Marsden Park Industrial Precinct

Post Exhibition
Water Cycle Management Strategy Report
Including
Consideration of Climate Change Impacts




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POST EXHIBITION WATER CYCLE MANAGEMENT STRATEGY INCLUDING CONSIDERATION OF CLIMATE CHANGE IMPACTS

- DOCUMENT CONTROL SHEET -

Issue No.	Amendment	Prepared By & Date	Checked By and Date	Approved By and Date
A	First Draft	GH, JC, ER (May 10)	PM (May 10)	PM (May 10)
B	Second Draft (internal issue only)	GH, JC, ER (May 10)	PM (May 10)	PM (May 10)
C	Third Draft	GH (May 10)	PM (May 10)	PM (May 10)
D	Final Draft	GH (May 10)	PM (May 10)	PM (May 10)
E	Amended Final Draft	GH (June 10)	PM (June 10)	PM (June 10)
F	Replaced Prelim Eng Concept Cover Sheet	GH (July 10)	PM (July 10)	PM (July 10)
G	Respond to additional BCC Comments	GH (October 10)		
H	DOP Comments	GH (January 11)	PM (January 11)	PM (Jan 11)
I	Additional DOP Comments per ILP	JC (February 11)	GH (February 11)	PM (Feb 11)

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1 INTRODUCTION

This report details the procedures used and presents the results of investigations undertaken in developing a Water Cycle Management Strategy that incorporates an assessment of Climate Change Impacts on the hydrology of the proposed development of an Industrial Precinct adjacent to Richmond Road Marsden Park. It provides a refinement of the hydrological information, contained in *"Water Cycle Management Assessment (July 2009)"* prepared by Gutteridge Haskins and Davey (GHD) (Ref.1) to inform the exhibition of the Draft Indicative Layout Plan (ILP) in February 2010, and is intended to support the post-exhibition re-zoning process for the Marsden Park Industrial Precinct (MPIP).

The objective of this investigation is to prepare a Water Cycle Management Strategy for the site which takes into account the comments submitted on the previously exhibited ILP. This report details the components of an integrated approach to managing peak discharges and stormwater quality, generated by the proposed development. It also provides an assessment of the likely impact that predicted increases in rainfall intensities, as a consequence of Climate Change, may have on these hydrological investigations.

Reticulated sewerage and potable water systems will be provided for the site and there is the opportunity for recycled water to be supplied at some time in the future. This report only addresses the stormwater aspects of the water cycle, however the harvesting and reuse of roof water is included within the water quality assessment. Rainwater storage tanks used for this purpose will be provided with an alternative to using potable water for 'top up', which will be activated once a recycled water supply system becomes operational in the area, making the connection to the potable water supply redundant.

This strategy report details the outcomes from each of the following specific tasks:

- Hydrologic analysis which determines the peak 5 and 100-year Average Recurrence Intervals (ARI) pre development and post development flows;
- Consideration of the impact that increased rainfall intensities, due to altered rainfall patterns in anticipation of Climate Change, will have on the proposed stormwater strategy
- Determination of the minimum detention storage volumes required to restrict post development peak flows to pre development levels;
- A water quality analysis to determine a cost–effective stormwater treatment strategy that will comply with the Department of Planning's (DoP) and Blacktown City Council's (BCC) stormwater quality discharge criteria;
- A Stream Erosion Index (SEI) assessment was undertaken by GHD for the Marsden Park Industrial Precinct (Ref.1) and returned an SEI value of 4, which is within the acceptable range adopted by the DoP for Growth Centre Release Areas;
- Preparation of concept designs for the Detention Basins, Trunk Drainage Channels and Bio-filtration water quality control measures;
- Extraction of a Bill of Quantities and construction costs for the major stormwater management and conveyance components of the Strategy.

The proposed Water Cycle Management Strategy expands and refines the Water Cycle Management Assessment (Ref.1) prepared by GHD (July 2009), which provided background information for the exhibition of the Draft ILP.

2 PREVIOUS REPORTS / STUDIES

BCC has constructed an XP-RAFTS hydrological model for the Bells Creek catchment. This model has been adopted as the basis for the hydrological assessments in the proposed Water Cycle Management and the analysis of the impact that the MPIP would have on peak flows in Bells Creek.

GHD prepared *Water Cycle Management Assessment: Flooding Stormwater and Water Sensitive Urban Design (July 2009)*, (Ref.1) which provided the background hydrological and water quality information necessary to inform the exhibition process for the Indicative Layout Plan in early 2010.

The background parameters and coefficients used in the preparation of this Water Cycle Management Strategy have been based on the BCC *Draft Integrated Water Cycle Management Development Control Plan (June 2009)* (Ref.6) and discussion with BCC staff.

In the absence of specific guidelines from BCC and the Department of Environment Climate Change and Water (DECCW), the primary reference sources adopted for our assessment of the hydrological impacts associated with Climate Change, were:

1. *NSW Climate Change Action Plan: Summary of Climate Change Impacts Sydney Region, October 2008*, (Ref.2) prepared by the NSW Department of Environment and Climate Change;
2. *Practical Consideration of Climate Change – Floodplain Risk Management Guideline, October 2007*, (Ref.3) prepared by the NSW Department of Environment and Climate Change;
3. *Climate Change in the Hawkesbury-Nepean Catchment, 2007*, (Ref.4) prepared by the Commonwealth Scientific and Industrial Research Organisation, were adopted as the primary reference documents for this assessment; and
4. *Climate Change in Australia – Observed Changes and Projections, October 2007*, (Ref.5) prepared by Australian Government Bureau of Meteorology.

A sensitivity analysis of the impact that increased peak flows (resulting from the Climate Change assessment) would have on freeboard allowances was undertaken in accordance with *Draft Flood Risk Management Guide: Incorporating sea level rise benchmarks in flood risk assessments, October 2009* Department of Environment, Climate Change and Water as well as *Blacktown City Council Engineering Guide for Development – 2005.* (Ref.13)

3 THE EXISTING ENVIRONMENT

3.1 Site Description

The site is gently undulating and located on the western side of Bells Creek between South Street and the proposed Castlereagh Freeway, which separates it from the suburbs of Hassall Grove and Bidwill. It has an area of approximately 500 hectares which is made up of open grassland, a quarry, a caravan park and a mosque to the west of Richmond Road and a paintball centre, nursery, mini golf course and boarding kennels to the east of Richmond Road (see Plate 1).

Apart from some isolated remnant stands of trees, in the northwest and southeast of the site and along Bells Creek and its tributaries, the majority of the vegetation on the site is grassland. Consequently the vegetation cover was represented, for the purposes of the hydrologic assessments, by runoff coefficients ranging from 0.05 for grasslands through to 0.08 for a copse of trees.

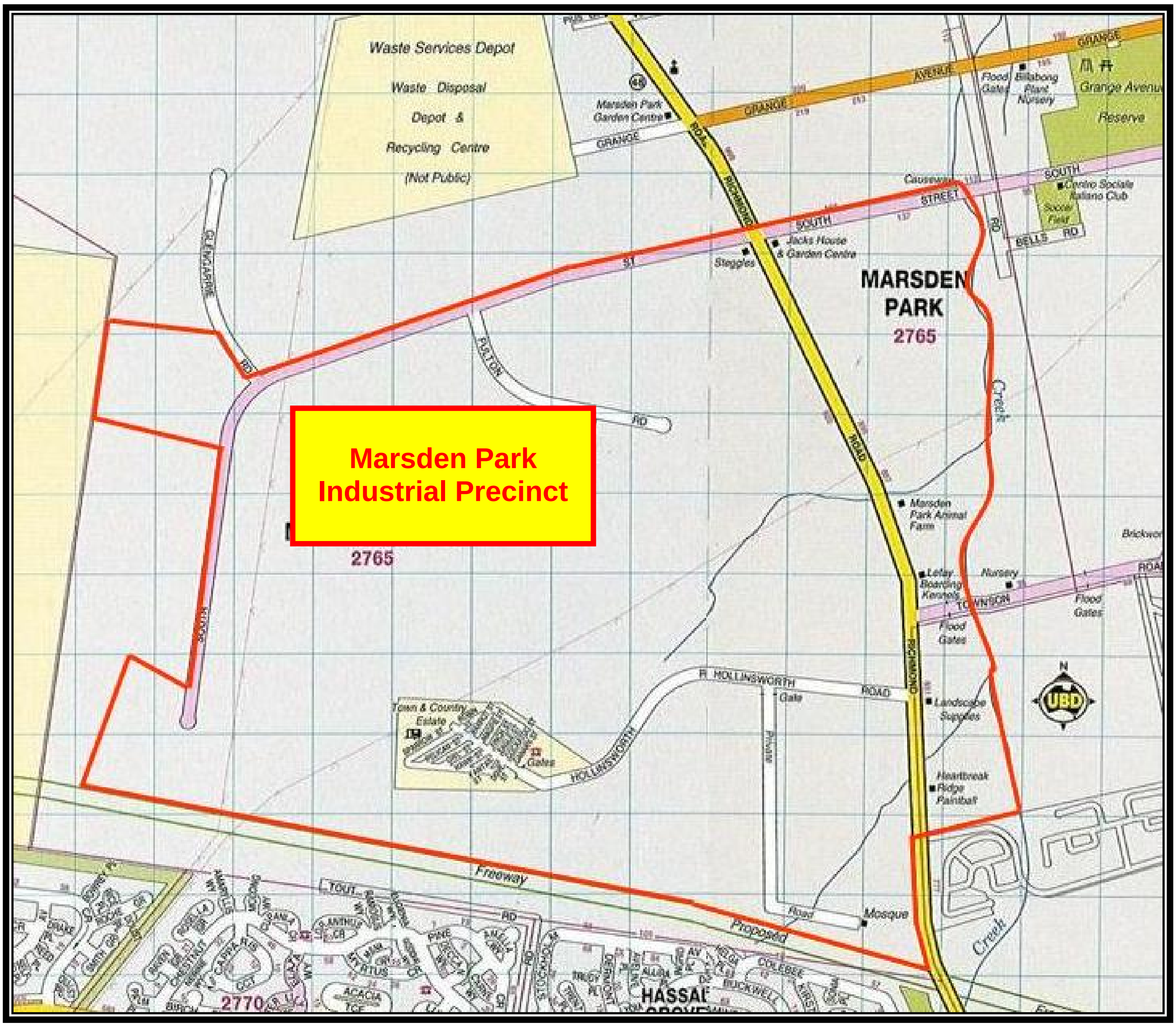

Plate 1: Location of the Marsden Park Industrial Precinct

3.2 Existing Drainage Configuration

Farm Dams with interconnecting depressions have been strategically located along the natural overland flowpaths through the site. Although the landform has been significantly modified for agricultural and quarrying pursuits: the western section of the site still generally drains in a north westerly direction into Little Creek within Shanes Park; the centre of the site drains to the north into an unnamed watercourse through Marsden Park; and the eastern areas drain through culverts under Richmond Road into tributaries of Bells Creek (see Plate 2 and Figure 2 Catchment Layout Plan Pre-developed Conditions).

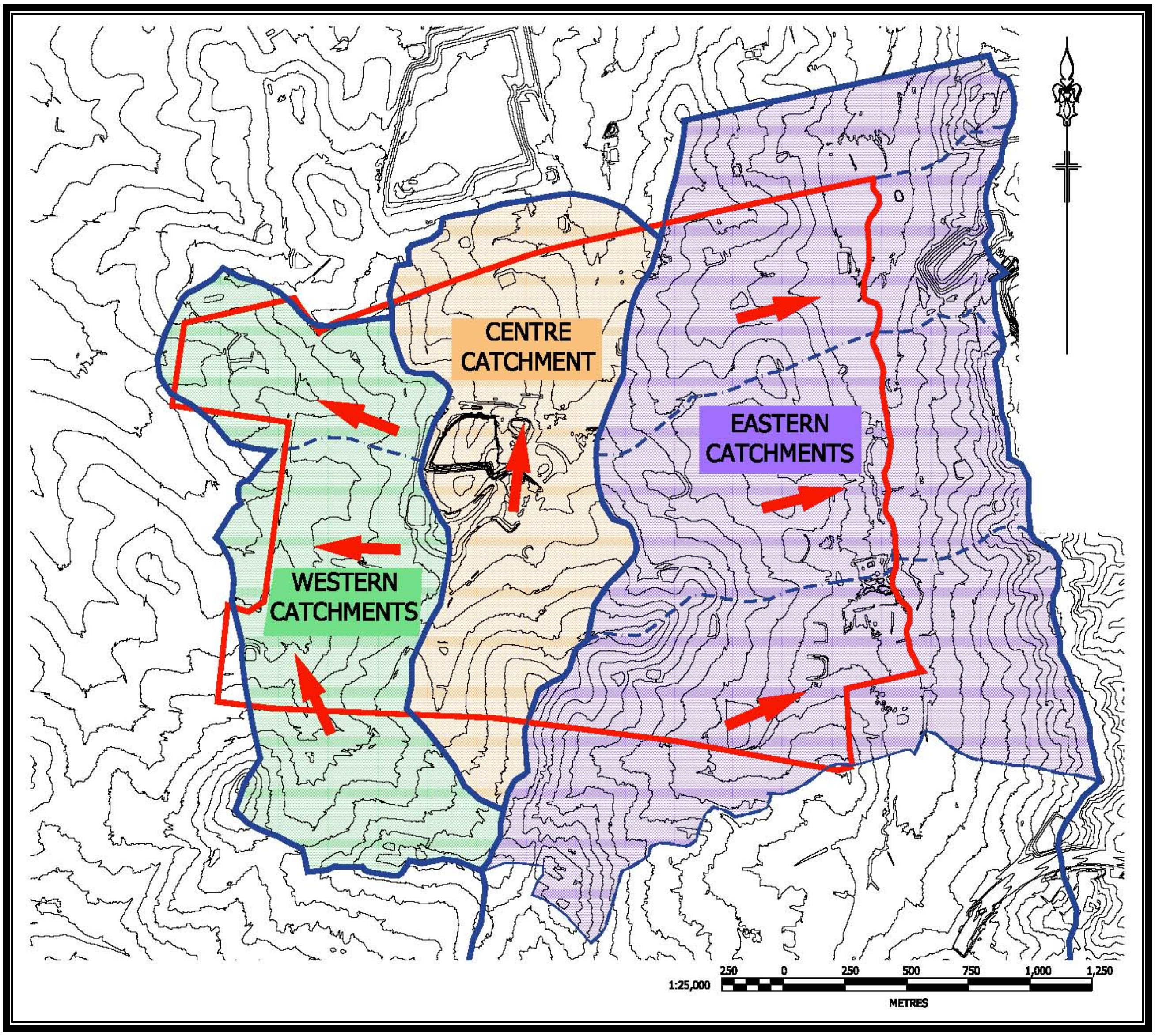

Plate 2: Existing Site Drainage Patterns

3.3 Mitigation Description

The Indicative Layout Plan (ILP) at Plate 3 provides an indication of the type and extent of the various land uses proposed for the Marsden Park Industrial Precinct. This plan formed the basis for determining the impervious percentages for each sub-catchment within the developed hydrological model.

To represent the change in runoff characteristics once the site is developed the following fractions were used to denote the impervious portions of the catchments:

- Existing (pre-development) 0.05;
- Future (post-development) 0.85 for residential and 0.9 for all other urban land uses, with 0.05 in the riparian corridors and 0.8 in areas nominated as Detention Basin.

Once developed the runoff from the site will impact on the hydrologic response of the local sub-catchments as well as the quality of the stormwater leaving the site. The following sections of this report describe the strategies to be employed to mitigate these impacts.

- Chapter 6 “Hydrologic Analysis” sets out in detail the estimated differences between the pre-development and post-development peak flows and the strategy to be employed to mitigate the post-development peak flows.
- Chapter 8 “Water Quality Analysis” sets out the stormwater treatment strategy to be employed at a lot level (industrial and commercial allotments only) and at a regional level (runoff from the public domain infrastructure in combination with the already treated runoff from the industrial/commercial catchments). Additional treatment is included in the regional treatment strategy to compensate for the lack of treatment in the residential catchments. This chapter also includes the calculations for the stormwater harvesting and reuse components of the strategy.
- Chapter 9 “Trunk Drainage Channel Analysis” describes the design of the Trunk Drainage Channel and road crossings which are intended to control runoff through the site from major storm events and prevent flooding of allotments up to the 100-year Average Recurrence Interval (ARI) storm events.

The goal of this Water Cycle Management Strategy is to mitigate the impacts of the increased peak discharges and reduced water quality in stormwater runoff generated by the development on downstream watercourses. Consequently the points of comparison, for the pre and post-development impacts, have been determined as the point where the discharge outlets meet with the existing watercourses, i.e. immediately downstream of each of the six (6) sub-catchments.

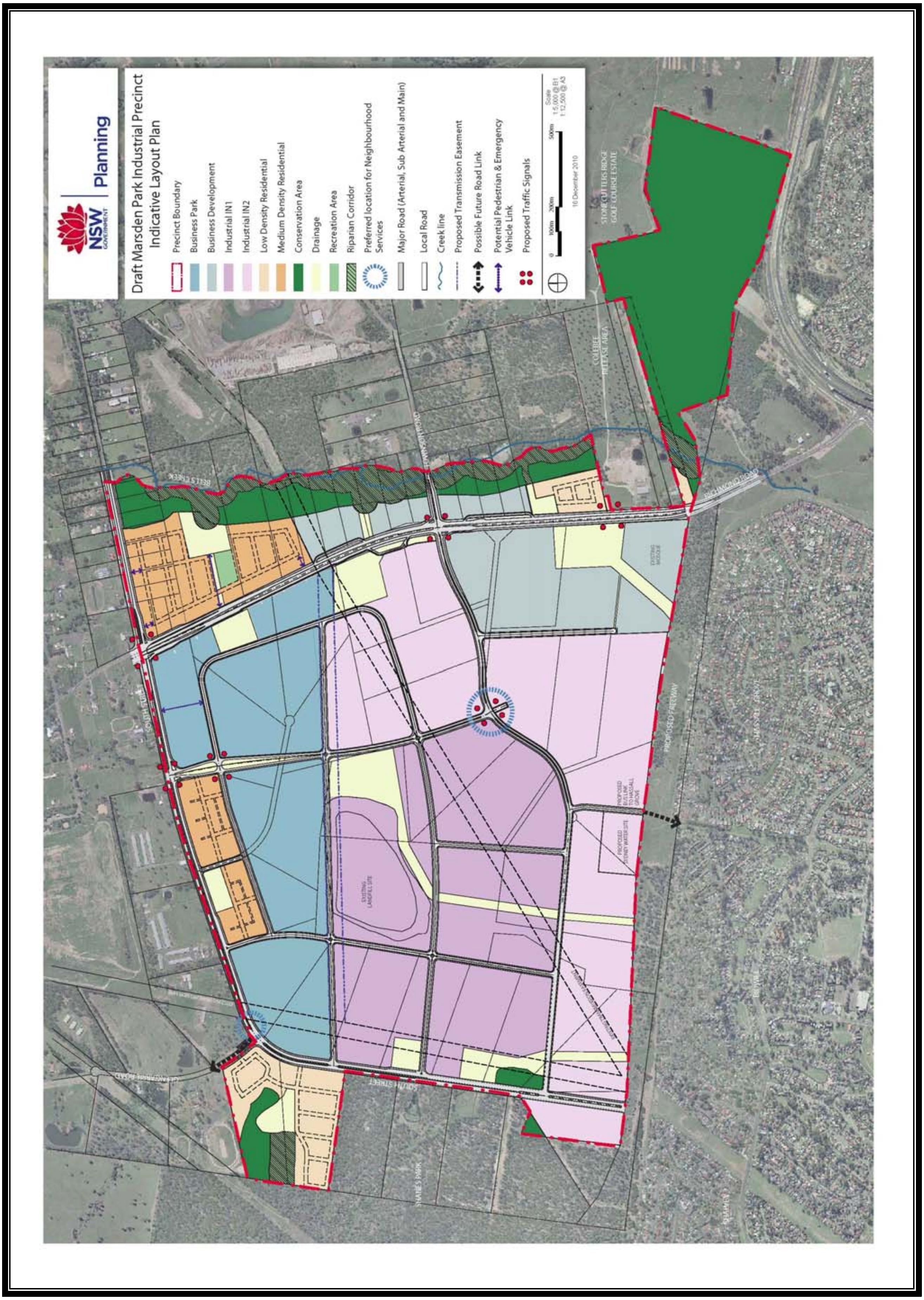

Plate 3: Marsden Park Industrial Precinct Indicative Layout Plan

4 DEVELOPMENT GUIDELINES, OPPORTUNITIES AND CONSTRAINTS

The *Draft Blacktown City Council Growth Centre Precincts Development Control Plan 2009* prepared by the NSW Department of Planning, October 2009 (Ref.7) formed the principle reference document for assessing the performance of the mitigation measures proposed in this Water Cycle Management Strategy.

Where it was necessary to source specific criteria or targets from elsewhere, the following documents were used as secondary references:

- *Draft Integrated Water Cycle Management Development Control Plan(June 2009,* Blacktown City Council (Ref.6); and
- *Engineering Guide for Development – 2005,* Blacktown City Council (Ref.13).

4.1 Draft Blacktown City Council Growth Centre Precincts Development Control Plan

Heading *2.3.1 Flooding and water cycle management* establishes the stormwater management objectives for the new subdivisions as:

a. *to manage the flow of stormwater from urban parts of the Precinct to replicate, as closely as possible, pre-development flows;*

b. *to define the flood constraints and standards applicable to urban development in the Precinct;*

c. *to minimise the potential flooding impacts on development.*

In summary the key targets against which to measure the success of the Water Cycle Management Strategy are:

- *Where practical, development shall attenuate up to the 50% AEP peak flow for discharges in the local tributaries, particularly Category 1 and 2 creeks.* (Attenuation afforded by Detention Basins determined on basis of 20% and 100% AEP discharges. Optimisation of the Basin outlets to be undertaken as part of the more detailed designs required for individual Development Applications).
- *The developed 1% AEP peak flow is to be reduced to pre-development flows through the incorporation of stormwater detention and management devices.*
- *Achieve water quality targets set by the Department of Environment and Conservation* (Ref.16). (See table below).

	WATER QUALITY % reduction in pollutant loads				**ENVIRONMENTAL FLOWS Stream erosion control ratio[1]**
	Gross Pollutants (>5mm)	Total suspended solids	Total phosphorous	Total nitrogen	
Stormwater management Objective	90	85	65	45	3.5-5.0: 1
'Ideal' stormwater outcome	100	95	95	85	1:1

[1] *This ratio should be minimised to limit stream erosion to the minimum practicable. Development proposals should be designed to achieve a value as close to one as practicable, and values within the nominated range should not be exceeded. A specific target cannot be defined at this time.*

4.2 Blacktown City Council Draft Integrated Water Cycle Management DCP

Although the Integrated Water Cycle Management DCP is still in draft form and has not yet been adopted by Council, it has been considered in preparation of the MPIP Water Cycle Management Strategy.

Blacktown City Council's Draft Integrated Water Cycle Management DCP (Ref. 6) aims to:

- Protect and enhance natural river systems.
- Minimise potable water demand and wastewater generation.
- Match the natural water runoff regime as closely as possible.
- Mitigate the impacts of development on water quality.
- Mitigate the impacts of development on groundwater.
- Ensure any changes to the existing groundwater regime do not adversely impact upon adjoining properties.
- Integrate water cycle management measures into the landscape and urban design to maximise amenity.
- Minimise the potential impacts of development and other activity on the aesthetic, recreational and ecological values of receiving waters.
- Minimise soil erosion and sedimentation resulting from site disturbing activities.
- Ensure the principles of ecologically sustainable development are applied in consideration of economic, social and environmental values in water cycle management.

General requirements for the design of trunk drainage systems in new release areas, to achieve these aims are detailed in the BCC Engineering Guide for Development – 2005 (Ref.13). This document was the principle reference for the hydrological modelling referred to in this strategy.

4.3 Adopted Performance Targets for the MPIP

4.3.1 Water Conservation

- New residential dwellings, including a residential component within a mixed use building and serviced apartments intended to, or capable of being strata titled, are to demonstrate compliance with the NSW Government Building Sustainability Index initiative (BASIX).
- BCC require that all *industrial and commercial developments must install rainwater tanks to meet a minimum of 50 per cent of their own potable water demand for outdoor use, toilets, laundry or hot water*. (Ref.6)
- BCC require that water use within public open space (e.g. irrigation, water features, open water bodies / pools) should be supplied from non potable sources to meet a minimum of 80% of this demand. (Ref.6)

4.3.2 Stormwater Quality

- 90% reduction in the post development average annual gross pollutant (>5mm) load.
- 85% reduction in the post development average annual load of Total Suspended Solids (TSS) load.
- 65% reduction in the post development average annual load of Total Phosphorus (TP) load.
- 45% reduction in the post development average annual load of Total Nitrogen (TN) load.
- Environmental Flows - Stream Forming Flow Ratio 3.5-5.0: 1, where "the 'stream forming flow' is defined as 50% of the 2-year flow rate estimated for the catchment under natural conditions" (DECCW advice to GCC).

4.3.3 Peak Flow Management

- The developed 5-year and 100-year peak flows attenuated to match the pre-development peak flows where the discharge point from the MPIP meets the existing natural watercourses and creeks.
- Freeboard on the proposed Trunk Drainage system will comply with the values specified under Section 1.5 Design Freeboard (BCC 05), i.e. the finished floor level will be at least 0.5 m above the design 100-year top water level.

4.3.4 Consideration of Climate Change Impacts

An assessment, based on the guidelines outlined in the contemporary references identified in Heading 2 Previous Studies/Reports of this report, was undertaken and submitted to the DECCW for consideration (see Appendix A for a copy). The comments from DECCW were that the assessment undertaken "provides a pragmatic approach to the consideration of Climate Change impacts on the urban hydrology for the MPIP.

In summary the final parameters adopted as a consequence of the sensitivity analysis of the changes to the hydrology and freeboard allowances included:

- Increasing the rainfall intensities by 15%.
- A maximum reduction of 0.2 m in the 0.5 m freeboard allowance, based on the depth of flow in channels and the top water level in basins, generated by the existing 100-year ARI critical storm (i.e. existing freeboard allowance reduced from 0.5 m to 0.3 m as a consequence of increasing the existing 100-year rainfall intensities by 15%).
- Detention Basins to be provided with flexible outlet structures which can be adjusted on the basis of anticipated decadal changes to rainfall intensities. The design of these outlet structures is to be undertaken as part of the more detailed designs required for Development Approval and Construction Certification.

4.4 Salinity

Salinity mapping prepared by NSW Department of Infrastructure Planning and Natural Resources in 2002 (Ref.22) identifies the MPIP as having a *“Moderate Salinity Potential”*, increasing to *“High”* in watercourses and *“Known”* to the north of the site. GHD (Ref. 23) referred to groundwater salinity at >14,000 mg/L and generally relatively shallow saline groundwater is to be expected in soils overlying Wianamatta shales (Ref.24). However, much of the development falls within the area of *“Moderate Salinity Potential”* and measures designed to mitigate the impact of the MPIP on groundwater levels include:

- Minimisation of water infiltration;
- Inclusion of deep rooted vegetation;
- Minimisation of cut and fill operations which inappropriately alter natural drainage patterns;
- Design of impermeable liners on all measures that retain water for in excess of 24 hours.

4.5 Stormwater Management Objectives

The Water Cycle Management Strategy proposed for the MPIP has been prepared in accordance with the principles of Ecologically Sustainable Development (ESD). To comply with these principles, the final form of the structural stormwater control elements must be designed, developed and maintained in accordance with the following objectives:

- *Precautionary Principle* – although there is uncertainty with respect to the magnitude of climate change impacts, the drainage system must be designed with sufficient redundancy to accommodate the changes to rainfall intensities considered in the Climate Change Impact assessment. Restriction of urban development to 0.5 m above the estimated 100-year flood levels provides a safe guard against uncertainty in Flood Level estimations.
- *Inter-generational Equity* – incorporation of Water Sensitive Urban Design principles within the development such that the post-development water quality discharges comply with the DECCW advice. The targets, included in this advice, were intended as minimum standards required to maintain the environmental integrity of receiving water bodies.
- *Conservation of Biological Diversity and Ecological Integrity* – achieved through the conservation of existing riparian areas, the use of vegetated ‘naturalised’ Trunk Drainage networks and compliance with the DECCW Environment Flow advice.
- *Improved Pricing and Valuation of Assets* – the Water Cycle Management Strategy has been prepared as a cost-effective control measure, for inclusion in a S94 Contributions Plan, to reduce the impacts of stormwater discharges on the receiving environment. It includes consideration of the costs associated with viability of the public domain infrastructure, in particular: ease of access to the open space areas; vegetation management; removal of litter and gross pollutants; and the control of sediment and nutrients.

4.6 Response to Blacktown City Council Comments

BCC made comment on the original submission of the GHD Water Cycle Management Assessment in August 2009. The following table contains a brief explanation of each BCC comment and the proposed element(s) within the Water Cycle Management Strategy (WCMS) which will address each comment.

BCC Comment	Response
a) RTA concurrence with the Stormwater Strategy proposed in Richmond Road.	Proposal for spill containment within the road reserve and compensatory storage and treatment within the MPIP. Still awaiting written concurrence fro RTA.
b) Assessment of the durations used in the TUFLOW model to determine peak flows in the Little Creek catchment.	Copy of the existing GHD models were provided by BCC. The 100 yr ARI Storm duration which currently produces peak flows in this catchment has been confirmed as 120 mins for both the existing and developed conditions. (Refer Table 6.3 for the 5 yr and Table 6.4 for the 100 yr – Link Label W-1.1.03)
c) Water quality options analysis for the Little Creek catchment.	See Section 7 “Water Quality Analysis.”
d) Water quality model to include bypass of high flows at the GPTs.	See Section 7 “Water Quality Analysis.”
e) TUFLOW modelling to account for existing Bells Creek Flood Models and downstream boundary conditions.	Copy of the existing GHD TUFLOW model for Bells Creek was provided by BCC.
f) Climate Change Impacts to be identified across the MPIP.	See Section 6.10 “Climate Change Sensitivity Assessment.”
g) Report on the Stream Erosion Index at each discharge location.	See Section 9 “Stream Erosion Index.”
h) Determination of Flood Planning Levels throughout the MPIP	To be identified as a constraint for the determination of individual DAs in accordance with Section 6 “Hydrologic Analysis” and Section 8 “Trunk Drainage Channel Analysis.”
i) Locate each modelling node to allow a comparison of pre and post development conditions.	See Section 6 “Hydrologic Analysis” and Section 7 “Water Quality Analysis.”
j) Hydraulic assessment of the Riparian corridors adjacent to Basins A, C and G as well as TC1 and TC8.	No longer applicable as the Basins are located on line. See Section 6 “Hydrologic Analysis” and Section 8 “Trunk Drainage Channel Analysis.
k) Identify local overland flowpaths and match with final road and drainage alignments.	Adjustments to ILP will address this issue. Each DA to be constrained by the nominated flowpaths. See attached Figures.
l) Assess options to locate TC7 outside of the road median	Still to be resolved.
m) Include options to use recycled water within the rainwater harvesting and reuse scheme	See Section 1 “Introduction.”
n) Estimation of bulk earthworks to include dewatering and desilting of existing farm dams and disposal of unsound material.	See Section 10 “Costing of Detention Basin Major Works” and the attached Bill of Quantities.
o) Water Cycle Management Report detailing the parameters used and the assessment outcomes.	Addressed by the final “Water Cycle Management Strategy” report.

5 STORMWATER MANAGEMENT CONCEPT

The proposed Water Cycle Management Strategy for the MPIP has been prepared with consideration of the statutory requirements and guidelines listed under Heading 4 Development Guidelines, Opportunities and Constraints.

In summary it consists of three (3) integrated components:

1. On Lot water quality controls for the industrial/commercial areas, and medium/high density residential sites.
   A rainwater tank or tanks will be strategically placed within each allotment to harvest, store and allow roof runoff to be reused for toilet flushing, hot water and external irrigation. All external hardstand areas will be directed to a Gross Pollutant Trap (GPT), which will pre-treat the runoff before it is discharged into a raingarden (bio-retention/filtration) system representing at least 1% of the allotment. The remaining pervious areas of the allotment, where elevations permit, will be directed into the raingarden, which will be connected to the formal drainage network.

2. Roof water harvesting and reuse on all residential allotments.
   The roof runoff will be collected in the roof guttering and collected in rainwater tanks connected to a BASIX compliant reuse system. There will be no other on lot treatment or on site detention of runoff from the low density residential catchments. All runoff will drain to the formal drainage network where it will be discharged into the precinct based Detention Basins and their co-located raingardens.

3. Water quality control raingardens co-located within the Precinct Based Detention Basins.
   The formal drainage network, consisting of underground pipes and densely vegetated channels, has been sized to convey the runoff from the public domain infrastructure, residential catchments and the pre-treated site runoff from the industrial/commercial catchments into the precinct based Detention Basins and their co-located raingardens.

Runoff from external catchments within Hassall Grove and Bidwill, which drain through the MPIP, are detained within the internal precinct based Detention Basins. However their low flows are diverted around the water quality control systems, co-located within the Basins, and 'shandied' with the treated runoff from the MPIP before being discharged downstream.

A more detailed description of the peak flow management and water quality control elements of the Strategy can be found in Heading 6 "Hydrologic Analysis" and Heading 8 "Water Quality Analysis".

6 HYDROLOGIC ANALYSIS

The hydrologic analyses for this study were undertaken using the rainfall - runoff flood routing model XP-RAFTS (Runoff and Flow Training Simulation with XP Graphical Interface) (Ref. 11 & 12).

6.1 Sub-Catchments (Pre and Post Development)

Sub-catchment areas contributing to this drainage system were established through site investigations, detail survey and Aerial Laser Scan (ALS) survey covering the catchment and consideration of the Masterplan for the site. Catchment boundaries for the existing and developed areas contributing to the drainage system are shown on Figure 3 and the catchment details are provided in Tables 6.3 and 6.4.

NOTE Developed catchment boundaries have been determined on the best information available with regard to final site grading and levels. Final catchment boundaries and areas contributing to each Detention Basin must be confirmed as part of the Development Approval process for each stage of the development.

6.2 Rainfall Data

6.2.1 Intensity-Frequency-Duration (I.F.D.)

Design rainfall intensity-frequency-duration (I.F.D.) data for the site were obtained from *Blacktown City Council's Engineering Guide for Development 2005* (Ref. 13). A summary of the rainfall intensities adopted in this study is provided in Table 6.1. The critical storm durations were determined using these values for each sub-catchment.

Table 6.1

BLACKTOWN RAINFALL INTENSITIES (mm/hr)

Storm Duration (min.)	Rainfall Intensities (mm/hr) ARI	
	5	100
5	129	219
10	98	167
15	82	139
20	71	121
25	64	108
30	58	98
45	46.2	78
60	39.2	66
90	30.7	52
120	25.7	43.4
180	20.0	33.8
270	15.5	26.2
360	13.0	21.9
540	10.1	17.1
720	8.45	14.3
1080	6.61	11.4
1440	5.54	9.66

The models used to examine the catchment rainfall/runoff relationships incorporated temporal patterns for synthetic design storms determined using procedures detailed in *Australian Rainfall and Runoff, Engineers Australia (1987)* (Ref. 14).

6.3 XP-Rafts Parameters

The Pern (n) values and losses adopted for the catchments in the XP-RAFTS modelling are listed in Table 6.2.

Table 6.2

ADOPTED XP-RAFTS PARAMETERS (BCC Standard Parameters)

Parameter	Catchment Condition	Adopted Value
Pern		
	Existing Pervious	0.05 - 0.08
	Urban Pervious	0.025
	Urban Impervious	0.015
Initial/Continuing Losses - Adopted in Undeveloped Subcatchments		
Initial Loss	Pervious Catchment	15.0
Continuing Loss	Pervious Catchment	2.5
Initial Loss	Impervious Catchment	1.0
Continuing Loss	Impervious Catchment	0.0
ARBM - Adopted in Developed Subcatchments		
ARBM Loss Process	Routing Time Step (min)	1
Storage Capacities		
Impervious Zone	Capacity (mm)	1.5
	Initial (mm)	0.5
Interception Zone	Capacity (mm)	1.5
	Initial (mm)	0.5
Depression Zone	Capacity (mm)	5
	Initial (mm)	0
Upper Soil	Capacity (mm)	25
	Initial (mm)	20
Lower Soil	Capacity (mm)	100
	Initial (mm)	80
Groundwater	Initial (mm)	0
Infiltration		
Upper Soil	Dry Sorpivity (mm/min^0.5)	3
	Hydraulic Conductivity (mm/min)	0.33
Lower Soil	Drainage Factor	0.05
	Constant Rate	0.94
	Variable Rate	1
Evapotranspiration		
Proportion of Rainfall Intercepted by Vegetation		0.7
Max Potential Evapotranspiration	Upper Soil (mm/day)	10
	Lower Soil (mm/day)	10
Proportion of Evapotranspiration from Upper Soil		0.7
Ratio of Potential Evaporation to A Class Pan		0.7

6.4 Calibration

It is normal practice for flood routing models such as XP-RAFTS to be calibrated with historical rainfall and stream flow data for the catchment being investigated in order to produce the most reliable results. The model parameter values (in particular Bx) are adjusted so that the model adequately reproduces observed hydrographs. However, no stream flow records were available for the site and a Bx value of 1.00, which is consistent with the value used in the BCC XP-RAFTS model for Bells Creek, was adopted for the hydrologic analysis of the MPIP.

6.5 XP-RAFTS Model

There are a number of basins attenuating discharges from catchments upstream of Richmond Road, which drain tributaries of Bells Creek. To ensure consistency with existing hydrologic models BCC provided an XP-RAFTS model for the Bell Creek catchment, which was used as the basis for this assessment.

The BCC catchment information was altered to incorporate the MPIP catchment extents as determined by the ILP and recent survey information. Catchment areas upstream of the MPIP were left unaltered, whilst those catchments within and adjacent to the site, as far downstream as Grange Avenue, were updated to reflect the more recent survey information.

Applying estimated lag times between links in the model, rather than routing them by individual calculations was adopted for the MPIP XP-RAFTS model.

Stage Storage relationships for each detention basin were developed using a graphical interpolation of each stage based on the total storage volume, basin layout, and the design levels. The final configuration of each basin, and its outlet, will be included within the detailed design information accompanying the Development Application for each basin.

Basin O (see Appendix D 8955 SK1) was originally required to attenuate and treat discharges from an 8.26 ha catchment in the south western corner of the MPIP. However, this catchment is zoned Industrial IN2 and any subsequent development will be required to provide a site specific “on site-detention and water management” strategy. Consequently Basin O is no longer required as a Regional Stormwater Management Basin and has been removed from the ILP and Council’s Section 94 Plan (see explanatory notation on Plate 3). The XP-RAFTS hydrologic model demonstrates that the storage volumes available in the proposed Basins is sufficient to achieve the peak discharge targets for the MPIP (see Tables 6.5 & 6.6)

6.6 Tailwater Effects

Invert levels for the floor of the Detention Basins storage zones have been determined on the basis that they will be at or above the downstream 100-year flood levels immediately downstream of the proposed basin location. However the hydraulic performance of the outlet arrangements for the basins has been modelled, in XP-STORM (Ref.20), with a submerged outlet during the 100-year ARI regional flood event. This should provide a conservative estimation of the hydraulic performance of the basins, in particular Basin G, I, J and M.

6.7 Discharge Estimates

Discharge estimates were derived for the existing and developed catchments for storms with ARI’s of 5-years and 100-years. A range of storm durations from 25 minutes to 24 hours were analysed to determine the critical storm duration for each sub-catchment.

XP-RAFTS modelling was undertaken to determine the estimated peak discharges from the catchment for the following catchment conditions:

- Undeveloped Site under existing rural/quarry conditions.

- Site developed with detention systems provided.

The 5-year and 100-year ARI peak flows from the catchment are presented in Tables 6.3 and 6.4. XP-RAFTS outputs for the individual Basins are provided in Tables 6.5 and 6.6.

Table 6.3

SUMMARY OF PEAK FLOWS – 5 YEAR ARI

			Existing				Developed with Detention				Ratio
Link Label Exist	Link Label Dev	Location	Cum Area ha	Max Flow (m³/s)	Storm Dur. (min)	Time to Peak	Cum Area ha	52.67 0.2606 (m³/s)	540 540 540	330 330 330	Prop. / Exist
1.13	1.13	Bells Creek at Site Boundary	834.14	52.67	540	330	834.14	52.67	540	330	1.00
B-1.1.03	B-1.1.04	Tributary 1 at Richmond Road	118.34	13.86	25	15	116.66	9.63	120	53	0.70
B-1.1.04	B-1.1.06	Tributary 1 into Bells Creek (Basin G)	131.98	14.16	25	15	127.26	10.13	90	47	0.72
1.15	1.15	Bells Creek at Tributary 1	1003.75	63.77	90	35	999.03	63.51	540	330	1.00
B-2.1.01	B-2.1.02	Tributary 2 into Bells Creek (Undetained)	22.74	1.56	90	43	8.56	1.51	25	16	0.97
1.16	1.16	Bells Creek at Tributary 2	1045.09	66.29	120	101	1026.19	65.18	540	330	0.98
B-3.1.03d	B-3.1.04	Tributary 3 at Richmond Road	55.50	3.99	270	92	68.02	3.20	120	65	0.80
B-3.1.04	B-3.1.06	Tributary 3 into Bells Creek (Basin I)	62.17	4.26	360	145	75.52	3.62	90	32	0.85
1.17	1.17	Bells Creek at Tributary 3	1117.34	70.80	360	150	1111.79	69.43	540	335	0.98
B-3.3.00	B-3.8.02	Catchment north of Tributary 3 into Bells Ck	10.19	0.64	360	150	11.15	1.79	25	15	2.80
1.18	1.18	Bells Creek at d/s Tributary 3 Catchment	1144.07	72.26	360	155	1138.88	70.89	540	340	0.98
B-4.1.01	B-4.1.02	Tributary 4 at Richmond Road	31.04	2.07	540	330	43.30	2.98	270	90	1.44
B-4.1.02	B-4.1.05	Tributary 4 into Bells Creek (Basin M)	72.66	4.62	540	330	66.22	3.65	540	340	0.79
1.19	1.19	Bells Creek at Tributary 4	1213.98	75.87	540	330	1219.02	75.33	540	345	0.99
B-4.3.00	B-4.7.01	Catchment north of Tributary 4 into Bells Ck	16.67	0.89	270	94	9.46	1.61	25	17	1.81
1.20	1.20	Bells Creek at d/s Site Boundary (South St)	1249.49	77.64	540	330	1247.32	76.75	540	350	0.99
1.22	1.22	Bells Creek at Grange Avenue	1296.63	79.77	120	125	1294.46	79.12	540	360	0.99
N-2.01	N-8.02b	Minor Western Tributary of N/S Ck from Site	20.39	1.37	540	330	21.64	1.36	540	330	1.00
N-1.04	N-1.07	Main Tributary of N/S Ck from Site	118.21	7.69	90	30	119.17	7.69	90	30	1.00
N-S	N-S	Total Discharges from N/S Creek D/S of Site	163.61	10.69	540	330	162.25	9.54	540	330	0.89
W-1.1.03	W-1.1.05	Main Western Tributary 1 from Site	101.72	12.25	25	15	117.02	11.14	120	48	0.91
W-1.4.00	W-1.6.02	Minor Western Tributary 1 from Site	26.66	1.59	90	35	22.94	1.37	120	68	0.86
W-1	W-1	Main Western Tributary 1 D/S from Site	128.38	12.85	540	330	139.96	12.44	120	49	0.97
W-2	W-2	Main Western Tributary 2 D/S from Site	55.20	3.36	90	47	47.97	3.03	540	330	0.90

Table 6.4

SUMMARY OF PEAK FLOWS – 100 YEAR ARI

			Existing				Developed with Detention				Ratio
Link Label Exist	Link Label Dev	Location	Cum Area ha	Max Flow (m³/s)	Storm Dur. (min)	Time to Peak	Cum Area ha	52.67 0.2606 (m³/s)	540 540 540	330 330 330	Prop. / Exist
1.13	1.13	Bells Creek at Site Boundary	834.14	95.89	120	66	834.14	95.89	120	58	1.00
B-1.1.03	B-1.1.04	Tributary 1 at Richmond Road	118.34	25.60	25	15	116.66	21.78	120	50	0.85
B-1.1.04	B-1.1.06	Tributary 1 into Bells Creek (Basin G)	131.98	26.54	90	33	127.26	22.71	120	53	0.86
1.15	1.15	Bells Creek at Tributary 1	1003.75	121.32	90	35	999.03	120.85	120	61	1.00
B-2.1.01	B-2.1.02	Tributary 2 into Bells Creek (Undetained)	22.74	3.39	90	43	8.56	2.47	90	31	0.73
1.16	1.16	Bells Creek at Tributary 2	1045.09	126.25	120	101	1026.19	123.67	120	61	0.98
B-3.1.03d	B-3.1.04	Tributary 3 at Richmond Road	55.50	8.68	120	48	68.02	7.27	120	65	0.84
B-3.1.04	B-3.1.06	Tributary 3 into Bells Creek (Basin I)	62.17	9.21	120	58	75.52	8.02	540	330	0.87
1.17	1.17	Bells Creek at Tributary 3	1117.34	136.45	120	60	1111.79	132.95	120	66	0.97
B-3.3.00	B-3.8.02	Catchment north of Tributary 3 into Bells Ck	10.19	1.23	120	58	11.15	2.87	90	29	2.33
1.18	1.18	Bells Creek at d/s Tributary 3 Catchment	1144.07	139.67	120	63	1138.88	135.94	120	70	0.97
B-4.1.01	B-4.1.02	Tributary 4 at Richmond Road	31.04	4.22	120	68	43.30	6.62	120	41	1.57
B-4.1.02	B-4.1.05	Tributary 4 into Bells Creek (Basin M)	55.99	7.69	120	66	66.22	7.41	120	72	0.96
1.19	1.19	Bells Creek at Tributary 4	1213.98	148.65	120	65	1219.02	144.99	120	76	0.98
B-4.3.00	B-4.7.01	Catchment north of Tributary 4 into Bells Ck	16.67	1.64	120	53	9.46	2.60	90	32	1.59
1.20	1.20	Bells Creek at d/s Site Boundary (South St)	1249.49	151.91	120	67	1247.32	147.95	120	80	0.97
1.22	1.22	Bells Creek at Grange Avenue	1296.63	156.86	120	124	1294.46	153.04	120	87	0.98
N-1.00	N-1.00	U/S Site Tributary into North/South Creek	16.00	4.26	360	160	12.64	5.82	25	15	1.37
N-2.01	N-8.02b	Minor Western Tributary of N/S Ck from Site	20.39	2.85	90	37	21.64	2.14	120	68	0.75
N-1.04	N-1.07	Main Tributary of N/S Ck from Site	118.21	14.83	90	30	119.17	14.14	120	61	0.95
N-S	N-S	Total Discharges from N/S Creek D/S of Site	163.61	20.55	120	65	162.25	19.38	120	63	0.94
W-1.1.03	W-1.1.05	Main Western Tributary 1 from Site	101.72	22.31	90	30	117.02	21.90	120	47	0.98
W-1.4.00	W-1.6.02	Minor Western Tributary 1 from Site	26.66	2.99	90	35	22.94	1.92	120	68	0.64
W-1	W-1	Main Western Tributary 1 D/S from Site	128.38	24.10	360	150	139.96	23.68	120	48	0.98
W-2	W-2	Main Western Tributary 2 D/S from Site	55.20	6.39	90	47	47.97	6.72	120	46	1.05

NOTE: *The increase in outflow from node B-4.1.02 results from the combination of external flows generated by Richmond Road and an existing external catchment, both of which*

discharge through cascading Detention Basins J and M. The outflow from detention Basin M is represented by node B-4.1.05.
Final Peak Flow values are to be determined upon completion of the detailed designs and preparation of the Development Application for each basin.

6.8 Basin Performance

The performance of the basins for the 5-year and 100-year ARI storm events are detailed in Tables 6.5 and 6.6, respectively.

Table 6.5

DETENTION BASIN PERFORMANCE – 5 YEAR ARI

	Basin Inflows			Basin Outflows				
Basin Label	Max Flow (m^3/s)	Storm Dur. (min)	Time to Peak (min)	Max Flow (m^3/s)	Storm Dur. (min)	Time to Peak (min)	Storage Used (m^3)	Storage Depth (m)
A*	30.89	25	15	10.99	120	46	18511	0.96
O	2.25	25	15	0.40	120	66	1550	0.84
B	6.31	25	15	1.21	120	66	4560	0.96
P	9.34	25	16	2.30	120	51	6983	1.00
E	25.07	25	15	5.64	540	335	30604	1.06
K	6.93	25	15	1.36	540	330	6275	1.02
G	24.72	25	19	9.13	120	53	21197	0.98
I	19.60	25	15	3.61	540	335	19045	0.97
J	11.54	25	15	2.33	270	92	9186	0.95
M	7.61	25	15	3.43	120	68	5220	0.84

* Basin A configured assuming that Basin O does not exist

Table 6.6

DETENTION BASIN PERFORMANCE – 100 YEAR ARI

	Basin Inflows			Basin Outflows				
Basin Label	Max Flow (m^3/s)	Storm Dur. (min)	Time to Peak (min)	Max Flow (m^3/s)	Storm Dur. (min)	Time to Peak (min)	Storage Used (m^3)	Storage Depth (m)
A*	48.42	25	15	21.47	120	45	30420	1.20
O	3.45	25	15	0.68	120	66	2855	1.17
B	9.79	25	15	1.53	120	82	8795	1.19
P	14.59	25	16	4.39	120	50	11548	1.18
E	39.94	25	15	11.66	270	94	39954	1.20
K	10.66	25	15	2.14	120	68	9453	1.20
G	39.71	25	19	20.83	120	51	32947	1.20
I	30.38	25	15	7.04	120	66	27616	1.19
J	17.82	25	15	5.07	120	46	13943	1.20
M	13.32	90	30	6.86	120	67	10074	1.17

6.9 Discussion of Modelling Results

The XP-RAFTS modelling undertaken, has determined that the proposed detention storages are adequate to restrict post development peak discharges from the site, to pre-development levels for the 5-year and 100-year ARI storm events. The results of this modelling have been reported in Tables 6.3 and 6.4 and demonstrate compliance with the "Blacktown City Council Growth Centres Precincts Development Control Plan 2009" flooding and water cycle management objectives.

6.9.1 Bells Creek Catchment (Eastern)

Detention Basins G, I and J discharge into culverts under Richmond Road before continuing on in an easterly direction towards Bells Creek within open channels (TC8B, TC13 and TC11 respectively). Apart from the residential sub-catchments draining to Detention Basin M, all other catchments east of Richmond Rd. discharge directly into Bells Creek without detention. These discharge points are shown on Figure 3 and denoted as Nodes B-1.1.06, B-2.1.02, B-3.1.06, B-3.8.02, B-4.1.05 and B-4.7.01.

To compensate for the sub-catchments without detention, the storage capacities of Basins G, I, J and M have been increased. The Existing and Developed with Detention peak flows in Bells Creek can be compared in Tables 6.3 and 6.4 at Nodes 1.15, 1.16, 1.17, 1.18. 1.19 and 1.20.

The results from the XP-RAFTS model generally exhibit a slight reduction in peak flows in Bells Creek at these Nodes. Consequently, the Hydrologic Analysis confirms that the goal for the Eastern Catchment of "not increasing peak flows in Bells Creek" has been demonstrated.

NOTE This strategy has been based on limiting the number of discharge points from the developed catchment into Bells Creek and its tributaries. The location of each discharge point shall be included with each Development Application for the relevant stage and the total number of discharge points identified in this strategy should not be exceeded without the approval of Blacktown City Council.

6.9.2 North-South Catchment (Centre)

The storage volume for Basin E has been increased to compensate for 20.66 ha which discharges directly into Marsden Park to the east of Basin K (see Nodes N-1.06 and N-1.07 on Figure 3). The point of comparison for peak flows from the Existing and Developed with Detention catchments is Node N-S (see Tables 6.3 and 6.4).

The slight decrease in peak flows determined at Node N-S demonstrates compliance with the goal for the North-South Catchment of "not increasing peak flows through Marsden Park."

6.9.3 Western Catchment (Little Creek)

Approximately 40 ha of an external residential catchment from Bidwill drains into the Little Creek catchment through the southwest corner of the site. This catchment is denoted by Nodes W-1.1.00, W-1.2.00 and W-1.9.00 on Figure 3. Basin A has the storage capacity to attenuate this external catchment as well as the internal catchments within the south western corner of the MPIP. Refer to Heading 6.5 *XP-RAFTS Model* for a discussion of the on-site detention and water management required to be undertaken on the smaller 8.26 ha south western sub-catchment situated to the west of South Street (see Plate 3).

Basin B detains and treats a small internal catchment centrally located on the western fringe of the site. The catchment controlled by both Basins A and B crosses South Street and joins to the west of the MPIP in Shanes Park at Node W-1. See Tables G.3 and 6.4 for a comparison of Existing and Developed with Detention peak flows, which confirms compliance with "no increase in peak flows into the southern reach of Little Creek."

Detention Basin P controls runoff from the northwest corner of the site. A comparison of Existing and Developed with Detention peak flows can be found at Node W-2 in Tables 6.3 and 6.4, which identifies a slight reduction and confirms compliance with "no increase in peak flows into the north-eastern reach of Little Creek."

6.10 Climate Change Sensitivity Assessment

Preliminary assessments of hydrologic impacts, resulting from changes to rainfall patterns as a consequence of Climate Change, were undertaken to determine the impact of such changes on the performance of the proposed Trunk Drainage system. These assessments followed the sensitivity analysis procedures recommended in the *NSW Climate Change Action Plan, DECC (October 2008)* (Ref.2). A copy of the Climate Change assessment is provided in Appendix A.

In summary:

- Summer runoff depths are expected to increase by a maximum of 26%; and
- The 40-year 24-hour duration rainfall intensity is expected to increase by a maximum of 12%.
- The net average annual runoff is expected to fluctuate with an overall minor increase.

Consequently for the purposes of this assessment, the worst-case scenario of projected increased rainfall intensities (15% increase) and runoff depths (25% increase based on rainfall intensities increased by 15%), were adopted.

The hydrologic modelling determined that the volumes and surface areas for the Detention Basins could remain the same as those determined for existing conditions providing the pre and post development rainfall intensities are both increased by 15%. However it will be necessary, to account for the variability in the predicted decadal increases in rainfall intensities, to design a flexible outlet arrangement as part of the detailed engineering designs required for individual Development Applications.

This approach has been referred to DECCW for comment and has been confirmed as "a pragmatic approach to considering the impacts of Climate Change on urban drainage systems."

6.11 Summary of Release Area Detention Basins

A summary of the storage volumes and associated drainage reserve areas for each of the various detention basins proposed to service the Marsden Park Industrial Precinct is provided in Table 6.7. This table also summarises the changes in basin reserve areas that have occurred as a result of the refinement of the "pre-exhibition" basin strategy developed for the Precinct previously (Ref. 1).

Table 6.7

SUMMARY OF RELEASE AREA DETENTION BASINS

Basin	Volume (m³)	Drainage Reserve Area (ha)	Reserve Area Difference to Pre-Exhibition (ha)	Remarks
A	30,500	2.93	-1.3	On-line Basin
B	9,000	1.35	-1.1	
C		0.00	-1.7	Replaced by Basin P
D		0.00	-2.5	Replaced by larger Basin E
E	40,000	6.04	0.8	
G	33,000	4.60	-2.7	On-line Basin
I	28,000	3.85	-1.8	
J	14,000	1.96	-0.2	
K	9,500	1.35	0.0	
L		0.00	-1.5	Replaced by Basin P
M	10,500	1.52	-0.9	
O	*3,000*	*1.29*	*-0.3*	
P	12,100	1.66	1.7	Replaces Basins C and L
All Basins	189,600	26.55	-11.4	

NOTE: *Basin O is not required as a regional detention Basin. Its details have been included to assist with the determination of future on-site detention requirements. It has not been included in the ILP or the BCC Section 94 Plan.*

7 STORMWATER QUALITY ANALYSIS

The stormwater quality analysis for this study was undertaken using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC version 3.01) (Ref. 15). This water quality modelling software was developed by the Cooperative Research Centre (CRC) for Catchment Hydrology, which is based at Monash University and version 3.01 was released in May 2005.

MUSIC makes it possible to:

- estimate the potential nutrient reduction benefits of gross pollutant traps, constructed wetlands, grass swales, bio-retention systems, sedimentation basins, infiltration systems as well as model stormwater re-use as a treatment technique;
- evaluate compliance with water quality objectives; and
- assist in determining the Stream Erosion Index (refer to Section 9).

BCC and DECCW have established default parameters for use in MUSIC models to represent the generation of various pollutants by different land uses. A MUSIC model of the proposed MPIP development including the Water Cycle Management Strategy was constructed to demonstrate compliance with the adopted Environmental Stormwater Objectives adopted by the GCC and the post development annual load reductions in Part R of the Draft BCC DCP 2006.

7.1 Catchments

7.1.1 Generic On-Lot Treatment Layout for Commercial/Industrial and Medium/High Density Residential Developments

A generic treatment node, representing a 2 ha commercial/industrial lot, was constructed and incorporated into a MUSIC model. The layout of the on-lot treatment proposed for the generic commercial/industrial lot is shown on Plate 4. The benefit provided through the use of a rainwater tank on each allotment was modelled using the following design assumptions:

- Half the roofed areas (representing 30% of the site area) will discharge to the rainwater tanks for re-use, with overflows discharging into the on-lot raingarden.
- Half the roofed areas (representing 30% of the site area) have been estimated to by-pass the rainwater tanks and discharge directly to the on-lot raingarden.
- All hardstand areas (representing 30% of the site area) will direct runoff, up to the 3-month ARI, into a generic vortex-type GPT, outflows from which will discharge directly into the on-lot raingarden.
- Landscaped areas (representing the remaining 10% of the site area) will discharge directly into the bio-retention device.
- For the purpose of estimating the pollutant load reduction achieved by the generic on-lot raingardens, a filter media depth of 0.6 m was assumed with an average particle size diameter of 0.5 mm and a hydraulic conductivity of 100 mm/hr.

For the purposes of estimating the pollutant load reduction achieved by the on-lot generic treatment, the GPT and raingarden were sized on basis of treating only the runoff from events less than the 3-month ARI with flows in excess of this bypassed directly to the formal drainage network.

The surface area of the on-lot generic raingarden was adjusted until the required water quality targets were achieved. Compliance with the targets occurred when the surface area of the raingarden was configured at 1.0% of the total catchment area of the commercial/industrial lot i.e. when the surface area of the raingarden reached 200 m^2 for a total contributing catchment area of 2 ha.

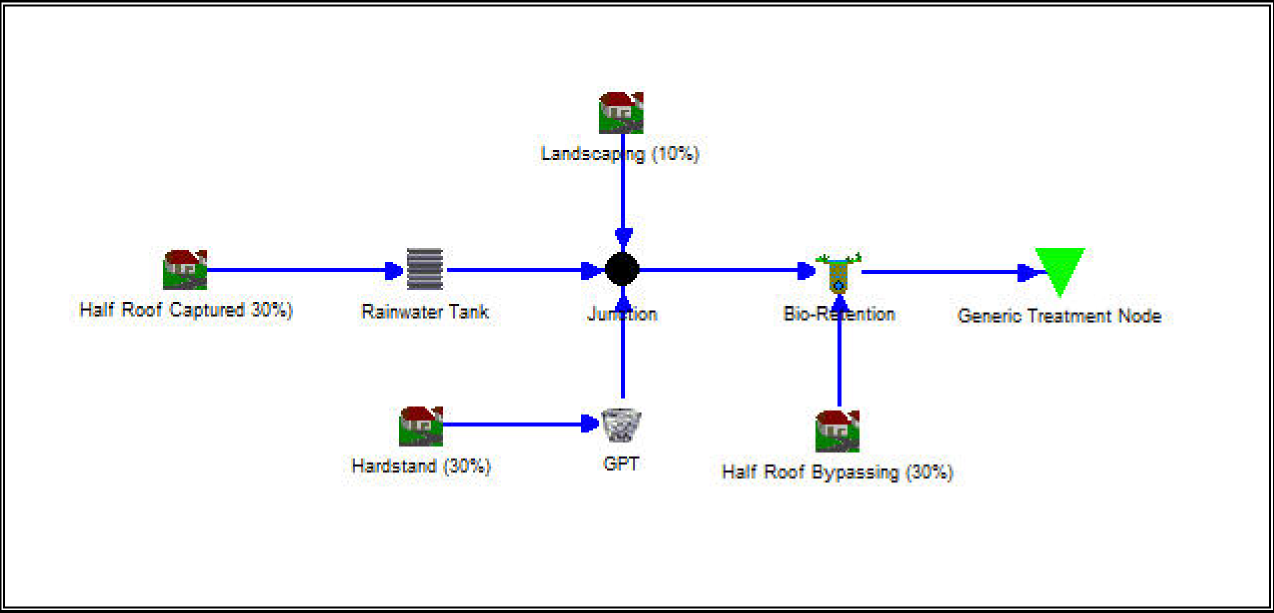

Plate 4: On-Lot MUSIC Model Layout
(8955 JWP Lot Treatment.sqz)

Part R of the Draft BCC DCP 2006 contains information to assist with the design of individual on lot treatment elements based on achieving their pollutant load reduction criteria.

7.1.2 Overall Release Area Treatment Layout

A MUSIC model was also established for the proposed Water Cycle Management Strategy proposed to treat the runoff from the public domain infrastructure. Figure 3 shows the extent of the catchments used in this model and Plate 5 shows the general arrangement and construction of the MUSIC model to determine compliance with the required water quality targets.

A fraction impervious of 0.90 was adopted for commercial, industrial and new high/medium density residential catchments (including half the road). A fraction of imperviousness of 0.85 was adopted for low-density residential developments (including half the road).

The runoff from all privately owned commercial/industrial and medium/high density residential catchments is intended to discharge through the on-lot water quality elements, as described earlier, before flowing into the formal drainage network. The public domain infrastructure, which includes the public roads, will be connected directly to formal drainage network before being treated in a vortex-type GPT prior to discharging into the either the downstream formal drainage network and/or the co-located raingardens and Detention Basins.

Where runoff, from the public domain infrastructure, bypasses the co-located raingardens and Detention Basins, the devices have been oversized to compensate by treating the runoff from the private sector to a higher standard. The point of comparison with the required treatment and peak flow reduction criteria is the point of connection with the existing watercourse for the respective catchment. Catchments that discharge directly into watercourses external to the MPIP have been minimised.

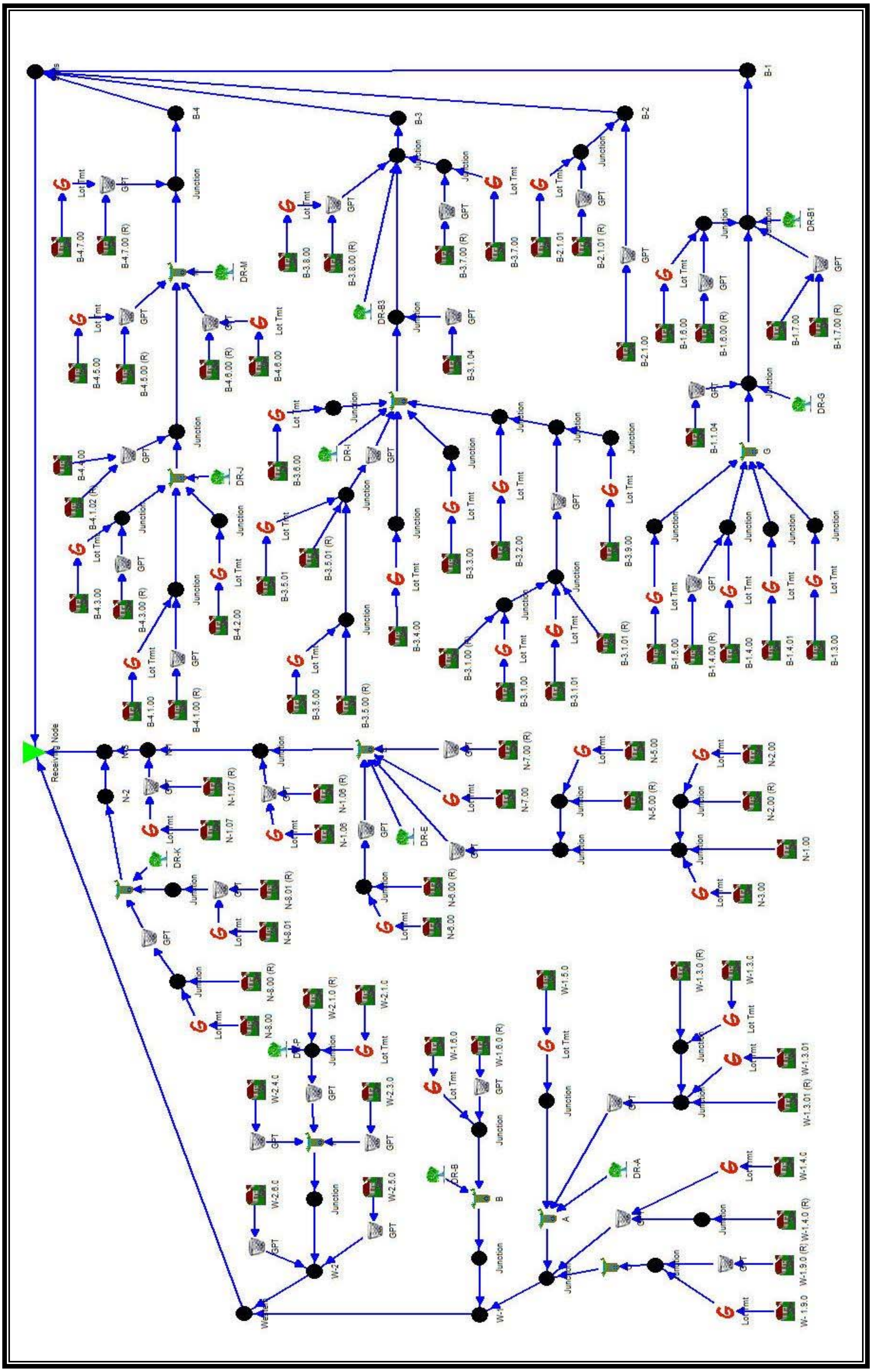

Plate 5: Overall Release Area MUSIC Model Layout (8955MU_06_Total.sqz)

7.2 Rainfall Data

The MUSIC model is able to utilise rainfall data based on 6 minute, hourly, 6 hourly and daily time steps. A 6 minute time step was chosen for this analysis, which is in accordance with the recommendations within the MUSIC Users Manual (Ref. 15).

Rainfall records for the area were obtained from Blacktown City Council. The station used and the years of record selected were determined by BCC and are tabulated below.

Station No	Location	Years of Record	Type of Data
67035	Liverpool (Whitlam Centre)	1967 - 1976	6 minute

Upon interrogation of the rainfall data provided by the Bureau of Meteorology for the Station No. 67035, it was noticed that a significant amount of data between 1974 and 1976 was missing. Consequently another data set was secured from BCC which included replacement rainfall data for this missing period. The amended 6 minute rainfall data was analysed and found to compare favourably with data from other local rainfall stations.

A summary of the amended rainfall data set (Liverpool 1967 – 1976) used in the MUSIC model for the MPIP and comparable rainfall data sets provided by the Bureau of Meteorology rainfall stations gauges in Seven Hills, Richmond and Orchard Hills (Penrith) is shown below in Table 7.1.

Table 7.1

**SUMMARY OF RAINFALL DATA
FOR THE SITE**

Property	Bureau of Meteorology Data - Seven Hills (1950 - 2009)	Bureau of Meteorology Data - Orchard Hills (1971 - 2009)	Bureau of Meteorology Data - Richmond (1881 - 2009)	Bureau of Meteorology Data - Average Between Three BOM Stations	MUSIC Model Data Set - Liverpool (Whitlam Centre) (1967 - 1976)
Mean Yearly Rainfall (mm)	915	801	801	839	835
Decile 1 Rainfall (mm)	627	503	529	553	638
Decile 5 Rainfall (mm)	900	782	793	825	833
Decile 9 Rainfall (mm)	1178	1036	1070	1094	1119
Mean No. Rain Days	112	101	116	109	110
Mean No. Rain Days > 1mm	85	77	77	80	77
Mean No. Rain Days > 10mm	26	23	22	24	23
Mean No. Rain Days > 25mm	8.8	7.8	7.2	7.9	8.2

7.3 Soil / Groundwater Parameters and Pollutant Loading Rates

In the absence of site specific data, the soil / groundwater parameters, adopted for the urban catchments of the Marsden Park site, were based on the recommended parameters provided by the Department of Environment and Climate Change for areas within Western Sydney (Ref. 16). The adopted parameters are also consistent with the values recommended by BCC (Ref. 6) and are presented in Table 7.2.

Table 7.2

ADOPTED SOIL / GROUNDWATER PARAMETERS FOR THE SITE
(Source: DECC Technical Note – Ref. 16)

	Units	Urban	Non-Urban
Impervious Area Parameters			
Rainfall threshold (Roof 0.5, Road 1)	mm/day	1.4	1.4
Pervious Area Parameters			
Soil storage capacity	mm	170	210
Initial storage	% of capacity	30	30
Field capacity	mm	70	80
Infiltration capacity coefficient - a		210	175
Infiltration capacity coefficient - b		4.7	3.1
Groundwater Properties			
Initial depth	mm	10	10
Daily recharge rate	%	50	35
Daily baseflow rate	%	4	20
Daily deep seepage rate	%	0	0

The pollutant loading rates adopted for the urban catchments within the MPIP are based on the recommended parameters provided by the Cooperative Research Centre for Catchment Hydrology (Ref.17). These values are consistent with the values recommended for use by BCC (Ref. 6) and have been presented in Table 7.3.

Table 7.3

ADOPTED EVENT MEAN CONCENTRATIONS
(Source: CRCCH – Ref. 17)

	Roofs		Roads		Remaining Urban		Drainage Corridor	
Pollutant	Base Flow (mg/L)	Storm Flow (mg/L)	Base Flow (mg/L)	Storm Flow (mg/L)	Base Flow (mg/L)	Storm Flow (mg/L)	Base Flow (mg/L)	Storm Flow (mg/L)
TSS	-	20.0	-	269	15.8	141	6.03	39.8
TP	-	0.129	-	0.501	0.141	0.251	0.032	0.079
TN	-	2.00	-	2.19	1.29	2.00	0.302	0.891

7.4 Treatment Device Performance

Each element of the series of treatment practice (commonly referred to as a treatment train), as represented in the MUSIC model for the MPIP, is described below.

NOTE: *Part R Blacktown Development Control Plan 2006 (Draft)* requires that the post development average annual load of Total Hydrocarbons be reduced by 90%. MUSIC is unable to estimate load reductions for Total Hydrocarbons. However research suggests that Bio-retention systems are very effective at reducing Total Hydrocarbon loads and raingardens have been adopted as the preferred treatment strategy for hydrocarbons throughout the MPIP. However, final Total Hydrocarbon treatment shall be determined as part of the Development Approval process for each stage of the MPIP.

7.4.1 Rainwater Tanks

The impacts of the use of rainwater tanks, provided on each allotment, were modelled in the generic On-Lot Treatment strategy using the "Rainwater Tank" node with the following design assumptions:

Minimum Connected Roof Area

It has been assumed that 50% of all of the roofed areas will be directly connected to rainwater tanks. The remaining 50% of the roof area is assumed to by-pass the rainwater tanks and discharge directly to the on-lot raingarden.

Average Rainwater Tank Size

The nominal rainwater tank size adopted in the investigation was based on 100,000 litres for every 12,000 m^2 of site roof area. (Ref.1, GHD WCMA, July 2009)

Average Reuse

The average reuse amount adopted in the investigation was split into two components based on internal and external usage. The following amounts were based on a typical 2.0 hectare site with a 12,000 m^2 development:

- Average Annual Internal Water Usage (Daily Demand) 2.0 kl/day
- Outdoor (Annual Demand scaled by daily PET) 1,500 kl/yr

Consequently the average total daily reuse, adopted in the MUSIC model, has been estimated as 6.1 kilolitres per day per 2.0 ha industrial/commercial development.

7.4.2 Litter and Sediment Control Structures

Drainage systems collecting runoff from local roads and hardstand areas, throughout the MPIP, have been modelled with GPTs to remove litter and coarse sediment prior to discharge into the downstream drainage systems, bio-retention raingardens and riparian corridors. GPTs are available as inlet pit filter inserts, purpose built cast in situ systems or precast proprietary traps using either dry or wet sump storage chambers.

BCC has a preference for proprietary wet sump GPTs which use a vortex technology to separate the pollutants out of the water column. The criterion, used to assess the performance of the GPTs in the MUSIC model, was based on the credit given to vortex-type GPTs (Ref.6, p.81) i.e. Total Suspended Solids (TSS) - 70% for concentrations > 75 mg/L, and Total Phosphorus (TP) - 30% for concentrations > 0.5 mg/L. No credit was given to the GPTs capacity to remove oils, other nutrients or metals. However, if required it is possible to incorporate oil skimming or oil absorbent materials within a wet sump GPT for the purpose of removing non-emulsified, free floating oils. It is expected that the site drainage strategy would require approximately 40 major GPTs (at least one per bio-retention system and at road connections into trunk drainage systems). Wherever possible, dewatering systems should be provided to facilitate de-watering of the wet sumps. These dewatering lines must be discharged to the raingardens or some other appropriate filtration system to allow nutrients and fine particulates to be stripped out of the supernatant water. The approximate locations of the proposed GPT units are indicated on the MUSIC Model Layout at Plate 5 and included on the individual Preliminary Engineering Concept drawing for each Detention Basin.

Since the effectiveness of pollutant load removal varies between different GPT devices, the MUSIC modelling assumed the indicative pollutant removal as documented in Council's WSUD DCP for vortex-type GPT's.

7.4.3 Bio-Retention Systems (Raingardens)

Ten (10) co-located raingarden bio-retention/filtration systems are proposed throughout the MPIP. The proposed development layout facilitates the provision of co-located raingardens within the Detention Basins. Wherever possible the co-located raingardens are located off-line from the major inflows into the Detention

Basins to limit scouring of the filter media; preserve the vegetation; and minimise the re-mobilisation of pollutants.

The media beds of the bio-retention systems are typically 600 mm deep with an average particle size of 0.5 mm and a hydraulic conductivity of 100 mm/hr with a minimum depth of storage above the media of 300 mm. A discharge control structure can be configured (during the Development Application process) to promote extended detention times if required.

Treatment is attained by detaining flows to promote sedimentation, direct filtration of particulate matter and nutrient stripping by bio-films which establish on the surface of the media bed and within the gravel layer. The organic sandy loam bed and plant system minimises evaporation losses and the raingarden will be constructed with an impermeable barrier to prevent seepage losses and to avoid groundwater salinity impacts.

The location and size of the proposed co-located raingarden systems are shown on the respective attached Preliminary Engineering Concept drawings for each Detention Basin. The general features and configuration of the proposed raingardens for the MPIP, as modelled in MUSIC, are detailed in Table 7.4.

Table 7.4

MAJOR BIO-RETENTION SYSTEMS
GENERAL FEATURES AND CONFIGURATIONS

Basin Name	A	O	B	P	E	K	G	I	J	M
Storage Properties										
Extended Detention Depth (m)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Surface Area (m^2)	1600	350	800	2500	7400	1000	6000	5000	3000	3000
Seepage Loss (mm/hr)	0	0	0	0	0	0	0	0	0	0
Infiltration Properties										
Filter Area (m^2)	1600	350	800	2500	7400	1000	6000	5000	3000	3000
Filter Depth (m)	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Filter Media Particle Diameter (mm)	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Saturated Hydraulic Conductivity (mm/hr)	100	100	100	100	100	100	100	100	100	100
Outlet Properties										
Overflow Weir Width (m)	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0

NOTE: *Basin O is not required as a regional stormwater Bo-retention system. Its details have been included to assist with the determination of future on-site water quality requirements. It has not been included in the ILP or the BCC Section 94 Plan.*

The expected sediment and nutrient removal performance of the bio-retention systems was determined using the default equations and parameters provided in the MUSIC model (Ref. 15). The water quality reduction mechanisms in MUSIC are based on an exponential decay equation referred to as the k – C* curve (refer to Wong et al. – Ref. 19).

The performance parameters used in the MUSIC model are summarised in Table 7.5. The viability of the raingarden and the longevity of its pollutant removal efficiency is dependent on the capacity of the pre-treatment GPTs to intercept and remove light litter, detritus and coarse sediment.

Table 7.5

MUSIC – PERFORMANCE PARAMETERS

Pollutant	Bio-Retention k (m/yr)	Bio-Retention C* (mg/L)	Rainwater Tanks k (m/yr)	Rainwater Tanks C* (mg/L)
TSS	8000	20.000	400	12.000
TP	6000	0.130	300	0.130
TN	500	1.400	40	1.400

The surface area of the individual co-located bio-retention systems for the MPIP has been determined using the MUSIC water quality model. A summary of the estimated performance of each system is detailed in Section 7.5 of this report, and their configuration is indicated on the respective attached Preliminary Engineering Concept drawing for each Detention Basin. (Refer to Appendix D)

Once the catchments upstream of the co-located raingardens are stabilised, their maintenance would generally involve plant replacement, weed control, repair of localised erosion and minor structural damage, the removal of localised sediment build-up, and checking for any reduction in infiltration capacity and hydraulic conductivity of the media. This would be undertaken on a quarterly basis on average with media and vegetation replacement budgeted for on a decadal cycle.

7.5 Pollutant Load Estimates

Total annual pollutant load estimates were derived from the results of a MUSIC model based on a stochastic assessment of the developed site incorporating the proposed water quality treatment system. The estimated annual pollutant loads and reductions for TSS, TP, TN and Gross Pollutants (for each sub-catchment) are presented in Tables 7.6, 7.7, 7.8 and 7.9, respectively with a summary for the whole of the MPIP presented in Table 7.10.

Table 7.6

SUMMARY OF ESTIMATED MEAN ANNUAL POLLUTANT LOADS AND REDUCTIONS – TOTAL SUSPENDED SOLIDS

Location (node)	Total Development Source Loads (kg/yr)	Minimum Reduction Required (kg/yr)	Total Residual Load from Site (kg/yr)	Total Reduction Achieved (kg/yr)	Total Reduction Achieved (%)
Basins A & O	77,200	65,620	11,000	66,200	**85.8%**
Basin B	25,000	21,250	2,420	22,580	**90.3%**
W-1	102,200	86,870	13,420	88,780	**86.9%**
W-2	47,200	40,120	6,890	40,310	**85.4%**
Western	149,400	126,990	20,200	129,090	**86.4%**
N-1	155,000	131,750	21,600	133,400	**86.1%**
N-2	30,000	25,500	3,140	26,860	**89.5%**
N-S	185,000	157,250	24,740	160,260	**86.6%**
B-1	73,700	62,645	9,960	63,740	**86.5%**
B-2	8,680	7,378	2,800	5,880	**67.7%**
B-3	106,000	90,100	12,500	93,500	**88.2%**
B-4	91,800	78,030	10,300	81,500	**88.8%**
Bells	280,180	238,153	35,560	244,620	**87.3%**
Total Site	614,580	522,393	80,500	533,970	**86.9%**

Table 7.7

SUMMARY OF ESTIMATED MEAN ANNUAL POLLUTANT LOADS AND REDUCTIONS – TOTAL PHOSPHORUS

Location (node)	Total Development Source Loads (kg/yr)	Minimum Reduction Required (kg/yr)	Total Residual Load from Site (kg/yr)	Total Reduction Achieved (kg/yr)	Total Reduction Achieved (%)
Basins A & O	126	81.9	43.9	82.1	**65.2%**
Basin B	40.7	26.455	12.2	28.5	**70.0%**
W-1	166.7	108.355	56.1	110.6	**66.3%**
W-2	77	50.05	26.8	50.2	**65.2%**
Western	243.7	158.405	82.9	160.8	**66.0%**
N-1	253	164.45	82.7	170.3	**67.3%**
N-2	48.8	31.72	13.7	35.1	**71.9%**
N-S	301.8	196.17	96.4	205.4	**68.1%**
B-1	120	78	41.3	78.7	**65.6%**
B-2	14.4	9.36	10.50	3.9	**27.1%**
B-3	174	113.1	54.4	119.6	**68.7%**
B-4	151	98.15	46.2	104.8	**69.4%**
Bells	459.4	298.61	152.4	307	**66.8%**
Total Site	1004.9	653.185	331.7	673.2	**67.0%**

Table 7.8

SUMMARY OF ESTIMATED MEAN ANNUAL POLLUTANT LOADS AND REDUCTIONS – TOTAL NITROGEN

Location (node)	Total Development Source Loads (kg/yr)	Minimum Reduction Required (kg/yr)	Total Residual Load from Site (kg/yr)	Total Reduction Achieved (kg/yr)	Total Reduction Achieved (%)
Basins A & O	849	382.05	425	424	**49.9%**
Basin B	265	119.25	127	138	**52.1%**
W-1	1114	501.3	552	562	**50.4%**
W-2	502	225.9	276	226	**45.0%**
Western	1616	727.2	831	788	**48.8%**
N-1	1700	765	814	886	**52.1%**
N-2	311	139.95	143	168	**54.0%**
N-S	2011	904.95	957	1054	**52.4%**
B-1	811	364.95	365	446	**55.0%**
B-2	73.9	33.255	58.9	15	**20.3%**
B-3	1110	499.5	510	600	**54.1%**
B-4	927	417.15	412	515	**55.6%**
Bells	2921.9	1314.855	1345.9	1576	**53.9%**
Total Site	6548.9	2947.005	3133.9	3418	**52.2%**

Table 7.9

SUMMARY OF ESTIMATED MEAN ANNUAL POLLUTANT LOADS AND REDUCTIONS – GROSS POLLUTANTS

Location (node)	Total Development Source Loads (kg/yr)	Minimum Reduction Required (kg/yr)	Total Residual Load from Site (kg/yr)	Total Reduction Achieved (kg/yr)	Total Reduction Achieved (%)
Basins A & O	10,700	9,630	174	10,526	**98.4%**
Basin B	3,350	3,015	26.1	3,324	**99.2%**
W-1	14,050	12,645	200	13,850	**98.6%**
W-2	6,260	5,634	229.0	6,031	**96.3%**
Western	20,310	18,279	418	19,881	**97.9%**
N-1	21,100	18,990	489	20,611	**97.7%**
N-2	3,820	3,438	30.6	3,789	**99.2%**
N-S	24,920	22,428	520	24,400	**97.9%**
B-1	10,300	9,270	1,090	9,210	**89.4%**
B-2	864	778	133	731	**84.6%**
B-3	13,500	12,150	406	13,094	**97.0%**
B-4	11,300	10,170	195	11,105	**98.3%**
Bells	35,964	32,368	1,824	34,140	**94.9%**
Total Site	81,194	73,075	2,762	78,421	**96.6%**

Table 7.10

SUMMARY OF ESTIMATED MEAN ANNUAL POLLUTANT LOADS – WHOLE OF THE MPIP

Pollutant	Existing Conditions (kg/yr)	Developed (no controls) (kg/yr)	Developed (with Controls) (kg/yr)	Percent Reduction	Reduction Target
Gross Pollutants	0	81,210	2,750	95%	90%
TSS	21,200	613,880	80,380	87%	85%
TP	100	1,000	330	65%	65%
TN	850	6,550	3,130	50%	45%

Note: *The pollutant reduction estimates, determined by the MUSIC model and displayed in Tables 7.6 to 7.10, include a reference to the combined reductions provided by the regional Bio-retention Basin A and the on-site Bio-retention Basin O, thereby demonstrating that the pollution reduction targets can be met by the proposed strategy.*

Loads have been rounded to the nearest 10 kgs and Percent Reduction to the nearest 5%.

They are intended for comparison purposes only.

Existing Loads are indicative and have been calculated on the basis of the generic loading rates described in Ref.17.

Developed Loads have been determined using a stochastic MUSIC model.

7.6 Discussion of Modelling Results

Table 7.10 provides a summary of the total load of pollutants anticipated to emanate from the site both before and after the proposed development. The Existing Conditions estimates are based on information provided by the CRCCH (Ref.17). It assumes that the site is being used for agricultural activities and that these activities do not generate Gross Pollutants.

The performance of the proposed water quality management strategy for the MPIP, as determined through a stochastic MUSIC assessment, is summarised in Tables 7.6, 7.7, 7.8 and 7.9 and estimates that:

- Total Suspended Solids (TSS) will be reduced by approximately 521,050 kg/yr which equals the 85% load reduction target of 521,050 kg/yr;
- Total Phosphorus (TP) will be reduced by approximately 650 kg/yr which equals the 65% load reduction target of 650 kg/yr;
- Total Nitrogen (TN) will be reduced by approximately 3,270 kg/yr which exceeds the 45% load reduction target of 2,940 kg/yr;
- Gross Pollutants will be reduced by approximately 77,150 kg/yr which exceeds the 90% load reduction target of 73,090 kg/yr;

Consequently it has been demonstrated, through the use of a MUSIC based assessment, that the combination of rainwater tanks, gross pollutant traps and raingarden bio-retention systems will, when configured according to the “treatment train” proposed for the Marsden Park Industrial Precinct, reduce priority pollutant loads to the required minimum pollution control targets required by Blacktown City Council and the NSW Department of Planning.

NOTE: Basin O has been included in the assessment in order to facilitate a comparison between existing and developed pollutant loads. Basin O is intended to manage stormwater runoff from an 8.26 ha catchment (zoned Industrial IN2) in the south western corner of the MPIP, and it will be the responsibility of the future land owner to both design and build this Basin. The on-site Basin O referred to herein has not been included on the ILP or in the BCC Section 94 Plan.

8 TRUNK DRAINAGE CHANNEL ANALYSIS

The existing flowpaths through the MPIP, which are proposed to be converted to open channels, will be vegetated in accordance with the BCC landscaping requirements for a Category 3 Stream. The twelve (12) channels identified will have the capacity to convey flows up to the 100-year ARI storm event including a 0.5 m freeboard above the 100-year water level estimated for each channel. Concept sizes for these channels have been determined for the purposes of establishing drainage corridor widths and assisting the process of preparing the Section 94 Contribution Plan (see Appendix C)

All the channels, except channel TC8B, were assessed using Manning's Equation to determine 'normal depths' in trapezoidal channel cross-sections. The use of 'normal depth' calculations is applicable for the channels west of Richmond Road which are expected to exhibit uniform flow regime characteristics. East of Richmond Road there are three (3) channels that are within close proximity to Bells Creek and are likely to be influenced by the water levels in Bells Creek, and where this occurs the affected channels have been analysed more robustly using HEC-RAS.

Channel TC11 discharges into Basin M and is therefore not subject to the influence of Bells Creek. Likewise, an initial check downstream of Basin I identified that the Bells Creek 100-year flood level of 26.4 m AHD is much lower than the outlet invert level of channel TC13. Consequently, 'normal depth' calculations were assessed as being appropriate to assess the conveyance capacity of channels TC11 (upstream of Basin M) and TC13 (downstream of Basin I).

The lower reach of TC8, downstream of Richmond Road, has been designated as reach TC8B and is affected by tailwater from Bells Creek which makes it unsuited to a 'normal depth' assessment using Manning's Equation. Consequently an assessment of its conveyance capacity and associated channel profile has been undertaken using HEC-RAS. The output from the HEC-RAS analysis includes flood levels, flow velocities and Froude numbers and these are shown graphically in Appendix C.

8.1 Use of Manning's Equation to Size Channels

The key parameters adopted for sizing the channels using Manning's equation were:

- Channels to convey 100-year ARI flow
- Maximum flow depth of 1 m
- Minimum Freeboard above the 100-year ARI top water level of 0.5 m to finished ground level with a further 0.2 m to finished floor level (FFL)
- Embankment side slopes of 4H:1V (based on the use of vegetated low maintenance batters)
- Channels to be vegetated to replicate a Manning's roughness coefficient of 0.07
- Flow velocity not to exceed 1.63 m/s (desirable)
- Minimum channel gradient of 0.5%

The channels will be vegetated and in the long term are assumed to have a Manning's roughness of 0.07, which replicates the style of landscaping required by the DECCW and BCC. Natural Channel Design Guidelines (Ref.21) recommends that the bankfull flow velocity within channels exhibiting these characteristics does not exceed 1.63 m/s. Velocities in excess of this allowable velocity may cause damage to the vegetation and possibly result in failure of the channel invert and embankments.

The 100-year flows used in the sizing of the channels were taken from the results detailed in Section 6 Hydrologic Assessment for the post-development scenario. A summary of the 100-yr Design Flows and the individual channel characteristics are provided in Appendix C.

The freeboard for channel TC12A was checked for post climate change 500-year ARI flows to ensure Richmond Road, being a flood evacuation route, would be flood free during a local 500-year flooding event. This check determined that a freeboard of 0.3 m remained from the 500-year top water level to the top of the channel embankment.

The depth of flow in all channels except TC12A and TC8B was restricted to 1 m. At this point in time it is likely that channel TC12A will be constructed as a box culvert. However, for the current intention of determining the potential land-take required for the flow path, the channel was assumed to be an open vegetated trapezoidal channel. Consequently the resulting width for a channel should result in a conservative estimate whereas the width required for a box culvert would be much narrower.

8.2 Modelling of Channel TC8B

Cross-sections for use in the HEC-RAS model were extracted from the existing terrain model, commencing from immediately downstream of Richmond Road down to a point approximately 50 m past the ILP boundary. Four sections, approximately 50 m apart, were determined for the hydraulic analysis.

A riparian corridor is indicated on the ILP for the entire reach of channel TC8B. Except for some filling and localised reshaping work, the intention is to retain the meandering alignment, incised channel and invert levels of the existing TC8B channel. To contain overbank flows, filling outside of the existing channel will be necessary with batter slopes to be no steeper than 4H:1V. As with the other channels, the side slopes of TC8B will be vegetated and hence a Manning's roughness of 0.07 was used to describe this reach. The grade of channel TC8B varies between 0.1 % and 2% with an average gradient of about 1.2%. The steepest sections are located in the upstream and downstream reaches whilst the flattest section is along the lower middle reach.

Steady flow analysis in HEC-RAS was performed assuming normal depth occurring in the upstream reach of the channel and with the downstream end controlled by the 100-year flood level in Bells Creek, which was adopted as 29.65 m AHD (based on GHD Tuflow model for Bells Creek). A slope of 0.92% was adopted for the Energy Gradient at the upstream end of the channel, which is consistent with the general bed slope at this location. These criteria formed the downstream control for the HEC-RAS analysis.

Flood levels, flow velocities and shearing stresses for selected sections along the creeks were determined from this analysis. This information is required to assist in assessing the capacity of the creek to convey 100-year flows without adversely impacting on the riparian vegetation. By increasing the 100-year top water levels by an additional 0.5 m to allow for freeboard it was then possible to determine the appropriate fill levels for development outside the riparian areas. Channel widths, depths, grades and freeboard are provided in Appendix C and Indicative Design Contours for the surrounding area are shown on the Engineering Concept Plan for CTC8B at Appendix D – 8955/SK6.

8.3 Maintenance Provisions

A 5 m wide access track and a 2.5 m alternative access track have been included in the determination of the width of the Trunk Drainage Reserve where the trunk drainage channel does not run parallel to an adjoining road. The 5m wide major access is for light truck maintenance access whilst the 2.5 m wide all weather access "along the alternate top of the channel (if not a road frontage)" is to provide emergency and rescue access.

BCC recommend that where channels do not run parallel to an adjoining road provision should be made, within the Drainage Reserve for an additional 7.5 m (5 m major and 2.5 m minor) in excess of the width required for conveyance capacity. Where an internal road runs to one side of the channel, an additional 2.5 m wide minor access track is also recommend along the opposite side. Suitable access provisions for each Trunk Channel will be determined prior to the lodgement of the Development Application for the respective Trunk Channel.

8.4 Discussion of Modelling Results

By fixing the depth of flow in the channel, the width required for hydraulic conveyance varies in proportion to the 100-year flow to be conveyed. Channels TC7A, TC7B, TC7C and TC11 have the narrowest width requirement of about 18 m (between top of banks) while channel TC8A, which conveys flows from an external catchment, requires the widest width of about 43 m to maintain conveyance capacity.

Results of the HEC-RAS analysis for channel TC8B show side channel velocities were generally less than 1.63 m/s with the highest velocity (2.43 m/s), recorded centrally within the existing channel, at River Station 100.00. This, higher than desirable velocity, was not considered a risk as this reach of TC8B was subject to downstream boundary conditions dominated by Bells Creek and local controls will be implemented as part of the relevant Development Application (see Appendix C and Plan No. 8955/SK17).

A summary of the channel characteristics, resulting from the 'normal depth' analyses and the required width of each channel is given in Appendix C. A tabulated summary of results for the HEC-RAS analysis of channel TC8B is presented graphically in Appendix C.

9 STREAM EROSION INDEX

9.1 Introduction

Both Blacktown City Council and the Department of Environment and Climate Change have recently released draft guidelines (Ref. 5 and 16) to address the risk of stream erosion from the urbanisation of catchments. Compliance with the guidelines is a method, acceptable to both DoP and BCC, for controlling the risks associated with discharges from developed catchments into informal watercourses and is generally referred to as a Stream Erosion Index (SEI) risk assessment.

The assessment procedure relies on calculating the increase in the relative frequency of flows from the site greater than the "stream forming flow" based on a continuous series of rainfall data over many years. Achieving a value at or below the recommended SEI value is considered to be an appropriate means of assessing and addressing the impacts that on a watercourse of the more frequent flows from the developed catchment.

The stream erosion index is defined by the Department of Environment and Climate Change *as the post-development duration of flows greater than the 'stream forming flow' divided by the natural duration of flows greater than the 'stream forming flow'.* The 'stream forming flow' is defined as 50% of the 2-year ARI flow rate estimated for the catchment under natural conditions. The DECC guidelines recommend a stream erosion index of between 3.5 and 5, while Council's Draft DCP (June 2009) proposes an SEI of between 3 and 5 for a 'stream forming flow' defined as 25% of the 2-year ARI.

9.2 SEI Simulation

GHD carried out an SEI assessment based on simulations for a natural and a developed (with mitigation) 35.34 ha sub-catchment. *"The simulations were undertaken with 6-minute data for the years 1988 to 1994, which includes an average year, and high and low rainfall years."*

Structural measures, in the form of a discharge control pit, were introduced into the calculations to replicate the mitigation afforded by the basin outlet structure. The simulations resulted in an SEI of 4.

9.3 Discussion of Modelling

The GHD simulation demonstrates that the use of raingardens, co-located within the Detention Basins, with a discharge control pit on the outlet from the basin, results in an SEI of 4, which complies with the *Draft Blacktown City Council Growth Centre Precincts Development Control Plan 2009* (Ref.7) and (Ref.16). .

Energy dissipation, discharge control pits and stream erosion protection is proposed on outlets from the Detention Basins and the formal drainage network. These structures, in combination with the dense vegetation proposed for the formal channels and riparian areas, have been determined by GHD as being capable of maintaining the Stream Erosion Index below the required maximum value of 5 (Ref.7 and Ref.16). It has been our experience with similar developments within Western Sydney that the inclusion of a co-located raingarden within the Detention Basin provides sufficient base flow attenuation to comply with the DEC recommended criteria for the Growth centres Commission (Ref.16).

10 COSTING OF DETENTION BASIN MAJOR WORKS

A S94 Contributions Plan is being prepared for the Trunk Drainage component of MPIP as part of the ILP process. Since the exhibition of the ILP, and as reported herein, there have been significant changes to the size, location and number of Detention Basins. Consequently it has been necessary to revise the costs associated with the construction of the Detention Basins Major Works in order to inform a S94 Contribution Plan.

Details of the methodology, and parameters used, to achieve compliance with the relevant peak flow attenuation and stormwater quality criteria, are discussed in the preceding headings of this report. The stormwater management components determined from this process were used in the development of preliminary engineering concept designs for the relevant trunk drainage elements (Appendix D).

From these preliminary concepts it has been possible to estimate the bulk earthworks and vegetation requirements associated with the construction of each Detention Basin. The rates used to determine the Amounts in Table 10.1 have been based on the construction of each Item as an independent element of the strategy, in isolation of other Trunk Drainage and site works (see Appendix B).

The above information resulted in the production of a Bill of Quantities, the individual items within which were multiplied by rates based on our experience from similar projects to provide a Construction Estimate for the Trunk Drainage Major Works.

A summary of the Costs associated with the construction of the Basins and their associated vegetation are presented in a simplified form in Table 10.1 which addresses only those items of significant cost. A more detailed breakdown of the Bill of Quantities and the associated Construction Estimate for each Basin, based on their construction being independent of all other works, is provided in Appendix B.

Table 10.1

PRELIMINARY CONSTRUCTION COST ESTIMATE
MPIP DETENTION BASINS AND ASSOCIATED VEGETATION

NO.	ITEM	AMOUNT Exc GST$
1	BASIN A	$1,265,000.00
2	BASIN B	$1,534,000.00
3	BASIN P	$2,075,000.00
4	BASIN E	$2,287,000.00
5	BASIN K	$930,000.00
6	BASIN G	$1,733,000.00
7	BASIN I	$1,836,000.00
8	BASIN J	$859,000.00
9	BASIN M	$1,788,000.00
	BASINS TOTAL	$14,307,000.00

11 SUMMARY & CONCLUSION

The Water Cycle Management Strategy proposed for the Marsden Park Industrial Precinct has been prepared to support the finalisation of the corresponding Indicative Layout Plan and S94 Contributions Plan. The strategy has been prepared to conform with the requirements of the NSW Department of Planning and Blacktown City Council as well as deliver industry best practice for stormwater management within the MPIP.

The Water Cycle Management Strategy described herein, builds upon the Water Cycle Management Assessment prepared by GHD which was developed to inform the preparation of the Draft ILP, and was placed on exhibition in early 2010. It proposes a series of individual elements arranged in what may be described as a "treatment train" consisting of on-lot treatment, street level treatment and precinct based public domain infrastructure treatment measures. The structural elements proposed for the development consists of:

- Rainwater tanks on each allotment, installed during building construction;
- Proprietary GPT units upstream of each raingarden and on each stormwater discharge point to trunk drainage channels;
- On-lot water quality bio-retention/filtration raingardens for commercial/industrial and medium/high density residential catchments;
- Precinct based bio-retention/filtration raingarden systems co-located within the precinct scale Detention Basins;
- Conservation and enhancement of the existing and future riparian areas.

Provision of the proposed detention basins within the development will ensure that peak post development discharges are restricted to pre development levels.

Provision of the proposed water quality treatment strategy within the development will ensure that the quality of the post development stormwater discharges meet the requirements of the NSW Department of Planning and Blacktown City Council.

Climate Change Impacts have been assessed on the basis of increasing the flows in the trunk drainage network by 15% and reviewing the associated risks to adjoining infrastructure and fill levels. The increase in peak flows resulting from a 15% increase in rainfall intensity resulted in a reduction of approximately 100 mm in the freeboard allowed for the 100-yr flood levels. According to *Draft Flood Risk Management Guide: Incorporating sea level rise benchmarks in flood risk assessments, October 2009* Department of Environment, Climate Change and Water, the 0.5 m freeboard allowance in floodplains is capable of absorbing small increases in flood levels resulting from Climate Change impact assessments.

Stream Erosion Index assessment criteria has been based on the Environmental Flows ratio advice provided by the Department of Environment and Conservation (DEC) to the Growth Centres Commission (Ref.16), which referred to a *"stream erosion control ratio of between 3.5 and 5.0"*. GHD, 2009 (Ref.1) carried out an SEI assessment for the MPIP using 6-minute rainfall data over a 6-year period for a 35.34 ha sub-catchment. Their assessment returned a ratio of 4, which demonstrated compliance with the advice from DEC (Ref.16).

The proposed Water Cycle Management Strategy for the MPIP provides a basis for the detailed design and development of the Precinct to ensure that the objectives for stormwater management and water quality are achieved.

12 REFERENCES

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2. NSW Department of Environment and Climate Change (2008) NSW Climate Change Action Plan: Summary of Climate Change Impacts Sydney Region, (October 2008)

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22. Department of Infrastructure Planning and Natural Resources (2002) Salinity Potential in Western Sydney (Map).

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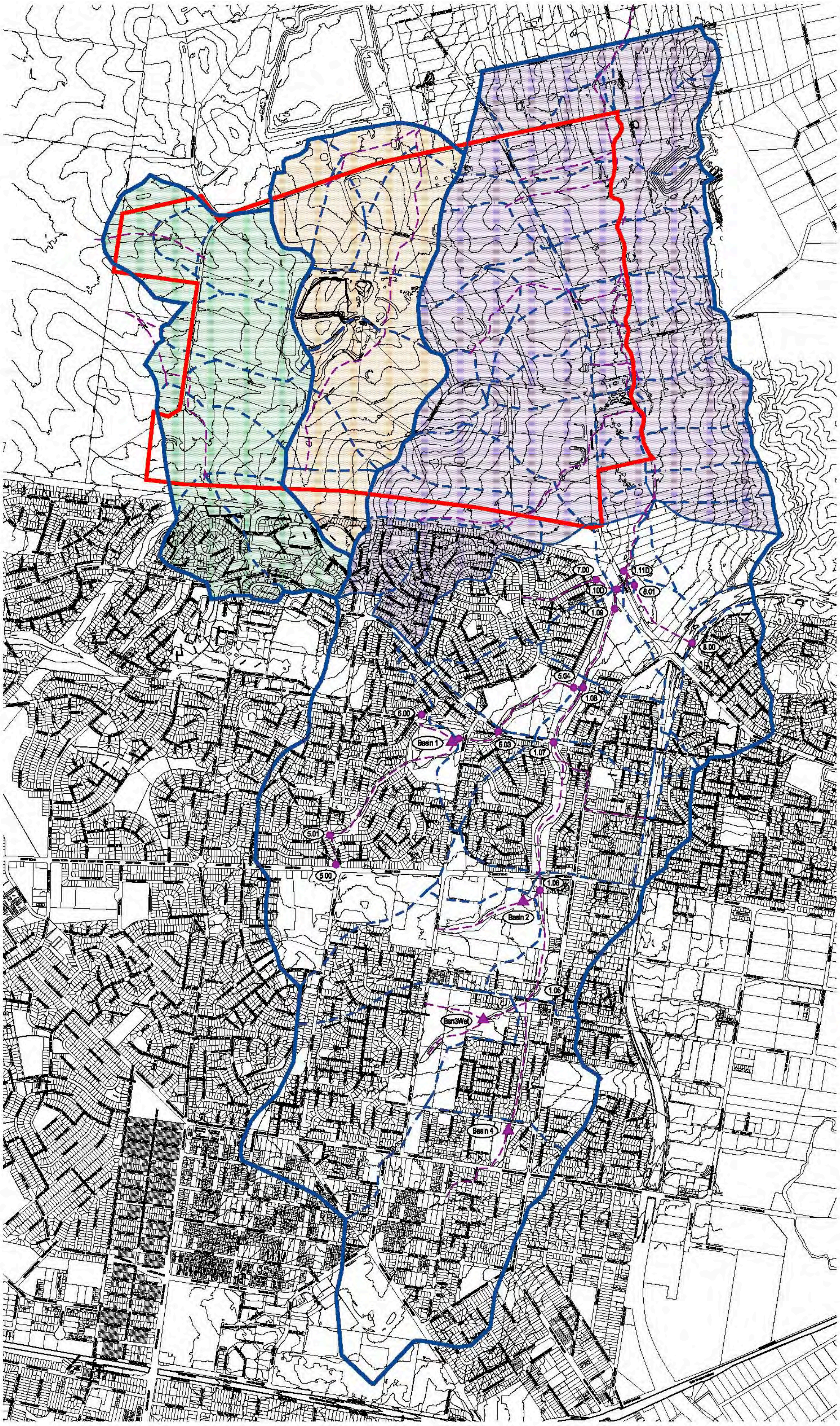
24. Bannerman, S.M. and Hazelton, P.A. (1990), Soil Landscapes of the Penrith 1:100 000 Sheet. Soil Conservation Service of NSW, Sydney.

FIGURES



REFER TO FIGURE 2 FOR THE SITE CATCHMENT LAYOUTS

LEGEND:

- SITE BOUNDARY
- SUB-CATCHMENT BOUNDARY
- N-1.01 XP-RAFTS NODE LABEL AND LOCATION
- DEVELOPMENT SITE CATCHMENT BOUNDARY BELLS CREEK CATCHMENT
- DEVELOPMENT SITE CATCHMENT BOUNDARY NORTH-SOUTH (CENTRAL) CREEK CATCHMENT
- DEVELOPMENT SITE CATCHMENT BOUNDARY WESTERN (LITTLE) CREEK CATCHMENT

1:25,000 — 250 0 250 500 750 1,000 1,250 METRES

FIGURE 1

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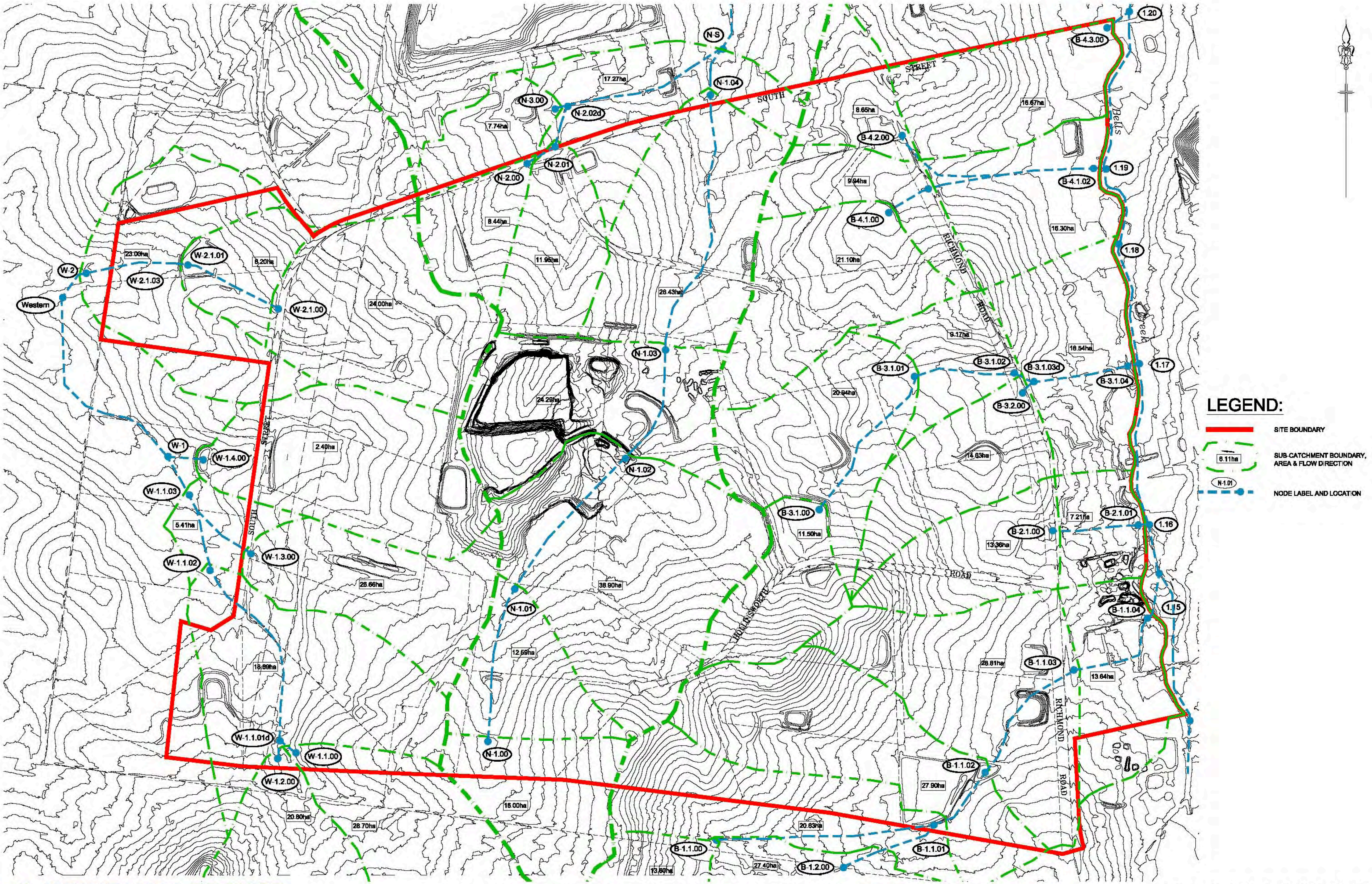
8955_SW01 A
SHEET 1 OF 3

MARSDEN PARK INDUSTRIAL PRECINCT

INDICATIVE BELLS CREEK CATCHMENT LAYOUT & PRE-DEVELOPMENT MPIP CATCHMENT EXTENTS



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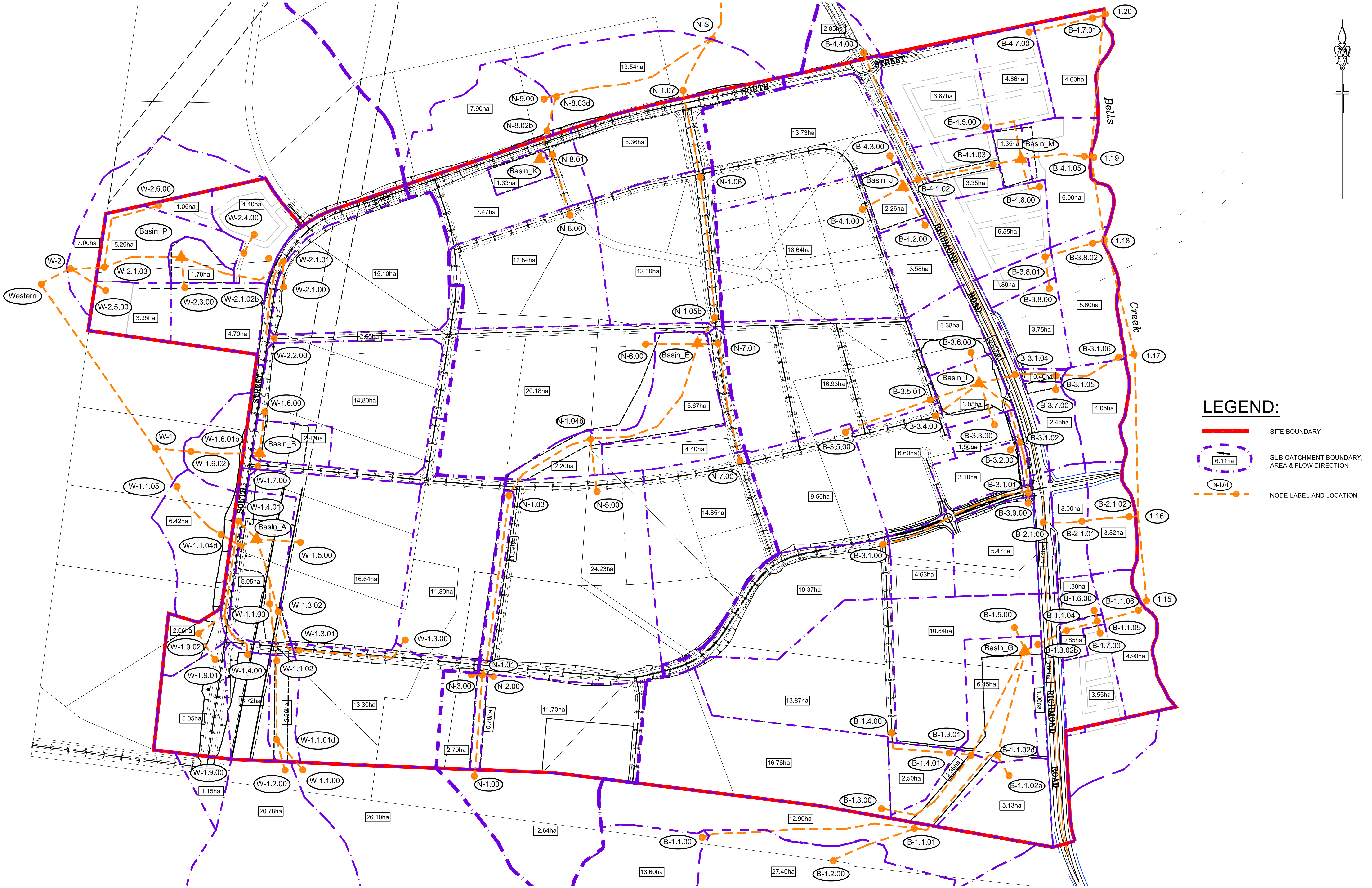
SHEET 2 OF 3

MARSDEN PARK INDUSTRIAL PRECINCT

1:10000 — 100 50 0 50 100 150 200 250 300 350 400 450 500 METRES

CATCHMENT LAYOUT PLAN
PRE-DEVELOPMENT CONDITIONS

FIGURE 2



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SHEET 3 OF 3

MARSDEN PARK INDUSTRIAL PRECINCT

CATCHMENT LAYOUT PLAN
POST-DEVELOPMENT CONDITIONS

FIGURE 3

APPENDIX A

Marsden Park Industrial Precinct

Climate Change Assessment

MARSDEN PARK INDUSTRIAL PRECINCT
CLIMATE CHANGE ASSESSMENT

The following information is offered as an explanation of our investigations into the anticipated impacts of Climate Change on the performance of the Drainage System proposed for Stage 1 of the Marsden Park Employment Precinct.

The objective of this assessment is to provide information on the possible impacts of Climate Change, and make recommendations which can be readily transferred to the design of the drainage systems in other Stages of the Marsden Park Employment Precinct.

BACKGROUND TO CLIMATE CHANGE ASSESSMENTS

When undertaking a risk assessment into the impact of flooding on urban infrastructure, as a consequence of Climate Change predictions, it is necessary to quantify the possible changes in rainfall intensity and assess the impact that these changes may have on the catchment hydrology. In the absence of specific quantifiable guidelines from Blacktown City Council (BCC) the primary reference sources agreed to, for this assessment, are:

1. *NSW Climate Change Action Plan: Summary of Climate Change Impacts Sydney Region, October 2008*, prepared by the NSW Department of Environment and Climate Change;
2. *Practical Consideration of Climate Change – Floodplain Risk Management Guideline, October 2007*, prepared by the NSW Department of Environment and Climate Change;
3. *Climate Change in the Hawkesbury-Nepean Catchment, 2007*, prepared by the Commonwealth Scientific and Industrial Research Organisation, were adopted as the primary reference documents for this assessment; and
4. *Climate Change in Australia – Observed Changes and Projections, October 2007*, prepared by Australian Government Bureau of Meteorology.

Prior to assessing the estimated impacts of Climate Change on the Marsden Park Employment Precinct, it is necessary to compare the various recommended increases to Rainfall Intensities identified in these documents, determine the most appropriate Rainfall Intensity increase and apply it to the hydrologic assessment for Stage 1.

This process is consistent with the “Management Strategies For Future Development” outlined in Reference 2. Table 1 summarises the State and Federal Government approaches to accounting for changes to predicted rainfall intensities and storm volumes associated with Climate Change. All documents predict increases in peak rainfall intensity with an associated increase in storm runoff volume. However the overall Average Annual Rainfall for the region is anticipated to reduce, whilst summer rainfall is predicted to increase. Drawing a direct comparison between each of the predictions, and relating a conclusion to a predicted increase in rainfall intensity is not as straightforward as it may seem and it has been necessary to relate the stated volumetric predictions to a more tangible Average Recurrence Interval (see Reference 2).

TABLE 1: Comparison of the Various Climate Change Strategies

Reference	Rainfall Intensity	Comment
1. Climate Change Impacts – Sydney Region, 2008 (DECC)	Summer runoff depths estimated to increase by 0% to 26% Summer rainfall volume projected to increase by 20% to 50%	Hydrologic change assessment based on seasonal variation estimates. The summer runoff depth increase is the largest.
2. Practical Consideration of Climate Change – Flood Risk Management, 2007 (DECC)	Sensitivity Analysis based on increases of: 10% peak rainfall & vol.; 20% peak rainfall & vol.; 30% peak rainfall & vol.	This approach relies on a risk analysis based on the potential impacts of the various increases. The lowest value with an acceptable Annual Average Damage (AAD) is

	Table of increases in Extreme Rainfall Intensities (40-yr, 24-hr) based on %age change in Intensity and Storm Volume.	then adopted. Consideration of the AAD where this value is exceeded must be included and a strategy to accommodate the additional risk identified.
3. Climate Change in the H-N Catchment, 2007 (CSIRO)	Projected max. Change in the 40-yr, 24-hr rainfall by: 2030 – 12%; 2070 – 10%.	Total annual rainfall is predicted to decline by about 80 mm with the possibility of seasonal increases in extreme rainfall events.
4. Climate Change in Australia, 2007 (BofM)	General increase in daily rainfall intensities in summer only.	Expected volumetric change is to be minimal but extreme daily rainfall is expected to increase.

A summary of the information contained in the above reference documents is outlined below.

- All references agree on a general increase in summer rainfall volume;
- Reference 1 determines the summer daily volumetric runoff depth to increase by 26%;
- Reference 2 refers to a sensitivity analysis of Climate Change based on the risks associated with an Annual Average Damage (AAD) analysis to determine the appropriate Flood Planning Levels, which can then be related to an Average Recurrence Interval (ARI). This approach accommodates a 10%, 20% and 30% increase in the rainfall intensities to determine revised flow rates and runoff depths.
- Reference 3 is the only reference to provide a quantifiable relationship between Climate Change and rainfall intensity for a particular Average Recurrence Interval (ARI). It estimates that the maximum projected change in rainfall intensity for the larger scale storms (40-yr, 24-hr) is about 12%;

These four (4) references were prepared as background documents to assist with Floodplain Risk Management planning. They provide limited guidance with respect to assessing the possible impacts of Climate Change on new urban developments and the costs associated with the subsequent increase in the land required for local flood control.

NOTE: Based on the 12% increase predicted in Reference 3, the rainfall depth hyetographs in the existing XP-RAFTS hydrologic computer models, constructed to represent the Basin I catchment, were 'conservatively' increased by 15%. The resulting increase in runoff depth, for the 100-yr ARI critical storm, was determined as approximately 25%, which approximates the summer seasonal runoff depth increases of 26% predicted in Reference 1.

SIZING OF DETENTION BASINS IN CONSIDERATION OF CLIMATE CHANGE IMPACTS

There are no specific parameters available which provide guidance for determining rainfall intensity increases when considering Climate Change Impacts (CCI) on the infrastructure within Urban Release Areas. To better understand the consequence of these impacts on the drainage structures and the land required for the Trunk Drainage system an assessment of two (2) configurations for the outlet structure of Basin I have been considered.

1. Flexible Outlet Arrangement

This option involves the Basin I outlet structure designed as a flexible multi-stage outlet which achieves a match between the post and pre development discharge rates. In calculating these peak discharge rates, the stage/discharge relationships were adjusted in proportion to the various increases in rainfall intensities, which were also applied as percentage increases on the post development and pre development rainfall intensities, in consideration of Climate Change. The peak discharge rates, storage volumes and land requirements for this option, which aims to match the post and pre development peak flows **at the confluence with Bells Creek**, are tabulated in Table 2. The management of the peak discharge rates for this option involves:

- Increasing the pre and post development rainfall intensities by the same percentage increase in consideration of Climate Change;
- Modifying the Basin I stage/discharge relationship to match post development peak discharges to pre-development peak discharges;
- Interrogating the downstream channel profile to determine its waterway area and capacity to convey the additional flows attributable to an increase in the pre-development rainfall intensity.

A flexible outlet structure will need to be designed, as part of the Development Application process, which takes into account a decadal variation in rainfall intensities and discharge rates as predicted in the earlier reference material. The additional peak discharge rates, attributed to predicted increases in the rainfall intensities associated with Climate Change, are to be accommodated within the freeboard allowance of the channel cross section downstream of Basin I (i.e. TC13).

TABLE 2: Basin I – Climate Change Sensitivity Analysis Matching Post Development to Pre Development Peak Discharges

100 Year ARI	**Pre-Development Catchment**	**Post-Development Catchment**			
Rainfall Intensity Increase	**Peak Discharge (m^3/s)**	**Peak Discharge (m^3/s)**	**Storage Volume (m^3)**	**Water Surface A. (m^2)**	**Basin Surface A. (m^2)**
0% (Current)	14.45	12.69	28,000	23,300	36,600
10%	16.77	16.65	28,000	23,300	36,600
15%	17.95	17.44	28,000	23,300	36,600
20%	19.18	18.19	28,000	23,300	36,600
30%	21.71	21.13	28,610	23,800	36,600

NOTE: Basin Surface A has been scaled from the ILP after overlaying with Basin I layout Water Surface A. & batters

2. Conventional Outlet Arrangement

This option involves a conventional multi-staged outlet for Basin I, designed to match between the post development discharge (including and allowance for CCI) with the **existing** pre development discharge rates (i.e. the basin is designed to mitigate future CCI). In calculating these peak discharge rates the stage/discharge relationships were adjusted in proportion to the various increases in post development rainfall intensities only, in consideration of Climate Change. The peak discharge rates, storage volumes and land requirements for this option, which aims to maintain the existing peak flows **at the confluence with Bells Creek**, are tabulated in Table 3. The management of the peak discharge rates for this option involves:

- Determination of the existing peak discharge rate;
- Increasing the post development rainfall intensities by the respective percentages in consideration of Climate Change;
- Modifying the Basin I stage/discharge relationship to match post development peak discharges to the existing pre-development peak discharges;
- Interrogating the downstream channel profile to determine its waterway area and capacity to convey the existing flows regardless of any CCI.

A conventional multi-staged outlet structure for Basin I will need to be designed, as part of the Development Application process, which takes into account a decadal variation in rainfall intensities for the post developed catchment as predicted in the earlier reference material. The additional peak discharge rates, attributed to predicted increases in the rainfall intensities associated with Climate Change, are to be accommodated in the increased storage volume within Basin I. The dimensions of the channel cross section downstream of Basin I (i.e. TC13) will remain constant for all increases in rainfall intensities that may be attributable to CCI.

TABLE 3: Basin I – Climate Change Sensitivity Analysis Matching Post Development to Existing Pre Development Peak Discharges

100 Year ARI	Pre-Development Site Conditions	Post-Development Site Conditions			
Rainfall Intensity Increase	**Peak Discharge (m^3/s)**	**Peak Discharge (m^3/s)**	**Storage Volume Adopted (m^3)**	**Water Surface A. (m^2)**	**Basin Surface A. (m^2)**
0% (Current)	14.45	10.93	28,000	23,300	36,600
10%	14.45	12.75	30,000	25,000	38,300
15%	14.45	12.88	32,000	26,700	40,000
20%	14.45	13.10	34,000	28,300	41,600
30%	14.45	12.52	42,000	35,000	48,300

NOTE: Basin Surface A has been increased proportionally, over the areas shown in Table 2, based on the corresponding increase in Water Surface A

Discussion of the Land Required to Manage Peak Discharges

The above comparison identifies the basin size required to match the post to pre development peak flows for various scenarios with and without accounting for Climate Change. In considering the impact of Climate Change on the size of Basin I and the Trunk Channel TC13 (see Table 4). An increase in rainfall intensities of 15% has been adopted for comparison purposes as it compares favourably with the Risk Assessment approach referred to in references 1, 2 and 3 and approximates the values referred to in References 1 and 3.

Basin I

Based on a 15% increase in rainfall intensity the analysis identifies that; to achieve a match between the existing post and pre development discharges, a Storage Volume of approximately 28,000 m^3 is required which equates to a Basin Surface Area of approximately 36,600 m^2. Whereas matching the post development discharge (rainfall intensity increased by 15%) to the existing undeveloped discharge requires a Storage Volume of approximately 32,000 m^3 and a Basin Surface Area of approximately 40,000 m^2. *NOTE: The discrepancy between the peak discharges in Table 2 (12.69 m³/s) and Table 3 (10.93m³/s) has resulted from the discharges out of the multi-staged outlet in Option 1, being sized to match the pre-development 5-year ARI peak discharges increased to include CCI.*

Trunk Channel (TC13)

The downstream channel reach between Richmond Road and Bells Creek is known as TC13. Its dimensions, determined in the GHD Report dated July 2009, include a minimum slope of 0.5%, batter slopes of 1:6 and a base width of 2 m, with a 5 m access track along the western side, for a total width of 25 metres.

Table 9 (p.25) of the GHD Report identifies the peak flow downstream of Basin I as 7.9 m^3/s (Initial Loss of 25 mm), this compares favourably with our own assessment of existing peak flow rate at Richmond Road, of 8.68 m^3/s. However, the information in Appendix G 'Water Sensitive Urban Design Strategy' identifies the Q100 design flow for TC13 as 4.6 m^3/s, which appears to be the flow rate used by GHD in their design of TC13. Our review and assessment of the amended developed catchment, at the confluence of TC13 and Bells Creek, has identified the existing 100-year peak flow as 14.45 m^3/s (Initial Loss of 15 mm) and it is this flow rate that has been adopted to determine Drainage Reserve width between Richmond Road and Bells Creek.

Table 4 provides a comparison of Drainage Reserve widths for the TC13 channel reach based on a: maximum depth of flow of 1 m; 0.5 m freeboard; 1:4 batter slopes. A 1:4 batter slope has been adopted in accordance with the requirements of a Category 3 Vegetation Management Plan as agreed with BCC and consistent with the BCC agreed approach for the North West Transport Hub site at Riverstone.

TABLE 4: Comparison of TC13 Drainage Reserve Widths for Various Peak Flows

100 yesr ARI	Q100-yr m^3/s	Base m	Depth m	Width m	Area m^2
Existing	14.45	11	1.5	23	2,300
2010	12.69	11	1.5	23	2,300
+10%	16.65	15	1.5	27	2,700
+15%	17.44	16	1.5	28	2,800
+20%	18.19	17	1.5	29	2,900
+30%	21.13	20	1.5	32	3,200

Existing refers to the TC13 profile required to convey the existing peak flow from the upstream undeveloped catchment.
Drainage Reserve Width includes an allowance for 0.5 Freeboard but no separate 5 m wide Access Track.
Batter Slopes 1:4 based on a Category 3 Vegetation Management Plan.
Depth is based on a "normal depth" calculation using a Manning's roughness coefficient of 0.07 plus 0.5 m Freeboard.
Area of the Drainage Reserve based on a length scaled from the ILP of 100 m
The depth of flow in TC13 with a 16 m base width, based on Option 1 with a 30% increase in rainfall intensity is 1.1 m.

Summary

- Option 1 requires a "Drainage Reserve" (combine Basin and Channel Reserve) allocation of approximately 39,400 m^2;
- Option 2 requires a "Drainage Reserve" allocation of approximately 42,300 m^2. (2,900 m^2 or 7.4% larger).

DISCUSSION

The Sensitivity Analyses outlined above provides information to assist in determining appropriate parameters to be used when considering the impact of an anticipated increase in rainfall intensities as a result of Climate Change predictions. A discussion of the results follows:

- The peak discharge generated by a 15% increase in rainfall intensity approximates a peak discharge rate midway between the existing peak discharge and that generated by a 30% increase in rainfall intensity. Reference 2 predicts a 12% rainfall intensity increase by 2030 with a reduction to a 10% increase by 2100, over present day rainfall intensities. Further, a 15% increase in rainfall intensity results in a 25% increase in the peak runoff depth for the Basin I catchment. This increase in peak runoff depth approximates the 26% increase anticipated in the seasonal summer runoff depth referred to in reference 1.
- Maintaining the existing peak flow rate in the downstream channel reach (TC13), whilst accounting for a 30% increase in post development rainfall intensities, requires a: 50% increase in the storage capacity of Basin I over a conventional design (see Table 3). Providing this level of protection is considered to exceed the requirements of the Sensitivity Analysis referred to in reference 2, especially when the freeboard allowance of 0.5 m for Basin I provides an potential extended detention storage capacity of 11,500 m^3, which equates to an additional 40% in storage capacity.
- In our opinion, adoption of a 15% increase in rainfall intensities provides a reasonable estimate of CCI.
- To achieve the objectives defined in Option 1, Basin I must have a minimum Storage Volume of 28,000 m^3, which can almost accommodate a 30% increase in rainfall intensities (see Table 2),.
- To achieve the objectives defined in Option 1 Trunk Channel TC13 must have a minimum Drainage Reserve width of 28 m, which can accommodate a 30% increase in rainfall intensities if the freeboard allowance is reduced to 0.3 m (see Table 4).
- The Drainage Reserve width for TC13 would reduce by about 1.5 m if the freeboard allowance were reduced from 0.5 m to 0.3 m. A freeboard allowance of 0.3 m is consistent with the minimum Typical Freeboard Requirements for Industrial/Commercial development identified on Figure 1.1 of Blacktown City Councils "Engineering Guide for Development – 2005".

RECOMMENDATION

Rainfall Intensity –increased by 15% for the 100-year critical storm in consideration of the possible impact of Climate Change. Table 5 compares land requirements for a Drainage Strategy that matches the existing peak flow rates and one which includes a 15% increase in rainfall intensity utilising both Options 1 and 2 to control peak discharges.

Basin Size – determined on the basis of attenuating the 100-year post-development peak flow to the 100-year pre-development peak flow rate, including an increase of 15% in rainfall intensity to account for Climate Change for both (per Option 1). (See Table 5 for a comparison of Water Surface Areas).

Trunk Channel Waterway Area – profile to be based on Option 1 with a 15% increase in rainfall intensity. A similar approach is to be taken for Trunk Channels draining the local catchments i.e. the capacity of the channel must contain the runoff generated by a 15% increase in the 100-year peak flow rate for the developed catchment.

Freeboard – adoption of 0.5 m clearance over and above the flow depth generated by the existing 100-year peak flow from the developed catchment. This freeboard allowance includes a maximum of 0.2 m to accommodate CCI (see discussion in Attachment).

TABLE 5: Comparison of Land Requirements for Basin I and TC 13

Options	**Estimated Drainage Reserve Area Basin I**	**Drainage Reserve Dimensions TC13**	**Total Drainage Reserve Area Required**
Match Existing Post Development to Existing Pre Development Peak Discharge Rates	36,600 m^2	23 m c. 2,300 m^2 (scaled length of 100m)	38,900 m^2
Match Post Development to Pre Development Discharge with 15% increase in Rainfall Intensity for both. **Option 1**	36,600 m^2	28 m c. 2,800 m^2 (scaled length of 100m)	39,400 m^2 This can be reduced to 38,900 m^2 if CCI is allowed to encroach by 0.1 on the 0.5 m Freeboard (see Attachment)
Match Post Development (with 15% increase in Rainfall Intensity) to existing Pre Development Peak Discharge Rates **Option 2**	40,000 m^2 (pro-rata)	23 m c. 2,300 m^2 (scaled length of 100m)	42,300 m^2

Basin I – refers to the water surface area only (i.e. no allowance for batter slopes or freeboard) based on a mean depth of 1.5 m.
Drainage Reserve Width TC13 – includes 1:4 batter slopes, 0.5 m freeboard and a 5 m access track.

<u>These analyses relate to the Basin I catchment only, and may be subject to change once the whole of the Marsden Park Precinct is analysed and its relationship to flooding in Bells Creek defined.</u>

APPENDIX B

Marsden Park Industrial Precinct

Trunk Drainage Components

Preliminary Construction Cost Estimate

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN A**

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN A**				
	Establishment	1	Item	$50,400.00	$50,400.00
	Clearing (Allowance Only)	1	Item	$50,000.00	$50,000.00
	Strip & stockpile topsoil	38,700	sq.m.	$1.50	$58,050.00
	Respread topsoil	29,200	sq.m.	$2.50	$73,000.00
	Earthworks - Cut to Fill On Site	8,710	cu.m.	$5.00	$43,550.00
	Earthworks - Import material	11,400	cu.m.	$30.00	$342,000.00
	Trim and Compact	38,700	sq.m.	$1.00	$38,700.00
	Maintain Integrity of Existing Conservation Area	7,500	sq.m.	$0.50	$3,750.00
	Core Riparian Planting Bush Regeneration (incl 2 yr maint)	6,300		$10.00	$63,000.00
	Planting (incl maintenance for 2 years)	22,900	sq.m.	$9.50	$217,550.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	400	sq.m.	$750.00	$300,000.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$1,265,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN B**

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN B**				
	Establishment	1	Item	$78,450.00	$78,450.00
	Clearing (Allowance Only)	1	Item	$10,000.00	$10,000.00
	Strip & stockpile topsoil	12,500	sq.m.	$1.50	$18,750.00
	Respread topsoil	11,600	sq.m.	$2.50	$29,000.00
	Earthworks - Excavate and Dispose of Unsound Material	6,200	cu.m.	$200.00	$1,240,000.00
	Earthworks - Import material	230	cu.m.	$30.00	$6,900.00
	Trim and Compact	12,500	sq.m.	$1.00	$12,500.00
	Planting (incl maintenance for 2 years)	11,600	sq.m.	$9.50	$110,200.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	100	sq.m.	$25.00	$2,500.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$1,534,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14	BASIN E

Western embankment based on Court Approved Landform

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN E**				
	Establishment	1	Item	$159,950.00	$159,950.00
	Clearing (Allowance Only)	1	Item	$10,000.00	$10,000.00
	Strip & stockpile topsoil	50,000	sq.m.	$1.50	$75,000.00
	Respread topsoil	41,800	sq.m.	$2.50	$104,500.00
	Earthworks - Cut to Fill On Site	11,550	cu.m.	$5.00	$57,750.00
	Earthworks - Import material	46,250	cu.m.	$30.00	$1,387,500.00
	Trim and Compact	50,000	sq.m.	$1.00	$50,000.00
	Planting (incl maintenance for 2 years)	41,800	sq.m.	$9.50	$397,100.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	800	sq.m.	$25.00	$20,000.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$2,287,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN G**

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN G**				
	Establishment	1	Item	$117,200.00	$117,200.00
	Clearing (Allowance Only)	1	Item	$50,000.00	$50,000.00
	Strip & stockpile topsoil	31,000	sq.m.	$1.50	$46,500.00
	Respread topsoil	34,120	sq.m.	$2.50	$85,300.00
	Earthworks - Excavate and Dispose of Unsound Material	4,000	cu.m.	$200.00	$800,000.00
	Earthworks - Cut to Fill On Site	19,480	cu.m.	$5.00	$97,400.00
	Earthworks - Import material	7,520	cu.m.	$30.00	$225,600.00
	Trim and Compact	41,000	sq.m.	$1.00	$41,000.00
	Planting (incl maintenance for 2 years)	23,120	sq.m.	$9.50	$219,640.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	1,000	sq.m.	$25.00	$25,000.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$1,733,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN I**

Layout per DA

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN I**				
	Establishment	1	Item	$92,700.00	$92,700.00
	Clearing (Allowance Only)	1	Item	$200,000.00	$200,000.00
	Strip & stockpile topsoil	32,000	sq.m.	$1.50	$48,000.00
	Respread topsoil	24,500	sq.m.	$2.50	$61,250.00
	Earthworks - Cut to Fill On Site	7,100	cu.m.	$5.00	$35,500.00
	Earthworks - Cut to Dispose off Site (incl. disposal fees)	14,000	cu.m.	$75.00	$1,050,000.00
	Trim and Compact	32,000	sq.m.	$1.00	$32,000.00
	Planting (incl maintenance for 2 years)	24,500	sq.m.	$9.50	$232,750.00
	Spillway Weir - 300x500 concrete edge strip	90	lin.m.	$150.00	$13,500.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	1,800	sq.m.	$25.00	$45,000.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$1,836,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN J**

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN J**				
	Establishment	1	Item	$48,400.00	$48,400.00
	Clearing (Allowance Only)	1	Item	$50,000.00	$50,000.00
	Strip & stockpile topsoil	20,000	sq.m.	$1.50	$30,000.00
	Respread topsoil	16,300	sq.m.	$2.50	$40,750.00
	Earthworks - Excavate and Dispose of Unsound Material	1,972	cu.m.	$200.00	$394,400.00
	Earthworks - Cut to Fill On Site	3,683	cu.m.	$5.00	$18,415.00
	Earthworks - Import material	1,972	cu.m.	$30.00	$59,160.00
	Earthworks - Cut to Dispose off Site (incl. disposal fees)		cu.m.	$75.00	
	Trim and Compact	20,000	sq.m.	$1.00	$20,000.00
	Planting (incl maintenance for 2 years)	16,300	sq.m.	$9.50	$154,850.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	700	sq.m.	$25.00	$17,500.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$859,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN K**

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN K**				
	Establishment	1	Item	$30,400.00	$30,400.00
	Clearing (Allowance Only)	1	Item	$10,000.00	$10,000.00
	Strip & stockpile topsoil	13,000	sq.m.	$1.50	$19,500.00
	Respread topsoil	11,820	sq.m.	$2.50	$29,550.00
	Earthworks - Excavate and Dispose of Unsound Material	3,380	cu.m.	$200.00	$676,000.00
	Earthworks - Cut to Fill On Site	1,820	cu.m.	$5.00	$9,100.00
	Trim and Compact	13,000	sq.m.	$1.00	$13,000.00
	Planting (incl maintenance for 2 years)	11,820	sq.m.	$9.50	$112,290.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	180	sq.m.	$25.00	$4,500.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$930,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN M**

NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN M**				
	Establishment	1	Item	$20,700.00	$20,700.00
	Clearing (Allowance Only)	1	Item	$50,000.00	$50,000.00
	Strip & stockpile topsoil	15,000	sq.m.	$1.50	$22,500.00
	Respread topsoil	11,999	sq.m.	$2.50	$29,997.50
	Earthworks - Excavate and Dispose of Unsound Material	1,800	cu.m.	$200.00	$360,000.00
	Earthworks - Cut to Fill On Site	6,200	cu.m.	$5.00	$31,000.00
	Trim and Compact	15,000	sq.m.	$75.00	$1,125,000.00
	Planting (incl maintenance for 2 years)	11,800	sq.m.	$10.00	$118,000.00
	Spillway Weir - 300x500 concrete edge strip	12	lin.m.	$9.50	$114.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	200	sq.m.	$150.00	$30,000.00
	Soil & Water Management (Allowance Only)	1	item	$25.00	$25.00
				TOTAL	**$1,788,000.00**

PRELIMINARY COST ESTIMATE

PROJECT: Marsden Park Section 94 Basins
Bulk Earthworks & Vegetation Only

CLIENT: Department of Planning C/- APP

JWP Plan Number/Version: 8955SK1-SK14 **BASIN P**

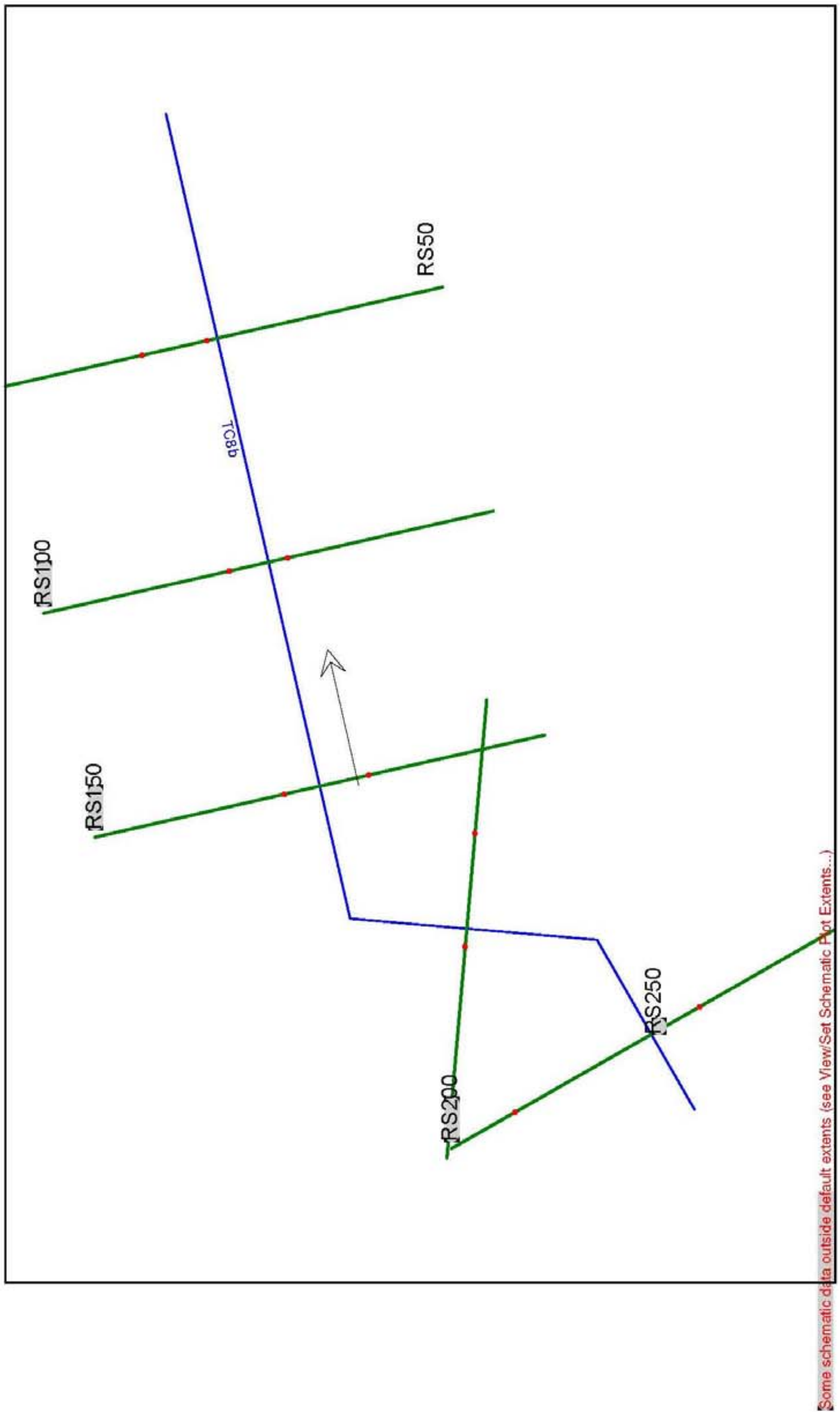
NO.	ITEM	QTY.	UNIT	RATE Exc GST$	AMOUNT Exc GST$
1	**BASIN P**				
	Establishment	1	Item	$66,700.00	$66,700.00
	Clearing (Allowance Only)	1	Item	$20,000.00	$20,000.00
	Strip & stockpile topsoil	9,000	sq.m.	$1.50	$13,500.00
	Respread topsoil	6,060	sq.m.	$2.50	$15,150.00
	Earthworks - Excavate and Dispose of Unsound Material	8,900	cu.m.	$200.00	$1,780,000.00
	Earthworks - Cut to Fill On Site	2,700	cu.m.	$30.00	$81,000.00
	Trim and Compact	9,000	sq.m.	$1.00	$9,000.00
	Planting (incl maintenance for 2 years)	6,160	sq.m.	$9.50	$58,520.00
	Reinforced Turfing on Weir and Spillway (incl 2 yr maint)	240	sq.m.	$25.00	$6,000.00
	Soil & Water Management (Allowance Only)	1	item	$25,000.00	$25,000.00
				TOTAL	**$2,075,000.00**

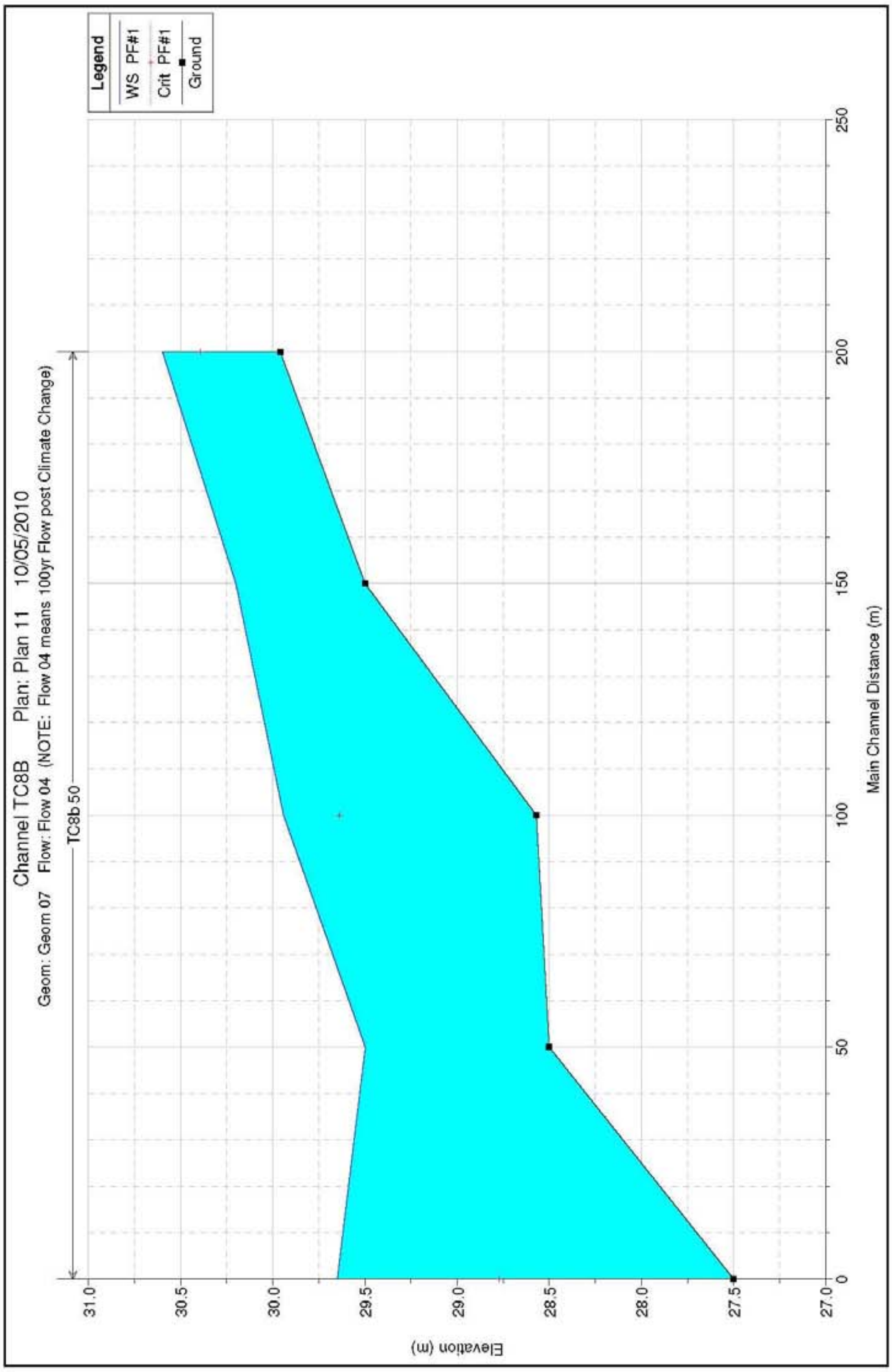
APPENDIX C

Marsden Park Industrial Precinct

Trunk Drainage Channel

HEC-RAS Assessment

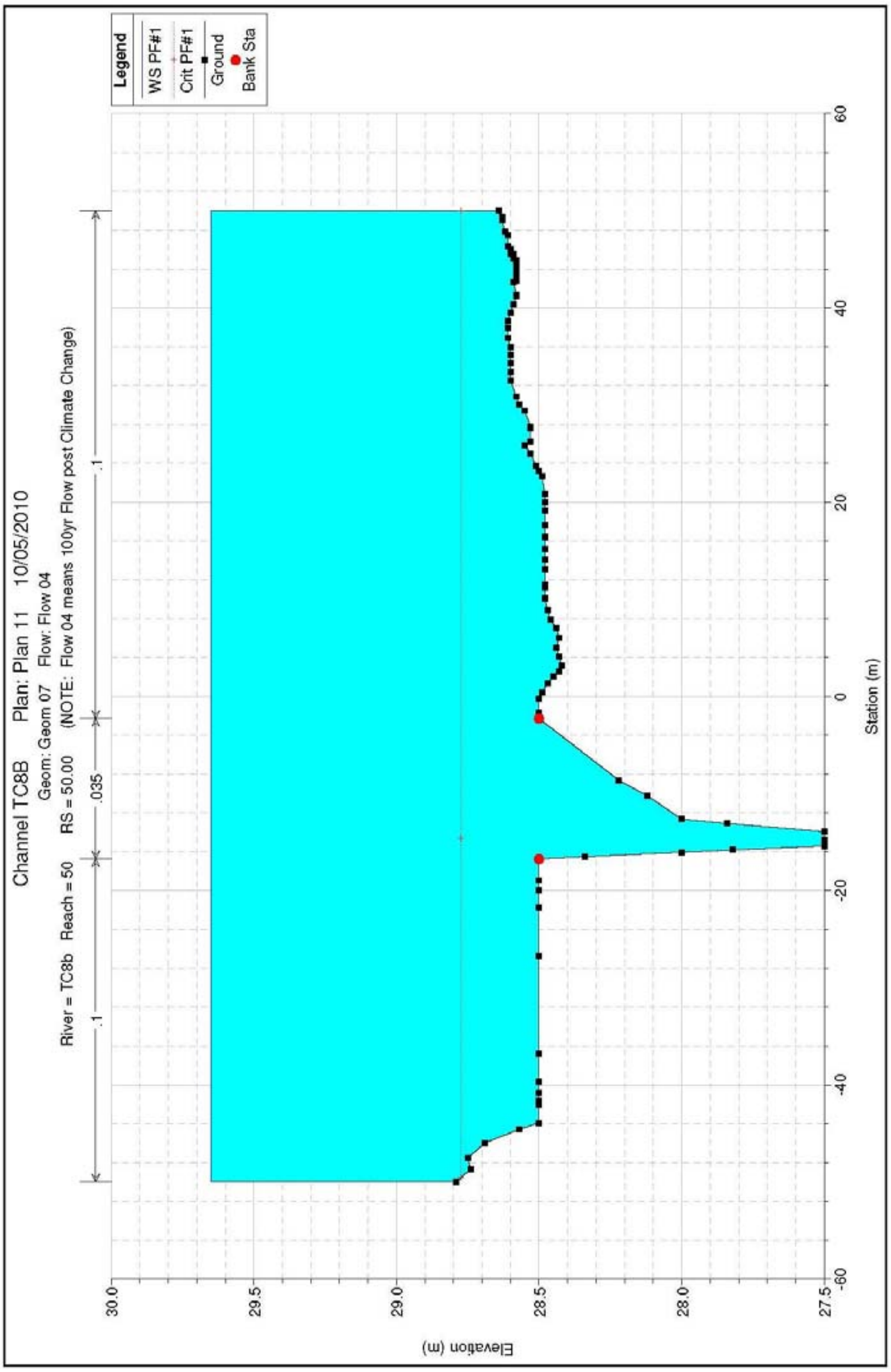

RS50
RS100
RS150
RS200
RS250
TC8b
Some schematic data outside default extents (see View/Set Schematic Plot Extents...)

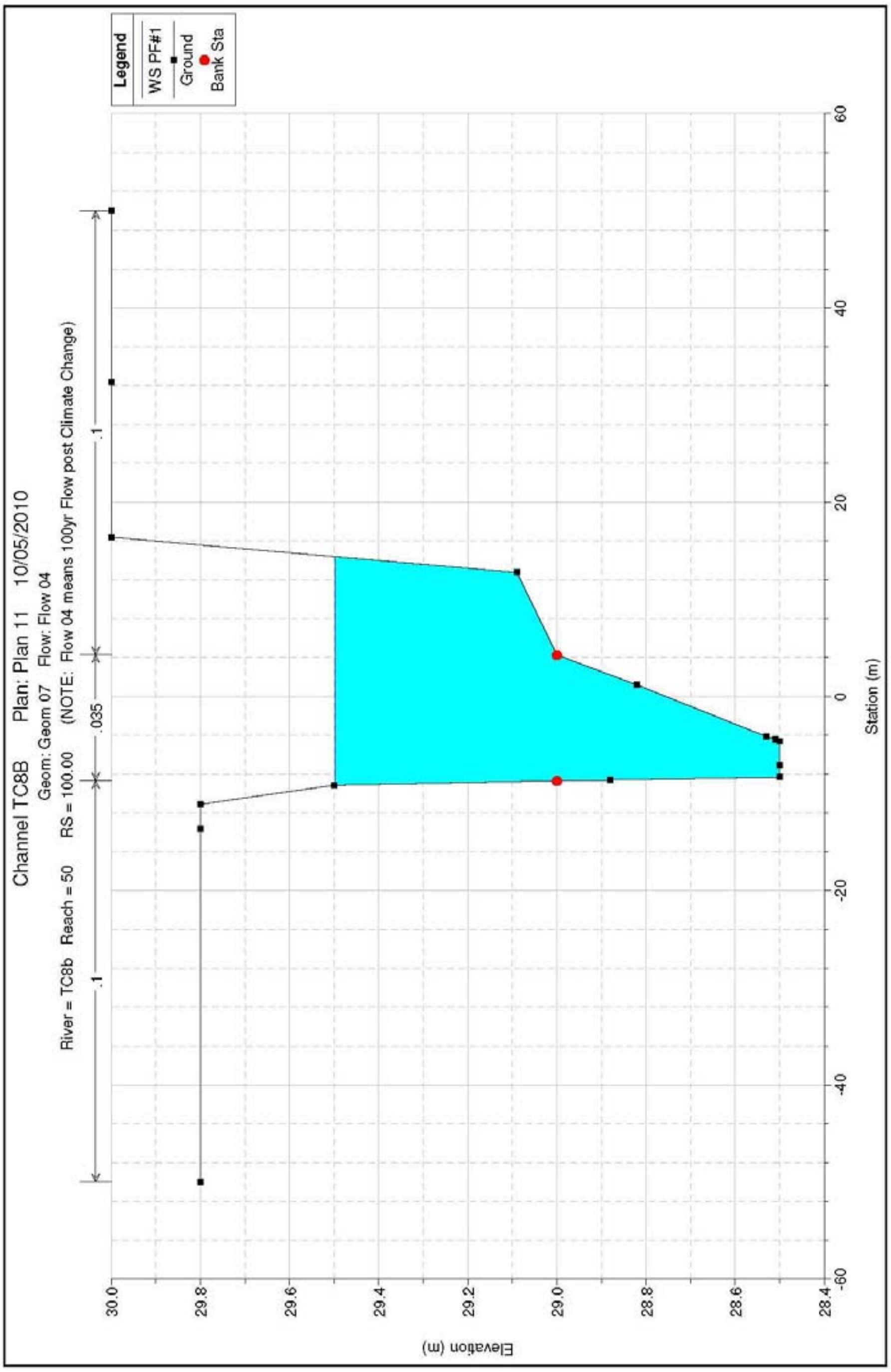

Channel TC8B Plan: Plan 11 10/05/2010
Geom: Geom 07 Flow: Flow 04 (NOTE: Flow 04 means 100yr Flow post Climate Change)
TC8b 50
Legend
WS PF#1
Crit PF#1
Ground
Elevation (m)
Main Channel Distance (m)
31.0
30.5
30.0
29.5
29.0
28.5
28.0
27.5
27.0
0
50
100
150
200
250

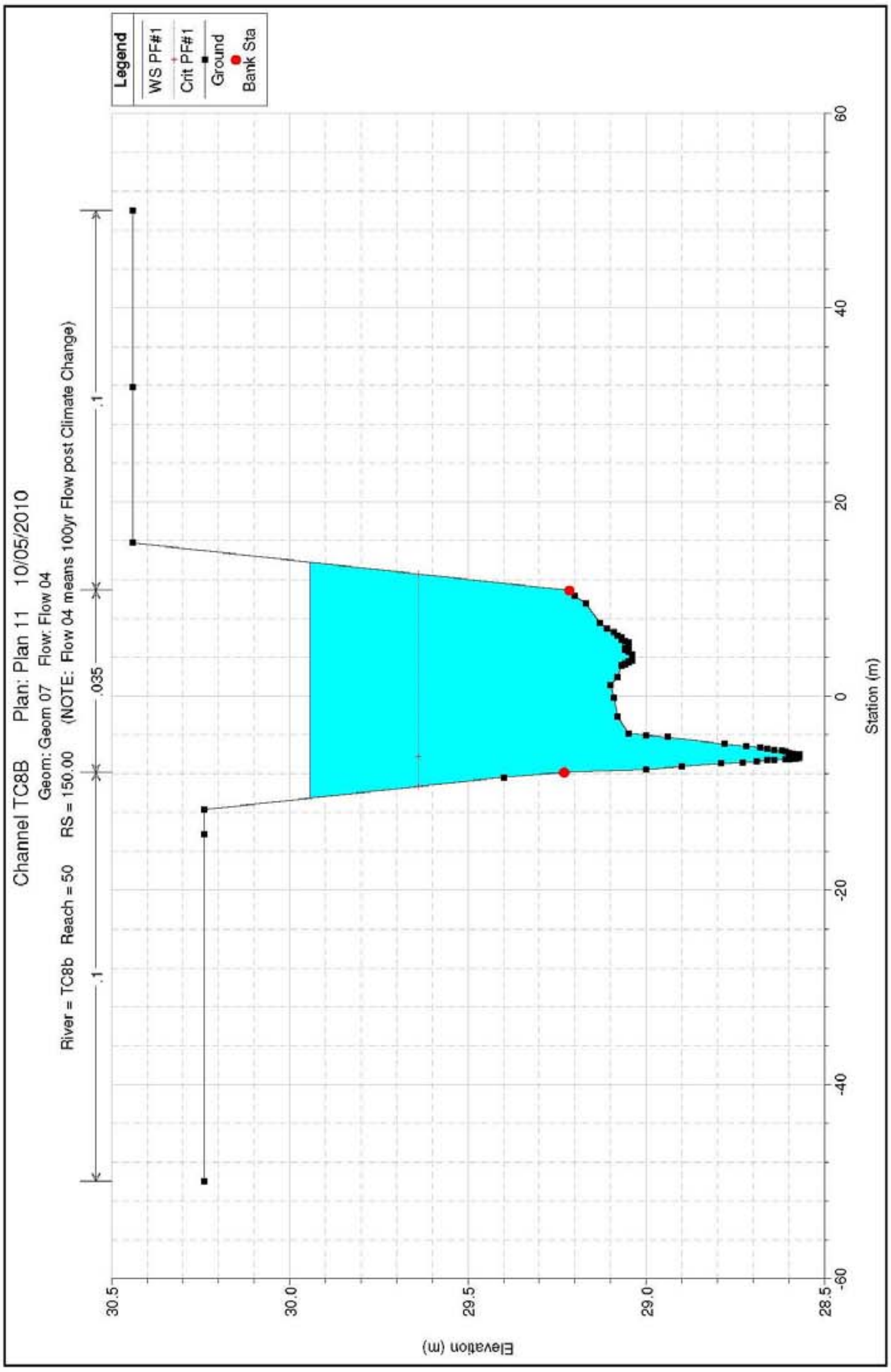
HEC-RAS Plan: Plan11 River: TC8b Reach: 50 Profile: PF#1

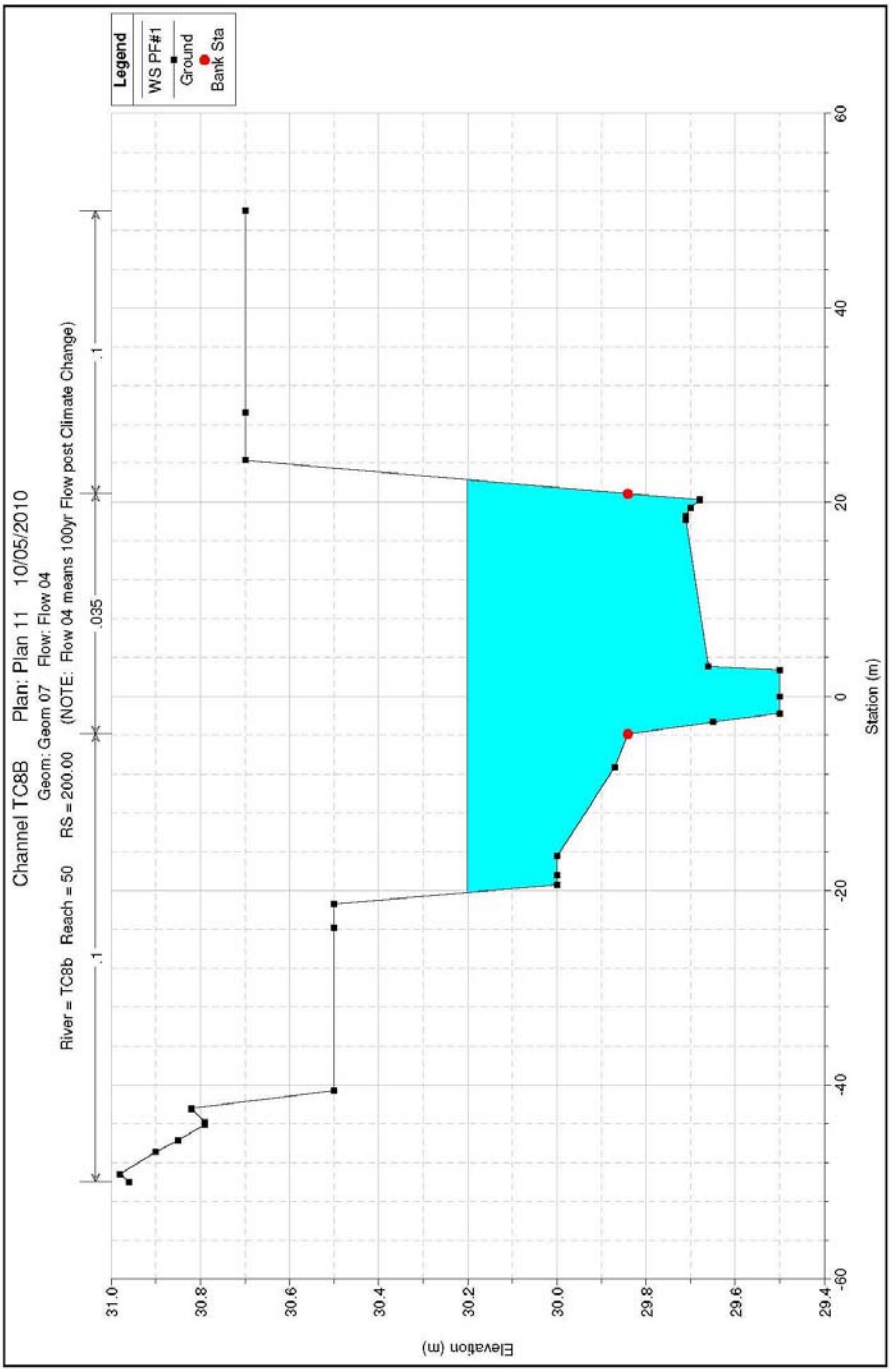
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Vel Left	Vel Chnl	Vel Right	Top Width	Froude # Chl	Shear LOB	Shear Chan	Shear ROB
			(m3/s)	(m)	(m)	(m/s)	(m/s)	(m/s)	(m)		(N/m2)	(N/m2)	(N/m2)
50	250.00	PF#1	28.20	29.96	30.60	0.14	1.06	0.15	54.54	0.47	3.99	16.76	4.51
50	200.00	PF#1	28.20	29.50	30.20	0.42	1.94	0.32	42.47	0.84	26.53	55.28	17.70
50	150.00	PF#1	28.20	28.57	29.94	0.29	1.62	0.30	24.31	0.54	11.76	32.56	12.77
50	100.00	PF#1	28.20	28.50	29.50	0.30	2.43	0.54	23.54	0.86	15.80	76.23	38.95
50	50.00	PF#1	28.20	27.50	29.65	0.16	0.57	0.16	100.00	0.15	2.49	3.42	2.52

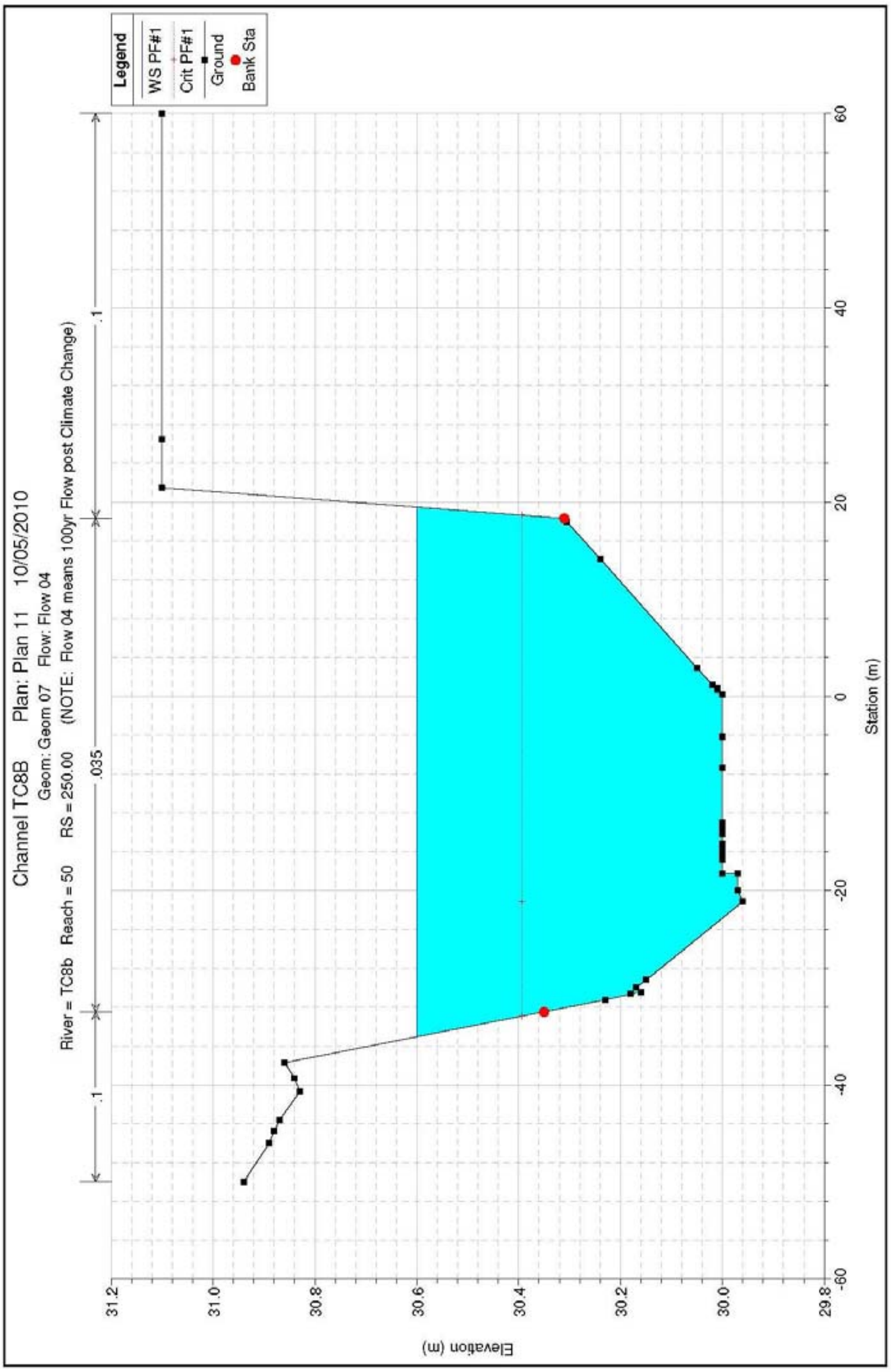
NOTE: River Station 100.00 has an average velocity of 1.9 m/s.


Channel TC8B Plan: Plan 11 10/05/2010
Geom: Geom 07 Flow: Flow 04
River = TC8b Reach = 50 RS = 50.00 (NOTE: Flow 04 means 100yr Flow post Climate Change)
.1
.035
.1
Legend
WS PF#1
Crit PF#1
Ground
Bank Sta
Station (m)
Elevation (m)


Channel TC8B Plan: Plan 11 10/05/2010
Geom: Geom 07 Flow: Flow 04
River = TC8b Reach = 50 RS = 100.00 (NOTE: Flow 04 means 100yr Flow post Climate Change)
Legend
WS PF#1
Ground
Bank Sta
.1
.035
.1
Elevation (m)
Station (m)
30.0
29.8
29.6
29.4
29.2
29.0
28.8
28.6
28.4
-60
-40
-20
0
20
40
60

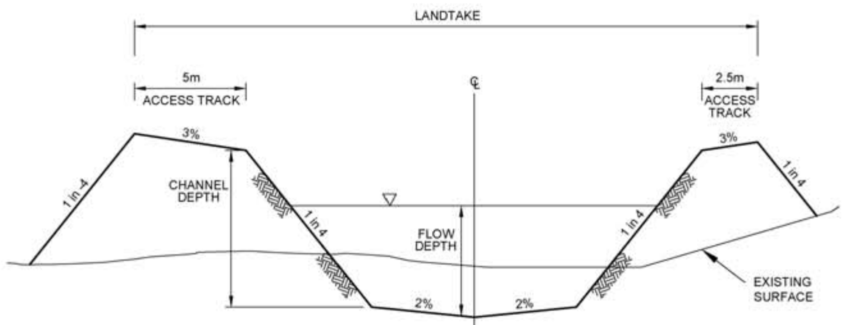

Channel TC8B Plan: Plan 11 10/05/2010
Geom: Geom 07 Flow: Flow 04
River = TC8b Reach = 50 RS = 150.00 (NOTE: Flow 04 means 100yr Flow post Climate Change)
Legend
WS PF#1
Crit PF#1
Ground
Bank Sta
.1
.035
.1
Elevation (m)
Station (m)


Channel TC8B Plan: Plan 11 10/05/2010
River = TC8b Reach = 50 RS = 200.00 Geom: Geom 07 Flow: Flow 04
(NOTE: Flow 04 means 100yr Flow post Climate Change)
.1
.035
.1
Legend
WS PF#1
Ground
Bank Sta
Elevation (m)
31.0
30.8
30.6
30.4
30.2
30.0
29.8
29.6
29.4
Station (m)
-60
-40
-20
0
20
40
60


Channel TC8B   Plan: Plan 11   10/05/2010
Geom: Geom 07   Flow: Flow 04
River = TC8b   Reach = 50   RS = 250.00   (NOTE: Flow 04 means 100yr Flow post Climate Change)
Legend
WS PF#1
Crit PF#1
Ground
Bank Sta
.1
.035
.1
Elevation (m)
Station (m)
31.2
31.0
30.8
30.6
30.4
30.2
30.0
29.8
-60
-40
-20
0
20
40
60

Channel	Channel Length (m)	Channel Grade (%)	Manning's n	Base Width (m)	Batter Slope (__H: 1V)	100yr Design Flow (m^3/s)	Flow Depth (m)	Freeboard (m)	Channel Depth (m)	Channel Width (m) Hydraulic Conveyance	Overall Width (m)	Maintenance Provisions (looking downstream)	
												5m wide Major Access	2.5m wide Minor Access
TC1	310	1.3%	0.07	14.0	4	21.6	1.0	0.5	1.5	25.0	36	Left	Right
TC4A	280	0.6%	0.07	10.7	4	12.0	1.0	0.5	1.5	21.8	30	Right	Left
TC4	480	0.6%	0.07	18.8	4	18.3	1.0	0.5	1.5	29.3	32	Internal Road at Right	Left
TC6	400	0.5%	0.07	21.3	4	18.3	1.0	0.5	1.5	31.6	40	Left	Right
TC7A	200	0.8%	0.07	9.5	4	12.7	1.0	0.5	1.5	20.7	21	Internal Road at Left	Internal Road at Right
TC7B	200	0.8%	0.07	10.0	4	13.2	1.0	0.5	1.5	21.2	22	Internal Road at Left	Internal Road at Right
TC7C	175	0.8%	0.07	11.0	4	14.2	1.0	0.5	1.5	22.2	23	Internal Road at Left	Internal Road at Right
TC8A	300	0.8%	0.07	33.2	4	32.0	1.0	0.5	1.5	42.7	51	Left	Right
TC8B	215	existing grade retained	0.07	varies	varies (4H:1V max)	28.2*	varies	0.5	varies	varies	varies	Right	Left
TC11	210	0.5%	0.07	6.0	4	7.3	1.0	0.5	1.5	17.5	20	Internal Road at Left	Right
TC12A	272	0.5%	0.07	2.0	4	10.4	1.5	0.5	2.0	17.8	23	Left	Richmond Road at Right
TC13	103	0.5%	0.07	7.1	4	8.2	1.0	0.5	1.5	18.6	27	Left	Right

*- POST CLIMATE CHANGE



CHANNEL TYPICAL SECTION
1:200 H
1:40 V

ADVANCE COPY ONLY
NOT FOR CONSTRUCTION

CLIENT: DEPARTMENT OF PLANNING C/- APP

MARSDEN PARK
DRAFT S94 BASIN REVIEW
CHANNEL TYPICAL SECTION & CHANNEL SUMMARY

PLAN No. 8955/SK17 C
FILE No. 8955_SK17

J. WYNDHAM PRINCE CONSULTING CIVIL INFRASTRUCTURE ENGINEERS & PROJECT MANAGERS

ISSUE	AMENDMENT	DATE	BY
C	ISSUES FOR EXHIBITION	01/02/11	GH
B	COUNCIL COMMENTS AMENDMENTS	30/11/10	MP
A	ISSUED FOR REVIEW	12/05/10	TG

DESIGNED DJH/ER DRAWN DJH/TG CHECKED
DATUM: AHD ORIGIN:
SCALES: H 1:200 V 1:40

THIS DRAWING MUST NOT BE USED FOR CONSTRUCTION UNLESS SIGNED AS PART OF AN APPROVED CONSTRUCTION CERTIFICATE

APPENDIX D

PRELIMINARY ENGINEERING CONCEPTS

Individual Basins and Channels have been depicted in isolation to all other Site Works. The Bulk Earthwork estimates have been based on these works being undertaken independent of each other and any other works in the MPIP.

Marsden Park Industrial Precinct