Concept Surface Water & Water Cycle Management Study

For Proposed Rezoning of Lot 4 DP 834254, 510 Beach Road, Berry

Prepared by: Jason Armstrong

SEEC Reference 19000408-WCMS-01B

10th March 2020

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Document Certification

This report has been developed based on agreed requirements as understood by SEEC at the time of investigation. It applies only to a specific task on the nominated lands. Other interpretations should not be made, including changes in scale or application to other projects. Any recommendations contained in this report are based on an honest appraisal of the opportunities and constraints that existed at the site at the time of investigation, subject to the limited scope and resources available. Within the confines of the above statements and to the best of my knowledge, this report does not contain any incomplete or misleading information.

Jason Armstrong
SEEC

10th March 2020

Document Table

Issue Number	By	Reviewed by	Date
1900408-WCMS-01A	JA	-	06/03/2020
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1 Introduction

SEEC have been commissioned by Mrs. Enid Hall owner of Lot 4 DP 834254 (Figure 1) to prepare this Water Cycle Management Study (WCMS). It is required to accompany a development application submission to Shoalhaven City Council to permit a large lot residential development over much of the site. This area was previously zoned RU1 (Primary Production) and has recently been rezoned to R5 (Large Lot Residential). There is a portion of the land that has been rezoned from E2 (Environmental Conservation) E1 (National Parks and Nature Reserves)

This WCMS shows how the requirements of Chapter G2 in Shoalhaven Council's Development Control Plan (DCP) (2014) can be met.

SEEC staff inspected the site on 5th May 2015. Weather on that day was cool and dry but the ground was sometimes wet following significant recent rainfall. Although the site inspection was undertaken some time ago SEEC have been informed that there have been no changes to the site since this initial inspection.

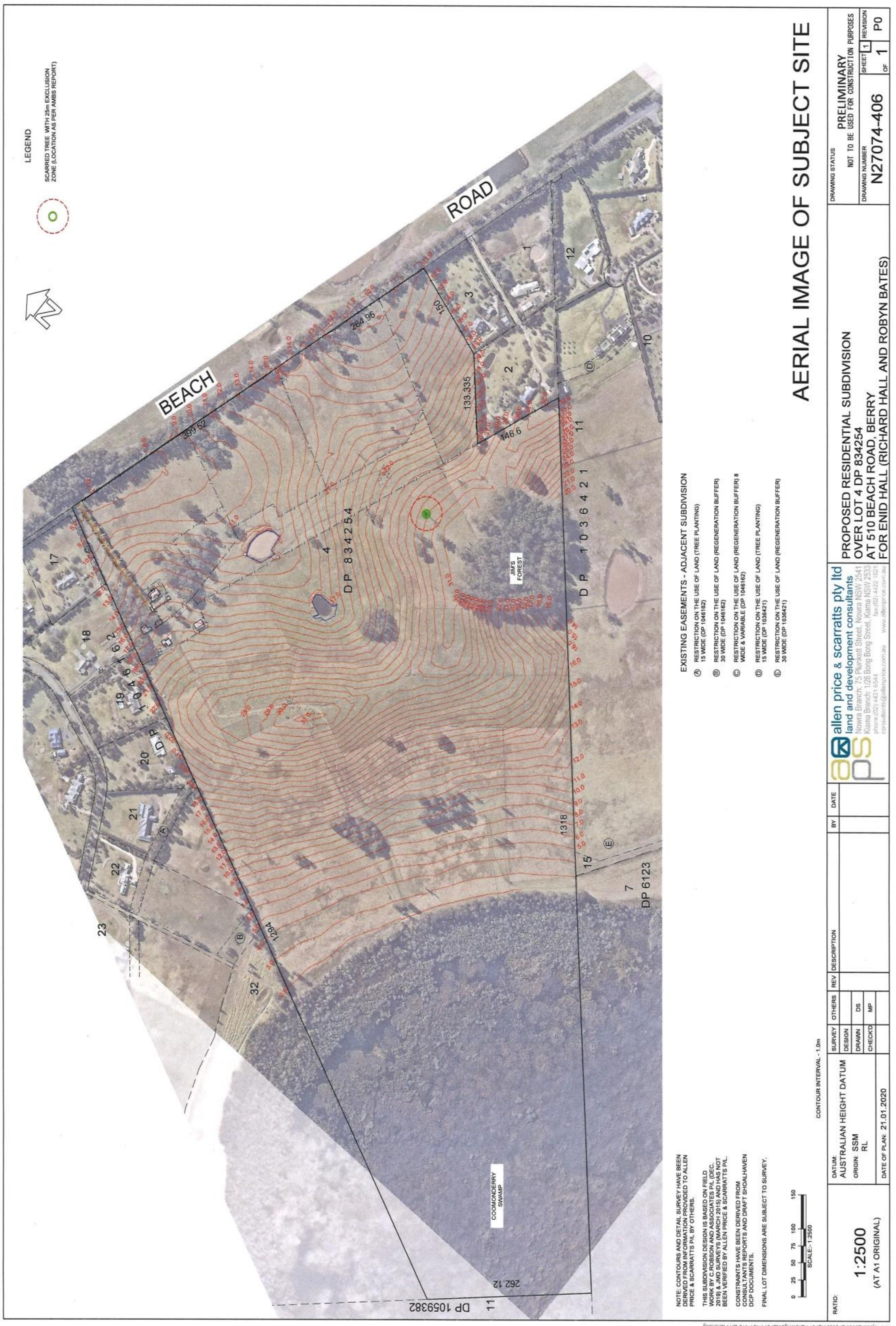

Figure 1 - Site Location

2 Project Description

49.3ha of the site has recently been rezoned from RU1 (Primary Production) to R5 (Large Lot Residential) and a section of the site to the south-west, approximately 25.6Ha, has been rezoned from E2 (Environmental Conservation) to E1 (National Parks and Nature Reserves).

Residential development would be confined to the R5 zoned land with lots ranging in size from 1 to 2 ha. They would be rural residential in nature, similar to neighbouring land to the west and the east (**Figure 1**). The proposed subdivision layout is shown in **Figure 2**. All the residential lots would require onsite wastewater management and rainwater tanks to supply potable water.

A road network would provide access to all residential lots. It is envisaged that the new development would drain to grassed table drains (swales), similar to the neighbouring developments to the east and west.

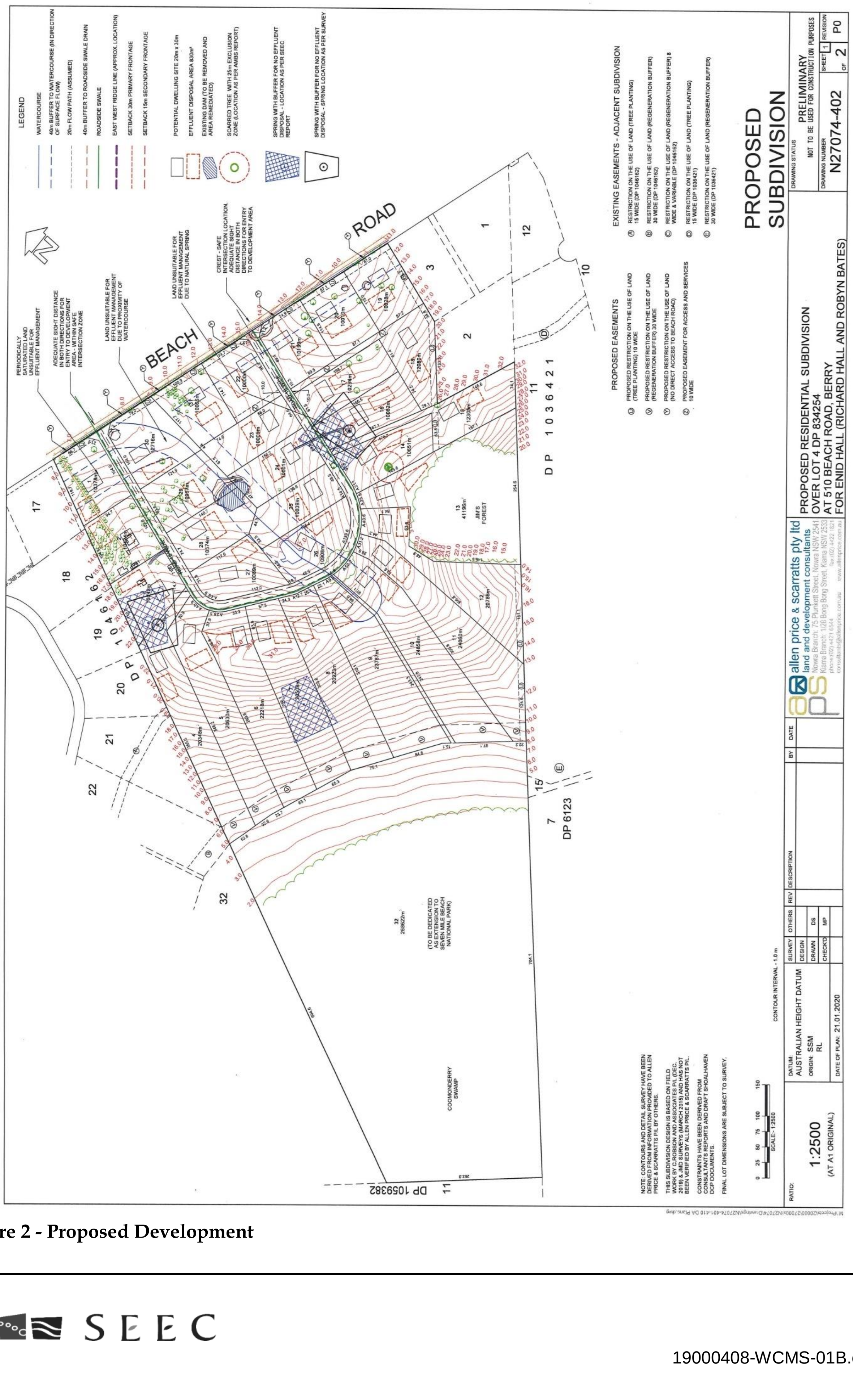

Figure 2 - Proposed Development

3 The Site

3.1 General Conditions

Lot 4 is a large rural property comprising about 55ha of pasture grass with the remainder (20ha) being part of Commonderry Swamp (**Figure 1**). At the time of inspection the pasture lands were used to graze cattle but in the past it has been used as a dairy farm. Other than Commonderry Swamp, minor remnant native vegetation occurs in parts but generally the land has been cleared. Aerial photography of the site over a number of years from 1945 shows the site has been in a similar condition since that time.

A farm house and associated sheds are located in the northwest part of the property. The house is serviced by a septic tank that discharges to an absorption bed located to the northwest and by a greywater trench located just to the east. **Figure 3** shows the typical conditions across the site.

Figure 3 - Looking south approximately along the alignment of a depression in the northwest corner of the site.

3.2 Climate

The area has a warm temperate climate with summer-dominated rainfall. Nearby Berry has a mean annual rainfall of 1,423 mm and nearby Kiama has a mean 97 wet days a year. Pan evaporation is relatively high (approximately 1,671 mm/year measured at Nowra RAN station).

3.3 Soils

According to mapping by The Department of Conservation and Land Management (Hazleton P.A., 1992) the site has three soil landscapes (**Figure 4**):

- The Coolangatta Soil Landscape which is a residual soil landscape and occupies most of that part of the site to be developed;
- The Shoalhaven Soil Landscape which is an alluvial soil landscape in the far northwest of the site; and
- The Seven Mile Soil Landscape which is an estuarine soil landscape found in Commonderry Swamp.

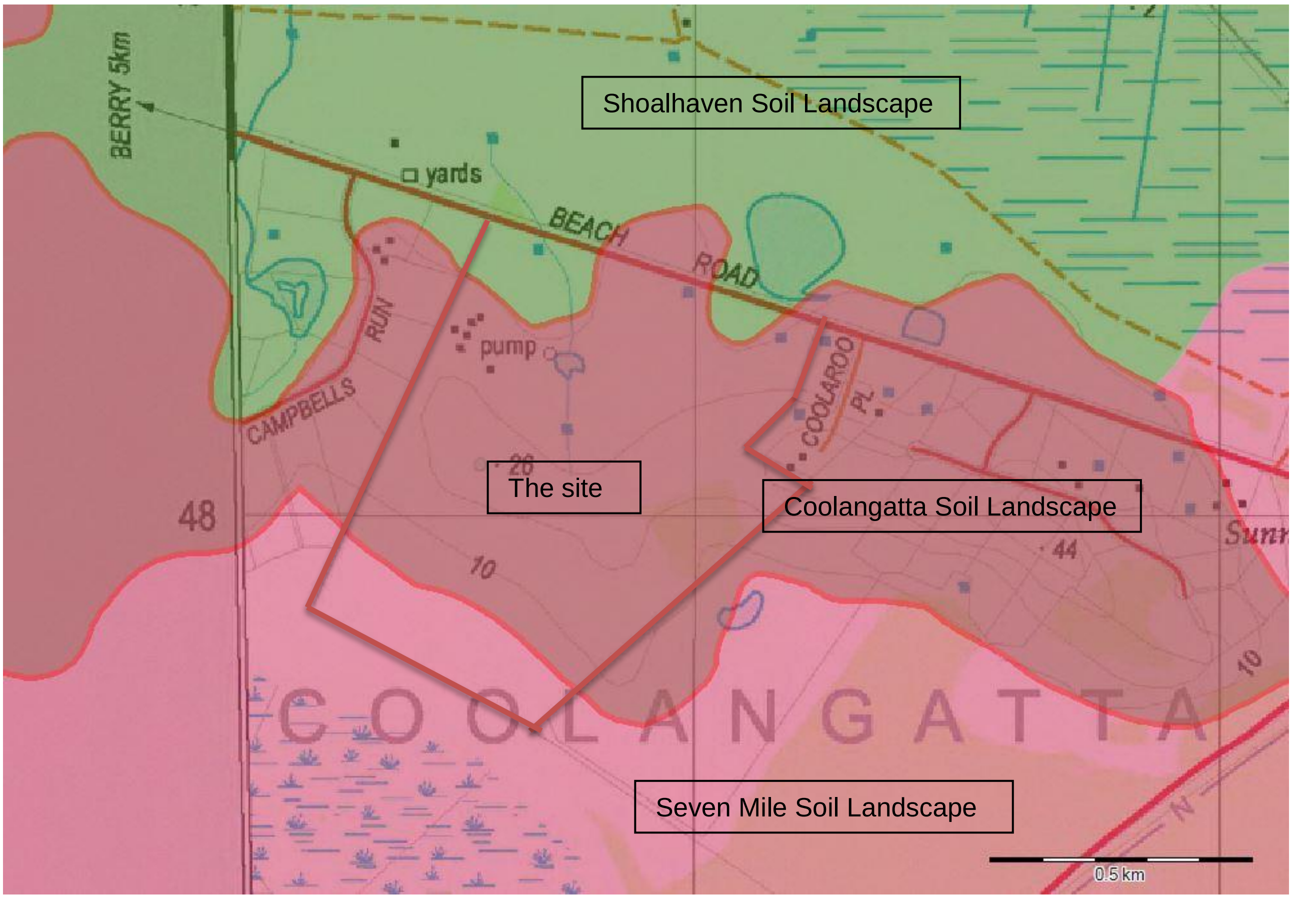

Figure 4 - Soil Landscape Mapping

A site specific soil investigation was undertaken by SEEC. A series of soil core samples were taken from boreholes across the site as shown in **Figure 5.** The results of that investigation suggest the extent of the Shoalhaven Soil Landscape in the far northwest is slightly smaller than mapped; as the Coolongatta Soil Landscape was present in this area. Lands in the far southwest would not be developed and were not investigated. All soil cores were taken on the Coolongatta Soil Landscape. The following soil profiles were observed:

Borehole 1:

0-200mm	Dark brown strongly pedal loam
200-400mm	Dark brown moderately pedal clay loam, sandy
400-900mm+	Light brown sandy clay loam with 10% fragments

Borehole 2:

0-150mm	Dark brown strongly pedal loam, saturated
150-600mm	Grey, weakly pedal, fine sandy clay loam to light clay
600-800mm	Mottled grey and orange brown sandy clay loam to light clay
800mm+	Bedrock (sandstone)

Borehole 3:

0-300mm	Dark brown strongly pedal loam
300-800mm	Dark brown moderately pedal clay loam to light clay, sandy
800-1,000mm	Dark brown moderately pedal clay loam, sandy with fragments
1,000mm+	Shale

Borehole 4:

0-200mm	Grey-brown loam
200-500mm	Brown light clay, moderately pedal
500-900mm	Mottled grey and orange medium clay, weakly pedal
900mm+	Shale

Borehole 5

0-100mm	Grey clay loam, weakly pedal
100-450mm	Grey mottled orange clay loam, moderately pedal
450-900mm	Grey mottled orange light to medium clay
900mm+	Shale

Figure 5 - Soil Sampling Locations

Soils were sent to NSW Department of Lands' Scone Research Laboratory and tested for a suite of effluent-disposal related tests. Topsoil and subsoil from BH2 and BH3 were composited separately and tested to give an indication of the average results. The results are given in **Table 1**. In summary, the boreholes and soil testing indicate that the soils at this site:

- Are consistently about 800 mm to 1,000 mm deep;
- Are moderately drained on the crest and higher side slopes but less well drained on footslopes where grey mottling occurs in the clay subsoil (*Minor to Moderate Limitation*[1]);
- Are slightly acidic, although this doesn't seem to affect grass growth (*Moderate Limitation*);
- Are not saline (*Minor Limitation*);
- Are not sodic (*Minor Limitation*);
- Are not significantly dispersive in the subsoil (EAT Class 2(1) *(Moderate Limitation);*
- Have good potential to sorb phosphorous *(Minor Limitation);*

[1] Limitations are those described in DLG (1998)

- Have moderate cation exchange capacity (about 20 cmol(+)/kg) *(Minor Limitation)*.

Table 1 - Laboratory Soil Test results

Lab No	Method	C1A/5	C2A/4	C2B/4	C5A/4 CEC & exchangeable cations (cmol (+)/ kg)						C8B/1	P9B/2	
	Sample Id	EC (dS/m)	pH	pH ($CaCl_2$)	CEC	Na	K	Ca	Mg	Al	P sorp (mg/kg)	EAT	Texture
1	15000106 BH 2 100 cm & 15000106 BH 3 100 cm	0.05	6.5	5.1	18.8	0.3	1.8	5.4	4.6	<0.5	290	8	Loam
2	15000106 BH 2 600 cm & 15000106 BH 3 500 cm	0.01	6.0	4.6	20.3	0.9	0.2	3.3	8.4	1.1	450	2(1)	Light clay

END OF TEST REPORT

3.4 Topography and Drainage

Total relief is about 20 m and much of the land slopes at about 10–15%. A ridge divides the site into lands that drain to Beach Road (about 28.5 ha) and lands that drain to Commonderry Swamp. Approximately 20 ha of Lot 4 is Commonderry Swamp which is a State Environmental Planning Policy No. 14 (SEPP14) Wetland. These lands would not be changed and so they are not discussed further in any detail.

Drainage on the southwest-facing slope (i.e. the land that slopes to Commonderry Swamp) is by sheet flow, there are no defined drainage depressions there. Drainage on most of the northeast-facing slope is via a broad depression which feeds two existing farm dams. A contour bank also feeds the northern-most of these dams. Although the depression is shown as a blue line on the Gerroa 1:25,000 topographic map, it does not have defined bed and banks (**Figure 9**) and so is not a watercourse that would require a controlled activity permit for any future works. On the map the depression is shown to discontinue north of Beach Road.

A third, small, dam is located in the far northwest. It is by-passed by the depression which drains just to its east and then enters the table drain of Beach Road. Finally, the depression drains under Beach Road through a culvert. A small portion of the northeast of the site also drains under Beach Road by another culvert; it then drains into a dam on a separate property.

3.5 Groundwater

A Groundwater Impact Assessment (GIA) was prepared by Larry Cook Consulting Pty Ltd for the subdivision (Rep. 17134-B dated 4 November 2017). The GIA identified three small springs (water features) on the Site. Two of these 'water features' had small head works historically constructed on them, namely a well in the north-eastern part of the Site and a simple PVC pipe in the northern central part.

An addendum to the GIA was published in August 2018 by Larry Cook Consulting in response to Council comments (**Appendix 1**). The locations of the springs and works are shown in Figure 1 of Appendix 1. The following is an extract from the addendum.

The site investigations and search of the WaterNSW computerised bore database did not reveal any registered bores within 100 m of the Site. However, any extraction works constructed on a spring such as the well in the north-western part of the Site and the crude PVC pipe driven into the spring in the central northern part are interpreted to be equivalent to a bore.

The shallow brick/concrete well is observed to be overgrown, largely invaded by vegetation (grass), in disrepair and non-operational. The PVC pipe work is a crude attempt to intercept the spring and is also in disrepair. The pipe appears to have been driven into the spring by hand. It is recommended that these unauthorised works be decommissioned. The shallow well on the north-western spring can be easily collapsed, filled in and the site levelled. The pipe work in the central northern spring is easy to decommission by simply extracting the pipe by hand and discarding.

The decommissioning of the unauthorised spring works effectively returns each of these sites to a spring (water feature). In this regard, the suggested minimum buffer setback distance between the springs and any Land Application Area (LAA) is a minimum 40 m, as required by Council.

In order to achieve an overall lower potential hazard the nominal buffer setback distance between a known spring on the Site and any proposed application of secondary treated wastewater (effluent management) should be no less than:

- *40 m sideslope*
- *40 m upslope*
- *50 m downslope*

The downslope setback is also nominated to prevent any spring flow migrating downslope and impacting the LAA.

The areas surrounding the known springs that are considered un-suitable for application of treated wastewater are annotated in Figure 2 of Appendix 1 and are reproduced in **Figure 9.**

Land Surface Changes

3.6 Subdivision Stage

3.6.1 Introduction

The site is divided into:

- Land that will remain as part of Coomonderry Swamp (and so is not discussed further)
- Lands that will drain to Commonderry Swamp; and
- Lands that drain to Beach Road.

The following land surface changes would occur when the subdivision works are done.

3.6.2 Lands that drain to Commonderry Swamp

- Approximately 18.1 ha of land that is currently agricultural lands would become large-lot residential land. That would represent an improvement in water quality, as runoff from large-lot residential land has a better quality than runoff from agricultural land (CMA, 2010 and SCA, 2012).
- Approximately 5.7 ha of land that is currently agricultural lands would become zoned E1 (National Parks and Nature Reserves) and is proposed for dedication as an extension to Coomonderry Swamp Nature Reserve. That would represent an improvement in water quality, as runoff from an environmental zone would have a better quality than runoff from agricultural land (CMA, 2010 and SCA, 2012).

3.6.3 Lands that drain to Beach Road

- Approximately 28.38 ha of land that is currently agricultural lands would become large-lot residential land and the road network; none of the road network would drain to Commonderry Swamp. Runoff from large-lot residential land has a better quality than runoff from agricultural land (CMA, 2010 and SCA, 2012) but runoff from any new road might offset that to some extent.
- The two existing dams on the north-facing slope would be removed.

3.7 Dwelling Stage (Future DAs)

As each lot is developed there would be an increase in impervious surfaces (roofs, paving and driveways). However as the lots are large, it would be permissible to manage stormwater overflow in bioretention basins (raingardens). The connectivity of these impervious surfaces to the stormwater drainage system would therefore be minimal. CMA

(2010) estimates the *effective* imperviousness[2] of these lots would only be about 5% although 20% was used as a conservative estimate for the MUSIC water quality model and also the post development catchment calculations for the site.

4 Hydrology

4.1 Rainfall Data

The Intensity Frequency Duration (IFD) rainfall data for the site is based on data presented in Australian Rainfall and Runoff (1987 and 2016 IFD data) and site specific calculations for Beach Road, Berry. All storm ensemble patterns were run for the various storm durations adopting the maximum flow of the median values at each storm duration. A copy of the IFD Chart and table for the site is attached in **Appendix 2**.

4.2 Catchment Areas

The post development catchment areas to the existing watercourse draining to Beach Road are shown in **Figure 6**. **Figure 7** shows the ultimate development catchments adopted for the MUSIC water quality model.

[2] i.e. the percentage of impervious surfaces that would be directly connected to the stormwater drainage system

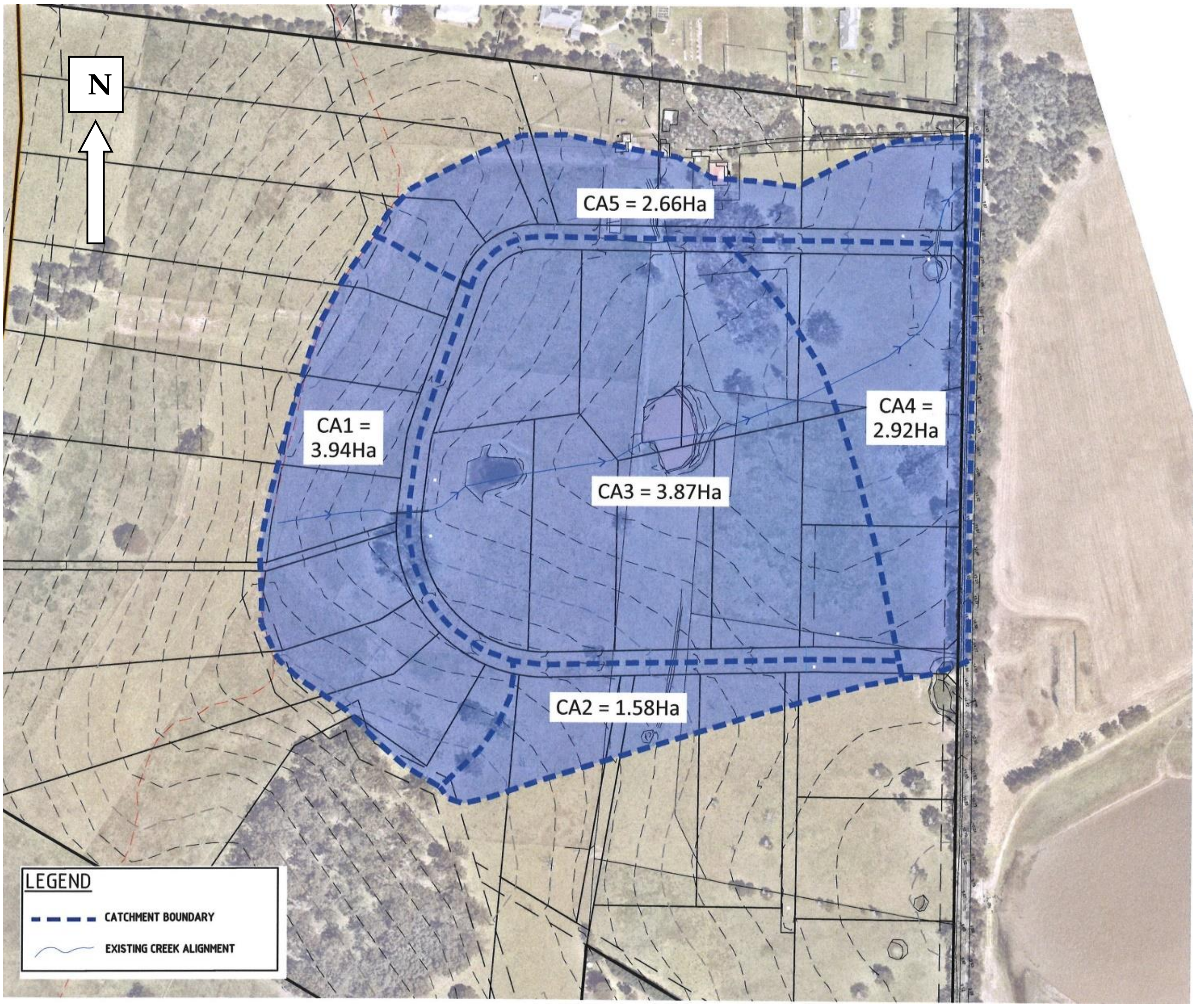

Figure 6 – Post Development Catchments Draining to the Existing Watercourse & Beach Road

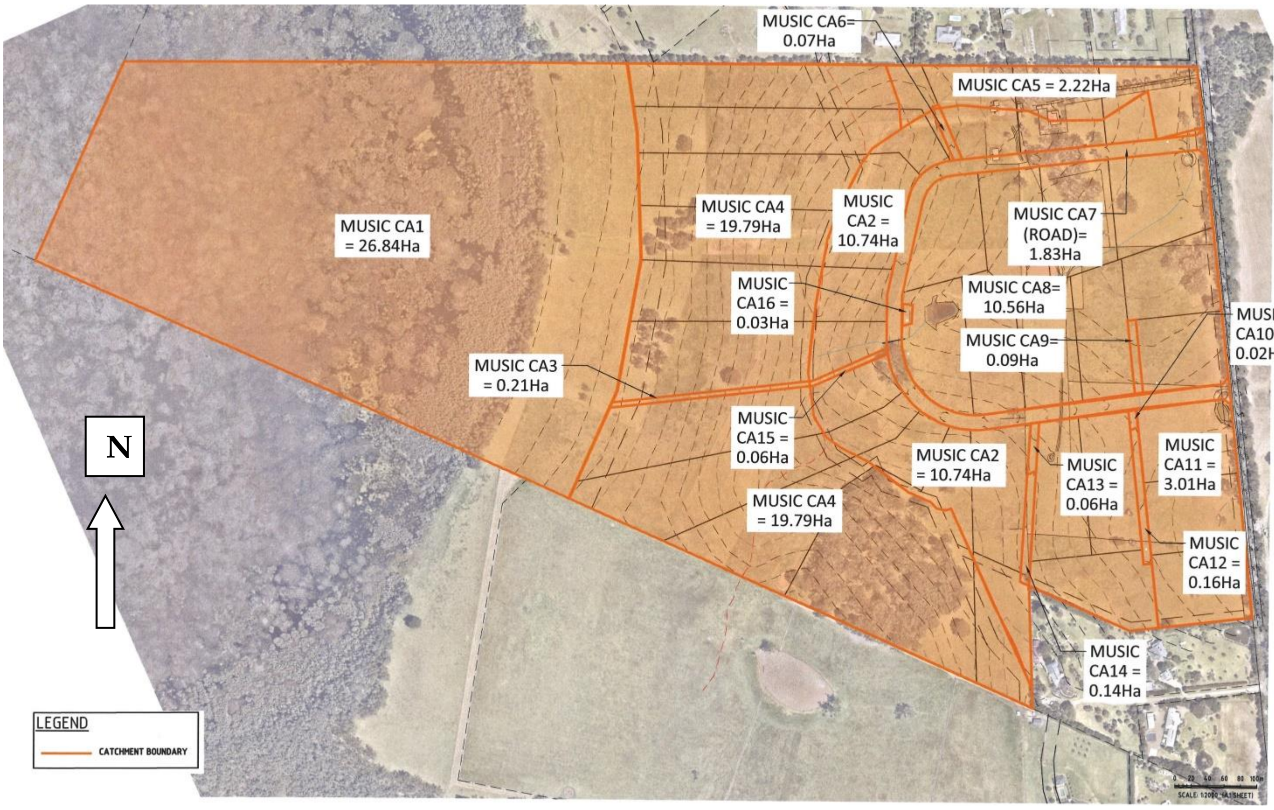

Figure 7 – Water Quality Modelling Catchment Plan

4.3 Site Catchment Peak Flow Calculations

4.3.1 Peak Flows to the Existing Watercourse

Flood flow calculations for the catchment are based on the Rational Method.

As used in the calculations, the formula of the Rational Method is:

$$Q_y = 0.278 \times C_Y \times I_{tc,y} \times A \qquad (1)$$

Where: Q_y = peak flow rate (*m³/s*) of average recurrence interval (ARI) of Y years

C_Y = runoff co-efficient, (dimensionless) for ARI of Y years

$I_{tc,y}$ = Average Rainfall Intensity, (*mm/hr*) for the design duration of t_c hours and ARI of Y years

A = area of catchment (*km²*)

Time of concentration (tc) was calculated using AR&R 1987 formula:

$tc = 0.76A^{0.38} \times A$ ....(2)

Runoff co-efficient (C_Y) was calculated using:

$C_Y = C_{10} x FF_Y$

Where: C_{10} = runoff co-efficient, (dimensionless) for ARI of 10 years

FF_y = flood frequency factor for Y years, Table 5.1 of ARR 1987 Volume 2.

The design calculation summary sheets are shown in **Tables 2 and 3** of this report.

4.4 Rainfall Data

The Intensity Frequency Duration (IFD) rainfall data for the site has been based on data presented in Australian Rainfall and Runoff (AR&R) and site specific calculations for Berry. IFD's from both AR&R 1987 and AR&R 2016 were compared and the AR&R 1987 charts showed higher rainfall intensities than the 2016 storms. The AR&R 1987 IFD's were therefore adopted for the peak flow calculations. A copy of the AR&R 1987 and AR&R 2016 charts and tables for the site are attached at **Appendix 2**.

4.5 Peak Flow Results

A summary of the 20% AEP (5 year ARI) and the 1% AEP (100 year ARI) flood flow calculations for all sub-catchments are shown in **Tables 2 and 3** for both the 1987 AR&R and the AR&R 2016:

Table 2 - Peak Flow Summary (Based on 1987 AR&R IFD Data)

Flow Calculations

Peak flow is given by the Rational Formula: $Q_y = 0.00278 \times C_{10} \times F_Y \times I_{y,\,tc} \times A$

where:
- Q_y is peak flow rate (m^3/sec) of average recurrence interval (ARI) of "Y" years
- C_{10} is the runoff coefficient (dimensionless) for ARI of 10 years.
- F_y is a frequency factor for "Y" years.
- A is the catchment area in hectares (ha)
- $I_{y,\,tc}$ is the average rainfall intensity (mm/hr) for an ARI of "Y" years and a design duration of "tc" (minutes or hours)

Time of concentration (t_c) = $0.76 \times (A/100)^{0.38}$ hrs

Note: For urban catchments the time of concentration should be determined by more precise calculations or reduced by a factor of 50 per cent. Place an x in the appropriate row below to automatically halve the time of concentration for that sub-catchment.

Structure Details

Name	CA1a	CA1b	CA2	CA3a	CA3b	CA4	CA5		Notes
Catchment Area (ha)	0.575	3.365	1.58	0.873	2.997	2.92	2.66		
Place an x here to halve tc									Place an x if disturbed catchment
Time of concentration (tc)	6	13	9	8	12	12	11		minutes

Rainfall Intensities

1-year, tc	104	75	90	95	77	77	80		Enter the relevant rainfall intensities (in mm/hr) for each of the nominated rainfall events. The time of concentration (tc) determines the duration of the event to be used
2-year, tc	134	95	112	120	100	100	105		
5-year, tc	170	122	145	160	130	130	140		
10-year, tc	191	140	165	175	147	147	150		
20-year, tc	219	168	190	200	170	170	180		
50-year, tc	255	183	220	235	200	200	210		
100-year, tc	282	220	240	260	230	230	235		
C10 runoff coefficient	1	1	1	1	1	1	1		Use AR&R or Table F3, pg F-6

Frequency Factors

FF, 1-year	0.62	0.62	0.62	0.62	0.62	0.62	0.62		Can use 0.8 for a construction site
FF, 2-year	0.78	0.78	0.78	0.78	0.78	0.78	0.78		Can use 0.85 for a construction site
FF, 5-year	0.9	0.9	0.9	0.9	0.9	0.9	0.9		Can use 0.95 for a construction site
FF, 10-year	1	1	1	1	1	1	1		Generally always 1
FF, 20-year	1.1	1.1	1.1	1.1	1.1	1.1	1.1		Can use 1.05 for a construction site
FF, 50-year	1.15	1.15	1.15	1.15	1.15	1.15	1.15		Can use 1.15 for a construction site
FF, 100-year	1.22	1.22	1.22	1.22	1.22	1.22	1.22		Can use 1.2 for a construction site

Flow Calculations

									Notes
1-year, tc (m^3/s)	0.103	0.435	0.245	0.143	0.398	0.388	0.367		
2-year, tc (m^3/s)	0.167	0.693	0.384	0.227	0.65	0.633	0.606		
5-year, tc (m^3/s)	0.245	1.027	0.573	0.349	0.975	0.95	0.932		
10-year, tc (m^3/s)	0.305	1.31	0.725	0.425	1.225	1.193	1.109		
20-year, tc (m^3/s)	0.385	1.729	0.918	0.534	1.558	1.518	1.464		
50-year, tc (m^3/s)	0.469	1.969	1.111	0.656	1.916	1.867	1.786		
100-year, tc (m^3/s)	0.55	2.511	1.286	0.77	2.338	2.278	2.12		

NB for flow calculations on sediment basin spillways, see Worksheet 3 (if required).

Table 3 - Peak Flow Summary (Based on 2016 AR&R IFD Data)

Flow Calculations

Peak flow is given by the Rational Formula: $Qy = 0.00278 \times C_{10} \times F_Y \times I_{y,tc} \times A$

where: Q_y is peak flow rate (m^3/sec) of average recurrence interval (ARI) of "Y" years
C_{10} is the runoff coefficient (dimensionless) for ARI of 10 years.
F_y is a frequency factor for "Y" years.
A is the catchment area in hectares (ha)
$I_{y,tc}$ is the average rainfall intensity (mm/hr) for an ARI of "Y" years and a design duration of "tc" (minutes or hours)

Time of concentration (t_c) = $0.76 \times (A/100)^{0.38}$ hrs

Note: For urban catchments the time of concentration should be determined by more precise calculations or reduced by a factor of 50 per cent. Place an x in the appropriate row below to automatically halve the time of concentration for that sub-catchment.

Structure Details

Name	CA1a	CA1b	CA2	CA3a	CA3b	CA4	CA5		Notes
Catchment Area (ha)	0.575	3.365	1.58	0.873	2.997	2.92	2.66		
Place an x here to halve tc									Place an x if disturbed catchment
Time of concentration (tc)	6	13	9	8	12	12	11		minutes

Rainfall Intensities

	CA1a	CA1b	CA2	CA3a	CA3b	CA4	CA5		Notes
1-year, tc	85.2	61.3	72.7	76.3	63.7	63.7	66.4		Enter the relevant rainfall intensities (in mm/hr) for each of the nominated rainfall events. The time of concentration (tc) determines the duration of the event to be used
2-year, tc	97.4	70.3	83.8	87.5	73.1	73.1	76.1		
5-year, tc	139	101	119	125	105	105	109		
10-year, tc	169	123	145	153	128	128	133		
20-year, tc	201	147	173	182	152	152	159		
50-year, tc	247	180	213	223	187	187	195		
100-year, tc	285	208	245	257	216	216	225		

C10 runoff coefficient	1	1	1	1	1	1	1		Use AR&R or Table F3, pg F-6

Frequency Factors

	CA1a	CA1b	CA2	CA3a	CA3b	CA4	CA5		Notes
FF, 1-year	0.62	0.62	0.62	0.62	0.62	0.62	0.62		Can use 0.8 for a construction site
FF, 2-year	0.78	0.78	0.78	0.78	0.78	0.78	0.78		Can use 0.85 for a construction site
FF, 5-year	0.9	0.9	0.9	0.9	0.9	0.9	0.9		Can use 0.95 for a construction site
FF, 10-year	1	1	1	1	1	1	1		Generally always 1
FF, 20-year	1.1	1.1	1.1	1.1	1.1	1.1	1.1		Can use 1.05 for a construction site
FF, 50-year	1.15	1.15	1.15	1.15	1.15	1.15	1.15		Can use 1.15 for a construction site
FF, 100-year	1.22	1.22	1.22	1.22	1.22	1.22	1.22		Can use 1.2 for a construction site

Flow Calculations

	CA1a	CA1b	CA2	CA3a	CA3b	CA4	CA5		Notes
1-year, tc (m^3/s)	0.084	0.356	0.198	0.115	0.329	0.321	0.304		
2-year, tc (m^3/s)	0.121	0.513	0.287	0.166	0.475	0.463	0.439		
5-year, tc (m^3/s)	0.2	0.85	0.47	0.273	0.787	0.767	0.725		
10-year, tc (m^3/s)	0.27	1.151	0.637	0.371	1.066	1.039	0.984		
20-year, tc (m^3/s)	0.353	1.513	0.836	0.486	1.393	1.357	1.293		
50-year, tc (m^3/s)	0.454	1.936	1.076	0.622	1.792	1.746	1.658		
100-year, tc (m^3/s)	0.556	2.374	1.313	0.761	2.196	2.139	2.03		

NB for flow calculations on sediment basin spillways, see Worksheet 3 (if required).

4.5.1 Total Site Post Development Drainage Model

A DRAINS model (**Figure 8**) was set-up using the ILSAX hydrological model to determine the total pre-development and post development flows for the proposed development including on-site detention design. Four storm events were modelled for the total pre and post development site scenario ranging from the 50% Annual Exceedance Probability (AEP) Storm to the 1% (AEP) using the following parameters:

- Paved (impervious) area depression storage (mm) = 1
- Supplementary area depression storage (mm) = 1
- Grassed (pervious) area depression storage (mm) = 5
- Soil Type = 3
- AMC (Antecedent Moisture Condition) = 3
- Development Percentage Impervious Area = 20%

Pre and post development flows including on-site detention are discussed further in **Section7.2.5**.

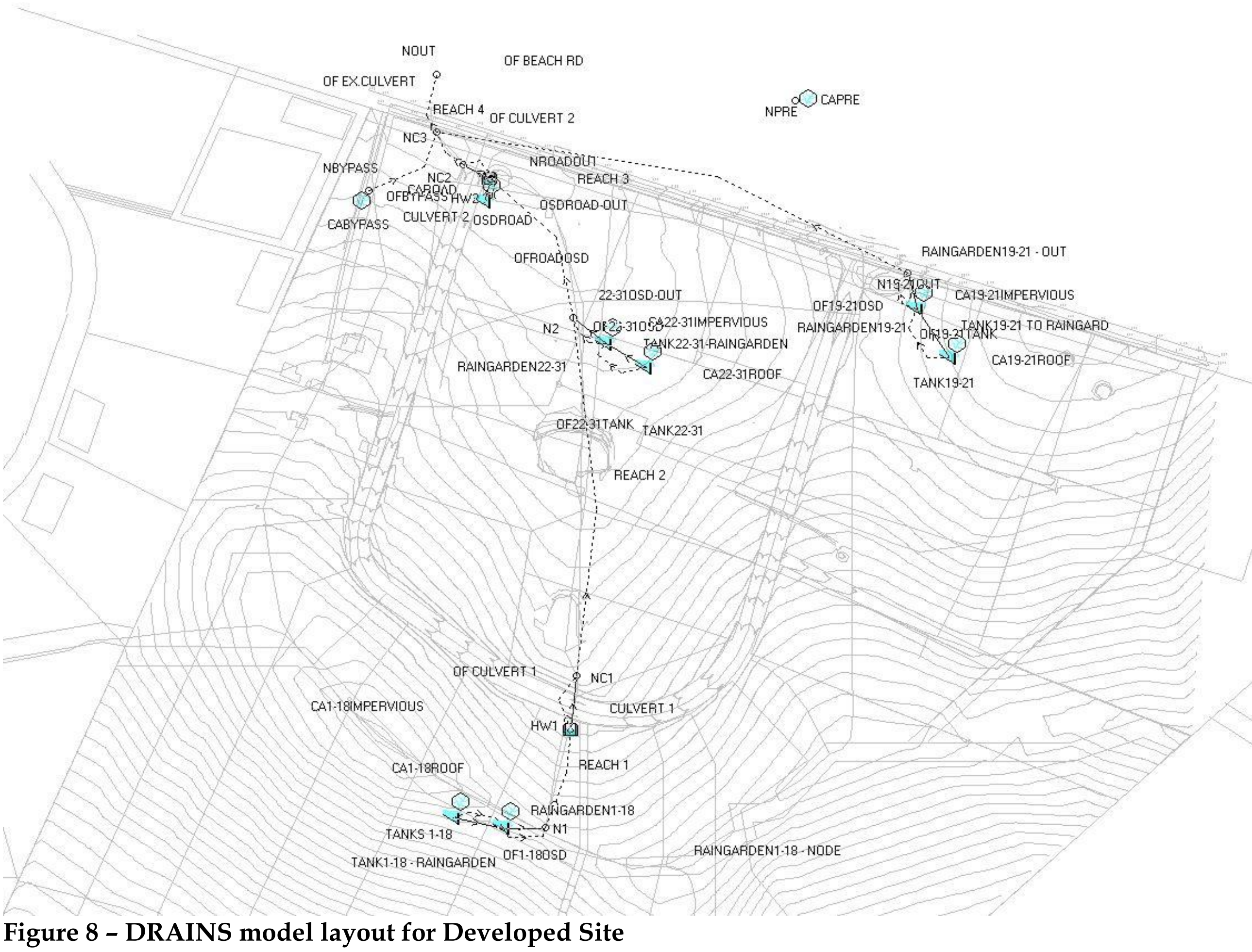

Figure 8 – DRAINS model layout for Developed Site

5 Flooding

The site contains an intermittent water course flowing from south to north through the site towards Beach road. A flood model was developed to determine the estimate flood extents for both the 5 year ARI and 100 year ARI storm events.

5.1 Flood Modelling

5.1.1 HEC-RAS

The computer modelling program HEC-RAS (Hydrologic Engineering Centers River Analysis System) has been used to analyse the flood level, velocity and extents for the post development scenario.

From the survey data a 3D digital terrain model was produced using *Autodesk Civil 3D*. This model included a design reach along the watercourse so that a series of cross sections could be extracted and exported into HEC-RAS. Refer to drawing 19000408_P01_FS100 (**Appendix 3**) for the location of the reach alignment and cross sections.

5.1.2 Model parameters

A post-development model was prepared to analyse the existing watercourse conditions. The provided site survey was used to produce the geometry of the watercourse in HEC-RAS with the concept design for the proposed road incorporated to provide a combined surface. It is not proposed to change the topography of the existing watercourse for the majority of its length except for where the proposed new road will cross it at the two locations shown of the concept stormwater plans at flood chainages 60 and 520.

Two 1.8m wide x 0.6 high culverts were added into the model at flood chainage 60 and three x 600mm Diameter RC pipes were included at the road crossing at flood chainage 520. Both were modelled with a 30% blockage allowance and have been sized to conveyor the 20% AEP (5 year ARI) flood event.

A Manning Roughness (n) of 0.04 was adopted for the watercourse and 0.04 was adopted for the over bank areas. The site survey and aerial photography shows that the existing watercourse is wide and shallow with minimum vegetation other than pastural grass in open paddocks. The watercourse varies in grade with an approximate base grade ranging from 8% at the start and flattening out to grades of 3% to 1.5% towards the end of the site towards Beach Road.

The calculated peak flows were entered into the *HEC-RAS* Model and a Mixed Flow Regime was adopted to determine the flow extents within the watercourse for the 5 year and 100 year ARI storm event. This was then exported back into *Civil 3d* to produce a flood plan, refer to drawing **19000408-FS100 (Appendix 3)**.

5.1.3 Flood Modelling Results

The flood extents across the site for the 100 year ARI storm event have been determined and are displayed on Drawing 19000408-FS200 (**Appendix 3**). It can be seen that this major storm event is contained wholly within the watercourse creek alignment except at the Beach Road end were flood events greater than the 5 year ARI would likely inundate and cross Beach Road. Beach Road would also most likely be affected by any ponding of significant storms within the adjoining catchment on the northern side of Beach Road in the area of Foys Swamp.

A summary of the modelling results including water levels are shown in **Tables 4 and 5** below for the 1% AEP (100 Year ARI) and the 20% AEP (5 Year ARI).

Table 4 - 1% AEP (100 Year ARI)

River Sta	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
619.375	0.55	27.04	27.15	27.15	27.17	0.041181	0.72	0.76	14.22	0.99
600	0.55	25.51	25.59	25.62	25.67	0.19316	1.27	0.43	10.92	2.05
550	0.55	21.19	21.36	21.32	21.37	0.010343	0.49	1.11	12.93	0.54
538.282	3.06	20.3	21.37	20.53	21.37	0.000013	0.09	40.56	59.05	0.03
524	Culvert									
510.515	3.06	18.27	18.44	18.44	18.5	0.03182	1.11	2.75	22.1	1.01
500	3.83	17.54	17.73	17.79	17.91	0.098071	1.91	2.01	16.68	1.76
450	3.83	15.57	15.89	15.89	16	0.02644	1.49	2.58	11.63	1.01
400	3.83	13.52	13.63	13.67	13.76	0.091957	1.59	2.41	25.46	1.64
359.742	3.83	12.03	12.34	12.29	12.38	0.012151	0.87	4.4	24.71	0.66
350	3.83	11.66	12.15	12.15	12.21	0.026575	1.12	3.55	29.99	0.94
300	3.83	10.48	10.97	10.95	11.04	0.019052	1.2	3.49	20.17	0.84
250	3.83	9.79	9.92	9.91	9.96	0.024385	0.47	4.56	52.25	0.73
208.97	7.45	9.07	9.27		9.31	0.01148	0.57	9.29	61.29	0.58
200	7.45	8.97	9.13	9.11	9.18	0.018958	0.72	7.93	58.64	0.74
150	7.45	8.18	8.47		8.5	0.010058	0.82	9.54	58.56	0.6
123.726	7.45	7.85	8.15		8.18	0.010841	0.86	9.33	56.24	0.63
100	7.45	7.57	7.96		7.99	0.006399	0.75	10.28	47.15	0.5
72.806	9.73	7.2	7.7	7.69	7.73	0.013694	1.08	15.56	198.65	0.73
62.5	Culvert									
50	9.73	7.1	7.71		7.71	0.000256	0.22	56.7	206.22	0.11
17.341	11.85	7.28	7.61	7.58	7.68	0.014008	1.19	11	61.7	0.75

Table 5 – 20% AEP (5 Year ARI)

River Sta	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
619.375	0.25	27.04	27.12	27.12	27.14	0.035079	0.55	0.44	10.82	0.88
600	0.25	25.51	25.57	25.59	25.64	0.285709	1.2	0.2	7.56	2.34
550	0.25	21.19	21.28	21.29	21.31	0.041049	0.68	0.36	7.38	0.98
538.282	1.27	20.3	21.24	20.46	21.24	0.000004	0.04	33.19	55.63	0.02
524	Culvert									
510.515	1.27	18.27	18.38	18.38	18.41	0.036751	0.86	1.47	19.29	1
500	1.62	17.54	17.66	17.7	17.78	0.096785	1.54	1.05	12	1.66
450	1.62	15.57	15.77	15.77	15.84	0.029861	1.21	1.33	8.93	1
400	1.62	13.52	13.59	13.61	13.65	0.070119	1.08	1.5	22.75	1.34
359.742	1.62	12.03	12.26	12.21	12.28	0.010378	0.64	2.55	20.45	0.57
350	1.62	11.66	12.05	12.05	12.11	0.031747	1.06	1.53	13.09	0.99
300	1.62	10.48	10.9	10.87	10.93	0.012884	0.82	2.15	16.57	0.66
250	1.62	9.79	9.86	9.86	9.89	0.038851	0.47	2.09	34.74	0.87
208.97	3.17	9.07	9.2	9.15	9.22	0.009208	0.34	5.38	50.13	0.47
200	3.17	8.97	9.05	9.05	9.09	0.025793	0.47	3.81	46.44	0.75
150	3.17	8.18	8.4	8.35	8.42	0.008047	0.54	5.83	48.81	0.5
123.726	3.17	7.85	8.05		8.07	0.01646	0.76	4.33	40.15	0.72
100	3.17	7.57	7.87		7.89	0.004485	0.54	6.34	39.93	0.4
72.806	4.12	7.2	7.65	7.65	7.68	0.012267	0.89	7.55	147.38	0.66
62.5	Culvert									
50	4.12	7.1	7.58		7.59	0.000276	0.2	31.78	198.67	0.11
17.341	5.05	7.28	7.51	7.48	7.55	0.014007	0.88	5.99	44.57	0.7

The results shown for the 1% (AEP) 100 year should be used as a preliminary assessment for setting flood planning levels for any proposed dwellings within the vicinity of the watercourse, that being the 100 year flood level plus 0.5m freeboard. Future dwellings should be set above the flood planning at a minimum.

6 Onsite Wastewater Management

6.1 Introduction

The site would not be connected to sewer and so wastewater generated in each new home would be managed on each lot. Much of the site is relatively unconstrained for wastewater management and most of the soils across the site are reasonably well suited to disposal of secondary treated effluent by either irrigation or absorption. However, there are some existing constraints and there would be others post development:

(i) Existing Constraints (Refer to **Figure 9**).

- There are areas of low-lying land along the frontage with Beach Road that are subject to poor drainage and high run-on; they represent the Shoalhaven Soil Landscape and are to be avoided for the purposes of wastewater management.
- There is a drainage depression on the north-facing slope that drains to the far northwest corner of the site and thence under Beach Road; it requires a 40 m overland flow buffer from any effluent management area.
- There are three springs located on proposed Lots 3, 9 and 16; these require buffers in accordance with the Groundwater Impact Assessment Report No. 17134-B (Appendix 11.1).
- The table drain(s) of Beach Road requires a 40 m buffer.

(ii) Post-development Constraints

- The new road network would, most likely, be drained using grass-lined drainage swales, similar to neighboring developments. Depending on their location and orientation, they might require a 40 m overland buffer from any future effluent management area.
- Buffers would also be required to lot boundaries and to the built environment on each lot; these would vary depending on each separate development application.

6.2 Conceptual Wastewater Design

6.2.1 Introduction

For the purpose of this early assessment, it is assumed irrigation would be adopted for wastewater disposal. Moderate slope gradients and the reasonably wet climate both dictate that subsurface irrigation be adopted on all lots. In addition, it is assumed wet weather storage is not desirable, as it is difficult for home-owners to manage. Assuming a five-bedroom home on tank water supply the design load is 900 L/day.

6.2.2 Nutrient Balances

Assuming the Effluent Management Areas consist of poorly managed lawns[3], to balance the input and uptake of nutrients, an area of 830 m^2 would be required[4] for a design life of 50 years (**Table 6**).

6.2.3 Hydraulic Balances

Chapter G8 in Council's DCP requires a monthly water balance. The hydraulic *inputs* are retained median rainfall (i.e. median rainfall less an allowance for run-off) and applied effluent. The *outputs* are evapotranspiration (taken as pan evaporation reduced by a crop factor which varies through the year) and percolation (the ability of the soil to absorb water, 10L/m^2/day for a light clay soil (AS/NZS 1547:2012)). Median rainfall and pan evaporation values have been taken from Chapter G8.

The water balance is shown in **Table 7**; a minimum Effluent Application Area (EAA) of 100 m^2 is required. Council require an area equivalent to this EAA to be set aside as a reserve in case it is ever required.

AS/NZS 1547:2012 gives an alternative, more conservative, method of calculating the required hydraulic area. The required area hydraulically is given as the daily effluent load (900L/day) divided by the Design Irrigation Rate (DIR) which, for light clay on a 10-20% slope, is 2.4L/m^2/day. By this method, the required EAA is 900/2.4 = 375 m^2. Again, Council require an area equivalent to this EAA to be set aside as a reserve, in case it is ever required.

6.2.4 Summary

Although, hydraulically, the EAA can (conservatively) be 375 m^2, the nutrient balance requires a total Effluent Management Area (EMA) of 830 m^2 to be set aside for effluent application and nutrient uptake.

Therefore, a total typical EMA for a five-bedroom home would be 830 m^2, consisting of:

- 375m^2 of subsurface irrigation (the EAA); and
- 455 m^2 of Nutrient Uptake Area (NUA) located immediately downslope of the EAA. Note: the NUA would include the reserve area.

Given the minimum lot size would be 10,000 m^2, incorporating an EMA of 830 m^2 would not be problematic (subject to a detailed assessment of existing and future constraints).

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[3] i.e. lawns that are slashed with clippings not removed. The estimated plant nutrient uptakes would be 32.5 mg/m^2 and 3.25 mg/m^2 for nitrogen and phosphorous respectively (SCA, 2012). The insitu soil phosphorous sorption is estimated from site-specific soil testing to be 1861 kg/ha.
[4] Based on the nitrogen balance which is limiting.

Table 6- Nutrient Balances

Wastewater Volume 900 (L/day)

NOTE: The area required to uptake nutrients varies on what vegetation is adopted in the EMA.

Vegetation in EMA **Lawn - Unmanaged**

Nitrogen Balances
A = (C x Q) / Lx
Where:
A = Land Area (m2)

C = Concentration of Nutrient =	30	mg/L
Q = Wastewater Flow =	900	L/day
Lx = Critical Loading Rate =	32.5	(mg/m^2/day)

A = 831 m^2 of subsurface irrigation under Lawn - Unmanaged

Phosphorus Balances
Step 1: P Sorption Calculation

Psorb (topsoil)	loam	290	mg/kg	
Psorb (subsoil)	clay	450	mg/kg	
Bulk Density (topsoil)	loam	1500	kg/m3	
Thickness (topsoil)		200	mm	
Coarse Frags (topsoil)		0	%	
Bulk Density (subsoil)	clay	1300	kg/m3	
Thickness (subsoil)		800	mm	
Coarse Frags (subsoil)		5	%	
Calculated Psorb (topsoil)		870	kg/ha	
Calculated Psorb (subsoil)		4446	kg/ha	
Assumed P-sorb		1861	kg/ha	(insitu P-sorb is 35% calculated P-sorb)

Step 2: Determine the required area to sorb phorphorus (50 year design life) :

P absorbed	=	5316	x 0.35
	=	1861	kg/ha
	=	0.19	kg/m2
P uptake	=	3.25	mg/m2/day

Determine the amount of phosphorus generated over that time:

Concentration of phosphorus =		12	mg/L
Phosphorus generated =	Concentration x volume of wastewater =	197.1	kg

Area Required:
P generated / (P sorbed + P uptake) = 803 m^2 of Lawn - Unmanaged

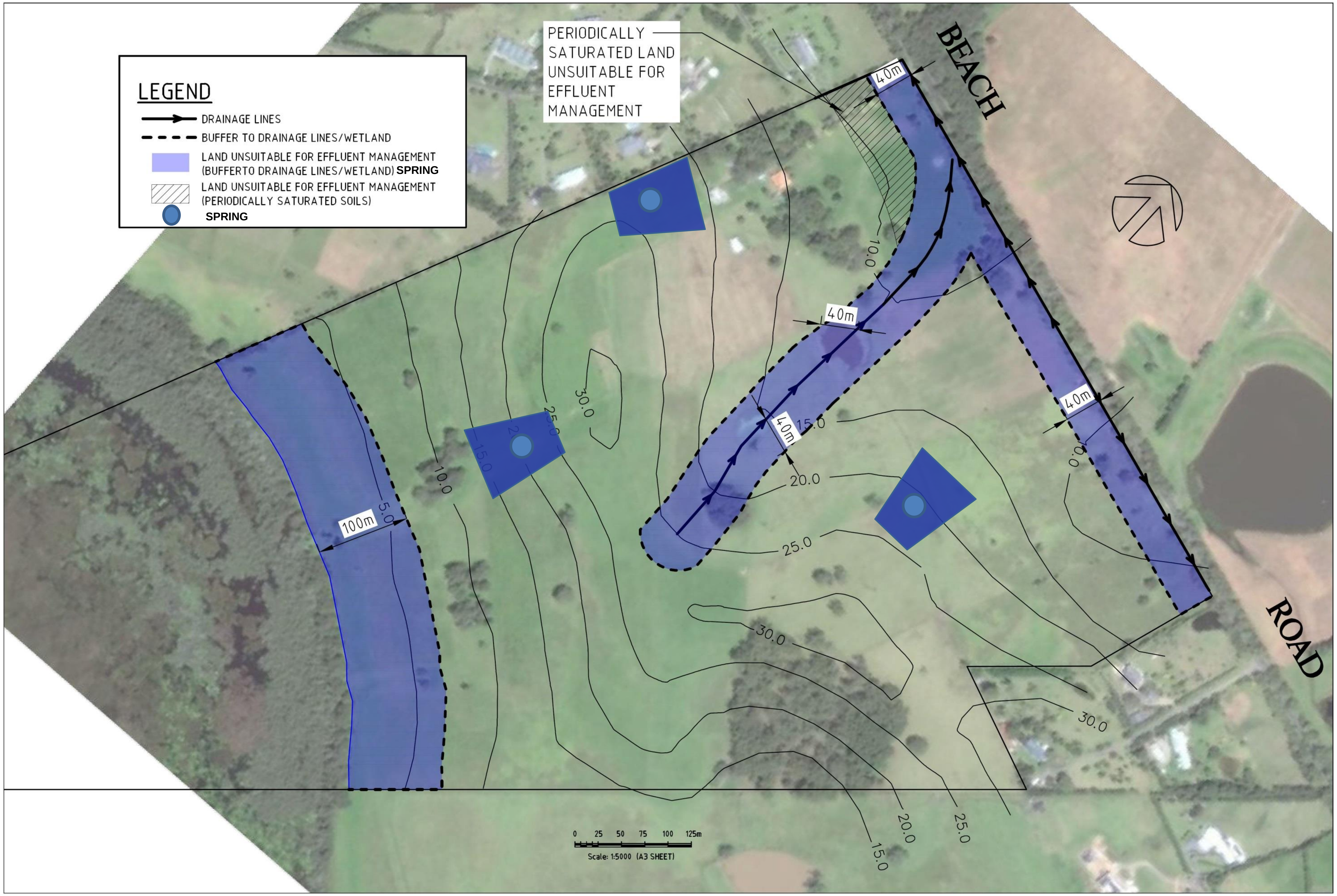

Figure 9 – Constraints to Effluent Management

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Table 7- Monthly Water Balance

Hydraulic Balance

	Rainfall Station	Berry		
	Evaporation Zone	Nowra		
	Wastewater Load	900	**L/day**	
	Percolation Rate	10	**mm/day**	
	Land Area	100	**m^2**	
	Storage Required	0	**m^3**	
Month	**Days in month**	**Median Precipitation (mm**	**Evaporation (mm)**	**Crop Factor**
Jan	31	99.8	194.4	0.8
Feb	28	108.7	161.1	0.8
Mar	31	116.2	144.6	0.8
Apr	30	98.1	118.7	0.7
May	31	79.8	95	0.7
Jun	30	90.2	85.8	0.6
Jul	31	60	94.7	0.6
Aug	31	46.2	127.6	0.7
Sep	30	59.1	148.1	0.7
Oct	31	70.3	194.4	0.8
Nov	30	77.6	161.1	0.8
Dec	31	87.4	144.6	0.8
INPUTS				
	Retained Rainfall (mm)	**Effluent Irrigation (mm)**	**Total Inputs (mm)**	
Jan	79.84	279.00	358.84	
Feb	86.96	252.00	338.96	
Mar	92.96	279.00	371.96	
Apr	68.67	270.00	338.67	
May	55.86	279.00	334.86	
Jun	54.12	270.00	324.12	
Jul	36	279.00	315.00	
Aug	32.34	279.00	311.34	
Sep	41.37	270.00	311.37	
Oct	56.24	279.00	335.24	
Nov	62.08	270.00	332.08	
Dec	69.92	279.00	348.92	
OUTPUTS				
	Evapotranspiration (mm)	**Percolation (mm)**	**Outputs (mm)**	**Storage (mm)**
Jan	155.52	310.00	465.52	-106.68
Feb	128.88	280.00	408.88	-69.92
Mar	115.68	310.00	425.68	-53.72
Apr	83.09	300.00	383.09	-44.42
May	66.5	310.00	376.50	-41.64
Jun	51.48	300.00	351.48	-27.36
Jul	56.82	310.00	366.82	-51.82
Aug	89.32	310.00	399.32	-87.98
Sep	103.67	300.00	403.67	-92.30
Oct	155.52	310.00	465.52	-130.28
Nov	128.88	300.00	428.88	-96.80
Dec	115.68	310.00	425.68	-76.76

7 Concept Surface Water Management Plan

The following stormwater management principles are to be adopted as part of the development. The concept stormwater management plans are attached **Appendix 3.**

7.1 Construction Phase Erosion and Sediment Control

7.1.1 During Subdivision

A construction-phase Soil and Water Management Plan (SWMP) will be required before commencing work on this development. This Plan will be prepared in accordance with the guidelines and recommendations in Landcom (2004). The SWMP is to be developed to incorporate the following generic principles:

(i) Sediment fencing is to be used downslope of any construction area until works are complete (Standard Drawing SD 6-8, Landcom, 2004).

(ii) Topsoil will be stripped off any construction areas and stockpiled following Standard Drawing SD 4-1 (Landcom, 2004) for later re-use.

(iii) The upslope catchment length of exposed soil areas will be kept below 80 m. Any slope length exceeding 80 m will have a berm installed to direct overland flows onto well-protected, vegetated lands.

(iv) Show the location and sizing of sediments basins, if required, depending on the size of the area of exposed soils and any stage construction of the development.

(v) Construction traffic access is to be limited to the minimum required for efficient construction. Areas not essential for construction purposes are to be protected from traffic entry through the use of barrier and/or sediment fencing. **Table 8** contains details of access limitations during construction in accordance with Landcom (2004).

(vi) Any required pipe/culvert outlets shall be stabilised with riprap and geo-textile underlay (Standard Drawing SD 5-8, Landcom, 2004). Downslope of the riprap a good cover of vegetation is necessary.

(vii) While C-factors are likely to rise to 1.0 during the work's program, they will not exceed those given in **Table 9**.

(viii) Diversion berms will be used to divert "clean" runoff from upslope of any construction areas away. Discharges are to be onto a stabilised, well-vegetated area, preferably using a level spreader or sill.

(ix) Rapidly rehabilitate disturbed lands to bring C-factors down to acceptable levels (see **Table 9**) and minimise the risk of erosion.

(x) Areas of concentrated flow (e.g. drainage pathways, swales etc.) are to be protected using appropriate erosion control measures. We suggest an appropriate biodegradable Rolled Erosion Control Product (RECP) such as coconut fibre matting or jute matting to provide stable ground cover until vegetation regenerates for low velocity concentrated flows.

(xi) Dust Control Measures during earthworks. This would include re-using water from sediment basins and spaying exposed areas via water cart.

The requirements of the SWMP would be implemented until at least 90 percent of the site was stabilised with hard surfaces or satisfactory vegetation.

Table 8 - Limitations to access during construction works

Land use	Limitation	Comments
Construction areas	Limited to 5 (preferably 2) metres from the edge of any essential construction activity as shown on the engineering plans	All site workers should clearly recognise these areas that, where appropriate, are identified with barrier fencing (upslope) and sediment fencing (downslope) or similar materials.
Access areas	Limited to a maximum width of 5 metres	The site manager will determine and mark the location of these zones on site. They can vary in position so as to best conserve existing vegetation and protect downstream areas while being considerate of the needs of efficient works activities. All site workers will clearly recognise these boundaries
Remaining lands, including re-veg areas	Entry prohibited except for essential management works	Thinning of growth might be necessary, for example, for fire reduction or weed removal

Table 9 - Maximum Acceptable C-Factors at Nominated Times During Works

Lands	Maximum *C*-factor	*Remarks*
Waterways and other areas subjected to concentrated flows (e.g. table drains), post construction	0.05	Applies after ten working days from completion of formation and before they are allowed to carry any concentrated flows. Flows will be limited to those shown in Table 5.2 of Landcom (2004). Foot and vehicular traffic will be prohibited in these areas
Stockpiles, post construction	0.1	Applies after ten working days from completion of formation. Maximum C-factor of 0.10 equals 60% ground cover
All lands, including waterways and stockpiles during construction	0.15	Applies after 20 working days of inactivity, even though works might continue later. Maximum *C*-factor of 0.15 equals 50% ground cover

7.1.2 Individual Houses

Each new home construction would implement a lot-scale Erosion and Sediment Control Plan (ESCP) to the requirements of Landcom (2004). It would be important this is done to protect any stormwater quality devices installed as part of the development.

7.2 Stormwater Drainage System

7.2.1 Earthworks

It is envisaged that some minor site earthworks and re-grading would be required for the installation of the new road, driveways, stormwater drainage, water quality infrastructure and backfilling of the existing farm dams on proposed lots 24, 26, 28 and 29. All lots would drain to the road, interalottment drainage system or existing watercourse located within dedicated drainage easements.

7.2.2 Road Drainage

(i) Beach Road

Beach road will require some minor upgrade works around the location of the two new intersections with the new road to the subdivision. This may also include some shoulder widening works and establishment of a table drain along the frontage of the site to convey storm flows from the 20% AEP (5 year ARI) design event.

Storm flows greater than 20% AEP and up to the 1% AEP (100 year ARI) would be conveyed overland within the road system.

(ii) Proposed New Road

The proposed new road is to be a full-road 8m wide pavement construction with flush kerb along the edges and grassed swale drains on each side as part of the water sensitive urban design (WSUD) for the development. It is also proposed to construct a new piped drainage system under the grassed swales with intermittently-spaced, grated inlet pits. The new trunk drainage system would be designed to convey all storms up to and including the 20% AEP (5 year ARI) event in accordance with Section D5 of Shoalhaven City Council's *'Engineering Design Specification'*.

Storm flows greater than 20% AEP and up to the 1% AEP would be conveyed overland within the grassed swales within the road network.

(iii) Proposed Driveways to Battle-axe Lots

It is proposed to construct 5m wide all-weather driveways for access to the battle-axe lots 4, 15, 18 and 32. The driveways are to be constructed as close to the natural ground levels as possible and drain to a 2.0m wide grassed buffer strips along the edges that would form part of the overall WSUD for the development.

7.2.3 Site and Inter-allotment Drainage & Flood Management

Surface water flows from individual lots would be conveyed to the proposed road and inter-allotment drainage lines located in dedicated drainage easements. These will also form dedicated overland flow paths for storms greater than the minor storm piped flow.

7.2.4 Rainwater Tanks

The development is not served by town water supply therefore each new dwelling would most likely install large rainwater tanks to cover their water usage requirements. Each new dwelling's rainwater tank would need to include the following at a minimum:

(i) A capacity of 40,000 L (minimum).

(ii) The top 20,000 L of the tank(s) dedicated to on-site detention.

(iii) A first-flush device.

(iv) A screen to prevent the entry of leaves, twigs and mosquitos.

(v) Be plumbed to toilet and laundry, and at least one outdoor tap.

(vi) Overflow to a 15m^2 bioretention basin (raingarden).

(vii) Be topped up from mains supply and that would require a back-flow prevention valve.

In addition to the above, each house is to employ water saving fittings with three-star rating or better. The tanks would be used to contribute to the total on-site detention requirements.

7.2.5 On-Site Detention

As discussed in Section 4.3, post development flows would increase due to the increased impervious area if on-site detention is not included. Flow from the site will be reduced by the provision of rainwater tanks for the capture and use of rainwater within each dwelling (Section 7.2.4) and an above-ground on-site detention (OSD) within the proposed bioretention basins within the proposed lots and also the four bioretention basins required to treat the new road stormwater run-off. Together, these measures are designed to limit post-development flows to no more than pre-development flows.

The indicative locations of the proposed bioretention basins with OSD are shown on the Concept Surface Water Management Plans (**Appendix 3**). The required OSD storage volume and post development outflow for all storms for the 50%, 20%, 5% and 1% AEP events have been summarised in **Table 10** below for the ultimate developed subdivision.

Table 10 – On-Site Detention Basin Size Summary – Ultimate Development

Storm Event ARI (Yr)		Total Catchment Flow (m3/s)		Total change in peak flow (m3/s)	Total Basin Storage Provided (m3)
	Pre-Develop	Post-Develop	Post-Develop with OSD		
50% AEP	1.54	1.657	1.15	-0.390	379.3
20% AEP	3.59	3.65	2.74	-0.85	398.3
5% AEP	7.75	8.458	7.38	-0.370	499.6
1% AEP	11.6	12.717	10.9	-0.700	610.0

Detailed design should be submitted to Council for the design of the four proposed bioretention/OSD basins for new road with a construction certificate application for the subdivision. For each lot, each owner would also need to provide a stormwater drainage plan with the submission of a development application to construct a dwelling on each lot incorporating the required rainwater tank and bioretention basin/OSD volumes as previous outlined in **Sections 7.2.4 and 7.2.5.**

7.3 Bioretention Basins (Raingardens)

7.3.1 Proposed Road

Four bioretention basins are to be constructed and located within the road reserve to treat stormwater run-off from the new road.

Each of these bioretention basins would have the following characteristics:

- A filter area of 125 m^2.
- A 300 mm ponding depth.
- An overflow weir at the surface.
- A 700 mm thick filtration layer of loamy sand (see spec. below).
- A 100 mm thick transition zone.
- A 300 mm thick drainage layer.

- Be unlined to allow infiltration into the naturally-permeable soils.
    - Be planted with moisture loving species such as *Carex* and *Juncus* at 8 plants/m²

7.3.2 *Proposed Lots*

A small bioretention basin is to be constructed on each lot.

Each of these bioretention basins would have the following characteristics (**Figure 10**):

- A filter area of 15 m².
- A 200 mm ponding depth.
- An overflow weir at the surface.
- A 500 mm thick filtration layer of loamy sand (see spec. below).
- A 100 mm thick transition zone.
- A 200 mm thick drainage layer.
- Be unlined to allow infiltration into the naturally-permeable soils.
- Be planted with moisture loving species such as *Carex* and *Juncus* at 8 plants/m²

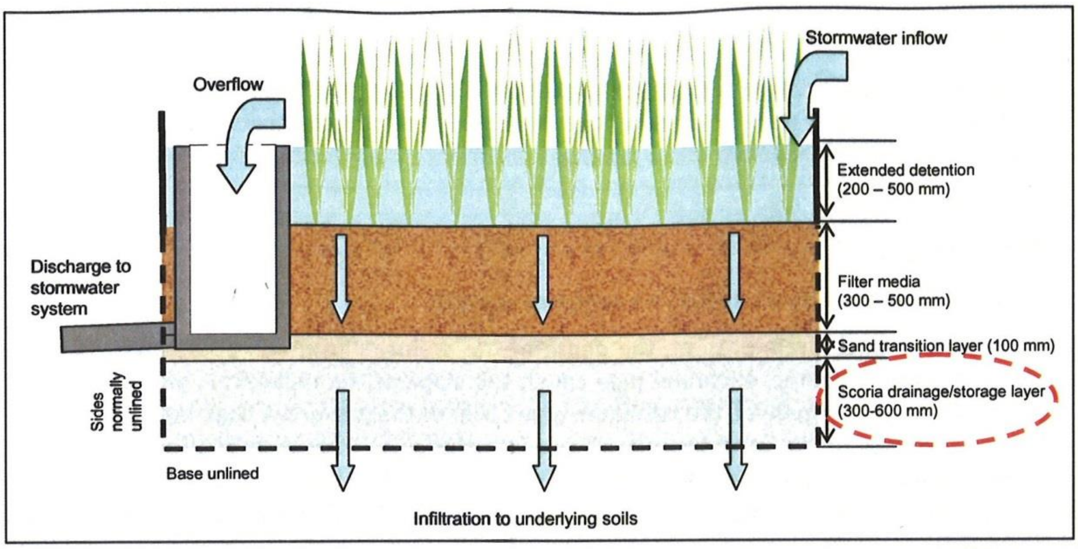


Figure 10 – Typical Section Through Bioretention Basin

Bioretention Basin Media Specification

The filtration media will be well-graded loamy sand with:

- *Hydraulic conductivity (ASTM F1815-06) between 250 and 300 mm/hour*
- *pH between 5.5 and 7.5*
- *Organic content less than 5 percent*

- *Electrical conductivity less than 1.2 dS/m*
- *Orthophosphate content less than 20 mg/kg*
- *Total nitrogen content <400 mg/kg*

Subject to adequate hydraulic conductivity the following particle size distribution is a guide:

Clay and silt	*< 3%*	*(<0.05 mm)*
Very fine sand	*5-30%*	*(0.05 - 0.15 mm)*
Fine sand	*10-30%*	*(0.15 - 0.25 mm)*
Med-Coarse sand	*40-60%*	*(0.25 - 1.0 mm)*
Coarse sand	*7-10%*	*(1.0 - 2.0 mm)*
Fine gravel	*<3%*	*(>2.0 mm)*

The filtration media will be compacted with one pass of a vibratory plate compacter or drum roller.

The transition layer shall be clean, well-graded sand containing little or no clay and silt (<2%). D15 of the transition layer must be <5 x D85 of the filter media.

The drainage layer shall be 2 - 7 mm washed screenings with <2% silt and clay.

7.4 Enviropod Pit Inserts or Gross Pollutant Traps (GPT's)

The water quality modelling includes the use of Enviropod pit inserts in all drainage pits located within the road side table drains. If this is not acceptable to Council then end of line Gross Pollutant Traps should be installed immediately upstream of the Bioretention basins located within the road reserve.

8 Water Quality Modelling

8.1 Pollutant Reduction Targets

The Shoalhaven City Council has a Water Sensitive Urban Design Policy (Chapter G2 – Sustainable Stormwater Management and Erosion/Sediment Control – Section 5.2.4 that states their required pollutant reduction targets as:

- Gross Pollutants – Retention of litter greater than 40mm for flows up to the 4 exceedances per year (EY) event (3-month ARI peak flow) and course sediment retention of sediment courser than 0.125mm for flows up to the 4EY peak flow.
- 80% reduction in average annual load of total suspended solids
- 45% reduction in average annual load of total phosphorous
- 45% reduction in average annual load of total nitrogen

8.2 Water Quality Modelling Introduction

Pre and post development sediment and pollutant loads were modelled for the development using MUSIC (Model for Urban Stormwater Improvement Conceptualisation), developed by the CRC for Catchment Hydrology (now eWater).

MUSIC contains algorithms based on the known stormwater runoff, pollutant generation from typical land uses and the performance characteristics of common stormwater quality treatment measures. These data are derived from research undertaken by eWater and others in Australia and overseas. The models have been developed using MUSIC default parameters (Table 12) for various land uses. Statistics are produced in MUSIC for the following parameters:

- Flow (ML/yr)
- TSS - Total Suspended Solids (kg/yr)
- TP - Total Phosphorus (kg/yr)
- TN - Total Nitrogen (kg/yr)
- Gross Pollutants (kg/yr).

The MUSIC model does not cater for hydrocarbons however it is assumed that if the total suspended solids are reduced by 80% the required reduction on hydrocarbons will also be achieved.

8.3 Climate Calibration

Creation of a MUSIC catchment file requires an associated meteorological data file. CMA (2010) recommends using data obtained from the Bureau of Meteorology's pluviougraph rainfall station at Nowra for the period 1964 to 1970. However, that data has a mean annual rainfall value of just 874 mm and so is not suitable. Therefore, Nowra data from

1970 to 1975 was used as that has a higher mean annual rainfall (1,179 mm). Basic rainfall and evapotranspiration statistics are in **Table 11** and the time-series graph is in **Figure 11**.

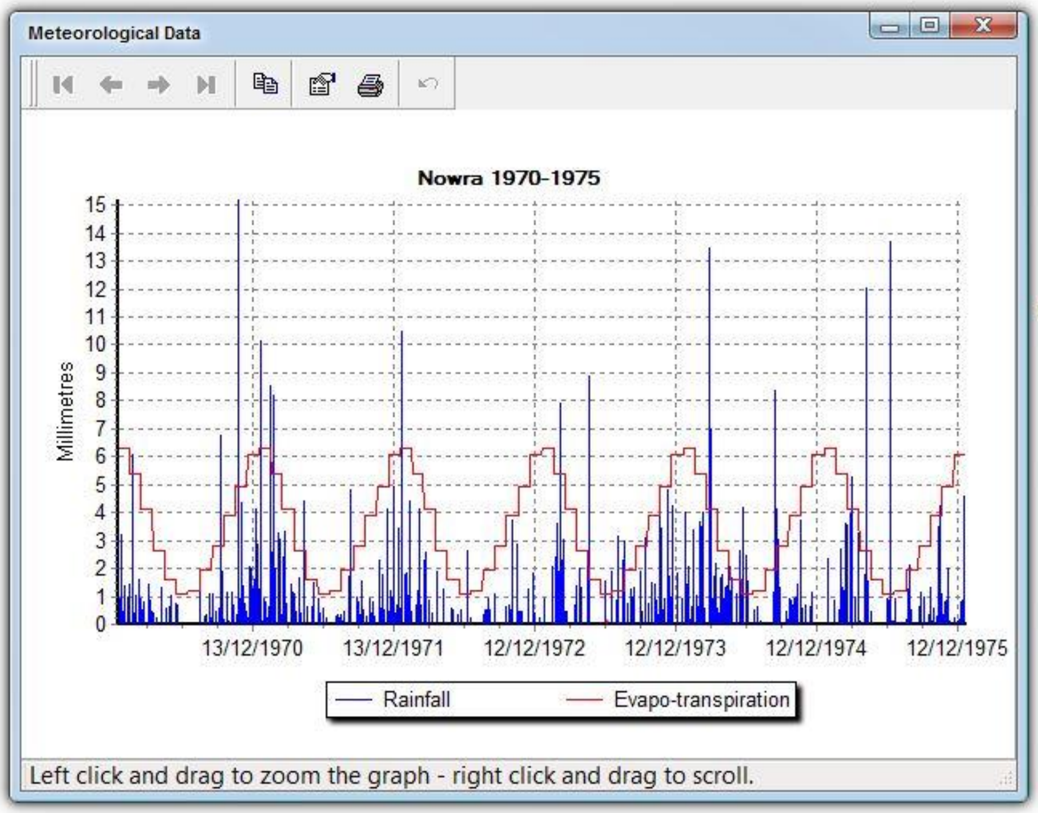
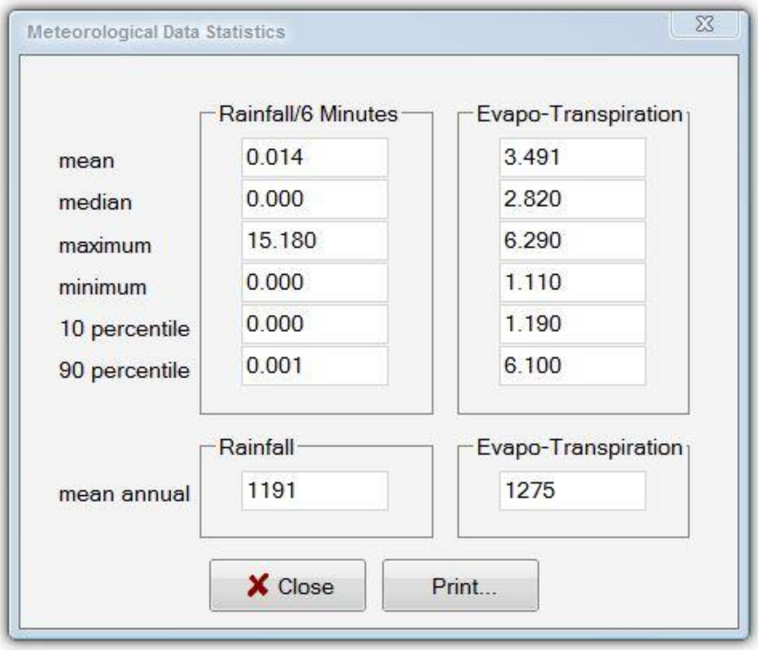


Figure 11 – Time Series Graph for Adopted Rainfall Data (6 minute timestep)

Table 11 – Adopted and Evapotranspiration Statistics (Nowra 1970-1975)



8.3.1 Node Data

Table 12 presents the storm flow concentration calibrations for the MUSIC model. They are derived from CMA (2010).

Table 12 - Storm flow concentration calibrations used in MUSIC

	TSS mean (log mean)	TSS std dev (log std dev)	TP mean (log mean)	TP std dev (log std dev)	TN mean (log mean)	TN std dev (log std dev)
Existing land (agricultural)	141 (2.15)	2 (0.31)	0.6 (-0.22)	2 (0.3)	6.31 (0.48)	1.82 (0.26)
Rural residential land	89 (1.95)	2.1 (0.32)	0.22 (-0.66)	1.8 (0.25)	2 (0.3)	1.55 (0.19)
Sealed road	269 (2.43)	2.1 (0.32)	0.5 (-0.30)	1.8 (0.25)	2.19 (0.34)	1.55 (0.19)
Conservation Zone	89 (1.95)	2.1 (0.32)	0.22 (-0.66)	1.8 (0.25)	2 (0.3)	1.55 (0.19)

The pervious area characteristics for both pre and post modelling are in **Table 13**. They are based on the method described in Section 3.6.3 of CMA (2010), see also **Section 8.3.2.**

Table 13 - Pervious area calibrations used in MUSIC

Parameter	Value
Soil storage capacity	210
Initial storage	30
Field capacity	66
Infiltration capacity coefficient	215
Infiltration capacity exponent	2.4
Groundwater initial depth	10
Daily recharge rate	55
Daily base flow rate	10[5]
Daily deep seepage rate	0

8.3.2 Catchment Hydrology Check

To check the model's hydrological output, the outflow from a pre-development, 100% pervious, node was checked against the Annual Runoff Fraction (**Figure 13**). The site's mean annual rainfall is 1,423 mm so the annual runoff fraction should be about 0.35 which equals 4.98 ML/ha/yr. The soil storage capacity and the soil field capacity were adjusted until this fraction was reached (+/- 10%); the model's value was 5.34 ML/ha/yr.

A similar check was made of the post development model, assumed to be 5% effective impervious (excluding the road). The annual runoff fraction should be about 0.38 which equals 5.41 ML/ha/ year. The model's value was 5.62 ML/ha/yr.

[5] Baseflow is specified as at least part of the site drains to the wetland and baseflow would enter it (CMA, 2010)

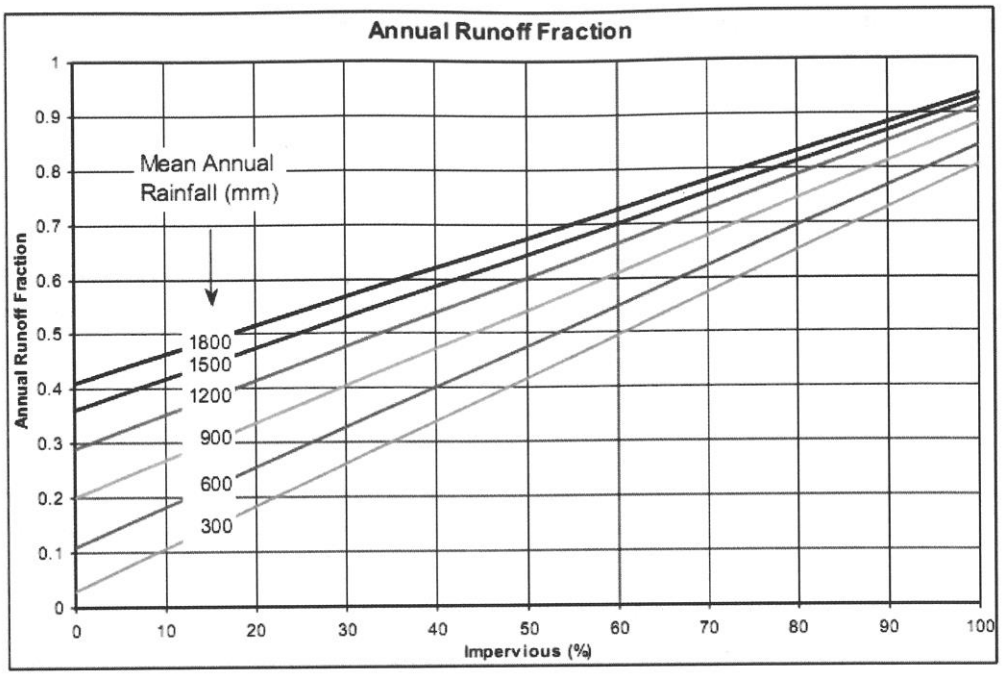


Figure 12 - Annual Runoff Fraction

8.4 Pre Development Modelling Assumptions

The pre-development model is comprised of a a agricultural source node, 100% pervious and adopting parameters as in **Table 12**:

- 28.38 ha that drains to Beach Road.

8.5 Post Development Assumptions

For the purpose of modelling we have assumed:

- Each new lot will be developed with a house having a roof area of 300 m^2;
- At least 80% of the roof would drain to its associated rainwater tank;
- The remaining building envelope of 300m^2 on each lot has an impervious area of 20% and would drain to a 15m^2 bioretention basin;
- Each new home would have a minimum 40 kL rainwater tank with 20 kL of that dedicated to domestic use. The anticipated demands on that 20 kL for use in the toilet and laundry, and at least one outdoor tap is 0.8 kL/day (SCA, 2012).
- The new road surface water run-off is to be directed to the grassed swale drains on each side of the carriageway and then into four bioretention basins, each with an area of 125m^2.

8.6 Treatment Train Effectiveness (Results)

A screenshot of the model is provided below for the development. It highlights the break-up of the catchment areas into roof areas that drain to a rainwater tank and areas that flow directly to the stormwater system.

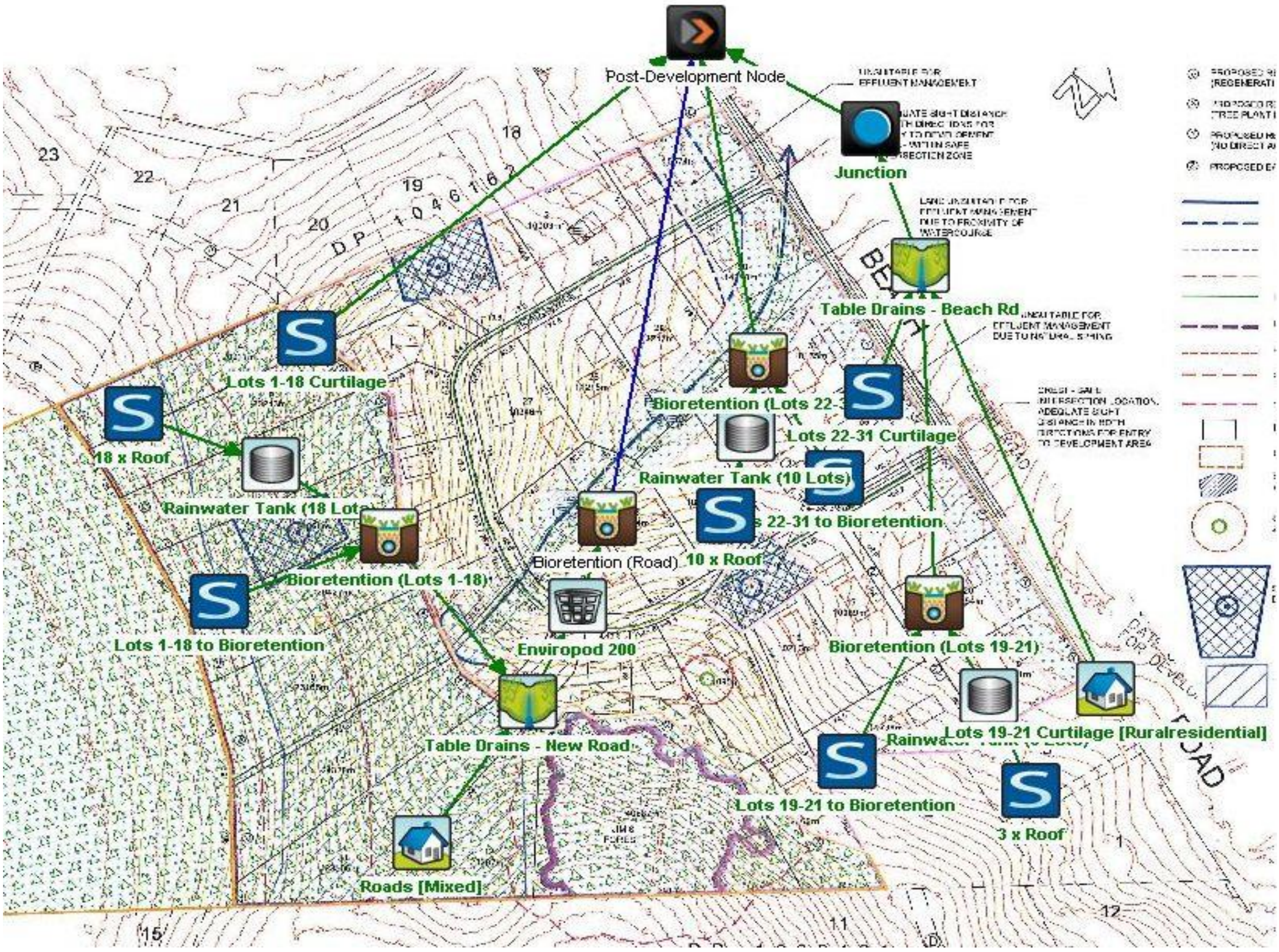


Figure 13 –Stage 3 MUSIC model layout

Table 14 provides the results of the MUSIC modelling. The target reductions for all parameters are met with post development reductions greater than the targets specified in **Section 8.1**.

Table 14 – Stage 3 MUSIC Results[6]

	Sources	Residual Load	% Reduction
Flow (ML/yr)	185	151	18.2
Total Suspended Solids (kg/yr)	13200	2420	81.7
Total Phosphorus (kg/yr)	31.7	13.1	58.7
Total Nitrogen (kg/yr)	284	153	45.9
Gross Pollutants (kg/yr)	2450	0.266	100

8.7 Monitoring and Maintenance

8.7.1 The Developer's Responsibilities

The developer shall be responsible for cleaning and maintenance of all water quality and detention systems including pits, swales and GPTs for a period (to be negotiated with Council) from the date of Certificate of Practical Completion for the development or for any period specified as part of a Voluntary Planning Period Agreement.

The construction of WSUD elements within the subdivision should be staged to minimise maintenance and prevent clogging by excessive sediment.

8.7.2 The Council's Responsibilities

Council would ultimately become responsible for the trunk drainage and the subdivision-scale water quality structures (i.e. the grassed swales and inlet filters).

8.7.3 The Residents' Responsibilities

The rainwater tank collection system (gutters, pipes, rainwater tanks, pumps and valves) would be maintained by each home owner. They would require periodic inspection and maintenance but the requirements are not great. Periodically the home owners would check all inlets, outlets and pumps for stability and operational performance.

[6] SEEC Internal reference = 19000408 Run 4

The homeowners would also be responsible for the maintenance of the bioretention basins on each lot including the additional bioretention basins required to treat the road stormwater run-off.

8.7.4 Operational Environmental Management Plan

An Operational Environmental Management Plan shall be submitted to Council for all stormwater treatment measures proposed, whether the asset is to remain in the private ownership or to be handed over to Council.

9 Summary and Conclusion

Stormwater runoff from large lot residential lands and environmental zones is expected to have better quality than stormwater runoff from agricultural lands (CMA, 2010). Therefore, the land use changes are expected to result in an improvement in stormwater quality.

A Concept Surface Water Management Plan (**Appendix 3**) has been prepared for the development and incorporates water sensitive urban design structures, including rainwater tanks, grassed swales, GPT and bioretention basins that provide an overall treatment train that meets the reduction targets as outlined in Shoalhaven City Council's DCP – '*Chapter G2 – Sustainable Stormwater Management and Erosion/Sediment Control'.*

On-site detention is also to be provided within the rainwater tanks and bioretention basins to ensure that post development peak flows are no greater than the pre-development flows.

All future residential lots would require onsite wastewater management and rainwater tanks to supply potable water. An assessment of the constraints and opportunities for onsite wastewater management has found that much of the site is unconstrained and so effluent management would not be problematic. However, there are some areas that are constrained and would be avoided (Figure 10); most notably:

- A 100m buffer from Commonderry Swamp.
- A 40m wide strip of land along Beach Road representing a buffer to Beach Road's table drain (which carries significant flow in rainfall).
- A 40 m buffer from a depression on the north-facing slope (currently containing two farm dams).
- An area of land in the far northwest representing the Shoalhaven Soil Landscape and which is low lying and prone to saturation.
- Land generally within 40-50m of a spring.

Subsurface effluent disposal would be required because of the relatively wet climate and the moderately steep topography. The proposed EMA's taking into account the abovementioned site constraints are shown in the proposed development plan (**Figure 2**).

10 References

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11 Appendices

11.1 Appendix 1 - Amendment to Groundwater Impact Assessment. Part Lot 4 in DP834254 No. 510 Beach Road Berry

overpage

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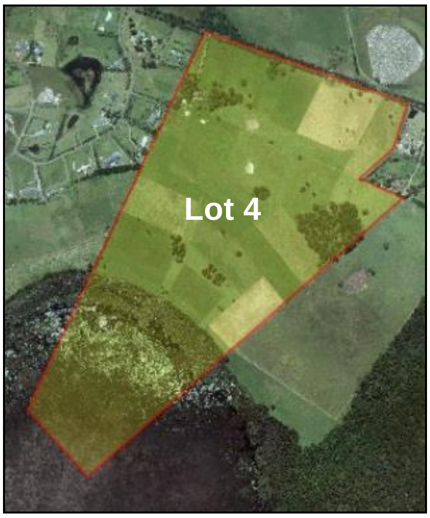
GROUNDWATER IMPACT ASSESSMENT

Part Lot 4 in DP834254
510 Beach Road Berry

for

E. Hall



Distribution

E. Hall (1) **Rep.No. 17134-B**

File (1) **8th March 2018**

CONTENTS

TABLES

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PLATES

1. INTRODUCTION AND BACKGROUND

As requested by *E. Hall* (the owner), *Larry Cook Consulting Pty Ltd* has undertaken a site inspection and carried out an assessment of potential impacts on the shallow groundwater system from proposed on-site wastewater (effluent) disposal on Part Lot 4 in DP834254 No. 510 Beach Road Berry South Coast New South Wales (the Site). The Site is shown on a lot plan in **Figure 1** and on a topographic plan presented in **Figure 2**. A plan of the Site supplied by *Strategic Environmental & Engineering Consulting* (SEEC) is presented in **Figure 3**.

It is understood that the owner proposes to rezone part of the Site from RU1 (Primary Production) to R5 (Large Lot Residential). In this regard, Shoalhaven City Council (Council) requires an assessment of on-site disposal of treated wastewater on shallow groundwater systems that may be present on the Site. It is understood that SEEC has prepared a conceptual Water Cycle Management Study (WCMS) that incorporates a conceptual design for on-site wastewater disposal.

An On-Site Sewerage Management system (OSSM) is designed to:

1. Dispose of treated effluent on-site using an approved and effective methodology in accordance with the Environmental Health Protection Guidelines (DLG 1998) and AS/NZS 1547:2012 (SAI & NZS 2012).
2. Meet the environmental and health *Performance Objectives* documented in the Environmental Health Protection Guidelines (DLG 1998) which ensure that on-site sewage management for single households is appropriate and will not affect public health or the environment.
3. Satisfy the requirements documented in Chapter G8 (Onsite Sewage Management) of Council's DCP (2014).

The objectives are:

- **Prevention of public health risk**. Contact with effluent should be minimised or eliminated, particularly for children. Residuals, such as composted material, should be handled carefully. Treated sewage should not be used on edible crops that are consumed raw
- **Protection of lands** - on-site sewage management systems should not cause deterioration of land and vegetation quality through soil structure degradation, salinisation, waterlogging, chemical contamination or soil erosion
- **Protection of surface waters** - on-site sewage management systems should be selected, sited, designed, constructed, operated and maintained so that surface waters are not contaminated by any flow from treatment systems and land application areas (including effluent, rainfall run-off and contaminated groundwater flow)

2. EXISTING DEVELOPMENT

The Site is a mostly cleared and partly developed parcel of land. A *Google Earth* image over the Site showing existing conditions is presented in **Figure 1**. Fenced paddocks cover the northern 60% of the Site. Three small farm dams are located in the northern part, two of these on the north-eastern fall and the third in the north-western corner. A wind mill is located on the northern edge of Coomonderry Swamp in the southern part of the Site (**see Plate 1**).

3. SITE INFORMATION

The Site is an approximate 74.8 hectare multi-sided but broadly wedge-shaped parcel of rural land oriented north-south. The Site is in the Shoalhaven City Council local government area and borders the southern side of Beach Road approximately 3.8 km east-southeast of its intersection with Agars Lane and 1.3 km west of its intersection with Gerroa Road.

The Site is surrounded by rural/residential properties to the east and west. There is mains power available to the Site but <u>no</u> town water or municipal sewerage system.

4. SITE ASSESSMENT

The Site is located within undulating low coastal hills and coastal wetlands. The Site is separated into two watersheds by a low-lying northwest-southeast trending ridge system that dissects the central northern part of the Site (**Figure 3**). Although no defined drainages occur on the Site, Coomonderry Swamp covers the southern 30 %. Overland flow north of the divide drains to Beach Road and south of the divide to Coomonderry Swamp. The elevation of the Site ranges from close to sea level at Coomonderry Swamp to about 30.0 m in the central part atop the divide.

The average slope of the Site away from the divide is approximately 10 to 15 %.

5. GEOLOGY

The Site is directly underlain by sedimentary rocks which belong to the Lower Permian-age Shoalhaven Group, in particular the Berry Siltstone. Colluvial deposits on the hillsides in the area are the products of erosion from the Berry Siltstone. The Berry Siltstone consists of interbedded, dominantly flat-lying, mid grey to dark grey siltstone and fine sandstone. Strongly weathered outcrop was observed on the more elevated central parts off the Site.

The Berry Siltstone on the flanks of coastal valleys and hillsides in the district is variably deeply weathered and generally covered with a silty sandy colluvial and residual silty sandy loam to sandy-silty clay soil profile which can vary in thickness from 0.2 m to greater than two metres. The soils are predictably thicker in the lower parts of the valleys and on the flanks of the ridge systems where deep colluvium (and alluvium) can be developed, such as is the case in the northern and southern parts of the Site.

6. SOIL ASSESSMENT

The reader is referred to the soil mapping carried out by The Department of Conservation and Land Management (Hazelton P.A., 1992). The soils beneath the Site are grouped into three soil landscapes:

- The Coolangatta Soil Landscape which is a residual soil landscape developed on undulating rises and low rolling hills. This soil covers the majority of the Site.
- The Shoalhaven Soil Landscape which is an alluvial soil landscape mapped in the far north-western part of the Site.
- The Seven Mile Soil Landscape which is an estuarine soil landscape occupying Coomonderry Swamp in the southern part of the Site.

Soil investigations recently carried out by SEEC in the central part of the Site revealed that this area is directly underlain by residual soils belonging to the Coolangatta Soil Landscape. The soil profile comprises approximately 0.2 to 0.3 m of dark brown strongly pedal loam overlying 0.2 to 0.5 m dark brown moderately pedal sandy clay loam. Strongly weathered shale to fine sandstone was intersected between approximately 0.8 and 1.0 m depth.

Soil investigations in the north-western part of the Site intersected a sequence consisting of interlayered grey weakly pedal clay loam and mottled grey-orange clay loam grading down into light/medium clay. Strongly weathered shale was recorded at about 0.9 m depth in the north-western corner.

Groundwater was not encountered in the SEEC investigations.

7. HYDROGEOLOGY

7.1 Aquifers

The sedimentary rocks belonging to the Shoalhaven group generally possess low porosity and permeability resulting in restricted groundwater flow. Localised water bearing zones (aquifers) may occur. Groundwater hosted by the Shoalhaven Group is generally brackish with very low yields.

7.2 Aquifer Recharge

Aquifer recharge is primarily by way of excess precipitation (rainfall), in particular the water that infiltrates the vadose (unsaturated) zone and is not consumed by evapotranspiration. A proportion of this recharge will provide base flow to the drainages in the district.

7.3 Aquifer Discharge

The reported existence of springs in the more elevated part of the Site indicates the presence of 'shallow' groundwater which is often collectively referred to as 'water features' or 'springs'. Three discharge sites were identified by the owner and inspected in September 2017. The locations of the known springs are shown in **Figure 4**:

1. An area of 'dry' exposed, disturbed soil located on the southern elevated side of the central ridge system at about 22 m RL (**see Plate 2**). The spring is located within a 30 m-wide corridor regularly travelled by dairy cows. There was no evidence of any groundwater seeps at the time of the site inspection.
2. A grassed covered area in a fenced paddock on the northern side of the central ridge south of the dwelling in the north-western part of the Site. The elevation of the spring is approximately 22 m RL (**see Plate 3**). A relatively small covered concrete tank has been historically buried in this location to presumably intercept and detain some of the shallow groundwater discharging at this point.
3. An area of 'dry' exposed disturbed soil located on the northern elevated side of the central ridge system at about 22 m RL (**see Plate 4**). There was no evidence of any groundwater seeps.

Inspection of the 'spring' sites (groundwater discharge zones) in September 2017 did not reveal any surface groundwater discharge or wet areas.

Conceptually, the groundwater system associated with springs is simple. It consists of:

- A **recharge area** where water enters the subsurface
- An **aquifer or set of aquifers** through which the water flows, and
- A **discharge point** where water emerges at the ground surface as a spring

The existence of springs on the elevated part of the Site indicates that some of the recharge infiltrates down to very shallow zones, likely down to the base of the weathered zone where 'perching' of shallow groundwater may occur. This water then migrates laterally down gradient.

The existence of a spring requires that below the ground surface, the infiltrating water encounters a low-permeability zone and is unable to continue to percolate downward as fast as it is supplied at the surface. As a result, the water migrates laterally until it intersects the land surface potentially discharging as springs where erosion has lowered the topography to the water's level (e.g., on the side of a gully, hill or valley). This is the situation to the north and south of the ridge line in the central part of the Site. For many people, springs are the most obvious and interesting evidence of groundwater.

Although the discharges from these springs are believed to vary in response to seasonal and climatic factors, anecdotal evidence indicates that they are low volume intermittent flows and/or seeps sometimes in an amount large enough to form a pool or stream-like flow. Spring discharge of local subsurface flow systems is closely related to recharge of precipitation and can show wide fluctuations in flow. Anecdotal evidence indicates that the seep on the southern side of the ridge system sometimes renders the ground wet (muddy).

The apparent elevation control of these springs (about 22 m RL) may also be associated with a geological contact within the flat-lying siltstone and sandstone beds, that is, 'contact springs' associated with the permeability contrast between a relatively permeable upper sedimentary bed and less permeable ('tight') contact or lower sedimentary unit.

The importance of spring systems is that they can support *Groundwater Dependant Ecosystems* (GDEs) which can be established at these groundwater discharge points.

7.4 Groundwater Dependent Ecosystems

Groundwater Dependant Ecosystems (GDEs) are ecosystems with potential for reliance on either the surface or sub-surface expression of groundwater. Ecosystems can be classified into three main categories according to their dependence on groundwater:

- Non-dependent;
- Facultative with some degree of groundwater dependence. This category can be subdivided into opportunistic, proportional and highly dependent; and
- Highly dependent/obligate.

Inspection of the 'spring' sites (groundwater discharge zones) in September 2017 did not reveal any surface groundwater discharge or the presence of GDEs. No other groundwater discharges were noted during the site inspection and no other features have been reported on the Site by the owner in his residence over several decades.

8. PROTECTION OF SHALLOW GROUNDWATER FEATURES

Any re-development of the Site (proposed rezoning) must ensure protection of the known shallow groundwater features (springs) from any potential adverse impacts from the on-site application of treated effluent.

In this regard, the Environmental Health Protection Guidelines (DLG 1998) and AS/NZS 1547:2012 (SAI & NZS 2012) provide a set of guideline buffer setback distances from ‘site features’ including ‘bore/well’ as defined in Table R1 in AS/NZS 1547:2012 (page 185). Although the water features are relatively small scale with no apparent associated dependent ecosystems, they are nevertheless categorised as shallow groundwater systems. Therefore the setback distances are considered appropriate for their protection.

The Environmental Health Protection Guidelines (DLG 1998) suggest a setback distance of 250 m (domestic water well (bore)). However, it is noted that Table R1 in AS/NZS 1547:2012 provides concessions on buffer distances from a bore/well and groundwater. In this regard, a range of setback distances between 15 and 50 m is recommended for a ‘bore/well’ and 0.6 to greater than 1.5 m for ‘groundwater’ based on a variety of sensitivities, limitations and parameters listed in Table R2 of AS/NZS 1547:2012. The items in Table R2 requiring assessment against a scale of constraints are A, C, H & J for a ‘bore/well’ and A, C, F, H, I, J for ‘groundwater’. These are assessed in **Table 1**

Table 1
Assessment of Buffer Constraint Levels

Item	Assessed Constraint Level	Rationale
A	Lower	Any domestic wastewater on new allotments will be treated to minimum secondary standard as recommended and specified by SEEC (2017)
C	Lower	Groundwater pollution hazard: Category 4/5 (clay loam to light clay) soil overlying bedrock.
F	Lower	Drainage Issues: Category 4/5 (clay loam to light clay) soil overlying bedrock on overall gently sloping site.
H	Medium to high close to springs	Groundwater pollution hazard: Category 4/5 (clay loam to light clay) soil with water table predicted to be relatively ‘deep’ but intermittent spring activity at approximately 22 m RL.
I	High immediately downslope of springs	Groundwater pollution hazard: resurfacing hazard
J	Lower	Off-site export of effluent, surface water pollution

In order to achieve an overall lower potential hazard the nominal buffer setback distance between a known spring on the Site and any proposed application of secondary treated wastewater (effluent management) should be no less than:

- 15 m sideslope
- 30 m upslope
- 50 m downslope

The downslope setback is also nominated to prevent any spring flow migrating downslope and impacting the LAA. The areas surrounding the known springs not considered suitable for application of treated wastewater are annotated in **Figure 4**.

9. CONCLUSIONS AND RECOMMENDATIONS

It is understood that the owner proposes to rezone part of the Site to R5 (Large Lot Residential). An assessment of on-site disposal of treated effluent (wastewater) from any new residential allotments on shallow groundwater systems revealed the presence of three relatively small water features (springs) known on the Site. These springs are shallow groundwater discharge zones associated with a contact between two sedimentary rock units (layers) in the relatively flat-lying, low-permeability Permian Berry Siltstone, part of the Shoalhaven Group.

The springs are located on the north and south sides of a small ridge system in the central part of the Site at an elevation of approximately 22 m RL. Inspection of the ‘spring’ sites did not reveal any surface groundwater discharge or the presence of any GDEs. No other groundwater discharges were noted during the site inspection and no other features have been reported on the Site by the owner in his residence over several decades.

In order to protect the known springs on the Site from any potential impacts associated with possible on-site disposal of treated effluent, a set of buffer setback distances has been developed based on guideline distances documented in AS/NZS 1547:2012. The downslope setback ensures that any spring flow will not migrate downslope and impacting any Land Application Areas.

10. REFERENCES

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For and on behalf of
Larry Cook Consulting

Larry Cook
Hydrogeologist

PLATES

Plate 1 Windmill

Plate 2 Spring – Southern Side of Divide

Plate 3 Spring – Northern Side of Divide North-Western Part of Site

Plate 3 Spring – Northern Side of Divide Central Northern Part of Site

FIGURES

Larry Cook Consulting	**Hydrogeological Assessment**	Scale: As shown
PO Box 8146 **Tumbi Umbi NSW 2261** **Phone 02 4340 0193**	**Lot 4 in DP834254** **510 Beach Road Berry** Lot Plan	**FIGURE 1**

Larry Cook Consulting
PO Box 8146
Tumbi Umbi NSW 2261
Phone 02 4340 0193

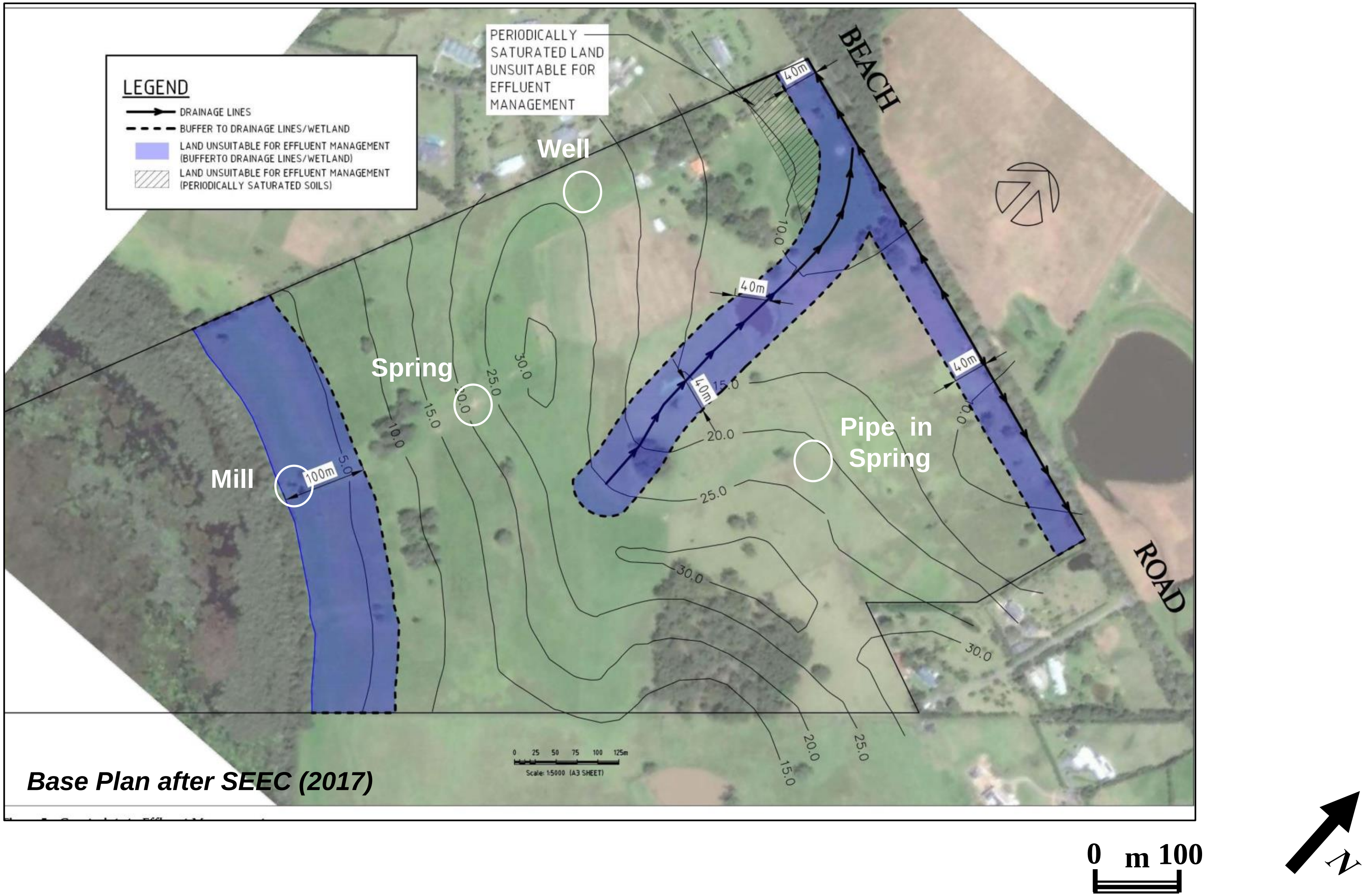
Hydrogeological Assessment

Lot 4 in DP834254
510 Beach Road Berry
Topographic Plan

N

Scale: As shown

FIGURE 2



Larry Cook Consulting
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Tumbi Umbi NSW 2261
Phone 02 4340 0193

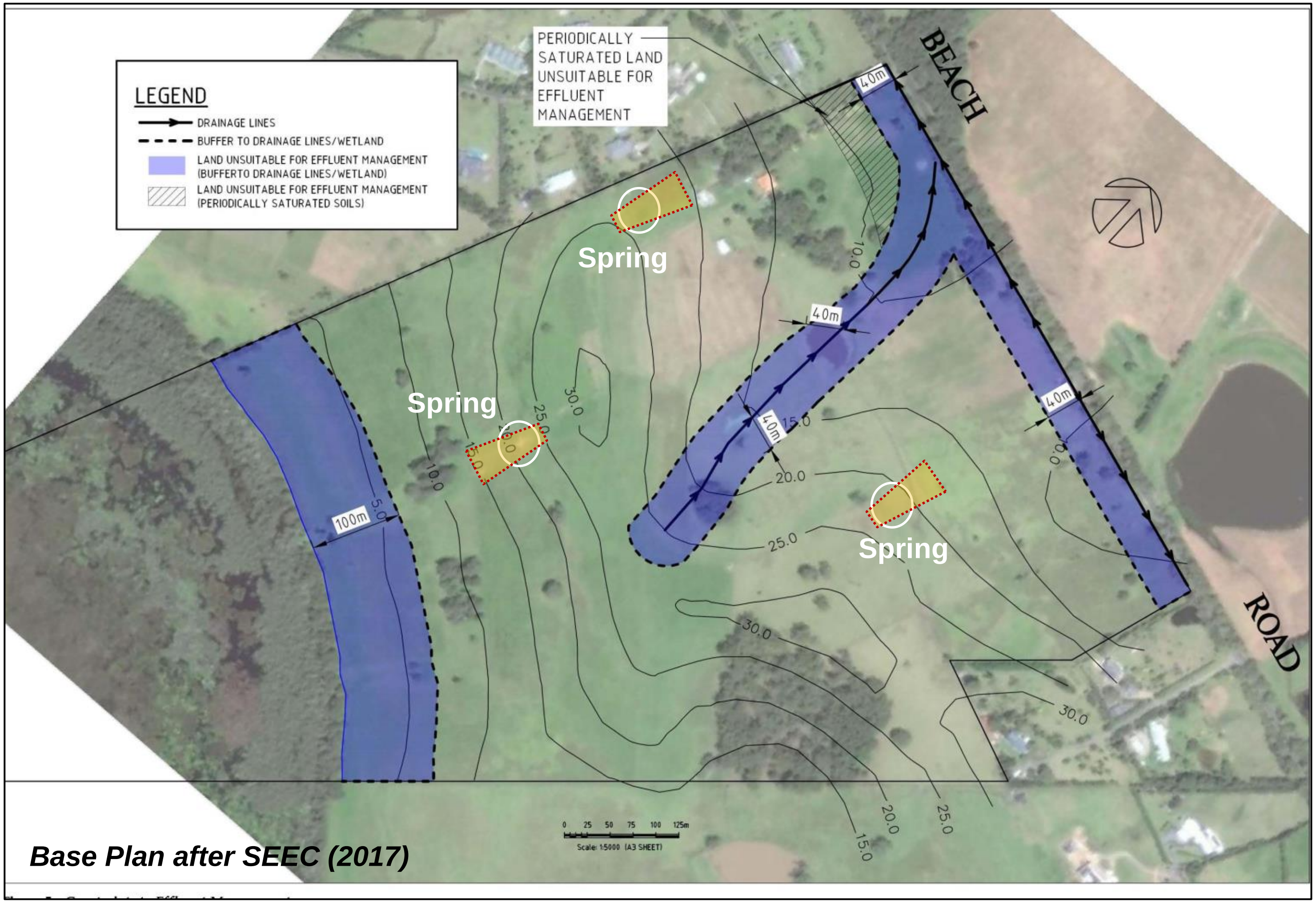
Hydrogeological Assessment

Lot 4 in DP834254
510 Beach Road Berry
Site Plan

N

Scale: As shown

FIGURE 3



Land Considered Unsuitable for Effluent Management (known springs)

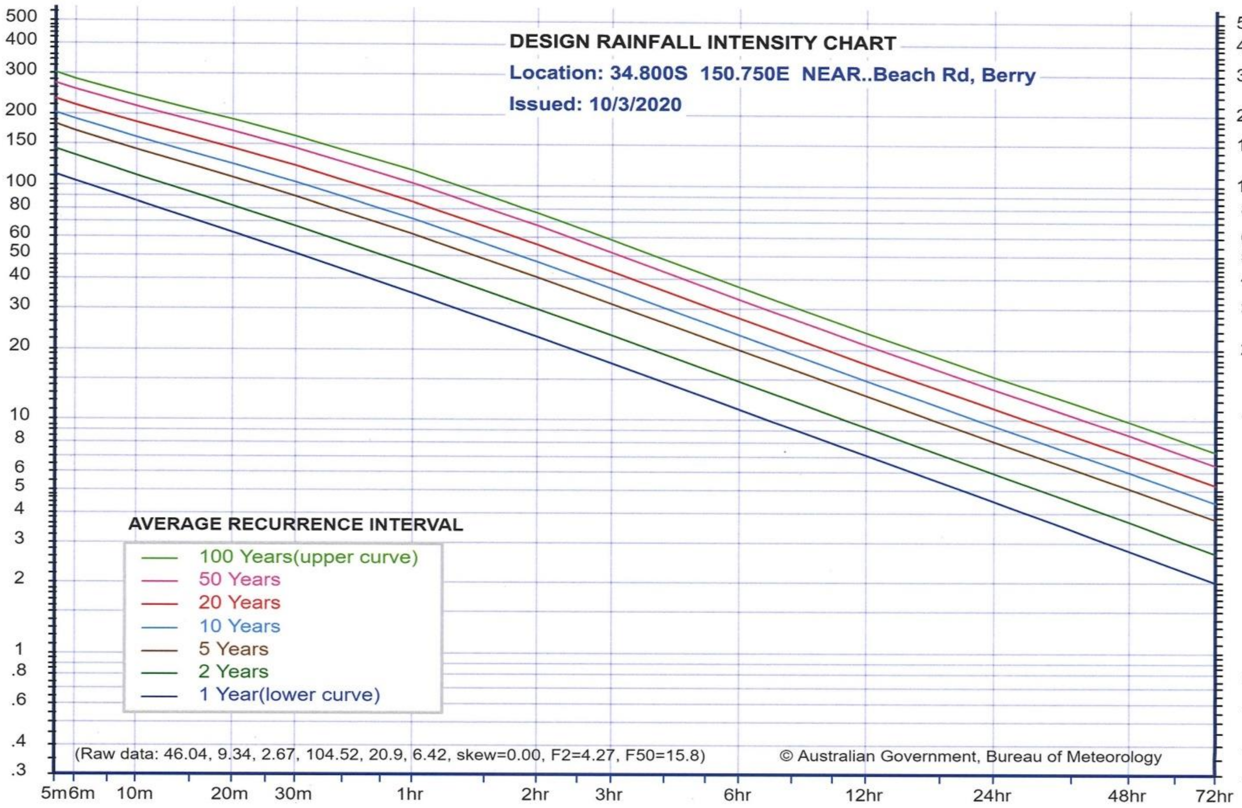
0 m 100

N

Larry Cook Consulting **PO Box 8146** **Tumbi Umbi NSW 2261** **Phone 02 4340 0193**	**Hydrogeological Assessment** **Lot 4 in DP834254** **510 Beach Road Berry** Areas Unsuitable for Effluent Disposal	N	Scale: As shown **FIGURE 4**

11.2 Appendix 2 – IFD Charts & Tables

AR&R 1987 IFD Chart & Table



Location: 34.800S 150.750E NEAR.. Beach Rd, Berry Issued: 10/3/2020

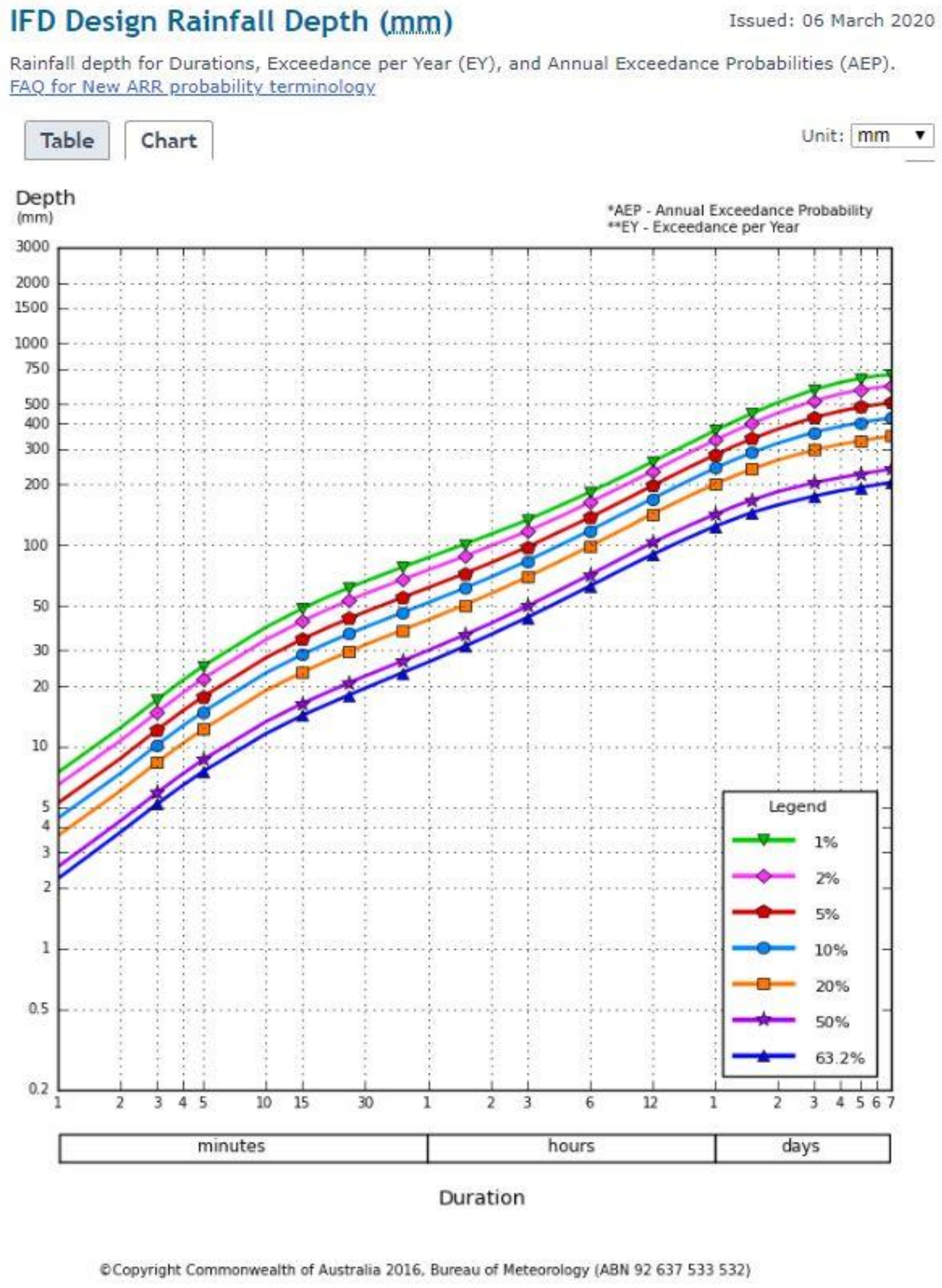
Rainfall intensity in mm/h for various durations and Average Recurrence Interval

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	111	143	182	204	233	271	300
6Mins	104	134	170	191	219	255	282
10Mins	85.4	110	142	160	185	217	241
20Mins	62.9	81.7	108	123	144	170	191
30Mins	51.3	66.9	89.5	103	121	144	162
1Hr	34.7	45.6	62.0	72.0	84.9	102	116
2Hrs	22.6	29.8	40.7	47.4	56.1	67.6	76.6
3Hrs	17.4	23.0	31.4	36.5	43.1	52.0	58.9
6Hrs	11.1	14.6	19.9	23.1	27.3	32.8	37.1
12Hrs	7.08	9.31	12.7	14.7	17.3	20.9	23.6
24Hrs	4.48	5.91	8.11	9.46	11.2	13.5	15.3
48Hrs	2.75	3.66	5.09	5.98	7.12	8.66	9.88
72Hrs	2.01	2.67	3.74	4.41	5.25	6.40	7.31

aw data: 46.04, 9.34, 2.67, 104.52, 20.9, 6.42, skew=0.00, F2=4.27, F50=15.8)

AR&R 2016 IFD Chart & Table



IFD Design Rainfall Depth (mm)

Issued: 06 March 2020

Rainfall depth for Durations, Exceedance per Year (EY), and Annual Exceedance Probabilities (AEP).
FAQ for New ARR probability terminology

Table Chart

Unit: mm

	Annual Exceedance Probability (AEP)						
Duration	**63.2%**	**50%#**	**20%***	**10%**	**5%**	**2%**	**1%**
1 min	2.21	2.53	3.61	4.41	5.24	6.44	7.43
2 min	3.76	4.28	6.04	7.36	8.73	10.7	12.4
3 min	5.19	5.91	8.36	10.2	12.1	14.8	17.1
4 min	6.44	7.35	10.4	12.7	15.1	18.5	21.4
5 min	7.54	8.62	12.2	14.9	17.7	21.8	25.1
10 min	11.6	13.3	19.0	23.2	27.6	33.9	39.1
15 min	14.3	16.4	23.4	28.7	34.2	42.0	48.5
20 min	16.3	18.7	26.8	32.8	39.1	48.1	55.6
25 min	18.0	20.7	29.6	36.2	43.1	53.1	61.3
30 min	19.5	22.4	32.0	39.1	46.6	57.3	66.1
45 min	23.2	26.5	37.8	46.1	54.8	67.2	77.5
1 hour	26.2	30.0	42.5	51.7	61.3	75.1	86.5
1.5 hour	31.4	35.8	50.4	61.1	72.2	87.9	101
2 hour	35.9	40.8	57.3	69.2	81.5	98.8	113
3 hour	43.7	49.7	69.3	83.4	97.6	117	133
4.5 hour	53.8	61.2	85.0	102	119	141	160
6 hour	62.6	71.2	98.7	118	137	163	183
9 hour	77.5	88.3	122	146	169	200	223
12 hour	89.8	103	143	170	197	232	259
18 hour	109	125	175	209	243	287	319
24 hour	124	142	201	241	280	331	370
30 hour	135	156	221	266	311	369	412
36 hour	145	167	238	287	336	400	449
48 hour	158	184	264	320	376	452	509
72 hour	175	204	297	363	430	520	591
96 hour	186	216	316	388	462	562	641
120 hour	193	225	329	405	483	589	672
144 hour	199	232	339	417	497	605	690
168 hour	205	238	347	426	508	616	700

Note:
The 50% AEP IFD **does not** correspond to the 2 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 1.44 ARI.
* The 20% AEP IFD **does not** correspond to the 5 year Average Recurrence Interval (ARI) IFD. Rather it corresponds to the 4.48 ARI.

11.3 Appendix 3 - Concept Surface Water Management & Flood Assessment Plans