URS

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RESOURCE CONSENT APPLICATION FOR THE STAGE 4 EXTENSION OF THE SOUTHERN LANDFILL, WELLINGTON

August 2013

Absolutely
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Me Heke ki Pōneke
Wellington City Council
Wellington

Report

Southern Landfill Stage 4

Hydrogeology

AUGUST 2013

Prepared for

Wellington City Council

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Introduction

1.1 Introduction

Wellington City Council (WCC) operate Southern Landfill, a municipal solid waste landfill, located at Happy Valley, Wellington. WCC wish to extend the existing landfill from the current Stage 3 area further up Carey's Gully into the Stage 4 area. WCC has engaged URS New Zealand Ltd (URS) to assist with the landfill resource consent application.

This report presents the findings of groundwater investigations and assessments of potential effects to groundwater and has been prepared to support the application for Southern Landfill Stage 4 resource consent. This includes details of the groundwater flow regime, hydrogeological parameters and aquifer conditions, and recommended measures to be incorporated into concept and detailed design.

1.2 Background

To undertake the proposed Stage 4 extension of the Southern Landfill, WCC will need to obtain a range of resource consents from the Greater Wellington Regional Council. The applications for these resource consents will in turn be supported by a comprehensive assessment of environmental effects. This report is one a series of technical assessments that have been prepared as supporting documentation to the assessment of environmental effects.

1.3 Summary of Works Completed

The following geological and hydrogeological work items have been undertaken by URS since July 2007:

- Aerial photography and digital terrain mapping with geomorphological mapping.
- Geological mapping.
- Drilling and monitoring well installation of boreholes (BH1/1A, BH2/2A, BH3A/3B/3C, BH4A/4B and BH5).
- Hydraulic conductivity testing of monitoring wells.
- Groundwater sampling of monitoring wells BH1/1A, BH2/2A, BH3A/3B/3C and BH4A/4B.
- Stream flow measurements of Carey's Stream.
- Pumping test at borehole BH5, with water level observations in surrounding monitoring wells.

This technical report describes the findings and interpretation of these works.

Proposed Activity

2.1 Filling Sequence and Landform

Filling is proposed to commence the top of Carey's Gully and progress downwards to the interface with the existing Stage 3. Fill areas will be developed and managed as cells, with cell size based on topographical and economic considerations.

2.1.1 Landfill Footprint and Access

The proposed Stage 4 expansion footprint is shown on **Drawings C-001 and C-002**. At the southern end of the Stage 4 expansion, adjacent to the interface with Stage 3, the footprint is approximately bounded at the periphery by the RL 175 m contours. From this interface the foot print extends to the north/northwest up Carey's Gully and is generally bounded by the RL 225-235 m contours at the furthest extent up the valley. The overall extent of the landfill footprint is approximately 28 hectares.

Permanent access roads to the Stage 4 landfill are proposed at the northern and southern periphery of the landfill footprint. These roads will allow access during landfill operation and long-term access to surface water drainage swales, landfill gas lines and cap maintenance. Cut-off drains will be provided along these roads to divert storm water from the catchment above Stage 4 around the landfill area.

Access to the landfill tip face and/or the toe bund of the active cell will be provided as follows:

- Across the landfill surface, utilising a built-up aggregate surface; or
- From the perimeter access roads via roads cut into the valley side slopes to the valley floor.

Limited access may be required at the valley floor for construction and/or maintenance/monitoring of leachate and storm water collection and conveyance systems.

2.1.2 Filling and Cover

For each cell, commencing from the lowest point of the landfill area practically accessible to landfill customers, refuse shall be placed and compacted on the prepared surface of the landfill to form a layer between 2 and 5 metres high. Compaction of the refuse will be carried out in accordance with accepted good practice for sanitary land filling in order to achieve an in-situ density of at least 1.0 tonne per cubic metre. Each cell of refuse shall be shaped to shed surface water run-off to the leachate drain/gas chimney. Fill shall then be successively placed, working from side to side of the working face and compacting each layer fully before placing the next.

Daily cover material shall be provided and compacted at the end of each day over the sloping active landfilling area to a compacted depth of not less than 100 mm, and not more than an average depth 250 mm. Cover material shall be spread and compacted over the top of the advancing lift or layer of refuse to a compacted depth of not less than 200 mm, and not more than an average depth of 300 mm.

Intermediate cover is proposed during the operational phase of the project. Intermediate cover will consist of the daily cover (compacted depth of not less than 200 mm, and not more than an average depth of 300 mm), which will then be grassed. Intermediate cover will be provided at areas where further refuse placement is not scheduled for approximately one year or more.

In areas where further filling is not proposed a final cover system is proposed comprising the following:

- 300 mm coarse aggregate gas collection layer.
- 600 mm soil cover compacted to a permeability of less than 1 x 10^{-7} m/s.
- 150 mm selected fill material layer
- 150 mm topsoil layer

The surface of the compacted cover material will be contoured to promote run-off of surface water to the gas chimneys and leachate drainage system. Only those areas of fill that have been completed to the design levels, and provided with the final cover system, shall be shaped so that surface water enters the storm water drainage system.

2.1.3 Final Landform

The final landform is proposed to rise from the landfill periphery access roads at a slope of approximately 1V:4H to a ridge or small plateau at the centre of the landfill. The final landfill ridge will have splays up the main tributary valleys. In general the landform will fall from the upper reaches of the valley to match the final grade at Stage 3. The final cap surface will be provided with flat grade contour drains directing storm water runoff to the open drainage at the periphery of the landfill.

The minimum slope angle for any finished surface will be 1V to 10H to ensure the finished surface remains free draining.

2.2 Leachate Management

The leachate management system primarily comprises the following engineered elements, with each of these described in more detail below:

- Valley floor liner.
- Sidewall drainage material
- Leachate collection drain.
- Leachate pond

2.2.1 Valley Floor Liner

A liner is proposed for the valley floor, which would intercept leachate collected on the steep side walls and direct it to the leachate collection system. This liner system would also minimise transport of leachate to the relatively higher permeability colluvial material present at the bottom of the valley. The liner would comprise a 900 mm thick compacted soil layer with a permeability of 10^{-9} m/s or alternatively, a composite geomembrane/geosynthetic clay liner and compacted soil system may be provided that delivers an equivalent hydraulic and chemical containment.

The liner system would either be keyed into competent greywacke rock at the valley floor or extend to a height of approximately two vertical metres above the base of the liner along the valley slopes. Refer to **Drawings C-015** and **C-016** for indicative liner options.

Due to the potential for artesian groundwater in the valley floors a subliner drainage layer is proposed to protect the liner system and reduce groundwater intrusion into the base of the landfill. This drainage system will include a perforated pipe system beneath the liner bedded in aggregate drainage material. The subliner drainage system will report to clean water (storm water) system. However, provision will be allowed for diversion of this water to the leachate system should monitoring of the subliner drainage system indicate that it is impacted with leachate. As such this subliner drainage system will provide secondary leachate control should the liner fail.

2.2.2 Sidewall Leachate Filter/Drainage

Due to the steepness of the side walls, drainage pipework to collect leachate is not proposed. However, a screened open-graded drainage layer or inclined chimney system is proposed, designed to intercept landfill leachate and groundwater discharging from the valley side walls and convey this downslope to the leachate collection system.

The sidewall drainage concept will involve trimming of loose material from the valley sidewalls and placement of drainage material, with hydraulic conductivity in the order of 1×10^{-4} m/s, on the side walls of the landfill in wedges as the waste level rises. Based on in situ permeability testing undertaken in groundwater monitoring wells (refer **Section 4.4**) it is expected that the permeability of the valley sidewall rock will be orders of magnitude less than the filter/drainage material and as a result the head of leachate on the valley side walls will be low. As such, any notable leachate infiltration into the sidewalls is not expected and additional containment will not be required (refer **Section 4**).

Where the lateral length of sidewall slopes exceeds 100 metres intermediate benching would be provided to collect leachate. The benches would comprise a drainage aggregate layer and collection pipe system. Leachate would be drained from the bench areas through provision of piped laterals, which discharge to leachate chimneys within the refuse, and ultimately report to the leachate collection system.

The drainage system would be designed to maintain a leachate head of 300 mm or less on the sidewalls in accordance with Landfill Guidelines (CAE, 2000).

2.2.3 Leachate Collection

A leachate collection system will be provided above the liner at the valley floor. The collection system will comprise a perforated drainage pipe system bedded in coarse drainage aggregate. A 300 mm drainage/filter layer, comprising graded sand, will be provided above the drainage aggregate layer. The drainage/filter layer will terminate at the valley sidewalls and will be tied into the sidewall filter/drainage system. The coarse drainage aggregate layer will provide secondary conveyance of the leachate should the perforated pipe system become blocked or otherwise fail. Non-perforated pipe risers will be provided along the valley walls to allow for flushing of the system.

The leachate collection system will be reticulated to, and cross connected with, the existing leachate system which comprises a non-perforated pipe system within the tunnel system beneath the existing landfill. This system ultimately discharges to the tradewaste system. Access to the tunnel system will be provided as appropriate to allow for maintenance and/or repair of the leachate conveyance system.

2.2.4 Leachate Ponds

Temporary leachate storage ponds (lined) will be provided at the toe of the active landfill cell. The purpose of these ponds is to provide attenuation of storm water accumulated from fill areas (without intermediate or final cover) and seepage prior to discharge to the leachate system.

2.3 Storm water Management

2.3.1 Perimeter Drainage

The Carey's Stream catchment above the landfill footprint will be captured by the cut-off drains described above, and diverted around the periphery of the landfill before discharging to Carey's Stream downstream of the landfill. Attenuation dams would be provided at the head of the main valley and major tributary valleys above the landfill footprint.

Carey's Stream between the active landfill stage in the upper part of the valley and the existing Stage 3 will be retained but progressively removed as landfilling advances down the valley to meet Stage 3.

A tunnel exists beneath Southern Landfill and is utilised for diversion of Carey's Stream and conveyance of leachate in a separate pipeline. The tunnel commences downstream of the landfill and terminates approximately 100 m beyond the toe of Stage 3 landfill toe. The existing tunnel entry, at the toe of Stage 3, will be retained until such time as landfilling begins in the area of the tunnel entrance. The existing tunnel will be used for diversion of Carey's Stream until this time.

When landfilling begins in the area of the tunnel entrance the existing tunnel cascade and entrance will be sealed and then covered. At this point the tunnel will cease to convey surface water but will still be utilised as a conduit to convey leachate, in separate pipeline(s), to the trade waste system.

Access to the tunnel will still be required for maintenance and inspection purposes. The proposed concept provides access to the tunnel through a new vertical access shaft (approximately 100 m deep) dug through rock offline from the existing tunnel alignment and in the roadway to the west of the tunnel (refer **Drawing C-006**). A lateral connection to the existing tunnel at the existing cascade is proposed from this vertical access shaft. This access shaft and lateral connection will be provided with the contingency to allow for diversion of upstream leachate to the access shaft for pumping to the surface should the downstream tunnel fail or otherwise become unusable.

The tunnel access shaft will be commissioned before landfilling begins in the area of the tunnel entrance.

2.3.2 Cell Storm water Management

Run-off and seepage from areas of active landfilling (without final cover) will be collected and treated as leachate.

Where practical cut-off drains will be provided around active landfill areas to divert run-on storm water flows to the storm water system. Storm water run-on to active landfill areas will be managed as leachate.

Existing Environment

3.1 Site Location

The proposed site for Southern Landfill Stage 4 is located in Carey's Gully catchment area, approximately 5 km to the southwest of central Wellington. **Drawing C-001** illustrates the location of the site and proposed Stage 4 landfill.

3.2 Topography

Carey's Gully is a deeply incised valley, with a series of smaller, steep sided gullies extending over the whole valley catchment. The valley walls are steeply sloping (typically greater than 30°) and high ridges bound it to the north, east and west (250 m above ordinance datum (AOD) to greater than 350 m AOD). The lowest point on the proposed Stage 4 site (120 m AOD) is situated at the southern boundary of Carey's Gully adjacent to the Stage 3 area, where the stream enters the culvert.

Dense vegetation is present across the majority of the site, which combined with steep soil and rock slopes limits access across the site. Access roads have been cut into the eastern and western valley sides, and part of the way up Carey's Gully. Earthworks and extensive rock cut slopes are present where the Stage 4 area overlaps with the Stage 3 area and in areas where the access roads have been cut.

Topography of the proposed Stage 4 landfill area is illustrated in **Drawing C-002**.

3.3 Climate

Wellington has a temperate climate with mild daytime temperatures and infrequent frosts. The area generally tends to receive high rainfall in winter and low rainfall in summer, but is prone to high-intensity rainfall and wind, which can occur at any time of the year. Annual rainfall is typically 1240 mm around Carey's Gully.

3.4 Geology

The Stage 4 area is underlain by Rakaia Terrane (Torlesse Group) indurated sandstones and mudstones, often referred to as "greywacke". At surface, superficial deposits comprise of Makara Soils. This typically consists of brown silty sand with some gravel and clay. Thicknesses are stated to vary widely from 0.5 m to 3 m and are generally thickest in mid slope areas of the valley. Angular greywacke gravel and scree slopes are common in steeper areas and there are no superficial deposits in the steepest areas, which comprise bedrock.

Alluvium, consisting of sub-rounded gravel of sandstone, occupies a narrow strip of limited depth in the bottom of the gullies.

In the vicinity of the proposed Stage 4 expansion, the bedrock is found to comprise predominantly unweathered-slightly weathered rock, with limited weathering at surface. The weathering identified on site was typically greatest at the ridgelines and where groundwater recharge was occurring. Highly weathered rock was only occasionally observed, forming a thin band (0.0 m to 0.5 m thick) at the top of road cuttings. Where moderately weathered rock is exposed in the road cuttings along the eastern and western valley sides, with the thickness of this material observed to range from about 0.0 m to about 7.0 m. Greywacke exposed in the valley floor typically comprises slightly weathered rock.

The major structural elements in the region include the Wellington Fault, which strikes approximately northeast and reaches within 1 km of the north-eastern boundary of the site. In addition, several north striking faults (mapped as possibly inactive) are inferred to pass through the site (**Drawing G-002**). The regional bedding orientation can be summarised as steeply inclined to vertical, dipping from northwest to southwest and east through to southeast

Shears or faults up to approximately 0.5 m in thickness are common in outcrops across the site (**Drawing G-001**). These comprise zones of intensely fractured rock and contain seams of sheared rock or clay 5 mm to 50 mm in thickness. The shears exhibit variable orientations and have an average spacing of 3 m to 10 m. Continuity of the shears or faults has been observed to be at least 20 m in places and some features are expected to be continuous for hundreds of metres.

3.5 Surface Hydrology

Carey's Gully has a surface water catchment in the order of 154 ha, with much of the flow to Carey's Stream, present in the base of the gully, sourced from run-off during rainfall events. Groundwater contributes a base flow to the stream, with artesian groundwater conditions present at the valley floor. These conditions lead to perennial stream flow conditions in the base of the gully. At least two tributaries into Carrey's Stream, with the confluence location indicated as SF1 and SF2 in **Drawing G-001**, also demonstrate a groundwater base flow. **Drawing G-014** indicates the inferred extent of perennial stream flow based on field observations.

During periods of high rainfall, temporary surface water flow can be present in minor gullies along the valley slopes, with these acting as tributaries feeding into Carey's Stream. Discrete catchments within the gully are illustrated in **Drawing C-003**.

Site Hydrogeology and Groundwater Quality

4.1 Groundwater Investigations

In order to characterise the groundwater system and develop a conceptual model of hydrogeology in the vicinity of the proposed Stage 4 landfill a number of hydrogeological investigations have been undertaken at the site. These investigations include installation of monitoring wells, hydraulic conductivity testing, a pumping test, groundwater sampling, monitoring of water levels and measurement of dry-weather flows in the Carey's Stream tributaries to allow estimation of base groundwater flow to the stream.

Previous investigations of hydrogeology in the general area have been undertaken by Woodward Clyde (1994) and Montgomery Watson (2001), with these investigations focussing on remedial actions required in the vicinity of Stage 2 of the landfill.

The investigations undertaken to date, both in the immediate vicinity of the proposed Stage 4 landfill and in previously consented areas, provide characterisation of the hydrogeological setting and specific parameters of the greywacke aquifer. Specific details of the most recent investigations undertaken by URS, which focus on the area of the proposed Stage 4 landfill, are provided below.

4.1.1 Monitoring Well Installation

In total, nine monitoring wells were installed in four locations (BH1, BH2, BH3 and BH4), with locations selected to provide coverage of the valley floor, ridgelines and shear zones in the vicinity of the site. Wells were paired to provide an assessment of groundwater at two depths at each location, with three wells of different depth installed at location BH3.

The grouping and distribution of these monitoring wells allowed assessment of vertical gradients at each location and characterisation of the likely groundwater flow paths. In addition, changes in aquifer properties with depth could be assessed.

A tenth borehole was installed at location BH5, with this bore used as an abstraction well for conducting a pumping test.

The locations of the monitoring wells and pumping well are illustrated in **Drawing G-001** and well completion details are summarised in **Table 4-1**.

4.1.2 Water Level Monitoring

Groundwater levels were measured in each of the monitoring wells on 7-9 March 2011 at the time of groundwater sampling and hydraulic testing, and on 23 June 2011 prior to the groundwater pumping test.

4.1.3 Groundwater Sampling

Groundwater samples were collected from each of the monitoring wells BH1-BH4 on 10 March 2011. Sampling was undertaken approximately 24-48 hours following the rising head tests, at which time the wells were developed and then purged.

Groundwater was sampled using disposable bailers, with three bailer volumes (approximately 3 L) removed prior to samples being collected into laboratory supplied containers. Samples were stored in a chilled state and submitted to Hill Laboratories Ltd, under standard URS chain of custody, for the analysis of the following parameters:

- pH and EC.
- Major anions and cations.
- Nutrients (nitrogen and phosphorous species).
- Select dissolved metals.

Table 4-1 Summary of monitoring well completion details

Monitoring Well	Top of Casing Elevation (m AMSL)	Total Depth (m)	Bottom of Screen Depth (m bgl)	Top of Screen Depth (m bgl)	Top of Filter Pack Depth (m bgl)
BH1A (Deep Valley Floor)	125.46	30	30	24	21.8
BH1B (Shallow Valley Floor)	124.96	11	11	2	1.3
BH2A (Deep Ridgeline)	369 (appx.)[1]	104.5	101.5	89.4	74.5
BH2B (Shallow Ridgeline)	370 (appx.)[1]	45	45	30	24
BH3A (Deep - Upper Northern Track)	205.87	60	59.5	50.5	45.5
BH3B (Medium - Upper Northern Track)	206.07	10	10	4	1.3
BH3C (Shallow - Upper Northern Track)	205.72	26.5	26	20	17
BH4A (Deep - Lower Northern Track)	189.79	54.6	54.6	45.6	42
BH4B (Shallow - Lower Northern Track)	189.7	24.5	24.5	15.5	12.5

Notes: 1) Elevations estimated from LIDAR data and well locations. Margin of error likely +/- 0.5 m.

4.2 Groundwater Levels

There is an extensive record of groundwater levels for monitoring wells in Carey’s Gully, with monitoring of the most recently installed monitoring wells (BH1-BH4) undertaken in March 2011 and June 2011. These results are outlined in **Table 4-2**.

Groundwater levels were found to be highest in the vicinity of the ridgelines and lowest at the base of the gully. Vertical gradients were evident in each of the well sets at each location, with strongly downward gradients evident below the ridgelines and upward artesian gradients evident at the gully floor.

Two cross sections of the gully and its surrounds are provided as **Drawings G-012 and G-013**, with the plan indicating cross section location provided as **Drawing G-011**.

Table 4-2 Measured Groundwater Levels

Monitoring Well	Top of Casing Elevation (m AMSL)	7-9 March 2011		4-6 July 2011	
		Depth to Groundwater (m top of casing)	Groundwater Elevation (m AMSL)	Depth to Groundwater (m top of casing)	Groundwater Elevation (m AMSL)
BH1A (Deep Valley Floor)	125.46	0.00	125.4	0.00	125.4
BH1B (Shallow Valley Floor)	124.96	0.30	124.6	0.12	124.8
BH2A (Deep Ridgeline)	369 (appx.)[1]	28.96	-	27.23	-
BH2B (Shallow Ridgeline)	370 (appx.)[1]	16.42	-	12.49	-
BH3A (Deep - Upper Northern Track)	205.87	0.53	205.3	0.33	205.5
BH3B (Medium - Upper Northern Track)	206.07	1.51	204.5	2.30	203.7
BH3C (Shallow - Upper Northern Track)	205.72	0.31	205.4	0.17	205.5
BH4A (Deep - Lower Northern Track)	189.79	7.85	181.9	7.77	182.0
BH4B (Shallow - Lower Northern Track)	189.7	2.77	186.9	2.68	187.0

Notes: 1) Elevations estimated from LIDAR data and well locations. Margin of error likely +/- 0.5 m.

4.3 Groundwater Quality

The groundwater sampling results are outlined in **Table 4-3**, with Stiff and Piper diagrams provided in **Appendix A**.

Groundwater composition appears to differ between wells as a function of well location and depth. Groundwater in recharge areas demonstrates a composition proportionally high in chloride (BH2A and BH2B), characteristic of rainwater in a region bounded by the sea and/or influenced by salt deposited to surface as sea spray. Deeper groundwater or groundwater in discharge areas demonstrates proportionally higher bicarbonate (BH1A and BH1B), with this reflecting mineral solubilisation during groundwater migration. This is also indicative of the groundwater residence time, with carbonate mineral dissolution occurring over time. As such, groundwater in transitional areas, prior to carbonate mineral saturation or discharge exhibits a more mixed composition.

Relative calcium content follows a similar trend; although, to a lesser extent than bicarbonate, implying mineral dissolution may include calcite. Calcium was found to increase in concentration with residence time, with newly recharged groundwater demonstrating relatively lower calcium content (BH2A and BH2B). The transition of water chemistry is illustrated in cross sections A and B provided as **Drawings G-012 and G-013**.

4.4 Hydraulic Conductivity

4.4.1 Rising Head Tests

Rising head tests were undertaken on each of the monitoring wells to assess hydraulic conductivity in the vicinity of the well screens. Water was evacuated from the wells using compressed air, with pressure transducers and manual water level measurement used to monitor water level recovery.

Based on the water level response and well construction details, hydraulic conductivity for geology in the immediate vicinity of the well screen was calculated[1]. For each monitoring well and rising head test, a minimum and maximum hydraulic conductivity was calculated, with the value adopted for each well selected based on consideration of the recovery curve. The calculation worksheets for each rising head test are provided in **Appendix B**.

Hydraulic conductivity for the greywacke aquifer materials in the immediate vicinity of the monitoring well screens, calculated from the results of rising head tests, were in the range 1×10^{-7} to 4×10^{-6} m/s (refer **Table 4-4**).

Conductivity appears to be greater where relaxation of the rock mass is likely to have occurred, such as near the surface of the ridges and side slopes (BH2B, BH3B, BH3C and BH4B). Conversely, it decreases with depth at these locations as the degree of relaxation decreases and fracture aperture or extent is tighter (BH2A, BH3A and BH4A). The opposite is evident at the valley floor, which may be explained by compression being greatest near the surface.

The presence of shear zones associated with the faults extending through the area are also likely to have a significant bearing on the spatial variability of aquifer hydraulic conductivity, with this creating a high degree of anisotropy within the aquifer. The degree of weathering does not appear to influence the measured hydraulic conductivity to any great degree, suggesting that infilling of fractures is not limiting interconnection of fractures or fracture apertures.

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[1] CANMET Pit Slope Manual, 1977

Table 4-3: Southern Landfill Groundwater Analytical Results

		Sample Details and Analytical Results								
		Southern Landfill								
Sample Location		BH1A	BH1B	BH2A	BH2B	BH3A	BH3B	BH3C	BH4A	BH4B
Laboratory Sample Reference		11/5472-09	11/5472-08	11/5472-04	11/5472-05	11/5472-03	11/5472-01	11/5472-02	11/5472-06	11/5472-07
Date Sampled		10-Mar-11	10-Mar-11	10-Mar-11	10-Mar-11	10-Mar-11	10-Mar-11	10-Mar-11	10-Mar-11	10-Mar-11
Total Bore Depth (m bTOC)		30	11	104.5	45	60	10	26.5	54.6	24.5
Physical Parameters	***Units***									
Alkalinity [1]	g HCO_3/m^3	119	117	55	33	194	127	51	117	92
Conductivity [2]	mS/m	38.3	41.3	43.9	30	44.6	35.6	26.5	38.9	39.2
Total dissolved solids	mg/l	238	247	337	210	292	236	163	259	251
Total hardness	g $CaCO_3/m^3$	115	126	117	55	159	115	51	104	93
Cations										
Calcium	mg/l	29.7	32.6	21.5	9.67	44.4	31.4	10.1	26.3	22.4
Magnesium	mg/l	6.48	7.15	12.2	5.9	6.82	5.36	4.83	6.23	6.22
Potassium	mg/l	0.91	1.21	2.43	1.88	1.92	1.72	1.54	3.19	2.37
Sodium	mg/l	37.1	39.5	39.6	33.7	40.7	31.7	30.4	41.5	47.1
Anions										
Chloride	mg/l	53.5	62.9	109	58.1	43.3	42	43	49.6	58.6
Nitrate nitrogen	mg/l	<0.01	0.46	<0.01	1.8	<0.01	0.18	1.09	0.01	2.73
Sulfate	mg/l	15	16	6.44	13	14.7	11.1	9.27	22.6	18.3
Ion Balance	%	1.62	1.18	2.69	1.94	1.2	1.15	1.24	1.02	0.05
Metals										
Arsenic	mg/l	0.008	0.007	0.001	<0.001	<0.001	0.002	<0.001	0.004	<0.001
Boron	mg/l	0.039	0.035	0.032	0.024	0.038	0.036	0.03	0.047	0.066
Cadmium	mg/l	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002
Chromium	mg/l	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Copper	mg/l	0.0009	0.0005	0.0005	0.0011	<0.0005	0.0006	0.0005	0.0026	0.0008
Iron	mg/l	<0.005	<0.005	0.771	0.011	0.014	<0.005	<0.005	0.008	0.078
Lead	mg/l	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005	<0.0005
Nickel	mg/l	0.0019	<0.0005	0.0016	0.001	0.0009	<0.0005	<0.0005	0.0011	0.0013
Zinc	mg/l	0.007	0.003	0.014	0.008	0.006	0.003	<0.002	0.021	0.006
Nutrients										
Ammonia nitrogen	mg/l	0.03	<0.01	0.01	<0.01	<0.01	<0.01	<0.01	0.02	0.01
Dissolved reactive phosphorus	mg/l	0.011	0.011	0.023	0.106	0.024	0.018	0.03	0.014	0.015
Total organic carbon	mg/l	0.4	<0.3	7.6	3.9	0.4	0.7	0.5	2.3	0.3

Notes:

[1] Titrated to pH 4.5

[2] At 25 degrees Celcius

Table 4-4: Rising Head Tests - Calculated Hydraulic Conductivity

Well	Top of Screen (m bgl)	Bottom of Screen (m bgl)	Calculated K (m/s)		Adopted K (m/s)	Geology
BH1A (Deep Valley Floor)	24	30	9.0E-07	3.3E-06	9.0E-07	Unweathered grey moderatly fractured GREYWACKE, 1 - 5 cm spacing of fractures. Dominant dips 30, 70, 80 degrees
BH1B (Shallow Valley Floor)	2	11	2.2E-07	1.2E-06	2.2E-07	Unweathered grey Highly fractured GREYWACKE. 2-5 cm spacing. Dominant dips 30, 70 degrees.
BH2A (Deep Ridgeline)	89.4	101.5	8.4E-09	5.8E-08	5.8E-08	Unweathered grey to grey/black highly fractured to broken GREYWACKE, 2 - 5cm fracture spacing.
BH2B (Shallow Ridgeline)	30	45	3.8E-06	1.9E-05	3.8E-06	Highly weathered brown highly fractured GREYWACKE. 1-5 cm spacing. Dominant dips 30,45, 70, 80 degree
BH3A (Deep - Upper Northern Track)	50.5	59.5	2.9E-08	3.0E-07	3.0E-07	Unweathered grey moderately fractured GREYWACKE. Interbedded argilite (20-30mm) and quartz veining. Fracture spacing 60 to 200 mm, with highly fractured zones and SHEAR ZONES. Some clay infilling
BH3B (Medium - Upper Northern Track)	20	26	2.3E-06	3.8E-06	3.8E-06	Unweathered grey highly fractured GREYWACKE. 20-200mm spacing. SHEAR ZONES with gravels, some clay infilling
BH3C (Shallow - Upper Northern Track)	4	10	3.4E-07	2.6E-06	2.6E-06	Moderately weathered grey highly fractured GREYWACKE. 20-120 mm spacing some clay infill.
BH4A (Deep - Lower Northern Track)	45.6	54.6	3.8E-07	9.0E-07	3.8E-07	Unweathered grey moderately fractured GREYWACKE with Interbedded argilite and quartz veining. Fracture spacing 20 to 400 mm, with highly fractured zones and SHEAR ZONES. Some clay infilling
BH4B (Shallow - Lower Northern Track)	15.5	24.5	2.6E-06	9.9E-07	2.6E-06	Slightly weathered grey highly fractured GREYWACKE. 20-200 mm spacing with some clay infilling. Occasional broken SHEAR ZONES with sand and gravel

4.4.2 Pumping Test

To provide a robust assessment of aquifer hydraulic conductivity in a shear zone, borehole BH5 was advanced and constructed as an abstraction bore, with a screen interval from 18 – 30 m bgl, for the purposes of undertaking a pumping test. The proximity of this bore to the BH3 series monitoring wells (approximately 17 – 20 m distant) allowed these existing monitoring wells to be used as observation wells for the pumping test. Pressure transducers were used, in conjunction with manual water level measurements, to record groundwater level response in all of the wells (BH1 to BH5). A barometric transducer was also deployed to allow barometric correction of the recorded pressures.

The pumping test was conducted by Webster Drilling Limited, who adjusted abstraction rates between 0.2 – 0.75 l/s over a 16 hour period, whilst monitoring drawdown in BH5. A sustainable pumping rate of 0.2 l/s was ultimately adopted and maintained for a period of 31 hours, at which time pumping was terminated. The recovery of water levels in BH5 and the surrounding monitoring wells was monitored for a further four days.

The pumping test analysis software AQTESOLV for Windows (Version 3.50, HydroSOLVE Incorporated) was used to analyse the water level data, and calculate aquifer transmissivity and storage coefficient. Analysis of water level response was carried out for measurements during the initial stages of pumping (0-100 minutes) and during the subsequent constant rate pumping (1000-2800 minutes).

The Cooper-Jacob (1946) solution was adopted in each case to predict hydraulic conductivity. The recovery data was assessed manually, also using the Cooper-Jacob solution, with two methods of assessing residual drawdown utilised to assess the recovery data. This generic solution (suitable for a porous, unconfined medium) was adopted as the water level response to pumping was not typical of a pumping test in fractured rock; primarily as a function of the variable flow rates. This precluded the use of a pumping test solution typically adopted for fractured rock aquifers.

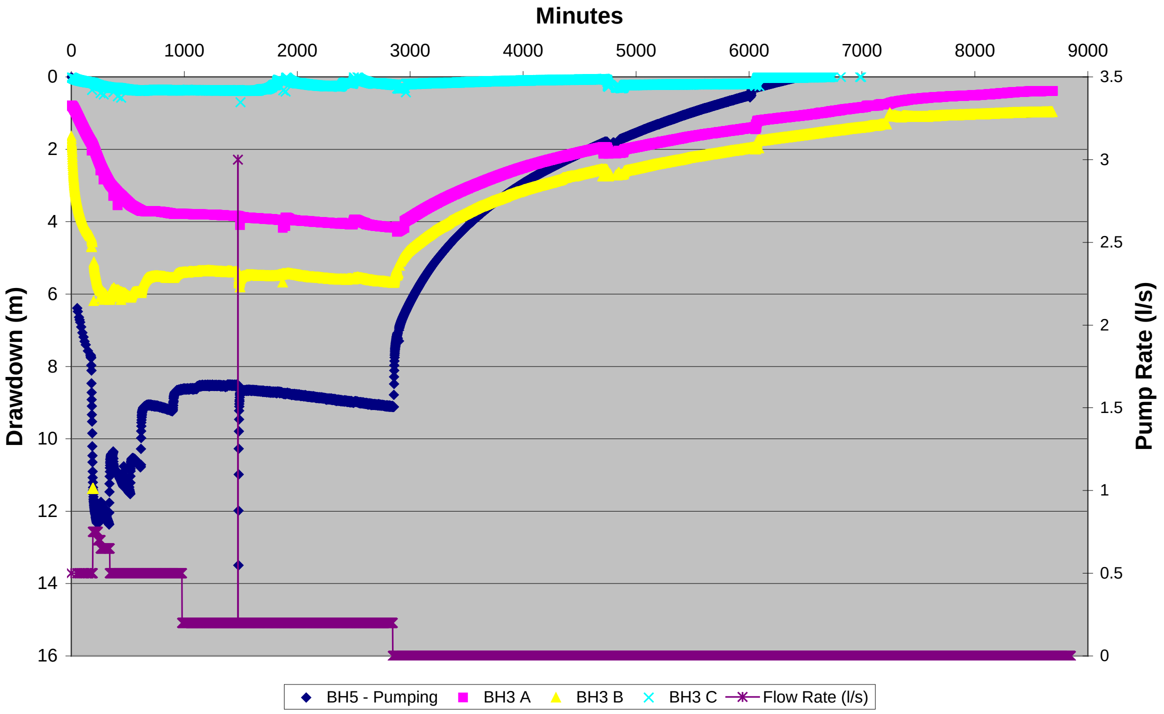
Where results vary for a given well, this variability may be a function of the following:

- Interpretive methodology.
- Differences in aquifer response to pumping and drawdown.
- Differences in interpretation associated with the assessment of different pumping rates.
- Error introduced through data correction and manual interpretation.

Drawdown of the abstraction well (BH5), in the order of 9 m below static water level, was sustained through the constant rate component of the pumping test, with drawdown in the order of 4 m and 6 m recorded in monitoring wells measured in monitoring wells BH3A and BH3B, respectively. These wells demonstrated rapid drawdown response indicating interconnection with the pumping well. The measured drawdown in these wells is illustrated in **Figure 4-1**.

4 Site Hydrogeology and Groundwater Quality

Figure 4-1 Pumping Test Water Level Response



Drawdown consistent with the pumping test was also recorded in monitoring well BH3C (shallow screen interval). However, measured drawdown was significantly less than the deeper wells at this location. No measurable response was apparent at other monitoring well locations (BH1, BH2 or BH4).

Interpreted transmissivities (outlined in **Table 4-5**) are consistent with the current understanding of the hydrogeological setting. On the basis that the aquifer is greater than 60 m depth, as indicated by the responses in shallow and deep wells at location BH3, then calculated aquifer hydraulic conductivity is in general agreement with that determined through rising head tests and approximately between 1.0 x 10^{-7} and 2.0 x 10^{-6} m/s.

Given the placement of monitoring wells at location BH3 and the pumping bore BH5, it is considered that the measured pumping test response and interpreted hydraulic conductivity is representative of the shear zone and as a result is most likely at the upper end of permeability.

Recovery of the observation bores was incomplete, with residual drawdown evident in both BH3A and BH3B. This incomplete recovery indicates that the aquifer in the vicinity of the bores is of limited extent. In the context of the fractured greywacke the limitation is most likely due to poor interconnection between fractures and/or fracturing not being continuous over a large distance.

Table 4-5 Pumping Test Calculated Hydraulic Conductivity

Monitoring Well	Time Period	Calculated Transmissivity (m^2/s)	Calculated Hydraulic Conductivity (m/s)
BH3A	0 – 800 min	3.2×10^{-5}	5.3×10^{-7}
	1000 – 2800 min	3.3×10^{-5}	5.5×10^{-7}
	Recovery	5.3×10^{-6} – 1.2×10^{-4}	8.8×10^{-8} – 2.0×10^{-6}
		7.4×10^{-6} – 1.1×10^{-4}	1.2×10^{-7} – 1.8×10^{-6}
		1.2×10^{-5} – 1.5×10^{-4}	2.0×10^{-7} – 2.5×10^{-6}
BH3B	0 – 140 min	5.4×10^{-5}	9.0×10^{-7}
	1000 – 2800 min	4.2×10^{-5}	7.0×10^{-7}
	Recovery	5.8×10^{-6} – 4.0×10^{-5}	9.7×10^{-8} – 6.7×10^{-7}
		7.0×10^{-6} – 4.0×10^{-5}	1.2×10^{-7} – 6.7×10^{-7}
		1.2×10^{-5} – 3.8×10^{-5}	2.0×10^{-7} – 6.3×10^{-7}

4.5 Aquifer Storage

Aquifer storage is expected to be low and consistent with a fractured rock aquifer, with water storage and flow limited to within fractures.

Pumping test analysis using the Cooper-Jacob solution provided interpreted storage coefficients, with the results outlined in **Table 4-6**. Manual calculation using the same solution was also carried out for the recovery data with these results in general agreement with the software calculated solution.

Table 4-6 Calculated Storage Coefficients

Monitoring Well	Time Period	Storage Coefficient (Unitless)
BH3A	0 – 800 min	3.8×10^{-4}
	1000 – 2800 min	7.78×10^{-6}
	Recovery	2×10^{-4}
BH3B	0 – 140 min	1.3×10^{-5}
	1000 – 2800 min	3.7×10^{-8}
	Recovery	2.9×10^{-5} – 1.3×10^{-4}

In deriving an indicative storage coefficient for the fractured greywacke greater emphasis has been placed on the values estimated from the recovery portion of the pumping test, owing to the extended period of recovery and more reliable data set. On this basis specific yield is expected to be on average in the order of 1×10^{-4}, with this likely varying with the degree and aperture of fracturing (a function of depth and proximity to shear zones). This value is generally consistent with that for fractured rock with minimal primary porosity.

4.6 Groundwater Flow to Carey's Stream

The flow in two tributaries reporting to Carey's Stream was measured during dry-weather periods between January to March 2008. These measurements were made at two locations SF1 and SF2 as illustrated in **Drawing G-001**. Measurements were made on four occasions, following at least six continuous days of no rainfall. It is noted that measurements at SF2 may underestimate the actual flow as a result of an absence of exposed bedrock in this area to allow accurate measurement of flow and the potential for a portion of the flow to be through alluvium. Stream flows measured during the summer of 2008 are provided in **Table 4-7**.

Table 4-7 Carey's Stream Base Flow

Date		Location SF1	Location SF2
Stream Flow Measurements	31 January 2008	0.37 L/s	1.08 L/s
	29 February 2008	0.38 L/ s	0.73 L/s
	19 March 2008	0.6 L/ s	1.18 L/s
	27 March 2008	0.57 L/ s	1.15 L/s
Average Flow (four measurements)		**0.48 L/s**	**1.03 L/s**
Catchment Area (ha)		**23.0**	**29.2**
Unitised Discharge ($m^3/s/m^2$)		**2.1×10^{-9}**	**3.5×10^{-9}**

The groundwater catchments that contribute flow to each of these tributaries, if considered to be equivalent to the surface water catchments as indicated in **Drawing C-003**, are in the order of 23 ha and 29 ha for SF1 and SF2, respectively. Assuming all groundwater recharged within this catchment reports as shallow groundwater to the tributary, recharge in the order of 2.1 – 3.5×10^{-9} $m^3/s/m^2$ is estimated or approximately 10% of rainfall. However, it is noted that these calculated recharge rates are based on seasonal low flow and where seasonal high flow (winter) is considered, overall recharge and groundwater flow is likely to be greater.

An estimation of base flow in Carrey's Stream is made by adopting the maximum recharge rate of 3.5×10^{-9} $m^3/s/m^2$ and applying this to the Carrey's Stream catchment areas. Based on groundwater contribution from the 11 catchments identified a total low-flow condition of approximately 8 L/s is estimated, with the change in estimated flow along the stream indicated in **Drawing G-014**.

4.7 Preferential Flow Paths

The hydraulic conductivity of shear zones is expected to be greater than the bulk rock mass, with these relatively discrete zones expected to provide preferential pathways for groundwater flow. This inference is supported by observations made during site inspection at which time groundwater was observed to be discharging from a shear zone.

Hydraulic testing of groundwater wells inferred to be screened in a shear zone, as outlined in **Section 4.4.1,** has indicated that hydraulic conductivity is low relative to other fractured rock environments.

Flow from shear zones is considered as a component of the predicted base flow to the stream. On the basis that the overall catchment base flow is relatively low (8 L/s), it is considered likely that the rate of preferential groundwater flow through these shear zones is also low, with any discrete zone contributing less than approximately 1 L/s.

4.8 Conceptual Hydrogeological Model

The hydraulic characteristics of the greywacke aquifer in the vicinity of the proposed Stage 4 landfill, is a function of the significant faulting activity in the area, which also influences the site topography and geomorphology. Groundwater flow occurs within fractures, and is likely to be greatest where these are more dense, extensive and open. Relaxation of the rock mass, promoting more open and extensive fracturing, is most likely along the ridges, whilst at depth and in the valley floor the degree of relaxation is likely less and compressive forces may be present. The differences in fracturing associated with the vertical spatial variability in relaxation/compressive forces are apparent in the hydraulic testing results, with rising head tests demonstrating generally lower permeability at depth.

Based on the hydraulic testing undertaken it is apparent that the transmissivity of the fractured rock aquifer is overall relatively low. Whilst significant fracturing is present, the permeability associated with this secondary porosity is limited, with this potentially the result of the following:

- Limited interconnection between fracture/discontinuities.
- Relatively small fracture aperture.

Residual drawdown following recovery from the pumping test suggests that the network of fractures is poorly connected, with this poor interconnection limiting overall groundwater flow within the greywacke. However, infilling of fractures from weathering products is not considered to significantly limit permeability, as demonstrated by generally higher permeability in the more weathered near surface zones.

The water table (piezometric surface) is a subdued expression of topography, being relatively high along the ridges and low in the gully floor. The significant differences in groundwater elevation create strong vertical hydraulic gradients that are likely to dominate groundwater flow. Recharge of groundwater is predicted to be greatest in the vicinity of the ridges, where the potential for soakage is greatest owing to rock exposures and higher permeability. This is supported by the greater extent of weathering observed along the ridges and the composition of the groundwater in these recharge zones. Water seeps through the surface and percolates downwards under strong vertical gradients. The incised gully floor, with low elevation relative to the surrounding groundwater levels, forms a discharge zone with strong upward gradients from the underlying aquifer providing artesian conditions. This discharge water, as with deeper groundwater outside the recharge zones, demonstrates a more mature composition, with a move towards proportionally greater calcium and bicarbonate content relative to groundwater in the recharge zones.

Under the conditions identified, Carey's Gully is hydraulically constrained, with piezometric highs beneath the ridgelines representing groundwater divides. As indicated, the hydraulic gradients in the valley catchment will direct groundwater ultimately toward the discharge zone at the valley floor.

Leachate Generation and Seepage

5.1 Assessment of Leachate Generation and Seepage

An estimate of landfill leachate generation rate was made using the 'Hydrologic Evaluation of Landfill Performance' (HELP) software endorsed by the USEPA. This one dimensional numerical model uses a water balance approach to assess run-off, evapotranspiration (ET) and drainage through the various soil layers that constitute the landfill cover to allow prediction of the partitioning of water flows through and across the cover.

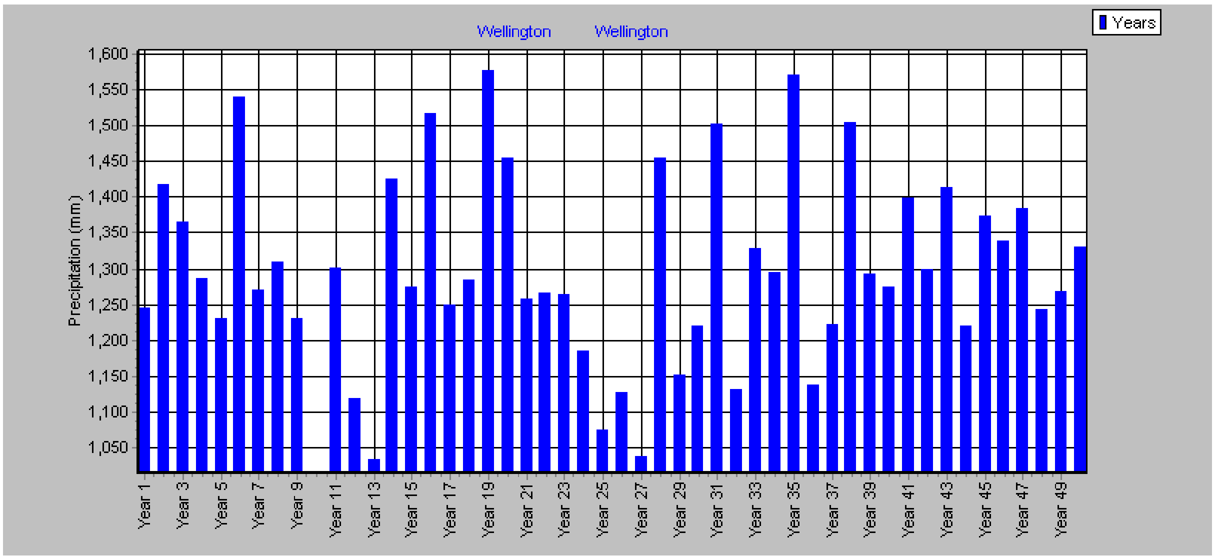
The HELP model incorporates local climate conditions into a weather model to simulate rainfall and evapotranspiration for the site. The physical and hydraulic properties of the cover are included as part of a one dimensional model, with this subjected to the predicted rainfall and ET.

5.2 Climatic Data

The New Zealand Weather Generator, developed by Lincoln University and University of Canterbury specifically for use with the HELP model, was used to derive synthetic weather data for the Wellington region. This model incorporates the rainfall, temperature, wind, solar radiation and growing conditions specific to the Wellington region.

Based on a synthetic 50 year period, the weather model generated an annual average rainfall of 1.29 m/year, which is consistent with regional weather station data. **Figure 5-1** illustrates the synthetically generated rainfall over the 50 year period.

Figure 5-1 Wellington Region Modelled Rainfall



An example of modelled monthly temperatures is illustrated in **Figure 5-2**, with an example of monthly solar radiation illustrated in **Figure 5-3**.

Figure 5-2 Modelled Monthly Temperatures

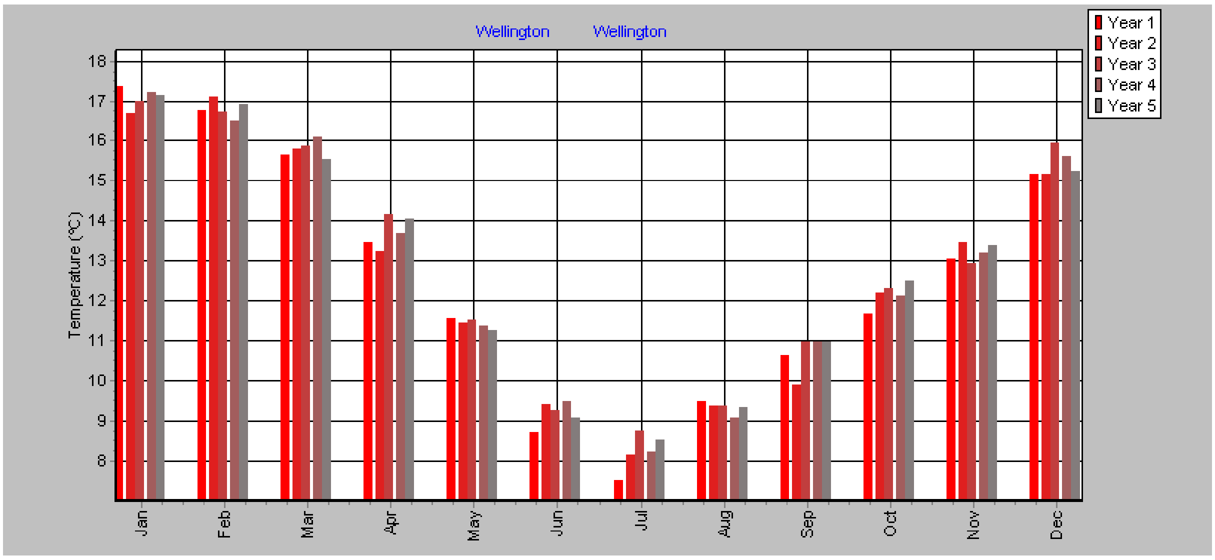
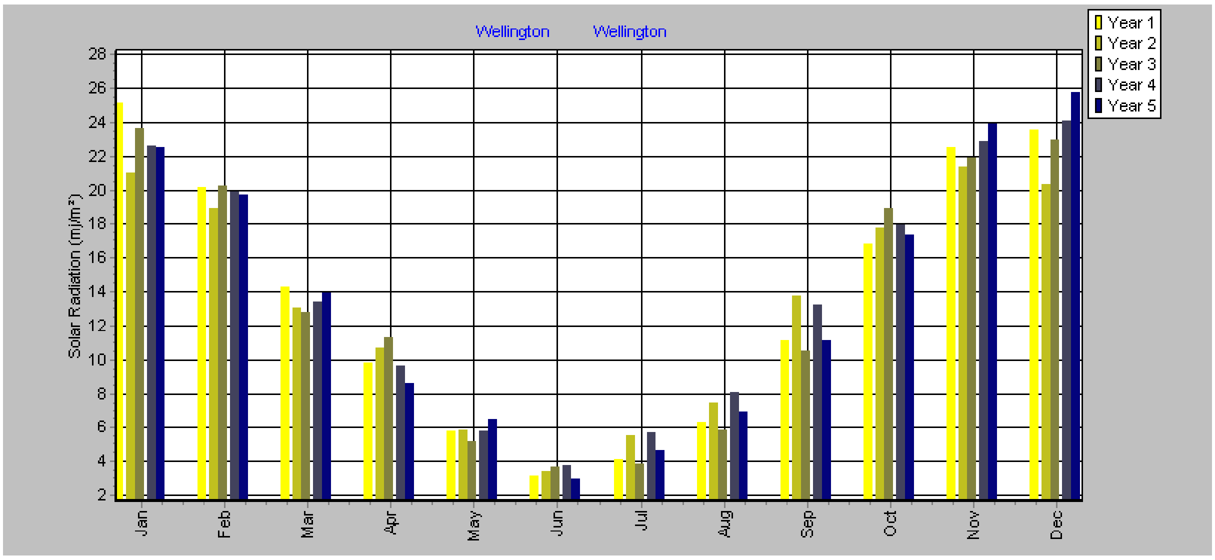


Figure 5-3 Modelled Monthly Solar Radiation



Evapotranspiration was predicted to be 0.75 m/year, with the model assuming the development of a grass surface on the cap topsoil, and a relatively limited evapotranspiration depth of 0.25 m.

5.3 Stage 4 Landfill Cover Design

The proposed Stage 4 landfill final cover would be as described in **Section 2.1.2**, and comprises a coarse gravel layer for landfill gas collection overlain by a 600 mm compacted soil cover. To protect this cover and support topsoil, the compacted soil will be overlain by approximately 150 mm, of selected fill materials, likely to comprise lightly weathered “rotten” greywacke rock. A final 150 mm topsoil layer will finish the cover design. Over much of the landfill, the cover will be graded to a slope of 4:1 (h:v). Adopted hydraulic conductivity and thickness of the landfill cover layers within the model were consistent with the design specifications. Other physical and hydraulic properties were sourced from the model internal material library and are consistent with the material types. Adopted properties are outlined in **Table 5-1**.

Table 5-1 Adopted Hydraulic and Physical Properties

Material	Thickness (m)	Hydraulic Conductivity, K (m/s)	Porosity, n	Wilting Point
Topsoil	0.15	1×10^{-6}	0.35	0.15
Selected Fill	0.15	1×10^{-4}	0.35	0.05
Compacted Soil Cover	0.6	1×10^{-7}	0.48	0.25
Gravel	0.3	1×10^{-3}	0.35	0.019

5.4 Leachate Generation

Model predictions for leachate generation are in the order of 11-16% (0.12 – 0.22 m/year) of rainfall, based on the modelled 50 year average. Modelled rates of rainfall and leachate generation are illustrated **Figure 5-4**.

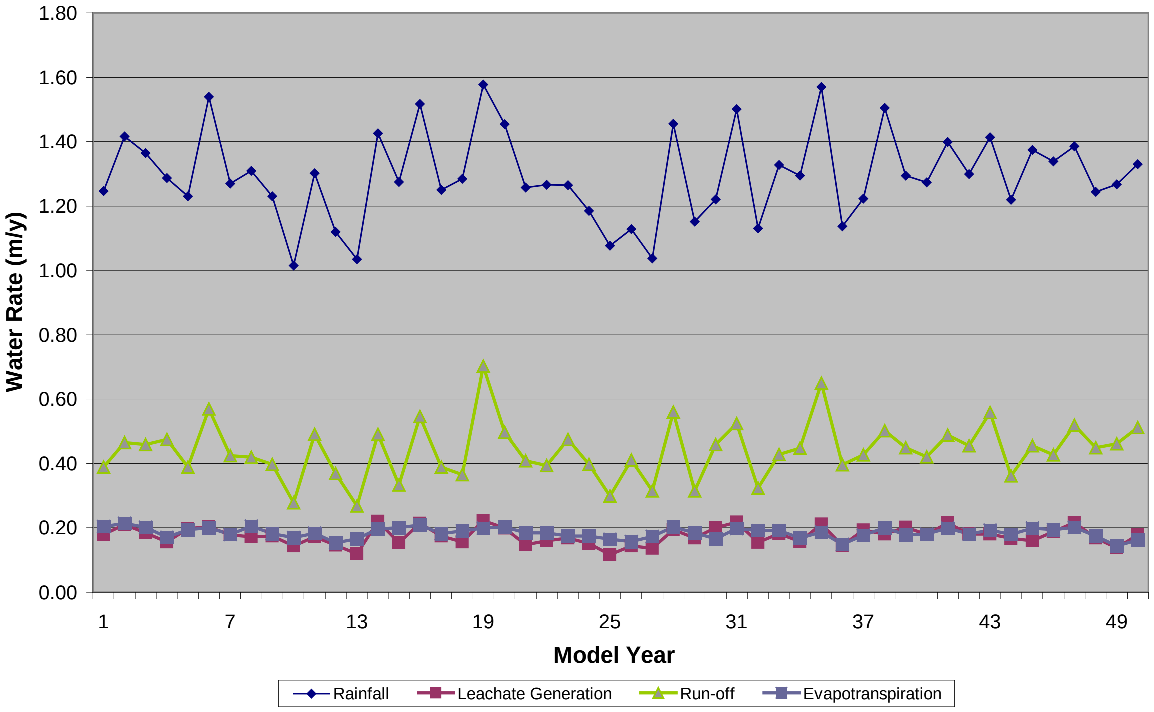
Based on a final landfill cap area of 28 ha, and the modelled rates of leachate generation, it is estimated that leachate generation would range between 33,000 m^3 (0.12 m/year) and 63,000 m^3 (0.22 m/year) per year at completion of the Stage 4 filling operations. Leachate generation may be higher prior to placement of the final cover owing to the exposed active areas of landfill without final cover.

Groundwater seepage from the side slopes would be expected in discrete areas where shear zones, which act as conduits for groundwater, intercept the gully. The majority of seepage reports to the base of the gullies, which would be controlled by the groundwater cutoff (sub liner drainage system). Under the current proposal if significant areas of side slopes seepage were encountered they would be grouted or sealed prior to filling. There are likely to be other minor shear zone areas where lining and draining are not practical. Seepage from these areas would provide dilution to the leachate and report to the leachate collection system, adding to the predicted volumes. Allowance would be made during detailed design for seepage contribution to the leachate flow.

5.5 Leachate infiltration to Ground

Seepage of leachate through the liner is predicted to occur as leachate contacts the landfill liner, but at a highly retarded rate relative to leachate generation. It is predicted that on average approximately 7-10% of the leachate generated (0.009 – 0.018 m/year) may infiltrate through the landfill liner. This seepage will mix with groundwater immediately beneath the landfill and report directly to the landfill sub-liner drainage. Owing to attenuation of contaminants across the liner, the high degree of dilution within groundwater, and the capture of any impacted groundwater by the landfill underdrains, it is considered that impacts to groundwater quality distant from the landfill footprint would be less than minor.

Figure 5-4 Modelled Leachate Generation



Assessment of Environmental Effects

6.1 Introduction

The proposed Stage 4 landfill has the potential to influence both groundwater levels and groundwater flow (particularly groundwater discharge to the receiving environment) and also groundwater quality.

The potential effects result primarily from the following:

- The presence of the landfill at the base of the gully, in the groundwater discharge zone, and the associated installation of sub-surface drain in the location of the existing Carey's Stream.
- The generation of leachate within the landfill and potential for seepage through the landfill liner.
- The large footprint area of the proposed Stage 4 landfill, limiting overland surface water flow to Carey's Stream.

6.2 Existing Environment

6.2.1 Hydrogeology

The hydrogeological units at the site, as detailed in Section 3, are summarised as follows:

- Rakaia Terrane (Torlesse Group) indurated sandstones and mudstones, often referred to as "greywacke". These materials are a fractured rock with low storage and low permeability.
- Alluvium, consisting of sub-rounded gravel of sandstone, occupies a narrow strip of limited depth in the bottom of the gullies. These materials constitute an unconsolidated deposit of limited extent, with likely high permeability and moderate storage.

6.2.2 Groundwater Levels

Groundwater levels in the footprint of the proposed Stage 4 landfill and its immediate surrounds are generally a subdued expression of the topography, being higher along the ridgelines and lower in the gully.

Groundwater levels closer to the ridgelines demonstrate a strong vertically downward gradient, with levels at depth significantly lower than those closer to the surface. In contrast, water levels in the gully demonstrate artesian conditions (vertically upwards gradient) with water levels at a higher elevation than the ground surface. **Drawings C-015 and C-016** illustrate the proposed construction of the landfill liner and liner underdrain relative to the existing surface.

6.2.3 Groundwater Recharge

Groundwater recharge is considered to occur in the vicinity of the ridgelines and upper areas of the gully side slopes. The strong downward gradients in these areas and greater depth of rock weathering in these areas support this inference.

6.2.4 Groundwater Flow Regime

The groundwater flow regime is typical of a steep ridge and gully environment. This includes recharge of groundwater near the ridgeline, downward percolation and flow through the subsurface to the gully, where it discharges under artesian conditions. A portion of the recharging groundwater is likely to migrate sufficiently deep to form part of the regional deep groundwater, with flow towards the coast.

Vertical flow directions are likely to dominate the early and late stages of groundwater flow, in the zones of recharge and discharge, respectively.

Shallow groundwater in the vicinity of the site, as investigated by installation of piezometers, is considered to be hydraulically constrained, with groundwater flow towards Carey's Stream.

6.3 Effects of Proposed Stage 4 Landfill on Groundwater Levels

The proposed Stage 4 landfill is located within the groundwater discharge zone (Carey's Stream) of Carey's Gully with filling of the gully resulting in the base of the landfill being present in the stream bed. A sub-liner/underdrain system is proposed to be located beneath the landfill liner at approximately the same level as the existing stream bed. This drainage system will allow groundwater to discharge at approximately the same elevation and rate as it would discharge to the stream. As such, the potential for the landfill to influence groundwater levels in the vicinity of Carrey's gully is considered to be minimal, with no significant drawdown of the aquifer expected.

In addition, as the landfill footprint is within the inferred discharge zone of the aquifer, the diversion of surface flows is not likely to significantly impact groundwater levels.

As such, likely effects to groundwater levels are considered to be less than minor.

6.4 Effects of Proposed Stage 4 Landfill on Groundwater Recharge

The proposed landfill footprint, which constitutes approximately 18% of the total Carrey's Gully catchment area, is generally inferred to be positioned outside of the groundwater recharge areas, considered to be present around the ridgelines and upper sections of the gully walls. Owing to the artesian conditions present in the gully floor and limited recharge potential in the lower gully it is considered unlikely that groundwater recharge will be reduced.

It is inferred that the landfill would not result in significant recharge as a result of the following:

- The hydraulic gradient in the area of the landfill base and sidewalls is inferred to be towards the landfill
- Leachate would be generally contained by the proposed containment, leachate management and sub-liner drainage systems.

So whilst contributing some recharge to shallow groundwater in the immediate vicinity of the landfill footprint, the extent is limited and the potential flow paths are constrained.

As such, any adverse effects to the recharge of groundwater in the vicinity of the proposed Stage 4 landfill will be less than minor.

6.5 Effects of Proposed Stage 4 Landfill on Groundwater Flow Directions

As the groundwater recharge and the groundwater discharge zones effectively being maintained, it is considered that the groundwater flow regime will remain approximately equivalent to the current setting, with the gully being hydraulically constrained by elevated groundwater in the vicinity of the surrounding ridges and groundwater flow toward the gully floor.

6.6 Effects of Proposed Stage 4 Landfill on Groundwater Quality

The landfill leachate collection system and the liner system are expected to contain and remove the majority of leachate generated by the proposed landfill. It is estimated that approximately 33,000 and 63,000 m^3/year of leachate will be generated on average after filling of Stage 4 is complete and the final landfill cover system has been installed.

6.6.1 Landfill Sidewalls

As indicated in **Section 2.2**, due to the steepness of the gully side walls within the proposed landfill footprint, lining and provision of leachate drainage pipework is generally not proposed. The potential for leachate discharge through the sidewalls is considered to be limited owing to the steepness of the sidewalls and the relatively low permeability of the sidewall rock (estimated to be in the order of 1×10^{-7} m/s). As a result, it is unlikely that a head of leachate would form on the sidewalls. In addition, where a minimal volume of leachate may infiltrate the rock mass, the groundwater flow directions are such that infiltrated leachate would either report directly to the landfill underdrains, or discharge back into the landfill.

6.6.2 Landfill Base

The head of leachate on the liner system will be managed by the leachate collection system, which will discharge collected leachate to the trade waste system for treatment. The proposed liner system will limit leachate transport to the underlying groundwater system to on average less than 10% of the leachate generated. However, the small proportion of leachate that does migrate through the liner system will be collected by the proposed liner underdrainage system. This underdrainage system will have the provision to divert flow from its normal proposed discharge as storm water to the leachate system should monitoring of the water quality in the underdrainage system indicate that this is required. In this manner the underdrainage system will serve as tertiary control (after the leachate collection and containment system) of leachate.

Groundwater flow at the base of the landfill is predicted to have a strong upwards vertical gradient towards the underdrain system. This is supported by the observed artesian conditions present in the gully floor. As a result there is limited potential for leachate infiltrating through the liner to interact with groundwater outside the immediate surrounds of the liner underdrain. This hydraulic containment is considered to mitigate potential contaminant migration away from the site.

6.6.3 Preferential Flow Paths

Groundwater flow through the greywacke aquifer is considered to occur predominantly through the shear zones, where fracturing is likely to be better connected, relatively more frequent and with greater apertures. These zones constitute preferential pathways for groundwater flow. The shear zones surrounding faults can be extensive and cross groundwater catchments, potentially providing pathways for groundwater to flow across the otherwise defined catchments. Groundwater investigations undertaken around the proposed landfill targeted likely shear zone areas, with the groundwater levels identified indicating that the Carey's Stream gully is hydraulically contained. Groundwater levels in the surrounding recharge zones are expected to limit the potential for impacted groundwater or leachate to flow away from the landfill and as such the potential effects associated with leachate flow along such preferential flow paths is considered to be less than minor.

6.6.4 Summary of Effects

As a result of the above it is considered that potential adverse effects on groundwater quality outside the immediate vicinity of the underdrain is expected to be less than minor.

6.7 Summary Assessment of Effects to Groundwater

The potential for the proposed Stage 4 landfill to adversely influence groundwater conditions is considered to be mitigated primarily through the following:

- Natural hydraulic containment within Carey's Gully;
- Low permeability of in-situ greywacke rock constituting the majority of the sideslopes within the landfill footprint;
- Location of the base of the landfill footprint within a groundwater discharge zone (Carey's Stream), which will limit downwards migration of leachate and comingling with surrounding groundwater;
- Provision of a drainage layer or inclined chimney system at the landfill sideslopes to limit leachate head to 300 mm in these areas;
- Provision of a leachate collection system to limit leachate head on the landfill liner;
- Provision of a low-permeability liner system at the landfill base to contain leachate;
- Provision of a liner underdrainage system, which will limit groundwater pressure on the liner system, provide a means for groundwater discharge similar to the existing groundwater flow regime, and allow for tertiary control of landfill leachate.

As such, adverse effects on groundwater associated with the proposal are expected to be less than minor.

Limitations

URS New Zealand Limited (URS) has prepared this report in accordance with the usual care and thoroughness of the consulting profession for the use of Wellington City Council and only those third parties who have been authorised in writing by URS to rely on the report.

It is based on generally accepted practices and standards at the time it was prepared. No other warranty, expressed or implied, is made as to the professional advice included in this report.

It is prepared in accordance with the scope of work and for the purpose outlined in the Proposal dated November 2009.

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It is the responsibility of third parties to independently make inquiries or seek advice in relation to their particular requirements and proposed use of the site.

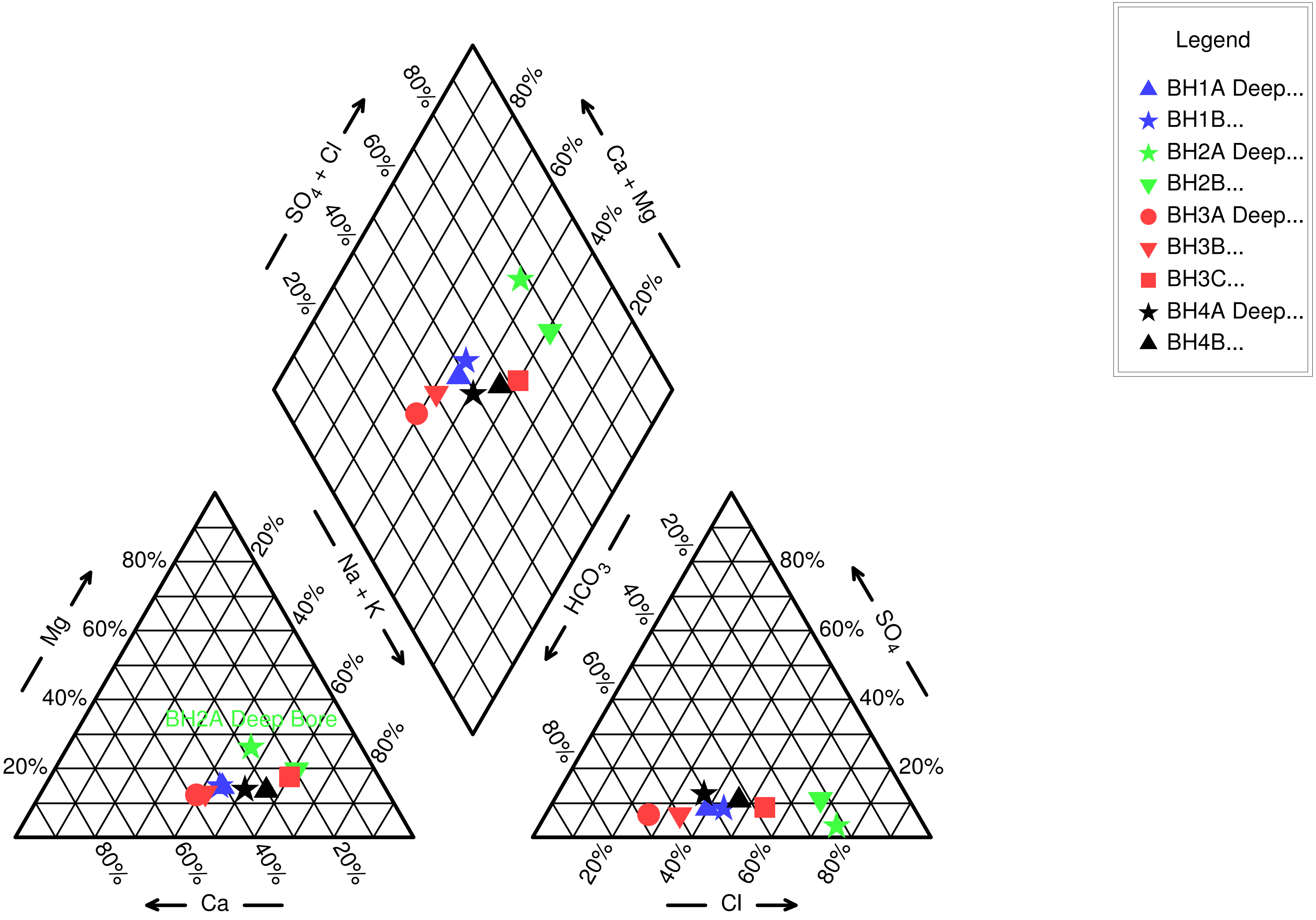
Any estimates of potential costs which have been provided are presented as estimates only as at the date of the Report. Any cost estimates that have been provided may therefore vary from actual costs at the time of expenditure.

Drawings

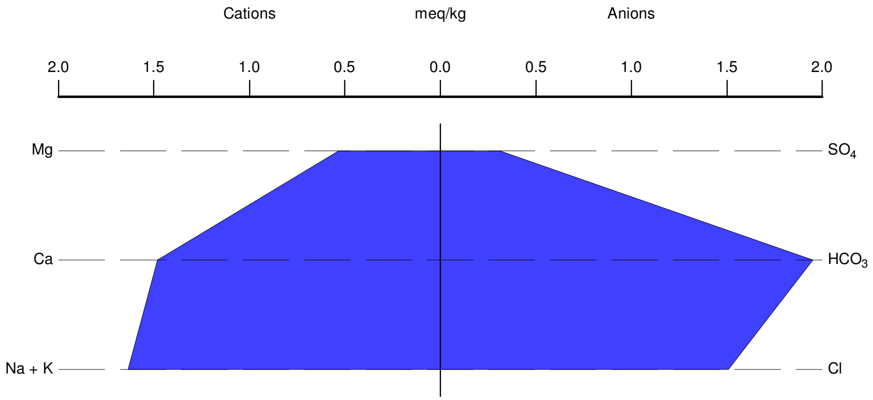
see separate pdf

Appendix A Groundwater Stiff and Piper Diagrams

Southern Landfill



BH1A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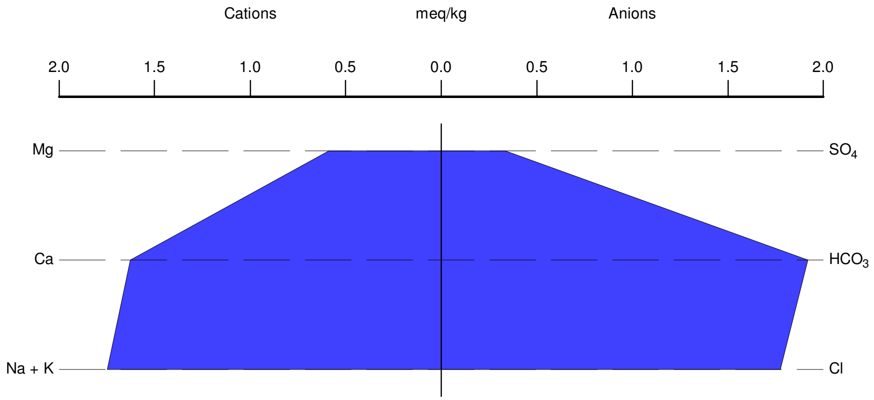
1.0
0.5
0.0
0.5
1.0
1.5
2.0
Mg
Ca
Na + K
SO_4
HCO_3
Cl

BH1B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meq/kg
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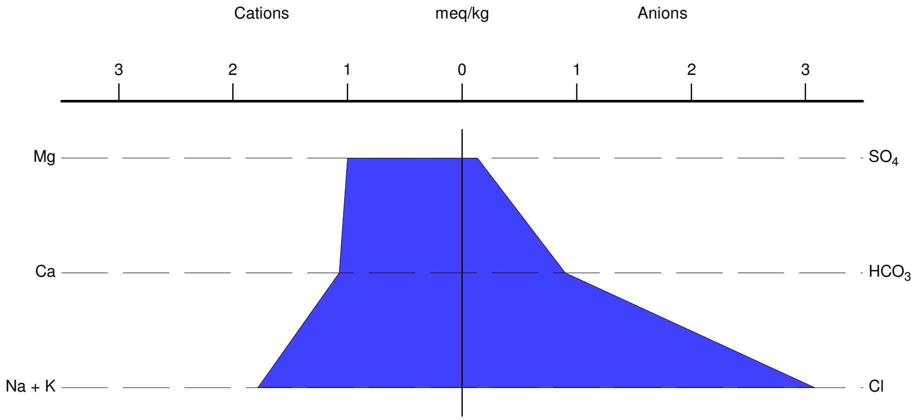
1.0
1.5
2.0
Mg
Ca
Na + K
SO_4
HCO_3
Cl

BH2A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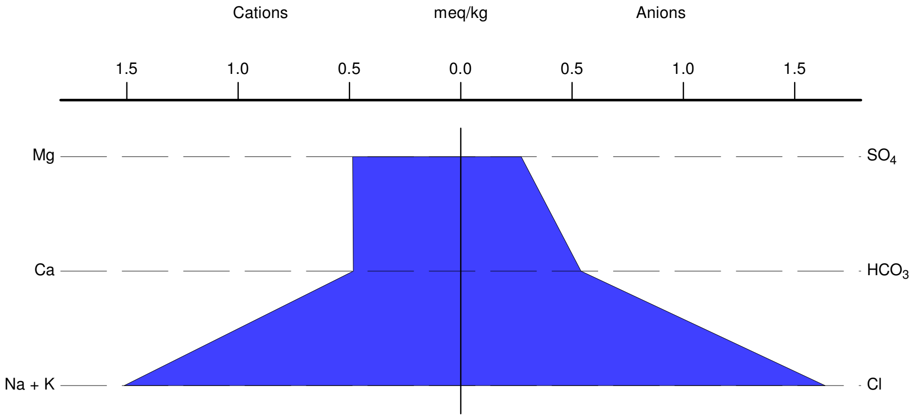
0
1
2
3
Mg
Ca
Na + K
SO4
HCO3
Cl

BH2B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meq/kg
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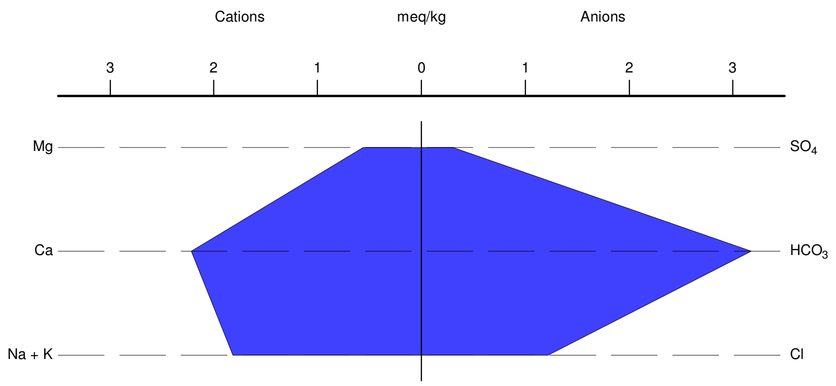
1.5
Mg
Ca
Na + K
SO4
HCO3
Cl

BH3A



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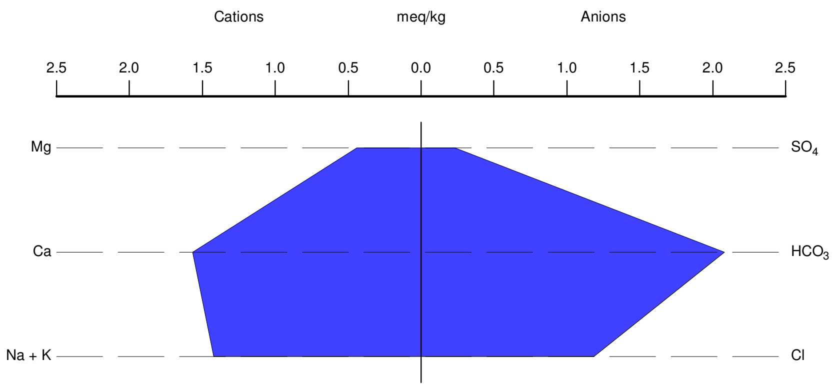

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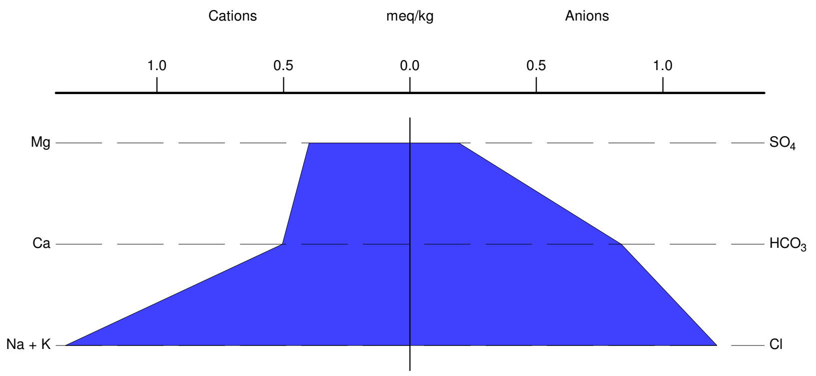

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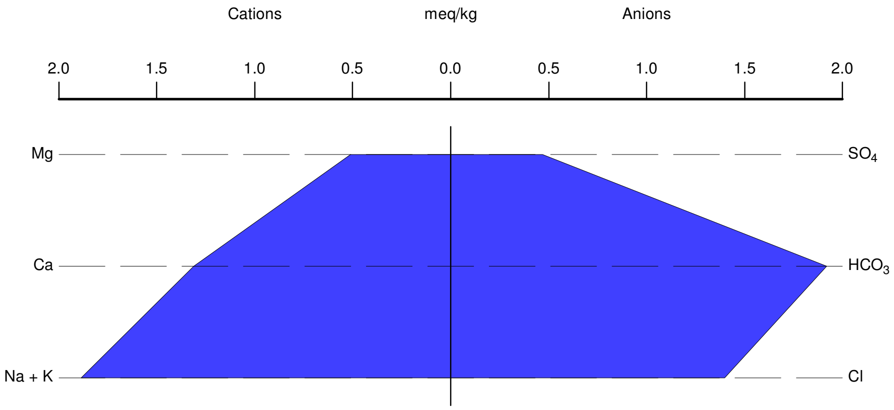
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meq/kg
Anions
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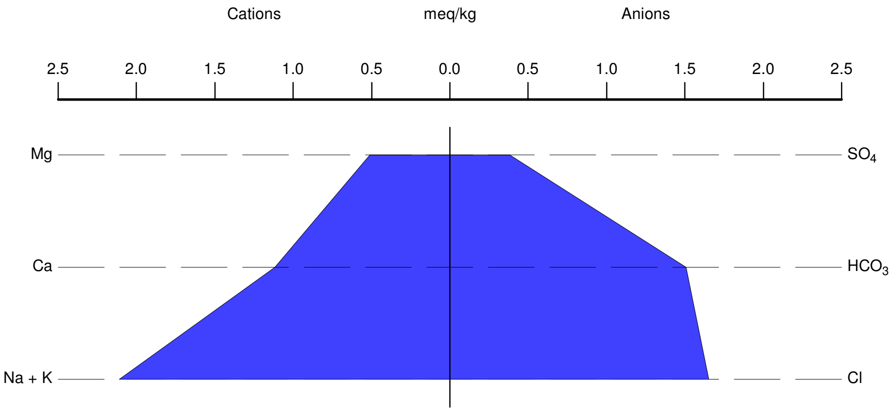
1.0
0.5
0.0
0.5
1.0
1.5
2.0
Mg
Ca
Na + K
SO4
HCO3
Cl

BH4B


Cations
meq/kg
Anions
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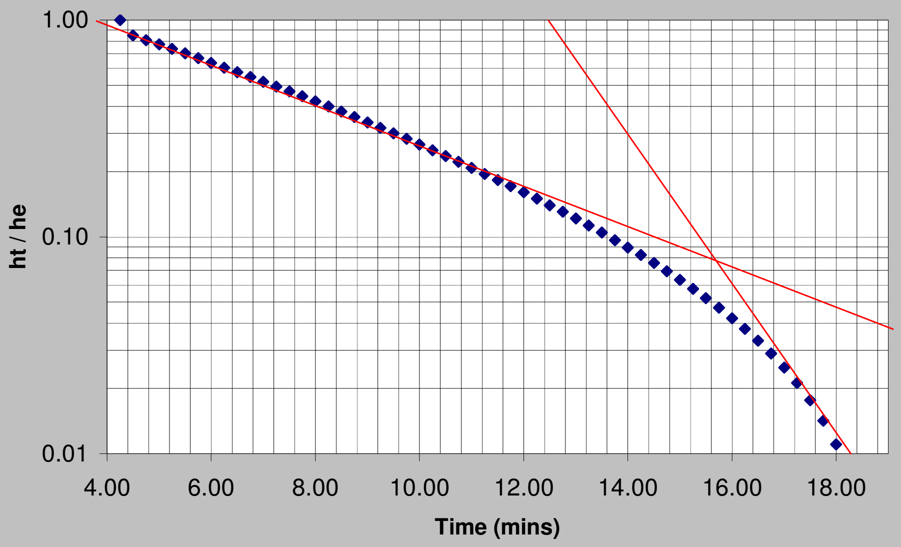
1.0
0.5
0.0
0.5
1.0
1.5
2.0
2.5
Mg
Ca
Na + K
SO4
HCO3
Cl

Appendix B Hydraulic Test Worksheets

BH1A Permeability Test

Static Water level	0	*m bgl*

Bottom of Test Section	30	*m bgl*
Top of Test Section	21.8	*m bgl*
Length of Test Section	8.2	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.048	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.001237002	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.001809557	*m3*	(area pipe ID)
Total unit volume	=	0.003046559	*m3*	
		0.00096975	*m3*	
Effective Radius	=	0.031140809	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

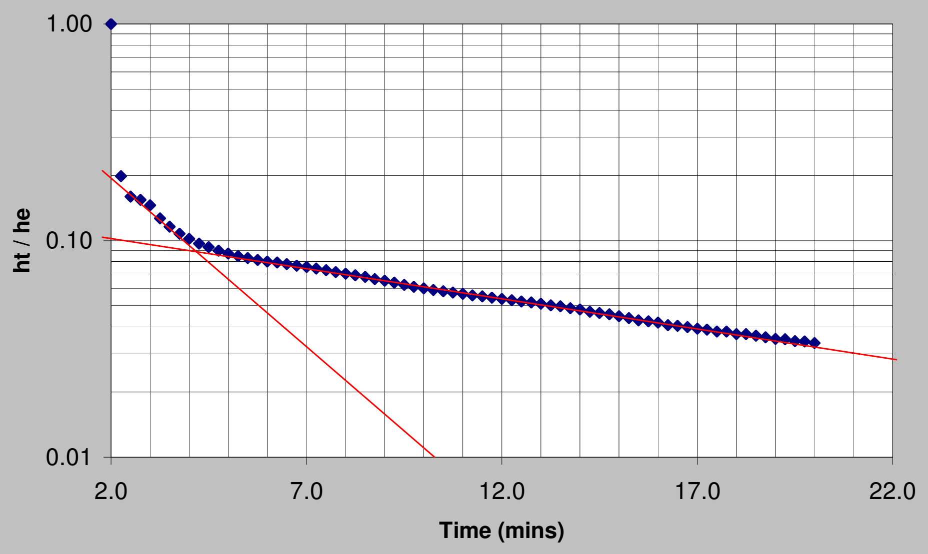
Permeability = 0.133S(r_e^2/L) m/s

	A		B	
InterceptHt/He	0.1		0.01	
Intercept Ht/He	1		0.1	
Intercept t	4		18.2	
Intercept t	14.4		15.4	
Slope	0.096153846		-0.357142857	
L	8.2		8.2	
Permeability	**8.98E-07**	*m/s*	**3.34E-06**	*m/s*

BH1B Permeability Test

Static Water level	0.3	*m bgl*

Bottom of Test Section	11	*m bgl*
Top of Test Section	1.3	*m bgl*
Length of Test Section	9.7	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.038	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.00056156	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.001809557	*m3*	(area pipe ID)
Total unit volume	=	0.002371117	*m3*	
		0.00075475	*m3*	
Effective Radius	=	0.027472714	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

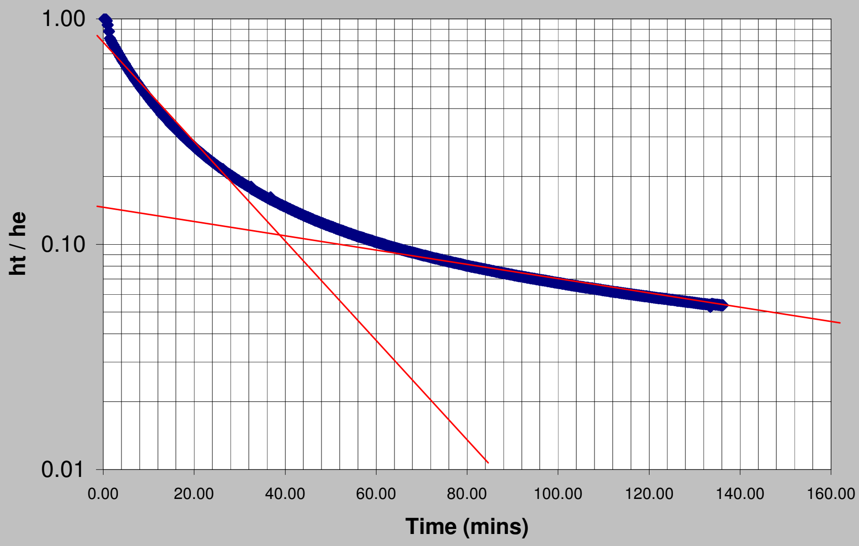
Permeability = $0.133S(r_e^2/L)$ m/s

	A		B	
InterceptHt/He	0.1		0.1	
Intercept Ht/He	0.01		0.04	
Intercept t	3.9		2.5	
Intercept t	10.3		17	
Slope	-0.15625		-0.02744414	
L	9.7		9.7	
Permeability	**1.2E-06**	*m/s*	**2.2E-07**	*m/s*

BH2A Permeability Test

Static Water level	28.96	*m bgl*

Bottom of Test Section	104.5	*m bgl*
Top of Test Section	74.5	*m bgl*
Length of Test Section	30	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.048	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.001237	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.00181	*m3*	(area pipe ID)
Total unit volume	=	0.003047	*m3*	
		0.00097	*m3*	
Effective Radius	=	0.031141	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

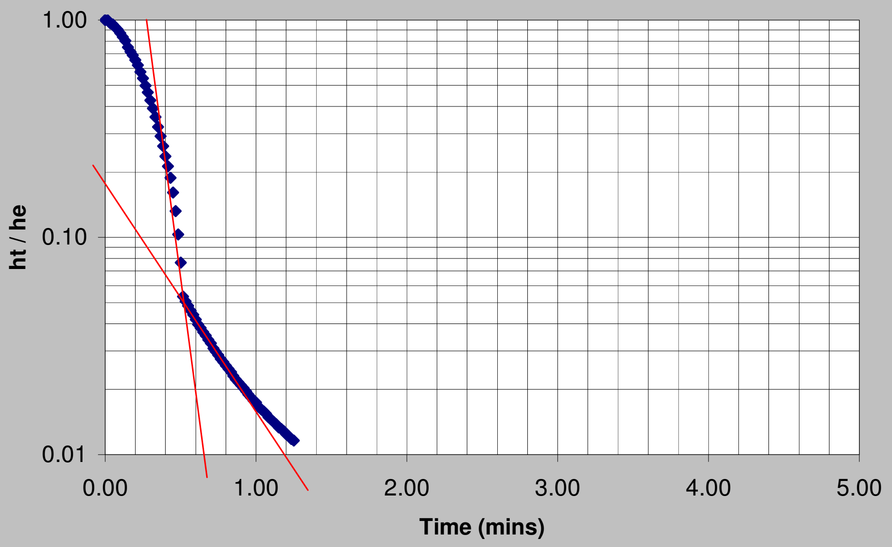
Permeability = 0.133S(r_e^2/L) m/s

	A		**B**	
InterceptHt/He	0.01		0.05	
Intercept Ht/He	0.8		0.1	
Intercept t	84		140	
Intercept t	0		46	
Slope	-0.022656		-0.003202	
L	30		30	
Permeability	**5.8E-08**	*m/s*	**8.2E-09**	*m/s*

BH2B Permeability Test

Static Water level	16.42	*m bgl*

Bottom of Test Section	45	*m bgl*
Top of Test Section	24	*m bgl*
Length of Test Section	21	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.038	*m*
Filter Pack Porosity	0.25	



Class E PVC ID	0.048	*m*	
Class E PVC OD	0.054	*m*	
Unit volume of Filter =	0.0005616	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe =	0.0018096	*m3*	(area pipe ID)
Total unit volume =	0.0023711	*m3*	
	0.0007548	*m3*	
Effective Radius =	0.0274727	*m*	(sq rt (total volume/pi))
r_e =	0.024	*m*	

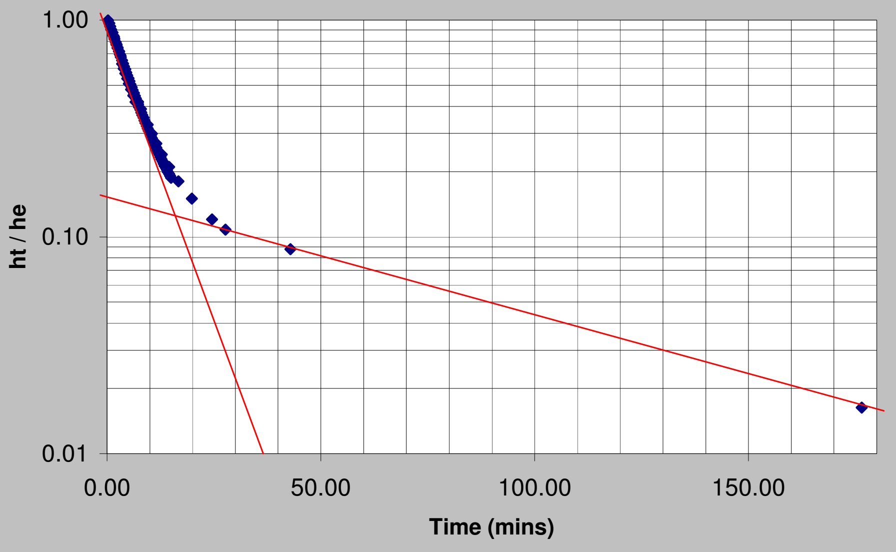
Permeability = $0.133S(r_e^2/L)$ m/s

	A	B
InterceptHt/He	0.01	0.01
Intercept Ht/He	1	0.1
Intercept t	0.68	1.2
Intercept t	0.3	0.25
Slope	-5.2631579	-1.0526316
L	21	21
Permeability	**1.9E-05** *m/s*	**3.8E-06** *m/s*

BH3A Permeability Test

Static Water level	0.476	*m bgl*

Bottom of Test Section	60	*m bgl*
Top of Test Section	45.5	*m bgl*
Length of Test Section	14.5	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.048	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.001237002	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.001809557	*m3*	(area pipe ID)
Total unit volume	=	0.003046559	*m3*	
		0.00096975	*m3*	
Effective Radius	=	0.031140809	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

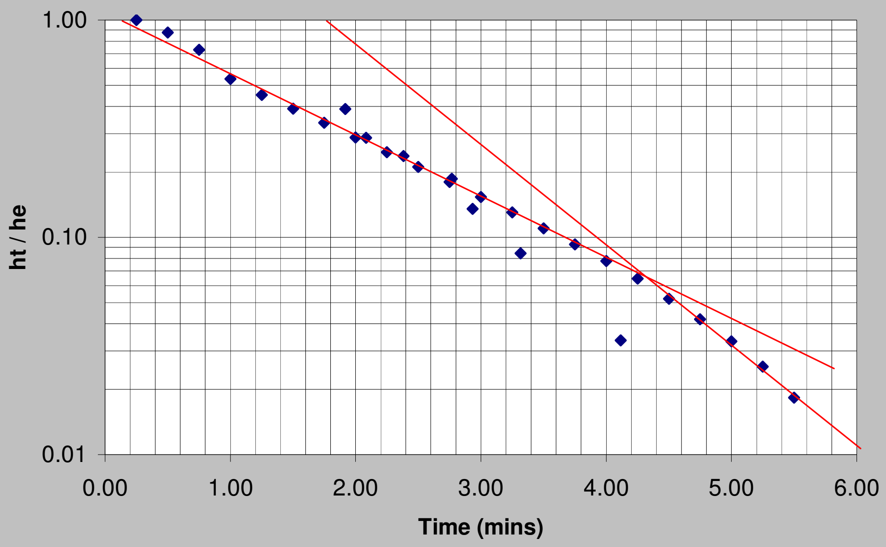
Permeability = $0.133S(r_e^2/L)$ m/s

	A		B	
InterceptHt/He	0.01		0.02	
Intercept Ht/He	1		0.1	
Intercept t	2.8		162	
Intercept t	38		35	
Slope	0.056818182		-0.0055037	
L	14.5		14.5	
Permeability	**3.0E-07**	*m/s*	**2.9E-08**	*m/s*

BH3B Permeability Test

Static Water level	1.92	*m bgl*

Bottom of Test Section	26.5	*m bgl*
Top of Test Section	17	*m bgl*
Length of Test Section	9.5	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.048	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.001237	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.0018096	*m3*	(area pipe ID)
Total unit volume	=	0.0030466	*m3*	
		0.0009698	*m3*	
Effective Radius	=	0.0311408	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

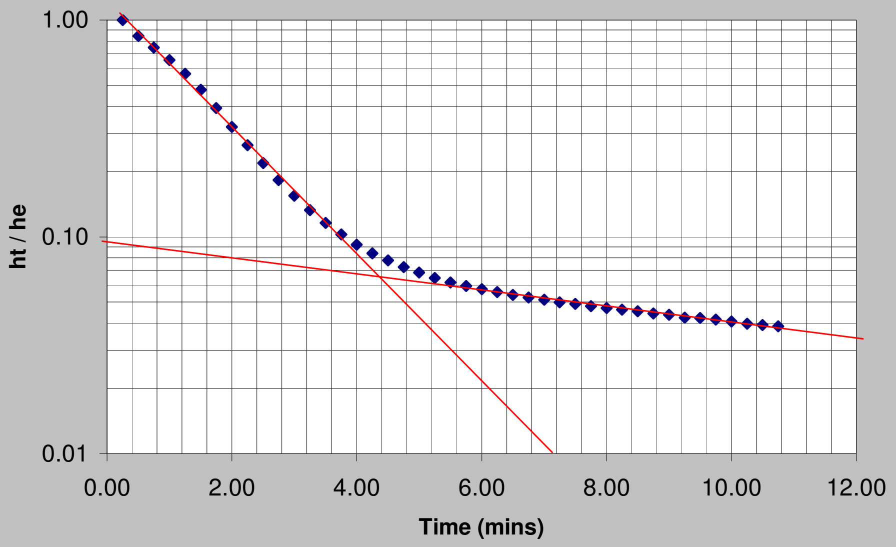
Permeability = $0.133S(r_e^2/L)$ m/s

	A		B	
InterceptHt/He	0.08		0.02	
Intercept Ht/He	0.9		1	
Intercept t	4		5.425	
Intercept t	0.3		1.8	
Slope	-0.284095		-0.4686814	
L	9.5		9.5	
Permeability	**2.3E-06**	*m/s*	**3.8E-06**	*m/s*

BH3C Permeability Test

Static Water level	0.2	*m bgl*

Bottom of Test Section	10	*m bgl*
Top of Test Section	1.3	*m bgl*
Length of Test Section	8.7	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.038	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.00056156	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.00180956	*m3*	(area pipe ID)
Total unit volume	=	0.00237112	*m3*	
		0.00075475	*m3*	
Effective Radius	=	0.02747271	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

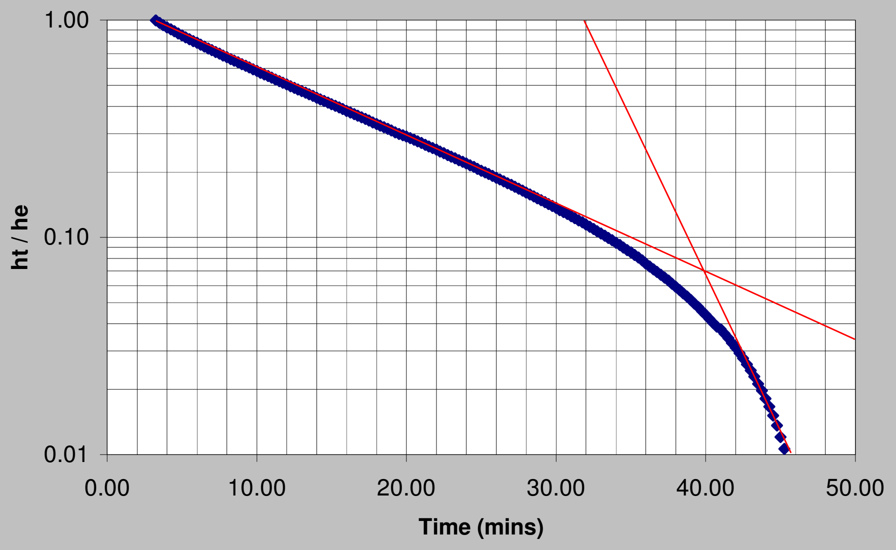
Permeability = $0.133S(r_e^2/L)$ m/s

	A		B	
InterceptHt/He	0.05		0.04	
Intercept Ht/He	1		0.09	
Intercept t	4.8		10	
Intercept t	0.4		0.8	
Slope	-0.2956886		-0.0382807	
L	8.7		8.7	
Permeability	**2.6E-06**	*m/s*	**3.4E-07**	*m/s*

BH4A Permeability Test

Static Water level	7.75	*m bgl*

Bottom of Test Section	54.6	*m bgl*
Top of Test Section	42	*m bgl*
Length of Test Section	12.6	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.048	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.001237	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.00180956	*m3*	(area pipe ID)
Total unit volume	=	0.00304656	*m3*	
		0.00096975	*m3*	
Effective Radius	=	0.03114081	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

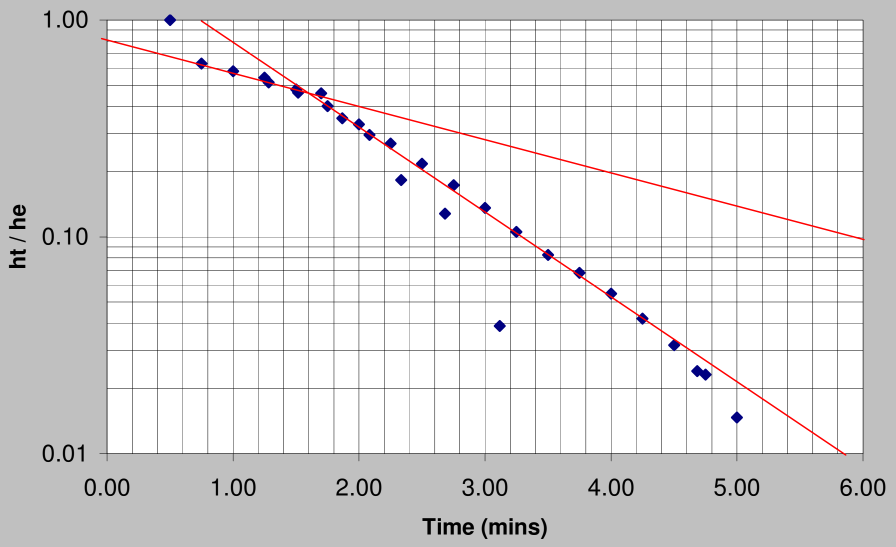
Permeability = $0.133S(r_e^2/L)$ m/s

	A		B	
InterceptHt/He	0.01		0.01	
Intercept Ht/He	1		1	
Intercept t	3.25		45.5	
Intercept t	35		32	
Slope	0.06299213		-0.1481481	
L	12.6		12.6	
Permeability	**3.8E-07**	*m/s*	**9.0E-07**	*m/s*

BH4B Permeability Test

Static Water level	2.78	*m bgl*

Bottom of Test Section	24.5	*m bgl*
Top of Test Section	12.5	*m bgl*
Length of Test Section	12	*m*
Casing Radius	0.024	*m*
Borehole Radius	0.038	*m*
Filter Pack Porosity	0.25	



Class E PVC ID		0.048	*m*	
Class E PVC OD		0.054	*m*	
Unit volume of Filter	=	0.00056156	*m3*	(area hole-area pipe OD) x Filter Porosity)
Unit volume of Pipe	=	0.00180956	*m3*	(area pipe ID)
Total unit volume	=	0.00237112	*m3*	
		0.00075475	*m3*	
Effective Radius	=	0.02747271	*m*	(sq rt (total volume/pi))
r_e	=	0.024	*m*	

Permeability = $0.133S(r_e^2/L)$ m/s

	A		B	
InterceptHt/He	0.1		0.01	
Intercept Ht/He	0.8		1	
Intercept t	5.9		5.8	
Intercept t	0.05		0.8	
Slope	-0.1543744		-0.4	
L	12		12	
Permeability	**9.9E-07**	*m/s*	**2.6E-06**	*m/s*

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