Eureka 1 Project 10 Pty Limited

Proposed North Medowie Residential Project Boundary Road, Medowie

Flood and Drainage Assessment

February 2010

Umwelt
Environmental Consultants

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Prepared by

Umwelt (Australia) Pty Limited

on behalf of

Eureka 1 Project 10 Pty Limited

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1.0 Introduction

Umwelt (Australia) Pty Limited (Umwelt) has been commissioned by Eureka 1 Project 10 Pty Limited to undertake a flood and drainage assessment of a proposed residential project at Medowie at Lots 93, 94, 95 and 96 of DP 753194 (refer to **Figure 1.1**).

This assessment has been undertaken to support an application to Port Stephens Council (PSC) to rezone the land for rural-residential purposes. This assessment follows on from two previous flood and drainage studies of the site prepared by Umwelt:

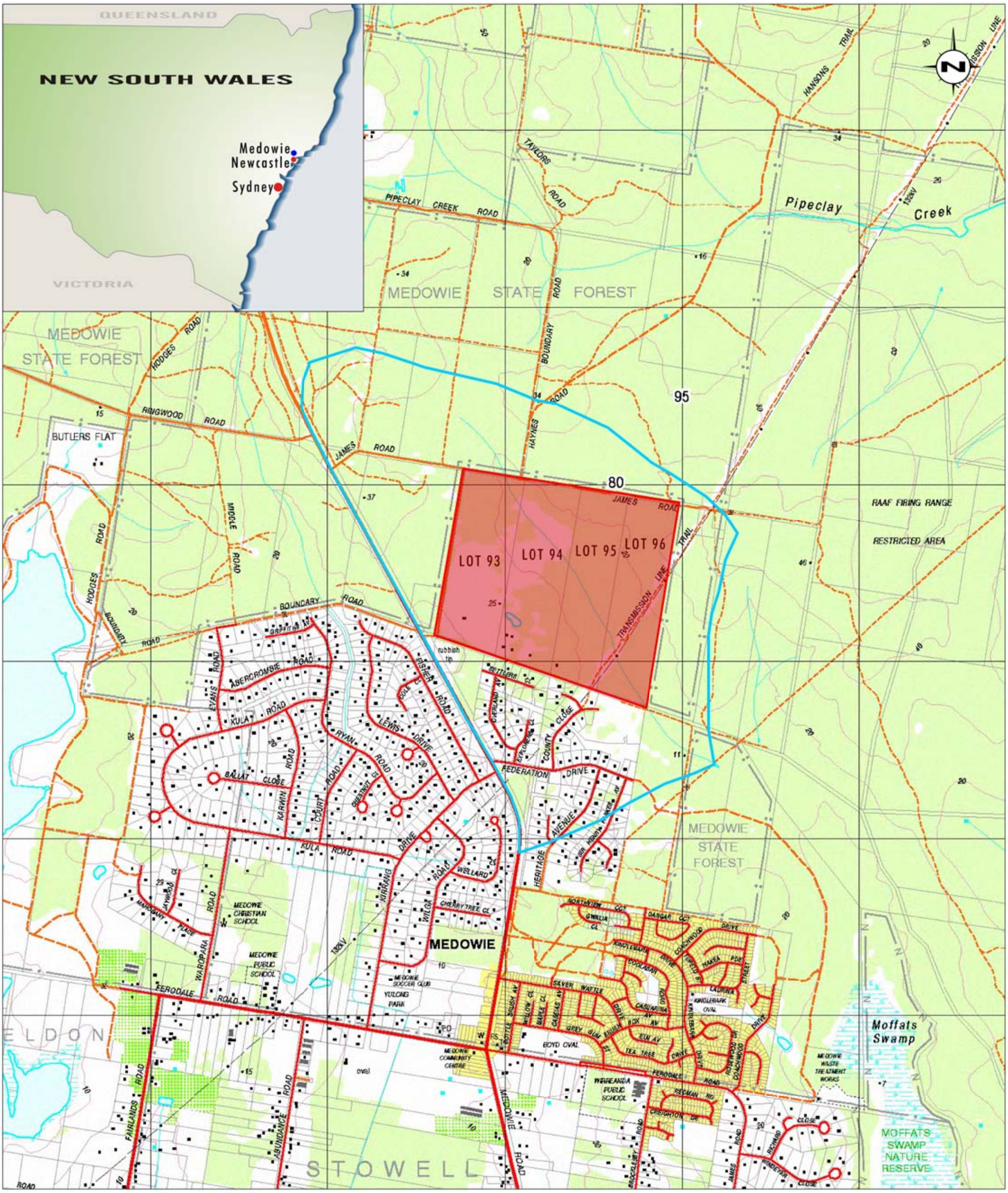
- *Flood and Drainage Study of Proposed North Medowie Residential Project, Boundary Road, Medowie*, 2006; and
- *Proposed North Medowie Residential Project Boundary Road, Medowie, Flood and Drainage Assessment*, 2009.

This report builds on information contained in the 2009 study and incorporates the assessment for a proposed development layout of 300 rural-residential lots.

1.1 Council Requirements

Council's requirements regarding stormwater controls for subdivisions are detailed in *Development Control Plan PS3 – Subdivision Guidelines* (DCP).

The DCP states that stormwater detention is to be provided to limit post-development runoff to that of the pre-developed site (i.e. natural) for storms up to the 1% AEP storm event.



Source: Department of Lands (2006)

Legend

Site Boundary

Catchment Boundary

FIGURE 1.1

Locality Plan

2.0 Site Description

2.1 Existing Site Description

The proposed North Medowie Residential Project site has an area of approximately 126 hectares. The site is bounded by James Road to the north, a private road to the west, dense vegetation to the east, Boundary Road to the south and residential development to the south of Boundary Road. The site is partially cleared and gently slopes from north-west to south-east. Elevations range from 24 metres AHD in the north-west to 13 metres AHD in the south-east, with a maximum of 32 metres AHD in the north-east (refer to **Appendix A)**. The majority of the site is relatively flat, with slopes ranging from 0% to 3% over most of the site increasing to 3% to 5% in the north-west of the site.

An unnamed creek (first order under the Strahler stream order system) traverses the site from the north-west to south-east, conveying stormwater from the upstream catchment areas. The catchment area of the drainage system extends outside the site some 920 metres to the north-west towards Hodges Road and 500 metres to the east and comprises a total catchment area of approximately 410 hectares. The creek is well-vegetated with grass and sedges and has longitudinal grades of approximately 0% to 3% through the site. The unnamed creek drains to Moffats Swamp (refer to **Figure 1.1**). Moffats Swamp is located approximately 2 kilometres south-east of the development area.

The catchment of Moffats Swamp is bounded to the north by a ridge within Medowie State Forest (i.e. to the north of the site), to the west along Medowie Road and Brocklesby Road and in the south by Richardson Road. The eastern boundary of the catchment is defined by a sand barrier, which adjoins the Tomago sand beds. Moffats Swamp is the lowest point within the Moffats Swamp catchment area. Runoff within the catchment is either stored within the swamp or released from any of the three outlets (PSC, 2003).

Two small dams are present in the south-western part of the site with some dwellings present in the southern part of the site. An electricity easement traverses the south-eastern portion and contains a high voltage transmission line (refer to **Figure 1.1**).

2.2 Post-Development Site Description

The proposed development is a rural-residential subdivision (refer to **Appendix B**) of between 300 and 350 lots with a median lot size of approximately 1200 m^2. The proposed lots will be accessed via a network of local streets 6.5 metres wide, with road reserves of 6.75 metres wide on both sides of the proposed roads. The site is divided into two main areas by a natural drainage path and associated vegetation buffers that are orientated from the north-west to south-east corners of the site. The proposed development is limited to the south-western side of the unnamed creek.

The 100 year Average Recurrence Interval (ARI) storm event flood extent was determined for the site in a previous study (Umwelt, 2006) (refer to **Appendix B**). The proposed development layout includes three proposed lots, in the north-western area of the site, that are within the modelled 100 year ARI storm event flood extent. The proposed building location zones for these three proposed lots are outside of the 100 year ARI storm event flood extent. As such no filling of the lots is currently proposed within the 100 year ARI storm event flood extent. In addition, floor levels of all proposed dwellings will be located a minimum of 500 millimetres above the 100 year ARI storm event flood level for the site (refer to **Appendix B**).

The pre-development 100 year ARI storm event flood extent also shows flooding along the overflow path from an existing on-site dam (refer to **Section 2.1**). This dam is not proposed to form part of the final development layout and as such overflows will not occur or have potential to influence downstream properties.

It is proposed that elements of Water Sensitive Urban Design (WSUD) will be incorporated into the development, including the use of rainwater tanks at each allotment for both flood mitigation and re-use on site, and swales and infiltration trenches used to control the surface water leaving the impervious areas such as the roads, driveways and roofs.

3.0 Proposed Stormwater Mitigation Strategy

3.1 Development Phase Stormwater Management Strategy

The stormwater mitigation strategy for the site has been developed with consideration of the flow regimes of the site and the potential impacts of the site on flood flows and velocities both within the site and downstream.

The proposed stormwater mitigation strategy includes:

- A dry detention basin (refer to **Figure 3.1**) that will be located downstream of the development area of the site to the north of the unnamed tributary. The proposed basin will be constructed to not hold any permanent water and would require construction of a 1.1 metre high grassed embankment approximately 205 metres in length.

- Rainwater tanks which are proposed for each allotment. The rainwater tanks will be installed to capture approximately 75% of roof runoff. A volume of 5 kL is proposed for the flood detention component of each rainwater tank.

- The rainwater tanks could also include a storage component (i.e. rainwater harvesting) zone. Installation of 10 kL rainwater tanks could provide 5 kL of flood detention and 5 kL for rainwater storage for re-use on site and in the home.

- Rainwater tanks would be installed in a manner consistent with the principles identified in Port Stephens Council's *Urban Rainwater Tank Policy* (PSC, 2003) and the Lower Hunter & Central Coast Regional Environmental Management Strategy's (LHCCREMS) *Fact Sheets for Water Sensitive Urban* Design – *Rainwater Tanks for dual water supply and stormwater management* (LHCCREMS, 2001).

Other WSUD elements that will be considered during future development applications (e.g. development applications for subdivision) that could improve the runoff response of the development would include infiltration trenches and grass swales to receive runoff from the impervious areas of the development such as the surplus roof area and tank overflow, driveway and road runoff. Lot-scale infiltration trenches could increase the soil moisture storage across the lot area. This would increase the soil moisture storage which would reduce the need for supplementary watering of plants and grasses. The consequent increases in infiltration and evapotranspiration losses would further reduce the runoff expected from each lot area. By using grass swales as the stormwater controls for the road water, further retardation of the post-development runoff response could be achieved. Such swales would allow for increased surface storage of stormwater, consequently increasing the rate at which water could infiltrate into the soil, and increase the opportunity for direct evaporation losses. Such structures could also further improve the soil moisture storage in the development area. A further advantage of such an approach to road water management could be the reduction (or potentially elimination) of underground stormwater pipes (i.e. a traditional minor drainage system), and the associated discharge structures required to dissipate the concentrated end-of-pipe flows before being released into the natural drainage path. The will be explored further as part of the future development proposals for the site.

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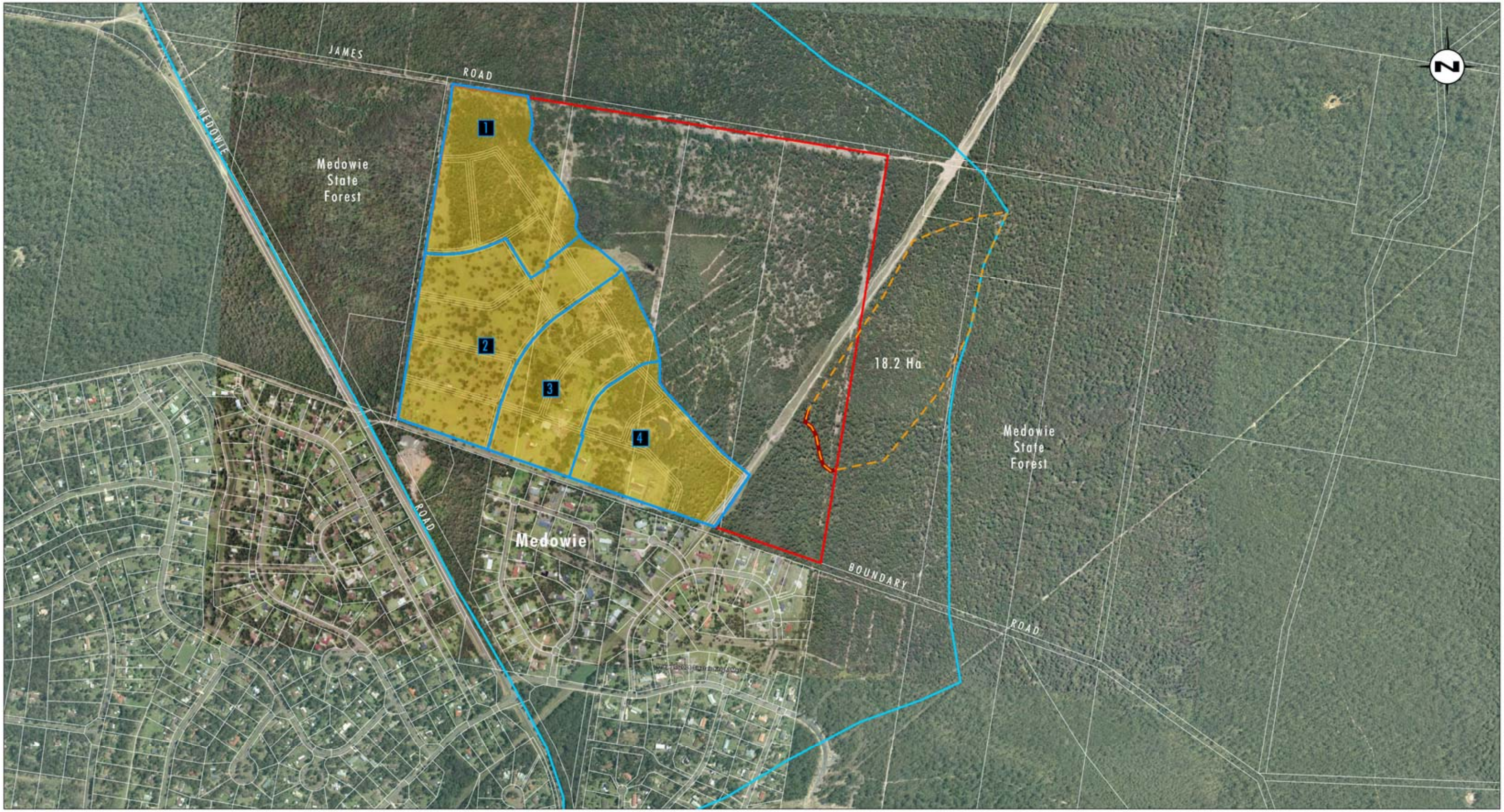

Source: Google Earth (2007), Port Stephens Council (2006), Umwelt (2006), Eureka 10 (2009)

Legend

- Site Boundary
- Dry Detention Basin Catchment
- Proposed Basin Embankment
- Land Use Plan
- Development Area Partition
- Catchment Boundary

FIGURE 3.1

Approximate Location of the Proposed Dry Detention Basin and Associated Subcatchment

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3.2 Construction Phase Stormwater Management Strategy

A detailed Erosion and Sediment Control Plan will be prepared for all construction activities in accordance with *Managing Urban Stormwater: Soils and Construction* (Landcom, 2004) and Council guidelines prior to commencement of construction. The Erosion and Sediment Control Plan will be suitable for construction purposes and will be developed in accordance with the controls and methods outlined in **Section 3.3**.

The proposed development site is typically on the foot slopes of the catchment area and adjacent to the vegetation buffer strips associated with the unnamed tributary (refer to **Appendix B**). The soils of the proposed development area are not dispersive and as such standard erosion and sediment control measures are considered suitable for use during the construction phase.

3.3 Construction Phase Erosion and Sediment Controls

Prior to site disturbance, erosion and sediment controls will be established to prevent sediment laden runoff from entering downstream creek systems. Specific erosion and sediment controls will be contained in the construction plans. These plans will include measures to be adopted to control the quality of runoff including the following:

- construction of control works such as sediment fences, hay bales and groynes at all stormwater inlets, around material stockpiles and at potential areas of increased sediment runoff prior to construction works commencing within the site area;
- ensuring that where possible all drainage and sediment and erosion control works are designed and constructed to be free draining to minimise the potential for ponding, infiltration and tunnel erosion;
- minimising all disturbed areas and stabilisation by progressive rehabilitation as soon as practicable;
- progressively staging earthworks to reduce the area that has potential to generate sediment;
- progressively stripping and stockpiling topsoil for later use in rehabilitation;
- constructing access road and earthworks cut and fill batters at slopes (of 1V:3H or less, where possible) to maximise long term stability;
- limiting the number of roads and tracks established;
- regular maintenance of all controls and inspection of all works weekly and immediately after storm events to ensure erosion and sediment controls are performing adequately;
- establishing a stable vegetative cover on all areas as soon as possible and regularly maintaining these areas; and
- bunding of fuel and oil storage areas and other pollutant-generating areas.

In addition, the construction plans for the site will detail the specific inspection, maintenance and revegetation requirements for each work area.

Sediment fences are to be designed in accordance with Landcom (2004). Where necessary, sediment fences are to be constructed immediately downslope of the areas to be disturbed and downstream of fill stockpiles to minimise the potential for sediment transport into receiving catchments and waterways. Fences are to be constructed using geotextile filter fabric with structural posts to be spaced no more than 3 metres apart.

4.0 Modelling Methodology

4.1 Approach

A one dimensional hydrodynamic XP-Storm model of the development area and unnamed tributary was previously developed to assess the potential impacts of a previously proposed residential subdivision on stormwater and flooding (Umwelt, 2006). This XP-Storm model has been updated to reflect the new proposed rural-residential subdivision plan for the site.

The previous modelling (Umwelt, 2006) of the site determined that the critical storm duration for the unnamed tributary downstream of the site is the 9 hour storm event. As the existing catchment conditions have not changed since the development of this model, the pre-development (i.e. existing) model was used as the basis for this assessment.

An additional model has been developed to model the potential impacts of the proposed source controls on stormwater and flooding (refer to **Section 4.2**). The modelling of source controls was undertaken using a stochastic spreadsheet allotment runoff model that can simulate tank storage and discharge. The output hydrographs from the spreadsheet model were used as input hydrographs to nodes within the XP-Storm model of the wider catchment area and natural drainage network (refer to **Section 4.3**).

Five scenarios were modelled, as follows:

1. existing (i.e. pre-development) conditions;
2. post-development conditions with no mitigation strategies;
3. post-development conditions with source controls;
4. post-development conditions with outlet controls; and
5. post-development conditions with both source and outlet controls.

The first scenario represents the baseline conditions to which all other simulations were compared. The second scenario provides an estimate of the maximum flood response from the site with no mitigation strategy, whilst the third, fourth and fifth scenarios enable the effectiveness of the proposed stormwater mitigation options to be assessed. The outputs from scenarios three and four were compared with the outputs from scenario one to determine the overall potential impact of the development with the proposed stormwater mitigation strategy (refer to **Section 3.0**).

4.2 Development Area Spreadsheet Model

A stochastic spreadsheet model was developed to model the runoff response for the development allotment area. The proposed development area was divided into four regions within the model (refer to **Figure 4.1**). A unit area, within each region, was modelled. Each unit area consisted of an average sized allotment for the region and a prorated area of roadway and road reserve within the region. Each allotment was assumed to consist of:

- a typical roof area of 250 m^2;and
- a driveway area of 40 m^2 (i.e. 4 metres wide by 10 metres long).

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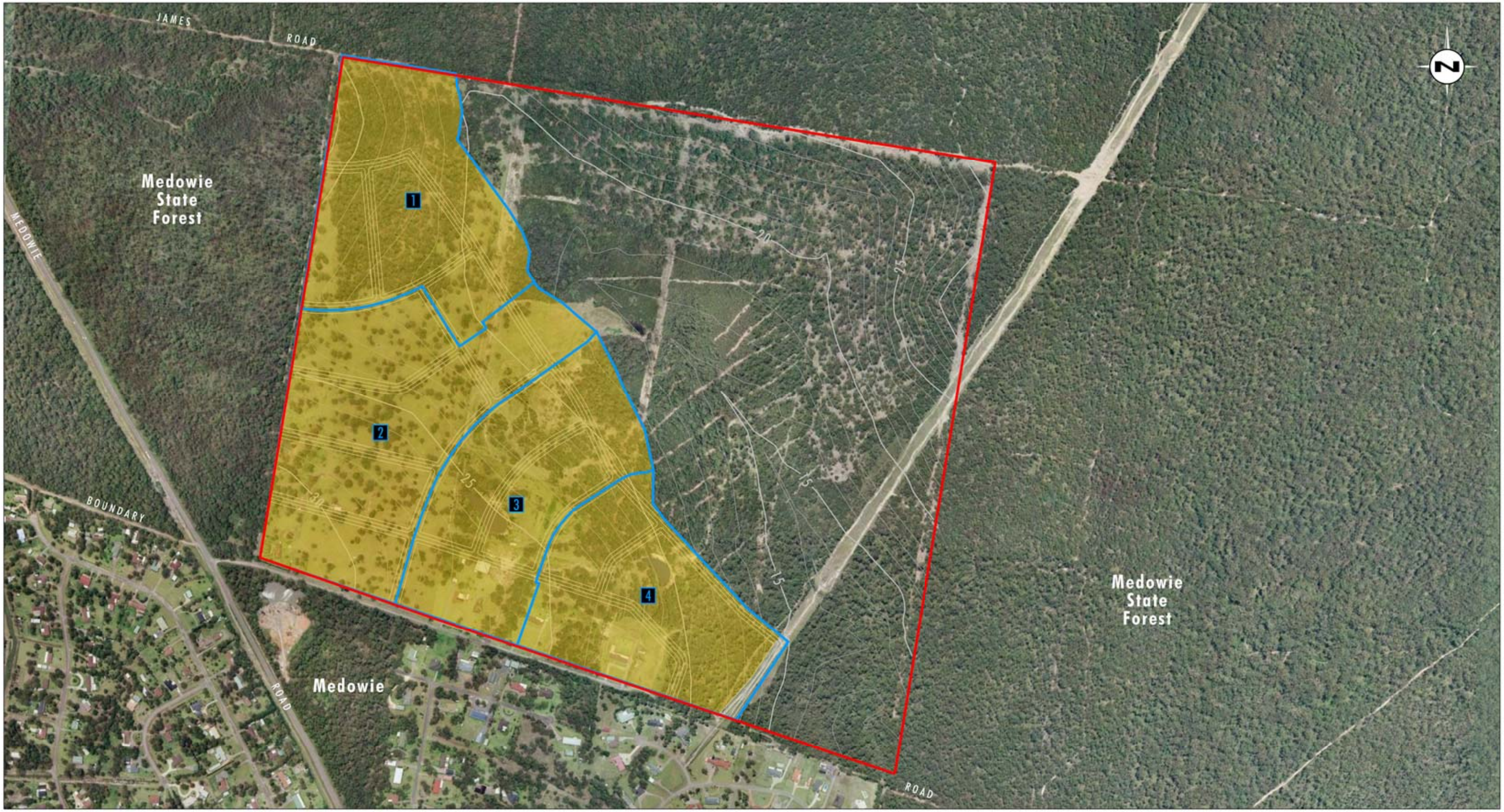


Source: Google Earth (2007), Port Stephens Council (2006), Eureka 10 (2009)
Note: Contour Interval 1m

0 100 250 500m
1:10 000

Legend
- Study Area
- Development Area Partition
- Land Use Plan

FIGURE 4.1

Partioning of Development Area

The impervious area within each unit area consists of the sum of the roof, driveway and prorated road areas. Whilst the pervious area of each unit area consists of the remaining lot area (i.e. average lot size minus roof area minus driveway area) and the prorated road reserve area. The breakup of the unit area models estimated for each region are summarised in **Table 4.1**. Where source controls, i.e. rainwater tanks are included, 75% of each roof area was assumed to drain into the rainwater tank, where it is captured and released via an outlet pipe when the tank overflows. An example of the stochastic spreadsheet model is included in **Appendix C**.

Table 4.1 – Regions Summary

Region	Region Estimates				Per Average Lot		
	Approximate Number of Lots	Area (ha)	Road Lengths		Area (m^2)	Pervious (m^2)	Impervious (m^2)
			Full Width (m)	Half Width (m)			
1	90	14.3	1353	302	1629	1230	399
2	110	18.7	1195	951	1740	1351	389
3	76	13.5	1057	944	1816	1396	420
4	60	12.5	1156	295	2123	1692	431

The unit area hydrographs for each of the four regions was multiplied by the number of lots in the region to produce a total runoff hydrograph for each region. The region hydrographs were then used as input hydrographs for the development area in the XP-Storm model (refer to **Section 4.3**).

4.3 XP-Storm Catchment Model

The XP-Storm model developed for the site (Umwelt, 2006) used data sourced from the 1:25,000 Land & Property Information Centre (LPIC) topographical maps and detailed survey of the site (refer to **Appendix A**).

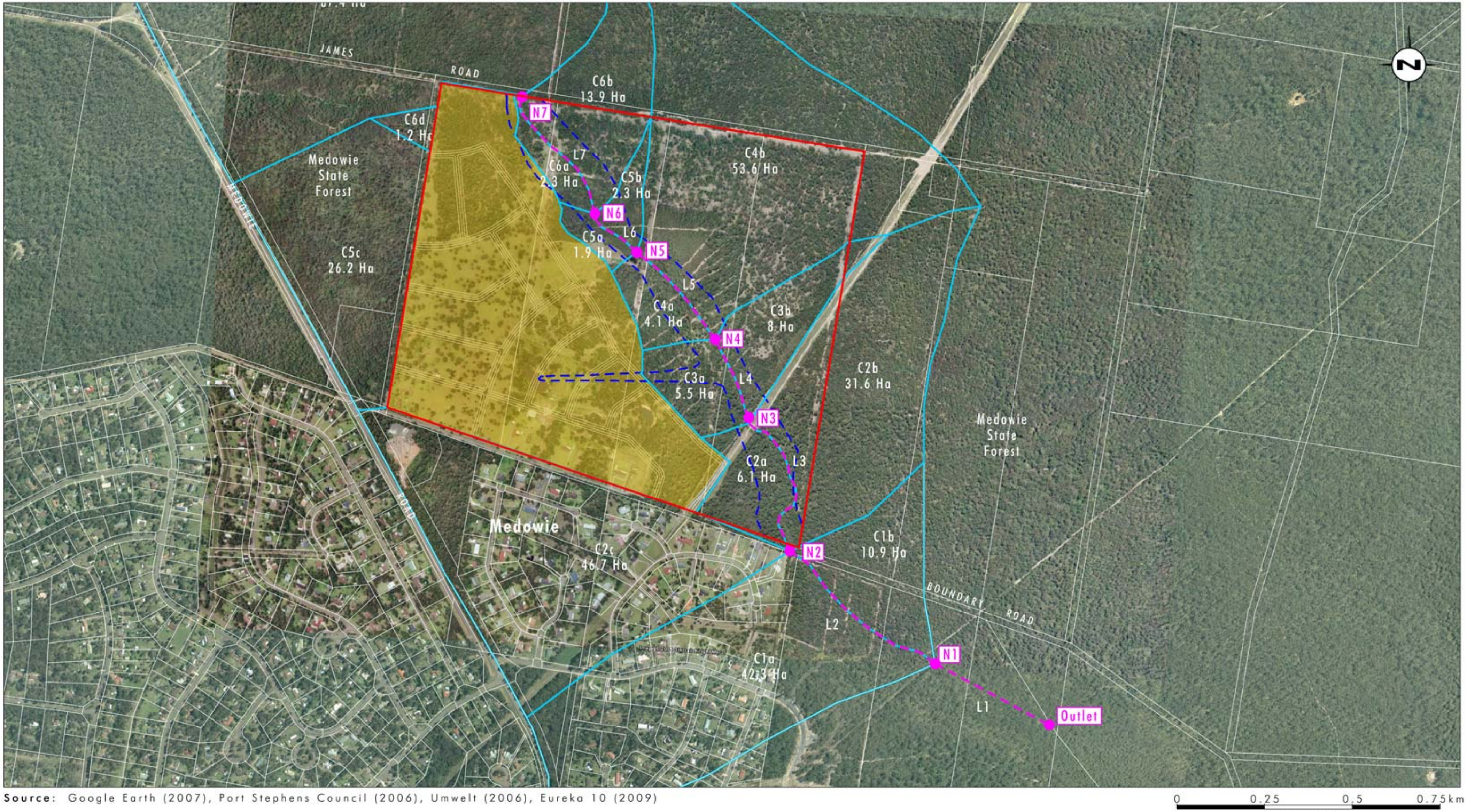
XP-Storm models a watercourse as a series of nodes and links along the drainage network. Surface runoff from subcatchments has been simulated as entering the drainage network via the nodes in the model. The hydraulic characteristics of the drainage channel reaches, including the cross-sectional shape, slope, height, depth, length and roughness (in the form of a Manning's n for both in channel and overbank flows) are described for each of the links.

Figure 4.2 shows the XP-Storm model layout used for the development area and surrounding catchment, including the delineation of the subcatchments areas outside of the development area. The relevant equations and associated parameters contained in the XP-Storm model are outlined in **Sections 4.3.1** to **4.3.3**.

4.3.1 Laurenson Equation

The Laurenson equation is a time-area routing function for the simulation of catchment runoff. It has been adopted in various models, including RORB, RAFTS and RSWM, as well as XP-Storm. The function works by subdividing a catchment into numerous sub-areas that sequentially discharge into the downstream catchment. The storage of each of these sub-areas is related to the discharge according to the equation:

$$S = BQ^{n+1} \qquad (1)$$



Source: Google Earth (2007), Port Stephens Council (2006), Umwelt (2006), Eureka 10 (2009)

Legend

- Site Boundary
- Land Use Plan
- XP-Storm Model Layout
- Catchment Boundary
- Existing 100 year ARI Flood Extent

FIGURE 4.2

XP-Storm Model Layout

Where S is the volume of storage (hours.m^3/s), Q is the discharge (m^3/s), B is a storage delay time coefficient and n is a storage non-linearity exponent. The default value of n is 0.285, which is used throughout this assessment. The value of B is determined by XP-Storm based on the catchment area, urbanised fraction and catchment slope.

4.3.2 Horton Infiltration

The Horton infiltration model is an empirically derived exponential decay function describing the changing infiltration loss rate of a catchment during a large storm event. Any difference between rainfall and the computed infiltration rate becomes the surface runoff that is routed by the Laurenson Equation described above. This flow is termed Hortonian or infiltration excess overland flow.

The Horton Infiltration function is:

$$F_P(t) = F_C + (F_0 + F_C)\, e^{-kt} \qquad (2)$$

Where $F_P(t)$ is the infiltration rate (mm/hr) at time t (s), determined from F_0 the maximum infiltration rate (mm/hr) and F_C the minimum infiltration rate (mm/hr), with a decay coefficient k (s^{-1}).

For this project, values of 25.4 mm/hr and 1.27 mm/hr were adopted for the maximum minimum infiltration rates respectively, with a decay coefficient of 0.002 s^{-1}. These parameters were used as they are indicative of the infiltration behaviour typical for this area.

4.3.3 Manning's Roughness

Manning's roughness (symbolised as n for channel flows and n* for overland flows) accounts for the influence that the surface roughness has on the flow of over it. Values of n used in this study range from 0.045 for in-channel flow to 0.06 for overbank flow, which are a reflection of the extent of vegetation present in the channel section. The value of n* was set to 0.30, which is typical for grasslands and sparse forests.

5.0 Flooding Assessment

5.1 Pre- and Post-Development Flood Behaviour

The modelled responses for the development area for the pre and post development scenarios were updated based on the proposed site layout and associated catchment areas (refer to **Section 4.0**). The modelled average lot runoff estimated for each of the four development area regions are shown in **Appendix C**. Graphs C1 to C4 show the pre-development responses generated using the spreadsheet model, whilst Graphs C5 to C8 show the post-development response with no source controls implemented.

A summary of the peak flood flow rates, velocities and depths for the modelled catchment area for the pre-development and post-development scenario with no source controls is presented in **Table 5.1**. The modelling indicates that downstream of the development site (Link/node L1/N1), flood peaks are predicted to increase from 23.7 m^3/s to 23.8 m^3/s (0.4% increase) with no mitigation measures in place. The modelled hydrographs downstream of the site for the pre-development and post-development (with no source controls) scenarios are included in **Appendix D** in Graphs D1 and D2 respectively.

Table 5.1 – Comparison of 100 year ARI Storm Event Modelled Flood Response for the Pre- and Post-Development Conditions with No Mitigation Strategy

Link / Node	Maximum Flow (m^3/s)			Maximum Velocity (m/s)			Maximum Depth (m)		
	Pre	Post	Δ	Pre	Post	Δ	Pre	Post	Δ
L1 / N1	23.7	23.8	0.4%	0.44	0.44	0.2%	0.45	0.45	0.0%
L2 / N2	19.0	19.1	0.5%	0.46	0.46	0.2%	0.32	0.32	0.0%
L3 / N3	13.5	13.6	0.7%	0.46	0.46	0.2%	0.28	0.28	0.0%
L4 / N4	11.7	11.8	0.7%	0.63	0.63	0.2%	0.61	0.61	0.0%
L5 / N5	8.5	8.6	0.6%	0.76	0.76	0.3%	0.64	0.64	0.0%
L6 / N6	4.9	4.9	0.5%	0.76	0.76	0.1%	0.41	0.41	0.0%
L7 / N7	3.4	3.4	0.0%	0.37	0.37	0.3%	0.37	0.37	0.0%

Note: Δ = difference calculated is (post – pre) / pre as percentage

Modelling indicates (refer to **Table 5.1**) that peak flood flows, flood depths and velocities would generally increase throughout the site with no mitigation strategy. This is to be expected as the increased impervious area of the post-development site will reduce infiltration losses and surface storage effects that naturally reduce the flood response of a catchment.

Modelling results also indicate (refer to **Table 5.1**) that peak velocities will be generally increased throughout the development area with no mitigation strategy employed. These increases are at most 0.3% above the existing conditions, which are relatively minor and still remain below levels that would cause issues with erosion and scouring.

5.2 Proposed Flood Mitigation Strategy

5.2.1 Source Control Performance

The potential benefits of the inclusion of 5 kL rainwater tanks that are emptied by a 20 millimetre diameter outlet on the runoff response of the development area was investigated. The stochastic lot model (refer to **Appendix C**) was used to estimate the runoff response of an average lot for each of the four development area regions (refer to **Figure 4.1**). The resulting runoff response hydrographs generated for each of these four average lots are included in **Appendix C** in Graphs C9 to C12.

The catchment flood response to the source control development estimated using the XP-Storm model is summarised in **Table 5.2**. The catchment outlet hydrographs that corresponds to this simulation are included in **Appendix D**, Graph D3.

Table 5.2 – Comparison of the 100 year ARI Storm Event Modelled Flood Response Estimated for the Pre- and Post-Development Conditions with 5 kL Rainwater Tanks

Link / Node	Maximum Flow (m^3/s)			Maximum Velocity (m/s)			Maximum Depth (m)		
	Pre	**Post**	**Δ**	**Pre**	**Post**	**Δ**	**Pre**	**Post**	**Δ**
L1 / N1	23.7	23.4	-1.1%	0.44	0.44	-0.5%	0.45	0.45	0.0%
L2 / N2	19.0	18.8	-1.6%	0.46	0.46	-0.9%	0.32	0.32	0.0%
L3 / N3	13.5	13.1	-2.7%	0.46	0.46	-0.9%	0.28	0.28	0.0%
L4 / N4	11.7	11.5	-1.6%	0.63	0.63	-0.6%	0.61	0.61	0.0%
L5 / N5	8.5	8.4	-1.4%	0.76	0.75	-0.4%	0.64	0.64	0.0%
L6 / N6	4.9	4.8	-0.9%	0.76	0.76	-0.4%	0.41	0.41	0.0%
L7 / N7	3.4	3.4	0.0%	0.37	0.37	0.0%	0.37	0.37	0.0%

Note: Δ = difference calculated is (post – pre) / pre as percentage

Modelling indicates (refer to **Table 5.2**) that the proposed source controls of 5 kL rainwater tanks are capable of reducing the modelled catchment outlet flood peak to 1.1% less than the pre-development peak downstream of the site, with similarly predicted decreases in the peak velocity, whilst flood depth remains unchanged. Throughout the development area, the peak flood flows are estimated to be reduced to at a minimum equal the pre-development flows, compared to the increases estimated with no source controls (refer to **Table 5.1**). The adoption of 5 kL rainwater tanks was also found to be able to reduce the peak velocities throughout the modelled area to values that are equal to or less than the pre-development values.

Based on these results, it is considered that the adoption of source controls as a flood mitigation strategy in the form of 5 kL rainwater tanks at each lot is, for the most part, capable of meeting the Council design requirements.

5.2.2 Outlet Control Performance

The proposed dry detention basin located on the northern edge of the vegetation buffer near the eastern boundary of the development area has a catchment area of approximately 18.2 hectares (refer to **Figure 3.1**). The runoff captured by the detention basin is proposed to be slowly released through a single 600 millimetre diameter pipe. This proposed basin will attenuate runoff for a large section of the total catchment area that would otherwise contribute to the downstream flood peaks, effectively offsetting the increases expected upstream in the development area. The catchment area for this conceptual dry detention basin is located mostly off site to the east of the development area.

The proposed basin embankment consists of a contour bank approximately 205 metres long, raised to a level of approximately 1.1 metres above the present ground level (i.e. a top of bank elevation of approximately 15.1 mAHD based on the location proposed in **Figure 3.1**). The proposed embankment height allows for containment of the peak storage depth, behind the embankment wall during a 100 year ARI 9 hour storm event with a free board of 0.5 metres.

As no source controls are adopted under this scenario, the allotment area runoff is the same as that for the post-development (no source controls) scenario (refer to **Section 5.1**). The modelled impact of the proposed dry detention basin on the flows through the unnamed tributary within the site are summarised in **Table 5.3**. The catchment outlet hydrograph generated by the XP-Storm model is included in **Appendix D** in Graph D4.

Table 5.3 – Comparison of the 100 year ARI Storm Event Flood Response Estimated for the Pre- and Post-Development Conditions with a Single Detention Basin with One Outlet Pipe

Link / Node	Maximum Flow (m^3/s)			Maximum Velocity (m/s)			Maximum Depth (m)		
	Pre	Post	Δ	Pre	Post	Δ	Pre	Post	Δ
L1 / N1	23.7	23.3	-1.6%	0.44	0.44	-0.7%	0.45	0.45	0.0%
L2 / N2	19.0	18.7	-2.0%	0.46	0.46	-1.1%	0.32	0.32	0.0%
L3 / N3	13.5	13.3	-1.0%	0.46	0.46	-1.1%	0.28	0.28	0.0%
L4 / N4	11.7	11.6	-0.9%	0.63	0.63	-0.3%	0.61	0.61	0.0%
L5 / N5	8.5	8.5	-0.5%	0.76	0.76	-0.1%	0.64	0.64	0.0%
L6 / N6	4.9	4.8	-0.5%	0.76	0.76	-0.3%	0.41	0.41	0.0%
L7 / N7	3.4	3.4	0.0%	0.37	0.37	-0.3%	0.37	0.37	0.0%

Note: Δ = difference calculated is (post – pre) / pre as percentage

Modelling (refer to **Table 5.3**) indicates that the proposed dry detention basin has the capacity to reduce the post-development flood flows, velocities and depths to values that are less than or equal to the pre-development conditions downstream of the site. As the proposed basin lies downstream of the proposed development area of the site the basin will provide no additional benefit in mitigating post-development flows within the site to those estimated for the no mitigation strategy.

Whilst the dry detention basin does not improve the modelled post-development flood flows expected through the majority of the site, modelling indicates that the proposed dry detention basin would reduce the downstream flows to pre-development conditions.

A further advantage of a dry detention basin would be the removal of nutrients entering the drainage system. As surface runoff is intercepted by the detention basin, nutrient laden sediment would settle, therefore reducing the amount of nutrients leaving the catchment and further improving the downstream condition of the catchment by improving water quality.

5.2.3 Combined Source and Sink Controls

Analysis indicates that either source controls or a dry detention basin could be used to ensure compliance with Port Stephens Council requirements. Alternatively, a combined approach, consisting of both a detention basin and source controls could be used to produce further improvements to the downstream flood conditions.

The catchment flood response to the combined source and outlet controlled development estimated using the XP-Storm model is summarised in **Table 5.4**. The catchment outlet hydrograph generated by the XP-Storm model is included in **Appendix D** in Graph D5.

Table 5.4 – Comparison of the 100 year ARI Storm Event Flood Response Estimated for the Pre- and Post-Development Conditions with 5 kL Rainwater Tanks and a Single Detention Basin with One Outlet Pipe

Link / Node	Maximum Flow (m^3/s)			Maximum Velocity (m/s)			Maximum Depth (m)		
	Pre	Post	Δ	Pre	Post	Δ	Pre	Post	Δ
L1 / N1	23.7	22.7	-4.2%	0.44	0.43	-1.8%	0.45	0.44	-2.2%
L2 / N2	19.0	18.0	-5.3%	0.46	0.45	-2.6%	0.32	0.31	-3.1%
L3 / N3	13.5	13.1	-2.6%	0.46	0.45	-2.6%	0.28	0.28	0.0%
L4 / N4	11.7	11.5	-1.6%	0.63	0.63	-0.6%	0.61	0.61	0.0%
L5 / N5	8.5	8.4	-1.4%	0.76	0.75	-0.4%	0.64	0.64	0.0%
L6 / N6	4.9	4.8	-0.9%	0.76	0.76	-0.4%	0.41	0.41	0.0%
L7 / N7	3.4	3.4	0.0%	0.37	0.37	0.0%	0.37	0.37	0.0%

Note: Δ = difference calculated is (post – pre) / pre as percentage

Modelling the influence that the combined source and outlet stormwater controls (refer to **Table 5.4**) indicates that the post-development flows downstream of the site could be reduced by approximately 4.2% below the present conditions. Similar reductions in the downstream flow velocity (1.8%) and flood depth (2.2%) could also potentially be achieved.

The use of a combined mitigation strategy would also ensure that flood flows, velocities and depth throughout the development area are not increased substantially above the pre-development conditions. It would also provide an inherent security through the provision of isolated, redundant flood controls, against failure and changes to the current conditions that may arise as a result of further site development, or changes to the prevailing meteorological conditions due to climate change.

A combined approach to stormwater mitigation would provide additional flexibility in the size requirements of the two control measures. The installation of the rainwater tanks would reduce the size requirement of the dry detention basin capacity, meaning that the embankment could be made shorter and lower. Conversely, as the dry detention basin provides sufficient downstream flood mitigation to meet with Councils requirements, the required rainwater tank sizing and outlet design could also be adjusted to better fit with other design requirements, and offer greater flexibility in water harvesting and usage.

6.0 Discussion and Conclusions

This assessment has demonstrated both that source controls and outlet controls are capable of meeting the requirements of Port Stephens Council that post-development runoff downstream of the development area is limited to the pre-development (i.e. natural) levels.

The implementation of a traditional dry detention dam is expected to provide sufficient flood attenuation to limit the downstream post-development peak flood flows, velocities and water depths to the estimated pre-development levels, with peak flows being reduced by 1.6% compared to the pre-development conditions. However, due to the proposed location of the dry detention basin downstream of the allotment area, the proposed basin will not reduce peak flows through the site. The proposed dry detention basin could also improve the downstream water quality by assisting the removal of sediment and the nutrients often carried by the stormwater runoff. The runoff water intercepted by the detention basin would be temporarily stored, giving time for any mobilised sediment carried to settle out of the runoff water before being released downstream.

Source controls in the form of 5 kL rainwater tanks were shown to reduce the peak flood flows throughout the development area to levels that were equivalent to the pre-development values, with the catchment outlet peak estimated to be 1.1% less than the pre-development value. Peak velocities and flood depths throughout the development area were shown to all be approximately equal to the pre-development values.

These two options represent two different but potentially complementary flood mitigation solutions for the site. It is expected that a combination of the two methods would exceed the Council design requirements for the peak flows leaving the development area, with peak flood flow reductions of approximately 4% below the pre-development flows expected. It would also introduce additional security due to the parallel management methods, and design flexibility by relaxing the design requirements for both the dry detention basin and rainwater tanks. In particular, a combined approach would allow for the primary use of the rainwater tanks to shift from flood mitigation to the capture and storage of rainwater for reuse on site. This approach could allow for a better integration with other design requirements of the site, further improving the amenity of the area.

The proposed 5 kL of detention volume in the rainwater tanks is small and provides an opportunity for dual-purpose rainwater tanks to be installed (i.e. to include storage component). Installing a larger rainwater tank (for example 10 kL) and placing the low-flow outlet at a height to maintain a potential harvest volume sufficient for stormwater mitigation, leaving the balance of the tank volume available for rainwater storage and re-use on site as part of a WSUD strategy.

7.0 References

Landcom, 2004. *Managing Urban Stormwater: Soils and Construction*.

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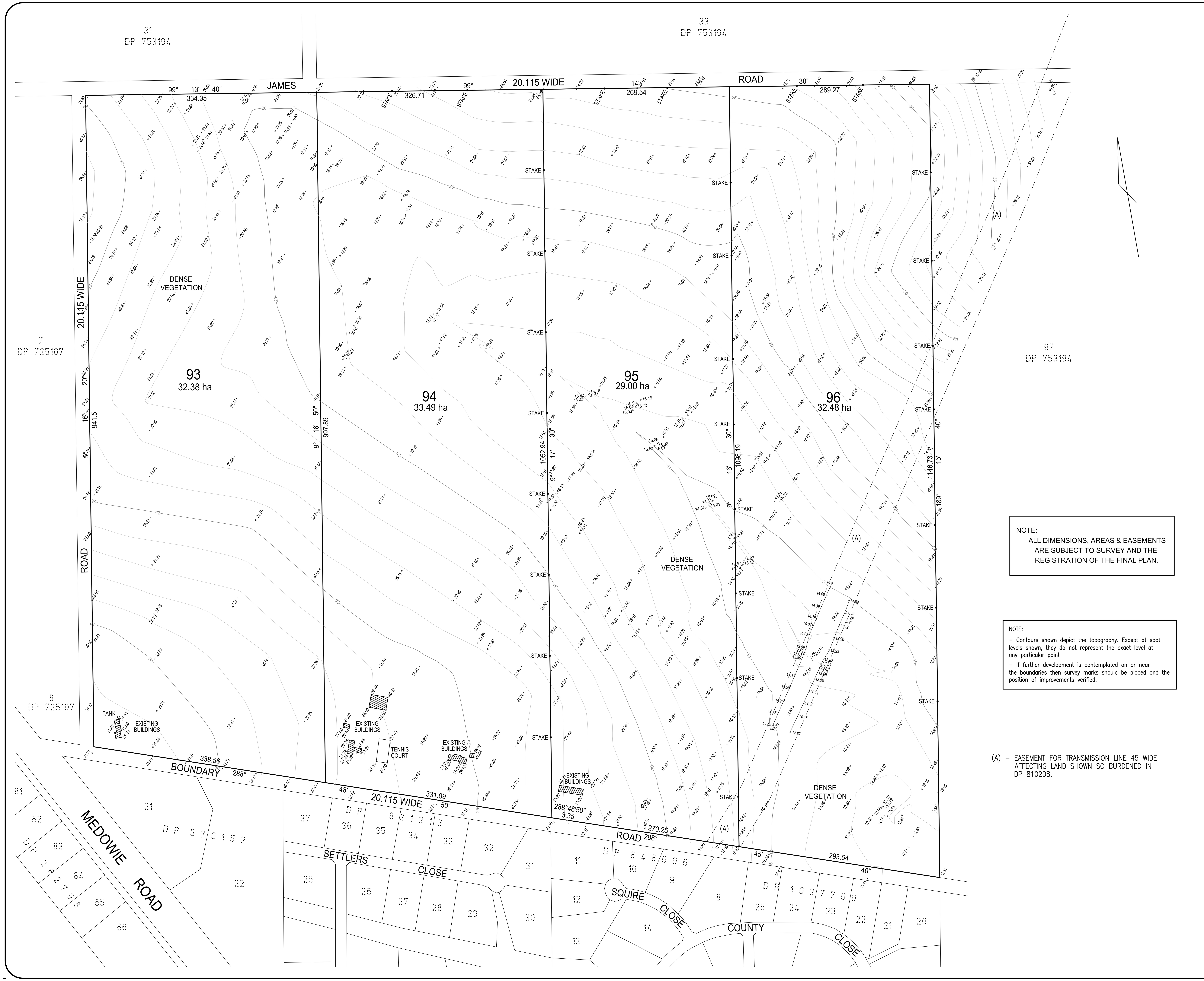
Umwelt, 2006. *Drainage Study of Proposed North Medowie Residential Project, Boundary Road, Medowie.* Prepared for Eureka 1 Project 10 Pty Limited.

Umwelt, 2009. *Proposed North Medowie Residential Project Boundary Road, Medowie, Flood and Drainage Assessment.* Prepared for Eureka 1 Project 10 Pty Limited.

APPENDIX A

Site Survey by Pulver Cooper & Blackley



50 25 0 50 100 150

Horizontal Scale 1:2500

INSTRUCTION NUMBER: 06/53
COMPUTER FILE: 0653_DETAIL_01_A.dwg
SURVEYED: LJB/KV
DESIGNED: ___
DRAWN: MAC
CHECKED: BDK
DATUM: Australian Height Datum
CONTOUR INTERVAL: 1.0m

REVISIONS	DATE
BUILDINGS & STAKES PLCD	26/06/06
INITIAL ISSUE	15/06/06

CLIENT:

BUILDEV GROUP

PCB
PULVER COOPER & BLACKLEY PTY. LTD.
SURVEYORS
TOWNPLANNERS
CIVIL ENGINEERS
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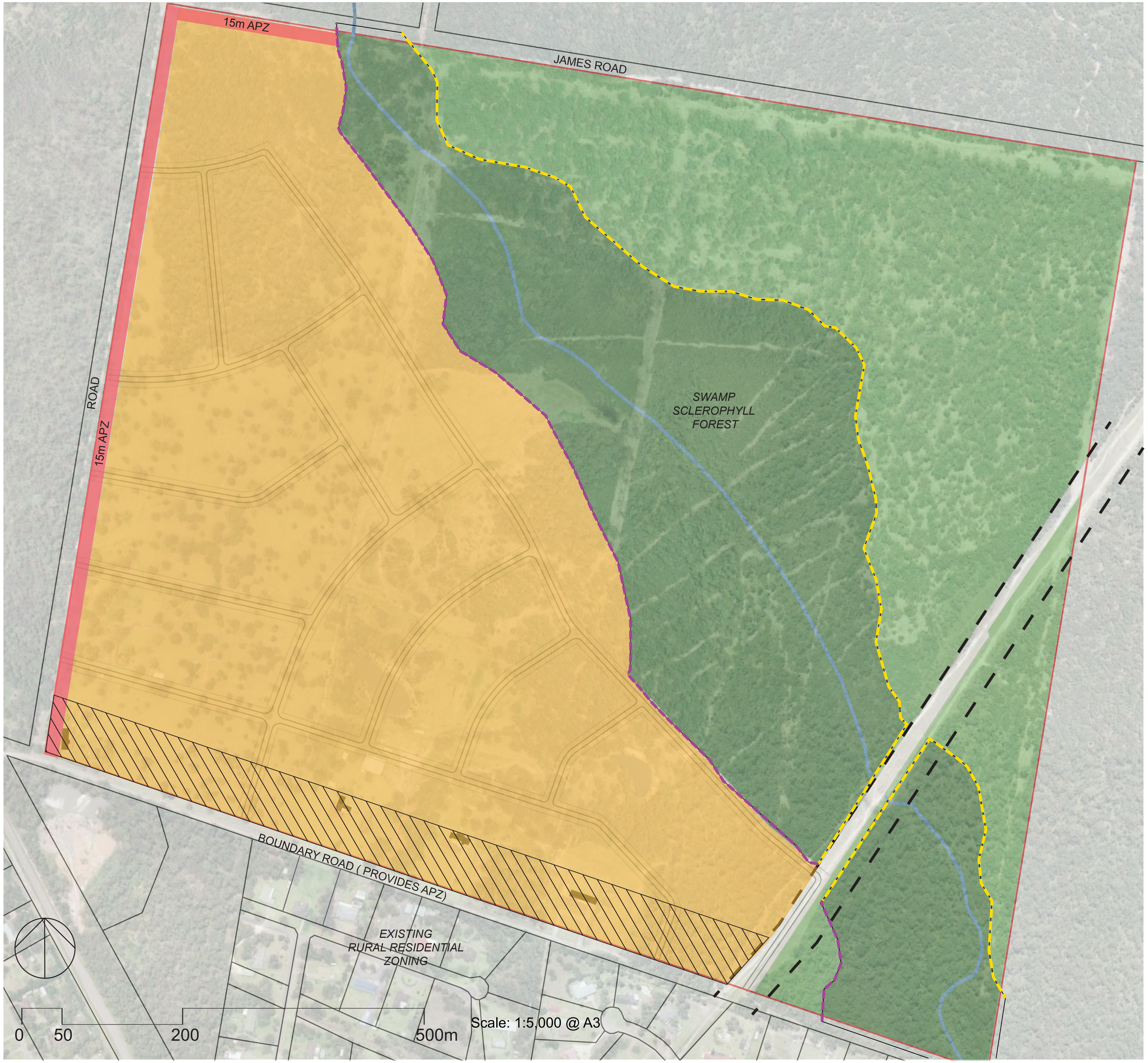
PLAN OF DETAIL SURVEY
OF LOTS 93, 94, 95 & 96
DP 753194
BOUNDARY ROAD
MEDOWIE

SHEET 1 OF 1 SHEETS

APPENDIX B

Proposed Site Layout Plans by Urbis

N O R T H   M E D O W I E



LEGEND

- EEC Boundary to undeveloped land
- Smoothed EEC boundary as confirmed with Council's offices
- Easement
- Drainage
- EEC
- Undeveloped Land
- APZ
- Existing Building
- Rural Small Holdings (1,000 - 1,500sqm) as per Council's Medowie Strategy
- Larger lots to be provided reflecting a similar frontage width and presentation to Boundary Road as those fronting the southern side of Boundary Road.

NOTES

1. Precise north-eastern boundary of developable area within Rural Small Holding Zoning to consider edge impacts and interface solutions such as roadways in determining an appropriate buffer to EEC (buffer of 0 - 50m, depending on solutions provided).
2. Road patterns shown is indicative only and subject to consideration of subdivision DA.
3. APZ along north-eastern boundary of developable area to be determined and subject to consideration of subdivision DA.

11 November 2009

North Medowie Neighbourhood:
Land use Plan

urbis

APPENDIX C

Lot Area Runoff Model

Appendix C – Unit Area Runoff Model

1.0 Example Model

1.1 Model Input

Key:	
Inputs	
Calculated	

1. Region Total Dimensions			
Number of Lots =	90		
Total Area =	143000	(m^2)	
Reserve Area =	0	(m^2)	
Lot Area =	143000	(m^2)	
Ave. Lot Area =	1255	(m^2)	
Roads:			
Full Length =	1353.0	(m)	
Half Length =	302.0	(m)	
Road Reserve Length =	1504.0	(m)	
Road Width =	6.5	(m)	
Reserve Width =	13.5	(m)	(2 x 6.75m)
Road Area =	9776	(m^2)	
Road Reserve Area =	20304	(m^2)	

2. Typical Lot Dimensions		
House Footprint =	250	(m^2)
Driveway:		
Length =	10	(m)
Width =	4	(m)

3. Effective Areas (per lot)		
Roof area into Tank:		
Fraction =	0.75	(-)
Area =	187.5	(m^2)
Impervious Areas:		
Road =	109	(m^2)
Balance of Roof =	62.5	(m^2)
Driveway =	40	(m^2)
Total =	211	(m^2)
Pervious Areas:		
Balance of Lot =	1005	(m^2)
Road Reserve =	226	(m^2)
Total =	1230	(m^2)
Total Effective Lot Area=	1629	(m^2)

4. Design Storm Details		
ARI =	100	(years)
Selected Duration =	540	(minutes)
IFD Depth =	180.45	(mm)

5. Pervious Area Details			
Initial Loss =	20	(mm)	* AR&R, 1987, Table 6.2 - Design Loss Rates for New South Wales
Continuing Loss =	2.5	(mm/hr)	
	0.0417	(mm/min)	

6. Tank Measurements			
Diameter =	2.92	(m)	
Area =	6.70	(m^2)	
Overflow Height =	2.15	(m)	
Volume =	14.40	(m^3)	
Outlet Diameter =	0.02	(m)	
Outlet Area =	0.0003	(m^2)	
Outlet height =	0.10	(m)	
Entrance Loss Coeff. =	0.50	(-)	* ~ 0.5 for squared edge inlet; ~ 0.04 Bell-Mouthed; ~ 0.8 Reentrant

1.2 Model Calculations

Time	Rainfall			Impervious Runoffs		Pervious Runoffs
Time (minutes)	Depth (mm)	Cumulative (mm)	After Losses (mm)	To Tank (m^3)	Balance (m^3)	(m^3)
0	0	0.00	0.00	0.00	0.000	0.000
1	1	0.14	0.14	0.00	0.027	0.030
2	2	0.14	0.29	0.00	0.027	0.030
3	3	0.14	0.43	0.00	0.027	0.030
4	4	0.14	0.58	0.00	0.027	0.030
5	5	0.14	0.72	0.00	0.027	0.030
6	6	0.14	0.87	0.00	0.027	0.030
7	7	0.14	1.01	0.00	0.027	0.030
8	8	0.14	1.15	0.00	0.027	0.030
9	9	0.14	1.30	0.00	0.027	0.030
10	10	0.14	1.44	0.00	0.027	0.030
11	11	0.14	1.59	0.00	0.027	0.030
12	12	0.14	1.73	0.00	0.027	0.030
13	13	0.14	1.88	0.00	0.027	0.030

Time	Tank					
Time (minutes)	Depth (m)	Outlet (m^3)	Overflow (m^3)	Outflow (m^3)	Depth (m)	Volume (m^3)
0	0.000	0.000	0.000	0.000	0.000	0.000
1	0.004	0.000	0.000	0.000	0.004	0.027
2	0.008	0.000	0.000	0.000	0.008	0.054
3	0.012	0.000	0.000	0.000	0.012	0.081
4	0.016	0.000	0.000	0.000	0.016	0.108
5	0.020	0.000	0.000	0.000	0.020	0.135
6	0.024	0.000	0.000	0.000	0.024	0.162
7	0.028	0.000	0.000	0.000	0.028	0.189
8	0.032	0.000	0.000	0.000	0.032	0.217
9	0.036	0.000	0.000	0.000	0.036	0.244
10	0.040	0.000	0.000	0.000	0.040	0.271
11	0.044	0.000	0.000	0.000	0.044	0.298
12	0.049	0.000	0.000	0.000	0.049	0.325
13	0.053	0.000	0.000	0.000	0.053	0.352

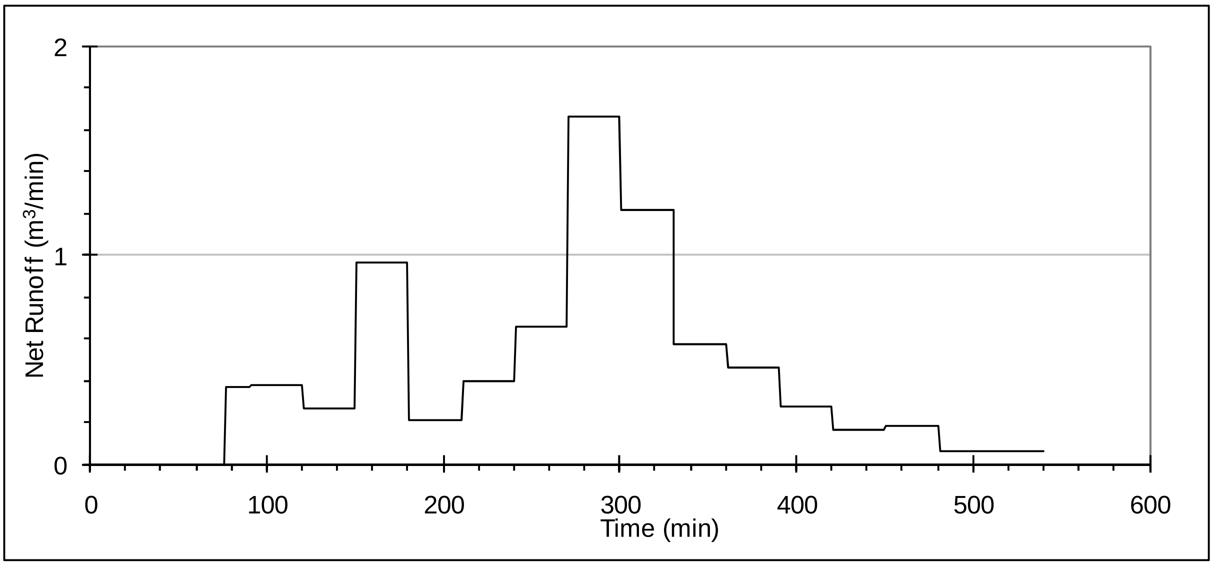
Time	Unit area Net Runoff			Unit area Cumulative Runoff		
Time (minutes)	Tank (m^3)	No Tank (m^3)	Natural (m^3)	Tank (m^3)	No Tank (m^3)	Natural (m^3)
0	0.000	0.000	0.000	0.000	0.000	0.000
1	0.030	0.058	0.000	0.030	0.058	0.000
2	0.030	0.058	0.000	0.061	0.115	0.000
3	0.030	0.058	0.000	0.091	0.173	0.000
4	0.030	0.058	0.000	0.122	0.230	0.000
5	0.030	0.058	0.000	0.152	0.288	0.000
6	0.030	0.058	0.000	0.183	0.345	0.000
7	0.030	0.058	0.000	0.213	0.403	0.000
8	0.030	0.058	0.000	0.244	0.460	0.000
9	0.030	0.058	0.000	0.274	0.518	0.000
10	0.030	0.058	0.000	0.305	0.575	0.000
11	0.030	0.058	0.000	0.335	0.633	0.000

Time (minutes)	Unit area Net Runoff Tank (m^3)	No Tank (m^3)	Natural (m^3)	Unit area Cumulative Runoff Tank (m^3)	No Tank (m^3)	Natural (m^3)
12	0.030	0.058	0.000	0.366	0.691	0.000
13	0.030	0.058	0.000	0.396	0.748	0.000

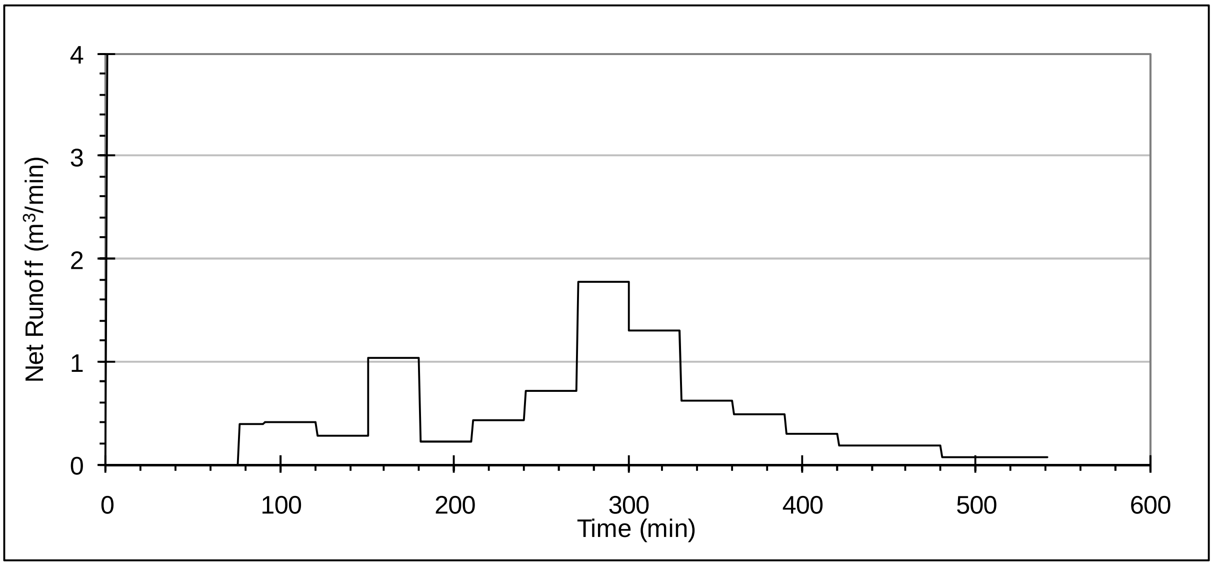
Time (minutes)	Region Total Net Runoff Tank (m^3)	No Tank (m^3)	Natural (m^3)	Region Total Cumulative Runoff Tank (m^3)	No Tank (m^3)	Natural (m^3)
0	0.000	0.000	0.000	0.000	0.000	0.000
1	2.743	5.179	0.000	2.743	5.179	0.000
2	2.743	5.179	0.000	5.486	10.358	0.000
3	2.743	5.179	0.000	8.229	15.537	0.000
4	2.743	5.179	0.000	10.972	20.716	0.000
5	2.743	5.179	0.000	13.715	25.895	0.000
6	2.743	5.179	0.000	16.458	31.074	0.000
7	2.743	5.179	0.000	19.201	36.253	0.000
8	2.743	5.179	0.000	21.944	41.432	0.000
9	2.743	5.179	0.000	24.687	46.612	0.000
10	2.743	5.179	0.000	27.430	51.791	0.000
11	2.743	5.179	0.000	30.173	56.970	0.000
12	2.743	5.179	0.000	32.916	62.149	0.000
13	2.743	5.179	0.000	35.659	67.328	0.000

2.0 Results

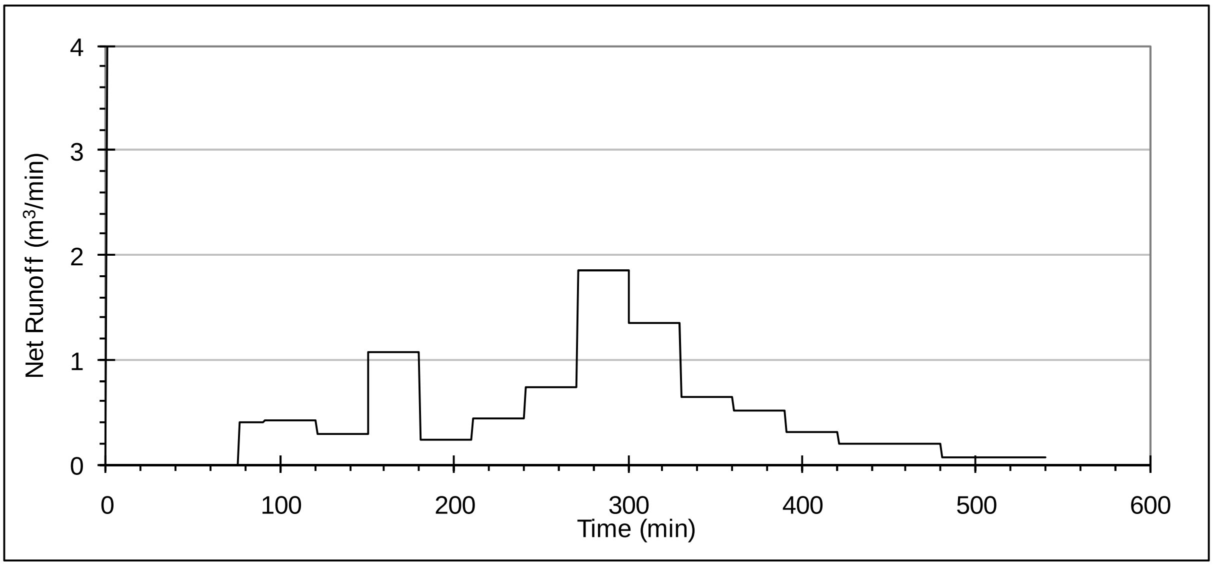
2.1 Pre-Development (100 year ARI storm event)



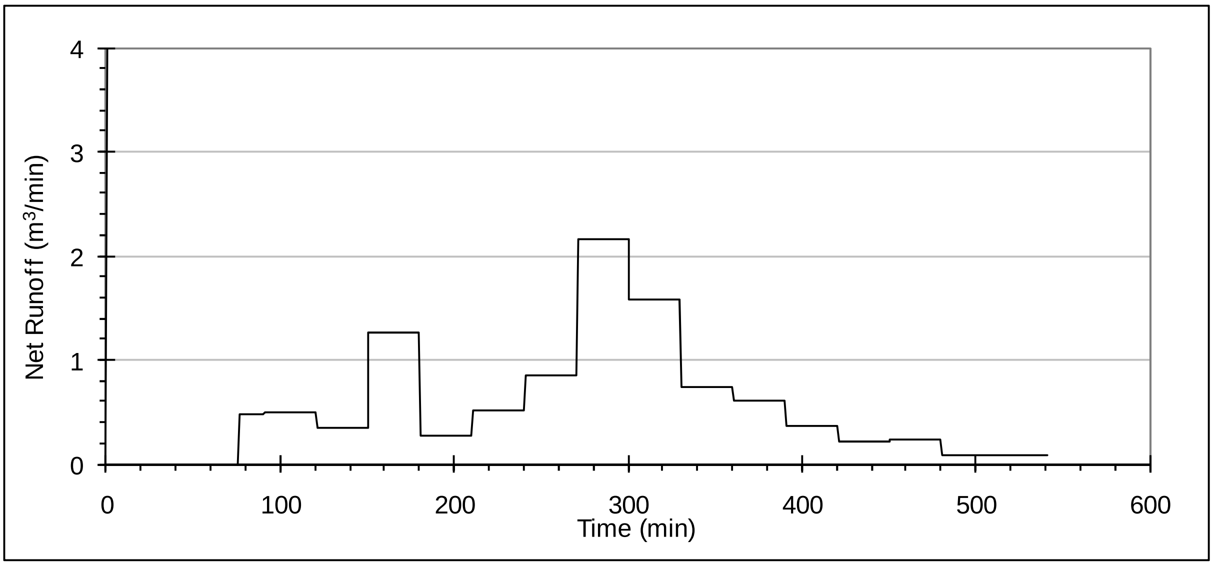
Graph C1 – Pre-development unit area runoff hydrograph for Region 1



Graph C2 – Pre-development unit area runoff hydrograph for Region 2

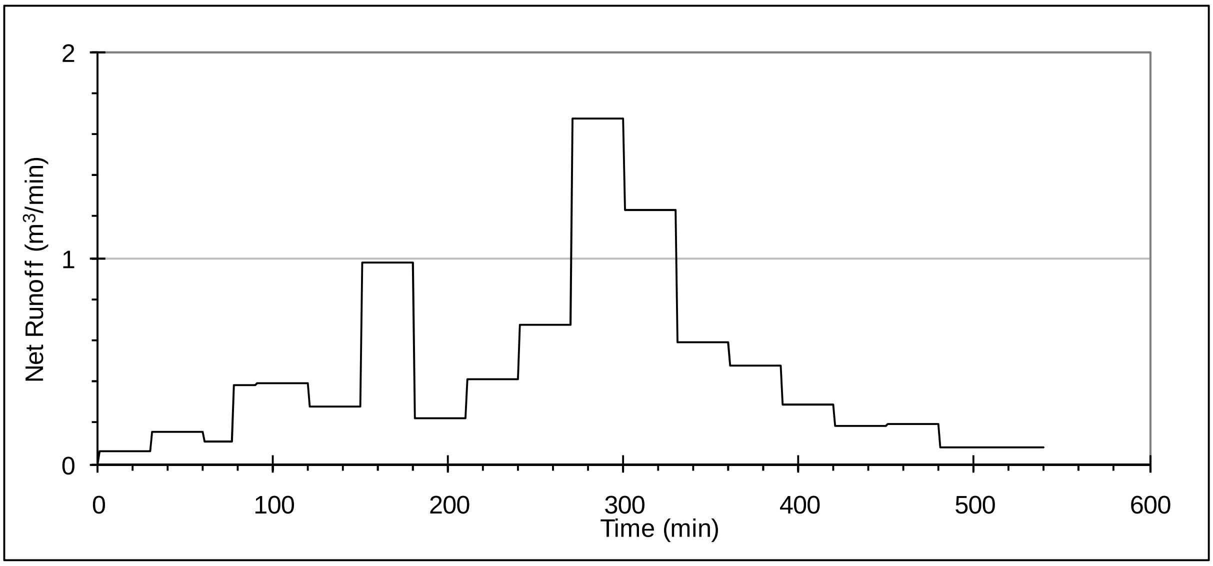


Graph C3 – Pre-development unit area runoff hydrograph for Region 3

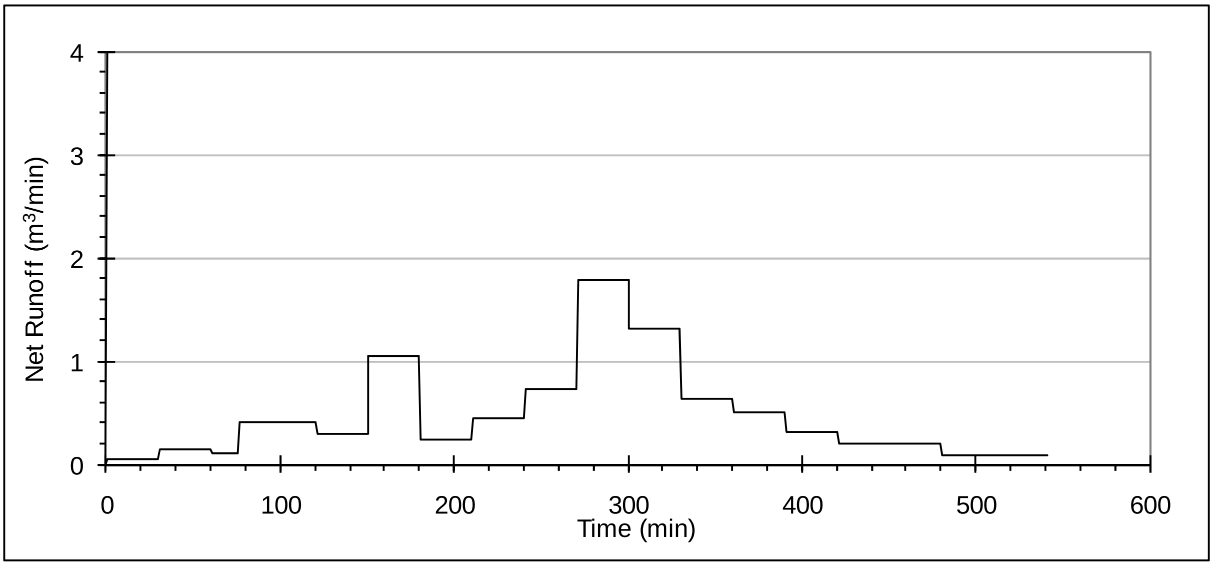


Graph C4 – Pre-development unit area runoff hydrograph for Region 4

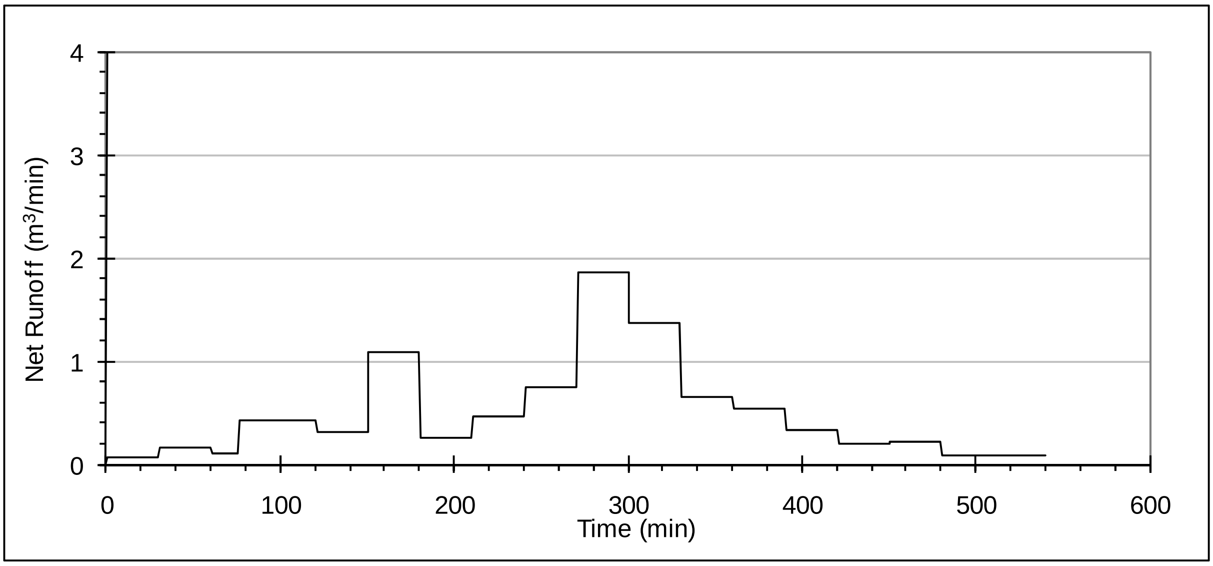
2.2 Proposed Development with No Mitigation (100 year ARI storm event)



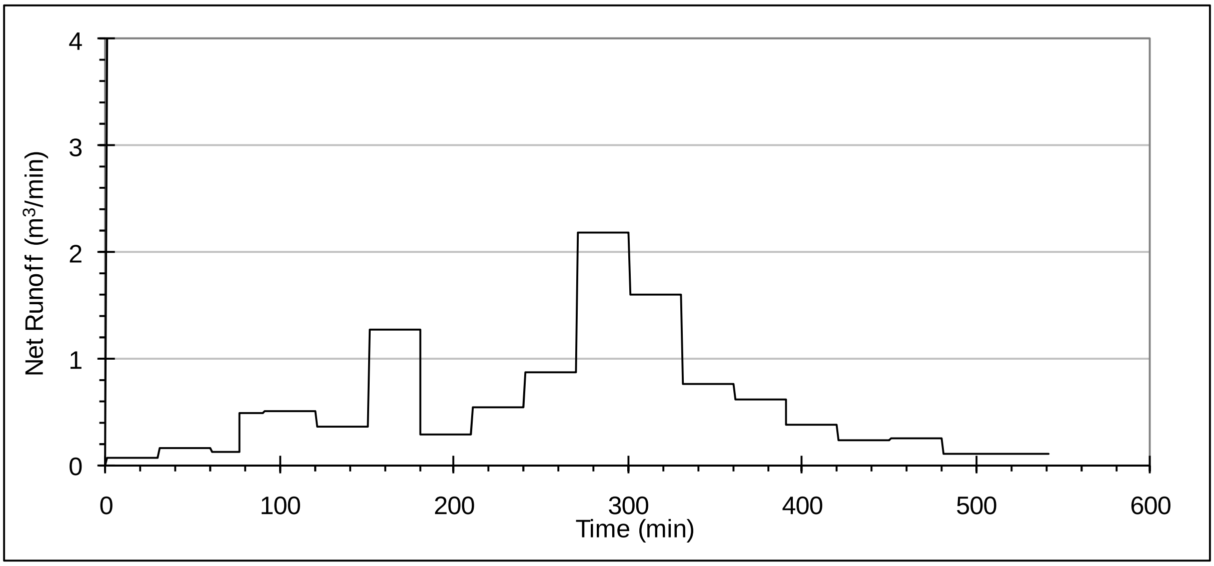
Graph C5 – Post-development unit area runoff hydrograph for Region 1



Graph C6 – Post-development unit area runoff hydrograph for Region 2

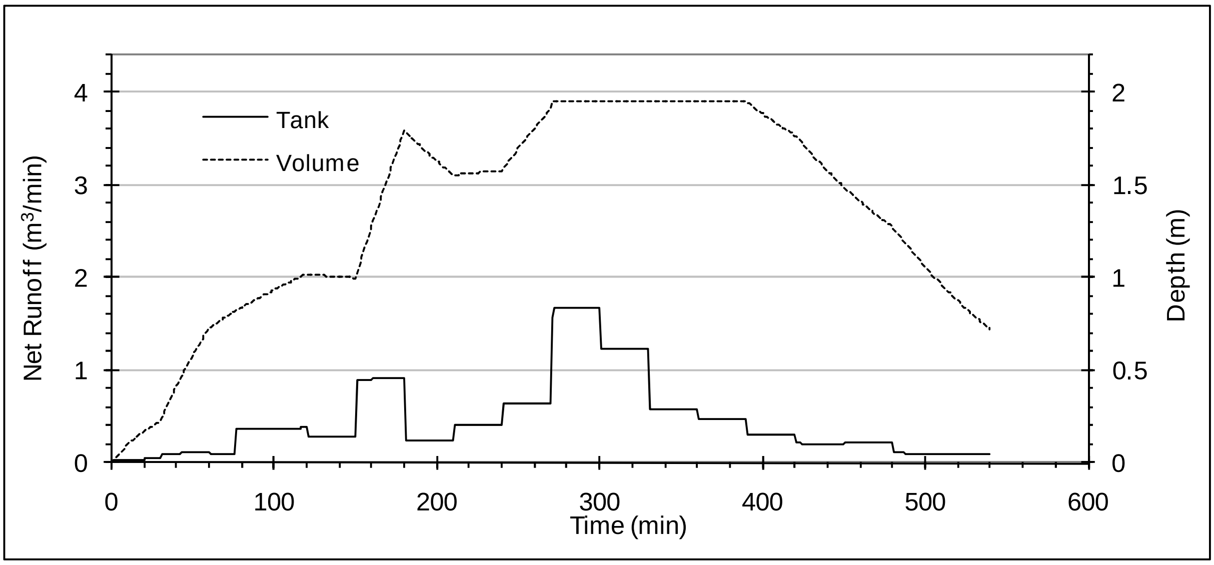


Graph C7 – Post-development unit area runoff hydrograph for Region 3

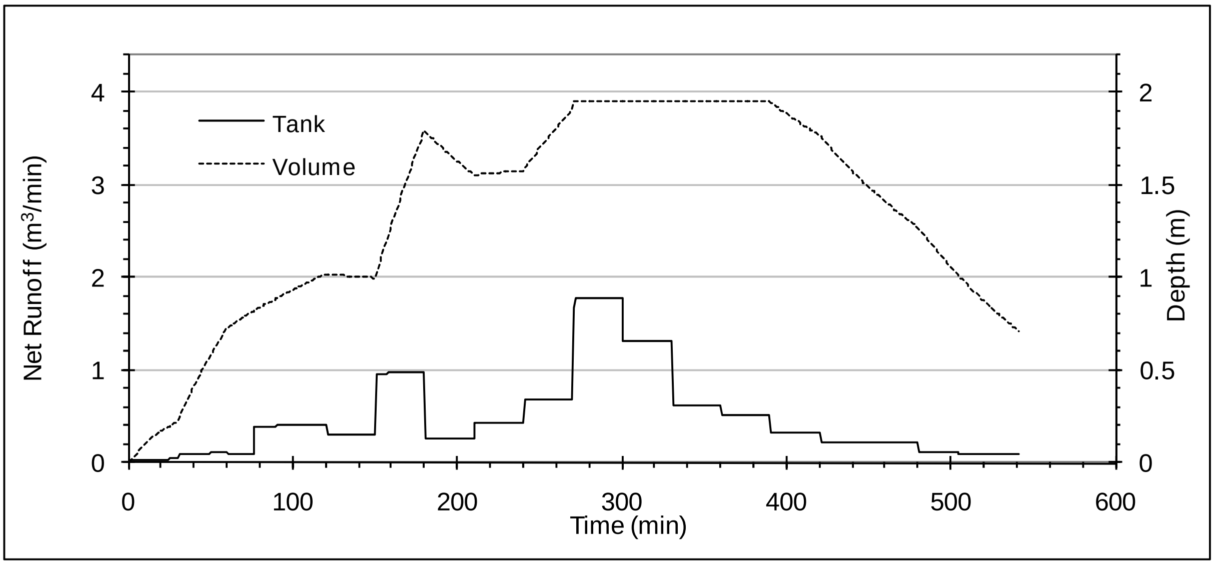


Graph C8 – Post-development unit area runoff hydrograph for Region 4

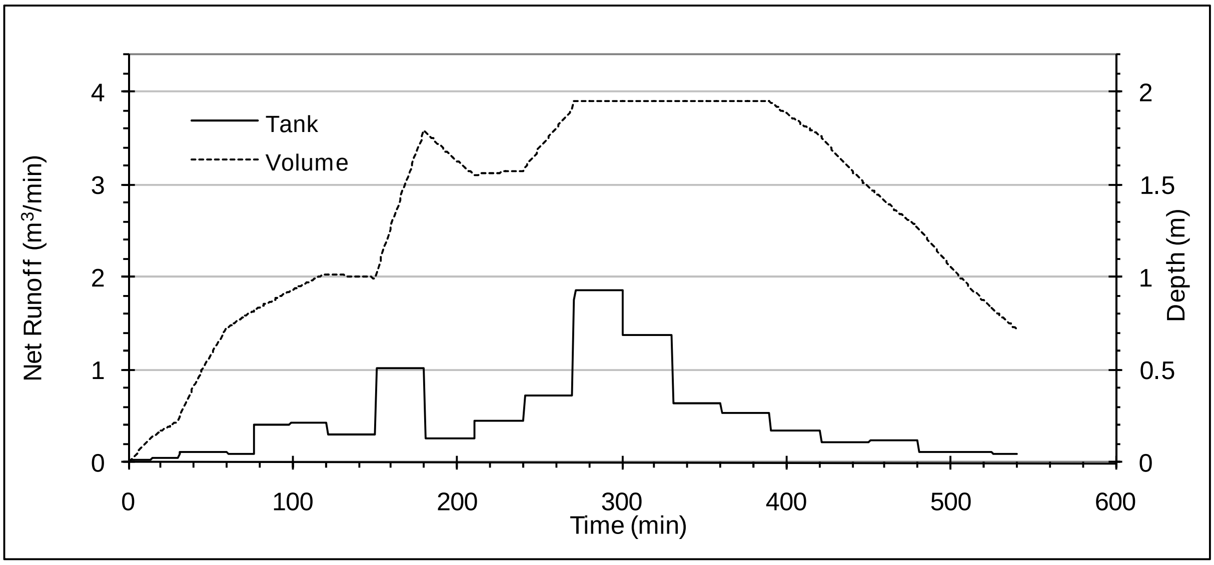
2.3 Proposed Development with 5 kL Rainwater Tanks (100 year ARI storm event)



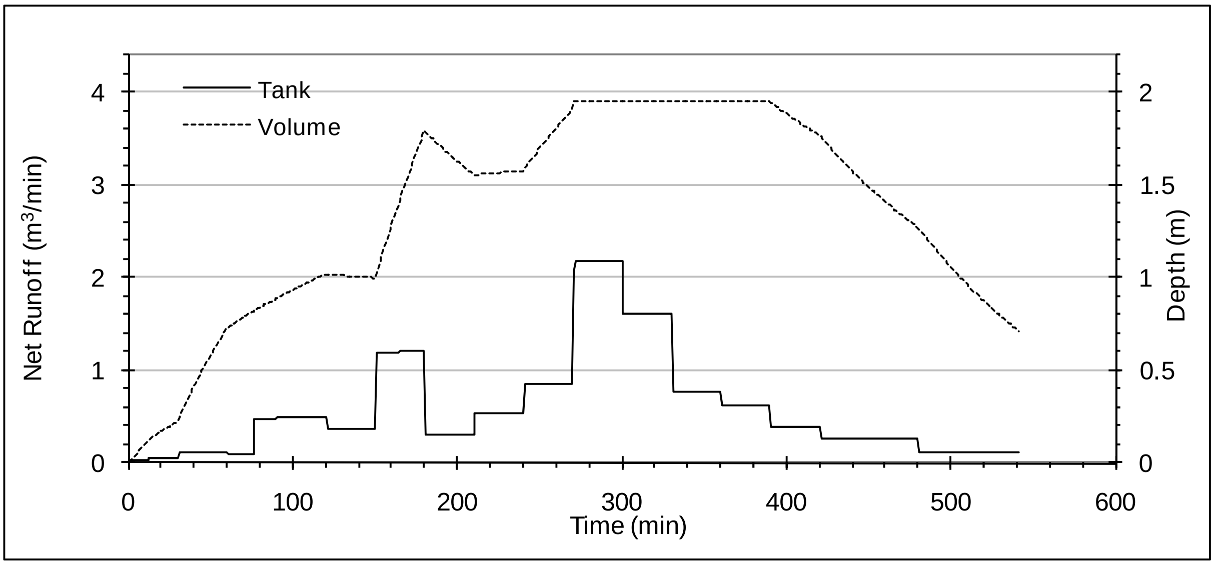
Graph C9 – Post-development unit area runoff hydrograph with 5 kL rainwater tanks for Region 1



Graph C10 – Post-development unit area runoff hydrograph with 5 kL rainwater tanks for Region 2



Graph C11 – Post-development unit area runoff hydrograph with 5 kL rainwater tanks for Region 3



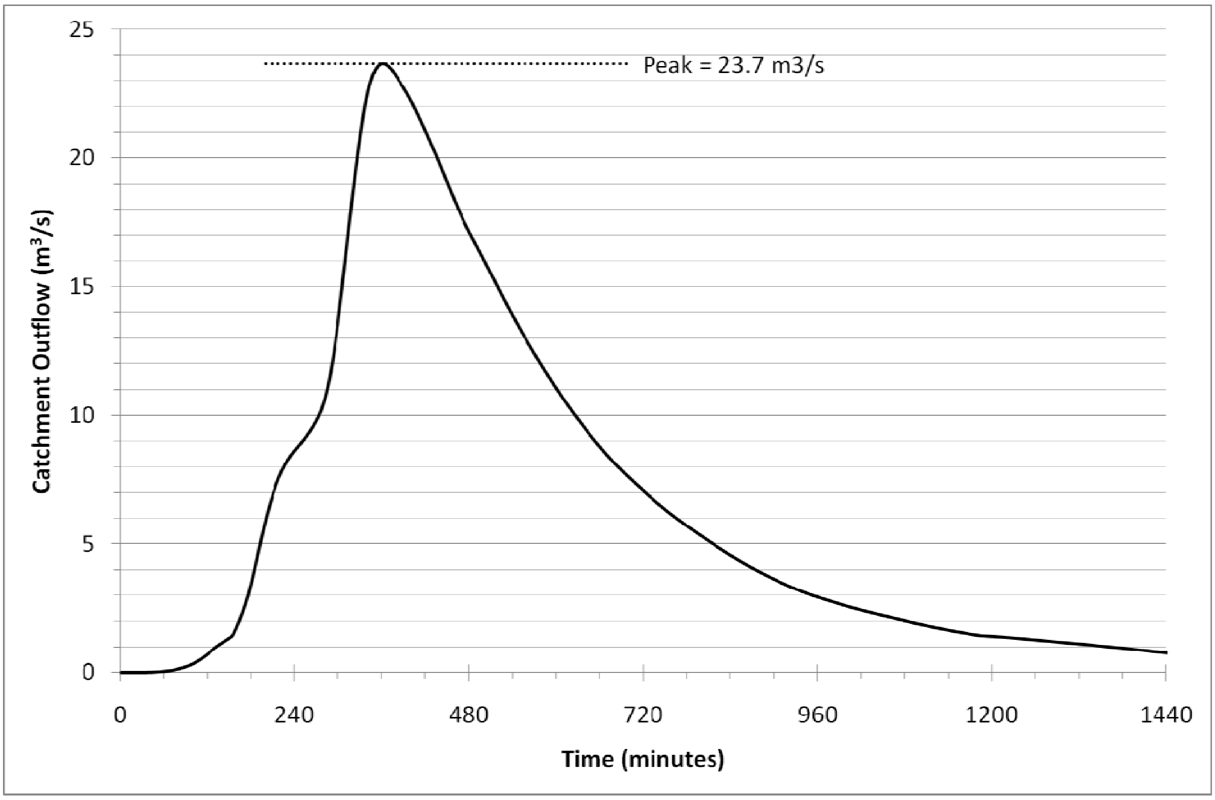
Graph C12 – Post-development unit area runoff hydrograph with 5 kL rainwater tanks for Region 4

APPENDIX D

XP-Storm Hydrographs

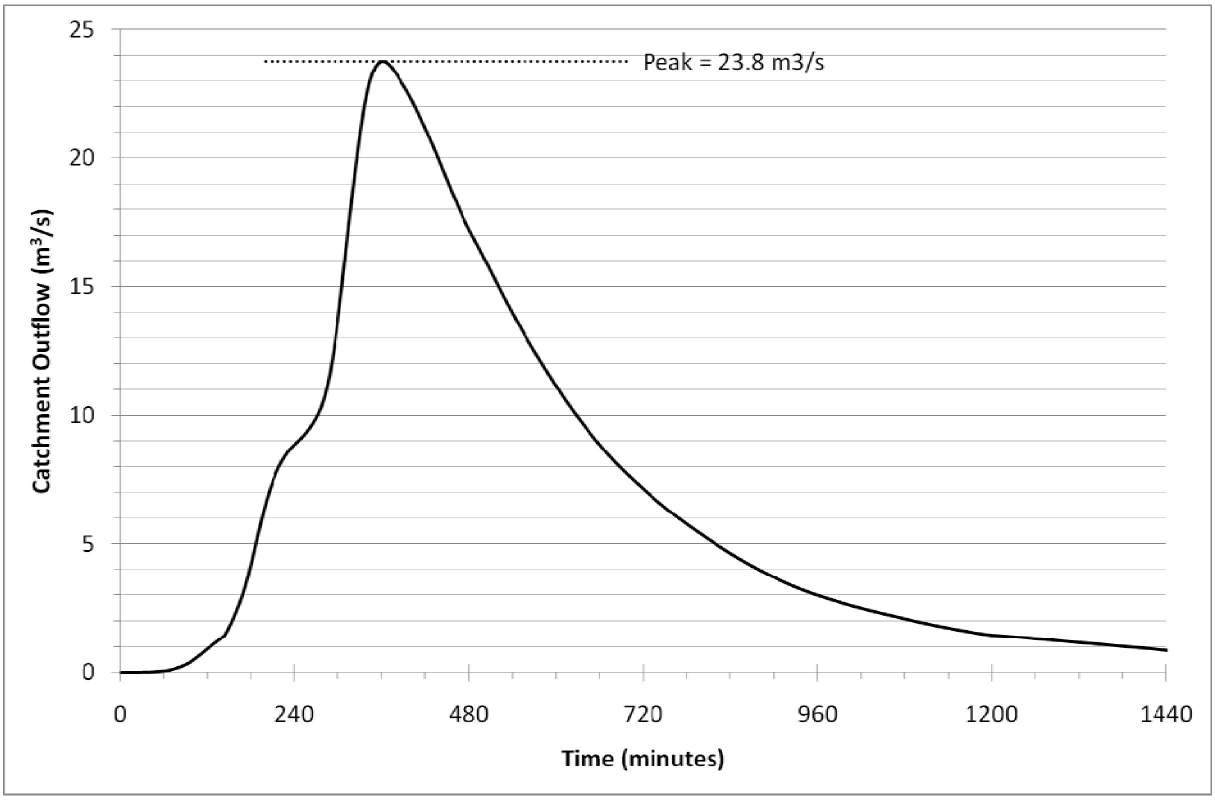
Appendix D – XP-Storm Hydrographs

1.0 Pre-Development (100 year ARI storm event)



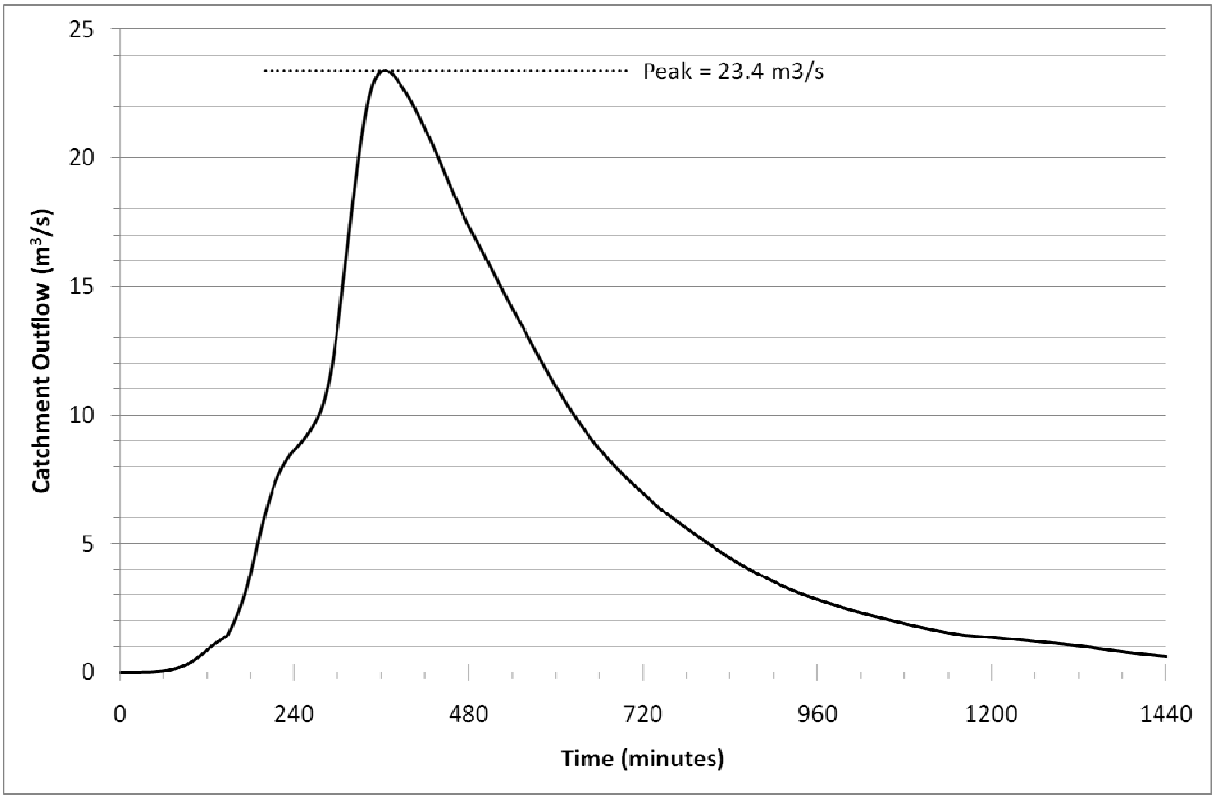
Graph D1 – Catchment outflow hydrograph calculated for the pre-development conditions

2.0 Post-Development (No Mitigation) (100 year ARI storm event)



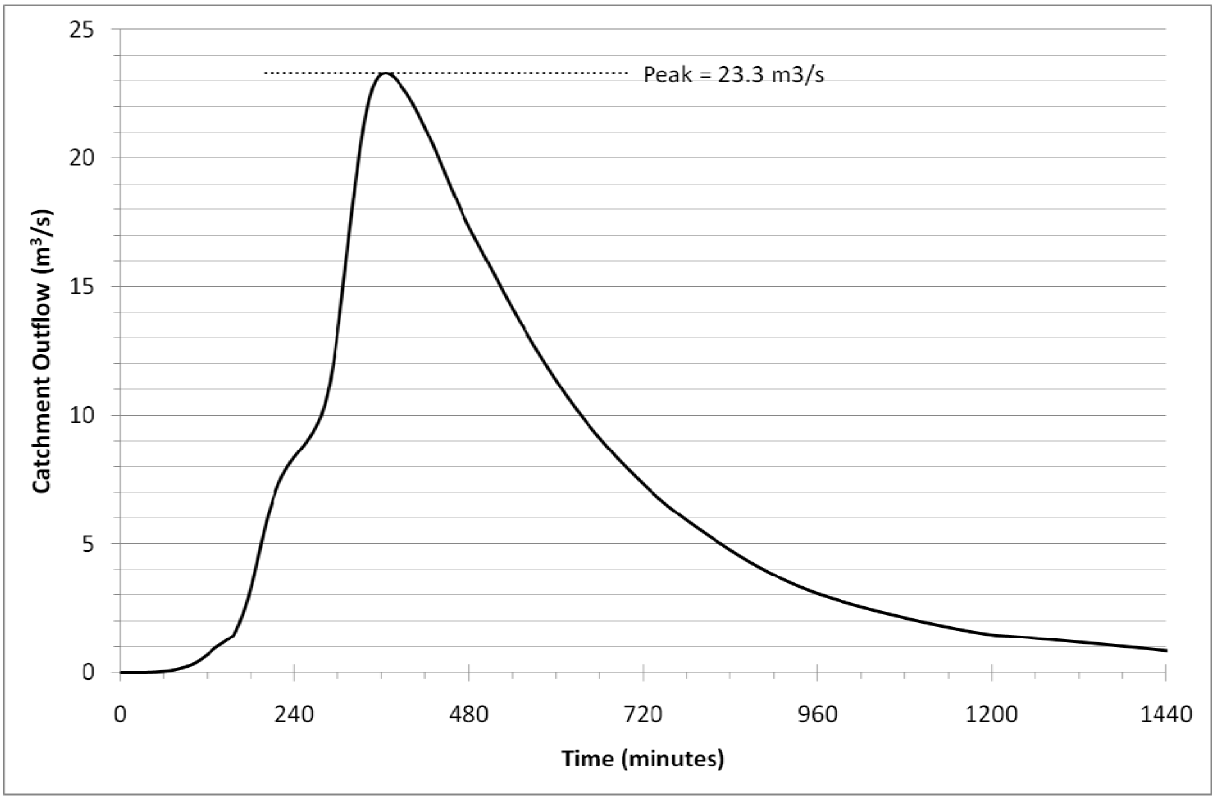
Graph D2 – Catchment outflow hydrograph calculated for post-development conditions with no mitigation strategy

3.0 Post-Development with Source Controls (100 year ARI storm event)



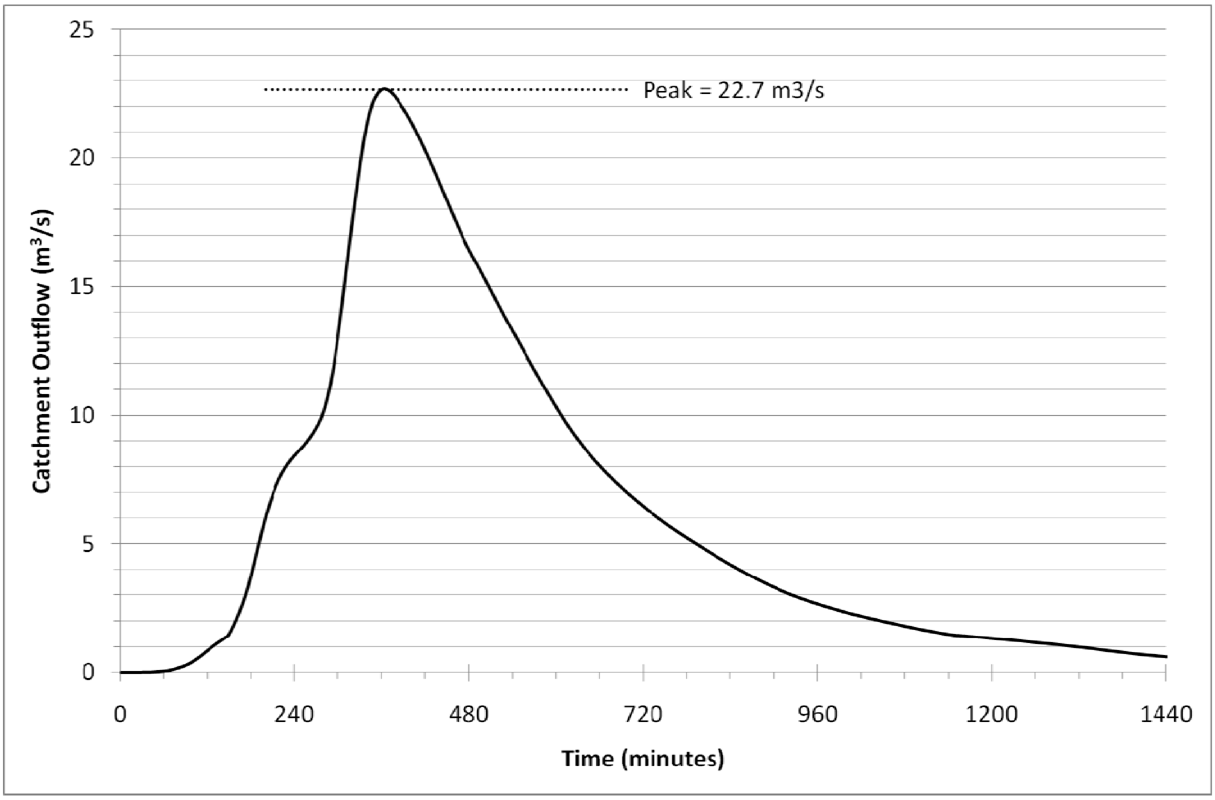
Graph D3 – Catchment outflow hydrograph calculated for post-development conditions with source controls (5 kL rainwater tanks) only

4.0 Post-Development with Outlet Controls (100 year ARI storm event)



Graph D4 – Catchment outflow hydrograph calculated for post-development conditions with outlet controls (dry detention basin) only

5.0 Post-Development with Source and Outlet Controls (100 year ARI storm event)



Graph D4 – Catchment outflow hydrograph calculated for post-development conditions with source (5 kL rainwater tanks) and outlet controls (dry detention basin)

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