Drainage and Stormwater Management Strategy

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WIPS Management

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Executive Summary

The proposal is for the development of a pre-fabricated building material manufacturing plant, namely internal and external wall panels for domestic and commercial construction.

The development comprises warehouse (17,800 m^2), manufacturing (9,240 m^2) and office (740 m^2) space to be housed under 1 roof area. Associated curtilage areas include hardstand driveway and parking areas, as well as landscaped potential future parking areas to accommodate future changes in use. The drainage facilities provide sufficient capacity for the ultimate conversion of this area to hardstand.

The site straddles the crest of a hill, with discharge points being in the north western corner and a natural gully to the south of the proposed building. A combined water quality dry basin and stormwater detention basin is proposed at each discharge point. The southern basin will be located on the opposite side of the gully, with gully flows being conveyed via an armoured channel at the toe of the batter to the building pad. A Humeceptor Gross Pollutant Trap (GPT) is proposed upstream of each pond to remove litter and suspended solids originating from the site.

A site drainage network is proposed to drain hardstand areas, with a drainage line running up the east side of the site to provide connection points for the roof drainage, to be designed by others. A 1200 m^3 concrete tank is to be provided on the roof drainage line, with 800m^3 being available for water reuse and 400m^3 to contribute towards on site detention in order to reduce peak flows from the site. The re use tank is to provide water for toilet flushing, truck washing and garden watering. It is estimated that the tank will fill with approximately 25mm of rainfall, and provide water for about 30 days. However, to ensure water is available for toilet flushing in extended drought periods, the tank is to be provided with a trickle top up.

It is also recommended that water efficient appliances, such as shower heads and dual flush toilets be installed in order to reduce the demands on waste water services to the site.

Modelling of the development with its proposed water management strategies demonstrates that:

- runoff quality will well exceed current recommended best practice of 80%, 45% and 45% removal for suspended solids, total phosphorous and total nitrogen respectively.
- peak flow rates will not exceed predevelopment peak flow rates, and
- stormwater will be re used and demands on sewer services will be minimised in accordance with best practice and the principals of ecologically sustainable development (ESD).

Accordingly, the objectives of the Hunter Economic Zone Water Cycle Management Strategy Development Study will be satisfied.

1. Introduction

Parsons Brinckerhoff Pty Ltd (PB) was engaged by WIPS Management to prepare a drainage and stormwater management strategy for a proposed industrial development to be located within the Hunter Economic Zone (HEZ). It is understood that the proposed industry will construct prefabricated walls and panels.

This report has been prepared to support a Part 3A Application to the Department of Planning.

1.1 Study Area

The study area is located within the HEZ in the Cessnock Local Government Area. The area is adjacent to the HEZ Spine Road approximately 3km from the intersection of the Spine Road and Kurri Kurri – Mulbring Road (Leggetts Drive), Pelaw Main, as shown in Figure 1. The site covers an area of approximately 7 hectares and currently comprises native bushland.

The proposed development consists of a single large shed, surrounded by large areas of hardstand (car parking) and some landscaping. The proposed development layout is illustrated in Figure 2.

Levels across the site vary from approximately 50m AHD to 60m AHD with variable grades. The site is located on the crest of a hill, with runoff draining to the north-western and south-eastern corners. Runoff from the north-western corner of the site drains to an existing catch drain constructed as part of the HEZ Spine Road civil works (Discharge Point 1). Runoff from the south-eastern corner drains into an existing creek line located to the east (Discharge Point 2). Runoff from the study area will ultimately discharge to Hebburn Dam.

Two existing culverts that drain catchments to the west of the Spine Road discharge water onto the site. The first culvert structure is located at the southern end of the site and comprises twin 1.2mW x 0.3mH reinforced concrete box culverts (RCBC). This culvert will not affect stormwater management on the site. The second culvert comprises a 900mm diameter reinforced concrete pipe (RCP) that discharges a third of the way along the western site boundary. The flow from this culvert is to be conveyed through the site via an open channel, to be located at the toe of the batter to the southern side of the proposed building platform.

1.2 Scope of Works

The scope of works includes:

- Site inspections for familiarisation purposes and allow identification of areas that may influence stormwater management.
- Hydrologic modelling of the site using XP-SWMM software to determine existing and developed peak flow rates for the 1, 10 and 100 year average recurrence interval (ARI) storm events.
- Water quality modelling using MUSIC software to determine existing pollutant loads from the site.

- Formulation of a stormwater management strategy and treatment train that maximises stormwater retention and reuse, reduces developed peak flow rates to existing levels and provides sufficient pollutant removal in line with current best practice technology.
- Analysis and sizing of stormwater management structures using XP-SWMM and MUSIC software to demonstrate achievement of the water quality and quantity objectives in accordance with current industry best practice.
- Preparation of stormwater management plan showing size requirements for major structures and integration of these structures throughout the built form.
- Preparation of a report documenting the proposed stormwater strategy.

1.3 Objectives

The objectives of the stormwater and water quality management assessment are to be in line with those contained in the HEZ Water Cycle Management Strategy (WCMS) prepared by PB (2122348A: PR_1459 WCMS Rev D.doc). Specific principles and objectives relating to stormwater management of both water quantity and quality are:

Water Quantity

- Reduce developed runoff volumes by maximising the capture and reuse of clean water from roof and hardstand areas.
- Reduce peak developed flow rates back to existing flow rates at the site discharge point.
- Ensure that site discharges are at velocities not conducive to erosion and scour of downstream watercourses.

Water Quality

- Discharges should not result in any deterioration of existing water quality.
- A treatment train of structures should be used to reduce pollutant loads to achievable levels based on current technology and performance data.
- Incorporate the principles of Water Sensitive Urban Design (WSUD) to ensure full integration of stormwater management with landscaping and the built form.

1.4 Available Data

Information used in the preparation of this stormwater management plan includes:

- Proposed development layout provided by Justin Long Design.
- Contour map of the proposed development provided by surveyors, Harper Somers O'Sullivan.
- 6 minute rainfall data recorded at Pokolbin (Somerset), which was sourced from the Bureau of Meteorology.

A site inspection was undertaken on 4 April 2006 for familiarisation purposes to allow identification of external areas that may influence stormwater management on the site.

2. Modelling Methodology

The methodology comprises quantitative analysis of available data to estimate existing and future flow behaviour from the development site. The analysis involved examination of surface hydrology and water quality to assess runoff characteristics from the site and sizing of stormwater mitigation devices to mitigate the impact of site development on existing flows and pollutant loads.

2.1 Hydrology

The catchments contributing to runoff within the study area have been identified from survey information. The impervious area for each sub-catchment has been estimated from the proposed layout and a nominated impervious area adopted.

Runoff hydrographs within the subject area have been estimated for the 1, 10 and 100 year ARI storm events using the RUNOFF mode of XP-SWMM. The Laurenson routing method was used for the routing of rainfall excess to each sub-catchment outlet.

Long-term flow data was not available for the existing networks draining the development site to enable calibration of catchment runoff. Thus the peak flows from the XP-SWMM model at the outlet of the catchment have been compared against the Probabilistic Rational Method (PRM) peak flow estimates, as described in Australian Rainfall and Runoff (1987) for the existing catchment. Calculated peak flow estimates from both methods were obtained for 1, 10 and 100 year ARI storm events.

2.2 Hydraulics

The routing of estimated flows throughout the existing catchment was undertaken using the HYDRAULICS mode of XP-SWMM.

The HYDRAULICS mode of XP-SWMM was also used to route estimated developed flows through proposed stormwater drainage and mitigation devices for the 1, 10 and 100 year ARI storm events.

2.3 Water Quality

Water quality monitoring was undertaken in 2003 to develop a baseline for future ongoing water quality monitoring as the HEZ site is progressively developed. However, the data collected in 2003 was not intended for use in development of a predictive water quality model for the HEZ site, or for the design of individual developments.

Accordingly, water quality modelling of local WIPS Management proposed industrial development was undertaken using the Model for Urban Stormwater Improvement Conceptualisation (MUSIC, Version 3.01). Baseline pollutant loads from the local site for suspended solids, total nitrogen and total phosphorous were predicted by the model and compared against published data for similar land uses.

Modelling was then undertaken to provide a relative assessment of the likely treatment performance of the proposed water quality treatment train.

3. Existing Hydrology

In order to determine the relative impact of site development on existing catchment hydrology, it is necessary to establish existing flow conditions. Provided below are estimates of existing flow rates from each discharge point.

3.1 Parameters

The study area was divided into two separate catchments due to the existence of a ridge through the centre of the site and therefore has two discharge points. The site has been considered in isolation from external areas and runoff from areas outside the site boundary will be kept separate to the site drainage system. The site has no existing impervious component, which was confirmed during the site inspection.

Rainfall intensity-frequency-duration (IFD) data for Cessnock were sourced from an existing model of the entire HEZ site. Design rainfall pluviographs for the XP-SWMM model were generated by the modelling software, which uses the temporal patterns from AR&R [2] (1987).

The XP-SWMM model accounts for rainfall losses across a sub-catchment by implementing an Initial-Continuing Loss Rate Model. The residual rainfall following the removal of all rainfall losses from a rainfall pluviograph is referred to as rainfall excess. Other sub-catchment parameters such as average slope, roughness and percentage impervious govern the rate at which rainfall excess is routed to each sub-catchment outlet. The result of this surface routing is a runoff hydrograph.

A summary of loss and roughness parameters for pervious land use (bushland) are provided in Table 3-1.

Table 3-1: Adopted Loss and Roughness Parameters for Existing Conditions

Parameter	Existing Conditions (Bushland)
Initial Loss (mm)	15
Continuing Loss Rate (mm/h)	2.5
Roughness	0.07

3.2 Existing Flow Rates

Estimated flow rates at the catchment outlet were compared with estimates obtained using the PRM estimated peak discharges. This was undertaken to confirm the magnitude of model predicted flows, and the validity of the catchment parameters selected. The comparison between estimated flow rates from the model and PRM estimated flow rates is shown in Table 3-2 for each discharge point.

Table 3-2: Comparison of Existing XP-SWMM and PRM Peak Flow Rates

ARI	Discharge Point 1 (DP1)			Discharge Point 2 (DP2)		
	PRM Estimate (m^3/s)	Existing Flow Rates (m^3/s)^	% Difference	PRM Estimate (m^3/s)	Existing Flow Rates (m^3/s)^	% Difference
1 Year	0.04	0.04 (540)	0%	0.13	0.15 (540)	13%
10 Year	0.11	0.10 (270)	-10%	0.39	0.41 (270)	5%
100 Year	0.22	0.20 (120)	-10%	0.75	0.75 (120)	0%

^ Shown in brackets is the critical storm duration (mins) that produced the highest peak flow rate.

4. Developed Hydrology

To assess the relative impact of the proposal on existing catchment hydrology, the development has been modelled to establish developed flow conditions (excluding mitigation structures). The following design parameters were adopted and estimates of developed flow rates at the catchment outlet calculated.

4.1 Parameters

The catchment of the proposed development was divided into sub-catchments to allow better representation of site development. Sub-catchment boundaries are defined by the preliminary drainage layout.

A single building is proposed for construction on the site. The building will be surrounded by extensive hardstand areas with pervious areas (landscaping) being minimal. The percentage impervious for each sub-catchment was calculated by measuring the impervious areas from the supplied building layout.

A summary of loss and roughness parameters for pervious and impervious land uses adopted in the developed model are provided in Table 4-1.

Table 4-1: Adopted Loss and Roughness Parameters for Developed Conditions

Parameter	Pervious	Roof	Hardstand
Initial Loss (mm)	10	0.5	1.5
Continuing Loss Rate (mm/h)	2	0.0	0.0
Roughness	0.07	0.011	0.014

4.2 Developed Flow Rates

A comparison between existing and developed peak flows expected from each catchment outlet during the 1, 10 and 100 year ARI storm events are provided in Table 4-2.

Table 4-2: Developed Peak Flow Rates at Catchment Outlet

ARI	Discharge Point 1			Discharge Point 2		
	Existing Flow Rates (m^3/s)^	Developed Flow Rates (m^3/s)^	Percentage Difference	Existing Flow Rates (m^3/s)^	Developed Flow Rates (m^3/s)^	Percentage Difference
1 Year	0.04 (540)	0.19 (15)	+375%	0.15 (540)	0.80 (20)	+433%
10 Year	0.10 (270)	0.29 (15)	+190%	0.41 (270)	1.48 (15)	+261%
100 Year	0.20 (120)	0.35 (15)	+75%	0.75 (120)	2.07 (15)	+176%

^ Shown in brackets is the critical storm duration in minutes that produced the highest peak flow rate.

Table 4-2 shows that the proposed development results in large increases in existing peak flow rates at both catchment outlets for the 1, 10 and 100 year ARI storm events. As the proposed development has led to an increase in peak discharge, the provision of storage structures such as rainwater tanks and detention storage will be required to mitigate flow from the developed site.

As previously stated, the flow from the existing 900mm diameter RCP located beneath the Spine Road will be piped through the site and discharge at DP2. Modelling indicates that the peak flow resulting from the 100 year ARI storm in this catchment is 1.64m^3/s, and that a 900mm diameter pipe is capable of conveying the required flow. It is assumed that any future development to the west of the Spine Road that drains to the 900mm diameter pipe will provide sufficient controls to attenuate developed flows back to existing levels. This will prevent the need to upgrade this pipe in the future.

5. Mitigated Hydrology

This section describes the stormwater management strategy for the proposed development and quantifies its performance. Storage within the site has been provided in the form of underground rainwater tanks and dry detention basins incorporating bio-retention.

5.1 Stormwater Strategy

A stormwater management strategy for the proposed development is required to manage runoff. A plan showing an indicative layout of proposed stormwater management structures is shown in Figure 3. This figure also shows the suggested drainage network for the proposed development.

The large building to be constructed will have a rainwater tank for the collection of clean roof water. Harvested water is to be reused onsite for landscape watering, toilet flushing and hardstand, plant and vehicle wash down.

Dry detention basins have been provided at two locations as shown in Figure 3. Conceptual sketches of BASIN 1 and 2 are provided in Figure 4 and Figure 5.

All pipe drainage outlets will have scour protection comprising dumped rock to prevent erosion and scour.

5.2 Parameters

The developed XP-SWMM model includes storage provided by the rainwater tank (TANK 1) and detention structures.

The proposed rainwater tank was modelled as a storage reservoir. TANK 1 is shown in Figure 3 and comprises a 40 metre long by 10 metre wide by 3 metre deep tank. The tank collects the first 800m^3 of runoff from the building roof area and temporarily detains a further 400m^3 of runoff, giving a total volume of 1,200m^3. It has been assumed that the tank contains 800m^3 of runoff at the beginning of the modelled storm events. TANK 1 will require connection to mains supply to ensure suitable top-up to meet anticipated reuse demand during periods of extended dry weather.

The detention basins (refer section 5.3) have been similarly modelled with volumes as described in Table 5-2.

5.3 Dry Detention Basin Characteristics

The characteristics of each dry detention basin are shown in Figure 4 and Figure 5. BASINS 1 and 2 drain principally via the in built bio-retention media, however basin 2 has an additional 225mm diameter outlet pipe to emulate natural catchment characteristics for larger events.

5.4 Mitigated Flow Rates

The mitigated peak flows at the catchment outlet following the introduction of mitigation measures for the 1, 10 and 100 year ARI storm events are provided in Table 5-1. Also provided for comparison are existing flow rates at each discharge point. Table 5-2 shows the peak storage, water level and water depths in each storage for the 1, 10 and 100 year ARI storm events.

Table 5-1: Mitigated Peak Flow Rates at each Discharge Point

ARI	Discharge Point 1			Discharge Point 2		
	Existing Flow Rates (m^3/s)^	**Mitigated Flow Rates (m^3/s)^**	**Percentage Difference**	**Existing Flow Rates (m^3/s)^**	**Mitigated Flow Rates (m^3/s)^**	**Percentage Difference**
1 Year	0.15 (540)	0.07 (540)	-53%	0.04 (540)	0.02 (90-540)	-50%
10 Year	0.41 (270)	0.38 (720)	-7%	0.10 (270)	0.10 (120)	+0%
100 Year	0.75 (120)	0.78 (120)	+4%	0.20 (120)	0.21 (120)	+5%

^ Shown in brackets is the critical storm duration in minutes that produced the highest peak flow rate.

Examination of Table 5-1 reveals that the proposed detention controls adequately attenuate developed flow rates back to existing levels.

Table 5-2: Peak Storage, Water Level and Depth in Storages

Storage	Parameter	ARI		
		1 Year	**10 Year**	**100 Year**
BASIN 1	**Peak Storage (m^3)**	866	1279	1444
	Peak Water Depth (m)	0.62	0.86	0.97
BASIN 2	**Peak Storage (m^3)**	179	250	302
	Peak Water Depth (m)	0.48	0.65	0.79