KNOBELCONSULTING
CIVIL ENGINEERS + HYDRAULIC ENGINEERS + PROJECT MANAGERS

PRELIMINARY STORMWATER MANAGEMENT PLAN

Proposed Tourist Accommodation
Lot 1 on DP1168904
Kirkwood Road, Tweed Heads South

for Proportional Property Investments Pty Ltd

29 August 2013

File No: K1868-0053-A

DOCUMENT CONTROL SHEET

Title:	Preliminary Stormwater Management Plan
Document No:	K1868-0053
Original Date of Issue:	20 July 2012
Project Manager:	Trent Purdon
Author:	Matt Godwin
Client:	Proportional Property Investments Pty Ltd
Client Contact:	Julie Nutting
Client Reference:	Kirkwood Road
Synopsis:	This *Preliminary Stormwater Management Plan* (SWMP-P) describes the existing site characteristics, proposed development of the site and corresponding stormwater quantity and quality management controls to be implemented during both the construction and operational phases of the development.

Revision/Checking History

Revision No	Date	Checked By	Issued By
Original	20 July 2012	Trent Purdon	Matt Godwin
A	29 August 013	Trent Purdon	Matt Godwin

Distribution

Recipient	No of Copies	Method
DA Consulting Pty Ltd	1	PDF
Proportional Property Developments Pty Ltd	1	PDF

CONTENTS

FIGURES

TABLES

APPENDICES

Appendix A - Paul Ziekulis Architect, *Site Plan* (Ref: SD-A01/09)
Appendix B - Knobel Consulting Pty Ltd, *Pre Development Internal Catchment Plan*, (Ref: K1868/P010/B)
Appendix C - Knobel Consulting Pty Ltd, *Post Development Internal Catchment Plan*, (Ref: K1868/P011/E)
Appendix D - Knobel Consulting Pty Ltd, *Detention and Bioretention Basin Plan* (Ref: K1868/P037/A)
Appendix E - Knobel Consulting Pty Ltd, *Bio Retention Basin Discharge Arrangement* (Ref: K1868/P044/A)
Appendix F - Knobel Consulting Pty Ltd, *Sediment and Erosion Control Layout Plan* (Ref: K1868/P014/C) and *Sediment and Erosion Control Details Sheet* (Ref: K1868/P015/A)

1.0 INTRODUCTION/OBJECTIVES

1.1 Background

Knobel Consulting Pty Ltd has been commissioned by Proportional Property Investments Pty Ltd to prepare a *Preliminary Stormwater Management Plan (SWMP-P)* and supporting engineering documentation for a proposed community title tourist accommodation development at Lot 1 on DP1168904 located on Kirkwood Road, Tweed Heads South (the subject site). In preparing this report Knobel Consulting Pty Ltd has considered the management of quality and quantity of stormwater during the operational phase.

This report has been updated in response to Tweed Shire Council's Information Request dated 22 March 2013.

1.2 Scope

The SWMP-P details the conceptual planning, layout and design of the stormwater management infrastructure for the operational phase of this development.

This SWMP-P aims to:

- Establish the required performance criteria for the proposed stormwater quantity and quality systems;
- Provide a conceptual design of stormwater infrastructure including stormwater quality improvement devices and stormwater quantity management controls where required; and
- To ensure stormwater runoff is conveyed from/through the site to a legal point of discharge without adversely affecting both upstream and downstream flood levels.

This SWMP-P has been prepared in accordance with the Tweed Shire Council (TSC) *Development Design Specifications* D5 and D7.

2.0 SITE DESCRIPTION

2.1 Location

Figure 1 shows the subject site that is located at Kirkwood Road, Tweed Heads South. The subject site has an area of 17.23 Ha however the proposed development is only proposed on a portion of that land with a significant portion of the site being retained for environmental purposes. The portion of land to be developed has been established by environmental constraints on the Eastern section of the site and proposed land dedication along the Northern and North Eastern boundaries for the future Council construction of Kirkwood Road and Pacific Motorway interchanges

Site details have been summarised within Table 1 and a UBD extract is presented as Figure 1.

Table 1: Site Details

Developer/Consultant	Property and Location	
Owner/Developer	**Lot and Property Description**	**Address**
Proportional Property Investments Pty Ltd	*Lot 1 on DP1168904*	*Kirkwood Road Tweed Heads South*

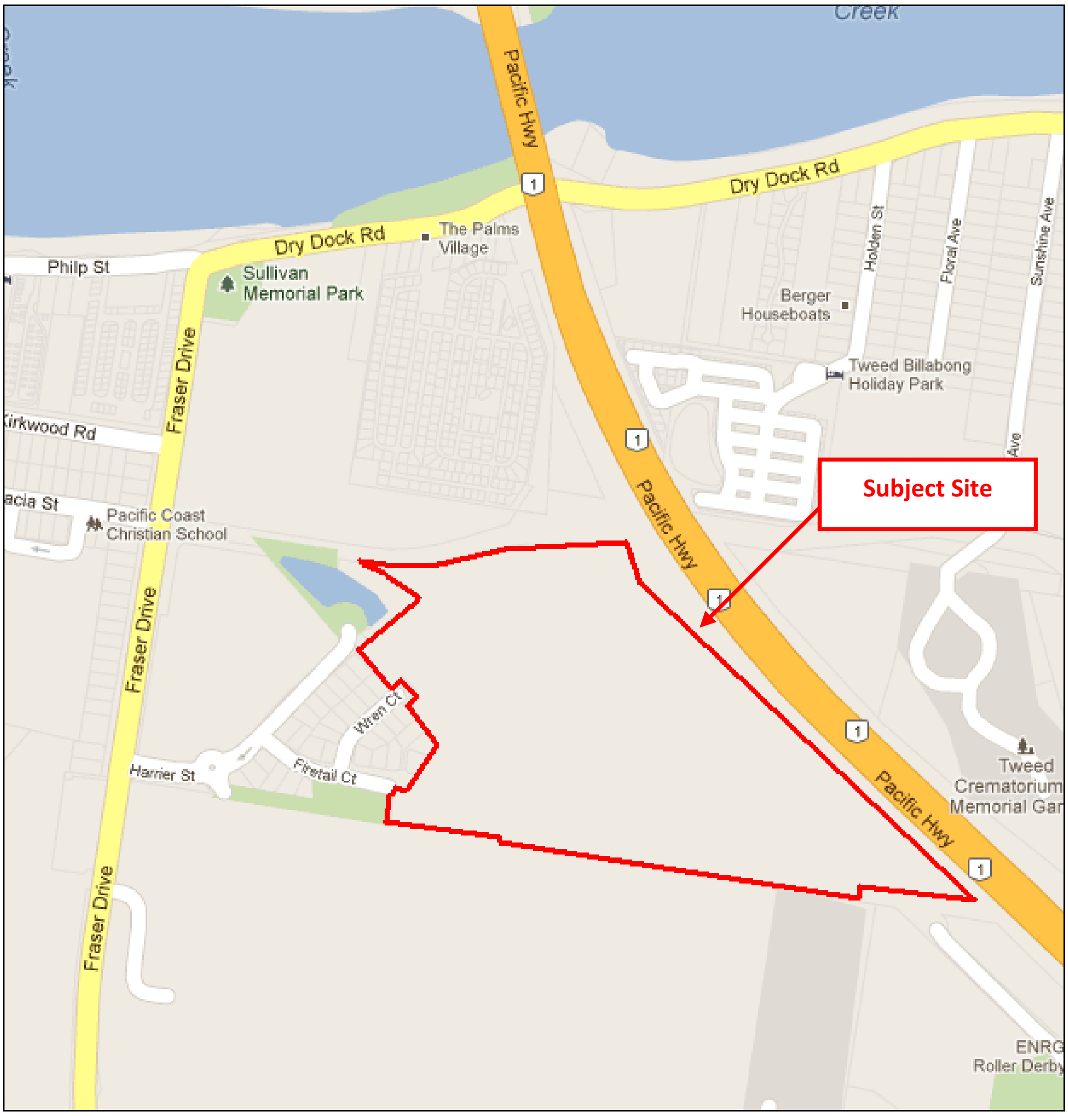

Figure 1: ***Google Extract***

2.2 Site Topography

The developable portion of the property contains a crest that runs roughly East to West. The crest has heights of approximately RL40 down to RL2 at the Northern corner of the site and RL1.5 at the South Eastern extremity. The site falls at approximately 25% to the North and approximately 27% to the South East. A large gully is evident through the south eastern portion of the site running from West to East.

Outside of the developable portion of the site contains low lying ground that extends to the far South Eastern corner of the site.

2.3 Vegetation and Land Use

The majority of the site is currently heavily vegetated with sparse vegetation evident on the Northern portion of the site.

An aerial photograph is illustrated in Figure 2.

Figure 2: ***Aerial Photograph of the Site***

2.4 Proposed Development

With reference to Paul Ziekulis Architect, *Site Plan* (Ref: SD-A01/09) included as Appendix A, a 355 dwelling community title tourist accommodation development is proposed on the site.

3.0 SITE HYDROLOGY AND HYDRAULICS

3.1 Background

The subject site is located adjacent to the proposed Kirkwood Road extension. Council have on file detailed plans for this road extension which includes an overpass over the adjacent Pacific Highway. This road extension includes the construction of culverts at the bottom of the northern catchment discharging flow into an existing gully located along the boundary of Lot 1 DP1074784 (Street address 136-150 Dry Dock Road) and Lot 201 DP1101907. The proposed Kirkwood Road will discharge stormwater runoff from the southern and northern side of the road to each side of the road respectively at the location of the culverts (i.e. the upstream and downstream end of the culverts).

The existing receiving gully conveys water north to a waterbody that discharges directly to Terranora Creek. For the purposes of stormwater quantity management this waterbody can be considered as part of Terranora Creek. The owner of the land containing the existing gully is not currently known and as such discussions regarding stormwater discharge through this property have not been able to be held. There is also no site survey for this parcel of land.

This report proceeds on the basis that discharge to the gully downstream of the proposed site outlet following construction of the development will need to be attenuated to existing peak discharge levels.

Refer to Knobel Consulting Pty Ltd, *Pre Development Internal Catchment Plan* (Ref: K1868/P010/B) included in Appendix B.

The southern catchment discharges to a large wetlands area. This land is protected from development due to designated environmental value and flood inundation. The land acts as a sink and does not represent a constriction on stormwater discharge. It is also located below the Q_{100} flood level and is subject to storm tide influence. Therefore provided that stormwater runoff is discharged in a manner that controls and prevents scour or erosion, there would not normally be a requirement to provide detention. However Council has stated in its Information Request that for compliance with its 'Ecological Assessment' stormwater detention must be provided if the post development peak discharge rate exceeds the pre development peak discharge rate. Therefore calculations have been carried out to ascertain this requirement.

The following sections define the parameters of the catchments. The Rational Method has been applied to define flow rates.

3.2 Legal Point of Discharge

The subject site currently discharges stormwater to two separate locations representing two Legal Points of Discharge (LPOD-A and LPOD-B). Refer to Knobel Consulting Pty Ltd, *Pre Development Internal Catchment Plan* (Ref: K1868/P010/B) included in Appendix B.

LPOD-A receives discharge from the northern catchment. The site boundary is offset from the receiving gully at the bottom of the hill which also accepts runoff from land adjacent to the LPOD-A. The gully initially conveys water west and then turns north to convey and discharge water into an existing waterbody that is connected without constriction to Terranora Creek (for the purposes of stormwater runoff).

LPOD-B receives discharge from the southern catchment and is represented by the existing vegetated open space. This area is protected from development with a number of environmental setbacks dictating the extent of the proposed development.

3.3 Legal Point of Discharge A

PRE DEVELOPMENT

3.3.1 Catchment Definition

A single catchment discharging to LPOD-A has been demarcated as shown on Knobel Consulting Pty Ltd, *Pre Development Internal Catchment Plan* (Ref: K1868/P010/B) included in Appendix B. The area has been confined to within the extent of the proposed works. A portion of land within the site adjacent to Wren Street (south west of the site) currently discharges southwest to the head of the cul de sac.

This land originally discharged north to LPOD-A but was altered during the construction of the adjacent subdivision such that it now grades to Wren Street. The current contours for this area have been used reflecting the current situation for pre development calculations.

3.3.2 Coefficient of Runoff

A coefficient of runoff (C year) was calculated for the site using the fraction impervious method from QUDM based on an analysis of the land coverage. The pre development site contains no impervious area. With a 1 hour 10 year rainfall intensity of 71.4 mm/hr, a C_{10} value of 0.70 is specified (Table 4.05.3(b) – QUDM) being for medium density bushland (average based on site inspection). Frequency factors have been determined from Table D5.2 of Tweed Shire Council's, *Stormwater Drainage Design.*

Table 2: LPOD-A Pre Development Coefficient of Runoff

	Area (ha)	Fraction impervious	C_2	C_{10}	C_{100}
Catchment A	2.405	0.00	0.57	0.70	0.90

3.3.3 *Time of Concentration*

The pre development time of concentration for Catchment A was derived from a flow path comprising initial sheet flow runoff and channel flow from the top of the catchment to the LPOD-A. This constitutes the longest time of concentration for the catchment. The sheet flow was calculated using Friend's equation (QUDM Eqn 4.06) as 20 metres of sheet flow over a 15% slope (n=0.06) equating to 10 minutes. Channel flow was estimated using Figure 4.09 (QUDM). 120 metres over a 34 metre fall (36 m AHD – 2 m AHD) with a factor of Δ=3 applied for natural channels equates to 1.5 minutes. This time equates to a velocity of 1.3 m/s which is reasonable for an average 25% slope over a natural channel.

Therefore the total time of concentration for the catchment equals 11.5 minutes.

3.3.4 *Design Flow Rates*

Design storm flow rates have been calculated for standard storms with an ARI of 2, 10, and 100 years using design rainfall intensities from TSC Development Design Specification D5. The Rational Method (Q = 2.78 x 10-3 CIA) has been used to calculate the required design flow rates for the subject site.

Table 3: LPOD-A Pre Development Flow Rate

Average Recurrence Interval	ARI	2	10	100
Coefficient of Runoff	C	0.57	0.70	0.90
Area of Catchment (ha)	A	2.405	2.405	2.405
Average Rainfall Intensity (mm/h)	I	121	162.4	227.1
Peak Flow Rate (m^3/s)	**Q**	**0.460**	**0.759**	**1.365**

POST DEVELOPMENT

3.3.5 *Catchment Definition*

The demarcation of the post development catchment plan includes the internal site area discharging to LPOD-A. Refer to Knobel Consulting Pty Ltd, *Post Development Internal Catchment Plan* (Ref: K1868/P011/E) included in Appendix C. The internal site is represented by two sub catchments. All runoff from Int A drains to the water quality basin in the northern corner of the site. The outflow from this basin will discharge directly to LPOD-A. Int C will direct pipe flow to LPOD-B. Overland flow from Int C surcharging the pipe system (flow > Q_{10}) will discharge to the proposed Kirkwood Road and on to LPOD-A and is considered to be an external sub catchment for stormwater management purposes.

It is intended to provide detention above the water quality basin within the proposed development. This basin will be sized to ensure that the total flow to LPOD-A, including the unattenuated runoff from Int C does not exceed the pre development peak flow rates. A hydraulic model is required to adequately assess this scenario and flow rates have been calculated using the rational method to calibrate model results for each sub catchment. The proposed Kirkwood Road to be constructed by Council external to the site has not been included in calculations.

3.3.6 *Coefficient of Runoff*

A coefficient of runoff (C year) was calculated using the fraction impervious method from QUDM based on an analysis of the land coverage as shown below. The C_{10} value is specified from Table 4.05.3(a) (QUDM) for a 1 hour 10 year rainfall intensity at the site of 71.4 mm/hr. Frequency factors have been determined from Table D5.2 of Tweed Shire Council's, *Stormwater Drainage Design.*

Table 4: LPOD-A Post Development Coefficient of Runoff

Catchment	Land Type	Area (ha)	Fraction impervious	Weighted Value	C_2	C_{10}	C_{100}
Int A	Lots	1.886	0.69	0.60	0.66	0.82	1.0
	Road	0.930	0.75				
	Park	0.488	0				
Int C	Mixed	0.323	Apply Int A	0.59	0.66	0.82	1.0

3.3.7 Time of Concentration

The post development time of concentration for each sub catchment comprises a mixture of sheet flow and pipe/channel flow. These have been estimated from QUDM Section 4.06. A t_c has been calculated for each sub catchment to generate respective peak flows and enabling the calibration of each sub catchment in the hydraulic model.

Table 5: LPOD-A Sub catchment time of concentration

Sub Catchment	Runoff Type	Parameters	Time	Total Tc	Discharge Point
Int A	Sheet	20 m @ 5%, n=0.045	9	11	Basin
	Pipe	210 m, 9 m fall, Δ=1	2		
Int C	Sheet	15 m @ 2%, n=0.035	8	10.5	U/S culverts
	Channel	340 m, 13 m fall, Δ=1	2.5		

3.3.8 Design Flow Rates

Design storm flow rates have been calculated for standard storms with an ARI of 2, 10, and 100-years using design rainfall intensities from TSC Development Design Specification D5. The Rational Method (Q = 2.78 x 10-3 CIA) has been used to calculate the required peak design flow rates for each sub catchment.

Table 6: LPOD-A Sub catchment Post Development Flow Rate

Sub Catchment	ARI	C	I (mm/h)	A (ha)	Q (m^3/s)
Int A	2	0.66	122.7	3.304	**0.743**
	10	0.82	164.6		**1.239**
	100	1.0	230.1		**2.119**
Int C	2	0.66	124	0.323	**0.073**
	10	0.82	167		**0.123**
	100	1.0	233		**0.209**

The hydraulic model has been calibrated such that the peak discharges for each sub catchment correspond with the calculated rational method flows in the table above. Note that for storm events of 1 in 10 year ARI or less the sub catchment Int C is excluded from contributing as pipe flow from this sub catchment (maximum pipe system design capacity of Q_{10}) is directed to LPOD-B.

For the Q_{100} storm event the sub catchment Int C was calibrated to discharge a peak flow rate equal to the Q_{100} flow – Q_{10} flow = 0.086 m^3/s (this is conservative as the pipe inlet system for Int C will be designed to cater for runoff with a shorter time of concentration than 10.5 minutes and hence larger flow rate).

3.4 Legal Point of Discharge B

PRE DEVELOPMENT

3.4.1 Catchment Definition

A single catchment discharging to LPOD-B has been demarcated as shown on Knobel Consulting Pty Ltd, *Pre Development Internal Catchment Plan* (Ref: K1868/P010/B) included in Appendix B. The area has been confined to within the extent of the proposed works.

3.4.2 Coefficient of Runoff

A coefficient of runoff (C year) was calculated for the site using the fraction impervious method from QUDM based on an analysis of the land coverage. The pre development site contains no impervious area. With a 1 hour 10 year rainfall intensity of 71.4 mm/hr, a C_{10} value of 0.70 is specified (Table 4.05.3(b) – QUDM) being for medium density bushland (average based on site inspection). Frequency factors have been determined from Table D5.2 of Tweed Shire Council's, *Stormwater Drainage Design.*

Table 7: LPOD-B Pre Development Coefficient of Runoff

	Area (ha)	Fraction impervious	C_2	C_{10}	C_{100}
Catchment A	4.427	0.00	0.57	0.70	0.90

3.4.3 Time of Concentration

The pre development time of concentration for Catchment B was derived from a flow path comprising initial sheet flow runoff and channel flow from the top of the catchment to the LPOD-B. This constitutes the longest time of concentration for the catchment. The sheet flow was calculated using Friend's equation (QUDM Eqn 4.06) as 20 metres of sheet flow over a 15% slope (n=0.06) equating to 10 minutes. Channel flow was estimated using Figure 4.09 (QUDM). 140 metres over a 36 metre fall (38 m AHD – 2 m AHD) with a factor of Δ=3 applied for natural channels equates to 1.5 minutes. This time equates to a velocity of 1.5 m/s.

Therefore the total time of concentration for the catchment equals 11.5 minutes.

3.4.4 Design Flow Rates

Design storm flow rates have been calculated for standard storms with an ARI of 2, 10, and 100-years using design rainfall intensities from TSC Development Design Specification D5. The Rational Method (Q = 2.78 x 10-3 CIA) has been used to calculate the required design flow rates for the subject site.

Table 8: LPOD-B Pre Development Flow Rate

Average Recurrence Interval	ARI	2	10	100
Coefficient of Runoff	C	0.57	0.70	0.90
Area of Catchment (ha)	A	4.427	4.427	4.427
Average Rainfall Intensity (mm/h)	I	121	162.4	227.1
Peak Flow Rate (m^3/s)	**Q**	**0.848**	**1.398**	**2.513**

POST DEVELOPMENT

3.4.5 Catchment Definition

Int B represents the majority of the catchment discharging to LPOD-B. The sub catchment Int C will discharge pipe flow only to LPOD-B (design capacity of Q_{10}) with any overland flow surcharging the pipe system being directed to LPOD-A. Therefore all flow from Int B and Int C is applied when calculating runoff rates for storm events up to and including the 1 in 10 year ARI event. All runoff from Int C exceeding the Q_{10} is subtracted from the calculated flow to LPOD-B for storm events greater than the Q_{10} event. In the hydraulic model the sub catchment of Int C is calibrated to discharge a maximum of 0.123 m^3/s equal to the Q_{10} runoff for any storm event greater than the 1 in 10 year ARI event. Refer to Knobel Consulting Pty Ltd, *Post Development Internal Catchment Plan* (Ref: K1868/P011/E) included in Appendix C.

3.4.6 Coefficient of Runoff

A coefficient of runoff (C year) was calculated for the site using the fraction impervious method from QUDM based on an analysis of the land coverage. The percentage of imperviousness is shown below. The C_{10} value is specified from Table 4.05.3(a) (QUDM) for a 1 hour 10 year rainfall intensity at the site of 71.4 mm/hr. Frequency factors have been determined from Table D5.2 of Tweed Shire Council's, *Stormwater Drainage Design.*

Table 9: LPOD-B Post Development Coefficient of Runoff

Catchment	Land Type	Area (ha)	Fraction impervious	Weighted Value	C_2	C_{10}	C_{100}
Int B	Lots	1.413	0.69	0.60	0.66	0.82	1.0
	Road	1.438	0.75				
	Park	0.557	0				
Int C	Mixed	0.323	Apply Int A	0.59	0.66	0.82	1.0

3.4.7 Time of Concentration

The post development time of concentration for Int B and Int C was derived from a flow path comprising a mixture of sheet flow and pipe/channel flow. These have been estimated from QUDM Section 4.06.

The sheet flow was calculated using Friend's equation (QUDM Eqn 4.06) as 20 metres of sheet flow over a 2.5% slope (n=0.035) equating to 8 minutes. Combined kerb and channel flow and pipe flow to the basin comprises 220 metres over a 10 metre fall (16.5 m AHD – 6.5 m AHD) with a factor of Δ=1 applied for drainage infrastructure equates to 2 minutes. Therefore the total time of concentration for the combined Int B and Int C catchments equals 10 minutes.

3.4.8 Design Flow Rates

Design storm flow rates have been calculated for standard storms with an ARI of 2, 10, and 100 years using design rainfall intensities from TSC Development Design Specification D5. The Rational Method (Q = 2.78 x 10-3 CIA) has been used to calculate the required design flow rates for the subject site.

Table 10: LPOD-B Post Development Flow Rate

Average Recurrence Interval	**ARI**	2	10	100
Coefficient of Runoff	**C**	0.66	0.82	1.0
Area of Catchment (ha)	**A**	3.731	3.731	3.731
Average Rainfall Intensity (mm/h)	**I**	126	169	236
Peak Flow Rate (m^3/s)	**Q**	**0.862**	**1.436**	**2.360***

***NOTE: The Q_{100} runoff rate has been reduced by 0.086 m^3/s equivalent to the Q_{100} – Q_{10} flow surcharging the pipe system from sub catchment Int C that is directed to LPOD-A**

The discharge arrangement for the water quality basin will be sized at the detailed design stage to ensure it has the capacity to safely discharge a minimum of 2.360 m^3/s to the downstream environment without causing adverse impacts.

3.5 Stormwater Detention

Calculations in section 3.4 show that stormwater detention is required for discharge to LPOD-A to attenuate post development runoff to pre development rates. The peak discharge at LPOD-B is practically the same and given the standard margin of error contained in assumptions and formula for stormwater runoff calculations, it is reasonable to conclude that onsite detention here is not warranted. However for conservatism detention has also been provided at LPOD-B to account for the minor increase in peak flow rates for events up to and including the Q_{10} event (calculations show that peak flow is reduced for the Q_{100} event due to the pipe network catchment as explained).

A detailed design analysis with a hydraulic model has been carried out at the DA stage to ensure the required volume and outlet arrangement to LPOD-A can be accommodated in the proposed area. To assess the storage and outlet requirements of the proposed detention basin, modelling software XPswmm - Sentinel Version (2011) has been selected for the hydrologic and hydraulic analysis. Details of the model set-up and detention basin design for LPOD-A are summarised in the following sections 3.5.1 to 3.5.3. Model results confirming adequate detention storage at LPOD-B are provided in section 3.5.4.

3.5.1 Model Set-up

A node has been designated for both sub catchments in the model. The parameters for each node have been calibrated such that the peak discharge rate for each node corresponds with the peak values calculated using the rational method earlier. Rainfall data for the subject site is based on the temporal patterns derived by Gold Coast City Council for the Gold Coast region which are notably different than those specified in Australian Rainfall and Runoff (1987) for Zone 3. Several storm events have been included in the modelling with durations ranging from 15 min to 1.5 hrs.

The 15 minute design storm was found to be the critical duration, generating the highest peak runoff rate. All Q_{100} hydrographs, with various durations are included as Figure 3.

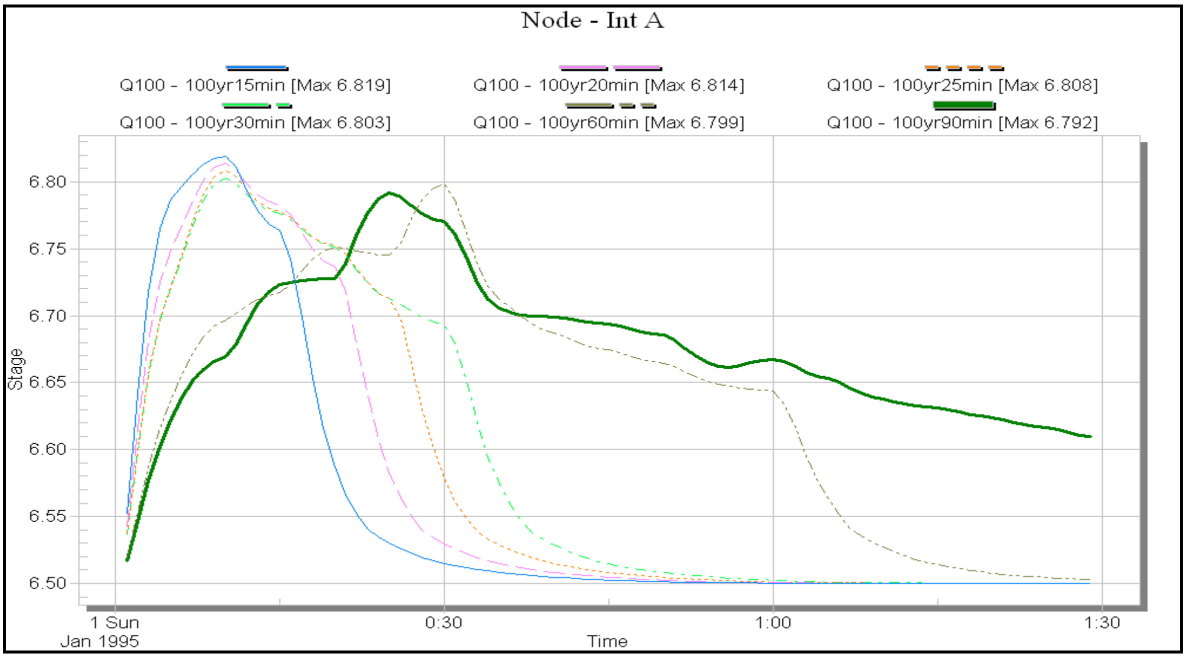

Figure 3: Int A Post Development Hydrograph – 100 year ARI, Various Durations

A detention basin was added to the node receiving the runoff from Int A. This basin was sized to attenuate outflow such that the total flow discharging further downstream to LPOD-A did not exceed the pre development peak flow rate discharging to LPOD-A calculated using the rational method.

3.5.2 Basin A Parameters

It is proposed to locate the detention on top of the bio retention basin. That is, the base of the detention basin is taken to be the top of the bio retention basin extended detention depth of 300 mm above the filter level. The minimum outlet of the detention basin is also set at this level. The filter area and batter slopes of the bio retention basin influence the base area of the detention basin. To achieve the required volume it will be necessary to construct the northern and eastern sides of the basin with a retaining wall while the southern and western ends can be a batter slope at 1:4. The maintenance access path will be located on these sides. The preliminary calculation of the stage-storage data for the detention basin is shown below.

Table 11 Detention Basin Stage-Storage Parameters

Stage (m AHD)	Basin Area (m2)
4.20	350
5.60	560

The outlet system comprises two pipe outlets. A raised field inlet within the bio retention basin set at the detention base level of 4.20 m AHD (300mm above the filter level) will discharge to a 375 mm diameter pipe. This pipe will convey minor storm event overflow and attenuate runoff in Q_2 storm events. 2x375 mm diameter pipes with an invert level set just below top water level reached in a Q_2 storm will discharge flow in larger events up to an including the Q_{100} storm event. All pipes will discharge a drainage channel leading to the proposed culverts under Kirkwood Road.

The design outlet arrangement has the following parameters:

- Low flow pipe: 1 x 375 mm diameter RCP
  USIL = 2.90 m AHD
  DSIL = 2.70 m AHD
  Serviced by field inlet with IL = 4.20 m AHD
- High flow pipe: 2 x 375 mm diameter RCP
  USIL = 4.52 m AHD
  DSIL = 2.70 m AHD
- Basin Spill Level: 5.60 m AHD

In storm events generating runoff in excess of the design Q_{100} runoff, or in cases where outlet pipe conveyance is reduced due to blockage, runoff will spill over the emergency weir level of 5.60 m AHD directly to the drainage channel. Refer to Knobel Consulting Pty Ltd, *Detention and Bioretention Basin Plan* (Ref: K1868/P037/A) included as Appendix D for a cross section of the basin.

3.5.3 Model Results

An analysis of all Q_{100} storm events indicates that the 20 minute design storm produces the highest peak flow and is also the critical duration for designing the detention basin storage. Figure 4 shows that a peak water level of 5.58 m AHD occurs within the detention basin during the 1 in 100 year, 20 min design storm event.

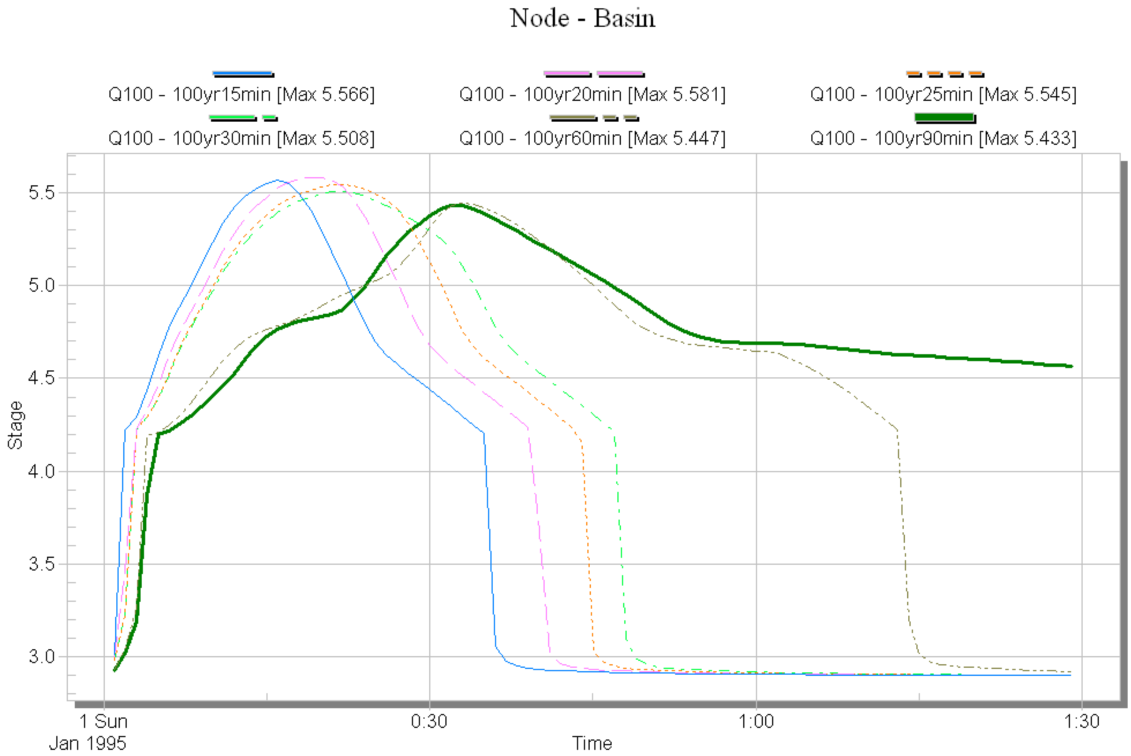

Figure 4: ***LPOD-A Detention Basin Peak Water Levels, 100 Year ARI, Varying Durations***

Figures 5 and 6 respectively show the post development detention basin outflow hydrographs for the Q_{100} and Q_2 design storms. The graphs demonstrates that the proposed detention basin and outlet system will achieve the required flow attenuation to reduce the peak post development flow rate to the pre development flow rate for all ARI design storms.

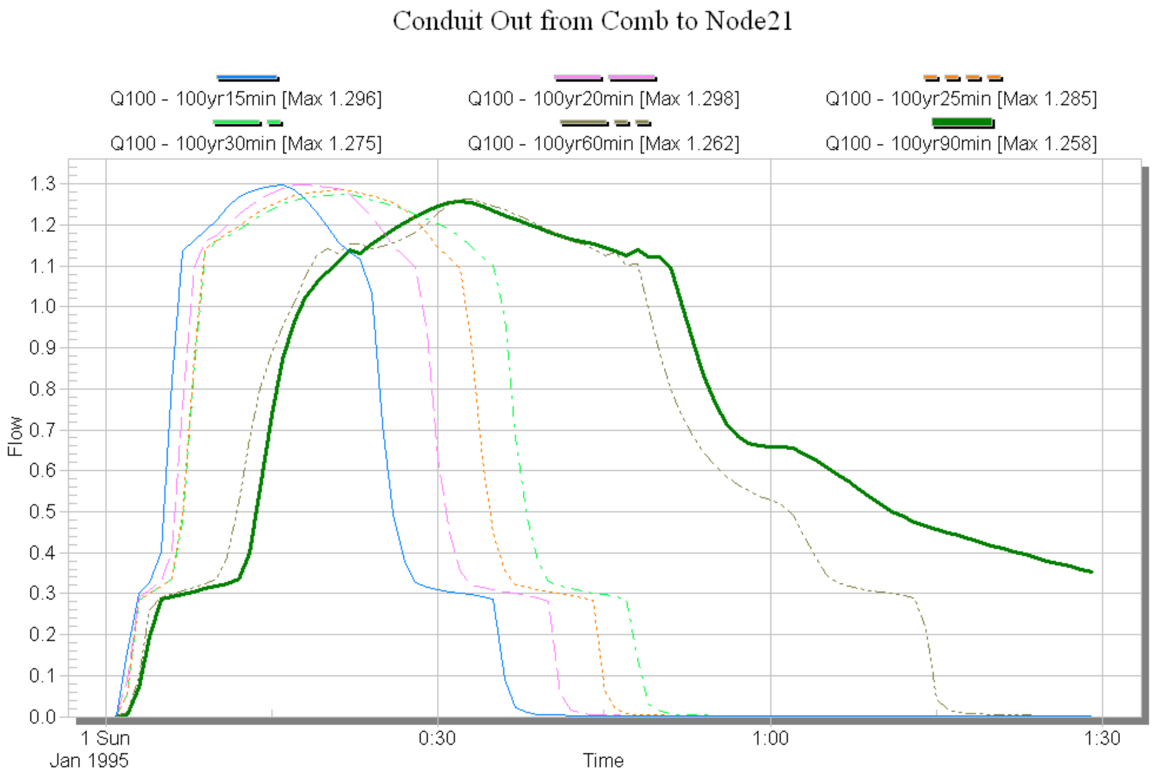

Figure 5: ***LPOD-A Detention Basin Outflow Hydrographs 100 Year ARI***

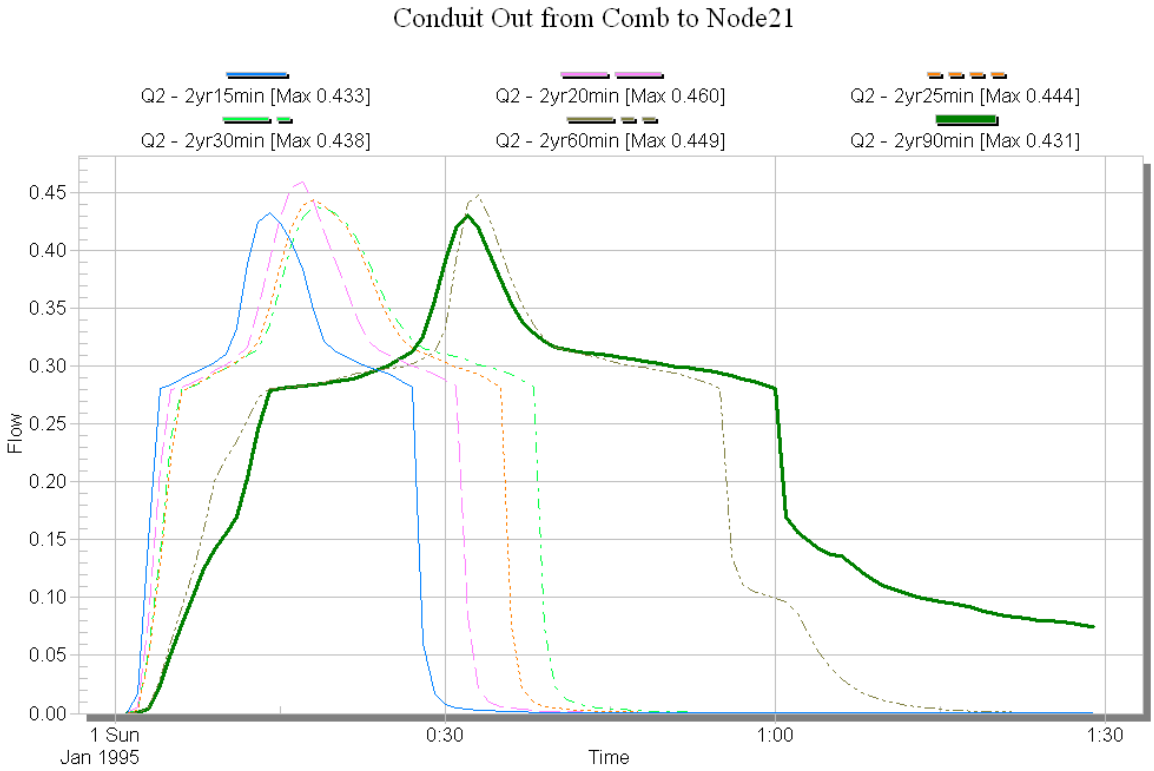

Figure 6: LPOD-A Detention Basin Outflow Hydrographs 2 Year ARI

The hydraulic model demonstrates that the objectives for stormwater runoff attenuation have been achieved. The proposed detention basis will reduce peak flow rates to below pre development levels for all storm events from 2 year ARI to 100 year ARI.

Detailed civil design of the basin and outlet arrangement generally in accordance with this report will be required at the Operational Works stage of the development. Note that future design amendments or refinements may require alteration of the basin and outlet parameters detailed in this preliminary report.

3.5.4 LPOD-B Detention

The proposed basin arrangement at LPOD-B has been modelled in the XP model. There is no above filter pipe outlet or field inlet for the basin with the outlet for this basin being 2 x 16m wide weirs (in addition to the slotted pipes at the base of the bio retention basin draining filtered runoff). For detention modelling purposes the basin has been assumed to be full at the time of the storm event. That is the 300mm bio retention extended detention depth is full such that the standing water depth is level with the weir outlet level. In effect this means that runoff will start discharging from the basin as soon as it enters.

The basin parameters and outlet have been modelled in accordance with the civil design plans as follows.

- Basin Area at Weir Level: 415 m^2
- Basin Area at Top of Weir: 592 m^2
- Weir Width: 16 m (x2 weirs for total of 32 m)
- Weir Side Slope: 1:4
- Weir Height: 0.3 m

Refer to Knobel Consulting Pty Ltd, *Bio Retention Basin Discharge Arrangement* (Ref: K1868/P044/A) included as Appendix E for a details of the outlet arrangement. Note that it is irrelevant what level the basin floor (and corresponding weir spill level) is for the purposes of modelling. For information a basin floor and weir spill level of 3.4 m AHD was used. This corresponds to an outlet level at the base on the bio retention batter of 2.0 m AHD.

The model results show that the amount of storage within the basin and the weir parameters result in a slightly reduced discharge of 1.384 m^3/s from the weir arrangement compared to the inflow for a Q_{10} storm event. This is less than the peak pre development rate of 1.398 m^3/s and demonstrates that the peak flow to LPOD-B will not be adversely affected by the development. A snapshot of the outflow hydrograph is shown below.

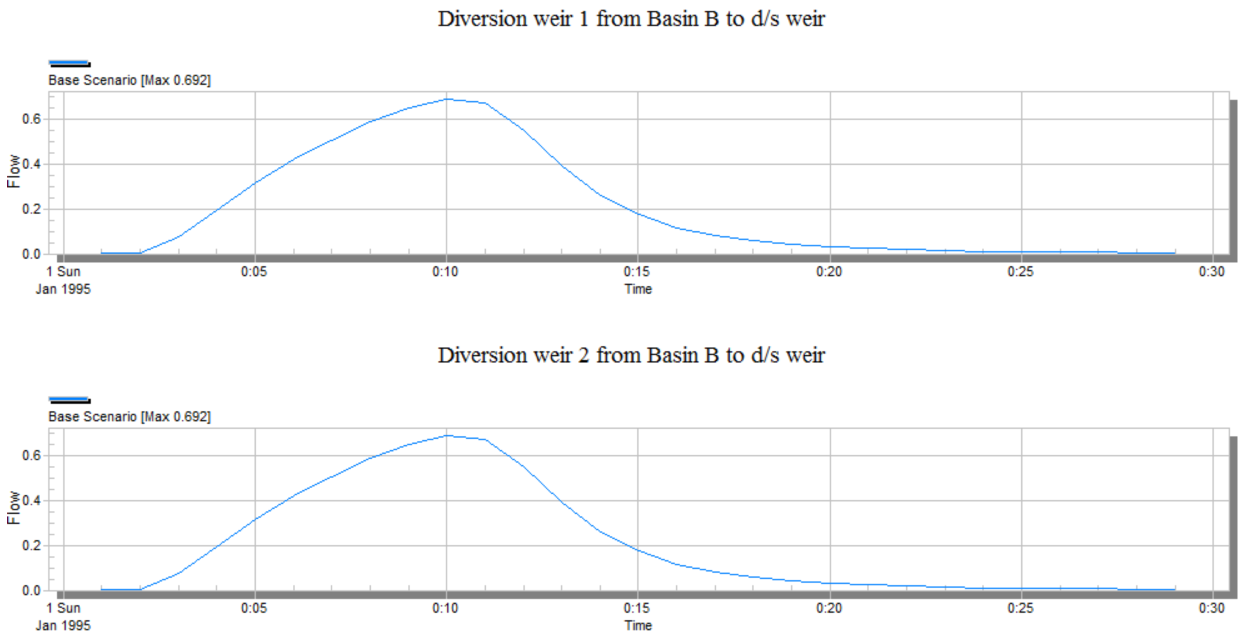

Figure 7: ***LPOD-B Detention Basin Outflow Hydrographs 10 Year ARI, 10 min critical duration***

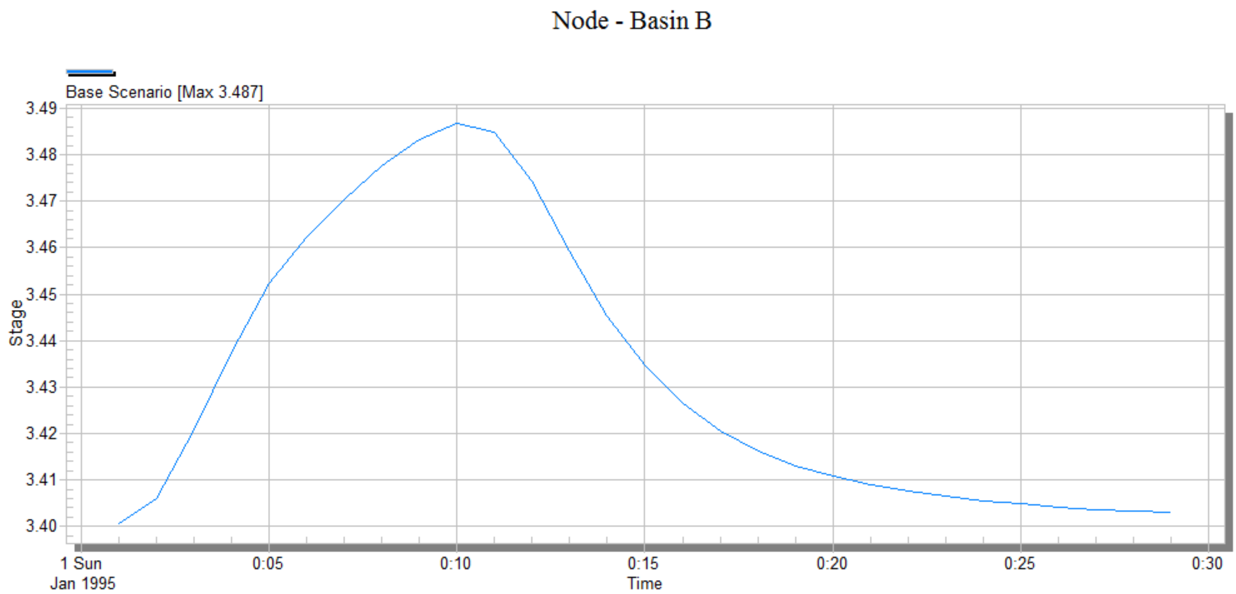

Figure 8: ***LPOD-A Detention Basin Peak Water Level, 10 Year ARI,10 min critical duration***

A check of the Q_2 storm event was made and confirmed that the peak discharge rate at 0.816 m^3/s is less than the peak predevelopment discharge rate of 0.840 m^3/s confirming all ARI storm events are compliant.

3.6 Emergency Weir Arrangement

Basin A

An emergency weir overflow will be provided in cases where inflow to the basin exceeds the design Q_{100} flow rate or there is blockage of the pipes. The weir has been sized to convey 50% of the Q_{100} flow rate of 2.119 m^3/s.

The design weir has the following parameters:

Weir Level:	5.60 m AHD
Weir Width:	6.50 m
Upstream Head:	0.30 m
Top Water Level:	5.90 m AHD
Design Flow Rate:	1.074 m^3/s

Basin B

An emergency weir overflow will be provided in cases where inflow to the basin exceeds the filtration capacity of the bio retention basin. The outlet arrangement comprises two separate weir structures to assist in dispersing the overflow over a wide area. The parameters of each weir are identical and together they will have the capacity to discharge to Q_{100} flow rate of 2.360 m^3/s.

Each weir has the following design parameters:

Weir Level:	300mm above the filter level
Weir Width:	16.0 m
Upstream Head:	0.20 m
Design Flow Rate:	1.23 m^3/s

Refer to Knobel Consulting Pty Ltd, *Bio Retention Basin Discharge Arrangement* (Ref: K1868/P044/A) included as Appendix E for details of the weir arrangement for Basin B. This outlet structure will discharge all runoff that surcharges to the 300mm extended detention depth of the bio retention basin and is designed to disperse the flow as sheet flow over a wide area at low velocity protecting the downstream environment from scour and erosion.

4.0 STORMWATER QUALITY ASSESSMENT

4.1 Background

The development of land has the potential to increase the pollutant loads within stormwater runoff and downstream watercourses. During the construction phase of the development disturbance to the vegetation on the site has the potential to significantly increase sediment loads entering downstream watercourses.

The operational phase of the development will change the sites current undeveloped open space land use to an urban residential land use potentially increasing the amount of sediments and nutrients washing from the site.

The following sections describe the predicted increase in pollutant loads generated by the proposed development and treatment devices to mitigate the potential increases.

4.2 Construction Phase

4.2.1 Key Pollutants

During the construction phase a number of key pollutants have been identified for this development. Table 12 illustrates the key pollutants that have been identified.

Table 12: Key Pollutants, Construction Phase

Pollutant	Sources
Litter	Paper, construction packaging, food packaging, cement bags, material off cuts.
Sediment	Exposed soils and stockpiles during earthworks and building works.
Hydrocarbons	Fuel and oil spills, leaks from construction equipment and temporary car park areas.
Toxic Materials	Cement slurry, asphalt primer, solvents, cleaning agents, and wash waters (eg, from tile works).
Acids or Alkaline substances	Acid sulphate soil, cement slurry and wash waters.

4.2.2 Performance Criteria

The following site discharge pollutant criteria have been identified during the construction phase of the development.

Table 13: Construction Phase, Water Quality Performance Criteria

Pollutant	Criteria
Total Suspended Solids	90^{th} percentile <50mg/L
pH	6.5 – 8.5
Dissolved Oxygen	90^{th} percentile >80% saturation or 6mg/L
Hydrocarbons	No visible sheen on receiving waters
Litter	No visible litter washed from site.

4.2.3 Sediment and Erosion Controls

Sediment and erosion control devices (S&EC) employed on the site shall be designed and constructed in accordance with the Institution of Engineers, *Soil Erosion and Sediment Control Guidelines*.

Details of the proposed controls are shown on the Knobel Consulting Pty Ltd, *Sediment and Erosion Control Layout Plan* (Ref: K1868/P014/C) *and Sediment and Erosion Control Details Sheet* (Ref: K1868/P015/A*)* included as Appendix F.

Pre Construction

- Stabilised site access/exits from Fraser Drive and entry to development site;
- Sediment fences to be located along the contour lines downstream of disturbed areas;
- Diversion drain to divert clean runoff around the construction site;
- Sediment basins located at the site of the future bio retention basins; and
- Educate site personnel to the requirements of the Sediment and Erosion Control Plan.

Initial Construction – Bulk Earthworks

- Maintain construction access/exit, sediment fencing, sediment basin, catch drains and all other existing controls as required; and
- Confine construction activities to stages to minimise areas of disturbance at any given time.

Second Stage Construction

- Maintain construction access/exit, sediment fencing, sediment basins, catch drains, dust fences and all other existing controls as required;

- Progressively revegetate and turf finished areas;
- Convert sediment basins to bioretention and detention basins; and
- Drainage structure protection around field inlets and gully pits.

During construction, all areas of exposed soils allowing dust generation are to be suitably treated. Treatments will include mulching the soil and watering. Road accesses are to be regularly cleaned to prevent the transmission of soil on vehicle wheels and eliminate any build up of typical road dirt and tyre dusts from delivery vehicles.

Adequate waste disposal facilities are to be provided and maintained on the site to cater for all waste materials such as litter hydrocarbons, toxic materials, acids or alkaline substances.

4.2.4 Bioretention Systems Construction Controls

Protection of the filtration media and vegetation within bioretention systems is important during the construction phase of the development; uncontrolled runoff can cause sedimentation and clogging of the filter media.

For the proposed bioretention basin the contractor shall adopt a staged implementation. The basins will act as sediment basins during the earthworks and major construction phases.

When the basins are converted to bio retention basins during the later stages of the construction phase, the filter media shall be covered with a layer of geo-textile, 50 mm of soil and turf strips laid perpendicular to the direction of flow.

4.2.5 Water Quality Monitoring and Inspections

To ensure that the water quality objectives are being met during the construction phase of the development water quality monitoring shall be conducted at one (1) monitoring station downstream of the subject site.

Water quality monitoring shall use a calibrated probe and/or sampling and testing at a NATA registered laboratory.

Location: Monitoring Station MS1 and MS2 at the downstream boundary of the proposed bioretention basins.

Parameters: Site discharge criteria.

Frequency: Following at least 30 mm of rainfall in a 24 hour period.

The contractor shall be responsible for the inspection and maintenance of all sediment and erosion control devices. Additional controls and review of existing controls shall be undertaken in response to the results of the above-mentioned monitoring program.

4.2.6 Reporting

An inspection report shall be written by a suitably qualified and experienced scientist/engineer following each water quality monitoring episode. The report shall include at least the following information.

- Name, address and real property description for the development site;
- Council file reference number (if known);
- Monitoring locations;
- Performance criteria;
- Results for each monitoring location, identifying any breaches of performance criteria, and
- Recommended corrective actions to be taken and additional sediment and erosion controls, if required.

Inspection reports shall be provided to the contractor for their action and compilation in an on-site register. If the above-mentioned performance criteria are exceeded and results from the downstream monitoring stations show significant deterioration from upstream results (if applicable), the contractor shall implement all recommendation of the inspection report within (1) working day of receipt of the report.

4.3 Operational Phase

4.3.1 Water Quality Objectives

Tweed Shire Council has advised that they are in the process of adopting the Water by Design water quality guidelines. As such, the following water quality objectives have been adopted for the subject site based on the Water by Design, *Water Sensitive Urban Design Technical Design Guidelines, 2006.*

Reduction in the average annual pollutant load discharging from the site:

- 80% for suspended solids;
- 45% for nitrogen;
- 60% for phosphorus; and
- 90% for gross pollutants

4.3.2 Land Use Parameters

In accordance with the recommendations of the Tweed Shire Council *Development Design Specification D7 (Version 1.3.)* the *Urban Tweed* Catchment Node Parameter has been applied to the subject site to predict the quality of the stormwater discharging from the site. Modelling has used one (1) year of rainfall data (1978) with a six (6) min time step. Evaporation data and Catchment Node Parameters have been applied in accordance with the above mentioned specification. Separate models have been constructed to model the discharge to LPOD-A and LPOD-B.

A breakdown of the various land uses within the developed site is shown in Table 14.

Table 14: Land Use Delineation

Land Use Category	Land Type	Area (m^2)	% Impervious
Typical Lot ($180m^2$ each lot)	Roof to Tank	40	100
	Roof to Drainage	40	100
	Driveway/Carparking	30	100
	Other Ground	70	20
Road Reserve	Roadway	-	75
	Verge	-	25
Open Space	All	-	0

4.3.3 Stormwater Quality Improvement Devices

Rainwater tanks shall collect a portion of roofwater runoff for reuse onsite and a portion of the allotment ground is assumed to travel over road verges acting as buffers. Runoff from impervious areas is designed to runoff across landscaped buffers where possible. Stormwater runoff shall be piped into bioretention basins located at the eastern and north western ends of the site. The piped flow will pass through a GPT device before entering the basins for preliminary treatment. A flow chart of the proposed stormwater quality treatment train is show as Figure 9 and 10 below.

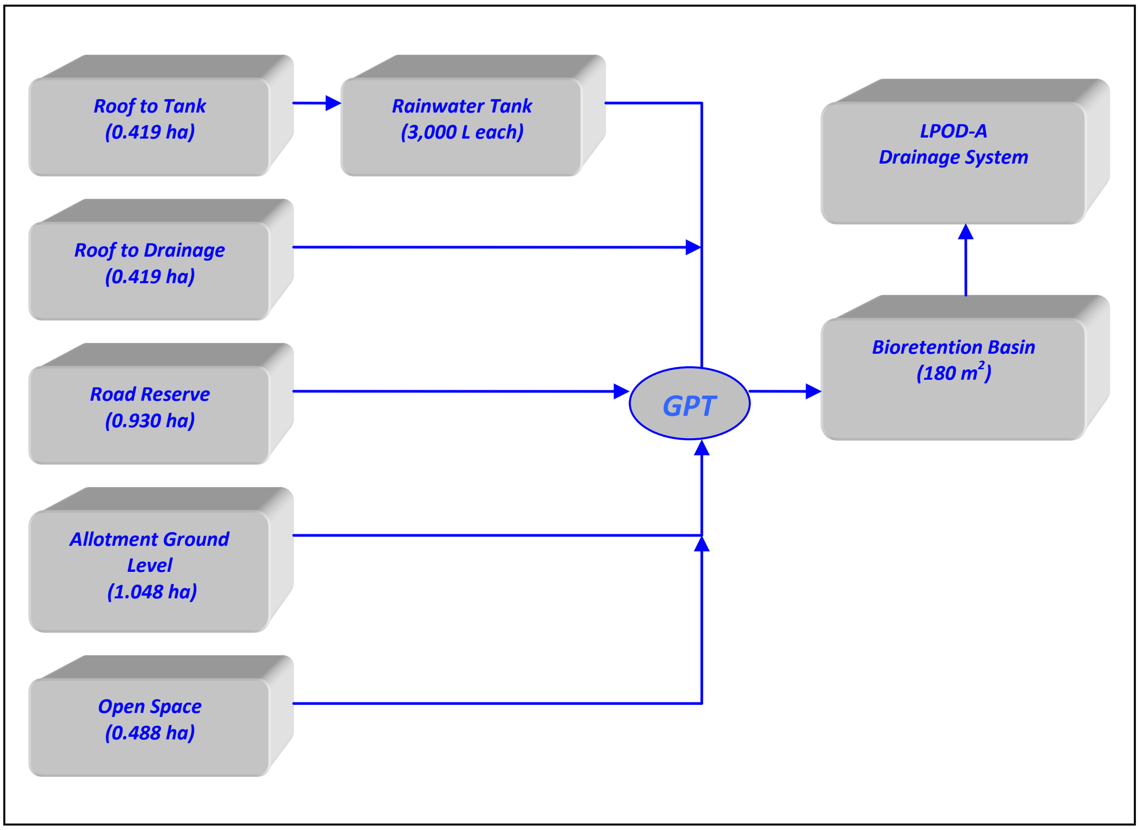

Figure 9: Stormwater Quality Treatment Train Flow Chart – LPOD-A

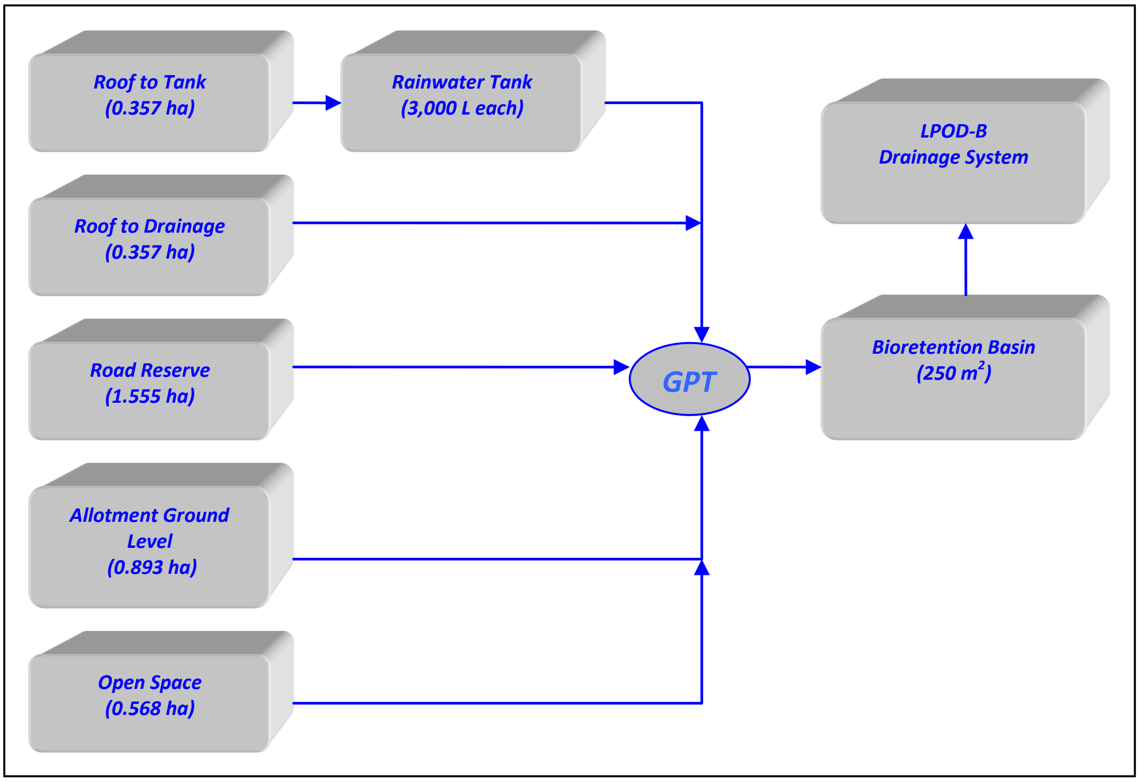

Figure 10: Stormwater Quality Treatment Train Flow Chart – LPOD-B

4.3.4 Design Parameters of the Stormwater Quality Improvement System

Detailed design of the stormwater quality treatment train shall be in accordance with the Healthy Waterways, *Water Sensitive Urban Design Technical Design Guidelines for South East Queensland (2006)*.

The bio retention basins will be located in the areas marked for stormwater management on the Knobel Consulting Pty ltd, *Post Development Internal Catchment Plan* (Ref: K1868/P011/E) included in Appendix C Note that the area demarcated exceeds the required area detailed below. The GPT units will be placed on the pipelines upstream of the outlet to the basins.

GROSS POLLUTANT TRAP

Gross pollutant traps (GPT) are proposed within the treatment train to remove sediment and gross pollutants from stormwater runoff upstream of the proposed bio-retention basins. Tweed does not currently have a standard specification for the modelling of GPT devices in MUSIC. Therefore the GPT has been modelled in accordance with the Gold Coast City Council, *MUSIC Modelling Guidelines 2006* by applying the following removal efficiencies:

- TSS 50%;
- TP 20%;
- TN 0%.

It is noted that these specifications are considered very conservative. Test results and specifications from the manufacturers of GPT devices state that actual removal efficiency rates are higher than these applied in the model.

BIORETENTION BASIN

A bioretention basin is designed to pond stormwater allowing it to percolate through a layer of filter media, typically sandy loam. Runoff passing through the filter media is collected within a perforated pipe discharging to downstream drainage infrastructure.

The bioretention basin has been modelled with the following properties:

<u>CATCHMENT INT A</u>

- Filter media - Sandy Loam
  - 5 – 10% Organic Content in accordance with AS12894.1.1
  - Average D_{50} 0.50 mm
  - K_{sat} 180 mm/hr
- Filter media depth - 0.60 m
- Water quality depth - 0.30 m
- Filter media area - 180 m^2
- Surface area - 200 m^2

<u>CATCHMENTS INT B AND INT C</u>

- Filter media - Sandy Loam
  - 5 – 10% Organic Content in accordance with AS12894.1.1
  - Average D_{50} 0.50 mm
  - K_{sat} 180 mm/hr
- Filter media depth - 0.60 m
- Water quality depth - 0.30 m
- Filter media area - 250 m^2
- Surface area - 295 m^2

Plant selection within the bio-retention basin is to be in accordance with the Landscape Architects plans and should generally use plant species listed in Table A1 and A2 of the Healthy Waterways, *Technical Design Guidelines for South East Queensland (2006)*.

SEDIMENT FOREBAY

A Coarse Sediment Forebay is proposed to remove coarse sediment from stormwater and provide scour and erosion protection prior to flowing across the surface of the filter material. The coarse sediment forebay will be sized in accordance with Healthy Waterways, *Technical Design Guidelines for South East Queensland (2006)* subject to requirements for flows passing through a GPT unit.

RAINWATER TANKS

Each detached dwelling will be connected to a minimum 3,000 L rainwater tank plumbed into the laundry, outdoor hose and toilet. A 5000 L rainwater tank has been assumed for connection to the community centre roof.

A reuse rate of 216 L/day per dwelling and the community centre has been assumed. This has been derived from 66% of a reuse rate of 324 L/day applied to standard detached dwellings.

LANDSCAPED BUFFERS

Stormwater runoff from a portion of impervious allotment areas such as paths and outdoor areas will be directed onto lawns, garden beds and across road verges acting as landscaped buffers before being collected within the proposed minor drainage system.

Buffers have been included within the MUSIC model with the following parameters

- Percentage of upstream area buffered: 50%
- Buffer area (% of upstream imp. area): 20%

4.3.5 Post Development Modelling Results - Mitigated

The SQID's described in the previous section are designed to achieve the Water Quality Objectives for the subject site only.

The MUSIC modelling results are shown in Table 15 below and demonstrate that the proposed water quality treatment train achieves the required pollutant removal rates are achieved.

Table 15: Catchment Int A Mitigated Pollutant Loads

Parameter	Post	Post Mitigated	Removal %
Flow (ML/yr)	41.0	37.1	9.6
TSS (kg/yr)	4270	726	83.0
TP (kg/yr)	10.5	4.04	61.4
TN (kg/yr)	63.6	34.4	45.9
Gross Pollutants (kg/yr)	811	0	100

Table 16: Catchment Int B and Int C Mitigated Pollutant Loads

Parameter	Post	Post Mitigated	Removal %
Flow (ML/yr)	46.6	43.3	6.7
TSS (kg/yr)	4780	777	83.7
TP (kg/yr)	11.6	4.45	61.7
TN (kg/yr)	72.1	39.4	45.3
Gross Pollutants (kg/yr)	922	0	100

A typical cross section of each of the bio retention basins is shown on Knobel Consulting Pty Ltd, *Detention and Bio-Retention Basin Plan* (Ref: K1868/P037/A) included as Appendix D (note there is no pipe outlet apart from the filtration pipes for Basin B).

5.0 MANAGEMENT AND MAINTENANCE PLAN

The proposed stormwater quality improvement devices will require maintenance and monitoring to ensure that they function as designed. The following section provides an outline of the necessary maintenance tasks for the proposed devices.

5.1 Swale Maintenance

The most intensive period of maintenance for a swale is during the grass establishment period (first six months) when weed removal and regular watering may be required. It is also the time when large flows could erode the unestablished grassed area. Inspection and removal of debris should be done regularly, and debris should be removed whenever it is observed on a site.

Typical maintenance of bioretention basin elements will involve:

- Routine inspection of the swale profile to identify any areas of obvious scouring from storm flows and damage to the profile from vehicles;
- Repairing any damage to the profile resulting from scour, rill erosion or vehicle damage by replacement of appropriate fill (to match onsite soils) and revegetating;
- Regular watering/ irrigation of the grass until it is established;
- Removal and management of invasive weeds (herbicides should not be used);
- Removal of grass sections that have died and replacement with grass of equivalent size and species;
- Pruning to remove dead or diseased vegetation material;

5.2 Gross Pollutant Trap (GPT) Maintenance

The management and maintenance requirements for the chosen device will be specified by the manufacturer and should be carried out accordingly.

6.0 ASSET HANDOVER

The proposed SQID's shall be dedicated to the Council following the time of sealing of survey plans for the proposed development. A twelve (12) month on-maintenance period is proposed for the devices. During this period the developer shall be responsible for the maintenance of the devices.

An asset Handover checklist is provided as Table 17 as a guideline for the necessary steps to be taken prior to the end of the on-maintenance period.

Table 17: Asset Handover Checklist

Asset Location:		
Construction By:		
On-maintenance period:		
Treatment:	**Y**	**N**
Actual treatment performance equivalent to design:		
Maintenance:		
Maintenance Plans Provided:		
Inspection and maintenance undertaken as per maintenance plan:		
Inspection and maintenance forms provided:		
Asset inspected for defects:		
Asset Information:		
Design assessment checklist provided:		
As constructed plans provided:		
Copies of all permits provided:		
Digital files provided:		

7.0 CONCLUSIONS

Knobel Consulting Pty Ltd has been commissioned by Proportional Property Investments Pty Ltd to prepare a *Preliminary Stormwater Management Plan (SWMP-P)* and supporting engineering documentation for a proposed community title tourist accommodation development at Lot 1 on DP1168904 located on Kirkwood Road, Tweed Heads South (the subject site). This report has been revised from the original issue in response to Tweed Shire Council's Information Request dated 22 March 2013.

The stormwater quality improvement system for the operational phase of the development consists of a Gross Pollutant Trap and Bio retention on treating all site runoff. Compliance with Council's water quality objectives has been demonstrated using the MUSIC software programme. Monitoring and maintenance details have been provided as well as a construction phase water quality control plan. Stormwater detention requirements have been identified and detailed modelling has been carried out using the XPswmm software programme. The required discharge controls have been confirmed in the model and plans provided for the detention basins. The proposed basin arrangements ensure that there is no adverse impact to the downstream environment due to the development.

APPENDIX

A

Paul Ziekulis Architect

Site Plan

(Ref: SD-A01/09)