



Oyster Reach Estate

PRECINCT 2A
Old Bar

Concept Stormwater Plan (July 2007)

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for Riverside Oyster Reach Estate Pty Ltd

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1.0 Introduction.

1.1 Development Proposal.

The proposal is to develop the land for residential development in a number of stages.

This report sets out the strategies for the management of stormwater runoff from the development site. The creation of residential allotments and the dwellings contained thereon increases the water runoff quantities and is said to increase the discharge of pollutants from the site.

The report develops detailed proposals to ameliorate these factors and proposes a number of control systems and devices to provide for stormwater quality and a degree of reduction in stormwater quantity discharging from the site.

Section 1 Gives background information. Outlines the Director General's Environmental Assessment Requirements, Greater Taree City Council Standard Conditions and Discussion Relating to DGEARS & GTCC Requirements

Section 2 Gives the location of site in context of Greater Taree LGA, Old Bar environs and local cadastre.

Section 3 Describes the existing drainage pattern, urban drainage system and proposed mitigation measures including protecting the Salt-Marsh community. Outlines increased Stormwater runoff, sediment load, nutrient load and expected 1% Flood Levels.

Section 4 Sets the criteria for modelling urban runoff including target volume, describes the catchment and pond layout, proposed outlet structures and details Old Bar IFD Chart, Runoff Coefficients and the Pipe Sediment Traps.

Section 5 Details concept designs of the pond systems and provides resultant Stage/Storage/Discharge tables for each proposed pond.

Section 6 Describes the preliminary hydraulic modelling.

Section 7 Provides a Soil and Water management plan including the Revised Universal Soil Loss Equation calculations, Acid Sulphate investigation, estimation of GPT sizing and describes the gross pollution traps.

Appendix 1 Provides the estimation of Total Runoff Volumes for calculation of target pond volumes.

Appendix 2 Provides the MUSIC Analysis and Pre and Post development drainage modelling for the proposed development

Appendix 3 Provides the Water Quality testing and results to date.

1.2 Background

The development is part of Precinct 2 contained in the Greater Taree Old Bar Wallabi Point Development Strategy 2001.

The eastern section of the site comprises Old Bar Precinct 2A which has been zoned for residential development in Greater Taree Local Environmental Plan 1995 (Amendment No 54) gazetted March 18, 2005.

The western portion of the site is within Old Bar Precinct 2B which has been the subject of a Local Environmental Study Precinct 2B Planning Report June 22, 2006. This LES has been publicly exhibited and is presently under review by greater Taree City Council.

This study comprises a discrete area of land that is to be developed. The land in this report overlaps the two precincts in order to provide an holistic approach to the development of the property.

1.3 Director Generals Environmental Assessment Requirements

The Department of Planning have authorised a concept plan (Application No 05_0106) and have issued the Director Generals Environmental Assessment Requirements (DGEARS) which include 2) Infrastructure Provision, 4) Water Cycle Management 8) Hazard Management and Mitigation and 9) Public Access & Community Services each of which comprise components that are addressed in this Stormwater Concept Plan.

This report addresses the components of these sections of the DGEARS.

1.4 Greater Taree City Council Development Standards.

This section of the report is aimed at pre-empting, addressing and fulfilling the requirements of the standard development conditions of Greater Taree City Council in presentation of this Concept Development.

1.5 GTCC Standard Development Conditions (Stormwater)

It is expected that Greater Taree City Council would apply the following standard development conditions to a development of this nature.

(a) “A **Stormwater Drainage Report** prepared by an Engineer whose qualifications are acceptable for membership of the Institution of Engineers Australia (or other professional approved by Council) is to be submitted to Council for approval prior to the issue of a Construction Certificate. “

“The report is to determine the extent of the 1% ARI event affecting the subject land, including the effects of the total development of the catchment, the effects on the downstream properties and the possible need for water detention facility. Additionally, the report is to analyse the impact of the upstream catchment developed to its full potential as well as the fully developed subject land on the downstream drainage system.”

(b) “**Water Sensitive Urban Design (WSUD)** principles are to be incorporated into the drainage design. The devices selected are to be submitted to Council for approval with the engineering drawings and are to be capable of retaining pollutants in accordance with the requirements of Councils Stormwater Management Plan and Australian Runoff Quality (ARQ).”

(c) “A **Soil and Water Management Plan** in accordance with Council’s Policy prepared by an Engineer whose qualifications are acceptable for membership of the Institution of Engineers Australia (or other professional approved by Council) detailing temporary and permanent measures proposed to be installed. The plan is to include analysis of the susceptibility of soil to erosion and is to be submitted with the Engineering plans.

“All erosion and sediment control measures undertaken on the site are to conform to the specifications and standards contained in the document ‘Managing Urban Stormwater – Soils & Conservation’, Department of Housing, 2004 Manual.”

Grant Calvin B.Surv MIS (Aust) of McGlashan & Crisp Pty Ltd is a registered surveyor with post graduate qualifications in urban stormwater hydraulics and 25 years of experience in urban road and drainage design. Grant has been registered with Greater Taree City Council as a professional approved by Council since the early 1990’s.

1.6 Greater Taree Urban Stormwater Management Plan

The Greater Taree Urban Stormwater Management Plan covers urban areas within the Greater Taree region, including the Old Bar area.

The aim of this plan, which has been adopted by Council, is:

‘to minimise future impacts on the stormwater environment and to provide a framework for mitigating existing impacts on the stormwater environment with the overall objective of helping to achieve Council’s commitment to ecologically sustainable development.’

The overall management objectives of the Greater Taree Urban Stormwater Management Plan include:

- To improve stormwater quality prior to it entering receiving waters;
- To improve knowledge of stormwater impacts within the Taree local government area; and
- To identify stormwater flow regimes that impact on the environment.

The proposed development will comply with the aims and objectives of Council’s Urban Stormwater Managements Plan and will manage stormwater in an ecologically sustainable manner minimising the impacts of the urban stormwater on aquatic ecosystems, public health and amenity.

Incorporated into the treatment train design is a series of storm water controls including household rainwater tanks, sediment retention basins, storm water pit inserts, gross pollutant trapping and constructed wetlands.

The treatment train has been described in Section 2.0.

1.7 Conclusions & Discussions Relating to DGEARS & GTCC Requirements

DGEARS 2 Infrastructure Provision

DGEARS 2.1 Address existing capacity and requirements of the subdivision for sewerage, water & electricity in consultation with relevant agencies having regard to Water Sensitive Urban Design (WSUD) principles.

The capacity of the Sewerage, Water and Electricity services are beyond the scope of this report, however the stormwater management principles of WSUD have been addressed in Section 3 of this report. The capacity of the stormwater system is discussed in Section 6.

DGEARS 4 Water Cycle Management

DGEARS 4.1 Describe the measures to mitigate direct and indirect impacts on the wetland site listed under State Environmental Planning Policy No 14 – Coastal Wetlands, including buffer areas, ownership and management.

The measures for the mitigation of the effects of this development are described in Section 3 of this report and include rainwater tanks, pit inserts, swales and bio-retention swales and wetland basins.

DGEARS 4.2 Demonstrate the impact and treatment of stormwater discharge including overland flows from the site to the local waterways and potential impacts of stormwater run off to adjoining Crown Reserves.

The effectiveness of the proposed measures has been modelled using the Model of Urban Stormwater Improvement Conceptualisation (MUSIC) The model results can be found in Appendix 2 Stormwater Quality Management Strategy (WBM 2007) Section 4 pp9. Overland flows are captured and directed using grassed swales. There are no adjoining Crown Reserves downstream of the site.

DGEARS 4.3 Provide details of Integrated Water Cycle Management (IWCM) for the site if proposed.

A non-potable water reuse system is not considered warranted for this location because of the proximity of the wetlands and inherent needs of the development. The coastal area has very good rainfall and is not unduly affected by extended drought. Roof water collected and stored rainwater tanks will adequately provide water for gardens and toilet flushing under the BASIX SEPP.

It is considered that a reticulated reuse system would increase the overall moisture regime of the development with non-potable water containing elevated levels of Nitrogen and Phosphorous. In this location, adjacent the sensitive wetlands, adding additional moisture and nutrient burden to the treatment train, prior to release directly into downstream waterways, is considered inappropriate.

DGEARS 4.4 *Provide details of the location and size of riparian buffers adjoining water courses and wetlands (including adjoining buffers).*

The Landscape Management Plan (ID Landscape Management 2007) provides the proposed tenure and management regimes for the open space areas of this development.

DGEARS 4.5 *Address the requirements of the relevant flooding data, having regard the NSW Floodplain Management Manual.*

Flooding:- The extent of the 1% flood has been determined and does not affect the proposed development lots. Refer section 3.7 & Figure 5

DGEARS 8 Hazard Management and Mitigation

8.3 *Identify the presence and extent of Acid Sulphate Soils on the site and appropriate mitigation measures.*

Testing for the presence of Acid Sulphate Soils has been done on the development site. The potential for ASS was below the level required to trigger the requirements for the development of an Acid Sulphate Management Plan. Refer Section 7.6

DGEARS 9 Public Access and Community Services.

8.1 *Illustrate pedestrian and cyclist linkages in the vicinity of the site including to Precinct 2B and provide additional walkway / cycleway facilities where appropriate.*

The creek side footway/cycleway has been continued around this development providing public access to the stream lines and creeks. This development also provides a footway cycleway connection from the Precinct 2B developments to the south and west to the Precinct 1 developments to the north and east. These linkages make the pedestrian and cycling connections from the existing and future village areas to the river and to the estuary and beach at Old Bar and can be seen on the Lot layout plans.

Greater Taree City Council Requirements

1. GTCC Stormwater Drainage Report

- i) Flooding has been addressed as above.
- ii) On Site Detention:- Peak discharge from the site will be limited to less than the natural runoff by the basin system proposed for all high frequency storm events. In the instance of ARI 100year or 1% storm events the basin system designed will attenuate the peak discharge for all short duration storms less than 6 hours duration. Due to the location of the development in the lowest segment of the catchment (ie not causing downstream flooding) mitigation measures for these very infrequent long duration storms is considered unwarranted. (Refer Stormwater Quality Management Strategy Report Section 5.3 pp21 WBM 2007)

2. GTCC Water Sensitive Urban Design

- i) The Stormwater treatment train that starts with rainwater tanks and includes pit inserts, bio-retention swales, gross pollution traps, sediment basins, wetland retention and final polishing through the existing bushland represents best practice WSUD principles. (Refer Section 3)
- ii) The objectives of Council's Urban Stormwater Management Plan will be achieved. (Refer: Stormwater Quality Management Strategy Report Section 4.5 pp15 WBM 2007)

3. GTCC Soil and Water Management Plan

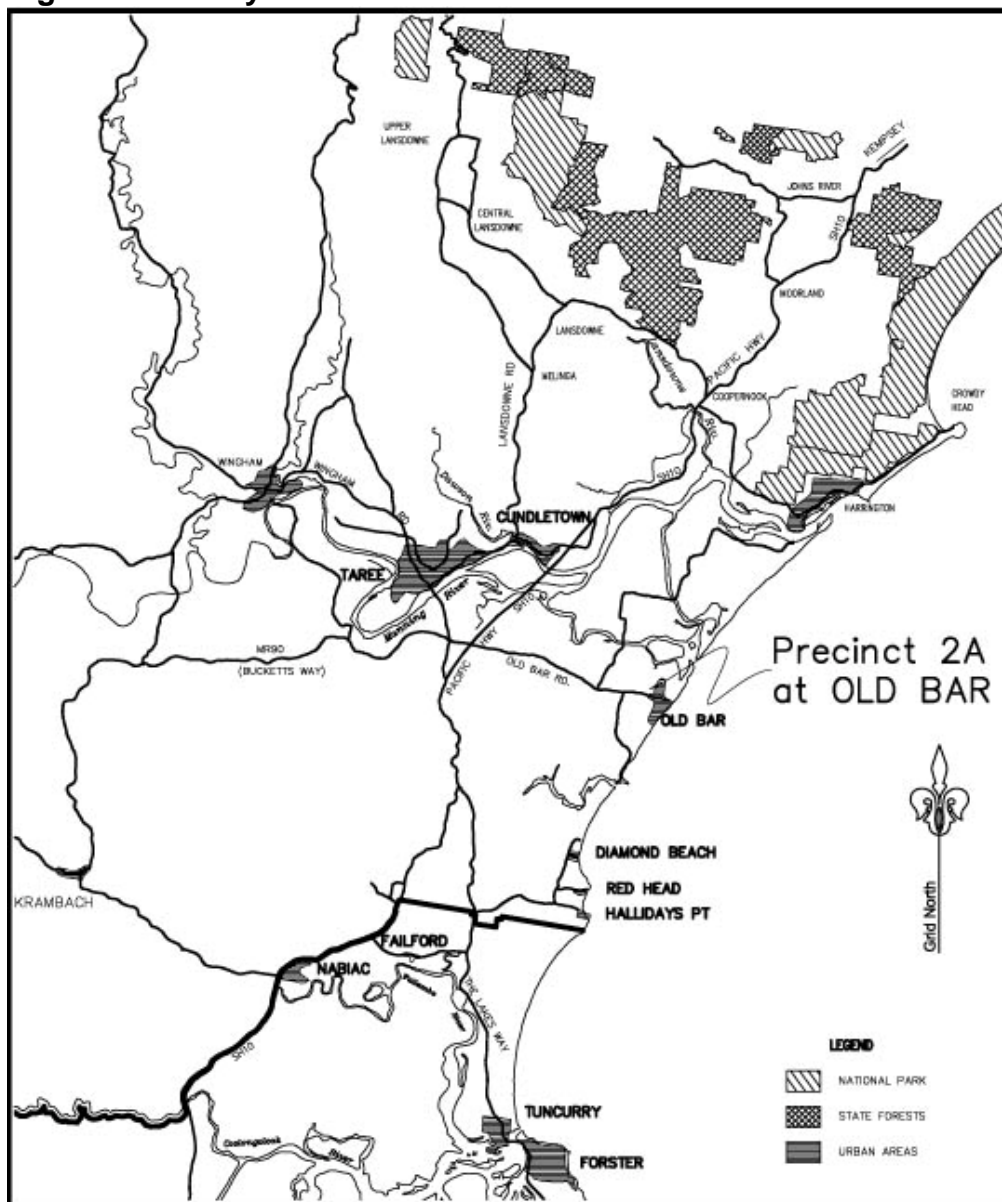
- i) Sections 7 of this report provides the basis for the Soil and Water Management plan including assessment of the soil and it's propensity for erosion.
- ii) Section 7.7 describes the Acid Sulphate testing and results
- iii) Detailed erosion and sediment control plans will form part of the Engineering Drawings for construction.

2.0 Location within GTCC LGA

Oyster Reach Estate is a subdivision development of approximately 200 lots on the northwestern outskirts of Old Bar in the Greater Taree Local Government Area. Oyster Reach development is Precinct 2A and part of Precinct 2B in Greater Taree City Councils' (GTCC) Old Bar Wallabi Point Development Strategy.

GTCC is a coastal LGA located between the LGA's of Great Lakes & Hastings on the Mid North Coast of NSW approximately 300 kilometres north of Sydney. The population of the GTCC LGA is centred on the towns of Taree and Wingham. Smaller villages are located around the LGA with significant villages of Harrington, Old Bar and Hallidays Point/Redhead dotted along the coast. The location of the Precinct 2A within general layout of the Greater Taree City LGA can be seen on the map below.

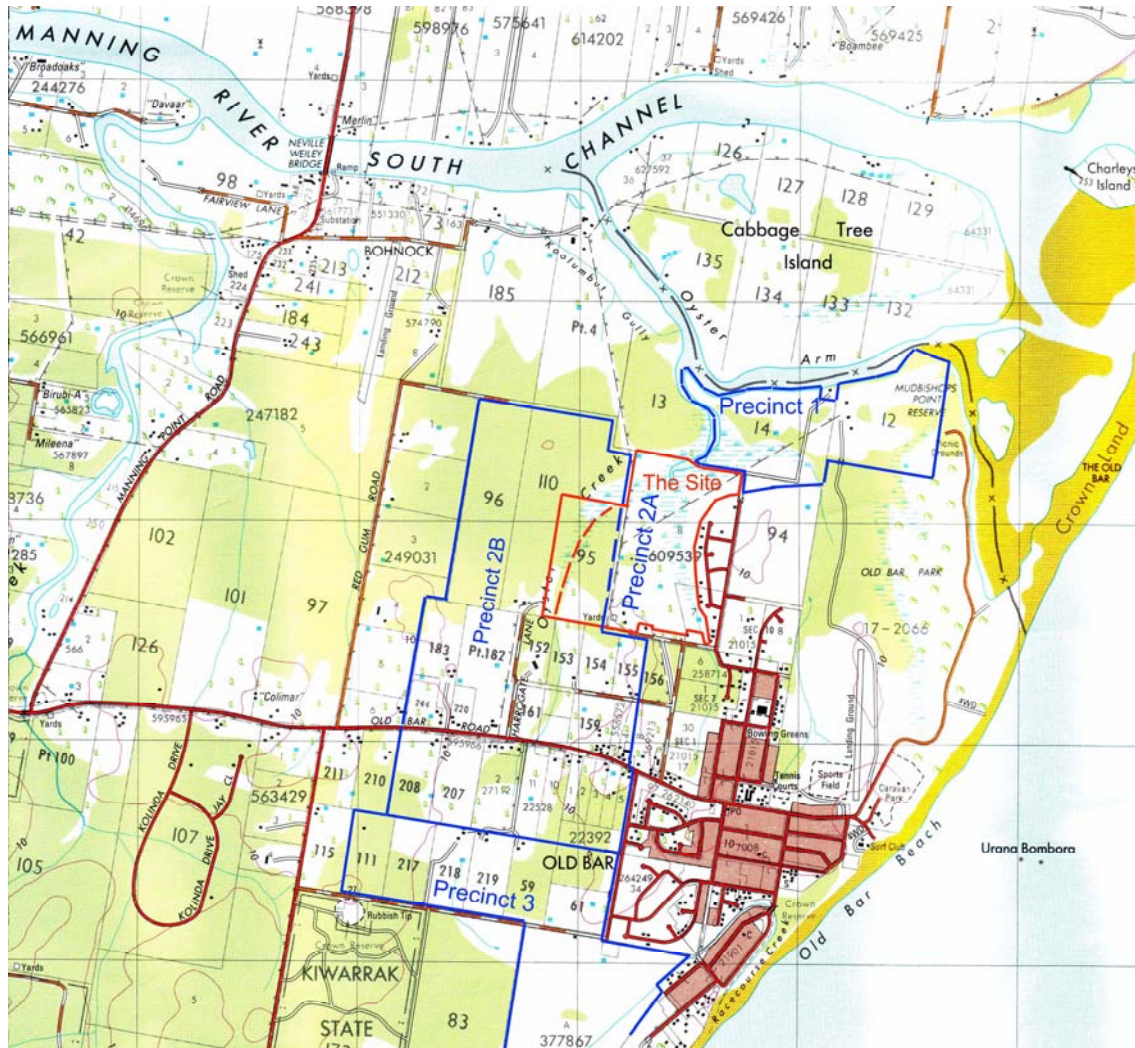
Figure 1 Locality within GTCC



2.1 Location relative to Old Bar Environs

The Locality of the Site in relation to the existing village of Old Bar, the Pacific Ocean, the Manning River, its tributaries and other Precincts in the Old Bar Wallabi Point Strategy Plan can be seen on the extract of the Cundletown Topo Map reproduced below

Figure 2 Locality in relation to Old Bar & Environs



The proximity of the Site to the Pacific Ocean, The Manning River and its tributaries places this development within the Coastal Development Zone of New South Wales.

As a result of this proximity this development is considered a Major Project under Part 3A of the Environmental Planning and Assessment Act 1979. An application has been made and Minister authorised a concept plan be lodged on June 30, 2006.

2.2 The Site

The site of Oyster Reach Estate is made up of three adjacent lots - Precinct2A being Lot 7 DP1069956 and part of Precinct 2B being Lot 1 DP48330, a closed road, and part of Lot 95 DP753149 east of Oyster Creek – (the Site). These lots can be seen on the plan below.

As noted in 1.2 above this report details the discrete section of land that is bounded on both sides by creek lines and has been prepared to provide an holistic review of the drainage factors that relate to the entire section of land. A small section of land off the northern end of Corkwood Street will be addressed in a separate Development Application to Greater Taree City Council.

Figure 3 Site Plan



The development site lies between Banyula Creek on the east and Oyster Creek on the west.

The Site is bounded :-

- to the east by the existing village of Old Bar,
- to the south by Banyula Drive and adjoining urban and rural lands that form part of Old Bar Precinct 2B.
- to the west by Oyster Creek. And
- to the north by the confluence of those creeks and a SEPP14 wetland mapped as No 571.

3.0 Existing Drainage Pattern

The Site is within the catchment of Oyster Reach. Two drainage lines pass through the land being Banyula Creek to the east and Oyster Creek to the west. A wetland nominated in SEPP 14 exists around the confluence of these drainage lines. The wetland includes areas of Coastal Salt-marsh, Swamp Oak Floodplain Forest and Swamp Sclerophyll Forest on Coastal Floodplains, which are listed as endangered ecological communities. Small channels exist, some man made, that carry runoff through the swamp areas and into the open water sections of Oyster Creek.

The development site rises gently from the lowlands to a maximum height of approximately RL 16.1m AHD at the crown of the hill adjacent Banyula Drive. Development of the hill will drain to the north, west and east. The contours, natural flow lines and proposed flow lines can all be seen on Figure 4.0 below

Figure 4 Drainage Flow Patterns



3.1 The Drainage System

The urban development of this site comprises a number of components.

Initially rainfall on the developed sections of the catchment falls on roofs and lawns.

- **Rainwater Tanks:** The roof areas of the development are directed into rainwater tanks attached to the dwellings. Rainwater tank installation is a requirement of the SEPP Building Sustainability Index (BASIX 2004).
- **Urban Lot Drainage System:** Once full, overflow from the tanks is directed into the urban drainage system. On the high side of the roadways it is discharged onto the kerb of the street via non-flexible kerb adaptors (NFKA). NFKA are installed in the high side kerb at the low side of each lot. On the low side of the road the roof water is directed to the Interallotment Drainage System (IAD). The IAD piped drainage system is installed in the rear of lots falling away from the road. Pits and junctions are installed for future connection of the dwellings rainwater tank overflow pipes when developed.
- **Road Kerbs:** Rainfall that lands on the driveway and lawn areas makes its way down slope and into the kerb on the high side of each roadway. Kerbs collect the runoff and direct it down slope along the roadways to drainage pits. The drainage pits are situated to collect water before the width of flow in the gutter enters the traffic lanes of the road. (typically 2.4m)
- **Pit Inserts:** “ECOSOL” or similar pit inserts are to be installed in a number of salient Stormwater pits during the subdivision construction and building development phases of the site.

These pit inserts collect sediment and other pollutants at source and are considered best practice in the initial phases of the development. The pits represent a significant maintenance burden and may be removed with the completion of the final urbanisation of the site.

- **Piped Drainage System:** The piped drainage system collects the water from the pits and conveys the water to the down stream outlets. A preliminary pipe drainage system has been modelled for this development and is described in Section 6.0 below. The piped drainage system has been designed to discharge into a series of swales and drainage basins situated around the site.
- **Swale & Pond System:** The geography of this site lends itself to the installation of longitudinal ponds. These ponds are designed to appear as natural levees that occur around coastal rivers and creeks with a small constructed wetland in the base.

The overall system is designed to provide for Water Sensitive Urban Design where: -

- Runoff is captured in rainwater tanks at source high in the catchment
- Water once captured is available for reuse on gardens and toilet systems
- Sediment and litter is caught at salient entry pits in the catchment
- Sediment and litter is further entrapped in swales and stormwater quality basins prior to discharging into downstream waters. And
- Nutrient loads in the urban runoff are reduced through a measure of hydraulic retention in the constructed wetlands within the stormwater basins.

3.2 Mitigation Measures

The close proximity of the SEPPP 14 wetlands raises the potential for indirect impacts on the wetlands by the development. Among the potential impacts are increases in stormwater runoff, stormwater sediment load and stormwater nutrient load. The mitigation measures are briefly described in this section of the report.

3.3 Protecting the Coastal Salt-marsh Community.

Fresh water runoff has the potential to leach the salt from the soil. In this way urban stormwater runoff can affect Coastal Salt-marsh by decreasing the salinity built up over time by the tidal inundation of the marshlands.

In order to mitigate against this process overland flow for this development it is intended to redirect the urban runoff flows to points upstream of the Coastal Salt marsh area. (Refer to Appendix 4 minutes of site meeting held Sept 16 2004 with Scott Carter NSW State Fisheries, Peter Adcock, Wetlands Australia Pty Ltd and Robert Baker, Greater Taree City Council). This redirection can be seen in Figure 4 above.

This involves the filling of the flat land at the northern extremity of the development in order to grade drainage from the urban development into constructed wetland basins higher up the catchment than the salt marsh communities.

Discharge from these wetland basins will be directed into the existing channels that pass through the salt marsh communities.

In this way the propensity for desalination of the salt marsh communities by fresh water runoff and overland flow from off this development will be reduced through the design of the drainage system.

3.4 Increased Stormwater Runoff

Some of the increases in stormwater runoff will be in part controlled by the installation of rainwater tanks on dwelling constructed within the development.

Primarily increases in stormwater runoff and the peak flow will be controlled through temporary storage of water in the wetland basins. The capacity of the wetland basins is targeted to temporarily store the peak runoff volume from an ARI 1 year storm.

Peak flows that result from lower frequency storms (>ARI 5 10 to 100 years) will also be significantly reduced by the volumes retained in the constructed wetland basins.

During some long duration, low frequency storms (ie ARI 100 year +6hr storms) there will be a marginal increase in the peak flow. Controlling the increases in runoff flows from these storms, is not considered warranted due to the infrequent nature of the storms and the location of this development at the bottom of catchment.

3.5 Increased Stormwater Sediment Load

Increases in stormwater sediment load will be reduced at source through the use of pit inserts such as “*ECOSOL RFS100*” in salient stormwater pits throughout the development. These pit inserts are particularly important in the initial development and building phase of any urban development. The inserts collect up to 95% of pollutants >Ø1.5mm including general litter.

Installed within the wetland basins there is also to be an end of pipe filtrations system or Gross Pollution Traps (GPT’s). In essence the GPT’s are an area, concrete lined that is designed to capture and store larger particulates for latter removal by mechanical means.

3.6 Increased Stormwater Nutrient Load

Increases in nutrient load are to be reduced through biological means through the wetlands plants planted in the constructed wetland basins. There wetland plants have the ability to breakdown and store the nutrient load from off the developed site prior to the water entering the downstream water chain.

Discharge from the catchment is directed into the stormwater basins. Discharge from these basins varies with height.

In the initial stages discharge from the basins is limited to small diameter pipes. As a result of this limited outflow water backs up in the basins. The limited discharge is designed to maintain water in the basin for a period of around 24 hours. In this time the plants of the constructed wetland area able to absorb many of the excess nutrients that may be found in the urban runoff.

3.7 ARI 100year Flood Levels or 1% Flood Levels

The 1% Flood Levels around the site have been calculated in separate LES reports for Precinct 2A (Stormwater Management Report, Terra Consulting September 2003) and Precinct 2B (Trunk Drainage and WSUD Strategy for Precinct 2B, Old Bar, WBM Oceanics April 2006). The table below summarises the Peak Water Levels for the ARI 100 year flood event. We note that the Peak Flood Levels depicted are a conservative worst-case scenario.

The scenario modelled is where an ARI 100 year flood in the Manning River catchment with a time of concentration 2-3 days peaks coincidentally with an ARI 100 year storm in the much smaller Old Bar catchments where the time of concentration is in the 2-3 hours range. This is an unlikely event with a frequency potentially far less than ARI 100 years possibly as low as ARI 500, or 1,000 years or less. (0.2% to 0.1%)

Table of ARI 100 Year Peak Water Levels (1% Flood Levels)

Reach or Creek Name	Creek Chainage	ARI 100 year Peak Water Level
Confluence of Creeks	779.6 Banyula Ck 00 Oyster Ck	2.42
Banyula Creek	900	2.44
Banyula Creek	1000	2.45
Banyula Creek	1100	2.45
Banyula Creek	1200	2.46
Banyula Creek	1300	2.48
Banyula Creek	1400	2.54
Banyula Creek	1500	3.2
Banyula Creek	Banyula Drive Culvert	3.24
Oyster Creek	76	2.42
Oyster Creek	220	2.42
Oyster Creek	300	2.43
Oyster Creek	360	2.43
Oyster Creek	450	2.44
Oyster Creek	550	2.45
Oyster Creek	685	2.48
Oyster Creek	745	2.51
Oyster Creek	805	2.59
Oyster Creek	892	2.87
Oyster Creek	914	2.92
Oyster Creek	950	3.12

These calculated flood levels have been superimposed over the detailed level survey of the property and the natural surface contours and an approximate flood inundation plan has been prepared. Refer Figure 5 overleaf

Figure 5 ARI 100 year Peak Flood Levels Approximate Inundation



The area proposed for urban development does not encroach on the predicted ARI 100 year inundation area as can be seen in the plan above.

This development complies with the requirements of the Greater Taree City Council Interim Flood Management Policy 1987. Clause 12 “Subdivisions” of that policy states:-

“Except in special circumstances, subdivisions will not be permitted where any lot to be created will be fully inundated by a 1% flood and the creation of such a lot will create potential for increased intensity of development on flood liable land. In assessing whether or not land would be wholly inundated by a 1% flood, Council will disregard any proposals to filling of that land.”

This is consistent with the NSW Floodplain Management Manual.

4.0 Modelling the Urban Runoff

The Greater Taree City Council its regulations, policies and planning instruments will govern the subdivision.

4.1 Old Bar IFD Chart

Drainage design in the GTCC LGA is specified in Councils AUSPEC No 1 Development Design Specification Edition 2 July 2006. Stormwater Drainage Design is specified in Chapter D5.

The rainfall and runoff in Old Bar is based on the Old Bar Intensity Frequency Duration (IFD) chart produced in accordance with Chapter 2 Australian Rainfall & Runoff 1987. The chart below is part of the IFD contained within in GTCC's AUSPEC No1.

Figure 6 IFD Chart for Old Bar

IFD ANALYSIS BASED ON AUSTRALIAN RAINFALL & RUNOFF (1987)

Site name: OLD BAR VILLAGE AREA

Site latitude = 31.59 degrees S
 longitude = 152.35 degrees E
 skewness = .03

2-year ARI, 1 hour intensity = 38.00 mm/hr
 12 hour intensity = 8.40 mm/hr
 72 hour intensity = 2.60 mm/hr

50-year ARI, 1 hour intensity = 70.00 mm/hr
 12 hour intensity = 17.00 mm/hr
 72 hour intensity = 5.60 mm/hr

IFD Table for Various ARIs and Durations

Duration	1 yr	2 yr	5 yr	10 yr	20 yr	50 yr	100 yr	200 yr	500 yr
5 min	96.23	122.12	152.07	168.96	191.97	221.77	244.24	266.86	297.19
6 min	90.18	114.47	142.62	158.49	180.13	208.13	229.26	250.53	279.05
10 min	73.81	93.74	116.96	130.09	147.94	171.07	188.53	206.11	229.71
12 min	68.25	86.69	108.24	120.42	136.98	158.44	174.64	190.97	212.88
15 min	61.70	78.40	97.96	109.03	124.07	143.56	158.28	173.12	193.03
18 min	56.61	71.95	89.96	100.16	114.00	131.95	145.52	159.19	177.54
20 min	53.78	68.36	85.50	95.22	108.40	125.49	138.41	151.43	168.91
24 min	49.09	62.41	78.12	87.02	99.10	114.77	126.61	138.55	154.58
30 min	43.72	55.61	69.66	77.64	88.44	102.47	113.08	123.77	138.14
45 min	35.07	44.63	56.00	62.47	71.22	82.58	91.18	99.85	111.51
1.0 hr	29.79	37.93	47.66	53.20	60.68	70.40	77.76	85.19	95.18
1.5 hr	23.38	29.84	37.72	42.24	48.31	56.23	62.23	68.30	76.49
2.0 hr	19.62	25.08	31.85	35.75	40.97	47.78	52.96	58.21	65.29
3.0 hr	15.28	19.58	25.02	28.18	32.38	37.88	42.08	46.34	52.10
4.5 hr	11.89	15.27	19.63	22.19	25.57	30.01	33.40	36.85	41.53
6.0 hr	9.95	12.80	16.54	18.73	21.62	25.44	28.35	31.33	35.36
9.0 hr	7.75	9.99	12.99	14.76	17.09	20.17	22.53	24.94	28.21
12.0 hr	6.49	8.38	10.95	12.47	14.47	17.11	19.14	21.22	24.05
18.0 hr	5.05	6.53	8.58	9.80	11.40	13.51	15.15	16.82	19.10
24.0 hr	4.21	5.46	7.20	8.24	9.60	11.41	12.80	14.23	16.18
30.0 hr	3.65	4.74	6.27	7.19	8.38	9.98	11.21	12.47	14.19
36.0 hr	3.24	4.21	5.58	6.41	7.49	8.92	10.03	11.17	12.72
48.0 hr	2.67	3.47	4.62	5.32	6.22	7.43	8.37	9.33	10.64
72.0 hr	1.99	2.59	3.48	4.01	4.71	5.64	6.36	7.11	8.13

4.2 Runoff Coefficients

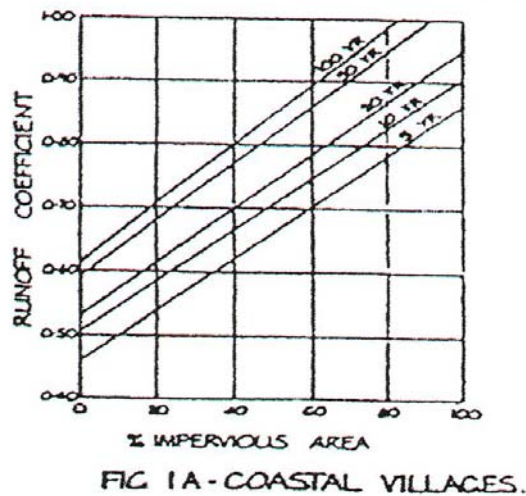
Runoff coefficients are based on AUSPEC No1 (D5.06), which specifies the percentage impervious to be used in lieu of detailed calculations. The project zoning is classified as Low Density Urban Residential 2 and the specified percentage impervious is 60%.

Percentage Impervious = 60%

Runoff Coefficients to be used are specified AUSPEC No1 (G.03) and the runoff coefficients for use in this project are based on Figure 1A – Coastal villages.

At 60% impervious the coefficients are read directly off this chart as reproduced in Figure 6 below.

Figure 7 Determination of Runoff Coefficients for Coastal Villages.



Runoff Coefficients

ARI	5 Year	10 Year	20 Year	50 Year	100 Year
Coefficient	0.7	0.75	0.78	0.85	0.89

For the purposes of this investigation the runoff coefficients for higher frequency storms have been adopted as **ARI 1 year = 0.6** **ARI 2 year = 0.65**

4.3 Target Pond Volumes.

The concept plan has been broken down into three (3) catchments. These catchments are labelled A, C & D and correspond with the proposed stormwater quality basins located on the east, southwest and northwest of the site.

Rainfall falling on these catchments will result in runoff being captured in the piped drainage system and directed through the road and street drainage into the proposed wetland ponds.

The catchment areas have been calculated and the standard rainfall patterns derived from the IFD chart above used to calculate the target volume for the various ponds, which is tabulated below.

Target Pond Volumes

Pond	Pond A	Pond C	Pond D
Catchment Area	11.57 Ha	6.06 Ha	4.86 Ha
Target Volume	4,650 m ³	2,436 m ³	1,955 m ³

The full calculation spreadsheets are printed in Appendix 1

The pond layout has been calculated from the available contour information.

The Base Water Level (BWL) 100 – 200mm deep has been set in relation to the surrounding channel to allow for drainage to the existing water course/s.

The Top Water Level TWL is set to be 1m above the Base WL thus the maximum depth in the pond system is set to about 1.2m and the side slopes are all designed to be 1V:6H for safety. This design is in accordance with AR&R 1987 Sec 14.10.4 Safety pp. 357-358.

The major storm outfall from the pond systems is over a concrete footpath that in effect creates a level weir that runs along the low side of the ponds. The length of the weir means that any overtopping of the basin in high flow situations is spread along the length of the basin.

4.4 Catchment and Pond Layout

The Catchment Pond Layout can be seen on figure 8 below. As noted the layout is based on the existing detail survey data. The detailed design of these ponds may result in a more dense survey of the area and may result in minor variations to the final design.

Figure 8 Catchment Layouts – Concept Pond Layout.



Detailed survey information at the engineering design phase may result in alteration of the current storm water concept plan or minor modification of the road layout.

4.5 Pond Discharge System

The ponds fill from runoff off the development. As noted earlier the target volume is designed to cater for the total runoff from an ARI 1 year storm.

Runoff water stored in the pond systems and released into the adjacent channels at a restricted rate through the outlet structure.

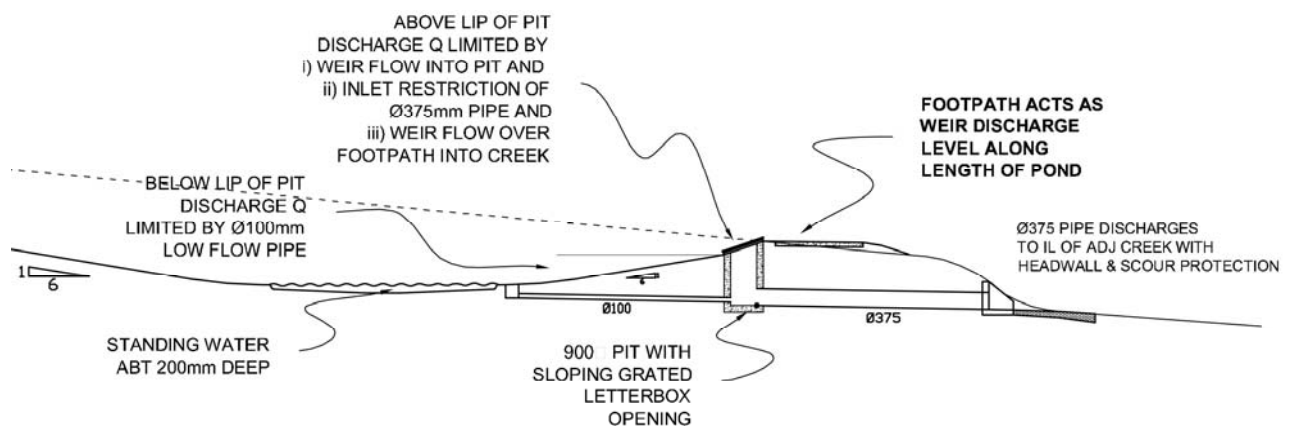
The hydraulic restrictions of outlet structures are described below.

- Initially a low flow pipe is via a Ø100mm diameter pipe set into a pit. The top level of the pit is set at the design Base Water Level (BWL). In effect these inlet pits will control the level of the permanent water bodies within the pond systems.

The small diameter outlet pipes restrict discharge from the pond systems until the depth of water is approximately 0.9m. (0.7m above base water level (BWL))

- When the pond fills above a depth of 0.9m the discharge is increased as water spills into the lower edge of sloping grated inlet pits. The outlet from the pit is a Ø375mm pipe.
- As the depth increases the discharge increases. Firstly as a weir restriction of the water entering the low side of the sloping pits and secondly, once the water level increases to a depth of 1.0m above BWL, the discharge is restricted by the inlet capacity of the Ø375mm pipes.
- Should the water level increase, to the level of the adjacent concrete footpath, the discharge can spill over the footpath, which in effect creates a long weir. As noted earlier the 1% peak discharge is calculated to not exceed a depth of 50mm over the weir. This depth is considered safe and has minimal erosion potential.

Figure 9 Sketch of Typical Outlet Structure



Typical Outlet Pit and Pipe Structure

Table of Depth / Discharge through Typical Outlet Structure

	Depth	RL	RL	Q m ³ /2	Q m ³ /2	Basin A		Basin C		Basin D	
	Of	Basins	Basin	Ø100mm	Ø375mm	Weir	Q	Weir	Q	Weir	Q
	Water	A & D	C	m ³ /s	m ³ /s	178m	m ³ /s	233m	m ³ /s	229m	m ³ /s
Invert of Pond 0.2m Deep	0.00	1.00	1.40	0.000		Long	0.000	Long	0.000	Long	0.000
Water Level of Pond	0.20	1.20	1.60	0.000			0.000		0.000		0.000
	0.40	1.40	1.80	0.013			0.039		0.026		0.026
	0.60	1.60	2.00	0.016			0.048		0.032		0.032
	0.80	1.80	2.20	0.020			0.060		0.040		0.040
Pit Entry Level	0.90	1.90	2.30	0.021			0.063		0.042		0.042
	1.00	2.00	2.40	0.025	0.048		0.219		0.146		0.146
	1.10	2.10	2.50		0.228		0.684		0.456		0.456
Footpath / Weir Level	1.20	2.20	2.60		0.301	0.00	0.903	0.000	0.602	0.000	0.602
	1.25	2.25	2.65		0.308	3.38	4.307	4.429	5.045	4.352	4.660
	1.30	2.30	2.70		0.317	9.57	10.520	12.525	13.159	12.310	12.627

Basin A is designed to have 3 Typical Outlet structures plus weir flow over 178m footpath at RL 2.2m

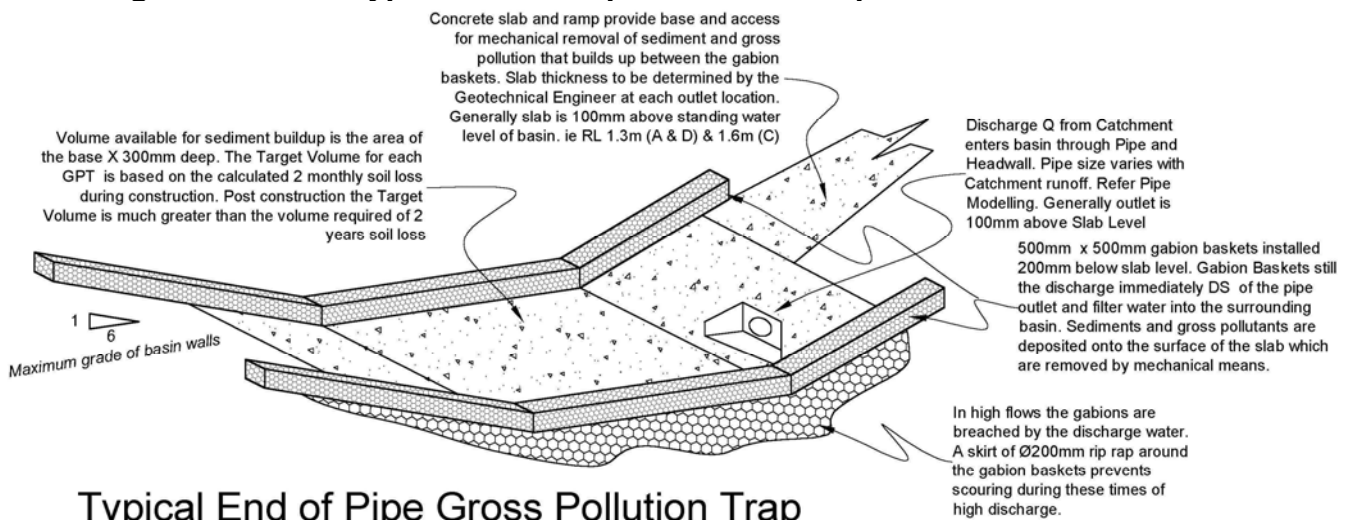
Basin C is designed to have 2 Typical Outlet structures plus weir flow over 233m footpath at RL 2.6m

Basin D is designed to have 2 Typical Outlet structures plus weir flow over 233m footpath at RL 2.2m

4.6 End of Pipe Sediment Traps

Where the piped drainage system enters the pond system a series of end of pipe sediment traps are to be constructed. These sediment and gross pollution traps consist of a concrete slab on the base and sides of the pond with gabion baskets on either side. Runoff exiting the piped drainage system is steadied or stilled by the small volume between the gabion walls. The runoff is then filtered through the gabion basket and into the pond. Any build up of sediment or litter is removed by mechanical means at regular intervals. This is further discussed in the Soil and Water Management Plan below. The Sketch below described a typical end of Pipe Gross Pollution Trap.

Figure 9 Sketch Typical End of Pipe Sediment Trap



Typical End of Pipe Gross Pollution Trap

The design volumes for the end of pipe Gross Pollution Traps GPT's has been calculated using the Revised Universal Soil Loss Equation for each Pipe that enters the basin system. The detailed calculations can be found in the Soil and Water Management Plan.

Summary of GPT target Volumes.

Basin	Catchment Pipe Outlet	Sub Catchment Area Ha	Design Volume m ³
A Eastern	P2B/10	1.26	4
	P2F/6	3.37	16
	P2J/5	1.63	5
	P2N/3	1.69	5
C South- Western	P3A/7	1.83	5
	P3C/10	3.31	9
D North- Western	P3H/5	1.27	4
	P3L/4	1.22	1
	P3N/2	0.76	0.3
	P3P/1	0.44	0.2

5.0 Pond Details

The catchments for the various pond systems described below are shown on Figure 7 above. Generally the ponds are based on a limit of 1.2m depth of water. This depth is made up of a permanent 200mm depth of water and 1 metre depth of storage.

A weir structure, made up of a footpath set at the ground level, is in essence the design Top Water Level (TWL) and is located on the downstream edge of each pond system. The long length of these weirs means that any depth of flow in a major storm event is limited. Pathways may be cut by overflow from the ponds but it will only be for short periods of time. A calculation of the possible depth of flow over the footpath weir is given in Section 6 pp 39.

It is considered that the infrequency of the overtopping does not represent dangerous or unacceptable nuisance to the amenity of the pathways in these locations.

The side slopes of the pond systems have been maintained at a maximum of 1 Vertical to 6 Horizontal for safety reasons. It is envisioned that the pond systems will be planted out with water based plant species in the base, and the batters planted out with shrubs and fire retardant bushes such as lilypilly etc.

The location of the end of pipe sediment traps is depicted on the sketches of the individual ponds below. Generally the sediment traps are located at the points where roadways will be designed to have low points where the piped drainage system will be collected and discharged into the pond systems. The 1H : 6V slope is considered an easy slope for mechanical maintenance and removal of any build up of litter and sediment collected by the sediment traps.

The detailed survey of the locations of the pond systems has been done. This data is sufficient to pick up the shape and level of the land sufficiently to provide conceptual design for the pond systems. Minor variations may occur at the detailed design phase of this process.

The pond outlets are typically located to provide for a maximum distance for the water to traverse from the inlet structure to the outlet structure. The elongated nature of these ponds is designed to emulate natural levee ponds that occur around coastal river systems.

5.1 Concept Pond A Details - Eastern

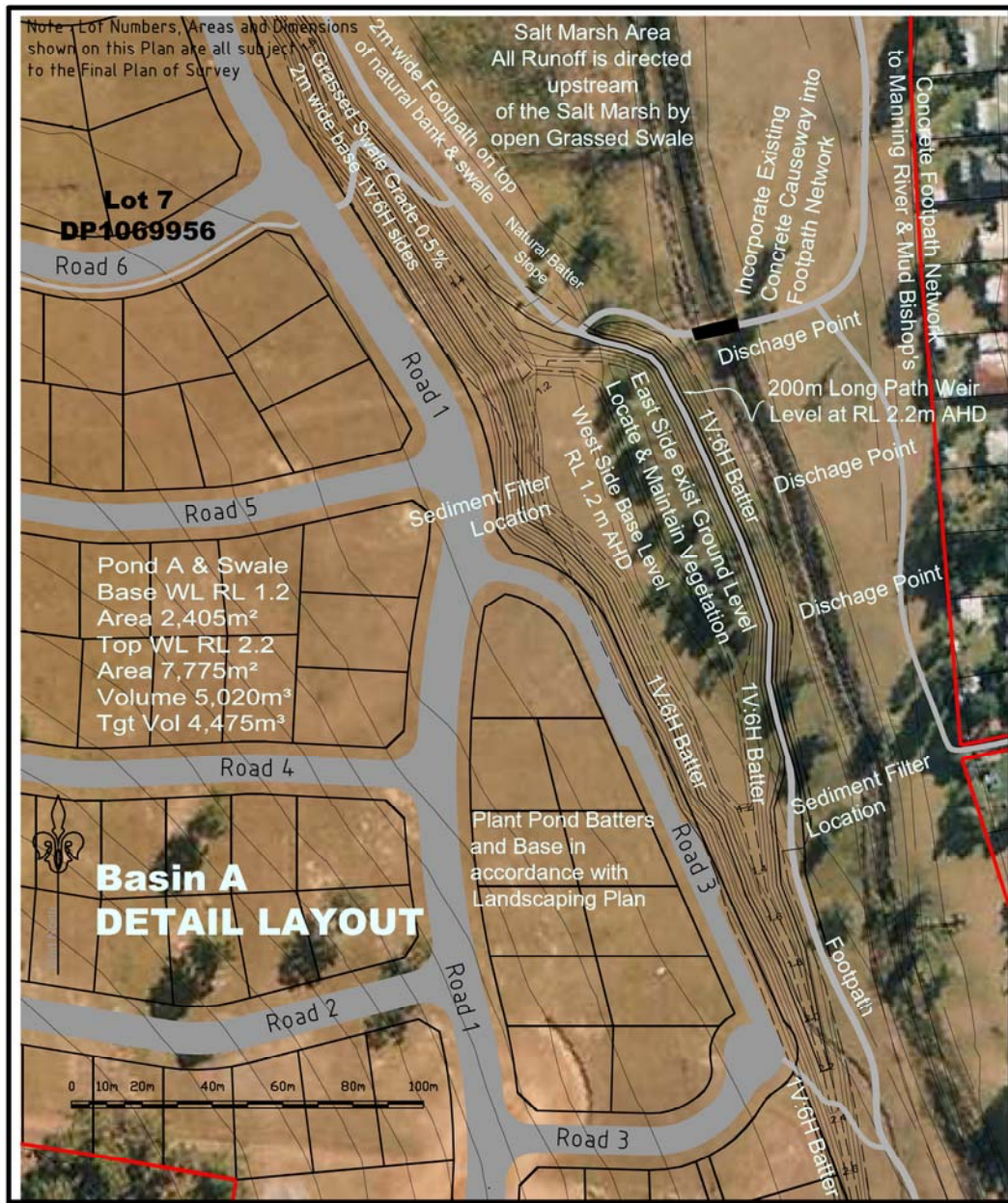
The base of Pond A will have two different levels.

The western side the base will be excavated to provide for free water 200mm deep at RL 1.2m AHD.

The eastern side will remain at the existing ground level in order to retain the existing vegetation. The level of the ground on the eastern section is in the range of 1.2m to 1.4m AHD. A bund top RL 2.2m (0.8 to 0.9 metres high) will be constructed. The bund will be shaped to pass between the existing vegetation as shown.

The location of the vegetation will define the final shape.

Figure 10 Detail Sketch of Pond A



The northern end of Pond A picks up the swale that collects runoff from the northern reaches of the development. The base of the swale has a very gentle slope of 0.5% grade.

However because of the length of the run the swale becomes quite deep (about 1.4m) by the time it reaches the northern end of the pond system.

The design location of the footpath to the north of Basin A is between the swale and the existing natural batter. The footpath location has been set out on the ground and marked. The marks have then been located by survey. The location chosen provides from 4 to 6 metres of flat land above the top of bank. The 2m wide concrete footpaths will be centrally located.

The footpath that surrounds the pond system is situated along the top of a low bund that creates the impoundment. The design level of the 2m wide footpaths will be RL 2.2metres.

The footpath is designed to be the weir control for the pond system. Based on the conservative rainfall and runoff figures under Greater Taree City Councils AUSPEC 1 the footpath will be broached every rarely.

Concept Pond A Stage / Storage Volume / Discharge Table

Level m	Area m ²	Vol m ³	Σ Vol m ³	Discharge Q m ³ /s	Description
1.20	2405		0	0.000	Standing Water Level 0.2m deep
		537			
1.40	2967		537	0.039	
		774			
1.60	4773		1311	0.048	
		1041			
1.80	5636		2352	0.060	
1.90		1226		0.063	Water Quality Storage level
2.00	6622		3578	0.219	
2.10		1440		0.684	
2.20	7775		5018	0.903	Footpath Weir Level
		395			
2.25	8030		5413	4.307	50mm over footpath
		408			
2.30	8290		5821	10.520	100mm over footpath

5.2 Concept Pond C Details - South-Western

Pond C is designed to have a weir outlet and TWL at the existing ground level of RL2.6m AHD. The Base water Level will be RL 1.6m AHD.

The Concept Layout of Pond C is depicted on Figure 12 below. The basin has inlet structures at either end of the pond system. These will be designed as described in section 4.6 above. The basin is designed to have two outlet structures as shown in the concept plan.

Figure 11 Detail Sketch of Pond C



Concept Pond C is constructed entirely by excavation of the existing natural surface.

The footpath than creates the downstream limit of the pond is designed to follow along the RL 2.6m contour. Variations in the existing natural contours may create minor changes in the layout of the pond at construction.

Concept Pond C Stage / Storage Volume / Discharge Table

Level m	Area m ²	Vol m ³	Vol m ³	Discharge Q m ³ /s	Description
1.60	1406		0	0.000	Standing Water Level 0.2m deep
		339			
1.80	1988		339	0.026	
		455			
2.00	2566		795	0.032	
		571			
2.20	3146		1366	0.040	
2.30		690		0.042	Water Quality Storage level
2.40	3753		2056	0.146	
2.50		814		0.456	
2.60	4390		2870	0.602	Footpath Weir Level
		223			
2.65	4542		3094	5.045	50mm over footpath
		231			
2.70	4694		3324	13.159	100mm over footpath

5.3 Concept Pond D Details - North-Western

Pond D is at the northwestern edge of the development area. Pond D is designed to have a weir outlet and TWL at the existing ground level of RL2.2m AHD. The Base Water Level (BWL) will be RL 1.2m AHD.

Like Pond A Pond C receives the grassed swale that collects urban runoff from the northern limits of the development and grades at 0.5% from the existing ground level to the base of the pond. Similarly because of the length of run of the swale it becomes quite deep by the time it reaches the northern end of the pond.

Figure 12 Detail Sketch of Pond D



The swale, as can be seen in the Figure 13 above, directs the runoff into the pond system. The discharge points for Pond D is upstream of the salt marsh communities and directs the treated fresh water to the channel effectively bypassing the salt marshes.

Concept Pond D will discharge into the down stream environs through two outlet structures as described in Section 4.5 above. The Concept pond system will have a resultant Stage, Storage Volume, Discharge regime as shown in the table below.

Concept Pond D Stage / Storage Volume / Discharge Table

Level m	Area m ²	Vol m ³	Vol m ³	Discharge Q m ³ /s	Description
1.20	1467		0	0.000	Standing Water Level 0.2m deep
		333			
1.40	1867		333	0.026	
		430			
1.60	2432		763	0.032	
		555			
1.80	3114		1318	0.040	
1.90		695		0.042	Water Quality Storage level
2.00	3839		2013	0.146	
2.10		847		0.456	
2.20	4628		2860	0.602	Footpath Weir Level
		236			
2.25	4814		3096	4.660	50mm over footpath
		245			
2.30	5000		3341	12.627	100mm over footpath

It should be noted that the contour interval of 0.2m depicted on all these concept pond plans represents only a small difference in height. The distance between the contours being 1.2m on a 1V : 6H slope actually represents a shallow grade that can easily be negotiated by ride on lawn mowers etc and is considered a safe slope for construction of pond systems.
(AR&R 1987 Sec 14.10.4)

6.0 Hydraulic Modelling

The stormwater pipe system has been developed for the preliminary concept plan and has been modelled using the ILSAX program.

This stormwater layout provides logical locations for pits and pipes that drain the development. Based on the stormwater layout the rainfall catchments of the subject land have been divided into smaller sub-catchments. In essence the sub-catchments represent stormwater pits and the water entering the piped drainage system.

The sub-catchments are modelled for impervious and pervious area based on Greater Taree City Councils AUSPEC 1 Design Manual the slopes and gradients existing on the land and expected of the road and drainage system.

Rainfall on the catchment is modelled over a number of Storm Events of different durations for each Frequency modelled. In this instance the Frequency modelled was Average Recurrence Interval (ARI) 5 years and ARI 100 years.

This information is contained in the Rainfall Files used.

At the ARI 5 year frequency 15 storms were modelled with durations from 5 min to 48 hours.

At the ARI 100 year frequency 15 storms were modelled with durations from 5 min to 12 hrs.

The catchment criteria used in the modelling includes

Temporal Patterns used are for the SE Coastal Zones (*Fig 3.1 AR&R 1987*)

Rainfall Intensities for Old Bar (*Refer Fig 5 IFD Chart Old Bar*)

Impervious Area Initial Loss = 1mm

Pervious (or grassed) Area Initial Loss = 5mm

Soil Type = 2.9 indicating slow infiltration rates in the soils.

Antecedent Moisture Condition = 2.5 indicating a median moisture condition exists at the time of the rainfall. (Approximately 15mm total rainfall in the 5 days preceding the storm)

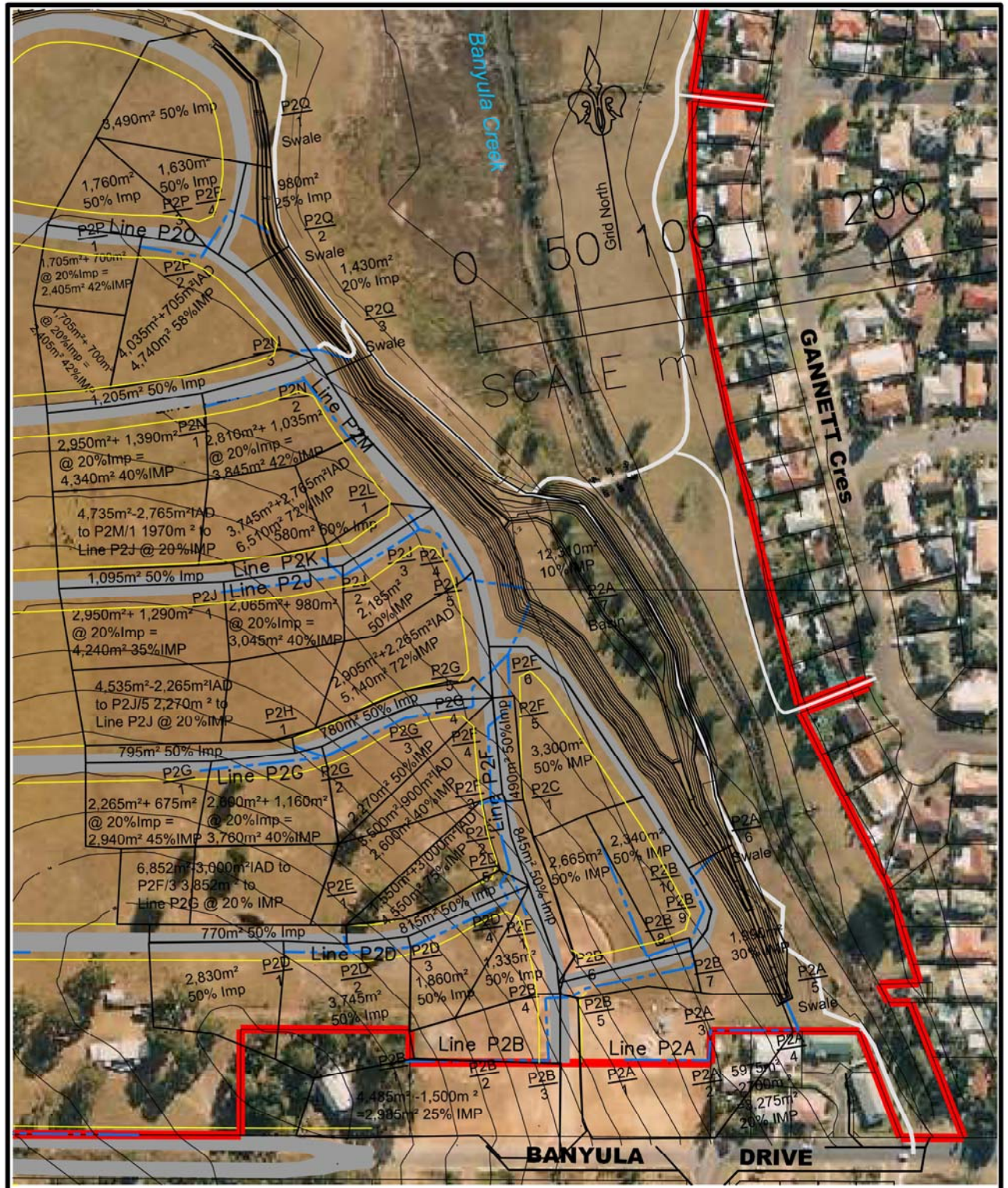
The Stormwater pipes are modelled using the Pipe Files.

The Pipe File nominates and models each sub catchments including percentage impervious, the slope and runoff characteristics.

At the downstream end of each sub catchment Pipe File provides the pipe length, size and grade of each section or reach within the piped drainage system. Where a swale is designed it also specifies and models the size, depth runoff characteristics and grade of the swale.

ILSAX modelling produces both extensive and summarised discharge analysis for each Rainfall Frequency modelled. The summarised version of the ARI 5 year analysis is provided below

**Figure 13 Eastern Pond A Detailed Modelling Plan
 Preliminary Layout Only**



Eastern Pond A Summary of Flows

Precinct 2A Eastern Basin A North Old Bar (Pipe File)

PEAK FLOWS AMONG RUNOFFS FROM 15 RAINFALL PATTERNS ARI 5year Storms

(The following lines give Branch and Reach names; Maximum surface flow arriving at an entry point or pit, Maximum flow in the pit before routing through downstream reach and Maximum surface overflow (m³/s); Diameter (mm) and Capacity (m³/s) for circular pipe reaches. Other information is given for non-circular reaches - refer to the program code for details.)

P2A	1	.012	.012	.000	150.	.042
P2A	2	.025	.037	.000	250.	.084
P2A	3	.000	.037	.000	250.	.109
P2A	4	.037	.074	.000	250.	.075
P2A	5	.064	.137	.000	0.	.028
Swale		.044	2.000	.167		
P2B	1	.012	.012	.000	150.	.044
P2B	2	.025	.037	.000	250.	.169
P2B	3	.070	.107	.000	375.	.283
P2B	4	.049	.153	.000	375.	.245
P2B	5	.025	.178	.000	375.	.245
P2B	6	.000	.177	.000	375.	.426
P2B	7	.049	.223	.000	450.	.396
P2B	8	.076	.298	.000	450.	.323
P2B	9	.000	.296	.000	450.	.323
P2B	10	.083	.376	.000	525.	.484
P2A	6	.057	.537	.000	0.	.109
Swale		.097	2.000	.167		
P2D	1	.078	.078	.000	375.	.347
P2E	1	.028	.028	.000	375.	.245
P2D	2	.000	.105	.000	375.	.440
P2D	3	.108	.199	.014	375.	.531
P2D	4	.064	.261	.000	450.	.458
P2D	5	.029	.290	.000	450.	.458
P2F	1	.042	.042	.000	375.	.283
P2F	2	.000	.330	.000	450.	.561
P2C	1	.030	.030	.000	375.	.245
P2F	3	.164	.480	.042	450.	.606
P2F	4	.093	.559	.006	525.	.768
P2F	5	.018	.576	.000	525.	.768
P2G	1	.080	.080	.000	375.	.329
P2H	1	.029	.029	.000	375.	.317
P2G	2	.107	.200	.014	375.	.539
P2G	3	.000	.199	.000	375.	.375
P2G	4	.068	.266	.000	450.	.512
P2G	5	.028	.293	.000	450.	.512
P2F	6	.086	.950	.000	600.	1.090
P2J	1	.107	.093	.013	375.	.329
P2K	1	.039	.039	.000	375.	.317
P2J	2	.081	.212	.001	375.	.329
P2L	1	.021	.021	.000	375.	.245
P2J	3	.063	.293	.000	450.	.532
P2J	4	.000	.293	.000	450.	.396
P2J	5	.173	.463	.000	525.	.594
P2N	1	.116	.098	.018	375.	.200
P2M	1	.215	.215	.000	450.	.323
P2N	2	.123	.433	.000	525.	.484
P2N	3	.031	.463	.000	525.	.484
P2P	1	.058	.058	.000	375.	.200
P2P	2	.138	.195	.000	375.	.200
P2P	3	.046	.240	.000	450.	.323
P2P	4	.042	.282	.000	450.	.323
P2Q	1	.091	.091	.000	0.	.019
Swale		.039	2.000	.167		
P2Q	2	.023	.385	.000	0.	.083
Swale		.091	2.000	.167		
P2Q	3	.032	.821	.000	0.	.184
Swale		.141	2.000	.167		
P2A	7	.344	2.640	.000	0.	.628
Pond		.267	2.000	.167		

Western Pond C Summary of Flows

ILSAX Precinct 2A South-Western Basin - North Old Bar (Pipe File)
PEAK FLOWS AMONG RUNOFFS FROM 15 RAINFALL PATTERNS ARI 5year Storms
(The following lines give Branch and Reach names; Maximum surface flow arriving at an entry point or pit, Maximum flow in the pit before routing through downstream reach and Maximum surface overflow (m3/s); Diameter (mm) and Capacity (m³/s) for circular pipe reaches. Other information is given for non-circular reaches - refer to the program code for details.)

P3A	1	.090	.085	.005	375.	.317
P3A	2	.106	.177	.013	375.	.370
P3A	3	.074	.250	.000	375.	.317
P3B	1	.045	.045	.000	375.	.317
P3B	2	.031	.076	.000	375.	.347
P3A	5	.000	.325	.000	375.	.492
P3A	6	.083	.405	.001	450.	.688
P3A	7	.029	.432	.000	450.	.512
P3A	8	.091	.522	.000	525.	.768
P3C	1	.077	.077	.000	375.	.364
P3D	1	.030	.030	.000	375.	.245
P3C	2	.081	.187	.001	375.	.364
P3C	3	.068	.253	.000	375.	.317
P3C	4	.016	.268	.000	375.	.317
P3C	5	.000	.267	.000	375.	.283
P3C	6	.123	.366	.021	450.	.512
P3C	7	.019	.385	.000	450.	.648
P3E	1	.096	.088	.008	375.	.364
P3F	1	.033	.033	.000	375.	.317
P3E	2	.070	.190	.000	375.	.426
P3E	3	.067	.255	.000	375.	.245
P3E	4	.012	.257	.000	375.	.245
P3C	8	.000	.628	.000	450.	.827
P3G	1	.145	.113	.033	375.	.245
P3C	9	.071	.810	.000	525.	.972
P3C	10	.111	.919	.000	600.	1.090
P3H	1	.083	.082	.002	375.	.329
P3J	1	.028	.028	.000	375.	.317
P3H	2	.083	.190	.002	375.	.375
P3H	3	.037	.225	.000	375.	.265
P3K	1	.079	.079	.000	375.	.265
P3H	4	.021	.323	.000	450.	.428
P3H	5	.050	.358	.000	450.	.428
P3A	9	.257	1.851	.000	0.	.398
Pond		.212	2.000	.167		
END						

Western Pond D Summary of Flows

ILSAX Precinct 2A North-West Basin North Old Bar (Pipe File)
 PEAK FLOWS AMONG RUNOFFS FROM 14 RAINFALL PATTERNS ARI 5year Storms
 (The following lines give Branch and Reach names; Maximum surface flow arriving at an entry point or pit, Maximum flow in the pit before routing through downstream reach and Maximum surface overflow (m3/s); Diameter (mm) and Capacity (m3/s) for circular pipe reaches. Other information is given for non-circular reaches - refer to the program code for details.)

P3L	1	.063	.063	.000	375.	.317
P3R	1	.026	.026	.000	375.	.317
P3L	2	.055	.142	.000	375.	.253
P3M	1	.037	.037	.000	375.	.245
P3L	3	.119	.296	.000	450.	.396
P3L	4	.055	.350	.000	450.	.512
P3N	1	.086	.083	.003	375.	.283
P3N	2	.128	.211	.000	450.	.512
P3Q	1	.101	.101	.000	0.	.022
Swale		.043	2.000	.167		
P3P	1	.112	.112	.000	375.	.317
P3Q	2	.028	.207	.000	0.	.051
Swale		.069	2.000	.167		
P3Q	3	.189	.791	.000	0.	.193
Pond		.145	2.000	.167		

END

From these peak output files we can see that that the maximum pipe size required in any of these catchments is Ø600mm in Catchment A - Reach P2F-6. This is considered a relatively small catchment and flow at 0.95m³/s.

During an ARI 5 year storm event depths of flow expected in the swales is not expected to exceed 150mm, which is considered safe.

Similarly Peak Flows have been modelled for ARI 100 year events. The Peak Flows entering the three basins is summarised below.

ARI 100 year Peak Flows entering Basins by ILSAX modelling

Basin	Peak Flow By ILSAX
Pond A Eastern	4.29 m ³ /s
Pond C South-Western	2.74 m ³ /s
Pond D North-Western	1.48 m ³ /s

To assess the safety of the footpath weir the ARI 100 year peak flow was used as the upper limit of discharge flowing over the weir. This assumes that all outlet structures are fully blocked and all discharge is over the weir.

The table indicates that the greatest depth of flow is 59mm and the velocity depth ratio indicates that the discharge would be safe.

Pond	Weir Length	Peak Flow	Depth of Flow	Velocity	V x D
Pond A Eastern	178m	4.29 m ³ /s	59mm	0.41m/s	0.024
Pond C South-Western	233m	2.74 m ³ /s	33mm	0.33m/s	0.012
Pond D North-Western	229m	1.48 m ³ /s	27mm	0.27m/s	0.006

7.0 Soil & Water Management Plan

In order to ascertain the soil types around this development, 12 boreholes have been dug over the site each providing a soil profile. In addition 5 laboratory BDR and 5 DCP tests were done and the soil analysed for its erosion potential. (Refer EGS Report A121006MCP2 dated 12/10/06)

E.G.S Chatham Laboratory carried out a site assessment for soil erosion on Old Bar Precinct 2A the location of the boreholes can be seen on, Figure 16. This assessment tested samples for the K factor of the soil at various locations around the site as shown in Figure 15.

The K factor is used in the Revised Universal Soil Loss Equation (RUSLE) to determine amount of erosion to be expected from the site.

The soil evaluation for erosion arrived at a stability factors of 0.16 to 0.21. We note that a conversion factor of $K' = 0.1317$ is used to convert this to SI units for the K Factor used in the Universal Soil Loss Equation.

The table below is a general summary of the soil profiles found on Site.

Hole No	Textural Classification	Soil Permeability	Soil Ph	International Clay Content	K' Factor
Bore Hole 1	Silty Clay	0.35 m/day	6.4	35%	0.017
Bore Hole 2	Silty Clay	0.36 m/day	6.2	31%	0.016
Bore Hole 3	Silty Clay	0.39 m/day	6.6	26%	0.021
Bore Hole 4	Silty Clay	0.40 m/day	6.5	27%	0.021
Bore Hole 5	Silty Clay	0.43 m/day	6.6	28%	0.018
Bore Hole 6	Silty Clay	0.43 m/day	7.0	25%	0.016


Figure 15 Site Evaluation for Erosion Test Report

E.G.S. CHATHAM LABORATORY
 ABN 98 750 367 147
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 ~ A MEMBER OF THE NATIONAL ASSOCIATION OF TESTING AUTHORITIES ~

PAGE No.: 1 OF 1.

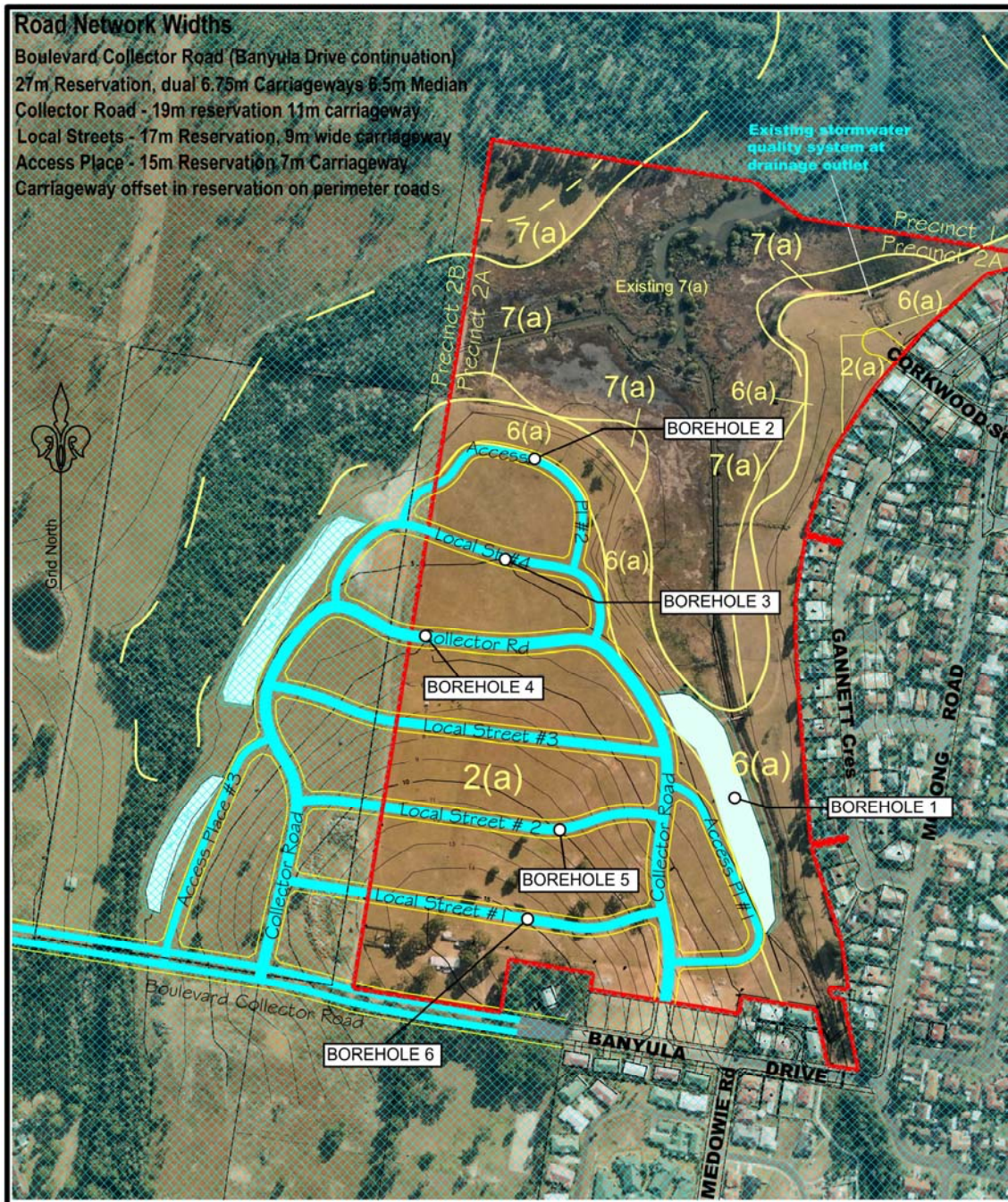
SITE EVALUATION FOR EROSION - TEST REPORT	
CLIENT: McGLASHAN CRISP & PARTNERS 117 VICTORIA STREET, TAREE NSW 2430	PROJECT No.: A121006MCP2
PROJECT: SITE ASSESSMENT FOR EROSION LOTS 7 DP 1069956, 95 DP 753149 AND 1/95 DP 48330, BANYULA DRIVE, OLD BAR NSW 2430	DATE TESTED: 12/10/2006

TEST No.:	1	2	3	4	5	6
SOIL PROFILE SAMPLE No.:	1A	2A	3A	4A	5A	6A
SAMPLE TAKEN; FROM (mm):	0	0	0	0	0	0
TO (mm):	500	500	500	500	500	500
TEXTURAL CLASSIFICATION:	SILTY CLAY	SILTY CLAY	SILTY CLAY	SILTY CLAY	SILTY CLAY	SILTY CLAY
SOIL STRUCTURE (PEDALITY):	HIGH	HIGH	HIGH	HIGH	HIGH	HIGH
SOIL PERMEABILITY (m / DAY) (P):	0.35	0.36	0.39	0.40	0.39	0.43
SOIL PERMEABILITY CATEGORY:	5A	5A	5A	5A	5A	5A
MOISTURE CONTENT (%):	12.8	14.6	12.3	10.3	8.8	9.4
LIQUID LIMIT (%):	31.7	29.4	28.5	27.7	27.4	27.2
PLASTIC LIMIT (%):	16.3	14.8	13.7	14.5	13.7	14.4
BULK DENSITY (t / m3):	1.71	1.67	1.69	1.70	1.71	1.69
SOIL pH:	6.4	6.2	6.6	6.5	6.6	7.0
ORGANIC MATTER CONTENT (%):	3.3	4.3	2.4	1.6	3.0	1.9
EMERSON CLASS:	5	5	5	5	5	5
PARTICLE SIZE DISTRIBUTION (-75 mm) (%):	---	---	---	---	---	---
(-53 mm) (%):	---	---	---	---	---	---
(-37.5 mm) (%):	---	---	---	---	---	---
(-26.5 mm) (%):	---	---	---	---	---	---
(-19 mm) (%):	---	---	---	---	---	---
(-13.2 mm) (%):	---	---	---	---	---	---
(-9.5 mm) (%):	---	---	---	---	---	---
(-6.7 mm) (%):	---	---	---	---	---	---
(-4.75 mm) (%):	---	---	---	---	---	---
(-2.36 mm) (%):	100	100	100	100	100	100
(-0.425 mm) (%):	85	82	75	78	78	80
(-0.075 mm) (%):	73	68	60	62	64	63
(-0.0135 mm) (%):	63	56	47	48	51	46
INTERNATIONAL CLAY CONTENT (%):	35	31	26	27	28	25
K FACTOR:	0.13	0.12	0.16	0.16	0.14	0.12
K':	0.017	0.016	0.021	0.021	0.018	0.016

APPROVED SIGNATORY:
 RON McCAFFREY
 LABORATORY MANAGER

 DATE: 02/11/2006

ISSUE DATE: 25/05/01. ISSUE No.: 4. FORM: 1547.

Figure 16 Location of Boreholes



7.1 Revised Universal Soil Loss Equation (RUSLE)

The Revised Soil Loss equation is used to calculate the predicted loss of material that may be eroded off the site during an average rainfall year.

The Equation is $A = R K L_s P C$ where: -

A = computed soil loss (tonnes/ha/yr)

R = rainfall erosivity factor

K = soil erodability factor

L_s = slope length / gradient factor

P = erosion control practice factor

C = ground cover and management factor

R = 3,500 - The rainfall erosivity factor is a measure of the ability of rainfall to cause erosion. The Rainfall Erosivity Factor for Site has been estimated to from the MUSSOC document. (Map 7 Hastings Appendix B pp B-85)

K = Geotechnical advice has evaluated the soil at various sites and determined the various K' values (E.G.S Chatham Labs).

L_s = The slope length / gradient factor is determines off Table A1 of the MUSSOC document pp A-5. The L_s factor has been determined by using the length of overland flow between roadways. This is used to model and simulate the road construction interrupting length of overland flow across the site.

P = 0.70 – The Erosion Control Factor is the ratio of soil loss from the nominated surface condition and the soil loss from land ploughed up and down the slope. The P factor of 0.70 is used to simulate the situation where only the roadways areas are being grossly disturbed during construction and, by virtue of the lot layout, the majority of the roadways and access tracks are/will be across the contour.

C = 0.32 - The C factor is the ratio of soil loss from land under specified crop conditions or mulch conditions to the corresponding loss from continually tilled, bare soil.

We note the Site under construction will be: -

65% basically undisturbed natural grasses C Factor <0.01

35% disturbed by road and drainage construction C Factor 1

(Reference MUSSOC Table A4 ppA-7)

7.2 Average Annual Soil Loss During Construction

The computed soil losses for the various sub-catchments of the Eastern basin during construction are as follow:

Catchment	Area (Ha)	R	P	C	LS	K	Soil Loss (m ³ /Yr)	Soil Loss (m ³ /2 mth)
P2B/10	1.26	3500	0.7	0.32	1.58	0.016	25	4
P2F/6	3.37	3500	0.7	0.32	2.02	0.018	96	16
P2J/5	1.63	3500	0.7	0.32	1.35	0.018	31	5
P2N/3	1.69	3500	0.7	0.32	1.03	0.021	29	5
Total Soil Loss (m³/Yr)							181	30

The computed soil losses for the various sub-catchments of the Southwestern basin during construction are as follow:

Catchment	Area (Ha)	R	P	C	LS	K	Soil Loss (m ³ /Yr)	Soil Loss (m ³ /2mth)
P3A/7	1.83	3500	0.7	0.32	1.4	0.016	32	5
P3C/10	3.31	3500	0.7	0.32	1.11	0.018	52	9
P3H/5	1.27	3500	0.7	0.32	1.4	0.018	25	4
Total Soil Loss (m³/Yr)							109	18

The computed soil losses for the various sub-catchments of the Northwestern basin during construction are as follow:

Catchment	Area (Ha)	R	P	C	LS	K	Soil Loss (m ³ /Yr)	Soil Loss (m ³ /2mth)
P3L/4	1.22	3500	0.7	0.32	0.58	0.016	9	1
P3N/2	0.76	3500	0.7	0.32	0.19	0.018	2	0
P3P/1	0.44	3500	0.7	0.32	0.16	0.021	1	0
Total Soil Loss (m³/Yr)							12	2

A monitoring & clearing program of 2 months and/or after each rainfall event is recommended during the construction and maintenance period.

In order to achieve this the minimum target volumes for the Gross Pollution Traps to be installed at the end of pipe outlets into the various basins will be approximated by the calculated 2-month soil loss from the tables above.

The removal of sediment and build up of gross pollutants during and construction and the project maintenance period is the responsibility of the Contractor.

7.3 Average Annual Soil Loss Post Construction

Post development, Old Bar Precinct 2A will have a vastly different soil loss regime this is simulated by and adjusting the variables in the Revised Universal Soil Loss Equation.

Variables R & K remain the same.

LS will decrease, simulating the installation of fences and the fact that now the runoff water is being directed into the Interallotment Drainage (IAD) system.

P = 0.01 reflecting the ratio between soil loss of “the nominated condition ploughed up and down the slope” and the soil loss one may expect from an urban lawn.

C = 0.01 reflecting the ratio between the soil loss from “continuously tilled, bare soil “ and the soil loss one may expect from a well established urban subdivision

The computed soil losses for the various sub-catchments of the Eastern basin post construction are as follow:

The computed soil losses for the various sub-catchments of the Southwestern basin post construction are as follow:

Catchment	Area (Ha)	R	P	C	LS	K'	Soil Loss (m ³ /Yr)	Soil Loss in 2 months (m ³)
P2B/10	1.26	3500	0.01	0.01	1.47	0.016	0.010	0.002
P2F/6	3.37	3500	0.01	0.01	1.7	0.018	0.036	0.006
P2J/5	1.63	3500	0.01	0.01	1.01	0.018	0.010	0.002
P2N/3	1.69	3500	0.01	0.01	0.71	0.021	0.009	0.001
Total Soil Loss (m³/Yr)							0.066	0.011

The computed soil losses for the various sub-catchments of the Southwestern basin post construction are as follow:

Catchment	Area (Ha)	R	P	C	LS	K'	Soil Loss (m ³ /Yr)	Soil Loss in 2 months (m ³)
P3A/7	1.83	3500	0.01	0.01	1.05	0.016	0.011	0.002
P3C/10	3.31	3500	0.01	0.01	0.81	0.018	0.017	0.003
P3H/5	1.27	3500	0.01	0.01	1.05	0.018	0.008	0.001
Total Soil Loss (m³/Yr)							0.036	0.006

The computed soil losses for the various sub-catchments of the Northwestern basin post construction are as follow:

Catchment	Area (Ha)	R	P	C	LS	K'	Soil Loss (m ³ /Yr)	Soil Loss in 2 months (m ³)
P3L/4	1.22	3500	0.01	0.01	0.43	0.016	0.003	0.000
P3N/2	0.76	3500	0.01	0.01	0.155	0.018	0.001	0.000
P3P/1	0.44	3500	0.01	0.01	0.15	0.021	0.000	0.000
Total Soil Loss (m³/Yr)							0.004	0.001

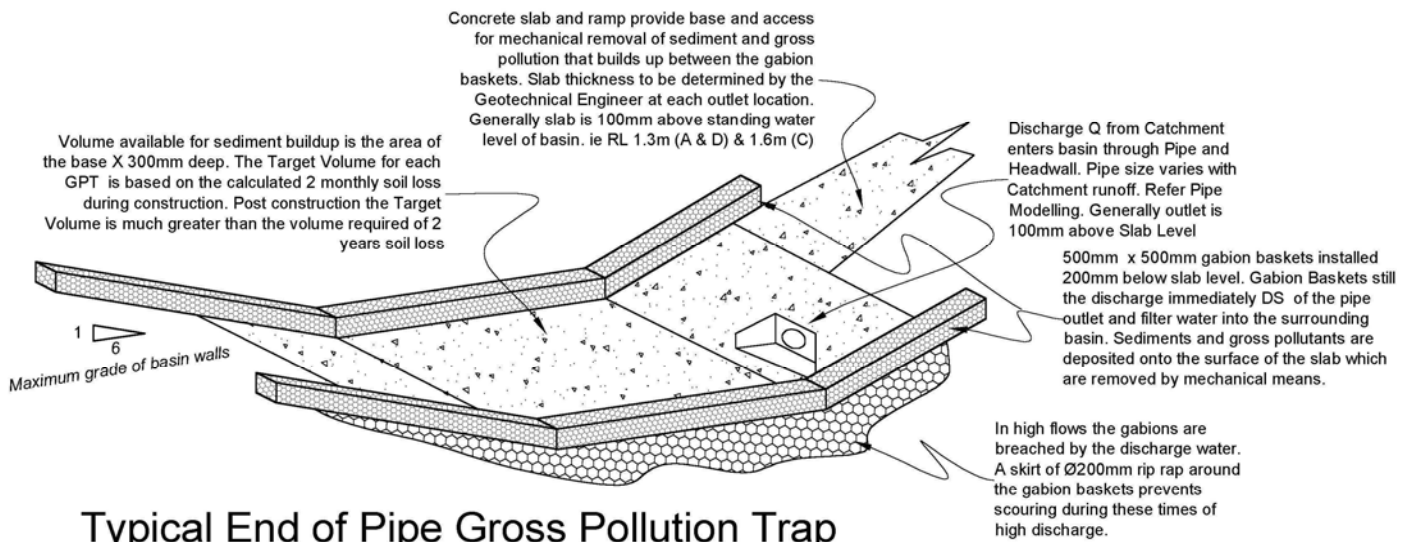
Whilst these numbers are extremely small never the less it reflects the fact that a continuing monitoring and clearing process at the end of pipe sediment control structures proposed for the development is needed.

Based on the information above a monitoring program that includes inspection and removal of build up will be required at a minimum of 12 monthly intervals. The gross pollution traps being sized for an entirely different soil loss regime means that removal works may not be required annually in accordance with the monitoring programme.

However when the maintenance work is required it will include removal and disposal of sediment build up at the end of pipe gross pollution trap plus the removal of any general litter, particularly floating flotsam such as plastic bottles etc that has been caught in the general surface area of the stormwater quality basin.

7.4 Design and Function of the Gross pollution Traps

A sketch of a typical Gross Pollution Trap is provided below.



Typical End of Pipe Gross Pollution Trap

Summary of the GPT Process

- 1) Runoff from the development is piped into the various basins and enters the basins via a series of gross pollution traps.
- 2) Water enters a closed portion of the basin with a concrete floor bounded by gabion baskets. (as in the sketch above) The closed section stills the runoff water and provides a point where larger particles settle out of the water train.
- 3) In most storm events water enters the main basin area by filtering through the gabion baskets leaving behind the gross pollutants that may come off the development.
- 4) When larger storms are encountered the volume between the gabions is overcome and water spills into the basin over the gabion walls. A Skirt of Ø200mm Rip Rap prevents scour when the water passes over the baskets.
- 5) The concrete slab of the GPT is provided with an access ramp. Periodically the build up of sediment is to be removed by mechanical means such as a bobcat or mini excavator.
- 6) The GPT should be monitored at least annually for sediment and gross pollutant build up. The RUSLE indicates that mechanical removal of sediment build up should occur at a period of 2 years or more once building and development work is completed in the catchment.
- 7) During the construction and construction maintenance period (GTCC maintenance period is 12 months) the GPT's are to be inspected and monitored every 2 months and/or after each rainfall event.

7.6 Conclusions relating to Soil and Water Management.

- 1) The system designed will cater for the expected increases in soil erosion during the construction period and during the future building processes on the site. An inspection & monitoring programme of every 2 months or after each rainfall event is recommended during the construction and construction maintenance period.
- 2) The system will also cater for the long-term entrapment and removal of gross pollutants from the site and a yearly monitoring and removal regime is recommended.
- 3) The site does not require a Acid Sulphate Soils Management Plan

APPENDIX 1

Calculation of Total Volume of Runoff Calculation

August 2006

Prepared by

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Location		Bob's Hill Catchment A						McGlashan & Crisp File F929-Job No 15804														
% Impervious		50%																				
Area (ha)		11.57						Runoff Volume Table														
ARI Years		1		2		5		10		20		50		100								
C dev		0.6		0.65		0.70		0.75		0.78		0.85		0.89								
						Initial Loss		347.1 m ³														
						Continuing Loss		1.25 mm/hr														
DEVELOPED	mins	Rainfall I	Discharge Qd	Volume Runoff	Rainfall I	Discharge Qd	Volume Runoff	Rainfall I	Discharge Qd	Volume Runoff	Rainfall I	Discharge Qd	Volume Runoff	Rainfall I	Discharge Qd	Volume Runoff	Rainfall I	Discharge Qd	Volume Runoff	Rainfall I	Discharge Qd	Volume Runoff
		mm/hr	m ³ /s	m ³	mm/hr	m ³ /s	m ³	mm/hr	m ³ /s	m ³	mm/hr	m ³ /s	m ³	mm/hr	m ³ /s	m ³	mm/hr	m ³ /s	m ³	mm/hr	m ³ /s	m ³
	5	96.00	1.827	201	122.20	2.527	411	152.07	3.393	671	168.96	4.043	866	191.97	4.781	1,087	221.77	6.024	1,460	244.24	6.950	1,738
	6	89.90	1.709	268	114.40	2.364	504	142.62	3.180	798	158.49	3.790	1,017	180.13	4.484	1,267	208.13	5.652	1,687	229.26	6.522	2,001
	7	84.80	1.611	330	108.00	2.230	590	136.21	3.036	928	151.39	3.619	1,173	172.08	4.282	1,452	198.87	5.399	1,920	219.08	6.231	2,270
	8	80.50	1.528	386	102.60	2.117	669	129.79	2.892	1,041	144.29	3.448	1,308	164.04	4.081	1,612	189.60	5.145	2,123	208.90	5.940	2,504
	9	76.80	1.457	440	97.80	2.017	742	123.38	2.748	1,137	137.19	3.277	1,422	155.99	3.879	1,748	180.34	4.892	2,295	198.71	5.648	2,703
	10	73.60	1.395	490	93.70	1.931	812	116.96	2.603	1,215	130.09	3.106	1,516	147.94	3.677	1,859	171.07	4.639	2,436	188.53	5.357	2,867
	11	70.60	1.337	536	90.00	1.854	877	112.60	2.505	1,306	125.26	2.989	1,626	142.46	3.540	1,989	164.76	4.467	2,601	181.59	5.158	3,057
	12	68.00	1.287	580	86.70	1.785	938	108.24	2.407	1,386	120.42	2.872	1,721	136.98	3.403	2,103	158.44	4.294	2,745	174.64	4.960	3,224
	13	65.70	1.243	622	83.70	1.722	996	104.81	2.330	1,470	116.62	2.781	1,822	132.68	3.295	2,223	153.48	4.159	2,897	169.17	4.803	3,399
	14	63.50	1.200	661	80.90	1.664	1,051	101.39	2.253	1,545	112.83	2.690	1,912	128.37	3.187	2,330	148.52	4.023	3,032	163.70	4.647	3,556
	15	61.50	1.162	699	78.40	1.612	1,103	97.96	2.176	1,611	109.03	2.598	1,991	124.07	3.079	2,424	143.56	3.888	3,152	158.28	4.492	3,695
	16	59.70	1.127	735	76.10	1.564	1,154	95.29	2.116	1,684	106.07	2.527	2,078	120.71	2.995	2,528	139.69	3.782	3,284	154.03	4.370	3,848
	17	58.00	1.094	769	74.00	1.520	1,203	92.63	2.056	1,750	103.12	2.455	2,158	117.36	2.911	2,622	135.82	3.676	3,403	149.77	4.248	3,986
	18	56.50	1.065	804	72.00	1.478	1,249	89.96	1.996	1,808	100.16	2.384	2,228	114.00	2.826	2,705	131.95	3.570	3,509	145.52	4.127	4,110
	19	55.00	1.036	834	70.10	1.438	1,293	87.73	1.946	1,871	97.69	2.325	2,303	111.20	2.756	2,795	128.72	3.482	3,623	141.97	4.025	4,242
	20	53.70	1.011	867	68.40	1.403	1,336	85.50	1.895	1,927	95.22	2.265	2,371	108.40	2.686	2,876	125.49	3.394	3,726	138.41	3.923	4,361
	30	43.72	0.819	1,127	55.61	1.136	1,697	69.66	1.539	2,423	77.64	1.841	2,967	99.10	2.453	4,068	114.77	3.101	5,235	126.61	3.586	6,107
	45	35.07	0.652	1,414	44.63	0.906	2,100	56.00	1.232	2,979	62.47	1.476	3,637	88.44	2.186	5,554	102.47	2.765	7,119	113.08	3.199	8,290
Hrs	1	29.79	0.550	1,634	37.90	0.766	2,409	47.70	1.045	3,415	53.30	1.255	4,170	60.80	1.493	5,027	70.60	1.895	6,473	78.00	2.195	7,556
	2	19.62	0.354	2,203	25.08	0.498	3,237	31.80	0.687	4,601	35.70	0.830	5,632	40.90	0.994	6,809	47.80	1.272	8,809	52.90	1.477	10,290
	6	9.95	0.168	3,277	12.80	0.241	4,865	16.50	0.343	7,063	18.70	0.421	8,738	21.60	0.510	10,672	25.40	0.660	13,903	28.40	0.777	16,427
	9	7.75	0.125	3,714	9.99	0.183	5,569	13.00	0.264	8,218	14.80	0.327	10,235	17.10	0.397	12,526	20.20	0.518	16,426	22.50	0.608	19,346
	12	6.49	0.101	4,018	8.38	0.149	6,087	10.90	0.217	9,032	12.50	0.271	11,368	14.50	0.332	14,002	17.10	0.433	18,358	19.10	0.511	21,710
	18	5.05	0.073	4,401	6.53	0.110	6,800	8.60	0.165	10,368	9.80	0.206	13,008	11.40	0.254	16,141	13.50	0.335	21,338	15.20	0.399	25,509
	24	4.21	0.057	4,584	5.46	0.088	7,252	7.20	0.134	11,218	8.20	0.168	14,127	9.60	0.209	17,738	11.40	0.277	23,610	12.80	0.330	28,197
	30	3.65	0.046	4,651	4.74	0.073	7,527	6.30	0.114	11,923	7.20	0.143	15,142	8.40	0.179	19,011	10.00	0.239	25,468	11.20	0.285	30,390
	48	2.67	0.027	4,385	3.47	0.046	7,667	4.60	0.075	12,676	5.30	0.098	16,522	6.20	0.124	21,095	7.40	0.168	28,684	8.40	0.205	34,993
	72	1.99	0.014	3,352	2.59	0.028	6,909	3.50	0.051	12,773	4.00	0.066	16,834	4.70	0.086	22,070	5.60	0.119	30,455	6.40	0.147	37,835
		Qd=Cdev x (I(mm/hr)-Continuing Loss) x Area/360						Initial Losses Grass = 5mm ; Concrete = 1mm mean 3mm x 112055m ² IL =						347.1 m ³								
		Vmin = Qd x time(min) x 60 - Initial Loss						Continuing Losses Grass =2mm/hr Concrete = 0.5mm/hr Mean CL = 1.25mm/hr														
		Vhrs = Qd x time(hrs) x 3,600 - Initial Loss																				

Location	Oyster Reach Catchment B																					
% Impervious	50%																					
Area (ha)	6.06																					
ARI Years	McGlashan & Crisp File F929-Job No 15804																					
C dev	Runoff Volume Table																					
	1			2			5			10			20			50			100			
	0.6			0.65			0.70			0.75			0.78			0.85			0.89			
							Initial Loss 181.8 m ³															
							Contiuing Loss 1.25 mm/hr															
DEVELOPED	Rainfall I mm/hr	Discharge Qd m ³ /s	Volume Runoff m ³	Rainfall I mm/hr	Discharge Qd m ³ /s	Volume Runoff m ³	Rainfall I mm/hr	Discharge Qd m ³ /s	Volume Runoff m ³	Rainfall I mm/hr	Discharge Qd m ³ /s	Volume Runoff m ³	Rainfall I mm/hr	Discharge Qd m ³ /s	Volume Runoff m ³	Rainfall I mm/hr	Discharge Qd m ³ /s	Volume Runoff m ³	Rainfall I mm/hr	Discharge Qd m ³ /s	Volume Runoff m ³	
mins																						
5	96.00	0.957	105	122.20	1.323	215	152.07	1.777	351	168.96	2.117	453	191.97	2.504	569	221.77	3.155	765	244.24	3.640	910	
6	89.90	0.895	141	114.40	1.238	264	142.62	1.666	418	158.49	1.985	533	180.13	2.349	664	208.13	2.960	884	229.26	3.416	1,048	
7	84.80	0.844	173	108.00	1.168	309	136.21	1.590	486	151.39	1.896	614	172.08	2.243	760	198.87	2.828	1,006	219.08	3.263	1,189	
8	80.50	0.800	202	102.60	1.109	350	129.79	1.515	545	144.29	1.806	685	164.04	2.137	844	189.60	2.695	1,112	208.90	3.111	1,311	
9	76.80	0.763	230	97.80	1.056	389	123.38	1.439	595	137.19	1.716	745	155.99	2.032	915	180.34	2.562	1,202	198.71	2.958	1,416	
10	73.60	0.731	257	93.70	1.012	425	116.96	1.363	636	130.09	1.627	794	147.94	1.926	974	171.07	2.430	1,276	188.53	2.806	1,502	
11	70.60	0.700	280	90.00	0.971	459	112.60	1.312	684	125.26	1.566	852	142.46	1.854	1,042	164.76	2.340	1,362	181.59	2.702	1,601	
12	68.00	0.674	304	86.70	0.935	491	108.24	1.261	726	120.42	1.505	901	136.98	1.782	1,101	158.44	2.249	1,438	174.64	2.598	1,689	
13	65.70	0.651	326	83.70	0.902	522	104.81	1.220	770	116.62	1.457	954	132.68	1.726	1,164	153.48	2.178	1,517	169.17	2.516	1,780	
14	63.50	0.629	346	80.90	0.872	550	101.39	1.180	809	112.83	1.409	1,002	128.37	1.669	1,220	148.52	2.107	1,588	163.70	2.434	1,863	
15	61.50	0.609	366	78.40	0.844	578	97.96	1.140	844	109.03	1.361	1,043	124.07	1.613	1,270	143.56	2.036	1,651	158.28	2.353	1,936	
16	59.70	0.590	385	76.10	0.819	604	95.29	1.108	882	106.07	1.323	1,089	120.71	1.569	1,324	139.69	1.981	1,720	154.03	2.289	2,016	
17	58.00	0.573	403	74.00	0.796	630	92.63	1.077	916	103.12	1.286	1,130	117.36	1.525	1,373	135.82	1.925	1,782	149.77	2.225	2,088	
18	56.50	0.558	421	72.00	0.774	654	89.96	1.045	947	100.16	1.249	1,167	114.00	1.480	1,417	131.95	1.870	1,838	145.52	2.161	2,153	
19	55.00	0.543	437	70.10	0.753	677	87.73	1.019	980	97.69	1.218	1,206	111.20	1.444	1,464	128.72	1.824	1,897	141.97	2.108	2,222	
20	53.70	0.530	454	68.40	0.735	700	85.50	0.993	1,009	95.22	1.186	1,242	108.40	1.407	1,506	125.49	1.778	1,951	138.41	2.055	2,284	
30	43.72	0.429	590	55.61	0.595	889	69.66	0.806	1,269	77.64	0.964	1,554	99.10	1.285	2,131	114.77	1.624	2,742	126.61	1.878	3,199	
45	35.07	0.342	740	44.63	0.475	1,100	56.00	0.645	1,560	62.47	0.773	1,905	88.44	1.145	2,909	102.47	1.448	3,729	113.08	1.675	4,342	
Hrs	29.79	0.288	856	37.90	0.401	1,262	47.70	0.547	1,789	53.30	0.657	2,184	60.80	0.782	2,633	70.60	0.992	3,390	78.00	1.150	3,958	
2	19.62	0.186	1,154	25.08	0.261	1,696	31.80	0.360	2,410	35.70	0.435	2,950	40.90	0.521	3,567	47.80	0.666	4,614	52.90	0.774	5,390	
6	9.95	0.088	1,716	12.80	0.126	2,548	16.50	0.180	3,700	18.70	0.220	4,577	21.60	0.267	5,590	25.40	0.346	7,282	28.40	0.407	8,604	
9	7.75	0.066	1,945	9.99	0.096	2,917	13.00	0.138	4,304	14.80	0.171	5,361	17.10	0.208	6,561	20.20	0.271	8,603	22.50	0.318	10,133	
12	6.49	0.053	2,105	8.38	0.078	3,188	10.90	0.114	4,730	12.50	0.142	5,954	14.50	0.174	7,334	17.10	0.227	9,615	19.10	0.267	11,371	
18	5.05	0.038	2,305	6.53	0.058	3,562	8.60	0.087	5,430	9.80	0.108	6,813	11.40	0.133	8,454	13.50	0.175	11,176	15.20	0.209	13,361	
24	4.21	0.030	2,401	5.46	0.046	3,798	7.20	0.070	5,876	8.20	0.088	7,399	9.60	0.110	9,291	11.40	0.145	12,366	12.80	0.173	14,769	
30	3.65	0.024	2,436	4.74	0.038	3,942	6.30	0.060	6,245	7.20	0.075	7,931	8.40	0.094	9,957	10.00	0.125	13,340	11.20	0.149	15,917	
48	2.67	0.014	2,296	3.47	0.024	4,016	4.60	0.039	6,639	5.30	0.051	8,654	6.20	0.065	11,049	7.40	0.088	15,024	8.40	0.107	18,328	
72	1.99	0.007	1,755	2.59	0.015	3,619	3.50	0.027	6,690	4.00	0.035	8,817	4.70	0.045	11,560	5.60	0.062	15,951	6.40	0.077	19,817	

Qd=Cdev x I(mm/hr)-Continuing Loss) x Area/360
Vmin = Qd x time(min) x 60 - Initial Loss
Vhrs = Qd x time(hrs) x 3,600 - Initial Loss

Initial Losses Grass = 5mm ; Concrete = 1mm mean 3mm x 101,350m² IL = 181.8 m³
Continuing Losses Grass =2mm/hr Concrete = 0.5mm/hr Mean CL = 1.25mm/hr

Location		Oyster Reach Catchment D																		McGlashan & Crisp File F929-Job No 15804																																																																																																											
% Impervious		50%																		50%																																																																																																											
Area (ha)		4.86																		4.86																																																																																																											
ARI Years		1																		2																		5																		10																		20																		50																		100																	
C dev		0.6																		0.65																		0.70																		0.75																		0.78																		0.85																		0.89																	
DEVELOPED		Rainfall Discharge Volume																		Rainfall Discharge Volume																		Rainfall Discharge Volume																		Rainfall Discharge Volume																		Rainfall Discharge Volume																		Rainfall Discharge Volume																																			
mins		I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff			I Qd Runoff																																																																																						
		mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³			mm/hr m³/s m³																																																																																									
5	96.00	0.767	84	122.20	1.061	173	152.07	1.425	282	168.96	1.698	364	191.97	2.008	457	221.77	2.530	613	244.24	2.920	730																																																																																																										
6	89.90	0.718	113	114.40	0.993	212	142.62	1.336	335	158.49	1.592	427	180.13	1.884	532	208.13	2.374	709	229.26	2.740	840																																																																																																										
7	84.80	0.677	138	108.00	0.937	248	136.21	1.275	390	151.39	1.520	493	172.08	1.799	610	198.87	2.268	807	219.08	2.617	953																																																																																																										
8	80.50	0.642	162	102.60	0.889	281	129.79	1.215	437	144.29	1.448	549	164.04	1.714	677	189.60	2.161	892	208.90	2.495	1,052																																																																																																										
9	76.80	0.612	185	97.80	0.847	312	123.38	1.154	477	137.19	1.376	597	155.99	1.629	734	180.34	2.055	964	198.71	2.372	1,135																																																																																																										
10	73.60	0.586	206	93.70	0.811	341	116.96	1.093	510	130.09	1.305	637	147.94	1.545	781	171.07	1.949	1,023	188.53	2.250	1,204																																																																																																										
11	70.60	0.562	225	90.00	0.779	368	112.60	1.052	549	125.26	1.256	683	142.46	1.487	836	164.76	1.876	1,093	181.59	2.167	1,284																																																																																																										
12	68.00	0.541	243	86.70	0.750	394	108.24	1.011	582	120.42	1.207	723	136.98	1.429	883	158.44	1.804	1,153	174.64	2.083	1,354																																																																																																										
13	65.70	0.522	261	83.70	0.723	419	104.81	0.979	618	116.62	1.168	765	132.68	1.384	934	153.48	1.747	1,217	169.17	2.018	1,428																																																																																																										
14	63.50	0.504	278	80.90	0.699	441	101.39	0.946	649	112.83	1.130	803	128.37	1.339	979	148.52	1.690	1,274	163.70	1.952	1,494																																																																																																										
15	61.50	0.488	293	78.40	0.677	463	97.96	0.914	677	109.03	1.091	836	124.07	1.293	1,018	143.56	1.633	1,324	158.28	1.887	1,552																																																																																																										
16	59.70	0.473	309	76.10	0.657	485	95.29	0.889	707	106.07	1.061	873	120.71	1.258	1,062	139.69	1.589	1,379	154.03	1.836	1,616																																																																																																										
17	58.00	0.460	323	74.00	0.638	505	92.63	0.864	735	103.12	1.031	906	117.36	1.223	1,101	135.82	1.544	1,429	149.77	1.784	1,674																																																																																																										
18	56.50	0.448	338	72.00	0.621	525	89.96	0.838	760	100.16	1.001	936	114.00	1.187	1,136	131.95	1.500	1,474	145.52	1.733	1,726																																																																																																										
19	55.00	0.435	351	70.10	0.604	543	87.73	0.817	786	97.69	0.976	967	111.20	1.158	1,174	128.72	1.463	1,522	141.97	1.691	1,782																																																																																																										
20	53.70	0.425	364	68.40	0.589	561	85.50	0.796	810	95.22	0.951	996	108.40	1.128	1,208	125.49	1.426	1,565	138.41	1.648	1,832																																																																																																										
30	43.72	0.344	473	55.61	0.477	713	69.66	0.646	1,018	77.64	0.773	1,246	99.10	1.030	1,709	114.77	1.303	2,199	126.61	1.506	2,565																																																																																																										
45	35.07	0.274	594	44.63	0.381	882	56.00	0.517	1,251	62.47	0.620	1,528	88.44	0.918	2,333	102.47	1.161	2,990	113.08	1.344	3,482																																																																																																										
Hrs	1	29.79	0.231	686	37.90	0.322	1,012	47.70	0.439	1,434	53.30	0.527	1,751	60.80	0.627	2,112	70.60	0.796	2,719	78.00	0.922	3,174																																																																																																									
	2	19.62	0.149	926	25.08	0.209	1,360	31.80	0.289	1,933	35.70	0.349	2,366	40.90	0.418	2,860	47.80	0.534	3,700	52.90	0.621	4,322																																																																																																									
	6	9.95	0.070	1,376	12.80	0.101	2,043	16.50	0.144	2,967	18.70	0.177	3,671	21.60	0.214	4,483	25.40	0.277	5,840	28.40	0.326	6,900																																																																																																									
	9	7.75	0.053	1,560	9.99	0.077	2,339	13.00	0.111	3,452	14.80	0.137	4,299	17.10	0.167	5,262	20.20	0.217	6,900	22.50	0.255	8,127																																																																																																									
	12	6.49	0.042	1,688	8.38	0.063	2,557	10.90	0.091	3,794	12.50	0.114	4,775	14.50	0.140	5,882	17.10	0.182	7,711	19.10	0.214	9,119																																																																																																									
	18	5.05	0.031	1,849	6.53	0.046	2,857	8.60	0.069	4,355	9.80	0.087	5,464	11.40	0.107	6,780	13.50	0.141	8,963	15.20	0.168	10,715																																																																																																									
	24	4.21	0.024	1,926	5.46	0.037	3,046	7.20	0.056	4,712	8.20	0.070	5,934	9.60	0.088	7,451	11.40	0.116	9,917	12.80	0.139	11,844																																																																																																									
	30	3.65	0.019	1,954	4.74	0.031	3,162	6.30	0.048	5,008	7.20	0.060	6,361	8.40	0.075	7,985	10.00	0.100	10,698	11.20	0.120	12,766																																																																																																									
	48	2.67	0.012	1,842	3.47	0.019	3,220	4.60	0.032	5,325	5.30	0.041	6,940	6.20	0.052	8,861	7.40	0.071	12,049	8.40	0.086	14,699																																																																																																									
	72	1.99	0.006	1,408	2.59	0.012	2,902	3.50	0.021	5,365	4.00	0.028	7,071	4.70	0.036	9,271	5.60	0.050	12,792	6.40	0.062	15,893																																																																																																									

Qd=Cdev x (I(mm/hr)-Continuing Loss) x Area/360

Initial Losses Grass = 5mm ; Concrete = 1mm mean 3mm x 101,350m² IL = 145.8 m³

Vmin = Qd x time(min) x 60 - Initial Loss

Continuing Losses Grass =2mm/hr Concrete = 0.5mm/hr Mean CL = 1.25mm/hr

Vhrs = Qd x time(hrs) x 3,600 - Initial Loss

APPENDIX 2

Stormwater Quality Management Strategy

Prepared by

WBM Pty Ltd (Newcastle office)

126 Belford Street
PO Box 266
Broadmeadow NSW 2292
AUSTRALIA

Phone: (02) 4940 8882

Fax: (02) 4940 8887

www.wbmpl.com.au

Includes:

Receiving Water Quality – commentary on Testing results

Stormwater Quality Management Modelling – MUSIC

Rainfall & Runoff Modelling - RAFTS

APPENDIX 3

Water Monitoring Programme

Prepared by

McGlashan & Crisp Pty Ltd
Taree NSW 2430

Testing by

MidCoast Water Laboratory
Sweet Pea Road
Locked Bag 4000
Forster NSW 2423
Phone: (02) 6555 4469
Fax: (02) 6554 8198

www.midcoastwater.com.au

A photographic record of the location of the testing sites and recording the base line measurements.

APPENDIX 4

Minutes of Meeting

Prepared by

McGlashan & Crisp Pty Ltd
Taree NSW 2430

Report of Site Meeting Thursday September 16, 2004

In Attendance:

Bob Lauder : Riverside Developments (BL)
Grant Calvin : McGlashan & Crisp (GC)
Scott Carter : State Fisheries (SC)
Peter Adcock : Australian Wetlands Pty Ltd (PA)
Robert Baker : Greater Taree City Council (RB)

Purpose of the Meeting.

To investigate existing wetland and stream environments on the periphery of proposed development area Precinct 2A at Old Bar and formulate engineering principles for best managing, protecting and or enhancing the existing environment.

A record of discussions which have been incorporated into the Precinct 2A development design.