City of Darwin

Stage 2 Expansion – Design Report

Shoal Bay Waste Management Facility

December 2024

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Stage 2 Expansion – Design Report
Shoal Bay Waste Management Facility

City of Darwin

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Rev	Date	Details
A	29 July 2024	Draft
B	17 October 2024	Draft – revised for an asbestos waste expansion
0	6 December 2024	Final

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WSP acknowledges that every project we work on takes place on First Peoples lands.
We recognise Aboriginal and Torres Strait Islander Peoples as the first scientists and engineers and pay our respects to Elders past and present.

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1 Introduction

The City of Darwin (CoD) has engaged WSP to prepare the Design Documentation for a landfill expansion to the existing Stage 2 Inert Landfill, to be constructed at the Shoal Bay Waste Management Facility (SBWMF) in Darwin, Northern Territory (the Site). The Stage 2 Landfill expansion is proposed to accept asbestos waste only.

The previous revision of the design was developed for an inert landfill expansion. CoD subsequently decided to change the expansion to accept asbestos waste only. The design has been updated based on an asbestos waste expansion.

The design of Stage 2A expansion is subject to audit by a Qualified Person (Environmental Auditor) to assess compliance of the design with the Northern Territory Environment Protection Authority (NT EPA) *Guidelines for the Siting, Design and Management of Solid Waste Disposal Sites in the Northern Territory* dated January 2013 (the NT EPA Guidelines).

This design report has been prepared to demonstrate conformance of the Stage 2A design with suggested measures of the NT EPA Guidelines, or, where design measures differ from the suggested measures, to provide rationale for difference and demonstrate that they meet the intent of the NT EPA Guidelines in terms of risk management.

The Stage 2A design is presented in the following documents, collectively referred to as the design documents:

- Technical Specification for Construction of Stage 2A, Shoal Bay Waste Management Facility – WSP document PS138768-006-S-Rev0 (the Technical Specification).
- Design Drawings for Shoal Bay Landfill Stage 2A, WSP reference PS138768-007-T-Rev0 (the Design Drawings).
- CQA Plan for Shoal Bay Landfill Stage 2A, Rev 0 – WSP reference PS138768-008-R-Rev0 (the CQA Plan).

This report has been prepared to document the basis of design as presented in the above documents, with supporting information presented in Appendix A to Appendix D.

2 Background information

2.1 General

SBWMF is located approximately 13 km north-east of Darwin's city, in the Northern Territory. The SBWMF lies within Lot 3952 with the site accessed by the Shoal Bay access road, which is connected to Vanderlin Drive, 1 km south of the site. The Stage 2A expansion area is located to the east of the municipal landfill area and south of the covered leachate ponds.

The key landfill operation facilities at the Site include:

- waste transfer station
- recycle shop and weighbridge
- current waste cells for burial activities of general waste (Stage 5 and 6)
- current waste cells for burial activities of inert waste and asbestos waste (Stage 2)
- green waste processing and storage area
- stormwater and leachate management facilities
- leachate treatment facilities
- exclusion area – explosive ordinance burial pit
- gas capture, recirculation, and power generation unit
- future waste cells.

2.2 Existing infrastructure

Stage 2A is an expansion to the exiting Stage 2 Inert Landfill to accept asbestos waste only. The relevant existing infrastructure within the general Stage 2 area at the SBWMF include the following:

- Buried (decommissioned) landfill gas pipes and wells within Stage 2 area.
- Temperature monitoring bores across the eastern portion of the Stage 2 landform. The existing temperature monitoring bores are outside the proposed Stage 2A cell design footprint.
- Leachate transfer pipe extending north to the covered leachate storage ponds located to the northwest of Stage 2.
- A site access road and overhead powerlines running north-south along the western edge of Stage 2 and the Stage 2A expansion.
- Inferred historical leachate trench to the east of the Stage 2 area.

2.3 Topography

The SBWMF is located on a low ridgeline that extends into the Shoal Bay floodplain. The ridge runs approximately north south and generally declines in elevation towards the north into Shoal Bay.

2.4 Geology and hydrogeology

The geotechnical investigation conducted on 5 and 6 December 2023 comprised of a field investigation, excavating a total of 19 test pits within the footprint of the Stage 2 expansion across the 3 days. The observations from the investigation provided a general surface profile comprising of Topsoil, Fill material, and weathered Porcellanite. The materials comprised of the following:

- Topsoil:
  - Silt, sandy silt, sandy clay, sandy silty clay and clayey sand with organic matter, roots and gravel. This was typically encountered at depths of 0.2 to 0.5 m below ground level.
- Fill Material:
  - Re-worked natural soil materials or other imported fill materials. Some waste present in the fill including timber, plastic, rubber and wire.
- Inferred weathered Porcellanite:
  - Generally highly weathered and inferred to be of very low to medium strength, with high strength zones increasing with depth. The depth to porcellanite varied across each of the test pits.

Refer to Geotechnical Factual Report Shoal Bay Landfill – Stage 2 (document reference: PS138768-WSP-MEL-GEO-REP-010-RevA, March 2024).

The Stage 2A expansion floor model has considered the depth to refusal at the porcellanite rock layer encountered during the field geotechnical investigations.

Hydrogeological characterisation of the site is described in the *Shoal Bay Waste Disposal Site, Hydrogeological Investigation* (URS, 2015) and *Shoal Bay Waste Disposal Site, Conceptual Site Model* (URS, 2016). The lithology encountered during the investigation generally comprised of topsoil (silts and clays) underlain by weathered siltstone, with claystone encountered east of Stage 1, underlain by sandstone between approximately 9 to 20 mbgl (metres below ground level). Groundwater exists within the unconsolidated overlying sediments, and fractured and weather underlying bedrock.

Groundwater levels and flows were analysed from site data for the period July 2016 – March 2020 and presented in Shoal Bay Waste Management Facility, Water Quality Monitoring Data Review 2016–2020 (EcOz, 2020). The following is noted from the report:

- The shallow (water table) aquifer is recharged by the direct infiltration of rainfall and runoff, and groundwater levels vary seasonally in accordance with rainfall totals.
- Seasonal groundwater levels were reported to be greater than 12 mbgl at the end of the dry season, and typically less than the 2 mbgl during the peak of wet season.
- Groundwater contours generally follow the topography of the site, with groundwater flowing in a western, eastern and north-eastern direction.
- Groundwater beneath Stage 2A flows to the north-east with groundwater monitoring bore GW4-12 located downstream of Stage 2A.

2.5 Climate and weather

The site is located in the tropical north of Australia and is typically defined as having distinct dry seasons and wet seasons.

The dry season (May to September inclusive) is characterised as having moderate temperatures, typically ranging from the high-teens to low-thirties, with mean rainfall between 0 mm/month and 20 mm/month.

The wet season (October to April inclusive) is characterised as moderate to high temperatures, typically ranging from the mid-twenties to mid-thirties, with high humidity and mean rainfall between 70 mm/month and 430 mm/month.

2.6 Existing Stage 2 Inert Landfill

The historic general and inert waste profiles are identified in the Landfill Development Plan for Stage 2 Inert Landfill Shoal Bay Waste Management Facility (Reference Number: 1653988-006-Rev1; Golder, 2017) (the Stage 2 Development Plan). The Stage 2 Development Plan states that general waste was placed from an elevation of approximately RL5 m AHD to an upper surface of approximately RL21 m AHD. In 2017, at the time of when the Stage 2 Development Plan was prepared, the maximum thickness of inert waste placed above the general waste was 6 m, with an upper waste surface of approximately RL27 m AHD. It is understood that any waste filled above RL 21 m AHD, is classified as inert waste or asbestos.

The proposed expansion area is offset more than 1,200 m from the nearest residential properties in the Holmes Jungle Nature Park to the southwest. The proposed Stage 2A expansion is for asbestos waste only. Stage 2 landfill is the only facility in Darwin that is Licensed to accept inert and asbestos waste.

CoD has indicated to WSP that the available airspace in Stage 2 is expected to be depleted by the end of 2024. Alternative measures are being considered by CoD until the Stage 2A expansion is commissioned to receive asbestos waste.

2.7 Key constraints

Key development constraints considered for the Stage 2A expansion include:

- The Highest Astronomical Tide (HAT) and Storm Surges that are experienced in the low-lying wetlands to the east of the Stage 2A area.

  The Department of Environment and Natural Resources provided via email dated 8 August 2022, the following vertical elevations for selected storm tide design intervals for the year 2100 predictions at the Shoal Bay Waste Management Facility (Lot 02952, Town of Sanderson):

  - Highest Astronomical Tide (HAT) – Max AHD WL = 4.21 m
  - Primary Storm Surge Zone (100-year ARI) – Max AHD WL = 4.98 m
  - Secondary Storm Surge Zone (1,000-year ARI) – Max AHD WL = 7.13 m
  - Extreme Storm Surge Zone (10,000-year ARI) – Max AHD WL = 9.02 m.
- The eastern embankment of the Stage 2A cell has been positioned (nominally) to be upstream of the Secondary Storm Surge Zone (1,000-year ARI level). The groundwater drainage outlet has been positioned (nominally) to discharge above the Primary Storm Surge Zone (100-year ARI level).

- Potential for remnant vegetation communities to be encountered during the construction works, noting that the proposed expansion area includes some vegetation and wetlands to the east. EPBC Self-Assessment has been completed. Details of the desktop review and outcomes can be found in WSP Report 'EPBC Self-assessment for Stage 2 Inert Landfill Design Service' Reference: PS138768-WSP-DRW-ECO-REP-00001 RevA.
- Shallow seasonal groundwater levels limit the ability to include an attenuation zone from potential seepages. This has resulted in two options, to lift the cell floor to achieve attenuation zone or to install a groundwater drainage system beneath the cell floor. The Stage 2A has been designed to incorporate a groundwater drainage system.
- Shallow hard rock conditions impact the maximum achievable excavation/reprofiling depth. The Stage 2A cell floor design has been optimised to increase volume of cut while maintaining the subgrade levels above or at rock level. This has been achieved by recording the depth to refusal at multiple locations across the project footprint during the geotechnical investigation.
- An Uncleared Unexploded Ordinance (UXO) Area approximately 200 m to the east of the proposed expansion boundary at the SBWMF.
- Maximum height of Stage 2A expansion is based on stability analysis and materials, with consideration to filling and access during operation. The Stage 2A expansion has been designed to allow future expansion of the Stage 2 landfill.
- Grassland fire risk within surrounding undeveloped land areas. The vegetation within the proposed Stage 2A footprint shall be cleared and disposed of prior to construction works commencing. The SBWMF Site Fire Plan shall be reviewed and updated by CoD to consider the expansion. A copy of the Fire Plan shall be made available to the Contractor and any other parties involved in the construction works.
- Release/generation of landfill gases within the existing waste cell. The risk of landfill gas release from the Stage 2 Landfill into the Stage 2A construction works area is low due to the waste type disposed into Stage 2 being inert waste and covered by a capping layer on the Stage 2 slope. However, workers and machines excavating or clearing near to the northern batter of Stage 2 shall wear/carry gas monitors as detailed in the Safety in Design Register (see Section 4 and Appendix A).
- CoD has opted to limit the waste stream of the expansion to asbestos only to protect the Stage 2A expansion from spread of known active hotspot conditions within existing Stage 2 waste mass. An interface layer was previously proposed for an inert waste expansion; however, this has since been removed for the proposed asbestos landfill expansion on the basis that asbestos waste is non-combustible. There are acceptance criteria for Asbestos Waste landfilled at Shoal Bay. CoD has informed WSP that it will manage the risk of other contaminant sources and agree this process with the Auditor. For the purposes of the cell design, the risk of a significant hot spots developing in low portions of potentially other materials sources within the Asbestos waste is low due to the Asbestos Waste acceptance criteria/operation controls at the site.

2.8 Stage 2A conceptual site model

A conceptual site model (CSM) has been prepared by URS Australia Pty Ltd (URS, 2016) which includes the Stage 2 Landfill. This CSM was considered in developing potential sources of emissions and discharges associated with the Stage 2A expansion, migration pathways and receptors.

2.8.1 Potential sources of contamination

Potential sources of contamination from Stage 2A include:

- Infiltration of rainwater into the landfill.
- Asbestos waste.

2.8.2 Potential migration pathways

Potential migration pathways from Stage 2A include:

- Migration of infiltrated rainwater through the Stage 2A landfill and lining system.
- Migration of infiltrated rainwater through the landfill into Stage 2 over the existing northern batter of Stage 2 leading to migration through the Stage 2 landfill.
- Asbestos migration through surface water from erosion of encapsulating materials from stormwater runoff.
- Asbestos migration through air.

2.8.3 Receptors

Potential receptors include (but are not necessarily limited to):

- The waters of the flood plain east and west of the site (Mickett Creek and Buffalo Creek flood plains respectively) and their flora and fauna ecosystems.
- On-site personnel and visitors.

2.8.4 Existing and proposed mitigating factors

The following are existing and proposed mitigating factors associated with the potential migration pathways listed above for the Stage 2A landfill expansion:

- Migration of infiltrated rainwater through the Stage 2A landfill and lining system. A basal liner system is included in the design that limits seepage through the base of the landfill. Asbestos waste has no chemical signature therefore seepage through Stage 2A is not expected to impact groundwater quality. The liner system is designed to limit seepage from the Stage 2A cell, which will limit the potential of asbestos migrating from the cell.
- Migration of infiltrated rainwater through the landfill into Stage 2 over the existing northern batter of Stage 2 leading to migration through the Stage 2 landfill. WSP developed seepage modelling to compare the current infiltration through the outer slope of Stage 2 relative to the infiltration into Stage 2 from the filling operation of an asbestos waste cell in Stage 2A. A copy of the technical memorandum presenting the analysis is provided in Appendix B. Based on the results of the analysis, the Stage 2A expansion over the existing Stage 2 waste slope was considered to result in less infiltration into the existing Stage 2 area than the modelled current conditions for the existing northern batter of Stage 2. Therefore, Stage 2A expansion is not expected to negatively impact the existing CSM (URS, 2016) for Stage 2 with regards to leachate within the waste mass. Key operational conditions, relative to the modelling undertaken, are included in Section 12.
- Asbestos migration through surface water from stormwater erosion of encapsulating materials. Asbestos waste is required to be covered by a minimum 1 m thick of cover soils in accordance with the License (amongst other requirements). This controls potential migration from surface water erosion and is consistent with current operating conditions for Stage 2.
- Asbestos migration through air – asbestos waste is required to be covered by a minimum 1 m thick of cover soils in accordance with the License (amongst other requirements). This controls potential migration though air and is consistent with current operational conditions for Stage 2.

3 Regulatory requirements and guidelines

3.1 NT EPA Landfill Guidelines

Extracts from the NT EPA landfill guidelines considered relevant to the design of a landfill cell are presented in Table 3.1.

Table 3.1 NTEPA Landfill Guideline Extracts

Page reference	Guideline extract	Design outcome
27	"Cells should be open for a maximum period of two years, then closed, covered and rehabilitated."	The Stage 2A expansion cell is small in size, when compared to other landfill cell construction footprints. The recommended timeline of filling of two years may be exceeded due to the rate at which CoD are currently receiving the asbestos waste stream. The cell will be filled more quickly if the demand for asbestos waste airspace increases during its service life. Section 7 details a temporary sub-cell which will allow CoD to place waste into a smaller portion of the cell and temporarily detain stormwater from the waste placement area. This will reduce generation of asbestos contact water in the initial filling stages of the cell. As per the License, asbestos waste is to be buried or covered by 1 m for stabilised asbestos and 3 m for asbestos fibres waste. CoD has indicated that the stabilised asbestos waste is currently immediately covered with 1 m of clean soil.
32	"Groundwater management is an important consideration in the design and operation of a landfill. The groundwater needs to be managed so that: — the groundwater does not adversely affect the landfill, in particular the liner system; — the normal flow of groundwater is not adversely affected by the landfill; and — the quality of the groundwater is not adversely affected by the landfill."	The design includes a groundwater depressurisation system incorporating a 100 mm thick drainage blanket of gravel, draining towards a gravity outlet which exits the cell alongside the sump, to the northeast of the cell floor. The groundwater drainage discharge point is proposed to become a water monitoring location as part of operational monitoring, consistent with other infrastructure on site to monitor for any changes in groundwater analytes.

Page reference	Guideline extract	Design outcome
32	"The base of the landfill and liner system should be located above the groundwater table with an unsaturated zone immediately below the liner of at least 2 metres."	Due to shallow seasonal groundwater levels reaching approximately the ground surface level during wet season, a minimum separation to the underside of the liner system of 0.5 m has been incorporated into the design. Based on EcOz (2020), groundwater levels were reported to be greater than 12 mbgl at the end of the dry season across the site.
32	"Surface water management is required to ensure that: — contaminated surface run-off from the active fill area does not enter water courses; — rainfall run-off from surrounding areas does not drain into the landfill; — surface water and stormwater does not generate excessive quantities of leachate; and — ponding and erosion on filled and capped landfill surfaces is minimised."	A surface water drain has been designed to divert potentially sediment impacted water along the perimeter of the Stage 2A expansion, to a level spreader located to the east of Leachate Pond 1. During operations, a stormwater bund is proposed to be progressively raised on the Stage 2 existing north slope ahead of the waste elevation to divert stormwater runoff from the cell.
33	"Rainfall falling on the active fill area should be collected and managed as leachate via the leachate collection, treatment and disposal system."	Rainfall within the operating landfill area will be captured by the leachate collection system. Leachate will be pumped to the covered leachate storage ponds and the leachate treatment plant, north of the Stage 2A area.
35	"Lateral extensions of existing landfills need to provide an appropriate level of retention to protect the environment from the adverse effects of leachate entering the groundwater and surface waters. This would generally comprise: a leachate retention or liner system; and a leachate collection system."	The Stage 2A expansion comprises a baseliner and a leachate collection system for extraction of leachate.
36	"A single liner system should generally comprise 900 mm of clay or other low permeability soil compacted in layers a maximum of 150 mm thick, to achieve a coefficient of permeability not exceeding 1×10^{-9} m/sec."	The design comprises a Geosynthetic Clay Liner (GCL) layer as the landfill baseliner which has a lower estimated seepage rate than the 900 mm clay layer recommended in the NT EPA Guidelines (see Section 6.2.8).

Page reference	Guideline extract	Design outcome
39	"A typical leachate collection system consists of the following components: — A drainage layer consisting of highly permeable aggregate material, either sand or gravel; — A network of perforated pipes laid within the aggregate layer; — Filter layers of aggregate or geotextile fabric where needed to prevent clogging; and — sump(s) at low points within the system from where leachate can be collected."	The Stage 2A design incorporates a leachate collection system comprising a geocomposite drainage medium with a network of perforated pipes to convey leachate to the sump. The leachate collection system grades towards the low point in the northeast corner of the cell, where there is a leachate sump with a leachate riser pipe (see Section 6.3).
44	"The CQA plan must contain the material/construction specifications, testing methods, testing frequency, corrective action and provide for appropriate documentation procedures."	The design documentation incorporates a CQA plan containing the items identified in the NT EPA Guidelines.

3.2 Environmental Protection Licence requirements

The Site is leased by Council from the Department of Defence and operated under the Northern Territory Environment Protection Authority (NT EPA) Environment Protection Licence EPL 188-04 dated 02 July 2021 (the Licence). The Licence requires that the site be operated in accordance with the Environmental Management Plan for Shoal Bay Waste Disposal Site, document reference 189753-43 (Rev 2), dated 21 August 2020 (the EMP).

Licence conditions considered relevant to the design of the Stage 2A inert landfill expansion are presented in Table 3.2.

Table 3.2 Licence conditions

Condition no.	Description	Design outcome
5.5.1	"Sediment impacted water will be discharged into a sediment basin located within the existing disturbed area north of Stage 3/4 general waste landfill."	Due to lower ground elevations at the Stage 2A location in comparison to the sediment basin location, Condition 5.5.1 is not considered viable. To remove the need to pump sediment impacted water west towards the disturbed area north of Stage 3/4, sediment impacted water shall be directed towards the north of Stage 2A. The required stormwater drainage infrastructure was designed to effectively convey surface runoff along the western and northern perimeters of Stage 2A, modifying the existing drainage pathway that currently services runoff from this portion of the Stage 2 landfill. Erosion and scour protection features were incorporated into the drain design to reduce sediments in the discharging water prior to its release into the environment. The Environment Management Plan (EMP) (CoD, 2023) assesses the risk of sediment laden runoff on receiving surface waters with proposed controls as low. The proposed treatment train approach for Stage 2A is consistent with the proposed approach for the general waste landfills onsite. The Stormwater Management Strategy Document (ref.: 20140796-004-R-Rev2) notes a constraint on the low-lying land to the east of Stage 2 is unsuitable due to inundation during highest astronomical tides storm surge events, but the proposed controls for Stage 2A are positioned above this inundation zone. Additionally, the document notes the existing native habitat to the east provides a large buffer to waterways located outside the Shoal Bay Landfill boundary and will provide some natural attenuation for sediment and nutrients released in stormwater runoff. On the basis of the above, the approach is considered to be consistent with other areas onsite and is not considered to change the current risk assessment in the EMP.
5.6.1	"Leachate that moves laterally from Stage 1 and 2 waste cells is recovered using interception trenches along the eastern toe of both stages. A gravity drain transfers the leachate to a single sump that is pumped to an irrigation area over the eastern side of Stage 1."	This condition is referring to the historical unlined landfill (Stage 1 and 2). The leachate generated by the Stage 2A expansion will be managed by the low permeable baseliner and leachate collection system. CoD has confirmed that the location of this interception trench is unknown and not documented. The location and performance of the inferred leachate interception trench shall be investigated and considered as part of any future development to the east of the Stage 2 landfill. This trench is not affected or relied upon for Stage 2A.

4 Safety in Design

The Stage 2A expansion has incorporated the principles of Safety in Design. The risk register included in Appendix A summarises risks identified during design, management measures included in the design, and management measured to be implemented during construction and operation. The assessment is not intended to consider all risks associated with construction and operation of Stage 2A, but to present considerations for key risks associated with the design, construction and operation of the cells. The Contractor must undertake its own assessment of risks associated with the construction of the works and mitigate these risks accordingly.

5 Sustainability in Design

WSP is committed to integrating and innovating the principles of Sustainability in Design (SustID) into our work across all stages of a project. By helping our clients design lasting solutions for the advancement of our built and natural environments, we are building a powerful legacy.

Resolving a project's design response to climate change, resource efficiency, whole of life impacts, certification and compliance, and governance and procurement is our priority. Our teams coordinate the multi-faceted and complex sustainability requirements in landfill projects, with the aim of knitting together disparate discipline teams with varying design objectives.

The Stage 2A landfill expansion incorporates a Geosynthetic Clay Liner (GCL) in the design instead of a Compacted Clay Liner (CCL) which has sustainability benefits. The construction of a conventional CCL has a considerably higher carbon footprint compared to a GCL, particularly when implemented at a location with limited clay sources located remote from the site. The inclusion of a thin barrier system also improves the facility efficiency by minimising the consumption of airspace volume by the volume of conventional materials.

WSP understands that CoD has a variety of environmental sustainability goals they are targeting in the coming years, by implementing these sustainability considerations and substituting more sustainable options where possible. Carbon emissions will be reduced during the procurement and construction of the Stage 2A expansion.

6 Detailed design elements

6.1 Groundwater drainage system

6.1.1 General

The subgrade drainage layer is designed to separate groundwater from the base of waste. The proposed separation distance for Stage 2A is 500 mm. The subgrade drainage system also manages the risk of potential groundwater uplift on the cell liner (prior to waste placement) by draining groundwater that may rise to within 0.5 m of the liner during the wet season during high seasonal groundwater levels.

The groundwater drainage system in the expansion comprises of:

- Filter geotextile,
- 0.1 m thick drainage gravel layer,
- Perforated DN110 SDR11 HDPE pipes running along the northern and eastern embankment toe towards the sump,
- A solid DN110 SDR11 HDPE outlet pipe running from under the sump location to the discharge point.

The groundwater drainage system has been designed to manage and drain the groundwater. The system will capture the groundwater during the wet season where it is known the groundwater fluctuates and can fully saturate surface soils. The groundwater drainage pipe outlet has been designed to discharge to environment above the Highest Astronomical Tide (HAT) level and above the 1 in 100-year storm surge level to provide a free draining system.

6.1.2 Groundwater flow estimation

The groundwater drainage layer is designed to accommodate the groundwater flow from the western sloping ground soil layer. The groundwater flow has been estimated using Darcy's Law and parameters selected are based on a current understanding of site conditions. A hydraulic conductivity of the soil 2.5 x 10^{-6} m/s, a thickness of 3.6 m measuring from existing ground level to porcellanite rock depth (maximum depth noted from test pit investigation on 2 December 2023) and a hydraulic gradient of 1 was used to estimate the groundwater flow rate drawing down the water level in the soil layer and flowing into the drainage layer.

The flow rate was estimated at approximate 1.6 L/s (140 m^3/day) into the groundwater drainage layer. This is considered to be a conservative estimate for the temporary peak flow rate when the entire soil layer above the rock level is fully saturated.

6.1.3 Groundwater drainage gravel

The specified particle size distribution for the groundwater drainage gravel limits sand or smaller size particles (fines) to less than 3%. The proposed gravel particle size distribution was compared to design charts (NAVFAC DM 7 Part 1: Soil Mechanics, by the US Naval Facilities Engineering Command, September 1986) to assess its expected hydraulic conductivity. The assessment indicates gravel with the proposed particle size distribution is expected to have a permeability of between 2.7 x 10^{-2} m/s and 3.7 x 10^{-1} m/s, which is similar to typical permeabilities of drainage gravel and is considered suitable for the estimated peak flow rate in the 100 mm thick layer. The flowrate capacity within the gravel layer results in 9.9 L/s which is greater than the estimated 1.6 L/s of groundwater inflow. Due to the cell floor geometry and dimensions, the proposed perforated pipe layout has a maximum flow length of 55 m on the 2.2% grade surface and 65 m on the 4.9% graded surface. The proposed spacing results in a maximum flow depth within the layer of 20 mm.

The Technical Specification also requires that the subgrade drainage gravel have a maximum Los Angeles abrasion of 25% to manage risks of break-down of particles during construction.

6.1.4 Groundwater drainage pipes

Perforated groundwater drainage pipes are proposed to be installed along the toe of the northern and eastern embankments, carrying groundwater from the groundwater drainage gravel blanket towards the outlet pipe, as shown on the Design Drawings.

The groundwater drainage pipes require a 110 mm nominal diameter, standard dimensional ratio (SDR) of 11 with a minimum 1% grade. This size has been selected based on assessment of the flow capacity of a pipe this size, using the Mannings Equation, relative to estimated flow from the groundwater flow estimation. It assumes a 1% grade on the pipe, and flow capacity reduced by 20% due to clogging of the pipe.

The assessment indicates that the proposed pipe in the design scenario is expected to have sufficient capacity of 4.5 L/s for drainage of 1.6 L/s of groundwater, resulting in a factor of safety of 2.8 which is greater than the minimum design factor of safety of 2.0 for pipes and considered acceptable for the application.

The pipes are to be constructed with 5 mm circular perforations, specified at 45 degrees to the vertical axis of the pipe, to avoid perforations in high bending stress zones of the pipe wall. An assessment of pipe strength was completed assessing the perforated drainage pipes under an overburden pressure comprising a maximum of 6 m of soil and waste as well as a conservative construction load. The assessment results in a factor of safety of 3.3 which is greater than the minimum design factor of safety of 2.0 for pipes.

6.1.5 Groundwater outlet pipe

A solid groundwater outlet pipe is proposed to be installed at the north-east corner of the cell, carrying groundwater from the groundwater drainage system to the discharge point, as shown on the Design Drawings.

The outlet pipe requires a 110 mm nominal diameter, standard dimensional ratio (SDR) of 11 with a minimum 0.5% grade. This size has been selected based on assessment of the flow capacity of a pipe this size, using the Mannings Equation, relative to estimated flow from the groundwater flow estimation. It assumes a 0.5% grade on the pipe, and flow capacity reduced by 20% due to clogging of the pipe.

The assessment indicates that the proposed pipe in the design scenario is expected to have capacity of 3.2 L/s for drainage of 1.6 L/s of groundwater, resulting in a factor of safety of 2.0 which meets the minimum design factor of safety of 2.0 for pipes and considered acceptable for the application.

A valve has been included at the outlet to allow a future connection if needed to remove and further manage collected groundwater from below the cell.

6.2 Liner system components

6.2.1 General

Existing site knowledge indicated that there are limited viable clay sources near the site that may be suitable for the construction of a compacted clay liner in accordance with the NT EPA guidelines. Hence an alternative liner system has been adopted for the majority of the cell area with a wedge of compacted low permeability soil liner proposed along the toe of the existing northern batter of Stage 2 to provide an offset to the geosynthetics from potential heat source within the existing landfill (refer Section 6.2.7).

The proposed main lining system comprises of (from bottom to top):

- a uniform and firm subgrade
- filter geotextile beneath the groundwater drainage gravel layer
- 100 mm groundwater drainage blanket
- filter geotextile across the groundwater drainage gravel layer
- 500 mm compacted Select Fill on the floor

- Geosynthetic Clay Liner (GCL)
- geocomposite drain with perforated leachate collection pipes at spacings
- 300 mm of a soil protection layer.

These elements complement each other to achieve the primary function of collecting landfill leachate and limiting potential leachate migration to groundwater.

6.2.2 Geosynthetic Clay Liner

A GCL is to be used on the cell floor and batters. The GCL will be installed over the prepared 500 mm thick Unit 2 Select Fill. The GCL is to be reinforced between the non-woven cover geotextile through the clay core and carrier geotextile.

The GCL is specified to be supplied with additional bentonite applied to the overlaps and manufacturer testing of rolls to be delivered to site will be required to demonstrate conformance of the supplied materials with this requirement. The Technical Specification requires that GCL be installed with minimum 300 mm wide overlaps.

The Technical Specification for the GCL requires a permeability of $\leq 2.5 \times 10^{-11}$ m/s using water as a permeant. The Technical Specification also requires that the underlying soils be moisture conditioned to near or wet of the Standard Optimum Moisture Content before being covered with GCL to provide moisture to hydrate the GCL with water and keep it hydrated under the expected landfill conditions.

The material specified for the GCL including the geotextile components are typical of materials used in the landfill industry and are chemically resistant to leachate.

The Technical Specification includes the requirement for the GCL to be covered with the geocomposite drain and soil protection layer on the same day it is deployed. This is intended to manage hydration and potential for GCL unconfined swelling and/or degradation due to hydration under lack of the confining pressure and potential damage from rainfall events. The Technical Specification allows the Contractor to propose alternative temporary protection controls to extend the timeframe for placement of the soil protection layer, subject to approval by the CQAI and Auditor. The 300 mm thickness of a soil also provides protection layer over the liner system. Provision of confinement loading is relevant to construction with GCL to control hydration and to manage risks associated with shrinkage and erosion of bentonite on side slopes.

There is potential for a delay in the waste filling over the western portion of the cell. The overlying geocomposite drain layer (which forms an air gap from the geonet component) provides a suction break above the GCL which controls the potential for drying of the GCL that could otherwise lead to desiccation and cracking of the bentonite in the GCL. The 300 mm thick overlying protection soil also provides a temperature buffer to the underlying materials. The specified sandy nature of the protection layer also presents a low potential soil suction profile.

6.2.3 Geocomposite drain

A geocomposite drain is included in the design to overlay the GCL on the cell floor to provide drainage of leachate towards the sump and to provide further protection of the GCL from placement of the soil protection layer. As asbestos has no biological clogging effects, a geosynthetic drainage material (geonet) has been proposed to control the leachate head instead of a gravel layer. Drainage design details for the geocomposite drain are discussed in Section 6.3.4.

The proposed geocomposite drain is a HDPE drainage core with geotextile bonded to each side of the geonet core. The requirements of the geocomposite drain are outlined in the Technical Specification.

6.2.4 Filter geotextile

Filter geotextile is required around the leachate drainage gravel mounds which surround the perforated leachate collection pipes and the gravel in the sump. A filter geotextile is also included on the batters above the GCL where the geocomposite drain does not extend to provide protection of the GCL from placement of the soil protection layer.

The design has considered particle retention, clogging and permeability of the filter geotextile and requires a geotextile in accordance with Giroud (2000) with the following properties:

- Minimum mass: 200 g/m^2
- Apparent opening size (O^{95}): less than 0.11 mm
- Permittivity: at least 1.8 s^{-1}

The filter geotextile is specified to be a non-woven needle-punched manufacture.

The filter geotextile is proposed to be covered in the design and is therefore not exposed to UV degradation.

The specified filter geotextile is typical for materials used in the landfill industry and is chemically resistant to leachate.

6.2.5 Soil protection layer

A 300 mm thick soil protection layer is included in the design over the liner system to provide confinement loading on the GCL and to provide a protection layer over the geocomposite drain and GCL from initial waste placement in the cell.

A sand rich material is specified for the soil protection layer. It is specified to enable drainage to the geocomposite layer and to limit the risk of large particles puncturing or deforming the GCL.

The interface friction between the GCL and the soil protection layer is expected to be greater than the angle of the perimeter embankment internal batters (18 degrees), therefore veneer stability is not expected to be an issue for the period the soil protection layer is exposed before being covered by waste. Based on Koerner, G.R., and Narejo, D. (2005), the interface friction between non-woven geotextile (top component of the GCL) and a granular soil (soil protection layer) is in the order of 33 degrees.

6.2.6 Anchor trench

An anchor trench is to be constructed at the crest of the northern, eastern and western embankments, and the southern bund, to secure the filter geotextile and underlying GCL.

Wind uplift has not been accounted for in the design of the anchor trenches due to the condensed construction timing. The soil protection layer will be placed immediately following GCL installation, resulting in ballasting the floor. The slope lengths vary in size ranging between approximately 3 m to 8 m (maximum slope length measuring from the bottom of sump to the embankment crest).

Based on the self-weight of the soil protection layer, the tensile load is minimal and the size of the anchor trenches has been driven by constructability considerations. The anchor trench size considers the practicality of construction and is specified to be 0.5 m deep and 0.5 m wide with a 1 m offset from the crest, as shown in the Design Drawings.

The anchor trench is to be excavated into Select Fill (on north, west and eastern embankments) and Low Permeability Soil (on the southern embankment) at the crest of the perimeter embankments. The purpose of the wedge of Select Fill is to enable construction of anchor trenches with surfaces suitable for placement of geosynthetics over the excavated surface. The wedge extends to the level of the base of the anchor trench. The Technical Specification includes requirements that the inside crest of the anchor trench have a uniform and regular geometry, to manage risks associated with potential crumpling and related stress concentration at this location. The Technical Specification also requires that sharp or angular particles be removed from the trenches prior to installation of geosynthetics, to provide a suitable surface for placement of geosynthetics in close contact with these surfaces.

The anchor trench is to be backfilled with Select Fill (on north, west and eastern embankments) and Low Permeability Soil (on the southern embankment), which is to be moisture conditioned and compacted in layers to achieve a minimum dry density ratio of 95% relative to Standard compaction. This will also provide a relatively low permeability zone to manage risks associated with ingress of surface water into the anchor trench.

6.2.7 Compacted low permeability soil liner

A layer of compacted low permeability soil liner is included along the toe of the existing northern batter of Stage 2 to provide an offset to the geosynthetic liner system from a potential heat source within the existing landfill and to continue the liner system for the new cell to the Stage 2 northern toe. The offset is based on the results of the heat transfer assessment (Appendix C) whereby a 3.5 m thick (perpendicular to the existing landform surface) was proposed. The offset to the geosynthetic liner system is based on a projection of the 3.5 m thick layer into the new cell (approximately 20 m horizontal distance from the toe).

The compacted low permeability soil liner has been specified to meet a permeability of less than or equal to 1 x 10^{-9} m/s to achieve the NT EPA recommended single liner system of 900 mm of compacted low permeability soil with a coefficient of permeability not exceeding 1 x 10^{-9} m/s. The layer has been designed to provide a nominal thickness of 1 m over the inferred existing waste extent edge and into the new cell. The layer extends to tie-in with the existing Stage 2 northern batter with grade towards the new cell to promote drainage of seepage into the new cell where it can be collected and managed.

CoD has indicated through their Contractors that the compacted low permeability soil layer should be achievable with select soils from local sources and potentially higher compactive effort (if required). The technical specification has included an option for higher compactive effort to help achieve the permeability criteria.

The geosynthetic drainage system (geocomposite drain) that is proposed over the floor of the cell is not proposed to be extended onto the compacted low permeability soil layer due to the potential heat source within the existing landfill. The following has been included to limit seepage and promote drainage over this area into the new cell:

- minimum 900 mm compacted low permeable material
- an approximate 3% grade into the new cell has been included on the crest of the low permeability soil layer
- a 300 mm thick layer of the soil protection layer, which is a moderate to high permeable material, has been extended over the compacted low permeability soil layer to promote drainage into the new cell.

The soil protection layer, which comprises a minimum of 90% sand particles, provides a suction break above the compacted low permeability soil liner which limits the potential for desiccation cracking and softening if left exposed to the weather before being covered by waste.

The requirements for the compacted low permeability soil layer are outlined in the Technical Specification.

6.2.8 Liner seepage

An estimate was prepared for the seepage rate through a conventional GCL using Darcy’s Law. The seepage rate was compared to the NT EPA recommended single liner system comprising 900 mm compacted clay liner. Assessing a 300 mm leachate head on the 900 mm compacted clay liner results in a seepage rate of 1152 L/day per hectare, while the conventional GCL results in a seepage rate of 670 L/day per hectare. Hence the conventional GCL meets the seepage criteria. The cation exchange capacity aspect of the liner system is not considered relevant to asbestos contact leachate and was therefore not considered in the equivalency assessment of the liner systems presented in the NT EPA Landfill Guideline.

6.3 Leachate collection system

6.3.1 General

The geometry of the cell has been designed to facilitate gravity drainage of leachate towards a sump located at the low point. The leachate collection system sump is located in the north-eastern corner of the cell. The system has been designed to manage and extract leachate collected from the cell. Leachate from the sumps will be removed by pumping from the riser pipe. The normal operating maximum depth of leachate on the cell floor is 300 mm. This depth will be controlled by an automated control of the pump in the sump.

The leachate collection system in the expansion will comprise of:

- geocomposite drainage layer
- HDPE perforated pipes wrapped in aggregate mound and filter geotextile
- leachate collection sump filled with aggregate with riser and pump.

The leachate collection system has been sized to manage a leachate generation rate of 5.85 mm/hr. This value is 50% of rainfall in a 1 in 20-year, 24-hour design rainfall event (BOM, 2024). Assuming 50% of rainfall from this design rainfall event will become leachate in the landfill is considered a conservative assumption based on the following:

- Filling of the first lift of waste across the cell floor is proposed to occur in the dry season. Commencing filling in the dry season is intended to manage the risk of temporary high leachate generation rate during heavy rainfall events due to the distinct wet season in Darwin.
- A temporary sub cell has been included in the Stage 2A design to allow waste placement in the lower portion of the cell while diverting clean water which falls in the unoccupied portion upstream of the separation. This segregation of clean water from leachate during the first wet season of operation will reduce the overall leachate generation in the early stages of operation.
- Waste placed in the cell will be shaped with side slopes graded at 5H:1V to 3H:1V and operating area at 20H:1V and covered with required asbestos cover soil layer to form a free-draining platform to promote sediment water runoff into the operational stormwater chute.
- A temporary stormwater bund will be built to the south of the cell (this will be constructed concurrently as the landfill is raised), diverting runoff from the existing Stage 2 slope around the cell expansion.

6.3.2 Geocomposite drain

The geocomposite drain has been design based on the following:

- Leachate infiltration rate of 5.85 mm/hr as per Section 6.3.1.
- Floor grade of 2% and 3% for the eastern and western portion of the cell respectively.
- Reduction factors:
  - Intrusion: 2 (based on intrusion of geotextile from cover soil and loading).
  - Creep: 2 (based on loading of Stage 2A).
  - Chemical clogging: 1 (based on asbestos contact water).
  - Biological clogging: 1 (based on asbestos contact water).
- Factor of safety of 1.5.
- Spacing between collection pipes of 12 m and 16 m for the eastern and western portion of the cell respectively.

Based on the above, the specified minimum flow capacity for the geocomposite drain was estimated to be approximately 10109 L/day/m for the 2% floor grade at 12 m pipe spacings. The maximum inflow rate per unit width without the reduction and safety factors was estimated to be approximately 1685 L/day/m for the 12 m pipe spacing.

The required geocomposite drain flow rate was compared with the flow capacity of off the shelf products which indicates geonets in the range of 8 mm thick geonets will likely meet the required flow capacity.

The requirements of the geocomposite drain are outlined in the Technical Specification.

6.3.3 Leachate collection pipes

The leachate collection pipes have been designed to allow for the collection and removal of landfill leachate. Spacings of the leachate pipes are based on the drainage capacity of the geocomposite drain (refer Section 6.3.2) and have been spaced at a maximum of 12 m and 16 m for the eastern and western portion of the cell, respectively, to suit the specified required flow capacity of the geocomposite drainage material.

Leachate collection pipes are proposed to be installed running from south to north, connecting to a primary collection pipe, running west to east along the toe of the northern embankment. Another primary collection pipe is located at the toe of the eastern embankment, running from south to north. Both primary collection pipes drain directly into the sump, as shown on the Design Drawings.

Flow capacity calculations indicate that the collection pipes require a 140 mm nominal diameter, standard dimensional ratio (SDR) of 11 with a minimum 1% grade. The pipe alignments have been positioned to provide a minimum of 1% grade. The leachate collection pipes have a flow capacity factor of safety of 2.7 which is greater than the minimum design factor of safety of 2.0 for pipes.

The pipes are to be constructed with 10 mm circular perforations, specified at 45 degrees to the vertical axis of the pipe, to avoid perforations in high bending stress zones of the pipe wall. The leachate pipes are designed to have a factor of safety of 3.0, which is greater than the design factor of safety of 2.0, against buckling/crushing under 20 m of waste loading (the maximum thickness of waste in the Stage 2A expansion). The assessment considers the unit weight of waste to be 18 kN/m^3.

The leachate collection pipes are wrapped in aggregate mound and filter geotextile to convey leachate from the geocomposite drain into the pipes.

6.3.4 Leachate sump and riser pipe

The sumps have been designed to retain a minimum 0.5 m separation between base of the sump and groundwater. The perimeter leachate collection pipes direct flow to a sump located in the north-eastern corner. The sump has been nominally sized to suit construction, 1 m deep with a 3 m by 3 m base included in the drawings.

A sloping riser pipe (630 mm nominal diameter, SDR of 11) is included in the sump for leachate extraction. The pipe has been sized to account for the leachate reporting from the cell floor and interface layer batter. The pipe diameter has been selected to account for the pump size as described in Section 6.3.5 and is consistent with sump riser pipes in Stages 5 and 6 at the SBWMF.

The riser pipe shall have 10 mm circular perforations in the lower portion of the riser (in the sump), specified at 15 degrees, 45 degrees and 75 degrees to the vertical axis of the pipe at 300 mm centres along the pipe. The riser pipe has been designed to have a factor of safety greater than 2.0 against buckling/crushing for up to 6 m of waste based on the concept capping shape at this location. An estimated 18 kN/m^3 unit weight of waste has been considered in the assessment.

The sump is filled with a drainage aggregate to convey leachate collected in the sump to the riser pipe and pump. The leachate drainage gravel is a nominal 26.5 mm uniform graded material which is aligned with previous designs at SBWMF.

6.3.5 Leachate pump design

The design leachate pumping rate of 36 m^3/day has been based on actual leachate pumping data from the lined landfill stages at the SBWMF. The maximum monthly pumping rate measured by flowmeter, in Stage 3 and 4 in 2023 (which is geomembrane lined) has been considered in the Stage 2A pump design. The lined area reporting to each sump in Stage 3 and 4 was compared to the lined area reporting to the Stage 2A sump, this comparison found that the area reporting to each pump in Stages 3 and 4 was within 1% of the area reporting to the Stage 2A pump. Therefore, the pumping rate of each sump/pump in Stage 3 and 4 was used as a lower bound pumping rate for the Stage 2A expansion.

The leachate pump selected to achieve the leachate pumping rate is the Airwell BF6010 series submersible pneumatic pump. This pump has been selected based on:

- Current use of Airwell pneumatic pumps within other landfill stages across the site. Consistent pump type use allows for ease of maintenance, consistent spare parts inventory and operation understanding by maintenance contractor.
- 6” (154.2mm) diameter, 1m long, bottom filling: short pump length and bottom filling allows the pump to be installed as deep as possible within the leachate riser pipe so that the overall standing level of leachate with the sump is kept to a minimum.
- Flowrate of 0.56 L/s (2.016 m^3/hr) @ 40m head: meets the design intent of 1.5 m^3/hr. Pressure loss calculations show friction losses of 4.21m @ 3.6 m^3/hr (max pump performance) with DN50 HDPE PE100 SDR11 PN16 leachate extraction (estimated 10m length), transfer pipe (140m length) and main header pipe DN200 HDPE PE100 SDR11 PN16 (150m length to discharge point). Calculations also account for losses in the leachate extraction manifold (DN40 Schedule 80 PVC). Including the elevation difference of 4.2m the total dynamic head losses are approximately 8.4m. At the pump flowrate of 2 m^3/hr it leaves approximately 31.6m of head pressure at the discharge point.
- Air consumption at the maximum pump performance of 1 L/s (3.6 m^3/hr) is estimated to be 10.7 CFM @ 350kPa. WSP has not undertaken an assessment of the compressed air network onsite, current usage rates or the potential capacity available in the existing network to supply compressed air to the leachate pump. It is understood this assessment will be undertaken by the Contractor.
- Material of construction is 316 stainless steel throughout with urethane internal seating balls. The 316 SS is considered to provide acceptable corrosion resistance against landfill leachate.

6.3.6 Leachate pump controller

The Airwell solar pump controller with S/S valve (SCICC-V9) has been selected to control the operation of the Airwell submersible pump by controlling when compressed air is released to the pump to displace leachate within the pump chamber. The pump controller uses a 3-way valve with solenoid connected to an internal circuit board which relays a signal from the conductivity sensors installed within the pump chamber. When the chamber fills with leachate it triggers the high-level sensor, the solenoid opens and compressed air is allowed to flow through to the pump chamber, when the chamber empties the leachate level reaches the low level sensor and compressed air supply is stopped. When the chamber is not under pressure it fills with leachate again and the process is repeated.

The pump controller has been selected based on the following:

- Current use of Airwell equipment onsite and compatibility with the Airwell pneumatic pump.
- Off the shelf item which is solar powered so no requirement for mains electricity or installation by a licenced electrician.
- Internal LiFePO4 12V 4.5AH battery and 5W solar panel.
- Digital cycle counter which can be used to estimate the volume of leachate pumped.

No remote telemetry has been included with the pump controller as it is expected that regular routine inspections of the leachate extraction system would be undertaken by site operations personnel.

No hazardous area classification or assessment has been undertaken as part of the design by WSP. The Contractor is to review and assess potential hazardous areas created by the leachate riser pipe, sampling points and connections within the leachate extraction and transfer pipes and ensure non intrinsically safe equipment and instrumentation (such as the pump controller and flow meter) are positioned outside these areas.

6.3.7 Leachate extraction manifold and flow meter

The leachate extraction manifold is to be positioned at ground surface next to the leachate pump controller. The manifold is connected to the outlet of the leachate extraction pipe and is constructed of DN40 Schedule 80 PVC pipe. The manifold consists of the following valving and instrumentation:

- Non return valve: to prevent leachate backflow from other leachate extraction areas onsite from flowing into the Stage 2A leachate collection sump. The leachate extraction pump also includes a urethane ball on the outlet that also acts as a non-return valve.
- Two ball isolation valves: to allow the isolation of the leachate extraction manifold at both ends for maintenance purposes.
- Pressure indicator: to confirm pressure within the manifold, assist with assessing pump operation/ condition and confirmation of no pressure before conducting maintenance activities.
- Battery powered electromagnetic flow meter (Endress and Hauser Proline Promag W800) which consists of the following specifications:
  - DN40 with volume flow measurement error of +/- 0.5% and max process pressure of PN40.
  - Battery powered so that no mains power is required to operate. Battery estimated to last up to 15 years.
  - Materials of construction: 316 SS electrode, PTFE internal liner. Materials compatibility with landfill leachate is considered acceptable.
  - IP66/67 enclosure which is suitable for outdoor use
  - Magnetic flow meter with no moving parts and no pressure loss due to no cross-section constriction. Means longer term reliability, efficiency, and less maintenance than a conventional mechanical flowmeter.
  - Signal output available but no communications back to the pump controller or external source not proposed for this design.
- T-piece with ball isolation valve for leachate sample collection and monitoring (if required).
- Y-strainer to remove particulate matter and foreign objects that may block manifold instruments or valves.

6.3.8 Leachate extraction and transfer pipelines

The leachate extraction pipe is designed to connect to the 2” leachate pump outlet to the leachate extraction manifold. From the leachate extraction manifold the leachate transfer pipe will convey leachate to the main leachate header pipe for discharge to the leachate ponds onsite. The pipelines are to be installed above ground and be routed as shown on the Design Drawings. The leachate extraction and transfer pipe details include:

- DN50, HDPE, PE100, SDR11, PN16.
- Pipe diameter matches the outlet diameter of the leachate pump and friction losses are within an acceptable limit to convey leachate to the main header pipe within the required pump duty (refer to Section 6.3.5).
- The pressure rating of the pipes is 16 Bar which is above the maximum discharge pressure of the leachate pump which in the event of a possible dead head would not burst the pipe. Fittings for the pipe connections to be of similar (or higher) pressure rating.
- Materials of construction appropriate for submergence/ contact with landfill leachate.

- HDPE pipes can generate static electricity with fluid flow which will need to be earthed either side of the magnetic flowmeter via earthing discs to prevent measurement interference.
- The Contractor needs to assess and confirm that the site mains leachate header pipe has sufficient capacity for the estimated flowrate of 36 m^3/hr from the leachate extraction pump and does not have an internal pressure (generated from other leachate extraction areas onsite) greater than the leachate transfer pipe.

6.3.9 Compressed air supply and transfer pipelines

The main compressed air transfer pipe from the compressed air header line onsite to the pump controller will be DN25, HDPE, PE100, SDR11, PN16. Connection to the header line to be made using HDPE tapping band with electrofusion join.

The compressed air supply line from the pump controller to the pump will be DN20, HDPE, PE100, SDR11, PN16. Connection to the leachate pump to be made using a Swagelok® fitting rather than the hose barb fitting that comes with the pump. The Swagelok will ensure a tight, secure connection rated for the pressure of the compressed air supply. It will also aid in keeping the connection together in the event the compressed air supply line is pulled to retrieve the pump from inside the riser pipe.

The Contractor needs to assess and confirm that the site mains compressed air supply is of sufficient volume and pressure to operate the proposed leachate extraction pump at maximum capacity.

6.4 Global slope stability

The global slope stability of the Stage 2A expansion waste landform has been assessed using the commercially available software SlopeW (Geostudio, 2023). The Stage 2A expansion conceptual waste profile extends to approximate RL28 m AHD. However, a higher crest elevation of RL36 m AHD has been modelled based on the Stage 2A Inert Landfill Expansion Concept Design (WSP, 2024) to be conservative.

The general and inert waste profiles identified in the model cross sections are based on the documentation in the Landfill Development Plan for Stage 2 Inert Landfill Shoal Bay Waste Management Facility (Reference Number: 1653988-006-Rev1, dated 2017).

6.4.1 Material parameters modelled

The material parameters applied in the slope stability analysis are presented in Table 6.1.

Table 6.1 Slope stability parameters

Material	Unit weight (kN/m^3)	Effective cohesion (kPa)	Effective friction angle (°)
New Waste – Asbestos(1)	18	0	28
Old Waste – Inert	12	10	25
Old Waste – General (2)	12	10	25
Liner	12	0	14
Existing Ground	20	5	35

Notes:

(1) CoD has indicated that the New Waste – Asbestos is mostly received as Asbestos in Soil (ABINS) materials.

(2) Waste parameters adopted are published values (Dixon, 2005).

6.4.2 *Modelled scenarios*

The slope stability analysis considers the following scenarios: one long-term (final landform) and one seismic loading.

The conceptual final waste landform profile for the Stage 2A batters is proposed to include batters at 1V:4H batter. Therefore, the model batters are at 1V:4H.

6.4.2.1 Long-term assessments

The Stage 2A expansion batter stability was assessed assuming a maximum waste elevation of 36 m AHD. Groundwater was modelled at 8 m AHD to reflect the seasonably variable conditions, where surface soils may become fully saturated during wet season conditions, forming localised perched water aquifers with standing water levels present at the ground surface. The model presents a 'worst case' scenario reflecting a non-functioning groundwater management system, hence the benefit of lowered groundwater has been omitted in the model. A separate phreatic surface was also modelled at 0.3 m above the liner system and interface layer to simulate the maximum leachate head.

Scenario 1 assesses the global stability with the slip surface analysed along the landfill liner system to represent potential movement along the weaker geosynthetic interfaces.

This scenario represents the critical slip surface for the long-term assessment.

6.4.2.2 Seismic loading

Scenario 2 assesses a design earthquake event for an annual probability of exceedance of 1 in 500, advised in Table 3.3 in AS1170.0-2002 Structural design actions Part 0: General principles for a structure with a design life of 50 years and importance level of 2. The importance level of 2 is described in Table 3.1 in AS1170.0-2002 as a structure for which the consequence of failure is considered medium for loss of human life, or considerable for economic, social or environmental consequences.

The horizontal coefficient of acceleration related to a design earthquake was obtained from information in AS4678-2002 Earth-retaining structures and AS1170.4-2007 Structural design actions Part 4: Earthquake actions in Australia.

- Probability factor of (k_p) of 1.0 for an annual probability of exceedance of 1 in 500 based on Table 3.1 in AS1170.4.
- Hazard factor (Z) of 0.09 for Darwin, based on Table 3.2 in AS1170.4.
- Site subsoil class B_e (rock), based on Section 4.2 in AS1170.4.
- Spectral shape factor for the period of zero seconds ($C_h(0)$) of 1.0, based on Table 6.4 in AS1170.4.
- Effective floor acceleration, $a_{floor} = k_p Z C_h(0)$, for a structure mounted on the ground, based on Section 8.2 in AS1170.4. A value of $a_{floor} = 0.09$ was assigned for the assessment.
- Horizontal coefficient of acceleration, $a_h = 0.5_a$, where a represents the peak ground acceleration coefficient for rock motions and represents a_{floor} in AS 1170.4. This approach is consistent with the approach given in USEPA (1995) and is related to the assessment of slope stability over the embankment slope length. The USEPA document indicates that the 50% coefficient of acceleration value can be used for global stability analysis with a target factor of safety of 1.0. WSP has adopted a target factor of safety of 1.1 for this assessment.

A value of $a_h = 0.045$ was assigned for the assessment and input as the horizontal coefficient for the seismic load in the model.

Scenario 2 is modelled based on the critical scenario, where the slip surface is along the liner. This modelled scenario also includes a 20% reduction to the liners shear strength parameter.

6.4.3 *Assessment results*

The results of the slope stability analysis are presented in Table 6.2, which indicate that for the modelled scenarios, the achieved Factors of Safety (FoS) are better than the conventionally adopted minimum values for engineered slopes. The outputs of the model with accompanying section analyses are presented in Appendix D.

Table 6.2 Slope stability assessment results

Scenario	Modelled Factor of Safety	Minimum target Factor of Safety
1 Post filling, static, slip surface along the liner	1.5	1.5
2 Post filling, slip surface along the liner, seismic loading, 20% reduction of liner shear strength	1.1	1.1

6.4.3.1 Sensitivity analysis

Sensitivity analysis has been modelled to understand the impact of elevated leachate levels, modelled at 1 m and variable shear strength of the geosynthetics within the Stage 2A expansion. The leachate level was modelled to the embankment crest within the Stage 2A expansion. The existing Stage 2 landform has been modelled with elevated groundwater levels at RL8 m AHD. The minimum target factor of safety was met for all analyses (Table 6.3).

Table 6.3 Slope stability elevated leachate sensitivity assessment results

Scenario	Modelled Factor of Safety	Minimum target Factor of Safety
1A – Post filling, static, slip surface along the liner, 1 m of leachate in exiting Stage 2 landform	1.5	1.5
2A – Post filling, static, slip surface along the liner, 1 m of leachate in exiting Stage 2 landform, 20% reduction of liner shear strength	1.1	1.1

Sensitivity analysis has been modelled to understand the long-term strength parameters of the lining system, refer to Table 6.4. The Fox and Stark paper reference (2015) refers to an unreinforced GCL strength relating to the long-term conditions with reliance on the needle punching strength, as the durability of the fibres may weaken over time. Typical strength parameters for a GCL were also selected from a GRI test results report. The analyses are a lower bound case, hence achieving a Factor of Safety above 1.3 is considered to represent a robust design. The design considers the internal strength as the long-term critical interface if the fibres degrade.

Table 6.4 Slope stability liner strength sensitivity assessment results

Scenario	Modelled Factor of Safety	Minimum target Factor of Safety
Friction angle of 10.2°, cohesion 2.4 kPa (Fox and Stark Paper, 2015)	1.3	1.3
Friction angle 6°, cohesion 12 kPa (GRI Report, 2005)	1.3	1.3

The chosen liner material parameters (friction angle of 14° and cohesion of 0 kPa) used in the modelling is the minimum strength target for all the interfaces in the lining system, hence it has been selected for this design.

6.4.3.2 Future development

Council has expressed interest in designing the Stage 2A expansion with consideration to future expansion. A scenario titled ‘Phase 3 development’ with a waste profile extending to RL59 m AHD has been considered in the stability model analysis to future proof the proposed final landform model. The critical slip surface through the liner has been modelled for this scenario, and the seismic loading scenario has also been applied to this critical slip surface. The achieved Factors of Safety’s are better than the conventionally adopted minimum values (Table 6.5).

Table 6.5 Slope stability Phase 3 development assessment results

Scenario	Modelled Factor of Safety	Minimum target Factor of Safety
3 – Phase 3 Development – Post filling, static, slip surface along the liner	1.5	1.5
4 – Phase 3 Development – Post filling, slip surface along the liner, seismic loading	1.1	1.1

7 Temporary sub-cell stormwater detention

7.1 General

The temporary sub-cell stormwater detention design has been developed to separate the eastern waste filling area (lower portion of the cell) from the western portion of the cell. A separation is a temporary vertical liner to separate the west area for approximately one wet season of waste placement. The temporary structure has been designed to comprise a vertical piece of polyethylene (PE) sheet to be connected to the surface of the GCL on the floor of the cell and up the northern and southern bunds to a vertical level approximately 300 mm higher than the chute invert level. Bags of leachate drainage gravel will ballast and support both the upstream and downstream faces of the barrier, up to the crest height elevation. The leachate collection pipe along the north side of the cell will be interrupted at the structure, with the intent of the pipe on the west and east sides of the barrier being reconnected by two pipe halves once the barrier is removed. In addition, the geocomposite drain will be interrupted at the structure, with the intent of the gravel in the ballast bags to be spread over the connection once the barrier has been removed to connect the leachate collection system.

The height of the temporary barrier has been proposed to be a maximum of 1.3 m high to tie into an elevation approximately 300 mm higher than the chute invert level at the location of the structure. Stormwater will be collected in the western portion of the cell. When the water level reaches the spillway level, the water will flow over the embankment in a controlled discharge to the stormwater drain to the north of the cell. Spillway/embankment crest erosion control at the location of the operational stormwater chute details can be found in Section 9.

With the revised liner from a coated GCL to a conventional GCL, the seal is less effective than an extrusion weld between polyethylene materials, which is not possible with the updated liner system. The overlap connection between the GCL and polyethylene sheet is expected to restrict flow across the connection to allow removal of water upstream of the temporary structure via pumping or runoff over the chute.

Refer to the Technical Specification for further details on the construction of the temporary sub-cell structure. It is likely that removal of the temporary structure will be carried out by the operations team and so the removal process has been discussed further in Section 12.

7.2 Stability of temporary structure

A stability analysis has been carried out to assess the requirements of the temporary structure supports to prevent the temporary structure from failing once stormwater begins to collect in the upstream zone (western portion of the cell). The support on the eastern face of the PE sheet shall be constructed from UV-resistant ballast bags filled with leachate drainage gravel. Leachate drainage gravel shall be used as the ballast material so that the gravel can be emptied onto the floor of the cell once CoD are ready to remove the temporary structure. To control the stability under the hydrostatic force of the water, 2300 kg/m^3 and 500 kg/m^3 of gravel is required in the ballast bags for water retaining heights of $\geq$ 1 m and < 1 m respectively.

The gravel shall be placed in maximum 20 kg bags (or alternative such as bulka bags) to make placement and removal more practical, with consideration to manual handling limits, as the temporary barrier will be constructed once the base liner system is in place. Leachate drainage gravel is proposed to be used as the ballast material so it can be emptied to fill the void when the temporary structure is removed providing a drainage connection.

The temporary structure should be installed as shown in the Design Drawings.

8 Surface stormwater management

8.1 General

WSP designed the stormwater management drainage infrastructure to effectively manage surface runoff generated by the completion of Stage 2A expansion and a portion of Stage 2B of the SBWMF. This includes mitigation of potential erosion and scour of the drainage infrastructure.

Based on the existing topography, surface runoff originating from the corresponding catchment(s) is currently being serviced by a drain located along Stage 2A footprint. This existing drain conveys runoff from most of the northwestern quarter of the Stage 2 landfill area to its north and discharges into the environment.

8.2 Stormwater management strategy

When designing the stormwater management drainage infrastructure, WSP referred to the NT EPA landfill stormwater management guidelines summarised in Table 3.1 and the landfill license conditions provided in Table 3.2. WSP also considered the provisions specified in the ‘Basis of Design’ document (WSP reference, PS137414-WSP-BNE-ENG-MEM-001 RevA, dated 21 November 2023) and the overarching landfill stormwater management strategy of *Water Balance Model, Leachate and Stormwater Management Strategies Shoal Bay Landfill, Darwin* (Golder (now WSP) reference, 20140796-004-R-Rev2, dated 11 October 2021). The drainage infrastructure design, including hydrological and hydraulic modelling, is also in accordance with the guidelines outlined in the Australian Rainfall and Runoff (ARR 2019) (Ball et al., 2016).

8.2.1 Design concept

The proposed drainage design includes improvements to the existing drain along the perimeter of the Stage 2A expansion footprint so that it will function as an engineered diversion drain. The drain will lead to a coarse sediment trap and a level spreader to dissipate flow energy and provide sediment treatment. Discharge from the drainage system will ultimately daylight into the natural environment, north of the northern boundary of Stage 2A.

This assessment adopted a conservative approach by considering the closure stage catchment in the drainage design. The catchment area is estimated based on the current and landfill design topography. It is noted that the existing culvert shown across the access road, immediately downstream of the northwestern corner of Stage 2A, is no longer in use, and thus no catchment to the west is included in this assessment.

The proposed stormwater management concept is shown in Figure 8.1.

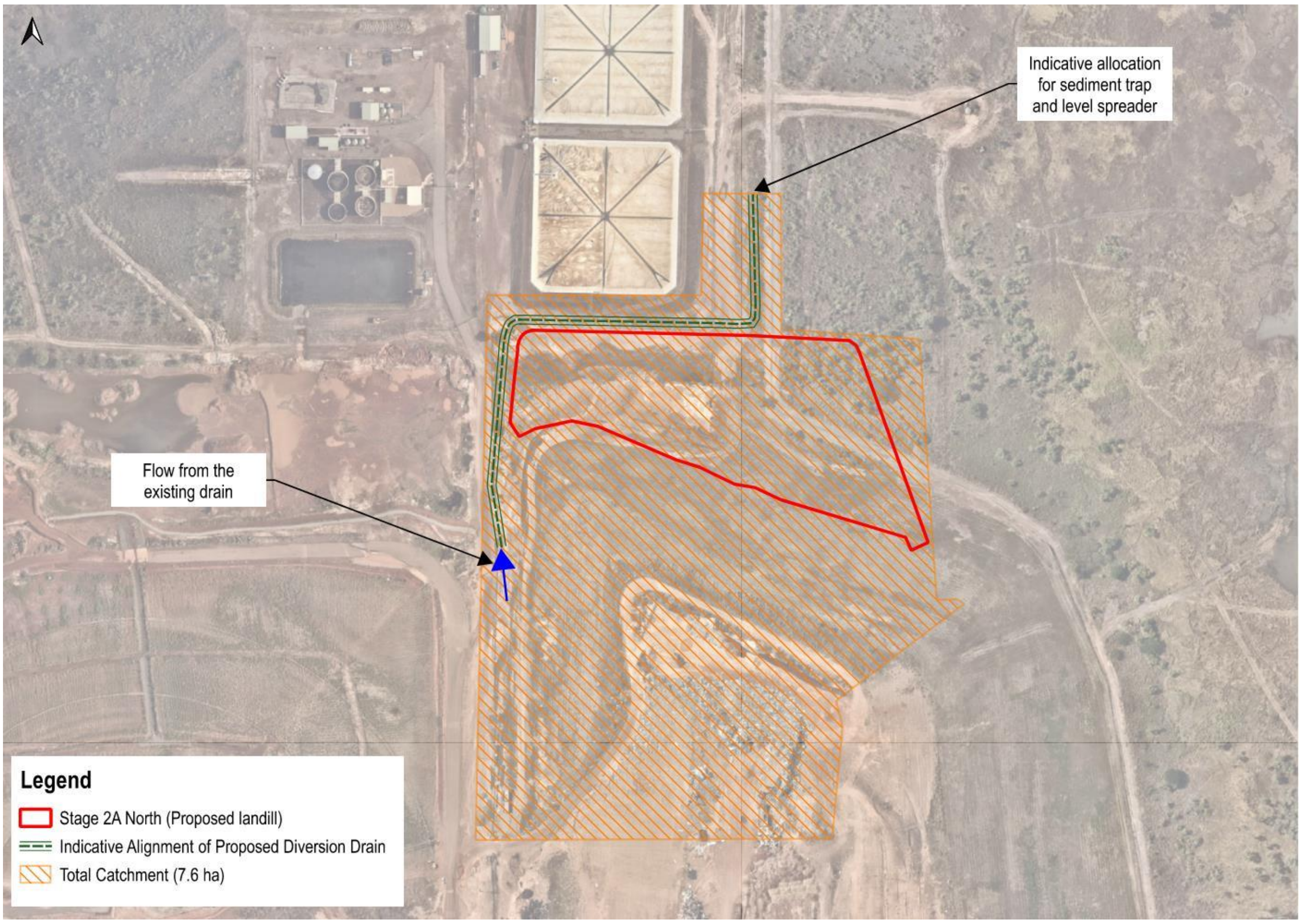

Figure 8.1 Proposed stormwater management concept plan

8.3 Hydrological and hydraulic modelling

To design the drainage infrastructure associated with the proposed Stage 2A expansion area, WSP developed a hydrological and hydraulic model using XPSTORM software (Innovyze, 2024). The XPSTORM is a widely used, robust model that runs catchment hydrological routing and drainage hydraulic routing simultaneously.

8.3.1 Hydrological assessment

WSP undertook the hydrological assessment for the drain design utilising the Laurenson routing method (Laurenson,1964) within the hydrologic platform of the XPSTORM software. The Laurenson method is a well-established technique for modelling the translation and attenuation of runoff hydrographs in a catchment. This method is particularly effective in urban and heavily modified catchments, where changes in land use and surface cover significantly impact runoff behaviour.

For the landfill catchments, WSP adopted a zero-perviousness assumption. In the Laurenson method, this translates into an urbanization factor of 0, meaning that the entire catchment is considered to be pervious.

8.3.1.1 Catchments and their properties

Figure 8.1 shows the extent of the runoff contributing catchment, totaling 7.6 ha, considered for this assessment.

The adopted catchment parameters are presented in Table 8.1. The Manning’s n adopted for the vegetated areas is towards the upper limit of the range of standard values suggested in Chow (1959). However, it is considered appropriate due to the vegetation growth visible in the contributing catchments.

The infiltration losses for landfill catchment areas were adopted considering the data extracted for the area from the ARR Datahub. The infiltration losses provided in the ARR Datahub generally are for the regional areas. Therefore, a conservative assumption was made for landfill areas reducing the losses by circa 75% from the regional loss values.

Loss values for hardstand areas were adopted based on project team's experience with similar land-uses.

Table 8.1 Land use and infiltration loss parameters

Parameter	Land use category	
	Landfill	Hardstand
Manning's n	0.035	0.015
Initial Loss (IL)	21.75	1
Continuing Loss (CL)	3.2	0.1

8.3.1.2 Design rainfall event

The rainfall intensity frequency duration (IFD) data for the site (nearest grid cell coordinates of 12.3827 S, 130.9375 E) were obtained from the Bureau of Meteorology's (BoM) computerised design IFD rainfall system (CDIRS). This system enables the automatic determination of a comprehensive set of IFD curves and associated data for any location in Australia.

Rainfall depths associated with design storms of 1%, 10%, and 63.2% annual exceedance probabilities (AEPs) for durations up to 72 hours are summarised in Figure 8.2 and Table 8.2. It should be noted that the 63.2% AEP event flows have only been utilised in designing the sediment trap and level spreader.

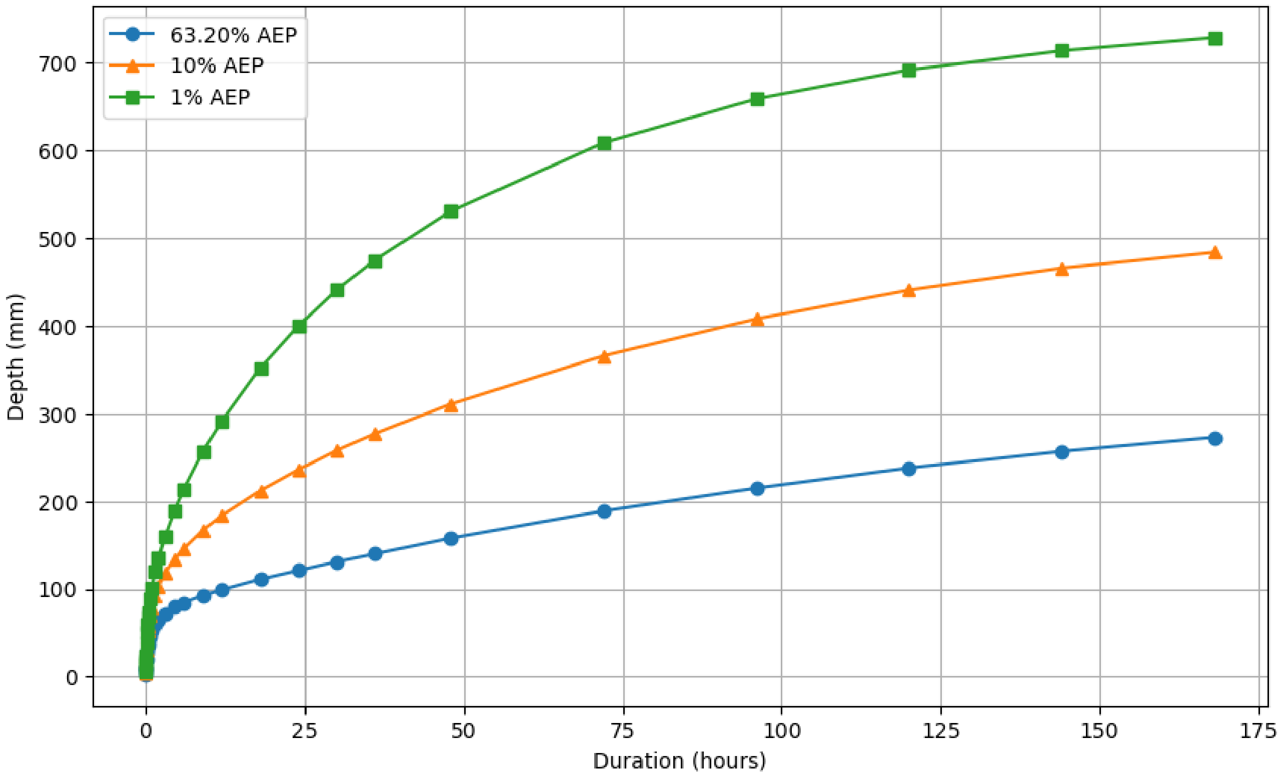

Figure 8.2 Rainfall depth (mm) for selected durations and AEPs

Table 8.2 Rainfall depth (mm) for selected durations and AEPs

Duration	63.20%	10%	1%
1 min	3.03	4.53	5.87
2 min	5.64	8.35	10.2
3 min	7.94	11.8	14.7
4 min	10	15	18.9
5 min	12	17.9	22.9
10 min	19.9	29.8	39
15 min	26	38.8	50.8
20 min	30.9	46.1	60
25 min	35	52.2	67.6
30 min	38.5	57.4	74.1
45 min	46.4	69.8	89.6
1 hour	52.2	79	101
1.5 hour	60	92.6	120
2 hour	65.3	103	135
3 hour	72.5	118	159
4.5 hour	79.5	134	189
6 hour	84.6	146	214
9 hour	92.5	167	257
12 hour	99.1	184	292
18 hour	111	212	352
24 hour	121	236	400
30 hour	131	258	441
36 hour	140	277	475
48 hour	158	311	531
72 hour	189	366	609

8.3.1.3 Critical duration analysis

The ARR 2019 guidelines recommend analysing various but likely storm ensembles to estimate peak catchment discharges for a particular AEP rainfall event (critical duration analysis). Therefore, the critical duration analysis was undertaken based on the latest rainfall and temporal pattern (TP) data. The 10% and 1% AEP rainfall events for durations from 10 minutes up to 6 hours and for different storm ensembles (10 TPs for each selected duration) sourced from the ARR Datahub, were considered for the critical duration analysis. The storm events with duration longer than 6 hours were not considered for the critical duration analyses, as preliminary modelling indicated that these longer events would not result in flows higher than those of the shorter durations considered. WSP believe that this is likely due to the small and steep catchment area and the adopted higher continuous losses.

The ARR 2019 also recommends adopting the critical duration based on maximum mean or median peak flow from the assessment. This assessment adopted the maximum mean peak flow from the modelling results.

The critical duration analyses plots for the Stage 2A expansion diversion drain (end section) for the 10% and 1% AEP events are presented in Figure 8.3 and Figure 8.4, respectively. A summary of critical duration analysis outcomes is presented in Table 8.3.

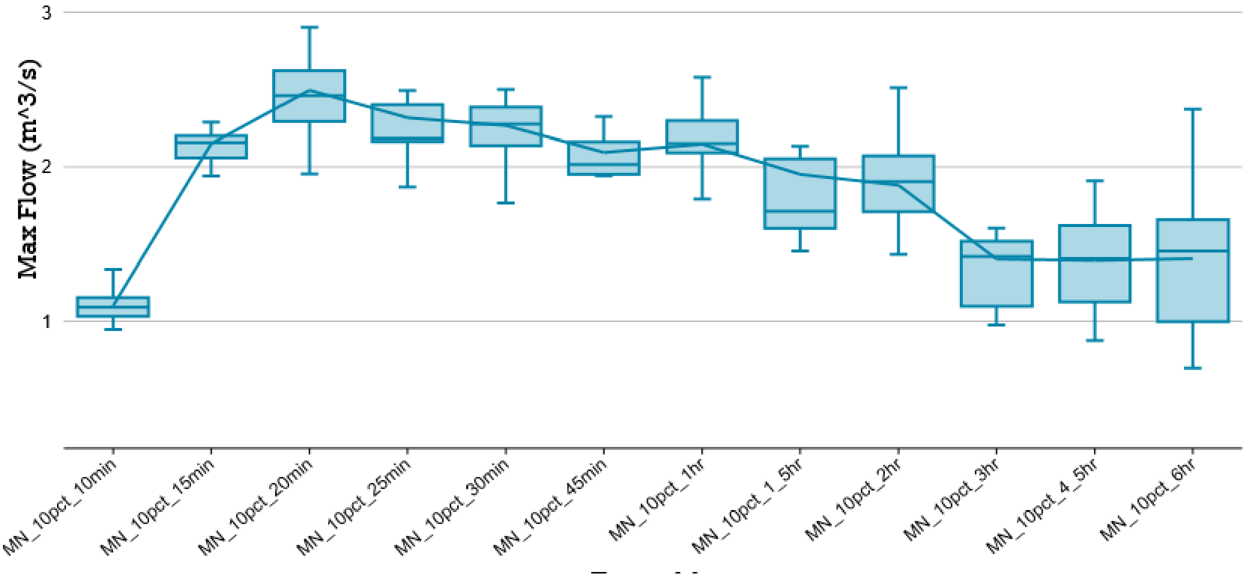

Figure 8.3 Critical duration analysis for 10% AEP event at the Stage 2A expansion diversion drain

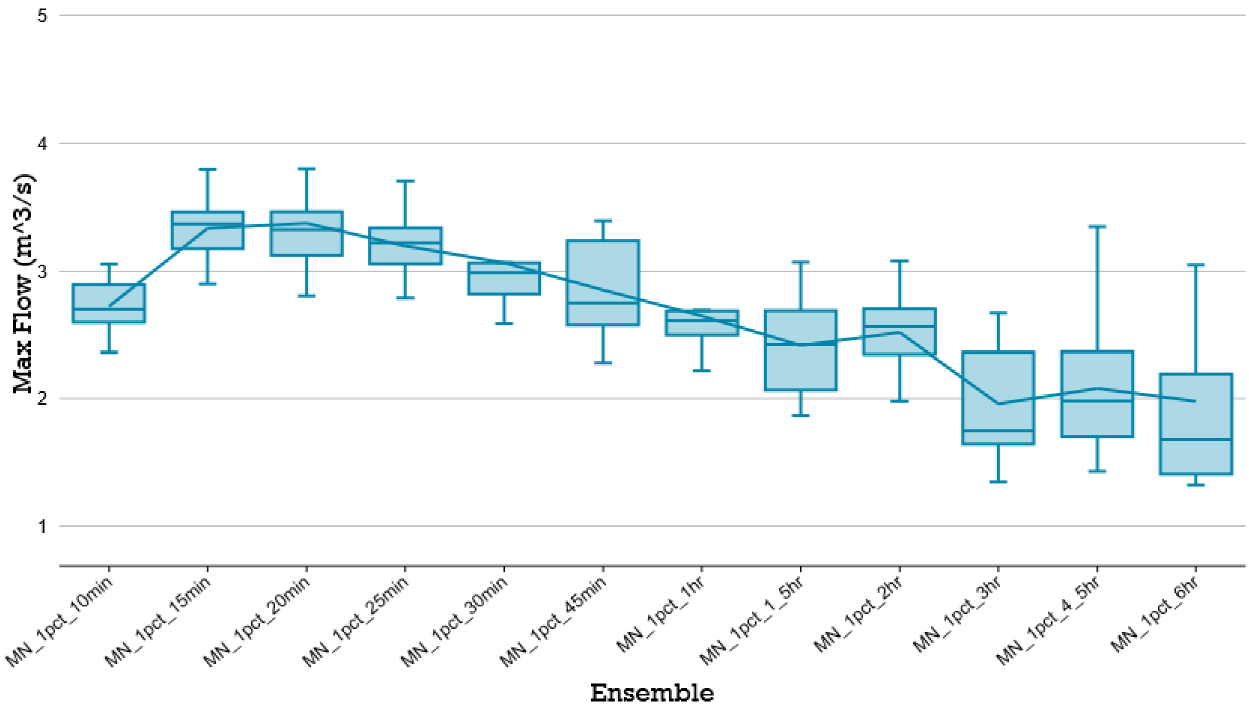

Figure 8.4 Critical duration analysis for 1% AEP event at the Stage 2A expansion diversion drain

Table 8.3 Critical duration analysis results

AEP Event	Peak Flow (m^3/s)	Critical Duration	Temporal pattern (TP)
10%	2.95	20 minutes	3
1%	3.37	20 minutes	9

8.3.2 Hydraulic assessment

WSP undertook a hydraulic assessment for the proposed Stage 2A expansion drain utilising the hydraulic platform of the XPSTORM software. The model was run for the resulted critical duration events (see Table 8.3). The existing slope along the alignment of the proposed Stage 2A expansion drain varies, as demonstrated in Figure 8.5.

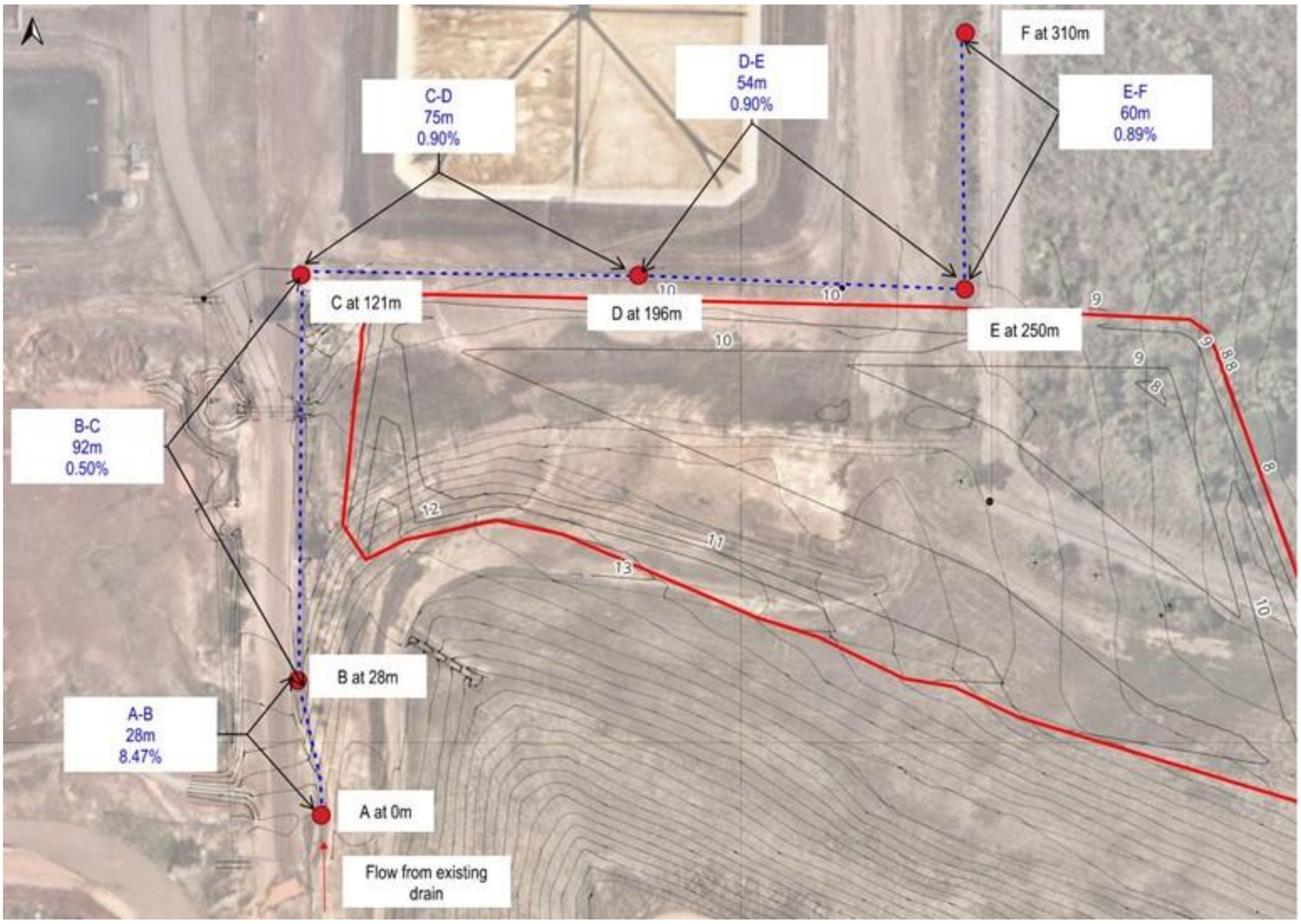

Figure 8.5 Existing topography (approximate only) along the proposed alignment of the Stage 2A expansion drain

The drain was modelled as a trapezoidal channel with a Manning's n of (0.035) for Section A-B to represent the riprapped profile, and a Manning's n of (0.03) for Section B to represent the short grass/earthen profile.

8.3.3 Modelling results

The XPSTORM model predicted the flow will be contained within the banks of a 1 m high trapezoidal drain with no overbank overflows for neither the 10% nor the 1% AEP design storm events. The estimated flow characteristics for each drain section (see Figure 8.5 for locations) for 10% AEP and for 1% AEP events are presented in Table 8.4.

Table 8.4 Stage 2A expansion diversion drain hydraulic modelling results

Section	Longitudinal Grade (%)	Side Slope(1V:H)	Bottom Width (m)	Estimated Maximum Flow Depth (m)		Freeboard (m)	Required Drain Depth (m)		Design Peak Flow (m^3/s)		Design Peak Velocity (m/s)	
AEP Event				10%	1%	10%	10%	1%	10%	1%	10%	1%
A-B	8.47	3	0.5	0.63	0.68	0.3	0.93	0.68	1.22	1.52	2.34	2.46
B-C	0.5	3	0.5	0.62	0.68	0.3	0.92	0.68	1.22	1.52	1.04	1.1
C-D	0.9	3	0.5	0.66	0.74	0.3	0.96	0.74	1.58	2.06	1.37	1.45
D-E	0.9	3	0.5	0.75	0.83	0.3	0.99	0.83	1.9	2.5	1.4	1.5

8.4 Drainage infrastructure design

8.4.1 Stage 2A expansion diversion drain design

Adhering to the clauses outlined in the “Basis of design” (WSP reference, PS137414-WSP-BNE-ENG-MEM-001 RevA, dated 21 November 2023) for stormwater design events, the results presented within Table 8.4 indicate that the minimum drain dimensions for each section to meet the requirement of accommodating 10% AEP storm event (with a minimum 300 mm freeboard) and 1% AEP storm event flows.

Conservatively and considering the stability of the bottom of the drain, a trapezoidal drain section with a depth of 1 m (including 300 mm freeboard) and 3H:1V side slopes is proposed for the entire drain.

It is noted that the diversion drain design presented in the drawings has been modified at the downstream end of the drain compared to the conceptual model presented in Section 8.2.1. The stormwater management design is considered conservative and includes additional 300 mm of freeboard above estimated flow depths. The modified stormwater diversion drain is assessed to the convey the design flow rate.

The Queensland Urban Drainage Manual (QUDM) guidelines (IPWEAQ, 2016) were referred to design the drain lining. According to QUDM, for easily erodible soils with 70% vegetation coverage (conservative approach), the permissible velocity is approximately 1.5 m/s. Considering that the resulted velocities are less than 1.5 m/s for most of the drain sections, a vegetated drain is proposed except for sections A-B.

Section A-B, with steep slopes appears to situate immediately upstream of the current design battery limit and is considered out of the current design scope. A nominal riprap/rock pitching is proposed at the entry of Section B-C or the starting point of current drain alignment where it meets the steep drain section with potentially higher velocities.

Table 8.5 summarises the diversion drain design requirements and proposed lining and their design specifications.

Table 8.5 Stage 2A expansion diversion drain design

Section	Longitudinal Grade (%)	Side Slope (1V:H)	Bottom Width (m)	Design Drain Depth, including 300 mm Freeboard (m)	Drain Lining	Remarks
A-B	8.47	3	0.5 (Trapezoidal)	1.0	Rip Rap	This section is situated external to the current design battery limit. However, if it/part of it still falls within the battery limit, it is recommended to adopt a suitable riprap/ rock lining design. It is recommended to adopt riprap with a minimum thickness of 300 mm with d_{50} 150 mm at the entrance/ transition from steep upstream drain to a minimum of 2 m from the transition in both direction (i.e., upstream and downstream).
B-C	0.50	3	0.5 (Trapezoidal)	1.0	Vegetated	
C-D	0.90	3	0.5 (Trapezoidal)	1.0	Vegetated	
D-E	0.90	3	0.5 (Trapezoidal)	1.0	Vegetated	

8.4.2 Treatment train

The *Water Balance Model, Leachate and Stormwater Management Strategies'* (document reference: 20140796-004-R-Rev2 dated 11 October 2021) provides recommendations on stormwater management for the site. A treatment train approach was proposed to manage sediment impacted runoff generated from the existing waste management operations at Shoal Bay Landfill. The treatment train is proposed to capture and treat runoff of up to and including 2 Exceedance per Year (2EY) rainfall event.

The treatment train consists of the following stormwater controls:

- Low Flow Drain: a drain designed to convey the 2EY rainfall event to allow for a low flow into the treatment train, the excavated sediment trap.
- Excavated Sediment Trap: a basin with a required surface area to allow for settling of sediment. Note that the sediment trap is only able to settle coarse sediment and less of the fine sediments.
- Level Spreader: an outlet structure to convey the partially treated flow from the excavated sediment trap into the existing grassed area.
- High-Flow Bypass Drain: a drain designed to convey rainfall event higher than 2EY. The aim being to reduce potential inundation and/or damage that may impact on the excavate sediment trap when the flow is higher than 2EY.

- Rock-lined Dissipation Basin: a rock lined dissipation basin to reduce the energy from the drain prior to discharging into the existing grassed area.

8.4.2.1 Low flow drain

The low flow drain is to convey 2EY rainfall event. Utilising the model described in Section 8.3, a peak flow of 0.45 m^3/s was estimated at the outlet of Section D-E of the drain. The low flow drain is estimated to have a geometry of 0.5 m base width, 3H:1V side slope and 0.4 m deep. The low flow channel will be connected to an excavated sediment trap. Based on the estimated velocity along the low flow drain, the drain is required to be grass lined with 70% coverage to be achieved prior to first flush. A high flow bypass channel is included to allow for overflow from flows exceeding the depth of the low flow channel.

8.4.2.2 Excavated sediment trap

Based on the stormwater management strategy, the proposed drain alignment from Stage 2 inert waste landfill is to connect into an excavated sediment trap to allow for settling of coarse sediment. The estimation on the requirement of the excavated sediment trap surface area was undertaken with the following assumptions and limitations:

- The sediment trap has been designed by assuming the water temperature as 20 degrees Celsius and a critical sediment size of 0.15 mm, resulting in a required sediment trap surface area of 62 m^2/m^3/s.
- Design will need to incorporate a high-flow bypass to convey flows greater than 0.45 m^3/s, peak flow for a 2EY rainfall event.

Sediment trap is expected to be frequently filled during periods of the wet season. The sediment trap will need to be maintained throughout operation to remove sediments as per the requirements in the EMP. A filtered drainage outlet has been included in the sediment trap to drain ponding water post rain event.

8.4.2.3 Level spreader details

A level spreader is proposed to be constructed downstream of the excavated sediment trap. The level spreader will distribute the flow from the excavated sediment trap across the sill length and release it as a sheet flow to the existing vegetated area. Whilst the critical operational parameter of the level spreader is the outlet sill, the length of the sill is governed by the design discharge and the allowable flow velocity of the downslope area. Based on the low flow of 0.45 m^3/s, the recommended minimum entrance width is 4.9 m, depth of 0.18 and end width of 0.9 m (Catchment & Creeks, 2010b). The adopted level spreader geometry is 5 m entrance width, end width of 0.9 m and a sill length of 5 m, resulting in estimated flow depth of 0.15 m based on the broad crested weir equation. The level spreader is to be vegetated to reduce potential erosion prior to discharging into the existing vegetated area.

8.4.2.4 High-flow bypass

A high-flow bypass is designed to limit potential inundation and damage on the excavated sediment trap. The high-flow bypass drain is designed to have the same geometry as the toe drain along the Stage 2A Expansion embankment. The high-flow bypass drain is to have a geometry of 0.5 m base width, 3H:1V side slope and 1 m deep (including a 0.3 m freeboard). A bund is to be located in between the low flow and high flow bypass to allow for flow higher than 2EY rainfall event to bypass the treatment train. The bund is required to have a crest width of 0.5 m, side slope of 2H:1V, height of 0.4 m and include erosion protection lining. Grouted rip rap is proposed for the erosion protection over the bund to allow the invert level of the bund to be constructed and to maintain separation of flows from the bypass drain during low flow event (i.e. rip rap lining would allow interstitial flow). The area downstream of the bund is also to be rock lined at least 2 m pass the toe of the bund to limit erosion potential occurring downstream of the bund during high flow.

8.4.2.5 Rock-lined dissipation basin

As the flow through the high-flow bypass is estimated to be higher than recommended for a level spreader of 0.85 m^3/s, a rock lined dissipation basin is proposed to be installed prior to discharging into the existing vegetated area with the aim of reducing the flow. Based on the requirement of the depth of the rock-lined dissipation basin from Catchment & Creeks (2010c), the rock lined dissipation basin is to be at least 0.25 m deep, 3H:1V side slope and 3 m long. It is to be rock lined with d_{50} of 150 mm and d_{90} of 300 mm. The flow from the rock lined dissipation basin will sheet flow to the existing vegetated area at a depth of approximately 50 mm.

9 Operational stormwater management

During the operational stage of the landfill, the stormwater runoff from the waste filling plateau which is covered by the interim cover soil is directed towards stormwater infrastructure to treat the sediment prior to discharging into the environment. CoD requested a dissipation basin for a stormwater drop structure be included in the Stage 2A expansion embankment to allow for this scenario in the future. The following assumptions and limitations have been adopted when estimating the operational stormwater infrastructure:

- A maximum of 3 ha catchment area of the waste filling plateau.
- A 100% impervious surface was adopted for a clay interim cover within the waste filling plateau.
- A slope of 0.5% within the plateau and channel flow prior to conveying the runoff into the chute.
- Manning's 'n' of 0.025.
- Based on the direction from CoD, the chute and apron are for operational purposes only. CoD to determine erosion controls for the chute during the operational stage consistent with current operations. Note that the access road spillway and apron into the diversion drain have been sized to convey flows based on above assumptions and limitations.

9.1 Flow estimation

To estimate the flow during the 10% and 1% Annual Exceedance Probability (AEP) rainfall event, a Rational Method assessment was undertaken. The Rational Method (The State of Queensland, 2013) is used to obtain the time of concentration and peak flow through the chute using the formula: $Q=(CIA)/360$, where Q is the peak flow (m^3/s), C is the runoff coefficient, I is the rainfall intensity (mm/hr), and A is the catchment area (ha). The rainfall intensity is determined using a site-specific Intensity-Frequency-Duration (IFD) data obtained from the Australian Bureau of Meteorology website. It should be noted that the latest available 2016 IFD is used within the Rational Method and is shown in Table 9.1. The coordinates adopted to obtain the IFD is latitude -12.39° and longitude 130.94°. The rainfall events adopted for estimating the peak flows are 10% and 1% AEP. Based on the flow estimation, the peak flows for 10% and 1% AEP are 1.13 m^3/s and 1.64 m^3/s, respectively.

Table 9.1 Rainfall depth (mm) for 10% and 1% rainfall events

Duration	10% AEP	1% AEP
1 min	4.53	5.87
2 min	8.35	10.2
3 min	11.8	14.7
4 min	15	18.9
5 min	17.9	22.9
10 min	29.8	39
15 min	38.8	50.8
20 min	46.1	60
25 min	52.2	67.6
30 min	57.4	74.1

Duration	10% AEP	1% AEP
45 min	69.8	89.6
1 hour	79	101

9.2 Chute

The flow from the waste filling plateau will be conveyed through a chute prior to flowing through the access track spillway and into the toe drain along the Stage 2A expansion embankment. The chute is proposed to have a 5 m base width with 3H:1V side slopes, 0.5 m depth and 4H:1V longitudinal slope.

The hydraulic jump has been estimated to occur within the chute using the CHUTE program (eWater, 2013). CoD to decide on erosion controls of the chute as required during operation consistent with current operations as per direction from CoD.

9.3 Apron

A riprap apron is proposed to be located at the end of the chute considering recommendations presented in Robinson *et al.* (1998) and Catchment & Creeks (2011). The estimation results in the following apron dimension:

- A concrete lined access road spillway with 5 m width, side slopes of 6H:1V with 0.5 m depth and at least 5 m long in the direction of flow from the chute.
- An apron of 10 m wide riprap-lined drain, perpendicular to the flow. The drain dimension is to remain the same as specified in Section 8.4.

9.4 Temporary diversion bund

A stormwater diversion bund is proposed to be constructed (and reconstructed periodically as waste filling elevation progresses) uphill of the southern crest of the Stage 2A cell, such that surface water runoff from the vegetated Stage 2 batter does not flow into Stage 2A and thus into the leachate collection system. It is envisaged that the diversion bund is constructed upstream of the waste lift, as it is progressively constructed up the existing Stage 2 batter as waste filling progresses in Stage 2A. The bund or swale is expected to discharge to the west, with runoff reporting to the existing site stormwater management infrastructure.

An initial stormwater diversion bund has been included in the design so it can be constructed as part of construction of the cell and be in place for initial operations. Peak flow was estimated for the 10% AEP using the same process described in Section 9.1 and based on the following assumptions:

- A maximum 1.5 ha catchment area of the existing northern batter of Stage 2 that needs to be diverted.
- Existing batter grade of approximately 20%.
- Initial diversion bund constructed at 1% grade to the east.
- A 100% impervious surface.
- Manning's 'n' of 0.025.

The estimated peak flow for the 10% AEP is 0.55 m^3/s. Hydraulic toolbox software was used to estimate the size of the diversion bund based on the following geometrical parameters:

- V-drain type.
- Upstream grade (batter) approximately equal to the existing Stage 2 northern batter of 20%.

- Proposed diversion bund batter angle of 1V:2H.

Based on the above, a flow depth of 0.38 m was estimated. A nominal 0.5 m high bund has been proposed to convey the estimated flow rate. A nominal 100 mm freeboard has been included based on the bund being temporary.

The upstream batter is to be vegetated to reduce potential erosion prior to discharging into the existing vegetated area to the east of Stage 2A. The diversion bund should be inspected and maintained as part of operations to ensure the diversion bund is functioning as intended.

10 Leachate generation

CoD indicated that the Leachate Treatment Plant (LTP) has a maximum treatment capacity of 51 ML per year. CoD confirmed in an email on 2 April 2024, that the total leachate generated in 2023 was 42.6 ML. Leachate generation in the Stage 2A expansion cell was estimated with reference to Table 11 of the '*Water Balance Model, Leachate and Stormwater Management Strategies'* Report prepared by Golder Associates, dated 2021 (Golder reference: 20140796-004-R-Rev2).

The Stage 2A expansion is considered for the conditions for 'Early Operation' of the above referenced report, for a typical rainfall (50th percentile) event (15% of total rainfall for the year forms leachate) and an extreme rainfall (90th percentile) event (12.5% of total rainfall for the year forms leachate). The Stage 2A expansion is estimated to generate 4.3 ML per year (50th percentile) and 4.5 ML per year (90th percentile), respectively. This leachate generation value is expected to decrease annually as the landfill is capped.

The LTP has capacity to service the leachate generated from the Stage 2A expansion if CoD continue to pump from the other cells on site at the rate that was identified in 2023. CoD's operational management team will be responsible for the ongoing maintenance and the additional leachate added to the system from the Stage 2A expansion.

11 Site access

The current access to the site through SBWMF will be maintained for the Stage 2A expansion. This access road extends from the front gate of the facility and passes the west embankment of the cell expansion through to the leachate ponds located north of the cell. Access to the cell for filling will be through the south-west corner of the Stage 2A expansion, in accordance with the current filling methods of Stage 2. Refer to Figure 11.1 below for the access plan.

The Stage 2A expansion has been designed to allow inspection and maintenance of the perimeter embankment crest to be undertaken from a 3 m wide access track around the perimeter of the cell. The crest will be locally widened at the north-eastern corner at the location of the sump for access and maintenance, as per the Design Drawings.

Figure 11.1 Site access

12 Operational considerations

The design of Stage 2A is based on a number of key points that need to be implemented and managed during operation of the expansion. These are summarised below, while some considerations relevant to operations are included in the Safety in Design register (refer to Section 4 and Appendix A).

- The intercepted groundwater seepage that will discharge via a gravity-draining pipe to the north-east of the Stage 2A footprint should be regularly monitored for water quality indicators. Trends should be monitored, and exceedances of the adopted trigger values observed in the monitoring data should be flagged during this regular onsite monitoring events, and responsive action should be taken.
- The design has incorporated a surface water diversion to intercept water that flows towards the Stage 2A footprint from the southwest, traveling along the eastern edge of the access road alignment, then to the west along the north side of Stage 2. The drain includes sediment control features that will need to be maintained throughout operation to remove sediments in accordance with the site EMP (CoD, 2023).
- The soil protection layer with geotextile over the top on slopes is exposed to rainfall until it is covered by waste, which could lead to erosion formations and/or slumping (on the batters) of the layer. The design includes a layer of sacrificial geotextile on the batters to reduce erosion on the batters. The surface of the soil protection layer should be regularly inspected by operations for signs of erosion or slumping on the batters, particularly following heavy rain fall events. Erosion that has formed deeper than 100 mm shall be repaired by in filling the void to surface level with a sand rich material. If slumping has occurred on the batters, the sacrificial layer of geotextile shall be removed and effected batter areas reinstated with sand rich material. Dewatering maybe required to repair effected areas. The methodology for repairs to the soil protection layer must consider protection of underlying materials, which may include use of low ground pressure equipment and formation of access track above the protection layer to the affected areas. If any of the underlying materials are damaged during maintenance works seek advice from the Designer.
- A stormwater diversion bund is proposed to be constructed (and reconstructed periodically as waste filling elevation progresses) uphill of the southern crest of the Stage 2A cell, such that surface water runoff from the vegetated Stage 2 batter does not flow into Stage 2A and thus into the leachate collection system. It is envisaged that the diversion bund is constructed upstream of the waste lift, as it is progressively constructed up the existing Stage 2 batter as waste filling progresses in Stage 2A. The bund or swale is expected to discharge to the west, with runoff reporting to the existing site stormwater management infrastructure.
- The methodology for waste placement must consider protection of the underlying materials. Low risk waste must be placed in a near-horizontal layer of approximately 1 m thickness across the entire base of the cell before filling to higher levels. This first layer of waste must be progressively tipped from the surface of the previously placed waste.
- Failure to place the first lift of waste appropriately may result in tearing of, or damage to, the leachate collection system; and potentially damage to the underlying cell liner system.
- CoD propose to access the cell from the south along the low permeability compacted clay layer by building a ramp/track above the soil protection layer to the required cell area. Access to the cells must consider controls to manage risks associated with access near slopes. CoD shall seek advice from the Designer for the geometry of the entry ramps, prior to waste filling, once the access location has been decided, to develop a stable platform. The cell walls are graded at 3H:1V; lower gradients are typically preferred for safe access by waste vehicles. As waste filling progresses, safe access to the top of waste plateau must also be planned and managed.
- The waste filling of the cell must be carried out in near horizontal layers across the entire floor footprint of the cell.

- As the landform progresses onto the existing Stage 2 northern batter, the follow operational conditions shall apply:
  - Asbestos waste and cover soils and the operating crest shall be placed with a nominal 5% crossfall grading downwards to the north. This shall occur as a minimum for the portion of the operating crest that overlies the existing Stage 2 northern batter.
  - Upslope runoff shall be diverted around the Stage 2A waste filling area.
  - Waste and cover soils placed in the cell shall be partially saturated, i.e. spadable.
- Leachate must be regularly extracted from the sump to maintain the maximum leachate head on the cell floor to less than 300 mm.
- Prior to filling the western sub cell, the temporary structure must be removed. The bags of the temporary structure need to be removed to reinstate the void to the following details. The geocomposite drain shall be pulled back to access the polyethylene sheet. The polyethylene sheet shall be carefully cut above the GCL at floor level and removed so as to not impede flow through the geocomposite drain (i.e. rest flat on the GCL). The void shall be filled with gravel wrapped in filter geotextile overlying the geocomposite strip drain. The bags must be cut with a retractable hooked bladed knife, releasing the gravel onto the filter geotextile and spread to fill the void (minimum 300 mm thick). The filter geotextile shall be overlapped a minimum of 500 mm and a 100 mm layer of fill placed over the exposed geotextile to restrain the geotextile in place. The break in the length of the leachate collection pipe shall be re-instated by two half pipes clamped around the ends of the existing pipe and gravel placed over the top and wrapped in filter geotextile. Any end caps at this location shall be removed.
- Stage 2A will be operated adjacent to the existing Stage 2 landfill. It is recommended CoD review and updated (if required) their Landfill Gas Management Plan to consider operations in Stage 2A.

13 Indicative landform and capping details

CoD has indicated that Stage 2A will be capped as part of the broader capping works for Stage 2. A bench is proposed to be incorporated into the external slopes to improve the ease of future capping works. External slopes are proposed at 1V:4H for the conceptual final landform for Stage 2A. The bench is proposed to be around 5 m wide to support geosynthetic anchorage and provision of surface water drains. It is expected that wider benches for access would be incorporated into the Stage 2 capping where the landfill is longer to support a ramp. The requirement for the bench on the east and west should be confirmed with the proposed future expansion to the east and west. The vertical position of the bench currently shown on Figure 13.1 is based on a future expansion providing a bench at nominal 15 m elevations to support cap construction.

The Stage 2A expansion conceptual waste profile shown in Figure 13.1 extends to approximate RL28 m AHD. It is noted that a higher crest elevation to RL36 m AHD was modelled in the Stage 2 Inert Landfill Concept Design (WSP, 2024); however, the Stage 2A expansion eastern edge was shifted west and therefore the finished crest level has been reduced to provide an operational area at the crest. The stability of landform has been assessed (Section 6.4) to a crest elevation of RL36 m to provide some flexibility in operations in the event there is a change to the landfill development plan.

The capping profile for rehabilitation of the broader Stage 2 may include a typical General Waste Landfill capping system due to the presence of general waste at the base of the Stage 2 landfill. A typical general waste landfill capping profile may comprise (from top to bottom): vegetation, cover soils, sub soil drain and a geosynthetic liner.

The default basal liner system from VicEPA BEPM has a design leakage rate of 1000 L/ha/day. We suggest a design leakage rate of 750 L/ha/day be the target for the capping system for the Stage 2A expansion

A conceptual landform profile and cross-section for Stage 2A is presented in Figure 13.1 and Figure 13.2. Depending on the broader Stage 2 capping design, stormwater management on the crest and benches of the Stage 2A landform could be directed north to a stormwater chute on the northern face to provide drainage off the landform and discharge offsite.

Based on the conceptual landform, the estimated airspace volume of the expansion is approximately 200000 m^3.

Figure 13.1 Indicative plan of Stage 2A landform

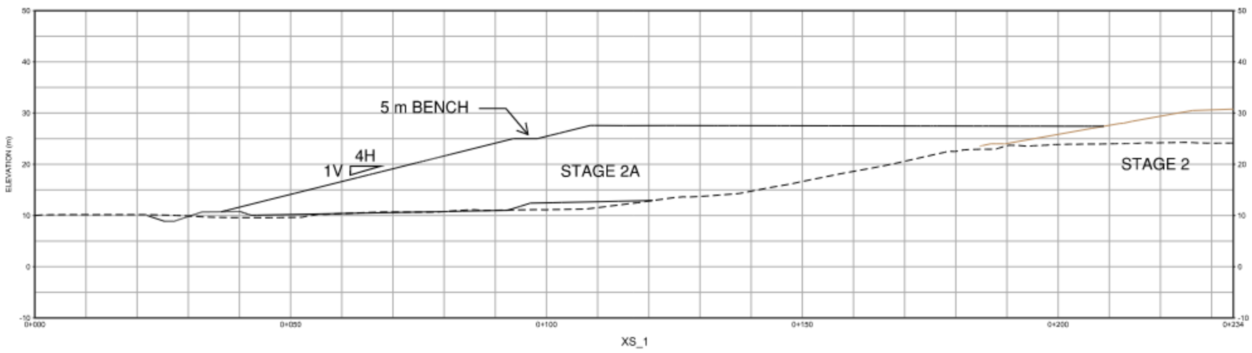

Figure 13.2 Indicative cross-section through Stage 2A landform

14 Limitations

This Report is provided by WSP Australia Pty Limited (WSP) for the City of Darwin in response to specific instructions from the Client and in accordance with WSP's proposal and agreement with the Client (Agreement).

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Appendix A

Safety in Design register

Safety in Design REGISTER

City of Darwin - Stage 2A Inert Landfill Expansion

PS138768 - SiD Register | 06-Mar-24

Item No.	Aspect	Risk Identification	Consequence(s)	Without Risk Controls				Risk Owner	Design Measures	With Risk Controls		
				Likelihood	Consequence	Risk				Likelihood	Consequence	Residual Risk
1.0	Groundwater	Groundwater conditions are noted to be seasonally variable. Surface soils may become fully saturated during wet season conditions from localised perched aquifers.	Potential for floor liner to uplift and become damaged during high seasonal groundwater levels prior to waste placement.	Likely - 4	Significant - 4	V 16	red		Design groundwater drainage system beneath the cell floor, comprising of a continuous 100 mm thick drainage gravel blanket and perforated collection pipes to discharge via a gravity flow outlet pipe, to a location northeast of the cell footprint.	Rare - 1	Significant - 4	H 4
2.0		Seasonally variable groundwater levels resulting in non-compliance with separation requirement in landfill guidelines.	Potential for contamination of groundwater due to interaction with waste and/or leachate within the cell footprint.	Almost certain - 5	Significant - 4	V 20	red		Groundwater outlet to be located above the Highest Astronomical Tide (HAT) and 1 in 100 year ARI storm surge levels.	Rare - 1	Significant - 4	H 4
3.0	Surface Water	Ineffective stormwater management.	Erosion of external embankments and ground surface.	Likely - 4	Minor - 2	H 8	orange		Design of a diversion drain on the western and northern sides of the cell. Design includes erosion control measures such as vegetation lining and riprap.	Unlikely - 2	Minor - 2	L 4
4.0		Rainfall into the cell.	Substantially increase leachate collection in cell during initial filling stages. Increase in quantity of leachate treatment.	Almost certain - 5	Medium - 3	V 15	red		A temporary subcell has been included in the design to separate the initial waste placement area and permit clean water collection and discharge from the empty upstream portion of the cell.	Unlikely - 2	Medium - 3	M 6
5.0		Surface water runoff from existing Stage 2 batter into the cell.	Substantially increase leachate collection in cell during waste filling. Increase in quantity of leachate treatment.	Almost certain - 5	Medium - 3	V 15	red		Diversion bund to be constructed and raised as part of operations as part of the interface layer construction.	Unlikely - 2	Medium - 3	M 6
6.0	Storm Surge Zone (tidal waters)	Flooding within storm surge zones.	Potential inundation of cell and erosion of the external cell embankment due to extreme storm surge levels.	Likely - 4	Significant - 4	V 16	red		Cell location and embankment crest levels selected such that the crest level of cell is located above Secondary storm surge zone (1,000 year ARI). Cell location selected such that the embankment toe level is located above Extreme storm surge zone (10,000 year ARI).	Rare - 1	Significant - 4	H 4
7.0	Geological Setting	Difficulties with excavating rock encountered during earthworks.	Reduced productivity. Delays due to design modifications during construction to minimise rock excavation. Reduced cell capacity.	Moderate - 3	Medium - 3	H 9	orange		Geotechnical investigations were undertaken to determine potential materials encountered during excavation. Cell subgrade optimised and located above levels identified as practical refusal during geotechnical site investigations.	Rare - 1	Medium - 3	M 3
8.0	Subgrade	Soft spots in subgrade leading to settlement.	Loss of grade on the cell floor.	Moderate - 3	Medium - 3	H 9	orange		Geotechnical site investigation undertaken to assess ground conditions. Technical Specification includes requirement for proof rolling of floor prior to fill placement.	Unlikely - 2	Medium - 3	M 6
9.0	Embankment Fill	Perimeter bund settlement due to poorly compacted engineered fill.	Potential compromise to stability.	Unlikely - 2	Medium - 3	M 6	yellow		Compaction control, testing and inspection of engineered fill placement to form the perimeter bund.	Rare - 1	Medium - 3	M 3
10.0	Slope Stability	Inappropriate design of embankment slopes.	Unsafe working conditions. Delays to project and impact on operations. Embankment failure.	Moderate - 3	Medium - 3	H 9	orange		Design includes a stability assessment demonstrating the minimum factor of safety for embankment stability is achieved for the proposed embankment. All embankments are constructed at 3H:1V slopes.	Rare - 1	Medium - 3	M 3
11.0		Progressive erosion of exposed external perimeter embankment batter leading to slope instability.	Slope failure.	Likely - 4	Medium - 3	H 12	orange		Design includes the placement of topsoil and vegetation establishment for erosion control on external batter. Design includes perimeter diversion drain to limit erosion of perimeter bund toe.	Unlikely - 2	Medium - 3	M 6
12.0		Inappropriate design of landfill batter slopes post filling.	Unsafe working conditions. Delays to project and impact on operations. Landfill batter failure.	Likely - 4	Significant - 4	V 16	red		Waste batters are to be graded at 4H:1V slopes. SlopeW model check on the stability of the northern expansion and the potential for further phases of development up to RL59m as per the Development Plan.	Unlikely - 2	Significant - 4	H 8
13.0	Liner Subgrade	Subgrade surface does not provide a smooth, firm platform for the installation of coated GCL.	Damage to the coated GCL from sharp edges/objects in the liner subgrade.	Likely - 4	Medium - 3	H 12	orange		Technical Specification requires subgrade to be smooth, compacted and free from sharp edges and objects.	Unlikely - 2	Medium - 3	M 6
14.0	Liner seepage	Seepage through liner	Groundwater contamination due to potential defects/holes in coating of GCL.	Moderate - 3	Significant - 4	V 12	red		Technical Specification and CQA Plan requires Level 1 Construction Quality Assurance for the installation of the lining system and overlying drainage gravel.	Unlikely - 2	Medium - 3	M 6
15.0	Coated GCL	Desiccation of bentonite component of GCL.	Reduced performance of the GCL and increased seepage rates from the cell.	Moderate - 3	Medium - 3	H 9	orange		Specification of Coated GCL limits desiccation through coating layer.	Rare - 1	Medium - 3	M 3
16.0	Coated GCL	Hydration of bentonite component of GCL under low confining pressure.	Reduced performance of the GCL and increased seepage rates from the cell.	Moderate - 3	Medium - 3	H 9	orange		Coated GCL to be confined by Leachate drainage gravel and soil cover immediately following liner installation.	Rare - 1	Medium - 3	M 3
17.0	Coated GCL	Ion exchange between leachate and sodium bentonite component of GCL.	Reduced performance of the GCL and increased seepage rates from the cell.	Moderate - 3	Medium - 3	H 9	orange		Specification of Coated GCL includes a polyethylene coating to separate the leachate collected in the overlying gravel and the sodium bentonite, reducing the potential for ion exchange in the GCL.	Unlikely - 2	Medium - 3	M 6
18.0	Coated GCL	Protection of Coated GCL from leachate drainage gravel and inlet and outlet structures.	Damage to Coated GCL resulting in increased liner seepage or project delays associated with repairs.	Likely - 4	Medium - 3	H 12	orange		Design includes liner protection materials such as Cushion Geotextile beneath leachate drainage gravel and pipework to reduce the likelihood of damaging the liner.	Unlikely - 2	Medium - 3	M 6
19.0	Geosynthetic Materials	Physical and chemical properties of the geosynthetics procured do not meet the specified minimum requirements.	Geosynthetics will not satisfy the design intent and will compromise the effectiveness/performance of the lining system.	Moderate - 3	Medium - 3	H 9	orange		Geosynthetics supplied should meet the material properties in the Technical Specification. Independent testing requirement in the Technical Specification to confirm properties meet manufacturer statements.	Unlikely - 2	Medium - 3	M 6
20.0		Wind uplift of geosynthetic materials.	Placed geosynthetics have potential to be lifted and deformed by wind, posing hazards to people and machinery.	Likely - 4	Medium - 3	H 12	orange		Technical specification requires construction of materials to occur concurrently. Temporary ballast (sandbags) are also required to be installed by the contractor in areas where leachate drainage gravel has not been placed for ballasting geosynthetics.	Unlikely - 2	Medium - 3	M 6
21.0	Drainage grades on cell floor	Cell floor is not graded sufficiently to direct leachate into sump.	Insufficient extraction of leachate.	Unlikely - 2	Medium - 3	M 6	yellow		Design includes a minimum 1% floor grade towards the sump. The sump is depressed below floor surface. Technical Specification requires a survey to confirm design grades are achieved.	Rare - 1	Medium - 3	M 3
22.0	Access to leachate riser	Insufficient space for vehicle to access sump riser for pump operation and maintenance.	Manual handling of pumps leading to injury.	Likely - 4	Minor - 2	H 8	orange		Embankment crest widened at sump location. Crest width has been locally widened to 9 m to support a light vehicle turning around.	Rare - 1	Minor - 2	L 2
23.0	Earthworks Quality Control and Quality Assurance	No quality control supervision of earthworks or inspection and testing of embankment subgrade and fill placement.	Failure of embankment/stability issues.	Likely - 4	Significant - 4	V 16	red		Level 1 Supervision of earthworks in accordance with AS 3798 - 2007. Inspection and testing in accordance with Technical Specification. GITA present during earthworks.	Unlikely - 2	Significant - 4	H 8
24.0	Geosynthetics Quality Control and Quality Assurance	Lack of QA/QC of geosynthetic lining system.	Failure of lining system.	Moderate - 3	Significant - 4	V 12	red		Appropriate QA/QC required by the Technical Specification for the works. GITA presence on site.	Unlikely - 2	Significant - 4	H 8
25.0	Existing Stage 2 Waste	Existing Stage 2 Waste Hotspot	Migration of hotspot to Stage 2A Northern expansion cell.	Likely - 4	Severe - 5	V 20	red		Design includes an 'interface layer' minimum thickness of 3.5 m located on the existing Stage 2 waste batter to reduce the potential for hotspot migration to the new cell.	Unlikely - 2	Significant - 4	H 8
26.0	Landfill gas	Existing Stage 2 Waste LFG venting	Potential exposure to hazardous odours and vapours which could cause harm to humans.	Moderate - 3	Significant - 4	V 12	red		Technical Specification requires any person working along the existing Stage 2 face to wear a gas monitor. Design does not require the existing Stage 2 waste to be stripped or exposed.	Unlikely - 2	Significant - 4	H 8
27.0		LFG venting from expansion cell	Potential exposure to hazardous odours and vapours which could cause harm to humans.	Moderate - 3	Significant - 4	V 12	red		The Stage 2 expansion cell will only accept inert waste, hence design allows venting to the atmosphere.	Rare - 1	Significant - 4	H 4
28.0	Exposed soil protection layer	Erosion/slumping of exposed soil protection layer from rainfall/ponding	Loss of confinement material above GCL in discrete areas leads to degradation of GCL.	Likely - 4	Medium - 3	H 12	orange		Sacrificial layer of geotextile included on batters to reduce erosion on batters. Operations to regularly inspect surface for erosion/slumping particularly following rainfall events and repair areas as required.	Unlikely - 2	Medium - 3	M 6
29.0	Sediment trap	Ponding of water	Drowning	Rare - 1	Severe - 5	H 5	orange		Drainage feature included at base to drain sediment trap following rainfall event. CoD/operations to consider installing delineation bunding around sediment trap.	Rare - 1	Severe - 5	H 5

Consequence	Likelihood				
	Almost certain - 5	Likely - 4	Moderate - 3	Unlikely - 2	Rare - 1
Severe - 5	V 25	V 20	V 15	V 10	H 5
Significant - 4	V 20	V 16	V 12	H 8	H 4
Medium - 3	V 15	H 12	H 9	M 6	M 3
Minor - 2	H 10	H 8	M 6	L 4	L 2
Negligible - 1	H 5	M 4	L 3	L 2	L 1

Red	Very high risk; immediate action required
Orange	High risk; management required
Yellow	Medium risk; specify required management
Green	Low risk; manage with standard operating procedures

Appendix B

Infiltration modelling

Memo

To:	Jedda Bennett-Kellam
From:	Jarod Laskazeski
Subject:	**Shoal Bay – Infiltration Modelling (HELP)**
Our ref:	PS138768-CIV-MEM-018 Rev0
Date:	**26 September 2024**

1. Introduction

WSP Australia Pty Ltd (WSP) is engaged for design services for a northern expansion to the Stage 2 Inert Landfill at the Shoal Bay Waste Management Facility, Darwin. WSP has recently submitted draft issue for tender (IFT) design documents.

City of Darwin (CoD) is assessing a potential alternative option for the Stage 2 northern expansion to operate the cell to receive asbestos waste only. At the request of CoD, WSP provided advice on key design considerations and potential changes to the current design for the alternative option[1]. A key outcome of this memorandum indicated the expansion may not require a separation layer due to asbestos waste being non-combustible. Seepage modelling was recommended to understand seepage potential from the expansion into the existing Stage 2 area.

WSP developed seepage modelling to compare the current infiltration through the outer slope of Stage 2 relative to the infiltration into Stage 2 from the filling operation of an asbestos waste cell in Stage 2A. The Hydraulic Evaluation of Landfill Performance (HELP) program was used to model two scenarios which are:

- The existing northern batter condition
- Stage 2A asbestos expansion while waste is being deposited in the cell.

The two infiltration rates were modelled to compare the results for the two scenarios.

2. Model Design Basis and Assumptions

Based on information provided by CoD, the following conditions were adopted:

- The Stage 2 northern expansion will be filled with a grade of 20H:1V towards the north over the existing batter of the Stage 2 footprint.
- 100 mm deep evaporative zone depth.
- Asbestos and cover soils are placed at a ratio of 1:1.07 respectively with approximately 1 m of cover soils placed over the asbestos waste during operation. The layer thicknesses for both asbestos and cover soils were assumed to be approximately 1.15 m (conservative).
- Initial moisture conditions assume saturated conditions of the waste.

[1] Memo: *Stage 2A Asbestos Landfill Expansion Design Considerations*, WSP document ref.: PS138768-WSP-BNE-ENG-MEM-016 RevA Asbestos Landfill Considerations, dated 28 August 2024.

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- Asbestos soils modelled as a lateral drainage layer due to expected higher permeability relative to cover soils.
- The cover soil layers were modelled as a vertical percolation layer.5
- The sloping geometry was modelled as multiple vertical ‘slices’ at various chainages in north south direction, with the average results adopted to estimate the comparative seepage quantities over the existing Stage 2 outer slope area.

2.1 Landfill Cross Section

Conceptual diagrams outlining the adopted ‘slices’ which have been modelled are provided in Figure 2.1 and Figure 2.2.

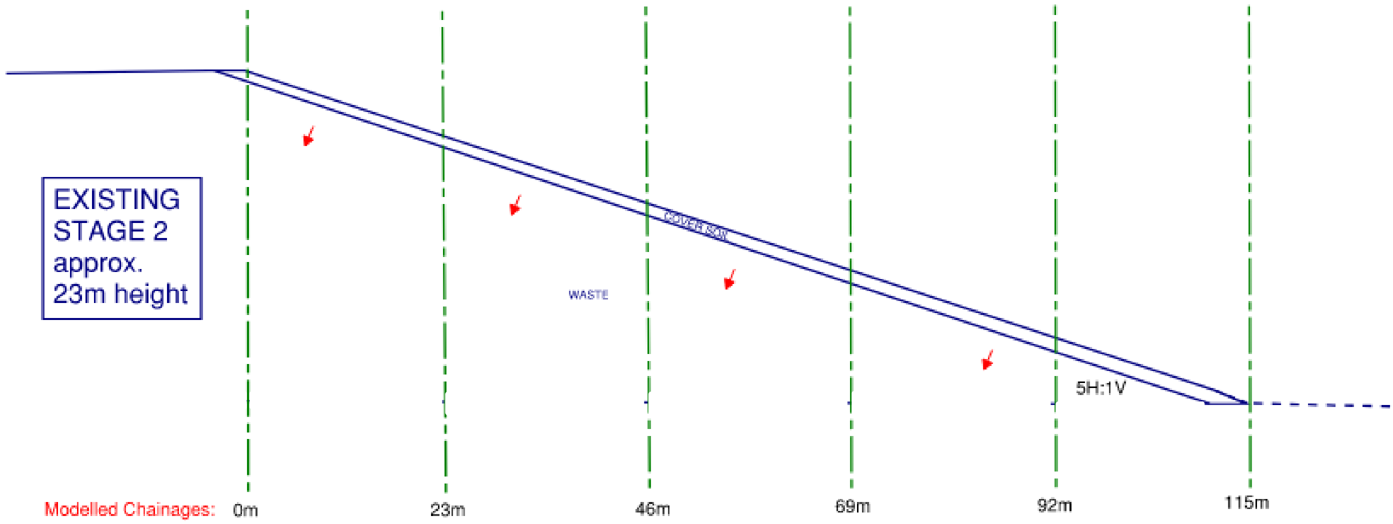

Figure 2.1 Existing Conditions – Scenario 1

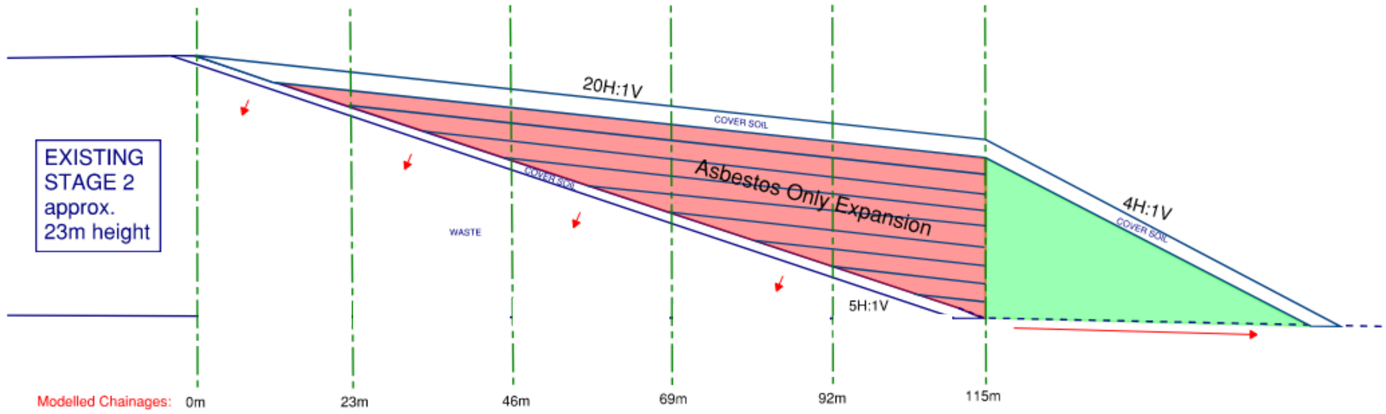

Figure 2.2 Landfill Expansion (Layout) – Scenario 2

The ‘green’ shaded area is not deemed to have any impact on the leachate generation into the existing Stage 2 area as it extends beyond the Stage 2 footprint and is over the new baseliner of Stage 2A.

2.2 HELP Input parameters

The HELP software version 3.95 incorporates location specific historic data to simulate weather conditions and was sourced to provide input to the comparative estimated seepage rates.

Historic data for precipitation, air temperature and solar radiation data were imported through ‘txt’ file, retrieved using the Queensland Government’s SILO database for the Shoal Bay location (-12.4, 131). The data available covers the date period from 1 January 2010 to 31 December 2023.

The HELP software calculates the evapotranspiration based on the following information:

- Location's latitude
- Evaporative zone depth
- Maximum leaf area index
- Yearly average wind speed
- Average relative humidity for each quarter
- Julian date (growing season start and end dates)

Sections 2.2.1 to 2.2.4 outline the data used for the HELP simulations.

Note the leaf area index was set at zero, so the growing season has been left at the program default values as this does not affect the results with no leaf area.

2.2.1 Precipitation Data

Precipitation data was retrieved using the SILO database described in Section 2.2.

2.2.2 Air Temperature Data

Temperature data was retrieved using the SILO database described in Section 2.2.

2.2.3 Solar Radiation Data

Solar radiation data was retrieved using the SILO database described in Section 2.2.

2.2.4 Evapotranspiration Data

The inputs required to calculate evapotranspiration were calculated using publicly available weather data retrieved from the Bureau of Meteorology (BOM).

Average Quarterly Relative Humidity

The monthly means for the 9am and 3pm relative humidity's for the period from 1991 to 2020 were retrieved and an average of those were taken for each 3 months in the quarter to estimate the quarterly average.

Table 2.1 Average Relative Humidity

Quarter 1 (Jan-Mar)	76%
Quarter 2 (Apr-Jun)	54%
Quarter 3 (Jul-Sep)	52%
Quarter 4 (Oct-Dec)	65%

Average Yearly Wind Speed

The same process was used for yearly average wind speed with the mean 9am and 3pm wind speed for each month between 1991 and 2020 averaged to estimate an overall average yearly wind speed. The resulting yearly average wind speed was calculated as 16 km/h.

2.2.5 Soil Input Data

The soil input parameters were derived from the material types within the HELP model software database. The cover soils were assigned the material properties of a moderately compacted Silty Loam and the asbestos was assigned material properties of a slightly compacted Sandy Clay Loam. The material for municipal waste was used for existing waste within Stage 2. Table 2.2 describes the HELP model parameters for each material.

Table 2.2 Material Parameters

HELP Material No.	HELP Description	Model Description	Total Porosity (vol/vol)	Field Capacity (vol/vol)	Wilting Point (vol/vol)	Saturated Hydraulic Conductivity (m/sec)
10	Sandy Clay Loam	Asbestos waste	0.398	0.244	0.136	1.2×10^{-6}
18	Municipal Waste	Existing Stage 2 waste	0.671	0.292	0.077	1.0×10^{-5}
23	Silty Loam	Existing cover soil on northern batter and new covers soils for Stage 2A	0.461	0.360	0.203	9.0×10^{-8}

2.3 HELP Computed Runoff Curve Number

The HELP software computes the runoff curve number (CN) from the upper most layer based on the slope, slope length and material type for a constant slope length. A length of 115 m was adopted representing the horizontal distance over the existing Stage 2 batter. The upper material was assumed to be a Sandy Clay Loam which resulted in runoff curve numbers of 94.13 and 93.93 for the 10% and 5% modelled slopes respectively.

3. Layer arrangement

Layer input is required to perform simulations. A schematic describing various layers is presented in Figure 3.1, with only the Cover System being applicable for this assessment. The layer arrangements for the seepage model are outlined in Sections 3.1 and 3.2.

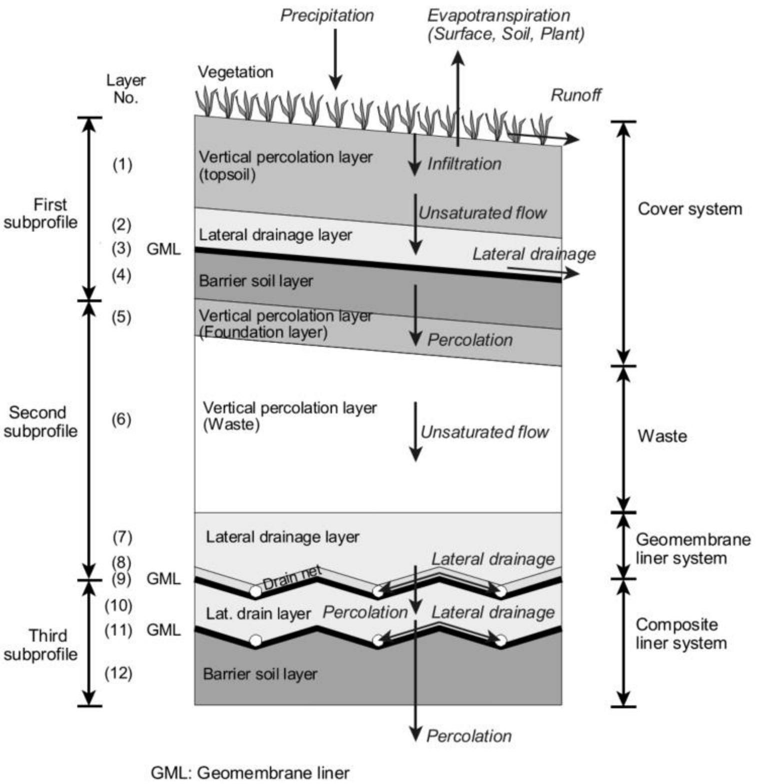

Figure 3.1 Schematic of Landfill Profile

3.1 Existing Conditions

2 layers were required to simulate existing conditions, the layer types for the soils are described below:

1. Cover Soil, 500 mm thickness – Vertical Percolation Layer
2. Waste Layer, varying thickness depending on chainage – Vertical Percolation Layer

The thickness of the waste layer varied depending on the modelled slice.

3.2 Expansion

The expansion of the Stage 2 landfill was assumed to be an asbestos only expansion and consist of 1.15 m layers (conservative) sloped at 10H:1V over the existing 5H:1V Stage 2 batter. The layer order for each chainage is presented in Table 3.1.

Table 3.1 Expansion Modelled Layer Arrangement

Chainage:	23 m		46 m		69 m		92 m	
	Thickness	Type	Thickness	Type	Thickness	Type	Thickness	Type
Layer 1 (Cover Soil)							1.15 m	VP
Layer 2 (Asbestos)							1.15 m	LD
Layer 3 (Cover Soil)					1.15 m	VP	1.15 m	BS
Layer 4 (Asbestos)					1.15 m	LD	1.15 m	LD
Layer 5 (Cover Soil)			1.15 m	VP	1.15 m	BS	1.15 m	BS
Layer 6 (Asbestos)			1.15 m	LD	1.15 m	LD	1.15 m	LD
Layer 7 (Cover Soil)	1.15 m	VP	1.15 m	BS	1.15 m	BS	1.15 m	BS
Layer 8 (Asbestos)	1.15 m	LD	1.15 m	LD	1.15 m	LD	1.15 m	LD
Layer 9 (Cover Soil)	0.5 m	BS	0.5 m	BS	0.5 m	BS	0.5 m	BS
Layer 10 (Waste)	17.9 m	VP	13.3 m	VP	8.7 m	VP	4.1 m	VP

1. VP = Vertical Percolation Layer, LD = Lateral Drainage Layer, BS = Barrier Soil Layer

4. Model Results

4.1 Existing Conditions

Resulting percolation through the Cover soil layer under existing conditions is presented in Table 4.1.

Table 4.1 Percolation Results (Averages, Existing Conditions)

Chainage:	23 m	46 m	69 m	92 m	Average
Percolation through layer (mm/year)	146.39	146.41	146.46	146.62	**146.47**

4.2 Expansion

Resulting percolation through the bottom of the Stage 2A asbestos filled wedge under the expansion scenario is presented in Table 4.2.

Table 4.2 Percolation Results (Averages, Expansion)

Chainage:	23 m	46 m	69 m	92 m
Percolation through bottom layer (mm/year)	Not modelled	149.94	150.15	Not modelled

Initial results at 46 m and 69 m chainages indicated a slightly higher seepage rate through to the existing Stage 2 area. Therefore, the other chainages were not modelled, and an additional assessment was conducted as described in Section 5.

5. Additional Assessment

Following the initial assessment and review of the preliminary results above, it was decided an additional assessment would be completed to assess the partially saturated condition of asbestos wastes, adopting a value of 10% below its saturation water content to represent the requirement that wastes transported into site being spadable, and hence partially saturated.

The filling operation for the expansion was also modelled at a surface grade of 20H:1V adopted for the asbestos waste over the existing 5H:1V Stage 2 batter.

5.1 Layer arrangement

The layer arrangement for the additional assessment was modelled with the same arrangement presented in Section 3.2 with the following variations:

- initial soil water content of the asbestos waste of 10% lower than its saturation soil water content.
- The surface grade of layers within the expansion was modelled at a slope of 20H:1V over the existing 5H:1V Stage 2 batter.

5.2 Results

The results from the additional assessment are outline in Table 5.1.

Table 5.1 Percolation Results (Averages, Expansion – Additional assessment)

Chainage:	23	46	69	92	Average
Percolation through bottom layer (mm/year)	143.06	141.00	138.97	136.67	**139.93**

The results of the additional assessment indicate the expansion results in reduced seepage rate into the existing Stage 2 area compared to the estimated infiltration rate through the existing Stage 2 outer slope.

6. Conclusion

Based on the adopted soil parameter for asbestos waste and cover soil as described in this report, the Stage 2A expansion over the existing Stage 2 waste slope is considered to result in less infiltration into the existing Stage 2 area than the modelled current conditions for the approximate 5H in 1V slope.

The key operational conditions for the expansion are:

- 5% cross fall on the asbestos waste and cover soils, grading downwards to the north.
- Diversion of upslope runoff around the Stage 2A waste filling area.
- Waste and cover soils placed in the cell to be partially saturated, i.e. spadable.

Laskazeski

Jarod Laskazeski
Senior Associate Civil Engineer

FGassner

Fred Gassner
Technical Director, GeoEnvironmental Engineering

Appendix C

Heat transfer assessment

Memo

To:	Jedda Bennett-Kellam (CoD)
From:	Amy Pyke
Subject:	**Interface layer to limit heat transfer**
Our ref:	PS138768-WSP-BNE-ENG-MEM-011 RevA Interface Layer for Heat Transfer
Date:	**20 March 2024**

1. Introduction

The City of Darwin (CoD) has engaged WSP to design a landfill expansion for the Stage 2 Inert Landfill, to be constructed at the Shoal Bay Waste Management Facility, Darwin.

The Stage 2 expansion project is intended to increase the available airspace of the operating landfill which receives waste materials generated within the greater Darwin area. The Stage 2 landfill expansion is intended to accept inert waste and asbestos only. CoD intends to expand to the north (Stage 2A) as part of the current development proposal, with potential expansions to the west (Stage 2B) and east (Stage 2C) in the future.

Due to risks associated with the current condition of the Stage 2 landfill, an interface layer was proposed in Technical Memorandum 'Potential Approach to Managing Stage 2 Risks on the Stage 2A Expansion' Ref: PS138768-WSP-BNE-ENG-MEM-003 RevA, dated 25 January 2024. The intent of the interface layer is to limit excessive heat transfer from the Stage 2 landfill into the Stage 2A northern expansion. WSP understands that the proposed intent of the interface layer has been generally agreed between CoD and the Auditor, and therefore, CoD has requested WSP to proceed with the assessments associated with the interface layer.

WSP has undertaken a heat transfer assessment to inform the design thickness of the interface layer. The purpose of this document is to:

- Provide the methodology and results of the heat transfer assessment.
- Present proposed material properties of the fill materials for the interface layer (developed in consultation with CoD) for agreement with the Auditor.

The intent is that this document is presented to the Auditor for comment before proceeding further. Should the Auditor agree with the proposed thickness and fill materials of the interface layer, the intent is to then progress with further assessments and detailed design associated with the interface layer.

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2. Heat transfer assessment

2.1 Method

Heat transfer modelling has been undertaken based on a simplified model for conductive heat flux described in Colls (2013)[1]. The method considers a series of concentric vertical cylinders of ground of different radii. The change in temperature is estimated for the various cyclinders from a defined initial starting temperature based on the net heat energy input or output into the cylinder volume. Over set time steps, there is radial heat transfer/heat flux between adjacent cylinders based on the temperature difference between each cylinder and their geometry.

The following assumptions have been made for the model:

- The landfill hotspot heat source has been assumed to be a constant temperature and an infinite source of energy. In reality, the heat source temperature will be a function of the heat power emitted by the source (which may be difficult to accurately assess given the inherent variability of landfill materials) and other factors such as the thermal conductivity of the surrounding materials that will affect how rapidly heat energy is conducted away from the source. The heat power emitted by the source and its temperature are also expected to change over time. For example, the waste materials may ignite and these parameters will increase until they reach a peak, and then decrease as the combustible material is consumed and as the hotspot moves through the waste. Assuming a constant temperature is considered a conservative assumption and allows simplification of the model.
- The temperature of the landfill hotspot heat source has been modelled at 239°C. This is equiavlent to the maximum temperature measured from subsurface temperature monitoring of the existing Stage 2 hotspot.
- The heat source is modelled as a single cylindrical column with a diameter of apprioxmately 1 m that extends to a depth of approximately 15 m, and is located at the outer edge of Stage 2 at the interface layer.
- It has been assumed that a waste temperature of less than 150°C would not lead to auto-ignition in inert waste materials. This assumption is supported by auto-ignition temperatures of common materials published online[2,3] which indicates auto-ignition temperatures for materials such as paper and wood to be greater than 233°C. Therefore, the allowable temperature of 150°C is considered conservative with respect to inert waste materials.
- Only conductive heat flux (generally, movement of heat energy by particle to particle contact) has been considered. If convective heat flux (e.g. movement of heat energy by air or water) takes place it would alter the model output, resulting in lower or higher temperatures than have been modelled depending on whether the convective heat flux removes or adds energy to the soil or waste around the heat source. Due to the compaction of both waste materials during landfilling operations and compaction during construction of the proposed interface layer, we consider this assumption representative of the field condition.
- The material surrounding the heat source is assumed to be at a fixed temperature of 60°C. The temperature of the surrounding material is based on the upper end of the general measured landfill temperature that has been recorded to range from about 30°C to 60°C (from temperature monitoring activites associated with the Stage 2). If the surrounding temperature is lower than the assumed assumed 60°C, then the heat flux results in a lower temperature at a given distance from the heat source, when compared to a fixed temperature of 60°C. By selecting the upper bound of measured ambient landfill temperatures, this is considered a conservative assumption.
- The ground surrounding the heat source has been assumed to have a thermal conductivity (TC) of 1 W/mK and a volumetric specific heat capacity (SHC) of 2 MJ/m^{3}K. Typical TC values for clay/silt soils are in the range of 1

---

1 Colls, S. (2013). “Ground heat exchanger design for direct geothermal energy systems”. PhD thesis, University of Melbourne Department of Infrastructure Engineering.

2 Fire and Safety Centre UK (2024). “Auto Ignition Temperature” accessed on 19 March 2024. https://www.fireandsafetycentre.co.uk/blogs/safety-storage/auto-ignition-temperature

3 TaylorEdge (2024). “Ignition Temperatures of Materials” accessed on 19 March 2024. https://www.tayloredge.com/reference/Science/ignition.html

W/mK to 2 W/mK and a SHC value of 2MJ/m^{3}K is a typical value for soils and weathered rock. Both of these paramaters are considered representative of soil-based materials proposed to form the interface layer. Generally, a higher TC is expected to result in lower temperatures near the heat source but higher temperatures further away (i.e. a shallower temperature gradient), and vice versa for a lower TC. A lower SHC would mean that the ground volume around the source changes temperature more rapidly for a given energy input than has been assumed, and vice versa.

- In reality, there is also likely to be heat flux from the heat source in an upwards and downward direction both upwards towards the ground surface (because the air temperature is lower than that of the heat source, and the air can transport heat energy away from the surface by convection) and towards the waste/ground below the heat source (which has a large volume therefore would not change temperature rapidly). If this axial heat flux takes place, then the temperatures around the heat source would be lower than modelled, as this direction of heat flux has not been considered in the model.

2.2 Results

The results of the heat transfer assessment indicate temperatures in surrounding materials adjacent to the landfill hotspot (heat source) to be less than approximately 150℃ at a horizontal separation distance of 3.5 m. Therefore, based on the above assessment, an interface layer (constructed from soil-based materials) that provides 3.5 m of separation from the waste edge in Stage 2, should limit the temperature at the northern face of the separation layer to less than 150℃. Assuming a maximum allowable temperature in the new waste of 150℃, which is understood to be below auto-ignition temperatures for inert waste/building materials, this should limit the potential for the migration of a hotspot from Stage 2 into the Stage 2A northern expansion.

3. Proposed fill materials

To make this a sustainable design it is proposed that asbestos in soil (ABINS) materials are used for the construction of the separation layer, on the basis that the asbestos fragments within the ABINS are non-combustible and have lower heat conductivity than soil. ABINS materials are currently received and landfilled in Stage 2 and could be used to supplement soil materials as part of a progressive construction of the interface layer.

The interface layer is proposed to be constructed in stages as the cell is being filled with waste, to provide stability and limit potential weather related desiccation of the interface layer. The use of ABINS material as part of the progressive construction of the interface layer throughout operations would likely require material handling and quality control of the separation layer fill materials by the Operation's Team as well as possibly third-party verification during its construction.

4. Closing

We seek agreement from the Auditor on the proposed thickness and fill materials for the construction of the interface layer.

We trust that this memorandum meets the requirements of CoD at this stage.

A.Pyke

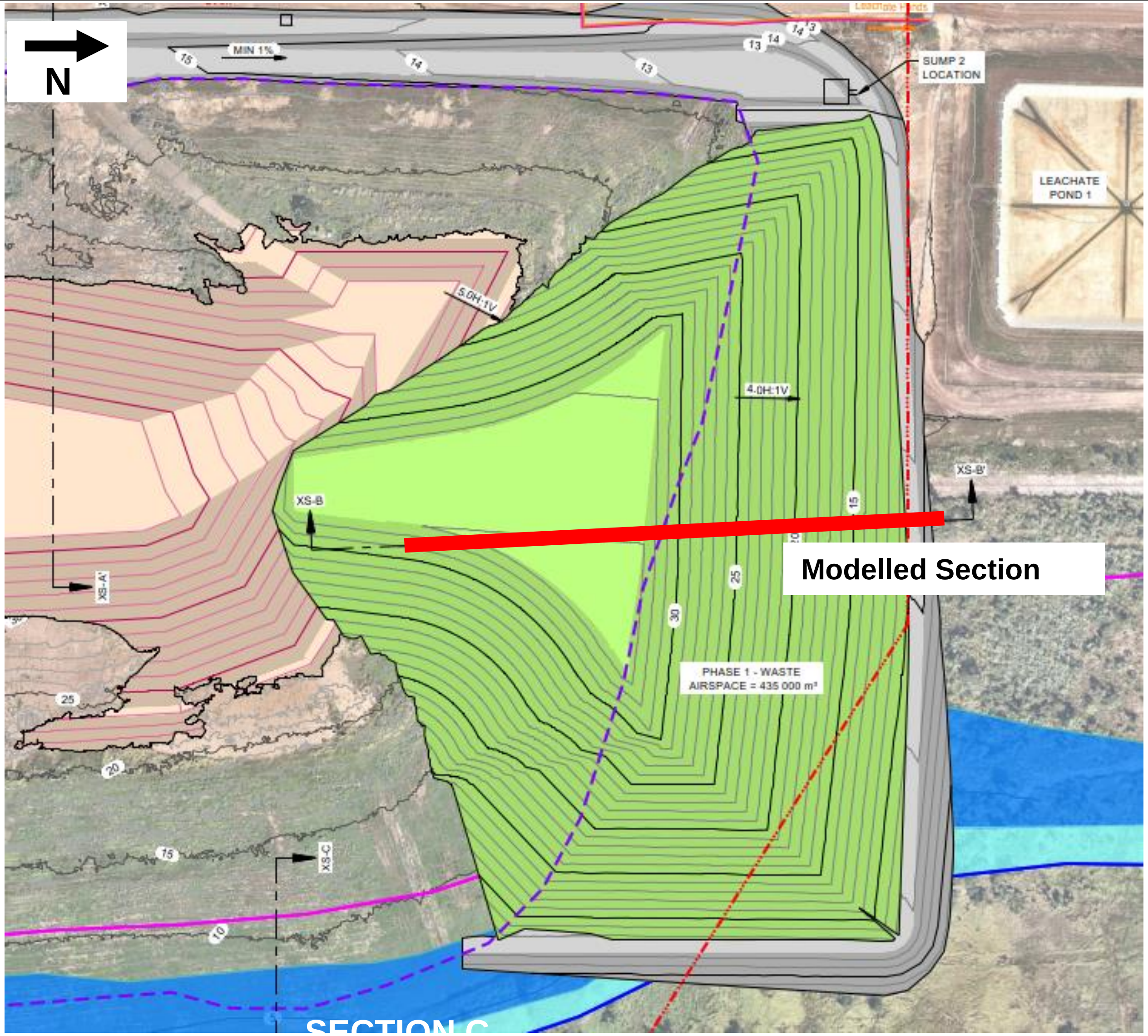
FGassner

Amy Pyke
Senior Civil Engineer

Fred Gassner
Technical Director, GeoEnvironmental Engineering

Appendix D

Slope stability analysis



Note: The Stage 2A Expansion Cell Footprint has been updated, refer to the Drawing Set provided with the Design Report.

CLIENT City of Darwin			PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024		TITLE Cross Section Layout	
CHECKED JL	DATE 04/12/2024			
SCALE As indicated on axes			PROJECT No PS138768	FIGURE No 1 / REV No 0

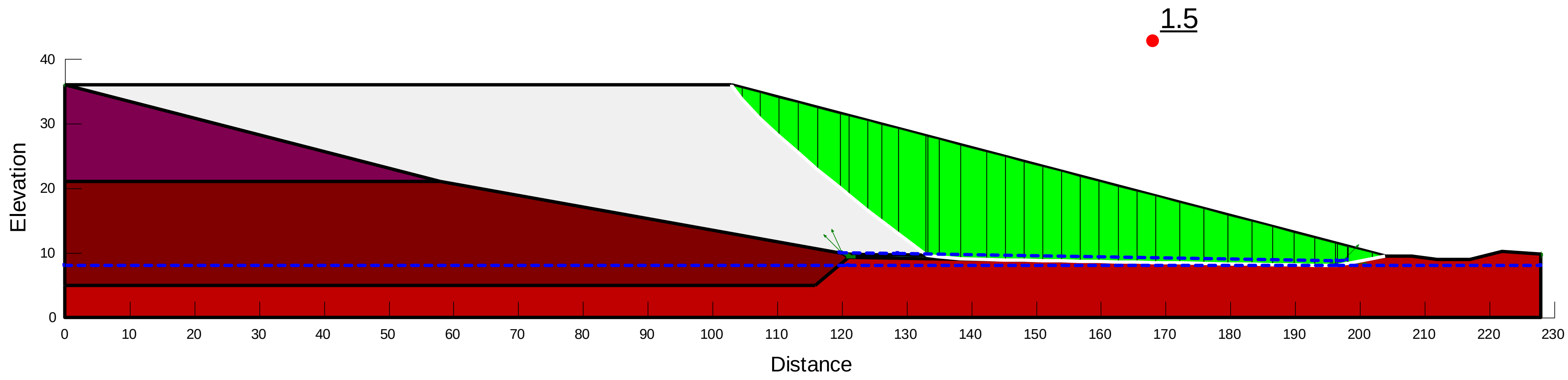
Assessment Results Summary

North-South Assessment	Modelled FoS	Minimum Target FoS
Assessment Results		
Post Filling, Static, Slip Surface Through Liner	1.5	1.5
Post Filling, Seismic Loading	1.1	1.1
Future Development Filling, slip through liner	1.5	1.5
Future Development Filling, Seismic Loading	1.1	1.1
Sensitivity Analysis		
Post Filling, 1m of Leachate in Existing Cell	1.5	1.5
Post Filling, 1m of Leachate in Existing Cell, Seismic Loading	1.1	1.1
Post Filling, GCL friction angle of 10.2°, cohesion 2.4 kPa (Fox and Stark Paper, 2015)	1.3	1.3
Post Filling, GCL friction angle 6°, cohesion 12 kPa (GRI Report, 2005)	1.3	1.3

CLIENT: City of Darwin		PROJECT: Stage 2A Inert Landfill Expansion	
COMPLETED: GM	DATE: 04/12/2024	TITLE: Summary of Results	
CHECKED: JL	DATE: 04/12/2024		
SCALE: As indicated on axes		PROJECT No: PS138768	FIGURE No: 2 / REV No: 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - GCL	Mohr-Coulomb	12	0	14	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	0	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

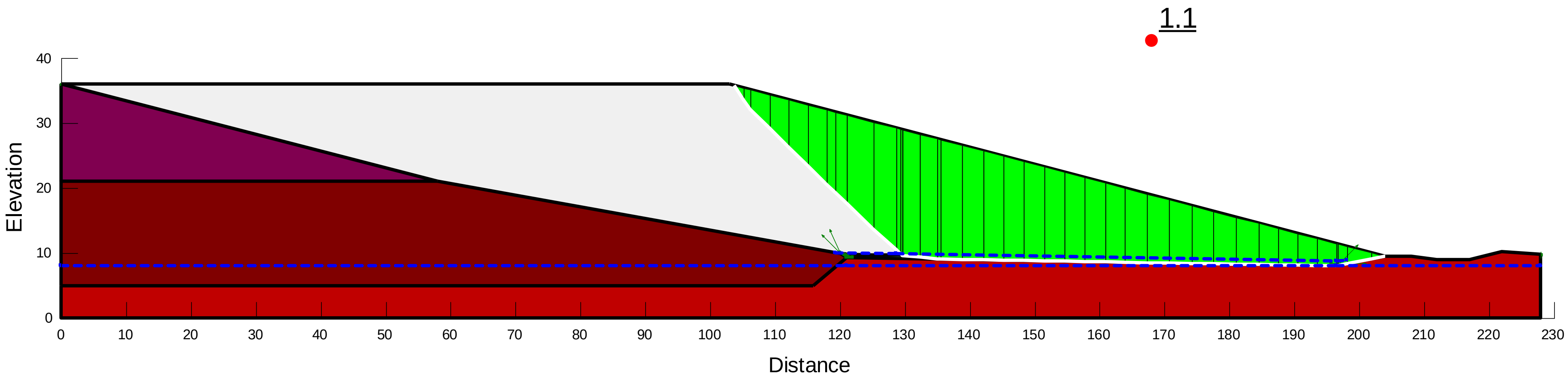
Horz Seismic Coef.:



wsp	CLIENT City of Darwin		PROJECT Stage 2A Inert Landfill Expansion
	COMPLETED GM	DATE 04/12/2024	TITLE Post Filling, Static, Slip Surface Through Liner
	CHECKED JL	DATE 04/12/2024	
	SCALE As indicated on axes		PROJECT No PS138768 / FIGURE No 3 / REV No 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - 20% seismic reduction	Mohr-Coulomb	12	0	11.2	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	0	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

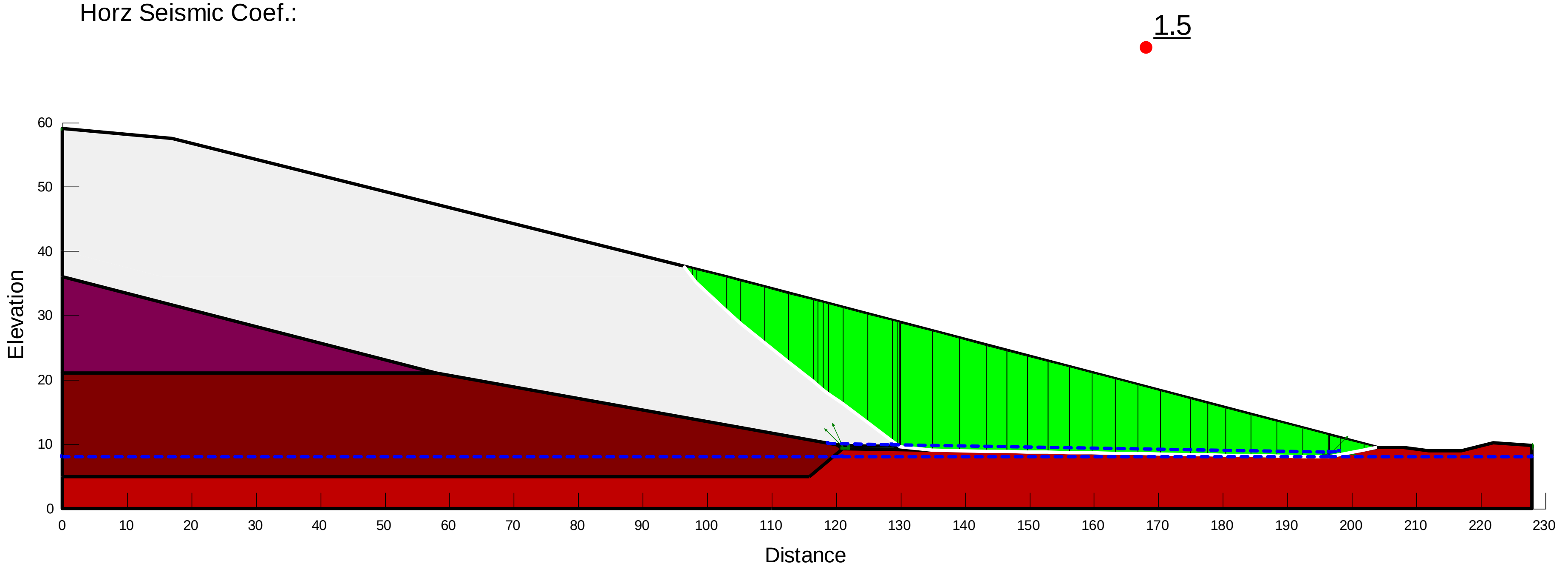
Horz Seismic Coef.: 0.045



CLIENT City of Darwin		PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024	TITLE Post Filling, Seismic Loading	
CHECKED JL	DATE 04/12/2024		
SCALE As indicated on axes		PROJECT No PS138768	FIGURE No 4 / REV No 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - GCL	Mohr-Coulomb	12	0	14	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	0	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

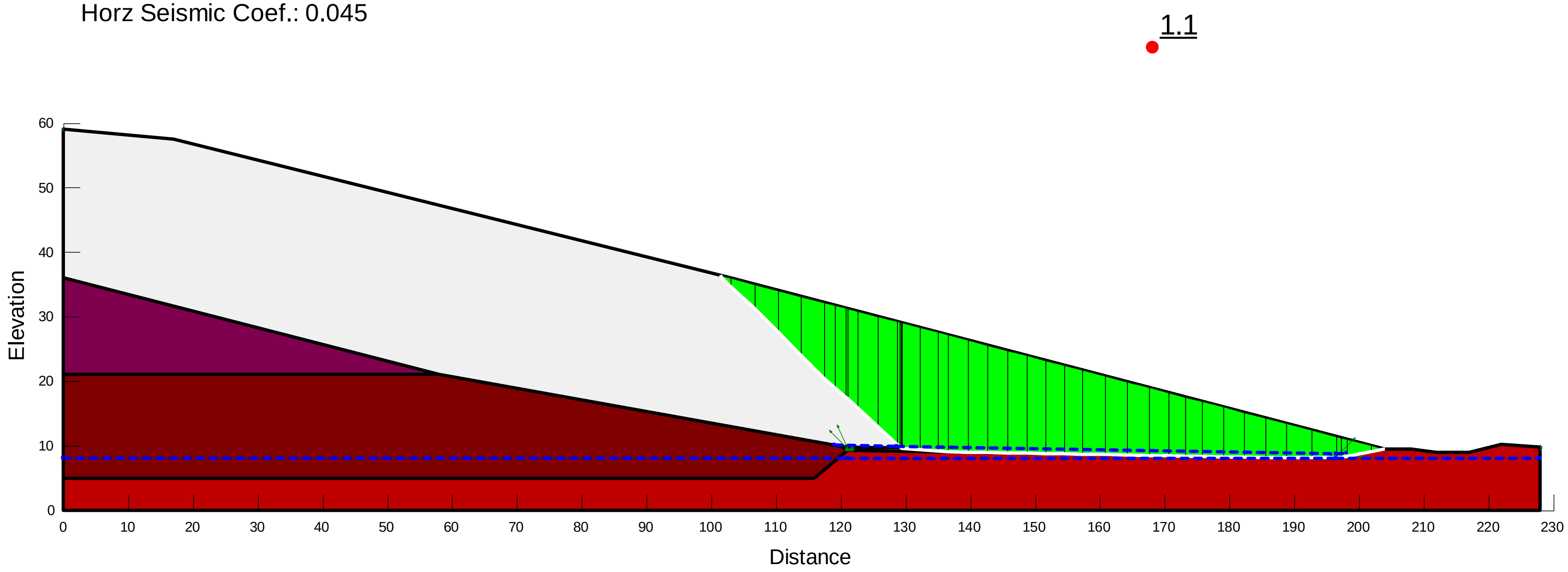
Horz Seismic Coef.:



CLIENT City of Darwin		PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024	TITLE Future Development Filling, slip through liner	
CHECKED JL	DATE 04/12/2024		
SCALE As indicated on axes		PROJECT No PS138768	FIGURE No 5 / REV No 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - 20% seismic reduction	Mohr-Coulomb	12	0	11.2	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	0	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

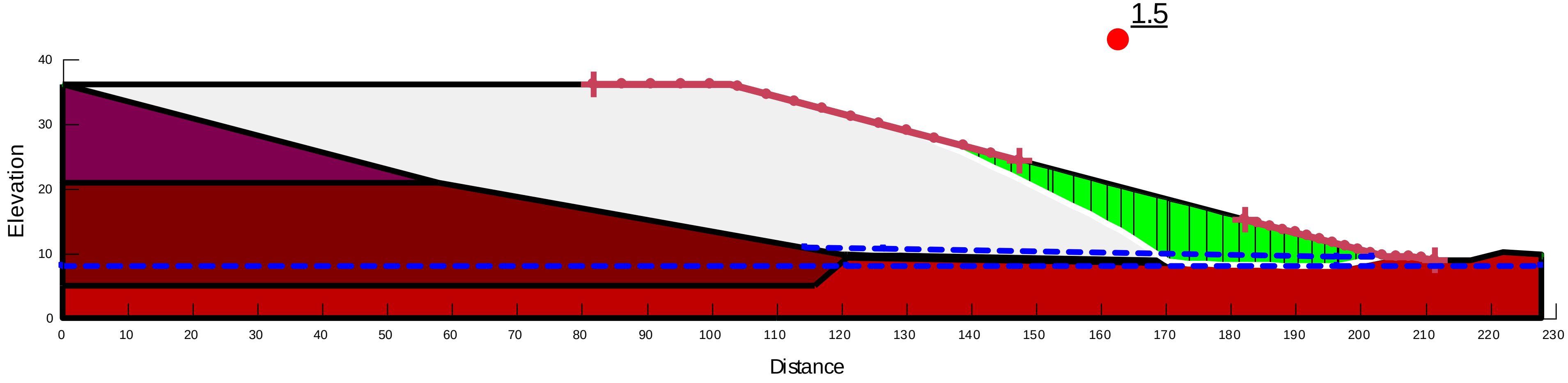
Horz Seismic Coef.: 0.045



CLIENT City of Darwin		PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024	TITLE Future Development Filling, Seismic Loading	
CHECKED JL	DATE 04/12/2024		
SCALE As indicated on axes		PROJECT No PS138768	FIGURE No 6 / REV No 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - GCL	Mohr-Coulomb	12	0	14	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	0	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

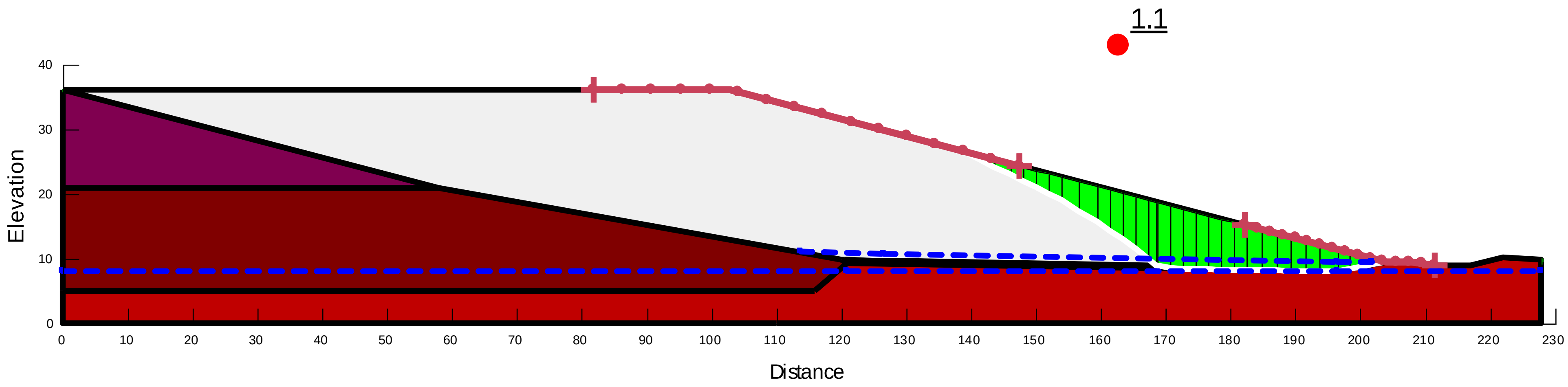
Horz Seismic Coef.:



CLIENT City of Darwin		PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024	TITLE Sensitivity Analysis - post filling, 1m of leachate in existing cell	
CHECKED JL	DATE 04/12/2024		
SCALE As indicated on axes		PROJECT No PS138768	FIGURE No 7 REV No 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - 20% seismic reduction	Mohr-Coulomb	12	0	11.2	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	0	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

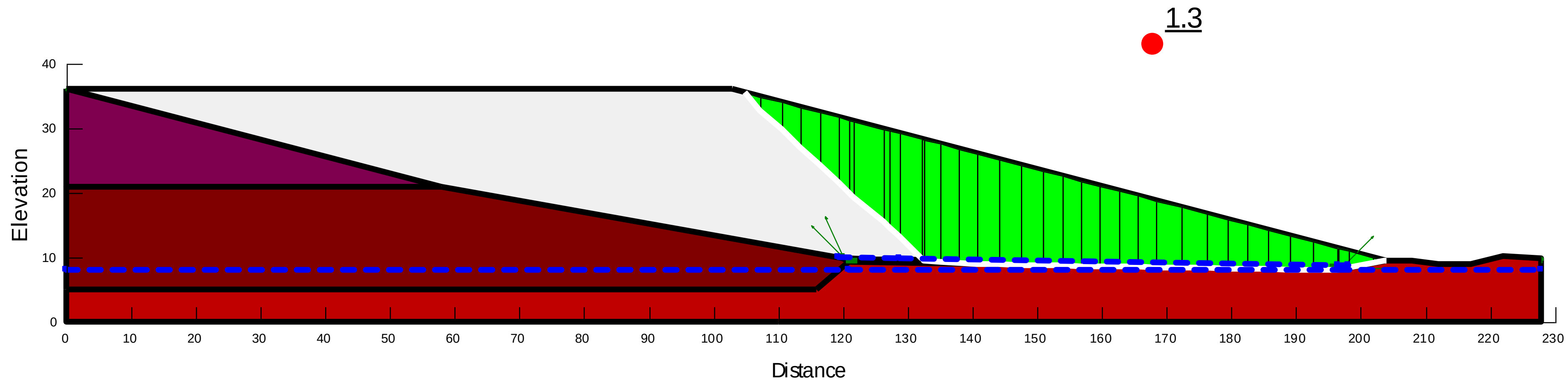
Horz Seismic Coef.: 0.045



CLIENT City of Darwin			PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024		TITLE Sensitivity Analysis - post filling, 1m of leachate in existing cell, seismic loading	
CHECKED JL	DATE 04/12/2024			
SCALE As indicated on axes			PROJECT No PS138768	FIGURE No 8	REV No 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - GCL Fox and Stark Paper	Mohr-Coulomb	12	2.4	10.2	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	0	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

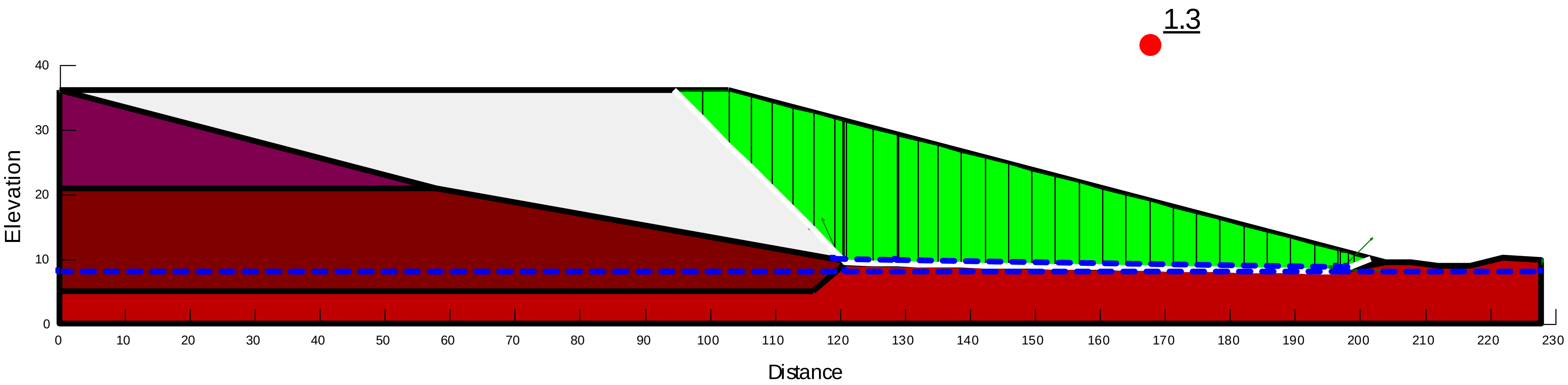
Horz Seismic Coef.:



CLIENT City of Darwin		PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024	TITLE Sensitivity Analysis - post filling, GCL friction angle of 10.2°, cohesion 2.4 kPa	
CHECKED JL	DATE 04/12/2024		
SCALE As indicated on axes		PROJECT No PS138768	FIGURE No 9 / REV No 0

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Existing Ground	Mohr-Coulomb	20	5	35	0	2
	Liner - GCL GRI Values	Mohr-Coulomb	12	12	6	0	1
	New Waste - Asbestos	Mohr-Coulomb	18	11	28	0	1
	Old Waste - General	Mohr-Coulomb	12	10	25	0	2
	Old Waste - Inert	Mohr-Coulomb	12	10	25	0	2

Horz Seismic Coef.:



CLIENT City of Darwin			PROJECT Stage 2A Inert Landfill Expansion	
COMPLETED GM	DATE 04/12/2024		TITLE Sensitivity Analysis - post filling, GCL friction angle 6°, cohesion 12 kPa	
CHECKED JL	DATE 04/12/2024			
SCALE As indicated on axes			PROJECT No PS138768	FIGURE No 10 / REV No 0

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