RYGATE AND WEST

RESIDENTIAL SUBDIVISION
DOLPHIN POINT, STAGES 2 & 3
WATER MANAGEMENT STRATEGY

Issue No. 3
SEPTEMBER 2006

Patterson Britton
& Partners Pty Ltd
consulting engineers

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1 INTRODUCTION

1.1 BACKGROUND

Patterson Britton and Partners (*PBP*) were engaged by Rygate and West to prepare a water management report for Stages 2 and 3 of the proposed residential development site at Dolphin Point.

This report is in support of the proposed subdivision application for the subject site. It is understood the application would be determined by the Department of Planning (*DOP*) under Part 3A of the Environmental Planning and Assessment Act.

It is understood that this report will be utilised for the preparation of the Environmental Assessment (*EA*) for the site. The requirements of the EA include meeting Council's current guidelines along with compliance with various other relevant planning policies.

1.2 SITE DESCRIPTION

The site has a total area of approximately 28 hectares and is bounded by the Princes Highway to the north and forested areas to the south, west and east. The site has been a rural holding and is generally undulating with pockets of remnant vegetation located along the existing creeklines.

The site is moderately graded and drains generally toward the existing dam on the site. The integrity of the embankment forming the dam is not covered in this report. An investigation into this would be necessary along with an appropriate spillway design. There are two creeklines and two overland flowpaths on the site. The creeks have been designated Creek 1 and Creek 2, while the overland flow paths have been designated as OL 1 and OL 2. Both the creeks and overland flow paths are shown on **Figure 1**.

The proposed development has been broken into three stages with Stage 1 now completed. Stage 1 covers an area of approximately 8 ha, whereas Stage 2 and Stage 3 are 15 hectares and 5 hectares respectively. The extents of stages are shown on **Figure 1**.

1.3 PROPOSED AND EXISTING DEVELOPMENT

The proposed development of Stages 2 and 3 consists of a residential subdivision with a total of 158 lots. Stage 1 of the development which has now been completed comprises 70 lots. The development would therefore have a total of 228 residential lots. The allotment boundaries and road layout has been provided by Rygate and West and is presented on **Figure 1**. The proposed lots would be distributed as follows:-

- Stage 1 – 70 residential lots (*existing*);
- Stage 2 - 135 residential lots (*proposed*); and
- Stage 3 - 23 residential lots (*proposed*).

1.4 BACKGROUND INFORMATION

A site inspection was carried out to determine the local conditions whilst 1:4000 orthophoto maps provided detail of the surrounding area. All relevant local government planning and engineering guidelines were obtained from Shoalhaven City Council.

Rygate and West have confirmed with Department of Natural Resources [(*DNR*)-formerly DIPNR] that OL 1 is not required to be maintained as an overland flow path, and as such could be piped. OL 2 has been identified by Shoalhaven Council for protection. However, it conveys a similar amount of flow as OL 1 and exhibits only minor environmental significance, and would therefore be piped.

A water quality report was prepared by Morse McVey and Associates in December 2002 (*attached as **Appendix A***), for Stage 1 of the development. Information provided in this report has been utilised in preparing the final water management strategy for the entire site.

1.5 ENVIRONMENTAL ASSESSMENT REQUIREMENTS

A preliminary application has been successfully lodged with the DOP and The Director General's requirements are incorporated in their letter dated 15th February 2006 including:

- Coastal Design Guidelines for NSW SEPP 71 and SEPP 65 in particular water efficiency;
- Drainage, Hydrological Regime and Flooding;
- Impacts on Water Quality and Sedimentation Control; and
- Impacts on Waterways and Estuary Management.

These issues are addressed in this report.

1.6 CERTIFICATION

The contents of this report are certified by Mark Tooker, who is a registered NPER engineer with the Institution of Engineers, to comply with the requirements of Shoalhaven City Council's *Development Control Plan 100* and *Engineering Design Specification D5*.

1.7 WATER MANAGEMENT APPROACH

Shoalhaven Council, Department of Natural Resources, and best management practice dictate that for the overall development:

- peak runoff flow rates from the development during storm events should not exceed existing values;
- average annual runoff volume after development should be minimised;
- average annual pollutant load in runoff following development should not exceed existing values; and
- industry best practice runoff water quality control measures should be implemented to achieve a minimum reduction in the annual pollutant load from the development

of 80% for suspended sediment and 45% for both total phosphorus (TP) and total nitrogen (TN).

In adherence to the above, PBP have incorporated the principles of Water Sensitive Urban Design (*WSUD*) and Ecologically Sustainable Development (*ESD*).

The development has therefore been designed with a water management strategy which incorporates stormwater detention *(to reduce localised peak runoff flow rates),* on-site retention/ reuse *(to mimic existing runoff volumes)* and pollutant removal devices *(to reduce pollutant load export).*

2 HYDROLOGICAL ANALYSIS

2.1 INTRODUCTION

The peak storm runoff rates generated by rainfall on the site were estimated using XP-RAFTS software.

RAFTS is a non-linear rainfall/runoff program developed by WP Software and can be used to estimate peak flows for catchments, using actual storm events, or design rainfall data derived from *Australian Rainfall and Runoff (AR&R)* (*IEAust, 1987*). All hydrologic analysis was undertaken in accordance with *AR&R*.

RAFTS was chosen for this investigation because it has the following attributes:

- it can account for spatial and temporal variation in storm rainfall across a catchment;
- it can be used to estimate discharge hydrographs at any location within the catchment;
- it can accommodate variations in catchment characteristics;
- it is able to route hydrographs though detention basins; and
- it has successfully been widely used across NSW.

Hydrologic analysis was undertaken for both the internal and external catchments that contribute runoff to the site. Internal analysis was completed to determine the detention volume required to allow peak post development flows to be maintained at or below peak pre development flows. Analysis of the external catchment hydrology was undertaken to estimate peak flows for each of the overland flow paths and creeks on the site.

The catchment and sub-catchments adopted for this investigation are presented on **Figure 2**. The catchment and sub-catchment parameters are presented in **Appendix B**.

2.2 INTERNAL CATCHMENT HYDROLOGY

Hydrologic analysis was carried out for Stages 2 and 3 of the development to facilitate the required stormwater detention calculations. The model was used to estimate design flows under both natural state and developed site conditions for the 100, 20, 5, and 1 year Average Recurrence Interval (*ARI*) events.

The following parameters were used in the RAFTS model:

	Pervious Areas	Impervious Areas
Initial Loss (IL)	25 mm	1.5 mm
Continuing Loss (CL)	2.5 mm	0 mm
Manning's n	0.035	0.015

2.2.1 Stormwater Detention

Council's *Shoalhaven Planning Policy No. 1 development Guidelines* and Councils *Subdivision Code* of *Development Control Plan 100* states that detention basins may be required where downstream stormwater drainage systems are inadequate and the cost of upgrading is excessive. This would include the existing culvert located on Dolphin Point Road.

It is also recognised that there is a wetland located downstream of the site and that significant impact on this must be avoided. Through the implementation of stormwater detention and retention measures the existing flow regime would be mimicked as closely as possible, therefore the existing downstream wetland is not expected to be significantly affected.

Council's requirements for on-site detention have not been formalised in the engineering specifications. However, they have indicated that it would be acceptable to reduce post-development flows to the pre-development levels for each of the 1, 5, 20 and 100 year average recurrence interval (*ARI*) storm events at the site boundary. This would ensure flows are maintained at their current level and hence avoid any adverse impacts on the downstream drainage system or wetland.

For the calculation of post-developed runoff, it was assumed that the development of the subdivision would result in approximately 55% impervious areas. This is greater than Council's minimum recommendations, which are described below:

Land Use	Area of Subject Site (m^2)	Percentage Impervious (Minimum Requirement)
Residential	119,350	40 %
Road Reserve	56,150	95 %
Recreation Areas	25,500	25 %
Total	*201,000*	*51 %*

Peak flows were derived for the 1, 5, 20 and 100 year ARI storm events for the site under pre- and post-development conditions. Storm durations of 30 minutes to 24 hours were simulated to determine the critical storm. It was determined that approximately **3,300m^3** of detention volume would be necessary to achieve control of peak flow rates for all storm events. **Table 2-1** presents the expected peak flows at the downstream boundary of the site for pre development conditions, post development conditions without detention and post development conditions with detention.

Table 2-1 Estimated Peak Flow Characteristics

Development Scenario	Average Recurrence Interval (ARI)			
	1yr	**5yr**	**20yr**	**100yr**
Pre development flow (m^3/s)	5.4	11.4	18.7	30.4
Critical storm (mins)	*720*	*270*	*120*	*120*
Post development flow (m^3/s)	5.5	13.0	22.3	33.2
Critical storm (mins)	*720*	*90*	*90*	*90*
Post development flow with detention (m^3/s)	5.4	10.6	17.9	29.3
Critical storm (mins)	*1440*	*270*	*120*	*120*
Detention Volume Required (m^3)	2,920	2,950	3,250	3,300

Table 2-1 shows that by providing approximately **3,300 m^3** of stormwater detention the post development peak flow rates would be expected to be maintained at or below the estimated pre development flow rates for all storms modelled.

It is proposed to implement the detention storage volume by providing extended detention as part of the proposed water quality control pond (*WQCP*) and bioretention basin. The proposed WQCP and bioretention basin are shown on **Figure 3**.

RAFTS model output files for the subject site under existing and post-developed conditions (*incorporating detention*) can be found in **Appendix B**.

2.2.1.1 Extended detention on WQCP

The proposed Water Quality Control Pond (*WQCP*) would primarily act as a stormwater quality treatment measure however, in addition to this primary function, extended detention of approximately 250mm over the area of the WQCP would be provided. The surface area of the WQCP has been estimated to be 12,500m^2 which would provide approximately **3,125m^3** of detention storage for the proposed development.

2.2.1.2 Extended Detention on Bioretention Basin

The proposed bioretention basin would primarily act as a stormwater quality treatment measure however, in addition to this primary function, extended detention of approximately 300mm on top of the bioretention basin would be provided. The detention volume on top of the bioretention basins has been estimated to be **285m^3**.

Extended detention on the WQCP and the bioretention basin would provide approximately **3,410m^3** of detention for the proposed development.

2.3 EXPECTED HYDROLOGICAL IMPACTS

The proposed development is expected to increase the amount of impervious area on the site to approximately 55% of the total site area. This increase in impervious area is expected to contribute to an increase in peak runoff during all storms up to and including the 1 in 100 year ARI event as presented in **Table 2-1**.

The expected increase in peak runoff rates would be managed by the implementation of stormwater detention on the site which is expected to allow post development peak flow rates to be maintained at or below pre development levels as shown in **Table 2-1**. The combination of rainwater tanks on each lot and the downstream water quality control pond would capture and reduce runoff rates from smaller rainfall events. Rainwater tanks would reduce the impact of development on the runoff regime by entirely capturing small rainfall events for reuse thereby reducing both the volume and frequency of runoff. Runoff from slightly larger rainfall events would be captured in the water quality control pond by implementing an appropriate outlet design. This would provide the opportunity for small to medium rainfall events to be captured thereby increasing the potential for infiltration and evaporation.

This combination of measures (*Rainwater tanks, water quality control pond outlet design, on site detention*) would ensure that the changes to the pre development hydrological regime are minimised and therefore would not significantly affect the downstream wetland or the downstream environment.

Maintenance of pre development peak runoff rates would comply with Council's Shoalhaven Planning Policy No. 1 and Subdivision Development Control Plan 100.

2.4 EXTERNAL CATCHMENT HYDROLOGY

Analysis of the external catchment hydrology was undertaken to estimate peak flows for each of the overland flow paths on the site. These estimations have been used to predict flood levels and extents for the overland flow paths that pass through the site.

The parameters as stated in **Section 2.2** were used in the RAFTS model.

2.4.1 Peak Flows

For the assessment of the overland flowpaths (*OL*) and creeks traversing the site, contributing upstream catchments were analysed. XP-RAFTS software was used to estimate peak flows for the 20 and 100 year ARI storm events for flows traversing the site. Storms of 30 minute to 24 hour duration were simulated for these events.

The total catchment was divided into eight subcatchments to determine runoff reaching each of the flowpaths and creeks. The overland flow paths, creeks and adopted subcatchments are shown on **Figure 2**.

The estimated peak flow rates determined for the overland flow paths and creeks on the site are presented in **Table 2-2**.

Table 2-2 Expected Peak Flows for Creeks and Overland Flow Paths (m^3/s)

Flow Path	Average Recurrence Interval			
	1 year	5 year	20 Year	100 Year
OL1	0.8	2.0	3.6	5.7
Creek 1	2.1	4.3	7.1	11.0
OL 2	0.9	2.0	3.7	5.9
Creek 2	1.6	3.4	4.9	7.9

It should be noted that this proposal would not develop the upstream catchments and that stormwater detention would be provided on the site so that the existing flow regime is maintained. Therefore, it is expected that flows in the overland flow paths and creeks modelled would not vary significantly between pre developed and post developed conditions.

Appendix B contains RAFTS output data for the external subcatchment analysis.

2.4.2 Trunk Drainage

The stormwater drainage element of Shoalhaven City Council's Subdivision Code states that the piped drainage system must accommodate peak flows generated from the 5 year ARI event, while overland flowpaths are to be provided for the 100 year ARI event.

As previously stated, overland flow paths 1 and 2 would be piped. As such, it is estimated that the upstream catchment feeding to the upstream site boundary would contribute a peak flow of $2.0m^3/s$ in the 5 year ARI storm for each of the overland flowpaths. A trunk drainage line would be required to convey this flow through the site and discharge to the proposed water quality control ponds as shown on **Figure 3**. The proposed roadway would be utilised to convey any overland flows from events larger than the 5 year ARI up to the 100 year ARI event. Further discussion of overland flows is presented in **Section 3**.

3 HYDRAULIC ANALYSIS

The flood profile of the creeks through the subject site were modelled using HEC-RAS, River Analysis System. HEC-RAS is a software package which allows modelling of one-dimensional flow in steady and unsteady state modes.

HEC-RAS was chosen for this investigation because it has the following attributes:

- it allows gradually varied flow along a flowpath;
- it produces graphical and tabular results of input data and water surface elevation calculations;
- it allows the user to determine water surface elevations at any location along the flowpath; and
- it is internationally recognized as the leading one-dimensional hydraulic modelling software.

Flows derived using RAFTS, as detailed in **Section 2**, were used to estimate the hydraulic behaviour during the 20 and 100 year ARI storm events.

3.1 EXISTING CONDITIONS

Under existing conditions, flows from upstream catchments enter the site at four points along the site boundary. Flows traverse the site via two existing creeks and two existing overland flow paths. Creek 1 and OL 1 naturally drain to the existing dam while Creek 2 and OL 2 discharge approximately 150m downstream of the existing dam. From the downstream site boundary, runoff flows via a natural creek line to Burrill Lake.

The minor flows from both overland flow paths on the site would be piped and provision included for major flows in the form of constructed overland flow paths.

Creeks 1 and 2 convey flows from two large external catchments through the site. Both of these creeks have been designated as significant by DNR and would remain.

An analysis of the existing creek lines has been completed using cross sections developed from survey data obtained by Rygate and West. The location of the existing dam was also taken from the survey data.

Normal depth was selected for the upstream boundary conditions, based on the natural surface slope at those locations. The downstream tailwater condition for Creek 1 was taken to be RL 3.0mAHD as the permanent water level of the existing dam. Normal depth was selected as the downstream tailwater condition for Creek 2. A representative Manning’s n value of 0.06 was adopted for overbank areas and 0.03 was adopted for channel flow.

A sensitivity analysis was completed to assess the degree of change in flood level due to changes in Manning's n (*roughness co-efficient*) values. Manning's n values from 0.05 to 0.1 for overbank areas and channel values from 0.03 to 0.05 where trialled. It was determined that altering the Manning's n values produced a maximum variance in modelled water surface elevation of approximately 50mm. This maximum variance was achieved by combining high Manning's n values of 0.1 for the overbank areas and 0.06 for the channel. Therefore, the existing creek line has been deemed to be relatively insensitive to changes in Manning's n.

The Manning's n values of 0.06 for overbank areas and 0.03 for channel flow were deemed acceptable for the hydraulic analysis of Creek 1 and Creek 2. Furthermore, it is recognised that Council's freeboard requirements would be able to absorb any variation in the adopted Manning's n values.

The resultant water surface profiles for both creeklines are included in **Appendix C**. **Figure 5** shows expected flood extents across the site for both the 100 and 20 year ARI events.

3.2 POST-DEVELOPMENT CONDITIONS

As previously mentioned, minor flows from both overland flowpaths on the site would be piped. As required by Council, minor flows up to the 5 year ARI flow would be piped whilst constructed overland flow paths would be provided to convey major flows up to the peak 100 year ARI flow. **Table 3-1** presents the flows that would be catered for by each system.

Table 3-1 Expected Peak Flows in Overland Flow Paths

Overland Flow Path	Flow (m^3/s)	
	Piped	Constructed Overland flow path
	(5yr ARI)	(100yr ARI)
1	2.0	3.7
2	2.0	3.9

3.2.1 Overland Flow Path 1

It is proposed to construct a roadway west of the existing overland flow path 1, as shown on **Figure 3**. A trunk drainage line would convey minor flows (*up to the 5 year ARI*), while the roadway would act as the major overland flowpath for events exceeding the 5 year ARI up to the 100 year ARI event. **Table 3-1** shows that approximately 2.0 m^3/s would be piped and 3.7m^3/s would be conveyed in the proposed roadway profile during the 100 year ARI storm event.

The proposed roadway would have a reserve width of 16m and a carriageway width of 12m. The grade of the road would be approximately 6.5%.

Manning's calculations predicted that the proposed roadway when combined with the proposed piped trunk drainage system would have sufficient capacity to convey the 100 year ARI event flows. Calculations also show that the velocity depth product would be approximately 0.3 m^2/s during the 100 year ARI event which is acceptable for safe pedestrian access.

3.2.2 Overland Flow Path 2

The proposed sub-division road located adjacent to the southern boundary of the site would covey major overland flow to the east, it would then be conveyed along the eastern perimeter road and Creek 2 to the bioretention basin, as shown on **Figure 3**. The proposed trunk drainage system would be designed to convey the 5 year ARI flow (*approximately 2.0*m^3/s) and the proposed road would convey flows exceeding the 5 year ARI flow up to the 100 year ARI flows (*approximately 3.9*m^3/s).

The current landform would not permit runoff to naturally flow towards the east. Therefore road regrading would be undertaken to enable the southern boundary roadway (*and other parallel roads*) to convey flows to the eastern perimeter road. The approximated area of regrading required is shown on **Figure 3**.

The proposed roadway would have a reserve width of 20m and a carriageway width of 12m. The maximum and minimum grades of the road would be approximately 8.0% and 1.0% respectively.

Manning's calculations have shown that the overland flow path as described would have capacity to convey 3.9m^3/s. Calculations also show that the estimated maximum velocity depth product would be approximately 0.3m^2/s during the 100 year ARI event which is acceptable for safe pedestrian access.

3.2.3 Creeks 1 and 2

Figure 5 shows flood extents across the site after development for the 20 and 100 year ARI events for Creek 1 and Creek 2. Water surface profiles for the floodways through the site are included in **Appendix C**.

Figure 5 shows that the majority of the proposed development would be flood free during the 100 year ARI event. A culvert would be constructed at the upstream boundary of the site where Creek 1 passes under the proposed boundary road. This culvert would have capacity to convey the 100 year ARI event as required by Council.

During the 100 year ARI event it is expected that Creek 2 would spill onto the adjacent boundary road. It is expected that runoff would pond to a maximum depth of approximately 250mm during the 100 year ARI event. The hazard (*depth x velocity*) is satisfactory for safe access and there are alternative routes for egress. This minor inundation would not affect the egress of occupants of nearby lots and the potential risk to residents is therefore not considered to be significant.

3.3 FLOODING

The site is relatively steep in nature and the nearest waterbody is Burrill Lake, approximately 1km to the north. It is therefore anticipated that there will be no regional flooding effects, and hence local rainfall events will govern flood levels across the site.

Analysis of the existing creeklines on the site has resulted in the expected flood extents presented on **Figure 5** which shows that all of the proposed lots would be flood free during the 20 and 100 year ARI storm events.

3.4 FLOOR LEVELS

The Council's Subdivision Code states that habitable floor levels require a freeboard above the 100 year ARI flood level of 500mm in floodways. Therefore proposed floor levels should be referenced against expected water surface levels presented in **Appendix C**.

The 100 year ARI flood levels presented in **Appendix C** could be utilised by Council to set suitable flood planning levels for the site.

4 RUNOFF WATER QUALITY

4.1 WATER QUALITY TARGETS

The *Stormwater Quality Management* element of Council's Subdivision Code states that there is to be no net increase in pollutant loads from the site. Implementation of rainwater tanks is also mentioned as being desirable, where appropriate.

The objective of the proposed water management strategy is to implement sufficient measures on the site to maintain existing annual pollutant loads and to also meet the DEC's minimum treatment requirements, which are:-

Pollutant	**% Reduction**
Suspended solids	80
Total Phosphorus	45
Total Nitrogen	45

In addition, the Director General's requirements include:-

- potential impacts on the quality on surface and ground water be addressed;
- consistency with any relevant statement of joint intent established by the Healthy Rivers Commission;
- the proposal is acceptable in terms of the achievement or protection of the river flow objectives and water quality objectives;
- the proposal take into account and complement Shoalhaven City Council's draft integrated water cycle management plan;
- an assessment of the accumulative impact on Burrill Lake be undertaken; and
- details of pollution controls be provided for both during and after construction.

4.2 MUSIC WATER QUALITY MODEL SET-UP

A long-term MUSIC model was established for the Dolphin Point site to assess the potential water quality impact of the proposed development. The model was used to estimate the annual pollutant load that would be generated under both existing and developed conditions.

MUSIC is a continual-run conceptual water quality assessment model developed by the Cooperative Research Centre for Catchment Hydrology (*CRCCH*). MUSIC can be used to estimate the long-term annual average stormwater volume generated by a catchment as well as the expected pollutant loads. MUSIC is able to conceptually simulate the performance of a group of stormwater treatment measures (*treatment train*) to assess whether a proposed water quality strategy is able to meet specified water quality objectives.

To undertake the water quality assessment component of the Stormwater Management Plan, a long-term MUSIC model was established for the proposed subdivision site. The model was used to estimate the annual pollutant load generated under existing state and developed conditions over a 5 year period of a range of rainfall years, including two below average, one average and one above average.

MUSIC was chosen for this investigation because it has the following attributes:

- it can account for the temporal variation in storm rainfall throughout the year;
- modelling steps can be as low as 6 minutes to allow accurate modelling of treatment devices;
- it can model a range of treatment devices;
- it can be used to estimate pollutant loads at any location within the catchment; and
- it is based on logical and accepted algorithms.

~~3.1~~4.2.1 Subcatchment Characteristics

The site has been divided into stages and ~~precincts~~ subcatchments as shown on **Figures ~~XX~~1** and **2**. The total impervious percentage ~~of~~ adopted for each subcatchment is shown below in **Table 4-1**. These impervious percentages are based on an assessment of the anticipated proportion of hard surfaces such as roads, roofs and paved areas across the proposed development.

Table 4-1 Adopted areas and imperviousness

Catchment	Area *(ha)*	Impervious Percentage Undeveloped *(%)*	Impervious Percentage Developed *(%)*
A	5.9	5	55
B	6.6	5	55
C	6.9	5	55
D	4.4	5	55

In addition to the subcatchments contained on the site there are also external catchments that contribute runoff to the site. External catchments which contribute runoff to the proposed water quality control pond have been included to assess their impact on the treatment efficiency of the pond. The locations of the external catchments are shown on **Figure 2**.

The properties adopted for these external catchments are presented in **Table 4-2**.

Table 4-2 Properties of external catchments

Catchment	Area (Ha)	Imperviousness (%)
E	5.7	5
F	25.0	5
G	5.0	5
H	22.8	5

4.2.2 Rainfall

In order to develop a model that could comprehensively assess the performance of water quality treatment devices such as bioretention systems, the use of 6 minute interval pluviograph data was considered necessary.

The nearest pluviometer (*i.e. 6 minute interval data instrument*) station to the site is located at Nowra RAN. The long term average annual rainfall for this rain station as provided by the Bureau of Meteorology is 1135mm. Based on the available data from the nearest rain gauges, it is considered that an annual average of approximately 1150mm is reasonable for the site.

For this study, the pluviograph record from -Nowra RAN from 1st January 1969 through until 31st December 1973 was selected for the MUSIC modelling because this period had an annual average rainfall of 1198~~7~~mm. This period contains wet and dry years as shown in **Table 4-34**. The selection of wet, dry and average years provides a more rigorous analysis of the treatment measures than an average year alone.

Table 4-3 Annual Rainfall Adopted

Year	Total Rainfall (mm)	Classification
1969	1587	Wet
1970	881	Dry
1971	1230	Average
1972	1063	Average
1973	1228	Average
Long Term Average	*1198*	

4.2.3 Evaporation

Monthly areal potential evapotranspiration values were obtained for the site from Bureau of Meteorology data and are shown in **Table 4-4**.

Table 4-4 Monthly Areal Potential Evapotranspiration

Month	Areal Potential Evapotranspiration (mm)
January	165
February	125
March	110
April	77
May	52
June	37
July	38
August	52
September	80
October	121
November	140
December	155
Annual Average	*1152*

The rainfall and evaporation data is represented graphically on **Figure 4-1**.

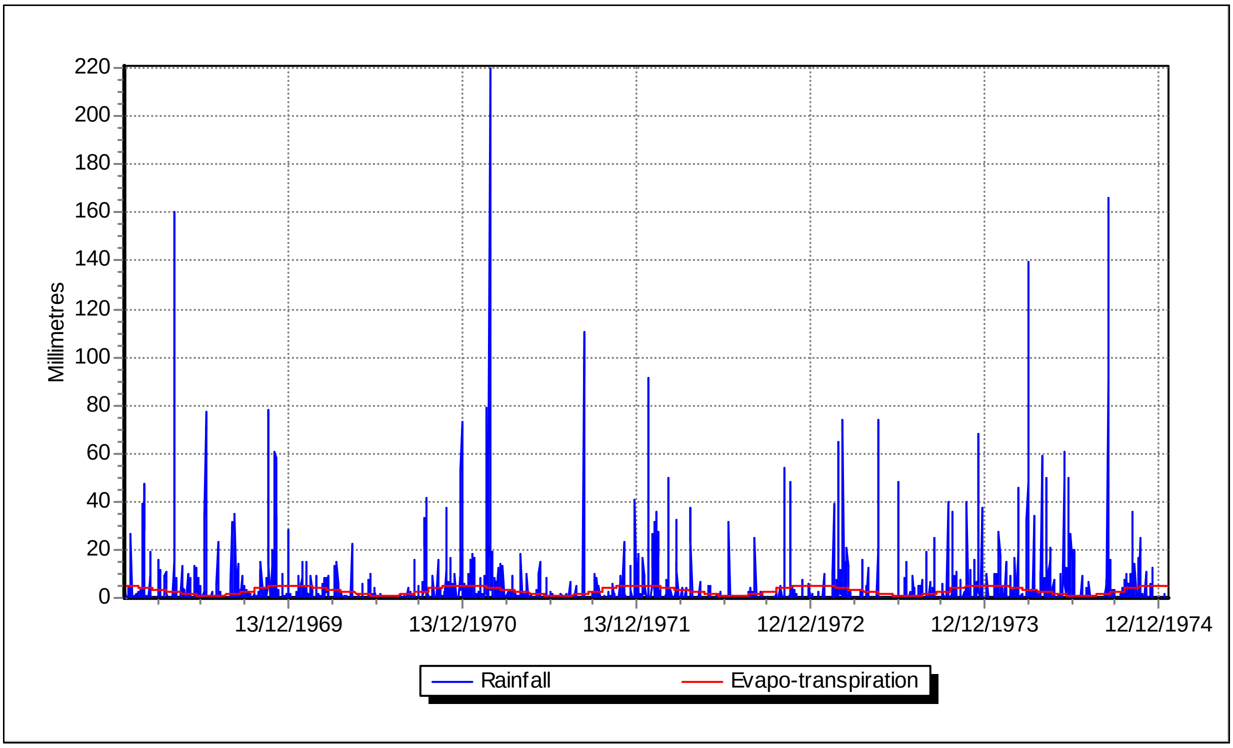

Figure 4-1 – Rainfall and Evaporation data

4.2.4 Soil Data and Model Calibration

A rainfall-runoff calibration was undertaken for existing site conditions. The default MUSIC values gave an existing state annual volumetric runoff coefficient of 0.33. This was considered appropriate for the site in its undeveloped state.

The default and adopted MUSIC rainfall run-off parameters along with the resulting run-off co-efficient are presented in **Table 4-5~~2~~**.

Table 4-5 Adopted rainfall run-off parameters

	Default Parameters
Impervious Area Properties	
Rainfall Threshold (mm/day)	1
Pervious Area Properties	
Soil Storage Capacity (mm)	120
Initial Storage (% of capacity)	30
Field Capacity (mm)	80
Infiltration Capacity Coefficient (a)	200
Infiltration Capacity Exponent (b)	1
Groundwater Properties	
Initial Depth (mm)	10
Daily Recharge Rate (%)	25
Daily Baseflow Rate (%)	5
Daily Deep Seepage Rate (%)	0
Runoff Co-efficient	
100% Pervious	0.33
55% Impervious	0.65

4.2.5 Pollutant Concentrations

Each catchment was divided into r~~R~~oofs and g~~G~~eneral u~~U~~rban areas to allow runoff from each area to be directed to specified treatment measures. For instance, runoff from roofs has been directed to rainwater tanks. The expected pollutant load from each catchment was determined by applying the pollutant concentrations or Event Mean Concentrations (*EMC's*).

The ~~applied~~ adopted EMC's for t~~T~~otal s~~S~~uspended s~~S~~olids (*TSS*), t~~T~~otal p~~P~~hosphorus (*TP*) and t~~T~~otal n~~N~~itrogen (*TN*) are given in **Table 4-6** and are sourced from the findings of a comprehensive review of stormwater quality in urban catchments undertaken by Duncan (*1999*) and adopted by the Department of Environment and Conservation (*DEC*) in

March 2004. Analysis by Duncan (*1999*) found event mean concentrations of TSS, TP and TN to be approximately log-normally distributed for a range of different urban land-uses.

Table 4-6 Adopted Pollutant Concentrations

Land Use	TSS	TP	TN
	mg/L	*mg/L*	*mg/L*
Rural	90	0.22	2.0
Roofs	20	0.13	2.0
General Urban	140	0.25	2.0

4.3 PROPOSED POST CONSTRUCTION TREATMENT STRATEGY

The stormwater management strategy to be implemented on the site would incorporate best practice water sensitive urban design (*WSUD*) measures. The water quality aspect of this strategy would include measures such as gross pollutant traps (*GPT*'s), a water quality control pond near the outlet of the catchment, rainwater tanks, and bio-retention basins.

As previously mentioned, water quality management for Stage 1 of the development was completed by Morse McVey and Associates. A copy of their report is attached in **Appendix A**. Stage 1 of the development utilised the following treatment strategy:-

- Sediment basins;
- Bio-retention swales; and
- Water quality control pond/wetland.

Details of these measures are included in their report attached as **Appendix A**.

Stages 2 and 3 of the development would adopt the following treatment measures:-

- Rainwater tanks;
- Gross pollutant traps;
- Water quality control pond; and
- Bio-retention basin.

The combination of the proposed treatment measures for Stage 1 and Stages 2 and 3 would create a framework of best management practices to achieve the treatment targets for the site.

The proposed measures for Stages 2 and 3 are described in more detail in the following sections.

4.3.1 Rainwater Tanks

Each dwelling is proposed to have a minimum 4,000L rainwater tank that will capture the stormwater collected on the roof. This water will be made available for re-use for toilet flushing, clothes washing and external irrigation. An average roof area of 200 m^2 per lot was assumed for modelling purposes.

The following daily consumption rates were used for modelling the rainwater tank re-use systems:

Toilet flushing	44 L/person/day
Clothes washing	40 L/person/day
Garden watering	72.5 L/person/day

Australian Bureau of Statistics Census data for 2001 indicates that the average household size for the Shoalhaven area is 2.5 persons. Hence, the consumption rates were multiplied by 2.5 for each tenement.

A summary of the proposed rainwater tank design is as follows:

- A minimum $4m^3$ volume rainwater tank designed to collect the majority of roof runoff and store it for irrigation, clothes washing and toilet flushing purposes would be installed for each of the dwellings on the site;
- The tanks are to incorporate a first flush device, inspection/cleanout hatch and cleanout valve;
- The tanks are to incorporate an outlet tap for connection to an irrigation system driven by the tank head (*if possible*);
- All tank overflow should be directed to the formal piped stormwater drainage system (*i.e. overflow to the street drainage system*) to prevent nuisance flooding;
- All rainwater tanks should be installed and maintained so as to prevent cross connection with the potable water supply;
- A "*topping up*" device (*from the potable water supply*) shall be provided to supplement roof runoff during periods of little rainfall or high water use.
- A *"backflow prevention device"* shall be installed;
- All rainwater services shall be clearly labelled "*Non Potable Water*" with appropriate hazard identification; and
- Pipe work used for rainwater services shall be coloured purple in accordance with AS1345. All valves and apertures shall be clearly and permanently labelled with safety signs to comply with AS 1319.

4.3.2 Gross Pollutant Traps

A Gross Pollutant Trap (*GPT*) captures litter, coarse sediment, some nutrients, oils and greases. While the pollutant capture efficiency of various traps may vary, the paper "Removal of Suspended Solids and Associated Pollutants by a Gross Pollutant Trap" (*Cooperative Research Centre for Catchment Hydrology, 1999*) suggests the following efficiencies for a CDS (*"Continuous Deflective Separation"*) unit.

- sediments up to 70%
- total phosphorous up to 30%
- total nitrogen up to 13%

These removal efficiencies have been adopted in the MUSIC model for all GPT's.

The proposed location of GPT's are shown on **Figure 3**.

~~4.1.3~~4.3.3 Water Quality Control Pond

A water quality control pond is a treatment measure that is generally used for the removal of suspended solids and ~~some~~ nutrients. Ponds generally use settlement and biological action to remove suspended solids and pollutants from stormwater runoff. An overflow weir is located at an elevation equal to the ~~the extended detention depth above the~~ permanent water level plus the extended detention depth.~~.~~

Apart from the water quality function of the water quality control ponds, they also provide a range of environmental benefits, extending from ecological functions, such as providing habitat, to aesthetic features, adding to the quality of life of the community.

Ponds are typically most effective when placed low in the catchment where a large proportion of runoff can be collected and treated.

It is proposed to upgrade the existing water storage dam on the site to a water quality control pond. ~~,~~ ~~t~~The ~~size and~~ size and approximate location of the pond is shown on **Figure 3**. The existing water storage dam has a surface area of approximately 12,500m^2. Remediation works to the water storage dam would include:

- planting suitable vegetation, including macrophytes;
- providing edge treatment to minimise mosquito habitat; and
- constructing suitable outlet/spillway.

The upgraded water storage dam would be designated WQCP 1 and treat runoff from the western portion of the site.

4.3.4 Bio-retention Basin

It is proposed that a bio-retention basin would be located downstream of OL 2 as shown on **Figure 3**. This would improve the quality of water discharging from the site, while also slowing down runoff and infiltrating low flows into the subsoil drainage media. This in combination with rainwater tanks and the water quality control pond will assist to mimic the natural pre-development frequency of runoff from the site.

The bio-retention basin required for the site would be 100m long and 11m wide.

The bio-retention systems will consist of the following:

- a subsoil drainage pipe system with a medium gravel surround;

- a transition layer of medium – coarse sand. The sizing of the sand and gravel is critical to prevent the overlying material including the bio-retention filter media from moving into the subsoil drain;
- bio-filter material consisting of sandy-loam;
- plantings generally consisting of native sedge plants along the central drainage medium;
- other vegetation such as shrubs and small trees outside central drainage medium; and
- a concrete overflow pit and pipe to accommodate trunk drainage and overflow requirements, where necessary.

A typical cross section through a bio-retention basin is provided on **Figure 4**.

The bioretention basin would allow stormwater to spill over a long (100m) weir to create a wide non erosive sheet flow.

4.4 WATER QUALITY MODELLING RESULTS

All stages of the development have been included in the modelling to allow accurate reporting of the performance of the proposed water management strategy for the site. Details of the treatment measures adopted for Stage 1 of the development have been extracted from the Morse McVey report attached as **Appendix A**.

4.4.1 Existing State Pollutant Export

The existing water storage dam on the site has been included in the existing state model, however, it has been assumed that the existing dam provides a role as a sedimentation basin only.

Table 4-7 presents the expected annual pollutant export for the site in its existing/undeveloped state.

Table 4-7 Existing State – Pollutant Export

	TSS	TP	TN
	kg/year	***kg/year***	***kg/year***
Without Treatment	16,631	44	336
With treatment (*existing water storage dam*)	14,305	42	329
% Reduction	14	5	2

The existing state pollutant export targets for the developed site as presented in **Table 4-7** are:-

- TSS – 14,305 kg/year
- TP – 42 kg/year
- TN – 329 kg/year

4.4.2 Developed (*No Treatment*) Pollutant Export

Table 4-8 presents the expected annual pollutant export for the site in its proposed developed state without water quality treatment.

The existing water storage dam has also been included in the developed state modelling. The pollutant removal efficiency of the existing dam has been assumed to be equal to that of a sedimentation basin.

Table 4-8 Annual Pollutant Export Loads – Developed State (*No Treatment*)

	TSS	TP	TN
	kg/year	***kg/year***	***kg/year***
Without Treatment	32,458	71	533
With treatment (*existing water storage dam*)	26,668	65	515
% Reduction	18	9	3

Comparison of **Table 4-7** and **Table 4-8** shows that a significant reduction in all pollutants is required to achieve existing state pollutant loads. Furthermore, to comply with DEC minimum treatment requirements the following reductions will be required:-

- TSS – 80% reduction (*5,334 kg/year*)
- TP – 45% reduction (*36 kg/year*)
- TN – 45% reduction (*283 kg/year*)

4.4.3 Developed (With Treatment) Pollutant Export

Table 4-9 presents the expected annual pollutant export for the site in its proposed developed state with the proposed water treatment strategy presented in **Section 4.3**.

Table 4-9 Annual Pollutant Export Loads – Developed State (*With Treatment*)

	TSS	TP	TN

	kg/year	kg/year	kg/year
Without Treatment	26,668	65	515
With Treatment	2,871	23	284
% Reduction	89	65	45

Table 4-9 shows that the water quality targets for the site would be met through the implementation of the proposed water treatment strategy. Furthermore, it is expected that pollutant export from the site would be significantly reduced when compared to the sites existing rate of pollutant export. The expected reductions when compared to the existing state are:-

- TSS – 80% reduction
- TP – 45% reduction
- TN – 14% reduction

These expected reductions would therefore lead to a long term improvement in receiving water quality.

4.5 MAINTENANCE OF WATER QUALITY CONTROL MEASURES

The proposed maintenance program for the sites water quality control measures would consist of the following:

- Periodic *(6 monthly)* inspection and removal of any gross pollutants & coarse sediment that is deposited in the water quality control pond and replacement of vegetation as necessary;
- Periodic *(3 monthly)* and episodic *(post storm greater than 1 yr ARI)* inspection and removal of trapped pollutants from all GPTs; and
- Periodic *(annually)* inspection (*and cleaning if required*) of rainwater tanks.

4.6 PROPOSED CONSTRUCTION PHASE WATER QUALITY TREATMENT

Figures 6 and **7** present conceptual erosion and sediment controls to be utilised on the site during construction. Erosion and sediment controls would be constructed and maintained in accordance with the Department of Housing's document "Managing Urban Stormwater – Soils and Construction", March 2004 (*otherwise known as the Blue Book*).

The Erosion and Sediment Control Plan (*ESCP*) has been prepared with the objective of minimising sediment movement off site and therefore minimising contamination of adjacent areas during the construction works.

The proposed erosion and sediment control measures are described in **Sections 4.7** and **4.8**.

4.7 SEDIMENT RETENTION PONDS

As the disturbed area will exceed 2,500 m^2, sediment retention ponds would be required during the construction phase.

Calculations to determine the required size of the sediment retention basins have been undertaken in accordance with the requirements of the NSW Department of Housing's publication Managing Urban Stormwater Soils and Construction, 2004 (*calculations are attached in* ***Appendix E).***

The proposed location of the sediment retention basins are presented on **Figure 6**.

Dispersive type soils occur on this site, therefore a Type D basin (*as described in the Blue Book*) is proposed. It is to be noted that the sediment basins would only be required during the earthworks and road construction stage of development. Upon completion of the roads the sediment ponds would no longer be required.

A summary of the sediment retention basin sizing calculations are outlined in the following sections.

4.7.1 Sediment Settling Zone

The sediment settling zone capacity for a Type D basin is based on a volume required to retain all runoff from a design storm event. In this instance the 80^{th} percentile, 5 day rainfall event was adopted as the design storm. It is to be noted that the Blue Book recommends the 75^{th} percentile design storm but because of the sensitive location of the site the 80^{th} percentile storm has been conservatively adopted.

Applying these criteria, the required sediment settling zone for the sediment basins is:

- Sediment Basin 1 – 1,050m^3; and
- Sediment Basin 2 – 350m^3.

4.7.2 Sediment Storage Zone

To determine the required sediment storage zone capacity, a calculation of the predicted soil loss was performed using the Revised Universal Soil Loss Equation (RUSLE) as described in the Blue Book. This calculation estimated that the 3 month soil loss will be approximately 63.1m^3. This calculation assumed that there will be no mulching of the surface after clearing and that the surface will be left compacted and smooth.

However, the Blue Book recommends that the sediment storage zone capacity for a Type D basin be not less than 30% of the settling zone volume

The adopted sediment storage zone volume for the basins will therefore be:

- Sediment Basin 1 – 315m^3; and
- Sediment Basin 2 – 105m^3.

4.7.3 Sediment Pond Volumes

These calculations result in the total minimum volume of the basins being:

- Sediment Basin 1 – 1,365m^3; and
- Sediment Basin 2 – 455m^3.

Indicative sizes of the basins are presented on **Figure 3**.

4.7.4 Outlet

Since the basins will be Type D, the captured water would be pumped out within a 5 day period following the rainfall after dosing with a chemical flocculant to achieve an acceptable turbidity level. Inflows exceeding the pond capacity would be discharged via an overflow weir and spillway. The weir and spillway will both be sized for the 20 year ARI design flow.

4.8 ADDITIONAL EROSION AND SEDIMENT CONTROLS

4.8.1 Stabilised Site Access

Site access/egress would be controlled through the designated site access points to reduce the likelihood of vehicles tracking soil onto public roads. The stabilised site access would be constructed of aggregate with nominal diameter of 30mm to a minimum depth of 200mm. Details of stabilised site access are presented on standard detail SD 6-14 in the Blue Book *(refer* **Figure 7***)*.

4.8.2 Diversion Drains

Clean runoff from areas upstream of the disturbed area would be diverted around the works area using diversion drains. Separate diversion drains would also be used to collect contaminated runoff and direct it to the sediment retention basin. **Figure 6** presents the conceptual diversion drain locations for collecting contaminated runoff, with sheet flow path lengths not exceeding 80m (*in accordance with the Blue Book requirements*).

The diversion drains would be of circular, parabolic or trapezoidal cross section rather than V-shaped. Details of diversion drains are presented on the standard detail SD 5-5 in the Blue Book and on **Figure 7**.

4.8.3 Sediment Fences

Sediment fences would be constructed to the general conceptual layout shown on **Figure 6**. Generally the sediment fences would be positioned parallel to the site contours at the downstream interface between disturbed and undisturbed areas. Details of sediment fences are presented on standard detail SD 6-13 in the Blue Book and on **Figure 7**.

4.8.4 Stockpile Protection

Stockpile protection would be required for excavated sediment, topsoil and other landscaping materials. The location of the designated stockpile sites are conceptually shown on **Figure 6**. The stockpiles would be constructed and protected in accordance with the standard detail SD 4-1 in the Blue Book. Stockpiled materials would be placed no closer than 2m from major drainage paths. Conceptual details of stockpile treatments are shown on **Figure 7**.

4.8.5 Maintenance of sediment and erosion control measures

The following outlines the proposed maintenance activities to maintain the effectiveness of the sediment and erosion control devices.

- sediment and erosion control devices would be regularly maintained and accumulated sediment removed before 50% of the capacity is used. Accumulated sediment would be re-used or disposed of in an acceptable manner off-site.

- sediment fences would be checked regularly for rips, excessive build up of sediment behind the fence, and breaches by construction activities. Damage to the fences would be repaired immediately on detection.

- surface water flows would be diverted around the designated site access to prevent sediment trapped within the access being re-suspended and transported offsite. Sediment that bypasses the stabilised site access, and is deposited on the nearby public streets would be cleaned up promptly by means other than washing into the drainage system.

- sediment and erosion control devices would be maintained until the disturbed areas have been adequately reinstated or new vegetation is sufficiently established.

4.9 EXPECTED WATER QUALITY IMPACTS

The Director General's requirements have outlined the following issues that should be addressed in regards to water quality.

4.9.1 Potential impacts on the quality on surface and ground water

The implementation of the proposed water quality treatment measures proposed in **Section 4.3** would improve surface runoff water quality from the site. This would lead to the long term improvement in receiving water quality.

There would be no significant impact on ground water quality. It is expected that the surface runoff that infiltrates to become groundwater would be of similar or better quality when compare to existing conditions.

4.9.2 Consistency with any relevant statement of joint intent established by the Healthy Rivers Commission

It is noted that the Healthy Rivers Commission has been dissolved and the Catchment Management Authority is responsible for tasks previously managed by the Healthy Rivers Commission. As such, the Draft Southern Rivers Catchment Action Plan has been used to replace a relevant statement of joint intent for this development.

The Draft Southern River Catchment Action Plan has developed the following relevant water targets:-

- Improving Water Quality – By 2016 water quality of all water bodies is maintained or progressively improved from 2005 benchmarks.

The proposed water quality treatment measures are expected to improve the long term water quality of the receiving waters. As such, consistency with this target was achieved.

- Water Conservation and Efficiency – Reduce residential potable water consumption from 2001/02 benchmark by 2016.

The proposed development would include a minimum rainwater storage tank of 4,000 L per lot. Harvested rainwater would be utilised for toilet flushing, clothes washing and garden watering. The inclusion of rainwater tanks would be in conjunction with water saving appliances which in combination would reduce the potable water demand of this development by approximately 40%. Therefore, consistency with this target would be achieved.

4.9.3 Achievement or protection of the river flow objectives and water quality objectives

The Department of Environment and Conservation’s River Flow and Water Quality objectives aim to achieve long-term goals for NSW's surface waters, such as:-

- water quality management to assess water quality in terms of whether the water is suitable for a range of environmental values (including human uses); and
- surface water flow management to identify the key elements of the flow regime that protect river health and water quality for ecosystems and human uses.

The proposed development is expected to improve the quality of surface runoff when compared to existing conditions and thus contribute to the long term improvement of receiving water quality. These improvements in water quality would contribute to improvement of the environmental values of Burrill Lake.

Stormwater harvesting, detention storage, bioretention swales and artificial wetlands have been integrated into the proposed development to allow the post development runoff regime to mimic the pre development runoff regime. These measures will slow down runoff from the site, it will reduce the runoff volume especially in small storms, it will better match the frequency of runoff compared with a rural site and will encourage infiltration to maintain throughflow in the shallow subsoil areas. Importantly, the base flows into the creeks and receiving waters would not be reduced.

4.9.4 Council’s draft integrated water cycle management plan

Council are currently developing an Integrated Water Cycle Management Plan to be adopted as part of their operating guidelines. It is envisaged that this management plan will promote the water saving appliances, rainwater harvesting, recycling of water and improved surface runoff quality.

All these strategies have been incorporated into the water cycle management for the proposed development.

Water saving devices and stormwater harvesting in rainwater tanks would reduce potable water use by a minimum of 40%. Roof runoff would be recycled for use in toilet flushing, washing machines and irrigation. The quality of surface runoff would be improved by incorporating industry best management practice measures which will lead to an improvement in the runoff quality compared to existing conditions.

4.9.5 Accumulative impact on Burrill Lake

The proposed development would utilise water sensitive urban design techniques to reduce runoff water quality to levels below that of existing conditions. Therefore, it is expected that the development would have significant beneficial accumulative impact on Burrill Lake.

It is expected that during construction, the proposed construction phase water management measures would minimise the risk of pollution of Burrill Lake.

5 WATERWAYS AND ESTUARIES

As part of the Environmental Assessment for this site the Department of Natural Resources (DNR) has requested that the proposed subdivision provide consistency with the following:-

- Rivers and Foreshores Improvement Act (1948);
- NSW State Rivers and Estuaries Policy;
- NSW Estuary Management Policy; and
- Burrill Lake Estuary and Catchment Management Plan.

The site is located approximately 0.5km west of the shores of Burrill Lake. A range of best practice management measures to be utilised within the proposed development would ensure that there is no significant impact on the ecology or sustainability of the lake.

5.1 BURRILL LAKE BACKGROUND

Burrill Lake has a total catchment area of approximately 78 km^2 and a total water surface area of approximately 4.1 km^2. Its entrance is generally open and untrained. The lake contains no mangroves but supports approximately 0.5 km^2 of seagrasses and 0.2 km^2 of salt marsh.

Burrill Lake is vital to the local oyster farming, boating and tourism industries. The lake is known as a urban recreational waterway.

5.2 RIVERS AND FORESHORES IMPROVEMENT ACT

As shown of **Figure 1**, Creeks 1 and 2 would be maintained as part of the proposed subdivision. A 20m corridor from the top of bank for Creeks 1 and 2 would be provided as riparian corridor. These existing corridors would be embellished with additional riparian vegetation were required.

It should be noted that no construction works would occur within the existing creeklines. The exception would be a culvert crossing of Creek 1. This culvert would be constructed to allow migration of fish species and native fauna. The culvert crossing would not exceed 15m continuos length without exposure to daylight.

Environmental buffers would be provided to the proposed water quality control ponds and the existing downstream wetland. Connectivity along riparian zones would be maintained.

The proposed environmental buffer zones would be in accordance with Council's development guidelines.

The proposed development would meet the requirements of the Rivers and Foreshore Improvement Act. Approval would be required under this Act for works within 40m of the top of bank for Creeks 1 & 2.

5.3 NSW STATE RIVERS AND ESTUARIES POLICY

It is understood that the NSW State Rivers and Estuaries Policy aims to reduce and where possible to halt:-

- declining water quality;
- loss of riparian vegetation;
- damage to river banks and channels;
- loss of biodiversity; and
- declining natural flood mitigation.

The proposed runoff water quality management measures are expected improve surface runoff quality when compared to existing conditions. The improvement in runoff quality would contribute to the improvement of the downstream environment including the downstream wetland which supports biodiversity in the area.

The peak flows from the site would be maintained at or below existing rates so that there would be no adverse impacts on creek bank erosion.

As part of the proposed development riparian buffer zones along Creeks 1 and 2 would be maintained and embellished with native riparian species. Constructed water quality control ponds would promote aquatic habitat and lead to improved biodiversity in the area. Pre development runoff regimes would be mimicked to minimise disruption of established aquatic and wetland habitats.

Natural flood mitigation behaviour would be enhanced with measures such as stormwater harvesting and additional detention storage. As such, the proposed development would meet the requirements of the NSW State Rivers and Estuaries Policy.

5.4 NSW ESTUARY MANAGEMENT POLICY

It is recognised that the states estuaries have significant ecological, social and economic importance and as such should be maintained and improved were ever possible.

The proposed development would not have significant adverse impacts on the Burrill Lake estuary system because:-

- Runoff quality post construction is expected to be improved when compared with pre development conditions. Best practice construction phase erosion and sedimentation controls would be utilised to minimise the risk of polluting downstream environments.

- Stormwater harvesting, infiltration in bioretention basins and detention storage would be provided on site to allow post development storm flows to more closely mimic the pre development runoff regime.

Therefore, it is expected that existing physical processes in the estuaries would not be significantly affected and, in fact, it would contribute to a long term improvement and positive accumulative affects due to a reduction of pollutants being exported from the site.

It is to be noted that the proposed development would not include any construction within 500m of the existing Burrill Lake foreshore.

All existing vegetation between the proposed development and the lake is located on land owned by others and it is assumed that it would be maintained.

The proposed development is expected to meet the requirements of the NSW Estuary Management Policy.

5.5 BURRILL LAKE ESTUARY AND CATCHMENT MANAGEMENT PLAN

The Burrill Lake Estuary and Catchment Management Plan raises three main issues in relation to stormwater management for the proposed development. These issues are:-

- Water Quality – to ensure that Burrill Lake meets the NSW government's interim water quality objectives for recreation, aquatic ecosystems, visual amenity, secondary and primary contact recreation, limited household and irrigation supply and cooked aquatic foods.

The proposed improvement in runoff quality from the proposed subdivision when compared to pre development rates would contribute to the improvement of water quality in Burrill Lake. In turn this would assist in sustaining the existing industry dependent on this waterway, including the oyster farming industry.

Construction phase erosion and sediment controls would be implemented to reduce the risk of pollution of the Dolphin Point wetland and Burrill Lake.

- Erosion and Sedimentation – to minimise erosion of soil from the catchments and to protect the lake from excessive sedimentation.

The proposed development would reduce the quantity of sediment exported from the site by the construction of water quality control ponds that would remove a large portion of sediment. The amount of sediment exported from the site post development is expected to be less than that exported pre development. This would assist in minimising sedimentation of Burrill Lake.

The design of the erosion and sedimentation controls for the construction phase has been based on the most stringent requirements of the state governments guidelines (Blue Book). Even for then, the design has been based on capturing a more severe storm runoff than recommended in the guidelines.

- Water Flows – to maintain the natural flow patterns of the creeks into the estuary.

Stormwater harvesting, infiltration in the bioretention basin and wetland and the detention storage would be provided as part of the proposed development to allow post development storm flows to more closely mimic pre development storm flows. Based flows in the creeks would not be reduced compared with existing conditions.

The proposed development is expected to be consistent with the objectives of the Burrill Lake Estuary and Catchment Management Plan.

6 STORMWATER DRAINAGE CONCEPT PLAN

A stormwater drainage concept plan has been developed for the site and is presented on **Figure 3** and detailed in the preceding sections.

The pipe drainage network is to accommodate peak runoff from all events up to the 5 year ARI storm event in accordance with Council guidelines. Flows in excess of the pipe capacity would be accommodated safely in the road reserves which would form the overland flow paths.

Two large upstream external catchments contribute to the two creeks that run through the site. In addition, there are two existing overland flow paths on the site. The external flows would be conveyed through the site by a combination of the trunk drainage system and constructed overland flow paths. Both the proposed overland flow paths and existing creek lines are able to convey the 100 year ARI flows without significant impact on the proposed lots.

Riparian corridors would be maintained and where necessary enhanced along the two creeks in the development. The creation of a wetland in the existing dam would contribute to the diversity of aquatic habitats on the site.

A stormwater detention strategy has been developed to ensure no impact on downstream drainage infrastructure. The strategy incorporates the use of extended detention on the proposed bioretention basin and water quality control pond to reduce peak flows to existing rates for a range of storm events. The stormwater collection in rainwater tanks would provide further attenuation of peak flows but this has not been accounted for in the provision of detention storage.

Water quality measures are proposed, which would improve the quality of the water leaving the site, thereby exceeding the Council objective of no net increase in pollution levels. The DEC's minimum recommended reduction in pollutant load would also been met.

Hydraulic analysis was carried out to determine flood levels across the site. The habitable floor levels would conform to Council's freeboard requirements above these levels. The freeboard for flood ways is 500mm and for flood storage areas is 300mm.

The proposed water management strategy for the proposed Dolphin Point subdivision is expected to contribute to the long term improvements in receiving water quality and maintain existing flow regimes to minimise any significant impact on the existing downstream environment.

The proposed development is expected to comply with all relevant stormwater management issues raised in the DOP Director General's Environmental Assessment requirements dated 15 February 2006.

7 REFERENCES

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