CAIRNCROSS WASTE MANAGEMENT FACILITY – RESPONSE TO SUBMISSIONS REPORT APPENDIX D

ADDENDUM HYDROGEOLOGICAL ASSESSMENT

06 DECEMBER 2018

Incorporating

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Response to Submissions Appendix D

Addendum Hydrogeological Assessment

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REVISIONS

Revision	Date	Description	Prepared by	Approved by
1	9/08/2018	Internal Draft	DJ/GH	CH
2	6/12/2018	Client Comments Addressed	DJ/BF	CH

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1 BACKGROUND

1.1 Introduction

Port Macquarie Hastings Council (PMHC) is seeking development approval to extend the Cairncross Landfill to cover the remaining area identified for landfilling in the 1999 Environmental Impact Statement (1999 EIS). The Proposal is for the expansion of the existing landfill at the Cairncross Waste Management Facility (Cairncross WMF), and would involve the progressive construction, operation and rehabilitation of three landfill stages (Stages 1-3) over approximately 36 years. Stage 1 would commence construction/operation in approximately 2019/2020 respectively and Stage 3 would reach capacity in approximately 2056 with a landfill closure period to follow.

An Environmental Impact Statement (EIS) was prepared for the Proposal seeking approval under Part 4, Division 4.7 of the E*nvironmental Planning and Assessment Act 1979* (EP&A Act). In particular, the EIS was prepared to address, and be consistent with, the Secretary's Environmental Assessment Requirements (SEARs) (SSD 13_5792) for the Proposal, which were issued on 7 May 2015.

The EIS was publicly exhibited, in accordance with Clause 83 of the *Environmental Planning and Assessment Regulations 2000* (EP&A Regulations) between 15 February 2018 and 16 March 2018. During this exhibition period submissions were invited from all stakeholders, including members of the community and government agencies. The submissions received included:

- A total of five submissions from government agencies
- One submission from a member of the community.

The submissions received during public exhibition of the EIS form the subject of formal response to submissions (RtS) report. Amendments are now proposed to the Proposal (the Amended Proposal) based on submissions provided by government agencies and the community, as part of design progression, and to provide additional clarity where relevant. A number of submissions received identified potential concerns with the groundwater management strategy proposed in the EIS. This report presents an amended conceptual mitigation design to manage groundwater ingress into the base of the landfill cell.

1.2 Objectives

The objective of this concept design is to:

- Provide an opinion on whether a base groundwater underdrainage collection system is needed at the landfill
- Provide detail on potential options for collecting groundwater at the proposed landfill
- Develop an amended concept design for groundwater underdrainage system, taking account of site geology, hydrogeology, lining system requirements and client specifications
- Recommend key requirements and functionality of the groundwater underdrainage collection system that aim to preserve the structural integrity and longevity of the base lining systems and leachate collection and control systems.

1.3 Key Features of Landfill

Item	Description
Waste input	General solid waste, including putrescible and non-putrescible materials and asbestos from domestic, commercial and industrial source.
Landfilling lifetime	Stages 1 to 2 are expected to be filled within 36 years commencing 2019/2020. While stage 3 is expected to each capacity in approximately 2056.
Nearest Surface Water Features	The upper reaches of Rawdon and Tommy Owers Creek which are tributaries of Hastings River
Surrounding topography	The Cairncross Landfill Site is located approximately 10 km northwest of Port Macquarie within the Cooperabung State Forest. The Landfill Site is located within the eastern foothills of the State Forest covering the elevated ridge and a sloping area to the east. The State Forest bounds the Landfill Site to the north and east, while the Rawdon Creek Nature Reserve lies to the south.
Geometry of void	The geometry of the void is trapezoidal in shape with the northern boundary abutting the existing closed landfill. The expansion area is split into three stages (1 through to 3). Each stage has the following areas and total landfill void volume: • Stage 1: 79,453 m^2 and 1,610,290 m^3 • Stage 2: 105,840 m^2 and 1,005,030 m^3 • Stage 3: 161,894 m^2 and 1,490,289 m^3

2 HYDROGEOLOGY

Site hydrogeology is described in detail in Appendix F of the EIS. A summary is provided in the sections below.

2.1 Geology

According to the current 1:250 000 Hastings geology map, the Landfill Site is underlain by Lower Permian Beechwood Beds comprising sandstone, siltstone, and mudstone (NSW Geological Survey, 1966). The Beechwood Beds are described as fissile blue grey mudstone and quartz-mica sandstone

The Port Macquarie Area coastal quaternary geology map 1:100 000 (Troedson and Hashimoto, 2005) identifies Carboniferous and Permian rock in the Project area.

Several boreholes have been completed across the Amended Proposal site and surrounding area. Three main geological units are identified within the Amended Proposal Site, generally:

- **Clay:** Clay/colluvium comprising silty medium to high plasticity clay:
  - The clay layer is discontinuous across the site with greater thickness in the valleys and lesser thickness at the ridges
  - The clay layer is reported to extend in several metre-wide strips in the northwest-southeast direction, varying in thickness from 1 to 5 m, or is absent.
- **Weathered Siltstone**:
  - The boundary between the weathered unit and underlying Fractured Shale depends on the depth and thickness of the overlying cover sediments.
- **Shale:** Fractured rock represented by shale:
  - The Shale unit occurs predominately as fresh rock, but is less competent in the upper section
  - The fractured rock is likely to be jointed and the presence of these joints may enhance permeability.

There are also minor areas of alluvium onsite. The alluvium is reported to be of very limited spatial extent and is associated with Rawdon Creek (which is oriented north-south to the west of the current landfill area).

2.2 Hydrogeology

2.2.1 Local Hydrogeology

The Amended Proposal site has been defined by two distinct hydrogeological units:

- **Clay/colluvium:** spatially discontinuous comprising silty medium to high plasticity clay:
  - Reported as a discontinuous clay sequence 2-5 m thick, varying in thickness from 2 m at the ridges to 5 m in the valleys
  - There are no bores installed in the overlaying clay layer as its thickness is not consistent and its spatial extent does not extend across the site
  - Constant head permeability test reported an average permeability of 4×10^{-10} m/s This low permeability may indicated the clay is not an aquifer and is only influenced by groundwater from the deeper geological units.
  - Its major characteristic is the retardation of recharge to the underlying aquifer.

- **Weathered and fractured rock:** associated with siltstone and shale:
  - The overlying clay is underlain by a 4-5 m thick weathered siltstone, which is sequentially underlain by fractured shale bedrock.
  - The hydrographs of groundwater elevations from coupled bores screened in weathered rock and fractured rock generally show very similar response to climate events and negligible difference in groundwater head. Furthermore, the geochemical composition of the two water bearing zones are generally similar, being sodium chloride dominant. These findings suggest that shallower weathered rock and deeper fractured rock are highly connected and therefore have been treated as a single hydrostratigraphic unit
  - Site specific testing reported an average hydraulic conductivity 3×10^{-7} m/s
  - Reported to be unconfined in the crests/ridges and confined in the slopes/valley floors (as a result of topography with the generally increasing clay overburden occurrence).

Groundwater recharge occurs via minor seepage through the clay or lateral flow through the shale/siltstone unit.

2.2.2 Regional Hydrogeology

It was reported (Appendix F of the EIS) that there are ten registered groundwater bores exist within three km from the site, all of which are located at distances greater than approximately two km from the site.

The bores are installed to depths ranging from 23 to 67 m and their purpose is mainly water supply. All bores are installed in hard rock aquifers either shale or basalt, with the yield ranging from 0.5 to 2.5 L/s. The water quality is fresh to slightly saline to brackish, ranging from 700 to 2500 mg/L

2.3 Piezometric Surface and Flow Direction

The groundwater monitoring network comprises nine monitoring points across the existing landfill site and within the Amended proposal site. The monitoring network includes standpipe monitoring bores installed in the upper weathered and lower fractured rock horizon of a fractured rock hydrostratigraphic unit.

Table 1: Summary of Monitoring Bore Construction Details (Trace Environmental 2016)

Monitoring Bore	Depth (mbgl)	Hydrostratigraphic Unit	Groundwater Head (mbgl) Sept 2015	Comment
CG102	21.41	Fractured rock	7.78	
CG103	11.56	Weathered horizon	10.11	
CG104	24.15	Fractured rock	3.26	Nested/Coupled Wells
CG105	9.85	Weathered horizon	3.31	
CG106	26.49	Fractured rock	4.05	
CG107	30+	Fractured rock	3.34	
CG108	30+	Fractured rock	2.78	
CG109	26.19	Fractured rock	7.93	Nested/Coupled Wells
CG110	12.44	Weathered horizon	6.57	

Historically, groundwater heads/pressures ranged from 2.8 mbgl in CG108 to just over 10 mbgl in CG103.

Deepest piezometric heads are found at the ridges and the shallowest in the low laying areas. Therefore, at the ridges the fractured rock hydrostratigraphic unit is unconfined and within the low laying areas it is confined with groundwater head above the top of the hydro-stratigraphic unit. The confinement is a result of topography with the generally increasing clay overburden occurrence in the valleys.

The flow in the fractured rock unit appears to follow the topography, with flow from elevated areas in the north and west to low lying areas in the south, southwest and southeast.

The hydraulic gradient is relatively steep (1 m fall over 50 m). Based on the hydraulic conductivity, gradient and estimated shale porosity of 10 per cent; the average groundwater velocity is approximately 0.0008 m/day.

2.4 Groundwater Chemistry

Groundwater quality samples have been collected during the period from December 2001 to March 2017. Between December 2001 and August 2015, groundwater samples were collected on a quarterly basis by PMHC Environmental Laboratory. Between November 2015 and March 2017, groundwater samples were collected on quarterly basis by Trace Environmental. Different sampling methods were used between the consultants over the periods.

It was reported that compared to baseline data collected in 1998 prior to landfill operation, current groundwater quality results are either improved or within the range of values measured before the start of the operation (Trace Environmental, 2016).

Geochemical quality data, as presented in the piper diagram below, shows that one groundwater type can be distinguished within the Project area, with strong mixing likely to be occurring between the upper and lower fractured rock horizons. The fractured rock system is typically sodium-chloride (Na-Cl) dominant.

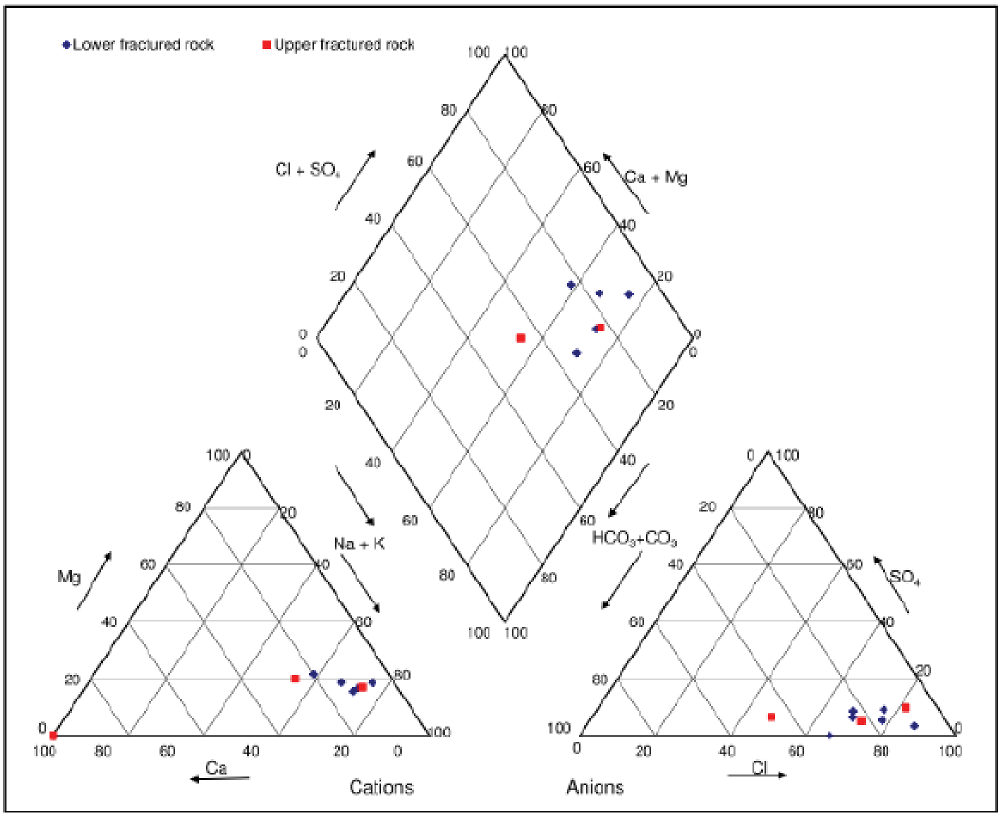


Figure 1: Piper Diagram of Geochemical Composition (Trace Environmental 2016)

Groundwater contaminant quality monitoring data from 2010/2011 relevant to the existing landfill cell (Stage E) was reported in the Water Quality Monitoring Summary Report for Dunbogan, Port Macquarie, Wauchope and Cairncross landfills for the 2010 and 2011 License Period (Connor and Smith, 2012). The report noted elevated levels for a number of parameters including Iron, Potassium, Ammonia and Nitrate. The reported concentrations of these parameters returned to within historical ranges in subsequent monitoring events. The report recommended further investigation to determine the cause of these results.

The Hydrogeological Assessment (Trace Environmental, 2016) (Appendix F of the EIS) found that:

> *The groundwater chemistry indicates that a single groundwater type heavy metal and nutrient concentrations below the ANZECC (2000) guideline values for 95% protection of freshwater species. Geochemical characteristics of the clay layer have not been investigated as this layer is spatially discontinuous.*
>
> *The Stage E landfill leachate has an entirely different chemical composition to groundwater with high salinity, high nutrient load and measurable phenol concentrations. Pre-operational baseline data has similar geochemical composition to groundwater collected over the past 15 years. Based on the chemistry of leachate and baseline groundwater data, it is concluded that there is presently no mixing of leachate with groundwater occurring at the site.*

It is noted that from November 2015, Trace Environmental commenced groundwater monitoring at the Amended Proposal Site, and prior to this the sampling was conducted by PMHC Environmental Laboratory. PMHC identified a change in sampling method from November 2015. Groundwater monitoring data collected at the Amended Proposal site prior to November 2015 had been collected using bailers and with limited or no purging of wells. Data collected by Trace Environmental was obtained using low flow methods, including monitored purging. Sampling of water without purging can result in collection of non-representative samples due to exposure to oxygen within the well resulting in water chemistry changes (pH and dissolved oxygen in particular). This chemistry change directly affects the dissolved contaminants in the water including ammonia and dissolved metals. The use of a bailer to sample can also result in entrainment of colloidal material and the disturbance and oxygenation of the water. Metals and organic compounds bind to colloidal clays and can then be detected in the water analysis giving falsely high concentrations. Volatile contaminants can be lost through the disturbance and oxygenation.

In the majority of wells there is no appreciable difference in reported concentrations obtained using the two methods. In CG107 and CG108 there is a clear increase in the reported ammonia concentration and the salinity corresponding with the sampling method change. In CG108, the salinity measurements went from a consistent 200 μS/cm to around 4,000 μS/cm and the ammonia concentration from 0.1 mg/L or less to approximately 1.4 mg/L. In CG107 the ammonia concentration went from <0.05 mg/L prior to November 2015 to approximately 1 mg/L after, although no change in salinity was reported.

It is considered likely that these two wells may be prone to infiltration from surface water or precipitation and that the historical sampling was not representative of the formation water chemistry. Therefore the more recent measurements should be taken as representative. Note that leachate is discussed in Section 8.5 of Appendix F of the EIS, representative leachate measurements at CL1 indicate ammonia in the leachate (used here as a tracer compound) is typically 1,000 – 1,300 mg/L. Concentrations of ammonia in groundwater in the order of 1 mg/L are not considered indicative of leakage of leachate as they represent less than 0.1 per cent of the leachate concentration.

It is further noted that whilst ammonia, albeit at low concentrations, was reported in CG107 and CG108 down gradient of the existing cell, it was not reported at concentrations above 0.1 mg/L in the boundary wells at the cell edge (CG104 and

CG105, monitoring data post November 2015). These wells are hydraulically aligned with CG107 and CG108 and would be expected to report similar or higher ammonia concentrations to those observed in down gradient wells if there were a breach of containment. This has not been observed.

A summary of the groundwater quality results (minimum and maximum), separated into the two periods of different sampling methods, is provided in Table 2 below. Further analysis on groundwater chemistry and quality is provided in the Addendum Surface Water and Groudnwater Quality Assessment (Appendix C of the RtS report).

Table 2: Ranges of Groundwater Contaminant Quality Measurements

Parameter	Units	Dec 2001 – Aug 2015		Nov 2015 – Mar 2017	
		Min	Max	Min	Max
pH	-	4.9	7.8	5.3	7.6
EC	µS/cm	108	3,910	138	4,320
Ammonia	mg/L	0.01	3.08	0.01	1.73
Nitrate	mg/L	0.01	2.0	0.01	1.2
Phenols	mg/L	0.05	0.42	0.05	0.05
Iron	mg/L	0.02	16.2	0.05	4.37
Manganese	mg/L	0.001	4.32	0.001	2.6

2.5 Estimated Groundwater Inflow

Assessment of groundwater head was completed based on data obtained over a 15-year period, including both dry (2004-2011) and wet (2012-2014) weather periods.

Based on the interpreted maximum groundwater contours, Trace (2016) reported that the proposed redesigned floor level of Stage 3 will generally be above the maximum groundwater head, with potential for maximum groundwater head interception in Stages 1 and 2. The extent of groundwater head above the landfill floor is shown in Figure 2 below.

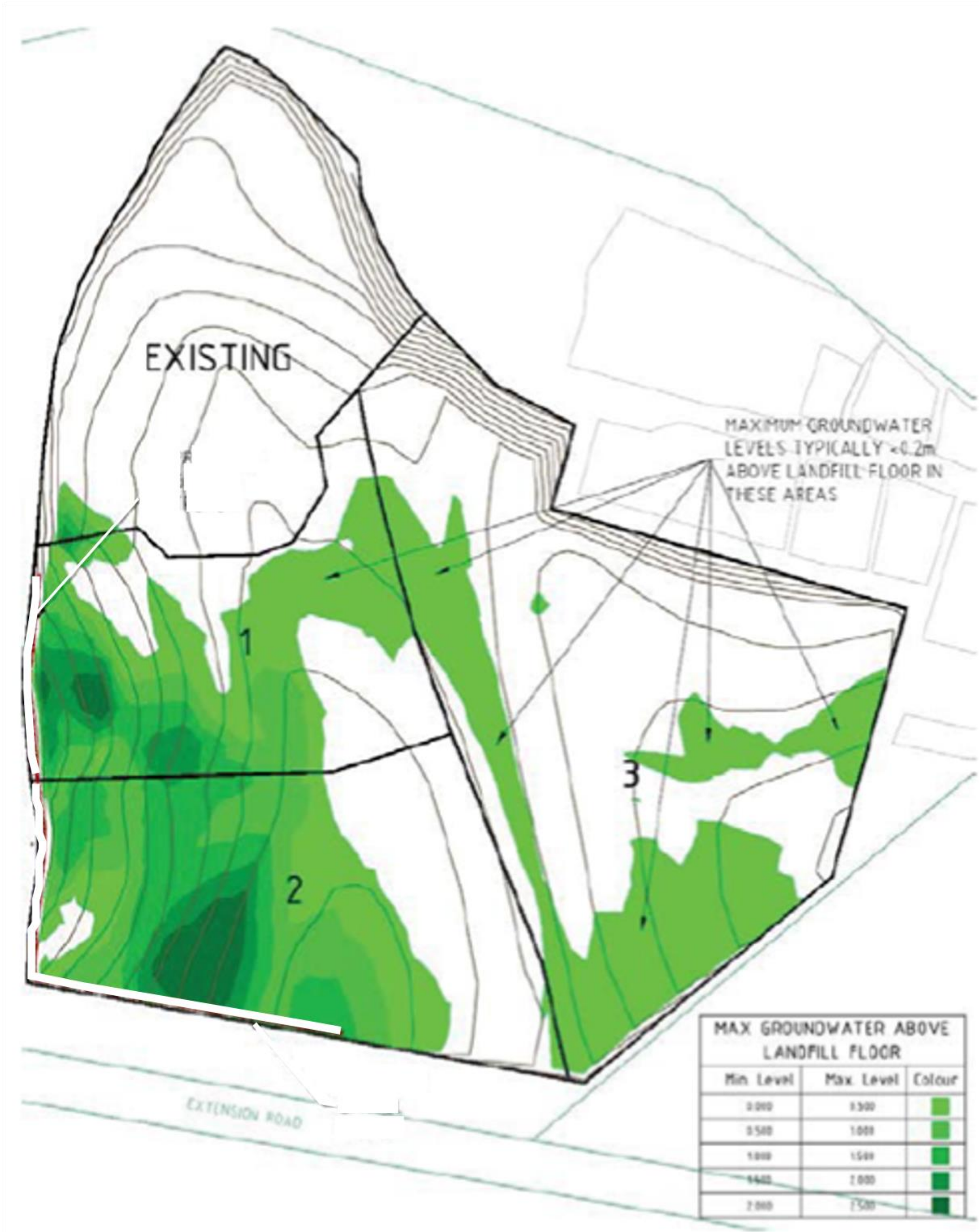


Figure 2: Elevation of Maximum Groundwater Heads above the Redesigned Landfill Floor (Trace Environmental 2016)

Within those areas the maximum groundwater heads are likely to be generally <0.2 m above the floor level. The exceptions are the western part of Stages 1 and 2 which are topographically elevated areas sloping to the east and groundwater recharge zones where the maximum groundwater head may exceed the landfill floor elevation by up to two m.

When above average groundwater heads occur, the groundwater table will intersect the floor of the landfill.

Trace Environmental (2016) completed an analytical groundwater assessment to estimate the short-term inflow into the planned landfill cell during the periods of maximum groundwater heads before any groundwater control is implemented.

The Dupuit –Thiem modified equation (Fetter, 1994) for confined aquifer was used to calculate the long term steady state groundwater inflow into the proposed landfill development.

In summary, the conservative inflow volume of groundwater **during excavation** has been predicted to be:

- **Stage 1** = 0.44 ML/year = 50.2 L/hr
- **Stage 2** = 0.53 ML/year = 60.5 L/hr
- **Stage 3** = <0.03ML/year = <3.4 L/hr

The predicted inflow of groundwater **during operation** (without any groundwater capture system) has been predicted to be:

- **Stage 1** = 0.03 to 0.3 ML/year = 3.4 to 34.2 L/hr
- **Stage 2** = 0.03 to 0.3 ML/year = 3.4 to 34.2 L/hr
- **Stage 3** = <0.03 ML/year = <3.4 L/hr

The estimated inflow rates are based on the modelled results of a long term steady state solution. The model contains assumptions which are detailed within the Hydrogeological Assessment report (Trace 2016). The assumptions are based on site specific and published data that was collected during the hydrogeological assessment and therefore provides a suitable estimation of inflow rates. The actual inflow rates that will be observed during construction may vary, however the estimated volumes are likely to be relatively low and the proposed groundwater capture system (described in Seciton 3 below) would be capable of managing the variances. The level of redundancy will be provided within the detailed design documentation.

3 GROUNDWATER COLLECTION SYSTEM

Management of groundwater associated with the original Proposal within the EIS was planned to be via the installation of a groundwater trench installed along the western boundary of Stages 1 and Stage 2 and the southern boundary of Stage 2. The trench was proposed to divert groundwater from recharge areas to the south and east of Stage 1 and 2, and allow it to discharge via natural flow to the south. Subsequent investigation has determined that the installation of a groundwater trench may not adequately mitigate potential upward hydrostatic pressure that may occur during the maximum potentiometric head conditions (described below). Consequently, the Amended Proposal has further considered the need for, and potential options for, an alternate groundwater management solution.

3.1 Need for groundwater Underdrainage collection

The lowest points of the landfill are reported to be located within the fractured rock geology of the Permian Beechwood Beds comprising sandstone, siltstone and mudstone. Hydraulic testing of the lower fractured rock indicated a permeability of less than 1×10^{-7} m/s thus functioning as a relatively impermeable barrier between the waste and the surrounding environment.

Potentiometric groundwater heads have been measured over time and have indicated the following:

- The base on the landfill is 2 m above the average potentiometric head of the historical groundwater level data;
- The maximum potentiometric head measured over a 15 year period indicated that sections of Stages 1, Stages 2 and Stages 3 may exceed the landfill floor by up to 2 m.

Based on the above, there would be the potential for upward hydrostatic pressure to occur during the maximum potentiometric head across each of the proposed stages that form the Amended Proposal.

A base groundwater underdrainage collection and control system would relieve the hydrostatic pressure that may be encountered during the maximum potentiometric head conditions. This is particularly important where the overlying clay materials are absent and upward flow of groundwater from the underlying fractured rock units is more likely. An underdrainage system would therefore mitigate against potential damage caused by hydrostatic uplift or wetting of the base clay liner. This is particularly important during construction of the low permeability base clay liner and the prevention of wetting / softening of these materials which could result in a long-term loss of hydraulic performance. Early operation of a groundwater underdrainage layer would normally be required until sufficient waste surcharge is in place to counter any upward groundwater pressure / seepage.

The need to depressurise and control any seepages beneath the base lining system cannot be predicted in terms of lateral extent until such time as the excavations have been undertaken and any seepages noted. The competency of the overlying clay and its thickness, together with the level of aquifer confinement at a particular location, would influence the final design and may therefore require a further assessment of shallow geology relative to base liner levels.

3.2 Underdrainage Collection / Control Layer

The Amended Proposal would incorporate the installation of an underdrainage groundwater collection layer beneath each of the proposed landfill stages. The underdrainage groundwater collection system would comprise the following key elements:

1. Installation of a collection/drainage layer: Two potential design options have been considered as suitable for effectively managing the hydrostatic pressures under the maximum potentiometric conditions:
   - Installation of a granular collection trenches, or
   - Installation of a geocomposite drainage net
2. Potential installation of a geotextile layer (if required), between the base of the landfill and underside of the clay capping layer
3. Installation of collection pipes
4. Installation and operation of sumps and risers

These components are described below.

3.2.1 Underdrainage Collection / Control Layer

The groundwater collection layer would need to have sufficient thickness to manage the inflow rates and hydrostatic pressures under the maximum potentiometric conditions. This must be achieved without pressurising the drainage layer; and to ensure an adequate "life expectancy" of the system from clogging. The maximum hydraulic head within the system is determined based on the horizontal permeability of the groundwater collection layer and the spacing of its associated piping. The advantage to minimizing the hydraulic head (both maximum and average) within the drainage layer material across the base would be to reduce the potential of hydrostatic uplift, prevent groundwater intrusion into the landfill cell and maximize the "life expectancy" of the system from clogging.

Depending on the lateral extent of any seepages and the intactness of overlying clays across the prepared cell subgrade surface, the installation of granular collection trenches may be the most effective at controlling seepages and depressurising the underside of the base liner. This will ensure that no localised soft spots form and the entire system can depressurise once the base liner is constructed across the cell levels.

Two potential options have been identified (in order of priority) that would effectively manage inflow rates and hydrostatic pressures under the maximum potentiometric conditions:

- Installation of granular collection trenches, or
- Installation of a geocomposite drainage net (CDN).

Option one: Granular Collection Trenches

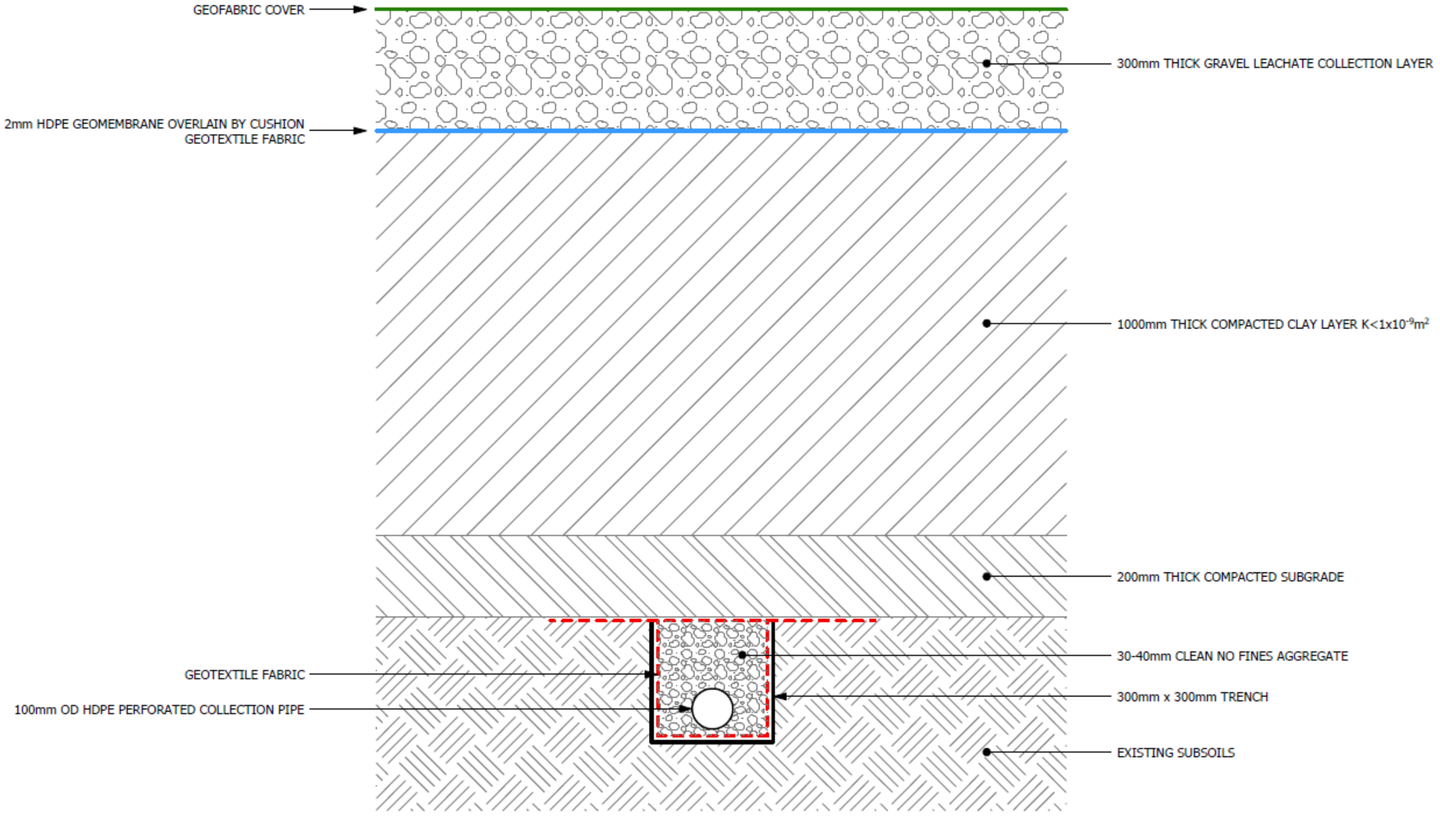
The most likely design option for the collection/drainage layer is through the installation of collection trenches containing a high-permeability granular material and perforated pipework (refer to Figure 3). A herringbone pattern of trenches would use gravity to drain the groundwater to a main header pipe and sump system for extraction.

The granular material would be comprised of predominantly rock (gravel/ cobbles) of greater than 25 mm diameter. In line with NSW EPA (2016), the drainage material would exhibit a coefficient of permeability K > 1 x 10-3 ms^{-1} and the gravel should be rounded, smooth surfaced and non-reactive in mildly acidic conditions. The material should be relatively uniform in grain size and free of carbonates that could form encrustations around collector pipes.

The longitudinal gradient on the landfill base would be greater than one per cent, and a transverse gradient of greater than three percent to ensure good drainage towards the header pipes and underdrainage collection sumps. The existing quarry floor already has sufficient fall. Trenches should also fall inwardly toward the main drainage pipe corridor. The hydraulic conductivity of the granular material would be sufficient to transport groundwater to the sump within a limited period of time (less than one day) from its appearance in the collection system.

In concept, the make-up of the trench system is a relatively uniform gradation stone (for example, a nominally 30-40 mm "single-size"). Such a stone would allow for high horizontal permeability and thus high velocities to the piping network; thus reducing retention time within the system and discouraging the development of biofilm and reducing the potential for biological clogging.

The suitability of the collection pipe design will be confirmed within the detailed design phase. An alternate option, using geo-composite drainage nets is also considered suitable and is discussed in the following sub-section. In addition, a geotextile layer can be placed between the base of the landfill and underside of the clay capping layer if the groundwater properties are found to be likely to result in clogging of the proposed system (and is discussed further in Section 3.2.2 of this RtS).



Proposed Groundwater Drainage Trench
Typical Cross Section

Figure 3: Proposed landfill base typical cross section – Option one

Option two: Geocomposite drainage net (CDN)

An alternative approach would be to use a geocomposite drainage net (CDN). A CDN consists of either a biaxial or triaxial high density polyethylene net bonded to an upper and lower nonwoven needle punched geotextile (composite drainage netting, CDN). This CDN allows for flow through the netting while preventing soil particles from migrating into the drainage core. The grading of the geotextile can be specified to match the required project specifications, in particular the soil types that the geotextile will be placed in contact with. A typical thickness of a CDN is between 5 and 8 mm.

The drainage netting would be installed between the base of the landfill liner and the final excavation level and drain into a main header pipe and sump (as shown in Figure 4). The drainage net would utilise the existing proposed grading of the landfill base, and the header pipe and sumps (described in Section 3.2.3 and 3.2.4) would be oriented in an offset layout to the leachate collection system. The offset layout would aid in preventing cross contamination and any potential differential vertical loading on the infrastructure.

Given the depth of the waste (up to 35 m) and the resulting vertical pressures, approximately 400kPa, this is within the parameters (refer to Appendix B for a typical specification sheet) that geosynthetic materials will remain functionally intact and therefore may be a practical option for groundwater management across the Amended Proposal.

The use of the CDN provides an open area and higher flow and less earthwork requirements when compared to the granular trench design. A CDN that adequately covers low spots and identified seepage areas will ensure the base lining system clay material does not suffer damage during compaction. CDNs placed in contact with underdrainage trenching will ensure that softening damage and depressurisation risks are managed during the critical early stages of waste placement. If the CDN is punctured or greater than expected compressive strengths are exerted upon the material they may be more susceptible to blockages. However, this risk is unlikely, given the current understanding of hydrostatic pressures and base liner design levels.

The suitability of the geosynthetic materials will be determined during detailed design and will consider the resistance of a groundwater collection system to clogging.

Should the detailed design phase determine that there may be potential for the geosynthetic layer to be subject to clogging an alternate option, described below, would be employed.

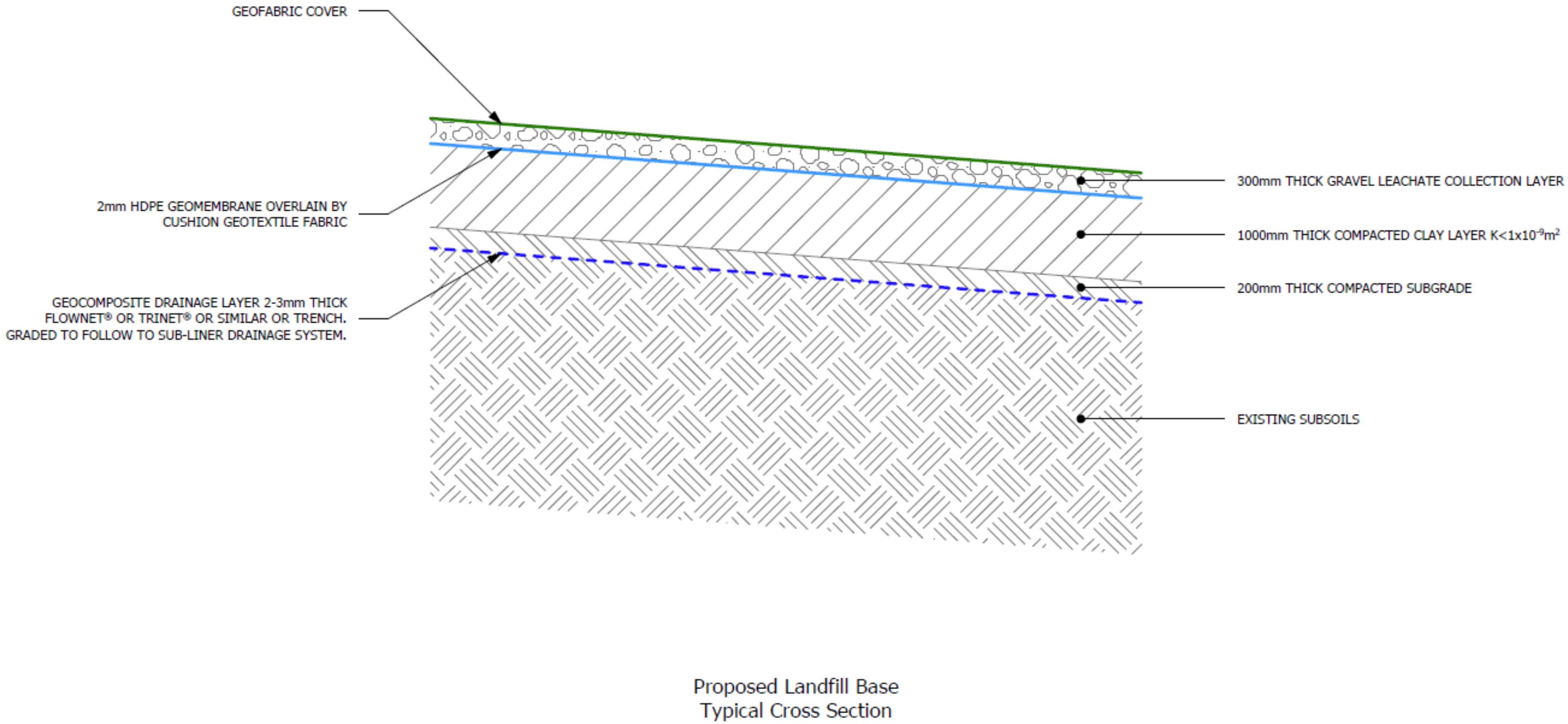


Figure 4 Proposed landfill base typical cross section – Option two

3.2.2 Geotextile

There are applications that suggest either inclusion or exclusion of a geotextile is acceptable dependent upon the nature of the geology/lithology, and proper design of the granular material/selection of the geocomposite drainage net.

In general, groundwater with higher chemical oxygen demand (COD), calcium and/or suspended solids are more susceptible to clogging of the collection system. Given any waste and leachate would be excluded by the liner system, the granular drainage layer material should be relatively uniform in size (single-sized) to maximize the horizontal permeability. A geotextile may be wrapped around the CDN or line the sides of the collection trenches. If a geotextile is employed, it should be placed as distant as practically possible from the collection piping network the drainage layer material to minimize the potential of clogging the entire drainage system.

The final decision of use of a CDN, granular drainage trench and/or the addition of a geotextile would be determined during the detailed design.

3.2.3 Underdrainage Collection Pipeworks

To ensure sufficient transport time (including allowance for redundancy) if collection trenches are used, a network of collection pipes in a chevron/herringbone pattern would be installed (refer Figure 5). The collection pipes would comprise 100 mm HDPE diameter laterals spaced nominally 50 m on centre, and a central 200 mm diameter header pipe. Trench size and spacing is to be confirmed by the engineer in the detailed design stage.

If the geocomposite drainage net is used then two main header pipes and associated trenches would be constructed. Stage 1 and Stage 2 would share a header trench/pipe and Stage 3 would have a dedicate header trench/pipe.

Pipes could be HDPE and pressure rated with an appropriate SDR and manufacturers quoted wall thickness, to maintain structural integrity. Perforated SDR11 or similar (10mm wall thickness) is generally acceptable. Thin walled vacuum formed HDPE pipework is not permitted for this application.

The pipe size and spacing, as shown, are based on typical previous landfill design experience and only intended as an indication of the Concept Design; with the actual size, spacing and configuration/orientation relative to the landfill base (within trenches placed into the landfill base) to be determined at later project design stages based on detailed analyses. It is further noted that these factors (pipe size, spacing and configuration/orientation) are dependent upon a number of considerations, including:

- The average vertical permeability of the underlying geological formation likely to be less than 10^{-8} m/s.
- The anticipated maximum hydrostatic pressure during periods of elevated groundwater levels of 2 m above the base of the landfill
- Inflow rates, maximum estimated at 60.5 L/hr as detailed in section 2.5
- The landfill base gradient
- The horizontal permeability of the drainage net / granular collection trench
- The pipe flow capacity; the vertical loading on the pipe
- The trench size/shape; and

- The pipe spacing and the degree of redundancy desired.

The level/degree of redundancy has not been fully considered in development of the above conceptual assessment; but as an example, the estimated flow could be managed by not only the pipe but also through the gravel or dimensioned trench (less than 300mm x 300mm) and/or the CDN.

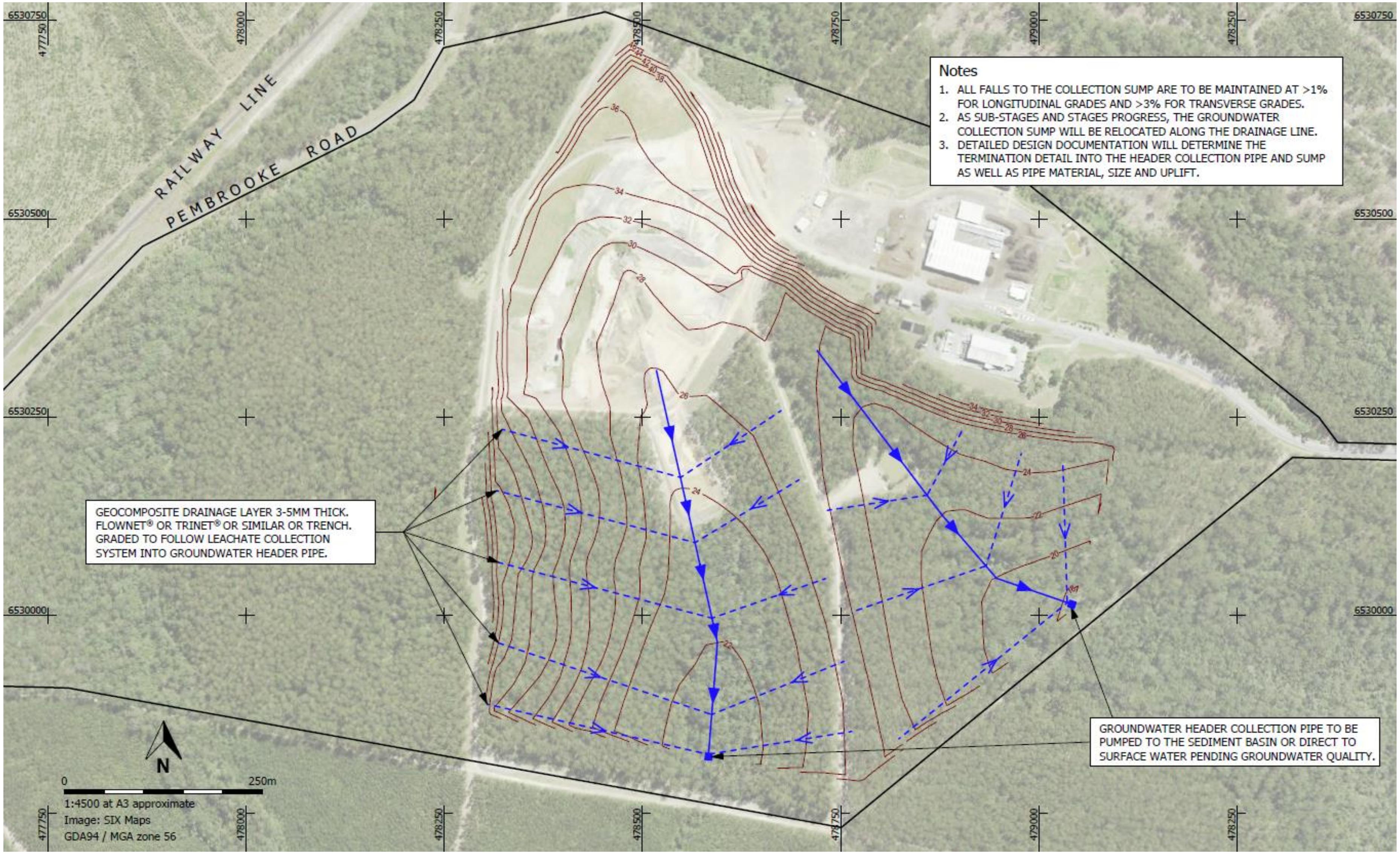


Figure 5 Proposed landfill base site layout

3.2.4 Sumps and Risers

A sump would be located at the lowest elevation of the base, serving to collect the groundwater in preparation for removal.

The sump would contain two risers and a housing for extraction pumps. The groundwater extraction pumps are to be sized with a capacity to maintain a hydraulic head that will be determined during detailed design, and would correlate to a level below the base of the landfill liner. A single pump would operate in one riser under normal conditions, while a second pump would serve as standby, for use if unusually high flow rates are reported (eg under high rainfall events) or during malfunction of the primary pump.

As the landfill cell sub-stages and stages progress the collection sump would be relocated along the main header trench/pipe to maintain the operation of the collection system. The sumps would also provide an accessible sampling point to test groundwater quality during landfilling operations and prior to discharge.

4 OPERATION

The operation of the groundwater management infrastructure described above would comprise:

- Discharge of groundwater
- Maintenance of the groundwater collection system.

4.1 Discharge Options

4.1.1 Primary Disposal

The primary disposal option for the collected groundwater is as surface discharge into the catchment. Prior to any surface water discharge of extracted groundwater, the NSW EPA will require that it meets the appropriate discharge limits and water quality guidelines that are protective of the receiving environments.

These discharge limits are detailed within Section 4 of the Addendum Surface Water and Groundwater Quality Assessment, provided as Appendix C of the RtS report.

4.1.2 Secondary Disposal Options

If following investigation of the water quality it is determined that collected groundwater cannot be discharged offsite then it will be either assessed for onsite re-use and re-used within the landfill cell for dust suppression or disposed of offsite to a licensed liquid waste facility. Given the maximum estimated volumes of groundwater inflow is predicted to be less than 2kL per day if the water quality is unsuitable for offsite discharge then it may also be piped to the sewerage treatment plant.

4.2 Maintenance of Groundwater Collection System

Development of biological clogging material typically begins with the formation of a soft biofilm which is relatively easily removed by flushing. With time, the biofilm ultimately results in the formation of a more solid material which is considerably more difficult or practically impossible to remove.

The underdrainage groundwater collection system should be routinely monitored, inspected and flushed as may be necessary, employing proven methods. Monitoring of the system can be accomplished by measuring and logging the volume of groundwater extracted as a function of time.

In general, the maintenance frequency should be sufficient to remove the biofilm and minimize, to the extent practically possible, the formation of the more solid, encrusted material. Industry practice suggests that this will likely be less frequent in clean groundwater environments that are not impacted by leachate. Iron fouling can also cause blockages within the pipework or drainage netting, review of the chemical composition of the groundwater indicates that the deeper shale zone has a relatively low concentration of iron with the average ranges between 0.2-2.3 mg/L and therefore is unlikely to cause significant iron fouling during the operational period of the landfill.

A reduction in the flow rate, compared to the predicted flow-rate over time, during the operational life of the landfill is typically an indication that the system is under the influence of clogging mechanisms. Inspection of the pipework is accomplished through closed-circuit television (CCTV), and may not be necessary if the system is closely monitored and maintained.

Maintenance of the system is practically limited to back flushing of the pipes and the perforations, and is typically accomplished through the application of high-pressure water jetting, with access provided through the removal (riser pipe) system.

The required frequency of system maintenance is dependent upon a number of factors, both general and specific details, including:

- The infiltration flow rate and hydrostatic pressure of the leachate vertically into the collection layer
- The thickness of the granular collection layer material
- The configuration including the size, spacing, use of trenches, use of geocomposite drainage layers within the collection layer
- The size of the particles composing the collection layer material; and the size and number of perforations in the collection pipe.

4.2.1 Contingency Measures

The principal areas where the groundwater collection system has the potential to fail are:

- Clogging of the drainage and pipe network and
- Pump failure.

Clogging can be prevented by good system design. Use of a suitable, open rock drainage material to prevent clogging or drainage netting (as above), ensuring gradients at the base are at least one per cent and providing a means of flushing the system will help to prevent this.

As a further contingency, the technology exists to flush groundwater collection pipe networks from the ground surface using water jets controlled by robotic systems.

There detailed design will comprise one service and one standby pump and two risers to ensure that there is always a means of removing a failed pump.

5 CONCLUSION AND RECOMMENDATIONS

The groundwater collection system will be designed to handle the estimated maximum groundwater inflow per day which will be based on the historical maximum potentiometric surface levels and measured hydraulic conductivities. The estimated inflow rates will be confirmed during detail detailed design and will inform the technical specifications of the final design. The current inflow rates ranges between 3 and 60 L/hr. The key considerations in designing the system will be:

- The depth of the void. This determines the amount of hydrostatic uplift and volume of water that will seep into the collection system
- The geometry/topography of the landfill void. This will determine how many drainage nets and intermediate sumps will be required as the three stages of the landfill progress
- The groundwater quality. This will determine how the collected groundwater is discharged and the required monitoring to be completed prior to and during discharge events.

Based on the above considerations a base groundwater collection system comprising either a geocomposite drainage net solution or installation of collection trenches will be incorporated into the detailed design for the Amended Proposal. Either design option would effectively mitigate the risk of hydrostatic pressure to occur during the maximum potentiometric head across each of the proposed stages that form the Amended Proposal.

Either solution would achieve the following performance:

- Separation of clean groundwater from waste, therefore reducing the creation of leachate
- Collect upward inflowing groundwater into a high permeability collection trench or drainage layer and convey it to sumps for extraction thus mitigating the risk of hydrostatic pressure uplift
- Prevent over wetting of the base landfill liner system during construction and early stages of landfilling
- Allow the staged construction of the underdrainage collection system as the landfill footprint progresses.

6 REFERENCES

NSW EPA (2016) ***Environmental Guidelines: Solid Waste Landfills*** Environmental Protection Authority, Chatswood

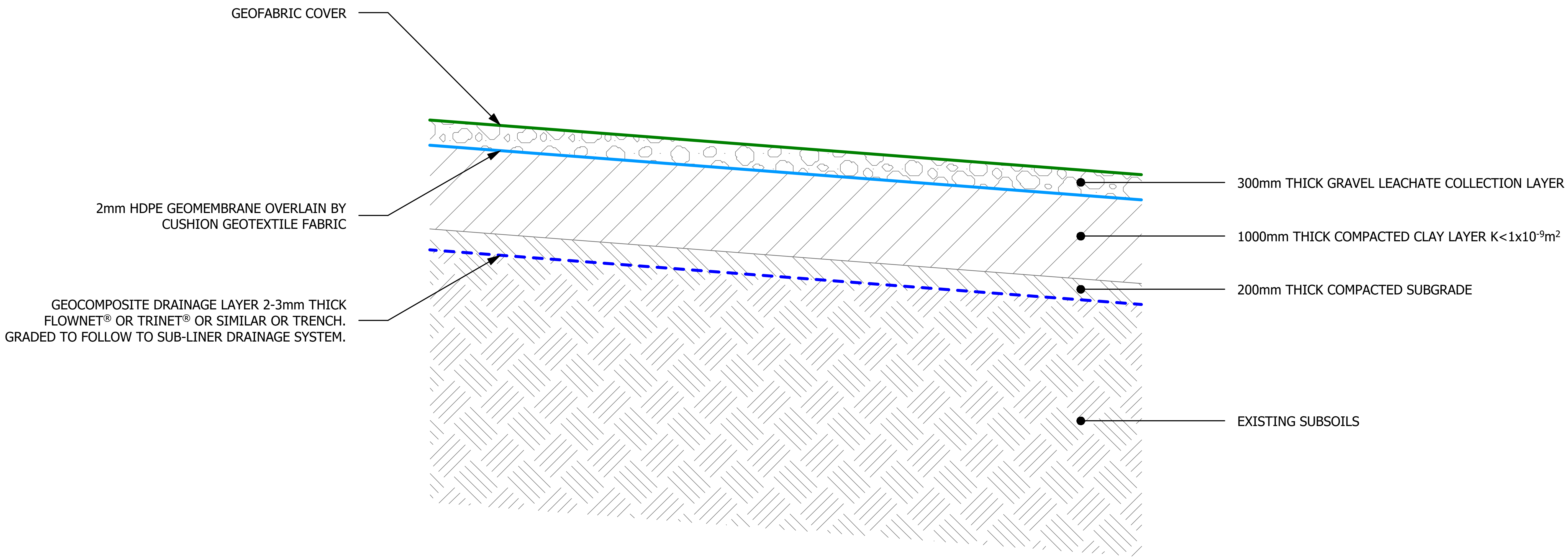
Trace Environmental ***Hydrogeological Assessment Cairncross Landfill Expansion***. 11 October 2016.

Arcadis ***Cairncross Landfill Expansion Environmental Impact Statement*** 11 November 2017.

ERM ***Environmental Impact Statement***. 1999. Note: The EIS was developed by ERM to support the development application, and subsequent approval, for the first stage of the Cairncross landfill.

APPENDIX A

Conceptual design drawings



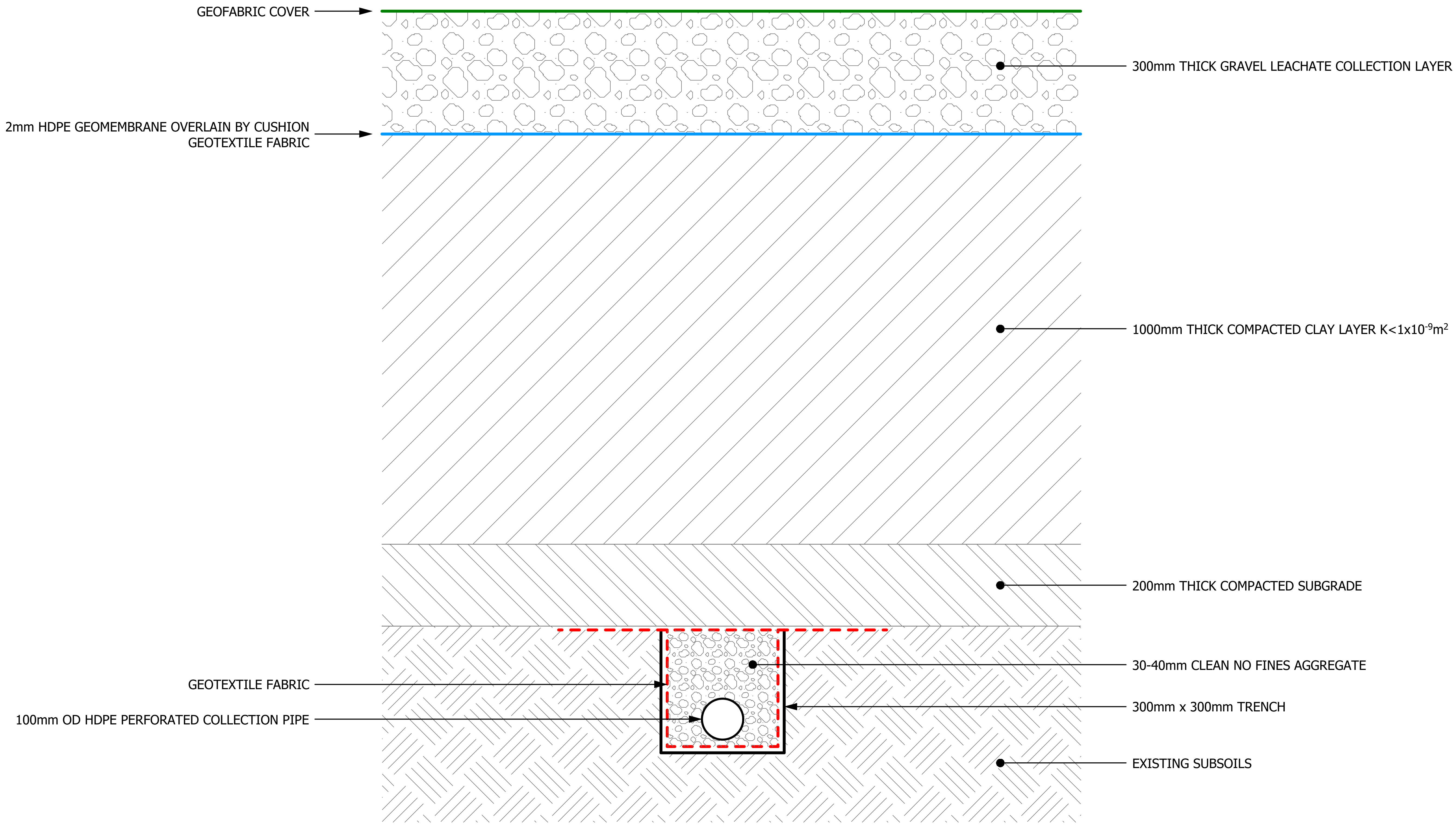
Proposed Landfill Base
Typical Cross Section

Notes:
1. Geocomposite transmissivity to be specified by others as part of detailed design to perform under the vertical surcharge of wastes and predicted water inflow rates
2. Geotextile protector to be designed and specified by others, subject to the rock type specified for leachate collection layer and vertical surcharges
3. Geofabric cover above leachate collection layer will act as a separator layer. The strength and pore size opening to be specified by others
4. Needlepunched staple fibre or continuous monofilament geotextile protection layer, mass and CBR to be determined during detailed design

Not to scale.

Key
- Geofabric
- Geomembrane overlain by Cushion Geotextile Fabric
- Geocomposite Drainange Layer

Title Cairncross Landfill		FIGURE 1 Proposed Landfill Base Typical Cross Section
Site Location 351 Telegraph Point Road Pembrooke NSW	Project No. 10004701	ARCADIS Design & Consultancy for natural and built assets



Proposed Groundwater Drainage Trench
Typical Cross Section

Not to scale.

Key
- Geofabric
- Geomembrane overlain by Cushion Geotextile Fabric
- Geocomposite Drainange Layer
- Drainage Trench
- Geotextile Fabric

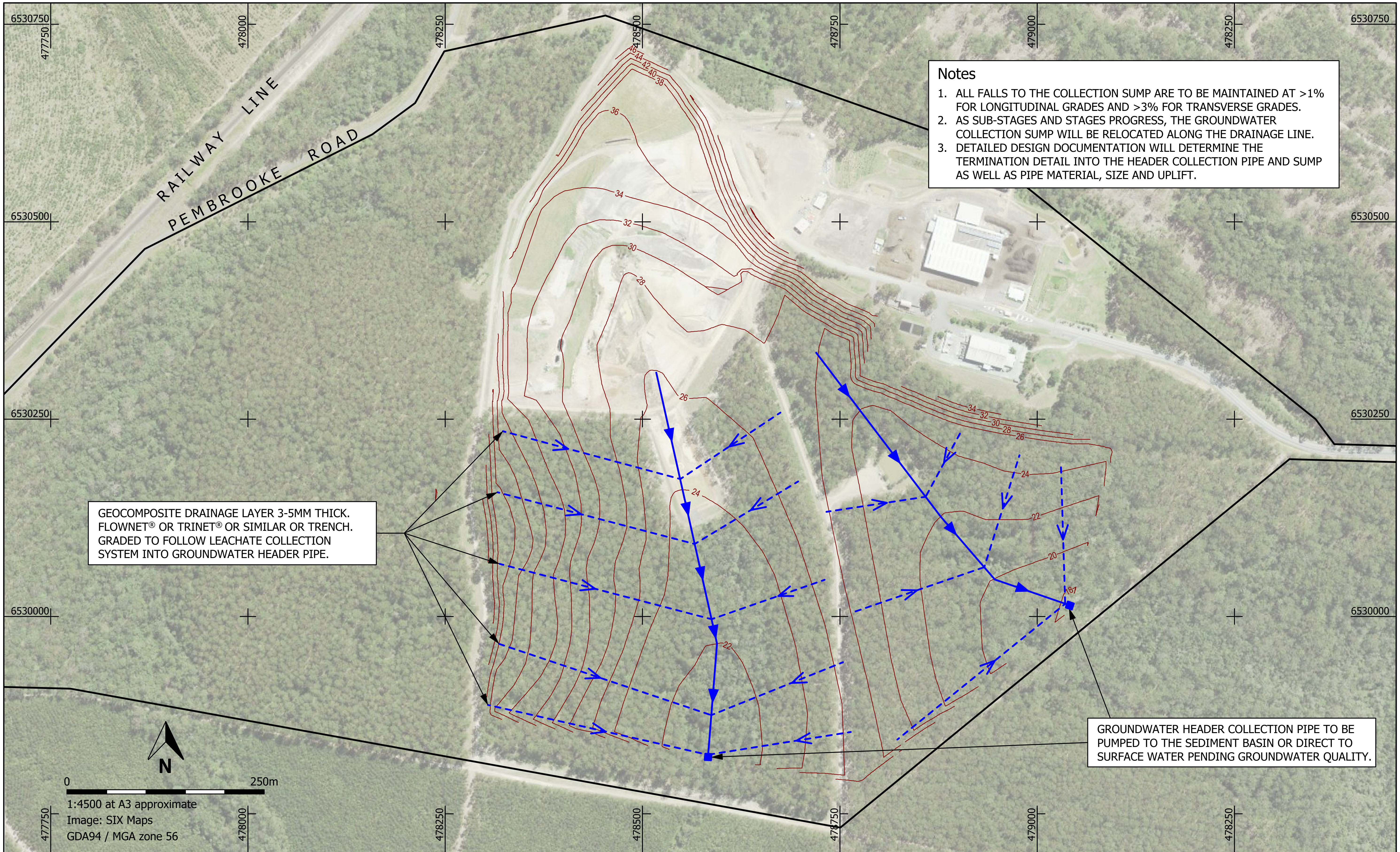
Title
Cairncross Landfill

Site Location
351 Telegraph Point Road
Pembrooke NSW

Project No.
10004701

FIGURE 2
Groundwater Drainage Trench Typical Cross Section

ARCADIS Design & Consultancy for natural and built assets



Key

- Site Boundary (approximate)
- ## Design Level Contour
- Groundwater Collection Header Pipe
- Geocomposite Drainage Layer or Trench
- Collection Sump

Title
Cairncross Landfill

Site Location
351 Telegraph Point Road
Pembrooke NSW

Project No.
10004701

FIGURE 3
Proposed Landfill Base Site Layout

ARCADIS Design & Consultancy for natural and built assets

APPENDIX B

Typical CDN specification sheet

Eco Line Solutions Pty Ltd
ABN 58 165 743 420

Office; Suite 601, 153 Walker St,
North Sydney, NSW, 2060

Postal: PO BOX 1040
Crows Nest, NSW, 1585

Product : EcoLine 270-2-270

We hereby certify that the TN 270-2-270 drainage geocomposite made from HDPE resin with nonwoven polypropylene geotextile fabric heat bonded on both sides of the geonet. and meets or exceeds properties listed in below Table:

Property	Test Method	Unit	Value	Qualifier
Geonet[3]				
Compressive Strngth	ASTM D 1621	kPa	990	MAV
Thickness @ 200 kPa	ASTM D 5199	mm	4.8	MAV
Carbon Black	ASTM D 4218	%	2.0 - 3.0	Range
Tensile Strength	ASTM D 7179	kN/m	7.5	MAV
Melt Flow	ASTM D 1238[2]	g/10 min	1	Maximum
Density	ASTM D1505	g/cm^3	0.94	MAV
Transmissivity[1]	ASTM D 4716	m^2/sec	7.4×10^{-5}	MAV[6]
Composite				
Ply Adhesion	ASTM D 7005	g/cm	184	MAV
Geotextile[3 & 4]				
Fabric Weight	ASTM D 5261	g/m^2	270	MARV[5]
Grab Strength	ASTM D 4632	N	900	MARV
Grab Elongation	ASTM D 4632	%	50	MARV
Tear Strength	ASTM D 4533	N	350	MARV
CBR Puncture	ASTM D 6241	N	2000	MARV
Permittivity	ASTM D 4491	sec^{-1}	0.5	MARV
AOS	ASTM D 4751	µm	430	MaxARV
UV Resistance	ASTM D 4355	%/hrs	70/500	MARV

Notes:

1. Transmissivity measured using water at $21 \pm 2\,^{o}C$ ($70 \pm 4\,^{o}F$) with a gradient of 1.0 and a confining pressure of 400 kPa (8,350 psf) between steel plates after 100 hours.
2. Condition 190/2.16
3. Geotextile and Geonet properties are prior to lamination.
4. Geotextile data is provided by the supplier.
5. MARV is statistically defined as mean minus two standard deviations and it is the value which is exceeded by 97.5% of all the test data.
6. Minium average value

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