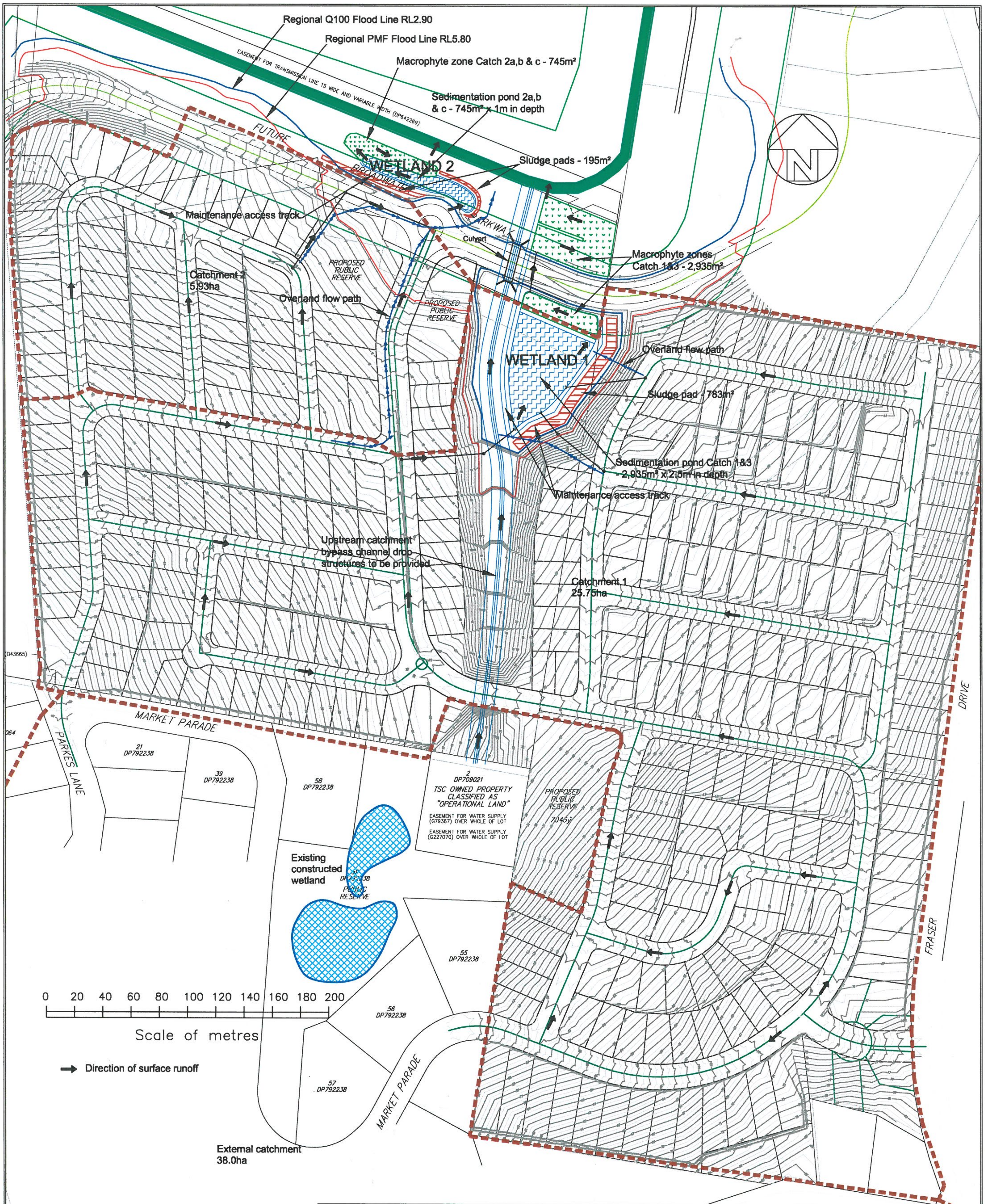


GILBERT+SUTHERLAND
agriculture · water · environment

Eastside
5/232 Robina Town Centre Drive, Robina, Qld. 4226
Phone 55789944 Mobile 0418 760919 Fax 55789945

PROJECT

METRICON QLD PTY LTD

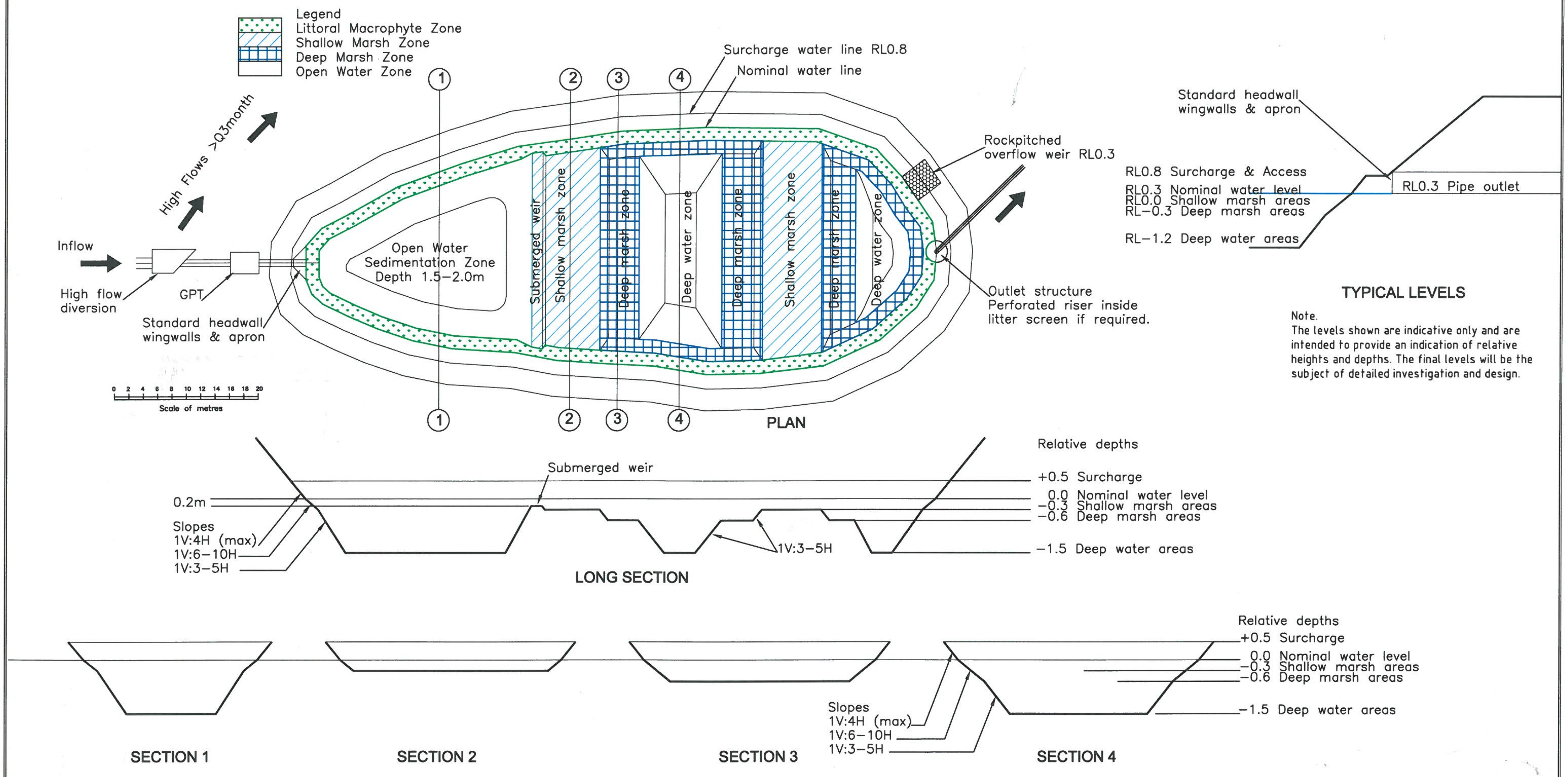
ALTITUDE ASPIRE, TERRANORA AREA E

PROPOSED DEVELOPMENT

STORMWATER MANAGEMENT DEVICES

FIGURED DIMENSIONS TO BE READ IN PREFERENCE TO SCALING.	APPROVED	SCALE AS SHOWN	DRAWN AGG
		DATE 23/11/10	CHECKED

DRAWING No. GJ0901.1.5a



Notes.
1. Where possible wetlands should be constructed "off–line" so that flows >Q3months are diverted away from the system.
2. Where practicable flows into the wetland should be pretreated by means of a Gross Pollutant Trap (GPT). Where the wetland is "on–line" the GPT may take the form of a trash rack.
3. If the wetland is required to provide stormwater detention functions, the outlet should take the form of a perforated riser so that water levels return to the normal operating level within 72 hours. In other cases the outlet may take the form of a simple overflow weir.
4. The inlet zone should be designed to trap particles >125um.
5. All areas to be planted with macrophytes to be covered with 150mm topsoil.

GILBERT+SUTHERLAND
agriculture · water · environment

Eastside
5/232 Robina Town Centre Drive, Robina, Qld. 4226
Phone 55789944 Mobile 0418 760919 Fax 55789945

FIGURED DIMENSIONS TO BE READ IN PREFERENCE TO SCALING.	APPROVED	PROJECT METRICON QLD PTY LTD ALTITUDE ASPIRE, TERRANORA AREA E STORMWATER MANAGEMENT CONSTRUCTED WETLAND DETAILS		
		SCALE AS SHOWN	DRAWN AGG	DRAWING No.
		DATE 24/11/10	CHECKED	GJ0901.1.6

4) Stormwater peak flow assessment method

A hydrological assessment was undertaken to assess the extent of flow attenuation measures required under a range of rainfall events using the Rational Method.

4.1 Rational Method

The Rational Method (Section 5.02 QUDM) is flexible in its data requirements and is able to produce satisfactory estimates of peak discharges from a site with the following data input:

- local intensity frequency duration data
- catchment areas
- runoff coefficients.

Discharge using the Rational Method is calculated by:

$$Q = \frac{F_Y C_{10} I A}{360}$$

where: Q = Peak flow (m^3/s)

F_Y = Frequency factor

C_{10} = Runoff coefficient (10yr)

I = Rainfall intensity (mm/hr)

A = Catchment area (ha)

Peak discharges were estimated for the pre-developed and post-developed case for events with average recurrence intervals (ARI) between 1 and 100 years.

4.1.1 Time of concentration

Pre-development
The time of concentration for the pre-developed site was calculated using the Bransby Williams equation in accordance with the recommendation contained in QUDM.

Details of the assumptions for the procedures are shown below.

$$t_c = \frac{92.7}{A^{0.1} S^{0.2}}$$

where: t_c = Time of concentration (min)

L = Longest flow path length (km)

A = Catchment area (ha)

S = Equal area slope (m/km)

Post-development
A drainage system would be installed to collect the roof runoff and convey it to rainwater storage tank.

Runoff from the remaining area and any overflow from the rainwater tanks would be collected and conveyed via the stormwater drainage system to the end-of-line constructed wetland. Discharge from the constructed wetland would be released at a legal point of discharge on the subject land.

Flow attenuation would be provided within the constructed wetland prior to discharge to ensure that flow leaving the site would be no greater than in the present (undeveloped) state.

4.1.2 Local intensity frequency

Rainfall intensities for the simulation of design rainfall events were calculated using the polynomial factors obtained from the Bureau of Meteorology. The values for the factors used are shown in Appendix 1 and the IFD Table used is provided in Appendix 2. It is noted that these intensities are slightly higher than those provided in Table D5.1 of Tweed Shire Council's Development Design Specification D5 Stormwater Drainage Design (TSC D5), particularly for 100 year ARI events.

4.1.3 Runoff coefficients

The runoff coefficient for the 10 year (C_{10}) average recurrence interval (ARI) was estimated as per recommendations in Section 4.05 of QUDM 2007.

For the existing upstream development which is low density residential, a fraction impervious of 0.40 was selected from Table 4.05.1. This value when entered in Table 4.05.3(a) provides a C_{10} value of 0.78.

For the proposed development site in its undeveloped state the fraction impervious is 0.0. Table 4.05.3(b) then indicates that for a site with good grass cover and medium permeability, a value of 0.70 should be adopted for C_{10}.

For the Developed Case, a fraction impervious of 0.8 was adopted for a Low Density Urban Residential development including roads. Table 4.05.3(a) then yields a value of 0.88 for C_{10}.

4.1.4 Frequency factors

Frequency factors have been adopted from TSC D5 Table D5.2 for coastal areas below 500m AHD, as presented in Table 4.1.

Table 4.1 Frequency factors

ARI (years)	Frequency Factor F_y
1	0.67
2	0.81
5	0.92
10	1.00
20	1.07
50	1.17
100	1.28

4.2 Detention storage estimate

An assessment was carried out to determine the detention volume required to reduce the estimated peak discharge of runoff from the developed catchment to a level equivalent to the estimated pre-developed rate. The analysis was carried out in accordance with Section 5.05.1 of QUDM 2007 for sizing of detention basins.

Sizing of the detention store was undertaken using each of the following relationships:

- Culp, 1948: $\frac{V_s}{V_i} = \frac{r}{3}(1 + 2r)$
- Boyd, 1989: $\frac{V_s}{V_i} = r$
- Carroll, 1990: $\frac{V_s}{V_i} = \frac{r}{8}(3 + 5r)$
- Basha, 1994: $\frac{V_s}{V_i} = \frac{r}{3}(2 + r)$

where: r = Reduction ratio
= $(Q_i - Q_o) / Q_i$
Q_o = Peak outflow rate (m^3/s)
Q_i = Peak inflow rate (m^3/s)
V_s = The required storage volume (m^3)
V_i = The inflow volume (m^3)
= $(4t_cQ_i) / 3$

Calculations were carried out using each method for the standard ARI events and the average detention size based on these methods was used to estimate for the required size of the detention store.

5) Stormwater quantity assessment results

5.1 Rational Method peak flows

5.1.1 Pre-developed Case

As it is intended that the upstream catchment would be conveyed through the site to a point of discharge, the flows to be conveyed by the bypass channel need to be estimated. The flows from the balance of the site will be attenuated and are estimated separately.

External Catchment (37.9ha)
The time of concentration for the existing developed external catchment was calculated using the Bransby-Williams equation with the following inputs.

$$t_c = \frac{92.7}{A^{0.1}S^{0.2}}$$

t_c = Time of concentration (min)

L = Longest flow path length (km) = 0.87

A = Catchment area (ha) = 38.0

S = Equal area slope (m/km) = 118.4

The time of concentration was estimated to be 21.6 minutes, which was rounded to 22.0 minutes.

The assumptions adopted to determine the peak flow rates discharging from each lot in its present condition under a range of rainfall events are given in Table 5.1.1.

Table 5.1.1 External catchment developed peak flow inputs for rational method

ARI (yrs)	C_{10}	$C_Y=F_Y.C_{10}$	I (mm/hr)
1	0.78	0.52	69.75
2	0.78	0.63	88.72
5	0.78	0.72	110.54
10	0.78	0.78	122.94
20	0.78	0.83	139.73
50	0.78	0.91	161.51
100	0.78	1.00	177.93

The resultant peak flows discharging from the site under a range of rainfall events are summarised in Table 6.1.2.

Table 5.1.2 External catchment peak flows

ARI (years)	Peak flow Q (m^3/s)
1	3.8
2	5.9
5	8.4
10	10.1
20	12.2
50	15.5
100	18.8

Proposed development
Catchment 1 (25.75ha)
Using the Bransby-Williams equation with the following inputs:

L = Longest flow path length (km) = 0.82

A = Catchment area (ha) = 25.75

S = Equal area slope (m/km) = 68.31

The estimated time of concentration is 23.7 minutes which was rounded to 24.0 minutes.

Table 5.1.1 Catchment 1 pre-development peak flow inputs for rational method

ARI (yrs)	C_{10}	$C_Y=F_Y.C_{10}$	I (mm/hr)
1	0.70	0.47	66.79
2	0.70	0.57	85.00
5	0.70	0.64	106.04
10	0.70	0.70	118.02
20	0.70	0.75	134.22
50	0.70	0.82	155.24
100	0.70	0.90	171.08

Catchment 2 (5.93ha)
Using the Bransby-Williams equation with the following inputs:

L = Longest flow path length (km) = 0.36

A = Catchment area (ha) = 5.93

S = Equal area slope (m/km) = 113.7

The estimated time of concentration is 11.0 minutes.

Table 5.1.1 Catchment 2 pre-development peak flow inputs for rational method

ARI (yrs)	C_{10}	C_Y=F_Y.C_{10}	I (mm/hr)
1	0.70	0.47	95.9
2	0.70	0.57	121.6
5	0.70	0.64	149.9
10	0.70	0.70	165.7
20	0.70	0.75	187.5
50	0.70	0.82	215.5
100	0.70	0.90	236.6

5.1.2 Post-development Case

Catchment 1

The time of concentration has been estimated using an assumed inlet time of 5 minutes and an estimated kerb and drainline flow time (using an assumed average velocity of 4.0m/sec) of 3.2 minutes to arrive at a time of concentration of 8.2 minutes.

The assumptions adopted to determine the peak flow rates discharging from the site in its developed condition under a range of rainfall events are given in Table 6.1.2.

Table 6.1.2 Post-developed peak flow inputs for rational method

ARI (yrs)	C_{10}	C_Y=F_Y.C_{10}	I (mm/hr)
1	0.88	0.590	109.3
2	0.88	0.713	138.4
5	0.88	0.810	169.9
10	0.88	0.880	187.3
20	0.88	0.942	211.5
50	0.88	1.000	242.6
100	0.88	1.000	266.0

Table 6.1.3 presents the resultant peak flows discharging from this catchment before and after the development for a range of rainfall events.

Table 6.1.3 Catchment 1 peak flows by rational method

ARI (years)	Pre-developed peak flow Q (m^3/s)	Post-developed peak flow Q (m^3/s)
1	2.2	4.6
2	3.5	7.1
5	4.9	9.8
10	5.9	11.8
20	7.2	14.3
50	9.1	17.4
100	11.0	19.0

A comparison of the pre-developed and post-developed flows indicates that peak flows would increase by up to 105% hence an attenuation device is required.

Catchment 2

The time of concentration has been estimated using an assumed inlet time of 5 minutes and an estimated kerb and drainline flow time (using an assumed average velocity of 4.0m/sec) of 3.0 minutes to arrive at a time of concentration of 8.0 minutes.

The assumptions adopted to determine the peak flow rates discharging from the site in its developed condition under a range of rainfall events are given in Table 6.1.2.

Table 6.1.2 Post-developed peak flow inputs for rational method

ARI (yrs)	C_{10}	C_Y=F_Y.C_{10}	I (mm/hr)
1	0.88	0.590	109.3
2	0.88	0.713	138.4
5	0.88	0.810	169.9
10	0.88	0.880	187.3
20	0.88	0.942	211.5
50	0.88	1.000	242.6
100	0.88	1.000	266.0

Table 6.1.3 presents the resultant peak flows discharging from this catchment before and after the development for a range of rainfall events.

Table 6.1.3 Catchment 1 peak flows by rational method

ARI (years)	Pre-developed peak flow Q (m^3/s)	Post-developed peak flow Q (m^3/s)
1	0.7	1.1
2	1.1	1.6
5	1.6	2.3
10	1.9	2.7
20	2.3	3.3
50	2.9	4.0
100	3.5	4.4

A comparison of the pre-developed and post-developed flows indicates that peak flows would increase by up to 43% hence an attenuation device is required.

5.2 Detention sizing

An estimate of the required storage volume was carried out as outlined in Section 5.2. Appendix 3 provides the results for the initial detention sizing for the site by each of the methods described.

The estimated volume of detention storage required for Catchment 1 and Catchment 2 is 3,200 to 5,200m^3 and 270 to 580m^3, respectively. It should be noted that these are preliminary estimates only based on the current proposed layout and the assumptions described above. These volumes would be refined during the detailed design process and full calculations would be provided in support of an application for a construction certificate.

5.3 Detention storage details

The required detention storage for the site will be provided within the proposed constructed wetlands described in Section 4.3. Table 5.3.1 provides a summary of the surface area and required depth for the detention store.

Table 5.3.1 Detention storage available

Catchment No.	Storage volume required (m^3)	Surface Area (m^2)	Depth (m)
1	3200 - 5200	4,500	0.71 – 1.15
2	270 - 580	1,480	0.18 – 0.39

Our assessments indicate that sufficient space is available to provide the required volume of detention storage. Provided the recommended detention storage is implemented, the peak flow rates for the total developed site would be sufficiently attenuated. Care should be taken to release these flows to the SEPP14 area as diffuse flows by means of appropriate flow spreaders.

These results are considered acceptable given that they are generally within the limits of accuracy of the methods and assumptions used.

6) Flooding assessments

6.1 Local flooding

As required by Tweed Shire Council's Development Design Specification D5 for Stormwater Drainage Design, and Section 7.03 of QUDM 2007, stormwater runoff from local catchment based storm events would be managed by the use of a major and minor system approach. This means that runoff from storm events up to and including 5 years Average Recurrence Interval (ARI) would be conveyed to a legal point of discharge by means of an underground piped drainage system.

Runoff from larger storms would be conveyed by the minor system together with a system of overland flow paths. It is the flow depth, velocity and safety issues in these overland flow paths that are considered in the local flooding assessment.

Potential overland flow paths are identified on Drawing No. GJ0901.1.4. This drawing indicates that sufficient space is available within the present design to provide adequate overland flow paths.

Details of flow depth and velocity are not available at this time. These details would be prepared as part of the detailed design process and would be submitted to Council for approval as part of an application for a Construction Certificate.

6.2 Regional flooding

As distinct from local flooding, regional flooding is the result of wider area storm events that cause simultaneous flooding in the Tweed River, Cobaki, Terranora and other creeks.

TSC has had regional flood studies completed. Based on the latest studies which incorporate 0.91m sea level rise and 10% increase in rainfall intensity, the anticipated 100 year ARI level in the northern portion of the site would be RL2.9m AHD. The Probable Maximum Flood (PMF) would be RL5.8m AHD. These flood lines have been added to Drawing No. GJ0901.1.5a enclosed.

The Broadwater Parkway would be designed to be above the 100 year ARI level to ensure that access would be available during a 100 year ARI event. During larger less frequent events when the Broadwater Parkway is not trafficable, access would be available up the hill via Market Parade.

The existing contours for the site indicate that approximately 3.6ha is below RL2.9m AHD. While there would be some loss of flood storage due to the construction of the Broadwater Parkway and the constructed wetlands within this area, this loss is inconsequential in terms of the regional flood storage and levels. This minimal impact would be demonstrated by updating the regional modelling during the detailed design process.

7) Erosion and sediment control

7.1 Objectives

A detailed Erosion and Sediment Control Plan (ESCP) would be prepared during the detailed design phase to accompany an application for a Construction Certificate. The main objective of this ESCP would be to implement the requirements of Tweed Shire Council's *'Code of Practice for Soil and Water Management on Construction Works'* as required in Council's *'Development Design Specification, D7, Stormwater Quality'*. Additionally, the ESCP would provide information on site-specific management issues to minimise potential environmental impacts from the development during the construction phase.

The control measures detailed in this ESCP would be developed to minimise impacts on the environment and achieve the following objectives:

- Minimise soil erosion and exposure.
- Minimise transportation of eroded soil by air and water.
- Limit suspended solids concentration in stormwater runoff to not more than 50mg/L.
- Limit/minimise the amount of site disturbance.
- Isolate the site by diverting clean upstream 'run on' water around the development.
- Control runoff and sediment at its source rather than at one final point.
- Stage ground disturbance/earthworks and progressively revegetate the site where possible to reduce the area contributing sediment.
- Retain topsoil for revegetation works.
- Locate sediment control structures where they are most effective and efficient.

7.2 Implementation

The ESCP would require the Proponent to mitigate the potential environmental impacts associated with the construction of the subdivision works.

It is noted that the owner of the land being developed is responsible for the erosion and sediment control throughout the construction phase. In addition, the owner shall also be responsible for all persons, including employees, plant operators, contractors, subcontractors, delivery drivers, etc who may cause erosion and sediment generation. This responsibility also extends to adjacent land where construction activities may have encroached upon and caused sedimentation and erosion.

It is intended that the ESCP would provide a set of performance criteria and guiding principles with which the engineering designs for the development would comply. The plans and specifications forming part of the construction contract for each stage should also include these performance criteria.

The estate would be developed in stages to minimise the potential for soil erosion and water pollution and this would enable the site to be progressively rehabilitated as the development proceeds. As soon as is practicable, after the completion of the earthworks in each stage, the lots would be topsoiled and reseeded to establish a fast growing cover crop which would minimise erosion and movement of sediment across and off the site. On steeper slopes, hydro-mulching may be required.

Where ever possible the site shall remain grassed and otherwise undisturbed until construction commences.

7.2.1 Self-auditing system

According to TSC guidelines, where more than 2,500m^2 of land is disturbed, a self-auditing program is to be developed for the site. A site inspection self-audit and monitoring program shall be undertaken.

The self-audit shall be undertaken systematically onsite including the recording of the following information:

- installation/removal of any erosion and sediment control device
- the condition of each device employed (particularly outlet devices)
- circumstances contributing to damage to any devices, accidental or otherwise
- storage capacity available in pollution control structures, including:
  - waste receptacles and portable toilets
  - trash racks
  - sediment barriers and traps

    - gross pollutant traps
    - wetlands/temporary sedimentation basin
- time, date, volume and type of any additional flocculants
- the volumes of sediment removed from sediment retention systems, and where this sediment has been disposed (if required)
- maintenance or repair requirements (if any) for each device
- circumstances contributing to the damage to device
- repairs affected on erosion and pollution control devices.

7.2.2 Construction phase control measures

Prior to commencement of bulk earthworks, temporary erosion and sediment controls shall be installed. In particular, the existing large dam in the northern portion of the central watercourse should be retained and modified for use as a temporary sedimentation basin during the initial earthworks phases.

Existing vegetation in the central watercourse should be retained as long as practicable to act as a filter zone. Silt fences should be installed and maintained along the perimeter of the proposed earthworks. Where runoff from disturbed areas cannot be directed to the existing dam, additional sedimentation basins should be constructed. Where practicable, runoff from undisturbed areas should be diverted around disturbed areas and away from the temporary sedimentation basin. Runoff from the filled areas should be diverted by means of surface slopes and V-drains to the temporary sedimentation basin.

Should the existing dam not be used as a temporary sedimentation basin, additional temporary sedimentation basins shall be provided. Temporary sedimentation basins shall be designed in accordance with requirements detailed within 'Managing Urban Stormwater, Soils and Construction' Landcom, fourth edition, March 2004 . The majority of the soils on the site have been classified as Ferrosols (or Krasnozems) Morand 1996[1]. These soils generally have a moderate erodibility but for estimating sediment basin sizes these have been classified as Type F soils. The size of the basin required for each stage is as shown on Table 7.2.2.1.

Table 7.2.2.1 Stage areas and associated sediment basin sizes

Stage No.	Area (ha)	Surface Area (m^2)	Minimum Volume (m^3)
1	5.10	1,180	1,090
2	1.68	460	410
3	1.91	510	460
4	3.20	790	720
5	2.40	620	560
6	2.23	600	540
7	2.56	660	600
8	2.74	710	640
9	3.18	780	720
10	3.30	810	740

Type F sedimentation basins are designed for fine grained materials, which require a much longer 'residence' time to settle in a sedimentation/retention basin. These types of sedimentation basins often require the addition of a flocculant to assist in the settling process.

Gypsum is the most commonly used flocculant. Gypsum application rates are site specific and the appropriate rate for this site will need to be determined once construction commences. As a guide, Landcom provides a maximum rate of 70kg of gypsum per 100m^3 of water. Previous experience with soils similar to those found on this site indicates that an application rate of 30kg per 100m^3 should be adequate.

Other control measures such as (but not limited to) silt fences, contour drains, and straw bales should be installed and maintained in accordance with the recommendations contained in 'Managing Urban Stormwater, Soils and Construction' Landcom, fourth edition, March 2004. A conceptual erosion and sediment control plan is shown on Drawing No. GJ0901.1.7.

The central dam would be converted to a wetland once the site has been revegetated after completion of the earthworks. However the other control measures mentioned above must be installed in disturbed areas during the construction phase and maintained until landscaping has been completed and becomes established.

[1] Morand DT, 1996, Soil Landscapes of the Murwillumbah-Tweed Heads.