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Hydrological Assessment

Integrated Live Export Facility (ILEF)

Report Number 23919. 80836

Prepared for

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This document provides information to address the intent of Project Number 23919 as agreed to by Wellard Rural Exports Pty Ltd.

Executive Summary

EnviroAg Australia Pty Ltd (EnviroAg) was engaged by Wellard Rural Pty Ltd (Wellard) to undertake a Hydrological Study at the proposed Livingstone Integrated Live Export Facility (ILEF) site in Darwin. The property is referred to as "Livingstone Valley" and is located on Lot 5544 Hundred of Strangways, 2658 Stuart Highway.

Based on the hazards and risks associated with the movement of potential surface water pollutants into the general environment, an ILEF catchment is normally considered to consist of four (4) functional components: support infrastructure (access roads, hard stands, offices) the ILEF itself and the ancillary facilities (feed mill, cattle handling and processing yards and manure stockpiles) servicing it, a wastewater utilisation area and a manure utilisation area.

The ILEF facility is defined within a Controlled Drainage Area (CDA). It has an area of just under 33 ha. The rainfall runoff from this area can contain entrained manures and thus contaminants. Most entrained manures are recovered in the sedimentation basins. The waste water is captured in a primary wastewater pond which is dewatered either directly to irrigation or alternately to a wet weather storage dam.

Current guidelines stipulate that the holding pond spillways be designed to handle a 1 in 50 year design storm event.

The FSIM model (Lott, 1998) was used to model the hydrology of the ILEF CDA. It found that a combined storage capacity of 220ML will safely accommodate all but extreme wet years.

The irrigable area is more than 40ha. The annual average years is estimated to be about 190ML/year and the 1 in 10 year wet season yield is estimated to be about 240ML/year. Thus the irrigation application rate is estimated to be about 4.75-6ML/ha/year. This is less than the expected crop water requirement of 10ML/ha/year. Some clean water will be available but this too will be limited. A crop water deficit is expect in late spring ("the build-up") and clean water will be held where ever possible to irrigate at this time to allow crop dry matter yields to be maximised.

The design dimensions of the sedimentation, holding and terminal ponds were compared using both the current National Feedlot guidelines (MLA, 2012) and the FSIM hydrology model (Lott, 1998).

The FSIM design would appear to be the more conservative of the approaches. These dimensions are to be adopted in the design of these structures in the proposed ILEF.

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1. Introduction

1.1 Site Description

EnviroAg Australia Pty Ltd (EnviroAg) was engaged by Wellard Rural Pty Ltd (Wellard) to undertake a Hydrological Study at the proposed Livingstone Integrated Live Export Facility (ILEF) site in Darwin. The property is referred to as "Livingstone Valley" and is located on Lot 5544 Hundred of Strangways, 2658 Stuart Highway (Figure 1).

The site of the proposed ILEF is located adjacent to Stuart highway as shown in Figure 1 below. It is near the eastern boundary of the Berry Spring Creek catchment, a tributary of the Blackmore River which feeds into the Middle Arm of Darwin Harbour in the north-west. Berry Springs is fed from dolomite aquifers and is a major source of water for the rural community.

The site is relatively flat and has been cleared, ploughed and sown to improve pasture. The land is currently used for grazing stock and pasture production. The site has an east to west slope of about 1.5% and a cross fall of about 0.5%. It is constrained by the railway line to the south – west of the site.

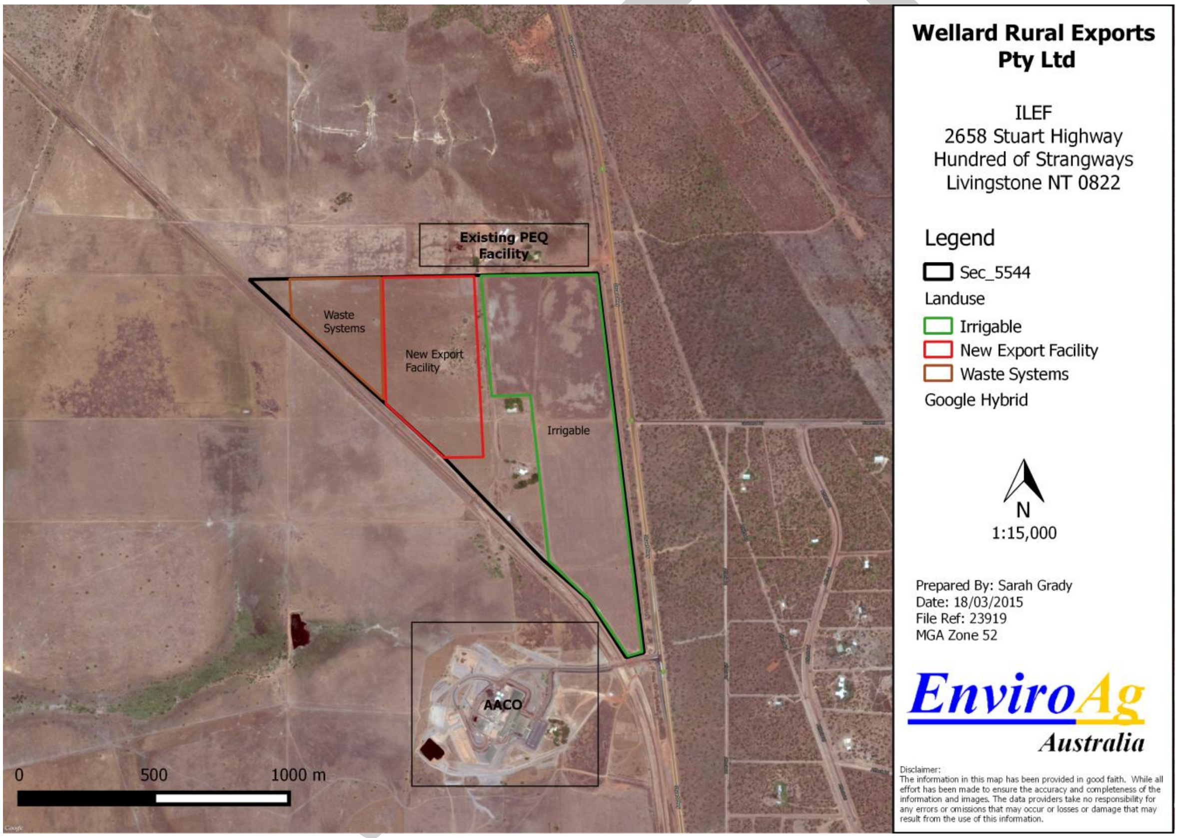


Figure 1 "Livingstone Valley" ILEF Site Satellite Image

1.2 Integrated Live Export Facility (ILEF)

An ILEF is characterised by several key land uses;

(i) Large areas of roof;

(ii) Large areas of road and hard stand; and ancillary land uses;

(iii) Open stock holding pen areas for the PEQ and small feedlot;

(iv) Sedimentation basins and waste water holding ponds; and,

(v) Irrigable area.

Clean water from the rooves, roads and hard stands are diverted away from the operation areas of the ILEF where rainfall runoff can become contaminated.

1.3 Site Hydrology

The site of the ILEF is at the top of a low ridge with no area lying within a flood zone. Areas at the bottom of the site have restricted drainage due to the change in slope as it becomes flat on the western slope toe, near surface soil types, and the impedance in drainage caused by the railway line. While these areas can be temporarily inundated in the wet season, due to their poor drainage, with improved drainage, surface waters can be better managed and safely removed from the site.

Figure 2 Topographic Cross section plan from irrigation area midpoint to nearest watercourses

Areas at the bottom of the site will be drained to the drainage line associated with the railway through construction of waterways on the property and its boundary with the railway line. Runoff passes under the railway line in existing large culverts.

Figure 3 Topographic cross section plan of Controlled Drainage Area Northern and Southern Boundary.

1.4 Design Guidelines

The pen areas are characterised by being areas of land that distinctively have manure as the principal form of "groundcover". Runoff from these areas typically carries quantities of dissolved nutrients and salts and may also entrain quantities of organic matter. If runoff containing these potential contaminants were to directly enter surface waters then pollution of that resource may result. Consequently, it is necessary for runoff to be adequately captured and safely stored. This typically involves the construction of runoff control structures and wastewater storages.

Guidelines exist that provide the design criteria for sizing runoff control structures in feedlots and, in particular, wastewater-holding ponds (Skerman, 2000; MLA, 2012). These guidelines have been based on research findings on the hydrology of feedlots under Australian climatic conditions. This research is described by Lott (1998). The research set out by Lott (1998) underpins current Australian National and State Government guidelines for the design and environmental management of the lot feeding industry.

The guidelines (MLA, 2012) can be applied to the hydrological assessment of the ILEF.

Australian design criteria for feedlots and other intensive livestock systems recommend or require that the holding pond design capacity be based on a modelled long-term (>10 yr) water balance undertaken for the ILEF catchment (ICIAI, 1997; SCARM, 1997, Skerman, 2000; MLA, 2012). This water balance adds, on a daily basis, sources of water such as direct precipitation and captured runoff, to the volume previously stored in the holding pond and then deducts transfers of water out of the holding pond by way of irrigation or evaporation.

Based on the output of the modelled water balance, a holding pond design capacity can be determined that will provide an allowable, probabilistic, design frequency for spills or overflow events. Other than in very sensitive environments the allowable frequency of overflow events is an average of less than once in every 10 years, for ponds from which waste water is routinely removed for land application (MLA, 2012).

Typically, wastewater captured in a waste water holding pond is used to irrigate crops that are able to utilise the water and assimilate the entrained nutrients. Guidelines require that any excess irrigation waters and some of the initial runoff from the irrigation area following a rainfall event be captured in a tailwater system. Captured tailwater is normally reapplied to the irrigation area once suitable conditions allow.

This report details the hydrology of the proposed Wellard Darwin ILEF. It describes the characteristics of the ILEF catchment, the design of the proposed runoff control structures and describes the methodology used to model the hydrological balance of the catchment.

2. Catchment Description & Characteristics

Based on the hazards and risks associated with the movement of potential surface water pollutants into the general environment, an ILEF or feedlot catchment is normally considered to consist of four functional components: the support land uses (access roads, hardstands and buildings), the ILEF itself, and the ancillary facilities (cattle handling and processing yards and manure stockpiles) servicing it, a wastewater utilisation area, and a manure utilisation area.

The measures taken in each of these areas to address the hazards will be commensurate with the risks posed and varies between the four areas and are site and land-use management specific.

2.1 The Controlled Drainage Area

2.1.1 Land usage in the controlled drainage area

The ILEF and its ancillary facilities typically pose the greatest risk to water quality in the external environment. To meet water quality objectives uncontaminated runoff water from any areas upslope of the ILEF should be prevented from entering the ILEF facility or the associated land areas where waste may be collected, stored or treated.

This runoff is typically excluded from these “controlled” areas by an upslope “clean” water diversion bank. Concurrently, runoff from within these “controlled” areas needs to be adequately captured and safely stored.

The resultant, controlled drainage area (CDA), can therefore be described as the area in which all wastewaters and runoff are to be controlled, captured and stored. In practical terms it is the land area between any upslope clean water diversion banks (or in their absence the top of the catchment) and the downslope wastewater holding ponds and typically is further delineated by the areal extent of drains catching and conveying ILEF runoff to a primary treatment and wet weather storage (waste water holding ponds).

In a CDA the majority of the land is normally used for ILEF pens. Other land uses in the CDA include, cattle processing facilities, roads and laneways, drains, manure stockpiles, hard stand areas and open or grassed areas between or around these facilities.

The layout of the proposed ILEF is detailed in the AutoCAD drawing provided in Figure 4 of this report. The relevant land use areas within the CDA of the proposed development are shown in Table 1. These areas were determined by using AutoCAD to determine areas within the plan . The CDA of the proposed ILEF compromises a catchment with a total area of nearly 33hectares (excluding roofs).

The layout of the ILEF is very compact considering the proposed capacity. This can be attributed to the favourable landform that has a natural fall of about 1-2% to the west and about 0.5% to the south. This allows the ILEF to be designed with pens aligned in straight rows on an east-west axis in a “back-to-back” configuration. In this configuration each row of ILEF pens is separated from the adjacent row of pens on its higher or “front” side by a common feed lane or alley (refer Figure 5).

A common catch drain on the lower or “back” side separates the next two rows of pens. The catch drains will discharge to three main drains that run down the slope at the northern and southern ends of the ILEF and through the middle. The natural fall to the west will result in a minor step down between each successive pair of ILEF pens. It is expected that the pens with a southerly aspect will have a grade of 2.5-4.5% whilst those with a westerly aspect will have a grade of 2.0-3.5%.

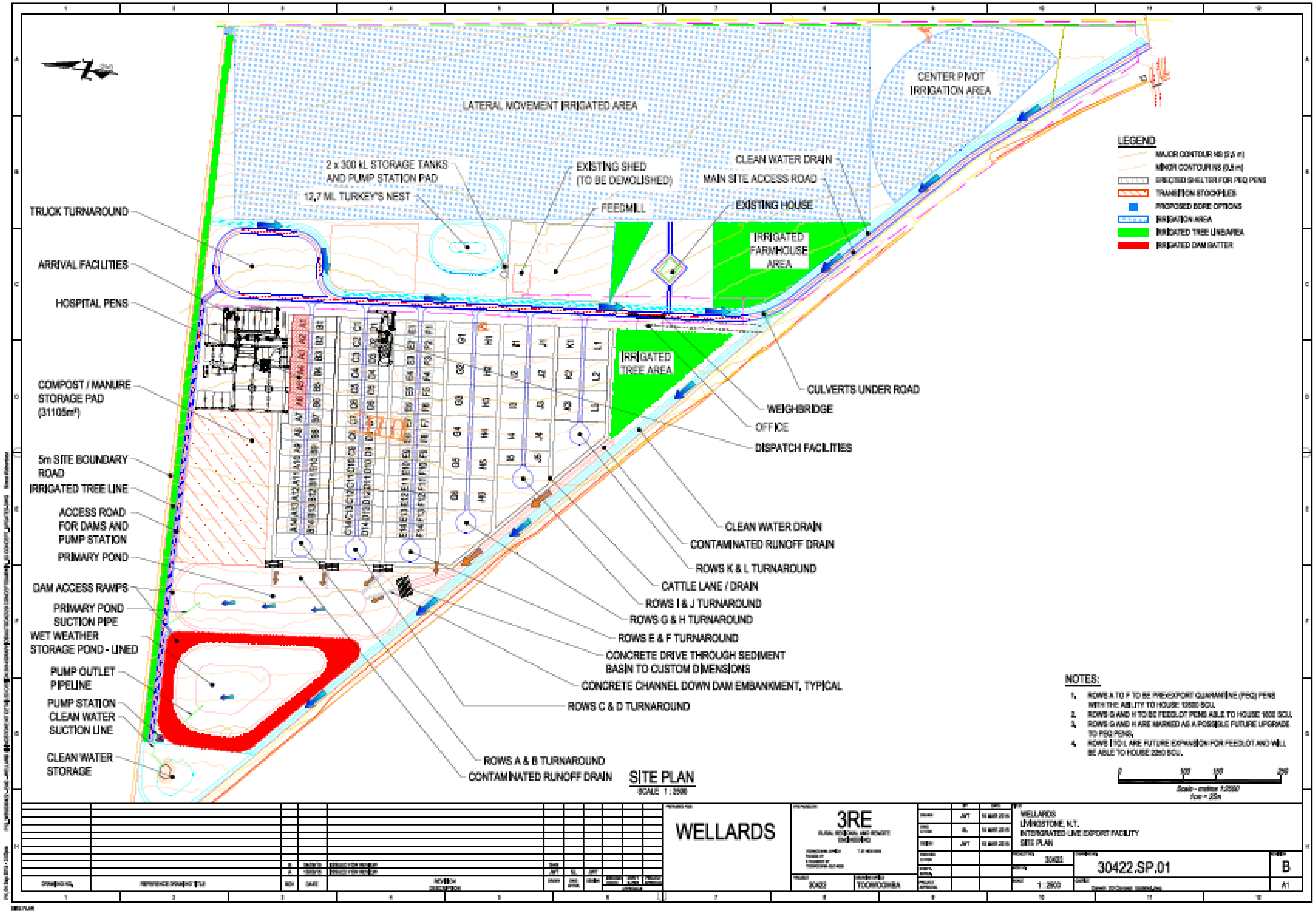


Figure 4 **Stage 1 site plan for Wellard's Integrated Live Export Facility**

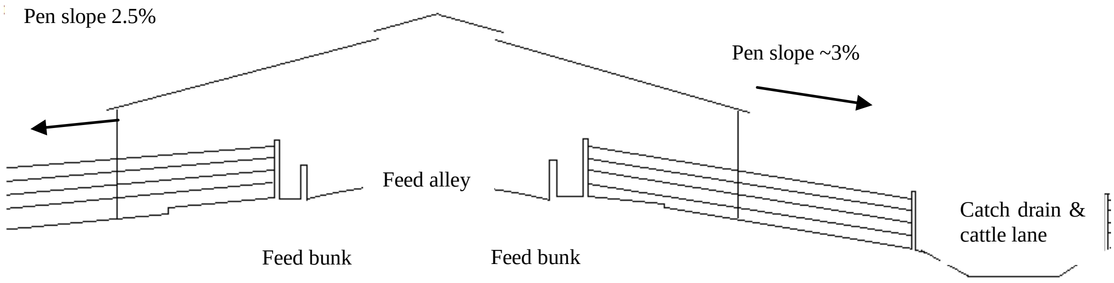


Figure 5 Notional north (left) south (right) transect (not to scale) through a row of "back-to-back" ILEF pens showing location of roof, feed alleys and catch drains

Table 1 Land usage within the controlled drainage area and non-controlled drainage area at the proposed ILEF

Controlled Discharge Areas	
Land Use	**Area**
Open pen area	96094
Cattle handling & processing facilities	23306
Combined cattle lanes and drains	20839
Manure stockpile	30591
Roads	28589
Undefined areas	52718
Hard stand areas	12906
Sediment basins	1289
Wet weather storage pond	25569
Primary wastewater pond	37445
Total	**329346**
Non Controlled Discharge Areas	
Land Use	**Area**
Pen rooves	45360
Clean water drains	26759
Buildings	1632
Irrigated house area	24903
Irrigated cropping area	333268
Irrigated tree line	2857
Irrigated tree line/area	23568
Freshwater runoff dam	15923
Freshwater supply turkey's nest	20531
Undefined area	81399
Total	**576200**
Combined Total	**905546**

2.1.2 Runoff Control Structures

The design principles of runoff control structures in ILEFs are discussed in detail in Lott (1994), Lott & Skerman (1995), ICIAI (1997), SCARM (1997), Skerman (2000) and MLA (2012).

With the proposed location of the ILEF yards and ancillary services on the upper slopes and over the crest of a low ridge (refer Figure 2), there will be no upslope runoff moving onto the site and as such diversion banks are not required in this instance.

Drains

Much of the pen areas are covered by roof. The rooves will discharge to a gutter and pipe and stormwater systems so that water does not enter the pens. This clean water will be conveyed to the clean water storages at the ILEF and below it.

Runoff from the exposed ILEF pen areas is to be collected in catch drains situated directly behind each ILEF pen. The pens are arranged in "back-to-back" rows (refer Figure 5, page 7). The configuration of the rows and the cross-slope gradient in the pens are designed to minimise the volume of runoff draining through adjoining pens.

The catch drains are also to serve as laneways providing access for cattle moving to and from the ILEF pens. The individual catch drains behind each row of pens are to discharge into main collection drains that will in turn discharge into a sedimentation system and ultimately the holding pond.

The catch drains and main drains need to be designed to both contain the flow volume and provide flow velocities that do not threaten channel stability at a peak flow rate equivalent to that from a design storm having an average recurrence interval (ARI) of 20 years (ICIAI, 1997, SCARM, 1997 and MLA, 2012). The maximum allowable flow velocity in channels is dependent on the characteristics of the material lining of the channel. High design velocities (>3 m/s) generally necessitate a concrete or masonry liner being applied. Where it is desirable to minimise any sedimentation of the entrained solids in the drains, minimum flow velocities (>0.3 – 0.5 m/s) may apply.

Sedimentation Basins

The aim of sedimentation system design is to provide flow velocities in the system low enough to allow for the settling of a minimum of 50% of the solids entrained in the CDA runoff in a design storm also having an ARI of 20 years (ICIAI, 1997 and SCARM, 1997). This level of sedimentation typically occurs when flow velocities are less than 0.005 m/s (Lott & Skerman, 1995). A performance standard requiring the settling of more than 50% of the entrained solids would require an exponential increase in detention time within the sedimentation system (as well as a correspondingly lower flow velocity) and therefore is generally impracticable and inefficient.

Peak Flow Velocities

To estimate the peak flow velocity in the catch drains, main drains and sedimentation systems it is necessary to determine the peak discharge of their respective catchments. The preferred method (ICIAI, 1997 and SCARM, 1997) for calculating the peak discharge of these catchments is the Rational Method as detailed by Pilgrim (2001). This methodology requires the prior estimation of the time of concentration of the catchments and the average rainfall intensity in the corresponding design storm.

Due to their relatively small size and the inability to derive observational data prior to construction, the Bransby Williams formula (Pilgrim, 2001) is often used to determine the time of concentration of ILEF catchments. This formula is given by:

Equation 1 Bransby Williams formula

$$t_c = \frac{58L}{A^{0.1} S_e^{0.2}}$$

where t_c = time of concentration (min)
L = mainstream length (km)

A = area of catchment (km²)
S_e = equal area slope (m/km).

Rainfall Intensity and Design

Having determined the time of concentration, the rainfall intensity of a design storm with an average recurrence interval of 20 years having a duration equal to the time of concentration can be derived for a location12.70080°E and 131.065°S. (Skew 0.36), using methodologies compatible with Canterford *et al*. (2001). The derived average rainfall intensity can then be applied to determine the peak volumetric flow using the formula for the Rational Method (Pilgrim, 2001) given by:

Equation 2 Rational method formula

$$Q_y = 0.278\, C\, {}^{y}I_t\, A$$

where Q_y = peak volumetric flow (m³/s) having an ARI of y years,
C = runoff coefficient (typically 0.8),
${}^{y}I_t$ = rainfall intensity (mm/h) of design storm having duration tc, and
A = catchment area (km²).

Home | IFD Table | IFD Chart | Coefficients | ARI | Print IFD table | Help IFD table

Intensity-Frequency-Duration Table

Location: 12.700S 131.075E NEAR.. Livingstone, NT Issued: 4/8/2015

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	142	180	220	244	279	325	362
6Mins	132	168	205	228	261	305	339
10Mins	109	139	169	187	214	249	277
20Mins	82.2	104	126	139	158	184	204
30Mins	67.6	85.6	103	114	130	151	167
1Hr	45.4	57.5	69.3	76.5	86.9	101	112
2Hrs	28.2	35.7	42.9	47.3	53.6	62.1	68.8
3Hrs	20.8	26.3	31.6	34.7	39.3	45.5	50.3
6Hrs	12.2	15.4	18.4	20.3	22.9	26.5	29.3
12Hrs	7.31	9.26	11.2	12.3	14.0	16.2	18.0
24Hrs	4.57	5.85	7.25	8.12	9.34	11.0	12.3
48Hrs	2.87	3.73	4.83	5.55	6.52	7.87	8.97
72Hrs	2.08	2.72	3.61	4.20	4.98	6.10	7.02

(Raw data: 59.85, 9.37, 2.8, 97.03, 15.07, 5.68, skew=0.36, F2=4.34, F50=18.5) © Australian Government, Bureau of Meteorology

Copy Table

Figure 6 Intensity Frequency Duration Table

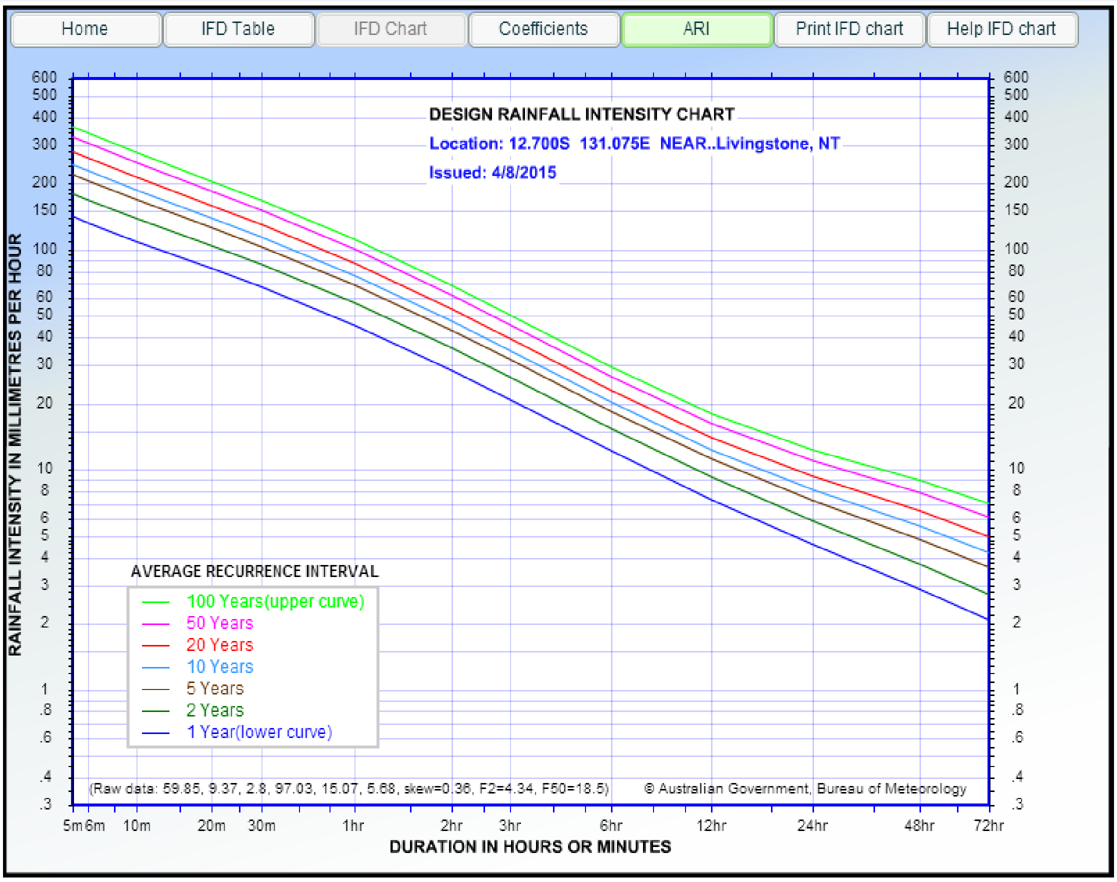


Figure 7 Design Rainfall Intensity

The channels formed by the catch drains and main drains are to be trapezoidal in cross-section. The bed width of the channel is usually determined by factors such as the operating width of the machinery cleaning and maintaining the drain. Using the peak volumetric flow rate determined above, the design dimensions of catch drains and main drains can be determined by solving for drain flow depth (*d*) the following equation (Skerman, 2000) derived from Manning's formula:

Equation 3 Manning's formula

$$Q_y = \frac{\left[W \times d + \dfrac{d^2 \times (z_1 + z_2)}{2}\right]^{5/3} \times S^{1/2}}{\left[W + d \times \left(\sqrt{1+z_1^2} + \sqrt{1+z_2^2}\right)\right]^{2/3} \times n}$$

where Qy = volumetric peak flow (m³/s) having an ARI of y years,
W = drain bed width (m),
d = drain flow depth (m),
z1 and z2 are the batter grades (1: z horizontal) of the channel sides,
S = gradient of the channel bed slope (m/m), and
n = a Manning's roughness coefficient.

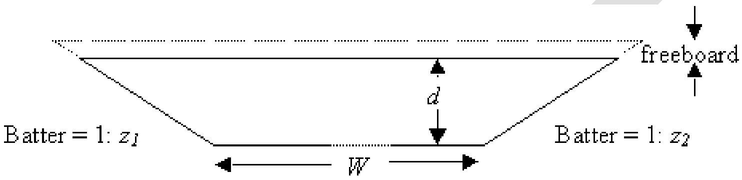
In the proposed development the catch drains are to function jointly as both drains and laneways for moving cattle to and from the pens. Accordingly, the drains will be lined with compacted gravel to provide a suitably durable surface for the dual purposes. Despite this form of lining, a conservative value for the Manning's roughness coefficient of 0.04 (Shaw, 1994; Skerman, 2000 & Loughlin & Robinson, 2001) should be applied to cater for any vegetative growth that might occur between cleaning operations in the drains.

Having established the drain flow depth it is then necessary to determine the flow velocity (*V*) in the channel using the following equation (Skerman, 2000) again derived from Manning's formula:

Equation 4 Manning's formula

$$V = \left[\frac{W \times d + \dfrac{d^2 \times (z_1 + z_2)}{2}}{W + d \times \left(\sqrt{1 + z_1^2} + \sqrt{1 + z_2^2} \right)} \right]^{2/3} \times \frac{S^{1/2}}{n}$$

The resultant design velocity should then be compared against tabulated maximum permissible flow velocities (Schwab *et al*., 1971 and Skerman, 2000) for similarly lined channels of various bed slopes. For channels lined with compacted gravel and having a bed slope gradient of 1 – 2%, a suitable maximum design velocity is 0.6 m/s. If the estimated design velocity exceeds the allowable maximum further iterations of the above calculations (Equations 3 and 4) can be undertaken using successively larger design widths for the drain bed until such time as the above design criteria are satisfied.



Freeboard

Where embankments are necessary to form the drains (eg irrigation tailwater drains), they need to be constructed to provide allowances for freeboard, settlement and minor undulations in addition to the calculated maximum drain flow depth. The degree of settlement will depend on soil type and the degree of compaction provided by construction equipment but can represent up to 20 to 25% reduction in finished embankment height (Skerman, 2000). An allowance of 0.15 m will normally account for undulations in most soils types (Skerman, 2000).

A suitable freeboard for ILEF drains is 0.5 metres (ICIAI, 1997). Side batter grades should less than 1:3 (ICIAI, 1997). Energy dissipaters may need to be placed where a catch drains terminate in the sediment basins and or main drain, so reducing the exit velocity from the channel (Lott, 1994). Design details for the catch and main drain are provided in Table 2.

Table 2 Design details of the ILEF pen catchments, ILEF main drains and Tailwater Drain

Parameter		Catch drain (Single open row)	CDA Main drain (all rows and pens)	Access Road Drain	North/South Main Drain	Upstream Tailwater Drain	Downstream Tailwater Drain
Mainstream length	L	0.147km	0.520km	0.351 km	1.153 km	1.153 km	1.03 km
Catchment area	A	0.0556 km²	0.0609 km²	0.119 km²	0.386 km²	0.506km²	0.621 km²
Equal area slope	S_e	13.4m/km	10.7m/km	10.25 m/km	12.0 m/km	12.0 m/km	8.1 m/km
Time of concentration	t_c	11.8 min	13.5min	15.8 min	44.7 min	44.7 min	44.7 min
Rational method formula							
Runoff coefficient	C	0.8	0.8	0.8	0.8	0.8	0.8
Rainfall intensity (20 yr ARI)	${}^{y}I_t$	197.7 mm/h	175 mm/h	176.1 mm/h	104.1 mm/h	104.1 mm/h	104.1 mm/h
Peak volumetric flow	Q_y	2.44 m³/s	3.23 m³/s	4.10 m³/s	8.94 m³/s	13.04 m³/s	15.58 m³/s
Manning's formula							
Lining	Material	Gravel	Gravel	Grass	Grass	Grass	Grass
Channel bed width	W	3 m	3 m	3 m	3 m	3 m	3 m
Upslope batter grade	z_1	0.025	0.33	0.25	0.25	0.25	0.25
Downslope batter grade	z_2	0.04	0.33	0.25	0.25	0.25	0.25
Channel bed gradient	S	0.015 m/m	0.011 m/m	0.005 m/m	0.0036 m/m	0.008 m/m	0.008 m/m
Manning's roughness coefficient	n	0.025	0.025	0.033	0.040	0.040	0.040
Channel flow depth	d	0.27 m	0.31 m	1.13 m	1.05 m	1.05 m	1.15 m
Channel flow velocity	v	0.928 m/s	1.755 m/s	0.485 m/s	1.183 m/s	1.725 m/s	1.783 m/s
Embankment height	$d + 0.5$	0.77 m	0.81 m	1.53 m	1.55 m	1.55 m	1.65 m

Sedimentation System Capacities

Sedimentation systems may be designed in the form of terraces, basins or ponds. These system types differ in respect to their aspect ratios and depth. Sedimentation terraces are shallow, relatively elongated structures with aspect ratios (*L*/*W*) of between 8:1 and 10:1. Sedimentation basins and ponds typically have similar aspect ratios (*L*/*W*) of between 2:1 and 3:1 but basins shallower (<1.5 m in depth) than ponds (>1.5 m in depth).

Both sedimentation terraces and basins are designed to drain freely after each runoff event so allowing the collected solids to be dried and removed at frequent intervals. Sedimentation ponds are designed to allow solids from a series of runoff events to accumulate with decanting of the captured solids typically occurring at intervals of one to five years. A scaling factor (λ) is applied to the design volume to account for the storage capacity required to store the solids captured in the various types of sedimentation system between decanting or cleaning operations. The required volume of sedimentation systems can be estimated using the formula provided by SCARM (1997) given by:

Equation 5 required volume of sedimentation systems

$$V = Q_y \frac{L}{W} . \frac{\lambda}{v}$$

where V = sedimentation system volume (m³),
Qy = volumetric peak flow (m³/s) having an ARI of y years,
L/W = aspect ratio of the system,
λ = a scaling factor, and
v = maximum design flow velocity (0.005 m/s).

The choice of sedimentation terrace, basin or pond is dependent upon factors such as available land areas, site topography and climate as well as the proximity of neighbours and other potential odour receptors. In this case the site of the proposed development *lends itself to the use of a sedimentation terraces* configured in a manner where terraces are concertinaed into joined three sections. A concrete lined terrace will be used so sediments can be recovered through the wet season.

The design details for the terrace are provided in Table 3. In a sedimentation terrace the aspect ratio (*L*/*W*) is typically around 10:1 while the applicable scaling factor (λ) can be 1 (SCARM, 1997). The embankments of the sedimentation terrace need to be constructed to provide an allowance for both freeboard and settlement. The minimum operational freeboard is 0.9 metres (ICIAI, 1997).

Table 3 Design details of sedimentation terrace

Parameter	Unit	Small Sediment Basin	Main Sediment Basin
Sedimentation terrace formula			
Peak volumetric flow	Q_y	2.44 m³	4.38 m³
Aspect ratio	L/W	10	10
Scaling factor	λ	1	1
Maximum flow velocity	v	0.005 m/s	0.005 m/s
Design volume	V	4,888m³	8,760 m³
Surface area	A	4,888 m²	8,760 m²
Depth	d	1.0 m	1.0 m
Minimum freeboard		0.9 m	0.9 m

Sediment Terrace Discharge: Weir and Channels

Runoff discharged from the sedimentation terraces need to be released in a controlled manner by way of weirs. The required dimensions of the weirs should able to accommodate the peak volumetric flow from a 20 year ARI design storm (MLA, 2012) and can be estimated iteratively by solving for weir crest length (*b*) and hydraulic head (*H*) the following equation (Isrealsen & Hansen, 1962; Shaw, 1994 and Jenkins, 2001):

Equation 6 determination of dimensions of a weir for peak volumetric flow - 20 year ARI storm

$$Q_y = C_d.bH^{3/2}$$

where Qy = volumetric peak flow (m³/s) having an ARI of y years,
Cd = a discharge coefficient,
b = weir crest length (m), and
H = hydraulic head of the approach flow (m).

A broad crested weir discharge coefficient (C_d) of 1.7, obtained from published values (Isrealsen & Hansen, 1962; Shaw, 1994 and Jenkins, 2001), can be considered suitable for preliminary design work such as this. The design dimensions for the weir are provided in Table 4.

Table 4 Design details for sedimentation pond weir

Parameter		Small Sediment Basin	Main Sediment Basin
Weir formula			
Peak volumetric flow	Q_y	2.44 m³/s	4.38 m³/s
Discharge coefficient	C_d	1.7	1.7
Weir crest length	b	5.2 m	9.4 m
Head of approach flow	H	0.5 m	0.5 m
Mean flow velocity	v	0.005 m/s	0.005 m/s

To reduce the likelihood of the weir being submerged due to subcritical flow conditions at the peak design discharge, the depth of the flow in the downstream channel at that time should generally be less than 80% or the hydraulic head of the approach flow in the sedimentation system (Isrealsen & Hansen, 1962 and WA Main Roads, 2004) and the spillway channel bed gradient should be greater than 0.5% (Schwab *et al*., 1971).

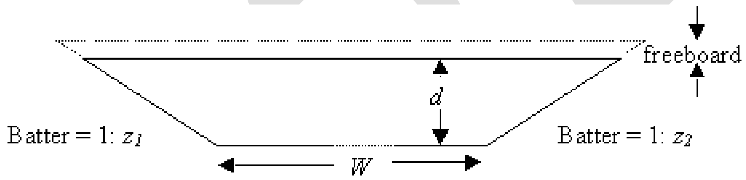
Any part of the spillway below the weir likely to be exposed to supercritical flows under ordinary conditions (less than a 20 year ARI design storm) will need a concrete or masonry lining applied. Some form of energy dissipater or stilling basin may also be necessary.

More detailed engineering design for the weir and spillway structures (based on unit hydrographs) will be undertaken as part of the detailed design work to be carried out prior to construction of the ILEF.

The spillway channel below the weir is to be trapezoidal in cross section and vegetated (mown grass). A Manning's roughness coefficient of 0.04 (Shaw, 1994 and Loughlin & Robinson, 2001) and maximum permissible flow velocity of 1.5 m/s (ICIAI, 1997 and Skerman, 2001) are applicable in this instance. The requirement for the spillway channel allowing critical or supercritical flow over the weir under normal flow conditions is an additional design criteria. Using these criteria the design dimensions and design flow velocity in the spillway channel can be estimated iteratively using Equations 3 and 4 and are provided in Table 5.

Table 5 Design details for sedimentation pond spillway channel (Concrete lined)

Peak volumetric flow	Q_y	2.44 m^3/s	4.38 m^3/s
Manning's formula (F.3 & F.4)			
Channel bed width	W	3 m	3 m
Batter grade (1)	z_1	0.33	0.33
Batter grade (2)	z_2	0.33	0.33
Channel bed gradient	S	0.025 m/m	0.025 m/m
Manning's roughness coefficient	n	0.013	0.013
Channel flow depth	d	0.36 m	0.5 m
Mean flow velocity	v	1.66 m/s	1.95 m/s



2.1.3 Primary Wastewater Pond

The design principles of ILEF holding ponds (referred to as the Primary Wastewater Pond) are discussed in detail in Lott (1994), ICIAI (1997), SCARM (1997), Lott (1998) and Skerman (2000).

The principal design function of holding ponds is to store ILEF runoff until such time as the pond effluent can be safely used for irrigating the wastewater utilisation area. Depending on the time for which the runoff is stored in the holding pond, microbial degradation (principally anaerobic) of the entrained organic matter may occur, a portion of any mineralised nitrogen may be lost to volatilisation and denitrification processes and a proportion of the water will be lost to evaporation (Lott, 1994 and ICIAI, 1997). Some sludge build-up may also occur through settlement of the entrained solids (Lott, 1994).

Until comparatively recent times, a commonly utilised approach to holding pond design was to treat the holding ponds as short-term retention systems. The applicable design criteria were for the pond to be capable of retaining the runoff from a major storm event (1 in 20 year 24 hour storm). Typically runoff coefficients of 0.8 were used for ILEF pens, laneways and hardstand areas and 0.4 for grassed areas (ICIAI, 1997; SCARM, 1997; MLA, 2012). The required storage volume using the "major storm" concept can be determined using the following relationship:

Equation 7 Holding pond formula

$$V = \left[\left(A_h \times C_h\right) + \left(A_s \times C_s\right)\right] \times {}^{y}I_t / 100$$

where V = required storage volume (ML),

${}^{y}I_t$ = rainfall intensity (mm/h) of design storm having duration t_c,
A_h = area of "hard" catchment (ha),
C_h = a hard catchment runoff coefficient,
A_s = area of "soft" catchment (ha), and
C_s = a soft catchment runoff coefficient.

An estimate of the required storage volume in the holding pond as determined using the "major storm" approach is shown in Table 6.

Table 6 Design details of holding pond (major storm event approach)

Parameter		Value
Holding pond formula		
1 in 20 yr 24 hour storm	${}^{y}I_t$	9.34mm/hr
"Hard" catchment area	A_h	27.7ha
Runoff coefficient	C_h	0.8
"Soft" catchment	A_s	5.27 ha
Runoff coefficient	C_s	0.4
Storage volume	V	22.35 ML

The "major storm" design concept used above is based on the premise that holding ponds are only used for the short-term storage of runoff and that the pond contents can be fully utilised in the wastewater utilisation area between significant rainfall events. Unfortunately, major rainfall events are often associated with episodic periods of wetter than normal weather and seasonal and climatic factors may necessitate the long-term storage of runoff until such time as it can be safely assimilated in the wastewater utilisation area. Further, sludge build-up may also reduce the effective storage volume. Consequently, holding ponds designed on the above basis have often been found to have an unacceptably high frequency of "spill" or overflow events (more than an average of once every 10 years) due to the effective storage capacity being insufficient to accommodate the accumulated runoff in a 90 percentile wet year (Lott, 1998).

A more robust alternative to the major storm event approach is that carried out by undertaking a water balance for the entire ILEF catchment (controlled drainage area and wastewater utilisation area). This water balance needs to be modelled on at least a monthly step basis using site representative metrological data. Using this approach the required storage volume is that capable of preventing the holding pond from overflowing in a 90 percentile wet year (ICIAI, 1997 and SCARM, 1997). In determining this capacity consideration also needs to be given for the storage of accumulated solids.

The FSIM model (Lott, 1998) is a daily step model developed specifically for ILEF catchments. The FSIM model simulates the material balance of both water and nutrients within a ILEF catchment using distributed parameters to describe the relevant system processes. Catchment hydrology is modelled using separate algorithms for pen areas, "hard" surfaces such as roadways and "soft", largely vegetated surfaces. The algorithms have been validated against standard methodologies used for catchment hydrology calculations (USDA, 1971 and Pilgrim, 1987). Model output has been verified by comparison with comprehensive hydrological measurements made in four catchments within three ILEFs in southern Queensland. The design capacity of the holding ponds as determined using the FSIM model is detailed in Section 3 of this report.

Holding Pond Spillway

Irrespective of the design concept used, any holding pond is likely to spill or overflow following extraordinary rainfall events. Current guidelines (ICIAI, 1997 and SCARM, 1997) stipulate that the holding pond spillways be designed to handle a 1 in 50 year design storm. The volumetric peak flow resulting from a 50 year ARI design storm can be calculated using Equations 1 and 2. The design values determined using these equations are provided in Table 7.

Table 7 Peak volumetric peak flow (Q_p) from a 1 in 50 year design storm

Parameter		Value
Bransby Williams formula (F.1)		
Mainstream length	L	0.450 km
Catchment area	A	0.3294 km²
Equal area slope	Se	15.5 m/km
Time of concentration	tc	16.86 min
Rational method formula (F.2)		
Runoff coefficient	C	0.8
Rainfall intensity (50 yr ARI)	${}^{y}I_{t}$	204.4 mm/h
Peak volumetric flow	Q_y	14.97 m³/s

The overflow from the primary holding pond needs to be released in a controlled manner by way of a weir. The required dimensions of a weir able to accommodate the peak volumetric flow from a 50 year ARI design storm (ICIAI, 1997) can be estimated by solving iteratively for weir crest length (*b*) and hydraulic head (*H*) the weir formula given in Equation 6. The resultant design details are provided in Table 8.

Constraints exist in terms of avoiding submergence of the weir due to subcritical flows at the peak design discharge apply. The extent of the spillway channel likely to be subjected to supercritical flows will need to be lined with a concrete or masonry liner. Again, more detailed engineering design for the weir and spillway structures will be undertaken as part of the detailed design work to be carried out prior to construction of the ILEF.

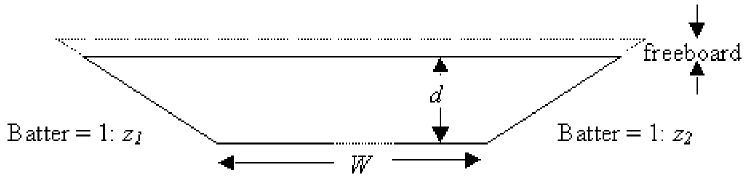
Table 8 Design details for Primary Pond Overflow Weir

Parameter		Value
Weir formula (F.6)		
Peak volumetric flow	Q_y	14.97 m³
Discharge coefficient	C_d	1.7
Weir crest length	b	25 m
Head of approach flow	H	0.5 m
Mean flow velocity	v	1.5 m/s

The required dimensions of a trapezoidal spillway channel to carry the 50 year ARI peak flow can be determined iteratively by solving for various channel bed width and depth the Manning's formula equations (3 & 4). The holding pond spillway channel below the weir is to be trapezoidal in cross section and vegetated (mown grass). A Manning's roughness coefficient of 0.04 (Shaw, 1994 and Loughlin & Robinson, 2001) and maximum permissible flow velocity of 1.5 m/s (ICIAI, 1997 and Skerman, 2001) are applicable in this instance. The requirement for the spillway channel allowing critical or supercritical flow over the weir under normal flow conditions is an additional design criteria. The resultant design dimensions and design flow velocity of the spillway channel are provided in Table 9.

Table 9 Design details for holding pond spillway channel

Peak volumetric flow	Q_y	Y m³/s
Manning's formula (F.3 & F.4)		
Channel bed width	W	25m
Batter grade (1)	z_1	0.25
Batter grade (2)	z_2	0. 25
Channel bed gradient	S	0.008 m/m
Manning's roughness coefficient	n	0.04
Channel flow depth	d	0.45 m
Mean flow velocity	v	1.241 m/s



2.2 Waste Water Irrigation Area

The runoff from a ILEF's controlled drainage area captured in the holding pond is to be irrigated on land adjacent to the ILEF complex where the nutrients and water can be utilised in plant production. The soil in the wastewater utilisation area provides a "sink" for the assimilation of applied nutrients.

The environmentally sustainable use of the wastewater utilisation area is directly related to the amount of nutrient applied to such areas, the amount of nutrient recovered in produce harvested or removed from the area and the amount of nutrient able to be safely stored in the soil. Some loss of nutrient (and salts) from the system will occur by way of leachate moving below the root zone of the crops and through processes such as erosive soil loss. It is also necessary for increased amounts of salt to be drained from the soil in the wastewater utilisation area by this means if salinization of the soil profile is to be avoided. This loss of nutrients and salts will not impact on the environmental value of any associated surface or groundwater resources.

Generally, one of the plant macronutrients (nitrogen, phosphorus or potassium), rather than either the hydraulic or the organic matter loading rate, is the limiting factor in determining the net annual application volume of wastewater in the utilisation area and, conversely, the required size of the utilisation area. The use of a source of "fresh" or "clean" irrigation water to supplement the applied wastewater will generally be necessary to help maximise crop yields and so maximise nutrient removal from the utilisation area. In the long term, rainfall, wastewater or irrigation water applications in excess of that utilised directly by the crops will be necessary to leach salts from the soil profile.

The amount and timing of both wastewater and fresh water applications will be largely determined by the irrigation requirement of the crops. In abnormally wet years or seasons, hydraulic loading may in the short term become the limiting factor on wastewater applications. Current guidelines (MLA, 2012) attempt to address this by stipulating that the wastewater utilisation area must be of sufficient size to allow wastewater irrigation in a 90 percentile wet year. Consistent with this, the FSIM model determines both the optimum size of the wastewater utilisation area and the optimum size of the holding pond necessary to provide sufficient storage capacity to safely store the wastewater in a 90 percentile wet year.

Under this proposal, irrigation of the wastewater utilisation area will be undertaken a large lateral move irrigator, a centre pivot irrigator and areas of drip. When properly designed and managed such irrigators generate a minimum of irrigation tailwater. Nevertheless, the potential does exist where a significant storm event occurs during or immediately after a wastewater irrigation application for stormwater runoff from the wastewater utilisation area to transport unacceptable amounts of nutrient and other potential contaminants off-site. Consequently, it is necessary to employ a terminal system to capture and recycle stormwater runoff from the wastewater utilisation area.

Current guidelines (ICIAI, 1997 and SCARM, 1997) stipulate that the terminal system must be capable of capturing and storing the runoff equivalent to a minimum of 12 mm over the entire wastewater utilisation area.

In consideration of the site to the Berri Springs Creek catchment, the ILEF design has additional storage capacity in the terminal system. It has a capacity of about 30ML. This storage has a capacity equivalent to greater than 75 mm of stormwater runoff from the 40 hectare wastewater utilisation area (about 6 times the guideline requirement). The same storage will collect some of the clean water runoff from other areas of the site. These will dilute tail-waters from the irrigable areas.

The required capacity of the terminal system can determined using the following equation (ICIAI, 1997; SCARM, 1997; MLA, 2012 and Skerman, 2001):

Equation 8 Terminal system formula

$$V = a + b$$

where V = volume of terminal system (m³),
a = irrigation tailwater (m³), and
b = stormwater runoff from the irrigation area (m³).

The design capacity of the terminal system determined using this approach is shown in Table 10.

Table 10 Design details for terminal system

Parameter		Min Value (approx.)
Terminal system formula		
Irrigation tailwater	a	0 m³
Rainfall runoff	b	20,000m³
Terminal system capacity	V	20,000m³

The total 20 ML design capacity of the terminal system would provide by a 30 ML tailwater pond to the northwest of the irrigation area. Runoff collected in the tailwater system would be recycled through the irrigation system. The FSIM model (Section 3) is able to accommodate this recycling in its water balance calculations.

Despite the proposed terminal system capacity, the tailwater ponds are liable to overflow more frequently than other components of the wastewater management system. Accordingly, the pond spillways should be designed to accommodate the runoff from at least a 1 in 20 year design storm for the wastewater utilisation area catchments (ICIAI, 1997 and SCARM, 1997).

The terminal pond will be largely cut below the natural surface. This will eliminate any prospect of catastrophic embankment failure. The by-wash and weir must be capable of handling a 1 in 50 year design storm. The by-wash is 25m and is cut on the lowest point on the terminal pond so water discharged to the natural flow line.

3. Hydrology of the Proposed ILEF Using the FSIM model

3.1 Introduction

The FSIM model (Lott, 1995 and Lott, 1998) simulates the hydrological mass balance of a ILEF complex with particular emphasis on the water balance of the pen surface. The model uses distributed parameters to describe the various aspects of the hydrological balance and has been developed to incorporate variables for factors such as land use and ILEF management practices.

Long-term daily climate data (precipitation and evaporation) for the site or a site representative station is a basic requirement. Output is in various forms and can be tailored to investigate the specific factors influencing the hydrology of the ILEF catchment. The model was developed using hydrological data collected in commercial ILEFs. The FSIM model has been subsequently calibrated and the accuracy of its predictions of catchment conditions and rainfall runoff in ILEF catchments has been verified and tested (Lott, 1998). The research data and model was used to derive the co-efficient used in the current State and National feedlot guidelines (MLA, 2012).

The FSIM model was used to simulate the hydrological performance of the Wellard Darwin ILEF catchment including the holding pond and effluent utilisation area. This section of report discusses the principles underlying the FSIM model, the input data used in the model and presents the output predictions, comparing and contrasting them with those provided in the previous sections of the report.

3.2 Climatic Data

3.2.1 Data Requirements

The climate data required for a FSIM simulation are precipitation, temperature, humidity, radiation, and potential evaporation.

3.2.2 Evaporation data

Evaporation can be demonstrated to be the most important climatic variable influencing the hydrological performance of the ILEF catchment, holding pond and wastewater utilisation area. To reliably model the hydrology of an ILEF, it is necessary to estimate, on a daily basis, the direct evaporation from the surface of the ILEF pen and the holding pond as well as the evapotranspiration from the wastewater utilisation area (Lott, 1998).

Lott & Skerman (1995) found hydrological balances based on daily variable evaporation estimates varied significantly from those based on monthly mean data with the two estimates of net annual evaporation varying by up to 30%. This has significant implications when issues such as the frequency of spill or overflow events are considered. Consequently, daily variable data is the preferred input for the FSIM model and should be used in preference to monthly mean data where available.

3.2.3 Climate Datasets

The site of the proposed development is located about 40 kilometres south of Darwin in the Northern Territory. The longitude and latitude of the site are respectively 12.70080°E and 131.08106°S.

Detailed climatic data are available for Bureau of Meteorology Darwin Airport (AP) some 30km to the north and at Berry Springs about 8 km from the site. Its data is similar to Darwin AP. Appendix B provides a summary of the Darwin climate.

The precipitation datasets for the other stations generally cover 100 years or more and are of reasonable quality (>99% original data & <1% patched data for missing values).

Given the intrinsic variability associated with climatic data, the length of the historical record used in the modelling of feedlot hydrology is an important consideration in determining the confidence that can be placed in modelled outcomes. This is particularly the case in determining the size of a holding pond and predicting the frequency of "spill" or overflow events.

Data records longer than 30 years are generally required to model spill events where the design criteria is a spill frequency less than one of 1 in 10 years. Ideally, more than 50 years of historical climate data should be used if available. To provide an acceptable level of accuracy and precision as well as conservative modelled outcomes representative of the development site, a composite meteorological dataset using precipitation data for 1889 through to 2015 was compiled for use in the FSIM modelling. The 126 year dataset was obtained from the Silo enhanced climate database hosted by the Science Delivery Division of the Department of Science, Information Technology and Innovation (DSITI), August 2015).

3.2.4 Runoff and Water Balance of a Manure Covered Pen

The accumulated manure (faeces and urine) on the surface of the ILEF pens acts as a significant store of water in the water balance a pen area catchment. The characteristics of the manure also influence the volume of runoff from rainfall events, the amount of nutrients and organic matter entrained on the runoff and the amount of odour generated.

The mass of faeces voided by cattle each day is typically equivalent to between 5 and 6% of the body weight of the animals and has a wet basis moisture content greater than 80%. Voided urine typically constitutes around 30% of the manure produced each day. In contrast manure can be air dried to a wet basis moisture content of around 6% (Watts *et al*., 1994 and Sweeten & Lott, 1994). Given an average bulk density of 750 kg/m³, the above range of potential moisture contents equates to a capacity for 100 mm of dry, compacted manure to store up to 280 mm of water. Storage of this amount of water would be associated with substantial expansion of particulate matter in the manure and water filling all the voids between the manure particles such that 100 mm of dry manure would become 300 mm of wet manure.

Due primarily to compaction resulting from cattle trampling the manure pad on the surface of the pen, the manure develops a stratified profile that is generally found to consist of up to three layers.

Immediately above the soil surface in the pen, an interface layer 25 to 50mm deep develops. This layer consists of organic matter from the manure mixed with the soil fabric. The trampling of cattle facilitates the mixing and compacts the manure and soil particles in this layer, so increasing the bulk density and reducing the hydraulic conductivity. Significantly, the manure is also a significant source of the monovalent cation forms of sodium and potassium. Overrepresentation of these cations on colloidal matter in this interface layer causes dispersion of the colloidal material. This exacerbates the compaction caused by the cattle trampling the manure, further increasing bulk density and reducing hydraulic conductivity. In addition, microbial decomposition of the organic matter releases complex carbohydrates and organic molecules that fill the voids between particulate matter and occlude pores increasing bulk density and further reducing hydraulic conductivity. The net result of these influences is that this interface layer usually has a bulk density of between 1,000 and 1,700 kg/m³. By comparison, the manure above this layer typically has a bulk density of between 750 and 930 kg/m³ while the underlying soil may have a bulk density of 1,400 to 1,600 kg/m³ (Lott, 1998).

The hydraulic conductivity of the interface layer has been found to be in the range of 5×10^{-13} m/s and 3×10^{-12} m/s (Walker *et al*., 1979 and Southcott & Lott, 1997). Consistent with this Mazurak (1976), in a study undertaken in a Nebraska feedlot, found the hydraulic conductivity of the interface layer to be less than 4% that of the soil 100 mm deeper. These characteristics mean that the interface layer can be considered to effectively provide a barrier to water in the manure pad infiltrating into the underlying soil profile. Similarly, the interface barrier also prevents water borne pollutants directly entering the soil profile from the manure pad (Lott, 1998).

The condition of the manure above the interface layer varies with time and is dependent on factors such as rainfall, evaporation, stocking density, cattle trampling (which has different effects depending on the moisture content) and the manure management practices of feedlot pens. Lott (1994 & 1998) found that the condition of this manure could be reliably classified using one of the following descriptions:

(1) Powdery-smooth-dry,
(2) Smooth-compact-moist,
(3) Rough-wet ("puggy"), and

(4) Smooth-saturated.

Each condition depends on manure moisture content and mechanical disturbance of the surface manure by cattle movement. Importantly:

- Maximum runoff occurs in conditions 2 and 4,
- Maximum sediment erosion occurs in conditions 1 and 4,
- Maximum odour nuisance and least runoff occurs in condition 3, and
- Minimum odour and maximum runoff occurs in condition 2.

The rainfall-runoff relationship of surface of a manure covered pen is discussed in detail in Lott (1998). Figure 8 shows the conceptual water balance of a pen. It accounts for the gains and losses of moisture by the pen surface. The manure on the pen surface represents a store of water and its characteristics (slope and roughness) may influence its water balance and the rainfall-runoff process from the pen surface. The parameters of interest, when understanding the water balance of the manure are:

- Stored water,
- Infiltration,
- Depression storage,
- Temporary storage),
- Evaporation, and
- Surface runoff.

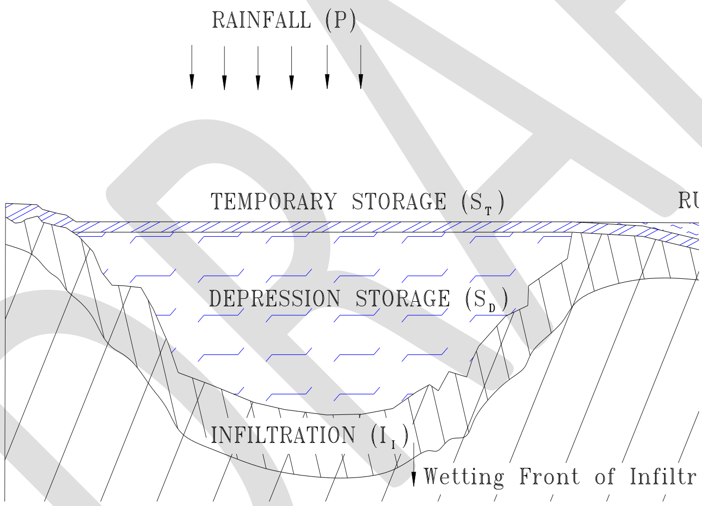


Figure 8 Conceptual model of the water balance of the cattle pen surface

The volume of runoff from a pen can be quantified by the following relationship (Lott, 1998):

Equation 9 Volume of Runoff

$$R = P - S_T - S_D - I - S_W - I_P$$

where R = runoff,
P = precipitation,
S_T = temporary storage,
S_D = depression storage,
I = infiltration,
S_W = stored water, and
I_P = percolation below the zone of stored water.

The FSIM model uses the four pen conditions (1 – 4) described above to characterise pen surface storage (*S*) and infiltration (*I*) in the above relationship.

A factor significantly impacting on the above relationship is the amount of water added to manure by the cattle (Watts, 1991). Cattle excrete faeces and urine that, when combined, have a mass equivalent to 5-6 % of the animal's body weight. It is anticipated that the mean live weight of cattle in the proposed ILEF will be about 450kg. An animal of this size can be estimated to produce about 25kg of manure (faeces and urine) per day. Of this, 28 kg is water and 4kg is dry matter. As a consequence, the amount of manure-derived water deposited on the pen surface can be seen to be dependent on the stocking density and the live-weight of the stock (refer Figure 9). The stocking density in the proposed PEQ yard is 5m2 per SCU and in the feedlot it is to be 10 m² per SCU. FSIM incorporates these additions when undertaking its daily step estimate of the pen surface water balance.

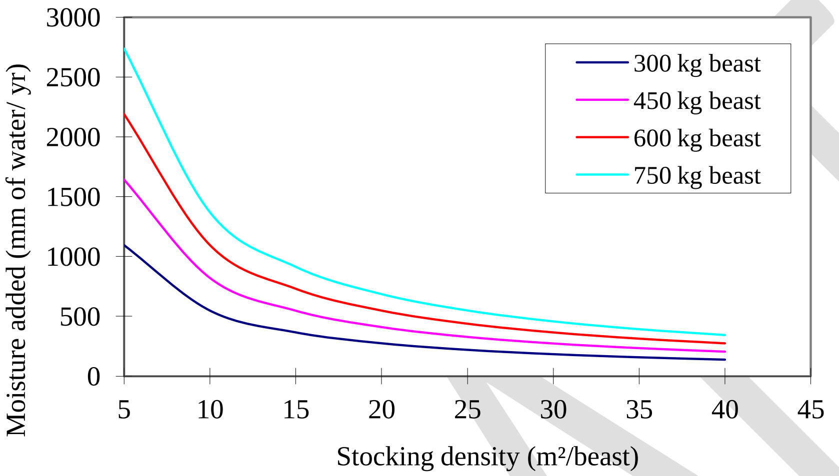


Figure 9 Moisture added to manure pad at various stocking densities of cattle of various live-weights (Sweeten & Lott, 1994)

The capacity of the pen surface to absorb water can vary with manure depth, manure condition and the gradient of the manure pad.

Empirically derived data on the rate of manure accumulation (Watts *et al*., 1994) is shown in Figure 10. At the proposed stocking density of 10 m² per head, the estimated rate of manure accumulation using this relationship is 130 mm/yr or 0.36mm/day (dry compact manure in pen condition 2). The amount of water able to be stored in the manure pad (S_W) increases with the mass of manure present on the pad. FSIM takes the accumulated depth of manure and its condition into consideration in undertaking the calculations for the pen surface water balance.

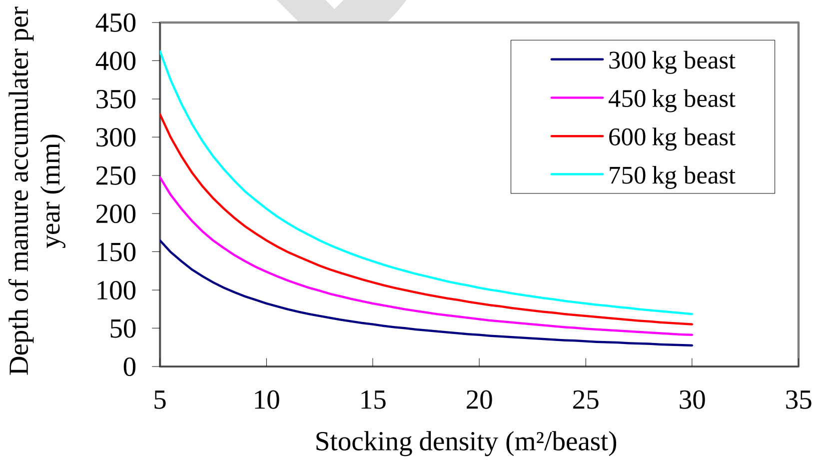


Figure 10 Depth of manure accumulated annually at various stocking densities of cattle of various live-weights (Watts et al., 1994)

Based on the above, at any given stocking density the amount of manure in the pens will be a function of the time since cleaning and the frequency of cleaning. This has implications for the condition of the manure in the pens. Lott (1998) found that compared to the regular intermittent cleaning of all of the pens, the continuous sequential cleaning of the pens reduces the incidence, extent and duration of pen condition 3 in the pen areas and the catchment that contributes to runoff to the waste water systems.

The depth of the manure and the moisture influences the amount of rainfall retained on the surface (S_T) and in depressions (S_D) in manure surface (Lott, 1994). This effect is not consistent across all pen conditions with manure depth being a significant influence on surface detention with pen condition 3 but not 1, 2 and 4. Condition3 is typified by “puggy” conditions where indentations made by the hooves of the cattle are more likely to form and persist on the surface of manure. Surface detention is also influenced by the gradient of the pen slope (Lott, 1994). However, this effect is least for pen condition 2 and 4.

Considering the above, it is evident that runoff from pens is determined by a multifarious relationship between factors as diverse as the live-weight of the cattle, climate, stocking density, the pen cleaning frequency and pen slope. A model such a FSIM allows all these factors to be integrated into the estimates of runoff from the cattle holding pens, so enhancing the precision and accuracy of the modelled outcomes.

It is important to note that most animals will be “empty” when they arrive and as such they are likely to void substantially less manure than shown in Figures 3 and 4. Consequently the FSIM model undertaken will be very conservative with regard to manure accumulations and characteristics.

3.3 Runoff from Other Land-Uses in the ILEF

While the ILEF open cattle holding pens have the most variable runoff yield and comprise the largest land-use in the ILEF controlled drainage area, other portions of the catchment can contribute significant amounts of runoff and have a significant effect on the hydrology of the catchment, the wastewater holding ponds and the irrigation areas.

Roadways, laneways and other hard stands generate significant runoff. After an initial abstraction of around 5mm, the remainder of a rainfall event can be considered to contribute directly to runoff from these areas. Similarly, an initial abstraction of around 7 mm can be expected for the drainage system within the controlled drainage area (Lott, 1998).

Harvested manure stockpiled in windrows has the capacity to store a substantial amount of rainfall. Lott (1998) found that an initial abstraction of around 25 mm was a reasonable approximation for windrowed manure. Hardstand areas with a compacted cover between the windrows can be expected to provide an initial abstraction of around 7 mm.

The runoff from grassed (“soft”) areas and vegetated waterways within the catchment are able to be reliably determined using the approach used in USSCS model (USDA, 1971). This model assigns “k” values based on catchment condition and antecedent rainfall to the various areas within the catchment. These k values are then used to estimate runoff based on daily precipitation data.

3.4 FSIM Modelling

3.4.1 Input data

The vales used for the major input variables in the FSIM model are provided in Table 11. The data for the parameters were either design values discussed elsewhere in the Environmental Assessment or derived from comparable production data for feedlots elsewhere. Catchment areas were estimated by scaling off the CAD drawing provided as Figure 4.

Table 11 Values for major input variables in the FSIM ILEF hydrology model

Parameter	Value
ILEF	
ILEF capacity	3850 head
Occupancy	80%
Mortality rate	0.3%
Market type	Export
Entry liveweight	250 kg
Exit liveweight	550 kg
Feeding period	6 days (Av)
Liveweight gain	0.2kg/d
Dry matter intake	2.8% liveweight
Pen capacity	150 head
Stocking density	5-10 m^2/head
Pen width	30 m
Pen depth	50 m
Pen slope	0.025 m/m
ILEF class	1
Maximum manure depth	50 mm
Cleaning frequency	4 times/yr
Catchment characteristics	
Area ILEF drains	20839m^2
Initial loss drains	7mm
Area roadways	28589 m^2
Initial loss roadways	5mm
Internal CDA waterways	1289 m^2
Grassed waterway K_1 K_2 K_3 values	35, 45, 55
Area grass	24903 m^2
Grass K_1 K_2 K_3 values	35, 45, 55
Manure stockpile area	30591 m^2
Manure bulk density	900 kg/m^3
Maximum stockpile height	10 m
Initial loss stockpile	25%
Initial loss pavement	7%
Primary wastewater pond maximum surface area	37445 m^2
Primary wastewater pond maximum depth	8 m
Wet weather storage pond maximum surface area	25569 m^2
Wet weather storage pond maximum depth	8m
Freshwater runoff dam maximum surface area	15923 m^2
Freshwater runoff dam maximum depth	5 m

3.4.2 ILEF Runoff and primary treatment pond and wet weather storage pond Capacity

The design of the ILEF includes a primary treatment pond and a wet weather storage pond. These combine to deliver a holding pond capacity. Using an iterative approach, numerous runs of the model were performed to derive an optimum design capacity for the holding pond able to satisfy the design criteria of overflowing or “spilling” at a frequency less than once every 10 years.

Iterations using a the 126 year composite dataset discussed in Section 3.2.3 (page 20) found that the optimum capacity was found to be a total of about 220 ML.

With this configuration of holding pond volume and surface area, twelve spill events were predicted occur the 126 year period of 1889 through to 2015 (less than 1 spill per 10 years)

The spill events are aligned with very significant wet seasons and major events; notably flood events of 1956, 1974 (per Cyclone Tracy), 1976, 1998 and 2011.

The volume of wastewater stored in the primary treatment pond and wet weather storage pond each day over a 115 year runtime from 1990 to 2015 of the simulation is shown in Figure 11 together with the spill events.

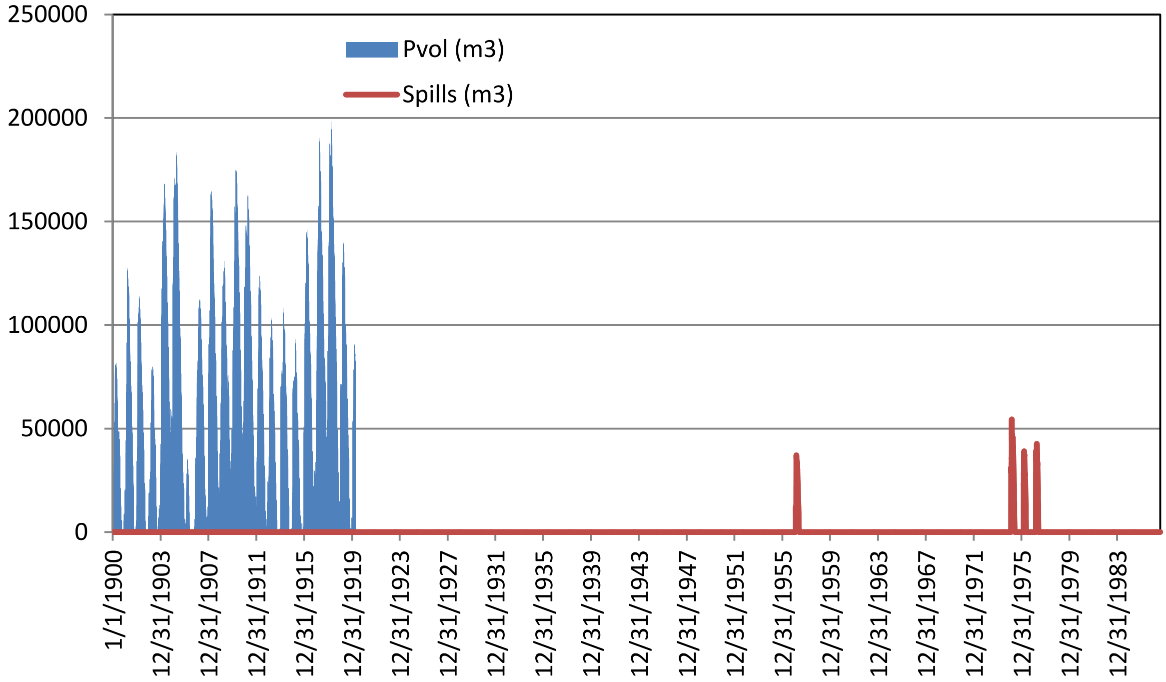


Figure 11 Volume of wastewater stored in the primary treatment pond and wet weather storage pond between 1900 to date in 2015

The estimated magnitude of the spills sensitive to the irrigation regime and, in particular, the amount of clean water allocated for irrigation. The model assumes that 100ML of clean water is available for irrigation; obviously this water would not be used in the situations where there is a significant wet season so the modelled results are somewhat conservative.

It is further noted that the spills are routed through the tailwater pond where significant dilution would occur. The routing would take place from the primary pond to the tailwater pond; most contaminated water would have been transferred to the holding pond at the start of any major event.

At the time of these spills the entire catchment is in a major flood. Any release from the site would pass into the easement next to the railway line, under the railway line, through a natural grassed waterway, then enter the swamp. Water from multiple sources enters the swamp. Overflow from the swamp does pass into the Berri Springs Creek. At the time of the simulated flood events and given the flooding at the time the dilution factor is greater than one in a billion (that is 1 in >1,000,000,000: ML for ML).

Figure 11 shows that the holding pond configuration spends much of its time containing less than 50 ML. The loss of wastewater by evaporation will be limited because the holding pond capacity is developed by using a two cell design. The two cell holding pond configuration allows quantities of water to be held in the first cell (the primary treatment pond) with a resultant reduction in surface area. The second cell (the wet weather storage pond) acts mainly as wet weather storage, only filling at times of significant rainfall. In addition to minimising evaporative loss, the smaller effective surface area provides a smaller interfacial area between pond contents and the atmosphere, minimising odour emissions from this source.

3.4.3 Sizing the Wastewater Irrigation Area

The interrelationship between the size of the wastewater irrigation area, the cropping program and the nutrient balance is discussed in detail in Appendices H, Solid and Liquid Waste Management Plan, M, Soils Assessment, N, Geotechnical Assessment and Section 10.6 Land and Soil Resource of the Environmental Assessment report.

The FSIM model provides output that allows the calculation of the average annual yield of runoff from the ILEF and the mean annual volume of wastewater available to irrigate the waste utilisation area. These data have been used as input data into the nutrient balance. The results would suggest that an irrigation area cropped to improved pasture and tree irrigation would need to be 40 ha in size to enable wastewater applications to be sustainable from a nutrient balance viewpoint.

Existing experience in hay production at this site provides useful information concerning factors such as crop management, crop yields and irrigation scheduling and has been incorporated in the modelling undertaken. Values applied to the key variables used in the FSIM modelling of the irrigation area are provided in Table 12.

Table 12 Values used in FSIM modelling of the wastewater irrigation area

Parameter	Value
Depth of Root Zone	0.9 m
Water holding capacity	150 mm/m
Plant available water	40 %
Total irrigable area	40.5 ha
Irrigated house area	2.49 ha
Irrigated cropping area (improved pasture)	33.3 ha
Irrigated tree line	0.29 ha
Irrigated tree line area	2.4 ha
Clean water irrigation allocation	100 ML

The modelling was undertaken on the basis that approximately 40 hectares of land was available for wastewater irrigation. This is based on utilising a lateral move irrigation area of approximately 30ha, a centre pivot area of over 3ha and additional areas comprising over 5.5ha used for improved pasture and trees.

The lateral move irrigation and centre pivot irrigation areas of over 33ha will be sown to an improved pasture. It will be cut for silage production. Other irrigable areas will be planted to trees for development of tree lines and some areas surrounding the facilities which will be irrigated to promote grass growth; these will be slashed. It is estimated that irrigated improved pasture will have a gross water requirement of 10-13ML/year respectively.

The irrigable area is approximately 40ha. The annual average years is estimated to be about 190ML/year and the 1 in 10 year wet season yield is estimated to be about 240ML/year. Thus the irrigation application rate is estimated to be about 4.75-6ML/ha/year. This is less than the expected crop water requirement of 10ML/ha/year. Some clean water will be available but too will be limited. A crop water deficit is expect in late spring ("the build-up") and clean water will be held where ever possible to irrigate at this time to allow crop dry matter yields to be maximised.

This level of wastewater usage exceeds the design capacity of the holding pond reflecting the general rapidity with which wastewater is utilised after an inflow. This results in less long-term storage in the holding pond and as a consequence less loss of water by evaporation and less concentration of the wastewater constituents.

4. General Civil Design Attributes of the ILEF Runoff Control Structures

4.1 Pens and Drains

The pens will be constructed of crushed and compacted ferricrete. The material has been shown to have permeability less than 1 x 10-9m/s when compacted to > 98% compaction. This engineered surface will be essentially impermeable and resistant to traffic by cattle and machinery.

4.2 Sedimentation Basins

The sediment basins will be constructed of concrete. The design has been used elsewhere and had proven to be effective. The concrete basin can be accessed in wet weather. Solids recovered from the basins are placed on a pad that drains back to the sedimentation basin.

The sedimentation basins will be designed and constructed so they have two parts to their storage capacity; a working volume and then peak discharge detention volume. The weirs will be accordingly designed as "stepped weirs".

4.3 Holding Ponds

4.3.1 Primary wastewater pond, and wet weather storage dam

The storage capacity is made up of two ponds; a primary waste water pond and a wet weather storage dam. Water will be transferred from the primary pond to the wet weather storage by the pumping station. It has a high capacity lifting pump.

The Primary pond will be 3-8 m deep, with an additional 0.75 m dead/storage space. It is designed for rapid dewatering.

All ponds will have compacted clay lining.

Where required a HDPE synthetic liner placed around the inner batter to prevent vegetation growth. All ponds will possess batters with a 33 degree angle and include a crest that can be accessed by a body truck so that sludges can be removed using a vacuums pump and/or front end loader.

The primary treatment pond will be constructed so that it is cut below the natural surface and will have an embankment of about 2-3m above the surface. The pond lining will be further "reinforced" to prevent lining 'push out' by any subsurface flows. Further, to reduce the risk of structural failure of the inner embankment and floor, a rock armouring will be implemented to improve stability during periods of heightened transient groundwater flow. Compacted material under the clay liner will undergo stability treatment. A piezometer should be placed above (upslope and up hydraulic gradient) and below the pond to monitor shallow groundwater depth and quality and to function as an early warning leak detection system.

The wet weather storage will be constructed largely above ground and will be HDPE lined if suitable soils for construction and lining cannot be sourced..

4.3.2 Freshwater / Tailwater Dam

This dam only contains the runoff from the irrigable area and other clean waters. It has a design capacity of 30ML. The dam is excavated below ground level. The spillway is cut at the lowest edge of the terminal pond on level to the path of the existing waterway. It is 25m wide and has a zero grade. Water flows to the existing flow line which is the drain on the western side of the railway line and then through the culvers tinder the railway line.

5. Conclusion

The site of the ILEF is above any flood level. It is elevated.

The design dimensions of the sedimentation, holding and terminal ponds were compared using both the current National Feedlot guidelines (MLA, 2012) and the FSIM hydrology model (Lott, 1998). The FSIM design would appear to be the more conservative of the approaches and these dimensions are to be adopted in the design of these structures in the proposed ILEF. A summary is shown in Table 16 below.

Table 13 Comparison of the *holding capacities* of the holding and terminal ponds derived using current Feedlot guidelines and the FSIM model

Methodology	Holding Pond Volume	Terminal Pond Volume
Feedlot Guidelines	25,000m³	20,000m³
FSIM	220,00m³	30,000m³

The total holding pond capacity will be 220ML. This exceeds the design criteria (25ML), considerably (almost by a factor 10). The capacity of the terminal pond is 30ML. This exceeds guideline values of 20ML

The final design dimensions of the proposed ILEFs catch and main drains and sedimentation, holding and terminal pond spillways channels will be determined using the current ILEF guidelines (ICIAI, 1997 and SCARM, 1997) and the parameters detailed in Table 2, Table 5, and Table 9 of this Appendix.

6. References

Department of Science, Information Technology and Innovation (DSITI), (August 2015), *climatic dataset.* SILO enhanced climate database hosted by the Science Delivery Division Qld DSITI.

Canterford,R.P., Pescod, N.R., Pearce, H.J. and Turner, L.H., (2001),*Design intensity-frequency-duration rainfall*, in *'Australian Rainfall and Runoff',* ed. Pilgrim, D.H., Institution of engineers Australia, Barton, ACT.

ICIAI, (1997). *The New South Wales Feedlot Manual*, The Interdepartmental Committee on Intensive Animal Industries (Feedlot Section), NSW Agriculture, Orange, NSW.

Jenkins, A., (2001) *the sensitivity of headwater streams in the Hindu Kush Himalayas to acidification.* Water, Air and Soil Pollution, 2, 163-181.

Lott, S., (1994), *Drainage*, in '*Designing Better Feedlots*', eds. Watts, P. and Tucker, R., Queensland Department of Primary Industries, Toowoomba, Qld., pp. 5.5–37.

Lott, S.C. and Skerman, A.G., (1995), *Design of feedlot sedimentation systems*, in '*Proceedings of the Feedlot Waste Management Conference*', Ashmore, Queensland, Department of Primary Industries and University of Southern Queensland, Toowoomba, Qld.

Lott, S.C. and Skerman, A.G.,(1995), *Design of feedlot sedimentation systems,* in '*Proceeding of the Feedlot Waste Management Conference,',* Ashmore Queensland, Department of Primary Industries and University of Southern Queensland, Toowoomba, Qld.

Lott, S.C., (1998), *Feedlot hydrology*, PhD thesis, University of Southern Queensland, Toowoomba, Qld.

Main Roads Western Australia (2004) *Environmental Guideline: Environmental Assessment and Approval*. Document No. 6707/001.

Mazurak A.P. and Mielke, L.N., (1976) *Infiltration of water on a cattle feedlot*. Trans. ASAE 19(2):341-344.

MLA (2012) *Lot feeding Brief: March 2012*. Meat and livestock Australia, Sydney.

Monteith, J.L., (1965), *Evaporation and environmental Physics*, 2nd Edition. Edward Arnold, London.

O'Loughlin, G.G and Robinson, D.K., (2001), *Urban stormwater drainage*, in '*Australian Rainfall and Runoff'* ed. Pilgrim, D.H., Institution of Engineers Australia, Barton ACT.

Penman, H.L., (1948), *Natural evaporation from open water, bare soil and grass.* Proceedings of the royal Society of London. Series A, Mathematical and Physical Sciences, Vol, 193, No.1032.

Pilgrim, D.H., ed., (2001), *Australian Rainfall and Runoff,* Institution of Engineers Australia, Barton, ACT.

SCARM (1997), *National guidelines for Beef Cattle Feedlots in Australia 2nd Edition*, Standing Committee on Agriculture and Resource Management, Collingwood, Victoria.

Schwab,G.O., Bamer, K.K., Frevert, R.K. and Edminter, T.W.,(1971) *Rainfall and Runoff*, In Elementary Soil and Water Engineering. John Wiley & Sons Inc., New York.

Israelsen, O.W., and Hansen V.E., (1962), *Irrigation principles and practices,* John Wiley and Sons, New York.

Shaw, E.M., (1994), *Hydrology in Practice*, 3rd Edition, Chapman and Hall, London.

Skerman, A., (2000), *Reference manual for the establishment and operation of beef cattle feedlots in Queensland*, Queensland Department of Primary Industries , Brisbane, Qld**.**

Southcott, P.H. and Lott, S.C., (1997), *Preliminary Results of Soil Permeability and Physical Characteristics of Feedlot Pens'*, 7th Australia, New Zealand Conference on '*Geomechanics: Geomechanics in a Changing* World',: Conference Proceedings.

Sweeten, J. M., and Lott S. (1994) *Dust management*. In Watts, P. and R. Tucker (eds.), Designing Better Feedlots. Toowoomba, Queensland, Australia. Queensland Department of Primary Industries. Conference and Workshop Series QC94002

Thornthwaite, C.W. (1948), *An Approach toward a rational classification of climate.* Geographical review 38.

Walker, K.F. (1979), *Regulated streams in Australia: the Murray Darling system*. In J.V Ward and J.A Stanford (eds), the Ecology of Regulated Streams, Plenum, N.Y:143-163.

Watts, P.J., Gardner, E.A., Tucker, R.W. & Casey K.D. (1994), *Mass-balance approach to design of nutrient management systems at cattle feedlots,* Proceedings of the Great Plains Animal Waste Conference on Confined Animal Production and Water Quality, Denver, Colorado, GPAC Publication 151, 27-33.

7. Appendices

Appendix A.	Intensity Frequency and Duration Data	A-1
Appendix B.	Summary of Darwin Climate	B-1

Appendix A. Intensity Frequency and Duration Data

Table 14 Intensity Frequency and Duration (IFD) Data : Darwin – Livingstone (mm/hr)

DURATION	1 Year	2 years	5 years	10 years	20 years	50 years	100 years
5Mins	142	180	220	244	279	325	362
6Mins	132	168	205	228	261	305	339
10Mins	109	139	169	187	214	249	277
20Mins	82.2	104	126	139	158	184	204
30Mins	67.6	85.6	103	114	130	151	167
1Hr	45.4	57.5	69.3	76.5	86.9	101	112
2Hrs	28.2	35.7	42.9	47.3	53.6	62.1	68.8
3Hrs	20.8	26.3	31.6	34.7	39.3	45.5	50.3
6Hrs	12.2	15.4	18.4	20.3	22.9	26.5	29.3
12Hrs	7.31	9.26	11.2	12.3	14	16.2	18
24Hrs	4.57	5.85	7.25	8.12	9.34	11	12.3
48Hrs	2.87	3.73	4.83	5.55	6.52	7.87	8.97
72Hrs	2.08	2.72	3.61	4.2	4.98	6.1	7.02

Appendix B. Summary of Darwin Climate

__Summary of Darwin Climate__

The climate of the site is best described as wet tropical. It is warm to hot through the year. The site has a defined dry season and wet season. The site has relative constant day time maximum temperatures. Detailed climatic data are available for Bureau of Meteorology Darwin Airport (AP) some 30km to the north and at Berry Springs about 8 km from the site. Its data is similar to Darwin AP.

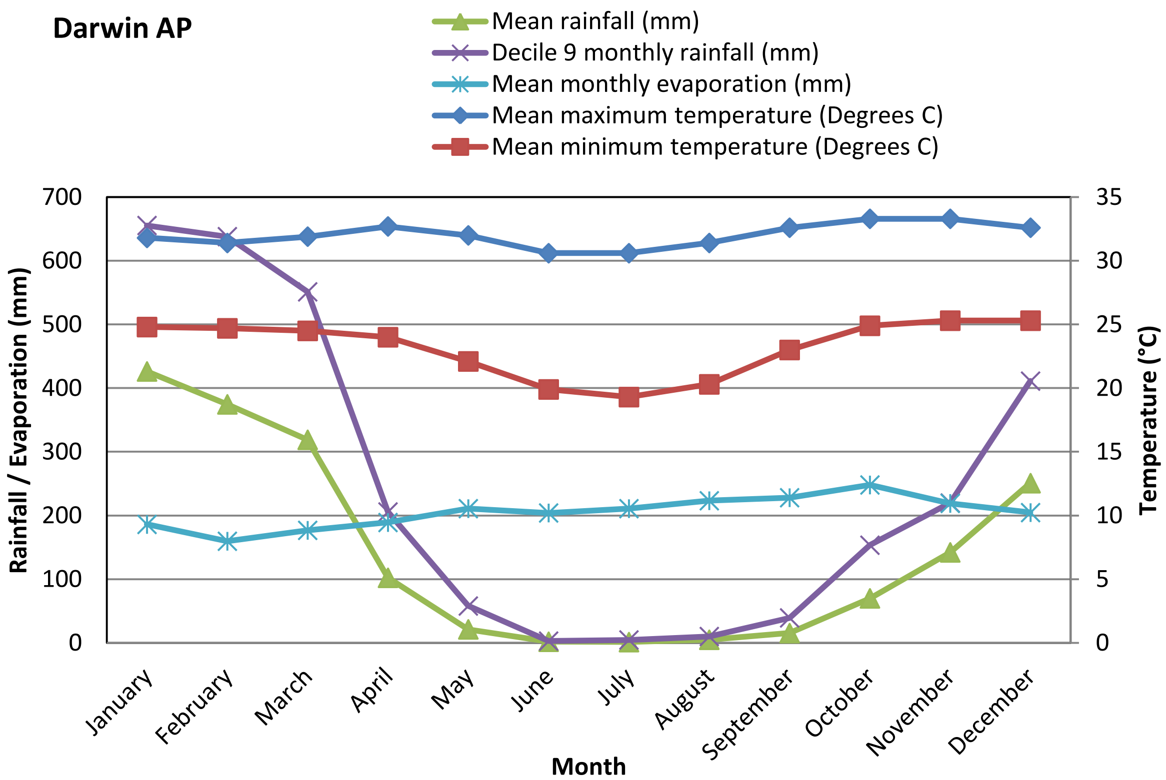


Figure 12 Graph of Temperature and Rainfall and Evaporation for Darwin AP

__Temperature__

Average temperature data are presented in Table 15 these show the low level of variation in temperatures. During winter minimums are as low as 15°C overnight.

Table 15 Monthly Average Temperatures for Berry Springs (~8km away) (1971-2013) (BoM, 2014)

Statistic	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Lowest	31.1	30.5	31.4	32.6	31.7	29.8	29.4	29.9	31.2	32.5	32.2	32.3
Highest	31.8	32.2	32.4	32.9	31.8	30.2	29.7	30.3	32.1	32.8	33.0	33.2

Rainfall

Rainfall data for Berry Springs are provided in Table 16. Most rain is received from October to April. The dry season is from May to September. It would reasonable to expect an average rainfall of 1,800mm per year.

Table 16 Rainfall data for Berry Springs (~8km away) (1971-2013) (BoM, 2014)

Statistic	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Mean	394.1	309.1	299.9	97.7	14.3	0.1	0.7	1.7	15.6	67.3	157.1	339.7	1779.9
Lowest	119.1	33.0	58.8	1.2	0.0	0.0	0.0	0.0	0.0	0.0	37.2	64.6	1156.1
5th %ile	235.1	73.2	71.9	17.1	0.0	0.0	0.0	0.0	0.0	6.7	46.9	70.4	1198.3
10th %ile	243.2	123.9	103.7	24.9	0.0	0.0	0.0	0.0	0.0	10.8	55.3	105.0	1283.0
Median	**371.6**	**328.0**	**309.6**	**60.2**	**2.2**	**0.0**	**0.0**	**0.0**	**5.0**	**53.9**	**145.1**	**343.4**	**1807.5**
90th %ile	554.5	449.4	480.5	160.3	30.9	0.0	0.0	6.1	43.1	142.9	240.2	541.9	2067.2
95th %ile	622.6	515.5	562.8	194.4	71.9	0.0	0.9	10.9	47.5	150.2	259.1	552.0	2169.8
Highest	797.6	534.8	814.4	675.3	85.2	1.6	14.8	15.2	92.7	179.5	367.6	630.0	2309.6

Evaporation

Evaporation data are collected at Darwin Airport. The annual average evaporation at Darwin is about 2460mm. Thus the site generally has a moisture deficit on an annualised basis of about 660mm. The greatest deficit occurs through the dry season; moisture surpluses occur in the wet season.

Moisture Deficit

The annual average rainfall for Berry Springs is 1807.5mm, and the annual average evaporation for Darwin is 2460mm. Thus the average moisture deficit at the site is in excess of 600 mm/year. This is equivalent to an annual average water deficit of 6ML/ha.

However this is misleading with regard to actual deficits applicable to the crop. While moisture surpluses occur over summer, every dry season has a significant deficit. The deficit over the dry season is the key variable in sustainable re-use of wastewater. The dry season moisture deficit is about 1375 mm (on average) (April to November).