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ALLAWUNA FARM LANDFILL

Surface Water, Groundwater and Leachate Management Plan

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REPORT

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Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 Overview

SITA Australia Pty Ltd (SITA) has appointed Golder Associates Pty Ltd (Golder) to provide engineering design services and supporting technical advice for the Allawuna Farm Landfill (Allawuna Landfill).

This document summarises the surface water, groundwater and leachate management assessments and plans for the Allawuna Landfill. Golder's scope of services for the work summarised in this document is outlined in Golder's proposal dated 11 December 2014 (P47645080-005-L-Rev0). The location of the Allawuna Landfill is shown in Figure C1 (Appendix C).

1.2 Purpose

The purpose of this document is to provide engineering information regarding stormwater and leachate management systems for the landfill site in relation to the development of Cells 1 and 2.

1.3 Objectives

The objectives for the study were to undertake investigation into the requirements and specifications for leachate and stormwater management at the Allawuna Farm landfill suite. More specifically, the following tasks were proposed to be undertaken through this study:

Leachate analyses to be undertaken as part of the leachate management study will include:

- Calculation of leachate generation from the landfill cells
- Sizing of leachate ponds to contain leachate from the landfill cells (taking into account a staged development plan)

Hydrological analyses to be undertaken as part of the stormwater management study will include:

- Sizing of a stormwater pond for retention of impacted runoff from the landfill site
- Development of a stormwater management plan taking into account staged development of the proposed site.

2.0 ABBREVIATIONS AND DEFINITIONS

The acronyms and abbreviations used in this document are defined in Table 1.

Table 1: Acronyms, Abbreviations and their Meanings

Name/Acronym	Definition
Allawuna Landfill	Allawuna Farm Landfill located south of the Great Southern Highway, approximately 20 km west from the town of York.
ARR	Australian Rainfall and Runoff – A Guide to Flood Estimation in Australia
AS	Australian Standard
ASTM	American Society for Testing and Materials
AWBM	Australian Water Balance Model – A rainfall-runoff model to assess surface water and catchment yield
BA	Bowman & Associates Pty Ltd
BoM	Bureau of Meteorology
Vic BPEM	Victorian EPA Best Practice Environmental Management
DEM	Digital Elevation Model
DER	Department of Environment and Regulation
Division Bunds	Short-term embankments that separate (divide) adjacent cells in the landfill
FoS	Factor of Safety
DTM	Digital Terrain Model
GCL	Geosynthetic Clay Liner
Golder	Golder Associates Pty Ltd
HDPE	High Density Polyethylene
LLDPE	Linear Low Density Polyethylene
MSW	Municipal Solid Waste
Perimeter Bunds	Long-term embankments that delineate the boundaries of the waste disposal area
SDS	Subsoil Drainage System
SILO	Not an acronym – Refers to the name of a climate information database
SITA	SITA Australia Pty Ltd
SMS	Sediment Management Structure
SRTM	Shuttle Radar Topographic Mission
V : H	Vertical : Horizontal
WAA	Works Approval Application
Waste	Municipal solid waste
XPSWMM	Hydrologic, hydraulic (1-D & 2-D) and quality modelling software

3.0 CLIMATE REVIEW

The aim of this assessment is to provide a robust review of available local and regional climate information and to provide recommendations for the determination of a long-term climate record to be adopted for the assessment and design of water management and storage options, the assessment of the site-wide water balance and for the development of stormwater management plan requirements. The use of a single climate record will provide consistency between each of these components of work. Average rainfall patterns are known to vary quite considerably in the region and therefore one of the key objectives of this assessment is to validate the reliability of the SILO (DSITIA, 2015) derived daily rainfall compared to long-term regional rainfall records.

The definitive climate series for the site will include, at this stage, the following climate variables:

- Daily rainfall (mm/d)
- Evaporation (Class A Pan equivalent) (mm/d).

3.1 Climate Data Availability

Rainfall and climate datasets for the region have been collated from the following sources:

- Bureau of Meteorology (BoM) database
- SILO data drill data for the site

The review of BoM climate stations included both open and closed stations with the objective of identifying long-term rainfall records and climate averages with sufficient duration to characterise the climatic regime of the area. Additionally, SILO drilled climate data (DSITIA, 2013) from 1900 have been downloaded for a location considered representative of the Allawuna study area

The locations and coverage of the available rainfall data are shown in Figure C2 (Appendix C) and details of the available rainfall records available in the region are summarised in Table 2.

Table 2: Summary of Available Regional Long-Term Rainfall Series

Location	Lat.	Long.	Period of Available Record	Elevation (m)	Distance from Site (km)	Median Annual Rainfall (mm)
SILO	-31.90	116.6	1900-2015	325	Within 1 km	590
Berry Brow	-31.82	116.53	1907-1950	276	12.5	614
Quadney	-31.79	116.63	1995-2015	310	14.0	397
York (Combined) [1]	-31.90	116.77	1877-2015	179	15.5	434
Muresk Agricultural College	-31.75	116.68	1926-1981	166	19.0	448
Mount Hardey	-31.92	116.82	2007-2015	283	20.5	366
Southbourne	-31.74	116.49	1907-2013	280	22.0	574
Bakers Hill	-31.75	116.46	1964-2015	330	22.5	574
Talbot House	-32.10	116.75	1904-1972	0	25.0	463
Quellington	-31.77	116.86	1909-2015	220	29.0	376
Northam	-31.65	116.66	1877-2015	170	29.5	407
Oakland	-32.30	116.64	1912-2013	240	43.0	517

[1] York combined rainfall series includes York Post Office (1877-1996) and York (1996 – 2015) rainfall station data.

3.2 Rainfall Data Analysis

3.2.1 Daily and Seasonal Rainfall

In order to undertake water management assessments for the proposed Allawuna Landfill site, a long-term, continuous daily rainfall record covering a range of climatic extremes, i.e. significantly dry and wet periods, is generally considered to be the minimum requirement. This is necessary to ensure that water management infrastructure can be designed with the required level of resilience for the life of the landfill.

The York rainfall station is located approximately 15.5 km north-east of the study area and has a long-term, consistent daily and monthly rainfall dataset extending back to 1877, which along with Northam, approximately 29 km to the north, represent two of the key rainfall and climate records for the Wheatbelt region. Due to the proximity and nature of these rainfall stations, these would generally be assumed to be appropriate for defining the prevailing daily and seasonal rainfall patterns for the Allawuna site.

However, a steep rainfall gradient exists across the region surrounding the study area and this is required to be considered in the assessment of a definitive rainfall series. The declining rainfall gradient follows a general west to east direction from the northern extent of the Darling Ranges (approximately 45 km west of Allawuna), This is one of the strongest rainfall gradients in Western Australia, as indicated in Figure C2 (Appendix C), with a marked rain shadow defined along the eastern side of the Darling Scarp and a distinct zone of markedly higher rainfall extending from Collie North to the Chittering Brook and Karnet. Note that the annual average rainfall contours presented are based on BoM 1961-1990 (30 year) averages and exclude the period from 1990 onwards which is widely referenced as exhibiting a notably drier climate across south-west Western Australia, particularly winter rainfall (Hope et al, 2006, Holper, 2011).

York is located just below the 500 mm BoM 1961-90 annual rainfall average and the median long-term annual rainfall (1931-2014) is 434 mm. This is consistent with calculated median annual rainfall for the climate stations at Northam (407 mm) and Muresk Agricultural College (448 mm) to the north and Talbot House (463 mm) to the south.

The Allawuna landfill site is located just above the BoM 1961-90 annual rainfall average of 600 mm and the extracted SILO data for the site has a long-term (1931-2014) median annual rainfall of 589 mm. This higher median rainfall is consistent with the long-term median rainfall statistics calculated for Bakers Hill (574 mm) and Southbourne (573 mm) just over 20 km to the north-west as well as the more limited rainfall series for Berry Brow (1931-1950) of 614 mm.

A more detailed comparison of the seasonal distribution of rainfall for the long-term (1931-2014) SILO and York rainfall records (Figure 1a and Figure 1b) indicate that monthly rainfall patterns for the dry season (Nov to Apr) are very similar. However, the monthly rainfall distributions for the wet season period (May-Oct) are notably higher at the SILO rainfall location and also exhibit greater variability in the monthly rainfall distributions. Figure 2 shows a comparison of the November–April (summer) and May–October 6-month seasonal rainfall totals for the SILO and York rainfall series. These data show that summer rainfalls at the two locations are almost identical, however winter rainfall totals are notably lower at York indicating that the rainfall gradient defined across the region has a significant influence on the distribution of winter rainfalls. Maximum monthly rainfall totals for the SILO record of just over 350 mm in July 1945 and June 1958 correspond with the highest monthly rainfall at York of 260 mm in June 1945 and 230 mm for July 1958.

The monthly and seasonal distributions of rainfall defined for the SILO data series (Figure 1a) are comparable to the distribution and magnitude of monthly rainfall statistics presented for the Bakers Hill rainfall station (Figure 1c). Both locations are in areas of similar annual average rainfall, based on BoM 1961-90 annual averages indicated in Figure C2 (Appendix C), i.e. above 600 mm, and experience median monthly rainfalls greater than 100 mm during June and July. The York and Northam rainfall stations are located in areas with annual average rainfall below 500 mm and exhibit similar monthly rainfall distributions (Figure 1b and Figure 1d, respectively). Median rainfall for June and July at the York and Northam locations ranges between 70 mm to 80 mm, which is approximately 25-30% lower than the maximum rainfall averages for the SILO and Bakers Hill locations.

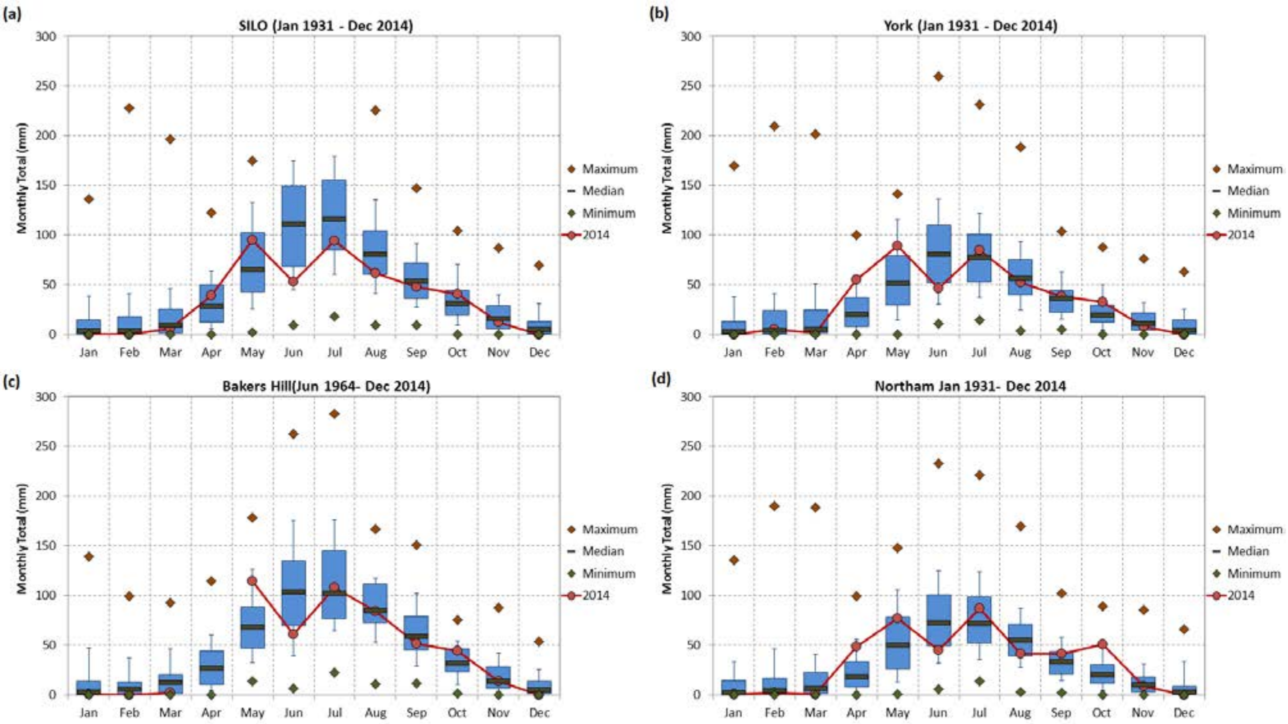

Figure 1: Long-Term Monthly Rainfall Distributions and 2014 Rainfall for (a) SILO, (b) York (Combined), (c) Bakers Hill and (d) Northam Rainfall Series

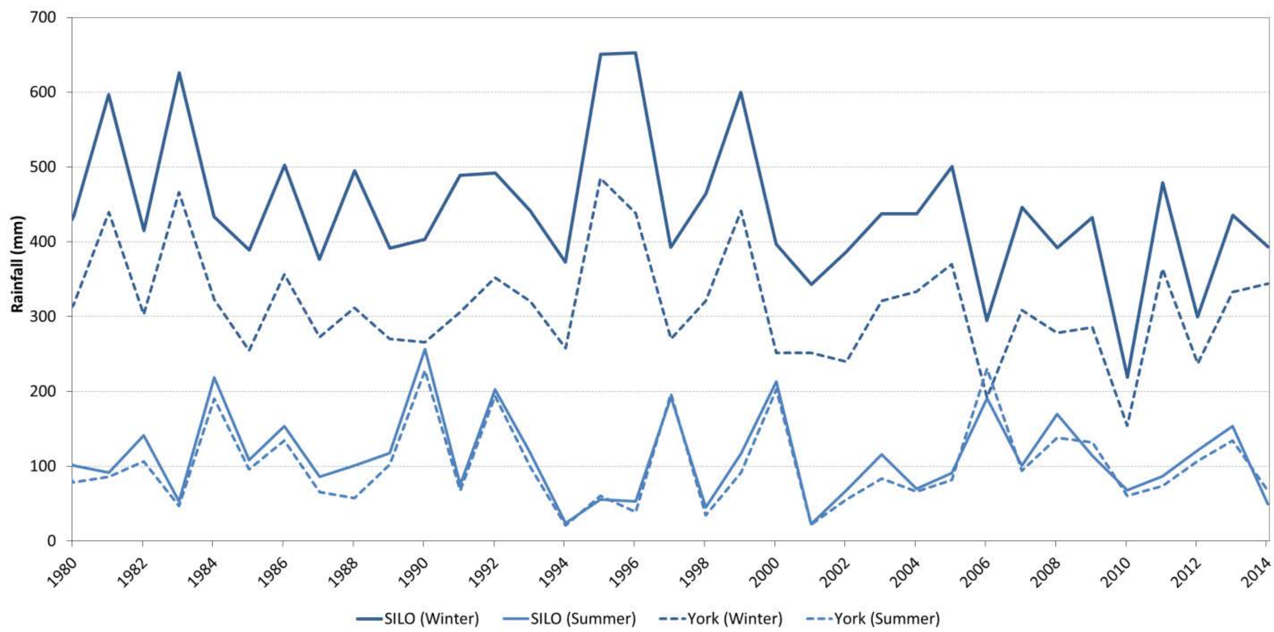

Figure 2: Comparison of SILO and York November–April (Summer) and May–October (Winter) Period Rainfall Totals (1980 to 2014)

Based on the rainfall data assessment, the adopted SILO dataset indicates that average annual rainfall for the Allawuna Landfill site over the period 1931-2014 was 599 mm (median annual rainfall of 589 mm) with annual rainfall showing a relatively high level of inter-annual variability ranging from a minimum of 286 mm (2010) to a maximum of 998 mm (1955). Monthly rainfall for the SILO series for the period Jan 1980 to December 2014 is presented in Figure 3.

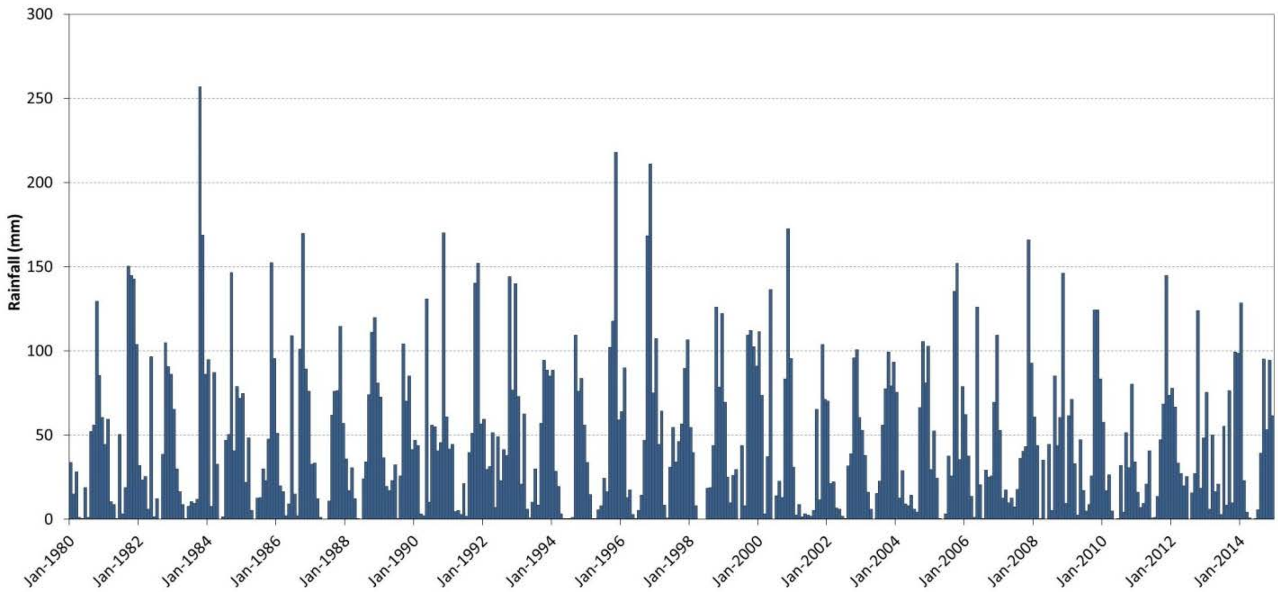

Figure 3: Monthly Rainfall for the SILO Drilled Rainfall Location (Jan 1980 to December 2014)

3.2.2 Short Duration Design Rainfall

Rainfall intensity-frequency-duration (IFD) data for the study area have been derived using the Bureau of Meteorology's (BoM) CDIRS (Computerised Design IFD Rainfall System), which allows automatic determination of a full set of IFD curves and associated data for any location in Australia. This approach is compatible with the manual procedures described in Australian Rainfall and Runoff (ARR): A Guide to Flood Estimation (Pilgrim, 1987).

Table 3 and Figure 4 summarise rainfall intensities associated with design storms with durations up to 72 hours and Average Recurrence Intervals (ARIs) up to 100 years applied in the estimation of design flood discharges for a range of ARI events.

Table 3: Rainfall Intensity (mm/h) for Standard Durations and Average Recurrence Intervals (ARIs)

Duration (mins)	1 Year ARI	2 Year ARI	5 Year ARI	10 Year ARI	20 Year ARI	50 Year ARI	100 Year ARI	Duration (hours)
10	34.7	46.3	62.2	73.7	89.7	114.0	135.0	**0.167**
20	24.2	31.9	41.7	48.7	58.5	72.9	85.2	**0.33**
30	19.2	25.1	32.4	37.5	44.6	55.1	64.0	**0.5**
60	12.5	16.3	20.6	23.6	27.8	33.9	39.0	**1**
120	8.1	10.5	13.1	14.8	17.4	21.0	24.0	**2**
180	6.3	8.1	10.0	11.4	13.3	16.0	18.2	**3**
360	4.1	5.2	6.4	7.3	8.4	10.1	11.5	**6**
720	2.6	3.4	4.1	4.6	5.4	6.4	7.3	**12**
1440	1.6	2.1	2.6	2.9	3.4	4.1	4.6	**24**
2880	0.99	1.28	1.58	1.78	2.08	2.51	2.87	**48**
4320	0.72	0.93	1.15	1.31	1.53	1.86	2.13	**72**

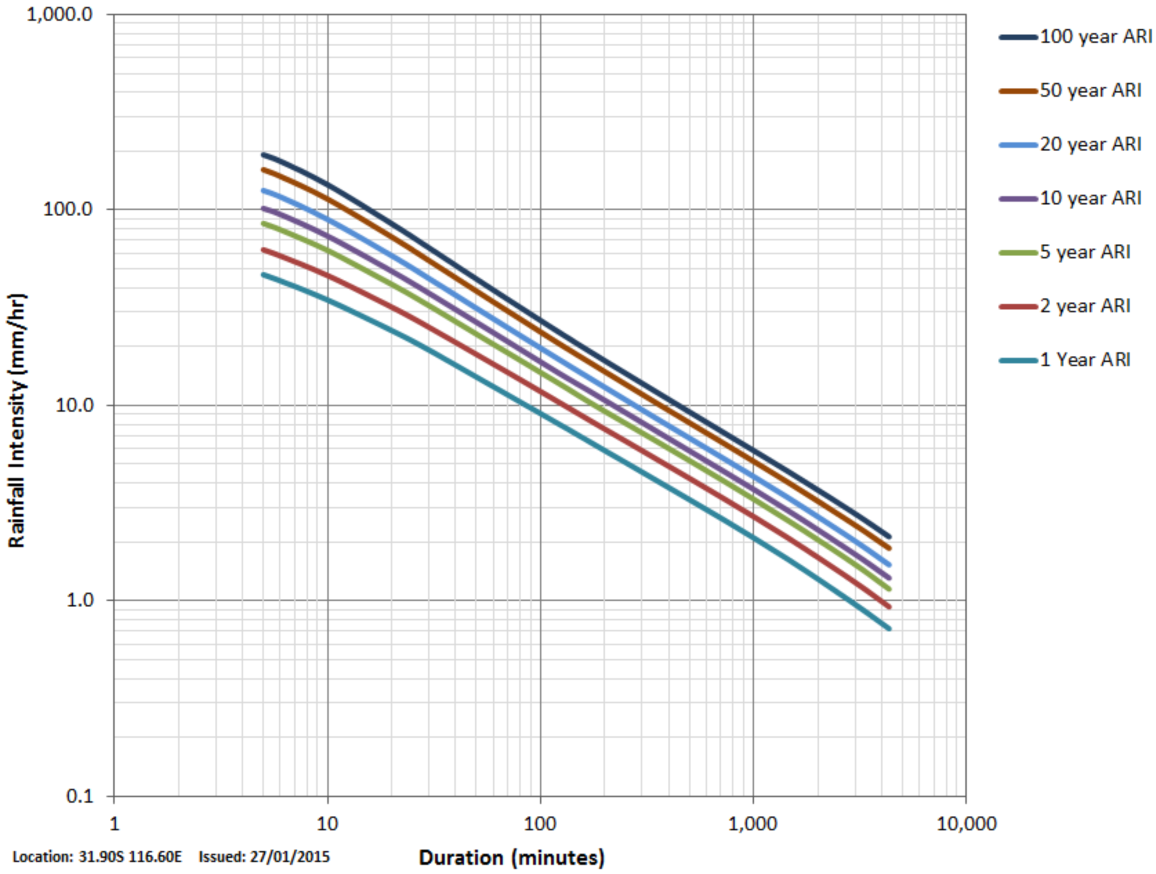

Figure 4: IFD Curves for the Allawuna Landfill Study Area

3.3 Estimated Evaporation

The SILO climate dataset also includes long-term estimates of daily evaporation (Class A Pan Evaporation) for the Allawuna Landfill site. These data represent an average 'synthetic' pan evaporation estimate pre-1970 and a daily pan evaporation estimate for the period from 1970.

Dam evaporation (E_{OD}) to Class A pan evaporation (E_{PAN}) coefficient relationships have proven accurate for small storages (depth less than 4 m) whereas, in larger dams, the effect of heat storage (particularly in higher latitudes) results in different dam/pan coefficients for different months.

While parts of Western Australia have reliable, though seasonal rainfall, large areas are extremely arid and across much of the state it is necessary, therefore, that water supplies be stored on site. Estimating the quantities required relies heavily on adequately estimating the evaporation loss demand. There have been a number of attempts to relate Class A pan evaporation to evaporation from a dam and Luke et al (1987) provides a concise summary of the spatial variations in the E_{OD}/E_{PAN} coefficients across Western Australia.

The evaporation estimates are presented in Table 4 with the monthly total evaporation loss profiles plotted in Figure 5. The estimated average annual dam evaporation for the site is approximately 1415 mm. An E_{OD}/E_{PAN} coefficient of 0.78 defined for Northam (Luke et al, 1987) has been adopted for the estimation of evaporative water losses for the stormwater dam water balance assessment.

Table 4: Estimated Monthly Average Evaporation Losses

Month	Evaporation (Class A Pan) (E_{PAN})	Estimated Open Water Dam Evaporation (E_{OD})
January	288	225
February	241	188
March	205	160
April	122	95
May	75	59
June	52	41
July	53	41
August	66	52
September	93	72
October	150	117
November	204	159
December	264	206
Annual	**1813**	**1415**

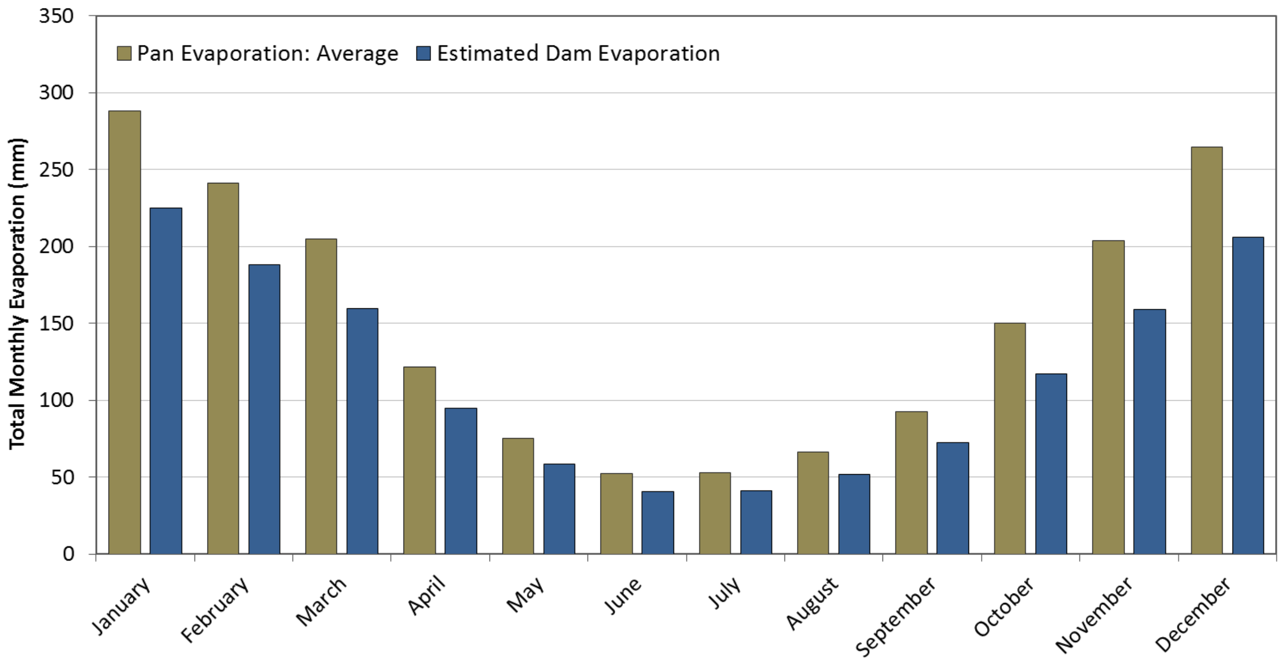

Figure 5: Estimated Monthly Average Evaporation (mm)

4.0 SURFACE WATER MANAGEMENT PLAN

The layout of proposed surface water management infrastructure for the Allawuna Landfill is presented in Figure C4 (Appendix C). The following sections describe the assessment of, specifications, sizing and operational requirements of the respective water management options.

4.1 Regional and Local Surface Water Systems

The location of the Allawuna Landfill site relative to local and regional surface water drainage features is shown in Figure C3 (Appendix C), indicating the proposed development site is located in the upper reaches of the Thirteen Mile Brook, close to the catchment divide with the adjacent Six Mile Brook. Both watercourses ultimately drain to the Avon River. A small, ephemeral creek is located directly adjacent to the proposed development site and flows into the Thirteen Mile Brook approximately 250 m to the south-west of the site.

Upstream of the development, near the headwaters of the Thirteen Mile Brook, a Rivercare project partnership between the Department of Water (DoW) and the Talbot Brook Land Management Association has been working to restore riparian vegetation along the banks of the Brook with the aim of reducing sediment and improving water quality (SITA, 2014).

The DoW has been consulted regarding the project and identified the following project requirements to be addressed:

- Development of Water Management Plan as part of the EPA and/or Local Government Approvals processes to prevent degradation of surface or groundwater systems
- Detailed protection of Thirteen Mile Brook from landfill leachate and impacted stormwater runoff
- Acquire a permit for a creek crossing under the *Rights in Water and Irrigation Act 1914*.

4.2 Stormwater Dam

4.2.1 Water Balance Modelling Approach

An assessment of the site water balance for the Allawuna Landfill has been carried out in order to develop an improved understanding of the water management requirements relating to the proposed stormwater dam. More specifically the development and application of the water balance model provides an opportunity to define key design specifications and management aspects of the water storage option, including:

- Assessment of the potential surface water yield of the upstream catchment areas
- Estimated water requirements and demands for site operation (over the life of the landfill), particularly to evaluate the likelihood of deficits of water availability during particularly dry periods
- Design specifications and sizing of water-related infrastructure, operational water management requirements and assessment of the potential scale and magnitude of downstream impacts

The water balance model was constructed using GoldSim, a graphical object-oriented modelling environment with a capacity to incorporate dynamic probabilistic simulations. For the purpose of the assessment of the potential stormwater dam yield and reliability, the simulation period is based on the 25 year period from Jan 1990 to December 2014. This includes the drier climate period from 2000 onwards and also includes the 2nd (2010), 4th (1994) and 5th (2001) driest years of the 115 year SILO rainfall record, therefore allowing the potential yield of the storage to be assessed for drought years.

The following sections describe the structure, inputs and assumptions applied in the water balance assessment.

4.2.1.1 Climate Data – Rainfall and Evaporation

The climate information adopted for the water balance modelling is based on the datasets described in Section 3.0. These include:

- Daily Rainfall: Based on the 25 year (Jan 1990 to Dec 2014) SILO daily rainfall series (Section 3.2).
- Monthly Evaporation: Based on the profiled estimate of open water evaporation (E_{OD}) (Section 3.3).

The simulation period adopted for the site wide water balance is defined by the length of the estimated daily rainfall sequence, i.e. the model runs were applied for the 25 year period (Jan 1990 to Dec 2014).

4.2.1.2 Rainfall-Runoff Modelling

Surface water drainage catchments surrounding the Allawuna Landfill site have been delineated using a Digital Elevation Model (DEM) generated from Shuttle Radar Topographic Mission (SRTM) data. The two contributing catchment areas upstream of the stormwater dam (shown in Figure C3 (Appendix C), delineated using the survey data, are listed below:

- Catchment East: 135 ha (1.35 km^2)
- Catchment North: 65 ha (0.65 km^2)

Note that the catchment area for the northern catchment has been adjusted to reflect the effective post-development catchment accounting for the reduction of runoff from the developed landfill areas.

The defined catchment areas are used to estimate surface water runoff inflows to the water storage using the Australian Water Balance Model (AWBM) rainfall-runoff model (Boughton et al, 2003). The adopted AWBM model parameters for simulating flow response in the upstream catchments are presented in Table 5 and an outline of the model structure is described in Appendix A.

Table 5: Adopted AWBM Parameters

Parameter	Abbreviation	Value
Small storage capacity (mm)	C1	30
Medium storage capacity (mm)	C2	85
Large storage capacity (mm)	C3	150
Small partial area portion	A1	0.2
Medium partial area portion	A2	0.45
Large partial area portion	A3	0.35
Baseflow index	BFI	0.35
Baseflow recession factor	K	0.95
Surface flow recession factor	KS	0.35

Modelled surface water runoff for the 25 year simulation period (Jan 1990 to December 2014) is presented in Figure 6. This clearly reflects the rapid response of surface water runoff which would be expected to be the dominant hydrological regime for this type of catchment.

A summary of annual catchment yield as a percentage of annual rainfall is presented in Table 6. This indicates that the lowest annual runoffs occurred in 2001, 2010 and 2012 with modelled catchment runoff estimated to be less than 13 mm, which equates to less than 3% of total annual rainfall for these years.

Based on the available rainfall data summarised in Table 6, 1994 has been identified as the 4th driest rainfall year of the 25 year simulation period; however, modelled runoff is predicted to be relatively high as a percentage of rainfall, i.e. 25 %. A more detailed review of the distribution of rainfall indicates that 1994 experienced an extremely dry November–April (summer) season of just 24 mm over the 6 month period; however, total May-October (winter) season rainfall was closer to the average of the 25 year period. The seasonal rainfall totals for the SILO rainfall series are presented in Figure 2. Catchment runoff and surface water yield for the stormwater dam catchment are particularly sensitive to dry winter periods when the majority of the runoff is typically generated.

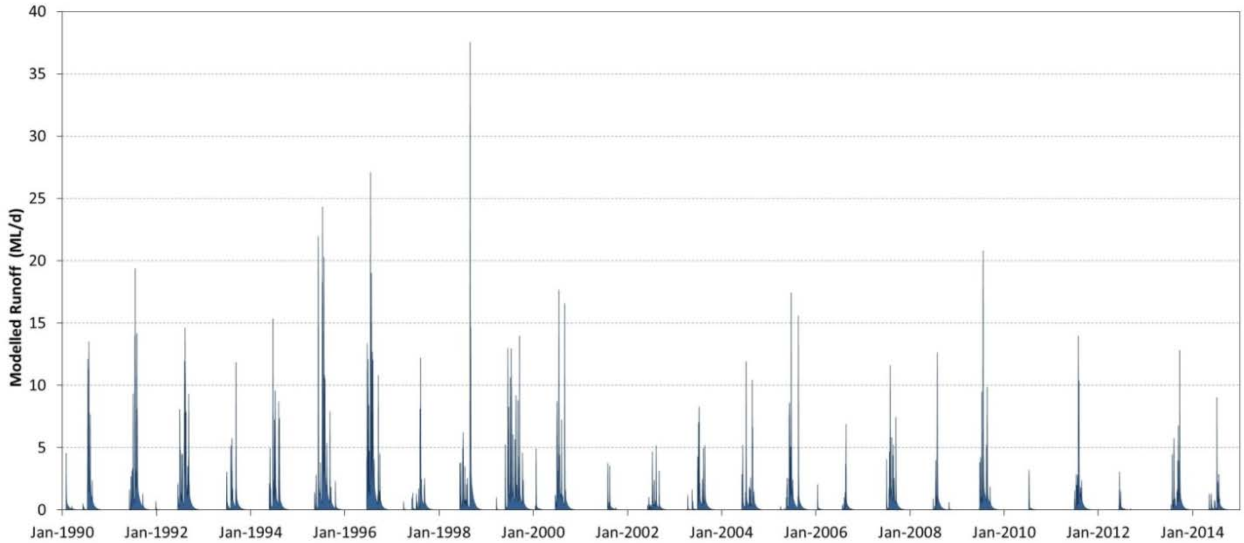

Figure 6: Modelled Daily Surface Water Runoff (ML/d)

Table 6: Annual Modelled Catchment Runoff Summary

Modelled Year	Rainfall (mm)	Rainfall Rank (out of 115 years)	Modelled Runoff (mm)	Runoff as a % of Rainfall
1990	665	80	95	14%
1991	637	68	127	20%
1992	681	81	138	20%
1993	514	26	55	11%
1994	376	4	92	25%
1995	736	96	218	30%
1996	749	98	219	29%
1997	521	28	57	11%
1998	538	35	131	24%
1999	721	94	176	24%
2000	580	50	124	21%
2001	385	5	13	3%
2002	448	13	34	8%
2003	569	47	67	12%
2004	494	22	70	14%
2005	582	51	104	18%
2006	498	24	21	4%
2007	555	40	76	14%
2008	562	43	42	7%
2009	542	36	107	20%
2010	286	2	7	2%
2011	595	57	71	12%
2012	427	10	10	2%
2013	527	31	63	12%
2014	451	14	32	7%

4.2.1.3 Estimated Water Demands

The water demands adopted for the modelling are outlined in Table 7.

Table 7: Site-Wide Water Demands

Demand Type	Demand	Notes
Dust Suppression	40 m^3/d	Based on two trucks each with a capacity of 20 000 L/d. Dust suppression applied based on a 5 day week. No dust suppression applied on days with more than 5 mm/day rainfall
Fire Water	500 m^3/y	Defined as an annual allocation/reserve
Construction Water Requirement	up to 12 500 m^3	Estimated water requirement for the construction of Cells 1 to 3 only. Construction water use is a function of volume of material required to be moisture conditioned as part of the construction of each cell (refer to Appendix B).

Vic BPEM (EPA, 2014) recommends that where reticulated water is not provided, at least 50 000 litres (50 m^3) should be stored on site for combating small fires. For a significant fire, this volume will need to be supplemented by another source of water, i.e. groundwater or stormwater in dams. The Fire Management Plan for Allawuna Landfill currently includes the following:

- 150 m^3 dedicated firefighting water tank
- 100 m^3 general site use tank.

All on-site tanks have the option to be refilled by an automatic pumping system from the stormwater dam, when required.

The construction water requirement has been estimated based on the maximum water demand for Cells 1 to 3 only at this stage and is not explicitly defined in the water balance modelling assessment. Details of the estimated construction water demand and associated calculation assumptions are provided in Appendix B. At this stage the duration and distribution of the construction water demand is not well defined, therefore, the water balance assessment will aim to advise periods when water availability for construction is at it optimum and identify potential constraints.

Construction water demand requirements, water availability and sources of supply will be addressed through an options study that will be carried out after the submission of the WAA. This options study will provide a more detailed assessment of the availability of construction water and the potential options available to supply suitable quantity and quality of construction water, specifically during the construction of Cell 1 and 2.

4.2.1.4 Seepage Losses

For the purpose of the water balance assessment potential seepage losses from the stormwater dam have also been accounted for. It is assumed that during construction of the dam, compaction and lining with clayey material, where available, will be utilised to minimise water losses through seepage throughout the year. However, an assumed vertical seepage rate of 1×10^{-8} m/s has been applied which is volumetrically accounted for based on the modelled surface area of the pond.

Total modelled seepage loss tends to vary seasonally between 10 and 30 m^3/d.

4.2.2 Stormwater Dam Sizing

Assuming a stormwater dam with an embankment approximately 4.80 m high at the proposed stormwater dam location presented in Figure C4 (Appendix C), this would equate to a total storage capacity of approximately 36 000 m^3 (36 ML) at the spillway elevation (RL 311.75 m). This maximum storage volume has been assessed against the demand requirements, potential losses and catchment runoff yield to assess the reliability of the water storage option over the life of the landfill site. The variation in modelled storage over the 25 year period is presented in Figure 7.

The storage water balance assessment indicates that based on the current estimates of upstream catchment runoff, water demand and storage losses, the stormwater dam only fails to provide sufficient water supply in one year (2011) out of the 25 year simulation period. This failure, due to the emptying of the storage during the summer of 2011 is a result of the very limited surface water runoff during the preceding winter period which did not allow the storage dam to fill sufficiently to maintain water supply for the following summer. Over the 25 year period the stormwater dam was predicted to not fill during the particularly dry winters of 2001, 2010 and 2012, highlighting the sensitivity of the water supply to winter period rainfall and runoff.

Excluding, 2011 and 2013, the water balance model indicates that approximately 5 ML (5000 m^3) of storage may be expected to be the minimum storage level during 'normal' operation. This unused water volume may be utilised and/or allocated as an emergency water supply, i.e. for fire water and construction water.

Construction water use from the stormwater dam should be utilised during the winter months when excess surface water yield is highest. Water availability for construction water requirements may be constrained during particularly dry years, i.e. during winter periods when the dam does not reach full storage capacity. Alternative water sources should be utilised where construction water is required during an extended dry period and these may include groundwater supplies or obtaining water from external sources.

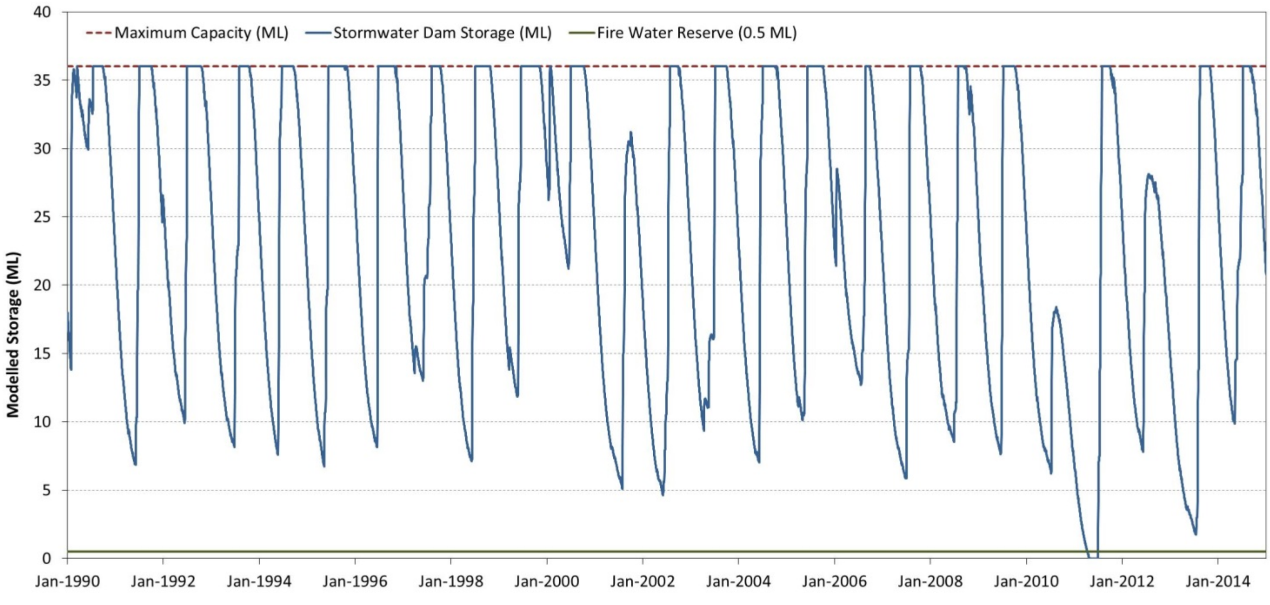

Figure 7: Modelled Stormwater Dam Storage (Jan 1990 to Dec 2014)

The management and operation of the stormwater dam as a water supply will need to be assessed in more detail in order to identify potential operational rules and restrictions in order to maximise the reliability of the water supply over the operational life of the landfill. Further surface water management and operational assessments will be carried out following the submission of the WAA. Alternative water supply options and water efficiency measures will be clearly defined for dry years, i.e. when the stormwater dam does not reach a full level by the end of the winter season. Additionally, an ongoing water monitoring plan including the measurement of surface runoff from the upstream catchment and variations in water storage within the stormwater dam will be carried out in order to better quantify the potential yield of the catchment.

4.2.3 Stormwater Dam Floor Preparation

The stormwater dam will not have a liner system. The floor of the dam will however be proof rolled and unsuitable material (such as sand) removed and replaced with compacted engineered clayey material. It is expected that some seepage losses may occur through the floor of the dam and hence allowance has been made in the water balance for seepage.

4.2.4 Spillway Sizing

The stormwater dam must have an appropriately sized spillway in order to safely convey discharges associated with more extreme storm events, or multiple storms occurring within a short period, that may generate excessive overflows from the dam. In this case the spillway has been designed based on the 100 year ARI peak discharge for the upstream catchment.

For the assessment of the spillway sizing, the following method for approximating storage routing relationships from small storage basins, as defined in ARR (Pilgrim, 1987), has been applied to determine the peak outflow discharge:

$$Q_P = I_P \left[1 - \frac{S_{max}}{V_I}\right]$$

where, S_{max} is maximum volume of temporary storage above spillway level (m^3), I_P is the peak inflow discharge (m^3/s), Q_P is the peak outflow discharge (m^3/s) and V_I is the total volume of the inflow flood (m^3).

Based on the 100 year ARI design flood hydrographs for upslope catchment area, with an estimated peak inflow discharge (Q_P) of 7.7 m^3/s, and the available temporary storage above the spillway level within the pond assuming a flood storage depth of 0.5 m, there is likely to be only minor attenuation in the inflow peak discharges. A freeboard of 0.25 m has been assumed to the crest of the dam above the 100 year ARI flood storage depth. Assuming a broad-crested weir type spillway, a maximum temporary flood storage depth of 0.5 m during the design flood event and spillway width of 10 m, the peak outflow discharge is estimated to be approximately 6.2 m^3/s. The design parameters for the stormwater dam spillway are presented in Table 8.

Table 8: Recommended Spillway Dimensions

Spillway Component	Dimension
Spillway Width (m)	10 m
Maximum Flood Depth (m)	0.5 m
Peak Inflow Discharge (m^3/s)	7.7 m^3/s
Peak Outflow Discharge (m^3/s)	6.2 m^3/s
Freeboard Allowance (m)	0.25 m

4.2.5 Potential Impact on Downstream Flows

As part of the water modelling analyses, an assessment of the potential impacts of the development and operation of the stormwater dam on the downstream creek has been carried out. It should be noted that the natural streamflow response of the local creek is based on assumed rainfall-runoff model parameters as there are currently no available measured flow data for the upstream catchment on which to base a model rainfall-runoff calibration. Therefore, the assessment presented here of the relative downstream impacts of the stormwater dam operation is high level.

Modelled monthly average natural and impacted streamflows, based on the 25 year simulation period, for the creek reach directly downstream of the proposed stormwater dam are presented in Figure 8. These data highlight the significant seasonal variation in streamflow responses ranging from extremely low flows through the summer dry season (October-May) compared to the relatively high winter season flows (June-September).

As a result of the development and operation of the stormwater dam the simulations indicate the impoundment will potentially intercept all local runoff during the months of November through to May. Assuming that the autumn/winter runoff response is sufficient to fill the stormwater dam, the months of July through to October show a slight reduction in the runoff volumes in the creek reach directly downstream of the dam during periods when the water storage is predicted to be at a full level and excess water overflows the spillway. On average, the most significant relative impact on downstream flow is predicted to occur during May, June and October when the dam storage is being replenished. During particularly dry years when the winter runoff may not be sufficient to fill the dam storage, i.e. 2001, 2010 and 2012 (see Figure 7), there may be no surface flow in the creek immediately downstream of the dam.

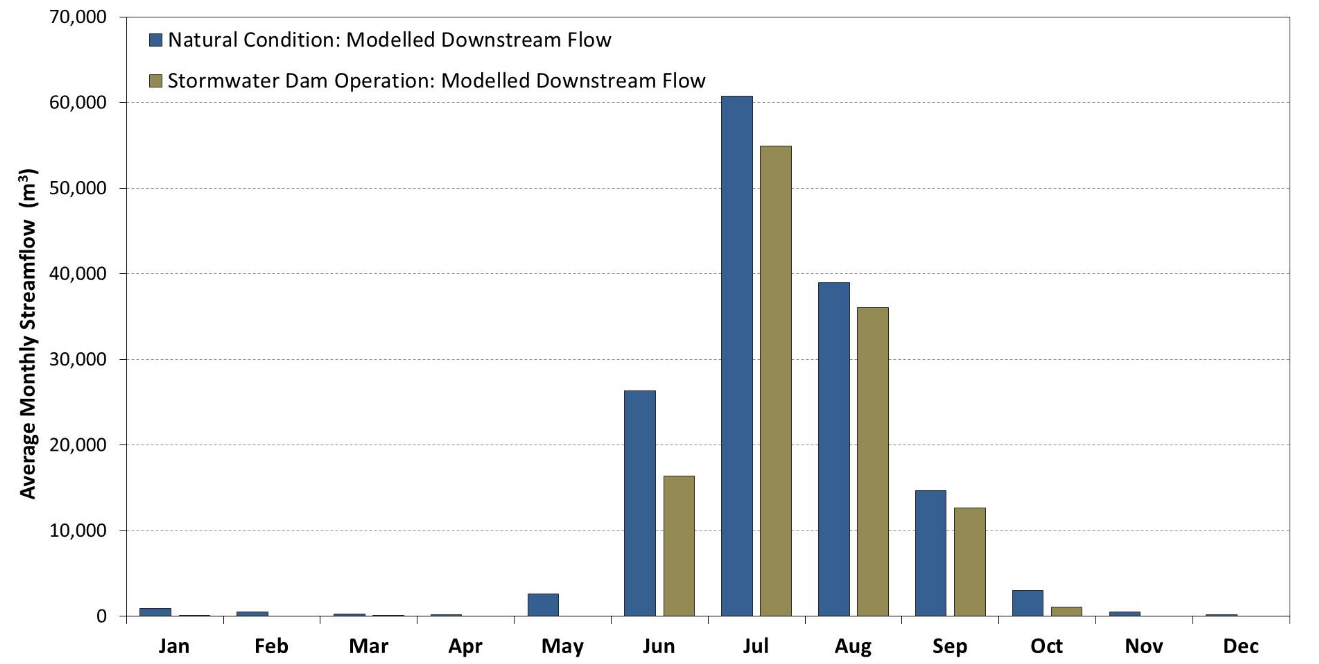

Figure 8: Modelled Average Monthly Streamflow: Impact of Storage Impoundment

4.3 Local Flood Risk Assessment

In addition to the assessment of the potential impact on average flows, a flood risk assessment has been carried out for the creek reach downstream of the stormwater dam to the confluence with the Thirteen Mile Brook. The estimated 100 year ARI design condition adopted for the design of the dam spillway has been used to identify the potential risk of flooding of landfill infrastructure and impacts on operation, i.e. road alignments, etc. It should be noted that the sediment control structure proposed to be located downstream of the stormwater dam is not proposed to be a permanent water retaining structure and will be appropriately designed to allow the bypass of extreme flood events.

Site-specific 2-dimensional (2D) hydraulic modelling of the study reach was undertaken using XPSWMM (XP Solutions, 2014), an integrated software package capable of simulating rainfall-runoff processes and the hydraulic performance of constructed/natural drainage systems. The existing topographic survey data were adopted to generate a digital terrain model (DTM) of the study reach and adjacent floodplain extents required to define the 2D hydraulic modelling domain. The defined model domain applies a modelled grid cell size of 1.0 m.

The estimated peak design flood discharge of 6.2 m^3/s, estimated as the 100 year ARI peak flood discharge from the dam spillway (described in Section 4.2.4), has been defined as the steady-state inflow to the hydraulic model at the upstream boundary. In the absence of flood levels for the downstream model boundary at the confluence of the local creek with Thirteen Mile Brook an assumed constant water level of approximately 3 m above creek invert level has been adopted. The application of this relatively conservative downstream boundary condition has been reviewed and does not have a significant influence on the upstream flood levels.

The modelled 100 year ARI flood extent and depths are presented in Figure 9 relative to proposed landfill infrastructure developments, i.e. stormwater dam (green), proposed road alignment (red) and landfill extent (light blue). Based on the existing topographic survey and road alignment it appears that the maximum flood extent for the 100 year ARI event is not expected to directly impact the proposed road alignment. Directly adjacent to the road alignment modelled flow depths and velocities are low, i.e. less than 0.5 m/s.

However, it would be recommended that additional profiling, stabilization and/or raising of the road profile along the potentially most-at-risk section be considered in order to increase the stability, integrity and serviceability of the road through the life of the landfill.

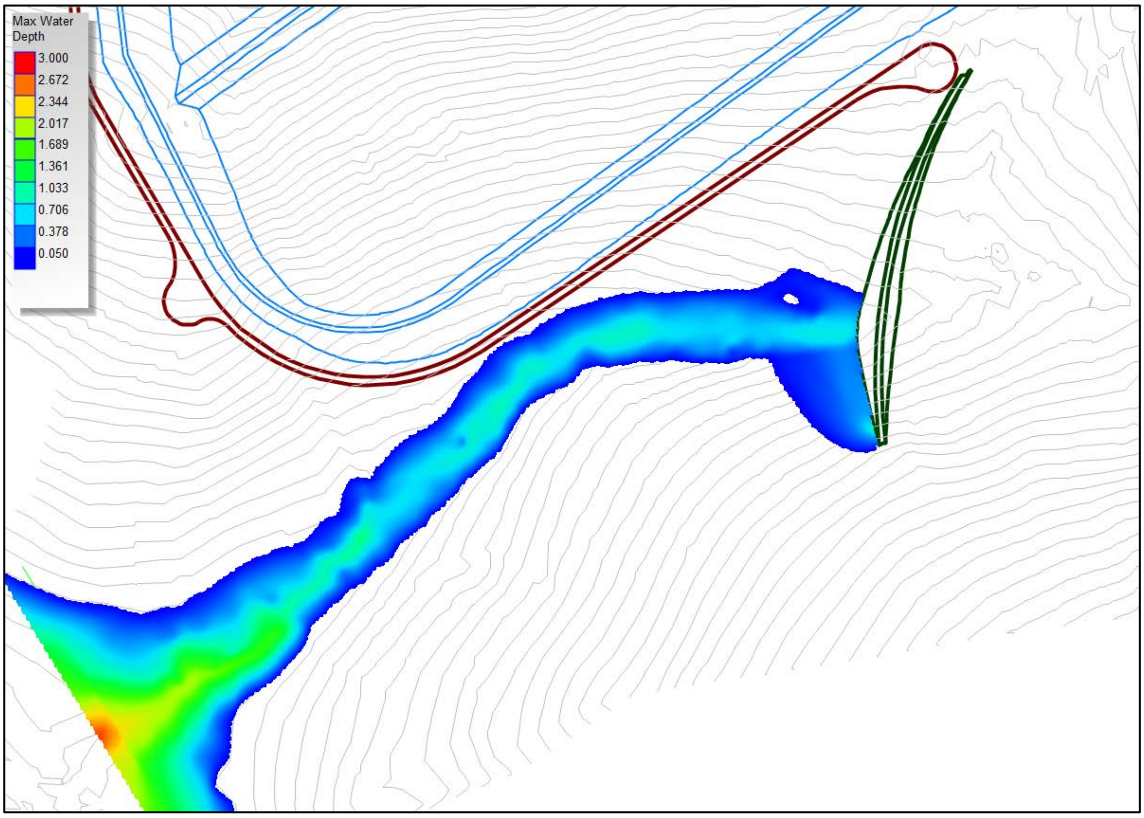

Figure 9: Modelled 100 Year ARI Flood Extent and Depth below the Stormwater Dam

4.4 Stormwater Diversion Bunds and Drains

Stormwater drains and swales are recommended to serve as the principal stormwater conveyance and surface runoff management system for the landfill site. The diversion bunds and drains aim to maximise the contributing catchment area for the stormwater dam as well as minimising the risk of uncontrolled stormwater runoff entering the operational landfill site from upslope areas. The upstream catchments are small and surface runoff responses are likely to be sheetflow runoff during significant storm events. Therefore, nominal diversion infrastructure is recommended, consisting of 0.5 m bunds with a diversion drain aligned with the upslope edge of the bund to control and divert runoff to the stormwater dam.

The arrangement of the proposed stormwater diversion system and contributing catchment to the stormwater dam during the operation of Cells 1 and 2 are shown in Figure C4 (Appendix C). Typical sections for the diversion bund/drain infrastructure are also presented. The proposed drains are aligned around the perimeter of the active landfill area providing a drainage pathway to the stormwater dam (described above).

4.5 Sediment Management

4.5.1 General

Sediment management requirements, specifications and designs have generally been based on the approaches recommended by the International Erosion Control Association (IECA, 2008). For the control of sediment likely to discharge from the Allawuna Landfill site the overall erosion and sediment control strategy therefore comprises:

- Drainage control measures aimed at preventing or reducing soil erosion caused by concentrated flows and to appropriately manage the movement and separation of ‘clean’ and ‘impacted’ water through the site; These are directly related to the development of the surface water management plan (SWMP) for the site.
- Erosion control measures aimed at preventing or reducing soil erosion caused by rain drop impact and sheet flow (i.e. the control of splash and sheet erosion).
- Sediment control measures aimed at trapping and retaining sediment either moving along the land surface (bed load) or contained within flowing water (suspended sediment).

4.5.2 Sediment and Erosion Management

Sediment from the site will be managed through a combination of sediment management options. These will include:

- Sediment fences
- Rock or sandbag check dams
- Sediment Management Structure (SMS)

The SMS is intended to capture sediments that escape from the other sediment capturing systems on the site and to prevent sediment from entering the Thirteen Mile Brook.

4.5.3 Design of Sediment Management Structure

A sediment management structure (SMS) is proposed to be constructed downstream of the stormwater dam and upstream of the inflow confluence to the Thirteen Mile Brook to minimise the release of sediment eroding from the landfill site to the downstream environment. The location of the SMS is shown in Figure C4 (Appendix C). The sediment impacted runoff from the landfill cells will discharge south through the culverts under the access road.

The primary function of SMSs is to retain sediment conveyed in the inflowing surface runoff above a defined particle size, not simply to retain a defined volume of water relating to a given ARI storm. It is therefore essential that the pond has the capability of controlled release of retained water prior to the inflow of a later event. This allows the settling zone within the pond to be restored to its full capacity within the required period.

If the capacity of the storage is not exceeded following a storm event conveyed sediment would be retained, while inflow will be allowed to gradually drain through the embankment. Should multiple storms occur within a short period, the inflow volume exceeding retention capacity would discharge over the rock embankment, which will act as a broad crested weir. The inflowing sediment above the recommended particle size would, however, still be deposited within the sediment storage zone of the pond.

The embankment is proposed to be a rock filter embankment constructed of aggregate with a particle size varying between 250 mm and 500 mm.

This rock filter will provide passive drainage of water retained in the pond over a period of hours. The design therefore allows for the controlled release of inflows to ensure the maximum available storage capacity can be maintained within the SMS.

5.0 SUBSOIL DRAINAGE MANAGEMENT

Seepage has been observed within areas of the proposed construction footprint for Cells 1, 4 and 6, therefore, the primary purpose of the subsoil drainage system is to reduce the impacts of phreatic surface mounding beneath the cell floor, i.e. preventing pressurization of the basal liner system from below, and the accumulation of pore pressures in the embankment fill.

5.1 Subsoil drainage system

5.1.1 Design overview

The subsoil drainage system (SDS) for the Allawuna Landfill will comprise a network of drainage pipes laid in seepage interception trenches and connected to a common sump. The trenches will be excavated into the stripped ground surface and fall at a grade determined by the topography, expected to be between 1% and 3%. The construction of the SDS will be progressive, preceding the construction of each cell serviced by the system.

The area to be serviced by the SDS is based on the area of intersection between potentiometric groundwater contours (consistent with hydrogeological studies – Refer report 147645033-009-R) and pre-construction topography. This equates to an area of approximately 271 600 m^2 across Cells 1, 4 and 6, representing the portions of the subgrade which may be subject to periodic groundwater seepage. This area is shown in Figure 10.

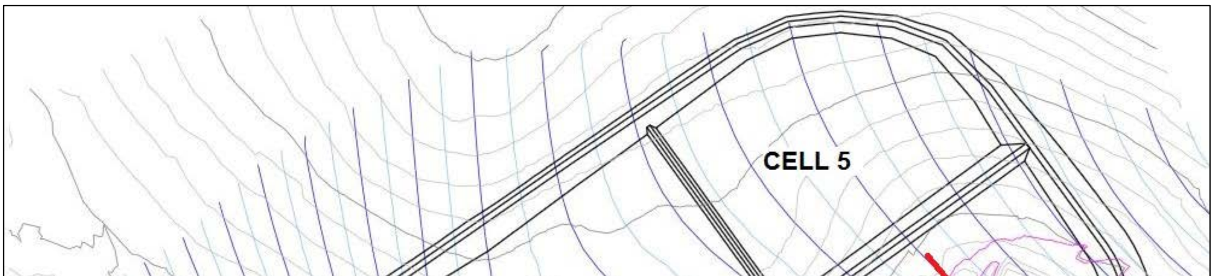

Figure 10: Pre-Construction Topography (Grey) Overlay with Potentiometric Groundwater Contours (Blue) and Area of Intersection (Magenta), Proposed Embankments Shown for Illustration

Figure 10 shows that Cell 6 has the largest area subject to seepage. A section through Cell 6 has therefore been adopted as the basis of design, as a conservative approach to provide significant contingency drainage for Cells 1 and 4. No SDS is proposed for Cells 2, 3 and 5, since the groundwater surface is significantly lower than the embankment design levels.

5.1.2 Approach and Staging

The approach taken to design the SDS was to prepare a seepage model using the finite element code SEEP/W[1] to estimate the required spacing of seepage interception trenches. This spacing was modified until a minimum distance of 2.5 m is maintained between the cell floor and the phreatic surface mound between drainage trenches. This spacing would then be applied consistently throughout the system.

The model was staged as follows:

1) Steady-state, pre-construction for 100 days

2) Transient post-construction for 1440 days

5.1.3 Trench Geometry

We have assumed a trench geometry of 0.7 × 0.6 m (W×D) for this assessment. Sensitivity analyses performed throughout the model development process indicate the phreatic conditions are not sensitive to changes in trench geometry.

5.1.4 Model Inputs

The following boundary conditions were applied to the model:

- Upper boundary (constant head): 323 m RL
- Lower boundary (constant head): 321 m RL

The *potential seepage face* boundary condition in SEEP/W was applied to the trench boundaries. Flux sections were placed inside the trenches to record the daily peak fluxes into the drainage system over the model duration.

Material properties used in the model were estimated based on the results of Golder geotechnical and hydrogeological studies and are summarised in Table 9.

Table 9: Summary of Material Properties Applied in Subsoil Drainage Model

Material	Model	Parameter	Value	Unit
Embankment fill	Saturated/Unsaturated	K_{sat} (isotropic)	1×10^{-8}	m/s
		Activation pressure	-100	kPa
Subgrade	Saturated only	Kh_{sat}	1×10^{-6}	m/s
		Kv_{sat}	1×10^{-7}	m/s
Interception trench backfill	Saturated only	K_{sat} (isotropic)	1×10^{-2}	m/s

The soil-water characteristic curve (SWCC) of the embankment fill material was estimated using the methods proposed by Perera and Zapata (2007) based on the average PSD and plasticity of the proposed materials. The graph of estimated SWCC for the embankment fill is presented in Figure 11.

[1] *Geostudio 2007* Version 7.23 Copyright © 1991-2013 Geo-Slope International, Ltd.

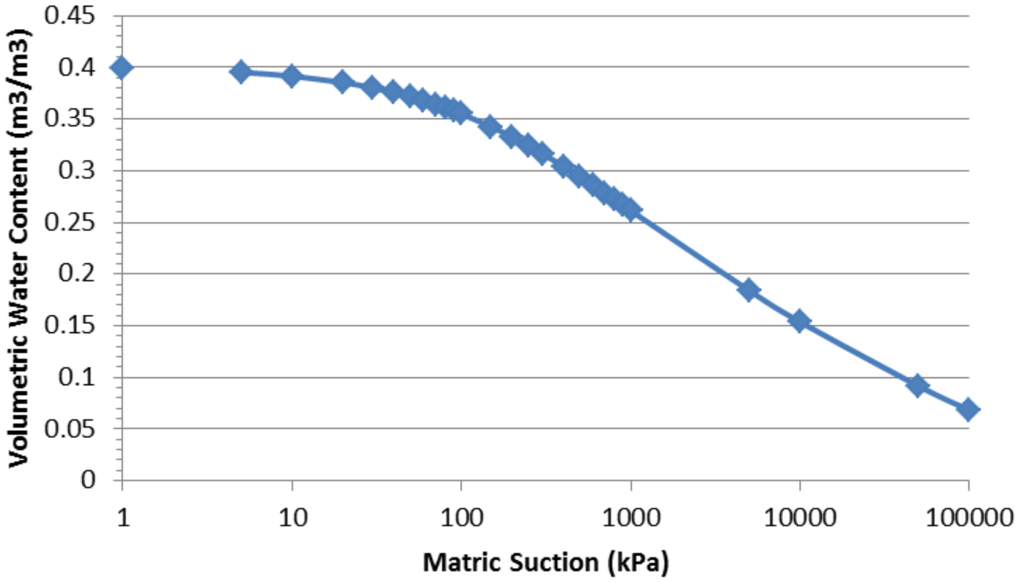

Figure 11: Estimated Soil Water Characteristic Curve for Embankment Fill Material

5.1.5 Assessment outcomes

The results of the seepage model and pipe flow calculations are summarised in Table 10

Table 10: Results of subsoil drainage model

Result	Value
Peak flow through drainage pipe – Stage 1AB	11.1 m^3/day*
Peak flow through drainage pipe – Final Landform	47.0 m^3/day*
Required minimum pipe spacing	40 m

*Flux corresponds to steady-state conditions assuming winter maximum groundwater level

Manning's equation for circular pipes was used to estimate the required minimum pipe diameter based on the peak flows for the 'final landform' condition above. Manning's equation is given by

$$Q = AR^{2/3}S^{1/2}/n$$

where Q is the maximum flow for the final landform (47 m^3/day), A is the section area of the pipe, R is the hydraulic radius (based on a water height of half the diameter), S is the slope of the pipe, (assumed to be 1%) and n is the roughness coefficient, which for HDPE drainage pipes has been assumed to be 0.025.

The minimum pipe diameter calculated from the above is 71 mm. We therefore propose 110 mm OD HDPE pipes spaced at 40 m centres throughout the identified seepage area for Cell 1.

The seepage areas for Cell 4 and 6 will be monitored and adjusted to suit site conditions prior to construction. The seepage collection pipes will be connected via a header pipe at the toe of the embankment to a collection sump, from where it will be pumped to the retention pond. The sump will be fitted with a permanent pump and level switch to allow automatic pumping to the retention pond should the water level in the sump build up to a height above the inflow pipes.

5.2 Retention Pond

5.2.1 Retention Pond Sizing

The location of the proposed retention pond is shown in Figure C4 (Appendix C). We have assumed that the retention pond receives water from the subsurface drainage system (via pumping).

A detailed assessment of the retention pond capacity requirements cannot be made directly from available subsoil drainage information available at this stage as the pipe sizing estimates described in section 5.1 is based on peak flow and not average flow. An initial retention pond size of 2000 m^3 is therefore proposed. Subsoil drainage water management and storage requirements will be monitored and reviewed throughout the operation of the Allawuna Landfill. Development of additional retention pond storage capacity or variations to the operational water management plan will be carried out, if necessary, based on an analysis of the recorded data. At this stage, it is assumed that the water stored within the retention pond will be managed and controlled primarily through direct evaporation losses with alternative water management options, such as use for dust suppression, to be implemented if there is an ongoing accumulation of stored water.

Discharge of water from the retention dam should only occur after confirmation that the water is not contaminated. This confirmation should at least be visual where the only possible contaminant source is sediment, but where other contaminants are possible, the water should be tested prior to discharging. The degree of testing will be determined by the risk of contamination and the sensitivity of the receiving environment. Water should not be discharged if suspected or found to be contaminated (EPA, 2014). The testing and stormwater discharge procedure should be defined based on the operational strategy for the landfill and will be dependent upon the nature of material being disposed of in the landfill cells.

Where discharge from the retention pond is permitted, this should be carried out through pumping of water from the pond. It is recommended that uncontaminated stormwater should be utilised for dust suppression, when possible, or pumped to the stormwater dam prior to release to the downstream environment over the spillway.

5.2.2 Retention Pond Liner System

The water quality on the retention pond is generally expected to be suitable for release to the environment, with the extent of contamination, should it occur, expected to be minimal. For this reason the proposed liner system for the retention pond consists of:

- 500 mm thick compacted engineered clayey fill material
- 2.0 mm smooth HDPE liner.

6.0 LEACHATE MANAGEMENT STRATEGY

The landfill design will incorporate a leachate collection system extending across the base of each stage and along the toe of the side walls. The leachate collection system will intercept vertical and lateral leachate seepage occurring through the waste. The leachate collection system will be designed in accordance with Vic BPEM (EPA, 2014).

The quantity of leachate produced within the landfill will typically be related to the amount of precipitation that percolates into or runs over deposited waste within the open working area and/or the amount that infiltrates through the final capped surface of the landfill. To minimise the amount of leachate produced, the landfill will be operated by keeping the exposed area of waste to a minimum with rehabilitation following shortly after completion of filling each cell. The volume of leachate generated in the landfill will be influenced by the size of the stage and the operational procedures adopted. Measures to reduce leachate generation will include:

- Diversion of stormwater away from the active waste disposal area to reduce leachate generation
- Progressive capping.

It is proposed that the landfill will be filled in sub-cell areas resulting in a high rate of rise and low risk of waste saturation before being capped. For the purpose of this leachate management assessment the leachate generation potential is based on the development of landfill cells 1A, 1B, 2A and 2B. The operation of these cells is estimated to cover the initial 7 years of the life of the landfill. Estimated areas and operational periods of the modelled landfill cells assumed for the leachate generation modelling are shown in Table 11.

Table 11: Modelled Landfill Cell Area and Operational Periods

Landfill Cell	Area (ha)	Start Year	Active Operation Period (years)
1A	4.1	1	2
1B	2.2	3	1
2A	3.6	4	2
2B	1.9	6	2

6.1 Leachate Collection System

The leachate collection gravel layer on the landfill base is generally sloped at 3 % to promote drainage towards the two valleys adjacent to the south and north bund walls, which contains perforated leachate collection pipes. A network of perforated leachate collection pipes located at 20 m spacing across the floor of the landfill will also aid in directing leachate towards the valleys.

The leachate header pipes direct leachate towards the leachate collection sumps at a grade of at least 1%. Leachate will be removed from the sumps, in Cells 1 and 2, by progressive pumping via a leachate riser pipe to an on-site leachate storage pond. The objective of this assessment aims to provide sufficient on-site storage for collected leachate which can then be managed passively through evaporation losses (or enhanced evaporative options). Where excess leachate generation occurs above the design capacity of the leachate management system, collected leachate will generally be transferred offsite for treatment at a licenced treatment facility. Leachate may also be re-circulated into the landfill as an emergency management measure.

6.2 Managing Leachate Head Over Liner

The hydraulic head of leachate over the landfill liner will be managed during the landfill operation in accordance with the BPEM requirements through pumping of the leachate sump. Leachate levels on the landfill base will be maintained to a maximum level of 300 mm above the landfill base liner.

6.3 Water Balance and Leachate Generation Estimates

A water balance analysis was undertaken to estimate the quantity of leachate that may be generated within the landfill, and included assessment of leachate generation resulting from rainfall infiltration through uncapped waste, rainfall seepage through the interim cap and from rainfall seepage through the final landfill cap areas.

Simulations were carried out using the Hydrologic Evaluation of Landfill Performance (HELP) computer program to estimate the quantity and rate of leachate generated. The HELP program requires input of daily climate data (rainfall, temperature and solar radiation) and details of landfilling and the final capping system. The model outputs include precipitation, estimated runoff, evapotranspiration, infiltration rates and changes in stored water within the landfill waste mass.

The following three operational stage scenarios were modelled to estimate leachate generation rates for

- the active landfill,
- the interim capped, and
- the final capped landfill.

Vic BPEM (EPA, 2014) recommends that for the assessment of leachate management options, a water balance should be modelled over at least 2 consecutive wet years, defined as a 90th percentile wet year, to ensure that the proposed system has sufficient capacity to deal with all leachate generated over the operational life of the landfill.

For the purpose of this assessment of leachate generation the simulation was applied for a 7 year period (covering the operational stages of landfill cells 1a, 1b, 2a and 2b) experiencing consecutive wet years. The weather data applied in the HELP model is based on the 90th percentile rainfall year (1995 – 736 mm), as assessed from the SILO rainfall series for the period between 1961 and 2014. The SILO rainfall data adopted for the leachate water balance assessment is described in more detail in Section 3.0 and the monthly rainfall for 1995 (90th percentile equivalent) is presented in Figure 12 relative to the 1961 to 2014 statistics. Rainfall records prior to 1961 were excluded as the data were considered to be less representative of the current (and future) climate in the region.

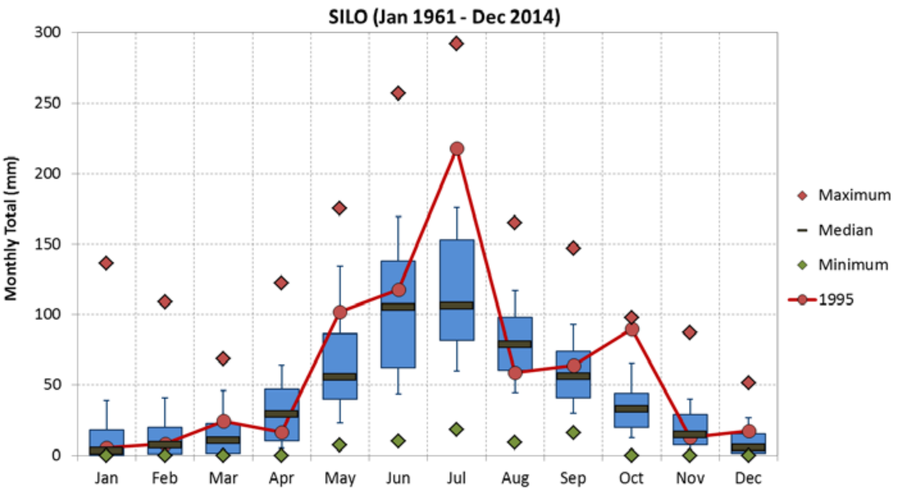

Figure 12: SILO Rainfall for 1995 (90th Percentile Year) Compared to 1961-2014 Average Monthly Rainfall Distribution

In addition to the defined rainfall series, the HELP software was used to create a synthetic climate record for the York and York Post Office weather stations (operational only up to 1996). Climate variables of temperature, solar radiation, wind speed and humidity do not experience significant spatial variability across the region, unlike rainfall (see Section 3.2), and therefore the York datasets are assumed to be representative of the Allawuna Landfill site.

The application of the dynamic leachate generation estimates over the 7 year operational period allows the maximum (critical) leachate generation water balance period to be identified for the purpose of sizing leachate ponds and management options.

6.3.1 Modelling Assumptions

- Waste filling will occur on a phase-by-phase basis with one active phase open at any one time.
- The final cap will be constructed over each separate stage once waste filling has been completed in the stage.
- The interim cap shall comprise of a 300 mm thick clay rich layer with a coefficient of permeability of 1×10^{-8} m/s.
- Runoff from the surface of deposited waste is classified as leachate
- Runoff from the interim cap and final landfill cap is classified as stormwater.
- A general landfill floor slope of 3% with leachate drains at 20 m spacing.
- The hydraulic conductivity of waste was assumed to be 10^{-5} m/s.
- The waste entering the landfill was assumed to be at 5% dry of Field Capacity[2].
- The interim cover soil overlying the waste was considered as bare ground.
- An evaporative zone depth of 250 mm was assumed for the bare ground surface.
- A surface slope on the interim cap of 4% over 20 m length with a runoff permitted from the interim cover soils of 100% of the landfill cell area.
- For the High Density Polyethylene (HDPE) liner the following were assumed:
  - Placement quality of 3 (Good)
  - Pinhole Density of 2 hole per ha
  - Installation Defects of 2 holes per ha
- The hydraulic conductivity of GCL as manufactured is typically 3×10^{-11} m/s. Field observations of the performance of GCL based composite liners suggest the permeability of these liners may increase by approximately an order of magnitude due to cation exchange processes that may occur following installation. The hydraulic conductivity of GCL adopted for the simulation was adjusted to 3×10^{-10} m/s for potential cation exchange effects.
- Design precipitation falling on the landfill is based on consecutive 90th percentile annual rainfall events.
- The thickness of the waste profile is 25 m.

6.3.2 Estimated Leachate Generation Rates

The estimated HELP modelled water balance and estimated leachate generation rates for year 1 to 7 covering the operational stages of landfill cells 1a, 1b, 2a and 2b are summarised in the following Table 12 to Table 18. It should be noted that the HELP model internally generates a synthetic rainfall input based on the defined 90th percentile rainfall year (736 mm) and does not explicitly replicate the design event. All leachate generation rates are summarised as a rate per hectare (L/ha/d), for relative comparison, and are summarised as estimated annual rates based on the landfill development schedule in Table 19.

[2] Field capacity is defined as the water content reached if a sample of the waste is initially saturated and then subjected to prolonged free drainage.

Table 12: Cell 1a Uncapped Waste Profile

Year	Precipitation (mm)	Change in Water Storage within Waste (m^3)	Lateral drainage collected at the base (m^3)	Runoff (m^3)	Leachate Generation (L/ha/d)
1	746.8	4317.8	0.0	167.5	460
2	801.8	4777.0	0.0	208.5	570

Table 13: Cell 1b Uncapped Waste Profile

Year	Precipitation (mm)	Change in Water Storage within Waste (m^3)	Lateral drainage collected at the base (m^3)	Runoff (m^3)	Leachate Generation (L/ha/d)
3	746.8	4317.8	0.0	167.5	460

Table 14: Cell 2a Uncapped Waste Profile

Year	Precipitation (mm)	Change in Water Storage within Waste (m^3)	Lateral drainage collected at the base (m^3)	Runoff (m^3)	Leachate Generation (L/ha/d)
4	746.8	4317.8	0.0	167.5	460
5	801.8	4777.0	0.0	208.5	570

Table 15: Cell 2b Uncapped Waste Profile

Year	Precipitation (mm)	Change in Water Storage within Waste (m^3)	Lateral drainage collected at the base (m^3)	Runoff (m^3)	Leachate Generation (L/ha/d)
6	746.8	4317.8	0.0	167.5	460
7	801.8	4777.0	0.0	208.5	570

Table 16: Cell 1a Waste Profile with Interim Cap

Year	Precipitation (mm)	Change in Water Storage within Waste (m^3)	Lateral drainage collected at the base (m^3)	Runoff (m^3)	Leachate Generation (L/ha/d)
3	746.8	2070.6	0	2809.7	0

Notes: [1]The moisture content of the waste at the end of the 2nd year (0.2784) has been considered the initial waste moisture content at the start of the 3rd year.

[2]We have assumed that the Cell 1a interim cap will be active for 1 year.

Table 17: Cell 1b Waste Profile with Interim Cap

Year	Precipitation (mm)	Change in Water Storage within Waste (m^3)	Lateral drainage collected at the base (m^3)	Runoff (m^3)	Leachate Generation (L/ha/d)
4	746.8	2070.6	0	2809.7	0

Notes: [1]The moisture content of the waste at the end of the 3rd year (0.2784) has been considered the initial waste moisture content at the start of the 4th year.

[2]I have assumed that the Cell 1a interim cap will be active for 1 year.

Table 18: Cell 1a Waste Profile with Final Cap

Year	Precipitation (mm)	Change in Water Storage within Waste (m^3)	Lateral drainage collected at the base (m^3)	Runoff (m^3)	Leachate Generation (L/ha/d)
4	746.8	357.0	44.0	398.6	120
5	801.8	268.8	0.00	369.9	0.0

Notes: [1]The moisture content of the waste at the end of the 3rd year with interim cap (0.2868) has been considered the initial waste moisture content at the start of the 4th year.

Table 19: Estimated Annual Leachate Generation Rates

Year	Cell 1a (m^3/ha)	Cell 1b (m^3/ha)	Cell 2a (m^3/ha)	Cell 2b (m^3/ha)	Total (m^3/ha)
1	167.5	NA	NA	NA	167.5
2	208.5	NA	NA	NA	208.5
3	0.00	167.5	NA	NA	167.5
4	44.0	0.00	167.5	NA	211.5
5	0.00	44.0	208.5	NA	252.5
6	0.00	0.00	0.00	167.5	167.5
7	0.00	0.00	44.0	208.5	252.5

6.3.3 Leachate Storage Pond Sizing

It is intended that leachate will be directed by pumping to a leachate storage pond located to the north of the landfill, as indicated in Figure C4 (Appendix C). The leachate storage pond will be constructed prior to the initial operation of Cell 1a. The sizing of the leachate storage pond considers a scenario of the maximum storage required for the number of closed, interim capped and operational cells, based on the estimated leachate generation rates defined in Table 19.

Additionally the water balance assessment for the leachate pond sizing includes the water volume resulting from incident rainfall on the leachate storage pond as well as direct evaporation losses. The 90th percentile rainfall year (1995) is applied as the rainfall input and the estimated open water evaporation (E_{OD}) profile (see Section 3.3) has been adopted to account for evaporation losses from the pond. The leachate pond will be externally bunded to prevent runoff entering the pond from surrounding upslope areas.

Based on the estimated leachate generation rates and cell development schedule presented in Table 19, the maximum 2 year leachate production periods of the presented scenarios are:

- **Year 4-5**: Total leachate generation volume of approx. 1750 m^3, and
- **Year 1-2:** Total leachate generation volume of approx. 1500 m^3.

Applying these leachate generation rates to the leachate pond water balance, using the GoldSim modelling platform, results in an operational storage requirement of around 2500 m^3, including the assumption that the leachate pond storage starts with 1000 m^3 of leachate storage at the start of the Year 4-5 scenario (see Figure 13). Therefore, a leachate storage pond storage requirement would have approximate dimensions of 40 m × 40 m with a capacity of around 2500 m^3 at 2.0 m depth and approximately 3500 m^3 at 2.5 m depth including a recommended 0.5 m freeboard.

Ongoing monitoring of leachate generation rates will be carried out as the site develops in order to ensure that sufficient leachate storage capacity is available and that the leachate management strategy remains robust and effective over the life of the landfill.

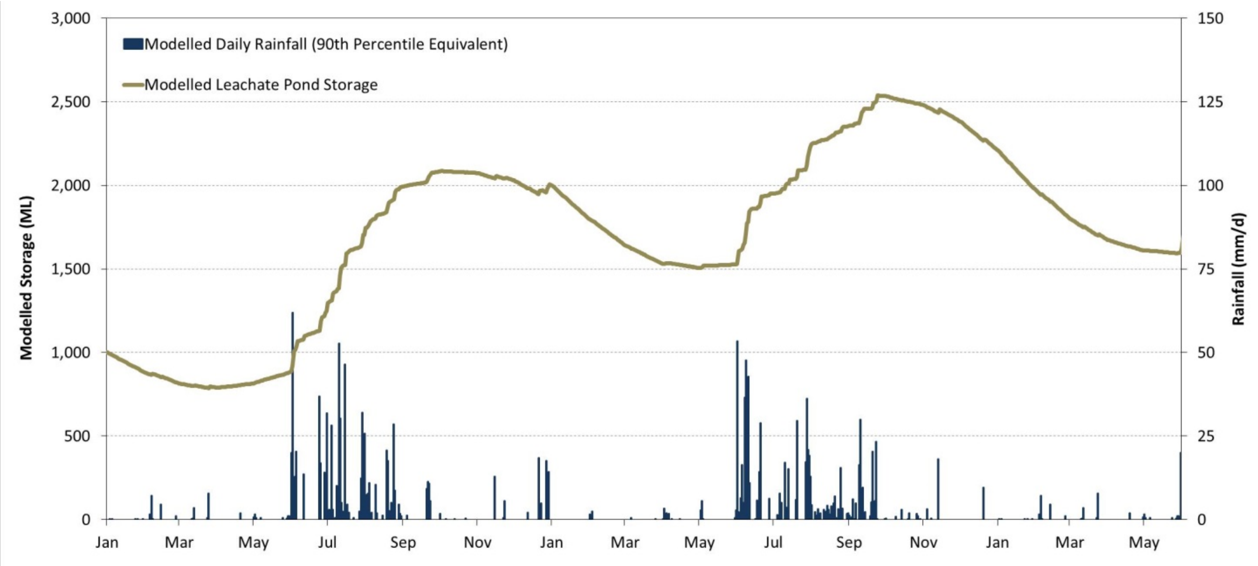

Figure 13: Modelled Leachate Pond Water Balance (Year 4-5 Scenario with Consecutive 90th Percentile Rainfall Years)

6.3.4 Leachate Pond Liner System

The proposed liner system for the retention pond is to the same standard as the liner system for Cell 1 and Cell 2 and consists of:

- 500 mm thick compacted engineered clayey fill material
- Geosynthetic clay liner
- 2.0 mm smooth HDPE liner.

6.4 Operational Leachate Management Plan and Monitoring Strategy

- Should the capacity of the leachate pond be exceeded, the retention pond can be used to contain leachate as the liner systems for both ponds are the same. This would require the temporary placement of a pump and pipe to transfer leachate from the leachate pond to the retention pond.
- A leachate management plan will be produced for the leachate and retention pond prior to construction of these ponds to capture the management strategy for these ponds.

7.0 RECOMMENDATIONS

Based on the detailed water management assessments outlined in this document, the following stormwater and leachate management infrastructure requirements have been identified for the Allawuna Landfill site:

- A Leachate Pond with a minimum operational storage capacity of at least **2500 m^3 (**a minimum 0.5 m freeboard should be applied to the pond design above the operational capacity), suitable for the operation of Cell 1 and Cell 2.
- The requirement for additional leachate storage capacity associated with the development of future cells should be assessed prior to cell construction. Should additional leachate storage capacity be required, the Works Approval Application for the relevant cells should include the development of another Leachate Pond.
- A Retention Pond with a recommended storage capacity of at 2000 m^3 (a minimum 0.5 m freeboard should be applied to the pond design above the operational capacity). Retention pond capacity requirements for the storage and management of subsoil drainage will be monitored and assessed continuously during the operation and further development of the landfill.
- A Stormwater Dam with a storage capacity of approximately **36 000 m^3** in order to provide water supply requirements, i.e. dust suppression, firefighting, construction water etc. A water resource operation and management plan will be developed in order to minimise the risk and impact of water deficits during dry years based on surface water flow and level data collected during site development and operation.
- Diversion bunds and interception drains will be required to control surface water runoff from surrounding areas and ensure that clean stormwater runoff remains separated from potentially impacted runoff within the landfill site. Nominal diversion bunds with a minimum height of 0.5 m are proposed to divert and control stormwater runoff along the eastern edge the landfill site and discharge to the stormwater dam.
- A sediment control pond is proposed to be constructed downstream of the stormwater dam and upstream of the inflow confluence to Thirteen Mile Brook to minimise the release of sediment eroding from the landfill site to the downstream environment.
- The alignment of diversion bunds and drains should be reviewed and adapted in a staged manner as the landfill site and operational cells develop. The external diversion bund alignments should be located in areas to maximise stormwater runoff to the stormwater dam and minimise runoff entering operational areas.
- The recommended arrangement of the proposed stormwater, sediment and leachate management infrastructure (described above) for the Allawuna Landfill during the operation of Cells 1 and 2 are shown in Figure C4 (Appendix C).

It should be noted that stormwater management and conveyance systems associated with road alignments and other landfill infrastructure developments, excluding the landfill cells, are not covered in this document.

7.1 Ongoing Assessments and Monitoring

The following water monitoring and surface water management assessments will be carried out in order to assess the reliability and effectiveness of the proposed stormwater management system and to assist in the development of an effective operational leachate management plan:

- Stream gauging and/or continuous flow monitoring will be required for the creek alignment flowing through the proposed stormwater dam location. These measurements should be carried out through at least one winter period (preferably starting in autumn/winter 2015). At this time the potential surface water runoff from the catchment upslope of the stormwater dam, described in Section 4.2, is based solely on assumed runoff responses for the contributing catchment which cannot be validated with site specific data.
- Construction water demand requirements, water availability and sources of supply will be addressed through an options study that will be carried out after the submission of the WAA. This options study will provide a more detailed assessment of the availability of construction water and the potential options available to supply suitable quantity and quality of construction water.
- Assessment of emergency and contingency water supply sources which may be utilised during extreme dry periods, i.e. groundwater bores. These may be linked to the findings of the construction water supply options assessment.
- Definition of an operation water management plan to define control conditions for water supplies, i.e. monthly or seasonal trigger and target water levels for the stormwater dam.
- Requirements for additional water storage capacity, further development of surface water management measures and the performance of the proposed drainage and sediment management infrastructure will be monitored and assessed continuously during the operation and further development of the landfill.

Water management assessments and operational planning investigations will be developed over time during the development and operation of the Allawuna Landfill site. All operational plans relating to water management will be required to apply to best practice management practices as well as align with conditions and requirements defined by SITA's operation licence.

8.0 REFERENCES

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Report Signature Page

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APPENDIX A

Australian Water Balance Model (AWBM) Description

GENERAL

Runoff was derived using the AWBM daily rainfall-runoff model (Boughton et al, 2003). The structure of the model is shown in Figure A1, which also provides definitions of the various model parameters.

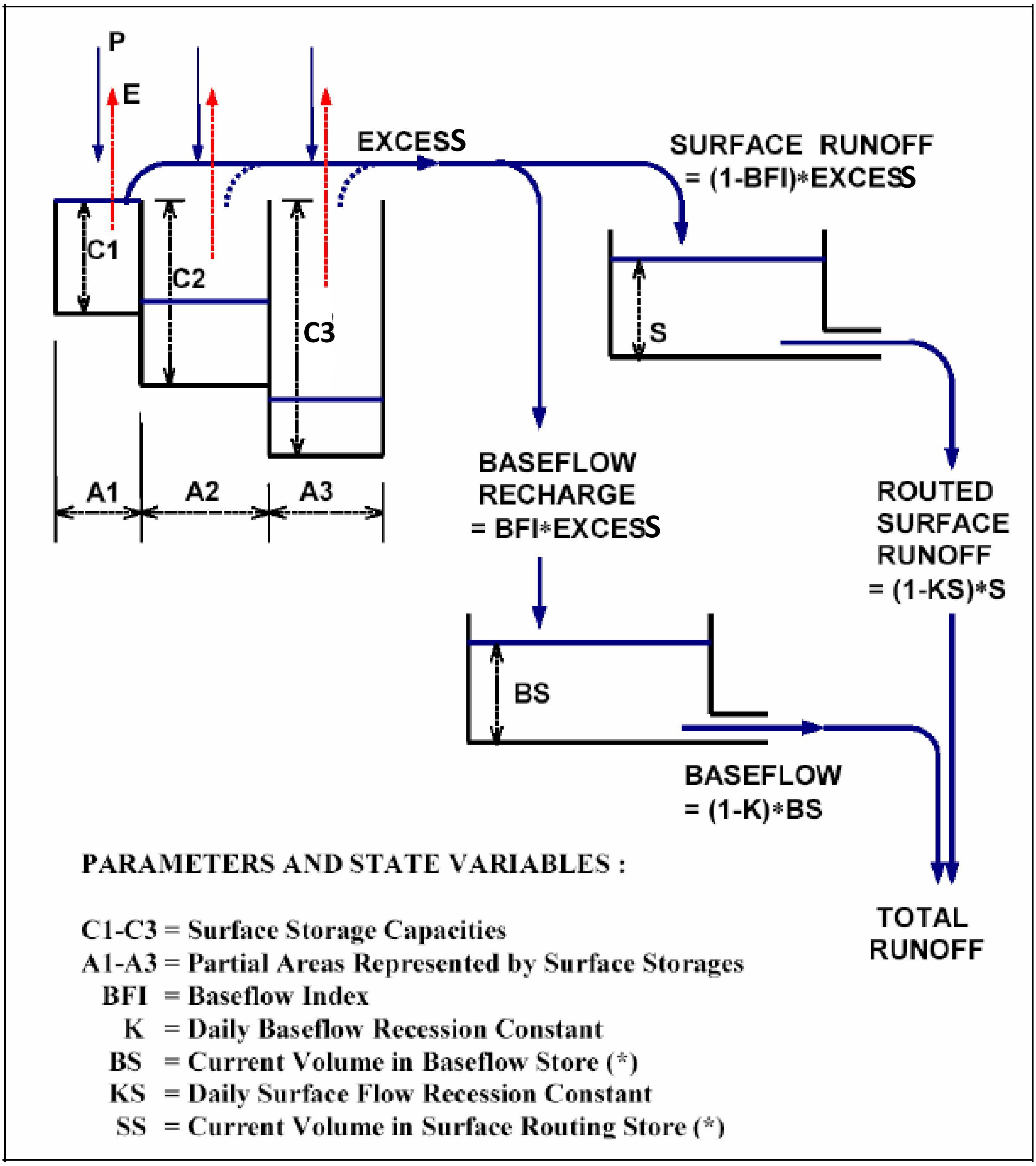


Figure A1: Structure of AWBM Rainfall-Runoff Model

As illustrated in Figure A1, the AWBM model uses three moisture stores to simulate the generation of excess rainfall within the catchment. The model calculates the moisture balance of each individual store (or partial area of the catchment) at daily intervals. At every time step, rainfall is added to each of the three stores C1, C2, and C3 while evaporation is subtracted. Thus, the daily water balance for these stores is given by:

$$store_n = store_{n-1} + rainfall - evaporation \ (n = 1 \text{ to } 3)$$

In the event of the moisture value of a particular store reducing below zero, it is reset to zero. Similarly, if the moisture value exceeds its capacity, the amount by which the capacity is exceeded becomes excess rainfall and the moisture store is set to its capacity.

Multiplying the excess by the base flow index (BFI) defines the fraction of the excess that recharges the baseflow store. The remaining portion of the excess is added to the surface runoff store. Outflows from the surface and baseflow stores occur on a daily basis (provided these stores are non-zero) with the outflows from each being a function of the surface flow and baseflow recession constants, respectively.

The overall runoff from a catchment is then the summation of surface flow plus baseflow. This resulting catchment runoff (in mm) is then converted to a daily discharge based on the overall contributing catchment area.

The AWBM model requires calibration of those model parameters listed in Figure A1 to provide reliable streamflow estimates for a catchment. As noted previously, no streamflow data are available in the landfill development area for model calibration. It was therefore considered valid to apply slightly modlfied default parameters which provide a good representation of the rapid runoff response and expected catchment yield of small headwater creeks in this region.

A summary of the AWBM parameters adopted for this site is presented in Table A1.

Table A1: Adopted AWBM Parameters

Parameter	Abbreviation	Value
Small storage capacity (mm)	C1	30
Medium storage capacity (mm)	C2	85
Large storage capacity (mm)	C3	150
Small partial area portion	A1	0.2
Medium partial area portion	A2	0.45
Large partial area portion	A3	0.35
Baseflow index	BFI	0.35
Baseflow recession factor	K	0.95
Surface flow recession factor	KS	0.35

APPENDIX B

Construction Water Demand Assumptions

SUMMARY

The following information and assumptions have been applied in the estimation of construction water demand requirements for the Allawuna Landfill water balance.

Table 1: Estimated Landfill Cell Construction Material and Water Requirements

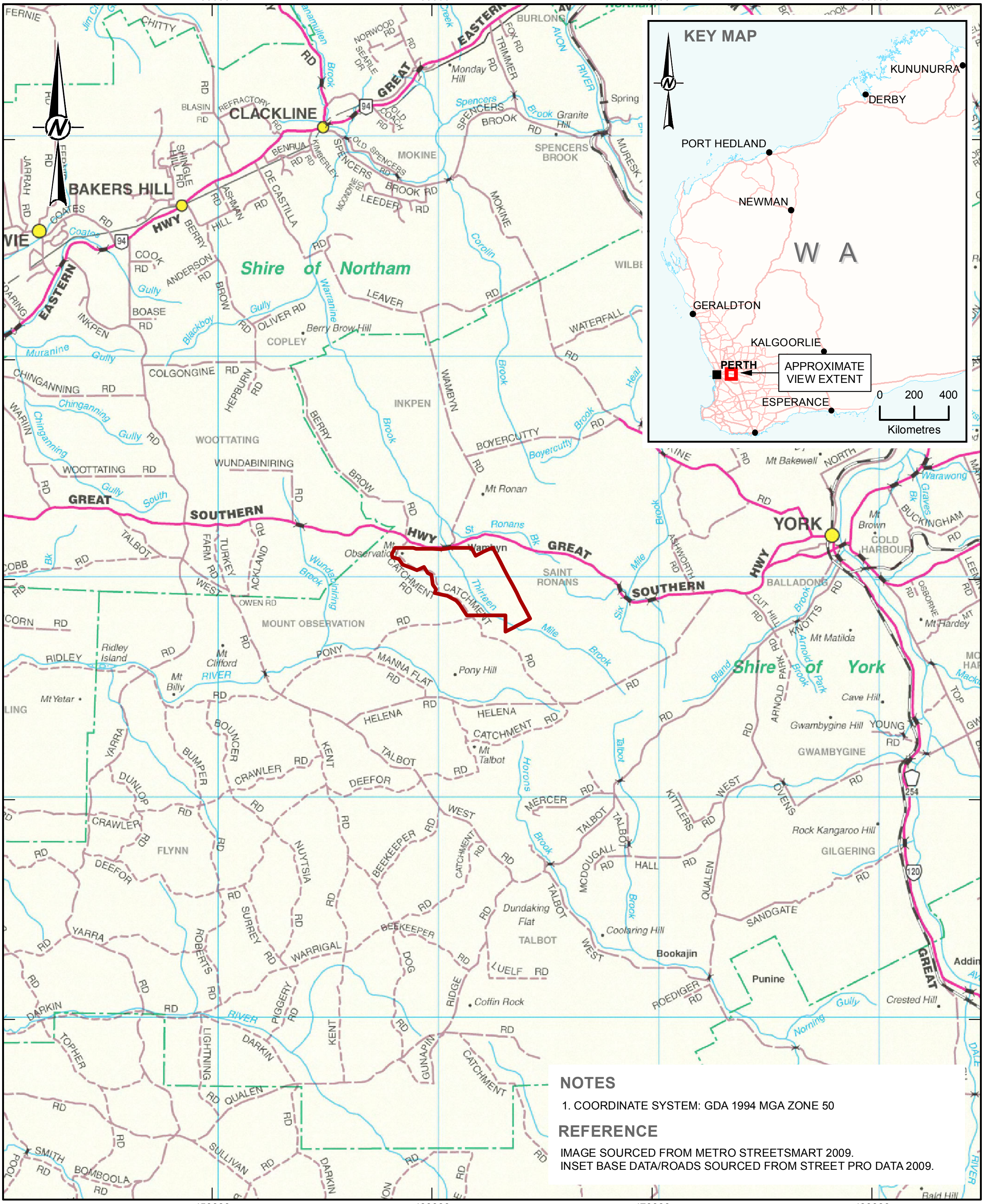
Cell Construction	Material Requirements (m^3)	Volume Water (m^3)
Cell 1	92 800	12 500
Cell 2	15 100	2 000
Cell 3	3 800	500
Cell 4	155 000	21 000
Cell 5	26 100	3 500
Cell 6	244 800	33 000

ASSUMPTIONS

- Compacted Dry Density = 1.8 t/m^3
- Moisture Deficit to bring soil to OMC = 5 % Mwater/Mdrysoild
- Moisture required to maintain soil moist during construction = 50 % Mwater/Mdrysoild

APPENDIX C

Figures and Maps



NOTES

1. COORDINATE SYSTEM: GDA 1994 MGA ZONE 50

REFERENCE

IMAGE SOURCED FROM METRO STREETSMART 2009.
INSET BASE DATA/ROADS SOURCED FROM STREET PRO DATA 2009.

LEGEND

SITE LOCATION

0 2 4 6 8 10 km

1:250,000

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REVIEW	GRH
APPROVED	GRH

TITLE
SITE LOCATION

PROJECT	CONTROL	Rev.	FIGURE
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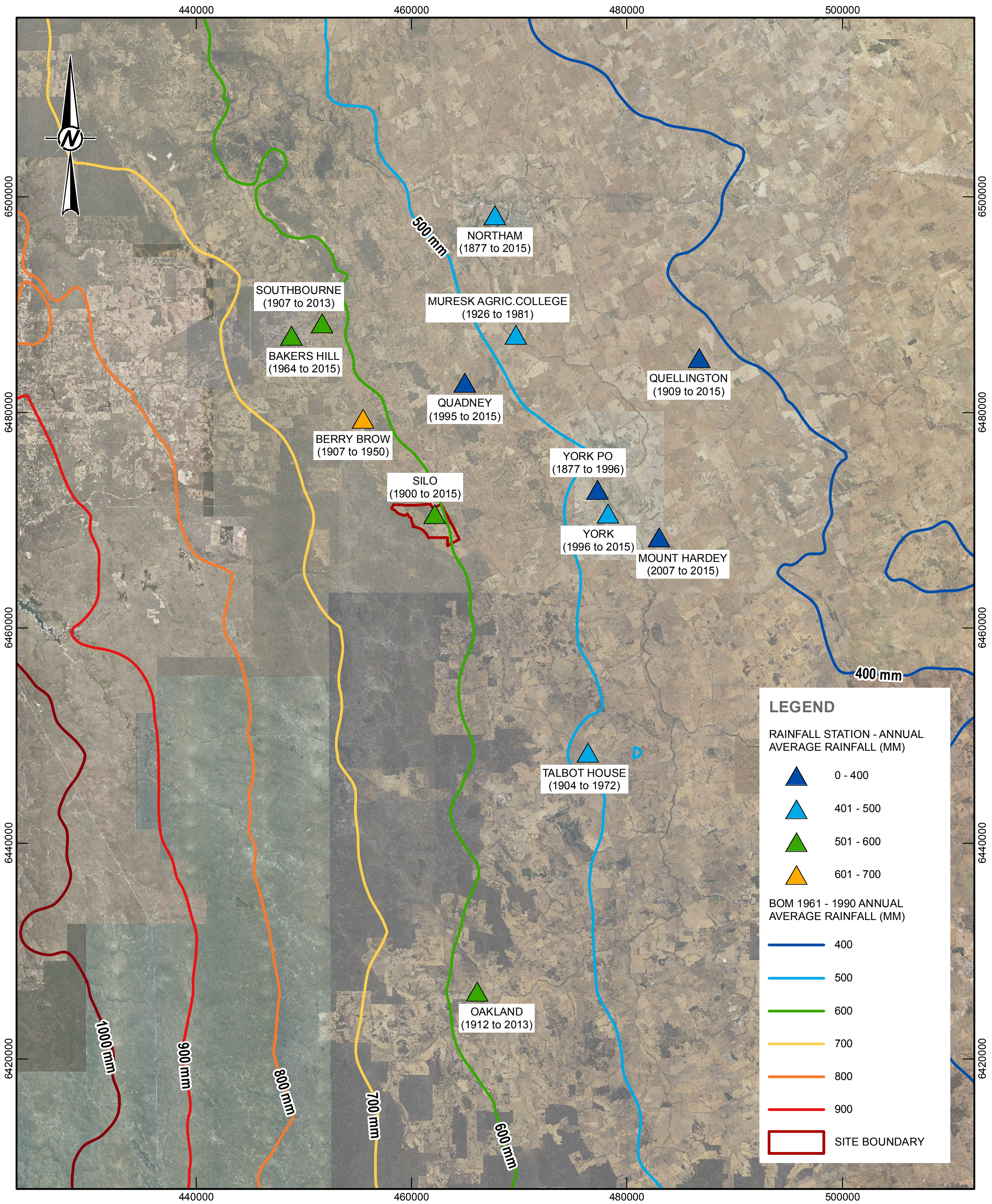

NOTES

1. COORDINATE SYSTEM: GDA 1994 MGA ZONE 50

REFERENCE

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1:500,000

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REGIONAL RAINFALL STATION LOCATIONS

PROJECT	CONTROL	Rev.	FIGURE
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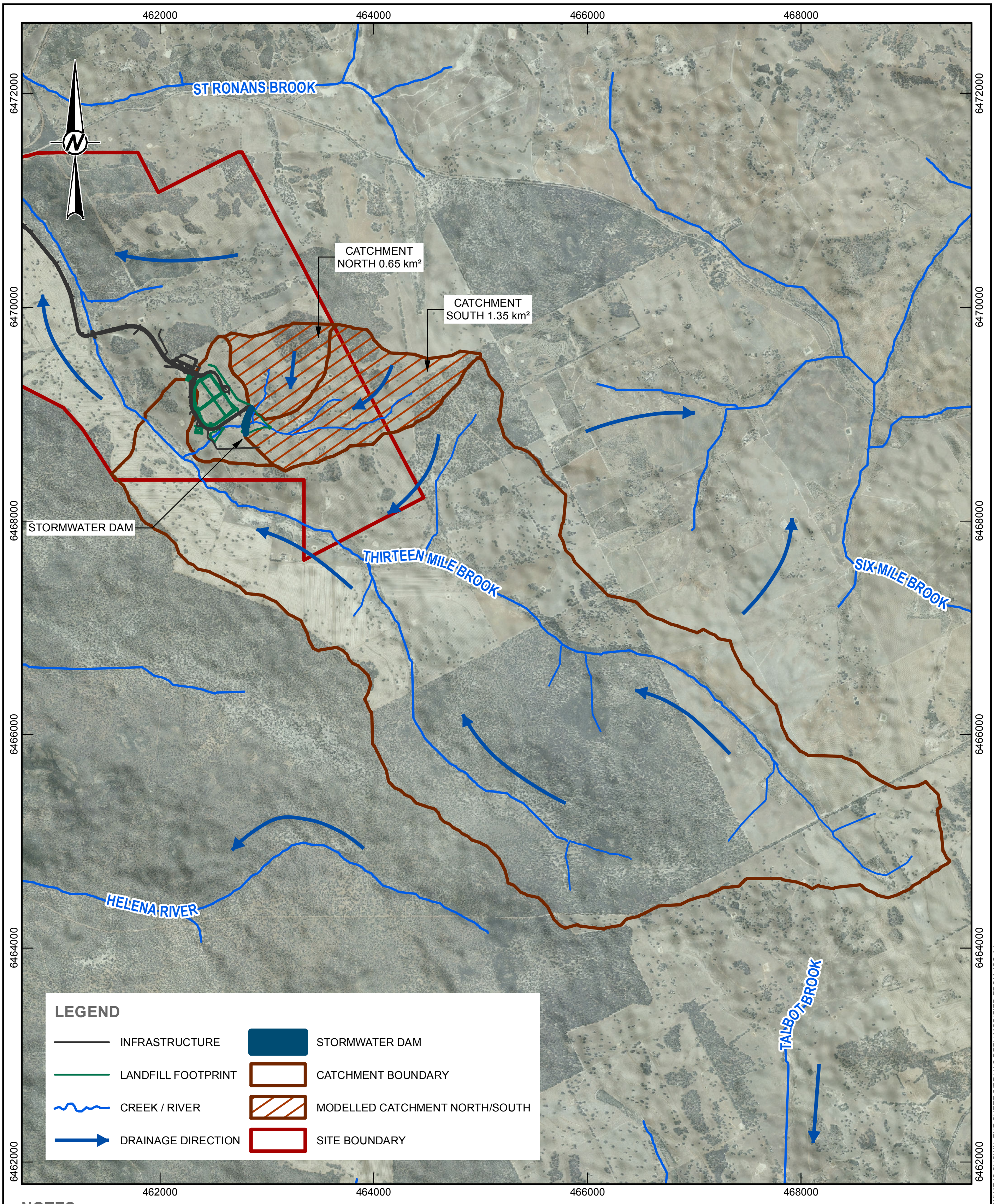

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1. COORDINATE SYSTEM: GDA 1994 MGA ZONE 50

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2012 AERIAL IMAGERY © WESTERN AUSTRALIAN LAND INFORMATION AUTHORITY TRADING AS LANDGATE (2015)

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LOCAL CATCHMENT MAP

PROJECT	CONTROL	Rev.	FIGURE
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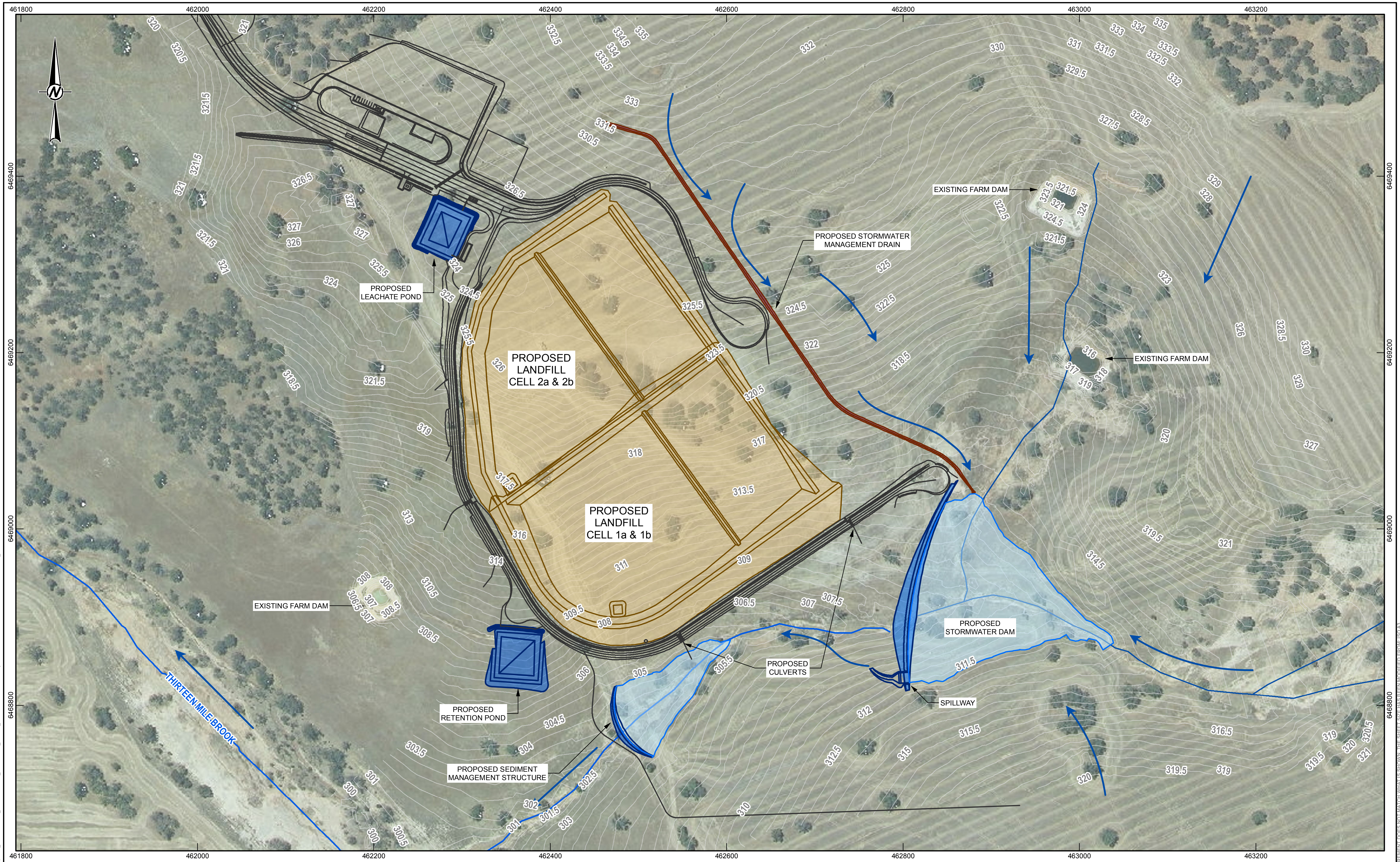

LEGEND

- INFRASTRUCTURE
- DIVERSION BUND
- EMBANKMENT
- POND
- DRAINAGE DIRECTION
- CREEK/RIVER
- CONTOUR

NOTES

1. COORDINATE SYSTEM: GDA 1994 MGA ZONE 50

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REVIEW	GRH
APPROVED	GRH
TITLE	SURFACE WATER AND LEACHATE MANAGEMENT PLAN
PROJECT No.	147645033
CONTROL	015 R
Rev.	0
FIGURE	C4

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