STORM
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Water Sensitive Urban Design for Common Street Business Park, Goulburn

Report Prepared for

Goulburn City Council

Prepared by:

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EXECUTIVE SUMMARY

Approximately 105 hectares of industrial development is planned in the Common Street Business Park area located in the south east of Goulburn. This area is within the Goulburn City Council local government area and is part of the Sydney Catchment Authority (SCA) drinking water catchments.

Goulburn City Council (Council) engaged Storm Consulting Pty Ltd (STORM) to undertake stormwater and water cycle investigations and make recommendations specific to developing the industrial zones within the Common Street Business Park. This information will guide Council so that they may prepare an appropriate Development Control Plan (DCP) that will satisfy its own requirements and those of the Sydney Catchment Authority (SCA).

This report documents the methodology and results:

- That will enable Council to form a broadly applicable on site detention (OSD) policy for Industrial Land in Goulburn Council. The determination of this policy represents a large reduction in the volume of storage required when compared with the Upper Parramatta River Catchment Management Trust (UPRCT) policy previously applied in Goulburn. The new policy developed by STORM policy may first be applied at Common Street.

- That will enable Council to form a broadly applicable on-site retention (OSR) policy for industrial land in Goulburn. In this report on site retention refers to the retention of stormwater in rain tanks within each development for use as a non-potable supply.

- That will enable Council to identify creek corridor widths to safely convey the 1 in 100 year ARI storm event through the site. This is based on a typical creek cross section which allows for revegetation throughout.

- That will enable Council to satisfy the SCA by instituting water quality controls to minimise the pollutant loads entering the downstream waterway.

- Of an investigation into the feasibility of capturing rainfall runoff in excess of the water that is to be reused on site. The investigation was to determine appropriate uses for the water, to determine an approach and approximate costs of storing, treating and delivering the water.

OSD rates that are particular to industrial development using Goulburn's planning, climatic, soil and slope conditions were developed to limit the post development flows to the predevelopment flows for storms up to the 100 year average recurrence interval (ARI) storm event. The Permissible Site Discharge (PSD) and Site Storage Requirements (SSR) were determined using appropriate computer software. The recommended PSD is 215 L/s/hectare and the recommended SSR is 140 m^3/hectare of proposed development. This compares very favourably with an SSR of 470 m^3/hectare of proposed development required under the UPRCT guidelines.

There is an approximate ten fold increase in the frequency and fourfold increase in the volume of runoff that would occur after development of land for industrial purposes. The use of rain tanks therefore has a twofold purpose. The first is to supply a viable source of water for non potable purposes. The second is to limit the frequency and volume of runoff post development. The optimum rain tank size was found to be a 20 kL/hectare of development, which is estimated to provide a high yield. The 20 kL rain tank per hectare of development also resulted in fewer spills when compared to the no-tank scenario. This is a minor benefit for low or environmental flows. The rain tanks alone therefore have a limited effect on flows. While this is not true for residential development it is true for industrial development with large roof areas and relatively low demands for water. Therefore the use of rain tanks will be complemented by the proposed on-site detention policy.

Thus two policies, an OSD policy to combat peak flows in rare storm events and OSR policy to combat low flows in frequent storm events will both work to limit the impacts of the proposed development.

Minimum corridor widths and channel profiles were developed for the Common Street Business Park Precinct to safely convey water throughout the site. The minimum widths required for the trunk drainage corridor in the upper catchments range between 31 and 46m. In these sections the depth of flow, velocities and vd's in a 100-year ARI storm event are less than 1.0. Lower down in the catchment the corridor width become much wider (44-66m) and the depth of flow, velocities and vd's increase, but are generally below 1.5. In addition to these widths, a typical creek cross section with 10m base and 1 in 6 batters are to be adopted throughout the upper sections of the creek system. Adjacent to the Mulwaree River this cross section changes to better represent natural creek design. These creek corridors are to be revegetated to reduce Council's maintenance burden and meet Department of Infrastructure Planning and Natural Resource's requirements for riparian management under the River's and Foreshores Improvement Act.

Various water quality management schemes have been identified and modelled for use within the precinct.

STORM has recommended that Stormfilters® be installed as the quality management option at source to satisfy Sydney Catchment Authority's requirements. An end of pipe wetland may also be constructed adjacent to the Mulwaree River in the future. This location already has some ponds acting as semi natural wetlands that can be replaced with a wetland system which will give further water quality benefits.

The feasibility of harvesting stormwater runoff over and above the non-potable needs of the industrial estate itself resulted in the following conclusions:

- Where the proposed end use is located close to a roof area, the cost of storage and pumping from the industrial development would exceed the cost of constructing a storage and pumping system at the end use. For example, the cost of constructing a large rain tank and pump system at the gaol would be in the order of $10,000 to $20,000. The cost of providing the infrastructure to store and pump this water from the Common Street Business Park would be in excess of $175,000. Therefore small-scale reuse appears not to be economically viable.

- Where the proposed end use is not located close to a roof area, for example the cemetery where it would not be possible to harvest large quantities of runoff even if a dam was constructed it would cost in the order of $180,000 to construct a storage and delivery system. Given that the water in these instances is water that is not used for essential purposes, the benefit is questionable. It would instead be far more beneficial to install rain tanks and small pump systems, at much lower cost, on a number of existing buildings – such as the prison or Police Academy.

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1.0 INTRODUCTION

Goulburn City Council (Council) has engaged STORM Consulting to prepare a stormwater management strategy for the Common Street Business Park.

1.1. BACKGROUND

The Common Street Business Park area is approximately 195 hectares of partially developed land located in the south east of Goulburn. Further industrial development is planned in three pockets (totalling approximately 69 hectares) within this area. This area is within the Goulburn Local Government Area and is part of the Sydney Catchment Authority (SCA) drinking water catchments defined by SEPP 58. Council is currently preparing a DCP and a Section 94 Contribution Plan to manage the future industrial development within the Common Street Business Park.

Given that the development is located within the highly sensitive SCA catchments there is a great need to protect those catchments from the impacts of such a development. It is well documented that industrial development can lead to the degradation of water quality in receiving waters and have a dramatic effect on the hydrological regime. Construction of large impervious surfaces reduces natural stores of water in the soil profile. This leads to significant increases in the frequency of runoff (often by a factor of 10), the peak flows for runoff events and the volume of runoff events.

Traditional approaches to stormwater management would implement either on-site or communal detention which would reduce peak flows and, to some degree, improve water quality by limiting erosion related to high velocities.

Council has requested that an on site detention (OSD) policy be prepared for Common Street and moreover that the policy is applicable not just to Common Street but also to the whole of the Goulburn LGA.

It is understood that detention alone has no impact on reducing the volume and frequency of runoff which may result in unsustainable consequences for the downstream water environments. The impact of increased flows is likely to lead to an alteration of the stability of receiving waters and local creeks that convey flows toward the Sydney Water drinking water off-takes. A traditional detention approach captures water but does not accommodate reuse of that water. In areas such as Goulburn where a consistent potable water supply is difficult to maintain, capture of rainfall and reuse of that water on-site and in the local area reduces the demand on the potable water supply.

Another traditional approach may be to pipe and concrete various components of the stormwater conveyance system. Although hydraulically efficient, these materials do not allow for the natural attenuation of pollution that occurs in both the soil and in natural creeks systems.

Therefore, to effectively address water management in new developments a departure from the traditional approach is required. Development of critical elements of a water cycle management plan that is cognisant of the various issues relating to the sustainable management of water is a step towards effective water management.

Additional runoff is created through the creation of large impervious surfaces that prevent the soil water stores from absorbing rainfall. This report presents the results of investigations into the application of on site controls (on site detention and retention) and also the opportunity to harvest rainwater runoff from the development, in excess of the predevelopment flow that would have left the development.

1.2. SCOPE OF REPORT

STORM has been commissioned to undertake 5 key tasks associated with the strategic planning for water sensitive urban design in Common Street Business Park. These include:

Task 1 – On-site retention of rainwater and on-site detention analysis.

Task 2 – Trunk Drainage corridor development and assessment

Task 3 – Water quality management

Task 4 – Broad stormwater reuse investigation.

Task 5 – Reporting.

Each of these tasks are detailed below, and presented in a format that can be easily incorporated into a DCP by Council.

2.0 LOCAL CONDITIONS

2.1. HYDROLOGY

Rainfall data for the Common Street Business Park has been obtained from the Bureau of Meteorology. Key hydrological statistics obtained from the Bureau are summarised below in Table 1. Approximately 27 years of good quality daily rainfall data was used that extended from 1971 to 2002 (portions of data between 1975 and 1979 were missing).

Table 1 - Rainfall statistics for Goulburn (Progress Street)

Statistic	Annual Average
Mean rainfall – mm	672.7
Median (5th decile) rainfall - mm	651.2
9th decile of rainfall - mm	854.3
1st decile of rainfall - mm	420.3
Mean no. of raindays	127.2

Source: the Bureau of Meteorology.

The average annual evaporation rate for Goulburn is 1289 mm/year. Goulburn is therefore in a net evaporation area.

2.2. SOILS

According to Council's Stormwater Management Plan (Goulburn City Council, 2000), Goulburn's soil profiles generally have poor drainage characteristics. Soil properties are described as having moderate permeability, moderate topsoil erodibility, low subsoil erodibility and moderate shrink-swell potential.

The Common Street Business Park overlays two different soil landscapes; the Bullamalita and Goulburn soil Landscapes. Their respective properties are presented in Table 2.

Table 2 – Soil Characteristics

Soil Landscape	Characteristics: Drainage	Permeability	Erodibility (top soil)	Erodibility (subsoil)	Shrink / Swell Potential
Bullamalita	Poor	Slow	High	High	Low
Goulburn	Mod/Poor	Slow/Mod	High	High	Low

Source: Goulburn City Council Stormwater Management Plan, 2000

2.3. VEGETATION

There is some vegetation within the Common Street Business Park area (as noted on the constraints map prepared by Council. The areas of most concern to potential development are the areas of endangered / protected vegetation adjacent to the drainage corridors in two of the three designated industrial zones.

The existing vegetated areas are shown in Figure 1.

2.4. EXISTING DEVELOPMENT

There is considerable existing development and supporting infrastructure in the area defining the Common Street Business Park DCP.

The existing development includes development along the Sydney Road, such as motels, a caravan park and McDonalds.

There is considerable infrastructure in place, such as water mains and roads that serve the existing development.

Existing development is shown in the background of Figure 1.

2.5. FUTURE SITE DEVELOPMENT

It is understood that the DCP for Common Street Business Park will include future site development standards that will stipulate a minimum landscaping requirement but not a minimum area to remain pervious. The work in this report has been based on a typical 1 hectare lot which assumes:

- Roof area of 5000 m^2;
- Access, parking and hard-stand area of 2000 m^2;
- Storage of about 1000 m^2; and
- Landscaping and protection of ecological services, 2000 m^2.

3.0 REGULATORY REQUIREMENTS

3.1. GOULBURN CITY COUNCIL

The Stormwater Management Plan (SMP) outlines Council's broader objectives in regard to new developments and stormwater quality management. Relevant objectives contained within the SMP include:

- Urban development should only occur in areas where a land capability study has indicated that area is physically capable of supporting the proposed type of development without causing significant soil erosion, land slip or water pollution;
- Water-sensitive urban design principles should be incorporated in the development;
- A strong emphasis should be placed on the management of stormwater at or near the source. This applies to both the quantity and quality of stormwater;
- The reuse of stormwater for non-potable purposes should be encouraged. This should be undertaken in the context of total water cycle management;
- Where appropriate "natural" channel designs should be adopted in preference to grass or concrete lined floodways, unless there are specific requirements for a lined channel;
- Site specific studies should be undertaken to identify the sustainable pollutant export from the development site. In the absence of these studies, there should be no net increase in the average annual load of pollutants critical to the health of receiving water ecosystems and human health, under post-development conditions. If this cannot be achieved, an 'offset' scheme could be developed where contributions are obtained from developers for rectifying existing problems affecting the 'health' of watercourse and water bodies within the catchment;
- Soil and water management practices should be implemented during the construction phase of the development to minimise soil erosion and sediment export;
- The applicable ANZECC water quality guidelines should be met for water bodies receiving stormwater runoff that is used for water supply purposes;
- The impact of urban stormwater on weed propagation and growth in bushland should be minimised;
- The impact of stormwater on public health and safety should be minimised;
- Opportunities for the multiple use of drainage facilities are to be encouraged, to the degree that they are compatible with other management objectives;

- The visual amenity and landscaping opportunities of stormwater systems are to be optimised;
- Peak flows from the development site should be attenuated so that there is no net increase in flows for event from the 1 year to 100 year average recurrence interval;
- The risk of property damage due to stormwater and groundwater should be minimised;
- The disruption to traffic and pedestrians during frequent storm events should be minimised;
- Protect and maintain natural wetlands, watercourses and riparian corridors; and
- Use of vegetated flow paths maximised.

Quantitative and qualitative stormwater management objectives that were generated for new development through the stormwater management planning process are presented in Tables 3 and 4 below.

Table 3 - Quantitative Stormwater Objectives for New Developments

Pollutant/Issue	Retention Criteria
Coarse Sediment	80% of average annual load for particles ≤ 0.5 mm
Fine Particles	50% of average annual load for particles ≤0.1 mm
Total Phosphorus	45% of average annual pollutant load
Total Nitrogen	45% of average annual pollutant load
Litter	90% of average annual litter load > 5 mm
Hydrocarbons, motor fuels, oils and grease	90% average annual pollutant load

Source: Goulburn Council Stormwater Management Plan, 2000

Table 4 - Qualitative Stormwater Objectives for New Developments

Pollutant/Issue	Management Objective
Runoff Volumes	Impervious areas connected to the stormwater drainage system are minimised.
	Reuse of stormwater for non-potable purposes maximised.
Stormwater Quality	Use of vegetated flow paths maximised.
	Use of stormwater infiltration 'at source' where appropriate.
Riparian Vegetation and Aquatic Habitat	Protect and maintain natural wetlands, watercourses and riparian corridors. All natural (or unmodified) drainage channels within the site which possess either: • base flow • defined bed and/or banks; or • riparian vegetation are to be protected and maintained. "Natural" channel design should be adopted in lieu of floodways in areas where there is no natural (or unmodified) channel.
Flow	Alterations to natural flow paths, discharge points and runoff volumes from the site to be minimised. The frequency of bank-full flows should not increase as a result of development. Generally, no increase in the 1.5 year and 100 year peak flows.
Amenity	Multiple use of stormwater facilities to the degree compatible with other management objectives.
Urban Bushland	Impact of stormwater discharges on urban bushland areas minimised.

Source: Council's Stormwater Management Plan, 2000

3.2. SYDNEY CATCHMENT AUTHORITY

SEPP 58 requires that the SCA assess and consent to development within the drinking water catchments. The SCA is required to assess the development proposal in relation to Clause 10 of SEPP 58 – matters for consideration, specifically:

(a) Whether the development or activity will have a neutral or beneficial effect on the water quality of rivers, streams or groundwater in the hydrological catchment, including during periods of wet weather;

(b) Whether the water quality management practices proposed to be carried out as part of the development/activity are sustainable over the long term; and

(c) Whether the development/activity is compatible with relevant environmental objectives and water quality standards for the hydrological catchment when these objectives and standards are established by Government.

Clearly there is an inconsistency between Goulburn Council (2002) objectives and the objectives of the SCA. Council's objectives are load based not outcome based. The SCA's objectives for new developments would override those of Council's and so it is suggested that Council objectives do not become the benchmark by which this water cycle management plan is to be measured.

The SCA does not have prescribed distances required to buffer natural watercourses from sewered urban developments.

3.3. DEPARTMENT OF INFRASTRUCTURE PLANNING & NATURAL RESOURCES

The Department of Infrastructure Planning and Natural Resources (DIPNR) administers the Rivers and Foreshores Improvement Act (RFIA). Some of the creeks within the Common Street Business Park will be subject to approval under this Act.

All waterways require a 40m setback from the top of the bank, however under Part 3(a), a permit can be issued to build structures within the setback (ie. GPTs, stream rehabilitation). The creeks shown in Figure 1 would be likely to be administered by the DIPNR.

The Water Management Act (2000) will not be applicable to the Common Street Business Park unless bores or water storage dams are installed.

4.0 ISSUES & CONSTRAINTS

4.1. CONSTRAINTS MAP

Council has undertaken constraints mapping for the Common Street Business Park area shown in Figure 1. This plan shows the following limitations:

- Drainage corridors
- Remnant vegetation
- 100-year flood extents

4.2. SOIL LIMITATIONS

Table 2 in Section 2.2 of this report details the characteristics of the soils found within the Common Street Business Park. Both the Bullamalita and Goulburn soil types have low to moderate drainage and permeability properties. This limits the types of measures applicable for use within the Common Street Business Park development. Infiltration is not recommended for use with the area unless soil permeability tests are undertaken to confirm suitable permeability rates.

According to the SMP, Goulburn has no identified areas suffering from salinity at the time of dry land salinity mapping by the DLWC in the 1980's. There is evidence of small areas of salinity in Goulburn's South (near Bungonia Road) at the effluent Irrigation Farm and on Kenmore Land.

Development on the site may require geotechnical investigations to determine the extents of these constraints with regard to their impact on soil acidity, foundation hazard and foundation design.

Figure 1 - Common Street Business Park Constraints Map

Source: Common Street Business Park DCP No.13 - Goulburn City Council

5.0 ON SITE STORMWATER QUANTITY MANAGEMENT

5.1. ON SITE RETENTION

5.1.1. Background

The use of rain tanks for the harvesting and use of stormwater for non potable purposes is recommended for Common Street. This is called on site retention and is aimed at reducing both the frequency and volume of runoff from a developed site. On site retention is a management tool that aims to:

1. Provide a viable alternative supply of water for non potable purposes.
2. Reduce the frequency and volume of runoff from the roof area of the developed site.

The benefits of on site retention are therefore twofold.

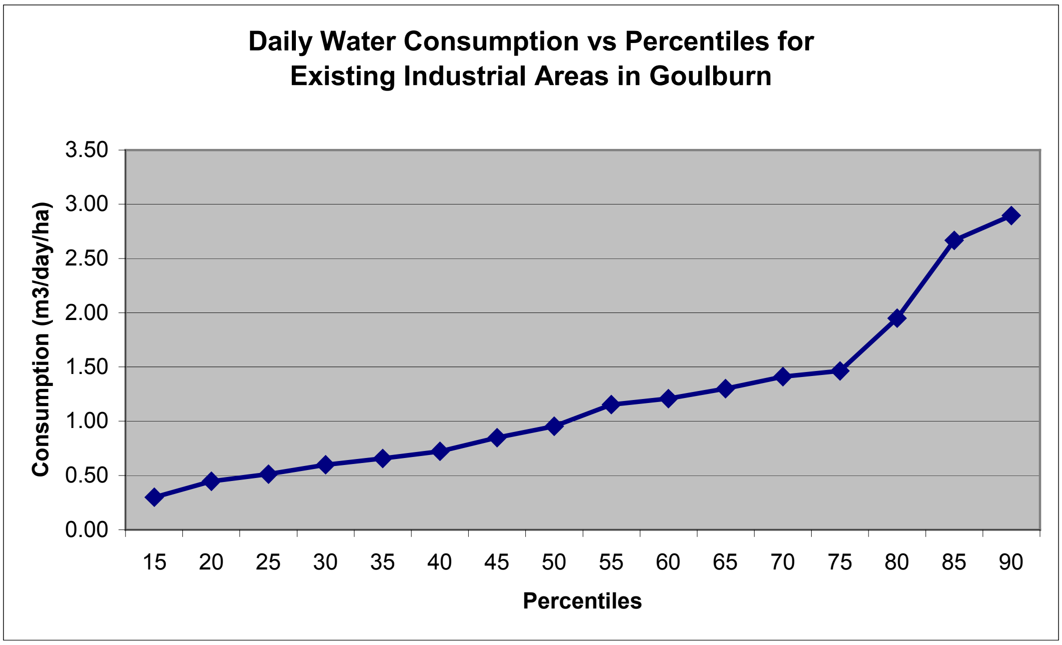
5.1.2. Methodology

5.1.2.1. Estimating Daily Water Demand

To assess both the hydrological benefit of using rain tanks and the potential yield from tanks on the development, it was necessary to estimate potential water consumption for the proposed industrial development.

Metered consumption rates for a similar industrial development in Goulburn were made available and used for the statistical analysis for the Common Street Business Park. The metered consumption was divided by the number of days included in the metering period and the gross lot size. A consumption rate expressed in cubic metres per day per hectare of lot was therefore obtained. The percentiles were then determined and graphed (Figure 2) for clarity and analysis.

Figure 2 - Water Consumption Percentiles for Common Street Business Park



From Figure 2 the following conclusions can be drawn:

- The 75 percentile consumption rate is about 1.50 m^3/day/Hectare of development, The 50 percentile value is less than 1.0 m^3/day/Hectare and the 25 percentile is about 0.50 m^3/day/Hectare;
- The extreme consumption rates (between the 75% and 100%) stem from organisations such as Tip-Top, Barry Burrows Engineering and Pirtex where it is assumed that they consume water in their production / manufacturing processes or use water for cleaning/washing purposes such as showering after work.
- The median consumption value is approximately 1.13m^3/day/Hectare and has been adopted in this study.

It is important to note that these values are rates of consumption on a per hectare basis.

5.1.2.2. Demands and Estimated Tank Performance Sensitivity

The sensitivity of the tank performance to changes in the water demand has been assessed using a spreadsheet model. This was done by comparing the storage volume in the tanks with high (75 percentile), low (25 percentile) and median water demand rates.

It was found that the draw down or performance of the tank did not change significantly when using the different consumption rates. This implies that the storage levels would be fairly close in tanks of the same configuration (and volume) on two different properties (with the same roof area) but with demand differing by up to 0.95m^3/day/Hectare.

Similar findings were reached in a recent study by STORM at a proposed industrial estate in North Wyong where tank performance was not highly sensitive to demand.

Table 5 includes a sensitivity analysis of the performance of rain tanks with low median and high levels of demand.

5.1.2.3. Modelling Rainfall Runoff

In order to model the hydrological effect of the proposed industrial development, a spreadsheet model was developed and run with 27 years of recorded daily rainfall data.

The site development standards identified in Section 2.4 were used to define the pervious and impervious areas on a typical 1-hectare lot. The direct runoff and surface runoff was calculated using runoff, drainage store and upper soil store values consistent with the soils found on site

The total number of runoff days each year was estimated to be approximately 6 days per year. This sets the benchmark for the post-development on-site detention. Further Council provided a graph of storage versus time for Pejar and Sooley water storages. A crude analysis of these graphs indicated that on 4 days per year on average there was major catchment water shedding. In this report runoff is defined as a situation where the soil moisture stores are completely saturated and any further water that falls on the ground is then shed from the catchment. Whether the number of runoff days is 4 or 6 is not significant – what is significant is that the number of runoff days is significantly less than that that would occur after development of an industrial estate.

5.1.2.4. Modelling Rain Tank Performance

It was assumed that a dual plumbed system would supply the water for the development in accordance with the system used to study the effect of rain tanks for the Upper Parramatta River Catchment Trust (Coombes 2002). That is, rainwater would be pasteurised in a hot water tank before use as a hot water supply. The first flush of rainfall would need to be bypassed from the system. Toilet water and irrigation water would also be supplied from the rain tank. Drinking water supply would come from the town water supply. The rain tank would function as a reservoir with augmentation from the town's supply to top up the tank to a minimum or "low" level.

If process water were required to be of potable standard then this demand would need to be supplied by town mains. The modelling undertaken does not consider possible process use of rain tank water, as this can not be defined at this stage.

While metered water consumption rates are available to estimate gross water consumption, there is very little data available that disaggregates water demand into its final end use. Disaggregated water demand data that is available is for residential development rather than for an industrial development. For residential developments it has been assumed that a maximum of 87% of the total indoor water demand could be potentially supplied from a large enough rain tank. On industrial sites this is likely to be greater as the percentage of potable use on an industrial estate is likely to be higher by comparison. For the purposes of this study 90% has been assumed.

Importantly as noted above, changes in the level of water demand between the 25% and 75% (ie from $0.51m^3$/day/ha to $1.46m^3$/day/ha) are not likely to alter the effectiveness of the tanks in terms of either reducing runoff or the level of dependence on the town's water supply. This is provided that a 20 kL tank size is adopted per hectare of development.

Each of these various parameters were compared in a spreadsheet model and are presented under Results in Section 6.1.3.

5.1.2.5. Good Practice

The town mains top up should be set to fill the tank to a level so that the pump intake is always below water. Only "effective" storage volumes, ie. storage above the top up level, were modelled in this study. Thus Council will be able to prescribe an effective tank volume as the geometry of tanks differ from manufacturer to manufacturer. Air space below the water supply inlet point must be left to prevent possible backflow into the supply pipe.

The Australian and New Zealand Standard, *AS/NZS 3500.1.2:* Water *Supply - Acceptable Solutions* provides guidance for the design of rainwater tanks with dual water supply (rainwater and mains water). It categorises cross connections between mains water supply and premises with a rainwater tank to be 'low hazard', thereby requiring a non-testable backflow prevention device.

After any extended dry period it is good practice to let the first runoff of rain bypass the tank. This first rain will wash or flush the roof catchment and usually contains higher amounts of accumulated dust, bird droppings, leaves and other debris.

Regular maintenance of the rainwater tanks is critical. Studies by Newcastle University have shown that any incidence of health related effects of drinking water from a rain water tank are usually related to a lack of maintenance. Reference should be made to the monograph: *Guidance on the Use of Rainwater Tanks* (National Environmental Health Forum 1998) for maintenance procedures. The catchment area should be kept clear of debris.

5.1.2.6. Rainwater for Drinking Water Purposes

The use of rainwater for drinking purposes (where potable water is available) is not recommended by the NSW Health Department and therefore not recommended in this report. While it is not prohibited, if rainwater is planned to be used for drinking purposes, then reference should be made to the monograph *Guidance on the Use of Rainwater Tanks* (National Environmental Health Forum 1998).

5.1.3. Results

The rain tank performance was analysed by using the average annual spills and percentage supply for each size of rain tank using a spreadsheet model. This analysis is summarised in Table 5 below.

The average annual spills is the number of times per year that the rain tank overflows into the stormwater system. The percentage supply is the percentage of total water demand supplied by the rain tank. The number of annual spills from a developed lot was also determined to compare the numbers of spills from a rain tank with a no-tank option. The number of annual run off days for the "no-tank" option was found to be 97.

This comparison allows us to determine the optimum rain tank size. A 20 kL tank was selected over a 10 kL tank because the 20 kL tank yields a further 11% water supply (medium consumption) from tanks while the 50 kL tank yields only a further 5% (more than the 20 kL tank) while more than doubling the tank size. The impact on runoff days is almost the same for any of the tanks analysed and so water yield was the primary differentiating factor.

Table 5 – Effect of Tank Sizes on Spills and Percentage Supply

		0kL tank	10kL tank	20kL tank	50kL tank
Post Development	Ave Ann Spills for Low Consumption	97	86	86	86
	% of supply	0	*87*	*90*	*90*
	Ave Ann Spills for Median Consumption	97	75	74	73
	% of supply	0	*74*	*85*	*90*
	Ave Ann Spills for High Consumption	97	73	70	68
	% of supply	0	*68*	*81*	*89*

From this analysis a number of conclusions can be drawn:

- The additional benefit in terms of the reducing the number of spills becomes marginal when the tanks are at about 10 kL. That is, the benefit of using a larger tank, from a stormwater point of view, is negligible in terms of affecting the frequency of runoff days.

- Any rain tank size presents an improvement on the number of annual spills when compared with the no-tank option of 97 days of runoff.

- The certainty of supply is relatively high – virtually at maximum yield. Recall that the maximum possible yield is 90%. In some scenarios this is achieved. This means that the development can go ahead without placing additional burden on the Goulburn water supply system and it will only take 4 mm of rainfall to fill a 20 kL tank. This assumes that no major water consumers develop on the site.

The effect of the roof size on the rain tank performance was also assessed to test the sensitivity. The typical lot breakdown (as described in Section 2.4) was compared with lots that had a “large” and “small” roof as detailed below in Table 6.

Table 6 – Lot Breakdowns used in the Tank Sensitivity Analysis

Typical	roof area =	0.5 ha
	landscaping area =	0.2 ha
	non roof impervious =	0.3 ha
Large	roof area =	0.7 ha
	landscaping area =	0.2 ha
	non roof impervious =	0.1 ha
Small	roof area =	0.3 ha
	landscaping area =	0.2 ha
	non roof impervious =	0.5 ha

The spreadsheet model was then run using the various lot breakdowns, with the results from each scenario presented in Table 7 below.

It can be seen from this table that roof size does not significantly impact on either the average annual spills or the percentage of demand supplied from the rain tank.

Table 7 – Effect of Roof Size on Annual Spills and Percentage Supply

	Median Use	
Roof Size	Ave Ann Spills with 20kL tank	% Supply
Typical	74	*85*
Large	79	*85*
Small	63	*83*

5.2. ON-SITE DETENTION

OSD systems are used to restrict the peak stormwater flows that would occur post development to that prior to development. OSD systems have three main elements:

1. Discharge Control – limiting the flow from the site by the use of a pipe, orifice or other means.

2. Storage – either a closed tank or above ground depression detained to contain the excess volume of stormwater unable to get through the discharge control.

3. Overflow Management –a spillway or dedicated flow path used to direct stormwater in extreme storm events or in system failures away from items that will be adversely affected by these flows.

5.2.1. Background

Council currently adopted the Upper Parramatta River Catchment Trust's OSD policy and is applicable to development and redevelopment sites within the city where under capacity drainage systems exist. Recently OSD has been required for industrial zones but there is no policy at present to support this. This report documents the creation of an OSD policy for industrial development in the Goulburn LGA. This policy does not consider the capacity of the downstream stormwater system. Instead it is intended as a broad policy to be used on new industrial developments where Council would like to limit the peak flows leaving the site post development to ensure that:

- The there is no loss of life or damage to property arising from an increase in peak flows leaving a development.

- Environmental harm or damage does not occur as a result of an increase in peak flows arising from the creation of large impervious areas.

The OSD policy documented herein aims to ensure that new developments and redevelopments do not increase peak stormwater flows in any downstream area during major storms up to and including the 100 year ARI (1% Annual Exceedence Probability) storm events.

The recommended 'Control Standards' are as follows:

- Permissible Site Discharge (PSD) – which specifies a maximum allowable discharge from the developed site.

- Site Storage Requirement (SSR) – the volume of storage that needs to be constructed and verified expressed as a rate per hectare.

- Minimum Outlet Size – designed to limit the potential blockage of the outlet.

- Maximum permissible surface ponding depths – various, maximum 600mm (potentially deeper where safety is ensured) for public health and safety reasons.
- Safety Fences – fencing required when gentle side slopes cannot be accommodated.
- Overland flow paths created to ensure a failsafe system is put in place.

The OSD analysis is based on a typical 1 hectare lot which assumes:

- Impervious roof area of 5000 m^2;
- Impervious access, parking and hard-stand area of 2000 m^2;
- Impervious storage of about 1000 m^2; and
- Pervious landscaping and protection of ecological services, 2000 m^2.

5.2.2. Methodology

5.2.2.1. Permissible Site Discharge

To ensure that new developments do not increase peak stormwater flows in any downstream area during major storms up to and including the 100 year ARI event, the stormwater discharged from a developed site must be the same as the stormwater discharged from the area in it's natural state. The peak stormwater flow from the site in its natural state therefore becomes the Permissible Site Discharge (PSD) for the developed site. This applies to storms of all duration, not just the "critical event". This statement has been made because it is not always the storm event that produces the largest peak flow that requires the largest volume of detention.

Note that matching predevelopment peak flows and post development peak flows may not be an adequate policy where there are existing flooding and drainage problems. In such a situation, a retrospective policy that requires detention volumes over and above that required to match predevelopment and post development peak flows may be required. This principle has been applied in some instances in Goulburn in the past.

The pre-development peak stormwater flows were determined by developing a RAFTS model to simulate the natural conditions on site. A typical 1-hectare rural catchment with an average slope of 7% was set up and run for a number of 100 year ARI storm events for 10 minute, 20 minute, 30 minute, 1 hour, 2 hour, and 3 hour duration's. The results are included in Table 8 below.

These pre-development peak flows form the benchmark against which to limit the post-development peak flows. Based on these flows a PSD was selected to ensure that the peak flow rates for the post-development scenario did not exceed the pre-development flow for each of the storm durations.

An orifice is to be fitted to the discharge point of the OSD tank or storage basin to control the amount of stormwater discharged from the site. This is sized to limit the stormwater discharge to the PSD. The size of the orifice is based on the depth of the storage. The orifice and pipe sizing formula that was used in the modelling is recommended for future adoption in the OSD policy (detailed below).

Orifice / Pipe Sizing:

$$A_o = Q / C_d \, (2g * H)^{0.5}$$

Where:

A_o = cross sectional area of orifice (m^2)
Q = Permissible Site Discharge = 0.215 (m^3/s/Hectare)
C_d = discharge coefficient (0.62 for sharp edged orifice)
g = acceleration due to gravity (9.8m/s^2)
H = head of water (m) above the orifice
(measured from the centre line of the orifice to the max storage level)

5.2.2.2. Site Storage Requirement

The Site Storage Requirement (SSR) is the volume of water to be detained to meet the PSD. The SSR is sized to contain the difference between the 100 year pre-development and post-development peak stormwater flows in all durations. This SSR is collected in storage located either above or below ground.

Three different storage configurations were tested to determine the sensitivity of the SSR to different storage configurations. A RAFTS model was set up for a typical 1-hectare industrial lot with site slope of 2% and impervious areas as per Section 2.4. A stage / discharge relationship was determined for each of the two following scenarios:

- storage tank with a depth of 1.0 m.
- storage tank with a depth of 0.5m.

5.2.3. Results

The peak flow rates in a 100 year ARI for pre-development and post-development are presented below in Table 8.

Table 8 – Results for the Pre and Post Development Scenarios

	Pre-development	Post-development Tank Storage (1m depth)		Post-development Tank Storage (0.5m depth)	
Storm Duration (minutes)	Peak Flow, Q (m^3/s)	Peak Flow, Q (m^3/s)	Basin Storage (m^3)	Peak Flow, Q (m^3/s)	Basin Storage (m^3)
10	0.193	0.192	111.1	0.192	111.5
20	0.271	0.212	135.9	0.212	136.4
30	0.252	0.203	124.5	0.203	125.0
60	0.260	0.200	121.6	0.201	122.1
120	0.238	0.188	107.1	0.188	107.5
180	0.218	0.181	99.2	0.181	99.5
360	0.142	0.134	54.1	0.134	54.4

The critical storm event (the event that requires the largest volume of storage) was found to be for the 20 minute storm duration. The 10 minute storm event however governed the determination of the PSD as can be seen from Table 8. Based on this critical storm event a PSD for the Goulburn area was determined (by trial and error) to be 0.215 m^3/s/hectare or 215 L/s/hectare. This PSD goes hand in hand with an SSR of 140m^3/hectare for industrial land.

The 1m deep storage scenario generated a maximum storage volume of 135.9m^3 and the 0.5 deep storage scenario generated a maximum storage volume of 136.4m^3. Adding a small margin of safety for other storage configurations lead to the recommendation that an SSR of 140m^3/hectare be adopted.

A third scenario was also tested for the sake of certainty. This was for the case of a 2m deep storage. The results were consistent with the previous two scenarios giving confidence that the PSD and SSR selected will achieve the desired objective irrespective of the storage configuration.

There is one exception to this case and that is the use of a high early discharge pit – which would enable a marginally smaller storage to be adopted. However high early discharge pits are not always possible to construct, having moving parts that tend not to be maintained. It is concluded that the extra cost of a marginally larger storage may outweigh the benefit of the marginally reduced storage that could be achieved with the use of a high early discharge pit.

In conclusion, if a PSD of 215 L/s/hectare together with an SSR of 140 m^3/hectare is adopted then the peak flows for storms of all duration will be reduced to equal to or less than their pre-development level (for the development scenario noted in section 5.2.1). Further refinement of the SSR and PSD requirements for different development scenarios will be undertaken as part of the Ducks Lane precinct modelling.

Note that work undertaken assumed that no overflow from the storage is allowed to occur in any storm event up to and including the 100 year ARI storm event.

It is strongly recommended that Council require a failsafe system to be put in place. That is, a defined overland flow path must be created to ensure that in events larger than the 100 year ARI storm event or in the case that when the orifice blocks that water can flow unimpeded out of the development.

It is also recommended that Council requires that orifice plates are to be tack welded into place to ensure that developers can not tamper and remove the orifice plate.

It is also recommended that the OSD storage have a restrictive covenant placed over it to ensure that Council has some legal recourse should the developer decide to partially prevent flow from entering the tank/storage etc or even to alter the system.

Council also needs to be conscious of the fact that roof and property drainage systems are not designed to convey the 100 year ARI flow. This means that overland flow will occur as gutter systems overflow in large events. Council therefore needs to ensure that all overland flow is to be directed into the OSD storage or it will not function as intended.

It is strongly recommended that Council take advantage of the relative ease of fitting "water quality" controls into OSD systems. For example a "maximesh" screen placed over the orifice plate will protect the orifice from blocking and also provide a water quality benefit. Achieving nutrient removal inside the OSD tank may also be possible through the use of filtration and a silt trap prior to the outlet. Water quality is addressed in detail within Chapter 7 of this report.

6.0 TRUNK DRAINAGE CORRIDOR DEVELOPMENT

6.1. BACKGROUND

A basic flood corridor assessment has been undertaken to determine minimum corridor widths to allow flood flows to be safely conveyed without the risk of damage to property or person.

The flood assessment has assumed that highly vegetated, relatively narrow, low maintenance channels would convey flood flows. The channel configuration used a typical base width of 10m, bank batters of 1 in 6, and depths of flow up to 1.5m depending on where the channel is located.

Currently, the flow regime east of Common Street can be described as broad, low, sheet flow. West of Common Street the flow regime changes with some of the channels becoming more incised closer to Mulwaree River with steeper banks, whilst others maintain broader channels.

Under the Rivers and Foreshores Improvement Act the Department of Infrastructure Planning and Natural Resources (DIPNR) requires a 40m set back from creeks that have defined beds and banks. This requirement will need to be assessed by Council to ensure riparian integrity under this Act.

6.2. FLOOD MODELLING USING RAFTS

6.2.1. Methodology

A desired level of flood immunity of 1 in 100 years Annual Recurrence Interval (ARI) has been assumed in accordance with Australian Rainfall and Runoff 1987 (AR&R).

A RAFTS computer model was constructed to estimate the peak flows generated from the site for the 100 year ARI storm event. This information was then used to define the corridor width. This assessment did not consider backwater effects from the Mulwaree River - it only considered flood flows generated from the Common Street precinct.

The corridor widths presented are based on the current development scenario and are therefore pre-development flows. The requirement for on-site detention and retention on all new developments, as documented in Chapter 5 will ensure that post-development flows match pre-development flows from the area.

The drainage sub-catchments were established by reviewing the existing site contours and locating a collection node along the creek line at the outlet of each sub-catchment. This is described in the attached plan entitled Stormwater Catchment Plan (drawing L268-P01).

Conservative loss rates of 1.5mm for impervious and 5mm for pervious areas were adopted in the RAFTS model. Impervious areas within the sub-catchment ranged from 5 – 30% depending on the current catchment development scenario.

A typical cross section with a base of 10m and 1 in 6 batters was used for the trunk drainage corridors throughout the precinct. It is known that the current creek profile does differ from this profile in some areas, particularly in the lower reaches of the creek system. We however recommend that the typical cross section used be adopted in the redevelopment (actual corridor base widths are shown in Table 9).

The velocity depth (vd) multiples were generally kept to below 1.0 except at confluence points of major trunk drainage lines and within the flood plain of the Mulwaree River. The Manning's 'n' adopted for these creeks was 0.15, which allows for mass planting to occur for low channel maintenance. This would also satisfy riparian corridor requirements.

The maximum flow from each node was established by taking the highest peak flow generated from a range of storm durations (20, 30, 60, 120, 180, 360, and 720 minutes). We then used Manning's Equation to determine the depth of flow, total width of flow and flow velocity at each node. From these Manning's Equation results, the velocity-depth ratio was calculated and the total corridor width determined by rounding up and adding a buffer of 10m to both sides of the total width of flow. This additional buffer of 10m was recommended by Goulburn Council to allow for bank stability and also to ensure a level of conservatism for the trunk drainage corridor. **This does not mean that flood planning levels do not need to be determined.** Council will need to ensure that adequate freeboard is allowed for in the design of floor levels adjacent to trunk drainage corridors.

6.2.2. Results

The results for each node are shown below in Table 9. Refer to both the Stormwater Catchment Plan (drawing L268-P01) and the Trunk Drainage Management Plan (drawing L268-P02) for presentation of relevant data.

Some nodes will be under the influence of backwater effects from Mulwaree River. This has not been accounted for in the modelling.

Table 9 - Rafts 100 year ARI flows and Manning's Equation Results

Link	Max Flow [m3/s]	Depth of Flow [m]	Width of Flow [m]	Flow Velocity [m/s]	Velocity Depth Ratio	Total Corridor Width [m]	Corridor Base Width [m]
1-1 - O1	56.943	1.268	45.22	1.19	1.514	66.00	35
1-2 - 1-1	18.982	1.134	23.61	1.00	1.130	44.00	10
1-3 - 1-2	18.747	1.040	22.48	1.11	1.154	43.00	10
1-4 - 1-3	18.368	1.125	23.50	0.97	1.097	44.00	10
1-5 - 1-4	17.661	0.990	21.88	1.12	1.108	42.00	10
1-6 - 1-5	16.504	1.333	25.99	0.69	0.917	46.00	10
1-7 - 1-6	13.695	1.005	22.06	0.85	0.854	43.00	10
1-8 - 1-7	8.791	0.833	20.00	0.70	0.586	40.00	10
1-9 - 1-8	5.560	0.516	16.20	0.82	0.424	37.00	10
1-10 - 1-7	3.192	0.378	14.54	0.69	0.260	35.00	10
2-1 -1-1	39.773	1.550	38.60	0.88	1.357	59.00	20
2-2 - 2-1	39.704	1.356	36.27	1.04	1.411	57.00	20
2-3 - 2-2	22.426	1.225	24.70	1.06	1.293	45.00	10
2-4 - 2-3	1.928	0.313	13.75	0.52	0.162	34.00	10
2-5 - 2-4	1.309	0.196	12.35	0.60	0.117	33.00	10
2-6 - 2-5	0.284	0.067	10.80	0.41	0.027	31.00	10
3-1 - 2-3	19.210	1.338	26.06	0.80	1.066	47.00	10
3-2 - 3-1	18.725	1.205	24.46	0.90	1.087	45.00	10
3-3 - 3-2	17.079	1.066	22.79	0.98	1.042	43.00	10
3-4 - 3-3	14.305	0.828	19.94	1.15	0.956	40.00	10
3-5 - 3-4	3.931	0.388	14.66	0.82	0.319	35.00	10
3-6 - 3-4	2.896	0.338	14.06	0.71	0.241	35.00	10
4-1 - 2-2	14.885	1.019	22.23	0.91	0.942	43.00	10
4-2 - 4-1	13.665	0.926	21.11	0.95	0.878	42.00	10
4-3 - 4-2	12.738	0.907	20.89	0.91	0.825	41.00	10
4-4 - 4-3	11.656	0.750	19.00	1.07	0.804	39.00	10
4-5 - 4-4	2.046	0.223	12.68	0.81	0.180	33.00	10
4-6 - 4-4	1.539	0.226	12.71	0.60	0.136	33.00	10
5-1 - O2	7.592	0.620	17.44	0.89	0.553	38.00	10
5-2 - 5-1	6.988	0.583	16.99	0.89	0.518	37.00	10
5-3 - 5-2	4.590	0.475	15.70	0.75	0.357	36.00	10
6-1 - O3	35.373	1.117	33.41	1.19	1.325	54.00	20
6-2 - 6-1	24.085	1.130	23.56	1.27	1.435	44.00	10
6-3 - 6-2	23.549	1.226	24.71	1.11	1.357	45.00	10
6-4 - 6-3	14.322	0.867	20.40	1.09	0.942	41.00	10
6-5 - 6-4	13.627	0.925	21.10	0.95	0.876	42.00	10
6-6 - 6-5	8.757	0.636	17.63	1.00	0.634	38.00	10
6-7 - 6-3	7.465	0.557	16.69	1.00	0.559	37.00	10
6-8 - 6-7	6.963	0.536	16.44	0.98	0.527	37.00	10
6-9 - 6-8	1.999	0.301	13.61	0.56	0.169	34.00	10
7-1 - 6-1	9.340	0.814	19.76	0.77	0.628	40.00	10
7-2 - 7-1	6.707	0.661	17.94	0.73	0.480	38.00	10
7-3 - 7-2	5.221	0.607	17.29	0.63	0.383	38.00	10

* Creek cross section has been altered from the typical arrangement.

Table 9 allows for the establishment of land take requirements for flood conveyance through the estate.

A contingency of 10m either side has been added to the total width of flow to provide for bank stability and a level of conservatism. The minimum widths required for the trunk drainage corridor in the upper catchments range between 31 and 46m. This reflects the relative steepness and the size of the catchments. In these sections the depth of flow, velocities and vd's in a 100-year ARI storm event are less than 1.0. Lower down in the catchment the corridor width become much wider (44-66m) and the depth of flow, velocities and vd's increase, but are generally below 1.5.

The trunk drainage corridors located adjacent to the Mulwarree River and within the 100-year ARI flood extents have an altered cross section to replicate natural river cross sections in the area. The amended cross sections also helped to maintain the depth of flow and velocities below the nominated thresholds above in these sections of the creek.

The installation of site controls such as on-site detention and reuse will reduce the post-development peak flows to pre-development peak flows. These combined controls will act to closely mimic the current flooding regime and ensuring that there will be no requirement for further community-based detention within the Common Street Business Park.

6.2.3. Discussion

The RAFTS model provides a suitably conservative flood estimation tool. As such if the corridors proposed are adopted then all flows up to the 100 year ARI are estimated to be able to be conveyed safely within the proposed drainage and riparian corridors.

These corridors are to be densely vegetated to minimise Council's maintenance requirement whilst providing significant water quality improvements. Adoption of these corridor widths will also increase the land yield in the upper catchments. Localised flooding has been identified lower in the catchment. In these areas adoption of the recommended cross section and corridor widths will ensure that all flows will be contained within the drainage corridor in a 100-year event.

No assessment of the existing culvert capacity has been undertaken within this study. Prior to the formation of the trunk drainage corridors, an assessment of the culvert capacity downstream of the corridor will be required. Adequate controls to prevent scouring and bank destabilisation around culvert inlets and outlets will also require further investigation.

STORM understands from Council that the current minor drainage systems are likely to be replaced with a conventional "pit and pipe" system drainage system. STORM recommends that the current rural road system be retained and upgraded. The current system includes swales on both sides of the road with drop structures to allow stormwater from the road system into the creeks. Swales provide a significant benefit to water quality and have previously been recommended for residential areas within the Goulburn area. The swale system in the Common Street Business Parkt will need to be assessed and sized appropriately and any pipes, crossing points and discharge controls installed where required.

6.3. TRUNK DRAINAGE CORRIDOR ASSESSMENT

STORM has undertaken a brief assessment of the trunk drainage system based on a site inspection and photographs taken from accessible locations adjacent to the current creek system. This assessment was used to determine a suitable post-development creek cross section and indicative costs associated with the installation of trunk drainage corridor so that Council can include the upgrade works within it's Section 94 apportionment for the Common Street precinct. Further detailed investigation of the current creek system will be required prior to undertaking any specific works.

The trunk drainage corridor assessment has identified two types of work to be carried out by Council and developers.

That is:

1. There are significant areas within the upper reaches of the Common Street precinct that are currently "unformed" drainage corridors. These areas will require formation as a "naturalised" drainage system (trunk drainage corridor). This formation work is to be undertaken by developers when they develop their respective landholdings. The developers will need to fill adjacent to the channels and or excavate the channels to ensure that the lots are constructed above the estimated 100 year ARI water levels. This work does not benefit or arise from development elsewhere in the catchment and so a nexus between the Section 94 and the work required can not be demonstrated. By way of further explanation, the channels could be left as they are today, and then revegetated for stability purposes.

   Vegetating the whole channel system on the other hand would provide a common water quality and environmental benefit that should therefore be funded under Section 94 works. Further, as developers would be undertaking earthworks to develop their subdivisions they would be in the best position to determine how to construct the requisite channel shape.

   In summary the earthworks and revegetation works associated with the trunk drainage system are to be developer funded.

   By constructing channels with the same geometric and vegetative characteristics as those recommended in Table 9 and the installation of on-site controls recommended in Chapter 5, the 100 year ARI flows could be contained wholly within the channel. Council would also need to ensure that adequate freeboard is allowed for in the final channel designs.

Plate 1 – Revegetation Opportunity

Plate 2 - Broad Drainage Corridor (with drop structure shown)

2. The lower sections of the creek system will also require works to accommodate the proposed creek cross sections and widths. There is observed evidence of erosion, bank instability and foreign materials dumped in the creek. These areas require removal of any dumped materials, stabilisation by revegetating the creek corridor and in some cases very minor regrading. Should development proceed it is recommended that these creeks be regraded and revegetated to fit the proposed cross section and corridor widths. The revegetation is to be undertaken by Council immediately after any channel formation works have been undertaken by potential developers. In the interim, it is recommended that there be a detailed creek assessment undertaken to identify works to be undertaken both in the short and long term to ensure stormwater conveyance and stream stability.

Plate 3 – Evidence of Erosion

Plate 4 – Evidence of Erosion & Instability

6.4. COST ESTIMATES

Based on the recommendations above the following information is provided so Council is able to apportion Section 94 contributions for the site.

Area	275,702 m^2
Rate	\$10 per m^2
Cost Estimate	**\$2,757,020**

Vegetation Management Plans will also be required for the creeks at an estimated cost of approximately \$30,000.

Based on a developable area of 105.4 Ha, the total cost per hectare of development is **$26,445**.

7.0 WATER QUALITY MANAGEMENT

7.1. JUSTIFICATION

The SCA requires that the Common Street precinct achieve a neutral or beneficial effect on the water quality leaving the site. In order to assess the water quality leaving the site a pre-development and post-development water quality model has been constructed.

7.2. LIMITATIONS OF WATER QUALITY MODELLING

Water quality modelling relies on a multitude of factors. There is a lack of calibrated data available within Australia and in the absence of calibrated data the best available information is used. This places limits on the accuracy of water quality modelling.

Water quality modelling is generally load based and to a lesser extent process based.

The water quality model adopted by STORM for this project is the MUSIC water quality model. The first addition of the model has some faults. However we accept those faults as the model is considered the best planning tool available at the current time.

7.3. MODELLING – MUSIC

As noted above pre and post development models were created in MUSIC.

The pre-development model represents the current site development - it is not the natural state environment. It therefore includes forested and agricultural areas with the impervious areas estimated using areal photography.

In developing the post-development models, STORM made an adjustment for nitrogen load rates from the roof catchments. Nitrogen values could be reduced significantly as atmospheric deposition of nitrogen contributes a small amount to the total nitrogen load within a catchment. The MUSIC reference manual does not state a nitrogen load rate from roofs; it does however quote a rate for both phosphorus and total suspended solids. STORM determined the phosphorus reduction achieved between roof catchments and typical industrial catchments and reduced the typical nitrogen values within MUSIC by the same amount.

The effect of rainwater tanks was entered into the model using a sedimentation pond node. We estimated the differences in flow that would arise from the use of rainwater tanks by using the results from the analysis of reuse potential from an industrial catchment undertaken in Chapter 5. The reuse potential was quoted as a per hectare rate and multiplied by the developable area within a particular sub-catchment.

STORM developed six post-development scenarios:

1. No controls;
2. Reuse only;
3. At source controls using Stormfilters®;
4. End of pipe control using a constructed wetland
5. At source controls using Stormfilters® together with a wetland at half the size used in scenario 4

The first scenario was developed to so as to be able to determine the percentage retained by comparing any proposed controls with a do nothing scenario.

The “reuse only” (scenario 2) included reuse at source as a control at a rate 20kL/ha. This option reduced the amount of stormwater leaving the developed site and provided some sedimentation benefit.

The “at source” (scenario 3) included both reuse and Stormfilters® (manufactured by Ingal Environmental Services) as part of the on-site detention system. A generic node developed by Ingal Environmental Services using studied Stormfilter® performance was included in the model to simulate the effect of the system.

The “end of pipe” (scenario 4) included reuse on-site and a wetland located at the end of the southern creek system. 1^{st} pass at wetland sizing of 2% of the catchment area resulted in a surface area of 7.3ha, permanent pool depth of 0.4m, volume of 29,240m^3 and an outlet pipe diameter of 375mm. This scenario was cross checked with the sizing recommendations in the Constructed Wetlands Manual (DLWC, 1998) which determined the wetland surface area required as 5.5ha. For the purposes of this study the wetland area adopted is 6.4ha.

The last scenario included reuse on-site together with the same at source controls used in scenario 3 and a wetland sized at half of scenario 4’s wetland.

The average areal potential evapotranspiration for the Goulburn area is 1200mm. This equates to an annual amount of 76.7ML/yr lost from the surface area wetland which has been incorporated into the model as a loss from the end of pipe scenario wetland node.

The water quality benefit derived from the revegetated and stabilised creek systems was not accounted for. Nor was the grass swales found as part of the current road system included. It is assumed that swales would be incorporated as part of the post-development system, therefore the pre and post-development road system would have the same quality benefit. If this is not the case, then the pollutant loads in the post-development scenarios will significantly higher.

Load rates from previous MUSIC models generated for Mary's Mount compared the outputs found with other benchmarks from "Strategic Land and Water Capability Assessments" for SCA and Sydney data derived for the Clean Waterways Program. It was found that the results from the MUSIC model were between the benchmark values and can be considered as reasonable. As the same meteorological base was used for the Common Street model, we are confident that the results obtained for this area will give a good representation of actual loads.

7.4. RESULTS

The load rates generated by the MUSIC models for the different scenarios are presented below in Table 10. Table 11 interprets these loads and presents the percentage retained

Table 10 – MUSIC Pollutant Loads from Total Catchment

Scenario	**Total Suspended Solids** (kg/yr)	**Total Phosphorus** (kg/yr)	**Total Nitrogen** (kg/yr)
0 Pre-Development (Current Development)	191,000	460	3,230
1 Post-Development (No Controls)	283,000	552	3,990
2 Post – Development (Reuse Only)	248,000	540	3,300
3 Post – Development (At Source)	190,000	428	3,020
4 Post – Development (End of Pipe)	92,500	272	2,360
5 Post – Development (At source + 0.5 End of Pipe)	78,600	248	2,360

It can be seen from Table 10 that all post-development scenarios demonstrate a reduction of pollutant loads compared with the no controls scenario. However, only the "at source" and "end of pipe" scenarios will satisfy SCA's requirements ie achieve a neutral or beneficial effect on the water quality leaving the site. The actual percentage retained through the use of water quality controls together with Council's recommended retention percentages is presented in Table 11.

Table 11 – Percentage Retained of Pollutant Load

Scenario	**Total Suspended Solids** (% retained)	**Total Phosphorus** (% retained)	**Total Nitrogen** (% retained)
2 Post – Development (Reuse Only)	12.4	2.2	17.3
3 Post – Development (At Source)	32.9	22.5	24.3
4 Post – Development (End of Pipe)	67.3	50.7	40.8
5 Post – Development (At source + 0.5 End of Pipe)	72.2	55.1	40.9
GCC SMP Retention Rates	50	45	45
Retention Rates to satisfy SCA (SEPP58)	32.5	16.7	19.0

There is a significant discrepancy between the retention rates specified in council's SMP and the SCA's requirement for SEPP58. The SMP is a guide for new developments whereas the SCA's conditions are based on protecting the quality of Sydney's drinking water. It is suggested that the SCA requirements be met as a minimum for this development.

It should also be noted that no roadside swales were included as part of the model. If swales were incorporated into the road profile to convey stormwater, then higher retention rates are expected.

Both the "at source" and "end of pipe" post-development scenarios satisfy SCA's requirements and resulted in significantly higher retention percentages compared to the reuse only scenario. Overall the 5th scenario which included both at source and end of pipe controls resulted in the best water quality outcome.

7.5. WETLAND ANALYSIS

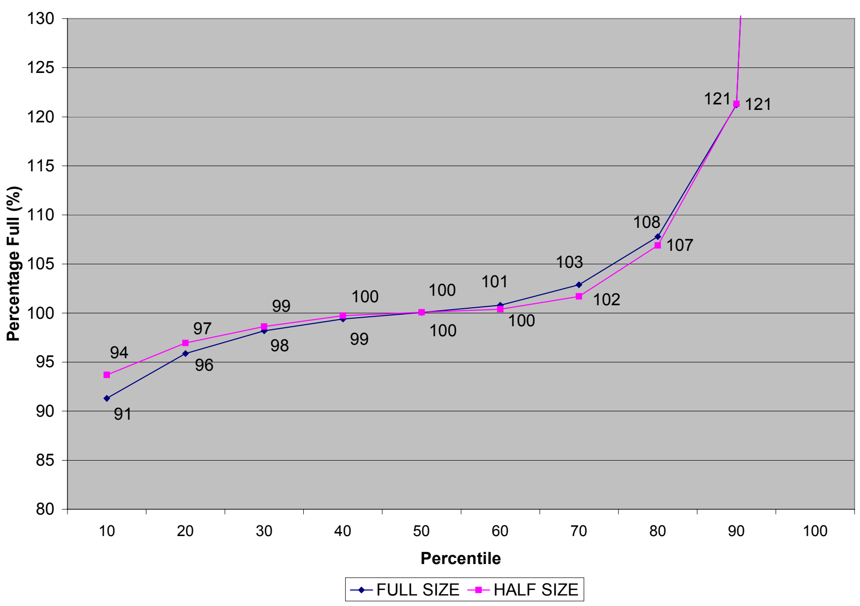
An analysis of the wetland was undertaken to establish the storage behaviour on long term basis. The flux file generated in MUSIC was inserted into excel and the percentile water depths and corresponding volumes were determined (Table 12). This analysis only incorporates evapotranspiration as a loss, it does not account for infiltration or other potential losses from the system. Should this option be pursued a more detailed water balance will be required.

Table 12 – Wetland Storage Behaviour

	Full Size Wetland (6.4 Ha)		Half Size Wetland (3.2 Ha)	
Percentile	Water Depth (m)	Volume (m^3)	Water Depth (m)	Volume (m^3)
10	0.3652	23355.6	0.3748	11984.2
20	0.3835	24526.0	0.3878	12399.9
30	0.3928	25117.5	0.3945	12615.4
40	0.3976	25427.1	0.3989	12755.1
50	0.4002	25596.3	0.4003	12799.9
60	0.4032	25783.9	0.4016	12839.9
70	0.4115	26316.7	0.4068	13007.8
80	0.4311	27570.1	0.4276	13671.9
90	0.4849	31009.5	0.4853	15517.5
100	1.1750	75144.8	1.1760	37602.6

It can be seen from Table 13 above that for the full sized wetland, 90% of the time the expected water depth is 0.3652m and the volume is 23,356m^3. This means that 90% of the time (10^{th} percentile) the wetland is 91% full. Figure 3 shows the wetland fill rates for the range of percentiles. Where as the wetland at half the size (and at source controls) has an expected water depth of 0.3748m and volume of 11,984m^3 90% of the time (or 94% full).

Figure 3 – Wetland Probabilistic Storage Behaviour



7.6. COST ESTIMATES

Based on the scenarios above the following information is provided so Council is able to apportion Section 94 contributions for the site.

At Source - Stormfilters®

Supply & Installation Stormfilters® cartridges and pre-cast vault to service 6 hectares each

Quantity	17.6
Rate	$25,000
Cost estimate	**$439,166**

Therefore the cost per hectare of development based on 105.4 developable hectares is $4,185.

This estimate does not include maintenance – this however will be undertaken by the owner/occupier.

End of Pipe - Wetlands

	Full Size	Half size
Quantity	63,950 m^2	31,975 m^2
Rate	$75 per m^2	$75 per m^2
Cost estimate	**$4,796,250**	**$2,398,125**

This figure does not include maintenance – this will have to be undertaken by Council. Based on an analysis of wetlands within Hornsby Shire Council the average cost to maintain wetlands within their Council area was approximately $250/ha of catchment/year. The total catchment feeding this wetland is approximately 365.5ha which equates to an annual maintenance cost of approximately $91,375 if using Hornsby Shire Council rates.

The costs given for constructing a wetland are based on a greenfield site. Rehabilitation of an existing system is expected to be significantly less as there is likely to be less earthworks.

7.7. DISCUSSION

7.7.1. General

It can be seen from the data presented in Section 7.4 that both the “at source” and “end of pipe” water quality management solutions would result in reduction of current pollutant loads leaving the Common Street precinct and satisfy SCA requirements.

7.7.2. End of Pipe Control

The “end of pipe” scenarios resulted in significantly higher pollutant retention than the other scenarios analysed. Critical to the effectiveness of a wetland is it’s design, water level, and maintenance. STORM recommends using the Constructed Wetlands Manual (DLWC, 1998) to design the wetland size and configuration should a wetland be pursued.

The installation of a wetland would be the most costly option for this site. The initial construction cost would be in the order of $4.7 million with an annual maintenance cost of approximately $90,000.

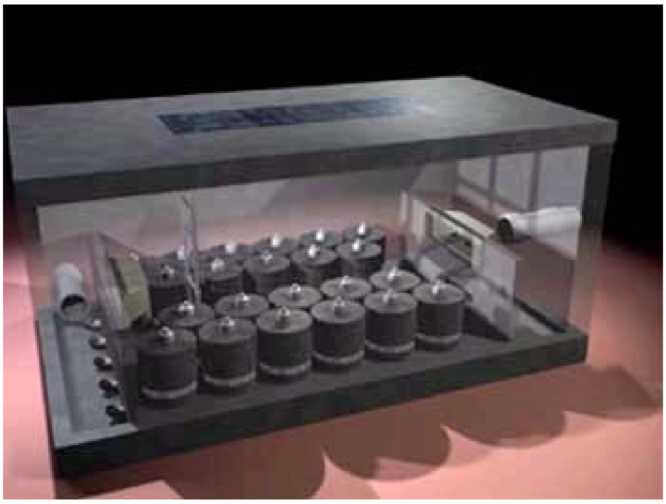
The suitability of an “end of pipe” solution only is questionable in that the pollutants generated on the development are allowed to be transported through the rehabilitated streams before being retained in the downstream wetland. From catchment management principles it is more appropriate to trap these pollutants as close to the source as possible. This will allow protection of the downstream environment from the effects of those pollutants. There is also a catchment in the north of the development area that does not feed into the proposed wetland location and therefore will not retain the pollutants as required.

An existing semi natural wetland occupies the nominated proposed location of a constructed wetland. There may be value in modifying this site to improve its pollutant trapping ability to augment “at source” controls. A functioning wetland system will also provide habitat value for the area We do not believe that Council can collect Section 94 contributions for this system as the implementation of at source controls alone meet SCA water quality requirements. This may be investigated further by council but is considered out of the scope of this study.

7.7.3. At Source Control

The “at source” scenario included reuse and Stormfilters® sourced through Ingal Environmental Services. Stormfilters® is a flow-through stormwater filtration system consisting of a tank that houses rechargeable cartridges filled with a variety of filter media such as gravel and perlite. These cartridges trap particulates and absorb pollutants such as dissolved metals, nutrients, and hydrocarbons. These devices can be installed on each lot as an additional chamber to the on-site detention system. This reduces Council’s maintenance commitment, however Council will need to install rigorous controls to ensure that owners properly maintain the device. Also recommended is the installation of Enviropods if the area includes above ground on-site detention. STORM has received a quote from Ingal to supply and install a Stormfilters® sized to treat a 6 hectare catchment. The total cost was in the order of $25,000 which has been used as the basis for calculating the costs for the installation of these devices throughout the Common Street Business Park Precinct. This information is to give Council an understanding of the typical cost to install these devices so as to be able to compare at source and of pipe control adequately.

Figure 4 – Stormfilter® by Ingal Environmental Services



The installation of these devices at source are consistent with treatment train and water sensitive principles. It also has the added benefit that if one system fails the whole system is not jeopardised, unlike the end of pipe scenario. At source treatment systems are also more easily isolated should any individual lot have a spill.

8.0 RAINWATER REUSE INVESTIGATION

8.1. BACKGROUND AND SCOPE

With the knowledge that industrial estates are associated with the construction of large impervious areas leading to significant changes in the volume of runoff that would occur post development together with Goulburn's current water shortages it was proposed to investigate the reuse potential of the excess stormwater to leave the industrial estate. STORM has therefore undertaken an investigation into the feasibility of capturing rainfall runoff that is in excess of the water to be reused on site. The investigation was to determine appropriate uses for the water, to determine an approach and approximate costs of storing, treating and delivering the water.

Storm both attended the site and liased with Council to identify possible users of the water. A number of options for capturing the excess runoff were identified as well as transport and more permanent storage options assessed.

8.2. METHODOLOGY

Each option involved capturing the runoff from the larger of the two major catchments in the Common Street DCP area. This is shown in Figure 3. Each option involved using a new storage as a permanent store for the water. We did consider using the disused quarry but now understand that this is not be feasible. The costs for this option have never the less been included in the final report as a record of the work undertaken.

As an alternative to the quarry, a new storage of 5,000 m^3 volume would need to replace the quarry storage. The storage may also be located in the area identified for the proposed 2 ML storage shown in Figure 3. This area is between the two creeks adjacent to the road, out of the floodplain and in an area identified as drainage reserve. The estimated cost for constructing the two storages is in fact cheaper than pumping to the quarry though the costs of distributing back to the cemetery would be higher as the distance is longer.

The options are described in detail below but essentially involve diverting the water from the creek to the storage or wetland (if there is to be one). A low flow structure would need to be put in place to ensure that environmental flows would continue to flow into the Mulwaree as they would prior to development. Each Option can be described as follows:

Table 13 - Stormwater Reuse Options

Option	Description
1	Is to capture water in an off-line 2 ML storage. To then pump, using a low flow rate pump, to the disused quarry for long term storage. The route of the rising main is to the railway line, along the side of the corridor, under Sydney Road inside the rail tunnel and then along the Sydney Road to the disused quarry.
2	Is to capture the treated stormwater after filtration in a wetland that may be required for water quality purposes anyway. Then to pump this water to the disused quarry for longer term storage. A similar route for the rising main was adopted for this option as a least cost route.
3	Is to capture the stormwater in a small storage (100 m^3) and use a high flow pump to deliver the water to the new storage. The route was the same as for Option 1.
4	Is to capture the water in the 2 ML storage leaving the storage empty for capture of runoff from the next storm event and to convey this water into a new 5 ML storage for distribution from that location.
A	Is to pump from the new storage to the cemetery for irrigation of the cemetery. This distance (and the cost) may also be similar to the distance required to pump to the May Street Wetland for distribution to the Golf course.
B	Is to pump across the river with the rising main attached to the road bridge as shown in Figure 3. Delivery and end use was intended for the gaol for non potable purposes including toilet flushing and irrigation. Costs would be marginally lower for a storage located where the new 2 ML storage is located rather than from the disused quarry as shown in the plan as the distance is smaller.

A low flow pump was assumed to be a pump that delivered 5 L/s and a high flow pump one that delivered at 20 L/s.

Cemetery water consumption was metered and Council provided STORM with the data summarised below:

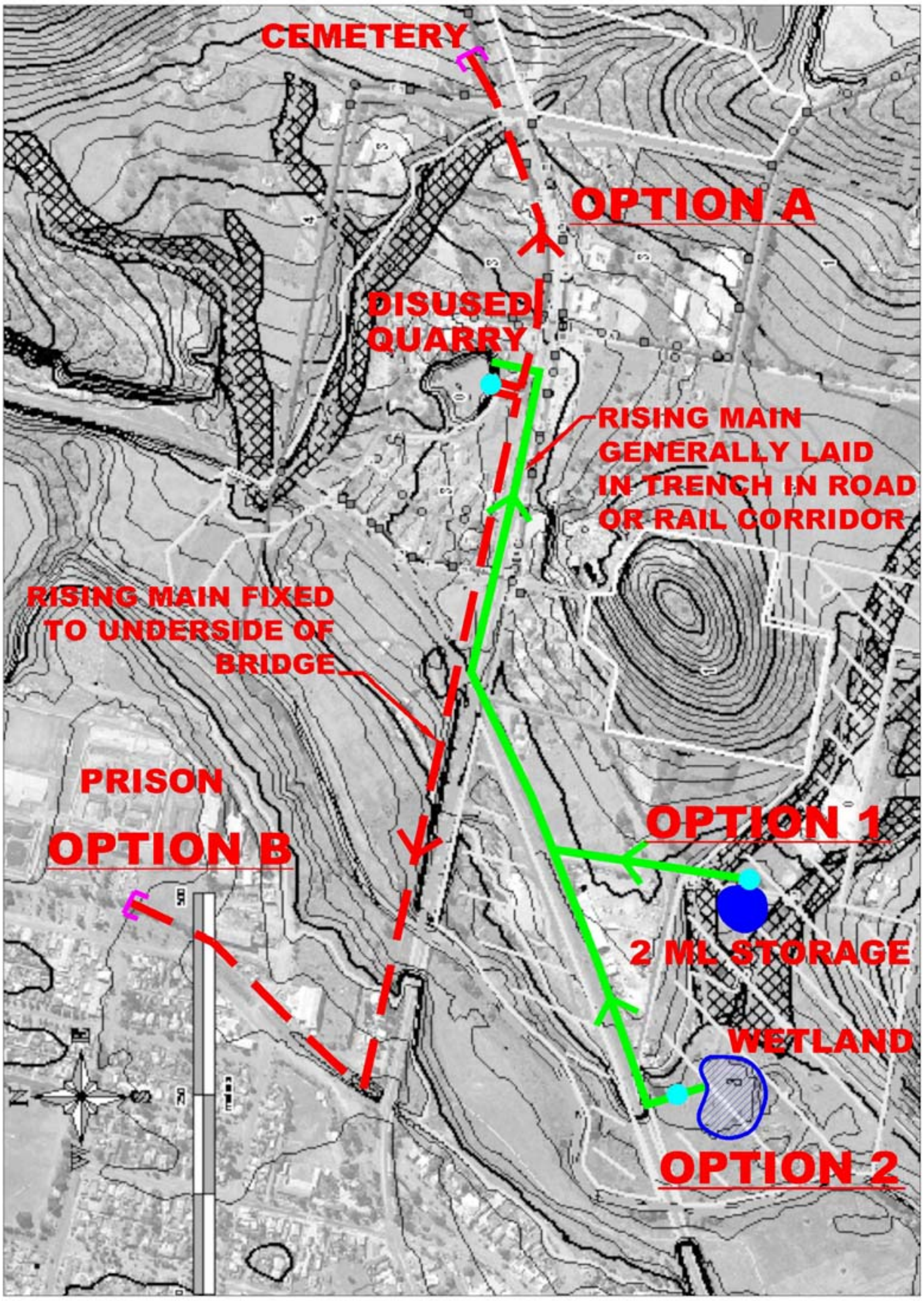
Table 14 - Average Metered Water consumption for the Cemetery

Season	Average Daily Consumption (kL/day)
Summer	30.35
Autumn	11.13
Winter	7.5*
Spring	11.69

* amplified slightly for the sake of ease of calculations

Analysis of the cemetery consumption showed that little consumption occurred during the latest drought in response to water restrictions. Thus it is assumed that irrigation of the cemetery is not essential.

Figure 5 - Water Reuse – Storage and Delivery Options Assessment



LEGEND

Rising main to Quarry

Rising main to end use

LINE TO TERMINATE AT BREAK TANK OR BRANCH OUT TO MEET LOCAL NEEDS

PUMP STATION

8.3. COSTS ESTIMATES

A basic cost estimate for each of the options was undertaken and is summarised in Table 15 below.

Table 15 - Cost Estimates for Options 1,2,3, 4, A and B

Item	Description	Quantity	Unit	Rate	Total
	Option 1: 2 ML storage to quarry				
1.1	Supply & install 75 mm dia PE pipe	900	m	55	$ 49,500
1.2	Restoration of road surfaces	200	m^2	100	$ 20,000
1.3	Supply & install 2 x GL55 (240V) submersible pumps	2	each	2000	$ 4,000
1.4	Construct 1.2m dia PE wet well	1	LS	5000	$ 5,000
1.5	Construct 2 ML storage	1	LS	6000	$ 9,000
	Option 1 Total				**$ 87,500**
	Option 2: wetland to quarry				
2.1	Supply & install 160 mm dia PE pipe	970	m	80	$ 77,600
2.2	Restoration of road surfaces	200	m^2	100	$ 20,000
2.3	Supply & install 2 x NP3127 (3 phase) submersible pumps	2	each	7000	$ 14,000
2.4	Construct 1.8m dia concrete wet well	1	LS	12000	$ 12,000
	Option 2 Total				**$ 123,600**
	Option 3: small storage to quarry				
3.1	Supply & install 160 mm dia PE pipe	900	m	80	$ 72,000
3.2	Restoration of road surfaces	200	m^2	100	$ 20,000
3.3	Supply & install 2 x NP3127 (3 phase) submersible pumps	2	each	7000	$ 14,000
3.4	Construct 1.8m dia concrete wet well	1	LS	12000	$ 12,000
3.5	Construct 100 m^3 storage	1	LS	2,000	$ 2,000
	Option 3 Total				**$ 120,000**
	Option 4: 2 dams 2 ML and 5 ML				
	Construct 2 ML earth dam storage				$ 9,000
	Construct separate 5 ML earth storage dam				$ 27,000
	Option 4 Total				**$ 36,000**
	Delivery Side				
	Option A: Quarry to cemetery				
A.1	Supply & install 63 mm dia PE pipe	550	m	50	$ 27,500
A.2	Restoration of road surfaces	150	m^2	100	$ 15,000
A.3	Supply & install 2 x GL55 (240V) submersible pumps	2	each	2000	$ 4,000
A.4	Construct 1.2m dia PE wet well	1	LS	5000	$ 5,000
	Sub Total				**$ 51,500**
	Option B: Quarry to prison				
B.1	Supply & install 63 mm dia PE pipe	1300	m	50	$ 65,000
B.2	Restoration of road surfaces	200	m^2	100	$ 20,000
B.3	Supply & install 2 x GL55 (240V) submersible pumps	0	eac	2000	$ -

			h		
B.4	Construct 1.2m dia PE wet well	0	LS	5000	$ -
	Sub Total				**$ 85,000**
	Option A&B Total				**$ 136,500**

These costs have been prepared on the basis of a highly simplistic “strategic analysis” which has not considered design and constraint issues to any extent. These costs should not be relied on as a decision making tool to help decide if further analysis is warranted. A feasibility study, the next stage of investigation, if undertaken, would be needed to ascertain appropriate cost estimates. Notwithstanding what has been said above we have endeavored to provide conservative cost estimates at all times.

8.4. WATER BALANCE ASSUMPTIONS AND RESULTS

A water balance model was constructed by STORM to assess the yield and bypass of each option.

8.4.1. Assumptions

The water balance included the following assumptions:

- The catchment to be included in the model was only to be the new developed area of industrial land estimated to be about 33.8 hectares. This catchment was selected, as it is likely to give the greatest yields.

- The end use of water was initially assumed to be for the cemetery only. The end use was determined by actual metered consumption rates from the cemetery shown Table 14. We found that the cemetery consumes water when there is supply available and vice versa. We derived average seasonal uses for inclusion in the water balance. In this report, meaningful reuse is defined as reuse that can provide some kind of payback even if over 20 years.

- Next the cemetery water use was multiplied by a factor to determine how much we could supply if we had the potential use.

- The water balance was based on 27 years of daily rainfall for Goulburn up to December 2002.

8.4.2. Results

The water balance results include:

- The result of this was that the cemetery did not consume enough water to allow for any meaningful reuse.

- At least three times the cemetery consumption could be supplied with irrigation quality water at 100% certainty of supply. Obviously we could supply a greater volume of water with less certainty of supply.

- The cost of irrigating the cemetery on average is about $3,300 per year. The infrastructure payback would be about 60 years and not considered acceptable. Further analysis on this scale of reuse was therefore not undertaken as it appeared unviable from an economic view point.

8.5. CONCLUSIONS

The conclusions below are for options 1,2 3 and 4 (Small Scale Reuse)

The feasibility assessment of harvesting stormwater runoff, over and above the non-potable needs of the industrial estate itself, enables the following conclusions to be drawn:

- Where the proposed end use is low (less than say 100 kL/day) and located close to a roof area, the cost of storage and pumping from the industrial development would exceed the cost of constructing a storage and pumping system at the end use. For example, the cost of constructing a large rain tank and pump system at the gaol would be in the order of $10,000 to $20,000. The cost of providing the infrastructure to store and pump this water from the Common Street Business Park would be in excess of $175,000.

- Where the proposed end use is not located close to a roof area, for example the cemetery where it would not be possible to harvest large quantities of runoff even if a dam was constructed it would cost in the order of $175,000 to construct a storage and delivery system. Given that the water in these instances is water that is not used for essential purposes, the benefit is questionable. It would instead be far more economical to install rain tanks and small pump systems, at much lower cost, on a number of existing buildings – such as the prison or Police Academy.

- It should also be noted that whilst water that is not used for essential purposes such as the cemetery irrigation provides little apparent advantage – that water is used never the less used during times when supply is available. At that point use would deplete the available supply. That is, not using the water supply at all for non essential purposes would actually leave more water in the storages for use for essential purposes.

- Council should further investigate the opportunity to construct 2 separate storages as per Option 4. The stormwater could then be distributed to the golf course for reuse. This option has not been further investigated at this stage and is beyond the scope of this project. This is likely however to be the most promising option as it overcomes the problems having a treated effluent store located in the floodplain.

9.0 RECOMMENDATIONS

9.1. ON SITE CONTROLS

Each industrial development should submit a “water management plan” at DA stage which addresses water quality, the on site retention (water reuse) and on-site detention policy to be developed by Council. This water management plan (WMP) should:

- nominate the type of development and therefore the expected water usage.
- nominate where rainwater (treated or non-treated) can be used in process or production.
- calculate the sizing and locate both the rain tank and the OSD storage and configuration for the site. The rain tank is to be sized on a requirement of 20 kL/hectare and the OSD based on a PSD of $0.215m^3$/s/hectare (215L/s/hectare) and an SSR of $140m^3$/hectare.
- identify on site water quality controls that are to be put in place whether inside the OSD system or not.

STORM recommends that the Stormfilters® be installed as the quality management option at source.

Control standards for minimum outlet size, ponding depths, safety fences, and internal drainage systems from the Upper Parramatta River Catchment Trust’s On-site Detention Handbook are recommended.

It is recommended that Council develop a formal and clear policy based on the recommendations and work undertaken and documented in this report.

9.2. TRUNK DRAINAGE

In order to safely convey the 1 in 100 year storm event the corridor widths in nominated in Table 9 need to be adopted throughout the Common Street Business Park Precinct. In addition to these widths, a typical creek cross section with 10m base and 1 in 6 batters are to be adopted throughout the creek system unless otherwise nominated in Table 9.

It is recommended that these corridors are to be densely vegetated to minimise Council’s maintenance requirement whilst providing significant water quality improvements.

An assessment of the culverts and associate drop structures will be required prior to any creek cross section alterations to ensure these structures are able to convey the stormwater appropriately.

When development proceeds it is recommended that these creeks be regraded and revegetated to fit the proposed cross section and corridor widths. The revegetation is to be undertaken by Council immediately after any channel formation works have been undertaken by potential developers.

STORM recommended that there be a detailed creek assessment undertaken and Stream Management Guidelines developed to identify works to be undertaken both in the short and long term to ensure stormwater conveyance and stream stability. It is also recommended that a Vegetation Management Plan be prepared for the trunk drainage in Common Street.

9.3. WATER QUALITY

STORM recommends that the Stormfilters® be installed as the quality management option at source.

Council may pursue construction of a smaller wetland adjacent to the Mulwaree River to enhance the pollutant retention capability of the existing semi natural wetland.

9.4. CENTRALISED STORMWATER REUSE

It is recommended that Council consider the potential for grant funding to construct the proposed reuse system.

A small scale reuse system however appears to have much greater economic cost than benefit. The benefits of the centralised system appear to become positive when the scale of the proposed system is increased.

An option involving potential reuse at the golf course was not investigated but is likely to provide the most environmental benefit at the least cost. This option is recommended for further investigation.

10.0 REFERENCES

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