REPORT

Tonkin+Taylor

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Auckland Regional Landfill

Engineering Report

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1 Introduction

Tonkin & Taylor Ltd (T+T) has been engaged by Waste Management NZ Ltd (WMNZ) to undertake an engineering concept design to support resource consent applications for a new landfill located in the Wayby Valley area north of Auckland. The proposed landfill is described herein as the Auckland Regional Landfill. The Auckland Regional Landfill will provide a new solid waste management and disposal facility to replace the Redvale Landfill which currently provides for disposal of a significant portion of Auckland’s solid waste.

This report describes the engineering concept design of the proposed landfill. The design has been undertaken to a level to provide a basis for all specialist assessments of potential effects associated with the landfill development, and to clearly define the general works proposed. The consent design presented herein will be developed into a detailed design for construction purposes, where the concepts described will be further developed and may change in detail from those presented. However, it is not anticipated that the overall concepts will change, other than to incorporate technology and practice developments that may occur over the relatively long life of the project. Any changes made will be within the scope of the resource consents or other approvals that have been given for the project.

This report covers the following aspects of the project:

- The proposed formation of the landfill, staging, finished fill profile and airspace volume provided;
- Construction of the landfill, including toe bund requirements, operational soil requirements and other construction aspects;
- The key environmental containment measures including the lining and capping systems.
- Leachate management systems;
- Stormwater management systems;
- Landfill gas management; and
- Ancillary works and infrastructure required for the landfill including access roads, the bin exchange area, staff facilities, wheel wash, water supply and wastewater disposal.

The description of these aspects is supported by a series of concept level drawings that are included in Volume 3.

2 Site description

The proposed landfill is located in the Wayby Valley area, approximately 6 km southwest of Wellsford and 70 km north of Auckland. The landfill is to be constructed in a northwest facing valley currently vegetated with pine forest. This portion of the site is described as the Eastern Block or Forestry Block. The landfill will be constructed entirely in the catchment of this valley, at the head of the catchment. Access roads and some ancillary facilities will be constructed in neighbouring catchments.

Access to the landfill is proposed off State Highway 1 just south of the Hōteo River bridge, where a sealed road will be constructed approximately 2 km in length commencing at an intersection on SH1 and climbing up a valley before crossing a ridge into the main landfill valley. The predominant vegetation in this access road valley is a plantation wattle forest. This portion of the site is described as the Southern Block.

Small portions of an adjacent farm block, described herein as the Western Block, will be utilised as required for the long-term stockpiling of soils needed for landfill operation, and as a source for obtaining suitable clay soils for landfill liner construction.

A plan showing the overall site is shown in Volume 3, Drawing ENG-01. A full description of the site can be found in the AEE.

The topography of the Eastern Block which will contain the landfill consists of a series of relatively steep-sided valleys, with natural slopes as steep as 1V:2H. North facing slopes are typically less steep. The Southern Block drains to the Waiteraire Stream which in turn flows to the Hōteo River. The Eastern Block drains to an unnamed tributary of the Hōteo River.

The site geology comprises Pakiri Formation sedimentary rocks of the Waitemata Group.

3 Project description

The project comprises the construction of a landfill with a capacity of approximately 25.8 Mm^3 to provide for the safe disposal of municipal solid waste for a period in excess of 35 years. The landfill will be designed to accept municipal solid waste in accordance with acceptance criteria described in a separate report. The overall project will comprise:

- All works associated with the development of an operating landfill on the identified footprint area including:
  - Earthworks to construct the required shape;
  - Construction of a lining system including a low permeability lining system to prevent leachate seepage into the surrounding environment;
  - Construction of a leachate collection system above the low permeability lining system;
  - Stormwater control around the constructed landfill and ultimate treatment of stormwater before it leaves the site;
  - A landfill gas (LFG) collection system to collect LFG from the placed waste;
- A leachate management system, including leachate storage, tanker loading facilities and leachate treatment facilities;
- LFG treatment by a LFG to energy plant, with any excess being flared;
- Provision of water supplies for operational (non-potable) and staff (potable) requirements;
- A bin exchange area near the site entrance, adjacent to SH1, where road vehicles will deposit bins for site vehicles to transport them to the landfill tip face;
- An access road from the site entrance to the main site, and all other roads required to access the various parts of the site.
- Operational infrastructure such as weighbridges and vehicle wheel wash;
- Facilities for site staff, including on-site wastewater disposal;
- Maintenance facilities for site plant and equipment.

The details of these works are described in subsequent sections of this report.

Development of a landfill is essentially a long term construction project. The landfill will be developed in stages, with one stage being filled with waste while the next stage is constructed.

4 Landfill design

4.1 Geotechnical design

The geotechnical design for the landfill is detailed in the Geotechnical Interpretive Report (GIR) (Techncial Report B, Volume 2). The concept level design for all works associated with the Auckland Regional Landfill have been developed in accordance with the recommendations of the GIR.

The geotechnical investigations show that the proposed landfill footprint and access road alignment are underlain by Pakiri Formation bedrock consisting of interbedded sedimentary sandstone and siltstone with some conglomeritic layers. The bedrock is overlain by a variable thickness of residual, colluvial and landslide soil. The site soils generally consist of silts and clays with fine sand of variable strength and plasticity. Overall, the GIR concludes that the land within the proposed project area is suitable for landfill development in accordance with the Technical Guidelines for Disposal to Land. In general, the landfill footprint overlies low permeability soils, which will provide good natural containment, is not close to any active faults and does not overlie Karst geology. Suitable soils are available on site for liner construction and landfill operation.

4.1.1 Formation stability

Slope stability analyses provided in the GIR for the proposed cut and fill design slopes generally indicate adequate slope stability for the proposed design. Design cut slopes should not exceed 1V:2H in residual soils and 1V:0.5H in rock. Fill slopes should not exceed 1V:2.5H. Existing landslide features identified within the Southern Block access road alignment and in the Valley 1 landfill footprint will require additional investigation, slope stability modelling and hazard and risk assessment during the detailed design of the works.

4.1.2 Landfill stability

The overall form and design of a landfill must be such that the landfill is stable over its lifetime. For a valley landfill, as proposed for Auckland Regional Landfill, the sides of the valley contain the fill so that once filled across the valley the overall landfill is stable on any cross section through the landfill. However, consideration needs to be given to the stability of the landfill along the length of the valley.

Overall stability of the landfill along the length of the valley is maintained by:

- Providing a toe bund at the base of the landfill to support the fill placed behind it;
- Constructing a fill face above the toe bund at a stable slope, in this case selected to be 1V:5H flattening to no steeper than 1V:10H on the upper surfaces;
- Providing materials with appropriate interface friction angles at the base of the landfill to protect against a base sliding failure or a potential circular slip failure through the base.

The overall stability of the landfill is considered in detail in Section 7.2 of the GIR. The stability of the landfill long section is summarised in Table 7.4 of that report, and the table is repeated below (Table 4.1) for ease of reference.

Table 4.1: Landfill long section stability

Analysis Case	Design Factor of Safety (FoS)	Calculated FoS
Static long term	1.5	2.7
Static short term (clay bund)	1.5	4.8
Elevated groundwater	1.2	2.7
Seismic ULS	1.0	1.0
Seismic SLS	1.0	1.6

4.2 Landfill formation and airspace

4.2.1 Landfill footprint

There are two valleys on the overall site capable of providing the target airspace volume: Valley 1, the southernmost valley of the Eastern Block and Valley 2, the northernmost valley. Valley 1 has been selected for landfill development, instead of the alternative Valley 2, predominantly because it is more accessible by likely access routes to the site, as described in Section 9.

The footprint has been selected to maximise the fill potential of Valley 1, the estimated airspace volume for Valley 1 being approximately 25.8 Mm^3.

The key considerations in determining the footprint were:

- The downstream extent was determined to allow sufficient area downstream of the landfill for long term stormwater treatment facilities, leachate handling and other operational requirements within Valley 1 prior to the confluence with the watercourse from Valley 2;
- The perimeter of the landfill was determined to allow for a perimeter road with drainage at approximately a minimum 2 % grade to the mouth of the valley. This requirement dictates the maximum extent of the southern side of the landfill footprint, with a low point in the southern ridge near the eastern end of the valley determining the maximum extent of the southern perimeter;
- The north and south perimeters were determined to fit closely to the existing contours, with practical “straightening” of the perimeter to facilitate access along the perimeter road and facilitate liner construction. This results in a series of cuts and fills along the perimeter;
- The eastern extent of the landfill determined to practically fit within the existing landform, while considering a practical final fill profile for the landfill.

The landfill footprint and overall site layout is shown on Drawing ENG-01 in Volume 3.

4.2.2 Basegrades

Valley 1 is a relatively narrow, steep sided valley with many spurs and side valleys along each side. Natural slopes along the northern side of the valley are typically approximately 1V:2.5H with some locally steeper areas. Slopes along the southern side of the valley are shallower, typically 1V:3H to 1V:4H, again with some areas with locally steeper or shallower slopes.

Soil is required for landfill operation throughout the life of the landfill, to be used for construction of the lining system, daily cover, intermediate cover and to form the final capping layer. Over the life of a landfill soil requirements amount to approximately 20 % of the total airspace volume. Therefore, an airspace volume of 25.8 Mm^3 requires approximately 5 Mm^3 of soil for operation of the landfill over its lifetime. Ideally the soil required for operation of one stage of the landfill will come from excavation within the footprint of the next stage to be constructed. However, this is not always possible over the life of a landfill and some excavated soil needs to be stockpiled for later use. The greatest soil requirement comes at final closure of the landfill when a significant quantity of soil is required for the landfill capping system, so some soils need to be held for a long period of time to meet this need.

In the case of the Auckland Regional Landfill, soils for operation of the landfill will need to be found from within or nearby to the landfill footprint. Therefore, the basegrade design needs to provide for significant excess cut over fill.

In addition to providing a net surplus of excavated material the following have been taken into consideration for establishing the basegrade geometry for the development:

- A minimum slope of 2 % anywhere within the footprint to facilitate leachate flow to the leachate collection pipe system;
- The proposed geocomposite lining system incorporates a low permeability clay layer overlain by geosynthetic components. While steeper slopes are possible, experience shows that lining system construction is easier on slopes no steeper than approximately 1V:3H. This allows for construction plant to operate up and down the slope for placing, compacting and trimming clay and allows for safer deployment and installation of the geosynthetic liner components;
- There is a limit to the slope length on which geosynthetic liner components can be practically deployed. For the Auckland Regional Landfill, a vertical height between benches of 20 m has been adopted. At 1V:3H this gives a slope length of 63 m. For typical 1.5 mm HDPE geomembrane roll lengths (140 m) this provides for two lengths per roll with allowance for anchoring at the top of the slope and run-out at the base of the slope;
- The benches need to provide for anchoring the geosynthetic components, perimeter drainage (both on the landfill side and the outer side of the bench), litter fences (where required) and access. A bench width of 12 m has been selected as the design bench width. Where a greater bench width may be required for two-way refuse truck access, the width can be temporarily widened by steepening the cut slope above. A typical layout of a bench is shown on Drawing ENG-21;
- The basegrade formation should be as planar as practicable. This is to facilitate forming the specified clay layer thickness and for installation of the geosynthetic components;
- Where practicable, excavation for the basegrade should be in weathered soils to facilitate excavation. However, some rock excavation may be considered. (Note: the extent of geotechnical investigations within the proposed footprint area is limited due to access limitations associated with the relatively steep terrain and the site being heavily forested. The depth of weathered material has been inferred from the investigations undertaken. The actual form of the basegrades may be modified during detailed design based on more detailed geotechnical information available at that time);
- A toe bund will be constructed at the base of the landfill in the order of 10 m height. This allows for containment of leachate at the toe, stability of the landfill and provides a practical starting point for filling from the toe of the landfill;
- The perimeter road, in combination with other access roads, will provide access for landfilling operations at some locations. Therefore, these portions of road need to be at a grade suitable for refuse vehicle traffic, typically not steeper than 1V:10H.

The proposed concept level final basegrade, designed to meet the above requirements, is shown by the top of liner levels on Drawing ENG-10. In summary the basegrade design comprises:

- A toe bund constructed to RL 93 m;
- Side slopes on the south and west sides of the footprint at 1V:3H;

- Side slopes on the north side of the footprint at 1V:2.5H. The slightly steeper formation slope has been selected to, as far as is practicable, match the existing slopes, minimise the cut/fill requirements and prevent excess break-through on the ridge above the slope;
- Inter-bench vertical height of 20 m;
- Bench width of 12 m;
- Longitudinal bench slope of approximately 2 %;
- Minimum floor slope of 2 %.

4.2.3 Final fill profile

The function and construction of the landfill final cap is described in Section 5.5 below, while the overall form of the final fill profile, and hence the final cap, is described here.

The final fill profile has been designed to maximise filling within Valley 1 and in accordance with the following parameters:

- The maximum overall slope after settlement will be 1V:5H. This is selected for overall stability of the landfill and to minimise potential erosion of the cap surface by overland flow;
- The minimum slope at any point of the landfill will be 1V:20H (5 %). This is selected to provide for stormwater runoff without ponding, even if some differential settlement has occurred.

Contour drains will be installed on the finished cap surface to divert stormwater to the landfill perimeter drains. These will typically be installed at intervals of 50 to 75 m down the slope to collect stormwater from the surface at regular intervals to avoid large sheet flow over the surface potentially causing erosion. These drains will be formed on benches, resulting in local steepening of the cap slope, while maintaining the overall post-settlement slope of no greater than 1V:5H.

After waste is placed in a landfill it undergoes significant settlement due to biodegradation of the organic fraction, consolidation as a result of new layers of waste placed above old, physical-chemical changes occurring within the waste and other changes that occur over time and as a result of additional loads (mechanical changes, ravelling, etc.). A large part of the overall settlement occurs during the operational life of a landfill. Typically, once fill has been placed to the final fill level, it could be expected that the final surface will settle by around 5 to 15 % of the landfill depth over the aftercare period of typically 20 to 30 years. The actual settlement will depend on:

- The nature of the waste material placed, particularly the organic fraction;
- The extent of the compaction achieved in the placement phase;
- The time taken to reach the final fill profile;
- The staging of the filling, and time delays between vertical stages.

The expected final contours after settlement are shown on Drawing ENG-12. To achieve these contours, the final lifts of waste will be placed above these levels to allow the surface to settle to approximately the levels shown.

4.2.4 Overall volumes and areas

The key overall physical parameters for formation of the landfill are:

- Airspace volume: 25.8 Mm^3
- Landfill footprint (liner area): 58.5 ha
- Area affected by landfill (incl perimeter road and cut batters): 71.87 ha
- Overall earthworks cut volume (excluding access roads): 5.08 Mm^3

- Overall earthworks fill volume (excluding access roads) 0.8 Mm^3
- Balance of cut over fill: 4.28 Mm^3

4.3 Landfill phasing

There are broadly two options for phasing a landfill within a valley system, namely either starting operations from the bottom of the valley and working up, or starting at the top of the valley and progressively forming cells down the valley. In regards to ultimate loss of stream habitat within the valley both scenarios result in the same outcome. The main difference is that with a top down approach the loss of stream habitat could potentially be managed to be progressive over time, rather than immediate.

Both approaches have operational challenges. Working from the top down results in leachate creation at the top of the valley. Managing the leachate from the cells above is difficult during the construction of subsequent cells down the valley, including having live leachate pipes crossing the construction area which are at risk of damage, and the need to connect new areas into live leachate pipes which can be difficult to achieve.

In contrast, with a bottom up approach, the main operational challenge in the early stages is managing the large volumes of stormwater from the catchments above the operational area of the landfill. However under this scenario, the discharge to be managed is uncontaminated stormwater rather than leachate.

The overall footprint of the landfill and therefore the footprint of stream reclamation within Valley 1 would not change under either approach. Under both scenarios the main stormwater treatment ponds would be constructed in the early stages of the initial construction and site establishment, sized for treatment of the entire Valley and will form part of the permanent infrastructure for the site. The ponds need to be online, due to the limited area available at the base of the Valley. Once constructed, the ponds would form a barrier to fish passage and the upstream migration of fish would be disrupted. Fish passage would not be provided through the stormwater ponds as the upstream environment will be covered by landfill development. As such, removal of fish would need to occur prior to construction of the stormwater ponds under both scenarios.

WMNZ’s experience on other sites has resulted in a preference for bottom up operation of landfills.

In the case of Auckland Regional landfill, due to the nature of this site, access to the bottom of the valley will be much easier in the initial stages of operation, favouring bottom up construction, compared to access requirements for a top-down operation.

A practical combination of the two approaches has been adopted for Auckland Regional Landfill. The phasing has been designed to provide significant capacity in the stormwater treatment system in the early stages of the landfill operation, with the proposed stormwater pond design providing in excess of the volumes typically required by good practice. This additional capacity will be provided on the lower landfill footprint, requiring that the first phase of development be upstream from the final landfill toe, working “bottom up” from that point with the final phase of development following a “top down” approach.

Drawing ENG-25 shows the expected phasing of development of the landfill. The conceptual phase plan is based on:

- An operational life of each phase of approximately five years (although this varies to suit the actual layout of each phase). It is likely that the larger phases shown would be constructed as a number of sub-stages;

- A waste input of approximately 500,000 tonnes per annum (tpa). Assuming approximately a 0.83 t/m^{3}airspace utilisation rate (allowing for compaction and cover soil) this will occupy approximately 600,000 m^3 per year;
- Commencement of filling approximately 300 m from the final toe of the landfill. This is proposed to allow for additional stormwater treatment on the landfill footprint during the early phases of the landfill operation (refer Section 6.3). When waste filling up to the first bench reaches the head of the valley, all stormwater systems will be above the landfill level and stormwater can be diverted around the landfill. Until that time, stormwater will be diverted through a pipe under the landfill;
- The vertical height of each phase is generally based on filling up to each of the benches at 20 m height intervals;
- The general phasing plan comprises:
  - **Phase 1:** filling to below the first bench, with filling progressing from the interim toe of the landfill heading up-valley in a west to east direction;
  - **Phase 2:** filling up to the next bench above Stage 1 whilst leaving the lower bench to the north to provide future drainage from the east of the valley. A pipe would need to be installed on the first bench below the northern side of Phase 2 to carry stormwater from areas of this bench to the west when Phase 3 is constructed;
  - **Phase 3:** located to the east of Stage 1, to complete filling on the floor of the landfill below the first bench. The agreement between WMNZ and Matariki Forests prevents access to the eastern portion of Phase 3 until 2031;
  - **Phase 4:** filling up to the second bench above Phase 3 along the northern side of the landfill, connecting to the eastern edge of Phase 2. The southern portion of Phase 4 would be completed to the perimeter road level;
  - **Phase 5:** completion of filling at the eastern end of the landfill, above Phase 4 to final cap levels;
  - **Phase 6:** filling above Phase 1 and 2 to fill to final cap levels in this area;
  - **Phase 7:** completion of filling from the toe of the landfill over the face of Phases 1, 2 and 6 to complete filling of the landfill. Any infrastructure on the footprint of Phase 7 would be relocated as part of the construction.

The phasing plan is indicative only and will be developed as landfilling proceeds. A summary of the identified phases are shown in Table 4.2.

Table 4.2: Landfill phasing details

Phase	Airspace volume (Mm^3)	Approximate life (years)	Liner area (ha)
1	1.2	2	9.8
2	2.2	3.7	4.4
3	0.8	1.4	6.0
4	6.6	11	13.2
5	4.3	7.2	9.9
6	6.5	10.8	6.0
7	4.2	7	9.2
TOTAL	**25.8**	**43.1**	**58.5**

4.4 Lining system

4.4.1 Description

The purpose of a landfill lining system is to contain any leachate within the landfill and prevent it from entering the underlying soils or groundwater. It provides a low permeability containment system on which leachate is collected and removed from the landfill.

For a Class 1 landfill, as proposed for Auckland Regional Landfill, the Technical Guidelines for Disposal to Land[1] describe the following two lining systems, comprising from top to bottom:

Type 1 lining system

- Leachate drainage material, with underlying cushion geotextile to protect the geomembrane;
- 1.5 mm HDPE geomembrane;
- 600 mm compacted clay with a coefficient of permeability $k < 1 \times 10^{-9}$ m/s.

Or

Type 2 lining system

- Leachate drainage material, with underlying cushion geotextile to protect the geomembrane;
- 1.5 mm HDPE geomembrane;
- Geosynthetic clay liner (GCL);
- 600 mm compacted clay with a coefficient of permeability $k < 1 \times 10^{-8}$ m/s.

These two lining systems are considered to be equivalent to each other, and either option will be used.

The geotechnical report (Technical Report B, Volume 2) and hydrogeological assessment (Technical Report E, Volume 2) identify that the underlying geology comprises areas of fractured rock and some areas where the underlying material is reasonably massive and competent sandstone/siltstone. The Technical Guidelines require that where a landfill is developed over fractured rock the design should incorporate a higher level of engineered containment. For the Auckland Regional Landfill this higher level of engineered containment will be provided by:

- Where the bottom of the lining system is less than 2 m above fractured rock the rock will be sub-excavated by 2 m and replaced with compacted soil excavated from site to provide an additional attenuation layer.

A “fluff layer” of selected waste is placed immediately above the lining system. This layer is usually from the household street collection or otherwise carefully selected waste to contain no large or bulky items and no strong chemical contaminants that may affect the lining or leachate collection system. This offers further protection to the lining system.

The proposed lining system is shown on Drawing ENG-20. All components of the lining system work together to contain leachate within the landfill and prevent leachate seepage. The combined system functions as follows:

- For there to be any leakage through a lining system there has to be a driving head (depth) of leachate. An effective drainage system above the main containment layers drains the leachate away before a significant depth of leachate can form above the containment layers, thus limiting the potential for any leakage;

[1] Technical Guidelines for Disposal to Land, Waste Management Institute New Zealand, August 2018

- The primary containment layer is the HDPE geomembrane. The individual sheets of the material are welded together using a robust welding system, with all welds tested for potential leaks. The HDPE geomembrane is essentially impermeable. There is low risk that the sheet is damaged during construction. However, this is mitigated by strict construction quality assurance (QA) procedures;
- The HDPE geomembrane is in intimate contact with the underlying clay layer, except for isolated wrinkles. Should there be a defect (damage) in the HDPE geomembrane, the potential seepage through this defect is controlled by the contact between the HDPE geomembrane and the underlying low permeability clay thereby significantly restricting leakage that can occur. If the underlying layer is a GCL then the bentonite (a very low permeability natural clay material in the GCL) swells when it becomes wet, filling the space between the HDPE geomembrane and the underlying clay layer, thereby limiting potential seepage even further.

Any potential seepage through a defect then has to flow through the GCL and/or 600 mm of a compacted clay liner before it flows out of the lining system. The time of travel through the system varies between 6 and 20 years depending on the actual permeability achieved for the compacted clay (six years for GCL plus clay with a permeability $k = 1 \times 10^{-8}$ m/s and 20 years for clay only with a permeability $k = 10^{-9}$ m/s). During this slow travel time contaminants in the leachate adhere to the clay particles and are removed from the liquid that may eventually seep from the bottom of the liner system, thereby significantly reducing the contaminant concentration.

Methods have been developed for calculating leakage through defects in a composite lining system making due allowance for the contact between the HDPE geomembrane and the underlying clay or GCL. Field and laboratory measurements of actual leakage through different lining system have been undertaken by Rowe at al, and the results are shown in Figure 4.1 below. The figure clearly shows the incremental benefit between a geomembrane (GM) or clay (CCL) liner alone versus a composite lining system in the bottom two examples.

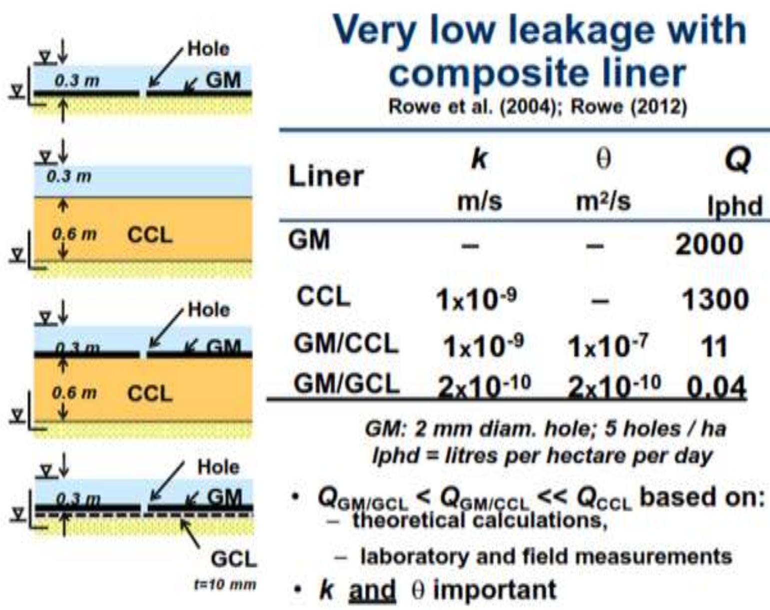

Figure 4.1: Laboratory and field measurements of liner seepage through known defects.

Soils investigations undertaken to date indicate that suitable clay soils are generally available on site, both within the general footprint area and elsewhere on the wider site, to meet the compacted clay liner objectives. The availability of these materials for the lining system construction will depend on:

- The amount of disturbance to potential low permeability soil layers near the surface during logging and stump removal operations;
- The degree of contamination of near surface clay soils by pine tree roots;
- The ability to stockpile low permeability soils excavated from the footprint for later use.

In relation to root contamination, all major roots will be removed from soils to be used for the lining system. However, where minor roots remain, provided the material otherwise meets permeability requirements, the soil may be used but only where a Type 2 lining system (with both GCL and geomembrane) is adopted.

The expected in-service life of the 1.5 mm HDPE geomembrane is expected to be well in excess of 300 years. This estimate is based on the expected temperature over the operating life of 20 to 30^{o} C and an extrapolation of the conservative values quoted by the Geosynthetic Institute[2] based on tests carried out by Drexel University in Pennsylvania. It includes the fact that the life is extended by the presence of a GCL under the HDPE geomembrane[3].

4.4.2 Lining system potential leakage

While every attempt is made to avoid leakage of leachate from a landfill, some leakage may occur through defects that may be present in the lining system, from either the manufacturing process or installation. The potential for defects is minimised by having good QA programmes in place, both for geosynthetic liner manufacture and for lining system construction. These are described in Section 4.4.3. However, to assess potential effects, it is standard international practice to assume that some defects will be present. In the case of the Auckland Regional Landfill we have assumed that a high level of construction QA will be provided, resulting in no more than the following assumed defects[4] in the geomembrane:

- Two manufacturing defects per ha (pinholes 1 mm in diameter);
- Four installation defects per hectare (1 cm^2 in area).

Potential leakage through the lining system (assuming the above defects) has been modelled using HELP (Hydrologic Evaluation of Landfill Performance, version 3.07). This is a model developed by the USEPA and widely used in landfill design to calculate leachate quantities and potential seepage/leakage from a landfill. It is a two dimensional water balance model that considers precipitation, runoff, water storage, evapotranspiration, and flow through the various layers of a landfill.

For the case of the Auckland Regional Landfill, the model has been based on the following:

- 50 years of simulated climatic data generated from average data from the nearest virtual weather station;
- Capping, refuse and lining system layers modelled generally as described for the landfill design. HELP will not allow two geosynthetic layers to be modelled together, i.e. it will not model a geomembrane overlying a GCL. Therefore, the base low permeability lining components have been modelled as geomembrane overlying clay with $k = 1 \times 10^{-9}$ m/s. The inclusion of a GCL, which contains bentonite, means that the actual leakage through any defect will be less than calculated as the bentonite swells when wet thus minimising any potential leakage through any hole that may be present in the geomembrane. In other words,

[2] GRI White Paper #6: Geomembrane Lifetime Prediction. Robert M Koerner, Y Grace Hsuan and George R Koerner, Geosynthetic Institute, February 2011.
[3] Long-term Performance and Lifetime Prediction of Geosynthetics, Y G Hsuan, H F Schroeder, K Rowe, et al, 2008.
[4] The Hydrologic Evaluation of Landfill Performance (HELP) Model, Users Guide for Version 3, Schroeder et al, USEPA, Sept 1994.

the model will over-estimate the likely leakage that would actually be expected to occur if the assumed defects were present;

- An intermediate cover layer has been assumed to be constructed from 600 mm of soil with a permeability of k = 1 x 10^{-7} m/s overlain by 100 mm of topsoil like material. Intermediate cover typically will be placed over areas of waste where no further filling is to take place in the immediate future. In the early stages of operation, once filling has progressed to final levels for a particular stage, this could represent almost 100 % of the cover for that stage;
- The geomembrane in the base lining system is assumed to have the defects described above and "good" placement quality. This assumes good field placement with a well-prepared smooth surface and geomembrane wrinkle control to provide good contact between the geomembrane and the underlying surface.

The outputs from the HELP modelling are included in Appendix A.

The results show little difference in leachate generation between the final capping system and an intermediate capping layer, so only results for the final capping system are reported. The HELP modelling estimates that, on average at full footprint extent and height, the potential seepage through the lining system is 2.63 m^3/year. This is equivalent to 7.2 L/day. The seepage for the maximum year during the 50 years of simulation is 3.01 m^3/year or an average of 8.2 L/day.

The analysis to predict potential leachate leakage assumes that the leachate is free to flow out of any defect. In practice, as this is a valley landfill and, particularly in the long term if/when the subsoil drainage system (ref Section 4.8) is sealed, it is likely that the groundwater level outside of the landfill will rise above the landfill lining system, at least over part of the landfill. In this case there will be an inward hydraulic gradient in these areas so that no leachate could leak out of any defect in such areas. The above estimates of potential leakage are thus conservative over the long term.

4.4.3 Construction quality assurance

Because of the importance of the lining system to the overall environmental performance of the landfill, as outlined above the lining system will be installed with a high standard of QA, typically undertaken by a party independent from the lining installer, contractor or landfill operator. The purpose of the QA process is to provide reliability that the lining system has been installed with no manufacturing or construction defects that may result in subsequent leakage.

The construction specification will specify the standards to be achieved and the quality control testing required by the contractor to demonstrate compliance with the specification. The QA process comprises an oversight of the testing undertaken by the contractor, regular or continuous observation of lining system placement and testing, and a review of all quality control documentation produced by the supplier and contractor.

Items that would be observed and reviewed as part of the QA process would include:

- All specified manufacturing QA documentation and/or independent testing of the geosynthetic materials supplied;
- All compaction testing associated with installing the clay liner (strength, density, moisture content, air voids);
- Permeability testing of the placed clay layer;
- Thickness of the layer;
- Approval of the clay surface for placing any geosynthetic lining components;
- Approval of the geosynthetic liner placement methodology and panel layout;
- Observation of placing, welding and testing of geosynthetic lining components to include:

- – Shear and peel testing of test weld samples at the commencement of each day;
- – Shear and peel testing of destructive test samples;
- – Air pressure testing of all dual track fusion welds;
- – Vacuum box or spark testing of all extrusion welds;
- – Visual inspection of the completed surface;
- Review of all construction records;
- Observation of placement of aggregate above the geosynthetic liner.

On completion, a report is prepared to include all of the test results, a description of the observations undertaken and certification that the lining system had been installed in accordance with the specification. This report would be submitted to an independent peer review panel (PRP) who would make recommendations to Auckland Council. Auckland Council would approve the liner placement/cell construction prior to any waste being placed in that cell.

4.5 Leachate collection

Leachate is the liquid produced when rain water percolates through the waste to the landfill lining system, collecting dissolved and/or suspended matter from the waste as it passes through. A landfill is managed to minimise the volume of leachate that is produced. This is achieved by:

- Minimising the size of the active tip area where waste is exposed to rainfall;
- Covering areas with intermediate or final cover as soon as is practicable so that as much water as possible is shed into the stormwater collection system and minimising percolation of water through these layers into the underlying waste;
- Providing well managed stormwater systems to separate all stormwater flow from areas where waste is placed, and ensuring all site stormwater is diverted away from rubbish.

All stormwater that comes into contact with waste will be treated as leachate and will not be discharged to the stormwater system.

Leachate generated within the landfill will flow to the leachate collection system at the base of the landfill from where it will be removed for treatment and disposal (refer Section 6.4).

The leachate collection system will comprise:

1 All landfill/lining system surfaces having a grade of no less than 2 % falling to leachate collection drains;

2 A high permeability aggregate layer on the floor areas of the landfill to collect leachate and direct it to the main collector drains. This will be a poorly graded (uniform size) aggregate nominally 20 mm particle size (or larger), in a layer with a minimum thickness of 300 mm;

3 A leachate drainage layer/liner protection layer on the side slopes to convey leachate to leachate collection pipes on the floor and bench areas. This layer can typically have a lower permeability than the floor leachate drainage layer due to the higher hydraulic gradient available on the slopes. Typically this layer may have a smaller particle size than the floor drainage layer and would have a permeability, k, in the order of 1×10^{-3} m/s. The aggregate will have sufficient durability for the life of the landfill in a static setting;

4 A primary leachate collection drain at the centre of the floor. This would comprise a perforated leachate collection pipe sized for the expected maximum leachate flows, with some redundancy. It would be surrounded by a coarse aggregate layer, typically with a 40 mm minimum particle size;

5 Leachate collection drains at the toe of all slopes, on the benches and on the floor, detailed similarly to the primary leachate drain;

6 Secondary leachate pipes on floor areas (if required) so that the drainage path through the drainage layer does not result in excessive hydraulic head over the liner system.

The system will be designed so that the leachate head on the liner does not exceed a selected target value, typically in the order of 300 mm. The leachate pipes will be HDPE PE100 for durability and strength. The minimum grade on any pipe will be 1.5 % with typically a target of 2 % minimum. A non-woven cushion geotextile will be placed beneath the aggregate layers to protect the geomembrane from puncturing as a result of the loads, i.e. weight of waste, on the aggregate.

Provision will be made for access to the ends of leachate pipelines for cleaning the pipes. Access to the downstream ends will provide for cleaning using flushing hoses, which would have a practical reach limit of 100 to 200 m going up the pipe. Access to the upstream ends (where practicable) will provide for launching inspection and monitoring equipment.

The leachate pipes will convey leachate to a common point at the toe of the landfill. Leachate will be pumped from this point out of the landfill to a leachate storage facility. Pumping is preferred rather than a gravity discharge because:

- It avoids a pipe penetration through the lining system on the toe bund;
- It allows greater options for storage of leachate, allowing for pond/tank water levels to be above the base of the landfill.

A second layer of HDPE geomembrane will be installed beneath any leachate sumps within the landfill for additional security against potential leakage.

The general layout of the proposed leachate collection system is shown on Drawing ENG-14.

4.6 Final cap

The purpose of the final cap on the landfill surface is:

- To control the seepage of water into the waste, thereby minimising leachate generation;
- To minimise the escape of LFG;
- To provide a separation between the waste and any future uses for the site;
- To provide a suitable growth medium for the covering vegetation, typically grass or shallow rooting shrubs. (Note: deep rooting trees will compromise the integrity of the landfill cap and, if large trees are proposed, special provision needs to be made for thickening the cap in localised areas for such planting).

Typically, landfill cap systems that are used throughout the world comprise one of two overall types:

- Barrier caps; or
- Evapo-transpirative caps (ET caps)

ET caps can provide an effective solution in low rainfall/arid climates. However, they are unlikely to be effective for the moderately high rainfall experienced in the Wayby Valley area and are not considered further.

Barrier caps generally fall into two categories. Those based on a low permeability soil layer and those incorporating a geomembrane or geosynthetic clay liner. The latter essentially excludes water from entering the landfill, resulting in a “dry tomb” landfill with little ongoing degradation of the waste. This approach is not commonly adopted in New Zealand, the preference being to allow some water to enter the landfill, at a controlled rate, to allow for ongoing biodegradation of the waste so that, ultimately, the waste will become benign. This approach has been adopted for the Auckland Regional Landfill.

The Technical Guidelines for Disposal to Land provide two options for a final cover design based on a soil barrier layer. These are described as, from top to bottom:

Minimum:

- 150 mm topsoil;
- 600 mm compacted soil ($k < 1 \times 10^{-7}$ m/s);
- Intermediate cover.

or

Enhanced minimum:

- 100 to 150 mm topsoil;
- 300 to 450 mm growth layer;
- 600 to 1000 mm compacted soil layer ($k < 1 \times 10^{-7}$ m/s).

WMNZ has a preference for the final capping layer to be approximately 2 m thick, to provide an effective barrier and to provide adequate thickness for a range of plantings on the final cap surface.

It is our experience, based on modelling of a range of cap types, that better cap performance is achieved with a reasonable thickness of higher porosity soil placed above the barrier layer. This allows for the storage of water entering the capping system and later evapotranspiration when conditions allow, resulting in less overall seepage through the capping system.

To meet these requirements the following final capping system is proposed for the Auckland Regional Landfill, from top to bottom:

- 100 to 150 mm topsoil;
- 450 to 900 mm subsoil/growth layer;
- 800 mm low permeability compacted soil 'barrier' layer ($k < 1 \times 10^{-7}$ m/s);
- 150 to 600 mm intermediate and/or daily cover soil.

This is shown in Drawing ENG-20.

As described in Section 8, the soils for construction of the final capping system will generally be soils obtained from excavation within the landfill footprint, stockpiled for later use. Careful separation of soils within stockpile areas may be required to provide for suitable soils to construct both the barrier layer and the growth layer.

4.7 Stormwater control

4.7.1 Overall stormwater management

Stormwater systems are required as part of a landfill operation to ensure that:

- Suitable diversion systems are in place to separate stormwater from waste to avoid contamination. With all stormwater that comes into contact with waste being treated as leachate;
- Practicable steps are taken to minimise erosion of soil and transport of sediment from earthworks areas. This is achieved through minimising exposed soil surfaces, installing cut-off drains to minimise flow over exposed earth surfaces, installing temporary measures where practicable to minimise the transport of sediment from earthworks areas, and stabilising these areas with vegetation or by other means as soon as practicable;

- Suitable conveyance systems (channels, pipes) are in place to carry the stormwater to suitable treatment devices to remove any sediment carried with the stormwater. These systems may comprise permanent systems (e.g. perimeter channels) or temporary systems as each stage is developed;
- Adequate treatment systems are in place to remove sediment from stormwater at all stages of development and operation of the landfill.

On this site, due to the narrowness of the lower portion of the valley where the stormwater treatment systems will be constructed, it is not possible to separate “clean” stormwater from vegetated catchments from stormwater from active areas. Thus all stormwater from the site will be combined into a common system.

The treatment of stormwater is described in detail in the separate report “Stormwater and Industrial and Trade Waste Assessment” Technical Report P, Volume 2). Stormwater management within the landfill footprint is described below.

4.7.2 Landfill stormwater systems

The stormwater collection and conveyance system at the landfill is based on:

- A system of temporary stormwater drains on the landfill surface, as required to suit the stage of operation, diverting all stormwater to the landfill perimeter;
- Drains on the landfill side of the benches to collect all stormwater from landfill surfaces. The water collected in these drains will be diverted to the larger drain on the other side of the bench at intervals so that these drains do not need to be very large;
- Permanent stormwater drains on the outside of each of the landfill benches. The benches fall eastwards at a gradient of 2 % so it is likely these drains will be unlined. Near the toe of the landfill the slopes increase to up to 15 % leading down to the stormwater ponds. These steeper drains will be lined to prevent erosion. These outer drains will collect runoff from above the landfill footprint;
- All stormwater passing through multiple treatment ponds for removal of sediment prior to discharging from the site.

Filling below the first bench within Phase 1 of the landfill will effectively block drainage of the upstream valley floor. This will last for a period of approximately 5 to 6 years. During this time the following interim drainage measures are proposed:

- Construction of a temporary stormwater pipe beneath the lining system, designed for the full flow from the upstream valley to minimise the potential for flooding of landfill areas. This pipe will need to be in the order of 1,350 mm internal diameter. It will be constructed from fully welded HDPE PE100 (or similar material) to prevent leachate seepage into the pipe;
- Construction of a second smaller diameter stormwater pipe above the lining system. This will drain stormwater from construction areas above the operating cell once construction has proceeded to a level where drainage is not possible through the main drainage pipe, e.g. during construction of the lining system for the Phase 3 up-valley from the operating cell in Phase 1.

Once filling has been completed across the entire floor of the landfill to the first bench level or higher (i.e. Phase 1), the pipes beneath the landfill will become redundant because stormwater will then be conveyed along the benches above waste fill levels. The under-liner pipe will be taken out of service and both ends sealed. The stormwater pipe laid above the liner level will be converted to a leachate conveyance pipe through Phase 1 to provide some redundancy for leachate collection and removal from the eastern portion of the landfill.

Phase 2 involves filling above the first and second bench prior to Phase 3 being constructed and will thus block drainage from areas to the east of Phase 2 (i.e. within the Phase 3 footprint). A temporary pipe along each bench below Phase 2 will be required to carry stormwater from the portion of the Phase 3 bench draining towards Phase 2.

Open channel stormwater drains will be provided on the benches to divert stormwater from catchments above waste fill areas. A typical section of the bench drains is shown on Drawing ENG-21. These bench drains form the essence of the site drainage system and will divert all site stormwater to the treatment ponds.

Stormwater systems will be designed for the following events:

- Temporary systems: 10 % Annual Exceedance Probability (AEP)
- Permanent systems: 1% AEP

The general layout of proposed stormwater systems is shown on Drawings ENG-40 to 42. Details of stormwater treatment are described in the separate stormwater report (Technical Report P, Volume 2).

The pipe beneath Phase 1 of the landfill will be constructed as follows:

- The pipeline will be designed for a ten year return period storm event;
- The catchment area to the east of Stage 1 is 42.3 ha (approximately 40 % of the overall landfill catchment) giving design flows of approximately 8.7 (10 % AEP) and 14.9 m^3/s (1 % AEP);
- The required pipe diameter is 1,350 mm;
- The upstream catchment will discharge into a sediment pond for coarse sediment removal prior to discharge into the pipeline. This pond will also be designed to attenuate flows larger than the 10 % AEP. A live volume of approximately 5,300 m^3 is required for flow attenuation for the 1 % AEP event. (The storage requirement would be 3,100 m^3 if a 1,500 mm diameter pipe is used);
- The pipeline will be laid with a minimum of 1 m cover beneath the landfill basegrade level;
- The downstream invert level of the pipe will be too low to discharge into the first stormwater treatment pond. It will therefore discharge into the second pond. This is described in detail in Section 6.3;
- Geotechnical investigations indicate rock near the surface in the valley floor. It is therefore expected that the pipe trench will be founded in rock;
- For a pipe trenched in rock, with 95 m waste depth above, preliminary indications are that a 1,500 mm OD HDPE PE100 SDR 17 pipe would be suitable;
- There is welding equipment available in New Zealand capable of welding joints of up to 1,650 mm HDPE pipe.

4.8 Subsoil drainage

It is expected that there will be a number of springs and groundwater seepages exposed within the valley when excavation to basegrade levels has been completed. Springs/seepages remaining beneath a lining system can result in uplift pressures and cause a local failure of the lining system, and therefore they must be controlled and drained away.

To control groundwater beneath the landfill a network of subsoil drains will be constructed beneath the lining system to be available to drain groundwater seepage under all stages of the landfill development. However, as subsoil drains provide a potential pathway for any leachate seepage through the lining system the drains will be progressively sealed when they are no longer required, i.e. when sufficient waste fill has been placed within each filling Phase such that uplift pressures are no longer of concern. The proposed subsoil drain system will comprise:

1 A main subsoil drain laid approximately central through the main valley beneath Phase 1, and later extended beneath Phase 3. This will drain to one of the downstream stormwater ponds at a point where a gravity discharge can be achieved. This will be extended into side valleys as required, and further extended to drain any springs identified during specific investigations for each stage of development. Any additional seepages found during construction will be connected to this system. Once filling has been completed to the first 20 m bench in Phase 3 this drain can be sealed.
2 Subsoil drains will be installed beneath each of the benches to intercept groundwater seeps from construction above each bench. These will drain to the site stormwater drainage system. Again, once all liner areas above the subsoil drains feeding to a bench subsoil drain have been filled the bench subsoil drain will be sealed.
3 A subsoil drain will again be installed along the centre of the main valley for Phase 7, similar to that for Phase 1 to 3.

The subsoil drains will be installed with trench dams at regular intervals along the drain so that the gravel surround does not provide a flow path after the drains are sealed. Where possible the last 50 m of drain upstream of the outlet will be a non-perforated pipe backfilled with clay to cut off this potential flow pathway.

5 Landfill gas management

The proposed landfill gas (LFG) management system will generally incorporate the following elements:

- A system to retain LFG within the landfill site and prevent off-site migration, i.e. lining and capping systems;
- A LFG collection system comprising a network of collection wells and pipework;
- A destruction system using flaring or electricity generation (or some other means of effective combustion); and
- Monitoring to confirm the effectiveness of the system, including LFG monitoring boreholes/wells outside the waste boundary (footprint) and regular surface methane emission monitoring on capped areas.

The LFG collection system will be installed progressively as the landfill is developed. It will be based on a series of vertical wells at approximately 50 m centres, installed once the depth of waste is sufficient and extended vertically as waste filling progresses. A series of horizontal collection pipes will be extended out from the vertical wells as required to control LFG and odour emissions between the wells as filling progresses. The wells will be connected to collection pipes to convey the LFG to the renewable energy centre where the LFG will be utilised to generate electricity or destroyed in a flare. A blower installed at the renewable energy centre will apply a vacuum pressure to the well field to extract LFG from the landfill and convey it to the engines or flare. Condensate traps (knock out pots) will be installed at any low points in the pipelines to remove potential condensate build-up in the pipelines.

Around the edges of the landfill, where the depth of waste is too shallow for the installation of a full well, pin wells (small diameter shallow wells) will be installed to provide additional coverage of the LFG collection system if required.

The general layout of the LFG collection system is shown in Drawing ENG-60.

6 Leachate management

6.1.1 Leachate quantities

Leachate generation within the landfill has been estimated empirically based on leachate generation data for other WMNZ landfill sites, specifically Redvale Landfill and Whitford landfill in Auckland. Analysis of leachate generation data compared with rainfall over the life of these facilities shows:

- Redvale: Leachate generation varies approximately 2 to 6 % of rainfall with an average of just under 4 %;
- Whitford: Leachate generation from the "new" landfill at Whitford varies between 8 and 20 % of rainfall with an average of 12.2 %. Only three years from a 21 year record exceeded 15 %.

Taking a conservative approach, leachate generation from the Auckland Regional Landfill has been assumed to be 15 % of rainfall.

To derive an appropriate annual average rainfall for Auckland Regional Landfill we have used daily rainfall data from NIWA's virtual climate station network (VCSN) database. The VCSN are spaced on a 5 km grid and the data is from the climate model that generates daily rainfall surfaces based on observed rainfall at surrounding rainfall stations. Based on these data, we have adopted an annual average rainfall for Auckland Regional Landfill of 1,564 mm.

For this average rainfall the expected annual leachate generation is 234.7 mm. Based on the recorded performance at Redvale and Whitford landfills, actual generation is expected to be less than this for most of the time.

Leachate generation is a function of the landfill footprint area, and will increase as the landfill area increases. Leachate generation from the landfill at each phase of development is summarised in Table 6.1.

Table 6.1: Estimated leachate generation

Phase	Liner area (ha)	Cumulative liner area (ha)	Daily leachate volume – avg year (m^3/day)	Daily leachate volume – peak month (m^3/day)
1	9.8	9.8	63	95
2	4.4	14.2	91	137
3	6.0	20.2	130	195
4	13.2	33.4	215	322
5	9.9	43.3	278	418
6	6.0	49.3	317	476
7	9.2	58.5	376	564

From a leachate management perspective it is important to be able to manage peak flow. Keeping the volume of leachate to be managed on a daily basis to a practical minimum requires balancing storage.

Table 6.2 shows the volume of balancing storage required to balance the peak monthly flow assuming leachate management systems are based on managing the average annual flow or a flow 20 % or 30 % greater than the annual average flow. Other combinations of flow versus storage could be adopted.

Table 6.2: Storage required for handling flows greater than annual average

Phase	Storage required based on managing annual average flow (m^3)	Storage required based on handling 1.2 x annual average flow (m^3)	Storage required based on handling 1.3 x annual average flow (m^3)
1	2,600	1,100	440
2	3,700	1,500	640
3	5,300	2,100	910
4	8,700	3,500	1500
5	11,300	4,500	1950
6	12,800	5,200	2220
7	15,200	6,100	2635
Closed	5,100	2,100	880

6.1.2 Leachate quality

Leachate quality varies significantly from one landfill to another, making leachate quality difficult to predict with any degree of accuracy for a new landfill. Leachate quality is influenced by the composition, age and depth of the waste, as well as the local climate and landfill operation practices. A new landfill operating at shallow depths will firstly undergo aerobic decomposition before changing to anaerobic decomposition as depth and cover increases. The anaerobic decomposition goes through an acetogenic phase then a methanogenic phase. The percentage of the landfill decomposing methanogenically will increase, and over time will become the dominant form of decomposition.

Typically, over time as the proportion of “older” leachate increases, the strength of the leachate will decrease and particularly the ammonia concentration and the BOD/COD will decrease.

Modern day practices to remove green waste and, in some cases, food waste from the waste stream create a significant change in leachate quality and complicate predictions of leachate quality for a new landfill when referencing older leachate quality data.

The parameters of interest when considering leachate quality are typically BOD/COD, pH, ammoniacal nitrogen and colour. The leachate will also contain a wide range of dissolved metals. Depending on where the leachate is to be discharged to, or whether it is to be treated on site, there may be other parameters of concern.

Based on an assessment of typical leachate composition from landfills in NZ it is expected that the concentration of the key leachate quality parameters from the Auckland Regional Landfill could be as outlined in Table 6.3.

Table 6.3: Expected leachate quality

Parameter	Range	Average
pH	7 – 8.5	7.5
BOD_5 (mg/L)	50 – 150	120
COD (mg/L)	600 – 5,000	2,000
NH_4-N (mg/L)	300 – 1,300	1,000

During the first one to two years of operation it would be expected that the leachate quality may be different, namely:

- It is likely to be more acidic - say pH 6;
- BOD_5 may be higher – say >3,000 mg/L;
- COD may be higher – say >10,000 mg/L; and
- NH_4-N may be marginally lower.

6.1.3 Leachate management options

Leachate management options potentially available for the Auckland Regional Landfill are:

1 Discharge to a local sewerage system;
2 Treatment on site and discharge to a watercourse;
3 Treatment and discharge by irrigation outside the landfill footprint;
4 Treatment and discharge by irrigation to the landfill cap;
5 Recirculation into the landfill;
6 Evaporation of leachate using landfill gas as a heat source;
7 Cartage to an off-site treatment plant;
8 Other technologies.

Each of these options and their relevance to the Auckland Regional Landfill are described below. The potential solution could be a combination of two or more of these options.

Option 1: Disposal to a local sewerage system

The nearest community sewerage system is the Wellsford system. Wellsford has a population of under 2,000 and therefore is likely to have a sewage flow in the order of 400 to 500 m^3/d. The leachate flow from the Auckland Regional Landfill would dominate the inflow to the plant and the high ammoniacal nitrogen (NH_4-N) and chemical oxygen demand (COD) concentrations in the leachate are unlikely to be able to be managed at the existing treatment plant, or in accordance with the consents for the existing plant. Therefore, this option is unlikely to be viable and could only be considered in the context of providing a new combined wastewater treatment plant to manage both the town sewage and leachate. This is not considered further.

Option 2: Treatment and discharge to surface water

For this to be viable a high level of treatment would be required, such as reverse osmosis (RO), to remove a high percentage of the contaminants in the leachate. RO produces a significant quantity of “reject water” which has a high concentration of contaminants. This is typically returned to the landfill. This option is not considered viable given the high standard that would be required to discharge either to the Hōteo River or directly to the stream downstream of the landfill.

Option 3: Treatment and discharge to adjacent land

For this option to be viable the treatment method needs to be able to reduce the nitrogen and COD concentrations in the leachate. This can be achieved through biological processes with sequencing batch reactors (SBR) having successfully been used for leachate treatment elsewhere. Some metals will be removed through this process as well.

Following treatment the treated leachate would be irrigated to land. Investigation of the soils in the vicinity would need to be undertaken to determine a site-specific irrigation rate. However, for feasibility assessment an irrigation rate of 3 mm/day is considered appropriate requiring an irrigation area of 3.2 ha at Phase 1 and 18.8 ha at full development, assuming design for peak month flows. Greater land areas may be required based on acceptable nitrogen loading rates.

Although this option is potentially viable it is not being considered at this stage given the significant level of treatment and the land area required.

Option 4: Treatment and discharge to the landfill cap

This option is preferable to Option 3 as potential effects on groundwater are controlled by the presence of the underlying landfill lining system. For this reason a lesser degree of treatment is required and an aerated lagoon system has successfully been used at other sites (e.g. Whitford Landfill). Similar land areas would be required as for Option 3.

The nature of the development of the Auckland Regional Landfill is such that it will be a long time after commencing operation until any significant areas of final cap are available for irrigation. Irrigation could occur on intermediate capped areas but similarly it will be some time until sufficient area is available. While this may be a possible option for the future it is not a viable option when the landfill first opens.

Option 5: Recirculation into the landfill

For this option leachate is discharged directly to the waste placed in the landfill, either through a series of trenches/distribution pipes in the waste or, more commonly, by irrigating onto newly placed waste each day. Waste typically arrives at the landfill below its field moisture content so it can permanently take up moisture. Therefore, in the early period of a recirculation management approach the quantity of leachate will reduce. Flow through the waste also typically provides some treatment benefit to the leachate and will help production of landfill gas (LFG) by raising the moisture content to the optimum level for LFG production. However, in the longer term the storage capacity for leachate will reduce and more and more leachate will require managing. Therefore, although it is proposed to potentially utilise leachate recirculation techniques throughout the operating life of the landfill it is not proposed as the primary leachate treatment method.

Option 6: Evaporation

This can be achieved in a purpose built thermal leachate evaporator using LFG as the heat source. This method of leachate management has been used by WMNZ at Redvale landfill for many years. Again, this option is not available during the early period of operation of a landfill as it takes some time before decomposition of the waste changes from aerobic through to acetogenic and then methanogenic when LFG starts to be produced. It is also necessary for a reasonable depth of waste to be placed before an effective gas collection system can be installed. Overall it is likely to be at least two years before thermal evaporation could start to be implemented.

Option 7: Cartage off-site

For many sites leachate is carted off site to a municipal wastewater treatment plant or to some other treatment facility. In the context of the Auckland Regional Landfill potential off-site disposal/treatment options comprise:

- Disposal into the Auckland sewerage system, either into a suitable manhole within the sewerage network or directly to a wastewater treatment plant – assumed to be either the Whangaparaoa wastewater treatment plant or Rosedale wastewater treatment plant. Such a discharge would need to be negotiated with Watercare and would be subject to trade waste charges in addition to transportation costs;
- Redvale landfill has a leachate evaporator. This could potentially be used for managing leachate in the early stages of the operation of the Auckland Regional Landfill subject to obtaining appropriate consents. In principle, the daily leachate at Redvale will be reducing as the landfill is finally capped, and its treatment plant will have growing spare capacity right at the time when Auckland Regional Landfill needs off-site treatment.

6.1.4 Proposed leachate management

From an assessment of the above options the following approach to leachate management is proposed at the Auckland Regional Landfill:

- Leachate collected from the landfill will be pumped directly to a balancing tank at higher elevations around the landfill;
- During the early stages of operation the holding tanks (approximately 500 m^3 capacity) will be located above the office area. Leachate in these tanks can be loaded into tankers by gravity and transferred off site for disposal to an approved facility (either the Auckalnd Sewarge system,the existing Redvale leachate evaporator or other suitable option);
- As far as is practicable, leachate will be recirculated into incoming waste at the landfill to minimise the quantity that needs to be transferred off site;
- Once sufficient LFG is available at the Auckland Regional Landfill a new evaporator or new technology will be installed on site. Once sufficient capacity is available, transfer of leachate off site will cease. At this time additional balancing tanks will be installed on the Renewable Energy Centre platform.

Some pertinent details of the system are:

- Leachate will be pumped through a fixed pipe system from the toe of the landfill to an elevated storage tank. From here, leachate will gravitate to a tanker loading area at the landfill services area where the access road enters the landfill valley. This will avoid the need for tankers to access the lower portion of the landfill where access is difficult. The tanker fill area will be bunded to capture any spills and bollards placed to prevent accidental damage;
- A branch from the fixed leachate pumping line will be diverted towards the active landfill face for the leachate recirculation system;
- Leachate generation is rainfall dependent and on some days there may be little leachate produced. However, at peak times, assuming 20 m^3 truck and trailer tankers, up to 4 or 5 tanker trips will be required each day, increasing to up to eight to nine trips at Stage 2 if leachate is still being tankered at that time;
- It is proposed that the flare and LFG to energy plant be located at the western end of the ridge to the north of the site. The evaporator would be located at this same location. The evaporator would be sized for the leachate flow at full development, or multiple units installed for a total capacity to manage to flow at full development.

7 Ancillary works

7.1 Bin exchange area and weighbridge

7.1.1 Overview

A bin exchange area will be located near the entrance to the site. This allows for standardised bins to be delivered full to the site and deposited in the bin exchange area near to the road. Road haulage trucks will deposit full bins and pick up empty bins, thereby allowing for immediate departure from the site. The full bins will be taken to the working face by site haulage/tipper vehicles (mules) for emptying, and subsequent return (empty) to the bin exchange area.

Through this approach the transport of waste from source to the site is disconnected from the landfill working face operating hours. This allows the landfill to operate efficiently, and provides a high level of control over waste arrival at the working face. The system also avoids disruption back in the communities, as waste can continue to be transported from off-site Refuse Transfer Stations (RTS) after hours as well as during the rare occasions that the working face may be closed due to high winds or other weather-related reasons. With access to the working face limited to the mules and non-WMNZ customers, the working face can be kept to a minimum size. Safety is optimised at the face by avoiding the manoeuvring activity of road haulage vehicles. Transport of waste to the facility can also take place over much longer hours each day than the working face operating hours, thereby optimising truck numbers and transport efficiency. This approach also minimises the number of trips required to the site from the RTS around Auckland.

7.1.2 Bin exchange

Waste from the RTS will be transported via a 2-bin or 3-bin hook truck and trailer. Each bin will be unloaded on to the ground by the road haulage truck or by dedicated bin handling units, and picked up by a mule truck that is permanently based on-site. The mule will take bins individually to the working face, discharge the waste, and return the empty bin to the bin exchange area to be swapped for a full bin.

There are two options for bin exchange at the bin exchange area:

1 Port-style container handling machinery is used to transfer hook bin containers; or

2 A conventional hook bin roll-on / roll-off system is used.

Both options may be used over time.

Compatible bins owned by third parties will be immediately picked up by a mule and taken to the working face for emptying while the truck waits within the bin exchange area for their bins to be returned. WMNZ bins will be standardised, so the haulage truck is able to make an exchange with any available empty bins in the exchange area, without needing to wait for the mule to return the same bins.

WMNZ compactor bins may be designed to be transferred between haulage truck and mule truck directly without placing them on the ground, using a container lifter machine rather than using the vehicle’s hook lift. Use of this direct bin transfer equipment may be extended to third parties if their bins are certified. Having this capability will mean the site needs to be designed for both machine lifting and regular hook exchange.

A shelter will be erected on part of the bin exchange area so that exposed open-top bins (used only for specific waste types) can be covered and will not be exposed to weather, or produce any adverse effects such as odour or leachate.

Some waste trucks will not have the standardised bin system and provision will be made for them to go direct to the working face.

7.1.3 Proposed operations

The bin exchange area will operate 24 hours a day, seven days a week, for pre-approved vehicles. Up to 50 full bins (covered) may need to be stored overnight.

The working face will be used by mule trucks transporting hook bins from the bin exchange area to the working face and special vehicles that do not use hook bins. Some road haulage vehicles from RTS that do not use a compactor method (such as Waitakere RTS) will continue to use the working face directly. However, it is anticipated that nearly all RTS haulage will eventually be standardised using hook compactor bins.

Other examples of trucks that will go direct to the working face include tip trucks with or without tip trailers carrying contaminated soil, and refuse collection trucks from the local area which, for any reason, do not or cannot use a transfer station.

7.1.4 Design

The layout of the proposed bin exchange area is shown on Drawing ENG-31. Key elements of the design include:

- No weighbridge will be provided at the bin exchange area, thereby avoiding queuing of arriving vehicles prior to entering the bin exchange area. Automation by use of RFID[5] tags will allow for bins to be weighed at the weighbridge located on the access road near where it enters the landfill area of the site;
- Provision will be made for public vehicles inadvertently entering the site from the roundabout on SH1 to turn around and exit the site before needing to go through any barrier arm;
- A one-way circulation system within the bin exchange area for trucks undertaking the bin exchange process, with up to five road-haulage vehicles at any time, with others waiting if needed;
- Separate access into the bin exchange area is provided for mule vehicles. Exclusive access will reduce traffic conflicts;
- Overnight parking is provided for the mule fleet, some haulage trucks, and tipper trucks;
- The bin exchange area will have provision for light vehicle parking for mule truck drivers, visitors, and some administrative staff. Parking for working face staff and site management staff will be provided up the access road nearer the working face. The bin exchange area will not provide for site administration activities;
- The facilities building at the bin exchange area will contain limited facilities mainly for drivers but will include a lunch room, washrooms, and one or two staff offices;
- The platform will be constructed by a combination of fill on the lower areas of the site and cut into the slope to the east of the site. The flood level at this location is at approximately RL 32 m and the minimum level of the platform will be at least 500 mm above this. Flood levels will be confirmed during detailed design and the platform level set accordingly;
- The entire bin exchange area will be paved with asphaltic concrete, with consideration to using concrete in the heavy duty areas (bin exchange bays). The entire surface will drain to a specific stormwater treatment device, likely to be a rain garden sized in accordance with Auckland Council GD01. No waste bins will be opened in this area so treatment requirements will be similar to any paved area subject to vehicle movements;

[5] Radio frequency identification

- Lighting will be provided for night operation and may include automatic sensor lights;
- The bin exchange area will be screened from SH1 by existing trees between the area and the road, and by the construction of an earth bund adjacent to the road that will also be planted.

7.1.5 Weighbridge

An inbound and outbound weighbridge will be located on the access road prior to it entering the landfill area, with by-pass lanes for non-refuse vehicles. The weighbridge will be automated with use of RFID tags to identify loads. Provision will be made for the weighbridge to be staffed with a kiosk installed between the incoming and outgoing weighbridge. This provides for manual/visual inspection of loads entering the landfill for compliance with the Waste Acceptance Criteria (Technical Report O, Valume 2). Ample road length is available for queuing in this area should this occur.

7.2 Site roading

7.2.1 Access road

7.2.1.1 Description

The main access to the site will be from SH1 through the valley of the Southern Block, then crossing the ridge into the landfill located in the Eastern Block. The length of the access road is just under 2 km. The road has been located along the more gently sloping southern side of the valley, avoiding the identified Significant Ecological Area (SEA) at the western end and the identified Natural Stream Management Area (NSMA) through the base of the valley. Towards the eastern end of the valley the road crosses the stream to rise to the ridge on the southern side of the valley to enter the landfill valley at approximately RL 140 m. A minor portion of the fill required for this crossing encroaches into the NSMA. The road bridges over the Waiteraire Stream near the entrance to the site.

This road will be used by all waste vehicles accessing the landfill, mostly the specialised waste haul vehicles (mules) but also some road haulage vehicles that will directly access the working face. It will also be used by all other landfill traffic, including operations staff, material deliveries, etc. and for all construction traffic entering the site. During logging operations by Matariki Forests, it will also be used by logging vehicles.

The road will be a two-way road, with 3.6 m width lanes and with a sealed surface. This provides for safety and efficiency of access for all vehicles on this primary access route onto the landfill site. The design objective is for the grade not to exceed 8 % to be suitable for hauling full waste vehicles up-hill.

The road will be constructed primarily in cut. As described in the GIR (Technical Report B, Volume 2), slopes above the road will be cut at:

- 1V:0.5H in rock
- No steeper than 1V:2H in residual soils

The concept level design shown on the drawings requires the following earthworks for the access road:

- Cut: 750,000 m^3
- Fill: 152,000 m^3

Optimisation of the design during detailed design will attempt to provide a better balance between cut and fill.

The GIR describes a large historic landslide feature within the Southern Block near the western end of the access road. Construction of the road is likely to require excavation into or near the toe of this landslide feature. Further specific investigation and design will be required during the detailed design for the road and some ground improvement work may be required to stabilise this area. Such works may include installation of subsoil drains and/or slide surface shear keys.

7.2.1.2 Stream crossings

Waiteraire Stream crossing (bridge)

A bridge is proposed where the access road crosses the Waiteraire Stream near SH1. An option for installing a culvert at this location was considered. However, from consideration of the presence of potentially liquefiable soils, flood levels, flood flow rates, fish passage, and construction footprint it was considered that a bridge was preferred at this location.

The bridge will be designed in accordance with the Bridge Manual (SP/M/022), NZTA. It is currently proposed as a 3-span bridge, two lanes wide, with spans between 20 and 25 m to provide for crossing the stream with minimal effect. The Bridge Manual requires that:

- Bridges of high importance be designed for floodwater actions associated with a 1 % AEP event, "normal" bridges for a 2 % AEP event and minor bridges for a 4 % AEP event;
- Future climate change impacts shall be taken into account in the design;
- State highway waterway crossings shall pass design floods without disruption to traffic. To meet this requirement a freeboard depth of 0.6 m shall be provided from the predicted flood level to the underside of the superstructure or, where there is a possibility that large trees may be carried down the waterway, the freeboard shall be 1.2 m.

Because of the importance of the bridge to the continued operation of the landfill WMNZ has elected to adopt state highway standards for the design. Therefore, at this concept level, the following applies:

- The expected flood level at the crossing is expected to be at approximately RL32 m based on the Auckland Council GIS. The flooding is caused by backwater effects from flooding in the Hōteo River rather than from flow in the Waiteraire Stream itself;
- A freeboard of 600 mm above this level will apply;
- A preliminary assessment of the bridge superstructure is that it will have a height of between 1.4 and 1.6 m;
- On this basis, the road level of the bridge is set at RL34.2 m;
- The actual flood level and the likely effect of climate change, and the actual depth of the bridge superstructure will be determined as part of detailed design and levels adjusted accordingly.

Tributary crossing (culvert)

As described above, the road also crosses the stream within the Southern Block near the eastern extent of the valley. It is proposed that this stream crossing be constructed as an earth fill with a culvert to maintain stream flow. The details of this culvert are:

- Upstream catchment area: 14.3 ha
- Design flow (1 % AEP): 5.08 m^3/s
- Culvert gradient: 9.5 %
- Culvert length: 104 m
- Culvert diameter: 1200 mm

A plan of the main access road is shown on Drawing ENG-30, with a long section and cross sections on subsequent drawings. The above culvert is identified as culvert No 8 on the plan and in the table below.

7.2.1.3 Access road stormwater

Stormwater management along the length of the road will comprise:

- Cut-off drains at the top of cut slopes (where required) to divert water to specific discharge locations at existing flow paths;
- Armoured channel or pipe down any cut slope to carry flow from existing flow paths to culverts under the access road;
- Inlet manhole at the location of each existing flow path to divert flow into culverts beneath the access road to discharge to the existing flow path on the downstream side of the road;
- Small collection drain on the upslope side of the access road to collect any runoff from the slopes above, discharging into the inlet manholes described above;
- The road surface will have a uniform cross-fall to shed all stormwater to a channel on the down-slope side of the road;
- Low flows from the road drain will be discharged for treatment through a series of filter strips;
- Overflows from the filter strip inlet will discharge to inlets to the culverts under the road to discharge to existing flow paths.

The general layout of this system is shown on drawings ENG-44 and 45 and the treatment is described in more detail in the stormwater report (Technical Report P, Volume 2). The culverts are scheduled in Table 7.1.

Table 7.1: Access road stormwater culverts

ID	Catchment (ha)	Flow (m3/s)	Slope (%)	Length (m)	Diameter (mm)
1	1.6	0.6	2.8	24.6	600
2	3.65	1.35	1.8	22.9	900
3	2.7	1.0	1.25	36.1	900
4	0.95	0.35	4.8	33.6	450
5	1.83	0.68	2.75	24.3	750
6	2.25	0.85	3.47	25.9	750
7	3.6	1.35	1.05	23.9	900
8	14.3	5.08	9.5	104	1200

7.2.2 Perimeter road

A road will be constructed at the top of the landfill basegrade slopes around the landfill perimeter as follows:

- Along the southern side of the landfill, the road will be constructed for two-way traffic and may be used for waste vehicles delivering waste to the landfill later in the landfill life. This two-way section of road will start in the general area of where the access road enters the main landfill valley;
- Along the eastern end, the road will be too steep for refuse or other road vehicles and will be used for maintenance purposes by 4-wheel drive or tracked vehicles. Access to place waste will make use of temporary roads on the landfill surface;

- Along the northern side, the road follows the ridge between Valley 1 and Valley 2, but the location of the existing road on this ridge will be modified to suit construction of the basegrades for the landfill in Valley 1. This road will be used for access to the flare and renewable energy centre, for general maintenance and by logging trucks hauling logs from the skid site at the western extent of this ridge.

The perimeter road will typically be a gravel road.

The perimeter road will be progressively constructed as the landfill basegrade formation progresses to the upper extent of the footprint. During the early phases of operation the landfill benches will form the landfill perimeter.

Earthworks volumes for the perimeter roads are included in the overall landfill earthworks volumes shown in Section 7.2.1, Table 7.1.

Stormwater beside the perimeter road will be managed in the same manner as stormwater on the landfill benches, with an open channel drain on the outside of the road diverting stormwater to the stormwater treatment ponds.

7.2.3 Stage access

The access roads described in this section are shown on Drawing ENG-01, 02 and 03.

Landfilling will occur below the first 20 m bench for the first phase of operation of the landfill and then continue at the eastern area of the landfill for some time. To provide access for filling and operation over the life if the landfill the following is proposed:

- A road will be constructed from the main access road where it first enters the site to connect with the first bench near the western extent of Phase 1. This will provide the main access route for waste vehicles entering the site. Access can continue over the waste fill or benches for filling at higher levels (Phase 2) or for subsequent phase located to the east of Phase 1;
- A side road from the main access road will be constructed along the side of the valley to the south west of the proposed office location, descending approximately 10 m below the ridge before cutting through the ridge to an access road that will join the landfill footprint at the first 20 m bench. The cut through the ridge will be crossed by a bridge or, more likely, a cut and cover tunnel to allow continuation of access along the existing forestry road on the ridge. This road will also connect to the road leading to Stockpile 1 and the clay borrow area. All sections of this road will be no steeper than 1V:10H. The road can be used by construction vehicles and could be used by waste vehicles when/if required;
- As landfilling progresses, access for waste vehicles will be constructed as required over the landfill surface.

The site roads will typically be gravel roads. WMNZ may consider sealing portions of road that will be used for extended periods.

The GIR notes a large historic landslide feature in the area below the office ridge towards the landfill. This will be buttressed to some extent by the formation of the landfill. However, the road down to the first 20 m bench crosses this landslide. Further specific investigation and design will be required during the detailed design for the road and some ground improvement work may be required to stabilise this area. Such works may include installation of subsoil drains and/or slide surface shear keys.

7.2.4 General site roading

In addition to the roads described above, the following site roading is proposed:

- A road from the "tunnel" beneath the office ridge to follow the ridge to the west to access Stockpile 1 and the clay borrow/stockpile. This road will also be used by forestry vehicles in future for logging in this area. This road will be located as far as possible to avoid any trees along this ridge line;
- The existing forestry road along the ridge to the south of the landfill will be kept continuously operational, mainly for forestry vehicles. Some changes to this road may be required where affected by landfill construction;
- A road off the existing forestry road (southern ridge) down to Stockpile 2;
- Additional roads that may be required from time to time for construction or operation activities.

7.3 Site facilities

7.3.1 Renewable energy centre

The renewable energy centre will be located on the ridge to the north of Valley. The platform constructed for this purpose will contain:

- LFG flare(s);
- Electricity generators;
- Leachate evaporator(s);
- Leachate storage tanks;
- Workshop facility for engine maintenance;
- Staff facilities.

The location of the centre is shown on Drawing ENG-02.

7.3.2 Site office

A building will be provided for offices for staff responsible for the operation of the landfill. It is anticipated that this would comprise:

- Four offices for landfill manager, construction manager, and others (four staff);
- One office for admin and accounts staff (five staff);
- One office for laboratory, safety and compliance personnel (three staff);
- One meeting room;
- One Lunch room/kitchen;
- One Toilet facilities.

This would be located in a single storey building of approximately 200 m^2 (say 25 x 8 m).

Parking will be provided adjacent to this building for 20 cars (staff and visitors).

The precise location for the building will be confirmed as part of detailed design. However, it is currently planned to be located on the ridge line above the landfill in the general area where the access road enters the landfill valley. This location is shown on Drawing ENG-02.

7.3.3 Workshop and staff amenities

A workshop will be provided for plant and general maintenance. This will be a building with a footprint of approximately 250 m^2 (25 x 10 m).

Hardstand area for plant of approximately 1,000 m^2 will be provided outside the building.

Amenities would accommodate:

- Two offices for supervisors (two staff)
- One lunchroom/ kitchen (ten staff)
- Two toilets, showers and locker facilities

It is currently proposed that this building will be constructed beside the access road to the lower area of the site as shown on Drawing ENG-02.

Additional small buildings will also be provided in this general area to house small plant, equipment, etc.

7.3.4 Gas plant staff amenities

Buildings at the flare site would include:

- Three offices for supervisors and gas plant operators (eight staff);
- One toilet block, showers and locker facilities.

7.3.5 Wheel wash

A wheel wash will be provided in the general area where the access road enters the landfill valley for cleaning the wheels of all vehicles leaving the site. The wheel wash will comprise as a minimum a ramp into a flooded basin with rumble bars through which vehicles drive, followed by a drip catcher pad at the outlet. It may also include fixed water jets and/or a hand held water blaster for manual cleaning of vehicles.

Sediment from the wheel wash will be removed from time to time by front-end loader and placed on the ground on the landfill to dry and/or be used as cover. Overflows from the wheel wash will be diverted to a sediment pond adjacent to the wheel wash for settling of any sediment. Water will be pumped from this pond to the wheelwash. Discharges from this sediment pond will flow into the landfill stormwater system and will pass through the landfill stormwater treatment system.

7.4 Water supply

7.4.1 Potable water supply

A potable supply will be provided for staff facilities and for make-up for odour suppression sprays. Water demands are typically based on 60 litres per capita per day (L/c/d) for office workers and 90 L/c/d where staff may use showers and other similar facilities. A composite rate of 70 L/c/d has been used for estimating flows at Auckland Regional Landfill.

Therefore, the expected water demand is:

60 staff (summertime) and bin truck drivers @ 70 l/c/d:	4.2 m^3/day
Odour sprays:	10 to 15 m^3/day
TOTAL (potable):	14 to 20 m^3/day

The source for potable water will be groundwater from a bore installed near the landfill facilities area at the location shown on Drawing ENG-02. Details of the bore are described in the hydrogeological report (Techncial Report E, Volume 2). The bore has been shown to have a sustainable yield in the order of 0.6 L/s which equates to 50 m^3/day, and is thus adequate to meet the potable demands described above. Water will be pumped from the bore to a tank (or multiple small tanks) located at a suitable elevation near the bore from where the supply will gravitate to the facilities on site. A pipeline will be laid along the main access road route to the facilities in the Bin Exchange Area.

Current indications are that the water is of suitable quality to not require any treatment. If shown to be required, under-sink UV disinfection units will be provided where water is used for human consumption.

This system would also be used for general water use in the office/workshop areas.

7.4.2 Dust suppression and road washing

The volume of water required for dust suppression and road washing is estimated to be in the order of 100 to 150 m^3/day.

Water for dust suppression, road washing and wheel wash will be sourced primarily from the stormwater ponds on site. When water is available from the bore supply, surplus to potable requirements, this may also be used. To maximise the use of bore water for this purpose, sufficient storage should be provided to optimise usage of the spare capacity of the bore supply.

7.4.3 Firefighting water supply

Any water required for firefighting will be drawn from the stormwater ponds and the bore supply. Additional sources may be available from other sedimentation ponds on site required from time to time, the Waiteraire Stream and the Hōteo River. These sources could be used in emergencies to fill water tankers for fire-fighting purposes. They would also be available for filling monsoon buckets carried by helicopters. No fire-fighting reticulation is proposed around the site.

7.5 Wastewater treatment and disposal

There is no reticulated sewerage system local to the site so all wastewater generated at the landfill will be treated and disposed of on site at a location near the point of generation. Wastewater will be generated from staff toilets, showers and lunch rooms on site at four separate locations:

- The bin exchange area;
- The main site office area;
- The main workshop and staff facilities area;
- The flare/renewable energy centre site.

The bin exchange area is remote from the main site and will be treated separately. The office and workshop areas are relatively close to each other and it is likely that these areas will be combined for treatment. The energy platform is relatively remote and it is proposed that a separate system be provided for this facility. However, at the detailed design stage consideration will be given to combining wastewater from this source with the office and workshop system.

The design of the wastewater systems will be in accordance with Auckland Regional Council Technical Publication No 58: On-site Wastewater Systems: Design and Management Manual (TP58).

Wastewater generation is based on 70 L/c/d (TP58 Table 6.2, increased as discussed above under water supply). Daily design wastewater volumes for each area are shown in Table 6.

Table 7.2: Wastewater volumes

Area	Design staff numbers	Design daily volume (L/d)
Bin Exchange Area	20	1,400
Site office and workshop areas	30	2,100
Energy platform	10	700

The design flow for each system falls within the permitted activity standard (E5.6.2.4) of the Auckland Unitary Plan. This allows for the discharge of treated domestic type wastewater onto land via one or up to three land application disposal systems within a site in circumstances where the systems cannot be reasonably combined, with a flow from each system not greater than 3,000 L/d.

In accordance with the permitted activity standard:

- Secondary wastewater treatment will be provided at each location;
- The land disposal will be a pressure compensating dripper irrigation system;
- 100 % reserve area is available on the site for each of the systems.

The soils on site indicate that a loading rate of 3 mm/d is applicable for the design of the disposal systems. Therefore the following primary disposal areas will apply:

- Bin exchange area: 470 m^2
- Office and workshop area: 700 m^2
- Renewable Energy Centre: 230 m^2

The general location for these primary disposal areasis shown on Drawings ENG-02 and ENG-31. In all cases there is land available immediately adjacent to the primary areas shown to accommodate a 100 % reserve area.

7.6 Services

Electricity, telecoms and data services are required at various locations on site. These will arranged with the applicable service provider.

Site telecommunications will use either mobile cell phone technology or a simplex radio system, and would involve small antennae and repeaters mounted on the site amenities buildings.

Electricity will be exported from the renewable energy centre at 11kV or 33kV by pylons and wires.

8 Landfill construction, operation and closure

8.1 Landfill construction activities

A landfill operation is effectively a long term construction project, being predominantly an earthworks and stormwater management operation. The bulk of these works are carried out progressively in conjunction with waste filling as waste is received over the life of the landfill. The activities on a landfill site fall into three general categories:

- Initial construction activities;
- Ongoing operational and phase development activities;
- Closure and aftercare activities.

Initial construction activities occur prior to the landfill accepting its first waste. On a complex site such as this the initial construction activities may be undertaken over a period of three or more construction seasons prior to the landfill accepting waste, with a construction season generally being defined as the period from October one year to May the following year.

Initial construction activities will include:

- Construction of permanent site stormwater controls downstream of the landfill and any other stormwater controls required for initial earthworks (e.g. at stockpile areas);
- Establishment of the site entrance and any works on SH1.
- Construction of the bin exchange area;
- Construction of the main site access road through the Southern Block;
- Site access roading to the first stage for landfilling and to all stockpile areas;
- Construction of the main site office area and workshop facilities;
- Formation of basegrades for Phase 1 of the landfill, construction of the toe bund, low permeability liner system and leachate collection system.

Details of construction activities and phasing of these works are described in “Sediment and Erosion Control Assessment” (Techncial Report S, Volume 2).

Operational activities include:

- Waste filling;
- Winning and placement of daily cover and intermediate cover as required. This may also include stockpiling soils close to where they may be required;
- Stormwater management and maintenance works;
- Construction of the next landfill phase and other required construction work.

During the operational period construction activities will be undertaken as required to develop the next landfill stage so that it is ready to accept waste when required. Wherever possible, soils required for operation of a stage will be taken from the footprint of the next or subsequent stages to minimise earthworks movements and the need for stockpiling of soils.

Closure activities include placing the final capping layer on completion, establishing any final landscaping and removing any facilities and infrastructure that is not required during the aftercare period, or modifying such infrastructure for the aftercare period.

Aftercare activities include maintenance of the cap and stormwater systems, management and maintenance of the leachate and LFG systems and ongoing site and environmental monitoring.

The following sections describe these activities in further detail.

8.2 Earthworks

8.2.1 General

The overall soil requirement for the operation of a landfill amounts to approximately 20% of the landfill airspace volume. Therefore, for a landfill airspace volume of approximately 25 Mm^3 a soil volume of approximately 5 Mm^3 is required for its operation. This soil is used for daily cover, intermediate cover, final cover and incidental construction works on the landfill site. This material is made available by providing for a surplus cut over fill when developing the landfill footprint, so that there is an overall soils balance over the life of the landfill. Where insufficient soil for operations is available on site it must be imported to the site. Some soil may become available for landfill operation by diverting any clean soils entering the site as waste. However, the supply of such materials cannot be relied upon. Therefore, it is planned for Auckland Regional Landfill that the operational soils be obtained from the landfill footprint.

Typically, any surplus soil from the construction of a stage or cell is carted to a stockpile on site. Soils required for the operation of a cell would be taken from the footprint of the next cell, to minimise double handling of materials and to optimise haulage distances. If there is no soil readily available from future development areas it would need to be taken from the soil stockpiles. At the end of the landfill life, soils for construction of the final cap would be taken from the soil stockpiles.

An approximate soils balance for the landfill is shown in Table 8.1. This is based on the current concept design. It is likely that actual earthworks quantities will differ once detailed design has been completed, leading into construction.

Table 8.1: Materials balance over life of landfill

Stage	Topsoil vol (m^3)	Cut vol (m^3)	Fill vol (m^3)	To daily cover (m^3)	To/from stockpile (m^3)	Cumulative stockpile volume (m^3)
Access roads					300,000	600,000
1	32,600	1,459,610	139,220	0	1,320,390	1,920,390
2	10,000	372,295	110,025	168,000	94,270	2,014,660
3	13,050	1,168,790	50,630	308,000	810,160	2,824,820
4	22,050	787,820	280,270	112,000	395,550	3,220,370
5	17,180	784,505	82,880	924,000	-222,375	2,997,995
6	5,330	232,970	8,195	602,000	-377,225	2,620,770
7	7,600	180,025	174,530	910,000	-904,505	1,716,265
Operation of final Phase				588,000	-588,000	1,128,265
Final Cap					-850,000	278,000
TOTAL	107,820	4,896,000	845,750	3,612,000		

Assumptions:

1 Liner material for each stage is available from the footprint of that stage

2 Topsoil depth of 150 mm. Topsoil is stockpiled for use on final cap

3 Daily cover is taken from footprint of subsequent cell where available

4 Unsuitables are included in the total cut volume. Unsuitables are deducted from the fill volume as this space needs re-filling.

Investigations on site have shown that the highly to moderately weathered rock is expected to be rippable with a large excavator. Such materials are expected over approximately 10 m depth from the surface. Greater depths of excavation are required in some areas for the proposed basegrade formation, particularly for the removal of a large spur near the eastern end of the valley where excavation up to 40 m depth is required. It is anticipated that blasting may be required in this area and other deep areas of excavation.

8.2.2 Site clearing

The landfill valley is currently covered in pine trees. WMNZ agreement with Matariki Forests is that these trees will be felled and removed by Matariki Forests prior to WMNZ commencing works.

It is expected that tree stumps and some “slash” will be present on site when WMNZ commences works. These will need to be cleared from the Stage 1 footprint prior to commencing the earthworks. Furthermore, all vegetation and stumps will need to be cleared from the access roads that are constructed as part of the initial site development.

Clearance and management of slash will be carried out in accordance with the permitted activity standards in the NES Plantation Forestry.

The cleared vegetation and strippings cleared during this early development will be coarsely shredded or mulched and placed on site in areas of exotic forest outside the landfill footprint, or used for erosion protection of cut slopes.

Strippings, stumps and slash removed from the site after the landfill is operational will be placed directly in the operating landfill or shredded/mulched and used elsewhere on site where there is a specific need.

8.2.3 Water management

Prior to any sediment generating works being undertaken on site the downstream sediment treatment ponds will be constructed (ref “Erosion and Sediment Control Assessment”, Technical Report S, Volume 2).

For the first phase of construction, once sediment controls are in place the main stormwater diversion pipe beneath the landfill will be installed to divert all runoff from upstream catchments past the construction area. Temporary diversions will be established within the construction area in accordance with the Erosion and Sediment Control Plan (ESCP).

For further phase development, once out of the valley floor, the primary stormwater diversion will be through surface drains on the benches around each phase of the landfill. These diversion drains will be established as early as practicable to divert stormwater to the main site ponds.

8.2.4 Topsoil stripping

Topsoil will be required for placing on the final cap for the establishment of vegetation. It is important that as much topsoil as possible be recovered during construction, to remove these weak/organic soils from the footprint and for the topsoil to be available for later use. Assuming a depth of 150 mm, there could be up to 100,000 m^3 of topsoil requiring removal. This material needs to be stockpiled for use in the cap. A potential stockpile is shown on Drawing ENG-02, and a number of smaller opportunities for stockpiling topsoil are expected to be found in side valleys above the southern footprint edge.

Topsoil will be excavated from the stockpile areas and other earthworks areas as required to be stockpiled with the balance of the topsoil, or re-used as part of the reinstatement of construction areas.

8.2.5 Unsuitables

It is likely that there are quantities of soft organic soils present in the valley floors. However, due to the steepness of the valley and rock known to be near the surface in the valley floor this is not expected to be a large quantity. If these materials were to remain in place they are likely to settle, causing unacceptable strains on the lining system. Therefore, these unsuitable soils will be removed. Depending on the nature of the materials they may be deposited in the main stockpiles or kept separate to allow them to dry or to allow a possible alternative use, for example as a growth layer in the final cap.

8.2.6 Subliner

The subliner material is the fill immediately under the lining system. The subliner layer consists of high strength controlled compacted fill, typically constructed by cutting to fill within the footprint of each cell.

In cut areas, i.e. where the lining system will be constructed directly on the exposed natural ground, surplus materials will be excavated and removed from the footprint to leave a trimmed surface at lining system subgrade level.

8.2.7 Liner soils

A 600 mm thick low permeability soil layer will be constructed above the prepared lining system subgrade (either natural ground or trimmed subliner fill) as the first of the lining system layers. Based on the site investigations to date, it is expected that much of this low permeability soil will be obtained from the footprint of each cell. As this material is likely to be near the existing ground surface, it will be carefully selected during the bulk excavation phase and stockpiled on the footprint of the next phase to be constructed, or at another convenient nearby location, for subsequent re-use for lining system construction. Any low permeability soils surplus to immediate requirements will be stockpiled in the clay borrow area (see below) for use in subsequent phases where there is a shortfall in suitable materials.

Where there are insufficient liner soils available within the footprint of a phase, suitable soils will be excavated from other areas of the site. Potential borrow areas for liner soils are:

- The base of Stockpile 1;
- The area on the neighbouring Western Block identified on Drawings ENG-01 and 02 as the main clay borrow area;
- The hillsides of the Western block around where excavation is required for road construction;
- Elsewhere on site where earthworks are required.

Liner materials excavated from the Stockpile 1 footprint will be placed on the identified clay borrow area for future use. These materials will be used prior to any further excavation of the borrow area.

8.2.8 Stockpiles

As discussed, soil stockpiles are required for:

- Surplus excavated materials, until they are needed for landfill operation or final capping;
- Low permeability (“clay”) soils;
- Topsoil;
- Unsuitables.

The location and form of the proposed stockpiles is shown on Drawings ENG-02 and 03.

Stockpiles 1 and 2

The soil cut/fill balance in Table 7 above shows that stockpiling for up to approximately 3.2 Mm^3 of soils is required. Two main stockpiling areas have been identified for this purpose:

- Stockpile 1, which is located in the Western Block, immediately to the west of the landfill footprint. This has a capacity to hold approximately 1.7 Mm^3 of soil;
- Stockpile 2, which is located at the head of the valley immediately to the south east of the landfill Valley 1. This has a capacity to hold approximately 1.3 Mm^3 of soil.

Access roads will be formed to the stockpile areas generally as shown on Drawings ENG-02 and 03.

Stockpiles will be formed as follows:

1. Sediment control ponds will be constructed downstream of the stockpile areas. These will be constructed and operated in accordance with GD 05;
2. Topsoil will be stripped from the portion of the site to be used for the works proposed;
3. Clay soils will be excavated from the stockpile area and transferred either directly to current construction or to the clay stockpile;
4. Filling with surplus excavated materials will commence, with compaction in accordance with the specification prepared as part of the detailed design, to ensure stability of the stockpiled soils;
5. Where practicable, filling will commence from the proposed final toe of the stockpile, with the front face formed and shaped as filling progresses. As soon as sufficient area is available remote from current filling works the surface of the front face will be vegetated. This will comprise covering with a layer of topsoil or other suitable growth layer and sowing grass seed, or hydroseeding the face;
6. On completion of filling at the end of each summer earthmoving season, all bare earth surfaces of the construction-related earth fills will be stabilised with grass, erosion mats or tarps.

The materials balance in Table 7 shows that the maximum stockpile requirements is a little greater than that available. Additional stockpiles will be used from time to time, typically on top of the waste surface, at a location convenient as a source for daily cover

Topsoil stockpile

The topsoil stockpile will be progressively added to as each cell is developed. This will be operated in a similar manner to the main stockpiles with appropriate sediment control in place prior to construction and revegetation as soon as is practicable after each seasonal episode of accumulation or depletion. The main topsoil stockpile will be located adjacent to the office area as shown on Drawing ENG-02

Clay borrow and stockpile area

The identified clay borrow area will be used as both a stockpile for clay soils, when surplus suitable soils are available from construction areas, and as a borrow area when there is a deficit of clay soils for liner construction.

Prior to any work in this area a sediment control pond will be constructed in accordance with GD05. Topsoil will be stripped and stockpiled nearby before any area is filled and/or excavated. Only sufficient area will be stripped as required for the activities being undertaken at that time, and the area will be progressively re-stabilised in accordance with the ESCP.

8.3 Operational activities

The operation of the landfill generally comprises receipt of the waste, placement and compaction within the landfill and covering. The operation of the landfill is described in the Assessment of Environmental Effects (Tonkin + Taylor, 2019).

8.4 Closure and aftercare activities

8.4.1 Closure Management Plan

Prior to the end of the life of the landfill a Landfill Closure Plan will be prepared to detail the activities required for closure of the landfill and during the aftercare period. In general terms, the activities required for closure and aftercare are described in the following sections.

8.4.2 Closure

8.4.2.1 Landfill cap

The final capping system will be constructed progressively after filling in any area has been brought up to final level. This work will generally comprise:

- Excavating soils from the soil stockpiles and placing in layers on the landfill cap in accordance with the design;
- Placing an upper topsoil and/or growth layer from materials stockpiled on site;
- Constructing surface contour drains to manage stormwater falling on the landfill cap, including connections to the perimeter drainage systems;
- Establishing vegetation (grass/shrubs, etc.) in accordance with an established planting plan.

On completion the stockpile sites will be graded to conform to the adjacent topography and re-vegetated. The sediment ponds downstream of the stockpiles will be removed on completion of the works, unless it is determined that they should be remain because of any established ecology.

8.4.2.2 Site stormwater systems

Work will be undertaken to ensure that all remaining stormwater systems required for the long term management of stormwater on site are in good working condition, any new works required constructed and all stormwater infrastructure no longer required is removed.

Any excess sediment will be removed from the stormwater ponds and the ponds left in a condition whereby they can operate with minimal attention.

8.4.2.3 Site facilities

All facilities not required during the landfill aftercare period will be removed. Such facilities will include:

- Bin exchange area facilities;
- Site office;
- General plant maintenance workshop;
- Removal of any leachate storage or other facilities no longer required.

It is proposed that all site roading remain in place, unless the adopted long term use for the site requires some alteration to the roading system.

8.4.3 Aftercare

Aftercare activities comprise:

- Ongoing operation and maintenance of the LFG extraction and treatment system;
- Ongoing operation and maintenance of the leachate collection, treatment and disposal system;
- Maintenance of the site stormwater systems;
- Maintenance of the landfill cap, including filling any areas that may have been subject to differential settlement, repair of any surface erosion and maintenance of vegetation as required;
- Maintenance of any remaining site infrastructure, including fences and the like;
- Ongoing environmental monitoring as required by consents;
- Any reporting required by consents;
- Responding to contingent events as set out in the Landfill Closure Plan.

9 Contingent events

A landfill is a large civil engineering project that continues over an extended period of time. As such, it may be subjected to possible extreme natural events that fall outside the range of anticipated design scenarios. Other potential “failures” could also occur. Such possible events are described below along with possible response actions.

9.1 Extreme weather

Extreme weather is considered in the context of an extreme rainfall event. Possible damage from such an event could comprise:

- Higher than expected erosion, resulting in high sediment loads to the treatment system and possible carry-over of sediment to the downstream environment;
- Erosion damage to site stormwater infrastructure;
- Flooding and/or erosion resulting in mixing of waste with stormwater;
- Erosion of surfaces not draining to the site stormwater treatment system.

In most cases the extensive stormwater treatment ponds will manage or limit potential environmental effects. After such an event, some or all of the following may be required:

- Immediately advise Auckland Council if there has been a breach of any consent condition;
- Take immediate action to prevent any discharges of waste or leachate to the stormwater system;
- Investigate whether there have been any downstream effects, and scope the extent of any remedial works that may be required;
- Repair site stormwater infrastructure, with a focus on areas that may cause further erosion and sediment transport to the ponds;
- Restore the treatment capacity of the ponds by removing excess sediment;
- Prepare a report to Auckland Council describing the event, its cause, environment effects and remedial action taken.

9.2 Earthquake

While extremely unlikely in this low seismicity zone, a large earthquake may result in:

- Instability of the permanent landfill face;
- Instability of any internal working face;
- Instability of the landfill toe bund;
- Lateral displacement of the waste pile;
- Possible excessive strains and/or rupture of the landfill lining system.

Following a significant earthquake the following should be undertaken:

- Inspect the landfill and surrounds for any visible sign of land damage, or damage to the lining system where exposed at the edge of any fill;
- If significant damage is observed, advise Auckland Council;
- If any evidence of damage is observed, scope a detailed investigation to determine the likely extent of any damage, and whether the lining system is likely to have been damaged. This investigation should also identify possible remediation works;

- Over the following months, carefully analyse groundwater monitoring results to identify any potential changes that may indicate new leakage through the lining system;
- Prepare a report to Auckland Council describing the event, its cause, environment effects and remedial action taken or to be taken.

9.3 Pipe blockage

Blockage of the main pipe beneath the landfill could result in water ponding against the landfill, additional leachate production and possibly a leachate spill. Blockage of other minor pipes or channels is likely to result in water flowing overland around the blockage and possible additional erosion. The potential effects of these events are generally described under “Extreme weather” and “Leachate spill”. As for all other contingent events it would be necessary to investigate appropriate remedial options and implement these as soon as is practicable.

9.4 Leachate spill

A leachate spill could occur as a result of:

1. A burst pipe or joint failure in the leachate rising main;
2. A leachate tank failure, with the tank contents overflowing from any bunded area;
3. Spillage at the tanker filling area, exceeding any containment in this area.

Any unintended discharge would be in the landfill stormwater catchment. Actions would include:

- Advise Auckland Council of the spill;
- Determine the extent of any discharge to the stormwater ponds and whether the concentration of any contaminants in the ponds are likely to cause any potential downstream environmental effects;
- If contaminant concentrations are unacceptable, hold any discharges from the pond system and develop a plan for emergency treatment of the contaminant(s) of concern;
- Clean up any surfaces impacted by the discharge;
- Continue to discharge from the ponds when safe;
- Prepare a report to Auckland Council describing the event, its cause, environment effects and remedial action taken or to be taken.

9.5 Landslip

A landslip in itself, depending on where it occurs, may impede operations but may not create an environmental effect. It may have an effect if the landslip occurred to:

- Impede any major stormwater infrastructure;
- Result in the exposure of waste or leachate.

Actions to be taken in these instances are generally as described in other sections.

A landslip around the landfill perimeter may impact on unfilled sections of lining system (it is unlikely to extend into the landfill where fill buttresses the slope). This becomes an operational/construction issue whereby the lining system would need to be repaired before filling in the cell could proceed.

9.6 Dam burst (sediment ponds)

A dam burst, particularly the dam for the downstream sediment pond, is likely to result in the discharge of large water flow and large quantities of sediment, with the potential for significant downstream effects. The actions to be taken include:

- Immediately advise Auckland Council of the event;
- Take immediate action to prevent any ongoing discharges of sediment into the downstream environment;
- Investigate the extent of downstream damage and effects and identify remedial works that can be undertaken;
- Prepare a report to Auckland Council describing the event, its cause, environment effects and remedial action to be taken;
- Undertake the remedial works;
- Repair the dam to reinstate the required stormwater treatment on site.

9.7 Landfill fire

The landfill management plan will have a section on how to manage landfill fires. Landfill fires fall into two general categories: surface (associated with incoming waste and underground fires. Deliberate fires are not a part of landfill operation and management procedures are put in place to minimise the risk of landfill fires.

Surface fires occur in recently placed waste. Typical causes of such fires include:

- A heat source within the incoming waste (e.g. hot embers);
- Spontaneous combustion of materials in the landfill when certain materials are mixed;
- Fire caused by human error or during maintenance work (smoking sparks from vehicles, etc);
- A source external to the landfill (e.g. an adjacent forest fire).

Surface fires can be managed by:

- Covering with soil to eliminate oxygen (soil stockpiles will generally be available adjacent to the working face resulting from cover soil stripped off at the beginning of each day, or stockpiles of fresh cover soil waiting to be used);
- Application of water (sourced from water tanks and ponds on site;
- Fire-fighting foams

In all cases the Fire Service will be called to manage the fighting of surface fires.

Undergorund fires occur deep below the surface and are generally caused by an increase in oxygen in the landfill at depth, resulting in aertobic digestion of the waste that can occur at high temperatures, with resulting hot-spots coming into contact with pockets of LFG. These can be difficult to extinguish and can result in damage to synthetic components of the lining system if close to the lining system. An underground fire is rare, particularly in landfills operated with appropriate use of daily and intermediate cover. Such fires are extinguished in a similar manner to surface fires but may also involve movement of quantities fo waste to expose the fire.

10 Alternatives considered

10.1 Landfill footprint and choice of valley

The overall landfill site identified from the site selection process contained two major valley systems. Conceptual layouts were prepared for a landfill in each of the valleys, and both valleys were found to be suitable, with a similar airspace capacity available in each.

In conjunction with considering which valley to develop, possible access routes were also being considered (see below). The favoured route from that exercise was through the Southern Block. Consequently, Valley 1 was favoured for development of the landfill due to:

- Both valleys presenting similar opportunities for landfill development;
- Valley 1 being closer to the preferred access route, minimising travel distances on site and associated infrastructure construction;
- To travel to Valley 2 would have greater energy requirements due to the need to either drop into Valley 1 and climb the intervening ridge, or to travel to the head of the valley around Valley 1.

10.2 Landfill access road

Selection of a suitable access road was undertaken in two stages. The first stage identified a range of potential options at feasibility level including:

- Access from Wayby Valley Road via Wilson Road to the top of the site;
- Access off Forestry Road and Wilson Road from the north;
- Access off Spindler Road and up the valley to the site;
- Access across the Western Block;
- Access up the valley on the Southern Block.

Considerations regarding each of these options were:

- Wilson Road was determined not to be feasible due to an extremely steep portion of the road exceeding design criteria for waste vehicles. There was no practicable way to reduce the slope of this portion of road;
- Similar difficulties were present with grades on the road from the north;
- The Spindler Road option was not carried forward primarily due to this being a quiet rural road with a number of houses/properties that would have been affected by the road being at this location;
- A range of options were considered for crossing the Western Block. All of these would have been very visible from SH1 heading south from Wellsford, and from areas in Wellsford;
- The Southern Block was favoured as it provided a route that was largely out of view. Also, it was an area of existing plantation wattle forest so could be developed without impacting on any native bush.

Once the Southern Block had been selected as the preferred route three options were considered for a road alignment through the valley. All options started at the same location off SH1, then:

- One option crossed the stream to join a spur near the western end of the valley then followed the ridge to Valley 1;
- The second option followed the left bank of the stream and wound around the head of the valley without crossing the stream;

- The third option was more-or less along the alignment of the current design, crossing the stream to link with a spur near the eastern end of the valley.

The third option was selected for meeting the objective of low visibility, achieving suitable grades for waste vehicles while minimising the earthworks required and the impact on the stream.

Once the basic route had been selected it was refined as follows:

- Grades were altered to reduce some of the steeper sections to make it more suitable for waste vehicles;
- Some of the bends were removed to make it safer for the trucks using the road;
- Consideration was given to realigning the road to minimise impacts on the side valleys draining to the stream;
- The stream crossing was moved approximately 45 m to the east with the effect of reducing the length of the culvert under the fill from about 120 m to 105 m.

10.3 Stockpiles

As previously discussed, large volumes of soil need to be stockpiled over the life of the landfill, requiring relatively large areas of land. Investigation of suitable stockpile areas included the following:

- Within Valley 1 where the landfill is located: This area was considered preferable as it would be within the catchment of the overall landfill. However, the landfill itself occupies most of the useable land within the catchment. The small valleys remaining above the perimeter road do not provide sufficient volume to meet requirements;
- Within the Southern Block in the area of the access road. Again, this is a relatively narrow valley with any remaining side valleys being insufficient to meet requirements. Consideration was given to using the upper reaches of this valley, upstream of where the access road crosses the stream, for stockpiling purposes. This was discarded because of the Natural Stream Management Area downstream;
- Two sites were considered on the Western Block. The first was at the head of the valley to the north east of where the main access road enters the landfill valley, to the west of the office area. This was not pursued due to the significant wetland downstream and the potential effects on the wetland. Also, it provided only limited stockpiling capacity;
- The other site in the Western Block was the head of the valley identified as Stockpile 1. This was favoured due to the large potential volume, manageable ecological effects and proximity to the landfill footprint. It was considered that the visual effects of this site could be managed. The initial footprint of the stockpile was altered to avoid some significant vegetation near the toe of the stockpile;
- The head of the valley identified as Stockpile 2 in the adjacent valley to the south. This also provided a large volume and is close to the eastern portions of landfill development, with no visibility from off site. While it is upstream of the Sunnybrook Reserve, that is some distance downstream through plantation pine forest. Appropriate management can control the downstream effects of stockpiling;
- Within Valley 2. This valley would be suitable for all stockpiling requirements at the head of the valley. The valley is currently precluded from consideration due to commercial agreements with Matariki Forests.

10.4 Leachate management

A wide range of options were considered for leachate management. These are described in Section 6.4.3.

11 Applicability

This report has been prepared for the exclusive use of our client Waste Management NZ Ltd, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Tony Bryce	Simonne Eldridge
Technical Director - Environmental Engineering	Project Director

agbb
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Appendix A: HELP Analysis

Valley 1 Landfill

Floor areas (including benches	133,480	sq m
Side slopes (1V:3H)	451,900	sq m
Total footprint	585,380	sq m

Seepage through liner: Final Cap case - 60 m refuse

	Floor areas	Sloping areas	Total landfill (m3)	Total Landfill (L/d)
Area (ha)	13.348	45.19	58.538	
Unit seepage (mm/ha)	0.01534	0.00128		
Average annual seepage (m3)	2.048	0.578	2.626	7.195
Peak day rate	0.000086	0.000010		
Peak day seepage (L)	11.479	4.519	15.998	
Maximum year (yr 23)	0.01722	0.00158		
	2.299	0.715	3.014	8.257

Seepage rates taken from HELP modelling

Final Cap
Side slopes (1V:3H)
20% surface slope
60 m refuse depth

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**              HELP MODEL VERSION 3.07 (1 November 1997)                   **
**              DEVELOPED BY ENVIRONMENTAL LABORATORY                       **
**              USAE WATERWAYS EXPERIMENT STATION                           **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:    C:\VHelp2203\data\P106.VHP\_weather1.dat
TEMPERATURE DATA FILE:      C:\VHelp2203\data\P106.VHP\_weather2.dat
SOLAR RADIATION DATA FILE:  C:\VHelp2203\data\P106.VHP\_weather3.dat
EVAPOTRANSPIRATION DATA:    C:\VHelp2203\data\P106.VHP\_weather4.dat
SOIL AND DESIGN DATA FILE:  C:\VHelp2203\data\P106.VHP\I_385381.inp
OUTPUT DATA FILE:           C:\VHelp2203\data\P106.VHP\O_385381.prt


TIME:  15:22     DATE:  11/12/2018


******************************************************************************

    TITLE:  CASE 1-60m refuse

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    NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
           COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                              LAYER  1
                              --------

                   TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER   6
          THICKNESS                  =     10.00    CM
          POROSITY                   =      0.4530 VOL/VOL
          FIELD CAPACITY             =      0.1900 VOL/VOL
          WILTING POINT              =      0.0850 VOL/VOL
          INITIAL SOIL WATER CONTENT =      0.0900 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  =  0.720000000000E-03 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY   4.20
               FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                              LAYER  2
                              --------

                   TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER   9
          THICKNESS                  =     45.00    CM
          POROSITY                   =      0.5010 VOL/VOL
          FIELD CAPACITY             =      0.2840 VOL/VOL
          WILTING POINT              =      0.1350 VOL/VOL
          INITIAL SOIL WATER CONTENT =      0.2332 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  =  0.190000000000E-03 CM/SEC


                              LAYER  3
                              --------

                      TYPE 3 - BARRIER SOIL LINER
                      MATERIAL TEXTURE NUMBER  28
          THICKNESS                  =    140.00    CM
```

POROSITY = 0.4520 VOL/VOL
FIELD CAPACITY = 0.4110 VOL/VOL
WILTING POINT = 0.3110 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.4520 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000E-04 CM/SEC

LAYER 4

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18
THICKNESS = 6000.00 CM
POROSITY = 0.6710 VOL/VOL
FIELD CAPACITY = 0.2920 VOL/VOL
WILTING POINT = 0.0770 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.2940 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000E-02 CM/SEC

LAYER 5

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 21
THICKNESS = 30.00 CM
POROSITY = 0.3970 VOL/VOL
FIELD CAPACITY = 0.0320 VOL/VOL
WILTING POINT = 0.0130 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.0350 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000 CM/SEC
SLOPE = 33.33 PERCENT
DRAINAGE LENGTH = 50.0 METERS

LAYER 6

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35
THICKNESS = 0.10 CM
POROSITY = 0.0000 VOL/VOL
FIELD CAPACITY = 0.0000 VOL/VOL
WILTING POINT = 0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.200000000000E-12 CM/SEC
FML PINHOLE DENSITY = 2.00 HOLES/HECTARE
FML INSTALLATION DEFECTS = 4.00 HOLES/HECTARE
FML PLACEMENT QUALITY = 3 - GOOD

LAYER 7

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 15
THICKNESS = 60.00 CM
POROSITY = 0.4750 VOL/VOL
FIELD CAPACITY = 0.3780 VOL/VOL
WILTING POINT = 0.2650 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.4750 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000E-06 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
SOIL DATA BASE USING SOIL TEXTURE # 6 WITH A
GOOD STAND OF GRASS, A SURFACE SLOPE OF 20.%
AND A SLOPE LENGTH OF 50. METERS.

SCS RUNOFF CURVE NUMBER = 65.28
FRACTION OF AREA ALLOWING RUNOFF = 100.0 PERCENT
AREA PROJECTED ON HORIZONTAL PLANE = 1.0000 HECTARES
EVAPORATIVE ZONE DEPTH = 25.0 CM
INITIAL WATER IN EVAPORATIVE ZONE = 2.875 CM
UPPER LIMIT OF EVAPORATIVE STORAGE = 12.045 CM
LOWER LIMIT OF EVAPORATIVE STORAGE = 2.875 CM
INITIAL SNOW WATER = 0.000 CM
INITIAL WATER IN LAYER MATERIALS = 1868.472 CM
TOTAL INITIAL WATER = 1868.472 CM
TOTAL SUBSURFACE INFLOW = 0.00 MM/YR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
DOME PROJECT AUST

STATION LATITUDE = -35.32 DEGREES
MAXIMUM LEAF AREA INDEX = 3.00
START OF GROWING SEASON (JULIAN DATE) = 274
END OF GROWING SEASON (JULIAN DATE) = 151
EVAPORATIVE ZONE DEPTH = 25.0 CM
AVERAGE ANNUAL WIND SPEED = 14.96 KPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY = 82.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY = 86.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY = 85.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY = 79.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR DOME PROJECT AUST

NORMAL MEAN MONTHLY PRECIPITATION (MM)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
94.3	96.0	117.9	118.4	142.8	183.1
184.5	170.7	142.6	107.4	99.5	107.3

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR DOME PROJECT AUST

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES CELSIUS)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
19.4	19.7	18.6	16.5	14.3	12.3
11.4	11.8	13.1	14.4	16.0	17.9

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR DOME PROJECT AUST
AND STATION LATITUDE = -36.33 DEGREES

HEAD #1: AVERAGE HEAD ON TOP OF LAYER 3
DRAIN #1: LATERAL DRAINAGE FROM LAYER 2 (RECIRCULATION AND COLLECTION)
LEAK #1: PERCOLATION OR LEAKAGE THROUGH LAYER 3
HEAD #2: AVERAGE HEAD ON TOP OF LAYER 6
DRAIN #2: LATERAL DRAINAGE FROM LAYER 5 (RECIRCULATION AND COLLECTION)
LEAK #2: PERCOLATION OR LEAKAGE THROUGH LAYER 7

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DAILY OUTPUT FOR YEAR 1

DAY	S A	O	RAIN	RUNOFF	ET	E. ZONE	HEAD	DRAIN	LEAK	HEAD	DRAIN	LEAK

```
RUNOFF
------
  TOTALS                  0.093     0.596     2.475     1.745     1.638     4.181
                          9.817     0.944     0.913     0.715     0.021     0.083

  STD. DEVIATIONS         0.407     3.123    11.644     8.223     3.879    18.459
                         26.761     3.535     2.292     3.408     0.094     0.313

EVAPOTRANSPIRATION
------------------
  TOTALS                 69.593    62.057    68.800    47.373    26.698    22.411
                         30.905    49.620    72.027    81.154    71.985    96.197

  STD. DEVIATIONS        34.303    31.857    32.825    17.649     4.927     2.898
                          2.281     2.647     7.317    20.758    29.977    31.112

PERCOLATION/LEAKAGE THROUGH LAYER  3
------------------------------------
  TOTALS                 18.5728   31.2917   45.7599   47.5603   93.8658  112.9037
                        135.3221  127.1701   97.0119   41.7175   23.4480   21.0796

  STD. DEVIATIONS        24.7066   37.8311   48.9013   54.2996   56.2546   63.3497
                         69.7680   69.2378   54.6378   41.7217   28.1840   25.4559

LATERAL DRAINAGE COLLECTED FROM LAYER  5
----------------------------------------
  TOTALS                 87.9140   73.5868   72.3814   64.9952   53.3451   41.4088
                         39.5528   46.3159   51.7007   76.5090   88.9447   91.4082

  STD. DEVIATIONS        20.4575   22.0998   26.9778   24.9210   25.2286   18.4250
                         18.9047   24.1811   22.1104   23.3692   20.9348   20.5139

PERCOLATION/LEAKAGE THROUGH LAYER  7
------------------------------------
  TOTALS                  0.0001    0.0001    0.0001    0.0001    0.0001    0.0001
                          0.0001    0.0001    0.0001    0.0001    0.0001    0.0001

  STD. DEVIATIONS         0.0000    0.0000    0.0000    0.0000    0.0000    0.0000
                          0.0000    0.0000    0.0000    0.0000    0.0000    0.0000


-------------------------------------------------------------------------------
              AVERAGES OF MONTHLY AVERAGED DAILY HEADS (CM)
-------------------------------------------------------------------------------


DAILY AVERAGE HEAD ON TOP OF LAYER  3
-------------------------------------
  AVERAGES                0.3969    1.3140    1.9818    1.7888    3.5158    4.6607
                          7.0994    4.7706    3.3038    1.4651    0.5755    0.4719

  STD. DEVIATIONS         0.7548    2.2380    3.4898    3.0676    3.5545    5.5323
                          7.6637    4.1281    3.3279    3.2591    1.0382    0.9162

DAILY AVERAGE HEAD ON TOP OF LAYER  6
-------------------------------------
  AVERAGES                0.2735    0.2513    0.2252    0.2090    0.1660    0.1331
                          0.1231    0.1441    0.1662    0.2381    0.2860    0.2844

  STD. DEVIATIONS         0.0637    0.0752    0.0839    0.0801    0.0785    0.0592
                          0.0588    0.0752    0.0711    0.0727    0.0673    0.0638

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    AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS   1 THROUGH   50
-------------------------------------------------------------------------------
                                     MM            CU. METERS     PERCENT
                               --------------    -----------    ---------
 PRECIPITATION                  1518.37   ( 207.202)     15183.7      100.00

 RUNOFF                           23.221  ( 34.4039)       232.21       1.529

 EVAPOTRANSPIRATION              698.820  ( 62.8843)      6988.20      46.024

 PERCOLATION/LEAKAGE THROUGH   795.70344 (180.79350)     7957.034    52.40504
   LAYER  3
```

AVERAGE HEAD ON TOP OF LAYER 3	26.120 (11.612)		
LATERAL DRAINAGE COLLECTED FROM LAYER 5	788.06258 (123.51409)	7880.626	51.90181
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.00128 (0.00019)	0.013	0.00008
AVERAGE HEAD ON TOP OF LAYER 6	2.083 (0.327)		
CHANGE IN WATER STORAGE	8.267 (8.1474)	82.67	0.544

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 50 and their dates (DDDYYYY)

	(MM)	(CU. METERS)	
PRECIPITATION	173.40	1734.00000	2090014
RUNOFF	89.452	894.51945	2090014
PERCOLATION/LEAKAGE THROUGH LAYER 3	11.997142	119.97142	1830026
AVERAGE HEAD ON TOP OF LAYER 3	544.013		
DRAINAGE COLLECTED FROM LAYER 5	6.46460	64.64604	2910022
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.000010	0.00010	2910022
AVERAGE HEAD ON TOP OF LAYER 6	6.236		
MAXIMUM HEAD ON TOP OF LAYER 6	12.414		
LOCATION OF MAXIMUM HEAD IN LAYER 5 (DISTANCE FROM DRAIN)	0.0 METERS		
SNOW WATER	0.00	0.0000	0
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4818		
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1150		

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 50

LAYER	(CM)	(VOL/VOL)
1	1.8799	0.1880
2	12.6523	0.2812
3	63.2800	0.4520
4	1802.4267	0.3004
5	1.0684	0.0356

6	0.0000	0.0000
7	28.5000	0.4750

SNOW WATER 0.000

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Final Cap
Floor areas (2 percent slope)
20% surface slope
60 m refuse depth

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**                                                                          **
**                                                                          **
**             HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                **
**             HELP MODEL VERSION 3.07 (1 November 1997)                    **
**             DEVELOPED BY ENVIRONMENTAL LABORATORY                        **
**             USAE WATERWAYS EXPERIMENT STATION                            **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     C:\VHelp2203\data\P106.VHP\_weather1.dat
TEMPERATURE DATA FILE:       C:\VHelp2203\data\P106.VHP\_weather2.dat
SOLAR RADIATION DATA FILE:   C:\VHelp2203\data\P106.VHP\_weather3.dat
EVAPOTRANSPIRATION DATA:     C:\VHelp2203\data\P106.VHP\_weather4.dat
SOIL AND DESIGN DATA FILE:   C:\VHelp2203\data\P106.VHP\I_386911.inp
OUTPUT DATA FILE:            C:\VHelp2203\data\P106.VHP\O_386911.prt


TIME:  16:31     DATE:  11/12/2018


******************************************************************************

    TITLE:  CASE 2-60m refuse

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    NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
           COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                           LAYER  1
                           --------

              TYPE 1 - VERTICAL PERCOLATION LAYER
                 MATERIAL TEXTURE NUMBER   6
    THICKNESS                 =    10.00    CM
    POROSITY                  =     0.4530 VOL/VOL
    FIELD CAPACITY            =     0.1900 VOL/VOL
    WILTING POINT             =     0.0850 VOL/VOL
    INITIAL SOIL WATER CONTENT  =   0.0900 VOL/VOL
    EFFECTIVE SAT. HYD. COND.   =  0.720001612800E-03 CM/SEC
   NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  4.20
          FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                           LAYER  2
                           --------

              TYPE 1 - VERTICAL PERCOLATION LAYER
                 MATERIAL TEXTURE NUMBER   9
    THICKNESS                 =    45.00    CM
    POROSITY                  =     0.5010 VOL/VOL
    FIELD CAPACITY            =     0.2840 VOL/VOL
    WILTING POINT             =     0.1350 VOL/VOL
    INITIAL SOIL WATER CONTENT  =   0.2332 VOL/VOL
    EFFECTIVE SAT. HYD. COND.   =  0.190000000000E-03 CM/SEC


                           LAYER  3
                           --------

                 TYPE 3 - BARRIER SOIL LINER
                 MATERIAL TEXTURE NUMBER  10
    THICKNESS                 =   140.00    CM
```

POROSITY = 0.3980 VOL/VOL
FIELD CAPACITY = 0.2440 VOL/VOL
WILTING POINT = 0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.3980 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000E-04 CM/SEC

LAYER 4

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 18
THICKNESS = 6000.00 CM
POROSITY = 0.6710 VOL/VOL
FIELD CAPACITY = 0.2920 VOL/VOL
WILTING POINT = 0.0770 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.2940 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000E-02 CM/SEC

LAYER 5

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 21
THICKNESS = 30.00 CM
POROSITY = 0.3970 VOL/VOL
FIELD CAPACITY = 0.0320 VOL/VOL
WILTING POINT = 0.0130 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.0778 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000 CM/SEC
SLOPE = 2.00 PERCENT
DRAINAGE LENGTH = 50.0 METERS

LAYER 6

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35
THICKNESS = 0.10 CM
POROSITY = 0.0000 VOL/VOL
FIELD CAPACITY = 0.0000 VOL/VOL
WILTING POINT = 0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.200000000000E-12 CM/SEC
FML PINHOLE DENSITY = 2.00 HOLES/HECTARE
FML INSTALLATION DEFECTS = 4.00 HOLES/HECTARE
FML PLACEMENT QUALITY = 3 - GOOD

LAYER 7

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 15
THICKNESS = 60.00 CM
POROSITY = 0.4750 VOL/VOL
FIELD CAPACITY = 0.3780 VOL/VOL
WILTING POINT = 0.2650 VOL/VOL
INITIAL SOIL WATER CONTENT = 0.4750 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.100000000000E-06 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
SOIL DATA BASE USING SOIL TEXTURE # 6 WITH A
GOOD STAND OF GRASS, A SURFACE SLOPE OF 20.%
AND A SLOPE LENGTH OF 50. METERS.

SCS RUNOFF CURVE NUMBER = 65.28
FRACTION OF AREA ALLOWING RUNOFF = 100.0 PERCENT
AREA PROJECTED ON HORIZONTAL PLANE = 1.0000 HECTARES
EVAPORATIVE ZONE DEPTH = 25.0 CM
INITIAL WATER IN EVAPORATIVE ZONE = 2.876 CM
UPPER LIMIT OF EVAPORATIVE STORAGE = 12.045 CM
LOWER LIMIT OF EVAPORATIVE STORAGE = 2.875 CM
INITIAL SNOW WATER = 0.000 CM
INITIAL WATER IN LAYER MATERIALS = 1862.157 CM
TOTAL INITIAL WATER = 1862.157 CM
TOTAL SUBSURFACE INFLOW = 0.00 MM/YR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
DOME PROJECT AUST

STATION LATITUDE = -35.32 DEGREES
MAXIMUM LEAF AREA INDEX = 3.00
START OF GROWING SEASON (JULIAN DATE) = 274
END OF GROWING SEASON (JULIAN DATE) = 151
EVAPORATIVE ZONE DEPTH = 25.0 CM
AVERAGE ANNUAL WIND SPEED = 14.96 KPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY = 82.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY = 86.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY = 85.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY = 79.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR DOME PROJECT AUST

NORMAL MEAN MONTHLY PRECIPITATION (MM)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
94.3	96.0	117.9	118.4	142.8	183.1
184.5	170.7	142.6	107.4	99.5	107.3

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR DOME PROJECT AUST

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES CELSIUS)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
19.4	19.7	18.6	16.5	14.3	12.3
11.4	11.8	13.1	14.4	16.0	17.9

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR DOME PROJECT AUST
AND STATION LATITUDE = -36.33 DEGREES

HEAD #1: AVERAGE HEAD ON TOP OF LAYER 3
DRAIN #1: LATERAL DRAINAGE FROM LAYER 2 (RECIRCULATION AND COLLECTION)
LEAK #1: PERCOLATION OR LEAKAGE THROUGH LAYER 3
HEAD #2: AVERAGE HEAD ON TOP OF LAYER 6
DRAIN #2: LATERAL DRAINAGE FROM LAYER 5 (RECIRCULATION AND COLLECTION)
LEAK #2: PERCOLATION OR LEAKAGE THROUGH LAYER 7

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DAILY OUTPUT FOR YEAR 1

DAY	A	S O	RAIN	RUNOFF	ET	E. ZONE	HEAD	DRAIN	LEAK	HEAD	DRAIN	LEAK

RUNOFF

TOTALS	0.093	0.596	2.474	1.745	1.638	4.177
	9.853	0.976	0.913	0.715	0.021	0.083
STD. DEVIATIONS	0.407	3.124	11.644	8.223	3.880	18.457
	26.840	3.718	2.292	3.407	0.094	0.313

EVAPOTRANSPIRATION

TOTALS	69.528	62.029	68.876	47.377	26.700	22.417
	30.909	49.628	72.059	81.117	71.927	96.311
STD. DEVIATIONS	34.281	31.933	32.897	17.647	4.921	2.900
	2.286	2.645	7.263	20.786	29.866	31.091

PERCOLATION/LEAKAGE THROUGH LAYER 3

TOTALS	18.5744	31.3687	45.6799	47.5312	93.8779	112.9399
	135.2903	127.0680	97.0355	41.7309	23.4939	20.9924
STD. DEVIATIONS	24.6382	37.8494	48.9312	54.2824	56.1435	63.4707
	69.7571	69.1691	54.7502	41.7150	28.3772	25.4349

LATERAL DRAINAGE COLLECTED FROM LAYER 5

TOTALS	88.8825	74.4210	73.7471	65.1830	56.6394	42.7793
	38.9346	46.2883	50.3572	70.7772	88.9048	90.9551
STD. DEVIATIONS	19.7841	21.0722	26.5218	24.3009	23.8013	17.9780
	17.2796	22.8082	21.5323	22.1640	20.3079	20.6024

PERCOLATION/LEAKAGE THROUGH LAYER 7

TOTALS	0.0017	0.0014	0.0014	0.0013	0.0011	0.0009
	0.0008	0.0009	0.0010	0.0014	0.0017	0.0017
STD. DEVIATIONS	0.0004	0.0004	0.0005	0.0004	0.0004	0.0003
	0.0003	0.0004	0.0004	0.0004	0.0004	0.0004

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (CM)

DAILY AVERAGE HEAD ON TOP OF LAYER 3

AVERAGES	0.3976	1.3181	1.9812	1.7909	3.5090	4.6580
	7.0948	4.7584	3.2967	1.4627	0.5723	0.4749
STD. DEVIATIONS	0.7543	2.2431	3.4882	3.0704	3.5562	5.5020
	7.6508	4.1216	3.3103	3.2555	1.0383	0.9185

DAILY AVERAGE HEAD ON TOP OF LAYER 6

AVERAGES	4.1500	3.8148	3.4434	3.1449	2.6446	2.0640
	1.8179	2.1613	2.4296	3.3047	4.2895	4.2468
STD. DEVIATIONS	0.9237	1.0806	1.2383	1.1725	1.1113	0.8674
	0.8068	1.0649	1.0389	1.0349	0.9798	0.9620

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 50

	MM		CU. METERS	PERCENT
PRECIPITATION	1518.37	(207.202)	15183.7	100.00
RUNOFF	23.284	(34.4792)	232.84	1.534
EVAPOTRANSPIRATION	698.877	(63.0629)	6988.77	46.028
PERCOLATION/LEAKAGE THROUGH LAYER 3	795.58295	(180.70598)	7955.830	52.39710

AVERAGE HEAD ON TOP OF LAYER 3	26.095 (	11.612)		
LATERAL DRAINAGE COLLECTED FROM LAYER 5	787.86940	(123.76228)	7878.694	51.88909
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.01534 (	0.00226)	0.153	0.00101
AVERAGE HEAD ON TOP OF LAYER 6	31.260 (	4.914)		
CHANGE IN WATER STORAGE	8.326	(8.1973)	83.26	0.548

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 50 and their dates (DDDYYYY)

	(MM)	(CU. METERS)	
PRECIPITATION	173.40	1734.00000	2090014
RUNOFF	89.442	894.42188	2090014
PERCOLATION/LEAKAGE THROUGH LAYER 3	12.007274	120.07274	1830026
AVERAGE HEAD ON TOP OF LAYER 3	545.654		
DRAINAGE COLLECTED FROM LAYER 5	4.61362	46.13616	2920022
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.000086	0.00086	2920022
AVERAGE HEAD ON TOP OF LAYER 6	66.779		
MAXIMUM HEAD ON TOP OF LAYER 6	107.722		
LOCATION OF MAXIMUM HEAD IN LAYER 5 (DISTANCE FROM DRAIN)	9.7 METERS		
SNOW WATER	0.00	0.0000	0
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4818		
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1150		

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 50

LAYER	(CM)	(VOL/VOL)
1	1.8799	0.1880
2	12.6524	0.2812
3	55.7200	0.3980
4	1802.5355	0.3004
5	2.4972	0.0832

6	0.0000	0.0000
7	28.5000	0.4750

SNOW WATER 0.000

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