 5,000m^3 detention basin at the outlet of the site.

The results indicate that the proposed development will not have an adverse impact on downstream property as a result of increased flows. The flow rates shown in Table 5.4 are representative of stormwater runoff being carried in low flow pipes as well as those travelling along overland flow paths.

Hydraulic modelling is completed in Section 6 to assess the depth and extent of flood inundation in both the post developed and existing scenarios. For the purposes of this study, the low flows pipes are assumed to operate at full capacity (in both scenarios). Consequently the overland flow rates used in Section 6 are expressed in Table 5.5.

The following summary table includes all those flowrates which are generated from RAFTS. Figures are then used for HECRAS modelling within Section 6.0.

Table 5.5 - Existing and Proposed 100 year ARI Flows for HEC-RAS

HEC-RAS Chainage (m)	EXISTING (m^3/s)	*RAFTS* Node	HEC-RAS Chainage (m)	PROPOSED (m^3/s)	*RAFTS* Node
1628.9	3.6	N8.2	1628.9	3.6	N8.2
1575.9	12.218	N1.6	1575.9	12.192	N1.6
			1527.2	12.9	N1.5 (Inflow)
1400	4.69	N1.5 (Outflow)	1400	4.667	N1.5 (Outflow)
1252.5	20.166	N1.4 (Inflow)	1252.5	20.945	N1.4 (Inflow)
1105.3	14.306	N1.4 (Outflow)	1105.3	16.802	N1.4 (Outflow)
			992.1	19.87	N1.3 (Inflow)
874.4	13.653	N1.3 (Outflow)	874.4	16.348	N1.3 (Outflow)
			744.1	17.075	N1.2 (Inflow)
641.4	14.227	N1.2	641.6	16.963	N1.2 (Outflow)
312.9	24.472	N1.1	312.9	32.588	N1.1
100	30.205	N1.0	100	32.588	N1.0
9.63	31.09	N0.0	9.62	28.545	N0.0

5.3.3. Comparison with other Results

The hydrologic model results were compared with a more approximate method described below to check that they were within the expected range.

2.2.2.3. Rational Method

The rational method is the most widely used empirical technique used for calculating design flow rates within Australia (as recommended in AR&R87). The rational method calculates the peak flow rate corresponding to the particular time of concentration for the catchment. These estimated flow rates are not related to any one specific storm event.

The position of the estimated peak flow rate was chosen as the outlet point of the eastern catchment (at the outflow of the watercourse adjacent to the Hume Highway). The result was then compared to the corresponding RAFTS node (DUMMY) – with <u>all detention basins removed</u> - as shown below in Table 5.6.

Table 5.6: Comparison of Results

Point / Node	**Location**	**AR&R Rational Method**	***RAFTS***
C1.1	Catchment Outlet (Hume Hwy)	42.1m^3/s	51.8m^3/s

Comparison has shown that the flowrates are within 23% and subsequently within a reasonable order of magnitude.

The difference in flowrates is explained by the following:

- the rational method is based on the theory of a basic triangular hydrograph,
- while RAFTS allows for generation of a varying rainfall hyetograph and therefore a more parabolic hydrograph.
- rational method does not consider any effect of minor and major (trunk) piped systems – which allow for flows to be conveyed much quicker towards the outlet, which in turn increases the peak flowrate.
- RAFTS allows for flowrates to be more accurately modelled through flowpaths, and
- the rational method assumes the catchment is uniformly defined, while RAFTS allows for more accurate delineation of sub catchments, which in turn allows for portions of the catchment to peak at different times or simultaneously.

5.3.4. Probable Maximum Flood

The 45 minute duration PMP storm produced the highest discharges. Estimated PMF discharges are up to 10 times the 100 year ARI flows with a flowrate of approximately 270m^3/s for the 45 minute duration storm. Detention Basin spillways will be designed at detailed construction certificated stage to convey half the PMF discharge in accordance with Campbelltown City Council's standards.

While it is recognised that an appreciation of the impacts of the PMF must be considered during the development assessment stage the critical parameters are the provision for evacuation and the structural integrity and safety of houses or other buildings subjected to high depth and velocities. It is accepted that the SES regional evacuation plans would address any need for evacuation. The structural integrity of any building impacted upon by the PMF event, needs to be addressed as part of the development application for the individual buildings.

The simple channel routing in the RAFTS model is inadequate to accurately assess flood behaviour with such high flows. However, in order to assess the structural integrity of dwellings during this event, RAFTS software is expected to provide results within an acceptable order of magnitude.

5.3.5. Overland Flow Depths and Velocities

The 100-year ARI flow depths and velocities are tabulated in Table 6.7. These depths and velocities are based on the assumption that the minor, piped drainage system conveys 100% of the 5-year ARI. Table 6.7 also compares the depths and velocities against the following criteria:

- A velocity-depth product of 0.4 m^2/s as recommended in AR&R
- Figure L1 of the NSW Floodplain Development Manual

In instances where RAFTS presented zero data was for overland flow, pipes were then excluded and the model re-run to be conservative. These are highlighted in yellow in Table 6.7.

Generally the upstream links (and the majority of proposed road corridors) in the central catchment satisfy the limits set above. Typically links at the upstream end of the catchments easily satisfy the velocity-depth product limits, but as flows increase down catchments the product is increased and in some instances the safety limits are exceeded.

Some potential hazard areas are identified as follows:

- Link 1.0-1.6 convey trunk overland flows through the central channel / watercourse, and as such the corresponding depth-velocity ratings for these channels exceed the standard maximum safe 0.4 depth to velocity rating. Due to the volume of flow, this was anticipated and has been excluded from the overland flow analysis. Flows through the watercourse have been assessed in the flood modelling (HEC-RAS) component of this report;
- Link 5.0 consists of a 1200dia pipe currently flowing through the existing Claymore School. This pipe has the capacity to convey the 1 in 5 year ARI peak event however; a 50% blockage on this pipe generates additional overland flow which does not satisfy the velocity depth criteria. Additional pits are recommended along Dobell Rd opposite the school to increase the inlet capacity to ensure the 1200dia pipe generally runs at capacity.

- Links 7.0, 6.1 and 6.2 are existing flow paths for external sections of the site to the North and North-West. Analysis of the existing minor system in these areas indicate that the pipes are undersized and may require upsizing to accommodate some additional flows above the 1 in 5 year ARI event to alleviate the potentially high overland flows. Generally these catchments to the North and North-West remain unchanged in the proposed scenario and therefore there is no additional increase in flows. Detailed DRAINS analysis of these areas is to be undertaken as part of future stages;
- Links 2.0-2.2 currently have oversized pipes (Link 2.0 has 2 x 1650dia pipes) which convey the 1 in 100 year ARI event;
- Links P16.4 through to P16.2 (these links convey flows from an upstream catchment and appear to present a problem);
- Links 3.0, 2.4 & 2.3 require upsizing to accommodate rainfall events greater than the 1 in 5 year ARI storm event. This is to be considered at the detailed design stage.

To enable the overland flows to be reduced in the areas mentioned above, the piped drainage system will need to be designed to a higher ARI than the 1 in 5 year ARI for the flow paths. The required ARI for the pipe drainage system at these locations will be confirmed at the detailed design stage. Refer to Figure W08 for positions.

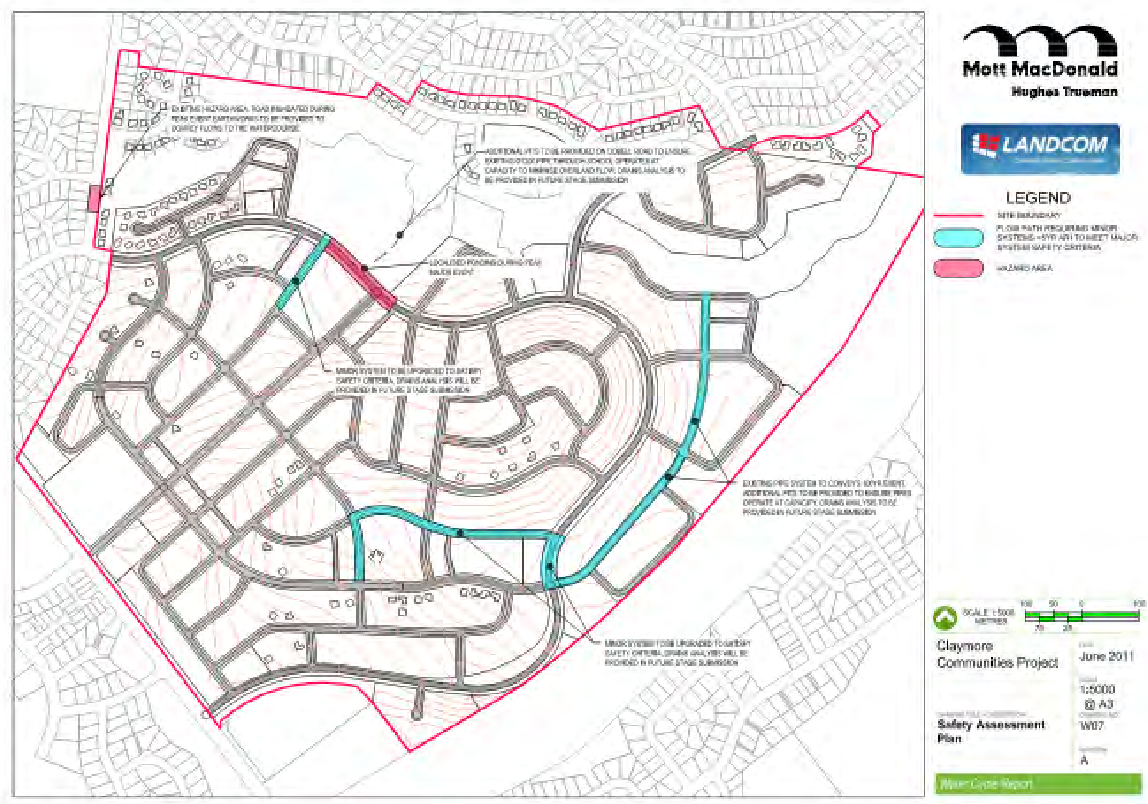

Figure 5.8 –Identified Potential Hazard Areas and Oversized Trunk Stormwater Lines

Table 5.7: Velocity – Depth Ratios

	Velocity (m/s)	Depth (m)	V*D	V*D<0.4
N10.0	1.35	0.08	0.11	YES
N11.0	1.3	0.17	0.22	YES
N13.0	1.01	0.34	0.34	YES
N2.0	2.72	0.33	0.89	NO
N2.1	1.49	0.46	0.69	NO
N2.2	1.81	0.38	0.69	NO
N2.3	1.87	0.32	0.60	NO
N2.4	2.72	0.18	0.50	NO
N3.0	1.9	0.30	0.57	NO
N3.1	1.18	0.24	0.29	YES
N4.0	3.23	0.12	0.39	YES
N4.1	1.47	0.17	0.25	YES
N5.0	2.58	0.44	1.13	NO
N5.1	2.34	0.15	0.35	YES
N5.2	2.01	0.23	0.46	NO
N5.3	1.15	0.28	0.32	YES
N5.4	2.46	0.14	0.34	YES
N5.5	2.42	0.11	0.26	YES
N6.0	1.39	0.10	0.13	YES
N6.1	2.33	0.21	0.50	NO
N6.2	3.09	0.21	0.65	NO
N7.0	1.96	0.71	1.39	NO
N8.0	1.83	0.13	0.24	YES
N9.0	2.76	0.13	0.35	YES

There are a number of existing pipes that are currently oversized for the 1 in 5 year ARI event. It is understood that these pipes were oversized to accommodate larger flows as they are at the tail end of the catchment. Here a blockage factor of 50% has been applied however it is likely that pipes of this size will not become blocked to this extent as pit blockages in the area only account for a small percentage of the catchment. In this instance during the detailed design phase appropriate pit blockage factors will be considered to alleviate any overland flow issues.

6. Hydraulics

6.1. The Model

The HEC-RAS hydraulic analysis program was used to analyse the effect of overland flows on both flood levels and the extent of inundation. The HEC-RAS Version 3.1.1 (May 2003) hydraulic model, the latest windows-based release from US Army Corps of Engineers, Hydrologic Engineering Centre, is widely used for analysis of hydraulic conditions where floodplain storage effects are small.

HEC-RAS is an integrated package of hydraulic analysis programs capable of performing one-dimensional, steady or unsteady flow, water surface profile calculations. The model can handle a full network of channels, a dendritic system or a single river reach. It is capable of modelling subcritical, supercritical and mixed flow water surface profiles.

The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's Equation). The effects of various obstructions such as bridges, culverts, weirs and obstructions in the floodplain are also considered in the computations.

6.2. Model Formulation

Formulating a HEC-RAS model involves defining river geometry, and boundary conditions.

HEC-RAS models were formulated to represent both the existing and proposed scenarios, subsequently allowing for assessment to be made on the extent of flood inundation, depths and the like. That is, the existing (pre-development) model is created to represent the extent of inundation and flood levels experienced by existing flows through the channel / basin system, while the proposed model encompasses any changes which may have been imposed on the flowpath by the new development footprint and extents for proposed basins, roads and the like.

6.2.1. River Geometry

The existing network contains one "reach" which extends from the sag at Drysdale Street, along the channel / piped system before discharging to the open channel to the North East.

The proposed HECRAS model includes a storage basin using the lateral weir function in order to ensure that the flow rates are not exceeded at the downstream most section in the 1 in 100year ARI event.

2.2.2.4. Existing

Surveyed cross-sections from 12d were used to model the existing surface profile of the watercourse (and surrounding land) where overland flow will occur during peak events. Here cross Sections were positioned at critical points, while other sections placed between at 50 metre intervals. Drawings W09 and W010 shows the cross-section locations for the existing scenario while diagrammatic layouts of the model are included in Appendix D.

Resistance to flow is a function of the surface roughness in the channel and overbank areas, and is affected by vegetation, development etc. Roughness was represented by Manning's 'n' values. Guides for the estimations of roughness parameters are given in several standard publications such as Australian Rainfall and Runoff (1987), HEC-RAS Hydraulic Reference Manual (2003), and Councils Engineering Design Guide for Development (2009). Values of Manning's 'n' were chosen on the basis of field inspection. Here 0.035 was used for grassed areas, 0.06 for concentrated areas of trees and 0.013 for roads and / or pathways.

Along the existing river alignment, there are a number of buildings that may impact on the effectiveness of overland flow. These buildings were incorporated into the HEC-RAS model as "obstructions" and include amenity buildings, dwellings, etc.

The existing network includes a series of detention basins (4) and a large underpass beneath Gould Rd. Each of the existing detention basins are formed via an earth embankment mound. The staged storage relationship on each of the basins was assessed in Section 5.0 and includes a low level piped outlet (typically 1650dia pipe) as well as a high level spillway (formed by the top of the embankment).

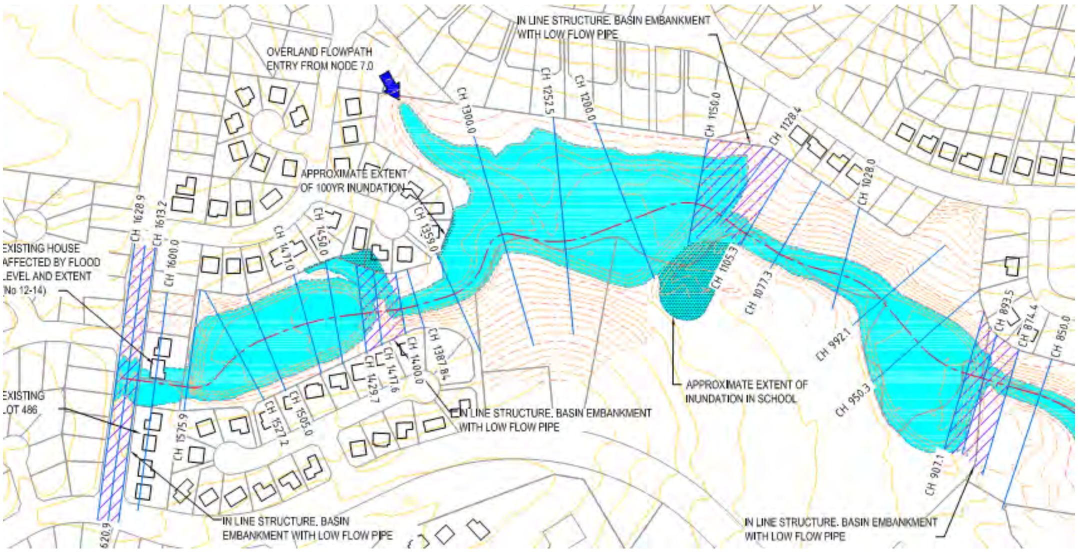

Figure 6.1 – Western Portion of Existing HEC-RAS

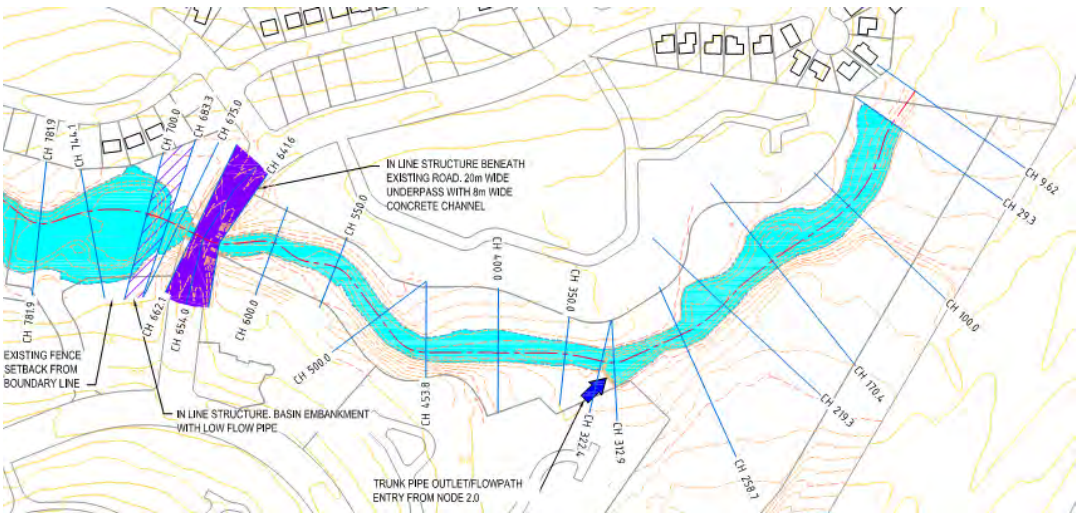

Figure 6.2 –Eastern Portion of Existing HEC-RAS

Detention Basins were modelled within HEC-RAS as “inline structures” in accordance with the HEC-RAS user manual. Ineffective areas were then assigned for each of the sections through the detention basins which may be affected by the downstream embankment. Flowrates from Section 5.0 were then assigned at the downstream section to represent the overland flows in excess of the piped system as well as flows from surrounding areas. Refer to Table 5.5.

The bridge crossing (under Gould Road) was modelled as a “bridge culvert” in accordance with the HEC-RAS user manual. This involved interpolation of cross sections just upstream and downstream, ineffective areas applied at 45deg from opening, decks extending for the surveyed width and level, and contraction and expansion losses applied as recommended. The decks (road above) were input from 12d sections.

2.2.2.5. Proposed

The proposed HEC-RAS network was extended to incorporate all proposed works adjacent to the central channel / basin system. Additional Cross Sections were also added at critical positions in order to model new proposed basins, new roads and the like. Cross Sections were also modified for the regraded portion of Creek between approx Ch312.9-Ch90.

The proposed detention basin (at Ch100 – Ch20.83) was modelled as a “storage area” in accordance with the HEC-RAS user manual, while a “lateral structure” was modelled to simulate the volume of flow which is directed into the proposed basin for attenuation.

An embankment across the channel is required in order to direct a portion of surface flows into the “offline” detention basin. The size of the embankment and low flow box culverts were determined by iteration in Section 5.0 and modelled as a “bridge culvert” in accordance with the HEC-RAS user manual.

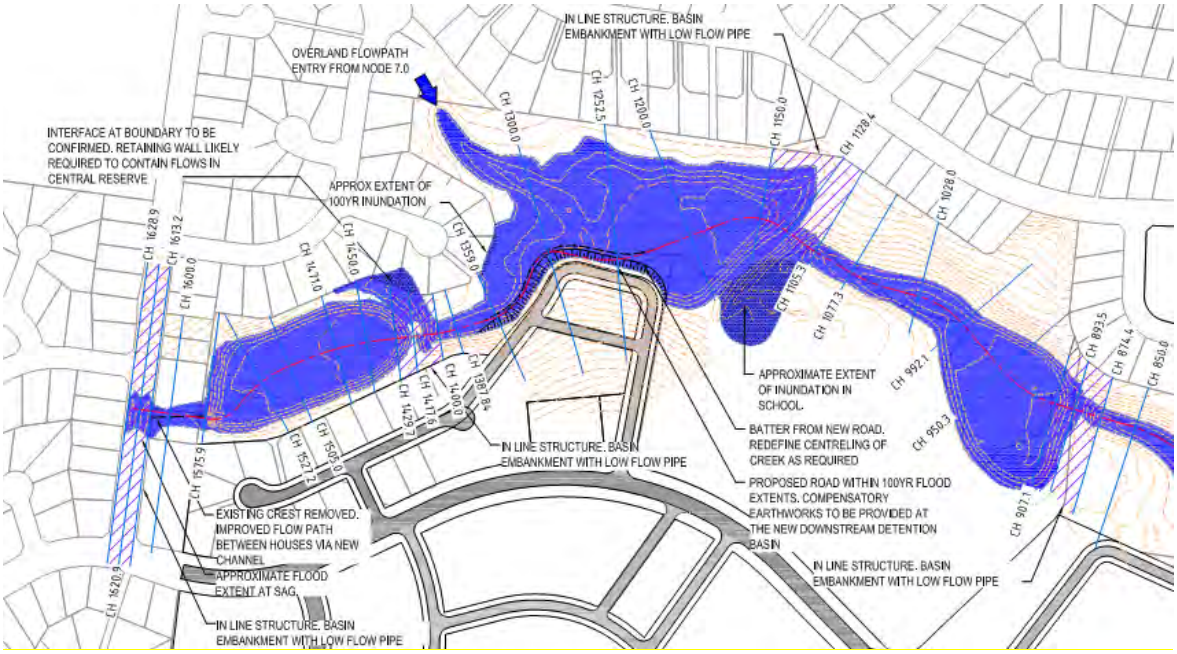

Figure 6.3 – Western Portion of Proposed HEC-RAS

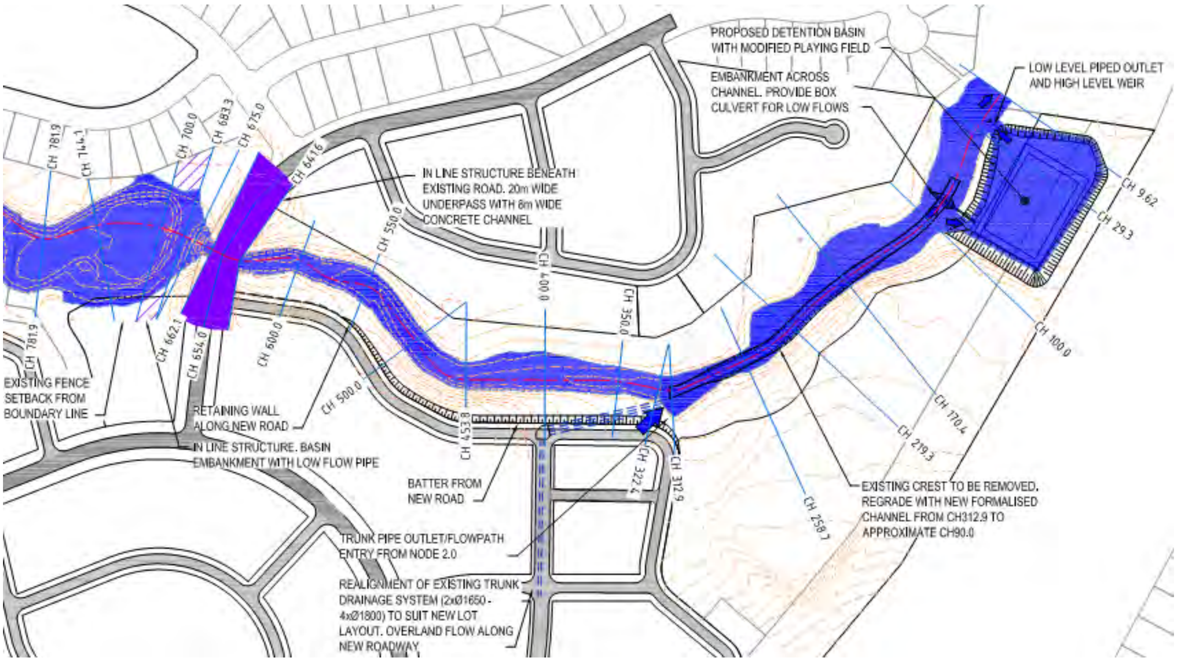

Figure 6.4 – Eastern Portion of Proposed HEC-RAS

6.2.2. Boundary Conditions

Discharges calculated from hydrologic modelling in Sections 5 were incorporated into the model as per Table 5.5. These were inserted at upstream locations as well as additional inflows along the branches at cross-sections corresponding to the hydrologic model nodes that were considered critical. Normal depth was used as the upstream and downstream boundaries (both at 1%).

6.3. Results

Full HEC-RAS results are included in Appendix D.

The following table shows the difference in 1 in 100 year ARI flood levels between the existing and proposed scenarios, while the extent of inundation is shown on drawings W08-W011. The quoted proposed results have incorporated the attenuation measures created by the new detention basin as well as the improved overland flowpaths.

Table 6.1 – 1 in 100 year ARI flood levels (existing versus proposed)

HECRAS CH	Existing W.S	Proposed W.S	Difference (m)
1628.9	73.52	73.29	-0.23
1613.2	73.52	73.28	-0.24
1600	73.47	73.25	-0.22
1575.9	73.01	73	-0.01
1527.2	71.75	71.75	0
1505	71.75	71.75	0
1471	71.75	71.75	0
1450	71.75	71.75	0
1429.7	71.75	71.75	0
1400	69.07	69.09	0.02
1387.84	69.08	69.1	0.02
1359	69.06	69.07	0.01
1300	69.06	69.08	0.02
1252.5	69.06	69.07	0.01
1200	69.06	69.07	0.01
1150	69.06	69.07	0.01
1105.3	65.7	65.75	0.05
1077.3	65.27	65.32	0.05
1028	64.87	64.94	0.07
992.1	64.86	64.92	0.06
950.3	64.86	64.94	0.08
907.1	64.86	64.94	0.08
874.4	62.5	62.57	0.07
850	62.04	62.11	0.07
781.9	62.07	62.12	0.05
744.1	62.07	62.12	0.05
700	62.07	62.12	0.05
675	61.7	61.7	0
662.1	59.05	59.11	0.06
641.6	59.57	59.63	0.06
600	59.24	59.36	0.12
550	58.93	59.06	0.13
500	58.59	58.66	0.07
453.8	58.22	58.29	0.07
400	58	58.07	0.07
350	57.6	57.66	0.06
322.4	57.29	57.35	0.06
312.9	57.13	56.99	-0.14
258.7	56.93	56.56	-0.37
219.3	56.31	56.34	0.03
170.4	56.32	56.28	-0.04
100	56.1	55.86	-0.24
88.35	56.04**	55.94	-0.1
78.37	55.98**	55.78	-0.2
29.3	55.68	55.59	-0.09
20.83	55.62**	55.65	0.03
9.63	55.37	55.37	0

** denotes interpolated from long section

6.4. Discussion

The following comments are provided:

- As discussed in Section 5.0, a portion of flows within extreme flood events are required to be directed into the new detention basin for attenuation in order to achieve pre-post requirements at the downstream most cross section (i.e 9.6m^3/s flow split in the 1 in 100 year event). Iterations were subsequently undertaken within HEC-RAS to determine a suitably sized embankment and overflow weir to direct such flows into the basin.
- Results indicate that by introducing a lateral weir approximately 18m long at RL5.1m, then 9.6m^3/s is directed into the basin. Refer to detail on drawing W014 and Figures in Appendix D.
- Site investigations and modelling within HEC-RAS indicate that a sag exists at Drysdale St, with ponding occurring prior to flows being directed into the channel / basin system (up to 110mm).
- The sag is currently drained by 5 kerb inlet pits within the roadway into a trunk pipe system (1800dia, 600dia and 1350dia) and onto the headwall outlet. Those flows which are not conveyed via this piped system then travel overland between the houses (No 12-14 and lot 486) through a 20.5m wide reserve. The capacity of the piped system is estimated 17.2 m^3/s. Hence by incorporating a 50% blockage factor, the remaining flowrate will be 3.6m^3/s.
- Results from the existing HEC-RAS show that the existing extent of flood inundation extends into No 12-14 with likely insufficient freeboard to floor level.
- While the proposed development does not extent into the western catchment, an opportunity does exist to improve the current situation prior to handover to Council. Proposed works subsequently incorporates localised regrading of the area from back for kerb to edge of basin. Here modelling has included removal of the crest in the reserve, with a new 13m wide, 0.4m deep channel with 1 in 4 side slopes introduced. Results in the proposed model show that it is likely for flood inundation to be clear of the existing house with freeboard to the existing floor level.
- Proposed flows through the development show a slight increase in flood levels across each of the basins and channels from existing (average 45mm increase from Ch1575 to Ch312.9 with a maximum of 130mm). Results do however indicate that the basins will continue to act as per the original design (by others).
- The design approach adopted in the proposed scenario is to accept these increases and maintain the existing basin configurations. The new detention basin will then allow for the overall flow rate to be reduced prior to discharge downstream.
- The extent of flood inundation in the 1 in 100 year ARI event is shown on drawings W08-W011. Generally the flood extent is restricted within the existing boundaries of the central channel / basin system. There are some isolated areas where the flood extent appears to extend into properties both in the existing and proposed situations. These appear at (a) Ch1440 – Ch1405 (b) Ch1105 – Ch1195 (c) Ch695 – Ch755. In each of these instances, the interface between the channel and the existing properties will be confirmed within detailed design. The following is also noted:

(a) Ch1440 – Ch1405 --> Interface to be confirmed. It is anticipated a retaining wall may likely be required along the property boundary to restrict flows to be wholly within the central reserve;

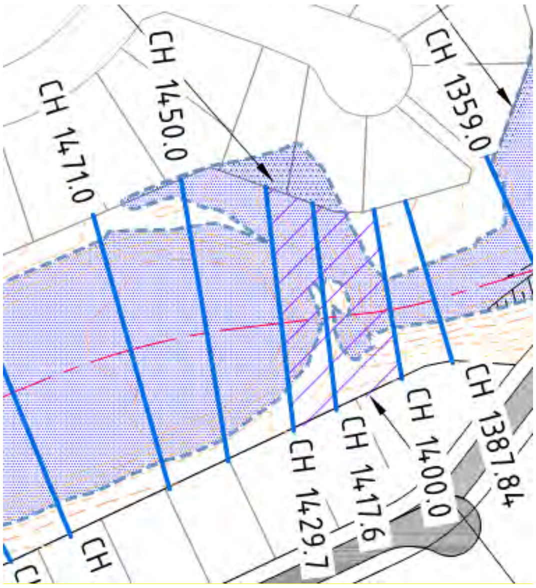

Figure 6.5 – Flood Interface with Existing Properties

(b) Ch1105 – Ch1195 --> Interface to be confirmed. Flooding appears to extend from the basin into the existing school. The boundary needs to be confirmed in this location as it appears that the school has constructed works outside of their property boundary and within the reserve. Flood levels are only increased by 10-50mm at this position – which is considered relatively minimal from existing; and

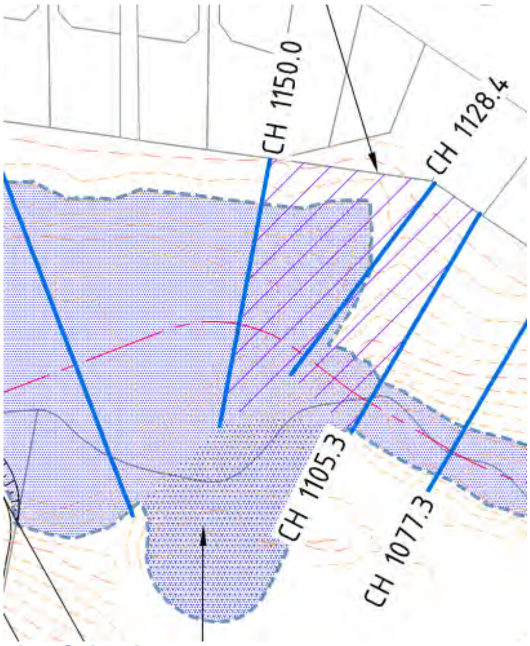

Figure 6.6 – Flood Interface with Existing School

(c) Ch695 – Ch755 --> Interface to be confirmed. An existing fence line is offset inside the property boundary. The flood extent is clear of this fence line, consequently any impact does not seem significant.

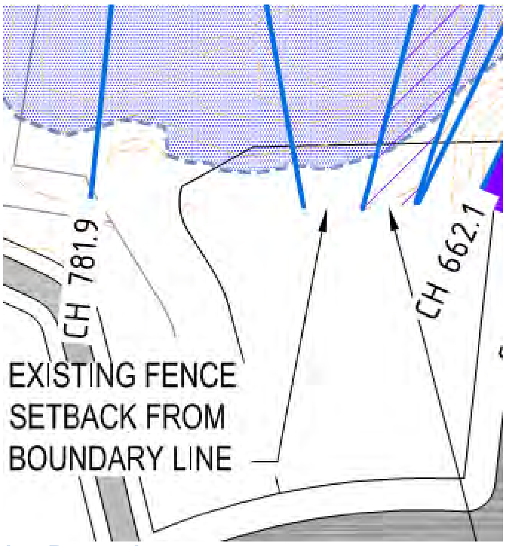

Figure 6.7 – Flood Interface with Existing Properties

- The proposed roadway between Ch1220 and Ch1350 does extend into the existing basin. Preliminary levels undertaken for the master planning have indicated that this roadway will be elevated with a suitable 1 in 4 batter into the basin. The height of the roadway will be at approximately RL69.07m and subsequently be situated above the 100year flood level. Freeboard to nearby houses will be achieved via driveways and the like.
- It is recognised that the basin storage will likely be reduced by the proposed road configuration. As discussed in Section 5, additional storage will consequently be provided via additional volume at the downstream proposed basin.
- Minor regrading will be required in order to re-define the centreline of the flowpath adjacent to the new batter.
- As discussed in Section 5.0, the existing scenario has an area of ponding immediately downstream from the large concrete headwall at Ch312.9 (up to 460mm). From site investigations, this ponding appears to have affected the overall performance of the trunk piped system with evidence of sedimentation occurring in the pipes up to approximately 300mm within the existing 4 x 1800dia. The proposed model has incorporated a new 6m wide channel to grade from Ch312.9 to Ch100 at 0.37%. Refer to drawing W06 and W013. For details, long section and extent. From 12d modelling, it appears that this flowpath can be improved while still maintaining the low flow pipe system in place (600dia) and losing only a few existing trees. Site investigations have indicated that these trees which are to be removed are not considered significant, however should be verified by a qualified arborist.

7. Water Quality Modelling

The stormwater management systems for the site shall comply with Campbelltown City Council's Development Control Plan. Council's policy requires improved water quality of the stormwater flow from the developed site prior to discharge into the authorities drainage system.

To demonstrate compliance with these objectives, treatment removal loads were analysed from pre to post development scenarios using MUSIC (Model for Urban Stormwater Improvement Conceptualisation) software. Model development and results are discussed below.

7.1. Model Parameters

The soil properties for the pervious areas of the catchment were taken from Draft NSW Music Modelling Guidelines (2008).

Table 7.1 - MUSIC Soil Parameters

Soil Properties:	Default Value for Urban Catchments
Impervious threshold (mm)	1.0
Soil storage capacity (mm)	170
Initial storage (% of capacity)	30
Field capacity (mm)	70
Infiltration coefficient 'a'	210
Infiltration coefficient 'b'	4.7
Initial groundwater depth (mm)	10
Daily recharge rate (%)	50
Daily base flow rate (%)	5.0
Daily deep seepage rate (%)	0.0

7.2. MUSIC Methodology

MUSIC software allows the modeller to assess the effectiveness of the water quality devices by measuring against a "base" model (which assumes that no water quality treatment measures are installed). The proposed developed site was compared with and without water quality treatment measures and subsequent pollutant reduction percentages calculated, based on the compared results.

These were then compared with pollutant removal objectives set out by Campbelltown City Council (CCC, 2009). As no other information regarding the watercourse system could be obtained, the following default removal rates were adopted from Council's standards (these rates are also consistent with those specified in Australian Runoff Quality).

Table 7.2 - MUSIC Pollutant Reduction Targets

Pollutant	Minimum Removal Rates
Gross Pollutants (GP)	90%
Suspended Solids (TSS)	80%
Nitrogen (TN)	45%
Phosphorous (TP)	45%

7.2.1. Base Catchment

The RAFTS model developed for detailed analysis and design of the proposed water management system divided the site into approximately 41 sub-catchments. This level of detail is required at the design stage for the site hydrologic and hydraulic analyses. However, this level of detail is not necessary for water quality modelling using MUSIC because the treatment devices capture runoff from large areas and sub-division of sub-catchments smaller than the treatment catchment will not achieve improved results.

The RAFTS sub-catchments were therefore consolidated into 8 sub-catchment areas based on the proposed drainage system layout (refer Figure 7.1 and 7.2).

Catchments were separated into three components for the purposes of the MUSIC model:

- Roof areas;
- pervious areas (including open space); and
- pavement areas (including roads, footpaths, etc.).

Roofed, pervious and impervious areas were measured as a percentage from the master plan documentation.

Claymore Public School has not been included as part of the site and therefore excluded from the water quality modelling.

Table 7.3 - Area Breakdown per MUSIC Sub-Catchment

Sub-Catchment	Roof Area (Ha)	Impervious Area (Ha)	Pervious Area (Ha)	TOTAL AREA (Ha)
M1	3.45	1.68	10.99	16.12
M2	5.32	4.66	7.28	17.26
M3	1.89	1.64	4.75	8.28
M4	6.03	3.88	6.49	16.40
M5	11.26	8.62	9.43	29.31
M6	8.27	6.31	11.98	26.56
M7	1.63	1.26	2.96	5.85
M8	0.66	0.42	0.72	1.80
			TOTAL	121.58

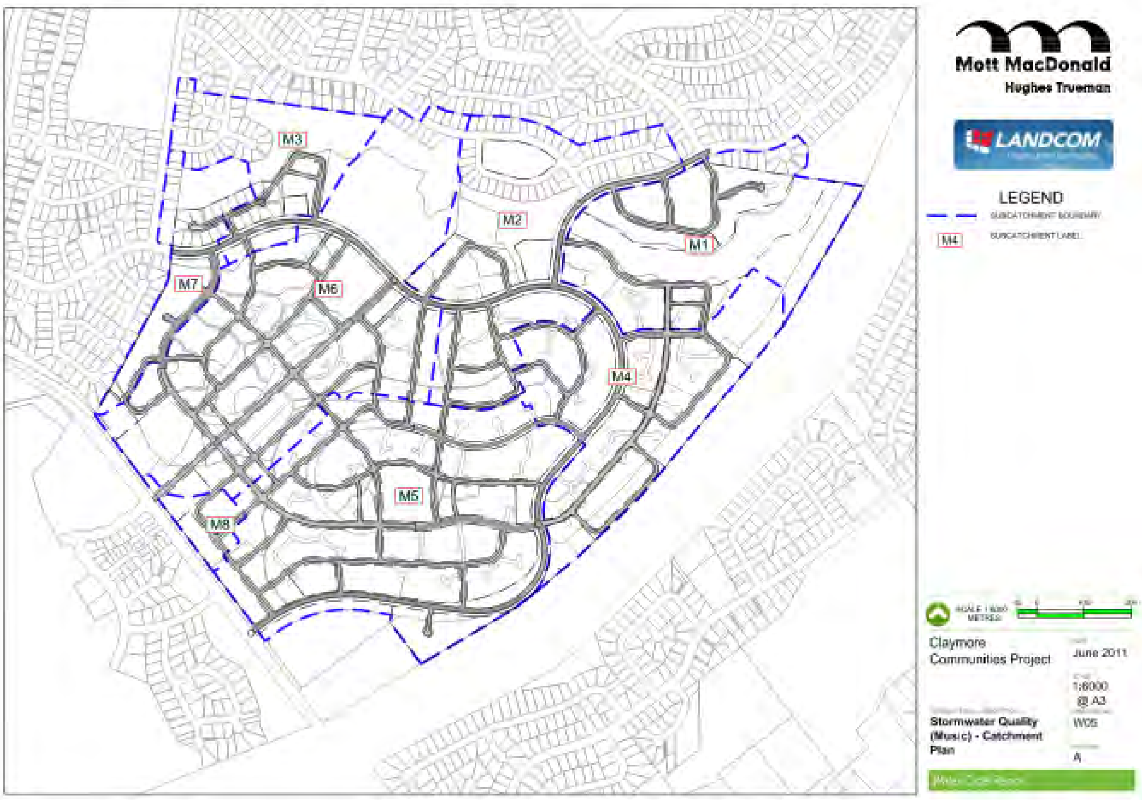

Figure 7.1 – MUSIC Catchment Plan

7.2.2. Proposed Catchment

The proposed catchment model was identical to the base model in terms of catchment area and break-up of roof, paved and pervious areas, but included the water quality management strategies outlined below.

7.3. Management Strategies

Storm runoff generated on the CURP site can be separated into 3 streams:

- Roof or rainwater runoff, which can be captured and reused for toilet flushing or irrigation;
- Road and pavement runoff, which can be treated by grassed swales or bio-retention devices; and
- Pervious surfaces will have reduced runoff due to a portion of infiltration, and water "lost" to groundwater.

The proposed treatment train is as follows:

- Rainwater tanks are to be provided on the proposed dwellings for at source treatment and re-use of roof water;
- Gross pollutant traps and trash racks to capture larger pollutants and sediments before discharge into the watercourse; and
- Native Grass Infiltration swales to provide online treatment for effective removal of fine sediments and nutrients.

The possibility of using the tree bays as an at source stormwater bio-retention device has not been considered as part of this proposal. The deviation of low flows from the road gutters into these tree bays would enable the at source water quality treatment of the low flows. This additional treatment would further improve any water quality results obtained during this modelling. The potential for this would be assessed as part of individual evaluation of each stage depending upon site parameters including road networks and grades.

7.3.1. BASIX Requirements/ Rain Water Tanks

A preliminary BASIX application was prepared (refer appendix F) to provide an approximate indication of the water quality and reuse targets required for individual dwellings to achieve a 40 point water saving rating. This has been based on a lot size of 450m^2, with a 270m^2 dwelling.

To achieve the 40 point rating the following assumptions have been made as the ideal minimum benchmark to be achieved for each lot. More efficient water saving devices may be implemented for dwellings to achieve a higher rating to encourage more sustainable development.

- Water Fittings
  - Minimum 3 Star Rating for Toilet Flushing Fittings
  - Minimum 3 Star Rating for Shower Heads and Tap Fittings
- Rainwater Tanks: 2500L (includes reuse for toilet flushing and gardening)

Rainwater reuse volumes for residential have been calculated based on the following:

- 86L/day for Toilet flushing, and
- 220L/day for general irrigation.

Refer appendix F for a more detailed breakdown of the BASIX point scheme.

7.3.2. Gross Pollutant Traps

For the purposes of MUSIC modelling on the CURP site, it was assumed that Gross Pollutant Traps (GPTs) would be located at the outflow from each discharge point into the watercourse. Additionally, GPTs are assumed upstream of any proposed water body or bio-retention devices to provide pre-treatment of gross pollutants and suspended solids.

Proposed positions of these Gross Pollutant Traps are shown in drawing W06. Here positioning has taken into consideration proposed catchments as well as both existing and proposed stormwater infrastructure.

MUSIC requires that transfer functions for the reduction in pollutants be entered. The pollutant reductions vary for different types of GPTs, estimates were therefore applied to the average advertised removal rates of the Rocla's "Cleansall", the CDS Unit and Humes' "Humeceptor":

Table 7.5 - GPT Pollutant Reductions

Pollutant	Rocla [1]	CDS [2]	Humes [3]	Adopted Rate
Total Suspended Solids (mg/L)	70%	70%	87%	**70%**
Total Nitrogen (mg/L)	-	23%	45%	**25%**
Total Phosphorus (mg/L)	-	30%	30%	**20%**
Gross Pollutants (kg/ML)	100%	98%	-	**95%**

[1] *Rocla Water Quality – Cleansall Gross Pollutant Trap* (Rocla Pty. Ltd. 2002)

[2] *Removal of Suspended Solids and Associated Pollutants by a CDS Unit* (CRC Catchment Hydrology 1999)

[3] *Humeceptor case study – Seatac, Washington USA* (http://www.humes.com.au/products/StormwaterQuality/humeceptor/seattle.pdf)

Table 7.6 - MUSIC Input - GPT Pollutant Reductions

Pollutant	Input	Output
Total Suspended Solids (mg/L)	1000	300
Total Nitrogen (mg/L)	50	25
Total Phosphorus (mg/L)	5	4.0
Gross Pollutants (kg/ML)	15	0.8

In accordance with statutory requirements, the GPTs will need to treat the maximum flow rate from their upstream catchments for all flows up to and including the 3-month ARI storm event. The following flow rates have been extracted from the RAFTS model.

Table 7.7 - GPT-Treatable Flow Rates

GPT No	Location	Rafts Node	Treatable Flow Rate (m^3/s) – 3 month
1	The Northern side of the watercourse and West of Gould Rd	N3.1	0.215
2	The Southern side of the watercourse and West of Gould Rd	N10.0	0.162
3	The Southern side of the watercourse and West of Gould Rd	N4.0	0.309
4	The North-East Corner of Davis Park	N5.0	1.960
5	The Southern side of the watercourse alongside the Carter Pl extension	N6.0 + 20% of C1.4	0.167

7.3.3. Trash Screens/Racks

At present catchments M5 and M4 discharge into the creek system via four 1800mm dia. culverts. It is desirable to pre-treat these flows to remove gross pollutants before discharging into the infiltration basin, however providing a GPT for these pipes is not desirable due to the size of the required unit and maintenance requirements.

In order to remove gross pollutants a trash screen is proposed at the downstream end of the headwall to filter the larger pollutants for the 3 month ARI flows. Providing a trash rack at this location will allow for easy access and maintenance.

It has been assumed that the trash screen will not remove any nutrients or fine sediments therefore the following removal rates have been adopted.

Table 7.8 - MUSIC Input – Trash Screen Removal Rates

Pollutant	Input	Output	Percent Reduction
Total Suspended Solids (mg/L)	1000	700	30%
Total Nitrogen (mg/L)	50	50	0
Total Phosphorus (mg/L)	5	5	0
Gross Pollutants (kg/ML)	15	0.75	95%

7.3.4. Infiltration Basin

An infiltration basin is proposed to treat runoff from sub-catchments M1, M4 and M5. Upstream flows from each sub-catchment will be directed to GPT's and trash racks to provide pre-treatment of gross pollutants and larger suspended solids prior to entry into the watercourse. The 3 month flows will be conveyed via the 4 x 1800mm dia. Pipes and headwall to the treatment facilities, with larger flows continuing along the stormwater network as bypass.

The following parameters were input into the MUSIC model:

Table 7.9 – Infiltration Basin MUSIC Parameters

Catchment	Swale Surface Area (m^2)	Extended Detention Depth (m)	Filter Area (m^2)	Depth of Infiltration (mm)
M1, M4, M5	2600	0.15	2300	600
M2, M3, M6, M7	2000	0.15	1700	600

7.3.5. SALINITY: WSROC Salinity Code of Practice and CCC Standards

The Western Sydney Salinity Code of Practice: Salinity Map identifies the Claymore site as at risk of high risk of salinity potential.

In accordance with council's guidelines bio-ribbons and wetlands are to be constructed with an impermeable membrane (specific reference is made to clay being unsuitable for use as an impermeable layer). In order to mitigate the effects of salinity a plastic membrane is proposed to provide a barrier reducing the infiltration of additional water into the groundwater system from the bio-retention and infiltration ponds, which if not implemented may potentially increase the groundwater level and subsequently cause saline to rise to the surface.

7.4. Results

7.4.1. Base Model

In accordance with the industry standards and assessment processes the base water quality MUSIC model for the site was developed assuming that no water quality treatment measures would be installed. This model provides the basis for pollutant generation from the site and the measure for pollutant removal under "treated" conditions.

7.4.2. Proposed Model

The "treated" site conditions model was developed incorporating the water quality treatment train as described above. Diagram 7.2 shows the layout of the model in MUSIC.

The results of the model are summarised in Table 7.10 below, and show that including a treatment train as described above, the water quality improvement objectives set out in Council’s Campbelltown (Sustainable City) *Development Control Plan 2009, Volume 2, Engineering Design for Development* – June 2009 are achieved.

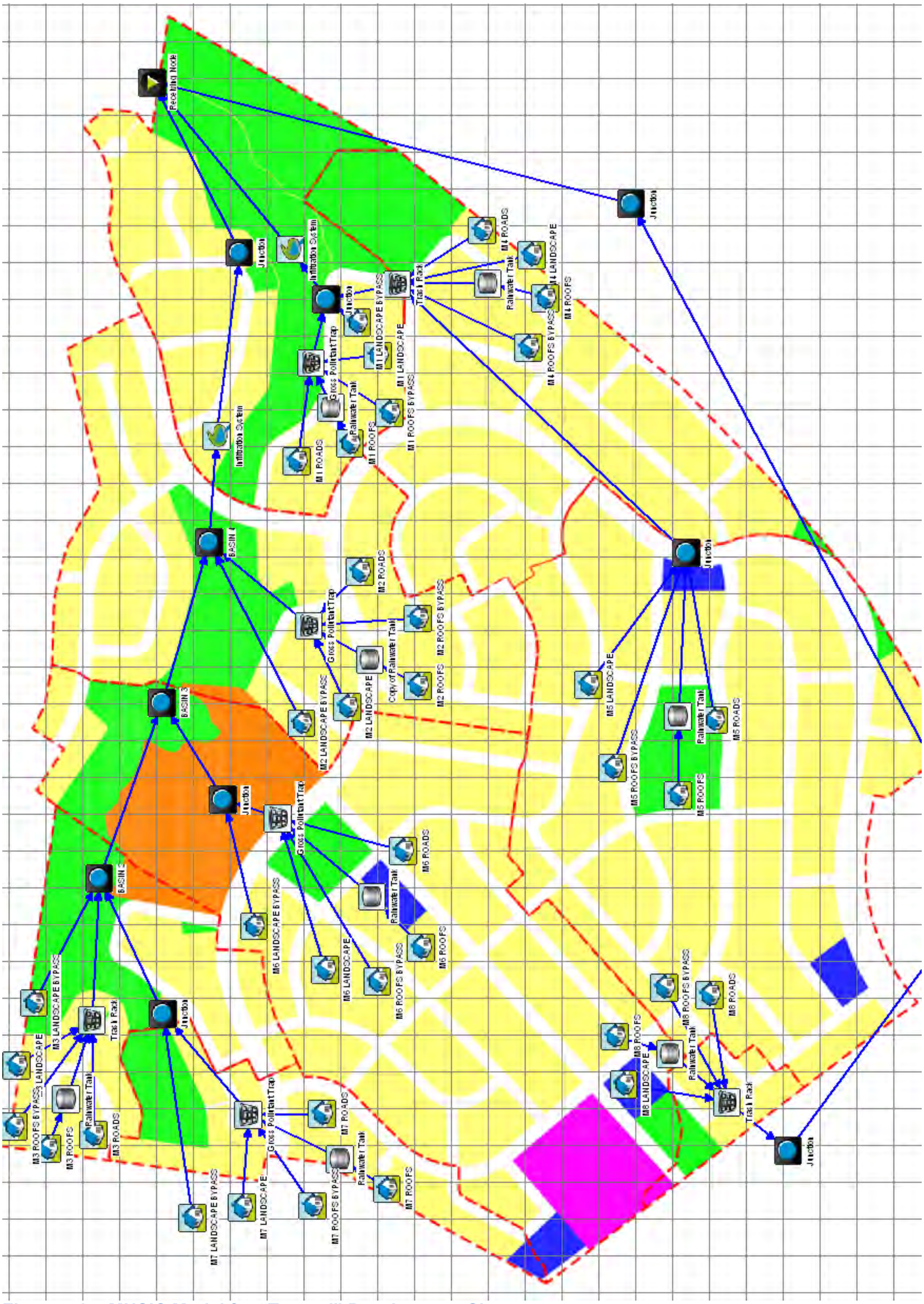

Figure 7.2 – MUSIC Model for "Treated" Development Site

Table 7.10 – MUSIC Model Results vs Objectives

	TTSS (kg/yr)	TP (kg/yr)	TN (kg/yr)	GPs (kg/yr)
GENERATION	120000	242	185000	21100
OUTPUT	23800	77.7	599	12.7
REDUCTIONS	80.1%	68%	67.6%	99.9%
OBJECTIVES	80%	45%	45%	90%

8. Conclusions

8.1. Water Quantity

RAFTS models of the proposed Claymore Urban Renewal Project were set up and run using design storms of various ARI's and durations. The model results were compared against the corresponding models used to represent the existing catchment development.

The model included retention of existing detention basins along the central watercourse and adjustments of Basin 3 to incorporate the developed footprint. A 5,000m^3 basin has been proposed at the outlet to the site, incorporated as part of Fullwood Reserve to ensure that downstream flows and flood damage risk would not increase in the 100-year ARI event as a result of the proposed development.

HEC-RAS models were also created to determine both the level and extent of flood inundation for the 100-year ARI event. An assessment identified potential hot spot areas and indicated those regions which require dwellings to sit higher than adjacent road levels. Analysis also showed that there would be no adverse effect on existing houses to be retained in the estate and downstream properties.

An assessment of flow depths and velocities in the 100-year ARI was also undertaken. A number of areas were identified where minor piped drainage system links would need to be designed for an ARI greater than Council prescribed criteria (i.e. 5-year ARI) so that overland flows in a major storm meet the safety criteria.

8.2. Water Quality

The water quality model set up using the MUSIC software provides an indication of the pollutant removal rates expected when applying a treatment train of measures. However, the model is limited to concept analysis and the detailed size and removal rates for the different treatment components should be developed at the detailed design stage of the project.

According to the results of the MUSIC analysis, a treatment train consisting of rainwater reuse, Infiltration facilities and GPTs/ trash racks will provide adequate treatment from the proposed development of the Claymore Urban Renewal Project site to exceed the statutory water quality objectives.

9. References

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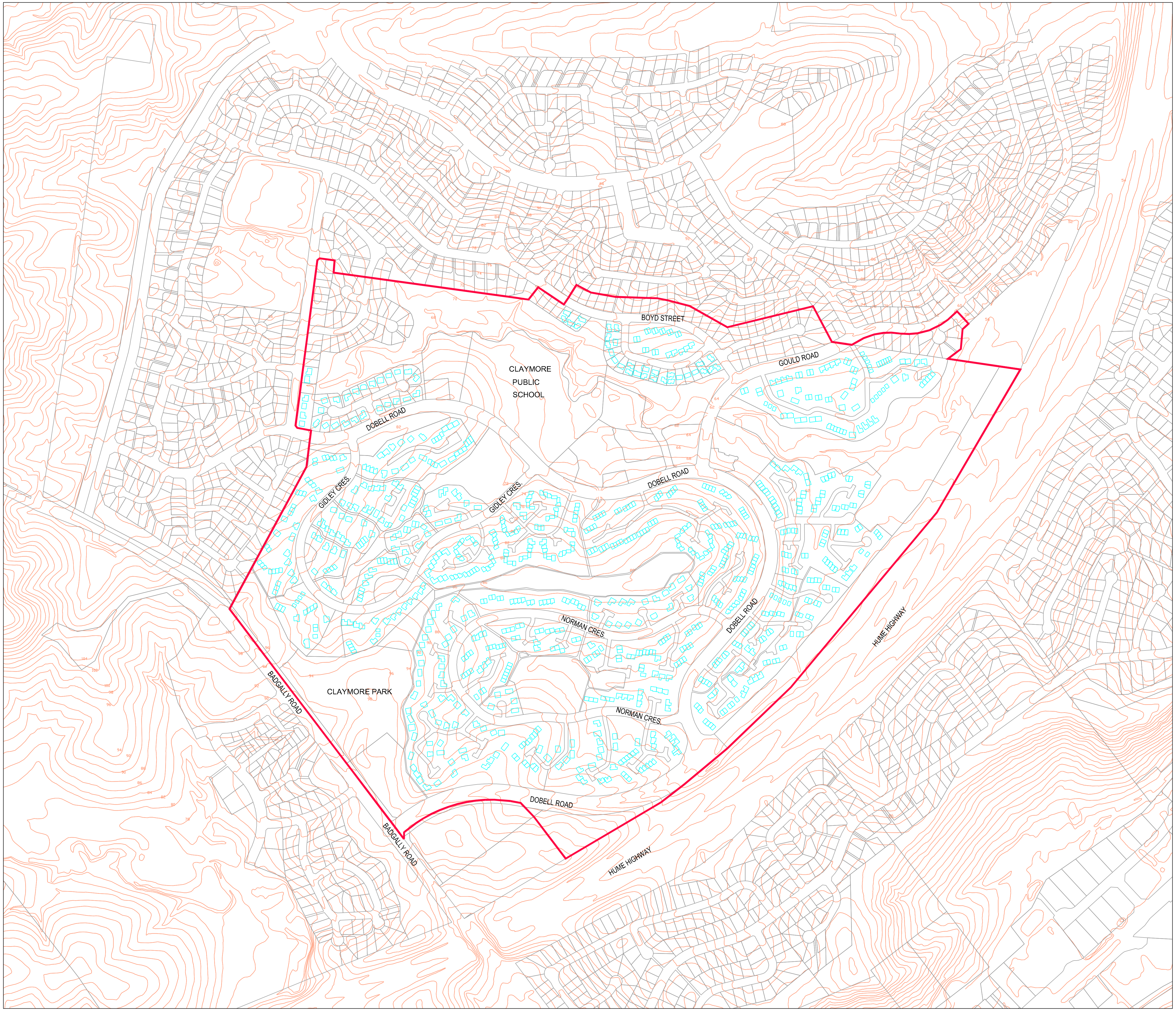
Appendix A: Drawings



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Hughes Trueman

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LEGEND

SITE BOUNDARY

SCALE 1:7500
METRES

150 75 0 150

Claymore Communities Project

DATE
June 2011

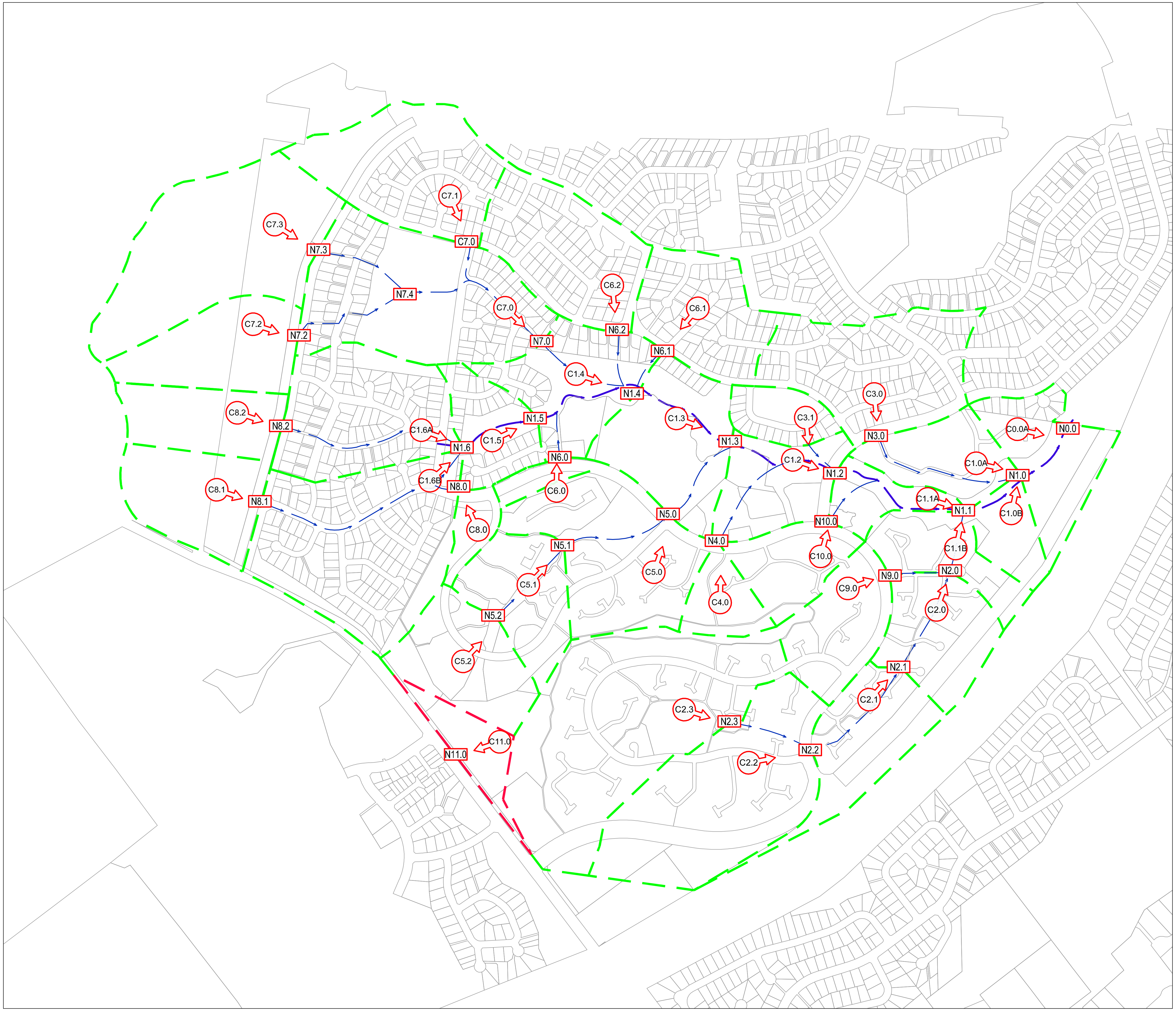
SCALE
1:7500
@ A3

DRAWING TITLE + DESCRIPTION
Existing Site Plan

DRAWING NO
W01

REVISION
A

Water Cycle Report



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LEGEND

EXISTING OVERLAND FLOW PATH

EXISTING CREEK

RIDGE LINE

NORTHERN CATCHMENT BOUNDARY

SOUTHERN CATCHMENT BOUNDARY

N1.2 RAFTS 'LINK' NODE

C1.3 RAFTS CATCHMENT NODE

SCALE 1:8000 METRES

100 50 0 200

N

Claymore Communities Project

DATE June 2011

SCALE 1:8000 @ A3

DRAWING TITLE + DESCRIPTION **Existing Catchment Plan**

DRAWING NO W02

REVISION A

Water Cycle Report

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LEGEND

- SITE BOUNDARY
- EXISTING DETENTION BASIN
- PROPOSED DETENTION BASIN
- PROPOSED INFILTRATION SWALE
- PROPOSED ROAD
- EXISTING ROAD
- SENIORS LIVING
- RETAIL CENTRE
- PARKS / OPEN SPACE
- GPT — GROSS POLLUTANT TRAP
- TRASH RACK — TRASH RACK
- PROPOSED GRASS SWALE

SCALE 1:6000 METRES
40 0 100 200

Claymore Communities Project

DATE: June 2011

SCALE: 1:6000 @ A3

DRAWING TITLE + DESCRIPTION: **Proposed Site Plan**

DRAWING NO: W03

REVISION: A

Water Cycle Report